GROUND
ENGINEERING

Preliminary Geotechnical Evaluation
Parker and Stroh Project
Douglas County, Colorado

Prepared For:

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a preliminary geotechnical evaluation performed by GROUND Engineering Consultants, Inc. (GROUND) to provide in support of planning of a proposed commercial development at the southeast corner Parker Road and Stroh Road near Parker, Colorado. Our study was conducted in general accordance with GROUND's Proposal No. 1705-0845 Revised2, dated August 23, 2017.

Field and office studies provided information obtained at the test hole locations regarding surface and subsurface conditions, including the existing site vicinity improvements. 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 our geotechnical analysis. Results of the field, office, and laboratory studies are presented below.

This report has been prepared to summarize the data obtained and to present our preliminary findings and conclusions based on the proposed development and the subsurface conditions encountered.

The preliminary or initial information presented in this report is not sufficient for final design of building or pavements, or for assessing depths of remedial earthwork, etc. Additional, structure-specific subsurface exploration and testing should be performed in support of development of geotechnical parameters for design.

PLANNED DEVELOPMENT

We understand that present plans envision 14 retail lots, a detention pond, retaining walls, and paved entrances/exits, drive lanes, and parking areas. Structural loads likely will be low, typical of these types of many commercial structures. Additionally, we anticipate that cuts and fills on the order of 30 feet may be required to achieve final lines and grades across portions of the site.

If the proposed development differs significantly from that described above, GROUND should be notified to re-evaluate the conclusions and parameters contained herein.

SITE CONDITIONS

At the time of our subsurface exploration program, the site consisted of a partially developed parcel that had been previously used as a farm. The site was bordered by Parker Road to the west, Stroh Road to north, and undeveloped land to the east and south. Commercial development, single-family residences, a golf course, construction operations, and undeveloped land further surrounded the site. Various streams and drainages also were observed near the project site, primarily a reach of Kinney Creek which followed from east to west near the southern margin of the project site and a reach of Stroh Gulch was observed to the north of the project site. Construction operations were observed to the north of the project site and appeared to be changing site grades and modifying the channel of Stroh Gulch.

The project site was covered with native grasses, weeds, and other relatively small vegetation. Several large, mature trees were observed near the southern margin of the property and near the existing buildings. Review of historic imagery suggested that development at the project site was generally limited to construction of the existing farm buildings and single-family residence at the project site. Additionally, we understand that two water wells registered with the state of Colorado are present on site.

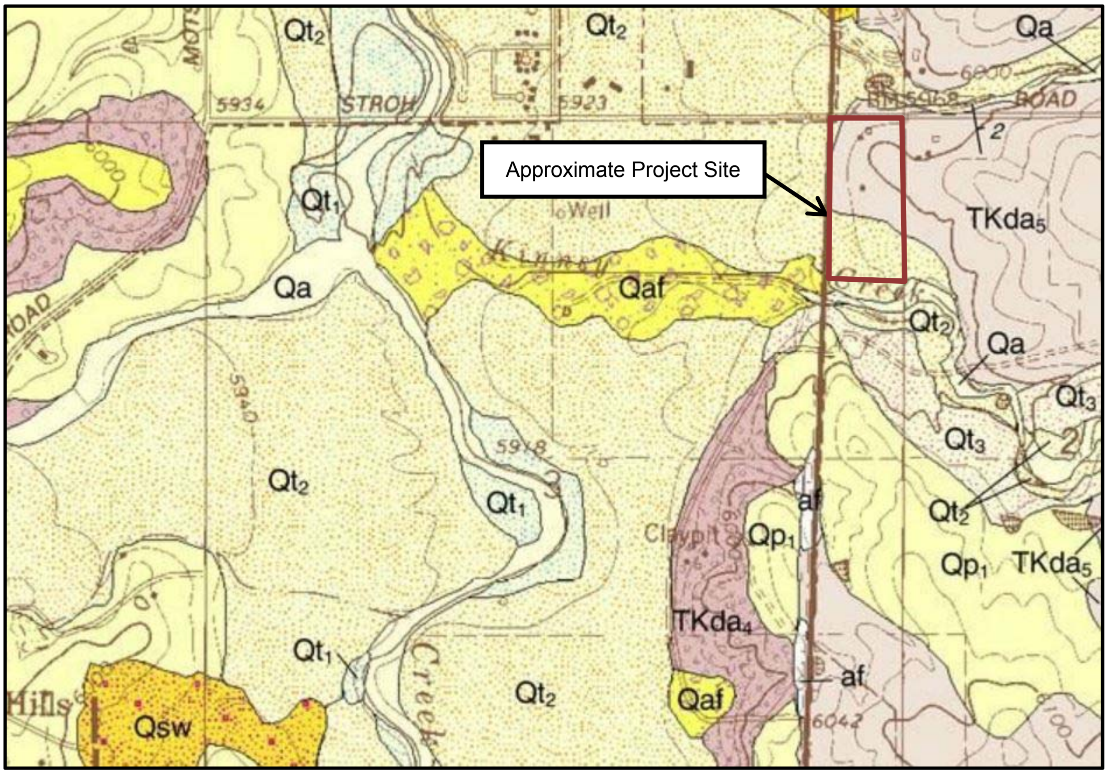
A hill was observed in the northeast portion of the site and slopes descended in all directions from this point. Topographic maps of the project area, including the Castle Rock North Quadrangle,[1] depict more than 40 feet of relief across the project site. The site slope was generally gentle to moderate; however, relatively steep slopes were observed near the hill. The greater project area generally sloped to the west and north towards the Cherry Creek channel.

