Geotechnical Engineering Report
804-812 North 4th Street Development
Wilmington, North Carolina
S&ME Project No. 1306-17-016

PREPARED FOR:

Hipp Architecture & Development, PC
228 North Front Street, Suite 201
Wilmington, North Carolina 28401

PREPARED BY:

S&ME, Inc.
3006 Hall Waters Dr, Suite 100
Wilmington, NC 28405

September 15, 2017

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Hipp Architecture & Development, PC
228 North Front Street, Suite 201
Wilmington, North Carolina 28401

Attention: Mr. Clark Hipp, AIA

Reference: **Geotechnical Engineering Report**
804-812 North 4th Street Development
Wilmington, North Carolina
S&ME Project No. 1306-17-016
NC PE Firm License No. F-0176

Dear Mr. Hipp:

S&ME, Inc. (S&ME) is pleased to submit this geotechnical engineering report for the proposed commercial development at 804-812 North 4th Street in Wilmington, North Carolina. Our work was conducted in accordance with S&ME Proposal No. 13-1700351, dated July 31, 2017. The purpose of the exploration was to evaluate subsurface conditions as they relate to site preparation, earthwork, and foundation support. This report presents a summary of pertinent project information, results of field exploration and our geotechnical recommendations. A Boring Location Plan, Generalized Borehole Profiles, and Boring Logs are included in the Appendices.

S&ME appreciates the opportunity to provide our professional engineering services on this project. Should you have any questions concerning this report or if we may be of further assistance, please contact us at your convenience.

Sincerely,
S&ME, Inc.

NORTH CAROLINA PROFESSIONAL SEAL 035244 ENGINEER BRIAN E. LADD

Brian C. Ladd

Brian Ladd, P.E.
Principal Geotechnical Engineer
N.C. Registration No. 035244

Sep 15 2017

Keith C. Brown

Keith C. Brown, P.E.
Vice President

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1.0 Project Information

The subject property is located on the east side of North 4th Street, mid-block between Brunswick Street to the south and Bladen Street to the north. A site plan sketch was provided that indicates that the site is a combination of three addresses, 804, 808, and 812 North 4th Street, with a total frontage along North 4th Street of 132 ft. The site is currently vacant, grass covered and relatively level.

The site sketch shows that a 7,000 sq ft, three story building is planned for the southeast corner of the site. We understand that potentially, a 5-story building could be constructed. Parking will be along the north property line. At the time of this proposal, structural loads and site grading plans have not been finalized. We assume that minimal earthwork will be necessary to prepare the building pad, say 2 to 3 ft, and that for a 5 story building the maximum column loads would be on the order of 450 to 500 kips.

S&ME previously performed four borings on the subject site. Most important to note is that an approximate 4-ft void was encountered along the northern property line from about 43 to 47 ft below grade. S&ME also previously performed two other exploration programs on the neighboring properties to the north and south. Two borings performed on the property to the south (802 N. 4th Street) encountered 12 ft of fill. Borings on the property to the south encountered very loose sand with organics, and peat.

2.0 Area Geology

The site is located within the Coastal Plain Physiographic Province of North Carolina. The Coastal Plain Province is typically characterized by marine, alluvial, and eolian sediments that were deposited during periods of fluctuating sea levels and moving shorelines. The soils and basal formations in the North Carolina Coastal Plain Physiographic Province are typical of those laid down in a shallow sloping sea bottom; interbedded sands and clays with irregular deposits of shells and layers of limestone and cemented sands. Alluvial sands, silts, and clays are typically present near rivers and creeks. Deposits of peat, organic silt, and organic clay are also typically present in or near current or former tidal marsh areas in the outer portion of the Coastal Plain.

According to the Geologic Map of North Carolina (1985), the site is underlain by the Pee Dee Formation of Cretaceous age. This formation consists of greenish gray to olive black sand, clayey sand, and clay. A light gray limestone "cap rock" layer is typically present at depths of 20 to 70 ft in New Hanover County. The limestone is more or less consolidated, varies from loose to consolidated, and is usually quite fossiliferous and in many places is composed of shells. Very loose or soft soils can be present immediately above the cap rock and solution cavities and voids may be present within the limestone.

