Preliminary Geotechnical Evaluation

Parker & Pine Retail
Southeast Corner of South Twenty Mile Road and Pine Lane
Parker, Colorado

EVT Parker Colorado, LLC

2710 E. Camelback Road, Suite 210 | Phoenix, Arizona 85016

April 4, 2019 | Project No. 501651001

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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1 INTRODUCTION

In accordance with your authorization and our proposal dated January 7, 2019, we have performed a preliminary geotechnical evaluation for the proposed retail development located at the southeast corner (SEC) of South Twenty Mile Road and Pine Lane in Parker, Colorado. The approximate location of the site is depicted on Figure 1.

The purpose of our study was to evaluate the subsurface conditions and to provide preliminary design and construction recommendations regarding the development of the referenced site. This report presents the findings of our subsurface exploration program, the results of our laboratory testing, conclusions regarding the subsurface conditions at the site, and our geotechnical recommendations for the design and construction of this project.

2 SCOPE OF SERVICES

The scope of our services for the project generally included:

- Review of the referenced background information, including aerial imagery, published geologic maps, in-house geotechnical data, and available topographical information pertaining to the project site and vicinity.

- Performing site reconnaissance and mark-out of the boring locations at the project site.

- Notification of Utility Notification Center of Colorado (UNCC) of the boring locations prior to drilling.

- Drilling, logging, and sampling of twelve small-diameter exploratory borings (Borings B-1 through B-12) across the project site. The borings were advanced to depths of approximately 25 to 45 feet below ground surface (bgs.) The boring locations are presented on Figure 2. Boring logs are presented in Appendix A. The ground elevations noted on the boring logs are approximate and were obtained from the preliminary grading plan provided (Kimley-Horn, 2018).

- Performance of laboratory tests on selected samples obtained from the borings to evaluate the engineering properties including in-situ moisture content and dry density, Atterberg limits, percent materials finer than the No. 200 sieve, gradation, swell/consolidation potential, and soil corrosivity characteristics (including pH, resistivity, water-soluble sulfates, and chlorides). The results of the in-situ moisture content and dry density tests are presented on the boring logs and the remainder of the test results are presented in Appendix B.

- Compilation and analysis of the data obtained.

- Preparation of this report presenting our findings, conclusions, and geotechnical recommendations regarding design and construction of the proposed project.

3 SITE DESCRIPTION AND BACKGROUND REVIEW

The site consists of an approximately 17-acre parcel located at the SEC of South Twenty Mile Road and Pine Lane in Parker, Colorado. The project site considered in this preliminary geotechnical evaluation includes Lots 1 through 6. Ninyo and Moore performed the subsurface exploration for Lot 7 on the western portion of the site in 2018. The project side is bounded by Pine Lane to the north, South Twenty Mile Road to the west, Parker Road to the east, and an existing detention pond and Baldwin Gulch to the south. The approximate location of the site is presented on Figure 1.

We reviewed several historic aerial photographs of the project site. Prior to 2002, the site was a mostly vacant lot with structures in the northwest and southeast corners of the site. By 2005, both structures had been demolished. Around 2005, the site was regraded during the installation of Pine Lane and South Twenty Mile Road. The grading included the construction of the detention pond and improvements to the Baldwin Gulch, located to the south of the project site. The site has remained in similar condition since 2005.

Existing utility lines are located in the northwest corner and south side of the lots. These lines were marked by Parker Water and Sanitation District prior to our drilling operations.

4 PROPOSED CONSTRUCTION

Based on the information provided, the development may include the design and construction of various retail buildings. We understand, based on preliminary grading plans (Kimley-Horn, 2018), site grading may consist of up to 9 feet of fill and 7 feet of cut to achieve the preliminary design grades. Ancillary construction of new roadways and utilities within the development is also anticipated. The preliminary finished grades are presented in Figure 2.

5 FIELD EXPLORATION AND LABORATORY TESTING

On March 4 and 5, 2018, Ninyo & Moore conducted subsurface exploration services at the project site to evaluate the existing subsurface conditions and to collect soil samples for visual observation and laboratory testing. The evaluation consisted of drilling, logging, and sampling of twelve exploratory borings (Borings B-1 through B-12) using a truck-mounted drill rig equipped with 4-inch continuous flight, solid-stem augers. The borings were advanced to depths of approximately 25 to 45 feet below ground surface (bgs). Borings B-1 through B-4, B-9, and B-11 were advanced in the proposed building footprints according to the preliminary grading plan

(Kimley-Horn, 2018.) Borings B-5 through B-8 were advanced in the proposed roadways and parking lots according to the preliminary grading plan. The approximate locations of the borings are presented on Figure 2. Relatively undisturbed and disturbed soil samples were collected at selected intervals.

The soil samples collected from our drilling activities were transported to the Ninyo & Moore laboratory for geotechnical laboratory analysis. Selected samples were analyzed to evaluate engineering properties including in-situ moisture content and dry density, Atterberg limits, percent material passing the No. 200 sieve, gradation, swell/consolidation potential, and soil corrosivity characteristics (pH, resistivity, water-soluble sulfates, and chlorides).

The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. A description of each laboratory test method and the remainder of the laboratory test results are presented in Appendix B.

6 GEOLOGY AND SUBSURFACE CONDITIONS

The geology and subsurface conditions at the site are described in the following sections.

6.1 Geologic Setting

The site is located in Parker, Colorado approximately 21 miles east of the Rocky Mountain Front Range, within the Colorado Piedmont section of the Great Plains Physiographic Province. The Laramide Orogeny uplifted the Rocky Mountains during the late Cretaceous and early Tertiary Periods. Subsequent erosion deposited sediments east of the Rocky Mountains, including the Dawson Formations in the area. As a result of regional uplift approximately 5 to 10 million years ago, streams such as the South Platte River down-cut and excavated into the Great Plains forming the Colorado Piedmont section (Trimble, 1980).

The surficial geology of the site vicinity is mapped by Maberry and Lindvall (1972) as Holocene age Piney Creek, Post-Piney Creek, and Broadway Alluviums. Dawson Formation bedrock is mapped underlying the project area at depth.

6.2 Subsurface Conditions

Our understanding of the subsurface conditions at the project site is based on our field exploration, laboratory testing, and review of published geologic maps, historic aerial imagery, and our experience with the general geology of the area. The following sections provide a generalized description of the subsurface materials encountered. More detailed descriptions are presented on the boring logs in Appendix A.

6.2.1 Fill

Fill was encountered at the surface in Borings B-1, B-11 and B-12, and extended to depths of approximately 2 to 5 feet bgs. Fill materials were also encountered in Boring B-8 to a depth of approximately 14.5 feet bgs. The fill materials encountered generally consisted of light brown to brown, dry to moist, lean clay with varying amounts of sand and clayey sand with varying amounts of gravel.

Based on the results of the laboratory testing, selected samples of the fill exhibited in-place moisture contents between approximately 4.0 and 15.2 percent and dry densities between approximately 107.8 and 127.2 pounds per cubic foot (pcf).

Considering the historic past-uses of the site, fill material may contain debris and deleterious materials, such as organics, although we did not encounter such materials in our borings during our subsurface exploration program.

6.2.2 Alluvial Deposits

Alluvial deposits were encountered beneath the fill in Borings B-1, B-11 and B-12, and at the surface in Borings B-2 through B-10. With the exception of Boring B-5, the alluvium extended to depths of approximately 9 to 19 feet bgs. In Boring B-5, the alluvium extended to a depth of approximately 42 feet bgs. The alluvium encountered generally consisted of reddish yellow to brown, dry to moist, loose to dense, fine to medium to fine to coarse sand with varying amounts of clay and stiff to hard, lean clay to fat clay with varying amounts of sand.

Based on the results of the laboratory testing, selected samples of the alluvium had in-place moisture contents between approximately 3.2 and 24.2 percent, and dry densities ranging between approximately 91.3 and 123.8 pcf.

6.2.3 Dawson Formation Bedrock

Dawson Formation bedrock, mapped by Maberry and Lindvall (1972), was encountered below the alluvial deposits and extended to the borings' termination depths of approximately 25 to 45 feet bgs. The Dawson Formation bedrock generally consisted of light brown, reddish brown, gray and olive, moist, moderately hard to hard, claystone and moderately to strongly cemented sandstone.

Based on the results of the laboratory testing, selected samples of the Dawson Formation bedrock had in-place moisture contents between approximately 8.3 and 29.2 percent, and dry densities between approximately 93.7 and 117.7 pcf.

6.3 Groundwater

Groundwater was encountered during drilling in Boring B-5 at a depth of approximately 29 feet bgs. Groundwater was not encountered in the remaining borings during drilling. Groundwater levels can fluctuate due to seasonal variations in precipitation, irrigation, groundwater withdrawal or injection, and other factors. The possibility of groundwater level fluctuations and perched groundwater should be considered when developing the design and construction plans for the project. In general, groundwater is not expected to be a constraint to the development of the site. However, depending on the depth of site utilities, groundwater may be encountered in deep utility excavations.

7 GEOLOGIC HAZARDS

The following sections describe potential geologic hazards at the site including faulting and seismicity, expansive soils, compressible/collapsible soils, and liquefaction potential.

7.1 Faulting and Seismicity

Historically, several minor earthquakes have been recorded around the Front Range area. Based on our field observations and our review of readily available published geological maps and literature there are no known active faults underlying or adjacent to the subject site. The faults closest to the project site include the Rocky Mountain Arsenal Fault and the Golden Fault.

The Rocky Mountain Arsenal Fault lies approximately 20 miles northeast of the site (Kirkham and Rogers, 1981). The most recent significant seismic movements associated with the Rocky Mountain Arsenal Fault occurred in the 1960s, with recorded earthquake magnitudes up to 5.5. United States Geological Survey (USGS) investigators concluded that a strong correlation existed between the seismic activity of this fault and pressure injection of liquid waste into a disposal well located at the nearby Rocky Mountain Arsenal. Pressure injection in the disposal well was discontinued in 1966 and only minor seismic movements along the fault have been recorded since. The risk of this fault giving rise to damaging, earthquake-induced ground motions at the site during the design life of the proposed structure is considered to be relatively low, based on the previously recorded low seismic magnitudes.

The Golden Fault lies approximately 26 miles northwest of the site. The fault is considered to be late Quaternary in age and has not shown displacement in Holocene time, as Pleistocene deposits overlie the fault (approximately 75 to 125 thousand years before the present [Kirkham, 1977]). Therefore, the probability of damage at the site from seismically induced ground surface rupture from this fault is considered to be low.

Design of any proposed improvements should be performed in accordance with the requirements of the governing jurisdictions and applicable building codes. Table 1 presents the preliminary seismic design parameters for the site in accordance with the 2015 International Building Code guidelines and adjusted maximum considered earthquake spectral response acceleration parameters evaluated using the web-based USGS ground motion calculator (USGS, 2018).

Table 1 – 2015 International Building Code Seismic Design Criteria

Seismic Design Factors	Value
Preliminary Site Class	D
Site Coefficient, Fa	1.6
Site Coefficient, Fv	2.4
Mapped Spectral Acceleration at 0.2-second Period, Ss	0.173 g
Mapped Spectral Acceleration at 1.0-second Period, S1	0.057 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, SMS	0.277 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, SM1	0.137 g
Design Spectral Response Acceleration at 0.2-second Period, SDS	0.184 g
Design Spectral Response Acceleration at 1.0-second Period, SD1	0.091 g

7.2 Expansive Soils

One of the more significant geologic hazards in Colorado is the presence of swelling clays in bedrock or surficial deposits. Moisture changes to bedrock or surficial deposits containing swelling clays can result in volumetric expansion and collapse of those units. Changes in soil moisture content can result from rainfall, irrigation, pipeline leakage, surface drainage, perched groundwater, drought, or other factors. Volumetric change of expansive soil may cause excessive cracking and heaving of structures with shallow foundations, concrete slabs-on-grade, or pavements supported on these materials. Construction on soils known to be potentially expansive could have a significant impact on the project.

A review of a Colorado Geological Survey map delineating areas based on their relative potential for swelling in the Front Range area by Hart (1973-1974) indicates soil and bedrock materials in the project vicinity typically exhibit low to moderate swell potential.

Based on the results of our laboratory testing, selected samples of the fill encountered on site exhibited swell percentages of less than 1 percent when inundated against a surcharge pressure of approximately 200 pounds per square foot (psf).

Selected samples of the alluvial deposits exhibited swell potentials up to approximately 2.1 when inundated under a surcharge pressure of approximately 200 psf, up to approximately 1.9 when inundated under a surcharge pressure of approximately 500 psf, and up to approximately 1.4 percent when inundated under a surcharge pressure of 1,000 psf.

Selected samples of the Dawson Formation bedrock exhibited swell potentials of up to 2.5 percent when inundated under a surcharge pressure of approximately 2,000 psf.

Based on the results of our subsurface exploration and laboratory testing, the swell potential for the overburden materials encountered are "LOW," based on the criteria in Table 2. The swell potential for the underlying Dawson Formation claystone bedrock is "LOW" to "MODERATE," based on the criteria presented in Table 2.

Table 2 – Slab Performance Risk Categories

Slab Performance Risk Category	Representative Percent Swell (500 psf Surcharge)	Representative Percent Swell (1,000 psf Surcharge)
LOW	0 to <3	0 to <2
MODERATE	3 to <5	2 to <4
HIGH	5 to <8	4 to <6
VERY HIGH	> 8	> 6

Note: Based on Colorado Association of Geotechnical Engineers, Guidelines for Slab Performance Risk Evaluation and Residential Basement Floor System Recommendations (Denver Metropolitan Area, 1996).

7.3 Compressible/Collapsible Soils

Compressible soils are generally comprised of soils that undergo consolidation when exposed to new loadings, such as fill or foundation loads. Soil collapse (or hydro-collapse) is a phenomenon where soils undergo a significant decrease in volume upon an increase in moisture content, with or without an increase in external loads. Buildings, structures, and other improvements may be subject to excessive settlement-related distress when compressible soils or collapsible soils are present.

Based on our subsurface evaluation and the results of our laboratory testing, the fill materials may compress under the loads of the proposed site grading. The alluvial deposits and the Dawson Formation bedrock encountered below the fill material have a low potential for compression or collapse.

7.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated soils lose shear strength under short-term (dynamic) loading conditions. Ground shaking of sufficient duration results in the loss of grain-to-grain contact in potentially liquefiable soils due to a rapid increase in pore water pressure, causing the soil to behave as a fluid for a short period of time.

To be potentially liquefiable, a soil is typically cohesionless with a grain-size distribution generally consisting of sand and silt. It is generally loose to medium dense and has relatively high moisture content, which is typical near or below groundwater level. The potential for liquefaction decreases with increasing clay and gravel content, but increases as the ground acceleration and duration of shaking increase. Potentially liquefiable soils need to be subjected to sufficient magnitude and duration of ground shaking for liquefaction to occur. Based on our subsurface exploration, laboratory testing, and the relatively low ground motion anticipated at the site, liquefaction is not considered a hazard at this site.

8 PRELIMINARY CONCLUSIONS

Based on the results of our subsurface evaluation, laboratory testing, and data analyses, it is our opinion that the proposed project is feasible from a geotechnical standpoint, provided the recommendations presented herein are implemented and appropriate construction practices are followed. Preliminary geotechnical design and construction considerations for the proposed project include the following:

- Fill was encountered at the surface in Borings B-1, B-11, and B-12, and extended to depths of approximately 2 to 5 feet bgs. Fill materials were also encountered in Boring B-8 to a depth of approximately 14.5 feet bgs. Due to the location of Boring B-8, it is anticipated the depth of fill is related to utilities along the adjacent Parker Road. The fill materials encountered generally consisted of light brown to brown, dry to moist, lean clay with varying amounts sand and clayey sand with varying amounts of gravel. Due to the relatively shallow thickness of the fill materials encountered, surficial compaction of these materials should be performed as a part of overall site grading.

- Our understanding of the proposed site grading is based on the preliminary site grading plans provided (Kimley-Horn, 2018). As indicated on the preliminary plans, mass earthwork for the site could involve fill thicknesses of up to approximately 9 feet and cut thicknesses of up to approximately 7 feet.

- Alluvial deposits were encountered beneath the fill in Borings B-1, B-11 and B-12, and at the surface in Borings B-2 through B-10. With the exception of Boring B-5, the alluvium extended to depths of approximately 9 to 19 feet bgs. In Boring B-5, the alluvium extended to a depth of approximately 42 feet bgs. The alluvium encountered generally consisted of reddish yellow to brown, dry to moist, loose to dense, fine to medium to fine to coarse sand with varying amounts of clay and stiff to hard, lean clay to fat clay with varying amounts of sand.

- Dawson Formation bedrock was encountered below the alluvial deposits and extended to the borings' termination depths of approximately 25 to 45 feet bgs. The Dawson Formation bedrock generally consisted of light brown, reddish brown, gray and olive, moist, moderately hard to hard, claystone and moderately to strongly cemented sandstone.

- Based on the results of our subsurface exploration and understanding of the development, it is our opinion lightly to moderately loaded retail structures could be supported on spread footings with slab-on-grade floors.

- Groundwater was encountered in Boring B-5 at a depth of approximately 29 feet bgs at the time of our subsurface exploration, but was not encountered in the remaining borings at that time. Groundwater is not anticipated to be a constraint to the development; however, groundwater levels will fluctuate due to seasonal variation from precipitation, irrigation, groundwater withdrawal or injection, and other factors.

- The on-site fill and alluvium deposits should generally be excavated with medium- to heavy-duty earthmoving or excavation equipment in good operating condition. Dawson Formation bedrock, if encountered, will be difficult to excavate, as it may contain lenses and layers of cemented sandstone. Use of heavy-duty excavation equipment in good operating condition supplemented, as needed, by heavy-duty excavation equipment with single-shank ripper attachments may be needed to excavate the Dawson Formation bedrock.

- Site soils generated from on-site excavation activities consisting of fill and alluvium deposits that are free of deleterious materials, and do not contain particles larger than 3 inches in diameter, can generally be used as engineered fill during site grading. Excavated Dawson Formation claystone bedrock should not be used as engineered fill.

- Based on our laboratory data, the sulfate content of the tested soils presents a negligible risk of sulfate attack to concrete. We recommend the use of Type I/II cement for construction of concrete structures at this site.

- Based on our laboratory data, the subgrade soils at the site are highly corrosive to ferrous metals. Therefore, special consideration should be given to the use of heavy gauge, corrosion-protected, underground steel pipe or culverts, if any are planned. As an alternative, plastic pipe or reinforced concrete pipe could be considered. A corrosion specialist should be consulted for further recommendations.

