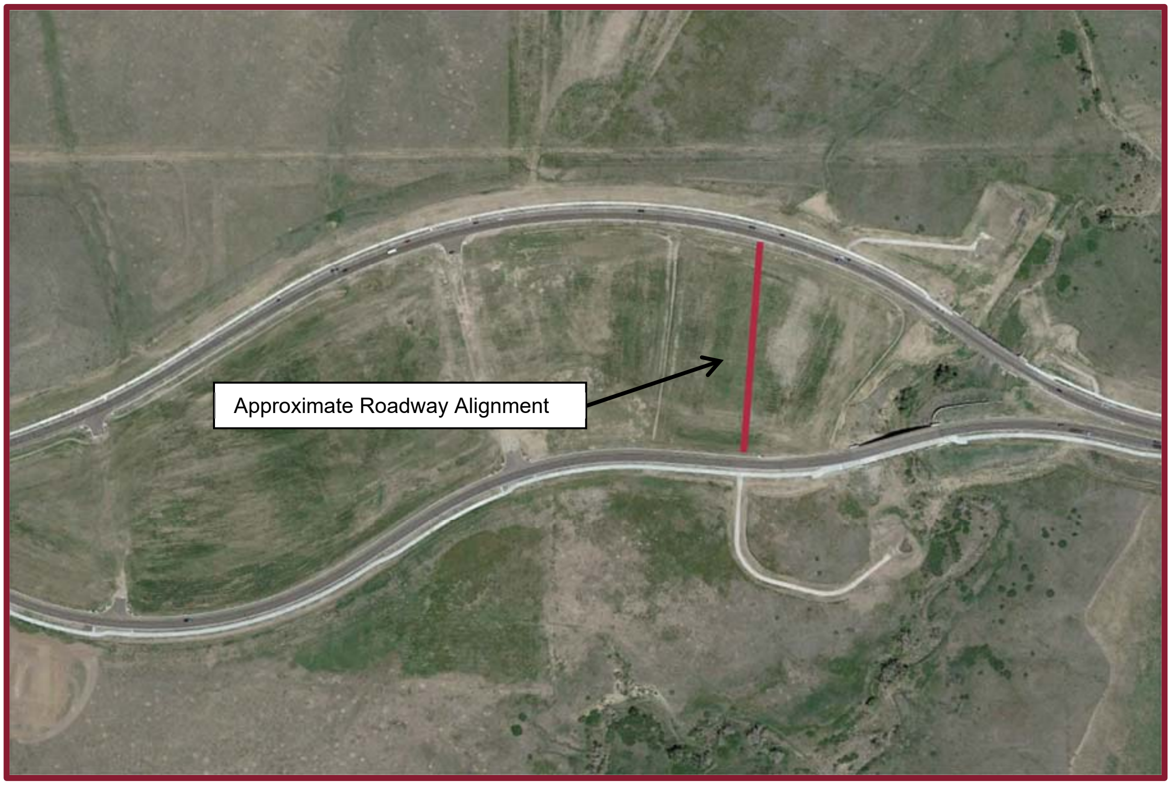
Geotechnical Evaluation
Ridgegate Parkway Couplet Road C
Pavement Sections
Lone Tree, Colorado



Prepared For:

Rampart Range Metropolitan District No. 5

8390 East Crescent Parkway, Suite 300
Greenwood Village, Colorado 80111

Attention: Denise Denslow

Job Number: 23-3031# **September 6, 2023**

41 Inverness Drive East | Englewood, CO 80112 | (303) 289-1989 | www.groundeng.com
ENGLEWOOD | COMMERCE CITY | LOVELAND | GRANBY | GYPSUM | COLORADO SPRINGS

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 Rampart Range Metropolitan District No. 5 in support of the design of the proposed couplet road "C" to connect east and westbound Ridgegate Parkway, approximately 5,500 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. 2307-1381 dated July 12, 2023 and associated contract documents.

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 Rampart Range Metropolitan District No. 5 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 road C is planned for construction to connect east and westbound Ridgegate Parkway; it will have a length of approximately 625 feet. A report[2] previously was prepared by GROUND to address the nearby Couplet Roads A and B. Information from that report was used to inform this report.

It is GROUND's understanding that a flexible pavement section is preferred for this couplet road. If rigid pavement sections 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.

ALIGNMENT CONDITIONS

At the time of our subsurface exploration, the alignment of the couplet road generally crossed an undeveloped plot of land with tall grasses and weeds. Topography of the greater alignment area was generally gently undulating while sloping overall to the northeast, showing approximately 15 feet of relief. Buried utilities were also present along the project alignment. The alignment extended from the westbound reach of Ridgegate Parkway to the eastbound reach of Ridgegate Parkway.

---

[1] Merrick. (June 2023) *Ridgegate East Filing No. 4 – Street and Storm Sewer – Plans – Road C Plan & Profile*
[2] GROUND Engineering Consultants, Inc. (January 2023) *Geotechnical Evaluation – Ridgegate Parkway Couplet Roads – Pavement Sections – Lone Tree, Colorado. Job Number: 22-3023.*

Fill was encountered in the 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 eastbound Ridgegate Parkway from 2019 through 2020. Therefore, fill soils should be anticipated at varying depths along the alignment. 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 July 2023 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. Three test holes were drilled during this exploration. A GROUND professional 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 inner diameter Modified California-type liner sampler. The sampler was 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 Rampart Range Metropolitan District No. 5, 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 T 89 and T 90
- Sulfate Tests — AASHTO T 290
- Swell Tests — ASTM D 4546

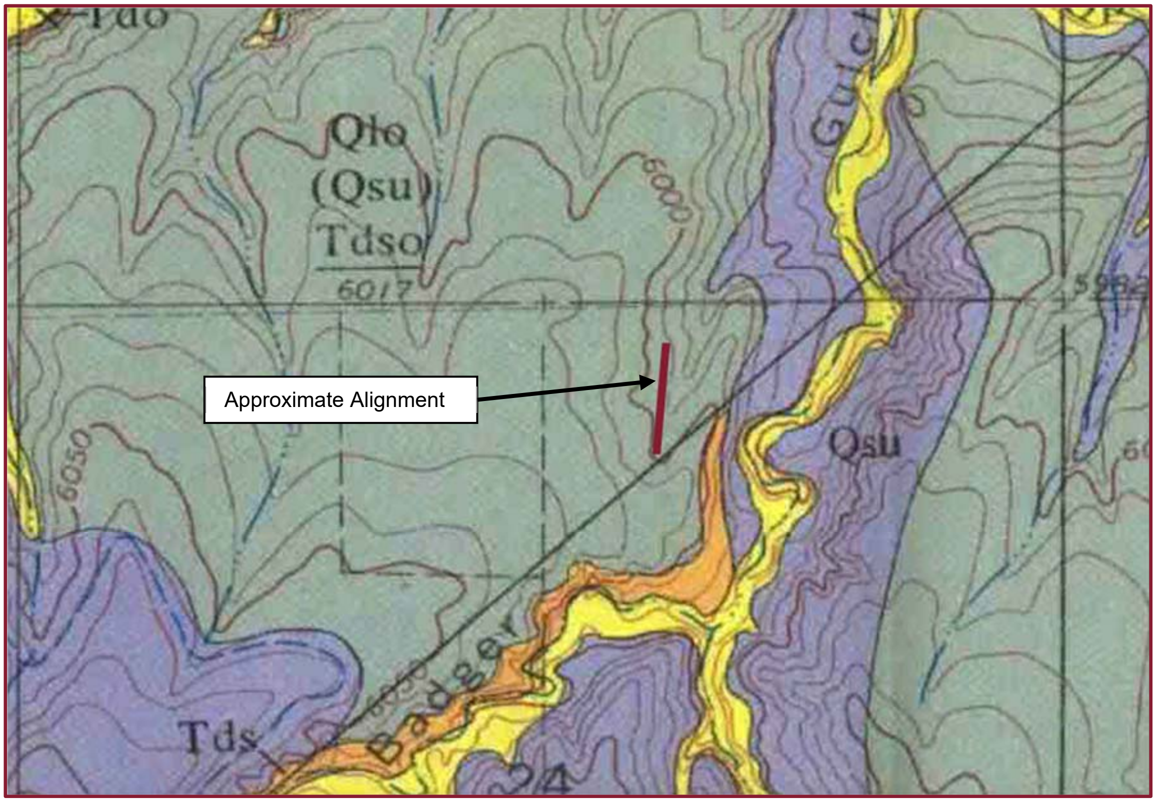
A suite of corrosivity tests also were performed on a selected sample. Results of the laboratory testing program are summarized in Tables 1 and 2.

SUBSURFACE CONDITIONS

Geologic Setting Published geologic maps, e.g., Maberry (1972),[3] 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.

---

[3] 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.



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

The subsurface conditions encountered in the test holes generally consisted of approximately 4 inches of poorly developed topsoil[4] overlying fills soils that were recognized to depths of about 3½ and 6 feet below existing grade. Native silts and clays were encountered beneath the fill, and extended to the depths explored.

[4] "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.

We interpret the fill soils to have been placed during the construction of Ridgegate Parkway and the installation of buried utilities within the lot between the Ridgegate Parkway alignments. The silts and clays are interpreted to be alluvial deposits.

Fill soils were recognized in the test holes, and are likely present elsewhere on site. 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 sampling apparatus used at this site 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. 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. Additional exploration utilizing alternate methods, such as test pits, should be considered if more information is desired.

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

Silts and Clays generally consisted of clays and silts with fine sands and local medium to coarse sands and gravels. They were moderately to highly plastic, dense and very stiff, dry to very moist, and light to brown in color. Caliche was noted commonly.

Swell-Consolidation Testing indicated a potential for heave in the on-site materials. Swells ranging from approximately 4.5 to 9.6 percent were measured upon wetting against a surcharge load approximating in-place overburden pressure. No significant consolidations were measured. 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 Rampart Range Metropolitan District No. 5 desires additional/repeated groundwater level observations, GROUND should be contacted to provide a cost estimate for this additional service.

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 Collector 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 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 clays and silts with sands and sandy clays. These materials were classified predominantly as A-7-6 with local A-6 soils in accordance with the AASHTO classification system, with Group Index values up to 15 in the upper 5 feet.

GROUND collected one composite bulk sample 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 alignment.

The material was compacted to approximately 95 percent of maximum dry density at approximately 3 percent above the optimum moisture content, 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 is often used for fine-grained soils that classify as A-4, A-6, or A-7.

Our testing yielded a resilient modulus value of approximately 10,000 psi for the on-site soils. To account for variability across the project site and our experience in the project area, a resilient modulus value of 6,500 psi was used for the design of the pavement sections.

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 A traffic study[5] was provided to GROUND. In this study, Couplet Road C, planned to be a two-lane residential collector roadway assumed to service multi-family residential, commercial, and business properties, is designated “East Road.” The 2045 total traffic volume of East Road was 3,300 ADT. This value was used to compute the ESAL value listed below.

An ESAL value of 1,033,577 was calculated based on a lane factor of 0.6 for a two-lane roadway and an ADT of 3,300 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 requirements. 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 sections

[5] Kimley Horn (April 2023) *Traffic Impact Study – Ridgegate Couplet Apartments – Lone Tree, Colorado.*

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. It exceeds the Douglas County specified minimum pavement section for a Collector road servicing commercial and residential traffic, 5 inches of asphalt over 8 inches of roadbase. The pavement design calculation is 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 SECTION

Subject Roadway	**Minimum Composite Section** (inches HMA / inches ABC)
Couplet Road C	5½ / 10*

*HMA = Hot-Mix Asphalt, ABC = Aggregate Base Course, * = Section does 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 Douglas County 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, which are reproduced below. 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 reestablish 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 reworking 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 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.

WATER-SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in a select sample of the site soils was approximately 0.04 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.

SOIL CORROSIVITY

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

Reduction-Oxidation testing during this evaluation indicated a red-ox potential of approximately -51 millivolts. Such a low potential typically creates a more corrosive environment.

Sulfide Reactivity testing during this evaluation indicated "trace" results in the local soils. The presence of sulfides in the soils suggests a more corrosive environment.

Soil Resistivity In order to assess the "worst case" for mitigation planning, samples of materials retrieved from the test holes were tested for resistivity in the laboratory, after being saturated with water, rather than in the field. Resistivity also varies inversely with temperature. Therefore, the laboratory measurements were made at a controlled temperature. Measurement of electrical resistivity during this evaluation indicated a value of approximately 29,120 ohm-centimeters in samples of site soils.

pH Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity.[6] Our testing during this evaluation indicated a pH value of 8.0.

Corrosivity Assessment The American Water Works Association (AWWA) has developed a point system scale used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. When the scale equals 10 points or higher, protective measures for ductile iron pipe are indicated. The AWWA scale is presented below.

The soil characteristics refer to the conditions at and above pipe installation depth. We anticipate that drainage at the site after construction will be effective. Nevertheless, based

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

on the values obtained for the soil parameters, the fill and native soils appear to comprise a moderately corrosive environment for ferrous metals (7 points).

If additional information or evaluation is needed regarding soil corrosivity, then the American Water Works Association or a corrosion engineer should be contacted. It should be noted, however, that changes to the site conditions during construction, such as the import of other soils, or the intended or unintended introduction of off-site water, might alter corrosion potentials significantly.

Table A.1 Soil-Test Evaluation

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

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

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 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 reuse 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 reuse.

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 60 percent or less passing the No. 200 Sieve and should have a plasticity index of 15 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 T 180.

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 T 99.

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 T 99.

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.

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 reestablished. 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, even though not encountered in our test holes at the time of exploration, well cemented lenses and beds of bedrock difficult to excavate could be encountered locally. The contractor and project team should anticipate that some of the site bedrock, should project excavation be advanced into 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 Rampart Range Metropolitan District No. 5, 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, unshored excavation slopes up to **10 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 Rampart Range Metropolitan District No. 5 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 Rampart Range Metropolitan District No. 5 as it pertains to design and construction of the proposed couplet road C 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 spring 2024. 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 reevaluation 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 Rampart Range Metropolitan District No. 5. 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 Rampart Range Metropolitan District No. 5, 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. Reuse 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 Rampart Range Metropolitan District No. 5 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

9/06/2023

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

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

GOOGLE EARTH AERIAL IMAGE (06/10/2021)

3

Indicates test hole numbers and approximate locations.

NOT TO SCALE

GROUND ENGINEERING

JOB NO.:	23-3031
FIGURE:	1

LOCATION OF TEST HOLES

LOGS OF THE TEST HOLES

GROUND ENGINEERING

PROJECT: Ridgegate Parkway Couplet Road C

CLIENT: Rampart Range Metropolitan District No. 5

JOB NO: 23-3031

SITE LOCATION: Lone Tree, CO

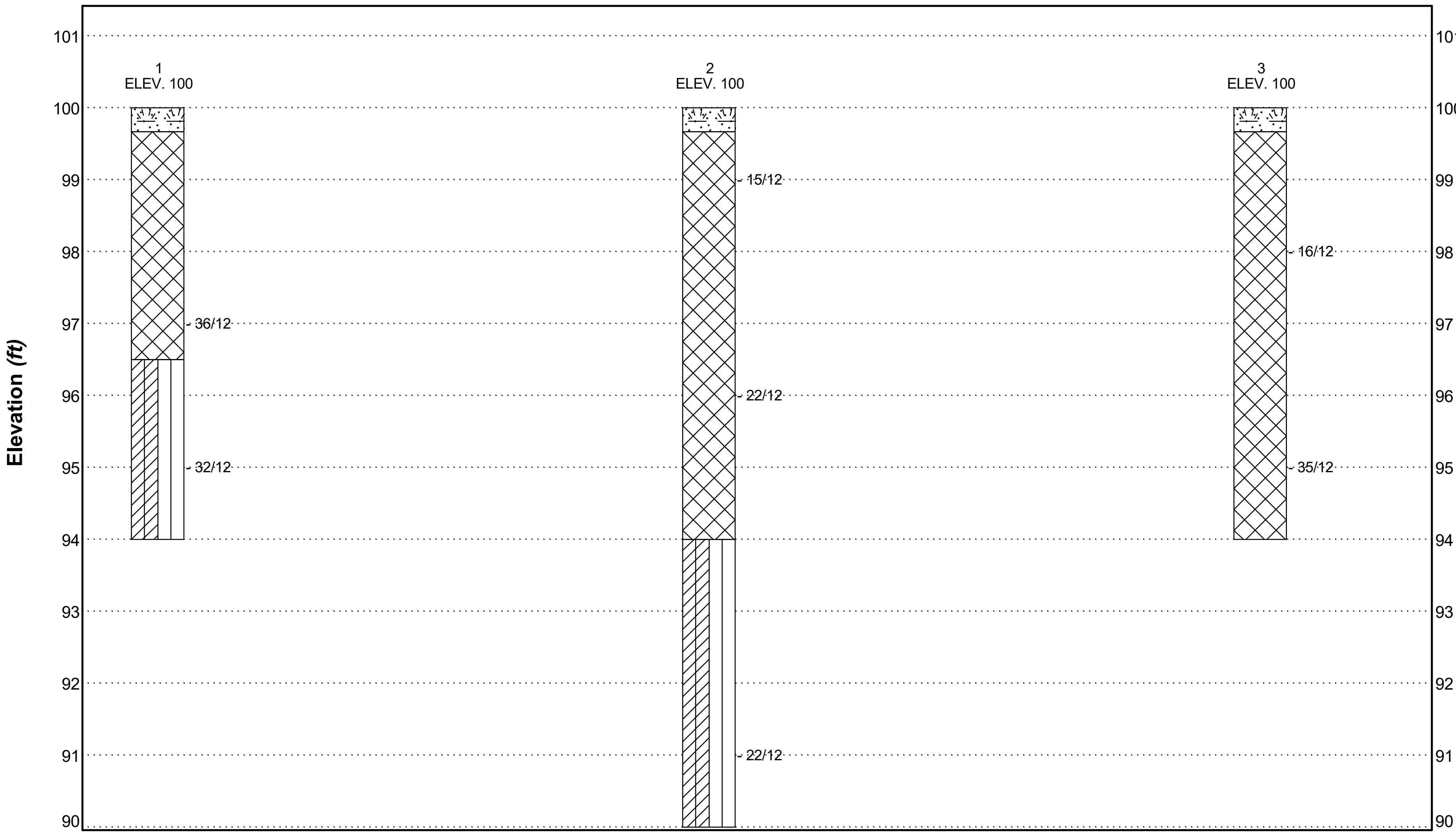


FIGURE: 2

GROUND ENGINEERING

LEGEND AND NOTES

PROJECT: Ridgegate Parkway Couplet Road C

JOB NO: 23-3031

CLIENT: Rampart Range Metropolitan District No. 5

SITE LOCATION: Lone Tree, CO

MATERIAL SYMBOLS

TOPSOIL

FILL

CLAYS and SILTS

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.

NOTES

1. Test holes were drilled on 7/21/2023 with 4" Solid Stem Auger.

2. Locations of the test holes were determined in the field using a hand held GPS device by GROUND.

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

Ridgegate Parkway Couplet Road C

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	3	15.0	113.5	1	31	67.9	45	24	9.6	200	s(CL)	A-7-6 (15)	FILL: Sandy CLAY
2	4	16.2	110.3	0	27	72.6	46	20	6.1	200	(CL)s	A-7-6 (14)	FILL: CLAY with Sand
3	2	18.4	105.8	0	27	73.2	47	20	4.5	200	(CL)s	A-7-6 (15)	FILL: CLAY with Sand
3	5	16.1	112.0	1	25	74.5	39	14	-	-	(CL)s	A-6 (10)	FILL: CLAY with Sand
1 - 3	0 to 5	-	-	-	-	72.8	46	18	-	-	(ML)s	A-7-6 (13)	FILL: SILT with Sand

Job No. 23-3031

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)*			
3	5	0.04	8.0	- 51	Trace	29,120	(CL)s	A-6 (10)	FILL: CLAY with Sand

Job No. 23-3031

Appendix A

Detailed Logs of the Test Holes

TEST HOLE 1

GROUND ENGINEERING

PROJECT: Ridgegate Parkway Couplet Road C **JOB NO:** 23-3031

CLIENT: Rampart Range Metropolitan District No. 5 **SITE 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	0.0		**TOPSOIL:** Approximately 4 inches of topsoil.										
			FILL: Silts and clays with fine sands and local medium to coarse sands and gravels. They were moderately plastic, dense or stiff to hard, dry to very moist, and brown in color. Caliche was noted regularly.										
97.5	2.5												
					36/12	15	113.5	68	45	24	9.6 (200)		s(CL)
			CLAYS and SILTS: Clays and silts with fine sands and local medium to coarse sands and gravels. They were moderately to highly plastic, dense and very stiff, dry to very moist, and light to brown in color. Caliche was noted commonly.										
95.0	5.0												
					32/12								

Bottom of test hole at approx. 6 feet.

GROUND ENGINEERING

TEST HOLE 2

PROJECT: Ridgegate Parkway Couplet Road C **JOB NO:** 23-3031

CLIENT: Rampart Range Metropolitan District No. 5 **SITE 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	0.0		**TOPSOIL:** Approximately 4 inches of topsoil.										
			FILL: Silts and clays with fine sands and local medium to coarse sands and gravels. They were moderately plastic, dense or stiff to hard, dry to very moist, and brown in color. Caliche was noted regularly.		15/12								
97.5	2.5												
					22/12	16.2	110.3	73	46	20	6.1 (200)		(CL)s
95.0	5.0												
			CLAYS and SILTS: Clays and silts with fine sands and local medium to coarse sands and gravels. They were moderately to highly plastic, dense and very stiff, dry to very moist, and light to brown in color. Caliche was noted commonly.										
92.5	7.5												
					22/12								
90.0	10.0												

Bottom of test hole at approx. 10 feet.

TEST HOLE 3

GROUND ENGINEERING

PROJECT: Ridgegate Parkway Couplet Road C

JOB NO: 23-3031

CLIENT: Rampart Range Metropolitan District No. 5

SITE 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	0.0		**TOPSOIL:** Approximately 4 inches of topsoil.										
			FILL: Silts and clays with fine sands and local medium to coarse sands and gravels. They were moderately plastic, dense or stiff to hard, dry to very moist, and brown in color. Caliche was noted regularly.										
97.5	2.5				16/12	18.4	105.8	73	47	20	4.5 (200)		(CL)s
95.0	5.0				35/12	16.1	112	74	39	14			(CL)s

Bottom of test hole at approx. 6 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

GROUND Job No. 23-3031

Couplet Road C

Composite Pavement Section

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	1,033,577
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	6,500 psi
Stage Construction	1
Calculated Design Structural Number	3.55 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	5.5	-	2.42
2	Roadbase	0.12	1	10	-	1.20
Total	-	-	-	15.50	-	3.62

Geotechnical Evaluation
Ridgegate East Infrastructure Improvements
Ridgegate Parkway Widening Pavements
Lone Tree, Colorado
Revised

Prepared For:

Merrick & Company

5970 Greenwood Plaza Boulevard
Greenwood Village, Colorado 80111

Attention: Brian Poling

Job Number: 17-3053A **May 30, 2018**

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 design of the proposed Ridgegate Parkway Widening project in Lone Tree, Colorado. Our study was conducted in general accordance with a portion of GROUND's Proposal No. 1704-0725, dated August 22, 2017.

