Report of GEOTECHNICAL EXPLORATION

AMERICAN ENGINEERS, INC.

CAMP NELSON NATIONAL CEMETERY WASH BAY & STORAGE BUILDING

NICHOLASVILLE, KY

JUNE 2014

NCA Project No. 833-CM3-026

DESIGNING YOUR FUTURE, TODAY. AEI

June 6, 2014

Mr. Mike Brandvold, P.E.
Anderson Engineering of Minnesota, LLC
13605 1st Avenue North Suite 100
Plymouth, MN 55441

RE: Report of Geotechnical Exploration
CAMP NELSON NATIONAL CEMETERY
CCNC Wash Bay & Storage Building
Nicholasville, Kentucky
AEI Project No. 214-085

Dear Mr. Brandvold:

American Engineers, Inc. Field Services Center is pleased to submit this geotechnical report that details the results of our geotechnical exploration performed at the above referenced site.

The attached report describes the site and subsurface conditions and also details our recommendations for the proposed project. The Appendices to the report contains a drawing with a boring layout, typed boring logs, and the results of all laboratory testing.

We appreciate the opportunity to be of service to you on this project and hope to provide further support on this and other projects in the future. Please contact us if you have any questions regarding this report.

Respectfully submitted,
AMERICAN ENGINEERS, INC.

Dennis Mitchell, P.E
Senior Geotechnical Engineer

REPORT OF GEOTECHNICAL EXPLORATION
CAMP NELSON NATIONAL CEMETERY
CCNC WASH BAY & STORAGE BUILDING
NICHOLASVILLE, KENTUCKY

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REPORT OF GEOTECHNICAL EXPLORATION
CAMP NELSON NATIONAL CEMETERY
CCNC WASH BAY & STORAGE BUILDING
NICHOLASVILLE, KENTUCKY

1 GENERAL SITE DESCRIPTION

The site is located off US HWY 27 within Camp Nelson National Cemetery in Nicholasville, Kentucky. The existing topography of the site is best described as level to gently rolling.

It is our understanding that a wash bay and storage building(s) are to be constructed at the referenced site. Due to existing septic field that crosses the site in a northwest to southeast direction; it is not known at this time whether the wash bay and storage bay will be housed together or split and constructed separately. Any structure constructed on the north side of the septic field is expected to have a finished floor elevation ranging from 944 to 945 and will receive one to two feet of fill to achieve subgrade elevation. On the south side of the septic field, the likely finished floor elevation will be lower and may require up to five feet of fill to achieve subgrade elevation. Exterior walls of the building may be CMU, wood or metal stud framing with veneer, a pre-fabricated system or some combination of these types. Anticipated isolated column and wall loads were unknown at the time of this report but are not anticipated to exceed four kips per linear foot (klf) with column loads not anticipated to exceed 50 kips.

2 GENERAL SITE GEOLOGY

Review of available geologic mapping for the area (*Geologic Map of the Little Hickman Quadrangle, Central Kentucky, Wolcott, 1969)* indicates that the site is underlain by the Tyrone Limestone Formation of the Highbridge group. The limestone is described as light brownish gray, laminated to thick bedded, and occurs with dolomite as irregular bodies or laminae. A persistent unit of greenish-gray argillaceous limestone six to seven feet thick is typically encountered in the upper part of the formation. The Kentucky River Fault was noted to lie within about one mile east of the site.

Karst potential mapping was also reviewed for the site. The Kentucky Geologic Survey identified the site and the surrounding areas as exhibiting high to very high potential for the development of karst features. It is impossible to investigate a site to fully identify the presence of or future development of geologic hazards during the course of a typical geotechnical investigation. It should be understood and accepted by the Owner that there is always some risk of future ground subsidence when building in any region where karst activity is known to historically exist.

3 SCOPE OF WORK PERFORMED

The geotechnical exploration consisted of drilling three soil test borings within the approximate limits of the potential building footprint areas. Each of the soil borings was drilled to depth to a maximum depth of 16.5 feet. Boring locations were roughly staked in the field by an AEI field geologist and driller; then subsequently located and elevated by Anderson Engineering of MN survey crew personnel.

The borings were drilled by an AEI drill crew using a truck-mounted drill rig equipped with continuous flight hollow-stem augers. Standard penetration tests (SPT's) and/or undisturbed tube samples were performed in each of the soil test borings at 2 ½ to 5 foot intervals. A geologist was on site throughout the investigation to log the recovered samples, with particular attention given to soil type, color, relative moisture content, primary constituents and soil strength consistencies. Recovered samples were returned to the lab and further classified by experienced laboratory personnel and verified by a Geotechnical Engineer.

The natural moisture content of the soil samples was determined in the laboratory. The natural moisture content is denoted as (W%) and shown as a percentage of the dry weight of the soil on the boring logs. In addition, Atterberg Limits and unconfined compressive strength tests were performed on samples representative of the predominant soil horizons. The results of the laboratory tests are summarized in Appendix C.

The soils were classified in the laboratory in general accordance with the Unified Soil Classification System (USCS). The Unified symbol for each stratum is shown on the legend for the typed boring logs. The testing was performed in accordance with the generally accepted standards for such tests.

4 RESULTS OF THE EXPLORATION

4.1 GENERAL

Information provided in the Appendices for this report includes a boring layout, typed boring logs, results of the laboratory tests and other relevant geotechnical information. A description of the subsurface soil, bedrock and groundwater conditions follows.

4.2 SUBSURFACE SOIL CONDITIONS

The generalized subsurface conditions encountered at the boring locations including descriptions of the various strata and their depths and thicknesses are presented on the Typed Boring Logs in Appendix B. Topsoil was encountered at the existing ground surface in each of the borings with thicknesses ranging from about 12 to 14 inches. Beneath the topsoil, low to moderate plasticity residual clays were encountered to the bottom of the hole depths. The clays were typically described as lean clay, containing trace amounts of fine sand, trace to some gravel, brown in color, moist of the anticipated optimum moisture content for compaction and stiff in soil strength consistency with isolated medium stiff and very stiff zones.

