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PURPOSE AND SCOPE OF STUDY

This report presents the results of an subsurface exploration program and laboratory testing performed by GROUND Engineering Consultants, Inc. (GROUND) to provide geotechnical recommendations for design and construction of the proposed University of Colorado at Boulder Williams Village Phase II Pod "C" Residence Hall project, in Boulder, Colorado. Our services were performed in general accordance with GROUND’s Proposal No. 0901-0133 dated February 2, 2009.

A field exploration program was conducted to obtain information on subsurface conditions. Material samples obtained during the subsurface exploration were tested in the laboratory to provide data on the classification and engineering characteristics of the on-site soils. The results of the field exploration and laboratory testing are presented herein.

This report has been prepared to summarize the data obtained and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to the construction of the proposed residence hall project are included.

DOCUMENT REVIEW

In addition to the data obtained from GROUND’s subsurface exploration and laboratory testing programs, GROUND has reviewed the following document that was provided to us by University of Colorado at Boulder:

- CTL/Thompson, Inc. report, titled, Geotechnical Investigation, Williams Village, Student Housing Project Phase I, 30th Street and Baseline Road, Boulder, Colorado, Job No. 34,469, prepared for American Campus Communities, dated March 27, 2002.

PROPOSED CONSTRUCTION

We understand that the proposed project will consist of the following:
Construction of an approximately 128,000-gross square foot, 5-story residence hall building without a below-grade (basement) level.

Construction of approximately 1,000 linear feet of paved access road.

Construction of utilities, retaining structures, exterior flatwork, and landscaping.

If the proposed construction differs significantly from that described above, GROUND should be notified to re-evaluate the recommendations contained herein.

SITE CONDITIONS

At the time of our exploration, the project site existed as paved and aggregate surfaced parking lots and private driveways. The general topography of the site was flat to gently sloping. The high point of the site was near the southwest corner of the property, gentle slopes descended down northerly, easterly, and northeasterly. Based on visual observation made at the time our field services, the approximate elevation difference across the building footprint was less than 5 feet. A Google Earth aerial image of the site is shown in the photo at right.

SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted on March 21 and 23, 2009. A total of thirteen (13) test holes were drilled. Of the thirteen test holes:

- Eight (8) test holes were drilled within the general footprint of the proposed residence hall building footprint to depths of 38 to 60 feet below the existing grades.
Five (5) test holes were advanced to depths of 5 to 10 feet within the proposed paved access road.

The test holes were drilled with truck-mounted, continuous flight, power auger rigs to evaluate subsurface conditions, including depths to groundwater and bedrock (where encountered), as well as to retrieve samples for laboratory testing and analysis. GROUND engineers directed subsurface exploration programs, logged the test holes in the field, and prepared the samples for transport to our laboratory.

Relatively undisturbed samples of the subsurface materials were taken with a 2-inch I.D. "California"-type liner sampler. The sampler was driven into the substrata with blows from a 140-pound hammer falling 30 inches. This procedure is similar to the Standard Penetration Test described by ASTM Method D1586. Penetration resistance values (blows per distance driven, typically 12 inches), when properly evaluated, indicate the relative density or consistency of soils and bedrock. A composite disturbed (bulk) sample of the shallow soils in the pavement areas was collected from the auger returns. Depths at which the samples were taken and associated penetration resistance values are shown on the test hole logs.

The approximate locations of the test holes are shown in Figure 1. Logs of the exploratory test holes are presented in Figures 2 through 4. Explanatory notes and a legend are provided in Figure 5.

LABORATORY TESTING

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil samples obtained from the subject site included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, liquid and plastic limits. Water-soluble sulfate content, corrosivity, swell-consolidation, soil suction, and unconfined compressive strength tests were performed on selected samples, as well. Compaction and resilient modulus tests were performed on a composite bulk sample obtained from the pavement test hole auger cuttings. Laboratory tests were performed in general accordance with applicable ASTM and AASHTO protocols.
Data from the laboratory-testing program are summarized on Table 1. Compaction test results are presented in Figure 6. Resilient modulus test result is presented in the *Pavement Sections* section of this report.

**SUBSURFACE CONDITIONS**

Test Holes 1 through 8, P1, and P2 were advanced through a layer of paved surface consisting of approximately 2 to 6 inches of asphalt (recently chip sealed) over approximately 0 to 3 inches of aggregate base type material. Test Holes P3 though P5 were advanced through approximately 4 inches of aggregate base type material.

The subsurface conditions encountered in the test holes generally consisted of overburden sands and clays with gravels that extended to depths of approximately 8 to 30 feet below the existing grades.

The upper 1 to 3 feet of the overburden sands and clays with gravels were interpreted to be man-made fill materials in several of the test holes. The exact extents, limits, and composition of any man-made fill were not determined as part of the scope of service addressed by this study, and should be expected to exist at varying depths and locations across the site.

Weathered claystone was encountered beneath the overburden sands and clays with gravels at depths of approximately 8 to 30 feet below the existing grades and extended to 17 to 39 feet below the existing grades.

Claystone bedrock was encountered beneath the weathered claystone at depths of approximately 17 to 39 feet below the existing grades and extended to test hole termination depths of 38 to 60 feet below the existing grades.

*Fill* materials encountered consisted of sandy clays with gravels. They were fine to coarse grained with gravels, low to moderately plastic, loose to compact, slightly moist to moist, and red-brown in color.

*Sands and Clays* encountered consisted of sandy clays with gravels and clayey sands with gravels. They were fine to coarse grained with gravels, low to moderately plastic, stiff/loose to very stiff/medium dense, moist to wet, and red-brown to gray-brown in color with occasional iron staining.
Weathered Claystone was fine grained, highly plastic, stiff to very stiff, moist to wet, and gray to gray-brown in color with iron staining.

Claystone Bedrock was fine grained, highly plastic, very hard, moist, and gray to gray-brown in color with iron staining.

Groundwater was encountered at depths of approximately 8 to 10 feet below the existing grades at the time of (March 2009) GROUND’s subsurface exploration. Water levels can vary, however, in response to annual and longer-term cycles of precipitation, irrigation, drainage, and other factors.

Swell-Consolidation Testing suggested a low potential for swell in the tested on-site materials. Percent swell values and surcharge pressures at which the swell/consolidation tests were performed are presented on Table 1.

SEISMIC CLASSIFICATION

We consider the site to fall within the parameters of a Seismic Site Class D site, in accordance with 2003/2006 IBC, based on extrapolation of available data to depth. Compared with other regions of Colorado, recorded earthquake frequency in the project area is moderate. If a quantitative assessment of the classification is needed, shear wave velocity testing will be required. A proposal for this additional service can be provided upon request.

FOUNDATION AND FLOOR SLAB SYSTEM OVERVIEW

At the time of report preparation finished floor of the building and building loads were not known. We assume building finished floor will be at or above the existing grades and the building will have no below grade (basement) level. We anticipate building loads will be moderate, on the order of 250 kips.

Based on the results of the laboratory testing and the subsurface conditions encountered, the proposed building could be supported on several different foundation and floor system types.

Common foundation systems used locally that appear feasible include drilled pier, driven pile, screw pier, spread footing, and reinforced mat. Recommendations for these foundation systems are provided in this report.
Floor systems used locally that appear feasible include slab-on-grade and structural floor systems. Recommendations for these floor systems are provided in this report.

DEEP FOUNDATION SYSTEMS

Drilled Pier Foundations

The design criteria presented below should be considered during drill pier foundation system design. The construction details and other considerations presented in this report should also be considered when preparing project documents.

If the measures outlined in this report are implemented effectively, the likelihood of acceptable building performance will be within the local industry standards (estimated to be less than ½-inch) for buildings supported on drilled pier foundation systems constructed on soils and bedrock of this nature. Differential movements likely will be of similar magnitude across distances of approximately 100 feet.

1. Piers should penetrate at least 8 feet or 2 pier diameters, which ever is greater, into competent bedrock and should have a minimum length of 25 feet. Both criteria for minimum bedrock penetration and minimum pier length should be met.

2. Based on the subsurface conditions encountered it will be necessary to advance piers to depths of 25 to 47 feet below existing grades to meet the above-stated geotechnical criteria. Moreover, because of the variable bedrock depth encountered at the site, the drilled pier Contractor should be prepared to drill piers at least an additional 5 feet.

The actual pier lengths may be longer, based on the design loads, grade changes, the requirement for minimum dead load pressure, etc., as determined by the Structural Engineer, and the actual conditions encountered in the field at each pier location during installation.

3. Lenses or layers of severely weathered, soft or loose material or lignite may be identified within the bedrock during drilled pier excavations. Where materials
not suitable for foundation support are identified, it will be necessary to deepen individual piers.

4. Locally the native overburden soils and some bedrock layers may be vulnerable to caving during the drilling process and casing may be required to complete the pier holes. Groundwater can increase the potential for caving. Cased zones should not be included in load calculations and the lengths of individual piers should be increased correspondingly.

5. Piers bearing in bedrock may be designed for an allowable end bearing pressure of 30,000 psf. The portion of the pier penetrating bedrock may be designed for an allowable skin friction of 3,000 psf. This allowable skin friction value is applicable to provide bearing support and resist uplift.

6. A minimum pier diameter of 18 inches is recommended to facilitate proper cleaning and observation of the pier hole. We suggest that the pier length to diameter ratio of 25 (length) to 1 (diameter) be maintained for constructability purposes based on our experience. However, the Structural Engineer should determine the actual length to diameter ratio.

7. Groups of relatively closely spaced piers placed to support concentrated loads will require an appropriate reduction of the estimated capacities.

Reduction of axial capacity can be avoided by spacing piers to a distance of at least 3 'diameters' center to center. At this spacing or greater, no reduction in axial capacities or horizontal soil modulus values is required. Pier groups spaced less than 3 diameters center to center should be studied on an individual basis to determine the appropriate axial capacity reduction(s).

In-line arrays of drilled piers, however, must be spaced at least 6 diameters apart, center to center, to avoid reductions in lateral capacity when loaded in line with the array (parallel to the line connecting the pier centers). Linear arrays of piers spaced more closely than 6 diameters center to center should be studied to determine the appropriate lateral capacity reduction(s).

8. Bedrock penetration in pier holes should be roughened artificially to assist the development of peripheral shear between the pier and bedrock. Artificially
roughening of pier holes should consist of installing shear rings 3 inches high and 2 inches deep in the lowest 8 feet of each hole. The shear rings should be installed 18 inches on centers.

The specifications should allow the Geotechnical Engineer to waive the requirement for shear rings depending on the conditions actually encountered in individual pier holes, however.

9. The parameters tabulated below may be used for analysis of drilled piers regarding their response to lateral loads using "L-Pile" or other programs using similar input parameters. The parameters were developed based on the field and laboratory data obtained for the subject site and GROUND’s experience with similar sites and conditions.

Soil parameters required for lateral load capacity analysis, including unit weights (γ), angles of internal friction (ϕ), undrained shear strengths (Cu), strains at 50 percent of maximum stress (ε50), and modulus of horizontal subgrade reaction (k) are summarized on the following table. Resistance to lateral loads in the upper five (5) feet of overburden soils and in all undocumented fill materials should be neglected. GROUND is available upon request to perform group lateral and axial load analyses.

<table>
<thead>
<tr>
<th>Material</th>
<th>γ</th>
<th>γe</th>
<th>ϕ</th>
<th>Cu</th>
<th>ε50</th>
<th>k</th>
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<tr>
<td>Overburden Soils and Weathered Claystone -</td>
<td>0.075</td>
<td>-</td>
<td>-</td>
<td>10</td>
<td>0.01</td>
<td>200</td>
</tr>
<tr>
<td>Above Groundwater</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overburden Soils and Weathered Claystone -</td>
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<td>0.038</td>
<td>-</td>
<td>5</td>
<td>0.01</td>
<td>200</td>
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<tr>
<td>Submerged</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bedrock - Submerged</td>
<td>-</td>
<td>0.039</td>
<td>-</td>
<td>15</td>
<td>0.005</td>
<td>2,000</td>
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10. Shallow groundwater was encountered at 8 to 10 feet below the existing grades at the time of subsurface exploration program. Therefore, the Contractor should be prepared to complete the piers in the presence of groundwater, including the use of casing.
The soils and bedrock below the water table may yield significant volumes of water when penetrated by the pier excavations. Seating of the casing in the upper layers of the bedrock may not create positive cutoff of water infiltration. In the event that casing is seated into the bedrock, the minimum bedrock penetration should be taken from the bottom of the casing.

