Report of:

Subsurface Investigation (Bridge) For Underpass of CSX Railroad Connecting Commerce Parkway and KY 146
Oldham County, Kentucky
Item #5-434.00

Prepared for:

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September 4, 2015

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REPORT

OF

SUBSURFACE EXPLORATION (BRIDGE ONLY)

FOR

UNDERPASS OF CSX RAILROAD CONNECTING

COMMERCE PARKWAY AND KY 146

OLDHAM COUNTY, KENTUCKY

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EXECUTIVE SUMMARY

This summary should be used in conjunction with the entire report for design purposes. It should be noted that details of the subsurface exploration are not presented in this summary section and that the report must be read in its entirety for a comprehensive understanding of the items contained herein. The following solely provides a brief description of the project and a brief summary of our findings and recommendations.

The proposed project consists primarily of lowering the existing Allen Lane at the CSX railroad crossing, and relocating the existing Allen Lane slightly to the west in order to provide a grade separation at the Allen Lane/CSX railroad intersection. As part of the proposed work, a new CSX bridge will be constructed over the lowered Allen Lane, and a temporary railroad diversion track (railroad runaround) will be installed north of the existing track in order to maintain railroad traffic during the Allen Lane/CSX railroad construction. Two bridge options are being considered. One option is a single-span bridge option with a span length of approximately 103 feet with breastwall abutments. The other option is an approximately 120 feet long, 3-span bridge, with span lengths of approximately between 49 and 64.5 feet. The bottom of the piers is anticipated to be located approximately at elevation 840. The initial design anticipated the use of spread footings for the support of the abutments for the single-span bridge and the piers for the 3-span bridge, while driven piles would be used for the support of the 3-span bridge abutments. For these foundation options, a temporary retaining structure would be required to retain the temporary railroad diversion track embankment to facilitate the excavation for the proposed bridge. However, the most recent design calls for the top-down method to be used for constructing the proposed bridge in order to eliminate the need for the temporary retaining structure. It is anticipated that tangent drilled shafts and drilled shaft-supported piers will be used for the support of the proposed bridge.

A field exploration consisting of drilling thirteen borings and installing two observation wells was performed to determine the subsurface conditions at the proposed bridge location. The borings were drilled to depths of between 6.8 and 40 feet below the existing ground surface. Rock soundings and rock coring were performed at selected boring locations. In general, the existing overburden soils consisted primarily of highly plastic to the top of bedrock at depths varying from 6 to 15 feet or at elevations varying from 835.5 to 843.7 feet. The underlying bedrock generally was very fine to coarse grained, argillaceous, fossiliferous, vuggy, brown and gray dolomite interbedded with gray shale. Rock with relatively poor quality was generally encountered in the upper 5 to 11 feet of rock core. Groundwater levels were encountered at depths of between 3.7 and 10.7 feet i.e., elevations between 837.7 and 850.7 feet.

It should be noted that the configuration of the proposed bridge has not been finalized at the time of this reporting, the borings drilled in the present field exploration may not be located at the final pier and abutment locations. The subsurface conditions at the final pier and abutment locations may be different from those encountered in the borings.

The proposed bridge piers or abutments can be supported on drilled shaft foundations deriving support within the underlying dolomite from a combination of base and side resistance. The drilled shafts should be embedded a minimum of 1.5 times the shaft diameter (above bedrock elevation) below the base of RDZ elevation approximately at 835 feet. A nominal (unfactored) unit tip resistance of 150 ksf and a nominal (unfactored) unit side resistance along the socket of 10 ksf may be used to design and proportion the size of drilled shafts. Side resistance of the top two feet of rock socket as well as the portion of the shaft embedded in soils and RDZ should be neglected. Resistance factors of 0.5 and 0.55 for tip and side resistance, respectively, should be applied to the nominal unit resistance values provided above to calculate the factored resistance of the shafts. However, where the substructure is supported by one or two shafts, a 20% reduction in the resistance factors should be applied due to non-redundancy. The design of the drilled shaft foundations should also include the evaluation for lateral resistance and uplift capacity, which may control the required shaft diameter and rock socket length.

Downdrag is anticipated in the soils above the top of RDZ or bedrock on the south side of the bridge abutments. The downdrag load shall be added to the total design loads for the design of drilled shaft foundations.

A groundwater level may be assumed to be 2 feet above the soil-rock interface approximately at elevation 835 feet for design purposes. A vertical live load surcharge of 1,880 psf (8’-6” continuous strip) should be used for each set of railroad tracks based on the recommendation indicated in the appendix of the “*CSXT Public Project Information*”, last revised on August 10, 2012.

All drilled shaft shall be constructed in accordance with the in accordance with the Kentucky Transportation Cabinet (KYTC) Special Note for Drilled Shafts, current edition and the recommendations in this report.

Approximately 15 feet of fill will be required to be placed at the south side of the proposed bridge abutments. Stability analyses and settlement evaluations were performed for the fill slopes at the south side of the bridge abutments with the anticipated maximum height of new fill. Stability analyses were also performed to determine the maximum steepness of the cut section for the spill-through slopes at the bridge abutments. In the analyses, the most critical section was generally considered to be the section constructed in the thickest layers of cohesive soils. The groundwater level was assumed to be 2 feet above the soil-rock interface in the analyses, and a vertical live load railroad surcharge of 1,880 psf (8’-6” wide continuous strip) was used in the stability analyses for the short-term (undrained) condition only. Based on the results of the analyses, the side slopes of the fill slopes and the cut section for the spill-through slopes at the anticipated tangent drilled shaft bridge abutments are recommended and summarized in the following tables.

Table 1. Recommended Side Slopes and Estimated Settlements at the Anticipated Tangent Drilled Shaft Bridge Abutments

	Anticipated Maximum Amount of (Cut)/Fill, ft	Recommended Side Slope	Estimated Settlement, inch
Cut Section/ Spill-through Slope	(26)	3H:1V or flatter	Not applicable
Fill Slope	15	2H:1V or flatter	2.3

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1.0 LOCATION AND DESCRIPTION

The proposed project consists primarily of lowering the existing Allen Lane at the CSX railroad crossing, and relocating the existing Allen Lane slightly to the west in order to provide a grade separation at the Allen Lane/CSX railroad intersection. Allen Lane is generally aligned in a north/northwest and south/southeast orientation, intersecting Commerce Parkway at the south end and KY 146 at the north end. The existing CSX railroad generally runs east and west across Allen Lane at approximately 500 feet south of KY 146 (Hwy 146). As part of the proposed work, a new CSX bridge will be constructed over the lowered Allen Lane, and a temporary railroad diversion track (railroad runaround) will be installed north of the existing track in order to maintain railroad traffic during the Allen Lane/CSX railroad construction. Two bridge options are being considered. One option is a single-span bridge option with a span length of approximately 103 feet with breastwall abutments. The finished grade of the proposed Allen Lane, immediately in front of the east abutment, is anticipated to be approximately at elevation 839 feet, while the proposed finished grade immediately in front of the west abutment is approximately at elevation 837 feet. The other option is an approximately 120 feet long, 3-span bridge, with span lengths of approximately between 49 and 64.5 feet. The finished grade of the proposed Allen Lane, immediately in front of the east pier, is approximately at elevation 839 and the proposed grade immediately in front of the west pier is approximately at elevation 840. The initial design anticipated the use of spread footings for the support of the abutments for the single-span bridge and the piers for the 3-span bridge, while driven piles would be used for the support of the 3-span bridge abutments. For these foundation options, a temporary retaining structure would be required to retain the temporary railroad diversion track embankment to facilitate the excavation for the proposed bridge. However, the most recent design calls for the top-down method to be used for constructing the proposed bridge in order to eliminate the need for the temporary retaining structure. It is anticipated that tangent drilled shafts and drilled shaft-supported piers will be used for the support of the proposed bridge.

The existing grade along Allen Lane between Commerce Parkway and KY 146 will also be modified. The proposed work along Allen Lane will begin at its intersection with Commerce Parkway (Sta. 137+75.11) and end at its intersection with KY 146 (Sta. 175+13.61), and the proposed work for the temporary railroad diversion track will begin at Sta. 1515+50 and end at Sta. 1541+50. Based on the current design information, it is anticipated that construction of the new Allen Lane will mostly require grade cuts of up to approximately 26 feet and placement of fills of up to approximately 7 feet. The larger amounts of grade cuts (greater than 14 feet) are generally required to accommodate the proposed alignment shift to the west and the proposed grade separation at the Allen Lane/CSX railroad intersection, approximately between Sta. 168+00 and Sta. 174+00. For the construction of the proposed CSX railroad bridge, approximately 15 feet of fill will be required at the south side of the bridge abutments. For the construction of the railroad runaround, it is anticipated that fills of up to approximately 10 feet thick will be placed within approximately 80 to 85 feet north of the proposed bridge location.

As part of the proposed work along Allen Lane, the existing Commerce Parkway will be widened to add turn lanes, which would impact the existing bike path along the north side of Commerce Parkway. A minimal amount of road improvement, consisting primarily of some “sliver fills” of relatively low heights and thicknesses, is also planned on the south side of Commerce Parkway, within the existing right-of-way. The proposed work along Commerce Parkway will begin at approximately 250 feet west of its intersection with Allen Lane (Sta. 42+50) and end at Sta. 57+00. It is anticipated that cuts of up to approximately 12 feet in the existing embankment along the north side of Commerce Parkway will be required.

The latitude and longitude coordinates at the intersection of Allen Lane and Commerce Parkway in decimal format is 38.392575^o and -85.394302^o, respectively, while the latitude and longitude coordinates at the proposed CSX railroad bridge is approximately 38.400767^o and -85.397939^o, respectively.

This report addresses the geotechnical issues for the proposed design and construction of the proposed CSX railroad bridge. The geotechnical report for the proposed roadway portion and the railroad runaround will be prepared under separate cover.

The purpose of this exploration was to 1) explore 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 and construction of the proposed CSX railroad bridge. The exploration presented in this report was performed essentially in accordance with DLZ Kentucky, Inc.’s (DLZ) proposal for the project last revised on July 3, 2014.

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 TOPOGRAPHY AND DRAINAGE

The project area is in the Outer Bluegrass Region of north-central Kentucky. The terrain is gently rolling to hilly. According to the existing contour lines shown on the design plan, the area within the anticipated footprint of the proposed railroad bridge generally slopes from the north approximately at elevation 860 to the south approximately at elevation 845, with approximately 15 feet of vertical relief. The site is situated in the Curry’s Fork watershed, which is located in the Salt River Basin. Drainage from the project area flows into the North Curry’s Fork, which is located approximately 1,000 feet south of the intersection of Allen Lane and Commerce Parkway. The North Curry’s Fork discharges into Floyds Fork, which discharges into the Salt River, a tributary of the Ohio River.

Based on the Geologic Map of Kentucky, Oldham County is the karst prone area of the outer Bluegrass Region. However, according to Kentucky Geological Survey, the project area is located in a moderate karst potential area, and is not known for significant karst topography.

However, one documented sinkhole is located approximately 300 feet east of Allen Lane and approximately 700 feet south of Artisan Parkway. Artisan Parkway is a side road near the north end of Allen Lane, approximately 600 feet south of the existing CSX railroad crossing. According to LiDAR data, three other potential sinkholes may be located in the immediate vicinity of this documented sinkhole. However, the karst topography does not play a major role in the Curry's Fork watershed or in the transport of groundwater.

According to the Geological Quadrangle Map of La Grange prepared in 1971, a few ponded areas were located approximately within 500 feet to the east, northeast, and southwest sides of Allen Lane, and a small stream ran north/northeast and south/southwest between the ponded areas on the southwest and northeast sides of the road. However, based on aerial imagery from Google Earth, taken on September 22, 2014, the small stream and the majority of the ponded areas on the east and west sides of Allen Lane appeared to have been filled. A field reconnaissance performed on August 28 and 29, 2014 did not identify the presence of any streams in the immediate vicinity of the proposed roadway alignments or in the areas identified on the historical mapping. However, a retention pond was observed at approximately 50 feet east of existing Allen Lane and approximately 150 feet south of Artisan Parkway (approximately Sta. 161+00).

3.0 GEOLOGY

The soils in the project area tend to delineate with the drainage patterns of the Curry's Fork. According to the Soil Survey of Oldham County, Kentucky, issued in November 1977, the majority of soil contained in the project area is classified primarily as clay loam with low organic content and high clay content.

According to the Geological Quadrangle Map of La Grange (GQ-901), the proposed railroad bridge is primarily underlain by Laurel Dolomite of Silurian age. A map showing the bedrock formation at the proposed railroad bridge is included in Appendix I. The La Grange Quadrangle Map showing the project location is also included in the appendix.

The Laurel Dolomite is described as consisting of dolomite of two types. One type of dolomite, which occurs in the upper portion of the unit, consists of greenish-gray to light-olive-gray dolomite, micro-grained to very finely crystalline-grained and weathers dark yellowish-orange. The second type is described as being more massive and somewhat porous, mottled dolomite in two bedding sets separated by dark-gray to olive-gray dolomitic clay shale situated approximately five to eight feet above the base of the formation.

No faults or other detrimental geologic features are noted to be present by the referenced mapping within the immediate vicinity of the proposed railroad bridge area.

In Oldham County, water is obtained from consolidated sedimentary rocks of Ordovician, Silurian, and Devonian ages, and from unconsolidated sediments of Quaternary age. The unconsolidated Quaternary sediments have primarily been deposited along the larger streams and rivers like the North Curry's Fork located approximately 1,000 feet to the south of the intersection of Allen Lane and Commerce Parkway. Wells installed in smaller creek valleys and some broad ridges in western and central Oldham County generally can produce enough water for a domestic supply, except during dry weather. However, in upland areas of the rest of Oldham County, where the project is located, most drilled wells generally cannot produce enough water for a dependable domestic supply, unless the wells are drilled along drainage lines or in areas where thick limestone is present, in which cases wells may produce up to 100 to 500 gallons per day, except during dry weather.

4.0 FIELD EXPLORATION

4.1 Summary

DLZ prepared the scope of field exploration based on the proposed work of the project. The field exploration plan was reviewed and subsequently approved by the Kentucky Transportation Cabinet (KYTC) Geotechnical Branch.

The field exploration for the proposed railroad bridge originally consisted of drilling sixteen borings (B-28 through B-43). Because of site constraints, borings B-29, B-33, B38, and B-42 were eliminated from the boring program during the field exploration. However, installation of two observation wells at borings B-32 and B-41 were added to the field exploration program to monitor the long-term groundwater levels at the proposed bridge location. The borings were drilled between September 2 and 10, 2014; however the observation wells were installed on December 29 and 30, 2014. The borings were drilled using either a truck-mounted or an all-terrain-vehicle (ATV) mounted rotary-type drill rig with automatic hammers.

In order to avoid existing underground utilities, existing site features and/or for accessibility, the as-drilled locations of some of the borings were field offset from the originally planned locations. A sketch showing the as-drilled boring locations with respect to the existing features is presented in Appendix I. The stations and offsets and latitude and longitudes of the as-drilled boring locations are on the boring logs in Appendix I. The locations and elevations of the borings were determined in the field by representatives of DLZ. All drilling and sampling were performed in general accordance with the current KYTC Geotechnical Manual, Section GT-300. Information concerning the general drilling procedures is presented in Appendix I.

The present field investigation was undertaken to develop engineering information to meet the purposes as described in Section 1, “Location and Description,” of this report. The intent of these services was not to uncover or identify any contaminated subsurface material that may contain hazardous or flammable substances. Identification of such substances requires specialized exploration techniques and analyses that were not employed in this investigation.

A field log was prepared for each boring. These logs contain visual classifications of the materials encountered during drilling as well as an interpolation of the subsurface conditions between samples. Final logs, included in Appendix I, represent our interpretation of the field logs and may include modifications based on laboratory observations and tests of the field samples. The final logs describe the materials encountered, their thicknesses, and the locations where samples were obtained. Ground surface elevations at the borings are presented on the final logs.

4.1.1 Rock Coring Borings

Rock cores were obtained in borings B-30, B-32, B-39, and B-41. The Standard Penetration Tests were performed generally in 1.5-foot increments at intervals not exceeding 5 feet through the overburden to the top of bedrock. Rock cores were obtained to the originally planned elevations; NQ size diamond coring tools were used with water as the circulating fluid. Borings B-30, B-32, B-39, and B-41 were drilled to a depth of 40 feet each. Upon completion of the borings, the soil samples and the rock cores were transported to DLZ's office and logged by the project geologist. The geologist determined the depth of the rock disintegration zone (RDZ) for each boring, and also determined the percent recovery and rock quality designation (RQD) for each core run. It should be noted that RDZ is the subsurface materials that are composed of weathered and decomposed bedrock in accordance with KYTC Geotechnical Guidance Manual, dated June 2005.

Because of the relatively low rock recovery and low Rock Quality Designation (RQD) in the first 10 feet of rock core in boring B-32, a rock sounding boring (B-32b) was drilled offset from boring B-32 to confirm the depth to the rock disintegration zone at boring B-32.

To facilitate the Shelby tube sampling, an undisturbed soil boring was drilled offset from each of the borings B-30, B-32, B-39, and B-41. Two Shelby tube samples were taken from boring B-30 at the depths of 3 and 6 feet, and two Shelby tube samples from boring B-32 at the depths of 2 and 5 feet. One Shelby tube sample was taken from each of borings B-39 and B-41 at the depth of 3 feet and 7 feet, respectively.

4.1.2 Rockline Soundings

Rockline soundings were performed in borings B-28, B-31, B-34, B-35, B-36, B-37, B-40, and B-43 by drilling through the overburden to top of bedrock. Upon auger refusal on bedrock, the Standard Penetration Test was performed at the refusal depth to obtain a bedrock sample for visual classification. The borings were drilled to depths of between 6.8 and 15.4 feet.

4.2 Generalized Subsurface Conditions

The following sections present the generalized subsurface conditions encountered by the borings. For more detailed information, please refer to the boring logs presented in the Appendix. Please note that the strata contact lines shown on the boring logs represent approximate boundaries between soil and rock layers. In the field, the actual transitions between soil and/or bedrock layers might be different both vertically and laterally.

4.2.1 Surface Material

Topsoil, approximately 1.0 to 2.0 inches thick, was encountered in all of the borings drilled near the proposed bridge location.

4.2.2 Probable Fill Materials

Below the topsoil, probable fill material consisting primarily of non-plastic silt with sand was encountered at depths of between 0 and 3.5 feet in boring B-41.

4.2.3 Lean Clays and Fat Clays

Beneath the topsoil or probable fill, the borings, except for boring B-30, generally encountered residual soils consisting primarily of highly plastic to the top of bedrock at depths varying from 6 to 19.5 feet. Below the topsoil, lean clay was encountered in boring B-30 at depths of between 0.1 and 2.5 feet. Hand penetrometer test values in the clay soils generally varied from 1.5 to 4.5 tons per square foot (tsf), indicating stiff to hard consistency.

4.2.4 Auger Refusal

Bedrock was encountered at depths of between 6 and 19.5 feet or at elevations between 835.2 and 843.7 feet. However, the upper few feet of the underlying bedrock generally were severely weathered or broken and auger refusal was encountered several feet below the top of rock in some of the borings. Given Oldham County is the karst prone area of the outer Bluegrass Region, the underlying bedrock could be pinnacled with crevices, mounds or valleys and such conditions might be present within the project areas although were not revealed in the borings drilled in this field exploration.

4.2.5 Bedrock

Rock coring was performed in borings B-30, B-32, B-39, and B-41. The rock core samples generally consisted of a RDZ and very fine to coarse grained, argillaceous, fossiliferous, vuggy, brown and gray dolomite interbedded with gray shale. Rock with relatively poor quality was encountered in the upper 5 to 11 feet of rock core in borings B-30, B-32, and B-39. Rock recovery in this zone ranged from 15 to 88 percent and the Rock Quality Designation (RQD) ranged from 0 to 40, which corresponds to a Rock Quality Description of very poor to poor according to Table 39 of FHWA-IF-02-034 *Geotechnical Engineering Circular No. 5 Evaluation of Soil and Rock Properties*. Below the poor quality rock zone, rock recovery generally ranged between 88 and 100 percent and the Rock Quality Designation (RQD) ranged between 70 and 100 percent, which corresponds to a Rock Quality Description of fair to excellent according to Table 39 of FHWA-IF-02-034.

At the anticipated abutment locations for the 3-span bridge option, rock soundings were performed in borings B-28 and B-43 while rock coring was performed in borings B-30 and B-41. The base of RDZ or auger refusal was generally encountered in these borings at depths of approximately between 10 and 17 feet or at elevations approximately between 839 and 840 feet.

At the anticipated breastwall abutment and pier locations, rock soundings were performed in borings B-31, B-32b, B-34 through B-37, and B-40, while rock coring was performed in borings B-32 and B-39. The base of RDZ or auger refusal was generally encountered in these borings at depths of approximately between 6.5 and 19.5 feet or at elevations approximately between 835 and 842 feet.

A summary of the top of bedrock elevations, auger refusal elevations, and bottom of RDZ elevation is included in Appendix I.

4.3 Observation Wells and Groundwater Levels

During the field exploration, measurable groundwater levels were reported in some of the borings. A few borings, where the site conditions allowed, were left open for 24 hours after completion for longer-term water level measurements. Observation wells were installed in offset locations from borings B-32 and B-41. The following table, Table 1, presents the elevations where groundwater was encountered during drilling (immediate water level) and where the groundwater level was encountered after the completion of drilling.

Table 1 – Groundwater Observations Summary

Boring No. (Ground Surface Elevation, ft)	Depth (Elevation) Water Level, ft			Depth (Elevation) to Bedrock, ft
	Depth (Elevation) to Immediate Water Level[1]	24 hrs After Completion	In Observation Well, Depth/ Elevation (Date Measured)[5]	
B-30 (850.2)	10.7 (839.5)	NM[3]	N/A[4]	8.5 (841.7)
B-31 (854.5)	NR[2]	NR[2]	N/A[4]	12.5 (842.0)
B-32 (854.6)	3.9 (850.7)	NM[3]	NR[2] (1-7-15)	11.0 (843.7)
B-35 (846.0)	8.3 (837.7)	4.8 (841.2)	N/A[4]	10.5 (835.5)
B-36 (855.4)	NR[2]	NR[2]	N/A[4]	13.5 (841.9)
B-39 (849.1)	8.5 (840.6)	NM[3]	N/A[4]	6.0 (843.1)
B-40 (843.7)	5.2 (838.5)	3.7 (840.0)	N/A[4]	6.5 (837.2)
B-41 (856.1)	NR[2]	NM[3]	NR[2] (1-7-15)	13.5 (842.6)

[1] Water level reported before adding drilling water, measured from the ground surface.
[2] NR: Measured, but none reported.
[3] NM: Boring was NOT left open for 24-hr reading (not measured).
[4] N/A: No observation well was installed.
[5] Measured from the ground surface.

Groundwater readings were taken from the observation wells on April 22, 2015. When DLZ personnel were at the site, part of the soil at the ground surface of the observation well was found to be missing. It should be noted that the soil at the ground surface was to prevent entry of surface water from entering the well. With the soil at the ground surface partially missing, the groundwater levels in the wells could likely include the surface runoff or rain water that might have entered into the wells, and therefore were not considered true groundwater levels. Consequently, the groundwater level measurements taken on April 22, 2015 are not included in this report.

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. Consequently, during construction or at other times during the project life, water levels may be higher or lower than observed at the time of this investigation.

5.0 LABORATORY TESTING AND RESULTS

5.1 General

Laboratory tests were performed in accordance with applicable AASHTO or Kentucky Methods of soil and rock testing specifications (KM) as specified in the KYTC Geotechnical Guidance Manual, dated June 2005. Where both AASHTO and KM standards are designated in the guidance manual, the KM procedure was used. The results of the laboratory tests are shown on the laboratory test summary tables in Appendix II. Appropriate soil profile sheets will be prepared and included in the final report.

5.2 Disturbed Soil Testing

The laboratory testing program consisted of visual classifications of all collected soil and decomposed rock samples, and general index tests on selected Standard Penetration Test samples. The general index tests consisted of grain-size analyses, moisture content, and plasticity determinations. The types of soils resulting from laboratory classification testing were mostly fat clay (AASHTO A-7-5 and A-7-6), which is in general agreement with the results of visual classification of all collected soil. However, the disturbed soil sample taken from boring B-41 at a depth of 1 foot was classified as non-plastic silt (ML or A-4). This sample was brown to dark brown in color and contained fine to coarse sand and trace organic, which was considered probable fill material. The in-place moisture contents of tested samples collected varied approximately from 13 to 44 percent with an average of approximately 27 percent.

It should be noted that fat clays classified as AASHTO A-7-5 have plasticity index equal to or less than the difference between the soils' liquid limit and 30 while fat clays classified as AASHTO A-7-6 have plasticity index greater than the difference between the soils' liquid limit and 30. The fat clays classified as A-7-5 may be highly elastic in addition to being prone to shrink/swell with change in moisture contents as well as relatively low long term strength like fat clays classified as A-7-6.

5.3 Rock Core Testing

The Slake Durability Index (SDI) and Jar Slake (JS) tests provide indications of the effects weathering will have on the bedrock when exposed in open cut faces. Shale recovered from the rock coring operations was subject to these tests. The following table, Table 2, summarizes the results of the SDI and JS tests on selected shale samples.

Table 2 – Slake Durability and Jar Slake Test Results Summary

Boring/ Depth, ft	SDI, %	Fragment Type	Classification	JS Category
B-30/ 18.5 – 19.5	63.9	III	Non-durable Class II	4
B-30/ 30 – 30.6	98.9	I	N/A*	N/A*
B-39/ 16.9 – 17.5	54.0	II	Non-durable Class II	3
B-39/ 26.3 – 27.5	95.1	I	N/A*	N/A*
B-41/ 27.8 – 28.4	79.8	II	Non-durable Class I	4

*The sample was re-classified as dolomite.

Unconfined compression tests were performed on ten cores of the Laurel Dolomite. The unconfined compressive strengths of the dolomite ranged from 1,720 to 15,748 psi; the average strength was 7,817 psi.

6.0 ENGINEERING ANALYSES

The proposed railroad bridge will be constructed at the existing railroad embankment. Based on the current design information, the top of the embankment elevation is approximately at 861 feet and the elevation of the south embankment toe is approximately at 846 feet. The existing railroad embankment has approximately 2H:1V side slopes. It is our understanding that the proposed finished grade of the new Allen Lane at the railroad bridge is anticipated to be approximately at elevation 835.5 and the proposed finished grade of the railroad bridge approximately at elevation 861 to match the existing grade of the railroad embankment. Based on the proposed finished grades of the bridge and the new Allen Lane and the existing grade at the railroad road embankment, placement of fills of up to approximately 15 feet will be required on the south side of the bridge abutments. However, to accommodate the proposed alignment shift of the existing Allen Lane to the west and the proposed grade separation at the Allen Lane/CSX railroad intersection, cuts of up to 26 feet are also anticipated. Based on the subsurface conditions encountered in the borings drilled at the bridge location, the new fill at the south side of the abutments will be placed on fat clay underlain by RDZ and bedrock and the cuts will be constructed in fat clay and the underlying RDZ and bedrock.

Two bridge options are being considered. One option is a single-span bridge option with a span length of approximately 103 feet with breastwall abutments. The finished grade of the proposed Allen Lane, immediately in front of the east abutment, is approximately at 839 feet, while the proposed finished grade immediately in front of the west abutment is approximately at elevation 837 feet. The other option is an approximately 120 feet long, 3-span bridge, with span lengths of approximately between 49 and 64.5 feet. The finished grade of the proposed Allen Lane, immediately in front of the east pier, is approximately at elevation 839 and the proposed grade immediately in front of the west pier is approximately at elevation 840. It is our understanding that the top-down method will be used for constructing the proposed bridge. It is anticipated that tangent drilled shafts and drilled shaft-supported piers will be used for the support of the proposed bridge.

It should be noted that the configuration of the proposed bridge has not been finalized at the time of this reporting, the borings drilled in the present field exploration may not be located at the final pier and abutment locations. The subsurface conditions at the final pier and abutment locations may be different from those encountered in the borings.

6.1 Correction of Standard Penetration Test Data

Standard correlations for the Standard Penetration Test (SPT) are based upon the blow counts (N-values) determined by a safety hammer (rope and cathead) system, which is generally estimated to be 60 percent efficient. Thus, the measured SPT N-values are termed as N_{60} data. The automatic hammers used for this exploration were tested and the average energy transfer efficiencies were reported to be between 74 and 89.9 percent based on the calibration test results in 2014. Consequently, the measured SPT N-values need to be corrected to normalized N_{60}- values. The measured SPT N-values and the normalized N_{60}-values for each boring is included in the summary table of laboratory test results in Appendix II.

6.2 Soil and Rock Parameter Selections

The subsurface profile for the analyses used the stratigraphy disclosed by the borings and material properties were estimated from normalized Standard Penetration Test blow counts (N_{60}-values), visual classification, index properties, unconfined compressive strengths, and engineering judgment. The federal government's Naval Facilities Design Manual (NAVFAC DM-7.2) and the FHWA's Soils and Foundations Workshop Manual were used as guide in estimating the soil and rock parameters used for the analyses. Additional published engineering literatures were also used, wherever applicable, to provide additional references for the soil and rock parameters. Generally, the parameters derived for the subsurface materials are typical of cohesive and bedrock materials found in the project area. The soil and rock properties selected for the analyses are presented in following sections.

6.3 Slope Stability Analyses

The global stability analyses, considering the short term, intermediate-term (if applicable), long-term, and seismic conditions, were performed for the selected critical sections. The short term analyses considered undrained shear strength parameters; whereas the intermediate-term, long-term, and seismic analyses considered drained shear strength parameters. The seismic analysis was also performed using a horizontal acceleration of 0.06, based on the seismic hazard at the site. The stability evaluations were performed using the GeoStudio 2012 software package with the SLOPE/W module. SLOPE/W is a software product for analyzing limit equilibrium to solve slope stability problems using the method of slices. Stability analyses were performed using Spencer's method, assuming potential circular failure surfaces. Furthermore, it was assumed that potential failure surfaces did not extend below the bottom of the rock disintegration zone.

Slope stability analysis for cut and fill sections took into account the soil and bedrock thickness at the sections and the subsurface conditions encountered in the field exploration. The most critical fill section is considered to be constructed in an area with the thickest layers of cohesive foundation soils while the most critical cut section to be constructed in the thickest layers of cohesive soils.

Based on the current design plan available, it is anticipated that placement of fills of up to approximately 15 feet will be required on the south side of the bridge abutments with the existing grade at 846 feet. To accommodate the proposed alignment shift of the existing Allen Lane to the west and the proposed grade separation at the Allen Lane/CSX railroad intersection, cuts of up to 26 feet are anticipated at approximate Sta. 170+25. The existing ground surface elevation at Sta. 170+25 is approximately at 863 feet and the proposed grade at Sta. 170+25 will be approximately at 837 feet. The borings drilled at the bridge and grade separation areas generally encountered cohesive soils of high plasticity underlain by bedrock approximately at elevations between 835.2 and 843.7 feet.

Given the existing grade at the south side of the bridge abutments at 846 feet and the lowest top of bedrock elevation at 835.2 feet, the new fill (up to approximately 15 feet) is anticipated to be placed on the subgrade materials consisting of up to approximately 10.8 feet (elevation 846 minus elevation 835.2) of fat clay underlain by RDZ and bedrock. Given the existing grade at Sta. 170+25 approximately at 863 feet, the proposed grade at the station approximately at 837 feet, and the lowest top of bedrock elevation at 835.2 feet, the cut at this station will be constructed in up to approximately 26 feet (elevation 863 minus elevation 837) of fat clay underlain by either the fat clay or RDZ and bedrock. In the analysis, the groundwater level was assumed to be 2 feet above the soil-rock interface and a vertical live load surcharge of 1,880 psf (8’-6” continuous strip) was used based on the recommendation indicated in the appendix of the “*CSXT Public Project Information*”, last revised on August 10, 2012. It should be noted that the vertical live load surcharge of 1,880 was applied in the stability analysis for the short-term (undrained) condition only.

After discussions with KYTC and using the empirical correlations for drained shear strengths from a literature search, the effective shear strength parameters consisting of a cohesion value of 200 psf and a friction angle of 20 degrees were selected to be used for the highly plastic soils at the site for the intermediate-term analysis. In the long-term stability analysis, a cohesion of 40 psf and friction angle of 20 degrees were used to account for the possible severe swelling or softening, or large potentials for sloughing failures. Calculation of the estimated drained shear strength is included in Appendix III.

6.3.1 Side Slopes at the South Side of the Abutment (Fill Slopes)

It is our understanding that the bridge approach embankments are planned to be constructed with 3H:1V to 2H:1V side slopes due to right-of-way constraints. Stability analyses were performed using a desired side slope of 2H:1V and assuming that cohesive soils with low plasticity are permitted for use in the railroad embankment construction. Highly plastic clay soils should not be used for embankment fill. The following tables summarize the soil and rock properties used and the results of the stability analyses.

Table 3 – Fill Slope Embankment Shear Strength Parameters

	Cohesive (Lean Clay) Fill for Proposed Side Slope Embankment			Existing Foundation Soils (Fat Clay)		
	c/c' (psf)	φ/φ'	γ /γ' (pcf)	c/c' (psf)	φ/φ'	γ /γ' (pcf)
Short-term (Undrained)	1,500	0°	125	1,000	0	130
Long-term (Drained)	270[1]	28[1]	125	40	20	130
Seismic (0.06g)	270[1]	28[1]	125	40	20	130

[1] The federal government's Naval Facilities Design Manual (NAVFAC DM-7.2).

Table 4 - Summary of Side Slope Embankment Stability Analyses

Location	Slope Geometry (H:V)			F.S./Target F.S.		
	Proposed Side Slope Embankment	Rock Disintegration Zone	Bedrock	Short-term	Long-term	Seismic
South Side of Bridge Abutments	2:1	N/A[1]	N/A[1]	1.6/1.2-1.4	1.6/1.6-1.8	1.4/1.0-1.2

[1] N/A: The bottom of the embankment is within overburden.

6.3.4 Cut Section for Spill-through Slopes at Bridge Abutments

It is our understanding that the spill-through cut slopes are planned to be constructed with 3H:1V to 2H:1V slopes due to right-of-way constraints. The preliminary design called for shallow footings and driven piles to be used for the support of the proposed bridge. Given the existing subsurface conditions consisting of predominantly highly plastic clays and the large railroad surcharge (1,880 psf), stability analyses indicated that the spill-through cut slopes at the proposed bridge abutments (assumed to be supported by shallow footings or driven piles) cannot be constructed with 2.5H:1V (or steeper) slopes without additional measures to stabilize the side slopes. In order to construct the proposed cut sections with 2.5H:1V and meet the required factors of safety of 1.2 to 1.4 for the short-term stability and 1.6 to 1.8 for the long-term stability, stability analyses indicated that approximately 30 to 35 feet (measured from the cut face horizontally back into the slope) of the existing cohesive materials in the cut slopes be removed and replaced with compacted low plasticity cohesive materials. The excavation and replacement of the existing cohesive materials should extend to the top of RDZ. The following tables, Tables 5 and 6 summarize the soil and rock properties used in the stability analyses.

Table 5 - Shear Strength Parameters (Spill-through Cut Slope)

	Cohesive (Lean Clay) Fill for Proposed Side Slope Embankment			Existing Cohesive Soils (Fat Clay) in Cut Slopes			RDZ		
	c/c’ (psf)	φ/φ’	γ /γ’ (pcf)	c/c’ (psf)	φ/φ’	γ /γ’ (pcf)	c/c’ (psf)	φ/φ’	γ /γ’ (pcf)
Short-term (Undrained)	1,500	0°	125	1,000	0	130	0	40	145
Long-term (Drained)	270[1]	28[1]	125	40	20	130	0	40	145
Seismic (0.06g)	270[1]	28[1]	125	40	20	130	0	40	145

[1] The federal government’s Naval Facilities Design Manual (NAVFAC DM-7.2).

Table 6 - Summary of Spill-through Cut Slope Stability Analyses With the Shallow Footing or Driven Pile Supported Abutments

Station	Slope Geometry (H:V)			F.S./Target F.S.		
	Cut Sections With Replacement of Existing Fat Clay[1]	Rock Disintegration Zone	Bedrock	Short-term	Long-term	Seismic
170+25	2.5:1	N/A[2]	N/A[2]	1.2/1.2-1.4	1.6/1.6-1.8	1.4/1.0-1.2

[1] **Proposed cut sections with replacement of existing highly plastic clay with low plasticity cohesive materials and the top of RDZ at elevation 840**. It should be noted that a 3H:1V slope was recommended in the draft report dated February 10, 2015, where the recommended 3H:1V slope was determined based on the top of RDZ at elevation 835, which should have been at elevation 840.

[2] N/A: The bottom of the cut is within overburden.

Additional slope stability analyses were performed using compacted granular materials in lieu of low plasticity cohesive soils for the replacement of existing highly plastic clay. Results of the analyses indicated that using granular materials for the replacement material would require larger areas of overexcavation and replacement of the highly plastic clay than using low plasticity cohesive soils would in order to meet the required factors of safety.

It is understood that the recent design calls for the top-down method to be used for constructing the proposed bridge and that tangent drilled shafts would be installed for the support of the bridge abutments prior to excavation. The elevation views of the recent design concepts are included in Appendix III. With the anticipated tangent drilled shaft walls at the proposed bridge abutments, additional slope stability analyses were performed to determine the required spill-through cut slope at the tangent drilled shaft supported abutments. In the analyses, any earth loads behind the tangent drilled shaft walls, surcharge loads, and superstructural loads were assumed to be fully supported by the tangent drilled shaft walls, thereby the spill-through slopes were considered as embankments structurally separated from the tangent drilled shaft walls. Consequently, the spill-through slope was essentially analyzed as a “stand alone” embankment. Tables 7 and 8 summarize the soil and rock properties used in the stability analyses and the analytical results.

Table 7 - Shear Strength Parameters (Tangent Drilled Shaft Abutment Walls and Spill-through Cut Slopes)

	Existing Cohesive Soils (Fat Clay) in Cut Slopes			RDZ		
	c/c’ (psf)	φ/φ’	γ /γ’ (pcf)	c/c’ (psf)	φ/φ’	γ /γ’ (pcf)
Short-term (Undrained)	1,000	0	130	0	40	145
Intermediate-term	200	20	130	0	40	145
Long-term (Drained)	40	20	130	0	40	145
Seismic (0.06g)	40	20	130	0	40	145

[1] The federal government’s Naval Facilities Design Manual (NAVFAC DM-7.2).

Table 8 - Summary of Stability Analyses for Spill-through Cut Slopes (Assuming Spill-through to be Structurally Separated from the Tangent Drilled Shaft-Supported Abutments, i.e., A Stand Alone Embankment)

Station	Slope Geometry (H:V)			F.S./Target F.S.[1]		
	Cut Sections With **Existing Fat Clay**[2]	Rock Disintegration Zone	Bedrock	Short-term	Inter-mediate-term	Long-term
170+25	3:1	N/A[3]	N/A[3]	3.5/1.2-1.4	2.1/1.2-1.4	1.4/1.4-1.6

[1] Factors of safety for cut slopes in soil.