Atterberg Limits testing was performed on representative samples and the results indicate that the near-surface clay soils classify as CL (Clay of Low plasticity), lean clay, in accordance with the USCS. Limit test results range of 40 and 41 were obtained with corresponding plasticity indices of 15 and 19 percent.

SPT-N values in the clay soils ranged from 5 to 25 blows per foot (bpf), with most values between 8 and 15 bpf. Corresponding Qp values range from 1.0 to 4.5+ tons per square foot (tsf), with most values between 1.5 and 3.25 tsf. Unconfined compressive strength results range from 2,691 to 7,036 psf. Together, the SPT-N, Qp and unconfined compressive strength values are generally indicative of stiff soil strength consistencies. Natural moisture contents of the clay soils range from about 22 to 41 percent. Results of Atterberg limits and moisture content testing indicate that the residual clays have a moisture content of approximately ten percent or more wet of the plastic limit; with a general increase in moisture content with increasing depth.

4.3 BEDROCK CONDITIONS

Refusal, as would be indicated by the Driller on the field boring logs, indicates a depth where either essentially no downward progress can be made by the auger or where the N-value indicates essentially no penetration of the split-spoon sampler. It is normally indicative of a very hard or very dense material such as large boulders or the upper bedrock surface. Each of the borings was drilled to a predetermined cutoff depth of 16.5' without encountering auger refusal.

4.4 GROUNDWATER CONDITIONS

Groundwater was not encountered in any of the three borings completed on the site during the subsurface investigation. To accurately determine the long-term groundwater level, as well as the seasonal and precipitation induced fluctuations of the groundwater level, it is necessary to install piezometers in the borings, and monitor them for an extended length of time. Frequently, groundwater conditions affecting construction in this region are caused by trapped or perched groundwater, which occurs within the soil materials or at the soil/rock interface in irregular, discontinuous locations. If these water bodies are encountered during excavation, they can produce seepage durations and rates that will vary depending on the recent rainfall activity and the hydraulic conductivity of the material.

4.5 Seismic Conditions

According to the Kentucky Building Code, 2012 Edition, and the subsurface conditions encountered in the borings, Site Class C should be utilized for any seismic structural design.

Soil liquefaction analysis was outside the scope of this investigation. Prior studies in this region on similar soil types indicate that the potential for liquefaction is low to moderate and is primarily dependent on the variability of site soils and earthquake severity.

Consideration for seismic loading and liquefaction potential beyond this level of investigation is left to the discretion of the structural framing and foundation design engineer.

5 ANALYSES AND RECOMMENDATIONS

The recommendations that follow are based on our conceptual understanding of the project. As the site design is advanced, please notify us of any significant design changes so that our recommendations can be reviewed and modified as necessary.

5.1 General Site Work

5.1.1 On-Site Soils

The near-surface soils on this site are residual clays which classify as low to moderate plasticity lean clay, CL, in accordance with the USCS. These soils exhibit low to moderate potential to swell or shrink when exposed to long-term increases or decreases in moisture content. These soils are suitable for use as fill material provided they are wetted or dried to a moisture content suitable for compaction.

5.1.2 General Fill Requirements

Any material, whether borrowed on-site or imported to the site, placed as engineered fill on the project site beneath the proposed building or other proposed on-grade structures such as pavement, parking lots, sidewalks, etc., should be an approved material, free of environmental contamination, vegetation, topsoil, organic material, wet soil, construction debris and rock fragments greater than six inches in diameter. We recommend that any borrow material, if needed, consist of granular or lean clay materials or mixtures thereof with Unified Classifications of CL, SC or GC. We further recommend high plasticity clays, known as fat clays (CH soils) not be *imported* to the site due to their potential for volume changes with fluctuations in moisture content.

The preferred borrow material should have a Plasticity Index (PI) less than 20 and a standard Proctor maximum dry density of at least 95 pcf. Engineering classification and standard Proctor tests should be performed on all potential borrow soils and the test results evaluated by an AEI Geotechnical Engineer to evaluate the suitability of the soil for use as engineered fill.

5.1.3 Topsoil Stripping/Removal of Existing Septic Field Drainlines

Prior to earthwork operations, any topsoil and surface plant material root mat should be stripped from both cut and fill areas and stockpiled for landscaping purposes. Any existing septic field drain lines within the limits of proposed structures or drives should be removed and the affected areas evaluated by proofrolling as outlined in Section 5.1.4. **Soft soils should be anticipated within the limits of the existing septic field.**

5.1.4 Subgrade Evaluation/Conditioning

Once the surface material is removed, areas to receive fill should be "proofrolled" under the observation of an AEI Geotechnical Engineer or Technician to evaluate the subgrade for suitability for fill placement. The proofrolling should be performed using heavy construction equipment such as a fully loaded single or tandem axle dump truck (approximately 20-25 tons), passing repeatedly over the subgrade at a slow rate of speed.

Subgrade soils that are considered unstable after proofrolling should be stabilized by additional compaction or by one or more of the following methods; in-place stabilization using chemical methods (lime/soil cement), removal and replacement with engineered fill, partial depth removal and replacement with a crushed (angular) aggregate layer, or partial depth removal and replacement with a geogrid and a crushed aggregate layer. The specific method of treatment will be based on the conditions present at the time the proofrolling is performed and local availability of materials and economic factors. The selection of the appropriate method to mitigate degrading subgrade soils is dependent on the time of year site work is anticipated, cost, anticipated effectiveness and scheduling impacts. AEI can assist in selecting this method considering all factors.

Once the subgrade is judged to be relatively uniform and suitable for support of engineered fill, fill areas should be brought to design elevations with on site soil and/or suitable off-site borrow material placed and compacted as specified in Section 5.1.5 Fill Placement.

5.1.5 Fill Placement

Suitable fill material placed under structural areas should be placed in maximum eight inch (loose thickness) horizontal lifts, with each lift being compacted to a minimum of 98 percent of the standard Proctor maximum dry density, at a moisture content within two percent of optimum. The compaction requirement may be reduced to 95 percent in proposed roadway and paved areas and to 92 percent in proposed field and landscape areas. At this site, wetting or drying of the soils will typically be necessary to achieve a moisture content suitable for compaction. Representative and adequate field density testing should be performed by AEI to verify that compaction requirements have been met.