11. In no case, should concrete be placed in more than 3 inches of water, unless placed through an approved tremie method.

12. Pier holes should be properly cleaned prior to placement of concrete.

13. Where water or slurry is present in the drilled pier hole, including outside of a casing that will be withdrawn from the hole, the concrete placed for the pier should have sufficient slump and be placed with sufficient head maintained above groundwater levels so that the concrete is not displaced in the body of the pier by water, soil, slurry, etc., leading to effective voids in the pier. Slurry, if used, must be fully displaced by the concrete.

14. Concrete utilized in the piers should be a fluid mix with sufficient slump so that it will fill the void between reinforcing steel and the pier hole wall. We recommend the concrete have a minimum slump in the range of 5 to 7 inches. Concrete should be placed by an approved tremie or other method to reduce mix segregation.

15. Concrete should be placed in piers the same day they are drilled. Failure to place concrete the day of drilling will normally result in a requirement for additional bedrock penetration. The presence of groundwater or caving soils may require that concrete be placed immediately after the pier hole drilling is completed.

16. The Contractor should take care to prevent enlargement of the excavation at the tops of piers, which could result in mushrooming of the pier top. Mushrooming of pier tops can increase uplift pressures on the piers.

17. GROUND recommends that sonic integrity testing be performed for an appropriate percentage (i.e. minimum 10 percent) of the drilled piers to assess
the effectiveness of the pier construction methods for installing the piers in accordance with project plans and specifications.

18. Resistant bedrock was encountered locally in the test holes and beds and lenses of highly cemented sandstone bedrock have been encountered in the project area. Difficult to very difficult drilling conditions may be encountered during pier hole drilling. The drilling Contractor should be prepared to core lenses and beds of highly cemented materials.

19. In general, the pier-drilling Contractor should mobilize equipment of sufficient size and operating capability to achieve the required penetration into the bedrock. We suggest the pier drilling contractor advance a test hole with the proposed equipment at least to the minimum anticipated depth prior to beginning pier installation to assess whether the equipment can reach the target depths. This test drilling should be performed sufficiently early in the construction process so that more powerful equipment can be mobilized if necessary without delaying the project. If refusal is encountered in these materials either during the test program or during actual installation, the Geotechnical Engineer should evaluate the conditions to establish that true refusal has been met with adequate drilling equipment.

A Geotechnical Engineer should be retained to observe pier-drilling operations on a full time basis.

**Driven Pile Foundations**

Preliminary recommendations for driven piles are provided below. If detailed recommendations for such systems are required, GROUND should be contacted for additional information.

The pile should consist of a heavy steel H-section. The pile tip should be reinforced with a commercial, heavy duty, pile tip. Based on the strength of the bedrock deposits underlying the site, piles may be designed for a maximum pile load up to a maximum allowable service stress of 12-ksi based on the pile cross-sectional area, however this should be verified by performing pile dynamic analysis.
Relatively competent bedrock was encountered beneath the proposed building at depths ranging from about 17 to 39 feet below the existing grades. We anticipate that piles will be driven 10 or more feet into competent bedrock to reach capacity. This would correspond to pile lengths of about 27 to 49 feet below existing grades (if finished floor elevations are higher or lower than the existing grades pile lengths will have to be adjusted accordingly). Moreover, because of the variable bedrock depth encountered at the site, the pile-driving Contractor should be prepared to drive piles at least an additional 5 feet. We recommend that a program of test pile installation be performed to refine anticipated driving depths/elevations.

**Screw Pile Foundations**

As an alternate deep foundation system to drilled piers and driven piles screw piles advanced into the underlying soils and bedrock could also be utilized to support the proposed additions. We suggest that you contact one or more reputable Suppliers / Contractors such as D&B Drilling Engineering, Wheat Ridge, Colorado (303-423-6834), Rocky Mountain Steel Piering, Littleton, Colorado (303-471-1155), Alpine Site Services, Inc., Denver, Colorado (303-420-0048), regarding site specific proposals for design and installation of such systems.

Screw piles are steel shafts with helices that resemble large screws. They are screwed into the ground using specialized equipment that is generally attached to a track-mounted excavator. Torque developed during installation has been correlated to foundation capacity. Screw piles emplacement depths will vary, based on the subsurface conditions encountered but we estimate that the depth of the elements could reach 22 to 44 feet or more in depth. However, actual screw pile depth, layout and design will be developed by the Supplier / Contractor. The use of battered piles may be necessary to resist the lateral loads. Actual screw pile length will also depend on the conditions encountered during installation. Note that for cost estimation, screw piles may not necessarily replace drilled piers or driven piles on a one to one basis. Two or more screw piles may be needed to replace each drilled pier or driven pile.
SHALLOW FOUNDATION SYSTEMS

Spread Footing Foundations

We estimate that potential post-construction movements, if the recommendations in this report are implemented effectively, to be approximately 1 inch. Differential movements likely will be of similar magnitude across distances of approximately 100 feet.

The remedial earthworks and related measures outlined below will not eliminate the potential for future settlement, but if implemented effectively and maintained, they will tend to make the movements more uniform, and reduce resultant damage to the facility if such movement occurs.

The criteria presented below may be observed for footing foundation systems. The construction details should be considered when preparing project documents.

1. Create a fill prism, consisting of imported approved compacted granular materials, with a uniform thickness of a minimum of 3-feet under the entire building main level foundation system.

2. The fill prism should extend, at full depth, at least 5 feet laterally beyond the perimeter of each footing. Specifications for imported granular fill materials, and recommendations proper placement and compaction of the fill soils are provided in the Project Earthworks section of this report.

3. The Contractor should provide surveyed elevations of the bottoms of the excavations beneath the building to the Project Team, verifying that the remedial excavation was advanced to a sufficient depth.

4. Due to shallow groundwater, soft and wet subgrade conditions may be encountered at the bottom of the excavations. In addition, firm materials may be disturbed by the excavation process or by the construction traffic. All such unsuitable materials should be stabilized prior to fill placement. The recommendations for possible stabilization techniques that may be used are provided in the Project Earthworks section of this report.

5. Footings bearing on 3 or more feet of properly compacted, uniform thickness granular fill, as described above, may be designed for an allowable soil bearing
pressure (Q) of 3,000 psf under drained conditions. This value may be increased by \( \frac{1}{3} \) for transient loads such as wind or seismic loading.

6. The allowable bearing capacity provided above is based on the assumption of well-drained conditions. If foundation soils become wet after the construction is complete, the effective bearing capacity could be reduced and the potential for settlement will be increased. For other estimated settlements, allowable bearing pressure values can be obtained from Figure 7.

7. Rammed aggregate piers could be utilized to increase the bearing capacity under the building footings, and reduce resultant footing sizes. If rammed aggregate piers are considered they should only be designed and installed by the specialty contractor. Additional recommendations for such systems can be provided upon request.

8. In order to reduce differential settlements between footings or along continuous footings, footing loads should be as uniform as possible. Differentially loaded footings will settle differentially.

9. Footings should have a minimum footing dimension of 16 inches for strip spread footings and 24 inches for isolated pad footings. Actual footing dimensions, however, should be determined by the Structural Engineer, based on the design loads.

10. Footings should be provided with adequate soil cover above their bearing elevation for frost protection. Footings should be placed at a bearing elevation 3 or more feet below the lowest adjacent exterior finish grades in un-heated areas.

11. Continuous foundation walls should be reinforced top and bottom to span an unsupported length of at least 10 feet.

12. The lateral resistance of spread footings will be developed as sliding resistance of the footing bottoms on the foundation materials. Sliding friction at the bottom of footings bearing on approved imported granular fill may be taken as 0.45 times the vertical dead load. An allowable passive soil pressure of 330 psf per foot of embedment may be used, to a maximum of 3,300 psf. The upper 1-foot of embedment should not be relied upon for passive resistance, however.
13. Compacted fill placed against the sides of the footings should be compacted to at least 95 percent relative compaction in accordance with the recommendations in the *Project Earthworks* section of this report.

14. Care should be taken when excavating the foundation to avoid disturbing the supporting materials. Hand excavation or careful backhoe soil removal may be required in excavating the last few inches.

15. Foundation soils may be disturbed or deform excessively under the wheel loads of heavy construction vehicles as the excavations approach footing levels. Construction equipment should be as light as possible to limit development of this condition. The use of track-mounted vehicles is suggested because they exert lower contact pressures. The movement of vehicles over proposed foundation areas should be restricted.

16. Foundation elevations at the proposed lot appear to be only slightly above local periodic groundwater levels. Therefore, it may be necessary to de-water the footing excavations during construction. De-watering should not be conducted by pumping from inside footing excavations. This may decrease the supporting capacity of the soils.

17. All footing areas should be compacted with a vibratory plate compactor prior to placement of concrete.

A Geotechnical Engineer must be retained to observe and test all footing excavations prior to placement of reinforcing steel or concrete.

**Reinforced Mat Foundations**

The design and construction criteria presented below should be observed for a mat foundation system. The construction details should be considered when preparing project documents.

We estimate that probable post-construction movements, if the recommendations in this report are implemented and maintained effectively, to be on the order of 1 inch or less for reinforced mat foundations. Differential movements likely will be of similar magnitude across distances of approximately 100 feet. The mat foundation systems, due to its
stiffness, will move as a rigid structure and significantly reduce differential movement between the load-bearing areas and floor slabs.

1. Create a fill prism, consisting of imported approved compacted granular materials, with a uniform thickness of a minimum of 5-feet under the entire reinforced mat foundation system.

2. The fill prism should extend, at full depth, at least 5 feet laterally beyond the perimeter of reinforced mat foundation. Specifications for imported granular fill materials, and recommendations proper placement and compaction of the fill soils are provided in the Project Earthworks section of this report.

3. The Contractor should provide surveyed elevations of the bottoms of the excavations beneath the building to the Project Team, verifying that the remedial excavation was advanced to a sufficient depth.

4. Due to shallow groundwater, soft and wet subgrade conditions may be encountered at the bottom of the excavations. In addition, firm materials may be disturbed by the excavation process or by the construction traffic. All such unsuitable materials should be stabilized prior to fill placement. The recommendations for possible stabilization techniques that may be used are provided in the Project Earthworks section of this report.

5. Mat foundations bearing on 5 or more feet of properly compacted, uniform thickness granular fill, as described above, may be designed for an allowable soil bearing pressure (Q) of 2,000 psf under drained conditions. This value may be increased by $1.0$ for transient loads such as wind or seismic loading. A modulus of vertical subgrade reaction equal to 48 tcf may be utilized.

6. Rammed aggregate piers could be utilized to increase the bearing capacity under the mat foundation. If rammed aggregate piers are considered they should only be designed and installed by the specialty contractor. Additional recommendations for such systems can be provided upon request.

7. Foundation system should be adequately reinforced (top and bottom) to account for the probable differential movements outlined above.
8. Depending on the design thickness of the mat, it may take a long time for the heat generated during concrete to release due to the thickness of the mat foundation, which may cause early-age thermal cracking. Depending on the design thickness of the mat foundation system, measures should be taken when selecting the concrete that will be placed to prevent early-age thermal cracking. In addition, cooling/heating systems, and monitoring of the temperature and moisture contents within the mat foundation is recommended.

9. Lateral resistance of the mat foundation will be a combination of the sliding resistance of the pad on the foundation materials and passive earth pressure against the side of the pad. Sliding friction at the bottom of footings bearing on approved imported granular fill may be taken as 0.45 times the vertical dead load. An allowable passive soil pressure of 330 psf per foot of embedment may be used, to a maximum of 3,300 psf. The upper 1-foot of embedment should not be relied upon for passive resistance, however.

10. The reinforced mat foundations should be provided with adequate soil cover or by other means above its bearing elevation for frost protection. The depth of frost protection in the area is 3 feet.

11. Flexible connections allowing 2 inches of vertical movement should be provided for all slab-bearing mechanical equipment that is connected to the existing building. Regardless of the type of flexible connection used periodic maintenance of the flexible connections should be performed throughout the life of the structure.

