GEOTECHNICAL ENGINEERING STUDY
PROPOSED CENTER FOR COMMUNITY
BUILDING AND PARKING STRUCTURE
INTERSECTION OF REGENERENT DRIVE
AND WILLARD LOOP DRIVE
BOULDER, COLORADO

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SUMMARY

1. Beneath existing asphalt pavement sections and thin layers of topsoil in Borings 4 and 8, the overburden soils encountered in the borings generally consisted of sandy lean clay to lean clay fill, which extended to depths ranging from 1 to 12 feet below the ground surface. The fills were underlain by natural, stiff to very stiff, lean clay to sandy lean clay extending to claystone bedrock at depths of about 13 and 18 feet, respectively.

Bedrock encountered beneath the fill and natural lean clays consisted of medium hard to very hard claystone bedrock extending to the maximum depths explored, ranging from 10 to 60 feet below the existing ground surface.

2. Unstabilized ground water was encountered in the exploratory borings at depths ranging from about 32 to 55 feet below the ground surface the time of drilling. Stabilized ground water was measured at depths ranging from about 2.5 to 10.5 feet below the ground surface when checked between 2 to 15 days after drilling. Based on our observations at the site and conditions encountered in the borings, it is our opinion the ground water is perched above the claystone bedrock. Fluctuations in the ground-water elevation may occur with time, irrigation and/or weather events.

3. Based on the subsurface conditions encountered at the site and anticipated relatively high structural loads, we recommend the proposed structure be supported on straight-shaft drilled piers. Piers should extend 8 feet or 3 pier diameters, whichever is greater, into the bedrock. In addition, a minimum total pier length of 20 feet is recommended.

Piers should be designed for an allowable end-bearing pressure of 50,000 psf, a skin friction of 2,500 psf for the upper 5 feet of bedrock penetration, and a skin friction of 5,000 psf for the portion of the pier below a 5-foot penetration. Uplift due to structural loadings on the piers can be resisted by using 75% of the allowable skin friction value plus an allowance for pier weight.

Piers should also be designed for a minimum dead load pressure of 25,000 psf based on pier end area only.

4. Based on a design resilient modulus of 3,025 psi and an 18-kip ESAL of 36,500 psi, the pavement section for the parking areas and driveways should consist of a minimum of 6.5 inches of full-depth hot mix asphalt.
PURPOSE AND SCOPE OF WORK
This report presents the results of a geotechnical engineering study performed for the proposed Center for Community (C4C) Building to be constructed at the University of Colorado (CU) Boulder Campus. The study was conducted for the purpose of providing geotechnical engineering recommendations and considerations related to the proposed project in accordance with the scope of work presented in our Proposal No. P-08-204 dated February 27, 2008. The project site is shown on Fig. 1.

This report has been prepared to summarize the data obtained during this study 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 construction of the proposed building are presented herein.

PROPOSED CONSTRUCTION
We understand that the proposed building will be located in the area immediately south of Hallett Hall and will consist of a 2- to 4-story structure supported above 2 to 3 levels of below-grade parking. We understand that the lowest parking level will have a finished elevation of 5366 feet. A wing of the building located at the northeast corner will be constructed at grade. Although the exact footprint of the proposed structure and lower floor elevations are not known, we understand the building will have a footprint area of approximately 130,000 square feet and a total gross square footage of 257,000 square feet. Building structure type is not known at this time; however, we understand that maximum column dead loads will be about 1,300 kips and maximum column dead plus live loads will be about 2,000 kips.

If the proposed construction varies significantly from that described above or depicted in this report, we should be notified to reevaluate the recommendations provided in this report.

SITE CONDITIONS
At the time of our field investigation the site was occupied by asphalt-paved parking areas and driveways, and landscaped areas. The site is bounded by Regent Drive to the south, Willard Loop Drive to the west and north, and developed driveway, parking and landscaped areas to the
east. The site is relatively flat with a gentle slope down to the east. The maximum elevation difference across the site is approximately 25 feet.

SUBSURFACE CONDITIONS

Information on the subsurface conditions was obtained by drilling sixteen borings at the approximate locations shown on Fig. 1. The boring logs are presented on Figs. 2 through 5 along with a legend and notes for the logs on Fig. 5. Results of laboratory tests performed on selected samples from the borings are presented to the right of the logs, and are summarized in Table I.

Beneath existing asphalt pavement sections and thin layers of topsoil in Borings 4 and 8, the overburden soils encountered in the borings generally consisted of sandy lean clay to lean clay fill, which extended to depths ranging from 1 to 12 feet below the ground surface. The fills were underlain by natural, stiff to very stiff, lean clay to sandy lean clay extending to claystone bedrock at depths of about 13 and 18 feet, respectively. The fill materials were generally moist and brown, and contained occasional to frequent claystone fragments and occasional zones of fat clay (possible claystone fill) and clayey sand with gravel. The natural lean clays were moist to very moist and brown.

Bedrock encountered beneath the fill and natural lean clays consisted of medium hard to very hard claystone bedrock extending to the maximum depths explored, ranging from 10 to 60 feet below the existing ground surface. The claystone was generally slightly moist to moist and brown to olive to gray.

Unstabilized ground water was encountered in the exploratory borings at depths ranging from about 32 to 55 feet below the ground surface the time of drilling. Stabilized ground water was measured at depths ranging from about 2.5 to 10.5 feet below the ground surface when checked between 2 to 15 days after drilling. Based on our observations at the site and conditions encountered in the borings, it is our opinion the ground water is perched above the claystone bedrock. Fluctuations in the ground-water elevation may occur with time, irrigation and/or weather events.