3.0 Field Exploration

3.1 Previous Exploration Programs

In March 2006, S&ME performed four test borings (B-1 through B-4) within the property. These borings were drilled to depths ranging from 50 to 74 ft below the existing grades. The boring logs were presented in our Geotechnical Exploration Report - Proposed Residential Condominium Building, dated April 5, 2006 (S&ME Project No. 1061-06-110). Boring B-1 encountered very soft silt at a depth of 30 ft, immediately above very dense sand.

Boring B-4 encountered a void from about 43 to 47 ft below grade as evidenced by a drop on the drill rods advancement of the borehole.

In October 2005, S&ME performed four test borings (also identified as B-1 through B-4) for immediately to the north of the site; borings B-3 and B-4 of this project are adjacent to the site. These borings were drilled to depths ranging from approximately 43.5 to 49.5 ft below the existing grades. The boring logs were presented in our *Geotechnical Report - Riverview Condominiums*, dated November 14, 2005 (S&ME Project No. 1061-05-528). Boring B-3 encountered very loose sand from about 27 to 32 ft below grade. Boring B-4 encountered a very loose/soft layer of peat and clayey sand about 32 to 43 ft below grade. The soft/loose materials encountered in B-3 and B-4 were immediately above very dense sand or limestone.

The boring logs from these previous exploration programs are incorporated into this report and are presented in Appendix I. The boring locations are shown on the Boring Location Plan, Figure 1.

3.2 2017 Field Exploration

The current exploration program for this project included a visual site reconnaissance by an S&ME staff professional and performance of three standard penetration test (SPT) soil test borings identified as S-1 through S-3. The boring locations are shown on the Boring Location Plan, Figure 1.

The soil test borings were performed on August 17-18, 2017 using a CME 45-C drill rig with a 2⅞ inch tri-cone roller bit and mud rotary drilling techniques.

All three borings were extended to a depth of 60 ft below the existing ground surface. Split-spoon samples of subsurface soils were taken at approximate 2½-ft intervals above a depth of 10 ft and at 5-ft intervals below 10 ft. Standard penetration tests were conducted using an automatic hammer in conjunction with split-spoon sampling in general accordance with ASTM D1586. Water levels were measured in the open boreholes at completion of drilling, then backfilled with auger cuttings.

Boring logs containing soil descriptions, SPT N-values, and observed water levels are included in Appendix II. Stratification lines shown on boring logs are intended to represent approximate depths of changes in soil types. Naturally, transitional changes in soil types are often gradual and cannot be defined at particular depths.

4.0 Subsurface Conditions

Subsurface conditions are discussed below in order of increasing depth. Generalized Borehole Profiles are presented in Figure 2.

4.1 Fill Soils

Approximately 3 ft of fill materials were encountered in each of the three boring locations. The fill materials generally consist of yellow and dark brown fine sand (SP). The sampled soils were observed to be moist. SPT N-values in the fill ranged from 3 to 6 blows per foot (bpf) indicating loose materials.

4.2 Coastal Plain Sands and Silt

Immediately below the fill materials, the upper 27 to 32 ft consisted orange-brown, reddish-brown, and light brown fine to medium sands (SP, SP-SM, SM). The sampled soils were described as wet with SPT N-values ranging from 3 to 30 bpf indicating a loose to dense deposit.

In borings S-1 and S-2, a 5-ft thick layer of reddish-brown fine sandy silt (ML) was present from roughly 27 to 32 ft below grade. The silts were also described as wet with SPT N-values of 1 and 2 bpf indicating a very soft deposit.

4.3 Pee Dee Formation Sands and Limestone Cap Rock

Underlying the brownish colored Coastal Plain deposits, the Pee Dee soils are distinguished by their transition to gray color. The Pee Dee soils consisted of gray, fine sand (SP) and clayey sand (SC). The upper Pee Dee sands are medium dense to dense and transition to loose with depth. The upper sands have SPT N-values of 45 to 48 bpf and decrease with depth.