[1] U.S. Geological Survey, 1994, *Castle Rock Quadrangle, Colorado, 7.5 Minute Series (Topographic),* DMA 5062 1V NE-Series V877. Scale 1:24,000.

GEOLOGIC SETTING AND HAZARDS

Site Geology

Published geologic maps, e.g., Thorson (2005)[2] depict the site as underlain by late Pleistocene Terrace Alluvium Two (**Qt$_2$**) other alluvial deposits are also mapped in the greater project area. The surficial deposits are shown as underlain by Paleocene to Eocene strata of the Dawson Formation (**TKda$_5$** and **TKda$_4$**).



In the project area, alluvial (stream-laid) deposits typically consist of fine to coarse sands and gravels with cobbles and scattered boulders. Lenses and beds of silts and clays are also commonly found in these deposits. The Dawson Formation, in the project area, consists largely of claystones and sandstones interbedded on various scales. The claystones typically are moderately to highly expansive and the sandstones include well-cemented beds that can be very hard and can be difficult to excavate.

[2] Thorson, J.P., 2005, *Geologic Map of the Castle Rock North Quadrangle, Douglas County, Colorado*: Colorado Geological Survey, Open-File Report OF05-02, scale 1:24,000.

We interpret the native sands to be alluvial terrace deposits. We interpret the native sands and clays encountered in the test holes to be colluvium. We interpret the underlying bedrock materials to be Dawson Formation materials.

Geologic Hazards

Expansive Soils Swelling clayey soils and bedrock change volume in response to changes in moisture content that can occur seasonally, or in response to changes in land use, including development. Expansion potentials vary with moisture contents, density, and details of the clay chemistry and mineralogy. The swell potential in any particular area can vary markedly both laterally and vertically due to the complex interbedding of the site soil and bedrock materials. Moisture changes also occur erratically, resulting in conditions that cannot always be predicted.

The shallow earth materials underlying the site generally consisted of sands and clays. The plasticity of the site soils ranged from non- to highly plastic. A swell of 0.9 percent was measured in a sample of site overburden materials. (See Table 1.) We understand that significantly higher swell results have been obtained by others in the greater project area. Swelling soils beneath a building's foundation can cause moderate to significant damage. Design level geotechnical evaluations of individual building sites should include an assessment of the possible presence of swelling materials in the foundation soils, so that appropriate, remedial design and construction can be implemented. For planning purposes, more severely swelling soils should be assumed.

Collapsible Soils Certain surficial deposits in the Denver metropolitan area, typically eolian (wind-blown) materials are known to be susceptible to local hydro-consolidation or "collapse." Hydro-consolidation consists of a significant volume loss due to re-structuring of the constituent grains of the soil to a more compact arrangement upon wetting.

The some of the surficial soils near the site are not mapped as eolian deposits and the soils encountered during our subsurface exploration program by have not been interpreted by GROUND to be eolian deposits. Additionally, the index parameters for site soils assessed for this study did not fall into the range typically associated with collapsible soils (e.g. Naval Facilities Engineering Command, 1986[3]).

Swell and consolidation testing performed on site materials indicated consolidations were about 0.2 percent. Design level geotechnical evaluations of individual building sites should include an assessment of the possible presence of collapsible materials in the foundation soils, so that appropriate, remedial design and construction can be implemented, where necessary.

Radon Testing for the possible presence of radon gas prior to project development does not yield useful results regarding the potential accumulation of radon in completed structures. Radon accumulations typically are found in basements or other enclosed portions of buildings built in areas underlain at relatively shallow depths by granitic crystalline rock. The likelihood of encountering radon in concentrations exceeding applicable health standards on the subject site, underlain by relatively deep soils and sedimentary bedrock, is significantly lower.

Radon testing should be performed in each building on-site, after construction is completed. Proper ventilation usually is sufficient to mitigate potential radon accumulations. Building designs should accommodate such ventilation for all building areas.

Seismic Activity / Faulting Neither site reconnaissance nor review of available geologic maps indicated the trace of an active or potentially active fault traversing or immediately adjacent to the site. Therefore, the likelihood of surface fault rupture at the site is considered to be relatively low.

