GEOTECHNICAL EXPLORATION
Bridge No. SCI-823-0917L
SR 823 over Portsmouth-Minford Road

SCI-823-6.81
Portsmouth Bypass – Phase 1
PID No. 19415

PREPARED FOR:
Ohio Department of Transportation
District 9
650 Eastern Avenue
Chillicothe, Ohio 45601

PREPARED BY:
HDR
HDR Engineering, Inc.
9987 Carver Road, Suite 200
Cincinnati, Ohio 45242
513-984-7500

May 2008

TABLE OF CONTENTS

1.0 INTRODUCTION

This report presents the results of HDR Engineering, Inc.'s geotechnical study for Bridge No. SCI-823-0917 L, SR 823 over Portsmouth-Minford Road, a component of Phase I of the Ohio Department of Transportation's Portsmouth Bypass project located in Scioto County. This study was undertaken in response to the Office of Structural Engineering's directive to modify the original two-span bridge design to four spans in order to eliminate the approximate 50-foot high MSE walls required to retain the approach embankments. This geotechnical report is intended to supplement the existing subsurface information at the site, and to amend, as necessary, the previous geotechnical recommendations provided by DLZ Ohio, Inc., (DLZ) in their "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" dated September 26, 2006.

The scope of work for this geotechnical study included

- a review of available soil, geologic and existing subsurface information at the site,
- site reconnaissance,
- the development and performance of a limited subsurface exploration program,
- laboratory testing on selected soil and rock samples in accordance with the requirements of the ODOT *Specifications for Geotechnical Exploration*,
- geotechnical engineering evaluations and analysis, and
- preparation of this report.

The purpose of this report is to present descriptions and interpretations of the subsurface conditions in the area of the proposed structure as they affect design, and to provide recommendations for geotechnical treatments and designs for the foundations of the substructure units.

2.0 PROJECT SETTING

The Portsmouth Bypass will be a four-lane limited access highway connecting U.S. Route 52 near Wheelersburg, Ohio to U.S. Route 23 north of Lucasville. The proposed bypass is intended to improve both regional mobility and economic development within the region, and will be constructed in three phases. Phase I of the project extends approximately 3.5 miles from Shumway Hollow Road to Lucasville-Minford Road (CR 28), passing through rough, hilly terrain. The steep hillsides and slopes located along the proposed alignment are typically wooded and undeveloped, while the more gradual slopes and valleys have for the most part been cleared for use as pasture land or have been developed as residential properties.

2.1 Proposed Structure

Figure 1 shows the planned location for Bridge No. SCI-823-0917 L. The proposed bridge is a 397-foot long, 4-span structure designed to carry traffic over Long Run Creek and Portsmouth-Minford Road (SR 139). The structure will be composed of 72-inch Modified AASHTO Type 4 prestressed concrete I-beams with a composite reinforced concrete deck supported on semi-integral abutments and T-type piers. As shown in Figure 2, the rear and forward abutments will be located at approximate Station 483+18 and Station 487+15, respectively, and are anticipated to be reinforced concrete semi-integral abutments supported on steel H-piles. Pier 1 will be located at Station 484+17 and Pier 2 at Station 485+17, on the opposite bank of Long Run Creek. Pier 3 will be located to the west of Portsmouth-Minford Road, at Station 486+17. Based on previous subsurface information gathered at the site, shallow spread footings bearing on rock appear to be viable options to support the bridge piers.

2.2 Soils

Review of the Natural Resources Conservation Service's "*Web Soil Survey*" (NRCS website, 2008) indicates several soil types within the project area, with the predominant soil associations consisting of the Shelocta-Brownsville and Omulga groups (see Figure 3). Specifically, soil types encountered within the immediate vicinity of Bridge No. SCI-823-0917 are listed below.

Skidmore Silt Loam, 0 to 3 percent slopes (Sk) – The Skidmore Silt Loam is typically found on flood plains, and as such, is occasionally flooded. These soils are well drained with high permeabilities and typically have a shallow water table. The depth to bedrock is also generally shallow in those areas overlain by the Skidmore Silt Loam. With a typical pH value ranging from 5.6 to 7.8, this unit represents a low risk of corrosion to uncoated steel and a moderate risk in regards to concrete.

Shelocta-Brownsville Association, 40 to 70 percent slopes (ScF) – The soils associated with the Shelocta-Brownsville Association are typically found along steep hillsides. They are well drained with moderately high to high permeabilities and available water capacities are moderate to high. The parent material for these soils is colluvium over residuum and the depth to water table is typically in excess of 80 inches. With a typical pH value ranging from 3.6 to 6.0, this unit represents a low risk of corrosion to uncoated steel and a high risk in regards to concrete. Additionally, this unit represents a very severe risk of erodibility due to the steepness of the slopes, particularly in regards to the Brownsville component.

Omulga Silt Loam, 1 to 8 percent slopes (OmB) – These soils are typically found along terraces and are moderately well drained. Permeabilities are moderately low to moderately high and available water capacities are considered moderate. The parent material is loess over alluvium over lacustrine deposits and the depth to water table is relatively shallow. With a typical pH value ranging from 3.6 to 7.3, this unit represents a moderate risk of corrosion to uncoated steel and a high risk in regards to concrete.

2.3 Site Geology

An overview of the site geology is found in the "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006) located in Appendix B. Please note that the potentially problematic Minford Silts do not appear to be present at the bridge site based upon our review of the previous test borings performed by DLZ at the site.

It should also be noted that slope instability was indicated by DLZ from Station 482+00 to Station 484+25 in their "*Report for Geology and Field Reconnaissance, Portsmouth Bypass Project, SCI-823-6.81, Phase I – Stage I, Scioto County, Ohio*" dated November 29, 2006. This instability was described by DLZ as relatively shallow soil creep contained within the overburden as a result of the toe of the nearly 1H:1V slope being eroded by Long Run Creek. The area was noted to exhibit signs of a massive landslide in the past at this location, but based on our meeting with DLZ and the Office of Geotechnical Engineering on December 20, 2007, it is our understanding that there is no evidence of a deep active slide in the area, and that the past slide at the site has removed the majority of the overburden on the slope. Evidence of the past slide and the more recent shallow soil creep were confirmed by HDR geotechnical personnel during their site reconnaissance on January 22, 2008.

3.0 SUBSURFACE EXPLORATION

A subsurface exploration program was developed using the site plans for the four span bridge option and the existing subsurface information available at the site. Eight test borings were previously drilled at the bridge site as part of DLZ's original geotechnical study for Bridge No. SCI-823-0837 L. As several of the previously drilled test borings are located at or near the proposed substructure units (see Figure 2), a single new test boring, designated as B-003-0-08 was located at the rear abutment of the structure. This boring was located and staked in the field by TesTech, Inc. with stations and offsets developed by HDR from the coordinates and elevation provided.

Drilling and sampling was performed on February 11, 2008. An ATV mounted CME 550 drill rig equipped with a 3¼" inside diameter hollow stem auger was used to advance the borings. The boring was drilled in general accordance with the "*Specifications for Geotechnical Explorations*" (ODOT, 2007) with sampling of the overburden soils accomplished in accordance with "*Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils*", ASTM D 1586. In the spilt-barrel sampling procedure, a standard 2-inch outside diameter split-barrel sampling spoon is driven into the ground with a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of an 18-inch penetration is recorded as the standard penetration test (SPT) resistance or N-value. The soils were sampled at 2.5-foot intervals until spoon refusal, defined as a minimum of 50 blows per 2 inches of penetration, was obtained on the underlying bedrock. It should be noted that as the soil/bedrock interface was generally transitional from residual soil to weathered rock, samples of this softer bedrock was achieved by overdriving the sampling spoon. Additional sampling of the bedrock at Boring B-003-0-08 was accomplished in accordance with the *"Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation"*, ASTM D 2113, using an NX-size double tube-swivel core barrel.

Water levels within Boring B-003-0-08 were measured when encountered during drilling, immediately upon completion of the boring, and again approximately 24 hours after completion. After obtaining the final water level reading, the boring was grouted in accordance with ODOT's "*Policy for Sealing of Geotechnical Exploratory Boreholes*".

4.0 LABORATORY TESTING PROGRAM

The recovered soil and rock samples were visually classified by an HDR geotechnical engineer and representative samples selected for laboratory testing to confirm the field classifications and to assess the various engineering properties of the encountered materials. The tests performed on representative soil samples included 7 natural moisture contents (ASTM D 2216), 2 Atterberg limit determinations (ASTM D 4318), 2 grain size analyses (ASTM D 422), and 2 unconfined compressive strength tests (ASTM D 2166). The results of the laboratory tests are presented on the laboratory summary sheets located in Appendix C, with individual copies of the laboratory test data sheets also provided in Appendix C.

5.0 ENCOUNTERED SUBSURFACE CONDITIONS AT THE STRUCTURE

This section summarizes the subsurface conditions encountered during the field exploration program. For a more detailed description of the subsurface conditions encountered during the previous subsurface exploration programs at the site, please refer to the "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006) located in Appendix B.

5.1 Previous Exploration Programs

Eight test borings were previously drilled at the bridge site as part of DLZ's original geotechnical study for the structure. Based upon review of their geotechnical report, five preliminary structural borings designated as TR-15 through TR-19 were performed by DLZ between July 9, 2004 and February 23, 2005, and three final structural borings, designated as B-10 through B-12, were performed between June 20 and 28, 2006. The locations of these eight borings as related to the current bridge plan are presented in Figure 2.

In general, the previous test borings at the site encountered 2 to 12 inches of topsoil overlying a relatively thin layer of primarily granular soils. The overburden typically extended from approximately 4.0 to 9.2 feet below the existing ground surface, and was described as gravel with sand (A-2-4), sandy silt (A-4a), and silt (A-4b) with a minor cohesive component. SPT N-values ranged from 2 to 18 blows/foot within the overburden material, with the granular soils noted to be loose to medium dense while the soils with a more appreciable cohesive component were typically described as medium stiff to very stiff.

The underlying bedrock was described as very fine to fine grained, argillaceous sandstone. Typically, the sandstone was described as medium hard to hard, moderately to slightly weathered, moderately to slightly fractured, and laminated to massively bedded. The amount of core recovery varied from 78 to 100 percent, with an average recovery of 95 percent. The rock quality designation (RQD) for the sandstone ranged between 57 and 97 percent, with an average RQD of 80 percent. Unconfined compressive strength tests performed on four intact core samples from the final structural borings indicated unconfined compressive strengths ranging from 9,709 to 11,829 psi, with an average unconfined compressive strength of 10,617 psi.

5.2 Recent Exploration Program (Rear Abutment)

This section summarizes the subsurface conditions encountered during HDR's field exploration program. The typed test boring log and photographs of the recovered rock core for boring B-003-0-08 is included in Appendix D.

Boring B-003-0-08 encountered a 12.5-foot thick layer of residual soil overlying sedimentary bedrock. The residuum was classified as silt (CL, A-4b) and silt and clay (CL, A-6a). SPT N-values within the overburden ranged from 9 blows/foot to over 50 blows/foot with depth.

The underlying sedimentary rock consisted of argillaceous silty shale and interbedded siltstone and sandy shale, with the top of rock (silty shale) encountered at approximate El. 663.0. The overlying argillaceous silty shale was described as completely to moderately weathered, with the degree of weathering decreasing with depth. The underlying interbedded siltstone and sandy shale was described as slightly weathered to unweathered. RQD values ranged from 82 to 94 percent, signifying very good quality rock. The core recoveries were generally good and ranged from 97 to 100 percent, with the lower recovery rates encountered within the upper rock stratum. The results of two unconfined compressive tests on intact core samples indicated unconfined compressive strengths (q_u) of 6,169 psi for the overlying argillaceous silty shale and 15,441 psi for the interbedded siltstone and sandy shale.

5.3 Summary of Subsurface Conditions

As noted previously, Bridge No. SCI-823-0917 L was modified from two spans to four spans in order to eliminate the approximate 50-foot high MSE walls required to retain the approach embankments. Under the new bridge design, several of the substructure units were repositioned and four new T-type piers added; however, the subsurface exploration program as performed by

DLZ had been completed under the original two-span bridge design. As shown in Table 1, these previously drilled test borings are located approximately 10 to 45 feet from the currently proposed substructure locations. As such, some variations in the estimated top of bedrock at the proposed substructure locations should be anticipated.

Table 1: Substructure and Boring Locations

Substructure		**Associated Borings**			
Description	**Station**	**Boring Number**	**Station**	**Top of Boring Elevation**	**Top of Rock Elevation**
Rear Abutment	483+16.0, CL	B-003-0-08	483+12.0, 11.4 ft. LT	675.5	659.4
Pier 1	484+14.7, CL	TR-18	484+38.6, 39.0 ft. LT	631.3	624.0
		TR-19	483+69.8, 46.5 ft. RT	633.0	624.3
Pier 2	485+14.7, CL	B-11	485+19.1, 48.6 ft. LT	632.7	624.2
		B-12	485+04.7, 9.0 ft. RT	632.5	624.0
		TR-17	485+26.9, 24.3 ft. RT	631.7	624.7
Pier 3	486+14.7, CL	B-10	486+01.5, 43.8 ft. RT	632.6	623.1
		TR-16	486+12.4, 32.3 ft. LT	631.9	623.4
Forward Abutment	487+13.4, CL	TR-15	486+83.3, 32.9 ft. RT	631.3	623.3

Table 2 presents the proposed design elevations for the individual substructure units and the top of rock as encountered in the nearby boring locations. Based on the encountered subsurface conditions at the site, the depth to bedrock varies from approximately 7 to 16 feet below the existing ground surface at the bridge site. The top of rock was encountered between elevations 623.1 and 624.7 along the valley floor at the locations of the bridge piers and the forward abutment. At the rear abutment as currently located on the valley wall, rock was encountered significantly higher at El. 659.4.

Table 2: Summary of Design Elevations for Individual Substructure Units

Substructure Unit	**Existing Grade at Centerline (Estimated)**	**Proposed Ground Surface At Centerline**	**Top of Rock[1] (El.)**	**Approximate Depth to Bedrock[2] (ft)**	**Proposed Bottom of Footing/ Concrete Cap**
Rear Abutment	665.0	699.8	659.4	40.5	685.9
Pier 1	634.0	643.0	624.0 - 624.3	18.5 to 19.0	622.5
Pier 2	631.6	631.6	624.0 - 624.7	7.0 to 8.0	622.7
Pier 3	631.9	635.4	623.1 - 623.4	12.0 to 12.5	622.8
Forward Abutment	633.2	691.1	623.3	68.0	676.8

Notes: 1. As encountered in the nearest test borings
2. Below proposed grade

6.0 ANALYSES AND DISCUSSIONS

Spread footings, drilled shafts and driven piles are all viable options for support of Bridge No. SCI-823-0917-L based upon the encountered subsurface conditions at the site as well as the economics of construction. As such, analyses were performed to determine the bearing capacity of shallow spread footings and the axial capacity of steel H-piles. Analyses for the drilled shafts were not performed as the recommendations provided by DLZ in their "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (2006) appear to be adequate. The results of these and other related analyses are presented in the appendices.

6.1 Rear Abutment

As shown in Table 2, the proposed bottom of footing/pile cap for the rear abutment is El. 685.87, approximately 21 feet above the existing ground surface (at the centerline) and roughly 27 feet above the top of rock based on boring B-003-0-08. Approximately 30 to 35 feet of fill will be required to attain the proposed profile grade (El. 699.8) at the abutment location based on the bridge plan provided in Figure 2. The overall depth of the embankment fill would preclude the use of spread footings bearing on rock, and excess differential settlement would be a concern if the spread footings were to be located within the fill. As such, steel H-piles driven to absolute refusal on bedrock appear to be the most feasible and cost effective foundation to support the rear abutment. For steel piles driven to bedrock, refusal is obtained when a minimum driving resistance of 20 blows per inch is achieved per Section 606.1 of the ODOT Bridge Design Manual.

Top of rock was encountered at El. 659.4 in boring B-003-0-08, with the bedrock consisting of decomposed to moderately weathered silty shale overlying interbedded siltstone and sandy shale. Refusal of the driven piles is expected to be obtained relatively quickly once the top of rock is encountered, with less than 12 inches of penetration into the overlying weathered rock expected. Hardened steel pile driving tips should be utilized per Section 202.2.2.2.a of the ODOT *Bridge Design Manual* to protect the H-piles from damage and to minimize slippage on the sloping bedrock surface.

For piles driven to refusal on competent rock, the structural capacity of the piles will control the design. Based on Section 4.5.7.3 of the *Standard Specifications for Highway Bridges* (AASHTO, 2002), an allowable axial stress of 12.5 ksi ($0.25f_y$) is recommended for a Grade 50 H-pile bearing on bedrock. Foundation settlement at the rear abutment as a result of elastic compression of the piles is anticipated to be negligible. It should be noted that lateral loads will be resisted by battered piles without relying on lateral resistance of the vertical piles.

Special construction measures will be required to allow for the installation of the driven piles through the approach embankments as the embankment material is expected to contain appreciable quantities of durable rock. It is recommended that the steel H-piles be installed through a pile window constructed during placement of the approach embankment fill. The pile window should extend 3 feet laterally beyond the outer limits of the piles in all directions, and extend from the bottom of the abutment pile cap to the existing ground surface. The pile window should be constructed of Granular Material Type C (Item 703.16 of the *Construction and Material Specifications*) as the maximum 3-inch particle size should not impede pile penetration and the requirement for prebored holes through the embankment material per Section 202.2.3.2.g of the ODOT *Bridge Design Manual* could be eliminated. It is anticipated that the Type C Granular Material can be processed on site using the hard, durable sandstone and siltstone from the nearby rock cuts.

6.1 Forward Abutment

As shown in Table 2, the proposed elevation for the bottom of footing/pile cap at the forward abutment is 676.78 feet, roughly 44 feet above the existing ground surface (at the centerline) and approximately 54 feet above the top of rock based on boring TR-15. The proposed profile grade at the abutment is El. 691.1, indicating that approximately 58 feet of embankment fill will be required at the abutment location based on the bridge plan provided in Figure 2. As such, steel H-piles driven to refusal on bedrock appear to be the most feasible and cost effective foundation to support the forward abutment as the overall depth of the embankment fill would preclude the use of spread footings bearing upon rock and excess differential settlement would be a concern if the spread footings would be located within the fill.

The top of rock was encountered at El. 623.3 in boring TR-15. The bedrock consists of medium hard to hard, very fine to fine grained sandstone. Refusal is expected to be obtained relatively quickly once the top of rock is encountered, with less than 12 inches of penetration into the sandstone expected. Hardened steel pile driving tips should be utilized per Section 202.2.2.2.a of the ODOT *Bridge Design Manual* to help protect the H-piles from damage during driving.

For piles driven to refusal on competent rock, the structural capacity of the piles will generally control the design. Based on Section 4.5.7.3 of the *Standard Specifications for Highway Bridges* (AASHTO, 2002), an allowable axial stress of 12.5 ksi ($0.25f_y$) is recommended for a Grade 50 H-pile bearing on bedrock. Foundation settlement at the forward abutment as a result of elastic compression of the piles is anticipated to be negligible. It should be noted that lateral loads will be resisted by battered piles without relying on lateral resistance of the vertical piles.

Special construction measures will be required to allow for the installation of the driven piles through the approach embankments as the embankment material is expected to contain appreciable quantities of durable rock. It is recommended that the steel H-piles be installed through a pile window constructed during placement of the approach embankment fill. The pile window should extend 3 feet laterally beyond the outer limits of the piles in all directions, and extend from the bottom of the abutment pile cap to the existing ground surface. The pile window should be constructed of Granular Material Type C (Item 703.16 of the *Construction and Material Specifications*) as the maximum 3-inch particle size should not impede pile penetration and the requirement for prebored holes through the embankment material per Section 202.2.3.2.g of the ODOT *Bridge Design Manual* could be eliminated. It is anticipated that the Type C Granular Material can be processed on site using the hard, durable sandstone and siltstone from nearby rock cuts.

6.3 Bridge Piers

Based on the subsurface conditions encountered at the pier locations, bedrock is expected to be encountered within approximately 7 to 19 feet below final grade at Piers 1, 2 and 3 (See Table 2). As such, both drilled shafts and spread footings bearing upon competent rock appear to be viable options to support the bridge piers. Recommendations are provided for both foundation types, with constructability and cost effectiveness expected to be the main factors in determining the most feasible foundation alternative.

6.3.1 Pier 1

Spread Footings

Based on Borings TR-18 and TR-19, the top of rock was encountered from El. 624.0 to El. 624.3 across Pier 1. The bedrock was described as medium hard to hard, very fine to fine grained, argillaceous, micaceous sandstone. The sandstone is moderately to slightly weathered, with fractures and broken zones noted from El. 624.0 to El. 623.3 in boring TR-18, and decomposed rock from El. 624.3 to El. 623.6 in boring TR-19. As such, it is recommended that the proposed bottom of footing be located at El. 623.3 or lower.

Analyses were performed to verify the allowable bearing capacity of 40 tsf for spread footings bearing upon competent bedrock as recommended by DLZ in their previous geotechnical report for the site (DLZ, 2006). These analyses were based upon the Geomechanics Classification System of Rock Mass Rating, and using the rock descriptions, RQD, and unconfined compression test data as provided in DLZ's final boring logs. As shown in the analyses presented in Appendix E, a reduced allowable bearing capacity of 35 tsf is recommended.

Due to the potential for variations in the top of bedrock beneath the footing from that encountered at Borings TR-18 and TR-19, provisions should be included in the construction plans for overexcavation and backfill with Class C concrete. If unacceptable bearing material is encountered at or below the proposed bottom of footing, the unacceptable materials should be removed to bedrock, and the minimum bottom of footing reestablished using Class C concrete. Any overexcavation should be stepped and have a level bottom.

Drilled Shafts

The use of drilled shafts should be explored as an alternative to a spread footing foundation at Pier 1 due to the size and depth of the excavation that will be required to construct the spread footing. It is currently understood that the approach embankment will be constructed under separate contract in advance of the bridge contract. As such, an approximate 21-foot deep excavation would be required to construct the footing at the proposed bearing elevation of 622.5. Temporary shoring, particularly on the upslope side of the excavation would likely be required, and/or the excavation sloped in accordance with applicable federal (OSHA) and state standards. A smaller footprint with less excavation is anticipated for the drilled shaft alternative as the cap for the drilled shafts is expected to be considerably smaller than the dimensions for a spread footing, and the bottom of the cap would likely be set higher, within the overburden material, rather than at the top of rock.

The drilled shafts should be designed following the recommendations provided in the "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006). Per DLZ's geotechnical report, the drilled shafts should be socketed a minimum of 5 feet into competent bedrock, and the shafts designed for tip resistance only, using an allowable bearing pressure of 40 tsf. Any side resistance provided by the overlying soils and from the shallow rock socket should be neglected. Per DLZ's report, deeper rock sockets (> 5 ft) can be utilized if adequate capacity cannot be developed through end bearing; however, it is recommended that the drilled shafts be designed such that the loads are carried entirely by the socket resistance and any end bearing ignored. DLZ recommends an allowable sidewall resistance of 7500 psf for the rock socket, and that any side resistance within the upper two feet of the rock socket be neglected.

6.3.2 Pier 2

Spread Footings

The top of rock varies from El. 624.0 to El. 624.7 across Pier 2, with the bedrock described as medium hard to hard, very fine to fine grained, argillaceous sandstone based on borings B-11, B-12 and TR-17. A highly fractured to broken zone was noted from El. 624.2 to El. 622.7 in Boring B-11 and a very soft, highly weathered zone to El. 624.3 in Boring TR-17. As such, it is recommended that the proposed bottom of footing be set at El. 622.7 or lower.

Analyses were performed to verify the allowable bearing capacity of 40 tsf for spread footings bearing upon competent bedrock as recommended by DLZ in their previous geotechnical report for the site (DLZ, 2006). These analyses were based upon the Geomechanics Classification System of Rock Mass Rating, and using the rock descriptions, RQD, and unconfined compression test data of the bedrock as provided in DLZ's final boring logs. As shown in the analyses presented in Appendix E, a reduced allowable bearing capacity of 35 tsf is recommended.

Due to the potential for variations in the top of bedrock beneath the footing from that encountered at Borings B-11, B-12 and TR-17 , provisions should be included in the construction plans for overexcavation and backfill with Class C concrete. If unacceptable bearing material is encountered at or below the proposed bottom of footing, the unacceptable materials should be removed to bedrock, and the minimum bottom of footing reestablished using Class C concrete. Any overexcavation should be stepped and have a level bottom.

Drilled Shafts

The use of drilled shafts should be explored as an alternative to a spread footing foundation at Pier 2 due to the close proximity of the bridge pier to Long Run Creek. A smaller footprint with less excavation is anticipated for the drilled shaft alternative, and could eliminate the need for construction of a temporary cofferdam within the creek and associated dewatering.

The drilled shafts should be designed following the recommendations provided in the "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006). Per DLZ's geotechnical report, the drilled shafts should be socketed a minimum of 5 feet into competent bedrock, and the shafts designed for tip resistance only, using an allowable bearing pressure of 40 tsf. Any side resistance provided by the overlying soils and from the shallow rock socket should be neglected. Per DLZ's report, deeper rock sockets (> 5 ft) can be utilized if adequate capacity cannot be developed through end bearing; however, it is recommended that the drilled shafts be designed such that the loads are carried entirely by the socket resistance and any end bearing ignored. DLZ recommends an allowable sidewall resistance of 7500 psf for the rock socket, and that any side resistance within the upper two feet of the rock socket be neglected.

6.3.3 Pier 3

Spread Footings

Based on Borings B-10 and TR-16, the top of rock was encountered from approximate El. 623.1 to El. 623.4 across Pier 3. The bedrock was described as medium hard to hard, very fine to fine grained, argillaceous, micaceous sandstone. With recovery rates ranging from 96 to 98 percent, it is recommended that the proposed bottom of footing for Pier 3 be located at El. 623.0 or lower.

Analyses were performed to verify the allowable bearing capacity of 40 tsf for spread footings bearing upon competent bedrock as recommended by DLZ in their previous geotechnical report for the site (DLZ, 2006). These analyses were based upon the Geomechanics Classification System of Rock Mass Rating (RMR), and using the rock descriptions, RQD, and unconfined compression test data of the bedrock as provided in DLZ's final boring logs. As shown in the analyses presented in Appendix E, a reduced allowable bearing capacity of 35 tsf is recommended.

Due to the potential for variations in the top of bedrock beneath the footing from that encountered at Borings B-10 and TR-16, provisions should be included in the construction plans for overexcavation and backfill with Class C concrete. If unacceptable bearing material is encountered at or below the proposed bottom of footing, the unacceptable materials should be removed to competent rock, and the minimum bottom of footing reestablished using Class C concrete. Any overexcavation should be stepped and have a level bottom.

Drilled Shafts

The use of drilled shafts should be explored as an alternative to a spread footing foundation at Pier 3 due to the size and depth of the excavation that will be required to construct the spread footing. It is currently understood that the approach embankment will be constructed under separate contract in advance of the bridge contract. As such, an approximate 13-foot deep excavation would be required to construct the footing at the proposed bearing elevation of 622.8. Temporary shoring, particularly on the upslope side of the excavation would likely be required, and/or the excavation sloped in accordance with applicable federal (OSHA) and state standards. A smaller footprint with less excavation is anticipated for the drilled shaft alternative as the cap for the drilled shafts is expected to be considerably smaller than the dimensions for a spread footing, and the bottom of the cap would likely be set higher, within the overburden material, rather than at the top of rock.

The drilled shafts should be designed following the recommendations provided in the "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006). Per DLZ's geotechnical report, the drilled shafts should be socketed a minimum of 5 feet into competent bedrock, and the shafts designed for tip resistance only, using an allowable bearing pressure of 40 tsf. Any side resistance provided by the overlying soils and from the shallow rock socket should be neglected. Per DLZ's report, deeper rock sockets (> 5 ft) can be utilized if adequate capacity cannot be developed through end bearing; however, it is recommended that the drilled shafts be designed such that the loads are carried entirely by the socket resistance and any end bearing ignored. DLZ recommends an allowable sidewall resistance of 7500 psf for the rock socket, and that any side resistance within the upper two feet of the rock socket be neglected.

6.4 Approach Embankments

As over 3 million cubic yards of waste material is currently estimated for Phase I of the Portsmouth Bypass project, consideration should be given to using durable rock fill to construct the bridge approach embankments. The use of durable rock rather than random fill materials will help to limit settlement at the bridge approaches (thus avoiding the bump that commonly occurs at the ends of the structure), as well as reduce the quarantine period for the embankments as settlement of the rock fill itself should occur relatively quickly. In addition, the stability of the embankment slopes will be improved as the rock fill provides a substantial increase in shear strength over that of random fill. The durable rock fill should be located within six times the height of the fill at the abutment location, and placed in accordance with Item 203 of the *Construction and Materials Specifications.*

6.4.1 Slope Stability

Based upon recommendations provided in the "*Report of Subsurface Investigation, Embankments (Station 416+00 to 509+50), Project SCI-823-6.81, Phase 1 – Stage 1, Scioto County, Ohio*" (DLZ, 2006), the embankment slope ratios beyond the ends of the bridge were set at 2H:1V. Stability analyses for the planned slopes were conducted in accordance with the guidelines and criteria established by the Ohio Department of Transportation using a minimum target factor of safety of 1.3 for both long and short term conditions as the abutments will be supported on pile foundations.

The soil and rock properties used in the stability analyses for the various strata encountered at the site are presented in Table 3. These parameters are based on previous values reported by DLZ in their "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" and their "*Response to Stage I Geotechnical Review Comments, Phase I*" dated March 7, 2008, as well as standard geotechnical correlations and engineering judgment.

Table 3: Soil Parameters Used in Stability Analyses

Zone	Soil Type	Unit Weight (pcf)	Strength Parameters			
			Undrained		Drained	
			c (psf)	Φ	c' (psf)	φ'
Fill	Compacted Embankment Fill	125	0	35	0	35[a]
Rock Toe	Select Rock Fill	130	0	38	0	38
Foundation Soil (Rear Abutment)	Medium Dense Sandy Silt	120	0	29	0	29
Foundation Soil (Forward Abutment)	Very Soft to Stiff Sandy Silt	120	1000	0	0	29
Bedrock	Sandstone and Siltstone	130	3500	45	3500	45

Note: a. Embankment fill consisting primarily of excavated rock (per DLZ).

The stability analyses were performed using the software package GSTABL7 with STEDwin. This program is a Windows version of the computer program STABL as developed by Purdue University through the support of the Indiana State Highway Commission. The program's capacity to analyze circular failure surfaces using the Modified Bishop's Method of Slices was used in these analyses. The results of the stability analyses are presented in Appendix E.

The planned 2H:1V embankment slopes at the forward abutment meet the minimum required factor of safety of 1.3 under both short and long-term conditions. As shown in the stability runs presented in Appendix E, factors of safety of 1.36 and 1.35 were calculated, respectively. However, at the rear abutment, the existing foundation soils encountered along Long Run Creek do not have sufficient strength for the planned embankment slopes to meet the targeted ODOT standard. As shown in Appendix E, a factor of safety of 1.24 was calculated for the rear approach embankment under both short and long term conditions. As such, an embankment toe key was modeled to lock the embankment into the relatively flat, existing ground and increase the shear strength of the foundation soils. Based on the stability analyses, the shear key will need to be constructed of durable rock fill and should extend to the top of bedrock. The base of the shear key should be a minimum of 8 feet in width, with front and back slopes of 1H:1V extending from the existing ground surface to the top of rock. As shown in the stability runs presented in Appendix E, the use of an embankment toe key is sufficient to increase the calculated factor of safety at the rear abutment to 1.31, exceeding the targeted ODOT standard of 1.3.

6.4.2 Embankment Settlement

Due to roadway design and grading requirements, the bridge abutments will be constructed on relatively large approach embankments. Based on the provided bridge plan (Figure 2), up to 35 feet of compacted fill is expected at the rear abutment, and over 58 feet of fill at the forward abutment. The magnitude of the embankment settlement will be a function of the consolidation of the existing foundation soils under the influence of the overlying fill and consolidation of the embankment fill itself under the influence of successive lifts. It is difficult to analyze settlement of the compacted embankment fill as the amount of settlement experienced will be dependent upon the materials, placement, and construction controls used to place the embankments. As such, a quarantine period and settlement monitoring is often recommended for critical embankment areas near project structures as inherent impacts such as downdrag and bending of piles, and rotation/differential stresses on the substructure units can occur if settlement is not allowed to progress to completion, or near completion, prior to substructure construction. Based upon research performed by the United States Bureau of Reclamation (Sherard et. al., 1963), consolidation within compacted embankment fill generally ranges between approximately one to four percent of the embankment height. Using proper placement and compaction of the embankment materials, and assuming one percent consolidation as the embankments will be constructed primarily of excavated rock, approximately 4 to 5 inches of settlement at the rear abutment and about 7 inches of settlement at the forward abutment can be expected. However, it is anticipated that most of this settlement will occur during construction of the embankment.

