GEOTECHNICAL REPORT

IL-15 OVER RICHLAND CREEK
FAP 103 (IL-15), SECTION 27-1BR-1
ABUTMENT RETAINING WALLS
WORK ORDER NO. 61
ST. CLAIR COUNTY, ILLINOIS
PTB 168-023

FARNSWORTH GROUP
2709 McGraw Drive
Bloomington, Illinois 61704
309-663-8435

Prepared by:

TSi geotechnical, inc

1340 North Price Road
St. Louis, Missouri 63132
314-373-4000

Authored by:
William J. Graham, PE
bgraham@tsigeotech.com
TSi Project Number 20175035.00

LICENSED PROFESSIONAL ENGINEER
WILLIAM JOSEPH GRAHAM
062-057873
STATE OF ILLINOIS

11-30-17

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1.0 PROJECT DESCRIPTION

1.1 INTRODUCTION

This report summarizes the results of a geotechnical exploration performed for the design of remedial measures for the existing retaining walls at the abutments for IL-15 over Richland Creek near Belleville, St. Clair County, Illinois. It is our understanding that IDOT has decided to not pursue a further geotechnical study for the bridge structure movement. However, this study entails providing geotechnical information for design measures to stabilize the existing abutment retaining walls. This report describes the exploration procedures used, presents the field and laboratory data, includes an assessment of the subsurface conditions in the area, and provides geotechnical recommendations for stabilization of the walls.

1.2 PROJECT DESCRIPTION

The project involves two bridges supporting Illinois Route 15 over Richland Creek, at the southern limits of Belleville, Illinois. The bridges, constructed in 1956-1958, consist of 4-span structures oriented at a sharp skew, approximately 65 degrees to the flowline of channelized Richland Creek. The abutments are supported on steel H-piles, both battered and vertical, while the interior piers are on wood piles. It appears that both pile types were driven to bedrock, located approximately 50 feet below the surrounding natural ground surface.

During construction, movements were noted of the east interior piers in relation to the east abutments. At that time, the bridge substructures were in place and the eastbound bridge deck had been placed, but not the westbound deck. The east retaining wall between the two bridge abutments had been constructed but not the west wall. These movements were noted in October, 1956, but were believed to have ceased. However, further movements apparently occurred the next spring, and measurements of the movements were initiated in April 1957 and continued through June 24, 1957. A memorandum dated mid-May notes that slope movements had occurred, with ground cracks in the vicinity of the east abutment(s), along with movement of the east interior pier(s) and the retaining wall between the abutments. These movements were in relation to the abutment(s), which were indicated not to have moved. This lack of movement of the abutments is reasonable, since these elements are supported on piles battered toward the creek, providing substantial resistance to such movement. The east retaining wall is supported on a shallow footing rather than on piles. Movements of 2 to 4.5 inches in the vicinity of the eastbound eastern abutment and interior pier were noted in a memorandum and accompanying data sheets dated June 25, 1957.

At that time, the site grade in the vicinity of the abutments had been filled to a level only about 4 to 5 feet above the natural grade, according to the construction drawings. Additional fill on the order of 6 to 8 feet was needed to be placed behind the abutments and the retaining wall in order to reach the final roadway grade. According to a memorandum dated May 19, 1958, up to 10 feet of fill still remained to be placed behind the east and west abutments and retaining walls.

The steep original slopes down to the creek channel beneath and adjacent to the structures had been graded back to an inclination of 2 feet horizontal to 1 foot vertical (2H:1V), and the slopes had been paved with concrete slabs. By mid-June 1957, the slabs had pulled away from the east abutments as much as 2 inches. At this time, the grade behind the east wall and abutments was at approximately Elev. 468 to 470, so that up to 10 feet of fill still had to be placed.

As a result of the slope movements during construction, misalignment of the girders on the abutment and pier supports had occurred, and recommendations were made to restore these members to their design locations, but no record is available to indicate that this was done.

Subsequent to the mid-May 1957 memo, an independent assessment of the slope movements was authorized by IDOT. This led to the conclusion that the movements resulted from a buildup of hydrostatic pressures behind the abutments, and the recommendation was made to install a drainage trench behind each of the abutments that would drain by gravity to the creek. The trenches were installed behind the east abutments, extending down to Elev. 450. These were later backfilled with granular material and piezometers were installed to monitor the levels in the soil behind the trenches. In the May 19, 1958 memorandum, the recommendation was made to control the rate of fill placement by continually monitoring the piezometers for any significant rise that would indicate a rise in pore water pressures, coupled with monitoring of any structure movement. No documentation of the fill placement is available.

In 1979, a contract was let to adjust and repair the bearing system for the bridges. It is assumed that these repairs would have resulted in a realignment such that the girders were centered on the bearing system, with the rockers centered on the plates between anchor bolts.

An inspection performed by Farnsworth in April, 2017 indicates that further movement has taken place, such that rockers have been moved to bear against the anchor bolts and girders have moved laterally to the extent that curvature is noted, and visible vertical twisting can be seen. The misalignment suggests that movement of the Pier 3 structures downslope toward the creek, in relation to the other substructure elements, has resulted in rotation of the deck superstructure around the central piers (Pier 2).

A brief site reconnaissance was made in conjunction with this report. The top of the east retaining wall has appeared to have moved outward in relation to the adjacent abutments by 12 inches for the westbound and 14 inches for the eastbound abutments. The wall itself has a 4- to 5-degree tilt toward the creek. The movements of the west retaining wall are much less, about 0.25 inch and 2 inches for the westbound and eastbound abutments, respectively, with little or no tilt. The original slabs along the west bank of the creek have been demolished and replaced with rip-rap. We understand that a similar removal and replacement is scheduled for the slabs along the east bank. Construction drawings indicate the retaining walls are supported on footings bearing slightly greater than 4 feet below the slabs at the base of the walls, such that the maximum free-standing heights of the walls should be about 8.4 feet for the east wall and 7.3 feet for the west wall.

2.0 Subsurface Exploration and Laboratory Testing

2.1 Subsurface Exploration

On July 11 and 12, 2017, TSi conducted a subsurface exploration at the site, consisting of two soil test borings, designated as Borings TH-1 and TH-2. TH-1 was drilled near the westbound abutment retaining wall and TH-2 was drilled near the eastbound abutment retaining wall. The boring locations were selected by and staked in the field by TSi prior to drilling. The boring locations were not surveyed at the time of this report. The ground surface elevations at the boring locations were estimated by TSi based on existing plans provided by Farnsworth. The logs from this exploration are included in Appendix B. The approximate locations of the borings are shown on Figure 2 in Appendix A. The approximate ground surface elevation at each boring is stated on the appropriate Log of Boring in Appendix B. Some of the borings were offset due to access restrictions.

Both of the borings were drilled using a Diedrich D-50 track-mounted drill rig with an automatic hammer efficiency of 91.3 %. The borings were advanced using hollow-stem auger drilling tools to depths of 50 feet, then advanced below 50 feet using mud rotary drilling methods. A geotechnical specialist from TSi directed the exploration procedures in the field, maintained a field log of the conditions encountered in the borings, and collected and classified the samples recovered. Split-spoon samples were recovered from the borings using a 2-inch outside-diameter, split-barrel sampler, driven by an automatic hammer in accordance with ASTM D 1586. The split-spoon samples were placed in glass jars for later testing in the laboratory. Shelby tube samples were obtained in accordance with ASTM D 1587. The Shelby tube samples were preserved by sealing the entire sample in the tube. Borings were backfilled with grout upon completion.

The results of the geotechnical field tests and measurements were recorded on field logs and appropriate data sheets. Those data sheets and logs contain information concerning the exploration methods, samples attempted and recovered, indications of the presence of various subsurface materials, and the observation of groundwater. The field logs and data sheets contain the field engineer's interpretations of the conditions between samples, based on the performance of the exploration equipment and the cuttings brought to the surface by the drilling tools.

2.2 Laboratory Testing

A laboratory testing program was conducted by TSi to determine selected engineering properties of the obtained soil samples. The following laboratory tests were performed on the samples recovered from the borings:

- visual descriptions by color and texture of each sample (ASTM 2488);
- natural moisture content of each cohesive sample (ASTM D 2216);
- Atterberg limits on selected cohesive samples (ASTM D 4318);
- unit weight of selected samples (ASTM D 7263);
- grain-size analysis of selected samples (ASTM D 422); and
- unconsolidated-undrained triaxial compression tests (ASTM D 2850).

Upon completion of the testing program, the remaining samples were carefully examined to check for the presence of slickensides, disruptions, or other features that would indicate the presence of soil movement at the boring locations.

Data and observations from laboratory tests were recorded on laboratory data sheets during the course of the testing program. The results of the tests are summarized on the Logs of Boring in Appendix B and on the Laboratory Test Reports in Appendix C. The boring logs are an interpretation of the subsurface conditions based on the field and laboratory data. Only data pertinent to the objectives of this report have been included on the logs; therefore, these logs should not be used for other purposes.

3.0 Subsurface Conditions

Details of the subsurface conditions encountered at the borings are shown on the boring logs. The general subsurface conditions encountered and their pertinent engineering characteristics are described in the following paragraphs. Conditions represented by the borings should be considered applicable only at the boring locations on the dates shown; the reported conditions may be different at other locations and at other times. Following the completion of the laboratory testing, the remaining samples were split apart and examined for indications of slope movement, such as slickensides or disrupted zones. No such features were observed.

3.1 Geology

The site is located on the broad floodplain of Richland Creek, having a width of about 2,000 feet in this vicinity. It appears that the creek has been channelized at some time in the past, since there are meander scars and abandoned channels at intervals downstream of the site. The site is underlain by alluvial floodplain deposits extending to bedrock at depths on the order of 50 feet below the adjacent natural ground surface. These deposits are generally fine-grained, consisting of lean clays and silts, with basal deposits of sands near the underlying bedrock. The sands that occur at depth beneath the floodplain were likely deposited as glacial outwash, granular material carried from receding glaciers by fast-flowing meltwater. The fine-grained materials making up the bulk of the alluvium are assumed to be eroded from the extensive loess deposits that form the surface of the adjacent uplands. These windblown materials were swept up from the broad floodplain of the Mississippi River during the latter stages of the last major glaciation, and deposited across the surrounding till plains. Published county-wide maps show the loess deposits in the uplands surrounding Richland Creek to have thicknesses of 10 to 25 feet.

The surficial loess is underlain by a unit of till that forms the upland plains of the area. The till was laid down and overrun by the advancing glacial ice of Illinoisan age. The plains of the area are overlain at intervals by moraines, ridges of material marking the farthest advances of glacial lobes as the glacier eventually receded.

The underlying bedrock consists of Pennsylvanian-age sedimentary deposits of shale and limestone, classified as the upper portion of the Carbondale Formation. A nearby coal mine encountered the Herrin Coal at depths of 70 to 75 feet. This may indicate that the limestone encountered in the site borings is the overlying Brereton Limestone. The Herrin seam has been extensively mined in the Belleville area during the late 19th and early to mid-20th centuries.

3.2 GENERALIZED SUBSURFACE PROFILE

The generalized subsurface profile consists of cohesive and granular alluvium overlying relatively deep bedrock.

Existing fill was encountered at both borings at depths extending to approximately 10.5 and 13.5 feet below the ground surface in Borings TH-1 and -2, respectively. The fill consists of lean clay (CL, in accordance with the Unified Soil Classification System), with secondary materials such as coal fragments varying in content. The fill encountered was placed for the existing roadway alignment. Standard penetration test (N) values in the fill range from 7 to 14 blows per foot (bpf). Moisture contents in the fill vary from 14 to 24%.

Alluvial soils consisting of cohesive and granular soil deposits were encountered beneath the existing fill. The cohesive soils include lean clay and silt (CL and ML), and contain varying amounts of sand and wood fragments. N-values in the cohesive soils range from 1 to 10 blows per foot (bpf). Moisture contents vary from 19 to 37%, with one value of 64% noted at a sample in Boring TH-2 containing a substantial amount of wood fragments. Shelby tube samples taken in the soil yielded dry unit weight values of 84 to 109 pounds per cubic foot (pcf), with undrained shear strengths of 0.40 to 1.61 tons per square foot (tsf).