The closest extent of a documented active fault to the site is the Perry Park – Jarre Canyon Fault, which is located approximately 15 miles to the southwest (Kirkham and Rogers, 1981[4]; Colorado Geological Survey, 2008[5]). This fault has a cumulative length of approximately 19 miles, trends generally north, and is generally considered to be a west dipping, moderate angle, range-front, Laramide reverse fault with late Cenozoic displacement.[4] The risk of this fault giving rise to damaging, earthquake-induced

[3] Naval Facilities Engineering Command, 1986, *Design Manual 7.01, Soil Mechanics*, 348 pp.
[4] Kirkham, R.M., and W. P. Rogers, 1981, *Earthquake Potential in Colorado, A Preliminary Evaluation*, Colorado Geological Survey, Bulletin 43.
[5] Colorado Geological Survey, 2008, *Preliminary Quaternary Fault and Fold Database, and Map Server*, http://geosurvey.state.co.us/Default.aspx?tabid=453.

ground motions at the site is considered to be relatively low give the magnitudes of previously recorded events in the project area and last known movements of the fault.

We consider the much of the northern portion of the site likely to qualify as a Seismic Site Class C site, in accordance with 2015 IBC based on extrapolation of available data to depth. Because the depth to bedrock appears to increase to the south, the southern portion of the site will likely qualify as Seismic Site Class D. Additionally, where relatively large volumes of fill are placed, a Seismic Site Class of D may also be appropriate. If a quantitative assessment of the classification is needed, shear wave velocity testing to 100+ feet or other quantitative method(s) will be required. A proposal for this work can be provided upon request. Compared with other regions of Colorado, recorded earthquake frequency in the project area is moderate.

Slope Stability and Erosion Colton and others (1975)[6], as well as larger scale geologic maps providing coverage of the site that were reviewed for this study, did not depict landslide deposits on or adjacent to the subject site.

The site is relatively gently to locally steeply sloping. During our preliminary reconnaissance of site area, no evidence was noted of mass-wasting processes associated with steep slopes, such as landslides, slumps, or unusual soil creep. Therefore, the likelihood of project developments being affected by existing large scale, unanticipated slope instabilities is considered low. Because relatively large cuts and fills are planned for the site the risk of slope instability may be increased. Therefore, constructed or modified slopes should be analyzed for stability, as appropriated.

Flooding The subject property abuts the north bank of the Kinney Creek and a portion of the site is mapped as Zone A by FEMA (2016).[7] Zone A indicates a 1 percent annual chance of flood (100-year flood), but base flood elevations with the zone have not been determined. The remainder of the site is mapped as Zone X which indicates a minimal risk of flooding. Significant fluctuation in the flow of streams can be expected during seasonal run periods and after large precipitation events.

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[6] Colton, R.B., J.A. Holligan, and L.W. Anderson, 1975, *Preliminary Map of Landslide Deposits, Denver 1° x 2° Quadrangle, Colorado*, U.S. Geological Survey, Miscellaneous Field Studies Map MF-705.

[7] Federal Emergency Management Administration, 2016, https://map1.msc.fema.gov/idms/IntraView.cgi?KEY=98996423&IFIT=1 accessed on 9/29/2017.

The site and proposed grade changes should be evaluated by a civil engineer regard to flooding.

Mining Activity and Subsidence Review of U.S. Geological Survey geologic maps covering the site, such as Turney and Murray-Williams (1983)[8] and other available, published maps depicting areas of coal extraction, do not indicate past mining activities on or adjacent to the subject parcel. Additionally, no evidence of past mining activity was identified at the project site during our subsurface exploration program, nor were coal and/or lignite lenses were identified within the bedrock in the test holes.

Considering the information presented above, the risk of potential for surface subsidence associated with consolidation of former mine workings at depth appears to be low.

Published geologic maps do not indicate formations underlying the site at shallow depths that include evaporite (salt, gypsum, etc.) deposits, limestones, or other materials vulnerable to subsurface dissolution. Therefore, the likelihood of subsidence (associated with such rock formations; not coal mine related) or other hazards related to subsurface dissolution appears to be low.

INITIAL SUBSURFACE EXPLORATION

Subsurface exploration for the project was conducted in September 2017. A total of four (4) test holes were drilled using a conventional truck-mounted, drilling rig advancing continuous flight auger equipment to depths of about 24 to 36 feet below existing grade near the proposed footprint areas. Test holes were advanced to their termination depths to evaluate the subsurface conditions as well as to retrieve samples for laboratory testing and analysis. A GROUND engineer directed the subsurface exploration, logged the test holes in the field, and prepared the 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 hammer falling 30 inches, in general accordance with (in the case of the 1⅜-inch sampler) the Standard Penetration Test described by ASTM Method D1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of soils. Depths at which the

[8] Turney, J.E., Murray-Williams L., 1983, *Colorado Front Range Inactive Coal Mine Data and Subsidence, Douglas County*, Colorado Geological Survey, Plate 4 of 12.

samples were obtained and associated penetration resistance values are shown on the test hole logs.

The approximate locations of the test holes are shown on Figure 1. Logs of the test holes are presented on Figures 2. Explanatory notes and a legend are provided on Figure 3.

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 included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, and Atterberg limits. Swell and consolidation testing and a suite of corrosivity tests were performed on select samples as well. Laboratory tests were performed in general accordance with applicable ASTM protocols. Results of the preliminary laboratory testing program are summarized in Table 1 and Table 2.