12. Compacted fill placed against the sides of the mat should be compacted in accordance with the recommendations in the Project Earthworks section of this report.

13. Care should be taken when excavating the foundation to avoid disturbing the supporting materials and adjacent buried utility lines. Hand excavation or careful backhoe soil removal may be required in excavating the last few inches.

14. Foundation soils may be disturbed or deform excessively under the wheel loads of heavy construction vehicles as the excavations approach footing levels. Construction equipment should be as light as possible to limit development of
this condition. The use of track-mounted vehicles is suggested because they exert lower contact pressures. The movement of vehicles over proposed foundation areas should be restricted.

15. Foundation elevations at the proposed lot appear to be only slightly above local periodic groundwater levels. Therefore, it may be necessary to de-water the footing excavations during construction. De-watering should not be conducted by pumping from inside footing excavations. This may decrease the supporting capacity of the soils.

16. All bearing areas should be compacted with a vibratory plate compactor prior to placement of concrete.

A Geotechnical Engineer should be retained to observe and test all footing excavations prior to placement of reinforcing steel or concrete.

**FLOOR SYSTEMS**

**Slab-on-Grade Floors**

If slab-on-grade construction is used in accordance with the following criteria, potential for slab movement will not be eliminated, but the following criteria will tend to reduce the magnitude of the movements, make them more uniform, and reduce resultant damage where movement does occur.

We estimate that probable post-construction movements, if the recommendations in this report are implemented and maintained effectively, to be on the order of 1 inch or less for slab-on-grade floors.

1. Create a fill prism, consisting of imported approved compacted granular materials, with a uniform thickness of a minimum of 5-feet under the entire slab-on-grade floor.

2. The fill section should extend at full depth at least 5 feet laterally beyond the perimeter of the building. Specifications for imported granular fill materials, and recommendations proper placement and compaction of the fill soils are provided in the *Project Earthworks* section of this report.
3. The Contractor should provide surveyed elevations of the bottoms of the excavations beneath the building to the Project Team, verifying that the remedial excavation was advanced to a sufficient depth.

4. Due to shallow groundwater, soft and wet subgrade conditions may be encountered at the bottom of the excavations. In addition, firm materials may be disturbed by the excavation process or by the construction traffic. All such unsuitable materials should be stabilized prior to fill placement. The recommendations for possible stabilization techniques that may be used are provided in the Project Earthworks section of this report.

5. A Geotechnical Engineer should be retained to observe the prepared surface on which the floor slab will be cast prior to placement of reinforcement. Loose, soft or otherwise unsuitable materials should be excavated and replaced with properly compacted fill, placed in accordance with the recommendations in the Project Earthworks section of this report.

6. The floor slab(s) should be separated from all bearing walls and columns with slip joints, which allow unrestrained vertical movement.

7. Joints should be observed periodically, particularly during the first several years after construction. Slab movement can cause previously free-slipping joints to bind. Measures should be taken to assure that slab isolation is maintained in order to reduce the likelihood of damage to walls and other interior improvements, including door frames, plumbing fixtures, etc.

8. Interior partitions resting on floor slabs should be provided with slip joints so that if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards and doorframes. Slip joints, which will allow at least 2 or more inches of vertical movement, should be considered. If slip joints are placed at the tops of walls, in the event that the floor slabs move, it is likely that the wall will show signs of distress, especially where the floors and interior walls meet the exterior wall.

9. All plumbing lines should be carefully tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided. Flexible
connections allowing 2 or more inches of vertical movement or more should be provided for slab-bearing mechanical equipment.

10. Concrete slabs-on-grade should be placed on properly prepared subgrade. They should also be constructed and cured according to applicable standards and be provided with properly designed and constructed control joints. The design and construction of such joints should account for cracking as a result of shrinkage, tension, and loading; curling; as well as proposed slab use. Joint layout based on the slab design may require more frequent, additional, or deeper joints, and should also be based on the ultimate use and configuration of the slabs. Areas where slabs consist of interior corners or curves (at column blockouts or around corners) or where slabs have high length to width ratios, high degree of slopes, thickness transitions, high traffic loads, or other unique features should be carefully considered. The improper placement or construction of control joints will increase the potential for slab cracking. ACI, AASHTO, and other industry groups provide many guidelines for proper design and construction of concrete slabs on grade and the associated jointing.

11. The floor slab should be adequately reinforced. Recommendations based on structural considerations for slab thickness, jointing, and steel reinforcement in floor slabs should be developed by the Structural Engineer.

12. GROUND recommends placement of a properly compacted layer of free-draining gravel, 4 or more inches in thickness, beneath the slabs. This layer will help distribute floor slab loadings, ease construction, reduce capillary moisture rise and aid in drainage. The free-draining gravel should contain less than 5 percent material passing the No. 200 Sieve, more than 50 percent retained on the No. 4 Sieve, and a maximum particle size of 2 inches.

13. Relatively shallow groundwater was encountered at the project site. Therefore, there is a potential for water to enter the buildings or water vapor to drawn into them. Both heating and air conditioning can accelerate the movement of water vapor into the building interiors. An effective barrier beneath a slab, however, can direct moisture released from the concrete and underlying gravel upwards, into the building interior. As noted above, this can be detrimental to the slab and/or to flooring materials placed on the slab.
Placement of a relatively impermeable floor covering on a concrete slab, together with an effective barrier beneath it, causes 'encapsulation' of the slab concrete. In such cases, the vapor pressure of trapped water within the concrete commonly causes floor coverings to lose adhesion to the slab. The Manual of Concrete Practice published by the American Concrete Institute recommends use of a cementitious covering or a flooring material which is permeable to water vapor to reduce this problem.

The Architect and the Flooring Contractor should consider the use of vapor and moisture barriers and the performance of proposed flooring products in light of the intended building use, groundwater conditions and possible moisture intrusion into the building, water vapor fluxes from the slabs, as well as the potential effects of moisture encapsulation of the slab concrete.

**Structural Floors**

Preliminary recommendations for structural floors are provided below. If detailed recommendations for such systems are required, GROUND should be contacted for additional information.

Structural floors should be supported on grade beams and straight-shaft drilled piers in the same manner as the building structure. Structural floors should be constructed to span above a well-ventilated crawl space to allow access and maintenance to utility piping.

A minimum 10-mil un-reinforced polyethylene vapor retarder (sheet material) should be considered for installation below all structurally supported below-grade floors and should be properly attached/sealed to foundation walls/caissons.

**FOUNDATION WALLS**

Foundation walls which are laterally supported and can be expected to undergo only a limited amount of deflection, i.e., an "at-rest" condition, should be designed to resist lateral earth pressures computed on the basis of an equivalent fluid unit weight of 55 pcf if imported approved granular materials is used to backfill the walls. Specifications for imported granular fill materials, and recommendations proper placement and compaction of the fill soils are provided in the Project Earthworks section of this report.
The at-rest lateral earth pressures should be computed using an equivalent fluid unit weight of 73 pcf where on-site materials are used as backfill.

GROUND recommends use of structural backfill behind the walls to achieve lower lateral earth pressures. To realize the lower equivalent fluid unit weight, structural fill should be placed behind the wall to a minimum distance equal or greater than half of the wall height. Where structural backfill is used, the upper 1-foot of the wall backfill should be a relatively impermeable soil or otherwise protected to reduce surface water infiltration into the backfill.

The parameters recommended above are for well-drained conditions with a horizontal upper backfill surface. The additional loading of an upward sloping backfill, hydrostatic loads if sufficient drainage is not provided, as well as loads from traffic, stockpiled materials, etc., should be included in foundation wall design.

Backfill soils should be thoroughly mixed to achieve a uniform moisture content, placed in uniform lifts not exceeding 8 inches in loose thickness, and properly compacted in accordance with the Project Earthworks section of this report. The Contractor should take care not to over-compact the backfill soils, which could result in excessive lateral pressures on the walls.

The Geotechnical Engineer should be retained to observe the exposed excavation prior to placement of backfill, observe earthwork operations, and test the soils.

Some settlement of wall backfills will occur even where the material was placed correctly. This settlement likely will be differential, increasing with depth of fill. Where shallowly founded structures and pavements must be placed on backfilled zones, structural design, pipe connections, etc., should take into account backfill settlement, including differential movement and the associated risks are understood by the Owner.

The above parameters are not recommended for use in retaining wall design. In the event that retaining walls are added once development begins, retaining wall parameters should be requested and the Client should realize that additional subsurface exploration may be necessary.
RETAINING WALLS

Wall Foundation Recommendations

Actual retaining wall design is not part of our scope of services. A registered Professional Engineer should be retained to design proposed project retaining walls. GROUND can provide a proposal for retaining wall design upon request.

Information regarding the proposed retaining wall(s) such as type, size, etc. of the wall(s) was not known at the time this report was prepared. General wall parameters for use in design are provided below and should be confirmed as construction progresses, mainly due to any grading changes and to verify materials types and parameters.

The shallow on-site soils, exclusive of topsoil or construction debris, encountered in the test holes in the areas proposed for retaining wall construction, and similar materials placed as properly compacted fill, are suitable to support the proposed structure on a shallow foundation system. Spread footing foundation recommendations are provided under the Shallow Foundations section of this report, which can be utilized for the proposed retaining wall foundations.

A Geotechnical Engineer should be retained to observe and test the footing bearing areas prior to placement of reinforcing steel or concrete.

Lateral Pressures

Walls which are laterally supported and can be expected to undergo only a limited amount of deflection, i.e., an "at-rest" condition, should be designed to resist lateral earth pressures computed on the basis of an equivalent fluid unit weight of 55 pcf if imported, granular, structural backfill (meeting the criteria presented below) is used to backfill the walls. The at-rest lateral earth pressures should be computed using an equivalent fluid unit weight of 73 pcf where on-site materials are used as wall backfill.

We assume that cantilevered retaining structures for the project will be designed to deflect sufficiently to mobilize the full, active earth pressure condition. Therefore, cantilevered structures may be designed for an active lateral earth pressure computed on the basis of an equivalent fluid unit weight of 35 pcf for the imported structural backfill and of 51 pcf where on-site soils are used as wall backfill.
The lateral earth pressures recommended above are for well-drained conditions with a horizontal finish grades. The additional loading of a sloping backfill, hydrostatic loads if sufficient drainage is not provided, as well as loads from traffic, stockpiled materials, etc., should be included in retaining wall design.

Select, granular materials imported for use as structural backfill should meet the criteria for CDOT Class 1 Structure Backfill as tabulated in the Project Earthworks section of this report. All imported soils should be tested and approved by the Geotechnical Engineer prior to transport to the site.

GROUND recommends the use of structure backfill behind the walls to achieve lower lateral earth pressures. To realize the lower equivalent fluid unit weight, structure backfill should be placed behind the wall to a minimum distance equal or greater than half of the wall height. Where structure backfill is used, the upper 1-foot of the wall backfill should be a relatively impermeable soil or otherwise protected to reduce surface water infiltration into the wall backfill.

Multiple tier walls (if utilized) for the project should be analyzed for global stability as a single retention structure, as well as individually.

**Mechanically Stabilized Earth (MSE) Wall**

Alternatively, flexible-type retaining walls, such as MSE walls, are able to accommodate movements with minimal adverse effects to the walls. Because of the flexibility of the wall system, an MSE wall can be supported at shallow depths.

For design of the MSE wall system, an estimated angle of internal friction of 26 degrees and a moist unit weight of 130 pcf may be used for the foundation materials and the on-site retained materials behind the wall backfill. We recommend that imported granular material be used in the reinforced zone and the active zone behind the reinforcement. Design parameters for the wall backfill in the reinforced zone and active zone should be determined by the Wall Design Engineer based on the material for which the MSE wall is designed.

MSE retaining walls bearing on firm native soils or properly compacted fill may be designed for an allowable soil bearing pressure of 1,500 psf. Where wall excavation
bottoms expose soft, loose or otherwise deleterious materials, such materials should be excavated and replaced with properly compacted backfill.

The estimated soil strength, density, and bearing pressure provided above are based on the conditions encountered at the time and locations of drilling and the assumption of well-drained conditions in the wall system, and can be affected by grading, drainage, construction, or other factors. It is Wall Design Engineer's responsibility to verify the soil design parameters of the reinforced backfill, wall backfill, retained material, and foundation material, other wall materials, as well as site conditions before and during the wall construction.

