GEOTECHNICAL EXPLORATION
Bridge No. SCI-823-0837L
SR 823 over Swauger Valley-Minford Road

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

PREPARED FOR:
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<u>APPENDICES</u>

1.0 INTRODUCTION

This report presents the results of HDR Engineering, Inc.'s geotechnical study for Bridge No. SCI-823-0837 L, SR 823 over Swauger Valley 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 60-foot high MSE walls required to retain the roadway embankment and provide lateral resistance for the pile-supported bridge abutments. 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 Swauger Valley-Minford Road, 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-823-0837 L. The proposed bridge is a 387-foot long, 4 span structure designed to carry traffic over Swauger Valley-Minford Road and Harrison Furnace Creek. 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 441+02 and Station 444+88, respectively, and are anticipated to be reinforced concrete stub abutments supported on steel H-piles. Pier 1 will be located at Station 442+00 and Pier 2 at Station 442+95, on the opposite sides of Swauger Valley-Minford Road. Pier 3 will be located to the west of Harrison Furnace Creek at Station 443+92. 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-823-0837 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-Wharton-Latham, 25 to 40 percent slopes (SfE) – The soils associated with the Shelocta-Wharton-Latham Association are typically found along steep hillsides. They are well drained with moderately high to high permeabilities and available water capacities are low to high. The parent material for these soils is colluvium over residuum and the depth to water table is typically from 18 inches to over 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 moderate risk of erodibility due to the steepness of the slopes, particularly in regards to the Shelocta component.

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 Swauger Valley-Minford Road, SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006) located in Appendix B. Please note that the geology overview indicates that the potentially problematic Minford Silts are present within the project area; however, these soils 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 432+00 to Station 442+00 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, and was attributed to erosion from logging activities within the area as well as intermittent streams running through the valley. No deep seated landslides were observed by DLZ along the proposed alignment, nor were signs of instability noted at the bridge site 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. Nine 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 near the proposed substructure units (see Figure 2), two new borings, designated as B-001-0-08 and B-002-0-08, were respectively located at the forward and rear abutments of the structure to supplement the existing information at the site. These test borings were located and staked in the field by TesTech, Inc. with stations and offsets developed by HDR from the coordinates and elevations provided.

Drilling and sampling of the new borings was performed on February 5, 2008. An ATV mounted CME 550 drill rig equipped with a 3¼" inside diameter hollow stem auger was used to advance the borings. The borings were 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 Borings B-001-0-08 and B-002-0-08 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 Borings B-001-0-08 and B-002-0-08 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 11 natural moisture contents (ASTM D 2216), 5 Atterberg limit determinations (ASTM D 4318), 5 grain size analyses (ASTM D 422), 2 unconfined compressive strength tests (ASTM D 2166) and 1 one-dimensional consolidation test (ASTM D 2435). 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 Swauger Valley-Minford Road, SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006) located in Appendix B.

5.1 Previous Exploration Programs

Nine 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, four preliminary structural borings designated as TR-20 through TR-23 were performed by DLZ between August 3, 2004 and February 24, 2005, and five final structural borings, designated as B-5 through B-9, were performed between June 15 and 16, 2006. The locations of these nine borings as related to the current bridge plan are presented in Figure 2.

In general, the previous test borings at the site encountered 1 to 8 inches of topsoil overlying a relatively thin layer of primarily granular soils. The overburden typically extended from approximately 1.5 to 7.5 feet below the existing ground surface, and was described as silt (A-4b), sandy silt (A-4a), and silt and clay (A-6a). SPT N-values ranged from 4 to 56 blows/foot within the overburden, with the granular soils noted to be medium dense to very dense while the soils with a more appreciable cohesive component were typically described as medium stiff to hard.

The underlying bedrock was described as very fine to fine grained, argillaceous, micaceous sandstone. Typically, the sandstone was described as medium hard to hard, moderately to highly weathered, slightly to highly fractured, and massively bedded with a few laminated zones. The amount of core recovery varied from 81 to 100 percent, with an average recovery of 94 percent. The rock quality designation (RQD) for the sandstone ranged between 66 and 100 percent, with an average RQD of 81 percent. Unconfined compressive strength tests performed on five intact core samples from the final structural borings indicated unconfined compressive strengths ranging from 7,966 and 13,418 psi, with an average unconfined compressive strength of 9,783 psi.

5.2 Recent Exploration Program (Forward and Rear Abutments)

This section summarizes the subsurface conditions encountered during HDR's field exploration program. The typed test boring logs and photographs of the recovered rock core for borings B-001-0-08 and B-002-0-08 are included in Appendix D.

Borings B-001-0-08 and B-002-0-08 encountered 10.1 to 11.2 feet of fine-grained residuum overlying sedimentary bedrock. The residual soils encountered at the abutment locations were classified as silt and clay (CL, A-6a), silty clay (CL, A-6b) and silt (CL/ML, A-4b). SPT N-values within the overburden ranged from 7 blows/foot to in excess of 50 blows/foot, with the soils becoming more competent with depth.

At Boring B-001-0-08, bedrock was encountered at El. 660.1 and consists of argillaceous siltstone with an approximate 1.7-foot thick interbed of silty shale encountered from approximate El. 652.77 to El. 651.07. The sedimentary rock was described as slightly weathered to unweathered, with unit RQD values ranging from 82 to 100 percent, signifying very good quality rock. The core recoveries were generally good and ranged from 87 to 100 percent, with the lower recovery rates encountered within the upper rock stratum. At Boring B-002-0-08, the underlying sedimentary rock consists primarily of siltstone (some argillaceous), with the top of rock encountered at approximate El. 659.3. The siltstone was described as highly weathered to unweathered, with the degree of weathering decreasing with depth. Unit RQD values ranged from 77 to 93 percent, signifying good quality rock, with no core loss reported. The results of two unconfined compressive tests on intact core samples indicated unconfined compressive strengths (q_u) within the siltstone ranging from 8,833 to 10,336 psi, with an average unconfined strength of approximately 9,628 psi.

5.3 Summary of Subsurface Conditions

As noted previously, Bridge No. SCI-823-0837 L was modified from two spans to four spans in order to eliminate the approximate 60-foot high MSE walls required to retain the roadway embankment and provide lateral resistance for the pile-supported bridge abutments. Several of the substructure units were repositioned and four new T-type piers added under the new bridge design; however, the subsurface exploration program as previously performed by DLZ had already been completed under the original two-span bridge design. As shown in Table 1, these previously drilled test borings are located from approximately 10 to 40 feet from the currently proposed substructure locations; therefore, variations in the estimated top of bedrock at the proposed substructure locations is 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	441+01.4, CL	B-001-0-08	441+05.0, 63.0 ft. RT	670.2	660.1
		TR-23	441+30.3, 48.1 ft. LT	661.0	653.5
Pier 1	441+98.3, CL	B-9	441+98.6, 66.2 ft. LT	647.5	643.5
Pier 2	442+95.0, CL	B-6	443+23.0, 34.6 ft. RT	635.9	629.9
		B-8	443+05.8, 34.6 ft. LT	638.4	630.9
Pier 3	443+91.8, CL	TR-21	443+67.0, 46.5 ft. LT	639.0	637.5
		B-5	444+30.2, 63.3 ft. RT	644.0	642.5
		B-7	444+00.8, 65.4 ft. LT	658.0	655.5
Forward Abutment	444+88.6, CL	B-002-0-08	444+75.0, 72.0 ft. LT	670.5	660.3
		TR-20	444+69.7, 42.1 ft. RT	650.0	645.0

Table 2 presents the proposed design elevations as noted in the Structure Type Study Report (KZF, 2008) for the individual substructure units and the top of rock as encountered at the nearby boring locations. Based on the encountered subsurface conditions, the depth to bedrock varies from approximately 2 to 11 feet below the existing ground surface at the bridge site. The top of rock was encountered from approximate El. 629 to El. 632 along the valley floor (Borings B-6, B-8, TR-22) and climbs to approximate El. 660 at both the rear and forward abutments as currently located on the valley wall (Borings B-001-0-08, B-002-0-08).

Table 2: 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	El. 665.1	El. 709.9	660.1 – 653.5	50 to 57	El. 696.57
Pier 1	El. 648.3	El. 655.0	643.5	12	El. 643.00
Pier 2	El. 636.9	El. 636.9	630.9 - 629.9	6 to 7	El. 630.40
Pier 3 (L)	El. 647.2	El. 647.2	655.5 - 637.5	2 to3	El. 631.20
Pier 3 (R)	El. 647.2	El. 647.2	642.5 - 629.9	2 to 6	El. 631.20
Forward Abutment	El. 660.7	El. 699.8	659.3 - 645.0	41 to 55	El. 686.45

Notes: 1. As encountered in the nearest test borings

2. Below proposed ground surface

6.0 ANALYSES AND DISCUSSSIONS

Spread footings and driven piles are viable options for support of Bridge No. SCI-823-0837-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 allowable axial stress of steel H-piles. The results of these and other related analyses are presented in Appendix E.

6.1 Rear Abutment

As shown in Table 2, the proposed bottom of footing/pile cap for the rear abutment is El. 696.57, approximately 31 feet above the existing ground surface (at the centerline) and roughly 50 to 57 feet above the top of rock based on borings B-001-0-08 and TR-23. Approximately 40 to 45 feet of fill will be required to attain the proposed profile grade (El. 709.9) 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 would be located within the fill. As such, steel H-piles driven to 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 achieved 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 ranging from El. 660.1 to El. 653.5 in borings B-001-0-08 and TR-23, respectively. The bedrock consists of slightly weathered to unweathered siltstone and decomposed to slightly weathered, very fine to fine grained sandstone with the degree of weathering decreasing with depth. Refusal of the driven piles is expected to be obtained relatively quickly once the top of rock is encountered, with approximately 0.5 to 2 feet of penetration into the overlying weathered rock anticipated. As such, hardened steel pile driving tips should be utilized per Section 202.2.3.2.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 from 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. As such, it is recommended that the steel H-piles be installed through pile windows constructed during placement of the approach embankment fills. 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 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.

6.2 Forward Abutment

As shown in Table 2, the proposed elevation for the bottom of footing/pile cap at the forward abutment is 686.45 feet, roughly 18 to 30 feet above the existing ground surface and approximately 27 to 42 feet above the top of rock. The proposed profile grade at the abutment is El. 699.83, indicating that approximately 30 to 45 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 from El. 659.3 to El. 645.0 at borings B-002-0-08 and TR-20, respectively. The bedrock consists of slightly weathered to unweathered siltstone and slightly weathered, very fine to fine grained sandstone. Refusal is expected to be obtained relatively quickly once the top of rock is encountered, with approximately 4 to 6 inches of penetration expected. As such, hardened steel pile driving tips should be utilized per Section 202.2.3.2.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 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 from 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. As such, it is recommended that the steel H-piles be installed through pile windows constructed during placement of the approach embankment fills. 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 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.

6.3 Bridge Piers

Based on the subsurface conditions encountered at the pier locations, bedrock is expected to be encountered within approximately 2 to 12 feet below final grade at Pier 1, 2 and 3 (See Table 2). As such, spread footings bearing upon competent rock are considered to be the most feasible foundation alternative at the bridge piers.

6.3.1 Pier 1

The top of rock at Pier 1 is anticipated at approximate El. 643.5 based on test boring B-9. However, some variation in the bedrock elevation should be expected as the boring is located approximately 40 feet from the left bridge pier and about 90 feet from the right pier. (See Figure 2.) The bedrock encountered at boring B-9 was described as medium hard to hard, very fine to fine grained, argillaceous, micaceous sandstone. The sandstone is moderately to highly weathered and noted to be highly fractured with decomposed argillaceous zones from El. 638.8 to El. 638.5. Based upon the bedrock description provided in the boring log, it is recommended that the proposed bottom of footing be located at El. 641.5 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 29 tsf is recommended.

Due to the potential for variations in the top of bedrock beneath the footing from that encountered in Boring B-9, 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.

6.3.2 Pier 2

Based on Borings B-6 and B-8, the top of rock was encountered from El. 629.9 to El. 630.9 across Pier 2, with the bedrock described as medium hard to hard, very fine to fine grained, argillaceous, micaceous, sandstone. At Boring B-8, the recovery rate was 81% for the first core run (El. 630.9 to El. 621.9), with SPT sampling terminated at El. 630.9 at a blow count of 50 blows for the last 4 inches of penetration. As such, the 21 inches of rock core that was not recovered likely represents decomposed to highly weathered sandstone from El. 630.9 to El. 629.2. At Boring B-6, several rust stained, low angle fractures were noted from El. 629.1 to El. 628.0.

A recommended bearing elevation of 627.9 at Boring B-8 and elevation 627.4 at Boring B-6 is provided in the "*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, 2006). Based upon review of the boring logs and the previously recommended bearing elevations, it is recommended that the proposed bottom of footing for Pier 2 be set at El. 627.9 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 29 tsf is recommended.

Due to the potential for variations in the top of bedrock beneath the footing from that encountered in Borings B-6 and B-8, 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.

6.3.3 Pier 3

As shown in Figure 2, Harrison Furnace Creek is located adjacent to Pier 3, with the elevation of the creek bed at approximate El. 633 and the top of bank at approximate El. 635. The existing ground surface varies from El. 635 to El. 655 at the pier location, with the top of bedrock varying from approximate El. 629.9 to El. 655.5 based on borings TR-21, B-5, B-6, B-7 and B-8. The bedrock was described as medium hard to hard, very fine to fine grained, argillaceous, micaceous sandstone. The bedrock at boring TR-21 was noted to be highly fractured to broken from El. 637.5 to El. 635.1, with a clay filled fracture noted from El. 635.7 to El. 635.6. At Boring B-5, the bedrock was noted to be highly to moderately weathered with a high angle fracture noted from El. 640.9 to El. 640.7. The bedrock at boring B-7 was noted to be highly fractured with a broken zone noted from El. 655.5 to El. 653.0. General information on the bedrock encountered at B-6 and B-8 is provided in Section 6.3.2.

Given the individual pier locations in relation to Harrison Furnace Creek and the existing bedrock conditions, it is recommended that the proposed bottom of footing for Pier 3 (L) be located at El. 633.0 or lower. For Pier 3 (R), the bottom of footing should be located at El. 628.0 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, 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 29 tsf is recommended.

Due to the potential for variations in the top of bedrock beneath the footing from that encountered at the test borings, 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.

6.4 Bridge 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. It is recommended that the durable rock fill 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 embankment 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 Swauger*

Valley-Minford Road, 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
Foundation Soil (Rear Abutment)	Medium Dense to Very Dense Silt	120	0	29	0	29
Foundation Soil (Forward Abutment)	Medium Dense to Very Dense Silt	120	0	29	0	29
Bedrock	Sandstone and Siltstone	130	3500	45	3500	45

Notes: Embankment fill will consist primarily of excavated rock (per DLZ reports)

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 for the planned 2H:1V embankment slopes are presented in Appendix E. As shown in Appendix E, the slopes are stable under both short and long-term conditions, exceeding the ODOT standard minimum required factor of safety 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 45 feet of compacted fill is expected at the centerline of the rear abutment, and over 38 feet of fill at the centerline of 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 5 to 6 inches of settlement at the rear abutment and 4 to 5 inches of settlement at the forward abutment can be expected. However, it is anticipated that most of this settlement will occur with load application during construction.

Settlement analyses were performed at Station 441+01 and Station 444+89 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 2.2 inches at Station 441+01 and approximately 1.0 inch at Station 444+89. The time needed to reach 90% consolidation is estimated at 130 to 106 days respectively.

Due to the estimated 1.0 to 2.2 inches of 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 3 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 is no longer a concern). As such, the embankments should be quarantined and monitored for a minimum of 90 days to allow the settlement to take place prior to the 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 90 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	440+60, 40 feet LT	Roadway Shoulder
	440+75, 40 feet RT	Roadway Shoulder
Forward	445+20, 40 feet LT	Roadway Shoulder
	445+30, 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 90 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 90 day period, than the quarantine period should be extended as appropriate.

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 designs.

7.1 Foundation Design

Table 5 provides a summary of the foundation design parameters for Bridge No. SCI-823-0837 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 spread footings are recommended at the bridge piers.

Table 5: Summary of Foundation Design Parameters

Substructure Unit	**Rear Abutment**	**Forward Abutment**	**Pier 1**	**Pier 2**	**Pier 3 (L)**	**Pier 3 (R)**
Foundation Type	Driven Piles	Driven Piles	Spread Footing	Spread Footing	Spread Footing	Spread Footing
Proposed Bottom of Footing/Pile Cap (El.)	696.57	686.45	641.5	627.9	633.0	628.0
Top of Bedrock (El.)	660.0 to 653.5	659.0 to 645.0	643.5	631.0 to 630.0	655.5 to 637.5	642.5 to 630.0
Estimated Tip Elevation (El.)	659.5 to 651.5	658.5 to 644.5	NA	NA	NA	NA
Estimated Pile Length[1,2]	42 ft	36 ft	NA	NA	NA	NA
Allowable Axial Stress[3,4]	12.5 ksi	12.5 ksi	NA	NA	NA	NA
Allowable Bearing Capacity	NA	NA	30 tsf	30 tsf	30 tsf	30 tsf

Notes: 1. Average Length based on encountered bedrock elevation at the test boring locations
2. Includes 1-foot embedment into cap
3. Allowable horizontal or lateral load to be developed in battered piles
4. Allowable Axial Stress does not include section loss due to corrosivity
5. NA = not applicable

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 42 feet is anticipated based on the encountered subsurface conditions at Borings B-001-0-08 and TR-23, 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.

7.1.2 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 36 feet is anticipated based on the encountered subsurface conditions at Borings B-002-0-08 and TR-20, 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 the pier be supported on spread footings bearing on rock. A bottom of footing elevation of 641.5 is recommended based on the subsurface conditions encountered at Boring B-9.
- The footings should be designed using an allowable bearing capacity of 29 tsf. For cast-in-place footings on sound bedrock, a friction factor of 0.7 recommended. Settlement of the pier footing is expected to be nominal.
- Due to the potential for variations in the top of bedrock beneath the footing from that encountered in Boring B-9, 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 material 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.

- As the approach embankment will be placed prior to construction of the substructure units, an excavation of approximately 16 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.

7.1.4 Pier 2

- It is recommended that the pier be supported on spread footings bearing on rock. A bottom of footing elevation of 627.9 is recommended based on the subsurface conditions encountered at borings B-6 and B-8.
- The footings should be designed using an allowable bearing capacity of 29 tsf. For cast-in-place footings on sound bedrock, a friction factor of 0.7 recommended. Settlement of the pier footing is expected to be nominal.
- Due to the potential for variations in the top of bedrock beneath the footing from that encountered in Borings B-6 and B-8, 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 material 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.

7.1.5 Pier 3

- Based on the subsurface conditions encountered at Pier 3, it is recommended that the pier be supported on spread footings bearing on rock. Given the individual pier locations in relation to Harrison Furnace Creek, the sloping bedrock surface at the pier locations, and the subsurface conditions encountered at borings B-5, B-6, B-7, B-8 and TR-21, it is recommended that the proposed bottom of footing be located at El. 633.0 for Pier 3 (L) and at El. 628.0 for Pier 3 (R).
- The spread footings should be designed using an allowable bearing capacity of 29 tsf. A friction factor of 0.7 is recommended for cast-in-place footings on sound bedrock. Settlement is expected to be nominal.
- Based on the existing ground surface at the site, an excavation of approximately 17 to 22 feet through soil and rock is expected in order to place the bottom of the footing at a consistent elevation. As such, the footing excavation for Pier 3 may require temporary shoring, particularly on the upslope side of the excavation.
- Due to the potential for variations in the top of bedrock beneath the footing from that encountered in the test borings, 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 material 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.

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 Swauger Valley-Minford Road, SCI-823-0.00 Portsmouth Bypass, Scioto County, Ohio*" (DLZ, 2006), seepage and/or groundwater was not encountered in any of the previous borings performed at the site. However, 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 Harrison Furnace Creek. As such, the Contractor should anticipate that the pier foundation excavations will likely require dewatering. Any excavations near Harrison Furnace 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 and forward approach embankments incorporate special benching in accordance with ODOT's Office of Geotechnical Engineering "*Geotechnical Bulletin GB2 - Special Benching and Sidehill Embankment Fills*" as the existing hillsides are steeper than 4H:1V. Per GB2, the special benching is to be 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 a minimum of 90 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 90 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 90 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-823-0837L crossing Swauger Valley-Minford Road 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. 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 – 17th Edition".*

DLZ, Inc. (2008). *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 Rate 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.

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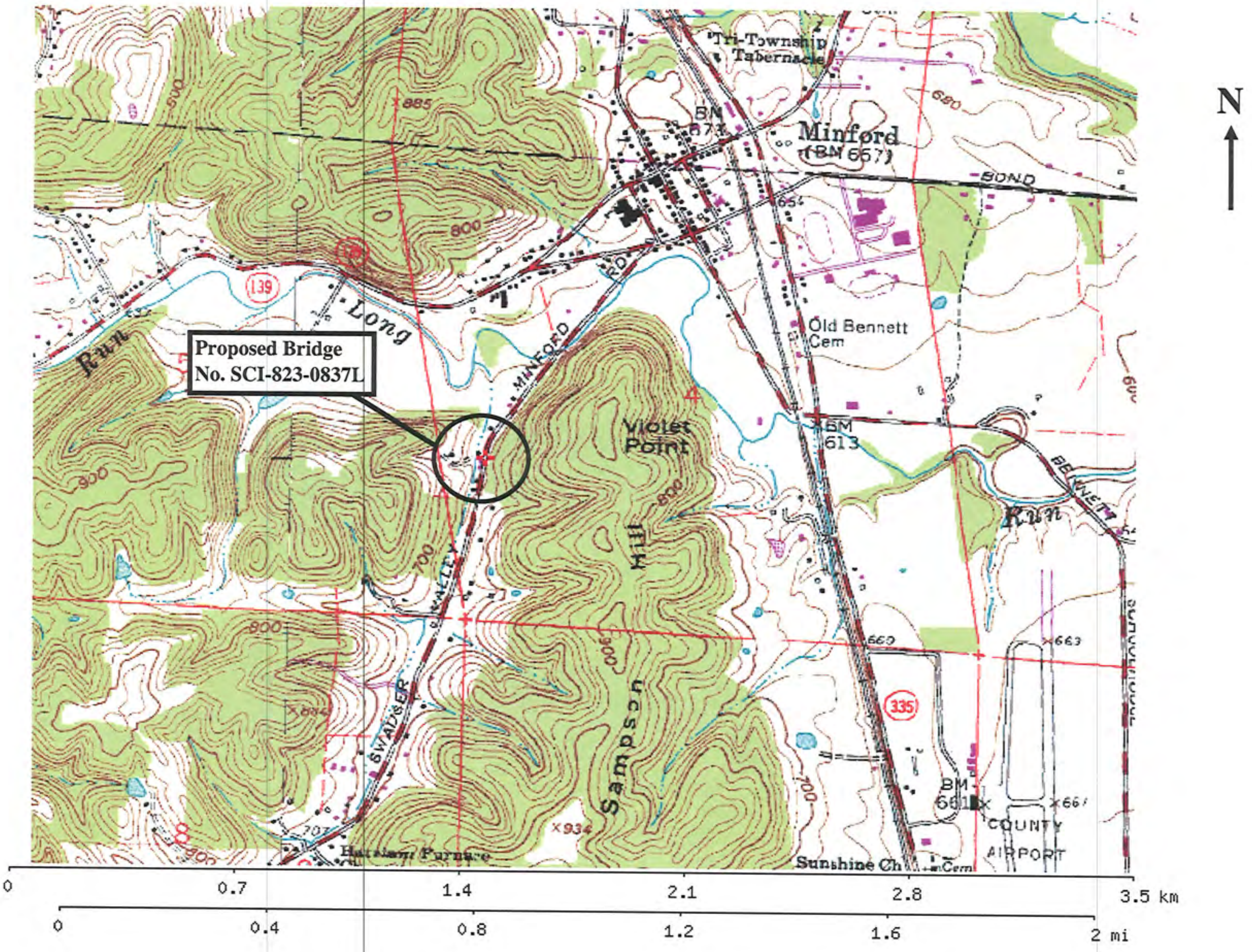
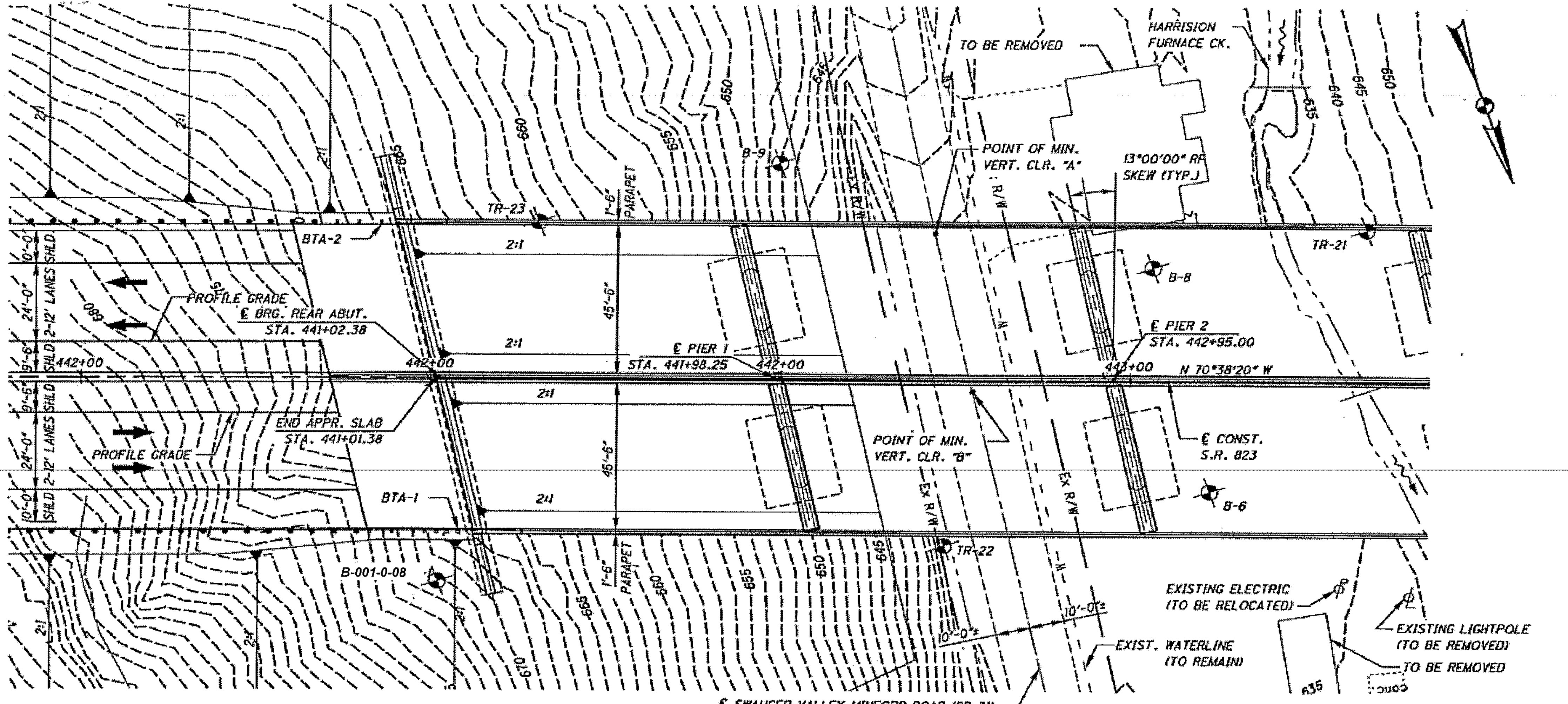
United States Department of Agriculture: Natural Resources Conservation Service (2008). *"Web Soil Survey".* <http://websoilsurvey.nrcs.usda.gov/app/>

Appendices

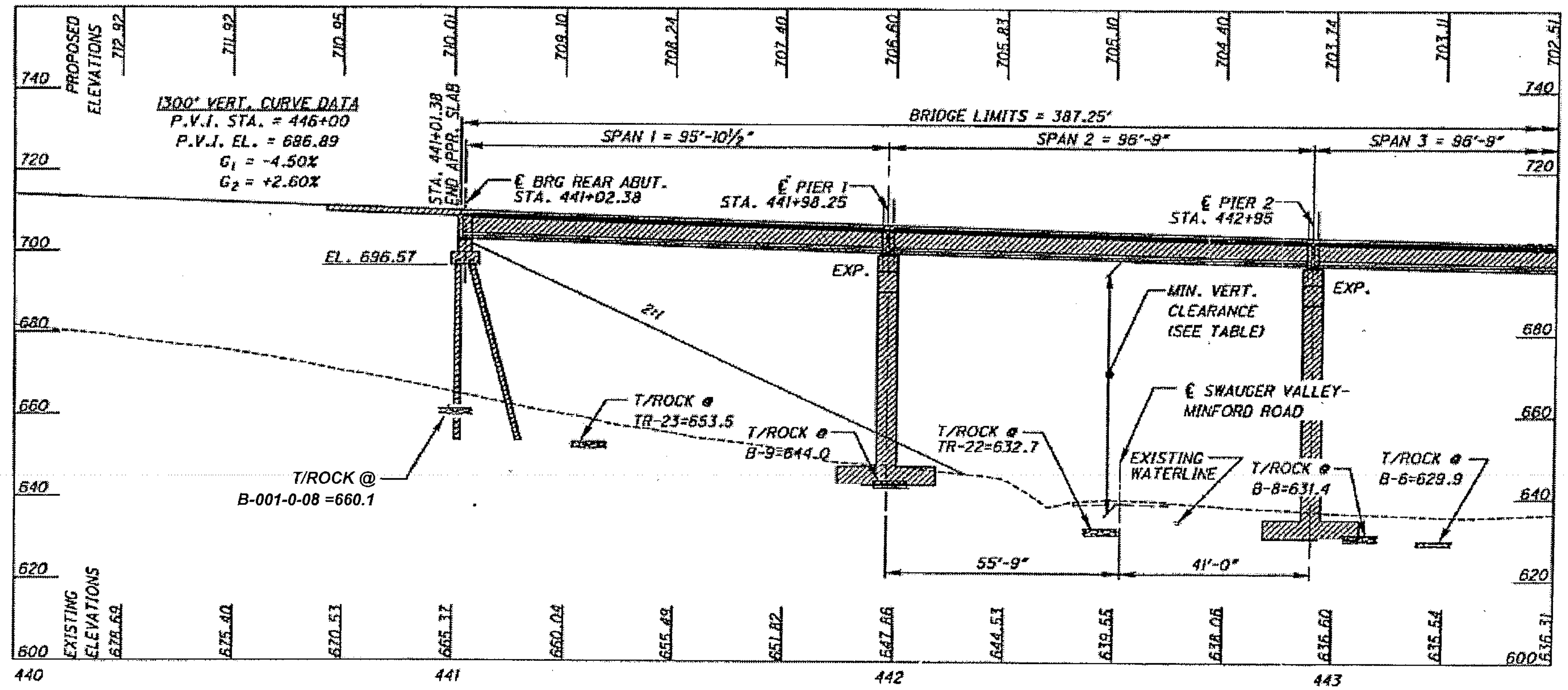
Appendix A
Figures

Figure 1 - Project Location Map





PLAN



PROFILE ALONG PROFILE GRADE S.R. 823 LEFT BRIDGE

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	OFFSET
REAR ABUT.	440+85.73	LT.
FWD. ABUT.	444+82.11	LT.

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

TRAFFIC DATA

(SR 823)

CURRENT YEAR ADT (2010) = 21,200
DESIGN YEAR ADT (2030) = 31,200
CURRENT YEAR ADTT (2010) = 2,968
DESIGN YEAR ADTT (2030) = 4,368

HYDRAULIC DATA

DRAINAGE AREA = 0.873 sq.mi. = 558.9 acres

Q_{50} = 493 cfs	Q_{100} = 581 cfs
V_{50} = 6.5 fps	V_{100} = 6.9 fps
EL 50= 638.2	EL 100= 638.5

OHWM: EL. 636.2
AREA BELOW OHWM: 0.17 ACRES
TEMP. FILL BELOW OHWM: 1130 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" DIA. C.I.P. REINFORCED CONCRETE PILES AND HAVE A MAXIMUM CAPACITY OF 70 TONS PER PILE

PROPOSED STRUCTURE

TYPE: 4 SPAN CONTINUOUS 72" MODIFIED AASHTO TYPE 4 PRESTRESSED CONCRETE I-BEAMS WITH COMPOSITE REINFORCED CONCRETE DECK ON SEMI-INTEGRAL ABUTMENTS AND T-TYPE PIERS.

SPANS: 94'-9", 94'-6", 94'-6", 94'-9" c/c BEARINGS

ROADWAY: 45'-6" TOE TO TOE OF PARAPETS

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

SKEW: 13°00'00" RF

CROWN: 0.016 FT./FT.

