C-17 Corrosion Control / Fuel Cell Hangar Project
Douglas International Airport - Charlotte, North Carolina

B-3 Final Design Submission

SUPPLEMENTAL INFORMATION

22 September 2017

Contract Number: W9133L-15-D-0002
Task Order Number: D303
PN: FJRP159062
Geotechnical Reports
The attached Geotechnical Investigation Reports that were completed in December-2016 (for Hangar), and March-2017 (for SIM Bldg.) for the NCANG C-17 Corrosion Control / Fuel Cell Hangar Project shall be considered Supplemental information only (e.g. non-binding) for the Contractor's use. The Contract Plans and Specifications have taken into consideration the discussions and recommendations in the reports and as such the Contract Plans and Specifications govern as construction requirements. The Contractor is required to verify all existing conditions necessary for proper installation, construction, and operation of all elements indicted in, and in accordance with the Contract documents.
REPORT
OF
SUBSURFACE EXPLORATION

NORTH CAROLINA AIR NATIONAL GUARD (NCANG) – C-17 HANGAR AND FLIGHT SIMULATION FACILITIES CHARLOTTE, NORTH CAROLINA

ECS PROJECT NO. 08-11868
DECEMBER 5, 2016
REPORT OF SUBSURFACE EXPLORATION

North Carolina Air National Guard (NCANG)
C-17 Hangar and Flight Simulation Facilities
Charlotte, North Carolina

Prepared For:

Mr. David Everett, AIA
Project Manager
Jacobs Global Buildings
1100 North Glebe Road, Suite 500
Arlington, VA 22201

Prepared By:

ECS CAROLINAS, LLP
1812 Center Park Drive
Suite D
Charlotte, NC 28217

ECS Project No:

08-11868

Report Date:

December 5, 2016
December 5, 2016

Mr. David Everett, AIA
Project Manager
Jacobs Global Buildings
1100 North Glebe Road, Suite 500
Arlington, VA 22201

Reference: Report of Subsurface Exploration
North Carolina Air National Guard (NCANG)
C-17 Hangar and Flight Simulation Facilities
Charlotte, North Carolina
ECS Project No: 08-11866

Dear Mr. Everett:

ECS Carolinas, LLP (ECS) has completed the subsurface exploration for the above referenced project. This project was authorized and performed in general accordance with ECS Proposal No. 08-19955P dated July 14, 2016. The purpose of this exploration was to determine the general subsurface conditions at the site and to evaluate those conditions with regard to foundation and floor slab support, along with general site development. This report presents our findings, conclusions, and recommendations for design and construction of the project.

ECS Carolinas, LLP appreciates the opportunity to assist you during this phase of the project. If you have questions concerning this report, please contact our office.

Respectfully,

ECS CAROLINAS, LLP

Michael R. Bailey, P.E.
Project Engineer
NC Registration No. 041906

Lee J. McGuinness, P.E.
Principal Engineer
1. INTRODUCTION

1.1. Project Information

The project site is located at the North Carolina Air National Guard (NCANG) at the Charlotte Douglas International Airport in Charlotte, North Carolina, as shown in the Site Vicinity Map (Figure 1) located in the Appendix. The project includes the construction of a 61,600 square feet C-17 Hangar Facility including an enclosed facility for washing aircraft, performing corrosion control, general maintenance, and shop areas to accommodate maintenance and training. In addition, a separate flight crew simulator training facility to support the C-17 mission will be located in the present location of Building #4. The Flight Simulation Facility is anticipated to be on the order of 10,000 square feet. It is our understanding that the current Building #4 will be razed. Additional components of the project will include the existing facility demolition, site preparation, stormwater detention and drainage systems, utilities, fencing, sidewalks, curbs, communications support, fire protection, flexible paving for POV access road, exterior lighting utilities, and Anti-Terrorism/Force Protection (AT/FP) measures.

The C-17 Hangar will consist of a metal framed building with a concrete slab-on-grade. The facilities may be 1 to 2 stories tall. Maximum column and wall footing loads are on the order of 450 kips and 5.5 kips per linear foot, respectively. The Simulator Building is a low-rise building with anticipated maximum column and wall footing loads on the order of 200 kips and 9.5 kips per linear foot, respectively. Anticipated loading conditions were provided by Jacobs.

1.2. Scope of Services

Our scope of services for this phase of the project included a subsurface exploration with soil test borings, engineering analysis of the foundation support options, and preparation of this report with our recommendations. The subsurface exploration included nine (9) soil test borings (B-1 through B-9) to depths ranging from approximately 15 to 40 feet below the existing ground surface. Approximate boring locations are shown on the Boring Location Diagram (Figure 2) included in the Appendix. The soil borings were performed using a Dietrich D-50 ATV drill rig using continuous-flight, hollow-stem augers.

2. FIELD SERVICES

2.1. Test Locations

The soil boring locations and depths were selected by others and located in the field by ECS using GPS and existing landmarks as reference. The approximate boring locations are shown on the Boring Location Diagram (Figure 2) presented in the Appendix of this report and should be considered accurate only to the degree implied by the method used to obtain them. Ground surface elevations at the boring locations were interpolated using Google Earth and should be considered approximate.

2.2. Soil Test Borings

Nine (9) soil test borings were drilled to evaluate the stratification and engineering properties of the subsurface soils at the project site. Standard Penetration Tests (SPT’s) were performed at designated intervals in general accordance with ASTM D 1586. The Standard Penetration Test is used to provide an index for estimating soil strength and density. In conjunction with the penetration testing, split-barrel soil samples were recovered for soil classification at each test interval. Boring Logs are included in the Appendix.
The drill crew also maintained a field log of the soils encountered at each of the boring locations. After recovery, each sample was removed from the auger and visually classified. Representative portions of each sample were then sealed and brought to our laboratory in Charlotte, North Carolina for further visual examination and potential laboratory testing. Groundwater measurements were attempted at the termination of drilling at each boring location.

2.3. Seasonal High Water Table

A licensed soil scientist was onsite during the drilling of the borings (B-1 through B-9) in order to determine the depth of the seasonal high water table (SHWT). The characteristics of the soil were observed, including texture, depth, slope, the presence of restrictive horizons, depth to seasonal high water table, etc. The assessment was conducted in accordance with current soil science practices and technology. The results of the SHWT study are included in the Appendix of this report.

3. LABORATORY SERVICES

Soil samples were collected from the borings and examined in our laboratory to check field classifications and to determine pertinent engineering properties. Data obtained from the borings, our visual/manual examinations, and laboratory testing are included on the respective boring logs in the Appendix.

3.1 Soil Classification

A geotechnical staff professional classified each soil sample on the basis of color, texture, and plasticity characteristics in general accordance with the Unified Soil Classification System (USCS). The staff professional grouped the various soil types into the major zones noted on the boring logs. The stratification lines designating the interfaces between earth materials on the boring logs and profiles are approximate; in situ, the transition between strata may be gradual in both the vertical and horizontal directions. The results of the visual classifications are presented on the Boring Logs included in the Appendix.

3.2 Laboratory Testing

In addition to visual classification, ECS performed four (4) natural moisture content tests and two (2) Atterberg Limits tests. The laboratory testing was performed in general accordance with the applicable ASTM standards. The results of the laboratory testing are presented on the respective Boring Log and presented within the Laboratory Testing Summary Sheet included within the Appendix.

4. SITE AND SUBSURFACE FINDINGS

4.1. Area Geology

The site is located in the Piedmont Physiographic Province of North Carolina. The native soils in the Piedmont Province consist mainly of residuum with underlying saprolites weathered from the parent bedrock, which can be found in both weathered and unweathered states. Although the surficial materials normally retain the structure of the original parent bedrock, they typically have a much lower density and exhibit strengths and other engineering properties typical of soil. In a mature weathering profile of the Piedmont Province, the soils are generally found to be finer grained at the surface where more extensive weathering has occurred. The particle size of the
soils generally becomes more granular with increasing depth and gradually changes first to weathered and finally to unweathered parent bedrock. The mineral composition of the parent rock and the environment in which weathering occurs largely control the resulting soil's engineering characteristics. The residual soils are the product of the weathering of the parent bedrock.

In addition, it is apparent that the natural geology within the site has been modified in the past by grading that included placement of fill materials. The quality of man-made fills can vary significantly, and it is often difficult to access the engineering properties of existing tests performed in soil test borings and the degree of compaction of existing fill soils; however, a qualitative assessment of existing fills can sometimes be made based on the N-values obtained and observations of the materials sampled in the test borings.

4.2. Subsurface Conditions

The subsurface conditions at the site, as indicated by the borings, generally consisted of fill and residual soil to the depths explored. The generalized subsurface conditions are described below. For soil stratification at a particular test location, the respective Boring Log found in the Appendix should be reviewed.

Varying amounts of asphalt was observed at the ground surface at Borings B-2, B-3, B-4, B-6, B-7, B-8, and B-9. Additionally, approximately 10 inches of concrete was observed beneath the asphalt at Borings B-3 and B-5. Approximately 10 inches of concrete was observed at the ground surface at Borings B-1 and B-5. The surficial material depths provided in this report and on the individual Boring Logs are based on driller observations and should be considered approximate. Please note that these reported values should not be used in determining removal quantities.

Fill soils were encountered below the surficial material at each of the boring locations. The fill extended to depths ranging from approximately 3 to 5½ feet below the existing ground surface. The fill soils encountered generally consisted of Sandy CLAY (CL), Plastic CLAY (CH), Elastic SILT (MH) and Sandy SILT (ML) exhibiting SPT N-values ranging from 3 to 13 blows per foot (bpf).

Residual soil was encountered below the fill soils at each of the boring locations. Residual soils are formed by the in-place chemical and mechanical weathering of the parent bedrock. The residual soils encountered in the borings generally consisted of Sandy SILT (ML), Elastic SILT (MH), Sandy CLAY (CL), and Silty SAND (SM), exhibiting SPT N-values ranging from 4 to 42 blows per foot (bpf) with a majority of the N-values ranging between 6 and 16 bpf. Each of the boring locations was terminated in the residual soils at depths ranging from 15 to 40 feet below the existing ground surface.

4.3. Groundwater Observations

Groundwater measurements were attempted at the termination of drilling at the time of our exploration. Groundwater was encountered at Borings B-6 and B-8 at depths of approximately 34.1 and 29.5 feet below the existing ground surface, respectively. Each of the remaining boring locations was dry when the groundwater measurements were taken. Borehole cave-in depths were observed at each boring location at depths ranging from approximately 16.1 to 34.7 feet below the existing ground surface. Cave-in of a soil test boring can be caused by groundwater hydrostatic pressure, weak soil layers, and/or drilling activities (i.e. drilling fluid circulation or advancement of bit).
Fluctuations in the groundwater elevation should be expected depending on precipitation, run-off, utility leaks, and other factors not evident at the time of our evaluation. Normally, highest groundwater levels occur in late winter and spring and the lowest levels occur in late summer and fall. Depending on time of construction, groundwater may be encountered at shallower depths and locations not explored during this study. If encountered during construction, engineering personnel from our office should be notified immediately.

4.4. Laboratory Test Results

Moisture content test results of the sampled soils range from approximately 25.5 to 38.0 percent. Atterberg Limits testing was performed on a selected soil samples from B-1 and B-4 resulting in liquid limits (LL) ranging from 48 to 58 and plasticity indices (PI) ranging from 14-15. The portion of the samples tested was USCS classified as Sandy SILT (ML) for B-1 and Elastic SILT (MH) for B-4. For laboratory test results at a particular test location, the Laboratory Test Summary sheet found in the Appendix should be reviewed.

5. CONCLUSIONS AND RECOMMENDATIONS

The borings performed at this site represent the subsurface conditions at the location of the borings. Due to inconsistencies associated with the prevailing geology and existing fill soils, there can be changes in the subsurface conditions over relatively short distances that have not been disclosed by the results of the test location performed. Consequently, there may be undisclosed subsurface conditions that require special treatment or additional preparation once these conditions are revealed during construction.

Our evaluation of foundation support conditions has been based on our understanding of the site, project information and the data obtained in our exploration. The general subsurface conditions utilized in our foundation evaluation have been based on interpolation of subsurface data between and away from the borings. In evaluating the boring data, we have examined previous correlations between penetration resistance values and foundation bearing pressures observed in soil conditions similar to those at your site.

5.1. Existing Fill

Fill soils were encountered at each of the boring locations and extended to depths ranging from approximately 3 to 5½ feet below the existing ground surface. ECS was not provided with documentation of the previous earthwork activities, thus the fill should be considered undocumented.

Undocumented fill poses risks associated with undetected deleterious inclusions within the fill and/or deleterious materials at the virgin ground fill interface that are covered by the fill. Deleterious materials can consist of significant amount of organics derived from organic rich stripplings, rubbish, construction or demolition debris, stumps and roots, and logs. If these materials are covered over by or are within undocumented fill, the organic materials tend to decompose slowly in the anaerobic conditions in or under the fill. Decomposition can occur over periods ranging from several years to several decades. As the organic materials decompose, a void is created which can create soft conditions and even subsidence in areas above the organics. Where these types of conditions exist under or within undocumented fill, they are sometimes in discreet pockets that can go undetected by normal subsurface exploration techniques, i.e., soil test borings and test pits.
Due to the depth and quality of the fill soils, we recommend removing and replacing the undocumented fill soils located within the building pad. The excavation should extend a minimum horizontal distance equal to the depth of excavation plus 5 feet. The risk of poor pavement performance can be lowered with the implementation of geogrid reinforcement to the pavement cross section or the use of a more robust pavement design; however, the pavement recommendations provided herein do not include the use of geogrids reinforcement. These methods limit the potential for poor pavement performance, but do not eliminate the risk.

5.2. Moisture Sensitive Soils (MH/CH)

Elastic SILTS (MH) and Plastic CLAYS (CH) were encountered within the fill soils at Borings B-2 and B-4 to depths of approximately 3 feet below ground surface and Elastic SILTS (MH) were encountered within the residual soils at Boring B-4 to a depth of approximately 5½ feet below ground surface. Soils classified as MH/CH are fine-grained and have a Liquid Limit greater than 50 percent. Additionally, MH/CH soils are moisture sensitive soils and tend to shrink and swell with moisture variations.

Residual MH soils with a plasticity index greater than 30 and CH soils should not be used for direct support of project foundations, slabs-on-grade, or pavements. Residual MH soils (PI's greater than 30) and CH soils encountered within proposed structural areas should be undercut and replaced with low plasticity engineered fill to a minimum depth of 2 feet below foundations and 2 feet below subgrade elevations in slab and pavement areas. Upon completion of the removal, the resulting subgrade soils should be evaluated for stability prior to placement of engineered fill.

5.3. Seismic Site Class

The North Carolina Building Code (NCBC) requires that the stiffness of the top 100-ft of soil profile be evaluated in determining a site seismic classification. The method for determining the Site Class is presented in Section 1615 of the code. The seismic Site Class is typically determined by calculating a weighted average of the N-values or shear wave velocities recorded to a depth of 100 feet within the proposed building footprint. Based on the SPT N-values obtained within the drilled depth of borings, a seismic site class of “D” is considered appropriate for this project.

5.4. Structure Foundations

The following section presents specific recommendations with regard to the foundation design of the proposed C-17 Hangar and Simulation Building. Each foundation recommendation has been made provided the site preparation and fill recommendations outlined within this report are implemented.

For this project, minimum wall and column footing dimensions of 18 and 24 inches, respectively, should be maintained to reduce the possibility of a localized, “punching” type, shear failure. Exterior foundations and foundations in unheated areas should be embedded deep enough below exterior grades to reduce potential movements from frost action or excessive drying shrinkage. For this region, we recommend footings bear at least 18 inches below finished grade.

As previously mentioned, MH soils with a plasticity index greater than 30 or CH soils should not be used for direct support of foundations or slabs-on-grade. MH and CH soils encountered within 2 feet of soil subgrade should be undercut and replaced with approved engineered fill to a minimum depth of 2 feet below foundations provided that the resulting subgrade is stable.
C-17 Hangar and Simulator Buildings

The proposed C-17 Hangar and Simulator Buildings can be adequately supported on a shallow foundation system consisting of spread footings bearing on firm undisturbed low plasticity residual soil or newly-placed engineered fill. A bearing capacity of up to 2,500 psf is recommended for foundations bearing on firm undisturbed low plasticity residual soil or newly-placed engineered fill.

Total settlement is anticipated to be less than 1 inch, while differential settlement between columns is anticipated to be less than ½ inch within 30 feet for shallow foundations bearing on low plasticity residual soil or newly-placed structural fill. Foundation geometry, loading conditions, and/or bearing strata different than those described in this report may result in magnitudes of settlement inconsistent with the previous estimates.

5.5. Slab-On-Grade Support

Slabs-on-grade can be adequately supported on undisturbed low plasticity residual soils or newly-placed engineered fill provided the site preparation (Section 6.1) and fill recommendations (Section 6.2) outlined herein are implemented. For a properly prepared site, a modulus of subgrade reaction \( (k_s) \) for the soil of 90 pounds per cubic inch for the soil can be used. This value is representative of a 30 inch diameter loaded area and may need to be adjusted depending on the size and shape of the loaded area depending on the method of structural analysis.

We recommend the slabs-on-grade be underlain by a minimum of 4 inches of granular material having a maximum aggregate size of 1½ inches and no more than 2 percent fines. Prior to placing the granular material, the floor subgrade soil should be properly compacted, proofrolled, and free of standing water, mud, and frozen soil. A properly designed and constructed capillary break layer can often eliminate the need for a moisture retarder and can assist in more uniform curing of concrete. If a vapor retarder is considered to provide additional moisture protection, special attention should be given to the surface curing of the slabs to minimize uneven drying of the slabs and associated cracking and/or slab curling. The use of a blotter or cushion layer above the vapor retarder can also be considered for project specific reasons.