Settlement analyses were performed at Station 483+17 and Station 486+98 to assess the magnitude and duration of the expected settlement for the encountered foundation soils at the site as a result of the new embankment loading. As shown in Appendix E, settlement as a result of primary consolidation is estimated to be approximately 2 inches at Station 483+17 and approximately 1 inch at Station 486+98. The time needed to reach 90% consolidation is estimated at 107 days and 221 days respectively.

Due to the estimated 1 to 2 inches of consolidation settlement expected at the approach embankments, additional loading due to downdrag on the pile supported abutments is a concern. It is estimated that consolidation will take approximately two months from completion of the embankments to progress to the point where less than ½ inch of settlement has yet to occur (the point at which loading due to downdrag on the abutment piles is no longer a concern). As such, the embankments should be quarantined and monitored for a minimum of 60 days to allow the settlement to take place prior to the start of substructure construction. Provisions should be included in the contract to allow for an extension of the monitoring period without penalty if the settlement has not slowed to an acceptable rate over the 60 days.

6.4.3 Settlement Monitoring

Settlement monitoring should consist of the placement and monitoring of surface monuments to establish the time-settlement characteristics of the embankment fill and the underlying foundation soils once the embankments are complete. Surface monuments typically consist of a 6-inch diameter augured hole that is backfilled with concrete. A section of steel rebar (minimum length of 36 inches) is centered in the concrete, with the top of the reinforcing bar approximately ½ inch above the ground surface. (See Figure 4.) Recommended locations for the surface monuments are provided in Table 4.

Table 4 Recommended Locations for Surface Monuments

Approach Embankment	Station	Location
Rear	482+90, 40 feet LT	Roadway Shoulder
	482+70, 40 feet RT	Roadway Shoulder
Forward	487+70, 40 feet LT	Roadway Shoulder
	487+50, 40 feet RT	Roadway Shoulder

Weekly settlement monitoring should be performed, and the survey data collected over the quarantine period reviewed by the District to establish the time-settlement characteristics of each approach embankment. The quarantine period could be refined and possibly shortened at the direction of the District should the data collected during the quarantine period show negligible settlement at a time less than the recommended 60 days. Conversely, if the data shows that settlement is continuing at a magnitude or rate deemed unacceptable by the District at the end of the 60 day period, the quarantine period should be extended as required.

7.0 RECOMMENDATIONS

General and specific recommendations are provided in this section and include foundation details as well as locations for geotechnical treatments for the approach embankments based on the proposed bridge design.

7.1 Foundation Design

Table 5 provides a summary of the foundation design parameters for Bridge No. SCI-823-0917 L, based on review of the previous geotechnical exploration programs at the site, the encountered subsurface conditions, laboratory tests performed on representative soil and rock samples, and our engineering analyses. Driven H-piles are recommended to support the rear and forward abutments, and recommendations for both spread footings and drilled shafts are provided for the bridge piers.

7.1.1 Rear Abutment

- It is recommended that the rear abutment be founded upon steel H piles driven to absolute refusal on the underlying bedrock. An allowable axial stress of 12.5 ksi is recommended for a Grade 50 H-pile bearing on bedrock.
- The allowable pile capacities provided in Section 202.2.3.2a of the *Bridge Design Manual* do not include section loss due to corrosion. As corrosivity testing was not performed on the potential embankment material, a corrosive environment should be assumed, and the pile dimensions should be reduced by 1/16 inch when computing the area of the pile.
- Standard pile tip reinforcement is recommended per Section 202.2.3.2.a of the ODOT *Bridge Design Manual.*
- An average pile length of 29 feet is anticipated based on the encountered subsurface conditions at Borings B-003-0-08 and the design elevations presented in Table 5.
- It is recommended that the steel H-piles be installed through pile windows constructed during placement of the approach abutment fill. The pile window should extend 3 feet laterally beyond the outer edges of the piles in all directions, with the vertical extent of the window from the bottom of the abutment pile cap to the existing ground surface. The pile window should be constructed of Type C Granular Material (Item 703.16 of the *Construction and Material Specifications*).
- The abutment should be designed based on an active earth pressure condition using a unit weight of 125 pcf and an angle of internal friction of 35 degrees plus any surface surcharge. To account for traffic loading, a surcharge equivalent to 2 feet of soil ($\gamma = 120$ pcf) should be applied. Please note that no hydrostatic pressure has been included in the recommended design earth pressure. As such, drainage provisions for the abutment should be provided.

Table 5: Summary of Foundation Design Parameters								
Substructure Unit	**Rear Abutment**	**Forward Abutment**	**Pier 1**	**Pier 2**	**Pier 3**	**Pier 1**	**Pier 2**	**Pier 3**
Foundation Type	Driven Piles	Driven Piles	Spread Footing	Spread Footing	Spread Footing	Drilled Shafts	Drilled Shafts	Drilled Shafts
Proposed Bottom of Footing/Pile Cap (El.)	685.87	676.78	623.3	622.7	623.0	TBD	TBD	TBD
Top of Bedrock (El.)	659.0	623.0	624.0 to 624.5	624.0 to 625.0	623.0 to 623.5	624.0 to 624.5	624.0 to 625.0	623.0 to 623.5
Estimated Tip Elevation (El.)	658.5	622.0	NA	NA	NA	618.3 (1)	617.7 (1)	618.0 (1)
Estimated Pile Length 2,3	29 ft	56 ft	NA	NA	NA	NA	NA	NA
Allowable Axial Stress 4,5	12.5 ksi	12.5 ksi	NA	NA	NA	NA	NA	NA
Minimum Length of Rock Socket 1,6	NA	NA	NA	NA	NA	5 ft	5 ft	5 ft
Allowable Side Resistance of Rock Socket 6,7,8	NA	NA	NA	NA	NA	7500 psf	7500 psf	7500 psf
Allowable Bearing Capacity	NA	NA	35 tsf	35 tsf	35 tsf	40 tsf (6)	40 tsf (6)	40 tsf (6)

Notes:
1. The design lengths for the rock sockets and corresponding tip elevations to be determined using the axial and lateral loads
2. Average Length based on encountered bedrock elevation at the test boring locations
3. Includes 1-foot embedment into cap
4. Allowable horizontal or lateral load to be developed in battered piles
5. Allowable Axial Stress does not include section loss due to corrosivity
6. Per "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006)
7. Neglect the upper two feet of the rock socket. (per DLZ recommendations)
8. If side resistance of the rock socket is utilized (length of socket >5 feet), the design load should be carried entirely by the side resistance, ignoring any end bearing. (per DLZ recommendations)
9. TBD = to be determined
10. NA = not applicable

7.1.2 Forward Abutment

- It is recommended that the forward abutment be founded upon steel H piles driven to absolute refusal on the underlying bedrock. As allowable axial stress of 12.5 ksi is recommended for a Grade 50 H-pile bearing on bedrock.
- The allowable pile capacities provided in Section 202.2.3.2a of the *Bridge Design Manual* do not include section loss due to corrosion. As corrosivity testing was not performed on the potential embankment material, a corrosive environment should be assumed, and the pile dimensions should be reduced by 1/16 inch when computing the area of the pile.
- Standard pile tip reinforcement is recommended per Section 202.2.3.2.a of the ODOT *Bridge Design Manual.*
- An average pile length of 56 feet is anticipated based on the encountered subsurface conditions at Borings TR-15, and the design elevations presented in Table 5.
- It is recommended that the steel H-piles be installed through pile windows constructed during placement of the approach abutment fill. The pile window should extend 3 feet laterally beyond the outer edges of the piles in all directions, with the vertical extent of the window from the bottom of the abutment pile cap to the existing ground surface. The pile window should be constructed of Type C Granular Material (Item 703.16 of the *Construction and Material Specifications*).
- The abutment should be designed based on an active earth pressure condition using a unit weight of 125 pcf and an angle of internal friction of 35 degrees plus any surface surcharge. To account for traffic loading, a surcharge equivalent to 2 feet of soil ($\gamma = 120$ pcf) should be applied. Please note that no hydrostatic pressure has been included in the recommended design earth pressure. As such, drainage provisions for the abutment should be provided.

7.1.3 Pier 1

It is recommended that drilled shafts be used to support Pier 1 as a drilled shaft foundation may be more cost effective than a spread footing given that the approach embankments will be in-place prior to the construction of the substructure units. For a drilled shaft foundation , a smaller footprint with less excavation is anticipated as the pier cap is expected to be smaller than a spread footing and the bottom of the cap would likely be located within the overburden, rather than at the top of rock. Please note that as a cost analysis of the foundation alternatives were not performed as part of this geotechnical study, recommendations for both drilled shafts and spread footings are provided should spread footings prove to be more economical.

Drilled Shafts

- Design recommendations for drilled shafts are located in Table 5 and in the "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00, Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006). It should be noted that deeper rock sockets (> 5 ft) can be utilized if adequate capacity cannot be developed through end bearing; however, the drilled shafts should be designed such that the loads are carried entirely by side resistance of the rock socket and any end bearing ignored.

Spread Footings Bearing on Rock

- A bottom of footing elevation of 623.3 is recommended based on the subsurface conditions encountered at Borings TR-18 and TR-19.

- The footings should be designed using an allowable bearing capacity of 35 tsf and a friction factor of 0.7 for cast-in-place footings on bedrock.
- Due to the potential for variations in the top of bedrock beneath the footing from that encountered at Borings TR-18 and TR-19, provisions should be included in the construction plans for overexcavation and backfill with Class C concrete. If unacceptable bearing material is encountered at or below the proposed bottom of footing, the unacceptable materials should be removed to the top of rock, and the minimum bottom of footing reestablished using Class C concrete. Any overexcavation should be stepped and have a level bottom.
- As the approach embankment will be placed prior to construction of the substructure units, an excavation of approximately 21 feet will be required to construct the footing. As such, the footing excavation for Pier 1 will require temporary shoring, particularly on the upslope side of the excavation.

7.1.4 Pier 2

The use of drilled shafts is recommended for Pier 2 due to the close proximity of the bridge piers to Long Run Creek. A smaller footprint with less excavation is anticipated for a drilled shaft foundation as compared to a spread footing, possibly eliminating the need for construction of a temporary cofferdam within the creek. However, a cost analysis of the foundation alternatives was not performed as part of this geotechnical study, and recommendations for both drilled shafts and spread footings are provided should spread footings prove to be more economical.

Drilled Shafts

- Design recommendations for drilled shafts are located in Table 5 and in the "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00, Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006). It should be noted that deeper rock sockets (> 5 ft) can be utilized if adequate capacity cannot be developed through end bearing; however, the drilled shafts should be designed such that the loads are carried entirely by the socket resistance and any end bearing ignored.

Spread Footings Bearing on Rock

- A bottom of footing elevation of 622.7 is recommended based on the subsurface conditions encountered at Borings B-11, B-12 and TR-17.
- The footings should be designed using an allowable bearing capacity of 35 tsf and a friction factor of 0.7 for cast-in-place footings on bedrock.
- Due to the potential for variations in the top of bedrock beneath the footing from that encountered at Borings B-11, B-12 and TR-17 , provisions should be included in the construction plans for overexcavation and backfill with Class C concrete. If unacceptable bearing material is encountered at or below the proposed bottom of footing, the unacceptable materials should be removed to the top of rock, and the minimum bottom of footing reestablished using Class C concrete. Any overexcavation should be stepped and have a level bottom.

7.1.5 Pier 3

Since the approach embankments will be in-place prior to the construction of the substructure units, it is recommended that drilled shafts be used to support Pier 3 as a drilled shaft foundation may be more cost effective due to the additional shoring and excavation costs associated with a spread footing. Recommendations for both drilled shafts and spread footing are provided should spread footings prove to be more economical.

Drilled Shafts

- Design recommendations for drilled shafts are located in Table 5 and in the "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00, Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006). It should be noted that deeper rock sockets (> 5 ft) can be utilized if adequate capacity cannot be developed through end bearing; however, the drilled shafts should be designed such that the loads are carried entirely by the socket resistance and any end bearing ignored.

Spread Footings Bearing on Rock

- A bottom of footing elevation of 623.0 is recommended based on the subsurface conditions encountered at Borings B-10 and TR-16.
- The footings should be designed using an allowable bearing capacity of 35 tsf and a friction factor of 0.7 for cast-in-place footings on bedrock.
- Due to the potential for variations in the top of bedrock beneath the footing from that encountered at Borings B-10 and TR-16, provisions should be included in the construction plans for overexcavation and backfill with Class C concrete. If unacceptable bearing material is encountered at or below the proposed bottom of footing, the unacceptable materials should be removed to the top of rock, and the minimum bottom of footing reestablished using Class C concrete. Any overexcavation should be stepped and have a level bottom.
- As the approach embankment will be placed prior to construction of the substructure units, an excavation of approximately 13 feet will be required to place the bottom of footing at a consistent elevation. As such, the footing excavation for Pier 1 may require temporary shoring, particularly on the upslope side of the excavation.

7.1.6 Temporary Construction Issues for Excavations

All temporary excavations at the site should comply with the requirements of OSHA 29 CFR, part 1926, Subpart P, "Excavations and Trenches" and other applicable codes. The excavations are anticipated to encounter natural silts and sands, as well as newly placed embankment fill. Temporary slopes should be observed daily for signs of distress as exposure to the environment may weaken the soils should the excavations remain open for extended periods of time.

7.1.7 Groundwater Considerations

Based on review of the geotechnical recommendations provided in the "*Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006), seepage was noted between approximate El. 624.5 and El. 626 at borings TR-15, TR-16 and TR-17, with no measurable water levels in the borings prior to rock coring. Based on experience, groundwater is likely to be encountered near the top of rock with some variation expected due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were completed. In addition, groundwater is expected to vary with the water level within nearby Long Run Creek. As such, the Contractor should anticipate that the pier foundation excavations will likely require dewatering. Any excavations near Long Run Creek should also be protected from stream and storm water flow.

7.2 Approach Embankments

The approach embankments at both the Forward and Rear Abutments should be constructed in accordance with the recommendations provided in the "*Report of Subsurface Investigation,* *Embankments (Station 416+00 to 509+50), Project SCI-823-6.81, Phase 1-Stage 1, Scioto County, Ohio*" (DLZ, 2006) with the following exceptions.

- It is recommended that the approach embankments be constructed of durable rock fill in order to limit settlement at the bridge approaches and potentially reduce the quarantine period for the embankments. The durable rock fill should extend a distance of six times the height of the fill (at the abutment) from the abutment location. The rock fill should be placed in accordance with Item 203 of the *Construction and Materials Specifications*.
- It is recommended that the rear approach embankment incorporate an embankment toe key and special benching in accordance with ODOT's Office of Geotechnical Engineering "*Geotechnical Bulletin GB2 - Special Benching and Sidehill Embankment Fills*".
    - The recommended shear key should be constructed of durable rock fill and extend completely through the foundation soils to the top of bedrock. The base of the shear key should be a minimum of 8 feet in width, with the front and back slopes of the key constructed at 1H:1V.
    - Special benching of the rear approach embankment will be required as the existing hillside is steeper than 4H:1V. Per GB2, the special benching is shown on the cross-sections in the project plans, and is performed in addition to, and in place of, standard specification benching (Item 203.05). In addition, Plan Note G110 from the ODOT *Location and Design Manual, Volume 3* needs to be included in the General Notes.
- It is currently anticipated that the approach embankments will be in-place prior to the start of construction of the proposed bridge structure. However, to ensure that settlement of the embankment fill and underlying soils has progressed sufficiently to avoid the effects of downdrag on the pile supported abutments, it is recommended that the embankments be quarantined and monitored for approximately 60 days after construction of the embankment fill is complete or prior to the start of pile driving for the abutments. A settlement monitoring program is recommended to establish the time-settlement characteristics of the embankment fill and underlying foundation soils. The recommended locations of the surface monuments are given in Table 4. If the data collected during the quarantine period shows negligible settlement at a time less than the recommended 60 days, than the quarantine period may be shortened at the direction of the District. Conversely, if the data shows settlement to be continuing at a magnitude or rate deemed unacceptable by the District at the end of the 60 day period, the quarantine period should be extended as appropriate.

8.0 LIMITATIONS

This report documents the findings and conclusions of HDR Engineering, Inc., for the geotechnical aspects related to the design of the proposed Bridge No. SCI-823-0917L SR 823 over Portsmouth-Minford Road (S.R. 139), in Scioto County, Ohio. The report has been prepared for the use of the Ohio Department of Transportation for specific application to the project, in accordance with generally accepted engineering practice and the parameters established by others from previous project geotechnical studies. No warranty, expressed or implied, is made. Any analyses or recommendations submitted are based on field explorations performed at the locations indicated, on specific laboratory tests on individual samples taken during the investigation, and information obtained from outside sources. The report and analyses do not reflect variations that could occur between borings or at other points in time. Variations in conditions, if any, may become evident during the construction period, at which time, a re-evaluation of the recommendations may become necessary. In the event of such changes, the recommendations and changes should be reviewed by HDR's geotechnical staff.

9.0 REFERENCES

American Association of State Highway and Transportation Officials (2002). *"Standard Specifications for Highway Bridges – 17e Edition".*

DLZ, Inc. (2006). *Subsurface Investigation Embankments (Station 416+00 to 509+50) Project SCI-823-6.81, Phase 1 – Stage 1 Scioto County, Ohio".*

DLZ, Inc. (2006). *"Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Swauger Valley-Minford Road, SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio".*

DLZ, Inc. (2006). *"Report for Geology and Field Reconnaissance, Portsmouth Bypass, Project SCI-823-6.81, Phase 1 – Stage 1, Scioto County, Ohio".*

DLZ, Inc. (2008). *SCI-823-6.81, "Portsmouth Bypass Project, PID 19415 Response to Stage 1 Geotechnical Review Comments, Phase 1".*

DLZ, Inc. (2008). *SCI-823-6.81, "Portsmouth Bypass Project, PID 19415 Addendum to Report: Embankments (Station 416+00 to 509+50) Phase 1 – Stage 1: Time Rage of Consolidation".*

Sherard, J. L., Woodward, R. J., Gizienski, S. F., Clevenger, W. A. (1963). *"Earth and Earth-Rock Dams"*, John Wiley and Sons, Inc.

State of Ohio Department of Transportation (2008). *"Construction and Material Specifications"*, Columbus Ohio.

State of Ohio Department of Transportation (2007) "*Specifications for Geotechnical Explorations*".

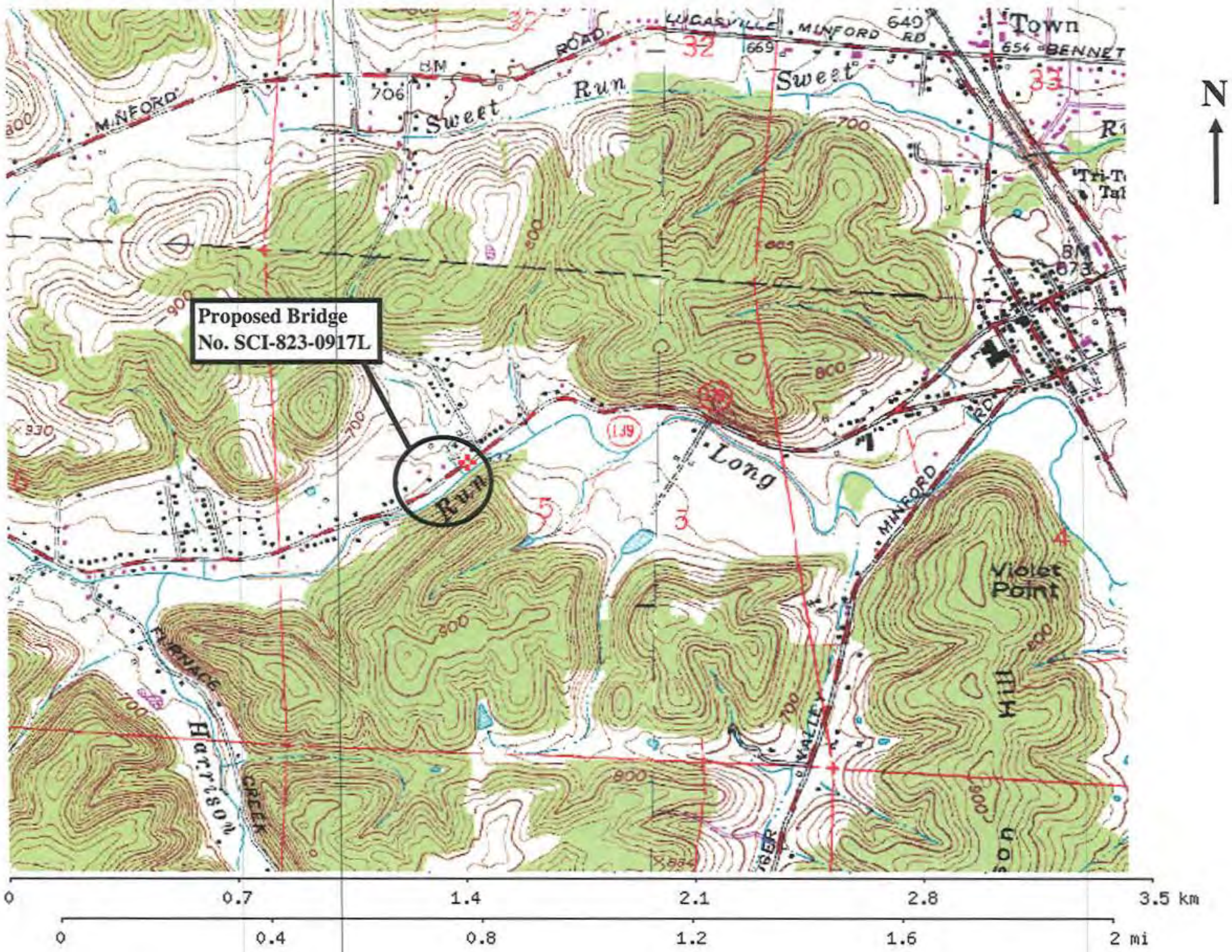
State of Ohio Department of Transportation – Office of Structural Engineering (2006). *"Bridge Design Manual".*

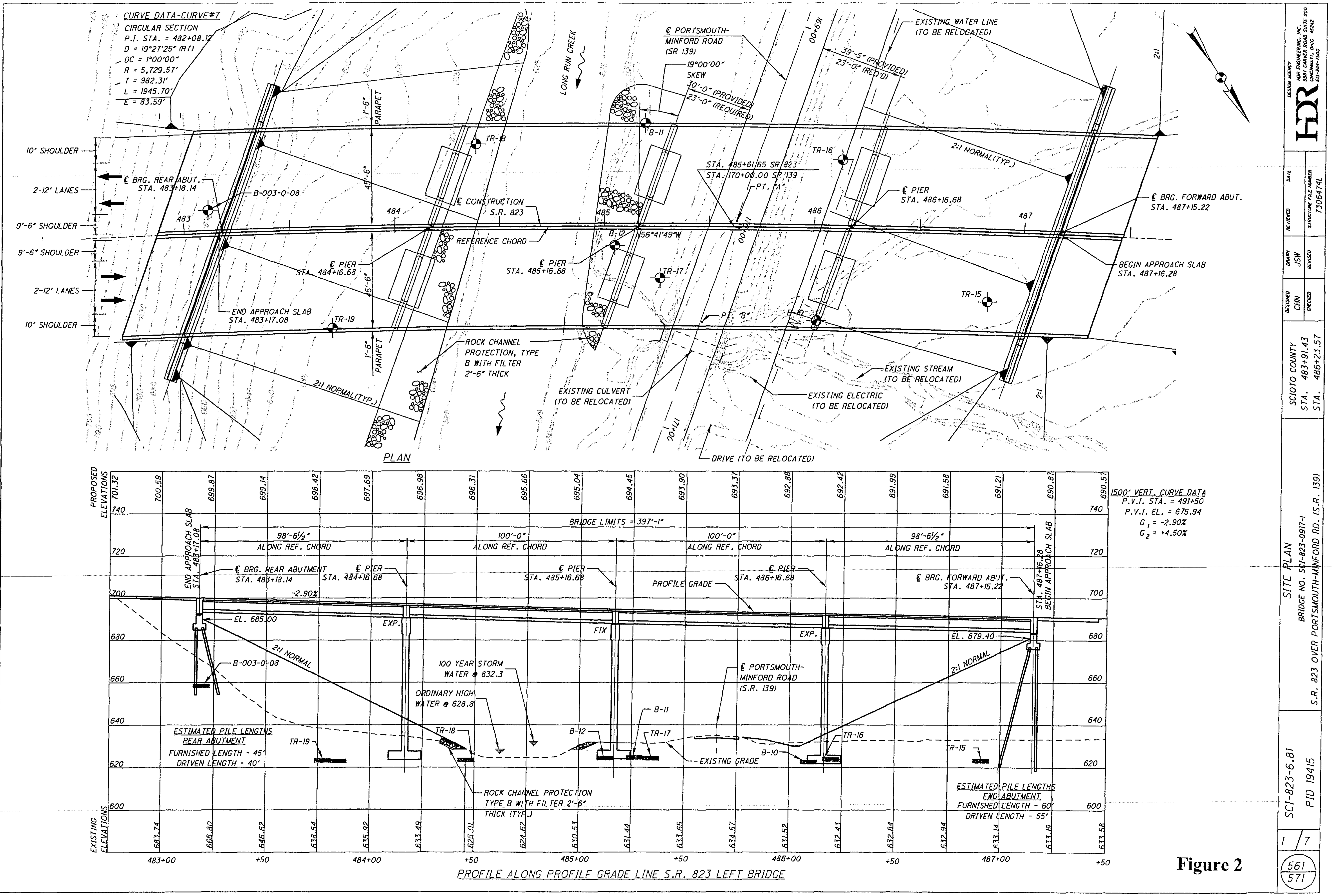
United States Department of Agriculture: Natural Resources Conservation Service (2008). *"Web Soil Survey".* <http://websoilsurvey.nrcs.usda.gov/app/>

Appendices

Appendix A
Figures

Figure 1 - Project Location Map

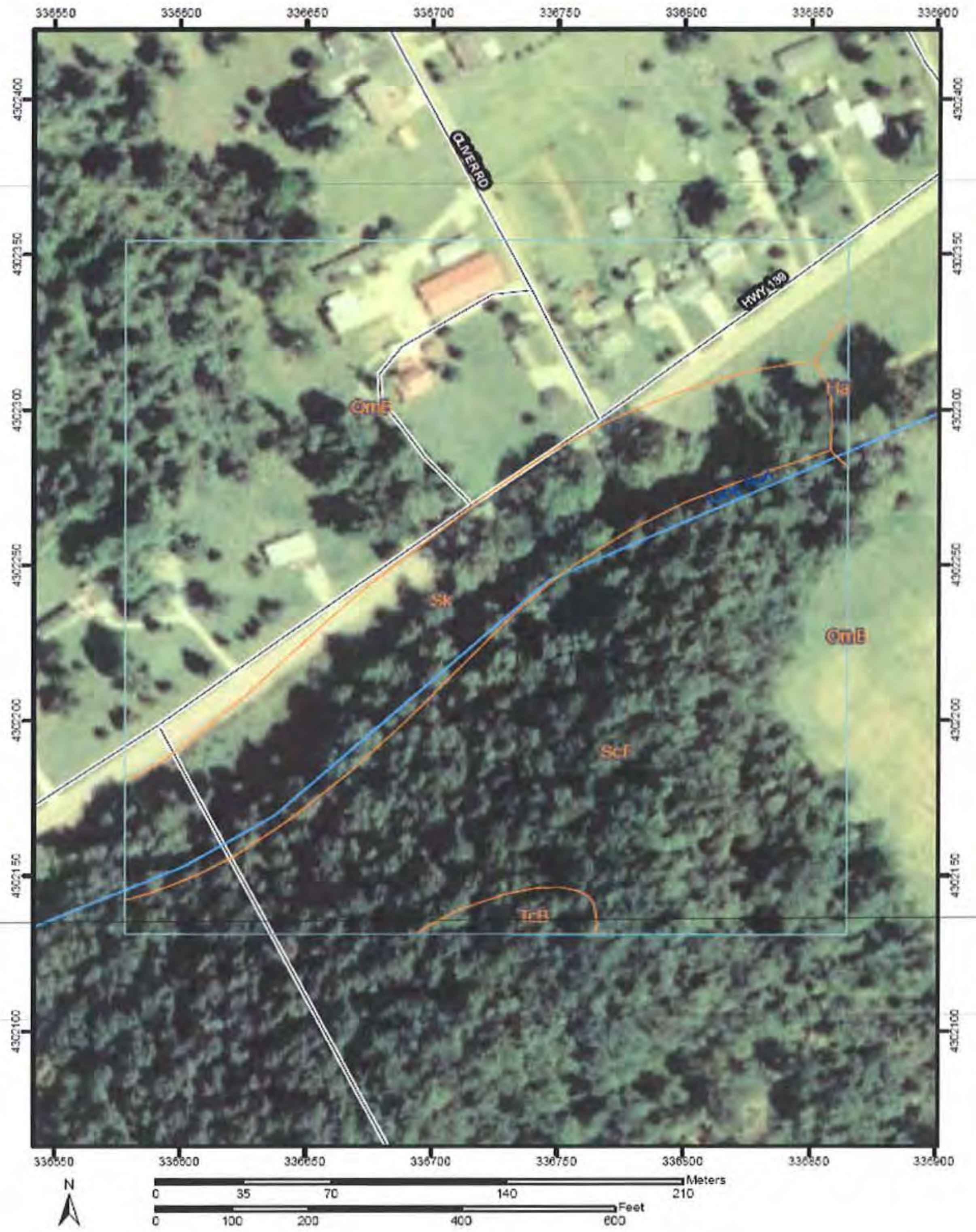




C:\PWWORKING\PITT\DMS84789\823_0917LSP001COMBINED REV1.DGN
4/30/2008 4:48:48 PM

Figure 2

Figure 3: NRCS Soil Map - Scioto County, Ohio



MAP LEGEND

Area of Interest (AOI)
- ☐ Area of Interest (AOI)

Soils
- ☐ Soil Map Units

Special Point Features
- ⊙ Blowout
- ⊠ Borrow Pit
- ✳ Clay Spot
- ◆ Closed Depression
- ✕ Gravel Pit
- ∴ Gravelly Spot
- ⊗ Landfill
- ʌ Lava Flow
- ⚶ Marsh
- ⚒ Mine or Quarry
- ◎ Miscellaneous Water
- ◉ Perennial Water
- ∨ Rock Outcrop
- + Saline Spot
- ∵ Sandy Spot
- ≡ Severely Eroded Spot
- ◇ Sinkhole
- ≫ Slide or Slip
- ∅ Sodic Spot
- ≣ Spoil Area
- ◊ Stony Spot
- ∞ Very Stony Spot
- ▼ Wet Spot
- ▲ Other

Special Line Features
- ∼ Gully
- ⋯ Short Steep Slope
- ∼ Other

Political Features

Municipalities
- ● Cities
- ☐ Urban Areas

Water Features
- 🟦 Oceans
- ∼ Streams and Canals

Transportation
- +++ Rails

Roads
- ∼ Interstate Highways
- ∼ US Routes
- ∼ State Highways
- ∼ Local Roads
- ∼ Other Roads

MAP INFORMATION

Original soil survey map sheets were prepared at publication scale. Viewing scale and printing scale, however, may vary from the original. Please rely on the bar scale on each map sheet for proper map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: UTM Zone 17N

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

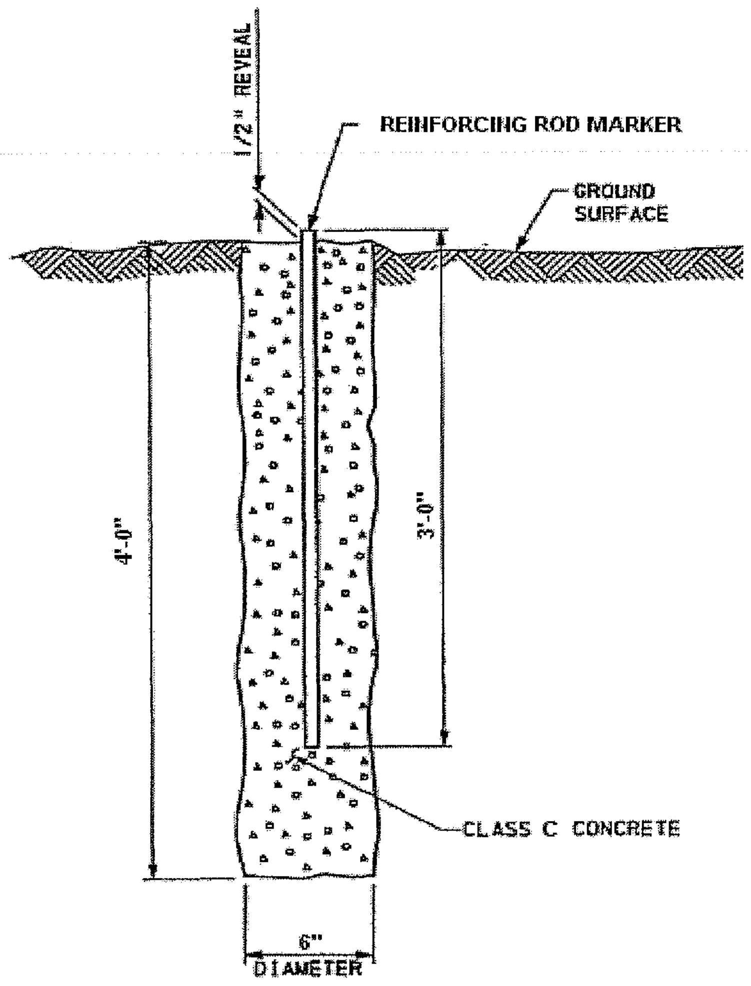
Soil Survey Area: Scioto County, Ohio
Survey Area Data: Version 7, Dec 13, 2007

Date(s) aerial images were photographed: 3/14/1995

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Scioto County, Ohio (OH145)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
Ha	Haymond silt loam, occasionally flooded	0.1	0.6%
OmB	Omulga silt loam, 1 to 8 percent slopes	8.7	40.7%
ScF	Shelocta-Brownsville association, very steep	8.2	38.5%
Sk	Skidmore silt loam, occasionally flooded	4.1	19.0%
TcB	Tilsit-Coolville association, undulating	0.3	1.2%
Totals for Area of Interest (AOI)		21.3	100.0%



NOT TO SCALE

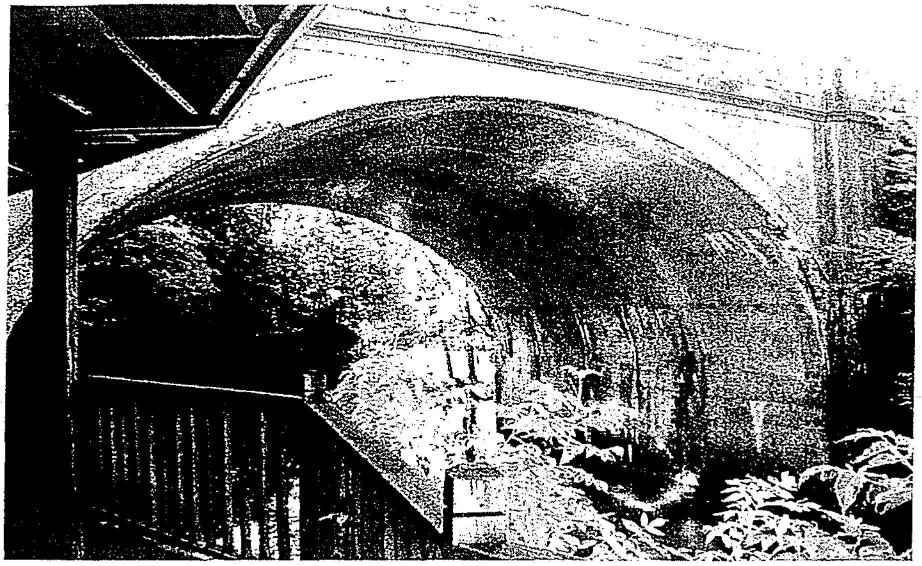
NOTE: EACH MONUMENT TO BE PROTECTED BY A 3 FOOT HIGH, FLOURESCENT PLASTIC, MESHED FENCE PLACED AT A 3 FOOT RADIUS FROM THE MONUMENT.