An approximate 3 to 7 ft thick stratum of very loose sands were encountered in all borings at depths of 37 to 47 ft below grade. These loose sands have N-values of ""weight-of-rod" (indicated as WOR on the boring log) to 3 bpf. The WOR was recorded in boring S-2 when the drill rods dropped under their own weight from about 48.5 to 51.5 ft. In borings S-2 and S-3, the loose sands were present immediately above a 6-inch to 3-ft thick weathered limestone cap rock; the cap rock was not present in boring S-1.

The materials underlying the cap rock in borings S-2 and S-3, and loose sands in boring S-1, were very stiff, greenish-gray, sandy silt (ML) and medium-dense to dense silty and clay sands (SM and SC). SPT N-values in the sands were 19 to 36 bpf and 19 to 20 bpf in the silts.

4.4 Water Levels

Water was measured at depths of 11.6 and 9.7 ft below grade at completion of drilling borings S-1 and S-2. These levels are influenced by the drilling fluids used to advance the boreholes. After 24 hours, water was measured in boring S-2 at a depth of 17 ft below grade. A water level was not measured in boring S-3 which was the last boring completed. Water levels can be expected to fluctuate due to seasonal variations in rainfall, evaporation, and other factors.

5.0 Design Recommendations

The following sections provide recommendations for the foundation system, slab-on-grade support, lateral earth pressures, and seismic site classification.

5.1 Comment on Solution Cavities and Loose Sands

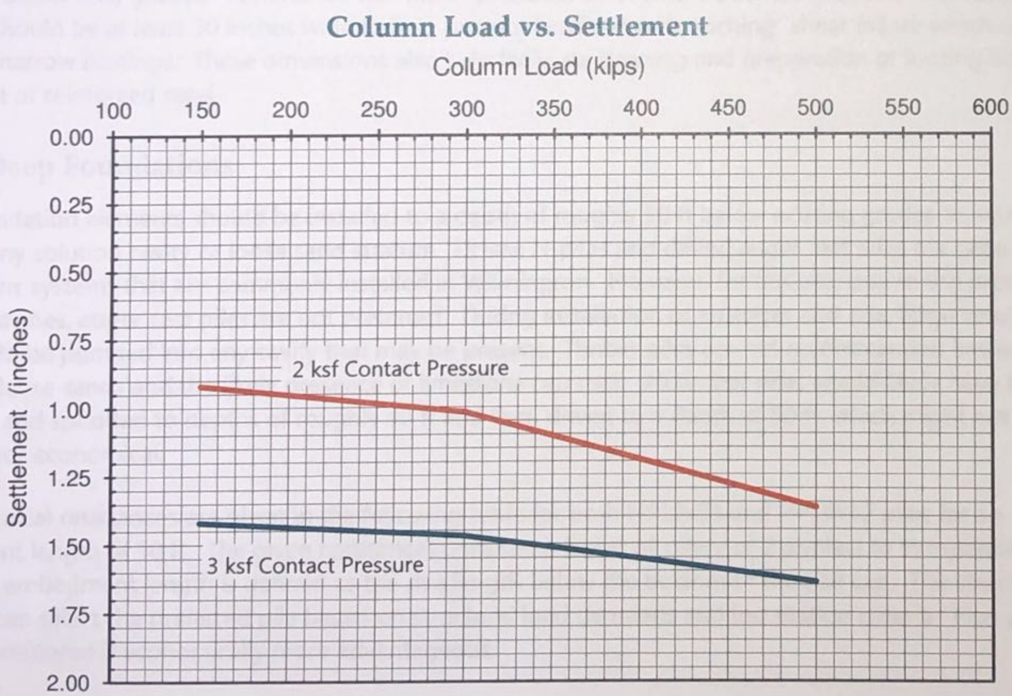
The voids and very loose sands that were revealed in the borings were encountered at depths of more than 40 ft below existing grades and are below the groundwater level. Solution cavities within a limestone geology are typically problematic when they occur near the surface above the groundwater level and are encountered during earthwork, and then require repair. Near surface cavities that are above the water can also expand or "drop out" when stormwater that percolates into the ground carries overlying soils into the void, resulting in ground loss that manifests at the ground surface. Ground loss can also result from a drawdown of the water table below the top of the void elevation or from stress increases from structural loads imposed by foundation elements (footings or piles) above the void.