ALIGNMENT: TANGENT
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30 FT LONG)
LATITUDE: 38°51'00"N
LONGITUDE: 82°52'03"W

TABLE OF VERTICAL CLEARANCES	
LOCATION	"A"
PROPOSED	56.82'±
PREFERRED	15.0'

LEGEND

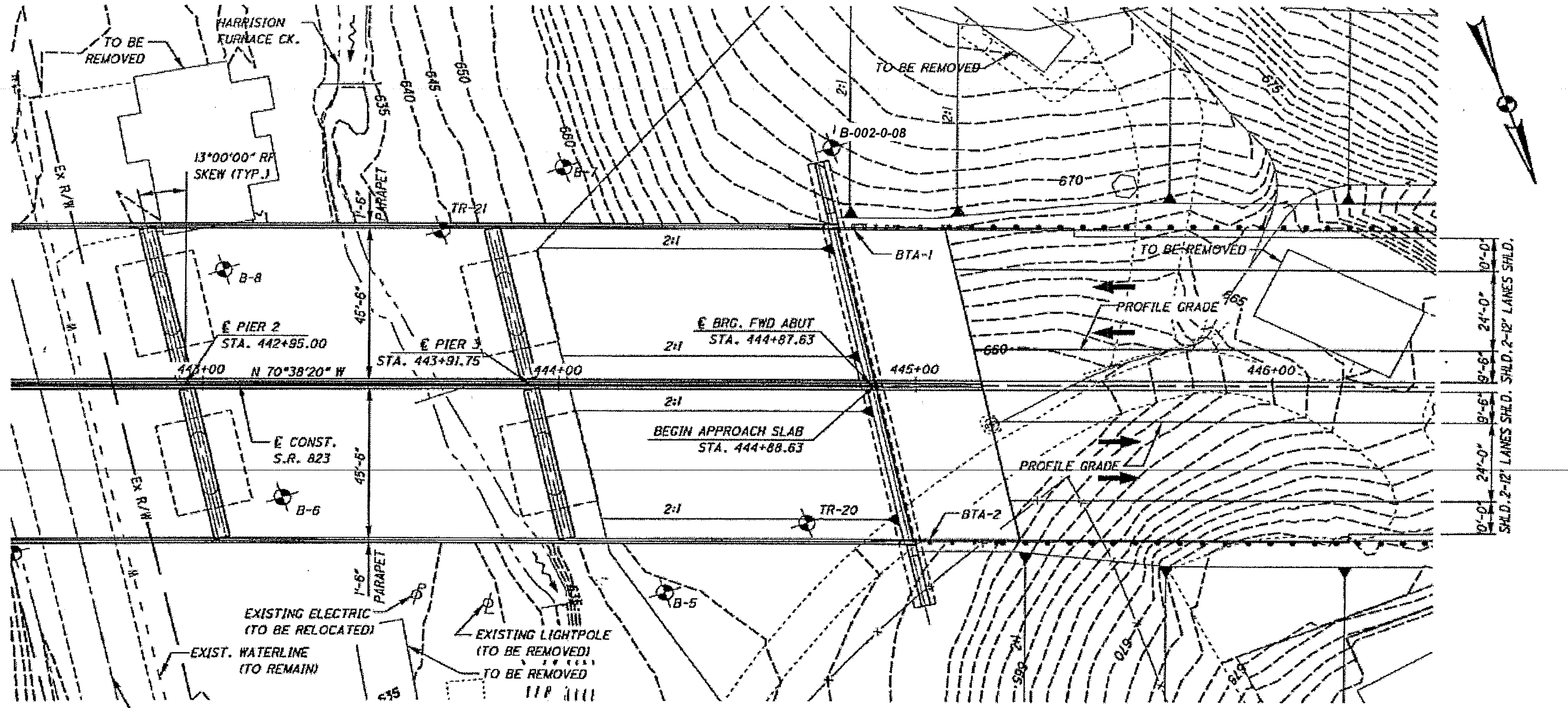
BTA-1 = BRIDGE TERMINAL ASSEMBLY TYPE 1
BTA-2 = BRIDGE TERMINAL ASSEMBLY TYPE 2
⊕ = BORING LOCATION

Figure Modified from Structure Type Study SCI-823-0837 L/R
Sheet 1/4 (KZF Design, February 2008)

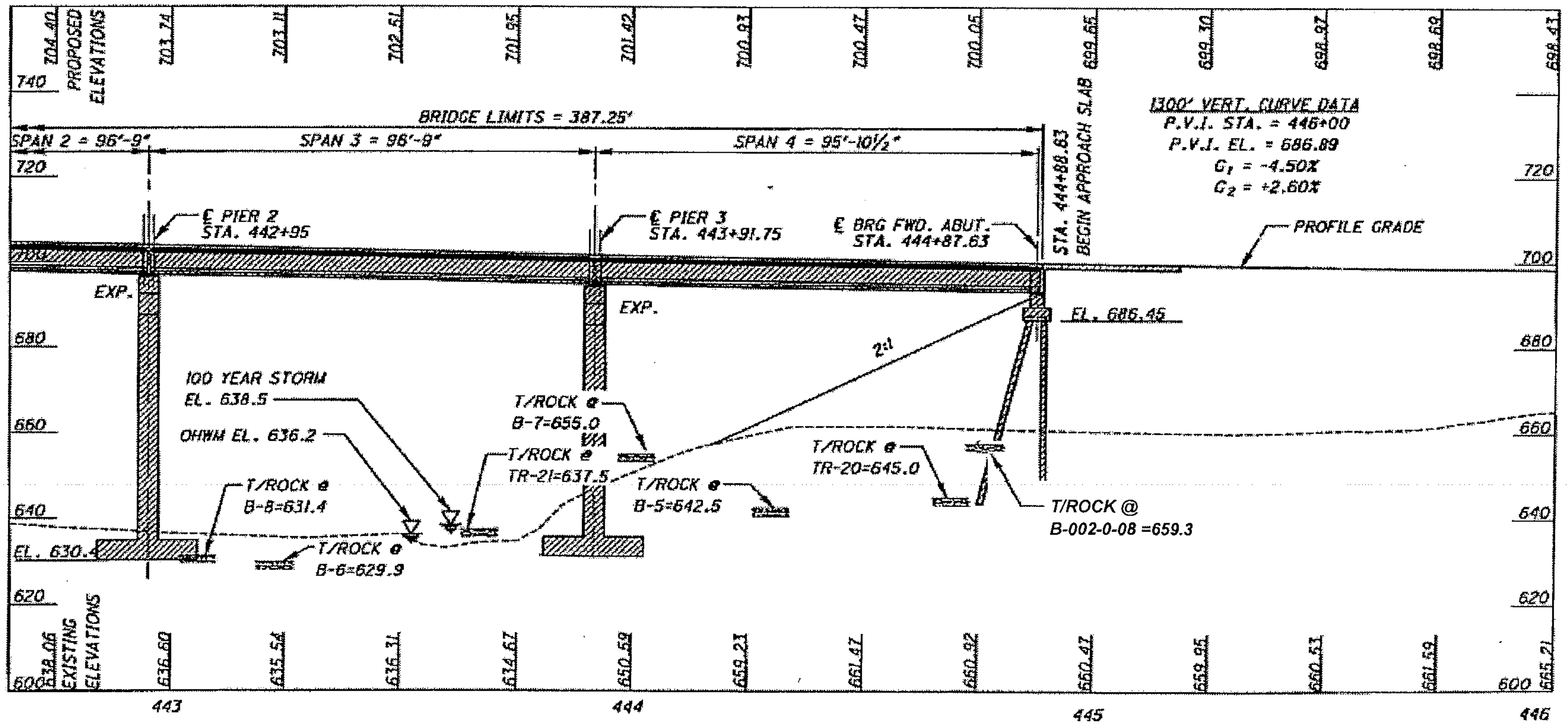
FIGURE 2 - SITE PLAN
BRIDGE NO. SCI-823-0837 L
S.R. 823 OVER SWAUGER VALLEY-MINFORD ROAD (CR-31)

SCI-823-6.81
PID 19415

1/4



PLAN



PROFILE ALONG PROFILE GRADE S.R. 823 LEFT BRIDGE

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	OFFSET
REAR ABUT.	440+85.73	LT.
FWD. ABUT.	444+82.11	LT.

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

TRAFFIC DATA

(SR 823)

CURRENT YEAR ADT (2010) = 21,200
DESIGN YEAR ADT (2030) = 31,200
CURRENT YEAR ADTT (2010) = 2,968
DESIGN YEAR ADTT (2030) = 4,368

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V_{50} = 6.5 fps	V_{100} = 6.9 fps
EL $_{50}$= 638.2	EL $_{100}$= 638.5

OHWM: EL. 636.2
AREA BELOW OHWM: 0.17 ACRES
TEMP. FILL BELOW OHWM: 1130 CY

NOTES:

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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" DIA. C.I.P. REINFORCED CONCRETE PILES AND HAVE A MAXIMUM CAPACITY OF 70 TONS PER PILE

PROPOSED STRUCTURE

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ROADWAY: 45'-6" TOE TO TOE OF PARAPETS

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

SKEW: 13°00'00" RF

CROWN: 0.016 FT./FT.

ALIGNMENT: TANGENT
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30 FT LONG)
LATITUDE: 38°51'00"N
LONGITUDE: 82°52'03"W

TABLE OF VERTICAL CLEARANCES	
LOCATION	"A"
PROPOSED	56.82'±
PREFERRED	15.0'

LEGEND

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

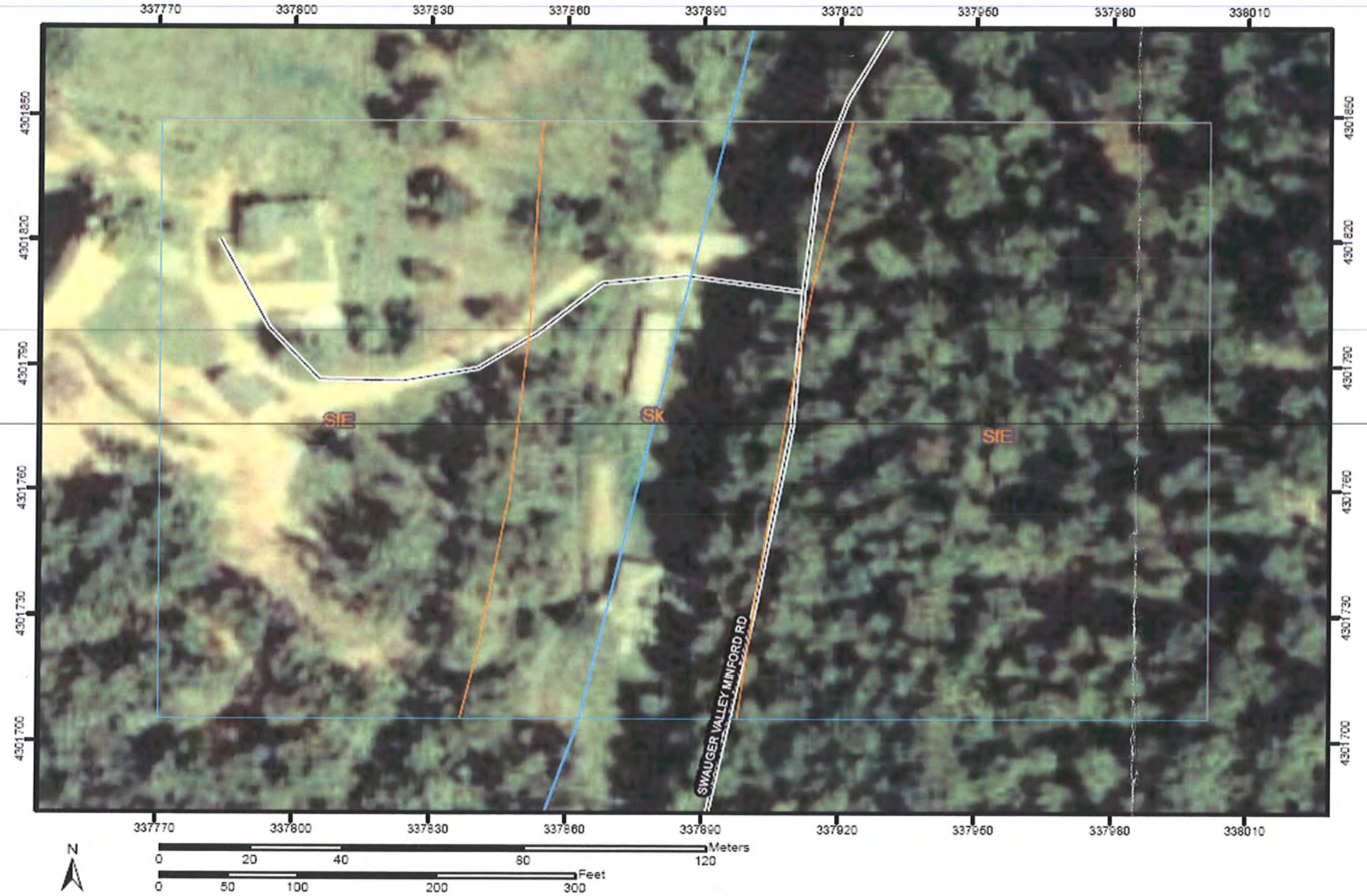
Figure Modified from Structure Type Study SCI-823-0837 L/R
Sheet 2/4 (KZF Design, February 2008)

FIGURE 2 - SITE PLAN
BRIDGE NO. SCI-823-0837 L
S.R. 823 OVER SWAUGER VALLEY-MINFORD ROAD (CR-31)

SCI-823-6.81
PID 19415

2/4

Figure 3: NRCS Soil Map - Scioto County, Ohio



Natural Resources Conservation Service

Web Soil Survey 2.0
National Cooperative Soil Survey

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 9, Feb 28, 2008

Date(s) aerial images were photographed: 3/23/1994

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 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
- 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

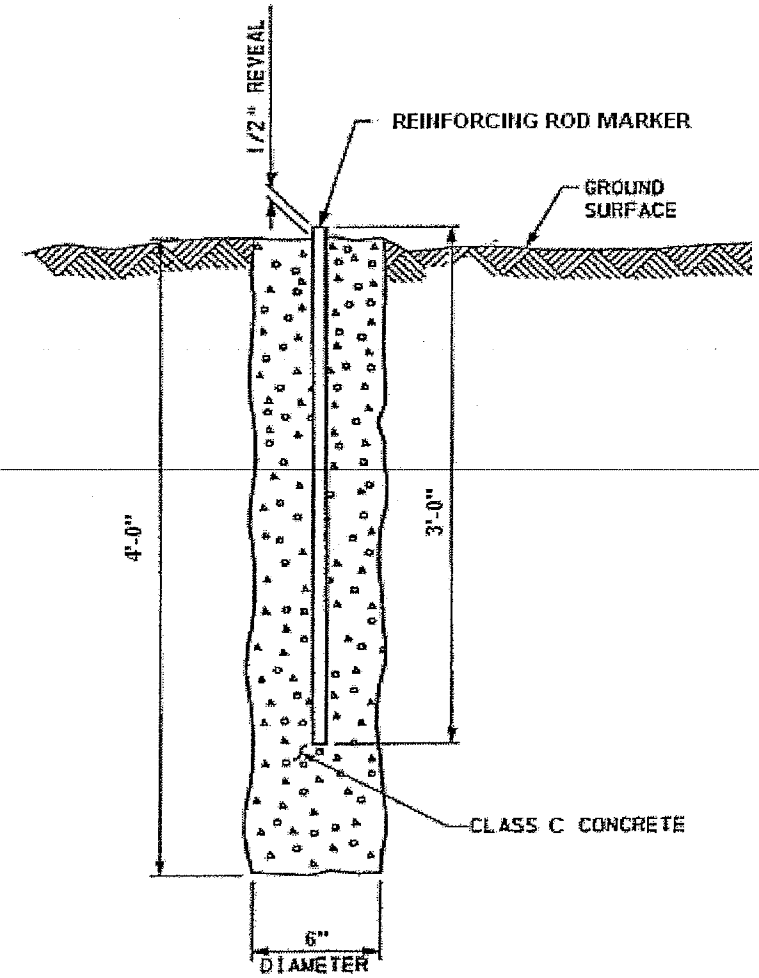
Water Features
- Oceans
- Streams and Canals

Transportation
- Rails

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

Map Unit Legend

Scioto County, Ohio (OH145)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
SfE	Shelocta-Wharton-Latham association, steep	6.0	73.7%
Sk	Skidmore silt loam, occasionally flooded	2.2	26.3%
Totals for Area of Interest (AOI)		8.2	100.0%



NOT TO SCALE

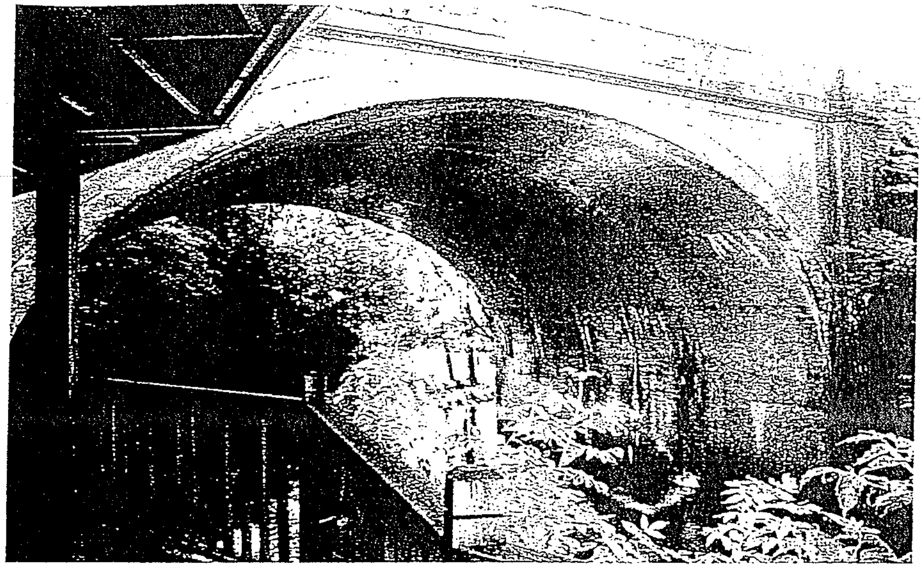
NOTE: EACH MONUMENT TO BE PROTECTED BY A 3 FOOT HIGH, FLOURESCENT PLASTIC, MESHED FENCE PLACED AT A 3 FOOT RADIUS FROM THE MONUMENT.

HDR | ONE COMPANY
Many Solutions℠

Figure 4 - Surface Monument Detail

Appendix B

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

Prepared for:

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

Ohio Department of Transportation
District 9

DLZ Ohio, Inc.
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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 SWAUGER VALLEY – MINFORD ROAD
SCI-823-0.00 PORTSMOUTH BYPASS
SCIOTO COUNTY, OHIO

For:

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

By:

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REPORT
OF
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SR 823 OVER SWAUGER VALLEY – MINFORD ROAD
SCI-823-0.00 PORTSMOUTH BYPASS
SCIOTO COUNTY, OHIO

1.0 INTRODUCTION

This report includes the findings of evaluation 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 Swauger Valley – 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 Swauger Valley – Minford Road (CR-31). 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 the proposed SR 823 over Swauger Valley – Minford Road 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 442+17 and 444+06 to contain the abutments and hold back the roadway embankment for the 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 442+17 (Rear Abutment) and 444+06 (Forward Abutment) will be approximately 63.0 and 58.5 feet, respectively. Those heights are based upon the maximum difference between the proposed grade of SR 823 and the approximate existing grade along the Swauger Valley – Minford Road.

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 five final and four preliminary structural borings. Borings B-5 through B-9 were drilled for the final bridge plan, essentially consisting of proposed SR 823 passing over the Swauger Valley – Minford Road (CR-31). The borings were drilled between June 15 and 16, 2006. Preliminary structural borings (TR-20 through TR-23) were drilled for a previous design configuration. The preliminary borings were drilled between August 3, 2004 and February 24, 2005. A boring plan is presented in Appendix I. Boring logs for borings TR-20 through TR-23, and B-5 through B-9 are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

Final Borings B-5 through B-9 and TR-22 are considered most representative of the conditions near the proposed structures. Other preliminary borings are included for informational purposes.

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). It should be noted that as-per-plan coordinates and elevations were used for borings B-5, B-7, B-9, and TR-21 in lieu of as-drilled survey information.

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 lacustrine soils were encountered. Lacustrine soils in this area are commonly known as "Minford Silts" or the Minford Complex. These deposits were formed during the early to middle Pleistocene age when the northward flowing Teays River system was blocked by the southward advance of the Kansan aged ice sheets. As the glaciers advanced, the course of the Teays River was blocked south of Chillicothe and a large lake was formed from the impoundment of the waterways. As a result of the impoundment, vast quantities of sediments were deposited ranging from 10 to 80 feet in thickness, thinning towards the margins. Bedrock within the structure area is primarily sandstone of the Logan

Formation of Mississippian age. Bedrock of the Pennsylvanian Breathitt Formation can be found at the top of the slopes to the west of the structures roughly above elevation 880. In the area of the structure, the bedrock was covered by a thin soil overburden ranging in thickness between 1.5 and 7.5 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-20, TR-21, B-5, and B-7 were drilled for the west (forward) abutment. Borings TR-22, TR-23, and B-9 were drilled for the east (rear) abutment, while borings B-6 and B-8 were drilled for the piers.

Borings TR-20, TR-22, B-5, B-7, and B-9 encountered surficial material consisting of 1 to 8 inches of topsoil. The topsoil in borings B-5, B-7, and B-9 was underlain by bedrock. Borings TR-20 through TR-23, B-6, and B-8, encountered native cohesive and granular soil deposits below the surficial material or the ground surface. The cohesive deposits consisted mainly of medium stiff to hard silt and clay (A-6a), stiff to hard sandy silt (A-4a), very stiff silt (A-4b), while the granular soil deposits consisted mainly of loose gravel (A-1-a) and very dense sandy silt (A-4a). The native soil deposits extended to an approximate depth ranging between 1.5 and 7.5 feet below the ground surface where bedrock was encountered.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in all borings. The bedrock consisted of medium hard to hard, slightly to highly weathered, slightly to moderately fractured sandstone. The amount of rock recovered in each core run varied between 81 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 17 and 100 percent with an average of 81 percent indicating good rock.

Unconfined compressive strength of tested cores ranged between 7,966 psi and 13,418 psi. The tested cores correspond to samples at depths between 3.5 feet and 18.5 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1, on the following page.

Table 1-Unconfined Compressive Strength Results

Boring	Depth (ft)	Unconfined Compressive Strength (psi)
B-5	3.5-4.0	8,382
B-6	18.0-18.5	13,418
B-7	6.5-7.0	7,966
B-8	17.0-17.5	10,997
B-9	7.2-7.7	8,153

4.2.3 Groundwater Conditions

Seepage was not encountered in any boring during drilling. 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-6 and B-8 upon the completion of coring between approximate depths of 0.5 and 12.5 feet. Boring TR-21 was drilled in a streambed and hence was completely submerged in water.

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 bearing on rock have been evaluated. On the other hand, the site is well suited for the use of MSE wall 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 pile 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 of the Geotechnical Engineer should field verify that the drilled shafts are founded on competent bearing materials and the installation procedures meet specifications.

If adequate capacity cannot be developed with reasonable shaft diameter, 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. Although the borings did not encounter significant seepage, 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 cavein. 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 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 bedrock or granular fill placed on bedrock. As such, the anticipated settlements of spread footings bearing on the fill are anticipated to be 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 of the Geotechnical Engineer should field verify that the drilled shafts are founded on competent bearing materials and the installation procedures meet specifications.

If adequate capacity cannot be developed with reasonable shaft diameter, 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 below 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 Recommendation

Structural Element	Structure / Boring	Existing Ground Surface Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	Left / B-9	647.5 $^{+}$	Pipe Piles	636.5 *	Pile Capacity^{++}
			Drilled Shafts	636.5 *	80 ksf^{+++}
			Spread Footings	MSE Fill**	4 ksf
	Right / TR-22	636.2	Pipe Piles	625.2 *	Pile Capacity^{++}
			Drilled Shafts	625.2 *	80 ksf^{+++}
			Spread Footings	MSE Fill**	4 ksf
Pier	Left / B-8	638.4	Spread Footings	627.9	80 ksf
			Drilled Shafts	622.9 *	80 ksf^{+++}
	Right / B-6	635.9	Spread Footings	627.4	80 ksf
			Drilled Shafts	622.4 *	80 ksf^{+++}
Forward Abutment	Left / B-7	658.0 $^{+}$	Pipe Piles	647.0 *	Pile Capacity^{++}
			Drilled Shafts	647.0 *	80 ksf^{+++}
			Spread Footings	MSE Fill**	4 ksf
	Right / B-5	644.0 $^{+}$	Pipe Piles	635.5 *	Pile Capacity^{++}
			Drilled Shafts	635.5 *	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.

$^{+}$ Ground surface elevation was estimated from the established topographic mapping in lieu of as-drilled survey information.

$^{++}$ 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)	Compacted Granular Fill	120	0	34	0	34
Foundation Soil (Forward Abutment)	Compacted Granular Fill	120	0	34	0	34

5.2.2 MSE Wall Evaluations and Recommendations

The MSE wall at the rear abutment (station 442+17) is understood to have a maximum height of approximately 63 feet. The overburden in this area is very thin. It is recommended that the leveling pad be extended to bedrock or soil be excavated to bedrock and replaced with compacted granular fill to the leveling pad elevation. If founded on bedrock, no embedment into the rock is required. The compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. 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. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to ODOT BDM Item 204. In addition, because the wall will be founded on or near bedrock, stability should be adequate. For stability, calculations have shown that a minimum reinforcement length of (H+D) times 0.7, or 44.1 feet, must be used for 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 significantly. If soft soils are encountered while excavating for the MSE wall-leveling pad, these soils should be removed and replaced with compacted granular fill. 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.

The MSE wall at the forward abutment (Station 444+14) is understood to have a maximum height of approximately 58.5 feet. The overburden in this area is relatively thin (1.0 to 4.5 feet). It is recommended that the leveling pad be extended to bedrock or soil be excavated to bedrock and replaced with compacted granular fill to the leveling pad elevation. If founded on bedrock, no embedment into the rock is required. The compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. 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. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to BDM Item 204. In addition, because the wall will be founded on or near bedrock, stability should be adequate. For stability, calculations have shown that a minimum reinforcement length of (H+D) times 0.7, or 41.0 feet must be used for the proposed MSE wall at this location.

It should be noted that the foundation leveling pad of the MSE wall at the forward abutment is in close proximity to a creek, which is running essentially parallel to Swauger Valley – Minford Road. The approximate elevation of bedrock under the MSE wall at the forward abutment ranges from 642.5 to 654.5 feet, which is near the bottom of the creek at elevation 631. If scour and erosion near the toe of the MSE wall are a concern, then slope protection should be provided with riprap.

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 compacted granular fill foundations. Drawings illustrating the typical soil and rock benches are presented in Appendix IV.

A summary of soil properties, summary of the results of calculations, and MSE retaining wall parameters are presented in Tables 4 and 5 on the following pages.

Table 4-MSE Retaining Wall Parameters and Analyses Results
(Rear Abutment)
Borings TR-23 & B-9

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} = 15,893 psf

Allowable Bearing Capacity – Drained Condition
q_{all} = 15,893 psf

Global Stability
Factor of Safety – Undrained Condition > 1.5 (Founded on Bedrock)
Factor of Safety – Drained Condition > 1.5 (Founded on Bedrock)
Factor of Safety – Seismic Condition > 1.3 (Founded on Bedrock)

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

Approximate Maximum Height of MSE Wall = 63.0 feet
Approximate Embedment Depth = 0.0 feet (Bedrock)
Minimum Length of Reinforcement for External Stability = 44.1 feet

Table 5-MSE Retaining Wall Parameters and Analyses Results
(Forward Abutment)
Borings TR-20 & B-5

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 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} = 14,734 psf

Allowable Bearing Capacity – Drained Condition
q_{all} = 14,734 psf

Global Stability
Factor of Safety – Undrained Condition > 1.5 (Founded on Bedrock)
Factor of Safety – Drained Condition > 1.5 (Founded on Bedrock)
Factor of Safety – Seismic Condition > 1.3 (Founded on Bedrock)

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

Approximate Maximum Height of MSE Wall = 58.5 feet
Approximate Embedment Depth = 0.0 feet (Bedrock)
Minimum Length of Reinforcement for External Stability = 41.0 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,

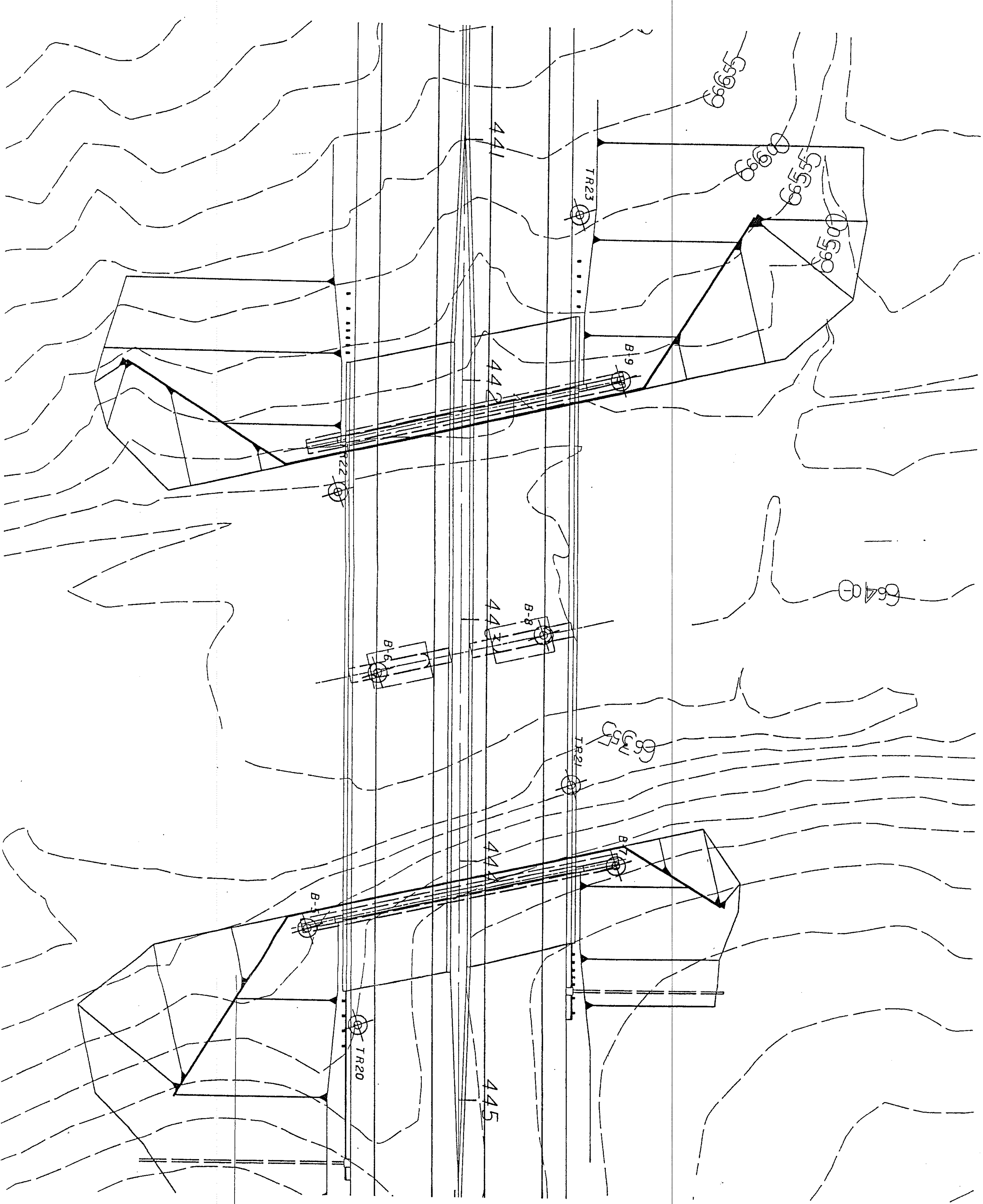
DLZ OHIO, INC.