HDR | ONE COMPANY *Many Solutions*℠

Figure 4 - Surface Monument Detail

Appendix B

"Report of Subsurface Exploration, Bridge and MSE Retaining Walls, SR 823 Over Portsmouth-Minford Road (SR 139), SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio" (DLZ, 2006)



Report of:

Subsurface Exploration
Bridge and MSE Retaining Walls
SR 823 Over Portsmouth-Minford Road (SR 139)
SCI-823-0.00 Portsmouth Bypass
Scioto County, Ohio

Prepared for:

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

Ohio Department of Transportation
District 9

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

DLZ Job No. 0121-3070.03

September 26, 2006

Prepared by:

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

SR 823 OVER PORTSMOUTH-MINFORD ROAD (SR 139)

SCI-823-0.00 PORTSMOUTH BYPASS

SCIOTO COUNTY, OHIO

For:

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

By:

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

DLZ Job. No. 0121-3070.03

September 26, 2006

TABLE OF CONTENTS

REPORT
OF
SUBSURFACE EXPLORATION
FOR
BRIDGE AND MSE RETAINING WALLS
SR 823 OVER PORTSMOUTH - MINFORD ROAD
SCI-823-0.00 PORTSMOUTH BYPASS
SCIOTO COUNTY, OHIO

1.0 INTRODUCTION

This report includes the findings of evaluations of foundations and mechanically stabilized earth (MSE) retaining walls for the structure at the above-referenced location of the project. The findings included in this report pertain to the structure at the intersection of the proposed SR 823 and Portsmouth – Minford Road only. The findings of other structure evaluations will be submitted in separate documents.

The project consists in part of placing two structures for the proposed SR 823 over Portsmouth – Minford Road (SR 139). The two structures as planned, are two-span structures using MSE walls to hold back the roadway embankments and contain the abutments.

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

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

2.0 GENERAL PROJECT INFORMATION

It is understood that the plan location of the bridge structure for proposed SR 823 over Portsmouth – Minford Road (SR 139) has not changed from the approved location, as shown on the Plan and Profile drawing in Appendix I. It is understood that MSE walls will be placed at approximate stations 483+97 and 486+15 to contain the abutments and hold back the roadway embankment for proposed SR 823. Furthermore, it is understood that pile foundations will be used to support the abutments of the proposed structures.

Based upon the structure plan and profile drawing, it is assumed that the maximum height of the embankment at stations 483+97 (Rear Abutment) and 486+15 (Forward Abutment) will be approximately 65 and 61 feet, respectively. Those heights are based upon the maximum difference between the proposed grade and the approximate existing grade along the Portsmouth – Minford Road (SR 139).

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

3.0 FIELD EXPLORATION

The field exploration consisted in part of three final and five preliminary structural borings. Borings B-10 through B-12 were drilled for the final bridge plan, essentially consisting of proposed SR 823 passing over the Portsmouth – Minford Road (SR 139). The borings were drilled between June 20 and 28, 2006. Preliminary structural borings (TR-15 through TR-19) were drilled for a previous design configuration. The preliminary borings were drilled between July 9, 2004 and February 23, 2005. A boring plan is presented in Appendix I. Boring logs for borings TR-15 through TR-19, and B-10 through B-12 are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

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

4.0 FINDINGS

4.1 Geology of the Site

The 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. Soils at the rear abutment location are composed primarily of residual and colluvial soils. These soils are generally thin, covering moderate to steep slopes. At the forward abutment residual and colluvial soils were also encountered. Lacustrine soils have also been encountered on this project. However, no lacustrine soils were encountered in borings near this proposed structure. 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 north and south of the structures roughly above elevation 880. In the area of the structure, the bedrock was covered by a relatively thin soil overburden ranging in thickness between 4.0 and 9.2 feet.

4.2 Subsurface Conditions

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

4.2.1 Soil Conditions

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

Borings TR-15, TR-16, and B-10 were drilled for the west abutment. Borings TR-18 and TR-19 were drilled for the east abutment, while borings TR-17, B-11, and B-12 were drilled for the piers. Borings TR-16, TR-18, TR-19, and B-10 through B-12 are considered most representative of the soil and bedrock in the area of the proposed structures. However, borings TR-15 and TR-17 are included for informational purposes.

All borings except boring TR-16 encountered surficial material consisting of 2 to 12 inches of topsoil. Boring TR-16 encountered native soil at the ground surface level. All borings encountered native cohesive and granular soil deposits below the surficial material or the ground surface. The cohesive deposits consisted mainly of medium stiff to very stiff sandy silt (A-4a) and medium stiff to stiff silt (A-4b), while the granular soil deposits consisted mainly of loose to medium dense gravel with sand (A-2-4), loose to very dense sandy silt (A-4a), and medium dense silt (A-4b). The native soil deposits extended to an approximate depth ranging between 4.0 and 9.2 feet below the ground surface where bedrock was encountered.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in all borings. The bedrock consisted mainly of medium hard to hard, slightly weathered, slightly to moderately fractured sandstone. The amount of rock recovered in each core run varied between 78 and 100 percent with an average of 95 percent. The rock quality designation (RQD) of the bedrock ranged between 57 and 97 percent with an average of 80 percent indicating good rock.

Unconfined compressive strength of tested cores ranged between 9,709 and 11,829 pounds per square inch. The tested cores correspond to samples at depths between 13.0 feet and 25.0 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1.

Table 1-Unconfined Compressive Strength Results

Boring	Depth (ft)	Unconfined Compressive Strength (psi)
B-10	16.5-17.0	10,393
B-11	13.5-14.0	10,537
B-12	24.5-25.0	9,709
B-12	13.0-13.5	11,829

4.2.3 Groundwater Conditions

Seepage was encountered only in borings TR-15, TR-16, and TR-17 between approximate depths of 6.0 and 7.0 feet. There were no measurable water levels in the borings prior to rock coring. Water was used during rock coring and masked any seepage zones that might exist in the rock. Measurable water levels were present in all test borings except borings B-11 and TR-15 upon the completion of coring between approximate depths of 1.6 and 28.5 feet.

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

5.0 CONCLUSIONS AND RECOMMENDATIONS

It is anticipated that the existing bridge will be constructed as described in Sections 1 and 2 of this report. It is understood through comments from ODOT's Office of Structural Engineering that pipe piles will be used to support the abutments. The use of drilled shafts and spread footings has also been considered to support the abutments. In addition, to support the piers, spread footings and drilled shafts bearing on rock have been evaluated. Furthermore, the site is well suited for the use of MSE walls to contain the abutments and hold back the roadway embankment. Recommendations for the piles, drilled shafts, spread footings, and MSE walls are presented in the following sections.

5.1 Bridge Foundation Recommendations

5.1.1 Rear and Forward Abutments

It is understood through comments from the ODOT Office of Structural Engineering (OSE) that pipe piles are to be used to support the abutments. It is understood that the abutments will be supported by steel pipe piles placed in prebored holes 12 inches larger than the diameter of the pile and 5 feet deep into bedrock. After installing the steel pipe pile in the prebored hole, grout or cement should be placed in the void area around the pile in the prebored hole prior to constructing the embankment granular fill (per OSE). Therefore, a pile sleeve may not be required for the installation of the piles. However, consideration should be given to the use of pile sleeves to mitigate down drag effects from compaction and to protect the pile during the embankment and MSE wall construction. The allowable pile capacity as per ODOT BDM 202.2.3.2.b may be utilized in this configuration. Excessive lateral loading and uplift is not anticipated to be a concern at this site. However, if these forces are determined to be significant, longer socket lengths may be required.

Due to the relatively small rigidity of the steel pipe piles compared to drilled shafts, the steel pipe piles are anticipated to provide low lateral resistance to lateral earth pressures that can be induced in high embankment fills such as those at the proposed structure. Therefore, the prebored and socketed steel pipe piles foundation system may be a concern if significant lateral loads are present.

As mentioned above, drilled shafts have also been considered for the support of the abutments. Due to the large amount of embankment fill, it appears that drilled shafts socketed a minimum of 5 feet into competent rock will be well suited for the support of the proposed structural abutments. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 80 ksf (40 tsf).

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

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.

Precautions should be taken to permit the shafts to be drilled and the concrete placed under relatively dry conditions. Some borings did encounter significant seepage at this site. Water could flow into the drilled shafts during installation, particularly below the stream level 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.

Spread footings bearing in the MSE wall fill may also be considered to support the abutments. As per the Bridge Design Manual (BDM) 204.6.2.1 an allowable bearing capacity of 4 ksf may be used to design the footings. The MSE walls as proposed will be founded on or near bedrock. As such, the anticipated settlements of spread footings bearing on the fill are anticipated negligible.

5.1.2 Piers

Spread footings can be constructed on the rock encountered by the borings to support the piers. Competent bedrock was generally encountered within two to three feet of the soil-rock interface. Spread footings bearing on competent bedrock may be designed using an allowable bearing capacity of 80 ksf (40 tsf).

Currently, lateral loading and uplift is not anticipated to be a concern at this site. However, if spread footings cannot be used at the piers, drilled shafts may be considered to support the piers. If drilled shafts are used to support the foundation of the piers, a minimum of 5-foot deep socket into competent rock is required. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 80 ksf (40 tsf).

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

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.

Precautions should be taken to ensure appropriate drilled shaft construction practices are followed. See Section 5.1.1 for more information.

Table 2, on the following page summarizes the site conditions and foundation recommendations. It should be noted that the bedrock surface varies widely across the project area. The approximate bearing elevations presented below indicate the elevations at the boring locations only. Variations in the elevation at which competent bedrock is encountered should be anticipated.

Table 2-Summary of Foundation Recommendations

Structural Element	Structure / Boring	Existing Ground Surface Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	Left / TR-19	633.0	Pipe Piles	618.3*	Pile Capacity^{+}
			Drilled Shafts	618.3*	80 ksf^{++}
			Spread Footings	MSE Fill**	4 ksf
	Right / TR-18	631.3	Pipe Piles	619.0*	Pile Capacity^{+}
			Drilled Shafts	619.0*	80 ksf^{++}
			Spread Footings	MSE Fill**	4 ksf
Pier	Left / B-11	632.7	Spread Footings	624.7***	80 ksf
			Drilled Shafts	619.7*	80 ksf^{++}
	Right / B-12	632.5	Spread Footings	624.0***	80 ksf
			Drilled Shafts	619.0*	80 ksf^{++}
Forward Abutment	Left / TR-16	631.9	Pipe Piles	617.7*	Pile Capacity^{+}
			Drilled Shafts	617.7*	80 ksf^{++}
			Spread Footings	MSE Fill**	4 ksf
	Right / B-10	632.6	Pipe Piles	~~622.6*~~ 617.6	Pile Capacity^{+}
			Drilled Shafts	617.6*	80 ksf^{++}
			Spread Footings	MSE Fill**	4 ksf

* Includes 5-foot socket into competent rock.

** Bearing elevation should be determined by a qualified engineer as the foundation alternative is selected.

*** Assuming competent rock at the soil-rock interface.

$^{+}$ Pile capacity should conform to ODOT BDM 202.2.3.2.

$^{++}$ End bearing capacity only.

5.2 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

It is understood that MSE walls would be used to construct the embankments and contain the abutments. Recommendations for the MSE wall are presented in the following sections. The MSE wall should be constructed per the recommendations presented in this report and in conformance with the manufacturer's specifications.

5.2.1 MSE Walls: General Information

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

A global stability analysis and bearing capacity analysis were performed for the MSE walls at this bridge location in accordance with ODOT and AASHTO guidelines. The MSE walls were also analyzed for sliding and overturning. At the time this report was prepared, it was understood that pipe piles socketed into bedrock would be used at this site to support the bridge abutments. If the foundation type should change, DLZ should be informed so that the analyses may be revised as necessary.

Calculations for bearing capacity, sliding, and overturning as well as the results of the global stability analyses are attached. Other external and internal stability analyses are required for the design of an MSE wall, but are considered outside the scope of this report. The parameters required to perform the stability analyses are presented in Table 3 below. 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. However, 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)	Medium Dense Sandy Silt	120	0	29	0	29
Foundation Soil (Forward Abutment)	Very Soft to Stiff Sandy Silt	120	1000	0	0	29
Foundation Soil (Undercut and Replace)	Compacted Granular Fill	120	0	34	0	34

5.2.2 MSE Wall Evaluations and Recommendations

The MSE wall at the rear abutment (station 483+97) is understood to be approximately 65 feet high. The minimum required embedment depth for this wall is or 3.0 feet assuming that the wall will be bearing on the native soil deposits.

Borings TR-18 and TR-19 were drilled for the rear abutment location. These borings generally encountered cohesionless silt (A-4b) and sandy silt (A-4a) to a depth of 7.3 to 8.7 feet below the ground surface.

Bearing capacity, stability, and global stability calculations have been performed assuming the above parameters. All calculated factors of safety for bearing capacity, sliding, overturning, and global stability were above the minimum recommended values. Therefore, it is recommended that the MSE wall at the rear abutment be built using a minimum embedment of 3.0 feet. Alternatively, soils may be overexcavated to shallow bedrock and replaced with compacted, granular fill to the leveling pad elevation. If soft or highly compressible soils are encountered while excavating for the leveling pad, these soils should be removed and replaced with compacted granular fill. The limits of the "remove and replace" area should extend beyond the edge of the MSE wall/select granular footprint by the depth of the aggregate base as per ODOT BDM Figure 330. The compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to BDM Item 204. For stability, calculations have indicated that a minimum reinforcement length of 0.8H, or 54.8 feet, is required for stability of the proposed MSE wall at this location.

It should be noted that variations in the topography will be encountered within the proposed footprint of the proposed MSE wall, causing the bedrock elevation to vary. Significant rock excavations may be required to accommodate the reinforcing straps for the MSE wall panels. In areas where bedrock is to be excavated, compacted granular fill is to be placed on bedrock, and a level bench must be cut into the rock to place the fill for stability purposes.

In addition, the foundation leveling pad of the MSE wall at the rear abutment is in close proximity to Long Run Creek, which is running essentially parallel to Portsmouth-Minford Road (SR 139). The approximate elevation of bedrock under the MSE wall is 624 feet, which is near the bottom of the creek. If scour and erosion near the toe of the MSE wall are a concern, then slope protection should be provided with riprap. Alternatively, to mitigate the threat of scour the MSE wall may be founded on bedrock, which is approximately 9 feet below the existing ground surface.

The MSE wall at the forward abutment (station 486+15) is understood to be approximately 61 feet high. The minimum required embedment depth for this wall is 3.0 feet.

Borings B-10 and TR-16 were drilled for the forward abutment. These borings generally encountered cohesive silt (A-4b) and sandy silt (A-4a) to a depth of approximately 9.0 feet below the ground surface.

Initial analyses for the MSE wall bearing on natural soils at this location yielded inadequate factors of safety for undrained bearing capacity, undrained sliding, and undrained global stability. Consequently, it is recommended that the soils beneath the proposed MSE wall be overexcavated to bedrock and replaced with compacted, granular fill to the leveling pad elevation. The limits of the "remove and replace" area should extend beyond the edge of the MSE wall/select granular footprint by the depth of the aggregate base as per ODOT BDM Figure 330. The compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to BDM Item 204.

It should be anticipated that variations in the topography may be encountered within the footprint of the proposed MSE wall, causing the bedrock elevations to vary significantly. In areas where compacted granular fill is to be placed on bedrock, a level bench must be cut into the rock to place the fill for stability purposes. A minimum reinforcing length of 0.8H, or 51.3 feet, is required for the MSE wall at this location.

Settlement calculations are not necessary for the MSE walls at this site. The MSE walls will bear on compacted granular fill or bedrock resulting in negligible settlement.

Calculations for bearing capacity, overturning, and sliding are attached for both the native soil and compacted granular fill foundations. A drawing showing the results of the global stability analyses is also attached. Tables 4 and 5, on the following pages summarize the MSE retaining wall parameters and results of analyses.

Table 4-MSE Retaining Wall Parameters and Analyses Results (Rear Abutment) *Natural Soil foundation*

Retained Soil (New Embankment)
Unit Weight = 120 pcf
Coefficient of Active Earth Pressure (K_a) = 0.33
(Based on $\Phi = 30^{o}$)

Sliding along base of MSE wall
Sliding Coefficient (μ)(0.67) = tan 29°(0.67) = 0.37
Use (μ)(0.67) = 0.35 as a maximum value as per AASHTO, BDM, 303.4.1.1

Allowable Bearing Capacity – Undrained Condition
q_{all} = 11,126 psf

Allowable Bearing Capacity – Drained Condition
q_{all} = 11,126 psf

Global Stability
Factor of Safety – Undrained Condition = NA (Sandy Silt – Drained Condition)
Factor of Safety – Drained Condition = 1.9
Factor of Safety – Seismic Condition = 1.8

Estimated Settlement of MSE volume
Total settlement = 0 inches
Differential settlement = 0 <1/100

Full Height of MSE Wall = 65.5 feet
Minimum Embedment Depth = 3.0 feet
Minimum Length of Reinforcement for External Stability = 54.8 feet

Table 5-MSE Retaining Wall Parameters and Analyses Results
(Forward Abutment) ***Compacted Granular Fill Foundation on Bedrock***

Retained Soil (New Embankment)
Unit Weight = 120 pcf
Coefficient of Active Earth Pressure (K_a) = 0.33
(Based on $\Phi = 30^{\circ}$)

Sliding along base of MSE wall
Sliding Coefficient (μ)(0.67) = tan 34°(0.67) = 0.45
Use (μ)(0.67) = 0.55 as a maximum value as per AASHTO, BDM, 303.4.1.1

Allowable Bearing Capacity – Undrained Condition
q_{all} = 21,873 psf

Allowable Bearing Capacity – Drained Condition
q_{all} = 21,873 psf

Global Stability *(Without undercut) [With "remove and replace", on bedrock]*
Factor of Safety – Undrained Condition = (**1.1**) [>1.5]
Factor of Safety – Drained Condition = (1.8) [>1.5]
Factor of Safety – Seismic Condition = (1.7) [>1.3]

Estimated Settlement of MSE volume
Total settlement = 0 inches
Differential settlement = 0 < 1/100

Full Height of MSE Wall = 61.1 feet
Minimum Embedment Depth = 3.0 feet
Minimum Length of Reinforcement for External Stability = 51.3 feet

5.3 Groundwater Considerations

Water seepage was not encountered in any of the borings. Groundwater was not noted prior to adding drill water. Representative final water levels could not be obtained due to the use of water during rock coring. Excavation for the pier foundation is expected to be limited to seven feet or less. Foundation construction on the rock is expected to encounter only minor seepage. Excavations or shafts extending below ground level may encounter more significant seepage through fractured zones in the rock. The contractor should be prepared to deal with seepage and water flow that may enter any excavations.

5.4 Anticipated Sequence of Construction

It is understood through comments from ODOT Office of Structural Engineering (OSE) that pipe piles are to be used to support the abutment. It is also understood that MSE walls will be used to retain the roadway embankment and contain the abutments. A brief outline of the anticipated construction sequence is provided here. This outline is general and is in no way inclusive of all of the procedures and precautions required during the construction process. The contractor is ultimately responsible for implementing sound construction practices to build the MSE wall and pile foundations as per plan and in accordance with ODOT specifications.

- Drill a 5-foot deep socket for each pile into competent bedrock.
- Place the pile into socket and grout or cement annular space in the socket. The unsupported length of piling shall be determined by the contractor. Stability of the unsupported pile must be maintained throughout the construction process. If the full length of the pile isn't installed initially, then splices shall be used.
- Although no appreciable consolidation is anticipated at this site, consideration should be given to the use of pile sleeves to mitigate down drag effects from compaction and to protect the pile during the embankment and MSE wall construction.
- Contractor is responsible for controlling the locations of the piles and ensuring that the locations conform to the plan location. This may be accomplished through bracing or other means.
- Place layers of select fill and/or MSE reinforcing straps per ODOT specifications and the MSE wall supplier's recommendations.
  Splice additional lengths of piling onto "in-place" piles as necessary.

6.0 CLOSING REMARKS

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

Respectfully submitted,

DLZ OHIO, INC.

Steven J. Riedy
Geotechnical Engineer

Wael Alkasawneh, P.E.
Geotechnical Engineer

sjr

M:\proj\0121\3070.03\Stability Analyses\Documents\MSE Wall letters\04 Portsmouth-Minford Road\Final\Portsmouth-Minford Road Structure Report 09-26-06 - sjr.doc

APPENDIX I

Structure Plan and Profile Drawing – 11”x17”
Boring Plan - 11”x17”

CURVE DATA
P.I. STA. = 482+08.11
D = 19°27'25" (RT)
DC = 1°00'00"
R = 5,729.57'
T = 982.31'
L = 1945.70'
E = 83.59'

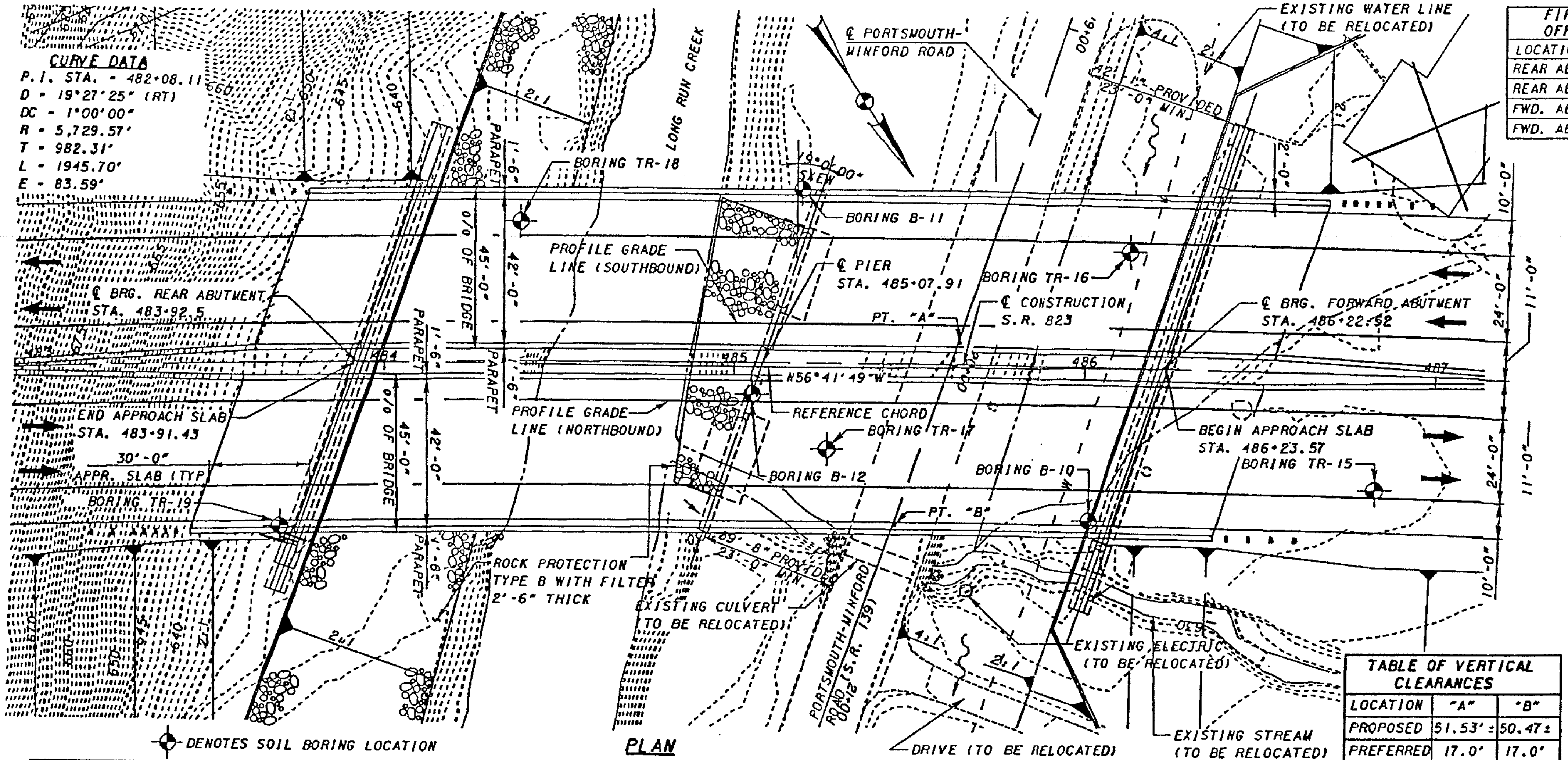
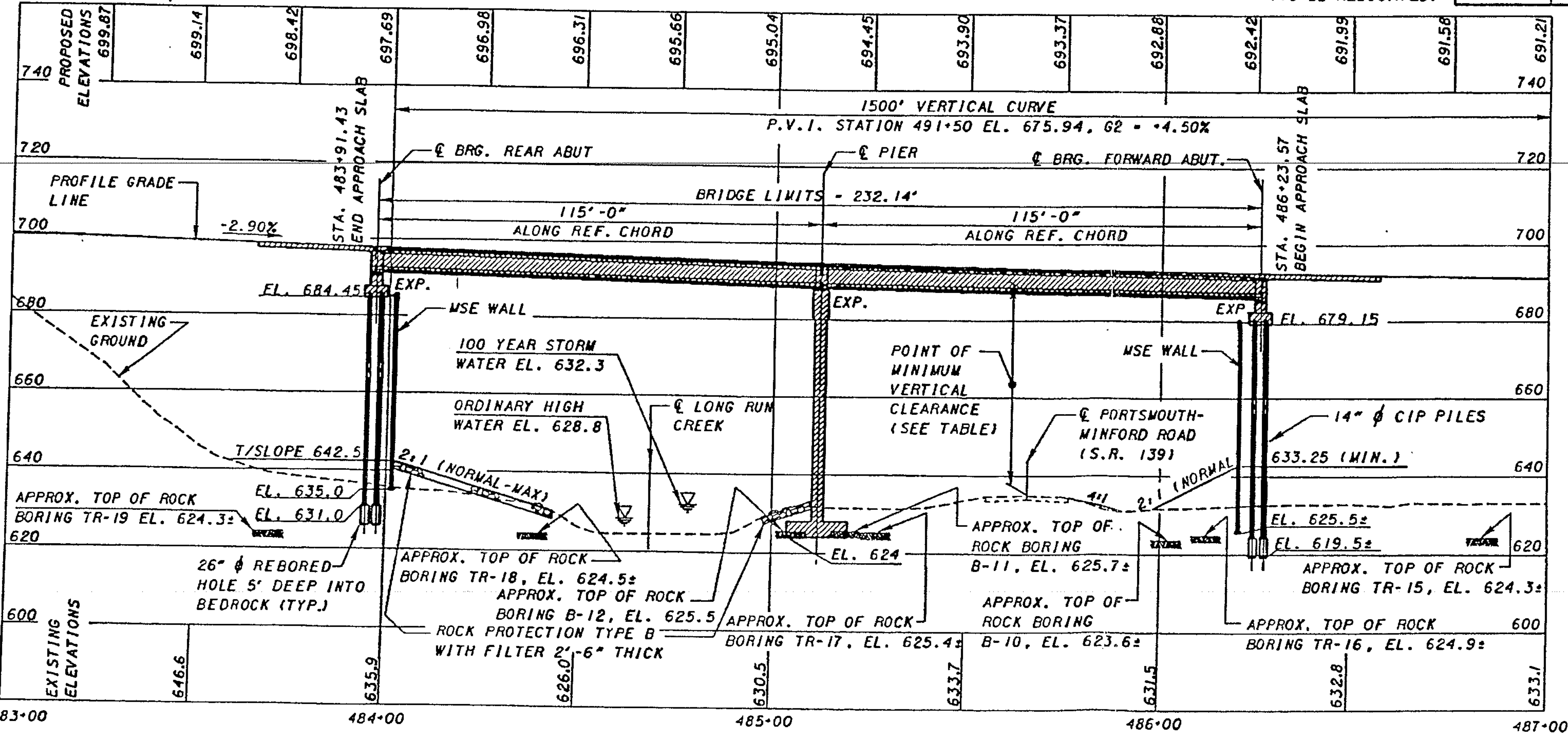


TABLE OF VERTICAL CLEARANCES		
LOCATION	"A"	"B"
PROPOSED	51.53'±	50.47'±
PREFERRED	17.0'	17.0'

PLAN



PROFILE ALONG LEFT PROFILE GRADE LINE

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	OFFSET
REAR ABUT.	483+39.53	47.00 RT.
REAR ABUT.	483+74.57	47.00 LT.
FWD. ABUT.	486+42.87	47.00 RT.
FWD. ABUT.	486+72.41	47.00 LT.

BORING LOCATIONS		
BORING No.	STATION	OFFSET
TR-15	486+82.23	36.94' RT.
TR-16	486+12.38	32.33' LT.
TR-17	485+26.88	46.98' RT.
TR-18	484+42.66	46.98' LT.
TR-19	483+60.89	42.99' RT.
B-10	486+01.99	43.79' RT.
B-11	485+18.99	48.57' LT.
B-12	485+04.99	8.98' RT.

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

TRAFFIC DATA

(SR 823)
CURRENT YEAR ADT (2010) = 19,800
DESIGN YEAR ADT (2030) = 26,000
CURRENT YEAR ADTT (2010) = 2,770
DESIGN YEAR ADTT (2030) = 3,640

HYDRAULIC DATA

DRAINAGE AREA = 13.424 sq.mi. = 8591 acres

Q_{50} = 2230 cfs	Q_{100} = 2572 cfs
V_{50} = 6.8 fps	V_{100} = 7.1 fps
EL_{50} = 631.7	EL_{100} = 632.1

OHWM: EL. 628.8
AREA BELOW OHWM: 0.13 ACRES
TEMP. FILL BELOW OHWM: 835 CY

NOTES:

1. ALL SHEETS WITH PLAN DIMENSIONS ARE SHOWN HORIZONTAL.
2. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
3. THE PROPOSED PROFILE GRADE IS WITHIN BRIDGE LIMITS. SEE ROADWAY PLANS FOR PAVEMENT ELEVATIONS BEYOND BRIDGE LIMITS.

