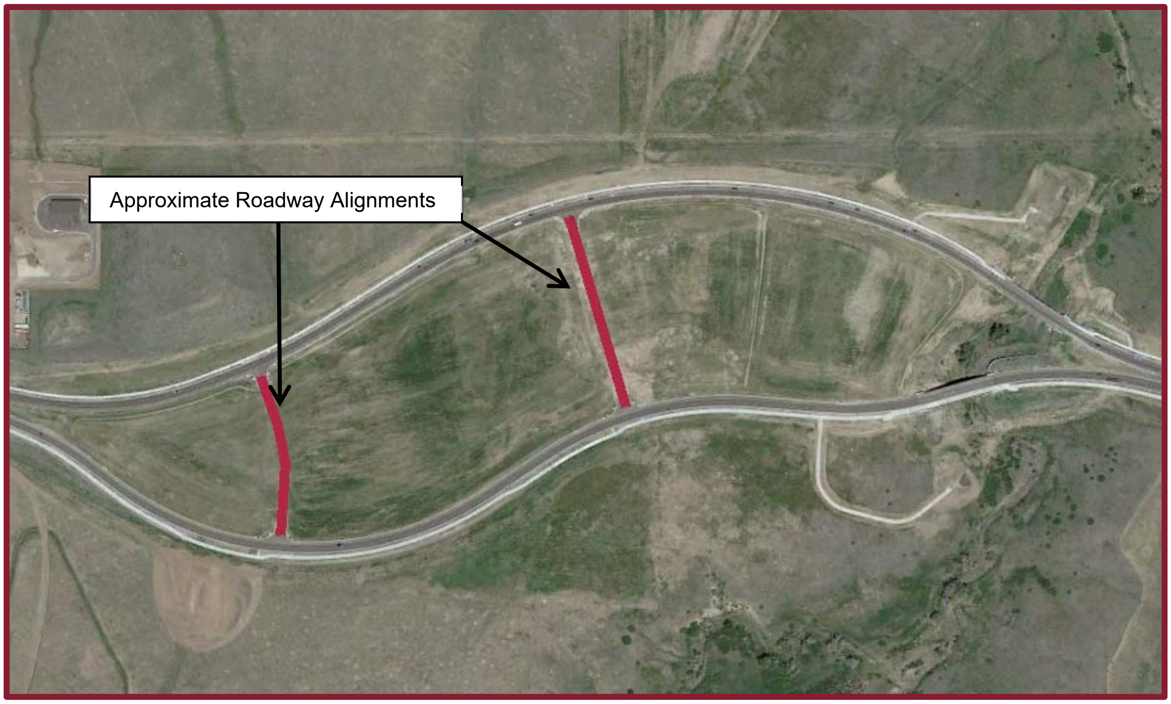
Geotechnical Evaluation
Ridgegate Parkway Couplet Roads
Pavement Sections
Lone Tree, Colorado
Revised



Prepared For:

Merrick & Company

5970 Greenwood Plaza Boulevard
Greenwood Village, Colorado 80111

Attention: Carson Besgrove

Job Number: 22-3023# **January 24, 2023**

TABLE OF CONTENTS

PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical evaluation performed by GROUND Engineering Consultants, Inc. (GROUND) for Merrick & Company in support of the proposed couplet roads to connect East- and West-bound Ridgegate Parkway that will be constructed approximately 4,000 feet east of the intersection of Ridgegate Parkway and South Peoria Street in Lone Tree, Colorado. Our study was conducted in general accordance with GROUND's Proposal No. 2203-0590 dated April 6, 2022.

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

This report has been prepared to summarize the data obtained and to present our findings and conclusions based on the proposed improvements and the subsurface conditions encountered. Design parameters and a discussion of engineering considerations related to the proposed improvements are included herein. This report should be understood and utilized in its entirety; specific sections of the text, drawings, graphs, tables, and other information contained within this report are intended to be understood in the context of the entire report. This includes the *Closure* section of the report which outlines important limitations on the information contained herein.

This report was prepared for design purposes of Merrick & Company based on our understanding of the proposed project at the time of preparation of this report. The data, conclusions, opinions, and geotechnical parameters provided herein should not be construed to be sufficient for other purposes, including the use by contractors, or any other parties for any reason not specifically related to the design of the project. Furthermore, the information provided in this report was based on the exploration and testing methods described below. Deviations between what was reported herein and the actual surface and/or subsurface conditions may exist, and in some cases those deviations may be significant.

PROPOSED CONSTRUCTION

Based on provided project documents[1] we understand couplet roads A1 and A2 are planned for construction to connect east- and west-bound Ridgegate Parkway. Couplet road A1 will have a length of approximately 650 feet and be located approximately 4,800 feet southeast of the intersection of South Peoria Street and Ridgegate Parkway. Couplet road A2 will have a length of approximately 500 feet and located approximately 4,000 feet southeast of the intersection of South Peoria Street and Ridgegate Parkway. Additionally, other anticipated improvements include a buried water line in the alignment of A2 and an expansion of a detention pond northwest of the intersection of Badger Gulch and Ridgegate Parkway.

It is GROUND's understanding that a flexible pavement section is preferred for these couplet roads. If rigid pavements are required, GROUND should be notified so that we may revise our report.

If our described understanding/interpretation of the proposed project is incorrect or project elements differ in any way from that expressed above, including changes to improvement locations, dimensions, orientations, loading conditions, elevations/grades, etc., and/or additional buildings/structures/site improvements are incorporated into this project, either after the original information was provided to us or after the date of this report, GROUND or another geotechnical engineer must be retained to re-evaluate the conclusions and parameters presented herein.

---

[1] Besgrove, Carson. *2202-0328 Couplet Typical Section Exhibit.pdf*. Email to GROUND Engineering Consultants, Inc.,dated March 29, 2022.

ALIGNMENT CONDITIONS

At the time of our subsurface exploration, the alignments of the couplet roads generally consisted of graded and grubbed parcels with intermittent short grasses and weeds. Topography of the greater alignment area was generally gently undulating while gently sloping to the west. Buried utilities were also present along the project alignments. The alignments crossed grubbed and graded unused land and extended from the westbound reach of Ridgegate Parkway to the eastbound reach of Ridgegate Parkway.

At the proposed site of the detention pond, vegetation consisted of short to medium grasses and weeds with sparse deciduous trees. Topographically, the site was generally sloping toward Badger Gulch. The slope was gentle from the east with a more pronounced slope from the west. A north-south aligned drainage pipe with a 45-foot by 30-foot rip-rap brake was separated from a concrete overflow outlet structure by a 100-foot by 10-foot rip-rap barrier. Standing water was visible at the time of drilling. Buried and above-ground utilities were also observed in the vicinity of the subject alignments.

Fill was encountered in several test holes during our exploration program, likely due to previous construction activities associated with the construction of Ridgegate Parkway including grading and utility installation. Review of aerial imagery available on Google Earth, as seen below, indicated that the site changed significantly during the construction

of east-bound Ridgegate Parkway from 2019 through 2020. Therefore, fill soils should be anticipated at varying depths along the alignments. The complete extents and compositions of fills along the alignments, however, were not determined as part of this scope of services.

SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted in May 2022, with a truck-mounted drill rig to evaluate the subsurface conditions as well as to retrieve soil and bedrock samples for laboratory testing and analysis. Six (6) test holes were drilled during this exploration. Two (2) test holes were drilled along the alignment of couplet A2, three (3) test holes were drilled along the alignment of couplet A1, and one (1) test hole was drilled within the proposed detention pond area. A GROUND engineer directed the subsurface exploration, logged the test holes in the field, and prepared the samples for transport to our laboratory.

Samples of the subsurface materials were taken with a 2-inch I.D. Modified California-type liner sampler and 1⅜-inch I.D. standard penetration sampler. The samplers were driven into the substrata with blows from a 140-pound hammer falling 30 inches. This procedure is similar to the Standard Penetration Test described by ASTM Method D1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of soils. Depths at which the samples were taken, and associated penetration resistance values are shown on the test hole logs.

GROUND utilized the site plan indicating existing features provided by Merrick & Company, Google Map imagery, and a hand-held GPS device to determine the locations of the test holes. The approximate locations of the test holes are shown in Figure 1. Summary logs are provided in Figure 2 and a legend and notes are provided in Figure 3. Detailed logs of the test holes are presented in Appendix A.

LABORATORY TESTING

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. As required by the City of Lone Tree, the following laboratory testing, as applicable, was performed:

- Gradation Analysis AASHTO T 27 / ASTM D422-63
- Moisture Density Curves AASHTO T 99 / ASTM D698
- Resilient Modulus AASHTO T-309
- R-Value AASHTO T-190
- Percent Passing No. 200 AASHTO T 11 / ASTM D1140
- Soil Classification AASHTO M 145 / ASTM D2487
- Atterberg Limits AASHTO T89 and T90
- Sulfate Tests AASHTO T 290
- Swell Tests ASTM D 4546

Results of the laboratory testing program are summarized in Tables 1 and 2. Gradation plots are provided in Figures 4 through 8.

SUBSURFACE CONDITIONS

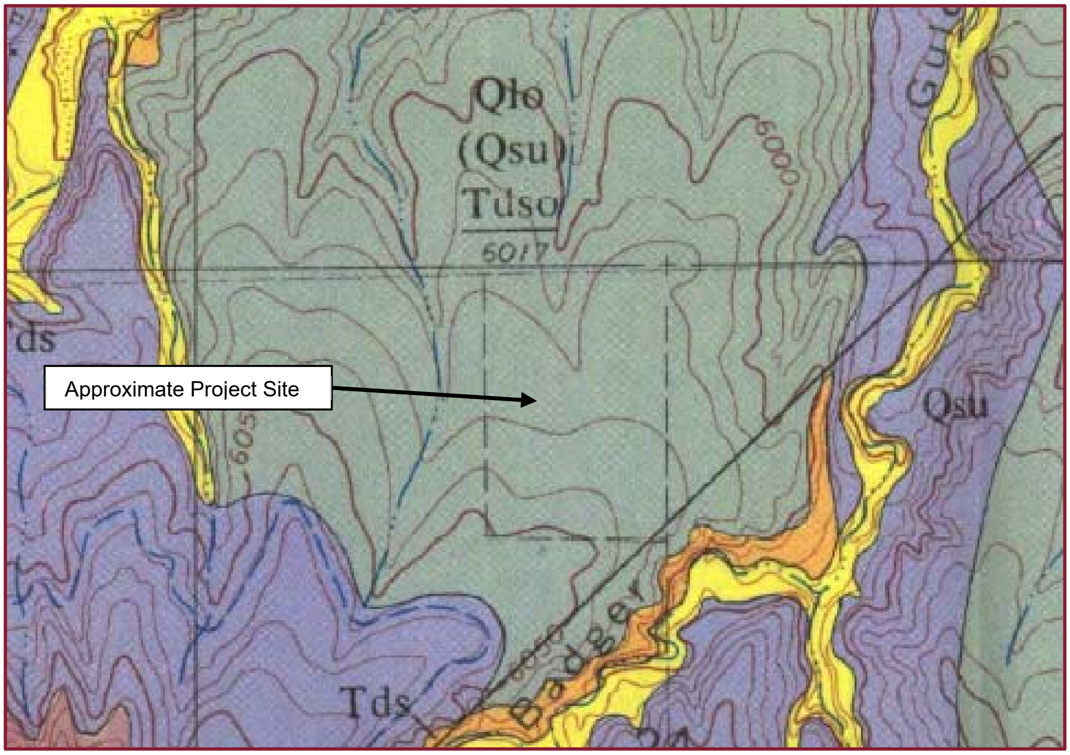
Geologic Setting Published geologic maps, e.g., Maberry (1972),[2] depict the site as underlain by the Pleistocene Louviers Alluvium (**Qlo**) and Pleistocene Slocum Alluvium (**Qsu**). These surficial deposits are mapped as being underlain by the Cretaceous to Tertiary Upper Dawson Arkose (**Tds**) with intertonguing Cretaceous to Tertiary Denver Formation (**Tde**). A portion of the that map is reproduced below.

In the project area, alluvial (stream-laid) deposits consist of sands and gravels with varying fractions of silts and clays. Cobbles and boulders are present locally as well. Some of the larger clasts present in alluvial deposits may not be appropriate for reuse in project fills.

---

[2] Maberry, J.O., and Lindvall, R.M., 1972, *Geologic Map of the Parker Quadrangle, Arapahoe and Douglas Counties, Colorado,* U.S. Geological Survey, Miscellaneous Geologic Investigations Map I-770-A, 1:24,000.

The Dawson Formation, in the project area, typically consists fine-grained, silty and clayey sandstone. The Denver Formation, in the project area, typically consists of claystone and siltstone. The siltstones and claystones can be moderately to highly expansive and the formations include well-cemented beds that can be very hard and difficult to excavate.



The subsurface conditions encountered in the A1 alignment test holes generally consisted of approximately 3 inches of topsoil[3] overlying fills soils that were recognized to depths of about 2½ and 3 feet below existing grade, or, in the case of Test Hole 3, native silts and clays that were recognized to a depth of about 7 feet below existing grade. Native sands were encountered beneath the fill and native silts and clays, and extended to depths of about 12 feet below existing grade in Test Holes 4 and 5 and the depths explored in Test Hole 3. Native silts and clays were encountered beneath the native sands, in Test Hole 4 and 5, and extended to the depths explored.

The subsurface conditions encountered in the A2 alignment test holes generally consisted of approximately 3 inches of topsoil overlying native clays and silts that extended to depths

[3] 'Topsoil' as used herein is defined geotechnically. The materials so described may or may not be suitable for landscaping or as a growth medium for plants that may be proposed for the project.

explored. Fill soils were recognized in Test Hole 1 and appeared to extend to about 6 feet below existing grade.

The subsurface conditions encountered in the detention pond test hole consisted of approximately 6 inches of topsoil overlying fill soils that were recognized to a depth of about 12 feet below existing grade. Native silts and clays were encountered beneath the fill and extended to the depths explored.

We interpret the sands and the silts and clays to be alluvial deposits. We interpret the sandstone to be Dawson Formation.

Fill soils were not recognized in all the test holes, but are present within both alignments and the detention pond. Delineation of the complete lateral and vertical extents of any fills at the site, and their composition, was beyond our present scope of services. If detailed soil compositions at the site are of significance, they should be evaluated using test pits.

The 2-inch diameter (or smaller) sampling apparatus inherently cannot sample undisturbed cobble and boulder materials due to their larger size. It should be understood that the samples obtained during drilling operations may not be representative of the larger sized earth materials that may be encountered during construction. Material sizes and descriptions are largely interpreted based on drilling advancement rates and other observations during the drilling operations. Additional exploration utilizing alternate methods, such as test pits, should be considered if more information is desired.

At this site, therefore, it should be anticipated that gravels and cobbles, and possibly boulders, may be present in the site soils, even where not included in the general descriptions of the site soil types below.

Fill generally consisted of fine to coarse sands, silts, and clays with local gravels. They were moderately plastic, medium dense or medium to very stiff, slightly moist to moist, and light brown to brown to light brown-gray in color. Caliche was noted locally.

Silts and Clays generally consisted of silts and clays with fine to coarse sands and local gravels. They were moderately to highly plastic, stiff to hard, dry to very moist, and light to dark brown to gray in color. Caliche and iron staining were noted locally.

Sands generally consisted of sands with gravels, silts, and clays. They were non- to slightly plastic, loose to medium dense, dry to slightly moist, and light brown to brown to light gray in color.

Weathered Sandstone generally consisted of weathered, fine grained, silty sandstones. It was moist, moderately plastic, medium hard, and gray in color. Iron staining was noted commonly.

Sandstone generally consisted of fine-grained silty sandstones. It was moderately plastic, hard, very moist, and gray brown in color. Iron staining was noted commonly.

Swell-Consolidation Testing indicated a potential for heave in the on-site materials. Swells ranging from approximately 1.7 to 10.2 percent were measured upon wetting against a surcharge load of approximating in-place overburden pressures. A consolidation of about 0.1 was measured in a sample of native soils against a surcharge load approximating the in-place overburden pressure. Swell-consolidation test results are presented on Table 1.

Groundwater was not encountered at the depths explored in the test holes at the time of drilling. The test holes were backfilled upon drilling completion per Code of Colorado Regulations (2 CCR 402-2). Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, nearby rivers and creeks, land use, and the development of transient, perched water conditions. The groundwater observations performed during our exploration must be interpreted carefully as they are short-term and do not constitute a groundwater study. In the event the Merrick & Company desires additional/repeated groundwater level observations, GROUND should be contacted to provide a cost estimate for this additional geotechnical evaluation.

PAVEMENT SECTIONS

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings. Per the City of Lone Tree, the standard practice in pavement design describes a typical flexible pavement section as a "20-year" design pavement for local roadways per Douglas County Roadway Design and Technical Criteria Manual (Section 5.3). However, most pavements will not remain in satisfactory condition without routine maintenance and rehabilitation procedures performed throughout the life of the pavement.

The pavement sections for couplets A1 and A2 were developed in general accordance with the applicable design guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO) and the Douglas County Roadway Design and Technical Criteria Manual updated August 2021 (City of Lone Tree specifications).

Subgrade Materials Based on the results of our field and laboratory studies, subgrade materials encountered in our test holes consisted predominantly of sandy silts and sandy clays. These materials were classified predominantly as A-7 with local A-2, A-4, and A-6 soils in accordance with the AASHTO classification system, with Group Index values up to 22 in the upper 5 feet.

GROUND collected two (2) composite bulk samples from the test hole auger returns. Resilient Modulus (M_R) testing (AASHTO T-307) was performed on composite samples of the subgrade materials encountered along the proposed alignments of couplets A1 and A2.

The material was compacted to approximately 95 percent of maximum dry density at approximately 3 and 3½ percent above the optimum, based on AASHTO T-99 (the "standard Proctor") for cohesive soils. The resilient modulus of a material at approximately 3 percent above optimum moisture content typically is often used for fine-grained soils that classify as A-4, A-6, or A-7.

According to our test results, a resilient modulus value of approximately 3,872 psi was indicated for the on-site soils within the proposed alignment of couplet A1 and a resilient modulus value of approximately 3,813 psi was indicated for the on-site soils within the proposed alignment of couplet A2.

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

Anticipated Traffic The City of Lone Tree requested that GROUND utilize an ADT of 7,550 vehicles for pavement section development. Couplets A1 and A2 are planned to be Residential Collector roadways and are assumed to service single-family residential, commercial, and business properties. We understand the roadways will have two traffic lanes.

An ESAL value of 2,045,913 was calculated based on a lane factor of 0.6 for a two-lane roadway and an ADT of 7,550 with a traffic breakdown of 2 percent combination trucks, 2 percent single unit trucks, and 96 percent passenger vehicles ("cars" and "pickups").