**Wall Drainage**

Retaining walls should be provided with drains at the heels of the walls, or with weep holes, or both, to help reduce the development of hydrostatic loads. The drain systems should consist of perforated PVC or HDPE collection pipe at least 4 inches in diameter, non-perforated PVC or HDPE discharge pipe at least 4 inches in diameter, free-draining gravel, and filter fabric. The free-draining gravel should contain CDOT No.57/No.67 clean crushed rock with a maximum particle size of 1.5 inches. Each collection pipe should be surrounded on the sides and top only with 6 or more inches of free-draining gravel. The gravel surrounding the collection pipe should be wrapped with filter fabric to reduce the migration of fines into the drain system. Typical, cross-section details of drain systems, as recommended above, can be provided upon request. The actual layout, outlets, and locations should be designed by the Civil Engineer.

The high point(s) of the drainpipe should be placed at least 6 inches below the bottom of the footings. The trench in which the drainpipe will be constructed should be graded at a minimum slope of 1 percent to one or more discharge pipe from which water can be gravity discharged.

In addition to surrounding the drainpipes with at least 6 inches of free-draining gravel, the gravel should extend upward to within 12 inches of the backfill surface behind the wall or the wall should be backed with a layer of geocomposite drainage medium, e.g., an appropriate MiraDrain® product or equivalent. The gravel or drainage product backing the wall should be in hydraulic connection with the wall heel drain. If gravel is selected, it should be separated from the enclosing soils by a layer of filter fabric to
reduce the migration of fines into the drainage system. Damp-proofing should be applied to the back side of rigid types of retaining walls.

Wall Construction Considerations

Backfill soils should be thoroughly mixed to achieve uniform moisture content, placed in uniform lifts not exceeding 8 inches in loose thickness, and placed in accordance with the recommendations provided in the Project Earthworks section of this report. The Contractor should take care not to over-compact the backfills, which could result in excessive lateral pressures on the walls.

The Geotechnical Engineer should observe the exposed excavation prior to placement of backfill, observe earthwork operations and drain installation, and test the soils.

Some settlements of wall backfills will occur even where the material is placed correctly. This settlement will likely be differential, increasing with depth of fill. Shallowly founded structures and pavements should not be located on or adjacent to the backfilled zones. Where improvements must be placed in the backfilled zones, structural design, pipe connections, etc., should take into account backfill settlements, including differential movement and the associated risks are understood by the Owner. Deep foundations may be appropriate for supporting structures, which must be located in the backfilled zone. The Geotechnical Engineer should provide recommendations for founding of improvements in such areas.

EXTERIOR FLATWORK AND PROJECT HARDSCAPING

Proper design, drainage, construction, and maintenance of the areas between the building and parking/driveway areas are critical to the satisfactory performance of the project. Sidewalks, raised planters, and other highly visible improvements commonly are installed within these zones, and distress in or near these improvements is often observed. Routinely, proper soil preparation in these areas receives little attention during construction because they fall between the building and pavement areas, which typically are built with heavy equipment. Subsequent landscaping and hardscape installation often is performed by multiple sub-contractors with light or hand equipment, and necessary over-excavation and soil processing is not performed. Consequently, subgrade soil conditions commonly deviate significantly from recommended ranges. Therefore, GROUND recommends that the Contractor take particular care with regard
to proper subgrade preparation in the immediate building exteriors. Exterior flatwork and other hardscaping placed on the site soils encountered at the site will experience post-construction movements as soil moisture contents increase after construction. Local settlements in the native overburden soils should be anticipated, as well as in filled and backfilled areas. Distress to rigid hardscaping likely will result. The following measures will help to reduce damages to these improvements.

GROUND recommends that shortly before installation, the subgrade soils beneath project sidewalks, paved entryways and patios, masonry planters and short, decorative walls, and other hardscaping be properly moisture-conditioned and compacted fill soils. The existing soils should be excavated to a depth of 12 inches or more below the bottom of the exterior flatwork, thoroughly mixed, moisture conditioned and replaced as properly compacted fill.

Greater depths of moisture-density conditioning of the subgrade soils beyond the above minimum values may improve hardscape performance. Recommendations for placement and compaction of fill soils are provided in the Project Earthworks section of this report. For areas such as hardscaping for the main entrance or other locations where hardscape movement tolerances are low, the fill prism recommended for slab-on-grade floors (see Floor Systems section) should be extended under the hardscaping.

Prior to placement of flatwork, a proof roll should be performed to identify areas that exhibit instability and deflection. (It should be noted that passing a proof roll is an additional requirement. Soils moisture-conditioned and compacted within the ranges recommended in the Project Earthworks section of this report may not necessarily pass proof rolling.)

Flatwork should be provided with effective control joints. Narrow flatwork, such as sidewalks, may require more closely spaced joints.

In no case should exterior flatwork extend to under any portion of the building where there is less than 2 inches of clearance between the flatwork and any element of the building. Exterior flatwork in contact with brick, rock facades, or any other element of the building can cause damage to the structure if the flatwork experiences movements.

As recommended in the Project Earthworks and Surface Drainage sections of this report, positive surface drainage away from all pavements and flatwork should be
included in project design. Proper drainage also should be maintained after completion of the project, and re-established as necessary.

**Concrete Scaling**

Surface scaling of sidewalks and other exterior concrete can result from poor workmanship during construction, such as 'over-finishing' the surface. It also can result from exposure to relatively severe weather conditions with repeated freeze-thaw cycles. In GROUND's experience, if reducing the potential for freeze-thaw scaling is a design consideration, the following measures are beneficial: a) maintaining a maximum water/cement ratio of 0.45 by weight for exterior concrete, b) including Type F fly ash in the mix for exterior concrete as 20 percent of the cementitious material, and c) use of exterior concrete that exhibits a minimum compressive strength of 4,500 psi. Inclusion of 'fibermesh' in the concrete mix also may be beneficial for reducing surficial scaling. (These concrete mix design criteria should be coordinated with other project requirements including the criteria for sulfate resistance presented in the Water-Soluble Sulfates section of this report.) In addition, the use of de-icing salts on exterior concrete flatwork during the first winter after construction will increase the likelihood of the development of scaling. Placement of flatwork concrete during cold weather so that it is exposed to freeze-thaw cycling before it is fully cured also increases its vulnerability to scaling. Concrete placing during cold weather conditions should be blanketed or tented to allow full curing. Depending on the weather conditions, this may result in 3 to 4 weeks of curing, and possibly more.

**PROJECT EARTHWORKS**

At the time this report was prepared, detailed development plans, final site grading plans and construction details were not available for our review. We anticipate cuts and fills that are about 5 feet or less, respectively, in nominal depth to construct the proposed structures, streets, and parking areas.

Site grading should be planned carefully to provide positive surface drainage away from all structures, paved surfaces, utility trench alignments and flatwork. Surface diversion features should be provided during construction to direct water into the proposed water-quality swales, detention ponds, or other appropriate structures as readily as possible.
Although not shown in project plans and details provided to GROUND prior to preparation of this report, if detention ponds are proposed for construction on this project site, we recommend that all detention ponds be lined with a impervious liner.

**General Considerations**

Site grading should be performed as early as possible in the construction sequence to allow settlement of fills and surcharged ground to be realized to the greatest extent prior to building construction.

Prior to earthwork construction, existing structures, vegetation and other deleterious materials should be removed and disposed of off-site. Relic underground utilities should be abandoned in accordance with applicable regulations, removed as necessary, and capped at the margins of the property.

Areas of previously placed fill soils were noted especially on the southwestern portion of the site. These uncontrolled fills should be excavated in their entirety and, if necessary to achieve project grades, replaced with properly moisture-conditioned and compacted fill. As previously mentioned, the exact extents, limits, and composition of the man-made fill were not determined as part of the scope of work addressed by this study, however and could exist at varying depths and locations across the site.

**On-Site Soils**

The granular site soils, that are free of organic materials, large cobbles and other deleterious materials appear suitable for placement as compacted fill. Cobbles and rock fragments larger than 3 inches in maximum dimension should not be incorporated into project fills.

Care should be taken with regard to achieving and maintaining proper moisture contents during placement and compaction. In our experience, achieving and maintaining compaction in such soils can be very difficult. We anticipate that on-site soils may exhibit significant pumping, rutting, and deflection at moisture contents near optimum and above. The Contractor should be prepared to handle soils of this type, including the use of chemical stabilization, if necessary.
Nearly all soils other than relatively coarse, clean, granular materials are susceptible to loss of density if allowed to become saturated and exposed to freezing temperatures and repeated freeze-thaw cycling. The formation of ice in the underlying soils can result in heaving of pavements, flatwork and other hardscaping ("frost heave") in sustained cold weather up to 2 inches or more. This heaving can develop relatively rapidly. A portion of this movement typically is recovered when the soils thaw, but due to loss of soil density, some degree of displacement will remain. This can result even where the subgrade soils were prepared properly.

Where hardscape movements are a design concern, e.g., at doorways, replacement of the subgrade soils with 3 or more feet of clean, coarse sand or gravel should be considered or supporting the element on foundations similar to the building and spanning over a void. Detailed recommendations in this regard can be provided upon request. It should be noted that where such open graded granular soils are placed, water can infiltrate and accumulate in the subsurface relatively easily, which can lead to increased settlement or heave from factors unrelated to ice formation. The relative risks from these soil conditions should be taken into consideration where frost heave is a concern. GROUND will be available to discuss these concerns upon request.

Where soils supporting foundations or on which foundation will be placed are exposed to these conditions during construction — commonly due to water ponding in foundation excavations — bearing capacity typically is reduced and/or settlements increased due to the loss of density in the supporting soils. After periods of freezing conditions, the Contractor should re-work areas affected by the formation of ice to re-establish adequate bearing support.

The site soils include low plasticity silty sands as well as some clays that due to their capillarity appear vulnerable to frost heave where an underlying source of water is present. Groundwater was encountered only at depths of 29 feet or more, however. Therefore, if surface drainage is effective, the likelihood of movement of pavements, flatwork and other hardscaping is relatively low. However, if other source(s) of water develop and are not drained effectively, frost heave may develop.

Topsoil should not be incorporated into common fill placed on the site. Instead, topsoil should be stockpiled during initial grading operations for placement in areas to be landscaped or for other approved uses.
Soft and Wet Subgrade Conditions

The following recommendations should be considered where soft, wet, and unstable subgrade conditions are encountered:

1. Areas of weak or pumping soils should be excavated and replaced with approved granular fill. The depth of overexcavation is anticipated to be on the order of 1 to 3 feet or more to provide a stable surface. The use of recycled concrete aggregate may be a cost effective alternative in this application.

2. Recommendations for compaction provided in the *Project Earthworks* section of this report should be followed.

3. In cases where placement of approved coarse granular fill or road base does not result in stable conditions, it will be necessary to place a woven geotextile (Mirafi® HP370 or equivalent) or a geogrid (such as Tensar BX1100 or equivalent).

   a. The subgrade should be leveled prior to woven geosynthetic reinforcement placement. Very weak or pumping soils should be excavated and replaced with approved coarse granular fill or road base, prior to geosynthetic reinforcement placement for best performance. The geosynthetic reinforcement should be placed directly on the prepared subgrade. Placement should be performed according to manufacturers recommendations.

   b. Geosynthetic reinforcement rolls should be overlapped as per manufacturers recommendations.

   c. Geosynthetic reinforcement will be disturbed under the wheel loads of heavy construction vehicles, especially track type vehicles, therefore no vehicle traffic should be allowed over the geosynthetic reinforcement until 8 or more inches of soil has been placed over.

   d. For very weak subgrades an approved coarse granular fill or road base may have to be placed in 18 to 24 inch lifts, in order to stabilize the subgrade.
As an alternate to placement of geosynthetic reinforcement and coarse granular fill, chemical stabilization techniques may be utilized to create stable surfaces. Additional recommendations regarding chemical stabilization techniques can be provided upon request.