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

We understand that proposed construction will consist of widening Ridgegate Parkway between Havana Street and the eastern city limit of the City of Lone Tree in that area. The widened road will include the addition of four lanes to the parkway for a total of six lanes, three in each travel direction. In the eastern portion of the alignment, we understand that additional, eastbound lanes will separate from the existing alignment and follow a new alignment to the south of the exiting alignment and reconnect with existing alignment near the City of Lone Tree city limit. Additionally turn lanes, bike lanes, sidewalks, landscaping, and curb and gutter are also planned for the project. We also understand that significant cuts and fills, on the order of 10 feet may be required to achieve project lines and grades.

The widening project also will include two new bridges at the Happy Canyon crossing and the Badger Gulch crossing. Geotechnical parameters and considers will be provided in separate reports.

Performance Expectations Based on our experience with similar facilities in the project area, we assume that post-construction movements on the order of 2 to 3 inches are acceptable and anticipated for project improvements. GROUND will be available to discuss the risks and remedial approaches outlined in this report, as well as other potential approaches, upon request if post-construction movements of these magnitudes are not acceptable and anticipated.

ALIGNMENT CONDITIONS

The project alignment extends eastward approximately 4,600 feet along the north side of existing Ridgegate Parkway alignment. At this point, it splits from the existing alignment to south and continues eastward another approximately 4,800 feet to a point where it connects back to the existing alignment Ridgegate Parkway alignment. From that point the new alignment continues to approximately 300 feet to the City of Lone Tree city limit.

At the time of our surface exploration, the proposed alignment crossed a variety of terrain and development types; however, much of the project alignment was largely undeveloped and had be used as ranch land. The ground slope was variable in steepness and direction, but the greater project area generally slopes to the east and north. Relatively steep slopes were observed along the Ridgegate Parkway roadway embankment and at stream and drainage crossings. The ground along the project alignment generally supported native grasses and weeds and shrubs and other similar vegetation was observed locally. Mature trees were observed at the stream and drainage crossings.

SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted in November, 2017. Thirty-four (34) test holes were drilled using a track- or truck-mounted drill rig advancing continuous flight auger equipment to evaluate the subsurface conditions as well as to retrieve soil samples for laboratory testing and analysis. The test holes were advanced to depths between approximately 13 to 16 feet below existing grade near the approximate proposed alignment. A GROUND engineer directed the subsurface exploration, logged the test hole in the field, and prepared the soil samples for transport to our laboratory.

Samples of the subsurface materials were retrieved with a 2-inch I.D. California liner sampler. 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 obtained and associated penetration resistance values are shown on the test hole log.

The project vicinity is shown in Figure 1 and the approximate locations of the test holes are shown in Figures 1A and 1B. Logs of the test holes are presented in Figures 2 through 5. Explanatory notes and a legend for the logs are provided in Figure 6.

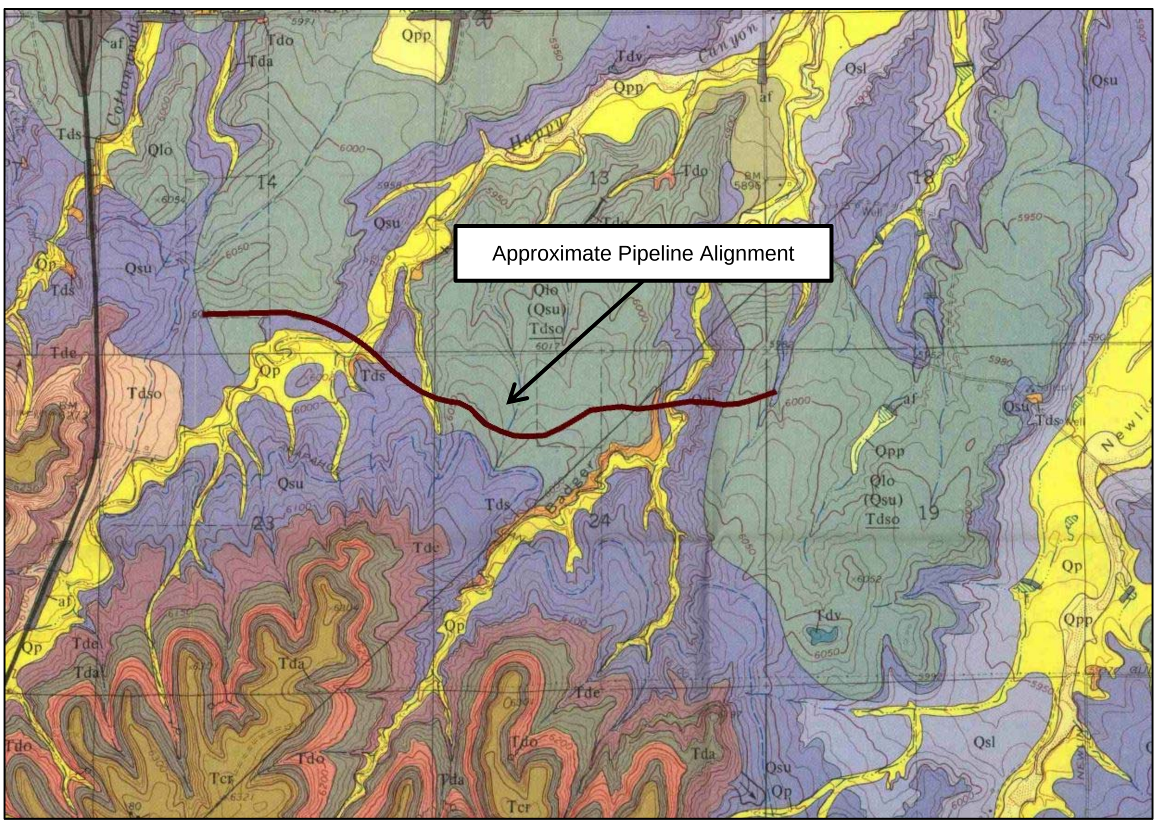
LABORATORY TESTING

Samples retrieved from our test hole were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil samples obtained from the subject site included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, and Atterberg limits. Swell and consolidation, water-soluble sulfate, and corrosivity tests were completed on selected samples of the soils, as well. Resilient modulus testing was performed on two composite samples retrieved from the test holes. Laboratory tests were performed in general accordance with applicable ASTM and AASHTO protocols. Results of the laboratory testing program are summarized on Tables 1 and 2. A gradation plot is provided on Figure 7.

SUBSURFACE CONDITIONS

Regional Geology

Published maps such Maberry and Lindvall (1972)[1] and depict the alignment and vicinity as underlain by Holocene Piney Creek Alluvium (**Qp**), Pleistocene Slocum Alluvium (**Qsl** and **Qsu**), and Pleistocene Loess (**Qlo**). A portion of that geologic map is reproduced below.



[1] 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, scale 1:24,000.

In the project area, alluvial deposits typically consist of fine to coarse sands, gravels, cobbles, and boulders with varying volumes of silts, clays. The cobbles and boulders in these deposits can often be awkward or difficult to handle. Loess deposits typically consist of fine sands and silts with varying fractions of clays. Weathering typically increases the clay contents in these deposits. Additionally, the eolian (wind-blown) deposits, such as loess, often are subject to hydro-consolidation or 'collapse.'

These surficial deposits are mapped as underlain by the Paleocene Dawson Formation sandstones and claystones (**Tdso** and **Tds**). Parts of the alignment are also mapped as being underlain by the Dawson Formation. Siltstones and conglomerates are also present in the Dawson Formation. This formation includes well cemented beds that can be very hard and difficult to excavate. Additionally the claystones and siltstones in the Dawson Formation can be moderately to highly expansive.

Subsurface Conditions Typically, the test holes penetrated approximately 2 to 6 inches of topsoil[2] before encountering native sands, silts, and clays that extended to depths between 2 and 15 feet below existing grade or to depths explored in the test holes. Sandstone and siltstone bedrock was encountered in underlying the native soils in test holes P-1, P-6, P-9 through P-22, P-25, P-26, P-28 and P-30 and extended to the depths explored. The upper several feet of bedrock was commonly weathered. Similarly in test holes P-13, P-22, and P-28 bedrock materials with comparably low penetration resistance values (fewer than 50 blows for 12 inches of penetration) where encountered between zones of comparably high penetration resistance values (50 blows for fewer than 12 inches of penetration). These zones have been indicated on the test hole logs as weathered bedrock. Additionally fill soils were recognized in Test Holes P-1, P-4, P-8, P-9, P-33, and P-34 and extended to depths between 3 to 10 feet below existing grade.
We interpret the fill soils to be materials placed during the construction of the roadway alignments. We interpret the native sands, silts, and clays to be interbedded colluvial and alluvial deposits. We interpret the sandstone, claystone, and siltstone bedrock to be Dawson Formation materials.

Given the existing development along the alignment, fill soils, in addition to those encountered in test holes noted above, likely are present elsewhere along the alignment;

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

particularly the areas along the existing Ridgegate Parkway alignment. Delineation of the complete lateral and vertical extents of fills at the site, or their compositions, however, was beyond our present scope of services. If fill soil volumes and compositions at the site are of significance, they should be evaluated by the contractor using test pits.

Similarly, coarse gravel, cobbles, and larger clasts (e.g. boulders) are not well represented in small diameter liner samples collected from the test holes. Therefore, such materials may be present even where not called out in the material descriptions herein.

Fill consisted of silts, clays, and fine to coarse sands with gravels locally. They were non- to highly plastic, medium dense or stiff or very stiff, dry to very moist, and light to dark brown in color.

Sands, Silts, and Clays consisted of silty to clayey, fine to coarse sands, silts, and clays with local gravels. They were non- to highly plastic, medium dense to very dense or stiff to very stiff, dry to wet, and red brown and pale to dark brown in color. Iron staining and caliche were observed commonly.

Weathered Sandstone, Siltstone, and Claystone consisted of interbedded, fine to medium grained, weathered sandstone, siltstone, and claystone. They were slightly to highly plastic, firm to medium hard, dry to wet, and green brown, brown, and gray in color. Iron staining was observed commonly.

Sandstone, Siltstone, and Claystone consisted of interbedded, fine to medium grained, sandstone, siltstone, and claystone. They were slightly to highly plastic, hard to very hard, dry to wet, and green brown, red brown, brown, gray, and blue in color. Iron staining and caliche were observed commonly.

Swell-Consolidation Testing indicated swells up to 7 percent or more, and consolidations up to nearly 1 percent in samples of alignment soils against various surcharge loads approximating in-place overburden pressures. (See Table 1.)

Groundwater was not encountered at the depths explored. The test holes were backfilled immediately after drilling operations due to safety concerns. Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, and the development of transient, perched water conditions. At this site, we anticipate that groundwater levels will rise and fall, in general, with the stages of the Happy Canyon Creek, Badger Gulch, other relatively minor drainages.

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. The standard care of practice in pavement design describes the flexible pavement section as a "20-year" design pavement; however, most flexible pavements will not remain in satisfactory condition without routine maintenance and rehabilitation procedures performed throughout the life of the pavement. Pavement sections for the roadway were developed in general accordance with the design guidelines and procedures of American Association of State Highway and Transportation Officials (AASHTO) and CDOT specifications.

Subgrade Materials Based on the results of our field and laboratory studies, the subgrade materials at the subject site consisted largely of silts, sands, and clays. These materials were classified largely as A-2-7 and A-7-6 soils in accordance with the AASHTO classification system, with Group Index values of 0 to 19. These soils typically provide fair to poor subgrade support.

Resilient modulus testing was performed on two composite samples of site soils. This testing yielded results of 9,837 psi and 6,934 psi. To account for variability across the project site, a resilient modulus value of 6,700 psi was used to develop the pavement sections.

It is important to note that significant decreases in soil support 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 GROUND Engineering was provided with a transportation analysis for Ridgegate East development performed by Felsburg, Holt, & Ullevig (FHU) [3] was provided to GROUND for use in developing the pavement sections. In that document, Ridgegate Parkway was shown to have an anticipated traffic of more than 16,000 vehicles per day per direction once the greater development is completed. Traffic counts available on the Denver Regional Council of Governments' (DRCOG) Regional Traffic Counts map[4] indicated single day traffic counts of 14,114 vehicles in February 2011 and 23,309 in April 2014 at location on Ridgegate Parkway east of Havana Street. This increase corresponds to a growth rate of approximately 16.7 percent per year. Given the history of the development in the greater project area, we anticipate this growth rate is higher than will be experienced from 2014 to 2018 and that a growth rate of 8 percent is appropriate to estimate the growth in traffic volume for this period. Similarly, we anticipate that traffic volume growths will further slow, and that an annual increase of 0.957 percent a year is appropriate for estimating traffic increases during the design life of the road.

To estimate current traffic volumes a growth rate of 8 percent per year was applied to a traffic count performed by the Douglas County Department of Public Works Engineering on June 19, 2014 at location approximately 2,500 feet east of Peoria Street. That count yielded a total traffic count of 21,087 vehicles in all lanes. Applying the 8 percent a year growth factor yielded a traffic count of 28,630 for all travel lanes. Based data available for similar roads in the greater project area, we estimate truck traffic makes up approximately 3.3 percent of the total traffic. Therefore, we estimated the current AADTT (Average Annual Daily Truck Traffic) to be 945 and this value was used in the pavement section calculations. CDOT Level 3 Traffic Cluster 2 was the assumed traffic mix with a 6-lane roadway and an operational speed of 45 mph.

The traffic loading values should be evaluated by City of Lone Tree and the project team to determine that they are acceptable for both current and future traffic on the roadway. Without accurate traffic loading information, the pavement sections indicated herein may

---

[3] Felsburg Holt & Ullevig, 2016, *RidgeGate East Ransportaion Anaylsis*, FHU Reference No. 16-231-01 dated September 28.
[4] Denver Regional Council of Governments, 2017, DRCOG Regional Traffic Counts, http://gis.drcog.org/trafficcounts/, accessed on 12/21/17.

be insufficient to support present and future traffic volumes. Premature deterioration of pavement including cracking and other distress may result.

If the traffic loadings utilized above differ significantly from actual values, GROUND should be notified to reevaluate the pavement sections.

Pavement Sections Pavement sections for the widening of the Ridgegate Parkway widening project were based on the CDOT 2017 M-E Pavement Design Manual utilizing the AASHTOWare Pavement M-E design software. The following table presents the pavement section for the Ridgegate Parkway from the Havana Street to City of Lone Tree city limit. Details of the 20-year flexible pavement sections ME calculations for Ridgegate Parkway are attached in Appendix A.

Composite Flexible Minimum Pavement Section (20-year design)

Layer Type	Material Type	Thickness (inches)
Flexible	**R6 Level 1 SX(100) PG 76-28**	**2**
Flexible	**R1 Level 1 S(100) PG 64-22**	**4½**
Non-Stabilized Base	**CDOT Class 6**	**10***
Subgrade	**Existing Sands, Silts, and Clays Placed as Fill**	**36***

*Properly Moisture Conditioned and Compacted

Pavement/Subgrade Properties

Hot Bituminous Asphalt (HBA): The asphalt pavement shall consist of a bituminous plant mix composed of a mixture of high quality aggregate and bituminous material, which meets the requirements of a job-mix formula established by a qualified engineer. The asphalt material used should be based on a SuperPave Gyratory Design Revolution (N_{DES}) of 100 for the lower lift(s) and SuperPave Gyratory Design Revolution (N_{DES}) of 100 for surface layer. Grading S is acceptable for the lower lift(s) using PG 64-22 asphalt cement binder and grading SX is acceptable for the surface layer using PG 76-28 asphalt cement binder. Note that the provided pavement binders could be adjusted depending on the market condition at the time of construction. Alternate binding types should be submitted by the contractor for review and approval by the City of Lone Tree and the project team prior to construction. Generally, pavement lift thicknesses should be between 2¼ to 3½ inches (S) for the lower lift(s), depending on the material type selected, and 2 inches for the top lift (SX).

Subgrade Preparation Although subgrade preparation to a depth of 12 inches is typical in the project area, pavement performance commonly can be improved by a greater depth of moisture-density conditioning of the soils. Soil moisture contents beneath existing pavements are commonly elevated and the contractor should be prepared to prepare the subgrade as outlined herein even where elevated subgrade moisture contents are encountered beneath the existing pavements.

Remedial Earthwork Shortly before paving, the pavement subgrade should be excavated and/or scarified to a depth of **36 inches or more**, moisture-conditioned and properly re-compacted. In certain areas, such as areas with unsuitable, existing fill materials or areas with very wet soils near drainages, it may be beneficial to deepen the fill section. Where existing utilities or hard, well cemented bedrock is present and remedial earthwork to 36 inches is impractical, then remedial earthwork should extend to the deepest practical depth. It also may be beneficial to remove relatively large cobbles for the subgrade soils.

Subgrade preparation should extend the full width of the pavement from back-of-curb to back-of-curb. Where existing utility or other improvements, limit the depth of earth practical, the fill section may be thinned provided that an increased risk of post-construction movement is accepted.

Criteria and standards for fill placement and compaction are provided in the *Project Earthwork* section of this report. The contractor should be prepared either to dry the subgrade materials or to moisten them, as needed, prior to compaction.

Where adequate drainage cannot be achieved or maintained, excavation and replacement should be undertaken to a greater depth and in addition to the edge drains/ underdrain included.

Proof Rolling 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. Establishment of a firm paving platform (as indicated by proof rolling) is an additional requirement beyond proper fill placement and compaction. It is possible for soils to be compacted within the limits indicated in the *Project Earthwork* section of this report and fail proof rolling, particularly in the upper range of specified moisture contents. Where a relatively thin pavement section is placed, it is particularly important to achieve a firm, stable platform on which to pave.

Additional Considerations 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. Where topography, site constraints, or other factors limit or preclude adequate surface drainage, pavements should be provided with edge drains to reduce loss of subgrade support. The long-term performance of the pavement also can be improved greatly by proper backfilling and compaction behind curbs, gutters, and sidewalks so that ponding is not permitted and water infiltration is reduced.

In our experience, the wetting of the subgrade soils commonly leads to loss of subgrade support for the pavement with resultant accelerating distress, loss of pavement life and increased maintenance costs. Heavy vehicle traffic over wetted subgrade commonly results in rutting and pushing of flexible pavements, and cracking of rigid pavements. Where the subgrade soils are expansive, wetting also typically results in increased pavement heave. In relatively flat areas where design drainage gradients necessarily are small, subgrade settlement or heave can obstruct proper drainage and yield increased infiltration, exaggerated distress, etc. (These considerations apply to project flatwork, as well.)

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

Most pavements will not remain in satisfactory condition without regular maintenance and rehabilitation procedures performed throughout the life of the pavement. Maintenance and rehabilitation measures preserve, rather than improve, the structural capacity of the pavement structure. Therefore, an effective program of regular maintenance should be developed and implemented to seal cracks, repair distressed areas, and perform thin overlays throughout the lives of the pavements. The greatest benefit of pavement overlaying will be achieved by overlaying sound pavements that exhibit little or no distress.

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

PEDESTRIAN FLATWORK

We understand the proposed Ridgegate East Infrastructure Improvements project will include pedestrian flatwork and may include a concrete paved bike path. Like other site improvements, flatwork will experience post-construction movements as soil moisture contents increase after construction and distress likely will result. The following measures will help to reduce damages to these improvements, but will not prevent all movements.

1) Remedial earthwork to prepare flatwork subgrades is subject to the same factors discussed in the *Pavement Sections* section of this report, and should be undertaken to the same depth.

2) Regardless of the depth of subgrade preparation, due to the collapse and swell potentials at this site, greater than typical maintenance, including the removal and replacement of portions of flatwork, should be anticipated for project flatwork. Greater depths of subgrade preparation will tend to reduce the extent and frequency of extra maintenance, however.

3) Prior to placement of flatwork, a proof roll should be performed to identify areas that exhibit instability and deflection. The deleterious soils in these areas should be removed and replaced with properly compacted fill. The contractor should take care to achieve and maintain compaction behind curbs to reduce differential sidewalk settlements. Passing a proof roll is an additional requirement to placing and compacting the subgrade fill soils within the specified ranges of moisture content and relative compaction in the *Project Earthwork* section of this report. Subgrade stabilization may be cost-effective in this regard.

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

5) In no case should exterior flatwork extend to under any portion of the building where there is less than **3 inches** of vertical clearance between the flatwork and any element of the building. Exterior flatwork in contact with brick, rock facades, or any other element of the building can cause damage to the structure if the flatwork experiences movements.

Concrete Scaling Climatic conditions in the project area including relatively low humidity, large temperature changes and repeated freeze – thaw cycles, make it likely that project sidewalks and other exterior concrete will experience surficial scaling or spalling. The likelihood of concrete scaling can be increased by poor workmanship during construction, such as ‘over-finishing’ the surfaces. In addition, the use of de-icing salts on exterior concrete flatwork, particularly during the first winter after construction, will increase the likelihood of scaling. Even use of de-icing salts on nearby roadways, from where vehicle traffic can transfer them to newly placed concrete, can be sufficient to induce scaling. Typical quality control / quality assurance tests that are performed during construction for concrete strength, air content, etc., do not provide information with regard to the properties and conditions that give rise to scaling.

We understand that some municipalities require removal and replacement of concrete that exhibits scaling, even if the material was within specification and placed correctly. The contractor should be aware of the local requirements and be prepared to take measures to reduce the potential for scaling and/or replace concrete that scales.

In GROUND's experience, the measures below can be beneficial for reducing the likelihood of concrete scaling. It must be understood, however, that because of the other factors involved, including weather conditions and workmanship, surface damage to concrete can develop, even where all of these measures were followed. Also, the mix design criteria should be coordinated with other project requirements including criteria for sulfate resistance presented in the *Water-Soluble Sulfates* section of this report.

1) Maintaining a maximum water/cement ratio of 0.45 by weight for exterior concrete mixes.

2) Include Type F fly ash in exterior concrete mixes as 20 percent of the cementitious material.

3) Specify a minimum, 28-day, compressive strength of 4,500 psi for all exterior concrete.

4) Including 'fibermesh' in the concrete mix also may be beneficial for reducing surficial scaling.

5) Cure the concrete effectively at uniform temperature and humidity. This commonly will require fogging, blanketing and/or tenting, depending on the weather conditions. As long as 3 to 4 weeks of curing may be required, and possibly more.

6) Avoid placement of concrete during cold weather so that it is not exposed to freeze-thaw cycling before it is fully cured.

7) Avoid the use of de-icing salts on given reaches of flatwork through the first winter after construction.

We understand that commonly it may not be practical to implement some of these measures for reducing scaling due to safety considerations, project scheduling, etc. In such cases, additional costs for flatwork maintenance or reconstruction should be incorporated into project budgets.

Frost and Ice Considerations Nearly all soils other than relatively coarse, clean, granular materials are susceptible to loss of density if allowed to become saturated and exposed to freezing temperatures and repeated freeze – thaw cycling. The formation of ice in the underlying soils can result in heaving of pavements, flatwork, and other hardscaping ("ice jacking") in sustained cold weather up to 2 inches or more. This heaving can develop relatively rapidly. A portion of this movement typically is recovered when the soils thaw, but due to loss of soil density, some degree of displacement will remain. This can result even where the subgrade soils were prepared properly.