<u>The City and Merrick and Company should review the above values, based on their knowledge and understanding of the roadway and current/potential use characteristics. If traffic loadings differ significantly from these values above, the pavement sections provided below should be re-evaluated.</u>

Pavement Sections All paved surfaces (parking areas, alleys, roads, highways, etc.), whether public or private, must be designed in accordance with the City of Lone Tree specifications and approved by Lone Tree prior to construction. The soil resilient modulus value and the ESAL values were used to determine the required design structural number for the project pavement. The required structural numbers were then used to develop the pavement sections. Pavement designs were based on the DARWin™ computer program that solves the 1993 AASHTO pavement design equations. A Reliability Level of 90 percent (Table 5.4), a serviceability loss of 2.0 (Table 5.3), and an overall standard deviation of 0.44 (Table 5.4) were used. A structural coefficient of 0.44 was used for hot bituminous asphalt and 0.12 was used for aggregate base course (Table 5.6).

The following table indicates the minimum pavement section thickness developed by GROUND. They exceed the Douglas County specified minimum pavement sections.

Pavement design calculations are provided in Appendix B. As indicated in the Douglas County Roadway Design and Technical Criteria Manual (Table 5.5), a full depth asphalt pavement section is not allowed for roadways designated as Collectors servicing residential developments.

PAVEMENT SECTIONS

Subject Roadway	Minimum Composite Section (inches HMA / inches ABC)
Couplet A1	7 / 14*
Couplet A2	7 / 14*

*HMA = Hot-Mix Asphalt, ABC = Aggregate Base Course, * = Sections do not include swell mitigation.*

Pavement Materials

Hot-Mix Asphalt (HMA): The asphalt pavement shall consist of a bituminous plant mix composed of a mixture of high-quality aggregate and bituminous material. The asphalt cement selected for the proposed pavement section should conform to requirements outlined in the Douglas County Roadway Design and Technical Criteria Manual.

Aggregate Base Course (ABC): The aggregate base material should meet the criteria of MGPEC aggregate base course. Base course should be placed in uniform lifts not exceeding 8 inches in loose thickness and compacted to at least 95 percent of the maximum dry density a uniform moisture contents within 1 percent of the optimum as determined by *MGPEC – Volume 1 – Pavement Design Standards & Construction Specifications – Item 13*.

Pavement Subgrade Preparation Remedial earthwork to any depth will not prevent pavement distress on these soils, but will tend to reduce it and improve perceived rideability.

Remedial Earthwork Based on the Douglas County Roadway Design and Technical Criteria Manual (Section 5.4.3), subgrade materials with swell potentials greater than 2 percent can be remediated in through one of three different methods. Such methods must be extended one (1) foot beyond the back-of-curb (if detached walk or no walk), or one (1) foot beyond to the back-of-walk (if attached or monolithic walk).

Section 5.4.3.1.1: **Over excavation and replacement of the swelling soil with an A-2 to A-6 soil group with less than 2% swell**. The over excavation shall be a minimum of **three (3) feet below the bottom of the approved pavement section**. Greater depths of moisture-density treatment will further reduce the potential for movement. **Upon removal of the three feet of material, the existing surface shall be scarified and reconditioned to a depth of 8 inches**. The reconditioning shall be moisture treated to **within 2 percent of optimum moisture content (optimum to +4% for A-6 soils) and compacted.**

5.4.3.1.2 : Remove the swelling soil to a depth of **one (1) foot below the bottom of the pavement section, then replace the excavated materials with one (1) foot of Class 6 Road Base**. If the road base option is used, this may require the use of an approved geotextile fabric between the native material and the Class 6 Road Base. Upon removal of the one foot of material, the existing surface shall be scarified and reconditioned to a depth of 8 inches. The reconditioning shall be moisture treated to within 2 percent of optimum moisture content (optimum to +4% for A-6 and A-7-6 soils) and compacted.

5.4.3.1.3: Other methods of swell mitigation could include the **use of lime or Portland cement**. Methods of mitigation to be used are **subject to approval by the City of Lone Tree**. The submittal of an alternative for swell mitigation as described above should include the requirements associated with the scarification and reconditioning of the subgrade below the proposed mitigation treatment.

The potential for pavement distress as a result of both heave and settlement still exists after properly following the pavement subgrade preparation provided in this report and recommended by the City of Lone Tree. This section assumes that significant total and differential pavement post-construction movements (on the order of several inches) and the associated maintenance costs that are necessary to re-establish effective drainage, replace distressed pavement, etc. are acceptable.

Immediately prior to paving, the subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle. Areas that show excessive deflection during proof rolling should be excavated and replaced and/or stabilized. Areas allowed to pond prior to paving will require significant re-working prior to proof-rolling. All subgrade preparation must ultimately comply with roadway inspection, testing, and construction procedures outlined in the Douglas County Roadway Design and Technical Criteria Manual.

Pavement subgrade materials should be compacted in accordance with the *Project Earthwork* section of this report. Subgrade preparation should extend the full width of the pavement from back-of-curb to back-of-curb and also extend under the adjacent sidewalks, exterior flatwork, etc.

Drainage The collection and diversion of surface drainage away from paved areas is extremely important to satisfactory performance of the pavements. The subsurface and surface drainage systems should be carefully designed to ensure removal of the water from paved areas and subgrade soils. Allowing surface waters to pond on pavements will cause premature pavement deterioration. <u>Roadway trench drains are required for composite sections per Douglas County Roadway Design and Technical Criteria Manual, 5.3.11</u>.

Additional Considerations GROUND's experience indicates that longitudinal cracking is common in asphalt-pavements generally parallel to the interface between the asphalt and concrete structures such as curbs, gutters or drain pans. Distress of this type is likely to occur even where the subgrade has been prepared properly and the asphalt has been compacted properly.

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

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

FROST HEAVE

Based on the results of the field exploration as well as the laboratory testing, it appears that clayey and silty soils requiring special design considerations for the purpose of addressing frost heave are present at the project. According to the US Army Corps of Engineers manual (1965), the soils on-site classify as F3 and F4 materials. Therefore, even if surface drainage is effective, the likelihood of movement of pavements, flatwork, and other hardscaping as a result of frost heave is relatively low to high, per the US Army Corps manual. Often times where frost heave is a concern, replacement of the subgrade soils with 3 or more feet granular material would be performed. However, due to the depth of the recently placed underground utilities, we understand that this may not be feasible. In GROUND's opinion, effective, positive surface drainage and routine maintenance operations to seal any cracks that will allow moisture to infiltrate the soils may reduce the potential for frost heave.

DETENTION PONDS

We understand that detention pond in planned as part of the project. Detention ponds can become locations of enhanced and concentrated infiltration into the subsurface, leading to wetting of foundation soils in the vicinity, with consequent increased heave or settlement. Therefore, it is GROUND's opinion that any detention pond should be provided with an effective, low permeability liner if it is not clearly down-gradient from all structures/improvements that are not tolerant of greater than anticipated post-construction movements. Either a clay liner or a synthetic (plastic membrane) liner may be used, but the selected liner should be able to tolerate several inches of differential settlement.

A clay liner should be at least 12 inches in thickness and exhibit an as-placed hydraulic conductivity of 10^{-6} cm/s or lower. In our experience, local claystone-derived fills compacted to 95 percent or more relative compaction at moisture contents above the optimum moisture content typically meet that criterion. The local clays reworked as compacted fill may meet that criterion. We suggest that a test section using the proposed liner soil be constructed so that the as-placed hydraulic conductivity may be evaluated.

A synthetic liner should be placed and lapped in accordance with the manufacturer's specifications.

Either type of liner should be protected by a cover layer of common fill at least **18 inches** in thickness.

WATER-SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in selected samples of the site soils ranged were approximately 0.01 and 0.03 percent by weight. (See Table 2.) Such concentrations of soluble sulfates represent a **negligible** environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of 'negligible,' 'moderate,' 'severe' and 'very severe' as described in the "Design and Control of Concrete Mixtures," published by the Portland Cement Association (PCA). The Colorado Department of Transportation (CDOT) utilizes a corresponding scale with 4 classes of severity of sulfate exposure (Class 0 to Class 3) as described in the published table below.

REQUIREMENTS TO PROTECT AGAINST DAMAGE TO CONCRETE BY SULFATE ATTACK FROM EXTERNAL SOURCES OF SULFATE

Severity of Sulfate Exposure	Water-Soluble Sulfate (SO_4) In Dry Soil *(%)*	Sulfate (SO_4) In Water *(ppm)*	Water Cementitious Ratio *(maximum)*	Cementitious Material Requirements
Class 0	0.00 to 0.10	0 to 150	0.45	Class 0
Class 1	0.11 to 0.20	151 to 1500	0.45	Class 1
Class 2	0.21 to 2.00	1501 to 10,000	0.45	Class 2
Class 3	2.01 or greater	10,001 or greater	0.40	Class 3

Based on our test results and PCA and CDOT guidelines, cement conforming to one of the following Class 0 requirements should be used in all concrete exposed to site soils and bedrock:

Class 0 (Negligible)

1) ASTM C150 Type I, II, III, or V.

2) ASTM C595 Type IL, IP, IP(MS), IP(HS), or IT.

PROJECT EARTHWORK

The earthwork criteria below are based on our interpretation of the geotechnical conditions encountered in the test holes. Where these criteria differ from applicable municipal specifications, e.g., for trench backfill compaction along a public utility line, the latter should be considered to take precedence.

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

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

Topsoil present on-site should not be incorporated into ordinary fills. Instead, topsoil should be stockpiled during initial grading operations for placement in areas to be landscaped or for other approved uses.

Existing Fill Soils Fill materials were recognized in some of the test holes during subsurface exploration and are likely are present elsewhere on the site, given it's the apparent prior grading. (See the *Site Conditions* section of this report.) Because not all the fill soils were sampled and tested, it is possible that some of the fill soils may not be suitable for re-use as compacted fill, due to the presence of deleterious materials such as trash, organic material, coarse cobbles and boulders, or construction debris, even though these materials were not recognized in the test holes. Therefore, excavated fill materials should be evaluated and tested, as appropriate, with regard to re-use.

Use of Existing Native Soils Based on the samples retrieved from the test holes, native soils that are free of organic material are suitable, in general, for placement as compacted fill.

Fragments of rock and cobbles larger than **3 inches** in maximum dimension will require special handling and/or placement to be incorporated into project fills. In general, such materials should be placed as deeply as possible in the project fills. A geotechnical engineer should be consulted regarding appropriate parameters for usage of such materials on a case-by-case basis when such materials have been identified during earthwork. Standard parameters that likely will be generally applicable can be found in Section 203 of the current CDOT Standard Specifications for Road and Bridge Construction.

Imported Fill Materials If it is necessary to import material to the site, the imported soils should be free of organic material, and other deleterious materials. **Imported material should consist of soils that have 45 percent or less passing the No. 200 Sieve and should have a plasticity index of 10 or less.** Representative samples of the materials proposed for import should be tested and approved prior to transport to the site.

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

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

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

Soils placed as fill in the roadway alignments should be compacted at the minimum densities and moisture content ranges as provided in Table 8.1 from the Douglas County Roadway Design and Technical Criteria Manual.

Soils that classify as **A-1, A-2-5, A-2-7, and A-3 through A-5** in accordance with the AASHTO classification system (granular materials) should be compacted to **95 or more percent of the maximum modified Proctor dry density at moisture contents within 2 percent of optimum moisture content** as determined by AASHTO T180.

Soils that classify as **A-2-4 and A-2-6** should be compacted to **95 percent of the maximum standard Proctor density at moisture contents within 2 percent of optimum moisture content** as determined by AASHTO T99. Soils that classify as **A-6 and A-7** should be compacted to **95 percent of the maximum standard Proctor density at moisture contents from the optimum moisture content to 4 percent above the optimum moisture content** as determined by AASHTO T99.

Additionally, **moisture treatment for swell mitigation** should comply with the moisture treatment requirements outlined in Chapter 5.4.3.1 of the Douglas County Roadway Design and Technical Criteria Manual. Mitigation is required for soils with a swell potential ≥2.0 percent under 200 psf surcharge pressures at 95 percent standard compaction from a swell test run on undisturbed samples in accordance with ASTM D 4546.

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

Care should be taken with regard to achieving and maintaining proper moisture contents during placement and compaction. Materials that are not properly moisture conditioned may exhibit significant pumping, rutting, and deflection at moisture contents near optimum and above. The contractor should be prepared to handle soils of this type, including the use of chemical stabilization, if necessary.

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

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

Excavated bedrock materials, including those present in the existing fill, will require a well-coordinated effort to moisture treat, process, place, and compact properly. In-place bedrock deposits were hard to very hard, and should be broken down in to a soil-like mass. Greater than typical watering, and compaction equipment that aids in breaking down such material (e.g., a Caterpillar 825 compactor-roller), likely will be needed. Crushing or other methods should be anticipated to sufficiently reduce sandstone bedrock, where encountered. Applied water will be taken up into the structures of the claystone. The contractor should anticipate that <u>handling and processing the excavated bedrock more than once</u> may be necessary to achieve the requirements herein.

Excavated bedrock, include those present in the existing fill, to be used as trench backfill, will require additional moisture conditioning and processing in an open area outside of trenches prior to placement as backfill.

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

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

Care should be taken with regard to achieving and maintaining proper moisture contents during placement and compaction. Materials that are not properly moisture conditioned may exhibit significant pumping, rutting, and deflection at moisture contents near optimum and above. The contractor should be prepared to handle soils of this type, including the use of chemical stabilization, if necessary.

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

Compaction Criteria Soils that classify as **GP, GW, GM, GC, SP, SW, SM, or SC** in accordance with the USCS classification system should be compacted to at least **95 or percent** of the maximum dry density at moisture contents **within 2 percent** of the optimum moisture content as determined by ASTM D1557, the ‘modified Proctor.’

Soils that classify as **ML, MH, CL, or CH** should be compacted to at least **95 percent** of the maximum dry density at moisture contents **between 1 percent below and 3 percent** above the optimum moisture content as determined by ASTM D698, the 'standard Proctor.'

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

Cut and Filled Slopes Permanent (final grading), unretained, graded slopes supported by local soils up to **10 feet** in height should be constructed no steeper than **3 : 1** (horizontal : vertical). Minor raveling or surficial sloughing should be anticipated on slopes cut at this angle until vegetation is well re-established. Surface drainage should be designed to direct water away from slope faces into designed drainage pathways or structures.

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

EXCAVATION CONSIDERATIONS

Excavation Difficulty Test holes for the subsurface exploration were advanced to the depths indicated on the test hole logs by means of conventional, truck-mounted, geotechnical drilling equipment. However, well cemented lenses and beds of bedrock that are harder than those encountered in the test holes could be encountered locally. The contractor and project team should anticipate that some of the site bedrock will be very hard and require greater than typical efforts to excavate and process.

Additionally, given the inherent nature of undocumented fill soils, materials that may be awkward or otherwise difficult to handle (e.g., relatively large pieces of construction debris) may be encountered the undocumented fill soils. The contractor and the project team should be prepared to handle such materials.

Temporary Excavations and Personnel Safety Excavations in which personnel will be working must comply with all applicable OSHA Standards and Regulations, particularly CFR 29 Part 1926, OSHA Standards-Excavations, adopted March 5, 1990. The contractor's "responsible person" should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. GROUND has provided the information in this report solely as a service to Tectonic Management Group, Inc., and is not assuming responsibility for construction site safety or the contractor's activities.

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

Temporary, un-shored excavation slopes up to **15 feet** in height, in general, should be cut no steeper than **2 : 1** (horizontal : vertical) in the on-site soils <u>in the absence of seepage</u>. Some surface sloughing may occur on the slope faces at these angles. Should site constraints prohibit the use of the above-indicated slope angle, temporary shoring should be used. GROUND is available to provide shoring design upon request. Stockpiling of materials should not be permitted closer to the tops of temporary slopes than 5 feet or a distance equal to the depth of the excavation, whichever is greater. Additionally, shallow granular soils should be cleared back from the tops of slopes.

Groundwater Groundwater was not observed in the test holes. Therefore, we anticipate that shallow project excavation will be unlikely to encounter shallow groundwater except for limited volumes of perched groundwater. However, deeper excavations could encounter groundwater.

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

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

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

CLOSURE

Geotechnical Review The author of this report or a GROUND principal should be retained to review project plans and specifications to evaluate whether they comply with the intent of the measures discussed in this report. The review should be requested in writing.

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

Materials Testing Merrick & Company or the City of Lone Tree should consider retaining a geotechnical engineer to perform materials testing during construction. The performance of such testing or lack thereof, however, in no way alleviates the burden of the contractor or subcontractor from constructing in a manner that conforms to applicable project documents and industry standards. The contractor or pertinent subcontractor is ultimately responsible for managing the quality of his work; furthermore, testing by the geotechnical engineer does not preclude the contractor from obtaining or providing whatever services that he deems necessary to complete the project in accordance with applicable documents.

Limitations This report has been prepared for Merrick & Company as it pertains to design and construction of the proposed couplet roads and related improvements as described herein. It may not contain sufficient information for other parties or other purposes.

In addition, GROUND has assumed that project construction will commence by summer 2023. Any changes in project plans or schedule should be brought to the attention of a geotechnical engineer, in order that the geotechnical conclusions in this report may be re-evaluated and, as necessary, modified. If our described understanding/interpretation of the proposed project is incorrect or project elements differ in any way from that expressed herein, including changes to improvement locations, dimensions, orientations, loading conditions, elevations/grades, etc., and/or additional buildings/structures/site improvements are incorporated into this project, either after the original information was provided to us or after the date of this report, GROUND or another geotechnical engineer must be retained to re-evaluate the conclusions and parameters presented herein.

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

If during construction, surface, soil, bedrock, or groundwater conditions appear to be at variance with those described herein, a geotechnical engineer should be retained at once, so that re-evaluation of the conclusions for this site may be made in a timely manner. In addition, a contractor who obtains information from this report for development of his scope of work or cost estimates may find the geotechnical information in this report to be inadequate for his purposes or find the geotechnical conditions described herein to be at variance with his experience in the greater project area. The contractor is responsible for obtaining the additional geotechnical information that is necessary to develop his workscope and cost estimates with sufficient precision. This includes current depths to groundwater, etc.

ALL DEVELOPMENT CONTAINS INHERENT RISKS. It is important that ALL aspects of this report, as well as the estimated performance (and limitations with any such estimations) of proposed improvements are understood by Merrick & Company. Utilizing these criteria and measures herein for planning, design, and/or construction constitutes understanding and acceptance of the conclusions with regard to risk and other information provided herein, associated improvement performance, as well as the limitations inherent within such estimates.

Ensuring correct interpretation of the contents of this report by others is not the responsibility of GROUND. If any information referred to herein is not well understood, then Merrick & Company, or other members of the design team, should contact the author or a GROUND principal immediately. We will be available to meet to discuss the risks and remedial approaches presented in this report, as well as other potential approaches, upon request.

GROUND makes no warranties, either expressed or implied, as to the professional data, opinions or conclusions contained herein. This document, together with the concepts and conclusions presented herein, as an instrument of service, is intended only for the specific purpose and client for which it was prepared. Re-use of, or improper reliance on this document without written authorization and adaption by GROUND Engineering Consultants, Inc., shall be without liability to GROUND Engineering Consultants, Inc.

