GEOTECHNICAL EXPLORATION
STERILITE CORPORATION FACILITY

HIGHWAY 72 WEST & CHARLOTTES ROAD
CLINTON, SOUTH CAROLINA
S&ME PROJECT NO. 1261-03-577

Prepared For:

LAURENS COUNTY
SOUTH CAROLINA

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November 2003

November 24, 2003

Laurens County
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Attention: Mr. Ernest B. Segars
Laurens County Administrator

Reference: **GEOTECHNICAL EXPLORATION**
Sterilite Corporation Facility
Highway 72 West and Charlottes Road
Clinton, South Carolina
S&ME Project No. 1261-03-577

Ladies and Gentlemen:

S&ME, Inc. is pleased to submit this Geotechnical Exploration report for the Sterilite Corporation facility in Clinton, South Carolina. The exploration was performed to evaluate subsurface conditions at the site pertinent to site preparation, excavation, and structural support. The report presents a brief confirmation of our understanding of the project, the exploration results, and our geotechnical conclusions and recommendations regarding the above considerations.

We appreciate the opportunity to work with Laurens County and Harrison, Walker & Harper by providing the geotechnical consultation for this project. Should you have any questions regarding the information in this report, please contact us.

Sincerely,
S&ME, Inc.

Michael Ulmer, P.E.
Project Engineer

Walker Birdsong, P.E.
Senior Consultant

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1.0 SUMMARY

For your convenience, this report is summarized in outline form below. This brief summary should not be used for design or construction purposes without reviewing the more detailed information presented in the remainder of the report.

1. The subject property was explored with 54 soil test borings, two CPT soundings, and routine laboratory testing. A front-end loader was used to clear access paths to borings located in wooded areas.

2. The majority of the borings initially penetrated a 2 to 6-inch thick layer of organic laden topsoil. In some areas, the surface soils appear to have been cultivated in the past. Below these surface materials, the borings penetrated moderate to high consistency residual soils. Nine borings encountered partially weathered rock at depths from 1 to 69 feet, and one boring encountered auger refusal at a depth of 77 feet. The refusal materials appears to be massive rock.

3. Subsurface water was measured in 32 borings at depths from 13 to 33.5 feet below the existing ground surface.

4. Site preparation will include clearing; stripping of surface vegetation, organic laden topsoil, and roots; and undercutting any unstable surface soils. The existing pond bottoms will be soft and wet and require undercutting, and possibly stabilization, prior to fill placement.

5. The clayey soils at this site are moderately plastic. They will require special handling during grading and their use will be greatly affected by prevailing weather conditions. When properly handled, these soils should not be susceptible to high swell pressures or volume changes.

6. The borings indicate excavation to the expected subgrade levels will encounter moderate to high consistency soils and possibly partially weathered rock. The excavation anticipated at this site is not expected to encounter massive rock.

7. All fill should be compacted to at least 95 percent of the soil's standard Proctor maximum dry density. This should be increased to 98 percent in the upper 18 inches below the building floor slab. The majority of the on-site soil are adaptable for use as well-compacted structural fill with moisture adjustment, primarily by drying.

8. Groundwater is near and potentially above the design grade in the west part of the building area. We expect the proposed drainage ditch to the west and south of the building will lower the groundwater level. However, groundwater control measures could be needed in this area.

9. Building foundations may be designed as conventional shallow spread footings bearing in residual soils and well-compacted fill. A net allowable bearing pressure of 3000 psf is available.

10. The site has an IBC 2000 site classification of D. A site-specific analysis indicates the design spectral response acceleration parameters are S_{DS} = 0.33g and S_{D1} = 0.18g and the Seismic Design Category for a Seismic Use Group I is C.

2.0 PROJECT INFORMATION

It is our understanding that Sterilite Corporation will construct a new manufacturing and warehouse facility on approximately 265 acres on Charlottes Road near Clinton, South Carolina. The manufacturing plant will be in the north portion of the building, and the warehouses will be in the south portion. A railroad spur will enter the north side of the site and then turn 90 degrees to parallel the north side of the manufacturing plant. Development will include paved drives and parking around the building and detention ponds to the east of the building. A water tower is planned adjacent to the northwest corner of the building. We understand the building will be constructed in two half-million square foot phases, but the entire site will be graded during the initial site development.