Currently, the encountered cavities and loose sands are below the water levels observed at time of the explorations. The cavities were also below the zone of influence were stress increases would occur due to shallow foundation loads. As such, new structures can be supported on shallow foundations provided that settlement criteria is met. The following sections discuss foundation recommendations.

However, if work in the vicinity of the project site results in a drawdown of the water level below the top of a cavity, there is a risk that the void could expand and ground loss occur. Ground loss occurring within the building footprint will result in settlement of the structure.

5.2 Shallow Foundations

As noted herein, construction of a 3 to 5-story building is being considered for the site. Maximum column loads for a 3 to 5-story structure were assumed to be 300 kips and 500 kips, respectively. A maximum column load of 150 kips was also evaluated. The figure below shows the settlement for the assumed column loads for contact pressures of 2 ksf and 3 ksf.



As shown on the figure, considering a tolerable settlement of 1 inch, column loads less than 300 kips can be supported on shallow spread footings designed using a net allowable bearing pressure of 2 ksf. Column loads greater than 300 kips will need to be supported on deep foundations and are discussed in subsequent sections.

5.2.1 *Additional Shallow Foundation Recommendations*

The shallow foundation recommendations contained herein are contingent upon subgrade preparation, and fill placement and compaction guidelines presented in this report being followed. Some repair of the near surficial silty soils may be required to obtain the recommended bearing pressure. An S&ME geotechnical engineer or his representative should observe foundation bearing conditions prior to placement of reinforcing steel and concrete. This evaluation should include the performance of shallow hand auger borings with dynamic cone penetrometer (DCP) testing to depths of 3 to 4 ft to confirm the suitability of near surface bearing soils for foundation support. Where loose soils exist at bearing elevations, neat line overexcavation, and replacement of foundation bearing soils with lean concrete or NCDOT No. 57 stone will be required.

Sandy bearing soils will be susceptible to loosening and strength loss if left exposed to construction traffic or if water is allowed to penetrate these materials. Therefore, rainwater must be prevented from wetting or ponding on these exposed subgrade surfaces. Foundation excavation and placement of concrete should be conducted on the same day, if practical. If placement of the foundation concrete is to be delayed, a lean concrete mud mat should be placed on exposed bearing soils. Any wet or loosened subgrade soils must be removed prior to concrete placement.

In order to provide frost protection and to develop bearing support, the foundation elements should bear at least 18 inches below final grades. Continuous wall footings should be at least 18 inches wide and individual column footings should be at least 30 inches wide to help prevent localized or "punching" shear failure which can occur with very narrow footings. These dimensions also help facilitate cleaning and preparation of footing bottoms and placement of reinforced steel.

5.3 Deep Foundations

Deep foundation elements should be installed to a depth of roughly 50 ft below existing grades to extend through any solution cavity or loose sand stratum. Driven H-piles and drilled auger cast piles are deep foundations systems that are commonly installed in Wilmington. However, for this site, due to the presence of solution cavities, auger cast piles are not preferred. During installation of an auger cast pile, large volumes of grout could be pumped into any cavity that may be present. Timber piles are not recommended because of the layers of dense sands and the likely presence of limestone cap rock. All timber piles would likely have to be predrilled and spudded to depths of roughly 40 ft and then driven to a depth of 50 ft, which might not be practical nor economical.

Allowable axial resistances are given in the following table for both HP12x53 and HP10x42 piles for an embedment length of 50 ft. The given resistances consider a factor of safety of 2 applied to the ultimate axial load. The embedment length is defined as the pile length below the bottom of the pile cap. The structural engineer can select the preferred pile based on structural load demands and installation criteria. Pipe piles can also be considered if economically more advantageous.