**Imported Fill Materials**

Imported granular fill materials to be used under the building foundations/floor slabs and behind the foundation and retaining walls should be free of topsoil, organic material, claystone and other deleterious materials. Imported granular fill materials should meet CDOT Class I Structural Backfill Requirements as tabulated below:

<table>
<thead>
<tr>
<th>Sieve Size or Parameter</th>
<th>Acceptable Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-inch Sieve</td>
<td>100% passing</td>
</tr>
<tr>
<td>No. 4 Sieve</td>
<td>30% to 100% passing</td>
</tr>
<tr>
<td>No. 50 Sieve</td>
<td>10% to 60% passing</td>
</tr>
<tr>
<td>No. 200 Sieve</td>
<td>5% to 20% passing</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>( \leq 35% )</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>( \leq 6% )</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>( \geq 34) degrees</td>
</tr>
</tbody>
</table>

If it is necessary to import additional materials to the site for non-structural use, those imported materials should also be free of topsoil, organic material, claystone and other deleterious materials and should have less than 65 percent passing the No. 200 Sieve and should have a plasticity index of less than 20.

All imported soils should be tested and approved based on the intended use by the Geotechnical Engineer prior to transport to the site. The Geotechnical Engineer should be notified at least 1 week prior to importing materials.
Fill Platform Preparation

Prior to filling, the top 12 inches of in-place materials on which fill soils will be placed should be scarified, moisture conditioned and properly compacted in accordance with the recommendations below to provide a uniform base for fill placement.

If surfaces to receive fill expose loose, wet, soft or otherwise deleterious material, additional material should be excavated, or other measures taken, to establish a firm platform for filling. The surfaces to receive fill must be effectively stable prior to placement of fill.

Fill Placement

Fill materials should be thoroughly mixed to achieve a uniform moisture content, placed in uniform lifts not exceeding 8 inches in loose thickness, and properly compacted.

If surfaces to receive fill expose loose, wet, soft or otherwise deleterious material, additional material should be excavated, or other measures taken, to establish a firm platform for filling.

No fill materials should be placed, worked, rolled while they are frozen, thawing, or during poor/inclement weather conditions.

To achieve adequate compaction near the outer faces of fill slopes, it may be beneficial to over-build the slopes and trim them back.

Soils that classify as GP, GW, GM, GC, SP, SW, SM or SC in accordance with the USCS classification system (granular materials) should be compacted to 95 or more percent of the maximum modified Proctor dry density at moisture contents within 2 percent of optimum moisture content as determined by ASTM D1557. Soils that classify as CL, CH, ML or MH (cohesive soils) should be compacted to 95 percent of the maximum standard Proctor density at moisture contents from 1 percent below to 3 percent above the optimum as determined by ASTM D698.

Quality Assurance

A Geotechnical Engineer should be retained to observe project excavations prior to placement of fill. The Geotechnical Engineer should also be retained to observe
earthwork operations and test the soils. The Geotechnical Engineer should provide a written declaration stating that the project site, including the building pad area, was filled with acceptable materials and was placed in accordance with the specifications outlined in this report.

Potential earthwork contractors should be made aware that on-site soils are very sensitive to moisture and extreme care will likely be required when moisture treating and compacting these soils. Moisture content and relative compaction testing should be conducted at a sufficient frequency in order to ensure proper placement.

It should be noted that in the later stages of large-scale projects, multiple subcontractors commonly are installing or adjusting/replacing components of the project simultaneously. These include utility laterals, electrical boxes, sidewalk access ramps, lighting fixtures, etc. Because several firms can be performing local earthworks on the site simultaneously, and because the same area can be excavated, backfilled and re-excavated and backfilled several times, achieving, maintaining and documenting adequate compaction in the backfill soils typically is very difficult. In order to facilitate proper observation and testing of the associated earthworks, GROUND recommends that the Contractor verify that his sub-contractors mobilize the necessary equipment and personnel to moisture-condition and compact disturbed or excavated soils effectively. The Contractor also should coordinate with his sub-contractors to ensure that these local earthwork operations are observed and tested with sufficient frequency by the Geotechnical Engineer.

Settlements

Settlements will occur in filled ground, typically on the order of 1 to 2 percent of the fill depth. For a 6-foot fill, this corresponds to settlement on the order of an inch, without imposition of foundation loads. If fill placement is performed properly and is tightly controlled, in GROUND’s experience the majority of that settlement will take place during earthwork construction. The remaining potential settlements likely will take several months or longer, to be realized.

Cut and Filled Slopes

Permanent site slopes supported by on-site soils up to 5 feet in height may be constructed no steeper than 3:1 (horizontal : vertical). Minor raveling or surficial
sloughing should be anticipated on slopes cut at this angle until vegetation is well re-established. Surface drainage should be designed to direct water away from slope faces.

**Additional Earthworks and Building Pad Caps**

Building fill prisms prepared for subsequent construction, as well as properly other compacted fills, that are allowed to stand for extended periods prior to construction of buildings, pavements, etc., will degrade over time. Moisture will be lost and the soils can become eroded or loosened, or shrink back to a denser and more expansive condition. The extent of degradation will increase with time. Therefore, where these conditions arise, the Geotechnical Engineer should be retained to test the soils and provide recommendations regarding an appropriate depth of moisture conditioning and re-compaction prior to construction.

**Squeegee**

Relatively uniformly graded fine gravel or coarse sand, i.e., "squeegee," or similar materials commonly are proposed for backfilling foundation excavations, portions of utility trenches and other areas where employing compaction equipment is difficult. In general, GROUND does not recommend this procedure for the following reasons:

> Although commonly considered "self compacting," uniformly graded granular materials require densification after placement, typically by vibration. The equipment to densify these materials is not available on many job-sites.

> Even when properly densified, uniformly graded granular materials are permeable and allow water to reach and collect in the lower portions of the excavations backfilled with those materials. This leads to wetting of the underlying soils and resultant potential loss of bearing support as well as increased local heave or settlement.

GROUND recommends that wherever possible, excavations be backfilled with approved, on-site soils placed as properly compacted fill. Where this is not feasible, use of “Controlled Low Strength Material” (CLSM), i.e., a lean, sand-cement slurry ("flowable fill") or a similar material for backfilling should be considered.
Where "squeegee" or similar materials are proposed for use by the Contractor, the
design team should be notified by means of a Request for Information (RFI), so that the
proposed use can be considered on a case-by-case basis.

**WATER-SOLUBLE SULFATES**

The concentrations of water-soluble sulfates measured in selected samples obtained
from the test holes ranged up to 0.02 percent by weight (see Table 2). Such
concentrations of water-soluble sulfates represent a negligible degree of sulfate attack
on concrete exposed to these materials. Degrees of attack are based on the scale of
'negligible,' 'moderate,' 'severe' and 'very severe' as described in the "Design and
Control of Concrete Mixtures," published by the Portland Cement Association.

Based on these data GROUND, makes no recommendation for use of a special, sulfate-
resistant cement in project concrete.

**SOIL CORROSION**

The degree of risk for corrosion of metals in soils commonly is considered to be in two
categories: corrosion in undisturbed soils and corrosion in disturbed soils. The potential
for corrosion in undisturbed soil is generally low, regardless of soil types and conditions,
because it is limited by the amount of oxygen that is available to create an electrolytic
cell. In disturbed soils, the potential for corrosion typically is higher, but is strongly
affected by soil conditions for a variety of reasons but primarily soil chemistry.

A corrosivity analysis was performed to provide a general assessment of the potential
for corrosion of ferrous metals installed in contact with earth materials at the site, based
on the conditions existing at the time of GROUND’s evaluation. Soil chemistry and
physical property data including pH, oxidation-reduction (redox) potential, sulfides, and
moisture content were obtained. Test results are summarized on Table 2.

**Soil Resistivity**

In order to assess the "worst case" for mitigation planning, samples of materials
retrieved from the test holes were tested for resistivity in the laboratory, after being
saturated with water, rather than in the field. Resistivity also varies inversely with
temperature. Therefore, the laboratory measurements were made at a controlled temperature.

Measurements of electrical resistivity indicated values from approximately 1,112 to 1,201 ohm-centimeters. The following table presents the relationship between soil resistivity and a qualitative corrosivity rating:

<table>
<thead>
<tr>
<th>Soil Resistivity (ohm-cm)</th>
<th>Corrosivity Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;20,000</td>
<td>Essentially non-corrosive</td>
</tr>
<tr>
<td>10,000 – 20,000</td>
<td>Mildly corrosive</td>
</tr>
<tr>
<td>5,000 – 10,000</td>
<td>Moderately corrosive</td>
</tr>
<tr>
<td>3,000 – 5,000</td>
<td>Corrosive</td>
</tr>
<tr>
<td>1,000 – 3,000</td>
<td>Highly corrosive</td>
</tr>
<tr>
<td>&lt;1,000</td>
<td>Extremely corrosive</td>
</tr>
</tbody>
</table>

**pH**

Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity. Testing indicated pH values of approximately 7.3.

The American Water Works Association (AWWA) has developed a point system scale used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. When the scale equals 10 points or higher, protective measures for ductile iron pipe are recommended. The AWWA scale is presented below. The soil characteristics refer to the conditions at and above pipe installation depth.

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2 American Water Works Association ANSI/AWWA C105/A21.5-05 Standard
Table A.1 Soil-test Evaluation

<table>
<thead>
<tr>
<th>Soil Characteristic / Value</th>
<th>Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistivity</td>
<td></td>
</tr>
<tr>
<td>&lt;1,500 ohm-cm</td>
<td>10</td>
</tr>
<tr>
<td>1,500 to 1,800 ohm-cm</td>
<td>8</td>
</tr>
<tr>
<td>1,800 to 2,100 ohm-cm</td>
<td>5</td>
</tr>
<tr>
<td>2,100 to 2,500 ohm-cm</td>
<td>2</td>
</tr>
<tr>
<td>2,500 to 3,000 ohm-cm</td>
<td>1</td>
</tr>
<tr>
<td>&gt;3,000 ohm-cm</td>
<td>0</td>
</tr>
<tr>
<td>pH</td>
<td></td>
</tr>
<tr>
<td>0 to 2.0</td>
<td>5</td>
</tr>
<tr>
<td>2.0 to 4.0</td>
<td>3</td>
</tr>
<tr>
<td>4.0 to 6.5</td>
<td>0</td>
</tr>
<tr>
<td>6.5 to 7.5</td>
<td>0*</td>
</tr>
<tr>
<td>7.5 to 8.5</td>
<td>0</td>
</tr>
<tr>
<td>&gt;8.5</td>
<td>3</td>
</tr>
<tr>
<td>Redox Potential</td>
<td></td>
</tr>
<tr>
<td>&lt; 0 (negative values)</td>
<td>5</td>
</tr>
<tr>
<td>0 to +50 mV</td>
<td>4</td>
</tr>
<tr>
<td>+50 to +100 mV</td>
<td>3½</td>
</tr>
<tr>
<td>&gt; +100 mV</td>
<td>0</td>
</tr>
<tr>
<td>Sulfide Content</td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>3½</td>
</tr>
<tr>
<td>Trace</td>
<td>2</td>
</tr>
<tr>
<td>Negative</td>
<td>0</td>
</tr>
<tr>
<td>Moisture</td>
<td></td>
</tr>
<tr>
<td>Poor drainage, continuously wet</td>
<td>2</td>
</tr>
<tr>
<td>Fair drainage, generally moist</td>
<td>1</td>
</tr>
<tr>
<td>Good drainage, generally dry</td>
<td>0</td>
</tr>
</tbody>
</table>

* If sulfides are present and low or negative redox-potential results (< 50 mV) are obtained, add three points for this range.

We anticipate that drainage at the site after construction will be good. Nevertheless, based on the values obtained for the soil parameters, the overburden soils/bedrock appear(s) to comprise a highly corrosive environment for metals.

Corrosive conditions can be addressed by use of materials not vulnerable to corrosion, heavier gauge materials with longer design lives, polyethylene encasement, or cathodic protection systems. If additional information or recommendations are needed regarding soil corrosivity, GROUND recommends contacting the American Water Works Association or a Corrosion Engineer. It should be noted, however, that changes to the

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3 American Water Works Association ANSI/AWWA C105/A21.5-05 Standard
site conditions during construction, such as the import of other soils, or the intended or unintended introduction of off-site water, may alter corrosion potentials significantly.