Please refer to ACI 302.1R96 Guide for Concrete Floor and Slab Construction and ASTM E 1643 Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs for additional guidance on this issue.

ECS recommends that the slab be isolated from the footings so differential settlement of the structure will not induce shear stresses on the floor slab. Also, in order to minimize the crack width of shrinkage cracks that may develop near the surface of the slab, we recommend mesh reinforcement as a minimum be included in the design of the floor slab. For maximum effectiveness, temperature and shrinkage reinforcements in slabs on ground should be positioned in the upper third of the slab thickness. The Wire Reinforcement Institute recommends the mesh reinforcement be placed 2 inches below the slab surface or upper one-third of slab thickness, whichever is closer to the surface.

Adequate construction joints, contraction joints and isolation joints should also be provided in the slab to reduce the impacts of cracking and shrinkage. Please refer to ACI 302.1R96 Guide for Concrete Floor and Slab Construction for additional information regarding concrete slab joint design.
5.6. Pavement Considerations

For the design and construction of exterior pavements for vehicular traffic only, the subgrades should be prepared in accordance with the recommendations in the “Site and Subgrade Preparation” and “Engineered Fill” sections of this report. This section does not include pavement recommendations for plane traffic or runway design. The plane apron design has been included in the appendix of this report.

Undisturbed low-plasticity natural soils or newly placed engineered fill can provide adequate support for a pavement structure designed for appropriate subgrade strength and traffic characteristics. Additionally, pavements can be supported on approved existing fill provided that the owner has accepted the risk associated with the existing fill as previously described in this report.

An important consideration with the design and construction of pavements is surface and subsurface drainage. Where standing water develops, either on the pavement surface or within the aggregate base course layer, softening of the subgrades and other problems related to the deterioration of the pavement can be expected. Furthermore, good drainage should help reduce the possibility of the subgrade materials becoming saturated during the normal service period of the pavement.

Based on our past experience with similar developments and subsurface conditions, we present the following design pavement sections, provided the recommendations contained in this report are strictly followed. Based on the soil types encountered in the soil test borings and provided the site grading recommendations outlined herein are implemented, we recommend a CBR value of 4 be used in design of the project pavements. Based upon our previous experience with similar projects, ECS has estimated the provided pavement sections based upon a 20 year life, with equivalent axle loadings of approximately 25,000 and 200,000 ESALs for light-duty and heavy-duty pavements, respectively.

<table>
<thead>
<tr>
<th>PAVEMENT SECTION RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Material Designation</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Asphalt Surface Course (S9.5B)</td>
</tr>
<tr>
<td>Intermediate Coarse (I19.0B)</td>
</tr>
<tr>
<td>Portland Cement Concrete</td>
</tr>
<tr>
<td>Aggregate Base Course</td>
</tr>
</tbody>
</table>

ECS should be allowed to review these recommendations and make appropriate revisions based upon the formulation of the final traffic design criteria for the project. It is important to note that the design sections do not account for construction traffic loading. In addition, individual retailers may have minimum pavement sections that were not accounted for during our pavement evaluation.

The aggregate base course materials beneath pavements and sidewalks should be compacted to at least 95 percent of their modified Proctor maximum dry density (ASTM D 1557). Front-loading trash dumpsters frequently impose concentrated front-wheel loads on pavements during loading. This type of loading typically results in rutting of bituminous pavements and ultimately pavement failures and costly repairs. Similarly, drive-thru lanes also create severe risk of rutting and scuffing. Therefore, we suggest that the pavements in trash pickup and drive-thru areas
utilize the aforementioned Portland Cement Concrete (PCC) pavement section. It may be prudent to use rigid pavement sections in all areas planned for heavy truck traffic. Such a PCC section would typically consist of 6 inches of 4,000 psi, air-entrained concrete over not less than 6 inches of compacted aggregate base course. Appropriate steel reinforcing such as welded-wire fabric and/or rebar and jointing should also be incorporated into the design of all PCC pavements.

It should be noted that these design recommendations may not satisfy the North Carolina Department of Transportation traffic guidelines. Any roadways constructed for public use and to be dedicated to the State for repair and maintenance must be designed in accordance with the State requirements.

5.7. Below Grade Excavation

Information regarding the depth of the planned underground utilities or site grades was not provided at the time of this report. Based on the results of the soil test borings and the anticipated cut/fill depths, we do not anticipate difficult excavation will be encountered.

As noted in the Geology section of this report, the weathering process in the Piedmont can be erratic and significant variations of the depths of the more dense materials can occur in relatively short distances. In some cases, isolated boulders or thin rock seams may be present in the soil matrix.

5.8. Cut and Fill Slopes

ECS does not anticipate cut or fill slopes greater than 10 feet in height. We recommend that permanent cut slopes with less than 10 feet crest height through undisturbed residual soils be constructed at 2:1 (horizontal: vertical) or flatter. Permanent fill slopes less than 10 feet tall may be constructed using engineered fill at a slope of 2.5:1 or flatter. However, a slope of 3:1 or flatter may be desirable to permit establishment of vegetation, safe mowing, and maintenance. The surface of all cut and fill slopes should be adequately compacted. All permanent slopes should be protected using vegetation or other means to prevent erosion.

A slope stability analysis should be performed on cut and fill slopes exceeding 10 feet in height to determine a slope inclination resulting in a factor of safety greater than 1.4. Upon finalization of site civil drawings, ECS should be contacted to perform slope stability analysis and determine if further exploration is necessary.

The outside face of building foundations and the edges of pavements placed near slopes should be located an appropriate distance from the slope. Buildings or pavements placed at the top of fill slopes should be placed a distance equal to at least 1/3 of the height of the slope behind the crest of the slope. Buildings or pavements near the bottom of a slope should be located at least 1/2 of the height of the slope from the toe of the slope. Slopes with structures located closer than these limits or slopes taller than the height limits indicated should be specifically evaluated by the geotechnical engineer and may require approval from the building code official.

Temporary slopes in confined or open excavations should perform satisfactorily at inclinations of 2:1. All excavations should conform to applicable OSHA regulations. Appropriately sized ditches should run above and parallel to the crest of all permanent slopes to divert surface runoff away from the slope face. To aid in obtaining proper compaction on the slope face, the fill slopes should be overbuilt with properly compacted structural fill and then excavated back to the proposed grades.
5.9. Lateral Earth Pressures

ECS understands that below grade walls/loading dock walls may be utilized for the building. Specifics regarding the below grade walls (i.e. location, height, length, loading, etc.) were unknown at the time of this report. Below grade walls should be designed to withstand the lateral earth pressures exerted upon them, and to resist additional lateral pressures generated by surcharge loads such as traffic loads, adjacent slab loads or from foundations bearing behind the walls.

For wall conditions where wall movement cannot be tolerated or where the wall is restrained at the top, such as the loading dock walls, the “At Rest” earth pressure should be used. For wall conditions where outward wall movement on the order of 0.5 percent of the wall height can be tolerated, the “Active” earth pressure should be used. In the design of loading dock walls to restrain compacted backfill, engineered fill or in-situ residual soils, the coefficient of lateral earth pressure can be used to determine lateral earth pressure loads. Please note that the values presented below are for on-site ML and SM materials. If the wall backfill is imported to the site, ECS should be contacted to review the lateral earth pressure coefficients provided. Moderately to highly elastic/plastic soils (CL, MH, and CH) should not be utilized behind earth retaining structures.

<table>
<thead>
<tr>
<th>Soil Parameter</th>
<th>Coefficient of Lateral Earth Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>“At Rest” Earth Pressure (K₀)</td>
<td>0.56</td>
</tr>
<tr>
<td>“Active” Earth Pressure (Kₐ)</td>
<td>0.39</td>
</tr>
<tr>
<td>“Passive” Earth Pressure (Kᵢ)</td>
<td>2.56</td>
</tr>
</tbody>
</table>

The lateral earth pressure values presented above assume level backfill fill behind the wall, and do not account for hydrostatic pressures against the walls or surcharge loads from overlying or nearby construction.

Resistance to sliding can be provided by friction between the bottom of the wall foundation and the underlying soils and by passive resistance of soil adjacent to the wall foundation. The passive resistance should only be used in situations where the soil adjacent to the toe of the wall will not be eroded or otherwise removed in the future. A coefficient of friction of 0.35 for concrete bearing on approved soils is recommended.

Drainage behind freestanding retaining walls is considered essential towards relieving hydrostatic pressures. Drainage can be established by providing a perimeter drainage system located just above the below grade/retaining wall footings which discharges by gravity flow to a suitable outlet. This system should consist of “perforated pipe” or “porous wall”, closed-joint drain lines. These drain lines should be surrounded by a minimum 6 inches of free-draining, granular filter material having a gradation compatible with the size of the openings utilized in the drain lines and the surrounding soils to be retained, or by gravel wrapped in filter fabric. The space between the interior face of the wall and the earth fill should be backfilled with a granular fill of porous quality or better extending from the perimeter drainage system to just below the top of the wall. To prevent frost heave effects from acting against these walls, the granular backfill should extend a minimum of 12 horizontal inches behind the wall. The granular backfill should be capped with pavement, concrete, or a 12-inch layer of low permeable silt or clay to minimize the seepage of water into that backfill from the surface. The ground surface adjacent to the below-grade walls should be kept properly graded to prevent ponding of water adjacent to the walls.
5.10. Mechanically Stabilized Earth (MSE) Wall Design

We understand that retaining walls may be utilized on this project. The performance of the MSE Walls is highly dependent upon sound design and construction practices. The design of the MSE Walls shall consider internal, external and global stability. The following table summarizes the recommended minimum factors of safety (FS) for static design criteria, as recommended by the National Concrete Masonry Association (NCMA).

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Static</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Sliding</td>
<td>1.5</td>
</tr>
<tr>
<td>Overturning</td>
<td>2.0</td>
</tr>
<tr>
<td>Internal Sliding</td>
<td>1.5</td>
</tr>
<tr>
<td>Tensile Overstress</td>
<td>1.5</td>
</tr>
<tr>
<td>Pullout</td>
<td>1.5</td>
</tr>
<tr>
<td>Connection</td>
<td>1.5</td>
</tr>
<tr>
<td>Internal Compound Stability</td>
<td>1.3</td>
</tr>
<tr>
<td>Bearing Capacity</td>
<td>2.0</td>
</tr>
<tr>
<td>Global Stability</td>
<td>1.3 to 1.5</td>
</tr>
</tbody>
</table>

The results of the required internal and geotechnical stability analyses are highly dependent upon the engineering properties of the retained, reinforced and foundation zone materials. Consequently, the design of the MSE Walls requires the assignment of specific engineering properties to the reinforced, retained and foundation zone soils. Required for design are the soil's total in-place unit weight and peak effective friction angle and cohesion. However, cohesion is typically ignored for all materials except the foundation zone materials.

Maintaining the integrity of the reinforced zone is critical to wall performance. Any below grade utilities should be situated outside the reinforced zone to limit potential conflicts between the reinforcement and below grade structures. The wall designer should contemplate the location and use of any below grade utilities during the design process, and should coordinate with the Civil Engineer where possible to relocate the utilities outside of the reinforced zone.

ECS has completed a subsurface exploration and laboratory testing program. The incorporation of subsurface information and laboratory test results into the design of a MSE Wall requires the evaluation of available information to develop design parameters. The laboratory results provided in this report represent the soil parameters for the materials selected and should be considered for information only. Interpretation of laboratory test results and development of design parameters (i.e. shear strength) is solely the responsibility of the MSE Wall designer.

Regardless of MSE Wall geometry, it is desirable to use select fill materials within the reinforced fill zone of the MSE Wall. Granular fill is typically easier to place and compact, have enhanced drainage characteristics, have greater strengths and are less susceptible to long term movements (creep). The following gradation is recommended for the reinforced zone fill materials.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 inch</td>
<td>100</td>
</tr>
<tr>
<td>No. 4</td>
<td>20 to 100</td>
</tr>
<tr>
<td>No. 40</td>
<td>0 to 60</td>
</tr>
<tr>
<td>No. 200</td>
<td>0 to 35</td>
</tr>
</tbody>
</table>
The wall designer should specify allowable backfill material including unit weight, relative compaction and shear strength requirements as well as a testing frequency to verify compaction and design shear strength properties.

In some instances it may be acceptable for low plasticity (LL<50 and PI<20) fine grain soils to be used in MSE Wall construction. However, the use of fine-grained soils can result in poor internal and surface drainage, as well as time dependent movements of the MSE Wall and related components. If fine grained low plasticity soils are used for MSE Wall backfill, we recommend additional internal drainage such as a chimney drain should be incorporated into the final wall design.

The preceding paragraphs and tables are intended to provide a general overview of the design and construction of the MSE Walls. Specific guidance regarding the design and construction of MSE Walls can be found in the current edition of the NCMA Design Manual for Segmental Retaining Walls. The information provided above does not alleviate the MSE Wall designer from any aspect of the design including selection of shear strength parameters, internal wall stability, external wall stability, global stability or settlement estimates.

6. CONSTRUCTION CONSIDERATIONS

6.1. Site Preparation

Prior to construction, the proposed construction area should be stripped of all topsoil, organic material, existing undocumented fill within the building footprint, and other soft or unsuitable material. Upon completion of these razing and stripping operations, the exposed subgrade in areas to receive fill should be proofrolled with a loaded dump truck or similar pneumatic-tired vehicle having a loaded weight of approximately 25 tons. After excavation, the exposed subgrades in cut areas should be similarly proofrolled.

Proofrolling operations should be performed under the observation of a geotechnical engineer or his authorized representative. The proofrolling should consist of two (2) complete passes of the exposed areas, with each pass being in a direction perpendicular to the preceding one. Any areas which deflect, rut or pump during the proofrolling, and fail to be remedied with successive passes, should be undercut to suitable soils and backfilled with compacted fill.

The ability to dry wet soils, and therefore the ability to use them for fill, will likely be enhanced if earthwork is performed during summer or early fall. If earthwork is performed during winter or after appreciable rainfall then subgrades may be unstable due to wet soil conditions, which could increase the amount of undercutting required. Drying of wet soils, if encountered, may be accomplished by spreading and disking or by other mechanical or chemical means. Drying and stabilizing of wet soils by chemical means can generally be achieved by the addition of 2 to 4% lime or cement; however, each case should be analyzed based on soil types and conditions during construction.

6.2. Fill Material and Placement

The project fill should be soil that has less than five percent organic content and a liquid limit and plasticity index less than 50 and 30, respectively. Soils with Unified Soil Classification System group symbols of SP, SW, SM, SC, and ML are generally suitable for use as project fill. Soils with USCS group symbol of CL that meet the restrictions for liquid limit and plasticity index are also suitable for use as project fill. Soils with USCS group symbol of MH or CH (high plasticity soil) or corrosive soils are generally not suitable for use as project fill.
The fill should exhibit a maximum dry density of at least 90 pounds per cubic foot, as determined by a standard Proctor compaction test (ASTM D-698). We recommend that moisture control limits of -3 to +2 percent of the optimum moisture content be used for placement of project fill with the added requirement that fill soils placed wet of optimum remain stable under heavy pneumatic-tired construction traffic. During site grading, some moisture modification (drying and/or wetting) of the onsite soils will likely be required.

Project fill should be compacted to at least 95 percent of its standard Proctor maximum dry density except within 24 inches of finished soil subgrade elevation beneath slab-on-grade and pavements. Within the top 24 inches of finished soil subgrade elevation beneath slab on grade and pavements, the approved project fill should be compacted to at least 100 percent of its standard Proctor maximum dry density. Aggregate base course (ABC stone) should be in accordance with NCDOT 2012 Standard Specifications Section 1005 Table 1005-1 and should be compacted to 95 percent of modified Proctor maximum dry density. However, for isolated excavations around footing locations or within utility excavations, a hand tamper will likely be required. ECS recommends that field density tests be performed on the fill as it is being placed, at a frequency determined by an experienced geotechnical engineer, to verify that proper compaction is achieved.

The maximum loose lift thickness depends upon the type of compaction equipment used. The table below provides maximum loose lifts that may be placed based on compaction equipment.

### LIFT THICKNESS RECOMMENDATIONS

<table>
<thead>
<tr>
<th>Equipment</th>
<th>Maximum Loose Lift Thickness, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large, Self-Propelled Equipment (CAT 815, etc.)</td>
<td>8</td>
</tr>
<tr>
<td>Small, Self-Propelled or Remote Controlled (Rammax, etc.)</td>
<td>6</td>
</tr>
<tr>
<td>Hand Operated (Plate Tamps, Jumping Jacks, Wacker-Packers)</td>
<td>4</td>
</tr>
</tbody>
</table>

ECS recommends that fill operations be observed and tested by an engineering technician to determine if compaction requirements are being met. The testing agency should perform a sufficient number of tests to confirm that compaction is being achieved. For mass grading operations we recommend a minimum of one density test per 2,500 SF per lift of fill placed or per 1 foot of fill thickness, whichever results in more tests. When dry, the majority of the site soil should provide adequate subgrade support for fill placement and construction operations. When wet, the soil may degrade quickly with disturbance from construction traffic. Good site drainage should be maintained during earthwork operations to prevent ponding water on exposed subgrades.

