Geotechnical Evaluation
West Lake Creek Road Bridge Over East Lake Creek
Edwards, Colorado

Prepared For:

Eagle County

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Attention: Mr. Richard Davies, P.E., PTOE

Job Number: 23-6012 **December 7, 2023**

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical evaluation performed by GROUND Engineering Consultants, Inc. (GROUND) in support of design of the foundations for the proposed prefabricated bridge to be installed to carry West Lake Creek Road over East Lake Creek in Edwards, Colorado. Our study was conducted in general accordance with the Agreement between Eagle County and GROUND Engineering Consultants, Inc., dated October 30, 2023, and GROUND's Proposal Number 2310-2057, dated October 12, 2023.

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 classification and 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 development/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 Eagle County, based on our understanding of the 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 the provided information,[1] we understand that present plans call for a prefabricated TrueNorth Steel Modular Vehicular Bridge Structure to replace the existing bridge structure that at East Lake Creek. The dimensions of the proposed bridge are anticipated to be about 60 feet in length and 32 feet in width. Abutment and wing walls are also anticipated. We understand that a geosynthetic reinforced soil (GRS) system could be used as part of the abutment and wing walls. We anticipate that the proposed bridge and associated abutment walls and wing walls will bear at depths of approximately 14 to 15 feet below existing road grades. Structural loads are anticipated to be relatively moderate, typical of this type of construction. No other improvements were included in this scope of services.

We understand that the existing bridge will be demolished and the new one constructed within the same approximate footprint.

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 reevaluate the conclusions and parameters presented herein.

Performance Expectations Based on our experience with other, similar projects, we understand that post-construction, bridge foundation on the order of 1 inch are acceptable to, and anticipated by Eagle County, as are the resultant distress and maintenance measures. 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.

[1] E-mail correspondence between GROUND and Richard Davies, October 10, 2023.

SITE CONDITIONS

At the time of our subsurface exploration, the project site was developed as a single-span, two-lane bridge carrying West Lake Creek Road over East Lake Creek. Relatively minor to moderate signs of distress were observed on the asphalt-paved bridge deck, including longitudinal and transverse cracking, small depressions, and local patch failures. Steel decking was visible on the edges of the bridge. The area beneath the bridge was generally inaccessible, but we observed that the bridge had concrete wing walls and abutments. The bridge deck appeared to be supported by steel I-beams.

Vegetation near the bridge consisted of a dense growth of deciduous and coniferous trees and short to medium grasses. Based on public utility marks, buried utilities including gas, water, and communication lines, were located near the bridge, with several utilities running across the bridge in conduits. Overhead utilities were also present.

The creek flowed northward with several feet of water in the channel. Stream stage was approximately 10 feet below the bridge deck at the time of our field work. The channel contained numerous boulders, up to about 4 feet in diameter.

Based on review of historical topographic maps, it appeared that West Lake Creek Road approached the crossing from the northeast prior to about 1980, rather than from the east as it did at the time of this report preparation. Although, the bridge appeared to be in approximately the same location. We are not aware if the bridge was replaced when the roadway was re-aligned. Based on review of Google Earth historical aerial imagery, the site did not appear to have undergone significant improvements since 1999, the earliest available images.

SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted in November 2023 with a truck-mounted drilling rig to evaluate the subsurface conditions as well as to retrieve soil samples for laboratory testing and analysis. Test Hole 1 was drilled near the west abutment by advancing ODEX air-percussion equipment to a depth of about 28 feet. Continuous flight auger was used at Test Hole 2 near the east abutment, where practical auger refusal was encountered at a depth of about 5½ feet below existing grade. Another attempt to drill was made at a location offset about 5 feet to the east from the original test hole location and refusal was encountered at a depth of 3 feet.

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 retrieved with a 2-inch inner diameter California liner sampler and a 1⅜-inch inner diameter Standard Penetration Test sampler. The samplers were driven into the substrata with blows from a 140-pound hammer falling 30 inches, in general accordance with (in the case of the 1⅜-inch sampler) the Standard Penetration Test described by ASTM Method D1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of soils. Depths at which the samples were taken, and associated penetration resistance values are shown on the test hole logs.

The approximate locations of the test holes are shown in Figure 1. Logs of the test holes is presented in Figure 2. Explanatory notes and a legend are provided in Figure 3. Detailed logs of the test holes are provided in Appendix A.

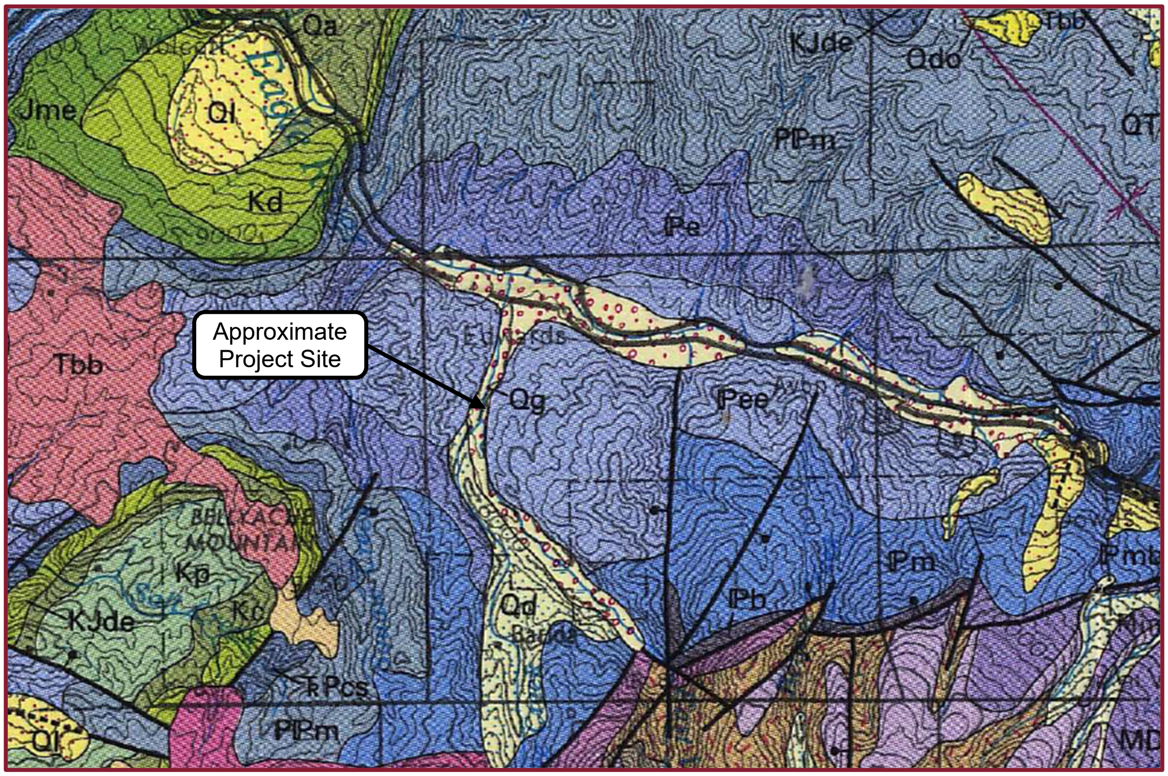
LABORATORY TESTING

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil samples included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, and Atterberg limits. Water-soluble sulfate content and a suite of corrosivity tests were completed on selected samples, as well. Laboratory tests were performed in general accordance with applicable ASTM protocols. Results of the laboratory testing program are summarized in Tables 1 and 2. Gradation plots are provided in Figures 4, 5, and 6.

SUBSURFACE CONDITIONS

Geologic Setting Published geologic maps, e.g., Tweto et al., 1978,[2] depicted the site as underlain by Pleistocene-aged alluvial gravels (**Qg**). Also mapped along the creek channel, near the project site, were Pleistocene-aged glacial drift (**Qd**). These surficial deposits were mapped as being underlain by the Pennsylvanian aged Eagle Valley Evaporite (**Pee**). A portion of the above-referenced geologic map is reproduced below.