The granular alluvium encountered consisted of sands (SP and SC). Occasional layers of lean clay and silt were also encountered within the zone of granular deposits. The sands yielded N-values that range from 3 to 16 bpf.

Auger refusal on apparent limestone bedrock was encountered at both borings for this study, at depths of 57.0 and 71.1 feet below the ground surface in Borings TH-1 and -2, respectively. Approximately 6 inches to 2.5 feet of weathered limestone was encountered above intact bedrock at both of the boring locations.

3.3 GROUNDWATER

At the time of drilling, groundwater was observed at both of the borings, at depths of 25 and 28 feet. The presence or absence of groundwater at a particular location does not necessarily mean that groundwater will be present or absent at that location at other times. Groundwater levels may vary significantly over time due to the effect of seasonal variations in precipitation, the water level in Richland Creek, or other factors not evident at the time of exploration.

4.0 Geotechnical Evaluations

4.1 General Assessment of Wall Stability

As indicated on the present boring logs and in the discussion of subsurface conditions in Section 3, the soils in the upper 16 feet along the west retaining wall and the upper 24 feet along the east wall are stiff to hard in consistency, with no indication that any slope movements had extended beyond the abutments. Assuming that similar conditions are present in the foundation soils for the walls, the performance of the walls should have no adverse deflections. The original test borings for the structures encountered soft clays at the interior bent locations, extending down to Elevation 450 near the west abutment, to as deep as Elevation 415 to 425 in the vicinity of the east interior pier. In contrast, the easternmost of the original borings encountered soils described as stiff, extending down to Elevation 445, then underlain by a zone of "medium" silty clay down to Elevation 435, with stiff silty clay below that until a basal sand was encountered. This is generally similar to the conditions encountered in the current Boring TH-2 at the east abutment.

One possible mechanism for the movements observed during construction would be that the soils close to the present channel of Richland Creek may be softer than encountered in the two test borings drilled for this investigation, so that the use of a 2H:1V inclination for the permanent slopes down to the creek resulted in an unstable condition. This could be the result of the channelized alignment of the creek crossing the original channel or a filled meander channel containing softer soils. A second possibility would be the occurrence of sudden drawdown conditions following a substantial rise of flood waters in the creek. A third possibility would be the occurrence of mine subsidence resulting from the collapse or failure of an underground mine opening.

Because available records during the construction of the bridges noted that sudden drawdown conditions could have occurred, this condition was assessed initially. To test this possibility, stability analyses were performed for the bank slopes using strength parameters appropriate for the conditions indicated by the present test borings. The indicated factors of safety for end of construction and long term groundwater conditions were well above 1.0, indicating slope movements would not be expected. Analyses were then performed for sudden drawdown conditions, assuming that Richland Creek was at bank-full levels following a storm event, then rapidly fell to normal levels so that the adjacent banks remained saturated to the maximum level, setting up hydrostatic forces causing instability. When this condition was modeled, factors of safety less than 1.0 were obtained, which could have resulted in progressive slumping and failures that could work up to the crest of the slope. Given the skews of the two bridges, the locations of the retaining walls would be buttressed by the intermediate support piers within the slope. The resultant slope movements would likely have been deep-seated, passing near to or below the base of these interior piers in order to involve the walls.

The occurrence of sudden drawdown conditions could be capable of causing the slope movements that would have led to instability of the east retaining wall, even without the presence of relatively weak soil conditions beneath the interior portions of the two bridge structures. Rainfall records indicating four episodes of rain resulting in bank-full levels in Richland Creek between July 4, 1956 and April 15, 1957 lend credence to the possibility that sudden drawdown conditions resulted in the observed slope movements during construction.

The observed movements of the east retaining wall during construction, and as noted during the TSi reconnaissance, would indicate that this retaining wall has been involved in the slope movements instituted during construction, and that may be continuing as soil creep during the life of the structure. The condition and consistency of the soils encountered in the test borings for this investigation would not result in the observed movements of the retaining walls, particularly those of the east wall. Consequently, it is likely that the movements noted during construction, and subsequent to the period of monitoring during construction, may have disrupted the supporting soils beneath the walls to some extent.

Available records indicate that a drainage trench was installed behind the east abutments and retaining wall during construction of the bridges. No record is available regarding the exact location of this trench or where the trench drains to daylight on the slope to Richland Creek. The present effectiveness of the drain could not be verified, so its existence was ignored in this analysis. The recommended repair system of tieback anchors inclined at gentle slope angles is intended to provide additional horizontal support for the wall without imposing a significant amount of additional vertical loading on the retaining wall foundations.

4.2 Stability Analyses

Slope stability analyses were performed for the existing abutment slopes of the bridges down to Richland Creek utilizing the SLOPE/W 2007 program. The purpose of the analyses was to assess the influence of the existing slopes on the stability of the retaining walls, for various equilibrium conditions including end-of-construction, long-term, and sudden drawdown. If none of these conditions would result in a factor of safety of 1.0 or less using strength parameters based on the conditions in the test borings, then it could be assumed that the slopes are underlain by soils significantly weaker than those encountered in the borings, or that other factors may have caused the slope movements, such as mine subsidence.

The stability analyses for each wall were performed using strength parameters conservatively based on the conditions encountered in the adjacent test boring. The specific parameters are given in the output sheets given in Appendix C for each equilibrium condition analyzed. These sheets also show the location of the critical failure surfaces and the corresponding factors of safety. The results are summarized in Table 1.

TABLE 1.
CALCULATED CRITICAL FACTOR OF SAFETY

Location	Calculated Factor of Safety		
	End-of-Construction (Drained)	Long-Term (Drained)	Sudden Drawdown
East Retaining Wall	1.77	1.23	0.84
West Retaining Wall	1.24	1.17	0.93

The results indicate that a sudden drawdown condition could result in slope movements that could affect the wall, and that the instability would occur without assuming the presence of relatively weak soils beneath the slopes. Further analyses were made to model the situation where the specific retaining wall was in a condition of incipient failure, with a factor of safety of 1.0, in order to calculate a horizontal resisting force on the wall that would provide a factor of safety of 1.5. This approach did not give reasonable results, so conventional sliding and overturning analyses were performed as a check. These analyses were made assuming that no passive resistance was present at the toe of the wall, and that the soils supporting the wall footing were weakened as a result of the adjacent slope movements. For the overturning analysis, it was necessary to impose a height of water behind the wall of 7.2 feet above the base of the wall footing to provide a factor of safety less than 1.0. For this condition, the required moment acting at the toe of the wall that would provide a factor of safety of 1.5, was calculated to be approximately 10.2 kip-foot per foot of wall length.

Because of the potential that the integrity of the foundation soils have been affected by slope movements, we recommend that the stabilizing repair consist of tieback soil anchors that are installed horizontally or at a maximum inclination of 20 degrees, to minimize the horizontal component of the resisting force that would be imposed in addition to the vertical load on the existing wall foundation. The location and number of anchors required to provide the appropriate horizontal force to stabilize the wall is dependent on the structural capacity of the wall itself, and on the restricted headroom and available space for the installation equipment.

The installation of the anchor system would be performed by a specialty contractor, using proprietary equipment and procedures. On this basis, the recommended contract basis would be a performance specification to provide the required stabilizing moment, given specific requirements including the moment, a defined no-load zone, a minimum anchor depth, a maximum anchor inclination from the horizontal, and a maximum anchor spacing and anchor plate shear capacity based on the structural capacity of the wall. The stabilizing moment would be the calculated resisting moment of 10.2 kip-foot per foot of wall length, a no-load zone defined by a line inclined and extending upward from the heel of the footing base at 45 degrees, an allowable soil-grout bond strength of 450 pounds per square foot (psf), a maximum anchor inclination of 20 degrees from the horizontal, and a minimum anchor embedment of 5 feet beyond the no-load zone, into the intact stiff to hard soils behind the wall. If chosen, the installation of helical anchors through the no-load zone and into these clay soils may be difficult to accomplish without predrilling, depending on the torque developed by the installation equipment. Any system that is chosen should be considered to be permanent and therefore requires some form of corrosion protection.

The design of the stabilizing repair using shallow tieback anchors should provide a satisfactory solution for the walls, whether the slope soils are similar to those encountered in the borings or if weaker soils are present. However, if the slope movements are due to subsidence caused by the on-going failure of underground mine openings, the repair would likely not be effective. The potential for mining activity affecting the site is discussed in the following section.

4.3 Mining Activity

Based on information available through the Illinois State Geological Survey (ISGS), the nearest abandoned coal mine appears to be located very close to the project site. The mine is known as the Vulcan No. 1, which operated from about 1888 to 1908. The mine operations removed coal primarily from the Herrin Coal Seam, which is approximately 70 to 75 feet below the ground surface at the project site. In this area, the coal seam is about 7 feet thick.

It should be noted that the ISGS includes a disclaimer with the published information, which states that the location of features, including mine boundaries, may be offset by 500 feet or more. In addition, the plotted mine boundaries are not always based on a final mine map, and undocumented extents of mines are frequently discovered throughout the state.

As a result of this close proximity of the site to an underground mine, there is a possibility that the distress and movements noted at the bridge site could be related to mine subsidence. Mine subsidence is the surface manifestation of the collapse or failure of the structural support at the mine level. Subsidence may manifest itself as vertical movements ranging from a few inches to 2 or 3 feet, and as lateral or rotational ground movements, that can result in significant surface movement and structural damage. The Belleville area has seen frequent instances of mine subsidence, causing damage to transportation structures and roadways as well as numerous public, commercial, and residential developments.

The risk of subsidence is difficult to quantify without extensive studies. A study of the mine workings would require drilling several borings into the mine and viewing the mine openings with a borehole camera. Soil and rock samples could be taken at each borehole and the engineering properties of the materials could be measured. Geophysical techniques, such as seismic reflection or refraction techniques, could also be used to help define the mine limits. A study of this type is costly and is rarely performed.

5.0 Report Limitations

This geotechnical report has been prepared for the exclusive use of **Farnsworth Group** and the **Illinois Department of Transportation** for the specific application to the subject project. The information and recommendations contained in this report have been made in accordance with generally accepted geotechnical and foundation engineering practices; no other warranties are implied or expressed.

The assessments and recommendations submitted in this report are based in part upon the data obtained from the borings. The nature and extent of variations between the borings may not be evident at this time. If variations appear evident at a later date, it may be necessary to re-evaluate the recommendations of this report.

We emphasize that this report was prepared for design purposes only and may not be sufficient to prepare an accurate construction bid. Contractors reviewing this report should acknowledge that the information and recommendations contained herein are for design purposes.

If conditions at the site have changed due to natural causes or other operations, this report should be reviewed by TSi to determine the applicability of the analyses and recommendations considering the changed conditions. The report should also be reviewed by TSi if changes occur in the structure locations, sizes, and types, in the planned loads, elevations, grading and site development plans or the project concepts.

TSi requests the opportunity to review the final plans and specifications for the project prior to construction to verify that the recommendations in this report are properly interpreted and incorporated in the design and construction documents. If TSi is not accorded the opportunity to make this recommended review, we can assume no responsibility for the misinterpretation of our recommendations.