We recommend at least one test per 1 foot thickness of fill for every 100 linear ft of utility trench backfill. Where fill will be placed on existing slopes, we recommend that benches be cut in the existing slope to accept the new fill. All fill slopes should be overbuilt and then cut back to expose compacted material on the slope face. While compacting adjacent to below-grade walls, heavy construction equipment should maintain a horizontal distance of 1(H):1(V). If this minimum distance cannot be maintained, the compaction equipment should run perpendicular, not parallel to, the long axis of the wall.
6.3. Foundation Construction & Testing

Foundation excavations should be tested to confirm adequate bearing prior to installation of reinforcing steel or placement of concrete. Unsuitable soils should be undercut to firm soils and the undercut excavations should be backfilled with compacted controlled fill. Exposure to the environment may weaken the soils at the footing bearing level if the foundation excavations remain open for too long a time; therefore, foundation concrete should be placed the same day that foundations are excavated. If the bearing soils are softened by surface water intrusion or exposure, the softened soils must be removed from the foundation excavation bottom immediately prior to placement of concrete. If the excavation must remain open overnight, or if rainfall becomes imminent while the bearing soils are exposed, a 1- to 3-inch thick "mud mat" of "lean" concrete may be placed on the bearing surface to protect the bearing soils. The mud mat should not be placed until the bearing soils have been tested for adequate bearing capacity. Foundations undercut should be backfilled with engineered fill. If lean concrete is placed within the undercut zone, the foundation footprint does not require oversizing. However, if soil or ABC stone is used in lieu of lean concrete, the foundation footprint should be oversized on a 1V:1H scale.

We recommend testing all shallow foundations to confirm the presence of foundation materials similar to those assumed in the design. We recommend the testing consist of hand auger borings with Dynamic Cone Penetrometer (DCP) testing performed by an engineer or engineering technician.

7. GENERAL COMMENTS

The borings performed at this site represent the subsurface conditions at the location of the borings only. Due to the prevailing geology and presence of existing undocumented fill, changes in the subsurface conditions can occur over relatively short distances that have not been disclosed by the results of the borings performed. Consequently, there may be undisclosed subsurface conditions that require special treatment or additional preparation once these conditions are revealed during construction.

Our evaluation of foundation support conditions has been based on our understanding of the site and project information and the data obtained in our exploration. The general subsurface conditions utilized in our foundation evaluation have been based on interpolation of subsurface data between and away from the test holes. If the project information is incorrect or if the structure locations (horizontal or vertical) and/or dimensions are changed, please contact us so that our recommendations can be reviewed. The discovery of any site or subsurface conditions during construction which deviate from the data outlined in this exploration should be reported to us for our evaluation. The assessment of site environmental conditions for the presence of pollutants in the soil, rock, and groundwater of the site was beyond the scope of this exploration.

The recommendations outlined herein should not be construed to address moisture or water intrusion effects after construction is completed. Proper design of landscaping, surface and subsurface water control measures are required to properly address these issues. In addition, proper operation and maintenance of building systems is required to minimize the effects of moisture or water intrusion. The design, construction, operation, and maintenance of waterproofing and dampproofing systems are beyond the scope of services for this project.
APPENDIX

Site Vicinity Map
Boring Location Diagram
Laboratory Testing Summary
Borelogs
Pavement Design Analysis
SHWT Report
ASFE Documents
## Laboratory Testing Summary

<table>
<thead>
<tr>
<th>Sample Source</th>
<th>Sample Number</th>
<th>Depth (feet)</th>
<th>MC (%)</th>
<th>Soil Type</th>
<th>Atterberg Limits</th>
<th>Percent Passing No. 200 Sieve</th>
<th>Maximum Density (pcf)</th>
<th>Optimum Moisture (%)</th>
<th>CBR Value</th>
<th>Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>S-2</td>
<td>3.50 - 5.00</td>
<td>25.5</td>
<td>ML</td>
<td>48</td>
<td>33</td>
<td>15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-2</td>
<td>S-2</td>
<td>3.50 - 5.00</td>
<td>27.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-4</td>
<td>S-2</td>
<td>3.50 - 5.00</td>
<td>38.0</td>
<td>MH</td>
<td>58</td>
<td>44</td>
<td>14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-7</td>
<td>S-3</td>
<td>6.00 - 6.00</td>
<td>30.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**

**Definitions:**
- MC: Moisture Content
- Soil Type: USCS (Unified Soil Classification System)
- LL: Liquid Limit
- PL: Plastic Limit
- PI: Plasticity Index
- CBR: California Bearing Ratio
- OC: Organic Content (ASTM D 2974)

---

**Project No.:** 11868  
**Project Name:** NCANG C-17 Hangar Project - GEO  
**PM:** Michael R. Bailey  
**PE:** Lee J. McGuinness  
**Printed On:** Monday, November 07, 2016
Concrete Depth [10.5"]

(ML) RESIDUAL- SANDY SILT, Reddish Brown, Moist, Stiff

(ML) SANDY SILT, Reddish Brown to Brown, Moist, Medium Stiff to Stiff

SHWT Identified at Approx. 11.6FT

END OF BORING @ 20.0'

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.
The stratification lines represent the approximate boundary lines between soil types. In-situ the transition may be gradual.

**Description of Material**
- **ASPHALT** (CH FILL) PLASTIC CLAY, Reddish Brown, Moist, Stiff
- **(ML) RESIDUAL- SANDY Silt**, Brown to Light Brown, Moist, Medium Stiff to Stiff
- **(ML) SANDY Silt**, Light Brown, Moist, Medium Stiff

**No SHWT Identified to 20FT**
The stratification lines represent the approximate boundary lines between soil types. In-situ the transition may be gradual.

<table>
<thead>
<tr>
<th>CLIENT</th>
<th>JOB #</th>
<th>BORING #</th>
<th>SHEET</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jacobs Global Buildings</td>
<td>11868</td>
<td>B-3</td>
<td>1 OF 1</td>
</tr>
</tbody>
</table>

**PROJECT NAME**
NCANG C-17 Hangar Project - GEO

**SITE LOCATION**
1st Union Road, Charlotte, Mecklenburg County, NC

**DEPTH (FT)**
<table>
<thead>
<tr>
<th>SAMPLE NO.</th>
<th>SAMPLE TYPE</th>
<th>SAMPLE DIST. (IN)</th>
<th>RECOVERY (IN)</th>
<th>DESCRIPTION OF MATERIAL</th>
<th>ENGLISH UNITS</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>ASPHALT</td>
<td></td>
</tr>
<tr>
<td>S-2</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>CONCRETE [10.5&quot;]</td>
<td></td>
</tr>
<tr>
<td>S-3</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>(CL FILL) SANDY CLAY, Reddish Brown, Moist, Stiff</td>
<td></td>
</tr>
<tr>
<td>S-4</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>(ML FILL) SANDY SILT, Reddish Brown, Moist, Medium Stiff</td>
<td></td>
</tr>
<tr>
<td>S-5</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>(ML) RESIDUAL-SANDY SILT, Brown to Light Brown, Moist, Medium Stiff to Stiff</td>
<td></td>
</tr>
</tbody>
</table>

**END OF BORING @ 20.0’**

No SHWT Identified to 20FT

**BORING STARTED** 10/28/16

**CAVE IN DEPTH @ 16.6’**

**HUMMER TYPE** Manual

**DRILLING METHOD** 2.25 H.S.A.
**NCANG C-17 Hangar Project - GEO**

**SITE LOCATION**
1st Union Road, Charlotte, Mecklenburg County, NC

**CLIENT**
Jacobs Global Buildings

**JOB #**
11868

**BORING #**
B-4

**PROJECT NAME**
NCANG C-17 Hangar Project - GEO

---

**ASPHALT**
(MH FILL) ELASTIC SILT, Reddish Brown, Moist, Medium Stiff

**RESIDUAL-ELASTIC SILT**
(MH) Brownish Red to Brown, Moist, Medium Stiff to Stiff

**SILTY FINE TO MEDIUM SAND**
(SM) Brown, Moist, Loose

---

END OF BORING @ 20.0'

No SHWT Identified to 20FT

---

**THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.**

---

**BOARING STARTED**
10/28/16

**CAVE IN DEPTH @ 17.2'**

**BOARING COMPLETED**
10/28/16

**RIG**
ATV

**FOREMAN**
Trevor

**DRILLING METHOD**
2.25 H.S.A.
**Project Name**: NCANG C-17 Hangar Project - GEO

**Site Location**: 1st Union Road, Charlotte, Mecklenburg County, NC

---

**Table**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Sample Dist. (in)</th>
<th>Recovery (in)</th>
<th>Surface Elevation (ft)</th>
<th>Description of Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>S-1</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td>727</td>
<td>CONCRETE [10.5&quot;]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(CL FILL) SANDY CLAY, Reddish Brown, Moist, Medium Stiff</td>
</tr>
<tr>
<td>5</td>
<td>S-2</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td></td>
<td>(ML) RESIDUAL-SANDY SILT, Reddish Brown to Brown, Moist, Stiff to Very Stiff</td>
</tr>
<tr>
<td>10</td>
<td>S-3</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>S-4</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td></td>
<td>(ML) SANDY SILT, Grayish Brown, Moist, Stiff</td>
</tr>
<tr>
<td>20</td>
<td>S-5</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td></td>
<td>END OF BORING @ 20.0'</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Notes**

- The stratification lines represent the approximate boundary lines between soil types. In-situ the transition may be gradual.
- No SHWT Identified to 20FT

---

**Additional Information**

- **Client**: Jacobs Global Buildings
- **Job #**: 11868
- **Boring #**: B-5
- **Sheet**: 1 OF 1
- **Architect-Engineer**: ECS
- **Project Name**: NCANG C-17 Hangar Project - GEO
- **Site Location**: 1st Union Road, Charlotte, Mecklenburg County, NC
- **Boring Started**: 10/28/16
- **Cave in Depth**: @ 17.4'
- **Hammer Type**: Manual
- **Drilling Method**: 2.25 H.S.A.
**CLIENT:** Jacobs Global Buildings  
**JOB #:** 11868  
**BORING #:** B-6  
**SHEET:** 1 OF 2

**PROJECT NAME:** NCANG C-17 Hangar Project - GEO

**SITE LOCATION:** 1st Union Road, Charlotte, Mecklenburg County, NC

<table>
<thead>
<tr>
<th>DEPTH (FT)</th>
<th>SAMPLE NO.</th>
<th>SAMPLE TYPE</th>
<th>SAMPLE DIST. (IN)</th>
<th>RECOVERY (IN)</th>
<th>DESCRIPTION OF MATERIAL</th>
<th>ENGLISH UNITS</th>
<th>WATER LEVELS (FT)</th>
<th>BLOWS/6&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>S-1</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>ASPHALT PROBABLE FILL: Driller Classified as (CL) SANDY CLAY, Reddish Brown, Moist, Medium Stiff</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>S-2</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>PROBABLE RESIDUAL: Driller Classified as (ML) SANDY SILT, Brown, Medium Stiff</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>S-3</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>S-4</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>S-5</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>S-6</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>S-7</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>S-8</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**CONTINUED ON NEXT PAGE.**

---

**The stratification lines represent the approximate boundary lines between soil types. In-situ the transition may be gradual.**

**WL** 34.1  
**WS**  
**WD**  
**BORING STARTED:** 10/28/16  
**CAVE IN DEPTH:** @ 34.5'

**WL(SHW)**  
**WL(ACR)**  
**BORING COMPLETED:** 10/28/16  
**HAMMER TYPE:** Manual

**WL**  
**RIG** ATV  
**FOREMAN:** Trevor  
**DRILLING METHOD:** 2.25 H.S.A.
### NCANG C-17 Hangar Project - GEO

**Client:** Jacobs Global Buildings  
**Job #:** 11868  
**Boring #:** B-6  
**Sheet:** 2 OF 2

**Project Name:** NCANG C-17 Hangar Project - GEO

**Site Location:** 1st Union Road, Charlotte, Mecklenburg County, NC

**Northing** | **Easting** | **Station**
---|---|---

---

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Sample Dist. (in)</th>
<th>Recovery (in)</th>
<th>Description of Material</th>
<th>English Units</th>
<th>Boring Started</th>
<th>Loss of Circulation</th>
<th>Loss of Circulation</th>
<th>Water Levels</th>
<th>Boring Completed</th>
<th>Hammer Type</th>
<th>Driller Classified as (ML) SANDY SILT, Brown, Medium Stiff</th>
<th>Driller Classified as (ML) SANDY SILT, Brown, Very Stiff</th>
<th>Driller Classified as (ML) SANDY SILT, Brown, Stiff</th>
<th>End of Boring @ 40.0'</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>S-9</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>S-10</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Surface Elevation:** 733

**CALIBRATED PENETROMETER TONS/FT²**

**ROCK QUALITY DESIGNATION & RECOVERY**

**PLASTIC LIMIT %**

**WATER CONTENT %**

**LIQUID LIMIT %**

---

**THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.**

**WL** 34.1 **WS** | **WD**  | **BORING STARTED** 10/28/16 | **CAVE IN DEPTH** @ 34.5' **WL** (SHW) | **WL(ACR)** | **BORING COMPLETED** 10/28/16 | **HAMMER TYPE** Manual | **RIG** ATV | **FOREMAN** Trevor | **DRILLING METHOD** 2.25 H.S.A.
### Soil Description

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Sample Dist. (in)</th>
<th>Recovery (in)</th>
<th>Surface Elevation</th>
<th>Description of Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>S-1</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>733</td>
<td>ASPHALT (CL FILL) SANDY CLAY, Brownish Red, Moist, Medium Stiff to Soft</td>
</tr>
<tr>
<td>5</td>
<td>S-2</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td>(ML) RESIDUAL-SANDY SILT, Reddish Brown, Moist, Medium Stiff</td>
</tr>
<tr>
<td>10</td>
<td>S-3</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td>(ML) SANDY SILT, Reddish Brown, Moist, Very Stiff to Stiff</td>
</tr>
<tr>
<td>15</td>
<td>S-4</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>S-5</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>S-6</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>S-7</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

SHWT Identified at Approx 23.5FT
**SITE LOCATION**

1st Union Road, Charlotte, Mecklenburg County, NC

**DEPTH (FT) | SAMPLE NO. | SAMPLE TYPE | SAMPLE DIST. (IN) | SURFACE ELEVATION**

| 35 | S-9 | SS | 18 | 733 |
| 40 | S-10 | DD | 18 | 733 |

**DESCRIPTION OF MATERIAL**

- (ML) SANDY SILT, Reddish Brown, Moist, Medium Stiff to Soft
- (ML) SANDY SILT, Reddish Brown, Moist, Stiff

**END OF BORING @ 40.0’**
THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

CONTINUED ON NEXT PAGE.
(ML) SANDY SILT, Brownish Red, Moist, Medium Stiff

(ML) SANDY SILT, Brownish Red Brown, Moist, Medium Stiff to Hard

END OF BORING @ 40.0'

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

WL 29.5 WS WD BORING STARTED 10/28/16 CAVE IN DEPTH @ 34.7'

WL(SHW) WL(ACR) BORING COMPLETED 10/28/16 HAMMER TYPE Manual

WL RIG ATV FOREMAN Trevor DRILLING METHOD 2.25 H.S.A.
1st Union Road, Charlotte, Mecklenburg County, NC

**Depth (ft)** | **Sample No.** | **Sample Type** | **Sample Dist. (in)** | **Recovery (in)** | **Water Levels** | **Surface Elevation** | **Bottom of Casing** | **Loss of Circulation** | **Boring Started** | **Boring Completed** | **Hammer Type** | **Drilling Method** |
---|---|---|---|---|---|---|---|---|---|---|---|---|
0 | S-1 | SS | 18 | 18 | 725 | 727 | | | | | Manual | 2.25 H.S.A. |
5 | S-2 | SS | 18 | 18 | | | | | | | | |
10 | S-3 | SS | 18 | 18 | | | | | | | | |
15 | S-4 | SS | 18 | 18 | | | | | | | | |
30 | S-5 | SS | 18 | 18 | | | | | | | | |

No SHWT Identified to 15FT

END OF BORING @ 15.0'

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.
December 5, 2016

ECS Carolinas, LLP
1812 Center Park Drive, Suite D
Charlotte, NC 28217

Attn: Mr. Michael Bailey, P.E.

RE: Roanoke – NCANG Charlotte-Douglas ANGB - C-17
Corrosion Control Hangar & SIM Building
Eden & Associates, P.C.

Mr. Bailey,

Per our recent discussions, included with this letter is a FAA FAARFIELD pavement design analysis for the above referenced project. The FAA software FAARFIELD is based on the cumulative damage factor (CDF) concept. The contribution of each airplane, in a given traffic mix, to total damage is separately analyzed. As provided by ECS, we’ve used the UFC Pavement Design for Airfields for providing design criteria and fleet mix expectations for the new pavement section. In particular, Table 3-1 Design Gross Weights and Pass Levels for Airfield Pavements:

<table>
<thead>
<tr>
<th>Design Aircraft</th>
<th>Weight / Pounds</th>
<th>Passes</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-15 E</td>
<td>81,000</td>
<td>100,000</td>
</tr>
<tr>
<td>C-17</td>
<td>580,000</td>
<td>400,000</td>
</tr>
<tr>
<td>B-52</td>
<td>400,000</td>
<td>400</td>
</tr>
</tbody>
</table>

Type C Traffic Areas - (2) Medium-load and modified heavy-load airfields. (c) Hangar access aprons and floors and washrack pavements. At Air Mobility Command Installations, hangar access aprons shall be designed as Medium Load Type C Traffic Area for the main gear plus 3 meters (10 feet) on each side. The remainder of the access apron shall be Light Load Type C Traffic Area.

<table>
<thead>
<tr>
<th>Design Aircraft</th>
<th>Weight / Pounds</th>
<th>Passes</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-15 E</td>
<td>60,750</td>
<td>100,000</td>
</tr>
<tr>
<td>C-17</td>
<td>435,000</td>
<td>400,000</td>
</tr>
<tr>
<td>B-52</td>
<td>300,000</td>
<td>400</td>
</tr>
</tbody>
</table>
The analysis is based upon a 20 year life span. The FAARFIELD software maintains an inventory of aircraft including the F-15C and C-17A. For gear loading purposes, these inventory aircraft match the criteria specified for the aircraft listed above. The FAARFIELD inventory does not include a B-52. For this analysis, a generic labelled D-200 was used in place of the B-52 to match the aircraft’s quadricycle tandem landing gear configuration.

