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Revised Geotechnical Evaluation Report

Proposed Calhan Highway Bridge Replacement

South of Paint Mine Road

El Paso County, Colorado

VIVID Project No.: D20-2-333

VIVID Engineering Group

June 2, 2021

Report prepared for:

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1.0 INTRODUCTION

1.1 GENERAL

This revised report presents the results of a geotechnical investigation performed for the proposed Calhan Highway Bridge Replacement project, located on Calhan Highway approximately 1 mile south of Paint Mine Road in El Paso County, Colorado. An attached Vicinity Map (Figure 1) shows the general location of the project. Our investigation was performed for AECOM and was authorized by Mr. Craig Parent, PE.

This report includes our recommendations relating to the geotechnical aspects of project design and construction. The conclusions and recommendations stated in this report are based upon the subsurface conditions found at the locations of our exploratory borings at the time our exploration was performed. They also are subject to the provisions stated in the report section titled **Additional Services & Limitations**. Our findings, conclusions, and recommendations should not be extrapolated to other areas or used for other projects without our prior review. Furthermore, they should not be used if the site has been altered, or if a prolonged period has elapsed since the date of the report, without VIVID's prior review to determine if they remain valid.

1.2 PROJECT DESCRIPTION

The project includes removal and replacement of the existing two-lane bridge located on Calhan Highway approximately one mile south of Paint Mine Road in northeastern El Paso County, Colorado. VIVID evaluated the subsurface conditions by drilling two deeper borings in the area of the bridge replacement and two shallower borings (one on each pavement approach) in order to develop geotechnical and pavement section design recommendations for the bridge and pavement approaches.

The new bridge will be designed to withstand heavier loads intended to allow larger vehicles such as emergency vehicles and commercial trucks to cross the bridge legally. Along with the bridge reconstruction, improvements are to include drainage, transitions, shoulders and guardrails.

AECOM is considering several new bridge options, including the following:

- a single-span girder bridge on deep foundations with cantilevered wingwalls from the abutments;
- a single-span girder bridge on GRS-IBS foundations (pending AECOM scour analysis for feasibility and a final settlement analysis to be performed by VIVID);
- pre-cast arch(s) on either spread footings or deep foundations with either pre-cast or CIP wingwalls; and
- a multi-cell box culvert (however we understand the box culvert will not be the preferred alternative per direction from El Paso County).

Anticipated loading conditions and movement tolerances for the bridge structure were not provided at the time this report was published. Based on preliminary cross sections of the roadway alignment and for estimating purposes, we have assumed that, in general, planned cuts and fills to achieve finish site grades within the bridge area will be on the order of 1 to 3 feet or less, with isolated areas of embankment fill on the order of up to 9 feet in the area of the proposed bridge abutments. VIVID should be notified in order to review and revise our recommendations if the construction varies from that presented above.

1.3 PURPOSE AND SCOPE

The purpose of our investigation was to explore and evaluate subsurface conditions at various locations near the proposed project improvements and, based upon the conditions found, to develop recommendations relating to the geotechnical aspects of project design and construction. Our conclusions and recommendations in this report are based upon analysis of the data from our field exploration, laboratory tests, and our experience with similar soil and geologic conditions in the area.

VIVID's scope of services included:

- A visual reconnaissance to observe surface and geologic conditions at the project site and locating the exploratory borings;
- Notification of the Utility Notification Center of Colorado (UNCC)/Colorado 811 to identify underground utility lines at the boring locations prior to our drilling;
- The drilling of two exploratory borings near the existing bridge abutments and two exploratory borings within pavement approach locations, all of which were selected based upon the proposed construction plans, access, and the locations of existing utilities;
- Laboratory testing of selected samples obtained during the field exploration to evaluate relevant physical and engineering properties of the soils and bedrock;
- Evaluation and engineering analysis of the field and laboratory data collected to develop our geotechnical conclusions and recommendations; and
- Preparation of this report, which includes a description of the proposed project, a description of the surface and subsurface site conditions found during our investigation, our conclusions and recommendations as to bridge foundation design, pavement section thickness design, other related geotechnical issues, and appendices which summarize our field and laboratory investigations.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

The field exploration was performed on October 5, 2020 and included drilling four exploratory borings at the approximate locations indicated on the attached Exploration Location Plan shown on Figure 2. A summary of the explorations is presented below:

Table 1
Summary of Subsurface Exploration[1]

Boring Designation	Boring Depth [feet, below ground surface]	Depth to Groundwater [feet, below ground surface]	Depth to Weathered Bedrock [feet, below ground surface]	Depth to Competent Bedrock [feet, below ground surface]
B-1	40	None Encountered	19	25
B-2	39.5	25.5	28	35
P-1	10	None Encountered	None Encountered	None Encountered
P-2	10	None Encountered	None Encountered	None Encountered

1) All depths approximate and estimated from existing ground surface.

The borings were advanced with a truck-mounted CME-55 drill rig equipped with 4-inch outside diameter, continuous-flight auger. Samples were taken with a 2.5-inch O.D./2.0-inch I.D. California-type sampler, standard penetration (SPT) sampler, wireline rock coring methods, and by bulk methods. Penetration tests were obtained at the various sample depths as well.

Appendix A to this report includes logs describing the subsurface conditions. The lines defining boundaries between soil and rock types on the logs are based upon drill behavior and interpolation between samples and are therefore approximate. Transition between soil and rock types may be abrupt or may be gradual.

2.2 GEOTECHNICAL LABORATORY TESTING

Laboratory tests were performed on selected soil samples to estimate their relative engineering properties. Tests were performed in general accordance with the following methods of ASTM or other recognized standards-setting bodies, and local practice:

- Description and Identification of Soils (Visual-Manual Procedure)
- Classification of Soils for Engineering Purposes
- Moisture Content and Unit Weight
- Sieve Analysis of Fine and Coarse Aggregates
- Liquid Limit, Plastic Limit, and Plasticity Index
- Unconfined Compressive Strength
- Swell/Consolidation
- Hveem Stabilometer (R-Value)

Results of the geotechnical laboratory tests are included in Appendix B of this report. Selected test results are also shown on the boring logs in Appendix A.

2.3 ANALYTICAL LABORATORY TESTING

Analytical testing for soil corrosivity was performed on selected samples and included the following tests:

- pH
- Resistivity
- Redox Potential
- Water-soluble Sulfates
- Water-soluble Chlorides
- Sulfides

Results of the analytical laboratory tests are included in Appendix C of this report.

3.0 SITE CONDITIONS

3.1 SURFACE

At the time of our investigation, the area identified for the new bridge and pavement approaches was located on Calhan Highway approximately one mile south of Paint Mine Road. The area surrounding the site comprised predominantly vacant land with sparse residential properties.

Buried and overhead utility infrastructure was present along the shoulders of the alignment. An ephemeral drainage was present running in a general southeast to northwest trend, under the existing bridge. Surface water was not observed in the drainage at the time of our investigation.

No obvious areas of groundwater seepage were observed on the site. Numerous areas to the east and west of Calhan Highway appeared to have exposures of sandstone and claystone bedrock material.

3.2 GEOLOGY

Prior to drilling, the site geology was evaluated by reviewing available geologic maps including the USGS geologic map of Colorado (Tweto, 1979). Mapping indicates the surficial soils in the general area of the project site comprise predominantly of alluvium deposits of sand, gravel and clay underlain by claystone and sandstone bedrock of the Dawson Formation. The mapping is generally consistent with the materials encountered in our explorations. However, existing fill was encountered below the asphalt.

3.3 SEISMICITY

Based upon the geologic setting, subsurface soil conditions, and low seismic activity in this region, liquefaction is not expected to be a hazard at the site. Based on correlation of blow count data (N-values) from the borings advanced near the bridge during this evaluation, the subsurface bedrock profiles correspond with 2009 AASHTO Site Class C. The intermediate design acceleration values are presented below. The peak ground acceleration (PGA) for this site is 0.046 g.

Table 2
Design Acceleration for Short Periods

S_S	F_a
0.102	1.2

S_S = The mapped spectral accelerations for short periods (U.S. Geological Survey Web Page, 2020)

F_a = Site coefficient from Table 3.4.2.3-1, 2009 AASHTO

Table 3
Design Acceleration for 1-Second Period

S_1	F_V
0.031	1.7

S_1 = The mapped spectral accelerations for 1-second period (U.S. Geological Survey Web Page, 2020)

F_v = Site coefficient from Table 3.4.2.3-2, 2009 AASHTO

3.4 SUBSURFACE

VIVID explored the subsurface conditions by drilling, logging, and sampling four exploratory borings as near as possible to the general area to be occupied by the proposed bridge and new pavement areas as shown approximately on Figure 2. These borings were drilled to depths between approximately 10 and 40 feet below the existing ground surface. The general profile encountered in our borings consisted of:

Existing Asphalt and Granular Base Materials

Approximately 5 to 7 inches of existing asphalt was encountered at the ground surface in the borings advanced within the roadway alignment. No discernable granular base layer was encountered in any of the borings.

Existing Fill

Fill materials comprised predominantly of silty to clayey sand were encountered under the pavement section materials described above at all boring locations and extended to depths up to approximately 4.5 to 10 feet below the ground surface. The fill materials were predominantly brown to dark brown in color, moist, and field penetration testing indicated the relative density of the soils was loose to medium dense based on blow counts (N-value). Two samples of clayey sand fill soil were found to exhibit low swell potential (0.8 to 1.0 percent) when wetted under a 200 pounds per square foot (psf) surcharge load.

Native Sand and Clay

Native soils comprised predominantly of silty sand to well graded sand with silt were encountered under the fill materials described above in all borings and extended to depths between approximately 10 to 28 feet below the ground surface. Thin layers of sandy clay were encountered between depths of approximately 24.5 and 28 feet in boring B-2 and between 4.5 feet and 8 feet in boring P-1. The native sand soils were light brown to brown in color, moist, and field penetration testing indicated the relative density of the sand soils was medium dense based on blow counts (N-value). The sandy clay was gray to brown, moist, and field penetration testing indicated the relative density of the clay soils was stiff based on blow counts (N-value).

Claystone and Sandstone

Weathered to comparatively unweathered sandstone and claystone of the Dawson Formation was encountered at the boring locations and approximate depths shown in Table 1, above and extended to the maximum depths explored of approximately 40 and 39.5 feet below the ground surface in borings B-1 and B-2, respectively. The bedrock materials were predominantly comprised of claystone that was light gray to dark gray to brown, moist, and slightly to highly weathered. Layers of sandstone were present interbedded within the claystone. The sandstone was light gray to gray with iron oxide staining, and slightly weathered. Field penetration testing indicated the relative density of the bedrock materials was moderately hard to very hard based on blow counts (N-value). Three samples of claystone materials were found to exhibit low to moderate swell potential (0.4 to 3.1 percent) when wetted under a 1,000 pounds per square foot (psf) surcharge load.

The boring logs in Appendix A should be reviewed for more detailed descriptions of the subsurface conditions at each of the boring locations explored.

3.4.1 Groundwater

Groundwater was encountered at the boring location and approximate depth presented in Table 1, above, at the time of drilling. Due to the location of the proposed structure and roadway alignment adjacent to an existing drainage channel, groundwater will be a consideration for construction of bridge foundation elements. Groundwater levels commonly vary over time and space depending on seasonal precipitation, irrigation practices, land use, and runoff conditions. These conditions and the variations that they create often are not apparent at the time of field investigation. Accordingly, the soil moisture and groundwater data in this report pertain only to the locations and times at which exploration was performed. They can be extrapolated to other locations and times only with caution. It should also be noted that VIVID has not performed a hydrologic study to verify the seasonal high-water level.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GEOTECHNICAL FEASIBILITY OF PROPOSED CONSTRUCTION

VIVID found no subsurface conditions during this investigation that would preclude construction of the improvements essentially as planned, provided the recommendations in this report are incorporated into the design and construction of the project.

Deep Foundation Systems

Geotechnical design parameters for straight-shaft, drilled caissons and driven steel piles for the proposed bridge structure are presented in Section 4.3. Geotechnical design recommendations for deep foundation systems are in general accordance with AASHTO LRFD Bridge Design Specifications, 8th Edition. The existence of relatively clean granular soils encountered in both borings as well as groundwater encountered during drilling within the bedrock in boring B-2 will likely result in caving of caisson excavations. Therefore all, or a combination of, casing and mud slurry will likely be required to successfully advance the caissons through the zones where free groundwater or clean sand zones exist.

Shallow Foundation System

Geotechnical design parameters for shallow foundations are presented in Section 4.4. To create more uniform and stable subgrade conditions and facilitate construction in wet conditions for new foundations, we recommend a 24-inch zone of structural fill to be placed beneath the foundations of the replacement structure and any associated wall shallow foundation elements. The over-excavation and fill placement process is described in more detail below.

The potential presence of surface water and presence of groundwater creates loose/soft and very moist to wet soil conditions as found in our borings. Therefore, construction dewatering, shoring, and difficult construction conditions should be expected on this site. Recommendations for construction dewatering, subgrade stabilization, and shoring are provided below.

Pavement Design

Pavement section thickness design and construction recommendations are presented in Section 4.7.

Further detail regarding our geotechnical design and construction recommendations for site preparation, pavements, bridge and wall foundations, and other related construction topics are provided in the following sections.

4.2 CONSTRUCTION CONSIDERATIONS

4.2.1 General

All site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations and other local, State or Federal guidelines.

4.2.2 Subgrade Preparation for General Site Grading

Initial site work should consist of completely removing all existing structures, organic material and other deleterious materials from all areas to be filled and areas to be cut. All material should be removed for offsite disposal in accordance with local laws and regulations or, if appropriate, stockpiled in proposed landscaped areas for future use. Areas to receive fill should be evaluated by the geotechnical engineer prior to the placement of any fill materials.

After performing the required excavations and prior to the placement of compacted fill, processing of subgrade soils should be performed. This should include scarifying the subgrade to a depth of at least 6 inches, moisture conditioning, and compacting as recommended in Section 4.2.5 of this report. Where unstable conditions exist and proper moisture conditioning and compaction is not feasible, stabilization of the subgrade as described in Section 4.4 will be required. All fill materials should be placed on a horizontal plane and placed in loose lifts not to exceed 8 inches in thickness, unless otherwise accepted by the geotechnical engineer. Additional subgrade preparation and stabilization for specific project elements such as retaining wall foundations and pavement subgrade are provided in the sections of this report that specifically address these elements.

4.2.3 Excavation Characteristics

Proposed site grading plans were not provided to us prior to compilation of this report. We anticipate cuts and fills for general site grading may be on the order of about 1 to 3 feet or less, with deeper excavations necessary for bridge foundations if drilled caissons are utilized. In addition, areas of embankment fill on the order of up to 9 feet are anticipated in the area of the proposed bridge approaches. Boring logs should be reviewed to evaluate material type and groundwater conditions.

Soil Materials

We believe that excavation of the on-site soils can be readily accomplished using standard-duty excavating equipment. Due to the potential groundwater conditions as well as clean sand zones, caisson excavations will likely require casing and potentially mud slurry to advance caisson excavations.

Bedrock

Excavation into the bedrock, where required for deep foundation elements, could encounter soft to moderately hard conditions for more weathered bedrock zones to extremely difficult conditions for unweathered/formational sandstone and claystone bedrock zones. Relatively soft conditions could be encountered during excavation in the upper portions of weathered bedrock material.

A caisson drilling rig for excavation of the drilled caissons of sufficient size to penetrate the hard to very hard formational bedrock the required amount should be mobilized. We anticipate excavation in the harder materials could be relatively slow depending on the depth of excavation, the type of bedrock encountered, the type and site of equipment used, as well as the contractor's experience with similar excavation.

