JULY 2006

GEOTECHNICAL ENGINEERING REPORT
FOR
PROPOSED CENTER AT ROCKY MEADOWS
CARLISLE BOROUGH, DICKINSON & SOUTH MIDDLETON TOWNSHIPS
CUMBERLAND COUNTY, PENNSYLVANIA

PREPARED FOR

E P CARLISLEI, LLC
RENO, NEVEDA

PREPARED BY

CGC
CUMBERLAND GEOTECHNICAL CONSULTANTS, INC.
801 BELVEDERE STREET • CARLISLE, PA 17013-4002

CGC
CUMBERLAND
GEOTECHNICAL
CONSULTANTS, INC.

July 26, 2006

Mr. David Loring, CEO
E. P. CarlisleI, LLC
140 West Huffaker Lane
Suite 509
Reno, Nevada 89511

Re: **Proposed Center at Rocky Meadows**
Carlisle Borough, Dickinson Township
and South Middleton Township
Cumberland County, Pennsylvania

Dear Mr. Loring:

In accordance with your request, we have completed a geotechnical engineering study for the referenced project. The scope of our work is outlined in our March 28, 2006 proposal to your office. A copy of our Agreement for Geotechnical Drilling Services was executed by our office on March 28, 2006 and by your office on June 19, 2006.

It has been our pleasure performing geotechnical engineering services for E. P. CarlisleI, LLC. This report is intended to provide guidance regarding site grading as well as general foundation design and construction parameters at the proposed Center at Rocky Meadows. This report is not intended as a tool to estimate rock excavation quantities. Estimates of required rock excavation quantities can be made following additional auger boring studies upon request. Inasmuch as detailed site and building designs are not complete at the time of this writing, all reported results, discussions, conclusions and recommendations contained in this report should be considered preliminary until supplemental studies or construction activities confirm the preliminary results. To implement the guidance presented herewith, Cumberland Geotechnical Consultants, Inc. would be pleased to perform the recommended supplemental studies along with the recommended onsite construction observation and testing services, which should be provided by a geotechnical engineering firm.

801 BELVEDERE STREET • CARLISLE, PENNSYLVANIA 17013-4002
TELEPHONE 717-245-9100 • FAX 717-245-9656 • www.cumberlandgeo.com

If you should have any questions concerning the following report, or if you should desire further assistance, please do not hesitate to contact us.

Yours sincerely,

CUMBERLAND GEOTECHNICAL CONSULTANTS, INC.

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
DAVID C. GASSERT
ENGINEER
NO. 39105-E

David C. Gassert

David C. Gassert, P.E.

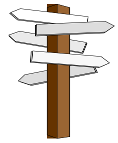
DCG/d
Attachments

TABLE OF CONTENTS

CUMBERLAND GEOTECHNICAL CONSULTANTS, INC.

<u>**TABLE OF CONTENTS (continued)**</u>

OBJECTIVE AND SCOPE



This report presents the results of our research, exploratory boring program and laboratory tests along with the interpretation and engineering analyses of this data. We do not intend our interpretations and analyses to address the relevance of potential environmental hazards such as contaminated soil, groundwater or biological pollutants. Furthermore, we have not addressed the delineation of any potential deep mines or wetland areas in this report. This report provides general guidance regarding our recommendations for foundation methodology and construction; however, construction estimates, such as the anticipated quantity of rock excavation for foundation construction or site grading should not be formulated from this report. Based upon the available test boring and laboratory testing data, we present our conclusions regarding potential subsurface material/foundation interactions along with recommendations concerning the design and construction of foundations and structural fill/backfill sections. The ASFE publication titled "Important Information About Your Geotechnical Engineering Report" in Appendix 5 presents additional details regarding the scope and limitations of this report.

PROJECT LOCATION

The site of the proposed Center at Rocky Meadows is situated in three (3) municipalities within Cumberland County including the Carlisle Borough, Dickinson Township and South Middleton Township. The site involves two properties positioned to the south of U.S. Route 11, to the north of Interstate 81 and to the west of Pennsylvania Route 465 (Allen Road). The project site encompasses all of the existing Rocky Meadows golf course as well as a tract owned by Mr. Frank Loney. The project location is shown on the Project Location Plan, Drawing No. LP-1, in Appendix 1 of this report. The position of the local streets relative to the project site is illustrated on the Test Boring Location Plan, Drawing No. TB-1 in Appendix 1 of this report.

SITE DESCRIPTION AND HISTORY

The 60 +/- acre site consists of the relatively undisturbed and wooded Loney tract along with the Rocky Meadows golf course. As implied by the name, many rock outcrops are visible along the golf course fairways, which are bordered by patches of woodland. Both the golf course and the Loney tract have rolling topography. High points include the western end of the golf course property at approximate elevation 550 feet and the southern end of the Loney tract at approximate elevation 521 feet. A wide swale traverses the central portions of the Loney tract, draining towards the western end of the Sheetz property where it reaches two closed depressions (and area with internal and no external drainage) at elevation 480 feet. The Sheetz property is positioned directly southwest of the intersection of Routes 11 and 465.

DESIGN CONCEPTS

Preliminary design concepts include approximately 15 single-story and two-story slab-on-grade structures with ancillary parking areas and landscaping. The multi-use independent commercial development will include office buildings, retail buildings, hotels and restaurants. Structural details for these buildings are not available at this time; nonetheless, we presume typical steel-frame and/or load-bearing masonry slab-on-grade structures will be used. The use of lightly loaded soil-bearing foundations is anticipated. We understand that there will be two entrances to the development along Allen Road and one along U.S. Route 11.

Significant cut and fill sections will be required to transition grades between the proposed south entrance at Allen Road and the existing golf course. Cut sections exceeding 10 feet thick will likely be required to establish rough subgrade levels in the areas of present topographic highs. Although complete site-grading plans have not been presented to our office, we anticipate that the proposed buildings will be supported by a combination of undisturbed soil, fill embankments and bedrock. As explained within the discussions to follow, precautions to restore soil-bearing conditions to foundations that would otherwise bear upon bedrock should be undertaken. We understand a wooded buffer zone around the perimeter of the site will be incorporated into the site stormwater management plan.