- No known or reported active faults are reported underlying, or adjacent to, the site. Based on the low ground motion hazard, the likelihood or potential for liquefaction is considered to be negligible and therefore not a design consideration.

9 PRELIMINARY RECOMMENDATIONS

The following sections present our preliminary geotechnical recommendations for the development. The recommendations made in this report are preliminary and final geotechnical engineering studies with additional borings, laboratory testing, and engineering analysis will be required once building locations and loads are known.

9.1 Earthwork

The following sections provide our earthwork recommendations for this project. In general, the Town of Parker and/or project specific earthwork specifications are expected to apply, unless noted.

9.1.1 Excavations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our subsurface exploration, our site observations, and our experience with similar materials. The on-site surface and near-surface soils (fill and alluvium deposits) may generally be excavated with moderate- to heavy-duty earthmoving or excavation equipment in good operating condition.

Dawson Formation bedrock, where encountered, will be difficult to excavate as it may contain lenses and layers of cemented sandstone. Use of heavy-duty excavation equipment in good operating condition supplemented, as needed, by heavy-duty excavation equipment with single-shank ripper attachments may be needed to excavate the Dawson Formation. Increased wear and tear on the equipment used could occur while excavating Dawson Formation bedrock.

Equipment and procedures that do not cause significant disturbance to the excavation bottoms should be used. Excavators and backhoes with buckets having large claws to loosen the soil should be avoided when excavating the bottom approximately 6 to 12 inches of excavations as such equipment may disturb the excavation bases.

Subgrade conditions should be observed by Ninyo & Moore during construction. Where unstable subgrade conditions develop, stabilization measures will need to be employed. Stabilization methods should be provided by the grading contractor, as needed, and may include the use of geogrids, geotextiles, pushing oversized rock into the subgrade, and chemical stabilization. The subgrade stabilization methods proposed should be discussed with the geotechnical engineer prior to implementation. The stabilization method selected should consider the effects of such stabilization on future utility installation and/or maintenance work.

Groundwater was encountered at the time of our subsurface exploration in Boring B-5 at a depth of approximately 29 feet bgs, but was not encountered in the remaining borings at that time. In general, groundwater is not expected to be a constraint during construction. However, groundwater levels will fluctuate due to seasonal variations from precipitation, irrigation, groundwater withdrawal or injection, and other factors.

9.1.2 Temporary Excavations and Shoring

Temporary excavations will be needed for this project to construct utilities. Based on the information obtained from our subsurface exploration and our experience with similar projects, we anticipate that the soil conditions and stability of the excavation sidewalls may vary with depth. Soils with higher fines content may stand vertically for a short time (less than 12 hours) with little sloughing. However, as the soil dries after excavation or as the excavations are exposed to rainfall, sloughing may occur. Soils with low cohesion (e.g., predominately sandy or gravelly material), may slough or cave during excavation, especially if wet or saturated.

The contractor should provide safely sloped excavations or an adequately constructed and braced shoring system, in compliance with Occupational Safety and Health Administration regulations (OSHA, 2005), for employees working in excavations that may expose them to the danger of moving ground. Reducing the inclination of the sidewalls of the excavations, where feasible, may increase the stability of the excavations. If construction material is stored, or equipment is operated near an excavation, flatter slope geometry or shoring should be used during construction.

In our opinion, the fill and alluvial deposits should generally be considered a Type C soil when applying OSHA regulations. For these soil conditions, OSHA recommends a temporary slope inclination of 1.5H:1V or flatter for excavations 20 feet or less in depth. Steeper cut slopes may be utilized for excavations less than 4 feet deep depending on the strength, moisture content, and homogeneity of the soils as observed in the field.

Excavations into the Dawson Formation, if anticipated, should generally be considered Type A when applying OSHA regulations. The Dawson Formation bedrock encountered did include varying levels of cementation in the sandstone. As such, flattening of the temporary excavations may be needed when excavating into the formational deposits.

Appropriate slope inclinations should be evaluated in the field by an OSHA-qualified “Competent Person” based on the conditions encountered.

9.1.3 Remedial Grading

Based on our understanding of mass earthwork to be performed (Kimley-Horn, 2018), site grading may consist of the placement of up to approximately 9 feet of fill and performance of cut thicknesses of up to approximately 7 feet.

On a preliminary basis, where shallow foundation systems are utilized, spread-footings should bear on a relatively uniform thickness of moisture conditioned and compacted engineered fill of approximately 12 or more inches below the bottom of the footings.

The buildings may be provided with slab-on-grade floors bearing 12 or more inches of moisture-conditioned and compacted engineered fill.

Private asphalt and concrete pavements and exterior flatwork may be placed on 12 or more inches of moisture conditioned and compacted engineered fill.

Remediation of subgrades for any public roadways planned within the development should follow Town of Parker standards. Stabilization with fly ash, lime, or cement to a depth of 12 or more inches may be required. The potential for stabilization of public roadways will depend on finish grades within the public pavement areas.

Ninyo & Moore should be retained to perform observations and construction testing during overlot grading/site excavations. Additional recommendations specific to the site conditions encountered may be provided at the time of construction. The project budget should include additional cost associated with the removal and replacement of unforeseen undocumented fill material.

Care should be taken to maintain the subgrade moisture content after fill placement but prior to construction of grade supported slabs and pavements. The site should be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become desiccated, saturated, frozen, or disturbed, the affected material should be removed or these materials should be scarified, moisture conditioned, and recompacted prior to slab and pavement construction.

The exposed subgrade materials should be firm and unyielding prior to fill placement. The extent of and depths of removal should be evaluated by our representative during the excavation work based on observation of the soils exposed. Subgrade materials that are disturbed during grading should be moisture conditioned and re-compacted according to the recommendations provided in this report.

The geotechnical consultant should be retained to observe the remedial excavations, and the elevations of the excavation bottoms should be surveyed by the project civil engineer.

9.1.4 Re-Use of Site Soils

Soils generated from on-site excavation activities consisting of fill and alluvium that are free of deleterious materials and organic matter, and do not contain particles larger than 3 inches in diameter, can generally be used as engineered fill as evaluated by the geotechnical consultant, provided they are compacted and moisture conditioned as recommended in this report.

Dawson Formation claystone should not be used as engineered fill. However, excavated sandstone may be utilized provided it is processed prior to placement. Processing of the excavated materials with disks will be needed to breakdown the bedrock fragments to create a uniform engineered fill. Such processing may result in additional handling of the excavated material prior to placement, and cost associated with this process should be included in the cost of overlot grading for the site.

Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) larger than 3 inches in diameter may be incorporated into the project fills in non-structural areas and below the anticipated utility installation depths. A Geotechnical Engineer should be consulted regarding appropriate recommendations for the usage of such materials on a case-by-case basis when such materials have been observed during earthwork. Care should be taken to avoid nesting of oversized materials during placement.

9.1.5 Fill Placement and Compaction

Granular soils (on-site soils that classify as SC, SW-SC, or import soils) used as engineered fill should be moisture-conditioned to within 2 percent of optimum moisture content. Fine-grained soils (on-site soils that classify as CL or CH) used as engineered fill should be moisture-conditioned to between 1 percent below and 3 percent over optimum moisture content. Engineered and select fills should be compacted to a relative compaction of 95 percent or more as evaluated by ASTM D698.

Fills should be compacted by appropriate mechanical methods. Lift thickness for fill will be dependent upon the type of compaction equipment utilized. Backfill should be placed in lifts not exceeding 8 inches in loose thickness in areas compacted by other-than hand operated machines. Backfill should be placed in lifts not exceeding 6 inches in loose thickness in areas compacted by hand operated machines.

Fill materials should not be placed, worked, or rolled while they are frozen, thawing, or during poor/inclement weather conditions.

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 CLSM should be considered in lieu of compacted fill for areas with low tolerances for surface settlements, for excavations that extend below the groundwater table and in areas with difficult access for compaction equipment. CLSM should be placed in lifts of 5 feet or less with a 24-hour or more curing period between each lift.

9.1.6 Imported Soil

Import soil for use as engineered fill should have between approximately 15 to 40 percent passing the No. 200 sieve. Additionally, import soils should exhibit low swell potential (approximately 1 percent or less when wetted against a surcharge pressure of 500 psf), and a low plasticity index (less than 20). Imported soil should not contain organic matter, clay lumps, bedrock (claystone, sandstone, etc.) fragments, debris, other deleterious matter, or rocks or hard chunks larger than approximately 3 inches' nominal diameter.

Import soil in contact with ferrous metals should have low corrosion potential. Import material in contact with concrete should have a soluble sulfate content of less than 0.1 percent.

We further recommend that proposed import soils be evaluated by the project's geotechnical consultant at the borrow source for its suitability prior to importation to the project site. Import soil should be moisture-conditioned and placed and compacted in accordance with the recommendations set forth in Section 9.1.5.

9.1.7 Utility Installation

The contractor should take care to achieve and maintain adequate compaction of the backfill soils around manholes, valve risers and other vertical pipeline elements where settlements commonly are observed. Use of "flowable fill," (e.g. CLSM, or a similar material) should be considered in lieu of compacted soil backfill for areas with low tolerances for surface settlements. This would also reduce the permeability of the utility trenches.

Pipe bedding materials placement and compaction should meet the specifications of the pipe manufacturer and applicable municipal standards. Materials proposed for use as pipe bedding should be tested for suitability prior to use.

Special care should be exercised to avoid damaging the pipe or other structures during the compaction of the backfill. In addition, the underside (or haunches) of the buried pipe should be supported on bedding material that is compacted as described above. This may need to be performed with placement by hand or small-scale compaction equipment.

Surface drainage should direct water away from utility trench alignments. Where topography, site constraints or other factors limit or preclude adequate surface drainage, the granular bedding materials should be surrounded by non-woven filter fabric (e.g., Mirafi® 140N or the equivalent) to reduce migration of fines into the bedding which can result in severe, isolated settlements.

Development of site grading plans should consider the subsurface transfer of water in sanitary sewer trenches and the pipe bedding. Sandy pipe bedding materials can function as efficient conduits for re-distribution of natural and applied waters in the subsurface. Cut-off walls in sanitary sewer trenches should be implemented to reduce the rates and volumes of water transmitted along the sanitary sewer alignment and toward buildings, pavements and other structures where excessive wetting of the underlying soils may be damaging.

9.2 Anticipated Foundations

Foundations for the structures within the development will depend on structural loads. However, for the purposes of this report and based on our understanding of the development, we anticipate construction will consist of primarily lightly- to moderately-loaded retails structures. We anticipate these types of structures may be supported on conventional spread footing foundations. The design recommendations made in this report are preliminary and final geotechnical studies with additional borings, laboratory testing, and/or engineering analysis will be required when information pertaining to the individual structures is known.

Perimeter footings should extend to 36 inches or more below the lowest exterior finished grade (for frost protection), and bear on a zone of adequately placed and compacted engineered fill as described in Section 9.1.3 of this report. Continuous wall footings should have a width of 18 inches or more and column footings should have a width of 24 inches or more.

Footings for lightly- to moderately-loaded structures may be designed using a net allowable soil bearing pressure of 3,000 psf for static conditions. The allowable end bearing may be increased by one-third when considering loads of short duration such as wind or seismic forces.

Uplift resistance can be developed from the weight of the footings, the effective weight of any overlying soil, and the weight of the supported structure itself. The effective unit weight of the soil can be assumed to be 125 pcf. Soil uplift resistance may be calculated as the weight of the soil prism defined by a diagonal line extending from the perimeter of the foundation to the ground surface at an angle θ of 20 degrees from the vertical. Under large moment and/or shear loading, the effective size of the uplift soil prism may be reduced. An appropriate safety factor should be applied.

The foundations should be proportioned such that the resultant force from design loads, including lateral loads, falls within the kern (i.e., middle one-third of the footing base).

The bottom surface of foundation excavations should be compacted with hand-held dynamic compaction equipment (i.e., jumping jack, flat-plate vibrator) prior to placement of forms and reinforcing steel. The base of foundation excavations should be free of ice, snow, water, frost, and loose soil prior to placing concrete. Concrete should be placed soon after subgrade compaction to reduce bearing soil disturbance. Should the soils at bearing level become excessively dry, disturbed, or saturated, the affected soil should be moisture conditioned and re-compacted. Ninyo & Moore should be retained to observe, test, and evaluate the soil foundation bearing materials. An "open hole" site visit letter will not be issued unless Ninyo & Moore representatives are retained to be on-site during footing excavation and subsequent engineered fill placement.

Based on the results of our subsurface exploration and laboratory testing, we estimate total and differential post-construction vertical movements of approximately 1-inch and ½-inch, respectively, may occur. Angular distortions of approximately 1-inch (vertical) over 40 feet (horizontal) are possible.

9.3 Slab-On-Grade Floors

Based on the preliminary grading information provided, it is anticipated that slab-on-grade floors may be utilized for the various structures within the development. The recommendations provided Section 9.1.3 should be followed during remedial grading associated with the preparation of the engineered fill below the building slab-on-grade floors.

The design of the floor slabs (including jointing and reinforcement) is the responsibility of the structural engineer. Joints should be constructed at intervals designed by the Structural Engineer to help reduce the random cracking of the slab. Floor slabs should be adequately reinforced. Recommendations based on structural considerations for slab thickness, jointing, and steel reinforcement should be developed by the Structural Engineer in accordance with the American Concrete Institute (ACI) recommendations. The proper placement of the reinforcement in the slab is vital for satisfactory performance.

The floor slabs should be constructed so that it "floats" independent of the foundations. Floor slabs should be separated from bearing walls and columns with expansion joints, which allow unrestrained vertical movement. Joints should be observed periodically, particularly during the first several years after construction. Slab movement can cause previously free-slipping joints to bind. Measures should be taken so that slab isolation is maintained in order to reduce the likelihood of damage to walls and other interior improvements. Utility lines entering the slab should be provided with positive bond breaks that allow 2 or more inches of differential movement.

Interior partitions resting on floor slabs should be provided with slip joints, so that if the slabs move, the movement cannot be transmitted to the upper structure, including wallboards and door frames. A slip joint that allows 2 or more inches of vertical movement is recommended for placement at the bottoms of the interior partitions. If slip joints are placed at the tops of walls, in the event that the floor slabs move, it is expected that the wall will show signs of distress, especially where the floors meet the exterior wall. Plumbing lines should be carefully tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided. Flexible connections allowing 2 or more inches of vertical movement should be provided for slab-bearing mechanical equipment.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates that any differential movement between the walls and slabs will probably be observed in adjacent slab expansion joints or floor slab cracks that occur beyond the length of the structural dowels. The Structural Engineer should account for this potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means.

The slab should be underlain by four or more inches of moist, clean sand and/or gravel. The need for a moisture-retarding system should be considered by the structural engineer or architect based on the moisture sensitivity of the anticipated flooring.

9.4 Earth Pressures and Retaining Walls

Earth pressures are used to compute the lateral forces acting on retaining walls. These pressures can be classified as at-rest, active, and passive. The direction and magnitude of the soil/wall movement just before failure affects the resulting pressure condition. At-rest conditions exist when there is no movement, such as for a restrained wall. Active stresses are exerted when the wall moves out and the soil moves toward the wall away from the soil mass, thereby mobilizing the shear strength of the soil. Passive stresses exist when the wall moves toward the soil mass.

The recommended equivalent fluid pressures in Table 3 assume the on-site materials with an angle of internal friction (ϕ) of 26 degrees, a unit weight of 125 pcf. The values listed below are for static conditions.

Table 3 – Lateral Earth Pressures			
Soil Condition	**Active Pressure (pcf)**	**At-Rest Pressure (pcf)**	**Passive Pressure (pcf)**
On-Site Materials	49	70	320

As indicated, lateral soil resistance developed against lateral structural movement may be obtained using a passive pressure of 330 psf per foot of depth for a level ground condition up to a value of 3,300 psf per foot. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 24 inches of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance. The passive value may be increased by one-third when considering loads of short duration, including wind and seismic loads. Further, for sliding resistance, an ultimate friction coefficient of 0.35 may be used between concrete and foundation soil. Retaining walls can be supported on spread footings following design recommendations presented in this report.

Measures should be taken so that moisture does not build up behind below-grade walls. Depending of grades and the location of retaining walls, it may be recommended to include perforated drains behind retaining walls. Additional recommendations could be provided once the location of retaining walls is known.

Below-grade walls should be damp proofed or waterproofed in accordance with the recommendations of the project architect and structural engineer. In addition, design considerations should be given to prevention of efflorescent development on the basement wall concrete. Additional recommendations and details can be provided upon request.

9.5 Pavement Design

The following sections are provided for privately maintained roadways and parking lots. Additional information relating to the grading and classification of public roadways will be required prior to providing a Pavement Design Report for public roadways. Pavement section alternatives are included herein for the paved surfaces, which include standard duty automobile parking areas and driveways, and heavy-duty drive lanes and fire lanes.

The pavement sections recommended below were developed in general accordance with the guidelines and procedures of the American Association of State Highway and Transportation

Officials (AASHTO) (AASHTO, 1993), the Colorado Department of Transportation (CDOT), and the Town of Parker Roadway Design and Construction Criteria Manual (Town of Parker, 2018).

9.5.1 Pavement Subgrade Support

The current subgrade soils encountered in our borings typically consisted of clayey sand, lean, and fat clay that classify as A-2-4 to A-7 in accordance with the AASHTO classification system. It is anticipated that fill imported to the site will classify as A-4 or better.

An R-Value of 10 was assumed for the pavement subgrade soils for the project. For purposes of new construction, it is assumed that soils placed within 2 feet of the finished pavement subgrade will exhibit an average R-Value of 10 or more. If during construction, the subgrade is found to vary from the expected soil conditions, we should be contacted so we may re-evaluate our recommended resilient modulus value.

9.5.2 Design Traffic and Pavement Design

Specific traffic loadings and intended use volumes for the project were not available at the time this report was prepared. Based on our current understanding of the project, we utilized an equivalent 18-kip single axle load value (ESAL) of 36,500 for standard duty parking areas and 219,000 for heavy-duty drive lanes for a 20-year design life. If design traffic loadings differ significantly from this assumed value, we should be notified to re-evaluate the pavement recommendations below.

