Subsurface Exploration Program and Geotechnical Recommendations
Fruita Recreation Center
Fruita, Colorado

Prepared for:

City of Fruita Engineering Department
325 E. Aspen Street
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Attention: Mr. Kenneth Haley

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a subsurface exploration program to provide geotechnical recommendations for the City of Fruita's proposed recreation center facility to be constructed east of Coulson Street, and south of Ottley Avenue in Fruita, Colorado. Our work was performed in general accordance with GROUND's Proposal No. 0903-0368, dated March 24, 2009.

Field and office studies provided information regarding surface and subsurface conditions. Material samples retrieved during the subsurface exploration were tested in our laboratory to assess the engineering characteristics of the site earth materials, and assist in the development of our geotechnical recommendations. Additional information was obtained from various published geologic maps and reports. Results of the field, office, and laboratory studies are presented in this report.

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 engineering considerations related to construction of the proposed facility are included herein. The several appendices following the text of this report provide significant additional discussion, observation and geotechnical recommendations relevant to the project on the subject site. They comprise an integral part of this report.

During the course of design and construction of this project, various ancillary structures may be designed and installed that may not be addressed directly by the geotechnical recommendations herein. The author of this report and/or the reviewing engineer should be contacted directly to provide additional geotechnical recommendations, as necessary.

PROPOSED CONSTRUCTION

We understand that present plans call for construction of an approximately 54,000-square foot, one- to two-story recreation center building surrounded by driveways, parking areas and limited landscaping. No below-grade (basement) levels are planned at this time other than the swimming pool that likely will extend approximately 5 feet below existing grades and possibly more. We anticipate that building loads will be low to moderate, typical of such structures. Underground utility laterals will be installed to service the new facility.

Because the site already has been graded, we anticipate only limited cuts and fills, likely on the order of 2 feet or less in thickness, to construct the building pad and drive lanes. Somewhat greater depths of excavation and backfilling will be required to install utility lines and the swimming pool, as well as comply with the remedial grading measures discussed herein.

If proposed grading, building construction or loadings are different than as described, GROUND should be contacted to re-evaluate the recommendations in this report.

SITE CONDITIONS

Topography and Surface Conditions The approximately 5-acre commercial site occupied a central portion of the block bounded by Coulson and Cherry Streets, and Ottley and Pabor Avenues. At the time of this evaluation, the site sloped gently to the west. Overall elevation differential across the site was estimated to be about 4 feet.

The ground surface was covered with fine to medium gravel, apparently as a parking lot wearing surface.

A commercial building was under construction to the north of the site. An older, single-family residence and a playground occupied the ground to the south.

Geologic Setting Published geologic maps, e.g., Tweto (1979)[1], depict the higher portions of the site as underlain by relatively recent alluvial (stream-laid) soils underlain by the upper Cretaceous Mancos Shale formation. In the project area, the Mancos Shale consists largely of silt to clay shales that commonly are moderately to highly expansive. Thin sandstone beds are interbedded locally with the shales. The bedrock strata typically dip northeastward at shallow angles.

SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted in April, 2009. A total of 11 test holes were drilled with a truck-mounted, continuous flight power auger rig to evaluate the subsurface conditions as well as to retrieve soil samples for laboratory testing and analysis. Nine of the test holes were advanced to depths of approximately 30 to 50 feet within the general proposed building footprint. The remaining two test holes were drilled to shallower depths in the areas proposed for pavements. A

[1] Tweto, O., 1979, *Geologic Map of Colorado*, U.S. Geological Survey.

GROUND engineer directed the subsurface exploration, logged the test holes in the field, and prepared the soil samples for transport to our laboratory.

Samples of the subsurface materials were retrieved with a 2-inch I.D. California liner sampler and a 1⅜-inch I.D. Standard Penetration Test sampler. The samplers were driven into the substrata with blows from a 140-pound, automatic hammer, falling 30 inches, in the case of the Standard Penetration Test sampler, in general accordance with ASTM Method D1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of soils and bedrock. A composite, bulk sample of the soils from the pavement areas was collected, as well. Depths at which the samples were obtained 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, 3 & 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 moisture contents, dry unit weights, grain size analysis and liquid and plastic limits. Consolidation, water-soluble sulfate content, and corrosivity tests were performed on selected samples, as well. Resilient modulus testing was performed on the composite sample collected from the pavement test holes. Laboratory tests were performed in general accordance with applicable ASTM and AASHTO protocols. Data from the laboratory-testing program are summarized on Tables 1 and 2.

SUBSURFACE CONDITIONS

In general, beneath the layer of surfacing gravel placed on the existing parking lot, the test holes penetrated a layered sequence of native alluvial soils that extended to depths of about 48 to 49 feet. The alluvial soils were underlain by clay shale bedrock interpreted to be Mancos Shale. Groundwater was encountered at depths of about 6½ to 8 feet below existing grades.

The sequence of earth materials can be generalized as follows:

- An upper layer of loose, silty to clayey sands extending to depths of about 4 to 8 feet.
- A layer of soft, saturated clays extending to depths of about 26 to 31 feet in the lower 5 to 10 feet of which silty to clayey sands were common.
- A layer of gravels and cobbles extending to depths of 48 to 49 feet that was relatively sandy in the upper portion and became coarser with depth to include at least scattered boulders in the lower portion.
- Hard clay shales of the Mancos Shale formation.

Other than the surficial gravels, fill soils were not recognized in the test holes. Because the site may have been graded previously, fill soils likely are present at least locally. Delineating the complete lateral and vertical extents of fills at the site, or their compositions, was beyond our present scope of services. If fill soil volumes and compositions at the site are of significance, the contractor should evaluate them using shallow test pits.

Sands and Clays consisted of silty to clayey sands with local beds and lenses of sandy silts and clays. The sands were fine to fine-to-medium. They were moist to wet, low to moderately plastic, loose / soft and red-brown in color.

Clays consisted of low to high plastic clays with local silts. The were clean to slightly sandy, wet, very soft to medium stiff, and red-brown to red in color.

Gravels and Cobbles ranged from sandy gravels to cobbles, generally coarsening downward. Boulders likely are present in the lowest 5 to 10 feet of the section. They were clean to silty, wet, low plastic to non-plastic, medium dense to dense, and light gray to gray in color.

Clay Shale Bedrock was clean to slightly sandy, very moist, highly plastic, very hard and dark gray in color.

Groundwater was encountered in the test holes at depths of about 6½ to 8 feet below existing grades at the time of drilling. When re-checked the next day, the water levels were similar. Groundwater levels can fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage and land use, and the development and drainage of transient, perched water conditions.

Time Consolidation Testing of samples of the soils and bedrock encountered in the test holes suggested low to moderate potentials for post-construction heave (See Table 1). Slight consolidations from less than 1 percent to about 2.7 percent were measured upon wetting under surcharges of 1,000 psf. The loose/soft shallow sands and clays are estimated to have a vertical hydraulic conductivity of approximately 10^{-4} to 10^{-5} cm/sec. The underlying soft clays are estimated to have a vertical hydraulic conductivity of approximately 10^{-5} to 10^{-6} cm/sec. We consider the shallow soils at the site to be Hydrologic Soil Group D materials.

GEOTECHNICAL CONSIDERATIONS FOR DESIGN

The shallow native soils were loose / soft and subject to consolidation under additional loading. As discussed in *Appendix A*, we estimate that vertical movements of 2 to 5 inches are likely where building elements or other improvements are supported directly on the site soils. Groundwater was relatively shallow and below grade construction deeper than about 5 feet likely will encounter wet soils and/or groundwater. These conditions, if not mitigated, can affect nearly all improvements at the site. Supporting structures that are not tolerant of settlements at depth are the principal geotechnical design considerations for the site. Specific geotechnical recommendations regarding these considerations are provided in subsequent sections of this report. Additional discussion and information regarding these recommendations and the geotechnical risks that they address are provided in *Appendix A*.

SEISMIC CLASSIFICATION

The project area falls within Seismic Performance Category A based on AASHTO guidelines, and is considered to have a low probability for large, damaging earthquakes. We consider it likely that the site would meet the parameters of a Seismic Site Class D site, in accordance with 2003/2006 IBC, based on extrapolation of available data to depth. If a quantitative assessment of the classification is needed, however, shear wave velocity testing will be required. A proposal for this additional service can be provided upon request. Compared with other regions of Colorado, recorded earthquake frequency in the project area is relatively low.

FOUNDATION SYSTEMS

Because of the soft alluvial soils underlying the site, GROUND recommends that the recreation center building (and other structures not tolerant of the settlements discussed in *Appendix A*) be supported on deep foundations. Geotechnical parameters for several types of deep foundations are provided below.

It should be noted that all depths indicated herein refer to depths below existing grades at the time of subsurface exploration. Likewise, the test hole elevations (estimated from a plan prepared by others) indicate the ground surface elevations at the time of subsurface exploration. The contractor should evaluate any grade changes between that time and the time of foundation installation, and adjust his anticipated installation depths, etc., accordingly.

Concrete-Filled Pipe Pile Foundations The following parameters are provided for a concrete-filled, steel, pipe pile foundation system. We assume that 8 or 10-inch diameter pipe piles will be used in construction of the addition foundations. Post-construction settlements of a properly designed and installed concrete-filled, pipe pile foundation system are estimated to be on the order of ½ inch or less.

1) The maximum pile load should not exceed a maximum service stress of 12,000 psi based on the steel pile cross-sectional area.

2) We estimate that piles will be driven to depths of more than 45 feet below existing grades into the dense gravels and cobbles or the underlying Mancos Shale to achieve capacity. Final pile depths should be based on the structural loads and the subsurface conditions encountered, however. Due to the variability in depth to the dense sands and gravels layer encountered during subsurface exploration, the contractor should anticipate advancing the pipe piles to at least 50 feet and be prepared to drive them an additional 5 to 10 feet. The actual depth of piles and penetration will depend of the size of pile used and the driving conditions encountered. Advancing one or more test piles and analyzing it/them with pile driving analysis equipment would provide a better estimate of final pile depths.

3) We anticipate that post-installation down-drag on the piles will be moderate. An adhesion coefficient of 0.1 appears appropriate for use in the overburden silts, clays and sands to estimate down-drag. However, we also recommend re-striking of the piles to evaluate their capacity at least 24 hours after (initial) driving has been completed.

4) Groups of piles required to support concentrated loads will require an appropriate reduction of the estimated bearing capacity based on the effective envelope area of the pile group.

Reduction of axial capacity can be avoided by spacing piles to a distance of at least 3 'diameters' center to center. Pile groups spaced less than 3 diameters center to center should be studied on an individual basis to determine the appropriate axial capacity reduction(s).

To avoid reduction of the capacity of piles to resist the component of lateral loading parallel to the line connecting the pile centers, piles should be spaced at least 6 diameters apart. Groups of piles spaced less than 6 diameters center to center should be studied to determine the appropriate lateral capacity reduction(s).

5) Lateral resistance to horizontal forces can be resisted by battered piles. It is normal to assume a battered pile can resist the same axial load as a vertical pile of the same type and size driven to the same depth. The vertical and horizontal components of the load will depend on the batter inclinations. Batters should not exceed 1:4 (horizontal : vertical).

6) Uplift on piles should be limited to 20 percent of the indicated vertical load capacities.