GROUND appreciates the opportunity to complete this portion of the project and welcomes the opportunity to provide Merrick & Company or the City of Lone Tree with a proposal for construction observation and materials testing.

Sincerely,

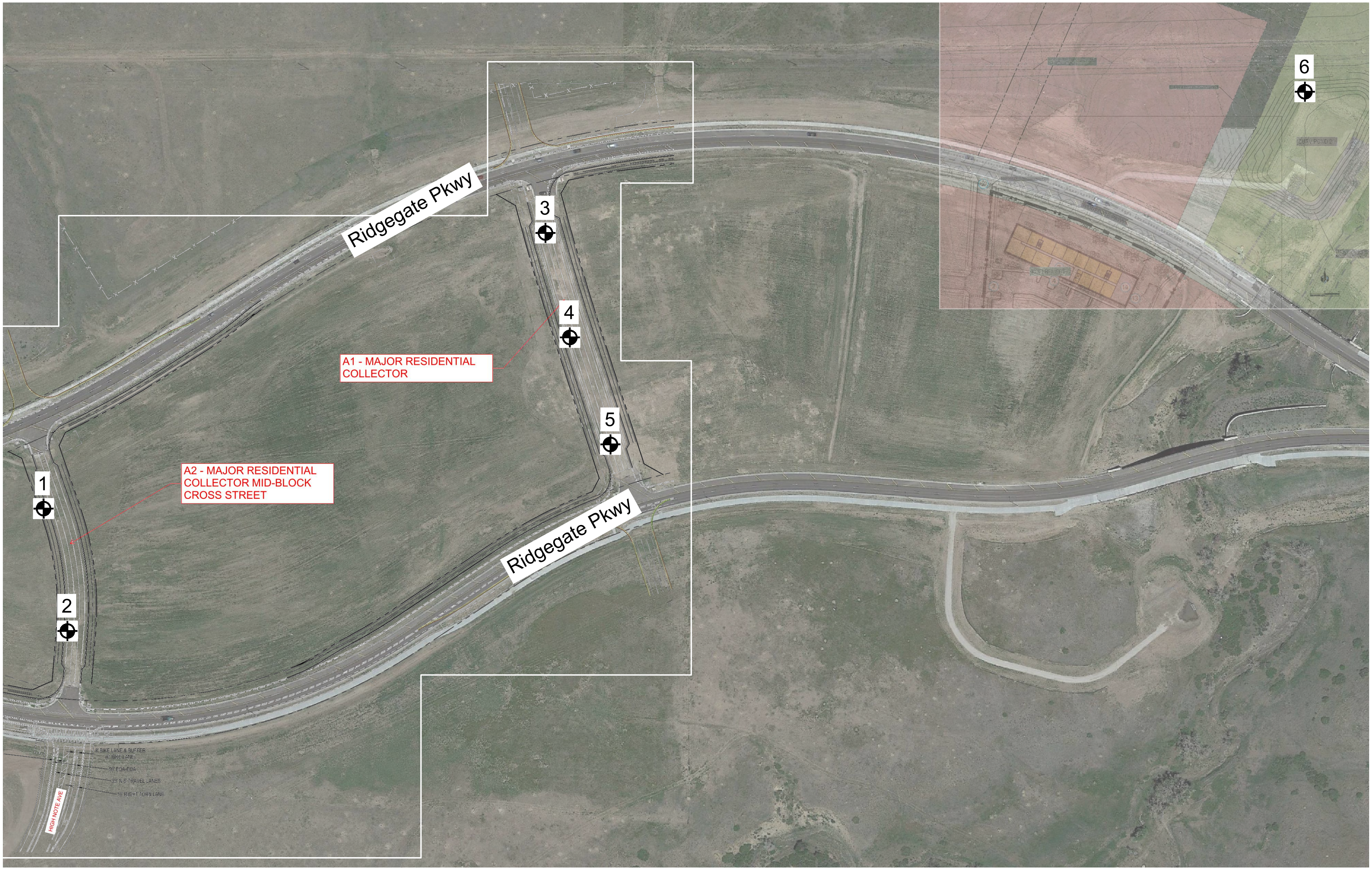
GROUND Engineering Consultants, Inc.

COLORADO LICENSED
BRIAN H. RECK
48265
PROFESSIONAL ENGINEER

1/24/2023

Ben Fellbaum, P.G., E.I.

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



1
Indicates test hole number and approximate location.

NOT TO SCALE

GOOGLE EARTH AERIAL IMAGE (6/10/2021)
SITE PLAN PROVIDED BY CLIENT

GROUND ENGINEERING	JOB NO.: 22-3023
	FIGURE: 1

LOCATION OF TEST HOLES

ENGINEERING

LOGS OF THE TEST HOLES

CLIENT: Merrick & Company

JOB NO: 22-3023

PROJECT NAME: Ridgegate Parkway Couplet Roads

PROJECT LOCATION: Lone Tree, CO

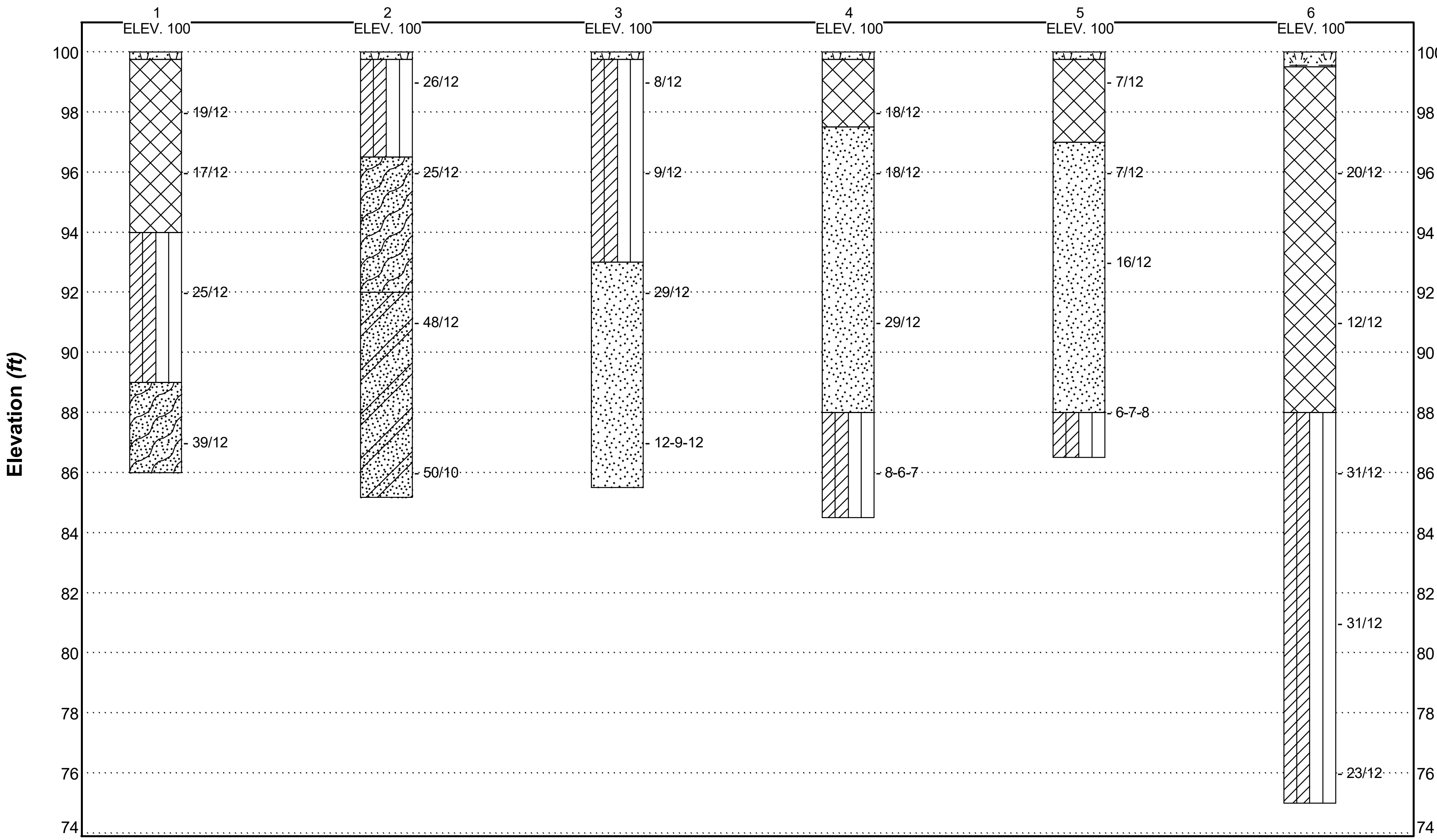


Figure 2

GROUND ENGINEERING

LEGEND AND NOTES

CLIENT: Merrick & Company

PROJECT NAME: Ridgegate Parkway Couplet Roads

JOB NO: 22-3023

PROJECT LOCATION: Lone Tree, CO

MATERIAL SYMBOLS

TOPSOIL

FILL

SAND

SILT and CLAY

WEATHERED SANDSTONE

SANDSTONE BEDROCK

NOTE: See Detailed Logs for Material descriptions.

SAMPLER SYMBOLS

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

Standard Penetration Test Sampler
20-25-30 Drive sample blow count, indicates 20, 25, and 30 blows of a 140 pound hammer falling 30 inches were required to drive the sampler 18 inches in three 6 inch increments.

NOTES

1. Test holes were drilled on 5/10 and 5/23/2022 with 4" solid stem auger.

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

3. Elevations of the test holes were not measured and the logs of the test holes are drawn to depth. Nominal elevation of "100 feet" indicates existing ground level at the test hole at the time of drilling.

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

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

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

7. The material descriptions on these logs are for general classification purposes only. See full text of this report for descriptions of the site materials & related information.

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

ABBREVIATIONS

▽ Water Level at Time of Drilling, or as Shown

▼ Water Level at End of Drilling, or as Shown

▼ Water Level After 24 Hours, or as Shown

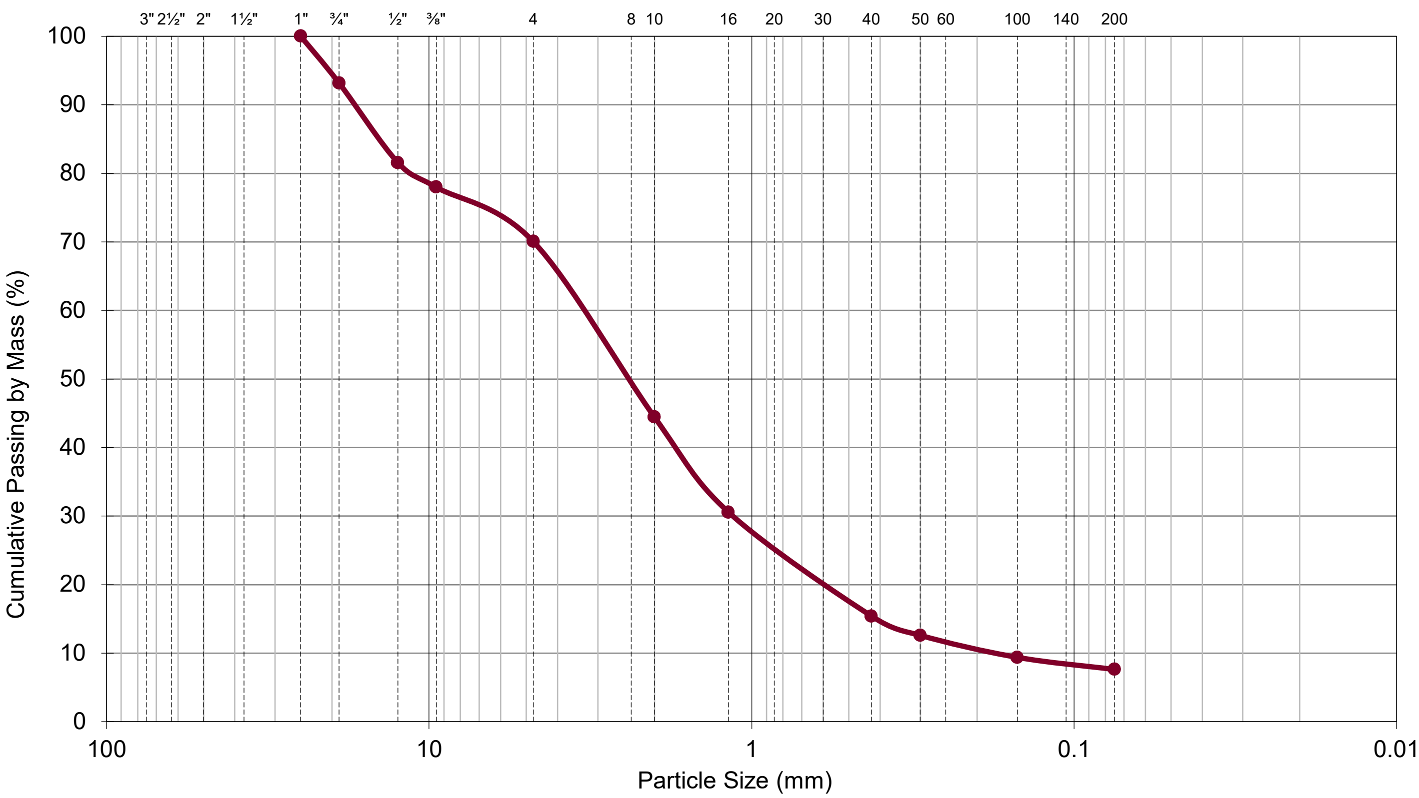
NV	No Value
NP	Non-Plastic

Figure 3

Project No.: 22-3023

Ridgegate Parkway Couplet Roads

Gradation (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	70	D90	16.954
5 in	125	-	No. 8	2.36	-	D85	14.158
4 in	100	-	No. 10	2.00	44	D80	11.097
3 in	75	-	No. 16	1.18	31	D60	3.382
2.5 in	63	-	No. 20	0.85	-	D50	2.413
2 in	50	-	No. 30	0.60	-	D40	1.689
1.5 in	37.5	-	No. 40	0.425	15	D30	1.136
1 in	25.0	100	No. 50	0.300	13	D15	0.405
3/4 in	19.0	93	No. 60	0.250	-	D10	0.172
1/2 in	12.5	82	No. 100	0.150	9	D05	-
3/8 in	9.5	78	No. 140	0.106	-	Cu	19.685
No. 4	4.75	70	No. 200	0.075	7.6	Cc	2.220

Location: 3 at 8 feet
Description: SAND with Gravel

Classification: (SP-SM)g / A-1-a (0)
Liquid Limit: NV
Plasticity Index: NP

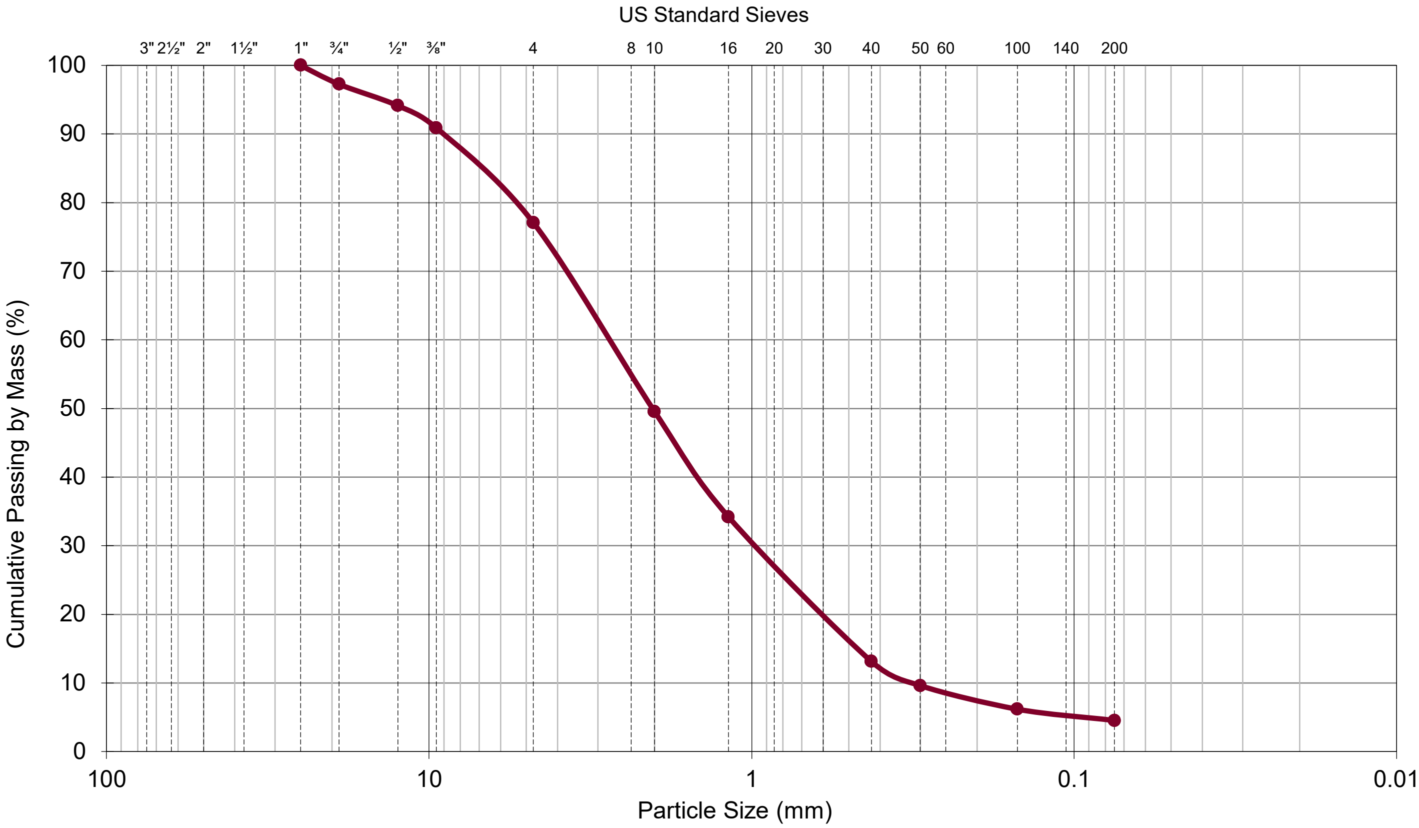
Gravel (%): 30
Sand (%): 62
Silt/Clay (%): 8