Dewatering/Shoring

Portions of the proposed earthwork and excavation operations for this project will be within or adjacent the existing drainage channel. **The existence and elevation of surface water flows and groundwater is highly dependent on time of year and runoff conditions. The groundwater elevation can vary significantly in this area.** Therefore, surface and possibly groundwater infiltration will occur below creek elevations during construction operations, requiring construction dewatering. Groundwater was also encountered at the boring location and approximate depth presented in Table 1 of this report. Other similar areas may exist along the alignment. Utilizing appropriate construction dewatering equipment/systems such as well points or sumps, and trenches, will be the responsibility of the contractor. In addition, trenching into unstable, saturated overburden soils will require temporary shoring, where construction of safe slopes is not feasible. OSHA requirements for excavation in unstable materials should be followed.

All excavations must comply with applicable local, State and Federal safety regulations, and particularly with the excavation standards of the Occupational Safety and Health Administration (OSHA). Construction site safety, including excavation safety, is the sole responsibility of the Contractor as part of its overall responsibility for the means, methods and sequencing of construction operations. VIVID's recommendations for excavation support are intended for the Client's use in planning the project, and in no way relieve the Contractor of its responsibility to construct, support and maintain safe slopes. Under no circumstances should the following recommendations be interpreted to mean that VIVID is assuming responsibility for either construction site safety or the Contractor's activities.

We believe that the <u>unsaturated</u> soils on this site will classify as Type C materials using OSHA criteria. OSHA requires that unsupported cuts in Type C materials be laid back to ratios no steeper than 1½:1 (horizontal to vertical). However, the hard and intact on-site claystone/sandstone may be classified as Type B material. OSHA requires that unsupported cuts up to 20 feet in height be laid back to ratios no steeper than 1H:1V (horizontal to vertical) for a Type B material. In general, we believe that these slope ratios will be temporarily stable under <u>unsaturated</u> conditions. Where groundwater occurs, flatter slopes will be required. Please note that the actual determination of soil type and allowable sloping must be made in the field by an OSHA-qualified "competent person."

<u>**Where new embankments tie into existing embankments, we recommend that new fill be benched.**</u> Benching of fills into the existing slopes is critical to the stability of new fills. Benching of the fill into the existing slopes as the fill is brought up in layers should be performed. The height and width of the benching is dependent on existing slope geometry, size of equipment utilized, and safety requirements. As a general rule, each bench should be cut 5-feet horizontally into the original ground line from the vertical side of the previous cut. We believe this will be possible where the existing slope is 1H:1V (horizontal:vertical) or shallower. Benching can be performed from bottom to top of slope as new layers of fill are placed. In areas where the existing slope is near vertical, the bucket of a track-hoe (or similar equipment) should be used to create "mini"- benches into the slope walls, if possible. Adjustments or other approaches will be required if slopes are unstable.

Although erosion analysis is beyond the scope of our analysis, it is generally recommended that embankment slopes be armored and/or well vegetated (with appropriate grass cover) to assist in reducing the influence of water that may flow over the face of the embankment, regardless of embankment material type. Water should be channeled away from the slope face to reduce the possibility of erosion due to water flow.

4.2.4 Fill Materials

Specific recommendations in regard to type of fill materials are presented in the following sections of this report for foundations, pavements, embankments, etc.

Foundation Fill Beneath Structural Elements

Imported structural fill for foundations (if required) at this site beneath structural elements such as foundation walls or other minor structures shall consist of a non-expansive, granular material meeting the specifications for CDOT Class I Structure Backfill.

Wall Drainage Zones

A 12-inch-wide zone of clean, "crushed", angular aggregate (No. 57 stone) should be placed adjacent abutment, box culvert, and wing walls to act as a wall drainage layer to facilitate groundwater movement around structures and limit build-up of hydrostatic pressures. The top of the drainage material behind the walls should not extend any closer than within 3 feet of the proposed ground surface. Use of a filter fabric will be required to prevent the fine site soils from clogging the No. 57 stone drainage layer against the walls. Alternatives such as a drainage board may be acceptable as well.

Wall Backfill

Fill placed adjacent and within the lateral earth pressure zone of influence to bridge abutments, box culvert walls or wingwalls must meet CDOT Class I Structure Backfill specifications. Due to the poor characteristics of the on-site clayey soils, on-site soils should not be used as structure backfill.

Fill materials should be compacted according to the recommendations in Section 4.2.5 of this report. We recommend that a qualified representative of VIVID visit the site during excavation and during placement of the fill to verify the soils exposed in the excavations are consistent with those encountered during our subsurface exploration and that proper foundation subgrade preparation and placement is performed.

Roadway/Embankment Fill

1. **Pavement Subgrade**: Defined as the subgrade soil within the 2-foot zone below the bottom of the pavement Aggregate Base Course layer. This material must meet the minimum R-value used for pavement design as shown in Table 11, in Section 4.7 of this report.
2. **Roadway Embankment**: Defined as roadway fill material located below the 2-foot Pavement Subgrade layer discussed above. On-site soils may be used as roadway embankment fill.

A sample of any imported fill material should be submitted to our office for approval and testing at least 1 week prior to stockpiling at the site. Fill materials should be compacted according to the recommendations in Section 4.2.5 of this report. We recommend that a qualified representative of VIVID visit the site during excavation and during placement of the structural fill to verify the soils exposed in the excavations are consistent with those encountered during our subsurface exploration and that proper foundation subgrade preparation and placement is performed.

4.2.5 Fill Placement and Compaction

Fill materials should be placed in horizontal lifts compatible with the type of compaction equipment being used, moisture conditioned, and compacted in accordance with the following criteria:

Table 4
Fill Placement and Compaction Criteria

Fill Location	Material Type	Percent Compaction[1] (ASTM D1557)	Moisture Content
Subgrade Preparation (after clearing, grubbing, excavation, and prior to placement of new fill and/or structural elements)	On-site Soils	92 minimum	$\pm$ 2 % of optimum
Foundation Subgrade (Abutment, Box Culvert, Wing Wall Structures)	CDOT Class I Structure Backfill (See Sections 4.2.4 and 4.4)	95 minimum	$\pm$ 2 % of optimum
12-inch Wall Drainage Zone (Behind Box Culvert Walls, Abutments and Wing Walls)	Clean, "Crushed" Aggregate (No. 57 Stone) (See Sections 4.2.4 and 4.4)	N/A[2]	N/A
Retaining Wall Backfill (Bridge Abutments, Box Culverts and Wing Walls)	CDOT Class I Structure Backfill (See Sections 4.2.4 and 4.4)	95 minimum	$\pm$ 2 % of optimum
Existing Pavement Subgrade	On-site soils	95 minimum	$\pm$ 2 % of optimum
Embankment Fill	On-site soils or Granular Imported Fill (See Section 4.2.4)	95 minimum	$\pm$ 2 % of optimum
Aggregate Base Course	CDOT Class 5 or 6 Aggregate Base Course	95 minimum	$\pm$ 2 % of optimum
Utility Trench Backfill/ Exterior Flatwork Subgrade	On-site Soils/Imported Granular Structural Fill (See Section 4.2.4)	95 minimum	$\pm$ 2 % of optimum

1) In non-structural/landscaped areas, the compaction specification may be reduced to 90 percent. The higher compaction criteria should be utilized where two or more "fill locations" coincide.
2) No. 57 stone material should be placed in maximum 8-inch lifts with compaction, but no testing required. Approved drain board materials may be acceptable in lieu of aggregate drainage layers.

Fill should be placed in level lifts not exceeding 8 inches in loose thickness and compacted to the specified percent compaction to produce a firm and stable surface. If field density tests indicate the required percent compaction has not been obtained, the fill material should be reconditioned as necessary and re-compacted to the required percent compaction before placing any additional material.

Fill against any site or foundation walls should be properly placed and compacted as recommended herein. Backfill should be mechanically compacted in layers (6 to 8 inches maximum loose lift thickness). Care should be taken when placing backfill so as not to damage the walls. Compaction of each lift adjacent to and near the walls should be accomplished with hand-operated tampers or other lightweight compactors. Over-compaction may cause excessive lateral earth pressures, which could result in wall movement, and potentially damage the walls. If required, wall designs may need to consider increased lateral pressures during construction/compaction.

4.2.6 Settlement of Embankment Fill

We anticipate that fills on the order of up to 9 feet will be constructed for roadway embankments, with the deeper fills in the area around the proposed bridge abutments. Deeper fills, even when properly compacted, can be expected to settle beneath their own weight, and also cause settlement in the underlying soils.

It is generally prudent to monitor settlement of the fill through surveyed benchmarks, settlement plates, or similar means to ensure the majority of the settlement (primary settlement) has occurred prior to construction of project elements that may be damaged due to settlement such as pavements. We can provide recommendations for the settlement monitoring approach, if desired.

4.2.7 Utility Trench Backfill

Backfill material should be essentially free of plant matter, organic soil, debris, trash, other deleterious matter and rock particles larger than 4 inches. However, backfill material in the “pipe zone” (from the trench floor to 1 foot above the top of pipe) should not contain rock particles larger than 1 inch. Strictly observe any requirements specified by the utility agency for bedding and pipe-zone fill. In general, backfill above the pipe zone in utility trenches should be placed in lifts of 6 to 8 inches, and compacted using power equipment designed for trench work. Backfill in the pipe zone should be placed in lifts of 8 inches or less and compacted with hand-held equipment. Compact trench backfill as recommended in Section 4.2.5 of this report.

4.2.8 Construction in Wet or Cold Weather

During construction, grade the site such that surface water can drain readily away from the structure area. Promptly pump out or otherwise remove any water that may accumulate in excavations or on subgrade surfaces and allow these areas to dry before resuming construction. The use of berms, ditches and similar means may be used to prevent stormwater from entering the work area and to convey any water off site efficiently.

If earthwork is performed during the winter months when freezing is a factor, no grading fill, structural fill or other fill should be placed on frosted or frozen ground, nor should frozen material be placed as fill. Frozen ground should be allowed to thaw or be completely removed prior to placement of fill. A good practice is to cover the compacted fill with a “blanket” of loose fill to help prevent the compacted fill from freezing.

If the structures are erected during cold weather, foundations, concrete slabs-on-grade, or other concrete elements should not be constructed on frozen soil. Frozen soil should be completely removed from beneath the concrete elements, or thawed, scarified and recompacted. The amount of time passing between excavation or subgrade preparation and placing concrete should be minimized during freezing conditions to prevent the prepared soils from freezing. The use of blankets, soil cover or heating as required may be utilized to prevent the subgrade from freezing.

4.2.9 Construction Testing and Observation

Testing and construction observation should take place under the direction of VIVID to support that engineer's professional opinion as to whether the earthwork does or does not substantially conform to the recommendations in this report. Furthermore, the opinions and conclusions of a geotechnical report are based upon the interpretation of a limited amount of information obtained from the field exploration. It is therefore not uncommon to find that actual site conditions differ somewhat from those indicated in the report. The geotechnical engineer should remain involved throughout the project to evaluate such differing conditions as they appear, and to modify or add to the geotechnical recommendations, as necessary.

4.2.10 Drainage

Positive drainage away from the proposed improvements is essential to the performance of foundations, retaining walls and pavements, and should be provided for the life of the structures. Drainage should be created such that water is diverted off the site and away from structures and other improvements.

4.2.11 Permanent Cut and Fill Slopes

If required, permanent cut and fill slopes exposing the materials encountered in our borings are anticipated to be stable at slope ratios as steep as 3:1 (horizontal to vertical) under dry conditions. The site soils are known to have significant erodibility characteristics. A 3:1 (horizontal to vertical) or shallower slope will result in less erosion and maintenance issues. New slopes should be revegetated as soon as possible after completion to reduce erosion problems. Slopes steeper than that recommended above are possible with proper earth retention and erosion control designs.

4.3 DEEP FOUNDATION SYSTEM RECOMMENDATIONS

Based on discussions with AECOM, we anticipate that the single-span bridge structure option will most likely be supported on either drilled caissons or driven steel piles bearing in the underlying competent bedrock materials. Depth to COMPETENT BEDROCK indicates the depth to which caisson "penetration" begins or the required driven pile tip elevations to achieve the geotechnical capacities, as shown on the attached boring logs and specified in the following sections of this report. It should be noted that due to the degree of weathering of the upper portions of the bedrock, the top of competent bedrock does not coincide with the top of the bedrock formation. The geotechnical design recommendations for deep foundations are based on the AASHTO LRFD Bridge Design Specification, 8th Edition manual, and are presented below.

4.3.1 Straight Shaft Drilled Caissons

Based on the available subsurface information, it appears the site is suitable for support of the bridge abutments on drilled caissons extending through the overlying soils and founded in competent bedrock. Competent bedrock was encountered in the borings at depths of approximately 25 and 35 feet below the existing ground surface in borings B-1 and B-2, respectively. The drilled caissons should be founded at a depth that provides a minimum penetration into the competent bedrock, as shown below in Table 5. Additional penetration may be required to meet lateral load resistance based on parameters provided in Section 4.3.3.

Drilled caissons founded as recommended in the competent bedrock may be sized using a factored side resistance and end bearing resistance as shown in Table 5, below. <u>Due to the differences in movement</u>

required to mobilize side resistance, side resistance is only provided for the portion of the caisson in contact with the competent bedrock. Side resistance for the "weathered sandstone" and overburden soils should be ignored.

Table 5
Drilled Caisson Design Parameters

<table>
<tr><th rowspan="2">Boring Location</th><th rowspan="2">Depth to Competent Bedrock

(below ground surface)</th><th rowspan="2">Minimum Drilled Caisson Embedment into Competent Bedrock (feet)[3]</th><th colspan="2">End Bearing Resistance (ksf)

(Competent Bedrock Only) [1,2]</th><th colspan="2">Side Resistance (ksf)

(Competent Bedrock Only) [2]</th></tr>
<tr><th>Nominal</th><th>Factored

($\phi_{r\text{-end}}$ = 0.55)</th><th>Nominal</th><th>Compression Factored

($\phi_{r\text{-side comp}}$=0.60)</th></tr>
<tr><td>B-1

(Near Existing North Abutment)</td><td>25</td><td rowspan="2">1.5B

or 6 feet minimum, whichever is greater

(B = caisson diameter)</td><td rowspan="2">31.6</td><td rowspan="2">17.4</td><td rowspan="2">5.2</td><td rowspan="2">3.1</td></tr>
<tr><td>B-2

(Near Existing South Abutment)</td><td>35</td></tr>
</table>

1) **Competent bedrock depth indicated in Table 5 and is shown on the boring logs in Appendix A. No end bearing resistance value is provided for weathered bedrock, as caissons must be socketed into competent bedrock.**
2) **The values for nominal and factored End Bearing and Side Resistance provided above assume 2 or more caissons will be utilized per structure.**
3) **Minimum embedment shown is that required to achieve the geotechnical parameters provided only. Structural engineer will determine additional embedment/penetration required to resist actual vertical and lateral structural loads when they are determined.**

- Calculations for uplift resistance, if required, should be based upon an uplift skin friction value of two-thirds of the factored compressive side friction for the portion of pier within the COMPETENT BEDROCK strata, which equates to 2.1 ksf.
- In order to achieve full design capacity for vertical loading conditions, drilled caissons should be installed at a minimum center-to-center spacing of three diameters. No axial resistance reduction in individual caisson capacity for group action is needed for this spacing. Adjacent drilled caissons with a clear space (edge to edge) of less than 3 caisson diameters should not be constructed on the same day.
- Caissons should be designed by a qualified structural engineer. Caissons should be reinforced their full length.