SUBSURFACE INVESTIGATION PROGRAM

As depicted on the Test Boring Location Plan, Drawing No. TB-1 in Appendix 1, 12 Standard Penetration Test (SPT)/Core Borings, Nos. B-1 through B-12, were drilled for the proposed development. SPT/Core Boring Nos. B-1 through B-6 were conducted on the Loney tract, and the remainder of the borings were drilled on the golf course. All borings were drilled by an in-house crew from Cumberland Geotechnical Consultants, Inc.

Each of the SPT/core borings was scheduled for a depth of 20 feet with the exception of Boring Nos. B-2 and B-6, which were drilled to depths of 18.0 feet and 18.5 feet respectively. The SPT/core borings totaled 236.5 lineal feet of drilling, including 128.6 lineal feet of diamond-bit coring through rock. The SPT/core boring locations were drilled at the point locations assigned by our office. Boring positions were visually assigned at locations relative to topographic landmarks, existing buildings or roadways. The approximate positions of these borings are illustrated on the Test Boring Location Plan, Drawing No. TB-1 in Appendix 1. The approximate ground surface elevation at each boring point was assigned relative to the site topographic contours shown on the Existing Conditions plan by Dawood Associates, Inc. The location and the elevation of the boring points are accurate only to the degree implied by the methods used.

The test borings were conducted in general compliance with the procedures established by the American Society for Testing and Materials (ASTM) Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils, Designation: D1586 and the Standard Practice for Diamond Core Drilling for Site Investigation, Designation: D2113. The SPT borings were cased via hollow stem augers or four-inch-diameter (outside diameter) heavy-duty steel casing in order to protect the boring walls from caving in. Down-hole water level readings were taken immediately after the completion of each boring (zero-hour reading) and again at 24 hours after drilling, provided the holes did not cave in and the drilling crew was still at the site. Detailed records of the subsurface conditions encountered by the borings are documented on the boring logs in Appendix 2 of this report. The stratification lines shown on these logs represent approximate boundaries between soil and rock types and transitions may be gradual.

GEOLOGY

In geologic terms, the site limestone bedrock is considered related to the Rockdale Run Formation of Ordovician age. The Ordovician Period occurred roughly 500 million years ago during the Paleozoic Era. The bedrock is considered to be a sedimentary rock formed from detrital (mineral or rock sediment which is transported), chemical, or biogenetic (created from living organisms) sediments and is primarily comprised of limestone. A reference entitled Engineering Characteristics of the Rocks of Pennsylvania (1982) by Alan R. Geyer and J. Peter Wilshusen of the Pennsylvania Geologic Survey describes the Rockdale Run Formation as "Very light gray, finely laminated, fine-grained limestone, with pink to brown lenses of chert."

Because the site is located within a carbonate bedrock formation, it should be considered to have karst terrain potential (containing deep bedrock fissures, caves, sinkholes and depressions). The Pennsylvania Geologic Survey's Open File Report No. 8902 compiled by William E. Kochanov (1988) does not indicate the presence of any closed depression (an area with internal and no external drainage) or sinkhole on the project site. Likewise, no apparent sinkholes or closed depressions were discovered during our observations of the existing site conditions. The previously mentioned closed depression near the Sheetz store appears to be on the Sheetz property. Report EG1 by Alan R. Geyer and J. Peter Wilshusen of the Commonwealth of Pennsylvania Topographic and Geologic Survey (1982) describe some of the subsurface characteristics of these formations as follows:

Formation	Drainage	Porosity and Permeability
Rockdale Run Formation	Good subsurface drainage; little surface drainage	Fractures and solution channels provide a secondary porosity of moderate to high magnitude; low to moderate permeability.

According to Educational Series 11 by William E. Kochanov of the Pennsylvania Geological Survey (1999), acid rainwater contributes to the creation of a groundwater plumbing network that promotes efficient flow of surface water to the groundwater table. Figure 6 of this series states the following: “Fractures behave as though they were pipes in limestone bedrock and convey water to the water table." This series goes on to describe how such subsurface drainage contributes to the formation of sinkholes, including: "If storm water, gathered over a specific area, is collected and directed into a karst area, the concentration of water may unplug one of the karst drains (Figures 15 and 16)."

The above references imply that good subsurface drainage and moderate to high secondary porosity via solution channels and fractures contribute to the vulnerability of the site bedrock and overburden to sinkholes. Our extensive experiences with these formations in the Carlisle area suggest that the site of the proposed Center at Rocky Meadows will be most vulnerable to sinkhole development during construction but will continue to be vulnerable to sinkholes in portions of the site subjected to concentrated water flow, such as stormwater management areas.

BEDROCK

According to geotechnical engineering convention, the position of the bedrock surface is defined as the depth where diamond-coring operations are necessary to advance the test borings or as the depth where tricone or auger refusal occurs in borings where no cores are procured. Boulders or other obstructions that are diamond-cored are not considered part of the "bedrock". Cumberland Geotechnical Consultants, Inc. can perform auger boring studies in order to provide details regarding the depth to bedrock profile. The depth to bedrock in each of the SPT/core borings is summarized in the table below and is shown on the boring logs in Appendix 2.

Although the depth to the bedrock surface at some of the SPT/core boring locations is indeterminate according to the definitions discussed above, it can sometimes be estimated based upon the SPT results. Refusal to further advancement of the split spoon sampler is considered a fair indicator of bedrock. Spoon refusal suggests that bedrock (by the definition above) could be close. The definition of the term "refusal" is relative to the number of blows required to advance the split-spoon sampler. In-house procedures define refusal as 50 or more blows for 6 inches of penetration. Consequently, when 50 or more blows are required to advance the split spoon sampler, one could declare that "soft" or "highly weathered" rock has been encountered. Where the spoon sampler did not encounter refusal, and no coring of bedrock was performed, the depth-to-bedrock is indeterminate but can sometimes be estimated based upon local or site trends. Accordingly, we have used this logic to estimate depths to rock where no diamond-coring occurred and have compiled the depth to bedrock table below based upon the test boring data along with our interpretations and extrapolations from the test boring data as follows:

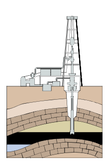
Depth to Bedrock Table

Boring Number	Ground Elevation (feet)	Depth to Rock * (feet)	Top-of-Rock Elevation * (feet)
B-1	496.5	30.0*	466.5*
B-2	506.5	1.0	505.5
B-3	489.5	30.0*	459.5*
B-4	483.0	5.0	478.0
B-5	491.0	6.0	485.0
B-6	521.0	2.0	519.0
B-7	510.0	4.9	505.1
B-8	508.5	10.5	498.0
B-9	530.0	9.0	521.0
B-10	513.0	8.0	505.0
B-11	525.5	3.5	522.0
B-12	512.5	6.0	506.5

