ADDENDUM NO. 2
DATE: September 03, 2016

RE: University of South Carolina
Athletic Village Improvements – Field House Conversion
State Project Number H27-6105-MJ-C

This addendum herein supplements, modifies, changes, deletes from or adds to the original bidding documents for the project noted above and is herein made a part of the contract documents. Drawings and General Provisions of the Contract, including General and Supplementary Conditions, shall apply to items incorporated in the Addendum.

This addendum consists of 37 pages including all attachments.

The following are general changes to the bid documents:

2.1 Bidder Questions – Responses are in bold

1. Can combustible powered equipment be used if monitoring is performed for CO2?

   Means and methods. The contractor will have the responsibility under OSHA to ensure that employee exposures remain below the PELs.

2. Is concrete slab under mercury containing track separated by expansion joint or will it need to be saw cut?

   See Report of Geotechnical Exploration prepared by GS2 Engineering dated July 16, 2014.

3. What is thickness of concrete under mercury containing track?

   See Report of Geotechnical Exploration prepared by GS2 Engineering dated July 16, 2014.

4. Does asphalt under tennis courts need to be disposed of along with asbestos containing surfacing?

   The tennis court surface consists of both a green surface layer and black asphalt like layer, both containing <1% asbestos (refer to photo # 7 in the specification). Both materials are to be removed down to the substrate.

2.2 Bid Date

1. The due date for bids has been changed from September 6, 2016 at 2:00 pm to **September 13, 2016 at 1:00 pm**

End of Addendum 2

USC Field House Conversion

1400 Whaley Street
Columbia, South Carolina

GS2 Project Number 14-1144-G
July 16, 2014

Report of Geotechnical Exploration

Prepared for:

University of South Carolina
Campus Planning & Construction
743 Greene Street
Columbia, South Carolina 29208

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Charleston Branch Office
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North Charleston, South Carolina 29405

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Florence Field Testing Office
2426 Third Loop Road, Suite A
Florence, South Carolina 29501

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July 16, 2014

University of South Carolina
Campus Planning & Construction
743 Greene Street
Columbia, South Carolina 29208

Attention: **Ms. Ann Derrick**

Reference: **Report of Geotechnical Exploration**
Proposed USC Field House Conversion
1400 Whaley Street
Columbia, South Carolina
OSE Project No.: H27-6015
GS2 Project No.: 14-1144-G

Dear Ms. Derrick,

This report presents our geotechnical exploration of the Proposed University of South Carolina Field House Conversion, in Columbia, South Carolina. Information obtained from our geotechnical exploration has been used to evaluate the existing site conditions for the use of developing design parameters for the proposed development. This work was performed in general accordance with industry standards and our proposal number GS2 P4119-14, dated January 10, 2014.

Recommendations detailed in this report are specific to the soil conditions in the immediate vicinity of the boring locations for this particular project. This report does not include any environmental assessment of soils, surface water or groundwater, the determination of wetlands, the determination of noise impact, the assessment of air quality, the identification of cultural resources, and the identification of endangered species. These services are beyond the scope of services of a geotechnical exploration.

PROJECT INFORMATION

Proposed Development

We understand that the existing field house structure is being considered for renovation. Although the construction details for the proposed structure were not known at the time of this exploration, we have made several assumptions from previous and similar project experience, for modeling purposes, which are detailed below.

From our review of the provided information, it is understood that the proposed development at the site is to include the construction of a new elevator at the southwest corner of the existing structure, consisting of a roughly 3 to 4-story elevator from the existing elevation of the northeastern quadrant of the intersection of Heyward and Marion Streets to the existing slab-on-grade elevation. In addition, it is understood that the proposed development is to also include the demolition of the existing tennis courts, football field and dividing wall and the construction of a new regulation size indoor running track and offices along the interior western wall within the existing field house structure. We have assumed that the structures will be constructed utilizing a combination of reinforced CMU block and structural steel framed walls and steel framed roof systems, with an exterior metal siding veneer.

Additionally, we have assumed that the interior office wall structures will be supported by a conventional shallow foundation system, with a cast-in-place concrete slab-on-grade. Maximum wall and column loads for this type of structure are typically on the order of 1 to 2 kips per linear foot (klf) and 10 to 20 kips, respectively. Furthermore, we have assumed that the exterior elevator structure will be supported will be supported by a conventional shallow foundation system, with a cast-in-place concrete slab-on-grade. Maximum wall and column loads for this type of structure are typically on the order of 3 to 5 kips per linear foot (klf) and 30 to 50 kips, respectively.

Site specific topographic information and planned finish floor elevations were not available at the time of our exploration. Therefore, we have assumed that cuts and fills will be minimal to level the interior development areas and assumed that cuts on the order of 15 to 20 feet will be necessary for construction of the exterior elevator development.

Finally, we have assumed that the design and construction of the proposed structures will be governed by the International Building Code, Edition 2012 (IBC 2012).

SITE SETTING

Site Location

The subject site is located within the southeastern quadrant of the intersection of Whaley Street and Marion Street, at 1400 Whaley Street, in Columbia, South Carolina. The location of the site relative to the nearby streets is shown in the *Site Location Map*, Figure 1 in Appendix A.

Site Description

The subject site is roughly 4.5 acres in area, is a generally rectangular in shape. At the time of our visit the site was observed to be developed with the existing field house within the southern portion of the site and an asphalt parking lot located north of the existing field house.

The subject site was further noted to be bordered by Whaley Street to the north, Heyward Street to the south, Bull Street to the east, and Marion Street to the west. Access was gained to the site via the existing concrete driveway emanating from Bull Street.

Site Topography

Topographically, the site is located along the western side slope of a broad ridge in the Upper Coastal Plain Physiographic Province. Ground surface elevations across the site appear to range from 239 to 290 feet mean sea level. Surface runoff in the vicinity of the site appears to drain to the south by southwest towards Heyward Street and into the city municipal storm sewer system. General topographic information was obtained from the USGS Southwest Columbia and Columbia North topographic quadrangles, shown in the *USGS Topographic Map*, Figure 2 in Appendix A and Google Earth.

SUMMARY OF FIELD EXPLORATION

The subsurface conditions within the vicinity of the ***proposed structures*** at the site were explored with two (2) mechanically-augered soil borings, with Standard Penetration Tests (SPT) taken at regular intervals, while the recovered samples were visually and texturally classified by ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). These soil borings were extended to termination depths of 20 feet below the existing ground surface.

The subsurface conditions within the area of the ***proposed structures*** were further explored with two (2) test pit excavations. The test pits were excavated with a tracked excavator while the recovered samples were visually and texturally classified by ASTM D2488 *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The test pits were excavated to the termination depths of 5 feet below the existing ground surface.

Furthermore, the subsurface conditions within the area of the ***proposed structures*** were explored with six (6) hand-augered soil borings, with Dynamic Cone Penetrometer (DCP) tests taken at regular intervals to the termination depths of 10 feet below the existing ground surface.

The approximate test boring locations are shown on the attached *Boring Location Plan*, Figure 3 in Appendix A. The borings were located in the field by measuring from estimated property corners and existing site features.

SITE SOIL CONDITIONS

<u>Site Geology</u>

The subject site is located in an old river terrace formed in the Upper Coastal Plain Physiographic Province of South Carolina, in downtown Columbia. The soils of this terrace are composed of a mixture of re-deposited material washed from upstream sources of ancient rivers, and are typically mixed with rocks that vary in size and depth which have been rounded through years of exposure to flowing water. The deposits in these areas are highly variable and may cover areas of the river bed and associated flood plains, which when deposited were established in very loose and wet conditions. Ultimately these terraces are underlain by firmer materials of the Piedmont Physiographic Province.

More specifically, the geology and geomorphology of the city of Columbia are dictated by several key factors of which the Fall Line and the local River Systems are the most dominant. Upstream from the Fall Line rivers and streams typically have very small floodplains, while downstream these floodplains widen greatly. T. Frank Johnson's 1972 mapping of the Columbia quadrangles depicts the near surface soil composition for areas along the east banks of the Broad River, to about Assembly Street, and west of the Broad River to consist of material that weathered from Phyllites and Granite, with the coastal plain sediments in this area typically 35 to 50 feet thick. Additionally, geological mapping of the Columbia quadrangles depicts the near surface soil composition for areas of Columbia east of Assembly Street to consist of coastal plain and river terrace sediments on the order of 80 to 90 feet thick. In both cases the coastal plain sediments are underlain by several feet of weathered rock and Potassium Feldspar-rich Granite. The granite underlying the surface deposits is known to be metamorphic in nature, and relatively weathered.

Soil Conditions

The subsurface conditions encountered at the boring and test pit locations are detailed on the attached *Soil Test Boring*, *Record of Hand Auger Boring, and Record of Test Pit Excavation* logs. These logs represent our interpretation of the subsurface conditions at the boring locations based on our visual and textural examination of the recovered soil samples. The horizontal lines in the Soil Description column of the boring logs represent an approximate interface between various soil strata. It is important to understand that these horizontal lines represent an estimated depth of soil variance where as the actual soil change may be gradual.

