

**NOTICE TO CONSULTANT ENGINEERS  
REGARDING A REQUEST FOR LETTERS OF INTEREST**

April 1, 2018

The City of Knoxville, an Equal Opportunity/Affirmative Action Employer, seeks to retain the services of a professional engineering consulting firm or team of firms with extensive experience in roadway design, structural design, and railroad coordination to provide professional design services to prepare bid specifications, contract documents, and construction plans for the Amherst Road Slope Stabilization Project.

The City of Knoxville intends to stabilize the south side of Amherst Road between 1950 Amherst Road and 2004 Amherst Road. The project will re-establish the roadway shoulder including the installation of guardrail and retaining wall within CSX railroad right-of-way. Results of previous subsurface exploration in this section of roadway shoulder and slope leading down to the railroad tracks are included with this solicitation on the Purchasing Division's website as background information. The scope of service for this project will include survey, design, and preparation of bid specifications, contract documents, and construction drawings.

The City of Knoxville will require interested firms have successfully completed roadway and structural design projects in the last five (5) years and provide information about these projects in their proposal. This project is funded locally by the City of Knoxville and not with Federal or State funds. As such, firms are not required to be Pre-Qualified by TDOT. However, in the interest of uniformity of submissions and simplicity in comparison/evaluation of the submissions, the City does require that submitters provide their statements of qualifications in the TDOT format. With this in mind, the City will require six (6) copies of TDOT prequalification forms DT0330, Parts I & II be completed and submitted with the letter of interest. The TDOT Form DT-0330 Parts I & II may be found on the Internet at: <https://www.tn.gov/content/tn/tdot/business-redirect/consultantinfo/consultantinfo-forms.html> .

***Firms may request consideration by submitting a letter of interest and completed TDOT Form DT0330, Part I & II to:***

**City of Knoxville  
Office of the Purchasing Agent  
City County Building, Suite 667-674  
400 Main Street  
Knoxville, TN 37902**

Letters of interest shall indicate the anticipated scope of services to be completed by subcontractors. **All letters of interest must be received on or before 11:00 AM (Eastern Time) Friday, April 27, 2018.** Late submissions will not be considered.

For additional details regarding this project, please visit the Purchasing Division's website at [www.knoxvilletn.gov/purchasing](http://www.knoxvilletn.gov/purchasing) under "Sealed Submissions."

All questions must be submitted in writing by end of business day April 20, 2018 to Mr. Boyce Evans, Purchasing Agent by letter, e-mail at [bhevans@knoxvilletn.gov](mailto:bhevans@knoxvilletn.gov) , or fax (865) 215-2277.

The factors that will be considered in evaluation of submissions are:

- a. Qualifications, relevant experience, past experience with the City of Knoxville, TDOT and other clients, especially as it relates to roadway design, structural design, and railroad coordination. **(25%)**
- b. Availability of staff to be assigned to work on this project. **(25%)**
- c. Demonstrated ability to meet schedules and perform work efficiently without compromising sound engineering practice. **(25%)**
- d. Evaluations on prior projects. **(20%)**
- e. Amount of work currently under contract with the City of Knoxville. **(5%)**

Evaluation proceedings will be conducted within the established guidelines regarding equal employment opportunity and nondiscriminatory action based upon the grounds of race, color, religion, national origin, sex, creed, age, and disability. Interested certified Disadvantaged Business Enterprise (DBE) firms as well as other minority and/or women-owned firms are encouraged to respond to all advertisements by the City.

City of Knoxville, Tennessee

\_\_\_\_\_  
Date

\_\_\_\_\_  
Purchasing Agent

# Subsurface Exploration

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Amherst Road  
Slope Stability Investigation  
Knoxville, Tennessee

5/10/2016  
FSE Project No. 315152

Prepared For: City of Knoxville

May 10, 2016

Mr. Shawn E. Fitzpatrick, P.E.  
City of Knoxville  
Department of Engineering – Civil Division  
1400 Loraine Street  
Knoxville, Tennessee 37921

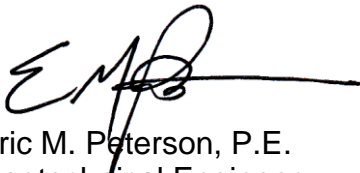
**RE: Subsurface Exploration  
Amherst Road  
Slope Stability Investigation  
Knoxville, Tennessee  
FSE Project No. 315152**

Dear Mr. Fitzpatrick:

As authorized by your acceptance of our proposal dated May 20, 2015, we have completed a subsurface exploration of the slope on the south side of Amherst Road, located approximately between 1950 and 2004 Amherst Road. The section of slope that was investigated is approximately 540 feet in length. The purpose of the exploration was to provide engineering recommendations for stabilization/repair of the south side of the road (shoulder) and slope leading down to the CSX rail line. The firms of Foundation Systems Engineering, P.C. and Construction Materials Laboratory completed this work.

Please give us a call if you have any questions concerning the data obtained or our recommendations. It has been a pleasure to be of service on this project.

Sincerely,  
**Foundation Systems Engineering, P.C.**



Eric M. Peterson, P.E.  
Geotechnical Engineer  
Tennessee No. 109536

JFL/EMP/sf  
Enclosures



Jack F. Lewellyn, Jr., P.E.  
Principal in Charge  
Tennessee No. 16187

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- III. Gabion Wall Analysis - GawacWin 2003
- IV. Boring Location Plan & Auger Boring Records
- V. Laboratory Test Results
  - a. Atterberg Limits
  - b. Natural Moisture Content
  - c. Triaxial Shear Test
- VI. ASFE

## **1.0 EXECUTIVE SUMMARY**

The focus of this subsurface exploration is the section of roadway shoulder and slope leading down to an existing CSX railroad track, located on the south side of Amherst Road. The section of roadway that was explored was between the addresses of 1950 Amherst Road and 2004 Amherst Road. Within this area, several slope face failures have occurred resulting in the loss of the roadway shoulder, including a loss of a portion of the roadway pavement at locations.

In order to investigate the stability of the slope, seven soil borings (A-1 through A-7) with Dynamic Cone Penetrometer testing and undisturbed Shelby tube sampling, were placed at the site. The borings were drilled to depths of 25 to 30 feet below existing grade or auger refusal. Laboratory plasticity, natural moisture content, and triaxial shear testing was performed on selected samples within the laboratory to determine soil plasticity and strength characteristics.

In general, the borings encountered approximately 4 to 4.5 inches of asphalt pavement at existing grade. The pavement was typically underlain by a combination of limestone aggregate base, limestone gravel, and/or additional layers of pavement. The limestone gravel was typically mixed with soil. Fill soils were encountered beneath the pavement and base at three of the boring locations (A-3, A-4, and A-6), ranging in depth from approximately 9 feet to approximately 22 feet below existing grade. The consistency of the fill at borings A-3 and A-4 was firm to stiff, whereas A-6 was soft to firm. Residual soils were encountered beneath the base and/or fill at each of the boring location. The consistency of the residual soil was typically stiff to hard. Borings A-1 and A-3 refused at depths of 7.5 feet and 13 feet below existing grade, respectively. All remaining borings were terminated at their predetermined drill depth of 25 or 30 feet.

The soil samples selected for testing are classified under the Unified Soil Classification System (USCS) as inorganic clays of low to medium plasticity (CL), high plasticity clays (CH) and inorganic silts (ML). The result of the Atterberg limits testing indicates that the soil liquid limit values for the samples selected for testing varied from approximately 22 percent to approximately 56 percent, with an average of approximately 39 percent. Soil plastic limit values varied from approximately 12 percent to approximately 30 percent, with an average soil plastic limit value of was approximately 22 percent. Soil natural moisture content values varied from approximately 11.4% to approximately 34.8%.

Five of the soil samples obtained from the Shelby tube sampling were selected for consolidated, undrained, effective stress triaxial shear testing. The triaxial shear testing

was performed to determine the strength parameter of the soil, including soil unit weight, friction angle, and cohesion. The results of the triaxial shear testing were used for slope stability analysis for the slope located on the south side of Amherst Road, between the road and CSX rail line.

Slope stability analysis was completed using Slide (version 5.044) 2D Limit Equilibrium Slope Stability Analysis Software by Rocscience, Inc. Three typical cross sections of the existing site was adapted from topographic data collected at the site by our firm. The existing slope geometry at each of the three cross sections along with estimated traffic loading was used in our analysis. Subsurface data utilized in our analysis is based on data obtained from site reconnaissance, soil auger borings, laboratory plasticity testing, and triaxial shear testing.

The stability of slopes is discussed in terms of factor of safety values. Factor of safety values are the ratio of the soil driving force to the soil resisting force. We recommend that a Factor of Safety of  $>1.5$  be utilized for minimum allowable slope stability. This recommendation considers that the slope provides support to a roadway, and slope failure could encroach onto CSX rail lines.

The results of our stability analysis indicated factor of safety values with respect to global slope stability ranging from 1.883 to 5.186. In our professional opinion the results of the testing performed indicate that the slope is stable with respect to mass instability. A global slope failure is not predicted to occur. However, we did observe raveling and sluffing of the slope surface (slope face failures). There are sections where the shoulder and edge of the paved portion of the road have been lost. As such, while the slope is stable with regard to mass movement, loss of the roadway due to erosion and sluffing of the exposed outer surface material is occurring.

We recommend the use of Gabion baskets to re-establish the roadway shoulder and protect the roadway from additional shoulder failures. In general, we recommend the following procedures be performed:

- We recommend that the southernmost (eastbound) traffic lane and proposed shoulder area be undercut down to firm, stable soil. We estimate undercut depths ranging from approximately 3 feet to approximately 6 feet below finished subgrade elevation in order achieve a suitable bearing surface area for the Gabion baskets.
- The temporary construction excavation behind the wall should be sloped no steeper than 0.5H to 1V, or otherwise braced.
- Once the required undercut has been performed, the upper 6 inches of the in-place soil at the undercut level should be reworked and recompacted. Fine grading



should be performed as necessary to ensure that the area has been adjusted to the proper elevation.

- The bottom and sides of the excavation should be lined with a minimum 8-ounce per square yard weight non-woven geotextile.
- The Gabion baskets should be constructed, filled, and set into place as per the Gabion basket manufacturer's specifications, and City of Knoxville technical specifications.
- A drain, consisting of a perforated pipe, surrounded by No. 57 sized stone, with stone and pipe encased in a non-woven drainage geotextile, should be placed behind the baskets to ensure no ponding of water occurs.
- Once the Gabion baskets have been constructed up to finished subgrade elevation as outlined above, the roadway undercut/backfilled area should be covered with a high modulus geotextile. The geotextile should be covered with 4 inches of No. 57 stone, compacted to 95%.
- The roadway should be repaved utilizing 8" thickness of TDOT 303D aggregate base, 3" thickness of TDOT 307BM Asphalt Base Course, and 1.5" thickness of TDOT 411D Asphalt Surface Course.
- The guardrail may be reinstalled and anchored into the Gabion shoulder utilizing a formed (sono-tube) concrete base.
- The remaining slope located south (down gradient) of the Gabion baskets and roadway should be protected from erosion utilizing a turf reinforcement mat, such as Propex TRM 1060.
- The attached sketch graphically depicts the above outlined recommendations.

Based on the results of the subsurface exploration, preliminary design work, and similar past projects, we estimate the cost for construction, construction administration, and quality control services for the above outlined recommendations to be approximately **\$357,500.00**. This estimate does not include the costs of permits, flagmen, insurance, etc. that will be required by the CSX railway.

Based on our familiarity with the site and the geotechnical considerations present, we believe that we are uniquely qualified to provide construction observation services during site grading and future building construction. We recommend that our firm be retained to provide quality control testing and observation services during construction.

*The above summary provides an overview only and should not be used as a separate document or in place of reading the entire report including the appendices. The summary is not a substitute for the following detailed sections of this report. A complete discussion of finding and recommendations are included in the following sections of this report.*

## **2.0 OBJECTIVE OF SUBSURFACE EXPLORATION**

The purpose of our exploration was to gather subsurface data to allow geotechnical engineering recommendations for repair of the Amherst Road south roadway shoulder and stabilization of the roadway slope leading down to a CSX railroad line. The section of roadway that was explored was between the addresses of 1950 Amherst Road and 2004 Amherst Road. To meet the objectives of the subsurface exploration, seven soil borings with Dynamic Cone Penetrometer testing and undisturbed Shelby tube sampling, were placed along the subject section of the roadway. The boring locations were identified/located in the field by our personnel. The borings were drilled to depths of 25 to 30 feet below existing grade or auger refusal. Laboratory plasticity, natural moisture content, and triaxial shear testing was performed to determine soil plasticity and strength characteristics.

## **3.0 SCOPE OF SERVICES**

Seven (7) soil auger borings with Dynamic Cone Penetrometer (DCP) testing were used to investigate the section of Amherst Road where the south roadway shoulder and slope leading down to the CSX rail line have experienced surface failure and sluffing. The borings were placed at locations as selected by our personnel. The borings were drilled to depths of 25 to 30 feet below existing grade, or auger refusal. The approximate locations of the borings may be seen on the attached *Boring Location Plan*. Upon completion, the borings were backfilled with concrete.

Dynamic Cone Penetrometer (DCP) testing, ASTM STP 399, was performed at 2.5 foot intervals to a depth of 10 feet below existing grade. Below a depth of 10 feet the penetrometer testing was performed at 5 foot intervals to auger refusal or proposed boring termination depth.

Twenty-three (23), 2-inch diameter, undisturbed, Shelby tube samples of the in-place soils were collected from the site as the borings were advanced. The Shelby tubes were sealed in the field and transported to the laboratory, at which location the soil was extruded from the tubes. The undisturbed samples were visually classified by an engineer. The locations and depths at which the undisturbed samples were taken may be seen in Table II of this report.

Mr. Shawn Fitzpatrick, P.E.  
May 10, 2016  
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A drilling log was prepared by a geotechnical engineer for the borings. The logs include the results of the DCP testing, Shelby tube sampling intervals, groundwater levels, and soil stratification with description.

Five (5) Atterberg limits and seventeen (17) natural moisture content determination tests were performed on selected samples of soil encountered at the boring locations. The Atterberg limits and moisture testing was performed to determine soil plasticity characteristics. The results of the Atterberg Limits test may be seen in Table III of this report, and the results of the natural moisture content testing may be seen in Table IV of this report. Laboratory Atterberg limits and natural moisture content data sheets may be found in the report appendix.

Five (5) consolidated, drained, effective stress, triaxial shear tests were performed on undisturbed samples of soil encountered as the borings were advanced. The triaxial shear test samples were selected from samples collected during undisturbed Shelby tube sampling. The results of the triaxial shear testing may be seen in Table V of this report. Laboratory triaxial shear test data sheets may be found in the report appendix.

A brief summary of the test procedures utilized on this project are located in the Appendix of this report.

#### **4.0 SITE LOCATION AND CONDITIONS**

Amherst Road parallels the CSX Railway for approximately 600 feet between the addresses of 1950 Amherst Road and 2004 Amherst Road. This section of the roadway is located on the Railroad right-of-way.

Topographically, the roadway is situated at the crest of a steep slope, at an elevation of approximately 15 to 20 feet above the railroad. The slope was measured to be sloped at an angle ranging from approximately 1.4H:1V to approximately 0.75H:1V. Steeper or gentler grades may be located between the measurement locations.

Slope surface failures along the southern edge of the roadway have left little to no shoulder. In some areas, the failures have extended into the roadway, undermining and failing the pavement. Some temporary measures have been taken to stabilize the shoulder in these areas, including the use of guardrail posts (as piling), guardrails (as lagging), and sacks of concrete mix.

## **5.0 SUBSURFACE STRATIFICATION**

### **5.1 Drilling And Sampling Procedures**

Seven soil auger borings (A-1 through A-7) were used to investigate the subject section of Amherst Road. The borings were placed using a Bobcat T 200 skid steer with auger head attachment. Dynamic Cone Penetrometer (DCP) testing, ASTM STP 399, was performed through nominal 8-inch diameter Hollow-stem augers as the borings were advanced. A brief summary of the soils encountered at the boring locations follows. Additional subsurface details may be seen on the attached *Auger Boring Records*.

Subsurface stratification indicated on the boring logs is approximate, and represents our interpretation of the soils encountered in the borings at the Shelby tube sampling intervals and on soil auger cuttings returned to the surface on the auger flights.

DCP testing was typically performed at 2.5-foot intervals to a depth of 10 feet below existing grade. Thereafter DCP testing was performed at 5-foot intervals. Twenty-three, 2-inch diameter, undisturbed, Shelby tube samples of the in-place soils were collected at the site as the borings were advanced. The locations and depths at which undisturbed Shelby tube sampling and DCP testing was performed may be seen on the attached *Auger Boring Records*.

The boring depths and approximate ground surface elevation at each of the boring locations may be seen in the following table.

**Table I – Boring Data Summary**

<b>Boring Number</b>	<b>Surface Elevation</b>	<b>Drill Depth, Ft.</b>		<b>Boring Number</b>	<b>Surface Elevation</b>	<b>Drill Depth, Ft.</b>
A-1	1100	7.5 R		A-5	1089	25 T
A-2	1100	30 T		A-6	1090	25 T
A-3	1095	13 R		A-7	1102	30 T
A-4	1090	30 T				

Elevation interpolated from topographic map. T = Terminated. R = Refusal

The locations and depths at which the Shelby Tube samples were taken may be seen in the following table.

**Table II – Shelby Tube Sample Locations**

Boring Number	Shelby Tube Depth, Ft.		Boring Number	Shelby Tube Depth, Ft.
A-1	4' – 6'		A-5	4' – 6'
A-2	1' – 3'		A-5	10' – 12'
A-2	8' – 10'		A-5	18' – 20'
A-2	16' – 18'		A-6	2.5' – 4.5'
A-2	22' – 24'		A-6	6' – 9'
A-3	2' – 4'		A-6	14' – 16'
A-3	6' – 8'		A-6	22' – 24'
A-3	11' – 13'		A-7	8' – 10'
A-4	2' – 4'		A-7	13' – 15'
A-4	9' – 11'		A-7	25' – 27'
A-4	15' – 17'			
A-4	20' – 22'			
A-4	26' – 28'			

## 5.2 Soil Overburden

The subsurface stratification indicated on boring logs is approximate, and was developed by a geotechnical engineer based on his interpretation of the driller's field log, soil auger cuttings returned to the ground surface on auger flights, and on undisturbed Shelby tube sampling.

The soil test borings were drilled vertically. The data indicated on the boring logs, and as summarized in the abbreviation below, indicates the findings at the boring locations only. The typical boring cross sections accompanying the boring logs is generalized, and briefly summarizes graphically the material encountered at the boring locations. The ground surface elevation indicated on boring logs is approximate. Following is a brief summary of the soils encountered at the boring locations.

## 5.3 Soil Overburden

Boring A-1 encountered approximately 4 inches of asphalt pavement at existing grade. The asphalt pavement is underlain by approximately 20 inches of alternating layers of aggregate base and asphalt pavement. Residual soil was encountered beneath the aggregate base/asphalt layers. The residual soils have been derived from the in-place weathering of the underlying bedrock unit. From a depth of approximately 2 feet (24

inches) to a depth of approximately 7.5 feet, stiff to very stiff, moist, tan, reddish tan and reddish brown, silty clay mixed with chert was encountered. The augers refused at a depth of 7.5 feet below existing grade. No groundwater was encountered at the time of drilling.

Boring A-2 encountered approximately 4.5 inches of asphalt pavement at existing grade. The asphalt pavement is underlain by approximately 7.5 inches of limestone base mixed with moist, brown silty clay. Residual soil was encountered at a depth of approximately 12 inches. The residual soil consists of very stiff to hard, moist, tan, reddish tan, and reddish brown, silty clays. The boring was terminated at a depth of 30 feet below existing grade. No groundwater was encountered at the time of drilling.

Boring A-3 encountered approximately 4 inches of asphalt pavement underlain by approximately 14 inches of crushed limestone base mixed with broken asphalt and topsoil. Fill soils were encountered at a depth of approximately 18 inches below grade. The fill soils were encountered to a depth of approximately 9 feet below existing grade. The fill soils consist of firm to stiff, moist, tan and reddish brown, silty clay. Residual soil was encountered at a depth of approximately 9 feet. The residual soil consists of very stiff, moist, tan and reddish brown, silty clay. The augers refused at a depth of 13 feet. No groundwater was encountered at the time of drilling.

Boring A-4 encountered approximately 4 inches of asphalt pavement underlain by approximately 5 inches of crushed limestone aggregate base. The aggregate base course was underlain by approximately 9 inches of limestone gravel mixed with gray and black, organic, silty clay. Soil fill was encountered beneath the soil/gravel mixture. The fill soils consist of stiff, very moist to moist, tan and reddish brown, silty clay mixed with chert. Residual soil was encountered at a depth of approximately 15 feet. The residual soil consists of stiff, moist, tannish gray, tan and reddish tan, silty clay mixed with chert. Boring A-4 was terminated in very stiff residual soil at a depth of 30 feet below existing grade. No groundwater was encountered at the time of drilling.

Boring A-5 encountered approximately 4.5 inches of asphalt pavement at existing grade. The asphalt pavement was underlain by approximately 7.5 inches of crushed limestone aggregate base mixed with moist to very moist, gray and black, organic, silty clay. Residual soil was encountered at a depth of approximately 12 inches below existing grade. From a depth of approximately 12 inches to a depth of approximately 25 feet, stiff to very stiff, moist, tan, reddish tan and reddish brown, silty clay mixed with fine to coarse chert was encountered. Boring A-5 was terminated in very stiff residual soil at a depth of 25 feet below existing grade. No groundwater was encountered at the time of drilling.