APPENDIX A

NOT TO SCALE

N

LEGEND

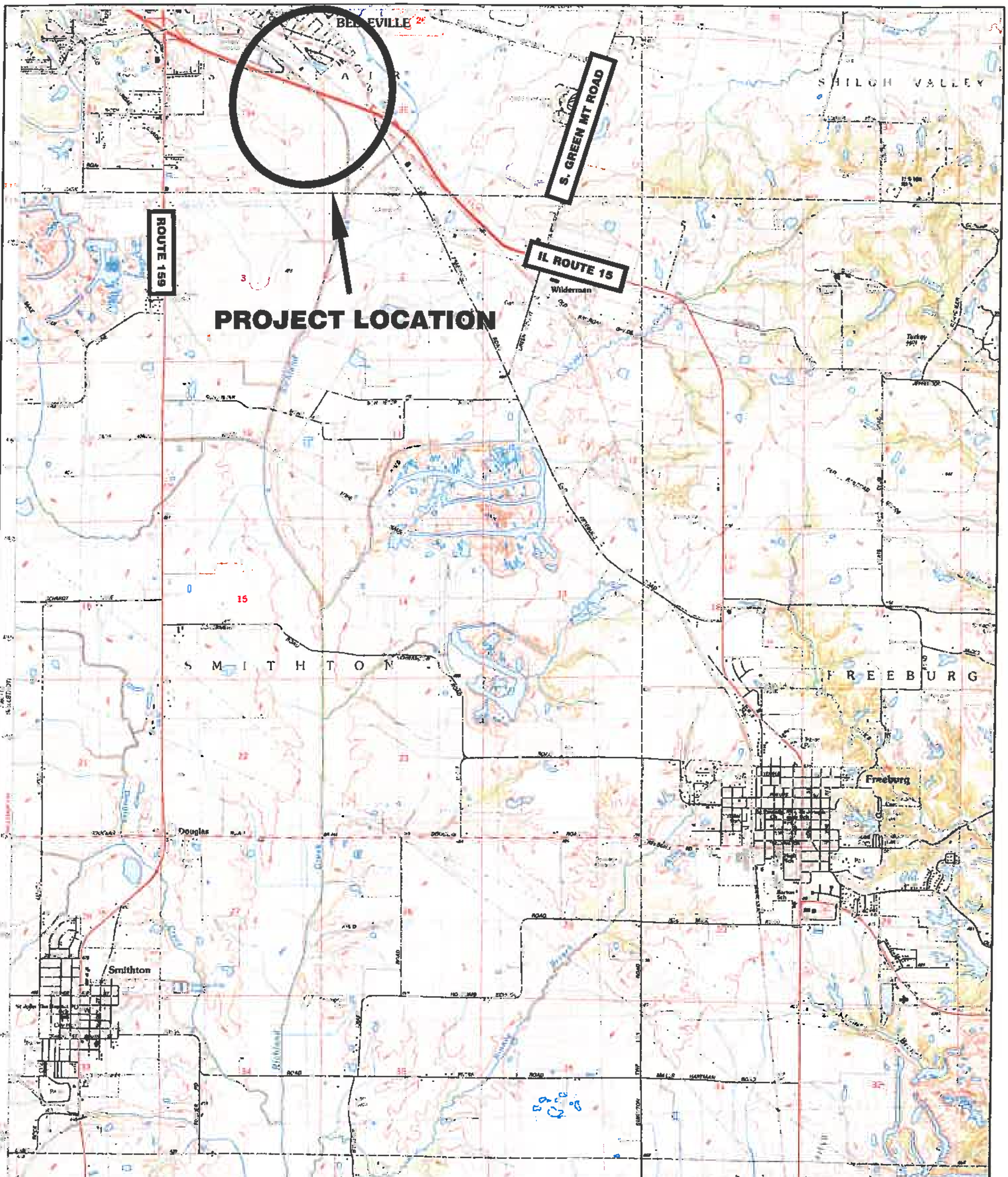
TH-1 APPROXIMATE BORING LOCATION AND NUMBER

NOTE: THIS PLAN WAS PREPARED FROM AN IMAGE OBTAINED FROM GOOGLE EARTH ON 8/9/2017.

TSi geotechnical, inc.
1340 N. PRICE ROAD
ST. LOUIS, MISSOURI 63132

SITE AND BORING LOCATION PLAN
IL ROUTE 15 OVER RICHLAND CREEK
RETAINING WALLS
ST. CLAIR COUNTY, ILLINOIS

Drawn By: ACE	Checked By: JAS	
Project No.: 20175035.00	Date: 8/9/17	Figure 2



NOT TO SCALE

N

NOTE:
DRAWING PREPARED FROM AN IMAGE OBTAINED FROM USGS ON 8/9/2017

TSi
geotechnical, Inc.
1340 NORTH PRICE ROAD
ST. LOUIS, MISSOURI 63132

VICINITY MAP IL ROUTE 15 OVER RICHLAND CREEK RETAINING WALLS ST. CLAIR COUNTY, ILLINOIS		
Drawn By: ACE	Checked By: JAS	
Project No.: 20175035.00	Date: 8/9/17	Figure 1

APPENDIX B

LOG OF BORING NO. TH-1

Project Description: **IL Route 15 over Richland Creek - Retaining Walls**
St. Clair County

TSi Geotechnical Inc.
1340 North Price Road
St. Louis, Missouri 63132
(314) 373-4000 (314) 227-6622 FAX

Surface El.: **Approx. 474**
Location: **See Site and Boring Location Plan**

Depth, feet	Samples	Sample #	Graphic Log	MATERIAL DESCRIPTION	Recovery %	RQD	Penetration Blows Per 6 inches	Hand Penetrometer, Qu TSF	Undrained Shear Strength, TSF	Unit Dry Weight, lb/cu ft.	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
				Brown and gray, lean CLAY (CL) (FILL)										
		SS-1			67		4 6 8	>4.5			14			
		ST-2			83			>4.5		107	19			
5														
		SS-3		- gray below 6.0 ft. - trace coal fragments below 7.0 ft. - more silty, trace fine roots below 8.0 ft.	78		5 6 7	3.50			21			
		ST-4			33			3.00		103	22			
10														
		SS-5		Gray, clayey SILT (ML), trace wood, coal fragments, sand and gravel	78		1 5 5	2.50			24			
				(86% Passing No. 200 Sieve)										
		ST-6			100			4.00	1.21	105	21	36	28	8
15														
		SS-7			100		2 2 2	0.50			27			
		ST-8		Gray, lean CLAY (CL), trace wood fragments, more silty	100			1.00	0.40	84	36			
				(99% Passing No. 200 Sieve)										
20														
		SS-9			94		1 2 2	0.50			37			
		ST-10			100			1.25	0.58	93	26			
				Greenish gray, clayey SILT (ML)										
25														

Completion Depth: 57.0
Date Boring Started: 7/12/17
Date Boring Completed: 7/12/17
Engineer/Geologist: FHH
Project No.: 20175035.00

Remarks: Boring drilled with D-50 using HSA and auto SPT. Groundwater encountered at 28.0 ft. during drilling. Began mud rotary at 50.0 ft. Hard drilling below 56.5 ft. Auger refusal at 57.0 ft.

The stratification lines represent approximate strata boundaries. In situations, the transition may be gradual.

Continued Next Page

LOG OF BORING NO. TH-1

Project Description: **IL Route 15 over Richland Creek - Retaining Walls**
St. Clair County

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St. Louis, Missouri 63132
(314) 373-4000 (314) 227-6622 FAX

Surface El.: **Approx. 474**
Location: **See Site and Boring Location Plan**

Depth, feet	Sample #	MATERIAL DESCRIPTION	Recovery %	RQD	Penetration Blows Per 6 inches	Hand Penetrometer, Qu TSF	Undrained Shear Strength, TSF	Unit Dry Weight, lb/cu ft.	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
		Greenish gray, clayey SILT (ML)*(continued)* -unterlayered with lean clay and lenses, and layers of silty and clayey fine sand - 1.0" sand seams at 26.5 and 27.8 ft.										
	SS-11		100		1 WH 1	<0.25			25			
	SS-12		100		1 2 3	0.50			25			
30												
		Greenish gray to dark gray, lean CLAY (CL), trace shell fragments										
	SS-13		100		1 2 2	0.75			32			
35												
		Gray, clayey SILT (ML) - trace organics from 38.5 to 40.0 ft. - trace organic odor below 38.5 ft. - brownish gray below 43.5 ft.										
	SS-14		100		2 4 4	2.00			32			
40												
	SS-15		100		2 4 6	2.25			27			
45												
		Gray, fine to medium SAND (SP)										
	SS-16		100		WR WR 3				19			
50												

Completion Depth:	57.0
Date Boring Started:	7/12/17
Date Boring Completed:	7/12/17
Engineer/Geologist:	FHH
Project No.:	20175035.00

Remarks: Boring drilled with D-50 using HSA and auto SPT. Groundwater encountered at 28.0 ft. during drilling. Began mud rotary at 50.0 ft. Hard drilling below 56.5 ft. Auger refusal at 57.0 ft.

The stratification lines represent approximate strata boundaries. In situations, the transition may be gradual.

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Surface El.: **Approx. 474**
Location: **See Site and Boring Location Plan**

Depth, feet	Samples	Sample #	Graphic Log	MATERIAL DESCRIPTION	Recovery %	RQD	Penetration Blows Per 6 inches	Hand Penetrometer, Qu TSF	Undrained Shear Strength, TSF	Unit Dry Weight, lb/cu ft.	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
				Gray, fine to medium SAND (SP)*(continued)*										
		SS-17		- fine to coarse, trace gravel, below 53.5 ft.	78		10 20 36				12			
55				Gray, SILT (ML)										
				Gray, weathered LIMESTONE										
				Boring terminated at 57.0 ft.										
60														
65														
70														
75														

Completion Depth:	57.0
Date Boring Started:	7/12/17
Date Boring Completed:	7/12/17
Engineer/Geologist:	FHH
Project No.:	20175035.00

Remarks: Boring drilled with D-50 using HSA and auto SPT. Groundwater encountered at 28.0 ft. during drilling. Began mud rotary at 50.0 ft. Hard drilling below 56.5 ft. Auger refusal at 57.0 ft.

The stratification lines represent approximate strata boundaries. In situations, the transition may be gradual.

LOG OF BORING NO. TH-2

Project Description: **IL Route 15 over Richland Creek - Retaining Walls**
St. Clair County

TSi Geotechnical Inc.
1340 North Price Road
St. Louis, Missouri 63132
(314) 373-4000 (314) 227-6622 FAX

Surface El.: **Approx. 478**
Location: **See Site and Boring Location Plan**

Depth, feet	Sample #	MATERIAL DESCRIPTION	Recovery %	RQD	Penetration Blows Per 6 inches	Hand Penetrometer, Qu TSF	Undrained Shear Strength, TSF	Unit Dry Weight, lb/cu ft.	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
		Brown and gray, lean CLAY (CL) (FILL)										
	SS-1		83		4 6 6	>4.5			15			
	SS-2	- more silty below 3.5 ft.	44		3 6 8	>4.5			16			
5												
	SS-3	- gray below 6.3 ft.	67		4 5 7	>4.5			19			
	ST-4		85			>4.5		104	21			
10												
	SS-5	- gray and greenish gray, trace coal fragments below 11.0 ft.	83		2 3 4	1.25			24			
	ST-6	Gray and greenish gray, lean CLAY (CL)	100			>4.5	1.61	109	19			
15		(97% Passing No. 200 Sieve)										
	SS-7		72		4 4 5	2.75			22			
	ST-8		88			2.00	1.03	97	24			
20												
	SS-9	- trace roots below 21.0 ft.	100		3 4 5	2.00			23			
	ST-10		100			1.25	0.65	100	24	41	20	21
25		Brown and gray, lean CLAY (CL)										

Completion Depth: 71.1
Date Boring Started: 7/11/17
Date Boring Completed: 7/12/17
Engineer/Geologist: FHH
Project No.: 20175035.00

Remarks: Boring drilled with D-50 using HSA and auto SPT. Perched groundwater at 25.0 ft. Groundwater encountered at 43.0 ft. during drilling. Began mud rotary at 50.0 ft. Rough drilling from 62.0 to 68.5 ft. Hard drilling below 70.5 ft. Auger refusal at 71.1 ft.

The stratification lines represent approximate strata boundaries. In situations, the transition may be gradual.