The Annual Departures printed in the FAARFIELD report is for year 1 with a 0% growth assumption. Accordingly, to reach the passes listed in the UFC, the total passes was divided by 20 to determine the initial annual departures. The FAARFIELD output does not print the total passes but does list them in the program itself. The total departures in FAARFIELD matches the total passes listed in the UFC.

We’ve provided two variations for each scenario, one utilizing a P-209 or crushed aggregate separation layer and a second utilizing a P-301 soil cement stabilized layer. Based upon the Geotech report, the in-situ moisture for the fill material and underlying subgrade tends to be higher than optimum moisture content. As such, a soil cement stabilized layer mixed in place may provide an improved platform for placement of the drainage layer and rigid PCC pavement. Otherwise, a crushed aggregate P-209 separation layer is fairly typical for airfield pavement construction.

The FAARFIELD output reports are included with this letter. In general, the pavement design results in:

**Type B areas**
- 17” PCC section
- 10” cement treated P-304 aggregate drainage layer
- 8-10” crushed aggregate or soil cement separation layer

**Type C areas**
- 15” PCC section
- 10” cement treated P-304 aggregate drainage layer
- 8-10” crushed aggregate or soil cement separation layer

Please let me know if you have any questions or desire to discuss the FAARFIELD analysis. Thanks for this opportunity and we look forward to working with you again in the near future.

Sincerely,

Mark McGuire, P.E.
Vice President
Eden & Associates, P.C.
FAARFIELD

Section NewRigid01 in Job CLT.
Working directory is C:\Users\Mark McGuire\Documents\FAARFIELD\n
The structure is New Rigid.
Design Life = 20 years.
A design for this section was completed on 12/02/16 at 15:07:01.

Pavement Structure Information by Layer, Top First

<table>
<thead>
<tr>
<th>No.</th>
<th>Type</th>
<th>Thickness in</th>
<th>Modulus psi</th>
<th>Poisson's Ratio</th>
<th>Strength R, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PCC Surface</td>
<td>17.20</td>
<td>4,000,000</td>
<td>0.15</td>
<td>650</td>
</tr>
<tr>
<td>2</td>
<td>P-304 CTB</td>
<td>10.00</td>
<td>500,000</td>
<td>0.20</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>P-209 Cr Ag</td>
<td>8.00</td>
<td>26,132</td>
<td>0.35</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>Subgrade</td>
<td>0.00</td>
<td>7,453</td>
<td>0.40</td>
<td>0</td>
</tr>
</tbody>
</table>

Total thickness to the top of the subgrade = 35.20 in

Airplane Information

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Gross Wt. lbs</th>
<th>Annual Departures</th>
<th>% Annual Growth</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-15C</td>
<td>81,000</td>
<td>5,000</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>C-17A</td>
<td>580,000</td>
<td>20,000</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>D-200</td>
<td>250,000</td>
<td>20</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Additional Airplane Information

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>CDF Contribution</th>
<th>CDF Max for Airplane</th>
<th>P/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-15C</td>
<td>0.00</td>
<td>0.00</td>
<td>4.40</td>
</tr>
<tr>
<td>2</td>
<td>C-17A</td>
<td>0.98</td>
<td>0.98</td>
<td>1.31</td>
</tr>
<tr>
<td>3</td>
<td>D-200</td>
<td>0.02</td>
<td>0.03</td>
<td>3.29</td>
</tr>
</tbody>
</table>

User is responsible for checking frost protection requirements.
<table>
<thead>
<tr>
<th>Layer Material</th>
<th>Thickness (in)</th>
<th>Modulus or R (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCC Surface</td>
<td>17.20</td>
<td>650</td>
</tr>
<tr>
<td>P-304 CTB</td>
<td>10.00</td>
<td>500,000</td>
</tr>
<tr>
<td>P-209 Cr Ag</td>
<td>8.00</td>
<td>26,132</td>
</tr>
</tbody>
</table>

Subgrade  
\[ k = 100.0 \]
\[ t = 7.463 \]

N = 1; PCC CDF = 1.00; t = 35.20 in
FAARFIELD

FAARFIELD v 1.41 - Airport Pavement Design

Section NewRigid01B in Job CLT.
Working directory is C:\Users\Mark McGuire\Documents\FAARFIELD\n
The structure is New Rigid.
Design Life = 20 years.
A design for this section was completed on 12/02/16 at 14:31:16.

Pavement Structure Information by Layer, Top First

<table>
<thead>
<tr>
<th>No.</th>
<th>Type</th>
<th>Thickness in</th>
<th>Modulus psi</th>
<th>Poisson's Ratio</th>
<th>Strength R, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PCC Surface</td>
<td>16.65</td>
<td>4,000,000</td>
<td>0.15</td>
<td>650</td>
</tr>
<tr>
<td>2</td>
<td>P-304 CTB</td>
<td>10.00</td>
<td>500,000</td>
<td>0.20</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>P-301 SCB</td>
<td>8.00</td>
<td>250,000</td>
<td>0.20</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>Subgrade</td>
<td>0.00</td>
<td>7,453</td>
<td>0.40</td>
<td>0</td>
</tr>
</tbody>
</table>

Total thickness to the top of the subgrade = 34.65 in

Airplane Information

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Gross Wt. lbs</th>
<th>Annual Departures</th>
<th>% Annual Growth</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-15C</td>
<td>81,000</td>
<td>5,000</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>C-17A</td>
<td>580,000</td>
<td>20,000</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>D-200</td>
<td>250,000</td>
<td>20</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Additional Airplane Information

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>CDF Contribution</th>
<th>CDF Max for Airplane</th>
<th>P/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-15C</td>
<td>0.00</td>
<td>0.00</td>
<td>4.40</td>
</tr>
<tr>
<td>2</td>
<td>C-17A</td>
<td>0.99</td>
<td>0.99</td>
<td>1.31</td>
</tr>
<tr>
<td>3</td>
<td>D-200</td>
<td>0.01</td>
<td>0.01</td>
<td>3.29</td>
</tr>
</tbody>
</table>

User is responsible for checking frost protection requirements.
<table>
<thead>
<tr>
<th>Layer Material</th>
<th>Thickness (in)</th>
<th>Modulus or R (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCC Surface</td>
<td>16.65</td>
<td>650</td>
</tr>
<tr>
<td>P-304 CTB</td>
<td>10.00</td>
<td>500,000</td>
</tr>
<tr>
<td>P-301 SCB</td>
<td>8.00</td>
<td>250,000</td>
</tr>
</tbody>
</table>

Subgrade: $k = 100.0$ in; $t = 34.65$ in

N = 1; PCC CDF = 1.00; $t = 34.65$ in
Section NewRigid02 in Job CLT.

Working directory is C:\Users\Mark McGuire\Documents\FAARFIELD\n
The structure is New Rigid.

Design Life = 20 years.

A design for this section was completed on 12/02/16 at 16:08:01.

Pavement Structure Information by Layer, Top First

<table>
<thead>
<tr>
<th>No.</th>
<th>Type</th>
<th>Thickness in</th>
<th>Modulus psi</th>
<th>Poisson's Ratio</th>
<th>Strength R, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PCC Surface</td>
<td>14.99</td>
<td>4,000,000</td>
<td>0.15</td>
<td>650</td>
</tr>
<tr>
<td>2</td>
<td>P-304 CTB</td>
<td>10.00</td>
<td>500,000</td>
<td>0.20</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>P-209 Cr Ag</td>
<td>8.00</td>
<td>26,132</td>
<td>0.35</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>Subgrade</td>
<td>0.00</td>
<td>7,453</td>
<td>0.40</td>
<td>0</td>
</tr>
</tbody>
</table>

Total thickness to the top of the subgrade = 32.99 in

Airplane Information

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Gross Wt. lbs</th>
<th>Annual Departures</th>
<th>% Annual Growth</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-15C</td>
<td>60,750</td>
<td>5,000</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>C-17A</td>
<td>435,000</td>
<td>20,000</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>D-200</td>
<td>250,000</td>
<td>20</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Additional Airplane Information

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>CDF Contribution</th>
<th>CDF Max for Airplane</th>
<th>P/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-15C</td>
<td>0.00</td>
<td>0.00</td>
<td>4.40</td>
</tr>
<tr>
<td>2</td>
<td>C-17A</td>
<td>0.02</td>
<td>0.02</td>
<td>1.31</td>
</tr>
<tr>
<td>3</td>
<td>D-200</td>
<td>0.98</td>
<td>0.98</td>
<td>3.29</td>
</tr>
</tbody>
</table>

User is responsible for checking frost protection requirements.
<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (in)</th>
<th>Modulus or R (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCC Surface</td>
<td>14.99</td>
<td>650</td>
</tr>
<tr>
<td>P-304 CTB</td>
<td>10.00</td>
<td>500,000</td>
</tr>
<tr>
<td>P-209 Cr Ag</td>
<td>8.00</td>
<td>26,132</td>
</tr>
<tr>
<td>Subgrade</td>
<td></td>
<td>k = 100.0</td>
</tr>
</tbody>
</table>
FAARFIELD

FAARFIELD v 1.41 - Airport Pavement Design

Section NewRigid02B in Job CLT.

Working directory is C:\Users\Mark McGuire\Documents\FAARFIELD\n
The structure is New Rigid.

Design Life = 20 years.

A design for this section was completed on 12/02/16 at 16:14:48.

Pavement Structure Information by Layer, Top First

<table>
<thead>
<tr>
<th>No.</th>
<th>Type</th>
<th>Thickness in</th>
<th>Modulus psi</th>
<th>Poisson's Ratio</th>
<th>Strength R, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PCC Surface</td>
<td>13.87</td>
<td>4,000,000</td>
<td>0.15</td>
<td>650</td>
</tr>
<tr>
<td>2</td>
<td>P-304 CTB</td>
<td>10.00</td>
<td>500,000</td>
<td>0.20</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>P-301 SCB</td>
<td>8.00</td>
<td>250,000</td>
<td>0.20</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>Subgrade</td>
<td>0.00</td>
<td>7,453</td>
<td>0.40</td>
<td>0</td>
</tr>
</tbody>
</table>

Total thickness to the top of the subgrade = 31.87 in

Airplane Information

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Gross Wt. lbs</th>
<th>Annual Departures</th>
<th>% Annual Growth</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-15C</td>
<td>60,750</td>
<td>5,000</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>C-17A</td>
<td>435,000</td>
<td>20,000</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>D-200</td>
<td>250,000</td>
<td>20</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Additional Airplane Information

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>CDF Contribution</th>
<th>CDF Max for Airplane</th>
<th>P/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-15C</td>
<td>0.00</td>
<td>0.00</td>
<td>4.40</td>
</tr>
<tr>
<td>2</td>
<td>C-17A</td>
<td>0.03</td>
<td>0.03</td>
<td>1.31</td>
</tr>
<tr>
<td>3</td>
<td>D-200</td>
<td>0.97</td>
<td>0.97</td>
<td>3.29</td>
</tr>
</tbody>
</table>

User is responsible for checking frost protection requirements.
**Layer Material** | **Thickness (in)** | **Modulus or R (psi)**
---|---|---
PCC Surface | 13.87 | 650
P-304 CTB | 10.00 | 500,000
P-301 SCB | 8.00 | 250,000
Subgrade | k = 100.0 | \( k = 7.463 \)

\( N = 0; \) PCC CDF = 1.00; \( t = 31.87 \text{ in} \)
October 31, 2016

Mr. David Everett, AIA
Project Manager
Jacobs Global Buildings
1100 North Glebe Road, Suite 500
Arlington, Virginia 22201

Reference:
Report of Seasonal High Water Table (SHWT)
NCANG C-17 Hangar Project
Charlotte, Mecklenburg County, North Carolina
ECS Project No: 49-3091

Dear Mr. Everett:

ECS Carolinas, LLP (ECS) is pleased to provide you with our Report of Seasonal High Water Table (SHWT) Study for the NCANG C-17 Hangar Site in Charlotte, North Carolina.

PROJECT UNDERSTANDING

The project is located at the existing North Carolina Air National Guard installation at Charlotte Douglas International Airport. We understand that the project will include a new 1-2 story, 61,000 SF, steel framed hangar facility and supporting space. A new 1-story, 10,000 SF flight simulation facility will also be constructed.

An aerial photograph was prepared by ECS identifying nine geotechnical boring locations; wherein, a SHWT determination was made (Figure 1). The soil investigation was conducted with a drill rig via split spoon, to a depth of approximately 15 feet to 40 feet below ground surface (bgs) or auger refusal.

SCOPE OF SERVICES

ECS conducted a study/investigation of the soils to identify the depth of the seasonal high water table, if present. The properties and characteristics of the soils retrieved from the boring were observed and recorded in field notes. The properties include texture, depth, the presence of restrictive horizons, depth to seasonal high water table, coarse fragments, etc. The assessment was conducted in accordance with current soil science practices and technology.

SEASONAL HIGH WATER TABLE STUDY

Below is a summary of the soils retrieved from the boring.

SHWT Boring 1 — Concrete and base were identified to a depth of approximately 14 inches bgs. The subsurface layer from 14 inches to approximately 150 inches bgs was red clay, with moderate, medium, sub-
angular blocky structure. The consistence was sticky, plastic, and firm. The sub-surface layer from 150 inches to approximately 240 bgs consisted of multi-colored sandy loam saprolite. The structure was structureless/massive.

**SHWT Boring 2** – Asphalt and base were identified to a depth of approximately 12 inches bgs. The sub-surface layer from 12 inches to approximately 162 inches bgs was red clay, with moderate, medium, sub-angular blocky structure. The consistence was sticky, plastic, and firm. The sub-surface layer from 162 inches to approximately 240 bgs consisted of multi-colored sandy clay loam to sandy loam saprolite. The structure was structureless/massive.

**SHWT Boring 3** – Concrete and base were identified to a depth of approximately 10 inches bgs. The sub-surface layer from 10 inches to approximately 102 inches bgs was red and yellow clay, with moderate, medium, sub-angular blocky structure. The consistence was sticky, plastic, and firm. The sub-surface layer from 102 inches to approximately 240 bgs consisted of multi-colored sandy clay loam saprolite. The structure was structureless/massive.

**SHWT Boring 4** – Asphalt and base were identified to a depth of approximately 8 inches bgs. The sub-surface layer from 8 inches to approximately 115 inches bgs was red clay, with moderate, medium, sub-angular blocky structure. The consistence was sticky, plastic, and firm. The sub-surface layer from 115 inches to approximately 240 bgs consisted of multi-colored sandy clay loam to sandy loam saprolite. The structure was structureless/massive.

**SHWT Boring 5** – Concrete and base were identified to a depth of approximately 14 inches bgs. The sub-surface layer from 14 inches to approximately 98 inches bgs was red and yellow clay, with moderate, medium, sub-angular blocky structure. The consistence was sticky, plastic, and firm. The sub-surface layer from 98 inches to approximately 240 bgs consisted of multi-colored sandy clay loam to sandy loam saprolite. The structure was structureless/massive.

**SHWT Boring 6** – Asphalt and base were identified to a depth of approximately 12 inches bgs. The sub-surface layer from 12 inches to approximately 75 inches bgs was red clay, with moderate, medium, sub-angular blocky structure. The consistence was sticky, plastic, and firm. The sub-surface layer from 75 inches to approximately 480 bgs consisted of multi-colored sandy clay loam to sandy loam saprolite. The structure was structureless/massive.

**SHWT Boring 7** – Asphalt and base were identified to a depth of approximately 7 inches bgs. The sub-surface layer from 7 inches to approximately 288 inches bgs was red clay, with moderate, medium, sub-angular blocky structure. The consistence was sticky, plastic, and firm. The sub-surface layer from 288 inches to approximately 480 bgs consisted of multi-colored sandy clay loam to sandy loam saprolite. The structure was structureless/massive.

**SHWT Boring 8** – Asphalt and base were identified to a depth of approximately 4 inches bgs. The sub-surface layer from 4 inches to approximately 255 inches bgs was red clay, with moderate, medium, sub-angular blocky structure. The consistence was sticky, plastic, and firm. The sub-surface layer from 255 inches to approximately 480 bgs consisted of multi-colored sandy clay loam to sandy loam saprolite. The structure was structureless/massive.
SHWT Study
NCANG C-17 Hangar SHWT
Charlotte, Mecklenburg County, North Carolina
ECS Project No. 49-3091

**SHWT Boring 9** – Concrete and base were identified to a depth of approximately 12 inches bgs. The sub-surface layer from 12 inches to approximately 102 inches bgs was red and yellow clay, with moderate, medium, sub-angular blocky structure. The consistence was sticky, plastic, and firm. The sub-surface layer from 102 inches to approximately 180 bgs consisted of multi-colored clay loam to sandy loam saprolite. The structure was structureless/massive.