The design of flexible pavements was based on the following input parameters:

Initial Serviceability:	4.5
Terminal Serviceability:	2.5
Reliability	80%
Overall Standard Deviation:	0.44
Resilient Modulus:	3,563 psi (R-Value of 10)
Stage Construction	1.0

The design of rigid pavements was based on the following input parameters:

Initial Serviceability:	4.5
Terminal Serviceability:	2.5

28-Day Mean PCC Modulus Rupture:	650 psi
28-Day Mean Elastic Modulus of Slab:	3.6 x 106 psi
Mean Effective k value:	175 psi/in
Reliability:	80%
Overall Standard Deviation:	0.35
Load Transfer Coefficient:	3.9
Overall Drainage Coefficient:	1.0

9.5.3 Pavement Section Recommendations

Based on the above-mentioned design traffic and input parameters, and following the AASHTO method of pavement design, the following structural sections were calculated for standard duty pavements and heavy duty pavements. Table 4 provides our recommended pavement section thicknesses for pavements supported on 12 or more inches of moisture conditioned and compacted engineered fill (recompacted in-situ deposits or engineered fill placed during site grading).

Table 4 – Recommended Pavement Thickness

Traffic Type	Full Depth AC (inches)	Composite AC / ABC (inches)	PCCP / ABC (inches)
Standard-Duty Areas	6.0	4.0 / 6.0	5.0
Heavy-Duty Areas	8.0	6.0 / 6.0	6.0

Notes: AC = Asphalt Concrete, ABC = Aggregate Base Course, PCCP = Portland Cement Concrete Pavement

We recommend PCCP be utilized in dumpster pads, loading areas, or other areas where extensive wheel maneuvering are expected. The dumpster pads should be large enough to support the wheels of the truck which will bear the load of the dumpster. Consideration should be given to the placement of 4 or more inches of aggregate base course (ABC) below PCCP. Although the use of ABC is not integral for structural support in PCCP pavements, their use will develop a more stable subgrade for concrete truck traffic associated with the pavement construction and help reduce potential slab curl, shrinkage cracking, and subgrade “pumping” through joints. Adequate joint spacing is recommended to prevent loss of load transfer across saw-cut crack control joints. Joints should be sealed to reduce water infiltration.

Where practical, we recommend "early-entry" cutting of crack-control joints in PCCP. Cutting of PCCP in its 'green" state typically reduces the potential for micro-cracking of the pavements prior to the crack-control joints being formed, compared to cutting the joints after the concrete has fully set. Micro-cracking of pavements may lead to crack formation in locations other than the sawed joints, and/or reduction of fatigue life of the pavement.

Adequate surface drainage should be provided to reduce ponding and infiltration of water into the pavement and subgrade materials. We suggest that the paved areas have a surface gradient of 2 percent or more. In addition, surface runoff from surrounding areas should be intercepted, collected, and not permitted to flow onto the pavement or infiltrate the subgrade. We recommend that perimeter swales, edge drains, curbs and gutters, or a combination of these drainage devices, be constructed to reduce the adverse effects of surface water runoff.

9.5.4 Pavement Materials

The AC pavement shall consist of a bituminous plant mix composed of a mixture of high-quality aggregate and bituminous material, which meets the requirements of a job-mix formula established by a qualified engineer. The asphalt material used should be based on a SuperPave Gyratory Design Revolution (NDES) of 75. Lower lifts should be constructed using an asphalt mix Grading S and asphalt cement binder grade PG 58-28. The top lift should be constructed using an asphalt mix Grading SX and asphalt cement binder grade PG 64-22 (the top lift performance in terms of temperature-related shrinkage and expansion could be improved if a modified asphalt cement binder grade PG 64-28 is utilized). Pavement layer thickness should be between 2 and 4 inches for the lower lift(s) and 2 inches for the top lift. The Geotechnical Engineer should be retained to review the proposed pavement mix designs, grading, and lift thicknesses prior to construction.

PCCP should consist of a plant mix composed of a mixture of aggregate, Portland cement and appropriate admixtures meeting the requirements of the Town of Parker.

Thickened edges should be used along outside edges of PCCP. Edges should thicken by 2 or more inches and taper to the recommended PCCP thickness 36 inches inward from the edge. Integral curbs may be used in lieu of thickened edges.

PCCP should have longitudinal and transverse joints that meet the applicable requirements of the Town of Parker.

The ABC material placed beneath pavements should meet the criteria of CDOT Class 6 aggregate base. Requirements for CDOT Class 6 aggregate base can be found in Section 703 of the current CDOT Standards and Specifications for Road and Bridge Construction.

9.5.5 Pavement Subgrade Preparation

For both the PCCP and AC pavement sections recommended above, we recommend the underlying subgrade soils be prepared as described in Section 9.1.3 of this report.

The contractor should be prepared either to dry the subgrade materials or moisten them, as needed, prior to compaction. Some site soils may pump or deflect during compaction if moisture levels are not carefully monitored. The contractor should be prepared to process and compact such soils to establish a stable platform for paving, including the use of geotextiles, where needed.

The prepared subgrade should be protected from the elements prior to pavement placement. Subgrades that are exposed to the elements may need additional moisture conditioning and compaction, prior to pavement placements.

Immediately prior to paving, the subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle and checked for moisture. Areas that show excessive deflection during proof rolling should be excavated and replaced and/or stabilized. Areas allowed to pond prior to paving may need to be re-worked prior to proof rolling.

9.5.6 Pavement Maintenance

The collection and diversion of surface drainage away from paved areas are vital to the satisfactory performance of the pavements. The subsurface and surface drainage systems should be carefully designed to facilitate removal of the water from paved areas and subgrade soils. Allowing surface waters to pond on pavements will cause premature pavement deterioration. Where topography, site constraints or other factors limit or preclude adequate surface drainage, pavements should be provided with edge drains to reduce the loss of subgrade support. The long-term performance of the pavement also can be improved greatly by backfilling and compaction behind curbs, gutters, and sidewalks so that ponding is not permitted and water infiltration is reduced.

The standard care of practice in pavement design describes the recommended flexible pavement section as a “20-year” design pavement; however, many pavements will not remain in satisfactory condition without routine, preventive maintenance and rehabilitation procedures performed during the life of the pavement. Preventive pavement treatments are surface rehabilitation and operations applied to improve or extend the functional life of the pavement. These treatments preserve, rather than improve, the structural capacity of the pavement structure. In the event the existing pavement is not structurally sound, the preventive maintenance will have no long-lasting effect. Therefore, routine maintenance programs to seal joints and cracks, and repair distressed areas are recommended.

9.6 Concrete Flatwork

Ground-supported flatwork, such as walkways, will be subject to soil-related movements resulting from heave/settlement, frost, etc. Thus, where these types of elements abut rigid building foundations or isolated/suspended structures, differential movements should be anticipated. We recommend that flexible joints be provided where such elements abut the main structure to allow for differential movement at these locations.

We recommend that exterior concrete flatwork be supported on improved subgrade as described in Section 9.1.3 of this report. Positive drainage should be established and maintained adjacent to flatwork. Water should not be allowed to pond or adjacent to flatwork.

Exterior walkways and flatwork should be 4 or more inches thick. The slab edges should be deepened by two or more inches where exterior slabs-on-grade are placed adjacent to landscaping areas and taper to the recommended thickness 12 inches inward from the edge. Formation of shrinkage cracks in concrete slabs, and other cracks due to minor soil movement, may be further reduced by utilizing steel reinforcement, such as welded wire mesh.

In no case should exterior flatwork extend 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.

The ground-supported flatwork should be provided with crack-control and expansion joints in accordance with the Town of Parker standards and specifications.

9.7 Corrosion Considerations

The corrosion potential of on-site soils to concrete and buried metal was evaluated in the laboratory using representative samples obtained from the exploratory borings. Laboratory testing was performed to assess the effects of sulfate on concrete and the effects of soil resistivity on buried metal. Results of these tests are presented in Appendix B. Recommendations regarding concrete to be utilized in the construction of proposed improvements and for buried metal pipes are provided in the following sections.

9.7.1 Concrete

The test for water-soluble sulfate content of the soils was performed using CDOT Test Method CP-L 2104. The laboratory test results are presented in Appendix B. Based on Table 601-2 of the CDOT 2011 Standard Specifications for Road and Bridge Construction, the on-site soils represent a Class 0 severity of sulfate exposure to concrete on a scale that ranges between Class 0 and Class 3. We recommend that the concrete used for this project should have a maximum water to cementitious material ratio of 0.45 and the cementitious materials should meet one of the below-outlined requirements.

- ASTM C 150 Type I, II or V
- ASTM C 595 Type IP, IP(MS) or IP(HS)
- ASTM C 1157 Type GU, MS or HS
- ASTM C 150 Type III cement if it is allowed, as in Class E concrete

The Structural Engineer should ultimately select the concrete design strength based on the project-specific loading conditions. However, higher strength concrete may be selected for increased durability, resistance to slab curling and shrinkage cracking. We recommend the use of concrete with a design 28-day compressive strength of 4,000 psi or more, for concrete slabs at this site. Concrete exposed to the elements should be air-entrained.

9.7.2 Buried Metal Pipes

The corrosion potential of on-site materials was analyzed to evaluate potential effects on foundations and structures. Corrosion potential was evaluated using the results of laboratory testing of samples obtained during the subsurface evaluation that were considered representative of soils at the subject site.

The results of the laboratory testing indicate the on-site materials have low resistivity and could be corrosive to ferrous metals. Therefore, special consideration should be given to the use of heavy gauge, corrosion-protected, underground steel pipe or culverts, if any are planned. As an alternative, plastic pipe or reinforced concrete pipe could be considered. A corrosion specialist should be consulted for further recommendations.

9.8 Scaling

Climatic conditions in the project area including relatively low humidity, large temperature changes, and repeated freeze-thaw cycles, may cause surficial scaling and spalling of exterior concrete. The occurrence of surficial scaling and spalling can be aggravated by poor workmanship during construction, such as “over-finishing” concrete surfaces and the use of de-icing salts on exterior concrete flatwork, particularly during the first winter after construction. The use of de-icing salts on nearby roadways, which can be transferred by vehicle traffic onto newly placed concrete, can be sufficient to induce scaling.

The measures below can be beneficial for reducing the concrete scaling. However, because of the other factors involved, including workmanship, surface damage to concrete can develop with the measures provided below being followed. The mix design criteria should be coordinated with other project requirements including the criteria for soluble sulfate resistance presented in Section 9.7.1.

- Curing concrete in accordance with applicable codes and guidelines.
- Maintaining a water/cement ratio of 0.45 by weight for exterior concrete mixes.
- Including Type F fly ash in exterior concrete mixes as 20 percent of the cementitious material.
- Specifying a 28-day, compressive strength of 4,500 or more psi for exterior concrete that may be exposed to de-icing salts.
- Avoiding the use of de-icing salts through the first winter after construction.
- If colored concrete is being proposed for exterior use at this site, Ninyo & Moore should be contacted for expansional consultation. Implementation of special design, construction, and maintenance precautions will be needed in these areas.

9.9 Frost Heave

Site soils are susceptible to frost heave 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 2 or more inches of heave of pavements, flatwork and other hardscaping in sustained cold weather. A portion of this movement may be recovered when the soils thaw, but due to loss of soil density, some degree of displacement will remain. Frost heave of hardscaping could also result in areas where the subgrade soils were placed on engineered fill.

In areas where hardscape movements are a design concern (i.e. exterior flatwork located adjacent to the building within the doorway swing zone), replacement of the subgrade soils with 3 or more feet of clean, coarse sand or gravel, or supporting the element on foundations similar to the building, or spanning over a void should be considered. Detailed recommendations in this regard can be provided upon request.

9.10 Construction in Cold or Wet Weather

During construction, the site should be graded such that surface water can drain readily away from the building area. Given the soil conditions, it is important to avoid ponding of water in or near excavations. Water that accumulates in excavations should be promptly pumped out or otherwise removed and these areas should be allowed to dry out before resuming construction. Berms, ditches, and similar means should be used to decrease stormwater entering the work area and to effectively convey it off-site.

Earthwork activities undertaken during the cold weather season may be difficult and should be done by an experienced contractor. Fill should not be placed on top of frozen soils. The frozen soils should be removed prior to the placement of fill or other construction material. Frozen soil should not be used as engineered fill or backfill. The frozen soil may be reused (provided it meets the selection criteria) once it has thawed completely. In addition, compaction of the soils may be more difficult due to the viscosity change in water at lower temperatures. In addition, materials with high moisture contents may not be suitable for use as compacted fill if weather conditions are not conducive to lowering the moisture content to recommended levels.

If construction proceeds during cold weather, foundations, slabs, or other concrete elements should not be placed on frozen subgrade soil. Frozen soil should either be removed from beneath concrete elements, or thawed and recompacted. To limit the potential for soil freezing, the time passing between excavation and construction should be minimized. Blankets, straw, soil cover, or heating may be used to discourage the soil from freezing.

9.11 Site Drainage

Infiltration of water into subsurface soils can lead to soil movement and associated distress, and chemically and physically related deterioration of concrete and masonry structures. It is imperative that the site drainage recommendations provided below are reviewed by project design and construction team and implemented as part of the overall site drainage. The property owner and/or property management firm should also understand that these drainage measures, once implemented by the project design and construction team, needs to be periodically maintained throughout the life of the structure. To reduce the potential for infiltration of moisture into subsurface soils at the site, we recommend the following:

- Positive drainage should be established and maintained away from the proposed building. Positive drainage may be established by providing a surface gradient for paved areas of 2 to 5 percent or more for a distance of 10 feet or more away from structures. For unpaved areas, positive drainage may be established by a slope of 5 to 10 percent for 10 feet or more away from structures, where possible.

- Adequate surface drainage should be provided to channel surface water away from on-site structures and off paved surfaces to a suitable outlet such as a storm drain. Adequate surface drainage may be enhanced by utilization of graded swales, area drains, and other drainage devices. Surface run-off should not be allowed to pond near structures or structure foundations.

- Building roof drains should have downspouts tightlined to an appropriate outlet, such as a storm drain or the street, away from structures, pavements, and flatwork. If tightlining of the downspouts is not practicable, they should discharge 5 feet or more away from structures and onto surfaces that slope away from the structure. Downspouts should not be allowed to discharge onto the ground surface adjacent to building foundations.

- The possibility of moisture infiltration beneath a structure, in the event of plumbing leaks, should be considered in the design and construction of underground water and sewer conduits. Permitting increases in moisture to the building supporting soils may result in a decrease in bearing capacity and an increase in settlement, heave, and/or differential movement. Incorporating a perimeter drainage system around the building foundations that will aid in the reduction of the moisture infiltration of subsurface soils may be considered.

- Area drains for landscaped and paved areas are recommended. Nearby landscaping should consist of drought tolerant plants, and landscape irrigation should be kept to a level just sufficient to maintain plant vigor. Overwatering should not be permitted.

- Irrigated plants should not be placed within 5 feet of the building. Low-water use (drip irrigated) landscaping should be utilized on site, particularly between 5 and 10 feet of the building exterior and within 5 feet of the exterior site improvements, including areas of concrete flatwork and masonry block walls. Irrigation heads should be oriented so that they spray away from building and block wall surfaces.

- Utility trenches should be backfilled with compacted, low permeability fill (i.e. permeability of 10^{-5} cm/s or less) within 5 feet of the building. Planters, if any, should be maintained 5 feet or more from the building and constructed with closed bottoms or with drainage systems to drain excess irrigation away from the building.

9.12 Construction Observation and Testing

A qualified geotechnical consultant should perform appropriate observation and testing services during grading and construction operations. These services should include observation of any soft, loose, or otherwise unsuitable soils, evaluation of subgrade conditions where soil removals are performed, evaluation of the suitability of proposed borrow materials for use as fill, evaluation of the stability of open temporary excavations, evaluation of the results of any subgrade stabilization or dewatering activities, and performance of observation and testing services during placement and compaction of engineered fill and structural fill soils.

The geotechnical consultant should also perform observation and testing services during placement of concrete, mortar, grout, asphalt concrete, and steel reinforcement. If another geotechnical consultant is selected to perform foundation inspection, and construction observation and materials testing services for the project, we request that the selected consultant provide a letter to the owner, with a copy to Ninyo & Moore, indicating that they fully understand our geotechnical recommendations and that they are in full agreement with the recommendations contained in this report. Qualified subcontractors utilizing appropriate techniques and construction materials should perform construction of the proposed improvements.

9.13 Plan Review

The recommendations presented in this report are based on preliminary design information for the proposed project and on the findings of our geotechnical evaluation. When finished, project plans and specifications should be reviewed by the geotechnical consultant prior to submitting the plans and specifications for bid. Additional field exploration and laboratory testing may be needed upon review of the project design plans.

9.14 Pre-Construction Meeting

We recommend a pre-construction meeting be held. The owner or the owner's representative, the architect, the contractor, and the geotechnical consultant should be in attendance to discuss the plans and the project.