* Note: The depth to rock is estimated – see explanation above

Core recoveries give an indication of the in-situ condition of the bedrock. The recorded core recoveries range from 15 to 100 percent, averaging 65 percent. Rock Quality Designations (RQD) illustrate the fractured condition of the recovered bedrock cores. Rock Quality Designation is defined as the total accumulated length of intact rock pieces greater than or equal to four inches expressed as a percentage of the total length of the core run from which the pieces originated. The recorded Rock Quality Designations range from zero to 72 percent, averaging 30 percent. Based upon all available bedrock core data, including fair core recoveries coupled with poor RQD values, the site bedrock is concluded to be a partly weathered mass containing some joints, fractures, dissolution channels and thin to thick soil seams. We anticipate that the materials labeled as "limestone" in this report, or on the boring logs, will require rock removal techniques including blasting and/or the use of pneumatic hammers. The significance of the site bedrock relative to potential conflicts with the proposed site development is highlighted above and addressed in the discussions below.

OVERBURDEN



Overburden is defined as any strata overlying the bedrock. Accordingly, overburden consists of soil, and/or decomposed or weathered rock that can be penetrated by augers or tricones. Overburden also consists of fill materials, obstructions and boulders that may have required limited diamond-coring. Therefore, the total apparent overburden thickness is reflected in the "Depth to Bedrock Table" above. Based upon the test boring and laboratory test results, a typical overburden profile is as follows:

Typical Overburden Profile

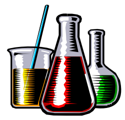
0 feet to 1 foot	**TOPSOIL – Dark brown organic SILT – moist – soft**
1 foot to 10 feet	**RESIDUUM: Orange-brown fat to lean CLAY – moist – stiff**

Specific information regarding the profile classification of the overburden is documented on the boring logs in Appendix 2. Below the topsoil layer, the site overburden typically consists of residual fat to lean clays. Residual soils are created by the in-place weathering of the native site bedrock. The terms “fat” and “lean” refer to the plasticity (stickiness) of the clay, with “fat” meaning highly plastic and “lean” meaning moderately plastic.

The blow counts used to calculate the Standard Penetration Test (SPT) N-value results obtained while sampling the site overburden with a split-spoon-sampling device are recorded on the boring logs. The SPT "N-value” is defined in ASTM D 1586 as follows: "*N-value* - the blowcount representation of the penetration resistance of the soil. The N-value, reported in blows per foot, equals the sum of the number of blows required to drive the sampler over the depth interval of 6 to 18 inches.” Refusal of the SPT split-spoon sampling device is determined by a maximum amount of blows for a given amount of penetration; therefore, “refusal” is an arbitrary term depending upon the number of blows and penetration distance specified. In-house procedures consider “refusal” to be 50 or more blows for six inches of penetration. Therefore, refusal is defined by inference of the blow counts and penetration distances recorded on the test boring logs. The ranges of SPT N-values recorded on the test boring logs (excluding N-values that involved refusals) are presented in the table below.

Minimum N-value	**Maximum N-Value**	**Median N-Value**	**Standard Deviation**
4	25	13	7

The site overburden soils generally exist in a moist and stiff state. Some zones of soft and unsuitable soils are present, especially near the ground surface where freeze-thaw cycles and previous site grading activities have destroyed the natural structure of the soil matrix, as well as near the overburden/bedrock interface where deep karst weathering processes have occurred. The significance of these Standard Penetration Test values, along with some lower or erratic SPT N-values occurring within isolated zones, are considered in the discussions to follow.

LABORATORY TESTS 

A series of general classification tests were conducted on representative samples of the overburden soils procured during the test boring program. These classification tests involved sieve and hydrometer analyses conducted in accordance with the American Society for Testing and Materials (ASTM) Standard D422 as well as Atterberg Limits determinations conducted in accordance with ASTM Standard D4318. The laboratory soil samples demonstrated high plasticity with all three (3) samples classifying as "CH" soils according to the Unified Soil Classification System (USCS) acknowledged by ASTM Standard D2487. According to USCS terminology, CH soils are defined as fat clays or fat clays with sand or gravel.

Natural water content tests were conducted on 20 selected samples in general compliance with ASTM Standard D2216. These tests indicate in-place soil moistures ranging from 7.8 to 33.5 percent, averaging 21.0 percent. A review of the natural moisture content results, coupled with the physical characteristics for each particular soil sample that was classified in the laboratory, suggest that the site soils at the time of drilling were typically roughly 4 to 10 percentage points wetter than their respective optimum moisture contents versus the ASTM D698 test procedure. Nonetheless, we believe the wet conditions implied by the three (3) tested samples are somewhat pessimistic as the average measured natural moisture content is 21.0 percent as opposed to 27 or 33 percent measured on the three (3) isolated samples. Based upon a visual review of the test boring samples, coupled with our local experiences, we believe that the majority of the onsite soils are a few to several percentage points wetter than their respective optimum moisture contents. Field and laboratory soil classification charts utilized by our firm are included in Appendix 3 of this report, and the results of our in-house laboratory soil tests are presented in Appendix 4. A summary of the results of our in-house laboratory tests is presented in the table below.

Physical Parameter Soil Test Results

Sample Location	USCS Class.	Liquid Limit	Plastic Limit	Plasticity Index	Passing No. 200 Sieve (%)	Natural Water Content (%)	Estimated Optimum Moisture (%)
B-3	CH	68	26	42	97.2	27.2	23.0
B-8	CH	50	24	26	98.8	27.1	20.0
B-10	CH	60	29	31	97.1	33.5	23.0

GROUNDWATER 

Water level readings have been made in drill holes at times and under conditions stated on the boring logs. This data has been reviewed and interpretations made in the text of this report. However, fluctuations in the level of the groundwater may occur due to variations in rainfall, local stream levels, temperature and other factors from the time measurements were made.