Where hardscape movements are a design concern, replacement of the subgrade soils with 3 or more feet of clean, coarse sand or gravel should be considered or supporting the element on foundations similar to the building and spanning over a void. Detailed guidance in this regard can be provided upon request. It should be noted that where such open graded granular soils are placed, water can infiltrate and accumulate in the subsurface relatively easily, which can lead to increased settlement or heave from factors unrelated to ice formation. Therefore, where a section of open graded granular soils are placed, a local underdrain system should be provided to discharge collected water. GROUND will be available to discuss these concerns upon request.

WATER-SOLUBLE SULFATES

The concentrations of water-soluble sulfates measured in selected samples obtained from the test hole were generally between 0.01 percent and 0.18 percent. However, a concentration of 1.00 percent was measured. (Table 2). Such concentrations of water-soluble sulfates represent a **severe** 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, sulfate-resistant cement should be used in all concrete exposed to site soil and bedrock, conforming to one of the following Class 2 requirements:

(1) ASTM C 150 Type V with a minimum of a 20 percent substitution of Class F fly ash by weight

(2) ASTM C 150 Type II or III with a minimum of a 20 percent substitution of Class F fly ash by weight. The Type II or III cement shall have no more than 0.040 percent expansion at 14 days when tested according ASTM C 452

(3) ASTM C 1157 Type HS; Class C fly ash shall not be substituted for cement.

(4) ASTM C 1157 Type MS plus Class F fly ash where the blend has less than 0.05 percent expansion at 6 months or 0.10 percent expansion at 12 months when tested according to ASTM C 1012.

(5) A blend of Portland cement meeting ASTM C 150 Type II or III with a minimum of 20 percent Class F fly ash by weight, where the blend has less than 0.05 percent expansion at 6 months or 0.10 percent expansion at 12 months when tested according to ASTM C 1012.

(6) ASTM C 595 Type IP(HS); Class C fly ash shall not be substituted for cement.

When fly ash is used to enhance sulfate resistance, it shall be used in a proportion greater than or equal to the proportion tested in accordance to ASTM C 1012, shall be the same source, and it shall have a calcium oxide content no more than 2.0 percent greater than the fly ash tested according to ASTM C 1012.

All concrete exposed to site soil and bedrock should have a minimum compressive strength of 4,500 psi.

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

To define more precisely the areas where sulfate-resistant cement may not be needed, additionally testing will need to be performed. Depending on the resolution desired, additional test holes may be necessary.

SOIL CORROSIVITY

The degree of risk for corrosion of metals in contact with soils commonly is considered to be in two categories: corrosion in undisturbed soils and corrosion in disturbed soils. The potential for corrosion in undisturbed soil is generally low, regardless of soil types and conditions, because it is limited by the amount of oxygen that is available to create an electrolytic cell. In disturbed soils, the potential for corrosion typically is higher, but is strongly affected by soil conditions for a variety of reasons, but primarily soil chemistry.

A corrosivity analysis was performed to provide a general assessment of the potential for corrosion of ferrous metals installed in contact with earth materials at the site, based on the conditions existing at the time of GROUND's evaluation. Soil chemistry and physical property data including pH, reduction-oxidation (redox) potential, sulfides, and moisture content were obtained. Test results are summarized in Table 2.

Soil Resistivity In order to assess the "worst case" for mitigation planning, a sample of materials retrieved from the test hole were tested for resistivity in the laboratory, after being saturated with water, rather than in the field. Resistivity also varies inversely with temperature. Therefore, the laboratory measurements were made at a controlled temperature. Measurement of electrical resistivity in selected samples indicated values of between approximately 1,146 and 2,264 ohm-centimeters.

pH Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity[5]. Testing indicated pH values of between approximately 7.3 and 8.0.

Reduction-Oxidation testing indicated re-dox potentials of approximately -48 and -94 millivolts. Low potentials typically create a more corrosive environment.

Sulfide Reactivity testing indicated “trace” results in the site soils. The presence of sulfides in the soils suggests a more corrosive environment.

The American Water Works Association (AWWA) has developed a point system scale used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. When the scale equals 10 points or higher, protective measures for ductile iron pipe should be used. The AWWA scale is presented below. The soil characteristics refer to the conditions at and above pipe installation depth.

We anticipate that drainage at the site after construction will be good. Nevertheless, based on the values obtained for the soil parameters, the overburden soils/bedrock appear(s) to comprise a **severely corrosive environment** for ferrous metals (about 20 points).

If additional information is needed regarding soil corrosivity, the American Water Works Association or a corrosion engineer should be contacted. It should be noted, however, that changes to the site conditions during construction, such as the import of other soils, or the intended or unintended introduction of off-site water, might alter corrosion potentials significantly.

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

Table A.1 Soil-test Evaluation [6]

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

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

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

PROJECT EARTHWORK

The considerations provided below were based on our interpretation of the geotechnical considerations at the site. Where they conflict with applicable City of Lone Tree, Douglas County, or other agency specifications, the latter should be considered to govern.

General Considerations Prior to earthwork construction, existing asphalt/concrete, 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.

Existing Fill Soils Fill materials were encountered in Test Holes P-1, P-4, P-8, P-9, P-33, and P-34 and fill materials likely are present elsewhere along the alignment; particularly along the existing Ridgegate Parkway alignments. Because the contents and composition of all fill materials are not known, some excavated fill materials may not be suitable for replacement as backfill. However, we anticipate that the major of these soils will suitable for reuse, provided they are free of deleterious materials. A geotechnical engineer should be retained during site excavations to observe the excavated fill materials and provide information for its suitability for reuse.

Use of Existing Native Soils Native site soils that are free of trash, organic material, construction debris, and other deleterious materials are suitable, in general, for placement as compacted fill. Organic materials should not be incorporated into project fills.

Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) 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 information for usage of such materials on a case-by-case basis when such materials have been identified during earthwork. Standard specifications 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 alignment as common fill, then the imported soils should be free of organic material, and other deleterious materials. **Imported material should consist of soils that have less than 50 percent passing the No. 200 Sieve and should have a plasticity index of less than 15.** 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. Trenches where pipe bedding is to be placed, this scarification and re-compaction is not necessary.

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.

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.

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

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

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 provided ranges are obtained.

Settlements Settlements will occur in filled ground, typically on the order of 1 to 2 percent of the fill depth. 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 will typically take place during earthwork construction, provided the contractor achieves the compaction levels provided 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 site slopes supported by on-site soils up to 10 feet in height may 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.

EXCAVATION CONSIDERATIONS

Excavation Difficulty The test holes were advanced to the depths indicated on the test hole logs by means of conventional truck-mounted drilling equipment. Therefore it appears likely that along significant reaches of the alignment, excavations to relatively shallow depths can be made without unusual excavation difficulties with conventional, heavy duty excavations equipment in good working condition.

However, large cobbles and boulders may be encountered in project excavations, primarily near the existing drainages. Such materials may be awkward or difficult to handle. The contractor should be prepared to handle such materials.

Additionally, at least locally and possibly for extended lengths of the alignment, the contractor should expect to excavate very hard bedrock that may be encountered at shallow depths in project excavations. Penetration resistance values as high as 50 blows for 0 inches of penetration were encountered at depths as shallowly as 9 feet below existing grades. Very hard and difficult to excavated bedrock is common in project area.

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 Merrick and Company, 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 **1½ : 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. We assume that the site will be de-watered effectively prior to excavation. The risk of slope instability will be increased significantly in areas of seepage along excavation slopes. The contractor should monitor de-watering operations and observe the slopes regularly for indications of seepage or instability.

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.

Additionally, the project alignment traversed an active stream channels, and water will be encountered in project excavations near drainages at and below stream stage. Even if surface flows are limited or absent prior to or at times during construction, significantly greater volumes of water should be anticipated after significant precipitation events, etc.

The contractor should be prepared to work in wet conditions, de-water, build cofferdams, etc., as necessary, and to handle, process and compact (or replace) wet soils. De-watering systems and cofferdams should be designed for the contractor by a qualified engineer.

Environmental or other non-geotechnical concerns regarding working in and near waterways should be address by other, qualified consultants.

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

Groundwater Groundwater was not encountered in the test holes at the depths explored at the time of drilling. Therefore, based on our experience, wet soils, and groundwater, other that local volumes of perched groundwater, should not be anticipated, in excavations shallower than about 16 feet below existing grade. However, at stream and drainage crossings, wet soils and local groundwater should be anticipated during and after periods of heavy precipitation and /or snow 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 DRAINAGE

Surface water should be directed to drainage from the proposed roadway alignment to help limit the wetting of subgrade soils. As mentioned elsewhere in this report, wetting of the subgrade soils will increase the likelihood of damage post-construction movements occurring. The following measures should be implemented as part of construction and after construction has been completed.

1) Project line and grades should be constructed to direct water away from the roadway alignment and direct water to appropriate drainage infrastructure as soon as practical.

2) Areas that pond water should be repaired, so that effective drainage can be restored.

3) Where existing natural or man-made drainages are crossed, appropriate measures should be taken so that the flow of water is not impeded.

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 owner 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 of the proposed Merrick & Company as described herein. It should not be assumed to contain sufficient information for other parties or other purposes. The Client has agreed to the terms, conditions, and liability limitations outlined in our proposal between Merrick & Company and GROUND. Reliance upon our report is not granted to any other potential owner, contractor, or lender. Requests for third-party reliance should be directed to GROUND in writing; granting reliance by GROUND is not guaranteed.

In addition, GROUND has assumed that project construction will commence by Fall 2018. 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.

The geotechnical conclusions in this report were based on subsurface information from a limited number of exploration points, as shown in Figures 1A and 1B, 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. In addition, a contractor who obtains information from this report for development of his scope of work or cost estimates does so solely at his own risk and 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 should obtain the additional geotechnical information that is necessary to develop his workscope and cost estimates with sufficient precision. This includes, but is not limited to, information regarding excavation conditions, earth material usage, current depths to groundwater, etc. Because of the necessarily limited nature of the subsurface exploration performed for this study, the contractor should be allowed to evaluate the site using test pits or other means to obtain additional subsurface information to prepare his bid.

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 our conclusions for this site may be re-evaluated in a timely manner and dependent aspects of project design can be modified, as necessary.

The materials present on-site are stable at their natural moisture content, but may change volume or lose bearing capacity or stability with changes in moisture content. Performance of the proposed structure and pavement will depend on implementation of the conclusions and information in this report and on proper maintenance after construction is completed. Because water is a significant cause of volume change in soils and rock, allowing moisture infiltration may result in movements, some of which will exceed estimates provided herein and should therefore be expected by Merrick & Company.

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 the geotechnical parameters 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, it is imperative that Merrick & Company or the owner 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.

This report was prepared in accordance with generally accepted soil and foundation engineering practice in the project area at the date of preparation. Current applicable codes may contain criteria regarding performance of structures and/or site improvements which may differ from those provided herein. Our office should be contacted regarding any apparent disparity.

GROUND makes no warranties, either expressed or implied, as to the professional data, opinions or conclusions contained herein. Because of numerous considerations that are beyond GROUND's control, the economic or technical performance of the project cannot be guaranteed in any respect.

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 owner with a proposal for construction observation and materials testing.

Sincerely,

GROUND Engineering Consultants, Inc.

COLORADO LICENSED
BRIAN H. RECK
48265
PROFESSIONAL ENGINEER

5/30/2018

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

GOOGLE EARTH AERIAL IMAGE (10/09/2015)
SITE PLAN PROVIDED BY CLIENT

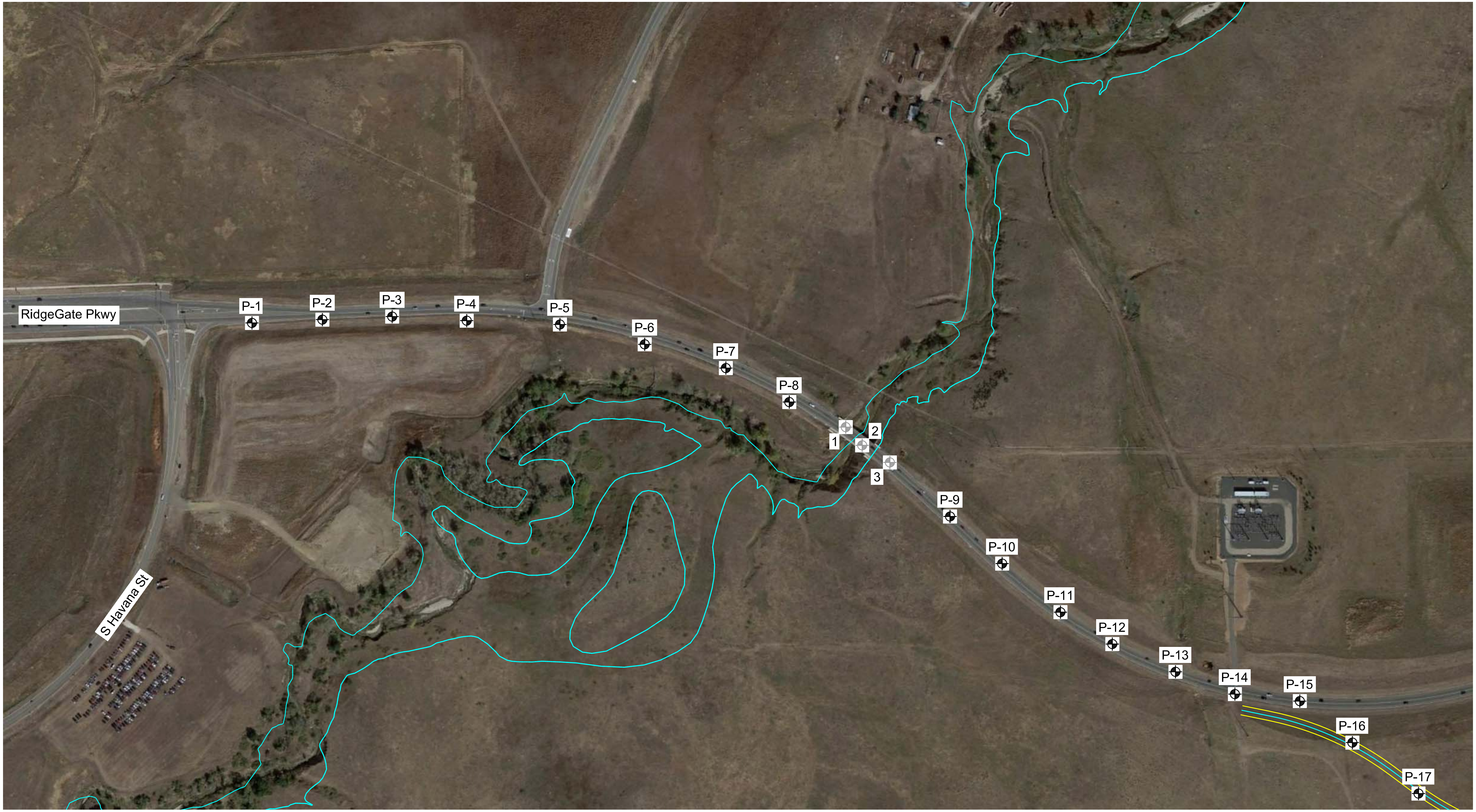
(Not to Scale)

GROUND
ENGINEERING CONSULTANTS

VICINITY MAP

JOB NO.: 17-3053 | FIGURE: 1A

CADFILE NAME: 3053SITE.DWG



GOOGLE EARTH AERIAL IMAGE (10/09/2015)
SITE PLAN PROVIDED BY CLIENT

1 Indicates test hole number and approximate location drilled under current scope.

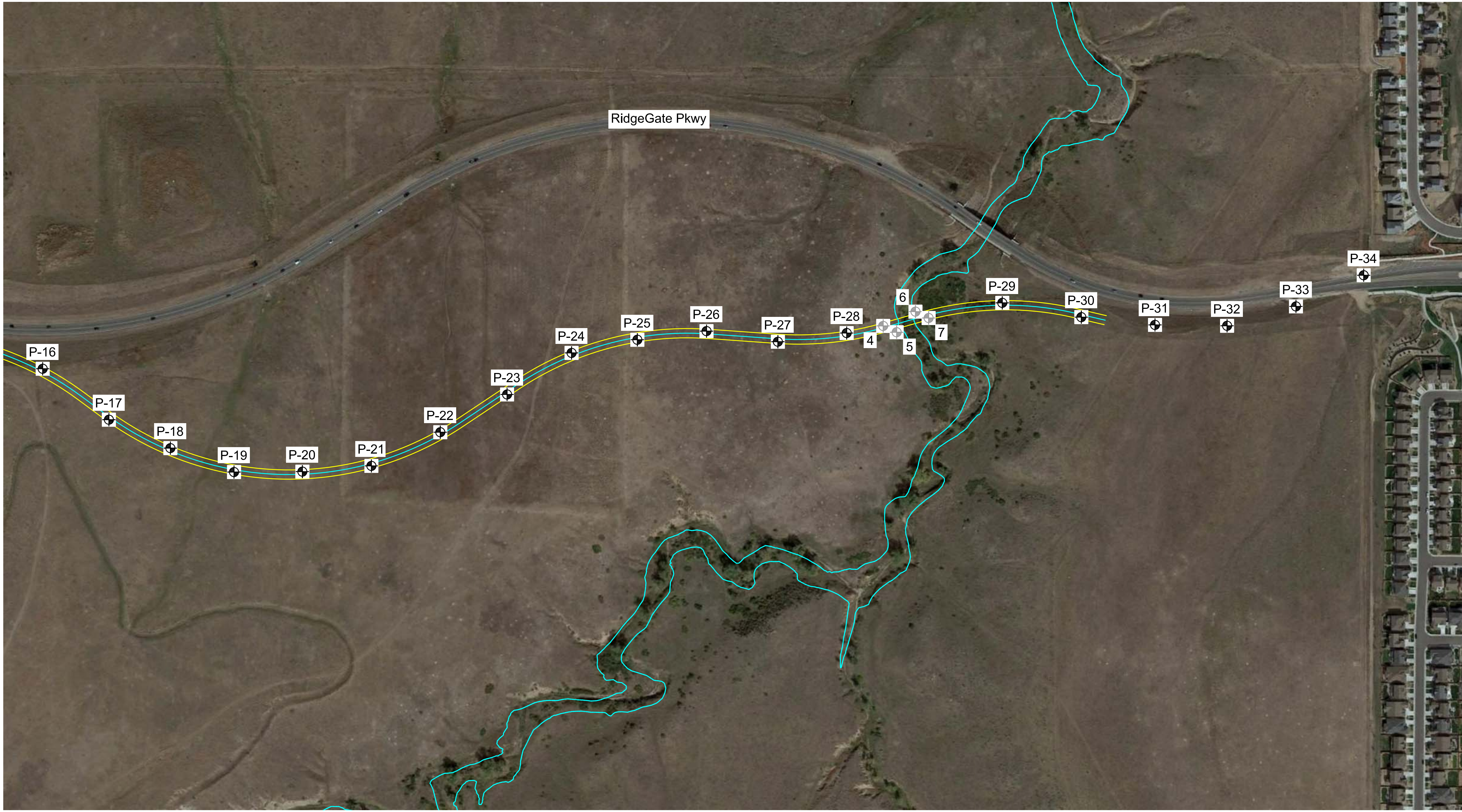
1 Indicates test hole number and approximate location drilled under separate scope.

(Not to Scale)

GROUND
ENGINEERING CONSULTANTS

LOCATION OF TEST HOLES

JOB NO.: 17-3053	FIGURE: 1B
CADFILE NAME: 3053SITE.DWG	



GOOGLE EARTH AERIAL IMAGE (10/09/2015)
SITE PLAN PROVIDED BY CLIENT

1
Indicates test hole number and approximate location drilled under current scope.

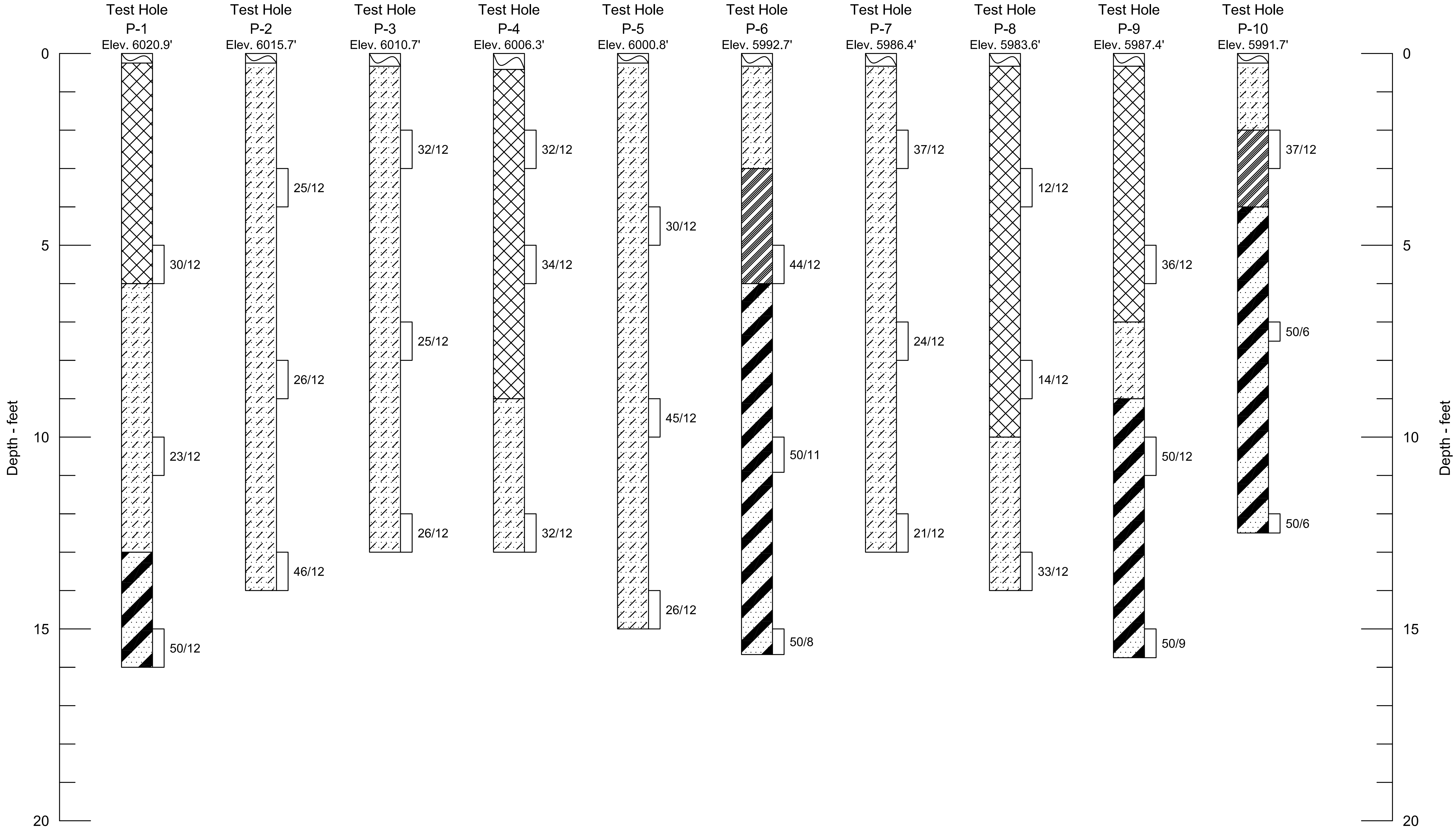
1
Indicates test hole number and approximate location drilled under separate scope.