10 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

11 REFERENCES

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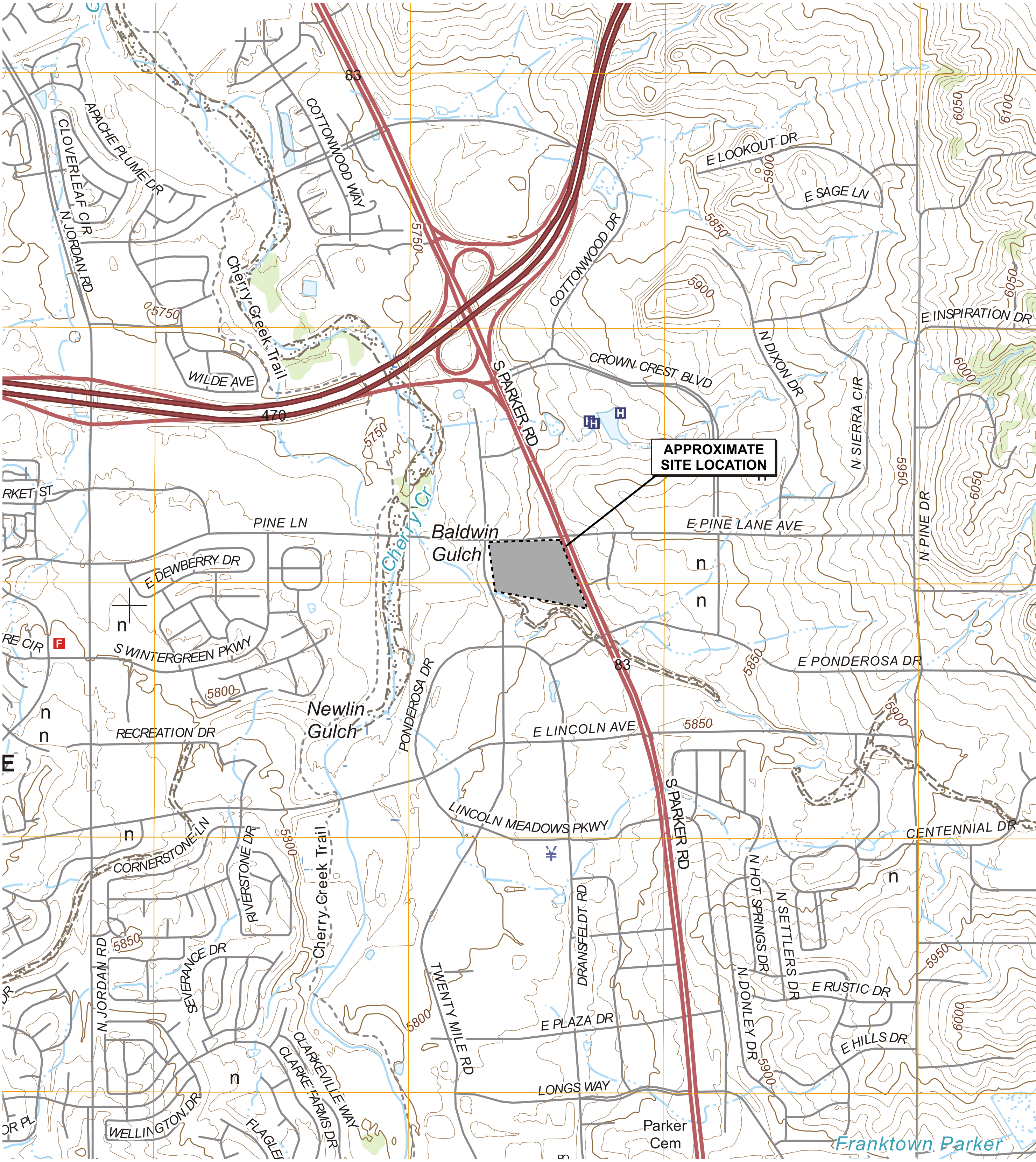
FIGURES



Source: US Geological Survey 7.5-minute topographic map, Parker, Colorado, 2016.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 1

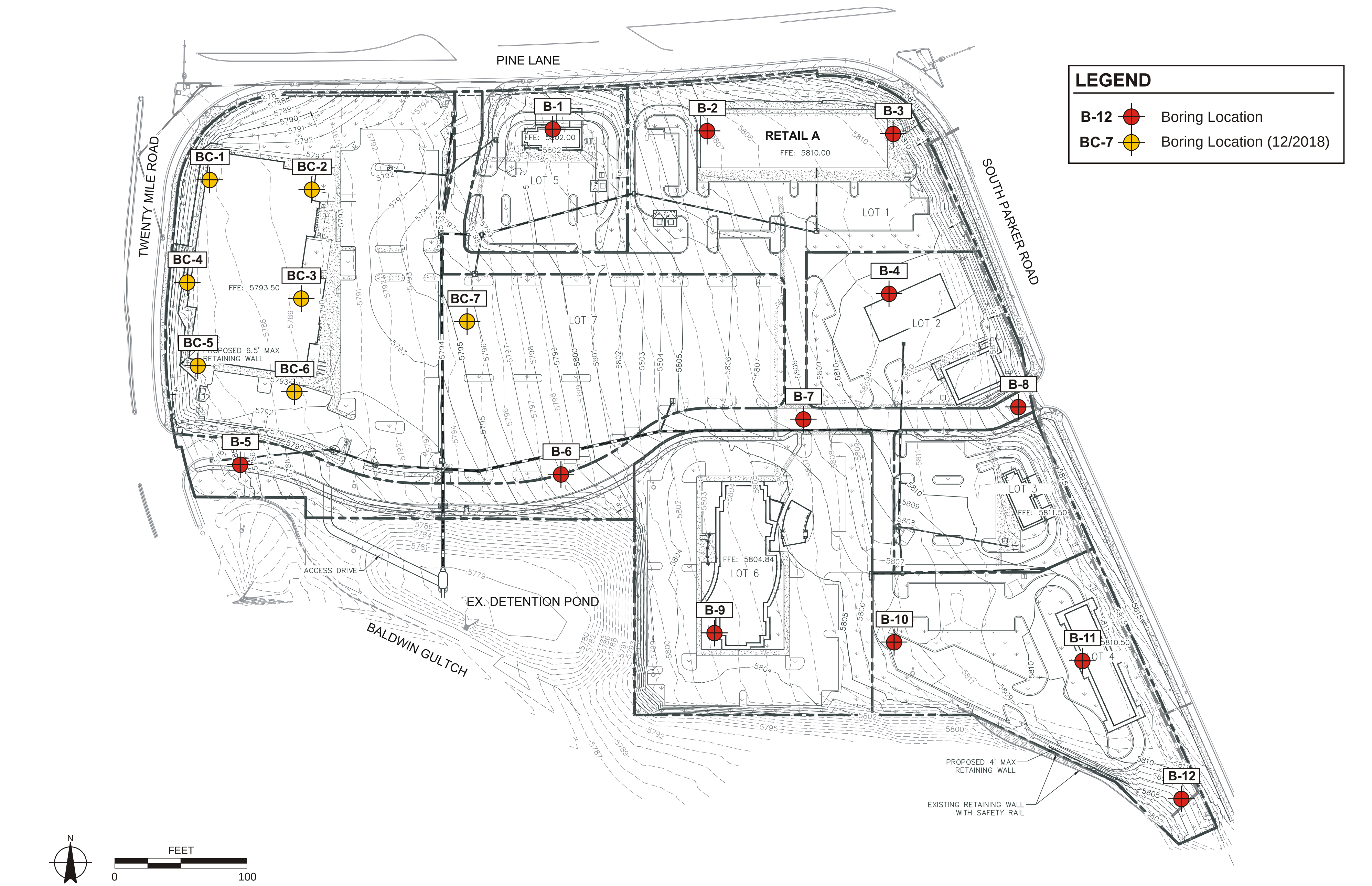
SITE LOCATION

PARKER AND PINE RETAIL PRELIM
PARKER, COLORADO

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

501651001 | 3/19

bsm file no: 1651vmap0319



LEGEND

B-12 Boring Location

BC-7 Boring Location (12/2018)

SOURCE: KIMLEY HORN, 11/30/18

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

bsm file no: 1651blm0319a

FIGURE 2

BORING LOCATIONS

PARKER AND PINE RETAIL PRELIM
PARKER, COLORADO

501651001 | 3/19

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory boring. The samples were bagged and transported to the laboratory for testing.

Field Procedure for the Collection of Ring-lined Samples
Ring-lined soil samples were obtained in the field using the following methods.

The Modified California Split-Barrel Drive Sampler
The sampler, with an external diameter of 3.0 inches, was lined with thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer or bar, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

The California Drive Sampler
The sampler, with an external diameter of 2.4 inches, was lined with four 4-inch long, thin brass rings with inside diameters of approximately 1.9 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

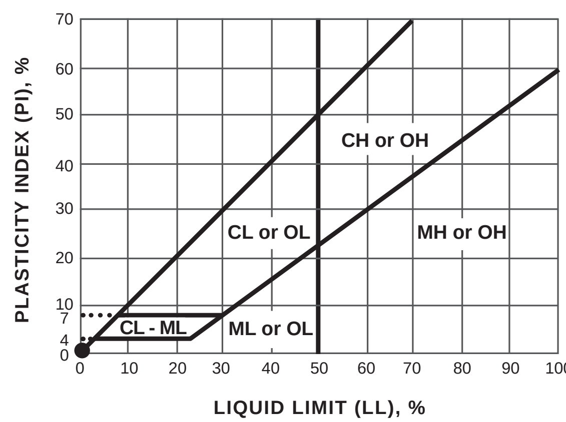
SOIL CLASSIFICATION CHART PER ASTM D 2488

PRIMARY DIVISIONS			SECONDARY DIVISIONS: GROUP SYMBOL	SECONDARY DIVISIONS: GROUP NAME
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

PLASTICITY CHART



APPARENT DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPOOLING CABLE OR CATHEAD: SPT (blows/foot)	SPOOLING CABLE OR CATHEAD: MODIFIED SPLIT BARREL (blows/foot)	AUTOMATIC TRIP HAMMER: SPT (blows/foot)	AUTOMATIC TRIP HAMMER: MODIFIED SPLIT BARREL (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

CONSISTENCY - FINE-GRAINED SOIL

CONSIS-TENCY	SPOOLING CABLE OR CATHEAD: SPT (blows/foot)	SPOOLING CABLE OR CATHEAD: MODIFIED SPLIT BARREL (blows/foot)	AUTOMATIC TRIP HAMMER: SPT (blows/foot)	AUTOMATIC TRIP HAMMER: MODIFIED SPLIT BARREL (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

Explanation of USCS Method of Soil Classification

PROJECT NO.	DATE	FIGURE

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								2-inch inner diameter split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.
								Sample retained by others.
5								Standard Penetration Test (SPT).
								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
10								Seepage.
								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	<u>MAJOR MATERIAL TYPE (SOIL)</u>: Solid line denotes unit change.
							CL	Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

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BORING LOG

Explanation of Boring Log Symbols

PROJECT NO.	DATE	FIGURE

DATE DRILLED	3/4/2019	BORING NO.	B-1
GROUND ELEVATION	5,801'	SHEET	1 OF 1
METHOD OF DRILLING	CME 55, 4" Solid-Stem Auger (Vine Laboratories)		
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)	DROP	30"
SAMPLED BY	DLH	LOGGED BY	DLH
REVIEWED BY	BFG		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	FILL: Light brown to brown, dry to moist, clayey fine to coarse SAND with gravel.
			31	7.0	102.7		SC	ALLUVIUM: Yellow to light brown, dry to moist, medium dense, clayey fine to coarse SAND.
			26	4.4	116.0			
10			50/9"	8.3	115.7			DAWSON FORMATION: Light olive and pink, dry, strongly cemented, clayey SANDSTONE.
			50/8"					Claystone interbeds.
20			50/11"	29.2	93.7			Brown to reddish gray, moist, hard, sandy CLAYSTONE; iron staining.
			50/10"					Red to reddish gray, moist, moderately cemented, clayey SANDSTONE; iron staining.
30			50/6"	10.7	104.4			Strongly cemented.
40								Total Depth = 29.5 feet. Groundwater was not encountered during drilling. Backfilled with on-site soils on 3/4/2019. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION

DATE DRILLED 3/4/2019 BORING NO. B-2
GROUND ELEVATION 5,807' SHEET 1 OF 1
METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories)
DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"
SAMPLED BY DLH LOGGED BY DLH REVIEWED BY BFG

DEPTH (feet)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0				SC	ALLUVIUM: Reddish brown to brown, moist, medium dense, clayey fine to medium SAND.
	20				
	21				
10	37	5.7	114.7		Reddish yellow and light brown; dry; fine to coarse.
	50/6"				DAWSON FORMATION: Light brown and gray, dry, strongly cemented, SANDSTONE.
20	50/7"	12.6	114.4		
	50/10"	26.7	98.8		Brown to reddish gray, moist, hard, sandy CLAYSTONE.
30					
40					

Total Depth = 24.8 feet.
Groundwater was not encountered during drilling.
Backfilled with on-site soils on 3/4/2019.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DATE DRILLED 3/4/2019 BORING NO. B-3

GROUND ELEVATION 5,810' SHEET 1 OF 1

METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY BFG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	ALLUVIUM: Light brown to brown, dry to moist, medium dense, clayey fine to medium SAND.
			25	6.7	101.9			
			16					Reddish brown with yellow; few clayey sand interlayers.
10			50/9"	19.9	106.4			DAWSON FORMATION: Fed and brown, dry, strongly cemented, clayey SANDSTONE with claystone interbeds.
			50/11"					Bluish gray and red; moderately cemented.
20			50/12"	24.6	102.3			Reddish brown to brown, dry to moist, hard, sandy CLAYSTONE.
			50/9"					Red with gray, dry to moist, strongly cemented, SANDSTONE; weathered.
30			50/9"	18.0	102.9			Iron staining.
								Total Depth = 29.8 feet. Groundwater was not encountered during drilling. Backfilled with on-site soils on 3/4/2019. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40								

FIGURE A - 3

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DATE DRILLED 3/4/2019 BORING NO. B-4

GROUND ELEVATION 5,809' SHEET 1 OF 1

METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY BFG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	ALLUVIUM: Light brown, dry, stiff, sandy lean CLAY; white calcium mineralizations.
			12	6.5	98.5			
			17					Very stiff.
10			28	20.4	103.3		SC	Reddish gray with yellow, moist, medium dense, clayey fine to medium SAND.
			41					DAWSON FORMATION: Brown to reddish gray, moist, moderately hard, sandy CLAYSTONE; weathered; iron staining.
20			50/11"	23.2	103.5			Hard.
			50/12"					Gypsum mineralizations.
								Total Depth = 25 feet. Groundwater not encountered during drilling. Backfilled with on-site soils on 3/4/2019.
30								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40								

DATE DRILLED	3/5/2019	BORING NO.	B-5
GROUND ELEVATION	5,786'	SHEET	1 OF 2
METHOD OF DRILLING	CME 55, 4" Solid-Stem Auger (Vine Laboratories)		
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)	DROP	30"
SAMPLED BY	DLH	LOGGED BY	DLH
REVIEWED BY	BFG		

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	Bulk					CH	ALLUVIUM: Brown with red, moist, hard, fat CLAY; trace sand.
	Driven	29	11.4	123.8			
	Driven	27	18.9	107.8			
10	Driven	16	13.0	91.3			Light brown; dry to moist; very stiff; few calcium mineralizations.
	Driven	17				CL	Light brown, dry, very stiff, sandy lean CLAY.
20	Driven	10				SC	Brown with red and gray, dry, loose, clayey fine to coarse SAND.
	Driven	9				CL	Light brown to brown, moist to wet, stiff, sandy lean CLAY with sand interlayers.
30	Driven	15	14.7	118.8		SP-SC	@29': Groundwater encountered during drilling. Light brown with red and gray, wet, medium dense, fine to coarse SAND with clay; trace gravel.
	Driven	10					
40	Driven	18					

FIGURE A - 5

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PARKER AND PINE RETAIL DEVELOPMENT
PARKER, COLORADO

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 3/5/2019 BORING NO. B-5 GROUND ELEVATION 5,786' SHEET 2 OF 2 METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories) DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30" SAMPLED BY DLH LOGGED BY DLH REVIEWED BY BFG **DESCRIPTION/INTERPRETATION**
40							CH	ALLUVIUM: (continued) Olive with reddish brown, moist to wet, very stiff, fat CLAY with sand.
								DAWSON FORMATION: Light gray to olive, moist, hard, CLAYSTONE; weathered.
			50/11"	18.5	112.8			
								Total Depth = 44.9 feet. Groundwater encountered at a depth of approximately 29 feet during drilling. Backfilled with on-site soils on 3/5/2019. Notes: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
50								
60								
70								
80								

DATE DRILLED 3/5/2019 BORING NO. B-6

GROUND ELEVATION 5,798' SHEET 1 OF 1

METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY BFG

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X						CL	ALLUVIUM: Light brown to reddish brown, dry, stiff, lean CLAY with sand.
	X	X	12	5.3	101.6			
	X	X	20					Yellow to light brown; very stiff.
10		X	29	15.7	105.5		SC	Reddish brown to dark brown, moist, hard, clayey fine to coarse SAND.
		X	40					DAWSON FORMATION: Light brown to olive brown, moist, moderately hard, CLAYSTONE; weathered; iron staining.
20		X	50/11"	24.2	101.6			Reddish brown to reddish gray; hard.
		X	50/9"					Bluish gray to bluish green; sandy.
30								
40								

Total Depth = 24.8 feet.
Groundwater was not encountered during drilling.
Backfilled with on-site soils on 3/5/2019.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	ALLUVIUM: Yellow to light brown, dry to moist, loose, clayey fine to medium SAND.
		X	14	3.2	106.9			
		X	14	7.7	100.1			
10		X	12	5.4	110.3			
		X	28	20.8	107.0		CH	Reddish brown to brown, moist, hard, fat CLAY; trace sand.
								DAWSON FORMATION: Brown to reddish brown, moist, hard, sandy CLAYSTONE; iron staining.
20		X	50/11"	19.2	110.9			
		X	50/11"					Reddish gray with reddish brown.
								Total Depth = 24.9 feet. Groundwater was not encountered during drilling. Backfilled with on-site soils on 3/4/2019.
30								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40								

DATE DRILLED 3/4/2019 BORING NO. B-7

GROUND ELEVATION 5,807' SHEET 1 OF 1

METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY BFG

DATE DRILLED 3/4/2019 BORING NO. B-8

GROUND ELEVATION 5,815' SHEET 1 OF 1

METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY BFG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	FILL: Brown, dry to moist, sandy lean CLAY.
			42	9.6	118.4			
			24	7.0	107.8			Light brown to brown.
10			25	8.9	116.8			
			33					DAWSON FORMATION: Brown to reddish gray, moist, moderately hard, sandy CLAYSTONE.
20			50/12"	19.5	108.1			Hard; iron staining.
			50/11"					
30			50/12"	22.1	105.7			Total Depth = 30 feet. Groundwater was not encountered during drilling. Backfilled with on-site soils on 3/4/2019. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40								

DATE DRILLED 3/5/2019	BORING NO. B-9	
GROUND ELEVATION 5,802'	SHEET 1 OF 1	
METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories)		
DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer)	DROP 30"	
SAMPLED BY DLH	LOGGED BY DLH	REVIEWED BY BFG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	ALLUVIUM: Yellow to light brown, dry, medium dense, clayey fine to medium SAND.
			20	3.3	106.1			
			23					
10			18	5.5	111.8			
			50/11"					DAWSON FORMATION: Reddish brown to reddish gray, moist, hard, CLAYSTONE; iron staining.
20			50/9"	22.7	104.4			Trace manganese staining.
			50/6"					
								Total Depth = 24.5 feet.
								Groundwater was not encountered during drilling.
								Backfilled with on-site soils on 3/5/2019.
30								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	ALLUVIUM: Light brown to olive brown, dry to moist, medium dense, clayey fine to medium SAND.
	X	X	20	4.8	104.7			
	X	X	18	4.9	105.5			
		X	8	6.7	103.2			
10							CH	Brown to grayish brown, moist, fat CLAY with sand.
		X	4				SC	Yellow, moist, very loose, clayey fine to medium SAND.
20		X	50/10"	15.3	117.7			DAWSON FORMATION: Yellowish brown to red, moist, hard, CLAYSTONE; iron staining.
		X	50/11"					Reddish brown.
30								
40								

DATE DRILLED 3/4/2019 BORING NO. B-10

GROUND ELEVATION 5,809' SHEET 1 OF 1

METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY BFG

Total Depth = 24.9 feet.
Groundwater was not encountered during drilling.
Backfilled with on-site soils on 3/4/2019.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DATE DRILLED	3/4/2019
BORING NO.	B-11
GROUND ELEVATION	5,811'
SHEET	1 OF 1
METHOD OF DRILLING	CME 55, 4" Solid-Stem Auger (Vine Laboratories)
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)
DROP	30"
SAMPLED BY	DLH
LOGGED BY	DLH
REVIEWED BY	BFG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	FILL: Brown, dry to moist, clayey fine to medium SAND.
		X	30	4.0	127.2			
		X	21	7.7	112.7		SC	ALLUVIUM: Light brown to reddish brown, dry to moist, medium dense, clayey fine to medium SAND.
10		X	19	19.2	111.0		CH	Yellowish brown to brown, moist, very stiff, fat CLAY with sand.
		X	13					Stiff.
20		X	50/12"	23.1	105.6			DAWSON FORMATION: Brown to reddish brown, moist, hard, CLAYSTONE; iron staining.
		X	50/7"					Light brown to red; iron staining.
30								
40								

Total Depth = 24.6 feet.
Groundwater was not encountered during drilling.
Backfilled with on-site soils on 3/4/2019.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	FILL: Yellow to dark brown, moist, lean CLAY with sand.
			30	13.2	112.2			
			39	15.2	114.8			
							CL	ALLUVIUM: Brown to dark brown, moist, lean CLAY with sand.
10			50/11"	17.6	111.7			DAWSON FORMATION: Brown to reddish brown, moist, hard, CLAYSTONE.
			50/7"					Yellow to yellowish brown; trace iron staining.
20			50/9"	19.5	111.9			
			50/10"					
30								Total Depth = 24.8 feet. Groundwater was not encountered during drilling. Backfilled with on-site soils on 3/5/2019. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40								

DATE DRILLED 3/5/2019 BORING NO. B-12

GROUND ELEVATION 5,808' SHEET 1 OF 1

METHOD OF DRILLING CME 55, 4" Solid-Stem Auger (Vine Laboratories)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY BFG

FIGURE A - 13

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PARKER AND PINE RETAIL DEVELOPMENT
PARKER, COLORADO

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classifications System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory excavations in Appendix A.