Continued Next Page

LOG OF BORING NO. TH-2

Project Description: **IL Route 15 over Richland Creek - Retaining Walls**
St. Clair County

TSi Geotechnical Inc.
1340 North Price Road
St. Louis, Missouri 63132
(314) 373-4000 (314) 227-6622 FAX

Surface El.: **Approx. 478**
Location: **See Site and Boring Location Plan**

Depth, feet	Samples	Sample #	Graphic Log	MATERIAL DESCRIPTION	Recovery %	RQD	Penetration Blows Per 6 inches	Hand Penetrometer, Qu TSF	Undrained Shear Strength, TSF	Unit Dry Weight, lb/cu ft.	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
				Brown and gray, lean CLAY (CL)*(continued)*										
		SS-11		- trace coal fragments and roots, more silty, from 26.0 to 30.0 ft.	100		2 2 3	1.00			27			
30		SS-12			100		2 2 3	1.25			27			
35		SS-13			100		1 2 2	0.50			30			
40		SS-14		- greenish gray, trace shell fragments, more silty, below 38.5 ft.	100		1 2 2	1.00			29			
				Greenish gray, clayey SILT (ML), trace shell fragments										
45		SS-15			100		WH 1 2	0.50			27			
				- unterlayered with lean clay, with wood pieces to 2 in., below 47 ft.										
50		SS-16			100		1 2 1	0.50			64			

Completion Depth:	71.1
Date Boring Started:	7/11/17
Date Boring Completed:	7/12/17
Engineer/Geologist:	FHH
Project No.:	20175035.00

Remarks: Boring drilled with D-50 using HSA and auto SPT. Perched groundwater at 25.0 ft. Groundwater encountered at 43.0 ft. during drilling. Began mud rotary at 50.0 ft. Rough drilling from 62.0 to 68.5 ft. Hard drilling below 70.5 ft. Auger refusal at 71.1 ft.

The stratification lines represent approximate strata boundaries. In situations, the transition may be gradual.

Continued Next Page

LOG OF BORING NO. TH-2

Project Description: **IL Route 15 over Richland Creek - Retaining Walls**
St. Clair County

TSi Geotechnical Inc.
1340 North Price Road
St. Louis, Missouri 63132
(314) 373-4000 (314) 227-6622 FAX

Surface El.: **Approx. 478**
Location: **See Site and Boring Location Plan**

Depth, feet	Samples	Sample #	Graphic Log	MATERIAL DESCRIPTION	Recovery %	RQD	Penetration Blows Per 6 inches	Hand Penetrometer, Qu TSF	Undrained Shear Strength, TSF	Unit Dry Weight, lb/cu ft.	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
				Greenish gray, clayey SILT (ML), trace shell fragments *(continued)*										
		SS-17		Gray, lean CLAY (CL)	78		9 11 13	>4.5			24			
55				Gray, fine to coarse clayey SAND (SC)										
60		SS-18		Gray, SILT (ML), trace shell fragments	94		2 3 4	1.00			31	37	30	7
65		SS-19		Gray, coarse SAND (SP), trace gravel	6		5 10 6							
70		SS-20		Gray, clayey SILT (ML)	72		9 4 3	0.25			20			
		SS-21		Gray, weathered LIMESTONE	17		50/1.5"							
				Boring terminated at 71.1 ft.										
75														

Completion Depth:	71.1
Date Boring Started:	7/11/17
Date Boring Completed:	7/12/17
Engineer/Geologist:	FHH
Project No.:	20175035.00

Remarks: Boring drilled with D-50 using HSA and auto SPT. Perched groundwater at 25.0 ft. Groundwater encountered at 43.0 ft. during drilling. Began mud rotary at 50.0 ft. Rough drilling from 62.0 to 68.5 ft. Hard drilling below 70.5 ft. Auger refusal at 71.1 ft.

The stratification lines represent approximate strata boundaries. In situations, the transition may be gradual.

General Notes

The number of borings is based on: topographic and geologic factors; the magnitude of structure loading; the size, shape, and value of the structure; consequences of failure; and other factors. The type and sequence of sampling are selected to reduce the possibility of undiscovered anomalies and maintain drilling efficiency. Attempts are made to detect and/or identify occurrences during drilling and sampling such as the presence of water, boulders, gas, zones of lost circulation, relative ease or resistance to drilling progress, unusual sample recovery, variation in resistance to driving split-spoon samplers, unusual odors, etc. However, lack of notation regarding these occurrences does not preclude their presence.

Although attempts are made to obtain stabilized groundwater levels, the levels shown on the Logs of Boring may not have stabilized, particularly in more impermeable cohesive soils. Consequently, the indicated groundwater levels may not represent present or future levels. Groundwater levels may vary significantly over time due to the effects of precipitation, infiltration, or other factors not evident at the time indicated.

Unless otherwise noted, soil classifications indicated on the Logs of Boring are based on visual observations and are not the result of classification tests. Although visual classifications are performed by experienced technicians or engineers, classifications so made may not be conclusive.

Generally, variations in texture less than one foot in thickness are described as layers within a stratum, while thicker zones are logged as individual strata. However, minor anomalies and changes of questionable lateral extent may appear only in the verbal description. The lines indicating changes in strata on the Logs of Boring are approximate boundaries only, as the actual material change may be between samples or may be a gradual transition.

Samples chosen for laboratory testing are selected in such a manner as to measure selected physical characteristics of each material encountered. However, as samples are recovered only intermittently and not all samples undergo a complete series of tests, the results of such tests may not conclusively represent the characteristics of all subsurface materials present.

NOTATION USED ON BORING LOGS

APPROXIMATE PROPORTIONS

TRACE	<15%
WITH	15-30%
MODIFIER	>30%

Clay or clayey may be used as major material or modifier, regardless of relative proportions, if the clay content is sufficient to dominate the soil properties.

PARTICLE SIZE

BOULDERS		>12 Inches
COBBLES		12 Inches – 3 Inches
GRAVEL		
	Coarse	3 Inches – ¾ Inch
	Fine	¾ Inch – No. 4 Sieve (4.750 mm)
SAND		
	Coarse	No. 4 – No. 10 Sieve (2.000 mm)
	Medium	No. 10 – No. 40 Sieve (0.420 mm)
	Fine	No. 40 – No. 200 Sieve (0.074 mm)
SILT		No. 200 Sieve - 0.002 mm
CLAY		< 0.002 mm

PENETRATION – BLOWS

Number of impacts of a 140-pound hammer falling a distance of 30 inches to cause a standard split-barrel sampler, 1 3/8 inches I.D., to penetrate a distance of 6 inches. The number of impacts for the first 6 inches of penetration is known as the seating drive. The sum of the impacts for the last 12 inches of penetration is the Standard Penetration Test Resistance or "N" value, blows per foot. For example, if blows = 6-8-9, "N" = 8+9 or 17.

OTHER NOTATIONS

Recovery % – length of recovered soil divided by length of sample attempted.

50/2"	Impacts of hammer to cause sampler to penetrate the indicated number of inches
WR	Sampler penetrated under the static loading of the weight of the drill rods
WH	Sampler penetrated under the static loading the weight of the hammer and drill rods
HSA	Hollow stem auger drilling method
FA	Flight auger drilling method
RW	Rotary wash drilling methods with drilling mud
AH	Automatic hammer used for Standard Penetration Test sample
SH	Safety hammer with rope and cathead used for Standard Penetration Test sample

GRAPHIC SYMBOLS

∇ Depth at which groundwater was encountered during drilling

▼ Depth at which groundwater was measured after drilling

Standard Penetration Test Sample, ASTM D1586

3-inch diameter Shelby Tube Sample, ASTM D1587

G Sample grabbed from auger

NX Size rock core sample

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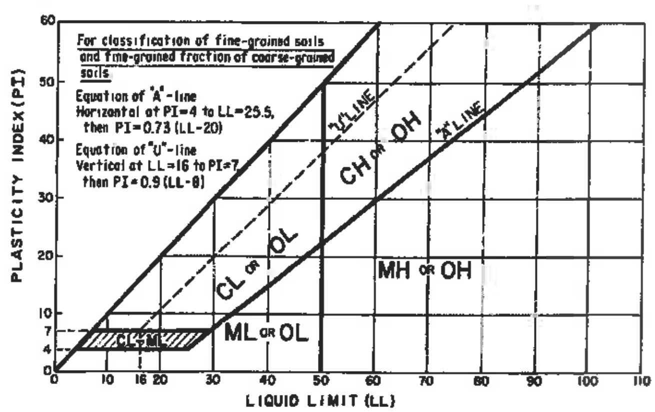
UNIFIED SOIL CLASSIFICATION SYSTEM, (ASTM D-2487)

<table>
<tr><th colspan="3">Major Divisions</th><th colspan="2">Group Symbols</th><th>Typical Names</th><th colspan="3">Laboratory Classification Criteria</th></tr>
<tr><td rowspan="10">Coarse-grained soils (More than half of materials is larger than No. 200 sieve size)</td><td rowspan="5">Gravels (More than half of coarse fraction is larger than No. 4 sieve size)</td><td rowspan="2">Clean gravels (Little or no fines)</td><td colspan="2">GW</td><td>Well-graded gravels, gravel-sand mixtures, little or no fines</td><td rowspan="10">Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:
Less than 5 per cent — GW, GP, SW, SP
More than 12 per cent — GM, GC, SM, SC
5 to 12 per cent — Borderline cases requiring dual symbols[b]</td><td colspan="2">$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3</td></tr>
<tr><td colspan="2">GP</td><td>Poorly graded gravels, gravel-sand mixtures, little or no fines</td><td colspan="2">Not meeting all gradation requirements for GW</td></tr>
<tr><td rowspan="3">Gravels with fines (Appreciable amount of fines)</td><td rowspan="2">GM[a]</td><td>d</td><td rowspan="2">Silty gravels, gravel-sand-silt mixtures</td><td rowspan="2">Atterberg limits below "A" line or P.I. less than 4</td><td rowspan="3">Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols</td></tr>
<tr><td>u</td></tr>
<tr><td colspan="2">GC</td><td>Clayey gravels, gravel-sand-clay mixtures</td><td>Atterberg limits below "A" line with P.I. greater than 7</td></tr>
<tr><td rowspan="5">Sands (More than half of coarse fraction is smaller than No. 4 sieve size)</td><td rowspan="2">Clean sands (Little or no fines)</td><td colspan="2">SW</td><td>Well-graded sands, gravelly sands, little or no fines</td><td colspan="2">$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3</td></tr>
<tr><td colspan="2">SP</td><td>Poorly graded sands, gravelly sands, little or no fines</td><td colspan="2">Not meeting all gradation requirements for SW</td></tr>
<tr><td rowspan="3">Sands with fines (Appreciable amount of fines)</td><td rowspan="2">SM[a]</td><td>d</td><td rowspan="2">Silty sands, sand-mix mixtures</td><td rowspan="2">Atterberg limits about "A" line or P.I. less than 4</td><td rowspan="3">Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols</td></tr>
<tr><td>u</td></tr>
<tr><td colspan="2">SC</td><td>Clayey sands, sand-clay mixtures</td><td>Atterberg limits about "A" line with P.I. greater than 7</td></tr>
<tr><td rowspan="7">Fine-grained soils (More than half of materials is smaller than No. 200 sieve size)</td><td colspan="2" rowspan="3">Silts and clays (Liquid limit less than 50)</td><td colspan="2">ML</td><td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity</td><td colspan="3" rowspan="7">
</td></tr>
<tr><td colspan="2">CL</td><td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td></tr>
<tr><td colspan="2">OL</td><td>Organic silts and organic silty clays of low plasticity</td></tr>
<tr><td colspan="2" rowspan="3">Silts and clays (Liquid limit greater than 50)</td><td colspan="2">MH</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td></tr>
<tr><td colspan="2">CH</td><td>Inorganic clays of medium to high plasticity, organic silts</td></tr>
<tr><td colspan="2">OH</td><td>Organic clays of medium to high plasticity, organic silts</td></tr>
<tr><td colspan="2">Highly organic soils</td><td colspan="2">Pt</td><td>Peat and other highly organic soils</td></tr>
</table>

[a]Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 26 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

[b]Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

T:\Geotechnical Group\Notes for Geotech Reports\Unified Soil Classifications System2.doc

APPENDIX C

Unconsolidated-Undrained Triaxial Compression Test - Q-test

Project Name: IL 15 over Richland Creek — Tested By: AdJ 7-19-17

Project No: 20175035.00 — Checked By:

Boring No. TH-1 — Sample No. ST-6-3 — Depth 14.0-14.5

Soil Description Brown, clayey SILT (ML)

Liquid Limit		%
Plastic Limit		%
Plasticity Index		%
USCS		
Specific Gravity	2.70	*

*assumed

Water Content Data:

Wet & Tare	56.93	gm
Dry & Tare	47.34	gm
Tare	2.64	gm
Water Content	21.5	%

Moisture content sample taken from: Trimmings

Specimen Data:

Height	5.570	in
Diameter	2.820	in
Hgt/Dia ratio	1.98	
Volume	570.09	cc
Wet Weight	1170.55	gm
Wet Density	128.2	pcf
Dry Density	105.5	pcf
Water Content	21.5	%
Saturation	97	%
Void Ratio	0.60	

Instrument Constants

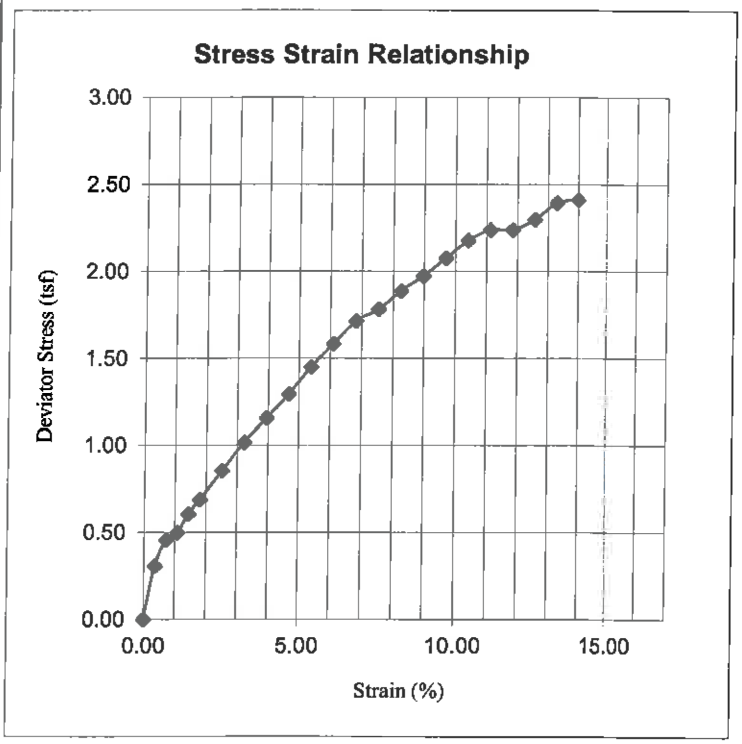
Deformation	0.0001	in/div
Load	1.9	lbs/div
Strain Rate	0.039	in/min
	0.70	%/min

Failure Sketch

Confining Pressure (psi) 11.5

Undrained Shear Strength (tsf) 1.21

Axial Strain (%) 14.0



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Unconsolidated-Undrained Triaxial Compression Test - Q-test

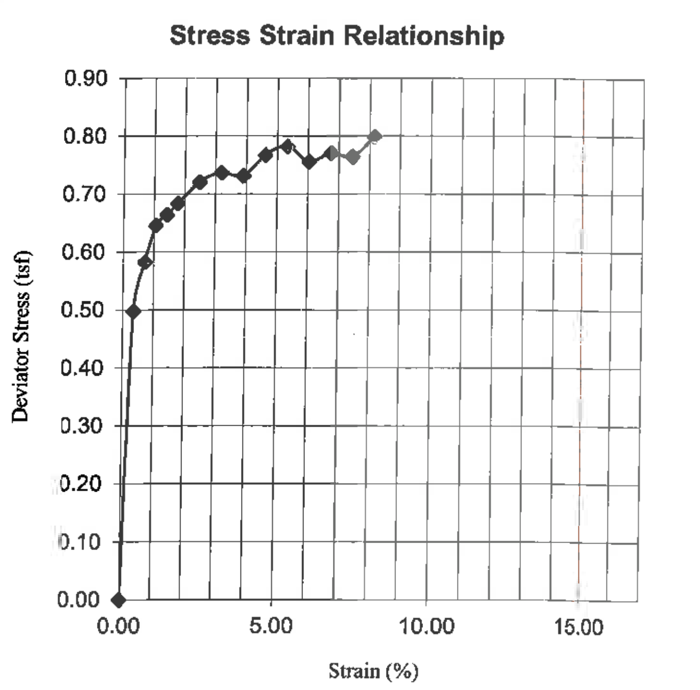
Project Name	IL 15 over Richland Creek			Tested By	AdJ 7-19-17
Project No.	20175035.00			Checked By	
Boring No.	TH-1	Sample No.	ST-8-3	Depth	19.0-19.5
Soil Description	Gray, lean CLAY (CL), trace wood fragments				

Liquid Limit		%	Specimen Data:			Instrument Constants		
Plastic Limit		%	Height	5.640	in	Deformation	0.0001	in/div
Plasticity Index		%	Diameter	2.830	in	Load	1.9	lbs/div
USCS			Hgt/Dia ratio	1.99		Strain Rate	0.039	in/min
Specific Gravity	2.70	*	Volume	581.36	cc		0.69	%/min
*assumed			Wet Weight	1061	gm			
Water Content Data:			Wet Density	113.9	pcf	Failure Sketch		
Wet & Tare	38.42	gm	Dry Density	83.8	pcf			
Dry & Tare	28.97	gm	Water Content	35.9	%			
Tare	2.64	gm	Saturation	96	%			
Water Content	35.9	%	Void Ratio	1.01				

Moisture content sample taken from: Trimmings

Undrained Shear Strength (tsf)	0.40
Axial Strain (%)	8.2

Confining Pressure (psi) 15.5



Unconsolidated-Undrained Triaxial Compression Test - Q-test

Project Name	IL 15 over Richland Creek	Tested By	AdJ 7-19-17
Project No.	20175035.00	Checked By	
Boring No.	TH-1	Sample No. ST-10-3	Depth 24.5-25.0

Soil Description: Gray, lean CLAY, trace wood fragments

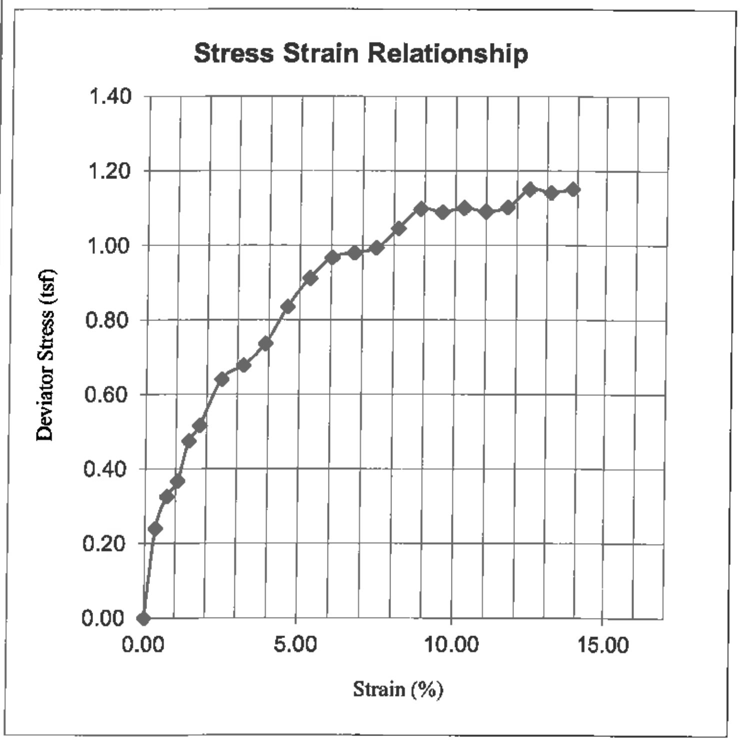
Liquid Limit		%	Specimen Data:			Instrument Constants		
Plastic Limit		%	Height	5.640	in	Deformation	0.0001	in/div
Plasticity Index		%	Diameter	2.820	in	Load	1.9	lbs/div
USCS			Hgt/Dia ratio	2.00		Strain Rate	0.039	in/min
Specific Gravity	2.70	*	Volume	577.26	cc		0.69	%/min
*assumed			Wet Weight	1081.42	gm			
Water Content Data:			Wet Density	116.9	pcf	Failure Sketch		
Wet & Tare	42.32	gm	Dry Density	92.5	pcf			
Dry & Tare	34.03	gm	Water Content	26.4	%			
Tare	2.59	gm	Saturation	87	%			
Water Content	26.4	%	Void Ratio	0.82				

Moisture content sample taken from: Trimmings

Undrained Shear Strength (tsf) 0.58

Confining Pressure (psi) 20

Axial Strain (%) 13.8



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Unconsolidated-Undrained Triaxial Compression Test - Q-test

Project Name	IL 15 over Richland Creek			Tested By	AdJ 7-19-17
Project No.	20175035.00			Checked By	
Boring No.	TH-2	Sample No.	ST-6-2	Depth	13.5-14.0
Soil Description	Gray, lean CLAY (CL)				

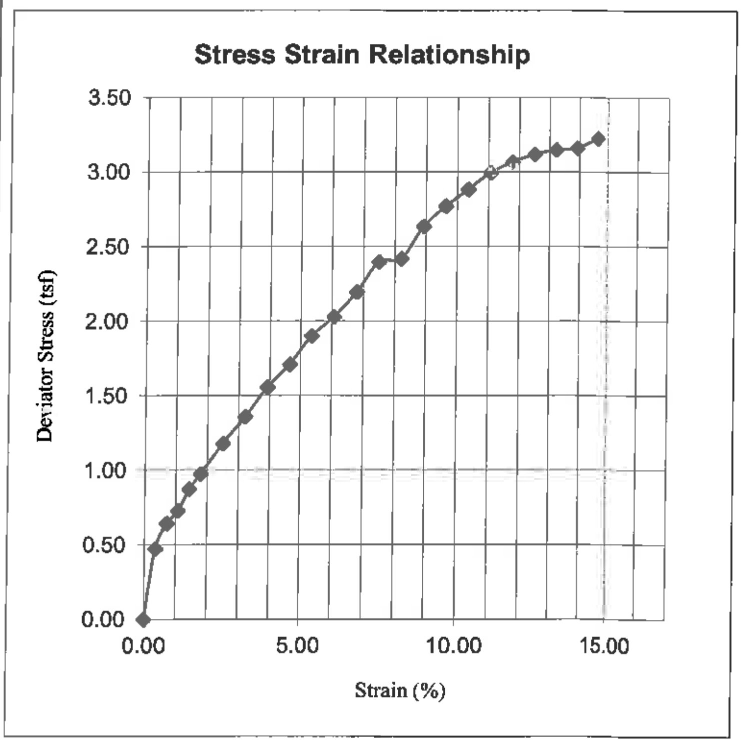
Liquid Limit		%	Specimen Data:				Instrument Constants		
Plastic Limit		%	Height	5.600	in		Deformation	0.0001	in/div
Plasticity Index		%	Diameter	2.840	in		Load	1.9	lbs/div
USCS			Hgt/Dia ratio	1.97			Strain Rate	0.039	in/min
Specific Gravity	2.70	*	Volume	581.32	cc			0.70	%/min
*assumed			Wet Weight	1203.37	gm				
Water Content Data:			Wet Density	129.2	pcf		Failure Sketch		
Wet & Tare	58.96	gm	Dry Density	109.1	pcf				
Dry & Tare	50.17	gm	Water Content	18.5	%				
Tare	2.66	gm	Saturation	92	%				
Water Content	18.5	%	Void Ratio	0.55					

Moisture content sample taken from: Trimmings

Undrained Shear Strength (tsf) 1.61

Confining Pressure (psi) 11.5

Axial Strain (%) 14.6



rev. 1.12.2000

TSi Geotechnical, Inc.