[2] Existing overburden.

[3] N/A: The bottom of the cut is within overburden.

With the anticipated tangent drilled shaft-supported bridge abutments, results of the stability analyses indicated that the spill-through slopes can be constructed with 3H:1V side slopes or flatter with the existing overburden (i.e., no soil stabilization needed). If a steeper spill-through cut slope is needed, in-situ ground improvement techniques, such as installing rammed aggregate piers (RAP) or other soil stabilization techniques, may be considered to improve the global stability of the spill-through cut slopes steeper than 3H:1V.

Using the soil and rock properties in Table 7, analyses were also performed to evaluate the stability of the tangent drilled shaft wall assuming that the spill-through slope was not present in front of the wall and the analytical results are summarized in Table 9.

Table 9 - Summary of Stability Analyses for the Tangent Drilled Shaft Abutment Wall Without the Spill-Through Slope

Station	F.S./Target F.S.		
	Short-term	Long-term	Seismic
170+25	2.1/1.2-1.4	2.9/1.6-1.8	2.6/1.0-1.2

6.4 Settlement Analyses

The settlement analyses were performed in general accordance with the FHWA's Soils and Foundation Workshop Manual.

Given the existing ground surface elevation (approximately between 846 and 848 feet) at the south side of the proposed bridge abutments and the proposed bridge grade approximately at elevation 861, placement of fills of up to 15 feet will be required at south side of the bridge abutments. Based on the soil types and thicknesses encountered in the borings drilled for the proposed bridge, the top of bedrock elevations in the boring locations varied from approximately 843.7 to 835.2 feet, the subsurface conditions at the new fill areas will likely consist of up to 11 to 13 feet of fat clay underlain by RDZ and bedrock. The soil profile used for the stability analysis as described in Section 6.3 of this report was used for the settlement analysis. In the analysis, the groundwater level was assumed to be 2 feet above the soil-rock interface. Result of the analysis indicated that the total settlement on the order of 2.3 inches was estimated due to the placement of 15 feet of new fill at the south side of the bridge abutments. Using an estimated coefficient of consolidation of 0.18 ft^2/day based on an average liquid limit of 70 for the soil (Ref. NAVFAC DM 7.1), a preliminary time dependent settlement calculation was performed to estimate the time required for 90% of the estimated total settlement to occur. Result of the analysis indicated that it may take up to 2¼ to 2½ years for 90% of the estimated total settlement to fully occur because of the inherent characteristics of fat clay and the underlying "impermeable" bedrock.

Consideration should be given to replacing the existing cohesive soils with granular soils to minimize the amount of settlement and time for it to occur due to the added embankment fill. Alternatively, in-situ ground improvement techniques, such as installing rammed aggregate piers (RAP) or other soil stabilization techniques, could be considered to minimize the amount of soil settlement if the estimated soil settlement is deemed excessive. Settlement platforms are recommended to monitor the actual amount and rate of settlement.

6.5 Deep Foundation Analyses

6.5.1 Design Parameters

It is anticipated that the proposed bridge piers or abutments will be supported on drilled shaft foundations deriving support within the underlying dolomite from base resistance, side resistance or a combination of both. However, using a combination of base and side resistance will require strain compatibility and settlement analyses, which require the anticipated drilled shaft geometry as well as the design loads. This information is not available at the time of this reporting. The settlement of the drilled shaft group should be evaluated as part of final design when the drill shaft geometry is finalized. The design of the drilled shaft foundations should also include the evaluation for lateral resistance and uplift capacity, which may control the required shaft diameter and rock socket length. Given the subsurface conditions and the preliminary design information available, design parameters for both axial end bearing and side resistance for rock socketed drilled shafts are provided in the following sections. Calculations for some of the design parameters are included in Appendix III.

Based on the groundwater level information collected during this field exploration, a groundwater level may be assumed to be 2 feet above the soil-rock interface approximately at elevation 835 feet for design purposes.

A vertical live load surcharge of 1,880 psf (8'-6" continuous strip) should be used based on the recommendation indicated in the appendix of the "*CSXT Public Project Information*", last revised on August 10, 2012.

6.5.2 Axial Loading

Based on the results of the field exploration, the top of bedrock elevations varied between 843.7 and 835.5 feet, and the base of RDZ elevations between 841.7 and 834.7 feet. The drilled shafts should be extended to a tip elevation a minimum of 1.5 times the shaft diameter (the diameter above top of bedrock elevation) below the base of RDZ, which may be assumed approximately at elevation 835 feet for design purposes. It should be noted that poor quality dolomite bedrock was encountered approximately between elevations 835 and 830 in boring B-32. The unconfined compressive strength of a core sample of the poor quality dolomite was 2,579 psi. For the drilled shaft foundations, it is recommended that a minimum shaft diameter above the top of bedrock of 3 feet should be used. If casing is used to install the drilled shafts, the rock socket diameter should be 6 inches smaller than the shaft diameter above the bedrock elevation.

The nominal side resistance of a single drilled shaft was calculated using Equation 10.8.3.5.4b-2 (Kulhawy et al., 2005 method) and the tip resistance Equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3 (Brown et al., 2010 method) as indicated in Section 10.8.3.5.4 of the AASHTO LRFD BDS 7th Ed (hereinafter referred to as AASHTO BDS). The nominal tip resistance was estimated to be 150 ksf and the nominal side resistance 13 ksf. These calculations are included in Appendix III.

According to Section C10.8.3.5.4d of the AASHTO BDS, the axial compression load on a drilled shaft socketed into rock is typically carried solely in side resistance until a total movement on the order of 0.4 inches occurs. However, when the rock is brittle in shear much of the side resistance could be lost as vertical movement increases to the value required to develop the full value of the tip resistance. Without the strain compatibility and settlement analyses, adding the full value of the calculated side and tip resistances directly is considered unconservative. Consequently, where combined side friction and end-bearing in rock is considered, relatively conservative values of the calculated side and tip resistances based on a lower bound compressive strength of the tested rock cores (2,000 psi) should be used for design. It is recommended that a nominal (unfactored) unit tip resistance of 150 ksf and a nominal (unfactored) unit side resistance along the socket of 10 ksf be used to design and proportion the size of drilled shafts. These recommended resistance values considered both the variability and the general conditions of the rock encountered in the borings and also took into account of the potential sidewall disturbance and strain compatibility issues, which would have a significant impact on design performance of drilled shafts. The recommended nominal side and tip resistances are generally typical of bedrock materials found in the project area. Side resistance of the top two feet of rock socket as well as the portion of the shaft embedded in soils and RDZ should be neglected. Resistance factors of 0.5 and 0.55 for tip and side resistance (Ref. Table 10.5.5.2.4-1 of the AASHTO BDS), respectively should be applied to the nominal unit resistance values provided above to calculate the factored resistance of the shafts. However, where the substructure is supported by one shaft, a 20% reduction in the resistance factors should be applied due to non-redundancy per Section 10.5.5.2.4 of the AASHTO BDS).

A single row of drilled shafts which develops their capacity from a combination of side resistance and end resistance should be installed with a minimum center-to-center spacing of three (3.0) times the shaft diameter to avoid reduction in axial capacity. If drilled shafts are spaced closer than 3.0 times the shaft diameter, then further evaluation to determine group effects will be needed. Adjacent shafts should not be constructed on the same day.

For tangent drilled shafts, the nominal bearing resistance of the shaft group should be the nominal resistance of an equivalent pier consisting of the shafts. The total vertical surface area of the equivalent pier should be used to determine the side resistance and the base area of the equivalent pier should be used to determine the end bearing resistance. The nominal bearing resistance of a shaft group should not be greater than the sum of the nominal bearing resistance of individual shafts.

Casing may be required in the overburden for the drilled shaft installation. However, the casing is not required to be left in-place after installation of the drilled shaft foundations is completed for geotechnical resistance and therefore may be removed, unless required for structural resistance. The appropriate means and methods should be employed by the contractor to provide integrity of the completed shafts and mitigate necking, voids, etc.

6.5.3 Lateral Loading

The drilled shafts are anticipated to be subjected to lateral loads from the superstructure, surcharge, earth pressure, and any anticipated seismic activities in the project area, and should be designed accordingly. The shaft lengths should be designed such that the lateral deflections are acceptable due to the anticipated lateral loads.

According to the current design information, the proposed bridge grade will be approximately at elevation 861. For the drilled shaft foundations, it is recommended that the drilled shafts be extended to a tip elevation a minimum of 1.5 times the shaft diameter (the diameter above top of bedrock elevation) below the base of RDZ elevation approximately at 835 feet. Given the existing ground surface elevation (approximately between 846 and 848 feet) at the south side of the proposed bridge abutments and the proposed bridge grade approximately at elevation 861, placement of fills of up to 15 feet will be required at south side of the bridge abutments. Based on the subsurface conditions encountered in the borings drilled at the proposed bridge location, recommended parameters for static lateral analysis of a single drilled shaft are provided in a table included in Appendix III. These parameters are intended to be used with the computer software LPILE by Ensoft, Inc.

Group effects due to loading should be considered when laterally loaded drilled shafts are used in closely spaced groups. It is recommended that group effects be considered when shaft spacing is less than six (6) diameters in any direction. A p-multiplier as indicated in Table 10.7.2.4-1 of the AASHTO BDS can be used to accommodate the group effects at the pier locations. For laterally loaded tangent drilled shaft supported abutments, a p-multiplier of 0.529 may be used to account for the group effects. The calculation for the p-multiplier of 0.529 is included in Appendix III.

6.5.4 Uplift Design

The nominal uplift resistance should be estimated in a manner similar to that for estimating the side resistance of drilled shafts in compression. However, a lower resistance factor should be used for uplift than axial compression. According to Table 10.5.5.2.4-1 of the AASHTO BDS, an uplift resistance factor of 0.4 should be used for single-drilled shafts installed in rock. It is recommended that an uplift resistance factor of 0.4 be used for group-drilled shafts.

6.5.5 Downdrag (Negative Skin Friction) Estimates

Approximately 15 feet of fill is anticipated to be placed on the south side of the proposed bridge abutments. Fill placement in the close vicinity of the abutment foundations will result in settlement of the foundation soils beneath the abutments. Settlement of these materials adjacent to the deep foundation elements will induce negative skin friction forces and apply downdrag loads to the piles.

According to Section 3.11.8 of the AASHTO BDS, if the settlement of the soil layers is 0.4 inches or greater relative to the shaft, downdrag can be assumed to fully develop. Results of the settlement analyses indicated that the total settlement on the order of 2 inches is anticipated due to the placement of 15 feet of new fill at the south side of the bridge abutments. This estimated settlement occurred in the approximately 13 feet thick fat clay soils above the top of RDZ or bedrock approximately at elevation 835 feet. Consequently, downdrag (negative skin friction) is anticipated in the soils above the top of RDZ or bedrock. It is recommended that a nominal downdrag load per unit area of shaft on the order of 1.5 ksf be used to estimate the downdrag loads and that a load factor of 1.25 be used for factored downdrag loads on drilled shafts in accordance with Table 3.4.1.2 of the AASHTO BDS. It should be noted that the total downdrag load is based on the drilled shaft diameter to be used in the final design. The following table presents the equation for calculating the total downdrag load on a single-drilled shaft.

Table 10 – Estimated Downdrag Load on a Single-Drilled Shaft at South Side of Abutments

Anticipated Bottom Elevation of Abutments[1], ft	**Top of Bedrock Elevation, ft**	**Estimated Shaft Length Subjected to Downdrag (H)[2], ft**	**Estimated Unfactored Downdrag Load[3], kips**	**Estimated Factored Downdrag Load[4], kips**
848	835	13	π x D x H x 1.5	π x D x H x 1.5 x 1.25

[1] The anticipated bottom elevation of abutments may vary in the final design.

[2] Estimated shaft length subjected to downdrag is equal to the difference between the bottom Elevation of the abutment and the top of bedrock elevation.

[3] D is the drilled shaft diameter and 1.5 ksf is the recommended nominal downdrag load per unit area of shaft.

[4] Load factor of 1.25 for downdrag on drilled shafts per Table 3.4.1-2 of AASHTO BDS.

6.5.6 Lateral Squeeze

FHWA guidelines suggest that if the pressure exerted by the weight of the embankment exceeds three times the undrained shear strength of the foundation soils, the potential for lateral squeeze exists. As indicated in Section 6.5.3, fat clay exists between the existing ground surface at the south side of the abutment (approximately elevation 846) and the anticipated top of RDZ or bedrock (approximately elevations between 835.5 and 844). Based on the boring information, an undrained shear strength of 1,000 psf was used for the fat clay. The pressure increase due to the placement of 15 feet of new fill at the base of the existing ground surface at the abutments is approximately 1,950 psf, using a unit weight of 130 pcf for the new fill. Using the FHWA guidelines, the pressure increase applied by the new fill (1,950 psf) does not exceed three times the undrained shear strength of the foundation soils (3 times 1,000 psf equal to 3,000 psf), indicating that the potential for lateral squeeze is low for the abutment foundations. It should be noted that the undrained shear strength of 1,000 psf was used for the existing highly plastic foundation soil. If the existing highly plastic clay foundation soils are replaced with compacted low plasticity cohesive soils as recommended above, a higher undrained shear strength of 1,500 psf may be used to evaluate the potential for of lateral squeeze, which results in a lower potential for lateral squeeze (i.e. higher factor of safety).

7.0 GEOTECHNICAL RECOMMENDATIONS

The following foundation recommendations are based upon reviews of available data, information obtained during the field exploration, results of laboratory testing, engineering analyses, and discussions with KYTC.

7.1 Foundation Recommendations

1. The proposed bridge piers or abutments can be supported on drilled shaft foundations deriving support within the underlying dolomite from a combination of base and side resistance. The drilled shafts should be extended to a tip elevation a minimum of 1.5 times the shaft diameter (above bedrock elevation) below the base of RDZ elevation approximately at 835 feet.

2. A nominal (unfactored) unit tip resistance of 150 ksf and a nominal (unfactored) unit side resistance along the socket of 10 ksf can be used to design and proportion the size of drilled shafts. Side resistance of the top two feet of rock socket as well as the portion of the shaft embedded in soils and RDZ should be neglected.

3. Resistance factors of 0.5 and 0.55 for tip and side resistance, respectively, should be applied to the nominal unit resistance values provided above to calculate the factored resistance of the shafts. However, where the substructure is supported by one shaft, a 20% reduction in the resistance factors should be applied due to non-redundancy.

4. A minimum shaft diameter above the top of bedrock of 3 feet should be used. If casing is used to install the drilled shafts, the rock socket diameter should be 6 inches smaller than the shaft diameter above the bedrock elevation.

5. A single row of drilled shafts which develop their capacity from a combination of side resistance and end resistance should be installed with a minimum center-to-center spacing of three (3.0) times the shaft diameter to avoid reduction in axial capacity. If drilled shafts are spaced closer than 3.0 times the shaft diameter, then further evaluation to determine group effects will be needed.

6. Adjacent shafts should not be constructed on the same day.

7. For tangent drilled shafts, the nominal bearing resistance of the shaft group should be the nominal resistance of an equivalent pier consisting of the shafts. The total vertical surface area of the equivalent pier should be used to determine the side resistance and

the base area of the equivalent pier should be used to determine the end bearing resistance.

8. Casing may be required in the overburden for the drilled shaft installation. However, the casing is not required to be left in-place after installation of the drilled shaft foundations is completed for geotechnical resistance and therefore may be removed, unless required for structural resistance. The appropriate means and methods should be employed by the contractor to provide integrity of the completed shafts and mitigate necking, voids, etc.

9. A groundwater level may be assumed to be 2 feet above the soil-rock interface approximately at elevation 835 feet for design purposes.

10. A vertical live load surcharge of 1,880 psf (8'-6" continuous strip) should be used based on the recommendation indicated in the appendix of the "*CSXT Public Project Information*", last revised on August 10, 2012.

11. The settlement of the drilled shaft group should be evaluated as part of final design when the drill shaft geometry is finalized. Based on the results of the settlement analyses, the recommended side and tip resistance in item #2 above may need to be revised, if deemed necessary.

12. The design of the drilled shaft foundations should also include the evaluation for lateral resistance and uplift capacity, which may control the required shaft diameter and rock socket length.

13. Group effects due to loading should be considered when laterally loaded drilled shafts are used in closely spaced groups. Group effects should be considered when shaft spacing is less than six (6) diameters in any direction. A p-multiplier as indicated in Table 10.7.2.4-1 of the AASHTO BDS can be used to accommodate the group effects at the pier locations. For laterally loaded tangent drilled shafts, a p-multiplier of 0.529 may be used to account for the group effects.

14. An uplift resistance factor of 0.4 should be used for single-drilled shafts installed in rock. An uplift resistance factor of 0.4 may be used for group-drilled shafts.

15. Downdrag is anticipated in the soils above the top of RDZ or bedrock on the south side of the bridge abutments. The downdrag load estimated in Section 6.5.5 of this report shall be added to the total design loads for the design of drilled shaft foundations. The downdrag load and length of drilled shaft subject to downdrag are a function of the shaft length and size. The estimated downdrag load is based on the assumptions made in Section 6.5.5. Should any of the assumptions indicated in Section 6.5.5 change, the estimated downdrag load would no longer be valid and should be adjusted accordingly.

16. All drilled shaft shall be constructed in accordance with the Special Note for Drilled Shafts, current edition and the recommendations in Section 7.0, Geotechnical Recommendations, of the Geotechnical Report.

7.2 Embankment Slope and Spill-through Slope at Abutments

1. The side fill slopes at the south side of the abutments shall be constructed with 2H:1V or flatter slopes. Embankment stability analyses were conducted using estimated soil strength parameters for cohesive soils with low swell potential cohesive soils (according to USCS or AASHTO soil classification) with liquid limit (as determined by AASHTO T-89) and plasticity index (as determined by AASHTO T-90) of less than 50 and 30, respectively. All fill materials shall be compacted in accordance with Section 206 of the current KYTC Standard Specifications for Road and Bridge Construction. All fill materials shall be approved by the Engineer prior to the placement of fill.

2. Approximately 2.3 inches of soil settlement was estimated due to the anticipated placement of 15 feet of new fill at the abutments. Consideration should be given to replacing the existing cohesive soils with granular soils to minimize the amount of settlement due to the added embankment fill. Alternatively, in-situ ground improvement techniques, such as installing rammed aggregate piers (RAP) or other soil stabilization techniques, could be considered to minimize the amount of soil settlement if the estimated soil settlement is deemed excessive.

3. Settlement platforms are recommended to monitor the actual amount of settlement due to added embankment fill at the south side of abutments.

4. The cut slopes for the spill-through slopes at the proposed tangent drilled shaft-supported bridge abutments shall be constructed with 3H:1V or flatter slopes. In-situ ground improvement techniques or other soil stabilization techniques, could be used to improve global stability of the spill-through cut slopes potentially requiring steeper slopes.

7.3 Foundation Geotechnical Notes

The designer shall include the following notes in the contract plans, where appropriate.

1. All drilled shaft shall be constructed in accordance with the in accordance with the Special Note for Drilled Shafts, current edition and the recommendations in Section 7.0, Geotechnical Recommendations, of the Geotechnical report.

2. The bedrock, except for those in RDZ, is strong dolomite/limestone with occasional interbedded shale. Hard rock excavation techniques will be necessary to remove the dolomite/limestone. **Rock blasting is not permitted for this project.** It is the Contractor’s responsibility to select the appropriate and acceptable means and methods of construction and adequate construction equipment based on the anticipated subsurface conditions and to prevent damage to adjacent structures or facilities.

3. Foundation preparation and backfilling shall be performed in accordance with Section 603 of the Standard Specifications for Road and Bridge Construction.

7.4 Roadway Geotechnical Notes

1. Clearing and grubbing of embankment areas shall be completed in accordance with Section 202 of the current KYTC Standard Specifications for Road and Bridge Construction.

2. Removal of existing structures and other obstructions shall, whether shown on the plans or not, shall be completed in accordance with Section 203 of the current KYTC Standard Specifications for Road and Bridge Construction.

3. All excavations shall be completed in accordance with Section 204 of the current Kentucky Department of Transportation Cabinet (KYTC) Standard Specifications for Road and Bridge Construction.

4. If applicable, all water wells within the limits of construction, whether shown on the plans or not, shall be plugged in accordance with requirements of Section 708 of the current KYTC Standard Specifications for Road and Bridge Construction.

5. If applicable, all catch basins and manholes shall be filled and capped, and all septic tanks, if any, shall be cleaned and filled in accordance with Section 708 of the current KYTC Standard Specifications for Road and Bridge Construction.

6. Any drainage swales, saturated, soft and unstable areas encountered within proposed embankment foundation limits and/or any other areas as specified by the Engineer shall be stabilized with a minimum of two feet (vertical thickness) of granular materials (KYTC Coarse Aggregate No. 2) in accordance with Section 805 of the current KYTC Standard Specifications for Road and Bridge Construction, and the materials shall be classified as non-erodible, as directed by the Engineer. Additional granular material may be required to stabilize the embankment foundations and to maintain positive drainage. The actual thickness and locations of granular material shall be determined by the Engineer during construction. The granular materials shall be wrapped (top and bottom) with Type IV Geotextile Fabric in accordance with Sections 214 and 843 of the current Standard Specifications. Positive drainage shall be maintained to prevent trapping water within the roadway embankment.

7. The Contractor is responsible for conducting any operations necessary in order to excavate the cut areas to the required typical sections. These operations shall be incidental to the roadway excavation price.

8. The Contractor shall conduct grading operations in such a manner that bedrock obtained from excavation below the base of RDZ shall be stockpiled separately or otherwise re-conditioned so as to be available for use in those areas requiring said material. No direct payment will be allowed for such necessary re-conditioning as stockpiling, re-handling the material, and/or hauling.

9. The construction of the embankment shall be completed in accordance with Section 206 of the current KYTC Standard Specifications for Road and Bridge Construction.

10. All channel changes, excavation of surface ditches, and construction of special ditches shall be performed in accordance with Sections 204 and 206 of the current KYTC Standard Specifications for Road and Bridge Construction, prior to placement of any embankment materials adjacent to them. The construction of the embankment shall be completed in accordance with Section 206 of the current KYTC Standard Specifications for Road and Bridge Construction. At the direction of the Engineer, materials excavated from these areas may be utilized in construction of the embankments, but may require aeration or other moisture adjustments to obtain proper moisture contents prior to compaction operations.

11. In accordance with Section 206 of the current KYTC Standard Specifications for Road and Bridge Construction, the moisture content of embankment material shall not vary from the optimum moisture content, as determined by KME 64-511, by more than plus or minus two percent. This moisture content requirement shall have equal weight with the density requirement when determining the acceptability of embankment or subgrade construction. Refer to the Family of Curves for moisture-density relationships.

12. Fat clay or shale shall not be used in construction of the embankments.

13. All soils, whether from roadway excavation or borrow, may require moisture adjustments to obtain proper moisture content prior to compaction. Direct payment shall not be permitted for re-handling, hauling, stockpiling and/or moisture adjusting soils.

14. All new fill materials shall be free of topsoil, organics, debris, or any deleterious material deemed by the Engineer. No frozen materials shall be incorporated into the fill, and no embankment, pavement, utilities, or fill shall be placed on top of frozen materials. Only suitable materials deemed by the Engineer shall be used as new fill materials.

15. No particle size larger than four inches in any direction, unless directed by the Engineer, shall be placed as fill within one foot of the finished subgrade elevation. Any particle size greater than four inches shall be broken down to less than four inches, or removed from the lift.

16. When rock, including shale, is present at the roadway subgrade, within 12 inches of the bottom of the DGA, it shall be removed in accordance with Section 204 of the current KYTC Standard Specifications for Road and Bridge Construction and replaced with soil fill in accordance with Sections 206 and 207 of the current Standard Specifications.

17. A minimum one-foot working platform (extending under the curb and gutter) will be required in areas where the roadway subgrade is soft and/or saturated. The platform will consist of Kentucky Coarse Aggregate #2 in accordance with Section 805 of the current KYTC Standard Specifications for Road and Bridge Construction. The working platform shall be wrapped with Type IV Geotextile Fabric in accordance with Sections 214 and 843 of the current Standard Specifications. The actual locations and thickness shall be determined by the Engineer during construction and may depend on seasonal fluctuations in the water table. The working platform can also serve as a drainage blanket by placing perforated drain pipe into the bottom of the granular material. Positive drainage of the perforated drain pipe shall be maintained to reduce the possibility of trapping water within the subgrade.

18. Pavement underdrains shall be provided to reduce the possibility of trapping water within the subgrade.

19. The cut slopes in the rock disintegration zones and bedrock shall be constructed with 2H:1V or flatter slopes.

20. Embankment stability analyses were conducted using estimated soil strength parameters for embankment material. Embankment stability analyses were conducted using estimated soil strength parameters for cohesive soils with low swell potential cohesive soils (according to USCS or AASHTO soil classification) with liquid limit (as determined by AASHTO T-89) and plasticity index (as determined by AASHTO T-90) of less than 50 and 30, respectively. All fill materials shall be compacted in accordance with Section 206 of the current KYTC Standard Specifications for Road and Bridge Construction. All fill materials shall be approved by the Engineer prior to the placement of fill.

21. Embankment foundation benches are not anticipated at this time. If requested by the Engineer, embankment foundation benches/slope serrations shall be constructed and perforated pipe underdrains shall be placed, as applicable, in accordance with the current KYTC Standard Drawings RDP-006 and RGX-010. The benches shall be constructed one at a time beginning with the lowest bench. Each bench shall be backfilled prior to excavation of the next bench. These procedures shall be followed to help maintain stability of the existing slopes.

22. Transverse benching is not anticipated at this time. If requested by the Engineer, transverse benching shall be constructed and perforated pipe underdrains shall be placed, as applicable, in accordance with the current KYTC Standard Drawings and RDP-005 and RDP-006, as applicable. Contrary to the Standard Drawing RDP-006, transverse benches and perforated pipe underdrains shall be installed in both uphill and downhill transition areas between cuts and fills. In addition, perforated pipe underdrains shall be installed in any areas showing signs of seepage during construction, as directed by the Engineer.

7.5 Design Recommendations

1. A rock swell factor of 15 percent is recommended for materials excavated below the rock disintegration zone (RDZ).

2. An average soil shrinkage value of five (5) percent is recommended for soils to be excavated on this project. This value is to be applied in calculating an "apparent" shrinkage value. This shrinkage value should be applied only to soil positioned above the top of rock. A shrink/swell value of zero should be applied to the weathered rock zone considered to be RDZ material.

7.6 Earthwork and Lateral Earth Pressure Recommendations

The abutments walls should be designed to resist the lateral loads imposed by the soil backfill, groundwater behind the wall, and surcharge effects of adjacent structures, trains, equipment, or materials. In order to minimize the lateral earth pressure caused by the existing fat clay behind the walls, the existing soils in the area immediately behind the abutment walls should be excavated and replaced with granular materials. It is recommended that the bottom of the excavations for the replacement of existing soils be extended a minimum of 2 feet outward from the outside edges of the abutment walls and rise from the base of the excavations at a maximum angle of 45 degrees from the horizontal. The following table presents suggested equivalent fluid unit weights for recommended granular backfill material, assuming a horizontal backslope behind the walls. The values given for submerged soil include the weight of water.

Table 11 - Suggested Equivalent Fluid Unit Weights for Granular Materials

Type of Material	**Equivalent Fluid Unit Weights (pcf)**			
	"Active" Condition		**"At Rest" Condition**	
	Unsaturated	**Submerged**	**Unsaturated**	**Submerged**
Moderately compacted granular backfill (ϕ=35°, γ_{moist} = 125 pcf, γ_{sat} = 130 pcf)	35	80	50	90

Adequate drainage should be provided behind the abutment walls. It is recommended that a wall drain consisting of 6-inch diameter perforated plastic pipes wrapped in filter fabric and bedded in a 24-inch layer of free-draining granular material (5 percent or less passing the No. 200 sieve) be installed immediately behind the abutment walls. The perforated plastic pipes should be placed near the bottom of the abutment walls and properly drained to a sump or daylighted. The 24-inch layer of free-draining granular material should be placed behind the walls up to a depth of one foot below the finished grade. The top one foot should be capped with an impervious material such as clay.

Extensive compaction of backfill against a rigid wall can result in excessive lateral earth pressures. It is recommended that the backfill material within a few feet of the walls should be placed in lifts not exceeding 8 inches loose thickness and compacted with hand-operated compaction equipment to 95 percent of the maximum dry density as determined by ASTM D698 (Standard Proctor).

If the retaining wall is restrained at the top and bottom, the “at rest” equivalent fluid unit weight should be used. If the backfill is compacted granular material and the retaining wall is allowed to yield at least 0.2 percent of the height of the wall, the equivalent fluid unit weight for the active condition may be used.

8.0 SEISMIC SITE COEFFICIENT

The AASHTO LRFD Bridge Design Specifications, Seventh Edition provides guidelines for determining the seismic hazard at a site. The seismic hazard for a site is characterized by the acceleration response spectrum and the site factors for the relevant site classification. Based on the results of this field exploration, the site is classified as Seismic Site Class C. The following table summarizes the seismic hazard coefficients and the site factors.

Table 12 - Seismic Hazard Considerations[1]

Acceleration Response Spectrum	
Peak Ground Acceleration (PGA)	0.06
Short Period Spectral Acceleration Coefficients (S_s)	0.14
Long Period Spectral Acceleration Coefficients (S_1)	0.058
Site Factors	
Factor at Peak Ground Acceleration Coefficient (F_{pga})	1.2
Factor at Short Period Range of Acceleration Spectrum (F_a)	1.2
Factor at Long Period Range of Acceleration Spectrum (F_v)	1.7

[1] Based on The AASHTO LRFD Bridge Design Specifications, Seventh Edition, 2014, Section 3.10

Based on the coefficients and factors listed above, the site classifies as Seismic Performance Zone 1.

9.0 SPECIAL CONSIDERATIONS

9.1 Excavations and Groundwater Considerations

All excavations should be constructed in accordance with applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards (29 CFR Part 1926), wherever applicable. Construction site safety generally is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. Additionally, the contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, and/or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom and to prevent damage to adjacent structures or buried installations.

Groundwater was encountered in borings B-30, B-32, B-35, B-39, and B-40 at elevations between 837.7 and 850.7 feet during the field exploration. The groundwater levels encountered were approximately 1.3 to 2.2 feet above the top of bedrock in borings B-35 and B-40, while the groundwater level encountered in boring B-32 was approximately 7 feet above the top of bedrock. However, the groundwater levels encountered in borings

B-30 and B-39 were approximately 2.2 to 2.5 feet below the top of bedrock in the borings. Observation wells were installed in borings B-32 and B-41 and dry when groundwater level measurements were taken on January 7, 2015.

It should be noted that groundwater conditions vary seasonally and with the passage of time. Furthermore, due to the shallow depths to the underlying bedrock, groundwater could flow through the fractures and joints in the bedrock and water levels during construction or at other times during the project life may be higher or lower than observed at the time of this investigation. Seepage and groundwater should be anticipated during construction; therefore, the Contractor should be equipped to deal with groundwater, seepage, and surface water that may accumulate in the open excavations. Side slopes of the excavations and adjacent structures should be constantly monitored by a competent person having knowledge relative to slope stability for signs of yielding and potential failures. The Contractor should not be allowed to have any side slopes sloughing into the excavation. If sloughing occurs, then mechanical methods of stabilization or flatter sides slopes should be utilized.

10.0 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 this report.
Respectfully submitted,

DLZ OHIO, INC.

Eric W. Tse, P.E.
Senior Geotechnical Engineer

Michael D. Kennedy, E.I.
Geotechnical Engineer

H. Jason Hughes, P.E. (Ohio)
Project Manager

EWT/MDK/HJH

M:\proj\0631\0006\02\Report (bridge)\REV Bridge Submittal (Sept 4, 2015)\Oldham County Bridge FINAL (Sept 2015).doc

APPENDIX I

Project Location Quadrangle Map
General Information - Drilling Procedures and Logs of Borings
Boring Location Plan
Geologic Map with Boring Locations
Boring Logs – Thirteen (13) Borings
Summary of Rock Soundings
Summary of Top of Bedrock Elevations, Auger Refusal Elevation, and Bottom of RDZ Elevations

DLZ

Underpass of CSX Railroad Connecting Commerce Parkway and KY 146

PROJECT LOCATION QUADRANGLE MAP

DRAWN EWT
DESIGNED EWT
APPRV'D HJH
DATE 09-02-2015

PROJECT NUMBER
0631-0006.02

REFERENCE NUMBER
N/A

DRAWING NUMBER
N/A

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.

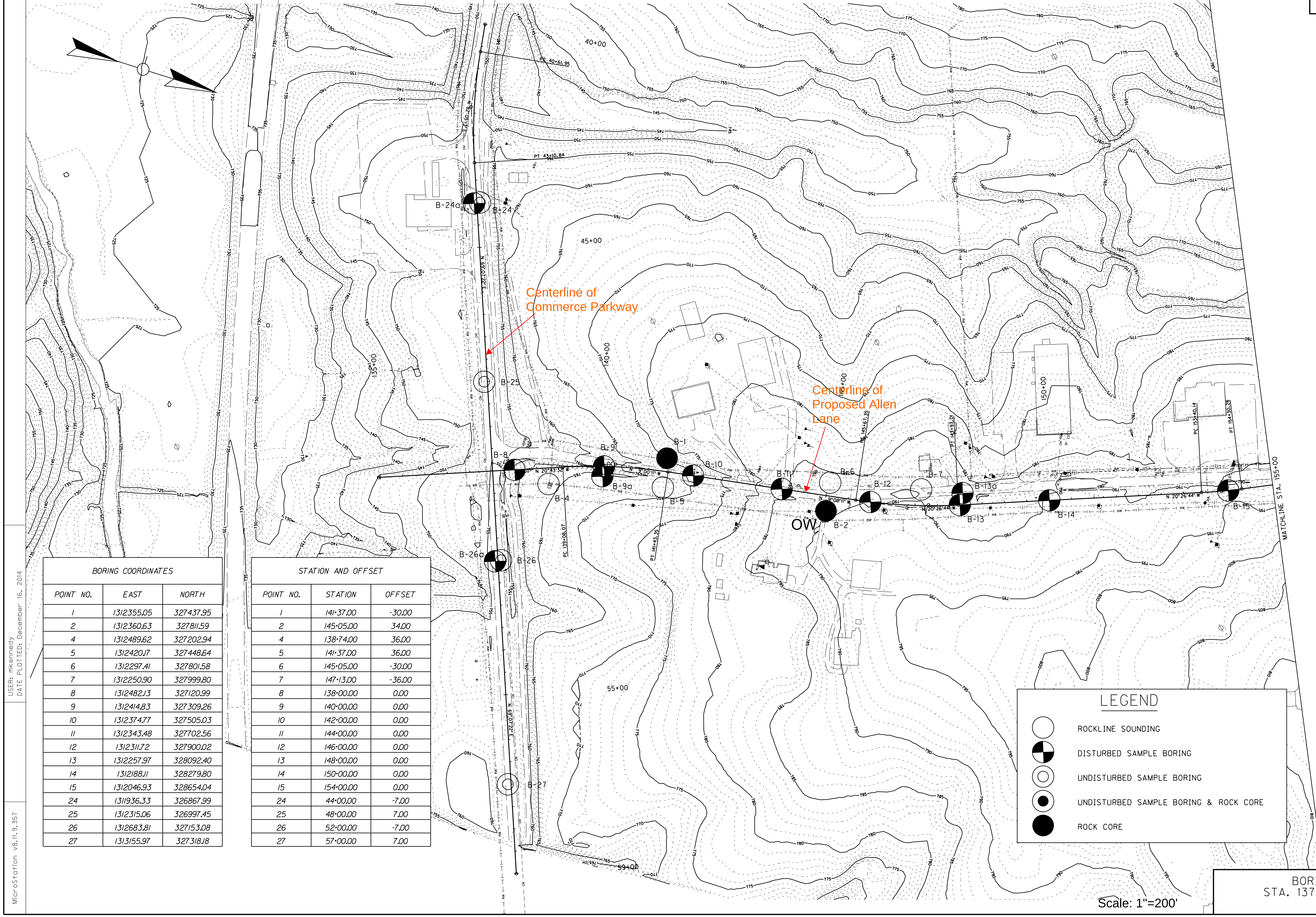
In the laboratory all samples were visually classified by a geotechnical engineer. Moisture contents of representative fine-grained soil samples were determined. A limited number of samples, considered representative of foundation materials present, were selected for performance of grain-size analyses 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.