Down-hole water level readings were taken at zero hours in all borings and at 24 hours in borings where the walls did not collapse. No water was observed within any boring unless the drilling crew introduced water into the boring. Water was introduced to flush out rock cuttings and to cool the diamond bit on the end of the core barrel. Circulation of drill water was lost in Boring Nos. B-4 and B-7, suggesting that open voids are present below the level where circulation was lost and indicating that the level of the free water surface is at a deeper level than where the water circulation was lost. "Dry" readings were recorded at zero hours within two (2) borings where no drill water was introduced into the borings as well as within four (4) borings where water was introduced. Consequently, we believe the majority of the water observed in the borings could be attributed to introduced drill water or the influx of rainwater, or perched surface water, and it is therefore not considered to accurately reflect the true level of the free water surface.

An interpolation of ground water contour intervals for this site presented on mapping within Water Resource Report No. W-50 compiled by the Pennsylvania Topographic and Geologic Survey indicates that the project site is positioned between groundwater contour intervals of 440 and 460 feet, or roughly 40 feet below the existing ground surface. Based upon all data discussed above, it appears that the level of the free water surface is positioned at greater than 20 feet below the existing ground surface; nonetheless, isolated zones of higher perched water could be present on a sporadic basis. For the most part, groundwater is anticipated to be positioned within the site bedrock and should be encountered only in unusually deep excavations within the bedrock.

CONCLUSIONS AND RECOMMENDATIONS

The 60 +/- acre site of the proposed Center at Rocky Meadows in the Borough of Carlisle, Dickinson Township and South Middleton Township, Cumberland County, Pennsylvania, as represented by our laboratory test and test boring results, is considered acceptable for the proposed construction provided the intent of all presentations, discussions, conclusions and recommendations within this report are incorporated into the final design and construction process. Due to the erratic bedrock profile and the variability of soil properties disclosed by the test boring results, separate studies should be performed at specific areas of proposed construction, such as individual building sites. If detailed depth-to-rock information is desired, additional auger boring studies should be conducted.

Rock Conflicts and Excavation

Unless the site grading plan largely reflects the existing site grades, rock excavation will likely be a significant factor during bulk earthwork activities along with foundation construction. Rock conflicts during bulk cut operations and with foundation trenching should be anticipated. In addition to uniform conflicts, isolated conflicts with steep rock ledges and pinnacles will likely occur at many locations. A Caterpillar E325 excavator (trackhoe), or equivalent, may be able to remove portions of the upper trench rock, which is jointed and weathered; however, equipment this large may be impractical for footing excavations. If a Caterpillar D8 Dozer (or equivalent) equipped with a ripper is used to remove bulk cut materials, the uprooted materials may become too large to reuse in fill sections and could be considered “rock” for excavation purposes (depending upon the precise contract definitions for “rock” that are specified). Consequently, the majority of the rock encountered by the test borings appears to be competent enough to dictate removal by pneumatic hammers and/or blasting.

Earthwork and Site Grading Design

As implied above, we believe that both soil and blasted and fractured rock will be removed from cut areas. We suggest that “shot rock” should be used as the initial fill material in deep fill embankments. Rockfills should not be used to construct detention basin berms where low permeability soils are desired. Some of the rock materials removed from cut areas will be too large for reuse in structural fill sections; therefore, offsite hauling of oversize rock, or the incorporation of designated onsite rock disposal areas (berms or landscaping rocks) should be anticipated. Alternatively, oversize rocks could be broken with pneumatic hammers and “downsized” such that they can be properly incorporated into rock fill zones. The earthwork contract should include pricing for breaking down boulders into reasonable sizes. Reasonable rock sizes are discussed under the rock fill section to follow. To provide the best fill material/structure interaction, the uppermost four (4) feet of all building area fill sections should be limited to soil materials that can pass a four-inch sieve whenever practically possible; however, available onsite materials may dictate that rock materials must be used up to the level of the building floor subgrade level. The targeted transition level from rock fill to soil fill may be raised to 12 inches below the finished proposed pavement levels outside of buildings. Above these levels, it is preferable to use onsite clay or offsite borrow materials. **As it would be unreasonable to require the Owner to reimburse contractors to haul significant quantities of rock offsite (and make up the subsequent deficit with offsite soil), some oversize fill materials (rocks) might conflict with the proposed foundation trenching. Therefore, contractors should anticipate overruns on foundation and utility trench excavation and backfill quantities versus neat line calculations. Contractors should anticipate that rocks as large as 48 inches could be removed from trenches.**

As previously stated, based upon the preliminary laboratory test results, we estimate that the onsite clays are roughly several percentage points above their respective optimum moisture contents (and in isolated cases much more) as necessary to achieve 100 percent compaction relative to the ASTM D698 laboratory compaction test procedure. Accordingly, air-drying of onsite soils that are reused as fill materials should be anticipated. Air-drying of this nature can be reasonably accomplished only in favorable weather conditions. Consequently, winter earthwork activities are not considered appropriate for quality construction practices. Furthermore, fast-track construction schedules are not compatible with extensive air-drying at any time of year. In order to air-dry soils, the earthwork contractor must be permitted to work large areas of the site at any given time. **If the earthwork contractor is confined to portions of parking lots or a small building pad, such that they must place several successive lifts of fill in one day, then the required air-drying and compaction might not be achieved.** It is important that construction managers preparing critical path method schedules recognize that earthwork schedules will not be maintained unless these principals are taken into account. Nevertheless, the obstacle of wet onsite borrow materials is a common problem in the

Cumberland Valley. This problem is typical, and it can be overcome when appropriate provisions are incorporated into the construction practices and schedule.

Some relief from wet soil conditions could be created by mixing blasted rock and clay. In such a case, the rock/soil mixture should consist of at least 60 percent rock by volume. When placed and compacted, the rock/soil mixture should be unyielding under the construction equipment traffic.

Blasted rock reused as fill material could be expected to swell to approximately 130 to 135 percent of its original volume of the undisturbed unblasted bedrock. When calculating earthwork quantities, onsite clay materials independently reused as fill materials can be expected to shrink to approximately 90 percent of their original undisturbed volume if air-dried to the extent necessary to achieve optimum moisture for 100 percent compaction. Where compaction specifications are relaxed for nonstructural areas, or where significant air-drying is not necessary, a shrinkage factor of roughly 95 percent is considered more appropriate. A minimum horizontal buffer zone (slightly pitched to allow surface drainage) of 10 feet in width, measured from any proposed structure, should be incorporated into the design prior to any break in the surface grade that would constitute a slope. All proposed final site grades should represent a surface profile with a conservatively assigned slope of no steeper than three horizontal to one vertical to provide for adequate slope stability of the low-friction onsite soils.