(Not to Scale)

GROUND
ENGINEERING CONSULTANTS

LOCATION OF TEST HOLES

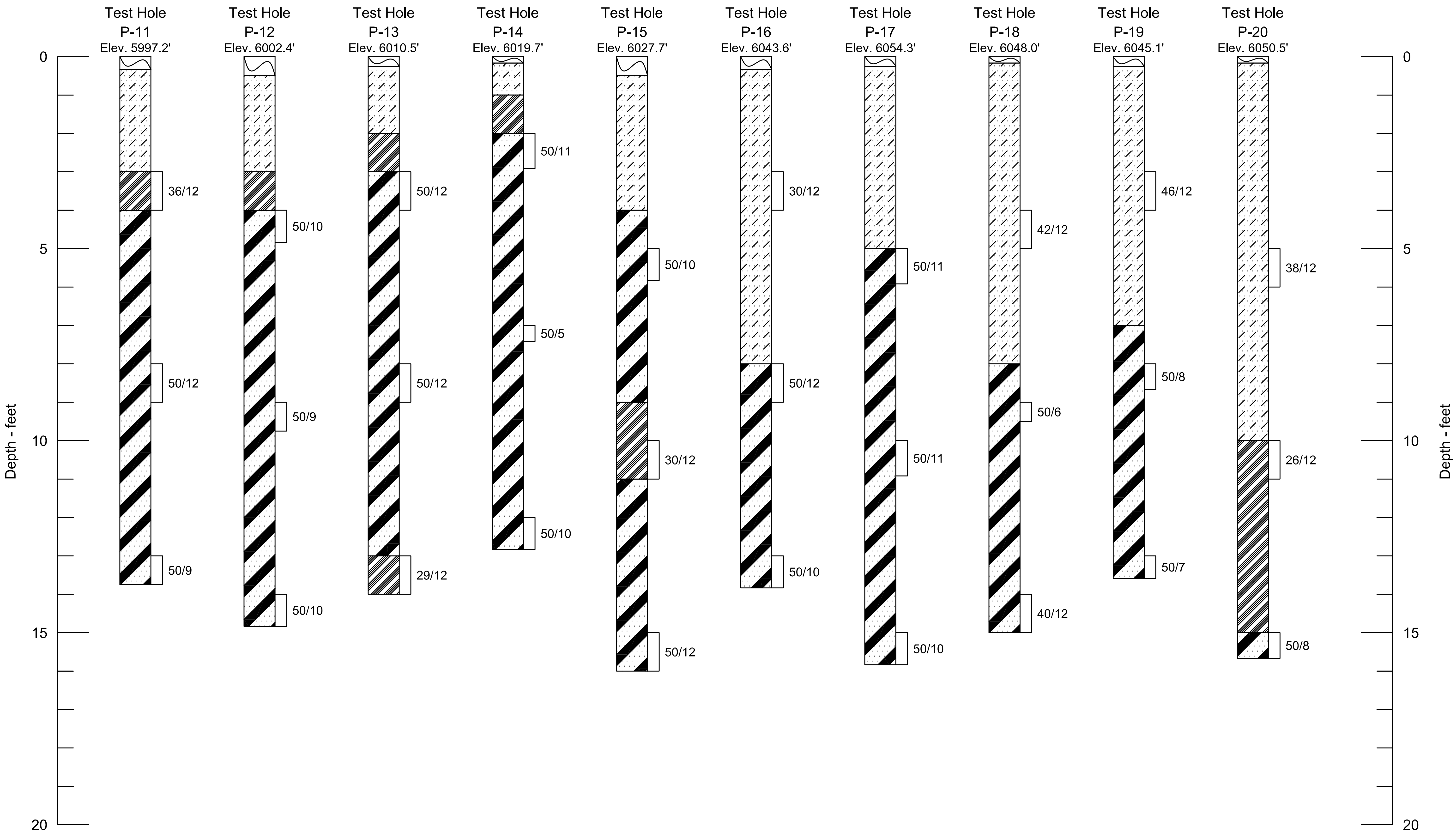
JOB NO.: 17-3053	FIGURE: 1C
CADFILE NAME: 3053SITE.DWG	



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

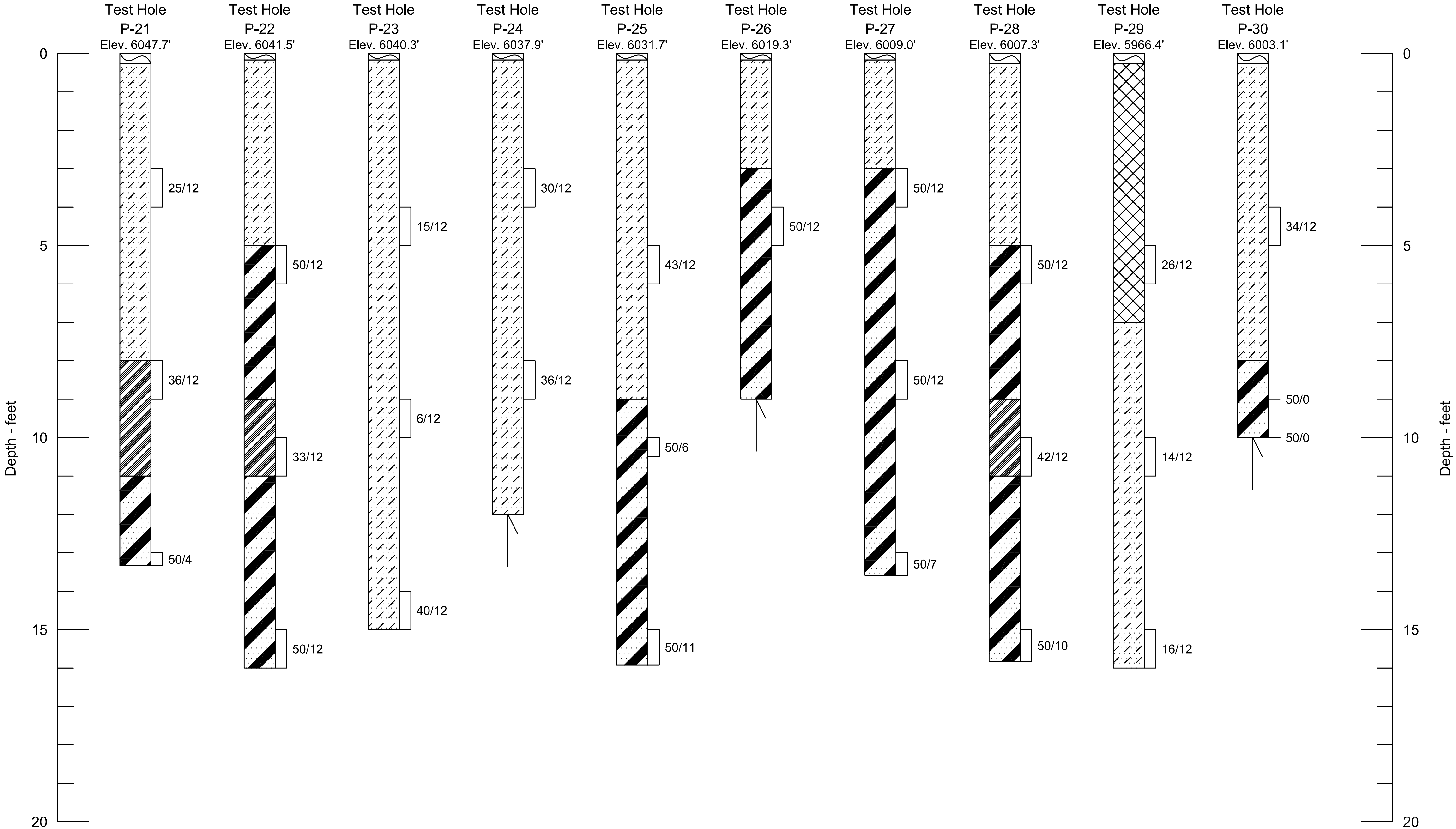
JOB NO.: 17-3053	FIGURE: 2
CADFILE NAME: 3053LOG.DWG	



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

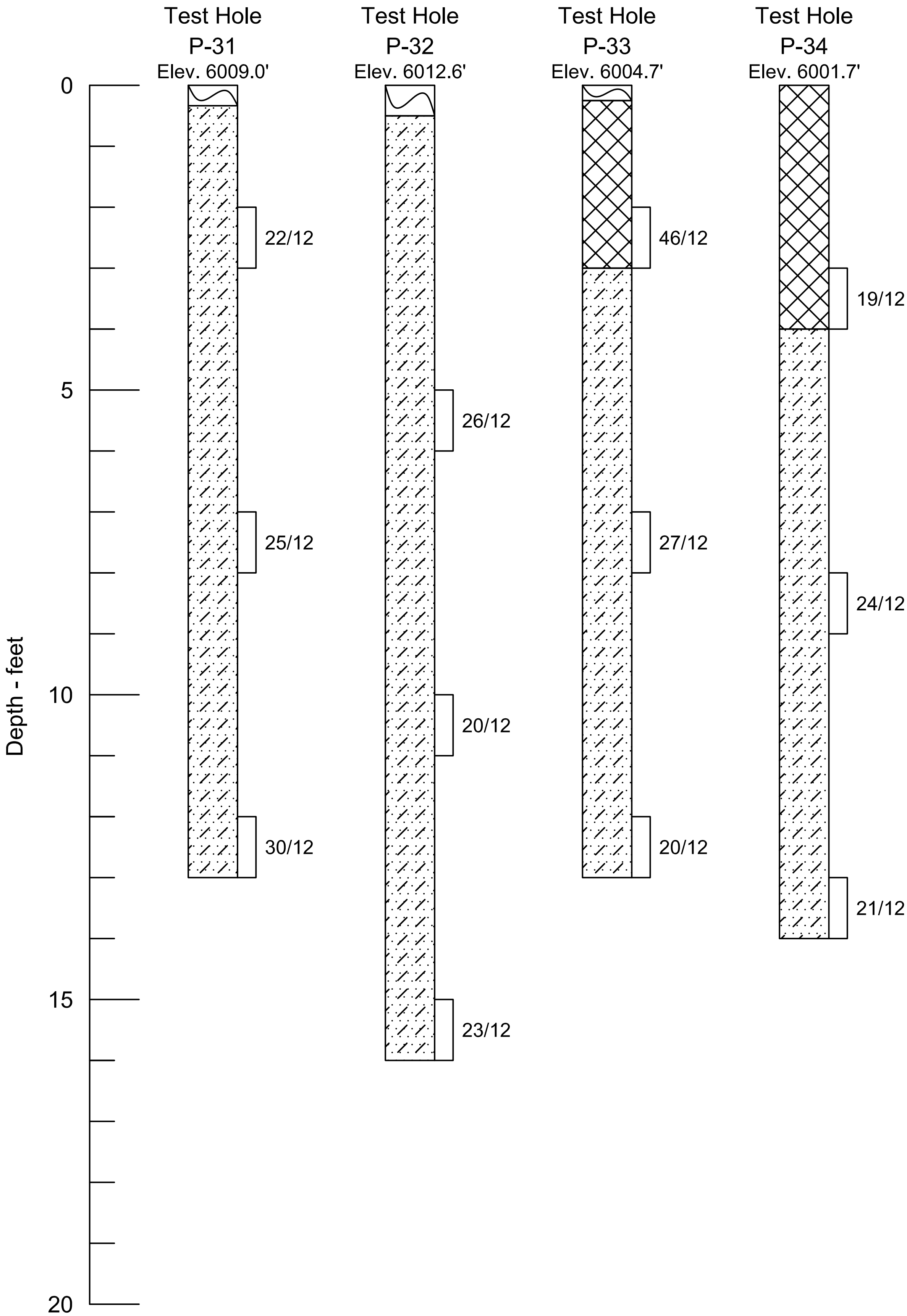
JOB NO.: 17-3053	FIGURE: 3
CADFILE NAME: 3053LOG.DWG	



GROUND ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

JOB NO.: 17-3053	FIGURE: 4
CADFILE NAME: 3053LOG.DWG	



GROUND ENGINEERING CONSULTANTS

LOG OF TEST HOLES

JOB NO.: 17-3053	FIGURE: 5
CADFILE NAME: 3053LOG.DWG	

LEGEND:

Topsoil

Fill: Silts, clays, and fine to coarse sands with gravels locally. They were non- to highly plastic, medium dense or stiff or very stiff, dry to very moist, and light to dark brown in color.

Sands, Silts, and Clays: Silty to clayey, fine to coarse sands, silts, and clays with local gravels. They were non- to highly plastic, medium dense to very dense or stiff to very stiff, dry to wet, and red brown and pale to dark brown in color. Iron staining and caliche were observed commonly.

Weathered Sandstone, Siltstone, and Claystone: Interbedded, fine to medium grained, weathered sandstone, siltstone, and claystone. They were slightly to highly plastic, firm to medium hard, dry to wet, and green brown, brown, and gray in color. Iron staining was observed commonly.

Sandstone, Siltstone, and Claystone: Interbedded, fine to medium grained, sandstone, siltstone, and claystone. They were slightly to highly plastic, hard to very hard, dry to wet, and green brown, red brown, brown, gray, and blue in color. Iron staining and caliche were observed commonly.

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

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

Practical Rig Refusal

NOTES:

1) Test holes were drilled on 11/15, 11/21, 11/30/2017 with 4-inch diameter continuous flight augers.

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

3) The logs of the test holes are drawn to depth. Elevations of the test holes were obtained from survey data for the anticipated test hole locations provided by the client and are shown with each log. The test holes were drilled in the nearest accessible location to the surveyed location. Therefore some deviation between the surveyed locations and the actual locations should be anticipated.

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 was not encountered during drilling. Ground water levels can fluctuate seasonally and in response to landscape irrigation.

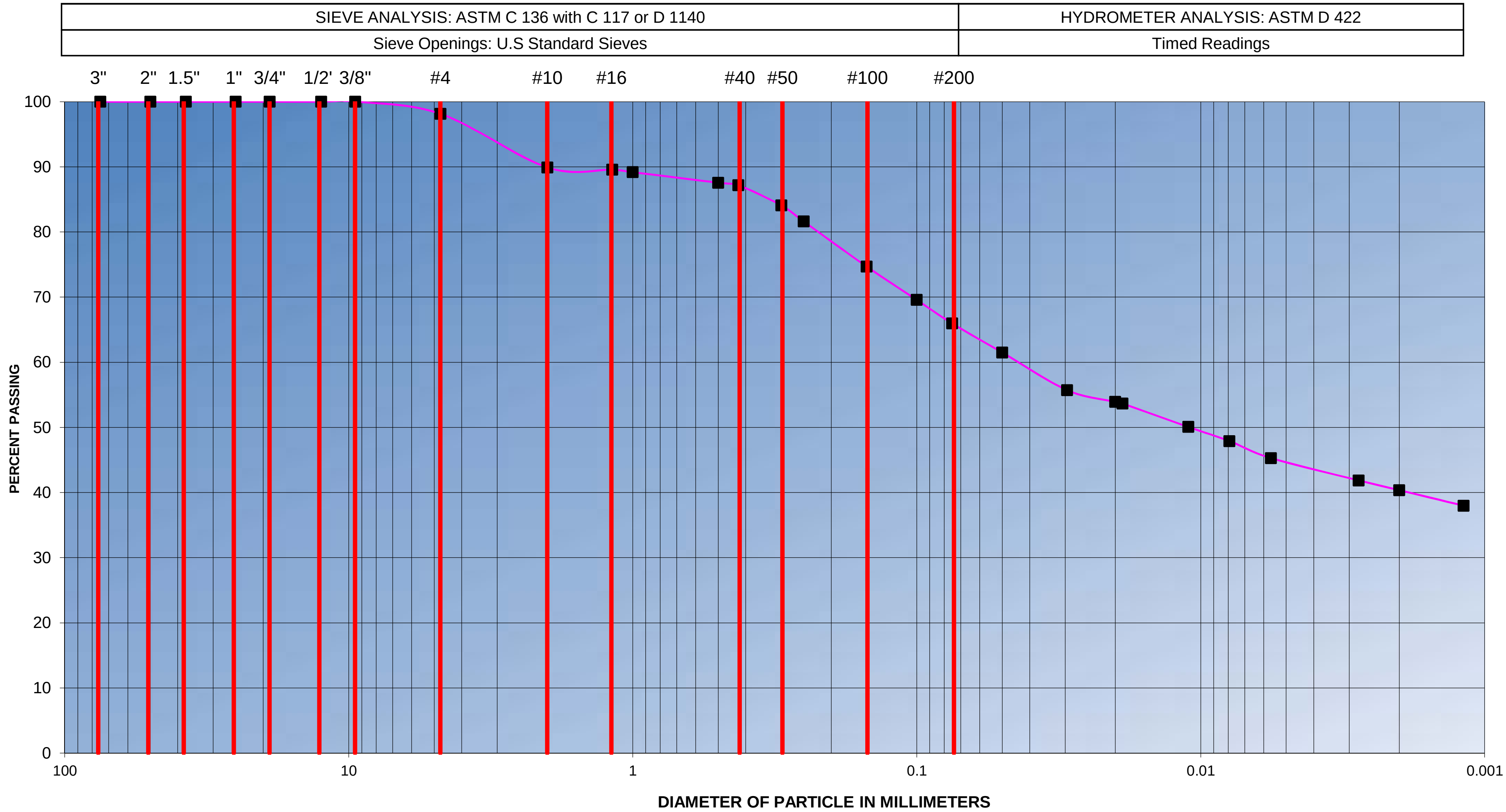
7) The material descriptions on this legend are for general classification purposes only. See the full text of this report for descriptions of the site materials and related information.

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

GROUND
ENGINEERING CONSULTANTS

LEGEND AND NOTES

JOB NO.: 17-3053	FIGURE: 6
CADFILE NAME: 3053LEG.DWG	



COBBLE	GRAVEL		SAND			SILT	CLAY
	Coarse	Fine	Coarse	Medium	Fine		

Sample of: FILL: Sandy Silt	Gravel 2%	Sand 32%	Silt and Clay 66%
From: P-9 at 5 feet	Liquid Limit 64	Plasticity Index 27	

GROUND ENGINEERING CONSULTANTS

GRADATION TEST RESULTS

JOB NO.: 17-3053

FIGURE: 7

GROUND ENGINEERING

TABLE 1

SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural	Natural	Gradation		Percent	Atterberg Limits		Percent	USCS	AASHTO	
Test Hole No.	Depth *(feet)*	Moisture Content *(%)*	Dry Density *(pcf)*	Gravel *(%)*	Sand *(%)*	Passing No. 200 Sieve	Liquid Limit	Plasticity Index	Swell (Surcharge Pressure)	Classifi-cation	Classifi-cation (GI)	Soil or Bedrock Type
P-1	15	29.4	83.5			36	59	15		SM	A-7-5 (1)	SANDSTONE Bedrock
P-2	3	14.8	SD			68	58	26		MH	A-7-5 (18)	Sandy SILT
P-3	2	11.4	95.2			65	46	18	1.0 (250 psf)	ML	A-7-6 (11)	Sandy SILT
P-3	8	21.4	95.3			72	50	21		MH	A-7-6 (16)	SILT with Sand
P-4	2	14.2	102.0			63		N P	2.6 (150 psf)	ML	A-4 (0)	FILL: Sandy Silt
P-5	9	20.8	105.3			76	56	26		MH	A-7-5 (21)	SILT with Sand
P-6	15	21.1	102.4			48	52	19		SM	A-7-5 (6)	SANDSTONE Bedrock
P-7	2	25.8	93.0			74	63	30		MH	A-7-5 (24)	SILT with Sand
P-8	8	34.8	80.7			61	64	28	-0.9 (1,000 psf)	MH	A-7-5 (17)	FILL: Sandy Silt
P-9	5	22.9	97.5	2	32	66	64	27		MH	A-7-5 (19)	FILL: Sandy Silt
P-10	12	6.5	104.4			31	52	13		SM	A-2-7 (0)	SANDSTONE Bedrock
P-11	3	24.0	SD			68	69	26		MH	A-7-5 (20)	Weathered SILTSTONE
P-12	4	30.5	92.3			47	55	17	3.7 (150 psf)	SM	A-7-5 (6)	SANDSTONE Bedrock
P-12	9	28.9	93.2			52	67	24		MH	A-7-5 (11)	SILTSTONE Bedrock
P-13	13	35.3	83.7			67	67	26		MH	A-7-5 (19)	Weathered SILTSTONE
P-14	2	35.4	85.3			37	61	15		SM	A-7-5 (2)	SANDSTONE Bedrock
P-15	10	44.3	68.9			52	73	24		MH	A-7-5 (11)	Weathered SILTSTONE
P-16	8	28.7	87.1			48	65	21	7.4 (150 psf)	SM	A-7-5 (8)	SANDSTONE Bedrock

GROUND ENGINEERING

TABLE 1

SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural	Natural	Gradation		Percent	Atterberg Limits		Percent	USCS	AASHTO	
Test Hole No.	Depth *(feet)*	Moisture Content *(%)*	Dry Density *(pcf)*	Gravel *(%)*	Sand *(%)*	Passing No. 200 Sieve	Liquid Limit	Plasticity Index	Swell (Surcharge Pressure)	Classifi-cation	Classifi-cation (GI)	Soil or Bedrock Type
P-17	15	25.8	91.0			40	48	12		SM	A-7-5 (2)	SANDSTONE Bedrock
P-18	9	14.5	104.7			78	45	19		CL	A-7-6 (15)	CLAYSTONE Bedrock
P-19	13	26.3	97.8			34	43	11		SM	A-2-7 (0)	SANDSTONE Bedrock
P-20	5	11.7	101.0			85	51	22		MH	A-7-6 (21)	SILT with Sand
P-20	15	33.3	80.1			31	63	17		SM	A-2-7 (0)	SANDSTONE Bedrock
P-21	8	25.8	84.5			28	55	19		SM	A-2-7 (0)	Weathered SANDSTONE
P-23	4	11.4	SD			79	49	20		ML	A-7-6 (17)	SILT with Sand
P-24	8	8.0	118.0			48	43	19	7.0 (150 psf)	SC	A-7-6 (6)	Clayey SAND
P-25	15	2.5	117.0			30	21	9		SC	A-2-4 (0)	SANDSTONE Bedrock
P-26	4	13.1	102.7			80	44	18		CL	A-7-6 (15)	CLAYSTONE Bedrock
P-27	8	11.2	101.4			71	44	21	-0.1 (150 psf)	CL	A-7-6 (4)	CLAYSTONE Bedrock
P-28	10	5.6	112.9			20	31	13	-	SC	A-2-6 (0)	Weathered SANDSTONE
P-29	5	14.5	97.9			78	51	20	2.9 (500 psf)	MH	A-7-5 (17)	FILL: Silt with Sand
P-30	4	5.4	108.7			25	42	17		SC	A-2-7 (0)	Clayey SAND
P-32	10	11.9	104.6			62	44	20		CL	A-7-6 (11)	Sandy CLAY
P-33	12	20.6	99.1			65	36	13	-0.1 (800 psf)	CL	A-6 (7)	Sandy CLAY
P-34	3	11.0	-			41	34	8		SM	A-4 (1)	FILL: Silty Sand

N P indicates "non-plastic."