Boring A-6 encountered approximately 4 inches of asphalt at existing grade. Fill was encountered beneath the asphalt pavement. From a depth of approximately 4 inches to a depth of approximately 5 feet, very soft, very moist, dark gray and black, organic silty clay mixed with limestone gravel was encountered beneath the asphalt pavement. Firm, very moist, brown and gray, silty clay fill was encountered at a depth of approximately 5 feet. This material extended to a depth of approximately 6 feet. From a depth of approximately 6 feet to a depth of approximately 14 feet the fill consists of firm, very moist, gray silty clay mixed with clayey silt. From a depth of approximately 14 feet to a depth of approximately 22 feet the fill consists of firm, very moist, gray silty clay mixed with fine to coarse chert. What we believe to be residual soil as encountered at a depth of approximately 22 feet below existing grade. The residual soil consists of stiff, very moist, tannish gray and light gray, sandy, clayey silt mixed with fine to coarse chert. Boring A-6 was terminated at a depth of 25 feet below existing grade. No groundwater was encountered at the time of drilling.

Boring A-7 encountered approximately 4 inches of asphalt pavement at existing grade. The asphalt pavement was underlain by approximately 14 inches of alternating layers of aggregate base and asphalt pavement. Residual soil was encountered at a depth of approximately 18 inches below existing grade. The residual soil consists of firm to very stiff, moist, tan, reddish tan and reddish brown, silty clay mixed with variable amounts of chert gravel. Boring A-7 was terminated in stiff residual soil at a depth of 30 feet below existing grade. No groundwater was encountered at the time of drilling.

#### **5.4 Ground Water**

By definition, ground water is the continuous body of subsurface water that fills the soil, rock voids and fissures and is free to move under the influence of gravity. The water table or phreatic surface is the level of zero (atmospheric) pressure in a continuous body of ground water. The ground water level is not a static level surface as the term *water table* implies. Instead, it is the sloping surface of a moving stream of water in the voids and fissures.

Ground water was not encountered at the boring locations at the time of drilling. For reasons of safety the bore holes were backfill with bag mix concrete after augers were removed from the ground. A deeper bedrock aquifer will typically be encountered at some depth into the bedrock below the ground surface. This water level is below the level of our 25 to 30 foot boring depths. A study of the deeper bedrock aquifer was beyond the scope of our exploration.

## **6.0 GEOLOGY**

Geologic mapping indicates that the site is underlain by the Knox Group, lower and middle part undifferentiated. The Knox Group was deposited in a pretidal environment on the Late Cambrian – Early Ordovician North American passive margin. The Knox Group has been divided into five formations based primarily on the characteristics of chert and sandstone blocks present in the residuum. The five formations are as follows: Mascot Dolomite, Kingsport Formation, Longview Dolomite, Chepultepec Dolomite and Copper Ridge. In some areas the Longview Dolomite and Chepultepec Dolomite are undifferentiated, and are referenced as the Newala. The Knox Group consists primarily of dolostone (dolomite) and secondarily of limestone. The middle and lower units consist of the Longview Dolomite, Chepultepec Dolomite and Copper Ridge. Mapping indicates that the site is underlain by the Copper Ridge member of the Knox Group.

The site is located on the Bearden Geologic Quadrangle map (Tennessee) GQ-126. This quadrangle has been geologically mapped at a 1:24,000 scale. The quadrangle was geologically mapped in 1954. Geologic mapping was published in 1960 by Mr. J. Mark Cattermole (State Geologist). Base mapping was completed by the Topographic Division of the U.S. Geological Survey, 1953.

Published mapping indicates that the crest line of a northeast to southwest plunging anticline is located to the southeast of the site. The mapped location of the crest line is approximate. Rock strike is northeast to southwest and rock dip is to the northwest.

The Copper Ridge Dolomite is described as follows: *Dolomite, medium- to coarse-crystalline, dark-gray, asphaltic, thick bedded; with stromatolite bioherms, thin sandstones, lower part' upper part is light- to medium- gray dolomite and generally not as thick bedded; cherty, with oolites, cryptozoans preserved in residuum.*

Weathering has resulted in an irregular, pinnacled, top of rock. Cavities and sinkholes are common. The unweathered limestone is hard to very hard. Data published by TVA indicates rock compressive strength values of up to 40,000 psi.

The Copper Ridge dolomite is a soluble carbonate bedrock unit that is subject to solutioning and sinkhole development. Open and soil filled cavities are common. Several very large sinkholes are located to the northeast and southwest of the subject sinkhole, along rock strike. The Copper Ridge is one of the formations responsible for much of the cavern development in Knox County.



The formation of sinkholes is the result of chemical solution of limestone, dolomite, marble and other carbonate rock units by percolating groundwater containing dissolved carbon dioxide that makes the water slightly acidic. During the early stages of solution, cavities will tend to form on joints and bedding along which the water flow is concentrated and the cavities may follow a reasonably regular pattern. As the solution process develops and the cavities enlarge, their size, location, and shape become impossible to predict with certainty.

Soil fines located at the soil/bedrock interface are carried by percolating groundwater into cavities within the rock. With time, a small dome shaped void forms at the location where soil is lost into the rock. As additional soil material is lost (eroded) from the roof of the dome, the soil cavity enlarges. When sufficient material is lost from the roof of the dome that the dome is unable to support the overlying material, a ground surface collapse occurs. It is also possible for soil fines that surround rock pinnacles to be eroded into cavities within the bedrock. Depending upon the amount of soil overlying the rock pinnacle, this may result in a roof dome collapse or the eroded area may increase in size based on the amount of erosion that occurs as surface water drains to the pinnacle.

Ground surface subsidence can also occur as a result of distortion (settlement) of the residual soil overlying a pinnacled bedrock surface, where the residual soils are impaled onto narrow rock pinnacles. The addition of significant new loading from fill or a building area, particularly in conjunction with site grading activities that result in removal of the stiff upper crust of residual soil, can result in the downward movement of the residual soil mass and subsidence (settlement) at the ground surface.

It is not unusual to discover that there are numerous sinkhole throat areas within and surrounding collapse areas. As some become clogged with soil fines, others develop, and the soil collapse overlying the bedrock enlarges. The volume of surface water flowing into an open sinkhole will typically flush partially filled cavities. This can result in an increase in size of existing sinkholes and result in development of dropouts.

Geologists and geotechnical engineers generally recognize three (3) different types of sinkholes. These are; Dissolution; Cover-subsidence; and Cover-collapse.

Dissolution sinkholes are typical in areas of thin soil overburden, where aggressive dissolution of limestone and dolostone occurs. A ground surface depression develops as a result of bedrock dissolution, with the thin layer of soil overburden washing/collapsing into cavities within the bedrock.

Cover-subsidence sinkholes tend to develop gradually where the covering soil overburden is permeable and contains sand. The slow downward erosion of the sandy overburden into cavities within the rock results in a ground surface depression.

Cover-collapse sinkholes may develop abruptly (over a period of hours) and cause catastrophic damages. They occur where the covering soil overburden contains a significant amount of clay. A cover-collapse sinkhole is the most common type of sinkhole in East Tennessee.

All structures (buildings, roadways, utility lines, basins, etc.) supported on soil overlying soluble carbonate rock units (such at this site) are at risk of damage due to sinkhole development. This risk can be minimized but can rarely be eliminated. As the degree of risk is minimized, the cost of the technique used to minimize the risk typically increases.

Sinkhole development (solutioning of the bedrock, roofing of the soil overburden, and ground surface subsidence) is an ongoing process. Sudden and dramatic sinkhole collapse is possible, as well as slower insidious development.

## **7.0 LABORATORY TESTING**

Construction Materials Laboratory (CML) has demonstrated proficiency for the testing of construction materials and has met the requirements of AASHTO R18 set forth by the AASHTO Highway Subcommittee on Materials. CML received a Certificate of Accreditation from the American Association of State Highway and Transportation Officials AASHTO Accreditation Program.

### **7.1 Atterberg Limits And Natural Moisture Content**

Five samples of soil obtained from the undisturbed Shelby tube sampling were selected for Atterberg Limits testing. The samples were collected from boring A-2, A-3, A-5, A-6 and A-7. Seventeen natural moisture content determination tests were performed on selected undisturbed samples obtained from all boring locations.

The soil liquid limit is the moisture content at which the soil passes from a plastic to a liquid state. Since soil cohesion retards flow, the liquid limit test is an index of soil cohesion. Soil cohesion is negligible at the liquid limit. High liquid limit values (greater than 50) indicate soils of high clay content and low load carrying characteristics.

The soil plastic limit is the moisture content at which the soil changes from a semisolid to a plastic state. Some silts and sands are non-plastic; however, most soils composed of silts and clays do have a plastic limit. The moisture content of silts and clays does have a direct bearing on their load carrying characteristics. A very important change in load carrying capacity of soil occurs at the plastic limit. Load carrying capacity decreases rapidly as the soil moisture content increases above the plastic limit.

**Table III - Atterberg Limits Data Summary**

<b>Boring Number</b>	<b>Depth, Feet</b>	<b>Liquid Limit, %</b>	<b>Plastic Limit, %</b>	<b>Plasticity Index</b>	<b>Natural Moisture Content, %</b>	<b>USCS Soil Classification</b>
A-2	22' – 24'	41	<b>30</b>	11	<b>32.7</b>	ML
A-3	2' – 4'	22	13	9	12.2	CL
A-5	18' – 20'	<b>56</b>	<b>29</b>	27	<b>29.9</b>	CH
A-6	6' – 8'	27	<b>12</b>	15	<b>16.9</b>	CL
A-7	13' – 15'	48	34	24	31.6	CL

The soil samples selected for testing are classified under the Unified Soil Classification System (USCS) as inorganic clays of low to medium plasticity (CL), high plasticity clays (CH) and inorganic silts (ML).

The low to medium plasticity clays (CL) typically have good to fair compaction characteristics, with compaction typically achieved using a sheepsfoot or rubber tired roller. In general, the required compaction (percentage of Standard Proctor maximum dry density) for the upper 8 feet of fill beneath modestly loaded structures is 100 percent. Such soils have medium compressibility and expansion characteristics, and poor drainage characteristics.

The inorganic silts (ML) have good to poor compaction characteristics, with compaction typically achieved using a sheepsfoot or rubber tired roller. In general, the required compaction for the upper 8 feet of fill beneath modestly loaded structures is 100 percent. Such soils have slight to medium compressibility and expansion and poor drainage characteristics.

The high plasticity clays (CH) typically have fair to poor compaction characteristics, with compaction typically achieved using a sheepsfoot roller. Such soil is not recommended for use beneath building and paved areas. When such soil must be used it is typically not placed within the upper 8 feet of fills beneath buildings. In general, the required

compaction (percentage of Standard Proctor maximum dry density) for the fill below a depth of 8 feet beneath building areas is 100 percent. Such soils typically have moderate to high compressibility and expansion characteristics, and poor drainage characteristics.

Soil strength values decrease rapidly as the soils reach and exceed the soil plastic limit. The soil will typically flow as a heavy viscous fluid at the liquid limit.

In general, the required compaction for fill soils placed beneath modestly loaded structures is 100 percent of its maximum dry density as determined by the Standard Proctor test, ASTM D 698.

**Table IV - Natural Moisture Content Data Summary**

<b>Boring Number</b>	<b>Depth, Ft.</b>	<b>Moisture Content - %</b>		<b>Boring Number</b>	<b>Depth, Ft.</b>	<b>Moisture Content - %</b>
A-1	4' – 6'	22.2		A-5	4' – 6'	13.4
A-2	8' – 10'	<b>31.3</b>		A-5	10' – 12'	14.2
A-2	16' – 18'	<b>33.0</b>		A-5	18' – 20'	<b>29.9</b>
A-2	22' – 24'	<b>32.7</b>		A-6	6' – 8'	16.9
A-3	2' – 4'	12.2		A-6	22' – 24'	17.0
A-3	11' – 13'	11.4		A-7	8' – 10'	<b>34.8</b>
A-4	2' – 4'	15.1		A-7	13' – 15'	<b>31.6</b>
A-4	9' – 11'	18.2		A-7	25' – 27'	<b>32.0</b>
A-4	15' – 17'	16.2				

**Bold** = Above Plastic Limit

The result of the Atterberg limits testing indicates that the soil liquid limit values for the samples selected for testing varied from approximately 22 percent to approximately 56 percent, with an average of approximately 39 percent. Soil plastic limit values varied from approximately 12 percent to approximately 30 percent, with an average soil plastic limit value of was approximately 22 percent. Soil natural moisture content values varied from approximately 11.4% to approximately 34.8%. The soil will typically flow as a heavy viscous fluid at the liquid limit. Soil strength values decrease rapidly as the soil moisture content exceeds the plastic limit.

## 7.2 Triaxial Shear Test Data Summary

Five (5) samples of soil obtained from the Shelby tube sampling were selected for consolidated, undrained, effective stress triaxial shear testing. Samples were collected from each of the borings. The results of the triaxial shear testing may be seen in the following table.

**Table V – Triaxial Shear Test Data Summary**

Boring Number	Sample Depth, Ft.	Wet Density, PCF	Water Content - %	Effective Cohesion – PSF	Effective Phi Angle – Degrees	Initial Void Ratio*
A-2	22' – 24'	118.8	32.7	1575	16.0	0.8841
A-3	2' – 5'	140.1	12.2	265	53.7	0.3244
A-5	18' – 20'	121.1	29.9	1828	16.8	0.8088
A-6	6' – 8'	135.3	16.9	616	25.8	0.4573
A-7	13' – 15'	119.5	31.6	1173	26.4	0.8555

\*Based on assumed specific gravity of 2.65.

The results of the triaxial shear testing were used for slope stability analysis for the slope located on the south side of Amherst Road, between the road and CSX rail line. The results of the slope stability analysis are discussed in the following section.

## **8.0 SLOPE STABILITY ANALYSIS**

Our stability analysis was completed using Slide (version 5.044) 2D Limit Equilibrium Slope Stability Analysis Software by Rocscience, Inc. Slide was developed to handle slope stability problems utilizing a number of methods. A circular search (global slope stability) using the Bishop Simplified and Spencer method of slices was utilized in the analysis of the slope. Three typical cross sections of the existing site was adapted from topographic data collected at the site by our firm. The existing slope geometry at each of the three cross sections along with estimated traffic loading was used in our analysis. Subsurface data utilized in our analysis is based on data obtained from site reconnaissance, soil auger borings, laboratory plasticity testing, and triaxial shear testing.

The stability of slopes is discussed in terms of factor of safety values. Factor of safety values are the ratio of the soil driving force to the soil resisting force. The driving force consists of the weight of the soil or rock mass, plus any surcharge loading such as from

roadway traffic. The resisting force is the strength of the in-place soil or rock mass (soil friction angle and cohesion).

Based on our past experience and data from published engineering texts, we offer the following general guidelines for evaluation of slope stability factor of safety values. At a factor of safety of less than 1, a slope failure can be expected. At a factor of safety of between 1 and 1.1, slope failures are common. At a factor of safety of between 1.1 and 1.25, slope failures do occur. At a factor of safety of greater than 1.25, slope failures almost never occur. The greater the factor of safety value, the less likely that a failure will occur. In their text entitled *Soil Mechanics*, T. W. Lambe and R. V. Whitman provide the following guidelines *“For intact homogenous soils, when the strength parameters have been chosen on the basis of good laboratory tests and a careful estimate of pore pressure has been made, a safety factor of at least 1.5 is commonly employed. With fissured clays and for non-homogenous soils larger uncertainties will generally exist and more caution is necessary.”* The on-site, in-place soils are non-homogenous. Data representing the in-place soil and rock is believed to be generally representative of the in-place soil.

Based on the guidelines as outlined above, we recommend that a Factor of Safety of >1.5 be established as the minimum allowable slope stability. This recommendation also considers that the slope provides support to a roadway, and slope failure could encroach onto CSX rail lines.

The results of our stability analysis of the existing slope are summarized in the following table. Illustrations of our analysis are included with this report.

**Table VI – Slope Stability Analysis Summary**

<b>Boring Location</b>	<b>Factor of Safety (FS)</b>
A-2	4.573
A-3	1.883
A-5	5.186
A-6	2.507
A-7	4.108

Based on the results of the testing performed, in our professional opinion the results of the testing performed indicate that the existing slope is stable with respect to mass instability. A global slope failure is not predicted to occur.

Mr. Shawn Fitzpatrick, P.E.  
May 10, 2016  
Page 17

As described above, we did observe raveling and sluffing of the slope surface. There are sections where the shoulder and edge of the paved portion of the road have been lost. As such, while the slope is stable with regard to mass movement, loss of the roadway due to erosion and sluffing of the exposed outer surface material is occurring.

As per our discussion with you, we understand that it is desirable to re-establish a portion of the roadway shoulder and the asphalt paved road. Also, it is desirable to construct a guard rail along the south shoulder, at the crest of the slope leading down to the railroad.

In this regard we have considered several methods of slope retention to allow the shoulder reconstruction. Most methods would allow re-establishment of the roadway shoulder and construction of a guard rail system, and would add to the slope stability factor-of-safety value. However; given the existing "good" slope stability factor-of-safety values, in our opinion many of the considered systems would be costly without meaningful benefit.

One system that was considered, and is recommended, is the use of Gabion Baskets placed at the crest of the slope to allow the roadway shoulder to be widened and a guard rail system to be constructed. The attached sketch graphically depicts our recommended stabilization technique.

A slope stability analysis was performed using the Maccaferri Inc. GawacWin slope stability analysis program. The results of the analysis indicates an overall stability safety coefficient of 1.80. With respect to sliding and overturning of the Gabion baskets, the results of the analysis indicates factors of safety of greater than 10.0. The results of our analysis are included within the appendix of this report.

A temporary construction slope with a slope geometry of 0.5H:1V and a maximum height of 10 feet was evaluated utilizing Slide. The temporary slope was modeled based on data obtained from soil auger borings, laboratory plasticity testing, and triaxial shear testing. The results of the temporary slope stability analysis indicate a factor of safety coefficient of 1.611. The results of our analysis are included within the appendix of this report.

## **9.0 RECOMMENDATIONS**

We offer the following geotechnical engineering recommendations for stabilization and reconstruction of the roadway shoulder, and placement of a guard rail system. Our recommendations are based on our site visit, review of available topographic and geologic mapping, field exploration, laboratory testing and engineering analysis.

### **9.1 Roadway Preparation**

We recommend that the roadway shoulder be widened and stabilized using Gabion baskets placed at the crest of the slope. A sketch that graphically depicts our recommended stabilization technique is included with this report.

In preparation for the areas where the roadway shoulder needs to be re-established, we recommend that the southernmost (eastbound) traffic lane and proposed shoulder area be undercut down to firm, stable soil. The depth of undercut will be dependent on existing site topography, and the required depth to establish a 3-foot wide shoulder. We estimate undercut depths ranging from approximately 3 feet to approximately 6 feet below finished subgrade elevation in order achieve a suitable bearing area for the Gabion baskets. The existing asphalt pavement should be saw-cut prior to excavation.

The temporary construction excavation behind the wall should be sloped no steeper than 0.5H to 1V, or otherwise braced. The temporary construction slope should be constructed no taller than 8 feet vertically. Traffic barriers should be placed along the crest of the excavation. Truck traffic and other heavy loads should not be allowed behind the temporary construction excavation. We recommend that truck traffic and other heavy traffic loads be diverted around the construction area.

Once the lane and shoulder has been undercut to the necessary depth, the upper 6 inches of the in-place soil at the undercut level should be reworked and recompacted to a minimum of 95% Standard Proctor maximum dry density. Once reworked and recompacted the surface of the subgrade should be smoothed/leveled to remove clods, low areas, etc. Fine grading should be performed as necessary to ensure that the area has been adjusted to the proper elevation. ASTM C 33 size No. 57 crushed limestone gravel should be used as bedding and backfill material. The No. 57 sized stone should be compacted to the equivalent of a minimum of 95% Standard Proctor maximum dry density. Backfill lift thickness should be limited to a maximum of 12 inches, loose. In-place density testing should be performed concurrent with fill placement to ensure that the required compaction is achieved.



Once the surface has been smoothed, the bottom and sides of the excavation should be lined with a minimum 8-ounce per square yard weight non-woven geotextile. The geotextile should be placed hand-tight, such that wrinkles and grabs are removed. The geotextile should be lapped a minimum of 2 feet between roll widths and at the ends of the rolls.

Once prepared as outlined above, the Gabion baskets should be constructed and filled. Gabion baskets are typically sized 3 feet wide; and range in height from 1 foot to 3 feet; and range in length from 4.5 feet to 12. The Gabion baskets should be constructed, filled, and set into place as per the Gabion basket manufacturer's specifications, and City of Knoxville technical specifications. The Gabion baskets should be filled with clean, crushed limestone cobbles (particle size range 5" - 12"). The Gabion baskets should be set into place, with the baskets placed lengthwise back beneath the roadbed.

A drain should be placed behind the baskets to ensure no ponding of water occurs. The drain system should consist of a perforated pipe, surrounded by No. 57 sized stone, with stone and pipe encased in a non-woven drainage geotextile. The drainage pipe should be sloped to ensure that positive drainage is maintained. All backfill material placed behind the Gabion baskets should consist of size No. 57 stone.