Surface Materials: Surface materials, in the form of topsoil, approximately one inch in thickness, were encountered at the ground surface in Boring B-1. Additionally, Borings B-3 through B-7 encountered approximately 4 to 6 inches of concrete, and Test Pits TP-1 and TP-2 encountered 4 to 5-1/2 inches of concrete, respectively.

Proposed Structure – SPT Borings: Below the surface materials, the borings performed within the proposed structures (Borings B-1 and B-7) generally encountered old and possible fill, consisting of clayey sands and clean sands (SC and SP), to depths of up to 12 feet below the ground surface. Beneath the near-surface old and possible fill, the borings encountered native Upper Coastal Plain deposits, consisting of layers of clay (CL), to depths of up to 20 (termination depth of Boring B-7) feet below ground surface. In Boring B-1 the deeper clayey soils were underlain by a layer of clean sands (SP), to depths of up to 20 (termination depth of Boring B-1) feet below ground surface.

The near-surface old and possible fill sandy (SC and SPM) soils, exhibited SPT N-values noted to range from 6 to 45 blows per foot (bpf), indicating loose to dense relative densities. The underlying clayey (CL) soils exhibited SPT N-values noted to range from 13 to 26 bpf, indicating stiff to very stiff relative consistencies. The deeper underlying clean sands (SP) exhibited SPT N-values noted be on the order of 50 blows for 0 inches (50/0") bpf, indicating very dense relative densities.

Proposed Structures – Hand Auger Borings: The borings performed within the vicinity of the proposed structures (Borings B-2A, B-2B and B-3 through B-6) encountered old fill, consisting of clayey, clayey silty, and silty sands (SC, SC-SM, and SM), to depths of up to 7-1/2 feet below the ground surface. Beneath the near-surface old and possible fill, the borings encountered native Upper Coastal Plain deposits, consisting of clayey-silty and silty sands (SC-SM, and SM) sands to the termination depths of roughly 10 feet below the existing ground surface.

The layers of sands (SC, SC-SM, and SM) soils exhibited average DCP blow counts noted to range from 0 to 25+ blows per increment (bpi), indicating very loose to very dense relative densities.

Proposed Structures - Test Pit Excavations: Beneath the surface materials the test pit excavations encountered 5 (termination depth of Test Pits TP-1, and TP-2) feet of possible fill soils, consisting of silty and clayey sands (SM and SC).

Groundwater

Free groundwater was encountered in Boring B-2B at a depth of approximately 4-1/2 feet below the existing ground surface at the time of drilling.

Additionally, borehole cave-in was observed in Borings B-1 and B-7 at depths ranging from approximately 11-1/2 to 14-1/2 feet below the existing ground surface at the time of drilling. Borehole cave-in is sometimes an indicator of groundwater elevation.

For safety the boreholes were backfilled upon completion, therefore, 24-hour stabilized groundwater readings were not obtained. Groundwater levels are dependent on many factors and can experience seasonal fluctuations and various other fluctuations due to precipitation, construction activities, and many other factors.

SEISMIC CONSIDERATIONS

Regional Seismic Conditions

This site is situated roughly 100 miles north-northwest of Charleston, South Carolina, which is the most prominent area of seismicity along the Atlantic Seaboard. The Charleston earthquake of 1886 was the largest seismic event that has occurred in this region and damage was extensive throughout the Charleston area. The epicenter was located approximately 15 miles northwest of Charleston between the town of Summerville and Middleton Place Plantation.

Recent discoveries of relict liquefaction in the Lowcountry region of South Carolina have expanded knowledge about seismicity in the area. Evidence indicates that at least five episodes of strong prehistoric ground shaking large enough to produce widespread liquefaction have occurred within the Charleston area within the last 7500 years. The Charleston region continues to experience earthquakes of smaller magnitudes yearly.

IBC 2012 Seismic Site Class

Our analysis of the soil seismic conditions was based on the information obtained from our SPT borings, previous CPT soundings with shear wave velocities, known site and vicinity geological conditions, known regional seismic conditions, and seismic design parameters established in data published in the International Building Code 2012 (IBC 2012), section 1613.3. Therefore, from the known regional conditions, the SPT N-values measured, and the parameters established in the IBC 2012, we have determined that the site is best defined to have a **Site Class C**.

Earthquake Ground Motion

Earthquake ground motion parameters at the bedrock for this site were obtained from the International Building Code (IBC 2012) section 1613.3. The values for this site are presented in Table 1. Ground motions were obtained utilizing the mapped accelerations, with the design responses for both ground motions represented in the following sections.

Table 1: Probabilistic Ground Motion Values

Spectral Response Acceleration	Ground Motion Values for Recurrence Period (g
	2% in 50 Years (2012)
0.2 sec Sa[1]	0.420
1.0 sec Sa	0.144

Note: 1. Sa is the Spectral Response Acceleration at the noted period.

Based on the information presented in the preceding table, and the IBC 2012 section 1613.3, the corresponding site coefficients for the site are calculated to be:

- $F_a = 1.200$
- $F_v = 1.656$

Design Spectral Response

Based on the information presented in the preceding table, and the corresponding site coefficients for the site, we have calculated the Design Spectral Response Acceleration Parameters, according to IBC 2012 section 1613.3.4, for this site to be:

- $S_{DS} = 0.336$
- $S_{D1} = 0.159$

CONCLUSIONS AND RECOMMENDATIONS

The borings performed during this exploration indicate that the existing sandy soils (SP, SM, SC, and SC-SM), are ***suitable***, and the clayey soils (CL) are ***marginally suitable*** for support of the proposed structure and pavements, as well as for use as structural fill due to their inherent characteristics.

Additionally, the borings performed during this exploration indicate that the existing near-surface site soils appear adequate to support the proposed structures on a shallow foundation system. However, due to the loose near-surface soil conditions encountered at the site, it may be necessary to use minor near-surface ground modifications in order to support the proposed structures on shallow foundations.

Alternately, in the event that the design does not allow the transference of new loads to the existing structures' foundations, the use of deep foundations also appears adequate to support the proposed structures. While final design loads were not available at the time this report was written, deep foundations options for micropiles are included herein.

It is important to note that fine-grained soils such as those found at this site may be sensitive to variations in moisture content with a relatively narrow range of workable moisture contents. Therefore, close control of moisture content will be necessary during grading and fill placement operations. In addition, the soils at this site may become difficult to work during periods of wet weather. Grading operations under wet conditions may result in deterioration of otherwise suitable soil conditions, or of previously placed and properly compacted fill. These inherent soil properties make these soils less desirable for support and for use as structural fill. However, if these soils are placed properly, suitable support of the structure and pavements is achievable.

These conclusions, and the associated recommendations, are provided in the assumption that the soil conditions at the site do not vary greatly from those encountered in our borings and that our recommendations presented in the following sections of this report are followed.

Suitability of Soils

As previously stated, the near-surface soils at the site have been identified to have a ***SP***, ***SM***, ***SC***, ***SC-SM,*** and ***CL*** USCS soil classifications. Most text includes soils with Unified Soil Classifications of SW, SP, SM, SC, SM-SC, ML and CL as suitable for support of structure or for use as structural fill, while soils with classifications of MH, CH, OL and OH are considered unsuitable. Therefore, it is important to note the site contains soils that are considered in the industry to be *suitable* (SP, SM and SC) and *marginally suitable* (CL). The following sections provide more insight into each soil classification, with emphasis placed on their workability and preferred structural loading.

Soils that have SP designations are considered clean and sandy in nature, meaning the soil particles pass the No. 4 sieve but are retained on the No. 200 sieve, meaning they are smaller in size than gravel particles but larger in size than silt and clay particles. SP soil designations are poorly graded in nature, meaning the particle sizes of the material are similar with little variance. These soils are well drained and show little effect due to moisture content. Subsequently, these soils are preferred as fill material, with good structural support characteristics under buildings and pavements.

Fine-grained sandy soils (SM and SC) are similar to clean SP soils in that fact that they are sandy in nature and pass the No. 4 sieve but are retained on the No. 200 sieve, meaning they are smaller in size than gravel particles, however, these soil designations also have fine-grained silty (M) and clayey (C) particles mixed in, resulting in these designations being preferable fill soils that exhibit good structural support characteristics under buildings and pavements, however, depending on the fines (silt and clay) content, may have less ease in workability, with less flexibility in achieving compaction at various moisture contents.

Soils that have CL designations are less preferable fill soils that exhibit fair to good structural support characteristics under buildings and pavements, less ease in workability, with little flexibility in achieving compaction at various moisture contents. Consequently, these soils are less preferred for use as roadway subgrade, with a fair to poor rating, due to their instability when exposed to excessive moisture. These soils may be used as roadway subgrade, if adequate moisture control is maintained during placement and if stormwater is not allowed to pond or penetrate these soils, ultimately preventing subgrade degradation due to over-saturation.