In the project area, alluvium typically consist of gravels, boulders, sand, and silt. Glacial drift typically consists of boulders, sands, and gravels. The coarse materials in these deposits will not be suitable for reuse as compacted fill without crushing.



The Eagle Valley Evaporite, in the project area consists largely of evaporites, including gypsum, anhydrite, and halite, interbedded with claystones, siltstones, and sandstones on various scales. Evaporites are subject to dissolution in the near surface when exposed to water. Sinkholes and other subsidence features may form as a result. The claystones

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[2] Tweto, Ogden, Moench, Robert H., and Reed Jr., John C. (1978). *Geologic map of the Leadville 1-degree x 2-degrees Quadrangle, Northwestern Colorado.* U.S. Geological Survey Miscellaneous Investigations Series I-999. 1:250,000.

typically are moderately expansive and the formation includes well-cemented beds that can be very hard and can be difficult to excavate.

Dissolution Related Subsidence Although not encountered at our test holes, evaporite minerals, such as gypsum, anhydrite, and halite, have been mapped underlying the project site. These materials are subject dissolution as water passes over or through them, creating voids that can collapse and generate sinkholes or other subsidence features. Evidence of such dissolution features were not observed at the bridge site, but have been noted in the greater project area. Sinkholes, including relatively large diameter sinkholes, are known to have formed in the greater project area. The likelihood of the development of a sinkhole or other subsidence feature at a given location is difficult to forecast. Additional geotechnical drilling and geophysical studies attempting to locate nascent sink holes in the near surface can be performed, but have been unreliable, in our experience. However, the likelihood of sinkholes and subsidence related to subsurface dissolution at the site appeared to be similar to other locations in the general area.

Additionally, given the bridge and associated roadway alignment at their present locations, GROUND assumed that the Eagle County was aware of and accepts the risks of dissolution-related subsidence. We are available to discuss further this risk upon request.

Local Conditions In general, the test holes penetrated about 5 and 6 inches of asphalt[3] and then 6 inches of a road base-like material before penetrating fill soils that were recognized to depths of about 4 and 5½ feet below existing grades. Beneath the fill soils, native sands, gravels, and boulders were encountered and they extended to the depths explored.

We interpret the fill materials to be materials placed in during the construction of the West Lake Creek Road and the associated bridge and during the development of the greater project area. (See the *Site Conditions* section of this report.) We interpret the native sands, gravels, and boulders to be alluvial (stream-laid) deposits.

Fill materials were recognized in the test holes and likely are present across the site. These fill soils may contain coarse gravels and boulders, as well as similarly sized pieces of construction, debris even though these items where not recognized in the test holes.

[3] Asphalt thicknesses are difficult to determine with precision in small diameter test holes. If existing pavement thicknesses are of significance to the project, then additional, larger diameter test holes should be drilled.

Delineation of the complete lateral and vertical extents of the fills at the site and their compositions was beyond our present scope of services. If more detailed information regarding fill extents and compositions at the site are of significance, they should be evaluated using test pits.

Similarly, coarse gravels and larger clasts 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 generally consisted of fine to coarse, clayey to silty sands and gravels with boulders. They were slightly plastic, moist, and dark-brown in color. Organic material was commonly observed.

Sands, Gravels, and Boulders generally consisted of fine to coarse, relatively clean sands, gravels, and boulders. They were non-plastic, moist to wet, dense to very dense, and brown to gray in color.

Groundwater was encountered in Test Hole 1 at a depth of 10 feet below existing grade at the time of drilling. The test holes were backfilled upon drilling completion per Code of Colorado Regulations (2 CCR 402-2). Specifically, it has been our experience that surface and groundwater levels fluctuate greatly in mountainous areas, primarily due to seasonal conditions such as spring runoff. This site is considered to be comparatively more vulnerable to fluctuating groundwater levels due to its proximity to the East Lake Creek. These conditions are often highly variable and difficult to predict. Although these conditions generally exist for 1 to 3 months annually, their impact on design can be significant. In the project area, it is common during construction to encounter dry conditions in the fall and wet conditions in the spring with relative groundwater fluctuations of 10 feet or more. This is particularly critical for foundation and deep utility excavations, cut slopes, culvert sizing, and for development adjacent to intermittently dry streams, rivers, and wetlands.

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 County desires additional/repeated groundwater level observations, GROUND should be contacted; additional exploration and fees will be necessary in this regard.

SEISMIC CLASSIFICATION

Based on extrapolation of available data to depth and our experience in the project area, we consider the areas of the proposed bridge location to likely meet the criteria for a Seismic Site Classification of **C** according to the AASHTO LRFD Bridge Design Specifications, Eighth Edition (Table 3.10.3.1-1). (Exploration and/or shear wave velocity testing to a depth of 100 feet or more was not part of our present scope of services.) If, however, a quantitative assessment of the site seismic properties is desired, then shear wave velocity testing should be performed. GROUND can provide a fee estimate for shear wave velocity testing upon request. We consider the likelihood of achieving a Site Class B to be low.

The following seismic parameters are applicable the bridge sites:

Peak Ground Acceleration (PGA):	0.086 g
Short Period Spectral Acceleration (S_S):	0.172 g
Long Period Spectral Acceleration (S_1):	0.042 g
F_{PGA}:	1.2
F_a:	1.2
F_V:	1.7
SD_S:	0.206 g
SD_1:	0.071 g
AASHTO Seismic Zone:	1

A seismic response spectrum for the site, based on these parameters are provided on Figure 7.

GEOTECHNICAL CONSIDERATIONS FOR DESIGN

One source of geotechnical risk at this site is the presence of undocumented fill soils at the site. Although these fill soil may have been placed in a controlled manner, documentation of the existing fill soils was not provided to GROUND at the time of report preparation. Therefore, these fill soils are considered to be undocumented fill soils. Undocumented fill soils are considered to be geotechnically unsuitable to support new construction given their unknown composition and consistency. Significant post-construction movements can result where improvements are supported directly on these materials.

The age of the fill soils, appear to be greater than 30 years, and Eagle County's experience with the structure, Eagle County may consider the risk associated with these fills to be lower than other similar sites.

We anticipate that the proposed bridge and associated abutment walls and wing walls will bear at depths of approximately 14 to 15 feet below existing road grades at the test hole locations. At these elevations, native sands, gravels, and boulders were encountered are anticipated to provide sufficient support for the proposed construction without unusual post-construction movements.

Post-construction movements for the bridge bearing directly on these materials are estimated by GROUND to be about 1 inch.

Foundation Systems In GROUND's opinion, supporting the proposed building on a driven pile foundation system will provide the lowest estimates of post-construction movement (about ½ inch) and will provide the least risk of excessive foundation movements. Driven piles should bear in the sands, gravels, and boulders at the site. Due to the large boulders encountered at our test holes, we anticipate that pile driving may be particularly difficult at this site. Pre-drilling might be necessary. Geotechnical parameters for driven piles can be provided upon request.

As an alternative to support the bridge and related abutment and wing walls on driven piles, a shallow foundation system could be used. Shallow foundations should bear on the undisturbed native sands, gravels, and boulders. Where boulders greater than 12 inches in diameter are encountered in foundation excavations, they should be removed and resulting void should be filled in with relatively clean, approximately ¾-inch to 1-½ inch nominal crushed rock or concrete. Crushed rock, if used, should be wrapped in filter fabric (MiraFi® 140N or equivalent).

We understand that a geosynthetic reinforced soil (GRS) system may be utilized to reconstruct the abutment area. In such a case the shallow bridge foundations likely will bear on the reinforced soil, which GROUND considers to be suitable geotechnically, provided that the bridge loads are appropriately accounted for.