Driven Steel H-Pile Foundations We assume that if H-piles are selected that HP 10 x 57 or 12 x 53 piles will be used. The design criteria below should be followed for design and construction of driven, steel, H-pile foundations for the light poles. The construction details should be considered when preparing project documents. Additional recommendations and criteria for construction of a conventional, driven pile, foundation system are provided in *Appendix B.*

1) 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 12,000 psi based on the pile cross-sectional area.

2) Relatively competent bedrock was encountered in the test holes for this project at depths of about 48 to 49 feet below existing grades. The gravels and cobbles below about 45 feet typically were dense to very dense. Therefore, we anticipate

that piles will be driven to depths of more than 45 feet. Due to the variability in depth to the dense sands and gravels layer encountered during subsurface exploration, the contractor should anticipate advancing the piles to at least 50 feet and be prepared to drive them an additional 5 to 10 feet.

3) Based on the saturated condition of the majority of the soils at the site and the largely non-cohesive nature of the coarse alluvial soils above the bedrock, we anticipate that post-installation down-drag on the piles will be low. An adhesion coefficient of 0.1 appears appropriate for use in the landfill materials to estimate down-drag. GROUND also recommends, however, re-striking of the piles to evaluate their capacity at least 24 hours after (initial) driving has been completed.

4) Groups of relatively closely spaced piles placed to support concentrated loads will require an appropriate reduction of the estimated capacities.

   Reduction of axial capacity can be avoided by spacing piles to a distance of at least 3 'diameters' center to center. Pile groups spaced less than 3 diameters center to center should be studied on an individual basis to determine the appropriate axial capacity reduction(s).

   To avoid reduction of the capacity of piles to resist the component of lateral loading parallel to the line connecting the pile centers, piles should be spaced at least 6 diameters apart. Groups of piles spaced less than 6 diameters center to center should be studied to determine the appropriate lateral capacity reduction(s).

5) Lateral resistance to horizontal forces can be resisted by battered piles. It is normal to assume a battered pile can resist the same axial load as a vertical pile of the same type and size driven to the same depth. The vertical and horizontal components of the load will depend on the batter inclinations. Batters should not exceed 1:4 (horizontal : vertical). Geotechnical parameters for lateral load analysis of vertical piles are provided below.

6) Uplift on piles should be limited to 20 percent of the indicated vertical load capacities.

'Screw Pile' Foundation Systems Use of 'screw piles' to support the proposed recreation center building is feasible geotechnically, if the elements can be installed to the necessary depths and design capacities achieved. It should be noted that somewhat greater strains commonly are required for an alternative foundation system of this type to mobilize its strength. Therefore, apparent settlements upon imposition of structural loads may be somewhat greater, on the order of ¾ inch or more.

'Screw pile' foundation systems are proprietary systems that must be designed by the specialty design/install contractor. We suggest that you contact one or more reputable contractors such as Alpine Site Services, Inc. (303-420-0048), regarding site-specific proposals for design and installation of this foundation system. The specialty foundation contractor and the structural engineer should coordinate to determine the type, number, layout, and necessary vertical and lateral capacities of an alternative foundation system.

We do not anticipate that 'screw piles' can replace driven pile elements on a one-for-one basis. Commonly several screw pile or helical pier elements are installed as a cluster to support a column in lieu of a single driven pile. Battered elements may be needed to resist lateral loads.

In general, the geotechnical parameters provided above for driven, concrete-filled, pipe piles can be used for design of an alternative, 'helical pier' or 'screw pile' foundation system.

'Screw piles' should be advanced so that they bear in the dense gravels and cobbles or the underlying Mancos Shale, at depths below 45 or more feet, not in the softer, overlying silts, clays and sands. We anticipate that this will involve advancing the elements to depths of 50 feet or more in order to achieve design capacities. The coarse cobbles and boulders may hinder advancement of screw piles, however. Potential installation difficulties should be anticipated by the contractor. We recommend that the specialty contractor install one or more test piers/piles prior to completing his cost estimate to evaluate his equipment, installation times, etc., and allow the pier capacities to be assessed.

Lateral Load Parameters Deep foundation elements piles should be designed to resist lateral loads. Based on the field and laboratory data generated in our geotechnical study of the site and our experience with similar sites and conditions, vertical piles may be designed to resist lateral loads taking a horizontal modulus of subgrade reaction (K_h) of 30 tons per cubic foot (tcf) to be characteristic of the soft soils at depths from 5 to 30 feet and 230 tcf to be characteristic of the dense gravels at depths from 30 feet to the top of the bedrock. A modulus of horizontal subgrade reaction of 400 tcf may be taken as characteristic of the clay shale bedrock. Resistance to lateral loads should be neglected in the upper 5 feet of the overburden soils.

Additional lateral load parameters for use in the L-Pile computer program or similar programs can be provided upon request.

FLOOR SYSTEMS

Structural Floors GROUND recommends the use of structural floors supported on drilled piers in a manner similar to the building structures and spanning over a void or well ventilated crawl space as the floor system entailing the lowest risk of post-construction floor movements.

Requirements for the number and position of piers to support a floor, etc., will depend upon the spans, design loads, etc., in the structural design and, therefore, should be developed by the Structural Engineer. Geotechnical recommendations for design and installation of drilled piers are provided in the *Foundation Systems* section of this report and in *Appendix B*.

Structural floors should be constructed to span above a void or a well-ventilated crawl space. A crawl space should be adequate to allow access and maintenance to utility piping. If a wooden structural floor system is used, particular care should be taken to design and maintain the under-floor ventilation systems in order to reduce potential deterioration of the wooden structural members.

Piping serving the building should be hung from a structural floor not placed in the ground beneath the building. (If the floor is constructed over a relatively thin void, this recommendation cannot be implemented, in which case an increased risk of post-construction utility pipe movement must be accepted.) Pipe penetrations through the floor should allow for differential movement between the piping and the floor system. Piping also should be provided with flexible connections where the pipes enter the building to accommodate differential movements.

A minimum 10-mil un-reinforced polyethylene vapor-retarder (a plastic sheet material) should be considered for installation in a crawl space below a structurally supported below-grade floors and should be properly attached/sealed to foundation walls. The plastic sheeting should not be attached to horizontal surfaces such that condensate might drain to wood or corrodible metal surfaces.

Additional discussion and recommendations regarding structural floors are provided in *Appendix B*.

Alternative Slab-on-Grade Floors The use of a slab-on-grade concrete floor for the recreation center building entails a higher risk of post-construction movements, as discussed in *Appendix A*. A slab-on-grade concrete floor may be used, together with remedial earthworks, where the floor is lightly loaded (150 psf or less) and if the owner understands and accepts the associated, increased risk of adverse post-construction floor movements. The criteria below may be followed if a slab-on-grade construction is selected. Additional geotechnical criteria for slab-on-grade floors are provided in *Appendix B*. Areas where equipment will be placed or the floor otherwise will be more heavily loaded, a structural floor should be constructed.

1) If a slab-on-grade floor is selected for the building, the floor system may bear on a properly compacted fill section at least 3 feet in depth, to achieve estimated, likely post-construction movements of about 1 inch, with similar differential movements over spans of about 50 feet.

   The existing soils should be excavated and replaced to a sufficient depth to allow the recommended fill section to be constructed .

   The fill section should underlie the entire building footprint and extend laterally at full depth at least 4 feet beyond the building perimeter.

   Recommendations for placement and compaction of fill soils are provided in the *Project Earthwork* section and *Appendix C*.

2) An allowable vertical modulus of subgrade reaction (K_v) of 45 tcf may be used for design of concrete slabs bearing on a properly prepared fill section.

3) The floor slabs should be separated from all bearing walls and columns with slip joints, which allow unrestrained vertical movement.

   Joints should be observed periodically by the Owner, 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.

4) Interior partitions resting on floor slabs should be provided with slip joints or tracks 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 1½ or more inches of vertical movement, should be considered.

5) Concrete slab-on-grade floors should be provided with properly designed and constructed control joints. ACI, AASHTO and other industry groups provide guidelines for proper design and construction of concrete slabs-on-grade, and associated jointing. The design and construction of such joints should account for cracking resulting from concrete shrinkage, curling, tension and applied loads, as well as other factors related to the proposed slab use. Joint layout based on slab design may require more frequent, additional or deeper joints than typical industry minimums, and should reflect the configuration and proposed use of the slab. Particular attention in slab joint design should be given to areas where slabs exhibit interior corners or curves, e.g., at column block-outs or reentrant corners, and slabs with high length to width ratios, significant slopes, thickness transitions, high traffic loads, or other unique features. The improper placement or construction of control joints will increase the potential for slab cracking.

6) A 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.

7) Moisture can be introduced into a slab subgrade during construction and additional moisture will be released from the slab concrete as it cures. Groundwater at the site is relatively shallow. Therefore, 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.

The capillary break and the drainage space provided by the gravel layer also may reduce the potential for excessive water vapor fluxes from the slab after construction as mix water is released from the concrete. We understand, however, that professional experience and opinion differ with regard to inclusion of a free-draining gravel layer beneath slab-on-grade floors. If these issues are understood by the owner and appropriate measures are implemented to address potential concerns including slab curling and moisture fluxes, then the gravel layer may be deleted.

8) A vapor barrier beneath a building floor slab can be beneficial with regard to reducing exterior moisture moving into the building, but can retard downward drainage of construction moisture. Uneven moisture release can result in slab curling. Elevated vapor fluxes can be detrimental to the adhesion and performance of many floor coverings and may exceed various flooring manufacturers' usage criteria.

   Therefore, in light of the several, potentially conflicting effects of the use vapor-barriers, the owner and the architect and/or contractor should weigh the performance of the slab and appropriate flooring products in light of the intended building use, etc., during the floor system design process and the selection of flooring materials. Use of a plastic vapor-barrier membrane may be appropriate for some building areas and not for others.

LATERAL EARTH PRESSURES

We understand that present plans do not call for foundation walls or other earth retaining structures. A geotechnical engineer should be retained to provide recommendations for design and construction of walls if they are added later to the project. Additional subsurface exploration may be appropriate to develop those recommendations.

Preliminarily, we suggest that walls that can be expected to undergo only a limited amount of deflection, i.e., an "at-rest" condition, be designed to resist lateral earth pressures computed taking an equivalent fluid unit weight of 75 pcf to be characteristic of the on-site materials used as backfill. For preliminary design of cantilevered structures designed to deflect sufficiently to mobilize the full, active earth pressure condition, an equivalent fluid unit weight of 55 pcf may be taken as characteristic of the on-site materials.

The ‘at rest’ and ‘active’ loads 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 wall design.

To resist lateral loads, preliminary design may be based on sliding friction at the bottom of footings calculated as 0.30 times the vertical dead load. An allowable passive soil pressure of 270 psf per foot of embedment may be taken as characteristic of the site soils, to a maximum of 2,700 psf.

SWIMMING POOL

Swimming Pool Foundation An in-ground or above-ground swimming pool at the site will be subject to the same geotechnical constraints discussed herein for other structures at the site and similar estimated post-construction movements. The pool decking likewise will perform similarly to the building floor if supported similarly. In addition to settlements of the pool shell, we anticipate that differential movements between the pool shell and the surrounding deck will be significant design and performance criteria.