**FINDINGS**

**SHWT Boring 1** – Evidence of a SHWT was identified at approximately 140 inches bgs.

**SHWT Boring 2** – No SHWT was identified to a depth of 240 inches bgs.

**SHWT Boring 3** – No SHWT was identified to a depth of 240 inches bgs.

**SHWT Boring 4** – No SHWT was identified to a depth of 240 inches bgs.

**SHWT Boring 5** – No SHWT was identified to a depth of 240 inches bgs.

**SHWT Boring 6** – Evidence of a SHWT was identified at approximately 270 inches bgs.

**SHWT Boring 7** – Evidence of a SHWT was identified at approximately 282 inches bgs.

**SHWT Boring 8** – Evidence of a SHWT was identified at approximately 275 inches bgs.

**SHWT Boring 9** – No SHWT was identified to a depth of 180 inches bgs.

The type of stormwater management facility designed is based on the depth of the SHWT or confining layer. The information above may be potentially utilized to determine the type of stormwater management facility best suited for this site according to the North Carolina Division of Water Quality Stormwater Best Management Practice Manual, dated July, 2007.
CLOSING

ECS is pleased to offer our professional services and look forward to assisting in any of your site analysis needs in the future. If you have any questions or require further assistance, please contact us at 704-525-5152.

Respectfully,

ECS CAROLINAS, LLP

David E. Valentine
Environmental Team Manager
dvalentine@ecslimited.com
704-525-5152

W. Brandon Fulton, LSS, PSC, PWS
Environmental Principal
bfulton@ecslimited.com
704-525-5152

Attachment: Figure 1 – SHWT Boring Location Map
## Reference Notes for Boring Logs

### Cohesive Silts & Clays

<table>
<thead>
<tr>
<th>Classification</th>
<th>Unconfined Compressive Strength, $Q_u$</th>
<th>SPT&lt;sup&gt;5&lt;/sup&gt; (BPF)</th>
<th>Consistency&lt;sup&gt;7&lt;/sup&gt; (Cohesive)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trace</td>
<td>&lt;0.25</td>
<td>&lt;3</td>
<td>Very Soft</td>
</tr>
<tr>
<td>Dual Symbol</td>
<td>0.25 - &lt;0.50</td>
<td>3 - 4</td>
<td>Soft</td>
</tr>
<tr>
<td>With</td>
<td>0.50 - &lt;1.00</td>
<td>5 - 8</td>
<td>Medium Stiff</td>
</tr>
<tr>
<td>Adjective</td>
<td>1.00 - &lt;2.00</td>
<td>9 - 15</td>
<td>Stiff</td>
</tr>
<tr>
<td></td>
<td>2.00 - &lt;4.00</td>
<td>16 - 30</td>
<td>Very Stiff</td>
</tr>
<tr>
<td></td>
<td>4.00 - 8.00</td>
<td>31 - 50</td>
<td>Hard</td>
</tr>
<tr>
<td></td>
<td>&gt;8.00</td>
<td>&gt;50</td>
<td>Very Hard</td>
</tr>
</tbody>
</table>

### Gravels, Sands & Non-Cohesive Silts

<table>
<thead>
<tr>
<th>SPT&lt;sup&gt;5&lt;/sup&gt;</th>
<th>Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;5</td>
<td>Very Loose</td>
</tr>
<tr>
<td>5 - 10</td>
<td>Loose</td>
</tr>
<tr>
<td>11 - 30</td>
<td>Medium Dense</td>
</tr>
<tr>
<td>31 - 50</td>
<td>Dense</td>
</tr>
<tr>
<td>&gt;50</td>
<td>Very Dense</td>
</tr>
</tbody>
</table>

### Water Levels<sup>6</sup>

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>WL</td>
<td>Water Level (WS)(WD)</td>
</tr>
<tr>
<td></td>
<td>(WS) While Sampling</td>
</tr>
<tr>
<td></td>
<td>(WD) While Drilling</td>
</tr>
<tr>
<td>SHW</td>
<td>Seasonal High WT</td>
</tr>
<tr>
<td>ACR</td>
<td>After Casing Removal</td>
</tr>
<tr>
<td>SWT</td>
<td>Stabilized Water Table</td>
</tr>
<tr>
<td>DCI</td>
<td>Dry Cave-In</td>
</tr>
<tr>
<td>WCI</td>
<td>Wet Cave-In</td>
</tr>
</tbody>
</table>

### Particle Size Identification

<table>
<thead>
<tr>
<th>Relative Amount&lt;sup&gt;7&lt;/sup&gt;</th>
<th>Coarse Grained (%)</th>
<th>Fine Grained (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trace</td>
<td>&lt;5</td>
<td>&lt;5</td>
</tr>
<tr>
<td>With</td>
<td>15 - 20</td>
<td>15 - 25</td>
</tr>
<tr>
<td>Adjective</td>
<td>25 - &lt;50</td>
<td>30 - &lt;50</td>
</tr>
</tbody>
</table>

### Drilling Sampling Symbols & Abbreviations

<table>
<thead>
<tr>
<th>Designation</th>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS</td>
<td>Split Spoon Sampler</td>
<td></td>
</tr>
<tr>
<td>ST</td>
<td>Shelby Tube Sampler</td>
<td></td>
</tr>
<tr>
<td>WS</td>
<td>Wash Sample</td>
<td></td>
</tr>
<tr>
<td>BS</td>
<td>Bulk Sample of Cuttings</td>
<td></td>
</tr>
<tr>
<td>PA</td>
<td>Power Auger (no sample)</td>
<td></td>
</tr>
<tr>
<td>HSA</td>
<td>Hollow Stem Auger</td>
<td></td>
</tr>
<tr>
<td>PM</td>
<td>Pressuremeter Test</td>
<td></td>
</tr>
<tr>
<td>RD</td>
<td>Rock Bit Drilling</td>
<td></td>
</tr>
<tr>
<td>RC</td>
<td>Rock Core, NX, BX, AX</td>
<td></td>
</tr>
<tr>
<td>REC</td>
<td>Rock Sample Recovery %</td>
<td></td>
</tr>
<tr>
<td>RQD</td>
<td>Rock Quality Designation %</td>
<td></td>
</tr>
</tbody>
</table>

### Material

<table>
<thead>
<tr>
<th>Code</th>
<th>Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS</td>
<td>Asphalt</td>
</tr>
<tr>
<td>CO</td>
<td>Concrete</td>
</tr>
<tr>
<td>GR</td>
<td>Gravel</td>
</tr>
<tr>
<td>TO</td>
<td>Topsoil</td>
</tr>
<tr>
<td>VO</td>
<td>Void</td>
</tr>
<tr>
<td>BR</td>
<td>Brick</td>
</tr>
<tr>
<td>AG</td>
<td>Aggregate base course</td>
</tr>
<tr>
<td>FI</td>
<td>Fill&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
<tr>
<td>GL</td>
<td>Well-graded gravel</td>
</tr>
<tr>
<td>GP</td>
<td>Poorly-graded gravel</td>
</tr>
<tr>
<td>GM</td>
<td>Silty gravel</td>
</tr>
<tr>
<td>GC</td>
<td>Clayey gravel</td>
</tr>
<tr>
<td>SW</td>
<td>Well-graded sand</td>
</tr>
<tr>
<td>SP</td>
<td>Poorly-graded sand</td>
</tr>
<tr>
<td>SM</td>
<td>Silty sand</td>
</tr>
<tr>
<td>SC</td>
<td>Clayey sand</td>
</tr>
<tr>
<td>ML</td>
<td>Silt</td>
</tr>
<tr>
<td>MH</td>
<td>Elastic silt</td>
</tr>
<tr>
<td>CL</td>
<td>Lean clay</td>
</tr>
<tr>
<td>CH</td>
<td>Fat clay</td>
</tr>
<tr>
<td>OL</td>
<td>Organic silt or clay</td>
</tr>
<tr>
<td>OH</td>
<td>Organic silt or clay</td>
</tr>
<tr>
<td>PT</td>
<td>Peat</td>
</tr>
</tbody>
</table>

2. To be consistent with general practice, “POORLY GRADED” has been removed from GP, GP-GM, GP-GC, SP, SP-SM, SP-SC soil types on the boring logs.
3. Non-ASTM designations are included in soil descriptions and symbols along with ASTM symbol [Ex: (SM-FILL)].
4. Typically estimated via pocket penetrometer or Torvane shear test and expressed in tons per square foot (tsf).
5. Standard Penetration Test (SPT) refers to the number of hammer blows (blow count) of a 140 lb. hammer falling 30 inches on a 2 inch OD split spoon sampler required to drive the sampler 12 inches (ASTM D 1586). "N-value" is another term for "blow count" and is expressed in blows per foot (bpf).
6. The water levels are those levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in granular soils. In clay and cohesive silts, the determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally employed.
7. Minor deviation from ASTM D 2488-09.
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 an existing 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
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 your ASFE-member geotechnical engineer for more information.
SUPPLEMENTAL REPORT
OF
SUBSURFACE EXPLORATION

NORTH CAROLINA AIR NATIONAL GUARD (NCANG) –
C-17 HANGAR AND FLIGHT SIMULATION FACILITIES
CHARLOTTE, NORTH CAROLINA

ECS PROJECT NO. 08-11868-A
MAY 19, 2017
SUPPLEMENTAL REPORT
OF
SUBSURFACE EXPLORATION

North Carolina Air National Guard (NCANG)
C-17 Hangar and Flight Simulation Facilities
Charlotte, North Carolina

Prepared For:

Mr. David Everett, AIA
Project Manager
Jacobs Global Buildings
1100 North Glebe Road, Suite 500
Arlington, VA 22201

Prepared By:

ECS CAROLINAS, LLP
1812 Center Park Drive
Suite D
Charlotte, NC 28217

ECS Project No:

08-11868-A

Report Date:

May 19, 2017
May 19, 2017

Mr. David Everett, AIA  
Project Manager  
Jacobs Global Buildings  
1100 North Glebe Road, Suite 500  
Arlington, VA 22201

Reference: Supplemental Report of Subsurface Exploration  
North Carolina Air National Guard (NCANG)  
C-17 Hangar and Flight Simulation Facilities  
Charlotte, North Carolina  
ECS Project No: 08-11868-A

Dear Mr. Everett:

ECS Southeast, LLP (ECS) has completed the subsurface exploration for the above referenced project. This project was authorized and performed in general accordance with ECS Proposal No. 08-20362P dated November 22, 2016. The purpose of this phase of exploration was to determine the general subsurface conditions at the proposed alternate Flight Simulation Building location and to evaluate those conditions with regard to foundation and floor slab support, along with general site development. This report presents our findings, conclusions, and recommendations for design and construction of the project. This report should be used in conjunction with ECS Report of Subsurface Exploration (ECS Project No. 08-11868) dated December 5, 2016.

ECS Southeast, LLP appreciates the opportunity to assist you during this phase of the project. If you have questions concerning this report, please contact our office.

Respectfully,

ECS SOUTHEAST, LLP

Michael R. Bailey, P.E.  
Project Engineer  
NC Registration No. 041906

Lee J. McGuinness, P.E.  
Principal Engineer

1812 Center Park Drive, Suite D, Charlotte, NC 28217 • T: 704-525-5152 • F: 704-357-0023 • www.ecslimited.com  
ECS Capitol Services, PLLC • ECS Florida, LLC • ECS Mid-Atlantic, LLC • ECS Midwest, LLC • ECS Southeast, LLP • ECS Texas, LLP
1. INTRODUCTION

1.1. Project Information

The project site is located at the North Carolina Air National Guard (NCANG) at the Charlotte Douglas International Airport in Charlotte, North Carolina, as shown in the Site Vicinity Map (Figure 1) located in the Appendix. The project includes the construction of a 61,600 square feet C-17 Hangar Facility including an enclosed facility for washing aircraft, performing corrosion control, general maintenance, and shop areas to accommodate maintenance and training. In addition, a separate flight crew simulator training facility to support the C-17 mission will be located to the east of the proposed C-17 Hangar Facility. The purpose of this supplemental report is to determine the general subsurface conditions at the proposed alternate Simulator Building Location. The Flight Simulation Facility is anticipated to be on the order of 10,000 square feet. Additional components of the project will include the existing facility demolition, site preparation, stormwater detention and drainage systems, utilities, fencing, sidewalks, curbs, communications support, fire protection, flexible paving for POV access road, exterior lighting utilities, and Anti-Terrorism/Force Protection (AT/FP) measures.

The Simulator Building is a low-rise building with anticipated maximum column and wall footing loads on the order of 150 kips and 8.5 kips per linear foot, respectively. Additionally, the simulator building will include a mat foundation approximately 20 feet by 20 feet in size with a maximum dynamic downward loading of 122 kips and dynamic horizontal loading of 118 kips. Anticipated loading conditions were provided by Jacobs.

1.2. Scope of Services

Our scope of services for this phase of the project included a supplemental subsurface exploration with soil test borings, engineering analysis of the foundation support options, and preparation of this supplemental report with our recommendations. The supplemental subsurface exploration included three (3) soil test borings (B-10 through B-12) to depths ranging from approximately 32.0 to 40 feet below the existing ground surface. The approximate boring locations are shown on the Boring Location Diagram (Figure 2) included in the Appendix. The soil borings were performed using a CME 550 Truck Mounted drill rig using continuous-flight, hollow-stem augers.

2. FIELD SERVICES

2.1. Test Locations

The soil boring locations and depths were selected by others and located in the field by ECS using GPS and existing landmarks as reference. The approximate boring locations are shown on the Boring Location Diagram (Figure 2) presented in the Appendix of this report and should be considered accurate only to the degree implied by the method used to obtain them. Ground surface elevations at the boring locations were interpolated using Google Earth and should be considered approximate.

2.2. Soil Test Borings

Three (3) soil test borings were drilled to evaluate the stratification and engineering properties of the subsurface soils at the project site. Standard Penetration Tests (SPT’s) were performed at designated intervals in general accordance with ASTM D 1586. The Standard Penetration Test is used to provide an index for estimating soil strength and density. In conjunction with the penetration testing, split-barrel soil samples were recovered for soil classification at each test interval. Boring Logs are included in the Appendix.
The drill crew also maintained a field log of the soils encountered at each of the boring locations. After recovery, each sample was removed from the auger and visually classified. Representative portions of each sample were then sealed and brought to our laboratory in Charlotte, North Carolina for further visual examination and potential laboratory testing. Groundwater measurements were attempted at the termination of drilling at each boring location.

2.3. Seasonal High Water Table

A licensed soil scientist was onsite during the drilling of the borings (B-10 through B-12) in order to determine the depth of the seasonal high water table (SHWT). The characteristics of the soil were observed, including texture, depth, slope, the presence of restrictive horizons, depth to seasonal high water table, etc. The assessment was conducted in accordance with current soil science practices and technology. The results of the SHWT study are included in the Appendix of this report.

3. LABORATORY SERVICES

Soil samples were collected from the borings and examined in our laboratory to check field classifications and to determine pertinent engineering properties. Data obtained from the borings, our visual/manual examinations, and laboratory testing are included on the respective boring logs in the Appendix.

3.1 Soil Classification

A geotechnical staff professional classified each soil sample on the basis of color, texture, and plasticity characteristics in general accordance with the Unified Soil Classification System (USCS). The staff professional grouped the various soil types into the major zones noted on the boring logs. The stratification lines designating the interfaces between earth materials on the boring logs and profiles are approximate; in situ, the transition between strata may be gradual in both the vertical and horizontal directions. The results of the visual classifications are presented on the Boring Logs included in the Appendix.

3.2 Laboratory Testing

In addition to visual classification, ECS performed four (4) natural moisture content tests and two (2) Atterberg Limits tests. The laboratory testing was performed in general accordance with the applicable ASTM standards. The results of the laboratory testing are presented on the respective Boring Log and presented within the Laboratory Testing Summary Sheet included within the Appendix.

4. SITE AND SUBSURFACE FINDINGS

4.1. Area Geology

The site is located in the Piedmont Physiographic Province of North Carolina. The native soils in the Piedmont Province consist mainly of residuum with underlying saprolites weathered from the parent bedrock, which can be found in both weathered and unweathered states. Although the surficial materials normally retain the structure of the original parent bedrock, they typically have a much lower density and exhibit strengths and other engineering properties typical of soil. In a mature weathering profile of the Piedmont Province, the soils are generally found to be finer grained at the surface where more extensive weathering has occurred. The particle size of the
soils generally becomes more granular with increasing depth and gradually changes first to weathered and finally to unweathered parent bedrock. The mineral composition of the parent rock and the environment in which weathering occurs largely control the resulting soil’s engineering characteristics. The residual soils are the product of the weathering of the parent bedrock.

In addition, it is apparent that the natural geology within the site has been modified in the past by grading that included placement of fill materials. The quality of man-made fills can vary significantly, and it is often difficult to access the engineering properties of existing tests performed in soil test borings and the degree of compaction of existing fill soils; however, a qualitative assessment of existing fills can sometimes be made based on the N-values obtained and observations of the materials sampled in the test borings.

4.2. Subsurface Conditions

The subsurface conditions at the site, as indicated by the borings, generally consisted of fill, residual soil, Partially Weathered Rock, and Auger Refusal materials to the depths explored. The generalized subsurface conditions are described below. For soil stratification at a particular test location, the respective Boring Log found in the Appendix should be reviewed.

Approximately 3 to 4 inches of asphalt underlain by approximately 3 to 6 inches of ABC stone was observed at the ground surface at each of the boring locations. The surficial material depths provided in this report and on the individual Boring Logs are based on driller observations and should be considered approximate. Please note that these reported values should not be used in determining removal quantities.

Fill soils were encountered below the surficial material at Borings B-10 and B-12. The fill extended to depths ranging from approximately 5½ to 12 feet below the existing ground surface. The fill soils encountered generally consisted of Plastic CLAY (CH), Elastic SILT (MH), and Sandy SILT (ML) exhibiting SPT N-values ranging from 4 to 13 blows per foot (bpf) with the majority of the blow counts above 8 bpf.

Residual soil was encountered below the surficial materials and/or fill soils at each of the boring locations. Residual soils are formed by the in-place chemical and mechanical weathering of the parent bedrock. The residual soils encountered in the borings generally consisted of Sandy SILT (ML), Sandy CLAY (CL), and Silty SAND (SM), exhibiting SPT N-values ranging from 6 to 64 blows per foot (bpf). Borings B-10 and B-11 were terminated in the residual soils at depths of approximately 40 feet below the existing ground surface.