The results of 15 swell-consolidation tests performed on selected samples of the existing fill, natural lean clay and claystone bedrock materials are presented on Figs. 6 through 13. The
results generally indicate a moderate swell potential for the fill, nil to low swell potential for the natural lean clay soils when wetted under surcharge pressures of 500 and 1,000 psf, and a low to occasionally high swell potential for the claystone bedrock when wetted under surcharge pressures of 500 and 1,000 psf.

Gradation test results for samples of the near surface sandy lean clay fill are presented on Fig. 14.

FOUNDATION RECOMMENDATIONS
Based on the subsurface conditions encountered at the site and anticipated relatively high structural loads, we recommend the proposed structure be supported on straight-shaft drilled piers.

The design and construction criteria presented below should be observed for a straight-shaft drilled pier foundation system. The construction details should be considered when preparing project documents.

1. Piers should extend 8 feet or 3 pier diameters, whichever is greater, into the bedrock. In addition, a minimum total pier length of 20 feet is recommended.

2. Piers should be designed for an allowable end-bearing pressure of 50,000 psf, a skin friction of 2,500 psf for the upper 5 feet of bedrock penetration, and a skin friction of 5,000 psf for the portion of the pier below a 5-foot penetration. Uplift due to structural loadings on the piers can be resisted by using 75% of the allowable skin friction value plus an allowance for pier weight.

3. Piers should also be designed for a minimum dead load pressure of 25,000 psf based on pier end area only. Application of dead load pressure is the most effective way to resist foundation movement due to swelling of expansive bedrock. However, if the minimum dead load requirement cannot be achieved and the piers are spaced as far apart as practical, the pier length should be extended beyond the minimum bedrock penetration and minimum length to mitigate the dead load deficit. This can be accomplished by assuming one-half of the skin friction value given above acts in the direction to resist uplift caused by swelling
bedrock near the top of the pier. The owner should be aware of an increased potential for foundation movement if the recommended minimum dead load pressure is not met.

4. Piers should be designed to resist lateral loads using a modulus of horizontal subgrade reaction of 250 tcf in the bedrock. Resistance to lateral loads should be neglected in the existing fills and natural clay soils. The modulus value given is for a one-foot wide pier and must be corrected for pier size. Alternatively, the lateral capacity of the piers may be analyzed using LPILE or COM824P computer programs and the parameters provided in the following table. The following parameters are intended for use only in the programs indicated above and may not be suitable for other applications.

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>c</th>
<th>φ</th>
<th>γ</th>
<th>k_s</th>
<th>k_c</th>
<th>C_50</th>
<th>Layer Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Properly Compacted Existing Fill and natural lean clays*</td>
<td>800</td>
<td>0</td>
<td>125</td>
<td>500</td>
<td>200</td>
<td>0.007</td>
<td>1</td>
</tr>
<tr>
<td>New Structural Fill</td>
<td>-</td>
<td>32</td>
<td>120</td>
<td>90</td>
<td>90</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>Claystone Bedrock</td>
<td>8,000</td>
<td>0</td>
<td>130</td>
<td>2,000</td>
<td>800</td>
<td>0.004</td>
<td>1</td>
</tr>
</tbody>
</table>

*Bedrock within 3 feet of the pier cap should be modeled as compacted fill.

Where,
- c - cohesion (psf)
- φ - internal friction angle (degrees)
- γ - moist unit weight (pcf)
- k_s - initial modulus of horizontal subgrade reaction (pci)
- k_c - initial modulus of horizontal subgrade reaction for cyclic loading (pci)
- C_50 - strain at 50% of peak shear strength

Layer types to be used in the program are as follows:
1. Stiff clay above the water table.
2. Granular soils above the water table

No modification of these values is required for pier size; the computer program requires pier size input.

5. Piers should be designed with additional reinforcement over their full length to resist an unfactored net tensile force from swelling soil pressure of least 80,000 pounds. Tensile forces are for a 1-foot diameter pier. For larger pier diameters, this force should be increased in proportion to the pier diameter. If the minimum dead load requirement is not met, the tensile force should be increased by the deficit between the required minimum dead load and the applied dead load. Similarly, the tensile force may be reduced if the design dead load exceeds the recommended minimum dead load. As a minimum, piers
should be reinforced with at least one No. 5 reinforced rod for each 16 inches of pier perimeter.

6. A 6-inch void should be provided beneath the grade beams to concentrate pier loadings and to separate the expansive soil and rock from the grade beams. Absence of a void space will result in a reduction in dead load pressure which could result in upward movement of the foundation system. A void should also be provided beneath necessary pier caps.

7. A minimum spacing requirement between piers of three diameters from center to center will require no reduction in axial load capacity and in horizontal soil modulus values for lateral loads applied perpendicular to the pier alignment. A minimum spacing of six pier diameters, center to center, will result in no required reduction in the pier capacity for lateral loads applied in the direction of the pier alignment. Piers grouped less than the recommended spacing should be studied on an individual basis to determine the appropriate reduction in both lateral and axial capacity.