SUBSURFACE CONDITIONS

In general, the test holes penetrated a thin layer of top soil,[9] typically 2 to 3 inches thick, before penetrating a variably thick layer of native sands that was underlain by sandstone and siltstone bedrock that extended to the depths explored. The upper 1 to 2 feet of the bedrock was commonly weathered. In Test Hole 1, approximately 3 feet fill soils were encountered overlying native sands and clays that extended to a depth of 9 feet below existing grade. At this depth the sandstone and siltstone bedrock was encountered and extended to the depths explored.

Fill materials were recognized in the Test Hole 1, but the delineation of the complete lateral and vertical extents of the fills at the site, and their composition, was beyond our present scope of services. If detailed fill soil compositions at the site are of significance, they should be evaluated using test pits.

Fill consisted silt, clay, and fine to medium sands and was slightly moist to moist, moderately to highly plastic, and brown in color.

[9] "Topsoil" as used herein is defined geotechnically. The materials so described may or may not be suitable for landscaping or as a growth medium for such plantings as may be proposed for the project.

Sands and Clays ranged from silty to clayey, fine to coarse sand and sandy clay. They were slightly to moist, moderately to highly plastic, and light brown to brown in color. Iron oxide staining and caliche were observed locally.

Sands consisted of silty, clayey, and clean, fine to coarse sands with gravels and silt and clay lenses locally. They were dry to wet, non- to slightly plastic, loose to medium dense, and light brown to brown. Iron oxide staining was noted locally.

Weathered Sandstone and Siltstone weathered fine to coarse grained sandstone and siltstone with claystone beds locally. They were moist to very moist, moderately to highly plastic, medium hard, and gray to yellow green. Iron staining was noted locally.

Sandstone and Siltstone Bedrock consisted of interbedded fine to coarse grained sandstones and siltstones with local claystone beds. They were moist to wet, moderately to highly plastic, hard to very hard, and brown to yellow green to gray in color. Iron staining was encountered locally.

Swell-Consolidation Testing of site materials indicated consolidations up to 0.2 percent and swells up to 0.9 percent upon wetting under surcharges equal to the approximate overburden load.

Groundwater was encountered in Test Hole 4 at a depth of 14 feet below existing grade at the time of drilling. When the depth to groundwater was measured in Test Hole 4 approximately 1 day after drilling, groundwater was measured to be approximately 13 feet below existing grade. The test hole was then backfilled. The remaining test holes were backfilled immediately after drilling due to safety concerns. Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, land use, and the development of transient, perched water conditions.

PRELIMINARY GEOTECHNICAL EVALUATION

Presented below are preliminary consideration and parameters regarding geotechnical aspects of the proposed retail development, which we have assumed will be relatively lightly loaded. The provided considerations and parameters are intended only to assist with preliminary project planning. Additional, structure-specific studies must be performed to develop design-level geotechnical considerations and parameters.

Construction on Existing Site Soils Undocumented fill soils were recognized in Test Hole 1 and may be present elsewhere on site. Undocumented fills present a largely unquantifiable risk for damaging post-construction movement that can affect nearly all of the proposed improvements to the site. Post-construction movements of improvements bearing on undocumented fill soils can be on the order of several inches and can include lateral movements.

Common methods of mitigating the risks to improvements posed by undocumented fill soils include the use of deep or intermediate foundation systems, and excavation and re-compaction of the undocumented fill soils. Deep foundations bearing below the zone of soil movement both support the structural loads and anchor a structure against heave or consolidation of undocumented fills. Intermediate systems such as rammed aggregate piers can improve the surrounding soils in place. Excavating the undocumented fills soils, processing them to remove unsuitable materials and achieve appropriate soils moisture contents, and re-compacting them as fill will reduce – but not eliminate – the potential for further heave or consolidation. The entire thickness of undocumented fill soils will likely need to removed and replaced. In the location of Test Hole 1, this would likely be a depth of about 3 feet; however deeper fills may be present.

The native soils encountered in the test holes, generally appear to be geotechnically suitable to support the proposed construction in their current state using shallow foundation systems. However, if expansive or collapsible soils are encountered remedial methods may be required to mitigate the risks of damaging post-construction movements. The mitigation methods described for undocumented fill soils will generally be appropriate for expansive and collapsible soils.

Anticipated Foundation and Floor Systems Based on the conditions encountered in our preliminary test holes, it appears shallow foundations and slab-on-grade floors will generally be appropriate for site construction bearing either on native soils or a section of properly compacted fill soils. Where structures are more sensitive to movement, where post-construction movements less than 1 inch are desired, deep foundation systems and structural floor system will likely be required.

Deep Foundation Systems Deep foundation systems will need to bear in the bedrock underlying the site. We anticipate that both drilled pier and driven pile foundations will be geotechnically feasible. Drilled pier lengths will depend on a variety of geotechnical and structural factors, but we anticipate that pier lengths will be between 25 and 35 feet. Allowable end-bearing pressures typically will be 30,000 psf or higher. Estimated post-construction movements of a properly designed and installed deep foundation likely will be on the order of ½ inch or less.