H-Pile Resistance Summary

Pile	Embedment Length	Allowable Resistance (FS=2)	
		Compression	Uplift
HP 12x53	50 ft	120 kips	70 kips
HP 10x42		100 kips	60 kips

Piles should be spaced on-center no closer than the three times the pile width; a center-to-center spacing of approximately 3 ft. The minimum spacing should be maintained to prevent the pile group compression load capacity from being significantly less than the summation of individual pile capacities. This spacing restriction also serves to limit surface heave and to reduce the possibility of damaging previously installed piles.

The pile hammer should have a rated energy of at least 25 ft-kips per blow. An impact hammer (air, hydraulic, or diesel) should be used to install the piles and verify that the design capacity is achieved. All equipment should be subject to the review of the geotechnical engineer.

5.3.1 *Load Testing*

Per Section 1810.3.3.1.1 of the 2012 North Carolina Building Code, piles with axial compressive loads greater than 40 tons require load testing in accordance with Section 1810.3.3.1.2. Load testing can be achieved by performing high-strain dynamic testing during the installation of the piles using a Pile Driving Analyzer (PDA) in general accordance with ASTM D4945. High-strain dynamic testing during pile driving can be used to establish pile driving criteria and also to evaluate driving stresses.

5.3.2 *Additional Deep Foundation Recommendations*

The geotechnical engineer should be retained to perform continuous Special Inspections of driven piles (per Chapter 17 of the 2012 North Carolina Building Code). The geotechnical engineer's representative should:

- Verify element materials, sizes and lengths comply with project requirements
- Determine capacities of test piles
- Observe driving operations and maintain complete and accurate records for each pile
- Verify placement locations and general plumbness
- Confirm type and size hammer used
- Record number of blows per foot of penetration
- Record tip and top elevations
- Document any damage to piles observed during driving

5.4 Slab-On-Grade Support

At least 6 inches of compacted select granular material should be placed beneath all ground floor slabs to provide a capillary break, provide more uniform slab support, and reduce damage to subgrade soils during construction. The select granular fill should classify as SP, SP-SM, SW, or SW-SM in accordance with the Unified Soil

Classification System (USCS), which requires that these soils have less than 12% passing the No. 200 sieve. Manufactured materials such as crushed aggregate base course (ABC) or processed fill (i.e., screenings) meeting this specification can be used. A modulus of subgrade reaction value of 180 psi/in may be used to design floor slab on subgrades consisting of these soils compacted to at least 98 % of the soil's standard Proctor maximum dry density.

Exposure to the environment and construction activities will weaken the floor slab subgrade soils. Therefore, the exposed subgrade soils in slab areas should be evaluated prior to placement of the select granular fill. If near surface deterioration of the soils has occurred, undercutting or reworking of the fill may be necessary.

Based on the results of our exploration and the assumed finish floor elevation, the floor slab will not be below the exterior grade and will not be subjected to hydrostatic pressure from groundwater. However, water vapor transmission through the slab is still a design consideration. Evaluating the need for and design of a vapor retarder or vapor barrier for moisture control is outside our scope of services and should be determined by the project architect/structural engineer based on the planned floor coverings and the corresponding design constraints, as outlined in ACI 302.1R-04 Guide for Concrete Floor and Slab Construction. Further, health and environmental considerations with respect to any potentially harmful vapor transmission are also outside of our scope.

5.5 Lateral Earth Pressures

Any below-grade walls should be designed to withstand loading from lateral earth pressures from surrounding soil and surcharge loads from nearby footings, floor slabs, or vehicle traffic, as appropriate. Recommended parameters for design of below-grade walls are given in the table below and are applicable for the existing on-site sandy soils.