It may be difficult to obtain the recommended pier spacings parallel to the direction of loading in small areas such as elevator pits. In such cases, we recommend the load-displacement curve (p-y curve) for an isolated pier be modified for closely spaced piers using p-multipliers to reduce all the p values on the curve. With this approach, the computed load carrying capacity of the pier in a group is reduced relative to the isolated pier capacity. The modified p-y curve should then be reentered into the LPILE or COM624P software to calculate the pier deflection. The reduction for the leading pier, the pier leading the direction of movement of the group, is less than that for the trailing piers. We recommend p-multipliers of 0.6 and 1.0 for pier spacing of 3 and 6 diameters, respectively, for the leading pier, and 0.4 and 1.0 for 3 and 6 diameter spacings, respectively, for the trailing piers. Reduction factors for spacings between 3 and 6 diameters may be obtained by linear interpolation. It will be necessary to determine the load distribution between the piers that attains deflection compatibility because the leading pier carries a higher proportion of the pier group load and the pier cap prevents differential movement between the piers.

8. Concrete used in the piers should have a slump in the range of 5 to 7 inches to fill the void between reinforcing steel and the pier hole.
9. Rock penetration in all pier holes should be provided with shear rings to assist the development of peripheral shear stress between the pier and the bedrock. Shear rings should be installed with a grooving tool in a pattern considered appropriate by the geotechnical engineer. Horizontal grooves at 1 to 2-foot centers (or helical grooves with a 1 to 2-foot pitch) are acceptable patterns for piers less than 5 feet in diameter. Horizontal grooves in piers greater than 5 feet in diameter should be 3 inches deep and 5 inches in height. Care should be taken that only the required bedrock penetration portion of the pier is provided with shear rings.

The specifications should allow the geotechnical engineer to eliminate the requirements for shear rings if it appears their installation is not beneficial. This could occur if the presence of water and/or weakly cemented materials results in degradation of pier hole during shear ring installation, or if the sides of the hole are naturally rough after drilling.

10. Based on the results of our field exploration, laboratory testing, and our experience with similar, properly constructed drilled pier foundations, we estimate pier settlement will be low. Generally, we estimate the settlement of a 2 to 4 foot diameter pier to be less than 3/4 inch when designed according to the criteria presented herein. The settlement of closely-spaced piers will be larger and should be studied on an individual basis.

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

12. The presence of water in the exploratory borings during drilling operations indicates the use of temporary casing to seal off the perched ground water above the bedrock, or dewatering equipment will be required. In no case should concrete be placed in more than 3 inches of water unless the tremie method is used. If water cannot be removed or prevented with the use of temporary casing and/or dewatering equipment prior to placement of concrete, the tremie method should be used after the hole has been cleaned.

13. The drilled shaft contractor should mobilize equipment of sufficient size and operating condition to achieve the required bedrock penetration. Although not detected in the exploratory borings, our experience with similar bedrock materials in the area indicates that localized zones of very hard bedrock could be encountered. Such zones may require the use of a core barrel or pilot hole to facilitate drilling with the required size auger.
14. Concrete should be placed in piers the same day they are drilled. The presence of water may require that concrete be placed immediately after the pier hole is completed. Failure to place concrete the day of drilling will normally result in a requirement for additional bedrock penetration immediately prior to concrete placement.

15. A representative of the geotechnical engineer should observe pier drilling operations on a full-time basis to assist in evaluation of adequate bedrock strata and monitor proper construction procedures.

SITE SEISMIC CRITERIA
The soil profile generally consists of 1 to 18 feet of primarily clay overburden soils underlain by claystone bedrock. The overburden soil materials generally classify as International Building Code (IBC) Site Class D and the claystone bedrock would classify as IBC Site Class B or C. Based on the relatively shallow depth to very hard bedrock, the design soil profile for the site may be at the borderline between IBC Site Class B or C, based on current data and procedures presented in the code. In order to establish if the site profile is IBC Site Class B, a site specific geophysical exploration will be necessary to obtain shear wave velocity data for the overburden soils and bedrock to a depth of 100 feet. In the absence of site specific geophysical data, we recommend a design soil profile for the site of IBC Site Class C be used.

If it is determined that seismic parameters based on soil profile type, or site class, will be a determining factor for structural design, it may be beneficial to perform a site specific geophysical study. We are available to discuss a supplemental geophysical study with you, if needed.

Based on the subsurface profile, site seismicity, and the anticipated depth of ground water, the subsoils are not subject to liquefaction.

FLOOR SLABS
Floor slabs present a problem where expansive materials are present near floor slab elevation because sufficient dead load cannot be imposed on them to resist the uplift pressure generated when the materials are wetted and expand. Based on the condition of the fill materials
encountered in our exploratory borings and the swell potential measured in samples of the fill and natural lean clay, it is our opinion that the existing overburden soils should not be used for support of slabs on grade. In the case of the fill, it should be entirely removed and replaced; in the case of the natural lean clays, we recommend these soils be overexcavated to a minimum depth of 3 feet below the slab level and replaced with nonexpansive, relatively impervious structural fill. However, we anticipate that in removing the existing fill materials from the site, the minimum 3-foot depth of overexcavation will be achieved across most of the building area. Local areas may require additional sub-excavation to meet the minimum depth requirement.

Because a below grade parking level is planned below the majority of the proposed structure, we anticipate that the slabs for the parking structure will be supported on the bedrock at an elevation of approximately 5366 feet. The swell potential of the claystone bedrock below this elevation is moderate to high. The most positive method to avoid damage as a result of floor slab movement is to construct a structural slab above a well-vented crawl or minimum 6-inch void space. The slab would be supported on grade beams and piers the same as the main structure.