COUNTY OF	ITEM NO.	SHEET NO.
OLDHAM	5-434.00	R__



BORING COORDINATES		
POINT NO.	EAST	NORTH
1	1312355.05	327437.95
2	1312360.63	327811.59
4	1312489.62	327202.94
5	1312420.17	327448.64
6	1312297.41	327801.58
7	1312250.90	327999.80
8	1312482.13	327120.99
9	1312414.83	327309.26
10	1312374.77	327505.03
11	1312343.48	327702.56
12	1312311.72	327900.02
13	1312257.97	328092.40
14	1312188.11	328279.80
15	1312046.93	328654.04
24	1311936.33	326867.99
25	1312315.06	326997.45
26	1312683.81	327153.08
27	1313155.97	327318.18

STATION AND OFFSET		
POINT NO.	STATION	OFFSET
1	141+37.00	-30.00
2	145+05.00	34.00
4	138+74.00	36.00
5	141+37.00	36.00
6	145+05.00	-30.00
7	147+13.00	-36.00
8	138+00.00	0.00
9	140+00.00	0.00
10	142+00.00	0.00
11	144+00.00	0.00
12	146+00.00	0.00
13	148+00.00	0.00
14	150+00.00	0.00
15	154+00.00	0.00
24	44+00.00	-7.00
25	48+00.00	7.00
26	52+00.00	-7.00
27	57+00.00	7.00

LEGEND

○	ROCKLINE SOUNDING
◕	DISTURBED SAMPLE BORING
◎	UNDISTURBED SAMPLE BORING
⦿	UNDISTURBED SAMPLE BORING & ROCK CORE
●	ROCK CORE

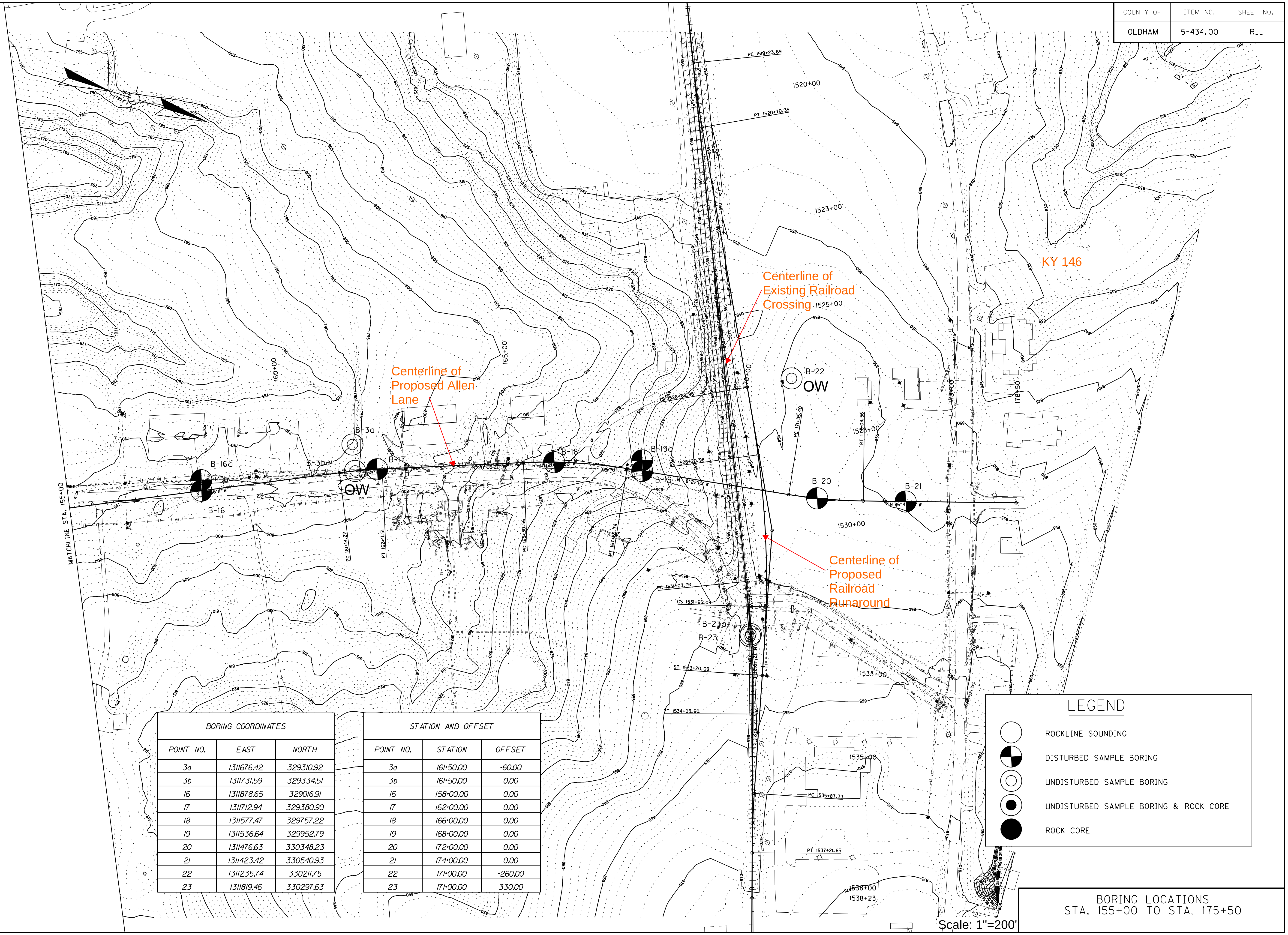
Scale: 1"=200'

BORING LOCATIONS
STA. 137+65TO STA. 155+00

USER: mkennedy
DATE PLOTTED: December 16, 2014

MicroStation v8.11.9.357

COUNTY OF	ITEM NO.	SHEET NO.
OLDHAM	5-434.00	R__



BORING COORDINATES		
POINT NO.	EAST	NORTH
3a	1311676.42	329310.92
3b	1311731.59	329334.51
16	1311878.65	329016.91
17	1311712.94	329380.90
18	1311577.47	329757.22
19	1311536.64	329952.79
20	1311476.63	330348.23
21	1311423.42	330540.93
22	1311235.74	330211.75
23	1311819.46	330297.63

STATION AND OFFSET		
POINT NO.	STATION	OFFSET
3a	161+50.00	-60.00
3b	161+50.00	0.00
16	158+00.00	0.00
17	162+00.00	0.00
18	166+00.00	0.00
19	168+00.00	0.00
20	172+00.00	0.00
21	174+00.00	0.00
22	171+00.00	-260.00
23	171+00.00	330.00

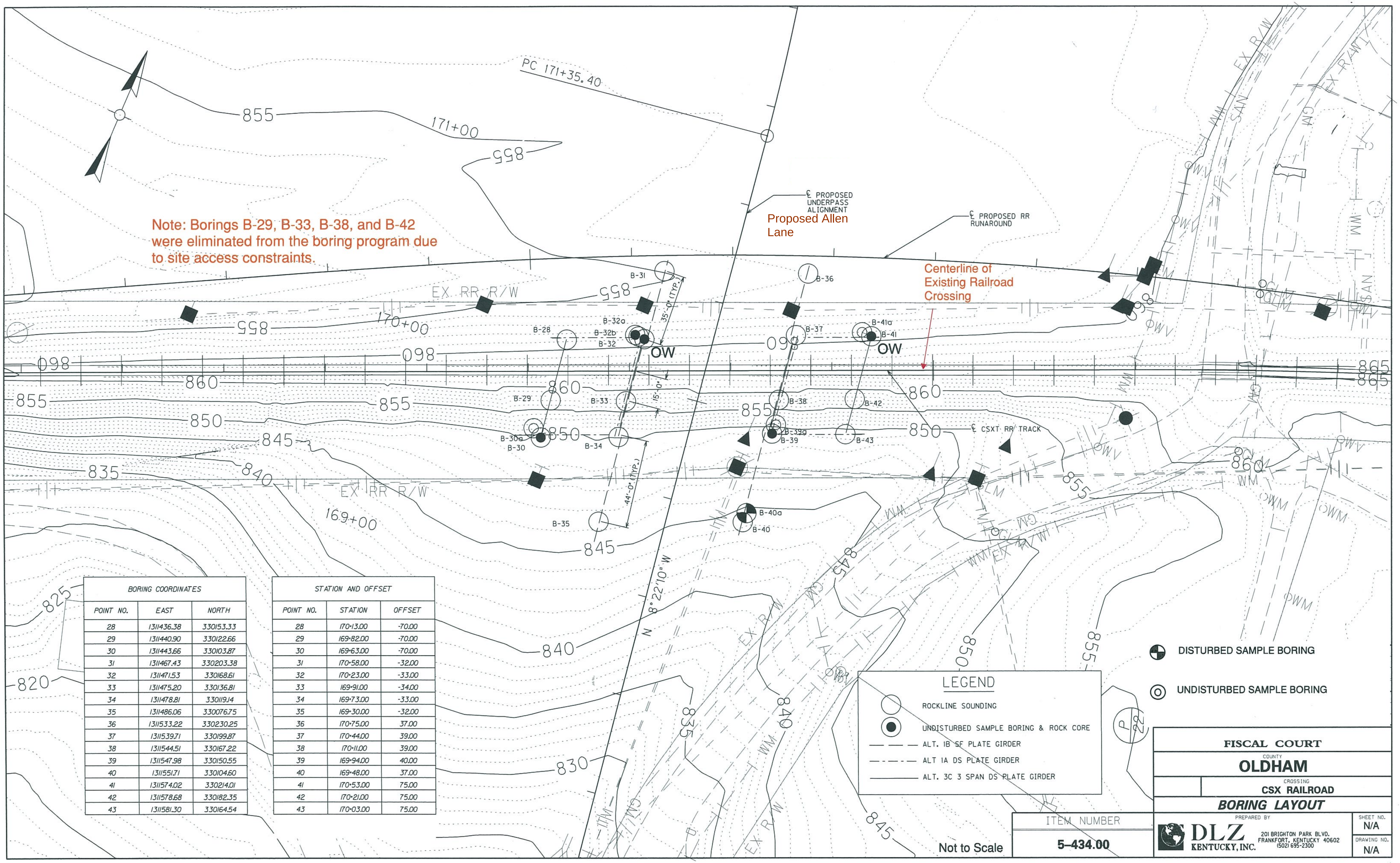
LEGEND

○	ROCKLINE SOUNDING
◕	DISTURBED SAMPLE BORING
◎	UNDISTURBED SAMPLE BORING
⊙	UNDISTURBED SAMPLE BORING & ROCK CORE
●	ROCK CORE

BORING LOCATIONS
STA. 155+00 TO STA. 175+50

USER: mkennedy
DATE PLOTTED: December 16, 2014

MicroStation v8.11.9.357



Note: Borings B-29, B-33, B-38, and B-42 were eliminated from the boring program due to site access constraints.

BORING COORDINATES		
POINT NO.	EAST	NORTH
28	1311436.38	330153.33
29	1311440.90	330122.66
30	1311443.66	330103.87
31	1311467.43	330203.38
32	1311471.53	330168.61
33	1311475.20	330136.81
34	1311478.81	330119.14
35	1311486.06	330076.75
36	1311533.22	330230.25
37	1311539.71	330199.87
38	1311544.51	330167.22
39	1311547.98	330150.55
40	1311551.71	330104.60
41	1311574.02	330214.01
42	1311578.68	330182.35
43	1311581.30	330164.54

STATION AND OFFSET		
POINT NO.	STATION	OFFSET
28	170+13.00	-70.00
29	169+82.00	-70.00
30	169+63.00	-70.00
31	170+58.00	-32.00
32	170+23.00	-33.00
33	169+91.00	-34.00
34	169+73.00	-33.00
35	169+30.00	-32.00
36	170+75.00	37.00
37	170+44.00	39.00
38	170+11.00	39.00
39	169+94.00	40.00
40	169+48.00	37.00
41	170+53.00	75.00
42	170+21.00	75.00
43	170+03.00	75.00

LEGEND

- ○ ROCKLINE SOUNDING
- ◉ UNDISTURBED SAMPLE BORING & ROCK CORE
- — — — ALT. 1B SF PLATE GIRDER
- —·—·— ALT 1A DS PLATE GIRDER
- ——— ALT. 3C 3 SPAN DS PLATE GIRDER

⊕ DISTURBED SAMPLE BORING

◎ UNDISTURBED SAMPLE BORING

Not to Scale

FISCAL COURT

COUNTY
OLDHAM

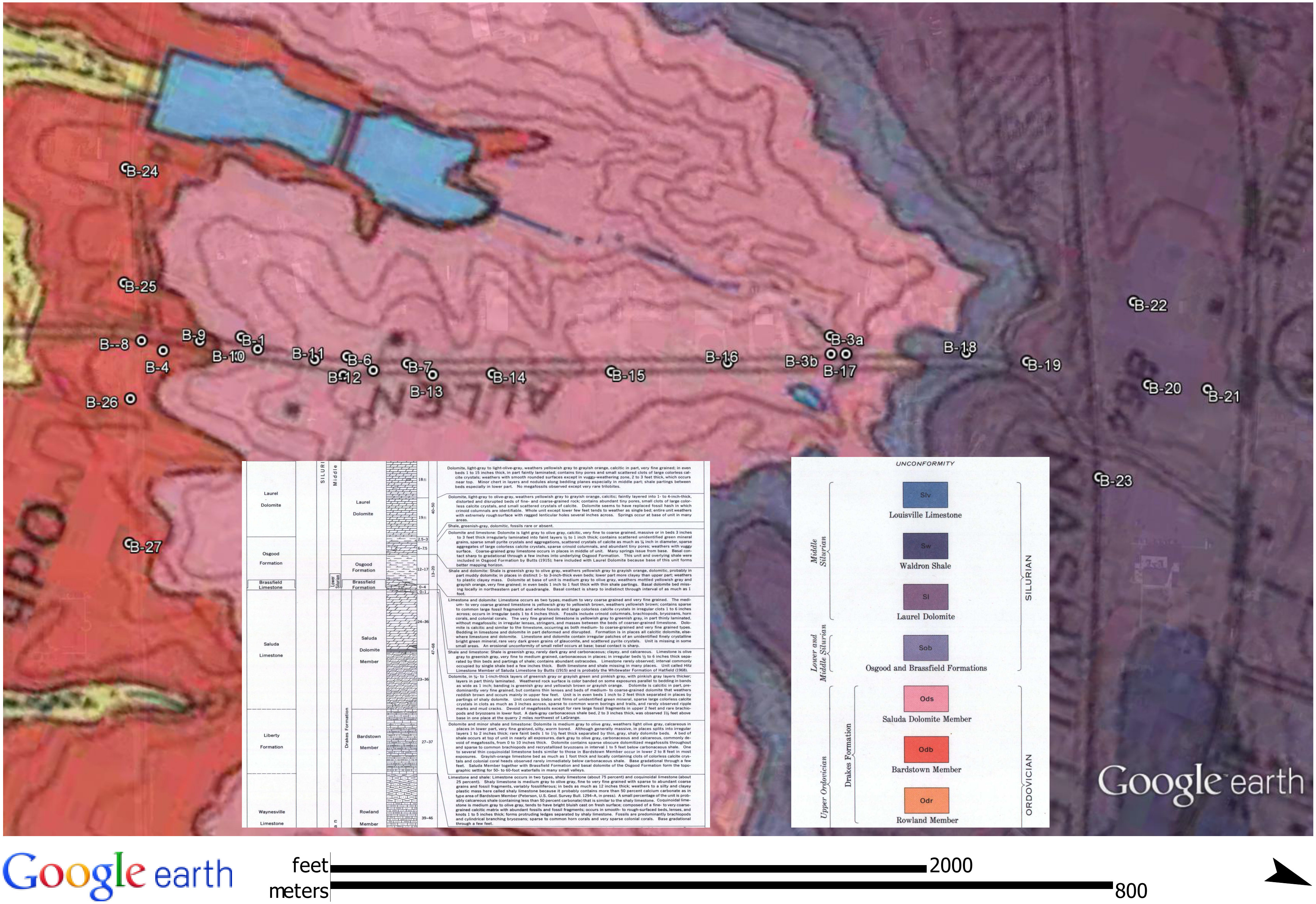
CROSSING
CSX RAILROAD

BORING LAYOUT

ITEM NUMBER
5-434.00

PREPARED BY
DLZ KENTUCKY, INC.
201 BRIGHTON PARK BLVD.
FRANKFORT, KENTUCKY 40602
(502) 695-2300

SHEET NO. N/A
DRAWING NO. N/A



Dolomite, light-gray to light-olive-gray, weathers yellowish gray to grayish orange, calcitic in part, very fine grained; in even beds 1 to 15 inches thick, in part faintly laminated; contains tiny pores and small scattered clots of large colorless calcite crystals; weathers with smooth rounded surfaces except in vuggy-weathering zone, 2 to 3 feet thick, which occurs near top. Minor chert in layers and nodules along bedding planes especially in middle part; shale partings between beds especially in lower part. No megafossils observed except very rare trilobites.

Dolomite, light-gray to olive-gray, weathers yellowish gray to grayish orange, calcitic; faintly layered into 1- to 4-inch-thick, distorted and disrupted beds of fine- and coarse-grained rock; contains abundant tiny pores, small clots of large colorless calcite crystals, and small scattered crystals of calcite. Dolomite seems to have replaced fossil hash in which crinoid columnals are identifiable. Whole unit except lower few feet tends to weather as single bed; entire unit weathers with extremely rough surface with ragged lenticular holes several inches across. Springs occur at base of unit in many areas.

Shale, greenish-gray, dolomitic, fossils rare or absent.

Dolomite and limestone: Dolomite is light gray to olive gray, calcitic, very fine to coarse grained, massive or in beds 3 inches to 3 feet thick irregularly laminated into faint layers ½ to 1 inch thick; contains scattered unidentified green mineral grains, sparse small pyrite crystals and aggregations, scattered crystals of calcite as much as ¼ inch in diameter, sparse aggregates of large colorless calcite crystals, sparse crinoid columnals, and abundant tiny pores; weathers with vuggy surface. Coarse-grained gray limestone occurs in places in middle of unit. Many springs issue from base. Basal contact sharp to gradational through a few inches into underlying Osgood Formation. This unit and overlying shale were included in Osgood Formation by Butts (1915); here included with Laurel Dolomite because base of this unit forms better mapping horizon.

Shale and dolomite: Shale is greenish gray to olive gray, weathers yellowish gray to grayish orange, dolomitic, probably in part muddy dolomite; in places in distinct 1- to 3-inch-thick even beds; lower part more clayey than upper part; weathers to plastic clayey mass. Dolomite at base of unit is medium gray to olive gray, weathers mottled yellowish gray and grayish orange, very fine grained; in even beds 1 inch to 1 foot thick with thin shale partings. Basal dolomite bed missing locally in northeastern part of quadrangle. Basal contact is sharp to indistinct through interval of as much as 1 foot.

Limestone and dolomite: Limestone occurs as two types; medium to very coarse grained and very fine grained. The medium- to very coarse grained limestone is yellowish gray to yellowish brown, weathers yellowish brown; contains sparse to common large fossil fragments and whole fossils and large colorless calcite crystals in irregular clots 1 to 6 inches across; occurs in irregular beds 1 to 4 inches thick. Fossils include crinoid columnals, brachiopods, bryozoans, horn corals, and colonial corals. The very fine grained limestone is yellowish gray to greenish gray, in part thinly laminated, without megafossils; in irregular lenses, stringers, and masses between the beds of coarser-grained limestone. Dolomite is calcitic and similar to the limestone, occurring as both medium- to coarse-grained and very fine grained types. Bedding in limestone and dolomite in part deformed and disrupted. Formation is in places all calcitic dolomite, elsewhere limestone and dolomite. Limestone and dolomite contain irregular patches of an unidentified finely crystalline bright green mineral, rare very dark green grains of glauconite, and scattered pyrite crystals. Unit is missing in some small areas. An erosional unconformity of small relief occurs at base; basal contact is sharp.

Shale and limestone: Shale is greenish gray, rarely dark gray and carbonaceous; clayey, and calcareous. Limestone is olive gray to greenish gray, very fine to medium grained, carbonaceous in places; in irregular beds ½ to 6 inches thick separated by thin beds and partings of shale; contains abundant ostracodes. Limestone rarely observed; interval commonly occupied by single shale bed a few inches thick. Both limestone and shale missing in many places. Unit called Hitz Limestone Member of Saluda Limestone by Butts (1915) and is probably the Whitewater Formation of Hatfield (1968).

Dolomite, in ¼- to 1-inch-thick layers of greenish gray or grayish green and pinkish gray, with pinkish gray layers thicker; layers in part thinly laminated. Weathered rock surface is color banded on some exposures parallel to bedding in bands as wide as 1 inch; banding is greenish gray and yellowish brown or grayish orange. Dolomite is calcitic in part, predominantly very fine grained, but contains thin lenses and beds of medium- to coarse-grained dolomite that weathers reddish brown and occurs mainly in upper few feet. Unit is in even beds 1 inch to 2 feet thick separated in places by partings of shaly dolomite. Unit contains blebs and films of unidentified green mineral, sparse large colorless calcite crystals in clots as much as 3 inches across, sparse to common worm borings and trails, and rarely observed ripple marks and mud cracks. Devoid of megafossils except for rare large fossil fragments in upper 2 feet and rare brachiopods and bryozoans in lower foot. A dark-gray carbonaceous shale bed, 2 to 3 inches thick, was observed 1½ feet above base in one place at the quarry 2 miles northwest of LaGrange.

Dolomite and minor shale and limestone: Dolomite is medium gray to olive gray, weathers light olive gray, calcareous in places in lower part, very fine grained, silty, worm bored. Although generally massive, in places splits into irregular layers 1 to 2 inches thick; rare faint beds 1 to 1½ feet thick separated by thin, gray, shaly dolomite beds. A bed of shale occurs at top of unit in nearly all exposures, dark gray to olive gray, carbonaceous and calcareous, commonly devoid of megafossils, from 0 to 10 inches thick. Dolomite contains sparse obscure dolomitized megafossils throughout and sparse to common brachiopods and recrystallized bryozoans in interval 1 to 5 feet below carbonaceous shale. One to several thin coquinoidal limestone beds similar to those in Bardstown Member occur in lower 2 to 8 feet in most exposures. Grayish-orange limestone bed as much as 1 foot thick and locally containing clots of colorless calcite crystals and colonial coral heads observed rarely immediately below carbonaceous shale. Base gradational through a few feet. Saluda Member together with Brassfield Formation and basal dolomite of the Osgood Formation form the topographic setting for 50- to 60-foot waterfalls in many small valleys.

Limestone and shale: Limestone occurs in two types, shaly limestone (about 75 percent) and coquinoidal limestone (about 25 percent). Shaly limestone is medium gray to olive gray, fine to very fine grained with sparse to abundant coarse grains and fossil fragments, variably fossiliferous; in beds as much as 12 inches thick; weathers to a silty and clayey plastic mass here called shaly limestone because it probably contains more than 50 percent calcium carbonate as in type area of Bardstown Member (Peterson, U.S. Geol. Survey Bull. 1294–A, in press). A small percentage of the unit is probably calcareous shale (containing less than 50 percent carbonate) that is similar to the shaly limestone. Coquinoidal limestone is medium gray to olive gray, tends to have bright bluish cast on fresh surface; composed of a fine- to very coarse-grained calcitic matrix with abundant fossils and fossil fragments; occurs in smooth- to rough-surfaced beds, lenses, and knots 1 to 5 inches thick; forms protruding ledges separated by shaly limestone. Fossils are predominantly brachiopods and cylindrical branching bryozoans; sparse to common horn corals and very sparse colonial corals. Base gradational through a few feet.

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

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Project ID: ***0631-000602*** Item Number: ***5-434.00***	***Oldham - Allen Lane*** ***Allen Lane***	Project Type: ***Roadway Railroad Bridge*** Project Manager: ***Eric Scott***

Hole Number ***B-28***	Immediate Water Depth ***NA***	Start Date ***09/09/2014***	Hole Type ***sounding***
Surface Elevation ***854.7'***	Static Water Depth ***NA***	End Date ***09/09/2014***	Rig_Number ***CME 750X***
Total Depth ***15.3'***	Driller ***Larry Bartlett***	Latitude(83) ***38.401029***	***GQ-***
Location ***170+32.37 77.2' Lt.***	Geologist ***Kyle Reinhart***	Longitude(83) ***-85.398030***	

Lithology		Description Overburden	Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks
Elevation	Depth	Rock Core	Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)	
5								5
10								10
15			1	15.0-15.3	0.3	50/0.30'	SPT	15
20		(Bottom of Hole 15.3') (Refusal @ 15)						20
25								25
30								30
35								35
40								40
45								45
50								50

Top of Rock = 15.0'
Elevation = 839.7'

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

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Project ID: ***0631-000602***
Item Number: ***5-434.00***

Oldham - Allen Lane
Allen Lane

Project Type: ***Roadway Railroad Bridge***
Project Manager: ***Eric Scott***

Hole Number ***B-30***
Surface Elevation ***850.2'***
Total Depth ***40.0'***
Location ***169+63.00 70.0' Lt.***

Immediate Water Depth ***10.7 (09/09/14)***
Static Water Depth ***NA***
Driller ***Keith Conrad***
Geologist ***Kyle Reinhart***

Start Date ***09/09/2014***
End Date ***09/09/2014***
Latitude(83) ***38.400845***
Longitude(83) ***-85.397967***

Hole Type ***core and sample***
Rig_Number ***CME 75***
GQ-1075
Laurel

Lithology		Description (Overburden / Rock Core)	Sample No. Std/Ky RQD	Depth (ft) Run (ft)	Rec. (ft) Rec (ft)	SPT Blows Rec (%)	Sample Type SDI (JS)	Remarks
Elevation	Depth							
850.1	0.1	**Overburden**: *Topsoil - 1".*						**Hand Penetrometer Reading (HP) = 4.5 tsf @ 1-2.5 HP = 4.0 tsf @ 3.5-5 HP = 3.5 tsf @ 6-7.5**
847.7	2.5	**Overburden**: *Hard, brown LEAN CLAY, some fine to coarse sand, damp.*	1	1.0-2.5	1.5	12-7-6	SPT	
844.2	6.0	**Overburden**: *Very stiff to hard, reddish-brown FAT CLAY, trace fine to coarse sand, trace organic, damp to moist.*	2	3.5-5.0	1.3	4-5-5	SPT	
841.7	8.5	**Overburden**: *Very stiff, reddish-brown FAT CLAY, trace fine to coarse sand, trace organic, moist.*	3	6.0-7.5	1.5	5-3-5	SPT	
840.2	10.0	**Overburden**: *Brown rock (dolomite) fragments. [@10' auger refusal].* (Begin Core)	4	8.5-9.3	0.8	38-50/0.30'	SPT	
835.4	14.8	**Dolomite**: *Brown and light gray, strong, fine to coarse grained, thinly to very thinly bedded, vuggy, fossiliferous, [@10.0'-10.8', lost recovery in broken zone; @10.8'-12.1', broken zone; @12.7', 13.6', 14.2', 14.6', 14.7', low angle, iron stained, narrow, slightly rough horizontal joints].*	38 / 38	5.0	1.9	38		15.0 **Uc=1,720 psi, Unit Wt.=144 pcf @ 13.5-13.85**
833.4	16.8	**Dolomite**: *Gray, strong, very fine to fine grained, thinly bedded, argillaceous, few pinhole vugs, [@16.3', slightly rough narrow horizontal joint].*	88 / 88	5.0	4.4	88	64@19 (4)	20.0
830.0	20.2	**Shale**: *Gray, slightly strong, (nondurable Class II), [@16.8'-16.9', clay filled zone; @17.1',17.9',18.5',19.5', clay filled, tight, slightly rough horizontal joints; @19.7'-19.8', broken zone;].*						
822.8	27.4	**Dolomite**: *Gray, strong, thinly to thickly bedded, argillaceous, vuggy, [@23.7'-24.1', iron stained, narrow, very rough vertical joint; @22.5',22.8', clay filled, tight, slightly rough horizontal joints; @25.2', slightly rough, narrow, horizontal joints].*	98 / 98	10.0	9.8	98		**Uc=12,743 psi, Unit Wt.=162 pcf @ 22.9-23.4**
811.5	38.7	**Dolomite**: *Gray, strong, thinly to thickly bedded, argillaceous, [@35.8', clay filled, tight, slightly rough horizontal joints; @29.5', slightly rough, narrow, horizontal joints].*	95 / 95	10.0	9.5	95	99@30 (5)	30.0 **Uc=12,716 psi, Unit Wt.=165 pcf @ 36.15-36.55** 40.0
810.2	40.0	**Shale**: *Gray, slightly to moderately strong; (nondurable based on visual classification), [@38.8',39.5',39.8', clay filled, tight, slightly rough horizontal joints, @38.8'-39.2', highly weathered zone w clay filling].*						
		(Bottom of Hole 40.0')						

Top of Rock = 8.5'
Elevation = 841.7'

Base Weathered Rock = 10.0'
Elevation = 840.2'

RDZ = 10.0'
Elevation = 840.2'

ABC is 20 tsf @ 10.0' (840.2')

Laurel Dolomite
Laurel Dolomite

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

Printed: 6/10/15

Project ID: **0631-000602**
Item Number: **5-434.00**

Oldham - Allen Lane
Allen Lane

Project Type: **Roadway Railroad Bridge**
Project Manager: **Eric Scott**

Hole Number *B-31*
Surface Elevation *854.5'*
Total Depth *12.8'*
Location *170+52.90 28.6' Lt.*

Immediate Water Depth *NA*
Static Water Depth *NA*
Driller *Larry Bartlett*
Geologist *Kyle Reinhart*

Start Date *09/09/2014*
End Date *09/10/2014*
Latitude(83) *38.401106*
Longitude(83) *-85.397874*

Hole Type *sounding*
Rig_Number *CME 750X*
GQ-

Lithology		Description	Overburden: Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks
Elevation	Depth		Rock Core: Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)	
			1	12.5-12.8	0.2	50/0.30'	SPT	

(Bottom of Hole 12.8')
(Refusal @ 12.5)

5 10 15 20 25 30 35 40 45 50

Top of Rock = 12.5'
Elevation = 842.0'

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

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Project ID: **0631-000602** Item Number: **5-434.00**	**Oldham - Allen Lane** **Allen Lane**	Project Type: **Roadway Railroad Bridge** Project Manager: **Eric Scott**

Hole Number *B-32*	Immediate Water Depth *3.9 (09/09/14)*	Start Date *09/09/2014*	Hole Type *core and sample*
Surface Elevation *854.7'*	Static Water Depth *NA*	End Date *09/09/2014*	Rig_Number *CME 750X*
Total Depth *40.0'*	Driller *Larry Bartlett*	Latitude(83) *38.401074*	*GQ-1075* *Laurel*
Location *170+43.21 41.7' Lt.*	Geologist *Kyle Reinhart*	Longitude(83) *-85.397914*	

Lithology		Description (Overburden / Rock Core)
Elevation	Depth	
854.5	0.1	**Overburden**: *Topsoil - 1.5".*
848.7	6.0	**Overburden**: *Very stiff to hard, reddish brown FAT CLAY, trace fine to medium sand, trace organic; damp to moist.*
846.2	8.5	**Overburden**: *Hard, reddish brown FAT CLAY, trace fine to medium sand, trace organic; damp.*
843.7	11.0	**Overburden**: *Hard, reddish brown FAT CLAY, trace fine to medium sand, trace organic; damp to moist.*
839.7	15.0	**Overburden**: *Brown rock (dolomite) fragments, [@15', auger refusal].* (Begin Core)
828.6	26.1	**Dolomite**: *Brown to gray, strong, fine to coarse grained, very thinly to thinly bedded, vuggy, fossiliferous;, [@15'-20', lost recovery in broken zones; @21.3',21.8',22.8',23.4',25.9', open, iron stained, very rough horizontal joints].*
825.2	29.5	**Dolomite**: *Gray, strong, very fine grained, thinly bedded, argillaceous, pin hole vugs, [@ 27'-27.5', 28.2-29.3' olive to dark gray calcareous shale bands; @26',26.1',26.5',26.9',28',28.1',28.3',28.5',29', slightly rough horizontal joints].*
819.5	35.2	**Dolomite**: *Gray, strong, very fine grained, very thinly to thickly bedded, argillaceous, [@31.1',33.7',36.4',36.8',39', narrow, slightly rough horizontal joints; @31.8',32.1', 60-degree tight, slightly rough joint].*
818.1	36.6	**Dolomite**: *Gray, strong, very fine grained, very thin to thinly bedded, argillaceous.*
817.5	37.2	**Shale**: *Gray, strong, calcareous (nondurable based on visual classification).*
814.7	40.0	**Dolomite**: *Gray, strong, very fine grained, laminated to thinly bedded, argillaceous.*
		(Bottom of Hole 40.0')

Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type
Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)
1	1.0-2.5	1.0	3-4-5	SPT
2	3.5-5.0	1.3	5-7-8	SPT
3	6.0-7.5	1.3	4-7-10	SPT
4	8.5-10.0	1.5	8-5-10	SPT
5	11.0-12.5	0.6	1-2-47	SPT
6	13.5-13.8	0.2	50/0.30'	SPT
0 / 0	5.0	0.8	15	
28 / 28	5.0	3.6	72	
74 / 74	5.0	5.0	100	
94 / 94	5.0	5.0	100	
100 / 100	5.0	5.0	100	

Remarks

Hand Penetrometer Reading (HP) = 3.5 tsf @ 1-2.5
HP = 4.5 tsf @ 3.5-5
HP = 4.5 tsf @ 6-7.5
HP = 4.5 tsf @ 8.5-10

20.0

Uc=2,579 psi, Unit Wt.=149 pcf @ 21.4-21.8

25.0

30.0 **Uc=15,748, Unit Wt.=167 pcf @ 30.05-30.45**

35.0

40.0

Top of Rock = 11.0'
Elevation = 843.7'

Base Weathered Rock = 20.0'
Elevation = 834.7'

RDZ = 20.0'
Elevation = 834.7'

ABC is 20 tsf @ 20.0' (834.7')

Laurel Dolomite
Laurel Dolomite

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

Printed: 9/3/15

Project ID: **0631-000602**
Item Number: **5-434.00**

Oldham - Allen Lane
Allen Lane

Project Type: **Roadway Railroad Bridge**
Project Manager: **Eric Scott**

Hole Number **B-32b**
Surface Elevation **854.7'**
Total Depth **19.6'**
Location **170+43.21 41.7' Lt.**

Immediate Water Depth **NA**
Static Water Depth **NA**
Driller **Larry Bartlett**
Geologist **Kyle Reinhart**

Start Date **09/10/2014**
End Date **09/10/2014**
Latitude(83) **38.401074**
Longitude(83) **-85.397914**

Hole Type **sounding**
Rig_Number **CME 750X**
GQ-

Lithology		Description	Overburden: Sample No. / Rock Core: Std/Ky RQD	Overburden: Depth (ft) / Rock Core: Run (ft)	Overburden: Rec. (ft) / Rock Core: Rec (ft)	Overburden: SPT Blows / Rock Core: Rec (%)	Overburden: Sample Type / Rock Core: SDI (JS)	Remarks
Elevation	Depth							
	20		1	19.5-19.6	0.1	50/0.10'	SPT	
		(Bottom of Hole 19.6') (Refusal @ 19.5)						

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

Top of Rock = 19.5'
Elevation = 835.2'

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

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Project ID: **0631-000602** Item Number: **5-434.00**	**Oldham - Allen Lane** **Allen Lane**	Project Type: **Roadway Railroad Bridge** Project Manager: **Eric Scott**

Hole Number **B-34**	Immediate Water Depth **NA**	Start Date **09/09/2014**	Hole Type **sounding**
Surface Elevation **849.9'**	Static Water Depth **NA**	End Date **09/09/2014**	Rig_Number **CME 75**
Total Depth **11.2'**	Driller **Keith Conrad**	Latitude(83) **38.400888**	**GQ-**
Location **169+73.00 33.0' Lt.**	Geologist **Kyle Reinhart**	Longitude(83) **-85.397845**	

Lithology		Description	Overburden	Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks
Elevation	Depth		Rock Core	Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)	
				1	11.0-11.2	0.2	50/0.20'	SPT	
		(Bottom of Hole 11.2') (Refusal @ 11)							

5 10 15 20 25 30 35 40 45 50

Top of Rock = 11.0'
Elevation = 838.9'

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

Printed: 6/10/15

Project ID: **0631-000602** Item Number: **5-434.00**	**Oldham - Allen Lane** **Allen Lane**	Project Type: **Roadway Railroad Bridge** Project Manager: **Eric Scott**

Hole Number **B-35**	Immediate Water Depth **8.3 (09/04/14)**	Start Date **09/04/2014**	Hole Type **sounding**
Surface Elevation **846.0'**	Static Water Depth **4.8 (09/05/14)**	End Date **09/05/2014**	Rig_Number **CME 75**
Total Depth **10.7'**	Driller **Keith Conrad**	Latitude(83) **38.400784**	**GQ-**
Location **169+32.30 18.1' Lt.**	Geologist **Kyle Reinhart**	Longitude(83) **-85.397772**	

Lithology		Description	Overburden	Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks	
Elevation	Depth		Rock Core	Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)		
5										5
10										10
				1	10.5-10.7	0.1	50/0.20'	SPT		
15		(Bottom of Hole 10.7') (Refusal @ 10.5)								15
20										20
25										25
30										30
35										35
40										40
45										45
50										50

Top of Rock = 10.5'
Elevation = 835.5'

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

Printed: 6/10/15

Project ID: **0631-000602** Item Number: **5-434.00**	**Oldham - Allen Lane** **Allen Lane**	Project Type: **Roadway Railroad Bridge** Project Manager: **Eric Scott**

Hole Number **B-36**	Immediate Water Depth **NA**	Start Date **09/09/2014**	Hole Type **sounding**
Surface Elevation **855.4'**	Static Water Depth **NA**	End Date **09/10/2014**	Rig_Number **CME 750X**
Total Depth **13.9'**	Driller **Larry Bartlett**	Latitude(83) **38.401181**	**GQ-**
Location **170+69.41 39.9' Rt.**	Geologist **Kyle Reinhart**	Longitude(83) **-85.397647**	

Lithology		Description	Overburden	Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks
Elevation	Depth		Rock Core	Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)	
				1	13.5-13.9	0.4	50/0.40'	SPT	
		(Bottom of Hole 13.9') (Refusal @ 13.5)							

5 10 15 20 25 30 35 40 45 50

Top of Rock = 13.5'
Elevation = 841.9'

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

Printed: 6/10/15

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Project ID: **0631-000602** Item Number: **5-434.00**	**Oldham - Allen Lane** **Allen Lane**	Project Type: **Roadway Railroad Bridge** Project Manager: **Eric Scott**

Hole Number **B-37**	Immediate Water Depth **NA**	Start Date **09/09/2014**	Hole Type **sounding**
Surface Elevation **855.9'**	Static Water Depth **NA**	End Date **09/09/2014**	Rig_Number **CME 750X**
Total Depth **15.4'**	Driller **Larry Bartlett**	Latitude(83) **38.401151**	**GQ-**
Location **170+59.24 34.7' Rt.**	Geologist **Kyle Reinhart**	Longitude(83) **-85.397660**	

Lithology		Description	Overburden	Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks
Elevation	Depth		Rock Core	Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)	
				1	14.5-15.4	0.8	37-50/0.40'	SPT	
		(Bottom of Hole 15.4') (Refusal @ 14.5)							

5

10

15

20

25

30

35

40

45

50

Top of Rock = 14.5'
Elevation = 841.4'

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

Printed: 9/2/15

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Project ID: **0631-000602** Item Number: **5-434.00**	**Oldham - Allen Lane** **Allen Lane**	Project Type: **Roadway Railroad Bridge** Project Manager: **Eric Scott**

Hole Number **B-39**	Immediate Water Depth **8.5 (09/09/14)**	Start Date **09/09/2014**	Hole Type **core and sample**
Surface Elevation **849.1'**	Static Water Depth **NA**	End Date **09/09/2014**	Rig_Number **CME 75**
Total Depth **40.0'**	Driller **Keith Conrad**	Latitude(83) **38.400976**	**GQ-1075** **Laurel**
Location **169+94.00 40.0' Rt.**	Geologist **Kyle Reinhart**	Longitude(83) **-85.397605**	

Lithology		Description — Overburden	Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks
Elevation	Depth	Rock Core	Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)	
849.0	0.1	**Overburden**: *Topsoil - 2".*						**Hand Penetrometer Reading (HP) = 3.5 tsf @ 1-2.5 HP = 1.5 tsf @ 3.5-5**
843.1	6.0	**Overburden**: *Stiff to very stiff, reddish brown FAT CLAY, trace fine to medium sand, trace organic; moist.*	1 2	1.0-2.5 3.5-5.0	1.0 1.2	1-3-5 2-3-5	SPT SPT	
839.1	10.0	**Overburden**: *Brown rock (dolomite) fragments, [@10', auger refusal].* (Begin Core)	3 4	6.0-7.5 8.5-8.8	1.4 0.2	3-5-15 50/0.30'	SPT SPT	
835.2	13.9	**Dolomite**: *Brown, strong, fine to coarse grained, very thinly to thinly bedded, agillaceous, vuggy, fossiliferous, [@10'-10.6', lost recovery in broken zone; @10.6'-12', 12.2'-12.5', broken zones].*	40 / 40	5.0	4.4	88		**Uc=3,518 psi, Unit Wt.=148 pcf @ 12.65-13**
833.5	15.6	**Dolomite**: *Gray, strong, very fine grained, thinly to thickly bedded, argillaceous, [@14.3'-14.4', broken zone, @15'-15.1', lost recovery in broken zone].*						15.0
831.1	18.0	**Shale**: *Dark gray, moderately strong, (nondurable Class II), [@15.7',15.9',16',17.2', slightly rough, narrow, clay filled horizontal joints, @17.5'-17.6', broken zone].*	84 / 84	5.0	4.8	96	54@17 (2)	20.0
823.3	25.8	**Dolomite**: *Light to dark gray, strong, very fine grained, thinly to thickly bedded, argillaceous, pin hole vugs, [@21.5',21.7',22.9', slightly rough, narrow clay filled horizontal joints].*	100 / 100	10.0	10.0	100	95@26 (5)	**Uc=7,676 psi, Unit Wt.=159 pcf @ 20-20.4** **Shale reclassified as Dolomite @ 26.3-27.5** 30.0
811.6	37.5	**Dolomite**: *Light to dark gray, strong, very fine grained, laminated to thinly bedded, argillaceous.*	98 / 98	10.0	10.0	100		**Uc-7,579 psi, Unit Wt.=166 pcf @ 31-31.4**
809.1	40.0	**Shale**: *Dark gray, moderately strong, thinly laminated, calcareous; (nondurable based on visual cllassification), [@37.7'-38.4', clay filled zone, @38.9'-39.1', broken zone].*						40.0
		(Bottom of Hole 40.0')						