## **9.2 Pavement Repair**

Once the Gabion baskets have been constructed up to finished subgrade elevation as outlined above, we recommend that the roadway undercut/backfilled area be covered with a high modulus geotextile, such as Propex Geotex 315ST. The geotextile should be placed and secured in tension to ensure that wrinkles and grabs are removed. Successive sheets should be lapped a minimum of 2 feet.

Once the geotextile has been placed and secured, the geotextile should be covered with 4 inches of ASTM C-33 No. 57 stone. The No. 57 stone should be compacted to the equivalent of a minimum of 95% Standard Proctor maximum dry density.

With the roadway prepared as outlined above, we recommend that the roadway asphalt pavement section be replaced utilizing the following section thicknesses.

**Table VII – Asphalt Pavement Section Thickness**

<b>Pavement Course</b>	<b>Thickness, inches</b>
<b>Tennessee Department of Transportation (TDOT) Specifications</b>	<b>Amherst Road</b>
Asphalt Surface Course – TDOT 411D	1.5”
Asphalt Base Course – TDOT 307BM	3.0”
Aggregate Base – TDOT 303D	8.0”

The aggregate base should consist of a crushed limestone meeting the requirement of the Tennessee Department of Transportation (TDOT) specification for Mineral Aggregate Base, Section 303, for Type “A” base, Class “A” aggregates, utilizing aggregate gradation “D”. The aggregate base should be compacted to a minimum of 100% of its maximum dry density as determined by the Standard Proctor test, ASTM D698.

The asphaltic base course should meet the specifications of TDOT, Section 307, Bituminous Plant Mix Base. The aggregates for the base course should meet the gradation requirements of Grading “B” Modified.

The asphalt surface course should meet the specifications of TDOT 411, Asphaltic Concrete Surface, the aggregates for the mixture meeting the requirements of Grading “D”.

The asphalt surface and base courses should be compacted to a minimum of 92% of their maximum theoretical density (MTD), ASTM D2041.

The materials and placement method for the aggregate base and asphaltic surface and base courses should meet the specifications of the Tennessee Department of Transportation (TDOT) Road and Bridge Specifications.

The new pavement should be sloped to taper/feather into the surrounding (existing) pavement such that the existing pavement slope and pavement drainage patterns are maintained. A slight crown in the pavement surface elevation over the center of the roadway is preferred; but, only if overall “existing” pavement drainage can be maintained. Under no circumstances should the repair area be left as a topographic “low” area, with water draining to and ponding in the repair area.

The guardrail may be reinstalled and anchored into the Gabion shoulder utilizing a formed (sono-tube) concrete base.

### 9.3 Erosion Control Measures

The remaining slope located south (down gradient) of the Gabion baskets and roadway should be protected from erosion. We recommend that the slope be protected utilizing a turf reinforcement mat, such as Propex, Inc. Landlok TRM 1060 or engineer-approved equal. Installation of the turf reinforcement mat should be performed in accordance with the manufacture's specifications. Fine grading should be performed on the down gradient slope to ensure that the slope is no steeper than 1H:1V prior to placement of the mat. The seed selected for growth on the on the slope should meet CSX Railway and City of Knoxville requirements.

## 10.0 ESTIMATED COSTS

Based on the results of the subsurface exploration, preliminary design work, and similar past projects, we offer the follow cost estimate breakdown for construction and quality control services in order to carry out the recommendations outlined within this report. This estimate does not include the costs of permits, flagmen, insurance, etc. that will be required by the CSX railway.

**Table VIII – Engineers Estimate**

<b>Service</b>	<b>Estimated Cost</b>
Construction	\$325,000.00
Quality Control Services	\$32,500.00
<b>Total</b>	<b>\$357,500.00</b>

## 11.0 ADDITIONAL RECOMMENDED WORK

Based on our familiarity with the site and the geotechnical considerations present, we believe that we are uniquely qualified to provide construction observation services during site grading and future building construction. We recommend that our firm be retained to provide quality control testing and observation services during construction.

## **12.0 GENERAL QUALIFICATIONS**

This report has been prepared for the exclusive use of the City of Knoxville. This report has been prepared in accordance with generally accepted geotechnical engineering practice. Any conclusions and recommendations contained in this report are based upon applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, expressed or implied, is made. Foundation Systems Engineering, P.C is not responsible for any claims, damages, or liability associated with any other party's interpretation of this report's subsurface data or reuse of this report's subsurface data or engineering analysis without our express written authorization, nor for the conclusions, opinions, or recommendations made by others based on these data.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from our site visit and the subsurface exploration. The nature and extent of variations between the borings will not become evident until repair/stabilization. If variations appear evident, then it will be necessary to re-evaluate the opinions and recommendations contained in this report. In the event that **any** change in the nature, design, location, evaluation, etc. of the roadway shoulder failure occurs, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing.

If any subsurface variations become evident, a reevaluation of the opinions contained in this report will be necessary after we have had an opportunity to observe the characteristics of the conditions encountered. Any and all variations from the above outlined recommendations are considered as "material" changes/variations, and the recommendations and opinions contained in this report will not be considered valid unless these changes/variations are reviewed and conclusions modified or verified in writing.

This report should not be made a part of project plans and specifications; but may be included with bidding documents for the convenience of the bidders.

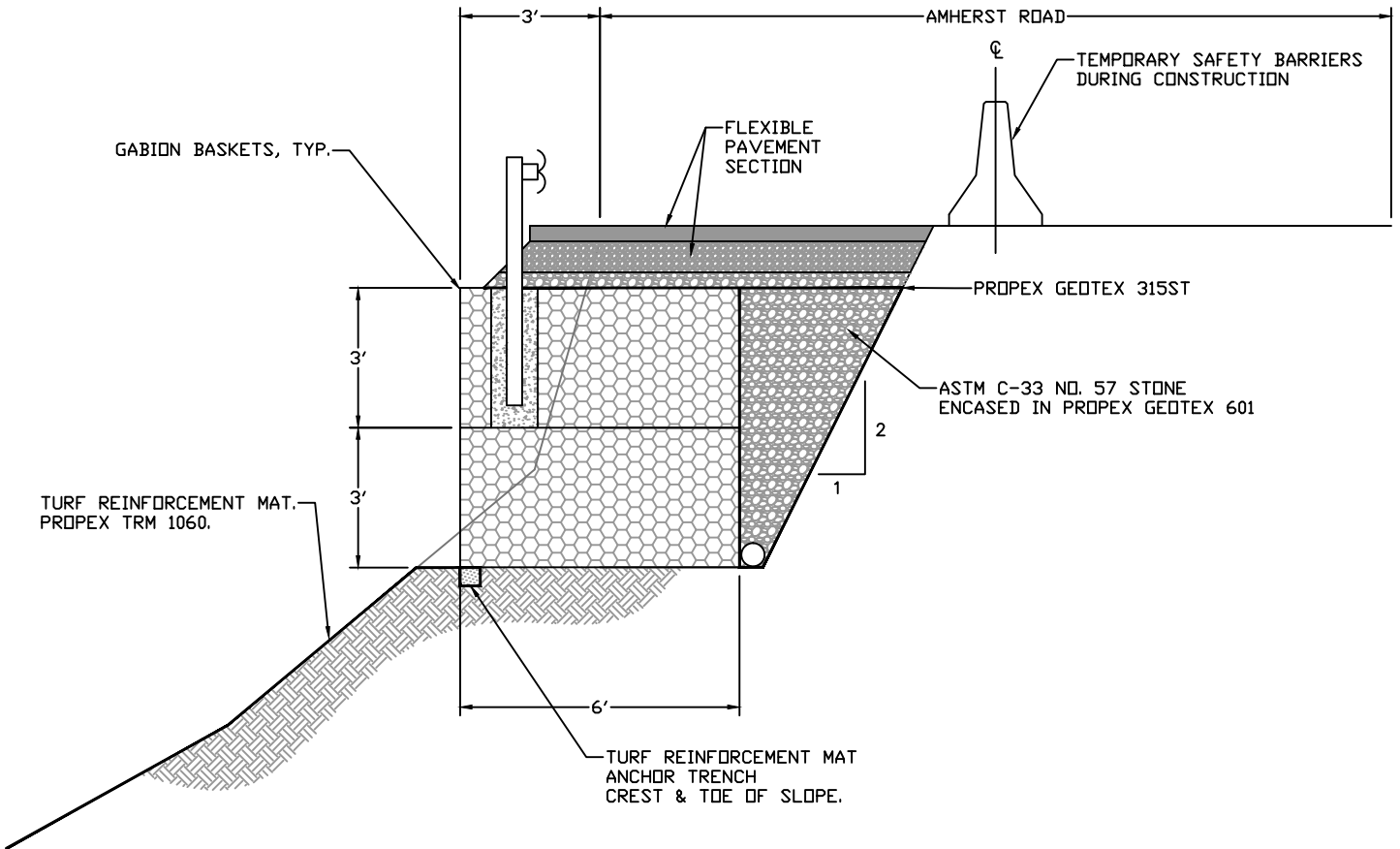
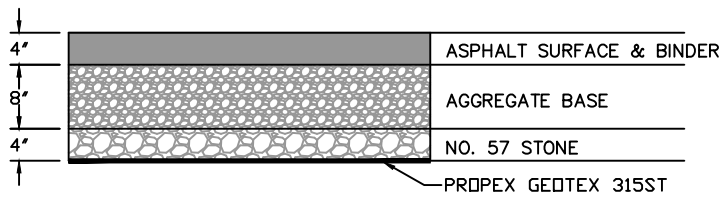
## **APPENDICIES**

- I. Typical Cross Section
- II. Slope Stability Analysis - Slide v. 5.0
- III. Gabion Wall Analysis - GawacWin 2003
- IV. Boring Location Plan & Auger Boring Records
- V. Laboratory Test Results
  - a. Atterberg Limits
  - b. Natural Moisture Content
  - c. Triaxial Shear Test
- VI. ASFE

## **APPENDIX I**

Typical Cross Section

FLEXIBLE PAVEMENT SECTION



TYPICAL CROSS SECTION



**FOUNDATION SYSTEMS  
ENGINEERING, P.C.**

Geotechnical Engineering and Consulting

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P.O. BOX 9449  
KNOXVILLE, TN 37940

865-577-3361  
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www.fsepc.com

TYPICAL CROSS SECTION  
AMHERST ROAD  
SHOULDER RESTORATION & STABILIZATION  
KNOXVILLE, TENNESSEE

FOR: CITY OF KNOXVILLE

DRAWN BY:  
EMP

NOTES: AS SHOWN

SCALE: NONE

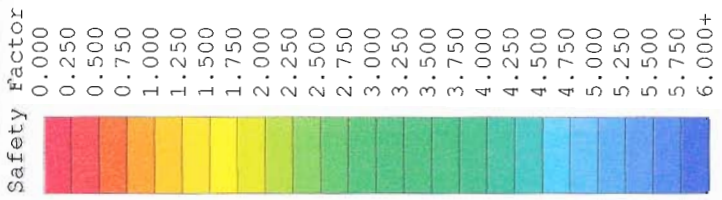
DATE: 05-05-2016

FSE #: 315152

## **APPENDIX II**

Slope Stability Analysis – Slide v. 5.0





Slope Stability Analysis - Critical Cross Section  
 Amherst Road  
 Knoxville, Tennessee

250.00 lb/ft<sup>2</sup>

Material Properties:  
 Material: A-2  
 Unit Weight: 120 lb/ft<sup>3</sup>  
 Cohesion: 1575 psf  
 Friction Angle: 16 degrees

Scale 1:100.0

50

40

30

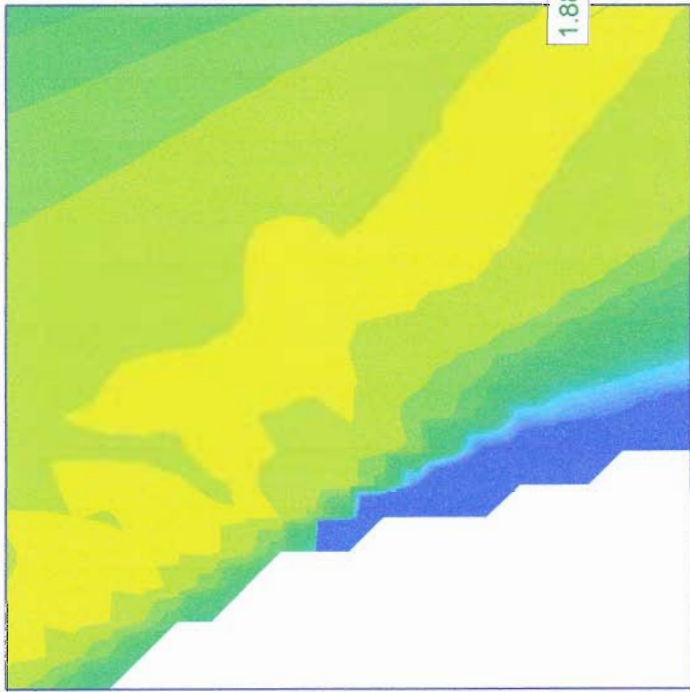
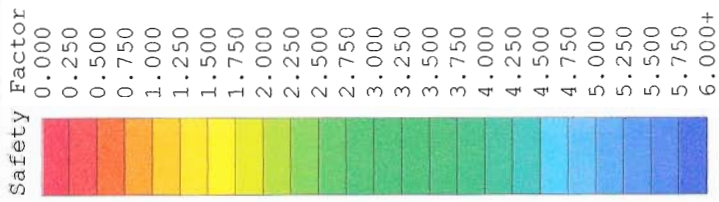
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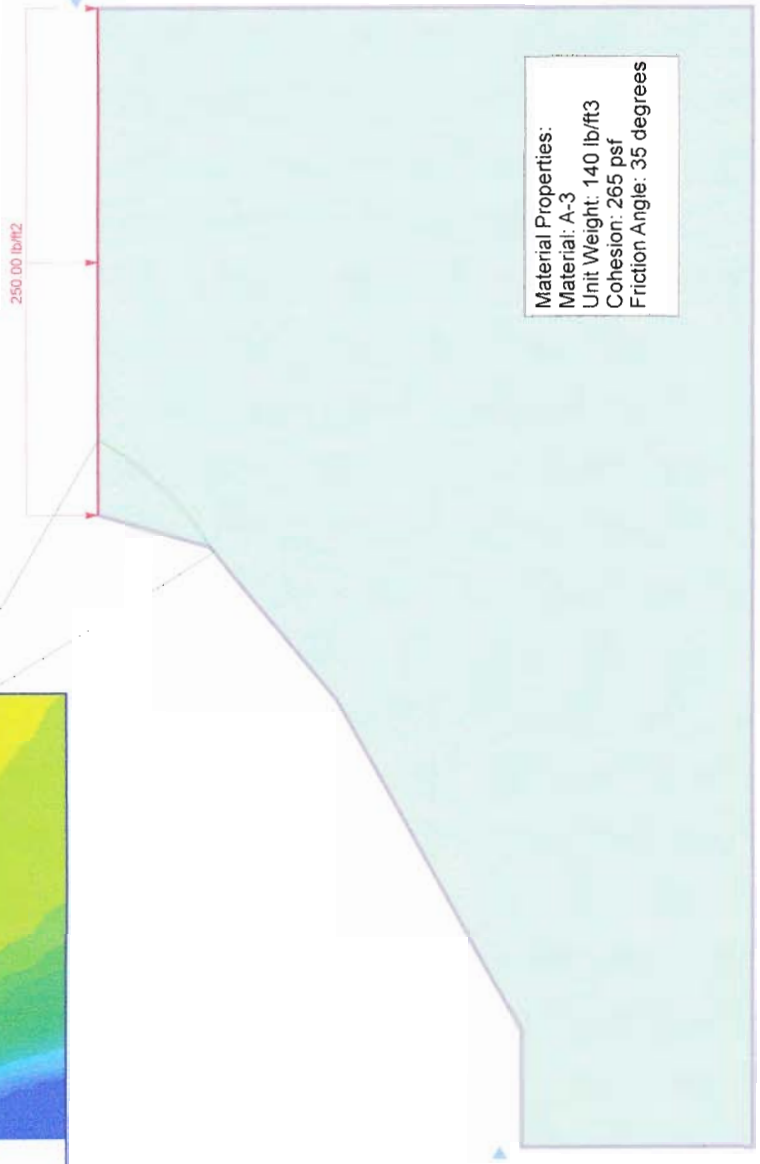
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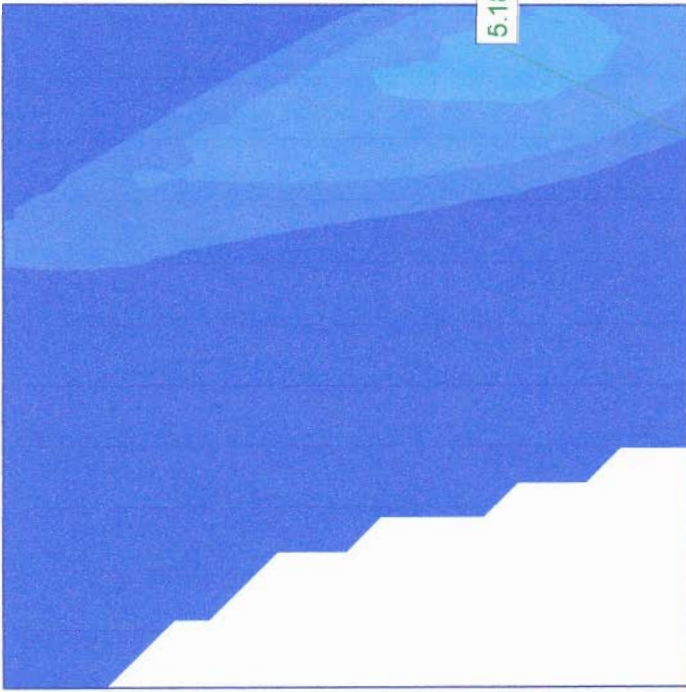
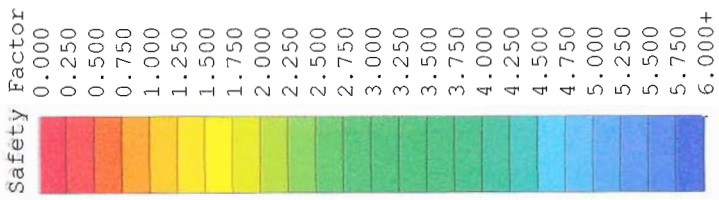
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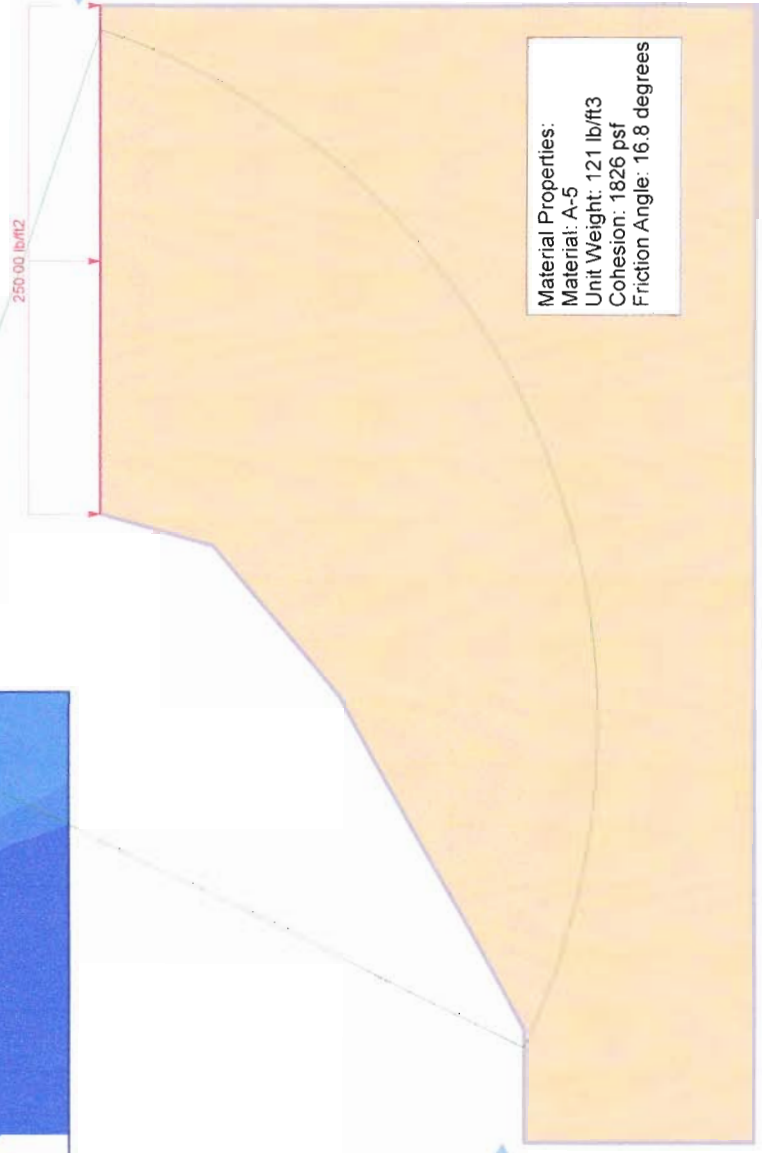
Slope Stability Analysis - Critical Cross Section  
 Amherst Road  
 Knoxville, Tennessee