The buildings will be of convention steel frame construction with a soil supported floor slab. The typical bay size is 30 by 70 feet, maximum column loads will be on the order of 55 kips, and maximum floor loads are assumed to be on the order of 250 psf. The manufacturing area will house large molding machines. The smaller machines will sit on the floor slab. The larger machines will require thicker slabs or isolated mat foundations. It is our understanding the machines do not produce significant dynamic forces or vibrations.

Based on the plans provided, the finished floor level will be 634 feet. Therefore, site grading will require excavation and fill placement depths on the order of 20 to 30 feet. Generally, the west side of the construction area parallel to Charlottes Road will be cut about 20 feet and the east site will be filled about 30 feet.

A ditch about 10 feet deep will be excavated at the toe of the cut slope in the west and south parts of the site. This ditch is to help lower the groundwater level in this area and to control rainwater runoff.

The above information is based upon our discussions with Mr. Gene Resch, P.E. and Mr. Tom Carson, P.E. with Carlisle Associates and Mr. Hunter Moore with Harrison, Walker & Harper. We have also reviewed grading plans provided by Carlisle Associates.

3.0 EXPLORATION

3.1 FIELD

The field exploration included a visual site reconnaissance by the Geotechnical Engineer and the performance of 54 soil test borings and two seismic cone penetrometer (CPT) soundings. The borings were performed in an approximately 300-foot grid across the proposed construction area, and locations were established in the field by our personnel using a hand-held GPS unit. The borings were made with truck and ATV-mounted drill rigs using hollow stem auger and wash boring techniques to advance the hole. Split-spoon samples and Standard Penetration Resistance (N) values were obtained in general accordance with ASTM D-1586, and two representative bulk soil samples were obtained for laboratory testing.

The CPT soundings were performed using a 25-ton truck-mounted CPT rig to hydraulically advance an electronically instrumented cone penetrometer. During penetration, the tip resistance, pore water pressure, and sleeve friction were measured and recorded in general accordance with ASTM D 5778. The method produces a nearly continuous record of information on subsurface conditions. The cone used for this exploration was instrumented with seismic sensors for measuring shear-wave velocity.

A Site Plan (Figure 1) showing the approximate testing locations, Boring and CPT Logs presenting the data obtained, and a brief description of the field testing procedures are included in the Appendix. Ground surface elevations shown on the Logs were interpolated from the furnished topographic survey and should be considered approximate.

3.2 LABORATORY TESTING

The bulk and split-spoon soil samples were transported to our laboratory where they were visually classified by a Geotechnical Engineer in accordance with the Unified Soil Classification System (USCS). Representative samples were subjected to the following tests:

- Natural Moisture Content
- Grain-Size Distribution
- Atterberg Limits
- Standard Proctor Compaction
- Remolded California Bearing Ratio (CBR)
- Electrical Resistivity and pH

The testing was performed in general accordance with ASTM standards. The results are presented on data sheets in Appendix C.

4.0 SITE CONDITIONS

4.1 SURFACE FEATURES

The subject property consists of approximately 265 acres of land in the north quadrant of the intersection of Highway 72 West and Charlottes Road south of Clinton, South Carolina. The property is bounded generally to the north by Byrd Drive and woodlands, to the east by the Bush River, to the southeast by SC Highway 72 West, and to the west by Charlottes Road. The majority of the property is presently open and used to pasture horses and grow hay. The rest of the property is wooded. There are a residence and several barns in the south corner of the property.

Topography generally slopes downward from a ridge line along Charlottes Road to the east. There are ponds in draws in the central and south portions of the site, and the Bush River flows along and through the east side of the site. Based on the topographic plans provided, elevations range from about 654 feet along Charlottes Road to about 580 feet along the Bush River. A sanitary sewer line parallels the Bush River, and overhead and underground utilities are present along SC Highway 72 West and Charlottes Road.