In addition, if the swimming pool extends near or below the water table, buoyant uplift of the pool is another significant design consideration.

The most positive way to limit post-construction pool shell settlements and resultant potential damage is to construct the pool shell as a structure and support it and the immediately surrounding decking on a deep foundation system in the same manner as the building. Such a system also can be used to resist buoyant forces. The recommendations in the *Foundation Systems* section of this report may be used to design a foundation for the swimming pool.

A cast-in-place swimming pool constructed with Gunnite or conventional concrete will be subject to greater settlements, including differential settlements with the resultant potential for cracking and other distress. If constructed at and below existing grades, we anticipate that a stabilized and drained excavation bottom as discussed in the *Project Earthwork* section of this report will be necessary to install the pool shell. It may be beneficial to construct a concrete mat to create a working platform. Longer term settlements

If fill is placed to raise grades in the area of the swimming pool, the potential for wet conditions can be reduced, but the increased loading will increase the magnitude of longer term, post-construction settlements. The magnitude of increased settlement will depend on the depth of fill placed, but for a 4-foot fill section, we estimate that longer term settlements will be increased by about ½ inch.

Swimming Pool Decking As with the remainder of the building floors, the most positive approach to supporting the pool decking is to construct it as a structural floor with deep foundations in the same manner as the building, and spanning over a void or well ventilated crawl space. Geotechnical recommendations regarding deep foundations are provided in the *Foundation Systems* section of this report.

The alternative, slab-on-grade floor system discussed in the Floor Systems section of this report, together with the increased post-construction settlements, may be applied to the pool decking as well. To limit differential movements in the immediate vicinity of the pool, we suggest that if a structural pool shell is constructed, the decking within 5 feet or more of the pool shell also be supported on deep foundations.

Regular maintenance and sealing of cracks and joints will be particularly necessary where a slab-on-grade pool deck is selected.

Swimming Pool Drainage Effective surface drainage is important to the proper performance of the swimming pool. drainage Concrete and Gunnite pools are susceptible to cracking and leaking. Moisture infiltration of the subsurface soils may result in additional movements, either from loss of bearing capacity, or from collapse or erosion of the underlying soils. Positive surface drainage and a regular maintenance program will be beneficial to the performance of the pool structures and surrounding decking.

The ground surface surrounding the pool should be sloped to drain away from the pool in all directions. A high quality joint sealant should be provided around the swimming pool perimeter edge and within the concrete deck construction joints surrounding the swimming pool. We recommend that the owner contact the swimming pool manufacturer/installer regarding appropriate products available for joint sealing.

Drain/fill piping, pool decking and other flatwork, appurtenant structures and improvements should be designed, maintained and, as necessary, (re-)sealed to minimize potential water infiltration into the swimming pool subgrade. The Owner should be aware that if settlement begins, particular care should be taken to repair leaks and seal opened joints to reduce further water infiltration.

Systems of perimeter and lateral underdrains beneath a pool, a free-draining gravel layer beneath or enclosing the shell, and other drainage recommendations provided by geotechnical engineers commonly are in conflict with the design and construction standards used by swimming pool designers and installers. Therefore, we recommend the owner contact the manufacturer/installer regarding the use and design of a perimeter + lateral underdrain system. It will be necessary to establish and maintain effective drainage around the swimming pool to prevent buoyant uplift if the shell extended to below the local water table.

WATER-SOLUBLE SULFATES

The concentrations of water-soluble sulfates measured in selected samples obtained from the test holes ranged up to approximately 0.5 percent by weight. (See Table 2.) Such concentrations of water-soluble sulfates represent a severe 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 and PCA and Colorado Department of Transportation (CDOT) guidelines, GROUND recommends use of sulfate-resistant cement in all concrete exposed to site soil and bedrock, conforming to one of the following requirements:

1) Type V, as specified by ASTM C150.

2) Type II with a maximum C3A content of 5 percent and a maximum content of (C4AF +2[C3A]) of 25 percent.

3) Type II or Type I/II, and 15 to 20 percent of the cement shall be replaced with an approved Type F fly ash.

4) A blended cement conforming to Type HS, as specified by ASTM C1157.

Other cement types or blends may be acceptable, however, if type-specific test data demonstrate equal or superior sulfate-resistance to Type V cement. Test data should be provided to the Geotechnical Engineer for review, and the cement approved, prior to use.

All concrete used should have a maximum water/cement ratio of 0.45 by weight. All concrete used should have a minimum compressive strength of 4,500 psi. Concrete mixes should be relatively rich and should be air entrained.

The Contractor should be aware that certain concrete mix components affecting sulfate resistance including, but not limited to, the cement, entrained air, and fly ash, can affect workability, set time, and other characteristics during placement, finishing and curing. The Contractor should develop mix(es) for use in for project concrete which are suitable with regard to these construction factors, as well as sulfate resistance. A reduced, but still significant, sulfate resistance may be acceptable to the Owner, in exchange for desired construction characteristics.

SOIL CORROSIVITY

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 chemistry and other factors.

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, reduction-oxidation (redox) potential, sulfide presence were obtained. Test results are summarized on Table 2.

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.[2] Testing indicated pH values of approximately 7.1 to 7.6.

Reduction-Oxidation testing indicated negative potentials: -8 to -35 milivolts. Such low potentials typically create a more corrosive environment.

[2] American Water Works Association ANSI/AWWA C105/A21.5-05 Standard

Sulfide Reactivity testing for the presence of sulfides indicated at least one 'positive" results in the shallow soils. The presence of sulfides in the site soils also suggests a more corrosive environment.

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 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 40 to 346 ohm-centimeters in samples of the site earth materials. The following table presents the relationship between soil resistivity and a qualitative corrosivity rating:[3]

Corrosivity Ratings Based on Soil Resistivity

Soil Resistivity (ohm-cm)	Corrosivity Rating
>20,000	Essentially non-corrosive
10,000 – 20,000	Mildly corrosive
5,000 – 10,000	Moderately corrosive
3,000 – 5,000	Corrosive
1,000 – 3,000	Highly corrosive
<1,000	Extremely corrosive

Corrosivity Assessment 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.

[3] ASM International, 2003, *Corrosion: Fundamentals, Testing and Protection,* ASM Handbook, Volume 13A.

Table A.1 Soil-test Evaluation [2]

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

* If sulfides are present and low or negative redox-potential results (< 50 mV) are obtained, add three (3) points for this range.

We anticipate that drainage at the site after construction will be good. With effective drainage, based on the values obtained for the soil parameters, the site soils appear to comprise a high risk environment for metals with regard to corrosion.

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 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.

PROJECT EARTHWORK

The project site appeared to have undergone previous, limited grading to prepare it for use as a parking lot. We anticipate only limited cuts and fills, likely on the order of 2 feet of less to construct the building pad and pavement areas. Deeper excavations and backfills will be needed to install utilities and the swimming pool, however.

Site grading should be planned carefully to provide positive surface drainage away from the building, and all pavements, utility alignments, and flatwork. Surface diversion features should be provided around paved areas to prevent surface runoff from flowing across the paved surfaces. 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.

Use of On-Site Materials as Fill Site soils, free of trash, organic material, construction debris and other deleterious materials are suitable, in general, for placement as compacted fill. Soils excavated from below about 4 typically will be wet and require drying to be placed as compacted fill.

Cobbles and fragments of inert construction debris (e.g., concrete or asphalt) larger than 6 inches in maximum dimension will require special handling and/or placement to be incorporated into project fills. In no case should these fragments be placed within 3 feet of finished grade. In general, such materials should be placed as deeply as possible in the project fills. A Geotechnical Engineer should be consulted regarding appropriate recommendations for usage of such materials on a case-by-case basis when such materials have been identified during earthwork. Standard recommendations that likely will be generally applicable can be found in Section 203 of the CDOT Standard Specifications for Road and Bridge Construction (2005).

Imported Fill Materials If it is necessary to import material to the site, the imported soils should be free of trash, organic debris or otherwise deleterious materials. Imported material should have less than 85 percent passing the No. 200 Sieve and should have a plasticity index of less than 15. Representative samples of all materials proposed for import should be tested and approved by the Geotechnical Engineer prior to transport to the site.

Fill Placement and Compaction Detailed geotechical recommendations for fill placement and compaction are provided in *Appendix C*.

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 constructed at this angle until vegetation is well re-established. Surface drainage should be designed to direct water away from slope faces.

Excavation Considerations The test holes were advanced to the depths indicated on the test hole logs by means of conventional truck-mounted drilling equipment. We anticipate no unusual excavation difficulties, in general, for the proposed construction in the site soils with conventional, heavy-duty excavating equipment in good working condition.

However, the shallow soils are soft and at depths below about 4 feet became very moist to wet. Groundwater was encountered during subsurface exploration below depths of about 6½ to 7 feet. Therefore, soft wet soils and groundwater will characterize all but the shallowest excavations at this site. The contractor should be prepared to work in wet soils and in the presence of groundwater. We anticipate that the contractor typically will need to stabilize and drain the bottoms of excavations deeper than about 4 or 5 feet to place fill or create a useful working platform. The contractor should anticipate placing coarse, open graded crushed rock, stabilization geo-textiles and/or using other methods to establish a stable excavation bottom.

A properly designed and installed de-watering system will be required during the construction near or below the water table. The de-watering system(s) should be designed for the contractor by a registered engineer. The risk of slope instability will be significantly increased in areas of seepage along the excavation slopes. If seepage is encountered, the slopes should be re-evaluated by the geotechnical engineer.

The contractor also should take pro-active measures to control surface waters during construction, to direct them away from excavations and into appropriate drainage structures.

Temporary Excavation Slopes We recommend that temporary, un-shored excavation slopes up to 10 feet in height be cut no steeper than 1½ :1 (horizontal : vertical) in the native soils and bedrock <u>in the absence of seepage</u>. Some surficial sloughing may occur on slope faces cut at this angle. Local conditions encountered during construction such as; loose, soft, 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. Temporary 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 taking an equivalent fluid unit weight of 55 pounds per cubic foot (pcf) to be characteristic of the site soils for a level adjacent ground condition in the absence of seepage. In addition to this lateral earth pressure, shoring design should include surcharge loads exerted by equipment, traffic, seepage forces, material stockpiles, etc. Actual shoring system(s) should be designed for the contractor by a registered engineer.

SURFACE AND SUBSURFACE DRAINAGE

Establishing and maintaining effective drainage is important for proper geotechnical performance of most building structures and other improvements. Effective drainage is particularly important at the subject parking facility site due to the heave potentials of the local soils.

Surface Drainage 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 will result primarily from the introduction of water into the soils underlying the proposed structure, hardscaping and pavements. The native soils are particularly vulnerable because of their capacity for relatively severe heave. 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 approach grade beam or floor elevations. Therefore, wetting of the soils likely will result from infiltrating surface waters (precipitation, irrigation, etc.), and water flowing along constructed pathways such as bedding in utility pipe trenches. In our experience, infiltration commonly is most pronounced at tree grates, planters, concrete / asphalt joints, behind small, decorative retaining walls and similar locations that commonly cannot be addressed readily in project-scale civil design. Project design should incorporate measures to inhibit water from wetting the project soils. Surface drainage gradients, pavements, flatwork, piping, drainage structures, etc., should be maintained during and after construction to inhibit 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.