In-Place Moisture and Density Tests
The moisture content and dry density of ring-lined samples obtained from the exploratory boring were evaluated in general accordance with ASTM D 2837. These test results are presented on the logs of the exploratory boring in Appendix A.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results and classifications are shown on Figures B-1 and B-2.

No. 200 Sieve Analysis
An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figures B-3 and B-4.

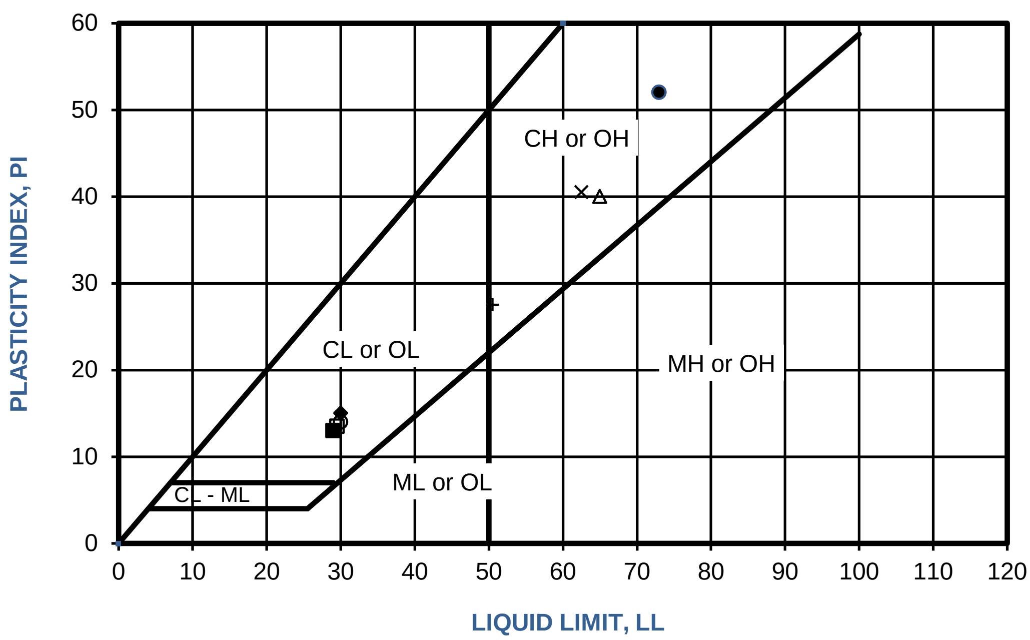
Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 6913. The grain-size distribution curves are shown on Figures B-5 through B-10. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Swell/Consolidation Tests
The consolidation and/or swell potential of selected materials were evaluated in general accordance with ASTM D 4546. Specimens were loaded with a specified surcharge before inundation with water. Readings of volumetric consolidation/swell were recorded until completion of primary consolidation/swell. After the completion of primary swell, surcharge loads were increased incrementally to evaluate swell pressure. The results of the consolidation/swell tests are presented on Figures B-11 through B-20.

Soil Corrosivity Tests
A soil pH test was performed on a representative sample in general accordance with ASTM Test Method D 4972. A soil minimum resistivity test was performed on a representative sample in general accordance with AASHTO T288. The sulfate content of a selected sample was evaluated in general accordance with CDOT Test Method CP-L 2103. The chloride content of a selected sample was evaluated in general accordance with CDOT Test Method CP-L 2104. The test results are presented on Figure B-21.

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	EQUIVALENT USCS
●	B-1	19.0-19.9	73	21	52	CH	CH
■	B-2	2.0-3.5	29	16	13	CL	SC
◆	B-2	4.0-5.0	30	15	15	CL	SC
○	B-3	2.0-3.5	30	16	14	CL	SC
□	B-4	2.0-3.5	30	16	14	CL	CL
Δ	B-4	9.0-10.0	65	25	40	CH	SC
x	B-5	4.0-5.0	63	22	41	CH	CH
+	B-6	9.0-10.0	51	23	28	CH	SC

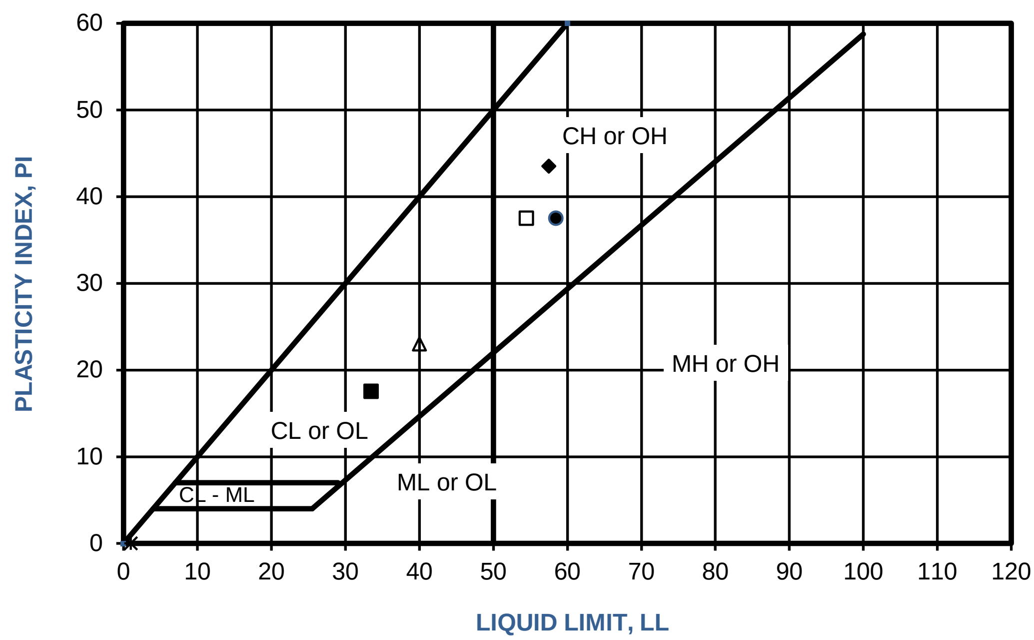
NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	EQUIVALENT USCS
●	B-7	14.0-15.0	59	21	38	CH	CH
■	B-8	2.0-3.5	34	16	18	CL	CL
◆	B-9	14.0-14.9	58	14	44	CH	CH
○	B-11	2.0-3.0	28	14	14	CL	SC
□	B-11	9.0-10.0	55	17	38	CH	CH
Δ	B-12	4.0-5.5	40	17	23	CL	CL

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	EQUIVALENT USCS
B-2	2.0-3.5	Reddish Brown to Brown Clayey SAND	100	44	SC
B-2	4.0-5.0	Reddish Brown to Brown Clayey SAND	100	36	SC
B-3	2.0-3.5	Light Brown to Brown Clayey SAND	100	42	SC
B-4	2.0-3.5	Light Brown Sandy Lean CLAY	100	60	CL
B-4	9.0-10.0	Reddish Gray with Yellow Clayey SAND	100	40	SC
B-5	4.0-5.0	Brown with Red Fat CLAY; Trace Sand	100	99	CH
B-6	2.0-3.0	Light Brown to Reddish Brown Lean CLAY with Sand	100	81	CL
B-6	9.0-10.0	Reddish Brown to Dark Brown Clayey SAND	100	34	SC
B-7	14.0-15.0	Reddish Brown to Brown Fat CLAY; Trace Sand	100	92	CH

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-3

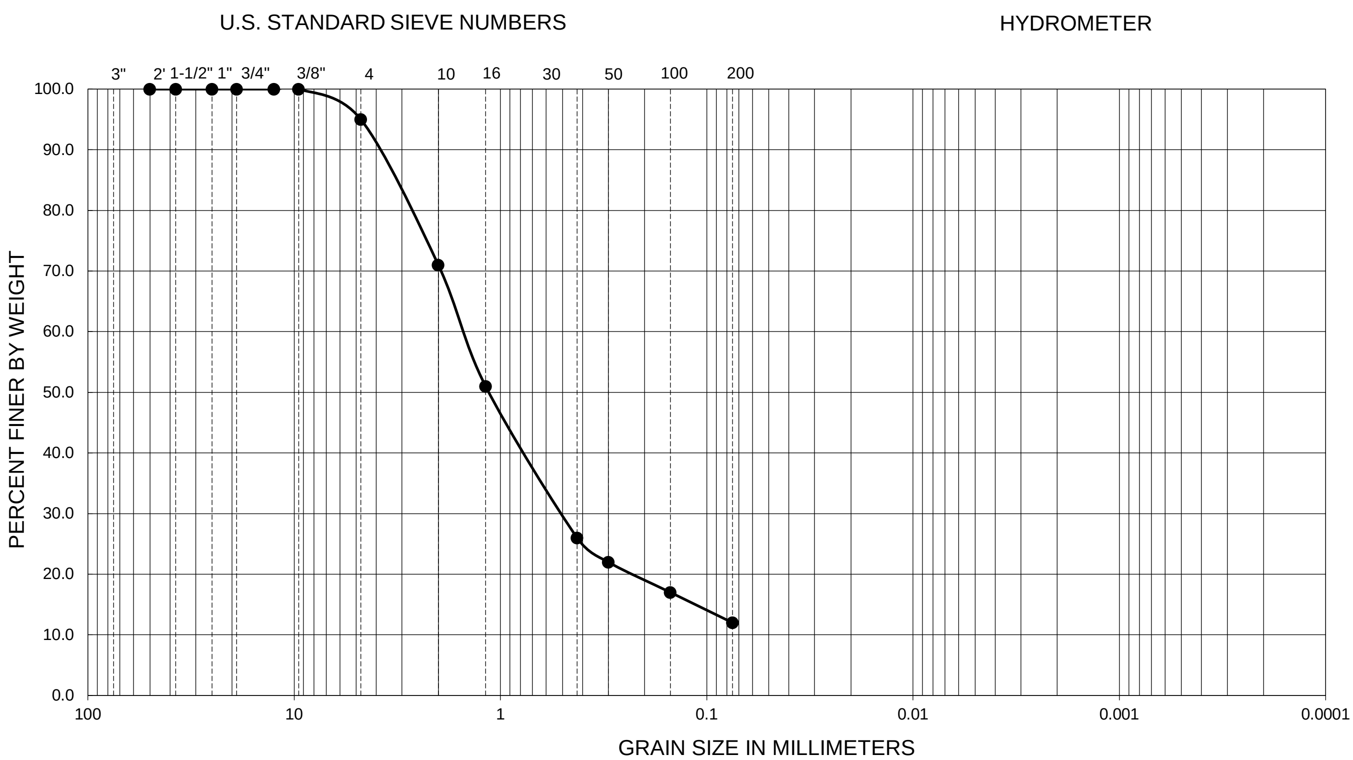
NO. 200 SIEVE ANALYSIS TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	EQUIVALENT USCS
B-8	2.0-3.5	Brown Sandy Lean CLAY	100	64	CL
B-9	2.0-3.0	Yellow to Light Brown Clayey SAND	100	18	SC
B-9	14.0-14.9	Reddish Brown to Reddish Gray CLAYSTONE; Dawson Formation	100	94	CH
B-11	2.0-3.0	Brown Clayey SAND	100	43	SC
B-11	4.0-5.0	Light Brown to Reddish Brown Clayey SAND	100	44	SC
B-11	9.0-10.0	Yellowish Brown to Brown Fat CLAY with Sand	100	81	CH
B-12	4.0-5.5	Brown to Dark Brown Lean CLAY with Sand	100	72	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

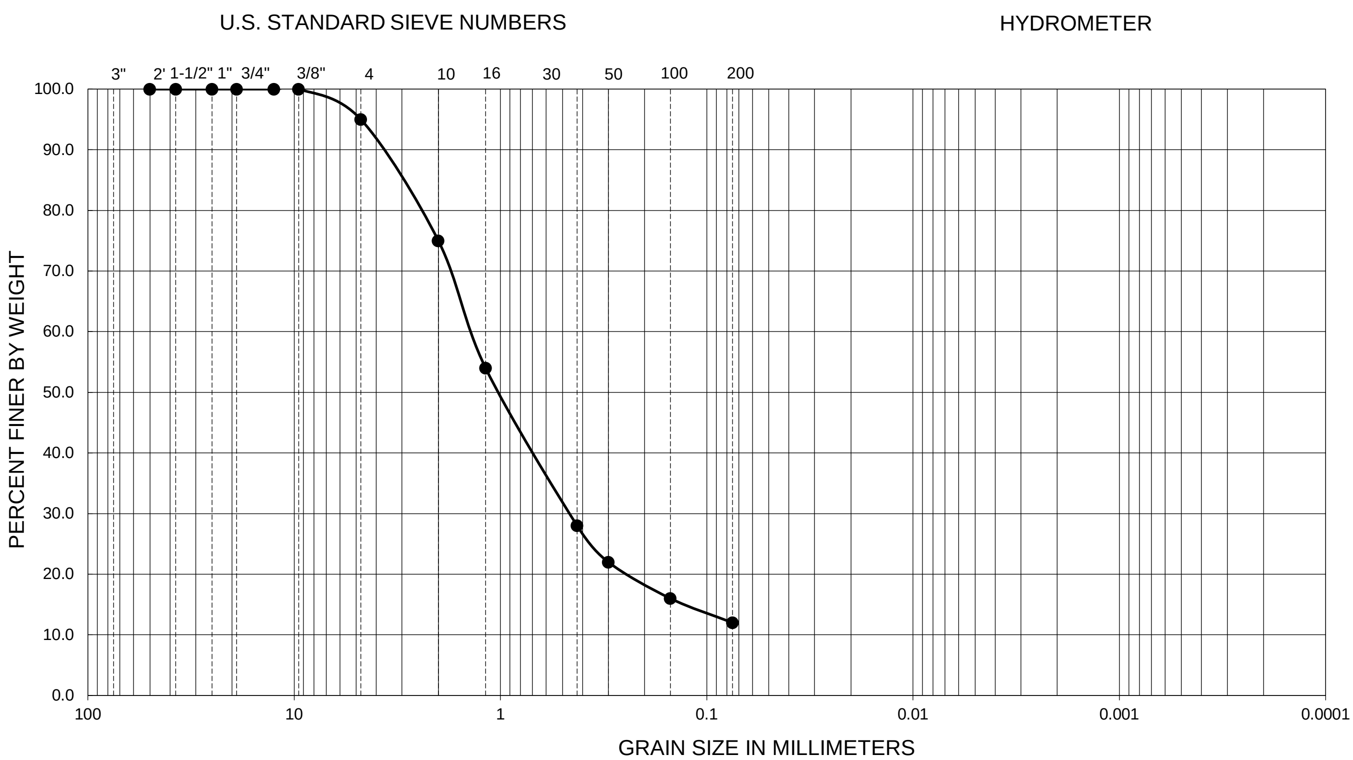
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-1	4.0-5.0	--	--	--	--	--	--	--	--	12	SC