FOUNDATION DATA:

ALL NEW PILES SHALL BE 14" ϕ CIP PILES AND HAVE A MAXIMUM CAPACITY OF 70 TONS PER PILE. SPREAD FOOTINGS SHALL HAVE AN ALLOWABLE BEARING CAPACITY OF 15 TSF.

PROPOSED STRUCTURE

TYPE: 2 SPAN 72" MODIFIED AASHTO TYPE 4 PRESTRESSED CONCRETE I-BEAMS WITH COMPOSITE REINFORCED CONCRETE DECK ON SEMI-INTEGRAL ABUTMENTS, T-TYPE PIERS, AND MSE WALL SUPPORTED EMBANKMENT.

SPANS: 115'-0", 115'-0" C/C BEARINGS (MEASURED ALONG REF. CHORD)

ROADWAY: 2-42'-0" TOE TO TOE OF PARAPETS.

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

SKEW: 19°00'00" (LF) WITH RESPECT TO REF. CHORD.

SUPER ELEVATION: 0.036 FT/FT.

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

WEARING SURFACE: 1" MONOLITHIC CONCRETE.

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

LATITUDE: 38° 51' 30"

LONGITUDE: 82° 53' 00"

DESIGN AGENCY
Systems
(address illegible)

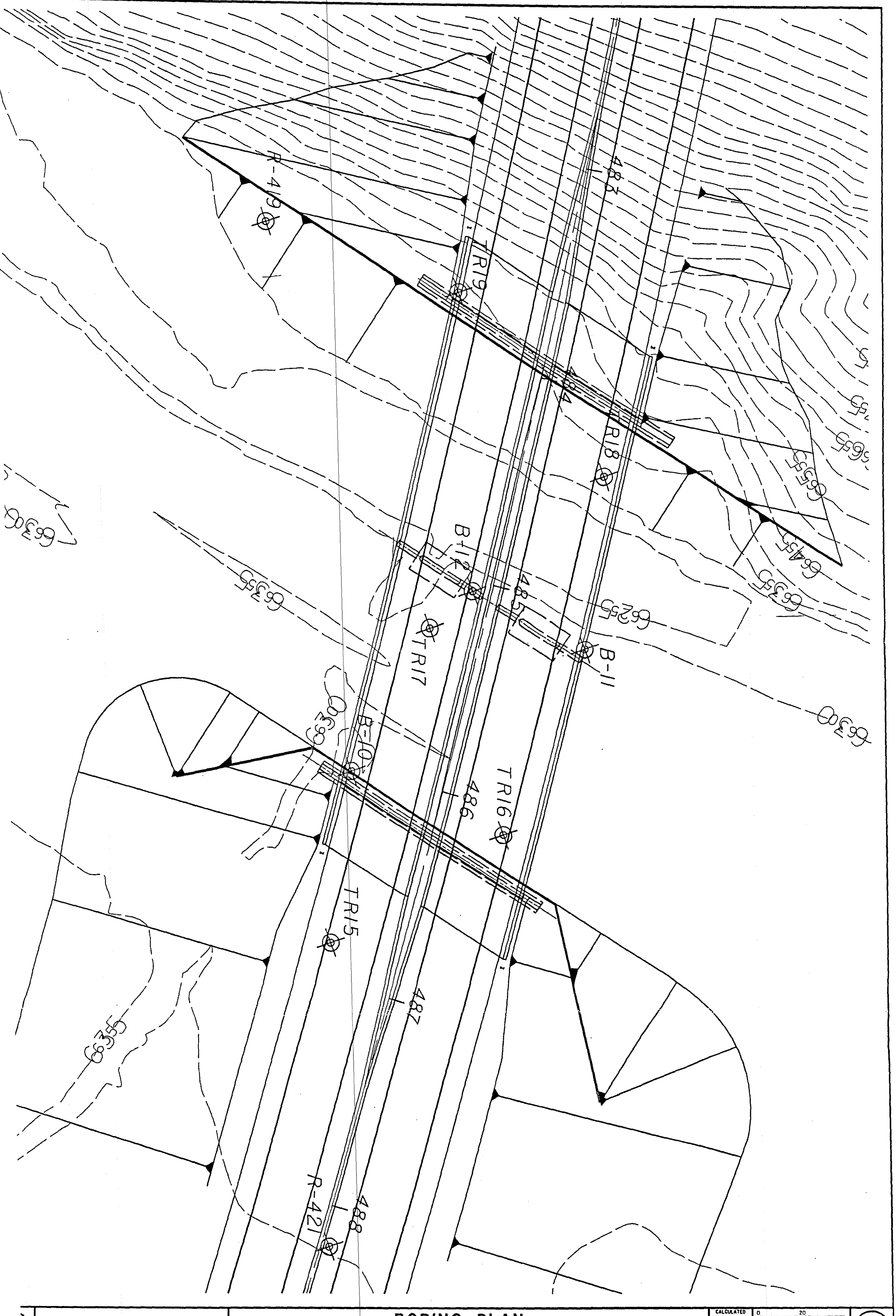
DESIGNED	DRAWN	REVIEWED	DATE
PJP	AWB	JRC	4/13/06
CHECKED MSL	REVISED	STRUCTURE FILE NUMBER	

SCIOTO COUNTY
STA. 483+91.43
STA. 486+23.57

SITE PLAN

BRIDGE NO. SCI-0814-R
S.R. 823 OVER PORTSMOUTH-MINFORD RD. (S.R. 139)

SCI-823-0.00
PID 19415



SCI-823

BORING PLAN
SR 823 OVER PORTSMOUTH-MINFORD ROAD

CALCULATED
CHECKED

0 10 20 40
HORIZONTAL
SCALE IN FEET

N

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Nine (9) Borings

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

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

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

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

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

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

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

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

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

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

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

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

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

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

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

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

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

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

9. Soil Description

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

   Granular Soils – Compactness

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

   Cohesive Soils – Consistency

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

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

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

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

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

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

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

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

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

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

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

10. Rock Hardness and Rock Quality Designation

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

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

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

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

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

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

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

ent: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

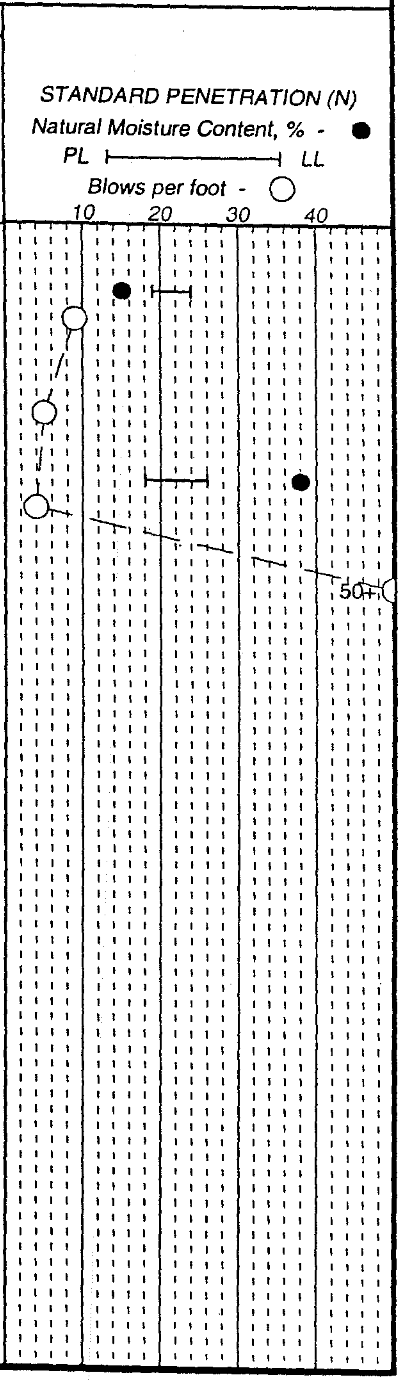
OG OF: Boring B-10

Location: Sta. 486+01.5, 43.8' RT of SR 823 CL

Date Drilled: 06/28/06

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

epth (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	632.6												
0.3	632.3						Topsoil - 3"						
		4 5 4	14	1		1.5	Stiff brown SILT (A-4b), little clay, trace to little fine sand; damp.	0	0	--	10	74	16
5		3 3 2	17	2		--							
		2 2 2	13	3		2.0	@ 6.0'-7.5', soft, wet.	0	0	--	11	74	15
8.5	624.1	10 50/2	6	4			Severely weathered gray SANDSTONE.						
9.5 10	623.1						Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, laminated to thinly bedded, moderately fractured.						
15		Core 120"	Rec 116"	RQD 87%	R-1								
							@ 16.5', qu = 10,393 psi.						
19.5 20	613.1						Bottom of Boring - 19.5'						
25													
30													



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

nt: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

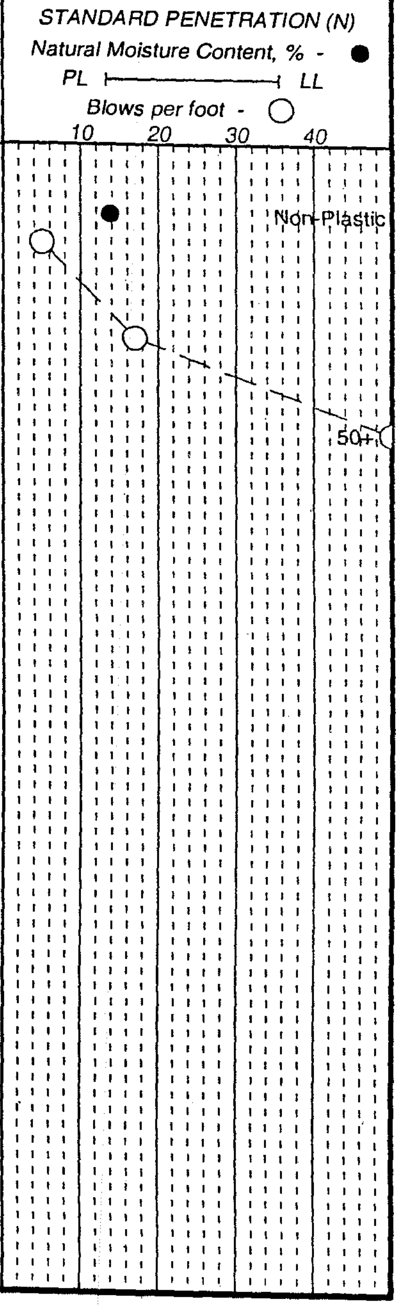
G OF: Boring B-11

Location: Sta. 485+19.1, 48.6 ft. LT of SR 823 CL

Date Drilled: 6/20/06

WATER OBSERVATIONS:
Water seepage at: none
Water level at completion: not reported

epth (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
	632.7												
0.2	632.5						Topsoil - 2"						
		3 3 2	13	1			Loose brown SANDY SILT (A-4a), trace clay; damp.	0	1	--	17	82	
4.0	628.7	4 8 9	15	2			Medium dense gray SANDY SILT (A-4a) ; damp. (Decomposed Rock)						
5													
		7 15 50/3	15	3			@ 8.5', auger refusal.						
8.5	624.2						Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, laminated to thinly bedded, moderately fractured. @ 8.5' to 10.0', highly fractured to broken.						
10													
15		Core 120"	Rec 93"	RQD 57%	R-1		@ 13.5', qu = 10,537 psi.						
20													
25		Core 120"	Rec 116"	RQD 97%	R-2								
28.5	604.2						Bottom of Boring - 28.5'						
30													



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

ient: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

OG OF: **Boring B-12**

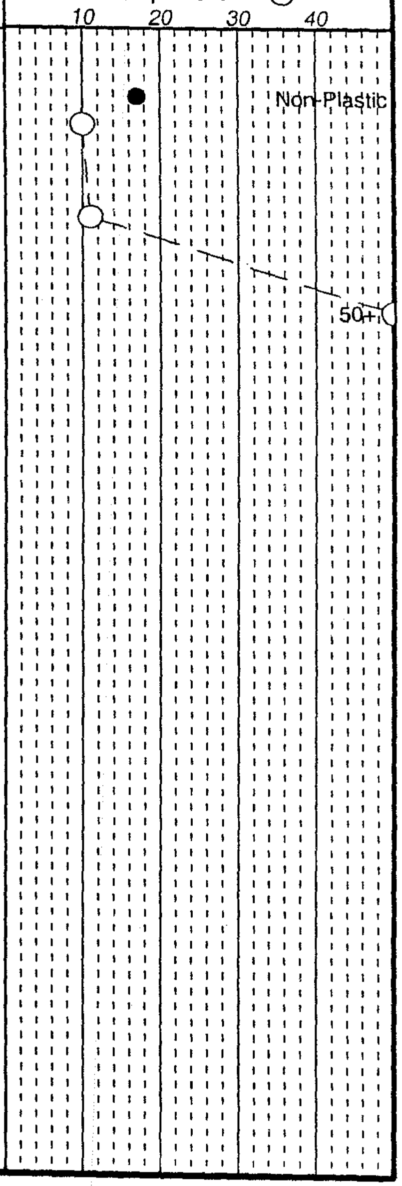
Location: Sta. 485+04.7, 9.0 ft. RT of SR 823 CL

Date Drilled: 6/20/06

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	632.5												
0.3	632.2						Topsoil - 3"						
		3 4 6	15	1			Loose to medium dense brown SANDY SILT (A-4a), trace clay; contains sandstone fragments; damp.	0	0	--	8	92	
4.0	628.5	10 7 4	9	2			Loose to medium dense reddish brown GRAVEL WITH SAND AND SILT (A-2-4); contains sandstone fragments; damp.						
5													
		4 5 50/3	15	3									
8.5	624.0						Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, laminated to thinly bedded, moderately fractured.						
10													
		Core 120"	Rec 97"	RQD 67%	R-1		@ 13.0', qu = 11,829 psi.						
15													
20													
		Core 120"	Rec 120"	RQD 85%	R-2								
25							@ 24.5', qu = 9,709 psi.						
28.5	604.0						Bottom of Boring - 28.5'						
30													

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



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

nt: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

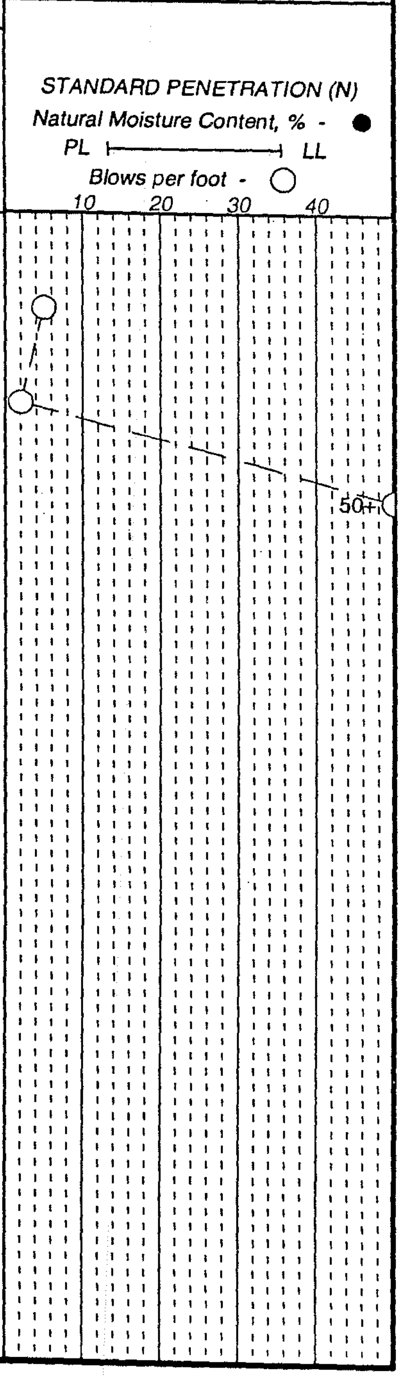
G OF: Boring TR-15

Location: Sta. 486+83.3, 32.9 ft. RT of SR 823 CL

Date Drilled: 7/9/2004

WATER OBSERVATIONS:
Water seepage at: 6.0'
Water level at completion: None

pth 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
	631.3												
.2	631.1						Topsoil - 2"						
		4 2 3	14	1		1.0	Stiff to very stiff brown SANDY SILT (A-4a), trace gravel; moist.						
5		2 1 1	13	2		<0.25	@ 3.5'-5.0', very soft.						
7.0	624.3	3 20 50/2	13	3A 3B		3.25	Severely weathered brownish-gray SILTSTONE fragments.						
8.0	623.3						Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, massively bedded, slightly fractured.						
10													
		Core 120"	Rec 99"	RQD 70%	R-1		@ 8.0'-9.0', probable core loss.						
15													
18.0	613.3						Bottom of Boring - 18.0'						
20													
25													
30													



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

nt: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

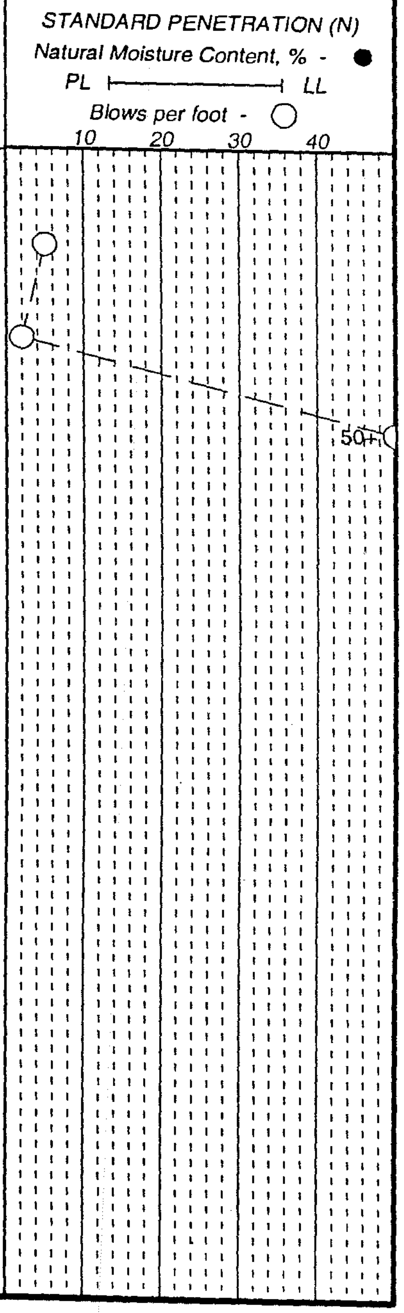
G OF: Boring TR-16

Location: Sta. 486+12.4, 32.3 ft. LT of SR 823 CL

Date Drilled: 7/9/04

WATER OBSERVATIONS: Water seepage at: 6.0'
Water level at completion: 6.5'

pth (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
	631.9												
.2	631.7						Topsoil - 2"						
		2 3 2	16	1		1.0	Medium stiff brown SANDY SILT (A-4a); moist.						
5		1 1 1	15	2		0.75							
		4 10 50/5	12	3		--	@ 6.0' to 7.4', contains rock fragments.						
8.5	623.4						Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, micaceous, argillaceous, massively bedded, slightly fractured.						
10													
		Core 120"	Rec 118"	RQD 85%	R-1								
15													
							@ 17.0', contains few argillaceous laminations.						
18.5	613.4						Bottom of Boring - 18.5'						
20													
25													
30													



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

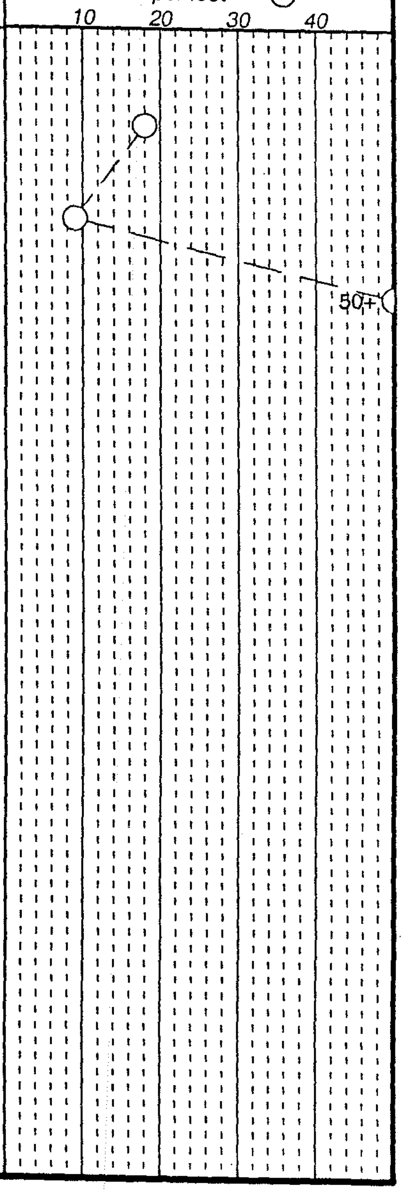
nt: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

G OF: Boring TR-17
Location: Sta. 485+26.9, 24.3 ft. RT of SR 823 CL
Date Drilled: 2/23/2005

WATER OBSERVATIONS:
Water seepage at: 6.3'-7.0'
Water level at completion: 1.6' (inside hollowstem augers, includes drilling water)

epth ('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	631.7						Topsoil - 5"						
.4	631.3	6 8 10	18	1			Medium dense brown SILT (A-4b), little fine to coarse sand, trace clay; damp.						
3.0	628.7	6 4 5	18	2			Loose brown GRAVEL WITH SAND AND SILT (A-2-4); damp.						
5.5	626.2						Very dense brown SANDY SILT (A-4a); wet.						
6.3	625.4	3 50/5	11	3A 3B			Severely weathered gray SANDSTONE.						
7.0	624.7	Core 120"	Rec 120"	RQD 83%	R-1		Medium hard brown and gray SANDSTONE; fine grained, moderately weathered, slightly micaceous, slightly fractured. @ 7.3'-7.4', very soft, highly weathered. @ 8.5', irregular fracture. @ 8.7', gray. @ 16.0', 1" soft, weathered zone.						
17.0	614.7	Core 120"	Rec 120"	RQD 97%	R-2		Hard brown and gray SANDSTONE; fine grained, slightly weathered, slightly micaceous, slightly fractured. @ 22.8'-23.0', very soft, highly weathered siltstone seam. @ 23.0'-23.2', siltstone seam.						
27.0	604.7						Bottom of Boring - 27.0'						

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



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

ent: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

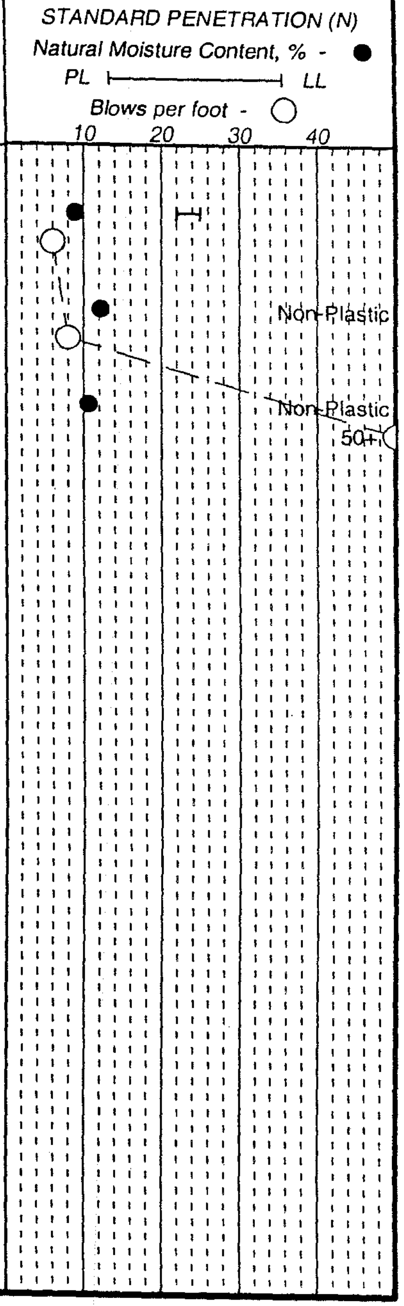
OG OF: Boring TR-18

Location: Sta. 484+38.6, 39.0 ft. LT of SR 823 CL

Date Drilled: 8/17/04

WATER OBSERVATIONS:
Water seepage at: None
Water level at completion: 9.4' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	631.3						Topsoil - 12"						
-1.0	630.3	2 3 3	18	1			Medium stiff brown SILT (A-4b), little clay, little fine to coarse sand, little gravel; contains roots; dry to damp.	13	7	--	9	58	13
-3.0	628.3	3 4 4	18	2			Loose brown SANDY SILT (A-4a), little clay, trace to little gravel; damp.	0	3	--	40	45	12
		6 7 50/3	12	3				11	20	--	28	31	10
-7.3	624.0	Core 84"	Rec 84"	RQD 88%	R-1		Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, slightly to moderately fractured. @ 7.3'-7.6', broken. @ 7.3'-7.8',8.0',8.6'-8.8', brown, rust-stained fractures. @ 7.3'-7.8', vertical fracture.						
		Core 72"	Rec 71"	RQD 94%	R-2								
-20.3	611.0						Bottom of Boring - 20.3'						



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

nt: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

G OF: Boring TR-19

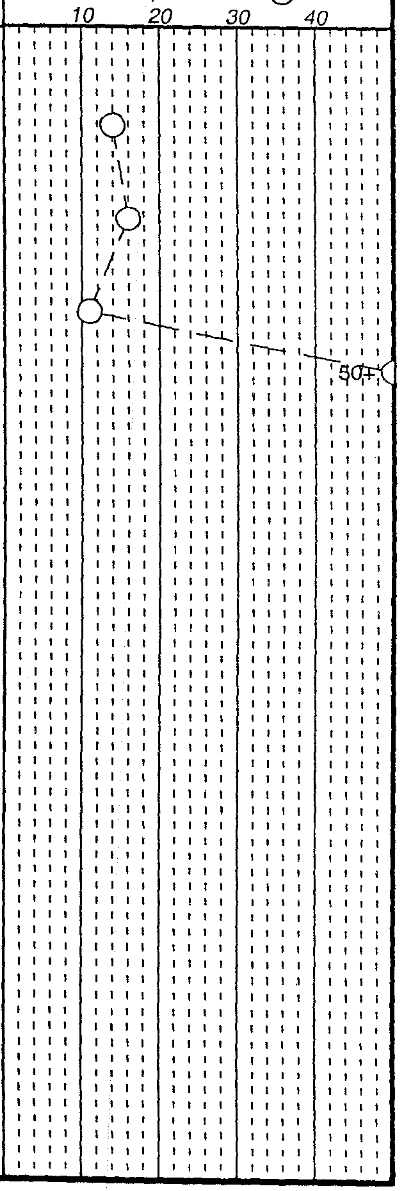
Location: Sta. 483+69.8, 46.5 ft. RT of SR 823 CL

Date Drilled: 8/16/04 to 8/17/04

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 16.3' (includes drilling water)

pth t)	*Elev.* *(ft)*	*Blows per 6"*	*Recovery (in)*	*Sample No.*		*Hand Penetro-meter (tsf) / * Point-Load Strength (psi)*	*DESCRIPTION*	*GRADATION*					
				Drive	*Press / Core*			*% Aggregate*	*% C. Sand*	*% M. Sand*	*% F. Sand*	*% Silt*	*% Clay*
0	633.0						Topsoil - 12"						
.0	632.0	3 7 7	18	1			Medium dense brown SANDY SILT (A-4a), trace gravel, trace clay; contains sandstone fragments; damp.						
5		4 7 9	18	2									
		4 5 6	18	3									
8.7	624.3	50/2	2	4			Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, massively bedded, slightly fractured. @ 9.2'-9.4', decomposed. @ 8.8'-9.0', brown. @ 13.1'-13.3', vertical fracture.						
10		Core 30"	Rec 30"	RQD 57%	R-1								
15		Core 108"	Rec 108"	RQD 70%	R-2		@ 13.9'-14.0', vertical fracture. @ 15.5', unfractured to slightly fractured. @ 14.7'-15.5', broken zone. @ 15.4'-15.5', clay filled fracture.						
20.2	612.8						Bottom of Boring - 20.2'						
25													
30													

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



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: 9/14/2006

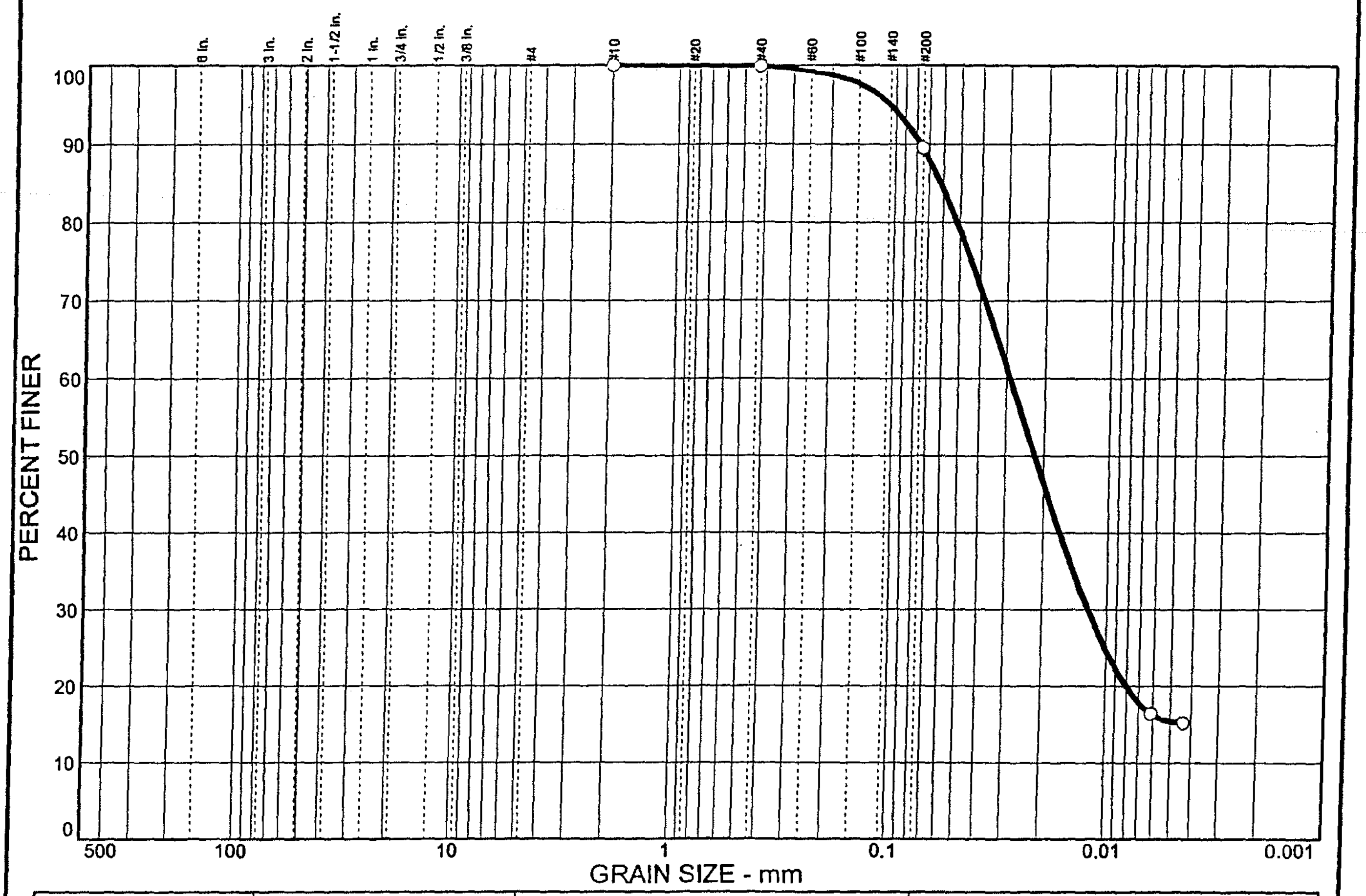
Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
B-10	2	16.5-17.0	1.861	1.865	1.865	1.864	4.665	4.656	4.652	4.658	2.499	0.0073493	523.59	157.07	28,240	**10,393**
			1.861	1.864	1.866											
B-11	1	13.5-14.0	1.555	1.866	1.869	1.815	4.468	4.468	4.471	4.469	2.462	0.0066869	491.85	162.16	28,630	**10,537**
			1.863	1.868	1.868											
B-12	2	24.5-25.0	1.865	1.855	1.861	1.862	4.497	4.505	4.487	4.496	2.415	0.0070808	513.72	159.95	26,380	**9,709**
			1.865	1.861	1.864											
B-12	1	13.0-13.5	1.867	1.867	1.869	1.867	4.615	4.615	4.612	4.614	2.472	0.0073039	490.11	147.94	32,140	**11,829**
			1.866	1.865	1.866											

DLZ

Engineers * Architects * Scientists

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.1	10.4	74.1	15.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	89.5		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 19 LL= 24 PI= 5

Coefficients

D_{85}= 0.0618 D_{60}= 0.0288 D_{50}= 0.0220
D_{30}= 0.0122 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