4.3.2 Driven Piles

As an alternative to drilled caissons, we anticipate that the proposed bridge may be supported on Grade 50 driven steel H-piles. We recommend that driven piles bear within the underlying competent bedrock.

However, we anticipate penetration into the bedrock with a driven pile will likely be minimal (limited to 5 to 10 feet) and that pile driving refusal (likely "absolute" refusal per Section 502.10 of Special Provision, 12/19/18) will take place within this approximate penetration into the competent bedrock.

Since piles will be driven into hard competent bedrock, it will be required that steel H-piles be driven with protective points to reduce the potential for damage to the pile during driving. Pre-drilling of piles, if required, should be performed in accordance with the latest version of Section 502 of the CDOT Standard Specifications, 2019. The pile driving contractor should be given the opportunity to review the boring logs to assess the appropriate driving technique.

At the time of publishing this document, the size of H-piles, pile driving equipment, and site survey elevations were unknown. **However, from a geotechnical end-bearing standpoint we recommend the pile tip elevations/depth be at or below the "competent bedrock" depth shown on the logs.** We anticipate deeper penetration may be required for lateral resistance. LRFD Bridge Design Specifications recommend a geotechnical resistance value of 0.65 for the evaluation of the factored geotechnical resistance if the project includes a dynamic pile testing program. However, if the foundations will include 5 piles or less at each abutment, the resistance factor should be reduced by 20 percent (i.e., to 0.52) as outlined in AASHTO 10.5.5.2.3 to reflect the potential for overstressing of an individual pile due to load sharing. The nominal and factored geotechnical structural pile capacities are summarized below. The estimated pile penetration is based on the subsurface exploration results at the location of the borings and variations in pile tip elevations should be anticipated during pile installations.

Table 6
Structural Steel HP Pile Capacities

Nominal Structural Capacity (ksi)	Factored Structural Capacity (ksi) ϕ = 0.65	Factored Structural Capacity (ksi)[1] ϕ = 0.52
50	32.5	26.0

1) If the foundations will include 5 piles or less at each abutment, the resistance factor should be reduced by 20 percent (i.e., to 0.52) as outlined in AASHTO 10.5.5.2.3 to reflect the potential for overstressing of an individual pile due to load sharing.

The depth at which the required LRFD nominal resistance will be achieved may vary due to variations in the bedrock type, weathering, and hardness. This should be considered when purchasing steel and planning splicing. PDA testing frequency will be controlled by the total number of piles installed for the structure. Based on the Table 10.5.5.2.3-1 of the AASHTO LRFD Bridge Design Specifications, dynamic testing of a minimum of 2 piles per site condition, but no less than 2 percent of the production piles, is required.

4.3.3 Lateral Resistance of Deep Foundations

We anticipate that the lateral load analyses will be performed by the structural engineer using the software program LPile developed by Ensoft, Inc. This program analyzes a single caisson considering deflection as a function of the design loads, foundation construction, and subsurface conditions. Table 7, below, provides LPile input parameters for the foundation soils and bedrock.

It is recommended that the lateral support from the soil within 3 feet of final design grade be ignored due to potential loss of support from frost penetration or other shallow ground disturbance. In addition, sloping grades, the influence of other structures, and scour depth must also be included in the lateral caisson analyses. The values provided are based on the existing subsurface conditions and were estimated, or calculated, based on generally accepted engineering correlations.

Table 7
LPile Parameters

Depth (ft)	P-Y Curve Model	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle (deg)	Strain Factor, ε50	Soil modulus, k (pci)
B-1 (Near Existing North Abutment)						
3 to 19	Sand (Loose) (Above Water)	120	n/a	28	n/a	25
19 to 22	Stiff Clay without Free Water (Weathered Claystone)	100	1,000	n/a	0.007	500
22 to 40	Very Stiff Clay (Claystone/Sandstone)	125	2,000	n/a	0.005	1,000
B-2 (Near Existing South Abutment)						
3 to 24.5	Sand (Loose) (Above Water)	120	n/a	28	n/a	25
24.5 to 33	Stiff Clay with Free Water (Weathered Claystone)	100	1,000	n/a	0.007	500
33 to 39.5	Very Stiff Clay (Claystone/Sandstone)	125	2,000	n/a	0.005	1,000

If the foundations are spaced closer than five times the diameter center to center, lateral resistances should be scaled by an appropriate multiplier from Table 10.7.2.4-1 of AASHTO LRFD, which is summarized in Table 8, below.

Table 8
LPILE P-Multiplier for Multiple Row Pile Groups

Pile Center to Center Spacing (in the direction of loading)	P-Multipliers, Pm		
	Row 1	Row 2	Row 3 and Higher
3B[1]	0.8	0.4	0.3
5B[1]	1.0	0.85	0.7

1) B = Caisson Diameter

4.3.4 Drilled Caisson Construction Considerations

- Drilled caissons should be installed in accordance with the latest version of Section 503 of the CDOT Standard Specifications for Road and Bridge Construction (CDOT, 2019). Excavation for the drilled caissons will encounter granular surficial soils overlying sandstone and claystone bedrock. Conventional drilling equipment should be able to penetrate the overburden soil and bedrock. The drilled caisson contractor should be given the opportunity to review the boring logs to assess the appropriate drilling technique.
- Concrete placement should be performed according to CDOT Standard Specifications for Road and Bridge Construction, 2019, Section 503 – Drilled Caissons. Per CDOT Standard Specifications, and to reduce disturbance of the bearing surface, concrete should be placed within four hours of drilling. The bottom of the drilled caisson excavation should be cleaned of water and loose material before placing reinforcing steel and concrete. Concrete placement should be continuous from the bottom to the top elevation of the caisson. For dry excavation, concrete can be placed by either tremie or free fall methods using hopper or other approved equipment.
- Groundwater was encountered in boring B-2 drilled during this investigation at the time of drilling, as well as zones of clean sand that will be prone to caving. The use of mud slurry and/or temporary casing, and potentially de-watering techniques will be required. If more than 3 inches of groundwater is present in the caisson hole at time of concrete is poured the concrete should be pumped from the bottom of the hole to the top in order to displace the water. Caissons should be filled with concrete immediately after they are drilled, cleaned, and inspected. Caissons holes should not be left open overnight.
- Since drilled caissons will be installed using casing and wet methods (i.e., using slurry to maintain excavation stability or control groundwater) at various locations, the contractor should prevent the slurry from “setting up”, control the sand content of the slurry to less than 4 percent by volume at any point in the excavation, and maintain the slurry level at a minimum of 5 feet above the highest expected piezometric head surface. Wet excavated caissons without temporary casing will require concrete placement using tremie methods. The tremie pipe should be clean and have a sufficient inside diameter for use with the specific concrete mix, but not less than 10 inches. The discharge end of the tremie should allow free radial flow of the concrete and be immersed at least 10 feet in concrete and maintain a positive pressure differential at all times during placement to prevent water or slurry intrusion.
- Drilled caisson installation should be observed by a VIVID representative to identify the bearing strata (e.g., top of “COMPETENT BEDROCK”), observe construction techniques, and confirm subsurface conditions are as anticipated from our exploratory borings. Should isolated areas of unsuitable material be encountered at planned depths, it will be necessary to deepen the drilled caisson to suitable bearing material.

4.3.5 Estimated Vertical Foundation Movement

Deep foundations designed in accordance with our recommendations should not produce long-term vertical movements in excess of about ½-inch.

4.4 SHALLOW FOUNDATION SYSTEMS

The possible replacement structures may include a multi-cell box culvert, pre-cast arch(s) structure, or single-span girder bridge. The foundation types of the replacement structure may include spread footings or GRS-IBS foundations. No plans of the possible replacement structures were provided to our office during this investigation. The feasibility of GRS-IBS foundations is currently pending a scour evaluation being performed by AECOM, as well as settlement analysis performed by VIVID. If GRS-IBS foundations are found to be a feasible alternative based on the scour and settlement analysis, VIVID should be contacted to provide revised recommendations.

The bearing conditions encountered in the borings around the anticipated replacement structure were generally loose to medium dense, moist, clayey to silty sand and moist, medium dense, silty to well-graded sand materials. Due to the nature of this site as a drainage and the proposed construction, the replacement structure foundations may be situated near or within groundwater or surface flows, depending on seasonal variability. It is this zone of material that will control design and performance of the replacement structure.

To help create a more uniform and stable platform on which to construct the replacement structure and any associated wing wall foundation elements, facilitate dewatering, if necessary, and reduce the potential for settlement of the proposed structures, we recommend the abutment foundations, box culvert and any associated wall structures be supported on shallow foundations bearing on a minimum 24-inch-thick zone of structural fill. In order to provide subgrade improvement, facilitate drainage/dewatering during construction (as necessary), and create a reasonable construction platform, this 24-inches of structural fill should comprise of material meeting CDOT Class I Structure Backfill specifications (see Table 703.08, Section 703, CDOT Standard Specifications for Road and Bridge Construction, 2019).

Additional subgrade stabilization may also be necessary. **Stabilization techniques can vary but may include use of a heavy-duty geogrid such as Tensar TX-7 with aggregate or rock (1 to 1.5-inch max aggregate size) or pushing rock (typically angular 3 to 12-inch rock) into the subgrade to minimize the instability.**

If the subgrade is unstable, we also highly recommend lightweight, tracked/low ground pressure equipment be utilized to perform earthwork operations for foundation preparation and to install fill and structural elements. This will help limit damage to the stabilized subgrade and reduce the required amount of stabilization.

Additional recommendations for shallow mat/footing foundation are presented below for preliminary design purposes for feasibility evaluation.

- Foundations bearing upon a zone of imported structural fill as described above may be designed for a maximum presumptive bearing resistance of 2,000 pounds per square foot (psf). This value was obtained from Table C10.6.2.6.1-1 of the AASHTO LRFD Bridge Design Specification, 8th Edition.
- Shallow foundation elements should have at least 36 inches of cover above the bottom of the foundation for frost protection, or that required by the local building code.

- **Once preliminary design provides estimated foundation size, dimension, and loading, settlement analysis will be required to evaluate if shallow foundations are feasible based on the magnitude of anticipated settlement.**

4.5 EARTH RETENTION SYSTEMS DESIGN CRITERIA

Various retaining wall structures may be utilized on this site, including bridge abutments/wing walls adjacent to the corners of the proposed bridge. Final wall type had not been finalized at the time of this report, but walls are anticipated to be either pre-cast or cast-in-place, reinforced concrete walls. The selection and design for retaining walls should be performed in general accordance with CDOT Bridge Design Manual and the AASHTO LRFD Bridge Design Specifications.

Based on our subsurface investigation, our earth retention system parameters provided herein assume dry subsurface conditions except where noted. If seepage or groundwater is encountered, we should be contacted to provide reduced soil strength values and additional wall drainage recommendations to protect the retained/reinforced zones of the wall systems. Because of hydrostatic pressure build-up, a drainage system is a pertinent part of any earth retention system and should be included in the final design of the selected wall type.

4.5.1 Structural Walls

Structural walls are defined as reinforced cast-in-place (CIP) or pre-cast concrete walls founded on shallow (cantilever) or deep foundation systems. The design and construction criteria presented below should be observed for structural retaining walls on this site.

To help create a more uniform foundation platform on which to construct the wall foundation elements (where shallow foundations are constructed) and reduce the potential for long-term settlement, we recommend the walls be supported on a 24-inch-thick zone of imported granular structural fill, as described in Section 4.2.4. Acceptable fill material for foundation subgrade and wall backfill as well as compaction requirements are provided in Sections 4.2.4 and 4.2.5. In addition, any soft/unstable subgrade soils should be stabilized prior to placement of the structural fill.

4.5.2 Wall Foundation Recommendations

Either a deep foundation system or a shallow footing foundation system may be utilized for retaining walls on this site. If drilled caissons are utilized the design should be based on the information provided in Section 4.3 of this report for bridge foundations. Where shallow foundations are desired the information provided above in Section 4.4 be utilized for design.

Estimated dimensions of the foundation elements and structural loads on the proposed footings for each wall were not provided. **Once final footing dimensions and structural loads are known, we should be contacted to review our recommendations.**

4.5.3 Uplift and Lateral Loads

Shallow foundations should be designed to resist uplift and lateral loading. Uplift loads can be resisted by the effective dead weight of the foundation plus the weight of any soil above the foundation. For design purposes, compacted soil above the foundation is estimated to have a moist unit weight of 120 pounds per cubic foot (pcf).

Lateral loads acting on the foundation may be resisted by passive resistance of the soils around the perimeter of the footing (at the toe of the wall) and sliding resistance acting on the base of the foundation between the concrete and soils. If the ground surface at the front face of each bridge abutment slopes down and away from the abutment greater than 5 (horizontal): 1(vertical), it is recommended that the passive earth pressure on the front side of the footing be reduced in computing the lateral resistance. Lateral resistance can be calculated utilizing the passive lateral earth pressure and sliding coefficient presented in Table 9, below.

4.5.4 Lateral Earth Pressures

Walls that retain earth on one side will be subjected to lateral earth pressures. Walls shall be backfilled with CDOT Class I Structure Backfill for the full width that influences lateral earth pressure. For design purposes, the following average soil parameters can be utilized:

- **CDOT Class I Structure Backfill:** Angle of Internal Friction (phi-angle) = 30 degrees; Unit Weight (unsaturated) = 120 pounds per cubic foot (pcf)
- **On-Site Sand Soils**: Angle of Internal Friction (phi-angle) = 28 degrees; Unit Weight (unsaturated) = 120 pounds per cubic foot (pcf)

Table 9
Lateral "Equivalent Fluid" Earth Pressure Parameter Summary

Parameter	CDOT Class I Structure Backfill (Above Groundwater)	CDOT Class I Structure Backfill (Below Groundwater)[4]	On-Site Sand Soils (Above Groundwater)	On-site Sand Soils (Below Groundwater)[4]
At-Rest[1]	60 pcf	91 pcf	64 pcf	93 pcf
Active[2]	40 pcf	82 pcf	43 pcf	83 pcf
Passive[3]	360 pcf	173 pcf	332 pcf	160 pcf
Unfactored Coefficient of Sliding Friction[3]	0.58	0.58	0.53	0.53

Notes: 1. Retaining walls that are laterally supported (structurally restrained from rotation) can be expected to undergo only a slight amount of deflection. These walls should be designed for an "at-rest" lateral earth pressure.

2. Retaining structures which can deflect sufficiently to mobilize the full "active" earth pressure condition should be designed for an "active" lateral earth pressure. For the medium dense sand soils encountered on this site, a wall deflection of at least 0.002H (where is H is the wall height) is required to mobilize active earth pressures.

3. Lateral loads may be resisted using these unfactored coefficients of sliding friction and unfactored passive earth pressures presented above. For passive resistance, a resistance factor (LRFD) of 0.5 should be used to minimize the lateral movement to mobilize full passive earth as this movement is generally well beyond structural movement tolerances. For sliding resistance of cast-in-place concrete on sand a resistance factor of 0.80 should be used, or for pre-cast concrete placed on sand a resistance factor of 0.90 should be used.

4. It should be noted that the hydrostatic water pressure (62.4 pcf) was already included in the pressure values for below groundwater condition.