Where a GRS system is not utilized and soft, wet, or unstable soils are exposed at footing bottom elevations, the footings could be supported on at least 2 feet of relatively clean, approximately ¾-inch to 1-½ inch nominal crushed rock wrapped on the top and sides with filter fabric. A layer of geotextile, e.g., Mirafi® RS580i, HP 570, or equivalent, between the crushed rock and on-site soils should also be installed at the base of the crushed rock section. Greater thicknesses of crushed rock may be necessary and is dependent on the stability of the foundation soils and the depth of scour during a flood event. The crushed rock should extend at least 2 feet beyond the edge of the bridge abutments on all sides. If this approach is followed, then post-construction movements of approximately 1 inch or more should be anticipated. In the event these movements cannot be tolerated, a thicker crushed rock section or a deep foundation system should be considered; GROUND should be contacted to provide additional design parameters in this regard.

Additional parameters for the design and construction of shallow foundations are provided in the *Shallow Foundations* section of this report.

Groundwater and Surface Water Groundwater was encountered at Test Hole 1 at a depth of about 10 feet below existing road grade. This is similar to the stream stage at the time of drilling. Therefore, unstable conditions should be anticipated that may result in greater than typical construction difficulties, where such activities extend below this depth at either abutment. The contractor should be prepared to dewater the excavations during construction. Similarly, efforts to divert surface waters carried in East Lake Creek likely will be necessary as well. Special environmental considerations and regulations likely apply to the design and execution of efforts to handle the groundwater and surface water at the site. An environmental consultant should be retained in this regard.

SHALLOW FOUNDATIONS

The geotechnical parameters indicated below may be used for design of shallow foundations for the proposed bridge and associated abutment and wing walls.

Geotechnical Parameters for Shallow Foundation Design

1) The footings should be placed on firm native soils or a layer of <u>at least</u> 2 feet of crushed rock wrapped on the top and sides with filter fabric. (See the *Geotechnical Considerations for Design* section of this report.) A layer of geotextile, i.e., Mirafi RS580i, HP 570, or approved equivalent, should be placed between the crushed rock and on-site soils should also be installed.

2) Footings placed on firm native soils or a GRS system may be designed for an allowable soil bearing pressure of **4,000 psf** for footings up to 8 feet in width (least lateral dimension).

   Footings placed on a layer of crushed rock may be designed for an allowable soil bearing pressure of **3,000 psf** for footings up to 8 feet in width (least lateral dimension).

   Compression of the bearing soils for the provided allowable bearing pressure is estimated to be ¾ inch, based on an assumption of drained foundation conditions. If foundation soils are subjected to an increase/fluctuation in moisture content, the effective bearing capacity will be reduced and greater post-construction movements than those estimated above may result.

   To reduce differential settlements between foundation elements, footing loads should be as uniform as possible. Differentially loaded footings will settle differentially.

3) Geotechnical parameters for lateral resistance to foundation loads are provided in the *Lateral Earth Pressure* section of this report.

4) Footings should be provided with adequate soil cover above their bearing elevation for frost protection. Therefore, footings should be placed at a bearing elevation of at least 4 feet below the lowest adjacent finish grades. Additional embedment may be required for scour protection. Hydrometers plots are provided in Figures 5 and 6 may be used for scour design.

5) Compacted fill placed against the sides of the footings should be compacted to at least 95 percent relative compaction in accordance with the *Project Earthwork*

section of this report. Use of controlled low strength material (CLSM), i.e., a lean sand-cement slurry, flowable fill, or a similar material in lieu of compacted soil backfill in these locations may be beneficial where access is restricted or when it can be placed more rapidly than properly compacted soil fill. CLSM should be placed in general accordance with Section 206.02 of the CDOT Standard Specifications for Road and Bridge Construction.

6) Care should be taken when excavating to avoid disturbing the supporting materials. Hand excavation or careful backhoe soil removal may be required in excavating the last few inches.

7) All bridge abutment bearing areas should be compacted with a vibratory plate compactor prior to placement of concrete.

8) A geotechnical engineer should be retained to observe excavations prior to placement of rock, stabilization materials, abutments, etc.

9) The contractor should take adequate care when making excavations not to compromise the bearing or lateral support for nearby improvements.

10) Headwalls should be designed with adequate provisions for drainage, facilitated by well-designed weep holes at the toes of the walls, or by underdrains consisting of gravel, perforated pipe and filter fabric at the heels of the walls, along the tops of the wall footings. Underdrains should be sloped at gradients of at least 1 percent to locations where they may discharge freely.

Shallow Foundation Construction

11) Care should be taken when excavating the foundations to avoid disturbing the supporting materials particularly in excavating the last few inches.

12) A crushed rock section thicker than 2 feet may be necessary if unsuitable materials including but not limited to saturated, near-saturated, muck-like or yielding bearing materials are exposed at the bottom of the excavation. Use of concrete in lieu of compacted soil backfill in these locations may be beneficial where access is restricted or when it can be placed more rapidly than properly compacted soil fill.

13) If boulders greater than 12 inches in diameter are encountered in the foundation excavations, they should be removed and the resultant void be backfilled with properly compacted soils or crushed rock. A clearance of 12 inches between the footing and large boulders should be achieved.

14) Foundation-supporting soils may be disturbed or deform excessively under the wheel loads of heavy construction vehicles as the excavations approach footing bearing levels. Construction equipment should be as light as possible to limit development of this condition. The movement of vehicles over proposed foundation areas should be restricted.

15) All foundation subgrade should be properly cleaned/compacted so that no loose soils remain, prior to placement of concrete.

LATERAL LOADS

Shallow Foundations Resisting Lateral Loads Values for equivalent fluid pressures and the coefficient for frictional resistance to sliding are provided below. These values were based on a moist unit weight (γ') of 130 pcf and an angle of internal friction (ϕ) of 28 degrees for site soils reworked as fill, and a γ' of 115 pcf and a ϕ of 40 degrees for free-draining crushed rock (ASTM C33 No. 57/67), and are unfactored. Appropriate factors of safety should be included in design calculations.

EQUIVALENT FLUID WEIGHTS (DRAINED CONDITION)

Backfill Material	Condition			Friction Coefficient
	Active	At-Rest	Passive	
Native Soils and Fill Re-Worked as Properly Compacted Fill	**47 pcf**	**69 pcf**	**320 pcf** (to a maximum of 3,200 psf)	**0.35**
Free-Draining Crushed Rock (ASTM C33 No. 57/67)	**26 pcf**	**42 pcf**	**-**	**0.56**

EQUIVALENT FLUID WEIGHTS (SUBMERGED CONDITION)

Backfill Material	Condition			Friction Coefficient
	Active	At-Rest	Passive	
Native Soils and Fill Re-Worked as Properly Compacted Fill	**87 pcf**	**99 pcf**	**245psf** (to a maximum of 2,450 psf)	**0.35**
Free-Draining Crushed Rock (ASTM C33 No. 57/67)	**74 pcf**	**82 pcf**	**-**	**0.56**

To realize the lower equivalent fluid unit weights, the selected structure backfill should be placed behind the wall to a minimum distance equal to half the retained wall height.

Additionally, where passive soil pressure is analyzed, the upper 1 foot of embedment should be neglected for passive resistance. Where passive soil pressure is used to resist lateral loads, it should be understood that significant lateral strains will be required to mobilize the full value indicated above, likely 1 inch or more. A reduced passive pressure can be used for reduced anticipated strains, however.

The lateral earth pressures presented above are for a horizontal upper backfill slope. The additional loading of an upward sloping backfill as well as loads from traffic, stockpiled materials, etc., should be included in the wall design.