Moisture Content= 15.1%

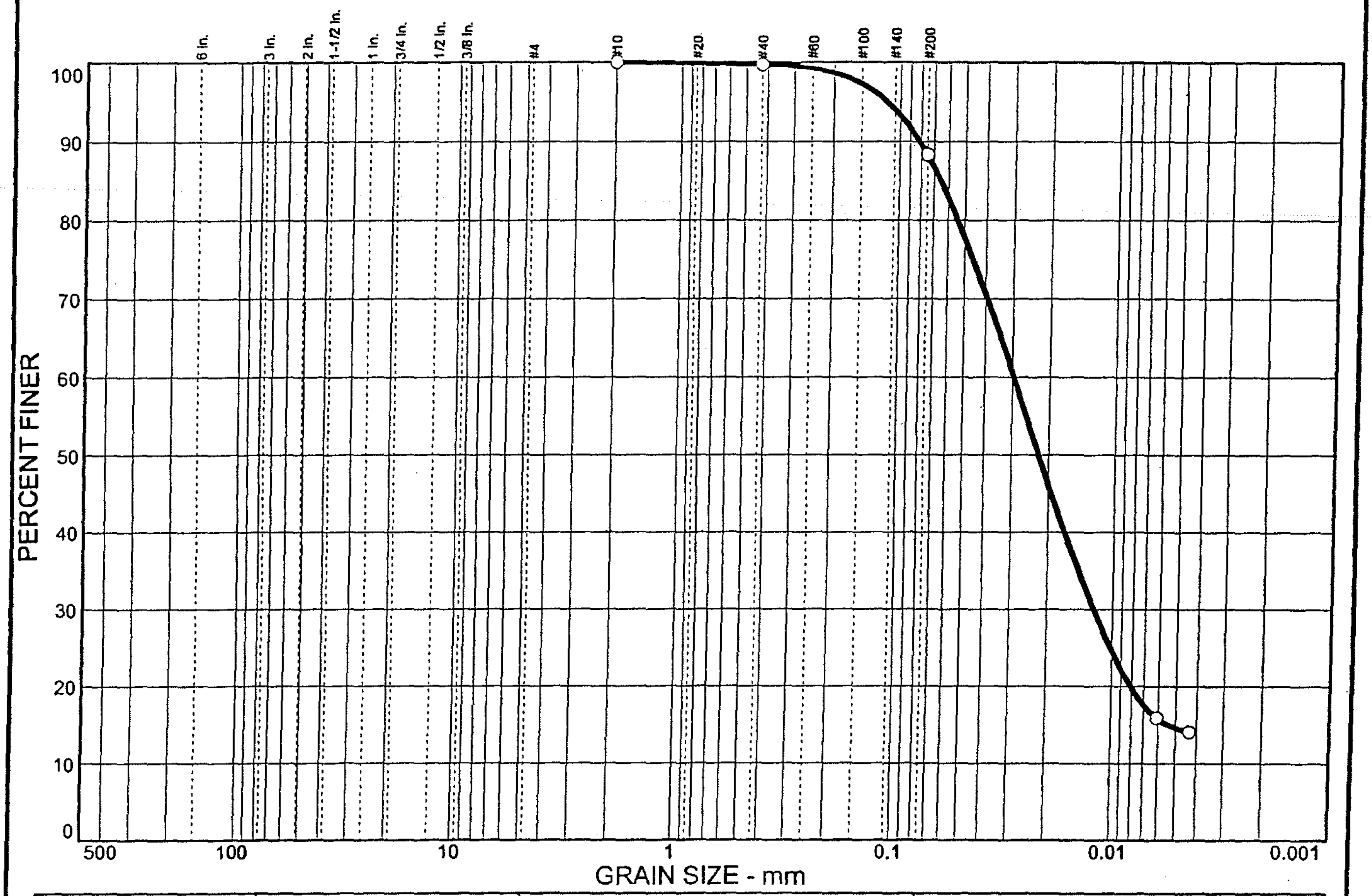
Sample No.: 1 **Source of Sample:** B-10 **Date:** 7/21/06
Location: **Elev./Depth:** 1.0

DLZ

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

Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.3	11.4	73.7	14.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.7		
#200	88.3		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 18 LL= 26 PI= 8

Coefficients

D_{85}= 0.0650 D_{60}= 0.0295 D_{50}= 0.0223
D_{30}= 0.0122 D_{15}= 0.0054 D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(5)

Remarks

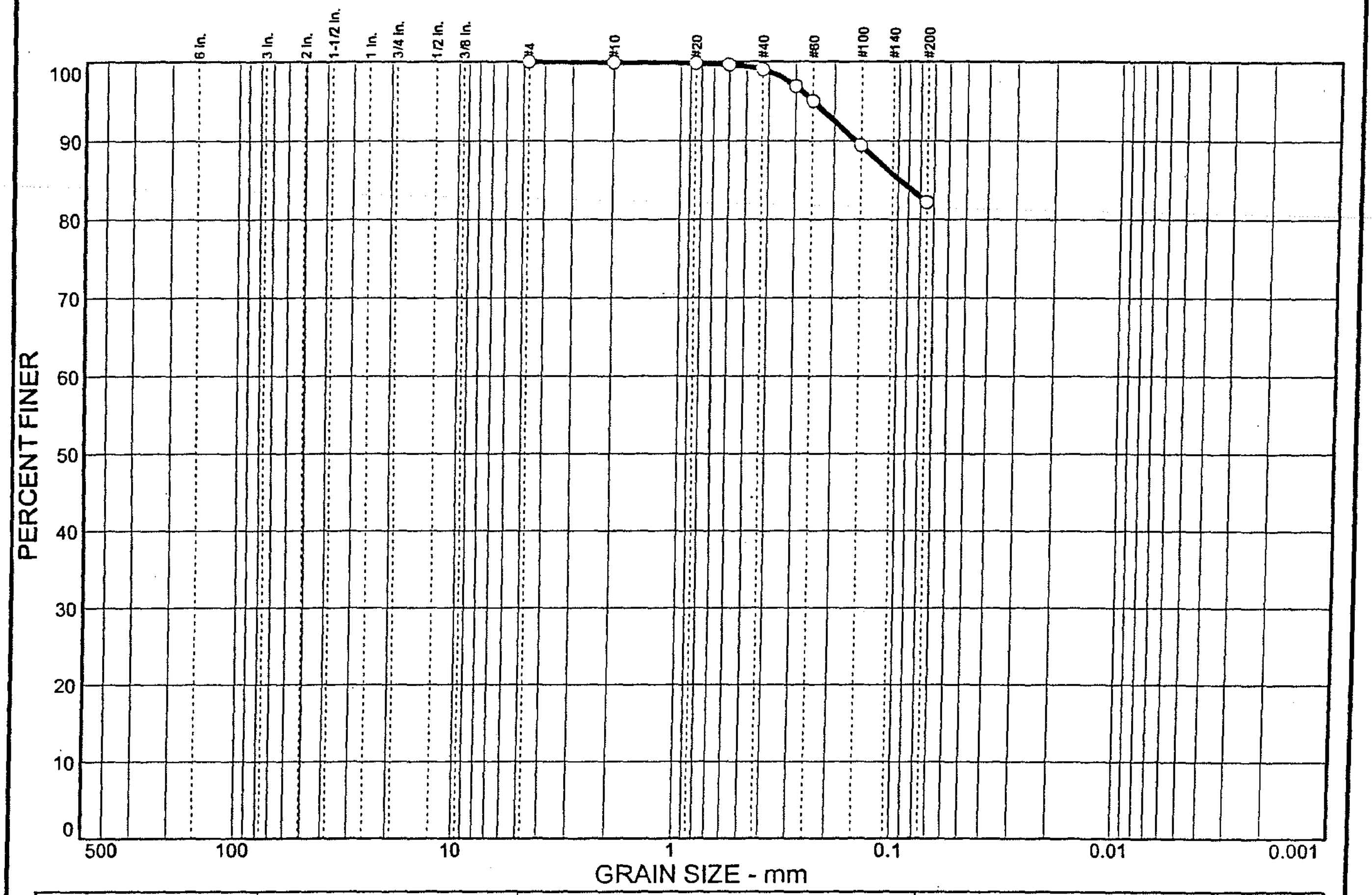
Moisture Content= 38.0%

Sample No.: 3 **Source of Sample:** B-10 **Date:** 7/21/06
Location: **Elev./Depth:** 6.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.1	0.9	16.8	82.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#20	99.8		
#30	99.6		
#40	99.0		
#50	96.9		
#60	95.0		
#100	89.4		
#200	82.2		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.0986 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

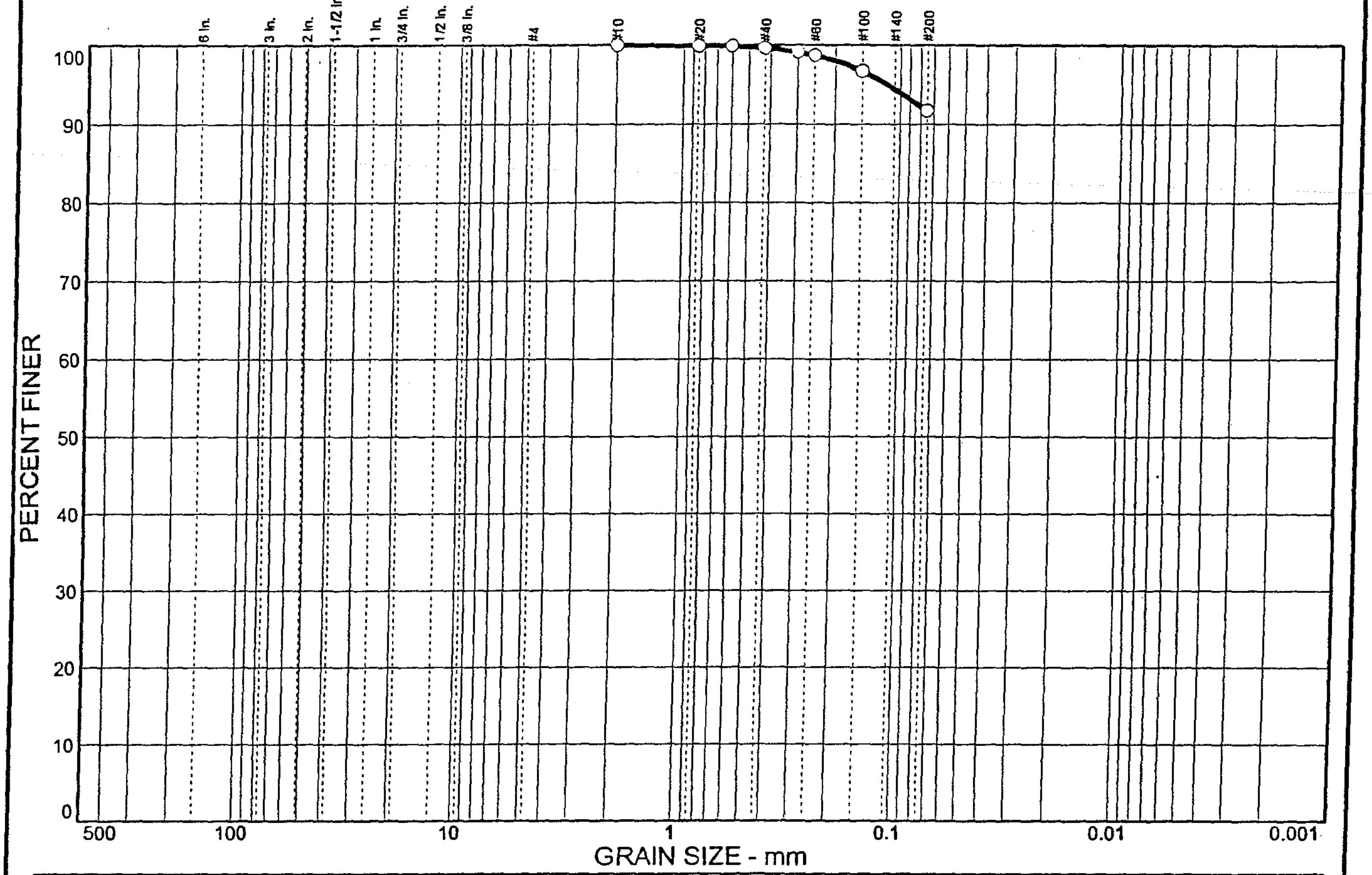
Moisture Content = 13.8%

Sample No.: 1 **Source of Sample:** B-11 **Date:** 07/12/06
Location: **Elev./Depth:** 1.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.4	7.9	91.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.9		
#30	99.9		
#40	99.6		
#50	99.1		
#60	98.7		
#100	96.7		
#200	91.7		

* (no specification provided)

Soil Description

Silt

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

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

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content = 17.0%

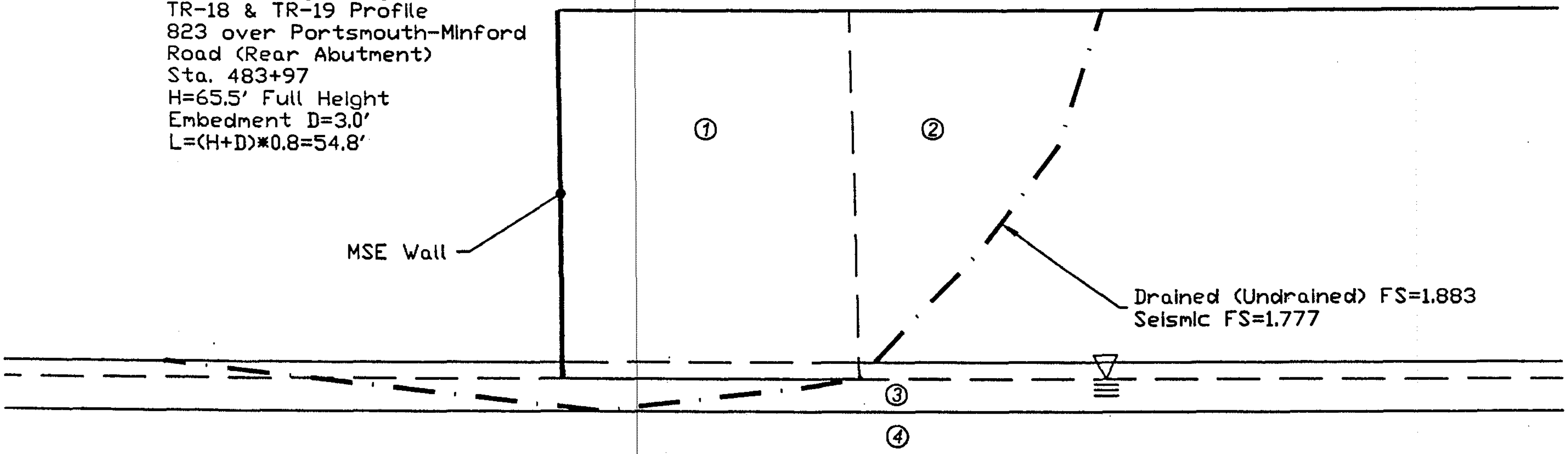
Sample No.: 1 **Source of Sample:** B-12 **Date:** 07/12/06
Location: **Elev./Depth:** 1.0

DLZ

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

			Undrained		Drained		
Material	Consistency	Soil Type	C (psf)	ϕ (deg)	C' (psf)	ϕ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Medium Dense	Silt/Sandy Silt	0	29	0	29	120
Material 4	Hard	Sandstone	10000	45	10000	45	145

MSE Stability Analysis
TR-18 & TR-19 Profile
823 over Portsmouth-Minford
Road (Rear Abutment)
Sta. 483+97
H=65.5' Full Height
Embedment D=3.0'
L=(H+D)*0.8=54.8'



823 OVER PORTSMOUTH - MINFORD ROAD
REAR ABUTMENT

MSE STABILITY ANALYSIS

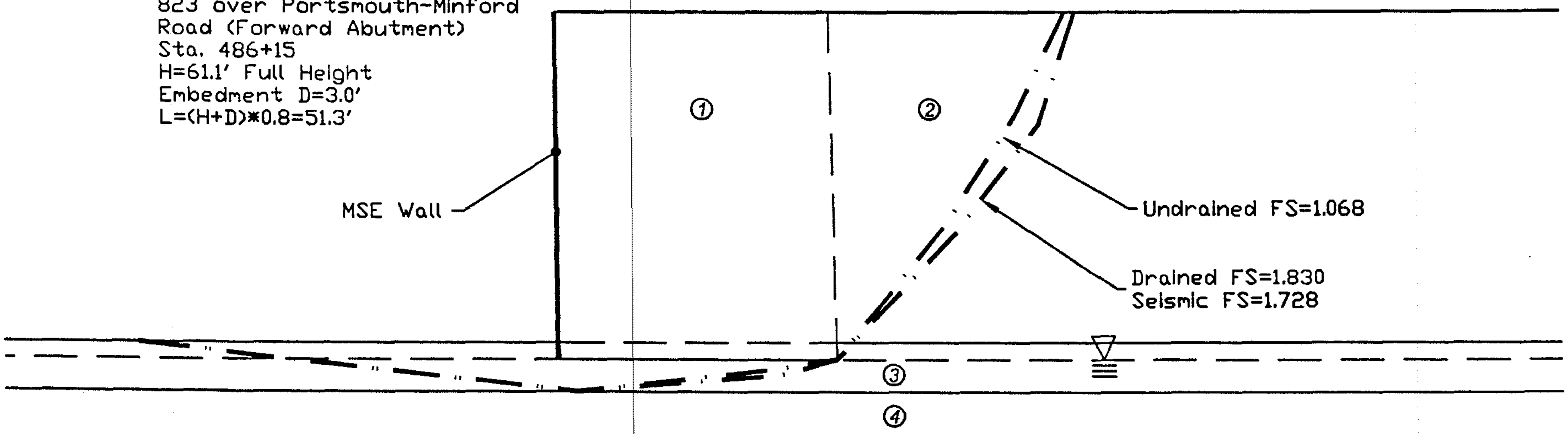
SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 9/14/06

roj\0121\3070.03\Stability Analyses\MSE Wall and Embankment Profiles.dwg, 9/26/2006 1:48:53 PM, \\Dizitree\Q_geotechhp\5100m

			Undrained		Drained		
Material	Consistency	Soil Type	C (psf)	φ (deg)	C' (psf)	φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Medium Dense	Silt/Sandy Silt	1000	0	0	29	120
Material 4	Hard	Sandstone	10000	45	10000	45	145

MSE Stability Analysis
TR-16 & B-10 Profile
823 over Portsmouth-Minford
Road (Forward Abutment)
Sta. 486+15
H=61.1' Full Height
Embedment D=3.0'
L=(H+D)*0.8=51.3'



823 OVER PORTSMOUTH - MINFORD ROAD
FORWARD ABUTMENT

MSE STABILITY ANALYSIS

SCI-823-0. 00

PROJECT NO. 0121-3070. 03 | CALC: SJR | DATE 9/14/06

\proj\0121\3070.03\Stability Analyses\MSE Wall and Embankment Profiles.dwg, 9/14/2006 1:03:47 PM, \\Dlztree\Q_geotechhplj5100tn

DLZ

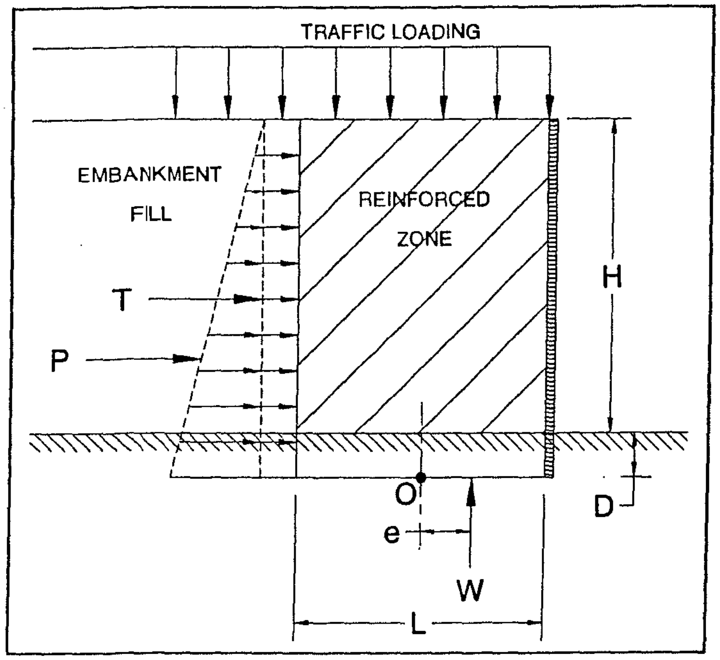
SUBJECT

Client TranSystems
Project SCI 823-0.00 Portsmouth Bypass
Item Bearing Capacity - MSE Wall
SR 823 over Portsmouth Minford Road
Rear Abutment

JOB NUMBER 0121-3070.03
SHEET NO. 1 OF 6
COMP. BY SJR DATE 9/13/06
CHECKED BY DAA DATE 9/14/06

BEARING CAPACITY OF A MSE WALL

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



Effective Bearing Pressure

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

σ_v = 10,339 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 27,816 psf

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

q_{ALL} = 11,126 psf

Factor of Safety = 2.69 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} = 27,816 psf

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

q_{ALL} = 11,126 psf

Factor of Safety = 2.69 OK

Soil Properties

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

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	54.8	ft	Length of MSE reinforcement
L factor	=	0.8		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	68.5	ft	
H	=	65.5	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	22.833	ft	Moment arm
Γ Wt	=	34.25	ft	Moment arm
B'	=	44.84	ft	
γ'	=	57.6	pcf	
W_t		13,152	lb/ft of wall	Weight from traffic
W_{mse}	=	450,456	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	27.86	N_c	27.86
N_q	16.44	N_q	16.44
N_γ	19.34	N_γ	19.34

Eccentricity of Resultant Force

e = 4.98 ft

Kern

e < L/6 = 9.13 ft

DLZ

SUBJECT	Client TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
	Project SCI 823-0.00 Portsmouth Bypass	SHEET NO.	2 OF 6
	Item MSE Wall Stability	COMP. BY	SJR DATE 09/13/06
	SR 823 over Portsmouth - Minford Road	CHECKED BY	DAA DATE 9/14/06
	Rear Abutment		

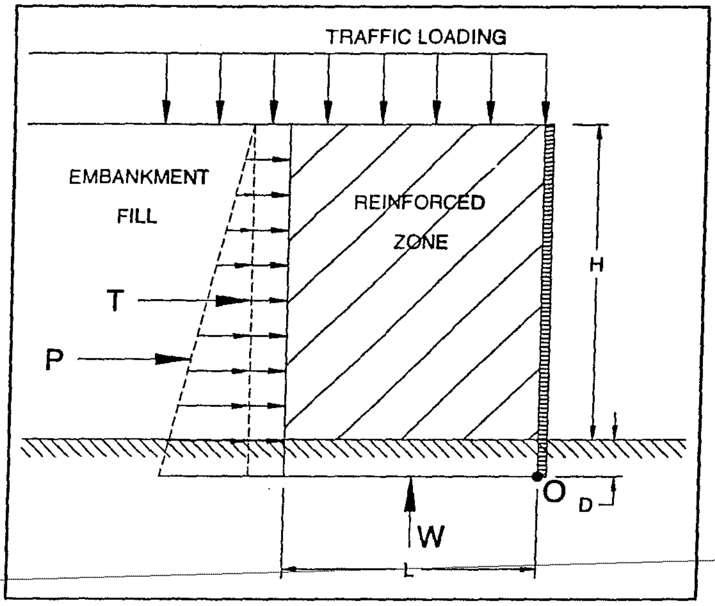
STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=65.5'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

Wall Properties			Foundational Soil Properties			
H+D =	68.5	feet	c =	0	psf	Cohesion
γ_{mse} =	120	pcf	ϕ' =	29	deg	Friction angle
L =	54.8	feet	ω_T =	240	psf	Traffic loading
L factor =	0.80		Length factor-range (0.7 - 1.0)			
ϕ =	30	deg	Friction Angle of Embankment Fill			

RESISTANCE AGAINST SLIDING ALONG BASE



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

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

P_a = 98,332 lbs per foot of wall

Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.37

0.67μ Max. = 0.35 (AASHTO, Bridge Design Manual, 303.4.1.1)

P_r = 157,660. lbs per foot of wall

USE THIS VALUE

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

P_r = 0 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$ Calculated FS = **1.60** Required FS = 1.50

Resistance Against Sliding is $\boxed{\text{OK}}$

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 12,342,494 lb-ft $\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 2,307,179 lb-ft $\Sigma M_{overturnig} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturnin\ g}}$ Calculated FS = **5.35** Required FS = 2.00

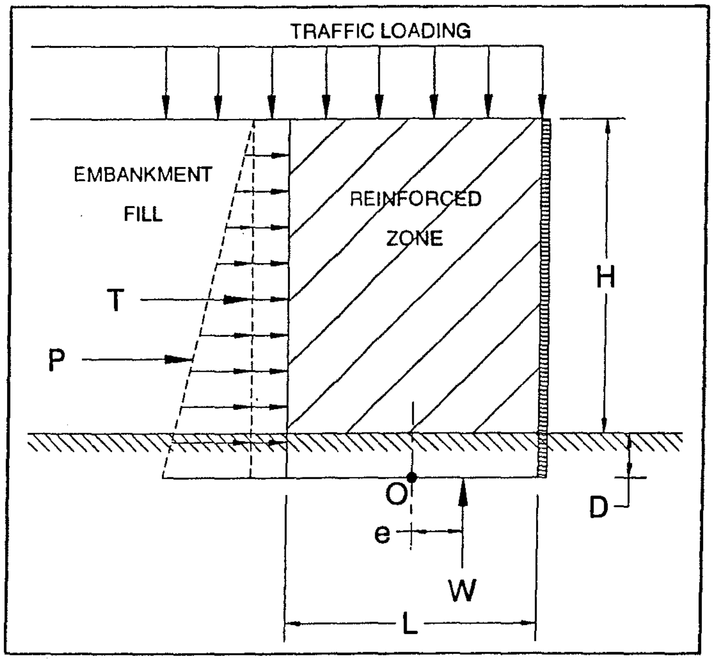
Resistance Against Overturning is $\boxed{\text{OK}}$

DLZ SUBJECT

Client TranSystems		JOB NUMBER	0121-3070.03
Project SCI 823-0.00 Portsmouth Bypass		SHEET NO.	3 OF 6
Item Bearing Capacity - MSE Wall		COMP. BY	SJR DATE 9/13/06
SR 823 over Portsmouth Minford Road		CHECKED BY	DAA DATE 9/14/06
Forward Abutment			

BEARING CAPACITY OF A MSE WALL

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



Effective Bearing Pressure

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

σ_v = 9,698 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 5,313 psf

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

q_{ALL} = 2,125 psf

Factor of Safety = **0.55** **No Good**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 26,201 psf

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

q_{ALL} = 10,480 psf

Factor of Safety = **2.70** OK

Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	1000	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	51.28	ft	Length of MSE reinforcement
L factor	=	0.8		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	64.1	ft	
H	=	61.1	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	21.367	ft	Moment arm
Γ Wt	=	32.05	ft	Moment arm
B'	=	41.94	ft	
γ'	=	57.6	pcf	
W_t		12,307	lb/ft of wall	Weight from traffic
W_{mse}	=	394,446	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 4.67 ft

Kern

e < L/6 = 8.55 ft

DLZ

SUBJECT

Client TranSystems

Project SCI 823-0.00 Portsmouth Bypass

Item Bearing Capacity - MSE Wall

SR 823 over Portsmouth Minford Road

Forward Abutment Granular Fill

JOB NUMBER 0121-3070.03

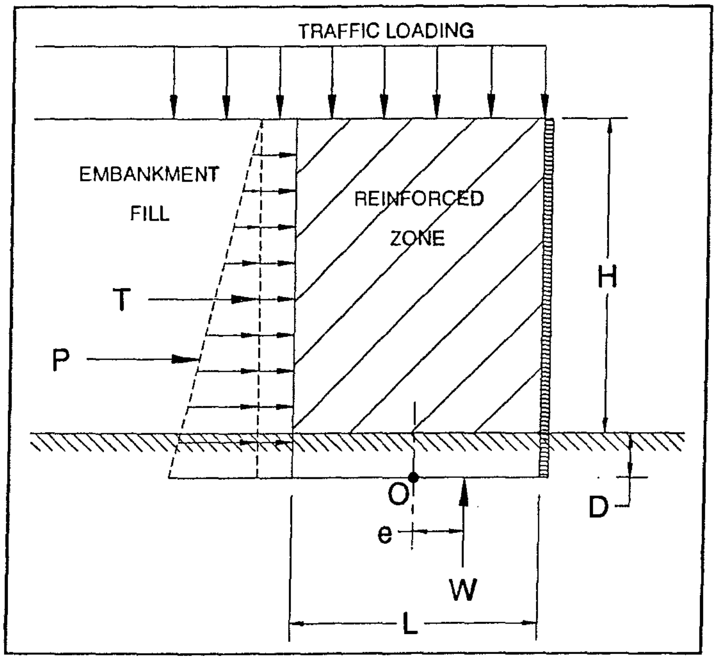
SHEET NO. 5 OF 6

COMP. BY SJR DATE 9/13/06

CHECKED BY DAA DATE 9/14/06

BEARING CAPACITY OF A MSE WALL

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



Effective Bearing Pressure

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

σ_v = 9,698 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 54,682 psf

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

q_{ALL} = 21,873 psf

Factor of Safety = **5.64** **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} = 54,682 psf

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

q_{ALL} = 21,873 psf

Factor of Safety = **5.64** **OK**

Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	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	=	51.28	ft	Length of MSE reinforcement
L factor	=	0.8		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	64.1	ft	
H	=	61.1	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	21.367	ft	Moment arm
Γ Wt	=	32.05	ft	Moment arm
B'	=	41.94	ft	
γ'	=	57.6	pcf	
W_t		12,307	lb/ft of wall	Weight from traffic
W_{mse}	=	394,446	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 4.67 ft

Kern

e < L/6 = 8.55 ft

DLZ

SUBJECT

Client TranSystems ODOT D-9
Project SCI 823-0.00 Portsmouth Bypass
Item MSE Wall Stability
SR 823 over Portsmouth - Minford Road
Forward Abutment Granular Fill

JOB NUMBER 0121-3070.03
SHEET NO. 6 OF 6
COMP. BY SJR DATE 09/13/06
CHECKED BY DAA DATE 9/14/06

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=65.5'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

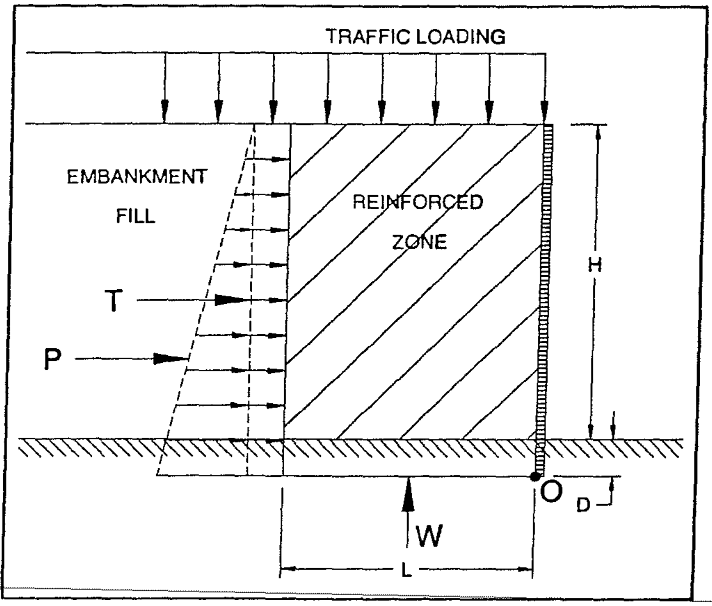
Wall Properties

H+D = 64.1 feet
γ_{mse} = 120 pcf
L = 51.28 feet
L factor = 0.80 Length factor-range (0.7 - 1.0)
ϕ = 30 deg Friction Angle of Embankment Fill

Foundational Soil Properties

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

RESISTANCE AGAINST SLIDING ALONG BASE



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

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

P_a = 86,431 lbs per foot of wall

Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.45

0.67μ Max. = 0.55 (AASHTO, Bridge Design Manual, 303.4.1.1)

P_r = 177,501 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

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

Calculated FS = **2.05**

Required FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 10,113,589 lb-ft

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

$\Sigma M_{overturning}$ = 1,900,982 lb-ft

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

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

Calculated FS = **5.32**

Required FS = 2.00

Resistance Against Overturning is **OK**

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT TranSystems Corp.
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Drilled Shaft - End Bearing
Portsmouth Minford Road

PROJECT NO. 0121-3070.03
SHEET NO. 1 OF 2
COMP. BY SJR DATE 9-14-06
CHECKED BY DAA DATE 9-14-06

* From lab testing rock core samples (lower bound) $q_u = 8,000$ psi

FHWA-IF-99-025   Eq$^=$ 116 $q_{max}(MPa) = 4.83[q_u(MPa)]^{0.51}$

End Bearing

For RQD between 70 — 100 and $q_u > 0.5$ MPa (5.2 tsf)

$q_u = 8000$ psi = 55.16 MPa

[Eq^n 11.6] $q_{max} = 4.83[q_u(MPa)]^{0.51}$

$q_{max} = 4.83[55.16 \text{ MPa}]^{0.51} = 37.34$ MPa

$q_{max} = 37.34$ MPa = 5416 psi = 780 ksf

$q_{allow} = \frac{q_{max}}{F.S.} = \frac{780 \text{ ksf}}{3.0} = 260$ ksf

For Competent Sandstone; Typically 2-3 feet below soil-Rock interface.