Fine-grained soils (CL) are typically sensitive to variations in moisture content with a relatively narrow range of workable moisture contents. Therefore, close control of moisture content will probably be necessary during grading and fill placement operations, where these soils are involved. In addition, these soils may become difficult to work during periods of wet weather. Grading operations under wet conditions may result in the deterioration of otherwise suitable soil conditions, or of previously placed and properly compacted fill.

Site Preparation

General Clearing and Grubbing: Any vegetation, root mat, organic laden topsoil, surface materials and debris should be stripped and grubbed from structurally loaded or fill areas and wasted off site or in areas to be landscaped, prior to placement of structure or fill. The removal of these materials should extend at least 5 feet beyond the perimeter of the proposed structurally loaded areas. It is important to note that roughly one inch of surface materials in the form of topsoil were encountered at the ground surface.

Additionally, as the future project site falls within a previously developed property, it is ***possible*** that surface debris and ***possible*** that buried debris and utilities will be encountered during excavation activities. Therefore, any surface and/or buried debris, or underground utilities encountered where it has not otherwise been identified in this report, will need to be removed from beneath and within a 5 foot perimeter of structure or fill area, and wasted off site or in areas to be landscaped prior to placement of structural fills.

As stated previously, due to the nature of the in-place possible fill (uncontrolled) soils encountered throughout the site, we recommend that the contractor be prepared to further explore the vertical and horizontal extents of the uncontrolled fill encountered, either prior to construction or at the onset of construction, as this material may be required to be undercut during grading activities. In either case, a bid item for unclassified excavation, haul off, and soil replacement should be presented by the contractor for this activity.

General Ground Modification Recommendations: As mentioned above, due to the size of the development, and the loose near-surface soil conditions encountered at the site, it will be necessary to use near-surface ground modifications in order to support the proposed structure. In general, the modification of the near-surface soils should incorporate the reworking or densification of at least the upper 5 feet of bearing soils beneath, and 5 feet beyond, the structures' perimeters.

Once the general stripping, clearing and grubbing are complete, methods of ground modification at this site may include in-place densification or undercut and replacement, with the most effective method highly influenced by the planned subgrade and finished floor elevations.

Foundation Subgrades: The in-place densification and proper placement of fill soil will likely provide a suitable footing subgrade beneath the foundations for the planned structure at this site. However, this does not alleviate the contractor from verifying that adequately dense bearing soils are present within the foundation subgrades, as stipulated in the recommendations provided in the *Foundation and Construction Recommendations* section of this report.

Alternate Ground Modification of Foundation Subgrades: If unstable bearing soils are encountered during footing excavation, an alternate ground modification technique that may be used to remedy the bearing soils includes the over-excavation of the bearing soils directly beneath the footprint of the foundations, and the backfilling the resulting excavation with properly compacted structural fill or washed No. 57 stone, to near original bearing elevations.

Pre-Pour Inspections: After achieving a stabilized subgrade, and prior to the construction of the finished slabs and/or pavements, assuming some time will pass where the grade slab and/or pavement subgrade is exposed, the prepared subgrade will need to be re-inspected and proofrolled in order to detect locally yielding soils.

General In-Place Densification Recommendations: In-place soil densification can be accomplished using a large smooth-drum vibratory roller by making several passes over the area to be densified in a crossing pattern, after the site has been stripped. Densification in-place of loose sands yields varying results in the field, and is highly dependent upon obtaining a sufficiently large roller, the in-situ moisture content, and the ability to achieve confinement on at least one side, (i.e. along one strip), prior to proceeding to the next. Obtaining confinement in sand is typically an iterative process and requires that multiple passes along well established rolling lanes be performed, the initial passes made with the vibratory setting used and the finishing passes made with a static roller. Upon achieving an optimal densification in one direction it is recommended that the rolling efforts be repeated in the perpendicular direction, until no noticeable improvements in densification are observed.

In-place soil densification is recommended for soils in which below optimum moisture contents are present, and where groundwater is greater than 3 feet below the depth of densification required. Densification of the on-site soils should continue until an SPT N-value of 8 or an equivalent Dynamic Cone Penetrometer (DCP) value of 11 is achieved, with a target density of 98 percent of the laboratory Standard Proctor maximum dry density (ASTM D698). The densification techniques and activities should be verified as the work progresses. In the event that adequate confinement for densification is not achieved, we recommend that over-excavation and replacement be conducted.

General Proofroll Recommendations: Proofrolling should be performed with a twenty-ton rubber-tired tandem axle vehicle or similarly loaded vehicle or construction equipment, and should be observed by a qualified geotechnical engineer. For mass graded areas, building pad areas, and paved parking areas, the designated vehicle should make at least four passes over each section of the exposed soils with the last two passes perpendicular to the first two. For paved roadways, the designated vehicle should make at least two passes over each section of the exposed subgrade soils, including the proposed curblines. A final proofroll is recommended to be performed within 24 hours of pavement construction. If inclement weather occurs or if the proofroll fails to yield favorable results within this 24-hour window, then reworking of the subgrade soils may be required to achieve a suitable subgrade.

Any localized areas of yielding, soft/loose and/or saturated soils identified during proofrolling will need to be densified in-place, undercut and the removed soil replaced with properly compacted structural fill, or be modified by the use of mechanical or chemical means. Any modification activities should be monitored and all fill should be placed in general accordance with the recommendations presented in the *Structural Fill* section of this report.

Stormwater and Groundwater Management

As previously stated, although free groundwater was encountered in Boring B-2B at a depth of approximately 4-1/2 feet below the existing ground surface at the time of drilling. If shallow groundwater or perched groundwater is encountered during grading and excavation operations, the contractor should be prepared to dewater any excavations that may be impacted by ditching or pumping. From our experience with similar projects and site conditions, the soil types encountered at this site will likely require several days to drain.

Any exposed subgrade soils and recently placed fill soils should be well drained to minimize the accumulation of stormwater runoff. If the exposed subgrade soils are not as anticipated, or become excessively wet, the geotechnical engineer should be consulted. Additionally, the working site grades should be graded to allow for positive run-off of the under-construction areas to help prevent destabilization of existing subgrades or recently placed fill soils.

Finally, the finished site grades should be designed by a registered civil engineer, and should promote positive drainage away from finished structures, to prevent water-softening of structural soils, or possible undermining of structures.

Structural Fill

On-site Sands: In general, the on-site, sandy soils (SP, SM, SC, and SC-SM) appear *suitable* for reuse as structural fill.

On-site Clays: As stated previously, the on-site, low-plasticity clayey soils (CL) are *marginally suitable* for reuse as structural fill.

As mentioned previously, the fine-grained nature of the on-site soils (fine sands and clays) indicates that they are typically sensitive to variations in moisture content, with a relatively narrow range of workable moisture contents. Therefore, close control of moisture content will be necessary during grading and fill placement operations. In addition, the soils at this site may become difficult to work during periods of wet weather. Grading operations under wet conditions may result in the deterioration of otherwise suitable soil conditions, or of previously placed and properly compacted fill.

Furthermore, these inherent soil properties (silts and clays) make these soils less desirable for use as structural fill; however, if placed properly, suitable support of structures is achievable, provided subgrade drainage is established and maintained throughout the service life of the structure. Alternate, more suitable, borrow soils should be sought in the event that the on-site soils are deemed, during grading activities by the geotechnical engineer, to be *unsuitable* for use as structural fill for the support of structures.

General Fill Recommendations: Prior to the placement of fill soils, representative soil samples should be obtained and tested to determine their classification and compaction characteristics. Optimum fill material should be free of debris and rocks larger than 2 inch diameter, and any fibrous organic material or organic soils. We recommend that fibrous organic material found in the fill materials be no more than 5 percent by weight. Additionally, we recommend optimum fill materials to have a Plasticity Index (PI) less than 15 and a maximum dry density, determined by laboratory Proctor testing, of at least 85 pounds per cubic foot (pcf). Compaction characteristics of the fill soils should be determined using the laboratory Standard Proctor density test, ASTM D698, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb. Rammer and 12-in. Drop*.

Fill material should be placed in no more than 8 to 10-inch thick lifts, loose measurement, and within +/- 2 percent of the optimum moisture content determined by ASTM D698. Fill material placed beneath the area of the structure and pavements, and 10 to 5 feet, respectively, beyond their perimeters should be compacted to a minimum **98** percent of the laboratory Standard Proctor maximum dry density (ASTM D698).

General Quality Control Recommendations: For mass grading beneath structures and pavements, compaction testing should be performed at a minimum frequency of one test per lift per 2000 square feet of fill placed. For utility trench backfill, compaction testing should be performed at a minimum frequency of one test per lift per 200 feet of fill placed within utility trenches, where these trenches are extended beneath pavement or structure. Upon completion of the mass grading and the installation of buried utilities and/or conduits, it will be necessary to re-test the compaction of the structural fill placed within all backfilled utility trenches, especially where they have been buried within a previously tested and approved grade slab or pavement area. Failure to re-inspect and re-test these trenches beneath grade slab and pavement areas may result in varying soil support of the loaded subgrade soils.