5.1.6 Soil Movement

Site grading should be maintained during construction so that positive drainage is promoted at all times. Final site grading should be accomplished in such a manner as to divert surface runoff and roof drains away from the foundation elements and paved areas. Precipitation runoff should be collected in storm sewers as quickly as possible. Maintenance should be performed regularly on paved areas to seal pavement cracks and reduce surface water infiltration into the pavement subgrade.

5.1.7 Site Soil Practices

Working with the on-site soils will demand sensible construction practices and techniques. Some of these include:

- Prevent stripping too far in advance of actual earthwork needs. Problems arise when broad areas of clay/silt mixtures are exposed and allowed to become wet and soft from rainfall. Once saturated, deep rutting can occur by movement of construction equipment.

- Strip areas to receive fill in small, sequential areas as needed. These areas should be limited to the contractor’s abilities to reasonably place and compact fill material.

- Schedule earthwork construction to take full advantage of a summer season. The on-site clays need to be placed within 2% of optimum moisture content to achieve compaction and reduce the potential for subgrade volume change. This moisture range is difficult to achieve in the winter and early spring when rainfall activity is more prevalent and soil drying is not always possible.

- Maintain good surface drainage during earthwork construction. Grade construction areas on a daily basis if necessary to promote sheet drainage of precipitation and seal all engineered fill placed with a smooth drum steel roller at the end of each day.

- Perform frequent density tests during fill placement to confirm achievement of proper compaction.

5.2 STRUCTURE FOUNDATIONS

5.2.1 Recommended Bearing Capacity Values

The on-site soils are suitable for support of isolated spread and continuous wall footings. Based on the borings drilled, the on-site soils are suitable for moderate bearing pressures. A net allowable soil bearing pressure of 2,000 psf is recommended for design of spread and continuous footings supported on the native soils or properly placed and compacted engineered fill.

These recommendations are provided in consideration of the field-testing, laboratory testing, local codes, and our experience with materials of similar description.

5.2.2 Recommended Minimum Footing Dimensions

The *minimum* recommended width of continuous wall footings is 18 inches. The minimum recommended plan dimension for isolated spread footings is 24 inches. Actual foundation sizes should be determined by the foundation engineer based on design structure loads and the net allowable bearing values presented in 5.2.1.

5.2.3 Footing Trenches

We recommend that the bottom of exterior continuous strip spread footings extend a minimum of 24 inches below finished exterior grade to provide protection against frost penetration related problems in normal winters. Interior foundations not exposed to severe drying, freezing temperatures, and/or severe moisture fluctuations can be constructed at relatively shallow depths as appropriate for construction. Foundation construction should follow these recommendations:

- Foundation concrete should be placed in the excavations the same day the trenches are cut.

- Exposed bearing surfaces should be protected from severe drying, freezing, and water accumulation. A concrete "mud-mat" may be constructed over the bearing materials if the excavation must remain exposed to the elements for an extended period of time.

- Any loose soil, debris, or excess water should be removed from the bearing surface by hand cleaning prior to concrete placement.

- The foundation-bearing surface should be level or appropriately benched.

- Foundation materials that have deteriorated as a result of the elements should be removed prior to concrete placement.

- Foundation trenches should be “clean-cut” where possible and constructed without the use of forms.

- Reinforcing steel should be placed in all footings to provide strength to distribute loads on the foundation that may be overlying weak or more compressible foundation materials to stronger adjacent materials.

5.2.4 Below Grade Walls

Below grade walls should include sand or gravel backfill. The sand or gravel backfill should be placed within a zone extending upwardly from the heel of the wall on a 1H:1V slope. The design should also include weepholes and perforated pipe foundation drains to prevent hydrostatic pressures behind the wall. For retaining walls free to rotate without top fixity, an equivalent fluid pressure of 40 pcf should be used for design. For below grade walls with top fixity restrained from rotation such as basement walls, an equivalent fluid pressure of 70 pcf should be used for design.

Earth pressure on the walls will result in a lateral load on the foundations. A passive earth pressure coefficient of 2.45 should be used along with a safety factor of 2.0 for determining the allowable passive pressure in front of the wall. For a unit weight of 125 pcf, this results in an equivalent fluid pressure of 150 pcf. A coefficient of friction of 0.35 can also be used between the concrete foundation and bearing materials when calculating resisting forces.

5.2.5 Grade Supported Floor Slab Recommendations

We recommend on-grade supported floor slabs be isolated from the building foundations and allowed to float free and settle differentially with the building. We have estimated an Effective Modulus of Soil Subgrade Reaction (k) of 125 pci.

The final floor slab design, including the amount of and type of steel reinforcement (welded wire mesh or bar reinforcing) will be dependent on the structural engineer’s evaluation of the final grade slab thickness, concrete compressive strength, and actual slab loadings. Additional design and construction recommendations are provided as follows:

- Proofrolling of the cut subgrade and existing subgrade should be performed to identify soft or unstable soil prior to engineered fill placement. Soft soils should be removed to the extent determined in the field by the AEI Geotechnical Engineer or Technician. Proofrolling of the final floor slab subgrade should also be performed prior to floor slab construction and any defects appropriately repaired as recommended in the field by AEI.

- The floor slab should be supported on a minimum 4-inch compacted layer of free draining granular base material to distribute concentrated loads, improve drainage, and reduce the risk of deterioration of the prepared subgrade during construction. The stone should be kept moist not wet, immediately before placement of concrete to limit differential curing conditions at the top and bottom of the slab.

- Typically, a vapor barrier is recommended to prevent water vapor transmissions that normally have the potential to adversely affect the floor coverings ability to bond to the slab. Based on the region and nature of the subsurface conditions, it is likely that the lack of a vapor barrier will not affect the performance of the slab and floor coverings. Recommendations from ACI 302.1 R 96, "Guide for Concrete Floor and Slab Construction", should be utilized. Joints between slab sections should contain keys or dowels to permit slab rotation but to reduce extreme vertical differential displacements.