Steven Riedy
Geotechnical Engineer

Wael Alkasawneh, P.E.
Geotechnical Engineer

sjr

M:\proj\0121\3070.03\Stability Analyses\Documents\MSE Wall letters\05 Swauger Valley - Minford Road\Final\Joint Structure Report\Swauger Valley Road Structure Report 09-26-06 - WMA_rev.doc



SCI-823

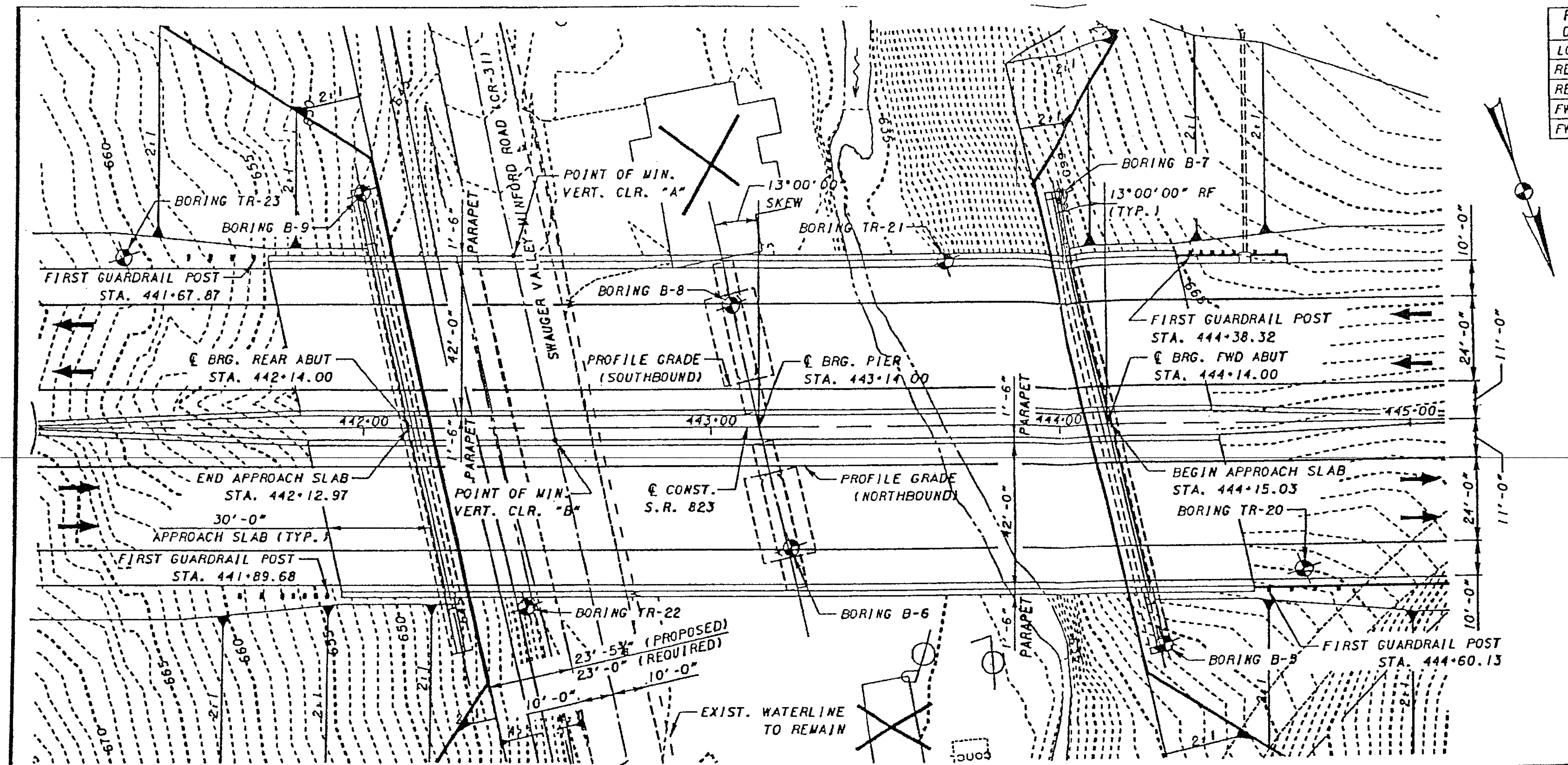
BORING PLAN
SR 823 OVER SWAUGER VALLEY-MINFORD ROAD

CALCULATED

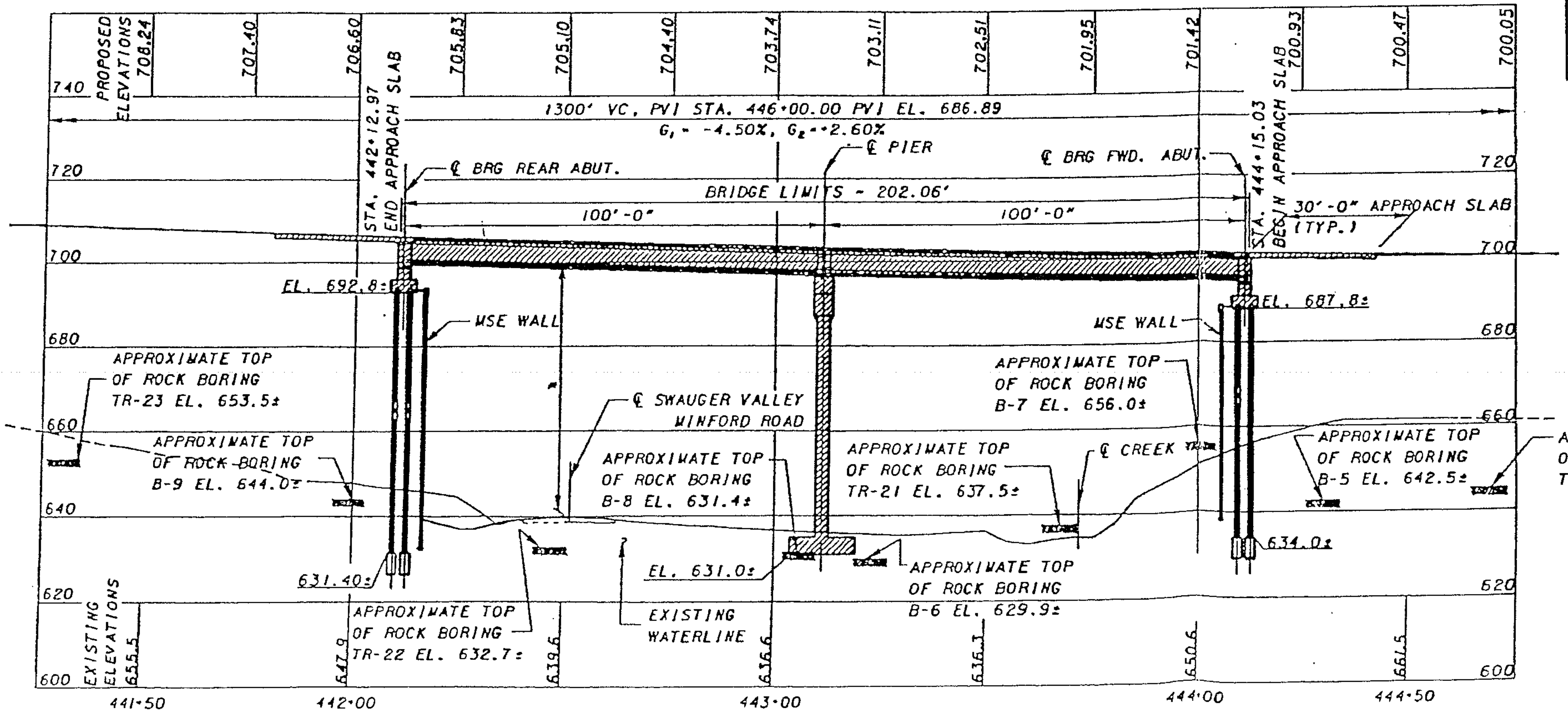
CHECKED

0 10 20 40
HORIZONTAL
SCALE IN FEET

N



PLAN



* SEE TABLE OF VERTICAL CLEARANCES

TABLE OF VERTICAL CLEARANCES		
LOCATION	"A"	"B"
PROPOSED	56.6±	58.7'±
PREFERRED	15.0'	15.0'

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	OFFSET
REAR ABUT.	441+89.68	RT.
REAR ABUT.	441+67.87	LT.
FWD. ABUT.	444+60.13	RT.
FWD. ABUT.	444+38.32	LT.

BORING LOCATIONS		
BORING No.	STATION	OFFSET
TR-20	444+69.72	42.10' RT.
TR-21	443+66.99	46.44' LT.
TR-22	442+46.91	51.49' RT.
TR-23	441+30.33	48.06' LT.
B-5	444+29.99	63.31' RT.
B-6	443+22.99	34.59' RT.
B-7	444+00.99	65.40' LT.
B-8	443+05.99	34.57' LT.
B-9	441+98.99	66.15' LT.

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

TRAFFIC DATA

(SR 823)
CURRENT YEAR ADT (2010) - 21,200
DESIGN YEAR ADT (2030) - 31,200
CURRENT YEAR ADTT (2010) - 2,968
DESIGN YEAR ADTT (2030) - 4,368

PROPOSED STRUCTURE

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

SPANS: 100'-0", 100'-0" c/c BEARINGS

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

LOADING: HS-25 AND ALTERNATE MILITARY LOADING; FWS - 60 PSF

SKEW: 13°00'00" RF

CROWN: 0.016 FT./FT.

ALIGNMENT: TANGENT
WEARING SURFACE: 1" MONOLITHIC SURFACE
APPROACH SLABS: AS-1-81 (30 FT LONG)
LATITUDE: 38°51'0"N
LONGITUDE: 82°52'3"W

HYDRAULIC DATA

DRAINAGE AREA - 13.424 sq.mi. - 8591 acres

Q_{50} - XXX cfs	Q_{100} - XXX cfs
V_{50} - XXX fps	V_{100} - XXX fps
EL 50 - XXX	EL 100 - XXX

OHWM: EL. XXX
AREA BELOW OHWM: XXX ACRES
TEMP. FILL BELOW OHWM: XXX 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 HP 14" DIA. C.I.P. REINFORCED CONCRETE PILES AND HAVE A MAXIMUM CAPACITY OF 70 TONS PER PILE

DESIGN AGENCY
Systems

DESIGNED	DRAWN	REVIEWED	DATE
MSL	MTH	JRC	07/31/06
CHECKED	REVISED	STRUCTURE FILE NUMBER	
PJP			

SCIOTO COUNTY
STA. 442+12.97
STA. 444+15.03

SITE PLAN
BRIDGE NO. SCI-823-XXXX
S.R. 823 OVER SWAUGER VALLEY-MINFORD ROAD (CR-31)

SCI-823-0.00
PID 19415

2

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.

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TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

OF: Boring B-5

Location: Sta. 444+30.2, 63.3 ft. RT of SR 823 CL *As Per Plan Date Drilled: 06/15/06

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

h	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / • Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
	644.0													
	643.9						Topsoil - 1"							
	642.5	18 50/0	6	1			Medium dense gray SANDY SILT (A-4a), damp. (Decomposed Sandstone)							
5		Core 120"	Rec 114"	RQD 83%	R-1		Medium hard to hard gray SANDSTONE, fine to very fine grained, moderately to highly weathered, argillaceous, micaceous, massive bedding, moderately fractured, contains few argillaceous laminations. @ 1.5'-7.2', 7.9'-8.3', 10.9', rust staining. @ 3.1'-3.3', high angle fracture. @ 3.5', qu = 8,382 psi.							
0														
.5	632.5						Bottom of Boring - 11.5'							
15														
20														
25														
30														

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: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

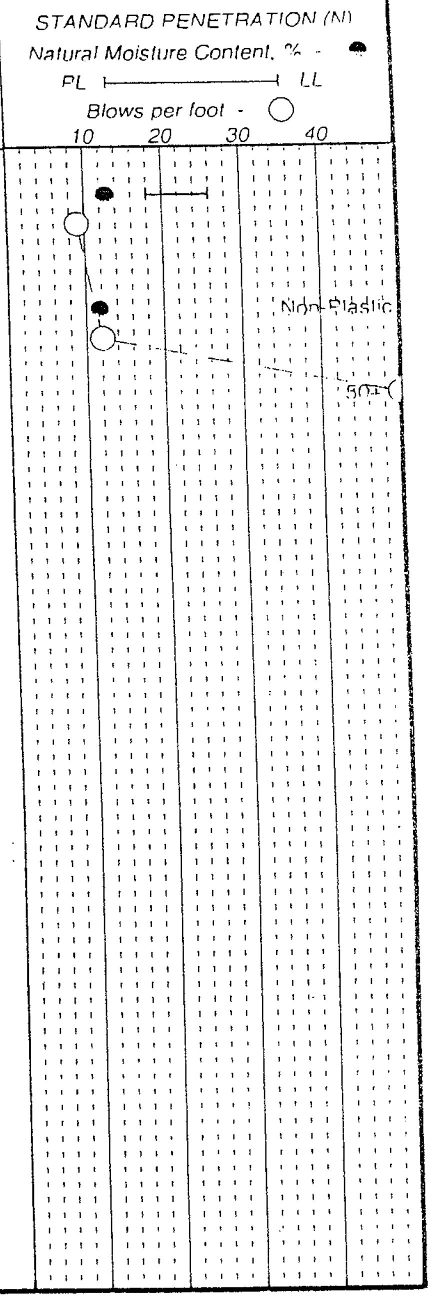
OF: Boring B-6

Location: Sta. 443+23.0, 34.6 ft. RT of SR 823 CL

Date Drilled: 06/15/06 to 06/16/06

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

th	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / • Point-Load Strength (psi)	DESCRIPTION	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	635.9	7 5 4	17	1		4.5+	Hard brown SANDY SILT (A-4a), little clay, trace to little gravel; damp.	16	23	--	15	28	18
		6 9 3		2		4.5+		10	25	--	14	40	11
5													
0	629.9	50/1	1	3			Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, micaceous, massively bedded, slightly fractured, contains moderate argillaceous laminations. @ 6.5'-7.4', rust staining. @ 6.8', 6.9', 7.1', 7.9', rust stained low angle fractures.						
10		Core 108"	Rec 105"	RQD 81%	R-1								
15													
							@ 18.0', qu = 12,418 psi.						
20		Core 120"	Rec 120"	RQD 89%	R-2								
25		Core 12"	Rec 10"	RQD 83%	R-3								
26.5	609.4						Bottom of Boring - 26.5'						
30													



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TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

OF: Boring B-7

Location: Sta. 444+00.8, 65.4 ft. LT of SR 823 CL *As Per Plan

Date Drilled: 06/15/06

WATER OBSERVATIONS: Water seepage at: none
Water level at completion: none

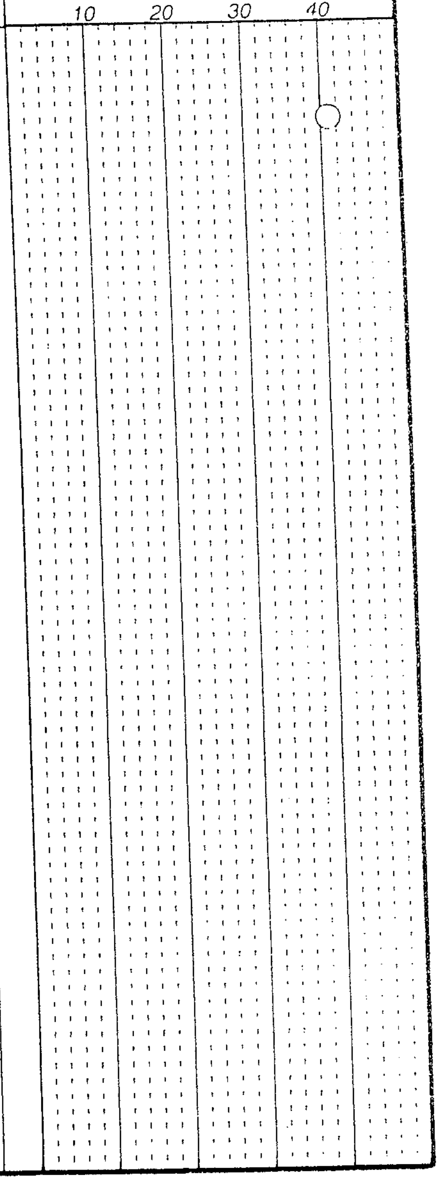
h	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	658.0 657.8						Topsoil - 2"						
		8 18 23	13	1			Medium dense gray SANDY SILT (A-4a), damp. (Decomposed Sandstone)						
	655.5	Core 120"	Rec 97"	RQD 76%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, micaceous, fossiliferous (trace fossils), massive bedding, highly fractured. @ 2.5'-9.3', rust staining. @ 2.5'-5.0', broken zone. @ 3.5', lost water circulation. @ 6.5', qu = 7,966 psi. @ 8.7'-8.9', high angle fracture.						
5													
0													
2.5	645.5						Bottom of Boring - 12.5'						
15													
20													
25													
30													

STANDARD PENETRATION (N)

Natural Moisture Content, % - ●

PL ├──┤ LL

Blows per foot - ○



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TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

OF: Boring B-8

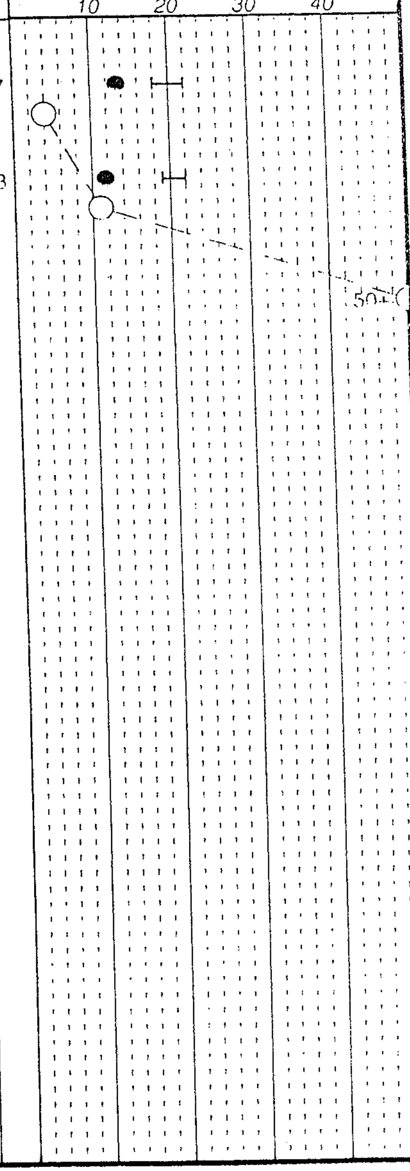
Location: Sta. 443+05.8, 34.6 ft. LT of SR 823 CL

Date Drilled: 06/16/06

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

h	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	638.4												
		3 2 2	11	1		4.0	Very stiff to hard brown SILT (A-4b), little clay, trace fine to coarse sand, trace gravel; damp.	4	12	--	15	52	17
	635.4												
		7 6 5	17	2		4.5+	Hard brown SANDY SILT (A-4a), little clay, little gravel; damp.	12	23	--	17	35	13
		9 15 50/4		3		4.5+							
	630.9												
0		Core 108"	Rec 87"	RQD 68%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, micaceous, massive bedding, highly fractured, contains few laminations. @7.5'-8.7', rust staining.						
15							@ 17.0', qu = 10,997 psi.						
20		Core 120"	Rec 120"	RQD 90%	R-2								
25		Core 12"	Rec 12"	RQD 100%	R-3								
27.5	610.9						Bottom of Boring - 27.5'						
30													

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



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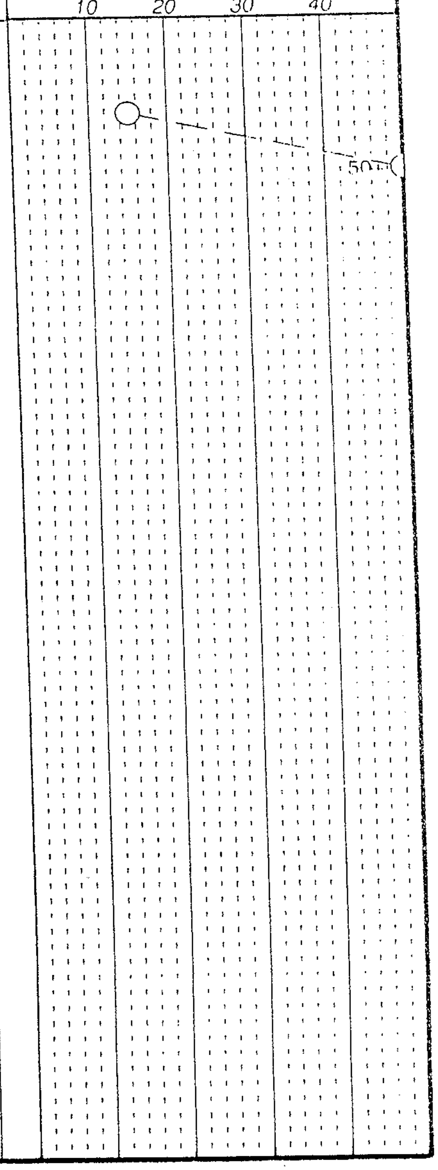
: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

OF: Boring B-9 | Location: Sta. 441+98.6, 66.2 ft. LT of SR 823 CL *As Per Plan | Date Drilled: 06/15/06

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

th	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
	647.5 647.4						Topsoil - 1"						
		7 7 8	4	1			Medium dense gray SANDY SILT (A-4a), damp. (Decomposed Sandstone) @ 4.0', auger refusal.						
0	643.5	50/0	0	2									
		Core 120"	Rec 117"	RQD 66%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, micaceous, massive bedding, highly fractured. @ 4.0'-11.4', rust staining. @ 7.2', qu = 8,153 psi. @ 8.7'-8.8', 8.9'-9.0'. Decomposed argillaceous zones.						
4.0	633.5						Bottom of Boring - 14.0'						

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



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TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

OF: Boring TR-20

Location: Sta. 444+69.7, 42.1 ft. RT of SR 823 CL

Date Drilled: 8/4/04

WATER OBSERVATIONS: Water seepage at: none

Water level at completion: 6.3' (Includes drilling water)

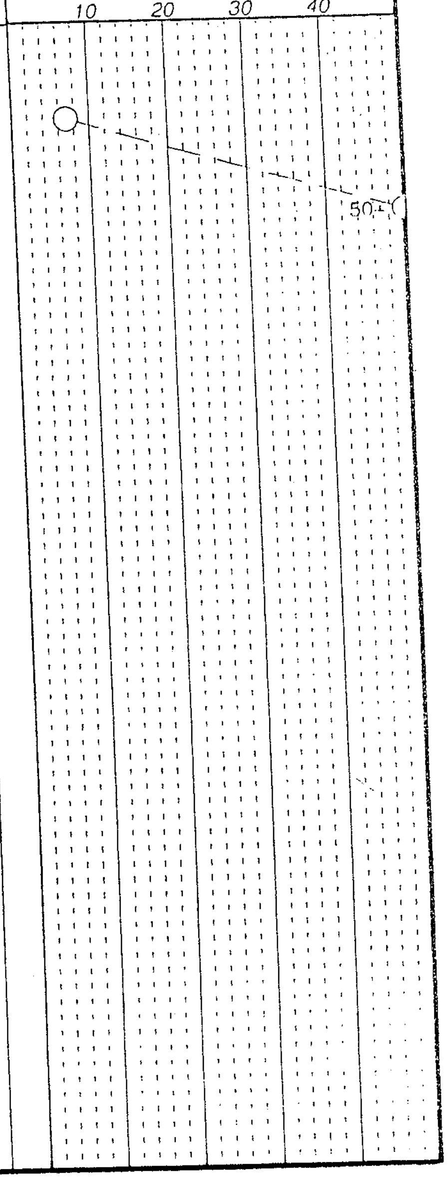
Depth	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / • Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	650.0 649.8						Topsoil - 2"						
		3 3 4	18	1		0.5	Medium stiff brown SILT AND CLAY (A-6a), little fine to coarse sand, little gravel; contains sandstone fragments; moist.						
		1 3 50/3	15	2									
	645.0	Core 96"	Rec 91"	RQD 88%	R-1		Hard gray SANDSTONE; very fine to fine grained, slightly weathered, micaceous, massively bedded, slightly fractured. @ 5.0'-5.3', broken. @ 9.3'-9.5', broken zone, possible core loss.						
15		Core 84"	Rec 84"	RQD 86%	R-2		@ 13.9' to 14.5', high angle fracture with reddish brown discoloration.						
20.0	630.0						Bottom of Boring - 20.0'						
25													
30													

STANDARD PENETRATION (N)

Natural Moisture Content, % - ●

PL ├──┤ LL

Blows per foot - ○



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

OF: Boring TR-21 *Location:* Sta. 443+67.0, 46.5 ft. LT of SR 823 CL *As Per Plan *Date Drilled:* 8/3/04

WATER OBSERVATIONS: Water seepage at: 0.0'

Water level at completion: 0.0' (includes drilling water)

ith)	*Elev. (ft)*	*Blows per 6"*	*Recovery (in)*	*Sample No.* *Drive*	*Sample No.* *Press / Core*	*Hand Penetro-meter (tsf) / * Point-Load Strength (psi)*	*DESCRIPTION*	*GRADATION* *% Aggregate*	*% C. Sand*	*% M. Sand*	*% F. Sand*	*% Silt*	*% Clay*	*STANDARD PENETRATION (N)* *Natural Moisture Content, % -* ● *PL* ├──┤ *LL* *Blows per foot -* ○ 10 20 30 40
	639.0						Gray GRAVEL (A-1-a); wet. (Auger sample - boring drilled in stream bed)							
5	637.5	Core 114"	Rec 114"	RQD 70%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, micaceous, argillaceous, massively bedded, slightly fractured. @ 1.5'-3.9', brown, highly weathered, highly fractured to broken. @ 3.3'-3.4', clay filled fracture.							
5														
10		Core 108"	Rec 108"	RQD 93%	R-2									
15														
20.0	619.0						Bottom of Boring - 20.0'							
25														
30														

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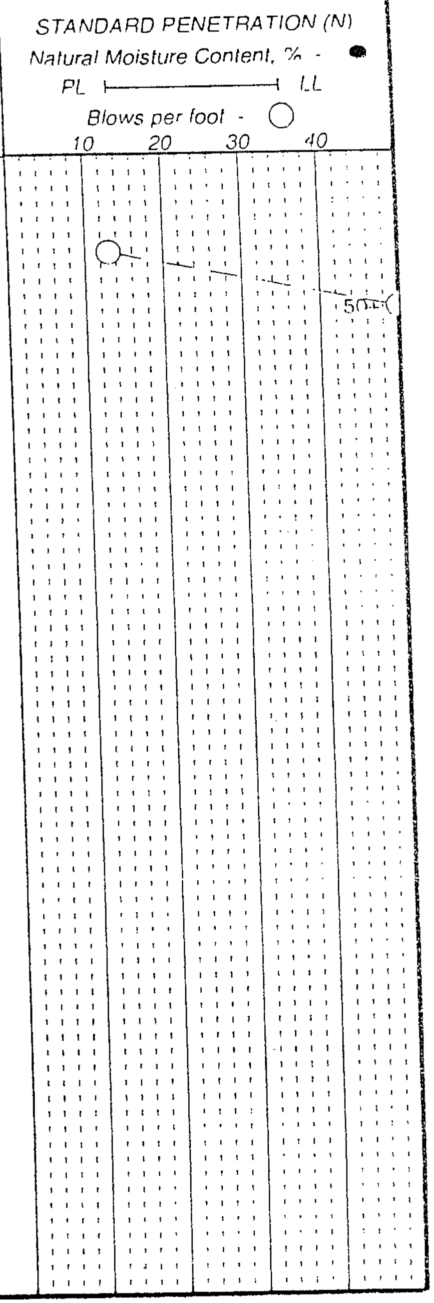
: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

OF: Boring TR-22 | *Location:* Sta. 442+46.9, 51.5 ft. RT of SR 823 CL | *Date Drilled:* 2/24/2005

WATER OBSERVATIONS: Water seepage at: none
Water level at completion: 4.5' (inside hollowstem augers, includes drilling water)

th)	*Elev. (ft)*	*Blows per 6"*	*Recovery (in)*	*Sample No.* *Drive*	*Sample No.* *Press / Core*	*Hand Penetrometer (tsf) / * Point-Load Strength (psi)*	*DESCRIPTION*	*GRADATION* *% Aggregate*	*% C. Sand*	*% M. Sand*	*% F. Sand*	*% Silt*	*% Clay*
0	636.2						Topsoil - 8"						
7	635.5	3 6 7	18	1		1.25	Stiff brown SANDY SILT (A-4a), trace gravel; organic; moist.						
.8	633.4	26 50/4	10	2A 2B			Very dense brown SANDY SILT (A-4a), trace gravel; organic; moist.						
.5	632.7						Severely weathered brown SANDSTONE.						
.0 5 10	632.2	Core 120"	Rec 120"	RQD 84%	R-1		Soft brown SANDSTONE; fine grained, moderately weathered, slightly micaceous, moderately fractured. @ 5.2'-5.7',7.1'-7.3',8.7'-8.9' very soft, highly weathered. @ 6.1', gray, medium hard. @ 10.9'-11.0', iron stained horizontal fractures. @ 12.0'-12.8', siltstone.						
14.0 15 20	622.2	Core 120"	Rec 120"	RQD 96%	R-2		Hard gray SANDSTONE; fine grained, slightly weathered, slightly micaceous, slightly fractured. @ 14.7'-15.3', very soft gray and brown SILTSTONE, highly weathered. @ 19.3'-19.4', irregular vertical fracture. @ 19.6', 1/2" clay filled fracture. @ 23.2'-23.5', siltstone.						
-24.0 25 30	612.2						Bottom of Boring - 24.0'						



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TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

OF: Boring TR-23

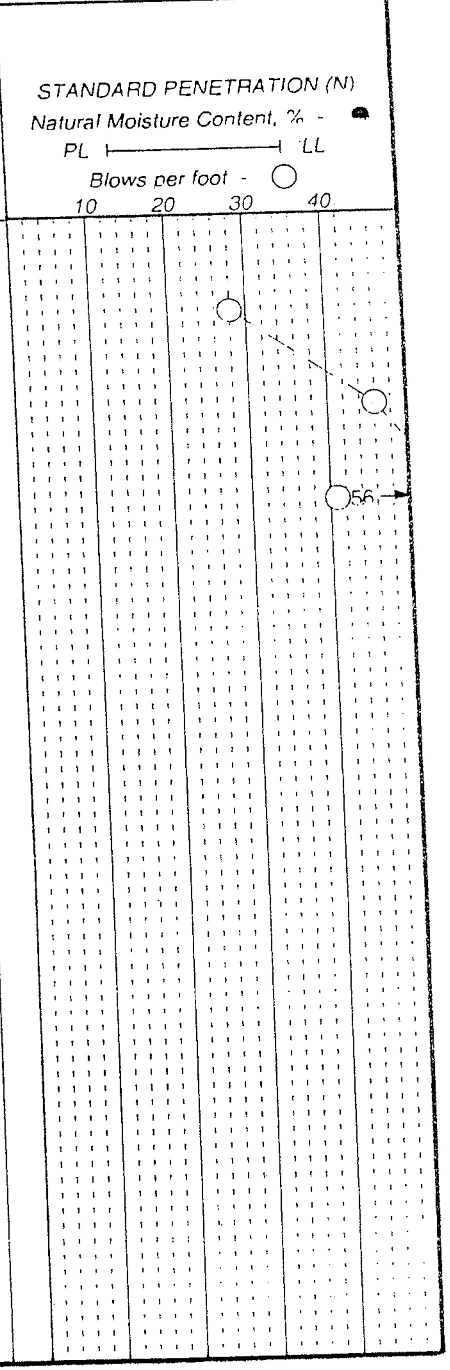
Location: Sta. 441+30.3, 48.1 ft. LT of SR 823 CL

Date Drilled: 8/9/04

WATER OBSERVATIONS: Water seepage at: none

Water level at completion: 2.0' (includes drilling water)

	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
	661.0 660.7						Topsoil - 4"						
		6 13 15	17	1		4.5+	Hard brown SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; contains sandstone fragments; damp.						
		11 26 20	17	2		4.5							
		11 16 40	16	3		4.5+							
5	653.5	Core 30"	Rec 26"	RQD 17%	R-1		Soft brown SANDSTONE; fine grained, decomposed.						
0	651.0	Core 120"	Rec 120"	RQD 84%	R-2		Hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, slightly fractured. @ 12.3',13.5', weathered fractures. @ 12.9' to 13.6', brown.						
15													
20.0	641.0						Bottom of Boring - 20.0'						
25													
30													



APPENDIX III
Laboratory Test Results

DLZ

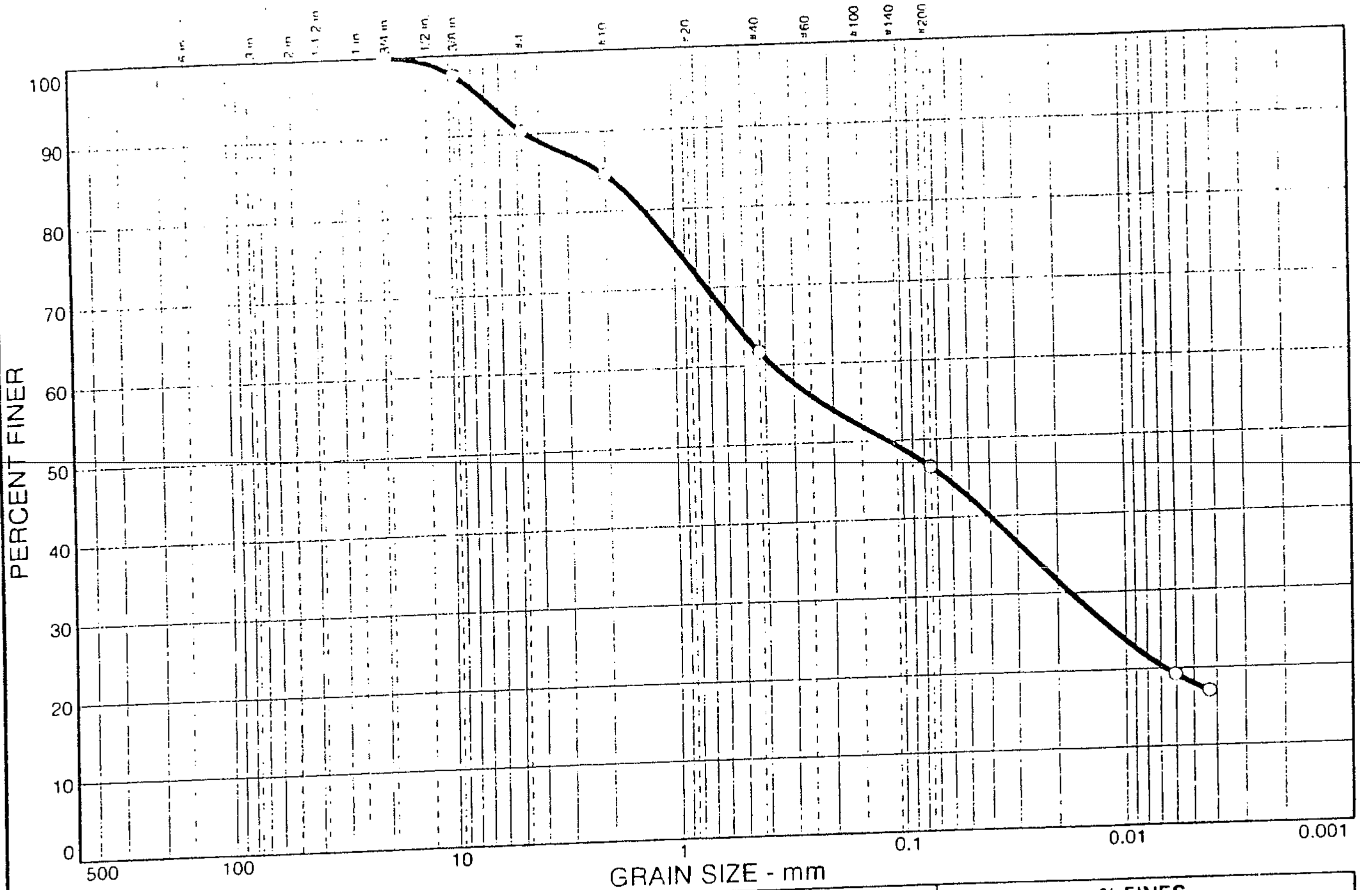
SUBJECT SCI-823 Portsmouth Bypass
Structures and MSE Walls
Unconfined Strength - Rock Results