General Compactive Equipment Recommendations: The soil types and lift thicknesses utilized during structural fill placement, as well as the type of fill placement that is taking place (i.e. mass fill, trench fill), will dictate the most appropriate type of compaction equipment to use for each activity. The following industry standard compaction equipment is suggested:

During mass grading activities smooth-drum rollers should be utilized for densification of the loosely placed structural fill lifts. These rollers can have both vibratory and non-vibratory settings, and can be utilized for densification of both granular (SP and SW) and semi-cohesive (SC and SM) soil types. Due to the particle size of the soils however, the compactive effort of these rollers can influence further into a granular soil lift than a semi-cohesive soil lift, therefore, the lift thickness of the structural fill placed, as well as the size, weight, and vibratory setting of the roller, play a large role in the effectiveness of this roller.

During utility trench backfill activities compaction wheels and jumping jack tamps, and vibratory plates can be utilized for densification of the loosely placed structural fill lifts. It is important to note that industry standard for these soil types allow for 6 to 8 inch loose lifts to be placed when a compaction wheel or jumping jack tamp are being utilized, due to the size, weight, and vibratory setting, when available, of the equipment.

Special Fill Placement Recommendations – Slope Fill: Where fill will be placed along existing slope embankments, we recommend that the areas to receive fill be benched into terraces and slightly over-built, in order to minimize the presence of a loose zone of poorly compacted soils near the slope face. Large terraces are recommended for the compaction activities along the slope in order to allow large earth moving and compacting equipment access to the work area, ultimately aiding in the ability and speeding the time required to achieve compaction. Further recommendations are provided in the *Slope Construction Recommendations* section of this report.

Slope Construction Recommendations

Permanent compacted fill and exposed cut slopes, if required, should be inclined no steeper than 2H:1V, for slopes greater than a height of 4 feet. Furthermore, we recommend that fill slopes constructed along existing slopes or embankments steeper than 4H:1V have a keyway constructed along the slope base to help counteract sliding failure. The keyway width should be at least ½ of the planned slope height, and the keyway should be embedded a minimum of 2 feet into a competent soil layer.

Furthermore, we recommend that any compacted fill slopes be benched and slightly over-built, (in order to minimize the presence of a loose zone of poorly compacted soils near the slope face), and then cut back to firm, well compacted soils prior to the placement of structure or vegetative cover. Cut slopes may require some reworking of the near surface soils in order to achieve a more sound slope surface. Upon construction of a competent slope face, it is critical that the slope face be protected from erosion, through the installation of a geotextile fabric or the application of a vegetative cover.

We caution against the installation of foundation, drop inlets or storm sewer lines within an improper embedment zone of the slope face, where possible over stressing and leakage may create maintenance problems or possible isolated slope failure. In general these structures need to be installed a minimum distance of 1½ times the height of the embankment, as measured from the crest and/or toe of the slope. Furthermore, proper embedment of foundations or buried utilities beneath slope faces should be established prior to construction, with a minimum embedment for foundation recommended to be 5 feet below the down gradient portion of the slope, while a minimum embedment for buried utilities is recommended to be 3 feet below the down gradient portion of the slope.

Soil Retainage System

We have assumed that retaining wall structures ***may*** be utilized to support lateral soils forces in portions of the site, and we have further assumed that other soil retainage systems ***may*** be required during excavation and foundation construction activities conducted on-site.

Therefore, we have estimated the earth pressure coefficients for each support condition in a drained situation, for the soils encountered at the site. The estimated values are dependent on the soil type, and the unit weight of the soil, as determined from laboratory testing, for the type of material actually retained, which should be verified upon exposing them, or upon the selection of the fill material.

Table 2: Earth Pressure Coefficients

Support Condition	***Pressure Coefficient***	
	Clayey Sands	***Sands***
Active (Wall deflects laterally away from retained soil).	Ka = 0.36	Ka = 0.33
At-rest (Wall is restrained from movement).	Ko = 0.53	Ko = 0.47
Passive (Wall deflects laterally toward retained soil).	Kp = 2.77	Kp = 3.26

1.) *A design unit weight of 115 pounds per cubic foot, cohesion of 50 pounds per square foot and a phi angle of 28 degrees are assumed for the existing clayey sandy soils, while a design unit weight of 110 pounds per cubic foot, cohesion of 0 pounds per square foot and a phi angle of 32 degrees are assumed for the existing sands. These values should be used for determining lateral earth pressures for the design of the permanent retainage structure's at this site.*

2.) *It is important to note that the presented coefficients are estimated for the in-situ conditions encountered at the time of drilling. Therefore, the on-site soils, if utilized, or any off-site fill, if utilized behind the walls of the structure, should both have samples presented to the laboratory for analysis, and the laboratory determined coefficients of the fill material should be utilized for design instead of the coefficients presented above.*

The design of the retainage structures should include an allowance for positive gravity drainage of the retained soils either using permanent toe drains or weep holes.

Additionally, compaction of fills behind retainage structures should be conducted with light, hand-held compactors. Heavy equipment, such as rollers or grading equipment should not be allowed to operate within 10 feet of the retaining wall during construction in order to avoid developing additional excessive lateral earth pressures.

We caution against the installation of structures, drop inlets or storm sewer lines within an improper offset zone of the retaining wall, where possible over stressing and leakage may create maintenance problems or possible wall failure. Proper offsets for construction behind and at the base of retaining walls should be established prior to construction. Minimum offset for the edge of structure or infrastructure should be at least 1 to 1½ times the height of the wall, with distances measured perpendicular and away from the top of the wall, starting at the crest and toe of the wall.

Shallow Foundation Recommendations

Provided that the site is properly prepared and that the ground modifications are complete and have been verified as stipulated in the *Site Preparation* of this report and that fill has been placed in accordance with the *Structural Fill* section of this report, the footings for the proposed structures may be proportioned for an allowable bearing pressure of **2,500** pounds per square foot (psf).

Furthermore, it appears that the structures at the site may be supported with a conventional system of shallow spread foundations. We recommend that the continuous foundations have a minimum width of **1-½** feet and the spread foundations have a minimum width of **3** feet, to avoid localized punching failure. The foundations should bear at a minimum depth of **18** inches below the final ground surface in order to ensure that the bearing surfaces are below the maximum frost depth.

General Foundation Recommendations: The actual depth of embedment of the foundations should be dictated by the ability to achieve the foundation and soil forces required to adequately resist up-lift and overturning for the subject structure. Soil forces reacting with embedded shallow foundations may be used to aid in the resistance of both uplift and overturn for this structure. The weight of the soil "wedge" above the footing may be used to aid in the resistance of uplift forces. We recommend that a unit weight of **115** pcf be used to compute the resisting soil weight. This unit weight has been estimated assuming select fill will be used as backfill and that the fill will be compacted to at least **98** percent of the Standard Proctor maximum dry density. The volume of the soil wedge may be calculated by assuming that the resisting soil section extends 45 degrees vertically from the outside top edge of the foundation to the ground surface. Additionally, passive earth pressure of the soils adjacent to the foundations, as well as soil friction at the foundation base and sides, may be used to develop shear to aid in the resistance of uplift and overturn. An ultimate friction coefficient between the foundation concrete and adjacent soil can be assumed to be on the order of 0.40.

The footings should be properly benched and the bearing soils free of loose debris or ponded water. If excavated bearing soils are exposed to the environment for extended periods of time or varying weather conditions, they may weaken. Foundation concrete should not be placed on bearing soils that have been weakened from the effects of the environment.

Therefore, we recommend that the footings be concreted shortly after excavation. If the footing excavation should remain open overnight, or if rain becomes imminent, we recommend that the bearing soils be covered with a 2 to 4 inch mud-mat of 2,000 psi concrete.

General Quality Control Recommendations: We strongly recommend that the footing excavations are observed and Dynamic Cone Penetrometer (DCP) values obtained by a qualified geotechnical engineer or engineering technician in order to confirm that the bearing soils are acceptable for the recommended bearing pressure. DCP testing should be conducted at a minimum frequency of 50 linear feet for continuous footings and at every pier footing, to minimum depths of twice the excavated foundation width. Unsuitable bearing soils, if encountered, will likely be required to be over-excavated and the resulting excavation to be backfilled with properly compacted fill, washed No. 57 stone or concrete. We recommend that the washed stone, if used, be wrapped with a non-woven filter fabric, where it will be submerged or partially submerged in groundwater.

Anticipated Settlement: Provided the foundation and construction recommendations presented in this report are followed, the total estimated settlement for the structure will likely be on the order of less than 1 inch. The differential settlement could be expected to be ½ of the total settlement for the semi-cohesive and cohesive type soils encountered at the site. It is important to note that these estimates do not account for any seismic induced settlements.