Top of Rock = 6.0' Elevation = 843.1'	Base Weathered Rock = 10.0' Elevation = 839.1'	RDZ = 10.0' Elevation = 839.1'	ABC is 20 tsf @ 10.0' (839.1')	Laurel Dolomite Laurel Dolomite

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

Printed: 6/10/15

Page 1 of 1

Project ID: **0631-000602** Item Number: **5-434.00**	**Oldham - Allen Lane** **Allen Lane**	Project Type: **Roadway Railroad Bridge** Project Manager: **Eric Scott**

Hole Number **B-40**	Immediate Water Depth **5.2 (09/04/14)**	Start Date **09/04/2014**	Hole Type **sounding**
Surface Elevation **843.7'**	Static Water Depth **3.7 (09/05/14)**	End Date **09/05/2014**	Rig_Number **CME 75**
Total Depth **6.8'**	Driller **Keith Conrad**	Latitude(83) **38.400848**	**GQ-**
Location **169+45.30 49.1' Rt.**	Geologist **Kyle Reinhart**	Longitude(83) **-85.397547**	

	Lithology		Description	Overburden	Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks	
	Elevation	Depth		Rock Core	Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)		
5											5
					1	6.5-6.8	0.1	50/0.30'	SPT		
10											10
			(Bottom of Hole 6.8') (Refusal @ 6.5)								
15											15
20											20
25											25
30											30
35											35
40											40
45											45
50											50

Top of Rock = 6.5'
Elevation = 837.2'

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

Printed: 6/10/15

Page 1 of 1

Project ID: ***0631-000602*** Item Number: ***5-434.00***	***Oldham - Allen Lane*** ***Allen Lane***	Project Type: ***Roadway Railroad Bridge*** Project Manager: ***Eric Scott***

Hole Number ***B-41***	Immediate Water Depth ***NA***	Start Date ***09/10/2014***	Hole Type ***core and sample***
Surface Elevation ***856.1'***	Static Water Depth ***NA***	End Date ***09/10/2014***	Rig_Number ***CME 750X***
Total Depth ***40.0'***	Driller ***Larry Bartlett***	Latitude(83) ***38.401193***	***GQ-1075*** ***Laurel***
Location ***170+68.91 72.4' Rt.***	Geologist ***Kyle Reinhart***	Longitude(83) ***-85.397535***	

Lithology		Description Overburden	Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks
Elevation	Depth	Rock Core	Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)	
856.0	0.1	**Overburden**: *Topsoil - 1.5".*						
852.6	3.5	**Overburden**: *Medium dense, brown to dark brown SILT with fine to coarse sand, little clay, trace fine gravel, trace organic; damp*, *[Probable Fill].*	1	1.0-2.5	0.8	6-8-9	SPT	
842.6	13.5	**Overburden**: *Hard, reddish brown FAT CLAY, little fine to coarse sand, trace fine gravel, trace organic, damp to moist*, *[@11'-12.5', stiff].*	2	3.5-5.0	1.0	10-14-12	SPT	**Hand Penetrometer Reading (HP) = 4.5 tsf @ 3.5-5** **HP = 4.5 tsf @ 6-7.5** **HP = 4.0 tsf @ 8.5-10** **HP = 1.5 tsf @ 11-12.5**
			3	6.0-7.5	0.5	3-7-6	SPT	
			4	8.5-10.0	0.9	2-5-7	SPT	
			5	11.0-12.5	0.7	2-4-6	SPT	
839.1	17.0	**Overburden**: *Brown rock (dolomite) fragments*, *[@17', auger refusal].* (Begin Core)	6	13.5-13.9	0.4	50/0.40'	SPT	
833.3	22.8	**Dolomite**: *Brown to gray, strong, fine to coarse grained, thinly bedded, vuggy*, *[@17.6',17.9',20.1',20.5',22.8', open, slightly rough horizontal joints; @19.1'-19.3', 21.1'-21.5', broken zones].*	82 / 82	5.0	5.0	100		**Uc=3,341 psi, Unit Wt.=154 pcf @ 17.2-17.6** 22.0
830.6	25.5	**Dolomite**: *Gray, strong, fine to very fine grained, thinly bedded, argillaceous*, *[@23.7',23.9', slightly rough narrow horizontal joints].*	96 / 96	5.0	5.0	100		27.0
827.4	28.7	**Shale**: *Dark gray, moderately strong, (nondurable Class I)*, *[@25.4'-25.5',28.3'-28.7', broken zones, @27.2',27.3', tight, narrow, slightly rough horizontal joints].*						
821.8	34.3	**Dolomite**: *Gray to dark gray, strong, very fine to fine grained, thinly bedded, argillaceous, pin hole vugs*, *[@30',30.2',30.6',31.2',31.4', tight, narrow, slightly rough horizontal joints, @33.4' 25-degree joint].*	90 / 90	10.0	10.0	100	80@28 (3)	**Uc=10,548 psi, Unit Wt.=164 pcf @ 29.5-29.95** 37.0
816.1	40.0	**Dolomite**: *Gray to dark gray, strong, very fine to fine grained, thinly bedded, argillaceous*, *[@39.5'-40', broken zones].*	100 / 100	3.0	3.0	100		40.0
		(Bottom of Hole 40.0')						

Top of Rock = 13.5'
Elevation = 842.6'

Base Weathered Rock = 17.0'
Elevation = 839.1'

RDZ = 17.0'
Elevation = 839.1'

ABC is 20 tsf @ 17.0' (839.1')

Laurel Dolomite
Laurel Dolomite

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

GEOLOGIST'S SUBSURFACE LOG

Printed: 6/10/15

Project ID: **0631-000602** Item Number: **5-434.00**	**Oldham - Allen Lane** **Allen Lane**	Project Type: **Roadway Railroad Bridge** Project Manager: **Eric Scott**

Hole Number **B-43**	Immediate Water Depth **NA**	Start Date **09/10/2014**	Hole Type **sounding**
Surface Elevation **849.8'**	Static Water Depth **NA**	End Date **09/10/2014**	Rig_Number **CME 75**
Total Depth **10.3'**	Driller **Keith Conrad**	Latitude(83) **38.401016**	**GQ-**
Location **170+03.00 75.0' Rt.**	Geologist **Kyle Reinhart**	Longitude(83) **-85.397490**	

Lithology		Description	Overburden	Sample No.	Depth (ft)	Rec. (ft)	SPT Blows	Sample Type	Remarks
Elevation	Depth		Rock Core	Std/Ky RQD	Run (ft)	Rec (ft)	Rec (%)	SDI (JS)	
				1	10.0-10.3	0.1	50/0.30'	SPT	
		(Bottom of Hole 10.3') (Refusal @ 10)							

5
10
15
20
25
30
35
40
45
50

Top of Rock = 10.0'
Elevation = 839.8'

Drilling Firm: DLZ
For: Division of Structural Design
Geotechnical Branch

Printed: 1/9/15

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Summary of Rock Soundings

Project ID: ***0631-000602***
Item Number: ***5-434.00***

County: ***Oldham***
Route: ***Allen Lane***

Project Type: ***Roadway Railroad Bridge***
Project Manager: ***Eric Scott***

Hole Number	Method	Latitude(83)	Longitude(83)	Station	Offset (ft)	Surface Elevation (ft)	Hole Depth (ft)	Refusal Depth / Elevation (ft)	Refusal Lithology	Notes
B-4	auger	38.392920	-85.394154	138+71.82	46.9' Rt.	764.1	8.4	8.0 / 756.1	weathered dolomite	
B-5	auger	38.393542	-85.394424	141+19.03	38.9' Rt.	768.1	7.8	7.5 / 760.6	weathered dolomite	
B-6	auger	38.394552	-85.394889	145+05.00	30.0' Lt.	787.3	9.4	8.5 / 778.8	weathered dolomite	
B-7	auger	38.395093	-85.395066	147+12.99	37.8' Lt.	789.4	7.3	7.0 / 782.4	weathered dolomite	
B-35	auger	38.400784	-85.397772	169+32.30	18.1' Lt.	846.0	10.7	10.5 / 835.5	weathered dolomite	
B-40	auger	38.400848	-85.397547	169+45.30	49.1' Rt.	843.7	6.8	6.5 / 837.2	weathered dolomite	
B-34	auger	38.400888	-85.397845	169+73.00	33.0' Lt.	849.9	11.2	11.0 / 838.9	weathered dolomite	
B-43	auger	38.401016	-85.397490	170+03.00	75.0' Rt.	849.8	10.3	10.0 / 839.8	weathered dolomite	
B-28	auger	38.401029	-85.398030	170+32.37	77.2' Lt.	854.7	15.3	15.0 / 839.7	weathered dolomite	
B-32b	auger	38.401074	-85.397914	170+43.21	41.7' Lt.	854.7	19.6	19.5 / 835.2	weathered dolomite	
B-31	auger	38.401106	-85.397874	170+52.90	28.6' Lt.	854.5	12.8	12.5 / 842.0	weathered dolomite	
B-37	auger	38.401151	-85.397660	170+59.24	34.7' Rt.	855.9	15.4	14.5 / 841.4	weathered dolomite	
B-36	auger	38.401181	-85.397647	170+69.41	39.9' Rt.	855.4	13.9	13.5 / 841.9	weathered dolomite	

Date: 10-7-14

Summary of Top of Bedrock Elevations, Auger Refusal Elevation, and Bottom of RDZ Elevations

5-434.00 **OLDHAM County**

Boring No.	Bridge Location (Boring Type)	Station	Offset	Surveyed Ex. Grade @ Boring	Depth to Top of Rock, ft	Top of Rock Elevation, ft	Depth to Auger Refusal, ft	Elevation of Auger Refusal, ft	Bottom of RDZ Elevation	Depth to Immediate Water Level, ft	Elevation of Immediate Water Level, ft	Depth to 24hr Water Level, ft	24hr Water Level Elevation, ft	Water Level in Observation Well*	Boring No.
28	Drilled Shaft Supporting Abutment (Sounding)	170+32	-77.15	854.7	15.0	839.7	15.0	839.7	839.4	NR	NR	--	--	--	28
30	Drilled Shaft Supporting Abutment (Rock Core)	169+63	-70	850.2	8.5	841.7	10.0	840.2	840.2	10.7	839.5	--	--	--	30
31	Drilled Shaft Supporting Abutment/Pier (Sounding)	170+53	-28.6	854.5	12.5	842.0	12.5	842.0	841.7	NR	NR	NFW	--	--	31
32	Drilled Shaft Supporting Abutment/Pier (Rock Core)	170+43	-41.72	854.7	11.0	843.7	15.0	839.7	834.7	3.9	850.8	--	--	NR	32
32b	Drilled Shaft Supporting Abutment/Pier (Sounding)	170+43	-41.72	854.7	19.5	835.2	19.5	835.2	835.1	--	--	--	--	--	32b
34	Drilled Shaft Supporting Abutment/Pier (Sounding)	169+73	-33	849.9	11.0	838.9	11.0	838.9	838.7	NR	NR	--	--	--	34
35	Drilled Shaft Supporting Abutment/Pier (Sounding)	169+32	-18.13	846.0	10.5	835.5	10.5	835.5	835.3	8.3	837.7	4.8	841.2	--	35
36	Drilled Shaft Supporting Abutment/Pier (Sounding)	170+69	39.92	855.4	13.5	841.9	13.5	841.9	841.5	NR	NR	NFW	--	--	36
37	Drilled Shaft Supporting Abutment/Pier (Sounding)	170+59	34.7	855.9	14.5	841.4	14.5	841.4	840.5	NR	NR	--	--	--	37
39	Drilled Shaft Supporting Abutment/Pier (Rock Core)	169+94	40	849.1	6.0	843.1	10.0	839.1	839.1	8.5	840.6	--	--	--	39
40	Drilled Shaft Supporting Abutmenting/Pier (Sounding)	169+45	49.12	843.7	6.5	837.2	6.5	837.2	836.9	5.2	838.5	3.7	840.0	--	40
41	Drilled Shaft Supporting Abutment (Rock Core)	170+69	72.35	856.1	13.5	842.6	17.0	839.1	839.1	NR	NR	--	--	NR	41
43	Drilled Shaft Supporting Abutment (Sounding)	170+03	75	849.8	10.0	839.8	10.0	839.8	839.5	NR	NR	--	--	--	43

NFW: No Free Water

*Date Measured = January 7, 2015

NR: Measured, but none reported

M:\proj\0631\0006\02\Report (bridge)\REV Submittal (June 2015)\Oldham Co Underpass Top of Rock Elev & Bottom of RDZ Elev REV.xlsx

APPENDIX II

Summary of Laboratory Test Results
Laboratory Testing Results

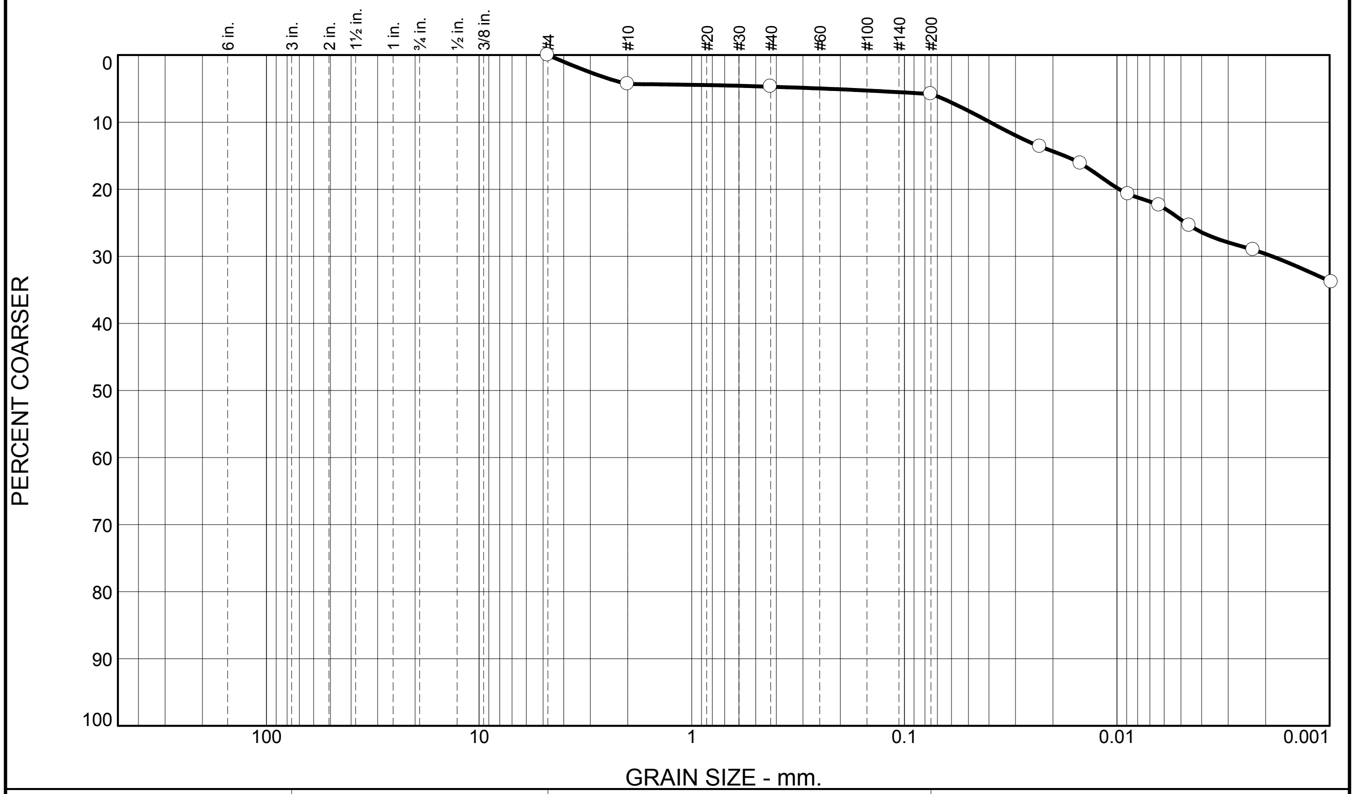
Summary of Laboratory Test Results - Index Properties

Boring Number	Sample Number	Depth (feet)	USCS Class.	AASHTO Class.	N Value	N60 Value	% AGG	% C. Sand	% M. Ssand	% F. Sand	% Silt	% Clay	LL	PL	PI	Moisture Content %	Liquidity Index	Soil Activity
B-30	S-1	1	-	-	13	18	-	-	-	-	-	-	-	-	-	15.17		
B-30	S-2	3.5	-	-	10	14	-	-	-	-	-	-	-	-	-	29.81		
B-30	S-3	6	CH	A-7-5(49)	8	11	0	4.3	0.4	1.1	23.8	70.4	78	35	43	44.19	0.21	0.62
B-31	S-1	12.5			R	R												
B-32	S-1	1	-	-	9	10	-	-	-	-	-	-	-	-	-	19.02		
B-32	S-2	3.5	-	-	15	17	-	-	-	-	-	-	-	-	-	19.29		
B-32	S-3	6	CH	A-7-5(39)	17	19	0	0	0.6	1.3	37.4	60.7	64	31	33	27.79	-0.10	0.55
B-32	S-4	8.5	-	-	15	17	-	-	-	-	-	-	-	-	-	34.28		
B-32	S-5	11			49	55												
B-34	S-1	11			R	R												
B-35	S-1	10.5			R	R												
B-36	S-1	13.5			R	R												
B-37	S-1	14.5			SR	50+												
B-39	S-1	1	-	-	8	11	-	-	-	-	-	-	-	-	-	26.19		
B-39	S-2	3.5	CH	A-7-6(46)	8	11	0	0.6	3.5	4.4	30	61.5	68	23	45	38.89	0.35	0.74
B-39	S-3	6			20	27												
B-40	S-1	6.5			R	R												
B-41	S-1	1	ML	A-4(0)	17	19	0.1	4	14.8	8.4	60.2	12.5	NP	NP	NP	13.43	NP	NP
B-41	S-2	3.5	-	-	26	29	-	-	-	-	-	-	-	-	-	13.39		
B-41	S-3	6	-	-	13	14	-	-	-	-	-	-	-	-	-	26.34		
B-41	S-4	8.5	-	-	12	13	-	-	-	-	-	-	-	-	-	29.72		
B-41	S-5	13.5	-	-	10	11	-	-	-	-	-	-	-	-	-	40.84		
B-43	S-1	10			R	R												

Summary of Laboratory Test Results
Unconfined Compressive Strength of Intact Rock Core Specimen

Boring Number	Run Number	Depth (feet)	Elevation (feet)	Description	Unit Weight, pcf	Unconfined Compressive Strength, psi
B-30	1	13.50 - 13.85	836.7 - 836.4	Brown to light gray dolomite	144.1	1,720
B-30	3	22.90 - 23.40	827.3 - 826.8	Gray dolomite	161.9	12,743
B-30	4	36.15 -36.55	814.1 - 813.7	Gray dolomite	165.1	12,716
B-32	2	21.40 - 21.80	833.3 - 832.9	Brown to gray dolomite	149.3	2,579
B-32	3	30.02 - 30.45	824.7 - 824.3	Gray dolomite	166.5	15,748
B-39	1	12.65 - 13.00	836.5 - 836.1	Brown dolomite	148.5	3,518
B-39	3	20.00 - 20.40	829.1 - 828.7	Light to dark gray dolomite	159.2	7,676
B-39	4	31.00 - 31.40	818.1 - 817.7	Light to dark gray dolomite	166.1	7,579
B-41	1	17.20 - 17.60	838.9 - 838.5	Brown to gray dolomite	154.2	3,341
B-41	3	29.50 - 29.95	826.6 - 826.2	Gray to dark gray dolomite	163.8	10,548
				minimum =	144.1	1,720
				maximum =	166.5	15,748
				Average =	157.9	7,817

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	4.3	0.4	1.1	23.8	70.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	95.7		
#40	95.3		
#200	94.2		

* (no specification provided)

Material Description

fat clay

Atterberg Limits

PL= 35 LL= 78 PI= 43

Coefficients

D_{90}= 0.0396 D_{85}= 0.0177 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-5(49)

Remarks

Moisture Content = 44.2%
Specific Gravity = 2.80

Source of Sample: B-30 **Depth:** 6.0'-7.5'
Sample Number: S-3

Date: 10-6-14

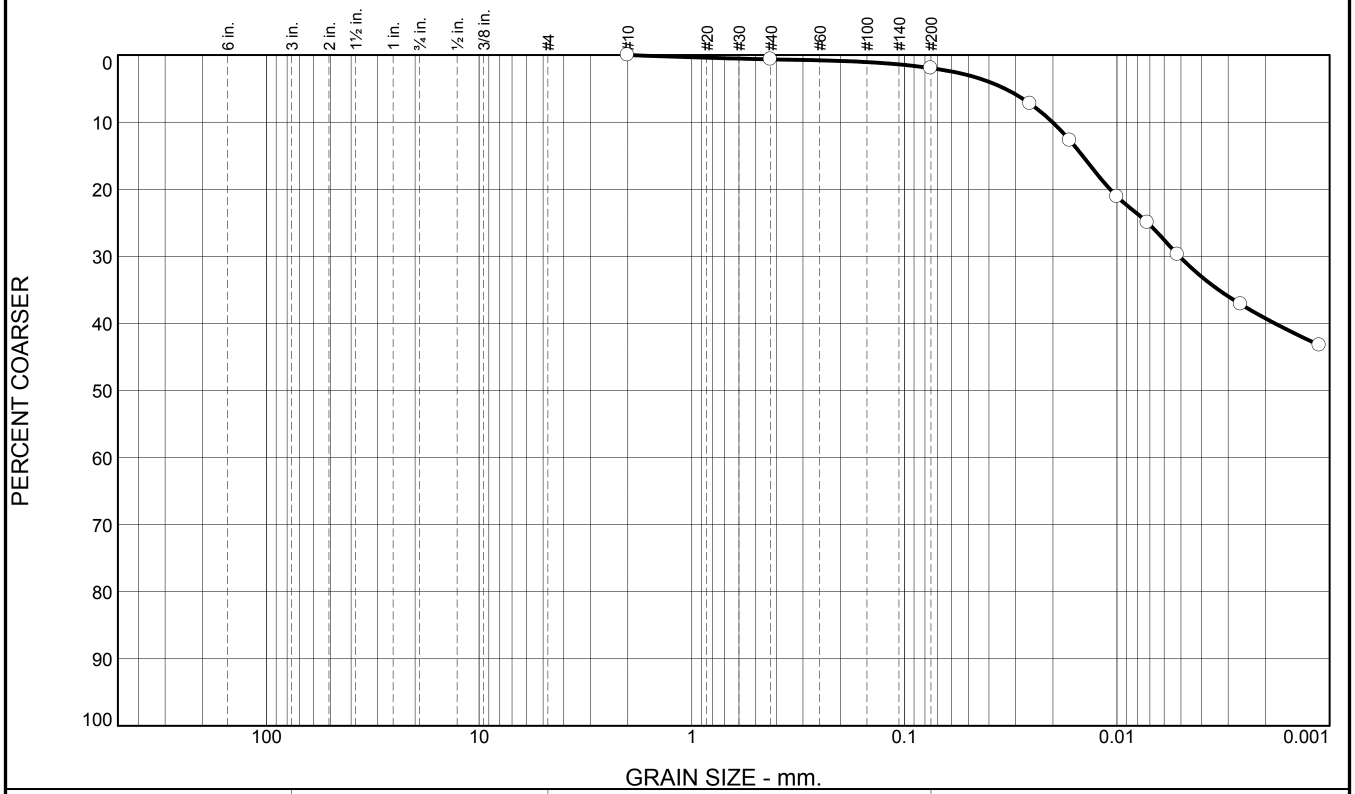
DLZ

Client: Kentucky Transp. Cabinet
Project: Oldham County Underpass

Project No: 0631-0006.02 **Figure**

Tested By: Sheena Marston **Checked By:** Steve Robinson

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.6	1.3	37.4	60.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.4		
#200	98.1		

* (no specification provided)

Material Description

fat clay

Atterberg Limits

PL= 31 LL= 64 PI= 33

Coefficients

D_{90}= 0.0201 D_{85}= 0.0145 D_{60}= 0.0018
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-5(39)

Remarks

Moisture Content = 27.8%
Specific Gravity = 2.74

Source of Sample: B-32 **Depth:** 6.0'-7.5'
Sample Number: S-3 **Date:** 10-7-14

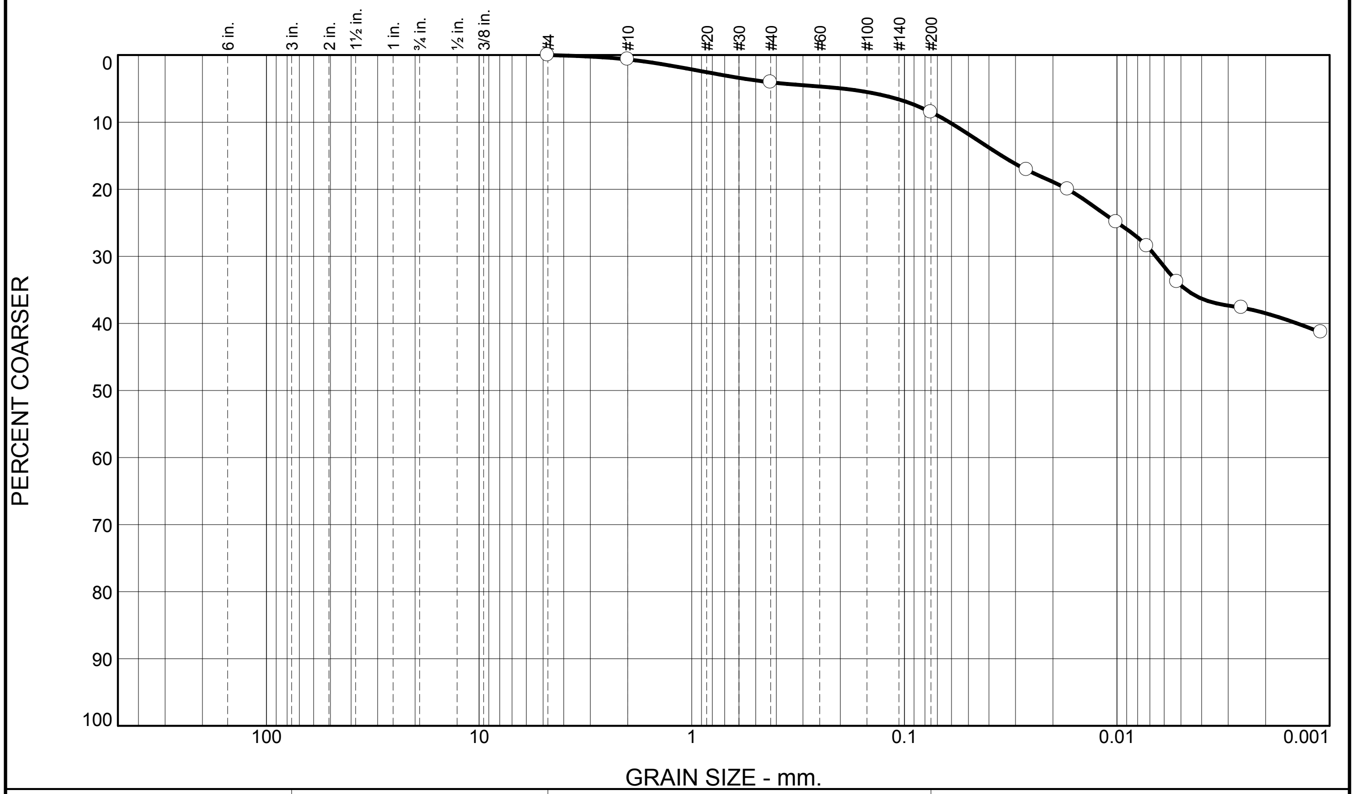
DLZ

Client: Kentucky Transp. Cabinet
Project: Oldham County Underpass

Project No: 0631-0006.02 **Figure**

Tested By: Sheena Marston **Checked By:** Steve Robinson

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.6	3.5	4.4	30.0	61.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.4		
#40	95.9		
#200	91.5		

* (no specification provided)

Material Description

fat clay

Atterberg Limits

PL= 23 LL= 68 PI= 45

Coefficients

D_{90}= 0.0615 D_{85}= 0.0349 D_{60}= 0.0014
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(46)

Remarks

Moisture Content = 38.9%
Specific Gravity = 2.78

Source of Sample: B-39 **Depth:** 3.5'-5.0'
Sample Number: S-2
Date: 10-7-14

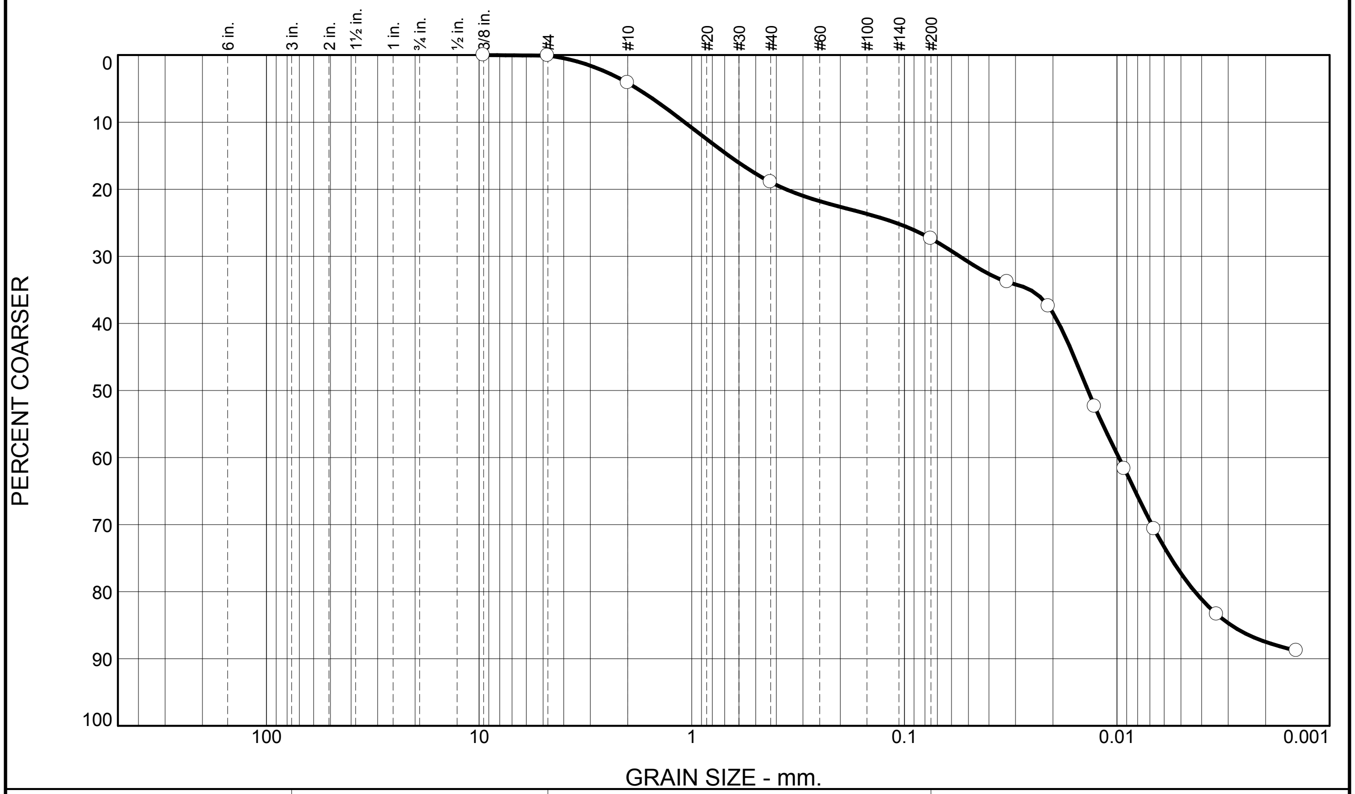
DLZ

Client: Kentucky Transp. Cabinet
Project: Oldham County Underpass

Project No: 0631-0006.02 **Figure**

Tested By: Sheena Marston **Checked By:** Steve Robinson

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.1	4.0	14.8	8.4	60.2	12.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100.0		
#4	99.9		
#10	95.9		
#40	81.1		
#200	72.7		

* (no specification provided)

Material Description

silt with sand

Atterberg Limits

PL= NP
LL= NP
PI= NP

Coefficients

D_{90}= 1.0809
D_{85}= 0.6687
D_{60}= 0.0188
D_{50}= 0.0137
D_{30}= 0.0069
D_{15}= 0.0029
D_{10}=
C_u=
C_c=

Classification

USCS= ML
AASHTO= A-4(0)

Remarks

Moisture Content = 13.4%
Specific Gravity = 2.43

Source of Sample: B-41 **Depth:** 1.0'-2.5'
Sample Number: S-1
Date: 10-7-14

Client: Kentucky Transp. Cabinet
Project: Oldham County Underpass

Project No: 0631-0006.02 **Figure**

Tested By: Sheena Marston **Checked By:** Steve Robinson

DLZ

SUBJECT: Slake Durability Index
GAGE ID:
PROCEDURE: ASTM D 4644

SHEET 1 OF 5
COMP. BY AA DATE 10/8/14
REV. BY SR DATE 10/10/14

REMARKS:
Jar Slake Category = 4
Classification = Non Durable Class 2

Project Name: Oldham Co.
DLZ Project Number: 0631-0006.02
Client: Oldham Co. Kentucky Fiscal Court
Sample ID: B-30, 18.5'-19.5'
Date Started: 10/7/2014
Date Completed: 10/8/2014

Pan # 570

Wet Wt. & Pan: 900.09
Dry Wt. & Pan: 879.21
Pan Wt.: 420.65
Dry Wt. of Sample (A): 458.56
% Moisture: 4.6

Slake Durability Index to nearest 0.1%

Trial 1

Date: 10/7/2014
Time: From 9:40 AM To 9:50 AM
Initials: AA

Water Temp Start: 22
Water Temp Finish: 21.8
Dry Wt, & Pan: 815.30
Pan Wt.: 420.65
Dry Wt. Trial 1: 394.65

Trial 2

Date: 10/8/2014
Time: From 1:05 PM To 1:15 PM
Initials: AA

Water Temp Start: 22.7
Water Temp Finish: 22.5
Dry Wt, & Pan: 713.76
Pan Wt.: 420.65
Final Dry Wt. (B): 293.11

Slake Durability Index = (B)/(A)*100 63.9
Fragment Type: III

Test Pictures

Initial	1st Trial	2nd Trial
		

DLZ

SUBJECT **Slake Durability Index**

PROCEDURE **ASTM D 4644**

GAGE ID

REMARKS

Jar Slake Category = 6

Classification = Durable (Reclassified as dolomite; therefore, shale classification is N/A

SHEET 2 OF 5

COMP. BY AA DATE 10/8/14

REV. BY SR DATE 10/10/14

Project Name Oldham Co.

DLZ Project Number 0631-0006.02

Client Oldham Co. Kentucky Fiscal Court

Sample ID B-30, 30.0'-30.6'

Date Started 10/7/2014

Date Completed 10/8/2014

Pan # SH

Wet Wt. & Pan 707.19

Dry Wt. & Pan 701.55

Pan Wt. 316.93

Dry Wt. of Sample (A) 384.62

% Moisture 1.5

Slake Durability Index to nearest 0.1%

Trial 1

Date: 10/7/2014

Time: From 10:10 AM

To 10:20 AM

Initials AA

Water Temp Start 22.2

Water Temp Finish 22.0

Dry Wt, & Pan 699.40

Pan Wt. 316.93

Dry Wt. Trial 1 382.47

Trial 2

Date: 10/8/2014

Time: From 11:00 AM

To 11:10 AM

Initials AA

Water Temp Start 22.0

Water Temp Finish 21.9

Dry Wt, & Pan 697.38

Pan Wt. 316.93

Final Dry Wt. (B) 380.45

Slake Durability Index = (B)/(A)*100 **98.9**

Fragment Type: **I**

Test Pictures

Initial	1st Trial	2nd Trial
 	 	

DLZ

SUBJECT Slake Durability Index

GAGE ID

PROCEDURE **ASTM D 4644**

SHEET 3 OF 5

REMARKS

Jar Slake Category = 3

Classification = Non Durable Class 2

COMP. BY AA DATE 10/8/14

REV. BY SR DATE 10/10/14

Project Name Oldham Co.

DLZ Project Number 0631-0006.02

Client Oldham Co. Kentucky Fiscal Court

Sample ID B-39, 16.9'-17.5'

Date Started 10/7/2014

Date Completed 10/8/2014

Pan # BV-1

Wet Wt. & Pan	794.96
Dry Wt. & Pan	783.84
Pan Wt.	325.64
Dry Wt. of Sample (A)	458.20
% Moisture	2.4

Slake Durability Index to nearest 0.1%

Trial 1

Date: 10/7/2014

Time: From 8:50 AM

To 9:00 AM

Initials AA

Water Temp Start 21.8

Water Temp Finish 21.4

Dry Wt, & Pan 710.70

Pan Wt. 325.64

Dry Wt. Trial 1 385.06

Trial 2

Date: 10/8/2014

Time: From 12:40 PM

To 12:50 PM

Initials AA

Water Temp Start 22.6

Water Temp Finish 22.2

Dry Wt, & Pan 573.08

Pan Wt. 325.64

Final Dry Wt. (B) 247.44

Slake Durability Index = (B)/(A)*100 **54.0**

Fragment Type: **II**

Test Pictures

Initial	1st Trial	2nd Trial
		

DLZ

SUBJECT **Slake Durability Index**

PROCEDURE **ASTM D 4644**

GAGE ID

REMARKS

Jar Slake Category = 6

Classification = Durable (Reclassified as dolomite; therefore, shale classification is N/A

SHEET 4 OF 5

COMP. BY AA DATE 10/8/14

REV. BY SR DATE 10/10/14

Project Name Oldham Co.