$\boxed{* \text{ Use } q_{allow} = 80 \text{ ksf}}$

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT TranSystems Corp.
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Drilled Shaft Side Resistance
Portsmouth - Minford Road

PROJECT NO. 0121-3070.03
SHEET NO. 2 OF 2
COMP. BY SJR DATE 9-14-06
CHECKED BY DAA DATE 9-14-06

* From lab testing rock core samples $q_u = 8000$ psi $f_c' = 4500$ psi

FHWA-IF-99-025 E_q^n 11.24 $f_{max} = 0.65\, P_a [q_u/P_a]^{0.5} \leq 0.65\, P_a [f_c'/P_a]^{0.5}$

Side friction - Smooth Rock Socket

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

$$f_{max} = 0.65\,(14.70 \text{ psi}) \left[\frac{8000 \text{ psi}}{14.70 \text{ psi}} \right]^{0.5} \leq 0.65\,(14.70 \text{ psi}) \left[\frac{4500 \text{ psi}}{14.70 \text{ psi}} \right]^{0.5}$$

$$f_{max} = 222.9 \text{ psi} > 167.2 \text{ psi} \qquad \text{Use } f_{max} = 167 \text{ psi}$$

$$f_{allow} = \frac{167 \text{ psi}}{3.0} = 55 \text{ psi} = 7920 \text{ psf}$$

$$\boxed{{}^{*}\text{Use } f_{allow} = 7500 \text{ psf}}$$

Appendix C
Laboratory Test Results

HDR

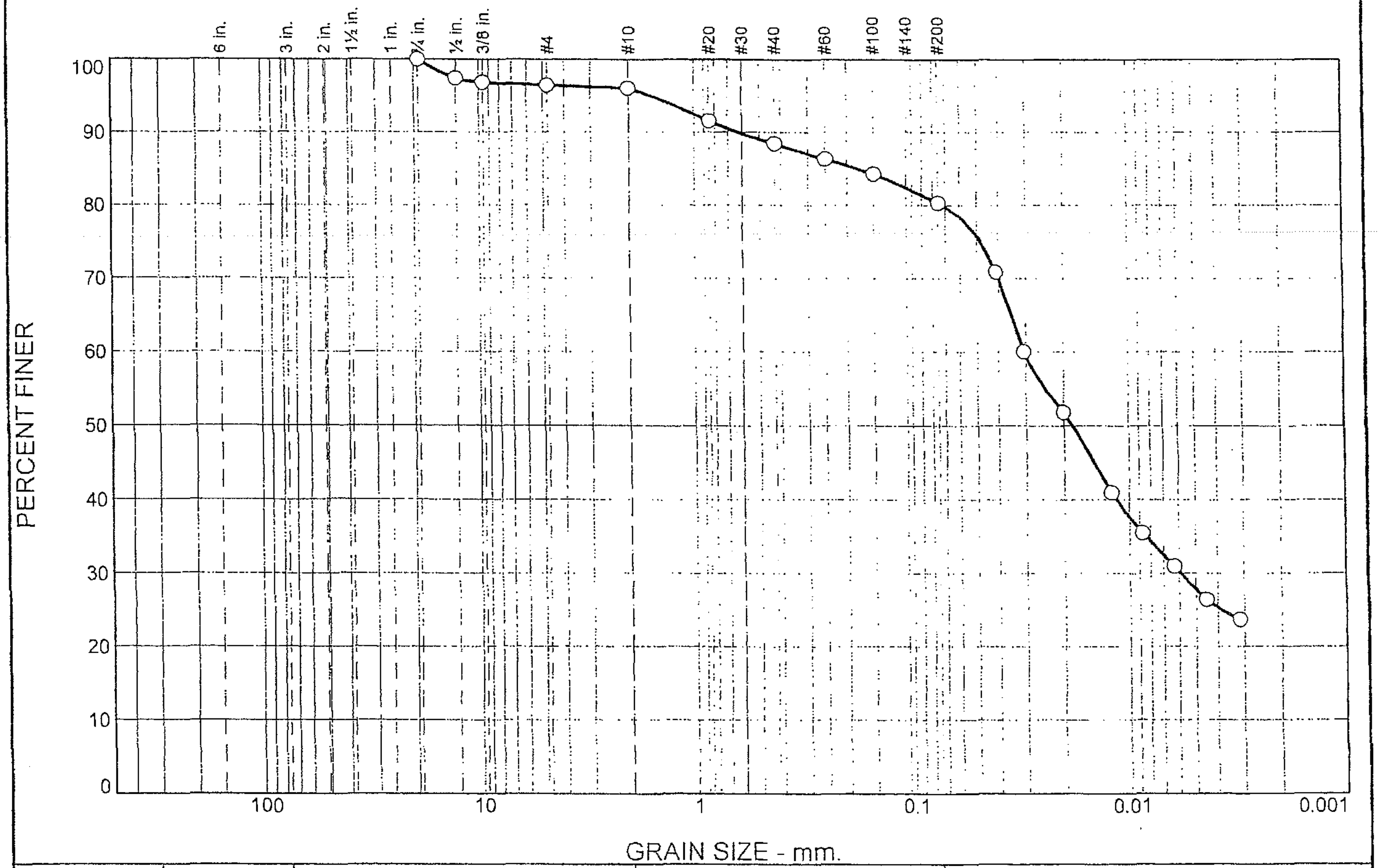
Classification Test Data Summary

Boring No.	Sample No. (SS)	Depth (ft)	Moisture Content (%)	Mechanical Analysis					Atterberg Limits			USCS Classification	ODOT Classification
				Gravel (%)	Coarse Sand	Fine Sand	Silt (%)	Clay (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)		
B-003-0-08	S-1	1.0-2.5	8.3										
B-003-0-08	S-2	3.5-5.0	11.9	4.0	7.6	8.1	52.6	27.7	29	19	10	CL	A-4b(8)
B-003-0-08	S-3	6.0-7.5	12.0										
B-003-0-08	S-4	8.5-10.0	11.1	0.5	3.2	4.9	59.2	32.2	32	20	12	CL	A-6a(9)
B-003-0-08	S-5	11.0-12.5	7.5										
B-003-0-08	S-6	13.5-15.0	5.6										
B-003-0-09	S-7	16.0-17.5	4.3										

Unconfined Compressive Test Data Summary

Boring No.	Depth (ft)	Length as Rec'd (in)	Diameter (in)	Mass (g)	Max Load (lb)	Uncorrected Strength (psi)	L/D Ratio	Corrected Compressive Strength (psi)
B-003-0-08	18.9-19.2	3.9	1.97	462.5	21410	7010	1.97	6996
B-003-0-08	19.3-19.6	3.9	1.97	477.6	18840	6169	2.00	6169
B-003-0-08	33.6-33.9	3.5	1.97	462.2	37070	121.5	1.75	11968
B-003-0-08	33.9-34.2	4.0	1.97	554.6	47070	15441	2.00	15441

Particle Size Distribution Report



% Boulders	% +3"	% Gravel		% Sand		% Fines	
		Coarse	Fine	Coarse	Fine	Silt	Clay
0.0	0.0	0.0	4.0	7.6	8.1	52.6	27.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100.0		
.5	97.4		
.375	96.8		
#4	96.4		
#10	96.0		
#20	91.6		
#40	88.4		
#60	86.4		
#100	84.3		
#200	80.3		

* (no specification provided)

Material Description

brown Silt

Atterberg Limits

PL= 19 LL= 29 PI= 10

Coefficients

D_{85}= 0.1764 D_{60}= 0.0306 D_{50}= 0.0182
D_{30}= 0.0059 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL ODOT= A-4b(8)

Remarks

Date Received: 2/14/08
Lab No.: 0128

Sample Number: 0128 **Depth:** 3.5' - 5.0'
Location: B-003-0-08 (S-2)

Date: 2/14/08

TES TECH

Dayton, Ohio

Client: HDR Engineering, Inc.
Project: ODOT District 9 Portsmouth Bypass, SCI-823-0.00/6.81, PID #19415 - Minford, Ohio
Project No: 25506 **File No.**

GRAIN SIZE DISTRIBUTION TEST DATA

3/5/2008

Client: HDR Engineering, Inc.

Project: ODOT District 9 Portsmouth Bypass, SCI-823-0.00/6.81, PID #19415 - Minford, Ohio

Project Number: 25506

Location: B-003-0-08 (S-2)

Depth: 3.5' - 5.0' **Sample Number:** 0128

Material Description: brown Silt

Date: 2/14/08 **PL:** 19 **LL:** 29 **PI:** 10

USCS Classification: CL **AASHTO Classification:** A-4(7)

Ohio DOT Classification (if different from AASHTO): A-4b(8)

Testing Remarks: Date Received: 2/14/08
Lab No.: 0128

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
227.69	14.51	0.00	.75	0.00	100.0
			.5	5.52	97.4
			.375	6.81	96.8
			#4	7.63	96.4
			#10	8.48	96.0
53.47	0.00	0.00	#20	2.48	91.6
			#40	4.23	88.4
			#60	5.37	86.4
			#100	6.54	84.3
			#200	8.76	80.3

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 96.0
Weight of hydrometer sample =53.469
Hygroscopic moisture correction:
Moist weight and tare = 49.88
Dry weight and tare = 49.72
Tare weight = 34.24
Hygroscopic moisture = 1.0%
Automatic temperature correction
Composite correction (fluid density and meniscus height) at 20 deg. C = -4.5
Meniscus correction only = 0.0
Specific gravity of solids = 2.65
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 x Rm

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	20.5	43.5	39.1	0.0136	43.5	9.2	0.0410	70.9
2.00	20.5	37.5	33.1	0.0136	37.5	10.1	0.0305	60.0
5.00	20.5	33.0	28.6	0.0136	33.0	10.9	0.0200	51.8
15.00	20.5	27.0	22.6	0.0136	27.0	11.9	0.0121	40.9
30.00	20.5	24.0	19.6	0.0136	24.0	12.4	0.0087	35.5
60.00	20.5	21.5	17.1	0.0136	21.5	12.8	0.0063	31.0
120.00	20.5	19.0	14.6	0.0136	19.0	13.2	0.0045	26.4
250.00	20.5	17.5	13.1	0.0136	17.5	13.4	0.0031	23.7

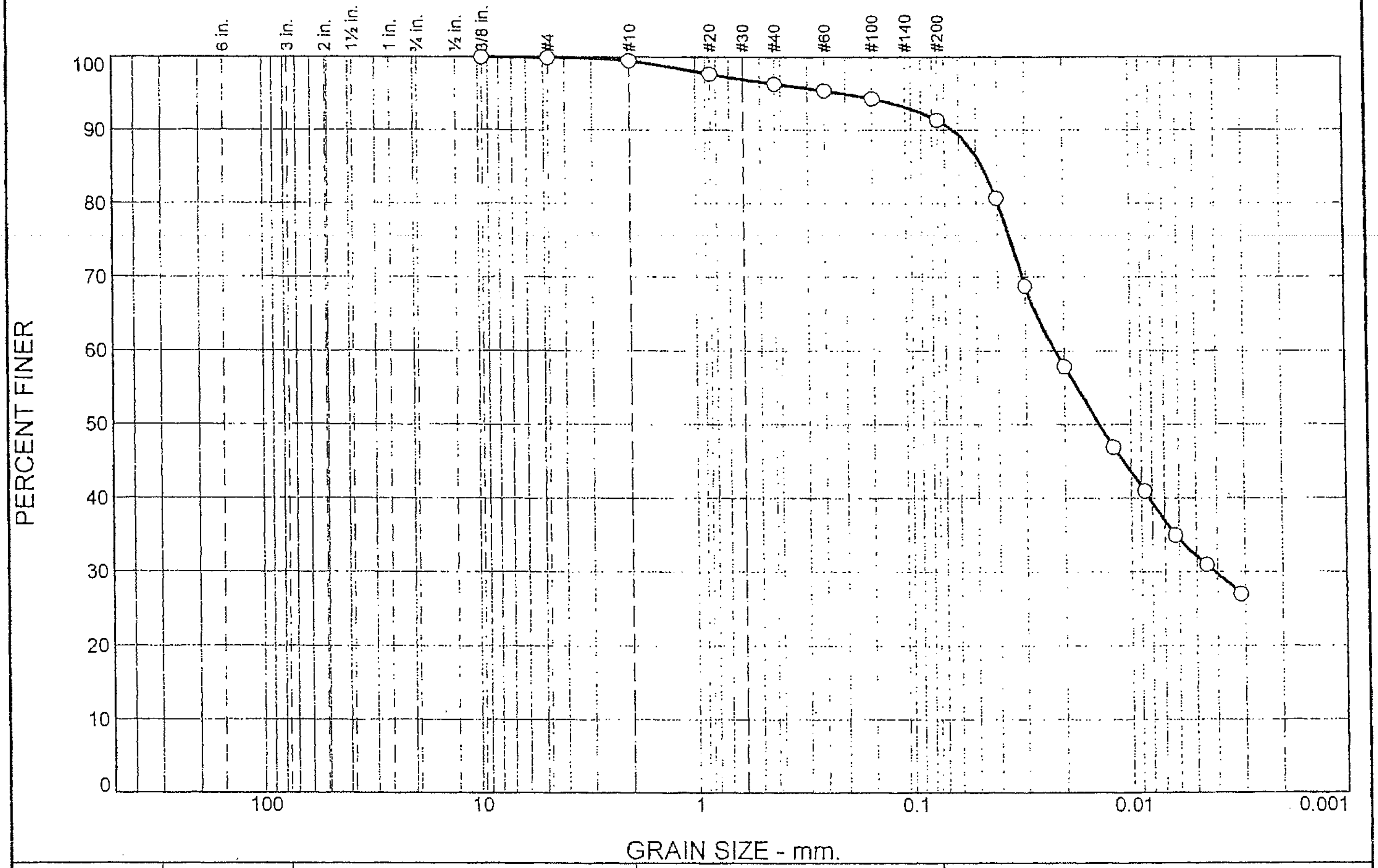
Fractional Components

Boulders	Cobbles	Gravel			Sand			Fines		
		Coarse	Fine	Total	Coarse	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	4.0	4.0	7.6	8.1	15.7	52.6	27.7	80.3

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0059	0.0182	0.0306	0.0718	0.1764	0.6197	1.5511

Fineness Modulus
0.56

Particle Size Distribution Report



% Boulders	% +3"	% Gravel		% Sand		% Fines	
		Coarse	Fine	Coarse	Fine	Silt	Clay
0.0	0.0	0.0	0.5	3.2	4.9	59.2	32.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	99.9		
#10	99.5		
#20	97.7		
#40	96.3		
#60	95.4		
#100	94.3		
#200	91.4		

* (no specification provided)

Material Description

brown Silt and Clay

Atterberg Limits

PL= 20 LL= 32 PI= 12

Coefficients

D_{85}= 0.0466 D_{60}= 0.0220 D_{50}= 0.0140
D_{30}= 0.0041 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL ODOT= A-6a(9)

Remarks

Date Received: 2/14/08
Lab No.: 0128

Sample Number: 0128 **Depth:** 8.5' - 10.0'
Location: B-003-0-08 (S-4)

Date: 2/14/08

TES TECH

Dayton, Ohio

Client: HDR Engineering, Inc.
Project: ODOT District 9 Portsmouth Bypass, SCI-823-0.00/6.81, PID #19415 - Minford, Ohio
Project No: 25506 **File No.**

GRAIN SIZE DISTRIBUTION TEST DATA

3/5/2008

Client: HDR Engineering, Inc.
Project: ODOT District 9 Portsmouth Bypass, SCI-823-0.00/6.81, PID #19415 - Minford, Ohio
Project Number: 25506
Location: B-003-0-08 (S-4)
Depth: 8.5' - 10.0' **Sample Number:** 0128
Material Description: brown Silt and Clay
Date: 2/14/08 **PL:** 20 **LL:** 32 **PI:** 12
USCS Classification: CL **AASHTO Classification:** A-6(10)
Ohio DOT Classification (if different from AASHTO): A-6a(9)
Testing Remarks: Date Received: 2/14/08
Lab No.: 0128

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
195.83	14.17	0.00	.375	0.00	100.0
			#4	0.15	99.9
			#10	0.97	99.5
50.57	0.00	0.00	#20	0.92	97.7
			#40	1.61	96.3
			#60	2.08	95.4
			#100	2.64	94.3
			#200	4.12	91.4

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 99.5
Weight of hydrometer sample =50.568
Hygroscopic moisture correction:
Moist weight and tare = 63.47
Dry weight and tare = 63.33
Tare weight = 51.02
Hygroscopic moisture = 1.2%
Automatic temperature correction
Composite correction (fluid density and meniscus height) at 20 deg. C = -4.5
Meniscus correction only = 0.0
Specific gravity of solids = 2.65
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 x Rm

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	20.5	45.0	40.6	0.0136	45.0	8.9	0.0405	80.7
2.00	20.5	39.0	34.6	0.0136	39.0	9.9	0.0302	68.8
5.00	20.5	33.5	29.1	0.0136	33.5	10.8	0.0199	57.8
15.00	20.5	28.0	23.6	0.0136	28.0	11.7	0.0120	46.9
30.00	20.5	25.0	20.6	0.0136	25.0	12.2	0.0086	40.9
60.00	20.5	22.0	17.6	0.0136	22.0	12.7	0.0062	35.0
120.00	20.5	20.0	15.6	0.0136	20.0	13.0	0.0045	31.0
250.00	20.5	18.0	13.6	0.0136	18.0	13.3	0.0031	27.0

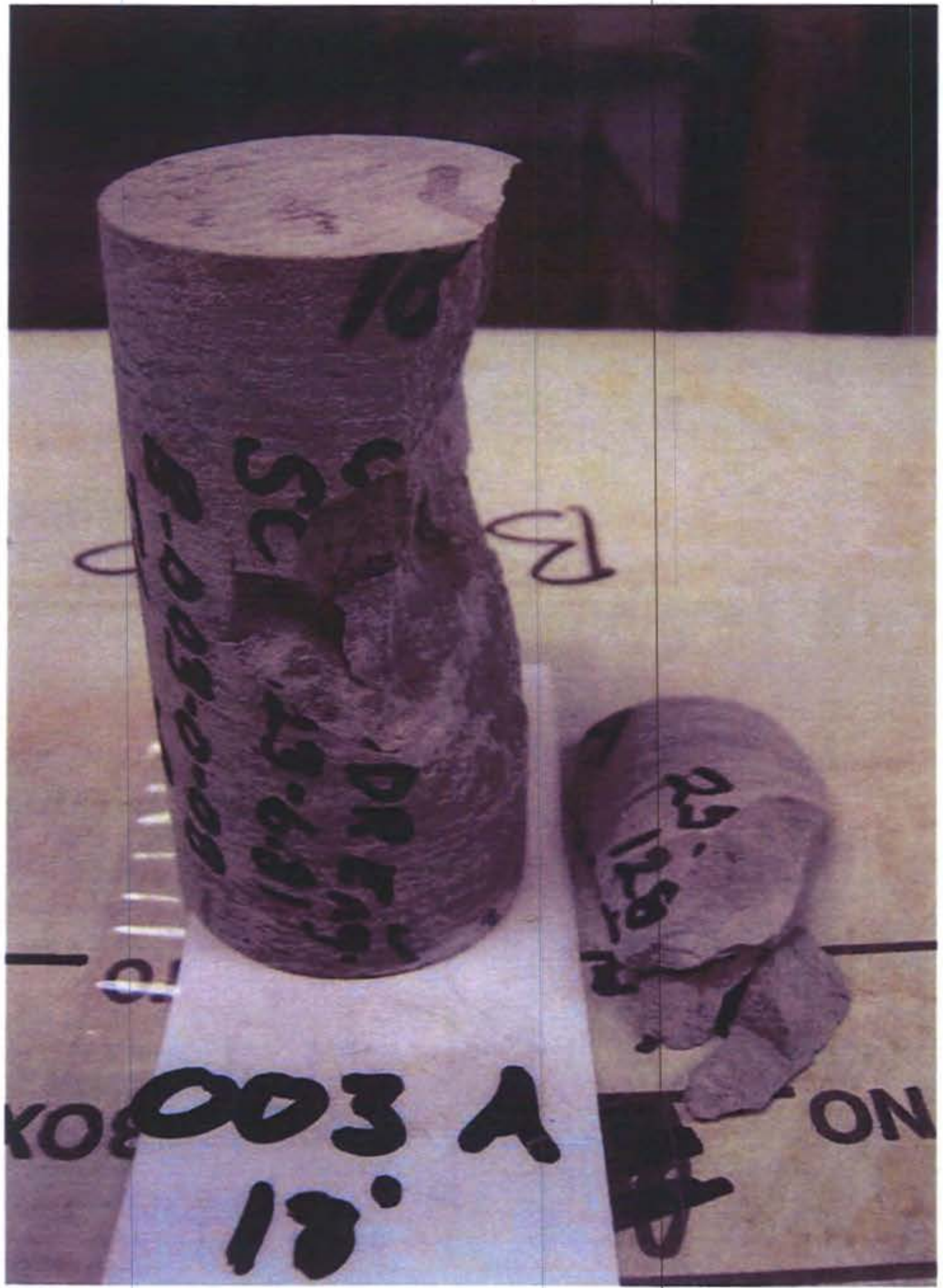
Fractional Components

Boulders	Cobbles	Gravel			Sand			Fines		
		Coarse	Fine	Total	Coarse	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.5	0.5	3.2	4.9	8.1	59.2	32.2	91.4

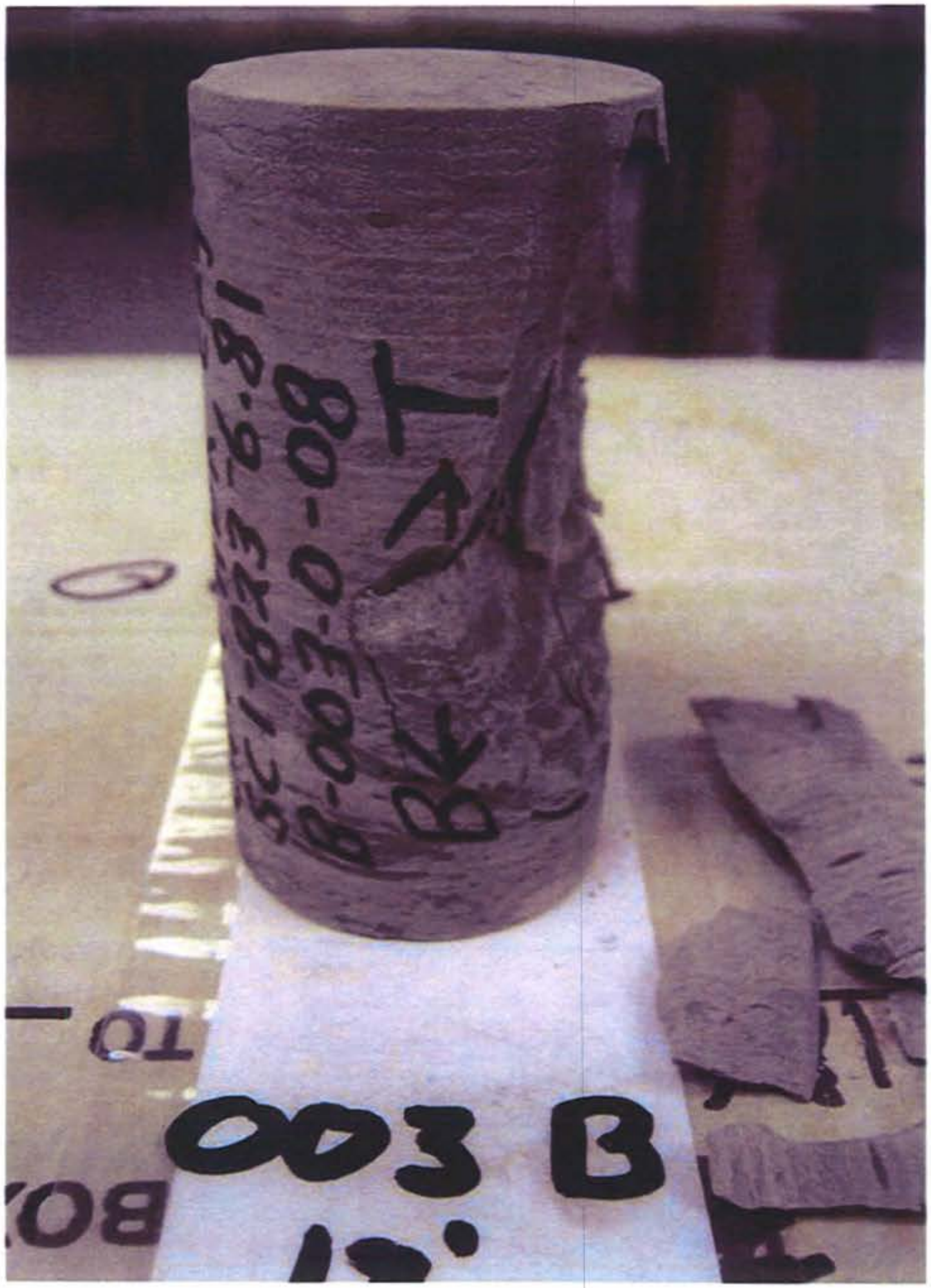
D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0041	0.0140	0.0220	0.0397	0.0466	0.0632	0.2057

Fineness Modulus
0.15

TT 25506 ODOT DISTRICT 9 PORTSMOUTH BYPASS MINFORD OH

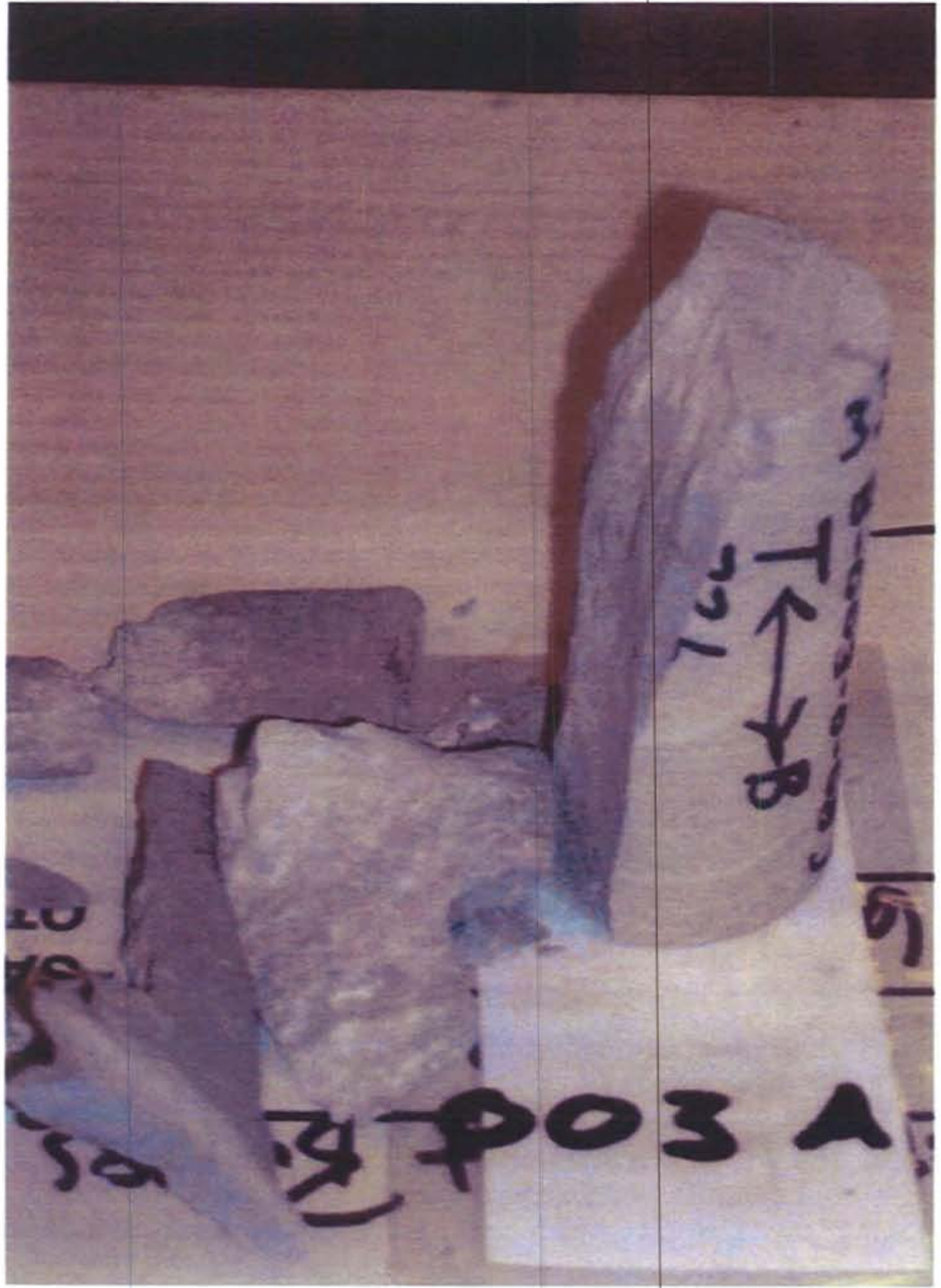


B-003-0-08 (18.9'-19.9') A

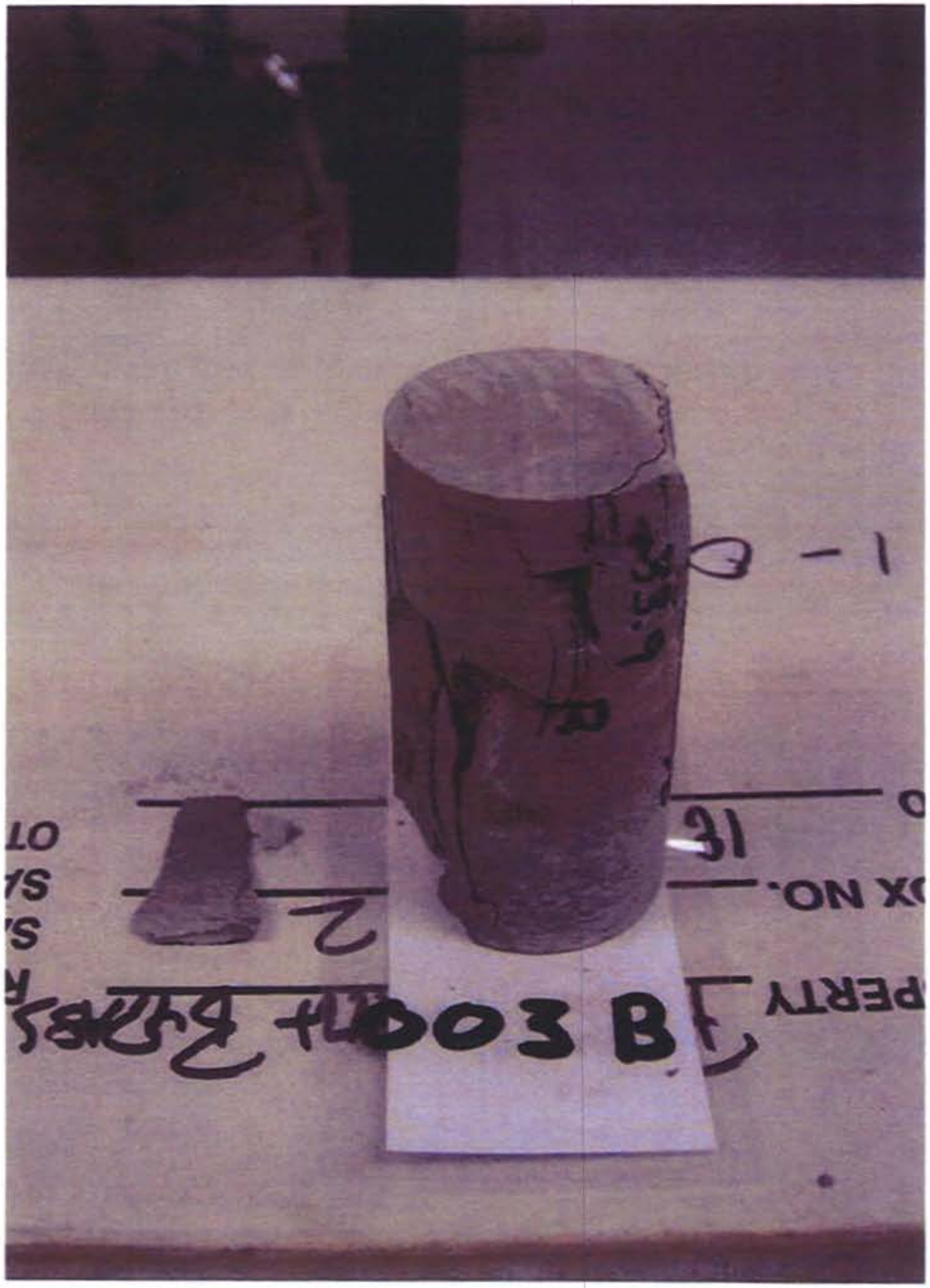


B-003-0-08 (18.9'-19.9') B

TT 25506 ODOT DISTRICT 9 PORTSMOUTH BYPASS MINFORD OH



B-003-0-08 (33.6'-34.5') A



B-003-0-08 (33.6'-34.5') B

Appendix D
Supplemental Boring Logs and Core Photos

LOG OF BORING

Boring No.: B-003-0-08
Date Started: 11-Feb-08
Date Completed: 11-Feb-08

Baseline, Station & Offset: 483+12, 11.38' LT
Coordinates:
Drilling Method: 3.25" HSA/NQ
Sealing & Backfilling Type: Cement
Quantity:

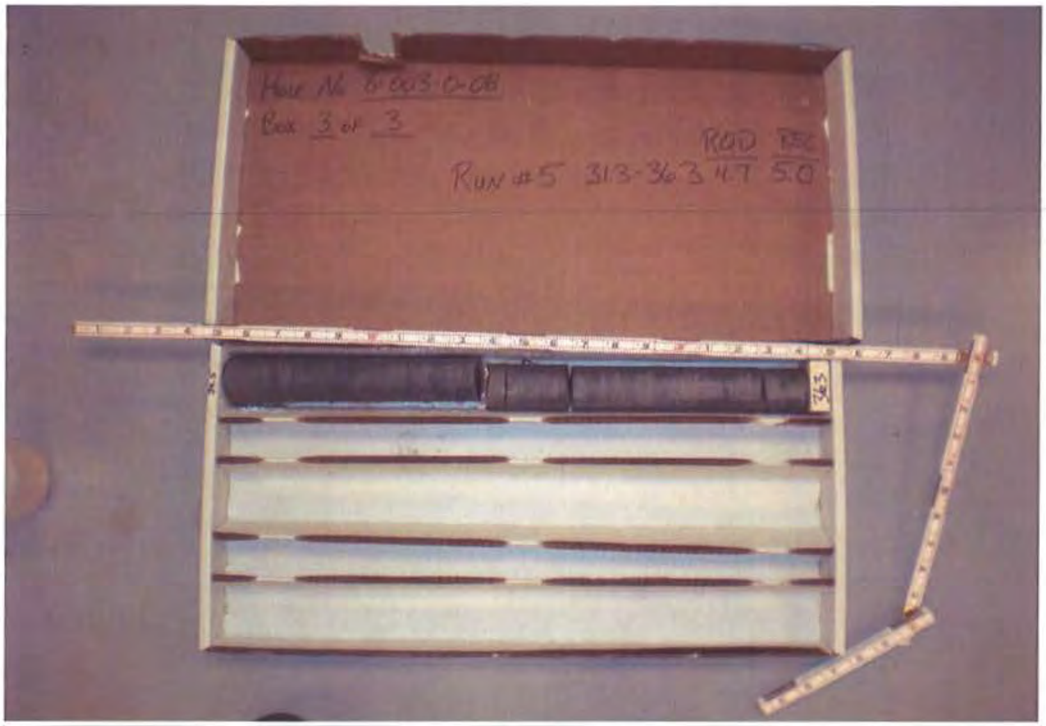
Surface Elev. (ft) 675.5
Water Elev. (ft) 663.5
Project ID.: SCI-823-6.81, PID 19415
Hammer System Type: CME Automatic
Calibration Date: 10-25-07
ER: 74%

Elev. ft 0.0	Depth ft 0	Std. Pen /RQD	N_{60}	Rec. ft	Loss ft	Description	Sample Type & No.	% Rec	% Agg	% C.S.	% F.S.	% Silt	% Clay	P.L	L.L	P.I	W.C.	ODOT Class
674.5		Augered				1.0'												
673.5	2	3/5/4	11				S-1					Same as S-2					8.3	A-4b (VISUAL)
672.5						Medium dence, light brown with red and black (manganese) staining, SILT, dry												
671.5	4	4/3/6	11				S-2		4	8	8	52	28	29		10	11.9	A-4b (8)
670.5																		
669.5	6					5.5'												
668.5		10/13/7	25				S-3					Same as S-4					12.0	A-6a (VISUAL)
667.5	8					Dense, light brown with red and black (manganese) staining, SILT AND CLAY, dry												
666.5							S-4		1	3	5	59	32	32		12	11.0	A-6a (9)
665.5	10	18/22/24	54															
664.5																		
663.5	12	21/25/26	63			12.5'	S-5					Same as S-4					7.5	A-6a (VISUAL)
662.5																		
661.5	14	50/.3				Silty Shale, gray, severely weathered (decomposed) very weak to weak, very fine sand texture, laminated to thinly laminated												
660.5																		
659.5	16	50/.1				Top of Rock 16.1'												16.1'
658.5						Argillaceous Silty Shale, moderately weathered, very weak to weak, very fine sand texture, laminated to thinly laminated, Unit RQD= 85%, Loss=3%												
657.5	18	90%		2.9	0.1													18.1'
656.5																		
655.5	20	100%		2.2	0.0													
654.5																		
653.5	22																	
652.5																		
651.5	24	84%		5.0	0.0													
650.5																		
649.5	26																	
648.5						Interbedded Siltstone & Sandy Shale, lightly gray with dark gray streaks, slightly to unweathered, moderately strong to strong, fine to very fine grained, Unit RQD= 89%, Loss=0%												
647.5	28																	
646.5		82%		5.0	0.0													
645.5	30																	
644.5																		
643.5	32																	
642.5																		
641.5	34	94%		5.0	0.0													
640.5																		
639.5	36																	

Bottom of Boring = 36.3'

B-003-0-08 - Box 1 of 3

B-003-0-08 - Box 2 of 3



B-003-0-08 - Box 3 of 3

Appendix E
Analyses

Bearing Capacity – Spread Footing
Embankment Consolidation Settlement
Slope Stability Analyses

HDR | ONE COMPANY *Many Solutions*℠

Project:	SCI-823-6.81 PID 19415	Computed:	DMV	Date:	11-Apr-08
Subject:	S.R. 823 over Portsmouth-Minford Road (SR-139)	Checked:	JSA	Date:	11-Apr-08
Task:	Spread Footing - Piers	Page:		of	
Job #:	45878	No:			

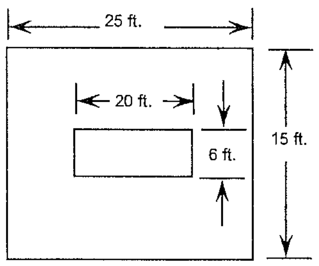
Objective: Determine Allowable Bearing Capacity

Reference: AASHTO 17th Edition (2002), Section 4.4.8.1
Report of Subsurface Exploration: Bridge and MSE Retaining Walls, SR 823 over Portsmouth-Minford Road, SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio; DLZ Ohio, Inc, 9/26/06

Given: Proposed Bottom of Footing (BOF) elevation = Varies

Assume: Spread Footings

f'_c = 4,000 psi
288 tsf
B_{min} = 25 feet
L_1 = 15 feet
L_2 = feet
T = 4 feet
D_f = 6 feet
Proposed BOF = Varies feet
L_1 / B = 0.6



(NTS)

Subsurface conditions are represented by Borings B-10, B-11 and B-12

	Strata	Top El. (feet)		Bottom El. (feet)	REC%avg	RQD%avg	Strength (tsf)	Strength (psi)
B-10	SNDST	623.1	to	613.1	97%	87%	748.3	10,393
B-11	SNDST	624.2	to	604.2	87%	77%	758.7	10,537
B-12	SNDST	624.0	to	614.0	90%	76%	699.0	9,709
						Average	735.3	10,213

Factor of Safety, FS = 3 [AASHTO, Section 4.4.8.1.3]

Compressive Strength of Bearing Strata, C_o = 699.0 tsf (AASHTO, Section 4.4.8.1.2)

Ground Water Table Varies feet

Rock "Category" = B [AASHTO, Table 4.4.8.1.2B]

RMR Rating = 55 [Geomechanics Classification System, see attached worksheet]
[using average value calculated for strata within 1B of BOF]

Nms = 0.15379 [Interpolated using AASHTO, Table 4.4.8.1.2A]

Calculation: $q_{ult} = N_{ms}\,C_o$ = 107.5 tsf = 215.0 ksf = 214,999 psf
[AASHTO 4.4.8.1.2-1]

=> $\boxed{q_{all} = q_{ult}/FS = 35.83 \text{ tsf} = 71.67 \text{ ksf} = 71,666 \text{ psf}}$

SAY 35 TSF

=> **Check Unconfined Compressive Strength of Rock (from testing)**

35.83 tsf < 699.0 tsf OK

=> **Check Allowable Stress in Concrete, $\sigma_{all} = 0.3 f'_c$**

35.83 tsf < 86.40 tsf OK

Data from AASHTO Table 4.4.8.1.2A (Group B)

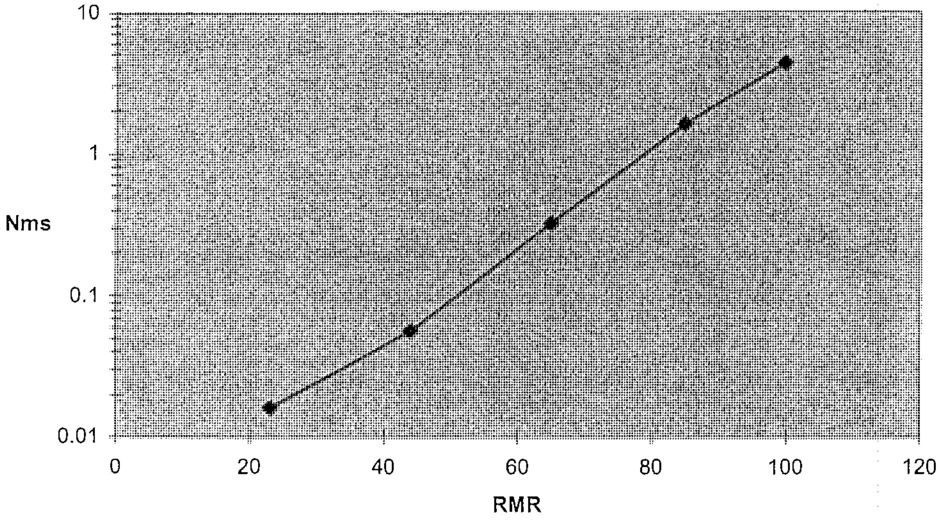
RMR	Nms	Rock Quality	RQD%
3	-	Very Poor	< 25
23	0.016	Poor	25 to 50
44	0.056	Fair	50 to 75
65	0.32	Good	75 to 90
85	1.6	Very Good	90 to 95
100	4.3	Excellent	95 to 100

Regression Calculation

RMR	Nms
55	0.15379

Values of Coefficient Nms for Estimation of Ultimate Bearing Capacity (Hoek, 1983)

"GROUP B"



SCI-823-0917 RMR
Input Data Sheet

Geomechanics Classification of Jointed Rock Masses, Tables B-2 & B-3									
General Rock Parameters	B-10	B-11	B-12		Value				Rating
					English		Metric		
Surface Elevation	632.60	632.70	632.50		632.60	feet	192.82	m	
Depth to Rock @ Bottom of Footing	9.50	8.50	8.50		8.83	feet	2.69	m	
Layer Thickness (feet)	10.00	20.00	20.00		18.00	feet	5.49	m	
Point-load Strength Index (psi)					0	psi	0.00	MPa	
Uniaxial Compressive Strength, Co (psi)	10,393	10,537	9,709		10,177	psi	70.19	MPa	7
Rock Quallity, RQD	87%	77%	76%		79%		79%		15
Spacing of Discontinuities (inch)	4.00	4.00	4.00		4.00	inch	101.60	mm	6
Condition of Discontinuities (rough/weathered)	Moderately Weathered								
Discontinuity Separation (inch)					0.00	Inch	0.00	mm	22
Ground water									7
Strike & Dip									-2
	SNDST	SNDST	SNDST			Fair Rock			55
		Weighted Average of individual strata (see below) =							55
Specific Rock Strata Parameters	B-10				Value				Rating
					English		Metric		
Surface Elevation	632.60				632.60	feet	192.82	m	
Depth to Top of Layer (feet)	9.50				9.50	feet	2.90	m	
Layer Thickness (feet)	10.00				10.00	feet	3.05	m	
Point-load Strength Index (psi)					0	psi	0.00	MPa	
Uniaxial Compressive Strength, Co (psi)	10393				10,393	psi	71.68	MPa	7
Rock Quallity, RQD	87%				87%		87%		17
Spacing of Discontinuities (inch)	4.00				4.00	inch	101.60	mm	6
Condition of Discontinuities (rough/weathered)	Moderately Weathered								
Discontinuity Separation (inch)	0.00				0.00	Inch	0.00	mm	22
Ground water									7
Strike & Dip									-2
	SNDST					Fair Rock			57
Specific Rock Strata Parameters		B-11			Value				Rating
					English		Metric		
Surface Elevation		632.70			632.70	feet	192.85		
Depth to Top of Layer (feet)		8.50			8.50	feet	2.59		
Layer Thickness (feet)		20.00			20.00	feet	6.10	m	
Point-load Strength Index (psi)		0			0	psi	0.00	MPa	
Uniaxial Compressive Strength, Co (psi)		10537			10,537	psi	72.67	MPa	7
Rock Quallity, RQD		77%			77%		77%		15
Spacing of Discontinuities (inch)		4.00			4.00	inch	101.60	mm	6
Condition of Discontinuities (rough/weathered)									
Discontinuity Separation (inch)		0.00			0.00	Inch	0.00	mm	22
Ground water									7
Strike & Dip									-2
		SNDST				Fair Rock			55
Specific Rock Strata Parameters			B-12		Value				Rating
					English		Metric		
Surface Elevation			632.5		632.50	feet	192.79	m	
Depth to Top of Layer (feet)			8.5		8.50	feet	2.59	m	
Layer Thickness (feet)			20		20.00	feet	6.10	m	
Point-load Strength Index (psi)			0		0	psi	0.00	MPa	
Uniaxial Compressive Strength, Co (psi)			9709		9,709	psi	66.96	MPa	7
Rock Quallity, RQD			76%		76%		76%		15
Spacing of Discontinuities (inch)			4.00		4.00	inch	101.60	mm	6
Condition of Discontinuities (rough/weathered)			0						
Discontinuity Separation (inch)			0		0.00	Inch	0.00	mm	22
Ground water									7
Strike & Dip									-2
			SNDST			Fair Rock			55

TABLE B-2. Geomechanics Classification of Jointed Rock Masses

<table>
<tr><th colspan="3">PARAMETER</th><th colspan="7">RANGES OF VALUES</th></tr>
<tr><td colspan="10">(a) Classification Parameters and their ratings</td></tr>
<tr><td rowspan="3">1</td><td rowspan="2">Strength of intact rock material</td><td>Point-load strength index</td><td>>10 MPa</td><td>4–10 MPa</td><td>2–4 MPa</td><td>1–2 MPa</td><td colspan="3">For this low range uniaxial compressive test is preferred</td></tr>
<tr><td>Uniaxial compressive strength</td><td>>250 MPa</td><td>100–250 MPa</td><td>50–100 MPa</td><td>25–50 MPa</td><td>5–25 MPa</td><td>1–5 MPa</td><td><1 MPa</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>12</td><td>7</td><td>4</td><td>2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">2</td><td colspan="2">Drill core quality RQD</td><td>90–100%</td><td>75–90%</td><td>50–75%</td><td>25–50%</td><td colspan="3"><25%</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>17</td><td>13</td><td>8</td><td colspan="3">3</td></tr>
<tr><td rowspan="2">3</td><td colspan="2">Spacing of discontinuities</td><td>>2 m</td><td>0.6–2 m</td><td>200–600 mm</td><td>60–200 mm</td><td colspan="3"><60 mm</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>15</td><td>10</td><td>8</td><td colspan="3">5</td></tr>
<tr><td rowspan="2">4</td><td colspan="2">Condition of discontinuities</td><td>Very rough surfaces Not continuous No separation Unweathered wall rock</td><td>Slightly rough surfaces Separation < 1 mm Slightly weathered walls</td><td>Slightly rough surfaces Separation <1 mm Highly weathered walls</td><td>Slickensided surfaces OR Gouge < 5 mm thick OR Separation 1–5 mm Continuous</td><td colspan="3">Soft gouge > 5 mm thick OR Separation > 5 mm Continuous</td></tr>
<tr><td colspan="2">Rating</td><td>30</td><td>25</td><td>20</td><td>10</td><td colspan="3">0</td></tr>
<tr><td rowspan="6">5</td><td rowspan="5">Ground water</td><td>Inflow per 10 m tunnel length</td><td>None</td><td>< 10 L/min</td><td>10–25 L/min</td><td>25–125 L/min</td><td colspan="3">> 125</td></tr>
<tr><td></td><td>OR</td><td>OR</td><td>OR</td><td>OR</td><td colspan="3">OR</td></tr>
<tr><td>Ratio $\frac{\textit{Joint water pressure}}{\textit{major principal stress}}$</td><td>0</td><td>0.0–0.1</td><td>0.1–0.2</td><td>0.2–0.5</td><td colspan="3">>0.5</td></tr>
<tr><td></td><td>OR</td><td>OR</td><td>OR</td><td>OR</td><td colspan="3">OR</td></tr>
<tr><td>General conditions</td><td>Completely dry</td><td>Damp</td><td>Wet</td><td>Dripping</td><td colspan="3">Flowing</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>10</td><td>7</td><td>4</td><td colspan="3">0</td></tr>
<tr><td colspan="10">(b) Rating Adjustment for Joint Orientations</td></tr>
<tr><td colspan="3">Strike and dip orientations of joints</td><td>Very favorable</td><td>Favorable</td><td>Fair</td><td>Unfavorable</td><td colspan="3">Very unfavorable</td></tr>
<tr><td colspan="2" rowspan="3">Ratings</td><td>Tunnels</td><td>0</td><td>−2</td><td>−5</td><td>−10</td><td colspan="3">−12</td></tr>
<tr><td>Foundations</td><td>0</td><td>−2</td><td>−7</td><td>−15</td><td colspan="3">−25</td></tr>
<tr><td>Slopes</td><td>0</td><td>−5</td><td>−25</td><td>−50</td><td colspan="3">−80</td></tr>
<tr><td colspan="10">(c) Rock Mass Classes Determined from Total Ratings</td></tr>
<tr><td colspan="3">Rating</td><td>100←81</td><td>80←61</td><td>60←41</td><td>40←21</td><td colspan="3"><20</td></tr>
<tr><td colspan="3">Class number</td><td>I</td><td>II</td><td>III</td><td>IV</td><td colspan="3">V</td></tr>
<tr><td colspan="3">Description</td><td>Very good rock</td><td>Good rock</td><td>Fair rock</td><td>Poor rock</td><td colspan="3">Very poor rock</td></tr>
<tr><td colspan="10">(d) Meaning of Rock Mass Classes</td></tr>
<tr><td colspan="3">Class number</td><td>I</td><td>II</td><td>III</td><td>IV</td><td colspan="3">V</td></tr>
<tr><td colspan="3">Average stand-up time</td><td>10 years for 15 m span</td><td>6 months for 8 m span</td><td>1 week for 5 m span</td><td>10 hours for 2.5 m span</td><td colspan="3">30 minutes for 1 m span</td></tr>
<tr><td colspan="3">Cohesion of rock mass</td><td>>400 kPa</td><td>300–400 kPa</td><td>200–300 kPa</td><td>100–200 kPa</td><td colspan="3">< 100 kPa</td></tr>
<tr><td colspan="3">Friction angle of rock mass</td><td>>45°</td><td>35–45°</td><td>25–35°</td><td>15–25°</td><td colspan="3"><15°</td></tr>
</table>

CHART A Ratings for Strength of Intact Rock

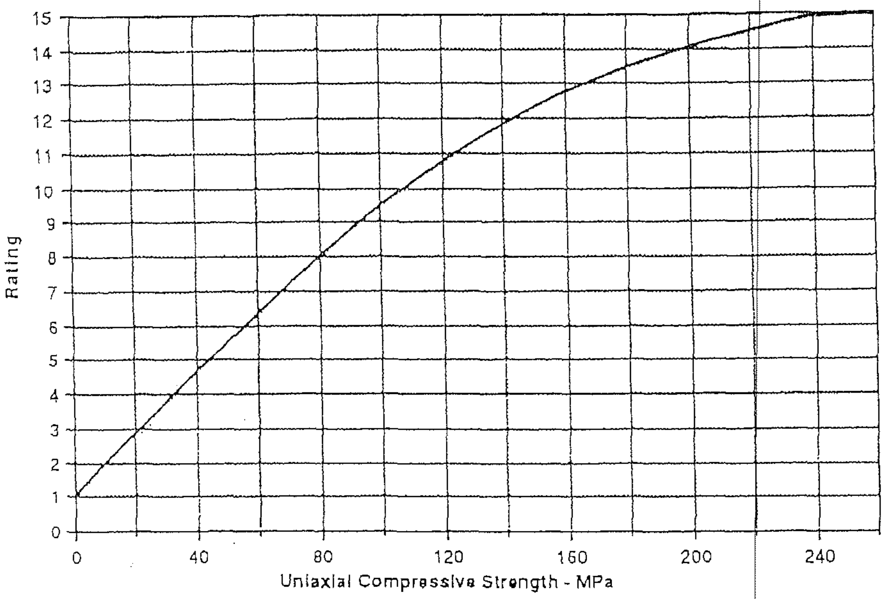


CHART B Ratings for RQD

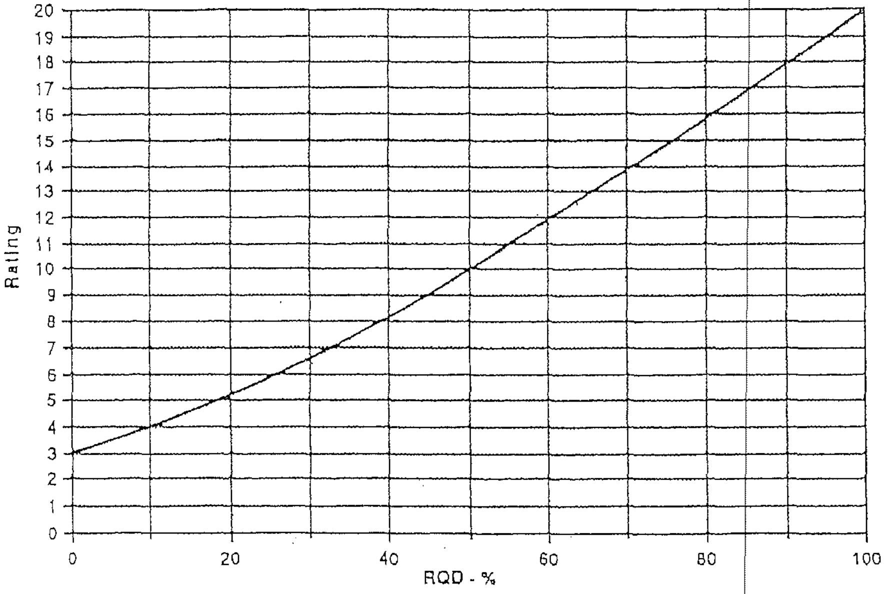


CHART C Ratings for Discontinuity Spacing

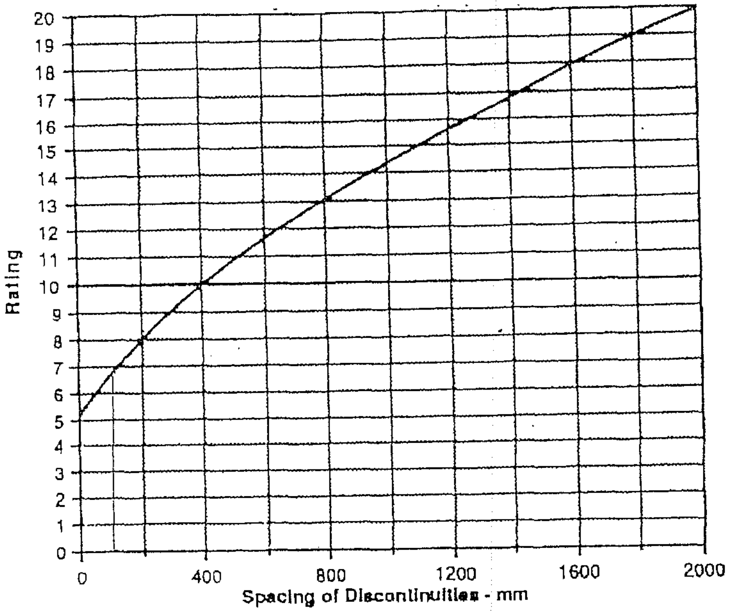
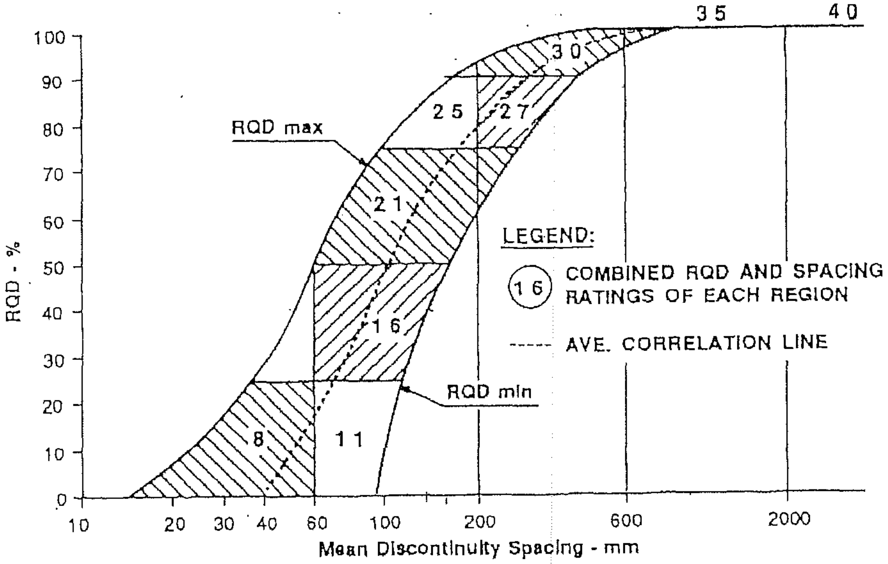


CHART D Chart for Correlation between RQD and Discontinuity Spacing



use lower range

Table 600-14. Degree of Fracturing in Bedrock

Description	Spacing
Unfractured	Greater than 10 ft.
Intact	3 ft. to 10 ft.
Slightly Fractured	1 ft. to 3 ft.
Moderately Fractured	4 in. to 12 in. 4"
Fractured	2 in. to 4 in.
Highly Fractured	Less than 2 in.

B-10, B-11, B-12

Condition of Discontinuities

B-10	moderately weathered	22.5
B-11	moderately weathered	22.5
C-12	moderately weathered	22.5

weighted average = 22.5

use 22

Groundwater

- assume wet due to presence of Long Run Creek

7

Strike and Dip Orientation

- assume favorable as relatively flat lying rock

-2

should be used to determine q_{ult}. Alternatively, Table 4.4.8.1.2B may be used as a guide to estimate C_o. For rocks defined by very poor quality, the value of q_{ult} should be determined as the value of q_{ult} for an equivalent soil mass.

4.4.8.1.3 Factors of Safety

Spread footings on rock shall be designed for Group 1 loadings using a minimum factor of safety (FS) of 3.0 against a bearing capacity failure.

4.4.8.2 Settlement

4.4.8.2.1 Footings on Competent Rock

For footings on competent rock, elastic settlements will generally be less than ½ inch when footings are designed in accordance with Article 4.4.8.1.1. When elastic settlements of this magnitude are unacceptable or when the rock is not competent, an analysis of settlement based on rock mass characteristics must be made. For rock masses which have time-dependent settlement characteristics, the procedure in Article 4.4.7.2.3 may be followed to determine the time-dependent component of settlement.

4.4.8.2.2 Footings on Broken or Jointed Rock

Where the criteria for competent rock are not met, the influence of rock type, condition of discontinuities and degree of weathering shall be considered in the settlement analysis.

The elastic settlement of footings on broken or jointed rock may be determined using the following:

- For circular (or square) footings;

$$\rho = q_o (1 - \nu^2) r I_p / E_m, \text{ with } I_p = (\sqrt{\pi})/\beta_z \qquad (4.4.8.2.2\text{-}1)$$

- For rectangular footings;

TABLE 4.4.8.1.2A Values of Coefficient N_{ms} for Estimation of the Ultimate Bearing Capacity of Footings on Broken or Jointed Rock (Modified after Hoek, (1983))

Rock Mass Quality	General Description	RMR[(1)] Rating	NGI[(2)] Rating	RQD[(3)] (%)	N_{ms}[(4)] A	B	C	D	E
Excellent	Intact rock with joints spaced > 10 feet apart	100	500	95-100	3.8	4.3	5.0	5.2	6.1
Very good	Tightly interlocking, undisturbed rock with rough unweathered joints spaced 3 to 10 feet apart	85	100	90-95	1.4	1.6	1.9	2.0	2.3
Good	Fresh to slightly weathered rock, slightly disturbed with joints spaced 3 to 10 feet apart	65	10	75-90	0.28	0.32	0.38	0.40	0.46
Fair	Rock with several sets of moderately weathered joints spaced 1 to 3 feet apart	44	1	50-75	0.049	0.056	0.066	0.069	0.081
Poor	Rock with numerous weathered joints spaced 1 to 20 inches apart with some gouge	23	0.1	25-50	0.015	0.016	0.019	0.020	0.024
Very poor	Rock with numerous highly weathered joints spaced < 2 inches apart	3	0.01	<25	Use q_{ult} for an equivalent soil mass				

(1)Geomechanics Rock Mass Rating (RMQ) System—Bieniawski, 1988.
(2)Norwegian Geotechnical Institute (NGI) Rock Mass Classification System, Barton, et al., 1974.
(3)Range of RQD values provided for general guidance only; actual determination of rock mass quality should be based on RMR or NGI rating systems.
(4)Value of N_{ms} as a function of rock type; refer to Table 4.4.8.1.2B for typical range of values of C_o for different rock type in each category.

TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	C_o[1] (ksf)	C_o[1] (psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800-45,000
		Limestone	500- 6,000	3,500-42,000
		Carbonatite	800- 1,500	5,500-10,000
		Marble	800- 5,000	5,500-35,000
		Tactite-Skarn	2,700- 7,000	19,000-49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200-21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600-28,000
	Based on description	Phyllite	500- 5,000	3,500-35,000
	+ visual classification →	Siltstone	200- 2,500	1,400-17,000
	of nearby rock cuts/outcrops	Shale[2]	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000-30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800-32,000
		Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000-26,000
		Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000-40,000
		Gabbro	2,600- 6,500	18,000-45,000
		Gneiss	500- 6,500	3,500-45,000
		Granite	300- 7,000	2,100-49,000
		Quartzdiorite	200- 2,100	1,400-14,000
		Quartzmonzonite	2,700- 3,300	19,000-23,000
		Schist	200- 3,000	1,400-21,000
		Syenite	3,800- 9,000	26,000-62,000

(1)Range of Uniaxial Compressive Strength values reported by various investigations.
(2)Not including oil shale.

$$p = q_o(1 - \nu^2)BI_p/E_m, \text{ with } I_p = (L/B)^{1/2}/\beta_z \qquad (4.4.8.2.2\text{-}2)$$

Values of I_p may be computed using the β_z values presented in Table 4.4.7.2.2B from Article 4.4.7.2.2 for rigid footings. Values of Poisson's ratio (υ) for typical rock types are presented in Table 4.4.8.2.2A. Determination of the rock mass modulus (E_m) should be based on the results of in-situ and laboratory tests. Alternatively, values of E_m may be estimated by multiplying the intact rock modulus (E_o) obtained from uniaxial compression tests by a reduction factor (α_E) which accounts for frequency of discontinuities by the rock quality designation (RQD), using the following relationships (Gardner, 1987):

$$E_m = \alpha_E E_o \qquad (4.4.8.2.2\text{-}3)$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \qquad (4.4.8.2.2\text{-}4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_o (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_E = 0.15$ should be used to estimate E_m.