4.2 AREA GEOLOGY / SOIL SURVEY

4.2.1 Area Geology

Laurens County is in the Piedmont Physiologic Province. The Piedmont Physiologic Province is a relatively broad strip extending from central Alabama across Georgia and the Carolinas into Virginia. Rocks of the Piedmont occur in belts that are some of the oldest formations in the United States. The rock types are primarily metamorphic gneiss and schist with some granite intrusions.

The major portion of the bedrock in the Piedmont is covered with a varying thickness of residual soil that has been derived by chemical decomposition and physical weathering of the underlying rock. Residual soils developed during the weathering of this bedrock, consist predominately of micaceous sandy silts and silty sands which grade to clayey silts with nearness to the ground surface. The thickness of the residual soils can vary from only a few feet to in excess of 100 feet.

The boundary between the residual soil and the underlying bedrock is not sharply defined. Generally, a transition zone consisting of very hard soil to soft rock, appropriately classified as "partially weathered rock", is found. Within the transition zone, large boulders or lenses of relatively "fresh" rock that are generally much harder than the surrounding material often exist. The irregular bedrock surface is basically a consequence of differential weathering of the various minerals and joint patterns of the rock mass.

4.2.2 Soil Survey

Review of the *Soil Survey of Laurens and Union Counties South Carolina* (USDA SCS, 1975) indicates the soils on the subject property are mapped predominantly as Cecil and Appling soils.

The soils in the floodplain along the Bush River are mapped as Cartecay-Toccoa complex. According to the *Soil Survey*, the depth to bedrock in the Cecil and Appling soils is typically more than five feet, the depth to groundwater is typically greater than six feet, and the Hydrologic Soils Group is B.

4.3 SUBSURFACE CONDITIONS

4.3.1 Topsoil and Cultivated Soils

The borings initially penetrated a 2 to 6-inch thick layer of organic laden topsoil. In some areas, cultivated soils up to 18 inches deep were encountered. These thicknesses will vary in unexplored areas. The borings were made during dry weather, and the cultivated soils were generally dry and stable.

4.3.2 Residual Soils

Beneath the topsoil and/or cultivated soils, the borings encountered residual soils common to the Laurens County area. These soils generally consist of an upper stratum of sandy clayey silt underlain by silty sands and sandy silts with varying mica content. The N values in the residual soils vary from 4 to 81 blows per foot. These values indicate a soft to very hard consistency for silt and clay and a very loose to very dense relative density for sand. With the exception of borings B-9 and B-48, all of the borings were terminated at depths from 10 to 100 feet below the existing ground surface without encountering auger refusal. The clayey silts have moderately high plasticity indexes (PI) and liquid limits (LL) and are moderately plastic. The USCS symbols are MH for clayey silt, ML for sandy silt, and SM for silty sand.

4.3.3 Partially Weathered Rock

Partially weathered rock in a layered and massive form was encountered by nine of the borings at depths varying from 1 to 69 feet below the existing ground surface. Partially weathered rock is a transitional material between very hard soil and rock that has a Standard Penetration Resistance value of at least 50 blows per 6 inches. This material contains boulders and rock lenses. Boring B-48 was terminated in the partially weathered rock at a depth of 100 feet.

4.3.4 Refusal Material

Refusal to auger advancement was encountered by boring B-9 at a depth of 77 feet below the existing ground surface. The refusal material appears to be massive rock.

4.3.5 Subsurface Water

Subsurface water was measured in 32 borings at depths from 13 to 33.5 feet below the existing ground surface. It should be noted that the actual groundwater level will fluctuate during the year due to such things as seasonal variations, withdrawal for irrigation purposes, and construction activity in the area.

The above description of subsurface conditions is relatively brief and general. For more detailed information, individual Boring Logs and contained in Appendix B may be consulted.

5.0 SITE GRADING CONCLUSIONS AND RECOMMENDATIONS

5.1 SITE PREPARATION

5.1.1 General

Site preparation should include the removal of all unsuitable surface materials. This will include surface vegetation, organic laden topsoil, roots, and any unstable surface soils. The borings indicate the organic laden topsoil thickness to be about 2 to 6 inches. Cultivated soils up to 18 inches deep were also encountered in some areas. These amounts will vary in unexplored areas.