JOB NUMBER 0121-3070.03
SHEET NO.
COMP. BY SJR
CHECKED BY

Unconfined Compression Test Results - Rock

Boring	Depth (ft.)	Avg Dia. (in.)	Avg L (in.)	L/D	Weight (g)	X-Section Area	Volume (ft^3)	Unit Weight (pcf)	Load (lb-f)	Calculated Stress (psi)	Rock Type
B-5	3.5-4.0	1.972	4.854	2.461	544.65	3.06	0.008575	140.0	25,550	**8,382**	Sandstone
B-6	18.0-18.5	1.986	4.753	2.393	606.38	3.10	0.008517	157.0	40,900	**13,418**	Sandstone
B-7	6.5-7.0	1.968	4.903	2.491	549.42	3.05	0.008627	140.4	24,280	**7,966**	Sandstone
B-8	17.0-17.5	1.986	4.811	2.422	603.89	3.10	0.008621	154.4	33,520	**10,997**	Sandstone
B-9	7.2-7.7	1.969	4.763	2.419	518.96	3.05	0.008389	136.4	24,850	**8,153**	Sandstone

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	9.7	5.8	22.8	15.2	28.5	18.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	97.5		
#4	90.3		
#10	84.5		
#40	61.7		
#200	46.5		

* (no specification provided)

Soil Description

Clayey sand

Atterberg Limits

PL= 18 LL= 26 PI= 8

Coefficients

D_{85}= 2.12 D_{60}= 0.372 D_{50}= 0.116

D_{30}= 0.0176 D_{15}= D_{10}=

C_u= C_c=

Classification

USCS= SC AASHTO= A-4(1)

Remarks

Moisture Content= 12.7%

Sample No.: 1 Source of Sample: B-6 Date: 7/14/06

Location: Elev./Depth: 0.5

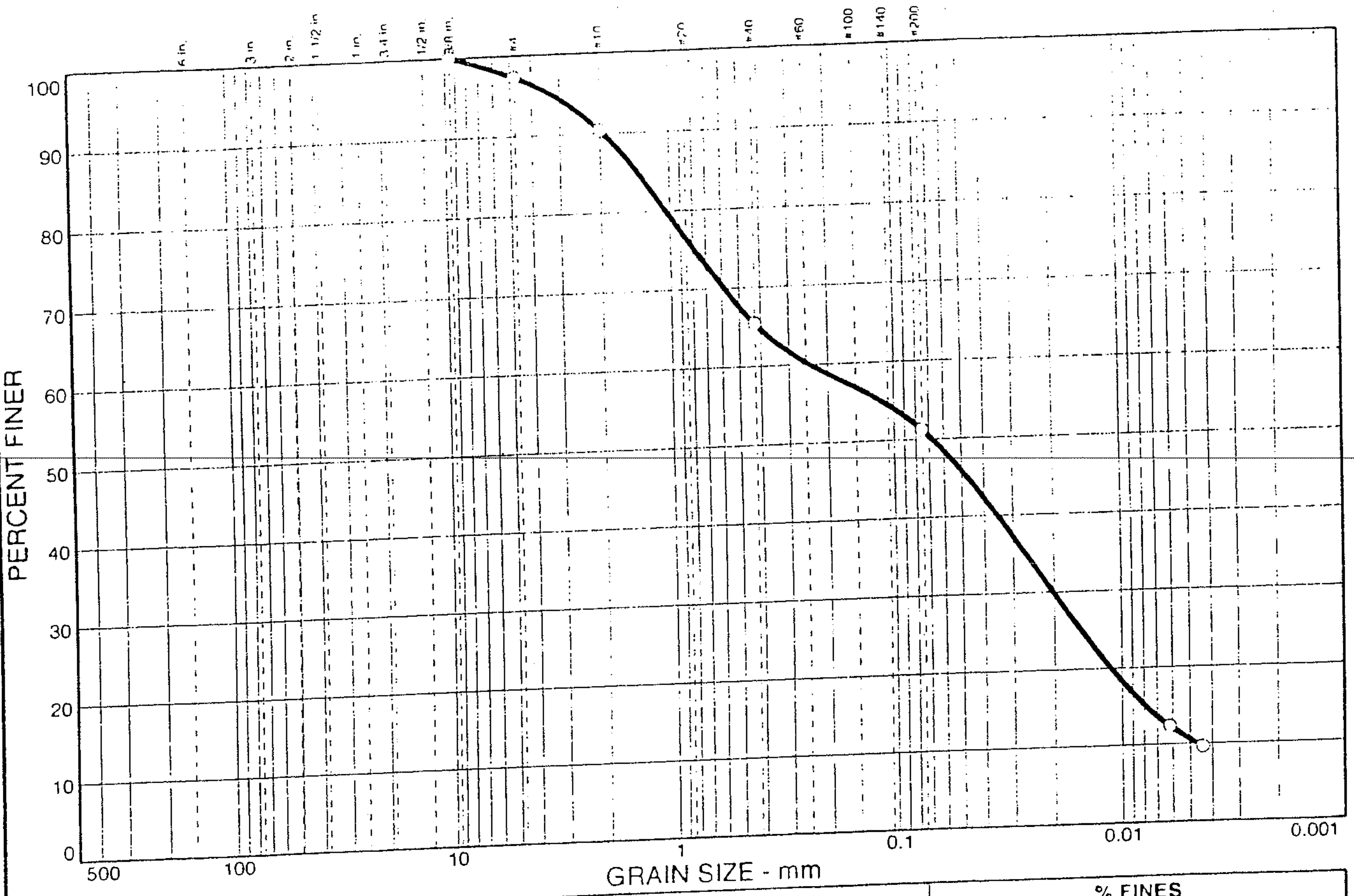
DLZ

Client: TranSystems, Inc.

Project: SCI-823-0.00

Figure

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	2.7	7.0	24.9	14.2	40.6	10.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	97.3		
#10	90.3		
#40	65.4		
#200	51.2		

* (no specification provided)

Soil Description

Sandy silt

Atterberg Limits

PL= NP
LL= NP
PI= NP

Coefficients

D_{85}= 1.39
D_{60}= 0.241
D_{50}= 0.0677
D_{30}= 0.0201
D_{15}= 0.0079
D_{10}= 0.0046
C_u= 51.92
C_c= 0.36

Classification

USCS= ML
AASHTO= A-4(0)

Remarks

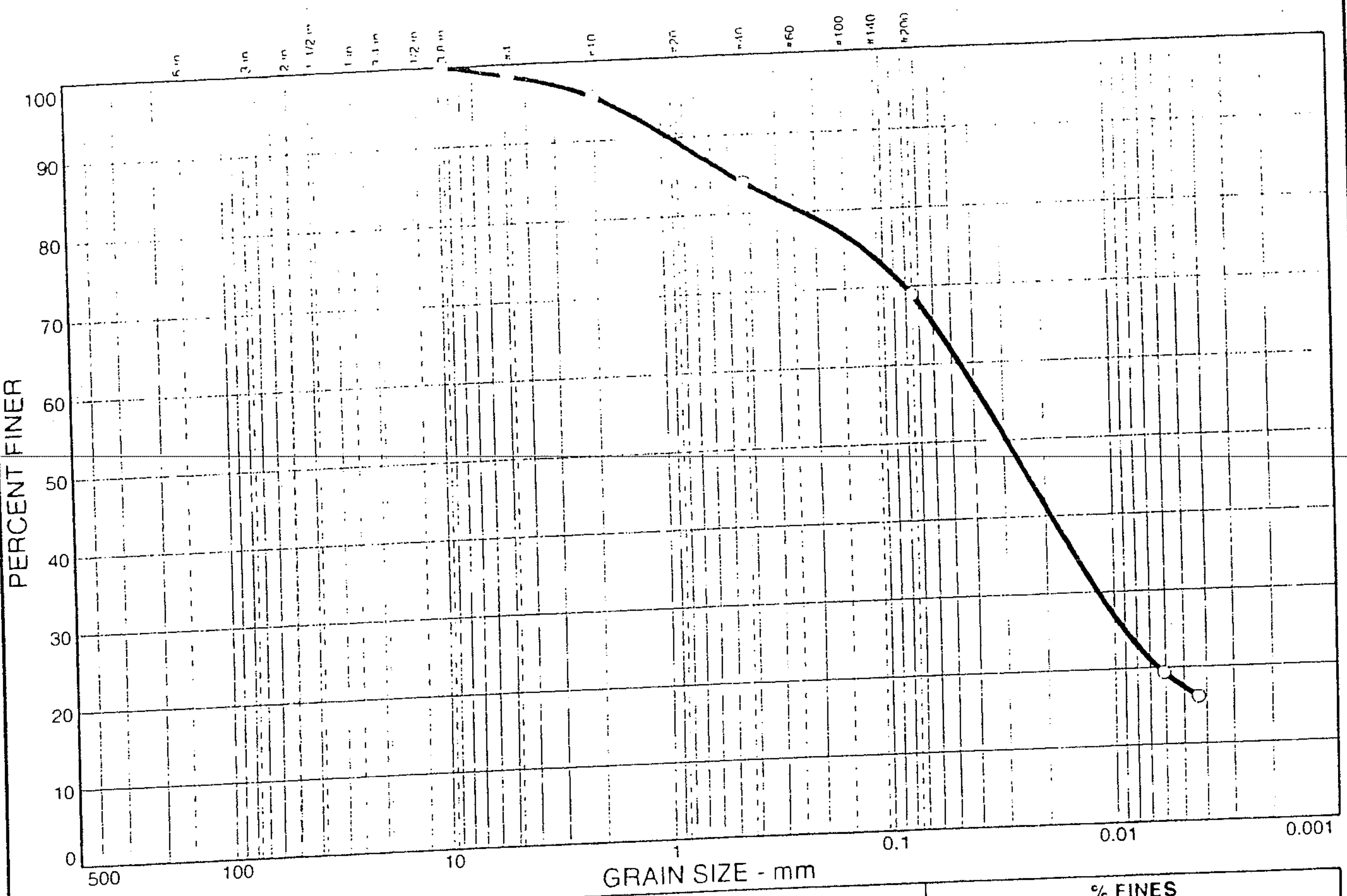
Moisture Content= 11.7%

Date: 7/14/06
Sample No.: 2
Source of Sample: B-6
Elev./Depth: 3.5
Location:

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.5	3.0	11.6	15.0	51.5	17.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	98.5		
#10	95.5		
#40	83.9		
#200	68.9		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 18 LL= 22 PI= 4

Coefficients

D_{85}= 0.494 D_{60}= 0.0460 D_{50}= 0.0292

D_{30}= 0.0118 D_{15}= D_{10}=

C_u= C_c=

Classification

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

Remarks

Moisture Content= 13.4%

Sample No.: 1

Location:

Source of Sample: B-8

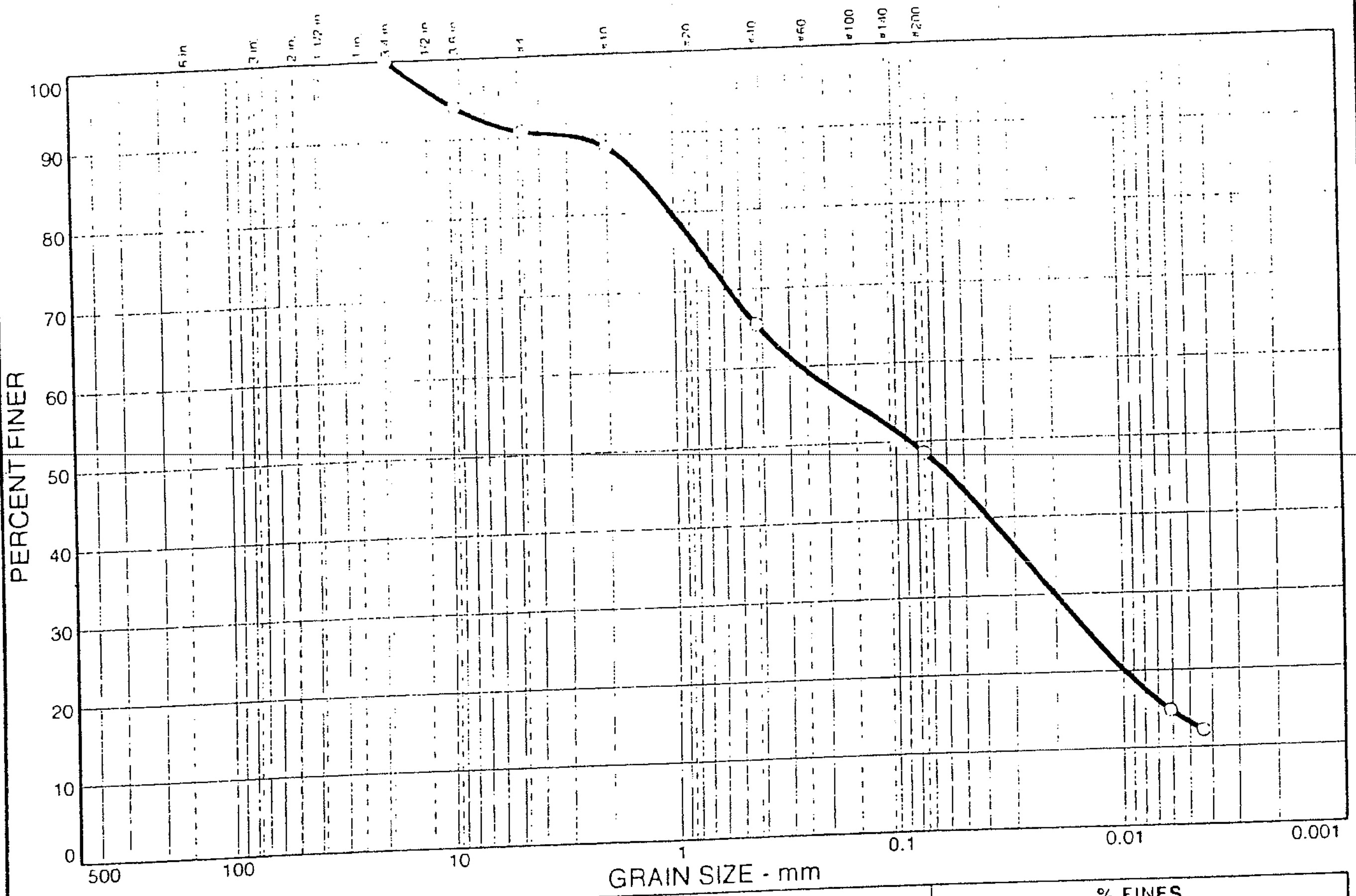
Date: 7/14/06

Elev./Depth: 1.0

Client: TranSystems, Inc.

Project: SCI-823-0.00

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	9.3	2.4	22.8	17.0	35.4	13.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	94.0		
#4	90.7		
#10	88.3		
#40	65.5		
#200	48.5		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= 19
LL= 22
PI= 3

Coefficients

D_{85}= 1.45
D_{60}= 0.267
D_{50}= 0.0866
D_{30}= 0.0200
D_{15}= 0.0063
D_{10}=
C_u=
C_c=

Classification

USCS= SM
AASHTO= A-4(0)

Remarks

Moisture Content= 11.7%

Date: 7/14/06
Sample No.: 2
Source of Sample: B-8
Elev./Depth: 3.5
Location:

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

APPENDIX IV

MSE Wall Stability Analysis Results
MSE Wall Bearing Capacity and Stability Calculations
Drilled Shaft – End Bearing and Side Friction Calculations

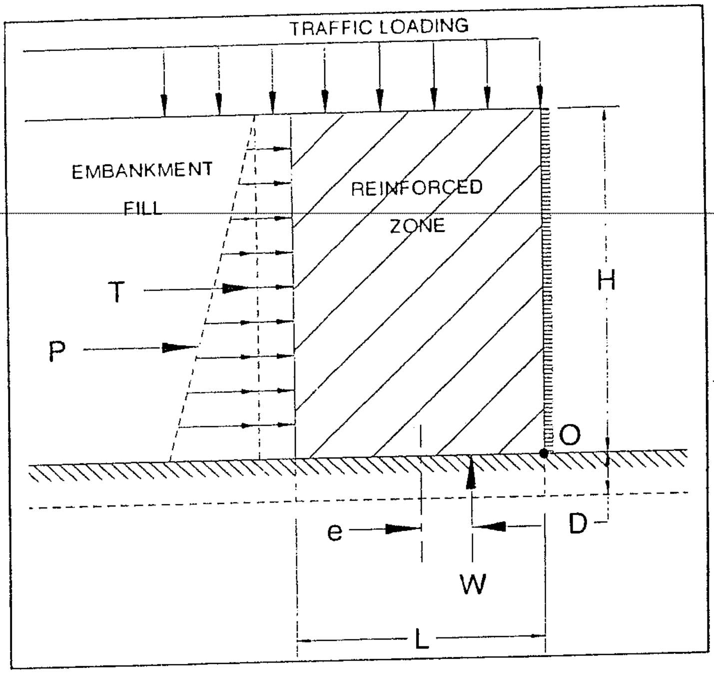
DLZ

SUBJECT
Client TranSystems
Project SCI 823-0.00
Item Bearing Capacity (Rear Abutment)
05 - 823 over Swauger Valley-Minford Rd TR-20
Bedrock / Granular Fill Foundation

JOB NUMBER 0121-3070.03
SHEET NO. 1 OF 4
COMP. BY SJR DATE 9/25/06
CHECKED BY WMA DATE 9/25/06

BEARING CAPACITY OF A MSE WALL *(non-coped)*

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



Effective Bearing Pressure

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

σ_v = 10,238 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 39,733 psf

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

q_{ALL} = 15,893 psf

Factor of Safety = 3.88 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} = 39,733 psf

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

q_{ALL} = 15,893 psf

Factor of Safety = 3.88 OK

Soil Properties

γ_{MSE}	=	120	pcf	unit weight	EMB/MSE
ϕ'_{MSE}	=	30	deg.	friction ang.	embankment
γ_{FDN}	=	120	pcf	unit weight	foundation soil
c_{FDN}	=	0	psf	cohesion	undrained
ϕ_{FDN}	=	34	deg.	friction ang.	undrained
c'_{FDN}	=	0	psf	cohesion	drained
ϕ'_{FDN}	=	34	deg.	friction ang.	drained

Loads and Parameters

ωt	=	240	psf	traffic loading
L=B	=	44.1	ft	length of mse block
L factor	=	0.7		Length factor-range (0.7 - 1.0)
D	=	0	ft	embedment depth
Dw	=	0	ft	groundwater depth
H+D	=	63	ft	
H	=	63	ft	height of wall
Ka	=	0.33		
Γ Pa	=	21	ft	moment arm
Γ Wt	=	31.5	ft	moment arm
B'	=	33.60	ft	
γ '	=	57.6	pcf	
W_t	=	10,584	lb/ft of wall	
W_{mse}	=	333,396	lb/ft of wall	

Bearing Capacity Factors for Equations

Undrained		Drained	
N_c	42.16	N_c	42.16
N_q	29.44	N_q	29.44
N	41.06	N	41.06

Eccentricity of Resultant Force

Kern

DLZ

SUBJECT
Client TranSystems ODOT D-9
Project SCI 823-0.00 Portsmouth Bypass
Item MSE Wall Stability (Rear Abutment)
05 - 823 over Swauger Valley - Minford Rd
Bedrock / Gran Fill

JOB NUMBER 0121-3070.03
SHEET NO. 2 OF 4
COMP. BY SJR DATE 09/25/06
CHECKED BY WMA DATE 9/25/06

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=63'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5

Wall Properties

H+D = 63 feet
γ_{mse} = 120 pcf
L = 44.1 feet
L factor = 0.70
Range (0.7-1.0)

Foundational Soil Properties

c = 0 psf cohesion
ϕ' = 34 deg friction angle
ω_T = 240 psf traffic loading

Embankment Soil Properties

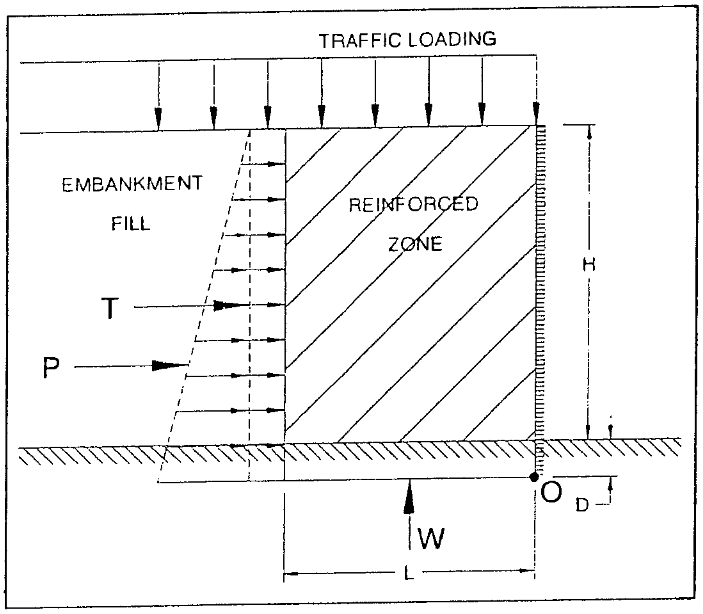
c = 0 psf cohesion
ϕ' = 30 deg friction angle

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 = 83,576 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 = 150,028 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 = **1.80**

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

$\Sigma M_{resisting}$ = 7,351,382 lb-ft

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

$\Sigma M_{overturning}$ = 1,807,483 lb-ft

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

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

Calculated FS = **4.07**

Required FS = 2.00

Resistance Against Overturning is **OK**

DLZ

SUBJECT

Client TranSystems

Project SCI 823-0.00

Item Bearing Capacity (Forward Abutment)

05 - 823 over Swauger Valley-Minford Rd TR-20

Bedrock / Granular Fill Foundation

JOB NUMBER 0121-3070.03

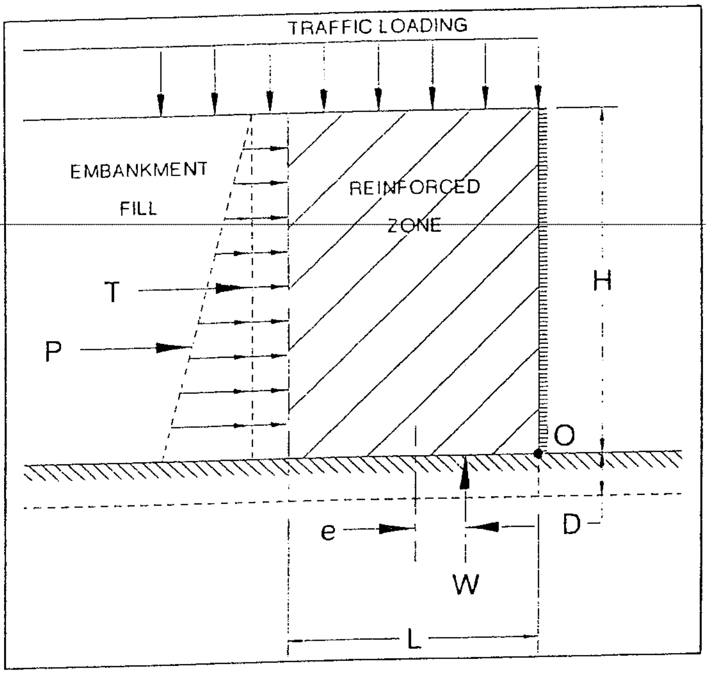
SHEET NO. 3 OF 4

COMP. BY SJR DATE 9/25/06

CHECKED BY WHA DATE 9/25/06

BEARING CAPACITY OF A MSE WALL *(non-coped)*

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



Soil Properties

γ_{MSE}	=	120	pcf	unit weight	EMB/MSE
ϕ'_{MSE}	=	30	deg.	friction ang.	embankment
γ_{FDN}	=	120	pcf	unit weight	foundation soil
c_{FDN}	=	0	psf	cohesion	undrained
ϕ_{FDN}	=	34	deg.	friction ang.	undrained
c'_{FDN}	=	0	psf	cohesion	drained
ϕ'_{FDN}	=	34	deg.	friction ang.	drained

Loads and Parameters

ωt	=	240	psf	traffic loading
L=B	=	40.95	ft	length of mse block
L factor	=	0.7		Length factor-range (0.7 - 1.0)
D	=	0	ft	embedment depth
Dw	=	0	ft	groundwater depth
H+D	=	58.5	ft	
H	=	58.5	ft	height of wall
Ka	=	0.33		
Γ Pa	=	19.5	ft	moment arm
Γ Wt	=	29.25	ft	moment arm
B'	=	31.15	ft	
γ'	=	57.6	pcf	
W_t	=	9,828	lb/ft of wall	
W_{mse}	=	287,469	lb/ft of wall	

Effective Bearing Pressure

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

σ_v = 9,544 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 36,836 psf

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

q_{ALL} = 14,734 psf

Factor of Safety = 3.86 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} = 36,836 psf

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

q_{ALL} = 14,734 psf

Factor of Safety = 3.86 OK

Bearing Capacity Factors for Equations

Undrained		Drained	
N_c	42.16	N_c	42.16
N_q	29.44	N_q	29.44
N	41.06	N	41.06

Eccentricity of Resultant Force

e = 4.90 ft

Kern

e < L/6 = 6.83 ft

DLZ

SUBJECT
Client TranSystems ODOT D-9
Project SCI 823-0.00 Fonsmouth Bypass
Item MSE Wall Stability (Forward Abutment)
05 - 823 over Swauger Valley - Minford Rd
Bedrock / Gran Fill

JOB NUMBER 0121-3070.05
SHEET NO. 4 OF 4
COMP. BY SJR DATE 09/25/06
CHECKED BY WMA DATE 9/25/06

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=58.5'
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

$H+D$ = 58.5 feet
γ_{mse} = 120 pcf
L = 40.95 feet
L factor = 0.70
Range (0.7-1.0)

Foundational Soil Properties

c	=	0	psf	cohesion
ϕ'	=	34	deg	friction angle
ω_T	=	240	psf	traffic loading

Embankment Soil Properties

c	=	0	psf	cohesion
ϕ'	=	30	deg	friction angle

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$_a$ = 72,394 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)

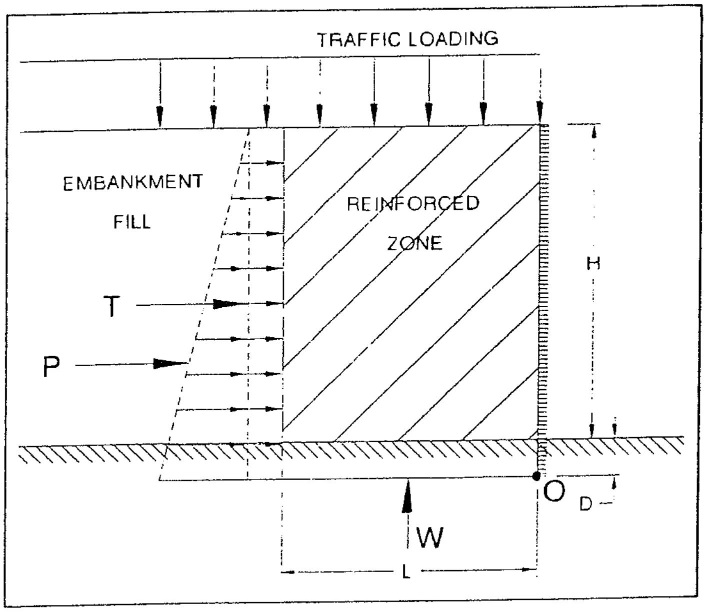
P$_r$ = 129,361 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P$_r$ = 0 lbs per foot of wall

Use Drained Value



	Calculated	Required
$FS = \frac{P_r}{P_a}$	FS = **1.79**	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

$\Sigma M_{resisting}$ = 5,885,928 lb-ft $\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

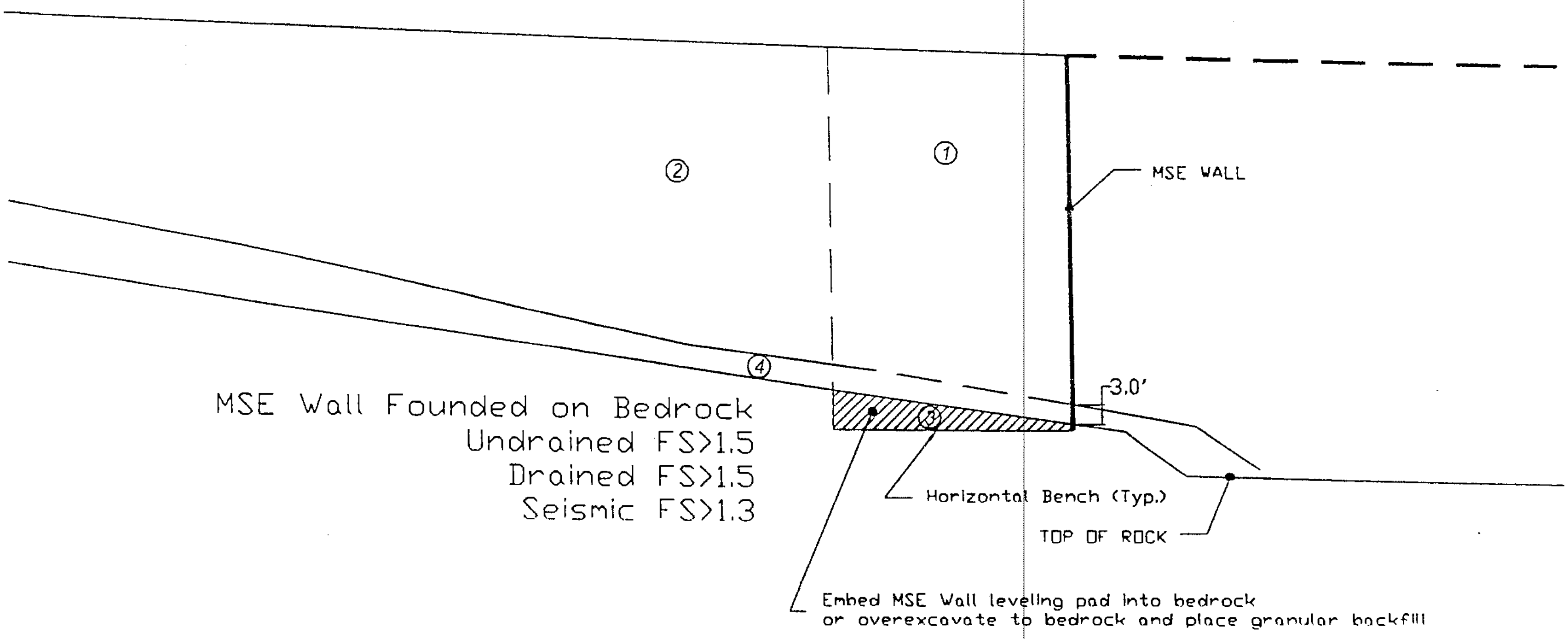
$\Sigma M_{overturning}$ = 1,456,852 lb-ft $\Sigma M_{overturnin} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

Resistance Against Overturning is OK

	Calculated	Required
FS = $\Sigma M_{resisting}$	FS = **4.04**	FS = 2.00

			Undrained		Drained		
Material	Consistency	Soil Type	C (psf)	Φ (deg)	C' (psf)	Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Compacted	Gran. Fill	0	34	0	34	120
Material 4	Hard	Silt and Clay	4500	0	0	29	120

MSE Wall STA: 442+17 (Rear Abutment)
Swauger Valley-Minford Road
Based on TR-23
H=63' (FROM PROPOSED GRADE TO PROPOSED
SR 823 GRADE)
Embedment=0.0' (BEDROCK)
Length=0.7(H+D)=44.1'



MSE Wall Founded on Bedrock
Undrained FS>1.5
Drained FS>1.5
Seismic FS>1.3

823 OVER SWAUGER VALLEY - MINFORD ROAD
REAR ABUTMENT

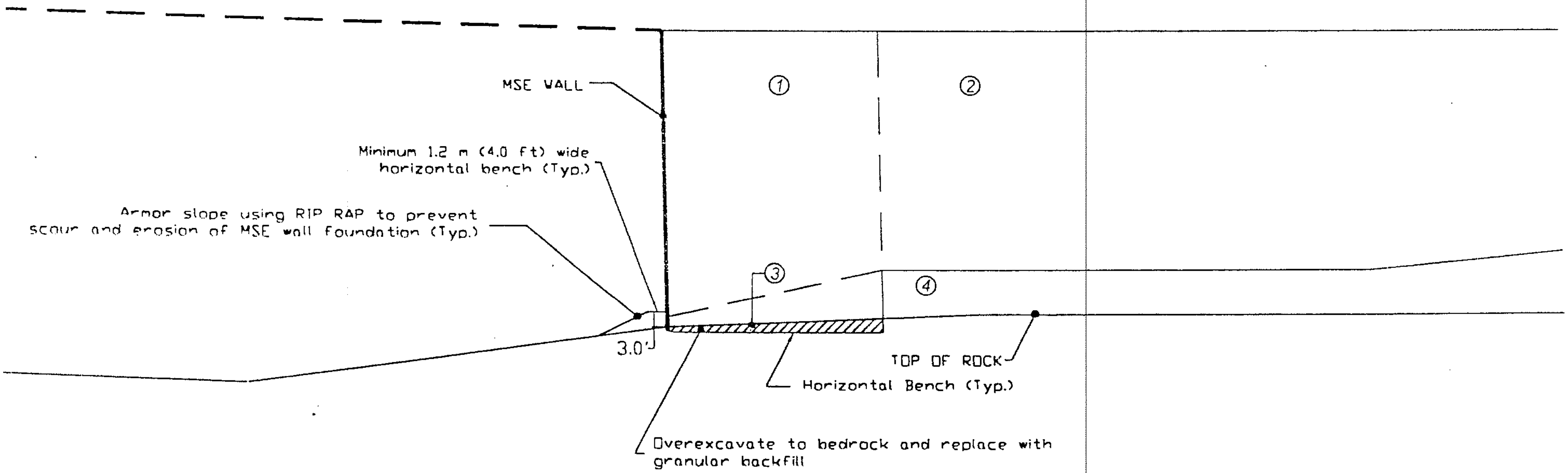
MSE STABILITY

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 9/25/06

MSE Wall STA: 444+14 (Forward Abutment)
Swauger Valley-Minford Road
Based on B-5, TR-20
H=58.5' (Proposed Grade at Base to Proposed Grade SR 823)
Embedment=0' (Bedrock)
Length=0.7(H+D)=41.0'

			Undrained		Drained		
Material	Consistency	Soil Type	C (psf)	Φ (deg)	C' (psf)	Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Compacted	Gran. Fill	0	34	0	34	120
Material 4	Soft	Clay	500	0	0	29	120



MSE Wall Founded on Bedrock
Undrained FS>1.5
Drained FS>1.5
Seismic FS>1.3

823 OVER SWAUGER VALLEY - MINFORD ROAD
FORWARD ABUTMENT

MSE STABILITY

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 9/25/06

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DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

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

PROJECT NO. 0121-3070.03
SHEET NO 1 OF 2
COMP. BY SJR DATE 9-12-06
CHECKED BY WMA DATE 9/12/06

*From lab testing rock core samples $q_u = 8000\ psi$

FHWA-IF-99-025 Eq^{n} 11.6 $q_{max}(MPa) = 4.83[q_u(MPa)]^{0.51}$

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]

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

$$q_{max} = 4.83[55.16\ MPa]^{0.51} = 35.87\ MPa$$

$$q_{max} = 35.87\ MPa = 5202\ psi = 749\ ksf$$

$$q_{allow} = \frac{q_{max}}{FS} = \frac{749\ ksf}{3.0} = 250\ ksf$$

* Rock is stronger than concrete, Use $q_u = f_c' = 4500\ psi$

$$q_{max} = 4038\ psi = 581\ ksf$$

$$q_{allow} = \frac{581\ ksf}{3.0} = 193.7\ ksf$$

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

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT: TranSystems Corp.