Lateral Earth Pressure Parameters

Parameter	Value
Soil Friction Angle (ϕ')	30°
At-Rest Earth Pressure Coefficient (K_o)	0.5
Active Coefficient Earth Pressure (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3.0
Moist Unit Weight of Backfill	125 pcf

Based on the parameters given above, fixed below-grade walls that are not allowed to rotate (at-rest condition) should be designed using a triangular earth pressure distribution having an equivalent fluid weight of 60 pcf per foot of wall height, assuming full wall drainage is provided. For walls that will be allowed to rotate (i.e. retaining walls), then the equivalent fluid weight would be 41 pcf. Retaining walls should have adequate factors of safety against overturning, sliding, and global failure. The structural engineer should select the appropriate loading condition based on the below-grade wall configuration. Only frictional resistance should be applied along the base of the wall, no adhesion should be considered. Lateral pressures from any surcharge loads (traffic, adjacent foundation elements) should be added as a uniform rectangular soil pressure equal to 50% the vertical pressure applied over the full height of the wall.

The lateral earth pressure distribution presented above assumes no wall friction between the wall and soil backfill (δ = 0 degrees), the backfill is level and has been properly placed and compacted, and that water is not allowed to accumulate behind foundation walls. Exterior grades should promote rapid drainage away from the structure. Although groundwater is well below the design subgrade elevations, free draining clean stone backfill (such as NCDOT No. 57) should be placed against the wall exterior and isolated from backfill soil with a non-woven geotextile. Alternatively, a prefabricated drainage panel could be applied to the exterior face of the below-grade walls to prevent the buildup of hydrostatic pressures (utility break, long rainfall event, etc.). The wall drain should be connected to a pipe that allows any collected water to drain away from the wall.

Backfill soils placed behind below-grade walls should be granular and compacted to at least 95% of the soil's standard Proctor maximum dry density (ASTM D698) and from at the optimum water content to 3% above optimum. Operating heavy compaction equipment within 5 ft of the walls can create lateral earth pressures in excess of those recommended for design. As such, hand-operated equipment should be used within 5 ft of below-grade walls. Bracing of the walls may be needed during backfilling operations.

5.6 Seismic Site Classification

Seismic site classification is based on the top 100 ft of a site's subsurface profile. Based on the recorded SPT N-values and knowledge of the area geology, the site is designated as Seismic Site Class D in accordance with Section 1613 of the *2012 North Carolina State Building Code*.

6.0 Earthwork Recommendations

Our recommendations for subgrade preparation, backfill and compaction, and construction quality control are presented in the sections below.

6.1 Site Preparation

Site preparation should be initiated by removing trees, vegetation, and root mats, then stripping topsoil from the proposed construction site. Measured topsoil thicknesses encountered in the borings were typically about 3 inches. Some areas of thicker topsoil may be encountered between boring locations. For budgeting purposes, an average stripping depth of 6 to 8 inches should be considered to remove root mats and topsoil across the site. The near-surface Coastal Plain soils are sandy in nature and will likely be loosened from the site preparation operations (vegetation removal and stripping). The exposed subgrade should be compacted using a smooth drum roller after completion of topsoil stripping.

After stripping, the site should be proofrolled with a partially loaded tandem axle dump truck under the observation of an S&ME geotechnical engineer or his representative. The proofrolling will help to define the limits of loose soils that will require repair prior to structural fill placement or foundation construction. The proof roller should make two complete passes at walking speed over the subgrade areas. Any areas that are loose or that are observed to rut, pump, or deflect excessively during the proof rolling process should be repaired.

Possible repair measures could include undercutting to stable soils and backfilling with well-compacted low-plasticity materials (either on-site or off-site borrow), discing of wet in-situ soils to dry to proper moisture content and then re-compacting, placement of a woven geotextile stabilization fabric on unsuitable materials and placement of crushed stone, or some combination of these. The most practical repair measure will be influenced by the degree of instability which exists, groundwater levels, and weather conditions. As such, actual repair measures should be determined in the field at time of construction. Based on the borings, 6.5 to 8.5 ft of loose sands were encountered across the site. Undercutting the upper 5 to 6 ft within the building area, stockpiling the undercut materials, densifying the exposed subgrade soils and then place the fill back in compacted lifts (as described in Section 6.2) may be more economical than repairing individual footing excavations. This would significantly reduce the amount of footing undercut and time spent observing footing repair.