4.5.5 Wall Drainage

Our earth retention system design recommendations provided herein assume dry subsurface conditions and a flat backfill surface except where noted otherwise. Drainage systems should be finalized when the earth retention system design has been selected. To prevent buildup of hydrostatic pressures behind retaining walls a drainage system must be constructed behind the earth retention system. If seepage or groundwater is encountered, we should be contacted to provide reduced soil strength values and wall drainage recommendations to protect the entire compacted fill zone of the wall systems.

Generally speaking, wall drain systems vary by wall type (e.g., CIP). CIP wall drain systems can incorporate a column of drain aggregate (typically minimum 12-inches thick and encapsulated in a suitable filter fabric (such as Mirafi 140N) to minimize intrusion of fines) or synthetic drain board in combination with weep holes or a drain pipe at or near the base of the walls to remove collected water from behind the face of the wall such that the fill and foundation soils are not negatively impacted. A less-permeable clayey cap should be placed above the free draining granular material to help mitigate infiltration of surface water.

4.6 METAL CORROSIVITY AND CONCRETE SULFATE DEGRADATION

Laboratory chloride concentration, sulfate concentration, sulfide concentration, pH, oxidation reduction potential, and electrical resistivity tests were performed on samples of onsite materials obtained during our field investigation. The results of the tests are included in Appendix C to this report and are summarized below in Table 10.

Table 10
Summary of Laboratory Soil Corrosivity Testing

Boring No.	Sample Depth (ft)	Lithology	Water Soluble Chloride (%)	pH	Redox Potential (mV)	Resistivity (ohm-cm)	Water Soluble Sulfate (%)	Sulfide Content
B-2	10	Silty SAND	0.0178	4.7	350.1	1302	0.004	Negative
B-2	34	Claystone	0.0004	6.6	389.4	741	0.002	Trace

4.6.1 Metal Corrosion

Laboratory testing was completed to provide data regarding corrosivity of onsite soils. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required.

Metal and concrete elements in contact with soil, whether part of a foundation system or part of a supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried metal and concrete elements should be designed to resist corrosion and degradation based on accepted practices.

Based on the "10-point" method developed by the American Water Works Association (AWWA) in standard AWWA C105/A21.5, the corrosivity test results indicate that the onsite sand soils have low corrosive potential, and the claystone bedrock has high corrosive potential. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures, if required.

4.6.2 Chemical Sulfate Susceptibility and Concrete Type

The degradation of concrete or cement grout can be caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) in their publication Guide to Durable Concrete (ACI 201.2R-08) provides guidelines for this assessment.

The concentration of water-soluble sulfates measured on subsurface materials submitted for testing represents a Class 0 exposure of sulfate attack on concrete exposed to the soils per CDOT Standard Specifications for Road and Bridge Construction, 2019, Section 601.04, Table 601-2.

4.7 PAVEMENT RECOMMENDATIONS

4.7.1 General

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. Soils are represented for pavement design purposes by means of a soil support value. Pavement design procedures are based on strength properties of the subgrade and pavement materials, along with the design traffic conditions.

We understand that new pavement areas on this site will include reconstruction of the existing bridge approach asphalt pavement, as well as areas of new asphalt pavement to support widening around the new bridge. Included herein is the pavement section thickness design that meets the El Paso County Engineering Criteria requirements, including the minimum required pavement section thickness for design based on the roadway classification/traffic loading (ESAL) and subgrade soil modulus.

4.7.2 Anticipated Pavement Subgrade Material

Our borings indicate the pavement subgrade soils comprise existing clayey sand materials. Under the AASHTO classification system, the soils tested predominantly classified as A-6 soils. Clayey sand soils are generally considered to provide fair to poor support for pavements. A Hveem stabilometer (R-Value) test was performed on a bulk soil sample of A-6 soils obtained from the borings and the resulting R-value was 9. A resilient modulus (M_R) value of 3,448 psi was calculated from the appropriate AASHTO R-value conversion formula referenced within the El Paso County Pavement Design Criteria Manual.

Two samples of clayey soils were found to exhibit low swell potential (0.8 and 1.0 percent) when wetted under a 200 pounds per square foot (psf) surcharge load. According to the El Paso County Pavement Design Criteria Manual, subgrade soils with swell percentages less than 2 percent do not require mitigation for expansive potential. We believe the potential risk for future vertical movement and associated impact to surface rideability is low in this area.

Our pavement investigation and thickness calculations were performed in general accordance with the El Paso County Pavement Design Criteria Manual, which is based on the 1993 American Association of State and Highway Transportation Officials (AASHTO) Guide for Design of Pavement Structures. Included herein are options for pavement section thickness design that meet the El Paso County Pavement Design Criteria

Manual requirements, including the minimum required pavement section thickness for design based on the provided roadway classification/traffic loading (ESAL) and subgrade soil modulus.

The following sections describe in more detail the pavement section thickness design recommendations for areas requiring new pavement section construction.

4.7.3 Pavement Design Parameter Summary

Based upon information provided by AECOM, the above-referenced pavement design manual, and the subgrade strength value based on materials obtained in the borings, the following table presents the pavement design parameters that were utilized in our design. These parameters were utilized to calculate required thickness of new Hot Mix Asphalt (HMA) and Aggregate Base Course (ABC) layers. The roadway classification of Minor Arterial (Rural) was utilized based on information provided in the project RFP document provided by El Paso County.

Table 11
Summary of Pavement Design Parameters
Areas Requiring New Flexible (HMA) Composite Pavement Section Construction

Flexible Pavement Design Parameters	
Roadway Classification[1]	Minor Arterial (Rural)
Required Minimum HMA [in.] [1]	4
Required Minimum ABC [in.] [1]	8
Initial Serviceability	4.5
Terminal Serviceability[1]	2.5
Overall Standard Deviation[1]	0.44
Reliability [%][1]	80
20-year, 18-kip ESAL[1]	689,850
Design R-Value	9
Resilient Modulus (M_R) [psi]	3,448
Strength Coefficients	
New Hot Mix Asphalt[1]	0.44
New Aggregate Base Course[1]	0.11

1) Indicates classification and pavement design parameter(s) obtained from the El Paso County Pavement Design Criteria Manual.

If traffic estimates vary significantly from those assumed, we should be contacted to re-evaluate our recommendations. The following pavement sections were designed using the AASHTO design methods for flexible pavements and El Paso County Pavement Design Criteria. All pavement thickness recommendations based on ESAL values for mainlines only. Specific adjustments for turn-lanes, acceleration/deceleration lanes, shoulders, etc. are not included.

4.7.4 Design Sections

Our recommended pavement sections below are for the new pavement proposed for the roadway areas on Calhan Highway. Material requirements and compaction specifications for HMA, ABC, and subgrade materials are presented in Sections 4.7.6 and 4.2.5, respectively. The following describes our recommended design sections that include the required thickness of HMA and ABC layers.

Table 12
Pavement Section Thicknesses - Mainline Calhan Highway(2)

Location	20-year Design Composite Flexible Section Thickness (HMA/ABC) (HMA 20-year ESAL = 689,850)
Calhan Highway – New Pavement Areas (Design R = 9)	7” HMA *over* 8.5” ABC

Notes:

1. The pavement sections will overlie a properly prepared subgrade as described in Sections 4.2.2 and 4.7.5.
2. Pavement Section Thicknesses were calculated using the El Paso County Pavement Design Criteria Manual, which is based on the 1993 American Association of State and Highway Transportation Officials (AASHTO) Guide for Design of Pavement Structures.
3. All pavement thickness recommendations based on existing pavement subgrade strength parameters and default ESAL values for mainlines only. Specific adjustments for turn-lanes, acceleration/deceleration lanes, shoulders, etc. not included.

4.7.5 Pavement Subgrade Preparation

Any obviously unsuitable materials present (e.g., debris, organic materials, waste) should be completely removed. Remove the stripped materials for offsite disposal in accordance with local laws and regulations.

Prior to placement of new pavement sections, processing of the subgrade should be performed as described in Sections 4.2.2 and 4.7.2. Prior to placing the pavement section, the prepared subgrade should be proof-rolled with a heavily loaded pneumatic-tired vehicle (such as a fully-loaded water truck) after preparation. Areas that pump or deform significantly under heavy wheel loads are not stable and should be stabilized prior to paving. The method and extent of stabilization should conform to the El Paso County Pavement Design Criteria Manual and Engineering Specifications. The final stabilization approach/method and depth shall be approved by the Engineer.

4.7.6 Pavement Materials

Hot Mix Asphalt (HMA) design and construction shall conform to the requirements of the current Pikes Peak Region Asphalt Paving Specifications (PPRAPS) manual. The asphalt pavement should consist of a bituminous plant mix composed of a mixture of aggregate and bituminous material that meets the requirements of a job-mix formula established by a qualified engineer. Based on Table 2.04 of the PPRAPS, the HMA binder grade shall be PG 58-28. Aggregate grading S or SX may be used. The HMA pavement should be placed in lifts not to exceed 3 inches in thickness, unless otherwise accepted by the project engineer, and be compacted to between 92 percent and 96 percent of its maximum theoretical (Rice) density.

Aggregate Base Course (ABC) materials should conform to Class 5 or 6 ABC specifications per Table D-6, in Appendix D of the El Paso County Engineering Criteria Manual. The ABC material should be placed in a uniform layer without segregation of size to a compacted maximum lift thickness of 6-inches. ABC should be moisture conditioned and compacted as described in Section 4.2.5 of this report.

Use of blankets, soil cover, or heating, may be required to help prevent the subgrade from freezing if construction occurs during cold weather.

4.7.7 Pavement Construction Considerations

All site preparation, earthwork operations and construction materials should be performed in accordance with applicable codes, safety regulations and other local, State or Federal guidelines as applicable including, but not limited to:

- El Paso County Engineering Standard Specifications;
- El Paso County Pavement Design Criteria Manual;
- Pikes Peak Region Asphalt Paving Specifications Manual, and;
- Colorado Department of Transportation (CDOT), as applicable, and included by reference.

Of particular importance are those specifications directed towards embankment construction, subgrade compaction, base course compaction, and utility trench compaction. Prior to pavement construction, the prepared subgrade should be proof-rolled with heavy construction equipment. A fully loaded water truck would be acceptable for this purpose. During proof-rolling, particular attention should be directed to the area immediately adjacent to manholes, valves, catch basins, and other similar surface features. Areas which exhibit excessive deflection during proof-rolling should be over-excavated and stabilized as required. If soil is imported to the subject site for final grading, the soil materials must be of a character similar to those described in this report.

Proper drainage is of paramount importance in enhancing pavement performance. To avoid distress to pavement from wet subgrade soils, we recommend the maintenance of good drainage away from all pavements. Possible water sources include storm runoff, irrigation of landscaping adjacent the pavement and localized groundwater seepage, among others. Landscaping adjacent to the pavements should be avoided. Joints in the pavement or at asphalt/concrete interfaces should be sealed. Any cracks or openings in the finished pavement surface should be sealed and/or repaired as quickly as possible.

5.0 ADDITIONAL SERVICES & LIMITATIONS

5.1 ADDITIONAL SERVICES

Attached to this report is a document by the Geoprofessional Business Association (GBA) that summarizes limitations of geotechnical reports as well as additional services that are required to further confirm subgrade materials are consistent with that encountered at the specific boring locations presented in this report. This document should be read in its entirety before implementing design or construction activities. Examples of other services beyond completion of a geotechnical report are necessary or desirable to complete a project satisfactorily include:

- Review of design plans and specifications to verity that our recommendations were properly interpreted and implemented.
- Attendance at pre-bid and pre-construction meetings to highlight important items and clear up misunderstandings, ambiguities, or conflicts with design plans and specifications.
- Performance of construction observation and testing which allows verification that existing materials at locations beyond our borings are consistent with that presented in our report, construction is compliant with the requirements/recommendations, evaluation of changed conditions.

5.2 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of VIVID's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. VIVID makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The work performed was based on project information provided by Client. If Client does not retain VIVID to review any plans and specifications, including any revisions or modifications to the plans and specifications, VIVID assumes no responsibility for the suitability of our recommendations.

Figures

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N

VIVID Engineering Group

Project No: D20-2-333	**VICINITY MAP**
Date: December 29, 2020	
Drawn by: BTM	Proposed Calhan Highway Bridge Replacement South of Paint Mine Road El Paso County, Colorado
Reviewed by: WJB	

Figure

1

LEGEND

= APPROXIMATE BORING LOCATION

N

Not to Scale. Base image dated 6/13/17 and obtained from Google Earth, 2020

VIVID Engineering Group

Project No: D20-2-333	**EXPLORATION LOCATION PLAN**	Figure 2
Date: December 29, 2020		
Drawn by: BTM	Proposed Calhan Highway Bridge Replacement South of Paint Mine Road El Paso County, Colorado	
Reviewed by: WJB		

Appendix A

Logs of Exploratory Borings

VIVID Engineering Group, Inc.
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Colorado Springs, CO 80907
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Fax: 719-896-4357

KEY TO SYMBOLS

CLIENT AECOM
PROJECT NAME Proposed Calhan Highway Bridge Replacement
PROJECT NUMBER D20-2-333
PROJECT LOCATION Calhan, Colorado

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

- ASPHALT
- CL: USCS Low Plasticity Clay
- CLAYSTONE
- FILL
- SANDSTONE
- SM: USCS Silty Sand
- SW-SM: USCS Well-graded Sand with Silt
- WEATHERED CLAYSTONE

SAMPLER SYMBOLS

- Grab Sample
- 2" I.D. Modified California Sampler (MC)
- Rock Core
- Standard Penetration Test (SPT)

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
MC - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
FINES- PERCENT PASSING NO. 200 SIEVE
UCS - UNCONFINED COMPRESSIVE STRENGTH

∇ Water Level at Time Drilling, or as Shown

VIVID Engineering Group, Inc.
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BORING NUMBER B-1

CLIENT AECOM
PROJECT NAME Proposed Calhan Highway Bridge Replacement
PROJECT NUMBER D20-2-333
PROJECT LOCATION Calhan, Colorado
DATE STARTED 10/5/20 **COMPLETED** 10/5/20
GROUND ELEVATION 6679.18 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Custom Auger Drilling (CME-55)
GROUND WATER LEVELS:
DRILLING METHOD 4" Solid Stem Auger
AT TIME OF DRILLING ---
LOGGED BY M. Ray **CHECKED BY** B. Mustain
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	TESTS
0–2	GB			MC = 11.4% LL = 32 PL = 15 Fines = 38.0%
~3	MC		6-5	
~4	GB			
~5.5	MC		4-4	MC = 13.0% DD = 112.7 pcf LL = 25 PL = 16 Fines = 36.0%
~8	MC		4-4	
~10.5	MC		8-8	MC = 9.0% DD = 107.3 pcf LL = NP PL = NP Fines = 18.0%
15	MC		8-8	
20	MC		10-8	MC = 28.6% DD = 95.4 pcf Swell = 1.9% when wetted under 1,000 PSF load
25	MC		25-50/6"	
25–30	RC	90 (64)		MC = 31.3% DD = 90.3 pcf UCS = 3,982 PSF, Shear Strength = 1,991 PSF
30–35	RC	100 (87)		MC = 20.5% DD = 108.3 pcf UCS = 25,297 PSF, Shear Strength = 12,648 PSF
35–40	RC	88 (87)		MC = 21.2% DD = 106.4 pcf UCS = 31,409 PSF, Shear Strength = 15,704 PSF

Depth (ft)	MATERIAL DESCRIPTION	Elevation (ft)
0.5	**Asphalt - 7 inches**	6678.7
9.0	**Existing Fill** Clayey SAND, brown, dark brown, moist, loose to medium dense	6670.2
19.0	Silty SAND with clayey lenses, light brown, brown, moist, medium dense -thin clay layer encountered at approximately 10 feet below the existing ground surface	6660.2
22.0	**Dawson Formation** Weathered CLAYSTONE, light gray, iron staining, moist, medium hard	6657.2
32.8	**Dawson Formation** CLAYSTONE, light gray, brown, iron staining, waxy, highly weathered, highly to intensely fractured	6646.4
34.5	**Dawson Formation** SANDSTONE, gray, slightly weathered, moderately fractured, iron staining along fractures	6644.7
37.1	**Dawson Formation** CLAYSTONE, dark gray, slightly weathered, moderately fractured	6642.1
39.6	**Dawson Formation** SANDSTONE, light gray, slightly weathered, moderately fractured	6639.6
40.0	**Dawson Formation** CLAYSTONE, dark gray, slightly weathered	6639.2

Top of Competent Bedrock (25' / Elev. 6654.2)

Bottom of borehole at 40.0 feet.