5.3 PAVEMENT CONSIDERATIONS

A flexible pavement design was performed for light-duty pavements. ESAL's (equivalent 18-kip single axle loads) of 20,000 were estimated for the light-duty pavement. Since it is our understanding that the heaviest equipment will be a 3,000 lb. front end loader and 5,000 lb. light duty pickup. An assumed CBR value of 5 was utilized for design; and is in general agreement with the CBR value from AEI's previous exploration on site and the prevalent material type encountered.

Our analysis was made using the *AASHTO Guide for Design of Pavement Structures (1993 Edition)* and the following parameters:

- Subgrade Resilient Modulus (M_r)= 7,500 psi
- CBR value = 5
- Initial Serviceability = 4.2
- Terminal Serviceability = 2.0
- Reliability = 90%
- Standard Deviation = 0.49
- ESAL's = 20,000 (Light Duty)
- Design Life = 20 Years
- Drainage Coefficient = 1.0
- Layer Coefficient = 0.44 for asphaltic surface, 0.40 for asphaltic base, 0.14 for crushed aggregate base.

Pavement performance is highly dependent on the support provided by the subgrade that can be greatly impacted by changes in the moisture condition of the subgrade. Measures that reduce the risk of the subgrade becoming saturated should be incorporated into the site design. Pavement slopes should have a minimum gradient of

2% where possible. Pavement edges should be "daylighted", or provided a means where water trapped in the aggregate base can escape by extending the aggregate base course through the sides of drainage channels. The use of underdrains in low areas, and/or spider drains at stormwater catch basins should also be considered within the paved area.

5.3.1 Flexible Pavement

Using the design parameters previously outlined, a recommended minimum *light-duty pavement* would consist of a 6 inch dense graded aggregate (DGA) or crushed stone base (CSB) aggregate base course, a 2 inch bituminous base course, and a 1 ¼ inch bituminous surface course. This design would be appropriate for car parking and travel areas only.

We recommend all bituminous paving materials and paving operations meet the requirements of Division 400 of the KYDOH Standard Specifications, 2008 Edition. Bituminous concrete for surface, base, and binder should meet the requirements for Superpave mixtures.

5.3.2 Aggregate Base Paving

We recommend the Bituminous Pavement aggregate base consist of dense graded aggregate (DGA) or crushed stone base (CSB) meeting the requirements of Section 805 of KDOH Standard Specifications, 2008 Edition. The aggregate base should be placed in maximum 4 inch thick horizontal lifts, with each lift being compacted in accordance with the control strip guidelines set forth in Section 302.03.04A of the KDOH Standard Specifications, 2008 Edition.

5.3.3 Rigid Pavement

We have estimated an Effective Modulus of Subgrade Reaction (k) of 125 pci and various other parameters for light duty rigid pavement design. A minimal recommended heavy-duty rigid pavement design would consist of 4 inches of Crushed Stone Base underlying 5 inches of Portland Cement Concrete Pavement with a 28-day compressive strength of 4,000 psi, and a corresponding modulus of rupture (MR) of 650 psi. The concrete used to construct the pavement should have four to six percent entrained air to improve the concrete's resistance to spalling from saturated freeze-thaw cycles. This section is a light-duty section.

Reinforcement for the rigid pavement should consist of No. 5 bars in both directions at 18 inches on center.

Control joints, filled with a fuel resistant seal to deter liquid intrusion into the subgrade, should be incorporated at 25-foot spacings.

5.4 General Considerations

5.4.1 Construction Monitoring/Testing

Field density and moisture content determinations should be made on each lift of fill with a minimum of one test per 3,000 to 5,000 square feet in building pad areas, one test per lift per 5,000 to 10,000 square feet in other fill areas and one test per lift per 100 to 200 linear feet of utility trench backfill. All construction operations involving earthwork and paving should be performed in the presence of an experienced representative of AEI. The representative would operate under the direct supervision of an AEI Geotechnical Engineer. Some adjustments in the test frequencies may be required based upon the general fill types, changes in the fill material and soil conditions at the time of placement.

Site problems can be avoided or reduced if proper field observation and testing services are provided. We recommend all foundation excavations, proofrolling, site and subgrade preparation, sinkhole remediation, subgrade stabilization (if used), and pavement construction be monitored by AEI. Density tests should be performed to verify compaction and moisture content for all earthwork operations. Field observations should be performed prior to and during concrete placement operations.

5.4.2 Construction Considerations

The surface soils at the site are susceptible to loss of bearing capacity (pumping) by the action of water and construction equipment. Once the subgrade has been stripped, cut to grade and performed adequately during proof-rolling, it should be sealed at the end of each filling day with a smooth drum roller and sloped to sheet drain rainwater. Any material disturbed by rainwater and construction operations should be undercut prior to placing the next lift of fill.

If the project is to begin in the fall and continue through the winter, care must be taken not to place frozen soil, as proper compaction will be impossible. Moisture contents must also be carefully monitored during the winter, as wet soil will be difficult to dry.

5.4.3 Limitations

Construction is accompanied by some risk that internal soil erosion and ground subsidence could occur in the future. During construction, the Contractor and earthwork inspection personnel should be alert for evidence of sinkholes that may form during earthwork or foundation construction. An AEI Geotechnical Engineer should evaluate any sinkholes detected. It is not possible to investigate a site to eliminate all potential future karst related problems. However, the recommendations presented herein can help reduce the risk of construction in karst areas to acceptable levels.

The conclusions and recommendations presented herein are based on information gathered from the borings advanced during this exploration using that degree of care and skill ordinarily exercised under similar circumstances by competent members of the engineering profession. No warranties can be made regarding the continuity of conditions between the borings.

We will retain samples acquired for this project for a period of 30 days subsequent to the submittal date printed on the cover of this report. After this period, the samples will be discarded unless otherwise requested.