6.2 Structural Fill

The on-site sandy soils are also suitable for use as structural fill provided that any debris, organics, and particles greater than 3 inches in diameter are removed prior to placement. The moisture condition of near surface soils will be influenced by prevailing weather conditions. The sampled soils were generally moist and likely near or slightly dry of their optimum water content. The contractor should submit samples of proposed imported fill material for evaluation by S&ME during construction.

If off-site material will be required to achieve the design subgrade elevation, the imported structural fill should be of low plasticity (PI<20 and LL<50), be free of debris and other deleterious materials, have a standard Proctor maximum dry density of at least 110 pounds per cubic foot (pcf), and should have a maximum particle size no greater than 2 inches. The water contents of imported materials should be within 3% of the standard Proctor optimum water content.

6.2.1 Structural Fill Placement

Minimal new structural fill is anticipated (i.e. less than 2 ft) at this site to reach finished grades. Where structural fill is required to reach finished grade and is placed above existing grade, a clean sand, a slightly silty sand, a slightly clayey sand, or a silty sand with less than 20% fines and having a Unified Soil Classification of SP, SP-SM, SP-SC, or SM should be used. The more clayey and silty sands (more than 12% passing No. 200 sieve) are moisture sensitive and will be difficult to work with during wet weather conditions. The fill should be free of organics and debris, be placed in 8 to 10-inch thick lifts, and should be compacted to at least 98% of their standard Proctor maximum dry density (ASTM D698).

In-place density testing should be performed during fill placement to confirm that the recommended degree of compaction is achieved. To achieve the desired engineering properties (strength and stability) of compacted fill, the soil's water content should be properly controlled during fill placement and compaction. If wet weather grading is attempted moisture conditioning of site soils will likely be required prior to placement and compaction as structural fill. Drying may be accomplished by spreading and discing to maximize exposure to sun and wind during favorable weather conditions (i.e. warm, dry months).

Surface stormwater should be diverted away from the exposed subgrades to help minimize deterioration. Even during drier periods of the year, exposed subgrades should be sloped and sealed at the end of each day to promote runoff and reduce infiltration from rainfall.

7.0 Additional Geotechnical Considerations

Geotechnical recommendations given herein should be incorporated into the project specifications. Earthwork such as subgrade preparation and proof rolling, fill placement and compaction, and foundation subgrade bearing surfaces (footings and slabs-on-grade) should be observed by S&ME who is familiar with the site subsurface conditions and design intent. A sufficient number of density tests should be performed during fill placement to confirm that the recommended degree of compaction is achieved. A foundation subgrade evaluation should include the performance of shallow hand augering with dynamic cone penetrometer testing to confirm the suitability of near surface bearing soils for foundation support.

S&ME would welcome the opportunity to provide geotechnical engineering and material testing services during construction of this project. This arrangement provides the continuity between design and construction that is important to verify recommendations are followed, soil and groundwater conditions are consistent with those presented in this report and to make sound field engineering decisions based on the knowledge gained from this exploration.

8.0 Qualifications of Report

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations contained in this report are based upon applicable standards of our practice in this geographic area at the time this report was prepared. No other representation or warranty either express or implied, is made.

We relied on project information given to us to develop our conclusions and recommendations. If project information described in this report is not accurate, or if it changes during project development, we should be notified of the changes so that we can modify our recommendations based on this additional information if necessary.

Our conclusions and recommendations are based on limited data from a field exploration program. Subsurface conditions can vary widely between explored areas. Some variations may not become evident until construction. If conditions are encountered which appear different than those described in our report, we should be notified. This report should not be construed to represent subsurface conditions for the entire site.

Unless specifically noted otherwise, our field exploration program did not include an assessment of regulatory compliance, environmental conditions or pollutants or presence of any biological materials (mold, fungi, bacteria). If there is a concern about these items, other studies should be performed. S&ME can provide a proposal and perform these services if requested.

S&ME should be provided the opportunity to review the final plans and specifications to confirm that earthwork, foundation, and other recommendations are properly interpreted and implemented. The recommendations in this report are contingent on S&ME's review of final plans and specifications followed by observation and monitoring of earthwork and foundation construction activities.