Figure 4

Project No.: 22-3023

Ridgegate Parkway Couplet Roads

Gradation (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	77	D90	9.087
5 in	125	-	No. 8	2.36	-	D85	7.072
4 in	100	-	No. 10	2.00	50	D80	5.504
3 in	75	-	No. 16	1.18	34	D60	2.778
2.5 in	63	-	No. 20	0.85	-	D50	2.029
2 in	50	-	No. 30	0.60	-	D40	1.440
1.5 in	37.5	-	No. 40	0.425	13	D30	0.962
1 in	25.0	100	No. 50	0.300	10	D15	0.465
3/4 in	19.0	97	No. 60	0.250	-	D10	0.312
1/2 in	12.5	94	No. 100	0.150	6	D05	0.091
3/8 in	9.5	91	No. 140	0.106	-	Cu	8.908
No. 4	4.75	77	No. 200	0.075	4.5	Cc	1.069

Location: 3 at 13 feet
Description: SAND with Gravel

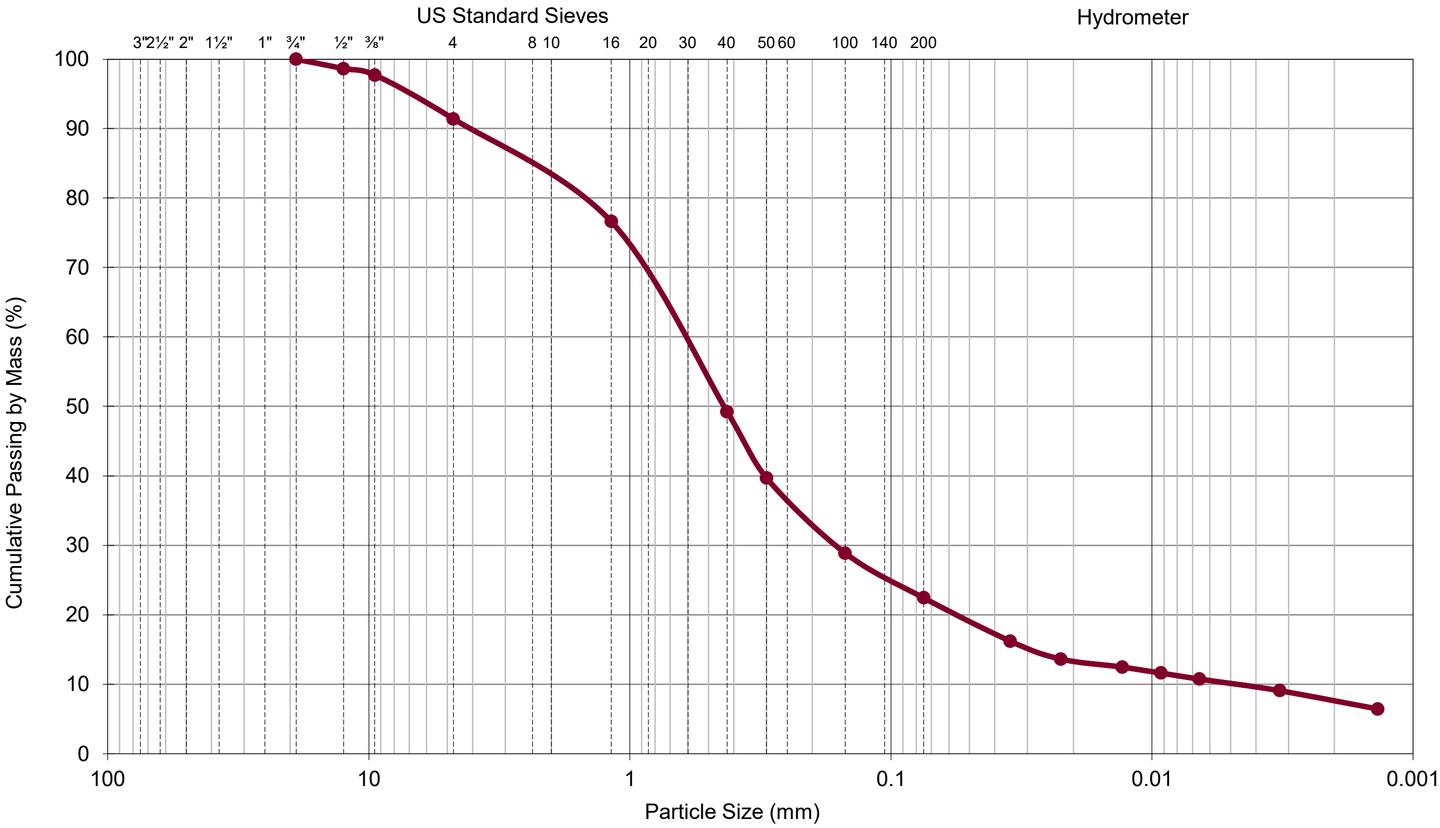
Classification: (SP)g / A-1-a (0)
Liquid Limit: NV
Plasticity Index: NP

Gravel (%): 23
Sand (%): 72
Silt/Clay (%): 5

Project No.: 22-3023

Ridgegate Parkway Couplet Roads

Gradation and Hydrometer (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	91	0.035	16	D90	4.161
5 in	125	-	No. 8	2.36	-	0.022	14	D85	2.597
4 in	100	-	No. 10	2.00	-	0.013	12	D80	1.621
3 in	75	-	No. 16	1.18	77	0.009	12	D60	0.634
2.5 in	63	-	No. 20	0.85	-	0.007	11	D50	0.437
2 in	50	-	No. 30	0.60	-	0.003	9	D40	0.303
1.5 in	37.5	-	No. 40	0.425	49	0.001	6	D30	0.161
1 in	25.0	-	No. 50	0.300	40	-	-	D15	-
3/4 in	19.0	100	No. 60	0.250	-	-	-	D10	-
1/2 in	12.5	99	No. 100	0.150	29	-	-	D05	-
3/8 in	9.5	98	No. 140	0.106	-	-	-	Cu	-
No. 4	4.75	91	No. 200	0.075	22.5	-	-	Cc	-

Location: 4 at 4 feet
Description: Clayey SAND

Classification: SC / A-2-4 (0)
Liquid Limit: 29
Plasticity Index: 10
Activity: 1.3

Gravel (%): 9
Sand (%): 69
Silt/Clay (%): 22
< .002 mm (%): 8

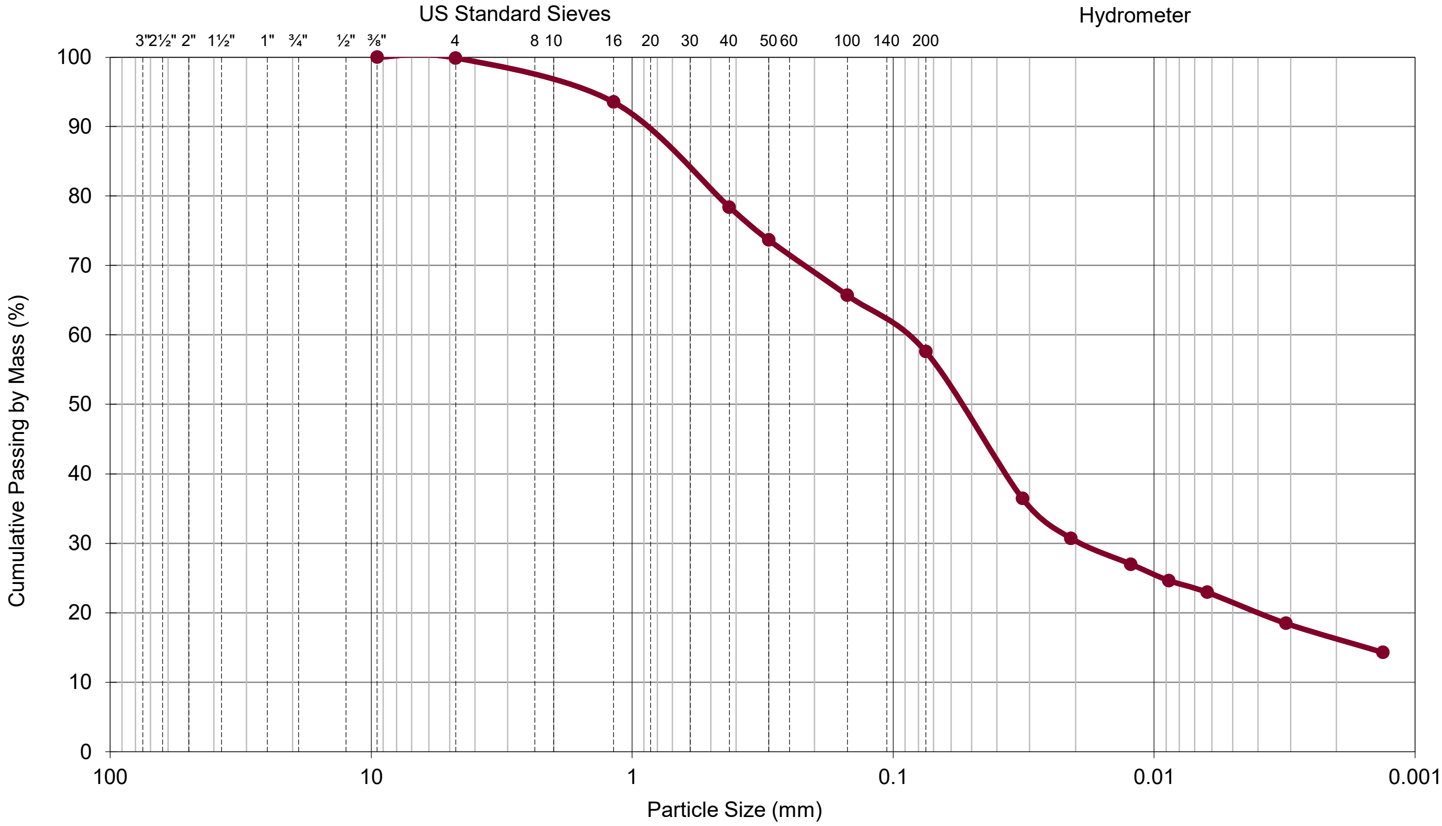
Results apply only to the specific items and locations referenced and at the time of testing. For the hydrometer portion of the test, a composite temperature correction and meniscus correction were applied to each reading. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

Figure 6

Project No.: 22-3023

Ridgegate Parkway Couplet Roads

Gradation and Hydrometer (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	100	0.032	36	D90	0.929
5 in	125	-	No. 8	2.36	-	0.021	31	D85	0.663
4 in	100	-	No. 10	2.00	-	0.012	27	D80	0.473
3 in	75	-	No. 16	1.18	94	0.009	25	D60	0.092
2.5 in	63	-	No. 20	0.85	-	0.006	23	D50	-
2 in	50	-	No. 30	0.60	-	0.003	19	D40	-
1.5 in	37.5	-	No. 40	0.425	78	0.001	14	D30	-
1 in	25.0	-	No. 50	0.300	74	-	-	D15	-
3/4 in	19.0	-	No. 60	0.250	-	-	-	D10	-
1/2 in	12.5	-	No. 100	0.150	66	-	-	D05	-
3/8 in	9.5	100	No. 140	0.106	-	-	-	Cu	-
No. 4	4.75	100	No. 200	0.075	57.6	-	-	Cc	-

Location: 4 at 14 feet
Description: Sandy CLAY

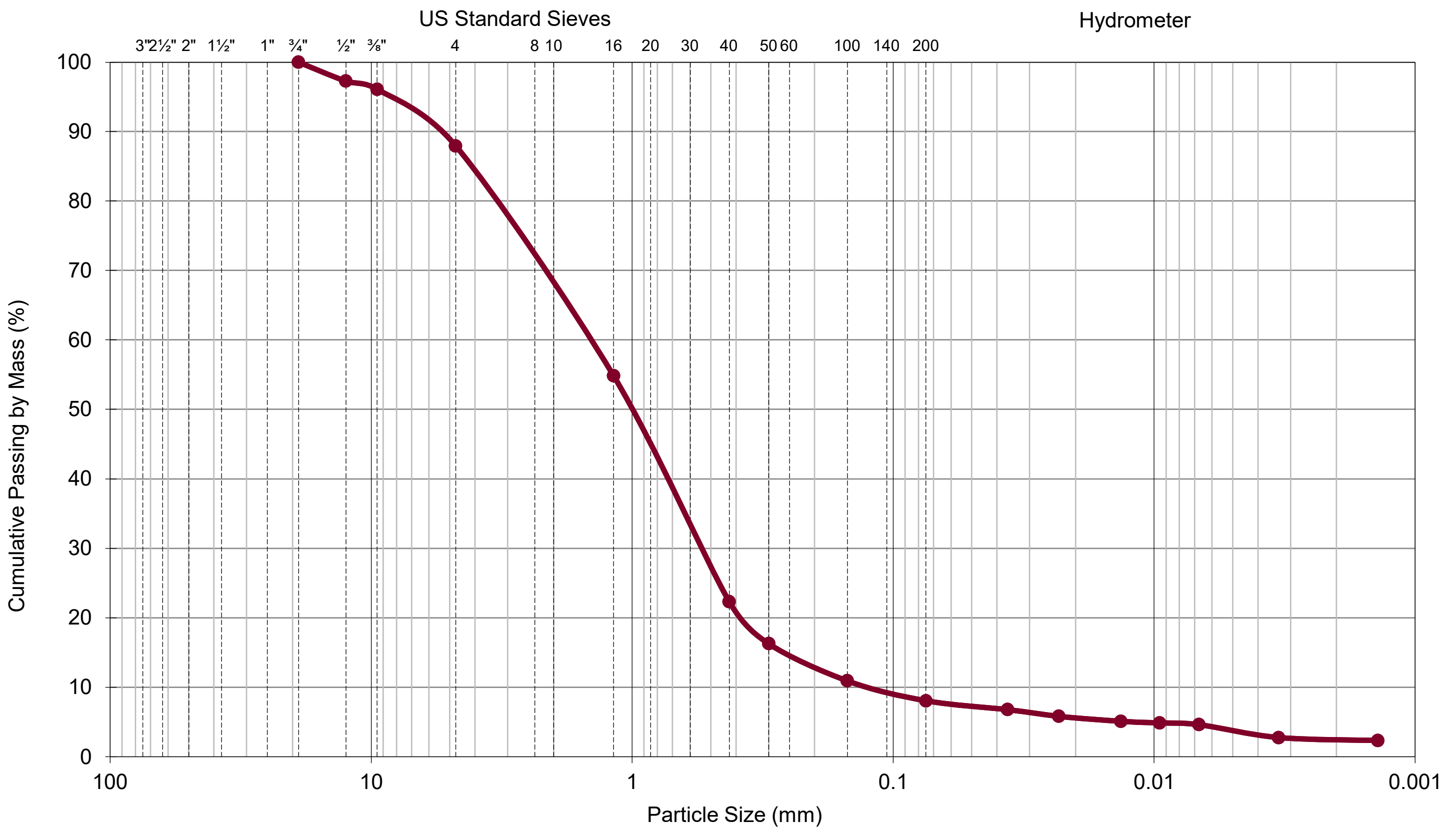
Classification: s(CL) / A-4 (3)
Liquid Limit: 29
Plasticity Index: 10
Activity: 0.6

Gravel (%): 0
Sand (%): 42
Silt/Clay (%): 58
< .002 mm (%): 16

Project No.: 22-3023

Ridgegate Parkway Couplet Roads

Gradation and Hydrometer (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	88	0.036	7	D90	5.658
5 in	125	-	No. 8	2.36	-	0.023	6	D85	4.197
4 in	100	-	No. 10	2.00	-	0.013	5	D80	3.400
3 in	75	-	No. 16	1.18	55	0.010	5	D60	1.465
2.5 in	63	-	No. 20	0.85	-	0.007	5	D50	1.013
2 in	50	-	No. 30	0.60	-	0.003	3	D40	0.740
1.5 in	37.5	-	No. 40	0.425	22	0.001	2	D30	0.541
1 in	25.0	-	No. 50	0.300	16	-	-	D15	0.254
3/4 in	19.0	100	No. 60	0.250	-	-	-	D10	0.120
1/2 in	12.5	97	No. 100	0.150	11	-	-	D05	-
3/8 in	9.5	96	No. 140	0.106	-	-	-	Cu	12.236
No. 4	4.75	88	No. 200	0.075	8.1	-	-	Cc	1.668

Location: 5 at 7 feet
Description: SAND

Classification: SP-SM / A-1-b (0)
Liquid Limit: NV
Plasticity Index: NP
Activity: -

Gravel (%): 12
Sand (%): 80
Silt/Clay (%): 8
< .002 mm (%): 3

GROUND ENGINEERING

Ridgegate Parkway Couplet Roads

TABLE 1: SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural Moisture Content	Natural Dry Density	Gradation			Atterberg Limits		Swell/Consolidation		USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth (feet)	(%)	(pcf)	Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plasticity Index	Volume Change (%)	Surcharge Pressure (psf)			
1	2	18.1	106.9	-	44	56	47	18	2.6	200	s(ML)	A-7-6 (8)	FILL: Sandy SILT
1	4	23.3	104.4	-	33	67	49	21	-	-	s(ML)	A-7-6 (14)	FILL: Sandy SILT
1	8	13.2	93.6	1	24	75	44	16	6.9	1000	(ML)s	A-7-6 (12)	SILT with Sand
1	13	22.1	94.0	-	63	37	46	9	-	-	SM	A-5 (0)	WEATHERED Silty SANDSTONE
2	1	14.6	95.2	-	14	86	53	22	10.2	200	MH	A-7-5 (22)	SILT
2	4	24.9	88.9	-	67	33	57	16	-	-	SM	A-2-7 (1)	WEATHERED Silty SANDSTONE
2	9	20.2	SD	-	71	29	46	11	-	-	SM	A-2-7 (0)	Silty SANDSTONE
2	14	25.3	96.8	-	73	27	48	13	-	-	SM	A-2-7 (0)	Silty SANDSTONE
3	1	15.7	94.9	-	45	55	34	9	-	-	s(ML)	A-4 (3)	Sandy SILT
3	4	12.4	108.8	-	-	50	37	15	-0.1	500	s(CL)	A-6 (5)	Sandy CLAY
3	8	1.5	-	30	62	8	NV	NP	-	-	(SP-SM)g	A-1-a (0)	SAND with Gravel
3	13	1.2	-	23	73	5	NV	NP	-	-	(SP)g	A-1-a (0)	SAND with Gravel
4	2	9.4	109.8	1	41	58	36	15	-	-	s(CL)	A-6 (6)	FILL: Sandy CLAY
4	4	6.4	106.8	9	69	23	29	10	-	-	SC	A-2-4 (0)	Clayey SAND
4	14	7.0	-	-	42	58	29	10	-	-	s(CL)	A-4 (3)	Sandy CLAY
5	1	18.2	96.1	1	28	71	47	21	1.7	200	(CL)s	A-7-6 (15)	FILL: CLAY with Sand
5	4	21.3	98.6	-	41	59	35	13	-	-	s(CL)	A-6 (5)	Sandy CLAY
5	7	2.7	SD	12	80	8	NV	NP	-	-	SP-SM	A-1-b (0)	SAND
5	12	14.6	-	1	44	55	57	29	-	-	s(CH)	A-7-6 (13)	Sandy CLAY
6	4	3.7	116.0	7	67	26	27	9	-	-	SC	A-2-4 (0)	FILL: Clayey SAND
6	9	8.7	105.1	4	45	51	32	14	-	-	s(CL)	A-6 (4)	FILL: Sandy CLAY
6	14	13.3	111.3	-	17	83	63	34	-	-	(CH)s	A-7-6 (31)	CLAY with Sand
6	19	15.4	106.6	1	27	72	48	21	-	-	(CL)s	A-7-6 (15)	CLAY with Sand
6	24	20.3	104.3	-	26	74	49	23	-	-	(CL)s	A-7-6 (17)	CLAY with Sand
3-5	0-5	-	-	-	-	72	39	15	-	-	(CL)s	A-6 (10)	CLAY with Sand
1 & 2	0-5	-	-	-	-	68	43	16	-	-	s(ML)	A-7-6 (10)	Sandy SILT

SD = Sample disturbed, NV = No value, NP = Non-plastic

Job No. 22-3023

TABLE 2: SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water Soluble Sulfates	pH	Redox Potential	Sulfide Reactivity	Resistivity	USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth *(feet)*	*(%)*		*(mv)*		*(ohm-cm)*			
1	4	0.01	8.4	- 73	Trace	2,416	s(ML)	A-7-6 (14)	FILL: Sandy SILT
5	4	0.03	8.7	- 90	Trace	2,615	s(CL)	A-6 (5)	Sandy CLAY

Appendix A

Detailed Logs of the Test Holes

GROUND ENGINEERING

TEST HOLE 1

CLIENT: Merrick & Company **PROJECT NAME:** Ridgegate Parkway Couplet Roads

JOB NO: 22-3023 **PROJECT LOCATION:** Lone Tree, CO

Elevation (ft)	Depth (ft)	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
100	0	**TOPSOIL**										
		FILL: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, moist, and brown. Caliche was noted locally.										
				19/12	18.1	106.9	56	47	18	2.6 (200)		s(ML)
95	5			17/12	23.3	104.4	67	49	21			s(ML)
		SILTS and CLAYS: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, dry, and brown.										
				25/12	13.2	93.6	75	44	16	6.9 (1000)		(ML)s
90	10											
		WEATHERED SANDSTONE: Weathered, fine grained, silty sandstones. Moist, moderately plastic, medium hard, and gray. Iron staining was noted commonly.										
				39/12	22.1	94.0	37	46	9			SM

Bottom of borehole at Approx. 14 feet.