WATER-SOLUBLE SULFATES

The concentrations of water-soluble sulfates measured in a selected sample obtained from the test hole was approximately 0.05 percent. Such a concentration of water-soluble sulfates represents a negligible environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of “negligible,” “moderate,” “severe,” and “very severe” as described in the “Design and Control of Concrete Mixtures,” published by the Portland Cement Association (PCA). 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, all concrete exposed to site soils should use sulfate-resistant cement conforming to one of the Class 0 requirements. However, Eagle Valley Evaporite deposits are mapped in close proximity to the project site. Where East Lake Creek carries sediments derived from the evaporite, there is an increased risk of elevated sulfate concentrations to be present, higher than measured in our laboratory. Therefore, it may be beneficial to consider the use of Class 1 or higher sulfate resistant cement. The requirements for Class 0 and Class 1 sulfate-resistant cement are presented below:

Class 0 (Negligible)

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

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

Class 1 (Moderate)

1) ASTM C150 Type II or V.

2) ASTM C595 Type IP(MS) or IP(HS).

3) ASTM C150 Type III. Type III shall have no more than 8 percent C3A.

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

Class C fly ash shall not be substituted for cement.

In addition, all concrete used shall have a minimum compressive strength of 4,000 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.

SOIL CORROSIVITY

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

Reduction-Oxidation testing indicated a red-ox potential of approximately -78 millivolts. Such a low potential typically creates a more corrosive environment.

Sulfide Reactivity testing indicated a "Positive" result in the local soils. The presence of sulfides in the soils suggests a more corrosive environment.

Soil Resistivity In order to assess the "worst case" for mitigation planning, samples of materials retrieved from the test holes were tested for resistivity in the laboratory, after being saturated with water, rather than in the field. Resistivity also varies inversely with temperature. Therefore, the laboratory measurements were made at a controlled temperature. Measurement of electrical resistivity indicated a value of approximately 2,206 ohm-centimeters in a sample of site soils.

pH Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity.[4] Our testing indicated a pH value of about 8.5.

Corrosivity Assessment The American Water Works Association (AWWA) has developed a point system scale used to predict corrosivity. The scale is intended for

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

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.

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.

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 on the values obtained for the soil parameters, the fill and native soils appear to comprise a severely corrosive environment for ferrous metals (10½ points). In addition, in our experience, landfill materials also comprise severely corrosive environments.

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.

PROJECT EARTHWORK

The earthwork criteria below are based on our interpretation of the geotechnical conditions encountered in the test hole. Where these criteria differ from applicable municipal specifications, the latter should be considered to take precedence.

Prior to earthwork construction, existing vegetation, topsoil, asphalt, and other deleterious materials should be removed and disposed of off-site. Relic underground utilities, if encountered, should be abandoned in accordance with applicable regulations, removed as necessary, and capped at the margins of the property.

Topsoil should not be incorporated into fill placed on the site. Instead, topsoil should be stockpiled during initial grading operations for placement in areas to be landscaped or for other approved uses.

Tree trunks and roots may be present within, under, or adjacent to the proposed improvements. The contractor should take care to assure that all tree roots are removed prior to filling or construction of improvements. Relatively deep excavations may be required to accomplish proper removal of roots and associated organic materials.

Use of Existing Fill Soils Fill materials were encountered in the test holes during subsurface exploration. We anticipate that these soils generally will be suitable for reuse as compacted fill for general purposes. However, because all of the fill soils were not sampled and tested, it is possible that some fill soils may not be suitable for reuse as compacted fill, due to the presence of deleterious materials such as trash, organic material, boulders, or construction debris. 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, we anticipate that the existing site soils free of organic materials, boulders, or other deleterious materials will be suitable, in general, for reuse as compacted fill for general fills.

Fragments of rock and boulders larger than **12 inches** in maximum dimension will require special handling and/or placement to be incorporated into project fills and may need to be wasted on-site beyond the proposed improvements or exported from the site. Specialized screening, sieving, or crushing equipment may be necessary to prepare excavated on-site soils for reuse beneath the proposed improvements. If such equipment is unavailable to the contractor, it should be anticipated that the import of soils to the site will be necessary to backfill excavations. 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 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 Materials imported to the site as (common) fill should be free of organic material, and other deleterious materials. Imported material should exhibit **15 percent or less than** passing the No. 200 Sieve and a plasticity index of **10 or less**. Materials proposed for import should be approved prior to transport to the site.

Fill Platform Preparation Prior to filling, the top **12 inches** of in-place materials on which fill soils will be placed 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.

Wet, Soft, or Unstable Subgrades Wet, soft, or unstable subgrades may be encountered at this site. The contractor must establish a stable platform for fill placement and achieving compaction in the overlying fill soils and to place foundations. Therefore, excavation of the unstable soils and replacing them with relatively dry or granular material, possibly together with the use of stabilization geotextile or geogrid, may be necessary to achieve stability. Although the stabilization approach should be determined by the contractor, GROUND offers the alternatives below for consideration. Proof-rolling can be beneficial for identifying unstable areas.

- Replacement of the existing subgrade soils with clean, coarse, aggregate (e.g., crushed rock or "pit run" materials) or road base. Excavation and replacement to a depth of 1 to 2 feet commonly is sufficient, but greater depths may be necessary to establish a stable surface.

  On very weak subgrades, an 18- to 24-inch "pioneer" lift that is not well compacted may be beneficial to stabilize the subgrade. Where this approach is employed, however, additional settlements of up to ½ inch may result.

- Where coarse, aggregate alone does not appear sufficient to provide stable conditions, it can be beneficial to place a layer of stabilization geotextile or geogrid (e.g., TenCate Mirafi® RS 580*i*, BXG 110, or other similar product) at the base of the aggregate section.

  The stabilization geotextile/geogrid should be selected based on the aggregate proposed for use. It should be placed and lapped in accordance with the manufacturer's recommendations.

  Geotextile or geogrid products can be disturbed by the wheels or tracks of construction vehicles; care should be taken to maintain the effectiveness of the system. Placement of a layer of aggregate over the geotextile/geogrid prior to allowing vehicle traffic over it can be beneficial in this regard.

When a given remedial approach has been selected, the contractor should construct a test section to evaluate the effectiveness of the approach prior to use over a larger area.

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

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

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

GROUND's experience within the project area suggests the frost depth to be approximately 3 feet, below ground surface.

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

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

Compaction Criteria Soils should be compacted to **95 or more percent** of the maximum dry density at moisture contents **within 2 percent** of the optimum moisture content as determined by ASTM D1557, the modified Proctor.

Use of Squeegee Where "squeegee" or similar materials are proposed for use by the Contractor, the design team should be notified by means of a Request for Information (RFI), so that the proposed use can be considered on a case-by-case basis. Where squeegee meets the project requirements for pipe bedding material, however, it is acceptable for that use.

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

Cut and Filled Slopes Permanent, graded slopes supported by local soils up to **15 feet** in height should be constructed no steeper than **3 : 1** (horizontal : vertical). Minor raveling or surficial sloughing should be anticipated on slopes cut at this angle until vegetation is well 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 Hole 1 for the subsurface exploration was advanced to the depth indicated on the test hole log by means of ODEX air-percussion equipment due to practical auger drilling refusal at a relatively shallow depth at Test Hole 2. The presence of boulders was interpreted within the fill and native soils based on drilling conditions and visual observations of the shallow portions of the test holes. Significant quantities of boulders should be anticipated by the contractor. Crushing or other size-reducing methods may be necessary to sufficiently reduce/process these materials adequately for use in site fills. If these methods are unavailable to the contractor, they should anticipate that fill soil will need to be imported to the site for use as backfill.

Additionally, construction debris (concrete, asphalt, rebar, tree limbs, wood, etc.) may be encountered within existing fill materials throughout the site. Boulders should also be anticipated to be encountered within these materials. These materials should be expected to be encountered by the contractor and should not be considered as an “unforeseen condition” at the time of construction.