Alternate Deep Foundation Recommendations – Micropiles

The following sections provide general design parameters for micropile construction considerations and monitoring and other design details for the use of micropiles at this site. Ultimately, the structural engineer should design the piles with consideration to the provided assumptions, applied loads and stresses for each pile, and overall economy. The micropile materials and construction requirements shall adhere to the specifications provided in the Federal Highway Administration Manual (FHWA-SA-97-070) *Micropile Design and Construction Guidelines, Edition June 2000*, or most recent edition and all applicable sub-sections for micropiles unless otherwise stipulated in the following sections of this report.

Advantages to using micropiles include their comparatively low initial costs, ability to be constructed in confined or limited access spaces, and ability to develop relatively high capacities. Disadvantages include; they are relatively difficult to install, a qualified specialty contractor is usually required, soil displacement (resulting in high initial skin friction that ultimately normalizes in wet conditions), and their vulnerability to inconsistencies during installation. The following sections include recommendations and construction details for micropiles.

Pile Groups: Both large and small pile groups are anticipated to support the proposed structure at this site. The following guidelines and recommendations typically apply to small pile groups:

- Installed piles in groups should have a minimum center-to-center spacing of 3 times the pile width. This is necessary to allow development of individual pile carrying capacity without a group reduction.
- Based on experience, it is known that the disturbed soil zone created by installation of a pile will require a waiting period to achieve maximum load carrying capability. Installation and testing guidelines presented below provide recommendations for allowing soil strength gain and increased pile carrying capacity with time.

Micropile Construction Details: The micropiles should be at least 8-inches in diameter and be designed for a minimum embedment depth of 15 feet. Additionally, the micropiles may require partial reinforcement with API schedule 80, threaded joint pipe casing. If so, the casing should have a minimum wall thickness of 0.375 inches to resist lateral shear forces. Furthermore, the steel reinforcement for the micropiles should have an minimum yield strength of 75 ksi.

During micropile construction, the borehole should remain free of cuttings and spoil material. If predrilling fluid is utilized, we recommend that it have a water/cement (w/c) ratio of no greater than 0.70. We further recommend that the structural pile grout have a minimum w/c ratio of 0.45 and a 28-day compressive strength of 5000 psi.

Finally, we recommend that the final design of the micropiles be submitted by the specialty pile contractor for approved by the structural engineer of record and our personnel prior to construction and testing.

Observation and Quality Control: The contractor should submit a list of proposed equipment before construction starts so that compatibility of each element may be checked. The geotechnical engineer should be retained to provide this service. Additionally, the geotechnical engineer should be retained to observe the installation of the micropiles, and their field report should include:

- A daily recording of the operation of installation equipment and any non-compliance with the project specifications,
- A record of the penetration achieved for each micropile including casing installation,
- A record of the dimensions, a location and any defects for each pile,
- A record of the grout volume installed in each pile,
- A record and report of movement of previously installed adjacent piles,
- A record of installed lengths for pay purposes.

Pile Load Testing: In order to verify the design and loading conditions of the piles for this project, a load test can be performed. Load testing would allow comparison of the results of the test to the validity of the estimates provided in this report, and a quick evaluation of the contractor's equipment and installation procedures prior to construction of the load bearing piles.

The test pile should be identical in design to those actually used for load bearing piles. The test pile location should be selected based on proximity of the actual support sites, with consideration to accessibility to installation and testing equipment. Load tests are commonly performed by jacking against a framework of girders spanning the piles from a series of reaction piles or piers installed nearby.

Load testing should be performed in general accordance with the procedures outlined in ASTM D1143, *Standard Method for Testing Piles Under Axial Compressive Loads and* ASTM D3689 *Standard Test Methods for Deep Foundations Under Static Axial Tensile Load* using the standard loading procedures outlined in the "Quick Load Test section of each ASTM. For the standard application, loads are applied in increments of approximately 10 to 15 percent of the ultimate shaft load and held until dial gauge readings indicate movement less than 0.25 mm/sec, or for a maximum of two hours/increment. A standard of at least three gauges are recommended at each vertical horizon instrumented, to allow some redundancy in the system in the event of failure or damage to one or more of the gauges during installation of the shaft.

The installation of the test pile and actual load testing should be conducted under the observation of the geotechnical engineer

Grade Slabs

We understand that the grade-slab for the structure will be soil supported. We therefore recommend that the slab be jointed, reinforced and/or doweled in appropriate locations in order to allow differential and rotational movement between parts of the slab without uncontrolled cracking or sharp vertical displacements.

We further recommend that a re-compacted modulus of subgrade reaction of **140** pounds per cubic inch (pci) for the on-site sandy soils, be used for design of slab reinforcement at this site. In addition, an underslab vapor barrier should be included where finished areas will receive floor coverings. Slab design and construction using vapor barriers should be performed using methods detailed in the ACI *Manual of Concrete Practice.*

Construction activities and exposure to the environment can cause deterioration of the prepared subgrades. Therefore, we recommend that the subgrades be observed and compaction tests performed by a qualified geotechnical engineer or engineering technician in order to confirm suitability of the soil subgrades.

BASIS FOR

RECOMMENDATIONS

The recommendations presented in this report are based on our understanding of the project information, our interpretation of the data obtained during our investigation and provided to us, as well as our experience with similar soil and project conditions. The Standard Penetration Tests (SPT) and Dynamic Cone Penetration (DCP) values obtained at the boring locations have been used to estimate existing soil conditions at this specific site. Regardless of the thoroughness of this investigation, it is possible that the soil conditions intermediate of the borings and sounding vary from the soil conditions encountered at the boring and sounding locations. Therefore, it will be necessary for a geotechnical engineer or qualified engineering technician to be present during grading operations in order to evaluate and document that the anticipated design conditions actually exist.

It is important to note that our investigation was performed to provide general observations and soil conditions. This report accounted for only assumed static or dynamic loading conditions that are typically modeled in a standard geotechnical exploration. We therefore strongly recommend that this report be updated when the actual static and dynamic design loads area established.

CLOSING

Once again we appreciate the opportunity to provide our services for your geotechnical consulting needs. If there are any questions concerning our recommendations or if additional information becomes available please contact us.

Sincerely,
GS2 ENGINEERING, INC.

John P. Lewis, E.I.T.
Geotechnical Project Manager

Shawn J. Etier, E.I.T.
Senior Geotechnical Consultant, VP

George A. Sembos, P.E.
Senior Geotechnical Engineer, President

APPENDIX A

Source: USGS Southwest Columbia and Columbia North Topographic Quadrangles, dated 1987

Prepared By\Date: JPL-02/14
Checked By\Date: GAS-02/14

GS2 ENGINEERING

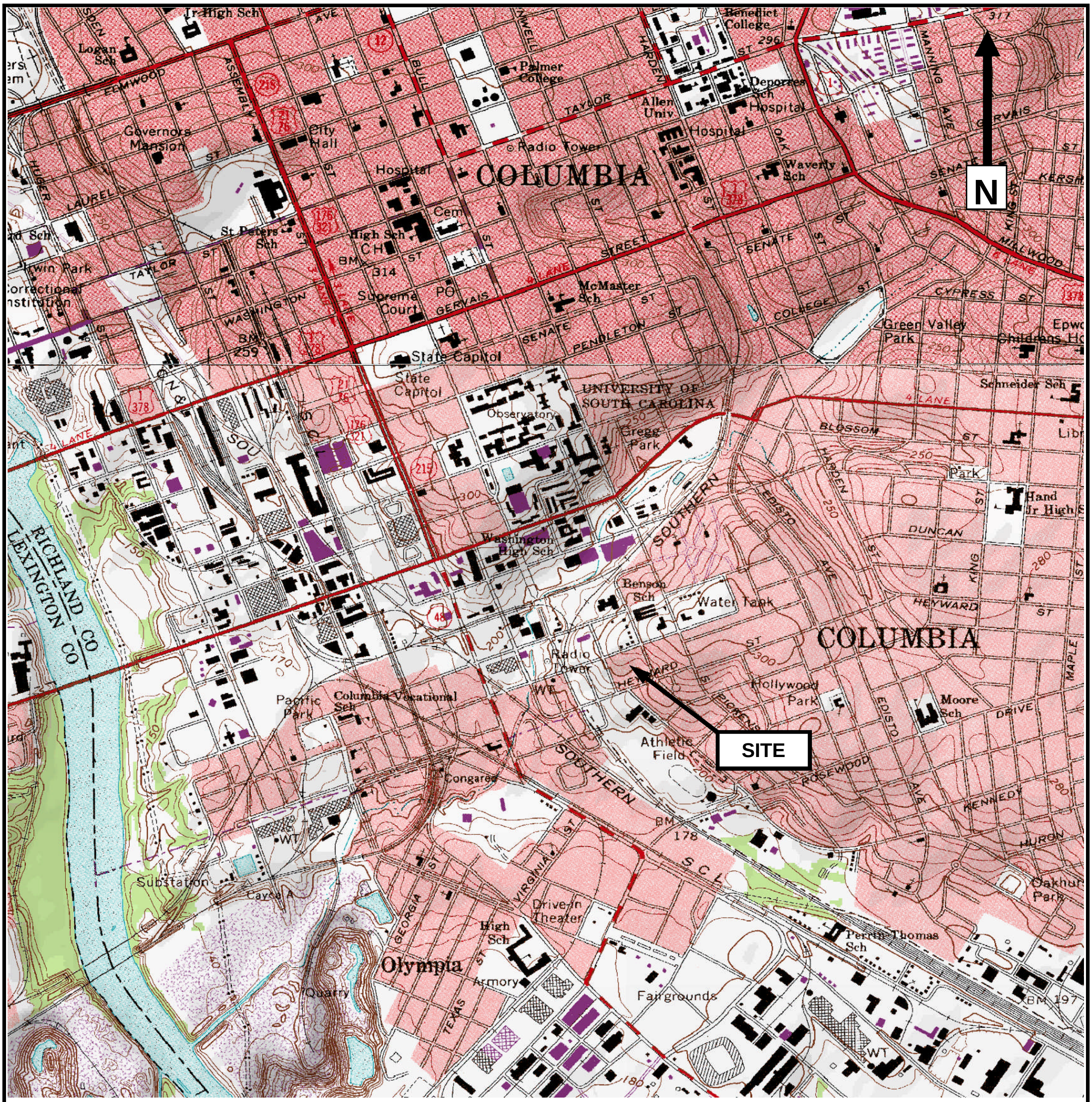
Site Location Map
Existing USC Field House Conversion
GS2 Project Number 14-1144-G
1400 Whaley Street
Columbia, South Carolina