APPENDIX A

Boring Layout

LEGEND

SOIL TEST BORING WITH STANDARD PENETRATION TESTS AND UNDISTURBED TUBE SAMPLES

REVISIONS

NO.	DATE	DESCRIPTION

BORING LAYOUT

CLIENT: ANDERSON ENGINEERING OF MN

PROJECT: CNNC WASH BAY AND STORAGE BUILDING

PLANS PREPARED AND SUBMITTED BY:
DESIGN CONSULTANT

AEI
AMERICAN ENGINEEERS, INC.
PROFESSIONAL ENGINEERING

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SCALE: 1"=40'

DATE: 6-3-14

DRAWN BY: J. CHILDRESS

CHECKED BY: D. MITCHELL

FILE: T:/14 Projects/214-085/CNNC Boring Layout.dgn

SHEET: B1

APPENDIX B

Boring Logs

FIELD TESTING PROCEDURES

The general field procedures employed by the Field Services Center are summarized in the following outline. The procedures utilized by the AEI Field Service Center are recognized methods for determining soil and rock distribution and ground water conditions. These methods include geophysical and in situ methods as well as borings.

Soil Borings are drilled to obtain subsurface samples using one of several alternate techniques depending upon the surface conditions. Borings are advanced into the ground using continuous flight augers. At prescribed intervals throughout the boring depths, soil samples are obtained with a split-spoon or thin-walled sampler and sealed in airtight glass jars and labeled. The sampler is first seated 6 inches to penetrate loose cuttings and then driven an additional foot, where possible, with blows from a 140 pound hammer falling 30 inches. The number of blows required to drive the sampler each six-inch increment is recorded. The penetration resistance, or “N-value” is designated as the number of hammer blows required to drive the sampler the final foot and, when properly evaluated, is an index to cohesion for clays and relative density for sands. The split spoon sampling procedures used during the exploration are in general accordance with ASTM D 1586. Split spoon samples are considered to provide *disturbed* samples, yet are appropriate for most engineering applications. Thin-walled (Shelby tube) samples are considered to provide *undisturbed* samples and obtained when warranted in general accordance with ASTM D 1587.

These drilling methods are not capable of penetrating through material designated as “refusal materials.” Refusal, thus indicated, may result from hard cemented soil, soft weathered rock, coarse gravel or boulders, thin rock seams, or the upper surface of sound continuous rock. Core drilling procedures are required to determine the character and continuity of refusal materials.

Core Drilling Procedures for use on refusal materials. Prior to coring, casing is set in the boring through the overburden soils. Refusal materials are then cored according to ASTM D-2113 using a diamond bit attached to the end of a hollow double tube core barrel. This device is rotated at high speeds and the cuttings are brought to the surface by circulating water. Samples of the material penetrated are protected and retained in the inner tube, which is retrieved at the end of each drill run. Upon retrieval of the inner tube the core is recovered, measured and placed in boxes for storage.

The subsurface conditions encountered during drilling are reported on a field test boring record by the driller. The record contains information concerning the boring method, samples attempted and recovered, indications of the presence of various materials such as coarse gravel, cobbles, etc., and observations between samples. Therefore, these boring records contain both factual and interpretive information. The field boring records are on file in our office.

The soil and rock samples plus the field boring records are reviewed by a geotechnical engineer. The engineer classifies the soil in general accordance with the procedures outlined in ASTM D 2487 and D 2488 and prepares the final boring records which are the basis for all evaluations and recommendations.

Representative portions of soil samples are placed in sealed containers and transported to the laboratory. In the laboratory, the samples are examined to verify the driller’s field classifications. Test Boring Records are attached which show the soil descriptions and penetration resistances.

The final boring records represent our interpretation of the contents of the field records based on the results of the engineering examinations and tests of the field samples. These records depict subsurface conditions at the specific locations and at the particular time when drilled. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the subsurface soil and ground water conditions at these boring locations. The lines designate the interface between soil or refusal materials on the records and on profiles represent approximate boundaries. The transition between materials may be gradual. The final boring records are included with this report.

Water table readings are normally taken in conjunction with borings and are recorded on the "Boring Logs". These readings indicate the approximate location of the hydrostatic water table at the time of our field investigation. Where impervious soils are encountered (clayey soils) the amount of water seepage into the boring is small, and it is generally not possible to establish the location of hydrostatic water table through water level readings. The ground water table may also be dependent upon the amount of precipitation at the site during a particular period of time. Fluctuations in the water table should be expected with variations in precipitation, surface run-off, evaporation and other factors.

The time of boring water level reported on the boring records is determined by field crews as the drilling tools are advanced. The boring water level is detected by changes in the drilling rate, soil samples obtained, etc. Additional water table readings are generally obtained at least 24 hours after the borings are completed. The time lag of at least 24 hours is used to permit stabilization of the ground water table which has been disrupted by the drilling operations. The readings are taken by dropping a weighted line down the boring or using as electrical probe to detect the water level surface.

Occasionally the borings will cave-in, preventing water level readings from being obtained or trapping drilling water above the caved-in zone. The cave-in depth is also measured and recorded on the boring records.

Sampling Terminology

Undisturbed Sampling: Thin-walled or Shelby tube samples used for visual examination, classification tests and quantitative laboratory testing. This procedure is described by ASTM D 1587. Each tube, together with the encased soil, is carefully removed from the ground, made airtight and transported to the laboratory. Locations and depths of undisturbed samples are shown on the "Boring Logs."

Bag Sampling: Bulk samples of soil are obtained at selected locations. These samples consist of soil brought to the surface by the drilling augers, or obtained from test pits or the ground surface using hand tools. Samples are placed in bags, with sealed jar samples of the material, and taken to our laboratory for testing where more mass material is required (i.e. Proctors and CBR's). The locations of these samples are indicated on the appropriate logs, or on the Boring Location Plan.

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CLIENT ANDERSON ENGINEERING OF MN LLC
PROJECT NAME CNNC WASH BAY & STORAGE BUILDING
PROJECT NUMBER 214-085
PROJECT LOCATION JESSAMINE COUNTY, KY
DATE STARTED 5/20/14 **COMPLETED** 5/20/14
GROUND ELEVATION 942.2 ft
DRILLER Don Cash
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY Zack Pennington **CHECKED BY** Dusty Barrett
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N-VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	REMARKS
0		Topsoil (15")									
		(CL) Lean CLAY, trace sand, trace gravel, brown, moist, stiff to very stiff									
			ST 1	90		3.75	23	41	22	19	
5			SS 2	93	4-7-8 (15)	2.5	26				
			ST 3	70		3.25					
10			SS 4	93	7-10-9 (19)	3.75	37				
15			SS 5	100	3-5-7 (12)	3.0	41				

Bottom of borehole at 16.5 feet.