EXCAVATION CONSIDERATIONS

Test holes for subsurface exploration were advanced to the depths indicated on the test hole logs by means of truck-mounted, flight auger equipment. We anticipate some excavation difficulties, generally caused by the presence of shallow groundwater, for the proposed construction in these materials with conventional, medium- to heavy-duty excavating equipment in good working condition.

Groundwater was encountered during subsurface exploration at depths of approximately 8 to 10 feet below the existing grades. Groundwater levels could also fluctuate due to variety of reasons as mentioned under the Subsurface Conditions section of this report. Based on the likely depths of earthworking and construction, groundwater and/or related soft and wet subgrade conditions is anticipated to be a significant factor for shallow earthworks during construction of this project. The Contractor should anticipate encountering water at or below these approximate elevations and be prepared to work in the presence of groundwater. In addition, shallower excavations may also expose wet soils or seepage.

A properly designed and installed de-watering system may be required during the construction of remedial earthworks (i.e. removal and replacement of native soils under the building), elevator shaft construction, utility installation, and other areas where excavation bottoms are near (i.e. within 5 feet) groundwater levels. The de-watering system should be designed by the Contractor. The dewatering should remain operational throughout construction.

In addition, the presence of water and the influence of heavy construction traffic may result in unstable conditions adjacent to excavation areas. We recommend construction traffic be limited and that any dewatering operations remove water completely off-site.

We recommend that temporary, un-shored excavation slopes up to 8 feet in height be cut no steeper than 1½:1 (horizontal : vertical) in the native soils in the absence of seepage. The risk of slope instability will be significantly increased in areas of seepage along the excavation slopes. Some surficial sloughing may occur on slope faces cut at this angle. Local conditions encountered during construction, such as loose, dry sand,
or soft or wet materials, or seepage will require flatter slopes. Stockpiling of materials should not be permitted closer to the tops of temporary slopes than 5 feet or a distance equal to the depth of the excavation, which ever is greater.

Should site constraints prohibit the use of the recommended slope angles, then temporary shoring should be used. Shoring designed to allow the soils to deflect sufficiently to utilize the full active strength of the soils may be designed for lateral earth pressures computed on the basis of an equivalent fluid unit weight of 51 pcf (89 pcf submerged) for a level adjacent ground condition. In addition to this lateral earth pressure, shoring design should include surcharge loads exerted by equipment, traffic, material stockpiles, etc. Actual shoring system(s) should be designed for the Contractor by a registered Engineer. The shoring and dewatering system designs must be coordinated by the Contractor.

Good surface drainage should be provided around temporary excavation slopes to direct surface runoff away from the slope faces. A properly designed drainage swale should be provided at the top of the excavations. In no case should water be allowed to pond at the site. Slopes should also be protected against erosion. Erosion along the slopes will result in sloughing and could lead to a slope failure.

Excavations in which personnel will be working must comply with all OSHA Standards and Regulations. The Contractor's "responsible person" should evaluate the soil exposed in the excavations as part of the Contractor's safety procedures. GROUND has provided the information above solely as a service to University of Colorado at Boulder and is not assuming responsibility for construction site safety or the Contractor's activities.

UTILITY INSTALLATION

Recommendations regarding utility trench excavation are provided in the Excavation Considerations section of this report.

On-site soils excavated from trenches are suitable, in general, for use as trench backfill. Due to the shallow groundwater levels, depending on the depth of utility installation, it may be necessary to dry the excavated native overburden soils prior to placing them as backfill or use other imported approved fill materials instead. Backfill soils should be free of vegetation, debris and other deleterious materials.
Pipe bedding materials, placement and compaction should meet the specifications of the pipe manufacturer and applicable municipal standards. Materials proposed for use as pipe bedding, should be tested for suitability prior to use. The Contractor should not anticipate that significant quantities of materials excavated on-site will be suitable for use where relatively free-draining bedding materials are called for.

Trench backfill materials should be placed in accordance with the recommendations in the Project Earthworks section of this report. Bedding should be brought up uniformly on both sides of the pipe to reduce differential loadings.

Some settlement of trench backfill materials should be anticipated, even where materials are placed and compacted correctly. To reduce these settlements, the Contractor should take adequate measures to achieve adequate compaction in the utility trench backfills, particularly in the lower portions of the excavations and around manholes, valve risers and other vertical pipeline elements where greater settlements commonly are observed. However, the need to compact to the lowest portion of the backfill must be balanced against the need to protect the pipe from damage during backfilling. Some thickness of backfill may need to be placed at compaction levels lower than recommended above to avoid damaging the pipe. Likewise, construction conditions may preclude density testing at specified frequencies in the lower portions of a trench.

Because of these limitations, we recommend the use of “controlled low strength material” (CLSM), i.e., a lean, sand-cement slurry, “flowable fill,” or similar material in lieu of compacted soil backfill for areas with low tolerances for surface settlements. Placement of CLSM in the lower portion of the trench and around risers, etc., likely will yield a superior backfill and provide protection for the pipe, although at an increased cost. Other means, e.g., use of smaller compaction equipment, also may be effective for achieving adequate compaction in these areas.

Based on the presence of ground water, it is our opinion that all bedding materials or rock used for stabilization should be protected from infiltration of fines from the backfill by using a separator geotextile, such as Mifafi 140N. If such protection is not provided, an additional risk of settlement will be inherent on this project, regardless of compactive effort. In some cases, the infiltration of fines into the bedding or other granular layers
used for stabilization, etc. can continue for many years resulting in a roadway surface that requires constant maintenance.

Development of site grading plans should consider the subsurface transfer of water in utility trenches and the pipe bedding. Sandy pipe bedding materials can function as efficient conduits for re-distribution of natural and applied waters in the subsurface. Cut-off walls in utility trenches or other water-stopping measures should be implemented to reduce the rates and volumes of water transmitted along utility alignments and toward buildings, pavements and other structures where excessive wetting of the underlying soils will be damaging. Incorporation of water cut-offs and/or outlet mechanisms for saturated bedding materials into development plans could be beneficial to the project. These measures also will reduce the risk of loss of fine-grained backfill soils into the bedding material with resultant surface settlement.

SURFACE DRAINAGE

The following drainage precautions should be observed during construction and maintained at all times after the facility has been completed. Establishment and regular maintenance of surface drainage is critical to the proper performance of the proposed structures and pavements. If the drainage measures below are not implemented and maintained effectively, the movement estimates provided in this report could be exceeded.

1. Wetting or drying of the foundation excavations and underslab areas should be avoided during construction.

2. Care should be taken during and after construction to identify local, concentrated water sources such as leaking hose bibs or pipes, downspouts from which water is not positively controlled, construction water tanks, etc., and repair the damage or make positive provision to capture the water or convey it off-site.

3. Positive surface drainage measures should be provided and maintained to reduce water infiltration into foundation soils. The ground surface surrounding the exterior of each building should be sloped to drain away from the foundation in all directions. We recommend a minimum slope of 12 inches in the first 5 feet in landscaped areas and 3 inches in the first 5 feet in areas where hardscaping covers the ground adjacent to the building. (It may be necessary to incorporate
ramps or other measures into project design to implement this recommendation while complying with access requirements.) In no case should water be allowed to pond near or adjacent to foundation elements.

4. Drainage measures also should be included in project design to direct water away from sidewalks and other hardscaping as well as utility trench alignments that are likely to be adversely affected by moisture-volume changes in the underlying soils or flow of infiltrating water. Routine maintenance of site drainage, including sealing of cracks and joints, should be undertaken throughout the design life of the project.

5. The ground surface near foundation elements should be able to convey water away readily. Ground coverings that direct water downward rather than away from the building should not be used to cover the ground surface near the foundations or other improvements sensitive to post-construction soil movements. Cobbles or other materials that tend to act as baffles and restrict surface flow should not be used.

Correspondingly, near other project improvements, such as hardscaping, where the ground surface does not convey water away readily, additional post-construction movements and distress should be anticipated. Shallow underdrain systems or flatwork edge drains can be beneficial in reducing movements and distress.

6. The buildings should be provided with roof drains/gutters and downspouts. Roof downspouts and drains should discharge well beyond the perimeters of the structure foundations, or be provided with positive conveyance off-site for collected waters. Providing piped discharge to the project storm sewer system is the most effective means of reducing infiltration into the site soils. Where collected waters are allowed to flow across the ground surface (whether paved or landscaped), the potential for post-construction movements are increased, particularly where joints, cracks, adverse local surface gradients, etc., are not well maintained.
7. Roof drains should not be routed inside the buildings. Roof downspouts and other surface water collectors should not discharge into the building underdrain systems.

8. Landscaping that requires watering should be located 10 or more feet from building perimeters. Irrigation sprinkler heads should be deployed so that applied water is not introduced into foundation soils. Landscape irrigation should be limited to the minimum quantities necessary to sustain healthy plant growth.

9. Use of drip irrigation systems can be beneficial for reducing the amount of water introduced to building foundation soils, but only if the total volumes of applied water are controlled with regard to limiting that introduction. Controlling rates of moisture increase beneath the foundations and floors should take higher priority than minimizing landscape plant losses.

10. We understand that municipal ordinances commonly require plantings in close proximity to buildings. Where plantings are desired within 10 feet of a building, GROUND recommends that the plants be placed in water-tight planters, constructed either in-ground or above-grade, to reduce moisture infiltration in the surrounding subgrade soils. Planters should be provided with positive drainage and landscape underdrains. Where watertight planters cannot be used, project design measures and post-construction maintenance routines should be developed to prevent infiltration.

11. We do not recommend the use of plastic membranes to cover the ground surface near the building without careful consideration of other components of project drainage. Plastic membranes can be beneficial to directing surface waters away from the building and toward drainage structures. However, they effectively preclude evaporation or transpiration of shallow soil moisture. Therefore, soil moisture tends to increase beneath a continuous membrane. Where plastic membranes are used, additional shallow, subsurface drains or other means to limit moisture increases should be considered.

12. Detention ponds commonly are incorporated into drainage design. When a detention pond fills, the rate of release of the water is controlled and water is retained in the pond for a period of time. Where in-ground storm sewers direct
surface water to the pond, the granular pipe bedding also can direct shallow groundwater or infiltrating surface water toward the pond. Thus, detention ponds can become locations of enhanced and concentrated infiltration into the subsurface, leading to wetting of foundation soils in the vicinity with consequent heave or settlement. Therefore, unless the pond is clearly down-gradient from the proposed building and other structures that would be adversely affected by wetting of the subgrade soils, including off-site improvements, GROUND recommends that the detention pond should be provided with an effective, low permeability liner. In addition, cut-off walls and/or drainage provisions should be provided for the bedding materials surrounding storm sewer lines flowing to the pond.

Regular maintenance of site drainage should be implemented throughout the design-life of the project. The site should be observed periodically for indications of ineffective drainage, settlement, ponding, erosion, damaged irrigation lines, etc., so that repairs can be effected. Necessary repairs/restoreation should be implemented as soon as possible. Where localized infiltration is allowed to continue, the risk of local settlements or heave will be increased.

In addition to design, construction and maintenance of effective, permanent surface drainage measures, effective surface drainage must be established and regularly maintained during the entire construction process. In GROUND's experience, it is common in areas of partially completed paving or hardscaping, bare soil surrounding buildings and behind curbs and gutters, and open utility trenches, etc., that water from irrigation or rain and snow melt events is allowed to pond. Because of the numerous trades that can be working on a site at various stages of construction, maintenance of effective surface drainage can require weekly or more frequent restoration. As has been discussed elsewhere in this report, wetting of the subgrade can result in loss of support and increased settlements. By the time that final grading has been completed, and permanent surface and subsurface drainage measures constructed, significant volumes of water can already have entered the subgrade, with consequent increased risks of movements, etc. GROUND recommends that the Contractor be directed to maintain the site regularly during the entire construction process so that water is directed into appropriate drainage structures during construction.
We understand that local, state and/or federal regulations regarding erosion control may require use of silt fencing, straw bundles or other erosion-control devices that can significantly impede surface drainage during construction, preventing effective implementation of the recommendations above. GROUND recommends that the various measures that could be implemented to comply with applicable regulations be assessed carefully with regard to minimizing interference with effective surface drainage. Use of local, lined sumps that can be pumped regularly, for example, may be necessary to prevent uncontrolled ponding and infiltration prior to completion of fine grading.