Scale 1:100.0



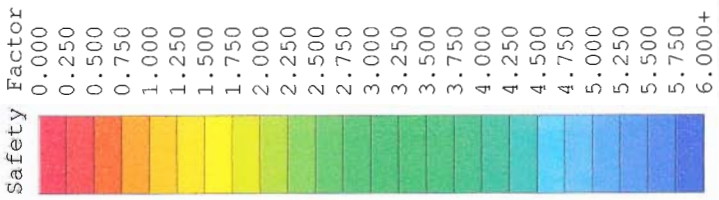
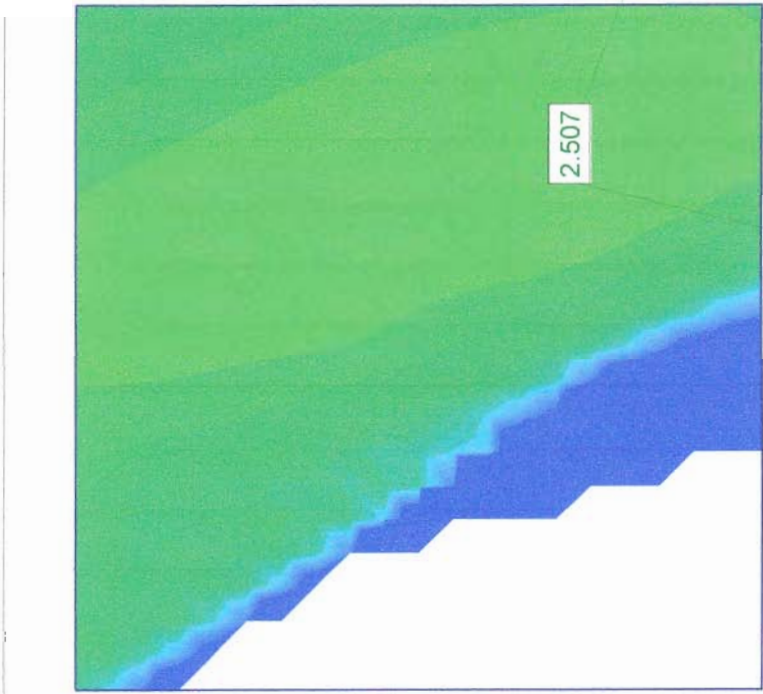
Slope Stability Analysis - Critical Cross Section  
 Amherst Road  
 Knoxville, Tennessee



Scale 1:100.0



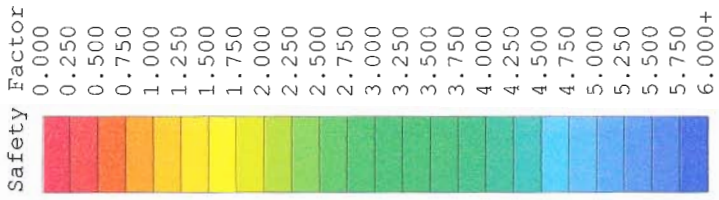
Slope Stability Analysis - Critical Cross Section  
Amherst Road  
Knoxville, Tennessee



Scale 1:100.0





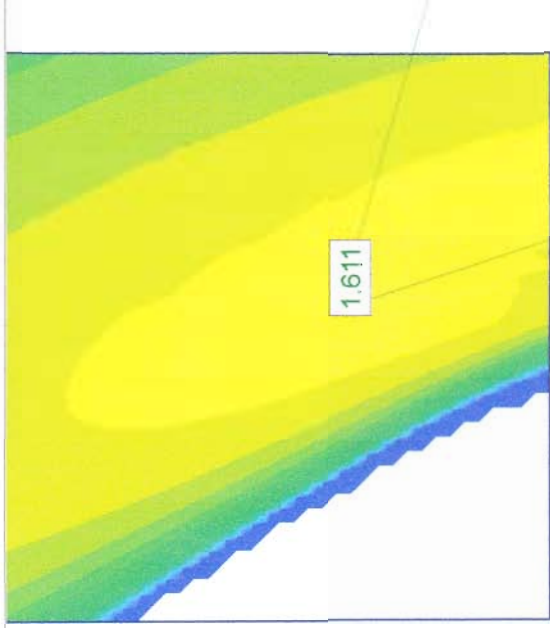
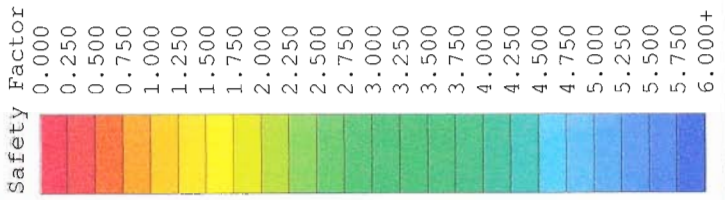


Slope Stability Analysis - Critical Cross Section  
 Amherst Road  
 Knoxville, Tennessee

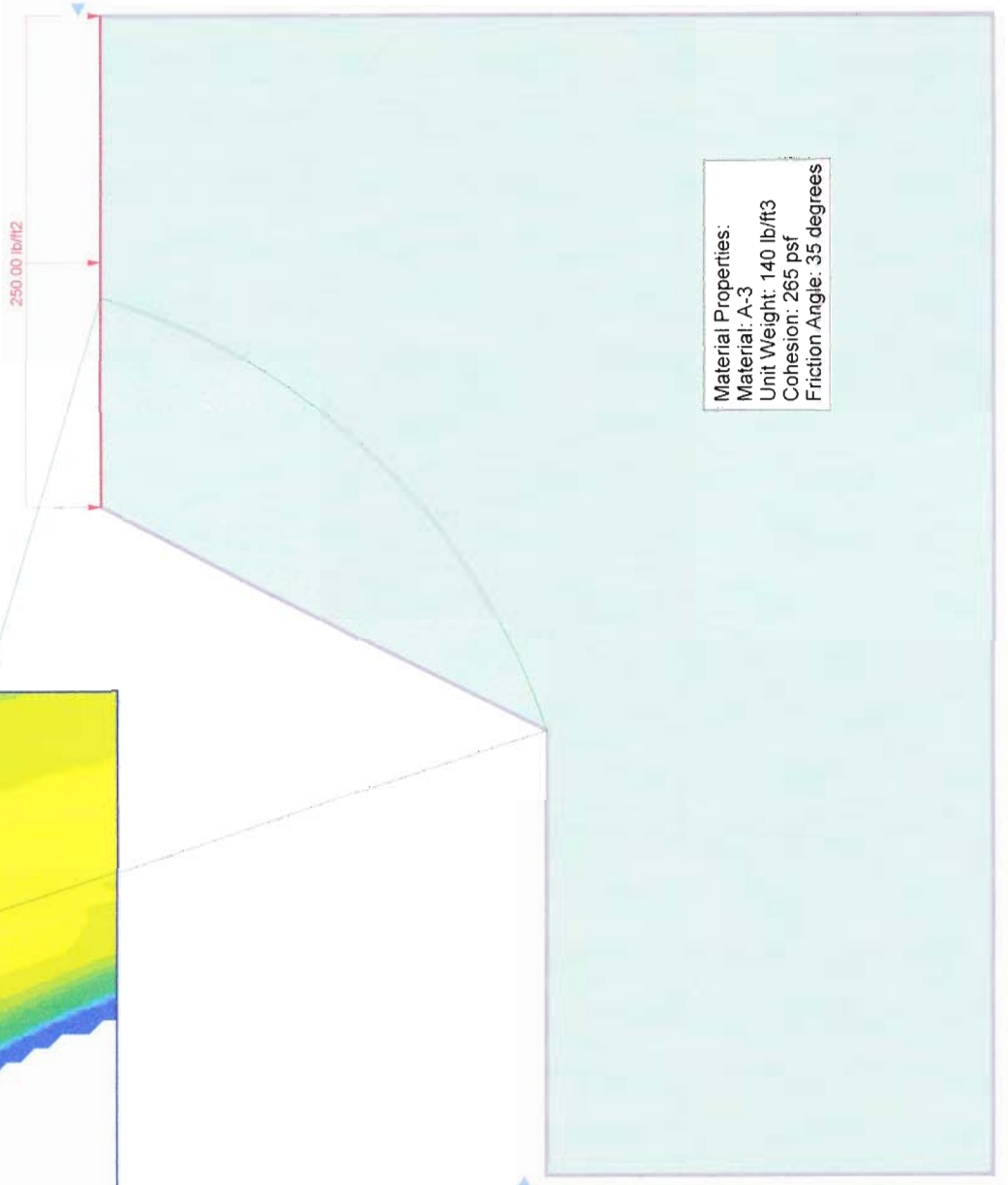


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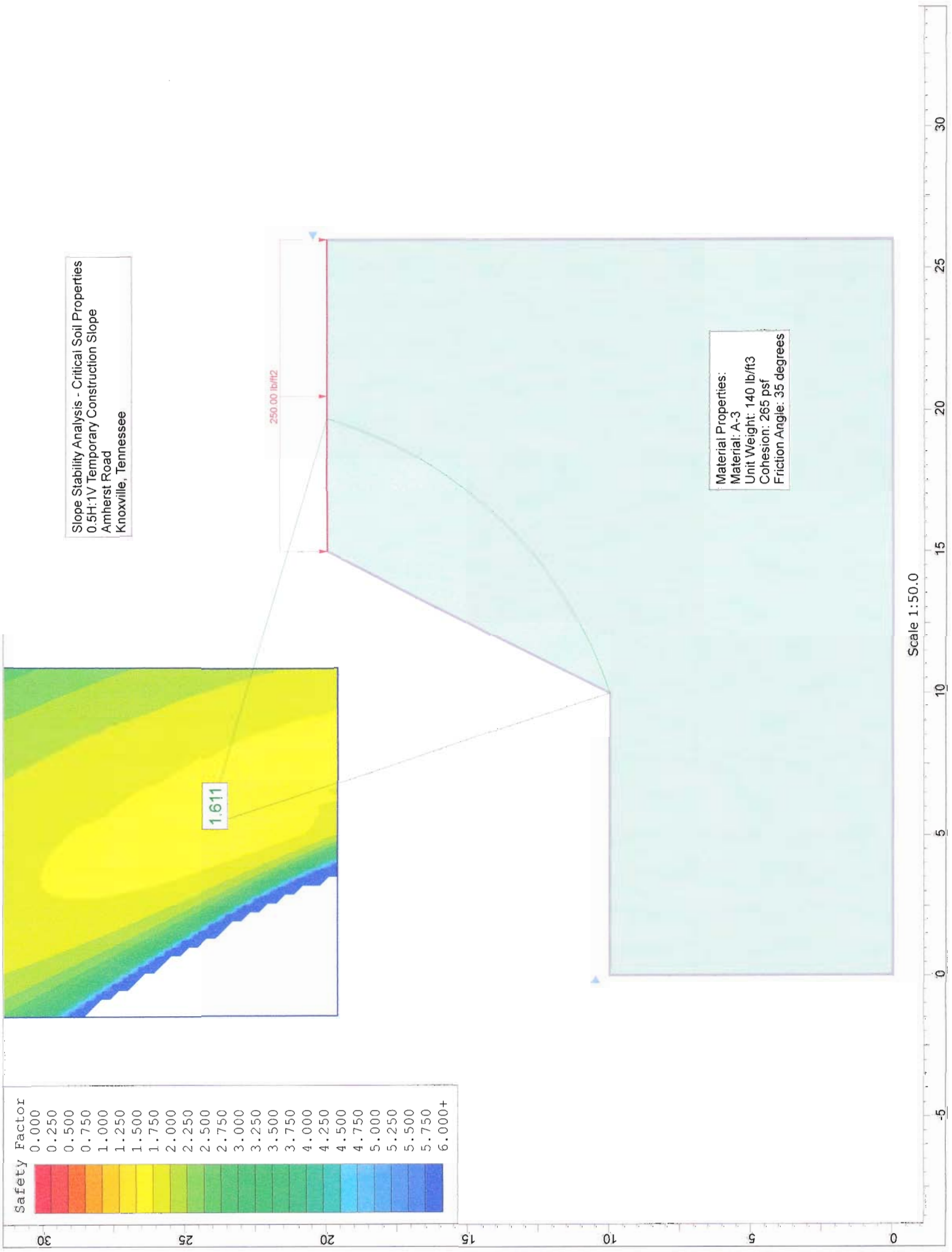


Slope Stability Analysis - Critical Soil Properties  
 0.5H:1V Temporary Construction Slope  
 Amherst Road  
 Knoxville, Tennessee



Material Properties:  
 Material: A-3  
 Unit Weight: 140 lb/ft<sup>3</sup>  
 Cohesion: 265 psf  
 Friction Angle: 35 degrees

Scale 1:50.0



## **APPENDIX III**

Gabion Wall Analysis – GawacWin 2003

Program released in license to: Eric Peterson

Project: Amherst

File: Amherst

Date: 5/10/2016

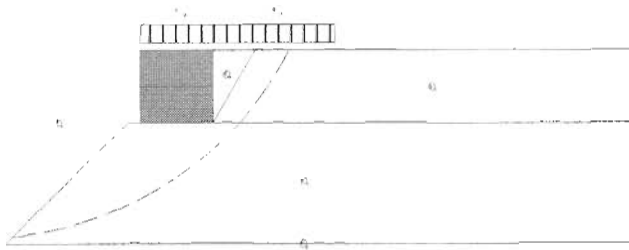
**INPUT DATA**

**Wall data**

Wall batter : 0.00 deg  
 Rockfill unit weight : 154.05 lb/ft<sup>3</sup>  
 Porosity of gabions : 30.00 %  
 Geotextile in the backfill : Yes  
 Friction reduction : 5.00 %  
 Geotextile on the base : Yes  
 Friction reduction : %  
 Mesh and the wire diam.: : 8x10, ø 2.70 mm

Layer	Length ft	Width ft	Offset ft
1	6.00	3.00	-
2	6.00	3.00	0.00

+



**Backfill soil data**

Inclination of Stretch 1 : 0.00 deg  
 Length of stretch 1 : 10.00 ft  
 Inclination of Stretch 2 : 0.00 deg  
 Soil unit weight : 100.00 lb/ft<sup>3</sup>  
 Soil friction angle : 38.00 deg  
 Soil cohesion : 0.00 lb/ft<sup>2</sup>

**Additional Backfill Layers**

Layer	Initial height ft	Incl. angle deg	Unit weight lb/ft <sup>3</sup>	Cohesion lb/ft <sup>2</sup>	Friction angle deg
1	0.00	60.00	140.00	265.00	35.00



Program released in license to: Eric Peterson

Project: Amherst

File: Amherst

Date: 5/10/2016

**Foundation data**

Top surface height : 0.00 ft  
 Top surface init. length : 1.00 ft  
 Top surface incl. angle : 45.00 deg  
 Soil unit weight : 120.00 lb/ft<sup>3</sup>  
 Soil friction angle : 16.00 deg  
 Soil cohesion : 500.00 lb/ft<sup>2</sup>  
 Foundation allowable pressure : lb/ft<sup>2</sup>  
 Water table height : ft

Additional Foundation Layers

Layer	Depth ft	Unit weight lb/ft <sup>3</sup>	Cohesion lb/ft <sup>2</sup>	Friction angle deg
1	0.00	140.00	265.00	35.00
2	10.00	150.00	20000.00	35.00

**Water profile data**

Initial height : ft  
 Inclination of the 1st stretch : deg  
 Length of the 1st stretch : ft  
 Inclination of the 2nd stretch : deg  
 Length of the 2nd stretch : ft

**Loads data**

Distributed loads on backfill  
 First stretch : 250.00 lb/ft<sup>2</sup>  
 Second stretch : lb/ft<sup>2</sup>

Distributed loads on wall  
 Load : 250.00 lb/ft<sup>2</sup>

Line loads on backfill  
 Load 1 : lb/ft Distance from wall face : ft  
 Load 2 : lb/ft Distance from wall face : ft  
 Load 3 : lb/ft Distance from wall face : ft

Line load on wall  
 Load : lb/ft Distance from wall face : ft

**Seismic action data**

Horizontal coefficient : Vertical coefficient :

---

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---

Project: Amherst

File: Amherst

Date: 5/10/2016

---

## STABILITY ANALYSIS RESULTS

### Active and Passive Thrust

Active Thrust	:	572.15 lb/ft
Point of application ref. to X axis	:	6.00 ft
Point of application ref. to Y axis	:	2.30 ft
Direction of the thrust ref. to X axis	:	36.10 deg
Passive Thrust	:	0.00 lb/ft
Point of application ref. to X axis	:	0.00 ft
Point of application ref. to Y axis	:	0.00 ft
Direction of the thrust ref. to X axis	:	0.00 deg

### Sliding

Normal force on the base	:	5719.17 lb/ft
Point of application ref. to X axis	:	2.99 ft
Point of application ref. to Y axis	:	0.00 ft
Shear force on the base	:	462.29 lb/ft
Resisting force on the base	:	4799.60 lb/ft

**Sliding Safety Coefficient** : **10.38**

### Overturning

Overturning Moment	:	1062.40 lb/ft x ft
Restoring Moment	:	18168.82 lb/ft x ft

**Overturning Safety Coefficient** : **17.10**

### Stresses Acting on Foundation

Eccentricity	:	0.01 ft
Normal stress on outer border	:	961.71 lb/ft <sup>2</sup>
Normal stress on inner border	:	944.68 lb/ft <sup>2</sup>
Max. allowable stress on the foundation	:	8572.02 lb/ft <sup>2</sup>

---

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---

Project: Amherst

File: Amherst

Date: 5/10/2016

**Overall Stability**

Initial distance at pivot leftside : ft  
 Initial distance at pivot rightside : ft  
 Initial depth referred to base : ft  
 Max depth allowed in calculation : ft  
 Center of the arch referred to X axis : -12.95 ft  
 Center of the arch referred to Y axis : 18.60 ft  
 Radius of the arch : 28.19 ft  
 Number of search surfaces : 286  
**Overall Stability Safety Coefficient : 1.80**

**Internal Stability**


---

Layer	H ft	N lb/ft	T lb/ft	M lb/ft x ft	$\tau$ Max lb/ft <sup>2</sup>	$\tau$ All lb/ft <sup>2</sup>	$\sigma$ Max lb/ft <sup>2</sup>	$\sigma$ All lb/ft <sup>2</sup>
1	3.00	3441.03	0.00	10323.09	0.00	788.80	573.50	

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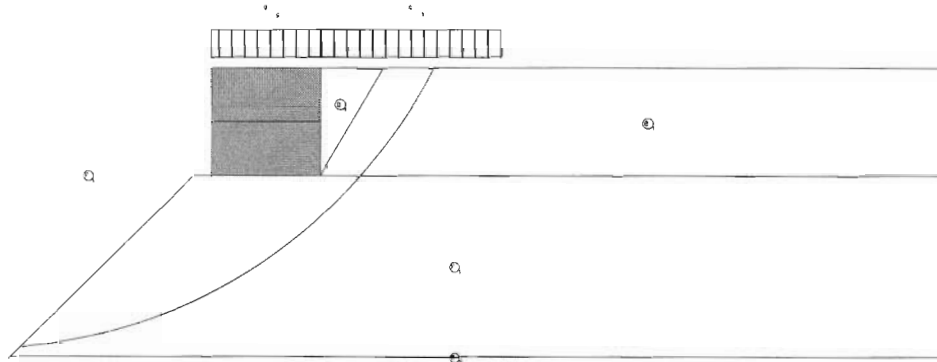
Program released in license to: Eric Peterson

Project: Amherst

File: Amherst

Date: 5/10/2016

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**SOIL DATA**

Soil	$\gamma$ lb/ft <sup>3</sup>	c lb/ft <sup>2</sup>	$\phi$ deg	Soil	$\gamma$ lb/ft <sup>3</sup>	c lb/ft <sup>2</sup>	$\phi$ deg
B <sub>s</sub>	100.00	0.00	38.00	F <sub>s</sub>	120.00	500.00	16.00
B <sub>1</sub>	140.00	265.00	35.00	F <sub>1</sub>	140.00	265.00	35.00
				F <sub>2</sub>	150.00	20000.00	35.00