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B-2

CLIENT ANDERSON ENGINEERING OF MN LLC
PROJECT NAME CNNC WASH BAY & STORAGE BUILDING
PROJECT NUMBER 214-085
PROJECT LOCATION JESSAMINE COUNTY, KY
DATE STARTED 5/20/14 **COMPLETED** 5/20/14
GROUND ELEVATION 942.2 ft
DRILLER Don Cash
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY Zack Pennington **CHECKED BY** Dusty Barrett
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N-VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	REMARKS
0		Topsoil (14")									
		(CL) Lean CLAY, trace sand, trace gravel, brown, moist, medium stiff to stiff									
			SS 1	100	3-4-6 (10)	1.75	24				
5			ST 2	70		4.5+	22	40	25	15	
			SS 3	100	6-11-14 (25)	4.5+	27				
10			SS 4	93	3-3-4 (7)	2.75	34				
15			SS 5	100	3-4-5 (9)	1.75	32				

Bottom of borehole at 16.5 feet.

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B-3

CLIENT ANDERSON ENGINEERING OF MN LLC
PROJECT NAME CNNC WASH BAY & STORAGE BUILDING
PROJECT NUMBER 214-085
PROJECT LOCATION JESSAMINE COUNTY, KY
DATE STARTED 5/20/14 **COMPLETED** 5/20/14
GROUND ELEVATION 935.2 ft
DRILLER Don Cash
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY Zack Pennington **CHECKED BY** Dusty Barrett
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N-VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	REMARKS
0		Topsoil (14")	SS 1	67	3-5-5 (10)	3.0	27				
		(CL) Lean CLAY, trace gravel, brown, dry to moist, medium stiff	SS 2	40	3-3-5 (8)	1.0	28				
5			ST 3	100		2.25	36				
			SS 4	100	3-2-3 (5)	1.5	41				
10			SS 5	100	2-3-5 (8)	1.5	39				
15		(CL) Lean CLAY, trace gravel, tan to gray, moist, stiff	SS 6	67	4-8-8 (16)	3.25	36				

Bottom of borehole at 16.5 feet.

APPENDIX C

Laboratory Testing Results

CLASSIFICATION SYSTEM FOR SOIL EXPLORATION

COHESIVE SOILS

(Clay, Silt, and Mixtures)

CONSISTENCY	SPT N-VALUE	Qu/Qp (tsf)
Very Soft	2 blows/ft or less	0 – 0.25
Soft	2 to 4 blows/ft	0.25 – 0.49
Medium Stiff	4 to 8 blows/ft	0.50 – 0.99
Stiff	8 to 15 blows/ft	1.00 – 2.00
Very Stiff	15 to 30 blows/ft	2.00 – 4.00
Hard	30 blows/ft or more	> 4.00

PLASTICITY

Degree of Plasticity	Plasticity Index (PI)
Low	0 – 7
Medium	8 – 22
High	over 22

NON-COHESIVE SOILS

(Silt, Sand, Gravel, and Mixtures)

DENSITY	SPT N-VALUE
Very Loose	4 blows/ft or less
Loose	4 to 10 blows/ft
Medium Dense	10 to 30 blows/ft
Dense	30 to 50 blows/ft
Very Dense	50 blows/ft or more

PARTICLE SIZE IDENTIFICATION

Boulders	12 inch diameter or more
Cobbles	3 to 12 inch diameter
Gravel	Coarse – 1 to 3 inch
	Medium – ½ to 1 inch
	Fine – ¼ to ½ inch
Sand	Coarse – 0.6mm to ¼ inch
	Medium – 0.2mm to 0.6mm
	Fine – 0.05mm to 0.2mm
Silt	0.05mm to 0.005mm
Clay	0.005mm

RELATIVE PROPORTIONS

Descriptive Term	Percent
Trace	1 – 10
Trace to Some	11 – 20
Some	21 – 35
And	36 – 50

NOTES

Classification – The Unified Soil Classification System is used to identify soil unless otherwise noted.

Standard "N" Penetration Test (SPT) (ASTM D1586) – Driving a 2-inch O.D., 1 3/8-inch I.D. sampler a distance of 1 foot into undisturbed soil with a 140-pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6-inches to seat the sampler into undisturbed soil, and then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the field drill long (e.g., 10/8/7). On the report log, the Standard Penetration Test result (i.e., the N value) is normally presented and consists of the sum of the 2nd and 3rd penetration counts (i.e., N = 8 + 7 = 15 blows/ft.)

Soil Property Symbols

Qu:	Unconfined Compressive Strength	N:	Standard Penetration Value (see above)
Qp:	Unconfined Comp. Strength (pocket pent.)	omc:	Optimum Moisture content
LL:	Liquid Limit, % (Atterberg Limit)	PL:	Plastic Limit, % (Atterberg Limit)
PI:	Plasticity Index	mdd:	Maximum Dry Density