Shallow Foundation Systems We anticipate that, in general, shallow foundation systems will be able be bear on the existing native soils or a section of properly compacted fill where grades are changed. Where undocumented fill soils underlie proposed construction, those materials will need to be removed and replaced as properly compacted fill. Allowable bearing pressures likely will range from 2,000 to 3,000 psf for post-construction movements of approximately 1 inch.

Shallow foundation systems bearing on rammed aggregate piers commonly can be designed for allowable bearing pressure of 5,000 to 7,000 psf.

Structural Floors Where deep foundations are appropriate for a building, constructing the lowest floor as a structural floor also supported on deep foundations also generally will be appropriate. Estimated post-construction movements of a properly designed and installed structural floor likely will be on the order of ½ inch or less. A structural floor should be constructed over a well-ventilated crawl space. Hanging the utilities beneath a building with a structural floor from that floor system (rather than installing them in the underlying soils) can limit differential structure / pipe movements on an expansive soil site.

Slab-on-grade Floors Similar to shallow foundation systems, conventional, slab-on-grade concrete floors may bear on existing native soils or a section of properly compacted fill where grades are raised. Again, where undocumented fill soils underlie the proposed construction, those materials will need to be removed and replaced as properly compacted fill. In our experience, estimates of post-construction floor movements bearing such soils will be on the order of 1 inch.

Flatwork It may be beneficial to support critical flatwork, such as flatwork at entrance and exits of buildings, on deep foundation systems and structural floors (as appropriate), or on a fill section like a slab-on-grade floor where there are low tolerances for movements. Less critical flatwork can be supported on lesser section of compacted fill in order to reduce post-construction movements. Increased maintenance, however, including periodic removal and replacement of flatwork, should be anticipated for this site.

Preliminary Water-Soluble Sulfates The concentration of water-soluble sulfates measured in a selected sample of site soils was approximately <0.01 percent by weight. (See Table 2.) Such a concentration of soluble sulfates represents a **negligible** environment for 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 (PCA). Based on these preliminary data, sulfate-resistant cement likely will not be necessary in project concrete. Additional testing for the presence of water-soluble sulfates should be performed as part of design-level geotechnical studies, however.

Preliminary Soil Corrosivity Data were obtained to support an initial assessment of the potential for corrosion of ferrous metals in contact with earth materials at the site, based on the conditions at the time of GROUND’s evaluation. The test results are summarized in Table 2.

Reduction-Oxidation testing in selected samples indicated a red-ox potential of -73 millivolts. Such a low potential typically create a more corrosive environment.

Sulfide Reactivity testing indicated a ‘positive’ results in the local soils. The presence of sulfides in the soils 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 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. Measurement of electrical resistivity indicated a value of approximately 12,788 ohm-centimeters in a selected sample of site soils.

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.[10] Testing of a selected sample of site soils indicated pH values of about 7.8.

Preliminary Corrosivity Assessment Taken together, the above data suggest a moderately corrosive site for ferrous metals. In our experience, however, the local claystones and claystone-derived soils commonly represent a relatively severely corrosive environment. Additional, structure-specific corrosivity assessments should be performed for final design.

Surface and Subsurface Drainage Wetting of the bearing soils can lead to loss of support and greater than anticipated settlements on most sites. Project plans should include establishment and maintenance of effective drainage (both surface and subsurface).

Underdrains can be a beneficial part of the site drainage program, and given the relatively clayey site soils, underdrains likely will be necessary to develop effective drainage. Additionally, buildings will below-grade or partially below-grade levels should be provided with underdrains.

Preliminary Pavements Sections *The following initial or preliminary pavement sections are based on current grades and the shallow on-site soils. Final pavement design must be completed once final grades are established.*

Preliminary Pavement Thicknesses Pavement sections supported by the local soils likely will consist of 4 to 5 inches of asphalt (or an equivalent composite section) in parking areas and 5 to 6 inches of asphalt (or an equivalent composite section) in the driveways. Areas of heavy truck traffic such as trash collections zones and shipping and receiving routes will likely require 6½ to 7½ inches of concrete pavement, placed over 6 inches of aggregate base course

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

Remedial Earthwork Based on CDOT guidelines and the soil conditions encountered during this preliminary evaluation, the subgrade would need to be excavated to a depth of 1 to 2 feet or more (depending on Plasticity Index of soils / bedrock exposed near final surface grades), moisture-conditioned and replaced as properly re-compacted fill. Greater depths of subgrade preparation may be appropriate in some areas, however, due to the presence of collapsible and swelling native soils.

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.

OVERLOT GRADING

The earthwork criteria below are based on our interpretation of the geotechnical conditions encountered in the test holes and are only considered appropriate for use in overlot grading at this time. Earthwork parameters and considerations for construction at individual pads, project pavements, etc. must be addressed by project (pad) specific geotechnical evaluations. Additionally, where these criteria differ from applicable municipal specifications, e.g., for trench backfill compaction along a public utility line, the latter should be considered to take precedence.

General Considerations Project 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 subsequent construction.

Prior to earthwork construction, existing construction debris, 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 properly capped.

Use of Existing Native Soils Based on the samples retrieved from the test holes, we anticipate that the existing site soils free of organic materials, coarse cobbles, boulders, or other deleterious materials will be suitable, in general, for re-use as compacted fill.