Partially weathered rock (PWR) was encountered below the residual soils at Borings B-12. The top of the PWR was encountered at depth of approximately 27 feet below the existing grade. Additionally, a lense of PWR was encountered between a depth of 32 to 37 feet at Boring B-11. PWR is defined as residual material exhibiting SPT N-values greater than 100 bpf. The PWR encountered in the borings generally consisted of Sandy SILT (ML) and Silty SAND (SM) exhibiting SPT N-values ranging from 50 blows per 3 inches of penetration to 50 blows per 2 inches of penetration.

Auger refusal was encountered at Boring B-12 at a depth of approximately 32 feet below the existing ground surface. Auger refusal indicates the presence of material that permitted no further advancement of the hollow stem auger or split spoon sampler. Rock coring would be
required to evaluate the character and continuity of the refusal materials. However, rock coring was beyond the scope of this exploration.

4.3. Groundwater Observations

Groundwater measurements were attempted at the termination of drilling at the time of our exploration. Groundwater was encountered at each of the boring locations at depths ranging from approximately 15.0 to 37.5 feet below the existing ground surface, respectively. Borehole cave-in depths were observed at each boring location at depths ranging from approximately 17.0 to 26.0 feet below the existing ground surface. Cave-in of a soil test boring can be caused by groundwater hydrostatic pressure, weak soil layers, and/or drilling activities (i.e. drilling fluid circulation or advancement of bit).

Fluctuations in the groundwater elevation should be expected depending on precipitation, run-off, utility leaks, and other factors not evident at the time of our evaluation. Normally, highest groundwater levels occur in late winter and spring and the lowest levels occur in late summer and fall. Depending on time of construction, groundwater may be encountered at shallower depths and locations not explored during this study. If encountered during construction, engineering personnel from our office should be notified immediately.

4.4. Laboratory Test Results

Moisture content test results of the sampled soils range from approximately 9.6 to 21.8 percent. Atterberg Limits testing was performed on a selected soil samples from B-10 and B-12 resulting in liquid limits (LL) ranging from 52 to 68 and plasticity indices (PI) ranging from 20 to 36. The portion of the samples tested was USCS classified as Plastic CLAY (CH) for B-10 and Elastic SILT (MH) for B-12. For laboratory test results at a particular test location, the Laboratory Test Summary sheet found in the Appendix should be reviewed.

5. CONCLUSIONS AND RECOMMENDATIONS

The borings performed at this site represent the subsurface conditions at the location of the borings. Due to inconsistencies associated with the prevailing geology and existing fill soils, there can be changes in the subsurface conditions over relatively short distances that have not been disclosed by the results of the test location performed. Consequently, there may be undisclosed subsurface conditions that require special treatment or additional preparation once these conditions are revealed during construction.

Our evaluation of foundation support conditions has been based on our understanding of the site, project information and the data obtained in our exploration. The general subsurface conditions utilized in our foundation evaluation have been based on interpolation of subsurface data between and away from the borings. In evaluating the boring data, we have examined previous correlations between penetration resistance values and foundation bearing pressures observed in soil conditions similar to those at your site.

5.1. Existing Fill

Existing fill soils were encountered below the surficial materials at Borings B-10 and B-12 and extended to depths ranging from approximately 5.5 to 12 feet below the existing ground surface. ECS has not been provided with test records (such as previous subsurface exploration, proofrolling, compaction testing, etc.) at the time of this report. Standard penetration resistances in the existing fill ranged from 4 to 13 blows per foot (bpf) with the majority of the blow counts
over 8 bpf. Based on our findings, it appears that the soils have generally been placed with some compactive effort. While ECS does not possess testing records of the existing fill soils, the qualitative properties of the soils are generally indicative of engineered fill. Therefore, our recommendations provided in this report are based on the assumption the existing fill soils were placed and compacted under the testing and observations of a third party testing firm. If compaction data from previous earthwork activities is available, ECS should be given the opportunity to review this data and make revisions to this report if needed.

Undocumented fill poses risks associated with deleterious inclusions within the fill and/or deleterious materials at the virgin ground fill interface that are covered by the fill. Deleterious materials can consist of significant amounts of organics derived from organic rich strippings, rubbish, construction or demolition debris, stumps and roots, and logs. If these materials are covered over by or are within undocumented fill, the organic materials tend to decompose slowly in the anaerobic conditions in or under the fill. Decomposition can occur over periods ranging from several years to several decades. As the organic materials decompose, a void is created which can create soft conditions and even subsidence in areas above the organics. Where these types of conditions exist under or within undocumented fill, they are sometimes in discreet pockets that can go undetected by normal subsurface exploration techniques, i.e., soil test borings and test pits.

The magnitude of settlement or subsidence associated with the organic materials is generally related to the volume of organic materials. In addition to settlement concerns related to decomposition of organic material, undocumented fill will be subject to settlement as a result of the addition of new loads.

5.2. Moisture Sensitive Soils (MH)

Elastic SILTS (MH) were encountered within the fill soils at Borings B-10 and B-12 to depths ranging from approximately 5.5 to 12 feet below ground surface. Soils classified as MH are fine-grained and have a Liquid Limit greater than 50 percent. Additionally, MH soils are moisture sensitive soils and tend to shrink and swell with moisture variations.

MH soils with a plasticity index greater than 30 should not be used for direct support of project foundations, slabs-on-grade, or pavements. Residual MH soils (Pl’s greater than 30) soils encountered within proposed structural areas should be undercut and replaced with low plasticity engineered fill to a minimum depth of 2 feet below foundations and 2 feet below subgrade elevations in slab and pavement areas. Upon completion of the removal, the resulting subgrade soils should be evaluated for stability prior to placement of engineered fill.

5.3. Seismic Site Class

The North Carolina Building Code (NCBC) requires that the stiffness of the top 100-ft of soil profile be evaluated in determining a site seismic classification. The method for determining the Site Class is presented in Section 1615 of the code. The seismic Site Class is typically determined by calculating a weighted average of the N-values or shear wave velocities recorded to a depth of 100 feet within the proposed building footprint. Based on the SPT N-values obtained within the drilled depth of borings, a seismic site class of “D” is considered appropriate for this project.
5.4. Structure Foundations (Simulation Building)

The foundation design at this site requires special consideration to address the presence of the relatively deep undocumented fill at the site. The following sections present foundation support options for:

- Option 1: Supporting the structure on spread footing foundations on the existing undocumented fill.
- Option 2: Supporting the structure on spread footing foundations with the undercut of existing fill.
- Option 3: Supporting the structure on a ground improvement system such as aggregate piers

The selection of the most appropriate foundation system should weigh the financial cost of the system with the intended use, settlement tolerances of the structure, and level of risk.

Option 1 – Spread Footing Foundations on Undocumented Fill

Provided the recommendations outlined herein are implemented, the proposed building can be adequately supported on a shallow foundation system consisting of spread footings bearing on approved existing fill soils, provided the owner has accepted the risk associated with bearing the structures on undocumented fill soils. While ECS does not possess testing records of the existing fill soils, the qualitative properties of the soils are indicative of engineered fill.

A bearing capacity of up to 2,000 psf is recommended for foundations bearing on approved existing fill. In areas where MH/CH soils are encountered at foundation bearing grades, a minimum of 2 feet should be undercut from beneath the footing and replaced with either additional concrete or engineered fill. If soil backfilling is performed, oversizing of the footing should be performed at a minimum of 1H:1V.

For this project, minimum wall and column footing dimensions of 18 and 24 inches, respectively, should be maintained to reduce the possibility of a localized, “punching” type, shear failure. Exterior foundations and foundations in unheated areas should be embedded deep enough below exterior grades to reduce potential movements from frost action or excessive drying shrinkage. For this region, we recommend footings bear at least 18 inches below finished grade.

Total settlement is anticipated to be less than 1 inch, while differential settlement between columns is anticipated to be less than ½ inch for shallow foundations bearing on undisturbed low plasticity residual soil or newly-placed engineered fill. Foundations bearing in existing fill may exceed the anticipated settlement provided within this section. Foundation geometry, loading conditions, and/or bearing strata different than those described in this report may result in magnitudes of settlement inconsistent with the previous estimates.

Option 2 – Spread Footing Foundations with Undercutting the Existing Fill

The proposed building can be adequately supported on a shallow foundation system consisting of spread footings bearing on firm residual soils or newly placed engineered fill. This option would require undercutting up to 12 feet to remove the existing fill. However, this option may be cost and schedule prohibitive.

A bearing capacity of up to 3,000 psf is recommended for foundations bearing on residual soils or newly placed engineered fill.
For this project, minimum wall and column footing dimensions of 18 and 24 inches, respectively, should be maintained to reduce the possibility of a localized, “punching” type, shear failure. Exterior foundations and foundations in unheated areas should be embedded deep enough below exterior grades to reduce potential movements from frost action or excessive drying shrinkage. For this region, we recommend footings bear at least 18 inches below finished grade.

Total settlement is anticipated to be less than 1 inch, while differential settlement between columns is anticipated to be less than ½ inch for shallow foundations bearing on undisturbed low plasticity residual soil or newly-placed engineered fill. Foundation geometry, loading conditions, and/or bearing strata different than those described in this report may result in magnitudes of settlement inconsistent with the previous estimates.

Option 3 – Shallow Foundations with Ground Improvement

It may prove cost prohibitive to remove up to 12 feet of existing fill from within the influence of foundations. An alternate approach would incorporate a ground improvement system to reinforce the undocumented fill soils. For this project we recommend the ground improvement consist of aggregate piers.

After installation of the aggregate piers, traditional spread footing foundations are constructed on the improved soil and aggregate piers. Aggregate piers are typically installed by a design/build contractor. Should this option be pursued, the bid documents should specify a minimum allowable bearing pressure and allowable total and differential settlement tolerances.

The design of the ground improvement system should account for differential settlements between un-reinforced and reinforced soils. If ground improvement is selected, the contract documents should specify a maximum allowable total and differential settlement of the ground improvement system. These values will depend on the structural tolerances of the building.

The drilled aggregate pier system should be designed by a design-build contractor and the proposed soil improvement plan should be reviewed by the Geotechnical Engineer of Record (GER) before construction begins. While design of this system would be performed by others, the design should be such that total settlements are limited to 1 inch and differential settlements are limited to ½ inch. The design-build contractor should also be made aware of changes in site grades required to achieve final site grades and should plan construction sequencing accordingly.

5.4.1. Horizontal Dynamic Loading Conditions (Soil Springs)

Soil spring values at the simulator mat foundation were requested. We anticipate that the simulator foundation will be supported on residual soils, structural fill, or aggregate piers. ECS was provided with loading and limited foundation information for the simulator foundation.

For the analysis, ECS considered a rectangular mat foundation approximately 32½ ft by 36 ft in plan view. ECS estimated shear modulus (G) values and shear wave velocities (V_s) based on the soils encountered during the exploration and correlations to the SPT N-value. The dynamic springs presented below could be refined with in-situ shear wave velocity testing and/or laboratory tests. For the structural fill soil spring values our assumptions are based on estimated shear modulus and shear wave velocities based on well compacted granular fill.
Based on the analysis, the recommended soil spring values for the corresponding boring locations are shown in the table below.

<table>
<thead>
<tr>
<th>Boring</th>
<th>N-Value*</th>
<th>Shear Wave Velocity**</th>
<th>Shear Modulus**</th>
<th>Dynamic Coefficient</th>
<th>Dynamic Stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-10</td>
<td>8</td>
<td>596</td>
<td>1270</td>
<td>0.8</td>
<td>2.0E+08</td>
</tr>
</tbody>
</table>

*Uncorrected averaged N-Value

**"Foundation Engineering Handbook" by Hsai-Yang Fang, equations 15.11b and 15.13.

The static and dynamic soil stiffness value is a function of the foundation element dimension and shape. At the time of this report, ECS was not provided with the location of the simulator foundation. The estimated spring values presented herein may not be valid depending on final foundation layout. ECS should be given the option to review the foundation plan to determine if the values presented herein are adequate.

5.5. Slab-On-Grade Support

Slabs-on-grade can be adequately supported on undisturbed low plasticity residual soils, approved existing fill, or newly-placed engineered fill provided the site preparation (Section 6.1) and fill recommendations (Section 6.2) outlined herein are implemented. For a properly prepared site, a modulus of subgrade reaction ($k_s$) for the soil of 90 pounds per cubic inch for the soil can be used. This value is representative of a 30 inch diameter loaded area and may need to be adjusted depending on the size and shape of the loaded area depending on the method of structural analysis.

We recommend the slabs-on-grade be underlain by a minimum of 4 inches of granular material having a maximum aggregate size of 1½ inches and no more than 2 percent fines. Prior to placing the granular material, the floor subgrade soil should be properly compacted, proofrolled, and free of standing water, mud, and frozen soil. A properly designed and constructed capillary break layer can often eliminate the need for a moisture retarder and can assist in more uniform curing of concrete. If a vapor retarder is considered to provide additional moisture protection, special attention should be given to the surface curing of the slabs to minimize uneven drying of the slabs and associated cracking and/or slab curling. The use of a blower or cushion layer above the vapor retarder can also be considered for project specific reasons.

Please refer to ACI 302.1R96 Guide for Concrete Floor and Slab Construction and ASTM E 1643 Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs for additional guidance on this issue.

ECS recommends that the slab be isolated from the footings so differential settlement of the structure will not induce shear stresses on the floor slab. Also, in order to minimize the crack width of shrinkage cracks that may develop near the surface of the slab, we recommend mesh reinforcement as a minimum be included in the design of the floor slab. For maximum effectiveness, temperature and shrinkage reinforcements in slabs on ground should be positioned in the upper third of the slab thickness. The Wire Reinforcement Institute recommends the mesh reinforcement be placed 2 inches below the slab surface or upper one-third of slab thickness, whichever is closer to the surface.
Adequate construction joints, contraction joints and isolation joints should also be provided in the slab to reduce the impacts of cracking and shrinkage. Please refer to ACI 302.1R-96 Guide for Concrete Floor and Slab Construction for additional information regarding concrete slab joint design.

5.6. Pavement Considerations

For the design and construction of exterior pavements for vehicular traffic only, the subgrades should be prepared in accordance with the recommendations in the “Site and Subgrade Preparation” and “Engineered Fill” sections of this report. This section does not include pavement recommendations for plane traffic or runway design.

Undisturbed low-plasticity natural soils or newly placed engineered fill can provide adequate support for a pavement structure designed for appropriate subgrade strength and traffic characteristics. Additionally, pavements can be supported on approved existing fill provided that the owner has accepted the risk associated with the existing fill as previously described in this report.

An important consideration with the design and construction of pavements is surface and subsurface drainage. Where standing water develops, either on the pavement surface or within the aggregate base course layer, softening of the subgrades and other problems related to the deterioration of the pavement can be expected. Furthermore, good drainage should help reduce the possibility of the subgrade materials becoming saturated during the normal service period of the pavement.

Based on our past experience with similar developments and subsurface conditions, we present the following design pavement sections, provided the recommendations contained in this report are strictly followed. Based on the soil types encountered in the soil test borings and provided the site grading recommendations outlined herein are implemented, we recommend a CBR value of 4 be used in design of the project pavements. Based upon our previous experience with similar projects, ECS has estimated the provided pavement sections based upon a 20 year life, with equivalent axle loadings of approximately 25,000 and 200,000 ESALs for light-duty and heavy-duty pavements, respectively.

<table>
<thead>
<tr>
<th>Material Designation</th>
<th>Light Duty Asphalt Pavement</th>
<th>Heavy Duty Asphalt Pavement</th>
<th>Portland Cement Concrete (PCC) Pavement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt Surface Course (S9.5B)</td>
<td>3 inches</td>
<td>1.5 inches</td>
<td>-</td>
</tr>
<tr>
<td>Intermediate Coarse (I19.0B)</td>
<td>-</td>
<td>2.5 inches</td>
<td>-</td>
</tr>
<tr>
<td>Portland Cement Concrete</td>
<td>-</td>
<td>-</td>
<td>6 inches</td>
</tr>
<tr>
<td>Aggregate Base Course</td>
<td>6 inches</td>
<td>8 inches</td>
<td>6 inches</td>
</tr>
</tbody>
</table>

ECS should be allowed to review these recommendations and make appropriate revisions based upon the formulation of the final traffic design criteria for the project. It is important to note that the design sections do not account for construction traffic loading. In addition, individual retailers may have minimum pavement sections that were not accounted for during our pavement evaluation.

The aggregate base course materials beneath pavements and sidewalks should be compacted to at least 95 percent of their modified Proctor maximum dry density (ASTM D 1557). Front-
loading trash dumpsters frequently impose concentrated front-wheel loads on pavements during loading. This type of loading typically results in rutting of bituminous pavements and ultimately pavement failures and costly repairs. Similarly, drive-thru lanes also create severe risk of rutting and scuffing. Therefore, we suggest that the pavements in trash pickup and drive-thru areas utilize the aforementioned Portland Cement Concrete (PCC) pavement section. It may be prudent to use rigid pavement sections in all areas planned for heavy truck traffic. Such a PCC section would typically consist of 6 inches of 4,000 psi, air-entrained concrete over not less than 6 inches of compacted aggregate base course. Appropriate steel reinforcing such as welded-wire fabric and/or rebar and jointing should also be incorporated into the design of all PCC pavements.

It should be noted that these design recommendations may not satisfy the North Carolina Department of Transportation traffic guidelines. Any roadways constructed for public use and to be dedicated to the State for repair and maintenance must be designed in accordance with the State requirements.