Foundation Conditions

We believe zones of soft and wet near-surface soils are present throughout the project site. These zones should be readily identified by a thorough proofrolling program. In many cases, the proofrolling program will sufficiently rehabilitate and densify these soil zones. Following the proofrolling, any remaining soft and yielding soils can subsequently be removed and replaced with well-compacted offsite processed aggregates, or onsite "shot rock" before any foundation construction begins.

Conventional shallow-depth spread footing foundation systems are anticipated as the most cost efficient foundation configurations for slab-on-grade structures of three (3) stories or less. An analysis of the SPT N-values for Boring Nos. B-1 through B-12 suggests that the undisturbed site residual soils are capable of supporting unit foundation soil contact pressures of approximately 2,000 to 4,000 p.s.f., depending upon the location. Weak soil zones could be rehabilitated by proofrolling, along with selective replacement with structural aggregate zones. Foundations will likely be designed for construction across the cut/fill lines such that both near-surface and deep seated soft soil zones will conflict directly with the proposed footing subgrade levels. Consequently, a bearing pressure equal to 3,000 p.s.f. is judged appropriate as a preliminary value; however, site specific adjustments for specific building parameters are necessary to identify isolated soft soil zones that will not be appropriate for this suggested foundation ground contact pressure. When appropriate bearing pressures and good construction procedures are used, potential total and differential settlements will be maintained within tolerable limits. Whenever practically possible, no foundation should be positioned to bear upon any existing or proposed utility, storm sewer, or overlying trench backfill material. Wall penetrations should be utilized instead of routing pipes below proposed foundations. Rock encountered at the foundation subgrade levels will require overexcavation and cushioning to promote uniform foundation performance. Cushioning can be accomplished by removing any rock that conflicts with proposed footings to a level of at least 12 inches below the concrete subgrade level and backfilling the resulting void with compacted onsite soil. Footing bases should be positioned at least 42 inches below the finish exterior grades for protection from frost penetration where applicable.

A conservative frost depth will help to protect the onsite soils from undesirable swelling due to fluctuations in moisture content. We estimate that surcharge pressures of roughly 500 p.s.f. will be necessary to counteract potential swelling pressures generated by onsite soils that become saturated after construction is complete. Therefore, precautions to protect building walls that do not produce foundation ground contact pressure of at least 500 p.s.f. should be incorporated into the structural and architectural designs. Overexcavation of residual clay from below the proposed footing areas, followed by backfill with compacted coarse aggregate may be a potential solution to this problem. Specific recommendations should be formulated based upon the estimated structural loading and the localized soil conditions. The use of moisture barriers below the slab-on-grade will also help to prevent moisture fluctuations within the subsurface soils.

Based upon the test boring results coupled with procedures outlined in the International Building Code (IBC), the site subsurface profile will typically coincide with Site Class C or D, depending upon the specific proposed building location. These site classes assume that foundation-bearing conditions are prepared in accordance with the Recommendations-Construction section to follow and further assume the proposed structures have shallow-depth spread footings.

Pavement Design

A minimum subgrade CBR value of 5 should be used for pavement design in order to account for fill sections yet to be constructed and areas with high natural moisture contents, especially near the proposed cut/fill lines. Cumberland Geotechnical Consultants, Inc. can provide structural thickness designs, or design reviews, of proposed pavement sections if requested. Due to the potential expansive nature of some of the onsite clay, we recommend a minimum asphalt pavement thickness of four (4) inches, regardless of how light the anticipated traffic loading may be.

Sinkholes

As previously discussed, the project site should be considered to have karst terrain potential, including sinkholes, caves, subsurface voids and deep-seated soft soil zones. This condition is common in the Cumberland Valley, and it should not disqualify the project site from potential development. Nonetheless, appropriate precautions to prevent sinkhole development should be taken.

As protection against future sinkhole development, all stormwater collection from the proposed structures and pavements (roof drains, downspouts, etc.) should be close-piped and discharged to appropriate storm sewers leading to the stormwater collection system as approved by the site engineer. **All storm water collection and transmission piping should be designed with flexible watertight connections and/or collars to prevent the infiltration of water into the ground. Consideration should be given to using pressure-rated piping for below-grade roof water collection conduits.**

To discourage future sinkhole development under the proposed buildings, no water should be impounded (i.e. detention basins) within 20 feet of a proposed building footprint area. The stormwater drainage system for the site should be installed as soon as possible during the appropriate phases of the construction. The site will be most vulnerable to sinkhole development during construction. We anticipate that the majority of sinkholes could be addressed by typical excavation and backfill procedures. These procedures involve excavation to the bedrock surface, or the maximum depth limits of a large hydraulic excavator (trackhoe), followed by backfill with rock or concrete and/or well-compacted low-permeability soils. Unusual circumstances involving sinkholes may dictate the use of grouting or structural underpinning. Cumberland Geotechnical Consultants, Inc. will be available to provide supplemental recommendations specific to any sinkholes that may be disclosed during construction.

Stormwater Controls

In our opinion, subsurface stormwater infiltration systems are not compatible with the subsurface features at the site of the proposed Center at Rocky Meadows. Large diameter perforated pipes with aggressive infiltration rates will promote sinkhole formation. In order to diminish the potential for sinkhole development within proposed stormwater management facilities, flow velocities and head pressures should be maintained at levels that are as low as reasonably achievable. Sinkhole formation along the rock/soil overburden interface is common and exposed rock along the bottom of stormwater management facilities should be discouraged. As it will be difficult to predict the precise location of bedrock below any such facilities, consideration should be given to providing clay liners within stormwater management holding basins and/or infiltration basins. If credit for infiltration were taken, the design would need to accommodate very slow infiltration rates through any clay lining system. Infiltration rates of any clay lining should be somewhat slower than infiltration rates through undisturbed site soils. Rock excavation below the plan bottom-of-basin levels will be necessary to provide clearance for any such clay liners.

The total thickness of any clay linings should be a minimum of 24 inches (including 18 inches of clay and 6 inches of topsoil cover), placed in maximum 6-inch-thickness lifts, and could consist of select onsite clay soils that classify as “CL” or “CH” soils according to the Unified Soil Classification System. Onsite clay should be free of rock fragments larger than 3/8-inch in size and compacted to minimum of 95% Standard (ASTM D698) Proctor density at a moisture content from 2 to 5 percent wet of optimum. A padfoot roller with minimum 7”-long cleats (such as a Caterpillar 815 Compactor) should be utilized for all clay liner construction in order to knit individual lifts together. The finished surface should be completed with a smooth drum roller and covered with a minimum 6-inch-thick layer of topsoil to prevent desiccation cracking of the soil liner material.