After stripping, the exposed subgrade should be evaluated by a representative of the Geotechnical Engineer to confirm that all unsuitable materials have been removed. To aid the Engineer during this evaluation, the exposed subgrade should be proofrolled with a heavily loaded tandem-axle dump truck or similar rubber-tired equipment. Proofrolling not only helps reveal the presence of any unstable or otherwise unsuitable surface materials, but also will densify the exposed subgrade for new fill placement and structural support. Any areas which deflect excessively under proofrolling should be undercut as recommended by the Geotechnical Engineer and backfilled as discussed in Section 5.3.

5.1.2 Previously Cultivated Soils

Previously cultivated soils up to 18 inches deep were encountered in isolated areas. During dry weather conditions, our experience indicates that these soils can typically be scarified and recompacted in place prior to fill placement. In wet weather conditions, these soils will likely be wet and unstable and should be removed prior to fill placement. The proofrolling measures discussed above will be helpful in determining the suitability of previously cultivated soils.

5.1.3 Subsurface Water Control

The boring data indicates excavation to proposed subgrade levels in the southwest part of the site will be near the groundwater level. Construction of the toe ditch discussed in Section 5.5 below will help lower the water level, but isolated areas may remain wet. Gravity drained ditches converted to French drains may be required to lower the water level and help dry these areas. The French drains should consist of a perforated pipe surrounded by washed crushed stone and wrapped with a geotextile fabric. The French drains should be installed as deep as possible to drain by gravity to an approved outfall area. A detail of a typical French drain is provided in the List of Figures, (Figure 3).

The location of wet areas and the need for French drains will not become apparent until grading begins. The optimum method is to locate the drains in the field during grading by joint consultation of the grading contractor and the Geotechnical Engineer.

5.1.4 Existing Ponds

The bottoms of the existing ponds in the central portion of the site are soft, wet, and unstable for fill placement. The bottom of the smaller, west pond appears to be above the proposed finished floor level; therefore, cutting to design subgrade levels will remove these soils.

The bottom of the larger pond is below the finished floor level. Therefore, the pond bottom should be undercut to stable materials prior to fill placement. Undercutting should be performed under the observation of the Geotechnical Engineer to help verify all unsuitable soils are removed and to prevent overexcavation of stable materials.

5.1.5 Special Considerations

Laboratory testing indicates that the more clayey residual soils have a PI and LL that are moderately high. When wet, these soils can be unstable and difficult to work with. These soils can also undergo volume change (shrink/swell) with significant changes in their moisture content. These soils comprise a large part of the soils that will be used in construction; consequently, they cannot be undercut in mass economically. These soils are suitably used in this area, but the measures discussed below are recommended:

- The soils will require more drying than typically expected during grading. This should be taken into consideration during the bidding phase of the project. Prevailing weather will greatly affect the use of the soils.

- The subgrade soils for slabs should not be allowed to dry significantly. This could require wetting and reworking the surface soils prior to slab installation.

- Rainwater runoff should be positively directed away from the buildings. Also, all utilities located below the floor slabs should not leak.

- Consideration should be given to separating the floor slabs from the subgrade soils with a 6-inch thick layer of granular material as discussed in the Building Floor Slab Section below.

When properly handled, these soils should not be susceptible to high swell pressures or significant volume changes.

5.2 EXCAVATION

The boring data indicates that excavation to the expected subgrade levels at the site will extend through moderate to high consistency soils and possibly partially weathered rock. Moderate to high consistency soils can normally be excavated by routine earthmoving equipment. That is, mass excavation can be accomplished by bulldozer pushed scrapers with light to moderate pre-loosening of the higher consistency soils by tractor drawn rippers. Local excavation for shallow utility trenches and foundations can be accomplished by a heavy backhoe or tracked excavator.

Partially weathered rock can normally be excavated by very hard ripping or by use of a heavy tracked excavator with light to significant difficulty. Rock layers or boulders in the partially weathered rock could require blasting, particularly for local excavation.