Use of Existing Fill Soils We anticipate that the great majority of the existing fill soils – if effectively cleaned of deleterious materials that may be present – can be replaced as properly compacted fill. The contractor should be prepared to screen and sort the excavated fill to remove deleterious materials. With regard to organic content, soils exhibiting more than **4 percent** (by weight of dry soil) organic material should be excluded from project fills as deleterious.

Cobbles and fragments of rock, as well as inert construction debris, e.g., concrete or asphalt, up to **3 inches** in maximum dimension may be included in project fills, in general. Such materials should be evaluated on a case-by-case basis where identified during earthwork.

Imported Fill Materials Materials imported to the site as (common) fill should be free of organic material, and other deleterious materials. Imported material should exhibit **less than 20 percent** passing the No. 200 Sieve and a plasticity index of **15 or less**. Materials proposed for import should be approved prior to transport to the site.

Fill Platform Preparation Prior to filling, the top **12 inches** of in-place materials on which fill soils will be placed (except for utility trench bottoms where bedding will be placed) should be scarified, moisture conditioned and properly compacted in accordance with the criteria 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. A surface to receive fill must be effectively stable prior to placement of fill, including trench bottoms prior to placement of bedding.

Where fill is to be placed on slopes steeper than 3 : 1 (horizontal : vertical), including both natural and previously constructed embankments, the slopes should be benched. The benches shall be cut 10 feet horizontally into the existing slope to create a stepped bench condition. (Where groundwater seepage is encountered or anticipated, the benches should be provided with back drains. In such cases, the bench surface should be sloped back toward the drain.) The vertical step should not exceed 4 feet between benches.

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

The site soils, especially the relatively large clasts of bedrock that may be generated by project cuts, will require a well-coordinated effort to moisture treat, process, place, and compact properly. Bedrock clasts should be broken down in to a soil-like mass with no bedrock fragments coarse than **3 inches**. Greater than typical watering, and compaction equipment that aids in breaking down such material (e.g., a Caterpillar 825 compactor-roller), likely will be needed for these materials. Crushing or other methods should be anticipated to sufficiently reduce bedrock where encountered. Applied water will be taken up into the structures of the mineral grains comprising the bedrock. The contractor should anticipate that handling and processing the excavated bedrock more than once may be necessary to achieve the requirements herein.

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

Where soils on which foundation elements 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 proper moisture contents during placement and compaction. Materials that are not properly moisture conditioned may exhibit significant pumping, rutting, and deflection at moisture contents near optimum and above. The contractor should be prepared to handle soils of this type, including the use of chemical stabilization, if necessary.

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

General Compaction Criteria 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 dry density at moisture contents **within 2 percent** of the optimum moisture content as determined by ASTM D1557, the ‘modified Proctor.’

Soils that classify as **ML, MH, CL, or CH** should be compacted to **at least 95 percent** of the maximum dry density at moisture contents between **one percent below and 3 percent above** the optimum moisture content as determined by ASTM D698, the 'standard Proctor.'

Settlements Settlements will occur in newly filled ground, typically on the order of 1 to 2 percent of the fill depth. This is separate from settlement of the existing soils left in place. For a 6-foot fill, for example, that corresponds to a total settlement of about 1 inch. If fill placement is performed properly and is tightly controlled, in GROUND's experience the majority (on the order of 60 to 80 percent) of that settlement typically will take place during earthwork construction, provided the contractor achieves the compaction levels indicated herein. The remaining potential settlements likely will take several months or longer to be realized, and may be exacerbated if these fills are subjected to changes in moisture content.

Cut and Filled Slopes Permanent, graded slopes supported by local soils up to **15 feet** in height should be constructed no steeper than **3 : 1** (horizontal : vertical). Minor raveling or surficial sloughing should be anticipated on slopes cut at this angle until vegetation is well re-established. Surface drainage should be designed to direct water away from slope faces into designed drainage pathways or structures.

Steeper slope angles and heights may be possible but will require detailed slope stability analysis based on final proposed grading plans. A geotechnical engineer should be retained to evaluate this on a case-by-case basis.

EXCAVATION CONSIDERATIONS

Excavation Difficulty Test holes for the subsurface exploration were advanced to the depths indicated on the test hole logs by means of conventional, truck-mounted, geotechnical drilling equipment. We anticipate no unusual excavation difficulties, in general, for the proposed construction in the relatively shallow site soils with conventional, heavy-duty excavating equipment in good working condition. However, due to the unknown nature of the fill soils, debris or other materials, that may require special handle or that may particularly awkward or difficult to handle, may be encountered in the fill soils. Similarly, gravel, cobbles, and boulders are often presence locally in alluvial deposits, such as those encountered on site, and may be encountered in project excavations.

Temporary Excavations and Personnel Safety Excavations in which personnel will be working must comply with all applicable 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 Parker & Stroh, LLC and is not assuming responsibility for construction site safety or the contractor's activities.