Contingencies

Contingencies should be provided for potential cost overruns due to unexpected subsurface conditions for all foundation and earthwork contracts. Additional construction costs exceeding the base bid by over 20 percent are considered typical for contracts involving natural conditions. Sinkholes could occur during construction that would require immediate attention. Rock excavation beyond base bid quantities for foundations and utilities could occur if contracts specify classified excavation. In order to incorporate a fair price structure into a competitive base bid, unit price contracts should be considered for excavation and backfill of sinkholes and/or soft subgrades. The use of appropriate equipment such as a Caterpillar E325 Excavator, a triaxle dump truck and a vibratory roller should be anticipated for this purpose. Unit prices should be available for at least the following items:

- Soil Excavation (cubic yard)
- Boulder, Rock or Concrete Obstruction Excavation (cubic yard)
- Backfill with 2,000 p.s.i. Concrete (cubic yard)
- Backfill with Run-of-Bank Riprap (ton)
- Backfill with AASHTO No. 1 Coarse Aggregate (ton)
- Backfill with AASHTO No. 57 Coarse Aggregate (ton)
- Backfill with PennDOT No. 2A Coarse Aggregate (ton)
- Backfill with Approved Offsite Borrow Material (Structural Fill)
- Backfill with Onsite Soil (cubic yard)
  (All backfill to be compacted per project specifications)
- Placement of PennDOT Class 4 Geotextile (square foot)
- Placement of Tensar BX1100 Bi-axial Geogrid

Construction Procedures

We consider the following general recommendations appropriate for the construction of the proposed Center at Rocky Meadows facilities:

Proofrolling

Proofrolling of the subgrade should be required as a prerequisite to any fill placement and should be conducted at on-grade areas and as subgrades are approached in cut areas. A minimum of six (6) complete coverages should be imparted from a loaded triaxle dump truck (or off-road equipment equivalent) having a gross vehicle weight of at least 60,000 lbs. as well as a vibratory roller meeting the minimum criteria of Required Rollers outlined in the discussions to follow. All building areas, retaining wall areas, sidewalk areas, pavement areas, etc. should be proofrolled and these regions should all include a ten-foot buffer zone beyond the actual structural area. Proofrolling should be repeated at the last practicable moment before subbase is placed on exposed subgrades and again at the subbase level before slabs, pavements or finished surfaces are placed. No additional proofrolling should be conducted behind any retaining walls. Any soft or unstable areas disclosed by the proofrolling should be repaired by using the procedures discussed below. All proofrolling should be conducted in the presence of a qualified geotechnical engineering technician.

Required Rollers

At a minimum, all compaction of bulk structural fill and proofrolling should be performed with an Ingersoll-Rand SD-150 smooth-drum vibratory roller, a Caterpillar CS-563 smooth-drum vibratory roller, a Hyster Hypac C850B smooth-drum vibratory roller, or equivalent roller with a minimum 84-inch drum width (with a static weight of at least 24,000 pounds and a centrifugal force of at least 50,000 pounds). A Caterpillar 815 Compactor or approved equivalent with minimum 7”-long pad feet should be utilized for any stormwater detention basin clay liner construction. Finish rolling in the basin should be with a smooth-drum roller.

Rock Definitions

If contracts for rock removal are performed on a classified basis, we recommend the following definitions for rock removal:

a. Trench rock in footing trenches or utility excavations should be defined as material which cannot be removed with a hydraulic excavator (trackhoe) with an operating weight of no less than 60,000 pounds, an arm force of no less than 22,000 pounds and a bucket tangential force of no less than 35,000 pounds. A Caterpillar E325 or a John Deere 270LC should meet these minimum requirements. In the event that such an excavator removes natural rock material with a volume exceeding three-quarters (¾) of a cubic yard after excavation, such material should be considered rock.

b. All rock removal that is not conducted in a trench should be considered bulk rock removal. Bulk rock in open cut areas should be defined as material that cannot be excavated with the ripper of a track-type tractor (bulldozer) with an operating weight of at least 80,000 pounds. A Caterpillar D-8 Dozer or a Komatsu D155AX Dozer should meet these minimum requirements. In the event that such a dozer removes natural rock material with a volume exceeding one-and-one-half (1½) cubic yards after excavation, such material should be considered rock.

Blasting

Blasting should be permissible for this project only under the following conditions:

a. Blasting should be done only with the permission of the local fire marshal and in accordance with all pertaining laws and regulations of the Carlisle Borough, Dickinson Township, South Middleton Township and the Commonwealth of Pennsylvania Department of Environmental Protection (DEP). A permit from the DEP will be necessary.

b. Blasting should be utilized on a controlled basis with appropriate drilling depths and charges necessary to prevent excessive "overshooting".

c. Under no circumstances should blasting be permitted within 50 feet of any new construction, existing building, home or other structure, swimming pool, railroad, roadway or existing utility. Blasting in close proximity to existing construction (but greater than 50 feet away) should incorporate appropriately scaled-down charges. This recommendation does not relieve the contractor of his responsibility to follow more stringent regulations as may be required by any party concerned with the blasting operations.

d. Where blasting must be utilized for rock removal in footing subgrade areas, all pre-drilling to set charges should terminate an appropriate distance above the proposed depth of rock removal to avoid "overshooting".

e. Blasting should be monitored with all appropriate geophysical instrumentation at the existing property lines.

f. The contractor must conduct pre-blast and post-blast surveys and present a written report to the Construction Manager and the Geotechnical Engineer demonstrating that no damage has occurred to the surrounding utilities, railroads, bridges or structures, homes, swimming pools, etc. The contractor shall be responsible for the protection of all such existing facilities during any limited blasting.

Rock Fill Criteria, Procedure and Compaction

Excavated and shot rock fragments not exceeding 36 inches in any dimension may be used as structural fill materials; however, they should not be used to construct any proposed pond berms or within four (4) feet of the final grade level within proposed building areas (plus a 10-foot buffer zone) or within 24 inches of any other final grade level. The top zone of fill embankments should be reserved for compacted soil fill to the greatest reasonable extent (see detailed discussions above). The following placement criteria for rock fill materials apply:

1. Rock materials should be choked off with sufficient soil fines and/or intermittent layers of soil fill from the onsite sources.