Massive rock that could not be penetrated by drill rig augering methods was encountered by one boring at a depths of 77 feet. The refusal material appears to be below the expected subgrade levels.

We would like to point out that rock in a weathered, boulder, and massive form varies erratically in depth and location in the Piedmont Geologic Province. Therefore, there is always a potential these materials could be encountered at shallower depths between the boring locations.

All excavations should be sloped or shored in accordance with local, state, and federal regulations, including OSHA (29 CFR Part 1926) excavation trench safety standards. The contractor is usually solely responsible for site safety. This information is provided only as a service and under no circumstances should S&ME be assumed to be responsible for construction site safety.

5.3 FILL PLACEMENT AND COMPACTION

All fill placed in building, pavement, and embankment areas should comprise soils free of organic matter, rock fragments greater than six inches in diameter, and other deleterious materials. The fill should be uniformly spread in relatively thin lifts (8 to 10 inches, loose) and compacted to at least 95 percent of the soil's maximum dry density, as determined by a laboratory standard Proctor compaction test (ASTM D-698). The fill in the upper 18 inches below floor slabs should be compacted to at least 98 percent. The moisture content should be controlled to within plus to minus 3 percent of optimum. The moisture content of the more clayey, plastic soils should be maintained between minus 1 and plus 3 percent of optimum.

The majority of the building will be supported on structural fill; therefore, it is very important that all of the fill is uniformly well compacted. Accordingly, fill placement should be monitored by a qualified Material Technician working under the direction of the Geotechnical Engineer. In addition to this visual evaluation, the Technician should perform a sufficient number of in-place field density tests.

5.3.1 Use of Excavated Soils as Structural Fill

Based on the results of the laboratory testing, the residual soils on-site are adaptable for use as a well-compacted structural fill to support the building and pavements. The laboratory testing indicates that the moisture content of the soils was generally above the optimum for compaction at the time the exploration was performed. Accordingly, some drying of the soils could be needed to obtain a high degree of compaction. It should be pointed out that the moisture content of the soils will be significantly affected by prevailing weather conditions. Significant drying should be expected unless grading is performed during hot and dry weather.

Partially weathered rock will also be suitable for use as structural fill. This material is typically excavated in the form of blocks. Normally, heavy self-propelled sheeps-foot type compaction equipment can suitable pulverize these blocks. Rock pieces and boulders larger than 6 inches should be placed in controlled fills below pavements.

Undercut cultivated and other soils are typically not suitable for use as structural fill. These soils should be wasted off site or used in non-structural areas.

5.4 CUT SLOPES AND FILL EMBANKMENTS

Cut slopes and fill embankments up to 30 feet in height will be required to grade this site. Generally, the west side of the site will be cut up to about 20 feet and the east side will be filled up to about 30 feet. Cut and fill slopes will be inclined at 3:1, horizontal to vertical. The boring data indicates subsurface water levels will be near the proposed subgrade level on the west and south sides of the building. An approximately 10-foot deep ditch will be excavated along the toe of the slope to intercept subsurface water and help lower the water level beneath the building.

S&ME has performed a slope stability analysis using the computer program PC STABL 5M and the modified Janbu Method. Based on this analysis and our past experience, cut slopes and embankments constructed at an inclination of 3:1 (horizontal to vertical) should be stable with the on-site materials. All fill placed in embankments should be compacted to similar requirement as discussed in Section 5.3. It is difficult to compact soil at the face; therefore, it may be necessary to construct the embankments outside their design limits, and then cut them back, leaving the exposed face well compacted.

The soils at this site will be highly susceptible to erosion from rainwater runoff, particularly when used as fill. Accordingly, we recommend that the faces of slopes and embankments be protected by establishing vegetation or other erosion control techniques as soon as practical after grading. Also, rainwater runoff should be diverted away from the crest of slopes.

5.5 TOE DITCH

The proposed toe ditch on the west and south sides of the building will intercept the subsurface water level. While our analysis indicates the slope will be stable, a continuous seepage condition will likely develop along the face of this ditch. Our experience indicates that this continuous seepage may cause soils to sluff off the face of the slope requiring repeated maintenance. We recommend that the lower portion of the slope and ditch that will be susceptible to seepage be reinforced with a layer of non-woven geotextile fabric and a layer of hand-placed riprap. This will allow continuous seepage through the face of the slope and help prevent the surface soils from continuously sluffing off into the ditch. The limits of this reinforcement are best determined in the field during grading by joint consultation of the grading contractor and Geotechnical Engineer.