Considering the intended use of the lowest level slab will be for parking, we believe slab-on-ground construction may be used, provided the risk of distress resulting from slab movement is accepted by the owner. In this case, we recommend below grade slabs be supported on a minimum of 4 feet of nonexpansive, relatively impervious structural fill for parking slabs and 6 feet for finished space floor slabs. Structural slabs underlain by crawl or void space should be considered for any areas sensitive to movement. For any below-grade, ground-supported slab, the perimeter underdrain should be deepened to key into the bedrock at least one foot below the elevation of the slab fill overexcavation.

Even with subexcavation and select underslab fill replacement, there would still be some risk of slab movement. To illustrate this risk, we have performed calculations, which are summarized in the following table, to demonstrate the potential for ground heave if the select fill zone and underlying bedrock should be thoroughly wetted to significant depth.
The heave estimate calculations demonstrate that significant heave should be expected if wetting of the bedrock underlying the slabs occurs to significant depth below the bottom of the prepared fill zone. However, our experience indicates that the large majority of similar structures underlain by similar subsoils and bedrock do not experience extreme moisture increases in the underlying soils and bedrock to significant depth provided that good surface and subsurface drainage is designed, constructed, and maintained, and that good irrigation practices are followed. Wetting can also occur as a result of unforeseeable influences such as plumbing leaks or breaks, or in some cases even due to off-site influences depending on geologic conditions. The intent of our recommendations is to provide for a condition where there is a good chance slab heave movements will not exceed 1 inch and it is unlikely they will exceed 2 inches unless extreme wetting is allowed. The following measures should be taken to reduce damage which could result from movement should the underslab materials be subjected to moisture changes.

1. Slabs should be separated from all bearing walls and columns with expansion joints which allow unrestrained vertical movement.

2. Interior non-bearing partitions resting on floor slabs should be provided with slip joints at the tops so that, if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards, stairways and door frames. Slip joints which will allow at least 2 inches of vertical movement are recommended.

If wood or metal stud partition walls are used, the slip joints should preferably be placed at the bottoms of the walls so differential slab movement won't damage the partition wall. If slab bearing masonry block partitions are constructed, the slip joints will have to be placed at the tops of the walls. If slip joints are provided at the tops of walls and the
floors move, it is likely the partition walls will show signs of distress, such as cracking. An alternative, if masonry block walls or other walls without slip joints at the bottoms are required, is to found them on grade beams and piers and to construct the slabs independently of the foundation. If slab bearing partition walls are required, distress may be reduced by connecting the partition walls to the exterior walls using slip channels).

Floor slabs should not extend beneath exterior doors or over foundation grade beams, unless saw cut at the beam after construction.

3. Floor slab control joints should be used to reduce damage due to shrinkage cracking. Joint spacing is dependent on slab thickness, concrete aggregate size, and slump, and should be consistent with recognized guidelines such as those of the Portland Cement Association (PCA) or American Concrete Institute (ACI). The joint spacing and slab reinforcement should be established by the designer based on experience and the intended slab use.

4. A minimum 4-inch layer of free-draining gravel should be placed beneath the slabs. This material should consist of minus 2-inch aggregate with less than 30% passing the No. 4 sieve and less than 5% passing the No. 200 sieve. The granular layer will help distribute concentrated loads and will be incorporated into the building underdrain system.

5. If moisture-sensitive floor coverings will be used, mitigation of moisture penetration into the slabs, such as by use of a vapor barrier, may be required. If an impervious vapor barrier membrane is used, special precautions will be required to prevent differential curing problems which could cause the slabs to warp. This topic is addressed by ACI 302.1R.

6. New structural fill placed below slabs should meet the following requirements:

<table>
<thead>
<tr>
<th>Percent Passing No. 200 Sieve</th>
<th>Minimum 25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit</td>
<td>Maximum 30</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>Maximum 10</td>
</tr>
</tbody>
</table>
Fill source materials not meeting the above liquid limit and plasticity index criteria may be acceptable (provided the minimum percentage passing the No. 200 sieve is satisfied) if the swell potential when remolded to 95% of the ASTM D 698 (standard Proctor) maximum dry density at optimum moisture content under a 200 psf surcharge pressure does not exceed 1%. Evaluation of potential sources would then require determination of laboratory moisture-density relationships and swell consolidation tests on remolded samples, thereby adding time and cost to evaluate proposed fill materials.

7. The geotechnical engineer should evaluate the suitability of proposed fill materials prior to use. Sub-slab structural fill should be placed and compacted to at least 95% of the ASTM D 698 (standard Proctor) maximum dry density within 2 percentage points of the optimum moisture content.

8. The existing fill, natural clays and claystone bedrock encountered during this study will be expansive when placed in a compacted condition. Consequently, it should not be used as fill beneath floor slabs.

10. All plumbing lines should be tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided. Flexible connections should be provided for slab-bearing mechanical equipment.

EXCAVATION AND GRADING CONSIDERATIONS

Excavation for the facility is expected to encounter existing clay fills, natural lean clays and medium hard to very hard claystone bedrock. In our opinion, excavation of the bedrock and existing fills and natural soils should be possible with conventional, heavy-duty earth-moving equipment. Some of the excavated existing fills may be suitable for use as wall backfill; the excavated claystone bedrock will not be suitable for use as fill.