VIVID Engineering Group, Inc.
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BORING NUMBER B-2

CLIENT AECOM

PROJECT NAME Proposed Calhan Highway Bridge Replacement

PROJECT NUMBER D20-2-333

PROJECT LOCATION Calhan, Colorado

DATE STARTED 10/5/20 **COMPLETED** 10/5/20

GROUND ELEVATION 6681.652 ft **HOLE SIZE** 4 inches

DRILLING CONTRACTOR Custom Auger Drilling (CME-55)

GROUND WATER LEVELS:

DRILLING METHOD 4" Solid Stem Auger

AT TIME OF DRILLING 25.50 ft / Elev 6656.15 ft

LOGGED BY M. Ray **CHECKED BY** B. Mustain

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	Elev.
0.5	**Asphalt - 5 inches**	6681.2
10.0	**Existing Fill** Silty, Clayey SAND, dark brown, moist, loose	6671.7
24.5	Well-Graded SAND with Silt, light brown, moist, medium dense	6657.2
28.0	CLAY, gray, brown, moist, stiff	6653.7
33.0	**Dawson Formation** Weathered CLAYSTONE, light gray, moist, medium hard, blocky, waxy, gradually hardens with depth into unweathered bedrock	6648.7
39.5	**Dawson Formation** CLAYSTONE, light gray, hard, moist	6642.2

Top of Competent Bedrock (35' / Elev. 6646.7)

Bottom of borehole at 39.5 feet.

SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS
GB		MC = 10.1%, LL = 32, PL = 16, Fines = 32.0%
MC	5-5	
GB		MC = 13.4%, LL = 23, PL = 17, Fines = 36.0%
MC	3-3	
MC	4-4	
MC	9-9	Chloride=0.0178%, pH=4.7, Redox Potential=350.1 mv, Resistivity=1302 ohm.cm, Sulfate=0.004%, Sulfide=Negative
MC	9-10	
SPT	6-8-11 (19)	MC = 6.9%, LL = NP, PL = NP, Fines = 12.0%
SPT	4-6-7 (13)	
MC	11-15	MC = 31.9%, DD = 86.7 pcf, Swell = 0.4% when wetted under 1,000 PSF load
MC	30-31	Chloride=0.0004%, pH=6.6, Redox Potential=389.4 mv, Resistivity=741 ohm.cm, Sulfate=0.002%, Sulfide=Trace
MC	50/7"	MC = 3.1%, DD = 106.9 pcf, Swell = 3.1% when wetted under 1,000 PSF load

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BORING NUMBER P-1

CLIENT AECOM
PROJECT NAME Proposed Calhan Highway Bridge Replacement
PROJECT NUMBER D20-2-333
PROJECT LOCATION Calhan, Colorado
DATE STARTED 10/5/20 **COMPLETED** 10/5/20
GROUND ELEVATION 6676.736 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Custom Auger Drilling (CME-55)
GROUND WATER LEVELS:
DRILLING METHOD 4" Solid Stem Auger
AT TIME OF DRILLING ---
LOGGED BY M. Ray **CHECKED BY** B. Mustain
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation
0.0					**Asphalt - 7 inches**	
0.5	GB		MC = 6.5% LL = 28 PL = 14 Fines = 45.0%		**Existing Fill** Clayey SAND, dark brown, moist, loose	6676.2
	MC	5-5				
2.5	GB					
4.5	MC	5-7	MC = 16.2% DD = 106.8 pcf Swell = 0.8% when wetted under 200 PSF load		Sandy CLAY, brown, moist, stiff	6672.2
5.0						
6.0					Silty SAND, brown, moist, medium dense	6670.7
7.5						
10.0	MC	8-9				6666.7

Bottom of borehole at 10.0 feet.

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BORING NUMBER P-2

CLIENT AECOM
PROJECT NAME Proposed Calhan Highway Bridge Replacement
PROJECT NUMBER D20-2-333
PROJECT LOCATION Calhan, Colorado
DATE STARTED 10/5/20 **COMPLETED** 10/5/20
GROUND ELEVATION 6683.426 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Custom Auger Drilling (CME-55)
GROUND WATER LEVELS:
DRILLING METHOD 4" Solid Stem Auger
AT TIME OF DRILLING ---
LOGGED BY M. Ray **CHECKED BY** B. Mustain
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					**Asphalt - 5 inches**	
0.5						6682.9
	GB		MC = 3.0% LL = 33 PL = 15 Fines = 48.0%		**Existing Fill** Clayey SAND, dark brown, moist, loose to medium dense	
	MC	6-7	MC = 22.7% DD = 96.1 pcf Swell = 1.0% when wetted under 200 PSF load			
2.5	GB					
5.0	MC	4-4				
7.5						
10.0	MC	12-7				6673.4

Bottom of borehole at 10.0 feet.

Appendix B

Geotechnical Laboratory Test Results

VIVID Engineering Group, Inc.
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SUMMARY OF LABORATORY RESULTS

CLIENT AECOM **PROJECT NAME** Proposed Calhan Highway Bridge Replacement

PROJECT NUMBER D20-2-333 **PROJECT LOCATION** Calhan, Colorado

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)		
B-1	0.5	32	15	17	9.5	38	SC	11.4			
B-1	5.0	25	16	9	4.75	36	SC	13.0	112.7		
B-1	10.0	NP	NP	NP	12.5	18	SM	9.0	107.3		
B-1	19.0							28.6	95.4		
B-1	28.8							31.3	90.3		
B-1	33.0							20.5	108.3		
B-1	38.9							21.2	106.4		
B-2	0.5	32	16	16	9.5	32	SC	10.1			
B-2	2.5	23	17	6	4.75	36	SC-SM	13.4			
B-2	19.0	NP	NP	NP	12.5	12	SW-SM	6.9			
B-2	29.0							31.9	86.7		
B-2	39.0							3.1	106.9		
P-1	0.5	28	14	14	4.75	45	SC	6.5			
P-1	4.0							16.2	106.8		
P-2	0.5	33	15	18	9.5	48	SC	3.0			
P-2	1.0							22.7	96.1		

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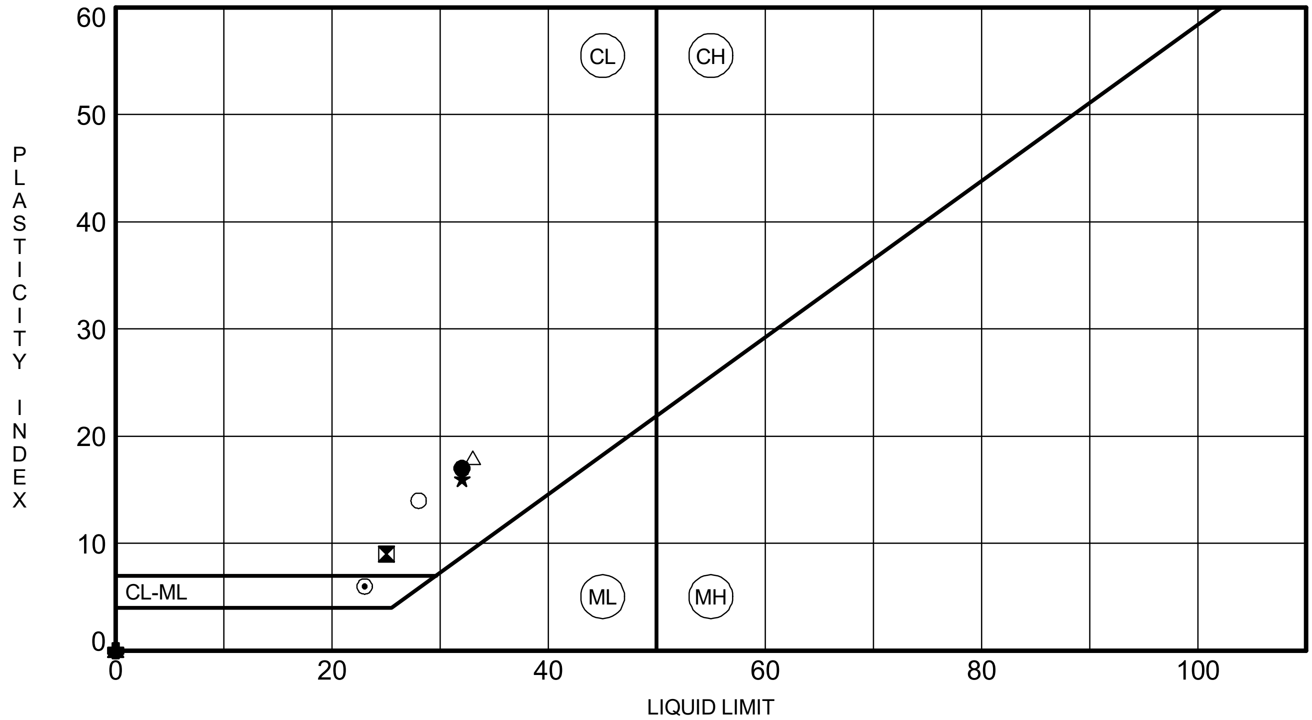
ATTERBERG LIMITS' RESULTS

CLIENT AECOM

PROJECT NAME Proposed Calhan Highway Bridge Replacement

PROJECT NUMBER D20-2-333

PROJECT LOCATION Calhan, Colorado



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	**B-1**	**0.5**	**32**	**15**	**17**	**38**	**CLAYEY SAND(SC)**
⧗	**B-1**	**5.0**	**25**	**16**	**9**	**36**	**CLAYEY SAND(SC)**
▲	**B-1**	**10.0**	**NP**	**NP**	**NP**	**18**	**SILTY SAND(SM)**
★	**B-2**	**0.5**	**32**	**16**	**16**	**32**	**CLAYEY SAND(SC)**
⊙	**B-2**	**2.5**	**23**	**17**	**6**	**36**	**SILTY, CLAYEY SAND(SC-SM)**
✿	**B-2**	**19.0**	**NP**	**NP**	**NP**	**12**	**WELL-GRADED SAND with SILT(SW-SM)**
○	**P-1**	**0.5**	**28**	**14**	**14**	**45**	**CLAYEY SAND(SC)**
△	**P-2**	**0.5**	**33**	**15**	**18**	**48**	**CLAYEY SAND(SC)**

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Telephone: 719-896-4356
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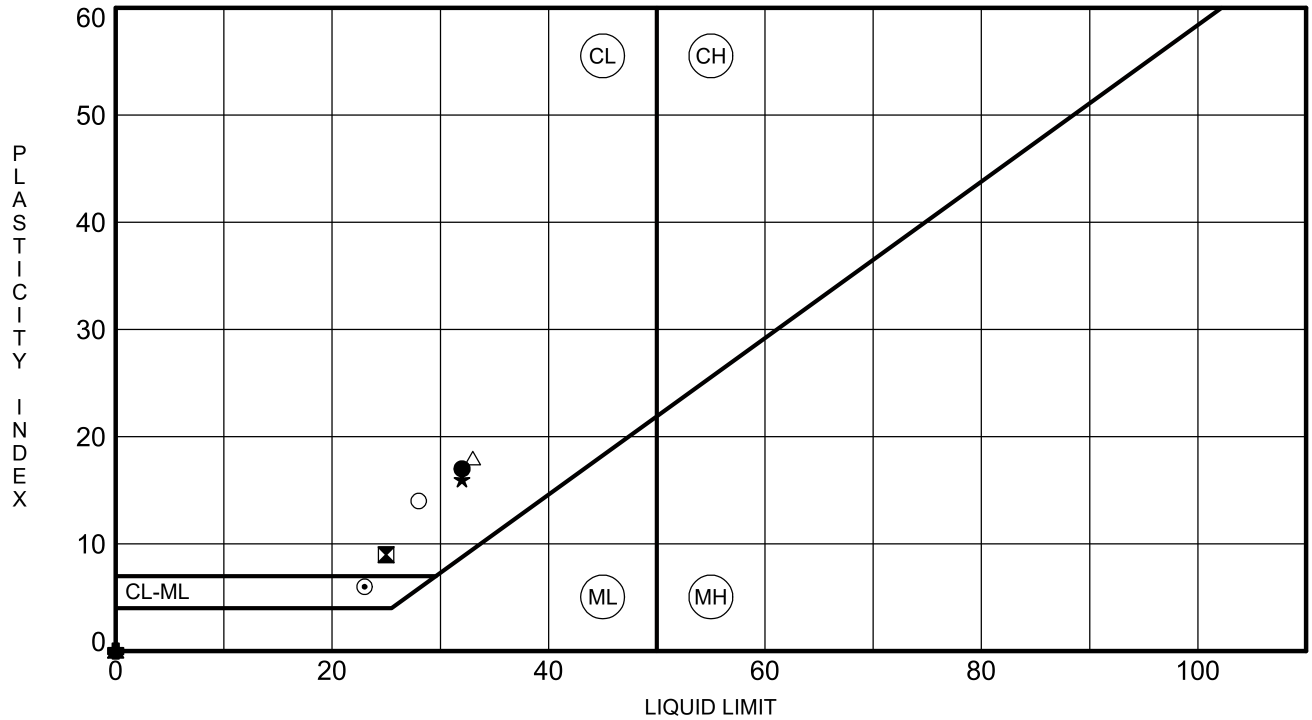
ATTERBERG LIMITS' RESULTS

CLIENT AECOM

PROJECT NAME Proposed Calhan Highway Bridge Replacement

PROJECT NUMBER D20-2-333

PROJECT LOCATION Calhan, Colorado



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	**B-1**	**0.5**	**32**	**15**	**17**	**38**	**(A-6)**
⧗	**B-1**	**5.0**	**25**	**16**	**9**	**36**	**(A-4)**
▲	**B-1**	**10.0**	**NP**	**NP**	**NP**	**18**	**(A-2-4)**
★	**B-2**	**0.5**	**32**	**16**	**16**	**32**	**(A-2-6)**
⊙	**B-2**	**2.5**	**23**	**17**	**6**	**36**	**(A-4)**
✿	**B-2**	**19.0**	**NP**	**NP**	**NP**	**12**	**(A-2-4)**
○	**P-1**	**0.5**	**28**	**14**	**14**	**45**	**(A-6)**
△	**P-2**	**0.5**	**33**	**15**	**18**	**48**	**(A-6)**