5.6 SUBGRADE REPAIR AND IMPROVEMENT METHODS

The exposed subgrade soil can deteriorate when exposed to construction activity and environmental changes such as freezing, erosion, softening from ponded rainwater, and rutting from construction traffic. We recommend that the exposed subgrade surface in the pavement and grade slab areas, that has deteriorated, be properly repaired by scarifying and recompacting immediately prior to construction. If this has to be performed during wet weather conditions, it would be worthwhile to consider undercutting the deteriorated soil and replacing it with crushed stone.

6.0 FOUNDATION DESIGN AND CONSTRUCTION

The exploration indicates that building foundations may be designed as shallow spread footings. The following presents our geotechnical conclusions and recommendations regarding structural support.

6.1 BEARING PRESSURE

Building foundations may be designed as conventional spread footings bearing in residual soils and well-compacted structural fill. A net maximum allowable bearing pressure of 3000 psf is available for design.

All foundation excavation bottoms should be evaluated by the Geotechnical Engineer prior to concrete placement. This evaluation will help verify that individual footings are directly underlain by suitable bearing material.

6.2 BEARING DEPTH AND DIMENSION

All footings should bear at least 18 inches below grade to prevent being adversely affected by frost penetration and to develop the design bearing pressure. Continuous wall footings should not be less than 24 inches wide and isolated column footings should not be less than 36 inches wide. This recommendation is made to help prevent a "localized" or "punching" shear failure condition that could exist with very narrow footings.

6.3 SETTLEMENT

A settlement analysis was performed using the boring and CPT data, and the anticipated foundations loads. Based on this analysis, we estimate properly designed and constructed foundations will experience maximum column settlement (due to the service loads) on the order of about 0.50 to 0.75 inches. Maximum differential settlement between any two adjacent columns is estimated to be about 0.50 inch. Based on our past experience, we expect 50 to 70 percent of the total settlement to take place upon load application or shortly thereafter. The majority of the remaining settlement is expected to occur over a 6 to 12 month time period.

Over half of the building area will be raised with as much as 30 feet of fill. The mass weight of the fill will cause the residual soils and lower parts of the fill to undergo some settlement. We expect that most all of the settlement will take place during grading; however, we recommend that at least two weeks be allowed to pass from the conclusion of grading to the construction of foundations. Settlement monitoring hubs should be established promptly after grading. Their movement should be monitored by precise surveying techniques, and the information should be furnished to the Geotechnical Engineer for review.

6.4 FLOOR SLAB

Concrete slabs may be soil supported provided that the recommendations discussed in Section 5 regarding site preparation and fill placement are followed. We suggest that an 6-inch thick layer of crushed stone be used to separate floor slabs from the subgrade soils. The crushed stone should consist of Macadam Base Course compacted to 100 percent of the standard Proctor maximum dry density. This layer will help reduce construction downtime during wet weather conditions and will provide a good leveling course. The crushed stone should consist of an aggregate base course as described in Section 7.1.

Based on the laboratory CBR results and our experience, a modulus of subgrade reaction (k) of 120 pci, based on the 30-inch diameter plate method, should be available for design of grade slabs on properly prepared subgrades. Using the six inches of crushed stone under the slabs will increase the subgrade modulus to 175 pci. Please note that using this material during construction will cause it to be contaminated with soil and lose some of its supporting value.

6.5 SEISMIC CONDITIONS

Based on Sections 1615.1.1 and 1615.1.5 of the IBC 2000, it is our interpretation this site is a Site Class D. The soils should not be subject to liquefaction. This is based on the boring data, the shear-wave velocity data, and our experience with geologic conditions in this area.