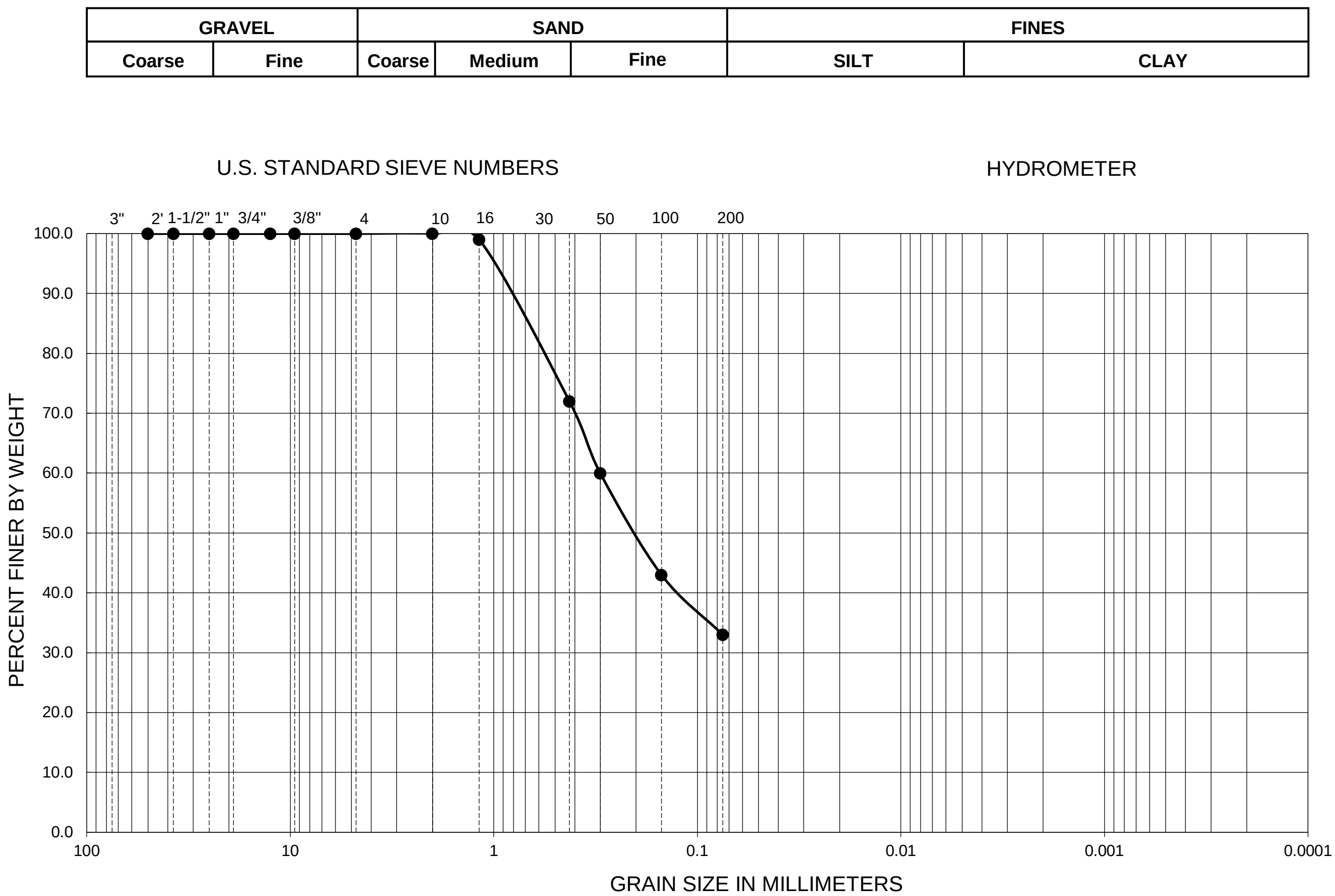
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-2	9.0-10.0	--	--	--	--	--	--	--	--	12	SC

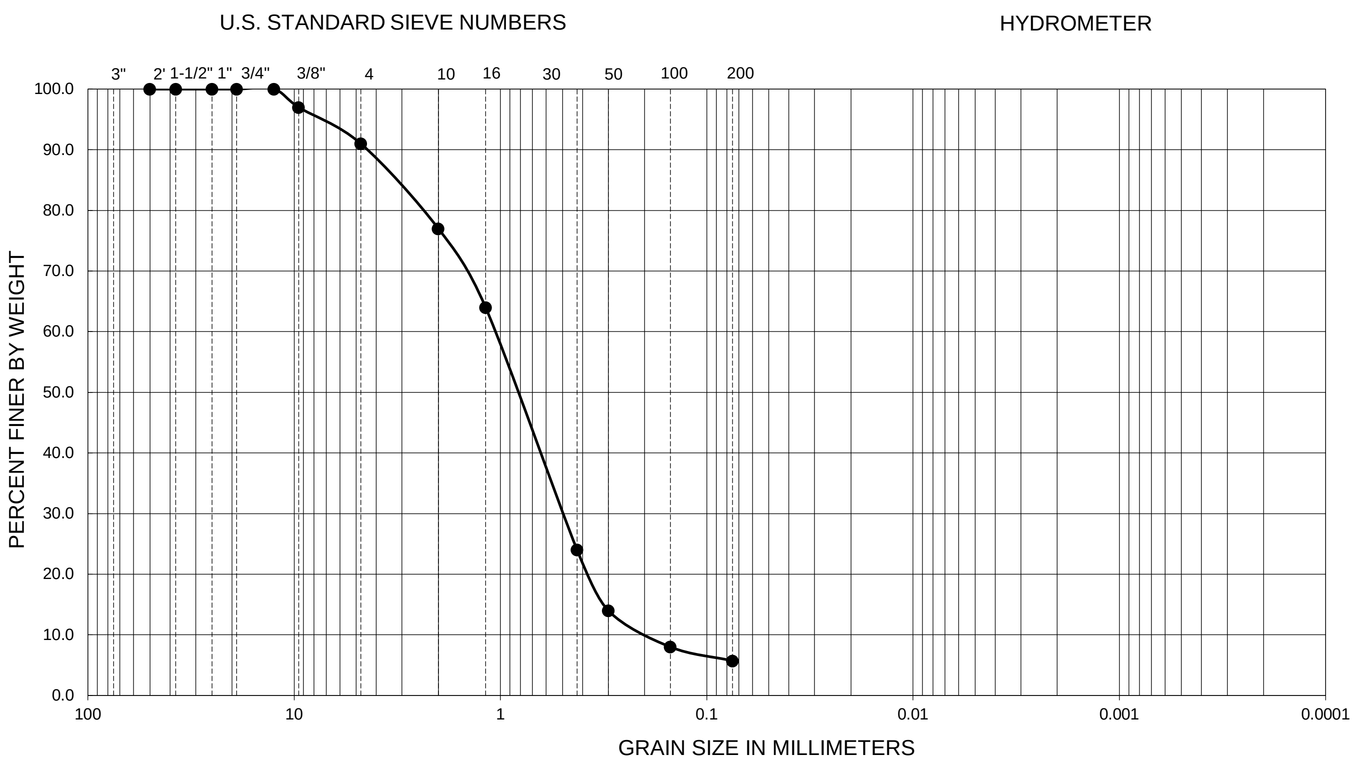
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-3	9.-9.75	--	--	--	--	--	--	--	--	33	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-5	29.0-30.0	--	--	--	0.21	0.40	1.20	5.7	0.6	5.7	SP-SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-8

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Geotechnical & Environmental Sciences Consultants

GRADATION TEST RESULTS
PARKER AND PINE RETAIL DEVELOPMENT

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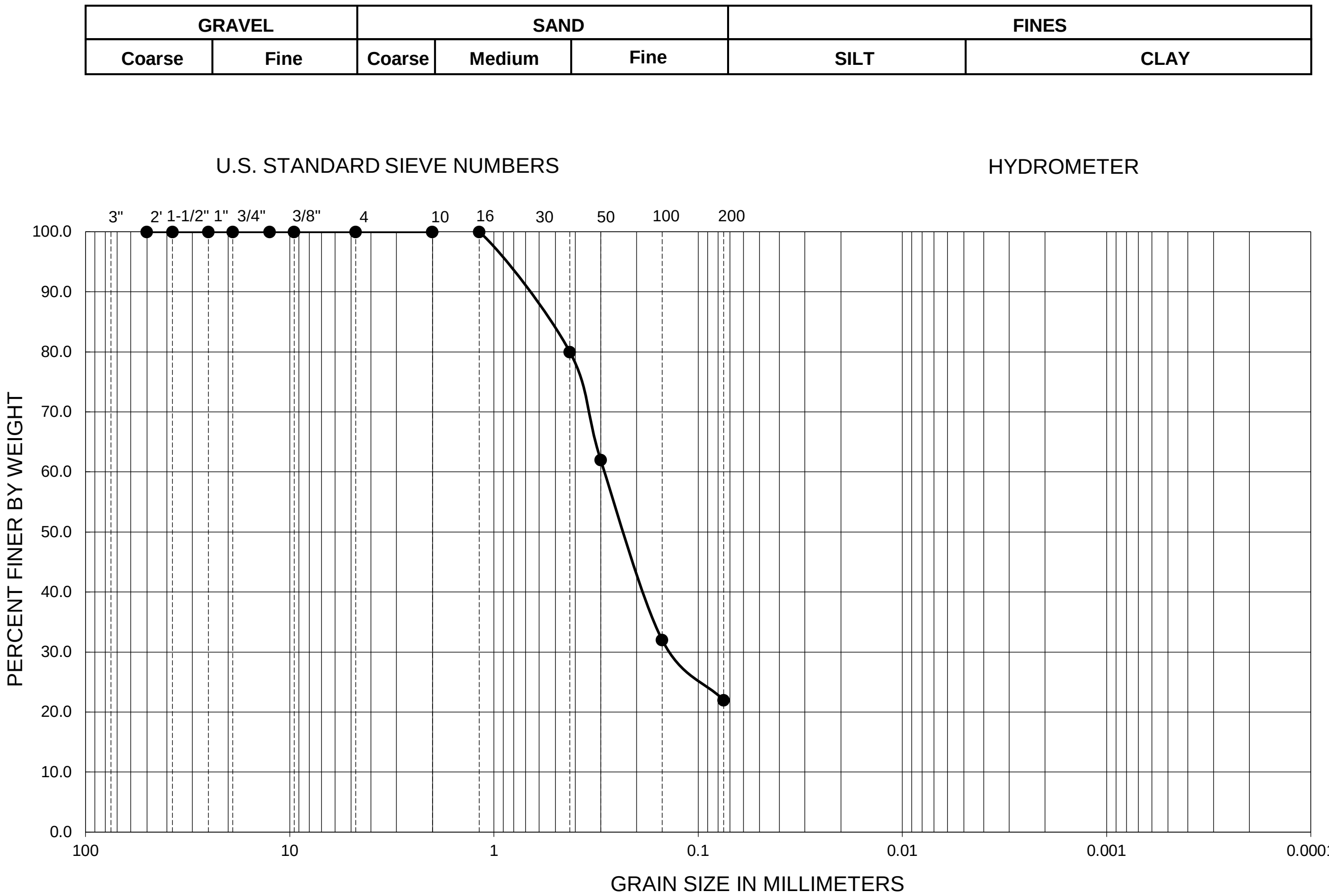

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-7	2.0-3.0	--	--	--	--	--	--	--	--	22	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-9

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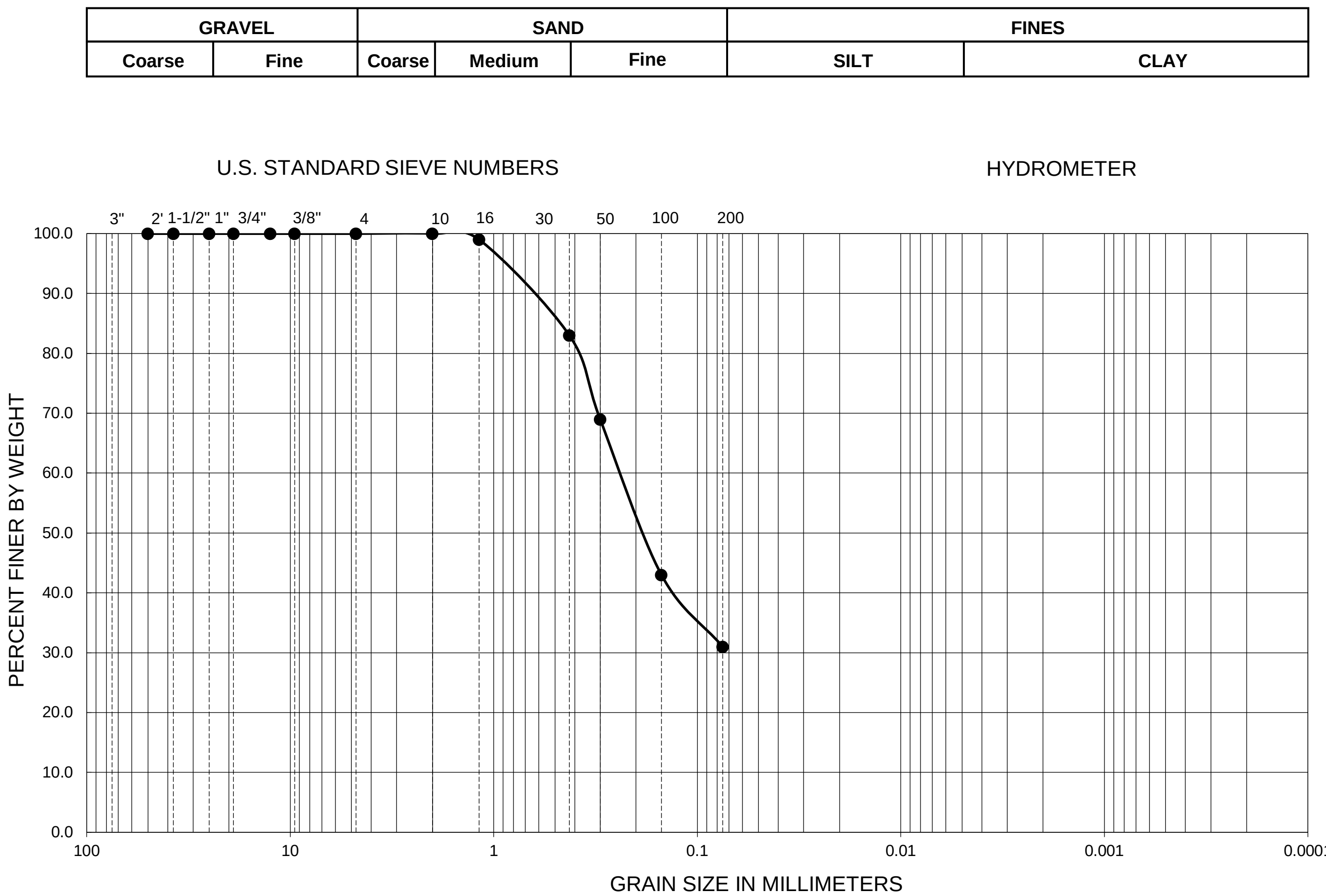
GRADATION TEST RESULTS
PARKER AND PINE RETAIL DEVELOPMENT



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-10	4.0-5.0	--	--	--	--	--	--	--	--	31	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

GRADATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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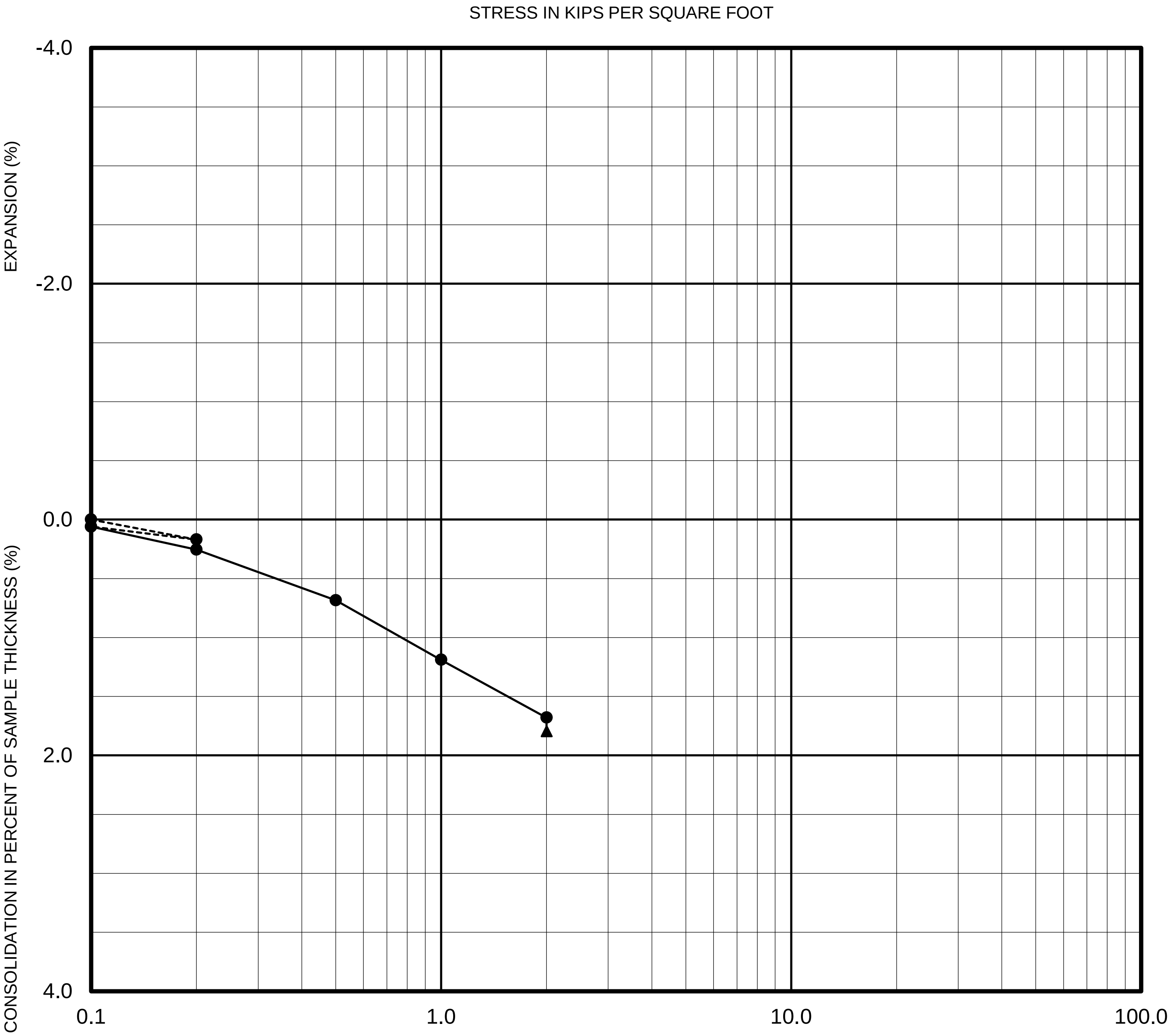

- Seating Cycle
- Loading Prior to Inundation
- Loading After Inundation
- Rebound Cycle

Moisture Content Before Test (%):	30.8%
Moisture Content After Test (%):	34.0%
Swell Percentage (%):	-0.1
Swell Pressure (psf):	-