TEST HOLE 2

GROUND ENGINEERING

CLIENT: Merrick & Company

PROJECT NAME: Ridgegate Parkway Couplet Roads

JOB NO: 22-3023

PROJECT LOCATION: Lone Tree, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
100	0		**TOPSOIL**										
			SILTS and CLAYS: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, dry, and brown.		26/12	14.6	95.2	86	53	22	10.2 (200)		MH
95	5		**WEATHERED SANDSTONE:** Weathered, fine grained, silty sandstones. Moist, moderately plastic, medium hard, and gray. Iron staining was noted commonly.		25/12	24.9	88.9	33	57	16			SM
90	10		**SANDSTONE BEDROCK:** Fine grained silty sandstones. Moderately plastic, medium-hard to hard, very moist, and gray in color. Iron staining was noted commonly.		48/12	20.2	SD	29	46	11			SM
					50/10	25.3	96.8	27	48	13			SM

Bottom of borehole at Approx. 14.83 feet.

TEST HOLE 3

GROUND ENGINEERING

CLIENT: Merrick & Company | **PROJECT NAME:** Ridgegate Parkway Couplet Roads

JOB NO: 22-3023 | **PROJECT LOCATION:** Lone Tree, CO

Elevation (ft)	Depth (ft)	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
100	0	**TOPSOIL**										
		SILTS and CLAYS: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, dry, and brown.		8/12	15.7	94.9	55	34	9			s(ML)
95	5			9/12	12.4	108.8	50	37	15	-0.1 (500)		s(CL)
		SANDS: Sands with gravels, silts, and clays. Non- to slightly plastic, medium dense, slightly moist, and light brown to brown to light gray.		29/12	1.5		8	NV	NP			(SP-SM)g
90	10			12-9-12	1.2		5	NV	NP			(SP)g

Bottom of borehole at Approx. 14.5 feet.

TEST HOLE 4

GROUND ENGINEERING

CLIENT: Merrick & Company

PROJECT NAME: Ridgegate Parkway Couplet Roads

JOB NO: 22-3023

PROJECT LOCATION: Lone Tree, CO

Elevation (ft)	Depth (ft)	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
100	0	**TOPSOIL**										
		FILL: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, moist, and brown. Caliche was noted locally.		18/12	9.4	109.8	58	36	15			s(CL)
		SANDS: Sands with gravels, silts, and clays. Non- to slightly plastic, medium dense, slightly moist, and light brown to brown to light gray.		18/12	6.4	106.8	23	29	10			SC
95	5											
90	10			29/12								
		SILTS and CLAYS: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, dry, and brown.		8-6-7	7.0		58	29	10			s(CL)
85	15											

Bottom of borehole at Approx. 15.5 feet.

GROUND ENGINEERING

TEST HOLE 5

CLIENT: Merrick & Company
PROJECT NAME: Ridgegate Parkway Couplet Roads
JOB NO: 22-3023
PROJECT LOCATION: Lone Tree, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
100	0		**TOPSOIL**										
			FILL: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, moist, and brown. Caliche was noted locally.		7/12	18.2	96.1	71	47	21	1.7 (200)		(CL)s
			SANDS: Sands with gravels, silts, and clays. Non- to slightly plastic, medium dense, slightly moist, and light brown to brown to light gray.		7/12	21.3	98.6	59	35	13			s(CL)
95	5												
					16/12	2.7	SD	8	NV	NP			SP-SM
90	10												
			SILTS and CLAYS: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, dry, and brown.		6-7-8	14.6		55	57	29			s(CH)

Bottom of borehole at Approx. 13.5 feet.

GROUND ENGINEERING

TEST HOLE 6

CLIENT: Merrick & Company

PROJECT NAME: Ridgegate Parkway Couplet Roads

JOB NO: 22-3023

PROJECT LOCATION: Lone Tree, CO

Elevation (ft)	Depth (ft)	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
100	0	**TOPSOIL**										
		FILL: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, moist, and brown. Caliche was noted locally.										
95	5			20/12	3.7	116.0	26	27	9			SC
90	10			12/12	8.7	105.1	51	32	14			s(CL)
		SILTS and CLAYS: Silts and clays with fine to coarse sands and local gravels. Moderately plastic, very stiff, dry, and brown.										
85	15			31/12	13.3	111.3	83	63	34			(CH)s
80	20			31/12	15.4	106.6	72	48	21			(CL)s
75	25			23/12	20.3	104.3	74	49	23			(CL)s

Bottom of borehole at Approx. 25 feet.

Appendix B

Pavement Section Calculations

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Flexible Structural Design Module

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	2,045,913
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	3,872 psi
Stage Construction	1
Calculated Design Structural Number	4.69 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Asphalt	0.44	1	7	-	3.08
2	Roadbase	0.12	1	14	-	1.68
Total	-	-	-	21.00	-	4.76

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Flexible Structural Design Module

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	2,045,913
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	3,813 psi
Stage Construction	1
Calculated Design Structural Number	4.71 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Asphalt	0.44	1	7	-	3.08
2	Roadbase	0.12	1	14	-	1.68
Total	-	-	-	21.00	-	4.76

Kumar & Associates, Inc.®
Geotechnical and Materials Engineers
and Environmental Scientists

An Employee Owned Company

800 Stockton Avenue, #4
Fort Collins, CO 80524
phone: (970) 416-9045
fax: (970) 416-9040
email: kaftcollins@kumarusa.com
www.kumarusa.com

Office Locations: Denver (HQ), Parker, Colorado Springs, Fort Collins, Glenwood Springs, and Summit County, Colorado

PRELIMINARY GEOTECHNICAL ENGINEERING STUDY AND PRELIMINARY PAVEMENT THICKNESS DESIGN PROPOSED REGENCY CENTERS DEVELOPMENT RIDGEGATE PARKWAY EAST OF PEORIA STREET LONE TREE, COLORADO

DRAFT SUBMITTAL

Prepared By:

Jacob A. Hanson, P.E.

Reviewed By:

Joshua L. Barker, P.E.

Prepared For:

Regency Centers
8480 East Orchard Road
Suite 6900
Greenwood Village, Colorado 80111

Attn: Mr. Thomas Metzger

Project No. 20-3-207

December 10, 2020

DRAFT

TABLE OF CONTENTS

DRAFT

SUMMARY

1. The subsurface conditions at the site were evaluated by drilling eight (8) exploratory borings to depths of approximately 12 to 35 feet below the existing ground surface. The borings generally encountered nil to approximately 3.5 feet of man-placed fill material consisting of lean clay with sand to clayey sand with gravel overlying natural clayey soils consisting of lean clay with sand to sandy lean clay with layers of fat clay with sand that extended to depths of about 5.5 to 29 feet below the ground surface. The natural overburden soils were generally underlain by interbedded claystone and sandstone bedrock that continued to the explored depths of about 20 to 35 feet below the ground surface. Boring 3 encountered approximately 4 feet of man-placed fill material overlying interbedded claystone and sandstone bedrock that continued to the explored depth of about 20 feet below the ground surface. Boring 4 encountered sandstone bedrock underlying the natural overburden soils which was in turn underlain by claystone bedrock that continued to the explored depth of about 25 feet. Borings 6 and 7 encountered layers of natural granular soils consisting of clayey sand to clayey sand with gravel within the natural overburden clayey soils. Boring 8 encountered natural clayey soils overlying natural granular soils that continued to the explored depth of about 12 feet where practical auger refusal was encountered.

   Groundwater was not encountered in the borings at the time of drilling or when subsequently checked 7 days after drilling.

2. Shallow spread footing foundations supported by a relatively thick zone of compacted fill should be feasible for the buildings. There is some risk of foundation movement associated with using shallow foundations on this site. The “Geotechnical Engineering Considerations” section of this report presents discussion of the risks associated with use of shallow foundations. Additional criteria are presented in the body of this report.

   Shallow foundations will likely require approximately 7 feet of subexcavation below the footing bearing elevation. Further discussion of the zone of subexcavation is provided in the Geotechnical Considerations section of this report. Spread footings bearing on properly compacted structural fill extending to natural soils or bedrock are expected to be designed for allowable soil bearing pressures between 2,000 and 4,000 psf.

3. The least risk of building movement due to soil expansion would be to support the structure(s) on deep foundations and structurally-supported floors. Our experience is such that structurally supported floors can be costly for structures of this size; therefore, we have also developed recommendations for spread footing foundations and slab on grade floors.

   Slab on grade floors, if selected, should be underlain by at least 10 feet of properly compacted fill material. The fill material should consist of moisture conditioned on-site soils or imported non-expansive fill material. Additional design considerations and recommendations are presented herein.

4. A deep foundation alternative is feasible for the structures on the kiosk structure site. Piers can be designed for allowable end bearing pressures ranging from 25,000 psf to 40,000 psf and bedrock side shear (skin friction) of 10% of end bearing, depending on the consistency of the bedrock at each particular building location. Piers should be designed for minimum dead load pressures between 15,000 and 30,000 psf based on pier end area only.

5. Due to the high swell potential of the overburden soils, structurally supported floors may be economical for the kiosk structure on the site.

6. Typical full-depth and/or layered composite asphalt pavement sections should be feasible at the site.

PURPOSE AND SCOPE OF WORK

This report presents the results of a preliminary geotechnical engineering study and preliminary pavement thickness design for the proposed Ridge Gate Parkway development located approximately 0.62 miles east of Peoria Street in Lone Tree, Colorado. The project site is shown on Fig. 1. The study was conducted to characterize the general site subsurface conditions and to provide preliminary geotechnical engineering recommendations to be used for planning and preliminary design information. This study was conducted in accordance with the scope of work presented in our Proposal No. P3-20-301 dated September 16, 2020.

This report has been prepared to summarize the data obtained during this study and to present our conclusions and preliminary recommendations based the subsurface conditions encountered. The information and conclusions presented herein are based on data obtained from widely-spaced exploratory borings drilled for this study in and around the proposed building site. Preliminary design parameters and a discussion of general geotechnical engineering considerations related to construction of future development are included in the report, along with preliminary pavement thicknesses.

PROPOSED DEVELOPMENT

We have been provided with a site plan showing a conceptual layout for the site development, which includes a 123,000 ft^2 King Soopers grocery store on the west side of the site and two 7,200 ft^2 retail buildings, a 4,000 ft^2 bank, and a 4,500 ft^2 restaurant on the east side of the site. The retail buildings range from approximately 4,000 to 7,200 ft^2. The area between the King Soopers store and the retail units will be paved for drive lanes and parking stalls. Part of the site development will likely include a fueling facility associated with the King Soopers store.

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

SITE CONDITIONS

At the time of drilling, the site was vacant of structures and was relatively flat from previous construction grading. The site was bordered to the west, south and east by vacant fields and to the north by Ridge Gate Parkway. The fields surrounding the site contained rolling hills.

SUBSURFACE CONDITIONS

The subsurface conditions at the site were explored by drilling a total of eight (8) exploratory borings to depths of about 12 to 35 feet below the ground surface at the approximate locations shown on Fig. 1. Graphic logs of the borings are presented on Fig. 2, and a legend and notes describing the soils encountered is presented on Fig. 3.

The borings generally encountered nil to approximately 3.5 feet of man-placed fill material consisting of lean clay with sand to clayey sand with gravel overlying natural clayey soils consisting of lean clay with sand to sandy lean clay with layers of fat clay with sand that extended to depths of about 5.5 to 29 feet below the ground surface. The natural overburden soils were generally underlain by interbedded claystone and sandstone bedrock that continued to the explored depths of about 20 to 35 feet below the ground surface. Boring 3 encountered approximately 4 feet of man-placed fill material overlying interbedded claystone and sandstone bedrock that continued to the explored depth of about 20 feet below the ground surface. Boring 4 encountered sandstone bedrock underlying the natural overburden soils which was in turn underlain by claystone bedrock that continued to the explored depth of about 25 feet. Borings 6 and 7 encountered layers of natural granular soils consisting of clayey sand to clayey sand with gravel within the natural overburden clayey soils. Boring 8 encountered natural clayey soils overlying natural granular soils that continued to the explored depth of about 12 feet where practical auger refusal was encountered.

The man-placed fill was fine to coarse grained with gravel, slightly moist to moist and brown to dark brown. The natural clayey soils contained a fine to coarse grained sand fraction and were moist and brown to light brown. The natural granular soils were fine to coarse grained, slightly moist to moist and light brown. The bedrock was generally fine to coarse grained, moist and brown to gray.

Based on sampler penetration resistance, the natural clayey soils were stiff to hard, the natural granular soils were dense to very dense and the bedrock was medium hard to very hard.

<u>Groundwater Conditions</u>: Groundwater was not encountered in the borings at the time of drilling or when subsequently checked 7 days after drilling.

Groundwater levels are expected to fluctuate with time, and may fluctuate upward after wet weather or subsequent to landscape irrigation.

LABORATORY TESTING

Laboratory testing was performed on selected samples obtained from the borings to determine in-situ moisture content and dry density, Atterberg limits, and swell-consolidation characteristics.

The results of the laboratory tests are shown next to the boring logs, graphically plotted on Figs. 4 through 11, and provided in the attached Summary of Laboratory Test Results, Table I. The testing was conducted in general accordance with recognized test procedures, primarily those of the ASTM International and the Colorado Department of Transportation.

Swell-Consolidation: Swell-consolidation tests were conducted on selected samples of the man-placed fill material, natural overburden soils and bedrock materials in order to evaluate their compressibility and swell characteristics under loading and when submerged in water. Each sample was prepared and placed in a confining ring between porous discs, subjected to a surcharge pressure of 200 or 1,000 psf, and allowed to consolidate before being submerged. The sample height was monitored until deformation practically ceased under each load increment.

Results of the swell-consolidation tests are plotted as a curve of the final strain at each increment of pressure against the log of the pressure and are presented on Figs. 4 through 9. A sample of the man-placed fill exhibited very high swell potential (9.7%) upon wetting under a surcharge pressure of 200 psf. Samples of natural overburden soils exhibited very high swell potential (12.5%) when wetted under a surcharge pressure of 200 psf and moderate to high swell potential (2.9% to 4.0%) when wetted under a surcharge pressure of 1,000 psf. The interbedded claystone and sandstone bedrock exhibited low swell potential (0.8%) when wetted under a surcharge pressure of 1,000 psf.

Moisture-Density Relationships: Results of moisture-density relationships from a composite sample of onsite soils, as determined by standard Proctor (ASTM D698), are presented on Fig. 6. The maximum dry density of the composite sample from the borings was 102.4 pcf at an optimum moisture content of 17.1 percent as determined by standard Proctor (ASTM D698).

Index Properties: Samples were classified into categories of similar engineering properties in general accordance with the Unified Soil Classification System. This system is based on index properties, including liquid limit and plasticity index and gradation characteristics. Values for moisture content, dry density, liquid limit and plasticity index, and the percent of soil passing the No. 200 sieves are presented in Table I and adjacent to the corresponding sample on the boring logs.

WATER-SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in samples of the natural clayey soils obtained from the exploratory borings ranged from 0.03% and 0.28%. These concentrations of water-soluble sulfates represent a Class S0 to Class S2 severity exposure of sulfate attack on concrete exposed to these materials. These degrees of attack are based on a range of Class S0, Class S1, Class S2, and Class S3 severity exposure as presented in ACI 201.2R-16.

Based on the laboratory test results, we recommend all concrete exposed to the on-site materials meet the cement requirements for Class S1 exposure as presented in ACI 201. Alternatively, the concrete could meet the Colorado Department of Transportation's (CDOT) cement requirements for Class S1 exposure as presented in Section 601.04 of the CDOT Standard Specifications for Road and Bridge Construction (2011).

GEOTECHNICAL ENGINEERING CONSIDERATIONS

Without documentation of placement conditions including density testing documenting the degree of compaction, the existing fill materials are considered non-engineered and generally not suitable for support of foundations or floor slabs. Based upon the results of the laboratory testing, the existing fill materials are estimated to have moisture contents well below the optimum moisture content, which in turn indicates a potential for movement of structures or slabs constructed on the undocumented fills should those fills be subject to post-construction wetting.

Building Foundations: We believe that shallow spread footing foundations are likely the most economical foundation type to support the proposed structures on the site and will provide acceptable performance for the proposed construction. However, subexcavation below the foundation elements will likely be required to mitigate swell in the overburden materials. Shallow foundations will likely require approximately 7 feet of subexcavation below the footing bearing elevation. The lower 5 feet of the zone of subexcavation may consist of moisture conditioned on-site overburden soils. The upper 3 feet of the zone of subexcavation should consist of imported non-expansive material meeting the criteria presented in the “Site Grading” section of this report. Spread footings bearing on properly compacted structural fill extending to natural soils or bedrock are expected to be designed for allowable soil bearing pressures between 2,000 and 4,000 psf. Designs with minimum dead load pressures of 500 to 1,000 psf may be necessary if the design-level investigations identify swell potential at foundation elevations. These conditions should be further evaluated in a design-level geotechnical investigation.