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

Temporary, un-shored excavation slopes up to **15 feet** in height, in general, should be cut no steeper than **3 : 1** (horizontal : vertical) in the on-site soils in the absence of seepage. Some surface sloughing may occur on the slope faces at these angles. Should site constraints prohibit the use of the above-indicated slope angle, temporary shoring should be used. GROUND is available to provide shoring design upon request.

Groundwater and Surface Water Groundwater was measured in Test Hole 4 to be as shallow as 13 feet. Therefore, groundwater (other than limited volumes of perched water) is not anticipated in shallow project excavations below at depth of 10 feet near Test Hole 4. It should be noted that there was significant elevation difference between Test Hole 4 and other test holes at the project site and appropriate adjustment regarding elevation should be made when considering this datum.

Where seepage or flowing groundwater be encountered in project excavations, the slopes should be flattened as necessary to maintain stability or a geotechnical engineer should be retained to evaluate the conditions. The risk of slope instability will be significantly increased in areas of seepage along excavation slopes.

The contractor should take pro-active measures to control surface waters during construction and maintain good surface drainage conditions to direct waters away from excavations and into appropriate drainage structures. A properly designed drainage swale should be provided at the tops of the excavation slopes. In no case should water be allowed to pond near project excavations.

Temporary slopes should also be protected against erosion. Erosion along the slopes will result in sloughing and could lead to a slope failure.

LIMITATIONS

This preliminary report has been prepared for Parker & Stroh, LLC as it pertains to planning of the proposed development of the subject site and related improvements as described herein. It may not contain sufficient information for other parties or other purposes. If any information referred to herein is not well understood, then Parker & Stroh, LLC, or anyone using this report, should contact the author or a GROUND principal immediately. GROUND will be available to meet to discuss the risks and remedial approaches presented in this report, as well as other potential approaches, upon request.

The geotechnical conclusions in this report relied upon subsurface exploration at a limited number of exploration points, as shown in Figure 1, as well as the means and methods described herein. Subsurface conditions were interpolated between and extrapolated beyond these locations. It is not possible to guarantee the subsurface conditions are as indicated in this report. The conditions exposed during design-level evaluations may differ from those encountered during subsurface exploration for this study.

This report provides the findings of our preliminary geotechnical evaluation of the site. It does not include sufficient geotechnical information or parameters for final design of buildings or other improvements. We assume additional, structure-specific, subsurface exploration and testing will be performed to provide information for design.

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

Sincerely,

GROUND Engineering Consultants, Inc.

Ben Fellbaum, G.I.T.

10/10/2017

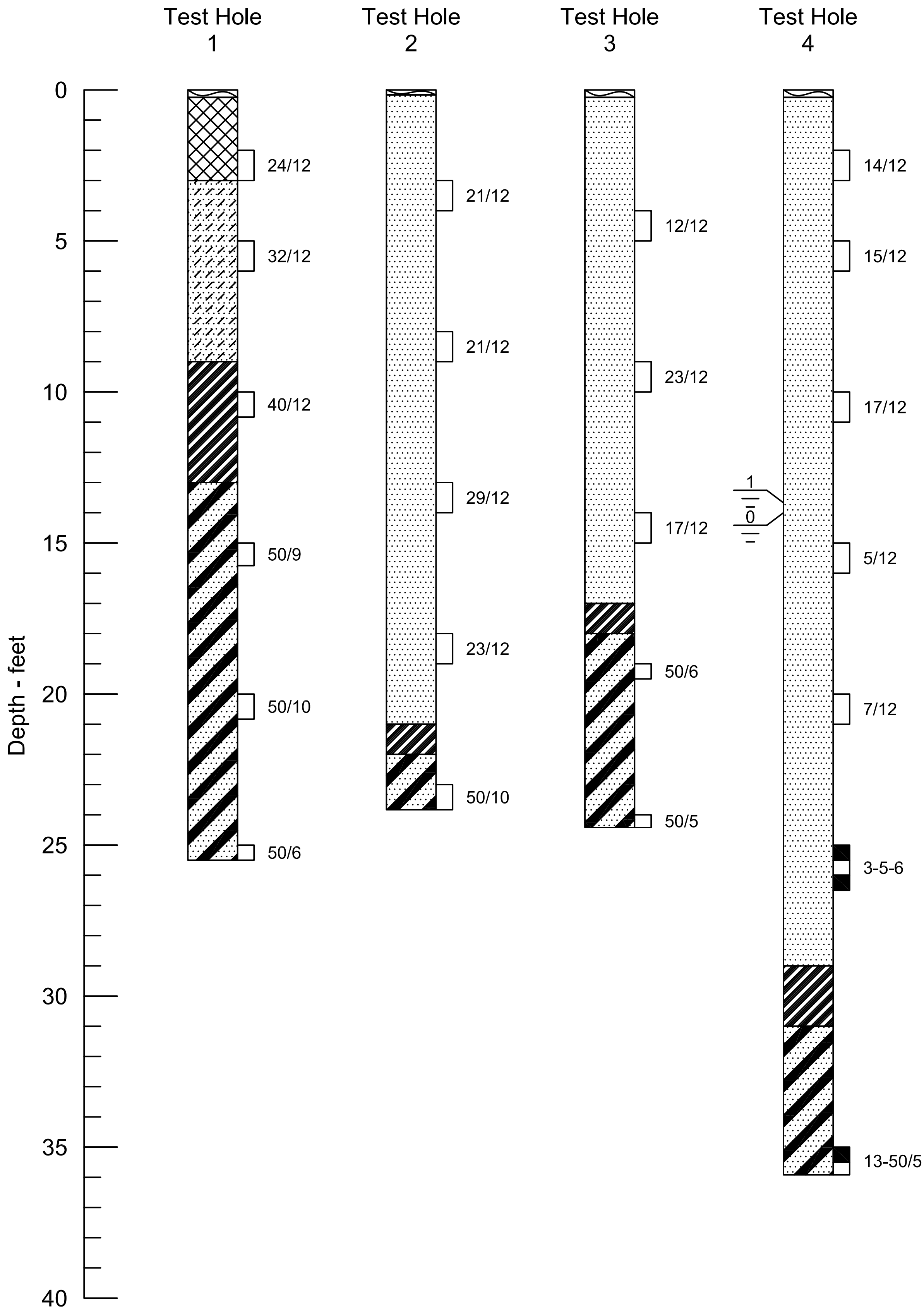
Reviewed by Brian H. Reck, P.G., C.E.G., P.E.