5.7. **Below Grade Excavation**

Information regarding the depth of the planned underground utilities or site grades was not provided at the time of this report. Based on the results of the soil test borings and the anticipated cut/fill depths, we do not anticipate difficult excavation will be encountered.

As noted in the Geology section of this report, the weathering process in the Piedmont can be erratic and significant variations of the depths of the more dense materials can occur in relatively short distances. In some cases, isolated boulders or thin rock seams may be present in the soil matrix.

5.8. **Cut and Fill Slopes**

ECS does not anticipate cut or fill slopes greater than 10 feet in height. We recommend that permanent cut slopes with less than 10 feet crest height through undisturbed residual soils be constructed at 2:1 (horizontal: vertical) or flatter. Permanent fill slopes less than 10 feet tall may be constructed using engineered fill at a slope of 2.5:1 or flatter. However, a slope of 3:1 or flatter may be desirable to permit establishment of vegetation, safe mowing, and maintenance. The surface of all cut and fill slopes should be adequately compacted. All permanent slopes should be protected using vegetation or other means to prevent erosion.

A slope stability analysis should be performed on cut and fill slopes exceeding 10 feet in height to determine a slope inclination resulting in a factor of safety greater than 1.4. Upon finalization of site civil drawings, ECS should be contacted to perform slope stability analysis and determine if further exploration is necessary.

The outside face of building foundations and the edges of pavements placed near slopes should be located an appropriate distance from the slope. Buildings or pavements placed at the top of fill slopes should be placed a distance equal to at least 1/3 of the height of the slope behind the crest of the slope. Buildings or pavements near the bottom of a slope should be located at least 1/2 of the height of the slope from the toe of the slope. Slopes with structures located closer than these limits or slopes taller than the height limits indicated should be specifically evaluated by the geotechnical engineer and may require approval from the building code official.

Temporary slopes in confined or open excavations should perform satisfactorily at inclinations of 2:1. All excavations should conform to applicable OSHA regulations. Appropriately sized
ditches should run above and parallel to the crest of all permanent slopes to divert surface runoff away from the slope face. To aid in obtaining proper compaction on the slope face, the fill slopes should be overbuilt with properly compacted structural fill and then excavated back to the proposed grades.

5.9. Lateral Earth Pressures

ECS understands that below grade walls may be utilized for the building. Specifics regarding the below grade walls (i.e. location, height, length, loading, etc.) were unknown at the time of this report. Below grade walls should be designed to withstand the lateral earth pressures exerted upon them, and to resist additional lateral pressures generated by surcharge loads such as traffic loads, adjacent slab loads or from foundations bearing behind the walls.

For wall conditions where wall movement cannot be tolerated or where the wall is restrained at the top, such as the loading dock walls, the “At Rest” earth pressure should be used. For wall conditions where outward wall movement on the order of 0.5 percent of the wall height can be tolerated, the “Active” earth pressure should be used. In the design of below grade walls to restrain compacted backfill, engineered fill or in-situ residual soils, the coefficient of lateral earth pressure can be used to determine lateral earth pressure loads. Please note that the values presented below are for on-site ML and SM materials. If the wall backfill is imported to the site, ECS should be contacted to review the lateral earth pressure coefficients provided. Moderately to highly elastic/plastic soils (CL, MH, and CH) should not be utilized behind earth retaining structures.

<table>
<thead>
<tr>
<th>Soil Parameter</th>
<th>Coefficient of Lateral Earth Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>“At Rest” Earth Pressure (K₀)</td>
<td>0.56</td>
</tr>
<tr>
<td>“Active” Earth Pressure (Kₐ)</td>
<td>0.39</td>
</tr>
<tr>
<td>“Passive” Earth Pressure (Kₚ)</td>
<td>2.56</td>
</tr>
</tbody>
</table>

The lateral earth pressure values presented above assume level backfill fill behind the wall, and do not account for hydrostatic pressures against the walls or surcharge loads from overlying or nearby construction.

Resistance to sliding can be provided by friction between the bottom of the wall foundation and the underlying soils and by passive resistance of soil adjacent to the wall foundation. The passive resistance should only be used in situations where the soil adjacent to the toe of the wall will not be eroded or otherwise removed in the future. A coefficient of friction of 0.35 for concrete bearing on approved soils is recommended.

Drainage behind freestanding retaining walls is considered essential towards relieving hydrostatic pressures. Drainage can be established by providing a perimeter drainage system located just above the below grade/retaining wall footings which discharges by gravity flow to a suitable outlet. This system should consist of “perforated pipe” or “porous wall”, closed-joint drain lines. These drain lines should be surrounded by a minimum 6 inches of free-draining, granular filter material having a gradation compatible with the size of the openings utilized in the drain lines and the surrounding soils to be retained, or by gravel wrapped in filter fabric. The space between the interior face of the wall and the earth fill should be backfilled with a granular fill of porous quality or better extending from the perimeter drainage system to just below the top of the wall. To prevent frost heave effects from acting against these walls, the granular backfill should extend a minimum of 12 horizontal inches behind the wall. The granular backfill should
be capped with pavement, concrete, or a 12-inch layer of low permeable silt or clay to minimize the seepage of water into that backfill from the surface. The ground surface adjacent to the below-grade walls should be kept properly graded to prevent ponding of water adjacent to the walls.

5.10. Mechanically Stabilized Earth (MSE) Wall Design

We understand that retaining walls may be utilized on this project. The performance of the MSE Walls is highly dependent upon sound design and construction practices. The design of the MSE Walls shall consider internal, external and global stability. The following table summarizes the recommended minimum factors of safety (FS) for static design criteria, as recommended by the National Concrete Masonry Association (NCMA).

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Static</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Sliding</td>
<td>1.5</td>
</tr>
<tr>
<td>Overturning</td>
<td>2.0</td>
</tr>
<tr>
<td>Internal Sliding</td>
<td>1.5</td>
</tr>
<tr>
<td>Tensile Overstress</td>
<td>1.5</td>
</tr>
<tr>
<td>Pullout</td>
<td>1.5</td>
</tr>
<tr>
<td>Connection</td>
<td>1.5</td>
</tr>
<tr>
<td>Internal Compound Stability</td>
<td>1.3</td>
</tr>
<tr>
<td>Bearing Capacity</td>
<td>2.0</td>
</tr>
<tr>
<td>Global Stability</td>
<td>1.3 to 1.5</td>
</tr>
</tbody>
</table>

The results of the required internal and geotechnical stability analyses are highly dependent upon the engineering properties of the retained, reinforced and foundation zone materials. Consequently, the design of the MSE Walls requires the assignment of specific engineering properties to the reinforced, retained and foundation zone soils. Required for design are the soil’s total in-place unit weight and peak effective friction angle and cohesion. However, cohesion is typically ignored for all materials except the foundation zone materials.

Maintaining the integrity of the reinforced zone is critical to wall performance. Any below grade utilities should be situated outside the reinforced zone to limit potential conflicts between the reinforcement and below grade structures. The wall designer should contemplate the location and use of any below grade utilities during the design process, and should coordinate with the Civil Engineer where possible to relocate the utilities outside of the reinforced zone.

ECS has completed a subsurface exploration and laboratory testing program. The incorporation of subsurface information and laboratory test results into the design of a MSE Wall requires the evaluation of available information to develop design parameters. The laboratory results provided in this report represent the soil parameters for the materials selected and should be considered for information only. Interpretation of laboratory test results and development of design parameters (i.e. shear strength) is solely the responsibility of the MSE Wall designer.

Regardless of MSE Wall geometry, it is desirable to use select fill materials within the reinforced fill zone of the MSE Wall. Granular fill is typically easier to place and compact, have enhanced drainage characteristics, have greater strengths and are less susceptible to long term movements (creep). The following gradation is recommended for the reinforced zone fill materials.
Gradation Requirements for Reinforced Zone Fill

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 inch</td>
<td>100</td>
</tr>
<tr>
<td>No. 4</td>
<td>20 to 100</td>
</tr>
<tr>
<td>No. 40</td>
<td>0 to 60</td>
</tr>
<tr>
<td>No. 200</td>
<td>0 to 35</td>
</tr>
</tbody>
</table>

The wall designer should specify allowable backfill material including unit weight, relative compaction and shear strength requirements as well as a testing frequency to verify compaction and design shear strength properties.

In some instances it may be acceptable for low plasticity (LL<50 and PI<20) fine grain soils to be used in MSE Wall construction. However, the use of fine-grained soils can result in poor internal and surface drainage, as well as time dependent movements of the MSE Wall and related components. If fine grained low plasticity soils are used for MSE Wall backfill, we recommend additional internal drainage such as a chimney drain should be incorporated into the final wall design.

The preceding paragraphs and tables are intended to provide a general overview of the design and construction of the MSE Walls. Specific guidance regarding the design and construction of MSE Walls can be found in the current edition of the NCMA Design Manual for Segmental Retaining Walls. The information provided above does not alleviate the MSE Wall designer from any aspect of the design including selection of shear strength parameters, internal wall stability, external wall stability, global stability or settlement estimates.

6. CONSTRUCTION CONSIDERATIONS

6.1. Site Preparation

Prior to construction, the proposed construction area should be stripped of all topsoil, organic material, existing undocumented fill within the building footprint (if this option is selected), and other soft or unsuitable material. Upon completion of these razing and stripping operations, the exposed subgrade in areas to receive fill should be proofrolled with a loaded dump truck or similar pneumatic-tired vehicle having a loaded weight of approximately 25 tons. After excavation, the exposed subgrades in cut areas should be similarly proofrolled.

Proofrolling operations should be performed under the observation of a geotechnical engineer or his authorized representative. The proofrolling should consist of two (2) complete passes of the exposed areas, with each pass being in a direction perpendicular to the preceding one. Any areas which deflect, rut or pump during the proofrolling, and fail to be remedied with successive passes, should be undercut to suitable soils and backfilled with compacted fill.

The ability to dry wet soils, and therefore the ability to use them for fill, will likely be enhanced if earthwork is performed during summer or early fall. If earthwork is performed during winter or after appreciable rainfall then subgrades may be unstable due to wet soil conditions, which could increase the amount of undercutting required. Drying of wet soils, if encountered, may be accomplished by spreading and disk ing or by other mechanical or chemical means. Drying and stabilizing of wet soils by chemical means can generally be achieved by the addition of 2 to 4% lime or cement; however, each case should be analyzed based on soil types and conditions during construction.
6.2. Fill Material and Placement

The project fill should be soil that has less than five percent organic content and a liquid limit and plasticity index less than 50 and 30, respectively. Soils with Unified Soil Classification System group symbols of SP, SW, SM, SC, and ML are generally suitable for use as project fill. Soils with USCS group symbol of CL that meet the restrictions for liquid limit and plasticity index are also suitable for use as project fill. Soils with USCS group symbol of MH or CH (high plasticity soil) or corrosive soils are generally not suitable for use as project fill.

The fill should exhibit a maximum dry density of at least 90 pounds per cubic foot, as determined by a standard Proctor compaction test (ASTM D-698). We recommend that moisture control limits of -3 to +2 percent of the optimum moisture content be used for placement of project fill with the added requirement that fill soils placed wet of optimum remain stable under heavy pneumatic-tired construction traffic. During site grading, some moisture modification (drying and/or wetting) of the onsite soils will likely be required.

Project fill should be compacted to at least 95 percent of its standard Proctor maximum dry density except within 24 inches of finished soil subgrade elevation beneath slab-on-grade and pavements. Within the top 24 inches of finished soil subgrade elevation beneath slab on grade and pavements, the approved project fill should be compacted to at least 100 percent of its standard Proctor maximum dry density. Aggregate base course (ABC stone) should be compacted to 95 percent of modified Proctor maximum dry density. However, for isolated excavations around footing locations or within utility excavations, a hand tamper will likely be required. ECS recommends that field density tests be performed on the fill as it is being placed, at a frequency determined by an experienced geotechnical engineer, to verify that proper compaction is achieved.

The maximum loose lift thickness depends upon the type of compaction equipment used. The table below provides maximum loose lifts that may be placed based on compaction equipment.

<table>
<thead>
<tr>
<th>Equipment</th>
<th>Maximum Loose Lift Thickness, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large, Self-Propelled Equipment (CAT 815, etc.)</td>
<td>8</td>
</tr>
<tr>
<td>Small, Self-Propelled or Remote Controlled (Rammax, etc.)</td>
<td>6</td>
</tr>
<tr>
<td>Hand Operated (Plate Tamps, Jumping Jacks, Wacker-Packers)</td>
<td>4</td>
</tr>
</tbody>
</table>

ECS recommends that fill operations be observed and tested by an engineering technician to determine if compaction requirements are being met. The testing agency should perform a sufficient number of tests to confirm that compaction is being achieved. For mass grading operations we recommend a minimum of one density test per 2,500 SF per lift of fill placed or per 1 foot of fill thickness, whichever results in more tests. When dry, the majority of the site soil should provide adequate subgrade support for fill placement and construction operations. When wet, the soil may degrade quickly with disturbance from construction traffic. Good site drainage should be maintained during earthwork operations to prevent ponding water on exposed subgrades.

We recommend at least one test per 1 foot thickness of fill for every 100 linear ft of utility trench backfill. Where fill will be placed on existing slopes, we recommend that benches be cut in the existing slope to accept the new fill. All fill slopes should be overbuilt and then cut back to expose compacted material on the slope face. While compacting adjacent to below-grade walls,
heavy construction equipment should maintain a horizontal distance of 1(H):1(V). If this minimum distance cannot be maintained, the compaction equipment should run perpendicular, not parallel to, the long axis of the wall.

6.3. Foundation Construction & Testing

Foundation excavations should be tested to confirm adequate bearing prior to installation of reinforcing steel or placement of concrete. Unsuitable soils should be undercut to firm soils and the undercut excavations should be backfilled with compacted controlled fill. Exposure to the environment may weaken the soils at the footing bearing level if the foundation excavations remain open for too long a time; therefore, foundation concrete should be placed the same day that foundations are excavated. If the bearing soils are softened by surface water intrusion or exposure, the softened soils must be removed from the foundation excavation bottom immediately prior to placement of concrete. If the excavation must remain open overnight, or if rainfall becomes imminent while the bearing soils are exposed, a 1- to 3-inch thick "mud mat" of "lean" concrete may be placed on the bearing surface to protect the bearing soils. The mud mat should not be placed until the bearing soils have been tested for adequate bearing capacity. Foundations undercut should be backfilled with engineered fill. If lean concrete is placed within the undercut zone, the foundation footprint does not require oversizing. However, if soil or ABC stone is used in lieu of lean concrete, the foundation footprint should be oversized on a 1V:1H scale.

We recommend testing all shallow foundations to confirm the presence of foundation materials similar to those assumed in the design. We recommend the testing consist of hand auger borings with Dynamic Cone Penetrometer (DCP) testing performed by an engineer or engineering technician.

7. GENERAL COMMENTS

The borings performed at this site represent the subsurface conditions at the location of the borings only. Due to the prevailing geology and presence of existing undocumented fill, changes in the subsurface conditions can occur over relatively short distances that have not been disclosed by the results of the borings performed. Consequently, there may be undisclosed subsurface conditions that require special treatment or additional preparation once these conditions are revealed during construction.

Our evaluation of foundation support conditions has been based on our understanding of the site and project information and the data obtained in our exploration. The general subsurface conditions utilized in our foundation evaluation have been based on interpolation of subsurface data between and away from the test holes. If the project information is incorrect or if the structure locations (horizontal or vertical) and/or dimensions are changed, please contact us so that our recommendations can be reviewed. The discovery of any site or subsurface conditions during construction which deviate from the data outlined in this exploration should be reported to us for our evaluation. The assessment of site environmental conditions for the presence of pollutants in the soil, rock, and groundwater of the site was beyond the scope of this exploration.