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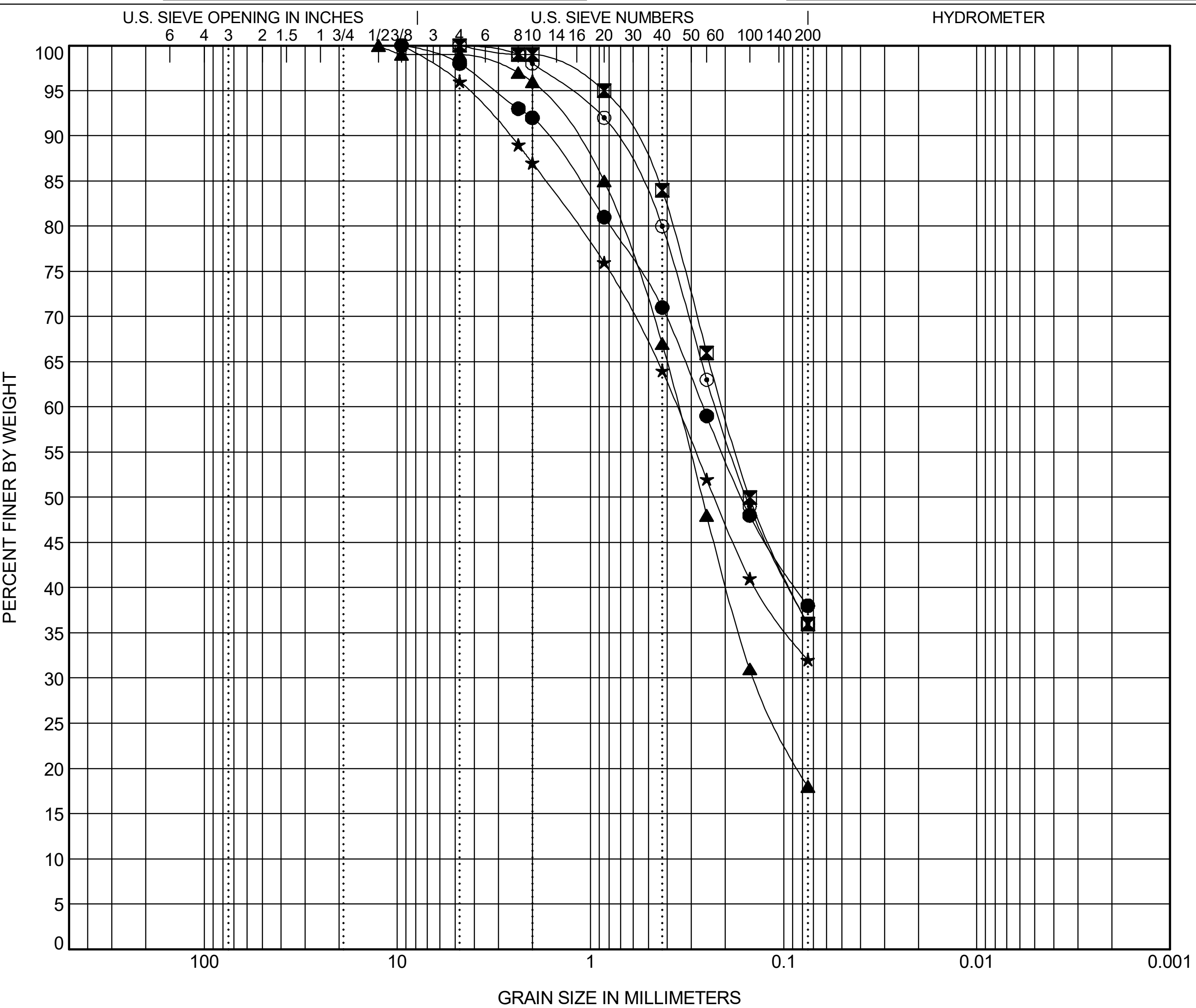
GRAIN SIZE DISTRIBUTION

CLIENT AECOM **PROJECT NAME** Proposed Calhan Highway Bridge Replacement

PROJECT NUMBER D20-2-333 **PROJECT LOCATION** Calhan, Colorado



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	B-1	0.5	CLAYEY SAND(SC)	32	15	17		
⊠	B-1	5.0	CLAYEY SAND(SC)	25	16	9		
▲	B-1	10.0	SILTY SAND(SM)	NP	NP	NP		
★	B-2	0.5	CLAYEY SAND(SC)	32	16	16		
⊙	B-2	2.5	SILTY, CLAYEY SAND(SC-SM)	23	17	6		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	0.5	9.5	0.261			2.0	60.0	38.0	
⊠	B-1	5.0	4.75	0.206			0.0	64.0	36.0	
▲	B-1	10.0	12.5	0.35	0.142		1.0	81.0	18.0	
★	B-2	0.5	9.5	0.356			4.0	64.0	32.0	
⊙	B-2	2.5	4.75	0.224			0.0	64.0	36.0	

C:\USERS\BRYSEN MUSTAIN\VIVID ENGINEERING GROUP\GEOTECH GROUP VIVID ENGINEERING - DOCUMENTS\PROJECTS_2020\D20-2-333_AECOM_CALHAN HIGHWAY BRIDGE_GEO\6 - DRAFTING\D20-2-333 GRAIN SIZE - GINT STD US LAB.GDT - 11/3/20 14:44

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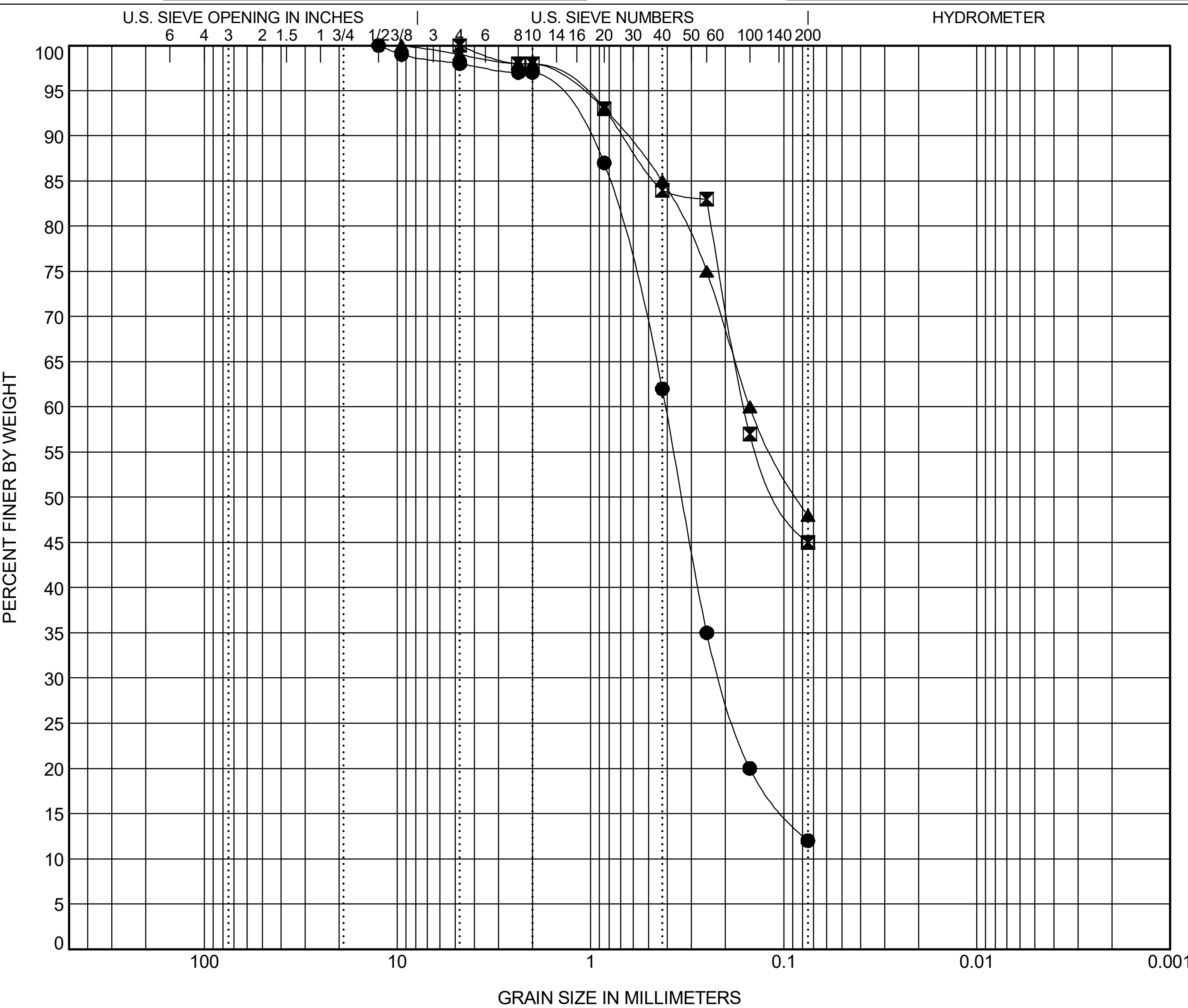
GRAIN SIZE DISTRIBUTION

CLIENT AECOM

PROJECT NAME Proposed Calhan Highway Bridge Replacement

PROJECT NUMBER D20-2-333

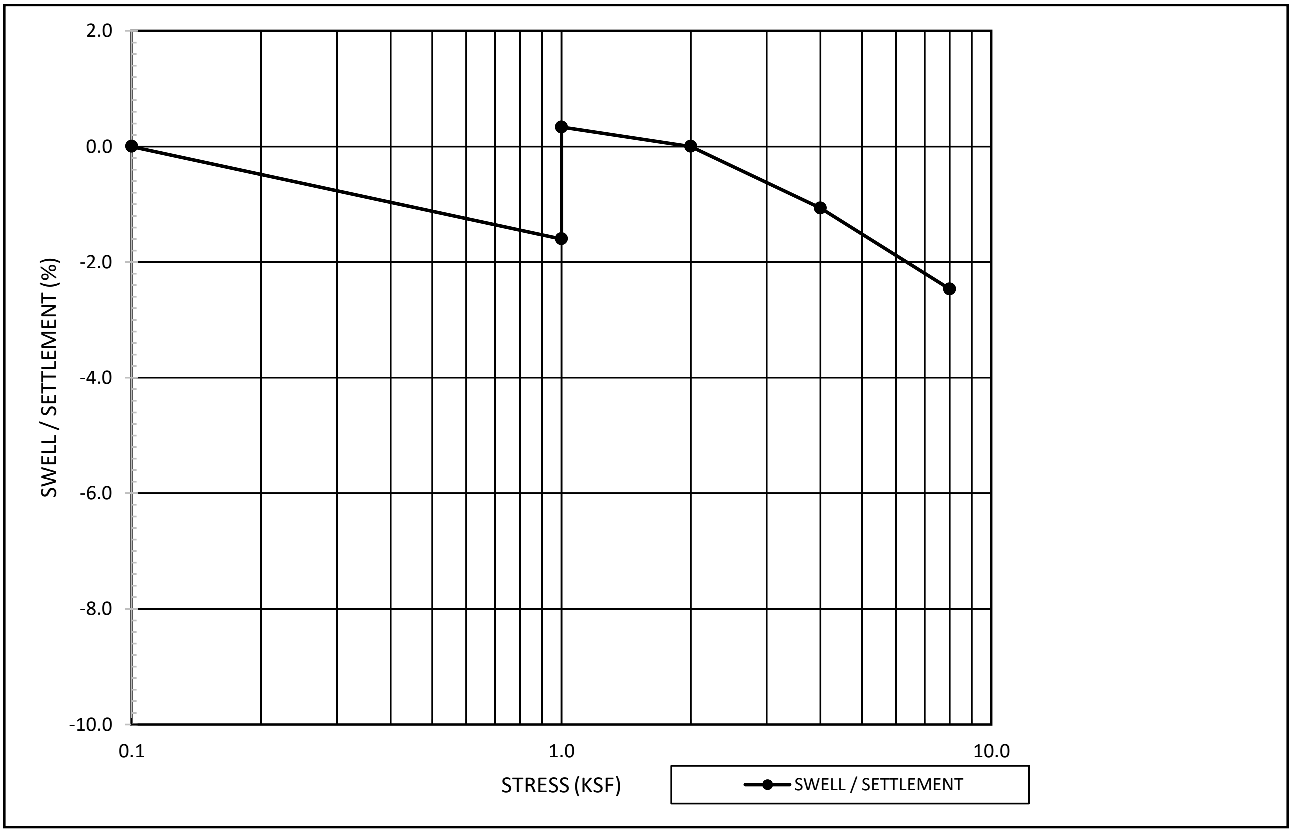
PROJECT LOCATION Calhan, Colorado



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-2**	**19.0**	**WELL-GRADED SAND with SILT(SW-SM)**	**NP**	**NP**	**NP**	**1.73**	**6.48**
⊠	**P-1**	**0.5**	**CLAYEY SAND(SC)**	**28**	**14**	**14**		
▲	**P-2**	**0.5**	**CLAYEY SAND(SC)**	**33**	**15**	**18**		

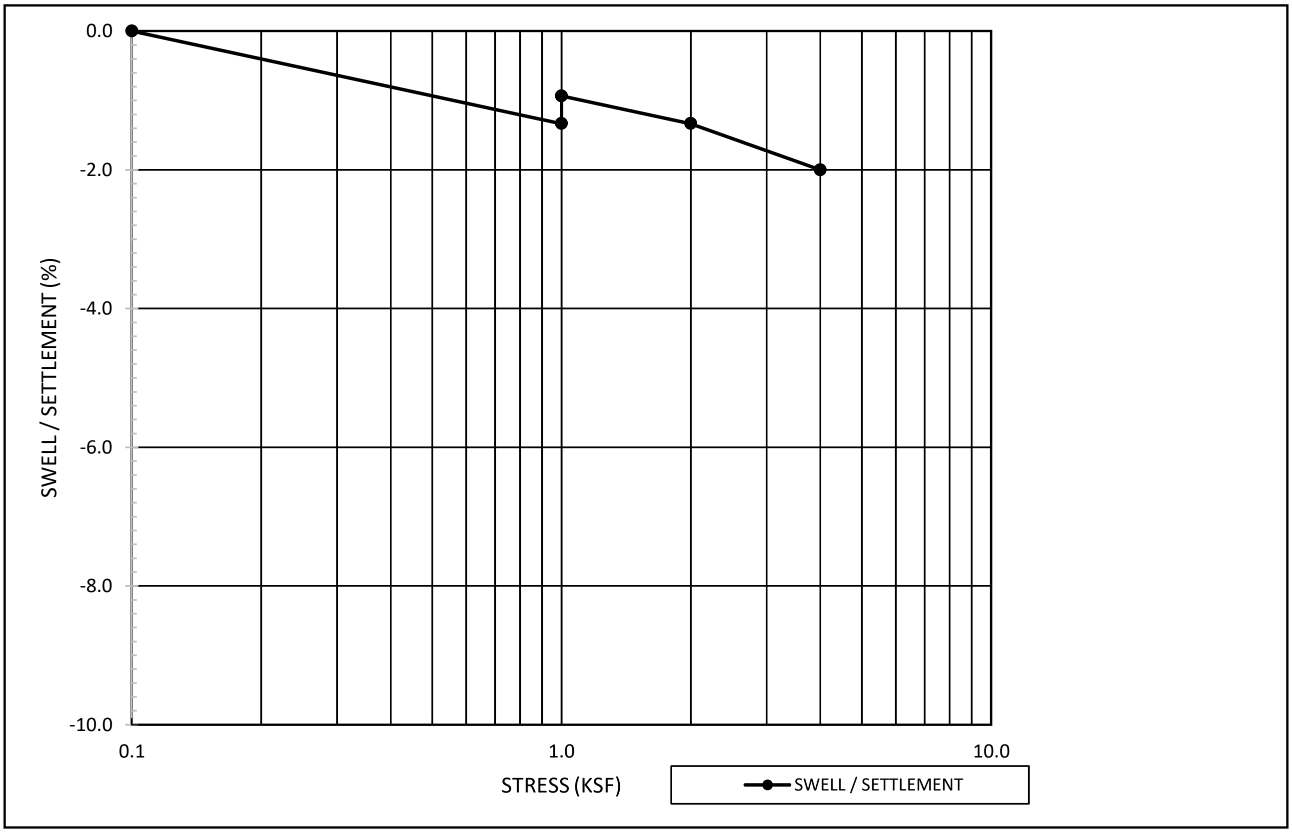
	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-2**	**19.0**	**12.5**	**0.409**	**0.211**		**2.0**	**86.0**	**12.0**	
⊠	**P-1**	**0.5**	**4.75**	**0.159**			**0.0**	**55.0**	**45.0**	
▲	**P-2**	**0.5**	**9.5**	**0.15**			**1.0**	**51.0**	**48.0**	