DLZ Project Number 0631-0006.02

Client Oldham Co. Kentucky Fiscal Court

Sample ID B-39, 26.3'-27.5'

Date Started 10/7/2014

Date Completed 10/8/2014

Pan # 576

Wet Wt. & Pan	764.95
Dry Wt. & Pan	756.34
Pan Wt.	312.27
Dry Wt. of Sample (A)	444.07
% Moisture	1.9

Slake Durability Index to nearest 0.1%

Trial 1

Date: 10/7/2014

Time: From 8:30 AM

To 8:40 AM

Initials AA

Water Temp Start 21.9

Water Temp Finish 21.0

Dry Wt, & Pan 748.00

Pan Wt. 312.27

Dry Wt. Trial 1 435.73

Trial 2

Date: 10/8/2014

Time: From 11:50 AM

To 12:00 PM

Initials AA

Water Temp Start 22.9

Water Temp Finish 22.5

Dry Wt, & Pan 734.75

Pan Wt. 312.27

Final Dry Wt. (B) 422.48

Slake Durability Index = (B)/(A)*100 **95.1**

Fragment Type: **I**

Test Pictures

Initial

1st Trial

2nd Trial

DLZ

SUBJECT Slake Durability Index
GAGE ID
PROCEDURE ASTM D 4644
SHEET 5 OF 5
COMP. BY AA DATE 10/8/14
REV. BY SR DATE 10/10/14

REMARKS
Jar Slake Category = 4
Classification = Non Durable Class 1

Project Name: Oldham Co.
DLZ Project Number: 0631-0006.02
Client: Oldham Co. Kentucky Fiscal Court
Sample ID: B-41, 27.8'-28.4'
Date Started: 10/7/2014
Date Completed: 10/8/2014

Pan # 577

Wet Wt. & Pan: 796.97
Dry Wt. & Pan: 785.97
Pan Wt.: 313.67
Dry Wt. of Sample (A): 472.30
% Moisture: 2.3

Slake Durability Index to nearest 0.1%

Trial 1

Date: 10/7/2014
Time: From 9:20 AM
To 9:30 AM
Initials AA

Water Temp Start: 21.8
Water Temp Finish: 21.4
Dry Wt, & Pan: 753.20
Pan Wt.: 313.67
Dry Wt. Trial 1: 439.53

Trial 2

Date: 10/8/2014
Time: From 11:25 AM
To 11:35 AM
Initials AA

Water Temp Start: 22.7
Water Temp Finish: 22.2
Dry Wt, & Pan: 690.59
Pan Wt.: 313.67
Final Dry Wt. (B): 376.92

Slake Durability Index = (B)/(A)*100 79.8 **Fragment Type:** II

Test Pictures

Initial	1st Trial	2nd Trial
		

DLZ

CLIENT **KYTC**

PROJECT **Oldham Co. Underpass**

SUBJECT

DLZ JOB NUMBER **0631-0006.02**

SHEET NO 1 OF 10

TEST COMP. BY SAR DATE 5/27/15

CHECKED BY JAN DATE 5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: B-30

Rock Description: Brown to Light Gray Dolomite

Run No.: 1

Depth 13.50'-13.85'

Diameter:	1.972	1.975	1.972	1.975	1.975	1.975	**1.974** in
	(D_1)	(D_2)	(D_3)	(D_4)	(D_5)	(D_6)	(D_{AVG})

Length:	4.038	4.013	4.009	**4.020** in	$\frac{L}{D}$ = 2.036
	(L_1)	(L_2)	(L_3)	(L_{AVG})	

Volume: 0.007116445 ft^3

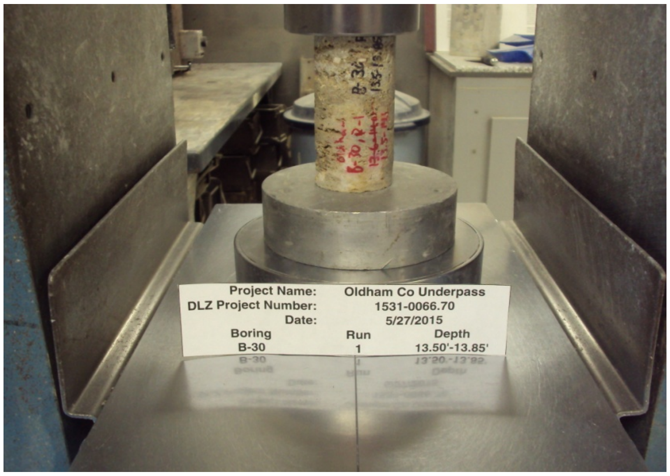
Mass: 464.99 g

Unit Weight: **144.05** pcf

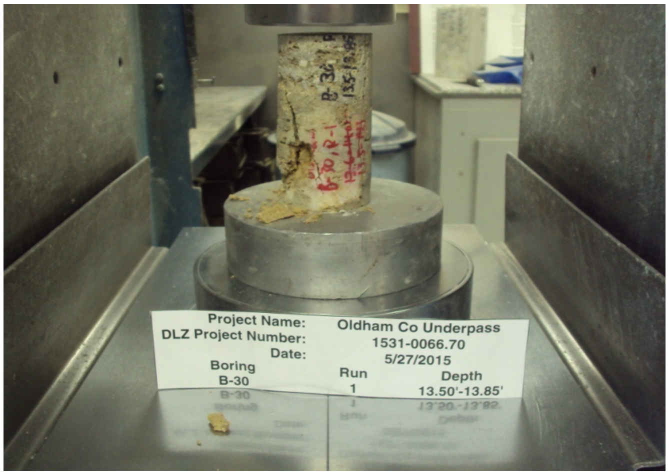
Failure 5,265 lbs

Stress: 1,720 psi

Original Specimen



Fractured Specimen



CLIENT	KYTC	DLZ JOB NUMBER	0631-0006.02		
PROJECT	**Oldham Co. Underpass**	SHEET NO	2	OF	10
SUBJECT		TEST COMP. BY	SAR	DATE	5/27/15
		CHECKED BY	JAN	DATE	5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: 30

Run No.: 3

Depth 22.90'-23.40'

Rock Description: Gray Dolomite

Diameter: 1.971 (D_1) 1.979 (D_2) 1.979 (D_3) 1.972 (D_4) 1.979 (D_5) 1.980 (D_6) **1.977** in (D_{AVG})

Length: 4.750 (L_1) 4.759 (L_2) 4.752 (L_3) **4.754** in (L_{AVG})

$\frac{L}{D}$ = 2.405

Volume: 0.008437977 ft^3

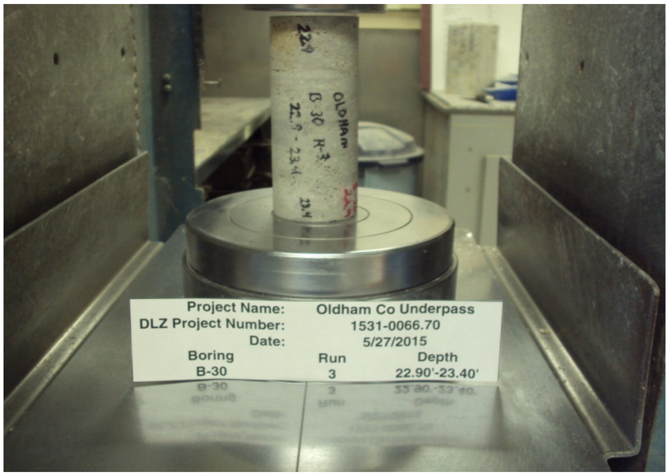
Mass: 619.50 g

Unit Weight: **161.86** pcf

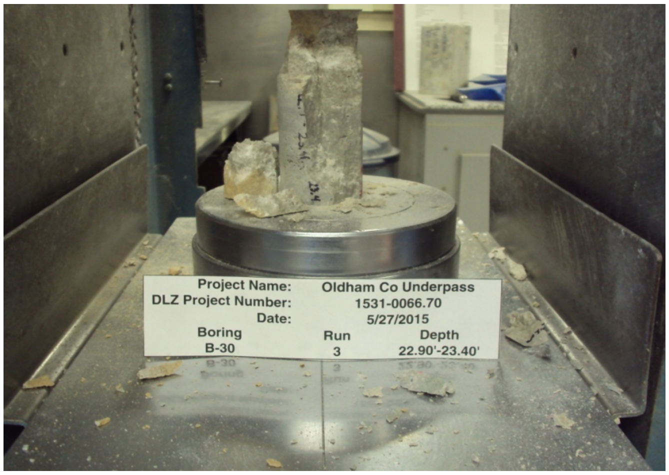
Failure 39,105 lbs

Stress: 12,743 psi

Original Specimen



Fractured Specimen



DLZ

CLIENT **KYTC**
PROJECT **Oldham Co. Underpass**
SUBJECT

DLZ JOB NUMBER **0631-0006.02**
SHEET NO 3 OF 10
TEST COMP. BY SAR DATE 5/27/15
CHECKED BY JAN DATE 5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: B-30
Run No.: 4
Depth 36.15'-36.55'

Rock Description: Gray Dolomite

Diameter:	1.971	1.974	1.972	1.972	1.978	1.973	**1.973** in
	(D_1)	(D_2)	(D_3)	(D_4)	(D_5)	(D_6)	(D_{AVG})

Length:	4.581	4.575	4.570	**4.575** in
	(L_1)	(L_2)	(L_3)	(L_{AVG})

$\frac{L}{D}$ = 2.319

Volume: 0.008094059 ft^3

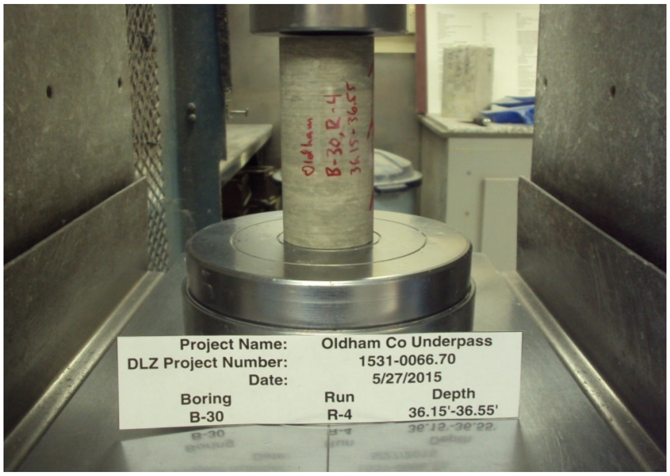
Mass: 606.30 g

Unit Weight: **165.14** pcf

Failure 38,890 lbs

Stress: 12,716 psi

Original Specimen



Fractured Specimen

DLZ

CLIENT **KYTC**
PROJECT **Oldham Co. Underpass**
SUBJECT

DLZ JOB NUMBER **0631-0006.02**
SHEET NO 4 OF 10
TEST COMP. BY SAR DATE 5/27/15
CHECKED BY JAN DATE 5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: B-32
Run No.: 2
Depth 21.40'-21.80'

Rock Description: Brown to Gray Dolomite

Diameter:	1.979	1.977	1.973	1.978	1.974	1.972	**1.976** in
	(D_1)	(D_2)	(D_3)	(D_4)	(D_5)	(D_6)	(D_{AVG})

Length:	4.719	4.706	4.724	**4.716** in
	(L_1)	(L_2)	(L_3)	(L_{AVG})

$\frac{L}{D}$ = 2.387

Volume: 0.008361829 ft^3

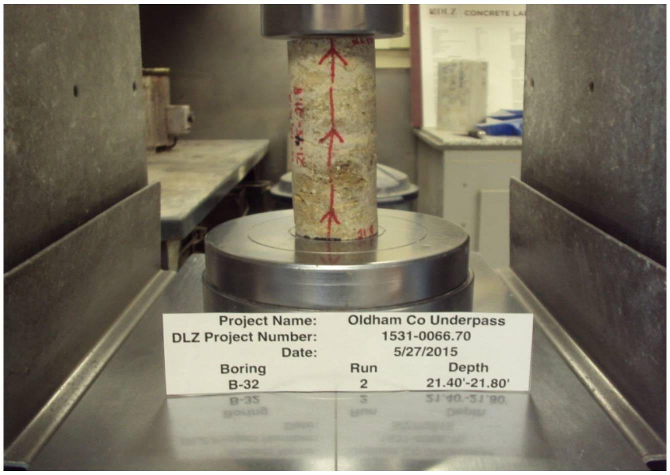
Mass: 566.36 g

Unit Weight: **149.32** pcf

Failure 7,905 lbs

Stress: 2,579 psi

Original Specimen



Fractured Specimen

DLZ

CLIENT **KYTC**
PROJECT **Oldham Co. Underpass**
SUBJECT

DLZ JOB NUMBER **0631-0006.02**
SHEET NO 5 OF 10
TEST COMP. BY SAR DATE 5/27/15
CHECKED BY JAN DATE 5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: B-32
Run No.: 3
Depth 30.05'-30.45'

Rock Description: Gray Dolomite

Diameter:	1.977	1.977	1.975	1.978	1.976	1.975	**1.976** in
	(D_1)	(D_2)	(D_3)	(D_4)	(D_5)	(D_6)	(D_{AVG})
Length:	4.762	4.752	4.761	**4.758** in			
	(L_1)	(L_2)	(L_3)	(L_{AVG})			

$\frac{L}{D}$ = 2.408

Volume: 0.008443412 ft^3

Mass: 637.70 g

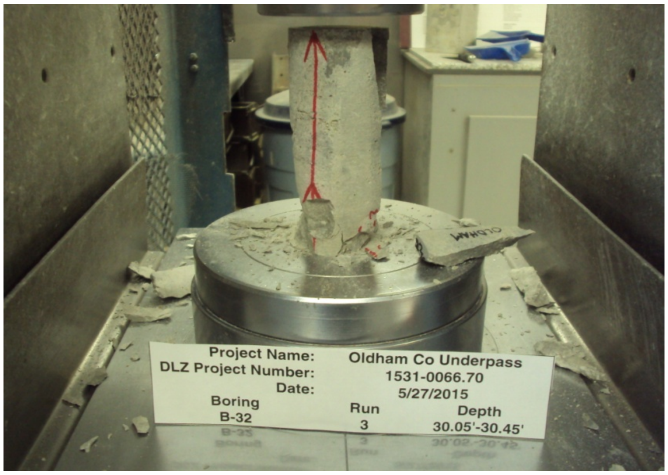
Unit Weight: **166.51** pcf

Failure 48,310 lbs

Stress: 15,748 psi

Original Specimen

Fractured Specimen



DLZ

CLIENT **KYTC**
PROJECT **Oldham Co. Underpass**
SUBJECT

DLZ JOB NUMBER **0631-0006.02**
SHEET NO 6 OF 10
TEST COMP. BY SAR DATE 5/27/15
CHECKED BY JAN DATE 5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: B-39
Run No.: 1
Depth 12.65'-13.00'

Rock Description: Brown Dolomite

	(D_1)	(D_2)	(D_3)	(D_4)	(D_5)	(D_6)	(D_{AVG})
Diameter:	1.973	1.961	1.977	1.978	1.966	1.975	**1.972** in

	(L_1)	(L_2)	(L_3)	(L_{AVG})
Length:	4.150	4.151	4.145	**4.149** in

$\frac{L}{D}$ = 2.104

Volume: 0.007326866 ft^3

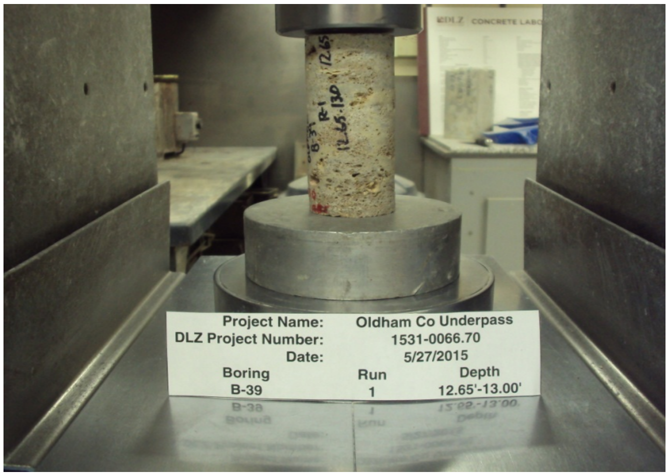
Mass: 493.42 g

Unit Weight: **148.47** pcf

Failure 10,740 lbs

Stress: 3,518 psi

Original Specimen



Fractured Specimen

DLZ

CLIENT **KYTC**
PROJECT **Oldham Co. Underpass**
SUBJECT

DLZ JOB NUMBER **0631-0006.02**
SHEET NO 7 OF 10
TEST COMP. BY SAR DATE 5/27/15
CHECKED BY JAN DATE 5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: B-39
Run No.: 3
Depth 20.00'-20.40'

Rock Description: Light to Dark Gray Dolomite

Diameter:	1.981	1.979	1.978	1.980	1.978	1.980	**1.979** in
	(D_1)	(D_2)	(D_3)	(D_4)	(D_5)	(D_6)	(D_{AVG})

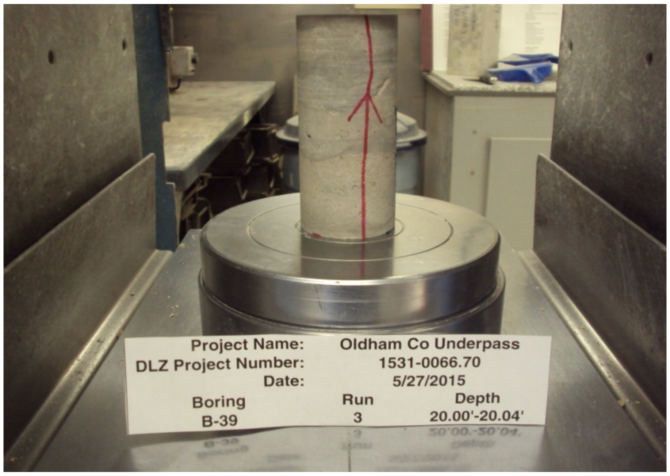
Length:	4.572	4.575	4.558	**4.568** in	$\frac{L}{D}$ = 2.308
	(L_1)	(L_2)	(L_3)	(L_{AVG})	

Volume: 0.008131896 ft^3 Mass: 587.18 g Unit Weight: **159.21** pcf

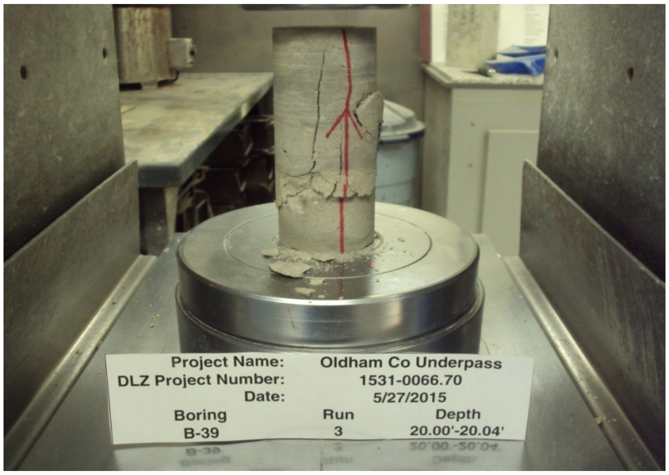
Failure 23,620 lbs

Stress: 7,676 psi

Original Specimen



Fractured Specimen



DLZ

CLIENT **KYTC**
PROJECT **Oldham Co. Underpass**
SUBJECT

DLZ JOB NUMBER **0631-0006.02**
SHEET NO 8 OF 10
TEST COMP. BY SAR DATE 5/27/15
CHECKED BY JAN DATE 5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: B-39
Run No.: 4
Depth 31.00'-31.40'

Rock Description: Light to Dark Gray Dolomite

Diameter:	1.971	1.972	1.973	1.975	1.974	1.975	**1.973** in
	(D_1)	(D_2)	(D_3)	(D_4)	(D_5)	(D_6)	(D_{AVG})

Length:	4.429	4.427	4.414	**4.423** in
	(L_1)	(L_2)	(L_3)	(L_{AVG})

$\frac{L}{D}$ = 2.242

Volume: 0.007825161 ft^3

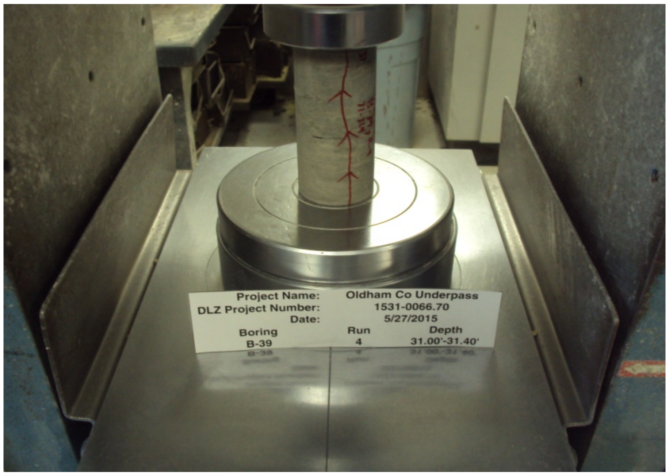
Mass: 589.58 g

Unit Weight: **166.11** pcf

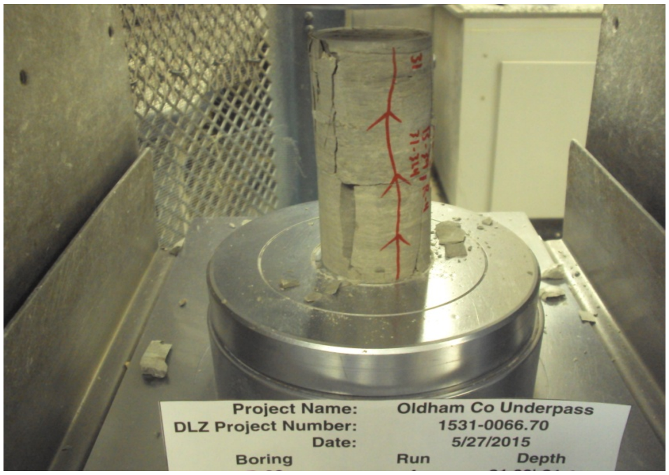
Failure 23,180 lbs

Stress: 7,579 psi

Original Specimen



Fractured Specimen



DLZ

CLIENT **KYTC**
PROJECT **Oldham Co. Underpass**
SUBJECT

DLZ JOB NUMBER **0631-0006.02**
SHEET NO 9 OF 10
TEST COMP. BY SAR DATE 5/27/15
CHECKED BY JAN DATE 5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: B-41
Run No.: 1
Depth 17.20'-17.60'

Rock Description: Brown to Gray Dolomite

Diameter:	1.970	1.970	1.979	1.972	1.975	1.978	**1.974** in
	(D_1)	(D_2)	(D_3)	(D_4)	(D_5)	(D_6)	(D_{AVG})

Length:	4.563	4.584	4.570	**4.572** in	$\frac{L}{D}$ = 2.316
	(L_1)	(L_2)	(L_3)	(L_{AVG})	

Volume: 0.008094218 ft^3

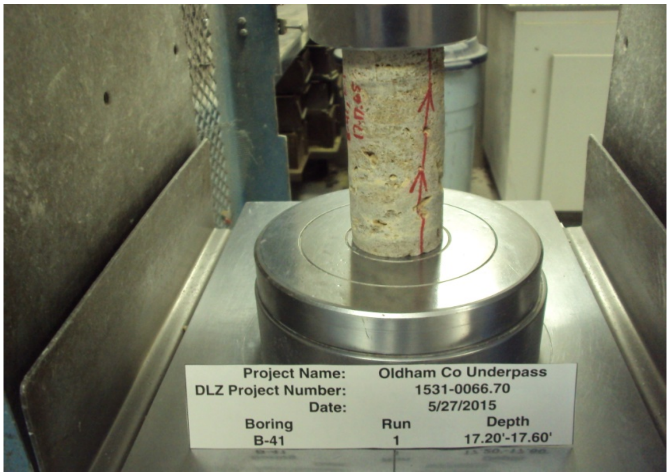
Mass: 565.98 g

Unit Weight: **154.16** pcf

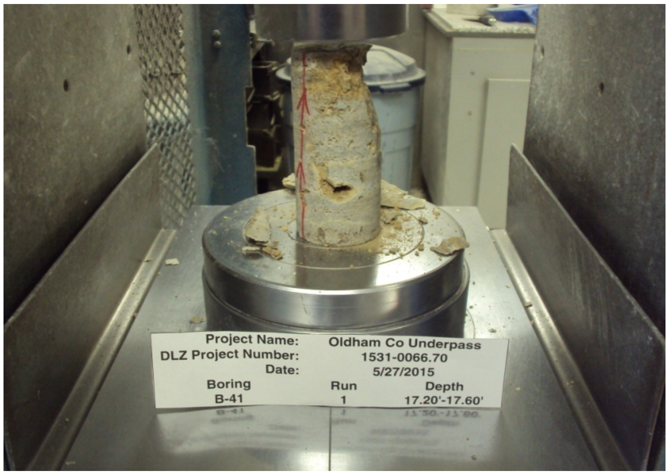
Failure 10,225 lbs

Stress: 3,341 psi

Original Specimen



Fractured Specimen



DLZ

CLIENT **KYTC**
PROJECT **Oldham Co. Underpass**
SUBJECT

DLZ JOB NUMBER **0631-0006.02**
SHEET NO 10 OF 10
TEST COMP. BY SAR DATE 5/27/15
CHECKED BY JAN DATE 5/28/15

Unconfined Compressive Strength of Intact Rock Core Specimen (ASTM D7012)

Boring No.: B-41
Run No.: 3
Depth 29.50'-29.95'

Rock Description: Gray to Dark Gray Dolomite

Diameter:	1.978	1.975	1.977	1.977	1.976	1.977	**1.977** in
	(D_1)	(D_2)	(D_3)	(D_4)	(D_5)	(D_6)	(D_{AVG})

Length:	4.699	4.708	4.701	**4.703** in
	(L_1)	(L_2)	(L_3)	(L_{AVG})

$\frac{L}{D}$ = 2.379

Volume: 0.00834745 ft^3

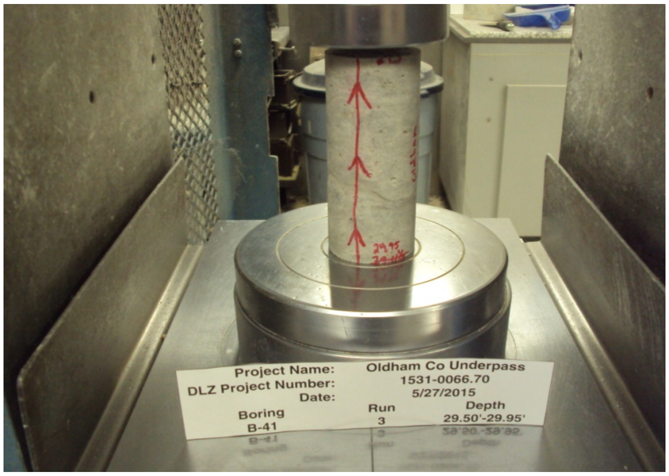
Mass: 620.20 g

Unit Weight: **163.80** pcf

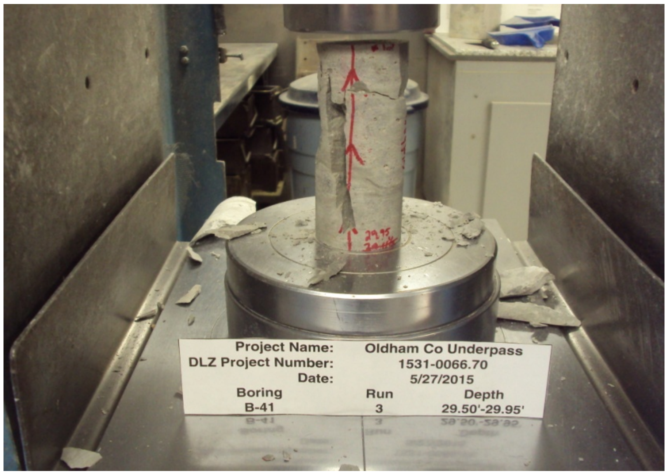
Failure **32,370** **lbs**

Stress: **10,548** **psi**

Original Specimen

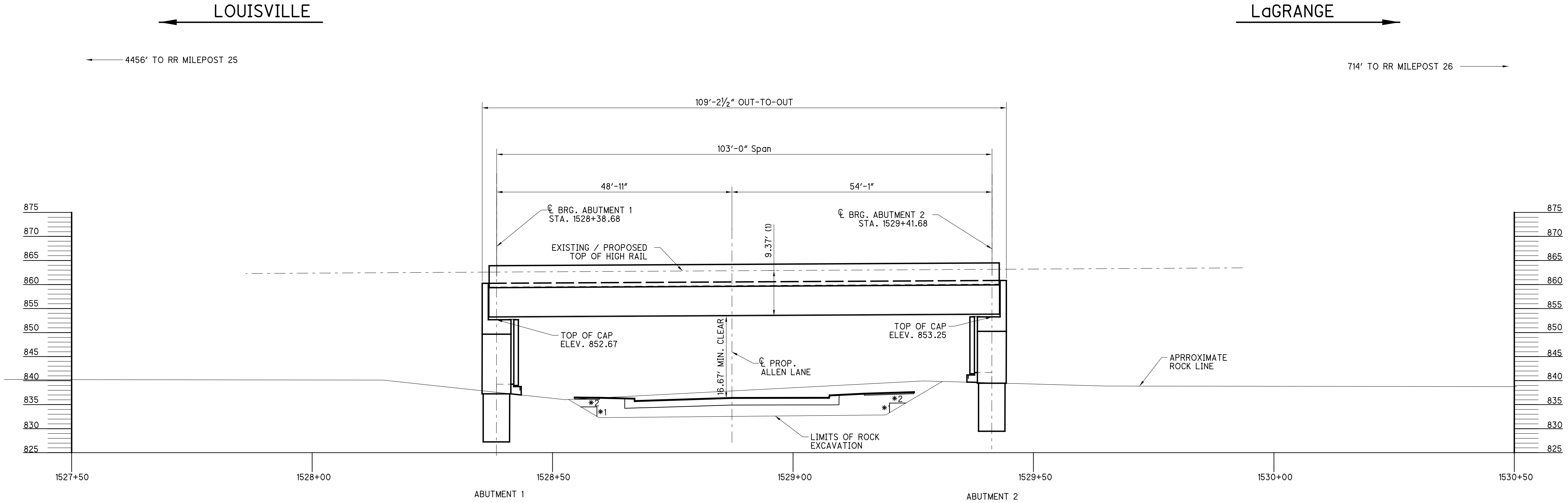


Fractured Specimen



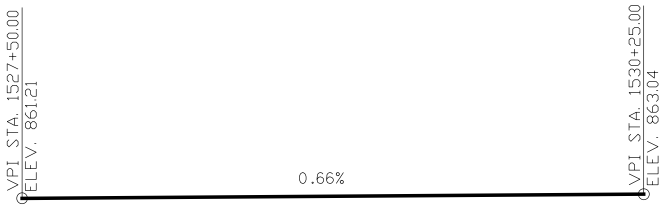
APPENDIX III

Elevation Views of the Recent Design Concepts for the Proposed Bridge
Calculations (Tip and Side Resistances, Elastic Modulus for Rock, Drained Strength Estimate for Fat Clay, Stability, Settlement, and Calculation for p-Multiplier)
Recommended Parameters for Single Drilled Shaft Static Lateral Analyses



ELEVATION

72" WELDED STEEL PLATE GIRDER ÷ ONE SPAN
103'-0" SPAN
COOPER E80 LOADING, 14° 57' 15" SKEW



PROFILE GRADE - CSX

PROFILE GRADE SHOWN IS FOR TOP OF TIE

*PERPENDICULAR TO ROADWAY
(1)SUPERSTRUCTURE DEPTH FROM
TOP OF RAIL TO BOTTOM OF BEAM

REVISION	DATE

DATE: APRIL - 2015	CHECKED BY
DESIGNED BY: ASP	MSC
DETAILED BY: MSC	ASP

OLDHAM COUNTY
FISCAL COURT

COUNTY
OLDHAM

ROUTE
CSX RR

CROSSING
ALLEN LANE

ELEVATION - SINGLE-SPAN ALTERNATE

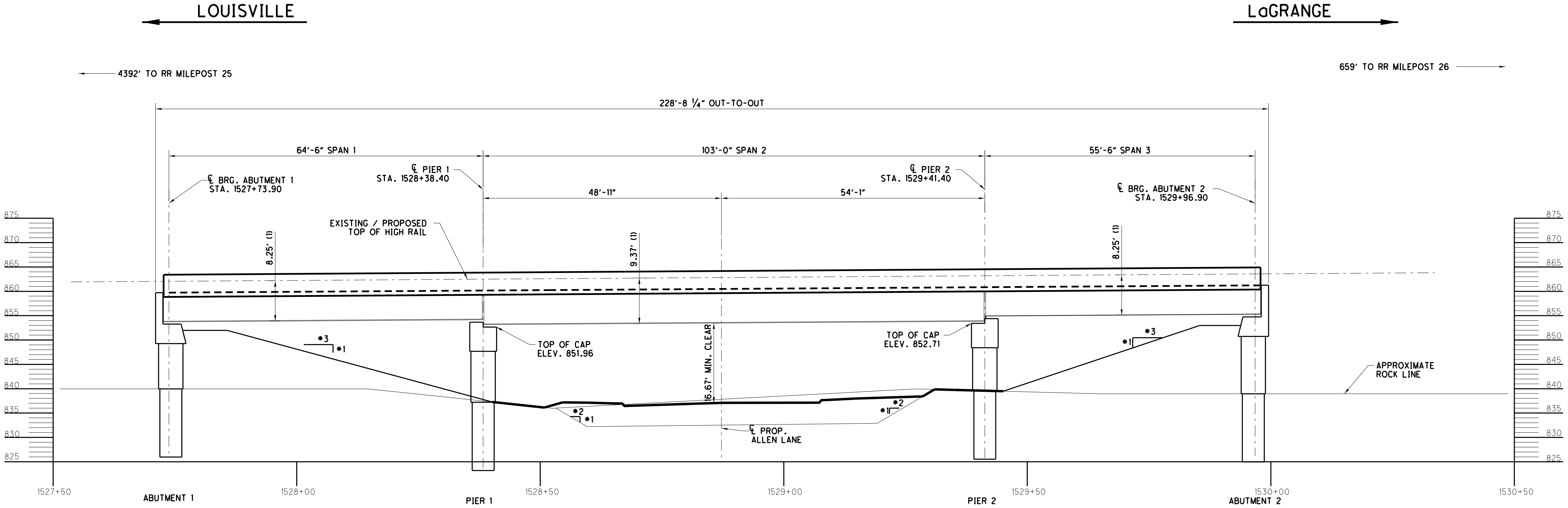
ITEM NUMBER
5-434.00

PREPARED BY
201 BRIGHTON PARK BLVD.
FRANKFORT, KENTUCKY 40602
(502) 695-2300

SHEET NO.

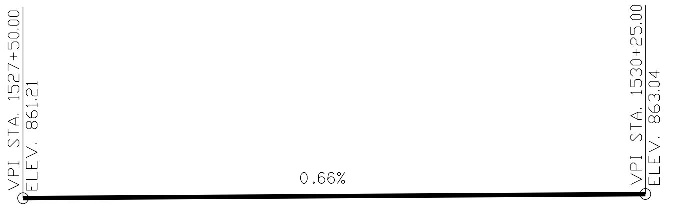
DRAWING NO.

FILE NAME: $$$$design$file$$specification$$$$
USERNAME: $$$$plotted$by$$$$
DATE: $$$$DATE$$$$
SHEET LOCATION:



ELEVATION

60" & 72" WELDED STEEL PLATE GIRDERS ÷ 3 SPANS
64'-6", 103'-0", 55'-6" SPANS
COOPER E80 LOADING, 14° 57' 15" SKEW



PROFILE GRADE - CSX

PROFILE GRADE SHOWN IS FOR TOP OF TIE

*PERPENDICULAR TO ROADWAY
(1) SUPERSTRUCTURE DEPTH FROM TOP OF RAIL TO BOTTOM OF BEAM

REVISION	DATE

DATE: APRIL - 2015	CHECKED BY
DESIGNED BY: ASP	MSC
DETAILED BY: MSC	ASP

OLDHAM COUNTY
FISCAL COURT

COUNTY
OLDHAM

ROUTE
CSX RR

CROSSING
ALLEN LANE

ELEVATION - THREE-SPAN ALTERNATE

ITEM NUMBER
5-434.00

PREPARED BY
201 BRIGHTON PARK BLVD.
FRANKFORT, KENTUCKY 40602
(502) 695-2300

SHEET NO.

DRAWING NO.

DLZ

CLIENT	Oldham County	JOB NUMBER	1121-3010.00		
PROJECT	Allen Lane Underpass	SHEET NO.	1	OF	1
SUBJECT	Tangent Drilled Shaft Capacity	COMP. BY	MDK	DATE	7/8/2015
		CHECKED BY	EWT	DATE	9/4/2015

Drilled Shaft - Side and Tip Resistance in Rock

Determine the side and tip resistance of the proposed Tangent Drilled Shaft Wall for the Allen Lane Underpass bridge.

Ref. AASHTO LRFD BDS, 7th Ed.