Depending on the layout of the proposed facility, it may be feasible for the facility excavation to be constructed by over-excavating the side slopes to stable configurations where enough space is available. However, temporary retaining structures, in combination with temporary excavation slopes, may be required where insufficient lateral space exists to lay back slopes, such as in the vicinity of the existing Hallet Hall, adjacent buried utilities, and along areas of flatwork and pavement that must remain in use during construction.
Temporary Excavations: All excavations 20 feet deep or less should be constructed in accordance with OSHA requirements, as well as state, local and other applicable requirements. The existing fills generally classify as OSHA Type C soils, and the natural lean clay and claystone bedrock generally classify as OSHA Type A materials. Excavations below groundwater, if encountered, in Type C soils may require significantly flatter side slopes or temporary shoring. A representative of the geotechnical engineer should evaluate excavations deeper than 20 feet.

Excavated slopes in on-site fills are considered susceptible to erosion from surface runoff. Measures to keep surface runoff from excavation slopes, including diversion berms, should be considered.

Temporary Retaining Structures: Temporary retaining structures include cantilevered systems and anchored or tied-back systems. Generally, cantilevered sheet pile walls can be used for temporary excavations up to about 10 to 15 feet in height. Temporary excavations up to 15 to 20 feet in height may be supported by cantilevered structures consisting of closely spaced drilled caissons, or micropiles or cantilevered steel soldier piles with wood lagging. Caissons typically consist of a mixed-in-place concrete slurry with a reinforcing cage.

Design of temporary retaining structures is typically the responsibility of the contractor or the earthwork subcontractor. Design of temporary retaining structures at the site must consider the expansive nature of the claystone bedrock, potential groundwater seepage, global stability of retained cuts, and the stability of exposed slopes, where present, above the retaining structures. Design must include any surcharge due to traffic, storage of materials, adjacent structures, etc. Temporary shoring provided in close proximity to existing pavements or structures, should be stiff to prevent movement of those pavement or structure foundations. Subsequent to design, we should be contacted to review and comment on the configuration of temporary retaining structures.

During construction, periodic review of site retaining systems should be undertaken to monitor their performance. In addition, a pre-construction survey should be performed to assess the existing condition of adjacent structures and areas of existing flatwork and pavement that could
be impacted by shoring movement. Contingency plans and materials should be immediately available in case excessive movement of the shoring occurs.

Excavation Dewatering: Because of the high ground water levels measured at the site, we believe construction-phase dewatering will likely be required once facility excavations advance below the ground-water elevation. For excavations extending only a few feet below the ground water level, we believe ground water flow into the excavation can be controlled by using sumps and drainage ditches within the excavation. Ground water encountered at greater depths may require perimeter cut-off trenches tied to sumps.

FOUNDATION WALLS AND SHORING SYSTEMS
At the time of this report, the construction approach for site excavations and the basement walls of the below-ground levels was not known. Permanent basement walls should be designed for the lateral earth pressure generated by the backfill, or in the case of shoring, the properties of the material behind the shoring, which is a function of the degree of rigidity of the retaining structure and the type of backfill material used. Retaining structures such as basement walls that are laterally supported and can be expected to undergo only a moderate amount of deflection should be designed for earth pressures based on the following equivalent fluid densities:

CDOT Class 1 (<20% passing No. 200 sieve) .................. 50 pcf
On-site lean clay/clayey sand backfill (PI<30) ............... 65 pcf

Cantilevered retaining structures that can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for earth pressures based on the following equivalent fluid densities:

CDOT Class 1 (<20% passing No. 200 sieve) .................. 40 pcf
On-site lean clay/clayey sand backfill (PI<30) ............... 55 pcf

Existing on-site clay fills may be used as wall backfill provided they have a maximum particle size of two inches and contain no organic material, debris, claystone fragments, or other deleterious materials.
The equivalent fluid density values recommended above assume drained conditions behind the walls and retaining structures, and a horizontal backfill surface. The buildup of water behind a wall or retaining structure, or an upward sloping backfill surface, will increase the lateral earth pressure imposed on the wall or retaining structure. Below-grade walls and retaining structures should also be designed for appropriate surcharge pressures due to adjacent structures, vehicle traffic, and construction activities.

The zone of backfill placed behind retaining structures to within 2 feet of the ground surface should be sloped upward from the base of the wall at an angle of no steeper than 45 degrees from horizontal. The upper 2 feet of the wall backfill should consist of a relatively impervious on-site soil or a slab or pavement structure to reduce surface water infiltration into the backfill. Backfill should be placed in uniform lifts and compacted to at least 95% of the standard Proctor (ASTM D 698) maximum dry density. Granular backfill materials should be compacted at a moisture content within 2 percentage points of optimum moisture content. Clayey backfill materials should be compacted at a moisture content within 0 to 3 percentage points above optimum moisture content. Care should be taken not to over-compact the backfill since this could cause excessive lateral pressure on the walls. Hand compaction procedures, if necessary, should be used to prevent lateral pressures from exceeding the design values. Some settlement of deep foundation wall backfill should be expected to occur even if the material is placed properly.

The lateral resistance of the foundation walls will be a combination of the lateral capacity of the drilled pier foundations and passive earth pressure against the embedded portion of the wall. Lateral pier capacity recommendations are provided in the Foundation Recommendations section of this report. Passive pressure resistance for permanent basement walls and incidental retaining walls should be calculated based on an allowable equivalent fluid unit weight of 200 pcf for select granular backfill, and 175 pcf for lean clay or clayey sand backfill. These are working values.