Greater than typical efforts should be anticipated by the contractor, as well as greater than typical equipment wear, to excavate, handle, and process these 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 Eagle County, and is not assuming responsibility for construction site safety or the contractor's activities.

Some surface sloughing may occur on the slope faces at these angles. Should site constraints prohibit the use of sloped excavation walls, 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.

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

Groundwater was encountered in Test Hole 1 at a depth of about 10 feet below existing road grade at the time of drilling. The test holes were backfilled upon drilling completion per Code of Colorado Regulations (2 CCR 402-2). Specifically, it has been our experience that surface and groundwater levels fluctuate greatly in mountainous areas, primarily due to seasonal conditions such as spring runoff. This site is considered to be comparatively more vulnerable to fluctuating groundwater levels due to its proximity to the East Lake Creek. These conditions are often highly variable and difficult to predict. Although these conditions generally exist for 1 to 3 months annually, their impact on design and construction can be significant. In the project area, it is common during construction to encounter dry conditions in the fall and wet conditions in the spring with relative groundwater fluctuations of 10 feet or more. This is particularly critical for foundation and deep utility excavations, cut slopes, culvert sizing, and for development adjacent to intermittently dry streams, rivers, and wetlands.

It is possible that groundwater may be encountered in project excavations at depths both shallower and deeper than those indicated above. The contractor should be prepared to dewater the excavation during construction. Pumps adequate to discharge water and/or well points to draw down the water level may be appropriate methods. Other methods may also be necessary. The dewatering approach should ultimately be determined by the contractor based on their means and methods experience. Dewatering operations may be necessary as both temporary and long-term/permanent installations. Wet and unstable subgrade likely will be encountered after de-watering. If seepage or groundwater is encountered during excavation or at any time during construction, the geotechnical engineer and project team should be contacted to evaluate the conditions. The presence of groundwater in these types of situations and associated potential design changes can have an impact to both the financial and schedule components of a project.

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.

Special measures to the divert or otherwise control surface waters carried by East Lake Creek should be anticipated. Environmental considerations regarding the discharge or handling of the waters likely will apply. An environmental consultant should be contacted for additional information.

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 earthworks 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 Eagle County 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 Eagle County as it pertains to design and construction of the proposed bridge 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 fall 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 additional buildings/structures, changes to improvement locations, dimensions, structural loading, site improvements, grades, etc., and are incorporated into this project, either after the original information was provided to us or after the date of this report, GROUND must be notified to re-evaluate the conclusions and parameters presented herein.**

The geotechnical conclusions in this report relied upon subsurface exploration at a single exploration point, 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 Eagle County. 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 Eagle County or other members of the design team, should contact the author or a GROUND principal immediately. We will be available to meet to discuss the risks and remedial approaches presented in this report, as well as other potential approaches, upon request.

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

GROUND appreciates the opportunity to complete this portion of the project and welcomes the opportunity to provide Eagle County with a proposal for construction observation and materials testing.

Sincerely,

GROUND Engineering Consultants, Inc.

Greg McCudden, P.G., E.I.

COLORADO LICENSED
BRIAN H. RECK
48265
PROFESSIONAL ENGINEER

12/07/2023

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

GOOGLE EARTH AERIAL IMAGE (09/13/2019)

2

Indicates test hole numbers and approximate locations.

NOT TO SCALE

GROUND ENGINEERING

JOB NO.: 23-6012

FIGURE: 1

LOCATION OF TEST HOLES

GROUND ENGINEERING

LOGS OF THE TEST HOLES

PROJECT: West Lake Creek Road - Bridge

JOB NO: 23-6012

CLIENT: Eagle County

SITE LOCATION: Eagle County, CO

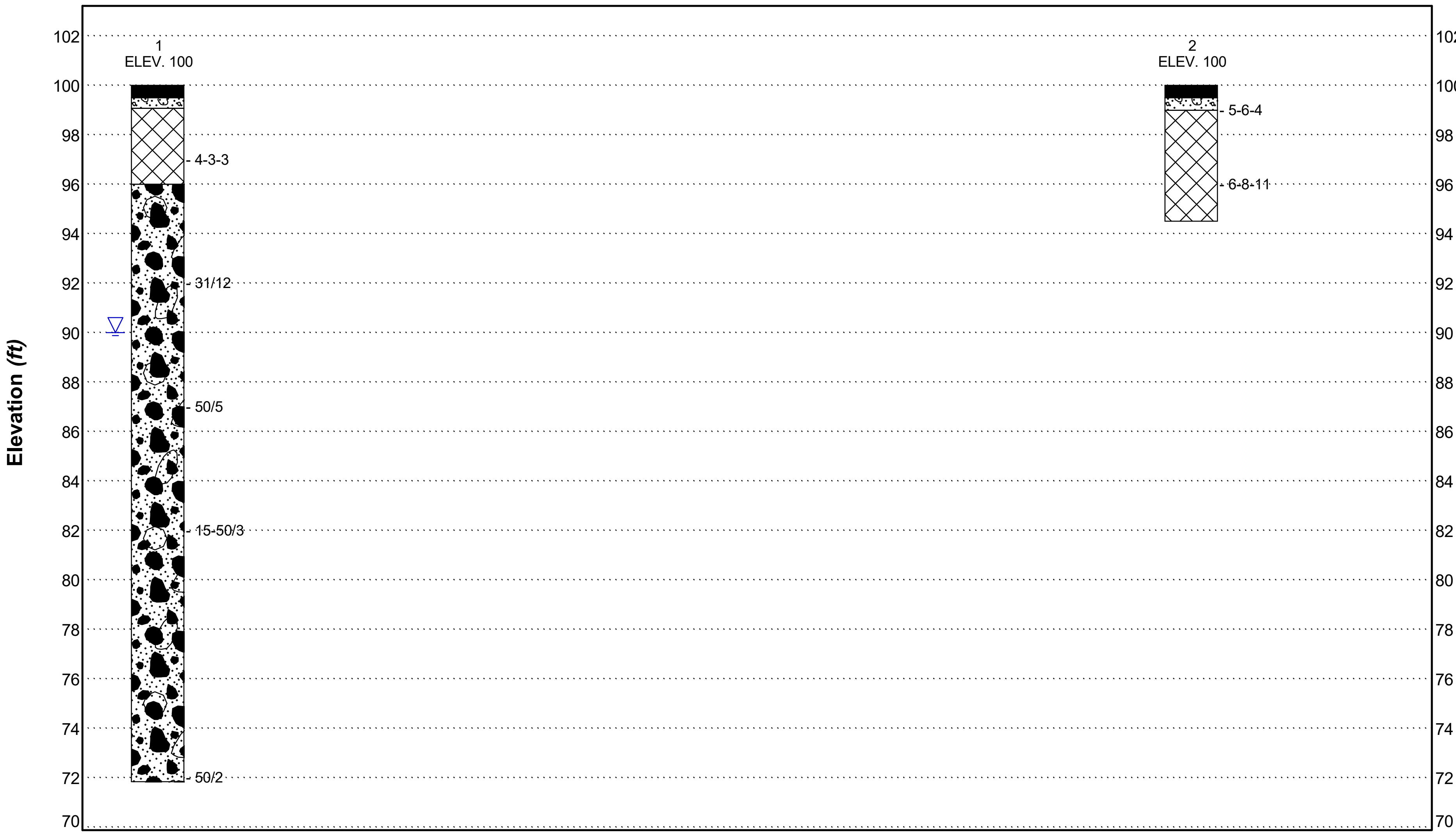


FIGURE: 2

GROUND ENGINEERING

LEGEND AND NOTES

PROJECT: West Lake Creek Road - Bridge **JOB NO:** 23-6012

CLIENT: Eagle County **SITE LOCATION:** Eagle County, CO

MATERIAL SYMBOLS

ASPHALT

ROAD BASE

FILL

SANDS, GRAVELS, and BOULDERS

NOTE: See Detailed Logs for Material descriptions.