University of South Carolina
Campus Planning and Construction
743 Greene Street
Columbia, South Carolina 29208

Scale

1 inch = 2300 feet

Figure 1



Source: USGS Southwest Columbia and Columbia North Topographic Quadrangles, dated 1987

Prepared By\Date: JPL-02/14
Checked By\Date: GAS-02/14

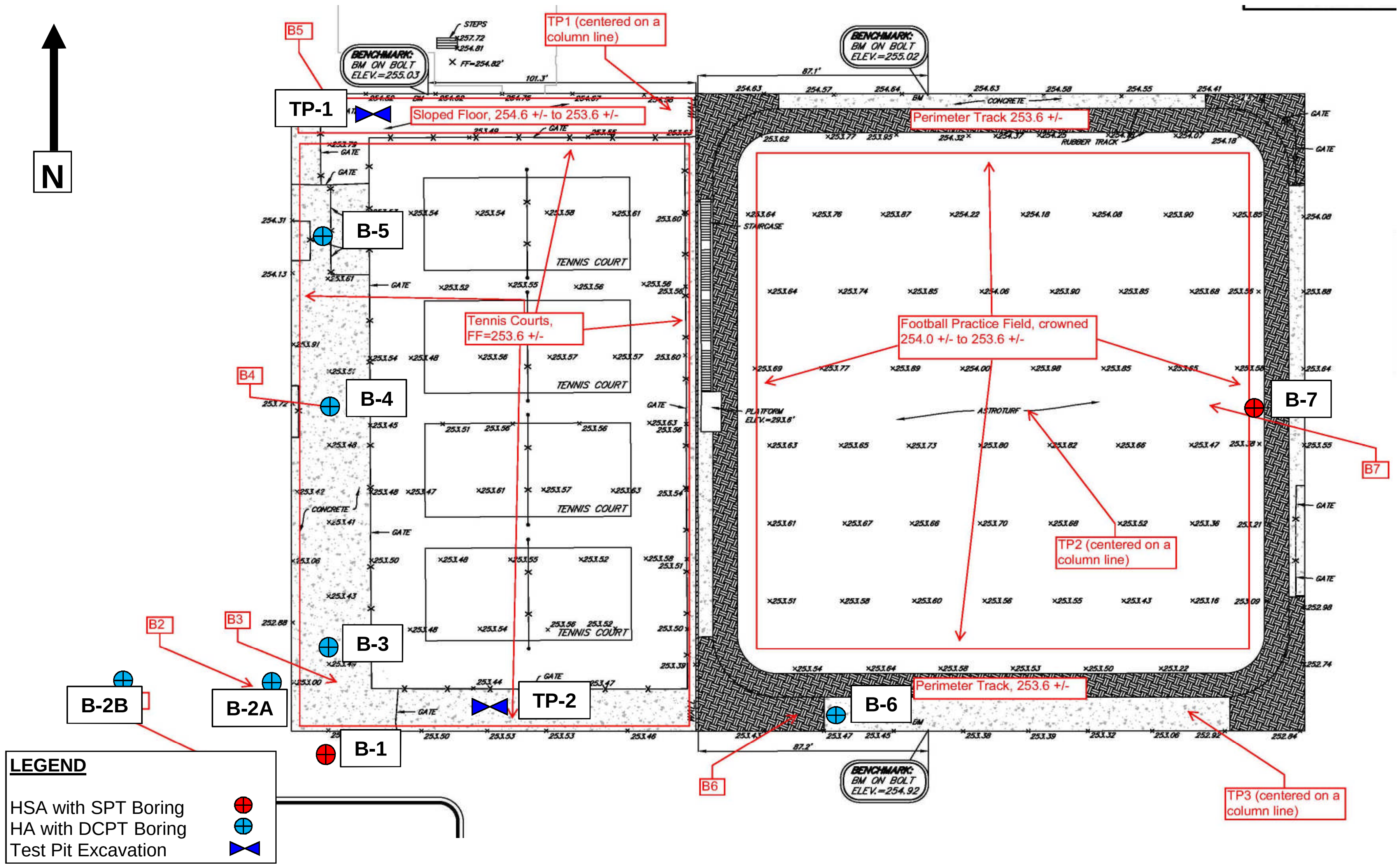
GS2 ENGINEERING

USGS Topographic Map
Existing USC Field House Conversion
GS2 Project Number 14-1144-G
1400 Whaley Street
Columbia, South Carolina

University of South Carolina
Campus Planning and Construction
743 Greene Street
Columbia, South Carolina 29208

Scale
1 inch = 2000 feet

Figure 2



Source: Boring Location Plan, dated 12/24/13, provided by CHA Sports

GS2 ENGINEERING

Boring and Test Pit Location Plan
Existing USC Field House Conversion
GS2 Project Number 14-1144-G
1400 Whaley Street
Columbia, South Carolina

University of South Carolina
Campus Planning and Construction
743 Greene Street
Columbia, South Carolina 29208

Prepared By\Date: JPL-07/14
Checked By\Date: GAS-07/14

Scale
NOT TO SCALE

Figure 3

APPENDIX B

Soil Test Boring Log Key

Soil Test Boring Logs

SOIL TEST BORING LOG KEY

COLOR SCHEME (Primary Soil Type):

SURFACE MATERIALS
Generally consist of Asphalt, Graded Aggregate Base Course, Concrete or Topsoils. Topsoils typicall combine a mixture of soils and organic materials. Topsoils are typically recognized through texture and odor.

SANDS
Sands are considered to be a granular soil type with no cohesive properties. Grain sizes are categorized as fine (falls between 0.075 and 0.420 mm. in diameter), medium (falls between 0.420 and 2 mm. in diameter) or coarse (falls between 2 and 4.75 mm. in diameter).

SILTS
Silt grain sizes typically fall between 0.002 and 0.075 mm. in diameter. The Atterberg's limits for silts typically plot below the A-Line on a Plasticity Chart. Silts are typically distinguished as having a Low Plasticity (P.I. is between 0 and 22) or as having a High Plasticity (P.I. is between 22 and 59). Silts exhibit some cohesive properties.

CLAYS
Clay grain sizes typically are smaller 0.002 mm. in diameter. The Atterberg's limits for clays typically plot on or above the A-Line on a Plasticity Chart. Clays are typically distinguished as having a Low Plasticity (P.I. is between 0 and 22) or as having a High Plasticity (P.I. is between 22 and 59). Clays exhibit strong cohesive properties.

COOPER MARL
The Cooper Marl Formation is typically olive green in color and classifies as a silty sand or sandy silt. It is composed of overconsolidated marine deposits and is highly reacitve to hydrochloric acid. The Cooper Marl sometimes contians cemented layers of phosphate. This formation is native to the Low Country Area of South Caroilina.

NO RECOVERY
Denotes that there was no recovery in the split-spoon sampler barrel upon its retrieval from the borehole. No recovery may be due to very hard soil layers that are unable to be penetrated by the barrel or super-saturated soils that are unable to be retained in the barrel.

PATTERN SCHEME: (Secondary Soil Type)

SANDY
Denotes a soil that has a percentage of sand. The portion of the soil that is sandy in nature is considered coarse-grained. When used in conjunction with the yellow color scheme, this pattern means the soil has more than 50% retained on the No. 200 sieve (i.e 0.075 mm in diameter).

SILTY
Denotes a soil that has a percentage of silt. The portion of the soil that is silty in nature is considered fine-grained. When used in conjunction with the gray color scheme, this pattern means the soil has more than 50% passing the No. 200 sieve (i.e 0.075 mm in diameter).