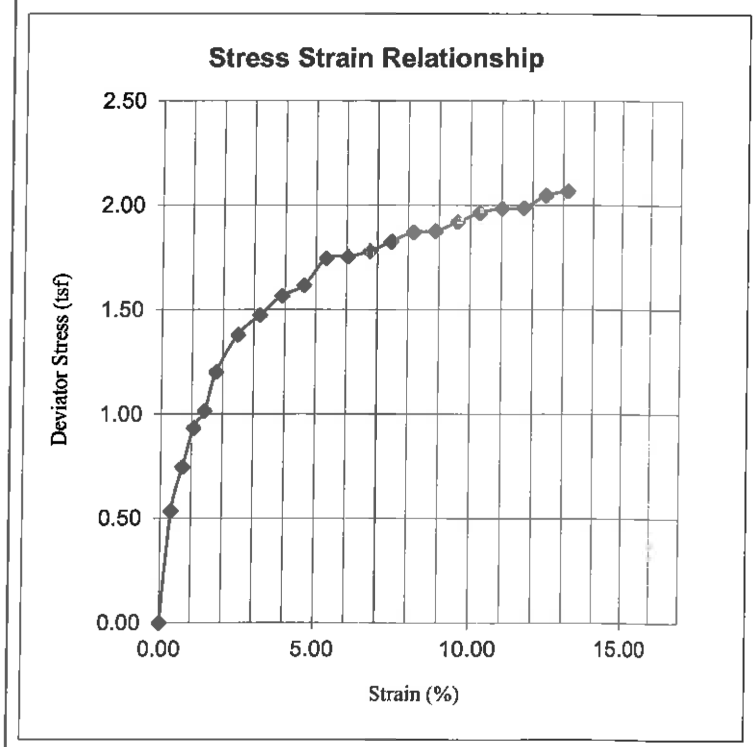
Unconsolidated-Undrained Triaxial Compression Test - Q-test

Project Name	IL 15 over Richland Creek			Tested By	AdJ 7-18-17
Project No.	20175035.00			Checked By	
Boring No.	TH-2	Sample No.	ST-8-2	Depth	18.5-19.0
Soil Description	Gray, lean CLAY (CL)				

			Specimen Data:			Instrument Constants		
Liquid Limit		%						
Plastic Limit		%	Height	5.630	in	Deformation	0.0001	in/div
Plasticity Index		%	Diameter	2.850	in	Load	1.9	lbs/div
USCS			Hgt/Dia ratio	1.98		Strain Rate	0.039	in/min
Specific Gravity	2.70	*	Volume	588.56	cc		0.69	%/min
*assumed			Wet Weight	1137.9	gm			
Water Content Data:			Wet Density	120.7	pcf	Failure Sketch		
Wet & Tare	66.73	gm	Dry Density	97.2	pcf			
Dry & Tare	54.24	gm	Water Content	24.2	%			
Tare	2.63	gm	Saturation	89	%			
Water Content	24.2	%	Void Ratio	0.73				

Moisture content sample taken from: Trimmings

Confining Pressure (psi)	15.5	Undrained Shear Strength (tsf)	1.03
		Axial Strain (%)	13.1



Unconsolidated-Undrained Triaxial Compression Test - Q-test

Project Name	Il 15 over Richland Creek	Tested By	AdJ 7-19-17
Project No.	20175035.00	Checked By	
Boring No.	TH-2	Sample No.	ST-10-3
Depth	24.0-24.5		
Soil Description	Gray, lean CLAY (CL)		

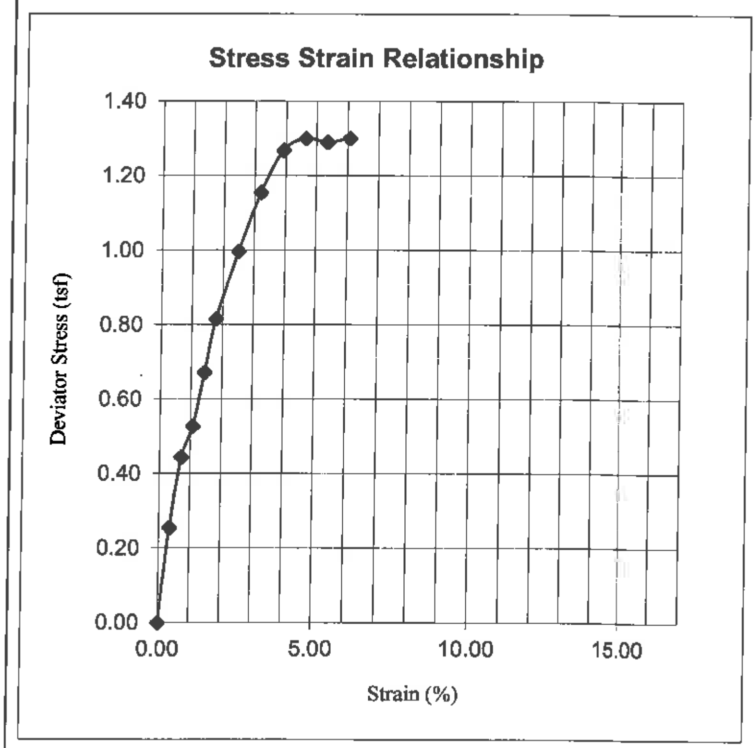
Liquid Limit		%	Specimen Data:			Instrument Constants		
Plastic Limit		%	Height	5.600	in	Deformation	0.0001	in/div
Plasticity Index		%	Diameter	2.860	in	Load	1.9	lbs/div
USCS			Hgt/Dia ratio	1.96		Strain Rate	0.039	in/min
Specific Gravity	2.70	*	Volume	589.54	cc		0.70	% min
*assumed			Wet Weight	1176.44	gm			
Water Content Data:			Wet Density	124.6	pcf	Failure Sketch		
Wet & Tare	80.96	gm	Dry Density	100.1	pcf			
Dry & Tare	65.55	gm	Water Content	24.5	%			
Tare	2.57	gm	Saturation	97	%			
Water Content	24.5	%	Void Ratio	0.68				

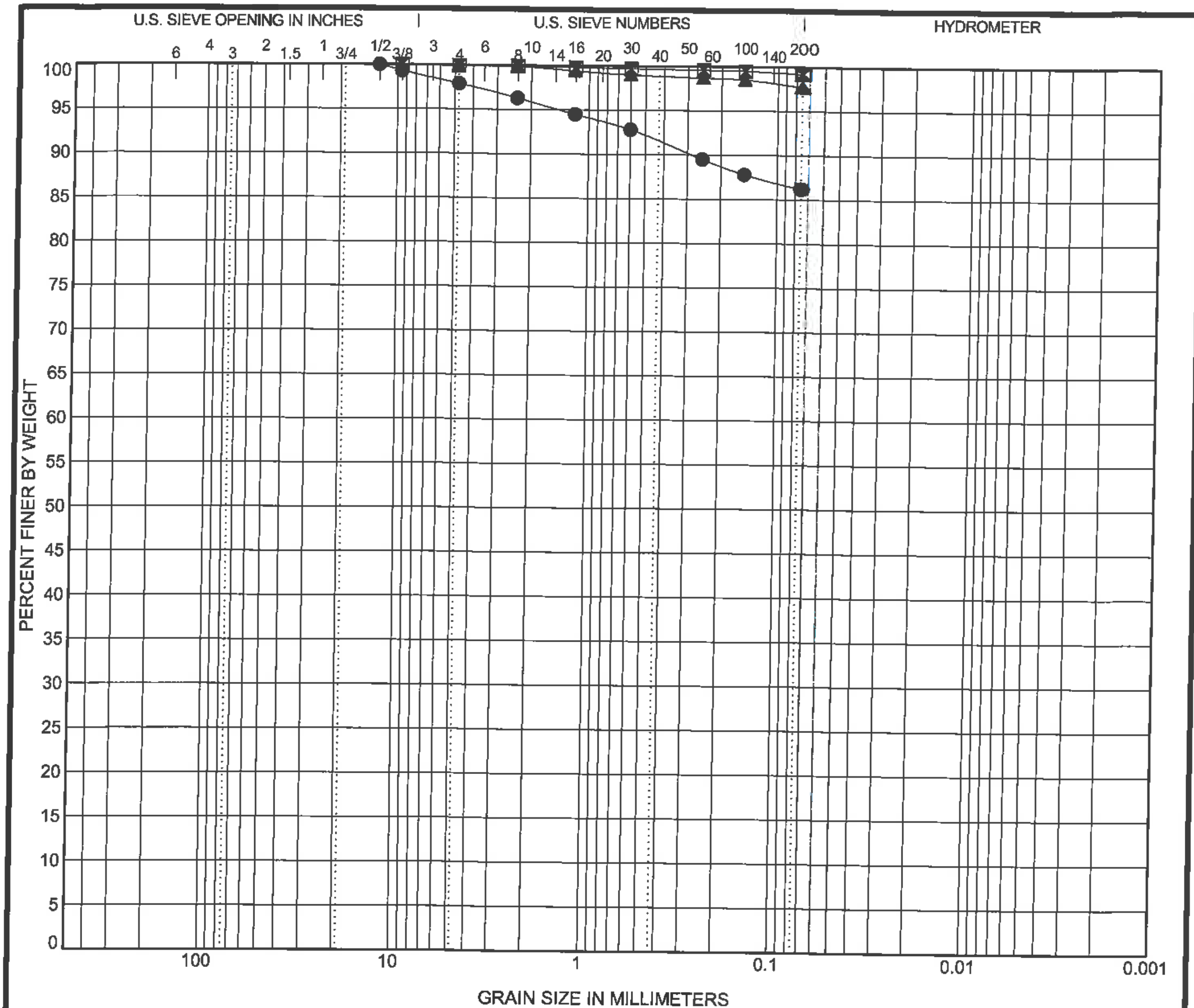
Moisture content sample taken from: Trimmings

Undrained Shear Strength (tsf) 0.65

Confining Pressure (psi) 20

Axial Strain (%) 6.1





COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification				Classification	LL	PL	PI	Cc	Cu
●	TH-1	ST-6	13.5	SILT(ML), trace sand and gravel	36	28	8		
⊠	TH-1	ST-8	18.5	Lean CLAY (CL)					
▲	TH-2	ST-10	23.5	Lean CLAY (CL)					

Specimen Identification				D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TH-1	ST-6	13.5	12.5				2.1	11.7	86.20	
⊠	TH-1	ST-8	18.5	9.5				0.1	0.7	99.20	
▲	TH-2	ST-10	23.5	4.75				0.0	2.3	97.70	