GROUND ENGINEERING

TABLE 2

SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water Soluble Sulfates	pH	Redox Potential	Sulfides Content	Resistivity	USCS Classification	Soil or Bedrock Type
Test Hole No.	Depth *(feet)*	*(%)*		*(mV)*		*(ohm-cm)*		
P-5	9	0.18	7.5	-58	Trace	2,264	MH	SILT with Sand
P-14	2	0.01	7.3	-48	Trace	1,146	SM	SANDSTONE Bedrock
P-17	15	1.00	7.5	-60	Trace	5,043	SM	SANDSTONE Bedrock
P-25	15	0.11	8.0	-94	Trace	1,164	SC	SANDSTONE Bedrock

Job No. 17-3053A

Appendix A

Pavement Section Calculations

Ridgegate Parkway - Full Depth Section REV

File Name: C:\Users\engineer\Desktop\17-3053A Ridgegate Parkway\Ridgegate Parkway - Full Depth Section REV.dgpx

Design Inputs

Design Life: 20 years
Design Type: FLEXIBLE

Base construction: May, 2019
Pavement construction: June, 2019
Traffic opening: September, 2019

Climate Data Sources (Lat/Lon): 39.57, -104.849

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	R6 Level 1 SX(100) PG 76-28	2.0
Flexible	R1 Level 1 S(100) PG 64-22	4.5
NonStabilized	A-1-a	10.0
Subgrade	A-7-6	36.0
Subgrade	A-7-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.1
Air voids (%)	5.2

Traffic

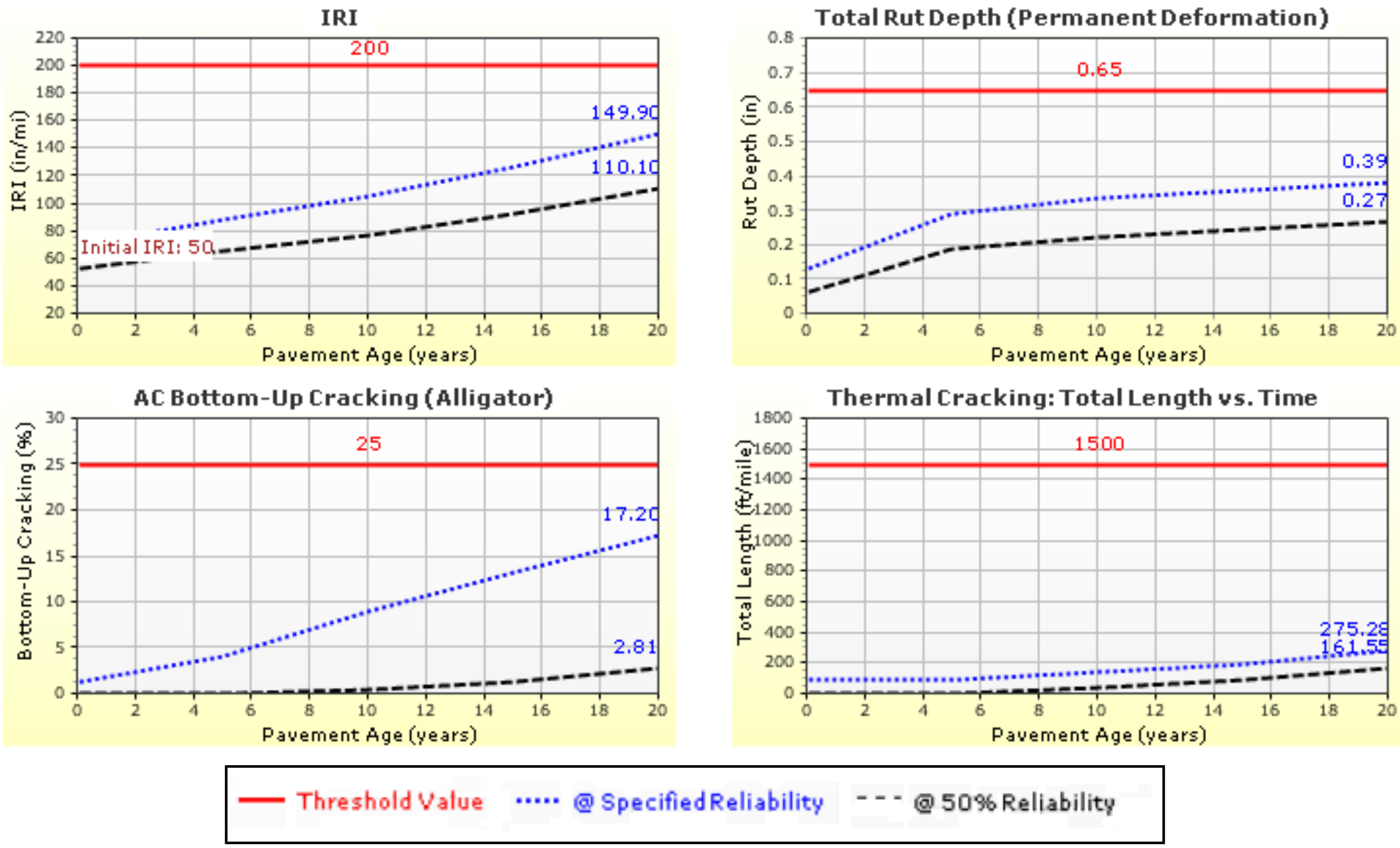
Age (year)	Heavy Trucks (cumulative)
2019 (initial)	945
2029 (10 years)	1,081,230
2039 (20 years)	2,270,510

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	200.00	149.88	90.00	99.81	Pass
Permanent deformation - total pavement (in)	0.65	0.38	90.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	25.00	17.20	90.00	97.59	Pass
AC thermal cracking (ft/mile)	1500.00	275.28	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2500.00	269.16	90.00	100.00	Pass
Permanent deformation - AC only (in)	0.50	0.25	90.00	100.00	Pass

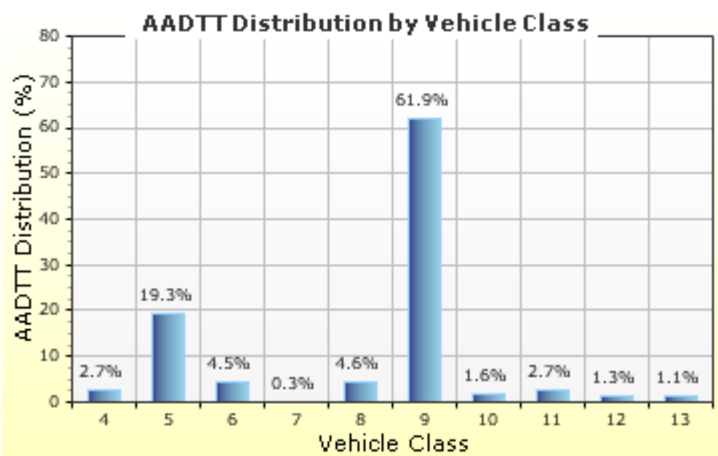
Distress Charts

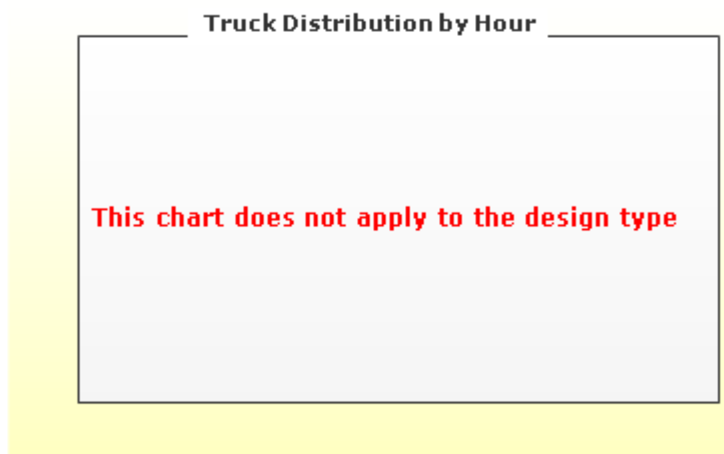


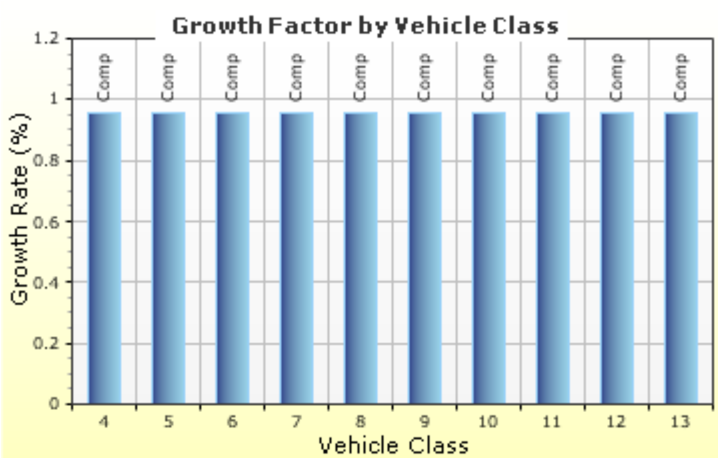
Traffic Inputs

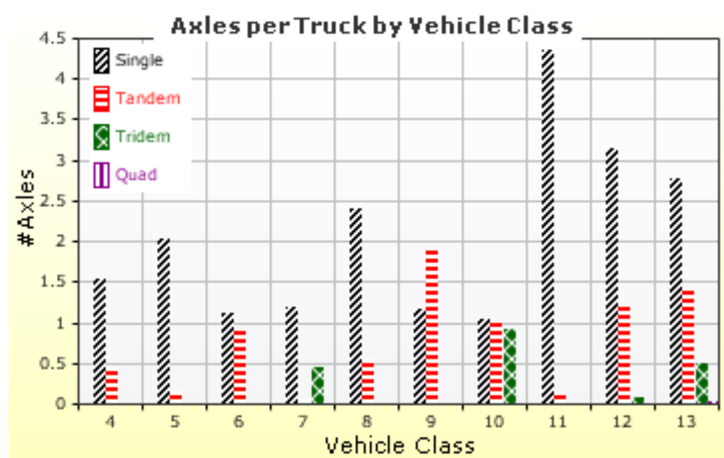
Graphical Representation of Traffic Inputs

Initial two-way AADTT:	945	Percent of trucks in design direction (%):	50.0
Number of lanes in design direction:	3	Percent of trucks in design lane (%):	60.0
		Operational speed (mph)	45.0

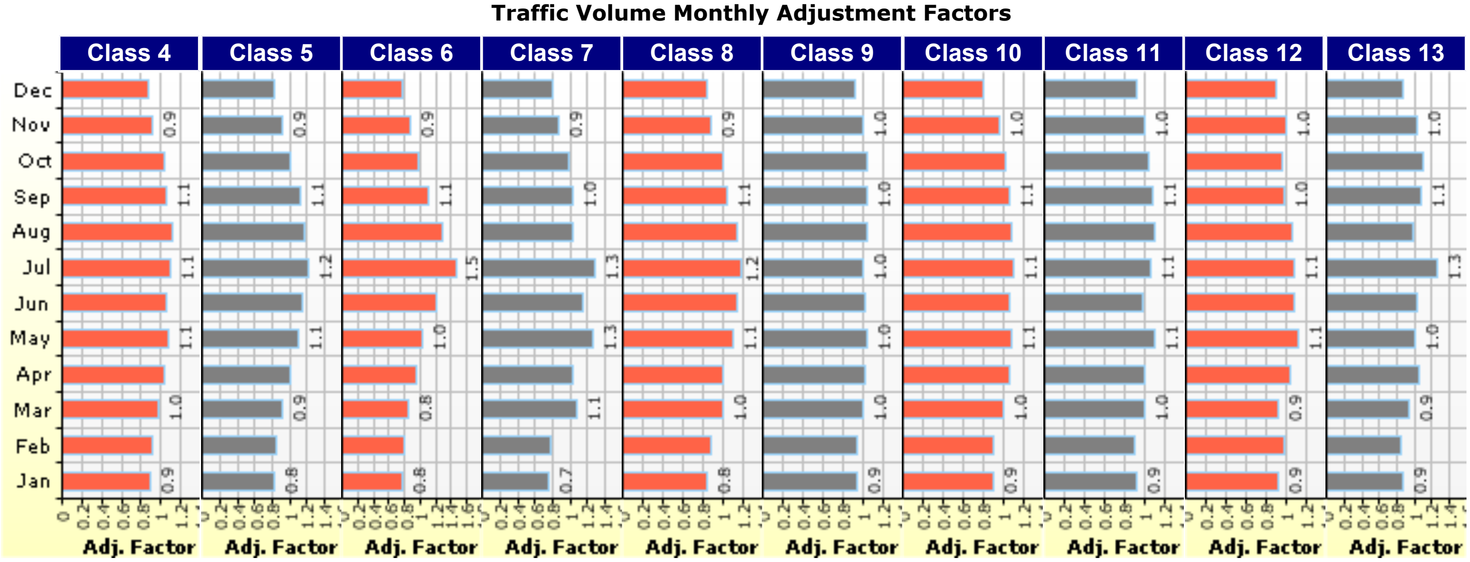








Traffic Volume Monthly Adjustment Factors



Tabular Representation of Traffic Inputs

Volume Monthly Adjustment Factors

Level 3: Default MAF

Month	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
January	0.9	0.8	0.8	0.7	0.8	0.9	0.9	0.9	0.9	0.9
February	0.9	0.8	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.8
March	1.0	0.9	0.8	1.1	1.0	1.0	1.0	1.0	0.9	0.9
April	1.0	1.0	0.9	1.0	1.0	1.0	1.1	1.0	1.0	1.1
May	1.1	1.1	1.0	1.3	1.1	1.0	1.1	1.1	1.1	1.0
June	1.1	1.1	1.2	1.1	1.1	1.0	1.1	1.0	1.1	1.0
July	1.1	1.2	1.5	1.3	1.2	1.0	1.1	1.1	1.1	1.3
August	1.1	1.2	1.3	1.0	1.1	1.0	1.1	1.1	1.1	1.0
September	1.1	1.1	1.1	1.0	1.1	1.0	1.1	1.1	1.0	1.1
October	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.9	1.1
November	0.9	0.9	0.9	0.9	0.9	1.0	1.0	1.0	1.0	1.0
December	0.9	0.8	0.8	0.8	0.8	0.9	0.8	0.9	0.9	0.9

Distributions by Vehicle Class

Vehicle Class	AADTT Distribution (%) (Level 3)	Growth Factor	
		Rate (%)	Function
Class 4	2.7%	0.957%	Compound
Class 5	19.3%	0.957%	Compound
Class 6	4.5%	0.957%	Compound
Class 7	0.3%	0.957%	Compound
Class 8	4.6%	0.957%	Compound
Class 9	61.9%	0.957%	Compound
Class 10	1.6%	0.957%	Compound
Class 11	2.7%	0.957%	Compound
Class 12	1.3%	0.957%	Compound
Class 13	1.1%	0.957%	Compound

Truck Distribution by Hour does not apply

Axle Configuration

Traffic Wander	
Mean wheel location (in)	18.0
Traffic wander standard deviation (in)	10.0
Design lane width (ft)	12.0

Axle Configuration	
Average axle width (ft)	8.5
Dual tire spacing (in)	12.0
Tire pressure (psi)	120.0

Average Axle Spacing	
Tandem axle spacing (in)	51.6
Tridem axle spacing (in)	49.2
Quad axle spacing (in)	49.2

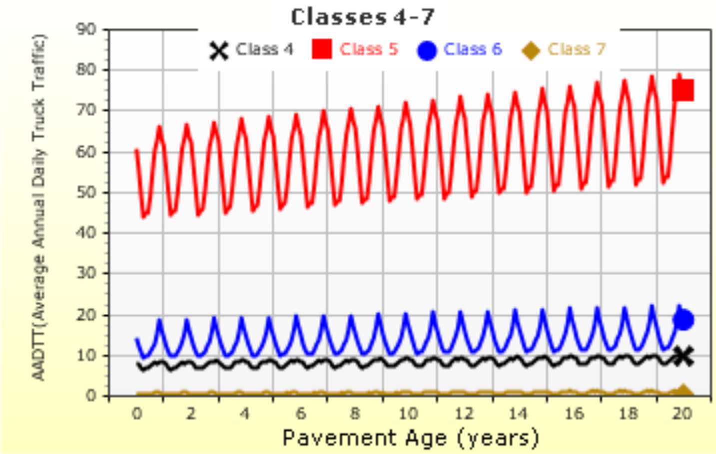
Wheelbase does not apply

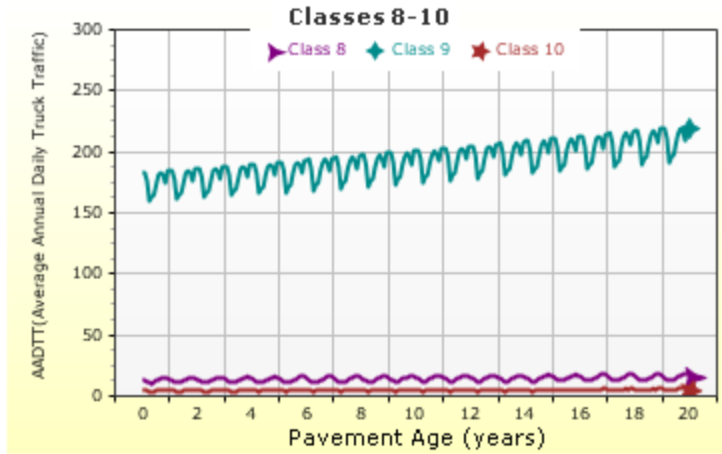
Number of Axles per Truck

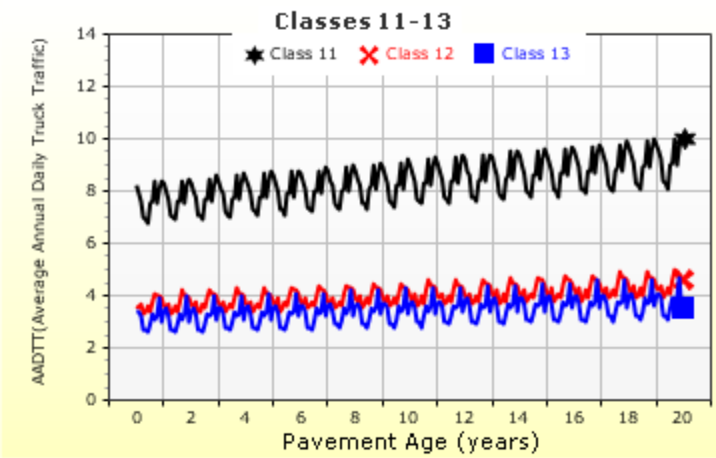
Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.53	0.45	0	0
Class 5	2.02	0.16	0.02	0
Class 6	1.12	0.93	0	0
Class 7	1.19	0.07	0.45	0.02
Class 8	2.41	0.56	0.02	0
Class 9	1.16	1.88	0.01	0
Class 10	1.05	1.01	0.93	0.02
Class 11	4.35	0.13	0	0
Class 12	3.15	1.22	0.09	0
Class 13	2.77	1.4	0.51	0.04

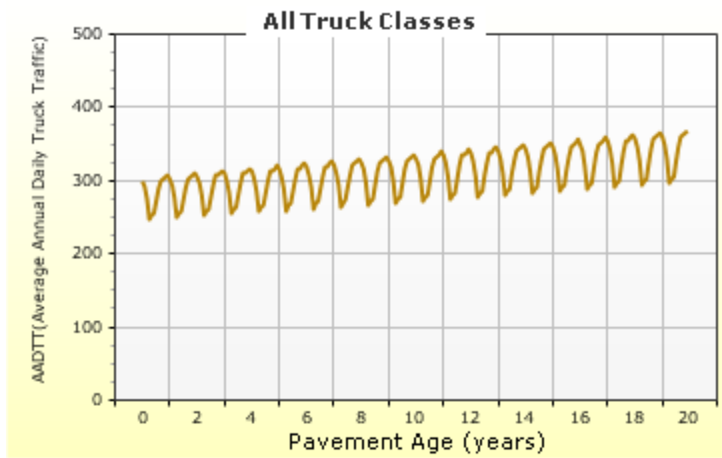
AADTT (Average Annual Daily Truck Traffic) Growth

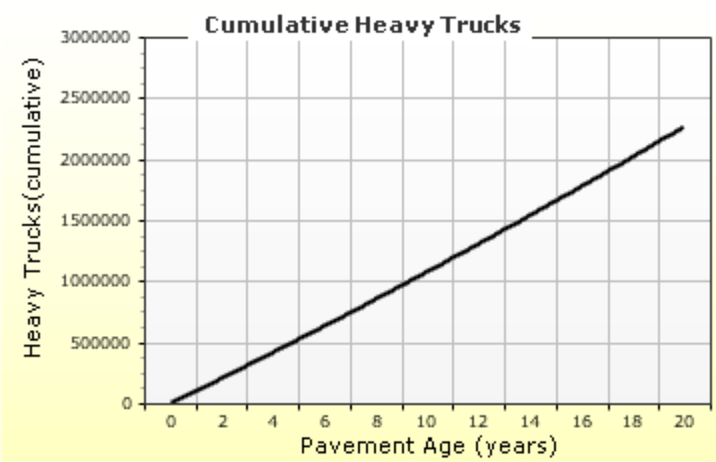
*** Traffic cap is not enforced**











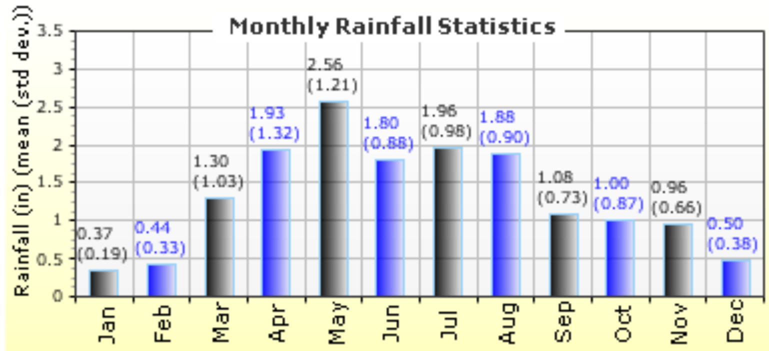
Climate Inputs

Climate Data Sources:

Climate Station Cities:	Location (lat lon elevation(ft))
CENTENNIAL, CO	39.57000 -104.84900 5828