Project Name:	Calhan Highway Bridge	**Date**	10/30/2020
Project No.:	D20-2-333		
Boring ID.:	B-1	**Sample Depth (ft)**	19
Sample Description:	WEATHERED CLAYSTONE, LIGHT GRAY TO LIGHT BROWN, MOIST		
		Swell @ Wetting Weight:	1.9 %



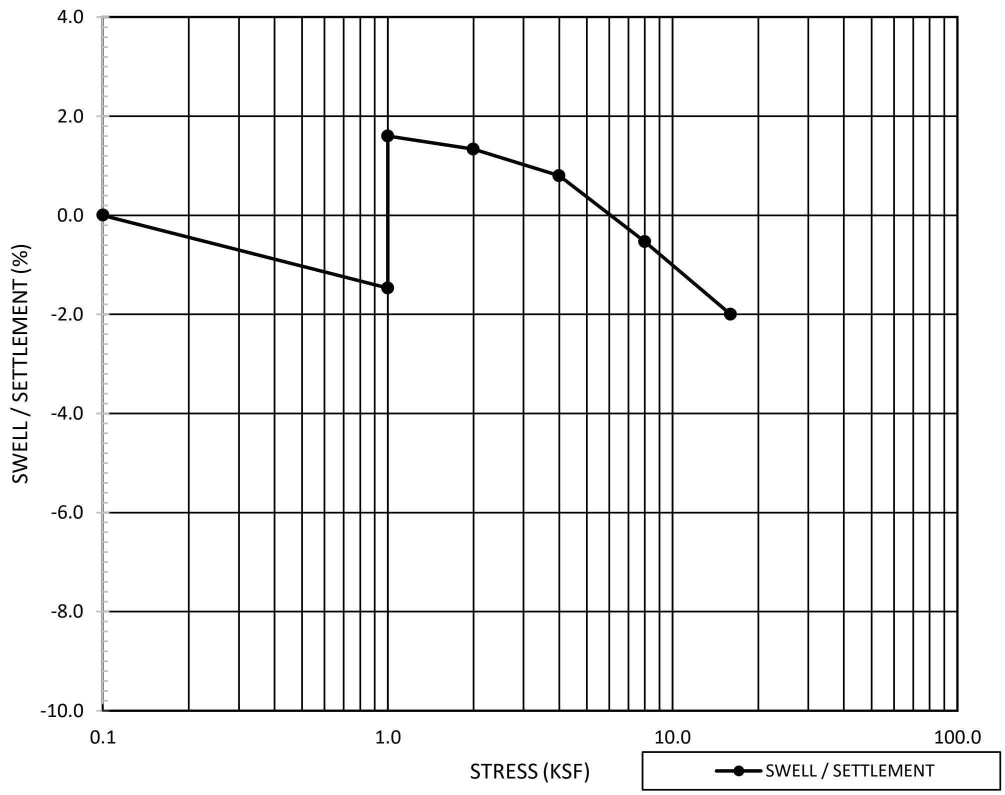
Initial Condition	
Moisture Content %	28.6
Dry Density (pcf)	95.4
Post-Swell Condition	
Moisture Content %	31.2

Project Name:	Calhan Highway Bridge	**Date**	10/30/2020
Project No.:	D20-2-333		
Boring ID.:	B-2	**Sample Depth (ft)**	29
Sample Description:	WEATHERED CLAYSTONE, LIGHT GRAY TO BROWN, MOIST		
		Swell @ Wetting Weight:	0.4 %



Initial Condition	
Moisture Content %	31.9
Dry Density (pcf)	86.7
Post-Swell Condition	
Moisture Content %	35.1

Project Name:	Calhan Highway Bridge	**Date**	10/30/2020
Project No.:	D20-2-333		
Boring ID.:	B-2	**Sample Depth (ft)**	39
Sample Description:	CLAYSTONE, LIGHT GRAY, MOIST		
		Swell @ Wetting Weight:	3.1 %

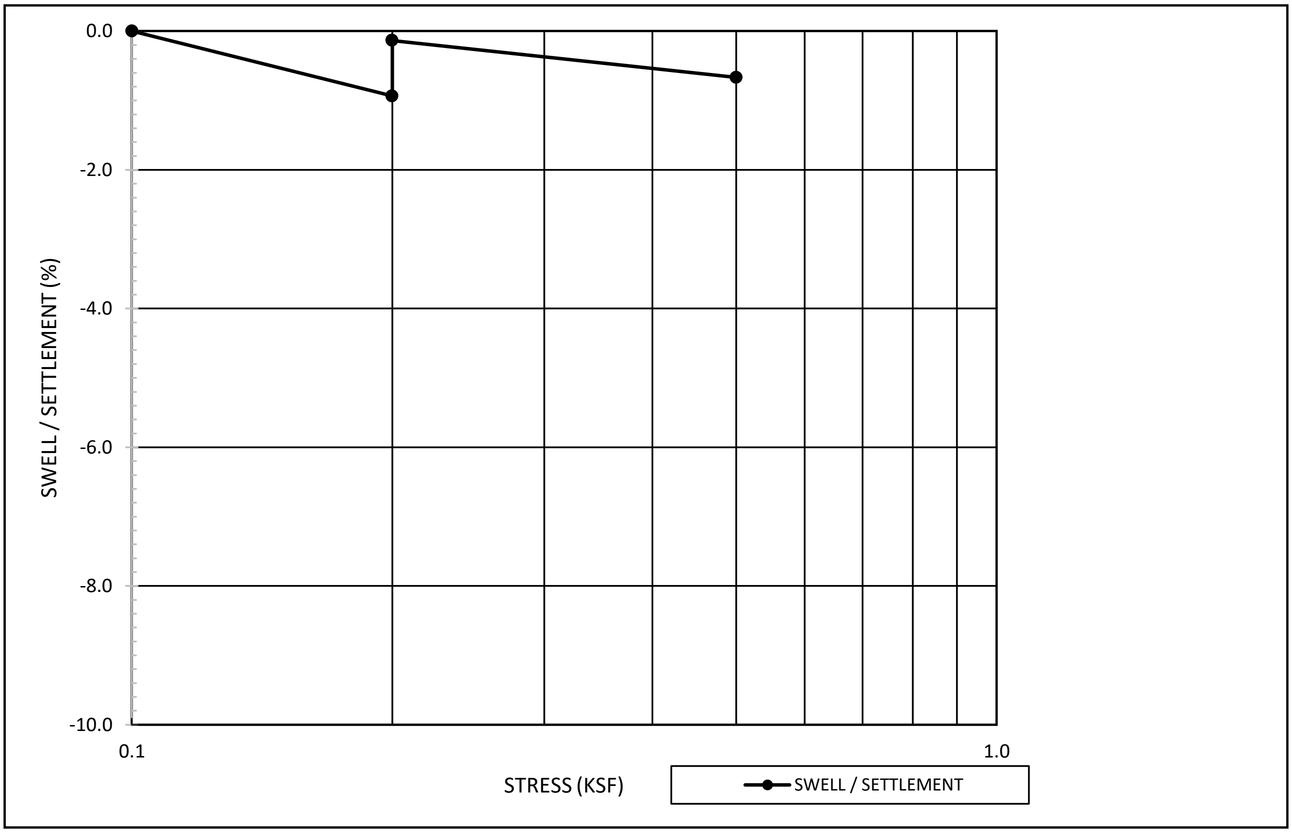


Initial Condition	
Moisture Content %	21.5
Dry Density (pcf)	106.9
Post-Swell Condition	
Moisture Content %	23.6

VIVID Engineering Group, Inc.

Project Name:	Calhan Highway Bridge	**Date**	10/30/2020
Project No.:	D20-2-333		
Boring ID.:	P-1	**Sample Depth (ft)**	4
Sample Description:	FILL: CLAYEY SAND, MOIST, DARK BROWN TO LIGHT BROWN		

Swell @ Wetting Weight: **0.8** %

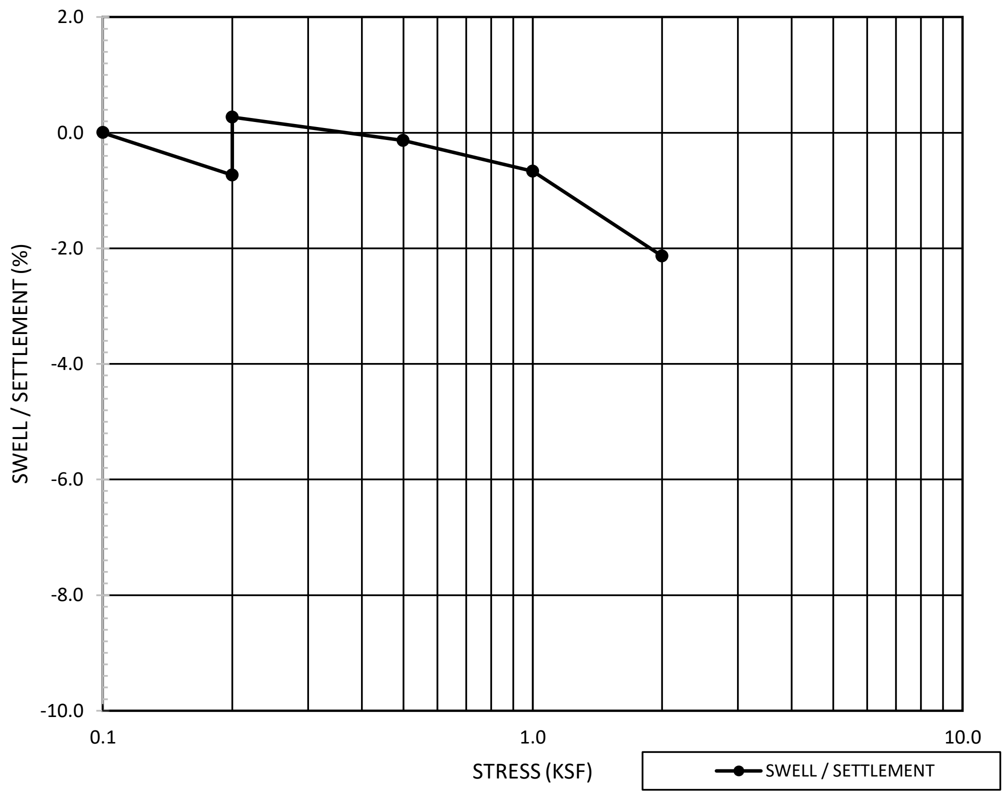


Initial Condition	
Moisture Content %	**16.2**
Dry Density (pcf)	**106.8**
Post-Swell Condition	
Moisture Content %	**18.9**

VIVID Engineering Group, Inc.

Project Name:	Calhan Highway Bridge	**Date**	10/28/2020
Project No.:	D20-2-333		
Boring ID.:	P-2	**Sample Depth (ft)**	1
Sample Description:	FILL: CLAYEY SAND, DARK BROWN, MOIST		

	%
Swell @ Wetting Weight:	**1.0**



Initial Condition	
Moisture Content %	**22.7**
Dry Density (pcf)	**96.1**
Post-Swell Condition	
Moisture Content %	**25.4**

UNCONFINED COMPRESSION TEST
ASTM D 2166

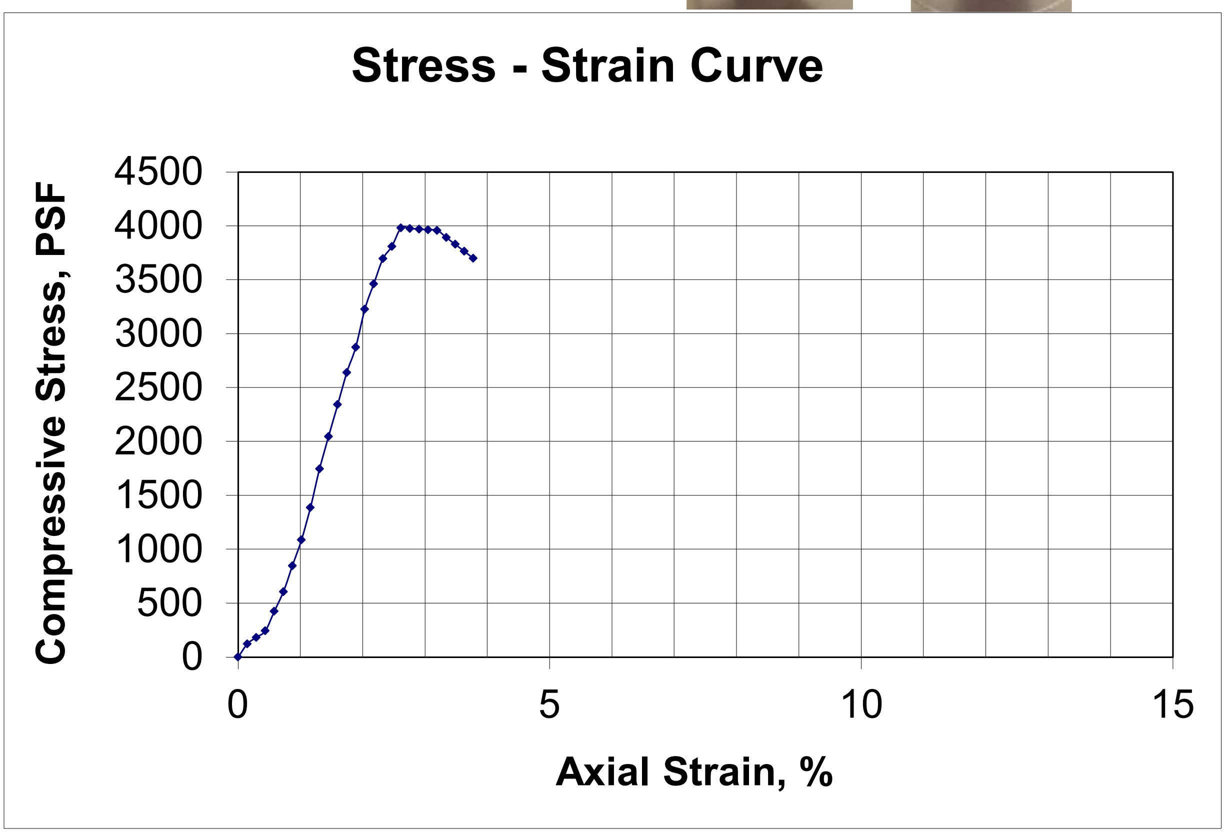
PROJECT NAME:	Calhan Highway Bridge	PROJECT ENG.:	MBR
PROJECT NO. :	D20-2-333	DATE RECEIVED:	10/19/2020
CLIENT NAME:	AECOM	DATE TESTED:	10/19/2020
		TESTED BY:	TK
BORING NO. :	B-1	DATA ENTRY:	TK
SAMPLE NO.:	N/A		
DEPTH, FT. :	28ft 9in to 29ft 6in	DESCRIPTION:	CLAYSTONE, light gray
TEST SPECIMEN NO.:	0		