PROJECT: SCI-823 Portsmouth Bypass

SUBJECT: Drilled Shaft - Side Resistance

Swauger Valley - Minford Road

PROJECT NO: 0121-3070.03

SHEET NO: 2 OF 2

COMP. BY: SJR DATE: 9-12-06

CHECKED BY: WMA DATE: 9/12/06

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

FHWA-IF-99-025 $\mathcal{E}_q \hat{=}$ 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

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

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

$$f_{max} = 222.9 \text{ psi} \leq 167.2 \text{ psi}$$

Use $f_{max} = 167$ psi

$$f_{allow} = \frac{167 \text{ psi}}{3.0} = 55 \text{ psi} = 7{,}920 \text{ psf}$$

$\boxed{{}^*\text{Use } f_{allow} = 7{,}500 \text{ psf}}$

Appendix C
Laboratory Test Results

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-001-0-08	S-1	1.0-2.5	24.7	0.1	2.8	13.5	41.8	41.8	37	21	16	CL	A-6b(10)
B-001-0-08	S-2	3.5-5.0	18.3	-	-	-	-	-	-	-	-	-	-
B-001-0-08	S-3	6.0-7.5	15.3	0.0	0.8	3.1	65.4	30.7	24	23	1	ML	A-4b(8)
B-001-0-08	S-4	8.5-10.0	8.8	-	-	-	-	-	-	-	-	-	-
B-001-0-08	S-5	10.0-10.1	10.4	-	-	-	-	-	-	-	-	-	-
B-002-0-08	S-1	1.0-2.5	15.8	2.0	2.7	3.1	61.6	30.6	26	19	7	CL-ML	A-4b(8)
B-002-0-08	S-2	3.5-5.0	19.7	-	-	-	-	-	-	-	-	-	-
B-002-0-08	S-3	6.0-7.5	12.4	0.0	5.0	4.4	62.9	27.7	32	21	11	CL	A-6a(8)
B-002-0-08	S-4	8.5-10.0	12.4	-	-	-	-	-	-	-	-	-	-
B-002-0-08	S-5	11.0-12.5	6.6	-	-	-	-	-	-	-	-	-	-

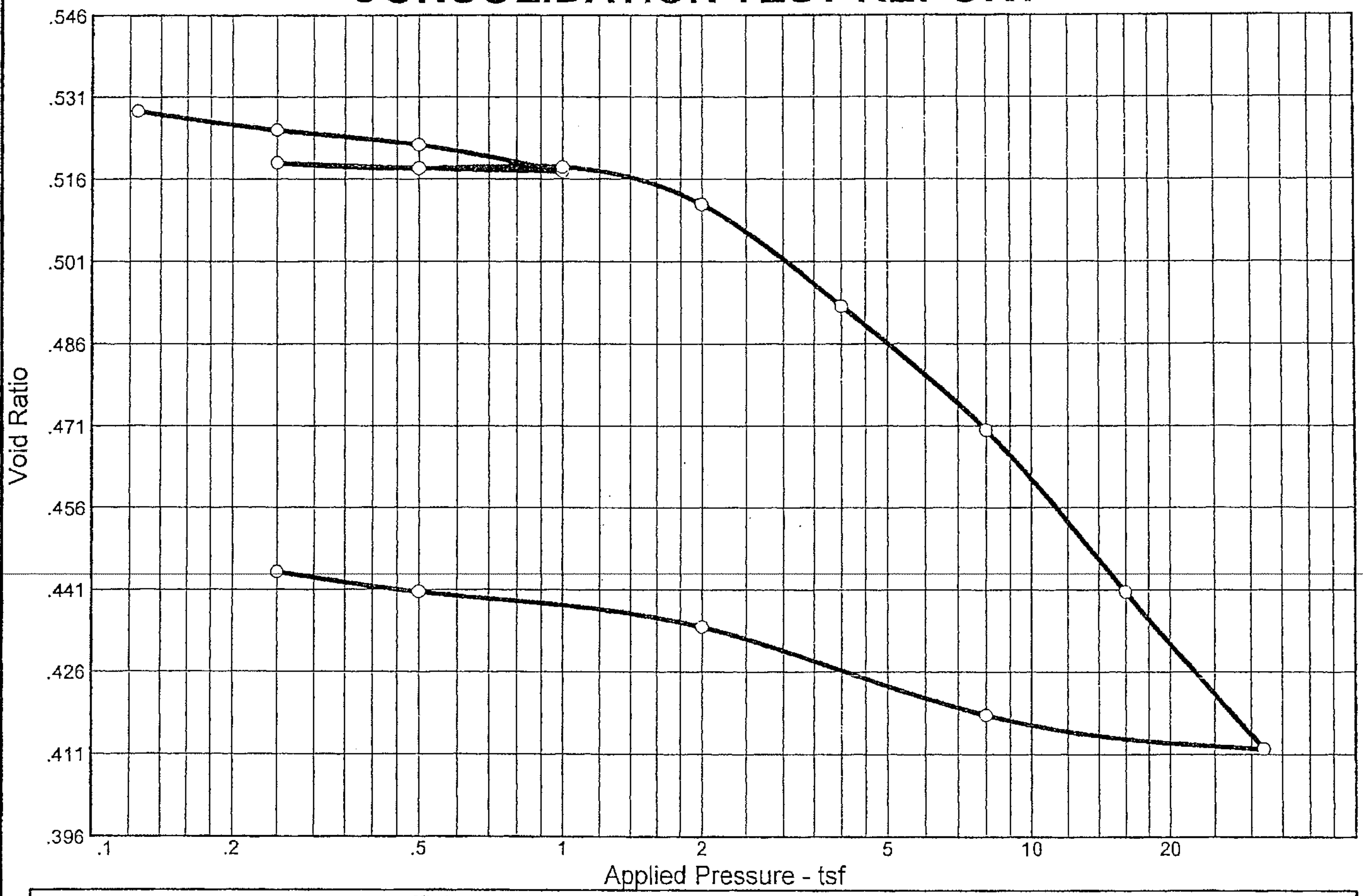
One-Dimmensional Consolidation Test Data Summary

Boring No.	Depth (ft)	Natural		Dry Density (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio	USCS	AASHTO
		Saturation	Moisture											
B-002-0-08	2.5-4.4	58.7%	11.6%	109.1	36	16	2.67	-	3.1	0.09	0.01	0.528	CL	A-6(13)

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-001-0-08	13.1-13.3	4.0	1.98	471.6	28390	9249	2.01	9249
B-001-0-08	13.3-13.7	4.0	1.98	461.6	27090	8833	2.01	8833
B-002-0-08	15.8-16.1	3.5	1.98	397.2	32380	10547	1.75	10336
B-002-0-08	16.1-16.4	3.9	1.97	458.9	30850	10092	2.00	10092

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation											
No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α
2	0.25	0.18									
3	0.50	1.29									
4	1.00	0.77									
7	0.50	0.31									
8	1.00	0.83									
9	2.00	1.10									
10	4.00	0.20									
11	8.00	0.57									
12	16.00	0.98									
13	32.00	0.75									

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
58.7 %	11.6 %	109.1	36	16	2.67		3.10	0.09	0.01	0.528

MATERIAL DESCRIPTION	USCS	AASHTO
brown Silty Clay	CL	A-6(13)

Project No. 25506 **Client:** HDR Engineering, Inc.

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

Location: Shelby Tube B-002-0-08 2.5'-4.4'

TES TECH

Dayton, Ohio

Remarks:

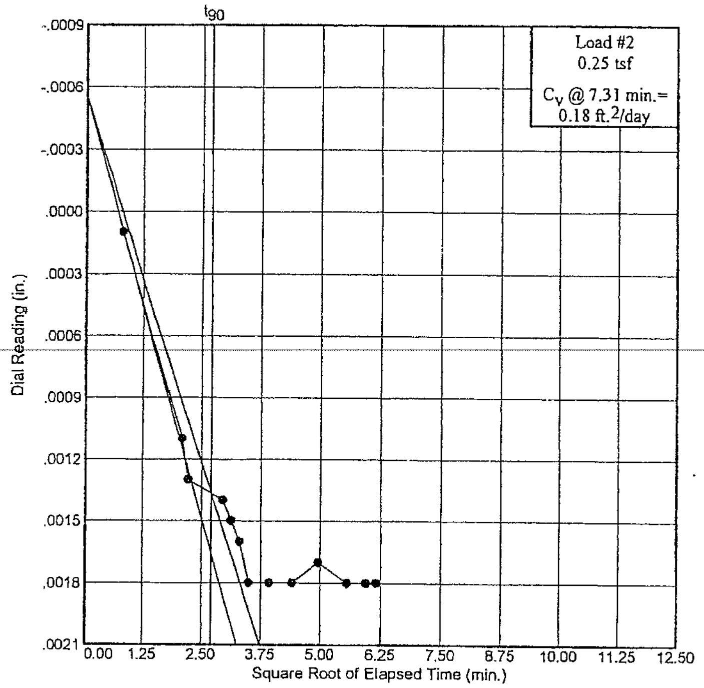
Date Received: 2/14/08
Lab No.: 0128

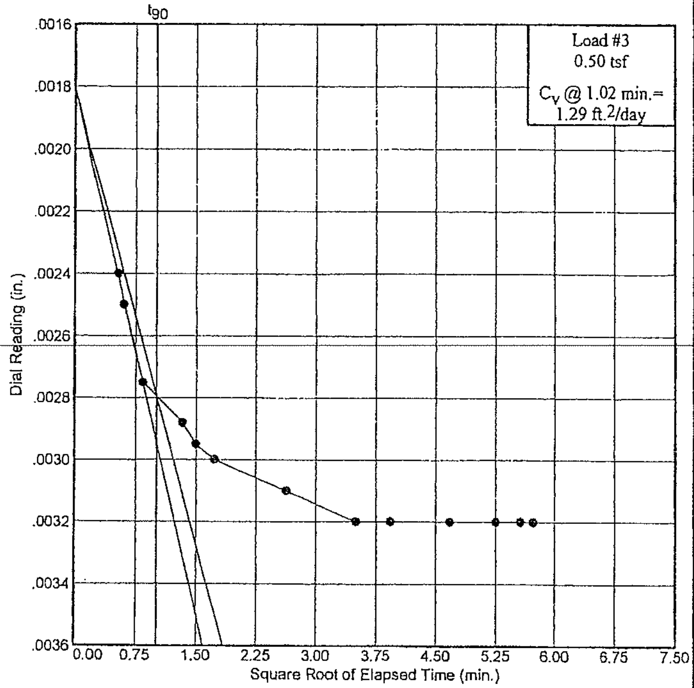
File No.

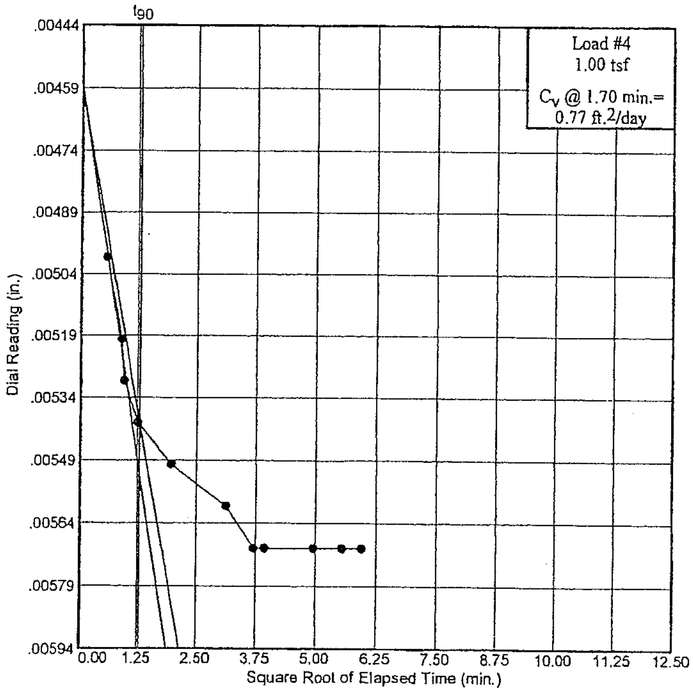
Dial Reading vs. Time

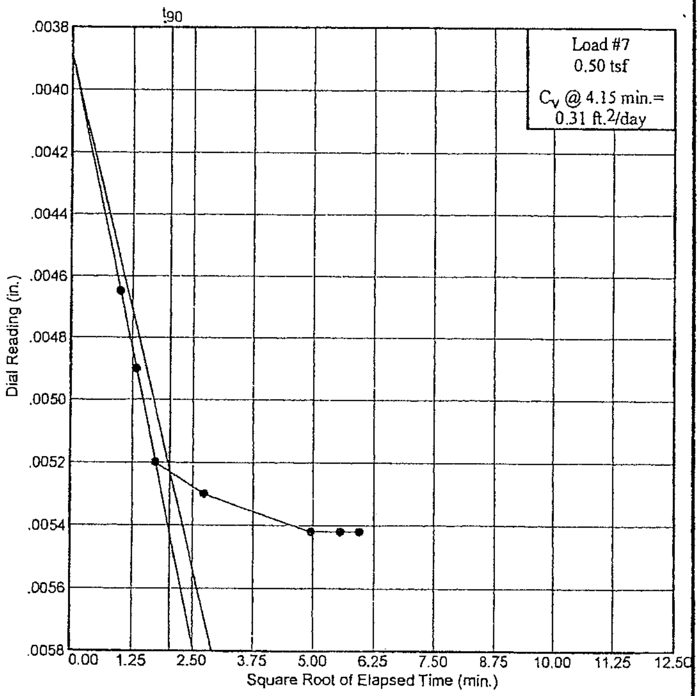
Project No.: 25506
Project: ODOT District 9 Portsmouth Bypass, SCI-823-0.00/6.81, PID #19415 - Minford, Ohio

Location: Shelby Tube B-002-0-08 2.5'-4.4'









TES TECH
Dayton, Ohio

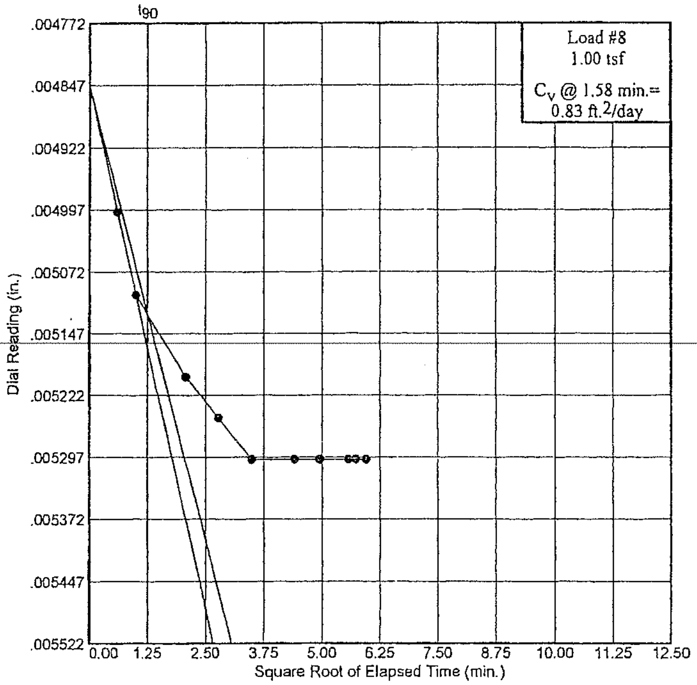
File No.

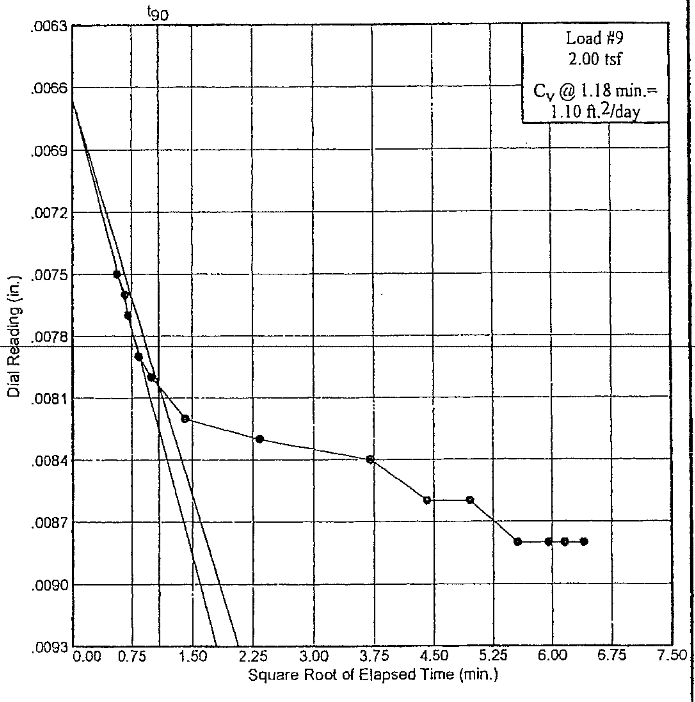
Dial Reading vs. Time

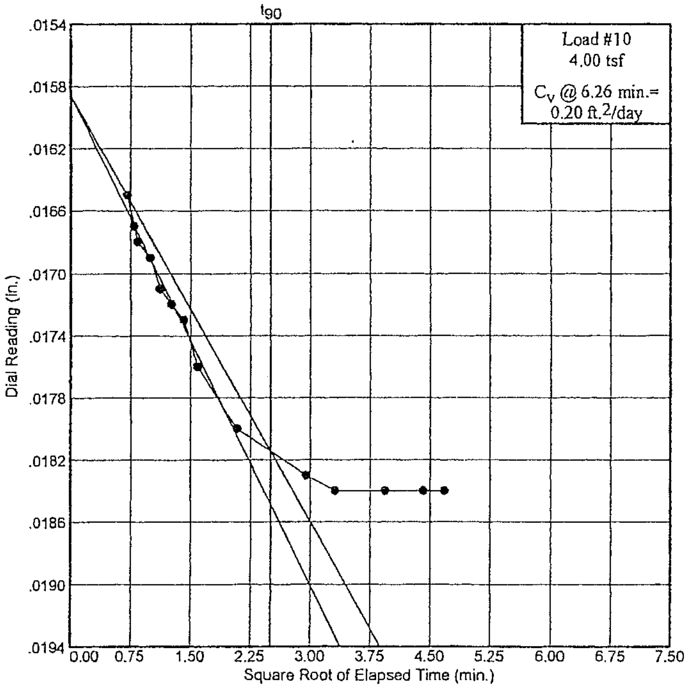
Project No.: 25506

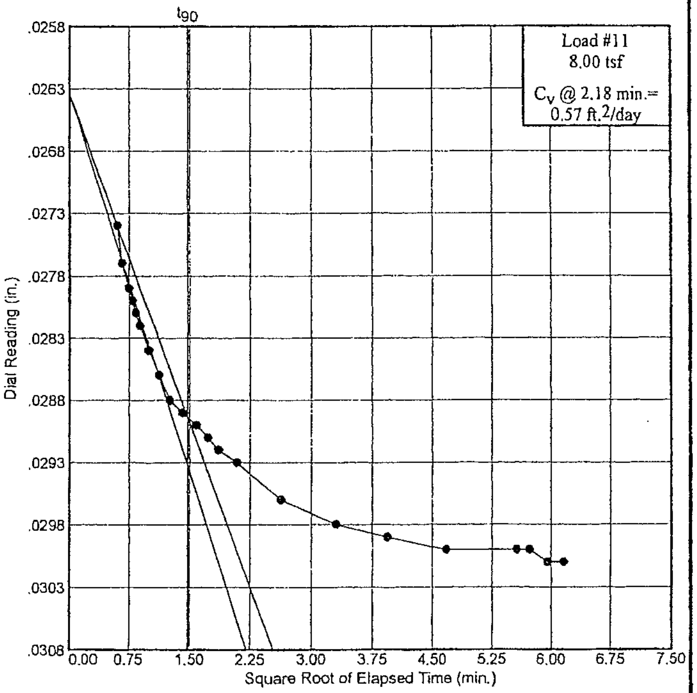
Project: ODOT District 9 Portsmouth Bypass, SCI-823-0.00/6.81, PID #19415 - Minford, Ohio

Location: Shelby Tube B-002-0-08 2.5'-4.4'









TES TECH

Dayton, Ohio

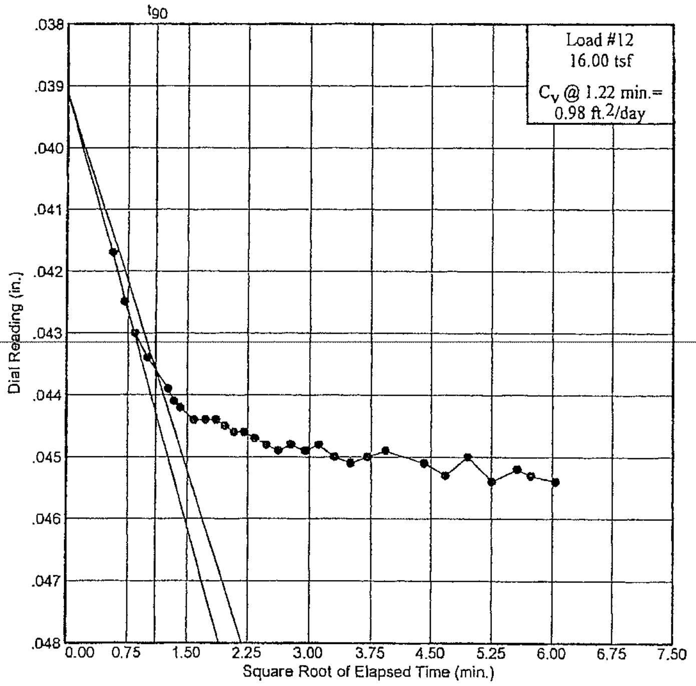
File No.

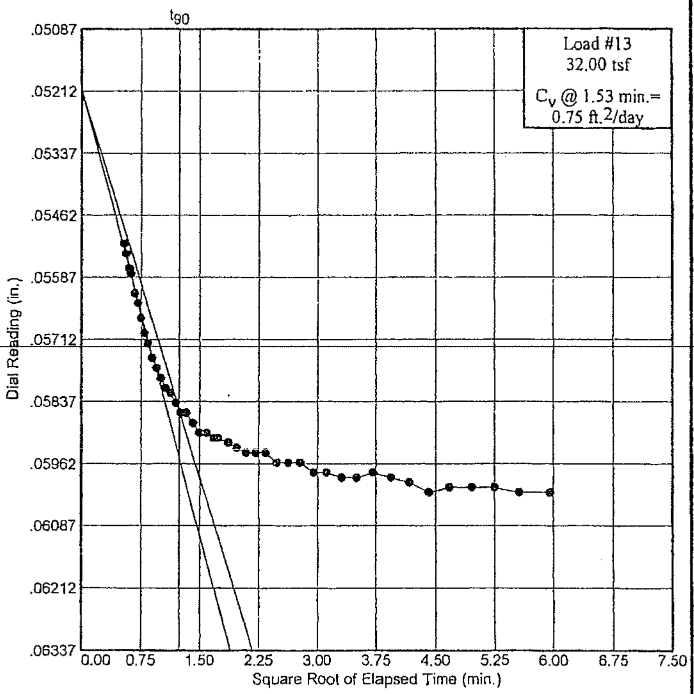
Dial Reading vs. Time

Project No.: 25506

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

Location: Shelby Tube B-002-0-08 2.5'-4.4'





TES TECH

Dayton, Ohio

File No.

CONSOLIDATION TEST DATA

Client: HDR Engineering, Inc.
Project: ODOT District 9 Portsmouth Bypass, SCI-823-0.00/6.81, PID #19415 - Minford, Ohio
Project Number: 25506

Sample Data

Source: 0128 GR 2/14/08
Sample No.: 0128
Elev. or Depth: 2.5' - 4.4'   **Sample Length (in./cm.):**
Location: Shelby Tube B-002-0-08 2.5'-4.4'
Description: brown Silty Clay
Liquid Limit: 36   **Plasticity Index:** 16
USCS: CL   **AASHTO:** A-6(13)   **Figure No.:**
Testing Remarks: Date Received: 2/14/08
Lab No.: 0128

Test Specimen Data

TOTAL SAMPLE

Wet w+t = 189.89 g.
Dry w+t = 177.00 g.
Tare Wt. = 65.99 g.
Height = .79 in.
Diameter = 2.50 in.
Weight = 123.90 g.

Moisture = 11.6 %
Wet Den. = 121.7 pcf
Dry Den. = 109.1 pcf

BEFORE TEST

Consolidometer # = 2

Spec. Gravity = 2.67
Height = .79 in.
Diameter = 2.50 in.
Defl. Table = Reference Set (inches/tsf)

Ht. Solids = 0.5169 in.
Dry Wt. = 111.01 g.*
Void Ratio = 0.528
Saturation = 58.7 %

AFTER TEST

Wet w+t = 137.72 g.
Dry w+t = 117.38 g.
Tare Wt. = .00 g.

Moisture = 17.3 %
Dry Wt. = 117.38 g.
Void Ratio = 0.444

*** Initial dry weight used in calculations**

End-of-Load Summary

Pressure (tsf)	Final Dial (in.)	Machine Defl. (in.)	C_v (ft.^{2}/day)	C_α	Void Ratio	% Compression /Swell
start	0.00000				0.528	
0.13	0.00000	0.00000			0.528	0.0 Swell
0.25	0.00220	0.00040	0.18		0.525	0.2 Comprs.
0.50	0.00400	0.00080	1.29		0.522	0.4 Comprs.
1.00	0.00730	0.00160	0.77		0.517	0.7 Comprs.
0.50	0.00620	0.00080			0.518	0.7 Comprs.
0.25	0.00530	0.00040			0.519	0.6 Comprs.
0.50	0.00622	0.00080	0.31		0.518	0.7 Comprs.
1.00	0.00690	0.00160	0.83		0.518	0.7 Comprs.
2.00	0.01120	0.00240	1.10		0.511	1.1 Comprs.
4.00	0.01840	0.00000	0.20		0.493	2.3 Comprs.
8.00	0.03010	0.00000	0.57		0.470	3.8 Comprs.
16.00	0.04540	0.00000	0.98		0.441	5.7 Comprs.

Pressure (tsf)	Final Dial (in.)	Machine Defl. (in.)	C_v (ft.^{2}/day)	C_α	Void Ratio	% Compression /Swell
32.00	0.06020	0.00000	0.75		0.412	7.6 Comprs.
8.00	0.05700	0.00000			0.418	7.2 Comprs.
2.00	0.05170	0.00300			0.434	6.2 Comprs.
0.50	0.04620	0.00080			0.441	5.7 Comprs.
0.25	0.04390	0.00040			0.444	5.5 Comprs.