The surface drainage measures recommended in *Appendix D* should be observed <u>during construction</u> <u>and maintained</u> at all times after the facility has been completed. If those measures are not implemented and maintained effectively, the movement estimates provided in this report could be exceeded.

Building Underdrains As a component of project civil design, properly functioning, subsurface drain systems (underdrains) can be beneficial for collecting and discharging saturated subsurface waters. Underdrains will not collect water infiltrating under unsaturated (vadose) conditions, or moving via capillarity, however. In addition, if not properly constructed and maintained, underdrains can transfer water into foundation soils, rather than remove it. This will tend to induce settlement of the subsurface soils, and may result in structure/floor slab distress. Underdrains can, however, provide an added level of protection against relatively severe post-construction movements by draining saturated conditions near individual structures should they arise, and limiting the volume of wetted soil.

Where underdrain systems are included in project drainage design, they should be designed in accordance with the detailed geotechnical recommendations in *Appendix D*. The actual underdrain layout, outlets, and locations should be designed by the Civil Engineer.

If below-grade or partially below-grade structures such as short foundation walls, elevator pits, etc., are included in the project, those structures should be damp-proofed on their exterior sides and provided with local underdrain systems.

UTILITY LATERAL INSTALLATION

Recommendations regarding excavation of utility lateral trenches are provided in the *Project Earthwork* section of this report. On-site soils excavated from trenches are suitable, in general, for use as trench backfill. Backfill soils should be free of vegetation, trash and other deleterious materials.

Pipe bedding materials, placement and compaction should meet the specifications of the pipe manufacturer and applicable municipal standards. The contractor should not anticipate that significant volumes of suitable materials will be available on-site where relatively free-draining bedding materials are called for. Imported materials proposed for use as pipe bedding should be tested and approved by a geotechnical engineer prior to transport to the site. Bedding should be brought up uniformly on both sides of the pipe to reduce differential loadings.

Trench backfill materials above the pipe bedding zone where CLSM is not used (See *Appendix C*.) should be conditioned to a uniform moisture content, placed in uniform lifts not exceeding 8 inches in loose thickness, and properly compacted. Recommendations for backfill placement and compaction are provided in *Appendix C*.

We assume that surface drainage will direct water away from trench alignments. Nevertheless, GROUND recommends that non-woven filter fabric (e.g., Mirafi® 140N, or the equivalent) should be placed around the granular bedding materials to reduce migration of fines into the bedding which can result in severe, local settlements. Where this protection is not provided, severe settlements can result as much as several months or years after construction is completed, even where backfill soils have be compacted properly.

Granular pipe bedding materials can function as efficient conduits for re-distribution of water in the subsurface. Therefore, GROUND recommends that clay or concrete 'cut-off's be installed in the utility lateral trenches between the building and the utility mains to interrupt the bedding and slow the rates of water movement through the bedding sections toward the building, pavements and other structures where excessive wetting of the underlying soils will be damaging. These measures also will reduce the risk of loss of fine-grained backfill soils into the bedding material – a process known as 'piping' – with resultant surface settlements.

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. Because the project pavements will be maintained by the City of Fruita, the recommended pavement sections were developed in general accordance with the guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO), the Colorado Department of Transportation (CDOT) and local construction practice. Geotechnical recommendations for pavement design are provided below. Additional recommendations for pavement materials, construction, etc., are provided in *Appendix E*.

Subgrade Materials Based on the results of our field and laboratory studies, subgrade materials in the proposed pavement areas consisted predominantly of clayey and gravelly sands to sandy silts, as well as weathered bedrock. These materials were classified typically as A-4, and A-6 soils in accordance with the AASHTO classification system, with Group Index values from 0 to 10.

Resilient Modulus (M_R) testing (AASHTO T-307) was performed on a representative composite sample of the subgrade materials encountered at the site. A M_R of 3,924 psi, obtained at 2 percentage points above the optimum moisture content, was taken to be characteristic of the subgrade soils. 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.

Anticipated Traffic An equivalent 18-kip daily load application (EDLA) value of 5 was assumed for automobile/light truck-only parking stalls. An EDLA of 10 was assumed for the parking area drive aisles. The EDLA values of 5 and 10 were converted to equivalent 18-kip single-axle load (ESAL) values of 36,500 and 73,000, respectively, for 20-year design lives. An EDLA of 50, corresponding to an ESAL value of 365,000, was assumed for trash collection zones and other areas subject to heavier traffic including large trucks.

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

Pavement Sections The soil resilient modulus and the anticipated ESAL values were used to determine the required structural number for the project pavements. The required structural number was then used to develop recommended, minimum, pavement sections. Pavement sections were based on the DARWin™ computer program that solves the 1993 AASHTO pavement design equation. Pavement parameters and calculations are summarized in *Appendix F*. A Reliability Level of 80 percent was utilized develop the pavement sections. Structural coefficients of 0.40 and 0.12 were used for hot bituminous asphalt and high quality aggregate base course, respectively. The resultant minimum pavement sections recommended by GROUND are tabulated below.

RECOMMENDED MINIMUM PAVEMENT SECTIONS

Location	***Full Depth Asphalt*** *(inches Asphalt)*	***Composite Section*** *(inches Asphalt / inches Aggregate Base)*
Parking Lot Drive Aisles	6.5	4 / 8
Automobile-Only Parking	6	4 / 6.5
High Turning Stresses & Heavy Traffic	6½ inches of pcc / 6 inches of aggregate base	

We recommend that primary internal truck routes serving the facility such as the trash collection and shipping / receiving areas as well as other pavement areas subjected to high turning stresses or heavy truck traffic be provided with rigid pavements consisting of 6½ or more inches of portland cement concrete, underlain by underlain by 6 inches of properly compacted aggregate base in accordance with City of Fruita requirements.

Concrete pavements should contain sawed or formed joints. CDOT and various industry groups provide guidelines for proper design and concrete construction and associated jointing. 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, found at the CDOT website: http://www.dot.state.co.us/DesignSupport/.

Subgrade Preparation Although subgrade preparation to a depth of 8 to 12 inches is typical in the project area, pavement performance commonly can be improved by a greater depth of moisture-density conditioning of the soils.

Remedial Earthwork GROUND recommends that shortly before paving, the pavement subgrade be excavated and/or scarified to a depth of at least 12 inches, moisture-conditioned and properly re-compacted. Recommendations for fill placement and compaction are provided in *Appendix C*. The contractor should be prepared either to dry the subgrade materials or moisten them, as needed, prior to compaction.

Subgrade preparation should extend the full width of the pavement from back-of-curb to back-of-curb. The subgrade for sidewalks and other project hardscaping also should be prepared in the same manner.

Where adequate drainage cannot be achieved or maintained, a greater depth of excavation and replacement is recommended, in addition to the edge drains recommended in *Appendix E*.

Proof Rolling Immediately prior to paving, the subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle. Areas where water that show excessive deflection during proof rolling should be excavated and replaced and stabilized. Areas allowed to pond prior to paving will require significant re-working prior to proof-rolling. Passing proof-rolling is an additional requirement for pavement subgrade soils; it may be possible for soils to be compacted within the limits indicated in *Appendix C* of this report and fail proof rolling, particularly in the upper range of recommended moisture contents.

Subgrade Stabilization Because of the silty nature of many of the site soils, they likely will “pump” or deflect during compaction and proof-rolling if moisture levels are not carefully controlled and achieving a stable platform for paving may be difficult. Chemical stabilization of the pavement subgrade may be necessary. Because of the water-soluble sulfates in the site soils, stabilization with lime does not appear feasible. We anticipate, however, that stabilization with portland cement would be effective.

EXTERIOR FLATWORK

Exterior flatwork and other hardscaping placed on the soils encountered at the site will experience post-construction movements as soil moisture contents increase after construction. Heave of the local earth materials should be anticipated and distress to rigid hardscaping likely will result. The following measures will help to reduce damages to these improvements:

1) Project sidewalks, paved entryways and patios, masonry planters and short, decorative walls, and other flatwork should be underlain by a section of properly moisture-conditioned and compacted fill soils at least 12 inches in thickness, constructed in accordance with the recommendations in *Appendix C*. Greater

depths of moisture-density conditioning of the subgrade soils may be needed locally, depending on the conditions exposed.

2) Prior to placement of flatwork, a proof roll should be performed to identify areas that exhibit instability and deflection. The deleterious soils in these areas should be removed and replaced with properly compacted fill. The Contractor should take care to achieve and maintain compaction behind curbs to reduce differential sidewalk settlements. As in the case of pavements, passing a proof roll is an additional requirement to placing and compacting the subgrade fill soils within the recommended ranges of moisture content and relative compaction presented in *Appendix C* of this report and subgrade stabilization may be cost-effective.

3) Flatwork should be provided with control joints extending to an effective depth and spaced no more than 10 feet apart, both ways. Narrow flatwork, such as sidewalks, likely will require more closely spaced joints.

4) 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.

Additional geotechnical recommendations regarding hardscape construction are provided in *Appendix E*.

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.

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 the City of Fruita, Colorado, as it pertains the design of the proposed recreational facility 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 Winter, 2009 – 2010. Changes in project plans or schedule should be brought to the attention of a 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 locations as shown in Figure 1. Subsurface conditions were interpolated between and extrapolated beyond these locations. Findings were dependent on the limited amount of direct evidence obtained at the time of this geotechnical evaluation. Our recommendations were developed for site conditions as described above. Actual conditions exposed during construction may be anticipated to differ, somewhat, from those encountered during site exploration.

If during construction, surface, soil, bedrock, or groundwater conditions appear to be at variance with those described herein, a geotechnical engineer should be advised at once, so that re-evaluation of the recommendations may be made in a timely manner. Contractors should review all available project information, including this report, prior to providing construction/service bids. In addition, the information in this report may be insufficient for a contractor to develop his scope of work or cost estimates 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 presented in this report are based on the current state-of-the-art for improvements placed on earth materials subject to consolidation and settlement. The owner should be aware that there is a risk in construction on these types of soils. Performance of the proposed structures and pavement will depend on implementation of the recommendations in this report and on proper maintenance after construction is completed. Any indications of distress to project installations should be brought to the attention of a geotechnical engineer in a timely manner.