Side Resistance

$$q_s = 0.65\alpha_E p_a \sqrt{q_u/p_a}$$

Eq. 10.8.3.5.4b-2

α_E = 0.85

Table 10.8.3.5.4b-1
(RQD = 100%, Open Joints)

q_u = 2,000 psi 288 ksf

Ref. UCS test results
(minimum value)

q_s = 13.65 ksf 7 tsf

Tip Resistance

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$

Eq. 10.8.3.5.4c-2

$$A = \sigma'_{vb} + q_u\left[m_b\left(\frac{\sigma'_{vb}}{q_u}\right) + s\right]^a$$

A = 36.71 ksf

Eq. 10.8.3.5.4c-3

$$s = e^{\frac{GSI-100}{9-3D}}$$

s = 0.0022

Eq. 10.4.6.4-2

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right)$$

a = 0.51

Eq. 10.4.6.4-3

$$m_b = m_i e^{\frac{GSI-100}{28-14D}}$$

m_b = 1.26

Eq. 10.4.6.4-4

GSI = 45 (range of 36-55, say 45)
Figure 10.4.6.4-1

D = 0
Ref. Hoek et al. 2002

m_i = 9
Table 10.4.6.4-1

σ'vb = 2.92 ksf

Ref. UCS test results

q_u = 2,000 psi 288 ksf
(Average value)

f'_c = 4,000 psi 576 ksf
Concrete Compressive Strength

(use the latter of the UCS test result and the concrete compressive strength for calculating tip resistance)

q_s = 150.83 ksf 75 tsf

Created By: EWT
Checked By: HJH

Elastic Modulus for Rock (AASHTO LRFD 7th Ed. 2014 Section 10.4.6.5)

Boring Number	Run Number	Depth (feet)	Elevation (feet)	Description	Unit Weight, pcf	Unconfined Compressive Strength, psi	Unconfined Compressive Strength, Mpa	Estimated GSI	Rock Mass Modulus Hoek (Em), GPa	Rock Mass Modulus Hoek (Em), ksi	Modulus of Intack Rock, (Er), ksi	Rock Mass Modulus Yang (Em), ksi	Preliminary Design Em, ksi	Recommended Design Em, ksi	Boring Number	Elevation (feet)
B-30	1	13.50 - 13.85	836.7 - 836.4	Brown to light gray dolomite	144.1	1,720	11.9	10	0.34	49.95	5700	90.37	49.95	Treated as RDZ above Elev. 835	B-30	836.7 - 836.4
B-30	3	22.90 - 23.40	827.3 - 826.8	Gray dolomite	161.9	12,743	87.9	50	9.37	1,359.49	5700	570.89	570.89	450*	B-30	827.3 - 826.8
B-30	4	36.15 -36.55	814.1 - 813.7	Gray dolomite	165.1	12,716	87.7	50	9.36	1,358.05	5700	570.89	570.89	450*	B-30	814.1 - 813.7
B-32	2	21.40 - 21.80	833.3 - 832.9	Brown to gray dolomite	149.3	2,579	17.8	20	0.75	108.76	5700	143.27	108.76	100	B-32	833.3 - 832.9
B-32	3	30.02 - 30.45	824.7 - 824.3	Gray dolomite	166.5	15,748	108.6	55	13.34	1,934.11	5700	718.82	718.82	450*	B-32	824.7 - 824.3
B-39	1	12.65 - 13.00	836.5 - 836.1	Brown dolomite	148.5	3,518	24.3	20	0.88	127.02	5700	143.27	127.02	Treated as RDZ above Elev. 835	B-39	836.5 - 836.1
B-39	3	20.00 - 20.40	829.1 - 828.7	Light to dark gray dolomite	159.2	7,676	52.9	35	3.07	444.95	5700	285.99	285.99	450*	B-39	829.1 - 828.7
B-39	4	31.00 - 31.40	818.1 - 817.7	Light to dark gray dolomite	166.1	7,579	52.3	35	3.05	442.13	5700	285.99	285.99	450*	B-39	818.1 - 817.7
B-41	1	17.20 - 17.60	838.9 - 838.5	Brown to gray dolomite	154.2	3,341	23.0	20	0.85	123.79	5700	143.27	123.79	Treated as RDZ above Elev. 835	B-41	838.9 - 838.5
B-41	3	29.50 - 29.95	826.6 - 826.2	Gray to dark gray dolomite	163.8	10,548	72.7	45	6.40	927.52	5700	453.40	453.40	450*	B-41	826.6 - 826.2
				minimum =	144.1	1,720	11.9									
				maximum =	166.5	15,748	108.6									
				Average =	157.9	7,817	53.9									

*Average of (570.89+570.89+718.82+285.99+285.99+453.4)/6 = 481 ksi, use 450 ksi

Reference: AASHTO 7th Edition, Sections 10.4.6.4 and 10.4.6.5

GSI: Geological Strength Index
Er: Used mean value for dolostone in AASHTO Table C10.4.6.5.1
Preliminary Design Em: Smaller of Hoek (Em) or Yang (Em), ksi

Created By: EWT
Checked By: HJH
Date: 9/2/2015

Summary of Laboratory Test Results (Highly Plastic Clay (CH) ONLY) - Index Properties
(For Drained Shear Strength Estimate for Fat Clay)

Boring Number	Sample Number	Depth (feet)	USCS Class.	AASHTO Class.	N Value	N60 Value	% AGG	% C. Sand	% M. Ssand	% F. Sand	% Silt	% Clay	LL	PL	PI	Moisture Content %	Liquidity Index	Soil Activity
B-1	S-1	1	CH	-	8	10												
B-1	S-2	3.5	CH	-	56	71												
B-3a	S-1	1	CH	A-7-6(28)	15	20	0	8	3.2	3.2	35.6	50	58	29	29	22.67	-0.22	0.59
B-20	S-2	3.5	CH	-	11	14												
B-20	S-3	6	CH	-	17	22												
B-20	S-4	8.5	CH	-	12	15												
B-21	S-2	3.5	CH	-	13	17												
B-21	S-3	6	CH	-	17	22												
B-21	S-4	8.5	CH	-	23	29												
B-22	S-1	1	CH	-	5	6												
B-22	S-2	3.5	CH	-	9	11												
B-22	S-3	6	CH	-	10	13												
B-22	S-4	8.5	CH	-	17	22												
B-23	S-2	3.5	CH	A-7-6(36)	9	12	0	0.5	1.2	1.8	48.4	48.1	59	27	32	29.28	0.07	0.68
B-24	ST-1	2	CH	A-7-6(28)			0	1.3	4.1	3.4	44.5	46.7	50	21	29	24.56	0.12	0.62
B-30	S-2	3.5	CH	-	10	14	-	-	-	-	-	-	-	-	-	29.81		
B-30	S-3	6	CH	A-7-5(49)	8	11	0	4.3	0.4	1.1	23.8	70.4	78	35	43	44.19	0.21	0.62
B-32	S-1	1	CH	-	9	10	-	-	-	-	-	-	-	-	-	19.02		
B-32	S-2	3.5	CH	-	15	17	-	-	-	-	-	-	-	-	-	19.29		
B-32	S-3	6	CH	A-7-5(39)	17	19	0	0	0.6	1.3	37.4	60.7	64	31	33	27.79	-0.10	0.55
B-32	S-4	8.5	CH	-	15	17	-	-	-	-	-	-	-	-	-	34.28		
B-39	S-1	1	CH	-	8	11	-	-	-	-	-	-	-	-	-	26.19		
B-39	S-2	3.5	CH	A-7-6(46)	8	11	0	0.6	3.5	4.4	30	61.5	68	23	45	38.89	0.35	0.74
B-41	S-2	3.5	CH	-	26	29	-	-	-	-	-	-	-	-	-	13.39		
B-41	S-3	6	CH	-	13	14	-	-	-	-	-	-	-	-	-	26.34		
B-41	S-4	8.5	CH	-	12	13	-	-	-	-	-	-	-	-	-	29.72		
B-41	S-5	13.5	CH	-	10	11	-	-	-	-	-	-	-	-	-	40.84		
	min=	1		min=	5	6						46.7	50					
	max=	13.5		max=	56	71						70.4	78					
	avg=	4.83		avg=	14.35	18						56.23	62.83					
	std=	3.09		std=	9.78	12.27						9.43	9.60					

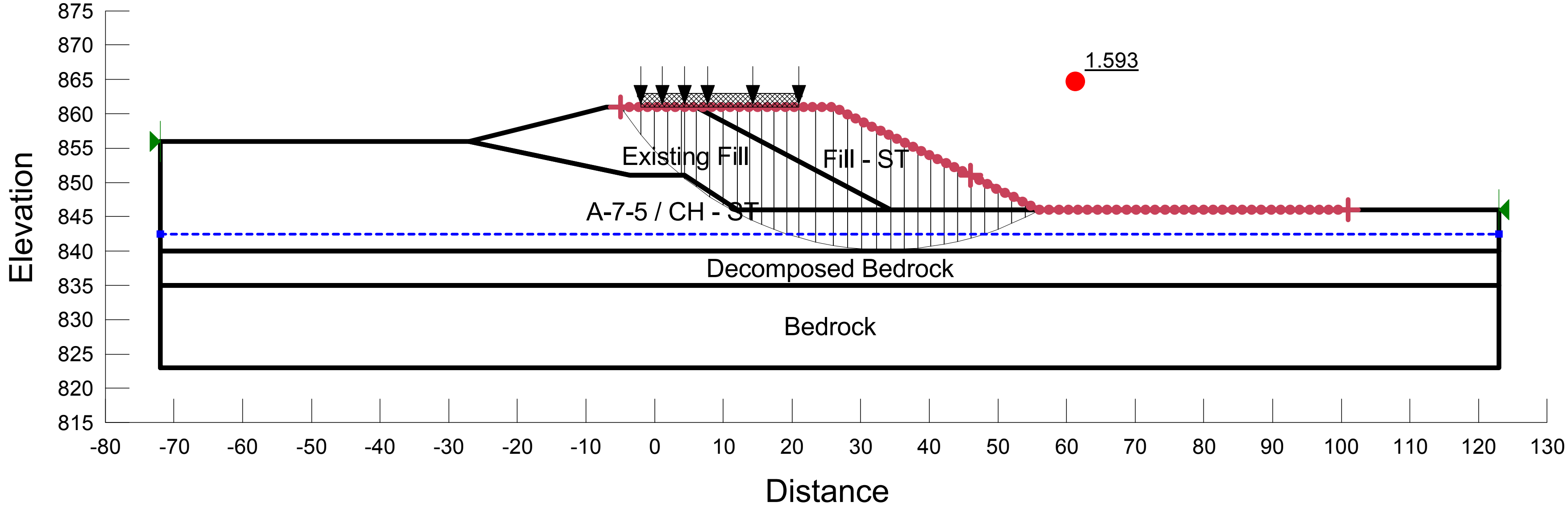
Step 1: Adjusting LL & %Clay values for samples using Equations 8 and 9 (per Stark method)

Eqn 8 LL#200= LL#40*0.003*LL#40 + 1.23*LL#40 where LL#40= 72
LL#200= 105

Eqn 9 CF#200= CF#40*0.0002*(CF#40)^2-0.0278*(CF#40)^2+2.15*CF#40 where CF#40= 66
CF#200= 78

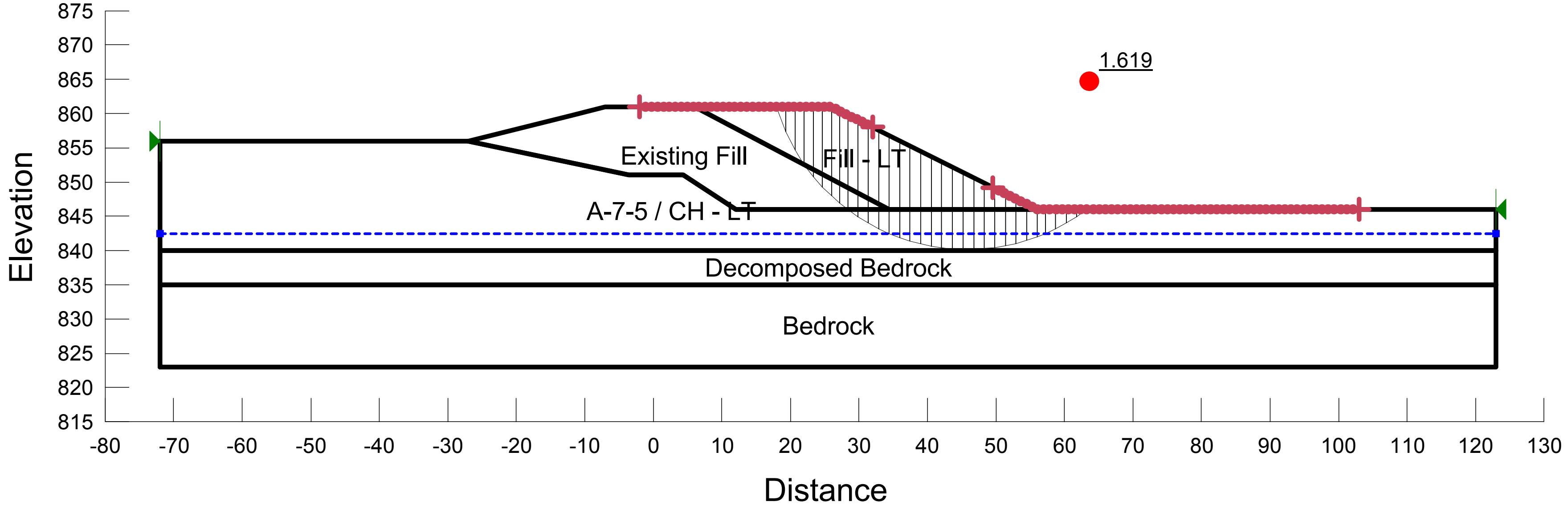
For an average depth of approximately 5 ft and a unit weight of soil of 130 pcf, the normal stress is approximately 650 psf or 31 Kpa.
Using the chart from Stark, the drained fully softened secant friction angle is approximately 24 degrees.
Reference: Stark, T.D., and Hussain, M., "Empirical Correlations - Drained Shear Strength for Slope Stability Analyses," Paper #GTENG-2476-Revision #2, August 14, 2012, A paper accepted for publication in the ASCE Journal of Geotechnical and Geoenvironmental Engineering. pp. 575-588.
After discussions with KYTC, a cohesion of 40 psf and a friction angle of 20 degrees are used for the drained shear strength of the highly plastic soils.

Side Slope Fill - Structure Slope Stability
Short Term



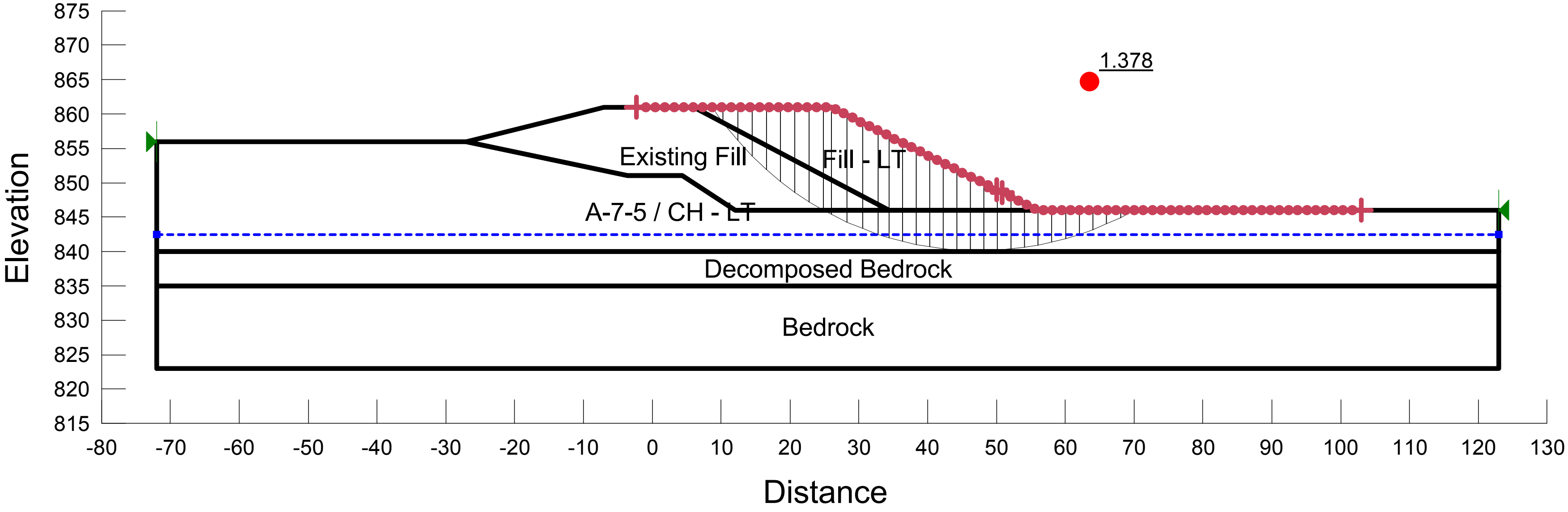
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Fill - ST Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 1,500 psf Phi': 0 °
Name: A-7-5 / CH - ST Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 1,000 psf Phi': 0 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °

Side Slope Fill - Structure Slope Stability
Long Term



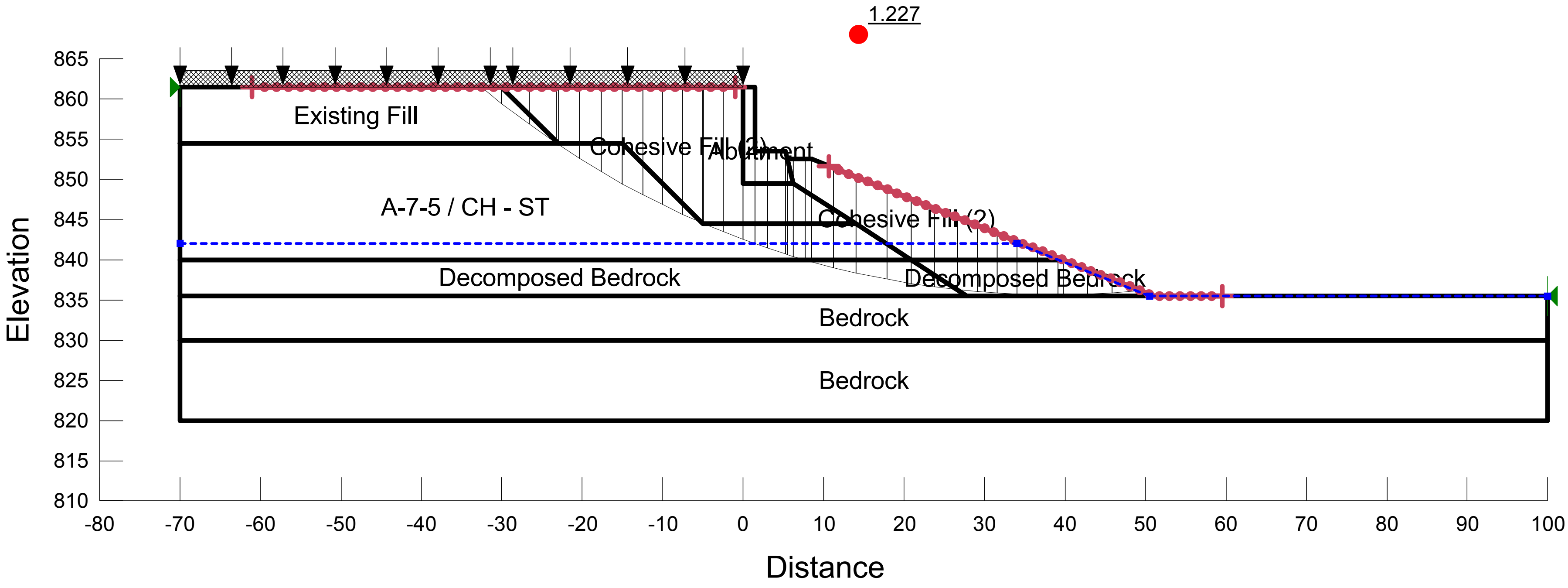
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Fill - LT Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 270 psf Phi': 28 °
Name: A-7-5 / CH - LT Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 40 psf Phi': 20 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °

Side Slope Fill - Structure Slope Stability
Seismic



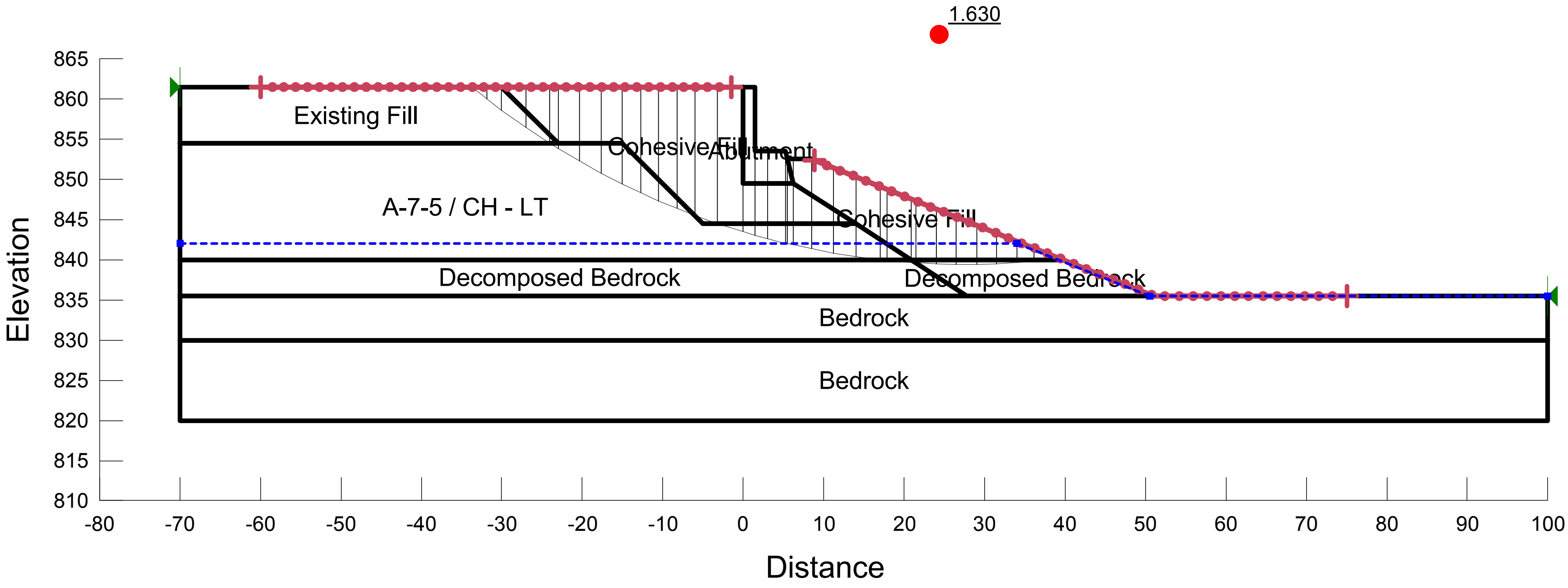
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Fill - LT Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 270 psf Phi': 28 °
Name: A-7-5 / CH - LT Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 40 psf Phi': 20 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °

Spillthrough Cut Stability
Short Term



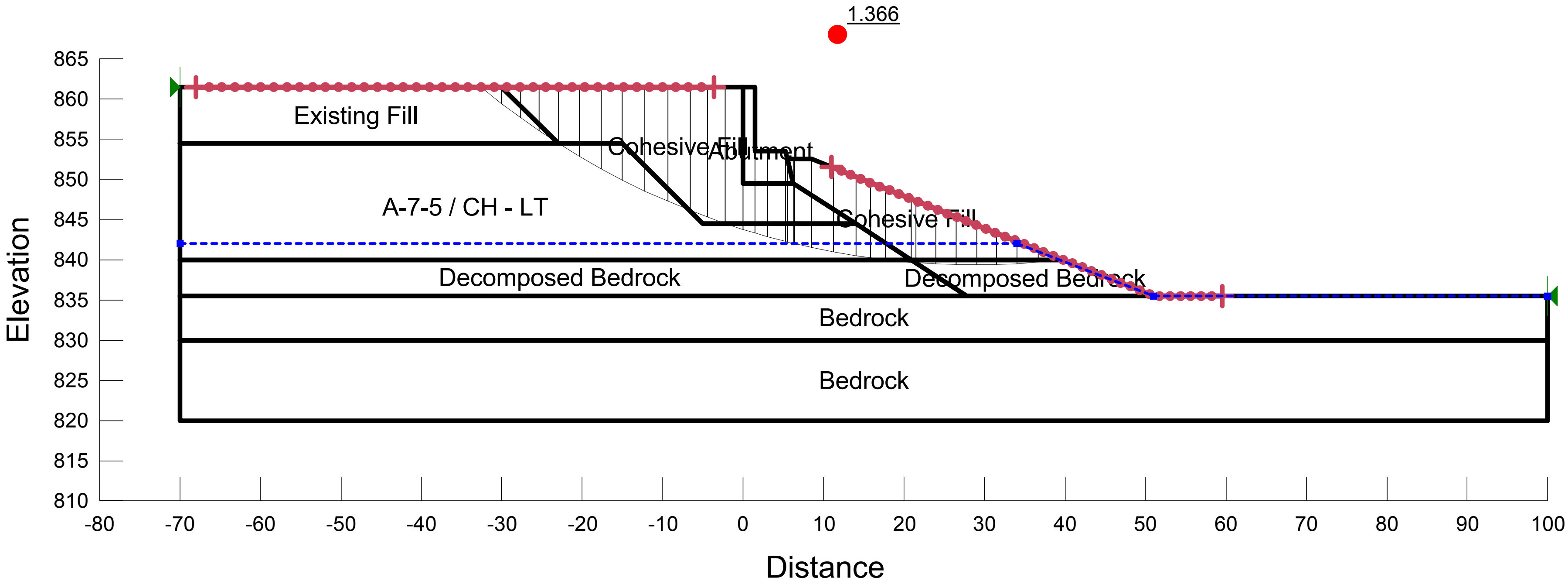
Name: Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 40 °
Name: A-7-5 / CH - ST Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 1,000 psf Phi': 0 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °
Name: Abutment Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 45 °
Name: Cohesive Fill (2) Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 1,500 psf Phi': 0 °

Spillthrough Cut Stability
Long Term



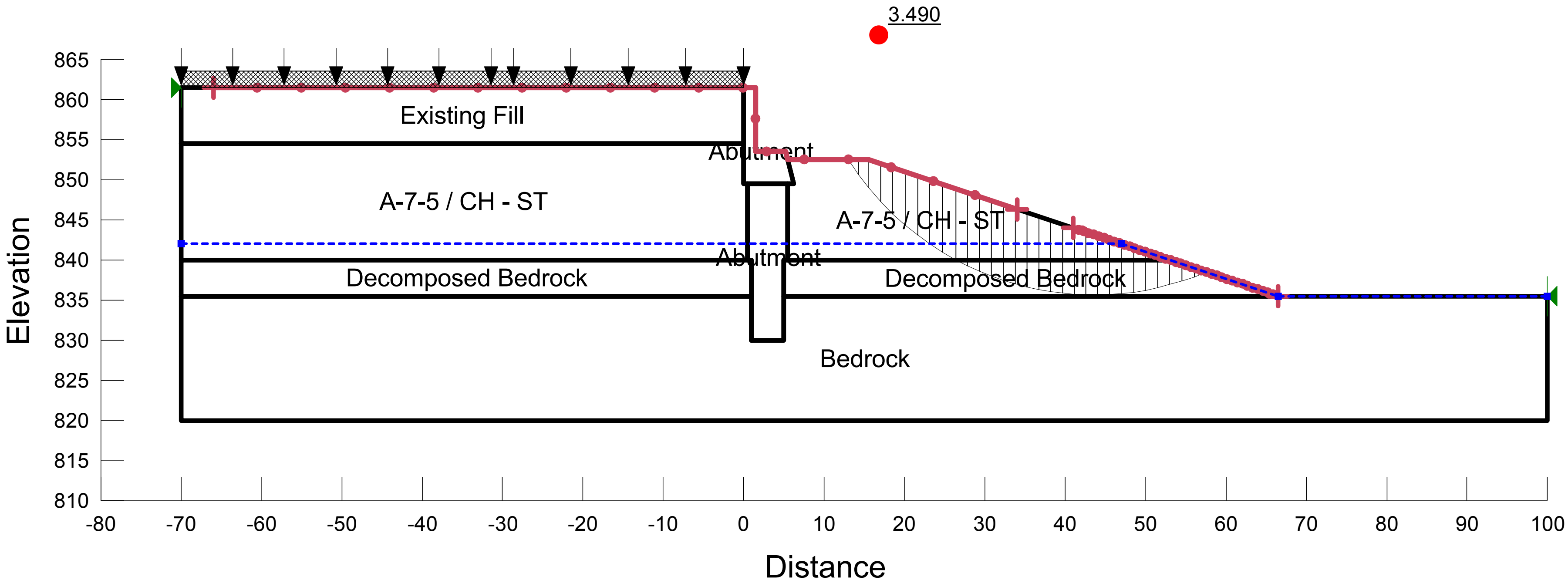
Name: Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 40 °
Name: A-7-5 / CH - LT Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 40 psf Phi': 20 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °
Name: Abutment Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 45 °
Name: Cohesive Fill Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 270 psf Phi': 28 °

Spillthrough Cut Stability
Seismic



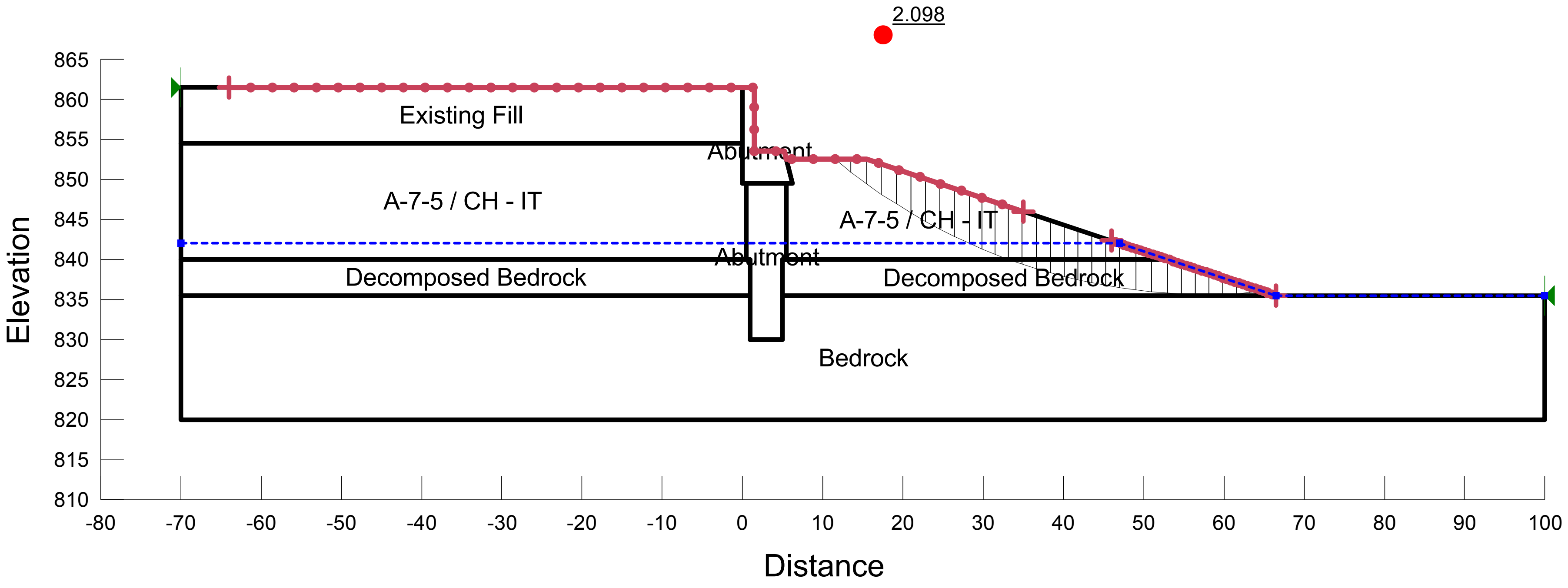
Name: Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 40 °
Name: A-7-5 / CH - LT Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 40 psf Phi': 20 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °
Name: Abutment Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 45 °
Name: Cohesive Fill Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 270 psf Phi': 28 °

Spillthrough Cut Stability
Short Term

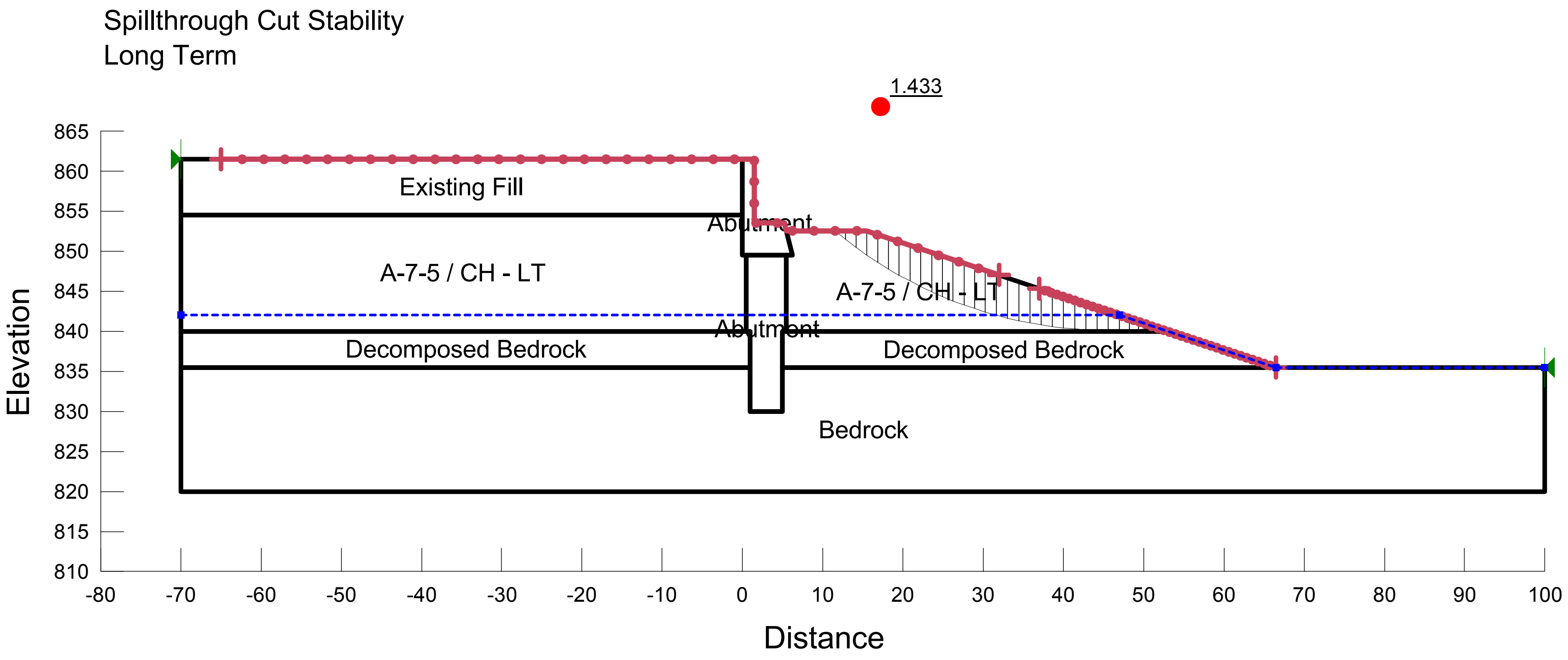


Name: Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 40 °
Name: A-7-5 / CH - ST Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 1,000 psf Phi': 0 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °
Name: Abutment Model: High Strength Unit Weight: 145 pcf

Spillthrough Cut Stability
Intermediate Term

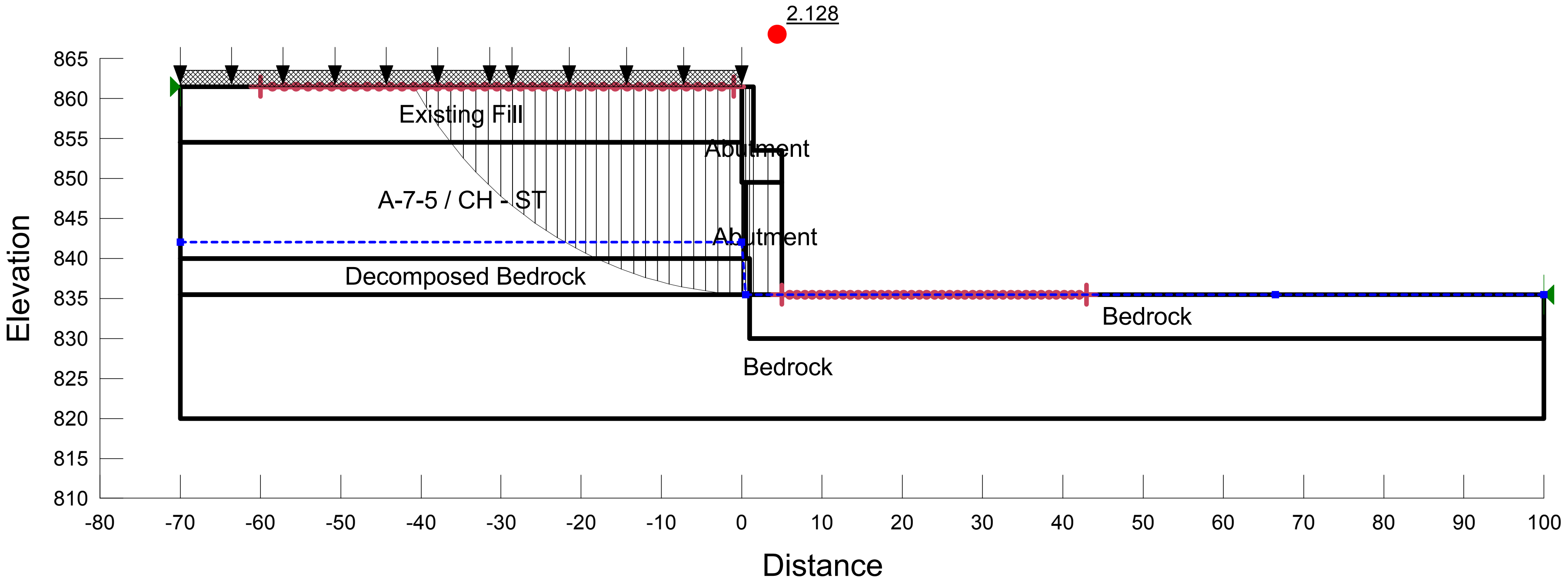


Name: Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 40 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °
Name: A-7-5 / CH - IT Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 200 psf Phi': 20 °
Name: Abutment Model: High Strength Unit Weight: 145 pcf



Name: Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 40 °
Name: A-7-5 / CH - LT Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 40 psf Phi': 20 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °
Name: Abutment Model: High Strength Unit Weight: 145 pcf

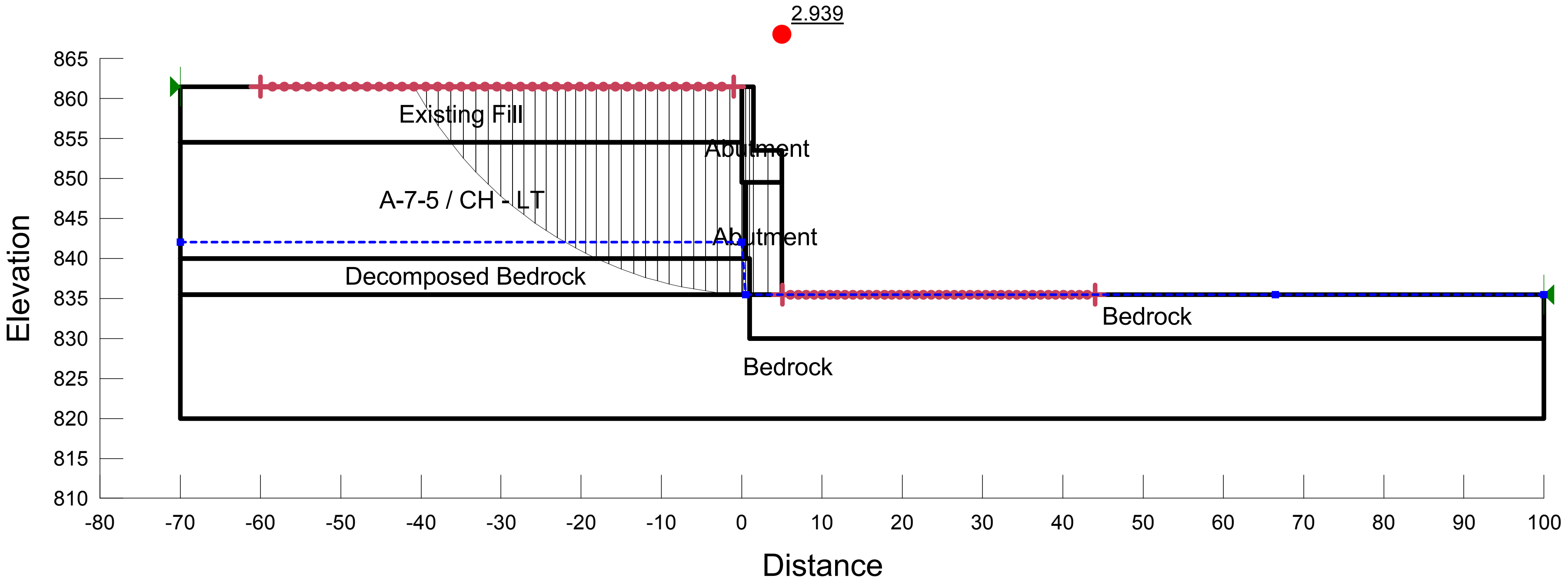
Spillthrough Cut Stability
Short Term



Name: Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 40 °
Name: A-7-5 / CH - ST Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 1,000 psf Phi': 0 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °
Name: Abutment Model: High Strength Unit Weight: 145 pcf

*Note: Stability analysis shown neglects support of the rock socket for the tangent drilled shaft wall.