SAMPLER SYMBOLS

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

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

NOTES

1. Test holes were drilled on 11/08/2023 with ODEX and 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

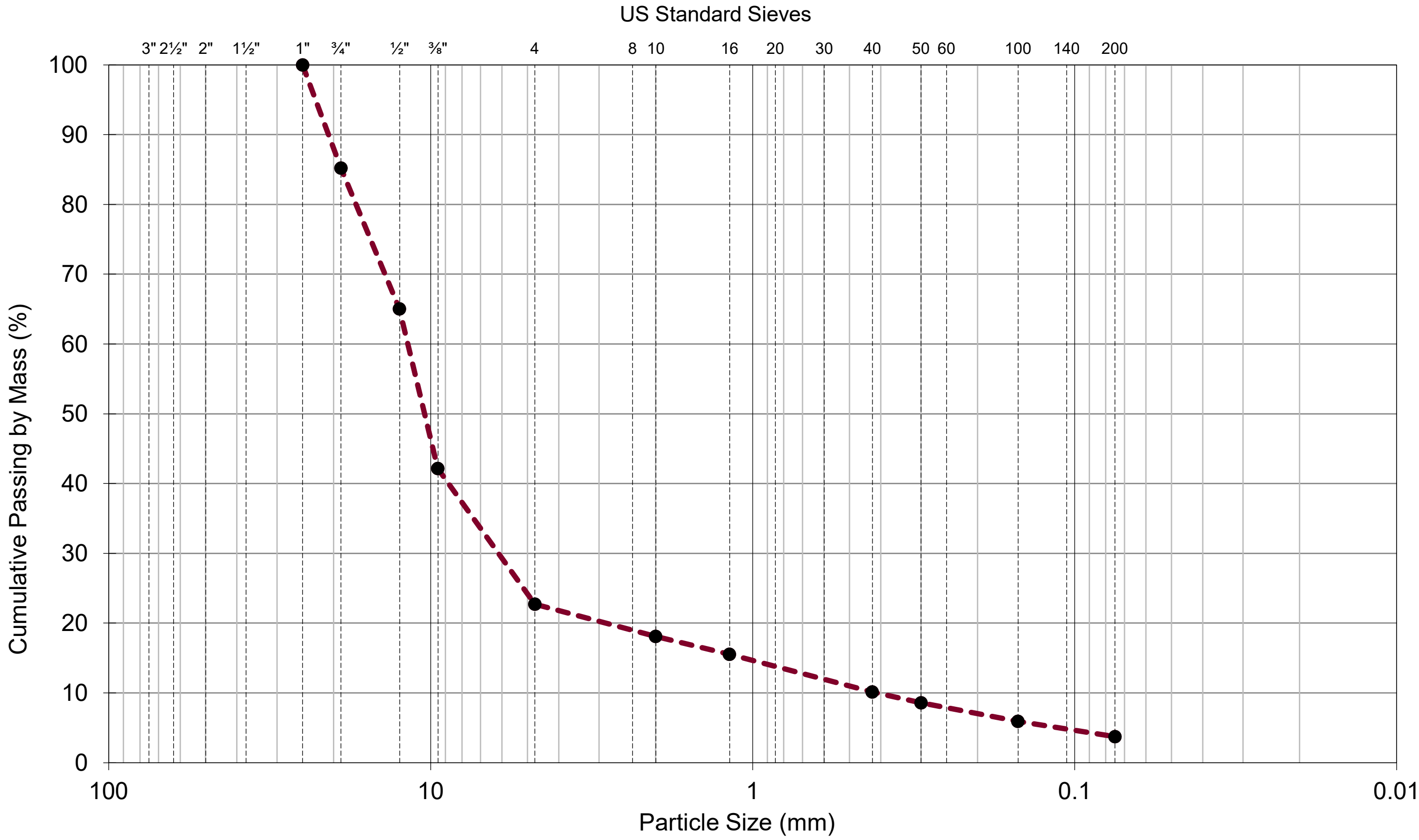
FIGURE: 3

ENGINEERING

Project No.: 23-6012

West Lake Creek Road - Bridge

Gradation (ASTM D422-63[2007])



Coarse Gradation			Fine Gradation			Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	23	D90	20.768
5 in	125	-	No. 8	2.36	-	D85	18.921
4 in	100	-	No. 10	2.00	18	D80	17.055
3 in	75	-	No. 16	1.18	16	D60	11.767
2.5 in	63	-	No. 20	0.85	-	D50	10.438
2 in	50	-	No. 30	0.60	-	D40	8.800
1.5 in	37.5	-	No. 40	0.425	10	D30	6.162
1 in	25.0	100	No. 50	0.300	9	D15	1.069
3/4 in	19.0	85	No. 60	0.250	-	D10	0.411
1/2 in	12.5	65	No. 100	0.150	6	D05	0.112
3/8 in	9.5	42	No. 140	0.106	-	Cu	28.639
No. 4	4.75	23	No. 200	0.075	3.7	Cc	7.852

Location: 1 at 8 feet
Description: GRAVEL with Sand

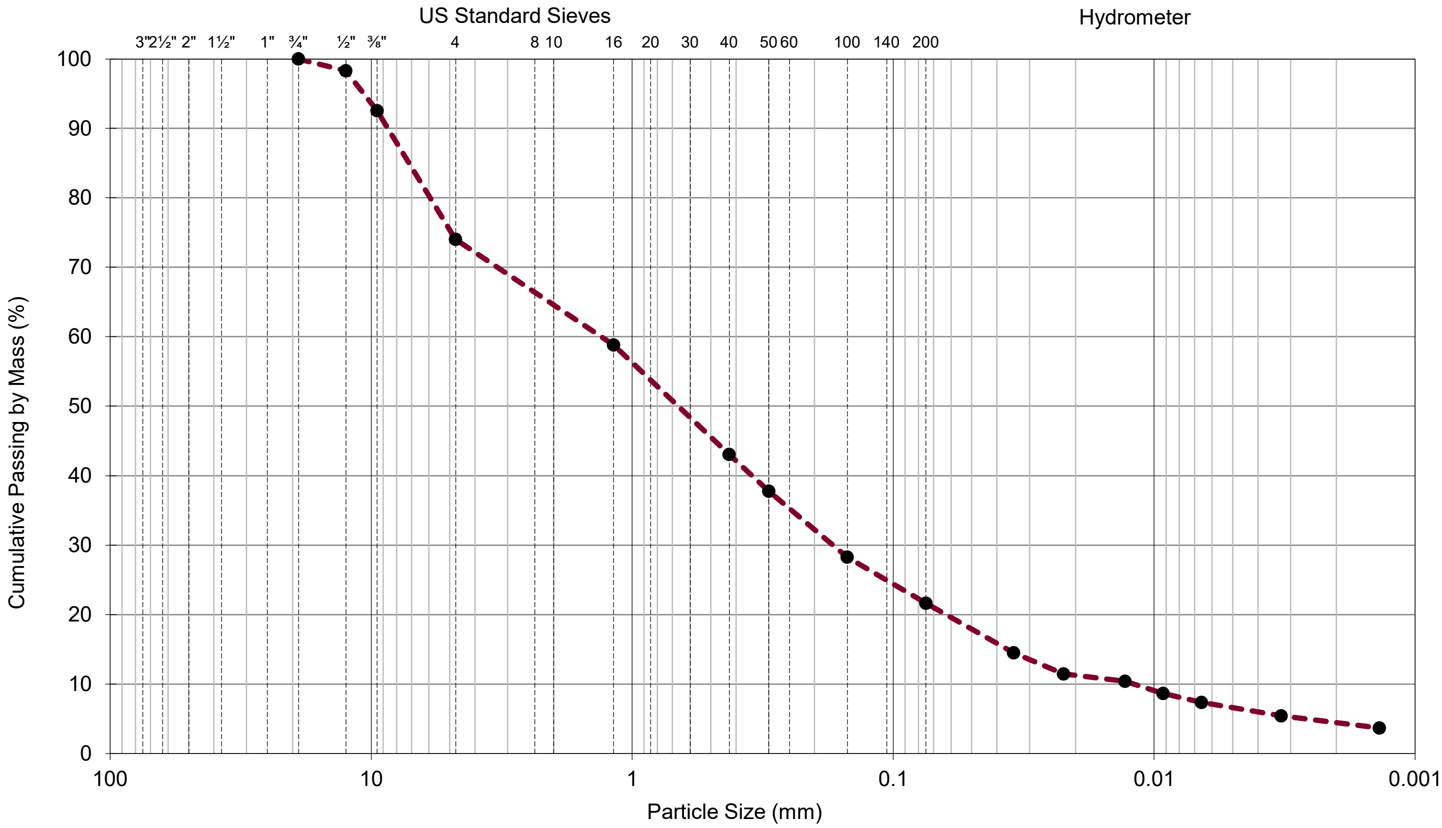
Classification: (GP)s / A-1-a (0)
Liquid Limit: NV
Plasticity Index: NP