Floor Slabs: Considering the above discussion, we believe slab-on-grade construction may be used for the project, provided that the risk of distress is recognized and accepted by the owner, and the measures herein are taken to reduce the damage which could result from movement should the underslab materials be subjected to excessive moisture increases. The intent of our recommendations is to provide for a condition where there is a good chance slab heave movements will not exceed 1 inch and it is unlikely they will exceed 2 inches unless extreme wetting is allowed. Barring unforeseen events, we do not believe extreme wetting is likely to occur if the surface drainage and irrigation recommendations presented in this report are followed. It is also very important to provide the recommended isolation between the structure and the slab-on-grade floors to reduce damage in the event that heaving occurs. Slab-on-grade floors should not be used if architectural finishes that will not tolerate upward slab movement are planned.

For the slab on grade alternative, we recommend the existing expansive soils be overexcavated to a depth of at least 10 feet below the floor slab subgrade elevation. The lower 5 feet of the zone of subexcavation may consist of moisture conditioned on-site overburden soils. The upper 5 feet of the zone of subexcavation should consist of imported non-expansive material meeting the criteria presented in the “Site Grading” section of this report. An underdrain system should be constructed at the base of the zone of subexcavation as described in the “Underdrain System” section of this report.

As an alternative, the upper 5 feet of the zone of subexcavation may consist of on-site lime-treated subgrade soils. The removed soil may be treated with a lime product to mitigate the swell potential of the soil. After properly treating the soil with lime, the soil should be moisture conditioned and properly compacted. All fill materials should be placed and compacted according to the material and placement criteria presented in the “Site Grading” section of this report. The zone of subexcavation should extend laterally outside of all building lines a minimum of 5 feet.

Fuel Station Kiosk Structure: A deep foundation alternative with a structurally supported floor may be an economical alternative for the kiosk structure due to the high swelling soils and relatively shallow bedrock encountered in the area. Founding the kiosk structure on drilled shafts would dramatically reduce the total and differential movements, thereby reducing the potential for aesthetic blemishes that can develop after construction. The floor may be supported on foundations the same as the building structure, and would be underlain by a void or full crawl space. Design of a crawl space or underfloor void should consider drainage and moisture control. We recommend a 6 to 12-inch void beneath structural floors.

Piers can be designed for allowable end bearing pressures ranging from 25,000 psf to 40,000 psf and bedrock side shear (skin friction) of 10% of end bearing, depending on the consistency of the bedrock at each particular building location. Piers should be designed for minimum dead load pressures between 15,000 and 30,000 psf based on pier end area only. A minimum penetration of 6 to 10 feet into bedrock and a minimum pier length of 20 to 25 feet will be required. Foundation walls and grade beams will require a 4-inch to 6-inch void space be provided beneath the walls to concentrate pier loadings and isolate the foundation from the expansive foundation materials. The presence of water in the exploratory borings indicates the use of temporary casing or dewatering equipment in the pier holes will likely be required to reduce water infiltration. In no case should concrete be placed in more than 3 inches of water.

Surface Drainage: The ground surrounding exteriors of buildings should be sloped to drain away from buildings in all directions. Therefore, during project planning, site development plans should attempt to place the buildings relatively high with relation to the surrounding ground surface. For preliminary planning, a slope of at least 6 inches within the first 10 feet of buildings should be assumed. The probability of obtaining foundations which remain stable for the life of the building will be significantly increased by planning a well-drained site without irrigation adjacent to buildings.

Moisture Conditioning of On-Site Soils: The on-site overburden soils and underlying bedrock materials generally appear to be about 3% to 7% below the optimum moisture content (OMC). The on-site soils had plasticity indices ranging from 9 to 33 with the majority of the tested samples in the range of about 29 to 33. Materials with plasticity indices in this range require substantially more processing time, water, and effort to thoroughly moisture condition this material. Elevating the moisture content in these materials by about 7% or more requires proper saturation time under ideal circumstances. Ideal circumstances in this case means that the material is broken down into particle sizes no larger than about ¼-inch and allowed to be in the presence of free water for at least 24 hours. An on-site pug mill or some other form of specific processing equipment should be utilized to achieve the proper particle sizes and moisture conditioning.

Presence of Fat Clays: As indicated natural fat clays were encountered in the borings. Fat clay can be susceptive to significant volume changes if the moisture content is allowed to vary. Therefore, we recommend all fat clay material be removed within building footprints and replaced with properly compacted structural fill.

SITE GRADING

Prior to placement of fill, vegetation and topsoil should be removed and the natural ground surface prepared by scarifying to a depth of 8 to 10 inches, moistening, and recompacting to provide a uniform base for fill placement. Large areas of fill were noted on the site and should generally be considered unsuitable in its current condition for support of foundations and slab. Further evaluation of the suitability of existing fill, for support of pavements may be performed during a design level geotechnical engineering study. Based on the samples obtained during this study, the majority of the onsite soils should be suitable for use as structural fill on the site. Some asphalt and brick debris was encountered in some of the samples of fill so there may be areas that contain trash or deleterious materials that would be unsuitable for use as structural fill.

In our opinion, subsoils similar to those encountered in the exploratory borings drilled for this study can be excavated with conventional construction equipment. Based on the proposed depths of cut and the subsurface conditions, it may be possible that a heavy-duty hydraulic excavator may be required at some locations. The production rates in bedrock may be somewhat diminished over that of the soil excavation.

Permanent Cut and Fill Slopes: The following guidelines should be observed for cut and fill construction:

1. The risk of slope instability will be significantly increased if seepage is encountered in cuts. If seepage is encountered, the Geotechnical Engineer should be retained to reevaluate requirements for stabilizing the slope, including slope flattening and dewatering if required.

2. Fills up to 20 feet in height can be used if the fill slopes do not exceed 3 horizontal to 1 vertical and the fills are properly compacted and drained. Fills should be benched into hillsides exceeding 4 horizontal to 1 vertical (4:1). A bench height between 2 feet and 4 feet should be used.

3. A slope stability evaluation should be performed for structures or other development located on existing or planned cut and fill slopes with inclinations greater than 25%.

4. Good surface drainage should be provided around all permanent cuts and fills to direct surface runoff away from the slope faces. Concentrated surface runoff onto slopes should not be allowed.

5. Fill slopes, cut slopes and other stripped areas should be protected against erosion by revegetation or other materials until the slope has had time to adequately revegetate.

UNDERDRAIN SYSTEM

Discussion of the necessity of a subsurface drainage (underdrain) system is discussed in the "Geotechnical Considerations" section of this report. The criteria presented below should be followed.

The base of the zone of subexcavation should be sloped to provide positive drainage towards the perimeter of the zone of subexcavation.

An underdrain system should consist of a drain around the perimeter of the zone of subexcavation and keyed at least 1 foot into the underlying materials below the lowest elevation of the underslab fill layer. Free-draining granular material used in the drain system should contain less than 5%

passing the No. 200 sieve, less than 30% passing the No. 4 sieve and have a maximum size of 2 inches.

The drain lines should consist of perforated, rigid, drain pipe placed in the bottom of a trench around the perimeter of the zone of subexcavation and surrounded above the invert level with free draining granular material. The free-draining granular material should extend at least 1 foot below and 3 feet above the base of the subexcavation zone. The entire drainage trench should be wrapped with a geotextile fabric to prevent migration of fines from the surrounding soil into the drainage material. The drain lines should be graded to sumps or gravity outlet at a minimum slope of ½%. Sumps should be provided with alarms in the event pumping equipment malfunctions.

Standby pump capacity should be provided in the event of pump failure. We also believe an overdesigned pump capacity is desirable in the event groundwater conditions change.

PRELIMINARY PAVEMENT DESIGN

Typical full-depth and/or layered composite asphalt pavement sections should be feasible at the site.

<u>Subgrade Materials</u>: Based on the results of the field exploration and laboratory testing programs, the majority of the near surface subgrade materials encountered at the subject site generally classify as A-7-6 with group indices between 20 to 27, in accordance with the AASHTO soil classification system. A-7-6 materials are generally considered to provide poor subgrade support.

<u>Preliminary Pavement Thickness Design</u>: The use of full-depth asphalt sections should be considered for drive lane and parking area pavements. Depending on traffic loading conditions, the design full depth pavements restricted to automobile traffic will probably range from 5.5 to 7 inches. Areas of heavy truck traffic will likely consist of full depth pavement sections in the range of 6 to 8 inches depending on the volume of truck traffic.

Generally, it should be assumed that the pavement subgrade will need to be moisture conditioned and recompacted to a depth of 24 to 36 inches below the finished subgrade elevation.

DESIGN LEVEL STUDY

The recommendations and considerations presented in this report are for preliminary design and layout purposes. A design level geotechnical engineering study and pavement thickness design should be performed on the site to more clearly establish construction requirements. Given the data obtained from this preliminary study, we recommend that a significant number of in-situ and remolded swell testing be performed to establish the subexcavation requirements for floor slabs as well as spread footing foundations, if applicable. We recommend that the design level exploratory borings be professionally surveyed to more accurately establish the bedrock elevations.

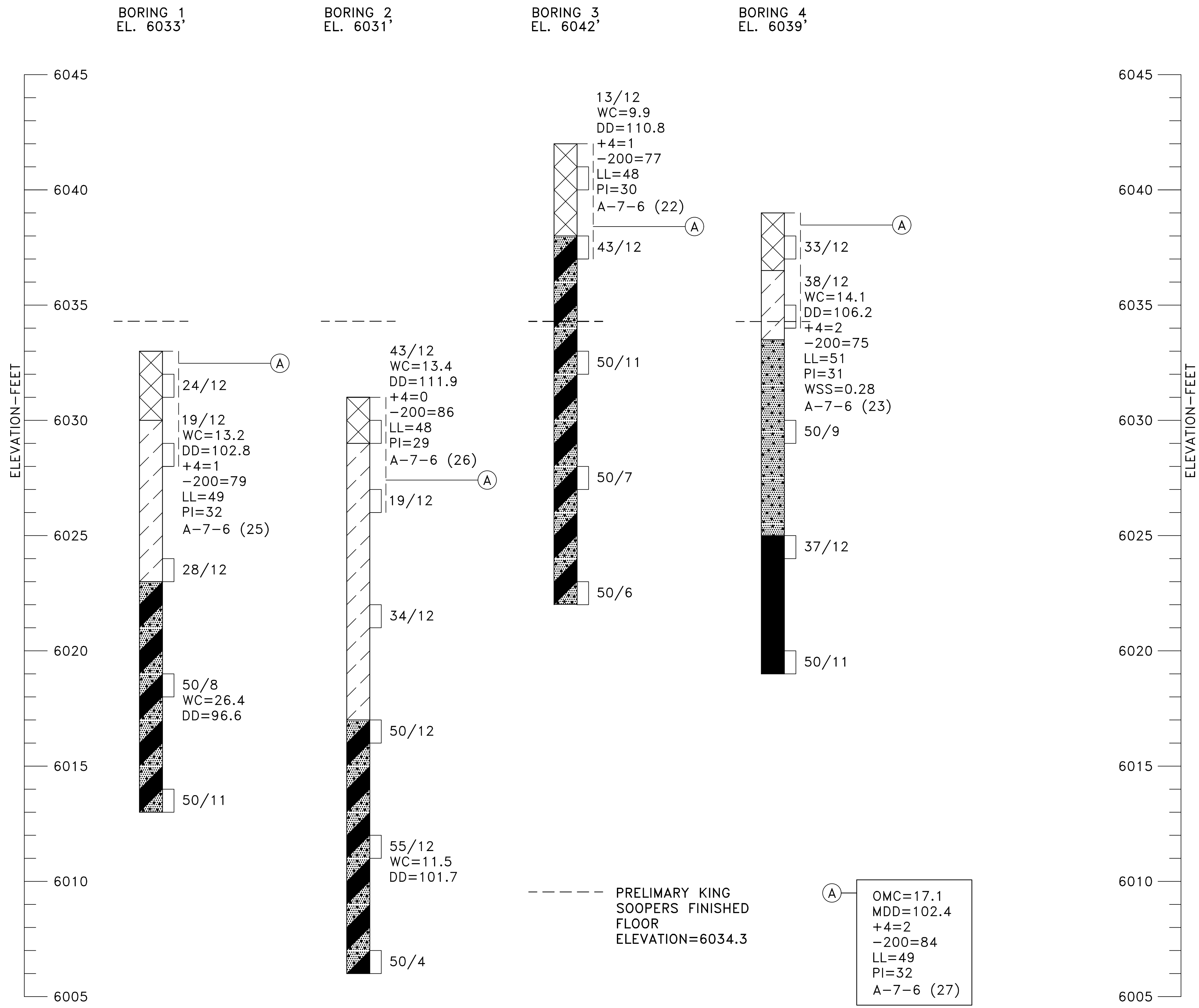
LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering practices in this area for use by the client for preliminary design and planning purposes. The preliminary conclusions and recommendations submitted in this report are based upon the data obtained from the widely spaced exploratory borings drilled at the locations indicated on the exploratory boring plan. Additional investigation must be conducted once building locations and floor elevations have been determined to provide final recommendations. We recommend on-site observation of site grading by a representative of the geotechnical engineer.

JAH/js
Rev. by: JLB
cc: book, file

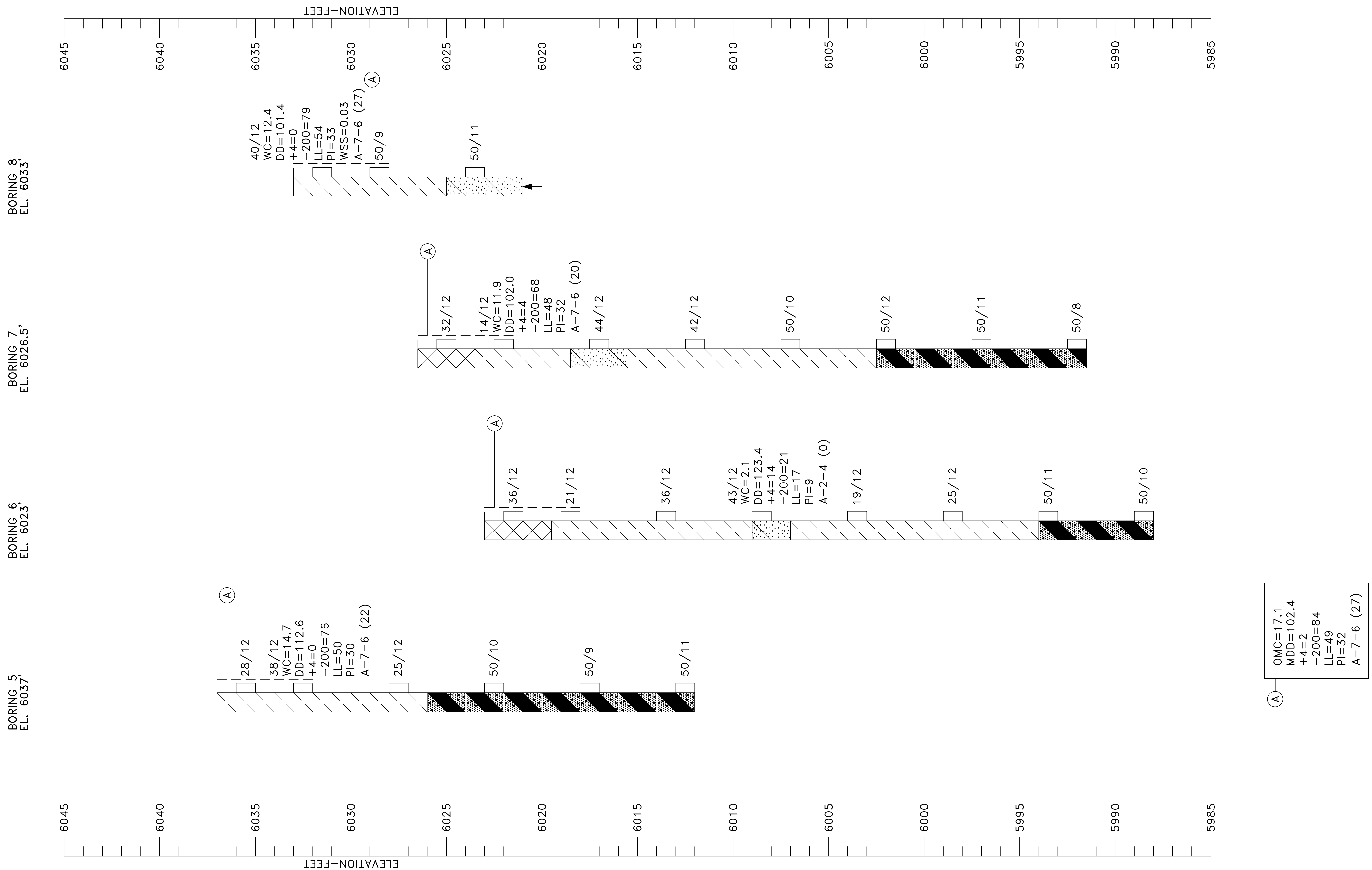
20-3-207	Kumar & Associates	REGENCY CENTERS DEVELOPMENT - RIDGE GATE	LOGS OF EXPLORATORY BORINGS	Fig. 1

December 03, 2020 - 10:18pm
V:\Projects\2020\20-3-207 King Soopers RidgeGate Preliminary\Drafting\203207-01.dwg



Dec 08, 20Y - 13:36pm
V:\Projects\2020\20-3-207 King Soopers RidgeGate Preliminary\Drafting\203207-02 to 04.dwg

20-3-207	Kumar & Associates	REGENCY CENTERS DEVELOPMENT - RIDGE GATE	LOGS OF EXPLORATORY BORINGS	Fig. 2



Dec 08, 20Y – 13:36pm
V:\Projects\2020\20-3-207 King Soopers RidgeGate Preliminary\Drafting\203207-02 to 04.dwg

20-3-207	Kumar & Associates	REGENCY CENTERS DEVELOPMENT – RIDGE GATE	LOGS OF EXPLORATORY BORINGS	Fig. 3

LEGEND

FILL; LEAN CLAY WITH SAND (CL) TO CLAYEY SAND WITH GRAVEL (SC), FINE TO COURSE GRAINED WITH GRAVEL, SLIGHTLY MOIST TO MOIST, BROWN TO DARK BROWN

LEAN CLAY WITH SAND (CL) TO SANDY LEAN CLAY (CL), WITH LAYERS OF FAT CLAY WITH SAND (CH), FINE TO COARSE GRAINED SAND FRACTION, STIFF TO HARD, MOIST, BROWN TO LIGHT BROWN

CLAYEY SAND (SC) TO CLAYEY SAND WITH GRAVEL (SC), FINE TO COARSE GRAINED, DENSE TO VERY DENSE, SLIGHTLY MOIST TO MOIST, LIGHT BROWN.