**LOADS**

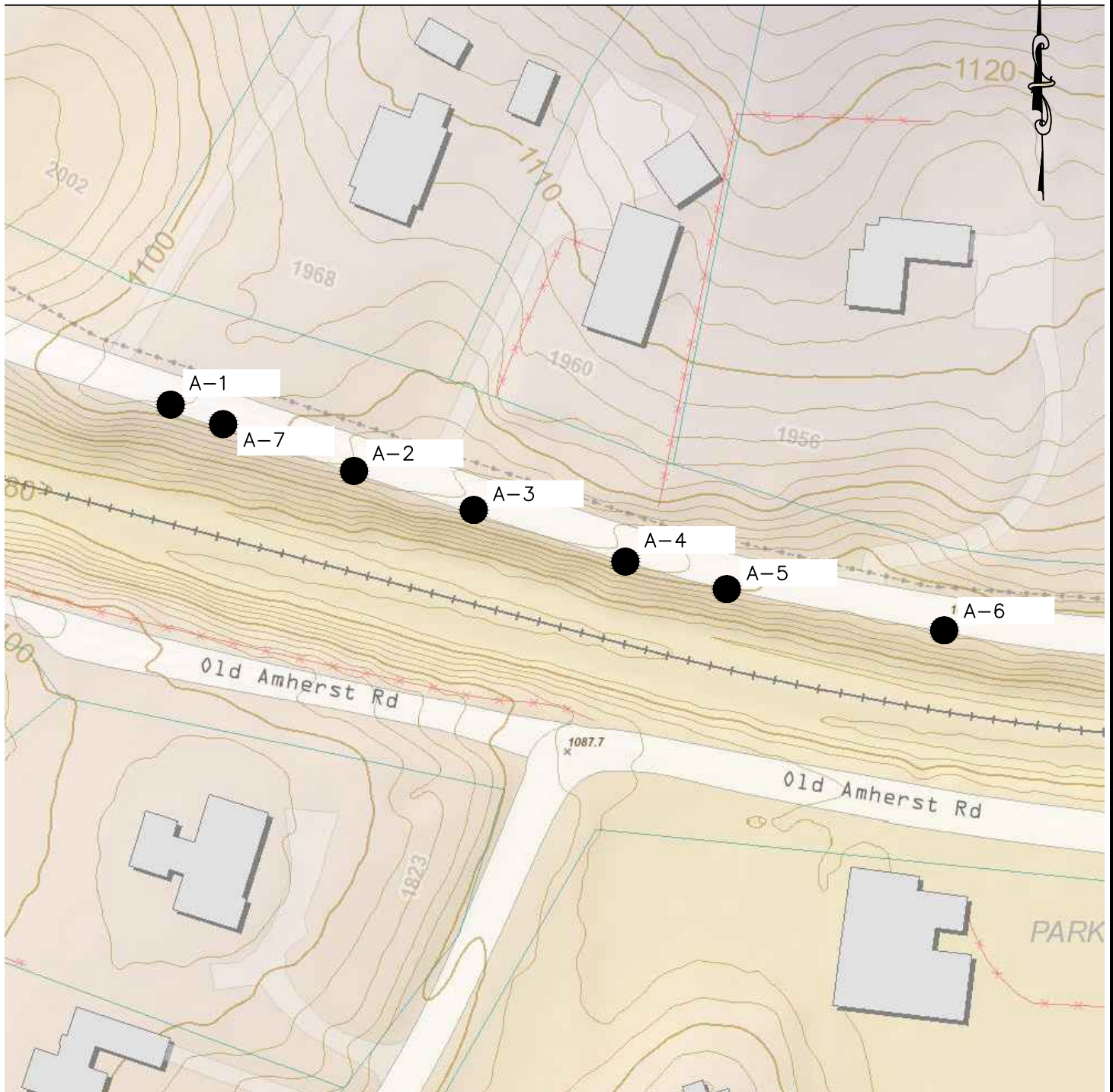
Load	Value lb/ft <sup>2</sup>	Load	Value lb/ft
q <sub>g</sub>	250.00		
q <sub>1</sub>	250.00		

**STABILITY CHECKS**

Sliding Safety Coefficient	10.38	Base normal stress (left)	961.71lb/ft <sup>2</sup>
Overturing Safety Coefficient	17.10	Base normal stress (right)	944.68lb/ft <sup>2</sup>
Overall Stability Safety Coefficient	1.80	Max. allowable stress	8572.02lb/ft <sup>2</sup>

## **APPENDIX IV**

Boring Location Plan & Auger Boring Records



**FOUNDATION SYSTEMS  
ENGINEERING, P.C.**

Geotechnical Engineering and Consulting

2203 ATCHLEY STREET  
P.O. BOX 9449  
KNOXVILLE, TN 37940

865-577-3361  
FAX 573-1817  
www.fsepc.com

**BORING LOCATION PLAN  
AMHERST ROAD  
SHOULDER RESTORATION & STABILIZATION  
KNOXVILLE, TENNESSEE**

FOR: CITY OF KNOXVILLE

DRAWN BY:  
EMP

NOTES: LOCATIONS ARE APPROXIMATE.  
ADAPTED FROM KGIS MAPPING.

SCALE: NONE

DATE: 05-06-2016

FSE # 315152

# FOUNDATION SYSTEMS ENGINEERING, P.C.

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 1427 Lakeside Lane  
 Kingsport, Tennessee 37663  
 (423) 239-9226

## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
1100	0.0	ASPHALT PAVEMENT (4").					N VALUES FROM DYNAMIC CONE PENETROMETER TESTING, ASTM STP 399.
1099.67	0.33	ALTERNATING LAYERS OF AGGREGATE BASE AND ASPHALT PAVEMENT (20").					
1098	2.0	STIFF TO VERY STIFF, MOIST, TAN, REDDISH TAN AND REDDISH BROWN, SILTY CLAY MIXED WITH CHERT (RESIDUAL).	2.5	8-10-11			
			6.0	13-17- 20			
1092.5	7.5	AUGER REFUSAL AT 7.5.  NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.					BORE HOLE BACKFILLED WITH CONCRETE UPON COMPLETION.

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

CR - 15 % CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

70  UNDISTURBED SAMPLE (ASTM D-1587)

100  WATER TABLE, TIME OF BORING

 WATER TABLE, 24 HOUR READING

 LOSS OF DRILLING FLUID

BORING NUMBER:     A-1    

DATE DRILLED:     1-12-16    

FSE NO.:     315152    

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**CITY OF KNOXVILLE  
 AMHERST ROAD  
 KNOXVILLE, TENNESSEE**

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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
1100	0.0	ASPHALT PAVEMENT (4.5").					N VALUES FROM DYNAMIC CONE PENETROMETER TESTING.
1099.62	0.38	LIMESTONE GRAVEL MIXED WITH					
1099	1.0	MOIST, BROWN, SILTY CLAY (FILL).					
		VERY STIFF, MOIST, TAN, REDDISH TAN, AND REDDISH BROWN, SILTY CLAY (RESIDUAL).	3.0	12-16- 20			
			5.0	14-17- 22			
			7.5	13-19- 26			
1092	8.0	VERY STIFF TO HARD, MOIST, TAN AND REDDISH BROWN, SILTY CLAY.	10.0	18-27- 46			
			15.0	16-25- 31			
1084	16.0	VERY STIFF, MOIST, TAN, REDDISH TAN, AND REDDISH BROWN, SILTY CLAY.	18.0	15-21- 33			
			20.0	17-19-			

Continued on page 2

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

CR - 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

5			
13	70		UNDISTURBED SAMPLE (ASTM D-1587)
20	100		WATER TABLE, TIME OF BORING
			WATER TABLE, 24 HOUR READING
			LOSS OF DRILLING FLUID

BORING NUMBER: A-2

DATE DRILLED: 1-12-16

FSE NO.: 315152

PAGE 1 OF 2

CITY OF KNOXVILLE  
 AMHERST ROAD  
 KNOXVILLE, TENNESSEE



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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
				25			
			24.0	14-18- 21			
			25.0	16-16- 20			
1070	30.0	BORING TERMINATED AT 30 FEET,  NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.	30.0	13-17- 21			BORE HOLE BACKFILLED WITH CONCRETE UPON COMPLETION.

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

CR - 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

 UNDISTURBED SAMPLE (ASTM D-1587)

 WATER TABLE, TIME OF BORING

 WATER TABLE, 24 HOUR READING

 LOSS OF DRILLING FLUID

BORING NUMBER: A-2

DATE DRILLED: 1-12-16

FSE NO.: 315152

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CITY OF KNOXVILLE  
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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
1095	0.0	ASPHALT PAVEMENT (4").					N VALUES FROM DYNAMIC CONE PENETROMETER TESTING, ASTM STP 399.
1094.67	0.33	LIMESTONE GRAVEL MIXED WITH BROKEN ASPHALT AND TOPSOIL.					
1093.5	1.5	FIRM TO STIFF, MOIST, TAN AND BROWN, SILTY CLAY (FILL).	4.0	5-6-6			
			5.0	7-8-11			
			8.0	6-9-13			
1086	9.0	VERY STIFF, MOIST, TAN AND REDDISH BROWN, SILTY CLAY (RESIDUAL).	10.0	14-17- 20			
1082	13.0	AUGER REFUSAL AT 13 FEET.  NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.					BORE HOLE BACKFILLED WITH CONCRETE UPON COMPLETION.

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

CR - 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

 UNDISTURBED SAMPLE (ASTM D-1587)

 WATER TABLE, TIME OF BORING

 WATER TABLE, 24 HOUR READING

 LOSS OF DRILLING FLUID

BORING NUMBER:     A-3    

DATE DRILLED:     1-12-16    

FSE NO.:     315152    

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 KNOXVILLE, TENNESSEE**

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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
1090	0.0	ASPHALT PAVEMENT (4").					N VALUES FROM DYNAMIC CONE PENETROMETER TESTING, ASTM STP 399.
1089.67	0.33	LIMESTONE AGGREGATE BASE					
1089.25	0.75	(5").					
1088.5	1.5	LIMESTONE GRAVEL MIXED WITH GRAY AND BLACK SILTY CLAY (FILL).  STIFF, VERY MOIST TO MOIST, TAN AND REDDISH BROWN, SILTY CLAY MIXED WITH CHERT (FILL).	4.0	14-17- 19			
			5.0	13-15- 21			
			7.5	15-19- 18			
			11.0	19-22- 29			
1075	15.0	STIFF, MOIST, TANNISH GRAY, TAN, AND REDDISH TAN, SILTY CLAY MIXED WITH CHERT.	17.0	17-20- 21			

Continued on page 2

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)


CR - 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

 UNDISTURBED SAMPLE (ASTM D-1587)

 WATER TABLE, TIME OF BORING

 WATER TABLE, 24 HOUR READING

 LOSS OF DRILLING FLUID

BORING NUMBER: A-4

DATE DRILLED: 1-11-16

FSE NO.: 315152

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CITY OF KNOXVILLE  
 AMHERST ROAD  
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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
			22.0	29-43*			* N VALUE ELEVATED DUE TO CHERT.
			25.0	23-39*			
1064	26.0	VERY STIFF, MOIST, REDDISH BROWN SILTY CLAY MIXED WITH FINE TO COARSE CHERT.					
1060	30.0	BORING TERMINATED AT 30 FEET.  NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.	30.0	47*			BORE HOLE BACKFILLED WITH CONCRETE UPON COMPLETION.


N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

CR - 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

70  UNDISTURBED SAMPLE (ASTM D-1587)

100  WATER TABLE, TIME OF BORING

 WATER TABLE, 24 HOUR READING

 LOSS OF DRILLING FLUID

BORING NUMBER:     A-4    

DATE DRILLED:     1-11-16    

FSE NO.:     315152    

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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
1089	0.0	ASPHALT PAVEMENT (4.5").					N VALUES FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399).
1088.62	0.38	LIMESTONE GRAVEL MIXED WITH MOIST TO VERY MOIST, GRAY AND BLACK, SILTY CLAY (FILL).					
1088	1.0	STIFF TO VERY STIFF, MOIST, TAN, REDDISH TAN AND REDDISH BROWN, SILTY CLAY MIXED WITH FINE TO COARSE CHERT (RESIDUAL).	2.5	8-17-20			
			6.0	16-21- 26			
			7.5	27-35*			* N VALUES ELEVATED DUE TO CHERT.
			12.0	31-47*			

Continued on page 2

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

5	CR - 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)
13	RQD - ROCK QUALITY
20	S - SYMBOLS DESCRIBED BELOW:
70	UNDISTURBED SAMPLE (ASTM D-1587)
100	WATER TABLE, TIME OF BORING
	WATER TABLE, 24 HOUR READING
	LOSS OF DRILLING FLUID

BORING NUMBER: A-5

DATE DRILLED: 1-11-16

FSE NO.: 315152

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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
1071	18.0	VERY STIFF, MOIST, REDDISH BROWN, SILTY CLAY MIXED WITH CHERT.	20.0	29-37*			
1064	25.0	BORING TERMINATED AT 25.0 FEET.  NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.	25.0	21-27-26			BORE HOLE BACKFILLED WITH SOIL AUGER CUTTINGS UPON COMPLETION.

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

CR - 15 % CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

70  UNDISTURBED SAMPLE (ASTM D-1587)

100  WATER TABLE, TIME OF BORING

 WATER TABLE, 24 HOUR READING

 LOSS OF DRILLING FLUID

BORING NUMBER:     A-5    

DATE DRILLED:     1-11-16    

FSE NO.:     315152    

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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
1090	0.0	ASPHALT PAVEMENT (4") .					N VALUES FROM DYNAMIC CONE PENETROMETER TESTING, ASTM STP 399. NO SAMPLE RECOVERY.
1089.67	0.33	VERY SOFT, VERY MOIST, DARK GRAY AND BLACK, ORGANIC, SILTY CLAY MIXED WITH LIMESTONE GRAVEL (FILL)	2.0	1-1-1			
1085	5.0	FIRM, VERY MOIST, BROWN AND GRAY, SILTY CLAY (FILL) .	5.0	4-6-6			
1084	6.0	SOFT TO FIRM, VERY MOIST, GRAY SILTY CLAY AND CLAYEY SILT (FILL) .	8.0	8-3-5			
			10.0	1-1-6			
1076	14.0	FIRM, VERY MOIST, GRAY, SILTY CLAY MIXED WITH FINE TO COARSE CHERT (FILL) .					





Continued on page 2

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

CR - 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

- 70  UNDISTURBED SAMPLE (ASTM D-1587)
- 100  WATER TABLE, TIME OF BORING
-  WATER TABLE, 24 HOUR READING
-  LOSS OF DRILLING FLUID

BORING NUMBER:     A-6    

DATE DRILLED:     1-11-16    

FSE NO.:     315152    

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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
			20.0	10-11-13			N VALUE ELEVATED DUE TO CHERT ROCK.
1068	22.0	STIFF, VERY MOIST, TANNISH GRAY AND LIGHT GRAY, SANDY, CLAYEY SILT MIXED WITH FINE TO COARSE CHERT (RESIDUAL?) .					
1065	25.0	BORING TERMINATED AT 25 FEET.  NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.	25.0	11-11-12			BORE HOLE BACKFILLED WITH CONCRETE UPON COMPLETION.

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

CR - 15 % CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

5		
13	70	UNDISTURBED SAMPLE (ASTM D-1587)
20	100	WATER TABLE, TIME OF BORING
		WATER TABLE, 24 HOUR READING
		LOSS OF DRILLING FLUID

BORING NUMBER:     A-6    

DATE DRILLED:     1-11-16    

FSE NO.:     315152    

PAGE     2     OF     2    

**CITY OF KNOXVILLE  
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 KNOXVILLE, TENNESSEE**



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## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
1102	0.0	ASPHALT PAVEMENT (4").					N VALUES FROM DYNAMIC CONE PENETROMETER TESTING, ASTM STP 399.
1101.67	0.33	ALTERNATING LAYERS OF AGGREGATE BASE AND ASPHALT PAVEMENT (14").					
1100.5	1.5	FIRM TO VERY STIFF, MOIST, TAN, REDDISH TAN, AND REDDISH BROWN, SILTY CLAY (RESIDUAL) .	2.5	7-7-11			
			5.0	6-9-10			
			7.5	11-14- 17			
			10.0	13-16- 19			
1090	12.0	VERY STIFF, MOIST, TAN AND REDDISH BROWN, SILTY CLAY, MIXED WITH CHERT.					
			15.0	19-27- 39			

Continued on page 2

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)

CR - 15 % CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)

RQD - ROCK QUALITY

S - SYMBOLS DESCRIBED BELOW:

 UNDISTURBED SAMPLE (ASTM D-1587)

 WATER TABLE, TIME OF BORING

 WATER TABLE, 24 HOUR READING

 LOSS OF DRILLING FLUID

BORING NUMBER:     A-7    

DATE DRILLED:     1-12-16    

FSE NO.:     315152    

PAGE     1     OF     2    

**CITY OF KNOXVILLE  
 AMHERST ROAD  
 KNOXVILLE, TENNESSEE**

# FOUNDATION SYSTEMS ENGINEERING, P.C.

2203 Atchley Street  
 P.O. Box 9449  
 Knoxville, Tennessee 37920

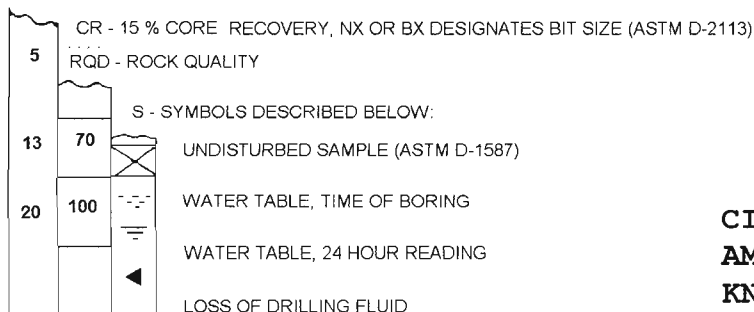
(865) 577-3361  
 FAX 573-1817  
 www.fsepc.com

Branch  
 1427 Lakeside Lane  
 Kingsport, Tennessee 37663  
 (423) 239-9226

## AUGER BORING RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	REMARKS
			20.0	37-40= 0"			N VALUE ELEVATED DUE TO CHERT GRAVEL TO COBBLES.
1077	25.0	STIFF, MOIST, REDDISH BROWN AND TAN, SILTY CLAY (RESIDUAL) .	27.0	10-8-12			
1072	30.0	BORING TERMINATED AT 30 FEET.  NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.	30.0	11-10-10			BORE HOLE BACKFILLED WITH CONCRETE UPON COMPLETION.

N - IS THE PENETRATION IN BLOWS PER FOOT (ASTM D-1586)



BORING NUMBER: A-7

DATE DRILLED: 1-12-16

FSE NO.: 315152

PAGE 2 OF 2

CITY OF KNOXVILLE  
 AMHERST ROAD  
 KNOXVILLE, TENNESSEE

## **APPENDIX V**

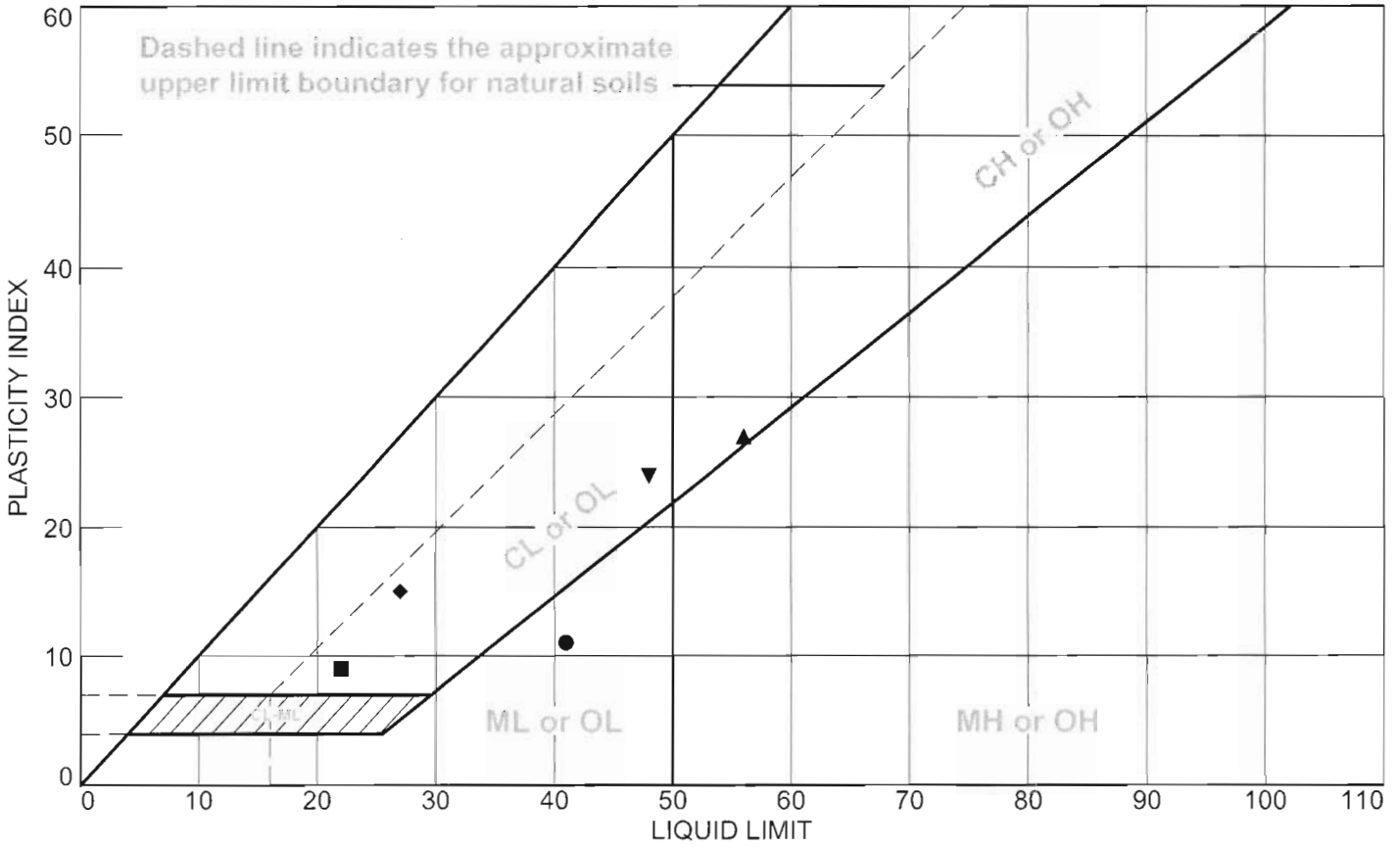
Laboratory Test Results

Atterberg Limits

Natural Moisture Content

Triaxial Shear Test

# LIQUID AND PLASTIC LIMITS TEST REPORT



## SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	A-2		22.0-24.0'	32.7	30	41	11	ML
■	A-3		2.0-4.0'	12.2	13	22	9	CL
▲	A-5		18.0-20.0'	29.9	29	56	27	CH
◆	A-6		6.0-8.0'	16.9	12	27	15	CL
▼	A-7		13.0-15.0'	31.6	24	48	24	CL