BURIED METAL CORROSION

The potential corrosive environment for metal placed beneath the ground surface at the site was evaluated based on data collected during our field exploration program and criteria presented in the Handbook of Steel Drainage and Highway Construction Products published by the American Iron and Steel Institute. The data includes soil group classification and electrical resistivity.
The materials encountered in the borings predominantly consist of lean clay fill and claystone bedrock having poor aeration and drainage, with water contents generally less than 20%. The general characteristics of the soils and bedrock indicate they are badly corrosive.

Laboratory electrical resistivity measurements conducted on two samples of the clay fills indicate resistivity values of between 720 and 910 ohm-centimeters at the in-situ moisture content. The laboratory test results are presented on Fig. 15. Soil resistivities of less than 2,000 ohm-centimeters are classified as having bad corrosion resistance.

The data obtained in our study indicates the subsurface conditions will be strongly aggressive towards iron and other buried metals based on a scale published in "Corrosion of Building Materials" by Dictbert Knofel. We recommend a qualified corrosion engineer review the information presented above to design an appropriate level of corrosion protection for buried metal.

WATER SOLUBLE SULFATES

The concentration of water soluble sulfates measured in samples obtained from the exploratory borings was less than 0.02%. This concentration of water soluble sulfates represents a negligible degree of sulfate attack on concrete exposed to these materials. The degree of attack is based on a range of negligible, moderate, severe and very severe as presented in the 2006 International Building Code. Based on this information, we believe special sulfate resistant cement will not be required for concrete exposed to the on-site soils.

UNDERDRAIN SYSTEM

The below-ground level of the proposed facility is anticipated to extend to an elevation below the site ground water and should either be protected by an underdrain system or be waterproofed and designed to resist hydrostatic uplift. Designing the structure to resist hydrostatic uplift is probably the lower risk alternative, but could be relatively complicated and costly. If an underdrain system is used, the owner should be aware of the risk of damage to the structure should the underdrain system become inoperative during the life of the building.
The design of an underdrain system should consider the consequences of hydrostatic uplift if the permanent dewatering system fails. Depending on the use of the lower floor level, hydrostatic uplift pressure could be avoided by providing blowout plugs in the floor of the lowest level to equalize the water pressure on the inside and outside of the structure should the water level rise above the floor elevation. Alternatively, if a crawl space is used, and depending on dewatering rates, the crawl space may be used as an emergency detention area for seepage accumulation.

An underdrain system should consist of a layer of free-draining granular material beneath the slab/floor connected to perimeter and lateral drains. If a crawl space is used, a gravel layer may not be required if the crawl space is sloped to the perimeter and lateral drains. Free-draining granular material used in the drain system should contain less than 5% passing the No. 200 sieve, less than 30% passing the No. 4 sieve and have a maximum size of 2 inches.

The drain lines should consist of perforated, rigid plastic pipe placed in a trench and surrounded with free-draining granular material wrapped in a geotextile filter fabric. The perforated drain pipes themselves should not be wrapped in geotextile due to the potential for clogging of the geotextile. If the drain trenches are not wrapped with a filter fabric, the granular material should then meet graded filter criteria and the drain pipe should be factory slotted in accordance with graded filter criteria. The free-draining backfill should extend over the drain pipe and to the surface of the crawlspace or to the base of the overexcavated zone.

The drain lines should be placed at least 1 foot below the surface of the crawl or void space and graded to a sump or sumps at a minimum slope of ½ percent where water can be removed by pumping or gravity flow. The drainage zone behind the foundation walls should discharge into the perimeter drain. The number and spacing of the lateral drains should be determined once the facility dimensions have been finalized. An over-designed pump capacity is desirable in the event that ground water or other subsurface conditions change. We also believe that standby pump capacity and standby generators should be provided in the event of pump or energy failure.

SURFACE DRAINAGE

Proper surface drainage is very important for acceptable performance of the building structure during construction and after the construction has been completed. Drainage recommendations
provided by local, state and national entities should be followed based on the intended use of the building. The following recommendations should be used as guidelines and changes should be made only after consultation with the geotechnical engineer.

1. Excessive wetting or drying of the foundation and slab subgrade should be avoided during construction.

2. Exterior backfill should be placed and compacted in accordance with the recommendations presented in the "Foundation Walls" section of this report.

3. The ground surface surrounding the exterior of the building should be sloped to drain away from the foundation in all directions. We recommend a minimum slope of 12 inches in the first 10 feet in unpaved areas. A minimum slope of 3 inches in the first 10 feet is recommended in the paved areas. These slopes may be changed as required for handicap access points in accordance with the Americans with Disabilities Act. Site drainage beyond the 10-foot zone should be designed to promote runoff and reduce infiltration. If used, drainage swales should be located no closer than 10 feet from building.

4. Ponding of water should not be allowed in backfill material or in a zone within 10 feet of the foundation walls whichever is greater.

5. Roof downspouts and drains should discharge well beyond the limits of all backfill.

6. Landscaping which requires relatively heavy irrigation and lawn sprinkler heads should be located at least 10 feet from foundation walls. Irrigation schemes are available which allow placement of lightly irrigated landscape near foundation walls in moisture sensitive soil areas. Drip irrigation heads with main lines located at least 10 feet from the foundation walls are acceptable provided irrigation quantities are limited. Excessive landscape irrigation should be avoided within 10 feet of the foundation walls.