INTERBEDDED CLAYSTONE AND SANDSTONE BEDROCK, FINE TO COARSE GRAINED, HARD TO VERY HARD, MOIST, BROWN TO GRAY.

SANDSTONE BEDROCK, FINE TO COARSE GRAINED, VERY HARD, MOIST, BROWN.

CLAYSTONE BEDROCK, FINE TO MEDIUM GRAINED, MEDIUM HARD TO VERY HARD, BROWN TO GRAY.

DRIVE SAMPLE, 2–INCH I.D. CALIFORNIA LINER SAMPLE.

DISTURBED BULK SAMPLE.

24/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 24 BLOWS OF A 140–POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

PRACTICAL AUGER REFUSAL.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON NOVEMBER 12, 2020 WITH A 4–INCH–DIAMETER CONTINUOUS–FLIGHT POWER AUGER.

2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY HANDHELD GPS DEVICE.

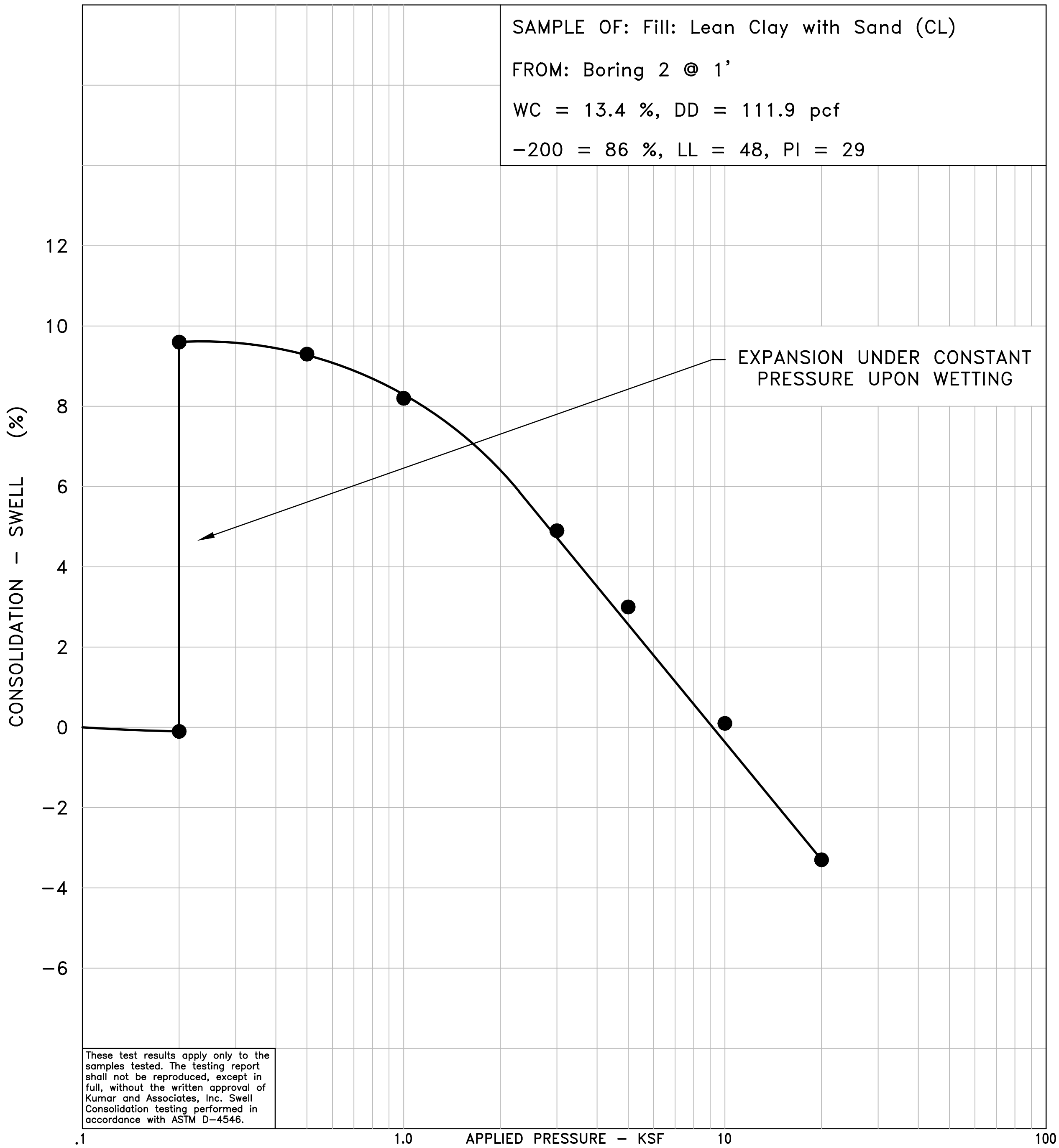
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.

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

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

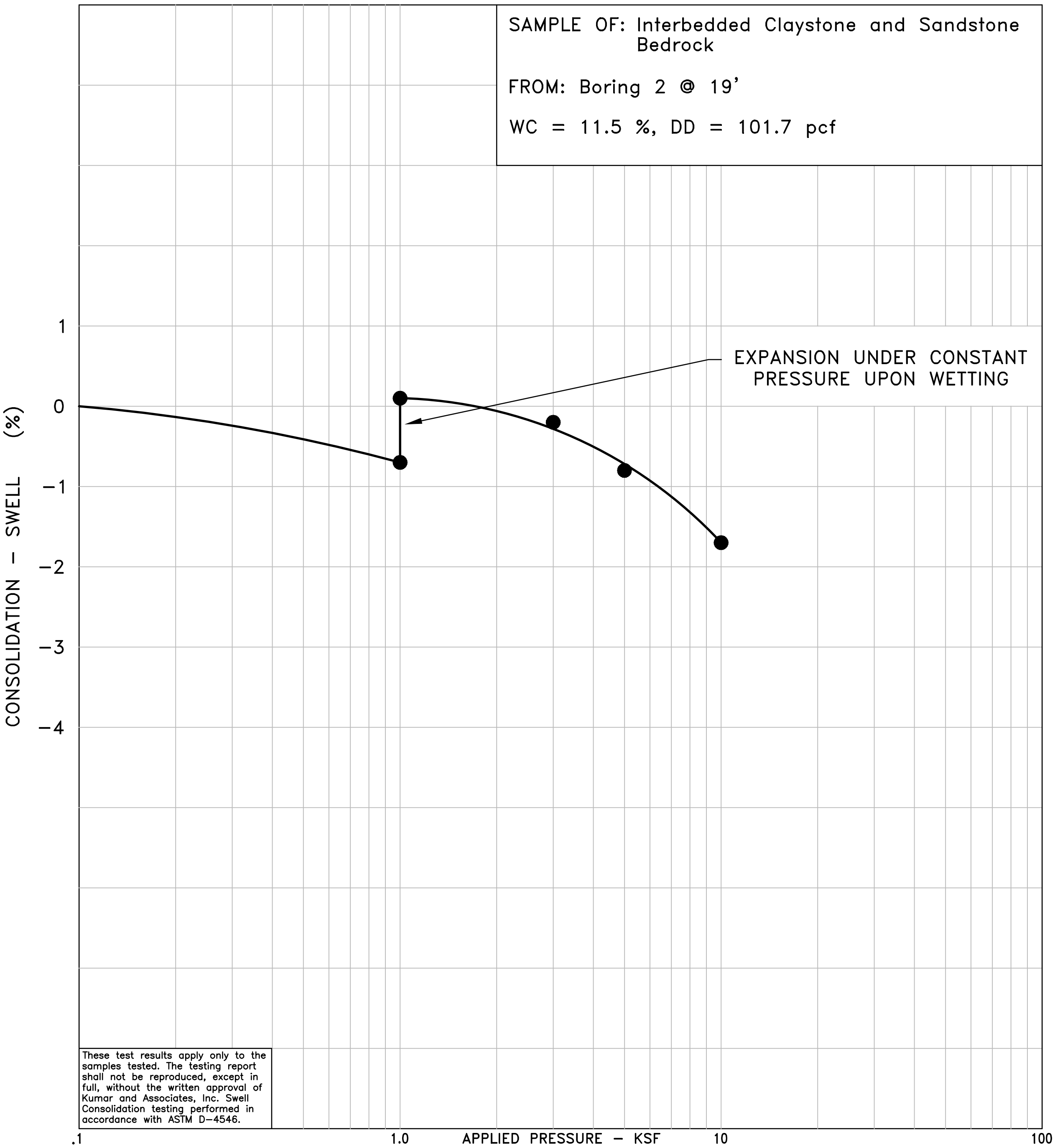
6. GROUNDWATER WAS NOT ENCOUNTERED IN THE BORINGS AT THE TIME OF DRILLING OR WHEN CHECKED 7 DAYS LATER.

7. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D2216);
   DD = DRY DENSITY (pcf) (ASTM D2216);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D6913);
   –200= PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP–L 2103);
   A–2–6 (0) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M145);
   OMC = OPTIMUM MOISTURE CONTENT (%) (ASTM D698);
   MDD = MAXIMUM DRY DENSITY (pcf) (ASTM D698).



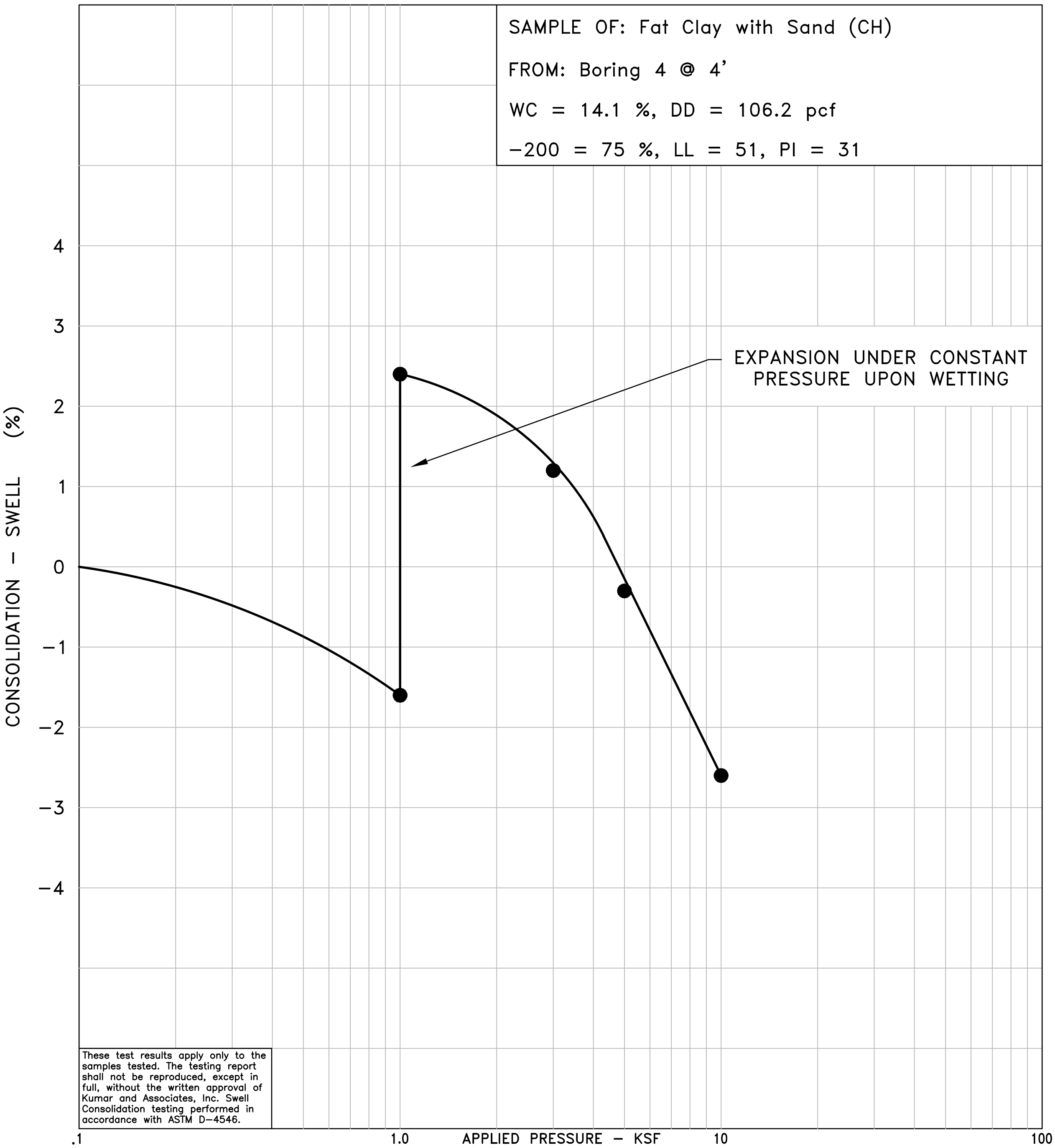
20−3−207	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 5

December 08, 2020 − 01:42pm
V:\Projects\2020\20-3-207 King Soopers RidgeGate Preliminary\Drafting\203207-05 to 10.dwg

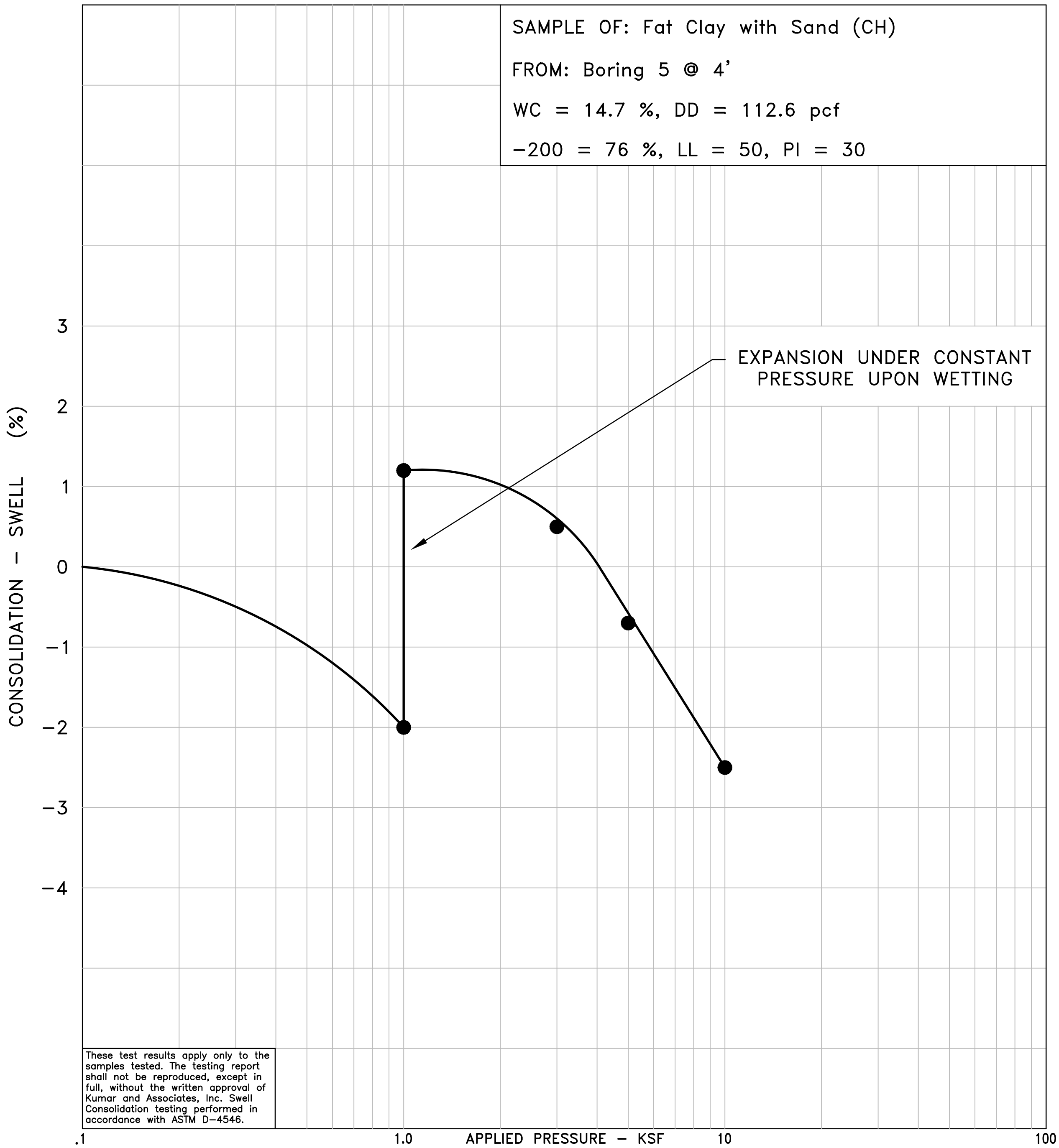


December 08, 2020 - 01:42pm
V:\Projects\2020\20-3-207 King Soopers RidgeGate Preliminary\Drafting\203207-05 to 10.dwg