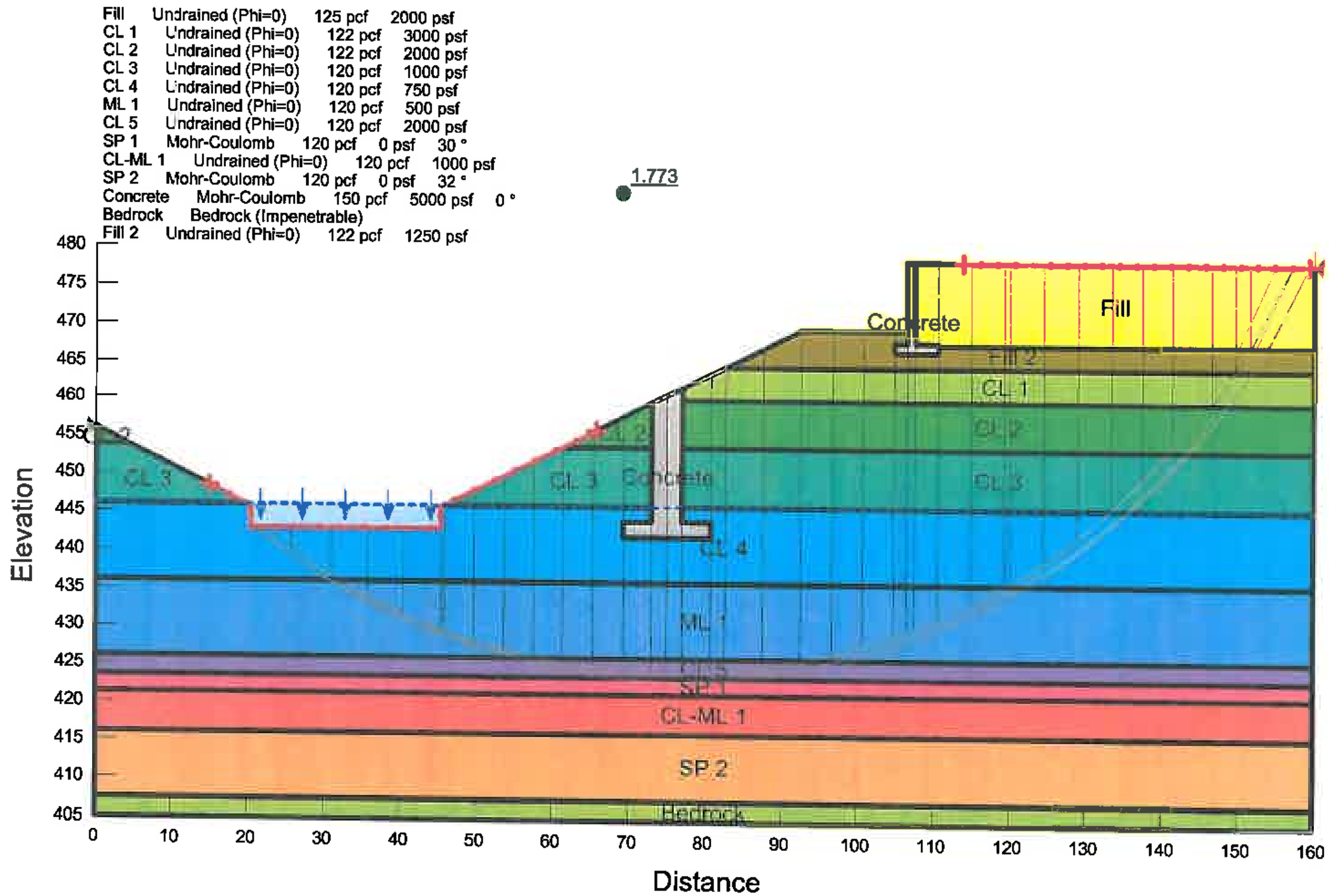
TSI
1240 NORTH PRICE ROAD
ST. LOUIS, MISSOURI 63132

GRAIN SIZE DISTRIBUTION

PROJECT NAME: IL Route 15 over Richland Creek - Retaining Walls	Tested By: KSE
PROJECT LOCATION: St. Clair County	Calculated By: KSE
PROJECT NUMBER: 20175035.00	Checked By: JAS

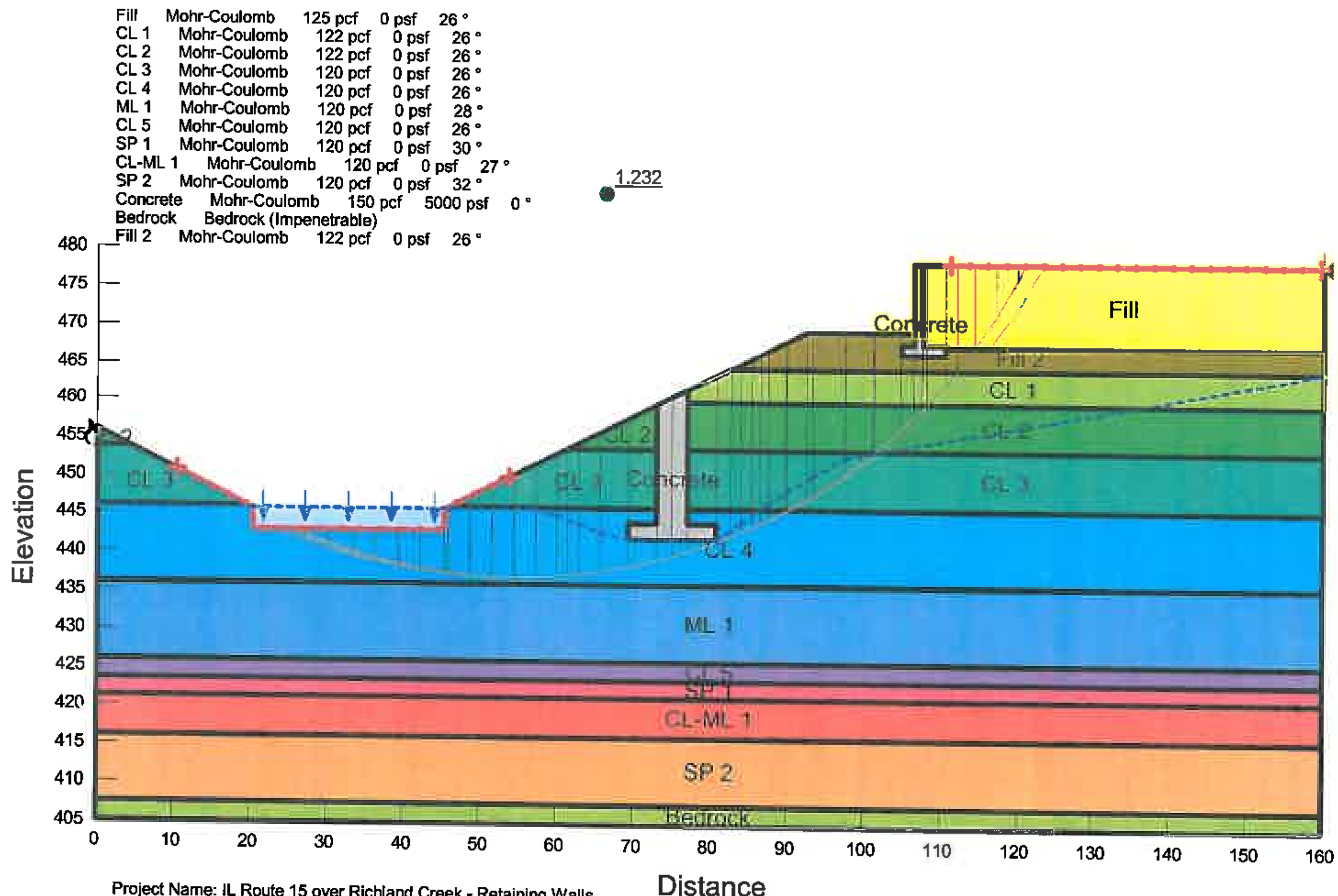
TSI GRAIN SIZE IL 15 OVER RICHLAND CREEK GINT.GPJ US_LAB.GDT 8/10/17

APPENDIX D



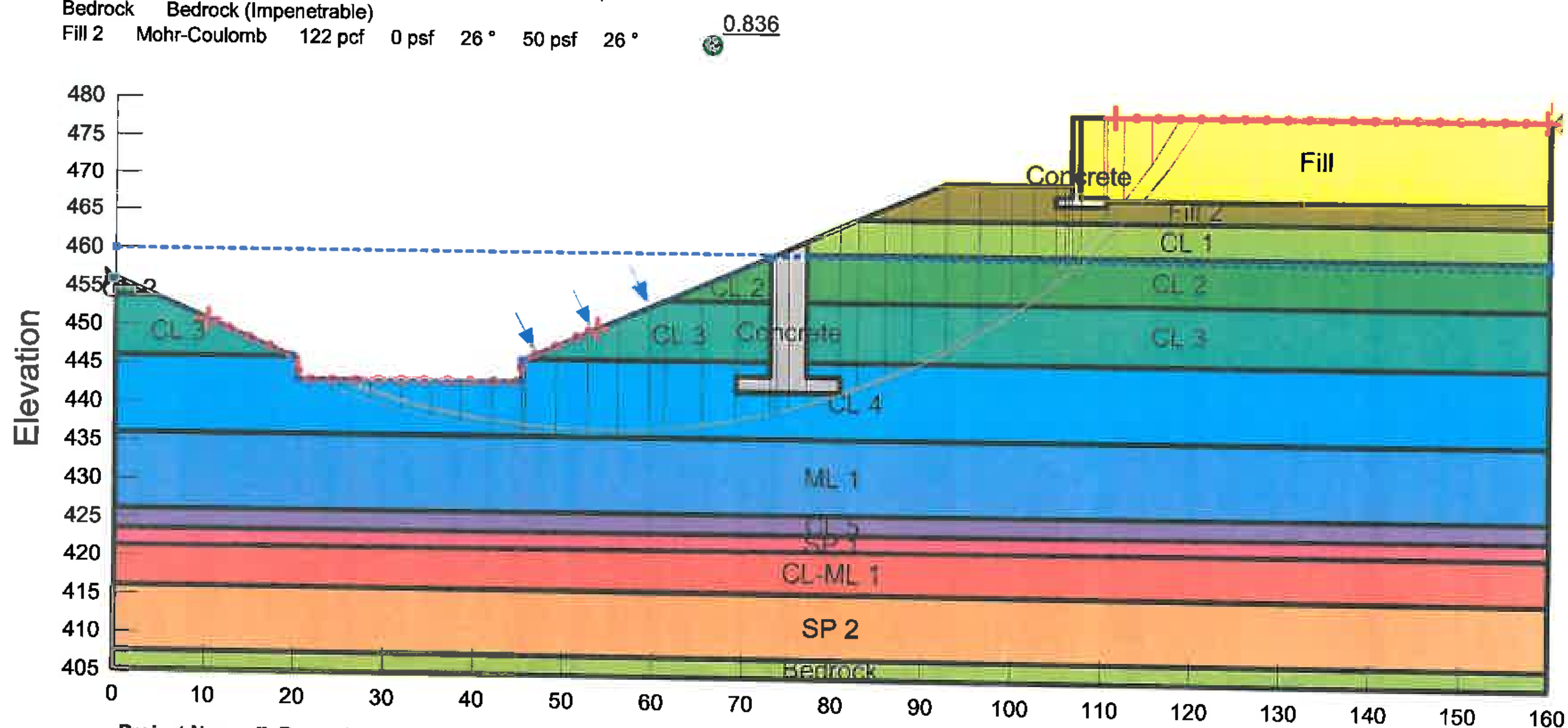
Project Name: IL Route 15 over Richland Creek - Retaining Walls
Project Number: 20175035.00
Analysis: Undrained Condition
Location: East Abutment, Perpindicular to Wall and Slope