**Foundation Systems Engineering**

**Knoxville, Tennessee**

**Client:** Foundation Systems Engineering P.C., Inc.

**Project:** City of Knoxville-Amherst Road

**Project No.:** 315152

**Figure**

Llewellyn & Associates, Inc. dba

# Construction Materials Laboratory

2204 Atchley Street  
Knoxville, Tn. 37920  
865-572-6031/-573-6122

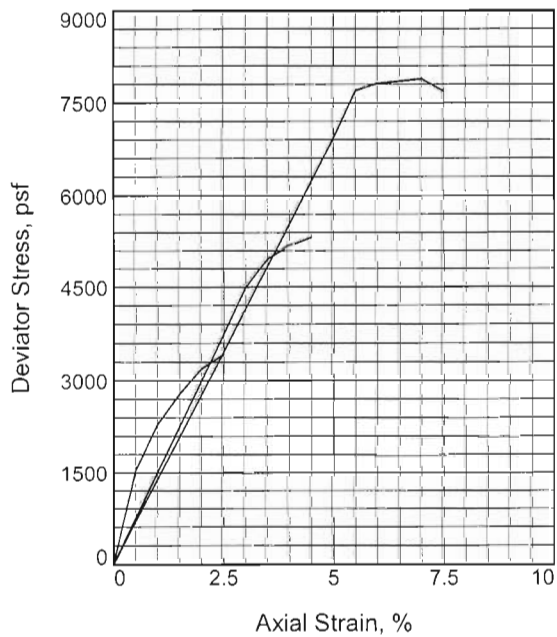
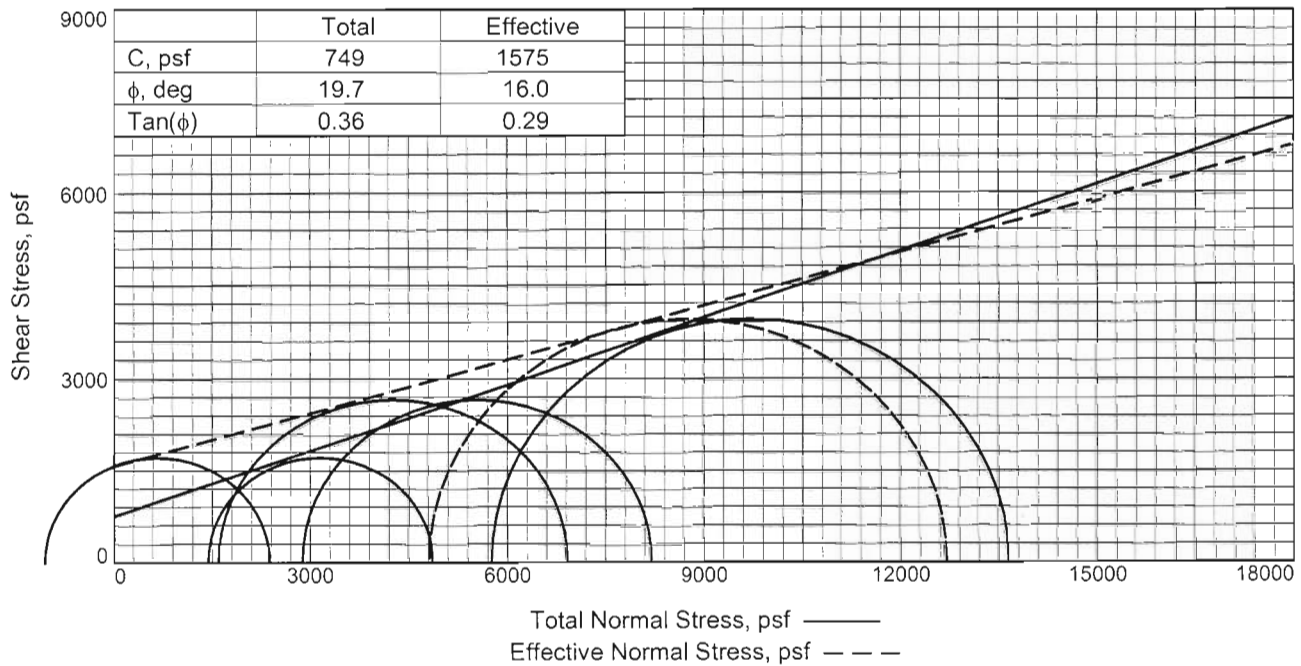
P. O. Box 5186  
Johnson City, Tn. 37603  
423-926-0762/926-3860

**Project:** City of Knoxville-Amherst Road  
**Client:** Foundation Systems Engineering, P.C.  
**Report to:** Foundation Systems Engineering, P.C.  
**Lab Number:** K-16264  
**FSE Job Number:** 315152

## Natural Moisture Contents

**Date Tested:** 2/3/2016

Boring Number	Depth	Natural Moisture Content %
A-1	4.0-6.0'	22.2
A-2	8.0-10.0'	31.3
A-2	16.0-18.0'	33.0
A-3	11.0-13.0'	11.4
A-4	2.0-4.0'	15.1
A-4	9.0-11.0'	18.2
A-4	15.0-17.0'	16.2
A-5	4.0-6.0'	13.4
A-5	10.0-12.0'	14.2
A-6	22.0-24.0'	17.0
A-7	8.0-10.0'	34.8
A-7	25.0-27.0'	32.0



Sample No.	1	2	3
<b>Initial</b>			
Water Content, %	32.7	32.7	32.7
Dry Density, pcf	89.5	89.5	89.5
Saturation, %	100.0	100.0	100.0
Void Ratio	0.8841	0.8841	0.8841
Diameter, in.	1.85	1.85	1.85
Height, in.	3.70	3.70	3.70
<b>At Test</b>			
Water Content, %	32.7	32.7	32.7
Dry Density, pcf	89.5	89.5	89.5
Saturation, %	100.0	100.0	100.0
Void Ratio	0.8841	0.8841	0.8841
Diameter, in.	1.85	1.85	1.85
Height, in.	3.70	3.70	3.70
Strain rate, in./min.	1.00	1.00	1.00
Back Pressure, psi	80.00	80.00	80.00
Cell Pressure, psi	90.00	100.00	120.00
Fail. Stress, psf	3426	5321	7892
Total Pore Pr., psf	14011	12802	12470
Ult. Stress, psf			
Total Pore Pr., psf			
$\bar{\sigma}_1$ Failure, psf	2375	6919	12701
$\bar{\sigma}_3$ Failure, psf	-1051	1598	4810

**Type of Test:**

CU with Pore Pressures

**Sample Type:** UNDISTURBED

**Description:** TAN & REDDISH BROWN, CLAY  
M/W CHERT.

**Assumed Specific Gravity=** 2.70

**Remarks:** CML LAB NO. K-16258

**Client:** CITY OF KNOXVILLE

**Project:** AMHERST ROAD  
KNOXVILLE, TENNESSEE

**Location:** A-2

**Depth:** 22

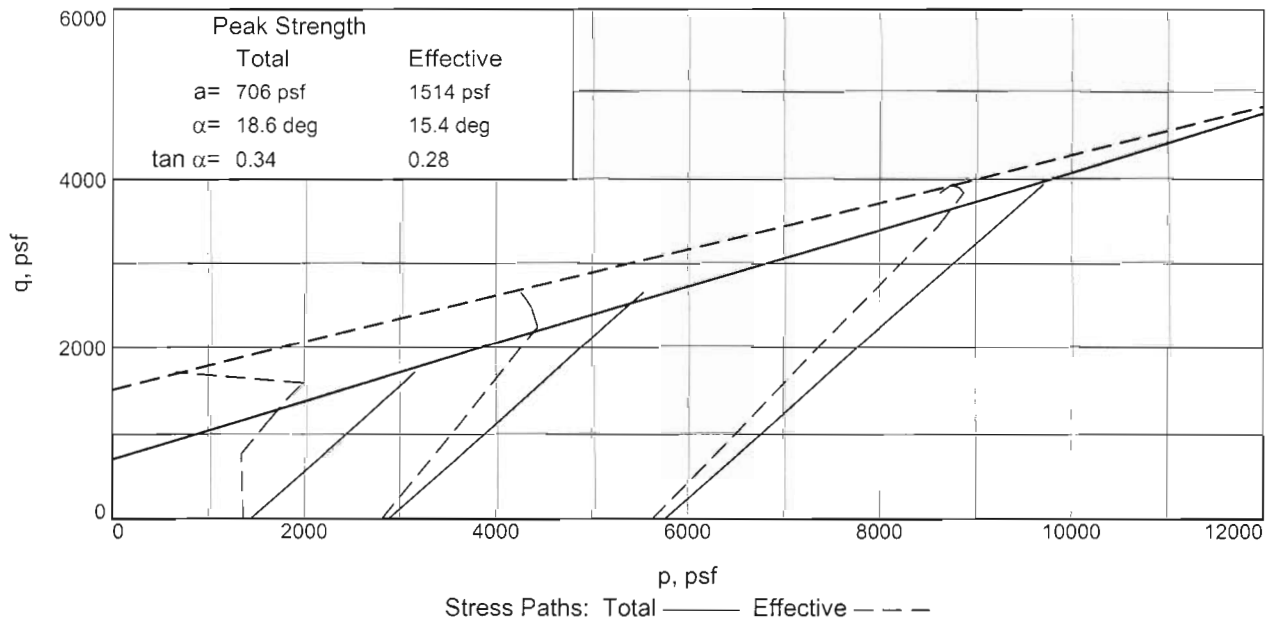
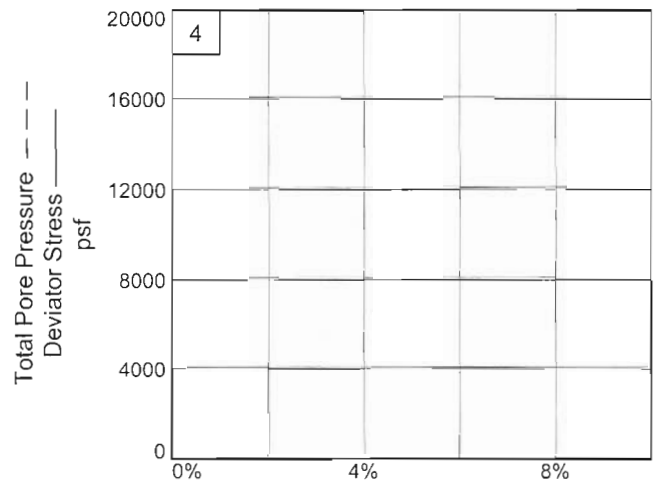
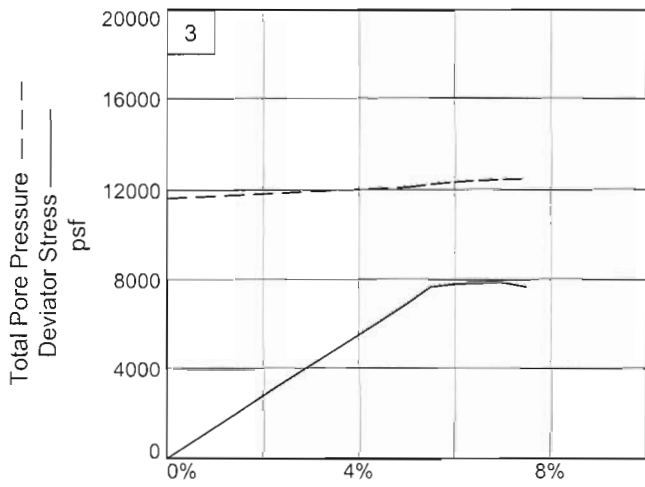
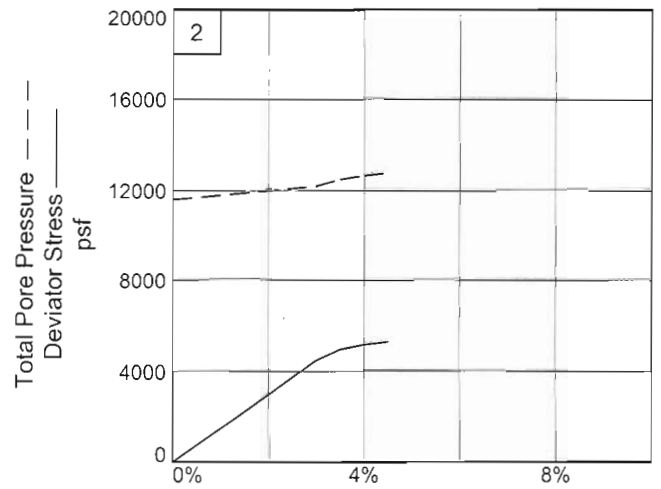
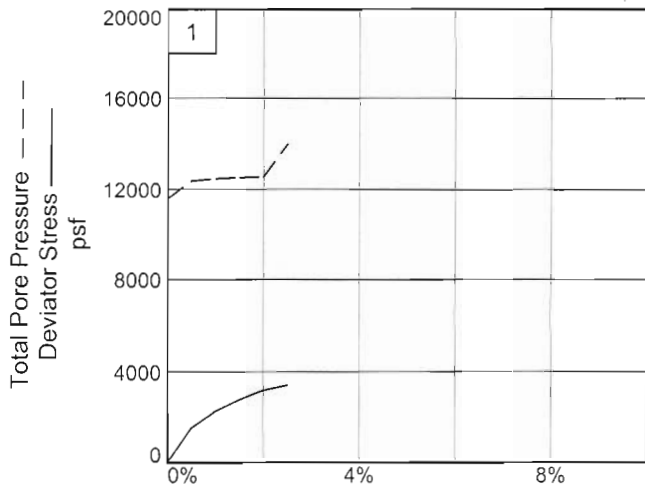
**Proj. No.:** 315152

**Date Sampled:** 1-12-16

TRIAXIAL SHEAR TEST REPORT

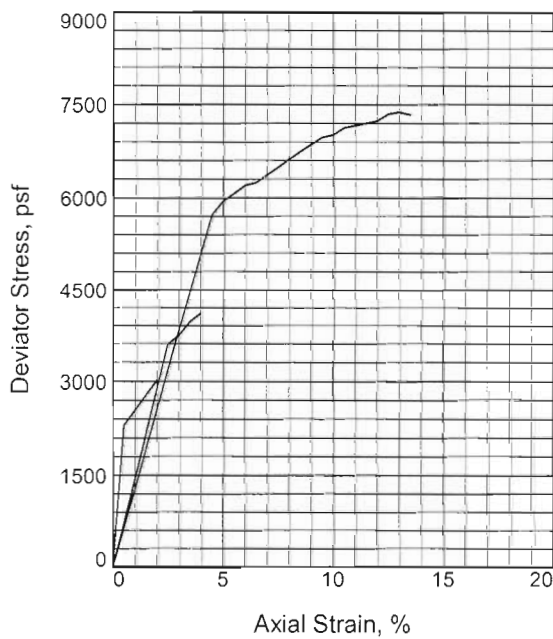
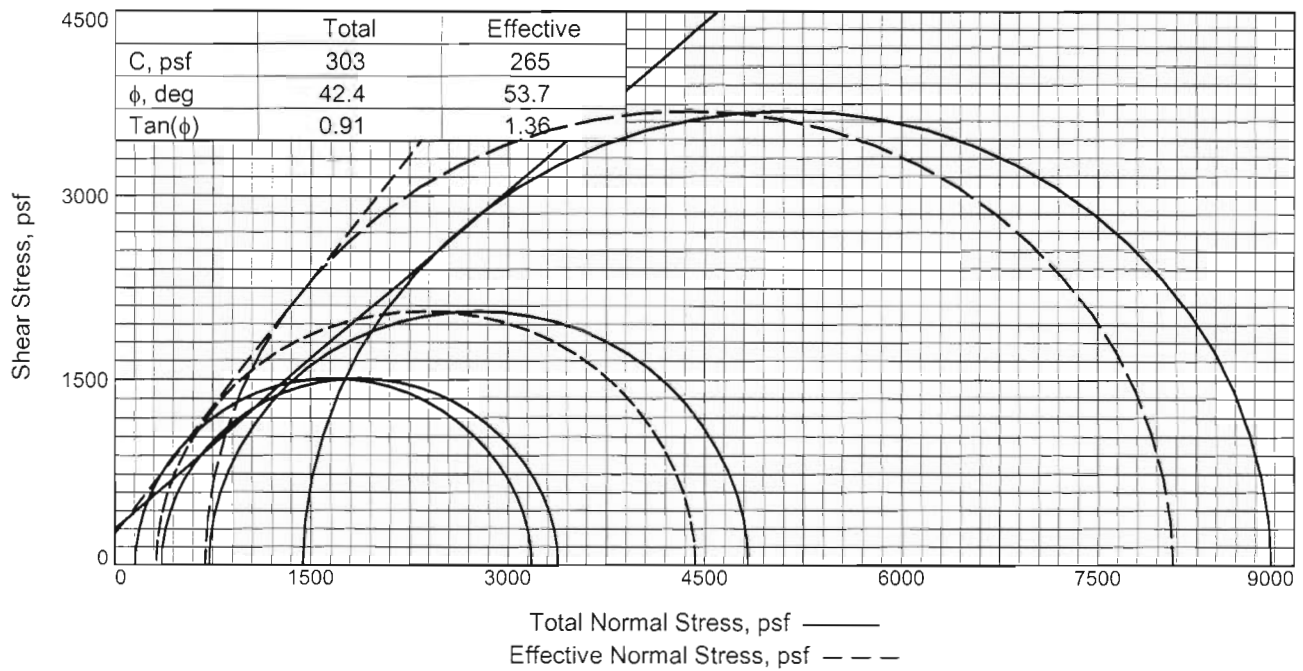
**Foundation Systems Engineering**

Figure 1



Client: CITY OF KNOXVILLE  
 Project: AMHERST ROAD  
 Location: A-2      Depth: 22  
 Project No.: 315152

Figure 1A



Sample No.	1	2	3
<b>Initial</b>			
Water Content, %	12.2	12.2	12.2
Dry Density, pcf	124.9	124.9	124.9
Saturation, %	99.9	99.9	99.9
Void Ratio	0.3244	0.3244	0.3244
Diameter, in.	1.85	1.85	1.85
Height, in.	3.70	3.70	3.70
<b>At Test</b>			
Water Content, %	12.2	12.2	12.2
Dry Density, pcf	124.9	124.9	124.9
Saturation, %	100.0	100.0	100.0
Void Ratio	0.3244	0.3244	0.3244
Diameter, in.	1.85	1.85	1.85
Height, in.	3.70	3.70	3.70
Strain rate, in./min.	1.00	1.00	1.00
Back Pressure, psi	90.00	90.00	90.00
Cell Pressure, psi	92.50	95.00	100.00
Fail. Stress, psf	3024	4114	7383
Total Pore Pr., psf	13162	13363	13709
Ult. Stress, psf			
Total Pore Pr., psf			
$\bar{\sigma}_1$ Failure, psf	3182	4431	8074
$\bar{\sigma}_3$ Failure, psf	158	317	691