CLAYEY
Denotes a soil that has a percentage of clay. The portion of the soil that is clayey in nature is considered fine-grained. When used in conjunction with the red color scheme, this pattern means the soil has more than 50% passing the No. 200 sieve (i.e 0.075 mm in diameter).

PARTIALLY WEATHERED ROCK
Denotes a soil that is considered Partially Weathered Rock (PWR). PWR is defined as residuum that exhibits SPT N-values in excess of 100 bpf

DEBRIS LADEN
Denotes a soil that is laden with debris. Debris may consist of anything man-made, including, but not limited to, house hold trash, construction debris (concrete, brick, metal, etc.) or may consist of natural debris, such as organics. Depending on the severity and type of the debris, these materials may require excavation and replacement.

OLD FILL
Denotes a soil that is assumed or known to be previously placed, possibly untested, old fill. As there is no known record of its placement, these soils are undocumented, and may require excavation and replacement.

Note:
The above detailed color schemes are indicative of the predominant pimary soil type observed in the indicated soil strata at the Boring locations for the subject site. Secondary soil types are detailed by the pattern scheme. Both the color and pattern scheme are detailed in the Remarks column of the SOIL TEST BORING LOG. All soil descriptions are based on visual and textural properties observed in the recovered soils. No laboratory tests were performed on the soils described in this report, unless noted within the remarks column of the logs.

SOIL TEST BORING LOG

Project Name: **Existing USC Field House Conversion**

Project Number: **14-1144-G**

Date of Test: **January 17, 2014**

Boring Number: **B-1**

Depth (feet)	Soil Description	Sample Interval	Blow Counts*	Remarks
	SURFACE MATERIALS: 1 Inch of TOPSOIL.			
1	OLD FILL: Firm Reddish-Brown Clayey Fine to Medium SAND with #57 Stone. (SC)	0 to 1-1/2'	11	
2				
3				
4	Loose to Firm, Red and Grayish-Brown, Clayey Fine to Medium SAND with Rocks. (SC)			
5		3-1/2' to 5'	6	
6				
7				
8		6' to 7-1/2'	11	
9	Firm Brown Clayey Fine to Medium SAND. (SC)			
10		8-1/2' to 10'	12	
11				
12				
13	UPPER COASTAL PLAIN: Very Firm Gray, Brown and Red Fine Sandy CLAY. (CL)			
14				BOREHOLE CAVE-IN
15		13-1/2' to 15'	22	
16				
17				
18				
19				
20	Very Dense Orange Fine to Medium SAND. (SP)	18-1/2' to 20'	50/5"	
21	Boring Terminated at 20 Feet.			
22				
23				
24				
25		23-1/2' to 25'		

Depth of Boring (ft): 20 Feet

Depth of Groundwater T.O.B. (feet): Not Encountered

Depth of Groundwater 24 hrs. (feet): Not Available

Location of Boring: see Boring and Test Pit Location Plan

Method of Drilling: Hollow Stem Auger

Performed By: GS2 Drilling

* The Blow Counts given above are recorded for a 140 pound hammer (falling 30 inches/blow) to drive a 2 inch O.D., 1.375 inch I.D. split-barrel sampler 12 inches, after an initial 6 inch seating increment.

Record Of Hand Auger Boring

Project Name: **Existing USC Field House Conversion** **Boring No.:** **B-2A**

Project Number: **14-1144-G** **Date:** **1/24/2014**

Depth From	Depth To	Soil Description	Depth of Test	DCP* Blow Counts 1st	2nd	3rd	Average** DCP (bpi)
0		OLD FILL: Loose to Very Firm Brown and Red Clayey Fine to Medium SAND. (SC)	0	3	6	6	6
			1'	6	8	9	9
			2'	12	12	12	12
			3'	9	9	9	9
			4'	6	9	7	8
			5'	5	6	6	6
			6'	10	13	20	17
	7-1/2'		7'	17	20	24	22
7-1/2'	8'	NATIVE: Very Firm Tan Silty Fine to Medium SAND with Pebbles. (SM)	8'	25+	-	-	25+
8'		Very Firm Tan Silty Fine to Medium SAND. (SM)	9'	14	18	18	18
	10'		10'	15	25+	-	25+
		Boring Terminated at 10 Feet.					

Method of drilling: Hand Auger

Performed By: G. Simonson

Depth of Groundwater T.O.B.: Not Encountered

Boring Location: see Boring and Test Pit Location Plan

Depth of Groundwater 24 hrs.: Not Available

Notes: 1. Please see attached report.

* DCP (or Dynamic Cone Penetrometer) tests were taken in general accordance with ASTM #T-399.
** The average DCP blow per increment (bpi) is arrived at by averaging the 2nd and 3rd blows.

Signature: John Lewis

John P. Lewis, E.I.T.
Staff Geotechnical Professional

Record Of Hand Auger Boring

Project Name: **Existing USC Field House Conversion** **Boring No.:** **B-2B**

Project Number: **14-1144-G** **Date:** **1/24/2014**

Depth			Depth of	DCP* Blow Counts			Average**
From	To	Soil Description	Test	1st	2nd	3rd	DCP (bpi)
0		OLD FILL: Loose Brown Clayey Fine to	0	3	6	7	7
	6"	Medium SAND. (SC)					
6"		Firm Tan Silty Fine to Medium	1'	8	9	8	9
	1-1/2'	SAND. (SM)					
1-1/2'		Firm Orange and Tan Clayey to Silty	2'	7	8	9	9
		Fine to Medium SAND. (SC-SM)					
	3'		3'	9	9	7	8
3'		NATIVE: Very Firm Tan, Orange and					
		Red Clayey to Silty Fine to Medium	4'	10	23	23	23
		SAND. (SC-SM)					
			5'	17	25+	-	25+
			6'	25+	-	-	25+
	7'		7'	25+	-	-	25+
7'		Very Firm Orange Silty Fine to					
		Medium SAND. (SM)	8'	18	25+	-	25+
	8'	Very Firm Tan and Orange Clayey to					
8'		Silty Fine to Medium SAND. (SC-SM)	9'	25+	-	-	25+
	10'		10'	18	25+	-	25+
		Boring Terminated at 10 Feet.					

Method of drilling: Hand Auger

Performed By: G. Simonson

Depth of Groundwater T.O.B.: 4'6"

Boring Location: see Boring and Test Pit Location Plan

Depth of Groundwater 24 hrs.: Not Available

Notes: 1. Please see attached report.

* DCP (or Dynamic Cone Penetrometer) tests were taken in general accordance with ASTM #T-399.

** The average DCP blow per increment (bpi) is arrived at by averaging the 2nd and 3rd blows.

Signature: John Lewis

John P. Lewis, E.I.T.

Staff Geotechnical Professional

Record Of Hand Auger Boring

Project Name: **USC Field House Conversion** **Boring No.:** **B-3**

Project Number: **14-1144-G** **Date:** **7/10/2014**

Depth From	Depth To	Soil Description	Depth of Test	DCP* Blow Counts 1st	2nd	3rd	Average** DCP (bpi)
		5-1/2 Inches of CONCRETE.	0'	4	4	6	5
0		OLD FILL: Loose to Firm Brown and					
	1'	Tan Silty SAND. (SM)	1'	14	13	14	14
1'		NATIVE: Firm Gray and Brown Silty					
	2'	SAND. (SM)	2'	21	17	13	15
2'		Firm to Very Firm Brown and Orange	2-1/2'	15	15	13	14
	2-3/4'	Clayey SAND with Rocks. (SC)	2-3/4'	12	21	25+	25+
		Auger Refusal at 2-3/4 Feet.					
			4'				
			5'				
			6'				
			7'				
			8'				
			9'				
			10'				

Method of drilling: Hand Auger

Performed By: J. Butler/R. Still

Depth of Groundwater T.O.B.: Not Encountered

Boring Location: Please see the attached Boring Location Plan

Depth of Groundwater 24 hrs.: Not Available

Notes: 1. Please see attached report.

* DCP (or Dynamic Cone Penetrometer) tests were taken in general accordance with ASTM #T-399.

** The average DCP blow per increment (bpi) is arrived at by averaging the 2nd and 3rd blows.

Signature: John Lewis

John P. Lewis, E.I.T.
Geotechnical Project Manager

Record Of Hand Auger Boring

Project Name: **USC Field House Conversion** **Boring No.:** **B-4**

Project Number: **14-1144-G** **Date:** **7/10/2014**

Depth From	Depth To	Soil Description	Depth of Test	DCP* Blow Counts 1st	2nd	3rd	Average** DCP (bpi)
		4-1/2 Inches of CONCRETE.	0'	11	13	12	13
0		OLD FILL: Loose to Firm Tan Silty					
	1'	SAND with Rocks. (SM)	1'	5	5	6	6
1'		Firm Brown Silty SAND with Rocks.					
	2'	(SM)	2'	11	11	11	11
2'		NATIVE: Very Firm Dark Brown					
	3'	Clayey SAND with Rocks. (SC)	3'	16	20	25+	25+
3'		Very Firm Gray and Tan Silty SAND					
	4'	with Rocks. (SM)	4'	25+	-	-	25+
		Auger Refusal at 4 Feet.					
			5'				
			6'				
			7'				
			8'				
			9'				
			10'				

Method of drilling: Hand Auger

Performed By: J. Butler/R. Still

Depth of Groundwater T.O.B.: Not Encountered

Boring Location: Please see the attached Boring Location Plan

Depth of Groundwater 24 hrs.: Not Available

Notes: 1. Please see attached report.