GOOGLE EARTH AERIAL IMAGE (10/09/2015)

1
⌖ Indicates test hole number and approximate location.

(Not to Scale)

GROUND ENGINEERING CONSULTANTS	
LOCATION OF TEST HOLES	
JOB NO.: 17-3054	FIGURE: 1
CADFILE NAME: 3054SITE.DWG	



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LOGS OF TEST HOLES

JOB NO.: 17-3054	FIGURE: 2
CADFILE NAME: 3054LOG.DWG	

LEGEND:

Topsoil

Fill: Silt, clay, and fine to medium sands and was slightly moist to moist, moderately to highly plastic, and brown in color.

Sands and Clays: Silty to clayey, fine to coarse sand and sandy clay. They were slightly to moist, moderately to highly plastic, and light brown to brown in color. Iron oxide staining and caliche were observed locally.

Sands: Silty, clayey, and clean, fine to coarse sands with gravels and silt and clay lenses locally. They were dry to wet, non- to slightly plastic, loose to medium dense, and light brown to brown. Iron oxide staining was noted locally.

Weathered Sandstone and Siltstone: Fine to coarse grained sandstone and siltstone with claystone beds locally. They were moist to very moist, moderately to highly plastic, medium hard, and gray to yellow green. Iron staining was noted locally.

Sandstone and Siltstone Bedrock: Interbedded fine to coarse grained sandstones and siltstones with local claystone beds. They were moist to wet, moderately to highly plastic, hard to very hard, and brown to yellow green to gray in color. Iron staining were encountered locally.

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 09/21/217 with 4-inch diameter continuous flight 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.

7) 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 information.

8) All test holes were immediately backfilled upon completion of drilling, unless otherwise specified in this report.

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LEGEND AND NOTES

JOB NO.: 17-3054	FIGURE: 3
CADFILE NAME: 3054LEG.DWG	

GROUND ENGINEERING

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural	Natural	Percent	Atterberg Limits		Percent	Unconfined	USCS	AASHTO	
Test Hole No.	Depth *(feet)*	Moisture Content *(%)*	Dry Density *(pcf)*	Passing No. 200 Sieve	Liquid Limit	Plasticity Index	Swell (Surcharge Pressure)	Compressive Strength *(psf)*	Classifi-cation	Classifi-cation (GI)	Soil or Bedrock Type
1	2	9.2	112.4	48	38	19	-	-	SC	A-6 (5)	FILL: Clayey Sand
1	15	22.3	97.0	60	50	16	-0.2 (1,850 psf)	-	MH	A-7-5 (9)	SILTSTONE Bedrock
1	25	21.8	102.3	70	61	26	-	23,600	MH	A-7-5 (19)	SILTSTONE Bedrock
2	8	4.3	109.7	19	NV	NP	-	-	SM	A-1-b (0)	Silty SAND
2	23	35.8	82.2	54	69	25	-0.2 (2,850 psf)	-	MH	A-7-5 (12)	SILTSTONE Bedrock
3	9	12.1	111.5	36	47	22	0.9 (1,100 psf)	-	SC	A-7-6 (3)	Clayey SAND
4	5	1.9	SD	3	23	7	-	-	SP	A-2-4 (0)	SAND
4	15	12.9	114.2	19	23	8	-	-	SC	A-2-4 (0)	Clayey SAND

SD = Sample Disturbed, NV = Non-Viscous, NP = Non-Plastic

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TABLE 2

SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		**Water Soluble Sulfates** *(%)*	**pH**	**Redox Potential** *(mV)*	**Sulfides Content**	**Resistivity** *(ohm-cm)*	**USCS Classifi-cation**	**Soil or Bedrock Type**
Test Hole No.	**Depth** *(feet)*							
1	2	<0.01	7.8	-73	Positive	12,788	SC	FILL: Clayey Sand

Job No. 17-3054