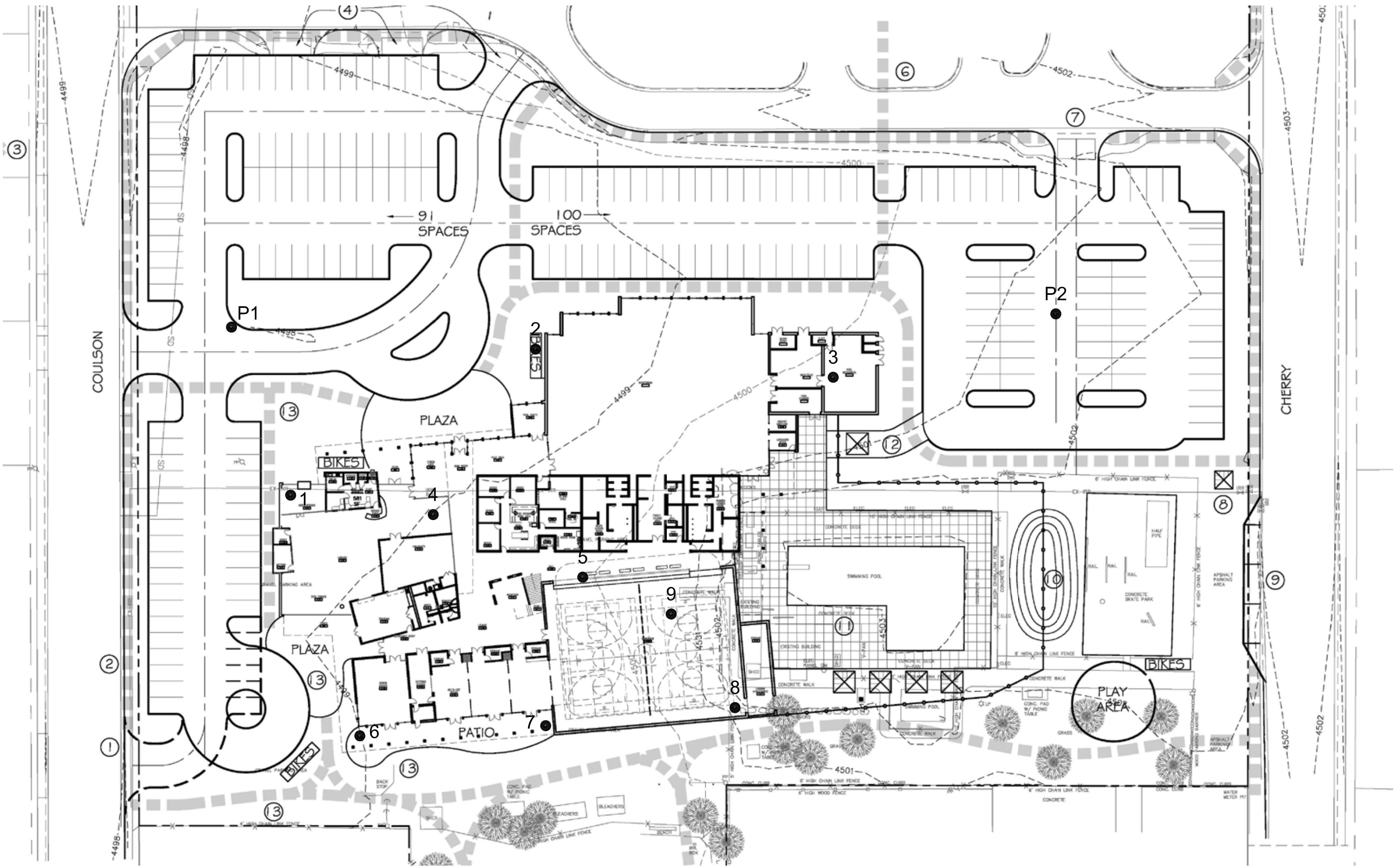
This report was prepared in accordance with generally accepted soil and foundation engineering practice in the Mesa County, Colorado, area, at the date of preparation. GROUND makes no warranties, either express or implied, as to the professional data, opinions or recommendations contained herein.

Sincerely,

GROUND Engineering Consultants, Inc.

Brian H. Reck, C.E.G.

Reviewed by Michael K. Wariner, P.E.



1
● Indicates test hole number and approximate location.

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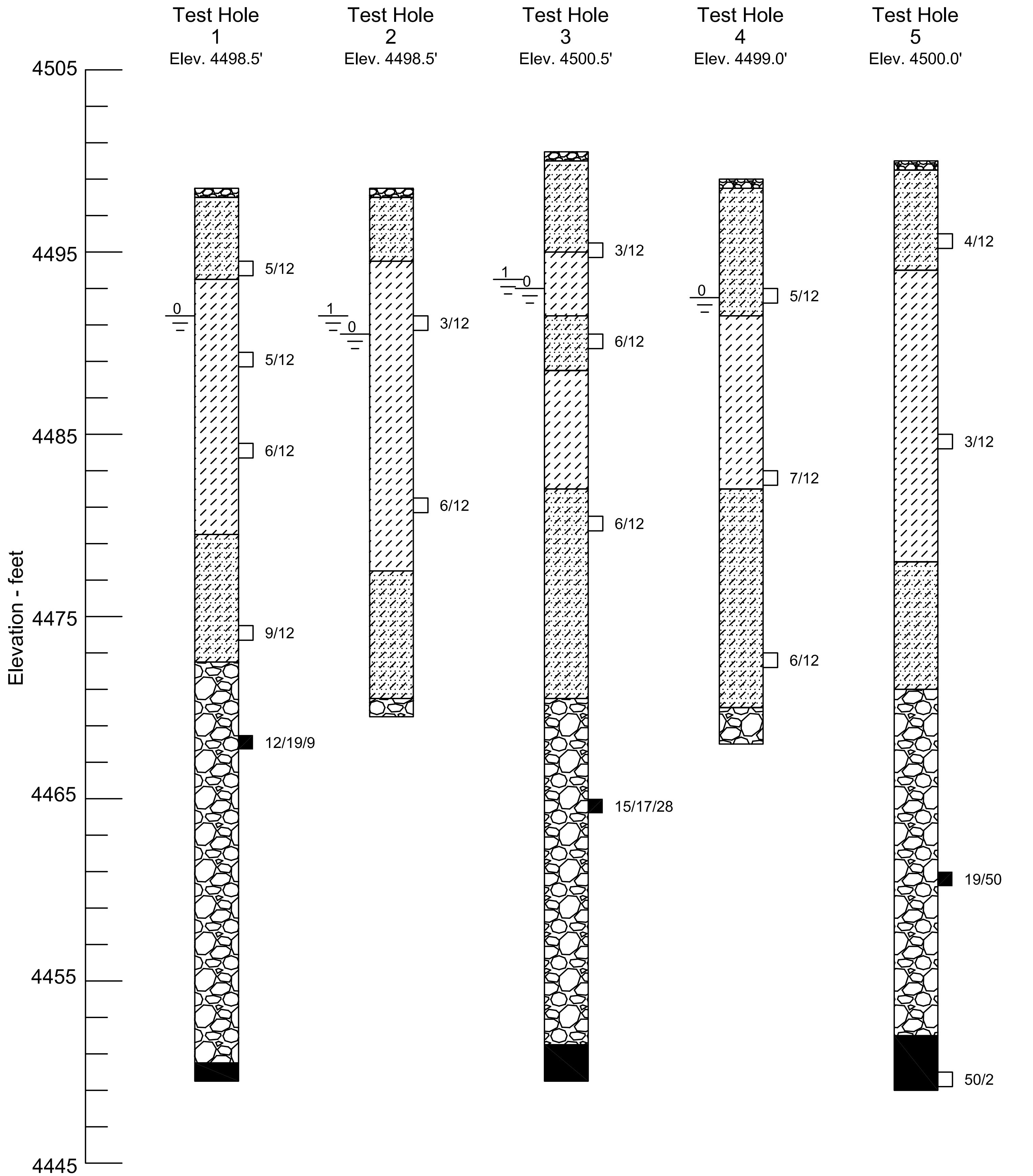
LOCATION OF TEST HOLES

JOB NO.: 09-6013	FIGURE: 1
CADFILE NAME: 6013SITE.DWG	



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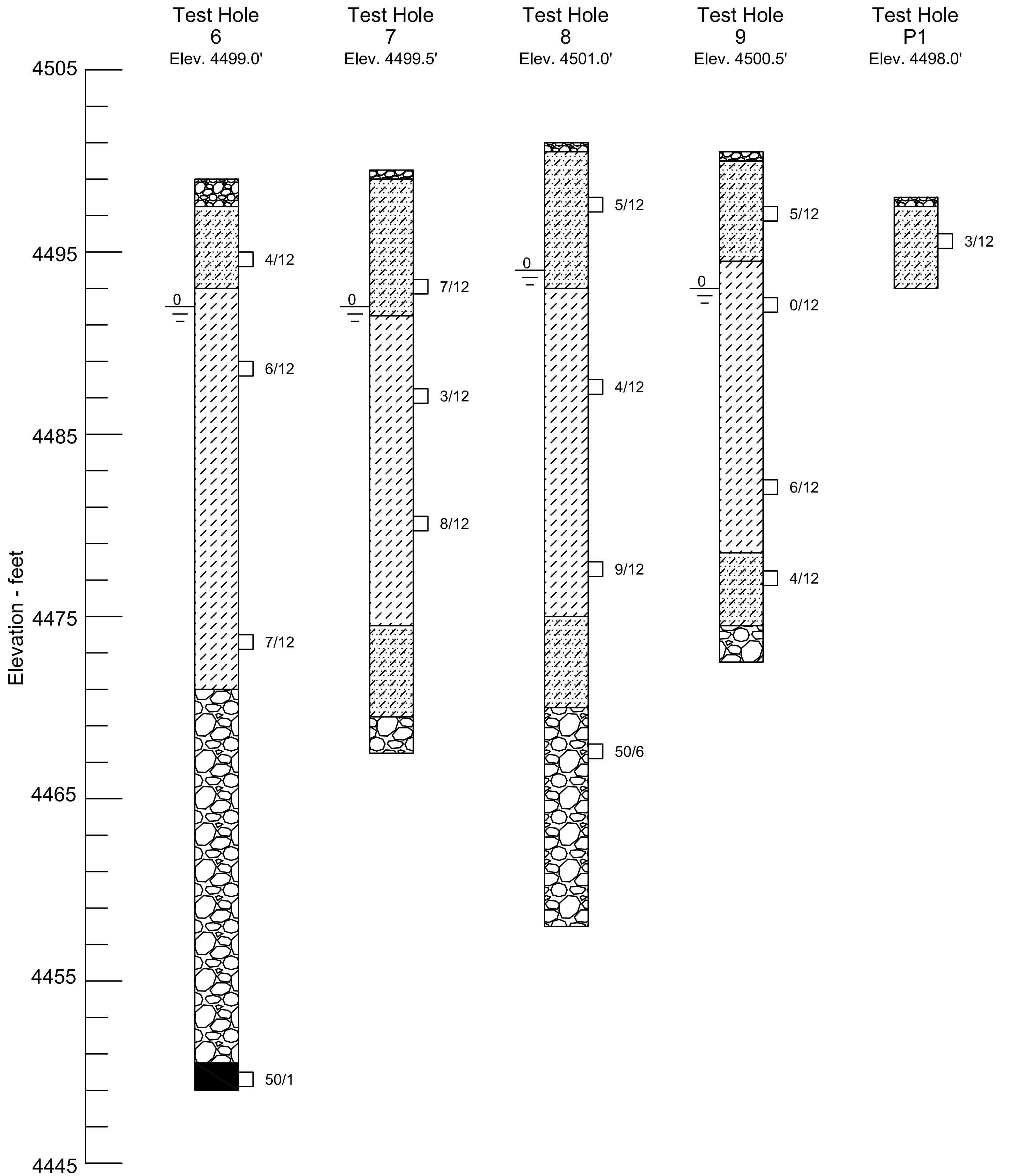
LOGS OF TEST HOLES