6.5.1 Site-Specific Analysis And Acceleration Response Spectrum

Based on Section 1615.2 of the IBC 2000, we have performed a seismic site-specific response analysis. This analysis is based on synthetic seismograms (base rock acceleration time histories) from the USGS National Seismic Hazard Mapping Project Interactive Deaggregation web page and the soil shear-wave velocities measured at the site. Modal and mean seismograms are scaled to a 2 percent probability of exceedance (PE) in 50 years and a spectral acceleration (SA) frequency of 1.0 Hz.

The computer program SHAKE 2000[1] was used to performed the site response analyses. SHAKE 2000 computes the response of a horizontally layered soil system subjected to a horizontal acceleration time history. The program uses index and dynamic soil properties to vertically propagate the response up the soil column. The analysis produces response spectra which are used for structural design. Acceleration response spectra (ARS) are presented in the attached Figure 2. As required by Section 1615.2.4 of the IBC 2000 code, the site-specific ARS shown on Figure 2 have been limited to a minimum value of 80 percent of the general response spectrum as determined in Section 1615.1.4.

The IBC 2000 design spectral response acceleration parameters S_{DS} and S_{D1} (for site class D) and the parameters based on the site-specific response spectra are presented in the table below. The design spectral response acceleration parameters S_{DS} and S_{D1} taken from the site-specific ARS

[1] SHAKE2000 (modification of the original SHAKE program by Schnabel, Lysmer & Seed) is a computer program for conducting equivalent linear seismic response analyses of horizontally layered soil deposits (Idriss & Sun).

indicate the Seismic Design Category for a Seismic Use Group I is C. The Seismic Use Group and Design Category should be determined by the Structural Engineer.

Method	S_{DS}	S_{D1}
General ARS	0.42	0.22
Site-Specific ARS	0.33	0.18

6.6 RETAINING WALL PARAMETERS

Retaining walls may be required to grade this site. All retaining walls must be capable of resisting lateral earth pressures that will be imposed on them. Lateral earth pressures to be resisted by the walls will be partially dependent upon the method of construction. Assuming that the walls are relatively rigid and structurally braced against rotation, they should be designed for a condition approaching the "at-rest" lateral pressure. However, in the event that the walls are free to deflect during backfilling, as for any exterior walls that are not restrained or rigidly braced, the "active" pressure conditions will be applicable for design. The following lateral earth pressure parameters are recommended for design. These parameters assume a level backfill, a frictionless wall, and no hydrostatic pressure.

LATERAL EARTH PRESSURE COEFFICIENT	VALUE
At-Rest Coefficient (K_o)	0.53
Active Coefficient (K_a)	0.36
Passive Coefficient (K_p)	2.8
UNIT WEIGHT OF SOIL (MOIST)	115 pcf
FRICTION FACTOR FOR FOUNDATIONS AND BEARING SOILS	0.36

The recommended lateral earth pressure coefficients do not consider the development of hydrostatic pressure behind the earth retaining wall structures. As such, positive wall drainage must be provided for all earth retaining structures. These drainage systems can be constructed of open-graded washed stone isolated from the soil backfill with a geosynthetic filter fabric and drained by perforated pipe, or several wall drainage products are made specifically for this application. Lateral earth pressures arising from surcharge loading and slopes above the walls should be added to the above earth pressures to determine the total lateral pressure.

The soil backfill placed behind retaining walls should be compacted to at least 95 percent of the soil's standard Proctor maximum dry density. We caution that operating compaction equipment directly behind the retaining structures can create lateral earth pressures far in excess of those recommended for design. Therefore, bracing of the walls may be needed during backfilling operations.

6.7 pH AND RESISTIVITY TESTING

Two pH and resistivity tests were performed on site soils to help determine the corrosion potential for pipes and the reactivity for concrete structures below grade. The pH values from the samples ranged from 5.2 to 5.7. Resistivity values ranged from 34,000 to 66,000 ohms/cm.

Based on the laboratory pH and resistivity results, underground metal pipes should not be susceptible to corrosion and Type I Portland cement may be used with sufficient concrete cover to protect reinforcing steel. The Structural Engineer should select the actual cement type.