* DCP (or Dynamic Cone Penetrometer) tests were taken in general accordance with ASTM #T-399.

** The average DCP blow per increment (bpi) is arrived at by averaging the 2nd and 3rd blows.

Signature: John Lewis

John P. Lewis, E.I.T.
Geotechnical Project Manager

Record Of Hand Auger Boring

Project Name: **USC Field House Conversion** **Boring No.:** **B-5**

Project Number: **14-1144-G** **Date:** **7/10/2014**

Depth From	Depth To	Soil Description	Depth of Test	DCP* Blow Counts 1st	2nd	3rd	Average** DCP (bpi)
		5 Inches of CONCRETE.	0'	8	10	11	11
0		Firm to Very Firm Brown and Tan					
		Silty SAND with Rocks. (SM)	1'	25+	-	-	25+
	1-1/2'		1-1/2'	25+	-	-	25+
		Auger Refusal at 1-1/2 Feet.	2'				
			3'				
			4'				
			5'				
			6'				
			7'				
			8'				
			9'				
			10'				

Method of drilling: Hand Auger

Performed By: J. Butler/R. Still

Depth of Groundwater T.O.B.: Not Encountered

Boring Location: Please see the attached Boring Location Plan

Depth of Groundwater 24 hrs.: Not Available

Notes: 1. Please see attached report.

* DCP (or Dynamic Cone Penetrometer) tests were taken in general accordance with ASTM #T-399.

** The average DCP blow per increment (bpi) is arrived at by averaging the 2nd and 3rd blows.

Signature: John Lewis

John P. Lewis, E.I.T.
Geotechnical Project Manager

Record Of Hand Auger Boring

Project Name: **USC Field House Conversion** **Boring No.:** **B-6**

Project Number: **14-1144-G** **Date:** **7/10/2014**

Depth		Soil Description	Depth of	DCP* Blow Counts			Average**
From	To		Test	1st	2nd	3rd	DCP (bpi)
		5 Inches of CONCRETE.	0'	11	12	12	12
0		OLD FILL: Loose to Firm Tan Silty					
	1'	SAND. (SM)	1'	8	7	6	7
1'		Very Loose Brown Silty SAND.					
		(SM)	2'	2	1	1	1
			3'	2	1	2	2
	4'		4'	W-O-H	-	-	W-O-H
4'		Loose Tan and Brown Silty SAND.					
		(SM)	5'	5	7	6	7
			6'	4	5	5	5
	7'		7'	4	4	5	5
7'		NATIVE: Firm Gray and Brown Silty					
		SAND. (SM)	8'	13	13	14	14
		Firm White and Tan Clayey SAND.					
		(SC)	9'	9	9	7	8
		Firm Yellow and Brown Clayey					
		SAND. (SC)	10'	13	15	13	14
		Boring Terminated at 10 Feet.					

Method of drilling: Hand Auger — Performed By: J. Butler/R. Still

Depth of Groundwater T.O.B.: Not Encountered — Boring Location: Please see the attached Boring Location Plan

Depth of Groundwater 24 hrs.: Not Available

Notes:
1. Please see attached report.
2. W-O-H = Weight of Hammer

* DCP (or Dynamic Cone Penetrometer) tests were taken in general accordance with ASTM #T-399.

** The average DCP blow per increment (bpi) is arrived at by averaging the 2nd and 3rd blows.

Signature: John Lewis

John P. Lewis, E.I.T.
Geotechnical Project Manager

SOIL TEST BORING LOG

Project Name: **Existing USC Field House Conversion**

Project Number: **14-1144-G**

Date of Test: **January 17, 2014**

Boring Number: **B-7**

Depth (feet)	Soil Description	Sample Interval	Blow Counts*	Remarks
	SLAB-ON-GRADE: 6 Inches of CONCRETE.			
1	OLD FILL: Dense Red Clayey Fine to Medium SAND with Concrete Debris. (SC)			
2		0 to 1-1/2'	45	
3				
4	Very Firm Brown Fine to Medium SAND with Concrete Debris. (SP)			
5		3-1/2' to 5'	30	
6				
7	POSSIBLE FILL: Loose Light Gray and Orangish-Brown Clayey Fine to Medium SAND. (SC)			
8		6' to 7-1/2'	9	
9	UPPER COASTAL PLAIN: Firm Red, Orange and Light Gray Clayey Fine to Medium SAND. (SC)			
10		8-1/2' to 10'	14	
11				
12				**BOREHOLE CAVE-IN**
13	Stiff Light Gray, Red and Brown Fine to Medium Sandy CLAY. (CL)			
14				
15		13-1/2' to 15'	13	
16	Very Stiff Brown, White and Red Fine Sandy CLAY. (CL)			
17				
18				
19				
20		18-1/2' to 20'	26	
21	Boring Terminated at 20 Feet.			
22				
23				
24				
25		23-1/2' to 25'		

Depth of Boring (ft): 20 Feet

Depth of Groundwater T.O.B. (feet): Not Encountered

Depth of Groundwater 24 hrs. (feet): Not Available

Location of Boring: see Boring and Test Pit Location Plan

Method of Drilling: Hollow Stem Auger

Performed By: GS2 Drilling

* The Blow Counts given above are recorded for a 140 pound hammer (falling 30 inches/blow) to drive a 2 inch O.D., 1.375 inch I.D. split-barrel sampler 12 inches, after an initial 6 inch seating increment.

GS2 ENGINEERING

RECORD OF TEST PIT EXCAVATION

Project Name: **Existing USC Field House Con.**

Project Number: **14-1144-G**

Pit No.: **TP-1** **Date:** **1/13/2014**

Equipment Utilized: Excavated by Hand

Depth From	Depth To	Material Description*
0	4"	CONCRETE SLAB-ON-GRADE.
4"	1'	Tan Silty Fine to Medium SAND. (SM)
1'	1-1/2'	Brown Clayey Fine to Medium SAND. (SC)
1-1/2'	5'	Red and Brown Clayey Fine to Medium SAND. (SC)
		Excavation Terminated at 5 Feet.

Test Pit Termination/Refusal at: 5 Feet

Performed By: J. Lewis and T. Turner

Depth of Groundwater T.O.E.: Not Encountered

Test Pit Location: see Boring and Test Pit Location Plan

Notes: Structural Cable Not Encountered

*Legend *(per ASTM D2487)*:

GW - Well-graded Gravel
GP - Poorly-graded Gravel
GC - Clayey Gravel
GM - Silty Gravel
SW - Well-graded Sand
SP - Poorly-graded Sand

SC - Clayey Sand
SM - Silty Sand
CL - Lean (low plasticity) Clay
ML - Lean (low plasticity) Silt
OL - Lean (low plasticity) Organics
CH - Fat (high plasticity) Clay

MH - Fat (high plasticity) Silt
OH - Fat (high plasticity) Organics
PT - Peat (heavy organic materials)
PWR - Partially Weathered Rock
DEBRIS - describe contents

Classification - Color/Less dominant soil component/Most dominant soil component

Signature: John Lewis

GS2 ENGINEERING

RECORD OF TEST PIT EXCAVATION

Project Name: **Existing USC Field House Con.**

Project Number: **14-1144-G**

Pit No.: **TP-2** **Date:** **1/17/2014**

Equipment Utilized: Excavated by Hand

Depth From	Depth To	Material Description*
0	5-1/2"	CONCRETE SLAB-ON-GRADE.
5-1/2"	5'	Tan Silty Fine to Medium SAND. (SM)
		Excavation Terminated at 5 Feet.

Test Pit Termination/Refusal at: 5 Feet

Performed By: J. Lewis and T. Turner

Depth of Groundwater T.O.E.: Not Encountered

Test Pit Location: see Boring and Test Pit Location Plan

Notes: Structural Cable Not Encoutered

*Legend *(per ASTM D2487)*:

GW - Well-graded Gravel
GP - Poorly-graded Gravel
GC - Clayey Gravel
GM - Silty Gravel
SW - Well-graded Sand
SP - Poorly-graded Sand

SC - Clayey Sand
SM - Silty Sand
CL - Lean (low plasticity) Clay
ML - Lean (low plasticity) Silt
OL - Lean (low plasticity) Organics
CH - Fat (high plasticity) Clay

MH - Fat (high plasticity) Silt
OH - Fat (high plasticity) Organics
PT - Peat (heavy organic materials)
PWR - Partially Weathered Rock
DEBRIS - describe contents

Classification - Color/Less dominant soil component/Most dominant soil component

Signature: John Lewis