**Type of Test:**

CU with Pore Pressures

**Sample Type:** UNDISTURBED

**Description:** BROWN & TAN, SILTY CLAY M/W SANDSTONE GRAVEL.

**Assumed Specific Gravity=** 2.65

**Remarks:** CML LAB NO. K-16259

**Client:** CITY OF KNOXVILLE

**Project:** AMHERST ROAD  
KNOXVILLE, TENNESSEE

**Location:** A-3

**Depth:** 2

**Proj. No.:** 315152

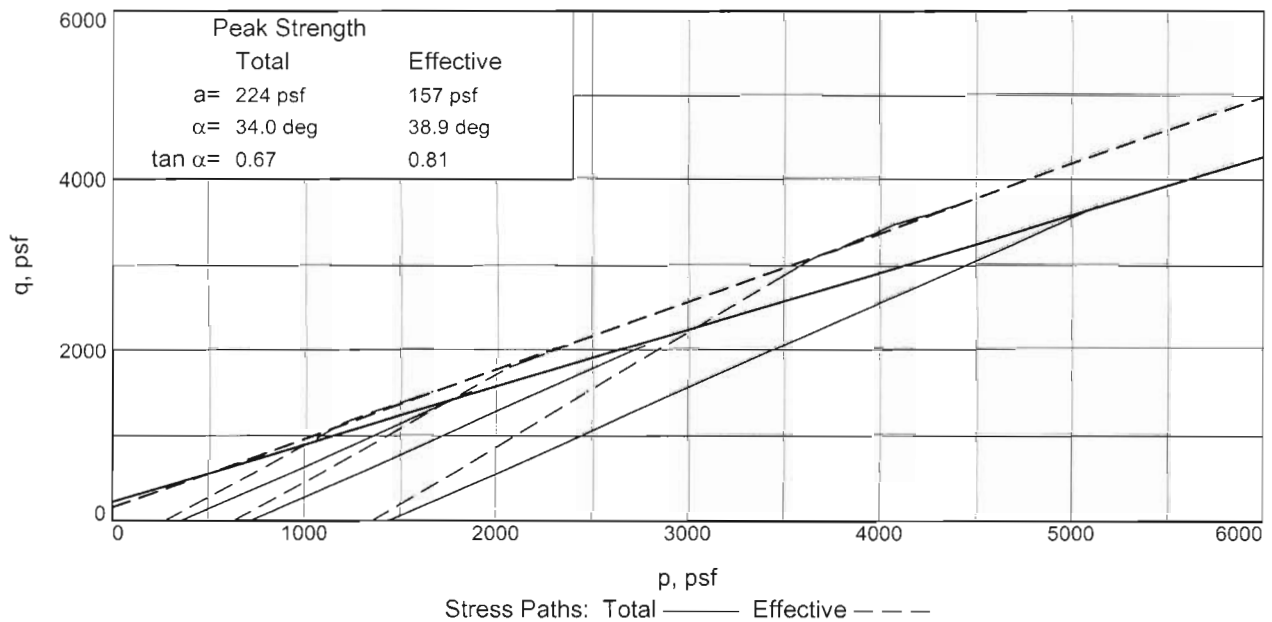
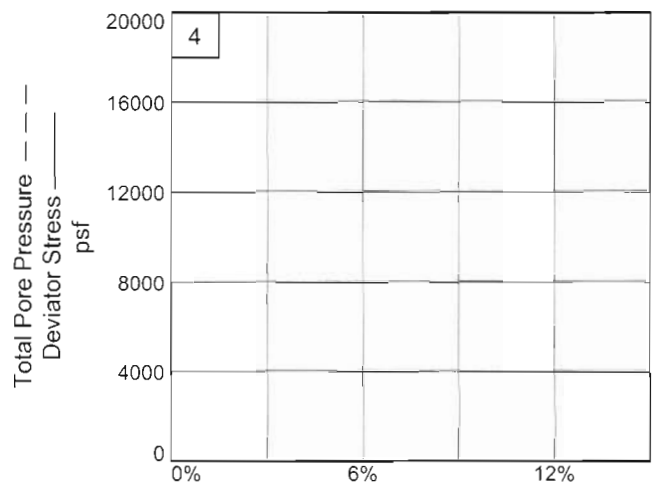
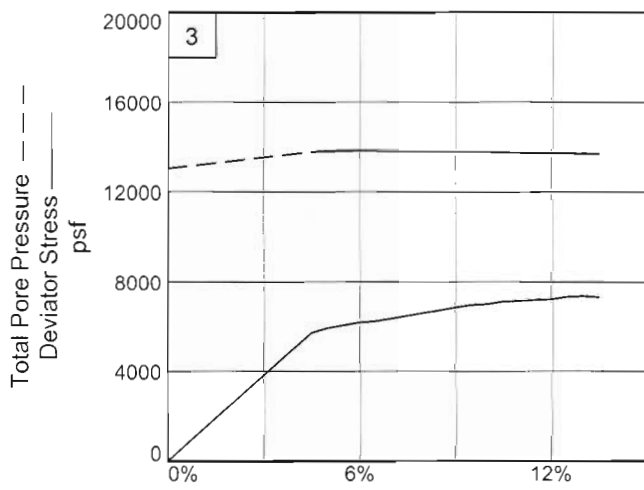
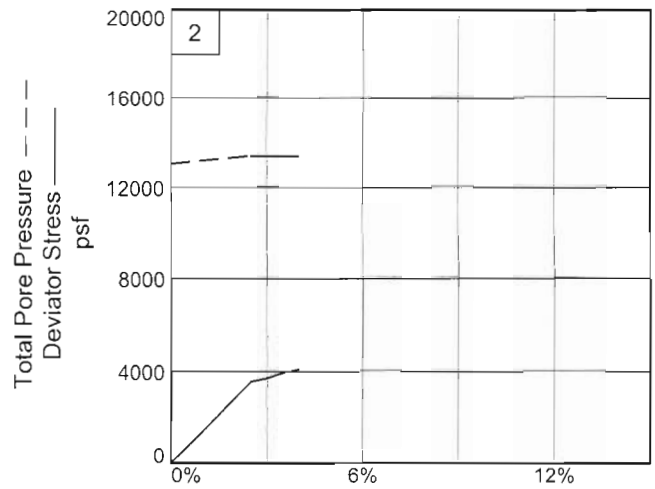
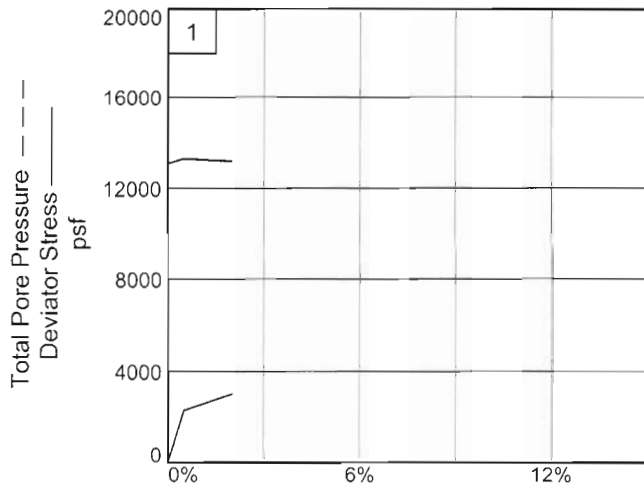
**Date Sampled:** 1-12-16

TRIAxIAL SHEAR TEST REPORT

**Foundation Systems Engineering**

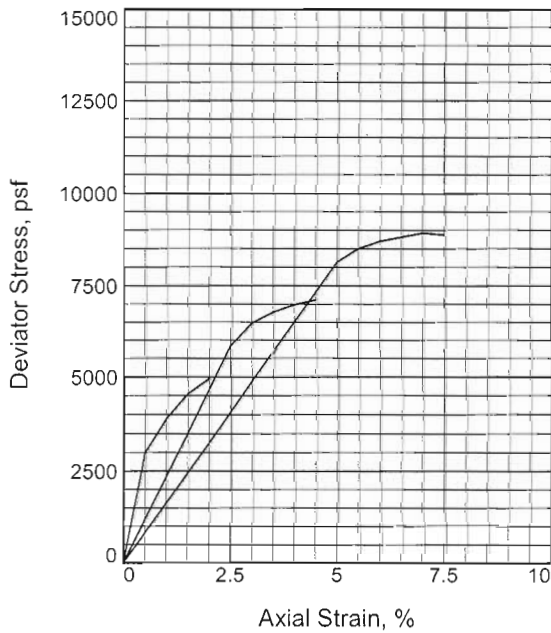
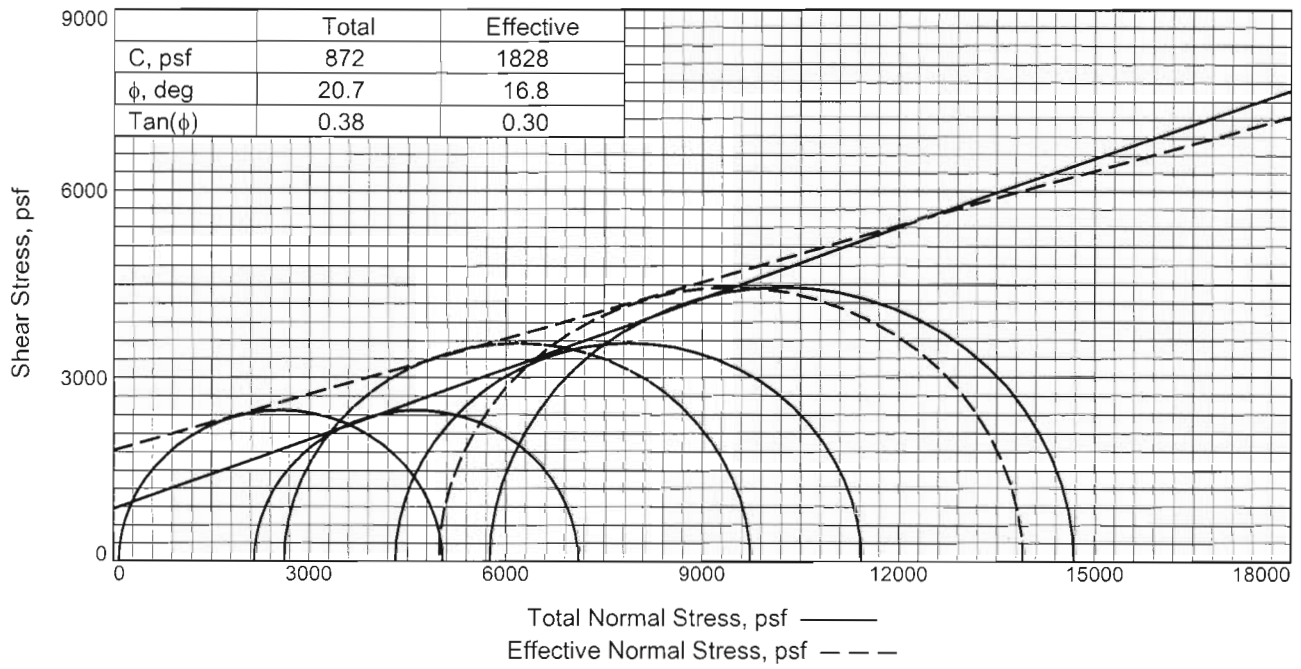
Figure 2





Client: CITY OF KNOXVILLE  
 Project: AMHERST ROAD  
 Location: A-3      Depth: 2  
 Project No.: 315152

Figure 2A



Sample No.		1	2	3
Initial	Water Content, %	29.9	29.9	29.9
	Dry Density, pcf	93.2	93.2	93.2
	Saturation, %	99.8	99.8	99.8
	Void Ratio	0.8088	0.8088	0.8088
	Diameter, in.	1.85	1.85	1.85
	Height, in.	3.70	3.70	3.70
At Test	Water Content, %	30.0	30.0	30.0
	Dry Density, pcf	93.2	93.2	93.2
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.8088	0.8088	0.8088
	Diameter, in.	1.85	1.85	1.85
	Height, in.	3.70	3.70	3.70
Strain rate, in./min.		1.00	1.00	1.00
Back Pressure, psi		80.00	80.00	80.00
Cell Pressure, psi		95.00	110.00	120.00
Fail. Stress, psf		4956	7122	8928
Total Pore Pr., psf		13594	13219	12298
Ult. Stress, psf				
Total Pore Pr., psf				
$\bar{\sigma}_1$ Failure, psf		5042	9742	13910
$\bar{\sigma}_3$ Failure, psf		86	2621	4982

**Type of Test:**

CU with Pore Pressures

**Sample Type:** UNDISTURBED

**Description:** REDDISH BROWN & TAN, CHERTY CLAY

**Assumed Specific Gravity=** 2.7

**Remarks:** CML LAB NO. K-16260

**Client:** CITY OF KNOXVILLE

**Project:** AMHERST ROAD  
KNOXVILLE, TENNESSEE

**Location:** A-5

**Depth:** 18

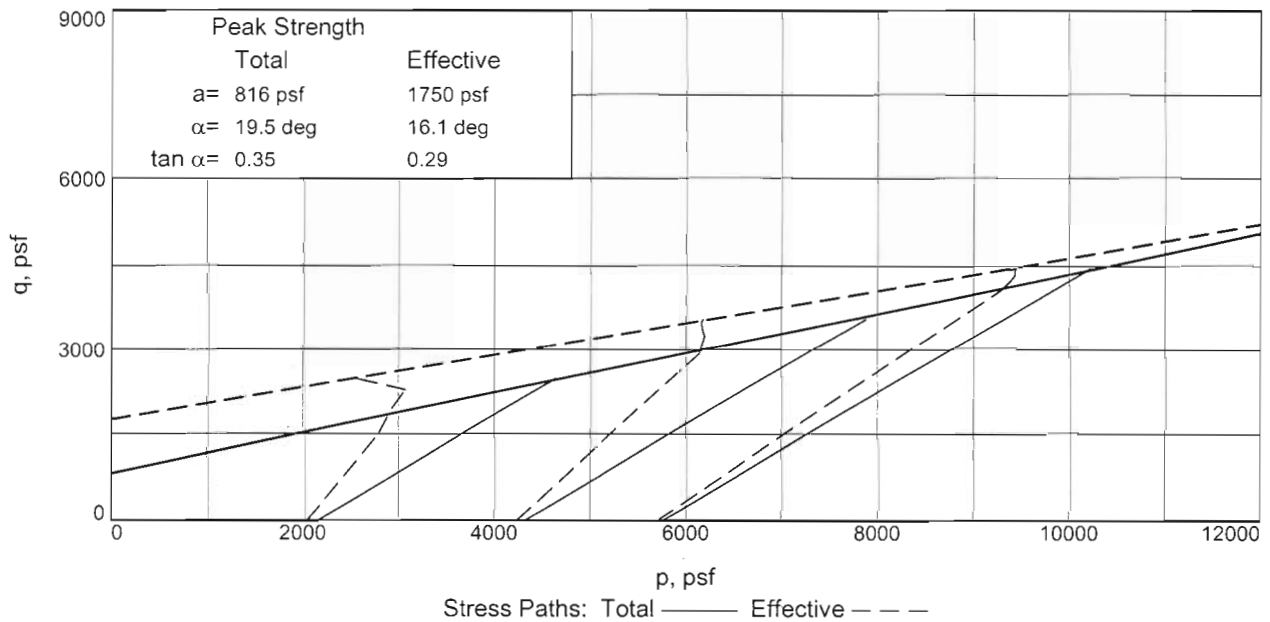
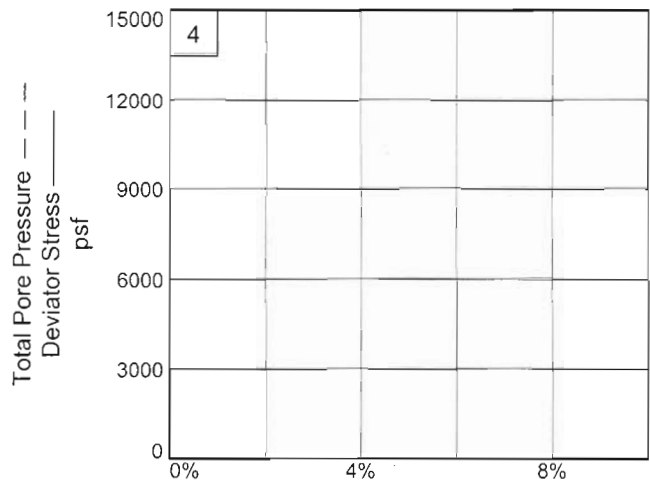
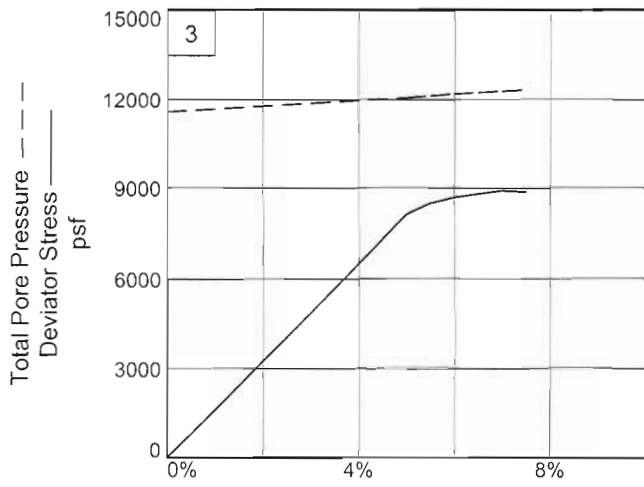
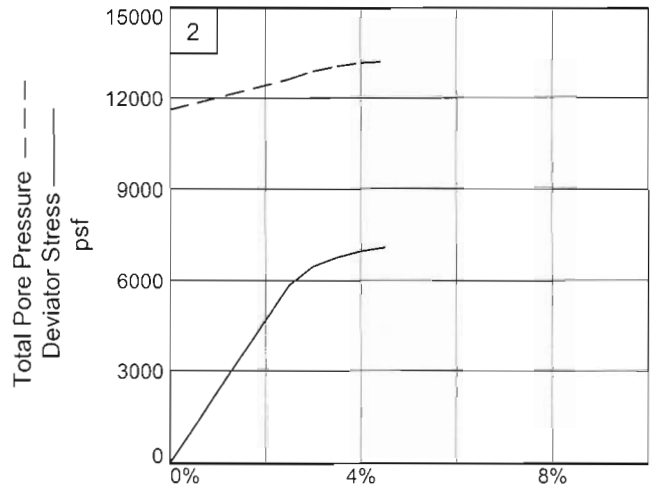
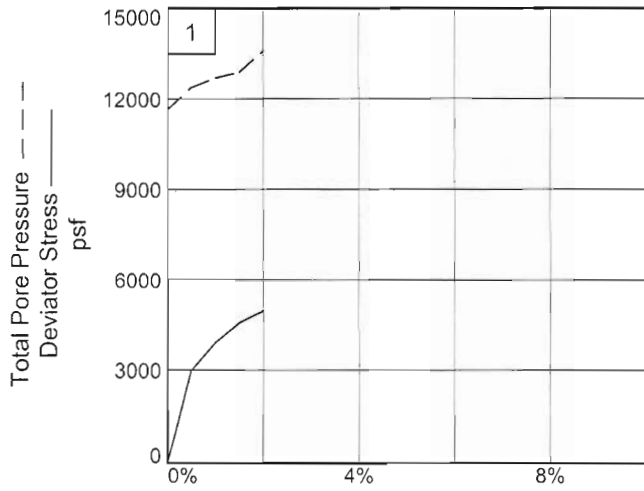
**Proj. No.:** 315152

**Date Sampled:** 1/11/16

TRIAxIAL SHEAR TEST REPORT

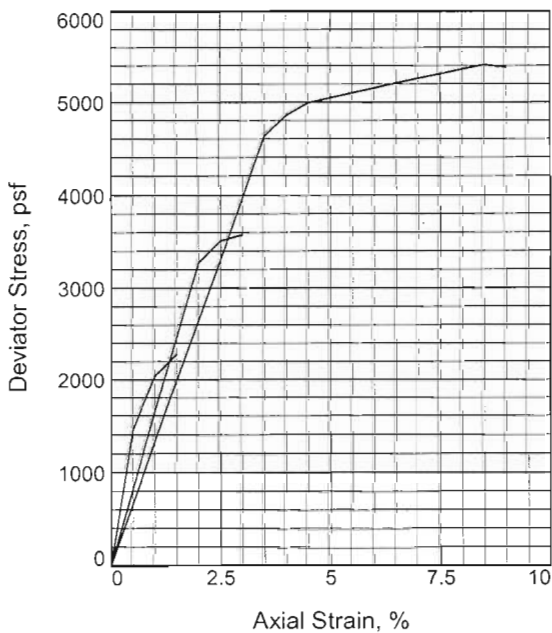
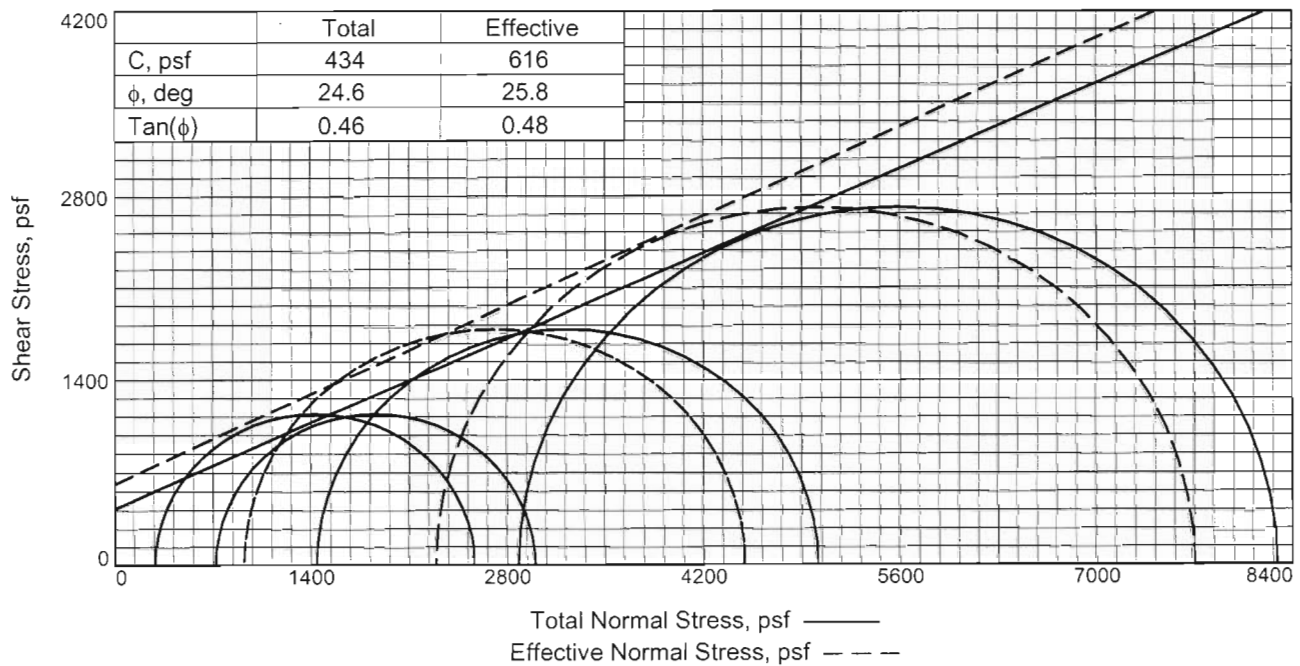
**Foundation Systems Engineering**

Figure 3



**Client:** CITY OF KNOXVILLE  
**Project:** AMHERST ROAD  
**Location:** A-5      **Depth:** 18  
**Project No.:** 315152