20-3-207	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 6

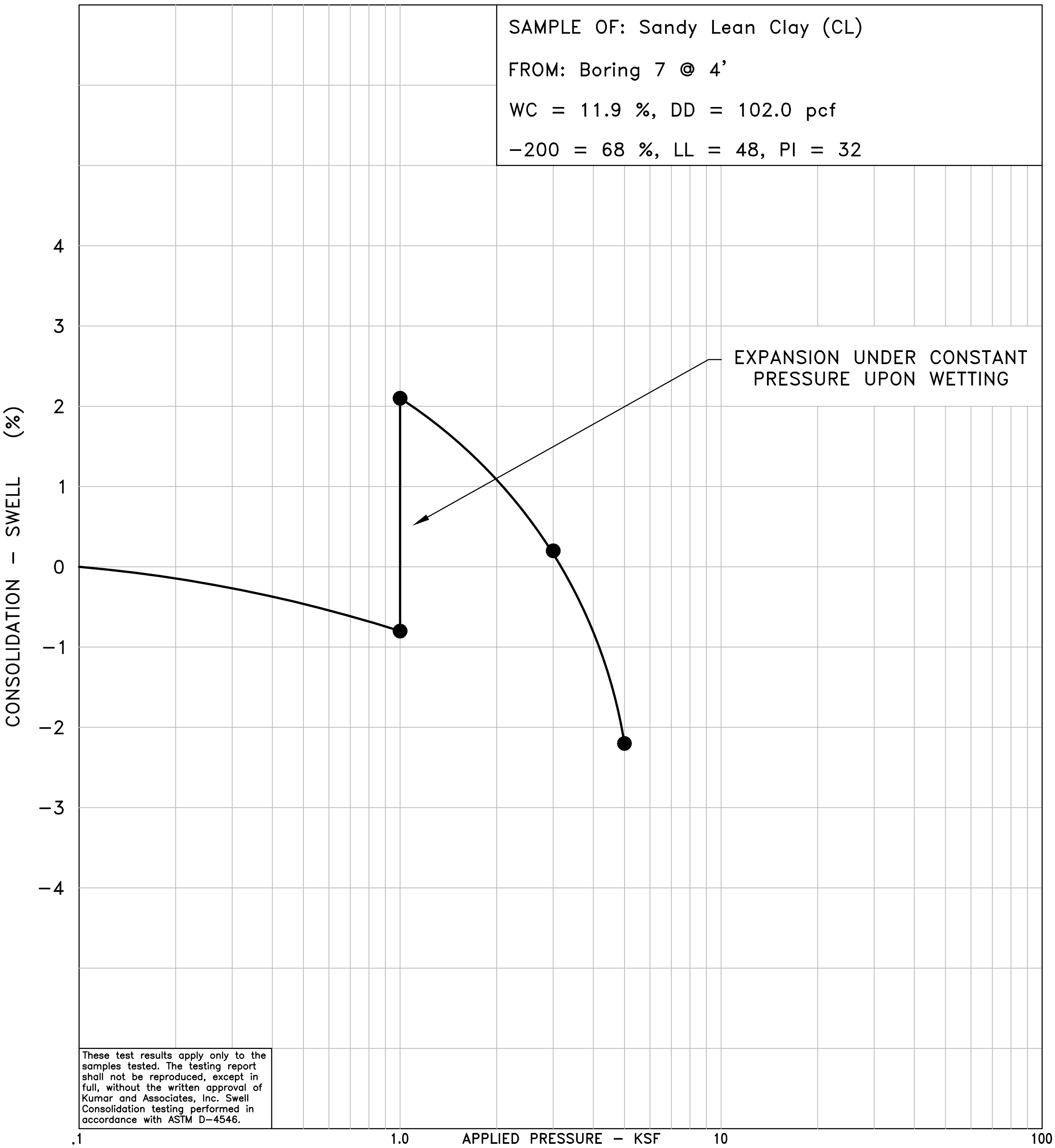


20−3−207	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 7

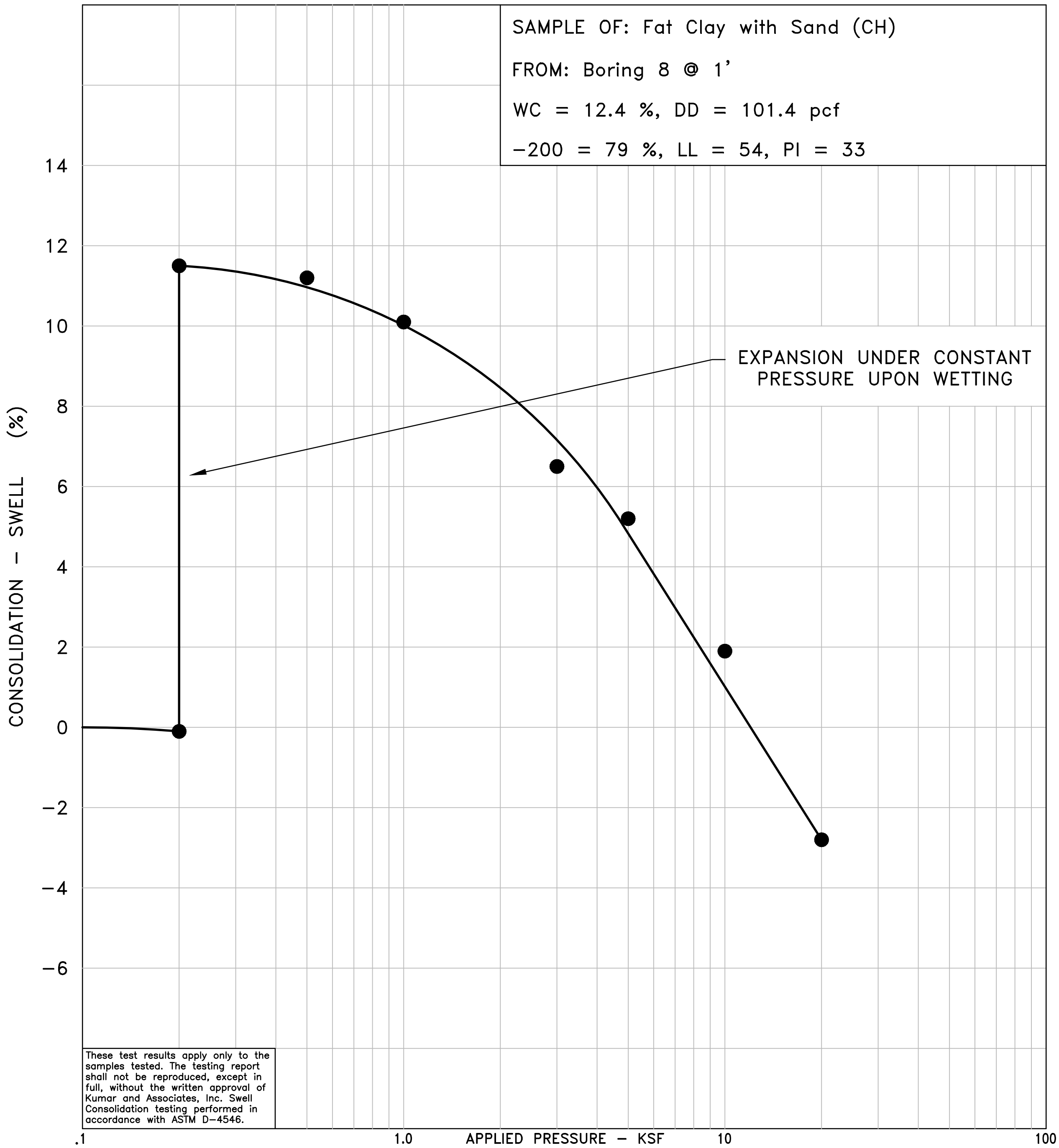


December 08, 2020 - 01:42pm
V:\Projects\2020\20-3-207 King Soopers RidgeGate Preliminary\Drafting\203207-05 to 10.dwg

20-3-207	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 8

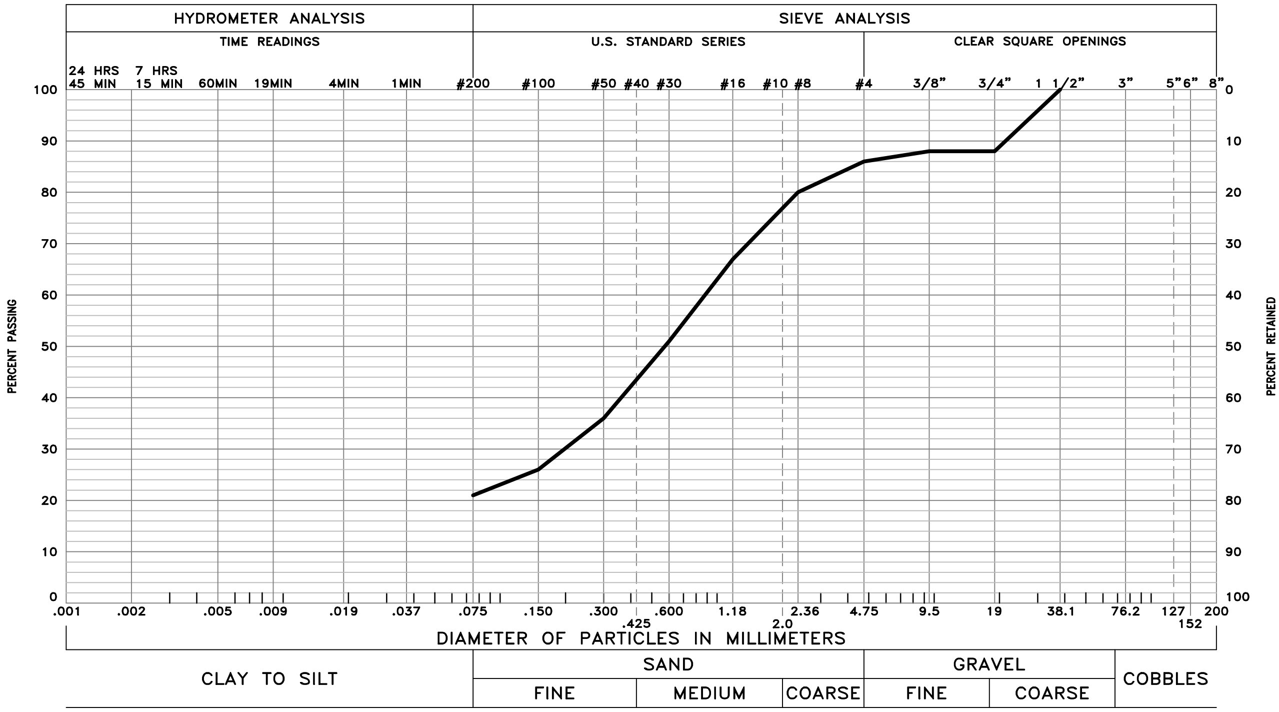


20−3−207	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 9



20−3−207	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 10

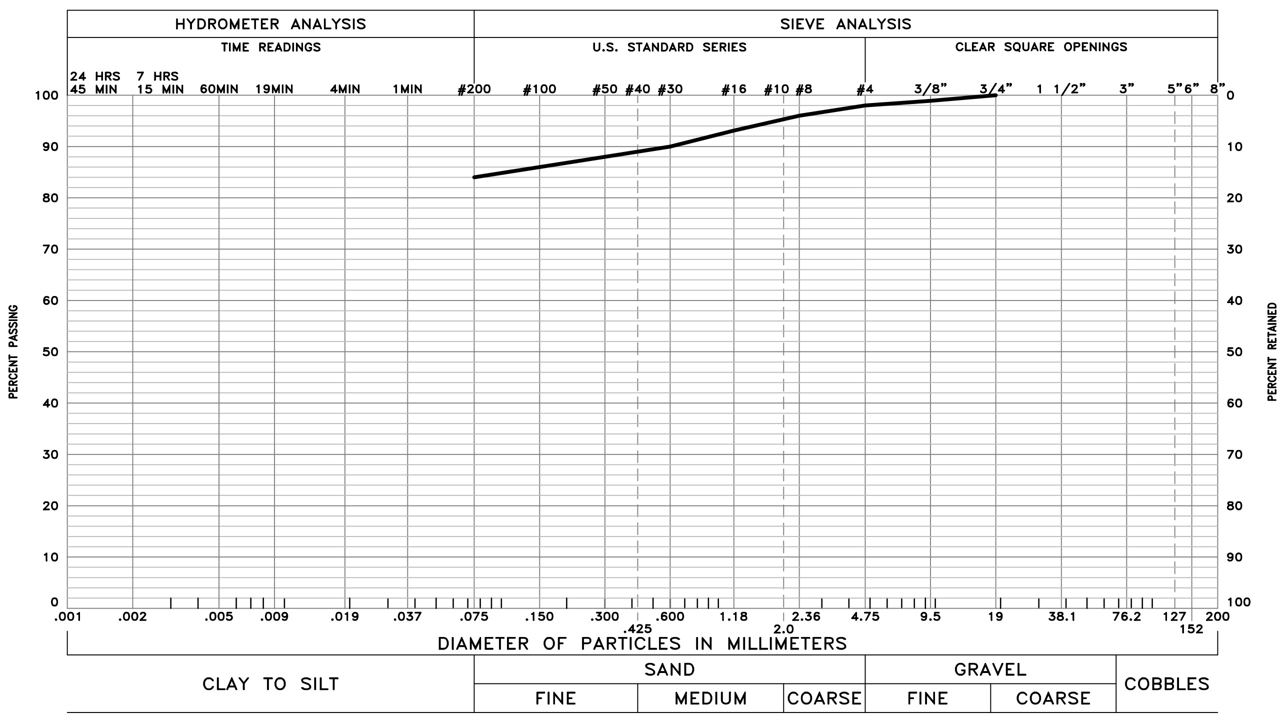
December 08, 2020 − 01:42pm
V:\Projects\2020\20-3-207 King Soopers RidgeGate Preliminary\Drafting\203207-05 to 10.dwg



GRAVEL 14 % SAND 65 % SILT AND CLAY 21 %

LIQUID LIMIT 17 PLASTICITY INDEX 9

SAMPLE OF: Clayey Sand (SC) FROM: Boring 6 @ 14'



GRAVEL 2 % SAND 14 % SILT AND CLAY 84 %

LIQUID LIMIT 49 PLASTICITY INDEX 32

SAMPLE OF: Lean Clay with Sand (CL) FROM: Boring 1-8 @ 0-5'

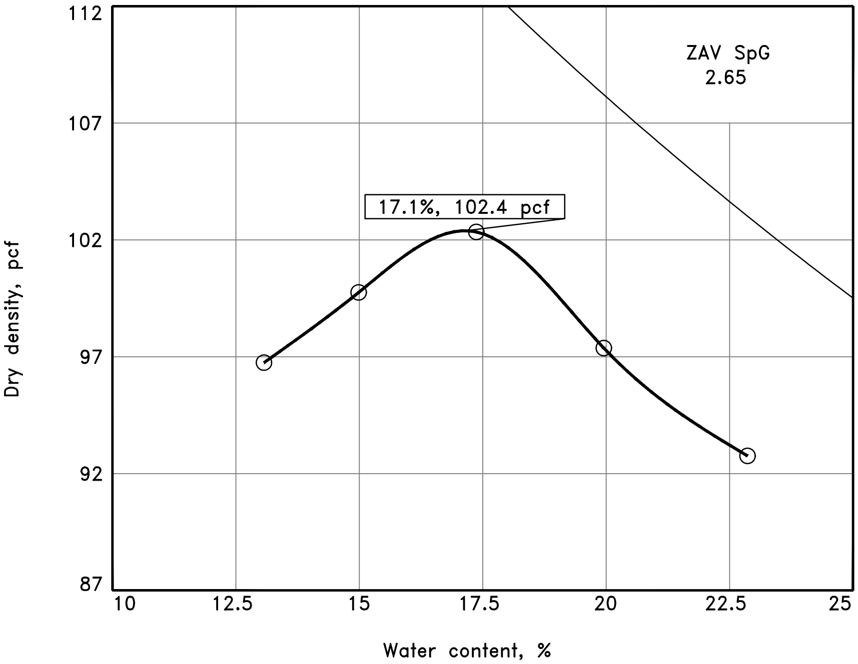
These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D6913, ASTM D7928, ASTM C136 and/or ASTM D1140.

December 08, 2020 - 01:42pm
V:\Projects\2020\20-3-207 King Soopers RidgeGate Preliminary\Drafting\203207-11.dwg

20-3-207	Kumar & Associates	GRADATION TEST RESULTS	Fig. 11

COMPACTION TEST REPORT

Curve No. 2454



Preparation Method	
Rammer: Wt.	5.5 lb.
Drop	12 in.
Type	manual
Layers: No.	three
Blows per	25
Mold Size	0.03322 cu. ft.
Test Performed on Material Passing ___ Sieve	#4
%>#4	2
%<No.200	84
Atterberg (D 4318): LL	49
PI	32
NM (D 2216)	
Sp.G. (D 854)	2.65
USCS (D 2487)	CL
AASHTO (M 145)	A-7-6(27)
Date: Sampled	11/13/20
Received	11/13/20
Tested	11/16/20
Tested By	JS

COMPACTION TESTING DATA
ASTM D 698-12 Method A Standard

	1	2	3	4	5	6
WM + WS	5940.2	6020.2	6101.8	6051.8	6009.1	
WM	4291.8	4291.8	4291.8	4291.8	4291.8	
WW + T #1	295.4	278.1	289.5	301.3	314.3	
WD + T #1	284.5	266.5	267.7	284.3	295.3	
TARE #1	201.1	189.1	142.2	199.1	212.2	
WW + T #2						
WD + T #2						
TARE #2						
MOIST.	13.1	15.0	17.4	20.0	22.9	
DRY DENS.	96.8	99.8	102.3	97.4	92.8	

SIEVE TEST RESULTS
ASTM D 422 ASTM D 1140

Opening Size	% Passing	Specs.
1 1/2"	100	
1"	100	
3/4"	100	
3/8"	99	
#4	98	
#8	96	
#16	93	
#30	90	
#50	88	
#100	86	
#200	84	

TEST RESULTS

Maximum dry density = 102.4 pcf

Optimum moisture = 17.1 %

Material Description

Lean Clay with Sand (CL)

Project No. 20-3-207 Client: Regency Centers

Project: King Soopers Ridge Gate Preliminary

○ Sample Number: 2454

Remarks:
These test results apply only to the samples which were tested. the testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Moisture/density relationships performed in accordance with ASTM D698, D1557. Atterberg limits performed in accordance with ASTM D4318 sieve analysis performed in accordance with ASTM D422, D1140.

Checked by: JH

Title: Lab Deputy

December 08, 2020 – 01:42pm
V:\Projects\2020\20-3-207 King Soopers RidgeGate Preliminary\Drafting\203207-12.dwg

20-3-207	Kumar & Associates	MOISTURE-DENSITY RELATIONSHIPS	Fig. 12

Table I
Summary of Laboratory Test Results

Project No.: 20-3-207
Project Name: Regency Centers Development-Ridge Gate
Date Sampled: November 12, 2020
Date Received: November 13, 2020

Sample Location		Date Tested	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation		Percent Passing No. 200 Sieve	Atterberg Limits		Water Soluble Sulfates (%)	AASHTO Classification (Group Index)	Soil or Bedrock Type
Boring	Depth (Feet)				Gravel (%)	Sand (%)		Liquid Limit (%)	Plasticity (%)			
1	4	11/18/20	13.2	102.8	1	20	79	49	32		A-7-6 (25)	Lean Clay with Sand (CL)
1	14	11/18/20	26.4	96.6								Interbedded Claystone and Sandstone Bedrock
2	1	11/18/20	13.4	111.9	0	14	86	48	29		A-7-6 (26)	Fill: Lean Clay with Sand (CL)
2	19	11/18/20	11.5	101.7								Interbedded Claystone and Sandstone Bedrock
3	1	11/18/20	9.9	110.8	1	22	77	48	30		A-7-6 (22)	Fill: Lean Clay with Sand (CL)
4	4	11/18/20	14.1	106.2	2	23	75	51	31	0.28	A-7-6 (23)	Fat Clay with Sand (CH)
5	4	11/18/20	14.7	112.6	0	24	76	50	30		A-7-6 (22)	Fat Clay with Sand (CH)
6	14	11/18/20	2.1	123.4	14	65	21	17	9		A-2-4 (0)	Clayey Sand (SC)
7	4	11/18/20	11.9	102.0	4	28	68	48	32		A-7-6 (20)	Sandy Lean Clay (CL)
8	1	11/18/20	12.4	101.4	0	21	79	54	33	0.03	A-7-6 (27)	Fat Clay with Sand (CH)
1-8	0-5	11/18/20	17.1*	102.4*	2	14	84	49	32		A-7-6 (27)	Lean Clay with Sand (CL)

* - Optimum moisture content and maximum dry density as determined by standard Proctor (ASTM D 698)