Sample Location: B-1
Depth (ft): 19.0-19.8
Soil Type: DAWSON FORMATION

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-11

CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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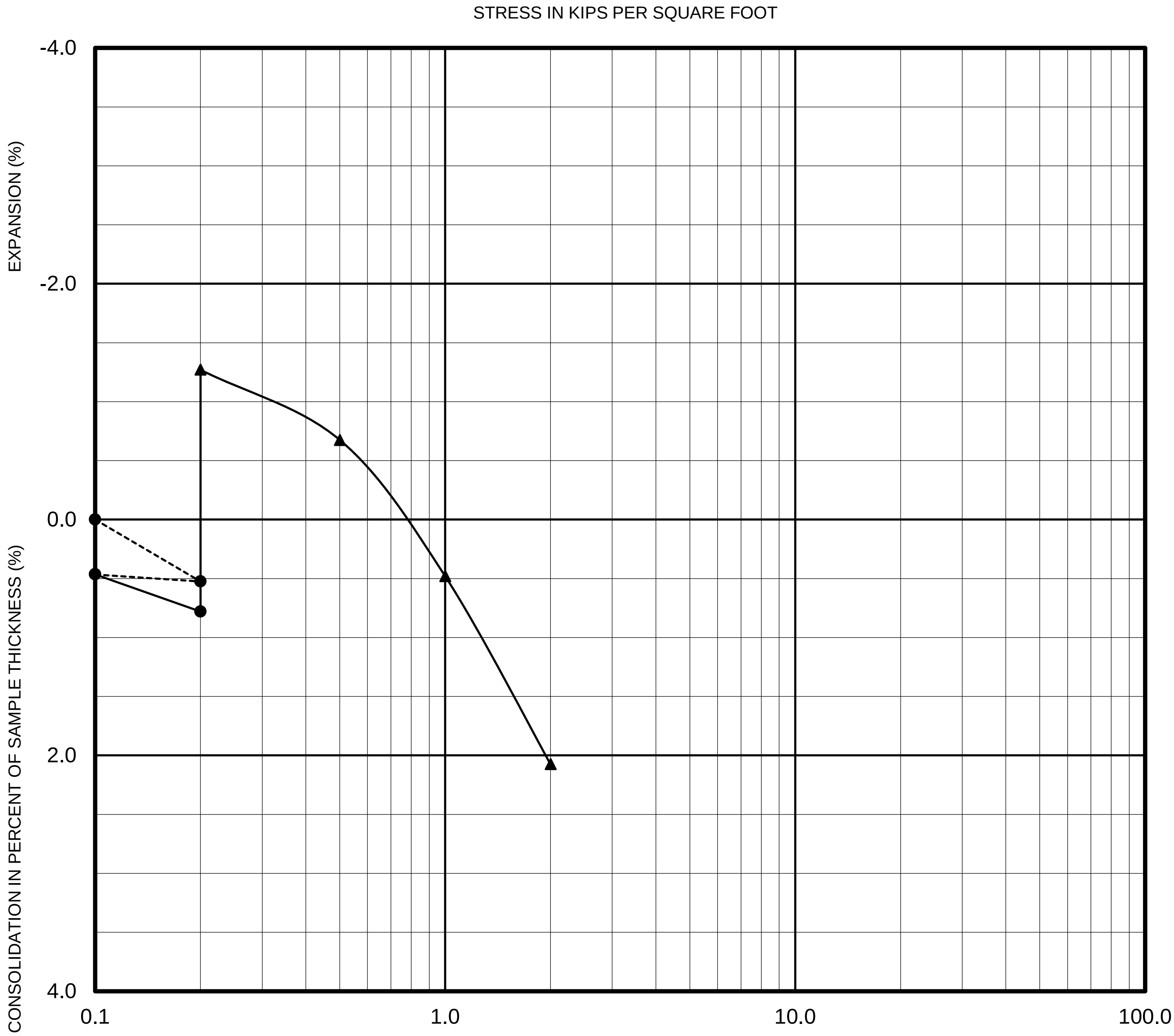

Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Moisture Content Before Test (%):	9.2%
Moisture Content After Test (%):	21.0%
Swell Percentage (%):	2.1
Swell Pressure (psf):	960.0

Sample Location: B-2
Depth (ft): 2.0-3.5
Soil Type: SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-12

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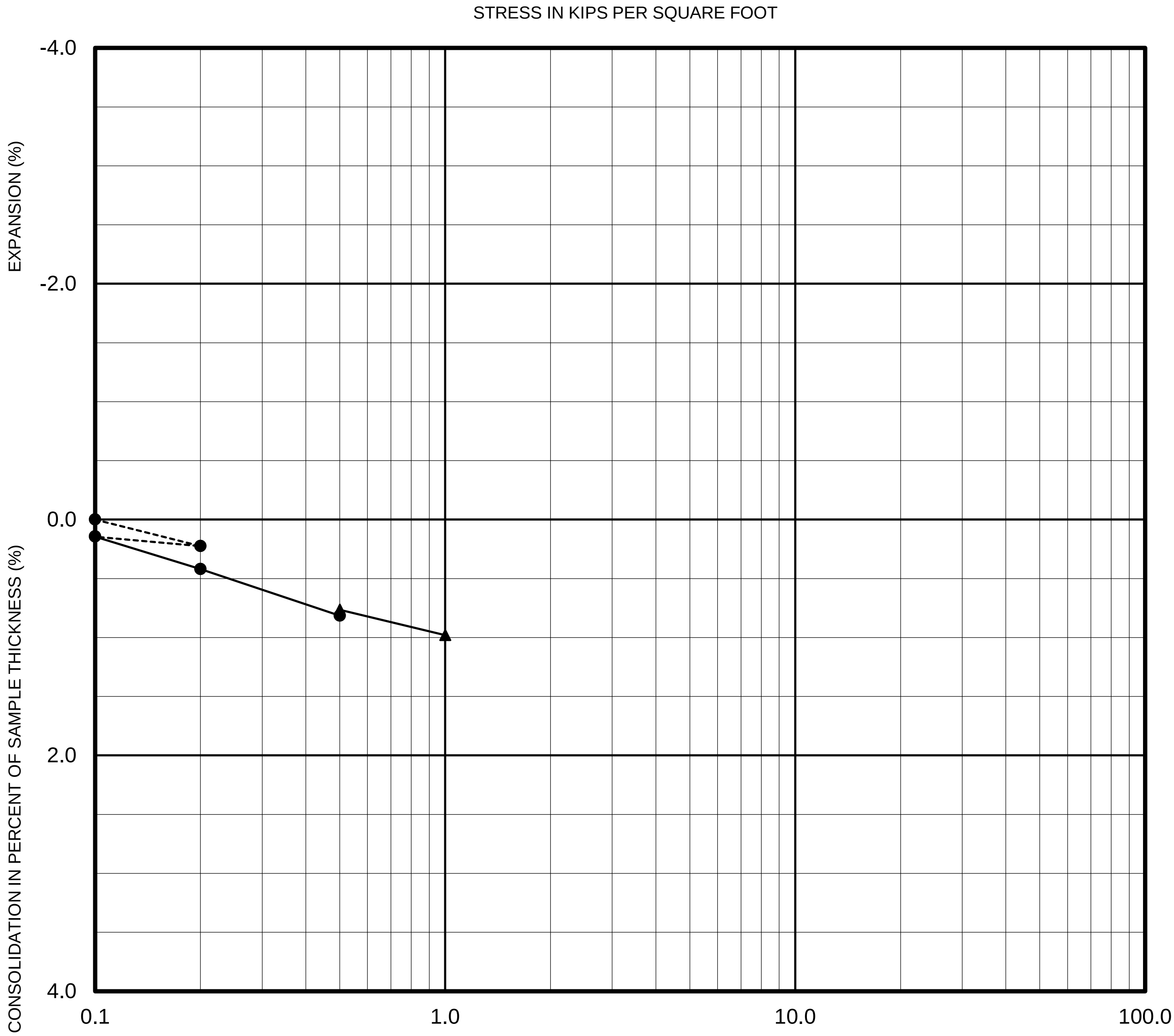
CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Moisture Content Before Test (%):	10.6%
Moisture Content After Test (%):	16.3%
Swell Percentage (%):	0.05
Swell Pressure (psf):	100.0

Sample Location: B-2
Depth (ft): 4.0-5.0
Soil Type: SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-13

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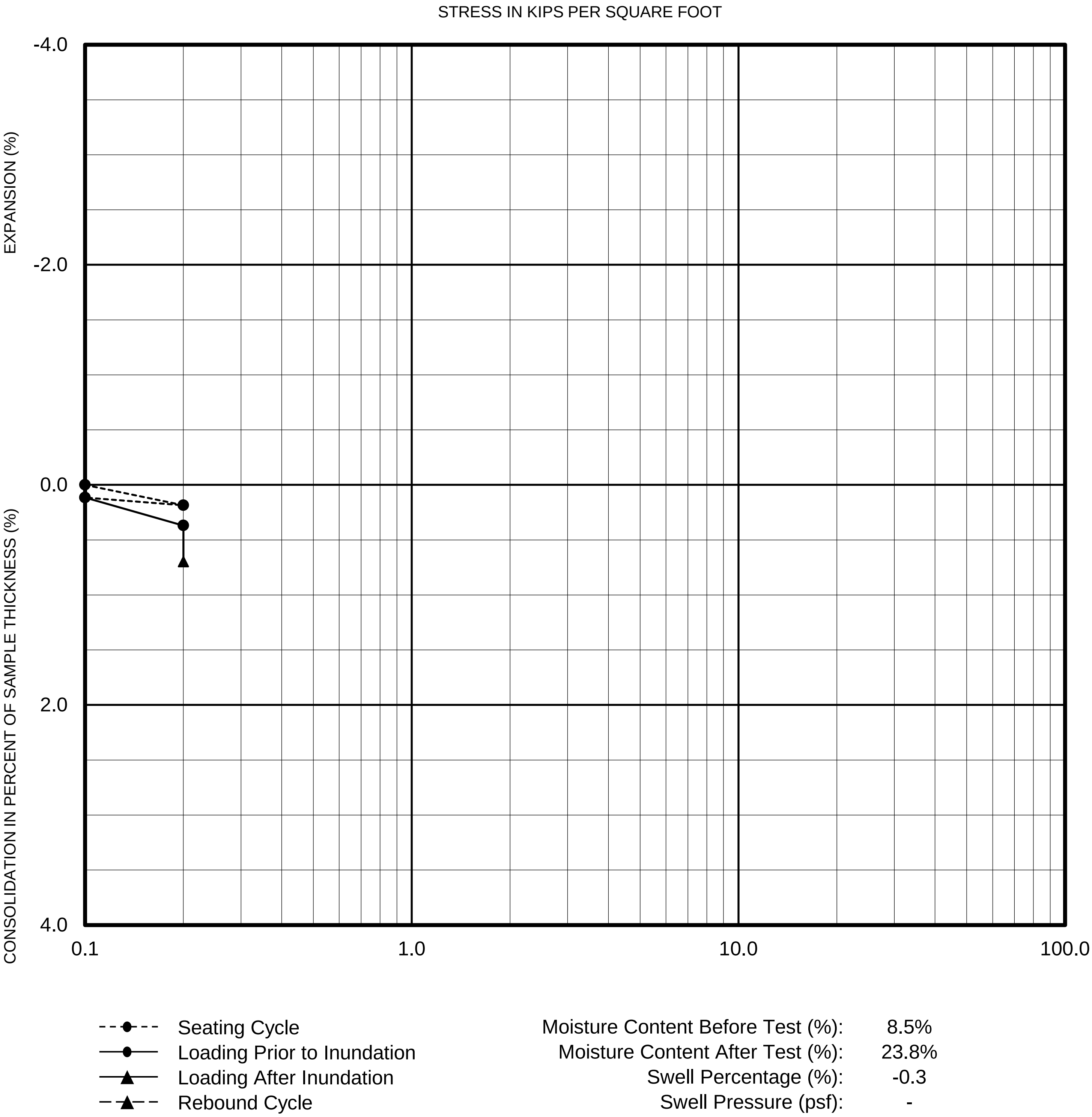
CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT



Moisture Content Before Test (%):	8.5%
Moisture Content After Test (%):	23.8%
Swell Percentage (%):	-0.3
Swell Pressure (psf):	-

Sample Location: B-4
Depth (ft): 2.0-3.5
Soil Type: CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-14

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CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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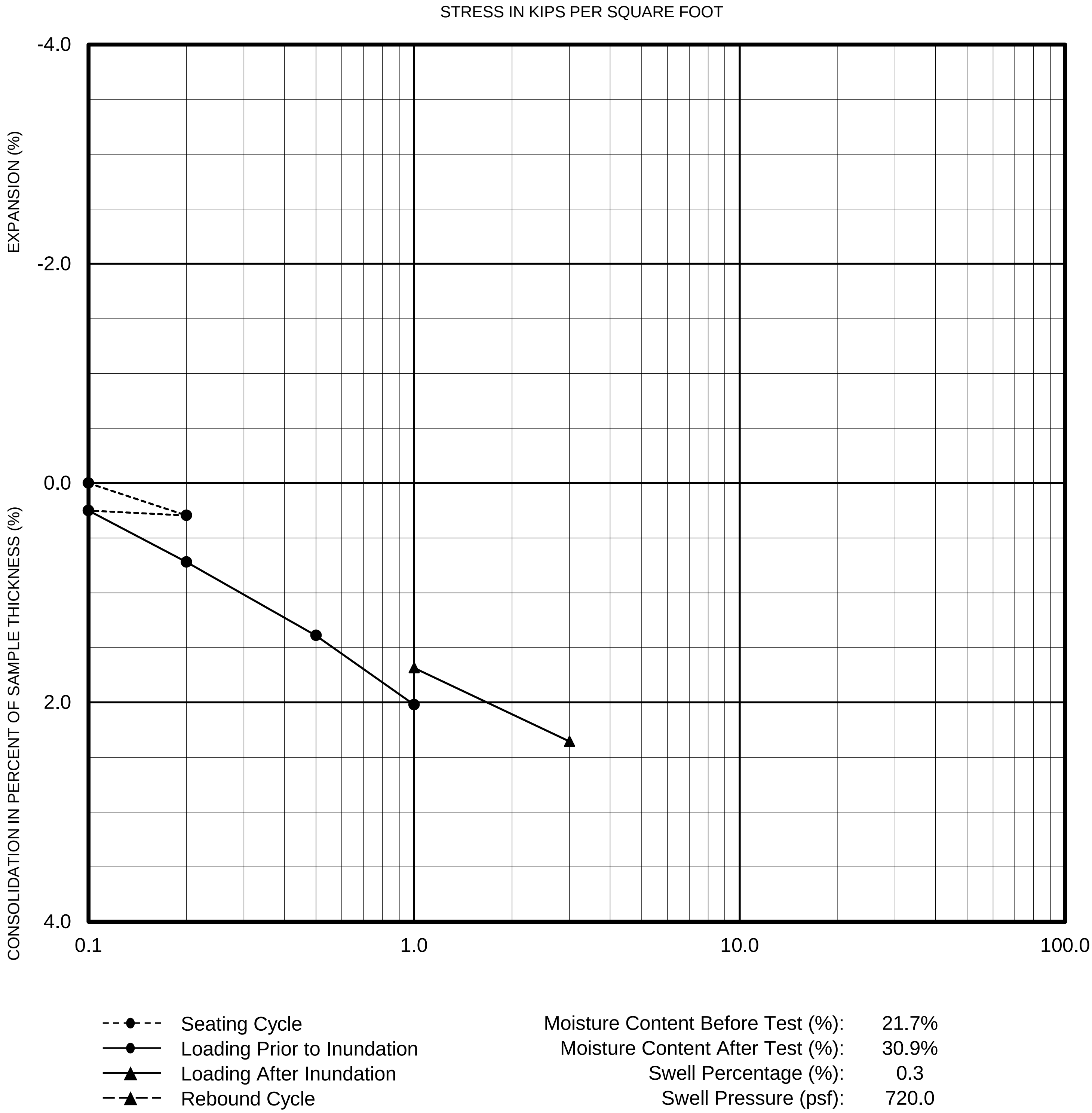

Moisture Content Before Test (%):	21.7%
Moisture Content After Test (%):	30.9%
Swell Percentage (%):	0.3
Swell Pressure (psf):	720.0

Sample Location: B-4
Depth (ft): 9.0-10.0
Soil Type: SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-15

CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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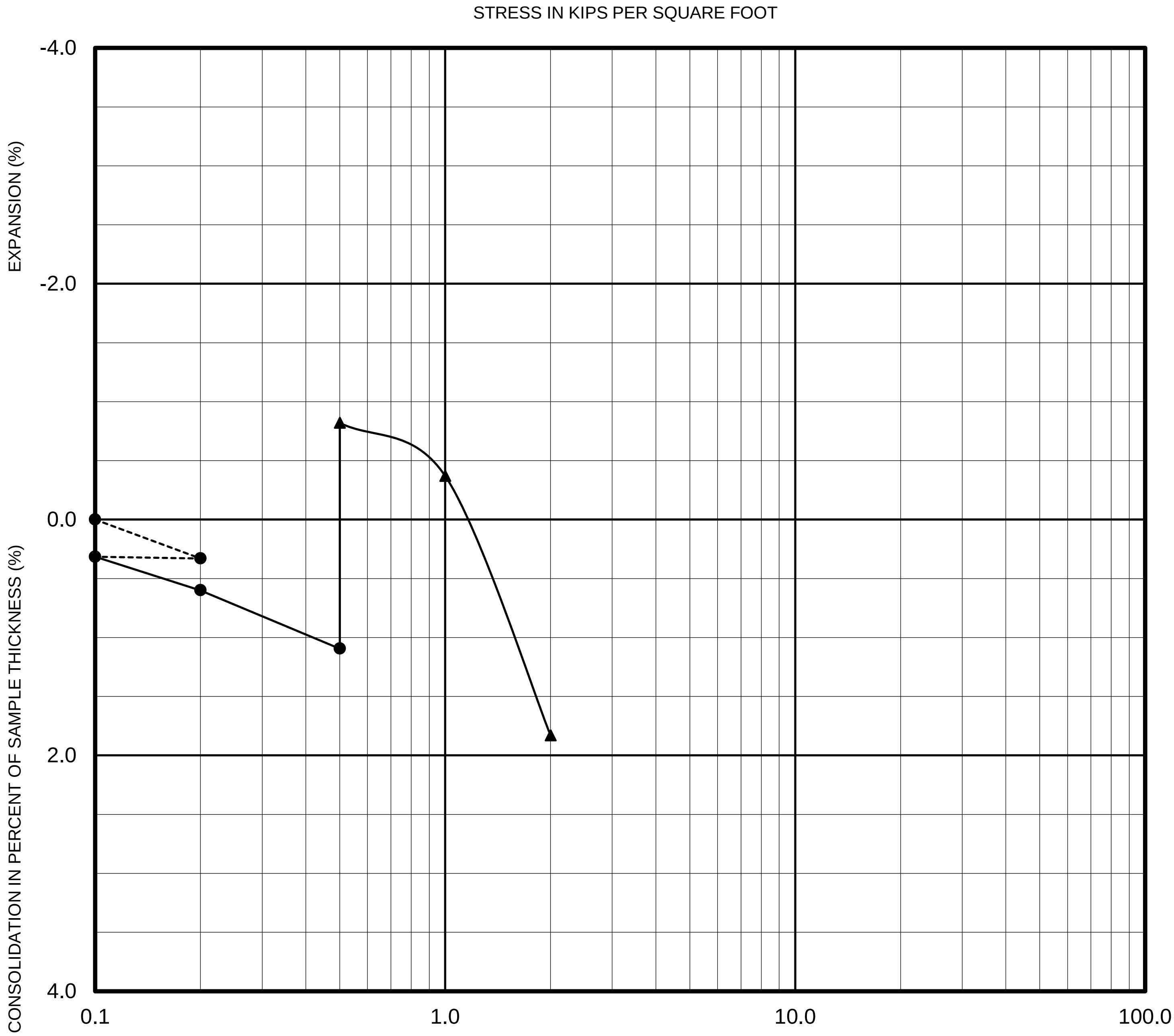

Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Moisture Content Before Test (%):	17.6%
Moisture Content After Test (%):	21.8%
Swell Percentage (%):	1.9
Swell Pressure (psf):	1600.0

Sample Location: B-5
Depth (ft): 4.0-5.0
Soil Type: CH

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-16

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CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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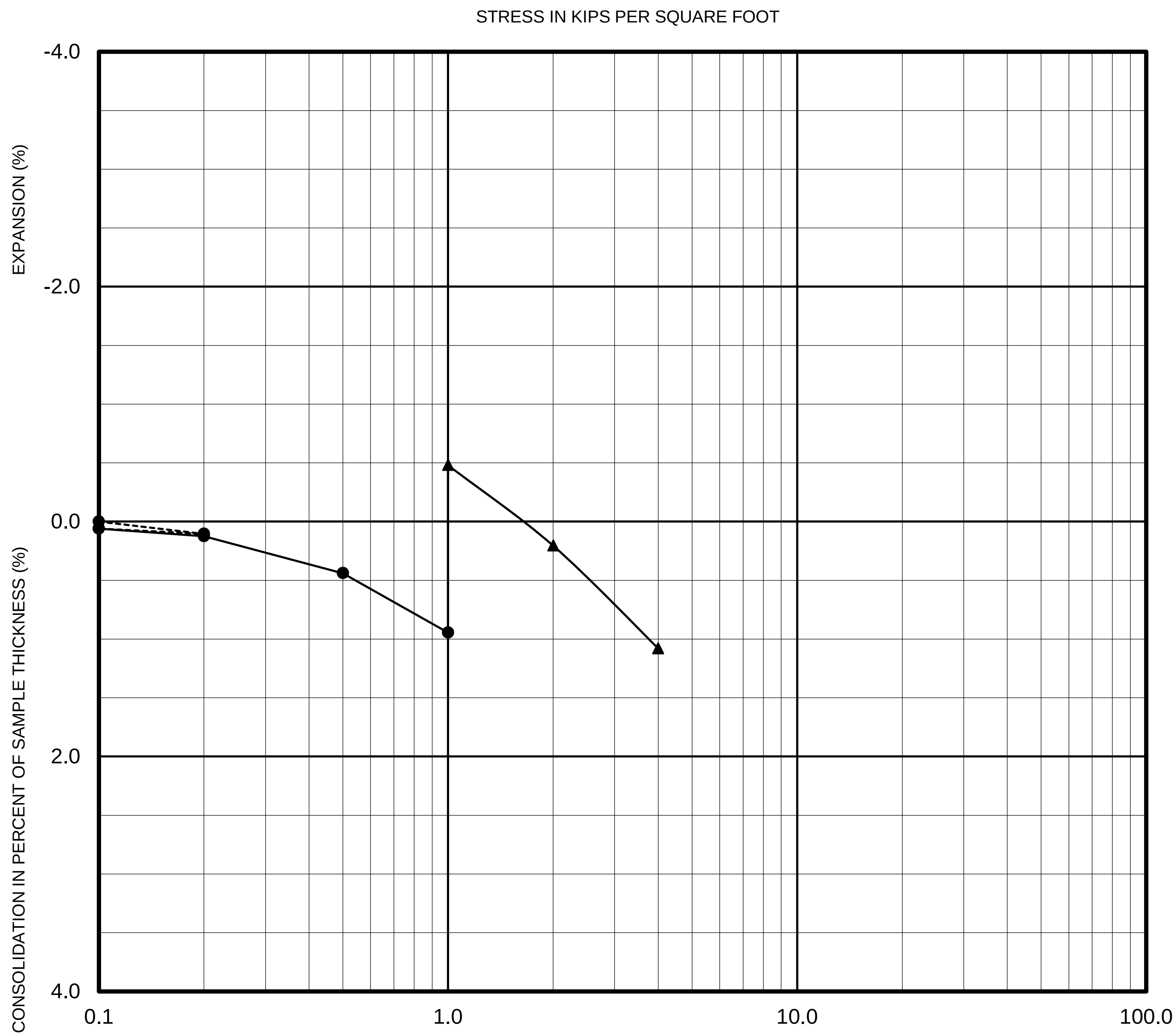

Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Moisture Content Before Test (%):	16.7%
Moisture Content After Test (%):	28.8%
Swell Percentage (%):	1.4
Swell Pressure (psf):	2600.0

Sample Location: B-6
Depth (ft): 9.0-10.0
Soil Type: SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-17

CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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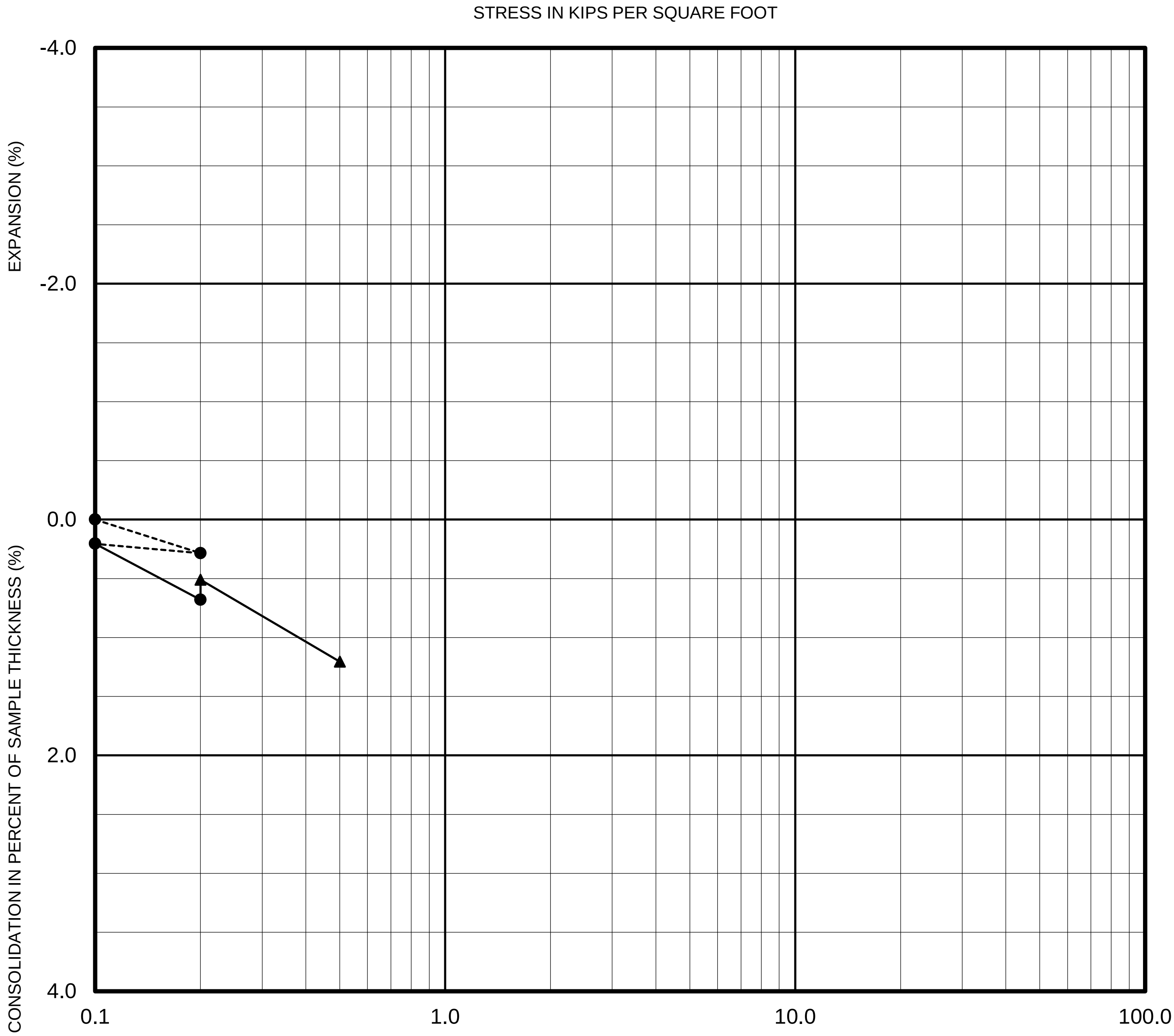

Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Moisture Content Before Test (%):	9.7%
Moisture Content After Test (%):	19.1%
Swell Percentage (%):	0.2
Swell Pressure (psf):	50.0

Sample Location: B-8
Depth (ft): 2.0-3.5
Soil Type: CL (fill)

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-18

CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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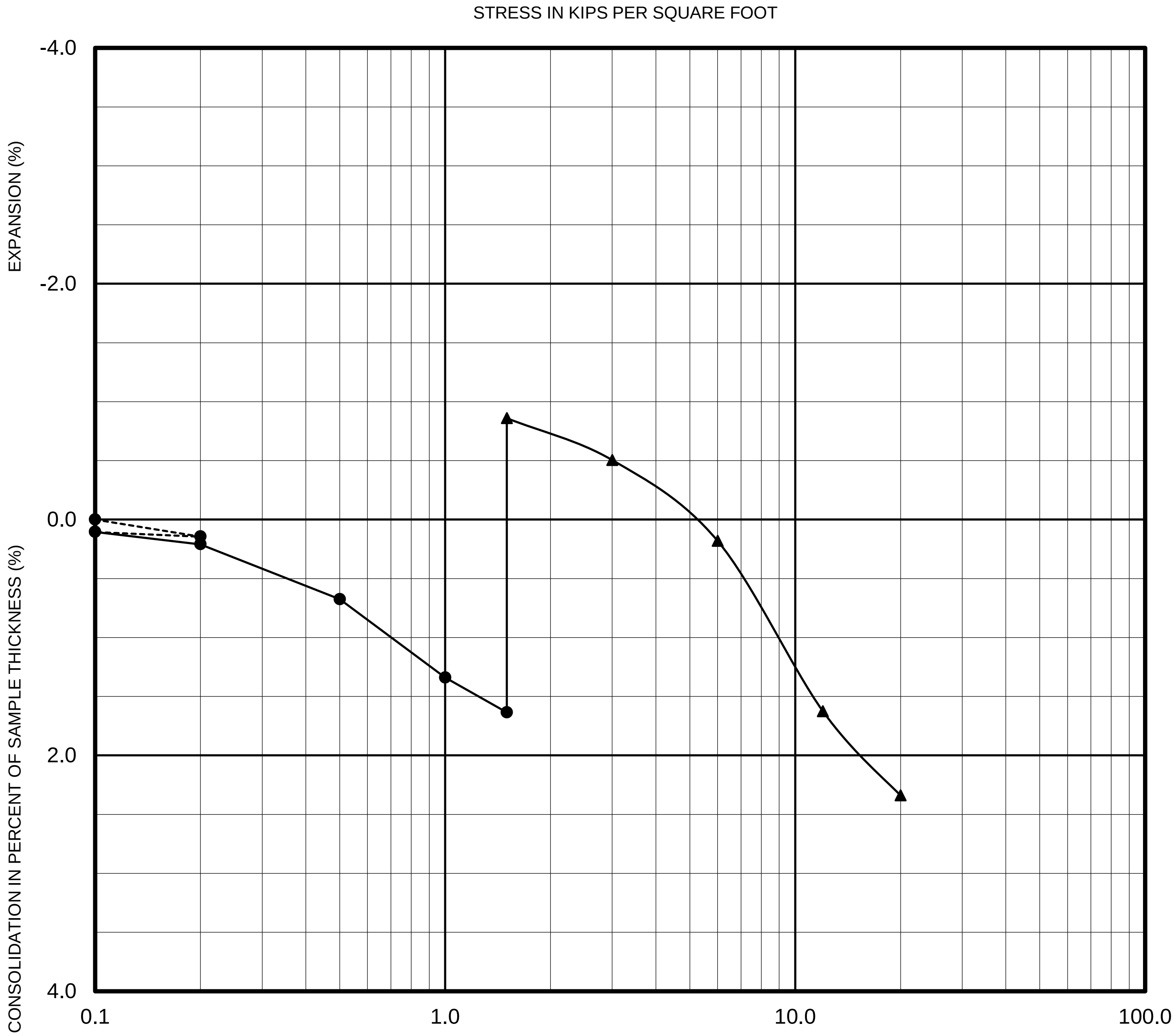

Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Moisture Content Before Test (%):	21.3%
Moisture Content After Test (%):	24.5%
Swell Percentage (%):	2.5
Swell Pressure (psf):	10400.0

Sample Location: B-9
Depth (ft): 14.0-14.9
Soil Type: DAWSON FORMATION

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-19

CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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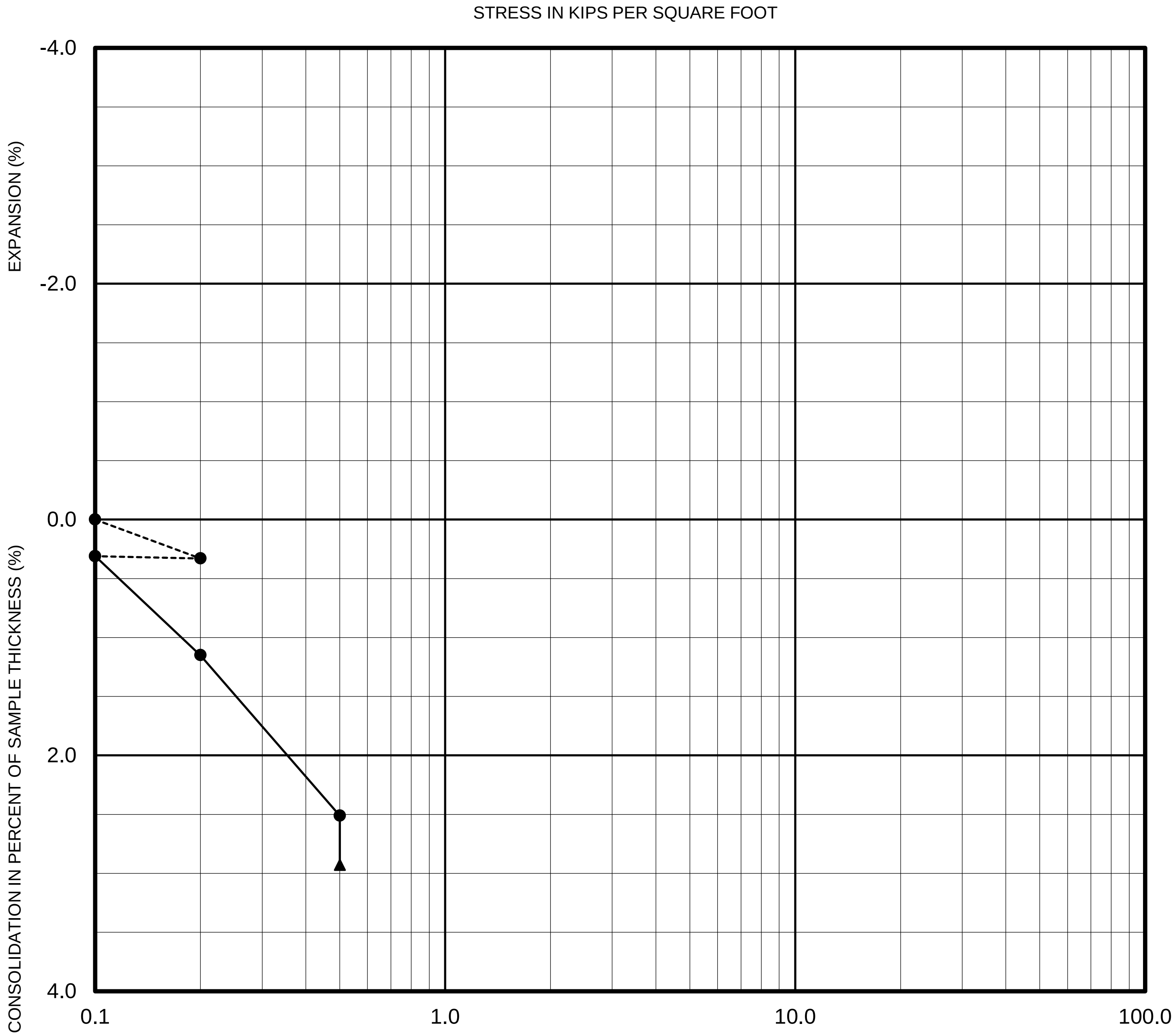

Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Moisture Content Before Test (%):	14.3%
Moisture Content After Test (%):	17.4%
Swell Percentage (%):	-0.4
Swell Pressure (psf):	-

Sample Location:	B-12
Depth (ft):	4.0-5.5
Soil Type:	CL (fill)

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-20

CONSOLIDATION TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [2] (ohm-cm)	SULFATE CONTENT [3] (ppm)	(%)	CHLORIDE CONTENT [4] (ppm)
B-1 to B-4, B-8, and B-10	0.0-5.0	5.8	3,125	33	0.003	20
B-5, B-6, B-9, and B-12	0.0-5.0	6.0	1,449	240	0.024	30

[1] PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4972
[2] PERFORMED IN GENERAL ACCORDANCE WITH AASHTO T288
[3] PERFORMED IN GENERAL ACCORDANCE WITH CDOT TEST METHOD CP-L 2103
[4] PERFORMED IN GENERAL ACCORDANCE WITH CDOT TEST METHOD CP-L 2104

FIGURE B-21

CORROSIVITY TEST RESULTS

PARKER AND PINE RETAIL DEVELOPMENT

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