JOB NO.: 09-6013	FIGURE: 2
CADFILE NAME: 6013LOG01.DWG	



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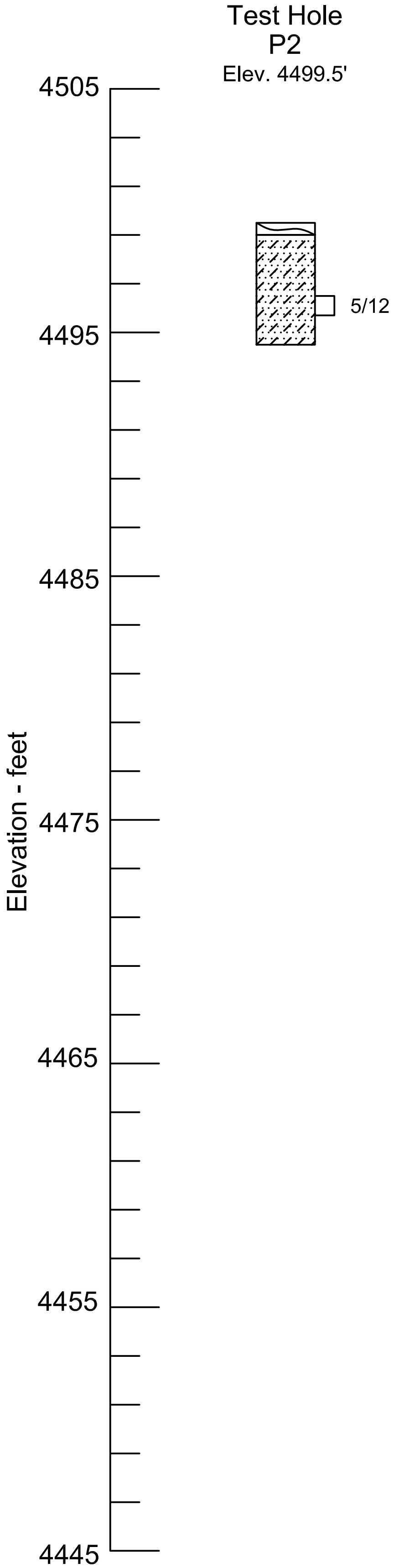
LOGS OF TEST HOLES

JOB NO.: 09-6013	FIGURE: 3
CADFILE NAME: 6013LOG02.DWG	



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LOGS OF TEST HOLES

JOB NO.: 09-6013	FIGURE: 4
CADFILE NAME: 6013LOG03.DWG	

LEGEND:

Sands and Clays: Consisted of silty to clayey sands with local beds and lenses of sandy silts and clays; fine to fine-to-medium. They were moist to wet, low to moderately plastic, loose / soft and red-brown in color.

Clays: Consisted of low to high plastic clays with local silts; clean to slightly sandy, wet, very soft to medium stiff, and red-brown to red in color.

Gravel and Cobbles: Ranged from sandy gravels to cobbles, generally coarsening downward. Boulders likely are present in the lowest 5 to 10 feet of the section. They were clean to silty, wet, low plastic to non-plastic, medium dense to dense, and light gray to gray in color.

Clay Shale Bedrock: Clean to slightly sandy, very moist, highly plastic, very hard and dark gray in color.

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

Drive sample, 1-3/8 inch I.D. standard 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.

20/25/30 Drive sample blow count, indicates 20, 25, and 30 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 18 inches.

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

NOTES:

1) Test holes were drilled on 04/23/09 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 provided.

3) Elevations of the test holes were not measured and the logs of the test holes are drawn to depth.

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.

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LEGEND AND NOTES

JOB NO.: 09-6013	FIGURE: 5
CADFILE NAME: 6013LEG.DWG	

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TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural Moisture Content *(%)*	Natural Dry Density *(pcf)*	Passing No. 200 Sieve *(%)*	Atterberg Limits		Swell* vs. 1,000 psf Surcharge *(%)*	USCS Classifi-cation	Soil or Bedrock Type
Test Hole No.	Depth *(feet)*				Liquid Limit *(%)*	Plasticity Index *(%)*			
1	4	13.3	97.4	38	18	4	-2.7	SM	Silty SAND
1	24	28.4	97.5	26	19	2		SM	Silty SAND
2	7	36.6	83.4	98	24	5		ML/CL	SILT and CLAY
3	5	23.8	99.6	81	26	10		CL	Sandy CLAY
3	20	25.7	97.4	35	21	6		SM/SC	Silty/Clayey SAND
5	15	27.7	92.7	97	39	18		CL	CLAY
6	4	15.7	-	37	19	4	-1.0	SM	Silty SAND
6	10	29.9	91.1	99	32	15		CL	CLAY
7	19	30.7	90.3	99	38	21		CL	CLAY
8	13	31.9	89.0	100	37	17		CL	CLAY
8	23	25.1	98.7	100	50	29		CH	CLAY
9	3	25.8	92.0	76	31	16		CL	Sandy CLAY
P-1	2	15.0	103.9	64	21	7	M_R = 3,924 psi	ML/CL	SILT and CLAY

* Negative values indicate consolidat

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TABLE 2
SUMMARY OF LABORATORY TEST RESULTS, CONTINUED

Sample Location		Water-Soluble Sulfates	Resisitivity	pH	Redox Potential	Sulfide Reactivity	Soil or Bedrock Type
Test Hole No.	Depth *(feet)*	*(%)*	*(ohm-cm)*		*(mV)*		
3	5	0.03	346	7.6	-35	Trace	Sandy CLAY
6	10	0.50	40	7.1	- 8	Positive	CLAY

Job No. 09-6013

APPENDIX A

Geotechnical Basis for Recommendations

GEOTECHNICAL BASIS FOR RECOMMENDATIONS

Geotechnical Risk

The data obtained for this study suggested moderate to severe potentials for longer term, post-construction settlements under increased loads in the on-site soils. These soil conditions have the potential to damage the proposed improvements. Native soils of this type and properly moisture-conditioned and compacted fill soils are generally stable with regard to their moisture-volume relationships so long as their moisture contents and loading conditions are kept unchanged. The loads imposed by new construction and post-construction wetting of the soils gives rise to soil consolidation and resultant surface settlements and damage to improvements. Pavements, utilities, and sidewalks, etc., all can be affected, in addition to the building. The actual extent and distribution of post-construction settlement realized at a given location (and the nature and degree of resultant damages) will depend on a variety of factors. These factors include the spatial distribution of the consolidation potentials in the underlying soils, the type and design of the structure supported on them, structural loads, etc. Because of the number and variability of these factors, the necessarily limited data set by which the geotechnical factors can be estimated, the variety of means by which estimates of post-construction settlement can be made, and the inherent assumptions involved, estimates of future settlement will tend to range significantly in magnitude.

Various quantitative and semi-quantitative methods are used by geotechnical engineers in the Colorado Western Slope area to estimate post-construction settlement of structures, pavements, etc., as a step toward development of recommendations for foundations, remedial earthworks, etc. Those typically used are based on practical engineering experience and judgment using measured values of engineering properties of the soils. 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 structures, and our engineering judgment with regard to the applicability of the data and methods of forecasting future performance. Several 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 generally effective, estimated to a reasonable degree of engineering certainty) as well as our assumptions about the Owner's willingness to accept geotechnical risk. <u>"Maximum possible" movement estimates necessarily will be larger than those presented herein</u>. <u>They also have a significantly lower likelihood of being realized</u> in our opinion, and generally require more expensive measures to address. We encourage the Owner and future prospective owners, upon receipt of this report, however, to discuss these risks and the geotechnical alternatives with us.

Engineering consulting and design practice always involves weighing the risks inherent in a given design approach against the construction 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 (or was) 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.

Likely Post-Construction Movements

Utilizing the above assumptions, data obtained for this study, and our experience on other projects in the vicinity, our estimates indicate post-construction vertical movements on the order of 2 to 5 inches where building elements are supported directly on the existing earth materials. (Lateral movements would result, as well.) The range in potential movement magnitude reflects, in part, the variable depth of soils subject to consolidation across the building site. Movements of this magnitude can cause severe cosmetic and/or structural distress to the proposed buildings. (The general potential for post-construction movement and damage also apply to project pavements, hardscaping, piping, and all other improvements supported on the site soils, generally proportionate to the loads that they impose.)

General Foundation Types

At the subject site, several types of foundation systems, in conjunction with differing extents of remedial earthworks, etc., can be employed to support the proposed building on the soils and bedrock encountered in the test holes. Each combination entails a different degree of risk of post-construction building movements. These range from utilizing shallow spread footings and slab-on-grade concrete floors bearing directly on the native soils (entailing the greatest risk) to supporting the structures, including floors, on deep foundations bearing in the underlying gravels or bedrock at depths of 50 feet or more (entailing the least risk among typically employed foundation types).

GROUND recommends that the proposed building and any ancillary structures not tolerate of significant settlement be supported on deep foundation systems such as driven steel H-piles, concrete-filled pipe piles or 'screw piles,' and provided with structural floors supported similarly. The geotechnical criteria provided in the *Building Foundations* section of this report for design of deep foundation system were developed accordingly. Although a deep foundation system incorporating these criteria will not eliminate the risk of post-construction building movement, if the measures outlined in this report are implemented effectively, the likelihood of acceptable building performance to a reasonable degree of engineering certainty will be within local industry standards for construction of a foundation system of the selected type on soils and bedrock of this nature. Based on the conditions encountered in GROUND's test holes, the assumptions outlined herein, including effective maintenance of site drainage, we estimate post-construction movement (apparent settlement) of a properly designed and installed driven steel H-pile or concrete-filled pipe pile foundation to be on the order of ½ inch as the structural loading is accommodated. We anticipate that 'screw piles' will exhibit apparent settlements of at least ¾ inch.

As a higher risk alternative, a slab-on-grade concrete floor can be used with a limited depth of remedial earthwork beneath the slab. Consolidation of the underlying soils will cause slab settlement, but where the slab is only lightly loaded, the estimated likely post-construction movement may be acceptable. Slab-on-grade floors together with compaction of the underlying soils have been used in the Colorado Western Slope area with varying degrees of success. If the alternative criteria are implemented effectively in design and construction, and effective site drainage is maintained, we estimate likely slab movements to be about 1 inch.

GROUND is available to meet, however, to discuss the risks and remedial approaches presented in this report, as well as other potential approaches, upon request.

APPENDIX B

Recommendations for Foundation and Floor System Construction

FOUNDATION AND FLOOR SYSTEM CONSTRUCTION

Driven Concrete-Filled Pipe Pile Foundations

I. The piles should consist of a heavy steel pipe section. The pile tip should be reinforced with a commercial, heavy duty, pile tip.

II. We recommend that the pile-driving hammer should develop a minimum of 20,000 foot-pounds of energy per blow for a 10-inch diameter pile. Minimum driving energies can be provided for other pipe pile types upon request.

III. After the actual pile type and proposed hammer have been selected, the Geotechnical Engineer should be retained to perform a Wave Equation analysis to determine if the driving hammer is sized adequately for the type of pile selected and the soils and bedrock materials into which the piles are driven.

IV. We suggest that a test pile installation program be performed to better define the driving conditions and installation depths and conditions.

V. A geotechnical engineer should be retained to observe all pile driving operations. We recommend that at the start of pile installation for the building, the geotechnical engineer perform pile dynamic testing. This testing should be performed in order to a) assess whether piles are being over-stressed relative to the maximum service stress of 12,000 psi recommended above, and b) develop virtual refusal criteria for bedrock penetration based on the design capacity of the piles.