ATTERBERG LIMITS RESULTS

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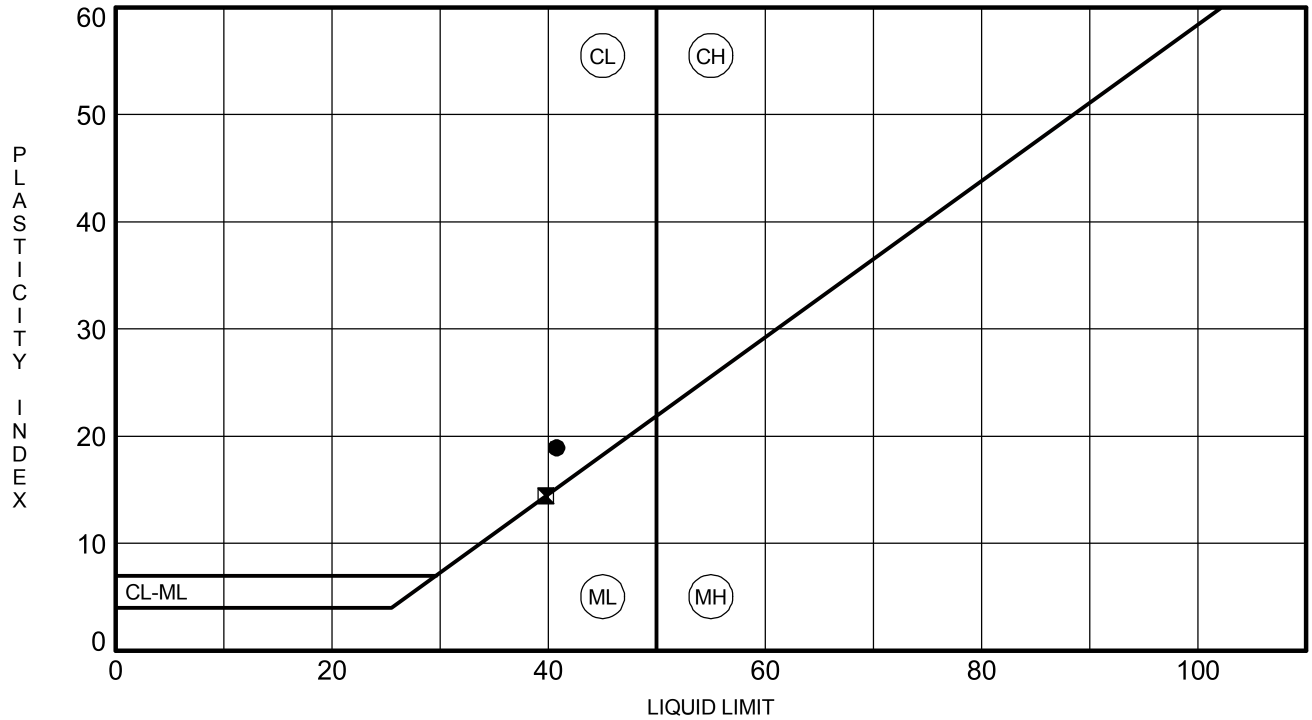
PROJECT NAME CNNC WASH BAY & STORAGE BUILDING

PROJECT NUMBER 214-085

PROJECT LOCATION JESSAMINE COUNTY, KY



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	**B-1**	**2.5**	**41**	**22**	**19**		**Brown lean clay**
⊠	**B-2**	**5.0**	**40**	**25**	**15**		**Brown lean clay**

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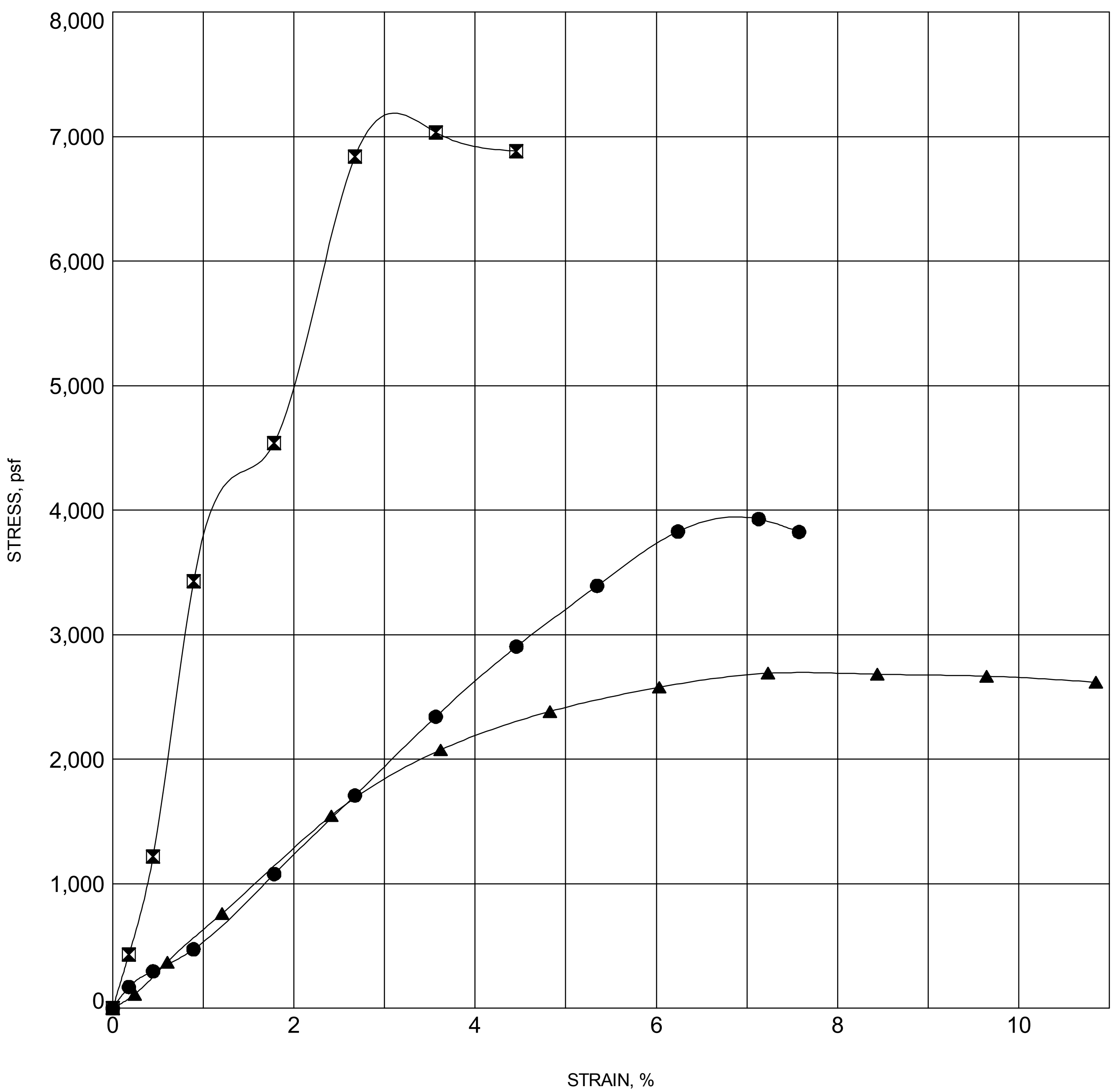
UNCONFINED COMPRESSION TEST