2. The rock borrow material (from “blast zones”) should be as well-graded as possible and spread in loose lift thicknesses of 12 to 36 inches corresponding to the approximate largest rock fragment size currently being placed as fill material. Larger rock fragments should be separated from the lift and discarded or mechanically broken (via pneumatic hammer, headache ball, onsite crushing plant, etc.) into smaller pieces not exceeding 24 inches in size prior to being placed as fill.

3. Compaction of each lift of rock should be accomplished by repeated passes of heavy track-mounted and rubber tired equipment supplemented by a vibratory roller with a minimum 84-inch drum width specified under "Required Rollers" above in tandem with a 25-ton pad-foot compactor (i.e. Caterpillar 815). The pattern of rolling should consist of a minimum of eight complete passes of each compaction machine.

4. All placement and compaction of rock material as fill should be conducted under the full-time resident observation of a qualified geotechnical engineering technician to review compaction and blending of the materials, and the construction of a dense, reasonably well graded and nonyielding rock mass.

5. Where in the opinion of the onsite Geotechical Engineering Technician rock borrow is poorly graded and does not contain sufficient rock fines, the finish rock fill surface should be thoroughly choked off with two four-inch-thick layers of fine rock fragments or processed AASHTO No. 1 Coarse Aggregate followed by PennDOT No. 2A or 2RC Coarse Aggregates (or onsite alternate material approved by the Geotechnical Engineer) prior to placing soil fill materials within the upper fill zones.

6. An oversize rock fragment (boulder) disposal area could be designated in a nonstructural site area (i.e. landscaped area) if desired. In this manner, presumably suitable material could be excavated from the designated area for use as fill material, and the resulting excavation could be backfilled with the unsuitable oversize rock fragments and unsuitable soils to save the cost of wasting such materials offsite.

7. Rockfill should not be reused as backfill material around walls or above utilities or other structures.

Soft Spot Repairs - Site Grading

If soft, excessively wet, organic, or otherwise unstable soil is disclosed by the proofrolling, all undesirable soil should be removed until a stable base is reached. The unstable, or unsuitable soil, should then be wasted off-site or at onsite nonstructural locations approved by the Owner and Civil Engineer. The resulting excavation should then be backfilled with quality-controlled aggregate that satisfies the gradations for AASHTO No. 1 (Old PennDOT No. 4) Coarse Aggregate and PennDOT No. 2A Coarse Aggregate specified by Section 703.2 in Publication 408 of the Commonwealth of Pennsylvania Department of Transportation. The PennDOT No. 2A material should be compacted in accordance with the recommendations for fill compaction below. Use of AASHTO No. 1 aggregate should be considered necessary only if the excavation penetrates a perched groundwater table or if a stable bearing area is not reached within three feet of the proposed floor slab or pavement subgrade level. If approved by a resident geotechnical engineering technician, onsite rock fill materials may be substituted for the PennDOT and AASHTO materials. A Class 4 geotextile as specified in PennDOT Specifications, Publication 408, Section 735 or a geogrid such as Tensar BX1100 could be used at the minus three-foot level and between successive lifts of aggregate backfill if additional stability and subgrade separation is considered necessary. Care should be taken not to place geotextiles or geogrids in floor slab areas scheduled for footings or utility installations. If backfill sections become unusually thick, well-compacted general structural fill may be used on top of the compacted PennDOT No. 2A Coarse Aggregate.

Backfill Behind Retaining Structures

No large excavation equipment or rollers should be utilized directly against an earth retaining structure. An imaginary line should be established from the bottom of the wall and extending upward on a one-to-one slope. Only hand-operated walk-behind rollers, or Wackers, should be used for compacting soil between this line and the wall. (A Wacker Model DPU 6055 vibratory plate tamper, Bomag BW-75AD roller or equivalent is recommended for this purpose.) Backfill materials between this line and the wall should be pushed or dumped into place with a hoe bucket. No front-end loaders, dump trucks, concrete trucks or any other large equipment should be permitted between this imaginary line and any earth-retaining structure.

Suitable Onsite Soil Fill and Backfill Materials

All bulk soil fill within structural fill zones including the building footprint and proposed pavement areas should consist of inorganic on-site soils which do not contain rock fragments which are retained on a six (6)-inch mesh screen and are of suitable moisture content, or imported off-site borrow materials to the best extent possible. All backfill within structural fill zones should meet these requirements as well; however, the maximum particle size should be limited to four (4) inches.

Suitable Offsite Borrow Materials

Materials originating from offsite sources of borrow required to balance onsite deficiencies should consist of inorganic and well-graded soils. Offsite borrow soils should meet the following gradation parameters:

- 100 percent of the soil particles should pass a 4-inch mesh screen
- At least 85 percent of the borrow soil particles should pass a 2-inch mesh sieve
- 5 to 45 percent of the borrow soil particles should pass the No. 200 mesh sieve

Liquid limits for offsite borrow soils should not exceed 40 and plasticity indices should not exceed 20. Borrow soils should classify as SC, SM, SW, GC, GM or GW according to ASTM D2487. Note that some clean and friable highly weathered shale materials (not a "bony shale or slate") could meet these requirements. Alternatively, PennDOT No. 2RC or No. 2A Coarse Aggregates would be ideal for use as backfill materials. No slag should be used.

Fill/Backfill Zone Definitions

Structural Fill Zone – Any area scheduled for a proposed building, retaining wall, courtyard, roadway, parking lot, sidewalk, other paved area or stormwater management basin plus a perimeter buffer zone of roughly 10 feet. Structural Fill Zones should include exterior perimeter wall backfill zones. Utility trenches in any location should be considered Structural Fill Zones.

Nonstructural Fill Zone – Any landscaped area for grass trees or shrubs that is not already included in the 10-foot buffer zone for a Structural Fill Zone.

Soil Compaction and Moisture Requirements

All Structural Fill Zone materials should be compacted to at least 100 percent of the Standard Compaction Test maximum dry density, ASTM Designation D 698. All Nonstructural Fill Zone materials should be compacted to at least 92% Standard Proctor density. All fill and backfill materials should be placed in eight-inch (maximum uncompacted thickness) lifts at a moisture content which is no more than 3.0 percentage points above, or 3.0 percentage points below, the optimum moisture content established for the material by ASTM Designation D 698. All fill and backfill placement should be closely observed by a qualified geotechnical engineering technician. No fill or backfill should be placed on frozen ground or during exceptionally wet periods of inclement weather. It should also be noted that the site clay soils are sensitive to changes in moisture content and will be difficult to compact outside of the required moisture control range in the wetter seasons of the year when moisture conditioning is especially difficult.