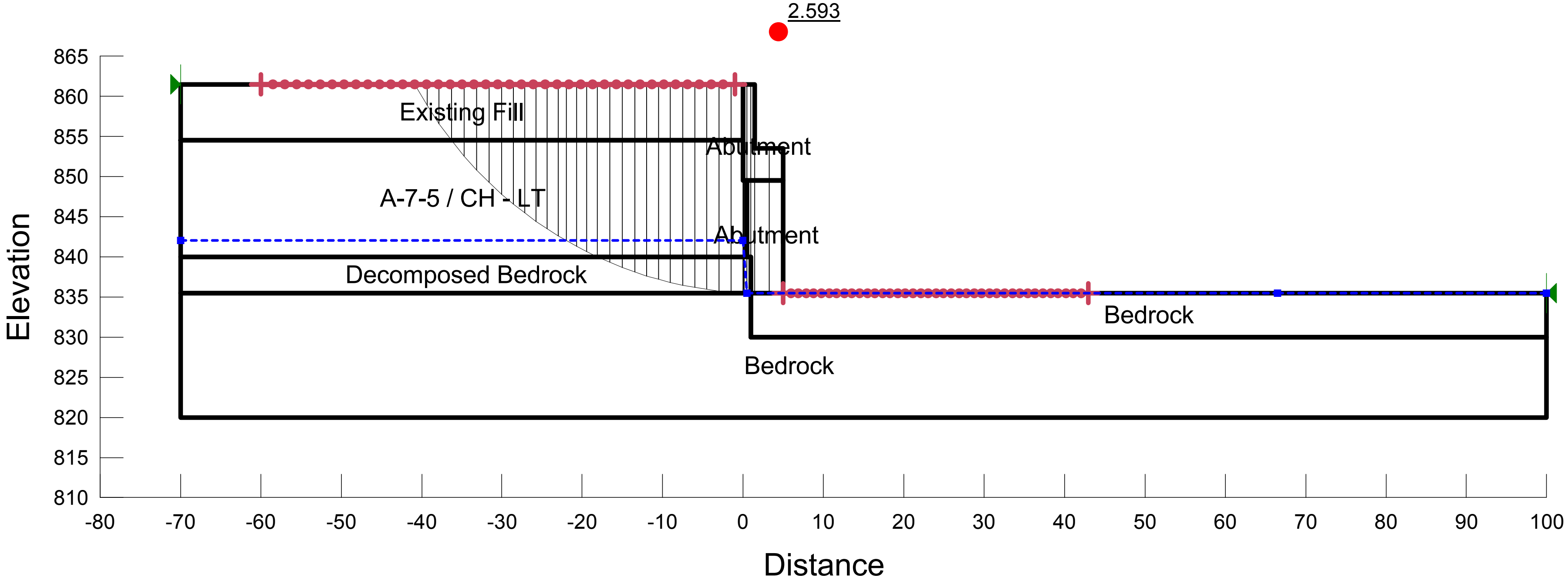
Spillthrough Cut Stability
Long Term



Name: Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 40 °
Name: A-7-5 / CH - LT Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 40 psf Phi': 20 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °
Name: Abutment Model: High Strength Unit Weight: 145 pcf

*Note: Stability analysis shown neglects support of the rock socket for the tangent drilled shaft wall.

Spillthrough Cut Stability
Seismic



Name: Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 6,000 psf Phi': 40 °
Name: A-7-5 / CH - LT Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 40 psf Phi': 20 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 30 °
Name: Decomposed Bedrock Model: Mohr-Coulomb Unit Weight: 145 pcf Cohesion': 0 psf Phi': 40 °
Name: Abutment Model: High Strength Unit Weight: 145 pcf

*Note: Stability analysis shown neglects support of the rock socket for the tangent drilled shaft wall.

SUBJECT

Client KYTC

Project Oldham County Underpass

Item Settlement of Bridge Side Slope

Without Soil Stabilization

JOB NUMBER 0631-0006.02

SHEET NO. OF

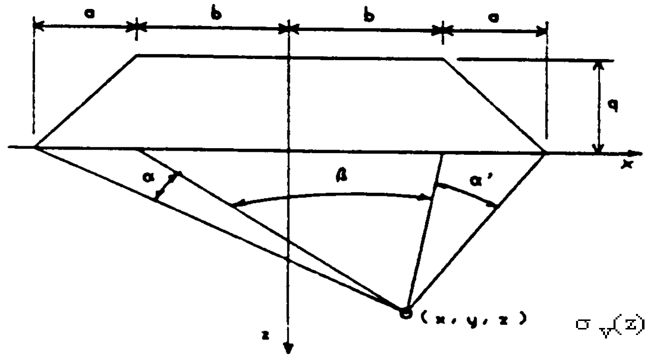
COMP. BY MDK DATE 01/17/15

CHECKED BY HJH DATE 01/17/15

REVISED BY MDK DATE 01/29/15

SETTLEMENT ANALYSIS - EMBANKMENT

Embankment Informaiton:



Groundwater Table: D= 11.0 ft

Embankment Height: H = 15 ft

Fill Unit Weight: γ_{emb} = 125 pcf q = 1,875 psf

Width of Slope: a = 45

Top half-width of Emb: b = 15

Distance from CL: x = 15

Output Range: z = 0 to 13 ft

**See Data output Attached*

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

$$\beta(z) := \operatorname{atan}\left[\frac{(b-x)}{z}\right]+\operatorname{atan}\left[\frac{(b+x)}{z}\right] \qquad \alpha'(z) := \operatorname{atan}\left[\frac{(a+b-x)}{z}\right]-\operatorname{atan}\left[\frac{(b-x)}{z}\right] \qquad \alpha(z) := \operatorname{atan}\left[\frac{(a+b+x)}{z}\right]-\operatorname{atan}\left[\frac{(b+x)}{z}\right]$$

Reference: US Army Corps of Engineers EM 1110-1-1904 "Settlement Analysis", Table C-1

Soil Properites: *Settlement is calculated at mid-point of layer*

								Cohesionless Soils	Cohesive Soils		
No.	Bot. of Laye	Soil Type	γ_{soil} (pcf)	σ'_c (psf)	σ'_o (psf)	$\Delta\sigma z$ (psf)	σ'_f (psf)	C'	C_r	C_c	e_o
1	5.0 ft	A-7-5/CH	130	2,200	325	1,842	2,116	0.0	0.05	0.54	0.990
2	13.0 ft	A-7-5/CH	130	3,120	1,170	1,755	2,749	0.0	0.05	0.54	0.990
3											
4											
5											
6											
7											
8											
9											
10											

`

No.	Settlement:	Total Settlement
1	0.110 ft	
2	0.081 ft	**0.191 ft**
3		
4		
5		**2.3 in**
6		
7		
8		
9		
10		

Reference: Geotechnical Engineering Principles and Practices; Coduto, 1999

Overconsolidated Soils - Case I ($\sigma'_0<\sigma'_c$) Eqn:11.24

$$(\delta_c)_{ult} = \Sigma\frac{C_r}{1+e_0}H\log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

Overconsolidated Soils - Case II ($\sigma'_0<\sigma'_c<\sigma_f$) Eqn:11.25

$$(\delta_c)_{ult} = \Sigma\left[\frac{C_r}{1+e_0}H\log\left(\frac{\sigma'_c}{\sigma'_0}\right)+\frac{C_c}{1+e_0}H\log\left(\frac{\sigma'_f}{\sigma'_c}\right)\right]$$

Normally Consolidated Soils ($\sigma'_0=\sigma'_c$) Eqn: 11.23

$$(\delta_c)_{ult} = \Sigma\frac{C_c}{1+e_0}H\log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

Reference: FHWA NHI-00-045

Cohesionless Soils ($\sigma'_0=\sigma'_c$)

$$(\delta_c)_{ult} = \Sigma\frac{1}{C'}H\log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

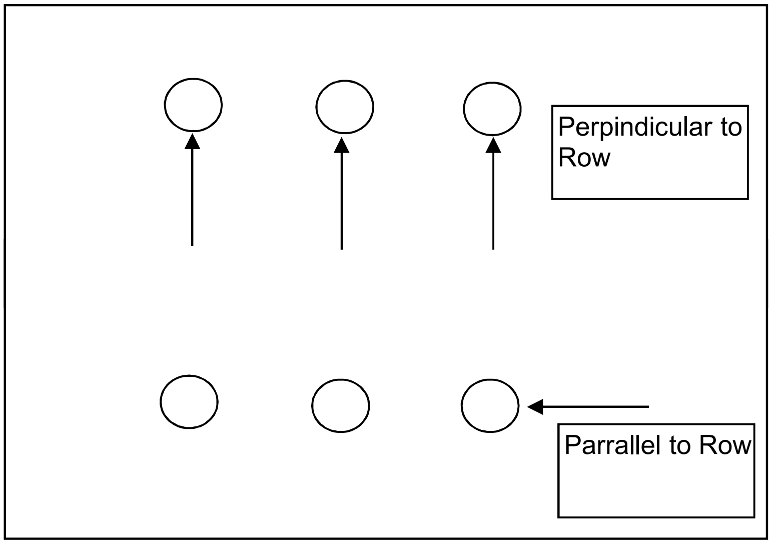
p-y reduction factor

Reference: Reese, GROUP 6.0 Technical Manual page 6-9

Pile Diameter =	3.5	feet	S/B =	1
B	42	inches		
Pile Spacing =	3.5	feet		
S	42	inches		
Loading Perpr to Row [1]	Y	Y/N	β_a =	**0.529**

$$\boxed{\beta_a \approx 0.5292\left(\frac{s}{b}\right)^{0.5659} \quad \text{for} \cdot 1.0 \le \frac{s}{b} \le 3.28}$$

$$\boxed{\beta_a \approx 1.0 \rightarrow \text{for} \cdot \frac{s}{b} \ge 3.28}$$



Recommended Parameters for Single Drilled Shaft Static Lateral Analysis

Approx. Elevation (ft)	Soil/Rock Type	Moist Unit Weight of Soil/Rock - ϒ (pcf)	Buoyant Unit Weight of Soil/Rock - ϒ' (pcf)	LPILE p-y Modulus - k (pci)	Internal Angle of Friction - ϕ (^{o})	Undrained Shear Strength - Su (psf)	RDQ of Rock (%)	Uniaxial Compressive Strength - qu (psi)	Initial Elastic Modulus of Rock Mass - E_i (psi)	Strain Parameter - ε_{50} or k_rm
861 - 855	Existing RR Embankment Fill[1,2]	125	--	90	30	0	--	--	--	--
855 - 842	In-situ Fat Clay[1,3]	130	--	300	0	1,000	--	--	--	0.007
842 - 840	Submerged In-situ Fat Clay[3,4]	--	68	300	0	1,000	--	--	--	0.007
840 - 835	Submerged RDZ[5]	--	83	125	40	0	--	--	--	--
835 -830	Submerged Dolomite[6]	--	83	--	45	--	25	2,000	100,000	0.0005
Below 830	Submerged Dolomite[6]	--	83	--	45	--	45	7,500	450,000	0.0005

[1] Approximately 15 feet of new fill is anticipated to be placed on the south side of the abutments. Assuming properly compacted lean clay with better soil strengths than the in-situ soils, the soil profile with in-situ soils is considered more critical

[2] Anticipated to be modeled as "sand (Reese)"

[3] Anticipated to be modeled as "medium to stiff clay without free water"

[4] Anticipated to be modeled as "medium to stiff clay with free water"

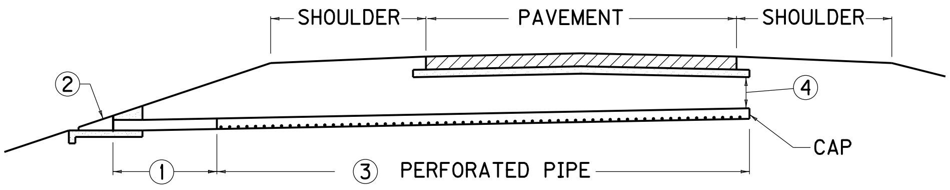
[5] Anticipated to be modeled as "sand (Reese)"

[6] Anticipated to be modeled as "weak rock"

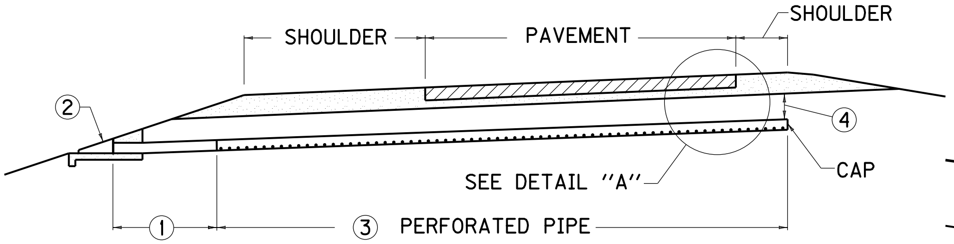
APPENDIX IV

KYTC Standard Drawings
RDP-005
RDP-006
RGX-010

TYPICAL SUBGRADE DRAINAGE LOCATIONS



TANGENT SECTION - TWO LANE



TANGENT SECTION - MULTI LANE

NOTES

SUBGRADE DRAINAGE, AS DEPICTED, IS INTENDED FOR USE WITH THE SURFACING PHASE OF CONSTRUCTION, AND SHALL BE INSTALLED ONLY AFTER THE SUBGRADE HAS BEEN COMPLETED, AND PRIOR TO CONSTRUCTING PAVING MATERIALS.

SUBGRADE DRAINAGE WILL NOT BE REQUIRED WHEN:
  a. ROCK SUBGRADE OR NATURAL BANK GRAVEL IS SPECIFIED.
  b. POROUS OR FREE DRAINING SUBGRADES ARE EVIDENT.
  c. DIRECTED BY THE ENGINEER.

THE CAP SHALL BE A STANDARD MANUFACTURED ITEM FURNISHED BY PIPE SUPPLIER.

① APPROXIMATELY 8 TO 12 FEET OF PIPE AT THE OUTLET SHALL BE NON-PERFORATED PIPE MEETING THE REQUIREMENTS OF THE PERFORATED PIPE, EXCEPT FOR PERFORATIONS.

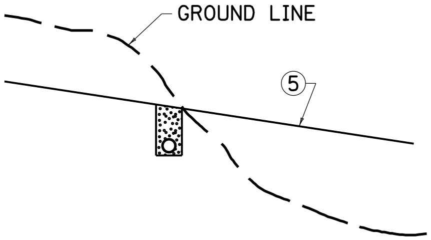
② PERFORATED PIPE HEADWALL REQUIRED AT OUTLET. SEE CUR. STD. DWG. RDP-010 .

③ SEE CURRENT STANDARD DRAWING RDP-001 FOR ALTERNATES.

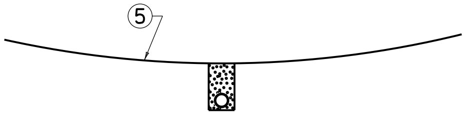
④ PIPE COVER: 2'-0" DESIREABLE MINIMUM, 1'-0" ABSOLUTE MINIMUM.

FLOW SHALL BE DIRECTED TOWARD THE FILL SIDE OF THE ROADWAY WHEN POSSIBLE.

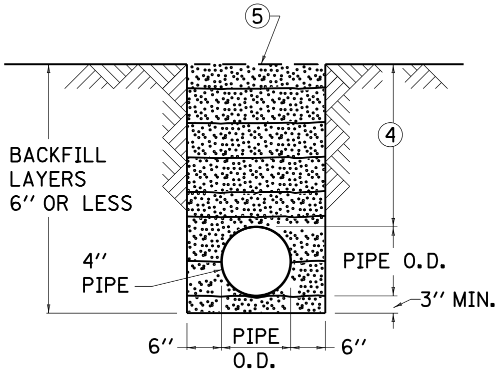
⑤ SUBGRADE ELEVATION.



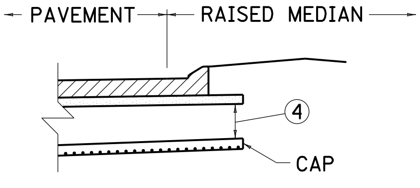
CUT TO FILL



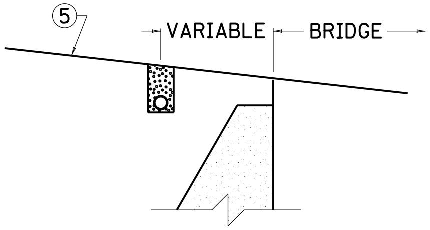
SAG VERTICAL CURVES



TRENCH DETAIL



RAISED MEDIAN DETAIL "A"



SUBGRADE DRAINAGE SHALL BE INSTALLED AT UPGRADE END OF BRIDGE ONLY.

BRIDGES

USE WITH CUR. STD. DWGS. RDP-001 AND RDP-010

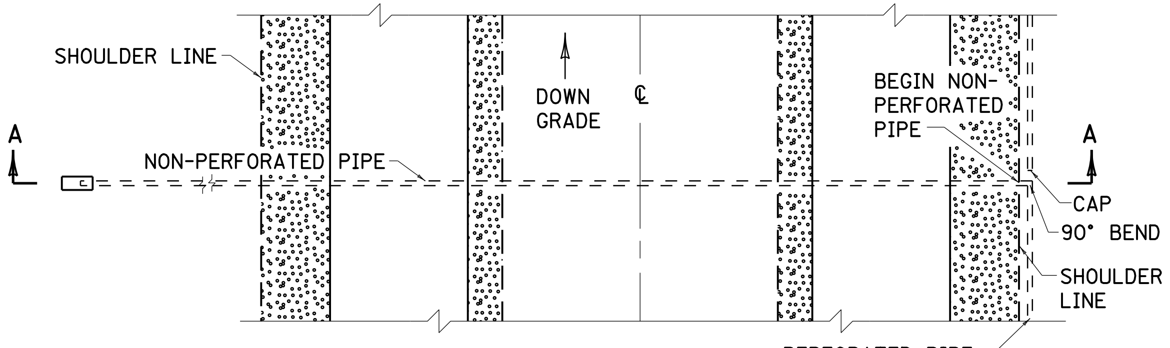
KENTUCKY
DEPARTMENT OF HIGHWAYS

PERFORATED PIPE FOR SUBGRADE DRAINAGE ON TWO-LANE (CLASS 2) AND MULTI-LANE ROADS

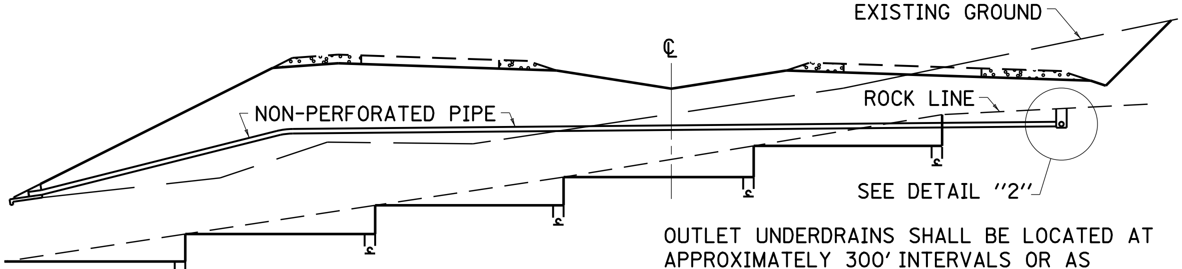
STANDARD DRAWING NO. RDP-005-04

SUBMITTED John B. Sackstider 12-1-99
DIRECTOR DIVISION OF DESIGN DATE
APPROVED J. M. Yowell 12-1-99
STATE HIGHWAY ENGINEER DATE

DETAIL FOR LONGITUDINAL UNDERDRAINS



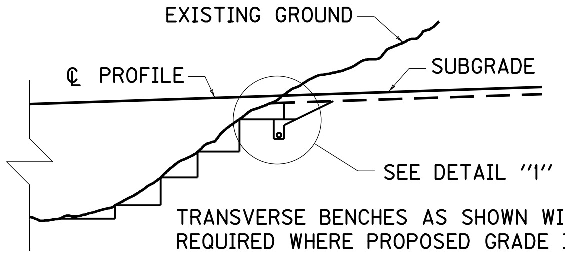
PLAN VIEW



OUTLET UNDERDRAINS SHALL BE LOCATED AT APPROXIMATELY 300′ INTERVALS OR AS DIRECTED BY THE ENGINEER. UNDERDRAINS MAY BE CONNECTED TO CROSS DRAINS.

SECTION A-A

DETAIL FOR TRANSVERSE UNDERDRAIN CUT TO FILL CONDITION



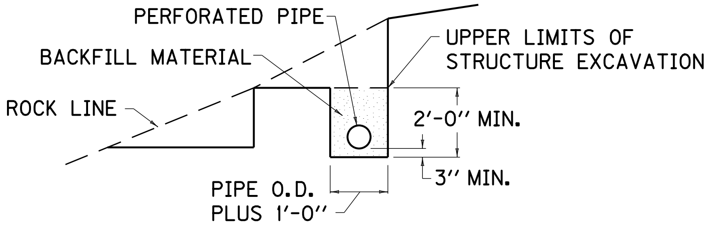
TRANSVERSE BENCHES AS SHOWN WILL BE REQUIRED WHERE PROPOSED GRADE INTERSECTS EXISTING GROUND.

THE EXCAVATION NECESSARY TO FORM THE BENCHES SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE BID FOR ROADWAY EXCAVATION.

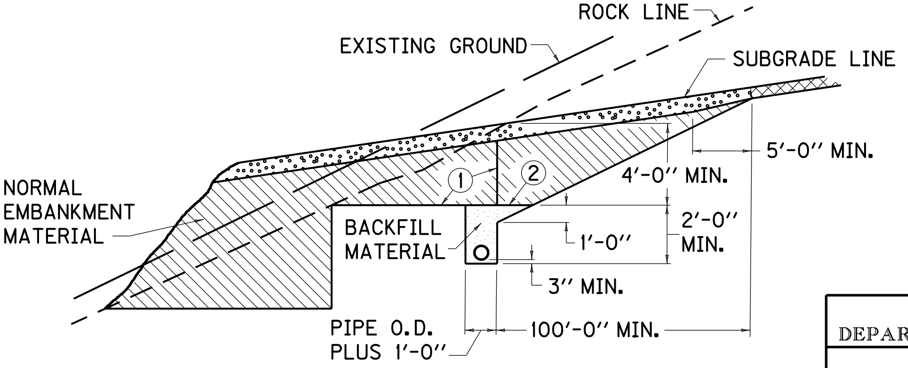
UNDERDRAINS WILL BE REQUIRED ON UPGRADE BENCH. THIS PERFORATED PIPE UNDERDRAIN SHOULD BE PLACED IN ROCK OR SHALE FORMATIONS IF POSSIBLE. PLAN LOCATIONS ARE FOR ESTIMATING PURPOSES ONLY. EXACT LOCATIONS TO BE DETERMINED BY THE ENGINEER ON CONSTRUCTION. THE FOOTAGE THUS INSTALLED SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE PER LINEAR FOOT FOR PERFORATED PIPE WHICH SHALL CONSTITUTE FULL COMPENSATION FOR FURNISHING AND INSTALLING PIPE INCLUDING ALL CONNECTIONS, FITTINGS, FURNISHING AND PLACING AGGREGATE, PLACING BACKFILL, AND FURNISHING ALL LABOR AND TOOLS NECESSARY TO COMPLETE THE WORK.

EXCAVATION FOR BOTH THE PERFORATED AND NON-PERFORATED PIPE SHALL BE MEASURED AND PAID FOR AT THE UNIT PRICE AS SET FORTH IN THE STANDARD SPECIFICATIONS.

BENCHING SHALL BE REQUIRED AT ALL TRANSITIONS FROM ROCK CUTS TO FILL WHETHER OR NOT UNDERDRAIN IS REQUIRED.



DETAIL "2"



DETAIL "1"

NOTES

① LIMITS OF FIRST BENCH.

② UPPER LIMITS OF STRUCTURE EXCAVATION.

ALL PERFORATED PIPE SHALL COMPLY WITH THE STANDARD SPECIFICATIONS.

ALL NON-PERFORATED PIPE SHALL BE THE SAME TYPE AS THE PERFORATED PIPE, EXCEPT WITHOUT PERFORATIONS.

KENTUCKY
DEPARTMENT OF HIGHWAYS

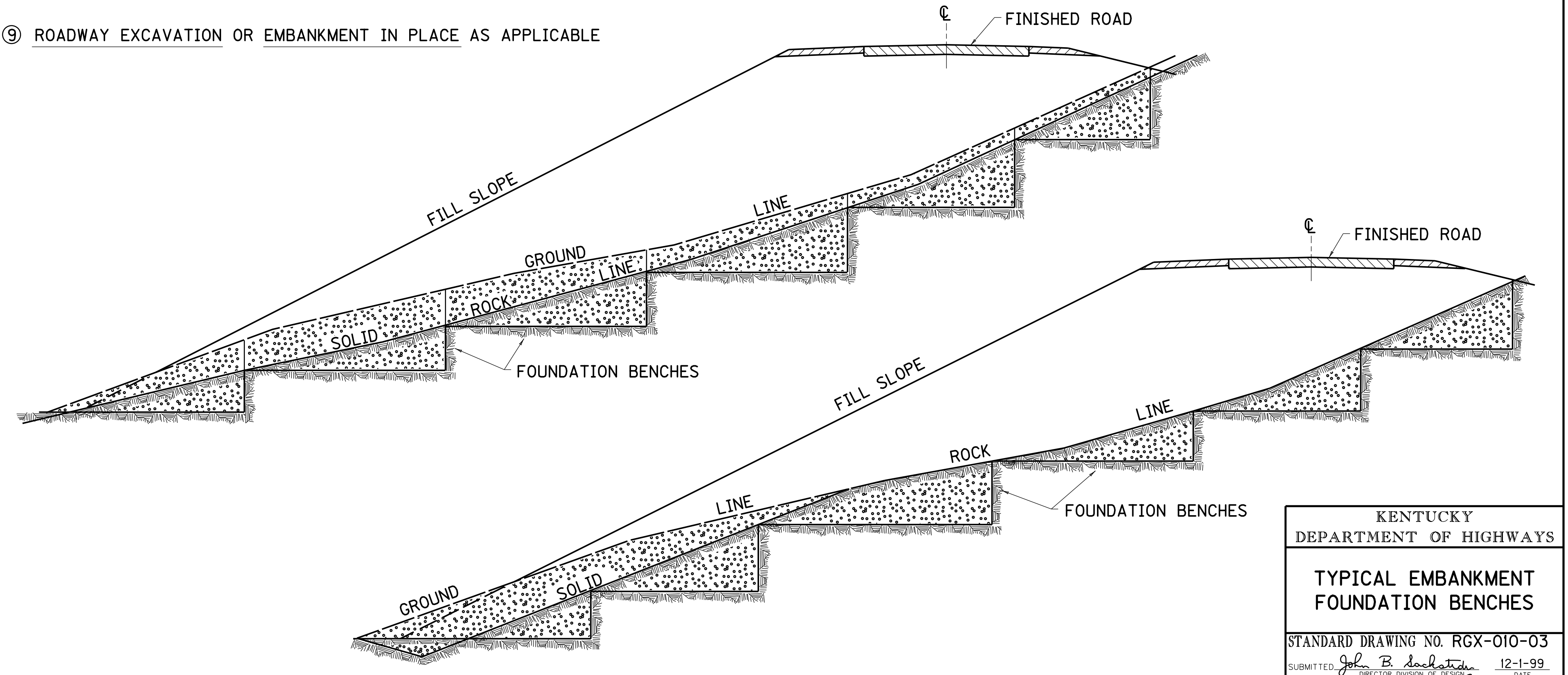
PERFORATED PIPE
UNDERDRAINS

STANDARD DRAWING NO. RDP-006-03
SUBMITTED ________ DIRECTOR DIVISION OF DESIGN 11-21-07 DATE
APPROVED ________ STATE HIGHWAY ENGINEER 11-21-07 DATE

TYPICAL EMBANKMENT FOUNDATION BENCHES

1. THIS TREATMENT FOR EMBANKMENT FOUNDATION BENCHES AS INDICATED ON THIS SHEET, SHALL BE ACCEPTED AS GUIDES FOR HIGHWAY DESIGN, HOWEVER, ALL THE CONDITIONS THAT WILL BE ENCOUNTERED CANNOT BE SHOWN, SO THE DESIGNER MUST GIVE CONSIDERABLE THOUGHT TO THE LOCATIONS AND DIMENSIONS OF THESE BENCHES.
2. DEFINITE DESIGN INFORMATION CANNOT BE ESTABLISHED AS TO SIZE OF THESE BENCHES, DUE TO THE IRREGULARITIES AND THE DIFFERENT RATES OF INCLINE OF THE EXISTING CROSS SECTION, HOWEVER, IT IS GENERALLY BELIEVED THAT A 6′ TO 12′ RISE AND A 20′ TO 35′ HORIZONTAL RUN IS FAIRLY TYPICAL WITH A 15′ HORIZONTAL RUN BEING THE MINIMUM.
3. WHEN THE INCLINE OF THE CROSS SECTION IS 15 PERCENT OR GREATER THESE EMBANKMENT FOUNDATION BENCHES SHALL BE CONSTRUCTED IN THE ORIGINAL SLOPE AS THE EMBANKMENT IS CONSTRUCTED IN COMPACTED LAYERS OR LIFTS.
4. WHEN EMBANKMENT FOUNDATION BENCHES ARE SHOWN ON THE CROSS SECTION, THE VOLUME SHALL BE COMPUTED AS ROADWAY EXCAVATION OR EMBANKMENT IN PLACE AS APPLICABLE AND SHOWN IN THE SHEET TOTALS AND BROUGHT FORWARD TO BE INCLUDED IN THE TOTAL EARTHWORK WITH THIS NOTE ″⑨ TOTAL INCLUDES ″X″ NUMBER OF CUBIC YARDS FROM EMBANKMENT FOUNDATION BENCHES.″
5. THE EXCAVATION FROM THESE BENCHES WILL NOT BE SHOWN IN THE DISTRIBUTION OF QUANTITIES BUT THEY WILL DEFINITELY BE A PAY QUANTITY BY VIRTUE OF THE FACT THEY ARE INCLUDED IN THE TOTAL OF ROADWAY EXCAVATION QUANTITIES.
6. NO QUANTITIES WILL BE ALLOWED FOR THE REFILLING OF THESE BENCHES, SINCE SUPPOSEDLY, THE MATERIAL THAT WAS EXCAVATED WILL BE PROCESSED AND PLACED BACK IN THESE BENCHES.
7. IF THE CROSS SECTION IS AN EARTH ONE, THAT IS IF NO ROCK IS SHOWN, THEN THE FOUNDATION BENCHES SHALL BE INDICATED ON THE CROSS SECTION AND CONSTRUCTED AS SHOWN BY THE DRAWING AND THE VOLUME OF EXCAVATION BECOMES A PAY ITEM AS ROADWAY EXCAVATION OR EMBANKMENT IN PLACE AS APPLICABLE, IN OTHER WORDS, SUPPORT BENCHING OF EARTH SECTIONS SHALL BE GIVEN SAME TREATMENT AS ROCK OR NEAR ROCK SECTION.
8. SHOULD IT BE EVIDENT, AT THE TIME OF CONSTRUCTION, THAT THE ENGINEER FINDS AND SO DIRECTS THAT EMBANKMENT FOUNDATION BENCHING IS NECESSARY AND IT IS NOT SO INDICATED ON THE DESIGN CROSS SECTIONS THE BASIS OF PAYMENT SHALL BE AS HEREIN BEFORE STATED.

⑨ ROADWAY EXCAVATION OR EMBANKMENT IN PLACE AS APPLICABLE



KENTUCKY
DEPARTMENT OF HIGHWAYS

TYPICAL EMBANKMENT FOUNDATION BENCHES

STANDARD DRAWING NO. RGX-010-03

SUBMITTED John B. Sackstader 12-1-99
DIRECTOR DIVISION OF DESIGN DATE

APPROVED J. M. Yowell 12-1-99
STATE HIGHWAY ENGINEER DATE

APPENDIX V

Soil Profile Sheets and Stability Analysis Sheets

GEOTECHNICAL SYMBOLS

COUNTY OF	ITEM NO.	SHEET NO.
OLDHAM	5-434.00	001

AASHTO Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35% or less passing 0.075 mm)							Silt-Clay Materials (More than 35% passing 0.075 mm)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, Percent Passing											
2.00 mm (No. 10)	50 max	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -
0.425 mm (No. 40)	30 max	50 max	51 min	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of Fraction Passing 0.425 mm (No. 40)											
Liquid Limit	- - -		- - -	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min

AI	Activity Index
LI	Liquidity Index
S+C	Silt + Clay (% finer than No.200 Sieve)
○	Rockline Soundings
◕	Disturbed Sample Boring
◎	Undisturbed Sample Boring
⦿	Undisturbed Sample Boring & Rock Core
●	Rock Core
⌖	Slope Inclinometer Installation typical applications: ⌖ ◕ ◎ ⦿ ●
OW	Observation Well
➡	Approximate Footing Elevation
▼ (Date)	Water Elevation

VS (psf)	Field Vane Shear Strength
■	Thin-walled Tube Sample
<	Standard Penetration Test Sample
N	Penetration Resistance
Qu (psf)	Unconfined Compressive Strength
UU (psf)	Unconsolidated Undrained Triaxial Strength
w%	Moisture Content
KY RQD	Rock Quality Designation (Kentucky Method)
STD RQD	Rock Quality Designation (Standard Method)
SDI(JS)	Slake Durability Index (Jar Slake Test)
REC	Core Recovery
$\emptyset$	Angle of Internal Friction (Total Stress)
$\overline{\emptyset}$	Angle of Internal Friction (Effective Stress)
c (psf)	Cohesion (Total Stress)
$\overline{c}$ (psf)	Cohesion (Effective Stress)
γ (pcf)	Total Unit Weight
RDZ	Rock Disintegration Zone
OB	Overburden Bench
IB	Intermediate Bench
R	Refusal
NR	Refusal Not Encountered

Unified Soil Classifications

MAJOR DIVISIONS		SYMBOL		NAME
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	▨	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	▨	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GM	▨	Silty gravels, gravel-sand-silt mixtures.
		GC	▨	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	▨	Well graded sands or gravelly sands, little or no fines.
		SP	▨	Poorly graded sands or gravelly sands, little or no fines.
		SM	▨	Silty sands, sand-silt mixtures.
		SC	▨	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 50	ML	▨	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays silty clays, lean clays.
		ML-CL	▨	Silty clay-silty clay with sand and or gravel, sandy silty clay, sandy silty clay with gravel, gravelly silty clay, gravelly silty clay with sand
	SILTS AND CLAYS LL IS GREATER THAN 50	MH	▨	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	▨	Inorganic clays of high plasticity, fat clays.

Unified Soil Classifications - Continued

MAJOR DIVISIONS		SYMBOL		NAME
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GP-GC	▨	Poorly graded gravel with clay (or silty clay), poorly graded gravel with clay and sand (or silty clay & sand)
		GP-GM	▨	Poorly graded gravel with silt, poorly graded gravel with silt and sand
		GW-GC	▨	Well graded gravel with clay (or silty clay), well graded gravel with clay and sand (or silty clay and sand)
		GW-GM	▨	Well graded gravel with silt, well graded gravel with silt and sand
		GC-GM	▨	Silty clayey gravel, silty clayey gravel with sand
	SAND AND SANDY SOILS	SW-SC	▨	Well graded sand with clay (or silty clay), well graded sand with clay and gravel (or silty clay & gravel)
		SP-SC	▨	Poorly graded sand with clay (or silty clay), poorly graded sand with clay and gravel (or silty clay and gravel)
		SP-SM	▨	Poorly graded sand with silt, poorly graded sand with silt and gravel
		SC-SM	▨	Silty clayey sand, silty clayey sand with gravel
		SW-SM	▨	Well graded sand with silt, well graded sand with silt and gravel
UNCLASSIFIED MATERIAL		OH		Organic (High Plasticity)
		OL		Organic (Low Plasticity)

▦	LIMESTONE	▨	TALUS, MINE WASTE, FILL MATERIAL, BOULDERS, & ETC.
⁙	SANDSTONE	■	COAL
▨	DURABLE SHALE (SDI ≥ 95)	▨	DOLOMITE
▨	NONDURABLE SHALE (SDI < 95)	▦	LIMESTONE (ARGILLACEOUS)
≈	GRANULAR EMBANKMENT	○	SLOPE PROTECTION
▨	STRUCTURE GRANULAR BACKFILL		

GEOTECHNICAL SYMBOL SHEET

FILE NAME: M:\PROJ\0631\0006\02\CADD\SHEETS\ROADWAY\R0010AGT.DGN

USER: mkennedy
DATE PLOTTED: September 3, 2015

MicroStation v8.11.9.357

GEOTECHNICAL NOTES

1. THE PROPOSED BRIDGE PIERS OR ABUTMENTS CAN BE SUPPORTED ON DRILLED SHAFT FOUNDATIONS DERIVING SUPPORT WITHIN THE UNDERLYING DOLOMITE FROM A COMBINATION OF BASE AND SIDE RESISTANCE. THE DRILLED SHAFTS SHOULD BE EXTENDED TO A TIP ELEVATION A MINIMUM OF 1.5 TIMES THE SHAFT DIAMETER (ABOVE BEDROCK ELEVATION) BELOW THE BASE OF RDZ ELEVATION APPROXIMATELY AT 835 FEET.

2. A MINIMUM SHAFT DIAMETER ABOVE THE TOP OF BEDROCK OF 3 FEET SHOULD BE USED. IF CASING IS USED TO INSTALL THE DRILLED SHAFTS, THE ROCK SOCKET DIAMETER SHOULD BE 6 INCHES SMALLER THAN THE SHAFT DIAMETER ABOVE THE BEDROCK ELEVATION.