VI. We suggest that a test pile installation program be performed to better define the driving conditions and installation depths and conditions.

Lateral loading tests also should be performed as appropriate.

VII. A geotechnical engineer should be retained to observe all pile driving operations.

VIII. We recommend that at the start of pile installation a geotechnical engineer should be retained to perform pile dynamic testing at each general location at which driven piles will be installed. Testing will be performed in order to a) assess whether piles are over-stressed relative to the maximum service stress of 12,000 psi recommended above, and b) develop driving criteria based on the design capacity of the piles. Dynamic pile testing should be performed by means of a Pile Driving Analyzer (PDA) to determine the driving criteria.

Driven Steel H-Pile Foundations

I. Piles should consist of a heavy steel H-section. The pile tip should be reinforced with a commercial, heavy duty, pile tip.

II. We recommend that the pile driving hammer should develop a minimum of 26,000 foot-pounds of energy per blow for both HP 10 x 87 piles, and HP 12 x 53 piles. Minimum driving energies can be provided for other H-pile types upon request.

III. After the actual pile type and proposed hammer have been selected, a geotechnical engineer should perform a Wave Equation analysis to determine if the driving hammer is sized adequately for the type of pile selected and the soils and bedrock materials into which the piles are driven. GROUND can provide dynamic pile testing during pile installation if requested.

IV. We suggest that a test pile installation program be performed to better define the driving conditions and installation depths and conditions.

V. A geotechnical engineer should be retained to observe all pile driving operations. We recommend that at the start of pile installation for the building, the geotechnical engineer perform pile dynamic testing. This testing should be performed in order to a) assess whether piles are being over-stressed relative to the maximum service stress of 12,000 psi recommended above, and b) develop virtual refusal criteria for bedrock penetration based on the design capacity of the piles.

VI. Dynamic pile testing should be performed by means of a Pile Driving Analyzer (PDA) to determine the virtual refusal criteria.

Lateral loading tests also should be performed at the start of pile installation.

VII. The test holes drilled for this evaluation were advanced to the depths indicated on the test hole logs by means of a conventional, truck-mounted, drilling rig using 4-inch diameter, solid-stem, flight auger equipment. Nevertheless, because of the cobbles and boulders in the site soils, the contractor should anticipate some difficulties in advancing the piles, and be prepared to work in these conditions. We anticipate the need for local pre-drilling of pile locations and/or splitting of boulders to advance the piles to the anticipated depths.

VIII. Additional pile footage should be included in project planning to allow for additional piles locally where offsets were required due to potential obstructions to pile driving in the landfill materials or damage to piles.

Where a pile cannot be advanced to at least the approximate, anticipated tip depth, it should be evaluated with regard to its capacity by the geotechnical engineer and the structural engineer.

Structural Floors

I. New buildings generally lack ventilation due primarily to systematic efforts to construct airtight, energy-efficient structures. Therefore, areas such as crawl spaces beneath structural floors are typically areas of elevated humidity which never completely dry. This condition can be aggravated in some locations by shallow groundwater or a perched groundwater condition, which can result in, saturated soils within close proximity of finished building pad grades.

Persistently warm, humid conditions in the presence of cellulose, which is the base material found in many typical construction products, creates an ideal environment for the growth of fungi, molds, and mildew. Published data suggest links between molds and negative health affects. Therefore, GROUND recommends that crawl spaces beneath structural floors be provided with adequate, positive <u>active</u> ventilation systems or other active mechanisms such as specially designed HVAC systems (as well as properly constructed and maintained underdrains) to reduce the potential for mold, fungus and mildew growth.

<u>Mold Growth Areas/Conditions for Growth for Structural Floors:</u>

1. Water damaged building materials or high moisture/humidity areas where cellulose-containing materials are used:
    - Wallboard/sheetrock
    - MDF/OSB/Plywood
    - Fibrous Ceiling Tiles
    - Paper-backed Insulation
    - Jute-backed carpet
    - Hardwood Flooring
2. Condensation inside buildings from pipes, baths, heaters, and dryer vents

3. Relative humidity greater than 55%
4. Temperatures of 36 to 104 ºF.
5. “Wet” areas that do not dry out after 24 hours.

Mold does not require a light source in order to grow and can grow inside walls, behind tubs/showers, under carpet and flooring undetected.

II. Crawl spaces should be inspected periodically so that remedial measures can be taken in a timely manner, should mold, fungus or mildew be present and require removal.

III. The Owner must be willing to accept the risks of potential mold, fungus, and mildew growth when electing to utilize a structural floor system. Additionally, the Contractor is solely responsible for the construction means and methods, and any observation or testing performed by a representative of a Geotechnical Engineer during construction does not relieve the Contractor of that responsibility.

Slab-on-Grade Concrete Floors

I. The existing soils should be removed to a sufficient depth beneath slab bearing elevation to accommodate construction of the recommended fill sections, in addition to scarification and re-compaction of the underlying 8 to 12 inches of material. The thickness of the fill section should be taken from the <u>bottom</u> of the slab + gravel layer system. (If the gravel layer is not installed, the fill section should be correspondingly thickened.)

II. The Contractor should survey the excavations beneath the building verifying that the remedial excavations were advanced to a sufficient depths and extents.

III. The Contractor should take care to construct a fill section of uniform depth and composition to reduce differential post-construction building, slab and flatwork movements. A differential fill beneath the building will tend to increase differential movements.

IV. The prepared surface on which a floor slab will be cast should be observed by a Geotechnical Engineer prior to placement of reinforcement. Exposed loose, soft or otherwise unsuitable materials should be excavated and replaced with properly compacted fill, placed in accordance with the recommendations in *Appendix C*.

V. Concrete slabs-on-grade should be constructed and cured in accordance with applicable industry standards and slab design specifications.

APPENDIX C

Recommendations for Earthwork Construction

EARTHWORK CONSTRUCTION

General Considerations

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.

Excavations

The contractor should take care when making excavations not to compromise the bearing or lateral support for the foundations of the adjacent, existing pavements or other improvements.

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 particularly CFR 29 Part 1926, OSHA Standards-Excavations, adopted March 5, 1990. 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 in this report solely as a service to the client, and is not assuming responsibility for construction site safety or the contractor's activities.

Fill Platform Preparation

Prior to filling, the top 8 to 12 inches of in-place materials on surfaces 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.

Excavations to depths greater than about 4 or 5 feet generally will encounter soft, wet conditions. The contractor should anticipate placing coarse, open graded, crushed rock, stabilization geo-textiles and/or using other methods to establish a firm platform for filling.

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. 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, the “modified Proctor.” Soils that classify as ML, MH, CL or CH 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, the “standard Proctor.”

No fill materials should be placed, worked, rolled while they are frozen, thawing, or during poor/inclement weather conditions. Where soils supporting foundations or on which foundation will be placed are exposed to freezing temperatures or repeated freeze – thaw cycling 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.

Care should be taken with regard to achieving and maintaining effective moisture contents during placement and compaction. We anticipate that the silts and silty sands comprising a significant proportion of the shallow site soils may exhibit significant pumping, rutting, and deflection at moisture contents above the optimum. In our experience, achieving and maintaining compaction in such soils can be very difficult if water contents are not monitored closely. The contractor should be prepared to handle soils of this type, including the use of chemical stabilization, if necessary.

Compaction areas should be kept separate, and no lift should be covered by another until relative compaction and moisture content within the recommended ranges are obtained.

Use of 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 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. Where “squeegee” meets the project requirements for pipe bedding material, however, it is acceptable for that use.

Utility Trench Backfilling

The recommendations above for fill placement and compaction are applicable to construction of trench backfills.

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 in this report to avoid damaging the pipe. Likewise, construction conditions may preclude density testing at specified frequencies in the lower portions of a trench. Such backfilling methods will lead to increased surface settlements.

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.

Quality Assurance

A geotechnical engineer should be retained to observe project excavations prior to placement of fill. That geotechnical engineer should observe earthwork operations and test the soils. That geotechnical engineer also should also provide a written declaration stating that the project site, including the building pad area, was filled with acceptable materials and was placed in general accordance with the requirements outlined in this report or otherwise specified for the project.

It should be noted that in the later stages of projects such as construction of the proposed facility, multiple sub-contractors commonly are installing or adjusting/replacing components of the project simultaneously. These can include utility laterals, electrical boxes, sidewalk access ramps, lighting fixtures and other components. 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 with sufficient frequency, and the soils tested, 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 3-foot fill, for example, this corresponds to settlement on the order of ½ inch, without imposition of foundation loads and in addition to any settlements due to consolidation of the underlying materials. 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.

APPENDIX D

Recommendations for
Surface and Subsurface Drainage

SURFACE AND SUBSURFACE DRAINAGE

Surface Drainage

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

II. Positive surface drainage measures should be provided and maintained to reduce water infiltration into foundation soils. 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 landscaped areas and 3 inches in the first 10 feet in areas where hardscaping covers the ground adjacent to the structures. (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. Ponding will lead to increased infiltration and post-construction building movements.

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

In GROUND's experience, it is common during construction that in areas of partially completed paving or hardscaping, bare soil behind curbs and gutters, and utility trenches, water is allowed to pond after rain or snow-melt events. Wetting of the subgrade can result in loss of subgrade support and increased settlements. By the time final grading has been completed, significant volumes of water can already have entered the subgrade, leading to subsequent distress and failures. The Contractor should maintain effective site drainage throughout construction so that water is directed into appropriate drainage structures.

III. The ground surface near foundation elements should be able to convey water away readily. Cobbles or other materials that tend to act as baffles and restrict surface flow should not be used to cover the ground surface near the foundations.

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.

IV. Roof downspouts and drains should discharge well beyond the perimeters of the structures foundations, or be provided with positive conveyance off-site for collected waters. Downspouts should not discharge into a building underdrain system.

V. Landscaping which requires watering should be located 10 or more feet from the building perimeter. 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.

Use of drip irrigation systems can be beneficial for reducing over-spray beyond planters. Drip irrigation also can be beneficial for reducing the amounts 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.

Where plantings are desired within 10 feet of the 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.

VI. 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 should be installed.

Underdrains

I. An underdrain system for the building should consist of perforated PVC collection pipe at least 4 inches in diameter, non-perforated PVC discharge pipe 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 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(s) should be wrapped with filter fabric (MiraFi 140N® or the equivalent) to reduce the migration of fines into the drain system.