7. Plastic membranes should not be used to cover the ground surface adjacent to foundation walls.
PAVEMENT DESIGN

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. Soils are represented for pavement design purposes by means of a soil support value for flexible pavements and a modulus of subgrade reaction for rigid pavements. Both values are empirically related to strength.

Pavement design procedures are based on strength properties of the subgrade and pavement materials assuming stable, uniform conditions. Certain soils, such as those encountered on this site, are potentially expansive and require additional precautions be taken to provide for adequate pavement performance. Expansive soils are problematic only if a source of water is present. If those soils are wetted, the resulting movements can be large and erratic. Therefore, pavement design procedures address expansive soils only by assuming they will not become wetted. Proper surface drainage is essential for adequate performance of pavement on these soils.

Subgrade Materials: Based on the results of the field and laboratory studies, the subgrade materials at the site classify as A-7-6 with group indices between 13 and 39 in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification. Soils classifying as A-7-6 would generally be considered to provide poor subgrade support. The R-Value of the subgrade materials was not measured; however, based on our experience, an R-value of 5 is generally representative for soils classifying as A-7-6. Based on CDOT correlation equations an R-value of 5 relates to a resilient modulus, $M_R$, value of 3,025 psi. Therefore, for design purposes an $M_R$ value of 3,025 psi was selected for the pavement section alternatives.

Design Traffic: Because anticipated traffic loading information was not available at the time of report preparation, an 18-kip equivalent single axle loading (ESAL) value of 36,500 was assumed for the parking areas and driveways at the site. The ESAL value was based on our past experience with similar facilities of this nature. We should be notified to reevaluate pavement thickness requirements if traffic will be different from the information indicated above.
Pavement Design: Based on a design resilient modulus of 3,025 psi and an 18-kip ESAL of 36,500 psi, the pavement section for the parking areas and driveways should consist of a minimum of 6.5 inches of full-depth hot mix asphalt.

Our experience indicates full-depth asphalt sections generally perform better on clayey subgrades than combined asphalt/aggregate base course sections. The reasons for the better performance of full-depth asphalt are not fully understood. However, the use of aggregate base course provides a pervious layer above a relatively impervious subgrade. The base course can transmit water causing changes in moisture content within the subgrade materials. Variations in the subgrade moisture content can be erratic and result in erratic volume changes which cause premature deterioration of the pavement. In addition, the thinner asphalt surface of a combined section can more easily allow water to penetrate through cracks and migrate through the aggregate base course. High moisture contents in the subgrade or base course will result in loss of strength.

Truck loading dock areas and other areas where truck turning movements are concentrated should be paved with 6 inches of Portland cement concrete. Concrete pavements may also be preferred for lower parking levels. The concrete pavement should contain sawed or formed joints to ¼ of the depth of the slab at a maximum distance of 15 feet on center.

Subgrade Preparation: If pavements are placed on the existing fill materials natural clays or claystone, we recommend at least 2 feet of the subgrade be removed, moisture conditioned and recomposted. Prior to placing the pavement section, the entire subgrade area should be scarified to a depth of 8 inches, adjusted to a moisture content near optimum and compacted to 95% of the maximum standard Proctor density. The pavement subgrade should be proofrolled with a heavily loaded pneumatic-tired vehicle. Pavement design procedures assume a stable subgrade. Areas which deform excessively under heavy wheel loads are not stable and should be removed and replaced to achieve a stable subgrade prior to paving.

Drainage: The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of pavement. Drainage design should provide for the removal of water from paved areas and prevent the wetting of the subgrade soils.
DESIGN AND CONSTRUCTION SUPPORT SERVICES

Kumar & Associates, Inc. should be retained to review the project plans and specifications for conformance with the recommendations provided in our report. We are also available to assist the design team in preparing specifications for geotechnical aspects of the project, and performing additional studies if necessary to accommodate possible changes in the proposed construction.

We recommend that Kumar & Associates, Inc. be retained to provide observation and testing services to document that the intent of this report and the requirements of the plans and specifications are being followed during construction, and to identify possible variations in subsurface conditions from those encountered in this study so that we can re-evaluate our recommendations, if needed.

LIMITATIONS

This study has been conducted in accordance with generally accepted geotechnical engineering practices in this area for exclusive use by the client for design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings at the locations indicated on Fig. 1, and the proposed type of construction. This report may not reflect subsurface variations that occur between the exploratory borings, and the nature and extent of variations across the site may not become evident until site grading and excavations are performed. If during construction, fill, soil, rock or water conditions appear to be different from those described herein, Kumar & Associates, Inc. should be advised at once so that a re-evaluation of the recommendations presented in this report can be made. Kumar & Associates, Inc. is not responsible for liability associated with interpretation of subsurface data by others.

TSB/mg
cc: book, file
LEGEND:
BUILDING LAYOUT SHOWN BASED ON CONCEPTUAL PLAN PROVIDED BY DAVIS PARTNERSHIP DATED 4-07-08.
LEGEND

- --- --- APPROXIMATE FINISH FLOOR ELEVATION OF PARKING GARAGE.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED BETWEEN APRIL 14 AND 18, 2008 WITH A 4-INCH DIAMETER CONTINUOUS FLIGHT POWER AUGER.

2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY TYPING FROM FEATURES SHOWN ON THE SITE PLAN PROVIDED.

3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE OBTAINED BY INTERPOLATION BETWEEN CONTOURS ON THE SITE PLAN PROVIDED.