C_c = 0.09 P_c = 3.10 tsf C_r = 0.01

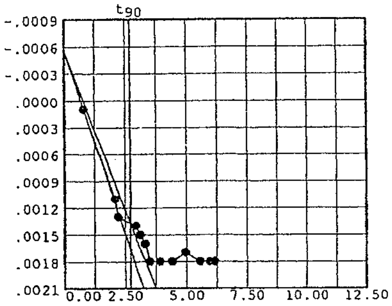
Pressure: 0.13 tsf **TEST READINGS** **Load No. 1**

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.00000	11	0.80	-0.00020	21	2.52	0.00000
2	0.29	0.00000	12	0.89	-0.00010	22	2.83	0.00000
3	0.32	0.00000	13	1.00	0.00000	23	3.00	-0.00010
4	0.37	0.00000	14	1.13	0.00000	24	3.46	0.00000
5	0.39	0.00000	15	1.26	0.00000	25	6.16	0.00000
6	0.45	0.00000	16	1.41	0.00000	26	9.75	0.00000
7	0.50	0.00000	17	1.59	0.00020	27	15.49	0.00000
8	0.56	-0.00010	18	1.78	0.00020	28	24.60	0.00000
9	0.63	-0.00010	19	2.00	0.00010	29	32.86	0.00000
10	0.71	-0.00010	20	2.24	0.00020	30	40.99	0.00000

Void Ratio = 0.528   **Compression** = 0.0 %

Pressure: 0.25 tsf **TEST READINGS** **Load No. 2**

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.00010	11	24.60	0.00210
2	0.63	0.00050	12	30.98	0.00220
3	4.36	0.00150	13	35.50	0.00220
4	4.90	0.00170	14	37.95	0.00220
5	8.72	0.00180			
6	9.75	0.00190			
7	10.95	0.00200			
8	12.29	0.00220			
9	15.49	0.00220			
10	19.52	0.00220			



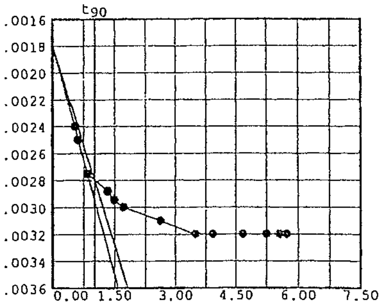
Void Ratio = 0.525   **Compression** = 0.2 %

D_0 = -0.00055 D_{90} = 0.00137 D_{100} = 0.00158

C_v **at 7.3 min.** = 0.18 ft.^{2}/day

Pressure: 0.50 tsf TEST READINGS Load No. 3

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.00210	11	21.91	0.00400
2	0.29	0.00320	12	27.60	0.00400
3	0.37	0.00330	13	30.98	0.00400
4	0.71	0.00355	14	32.86	0.00400
5	1.78	0.00368			
6	2.24	0.00375			
7	3.00	0.00380			
8	6.93	0.00390			
9	12.29	0.00400			
10	15.49	0.00400			



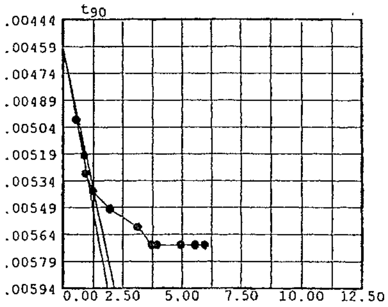
Void Ratio = 0.522 Compression = 0.4 %

D_0 = 0.00180 D_{90} = 0.00279 D_{100} = 0.00290

C_v at 1.0 min. = 1.29 ft.^{2}/day

Pressure: 1.00 tsf TEST READINGS Load No. 4

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.00390	11	30.98	0.00730
2	0.32	0.00660	12	35.50	0.00730
3	0.80	0.00680			
4	0.89	0.00690			
5	1.59	0.00700			
6	3.87	0.00710			
7	9.75	0.00720			
8	13.78	0.00730			
9	15.49	0.00730			
10	24.60	0.00730			



Void Ratio = 0.517 Compression = 0.7 %

D_0 = 0.00459 D_{90} = 0.00541 D_{100} = 0.00550

C_v at 1.7 min. = 0.77 ft.^{2}/day

Pressure: 0.50 tsf TEST READINGS Load No. 5

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.00680	8	0.56	0.00620	15	1.13	0.00640
2	0.29	0.00610	9	0.63	0.00610	16	1.59	0.00620
3	0.32	0.00620	10	0.71	0.00610	17	2.00	0.00620
4	0.37	0.00620	11	0.80	0.00620	18	2.24	0.00610
5	0.39	0.00620	12	0.89	0.00620	19	2.52	0.00610
6	0.45	0.00620	13	1.00	0.00630	20	2.83	0.00610
7	0.50	0.00620	14	1.13	0.00610	21	3.00	0.00620

Void Ratio = 0.518 Compression = 0.7 %

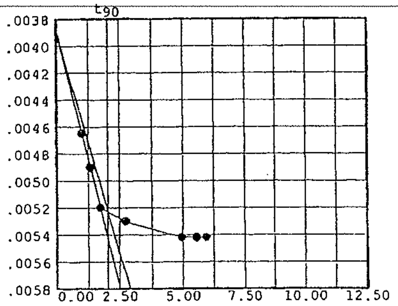
Pressure: 0.25 tsf — **TEST READINGS** — **Load No.** 6

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.00620	11	1.59	0.00550
2	0.32	0.00550	12	1.78	0.00540
3	0.45	0.00560	13	2.00	0.00540
4	0.50	0.00560	14	2.24	0.00530
5	0.56	0.00560	15	2.52	0.00530
6	0.63	0.00560	16	2.83	0.00530
7	0.89	0.00560	17	3.00	0.00530
8	1.00	0.00560			
9	1.13	0.00560			
10	1.26	0.00560			

Void Ratio = 0.519 **Compression** = 0.6 %

Pressure: 0.50 tsf — **TEST READINGS** — **Load No.** 7

No.	Elapsed Time	Dial Reading
1	0.00	0.00530
2	1.02	0.00545
3	1.78	0.00570
4	3.00	0.00600
5	7.48	0.00610
6	24.60	0.00622
7	30.98	0.00622
8	35.50	0.00622



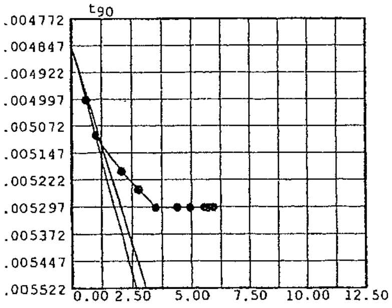
Void Ratio = 0.518 **Compression** = 0.7 %

D_0 = 0.00388 D_{90} = 0.00523 D_{100} = 0.00538

C_v **at 4.1 min.** = 0.31 ft.^{2}/day

Pressure: 1.00 tsf — **TEST READINGS** — **Load No.** 8

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.00620	11	35.50	0.00690
2	0.37	0.00660			
3	1.00	0.00670			
4	4.36	0.00680			
5	7.75	0.00685			
6	12.29	0.00690			
7	19.52	0.00690			
8	24.60	0.00690			
9	30.98	0.00690			
10	32.86	0.00690			



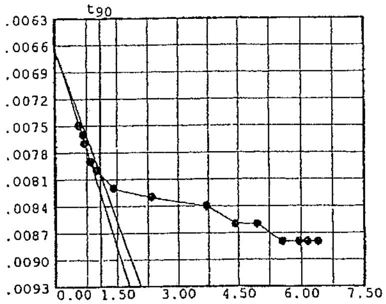
Void Ratio = 0.518 **Compression** = 0.7 %

D_0 = 0.00485 D_{90} = 0.00512 D_{100} = 0.00515

C_v **at 1.6 min.** = 0.83 ft.^{2}/day

Pressure: 2.00 tsf TEST READINGS Load No. 9

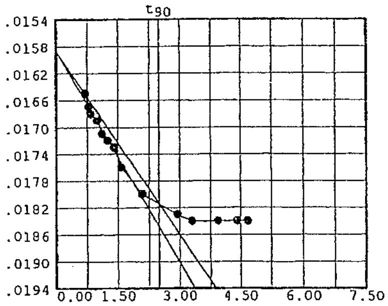
No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.00680	11	24.60	0.01100
2	0.32	0.00990	12	30.98	0.01120
3	0.45	0.01000	13	35.50	0.01120
4	0.50	0.01010	14	37.95	0.01120
5	0.71	0.01030	15	40.99	0.01120
6	1.00	0.01040			
7	2.00	0.01060			
8	5.48	0.01070			
9	13.78	0.01080			
10	19.52	0.01100			



Void Ratio = 0.511 **Compression** = 1.1 %
D_0 = 0.00666 D_{90} = 0.00804 D_{100} = 0.00820
C_v **at 1.2 min.** = 1.10 ft.^{2}/day

Pressure: 4.00 tsf TEST READINGS Load No. 10

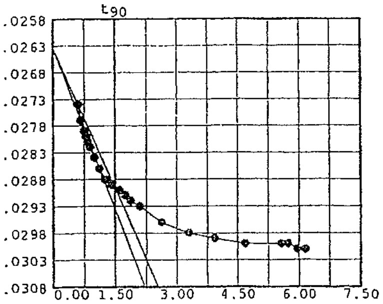
No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.01120	11	8.72	0.01830
2	0.50	0.01650	12	10.95	0.01840
3	0.63	0.01670	13	15.49	0.01840
4	0.71	0.01680	14	19.52	0.01840
5	1.00	0.01690	15	21.91	0.01840
6	1.26	0.01710			
7	1.59	0.01720			
8	2.00	0.01730			
9	2.52	0.01760			
10	4.36	0.01800			



Void Ratio = 0.493 **Compression** = 2.3 %
D_0 = 0.01586 D_{90} = 0.01814 D_{100} = 0.01840
C_v **at 6.3 min.** = 0.20 ft.^{2}/day

Pressure: 8.00 tsf TEST READINGS Load No. 11

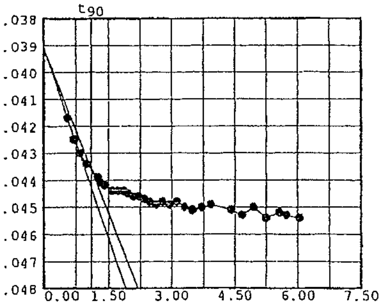
No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.01840	13	3.00	0.02910
2	0.37	0.02740	14	3.46	0.02920
3	0.45	0.02770	15	4.36	0.02930
4	0.56	0.02790	16	6.93	0.02960
5	0.63	0.02800	17	10.95	0.02980
6	0.71	0.02810	18	15.49	0.02990
7	0.80	0.02820	19	21.91	0.03000
8	1.00	0.02840	20	30.98	0.03000
9	1.26	0.02860	21	32.86	0.03000
10	1.59	0.02880	22	35.50	0.03010
11	2.00	0.02890	23	37.95	0.03010
12	2.52	0.02900			



Void Ratio = 0.470 **Compression** = 3.8 %
D_0 = 0.02634 D_{90} = 0.02894 D_{100} = 0.02922
C_v **at 2.2 min.** = 0.57 ft.^{2}/day

Pressure: 16.00 tsf TEST READINGS Load No. 12

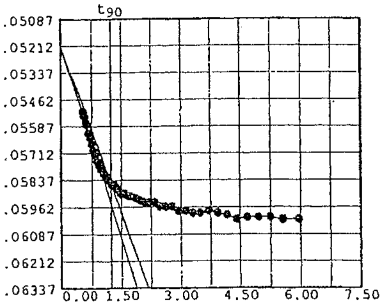
No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.03010	17	6.93	0.04490
2	0.32	0.04170	18	7.75	0.04480
3	0.50	0.04250	19	8.72	0.04490
4	0.71	0.04300	20	9.75	0.04480
5	1.00	0.04340	21	10.95	0.04500
6	1.59	0.04390	22	12.29	0.04510
7	1.78	0.04410	23	13.78	0.04500
8	2.00	0.04420	24	15.49	0.04490
9	2.52	0.04440	25	19.52	0.04510
10	3.00	0.04440	26	21.91	0.04530
11	3.46	0.04440	27	24.60	0.04500
12	3.87	0.04450	28	27.60	0.04540
13	4.36	0.04460	29	30.98	0.04520
14	4.90	0.04460	30	32.86	0.04530
15	5.48	0.04470	31	36.50	0.04540
16	6.16	0.04480			



Void Ratio = 0.441 **Compression** = 5.7 %
D_0 = 0.03911 D_{90} = 0.04360 D_{100} = 0.04410
C_v **at 1.2 min.** = 0.98 ft.^{2}/day

Pressure: 32.00 tsf TEST READINGS Load No. 13

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.04540	23	3.00	0.05910
2	0.29	0.05520	24	3.46	0.05920
3	0.32	0.05540	25	3.87	0.05930
4	0.37	0.05570	26	4.36	0.05940
5	0.39	0.05580	27	4.90	0.05940
6	0.45	0.05620	28	5.48	0.05940
7	0.50	0.05640	29	6.16	0.05960
8	0.56	0.05670	30	6.93	0.05960
9	0.63	0.05700	31	7.75	0.05960
10	0.71	0.05720	32	8.72	0.05980
11	0.80	0.05750	33	9.75	0.05980
12	0.89	0.05770	34	10.95	0.05990
13	1.00	0.05790	35	12.29	0.05990
14	1.13	0.05810	36	13.78	0.05980
15	1.26	0.05820	37	15.49	0.05990
16	1.41	0.05840	38	17.38	0.06000
17	1.59	0.05860	39	19.52	0.06020
18	1.78	0.05860	40	21.91	0.06010
19	2.00	0.05880	41	24.60	0.06010
20	2.24	0.05900	42	27.60	0.06010
21	2.52	0.05900	43	30.98	0.06020
22	2.83	0.05910	44	35.50	0.06020



Void Ratio = 0.412 Compression = 7.6 %

D_0 = 0.05212 D_{90} = 0.05853 D_{100} = 0.05925

C_v at 1.5 min. = 0.75 ft.^{2}/day

Pressure: 8.00 tsf TEST READINGS Load No. 14

No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	0.06020	9	0.71	0.05720	17	1.78	0.05700
2	0.32	0.05760	10	0.80	0.05720	18	2.00	0.05710
3	0.37	0.05750	11	0.89	0.05720	19	2.24	0.05700
4	0.39	0.05750	12	1.00	0.05710	20	2.52	0.05700
5	0.45	0.05740	13	1.13	0.05710	21	2.83	0.05700
6	0.50	0.05730	14	1.26	0.05700	22	3.00	0.05700
7	0.56	0.05720	15	1.41	0.05710			
8	0.63	0.05720	16	1.59	0.05700			

Void Ratio = 0.418 Compression = 7.2 %

Pressure: 2.00 tsf **TEST READINGS** **Load No.** 15

No.	**Elapsed Time**	**Dial Reading**	**No.**	**Elapsed Time**	**Dial Reading**	**No.**	**Elapsed Time**	**Dial Reading**
1	0.00	0.05700	9	0.63	0.05290	17	1.59	0.05210
2	0.05	0.28870	10	0.71	0.05280	18	1.78	0.05210
3	0.32	0.05360	11	0.80	0.05270	19	2.00	0.05190
4	0.37	0.05350	12	0.89	0.05260	20	2.24	0.05190
5	0.39	0.05340	13	1.00	0.05250	21	2.52	0.05180
6	0.45	0.05320	14	1.13	0.05250	22	2.83	0.05160
7	0.50	0.05310	15	1.26	0.05230	23	3.00	0.05170
8	0.56	0.05310	16	1.41	0.05230	24	3.46	0.05170

Void Ratio = 0.434 **Compression** = 6.2 %

Pressure: 0.50 tsf **TEST READINGS** **Load No.** 16

No.	**Elapsed Time**	**Dial Reading**	**No.**	**Elapsed Time**	**Dial Reading**	**No.**	**Elapsed Time**	**Dial Reading**
1	0.00	0.05170	9	0.63	0.04850	17	1.59	0.04720
2	0.29	0.04910	10	0.71	0.04830	18	1.78	0.04700
3	0.32	0.04910	11	0.80	0.04830	19	2.00	0.04680
4	0.37	0.04900	12	0.89	0.04810	20	2.24	0.04670
5	0.39	0.04900	13	1.00	0.04800	21	2.52	0.04640
6	0.45	0.04880	14	1.13	0.04780	22	2.83	0.04630
7	0.50	0.04870	15	1.26	0.04760	23	3.00	0.04620
8	0.56	0.04860	16	1.41	0.04740			

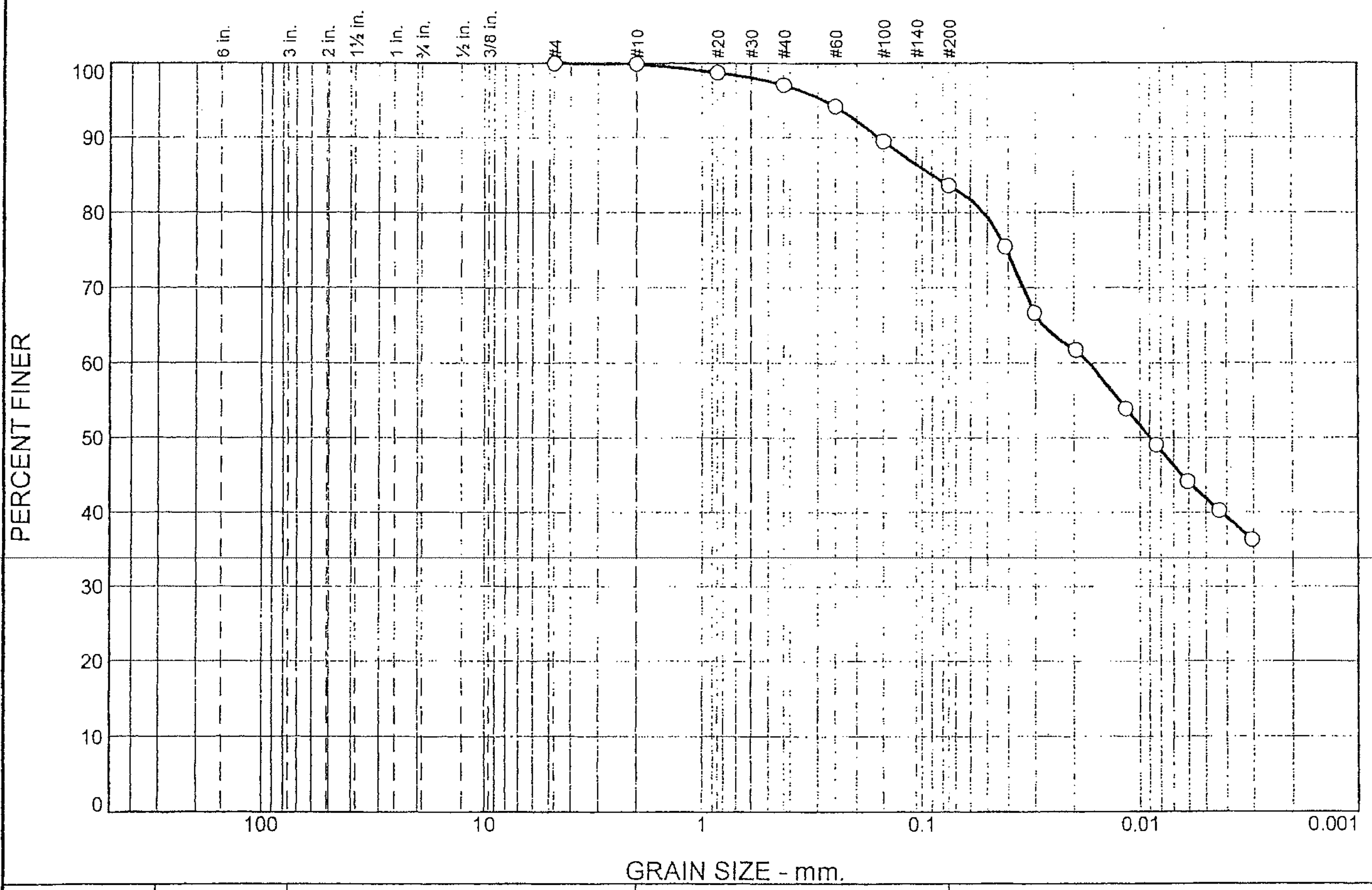
Void Ratio = 0.441 **Compression** = 5.7 %

Pressure: 0.25 tsf **TEST READINGS** **Load No.** 17

No.	**Elapsed Time**	**Dial Reading**	**No.**	**Elapsed Time**	**Dial Reading**	**No.**	**Elapsed Time**	**Dial Reading**
1	0.00	0.04620	9	0.63	0.04540	17	1.59	0.04490
2	0.29	0.04560	10	0.71	0.04530	18	1.78	0.04470
3	0.32	0.04550	11	0.80	0.04520	19	2.00	0.04450
4	0.37	0.04550	12	0.89	0.04510	20	2.24	0.04440
5	0.39	0.04540	13	1.00	0.04500	21	2.52	0.04420
6	0.45	0.04550	14	1.13	0.04510	22	2.83	0.04400
7	0.50	0.04540	15	1.26	0.04500	23	3.00	0.04390
8	0.56	0.04540	16	1.41	0.04490			

Void Ratio = 0.444 **Compression** = 5.5 %

Particle Size Distribution Report



% Boulders	% +3"	% Gravel		% Sand		% Fines	
		Coarse	Fine	Coarse	Fine	Silt	Clay
0.0	0.0	0.0	0.1	2.8	13.5	41.8	41.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#20	98.7		
#40	97.1		
#60	94.2		
#100	89.5		
#200	83.6		

* (no specification provided)

Material Description

orange / brown Silty Clay

Atterberg Limits

PL= 21 LL= 37 PI= 16

Coefficients

D_{85}= 0.0895 D_{60}= 0.0169 D_{50}= 0.0090
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL ODOT= A-6b(10)

Remarks

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

Sample Number: 0128 **Depth:** 1.0' - 2.5'
Location: B-001-0-08 (S-1)

Date: 2/20/08

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-001-0-08 (S-1)
Depth: 1.0' - 2.5' **Sample Number:** 0128
Material Description: orange / brown Silty Clay
Date: 2/20/08 **PL:** 21 **LL:** 37 **PI:** 16
USCS Classification: CL **AASHTO Classification:** A-6(13)
Ohio DOT Classification (if different from AASHTO): A-6b(10)
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
203.14	14.53	0.00	#4	0.00	100.0
			#10	0.22	99.9
51.71	0.00	0.00	#20	0.60	98.7
			#40	1.46	97.1
			#60	2.94	94.2
			#100	5.37	89.5
			#200	8.42	83.6

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 99.9
Weight of hydrometer sample =51.710
Hygroscopic moisture correction:
Moist weight and tare = 62.71
Dry weight and tare = 62.55
Tare weight = 49.45
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	43.0	38.6	0.0136	43.0	9.2	0.0412	75.4
2.00	20.5	38.5	34.1	0.0136	38.5	10.0	0.0303	66.6
5.00	20.5	36.0	31.6	0.0136	36.0	10.4	0.0195	61.7
15.00	20.5	32.0	27.6	0.0136	32.0	11.0	0.0116	53.9
30.00	20.5	29.5	25.1	0.0136	29.5	11.5	0.0084	49.0
60.00	20.5	27.0	22.6	0.0136	27.0	11.9	0.0060	44.1
120.00	20.5	25.0	20.6	0.0136	25.0	12.2	0.0043	40.2
250.00	20.5	23.0	18.6	0.0136	23.0	12.5	0.0030	36.3

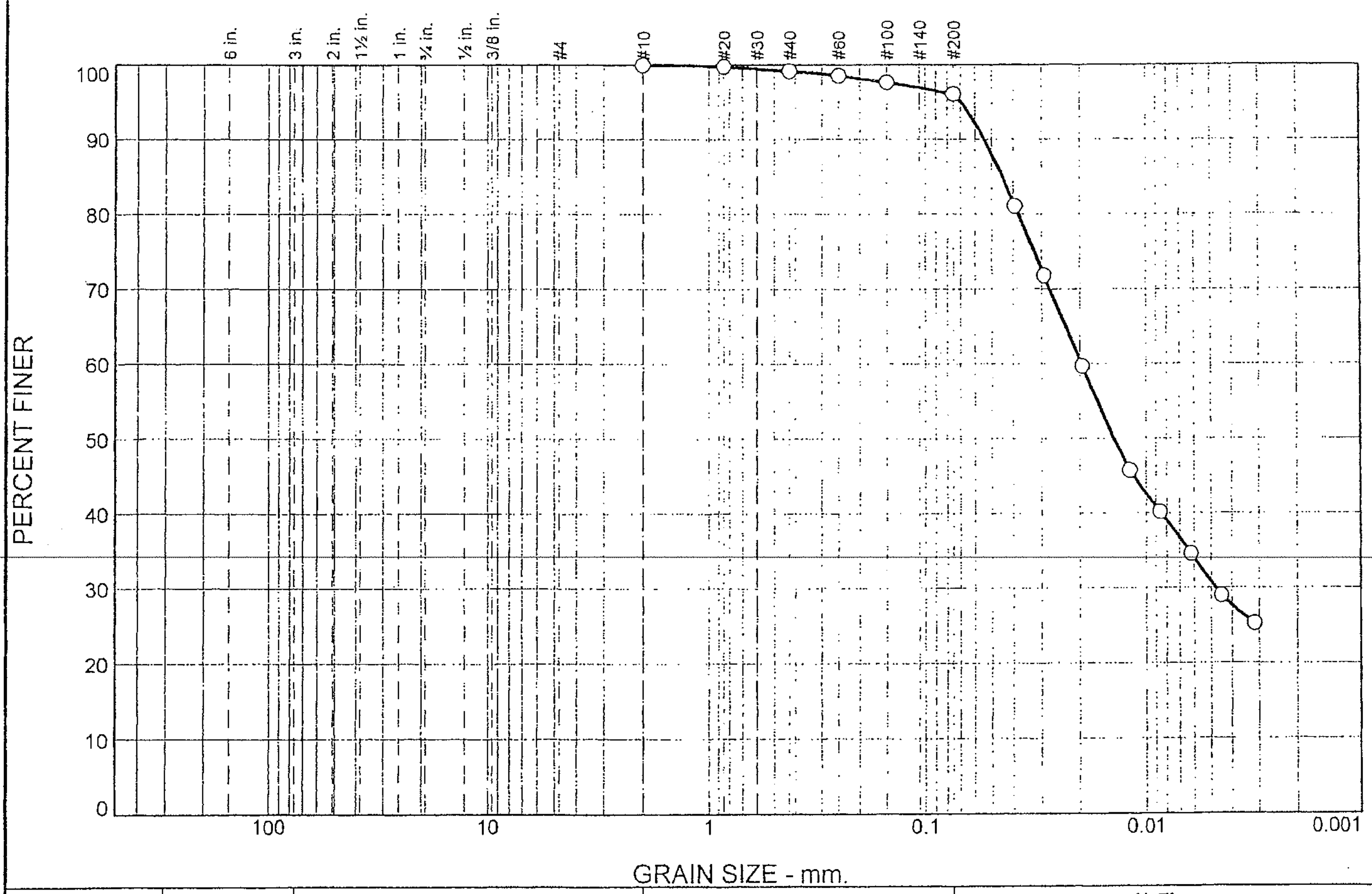
Fractional Components

Boulders	Cobbles	Gravel			Sand			Fines		
		Coarse	Fine	Total	Coarse	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.1	0.1	2.8	13.5	16.3	41.8	41.8	83.6

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
				0.0090	0.0169	0.0518	0.0895	0.1578	0.2796

Fineness Modulus
0.18

Particle Size Distribution Report



% Boulders	% +3"	% Gravel		% Sand		% Fines	
		Coarse	Fine	Coarse	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.8	3.1	65.4	30.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.8		
#40	99.2		
#60	98.5		
#100	97.7		
#200	96.1		

* (no specification provided)

Material Description

orange / brown Silt

Atterberg Limits

PL= 23 LL= 24 PI= 1

Coefficients

D_{85}= 0.0449 D_{60}= 0.0197 D_{50}= 0.0141
D_{30}= 0.0048 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

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

Sample Number: 0128 **Depth:** 6.0' - 7.5'
Location: B-001-0-08 (S-3)

Date: 2/21/08

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-001-0-08 (S-3)

Depth: 6.0' - 7.5' **Sample Number:** 0128

Material Description: orange / brown Silt

Date: 2/21/08 **PL:** 23 **LL:** 24 **PI:** 1

USCS Classification: ML **AASHTO Classification:** A-4(0)

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

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

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
249.98	14.53	0.00	#10	0.00	100.0
54.33	0.00	0.00	#20	0.13	99.8
			#40	0.46	99.2
			#60	0.81	98.5
			#100	1.27	97.7
			#200	2.14	96.1

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 100.0
Weight of hydrometer sample =54.334
Hygroscopic moisture correction:
Moist weight and tare = 49.24
Dry weight and tare = 49.08
Tare weight = 35.18
Hygroscopic moisture = 1.1%
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	48.0	43.6	0.0136	48.0	8.4	0.0394	81.1
2.00	20.5	43.0	38.6	0.0136	43.0	9.2	0.0292	71.8
5.00	20.5	36.5	32.1	0.0136	36.5	10.3	0.0195	59.7
15.00	20.5	29.0	24.6	0.0136	29.0	11.5	0.0119	45.7
30.00	20.5	26.0	21.6	0.0136	26.0	12.0	0.0086	40.1
60.00	20.5	23.0	18.6	0.0136	23.0	12.5	0.0062	34.6
120.00	20.5	20.0	15.6	0.0136	20.0	13.0	0.0045	29.0
250.00	20.5	18.0	13.6	0.0136	18.0	13.3	0.0031	25.2

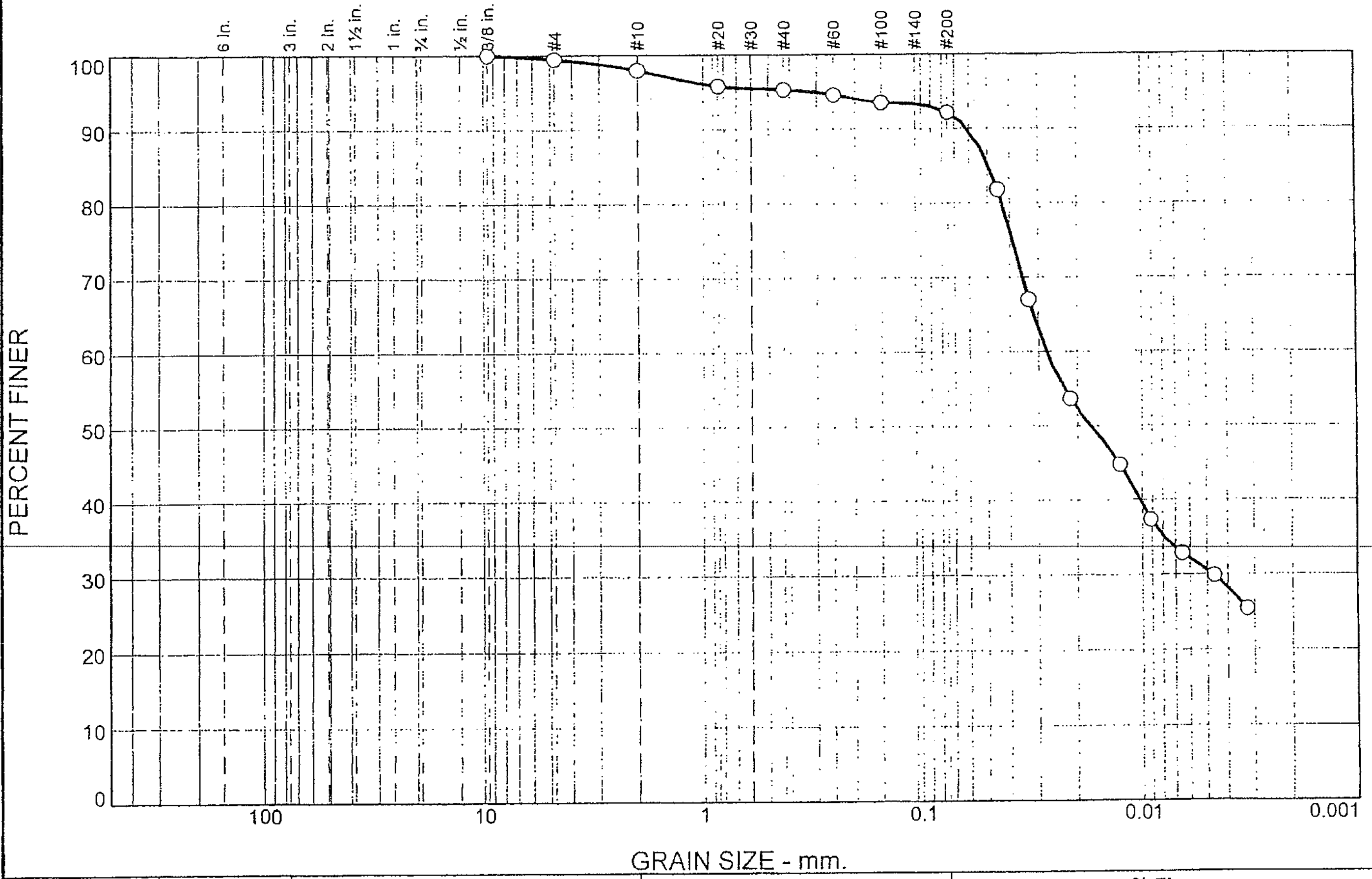
Fractional Components

Boulders	Cobbles	Gravel			Sand			Fines		
		Coarse	Fine	Total	Coarse	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	0.0	0.8	3.1	3.9	65.4	30.7	96.1

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0048	0.0141	0.0197	0.0380	0.0449	0.0542	0.0694

Fineness Modulus
0.04

Particle Size Distribution Report



% Boulders	% +3"	% Gravel		% Sand		% Fines	
		Coarse	Fine	Coarse	Fine	Silt	Clay
0.0	0.0	0.0	2.0	2.7	3.1	61.6	30.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	99.4		
#10	98.0		
#20	95.8		
#40	95.3		
#60	94.6		
#100	93.5		
#200	92.2		

* (no specification provided)

Material Description

brown Silt

Atterberg Limits

PL= 19 LL= 26 PI= 7

Coefficients

D_{85}= 0.0492 D_{60}= 0.0276 D_{50}= 0.0174
D_{30}= 0.0047 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

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

Sample Number: 0128 **Depth:** 1.0' - 2.5'
Location: B-002-0-08 (S-1)

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-002-0-08 (S-1)
Depth: 1.0' - 2.5' **Sample Number:** 0128
Material Description: brown Silt
Date: 2/14/08 **PL:** 19 **LL:** 26 **PI:** 7
USCS Classification: CL-ML **AASHTO Classification:** A-4(5)
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
222.00	14.32	0.00	.375	0.00	100.0
			#4	1.15	99.4
			#10	4.15	98.0
33.37	0.00	0.00	#20	0.75	95.8
			#40	0.92	95.3
			#60	1.17	94.6
			#100	1.52	93.5
			#200	1.99	92.2

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 98.0
Weight of hydrometer sample =33.369
Hygroscopic moisture correction:
  Moist weight and tare = 65.92
  Dry weight and tare = 65.75
  Tare weight = 50.63
  Hygroscopic moisture = 1.1%
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	32.0	27.6	0.0136	32.0	11.0	0.0451	81.8
2.00	20.5	27.0	22.6	0.0136	27.0	11.9	0.0330	67.0
5.00	20.5	22.5	18.1	0.0136	22.5	12.6	0.0215	53.6
15.00	20.5	19.5	15.1	0.0136	19.5	13.1	0.0127	44.7
30.00	20.5	17.0	12.6	0.0136	17.0	13.5	0.0091	37.3
60.00	20.5	15.5	11.1	0.0136	15.5	13.8	0.0065	32.9
120.00	20.5	14.5	10.1	0.0136	14.5	13.9	0.0046	29.9
250.00	20.5	13.0	8.6	0.0136	13.0	14.2	0.0032	25.4