Foundation Construction Sequence

For protection against sinkholes, freezing and wet weather, all footings should be constructed on a "prep and pour" basis whereby no more footing trenches should be opened up than the amount that can be poured on the same work day.

Preparation of Foundation Subgrades

When the bottom-of-footing level is reached by the excavation equipment and the exposed subgrade is determined to be stable, all loose or disturbed material which has fallen into the excavation or has been churned up by the excavation machinery (i.e. between backhoe teeth), should be removed by hand (with a shovel) until undisturbed material is found across the entire trench bottom. Onsite residual clay should not be mechanically tamped back into place (it must be shoveled out); however, sandy materials or gravel should be tamped with a “Wacker” or equivalent. Where excavations below plan grade are necessary to achieve compaction, the resulting excavations should be backfilled with PennDOT No. 2A Coarse Aggregate as discussed below.

Soft Spot Repairs in Foundation Trenches

If soft and unstable soil is encountered at, or detected below, the proposed footing subgrade level, it should be overexcavated until suitable bearing conditions are reached and replaced with well-compacted AASHTO No. 1 and PennDOT No. 2A Coarse Aggregates as described above. If suitable bearing conditions are not encountered within a distance of three (3) footing widths or ten (10) feet of the plan footing subgrade level (whichever come first), Cumberland Geotechnical Consultants, Inc. should be contacted to make supplementary recommendations. All backfill necessary to re-establish the design footing subgrade level should be compacted in accordance with the recommendations for structural fill and backfill above. We recommend any excavation and backfill below the plan footing subgrade level should be performed on a unit price basis at the owner's expense; however, the Architect and Owner may specify unclassified foundation excavation if they so desire.

Rock in Foundation Trenches or Below Floor Slabs

When rock is encountered above, at or within 12 inches of the design foundation subgrade level, it should be removed to a level of 12 or more inches below the plan foundation subgrade level. The resulting excavation should be backfilled with onsite clay soils compacted in accordance with the recommendations for structural fill above. Similarly, when rock is encountered at or within 6 inches of the design slab-on-grade subgrade level, it should be removed to a level of 6 or more inches below the plan floor slab subgrade level. The resulting excavation should be backfilled with onsite soils compacted in accordance with the recommendations for compaction of structural fill.

Dewatering and Drainage

Strict attention should be directed to providing adequate site drainage away from the structure during the construction period and permanently to further decrease the risk of sinkhole development. Protection of subgrades during construction will also be of utmost importance. Accordingly, preventive measures for the total control of water during construction should be outlined in a section of special requirements within the project specifications. The importance of these special requirements cannot be overemphasized because of the inherent vulnerability to sinkhole development that exists in the area, particularly during construction. Only a minimum amount of trench excavation should be permitted to be open at any one time. The maximum amount of excavation should be defined as the quantity of concrete that can be poured the same day. Small dikes should be constructed around any excavations to positively guarantee against any surface waters from entering them and no impoundments of water behind walls or in any trenches should be permitted. All puddles should be removed immediately by pumping or bucketing. All open trenches and excavations at the site should be backfilled as soon as possible. The stormwater drainage system for the proposed structure should be installed as soon as possible during the appropriate phases of the construction.

Sinkholes

Any sinkholes encountered, during or after construction, should immediately be reported to Cumberland Geotechnical Consultants, Inc. for appropriate recommendations.

Foundation Subgrade Soil Testing

All foundation subgrades should be reviewed by a qualified geotechnical engineering technician. All subgrades should be visually examined to determine the type and extent of the bearing stratum. The subgrades should also be reviewed for the presence of water or unacceptable bearing materials. The foundation subgrade materials should be tested with a penetrometer, drop-hammer device, nuclear density gauge or other appropriate testing equipment to confirm conditions consistent with those implied by the design foundation soil bearing pressure. Any soil below the foundation subgrade level should be further evaluated by probing to a depth of 30 inches or more with a steel pin or drop-hammer apparatus on a maximum spacing interval of 20 feet or at each individual foundation. If any softening trends are encountered, the exact depth and lateral extent of the soft zones should be defined by additional probing. Any soft zones encountered should be repaired in accordance with the recommendations above. All foundation excavation should be observed on a resident basis by a geotechnical engineering technician under contract to the owner.

Quality Control

Qualified geotechnical engineering observations and testing should be conducted on a full-time basis during all phases of the site preparation, foundation construction and pavement construction work to ensure its proper execution. Proofrolling of all subgrades should be witnessed and approved, all foundation subgrades should be approved before pouring foundations, and each lift of fill and backfill should be observed and tested on a layer-by-layer basis to ensure that the recommended degree of compaction is obtained and that the material is placed within the proper moisture content range. In-place moisture/density tests should be taken at a minimum rate of one test for each 50 lineal feet of wall or utility trench backfill for each lift of backfill material placed. Furthermore, all required overexcavation and backfill of localized soft soil zones (such as within two feet of the existing ground surface) should be as directed by a qualified geotechnical engineering technician. All construction of pavements and stormwater management facilities should be observed by the geotechnical engineering technician as well.

GENERAL TERMS AND CONDITIONS

This report has been prepared for the exclusive use of E P CarlisleI, LLC and their design team for specific application to the development of the Center at Rocky Meadows in the Borough of Carlisle, Dickinson Township and South Middleton Township, Cumberland County, Pennsylvania and in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

The document in Appendix 5 should be considered an integral portion of this report. Appendix 5 is intended to aid our client in the proper interpretation of this report. A supplemental rock profile study should be conducted. The conclusions and recommendations contained in this report shall not be considered valid unless design changes are reviewed and the conclusions and recommendations of this report are modified or verified in writing. The conclusions and recommendations submitted in this report are based in part upon the data obtained from 12 test borings and physical parameter laboratory tests conducted on 20 samples. The nature and extent of variations between the borings will not become evident until additional borings are conducted or until construction begins. Additional studies should be conducted for specific proposed buildings. Cumberland Geotechnical Consultants, Inc. will be available to conduct supplemental programs and to re-evaluate our current information and studies as specific designs progress.