SUBSURFACE MOISTURE INFILTRATION AND DRAINAGE

Standard practice for the combination of soil and foundation system proposed for this project includes the installation of an underdrain system. If properly constructed, a perimeter underdrain system can result in a reduction of moisture infiltration of the subsurface soils. However, an underdrain not properly functioning can induce settlement or heave of the subsurface soils and may result in structure/floor slab distress.

The site soils are relatively stable with regard to moisture content – volume relationships at their existing moisture contents. Other than the anticipated, post-placement settlement of fills, post-construction soil movements from consolidation will result primarily from the introduction of water into the soils underlying the proposed structure, hardscaping and pavements. Based on the site surface and subsurface conditions encountered in this study, we do not anticipate a rise in the local water table sufficient to cause adverse wetting of the soils supporting shallow foundations. Therefore, wetting of the soils likely will result from infiltrating surface waters (precipitation, irrigation, etc.), flow along constructed pathways such as bedding in utility pipe trenches. Project design should incorporate measures to prevent water from wetting the project soils. Surface drainage gradients, pavements, flatwork, piping, drainage structures, etc., should be maintained during and after construction to prevent infiltration. Pipes, below-grade drainage structures, etc., should be maintained during and after construction to prevent infiltration.

It is the responsibility of the design team and Ownership as well as the construction and maintenance Contractor(s) within their respective disciplines and in accordance with
their familiarity with the site conditions to evaluate the possible sources of water that could affect the project area and provide design and/or construction measures that address the conditions so that moisture is directed away from the foundations and supporting materials prior to being allowed to infiltrate the subsurface, both during and after construction. Wetting or drying of the foundation excavations and underslab areas should be avoided during and after construction as well as throughout the life of the facility. Permitting increases/variations in moisture to the supporting soils may result in a decrease in bearing capacity and an increase in total and/or differential movements.

If an underdrain is incorporated in the project design, the underdrain system should consist of perforated PVC drainpipe at least 4 inches in diameter, free-draining gravel, and filter fabric. The free-draining gravel should contain less than 5 percent passing the No. 200 Sieve and more than 50 percent retained on the No. 4 Sieve, and have a maximum particle size of 2 inches. Each collector pipe should be surrounded on the top and sides only with 6 or more inches of free-draining gravel. The gravel surrounding the drainpipe should be wrapped with filter fabric to reduce the migration of fines into the drain system.

Areas such as the elevator shafts, crawl spaces, or other below grade levels that may extend several feet below the floor system should be provided with separate drain systems that are located around their local perimeter, constructed in the same manner as mentioned above. Depending on the actual elevation of these localized below grade levels, it may be necessary to construct permanent dewatering systems. Additional recommendations in this regard can be provided upon request.

Wall drain measures, waterproofing, and water stops at joints or similar measures should be provided at the foundation walls that are below grade. The walls should be backed with a layer of synthetic drainage medium, e.g., an appropriate Miradrain® product or equivalent. Damp-proofing and a drain board –type product should be installed along the exterior of the foundation walls. The drain board should be in hydraulic continuity with the underdrain system.

The actual layout, outlets, and locations of underdrains should be designed by the Civil Engineer. The pipe(s) for underdrain systems should be graded at a sufficient slope to enable efficient discharge of collected waters to one or more sumps from which water can be removed by pumping, or to outlet(s) for gravity discharge. Design slopes should
consider potential post-construction soil movements, as discussed herein, which could affect the pipes, as well. Where collection segments of the underdrain system (with gravel and perforated pipe) transition to discharge segments with solid pipe, clay or concrete cut-off walls should be installed to reduce migration of water along the discharge line trenches outside of the pipes.

All subsurface drain systems should be tested by the Contractor after installation, and after placement and compaction of the overlying backfill, to verify that the systems function properly.

PAVEMENT SECTIONS

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings. Pavement sections were developed in general accordance with the design guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO), the CDOT and local construction practice.

Subgrade Materials

Based on the results of our field and laboratory studies, subgrade materials in the proposed pavement areas consisted predominantly of clayey sands and sandy clays. These materials were classified typically as A-4, A-6, and A-7-6 soils in accordance with the AASHTO classification system.

Resilient Modulus (M_r) testing (AASHTO T-307) was performed on a representative composite sample of the subgrade materials encountered at the site. Typically, the R-value, unconfined compressive strength, California Bearing Ratio (CBR), or other index properties of subgrade materials have been obtained and the resilient modulus obtained only by correlation. However, due to the variability in the correlations, subjecting representative samples of the subgrade to the actual resilient modulus testing is the most accurate way to determine soil support characteristics for use in pavement design.

A dynamic load test, the resilient modulus measures the elastic rebound stiffness of flexible pavement materials, base courses and subgrades under repeated loading. The loading cycles were applied under various confining and deviatoric stresses as specified
in AASTHO T-294. The material was compacted to 95 percent of maximum dry density at optimum moisture content, and at 2 percent and 4 percent above the optimum, based on AASHTO T-99 (the "standard Proctor") for cohesive soils, or AASHTO T-180 (the "modified Proctor") for granular soils.

The resilient modulus of a material at 2 percent above optimum moisture content typically is used for the pavement design for fine-grained soils that classify as A-4, A-6, or A-7. The resilient modulus at the optimum moisture content is typically used for granular soils that classify as A-1 or A-2. For the granular shallow site soils, the resilient modulus at optimum moisture content was taken as representative of the subgrade materials. Therefore, a resilient modulus 4,228 psi was used for pavement design obtained at 2 percent above optimum moisture content.

It is important to note that significant decreases in soil support as quantified by the resilient modulus have been observed as the moisture content increases above the optimum. Therefore, pavements that are not properly drained may experience a loss of the soil support and subsequent reduction in pavement life.

**Design Traffic**

Specific traffic loadings were not available at the time of this report preparation. Therefore, an equivalent 18-kip daily load application (EDLA) value of 20 was assumed for the project driveways. An EDLA of 10 was assumed for automobile-only parking stalls. The EDLA values of 20 and 10 were converted to equivalent 18-kip single-axle load (ESAL) values of 146,000 and 73,000, respectively, for 20-year design lives.

An EDLA of 50, corresponding to an ESAL value of 365,000, was assumed for all areas subject to heavy traffic and high turning stresses including but not limited to bus drive loop, loading docks, loading/unloading areas, delivery areas, etc.

If design traffic loadings differ significantly from these assumed values, GROUND should be notified to re-evaluate the pavement recommendations below.

**Pavement Design**

The soil resilient modulus and the design ESAL values were used to determine the required design structural number for the project pavements. The required structural
number was then used to develop recommended pavement sections. Pavement designs were based on the DARWin™ computer program that solves the 1993 AASHTO pavement design equation. Pavement design parameters and calculations are summarized in Appendix A. A Reliability Level of 85 and 90 percent was utilized for design of the flexible and rigid pavement (PCCP) sections, respectively. Structural coefficients of 0.40 and 0.12 were used for hot bituminous asphalt (HBA) and high quality aggregate base course (ABC), respectively. The table provided below summarizes the recommended minimum pavement sections:

<table>
<thead>
<tr>
<th>Pavement Design Summary</th>
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<tr>
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<td>(HBA / ABC)</td>
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<td>(PCCP / ABC)</td>
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<tr>
<td>-</td>
</tr>
<tr>
<td>6.5 / 6</td>
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</table>

Please refer to the Subgrade Preparation section below for additional information regarding pavement subgrade preparation.

Hot Bituminous Asphalt (HBA)

The asphalt pavement shall consist of a bituminous plant mix composed of a mixture of high quality aggregate and bituminous material, which meets the requirements of a job-mix formula established by a qualified engineer. The asphalt material used should be based on a SuperPave Gyratory Design Revolution.

Portland Cement Concrete Pavement (PCCP)

Concrete pavements should consist of a plant mix composed of a mixture of aggregate, portland cement and appropriate admixtures meeting the requirements of a job-mix formula established by a qualified engineer as well as applicable design requirements of University of Colorado at Boulder. Concrete should have a minimum modulus of rupture of third point loading of 650 psi. Normally, concrete with a 28-day compressive strength of 4,500 psi should develop this modulus of rupture value. The concrete should be air-entrained with approximately 6 percent air and should have a minimum cement content of 6 sacks per cubic yard. Maximum allowable slump should be 4 inches. These
concrete mix design criteria should be coordinated with other project requirements including the criteria for sulfate resistance presented in the Water-Soluble Sulfates section of this report. To reduce surficial spalling resulting from freeze-thaw cycling, we suggest that pavement concrete meet the requirements of CDOT Class P concrete. In addition, the use of de-icing salts on concrete pavements during the first winter after construction will increase the likelihood of the development of scaling. Placement of flatwork concrete during cold weather so that it is exposed to freeze-thaw cycling before it is fully cured also increases its vulnerability to scaling. Concrete placing during cold weather conditions should be blanket ed or tented to allow full curing. Depending on the weather conditions, this may result in 3 to 4 weeks of curing, and possibly more.

The concrete pavement should contain effective sawed or formed joints. In areas of repeated turning stresses we recommend that the concrete pavement joints be fully tied and doweled. We suggest that civil design consider joint layout in accordance with CDOT's M Standards. Standard plans for placement of ties and dowels, etc., (CDOT M Standards) for concrete pavements can be found at the CDOT website: http://www.dot.state.co.us/DesignSupport/

Aggregate Base Course (ABC)

The aggregate base material should meet the criteria of CDOT Class 6 aggregate base course. Base course should be placed in uniform lifts not exceeding 8 inches in loose thickness and compacted to at least 95 percent of the maximum dry density a uniform moisture contents within 3 percent of the optimum as determined by ASTM D1557 / AASHTO T-180, the “modified Proctor.”

Subgrade Preparation

Shortly before placement of pavement, including aggregate base, the exposed subgrade soils should be scarified to a depth of at least 12 inches, thoroughly mixed, moisture conditioned and replaced as properly compacted fill as outlined in the Project Earthworks section of this report. Subgrade preparation should extend the full width of the pavement from back-of-curb to back-of-curb. As recommended in the Exterior Flatwork and Project Hardscaping section of this report, sidewalks and other hardscaping also should be underlain by at least 12 inches of properly moisture conditioned and compacted fill.
Due to the nature of the majority of site soils, some post-construction movements of the pavements should be anticipated. A greater depth of excavation and replacement will result in improved pavement performance over its design life. Recommendations in this regard can be provided upon request.

Recommendations for placement and compaction of fill soils are provided in the *Project Earthworks* section of this report.

The Contractor should be prepared either to dry the subgrade materials or moisten them, as needed, prior to compaction. Some site soils likely will "pump" or deflect during compaction if moisture levels are not carefully controlled. The Contractor should be prepared to process and compact such soils to establish a stable platform for paving, including use of chemical stabilization, if necessary.

Immediately prior to paving, the roadway subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle. The proof roll should be performed in accordance with Section 203.09 of the current CDOT Standard Specifications for Road and Bridge. Areas that show excessive deflection during proof rolling should be excavated and replaced and stabilized. Areas allowed to pond prior to paving will require significant reworking prior to proof rolling.

**Pavement Performance**

The collection and diversion of surface drainage away from paved areas is extremely important to satisfactory performance of the pavements. The subsurface and surface drainage systems should be carefully designed to ensure removal of the water from paved areas and subgrade soils. Allowing surface waters to pond on pavements will cause premature pavement deterioration. Where topography, site constraints or other factors limit or preclude adequate surface drainage, pavements should be provided with edge drains to reduce loss of subgrade support.

GROUND's experience indicates that longitudinal cracking is common in asphalt-pavements generally parallel to the interface between the asphalt and concrete structures such as curbs, gutters or drain pans. Distress of this type is likely to occur even where the subgrade has been prepared properly and the asphalt has been compacted properly.
The standard care of practice in pavement design describes the recommended flexible pavement section as a “20-year” design pavement; however, most pavements will not remain in satisfactory condition without routine, preventive maintenance and rehabilitation procedures performed throughout the life of the pavement. Preventive pavement treatments are surface rehabilitation and operations applied to improve or extend the functional life of a pavement. These treatments preserve, rather than improve, the structural capacity of the pavement structure. In the event the existing pavement is not structurally sound, the preventive maintenance will have no long-lasting effect. Therefore, a routine maintenance program to seal cracks, repair distressed areas, and perform thin overlays throughout the life of the pavement is recommended.