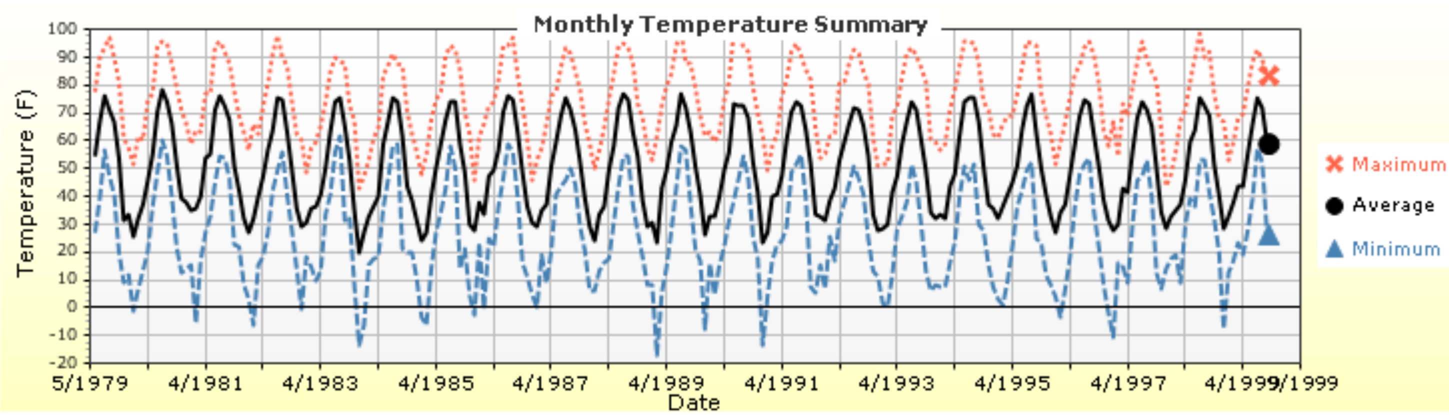
Annual Statistics:

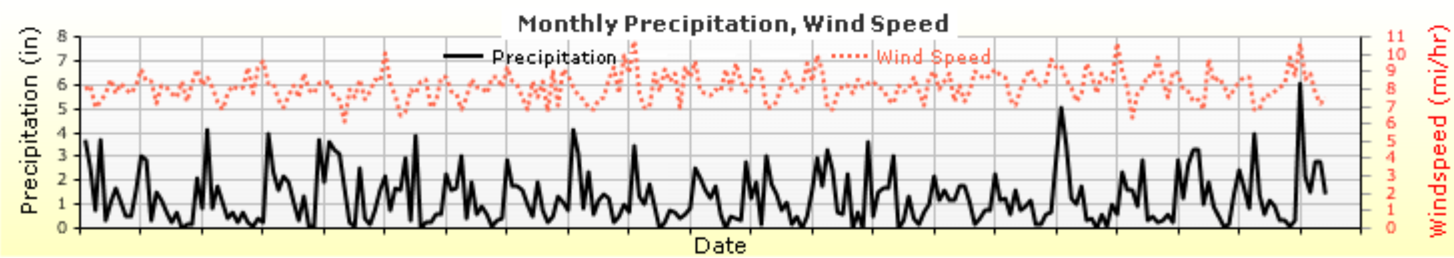
Mean annual air temperature (ºF)	50.66
Mean annual precipitation (in)	15.91
Freezing index (ºF - days)	493.72
Average annual number of freeze/thaw cycles:	77.45

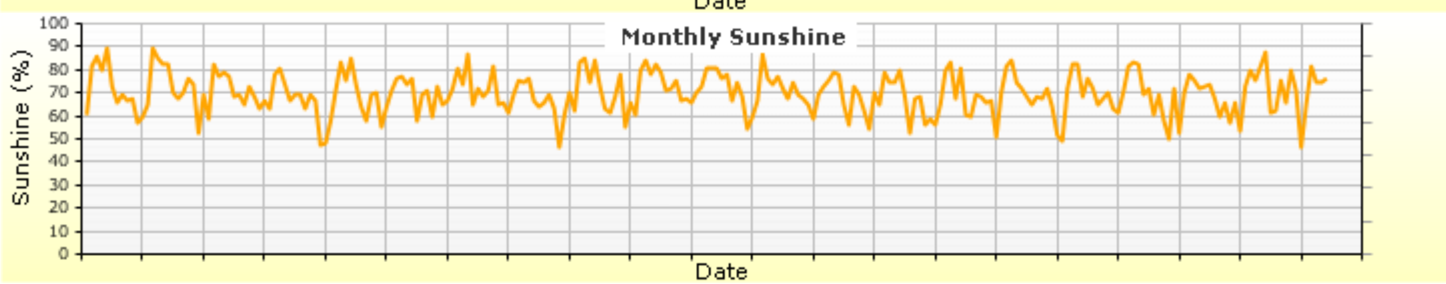


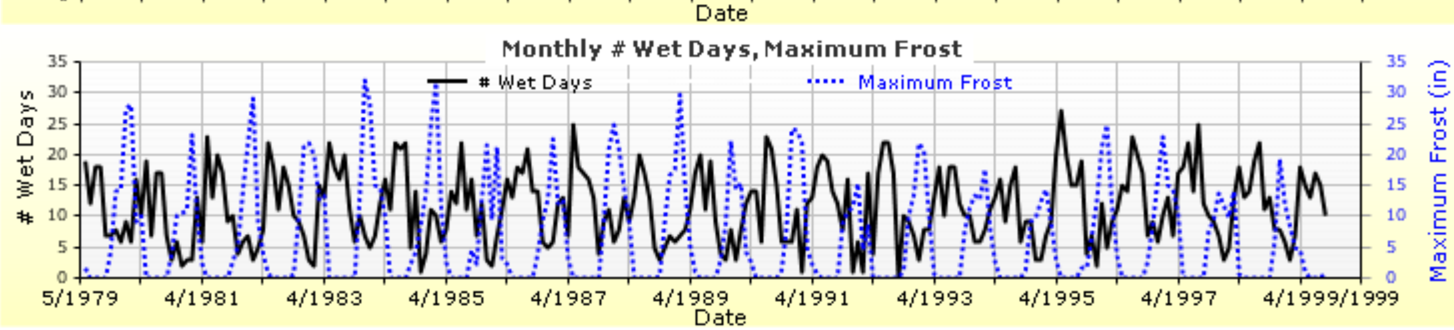
Water table depth (ft) 12.00

Monthly Climate Summary:





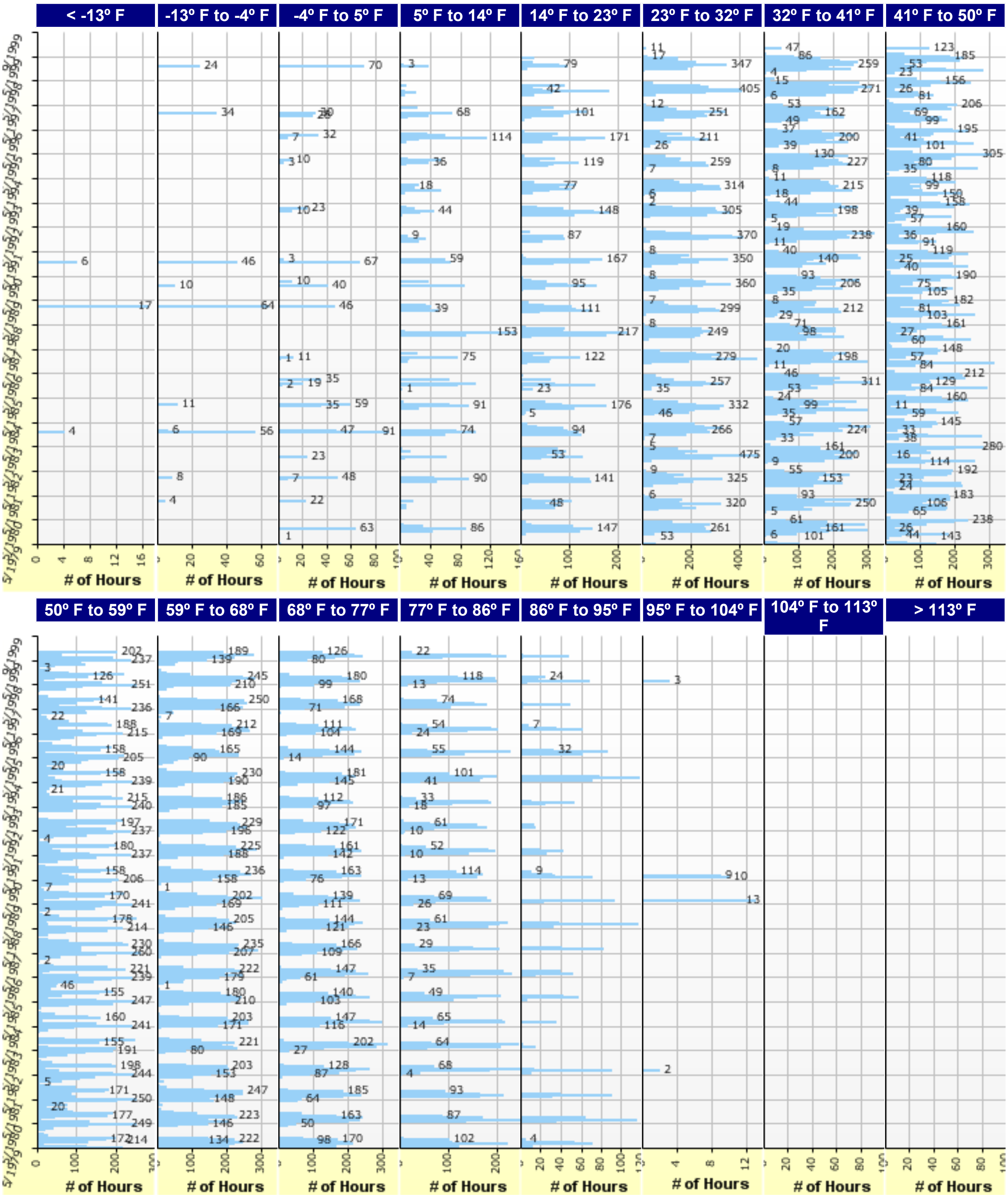




Ridgegate Parkway - Full Depth Section REV

File Name: C:\Users\engineer\Desktop\17-3053A Ridgegate Parkway\Ridgegate Parkway - Full Depth Section REV.dgpx

Hourly Air Temperature Distribution by Month:



Design Properties

HMA Design Properties

Use Multilayer Rutting Model	False
Using G* based model (not nationally calibrated)	False
Is NCHRP 1-37A HMA Rutting Model Coefficients	True
Endurance Limit	-
Use Reflective Cracking	True

Structure - ICM Properties	
AC surface shortwave absorptivity	0.85

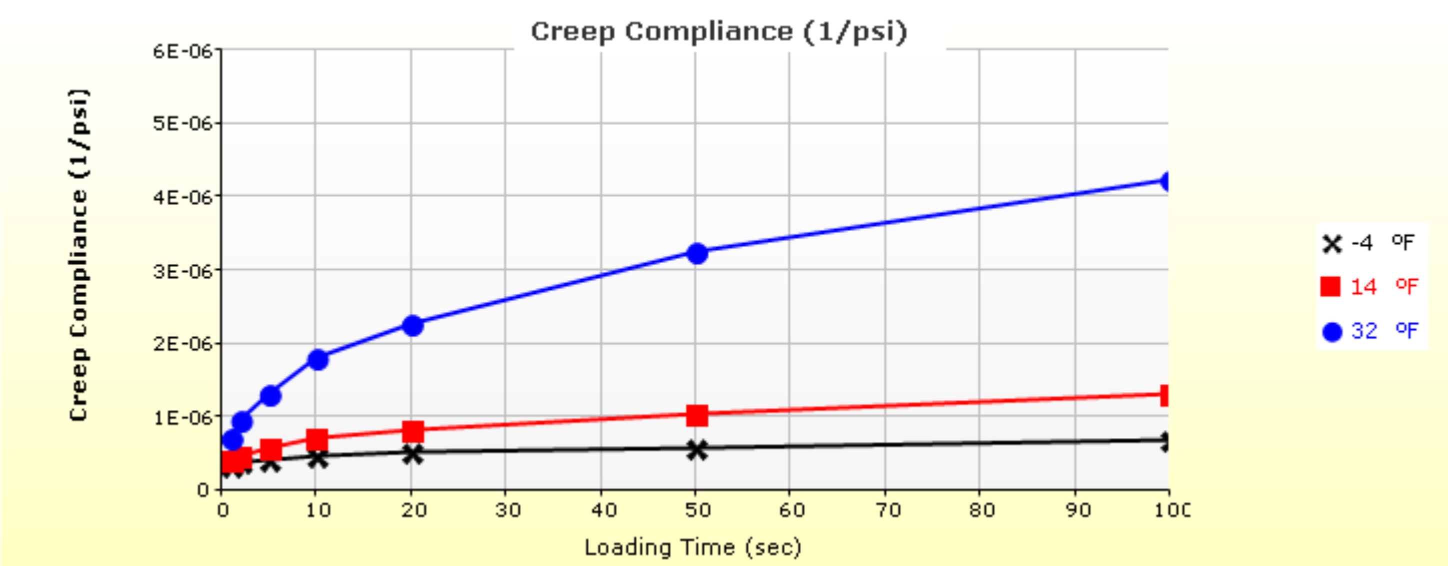
Layer Name	Layer Type	Interface Friction
Layer 1 Flexible : R6 Level 1 SX (100) PG 76-28	Flexible (1)	1.00
Layer 2 Flexible : R1 Level 1 S (100) PG 64-22	Flexible (1)	1.00
Layer 3 Non-stabilized Base : A-1 -a	Non-stabilized Base (4)	1.00
Layer 4 Subgrade : A-7-6	Subgrade (5)	1.00
Layer 5 Subgrade : A-7-6	Subgrade (5)	-

Thermal Cracking (Input Level: 1)

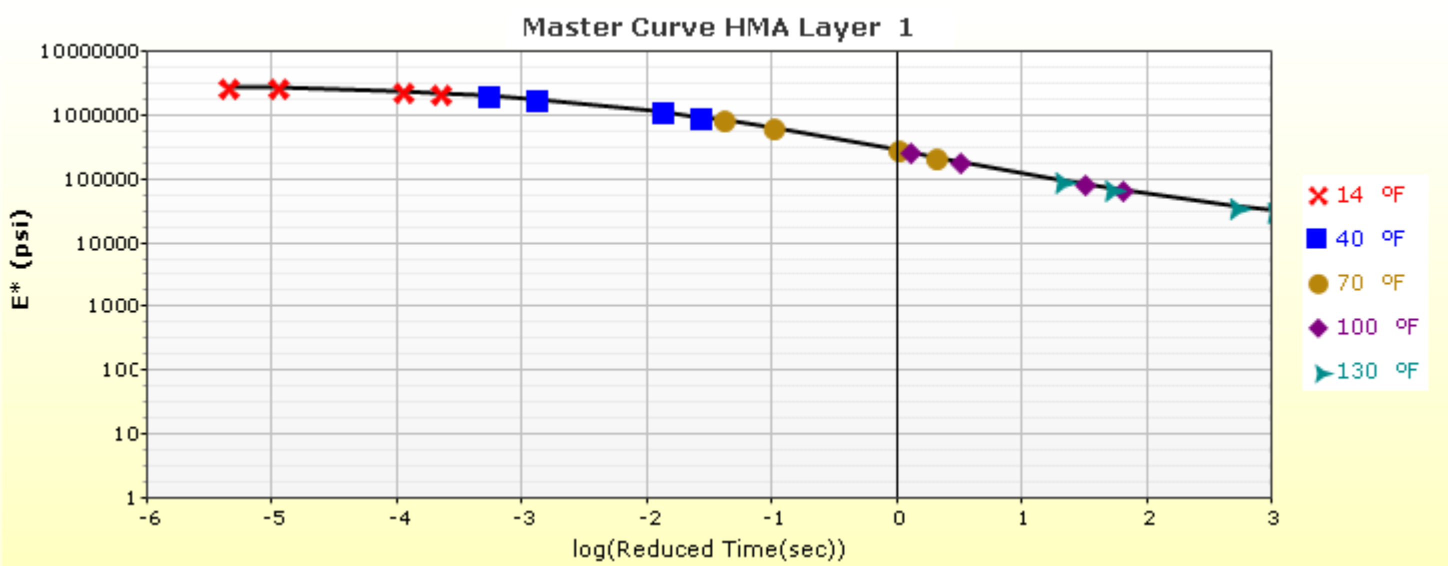
Indirect tensile strength at 14 ºF (psi)	595.00

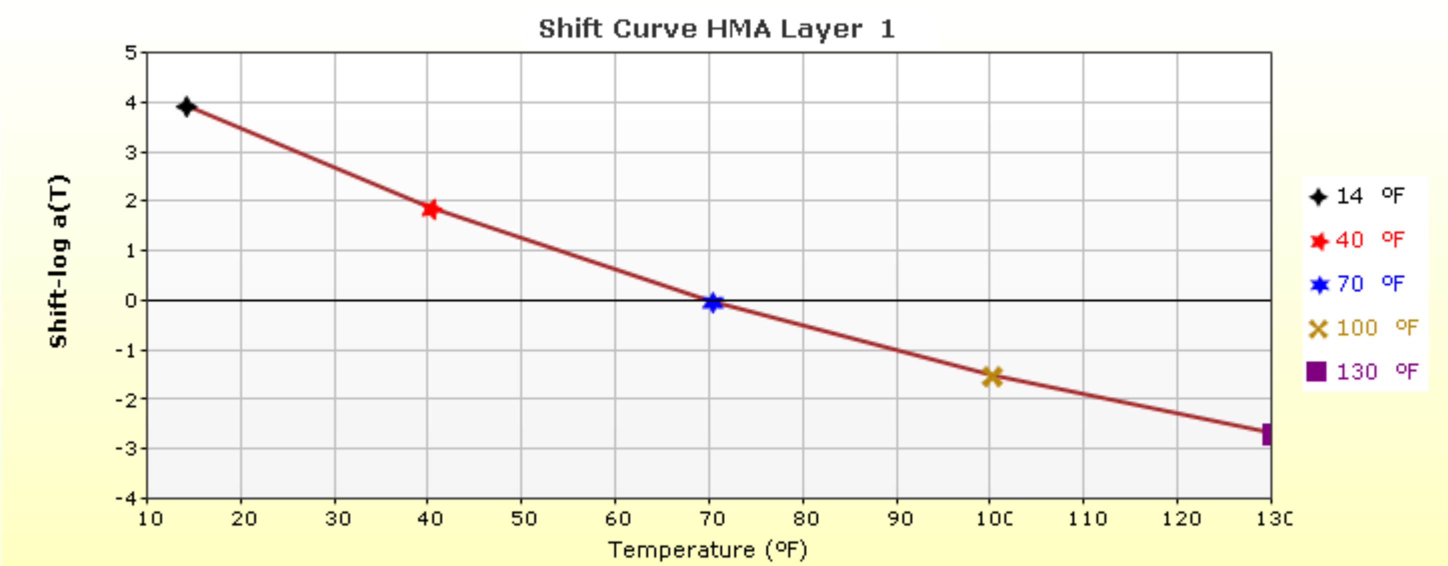
Thermal Contraction	
Is thermal contraction calculated?	True
Mix coefficient of thermal contraction (in/in/ºF)	-
Aggregate coefficient of thermal contraction (in/in/ºF)	5.0e-006
Voids in Mineral Aggregate (%)	16.3

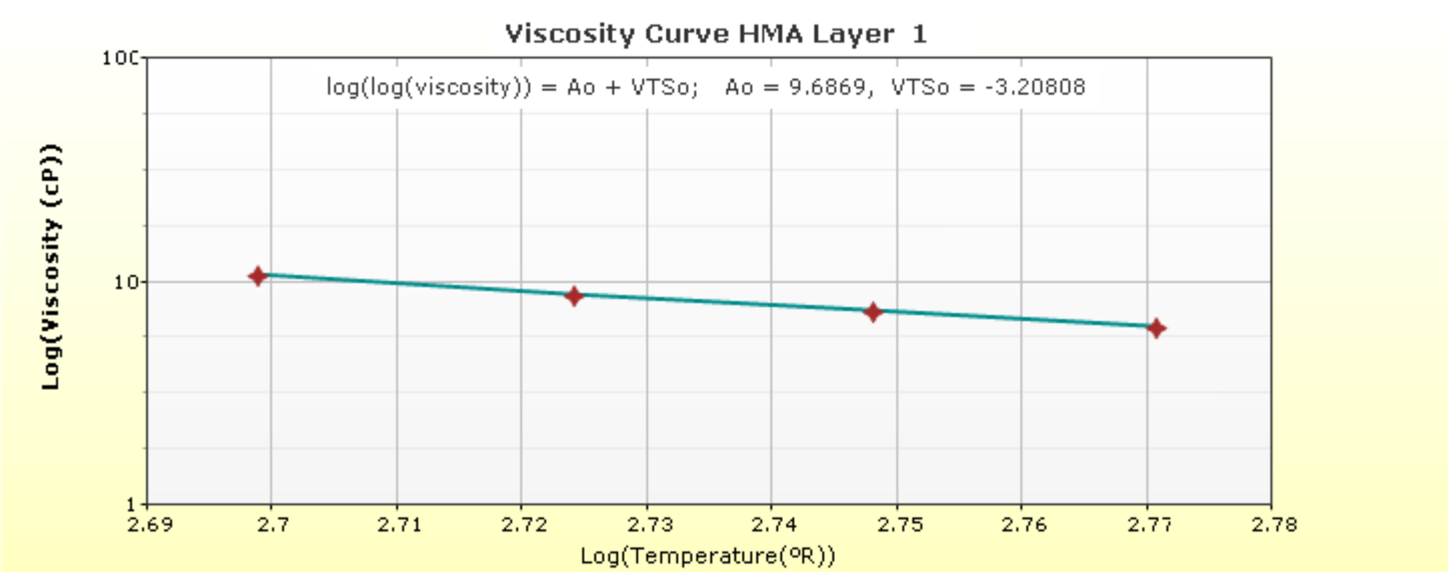
	Creep Compliance (1/psi)		
Loading time (sec)	-4 ºF	14 ºF	32 ºF
1	3.46e-007	4.12e-007	7.13e-007
2	3.83e-007	4.76e-007	9.57e-007
5	4.34e-007	5.97e-007	1.33e-006
10	4.85e-007	7.25e-007	1.80e-006
20	5.29e-007	8.45e-007	2.29e-006
50	5.99e-007	1.05e-006	3.25e-006
100	6.87e-007	1.32e-006	4.24e-006