7.0 PAVEMENT DESIGN

Two soaked laboratory California Bearing Ratio (CBR) tests were performed on representative bulk samples obtained from the borings. The samples were compacted (remolded) to approximately 95 percent of the standard Proctor maximum dry density near the optimum moisture content. The results of the laboratory testing indicate CBR values of 3.7 to 5.6 percent. A value of 4 percent was used for design.

Design procedures are based on the ASSHTO *Guide for Design of Pavement Structures* and associated literature. The materials recommended for the pavement design are referenced to the South Carolina Department of Transportation *Standard Specifications for Highway Construction* (2000 Edition). Based on the subsurface conditions and assuming our grading recommendations will be implemented as specified, the following presents our recommendations regarding typical pavement sections and materials.

It is our understanding that rigid pavements will be used for all truck drive and parking areas. The only area that may use asphalt pavement is the employee parking lot which will be subject to only automobile traffic. As requested, the pavement design is based on a design life of 20 years and an average of 100 heavy trucks per day. The following AASHTO design criteria were assumed:

- Initial serviceability = 4.2
- Terminal serviceability = 2.0
- Reliability = 85%
- Standard Deviation = 0.35

A rigid pavement design was performed using a compressive strength of the concrete of 4000 psi. Based on empirical relationships with CBR values and our past experience, a modulus of subgrade reaction of 175 pci was used for design. (This value assumes the use of a six inch thick aggregate base course below the concrete.) Based upon the supplied traffic data, the following represents our recommendations for the rigid pavements.

STERILITE CORPORATION CLINTON, SC	**RIGID PAVEMENT DESIGN (Minimum Thickness)**	
	Automobile Parking Areas (Inches)	**Heavy Truck Areas (Inches)**
Portland Cement Concrete	5.0	8.0
Aggregate Base Course	NA	6.0

The aggregate base course will help provide additional support, provide drainage, and will help with the long-term performance of the concrete pavements when subjected to freeze-thaw actions. The aggregate base course should consist of Macadam Base Course (Refer to SCDOT *Standard Specifications*, Section 305 page 209) compacted to at least 100 percent of the maximum dry density, as determined by the modified Proctor compaction test (ASTM D 1557-

90 or AASHTO T 180-90). Rigid pavements should meet all the requirements of Section 501 (page 332) of the SCDOT specifications. To confirm that the base course has been uniformly compacted, in-place field density tests should be performed by a qualified Materials Technician, and the area should be methodically proofrolled under his evaluation.

As an alternative, automobile parking areas may consist of a 3-inch thick course of hot mix asphaltic (HMA) concrete underlain by 6 inches of Macadam Base Course. The hot-mixed asphalt can consist of a Type 1 or Type 3 surface mix (Section 403, page 302). Type 3 has a somewhat smoother appearance, but if used, it should have a minimum 50-blow Marshall stability of 1400 pounds. The Macadam Base Course should be compacted as specified above.

All materials and workmanship should meet the SCDOT's *Standard Specifications for Highway Construction* (2000 Edition). Also, sufficient tests and inspections should be performed during pavement installation to confirm that the required thickness, density, and quality requirements of the specifications are followed.

7.1 GENERAL PAVEMENT CONSIDERATIONS

The performance of the flexible and rigid pavements will be influenced by a number of factors including the actual condition of subgrade soils at the time of pavement installation, installed thicknesses and compaction, and drainage. The subgrade soils should be reevaluated by thorough proofrolling immediately prior to paving, and any unstable areas should be repaired. This recommendation is very important to the long-term performance of the pavements and slabs. Areas adjacent to pavements (embankments, landscaped island, ditching, etc.) which can drain water (rainwater or sprinklers) should be designed to help reduce water seepage below the pavements. This may require the use of french drains or swales.

Typically, low safety factors are used for pavement design. Therefore, it is very important that all components of design are properly installed and that the expected traffic is not exceeded.

8.0 LIMITATIONS 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 warranty, express or implied, is made.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations between the borings will not become evident until construction. If variations appear evident, then we will re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the proposed building are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing.

We recommend that S&ME be provided the opportunity to review the final design plans and specifications in order that earthwork and foundation recommendations are properly interpreted and implemented.