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

The overall stability of footings, slopes, and foundation soil or rock shall be evaluated for footings located on

Job No. 000000000045878 PID No. 19415

HDR Computation

Project SCI-823-0.00/6.81 Portsmouth Bypass Phase I — Computed JSA — Date 3/17/2008

Subject Bridge No. SCI-823-0917L — Checked DMV — Date 3/28/2008

Task Primary Consolidation Settlement Evaluation (Rear Abutment) — Sheet 1 Of 2

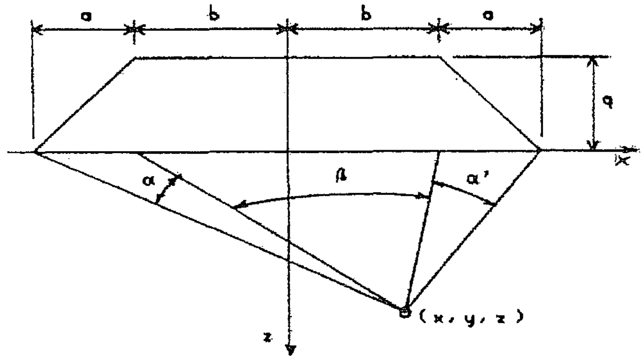
References:

1. EM 1110-1-1904 "Settlement Analyses"
2. Advanced Soil Mechanics (2nd Edition) - B. M. Das (1997)
3. Training Course in Geotechnical & Foundation Engineering - Publication No. FHWA HI-97-021 (1997)
4. DLZ Subsurface Investigation Embankments (Station 416+00 to 509+50) Project SCI-823-6.81 Phase 1 - Stage 1 Scioto County, Ohio

Assumptions:

1. Terzaghi's one-dimensional consolidation theory applies.

Stress Due to Additional Fill (Embankment)



Groundwater Table: D = 12 ft
Embankment Height: H = 38 ft
Fill Unit Weight: γ_{emb} = 125 pcf q = 4750 psf
Surcharge: P = 240 psf
Width of Slope: a = 76 ft
Top half-width of Emb.: b = 50
Distance from CL: x = 0
Output Range: z = 0 to 12.5 ft
Drainage: Single

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

$$\beta(z) = a\tan\left[\frac{(b-x)}{z}\right] + a\tan\left[\frac{(b+x)}{z}\right]$$

$$\alpha'(z) = a\tan\left[\frac{(a+b-x)}{z}\right] - a\tan\left[\frac{(b-x)}{z}\right]$$

$$\alpha(z) = a\tan\left[\frac{(a+b+x)}{z}\right] - a\tan\left[\frac{(b+x)}{z}\right]$$

Layer No.	Soil Description	Bottom Layer	z	α(z)	α'(z)	β(z)	σ_v (psf)	P (psf)	$\Delta\sigma_z'$ (psf)
1	Silt (A-4b)	5.5	2.75	0.0	0.0	3.0	4687.3	240	4927.3
2	Silt & Clay (A-6b)	12.5	9	0.1	0.1	2.8	4545.0	240	4785.0
3									
4									
5									
6									
7									
8									
9									
10									

Note: Profile based upon Boring B-003-0-08.

Job No. 000000000045878 | PID No. 19415

HDR Computation

Project SCI-823-0.00/6.81 Portsmouth Bypass Phase I | Computed JSA | Date 3/17/2008

Subject Bridge No. SCI-823-0917L | Checked DMV | Date 3/28/2008

Task Primary Consolidation Settlement Evaluation (Rear Abutment) | Sheet 2 | Of 2

Normally Consolidated Soil

$$S = \sum \left[\frac{C_c}{1+e_0} \cdot H \cdot \log \left(\frac{\sigma_f'}{\sigma_0'} \right) \right]$$

Overlyconsolidated Soil ($\sigma'_0 < \sigma'_c$)

$$S = \sum \left[\frac{C_r}{1+e_0} \cdot H \cdot \log \left(\frac{\sigma'_f}{\sigma_0'} \right) \right]$$

Overlyconsolidated Soil ($\sigma'_0 < \sigma'_c < \sigma_f$)

$$S = \sum \left[\frac{C_r}{1+e_0} \cdot H \cdot \log \left(\frac{\sigma_c'}{\sigma_0'} \right) + \frac{C_c}{1+e_0} \cdot H \cdot \log \left(\frac{\sigma_f'}{\sigma_c'} \right) \right]$$

Layer No.	Soil Description	Bottom Layer	γ_{soil} (pcf)	σ_o' (psf)	$\Delta\sigma_z'$ (psf)	σ_f' (psf)	σ_c' (psf)	C_c	C_r	e_o	S
1*	Silt (A-4b)	5.5	122	335.5	4927.3	5262.8	5262.8	0.09	0.01	0.238	0.048
2*	Silt & Clay (A-6b)	12.5	122	1098.0	4785.0	5883.0	5883.0	0.09	0.01	0.238	0.037
3											
4											
5											
6											
7											
8											
9											
10											

Notes:

1) γ_{soil} & w=9% based on Lab Testing.
2) Cc = w/100
3) C_r = w/1000
4) $e_0 = G_s$*w/100

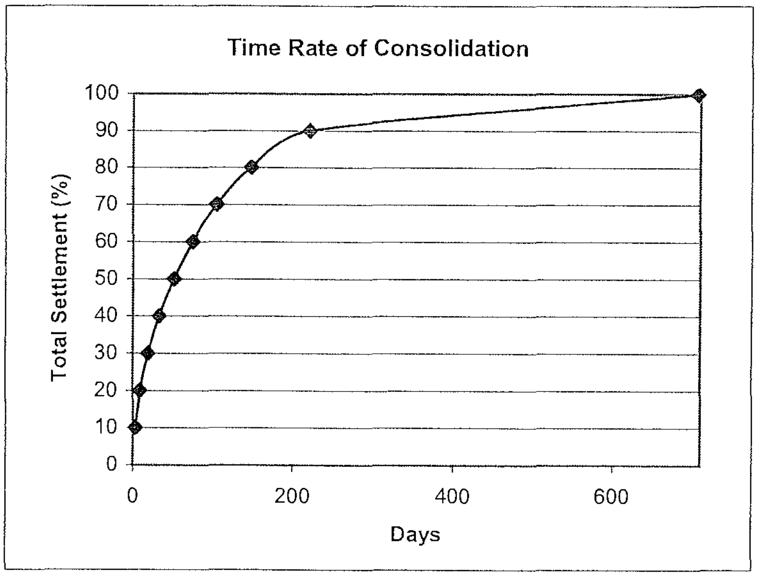
0.085 ft
Total Settlement **1.019** in

TIME RATE OF CONSOLIDATION

U_{av}	T_v	H_{dr}	C_v (ft^2/day)	Time
10	0.01	12.5	0.6	3
20	0.03	12.5	0.6	9
30	0.07	12.5	0.6	19
40	0.13	12.5	0.6	33
50	0.20	12.5	0.6	52
60	0.29	12.5	0.6	75
70	0.40	12.5	0.6	105
80	0.57	12.5	0.6	148
90	0.85	12.5	0.6	221
99.9	2.71	12.5	0.6	707

Note:

1) C_v from DLZ Subsurface Investigation Embankments Project SCI-823-6.81 (Sta 487+00).



Time Rate of Consolidation

	U_{av}		T_v		Time	
Time to reach 0.5 in settlement remaining:	51	→	0.20	→	53	days

Job No. 000000000045878

PID No. 19415

HDR Computation

Project SCI-823-0.00/6.81 Portsmouth Bypass Phase I

Computed JSA

Date 3/17/2008

Subject Bridge No. SCI-823-0917L

Checked DMV

Date 3/28/2008

Task Primary Consolidation Settlement Evaluation (Forward Abutment)

Sheet 1 Of 2

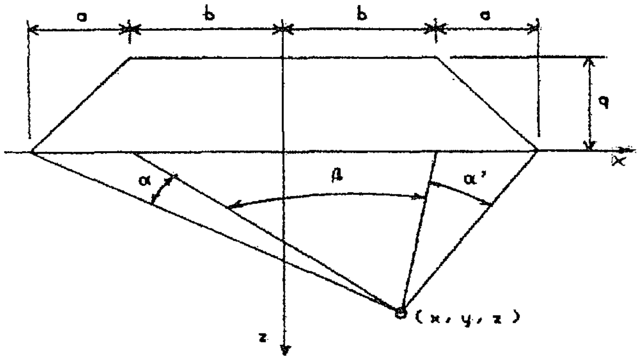
References:

1. EM 1110-1-1904 "Settlement Analyses"
2. Advanced Soil Mechanics (2nd Edition) - B. M. Das (1997)
3. Training Course in Geotechnical & Foundation Engineering - Publication No. FHWA HI-97-021 (1997)
4. DLZ Subsurface Investigation Embankments (Station 416+00 to 509+50) Project SCI-823-6.81 Phase 1 - Stage 1 Scioto County, Ohio

Assumptions:

1. Terzaghi's one-dimensional consolidation theory applies.

Stress Due to Additional Fill (Embankment)



Groundwater Table:	D =	6	ft			
Embankment Height:	H =	58	ft			
Fill Unit Weight:	γ_{emb} =	125	pcf	q =	7250	psf
Surcharge:	P =	240	psf			
Width of Slope:	a =	116	ft			
Top half-width of Emb.:	b =	50				
Distance from CL:	x =	0				
Output Range:	z =	0	to	8.7	ft	
Drainage:	Single					

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

$$\beta(z) = a\tan\left[\frac{(b-x)}{z}\right] + a\tan\left[\frac{(b+x)}{z}\right]$$

$$\alpha'(z) = a\tan\left[\frac{(a+b-x)}{z}\right] - a\tan\left[\frac{(b-x)}{z}\right]$$

$$\alpha(z) = a\tan\left[\frac{(a+b+x)}{z}\right] - a\tan\left[\frac{(b+x)}{z}\right]$$

Layer No.	Soil Description	Bottom Layer	z	α(z)	α'(z)	β(z)	σ_v (psf)	P (psf)	$\Delta\sigma_z'$ (psf)
1	Sandy Silt (A-4a)	8.7	4.35	0.1	0.1	3.0	7135.1	240	7375.1
2									
3									
4									
5									
6									
7									
8									
9									
10									

Note: Profile based on Boring B-10 (Critical Soil Profile).

Job No. 000000000045878

PID No. 19415

HDR Computation

Project SCI-823-0.00/6.81 Portsmouth Bypass Phase I

Computed JSA

Date 3/17/2008

Subject Bridge No. SCI-823-0917L

Checked DMV

Date 3/28/2008

Task Primary Consolidation Settlement Evaluation (Forward Abutment)

Sheet 2

Of 2

Normally Consolidated Soil

$$S = \sum\left[\frac{C_c}{1+e_0}\cdot H\cdot\log\left(\frac{\sigma_f'}{\sigma_0'}\right)\right]$$

Overlyconsolidated Soil ($\sigma'_0<\sigma'_c$)

$$S = \sum\left[\frac{C_r}{1+e_0}\cdot H\cdot\log\left(\frac{\sigma'_f}{\sigma_0'}\right)\right]$$

Overlyconsolidated Soil ($\sigma'_0<\sigma'_c<\sigma_f$)

$$S = \sum\left[\frac{C_r}{1+e_0}\cdot H\cdot\log\left(\frac{\sigma'_c}{\sigma_0'}\right)+\frac{C_c}{1+e_0}\cdot H\cdot\log\left(\frac{\sigma_f'}{\sigma_c'}\right)\right]$$

Layer No.	Soil Description	Bottom Layer	γ_{soil} (pcf)	σ_o' (psf)	$\Delta\sigma_z'$ (psf)	σ_f' (psf)	σ_c' (psf)	C_c	C_r	e_o	S
1*	Sandy Silt (A-4a)	8.7	122	271.0	7375.1	7646.1	7646.1	0.15	0.02	0.413	0.134
2											
3											
4											
5											
6											
7											
8											
9											
10											

Notes:

1) γ_{soil} & w% based on DLZ Lab Testing (Ref. 4)

2) Cc = w/100

3) C_r = w/1000

4) $e_0 = G_s$*w/100

w=15% G_s=2.75

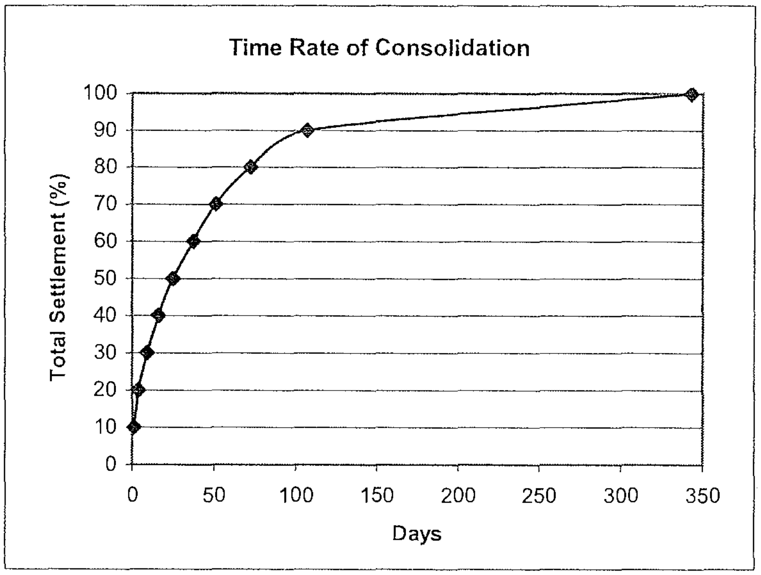
	0.134	ft
Total Settlement	**1.608**	in

TIME RATE OF CONSOLIDATION

U_{av}	T_v	H_{dr}	C_v (ft^2/day)	Time
10	0.01	8.7	0.6	1
20	0.03	8.7	0.6	4
30	0.07	8.7	0.6	9
40	0.13	8.7	0.6	16
50	0.20	8.7	0.6	25
60	0.29	8.7	0.6	37
70	0.40	8.7	0.6	51
80	0.57	8.7	0.6	72
90	0.85	8.7	0.6	107
99.9	2.71	8.7	0.6	343

Note:

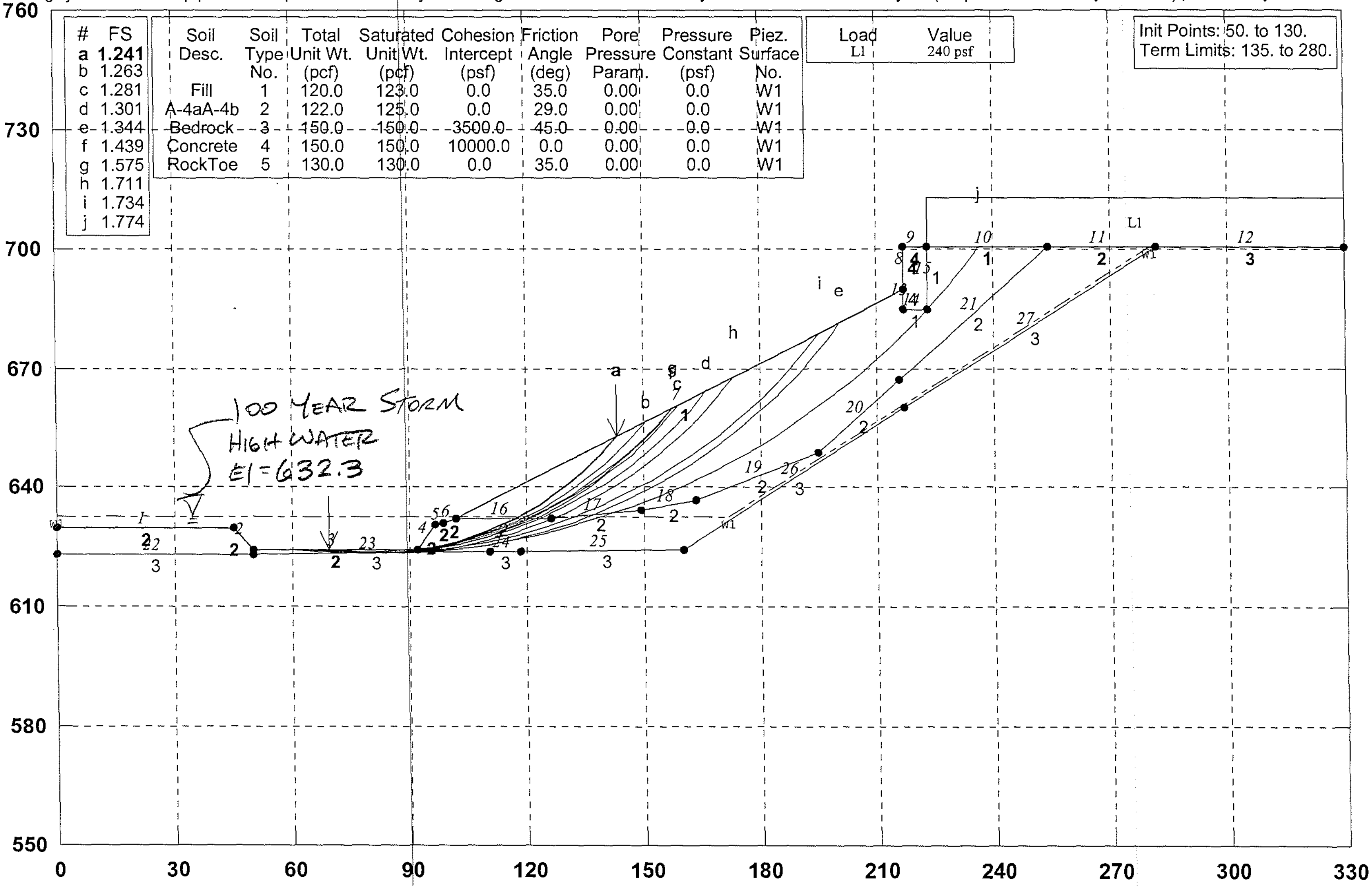
1) C_v from DLZ Subsurface Investigation Embankments Project SCI-823-6.81 (Sta 487+00).



	U_{av}		T_v		Time	
Time to reach 0.5 in settlement remaining:	69	→	0.39	→	49	days

Bridge No. SCI-823-0917-L Rear Abutment (Short-Term/Long-Term)

c:\documents and settings\juanders\desktop\portsmouth\portsmouth stability runs\bridge no. sci-823-0917-l stability runs\rear abutment\no key ltst (dlz parameters - 100 year storm).pl2 Run By: Username 4/24/2008 0



#	FS
a	**1.241**
b	1.263
c	1.281
d	1.301
e	1.344
f	1.439
g	1.575
h	1.711
i	1.734
j	1.774

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	120.0	123.0	0.0	35.0	0.00	0.0	W1
A-4aA-4b	2	122.0	125.0	0.0	29.0	0.00	0.0	W1
Bedrock	3	150.0	150.0	3500.0	45.0	0.00	0.0	W1
Concrete	4	150.0	150.0	10000.0	0.0	0.00	0.0	W1
RockToe	5	130.0	130.0	0.0	35.0	0.00	0.0	W1

Load	Value
L1	240 psf

Init Points: 50. to 130.
Term Limits: 135. to 280.

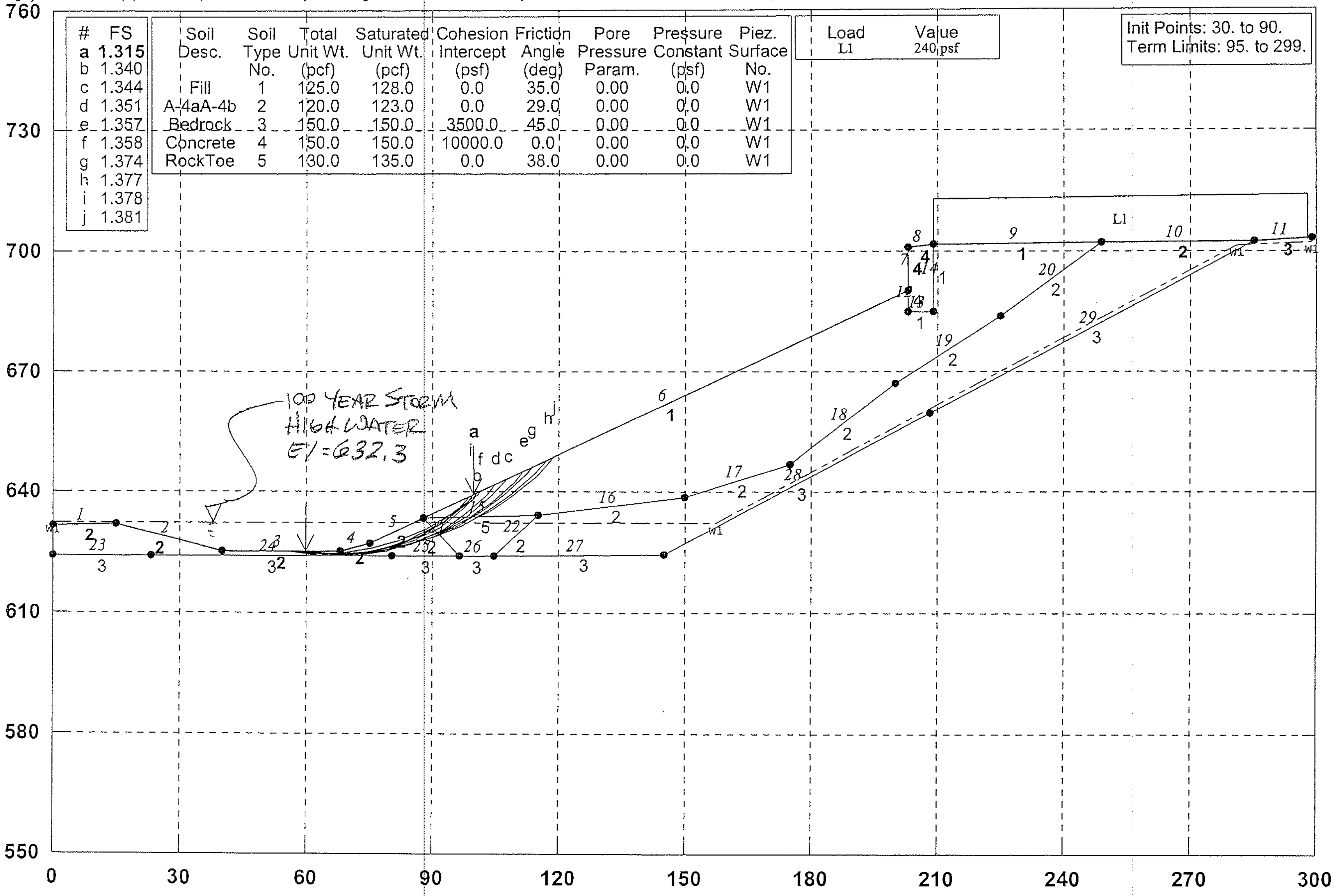
100 YEAR STORM
HIGH WATER
El=632.3

GSTABL7 v.2 FSmin=1.241
Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

Bridge No. SCI-823-0917-L Rear Abutment (Short-Term/Long-Term)

c:\documents and settings\juanders\desktop\portsmouth\portsmouth stability runs\bridge no. sci-823-0917-l stability runs\east abutment\add 10' x 10' shear key to toe (100 year storm high water)\east abutment - st (dlz parameters) 8'x8' key.pl2 Run By: Username 3/28/2008 1



#	FS
a	**1.315**
b	1.340
c	1.344
d	1.351
e	1.357
f	1.358
g	1.374
h	1.377
i	1.378
j	1.381

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	125.0	128.0	0.0	35.0	0.00	0.0	W1
A-4aA-4b	2	120.0	123.0	0.0	29.0	0.00	0.0	W1
Bedrock	3	150.0	150.0	3500.0	45.0	0.00	0.0	W1
Concrete	4	150.0	150.0	10000.0	0.0	0.00	0.0	W1
RockToe	5	130.0	135.0	0.0	38.0	0.00	0.0	W1

Load	Value
L1	240 psf

Init Points: 30. to 90.
Term Limits: 95. to 299.

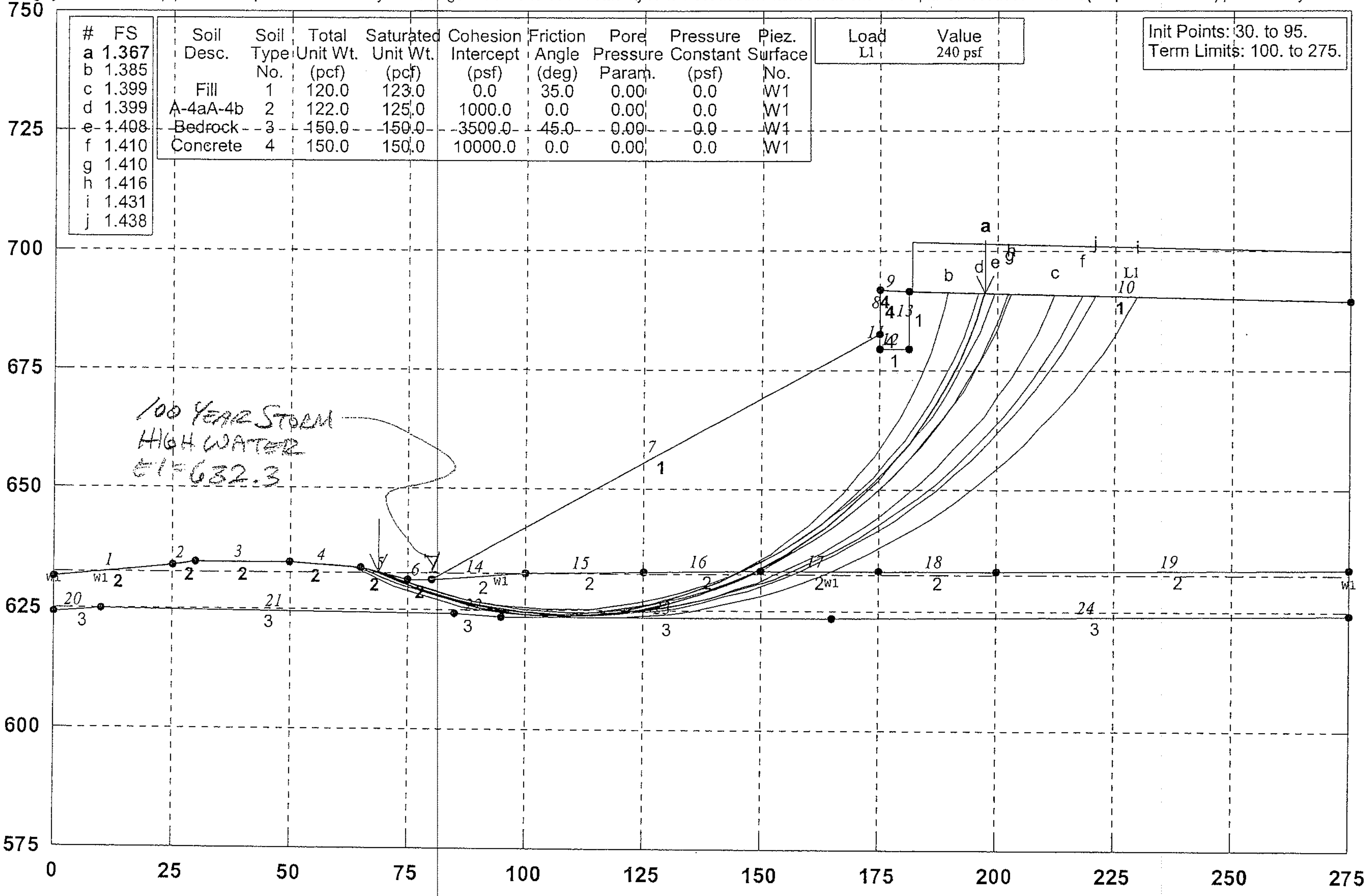
GSTABL7 v.2 FSmin=1.315

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

Bridge No. SCI-823-0917-L Forward Abutment (Short-Term)

c:\documents and settings\juanders\desktop\portsmouth\portsmouth stability runs\bridge no. sci-823-0917-l stability runs\west abutment\model as shown on plans\west abutment - st (dlz parameters - 2).pl2 Run By: Username 3/28/2008 1



#	FS
a	1.367
b	1.385
c	1.399
d	1.399
e	1.408
f	1.410
g	1.410
h	1.416
i	1.431
j	1.438

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	120.0	123.0	0.0	35.0	0.00	0.0	W1
A-4aA-4b	2	122.0	125.0	1000.0	0.0	0.00	0.0	W1
Bedrock	3	150.0	150.0	3500.0	45.0	0.00	0.0	W1
Concrete	4	150.0	150.0	10000.0	0.0	0.00	0.0	W1

Load	Value
L1	240 psf

Init Points: 30. to 95.
Term Limits: 100. to 275.

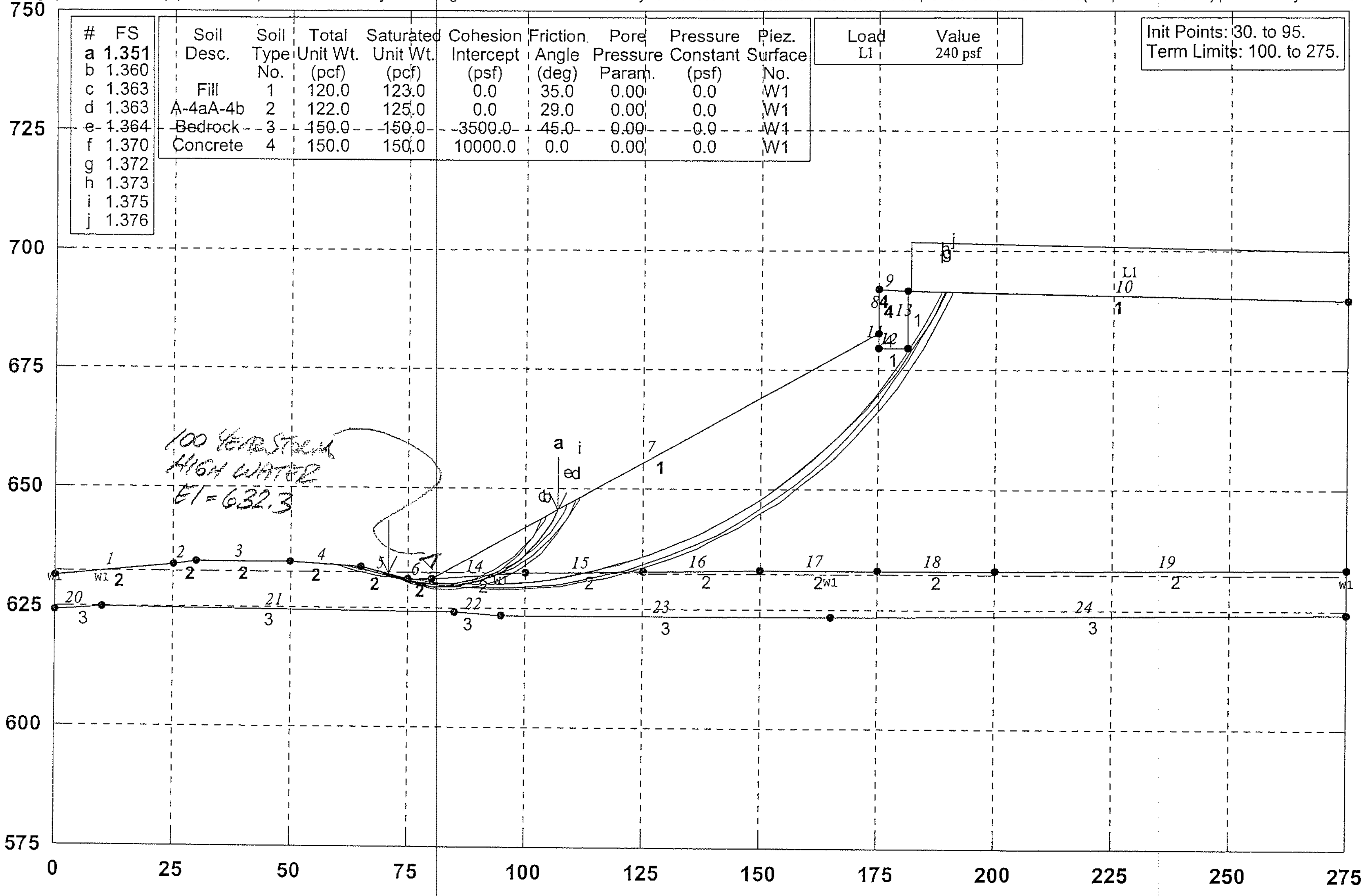
GSTABL7 v.2 FSmin=1.367

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

Bridge No. SCI-823-0917-L Foward Abutment (Long-Term)

c:\documents and settings\juanders\desktop\portsmouth\portsmouth stability runs\bridge no. sci-823-0917-l stability runs\west abutment\model as shown on plans\west abutment - lt (dlz parameters - 2).pl2 Run By: Username 3/28/2008 1



#	FS
a	**1.351**
b	1.360
c	1.363
d	1.363
e	1.364
f	1.370
g	1.372
h	1.373
i	1.375
j	1.376

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	120.0	123.0	0.0	35.0	0.00	0.0	W1
A-4aA-4b	2	122.0	125.0	0.0	29.0	0.00	0.0	W1
Bedrock	3	150.0	150.0	3500.0	45.0	0.00	0.0	W1
Concrete	4	150.0	150.0	10000.0	0.0	0.00	0.0	W1

Load	Value
L1	240 psf

Init Points: 30. to 95.
Term Limits: 100. to 275.

100 YEAR STORM
HIGH WATER
El = 632.3

GSTABL7 v.2 FSmin=1.351
Safety Factors Are Calculated By The Modified Bishop Method