3. ADJACENT SHAFTS SHOULD NOT BE CONSTRUCTED ON THE SAME DAY.

4. CASING MAY BE REQUIRED IN THE OVERBURDEN FOR THE DRILLED SHAFT INSTALLATION. HOWEVER, THE CASING IS NOT REQUIRED TO BE LEFT IN-PLACE AFTER INSTALLATION OF THE DRILLED SHAFT FOUNDATIONS IS COMPLETED FOR GEOTECHNICAL RESISTANCE AND THEREFORE MAY BE REMOVED, UNLESS REQUIRED FOR STRUCTURAL RESISTANCE. THE APPROPRIATE MEANS AND METHODS SHOULD BE EMPLOYED BY THE CONTRACTOR TO PROVIDE INTEGRITY OF THE COMPLETED SHAFTS AND MITIGATE NECKING, VOIDS, ETC.

5. ALL DRILLED SHAFT SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE SPECIAL NOTE FOR DRILLED SHAFTS, CURRENT EDITION AND THE RECOMMENDATIONS IN SECTION 7.0, GEOTECHNICAL RECOMMENDATIONS, OF THE GEOTECHNICAL REPORT.

6. THE SIDE FILL SLOPES AT THE SOUTH SIDE OF THE ABUTMENTS SHALL BE CONSTRUCTED WITH 2H:1V OR FLATTER SLOPES. EMBANKMENT STABILITY ANALYSES WERE CONDUCTED USING ESTIMATED SOIL STRENGTH PARAMETERS FOR COHESIVE SOILS WITH LOW SWELL POTENTIAL COHESIVE SOILS (ACCORDING TO USCS OR AASHTO SOIL CLASSIFICATION) WITH LIQUID LIMIT (AS DETERMINED BY AASHTO T-89) AND PLASTICITY INDEX (AS DETERMINED BY AASHTO T-90) OF LESS THAN 50 AND 30, RESPECTIVELY. ALL FILL MATERIALS SHALL BE COMPACTED IN ACCORDANCE WITH SECTION 206 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION. ALL FILL MATERIALS SHALL BE APPROVED BY THE ENGINEER PRIOR TO THE PLACEMENT OF FILL.

7. SETTLEMENT PLATFORMS ARE RECOMMENDED TO MONITOR THE ACTUAL AMOUNT OF SETTLEMENT DUE TO ADDED EMBANKMENT FILL AT THE SOUTH SIDE OF ABUTMENTS.

8. THE CUT SLOPES FOR THE SPILL-THROUGH SLOPES AT THE PROPOSED TANGENT DRILLED SHAFT-SUPPORTED BRIDGE ABUTMENTS SHALL BE CONSTRUCTED WITH 3H:1V OR FLATTER SLOPES. IN-SITU GROUND IMPROVEMENT TECHNIQUES OR OTHER SOIL STABILIZATION TECHNIQUES, COULD BE USED TO IMPROVE GLOBAL STABILITY OF THE SPILL-THROUGH CUT SLOPES POTENTIALLY REQUIRING STEEPER SLOPES.

9. THE BEDROCK, EXCEPT FOR THOSE IN RDZ, IS STRONG DOLOMITE/LIMESTONE WITH OCCASIONAL INTERBEDDED SHALE. HARD ROCK EXCAVATION TECHNIQUES WILL BE NECESSARY TO REMOVE THE DOLOMITE/LIMESTONE. ROCK BLASTING IS NOT PERMITTED FOR THIS PROJECT. IT IS THE CONTRACTOR$^{5}/_{32}$ S RESPONSIBILITY TO SELECT THE APPROPRIATE AND ACCEPTABLE MEANS AND METHODS OF CONSTRUCTION AND ADEQUATE CONSTRUCTION EQUIPMENT BASED ON THE ANTICIPATED SUBSURFACE CONDITIONS AND TO PREVENT DAMAGE TO ADJACENT STRUCTURES OR FACILITIES.

10. FOUNDATION PREPARATION AND BACKFILLING SHALL BE PERFORMED IN ACCORDANCE WITH SECTION 603 OF THE STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION.

11. CLEARING AND GRUBBING OF EMBANKMENT AREAS SHALL BE COMPLETED IN ACCORDANCE WITH SECTION 202 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION.

12. REMOVAL OF EXISTING STRUCTURES AND OTHER OBSTRUCTIONS SHALL, WHETHER SHOWN ON THE PLANS OR NOT, SHALL BE COMPLETED IN ACCORDANCE WITH SECTION 203 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION.

13. ALL EXCAVATIONS SHALL BE COMPLETED IN ACCORDANCE WITH SECTION 204 OF THE CURRENT KENTUCKY DEPARTMENT OF TRANSPORTATION CABINET (KYTC) STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION.

14. IF APPLICABLE, ALL WATER WELLS WITHIN THE LIMITS OF CONSTRUCTION, WHETHER SHOWN ON THE PLANS OR NOT, SHALL BE PLUGGED IN ACCORDANCE WITH REQUIREMENTS OF SECTION 708 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION.

15. IF APPLICABLE, ALL CATCH BASINS AND MANHOLES SHALL BE FILLED AND CAPPED, AND ALL SEPTIC TANKS, IF ANY, SHALL BE CLEANED AND FILLED IN ACCORDANCE WITH SECTION 708 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION.

16. ANY DRAINAGE SWALES, SATURATED, SOFT AND UNSTABLE AREAS ENCOUNTERED WITHIN PROPOSED EMBANKMENT FOUNDATION LIMITS AND/OR ANY OTHER AREAS AS SPECIFIED BY THE ENGINEER SHALL BE STABILIZED WITH A MINIMUM OF TWO FEET (VERTICAL THICKNESS) OF GRANULAR MATERIALS (KYTC COARSE AGGREGATE NO. 2) IN ACCORDANCE WITH SECTION 805 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION, AND THE MATERIALS SHALL BE CLASSIFIED AS NON-ERODIBLE, AS DIRECTED BY THE ENGINEER. ADDITIONAL GRANULAR MATERIAL MAY BE REQUIRED TO STABILIZE THE EMBANKMENT FOUNDATIONS AND TO MAINTAIN POSITIVE DRAINAGE. THE ACTUAL THICKNESS AND LOCATIONS OF GRANULAR MATERIAL SHALL BE DETERMINED BY THE ENGINEER DURING CONSTRUCTION. THE GRANULAR MATERIALS SHALL BE WRAPPED (TOP AND BOTTOM) WITH TYPE IV GEOTEXTILE FABRIC IN ACCORDANCE WITH SECTIONS 214 AND 843 OF THE CURRENT STANDARD SPECIFICATIONS. POSITIVE DRAINAGE SHALL BE MAINTAINED TO PREVENT TRAPPING WATER WITHIN THE ROADWAY EMBANKMENT.

17. THE CONTRACTOR IS RESPONSIBLE FOR CONDUCTING ANY OPERATIONS NECESSARY IN ORDER TO EXCAVATE THE CUT AREAS TO THE REQUIRED TYPICAL SECTIONS. THESE OPERATIONS SHALL BE INCIDENTAL TO THE ROADWAY EXCAVATION PRICE.

18. THE CONTRACTOR SHALL CONDUCT GRADING OPERATIONS IN SUCH A MANNER THAT BEDROCK OBTAINED FROM EXCAVATION BELOW THE BASE OF RDZ SHALL BE STOCKPILED SEPARATELY OR OTHERWISE RE-CONDITIONED SO AS TO BE AVAILABLE FOR USE IN THOSE AREAS REQUIRING SAID MATERIAL. NO DIRECT PAYMENT WILL BE ALLOWED FOR SUCH NECESSARY RE-CONDITIONING AS STOCKPILING, RE-HANDLING THE MATERIAL, AND/OR HAULING.

19. THE CONSTRUCTION OF THE EMBANKMENT SHALL BE COMPLETED IN ACCORDANCE WITH SECTION 206 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION.

20. ALL CHANNEL CHANGES, EXCAVATION OF SURFACE DITCHES, AND CONSTRUCTION OF SPECIAL DITCHES SHALL BE PERFORMED IN ACCORDANCE WITH SECTIONS 204 AND 206 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION, PRIOR TO PLACEMENT OF ANY EMBANKMENT MATERIALS ADJACENT TO THEM. THE CONSTRUCTION OF THE EMBANKMENT SHALL BE COMPLETED IN ACCORDANCE WITH SECTION 206 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION. AT THE DIRECTION OF THE ENGINEER, MATERIALS EXCAVATED FROM THESE AREAS MAY BE UTILIZED IN CONSTRUCTION OF THE EMBANKMENTS, BUT MAY REQUIRE AERATION OR OTHER MOISTURE ADJUSTMENTS TO OBTAIN PROPER MOISTURE CONTENTS PRIOR TO COMPACTION OPERATIONS.

21. IN ACCORDANCE WITH SECTION 206 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION, THE MOISTURE CONTENT OF EMBANKMENT MATERIAL SHALL NOT VARY FROM THE OPTIMUM MOISTURE CONTENT, AS DETERMINED BY KME 64-511, BY MORE THAN PLUS OR MINUS TWO PERCENT. THIS MOISTURE CONTENT REQUIREMENT SHALL HAVE EQUAL WEIGHT WITH THE DENSITY REQUIREMENT WHEN DETERMINING THE ACCEPTABILITY OF EMBANKMENT OR SUBGRADE CONSTRUCTION. REFER TO THE FAMILY OF CURVES FOR MOISTURE-DENSITY RELATIONSHIPS.

22. FAT CLAY OR SHALE SHALL NOT BE USED IN CONSTRUCTION OF THE EMBANKMENTS.

23. ALL SOILS, WHETHER FROM ROADWAY EXCAVATION OR BORROW, MAY REQUIRE MOISTURE ADJUSTMENTS TO OBTAIN PROPER MOISTURE CONTENT PRIOR TO COMPACTION. DIRECT PAYMENT SHALL NOT BE PERMITTED FOR RE-HANDLING, HAULING, STOCKPILING AND/OR MOISTURE ADJUSTING SOILS.

24. ALL NEW FILL MATERIALS SHALL BE FREE OF TOPSOIL, ORGANICS, DEBRIS, OR ANY DELETERIOUS MATERIAL DEEMED BY THE ENGINEER. NO FROZEN MATERIALS SHALL BE INCORPORATED INTO THE FILL, AND NO EMBANKMENT, PAVEMENT, UTILITIES, OR FILL SHALL BE PLACED ON TOP OF FROZEN MATERIALS. ONLY SUITABLE MATERIALS DEEMED BY THE ENGINEER SHALL BE USED AS NEW FILL MATERIALS.

25. NO PARTICLE SIZE LARGER THAN FOUR INCHES IN ANY DIRECTION, UNLESS DIRECTED BY THE ENGINEER, SHALL BE PLACED AS FILL WITHIN ONE FOOT OF THE FINISHED SUBGRADE ELEVATION. ANY PARTICLE SIZE GREATER THAN FOUR INCHES SHALL BE BROKEN DOWN TO LESS THAN FOUR INCHES, OR REMOVED FROM THE LIFT.

26. WHEN ROCK, INCLUDING SHALE, IS PRESENT AT THE ROADWAY SUBGRADE, WITHIN 12 INCHES OF THE BOTTOM OF THE DGA, IT SHALL BE REMOVED IN ACCORDANCE WITH SECTION 204 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION AND REPLACED WITH SOIL FILL IN ACCORDANCE WITH SECTIONS 206 AND 207 OF THE CURRENT STANDARD SPECIFICATIONS.

27. A MINIMUM ONE-FOOT WORKING PLATFORM (EXTENDING UNDER THE CURB AND GUTTER) WILL BE REQUIRED IN AREAS WHERE THE ROADWAY SUBGRADE IS SOFT AND/OR SATURATED. THE PLATFORM WILL CONSIST OF KENTUCKY COARSE AGGREGATE #2 IN ACCORDANCE WITH SECTION 805 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION. THE WORKING PLATFORM SHALL BE WRAPPED WITH TYPE IV GEOTEXTILE FABRIC IN ACCORDANCE WITH SECTIONS 214 AND 843 OF THE CURRENT STANDARD SPECIFICATIONS. THE ACTUAL LOCATIONS AND THICKNESS SHALL BE DETERMINED BY THE ENGINEER DURING CONSTRUCTION AND MAY DEPEND ON SEASONAL FLUCTUATIONS IN THE WATER TABLE. THE WORKING PLATFORM CAN ALSO SERVE AS A DRAINAGE BLANKET BY PLACING PERFORATED DRAIN PIPE INTO THE BOTTOM OF THE GRANULAR MATERIAL. POSITIVE DRAINAGE OF THE PERFORATED DRAIN PIPE SHALL BE MAINTAINED TO REDUCE THE POSSIBILITY OF TRAPPING WATER WITHIN THE SUBGRADE.

28. PAVEMENT UNDERDRAINS SHALL BE PROVIDED TO REDUCE THE POSSIBILITY OF TRAPPING WATER WITHIN THE SUBGRADE.

29. THE CUT SLOPES IN THE ROCK DISINTEGRATION ZONES AND BEDROCK SHALL BE CONSTRUCTED WITH 2H:1V OR FLATTER SLOPES.

30. EMBANKMENT STABILITY ANALYSES WERE CONDUCTED USING ESTIMATED SOIL STRENGTH PARAMETERS FOR EMBANKMENT MATERIAL. EMBANKMENT STABILITY ANALYSES WERE CONDUCTED USING ESTIMATED SOIL STRENGTH PARAMETERS FOR COHESIVE SOILS WITH LOW SWELL POTENTIAL COHESIVE SOILS (ACCORDING TO USCS OR AASHTO SOIL CLASSIFICATION) WITH LIQUID LIMIT (AS DETERMINED BY AASHTO T-89) AND PLASTICITY INDEX (AS DETERMINED BY AASHTO T-90) OF LESS THAN 50 AND 30, RESPECTIVELY. ALL FILL MATERIALS SHALL BE COMPACTED IN ACCORDANCE WITH SECTION 206 OF THE CURRENT KYTC STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION. ALL FILL MATERIALS SHALL BE APPROVED BY THE ENGINEER PRIOR TO THE PLACEMENT OF FILL.

31. EMBANKMENT FOUNDATION BENCHES ARE NOT ANTICIPATED AT THIS TIME. IF REQUESTED BY THE ENGINEER, EMBANKMENT FOUNDATION BENCHES/SLOPE SERRATIONS SHALL BE CONSTRUCTED AND PERFORATED PIPE UNDERDRAINS SHALL BE PLACED, AS APPLICABLE, IN ACCORDANCE WITH THE CURRENT KYTC STANDARD DRAWINGS RDP-006 AND RGX-010. THE BENCHES SHALL BE CONSTRUCTED ONE AT A TIME BEGINNING WITH THE LOWEST BENCH. EACH BENCH SHALL BE BACKFILLED PRIOR TO EXCAVATION OF THE NEXT BENCH. THESE PROCEDURES SHALL BE FOLLOWED TO HELP MAINTAIN STABILITY OF THE EXISTING SLOPES.

32. TRANSVERSE BENCHING IS NOT ANTICIPATED AT THIS TIME. IF REQUESTED BY THE ENGINEER, TRANSVERSE BENCHING SHALL BE CONSTRUCTED AND PERFORATED PIPE UNDERDRAINS SHALL BE PLACED, AS APPLICABLE, IN ACCORDANCE WITH THE CURRENT KYTC STANDARD DRAWINGS AND RDP-005 AND RDP-006, AS APPLICABLE. CONTRARY TO THE STANDARD DRAWING RDP-006, TRANSVERSE BENCHES AND PERFORATED PIPE UNDERDRAINS SHALL BE INSTALLED IN BOTH UPHILL AND DOWNHILL TRANSITION AREAS BETWEEN CUTS AND FILLS. IN ADDITION, PERFORATED PIPE UNDERDRAINS SHALL BE INSTALLED IN ANY AREAS SHOWING SIGNS OF SEEPAGE DURING CONSTRUCTION, AS DIRECTED BY THE ENGINEER.

DATE: 09/04/2015
DESIGNED BY: E. Tse
DETAILED BY: M. Kennedy
CHECKED BY
H.J. Hughes

OLDHAM COUNTY
FISCAL COURT

COUNTY
OLDHAM

ROUTE

CROSSING
ALLEN LANE

ITEM NUMBER
5–434.00

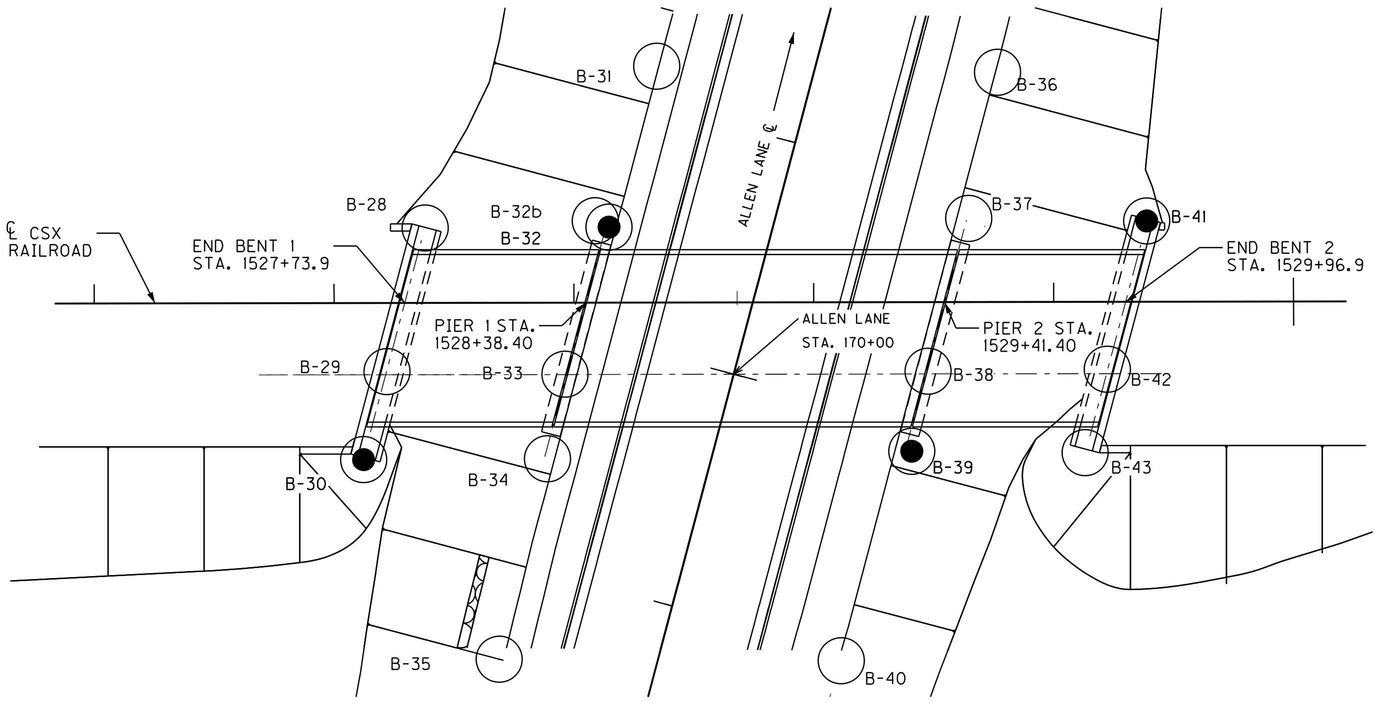
PREPARED BY
DLZ
KENTUCKY, INC.
201 BRIGHTON PARK BLVD.
FRANKFORT, KENTUCKY 40602
(502) 695-2300

SHEET NO.
002
DRAWING NO.

FILE NAME: M:\PROJ\0631\0006\02\CADD\SHEETS\STRUCTURE\SUBMITTAL SET\10-01-2015\S0000_002.DGN
USER: mkennedy
DATE PLOTTED: October 1, 2015
MicroStation v8.11.9.357

SUBSURFACE DATA

Plan Scale: 1" = 40'



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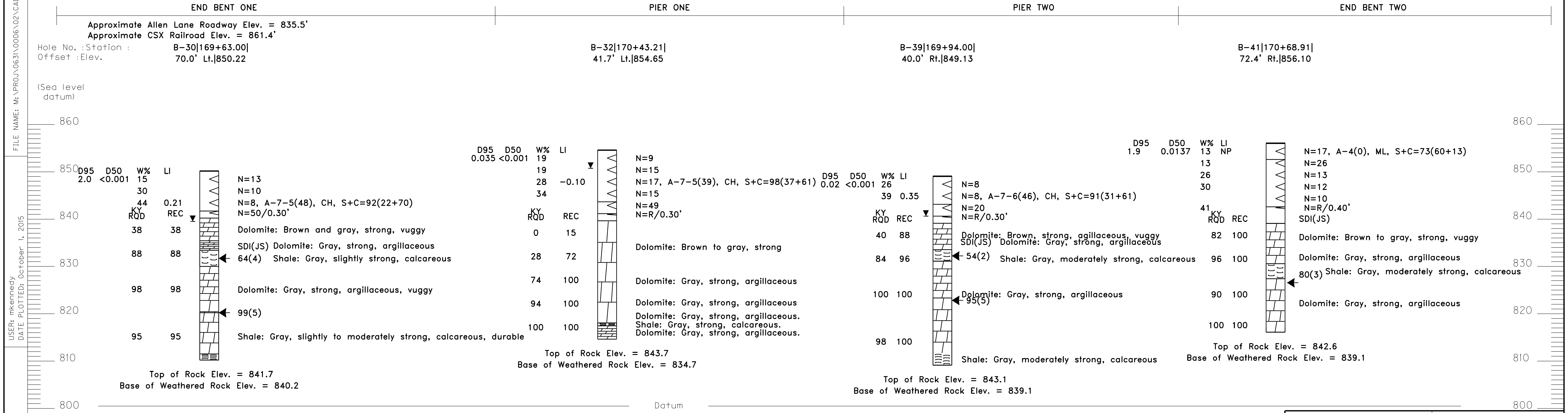
Vertical 1" = 20'
Horizontal not to scale



Approximate Allen Lane Roadway Elev. = 835.5'
Approximate CSX Railroad Elev. = 861.4'

Hole No. :Station :
Offset :Elev.

B-30|169+63.00|
70.0' Lt.|850.22

D95	D50	W%	LI	
2.0	<0.001	15		N=13
		30		N=10
		44	0.21	N=8, A-7-5(48), CH, S+C=92(22+70)
				N=50/0.30'

KY RQD	REC	
38	38	Dolomite: Brown and gray, strong, vuggy
		SDI(JS) Dolomite: Gray, strong, argillaceous
88	88	64(4) Shale: Gray, slightly strong, calcareous
98	98	Dolomite: Gray, strong, argillaceous, vuggy
		99(5)
95	95	Shale: Gray, slightly to moderately strong, calcareous, durable

Top of Rock Elev. = 841.7
Base of Weathered Rock Elev. = 840.2

B-32|170+43.21|
41.7' Lt.|854.65

D95	D50	W%	LI	
0.035	<0.001	19		N=9
		19		N=15
		28	-0.10	N=17, A-7-5(39), CH, S+C=98(37+61)
		34		N=15
				N=49
				N=R/0.30'

KY RQD	REC	
0	15	
28	72	Dolomite: Brown to gray, strong
74	100	Dolomite: Gray, strong, argillaceous
94	100	Dolomite: Gray, strong, argillaceous
100	100	Dolomite: Gray, strong, argillaceous. Shale: Gray, strong, calcareous. Dolomite: Gray, strong, argillaceous.

Top of Rock Elev. = 843.7
Base of Weathered Rock Elev. = 834.7

B-39|169+94.00|
40.0' Rt.|849.13

D95	D50	W%	LI	
0.02	<0.001	26		N=8
		39	0.35	N=8, A-7-6(46), CH, S+C=91(31+61)
				N=20
				N=R/0.30'

KY RQD	REC	
40	88	Dolomite: Brown, strong, agillaceous, vuggy SDI(JS) Dolomite: Gray, strong, argillaceous
84	96	54(2) Shale: Gray, moderately strong, calcareous
100	100	Dolomite: Gray, strong, argillaceous 95(5)
98	100	Shale: Gray, moderately strong, calcareous

Top of Rock Elev. = 843.1
Base of Weathered Rock Elev. = 839.1

B-41|170+68.91|
72.4' Rt.|856.10

D95	D50	W%	LI	
1.9	0.0137	13	NP	N=17, A-4(0), ML, S+C=73(60+13)
		13		N=26
		26		N=13
		30		N=12
				N=10
		41		N=R/0.40'

KY RQD	REC	
		SDI(JS)
82	100	Dolomite: Brown to gray, strong, vuggy
96	100	Dolomite: Gray, strong, argillaceous
		80(3) Shale: Gray, moderately strong, calcareous
90	100	Dolomite: Gray, strong, argillaceous
100	100	

Top of Rock Elev. = 842.6
Base of Weathered Rock Elev. = 839.1

FILE NAME: M:\PROJ\0631\0006\02\CADD\SHEETS\STRUCTURE\SUBMITTAL SET\10-01-2015\S00000_003.DGN
USER: mkennedy
DATE PLOTTED: October 1, 2015
MicroStation v8.11.9.357

DATE: 01-OCTOBER-2015	CHECKED BY
DESIGNED BY: E. Tse	H.J. Hughes
DETAILED BY: M. KENNEDY	

OLDHAM COUNTY
FISCAL COURT

COUNTY
OLDHAM

ROUTE
—
CROSSING
ALLEN LANE UNDERPASS

SUBSURFACE DATA

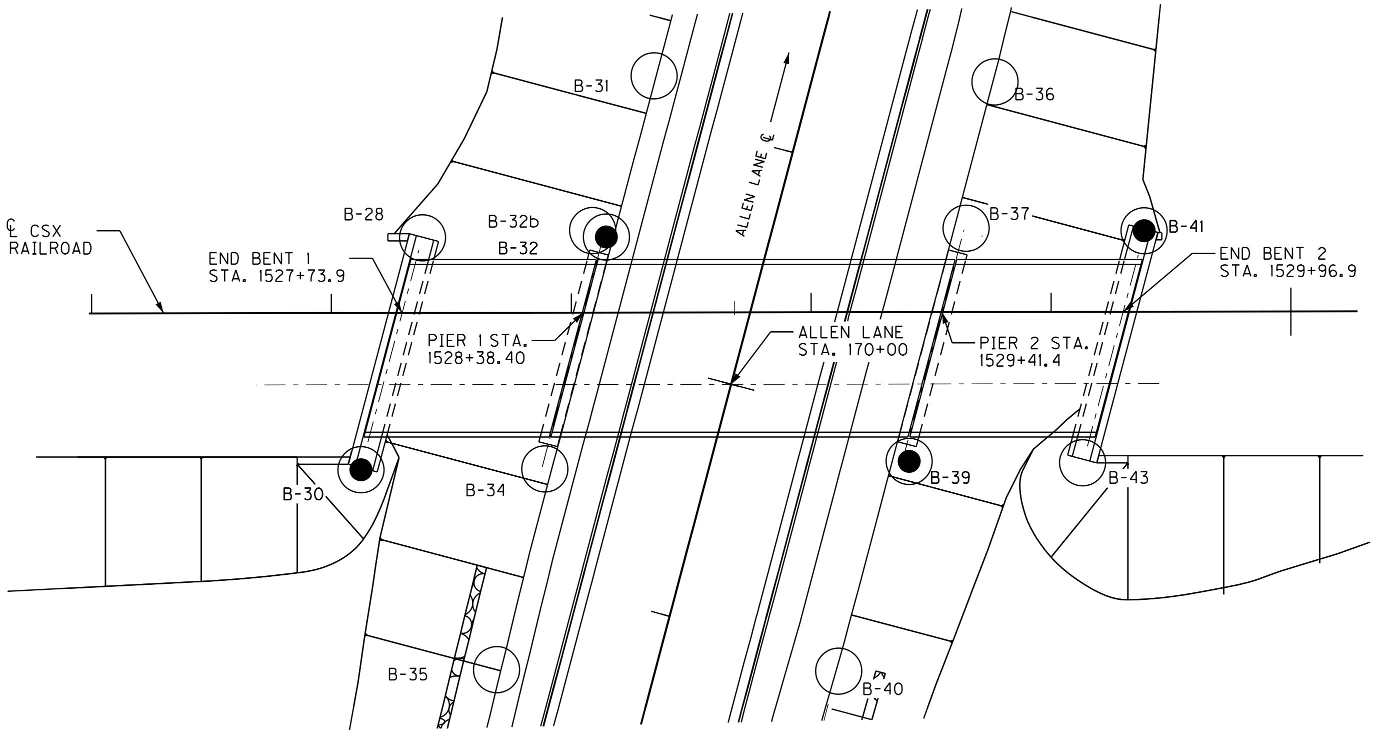
ITEM NUMBER
5-434.00

PREPARED BY
DLZ KENTUCKY, INC.
201 BRIGHTON PARK BLVD.
FRANKFORT, KENTUCKY 40602
(502) 695-2300

SHEET NO.
003
DRAWING NO.

SUBSURFACE DATA

Plan Scale: 1" = 40'



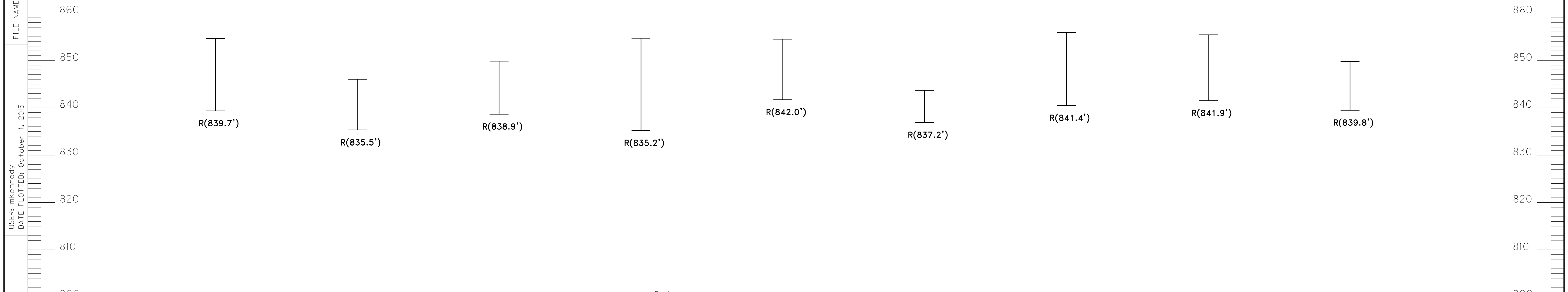
Profile Scale:
Vertical 1" = 20'
Horizontal not to scale

END BENT ONE — PIER ONE — PIER TWO — END BENT TWO

Approximate Allen Lane Roadway Elev. = 835.5'
Approximate CSX Railroad Elev. = 861.4'

Hole No. :Station :	B-28|170+32.37|	B-35|169+32.30|	B-34|169+73.00|	B-32b|170+43.00|	B-31|170+52.90|	B-40|169+45.30|	B-37|170+59.24|	B-36|170+69.41|	B-43|170+03.00|
Offset :Elev.	77.2' Lt.|854.65	18.1' Lt.|846.01	33.0' Lt.|849.86	41.7' Lt.|854.70	28.6' Lt.|854.51	49.1' Rt.|843.70	34.7' Rt.|855.88	39.9' Rt.|855.41	75.0' Rt.|849.78

(Sea level datum)



DATE: 01-OCTOBER-2015	CHECKED BY
DESIGNED BY: E. Tse	H.J.Hughes
DETAILED BY: M. KENNEDY	

OLDHAM COUNTY
FISCAL COURT

COUNTY
OLDHAM

ROUTE
—

CROSSING
ALLEN LANE UNDERPASS

SUBSURFACE DATA

PREPARED BY
DLZ
KENTUCKY, INC.
201 BRIGHTON PARK BLVD.
FRANKFORT, KENTUCKY 40602
(502) 695-2300

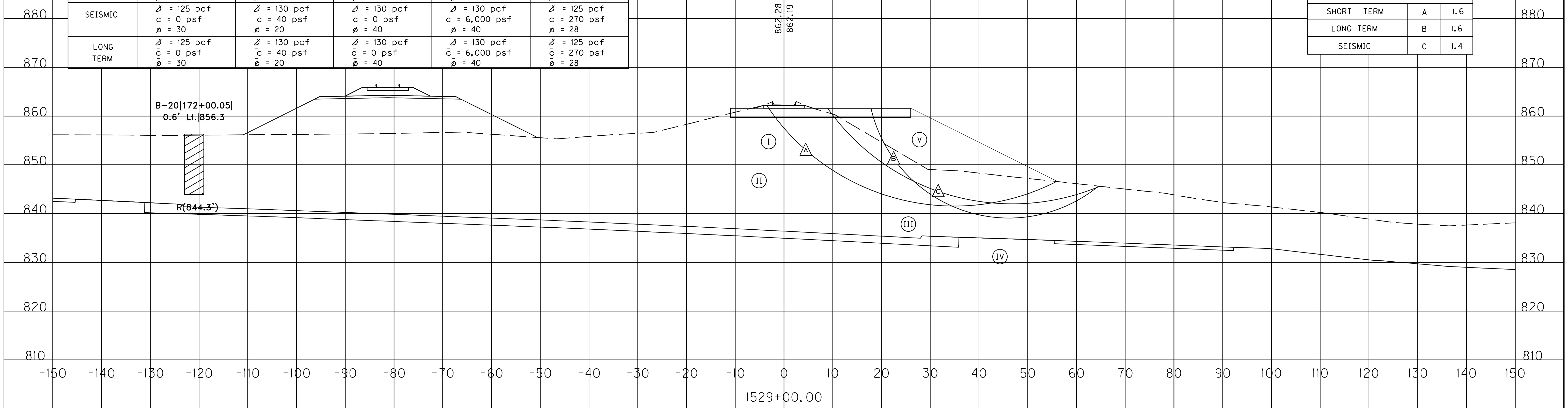
SHEET NO.
004
DRAWING NO.

ITEM NUMBER
5-434.00

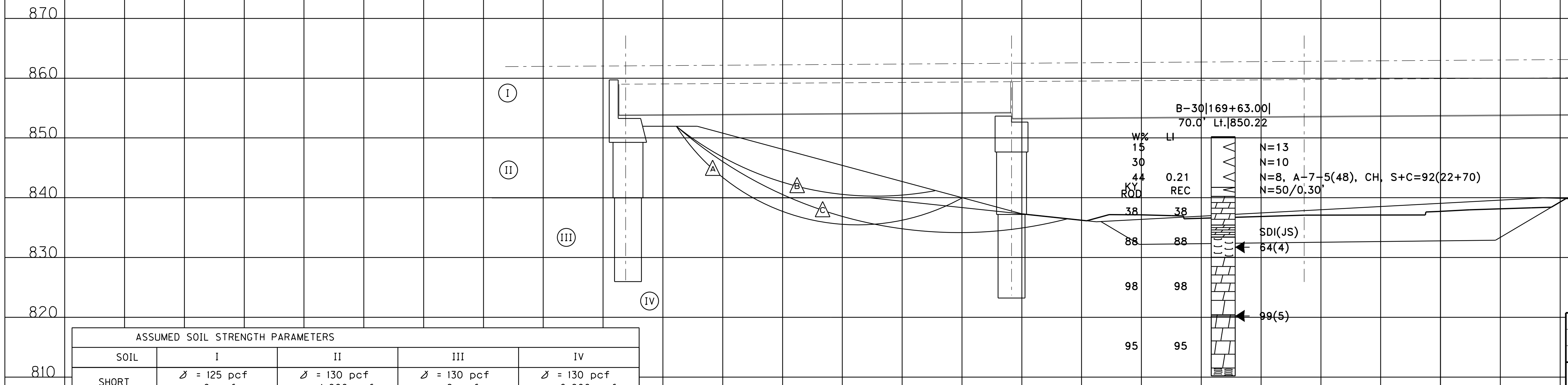
ASSUMED SOIL STRENGTH PARAMETERS					
SOIL	I	II	III	IV	V
SHORT TERM	γ = 125 pcf c = 0 psf ø = 30	γ = 130 pcf c = 1,000 psf ø = 0	γ = 130 pcf c = 0 psf ø = 40	γ = 130 pcf c = 6,000 psf ø = 40	γ = 125 pcf c = 1,500 psf ø = 0
SEISMIC	γ = 125 pcf c = 0 psf ø = 30	γ = 130 pcf c = 40 psf ø = 20	γ = 130 pcf c = 0 psf ø = 40	γ = 130 pcf c = 6,000 psf ø = 40	γ = 125 pcf c = 270 psf ø = 28
LONG TERM	γ = 125 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 30	γ = 130 pcf $\bar{c}$ = 40 psf $\bar{ø}$ = 20	γ = 130 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 40	γ = 130 pcf $\bar{c}$ = 6,000 psf $\bar{ø}$ = 40	γ = 125 pcf $\bar{c}$ = 270 psf $\bar{ø}$ = 28

Side Slope Embankment Fill Stability

FACTORS OF SAFETY		
SHORT TERM	A	1.6
LONG TERM	B	1.6
SEISMIC	C	1.4



Spill-Through Cut Slope Stability



Top of Rock Elev. = 841.7
Base of Weathered Rock Elev. = 840.2

ASSUMED SOIL STRENGTH PARAMETERS				
SOIL	I	II	III	IV
SHORT TERM	γ = 125 pcf c = 0 psf ø = 30	γ = 130 pcf c = 1,000 psf ø = 0	γ = 130 pcf c = 0 psf ø = 40	γ = 130 pcf c = 6,000 psf ø = 40
INTERMEDIATE TERM	γ = 125 pcf c = 0 psf ø = 30	γ = 130 pcf c = 200 psf ø = 20	γ = 130 pcf c = 0 psf ø = 40	γ = 130 pcf c = 6,000 psf ø = 40
LONG TERM	γ = 125 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 30	γ = 130 pcf $\bar{c}$ = 40 psf $\bar{ø}$ = 20	γ = 130 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 40	γ = 130 pcf $\bar{c}$ = 6,000 psf $\bar{ø}$ = 40

FACTORS OF SAFETY		
SHORT TERM	A	3.5
LONG TERM	B	1.4
INTERMEDIATE TERM	C	2.1

SCALE: 1" = 20 HORIZONTAL
1" = 20 VERTICAL

1527+50
1528+00
1528+50
1529+00

DATE: 01-OCTOBER-2015
CHECKED BY
DESIGNED BY: E. Tse
H. J. Hughes
DETAILED BY: M. KENNEDY

OLDHAM COUNTY
FISCAL COURT

COUNTY
OLDHAM

ROUTE
—
CROSSING
ALLEN LANE UNDERPASS

SUBSURFACE DATA

ITEM NUMBER
5–434.00

PREPARED BY
DLZ
KENTUCKY, INC.
201 BRIGHTON PARK BLVD.
FRANKFORT, KENTUCKY 40602
(502) 695-2300

SHEET NO.
005
DRAWING NO.

FILE NAME: M:\PROJ\0631\0006\02\CADD\SHEETS\STRUCTURE\SUBMITTAL SET\10-01-2015\S00000_005.DGN

USER: mkennedy
DATE PLOTTED: October 1, 2015

MicroStation v8.11.9.357