**Figure 3A**



Sample No.		1	2	3
Initial	Water Content, %	16.9	16.9	16.9
	Dry Density, pcf	115.7	115.7	115.7
	Saturation, %	99.8	99.8	99.8
	Void Ratio	0.4573	0.4573	0.4573
	Diameter, in.	1.85	1.85	1.85
	Height, in.	3.70	3.70	3.70
At Test	Water Content, %	16.9	16.9	16.9
	Dry Density, pcf	115.7	115.7	115.7
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.4573	0.4573	0.4573
	Diameter, in.	1.85	1.85	1.85
	Height, in.	3.70	3.70	3.70
Strain rate, in./min.		1.00	1.00	1.00
Back Pressure, psi		90.00	90.00	90.00
Cell Pressure, psi		95.00	100.00	110.00
Fail. Stress, psf		2280	3575	5412
Total Pore Pr., psf		13392	13478	13550
Ult. Stress, psf				
Total Pore Pr., psf				
$\bar{\sigma}_1$ Failure, psf		2568	4497	7701
$\bar{\sigma}_3$ Failure, psf		288	922	2290

**Type of Test:**

CU with Pore Pressures

**Sample Type:** UNDISTURBED

**Description:** BROWN & TAN, CHERTY CLAY

**Assumed Specific Gravity=** 2.7

**Remarks:** CML LAB NO. K-16261

**Client:** CITY OF KNOXVILLE

**Project:** AMHERST ROAD  
KNOXVILLE, TENNESSEE

**Location:** A-6

**Depth:** 6

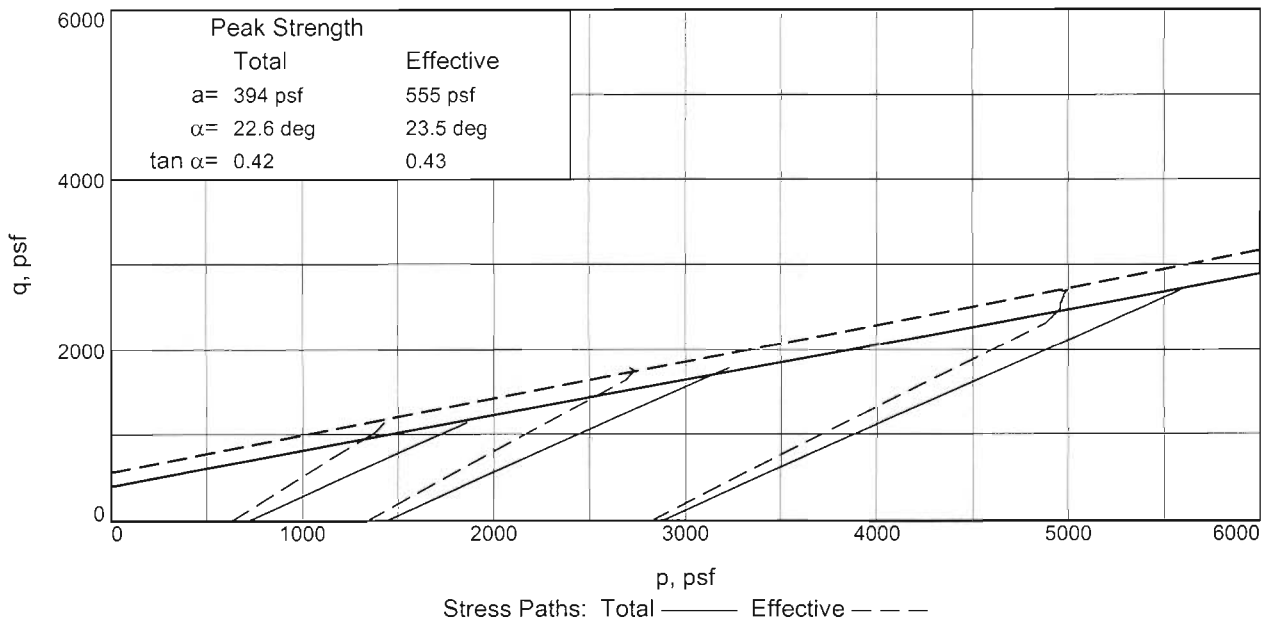
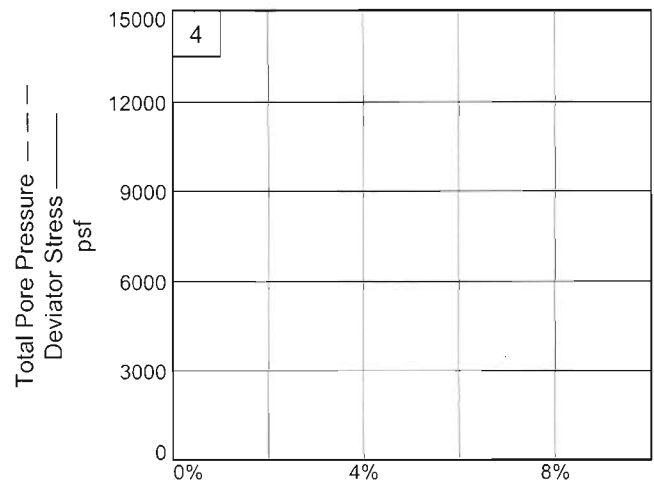
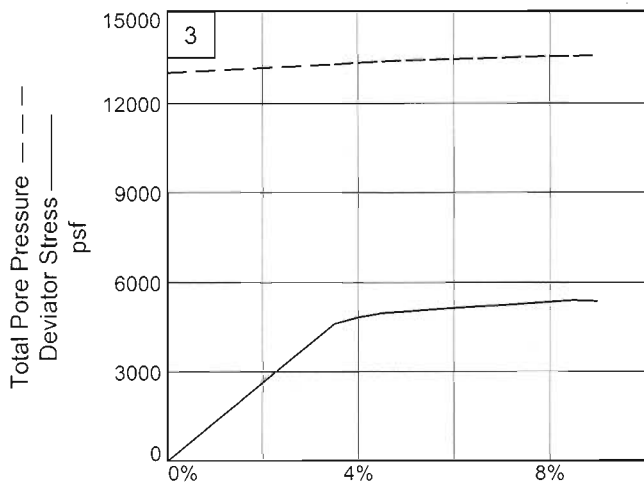
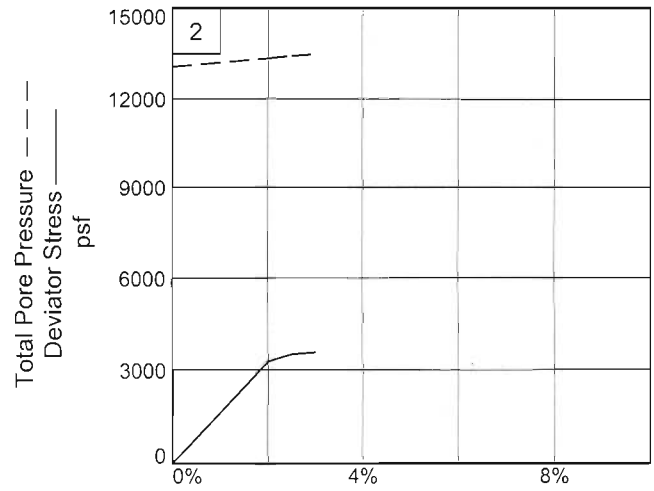
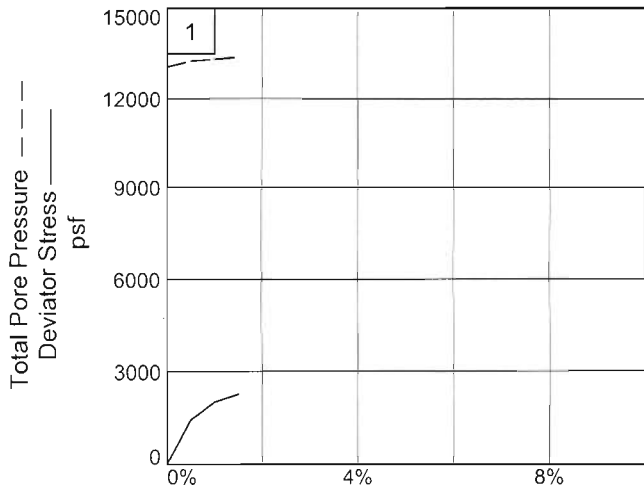
**Proj. No.:** 315152

**Date Sampled:** 1/11/16

TRIAXIAL SHEAR TEST REPORT

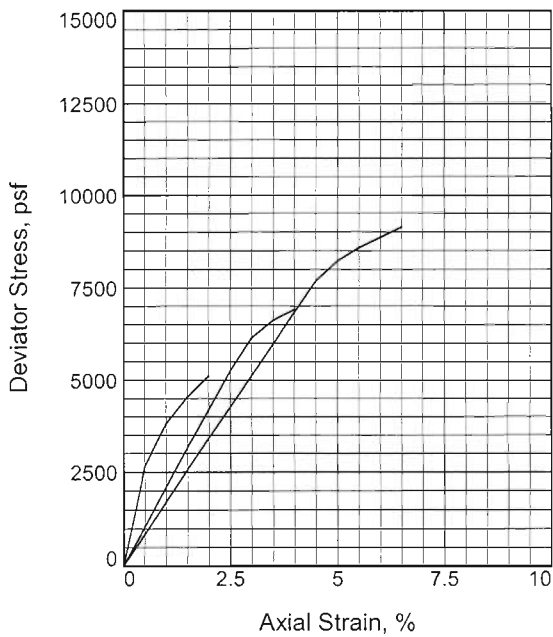
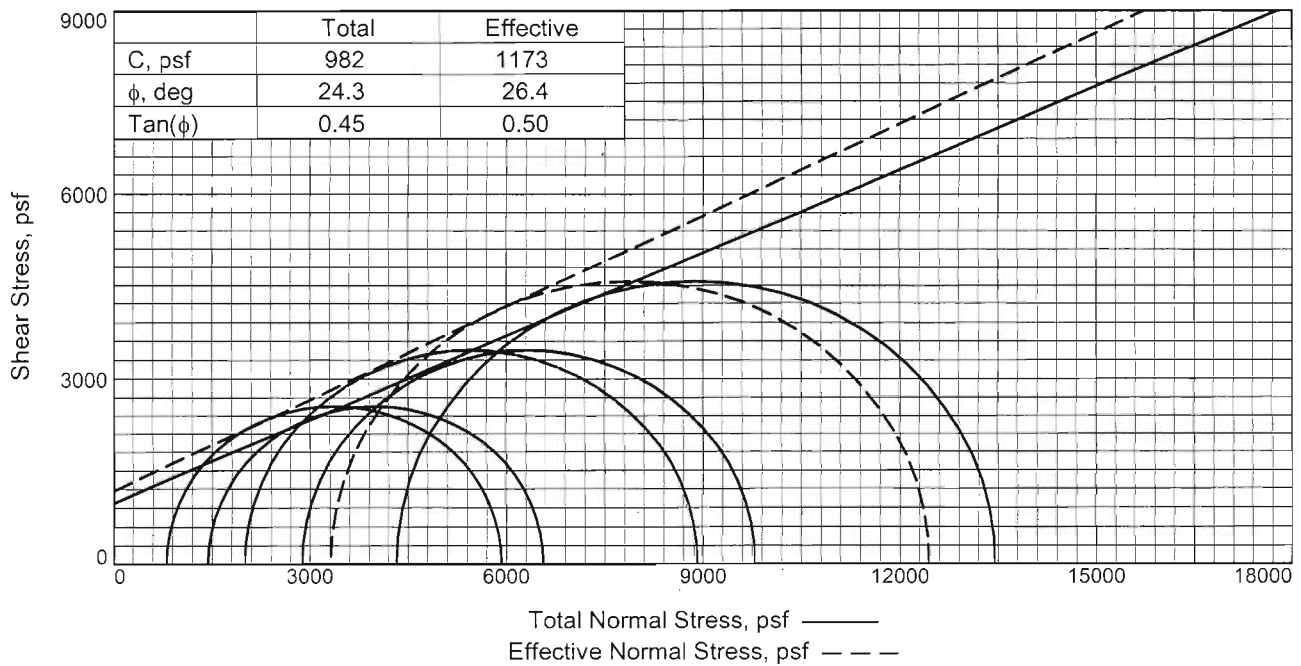
**Foundation Systems Engineering**

Figure 4



**Client:** CITY OF KNOXVILLE  
**Project:** AMHERST ROAD  
**Location:** A-6      **Depth:** 6  
**Project No.:** 315152

Figure 4A



Sample No.		1	2	3
Initial	Water Content, %	31.6	31.6	31.6
	Dry Density, pcf	90.8	90.8	90.8
	Saturation, %	99.6	99.6	99.6
	Void Ratio	0.8555	0.8555	0.8555
	Diameter, in.	1.85	1.85	1.85
	Height, in.	3.70	3.70	3.70
At Test	Water Content, %	31.7	31.7	31.7
	Dry Density, pcf	90.8	90.8	90.8
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.8555	0.8555	0.8555
	Diameter, in.	1.85	1.85	1.85
	Height, in.	3.70	3.70	3.70
Strain rate, in./min.		1.00	1.00	1.00
Back Pressure, psi		90.00	90.00	90.00
Cell Pressure, psi		100.00	110.00	120.00
Fail. Stress, psf		5124	6912	9136
Total Pore Pr., psf		13594	13838	13968
Ult. Stress, psf				
Total Pore Pr., psf				
$\bar{\sigma}_1$ Failure, psf		5930	8914	12448
$\bar{\sigma}_3$ Failure, psf		806	2002	3312

**Type of Test:**

CU with Pore Pressures

**Sample Type:** UNDISTURBED

**Description:** REDDISH BROWN & TAN, CLAY

**Assumed Specific Gravity=** 2.7

**Remarks:** CML LAB NO. K-16262

**Client:** CITY OF KNOXVILLE

**Project:** AMHERST ROAD  
KNOXVILLE, TENNESSEE

**Location:** A-7

**Depth:** 13

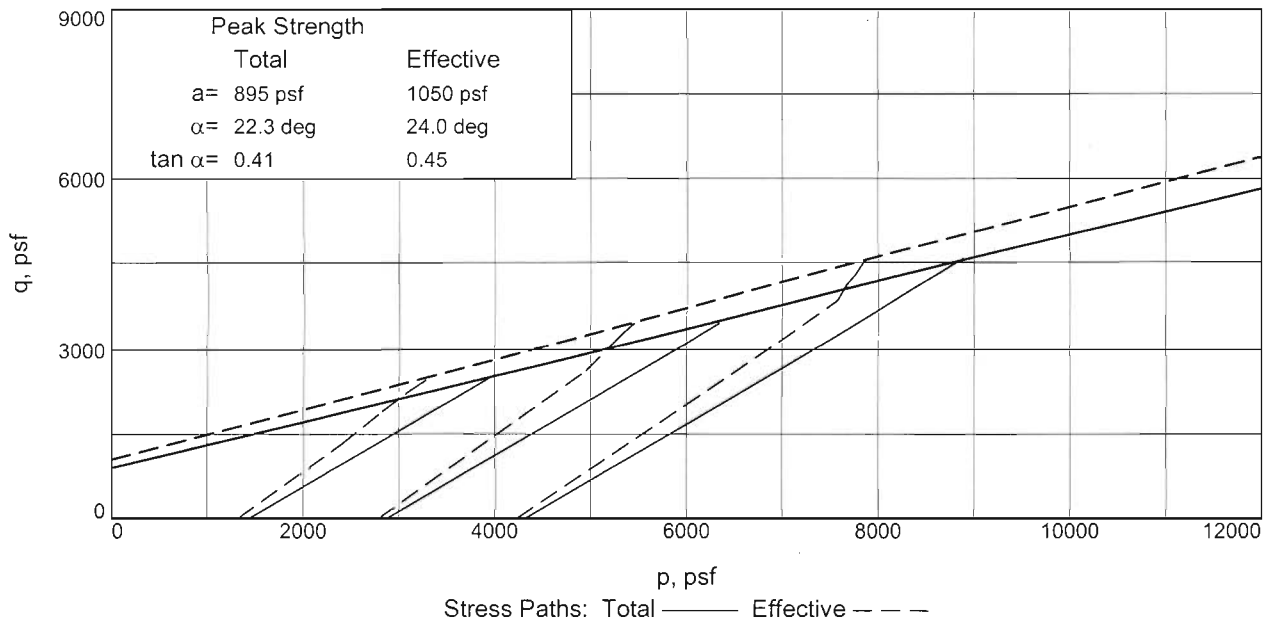
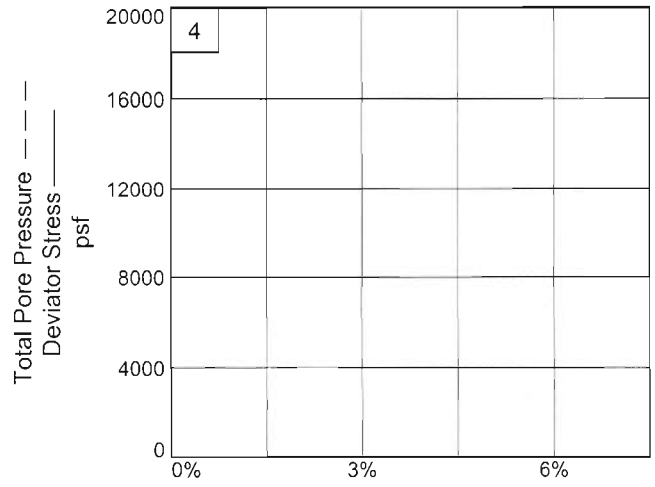
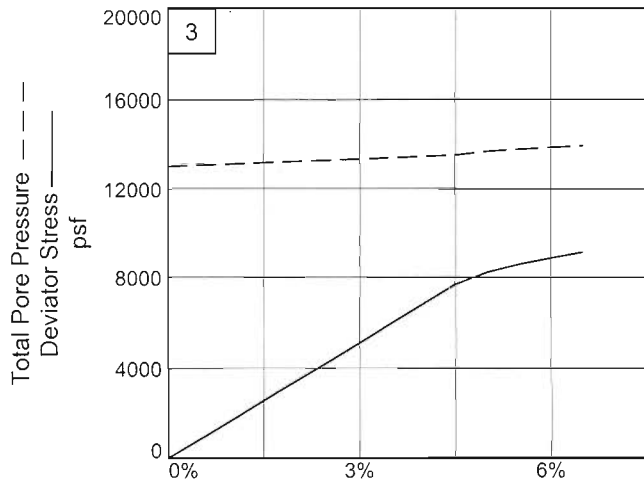
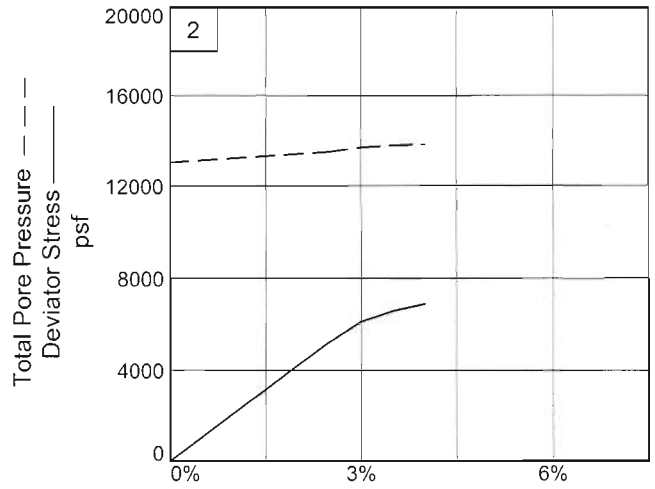
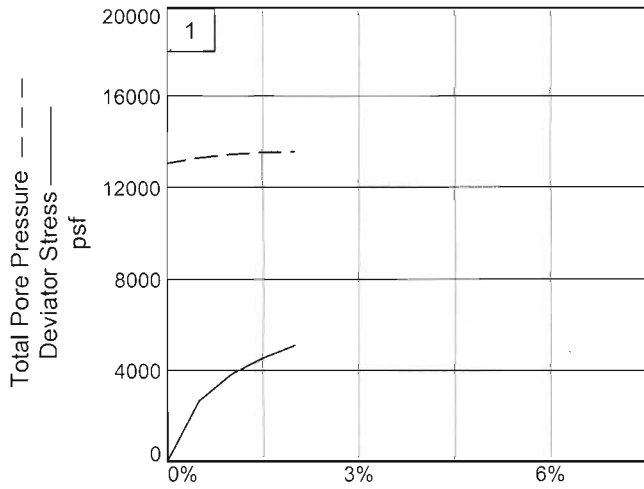
**Proj. No.:** 315152

**Date Sampled:** 1-11-16

TRIAXIAL SHEAR TEST REPORT

**Foundation Systems Engineering**

Figure 5



Client: CITY OF KNOXVILLE  
 Project: AMHERST ROAD  
 Location: A-7      Depth: 13  
 Project No.: 315152

Figure 5A

## **APPENDIX VI**

ASFE



# Important Information About Your Geotechnical Engineering Report

*Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.*

*The following information is provided to help you manage your risks.*

## **Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects**

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply the report for any purpose or project except the one originally contemplated.*

## **Read the Full Report**

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

## **A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors**

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,

- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

## **Subsurface Conditions Can Change**

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

## **Most Geotechnical Findings Are Professional Opinions**

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

## **A Report's Recommendations Are *Not* Final**

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

### **A Geotechnical Engineering Report Is Subject to Misinterpretation**

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

### **Do Not Redraw the Engineer's Logs**

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

### **Give Contractors a Complete Report and Guidance**

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time to perform additional study.* Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

### **Read Responsibility Provisions Closely**

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that

have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

### **Geoenvironmental Concerns Are Not Covered**

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

### **Obtain Professional Assistance To Deal with Mold**

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

### **Rely, on Your ASFE-Member Geotechnical Engineer for Additional Assistance**

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.



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