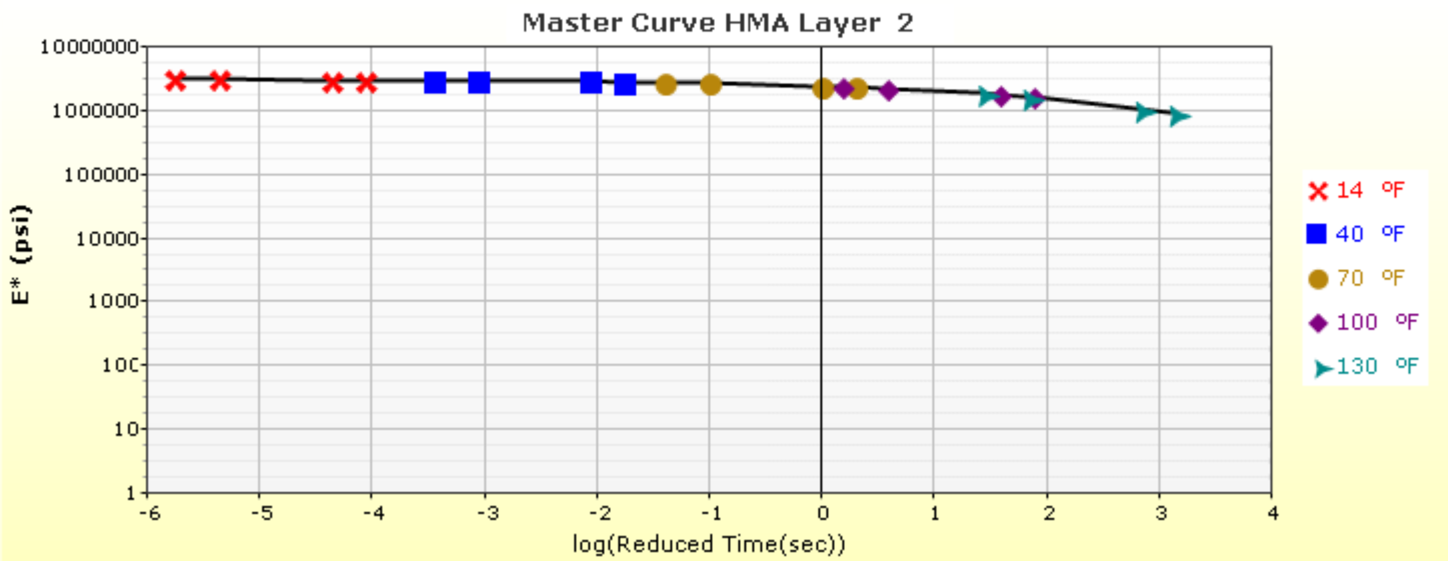
HMA Layer 1: Layer 1 Flexible : R6 Level 1 SX(100) PG 76-28

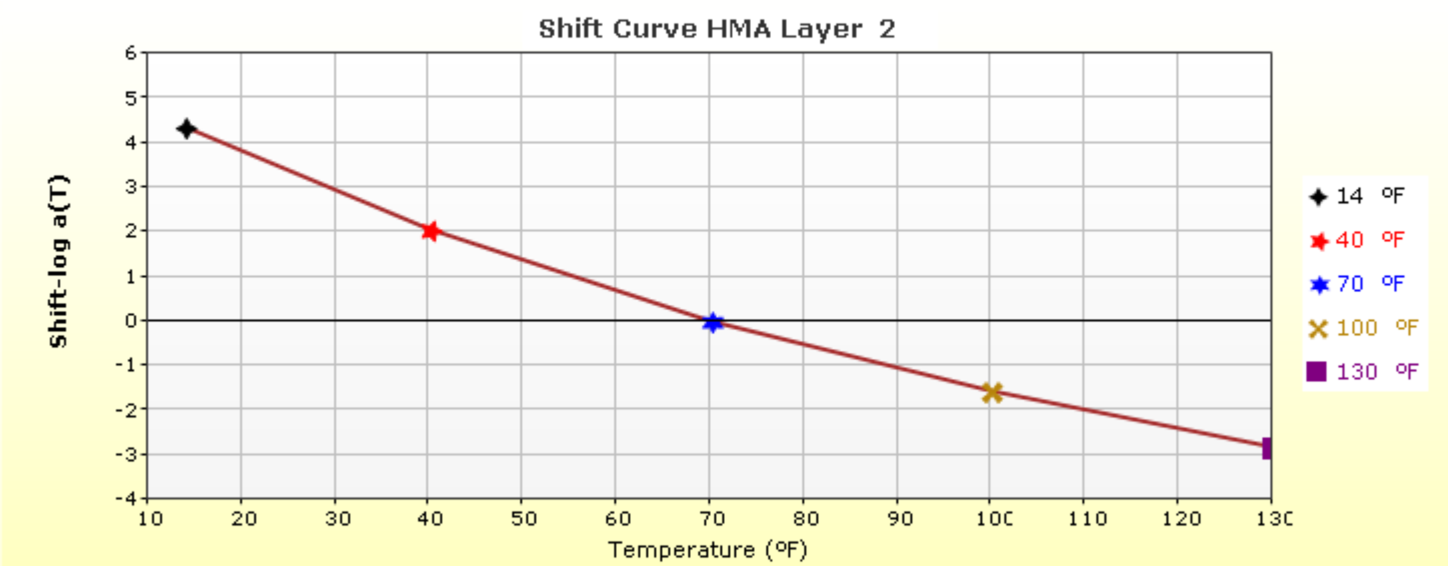


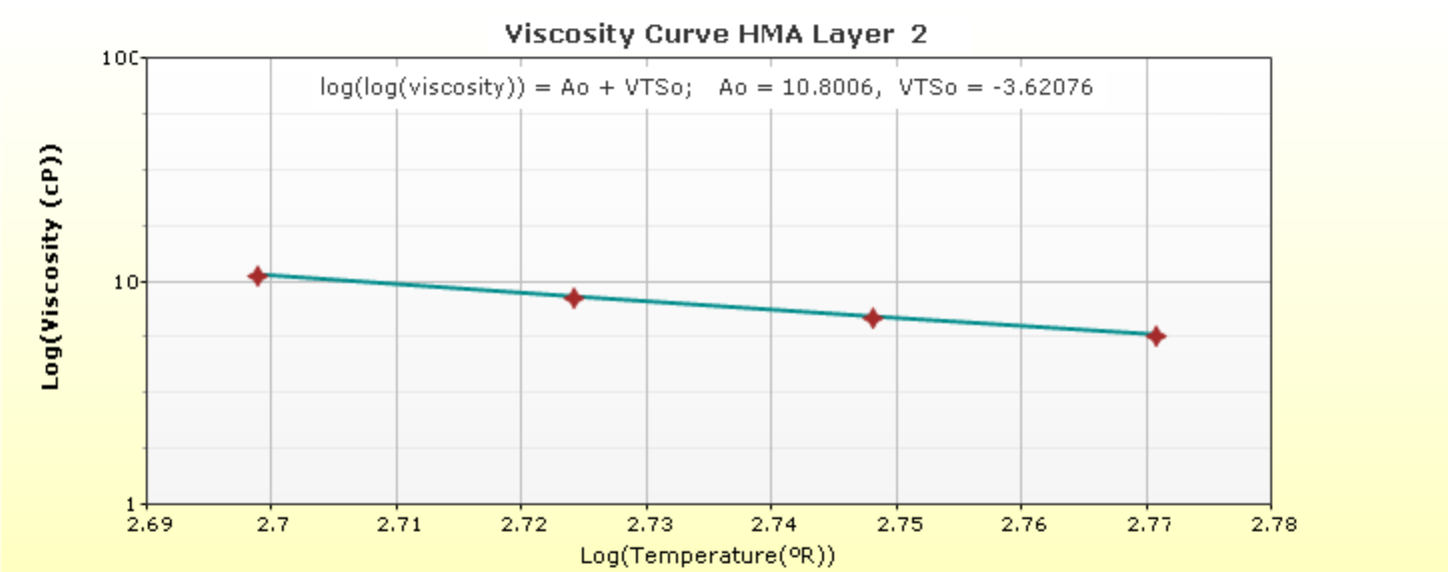




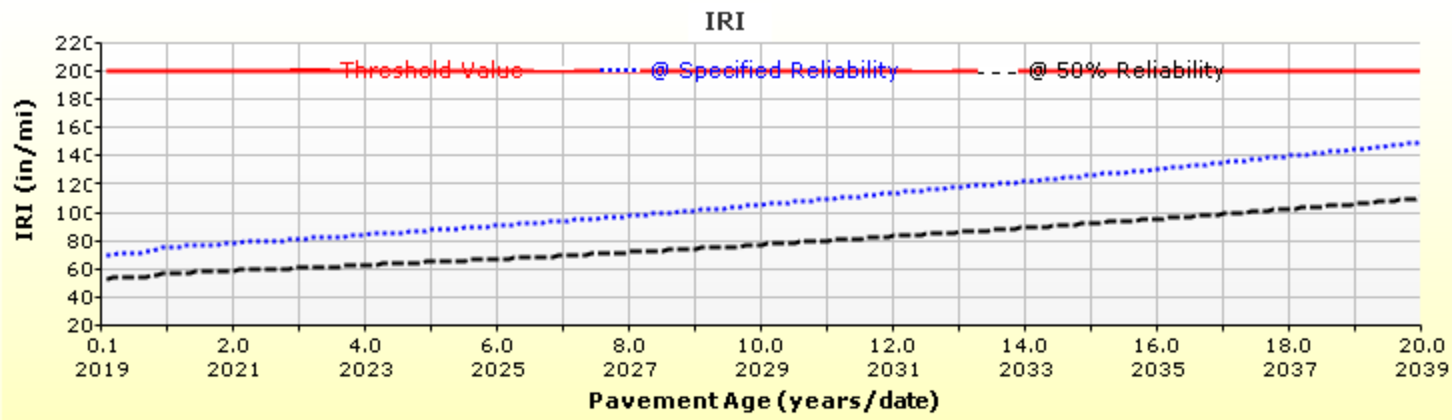
HMA Layer 2: Layer 2 Flexible : R1 Level 1 S(100) PG 64-22

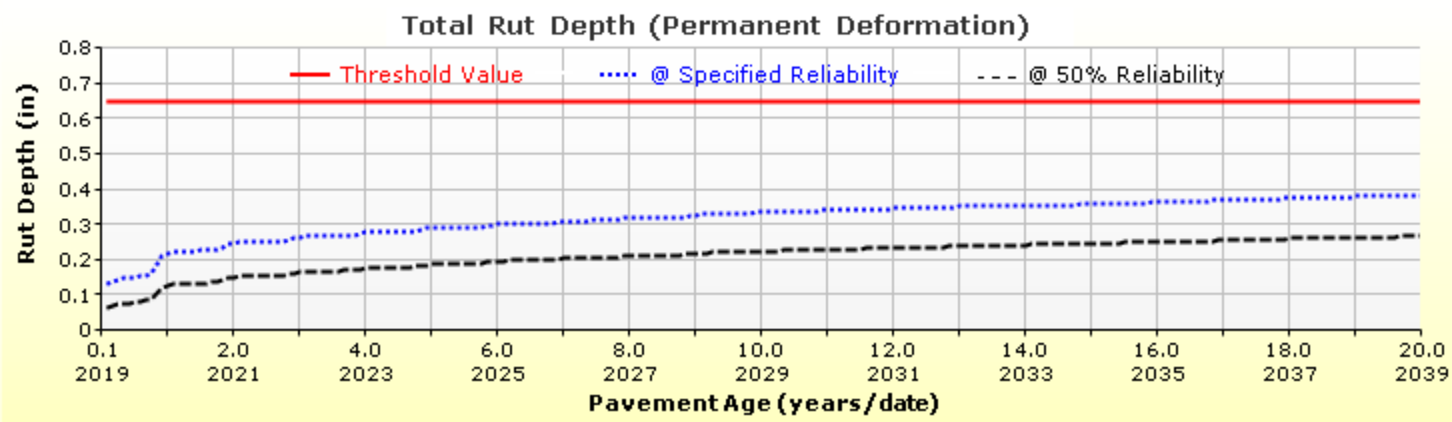


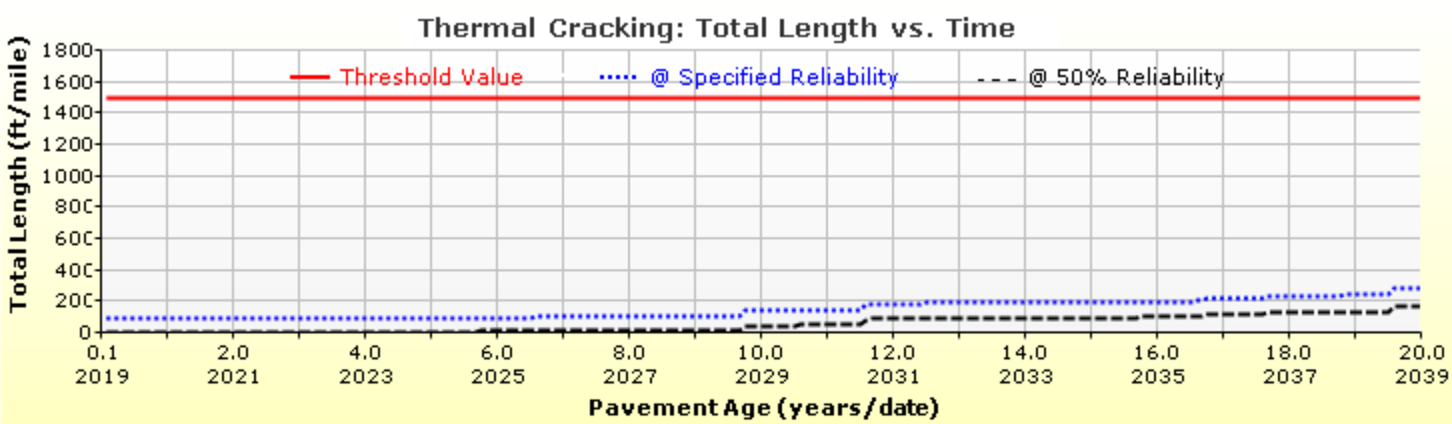


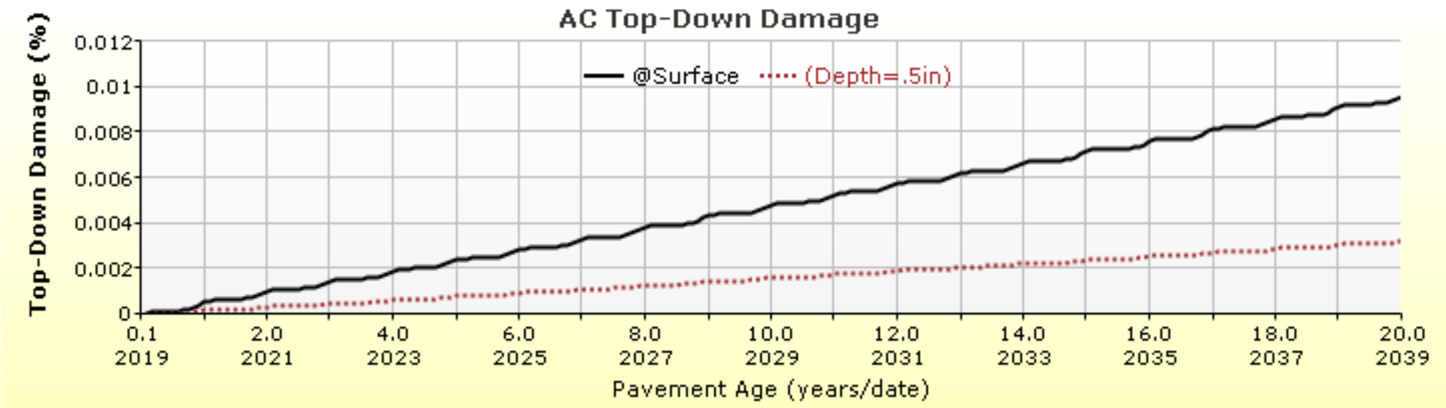


Analysis Output Charts

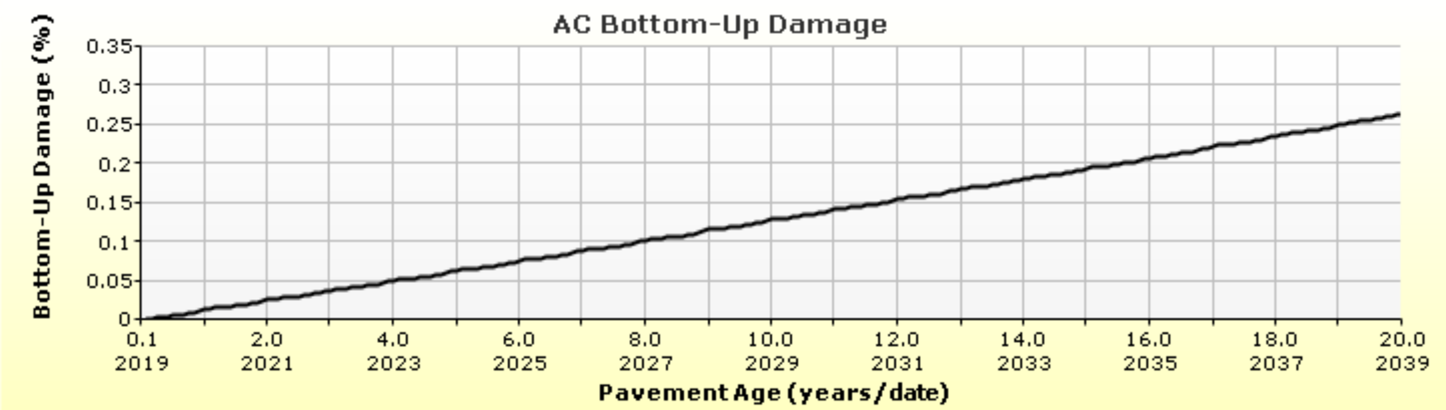




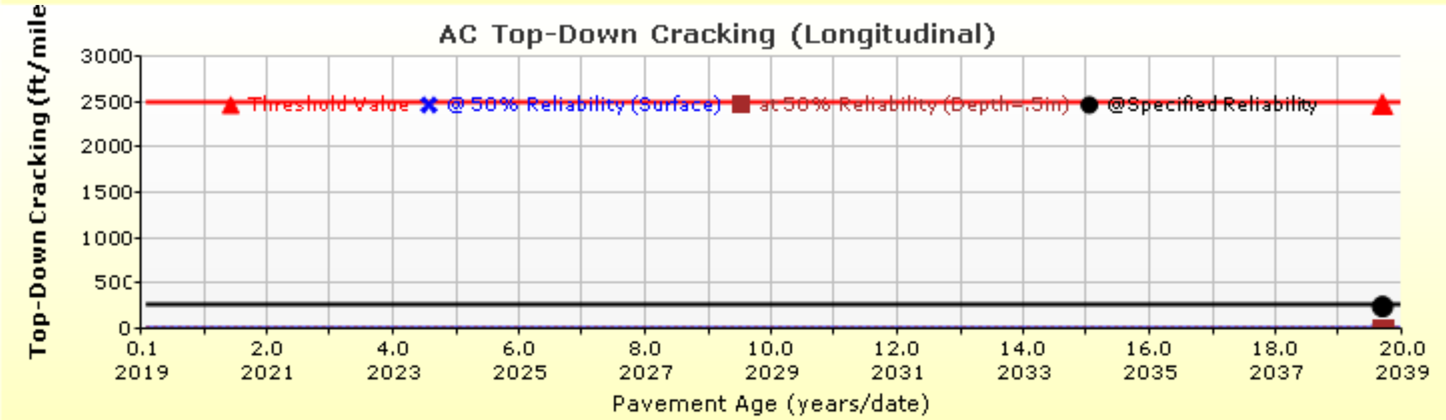




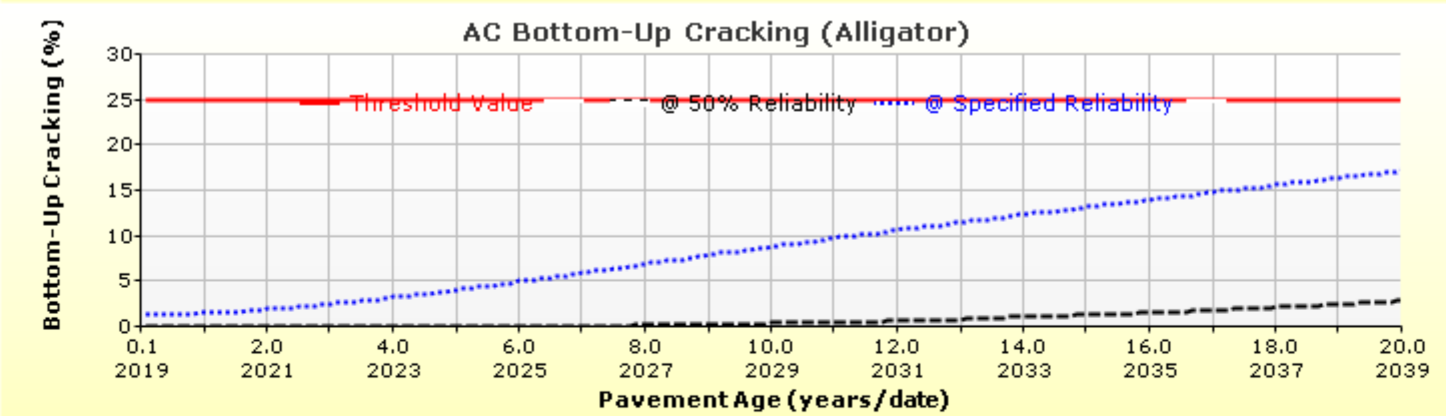
AC Top-Down Damage
@Surface
(Depth=.5in)
Top-Down Damage (%)
0.012
0.01
0.008
0.006
0.004
0.002
0
0.1 2019
2.0 2021
4.0 2023
6.0 2025
8.0 2027
10.0 2029
12.0 2031
14.0 2033
16.0 2035
18.0 2037
20.0 2039
Pavement Age (years/date)