TH-2 Undrained Case



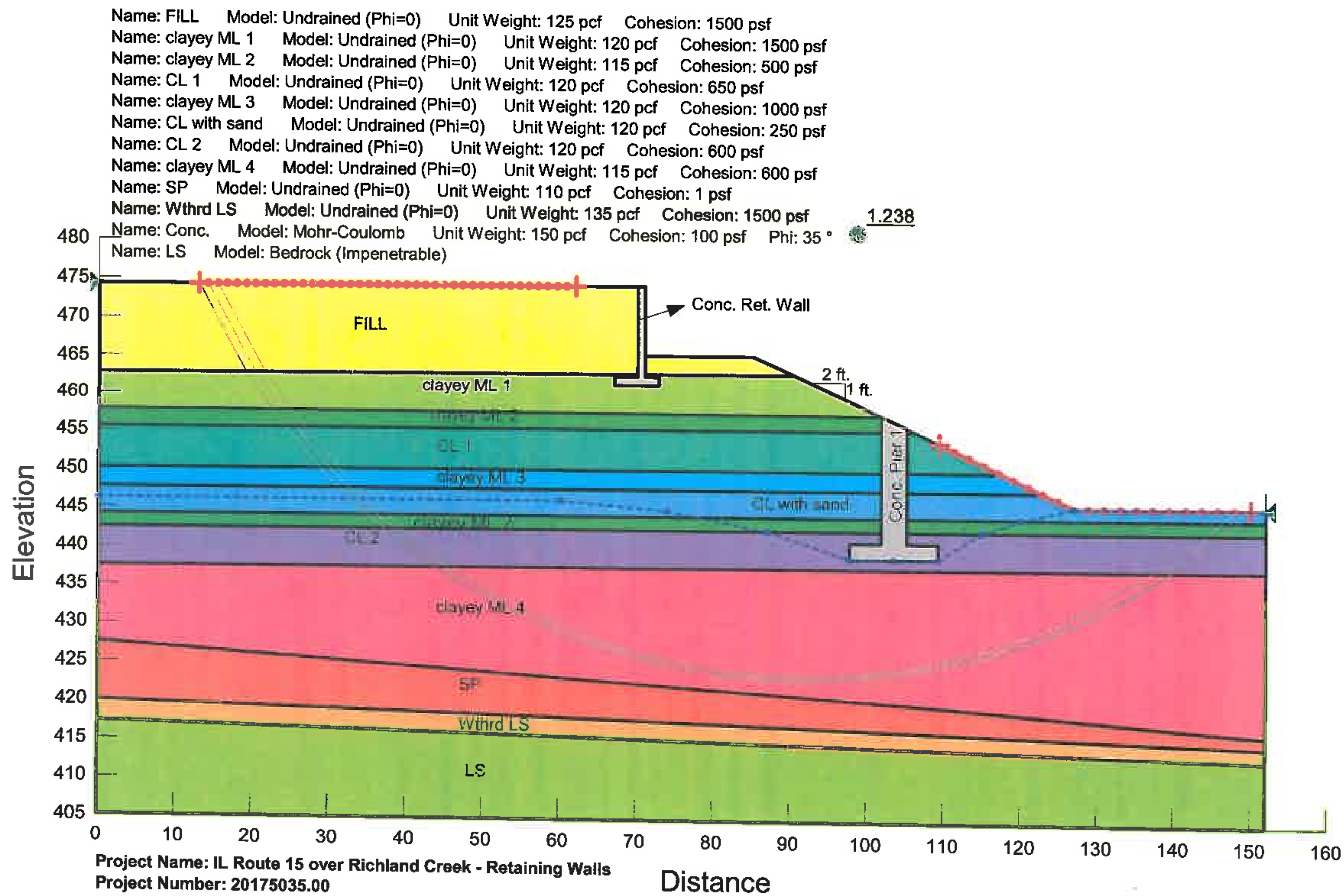
Project Name: IL Route 15 over Richland Creek - Retaining Walls
Project Number: 20175035.00
Analysis: Drained Condition
Location: East Abutment, Perpindicular to Wall and Slope

Fill	Mohr-Coulomb	125 pcf	0 psf	26 °	100 psf	26 °
CL 1	Mohr-Coulomb	122 pcf	0 psf	26 °	75 psf	26 °
CL 2	Mohr-Coulomb	122 pcf	0 psf	26 °	50 psf	26 °
CL 3	Mohr-Coulomb	120 pcf	0 psf	26 °	0 psf	26 °
CL 4	Mohr-Coulomb	120 pcf	0 psf	26 °	0 psf	26 °
ML 1	Mohr-Coulomb	120 pcf	0 psf	28 °	0 psf	28 °
CL 5	Mohr-Coulomb	120 pcf	0 psf	26 °	100 psf	26 °
SP 1	Mohr-Coulomb	120 pcf	0 psf	30 °	0 psf	30 °
CL-ML 1	Mohr-Coulomb	120 pcf	0 psf	27 °	50 psf	27 °
SP 2	Mohr-Coulomb	120 pcf	0 psf	32 °	0 psf	32 °
Concrete	Mohr-Coulomb	150 pcf	5000 psf	0 °	0 psf	0 °
Bedrock	Bedrock (Impenetrable)					
Fill 2	Mohr-Coulomb	122 pcf	0 psf	26 °	50 psf	26 °

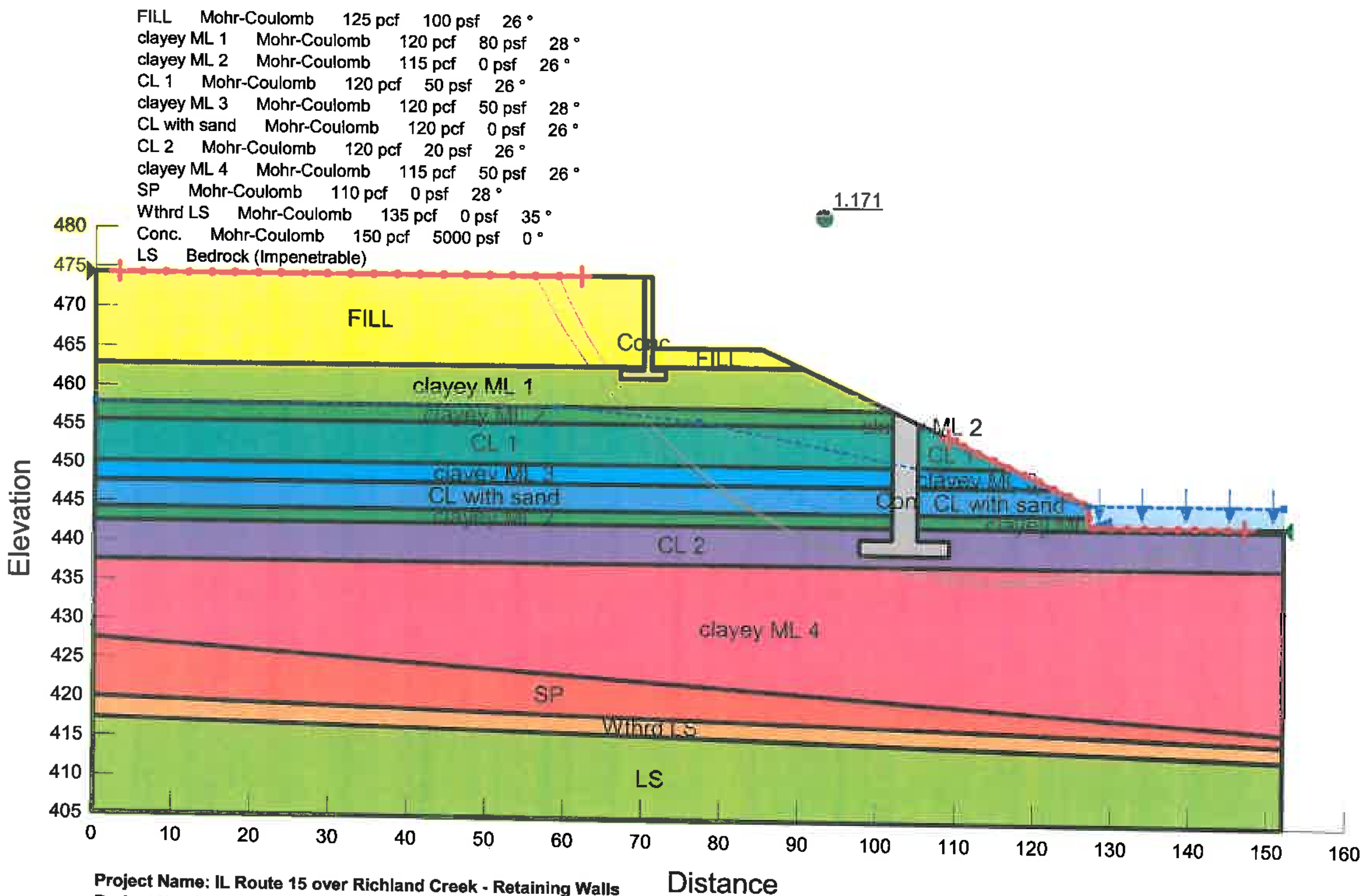


Project Name: IL Route 15 over Richland Creek - Retaining Walls
Project Number: 20175035.00
Analysis: Drawdown Condition
Location: East Abutment, Perpindicular to Wall and Slope

TH-2 Drawdown

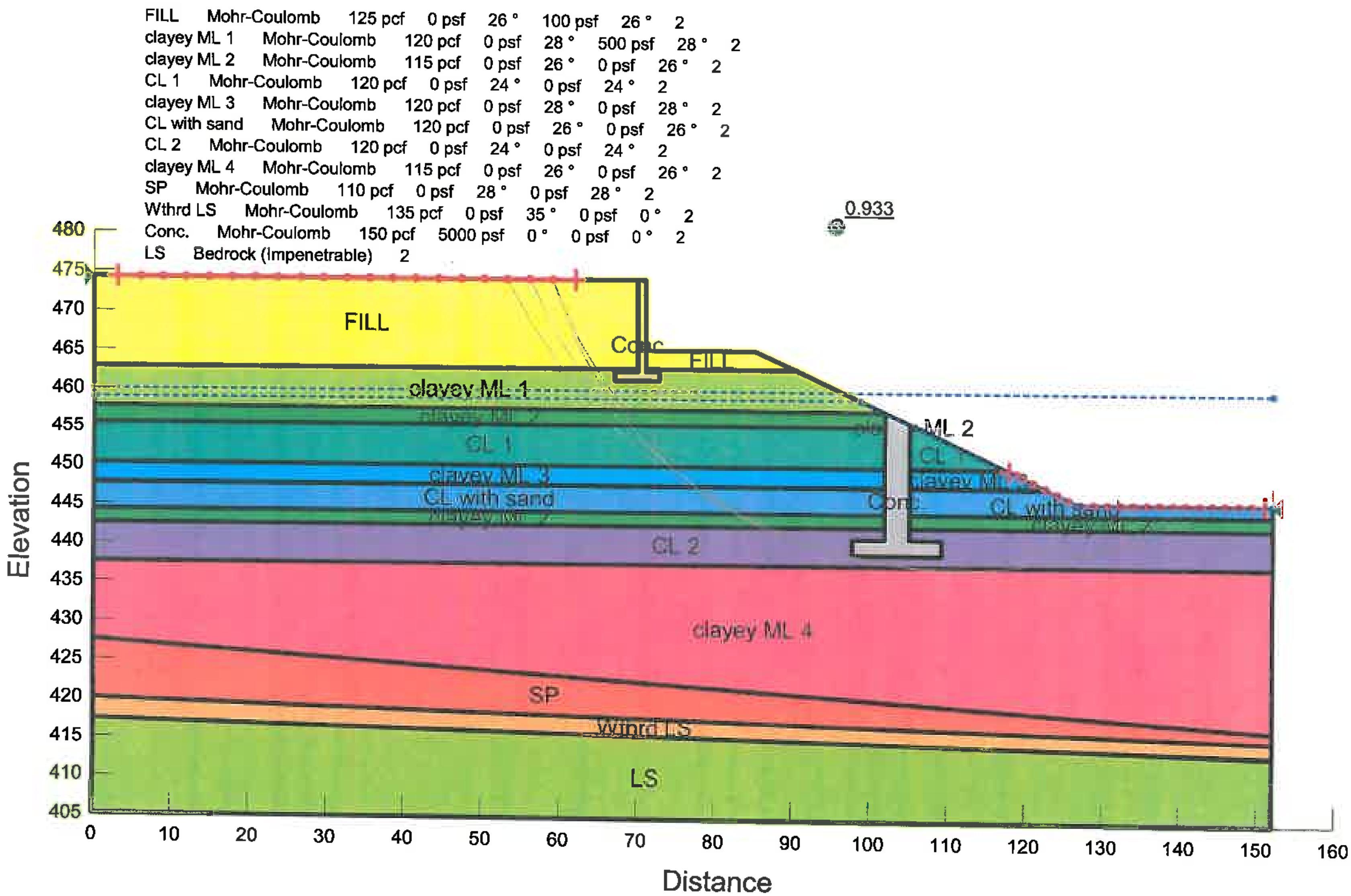


Project Name: IL Route 15 over Richland Creek - Retaining Walls
Project Number: 20175035.00
Analysis: Undrained Condition
Location: West Abutment, Perpindicular to Wall and Slope



Project Name: IL Route 15 over Richland Creek - Retaining Walls
Project Number: 20175035.00
Analysis: Drained Condition
Location: West Abutment, Perpindicular to Wall and Slope

TH-1 Drained Case



Project Name: IL Route 15 over Richland Creek - Retaining Walls
Project Number: 20175035.00
Analysis: Drawdown Condition
Location: West Abutment, Perpindicular to Wall and Slope

TH-1 Drawdown Case