CLIENT ANDERSON ENGINEERING OF MN LLC

PROJECT NAME CNNC WASH BAY & STORAGE BUILDING

PROJECT NUMBER 214-085

PROJECT LOCATION JESSAMINE COUNTY, KY



	BOREHOLE	DEPTH	Classification	γ_d	Qu
●	**B-1**	**2.5**	**Brown lean clay**	**83**	**3930**
⊠	**B-2**	**5.0**	**Brown lean clay**	**85**	**7036**
▲	**B-3**	**5.0**	**Brown lean clay**	**86**	**2691**

UNCONFINED - GINT STD US LAB.GDT - 6/5/14 17:02 - T:\14 PROJECTS\214-085 CNNC\CAMP NELSON NATIONAL CEMETERY.GPJ

Your Geotechnical Engineering Report

To help manage your risks, this information is being provided because subsurface issues are a major cause of construction delays, cost overruns, disputes, and claims.

Geotechnical Services are Performed for Specific Projects, Purposes, and People

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering exploration conducted for an engineer may not fulfill the needs of a contractor or even another engineer. Each geotechnical engineering exploration and report is unique and is prepared solely for the client. No one except the client should rely on the geotechnical engineering report without first consulting with the geotechnical engineer who prepared it. The report should not be applied for any project or purpose except the one originally intended.

Read the Entire Report

To avoid serious problems, the full geotechnical engineering report should be read in its entirety. Do not only read selected sections or the executive summary.

A Unique Set of Project-Specific Factors is the Basis for a Geotechnical Engineering Report

Geotechnical engineers consider a numerous unique, project-specific factors when determining the scope of a study. Typical factors include: the client's goals, objectives, project costs, risk management preferences, proposed structures, structures on site, topography, and other proposed or existing site improvements, such as access roads, parking lots, and utilities. Unless indicated otherwise by the geotechnical engineer who conducted the original exploration, a geotechnical engineering report should not be relied upon if it was:

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

Typical changes that can lessen 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 multi-story hotel to a parking lot
- finished floor elevation, location, orientation, or weight of the proposed structure, anticipated loads or
- project ownership

Geotechnical engineers cannot be held liable or responsible for issues that occur because their report did not take into account development items of which they were not informed. The geotechnical engineer should always be notified of any project changes. Upon notification, it should be requested of the geotechnical engineer to give an assessment of the impact of the project changes.

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that exist at the time of the exploration. A geotechnical engineering report should not be relied upon if its reliability could be in question due to factors such as man-made events as construction on or adjacent to the site, natural events such as floods, earthquakes, or groundwater fluctuation, or time. To determine if a geotechnical report is still reliable, contact the geotechnical engineer. Major problems could be avoided by performing a minimal amount of additional analysis and/or testing.

Most Geotechnical Findings are Professional Opinions

Geotechnical site explorations identify subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field logs and laboratory data and apply their professional judgment to make conclusions about the subsurface conditions throughout the site. Actual subsurface conditions may differ from those indicated in the report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risk associated with unanticipated conditions.

The Recommendations within a Report Are Not Final

Do not put too much faith on the construction recommendations included in the report. The recommendations are not final due to geotechnical engineers developing them principally from judgment and opinion. Only by observing actual subsurface conditions revealed during construction can geotechnical engineers finalize their recommendations. Responsibility and liability cannot be assumed for the recommendations within the report by the geotechnical engineer who developed the report if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject To Misinterpretation

Misinterpretation of geotechnical engineering reports has resulted in costly problems. The risk of misinterpretation can be lowered after the submittal of the final report by having the geotechnical engineer consult with appropriate members of the design team. The geotechnical engineer could also be retained to review crucial parts of the plans and specifications put together by the design team. The geotechnical engineering report can also be misinterpreted by contractors which can result in many problems. By participating in pre-bid and preconstruction meetings and providing construction observations by the geotechnical engineer, many risks can be reduced.

Final Boring Logs Should not be Re-drawn

Geotechnical engineers prepare final boring logs and testing results based on field logs and laboratory data. The logs included in a final geotechnical engineering report should never be redrawn to be included in architectural or design drawings due to errors that could be made. Electronic reproduction is acceptable, along with photographic reproduction, but it should be understood that separating logs from the report can elevate risk.

Contractors Need a Complete Report and Guidance

By limiting what is provided for bid preparation, contractors are not liable for unforeseen subsurface conditions although some owners and design professionals believe the opposite to be true. The complete geotechnical engineering report, accompanied with a cover letter or transmittal, should be provided to contractors to help prevent costly problems. The letter states that the report was not prepared for purposes of bid development and the report's accuracy is limited. Although a fee may be required, encourage the contractors to consult with the geotechnical engineer who prepared the report and/or to conduct additional studies to obtain the specific types of information they need or prefer. A prebid conference involving the owner, geotechnical engineer, and contractors can prove to be very valuable. If needed, allow contractors sufficient time to perform additional studies. Upon doing this you might 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.

Closely Read Responsibility Provisions

Geotechnical engineering is not as exact as other engineering disciplines. This lack of understanding by clients, design professionals, and contractors has created unrealistic expectations that have led to disappointments, claims, and disputes. To minimize such risks, a variety of explanatory provisions may be included in the report by the geotechnical engineer. To help others recognize their own responsibilities and risks, many of these provisions indicate where the geotechnical engineer's responsibilities begin and end. These provisions should be read carefully, questions asked if needed, and the geotechnical engineer should provide satisfactory responses.

Environmental Issues/Concerns are not Covered

Unforeseen environmental issues can lead to project delays or even failures. Geotechnical engineering reports do not usually include environmental findings, conclusions, or recommendations. As with a geotechnical engineering report, do not rely on an environmental report that was prepared for someone else.

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