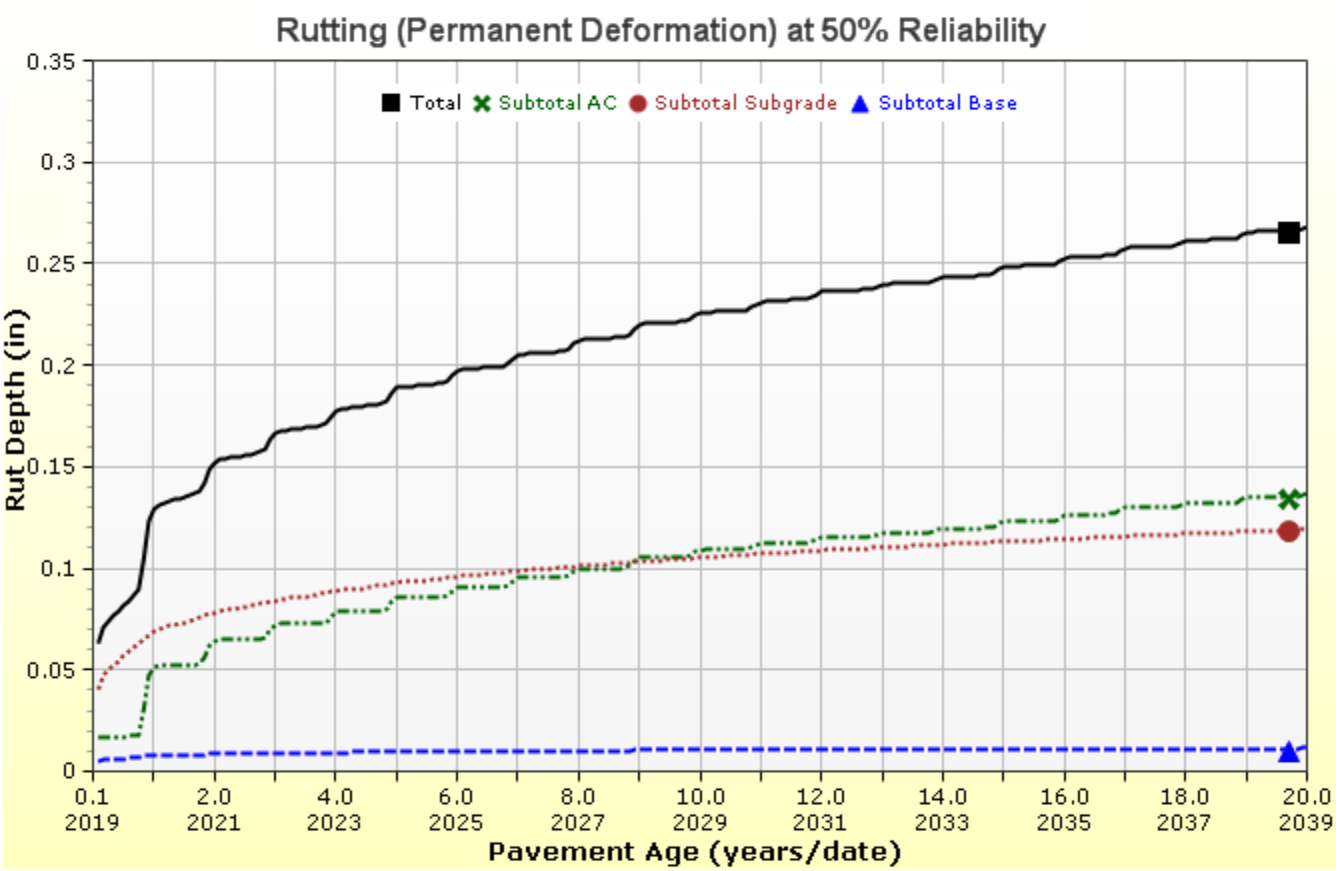
AC Bottom-Up Damage
Bottom-Up Damage (%)
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0.1 2019
2.0 2021
4.0 2023
6.0 2025
8.0 2027
10.0 2029
12.0 2031
14.0 2033
16.0 2035
18.0 2037
20.0 2039
Pavement Age (years/date)



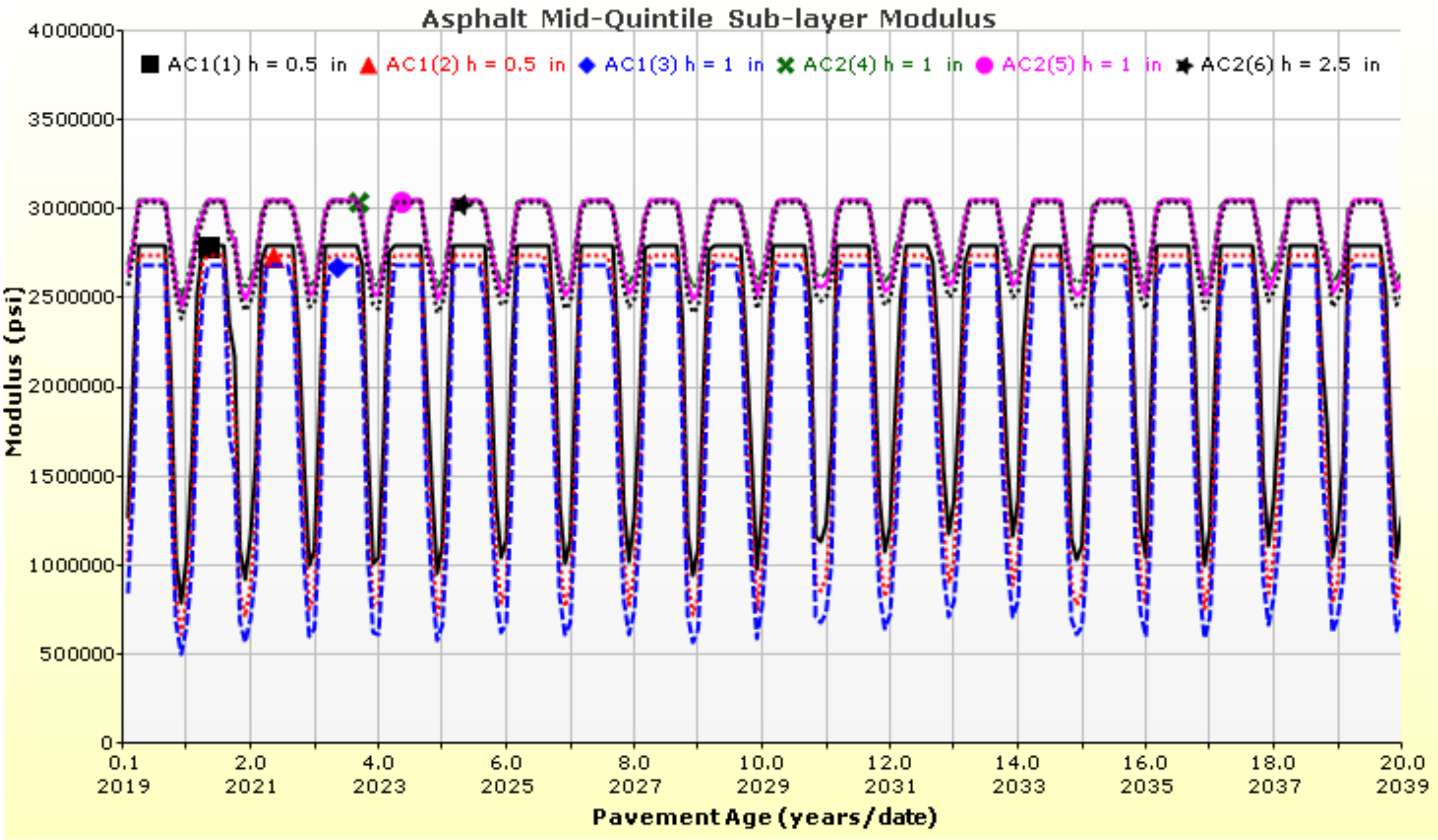
AC Top-Down Cracking (Longitudinal)
Threshold Value
@ 50% Reliability (Surface)
at 50% Reliability (Depth=.5in)
@Specified Reliability
Top-Down Cracking (ft/mile
3000
2500
2000
1500
1000
500
0
0.1 2019
2.0 2021
4.0 2023
6.0 2025
8.0 2027
10.0 2029
12.0 2031
14.0 2033
16.0 2035
18.0 2037
20.0 2039
Pavement Age (years/date)



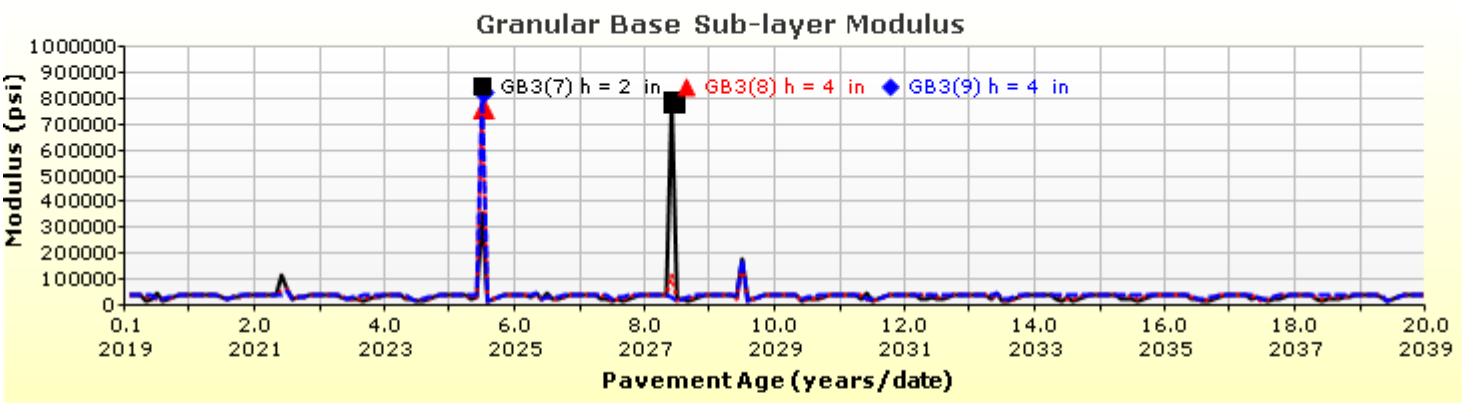
AC Bottom-Up Cracking (Alligator)
Threshold Value
@ 50% Reliability
@ Specified Reliability
Bottom-Up Cracking (%)
30
25
20
15
10
5
0
0.1 2019
2.0 2021
4.0 2023
6.0 2025
8.0 2027
10.0 2029
12.0 2031
14.0 2033
16.0 2035
18.0 2037
20.0 2039
Pavement Age (years/date)



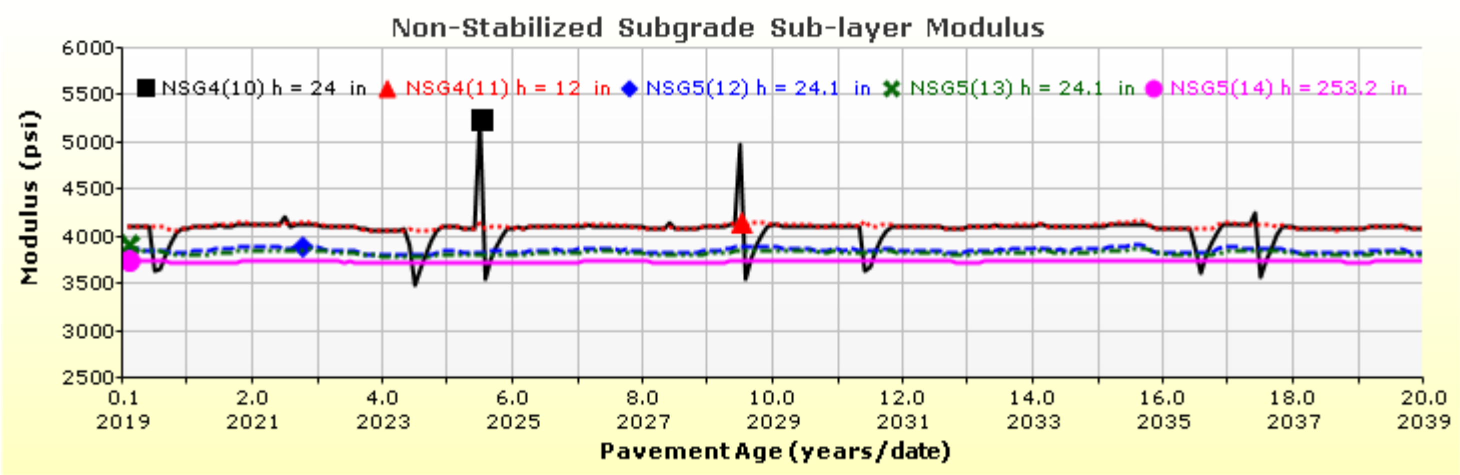
Rutting (Permanent Deformation) at 50% Reliability
Total
Subtotal AC
Subtotal Subgrade
Subtotal Base
Rut Depth (in)
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0.1 2019
2.0 2021
4.0 2023
6.0 2025
8.0 2027
10.0 2029
12.0 2031
14.0 2033
16.0 2035
18.0 2037
20.0 2039
Pavement Age (years/date)



Asphalt Mid-Quintile Sub-layer Modulus
AC1(1) h = 0.5 in
AC1(2) h = 0.5 in
AC1(3) h = 1 in
AC2(4) h = 1 in
AC2(5) h = 1 in
AC2(6) h = 2.5 in
Modulus (psi)
Pavement Age (years/date)



Granular Base Sub-layer Modulus
GB3(7) h = 2 in
GB3(8) h = 4 in
GB3(9) h = 4 in
Modulus (psi)
Pavement Age (years/date)



Non-Stabilized Subgrade Sub-layer Modulus
NSG4(10) h = 24 in
NSG4(11) h = 12 in
NSG5(12) h = 24.1 in
NSG5(13) h = 24.1 in
NSG5(14) h = 253.2 in
Modulus (psi)
Pavement Age (years/date)

Layer Information

Layer 1 Flexible : R6 Level 1 SX(100) PG 76-28

Asphalt		
Thickness (in)	2.0	
Unit weight (pcf)	145.0	
Poisson's ratio	Is Calculated?	True
	Ratio	-
	Parameter A	-1.63
	Parameter B	3.84E-06

Asphalt Dynamic Modulus (Input Level: 1)

T (ºF)	0.5 Hz	1 Hz	10 Hz	25 Hz
14	1821960	2284749	2635719	2743629
40	761414	1245330	1773800	1972669
70	186328	368894	694551	866370
100	59960	102426	195476	256712
130	32727	44234	68258	84345

Asphalt Binder

Temperature (ºF)	Binder Gstar (Pa)	Phase angle (deg)
158	1559	64
168.8	859	66
179.6	493	68

General Info

Name	Value
Reference temperature (ºF)	70
Effective binder content (%)	11.1
Air voids (%)	5.2
Thermal conductivity (BTU/hr-ft-ºF)	0.67
Heat capacity (BTU/lb-ºF)	0.23

Identifiers

Field	Value
Display name/identifier	R6 Level 1 SX(100) PG 76-28
Description of object	Mix ID # FS1939
Author	CDOT
Date Created	4/3/2013 12:00:00 AM
Approver	CDOT
Date approved	4/3/2013 12:00:00 AM
State	Colorado
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	SX
User defined field 2	
User defined field 3	
Revision Number	0

Layer 2 Flexible : R1 Level 1 S(100) PG 64-22

Asphalt		
Thickness (in)	4.5	
Unit weight (pcf)	142.9	
Poisson's ratio	Is Calculated?	True
	Ratio	-
	Parameter A	-1.63
	Parameter B	3.84E-06

Asphalt Dynamic Modulus (Input Level: 1)

T (ºF)	0.5 Hz	1 Hz	10 Hz	25 Hz
14	2969000	2996000	3060900	3078700
40	2743500	2800900	2942500	2982200
70	2279700	2389100	2676500	2761700
100	1618700	1773200	2227200	2376100
130	942200	1094700	1624700	1827300

Asphalt Binder

Temperature (ºF)	Binder Gstar (Pa)	Phase angle (deg)
147.2	1857	81.6
158	889	83.1
168.8	451	85

General Info

Name	Value
Reference temperature (ºF)	70
Effective binder content (%)	12.09
Air voids (%)	6.78
Thermal conductivity (BTU/hr-ft-ºF)	0.67
Heat capacity (BTU/lb-ºF)	0.23

Identifiers

Field	Value
Display name/identifier	R1 Level 1 S(100) PG 64-22
Description of object	Mix ID # FBR 044A-010
Author	CDOT MP
Date Created	5/3/2016 12:00:00 AM
Approver	CDOT
Date approved	5/3/2016 12:00:00 AM
State	Colorado
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	S
User defined field 2	
User defined field 3	
Revision Number	0

Layer 3 Non-stabilized Base : A-1-a

Unbound	
Layer thickness (in)	10.0
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 3)

Analysis Type:	Modify input values by temperature/moisture
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
25000.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	Value
Display name/identifier	A-1-a
Description of object	CDOT Class 6
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	6.0
Plasticity Index	1.0
Is layer compacted?	True

	Is User Defined?	Value
Maximum dry unit weight (pcf)	False	128
Saturated hydraulic conductivity (ft/hr)	False	1.93e-02
Specific gravity of solids	False	2.7
Water Content (%)	False	7.2

User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	5.5103
bf	2.0224
cf	0.7256
hr	115.0000

Sieve Size	% Passing
0.001mm	
0.002mm	
0.020mm	
#200	7.5
#100	
#80	
#60	
#50	
#40	
#30	
#20	
#16	
#10	
#8	40.0
#4	47.5
3/8-in.	
1/2-in.	
3/4-in.	95.0
1-in.	100.0
1 1/2-in.	
2-in.	
2 1/2-in.	
3-in.	
3 1/2-in.	

Layer 4 Subgrade : A-7-6

Unbound	
Layer thickness (in)	36.0
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 3)

Analysis Type:	Modify input values by temperature/moisture
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
6700.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	Value
Display name/identifier	A-7-6
Description of object	Properly Compacted Existing Site Soils
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	51.0
Plasticity Index	30.0
Is layer compacted?	True

	Is User Defined?	Value
Maximum dry unit weight (pcf)	False	98.6
Saturated hydraulic conductivity (ft/hr)	False	8.849e-06
Specific gravity of solids	False	2.7
Water Content (%)	False	22.2

User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	136.4179
bf	0.5183
cf	0.0324
hr	500.0000

Sieve Size	% Passing
0.001mm	
0.002mm	
0.020mm	
#200	79.1
#100	
#80	84.9
#60	
#50	
#40	88.8
#30	
#20	
#16	
#10	93.0
#8	
#4	94.9
3/8-in.	96.9
1/2-in.	97.5
3/4-in.	98.3
1-in.	98.8
1 1/2-in.	99.3
2-in.	99.6
2 1/2-in.	
3-in.	
3 1/2-in.	99.9

Layer 5 Subgrade : A-7-6

Unbound	
Layer thickness (in)	Semi-infinite
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 3)

Analysis Type:	Modify input values by temperature/moisture
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
6700.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	Value
Display name/identifier	A-7-6
Description of object	Existing Native Soils and Bedrock
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	51.0
Plasticity Index	30.0
Is layer compacted?	False

	Is User Defined?	Value
Maximum dry unit weight (pcf)	False	97.7
Saturated hydraulic conductivity (ft/hr)	False	8.946e-06
Specific gravity of solids	False	2.7
Water Content (%)	False	22.2

User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	136.4179
bf	0.5183
cf	0.0324
hr	500.0000

Sieve Size	% Passing
0.001mm	
0.002mm	
0.020mm	
#200	79.1
#100	
#80	84.9
#60	
#50	
#40	88.8
#30	
#20	
#16	
#10	93.0
#8	
#4	94.9
3/8-in.	96.9
1/2-in.	97.5
3/4-in.	98.3
1-in.	98.8
1 1/2-in.	99.3
2-in.	99.6
2 1/2-in.	
3-in.	
3 1/2-in.	99.9

Calibration Coefficients

AC Fatigue

$$N_f = 0.00432 * C * \beta_{f1} k_1 \left(\frac{1}{\varepsilon_1}\right)^{k_2 \beta_{f2}} \left(\frac{1}{E}\right)^{k_3 \beta_{f3}}$$

$$C = 10^M$$

$$M = 4.84\left(\frac{V_b}{V_a + V_b} - 0.69\right)$$

k1: 0.007566
k2: 3.9492
k3: 1.281
Bf1: 130.3674
Bf2: 1
Bf3: 1.217799

AC Rutting

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r1} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_3 \beta_{r3}}$$

$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth}$$

$$C_1 = -0.1039 * H_\alpha^2 + 2.4868 * H_\alpha - 17.342$$

$$C_2 = 0.0172 * H_\alpha^2 - 1.7331 * H_\alpha + 27.428$$

Where:

H_{ac} = total AC thickness(in)

ε_p = plastic strain(in/in)

ε_r = resilient strain(in/in)

T = layer temperature(°F)

N = number of load repetitions

AC Rutting Standard Deviation	0.1414 * Pow(RUT,0.25) + 0.001	
AC Layer	K1:-3.35412 K2:1.5606 K3:0.3791	Br1:4.3 Br2:1 Br3:1

Thermal Fracture

$$C_f = 400 * N\left(\frac{\log C / h_{ac}}{\sigma}\right)$$

$$\Delta C = (k * \beta t)^{n+1} * A * \Delta K^n$$

$$A = 10^{(4.389 - 2.52 * \log(E * \sigma_m * n))}$$

C_f = observed amount of thermal cracking(ft/500ft)
k = refression coefficient determined through field calibration
$N()$ = standard normal distribution evaluated at()
σ = standard deviation of the log of the depth of cracks in the pavments
C = crack depth(in)
h_{ac} = thickness of asphalt layer(in)
ΔC = Change in the crack depth due to a cooling cycle
ΔK = Change in the stress intensity factor due to a cooling cycle
A, n = Fracture parameters for the asphalt mixture
E = mixture stiffness
σ_M = Undamaged mixture tensile strength
β_t = Calibration parameter

Level 1 K: 6.3	Level 1 Standard Deviation: 0.1468 * THERMAL + 65.027
Level 2 K: 0.5	Level 2 Standard Deviation: 0.2841 * THERMAL + 55.462
Level 3 K: 6.3	Level 3 Standard Deviation: 0.3972 * THERMAL + 20.422

CSM Fatigue

$$N_f = 10^{\left(\frac{k_1 \beta_{c1}\left(\frac{\sigma_s}{M_r}\right)}{k_2 \beta_{c2}}\right)}$$

N_f = number of repetitions to fatigue cracking
σ_s = Tensile stress(psi)
M_r = modulus of rupture(psi)

k1: 1	k2: 1	Bc1: 0.75	Bc2:1.1

Subgrade Rutting

$$\delta_a(N) = \beta_{s_1} k_1 \varepsilon_v h \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \left| e^{-\left(\frac{\rho}{N}\right)^\beta} \right|$$

$\delta_a = permanent\ deformation\ for\ the\ layer$
$N = number\ of\ repetitions$
$\varepsilon_v = average\ veritcal\ strain(in/in)$
$\varepsilon_0, \beta, \rho = material\ properties$
$\varepsilon_r = resilient\ strain(in/in)$

Granular		Fine	
k1: 2.03	Bs1: 0.22	k1: 1.35	Bs1: 0.37
Standard Deviation (BASERUT) 0.0104* Pow(BASERUT,0.67) + 0.001		Standard Deviation (BASERUT) 0.0663* Pow(SUBRUT,0.5) + 0.001	

AC Cracking

AC Top Down Cracking

$$FC_{top} = \left(\frac{C_4}{1 + e^{(C_1 - C_2 * log_{10}(Damage))}}\right) * 10.56$$

c1: 7	c2: 3.5	c3: 0	c4: 1000

AC Cracking Top Standard Deviation

200 + 2300/(1+exp(1.072-2.1654*LOG10 (TOP+0.0001)))

AC Bottom Up Cracking

$$FC = \left(\frac{6000}{1 + e^{(C_1 * C_1' + C_2 * C_2' log_{10}(D*100))}}\right) * \left(\frac{1}{60}\right)$$

$$C_2' = -2.40874 - 39.748 * (1 + h_{ac})^{-2.856}$$

$$C_1' = -2 * C_2'$$

c1: 0.021	c2: 2.35	c3: 6000

AC Cracking Bottom Standard Deviation

1 + 15/(1+exp(-3.1472-4.1349*LOG10 (BOTTOM+0.0001)))

CSM Cracking

$$FC_{ctb} = C_1 + \frac{C_2}{1 + e^{C_3 - C_4(Damage)}}$$

C1: 0	C2: 75	C3: 5	C4: 3

CSM Standard Deviation

CTB*1

IRI Flexible Pavements

C1 - Rutting
C2 - Fatigue Crack
C3 - Transverse Crack
C4 - Site Factors

C1: 50	C2: 0.55	C3: 0.0111	C4: 0.02