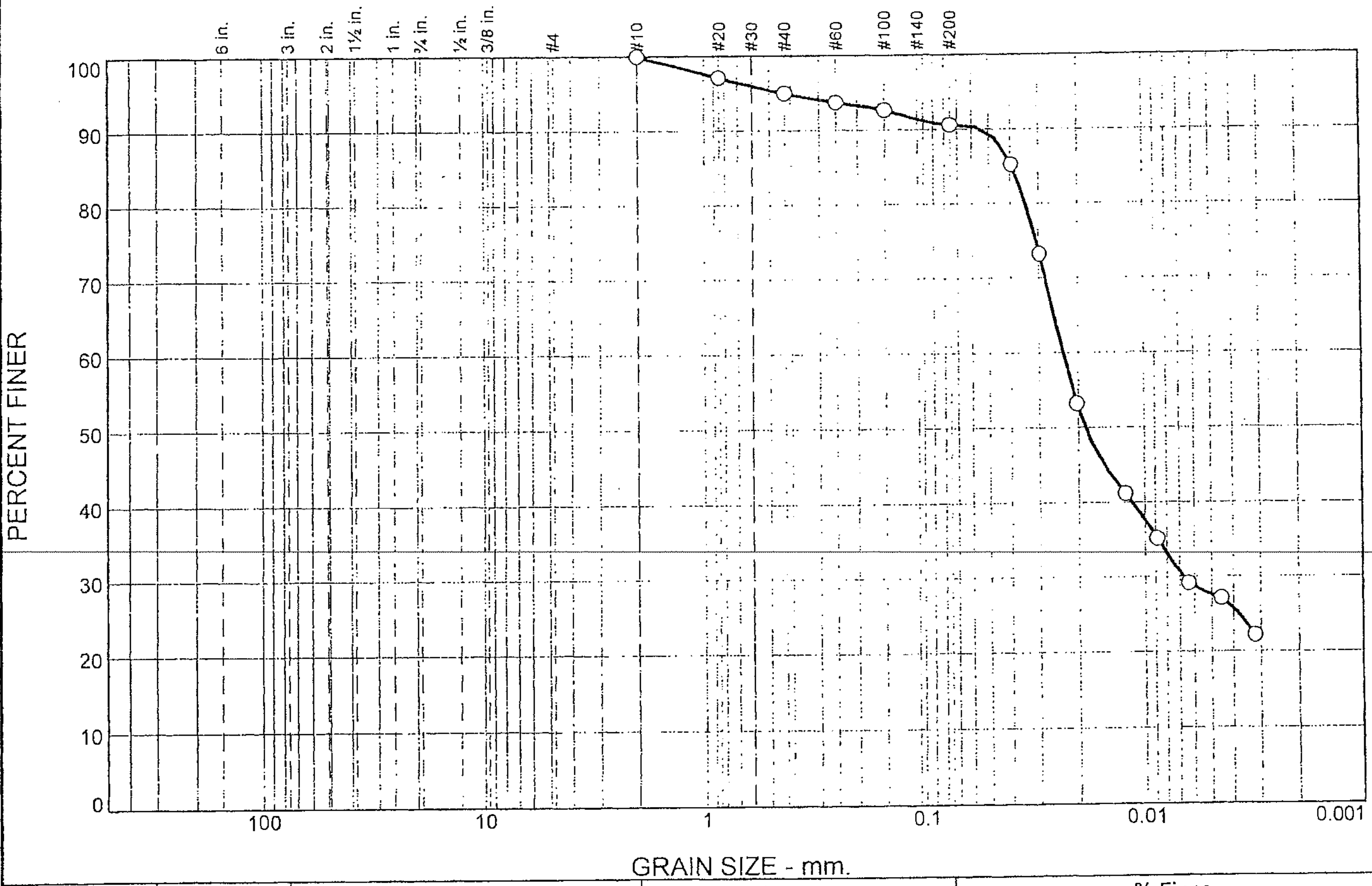
Fractional Components

Boulders	Cobbles	Gravel			Sand			Fines		
		Coarse	Fine	Total	Coarse	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	2.0	2.0	2.7	3.1	5.8	61.6	30.6	92.2

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0047	0.0174	0.0276	0.0432	0.0492	0.0611	0.3193

Fineness Modulus
0.22

Particle Size Distribution Report



% Boulders	% +3"	% Gravel		% Sand		% Fines	
		Coarse	Fine	Coarse	Fine	Silt	Clay
0.0	0.0	0.0	0.0	5.0	4.4	62.9	27.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	97.1		
#40	95.0		
#60	93.7		
#100	92.7		
#200	90.6		

* (no specification provided)

Material Description

brown Silt and Clay

Atterberg Limits

PL= 21 LL= 32 PI= 11

Coefficients

D_{85}= 0.0394 D_{60}= 0.0234 D_{50}= 0.0187
D_{30}= 0.0068 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL ODOT= A-6a(8)

Remarks

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

Sample Number: 0128 **Depth:** 6.0' - 7.5'
Location: B-002-0-08 (S-3)

Date: 2/20/08

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

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-002-0-08 (S-3)
Depth: 6.0' - 7.5' **Sample Number:** 0128
Material Description: brown Silt and Clay
Date: 2/20/08 **PL:** 21 **LL:** 32 **PI:** 11
USCS Classification: CL **AASHTO Classification:** A-6(10)
Ohio DOT Classification (if different from AASHTO): A-6a(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
231.71	14.36	0.00	#10	0.00	100.0
50.51	0.00	0.00	#20	1.45	97.1
			#40	2.53	95.0
			#60	3.16	93.7
			#100	3.71	92.7
			#200	4.75	90.6

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 100.0
Weight of hydrometer sample =50.514
Hygroscopic moisture correction:
Moist weight and tare = 51.34
Dry weight and tare = 51.15
Tare weight = 34.73
Hygroscopic moisture = 1.1%
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	47.0	42.6	0.0136	47.0	8.6	0.0397	85.2
2.00	20.5	41.0	36.6	0.0136	41.0	9.6	0.0297	73.2
5.00	20.5	31.0	26.6	0.0136	31.0	11.2	0.0203	53.2
15.00	20.5	25.0	20.6	0.0136	25.0	12.2	0.0122	41.2
30.00	20.5	22.0	17.6	0.0136	22.0	12.7	0.0088	35.2
60.00	20.5	19.0	14.6	0.0136	19.0	13.2	0.0064	29.2
120.00	20.5	18.0	13.6	0.0136	18.0	13.3	0.0045	27.2
250.00	20.5	15.5	11.1	0.0136	15.5	13.8	0.0032	22.2

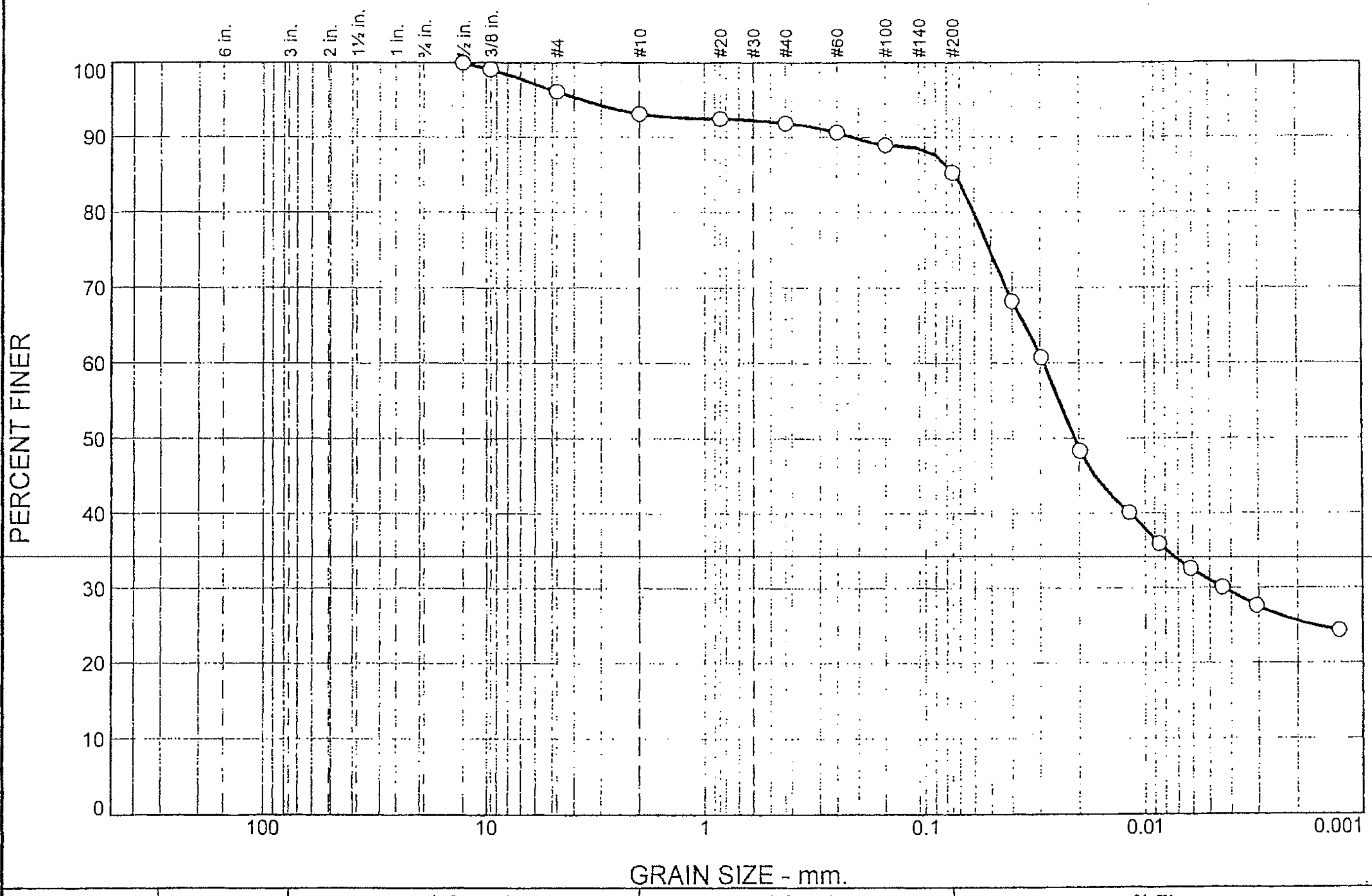
Fractional Components

Boulders	Cobbles	Gravel			Sand			Fines		
		Coarse	Fine	Total	Coarse	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	0.0	5.0	4.4	9.4	62.9	27.7	90.6

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0068	0.0187	0.0234	0.0343	0.0394	0.0537	0.4263

Fineness Modulus
0.19

Particle Size Distribution Report



% Boulders	% +3"	% Gravel		% Sand		% Fines	
		Coarse	Fine	Coarse	Fine	Silt	Clay
0.0	0.0	0.0	6.9	1.2	6.6	54.2	31.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.5	100.0		
.375	99.1		
#4	96.1		
#10	93.1		
#20	92.4		
#40	91.9		
#60	90.7		
#100	88.9		
#200	85.3		

* (no specification provided)

Material Description

brown Silty Clay

Atterberg Limits

PL= 20 LL= 36 PI= 16

Coefficients

D_{85}= 0.0740 D_{60}= 0.0287 D_{50}= 0.0210
D_{30}= 0.0043 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL ODOT= A-6b(10)

Remarks

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

Sample Number: 0128 **Depth:** 2.5' - 4.4'
Location: Shelby Tube B-002-0-08 2.5'-4.4'

Date: 2/22/08

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: Shelby Tube B-002-0-08 2.5'-4.4'
Depth: 2.5' - 4.4' **Sample Number:** 0128
Material Description: brown Silty Clay
Date: 2/22/08 **PL:** 20 **LL:** 36 **PI:** 16
USCS Classification: CL **AASHTO Classification:** A-6(13)
Ohio DOT Classification (if different from AASHTO): A-6b(10)
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
1364.90	973.32	0.00	.5	0.00	100.0
			.375	3.57	99.1
			#4	15.12	96.1
			#10	26.88	93.1
56.99	0.00	0.00	#20	0.42	92.4
			#40	0.78	91.9
			#60	1.52	90.7
			#100	2.59	88.9
			#200	4.82	85.3

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 93.1
Weight of hydrometer sample =56.992
Hygroscopic moisture correction:
  Moist weight and tare = 96.34
  Dry weight and tare = 95.71
  Tare weight = 49.68
  Hygroscopic moisture = 1.4%
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	21.0	45.5	41.2	0.0135	45.5	8.8	0.0401	68.2
2.00	21.0	41.0	36.7	0.0135	41.0	9.6	0.0295	60.8
5.00	21.0	33.5	29.2	0.0135	33.5	10.8	0.0198	48.3
15.00	21.0	28.5	24.2	0.0135	28.5	11.6	0.0119	40.0
30.00	21.0	26.0	21.7	0.0135	26.0	12.0	0.0085	35.9
60.00	21.0	24.0	19.7	0.0135	24.0	12.4	0.0061	32.6
120.00	21.0	22.5	18.2	0.0135	22.5	12.6	0.0044	30.1
250.00	21.0	21.0	16.7	0.0135	21.0	12.9	0.0031	27.6
1440.00	21.0	19.0	14.7	0.0135	19.0	13.2	0.0013	24.3

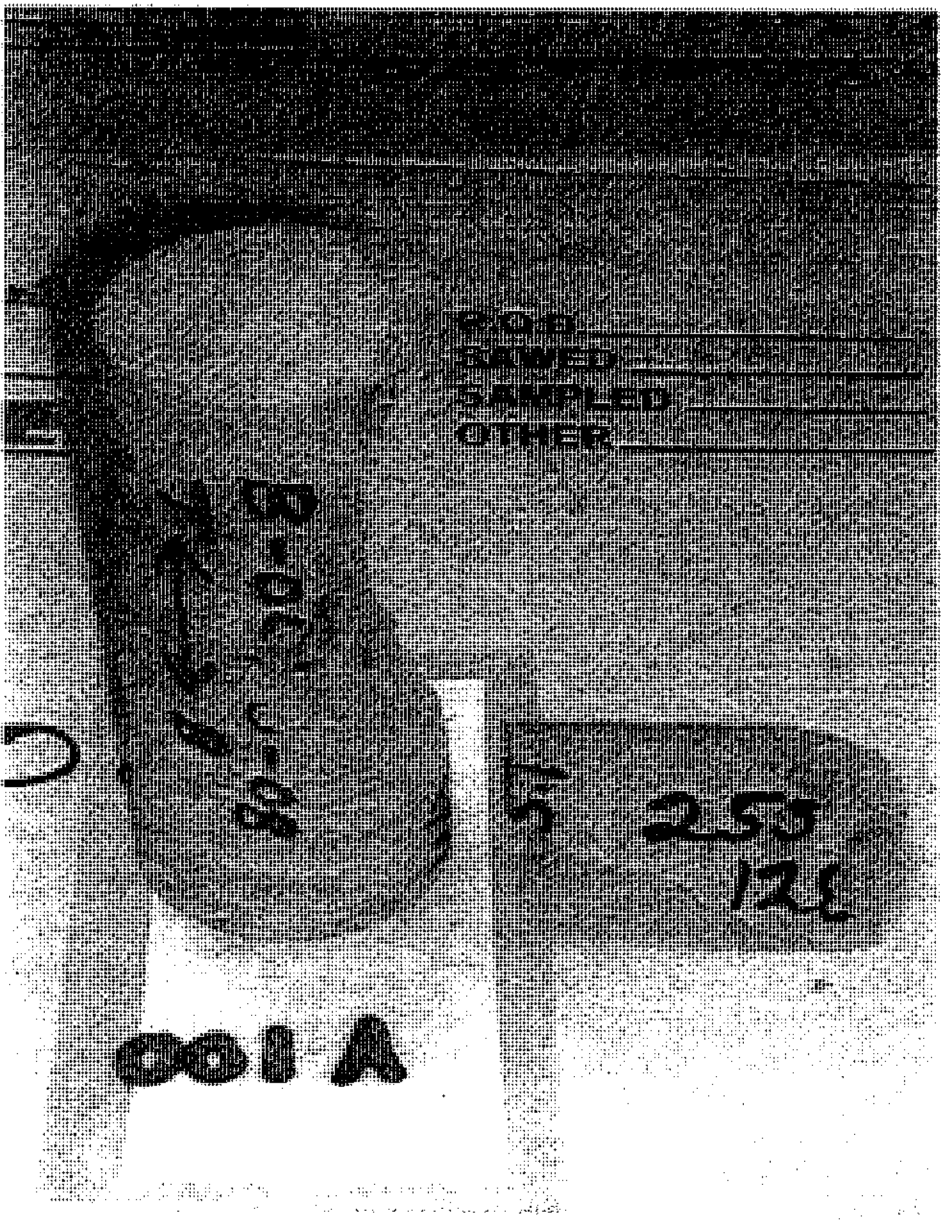
Fractional Components

Boulders	Cobbles	Gravel			Sand			Fines		
		Coarse	Fine	Total	Coarse	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	6.9	6.9	1.2	6.6	7.8	54.2	31.1	85.3

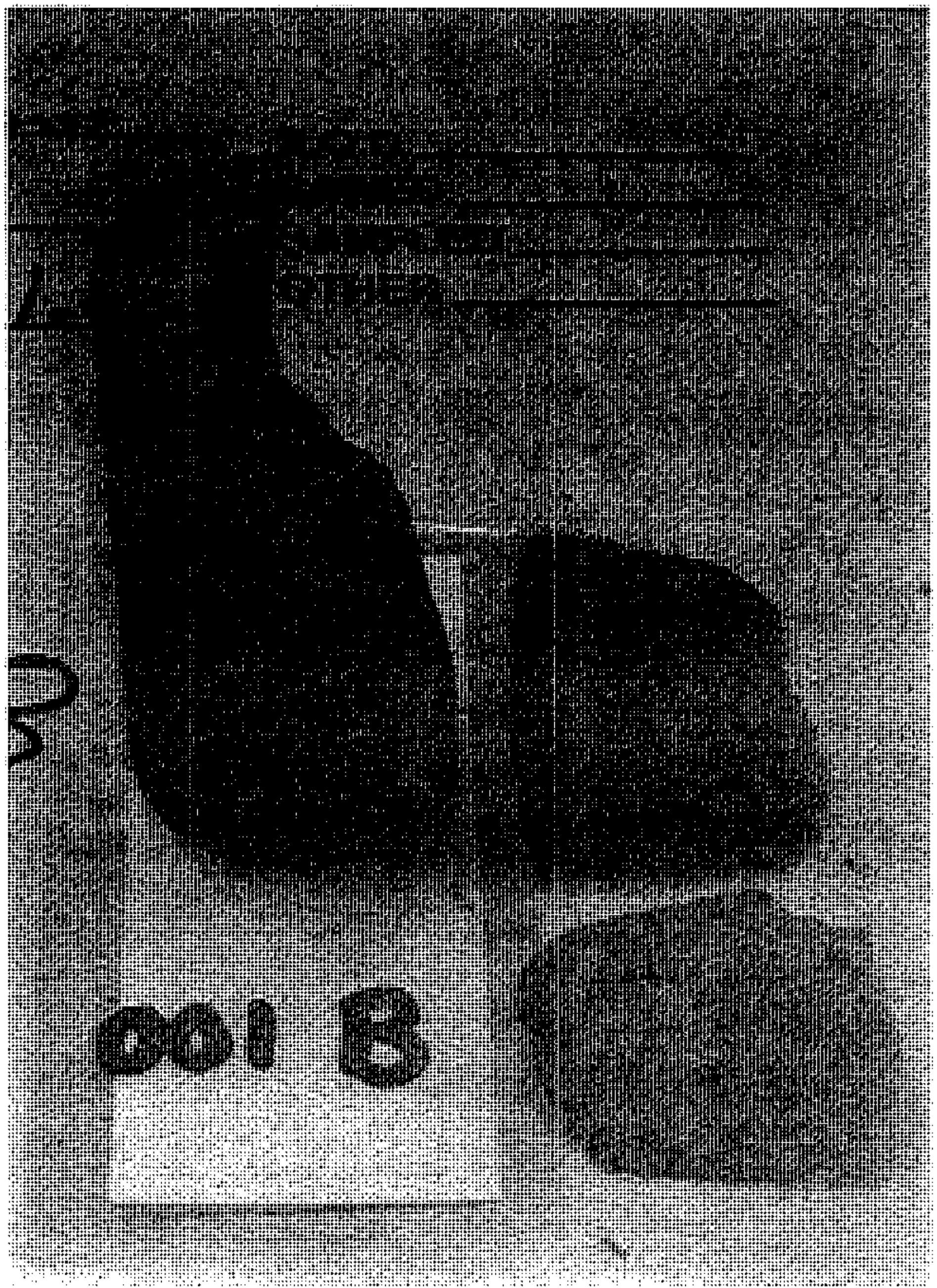
D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0043	0.0210	0.0287	0.0601	0.0740	0.2119	3.6322

Fineness Modulus
0.46

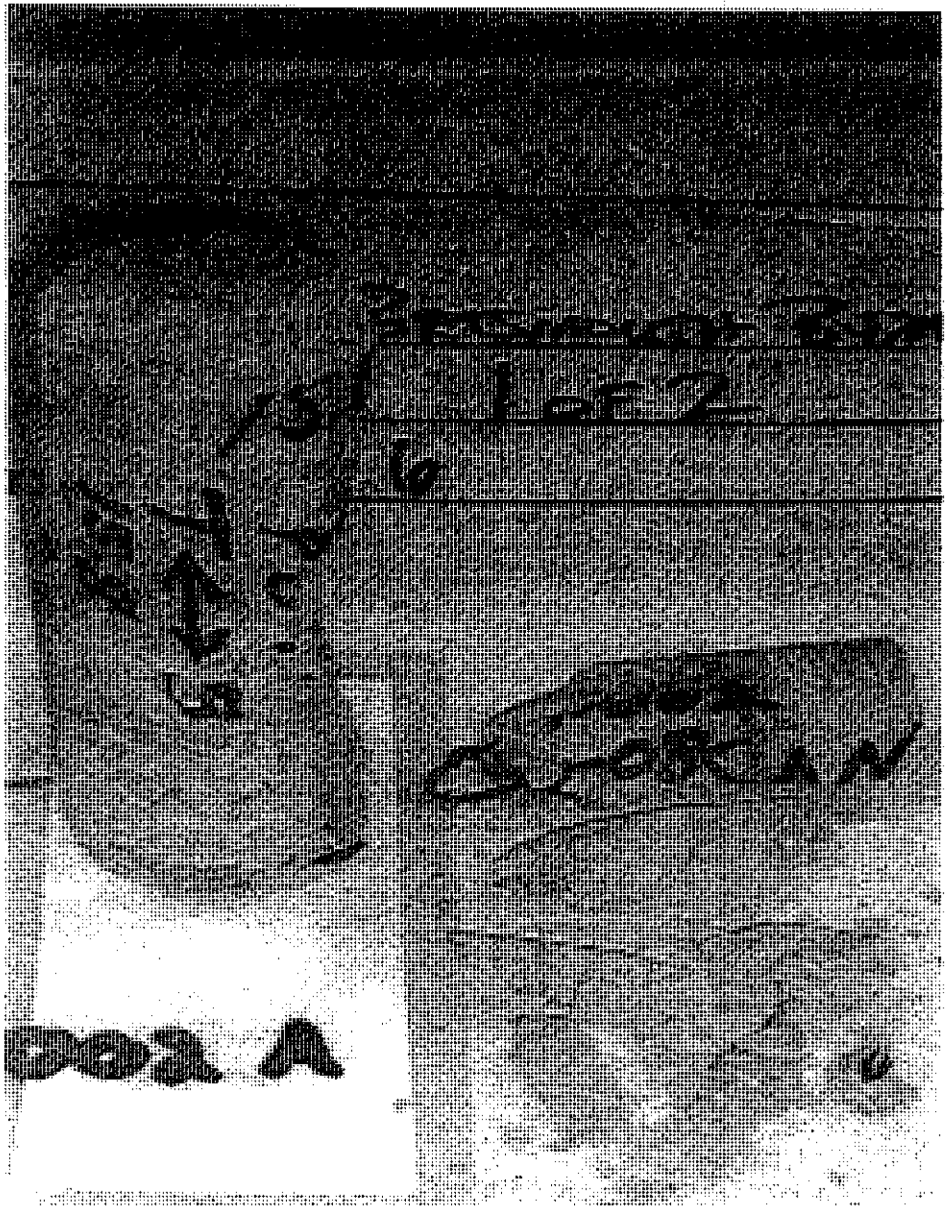
TT 25506 ODOT DISTRICT 9 PORTSMOUTH BYPASS MINFORD OH



B-001-0-08 (13.1'-13.8') A



B-001-0-08 (13.1'-13.8') B



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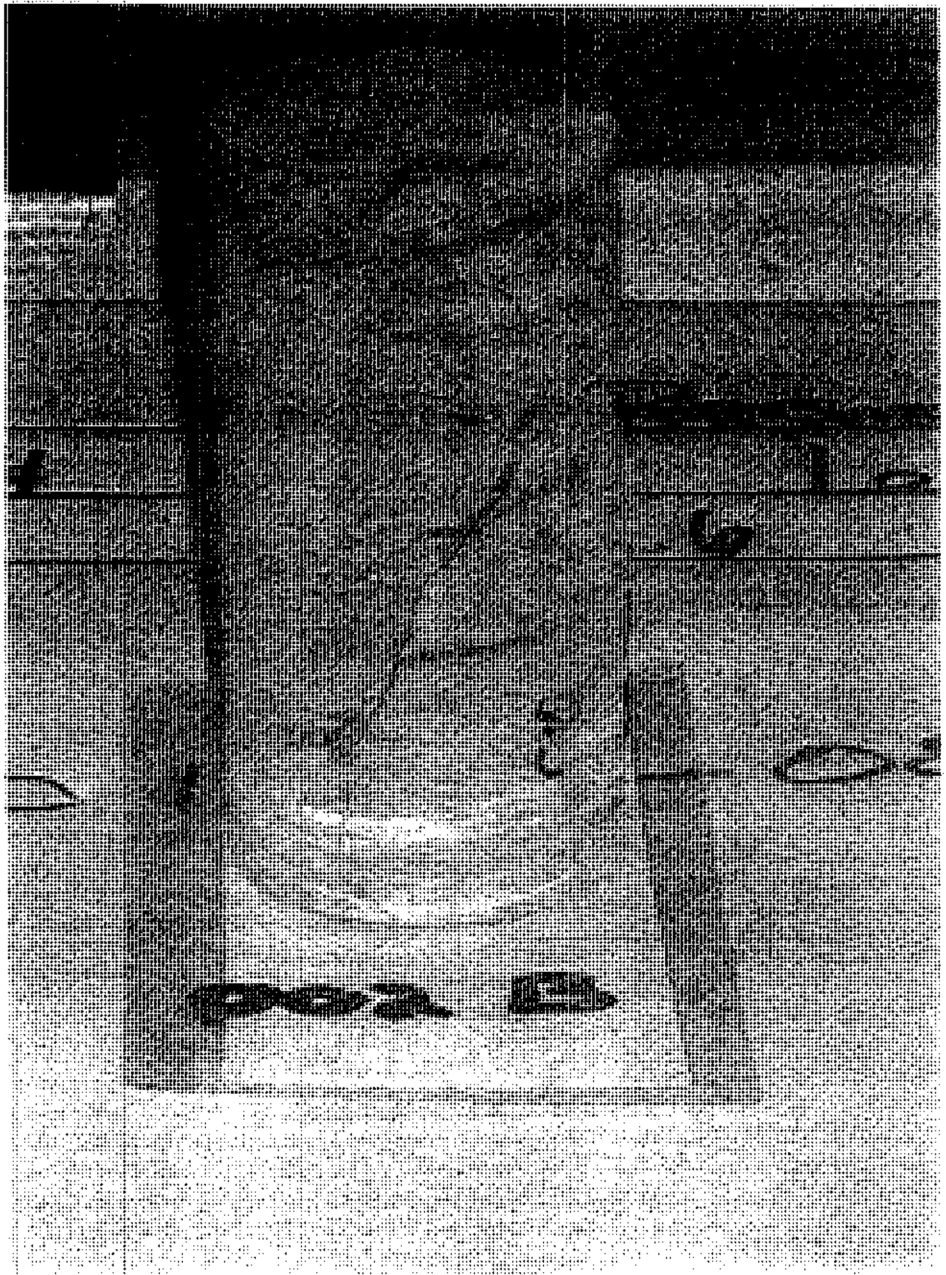
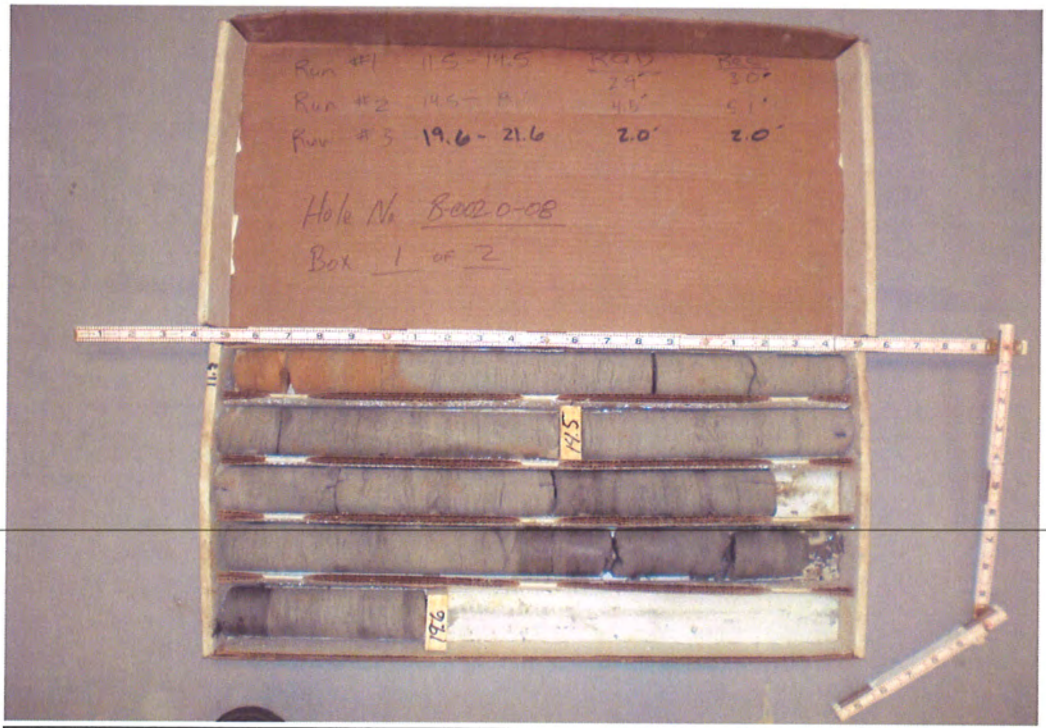


B-002-0-08 (15.8'-16.5') B

Appendix D
Supplemental Boring Logs and Core Photos



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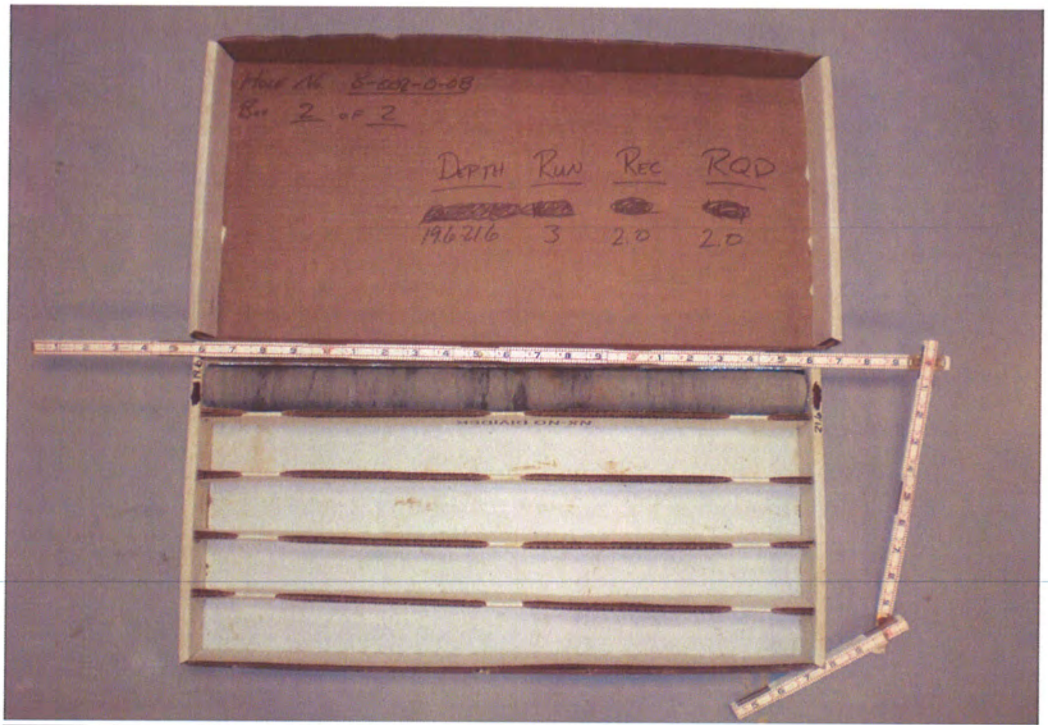


B-002-0-08 - Box 2 of 2

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 Swauger Valley-Minford Road (CR-31)	Checked:	JSA	Date:	11-Apr-08
Task:	Spread Footing - Piers	Page:		of	
Job #:	45878	No:			

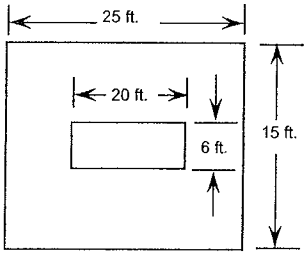
Objective: Determine Allowable Bearing Capacity

Reference: AASHTO 17th Edition (2002), Section 4.4.8.1
Report of Subsurface Exploration: Bridge and MSE Retaining Walls, SR 823 over Swauger Valley-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 feet

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-5, B-6, B-7, B-8 and B-9

	Strata	Top El.		Bottom El.	REC%avg	RQD%avg	Strength	
		(feet)		(feet)			(tsf)	(psi)
B-5	SNDST	642.5	to	632.5	95%	83%	603.5	8,382
B-6	SNDST	629.4	to	609.4	98%	85%	966.1	13,418
B-7	SNDST	655.5	to	645.5	81%	76%	573.6	7,966
B-8	SNDST	630.9	to	610.9	91%	80%	791.8	10,997
B-9	SNDST	643.5	to	633.5	98%	66%	587.0	8,153
						Average	704.4	9783.2

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

Compressive Strength of Bearing Strata, C_o = 573.6 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: $q_{ult} = N_{ms}\ C_o$ = 88.2 tsf = 176.4 ksf = 176,413 psf
[AASHTO 4.4.8.1.2-1]