Gravel (%): 77
Sand (%): 19
Silt/Clay (%): 4

Project No.: 23-6012

West Lake Creek Road - Bridge

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



Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	74	0.035	15	D90	8.638
5 in	125	-	No. 8	2.36	-	0.022	11	D85	7.162
4 in	100	-	No. 10	2.00	-	0.013	10	D80	5.938
3 in	75	-	No. 16	1.18	59	0.009	9	D60	1.317
2.5 in	63	-	No. 20	0.85	-	0.007	7	D50	0.666
2 in	50	-	No. 30	0.60	-	0.003	5	D40	0.347
1.5 in	37.5	-	No. 40	0.425	43	0.001	4	D30	0.170
1 in	25.0	-	No. 50	0.300	38	-	-	D15	-
3/4 in	19.0	100	No. 60	0.250	-	-	-	D10	-
1/2 in	12.5	98	No. 100	0.150	28	-	-	D05	-
3/8 in	9.5	93	No. 140	0.106	-	-	-	Cu	-
No. 4	4.75	74	No. 200	0.075	21.6	-	-	Cc	-

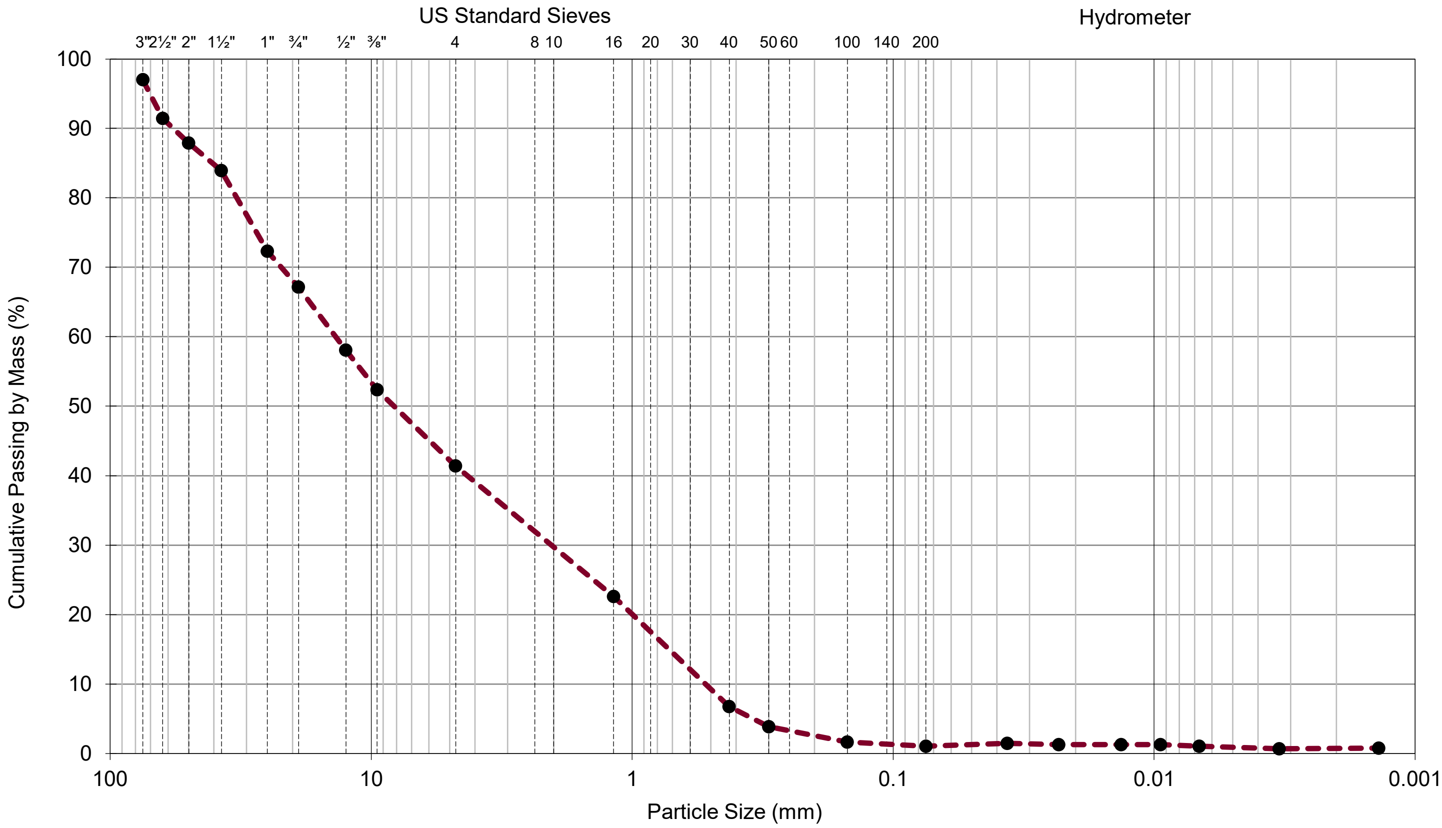
Location: Composite - Test Holes
Description: Silty SAND with Gravel

Classification: (SM)g / A-1-b (0)
Liquid Limit: 20
Plasticity Index: 3
Activity: 0.7

Gravel (%): 26
Sand (%): 52
Silt/Clay (%): 22
< .002 mm (%): 4

West Lake Creek Road - Bridge

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



Coarse Gradation			Fine Gradation			Hydrometer		Grading	
US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	US Standard Sieve	Particle Size (mm)	Passing by Mass (%)	Particle Size (mm)	Passing by Mass (%)	Coefficient	Value
6 in	150	-	No. 4	4.75	41	0.037	1	D90	57.367
5 in	125	-	No. 8	2.36	-	0.023	1	D85	40.540
4 in	100	-	No. 10	2.00	-	0.013	1	D80	32.698
3 in	75	97	No. 16	1.18	23	0.009	1	D60	13.645
2.5 in	63	91	No. 20	0.85	-	0.007	1	D50	8.177
2 in	50	88	No. 30	0.60	-	0.003	1	D40	4.278
1.5 in	37.5	84	No. 40	0.425	7	0.001	1	D30	2.040
1 in	25.0	72	No. 50	0.300	4	-	-	D15	0.722
3/4 in	19.0	67	No. 60	0.250	-	-	-	D10	0.523
1/2 in	12.5	58	No. 100	0.150	2	-	-	D05	0.344
3/8 in	9.5	52	No. 140	0.106	-	-	-	Cu	26.087
No. 4	4.75	41	No. 200	0.075	1.0	-	-	Cc	0.583