A crack sealing and fog seal / chip seal program should be performed on the roadway alignment on a regular basis. After approximately 8 to 10 years, patching, additional crack sealing, and asphalt overlay may be required. Prior to future overlays, it is important that all transverse and longitudinal cracks be sealed with a flexible, rubberized crack sealant in order to reduce the potential for propagation of the crack through the overlay. Traffic volumes that exceed the values utilized by this report will likely necessitate the need of pavement maintenance practices on a schedule of shorter timeframe than that stated above. “The greatest benefit of preventive maintenance is achieved by placing the treatments on sound pavements that have little or no distress”.

CLOSURE

Geotechnical Review

The poor performance of foundations and subsurface structures has been directly attributed to inadequate geotechnical review and earthwork quality control. Therefore, a Geotechnical Engineer should be retained to review project plans and specifications to evaluate whether they comply with the intent of the recommendations in this report. The author of this report and/or the reviewing engineer should be contacted directly to provide this review. The review should be reported in writing.

Project earthwork construction operations should be observed by the Geotechnical Engineer. All excavations should be observed by the Geotechnical Engineer prior to placement of fill or backfill soils, installation of shoring, or foundation construction.
Placement of fill/backfill soils should be observed by the Geotechnical Engineer, and the soils tested.

The geotechnical recommendations presented in this report are highly contingent upon observation and testing of project earthworks by representatives of GROUND. If another geotechnical consultant is selected to provide construction observation and quality control, then that consultant must assume all responsibility for the geotechnical aspects of the project by concurring in writing with the recommendations in this report, or by providing alternative recommendations.

**Limitations**

This report has been prepared for University of Colorado at Boulder as it pertains to design of the subject University of Colorado Boulder Williams Village Phase II Pod "C" Residence Hall project as described herein. It may not contain sufficient information for other parties or other purposes. In addition, GROUND has assumed that project construction will commence by Spring 2010. Changes in project plans or schedule should be brought to the attention of the Geotechnical Engineer, in order that the geotechnical recommendations may be re-evaluated and, as necessary, modified.

The geotechnical conclusions and recommendations in this report relied upon subsurface exploration at a limited number of exploration points, as shown in Figure 1. Subsurface conditions were interpolated between and extrapolated beyond these locations. It is not possible to guarantee the subsurface conditions are as indicated in this report. Actual conditions exposed during construction should be anticipated to differ, somewhat, from those encountered during site exploration.

Contractors should review all available project information prior to providing construction/service bids. If during construction, surface, soil, bedrock, or groundwater conditions appear to be at significant variance with those described herein, the Geotechnical Engineer should be advised at once, so that re-evaluation of the recommendations may be made in a timely manner. Findings were dependent on the limited amount of direct evidence obtained at the time of this geotechnical evaluation. Our recommendations were developed for the site conditions as described above.

A Contractor who relies upon this report for development of his scope of work or cost estimates may consider the geotechnical information in this report to be inadequate for
his purposes or find the geotechnical conditions described herein to be at variance with his experience in the greater project area. In such cases, the Contractor should obtain the additional geotechnical information that he considers necessary to develop his workscope and cost estimates with sufficient precision.

The recommendations and criteria provided in this report were based on the data presented herein, and our experience in the general project area with similar projects, and our engineering judgment with regard to the applicability of the data and methods of forecasting future performance. A variety of engineering parameters were considered as indicators of potential future soil movements. Our recommendations were based on our judgment of "likely movement potentials," (i.e., the amount of movement likely to be realized if site drainage is effective, estimated to a reasonable degree of engineering certainty) as well as our assumptions about the Owner's willingness to accept geotechnical risk. "Maximum possible" movement estimates necessarily will be larger than those presented herein. They also have a lower likelihood of being realized in our opinion, and generally require more expensive measures to address. We encourage the Owner or prospective future owners, upon receipt of this report, however, to discuss these risks and the geotechnical alternatives with us.

Engineering consulting and design practice necessarily involves weighing the risks inherent in a given design approach against the construction and maintenance costs associated with reducing those risks. The Owner (and subsequent prospective future owners) must, therefore, understand the risks and remedial approaches presented in this report (and the risk-cost trade-offs addressed by the Civil Engineer and Structural Engineer) in order to direct his Design Team to the portion of the Higher Cost / Lower Risk – Lower Cost / Higher Risk spectrum in which this project should be designed. If the Owner or a prospective future owner does not understand these risks, it is critical that he request additional information or clarification so that his expectations reasonably can be met.
This report was prepared in accordance with generally accepted soil and foundation engineering practice in the Boulder, Colorado area at the date of preparation. GROUND makes no other warranties, either express or implied, as to the professional data, opinions or recommendations contained herein.

Sincerely,

GROUND Engineering Consultants, Inc.

Serkan Sengul, P.E.

Reviewed by James B. Kowalsky, P.E.
LEGEND:

- Pavement (AP)
- Road Base (RB)
- Fill:
- Clay:
- Sand and Clay:
- Weathered Claystone:
- Claystone Bedrock:

Drive sample, 2-inch I.D. California liner sample

23/12 Drive sample blow count, indicates 23 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 12 inches.

0 Depth to water level and number of days after drilling that measurement was taken.

NOTES:

1) Test holes were drilled on 03/21/09 with 4-inch diameter continuous flight power augers.

2) Locations of the test holes were measured approximately by pacing from features shown on the site plan.

3) Elevations of the test holes were surveyed by the project surveyor (Martin/Martin).

4) The test hole locations and elevations should be considered accurate only to the degree implied by the method used.

5) The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.

6) Groundwater level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.

The material descriptions on this legend are for general classification purposes only. See the full text of this report for descriptions of the site materials and related recommendations.
COMPACATION TEST REPORT

Curve No.: 3141
Project No.: 09-3011
Project: Williams Village Phase II Pod "C" Residence Hall
Location: P1-P5
Elev./Depth: 0.5 to 3 feet
Sample No. 3141
Remarks:

MATERIAL DESCRIPTION

Description:

Classifications - USCS: s(CL-ML) AASHTO: A-4(1)
Nat. Moist. = Sp.G. =
Liquid Limit = 25 Plasticity Index = 6
% > No.4 = 5.4 % % < No.200 = 50.5 %

ROCK CORRECTED TEST RESULTS UNCORRECTED
Maximum dry density = 111.3 pcf 109.5 pcf
Optimum moisture = 16.1 % 16.9 %

Test specification:
ASTM D698 Method A Standard Compaction
Oversize correction applied to each point

100% SATURATION CURVES FOR SPEC. GRAV. EQUAL TO:
2.8
2.7
2.6

GROUND ENGINEERING CONSULTANTS, INC.
Q = 3,000 psf

Note: Design should be controlled by settlement. Estimated settlement values indicated above are based on drained conditions. If foundation materials become wet, the allowable bearing capacity will be reduced and result in larger estimated settlement. This relationship is based on footing widths of 1 to 4 feet. If the footing width is to be greater than 4 feet, we should be notified to reevaluate these recommendations.
<table>
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<th>Sample Location</th>
<th>Test Hole No.</th>
<th>Natural Moisture Content (%)</th>
<th>Natural Density (pcf)</th>
<th>Gradation</th>
<th>Percent Passing No. 200 Sieve</th>
<th>Afterberg Limits</th>
<th>Percent Swell (%)</th>
<th>Surcharge Pressure (psf)</th>
<th>Soil Suction (pF)</th>
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</table>

SD = Sample Disturbed
### TABLE 2
**SUMMARY OF LABORATORY TEST RESULTS**

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<th>Test Hole No.</th>
<th>Depth (feet)</th>
<th>Water Soluble Sulfates (%)</th>
<th>pH</th>
<th>Redox Potential (mV)</th>
<th>Sulfides Content</th>
<th>Resistivity (ohm-cm)</th>
<th>USCS Classification</th>
<th>Soil or Bedrock Type</th>
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<td>9</td>
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<td>-18</td>
<td>Trace</td>
<td>1,201</td>
<td>(CL)s</td>
<td>Sandy Clay</td>
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</tbody>
</table>

*Job No. 09-3011*
APPENDIX A

PAVEMENT SECTION CALCULATIONS
1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product
Network Administrator

Flexible Structural Design Module

Job No. 09-3021
University of Colorado at Boulder
Williams Village Phase II Pod "C" Residence Hall
Automobile-Only Parking
Full Depth Asphalt Section

Flexible Structural Design

| 18-kip ESALs Over Initial Performance Period | 73,000 |
| Initial Serviceability                     | 4.5    |
| Terminal Serviceability                     | 2.5    |
| Reliability Level                          | 85 %   |
| Overall Standard Deviation                 | 0.44   |
| Roadbed Soil Resilient Modulus             | 4,228 psi |
| Stage Construction                         | 1      |

Calculated Design Structural Number 2.64

Specified Layer Design

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<tr>
<th>Layer</th>
<th>Material Description</th>
<th>Struct Coef.</th>
<th>Drain Coef.</th>
<th>Thickness (Di)(in)</th>
<th>Width (ft)</th>
<th>Calculated SN (in)</th>
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1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHOWare Computer Software Product

Network Administrator

Flexible Structural Design Module

Job No. 09-3021
University of Colorado at Boulder
Williams Village Phase II Pod "C" Residence Hall
Automobile-Only Parking
Composite Section

Flexible Structural Design

18-kip ESALs Over Initial Performance Period 73,000
Initial Serviceability 4.5
Terminal Serviceability 2.5
Reliability Level 85%
Overall Standard Deviation 0.44
Roadbed Soil Resilient Modulus 4,228 psi
Stage Construction 1

Calculated Design Structural Number 2.64

Specified Layer Design

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<tr>
<th>Layer</th>
<th>Material Description</th>
<th>Struct Coef.</th>
<th>Drain Coef.</th>
<th>Thickness (in)</th>
<th>Width (ft)</th>
<th>Calculated SN (in)</th>
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1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Network Administrator

Flexible Structural Design Module

Job No. 09-3021
University of Colorado at Boulder
Williams Village Phase II Pod "C" Residence Hall
Driveways
Full Depth Asphalt Section

Flexible Structural Design

18-kip ESALs Over Initial Performance Period 146,000
Initial Serviceability 4.5
Terminal Serviceability 2.5
Reliability Level 85 %
Overall Standard Deviation 0.44
Roadbed Soil Resilient Modulus 4,228 psi
Stage Construction 1

Calculated Design Structural Number 2.95

Specified Layer Design

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<th>Layer</th>
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<th>Drain Coef.</th>
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<th>Width (ft)</th>
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1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Network Administrator

Flexible Structural Design Module

Job No. 09-3021
University of Colorado at Boulder
Williams Village Phase II Pod "C" Residence Hall
Driveways
Composite Section

Flexible Structural Design

18-kip ESALs Over Initial Performance Period  146,000
Initial Serviceability  4.5
Terminal Serviceability  2.5
Reliability Level  85%
Overall Standard Deviation  0.44
Roadbed Soil Resilient Modulus  4,228 psi
Stage Construction  1

Calculated Design Structural Number  2.95

Specified Layer Design

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1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Network Administrator

Flexible Structural Design Module

Job No. 09-3021
University of Colorado at Boulder
Williams Village Phase II Pod "C" Residence Hall
Heavy Traffic
Composite Section

Flexible Structural Design

18-kip ESALs Over Initial Performance Period 365,000
Initial Serviceability 4.5
Terminal Serviceability 2.5
Reliability Level 85%
Overall Standard Deviation 0.44
Roadbed Soil Resilient Modulus 4,228 psi
Stage Construction 1

Calculated Design Structural Number 3.40

Specified Layer Design

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1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Network Administrator

Rigid Structural Design Module

Job No. 09-3021
University of Colorado at Boulder
Williams Village Phase II Pod "C" Residence Hall
High Turning Stress and Heavy Traffic
Portland Cement Concrete Pavements

Rigid Structural Design

<table>
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<td>28-day Mean PCC Modulus of Rupture</td>
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<td>Mean Effective k-value</td>
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Calculated Design Thickness | 6.48 in