The recommendations outlined herein should not be construed to address moisture or water intrusion effects after construction is completed. Proper design of landscaping, surface and subsurface water control measures are required to properly address these issues. In addition, proper operation and maintenance of building systems is required to minimize the effects of moisture or water intrusion. The design, construction, operation, and maintenance of waterproofing and dampproofing systems are beyond the scope of services for this project.
APPENDIX

Site Vicinity Map
Boring Location Diagram
Laboratory Testing Summary
Borelogs
ASFE Documents
FIGURE 1
Boring Location Diagram
NCANG C-17 Hangar Project - Additional GEO
Charlotte, North Carolina

Source: Google Maps
LEGEND:

> = Approximate Location of Boring (February 2017)

> = Approximate Location of Boring (October 2016)

FIGURE 2

Boring Location Diagram
NCANG C-17 Hangar Project - Additional GEO
Charlotte, North Carolina
# Laboratory Testing Summary

| Sample Source | Sample Number | Depth (feet) | MC (%) | Soil Type | Atterberg Limits | Percent Passing No. 200 Sieve | Maximum Density (pcf) | Optimum Moisture (%) | CBR Value | Other |
|---------------|---------------|-------------|--------|-----------|------------------|------------------------------|-----------------------|----------------------|-----------|
| B-10          | S-3           | 6.00 - 7.50 | 19.9   | CH        |                  | 68                           | 32                    | 36                   |           |       |
| B-11          | S-2           | 3.50 - 5.00 | 16.0   |           |                  |                             |                       |                      |           |       |
| B-12          | S-1           | 0.00 - 1.50 | 21.8   | MH        |                  | 52                           | 32                    | 20                   |           |       |
|               | S-2           | 3.50 - 5.00 | 9.6    |           |                  |                             |                       |                      |           |       |

**Notes:**

**Definitions:**
- MC: Moisture Content
- Soil Type: USCS (Unified Soil Classification System)
- LL: Liquid Limit
- PL: Plastic Limit
- PI: Plasticity Index
- CBR: California Bearing Ratio
- OC: Organic Content (ASTM D 2974)

**Project No.:** 11868-A
**Project Name:** NCANG C-17 Hangar Project - Additional GEO
**PM:** Michael R Bailey
**PE:** Lee J. McGuinness
**Printed On:** Thursday, March 09, 2017
<table>
<thead>
<tr>
<th>DEPTH (FT)</th>
<th>SAMPLE NO.</th>
<th>SAMPLE TYPE</th>
<th>SAMPLE DIST. (IN)</th>
<th>SURFACE ELEVATION</th>
<th>DESCRIPTION OF MATERIAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>S-1</td>
<td>SS</td>
<td>18 8</td>
<td>699</td>
<td>Asphalt Depth [3.00&quot;]</td>
</tr>
<tr>
<td>5</td>
<td>S-2</td>
<td>SS</td>
<td>18 12</td>
<td></td>
<td>Gravel Depth [6.00&quot;]</td>
</tr>
<tr>
<td>5</td>
<td>S-3</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td>(MH FILL) ELASTIC SILT, Reddish Brown, Moist, Medium Stiff</td>
</tr>
<tr>
<td>5</td>
<td>S-4</td>
<td>SS</td>
<td>18 12</td>
<td></td>
<td>(ML FILL) SANDY SILT, Reddish Brown, Moist, Stiff</td>
</tr>
<tr>
<td>5</td>
<td>S-5</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td>(CH FILL) PLASTIC CLAY, Reddish Brown, Moist, Stiff</td>
</tr>
<tr>
<td>5</td>
<td>S-6</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td>(MH FILL) ELASTIC SILT, Reddish Brown, Moist, Soft</td>
</tr>
<tr>
<td>5</td>
<td>S-7</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td>(ML) RESIDUAL-SANDY SILT, Gray, Moist, Stiff to Very Stiff</td>
</tr>
<tr>
<td>5</td>
<td>S-8</td>
<td>SS</td>
<td>18 18</td>
<td></td>
<td>(SM) SILTY FINE TO MEDIUM SAND, Grayish White to Brownish Gray, Moist, Medium Dense</td>
</tr>
</tbody>
</table>

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.
**CLIENT**  
Jacobs Global Buildings

**JOB #**  
11868-A

**BORING #**  
B-10

**SHEET**  
2 OF 2

**PROJECT NAME**  
NCANG C-17 Hangar Project - Additional GEO

**SITE LOCATION**  
1st Union Road, Charlotte, Mecklenburg County, NC

**NORTHING | EASTING | STATION**

<table>
<thead>
<tr>
<th>DEPTH (FT)</th>
<th>SAMPLE NO.</th>
<th>SAMPLE TYPE</th>
<th>SAMPLE DIST. (IN)</th>
<th>SAMPLE RECOVERY (IN)</th>
<th>DESCRIPTION OF MATERIAL</th>
<th>ENGLISH UNITS</th>
<th>BOTTOM OF CASING</th>
<th>LOSS OF CIRCULATION</th>
<th>SURFACE ELEVATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>S-9</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>(SM) SILTY FINE TO MEDIUM SAND, Grayish White to Brownish Gray, Moist, Medium Dense</td>
<td></td>
<td></td>
<td></td>
<td>699</td>
</tr>
<tr>
<td>35</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>S-10</td>
<td>SS</td>
<td>18</td>
<td>18</td>
<td>(SM) SILTY FINE TO MEDIUM SAND, Brownish Gray, Moist, Loose</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(CL) SANDY CLAY, Gray, Moist, Very Stiff</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**END OF BORING @ 40.0'**

**THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.**

**WL**  
32.0

**WS**  

**WD**

**BORING STARTED**  
02/20/17

**CAVE IN DEPTH @ 17.0'**

**WL(SHW)**

**WL(ACR)**

**BORING COMPLETED**  
02/20/17

**HAMMER TYPE**  
Auto

**RIG**  
CME 55

**FOREMAN**  
LE

**DRILLING METHOD**  
2.25 HSA
The stratification lines represent the approximate boundary lines between soil types. In-situ the transition may be gradual.

**CLIENT:** Jacobs Global Buildings  
**PROJECT NAME:** NCANG C-17 Hangar Project - Additional GEO  
**SITE LOCATION:** 1st Union Road, Charlotte, Mecklenburg County, NC

**NORTHING:**  
**EASTING:**  
**STATION:**

<table>
<thead>
<tr>
<th>DEPTH (FT)</th>
<th>SAMPLE NO.</th>
<th>SAMPLE TYPE</th>
<th>SAMPLE DIST. (IN)</th>
<th>POLISHED SURFACE ELEVATION</th>
<th>DESCRIPTION OF MATERIAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>S-1</td>
<td>SS</td>
<td>18</td>
<td>702</td>
<td>Asphalt Depth [4.00&quot;]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Gravel Depth [3.00&quot;]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(ML) SANDY SILT, Orangish Brown, Moist, Stiff</td>
</tr>
<tr>
<td>5</td>
<td>S-2</td>
<td>SS</td>
<td>18</td>
<td></td>
<td>(SM) RESIDUAL- SILTY FINE TO MEDIUM SAND, Orangish Tan to Grayish Brown, Moist, Loose to Medium Dense</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(ML) SANDY SILT, Grayish Brown, Moist, Very Stiff to Hard</td>
</tr>
</tbody>
</table>

**WATER LEVELS**

- **WL:** 37.5
- **WS:**
- **WD:**

**LOSS OF CIRCULATION**

- **WL(SHW):**
- **WL(ACR):**

**BOTTOM OF CASING**

- **WL RIG:** CME 55
- **FOREMAN:** LE

**BORING STARTED:** 02/20/17  
**CAVE IN DEPTH:** @ 26.0'

**BORING COMPLETED:** 02/20/17  
**HAMMER TYPE:** Auto

**DRILLING METHOD:** 2.25 HSA

CONTINUED ON NEXT PAGE.
**NCANG C-17 Hangar Project - Additional GEO**

**SITE LOCATION**
1st Union Road, Charlotte, Mecklenburg County, NC

**NORTHING | EASTING | STATION**

<table>
<thead>
<tr>
<th>DEPTH (FT)</th>
<th>SAMPLE NO.</th>
<th>SAMPLE TYPE</th>
<th>SAMPLE DIST. (IN)</th>
<th>SAMPLE RECOVERY (IN)</th>
<th>DESCRIPTION OF MATERIAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>S-9 SS</td>
<td>15</td>
<td>12</td>
<td></td>
<td>(ML) SANDY SILT, Grayish Brown, Moist, Very Stiff to Hard</td>
</tr>
<tr>
<td>40</td>
<td>S-10 SS</td>
<td>18</td>
<td>18</td>
<td></td>
<td>(ML) SANDY SILT, Grayish Brown, Moist, Very Hard</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>END OF BORING @ 40.0'</td>
</tr>
</tbody>
</table>

**SURFACE ELEVATION**
702

**WATER LEVELS**

<table>
<thead>
<tr>
<th>670</th>
<th>665</th>
<th>660</th>
<th>655</th>
<th>650</th>
<th>645</th>
<th>640</th>
<th>635</th>
<th>630</th>
<th>625</th>
<th>620</th>
<th>615</th>
<th>610</th>
<th>605</th>
<th>600</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**WATER LEVELS ELEVATION (FT)**

<table>
<thead>
<tr>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DEPTH (FT) SAMPLE NO. SAMPLE TYPE SAMPLE DIST. (IN) SAMPLE RECOVERY (IN) DESCRIPTION OF MATERIAL STATION**

<table>
<thead>
<tr>
<th>SURFACE ELEVATION</th>
<th>BORING STARTED</th>
<th>CAVE IN DEPTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>702</td>
<td>02/20/17</td>
<td>@ 26.0'</td>
</tr>
</tbody>
</table>

**THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.**

**ENG. UNITS**

<table>
<thead>
<tr>
<th>BOTTOM OF CASING</th>
<th>LOSS OF CIRCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>(ML) SANDY SILT, Grayish Brown, Moist, Very Stiff to Hard</td>
<td></td>
</tr>
</tbody>
</table>

**PLASTIC LIMIT%**

<table>
<thead>
<tr>
<th>WATER CONTENT%</th>
<th>LIQUID LIMIT%</th>
</tr>
</thead>
<tbody>
<tr>
<td>20%</td>
<td>40%</td>
</tr>
<tr>
<td>60%</td>
<td>80%</td>
</tr>
<tr>
<td>100%</td>
<td></td>
</tr>
</tbody>
</table>

**ROCK QUALITY DESIGNATION & RECOVERY**

<table>
<thead>
<tr>
<th>RQD%</th>
<th>REC%</th>
</tr>
</thead>
<tbody>
<tr>
<td>702</td>
<td>50/3</td>
</tr>
</tbody>
</table>

**STANDARD PENETRATION**

<table>
<thead>
<tr>
<th>BLOWS/6&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
</tr>
</tbody>
</table>

**CALIBRATED PENETROMETER TONS/FT²**

<table>
<thead>
<tr>
<th>x STANDARD PENETRATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
</tr>
</tbody>
</table>

**WATER LEVELS**

<table>
<thead>
<tr>
<th>670</th>
<th>665</th>
<th>660</th>
<th>655</th>
<th>650</th>
<th>645</th>
<th>640</th>
<th>635</th>
<th>630</th>
<th>625</th>
<th>620</th>
<th>615</th>
<th>610</th>
<th>605</th>
<th>600</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DRILLING METHOD**

2.25 HSA

<table>
<thead>
<tr>
<th>RIG</th>
<th>FOREMAN</th>
</tr>
</thead>
<tbody>
<tr>
<td>CME 55</td>
<td>LE</td>
</tr>
</tbody>
</table>

**RIG | FOREMAN | 2.25 HSA**

**Hammer Type**

Auto

**WL**

37.5

**WL(SHW)**

02/20/17

**WL(ACR)**

02/20/17

**WL**

Cave in Depth @ 26.0'
Asphalt Depth [3.00"
Gravel Depth [3.00"
(MH FILL) ELASTIC SILT, Reddish Brown,
Moist, Stiff
(SM) Residual- SILTY FINE TO MEDIUM
SAND, Tannish Brown to Grayish White, Moist,
Loose to Medium Dense
(SM PWR) PARTIALLY WEATHERED ROCK
SAMPLED AS SILTY FINE TO MEDIUM SAND,
Grayish White

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

CLIENT Jacobs Global Buildings

PROJECT NAME NCANG C-17 Hangar Project - Additional GEO

SITE LOCATION
1st Union Road, Charlotte, Mecklenburg County, NC

NORTHING EASTING STATION

11868-A
B-12
1 OF 2

ENGLISH UNITS

DEPTH (FT) SAMPLE NO. SAMPLE TYPE SAMPLE DIST. (IN) RECOVERY (IN)

SURFACE ELEVATION

DESCRIPTION OF MATERIAL ENGLISH UNITS

BOTTOM OF CASING
LOSS OF CIRCULATION

WATER LEVELS

BLOWS

PLASTIC LIMIT % WATER CONTENT % LIQUID LIMIT %

ROCK QUALITY DESIGNATION & RECOVERY

RQD% REC %

STANDARD PENETRATION BLOWS/FT

CALIBRATED PENETROMETER TONS/FT²

Boring

BoRiNg Started 02/20/17

Cave in Depth @ 19.5'

Hammer Type Auto

Drilling Method 2.25 HSA
### NCANG C-17 Hangar Project - Additional GEO

#### Site Location

1st Union Road, Charlotte, Mecklenburg County, NC

#### Project Name

NCANG C-17 Hangar Project - Additional GEO

#### Client

Jacobs Global Buildings

#### Job #

11868-A

#### Boring #

B-12

#### Sheet

2 OF 2

---

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Recovery (in)</th>
<th>Bottom of Casing</th>
<th>Water Levels</th>
<th>Surface Elevation</th>
<th>Description of Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>SS</td>
<td>1</td>
<td></td>
<td></td>
<td>725</td>
<td>(SM PWR) PARTIALLY WEATHERED ROCK \nSAMPLED AS SILTY FINE TO MEDIUM SAND, \nGrayish White \nAUGER REFUSAL @ 32.0'</td>
</tr>
</tbody>
</table>

---

**Note:**

The stratification lines represent the approximate boundary lines between soil types. In situ the transition may be gradual.
**REFERENCE NOTES FOR BORING LOGS**

### MATERIALs

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>ASPHALT</th>
<th>CONCRETE</th>
<th>GRAVEL</th>
<th>TOPSOIL</th>
<th>VOID</th>
<th>BRICK</th>
<th>AGGREGATE BASE COURSE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**FILL**

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>MAN-PLACED SOILs</th>
<th>WELL-GRADeD GRAVEl</th>
<th>POORLY-GRADeD GRAVEl</th>
<th>SILTY GRAVEl</th>
<th>POORLY-GRADeD SAND</th>
<th>SILTY SAND</th>
<th>CLAYEY SAND</th>
<th>SILT</th>
<th>ELASTIC SILT</th>
<th>LEAN CLAY</th>
<th>FAT CLAY</th>
<th>ORGANIC SILT or CLAY</th>
<th>ORGANIC SILT or CLAY</th>
<th>PEAT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### DRILLING SAMPLING SYMBOLS & ABBREVIATIONS

- **SS** Split Spoon Sampler
- **ST** Shelby Tube Sampler
- **WS** Wash Sample
- **BS** Bulk Sample of Cuttings
- **PA** Power Auger (no sample)
- **HSA** Hollow Stem Auger
- **PM** Pressuremeter Test
- **RD** Rock Bit Drilling
- **RC** Rock Core, NX, BX, AX
- **REC** Rock Sample Recovery %
- **RQD** Rock Quality Designation %

### WATER LEVELS

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>WL</td>
<td>Water Level (WS)(WD)</td>
</tr>
<tr>
<td>(WS)</td>
<td>(WD) While Sampling</td>
</tr>
<tr>
<td>(WD)</td>
<td>(WD) While Drilling</td>
</tr>
<tr>
<td>SHW</td>
<td>Seasonal High WT</td>
</tr>
<tr>
<td>ACR</td>
<td>After Casing Removal</td>
</tr>
<tr>
<td>SWT</td>
<td>Stabilized Water Table</td>
</tr>
<tr>
<td>DCI</td>
<td>Dry Cave-In</td>
</tr>
<tr>
<td>WCI</td>
<td>Wet Cave-In</td>
</tr>
</tbody>
</table>

### PARTICLE SIZE IDENTIFICATION

#### COHESIVE SILTS & CLAYS

<table>
<thead>
<tr>
<th>UNCONFINED COMPRESSIVE STRENGTH, $Q_p$</th>
<th>SPT$^5$</th>
<th>CONSISTENCY$^7$</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.25</td>
<td>&lt;3</td>
<td>Very Soft</td>
</tr>
<tr>
<td>0.25 - &lt;0.50</td>
<td>3 - 4</td>
<td>Soft</td>
</tr>
<tr>
<td>0.50 - &lt;1.00</td>
<td>5 - 8</td>
<td>Medium Stiff</td>
</tr>
<tr>
<td>1.00 - &lt;2.00</td>
<td>9 - 15</td>
<td>Stiff</td>
</tr>
<tr>
<td>2.00 - &lt;4.00</td>
<td>16 - 30</td>
<td>Very Stiff</td>
</tr>
<tr>
<td>4.00 - 8.00</td>
<td>31 - 50</td>
<td>Hard</td>
</tr>
<tr>
<td>&gt;8.00</td>
<td>&gt;50</td>
<td>Very Hard</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>RELATIVE AMOUNT$^7$</th>
<th>COARSE GRAINED (%)</th>
<th>FINE GRAINED (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trace</td>
<td>&lt;5</td>
<td>&lt;5</td>
</tr>
<tr>
<td>Dual Symbol (ex: SW-SM)</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>With</td>
<td>15 - 20</td>
<td>15 - 25</td>
</tr>
<tr>
<td>Adjective (ex: &quot;Silty&quot;)</td>
<td>25 - &lt;50</td>
<td>30 - &lt;50</td>
</tr>
</tbody>
</table>

#### GRAVELS, SANDS & NON-COHOESIVE SILTS

<table>
<thead>
<tr>
<th>SPT$^5$</th>
<th>DENSITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;5</td>
<td>Very Loose</td>
</tr>
<tr>
<td>5 - 10</td>
<td>Loose</td>
</tr>
<tr>
<td>11 - 30</td>
<td>Medium Dense</td>
</tr>
<tr>
<td>31 - 50</td>
<td>Dense</td>
</tr>
<tr>
<td>&gt;50</td>
<td>Very Dense</td>
</tr>
</tbody>
</table>

---

2. To be consistent with general practice, "POORLY GRADED" has been removed from GP, GP-GM, GP-GC, SP, SP-SM, SP-SC soil types on the boring logs.
3. Non-ASTM designations are included in soil descriptions and symbols along with ASTM symbol (Ex: (SM-FILL)).
4. Typically estimated via pocket penetrometer or Torvane shear test and expressed in tons per square foot (tsf).
5. Standard Penetration Test (SPT) refers to the number of hammer blows (blow count) of a 140 lb. hammer falling 30 inches on a 2 inch OD split spoon sampler required to drive the sampler 12 inches (ASTM D 1586). "N-value" is another term for "blow count" and is expressed in blows per foot (bpf).
6. The water levels are those levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in granular soils. In clay and cohesive silts, the determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally employed.
7. Minor deviation from ASTM D 2488-09.

Reference Notes for Boring Logs (FINAL 08-23-2016).doc

© 2016 ECS Corporate Services, LLC. All Rights Reserved
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. Parebid 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 your ASFE-member geotechnical engineer for more information.