Location: Composite - Channel
Description: GRAVEL with Sand

Classification: (GP)s / A-1-a (0)
Liquid Limit: NV
Plasticity Index: NP
Activity: -

Gravel (%): 59
Sand (%): 40
Silt/Clay (%): 1
< .002 mm (%): 1

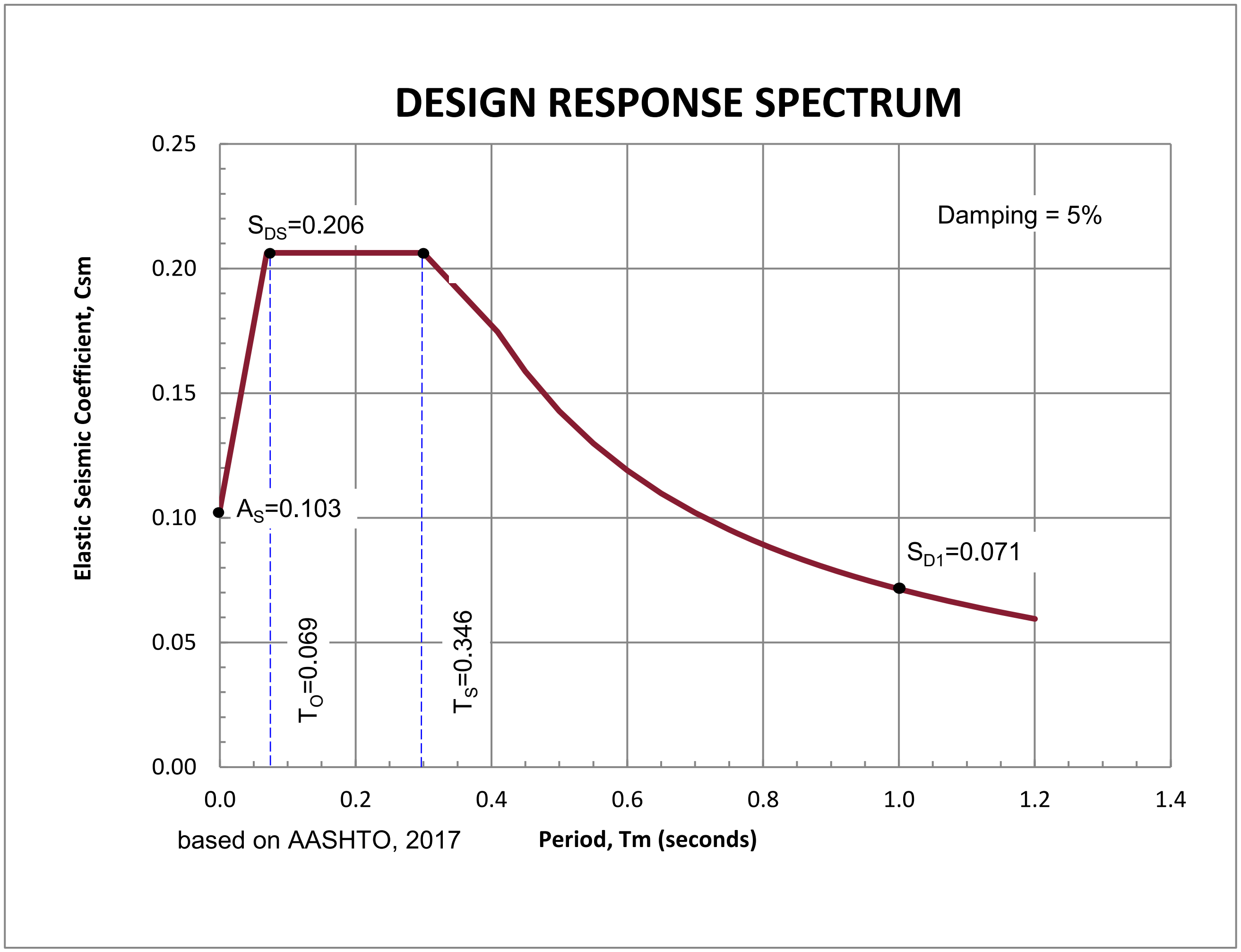

DESIGN RESPONSE SPECTRUM
Elastic Seismic Coefficient, Csm
0.25
0.20
0.15
0.10
0.05
0.00
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
S_{DS}=0.206
A_S=0.103
T_O=0.069
T_S=0.346
S_{D1}=0.071
Damping = 5%
based on AASHTO, 2017
Period, Tm (seconds)

Figure 7

West Lake Creek Road - Bridge

TABLE 1: SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural Moisture Content	Natural Dry Density	Gradation			Atterberg Limits		USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth *(feet)*	*(%)*	*(pcf)*	Gravel *(%)*	Sand *(%)*	Fines *(%)*	Liquid Limit	Plasticity Index			
1	3	10.3	*SD*	45	34	21.0	26	8	(GC)s	A-2-4 (0)	FILL: Clayey Gravel with Sand
1	8	6.3	122.4	77	19	3.7	NV	NP	(GP)s	A-1-a (0)	GRAVEL with Sand
2	1	6.1	*SD*	48	33	18.6	23	5	(GC-GM)s	A-2-4 (0)	FILL: Gravel with Clay, Silt, and Sand
Composite - Test Holes		-	-	26	52	21.6	20	3	(SM)g	A-1-b (0)	Silty SAND with Gravel
Composite - Channel		-	-	59	40	1.0	NV	NP	(GP)s	A-1-a (0)	GRAVEL with Sand

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

Job No. 23-6012

West Lake Creek Road - Bridge

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)*			
Composite - Test Holes		0.05	8.5	- 78	Positive	2,206	(SM)g	A-1-b (0)	Silty SAND with Gravel

Job No. 23-6012

Appendix A

Detailed Logs of the Test Holes

TEST HOLE 1

GROUND ENGINEERING

PROJECT: West Lake Creek Road - Bridge **JOB NO:** 23-6012

CLIENT: Eagle County **SITE LOCATION:** Eagle County, CO

Elevation (ft): 100 at Depth (ft): 0. Scale marks: 95 / 5, 90 / 10, 85 / 15, 80 / 20, 75 / 25.

Material Descriptions and Drilling Notes

ASPHALT: Approximately 6 inches of asphalt.

ROAD BASE: Approximately 5 inches of aggregate base coarse.

FILL: Fine to coarse, clayey to silty sands and gravels with boulders. They were slightly plastic, moist, and dark-brown in color. Organic material was commonly observed.

SANDS, GRAVELS, and BOULDERS: Fine to coarse, relatively clean sands, gravels, and boulders. They were non- plastic, moist to wet, dense to very dense, and brown to gray in color.

Large boulder from 5 to 7 feet

Moderate sized boulders were common below 8 feet

Groundwater encountered at 10 feet at the time of drilling.

Large boulder from 24 to 27 feet

Bottom of test hole at approx. 28.17 feet.

Sample Data

Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gravel %	Sand %	Fines %	Liquid Limit	Plasticity Index	USCS
4-3-3	10.3		45	34	21	26	8	(GC)s
31/12	6.3	122.4	77	19	4	NV	NP	(GP)s
50/5								
15-50/3								
50/2								

TEST HOLE 2

PROJECT: West Lake Creek Road - Bridge **JOB NO:** 23-6012

CLIENT: Eagle County **SITE LOCATION:** Eagle County, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation: Gravel %	Gradation: Sand %	Gradation: Fines %	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	USCS
100	0												
			ASPHALT: Approximately 6 inches of asphalt.										
			ROAD BASE: Approximately 6 inches of aggregate base coarse.	X	5-6-4	6.1		48	33	19	23	5	(GC-GM)s
			FILL: Fine to coarse, clayey to silty sands and gravels with boulders. They were slightly plastic, moist, and dark-brown in color. Organic material was commonly observed.										
95	5			X	6-8-11								

Refusal at approx. 5.5 feet.