4. THE EXPLORATORY BORING LOCATIONS AND ELEVATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.

6. GROUND WATER LEVELS SHOWN ON THE LOGS WERE MEASURED AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.

7. LABORATORY TEST RESULTS:
   - MC = WATER CONTENT (%) (ASTM D 2216);
   - DD = DRY DENSITY (pcf) (ASTM D 2216);
   - +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D 422);
   - −200 = PERCENTAGE PASSING NO. 200 SIEVE (ASTM D 1140);
   - LL = LIQUID LIMIT (ASTM D 4318);
   - PI = PLASTICITY INDEX (ASTM D 4318);
   - RES = MINIMUM LABORATORY RESISTIVITY (ohm-cm.) (ASTM G 57);
   - WSS = WATER SOLUBLE SULFATES (%) (AASHTO T 290);
   - A–2–6 (S) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145).
SAMPLE OF: Claystone
FROM: Boring 2 @ 24'
WC = 12.5%, DD = 109.3 pcf
-200 = 94%, LL = 40, PI = 23

EXPANSION UNDER CONSTANT PRESSURE UPON WETTING

CONSOLIDATION - SWELL (%)

APPLIED PRESSURE - KSF

100

10

.1

SAMPLE OF: Claystone
FROM: Boring 3 @ 19'
WC = 14.5%, DD = 106.2 pcf
-200 = 97%, LL = 43, PI = 24

EXPANSION UNDER CONSTANT PRESSURE UPON WETTING

CONSOLIDATION - SWELL (%)

APPLIED PRESSURE - KSF

100

10

.1

These test results apply only to the samples tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Swell Consolidation testing performed in accordance with ASTM D-4546.
SAMPLE OF: Claystone
FROM: Boring 3 @ 34'
WC = 13.1%, DD = 112.9 pcf
-200 = 96%, LL = 42, PI = 24

EXPANSION UNDER CONSTANT PRESSURE UPON WETTING

CONSOLIDATION - SWELL (%)

APPLIED PRESSURE - KSF

10 100

1 2

0 1

-1 -2

-3 -4

These test results apply only to the sample tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Swell consolidation testing performed in accordance with ASTM D-4546.
SAMPLE OF: Claystone
FROM: Boring 6 @ 19'
WC = 13.8%, LL = 42, PI = 23

CONSOLIDATION - SWELL

EXPANSION UNDER CONSTANT PRESSURE UPON WETTING

-200 = 99%

APPLIED PRESSURE - KSF

0.1 10 100

614.4x792.0

SAMPLE OF: Claystone
FROM: Boring 7 @ 19'
WC = 11.5%, DD = 116.8 pcf
-200 = 86%, LL = 41, PI = 22

CONSOLIDATION - SWELL

EXPANSION UNDER CONSTANT PRESSURE UPON WETTING

0.1 10 100

These test results apply only to the sample tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Swell consolidation testing performed in accordance with ASTM D-4546.

08-1-227 Kumar & Associates SWELL-CONSOLIDATION TEST RESULTS Fig. 8
SAMPLE OF: Claystone
FROM: Boring 7 @ 39'
WC = 11.9%, DD = 107.5 pcf
-200 = 95%, LL = 48, PI = 31

Expansion under constant pressure upon wetting

These test results apply only to the sample tested. The testing report
shall not be reproduced, except in full, without the written approval of
Kumar and Associates, Inc. Swell
Consolidation testing performed in accordance with ASTM D-4546.
SAMPLE OF: Claystone
FROM: Boring 12 @ 19'
WC = 14.3%, DD = 116.1 pcf
\(-200 = 98\%, \text{ LL } = 47, \text{ PI } = 28\)

EXPANSION UNDER CONSTANT PRESSURE UPON WETTING

SAMPLE OF: Claystone
FROM: Boring 12 @ 39'
WC = 15.0%, DD = 112.5 pcf
\(-200 = 99\%, \text{ LL } = 43, \text{ PI } = 25\)

NO MOVEMENT UPON WETTING

These test results apply only to the samples tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Swell Consolidation testing performed in accordance with ASTM D-4549.
SAMPLE OF: Fill: Sandy Lean Clay
FROM: Boring 13 @ 1'
WC = 14.2%, DD = 112.4 pcf
-200 = 59%, LL = 40, PI = 23

EXPANSION UNDER CONSTANT PRESSURE UPON WETTING

CONSOLIDATION - SWELL (%)

APPLIED PRESSURE - KSF

10 100

SAMPLE OF: Lean Clay
FROM: Boring 13 @ 9'
WC = 23.9%, DD = 100.9 pcf
-200 = 87%, LL = 46, PI = 29

EXPANSION UNDER CONSTANT PRESSURE UPON WETTING

CONSOLIDATION - SWELL (%)

APPLIED PRESSURE - KSF

10 100
SAMPLE OF: Lean Clay with Sand
FROM: Boring 13 @ 14'
WC = 25.5%, DD = 94.8 pcF
-200 = 85%, LL = 40, PI = 26

NO MOVEMENT UPON WETTING

CONSOLIDATION - SWELL (%)

10 APPLIED PRESSURE - KSF

SAMPLE OF: Claystone
FROM: Boring 13 @ 19'
WC = 13.8%, DD = 118.3 pcF
-200 = 98%, LL = 41, PI = 26

EXPANSION UNDER CONSTANT PRESSURE UPON WETTING

CONSOLIDATION - SWELL (%)

10 APPLIED PRESSURE - KSF
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<th>SAND (%)</th>
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