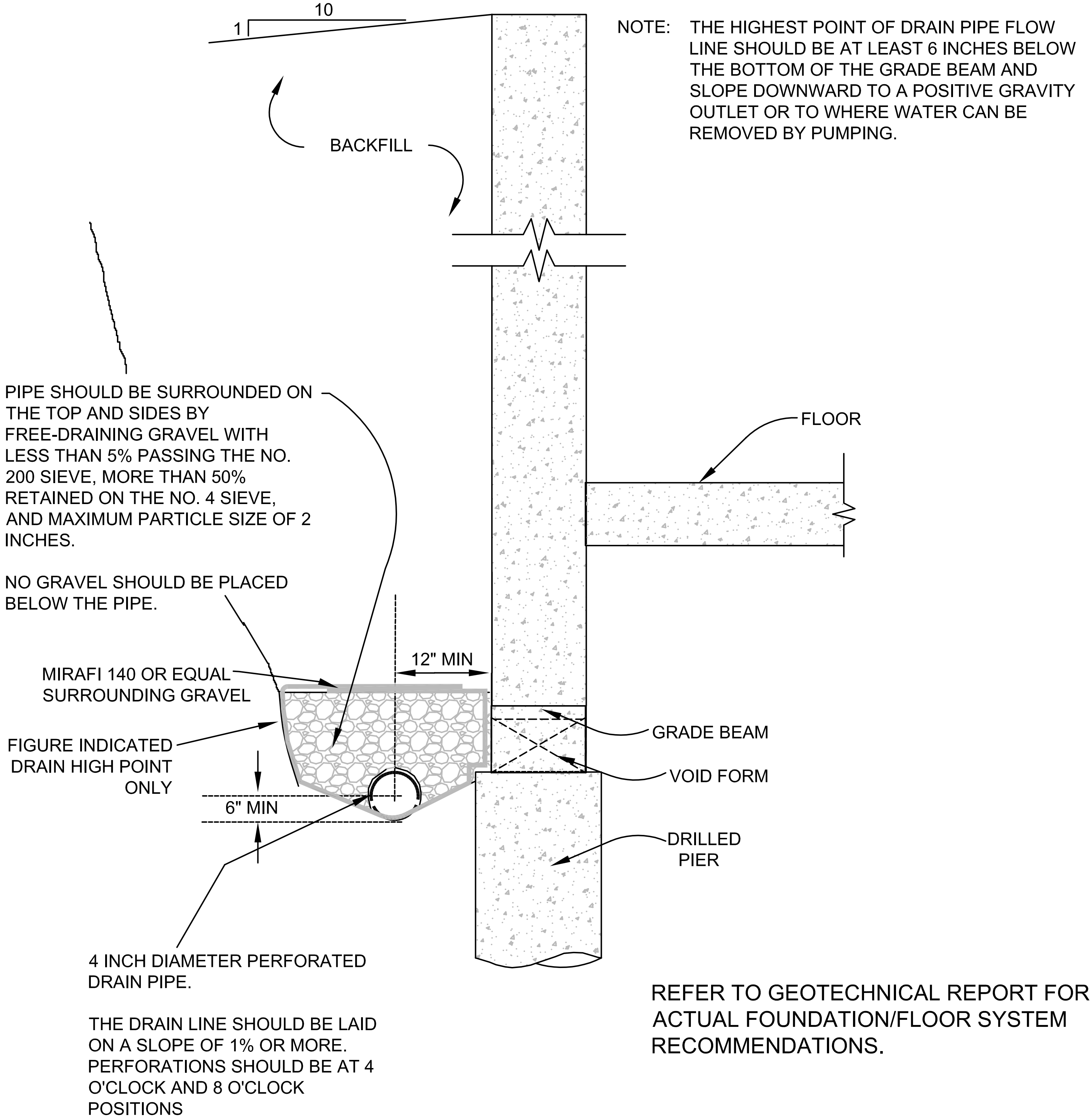
A typical, cross-section detail of an underdrain for projects of this type are provided below.

II. The high point(s) for the collection pipe flow lines should be at least 6 inches grade beam or footing bearing elevation. The collection and discharge pipe for the underdrain system should be laid on a slope sufficient for effective drainage. Pipe gradients should be designed to accommodate at least 1½ inches of differential movement after installation along a 50-foot run.

III. Underdrain 'clean-outs' should be provided at regular intervals to facilitate maintenance of the underdrains.

IV. The underdrain discharge pipes should be connected to one or more sumps from which water can be removed by pumping, or to outlet(s) for gravity discharge. We suggest that collected waters be discharged directly into the storm sewer system, if possible. The actual layout, outlets, and locations should be designed by the civil engineer.

V. The underdrain system should be tested by the contractor after installation and after placement and compaction of the overlying backfill to verify that the systems function properly.



NOTES:

1. THIS IS NOT A DESIGN-LEVEL DRAWING. IT SHOULD BE USED SOLELY FOR GENERAL INFORMATIONAL PURPOSES ONLY. ACTUAL UNDERDRAIN DESIGN SHOULD BE COMPLETED BY OTHERS.
2. THE UNDERDRAIN SYSTEM MUST BE TESTED BY THE CONTRACTOR AFTER INSTALLATION AND BACKFILLING TO VERIFY THAT IT FUNCTIONS PROPERLY.
3. INCLUSION OF THIS FIGURE IN CONSTRUCTION DOCUMENTS IS DONE SO AT THE DOCUMENT PREPARER'S RISK.

GROUND
ENGINEERING CONSULTANTS

TYPICAL UNDERDRAIN DETAIL

NON-PROJECT SPECIFIC	FIGURE: D1
CADFILE NAME: DRAIN.DWG	

APPENDIX E

Recommendations for
Pavement and Hardscape
Design and Construction

PAVEMENT AND HARDSCAPE DESIGN AND CONSTRUCTION

Pavement Materials

Asphalt pavement should consist of a bituminous plant mix composed of a mixture of aggregate and bituminous material. Asphalt mixture(s) should meet the requirements of a job-mix formula established by a qualified engineer as well as applicable municipal design requirements.

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 municipal design requirements. Concrete should have a minimum modulus of rupture of third point loading of 650 psi. Normally, concrete with a 28-day compressive strength of 4,200 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 7 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 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.

If composite flexible sections are placed, 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.”

Drainage and Maintenance

The collection and diversion of surface drainage away from paved areas is extremely important to satisfactory performance of the pavement. The subsurface and surface drainage systems should be carefully designed to ensure removal of the water from paved areas and subgrade soils. 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. The long-term performance of the pavement also can be improved greatly by proper backfilling and compaction behind curb, gutter, and sidewalk. Unless the interceptor drain and edge drains (where included) are installed properly and maintained, and site drainage in general is well maintained, there is an increased risk of poor pavement performance at this site due to the expansive subgrade materials and the local introduction of off-site irrigation water.

Landscape irrigation in planters adjacent to pavements and in "island" planters within paved areas should be carefully controlled or differential settlement and/or rutting of the nearby pavements will result. Drip irrigation systems are recommended for such planters to reduce over-spray and water infiltration beyond the planters. Enclosing the soil in the planters with plastic liners and providing them with positive drainage also will reduce differential moisture increases in the surrounding subgrade soils. In our experience, infiltration from planters adjacent to pavements is a principal source of moisture increase beneath those pavements. This wetting of the subgrade soils from infiltrating irrigation commonly leads to loss of subgrade support for the pavement with resultant accelerating distress, loss of pavement life and increased maintenance costs. This is particularly the case in the later stages of project construction after landscaping has been emplaced but heavy construction traffic has not ended. Heavy vehicle traffic over wetted subgrade commonly results in rutting and pushing of flexible pavements, and cracking of rigid pavements. In relatively flat areas where design drainage gradients necessarily are small, subgrade settlement can obstruct proper drainage and yield increased infiltration, exaggerated distress, etc. (These considerations apply to project flatwork, as well.)

Also, 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. This of this type is likely to occur even where the subgrade has been prepared properly and the asphalt has been compacted properly.

The anticipated traffic loading does not include excess loading conditions imposed by heavy construction vehicles. Consequently, heavily loaded concrete, lumber, and building material trucks can have a detrimental effect on the pavement. In areas where the maintenance traffic is turning, concrete pavement is recommended.

As noted above, 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 regular maintenance and rehabilitation procedures performed throughout the life of the pavement. Maintenance and rehabilitation measures preserve, rather than improve, the structural capacity of the pavement structure. Therefore, GROUND recommends that an effective program of regular maintenance be developed and implemented to seal cracks, repair distressed areas, and perform thin overlays throughout the lives of the pavements. The greatest benefit of pavement overlaying will be achieved by overlaying sound pavements that exhibit little or no distress.

Crack sealing should be performed at least annually and a fog seal/chip seal program should be performed on the pavements every 3 to 4 years. After approximately 8 to 10 years after construction, patching, additional crack sealing, and asphalt overlay may be required. Prior to overlays, it is important that all cracks be sealed with a flexible, rubberized crack sealant in order to reduce the potential for propagation of the crack through the overlay. If actual traffic loadings exceed the values used for development of the pavement sections, however, pavement maintenance measures will be needed on an accelerated schedule.

Construction and Drainage Between Buildings and Pavements

Proper design, drainage, construction and maintenance of the areas between individual buildings and parking/driveway areas are critical to the satisfactory performance of the project. Sidewalks, entranceway slabs and roofs, fountains, raised planters and other highly visible improvements commonly are installed within these zones, and distress in or near these improvements is common. Commonly, proper soil preparation in these areas receives little attention during overlot 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.

Frost and Ice Considerations

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 (“ice jacking”) 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 ice jacking is a concern. GROUND will be available to discuss these concerns upon request.

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.

APPENDIX F

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. 09-6013
Fruita Recreation Center
Automobile-Only Parking
Full Depth Asphalt

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	38,500
Initial Serviceability	4.5
Terminal Serviceability	2
Reliability Level	80 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	3,942 psi
Stage Construction	1
Calculated Design Structural Number	2.34 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Asphalt	0.4	1	6	-	2.40
Total	-	-	-	6.00	-	2.40

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

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Computer Software Product

Network Administrator

Flexible Structural Design Module

Job No. 09-6013
Fruita Recreation Center
Automobile-only Parking
Composite Section

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	38,500
Initial Serviceability	4.5
Terminal Serviceability	2
Reliability Level	80 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	3,924 psi
Stage Construction	1
Calculated Design Structural Number	2.34 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Asphalt	0.4	1	4	-	1.60
2	Roadbase	0.12	1	7	-	0.84
Total	-	-	-	11.00	-	2.44

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Network Administrator

Flexible Structural Design Module

Job No. 09-6013
Fruita Recreation Center
Drive Aisles
Full Depth Asphalt

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	73,000
Initial Serviceability	4.5
Terminal Serviceability	2
Reliability Level	80 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	3,942 psi
Stage Construction	1
Calculated Design Structural Number	2.58 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Asphalt	0.4	1	6.5	-	2.60
Total	-	-	-	6.50	-	2.60

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Network Administrator

Flexible Structural Design Module

Job No. 09-6013
Fruita Recreation Center
Drive Aisles
Composite Section

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	73,000
Initial Serviceability	4.5
Terminal Serviceability	2
Reliability Level	75 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	3,942 psi
Stage Construction	1
Calculated Design Structural Number	2.51 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Asphalt	0.4	1	4	-	1.60
2	Roadbase	0.12	1	8	-	0.96
Total	-	-	-	12.00	-	2.56

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Network Administrator

Rigid Structural Design Module

Job No. 09-6013
Fruita Recreation Center
Heavy Vehicle / High Turn Stress
Concrete Sections

Rigid Structural Design

Pavement Type	JPCP
18-kip ESALs Over Initial Performance Period	365,000
Initial Serviceability	4.5
Terminal Serviceability	2
28-day Mean PCC Modulus of Rupture	650 psi
28-day Mean Elastic Modulus of Slab	3,400,000 psi
Mean Effective k-value	22 psi/in
Reliability Level	80 %
Overall Standard Deviation	0.34
Load Transfer Coefficient, J	3.6
Overall Drainage Coefficient, Cd	1
Calculated Design Thickness	6.28 in