=> $q_{all} = q_{ult}/FS$ = 29.40 tsf = 58.80 ksf = 58,804 psf

SAY 29 TSF

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

29.40 tsf < 573.6 tsf OK

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

29.40 tsf < 86.40 tsf OK

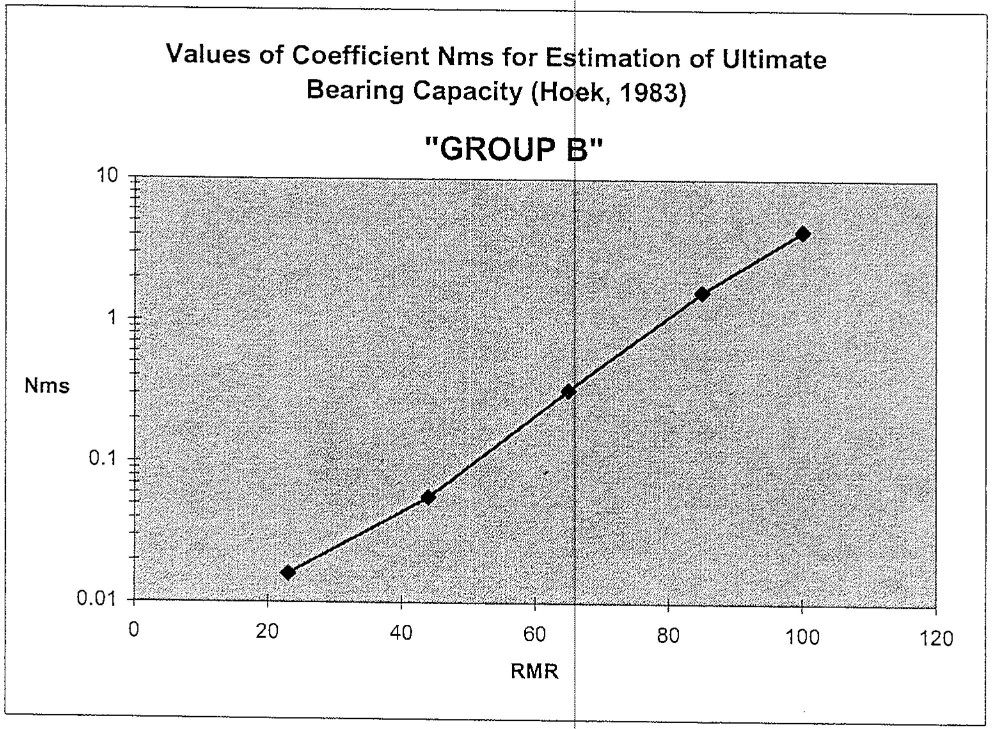
SCI-823-0837 RMR
Table 4.4.8.1.2A

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



4/16/2008

SCI-823-0837 RMR
Input Data Sheet

Sheet 1 of 1

Geomechanics Classification of Jointed Rock Masses, Tables B-2 & B-3

General Rock Parameters	B-5	B-6	B-7	B-8	B-9		Value				Rating
							English		Metric		
Surface Elevation	644.00	635.90	658.00	638.40	647.50		644.76	feet	196.52	m	
Depth to Rock @ Bottom of Footing	1.50	6.00	2.50	7.50	4.00		4.30	feet	1.31	m	
Layer Thickness (feet)	10.00	20.00	10.00	20.00	10.00		15.71	feet	4.79	m	
Point-load Strength Index (psi)							0	psi	0.00	MPa	
Uniaxial Compressive Strength, Co (psi)	8,382	13,418	7,966	10,997	8,153		10,476	psi	72.25	MPa	7
Rock Quallity, RQD	83%	85%	76%	80%	66%		79%		79%		15
Spacing of Discontinuities (inch)	4.00	12.00	2.00	2.00	2.00		5.14	inch	130.63	mm	7
Condition of Discontinuities (rough/weathered)	Moderately to Highly Weathered										
Discontinuity Separation (inch)							0.00	Inch	0.00	mm	21
Ground water											7
Strike & Dip											-2
	SNDST	SNDST	SNDST	SNDST	SNDST			Fair Rock			55
				Weighted Average of individual strata (see below) =							56

← use

Specific Rock Strata Parameters	B-5						Value				Rating
							English		Metric		
Surface Elevation	644.00						644.00	feet	196.29	m	
Depth to Top of Layer (feet)	1.50						1.50	feet	0.46	m	
Layer Thickness (feet)	10.00						10.00	feet	3.05	m	
Point-load Strength Index (psi)	0						0	psi	0.00	MPa	
Uniaxial Compressive Strength, Co (psi)	8382						8,382	psi	57.81	MPa	6
Rock Quallity, RQD	83%						83%		83%		16
Spacing of Discontinuities (inch)	4.00						4.00	inch	101.60	mm	6
Condition of Discontinuities (rough/weathered)	Moderately to Highly Weathered										
Discontinuity Separation (inch)	0.00						0.00	Inch	0.00	mm	20
Ground water											7
Strike & Dip											-2
	SNDST							Fair Rock			53

Specific Rock Strata Parameters		B-6					Value				Rating
							English		Metric		
Surface Elevation		635.90					635.90	feet	193.82		
Depth to Top of Layer (feet)		6.00					6.00	feet	1.83		
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)		13418					13,418	psi	92.54	MPa	9
Rock Quallity, RQD		85%					85%		85%		17
Spacing of Discontinuities (inch)		12.00					12.00	inch	304.80	mm	9
Condition of Discontinuities (rough/weathered)		Moderately Weathered									
Discontinuity Separation (inch)		0.00					0.00	Inch	0.00	mm	25
Ground water											7
Strike & Dip											-2
		SNDST						Good Rock			65

Specific Rock Strata Parameters			B-7				Value				Rating
							English		Metric		
Surface Elevation			658				658.00	feet	200.56	m	
Depth to Top of Layer (feet)			2.5				2.50	feet	0.76	m	
Layer Thickness (feet)			10				10.00	feet	3.05	m	
Point-load Strength Index (psi)			0				0	psi	0.00	MPa	
Uniaxial Compressive Strength, Co (psi)			7966				7,966	psi	54.94	MPa	6
Rock Quallity, RQD			76%				76%		76%		15
Spacing of Discontinuities (inch)			2.00				2.00	inch	50.80	mm	6
Condition of Discontinuities (rough/weathered)			Moderately to Highly Weathered								
Discontinuity Separation (inch)			0				0.00	Inch	0.00	mm	20
Ground water											7
Strike & Dip											-2
			SNDST					Fair Rock			52

Specific Rock Strata Parameters				B-8			Value				Rating
							English		Metric		
Surface Elevation				638.4			638.40	feet	194.58	m	
Depth to Top of Layer (feet)				7.5			7.50	feet	2.29	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)				10997			10,997	psi	75.84	MPa	7
Rock Quallity, RQD				80%			80%		80%		16
Spacing of Discontinuities (inch)				2			2.00	inch	50.80	mm	6
Condition of Discontinuities (rough/weathered)	Moderately to Highly Weathered										
Discontinuity Separation (inch)				0			0.00	Inch	0.00	mm	20
Ground water											7
Strike & Dip											-2
				SNDST				Fair Rock			54

Specific Rock Strata Parameters					B-9		Value				Rating
							English		Metric		
Surface Elevation					647.5		647.50				
Depth to Top of Layer (feet)					4		4.00				
Layer Thickness (feet)					10		10.00	feet	3.05	m	
Point-load Strength Index (psi)					0		0	psi	0.00	MPa	
Uniaxial Compressive Strength, Co (psi)					8153		8,153.00	psi	56.23	MPa	6
Rock Quallity, RQD					66%		66%		66%		13
Spacing of Discontinuities (inch)					2		2.00	inch	50.80	mm	6
Condition of Discontinuities (rough/weathered)		Moderately to Highly Weathered									
Discontinuity Separation (inch)					0		0.00	Inch	0.00	mm	20
Ground water											7
Strike & Dip											-2
					SNDST			Fair Rock			50

TABLE B-2. Geomechanics Classification of Jointed Rock Masses

(a) Classification Parameters and their ratings

	PARAMETER		RANGES OF VALUES				
1	Strength of intact rock material	Point-load strength index	>10 MPa	4–10 MPa	2–4 MPa	1–2 MPa	For this low range uniaxial compressive test is preferred
		Uniaxial compressive strength	>250 MPa	100–250 MPa	50–100 MPa	25–50 MPa	5–25 MPa 1–5 MPa <1 MPa
	Rating		15	12	7	4	2 1 0
2	Drill core quality ROD		90–100%	75–90%	50–75%	25–50%	<25%
	Rating		20	17	13	8	3
3	Spacing of discontinuities		>2 m	0.6–2 m	200–600 mm	60–200 mm	<60 mm
	Rating		20	15	10	8	5
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation <1 mm Highly weathered walls	Slickensided surfaces OR Gouge < 5 mm thick OR Separation 1–5 mm Continuous	Soft gauge > 5 mm thick OR Separation > 5 mm Continuous
	Rating		30	25	20	10	0
5	Ground water	Inflow per 10 m tunnel length	None	< 10 L/min	10–25 L/min	25–125 L/min	> 125
			OR	OR	OR	OR	OR
		Ratio $\frac{\textit{Joint water pressure}}{\textit{major principal stress}}$	0	0.0–0.1	0.1–0.2	0.2–0.5	>0.5
			OR	OR	OR	OR	OR
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	0

(b) Rating Adjustment for Joint Orientations

Strike and dip orientations of joints		Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Ratings	Tunnels	0	−2	−5	−10	−12
	Foundations	0	−2	−7	−15	−25
	Slopes	0	−5	−25	−50	−80

(c) Rock Mass Classes Determined from Total Ratings

Rating	100←81	80←61	60←41	40←21	<20
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

(d) Meaning of Rock Mass Classes

Class number	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of rock mass	>400 kPa	300–400 kPa	200–300 kPa	100–200 kPa	< 100 kPa
Friction angle of rock mass	>45°	35–45°	25–35°	15–25°	<15°

CHART A Ratings for Strength of Intact Rock

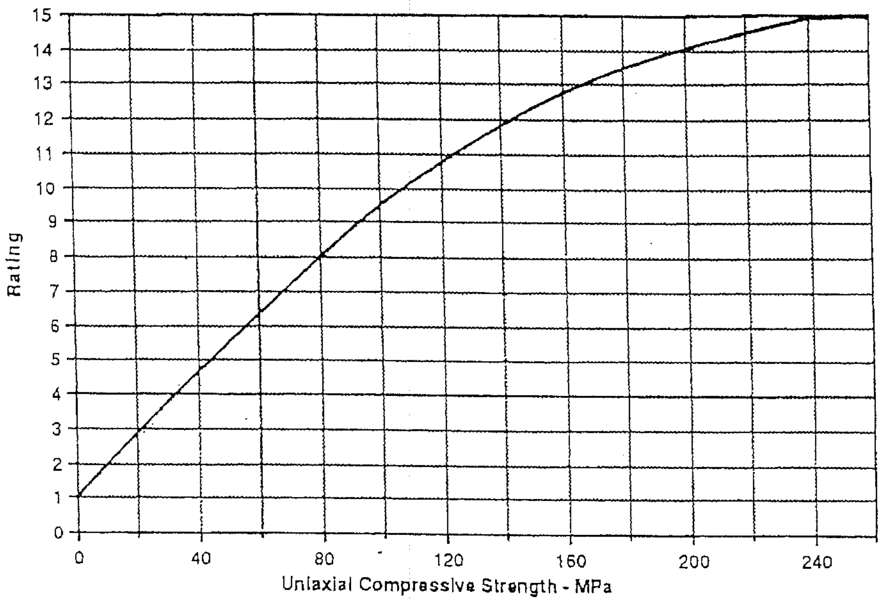


CHART B Ratings for RQD

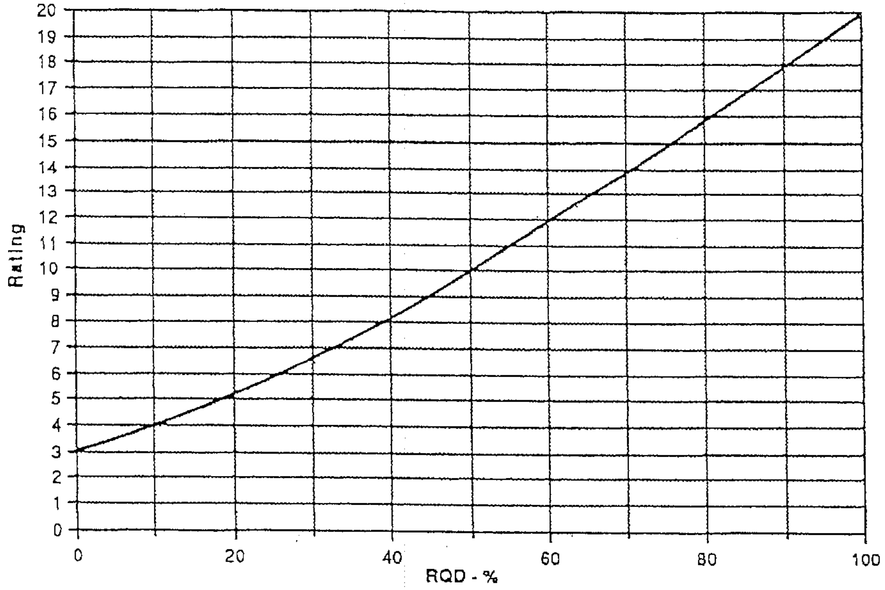


CHART C Ratings for Discontinuity Spacing

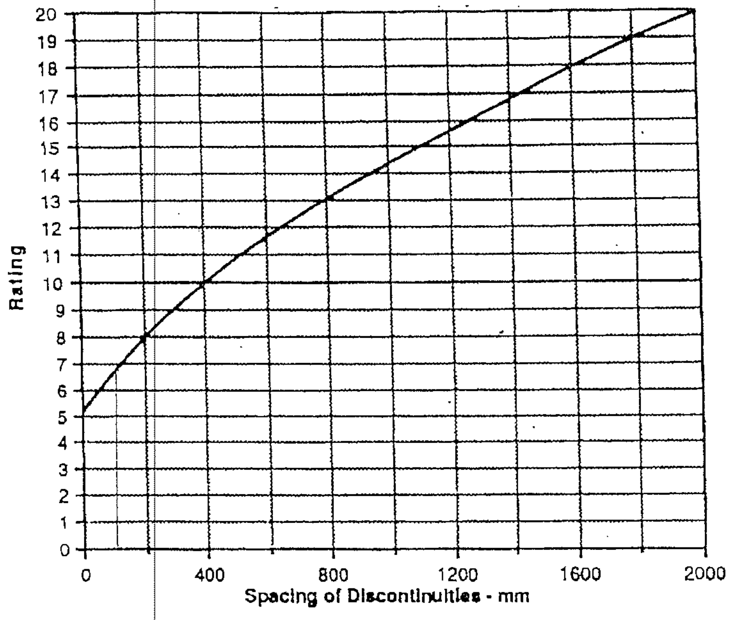
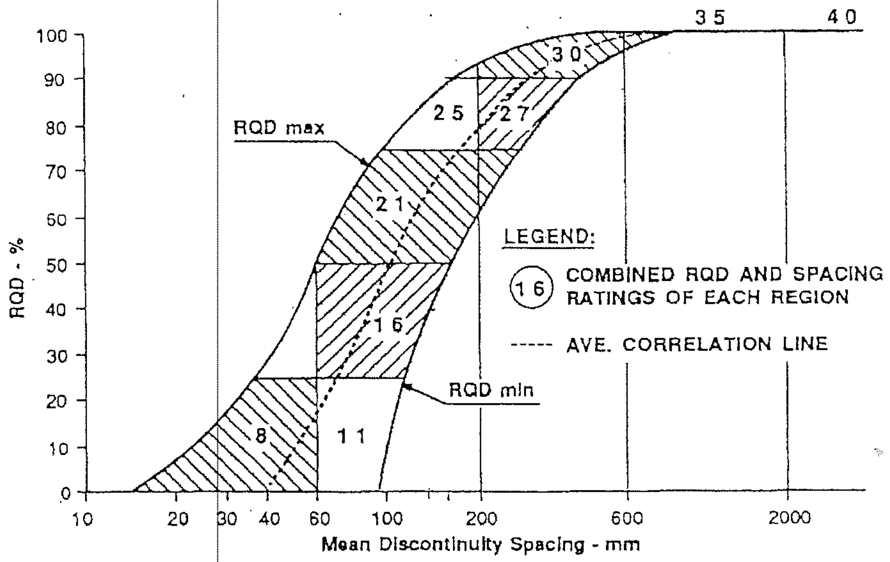


CHART D Chart for Correlation between RQD and Discontinuity Spacing



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. 12"	B-6
Moderately Fractured	4 in. to 12 in. 4"	B-5
Fractured	2 in. to 4 in.	
Highly Fractured	Less than 2 in. 2"	B-7, B-8, B-9

Condition of Discontinuities

B-5 moderately to highly weathered

B-6 moderately weathered 22.5

B-7 moderately to highly weathered

B-8 moderately to highly weathered

B-9 moderately to highly weathered 20

Weighted Average = 20.71

Use 21

Groundwater

- assume wet due to proximity of Harrison Furnace Creek

Strike and dip orientation

- assume favorable - relatively flat [illegible] 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.

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

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

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HDR Computation

Project SCI-823-0.00/6.81 Portsmouth Bypass Phase I
Computed JSA
Date 3/17/2008
Subject Bridge No. SCI-823-0837L
Checked
Date
Task Primary Consolidation Settlement Evaluation (Rear Abutment)
Sheet 1 Of 2

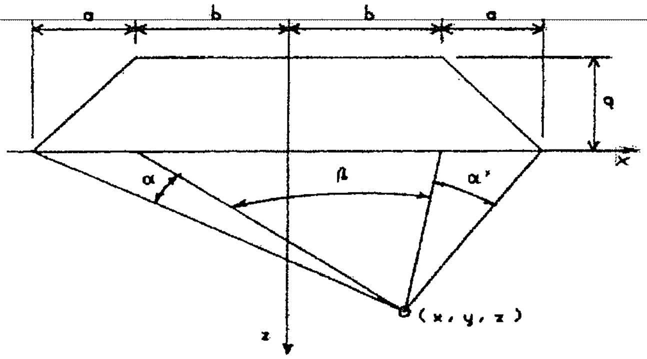
References:

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)

Assumptions:

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

Stress Due to Additional Fill (Embankment)



Groundwater Table: D = 4 ft
Embankment Height: H = 44 ft
Fill Unit Weight: γ_{emb} = 125 pcf q = 5500 psf
Surcharge: P = 240 psf
Width of Slope: a = 88 ft
Top half-width of Emb.: b = 50
Distance from CL: x = 0
Output Range: z = 0 to 11.2 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	Silty Clay (A-6a)	5.5	2.75	0.0	0.0	3.0	5433.7	240	5673.7
2	Silt & Clay (A-6b)	11.2	8.35	0.1	0.1	2.8	5298.9	240	5538.9
3									
4									
5									
6									
7									
8									
9									
10									

Note: Profile based on Boring B-002-0-08.

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

Computed JSA

Date 3/17/2008

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Date

Task Primary Consolidation Settlement Evaluation (Rear Abutment)

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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*	Silty Clay (A-6a)	5.5	122	335.5	5673.7	6009.2	6200.0	0.09	0.01	0.528	0.041
2*	Silt & Clay (A-6b)	11.2	122	917.1	5538.9	6456.0	6200.0	0.09	0.01	0.528	0.142
3											
4											
5											
6											
7											
8											
9											
10											

Note:

1) C_v, C_r, γ_{soil}, and e_0 from consolidation test with pore pressures at B-002-0-08.

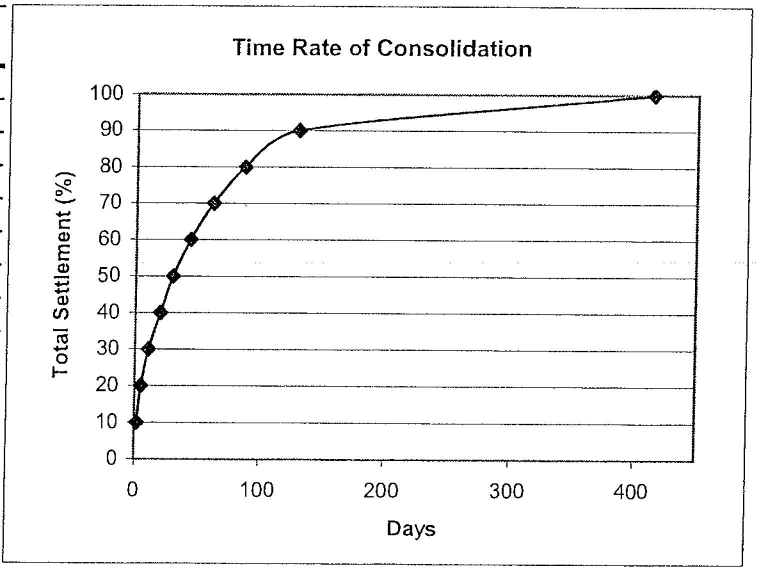
	0.183	ft
Total Settlement	2.202	in

TIME RATE OF CONSOLIDATION

U_{av}	T_v	H_{dr}	C_v (ft^2/day)	Time
10	0.01	11.2	0.821	2
20	0.03	11.2	0.821	5
30	0.07	11.2	0.821	11
40	0.13	11.2	0.821	20
50	0.20	11.2	0.821	30
60	0.29	11.2	0.821	44
70	0.40	11.2	0.821	62
80	0.57	11.2	0.821	87
90	0.85	11.2	0.821	130
99.9	2.71	11.2	0.821	415

Note:

1) C_v from Consolidation test with pore pressures at B-002-0-08.



	U_{av}		T_v		Time	
Time to reach 0.5 in settlement remaining:	77	→	0.52	→	79	days

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Date
Task Primary Consolidation Settlement Evaluation (Forward Abutment)
Sheet 1
Of 2

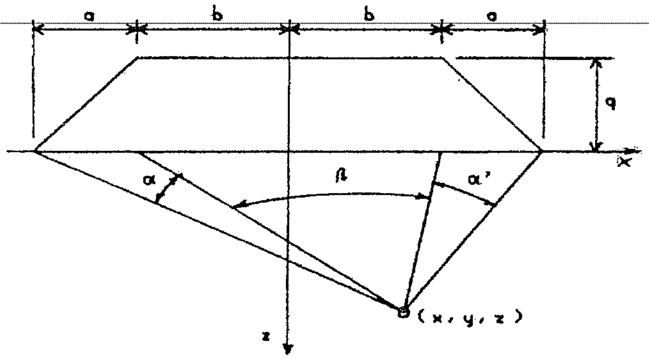
References:

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)

Assumptions:

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

Stress Due to Additional Fill (Embankment)



Groundwater Table: D = 8 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 10.1 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	Silty Clay (A-6b)	3	1.5	0.0	0.0	3.1	4715.8	240	4955.8
2	Silt (A-4b)	10.1	6.55	0.1	0.1	2.9	4600.8	240	4840.8
3									
4									
5									
6									
7									
8									
9									
10									

Note: Profile based on Boring B-001-0-08.

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Task Primary Consolidation Settlement Evaluation (Forward Abutment)

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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*	Silty Clay (A-6b)	3	122	366.0	4955.8	5321.8	6200.0	0.09	0.01	0.528	0.012
2*	Silt (A-4b)	10.1	122	799.1	4840.8	5639.9	5639.9	0.09	0.01	0.528	0.056
3											
4											
5											
6											
7											
8											
9											
10											

Note:

1) C_v, C_r, γ_{soil}, and e_0 from consolidation test with pore pressures at B-002-0-08.

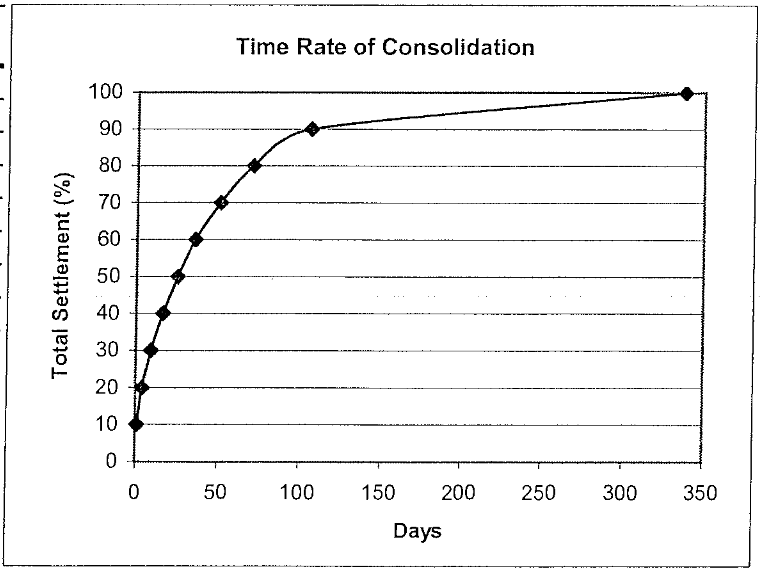
	0.069	ft
Total Settlement	0.822	in

TIME RATE OF CONSOLIDATION

U_{av}	T_v	H_{dr}	C_v (ft^2/day)	Time
10	0.01	10.1	0.821	1
20	0.03	10.1	0.821	4
30	0.07	10.1	0.821	9
40	0.13	10.1	0.821	16
50	0.20	10.1	0.821	25
60	0.29	10.1	0.821	36
70	0.40	10.1	0.821	51
80	0.57	10.1	0.821	71
90	0.85	10.1	0.821	106
99.9	2.71	10.1	0.821	338

Note:

1) C_v from Consolidation test with pore pressures at B-002-0-08.



	U_{av}		T_v		Time	
Time to reach 0.5 in settlement remaining:	39	→	0.12	→	15	days

Bridge No. SCI-823-0837-L Rear Abutment (Short-Term/Long-Term)

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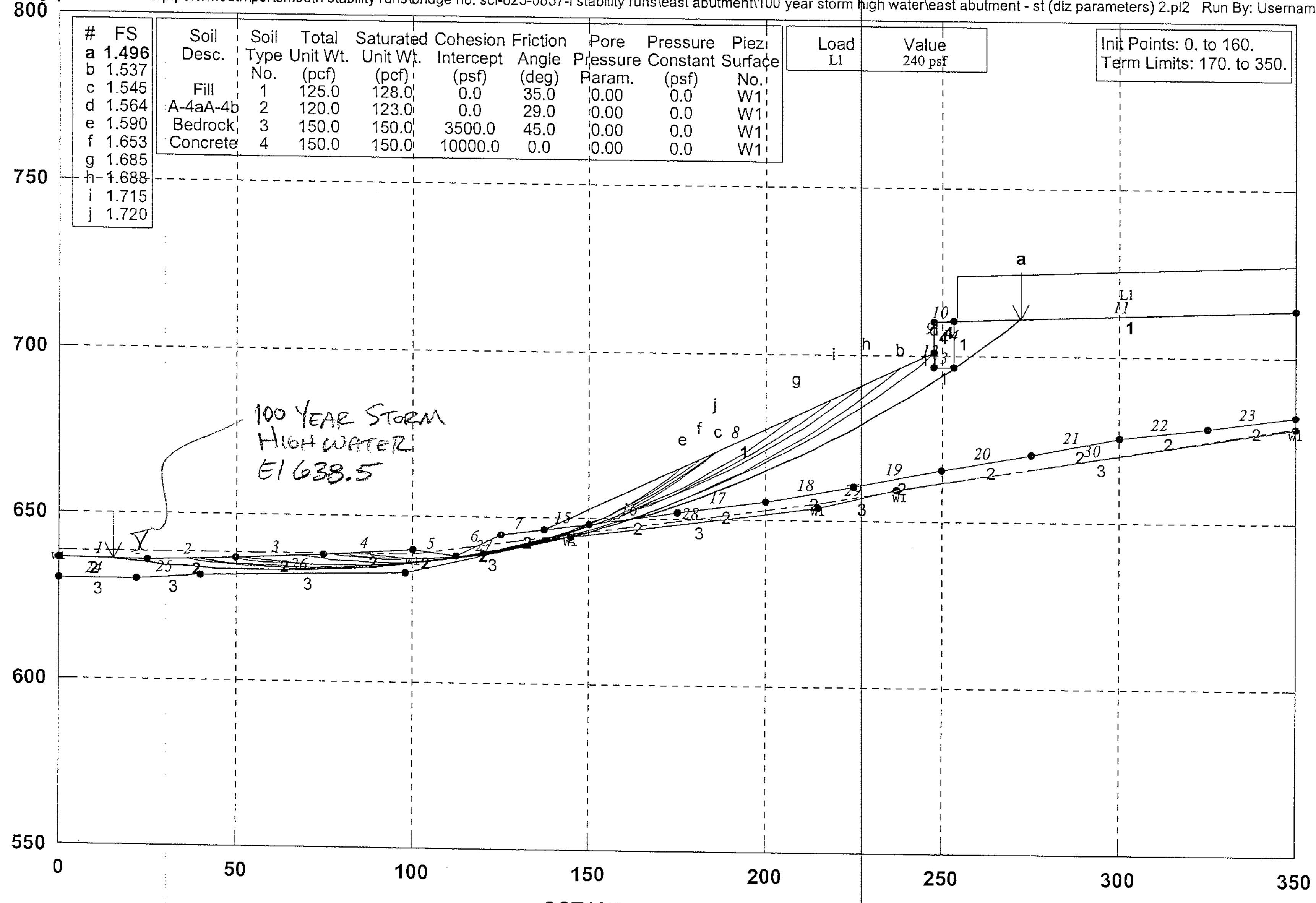

#	FS
a	1.496
b	1.537
c	1.545
d	1.564
e	1.590
f	1.653
g	1.685
h	1.688
i	1.715
j	1.720

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

Load	Value
L1	240 psf

Init Points: 0. to 160.
Term Limits: 170. to 350.

100 YEAR STORM
HIGH WATER
El 638.5

GSTABL7 v.2 FSmin=1.496
Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

Bridge No. SCI-823-0837-L Forward Abutment (Short-Term/Long-Term)

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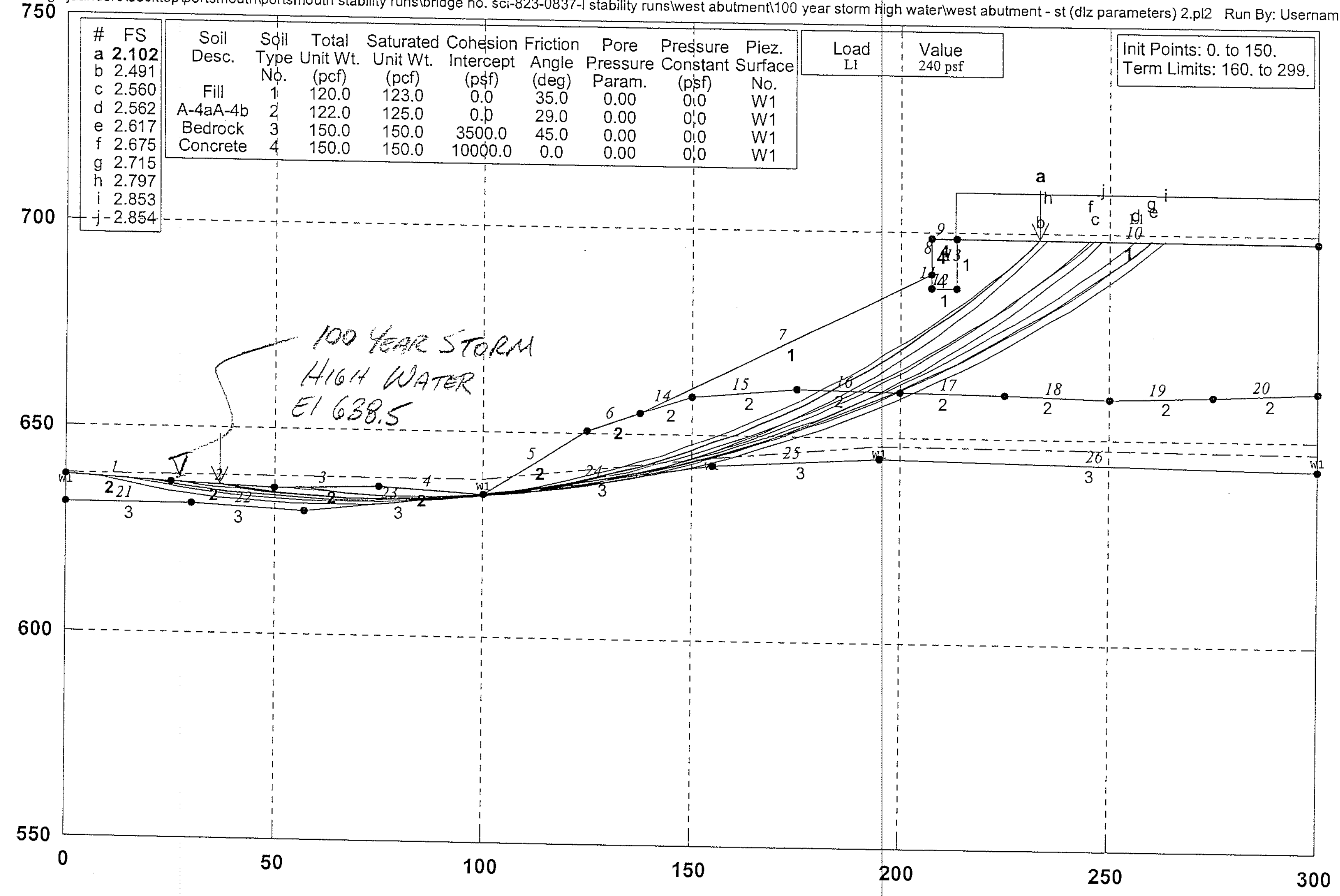

#	FS
a	**2.102**
b	2.491
c	2.560
d	2.562
e	2.617
f	2.675
g	2.715
h	2.797
i	2.853
j	2.854

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: 0. to 150.
Term Limits: 160. to 299.

100 YEAR STORM
HIGH WATER
El 638.5

GSTABL7 v.2 FSmin=2.102
Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7