INITIAL DATA

Avg. Height, In.:	3.443
Avg. Diameter, In.:	1.733
L/D Ratio:	2.0
Moisture Content, %: (Sample, After test)	31.3
Dry Density, pcf:	90.3
Assumed Specific Gravity:	2.7
Saturation, %:	97.5
Void Ratio:	0.867
Rate of Strain, %/Minute:	1.0

Photo:


	PSF	PSI
Compressive Strength @ Failure:	3982	28
Shear Strength @ Failure:	1991	14
Axial Strain @ Failure,%:	2.6	2.6



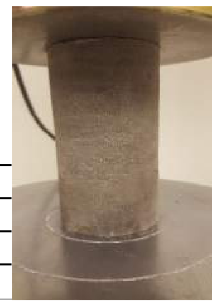
UNCONFINED COMPRESSION TEST
ASTM D 2166

PROJECT NAME:	Calhan Highway Bridge	PROJECT ENG.:	MBR
PROJECT NO. :	D20-2-333	DATE RECEIVED:	10/19/2020
CLIENT NAME:	AECOM	DATE TESTED:	10/19/2020
		TESTED BY:	TK
BORING NO. :	B-1	DATA ENTRY:	TK
SAMPLE NO.:	N/A		
DEPTH, FT. :	33ft 0in to 33ft 8in	DESCRIPTION:	SANDSTONE, gray
TEST SPECIMEN NO.:	0		

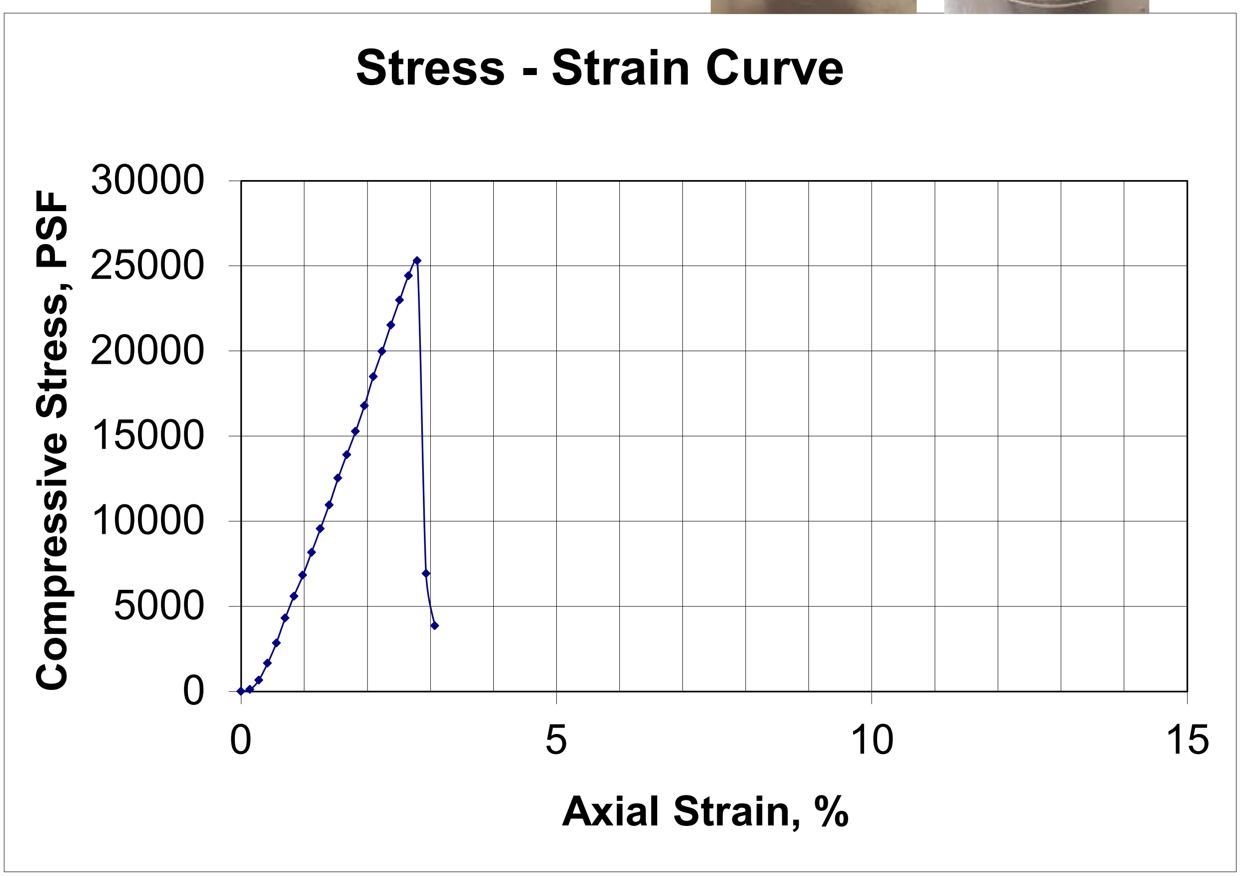
INITIAL DATA

Avg. Height, In.:	3.580
Avg. Diameter, In.:	1.757
L/D Ratio:	2.0
Moisture Content, %: (Sample, After test)	20.5
Dry Density, pcf:	108.3
Assumed Specific Gravity:	2.7
Saturation, %:	99.7
Void Ratio:	0.556
Rate of Strain, %/Minute:	1.0

Photo:




	PSF	PSI
Compressive Strength @ Failure:	25297	176
Shear Strength @ Failure:	12648	88
Axial Strain @ Failure,%:	2.8	2.8



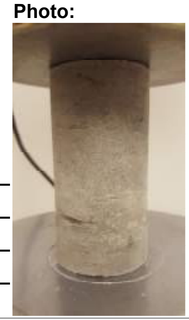
UNCONFINED COMPRESSION TEST
ASTM D 2166

PROJECT NAME:	Calhan Highway Bridge	PROJECT ENG.:	MBR
PROJECT NO. :	D20-2-333	DATE RECEIVED:	10/19/2020
CLIENT NAME:	AECOM	DATE TESTED:	10/19/2020
		TESTED BY:	TK
BORING NO. :	B-1	DATA ENTRY:	TK
SAMPLE NO.:	N/A		
DEPTH, FT. :	38ft 11in to 39ft 7in	DESCRIPTION:	SANDSTONE, light gray
TEST SPECIMEN NO.:	0		

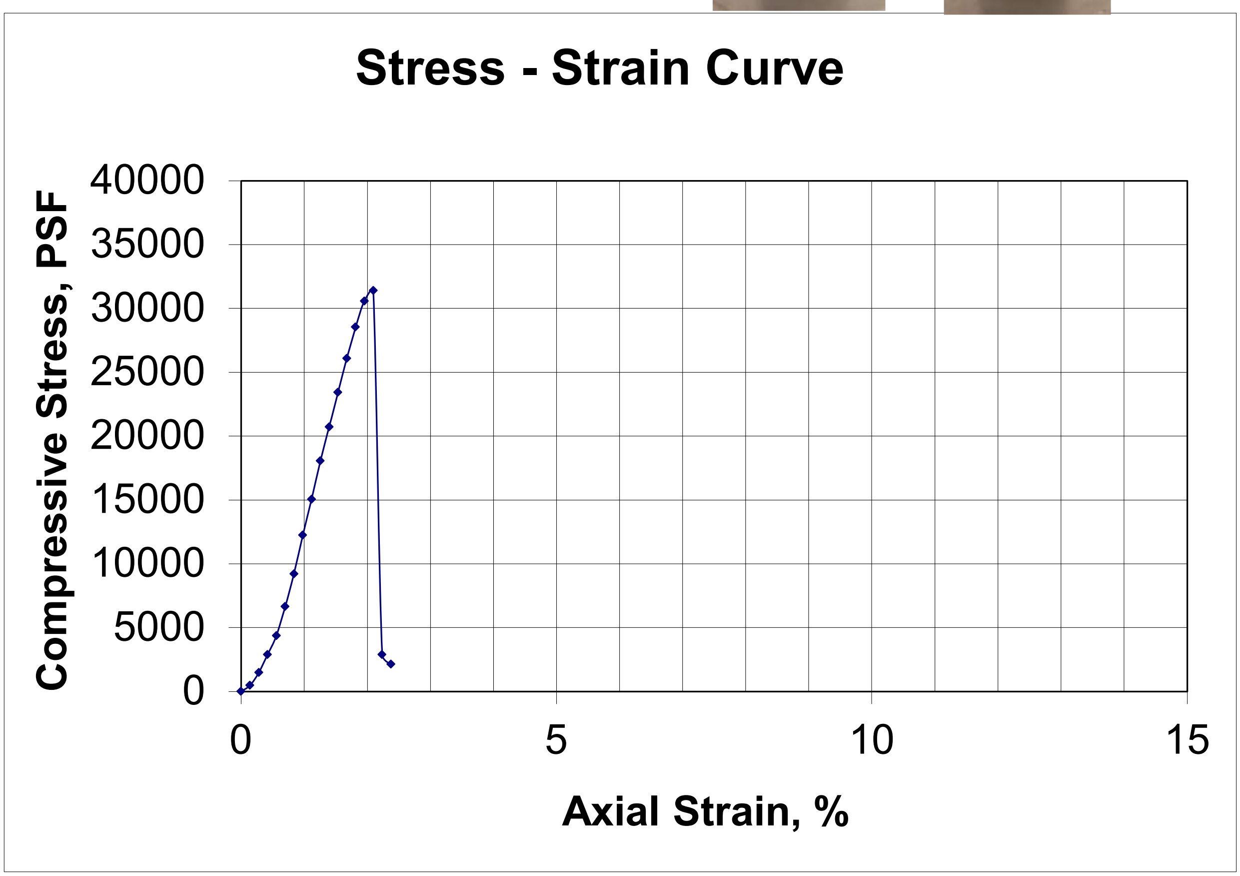
INITIAL DATA

Avg. Height, In.:	3.580
Avg. Diameter, In.:	1.760
L/D Ratio:	2.0
Moisture Content, %: (Sample, After test)	21.2
Dry Density, pcf:	106.4
Assumed Specific Gravity:	2.7
Saturation, %:	98.1
Void Ratio:	0.584
Rate of Strain, %/Minute:	1.0

Photo:



	PSF	PSI
Compressive Strength @ Failure:	31409	218
Shear Strength @ Failure:	15704	109
Axial Strain @ Failure,%:	2.1	2.1



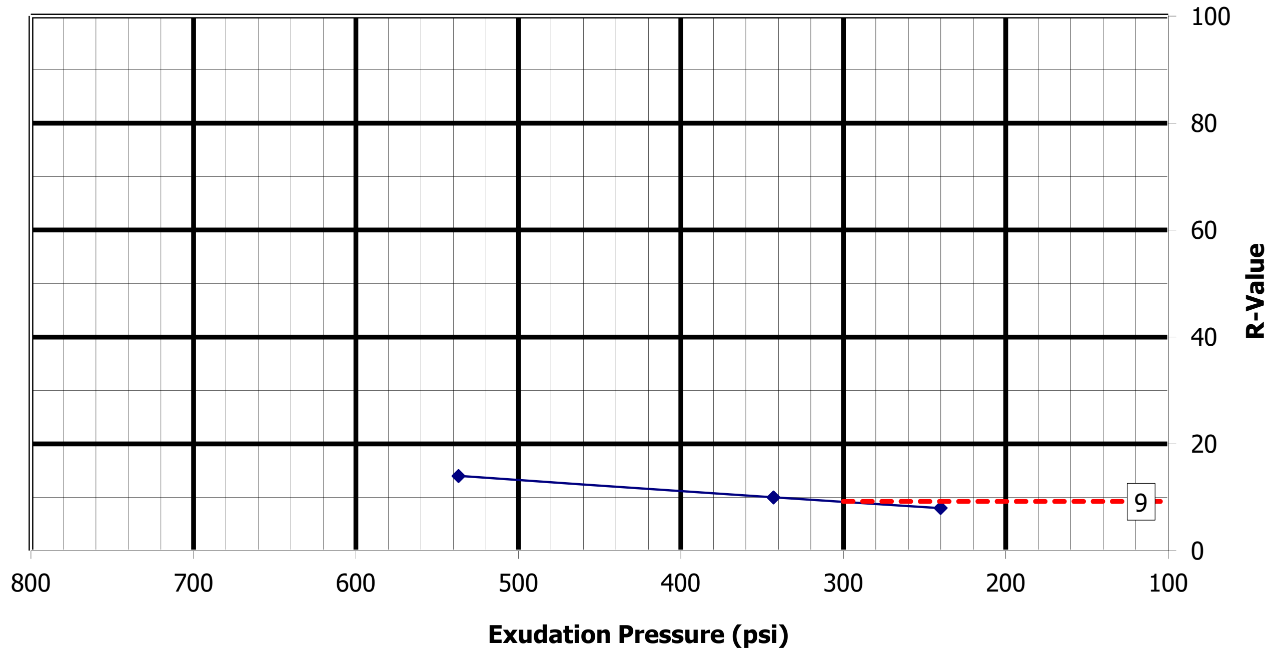
VIVID Engineering Group

CESARE, INC.
Geotechnical Engineers & Construction Materials Consultants

AASHTO ACCREDITED

R-VALUE TEST GRAPH (ASTM D2844)

Project Number:	20.019, Vivid Engineering Group	Date:	5-Nov-20
Project Name:	Calhan Highway Bridge (Vivid Project No. D20-2-333)	Technician:	J. Weinerth
Lab ID Number:	2021290	Reviewer:	G. Hoyos
Sample Location:	Composite No. 1: P-1 and P-2 at 0' to 4'		
Visual Description:	CLAY, sandy, brown		



R-Value @ Exudation Pressure 300 psi:	**9**
Specification:	

Test Specimen:	1	2	3
Moisture Content, %:	13.2	14.1	15.5
Expansion Pressure, psi:	0.15	0.03	-0.12
Dry Density, pcf:	121.6	118.3	112.7
R-Value:	14	10	8
Exudation Pressure, psi:	537	343	240

Corporate: 7108 South Alton Way, Building B • Centennial, Colorado 80112
Phone 303-220-0300 • www.cesareinc.com

Appendix C

Analytical Laboratory Test Results

Colorado Analytical
LABORATORIES, INC.

Analytical Results

TASK NO: 201009060

Vivid Engineering Group, Inc.
1053 Elkton Drive
Colorado Springs CO 80907

Task No.: 201009060
Client PO:
Client Project: Calhan Highway Bridge D20-2-333

Date Received: 10/9/20
Date Reported: 10/16/20
Matrix: Soil - Geotech

Customer Sample ID B-2 @ 10 Ft.
Lab Number: 201009060-01

Test	Result	Method
Chloride - Water Soluble	**0.0178 %**	AASHTO T291-91/ ASTM D4327
pH	**4.7 units**	AASHTO T289-91
Redox Potential	**350.1 mv**	ASTM D1498
Resistivity	**1302 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.004 %**	CDOT CP-L 2103 / ASTM D4327
Sulfide	**Negative**	AWWA C105

Customer Sample ID B-2 @ 34 Ft.
Lab Number: 201009060-02

Test	Result	Method
Chloride - Water Soluble	**0.0004 %**	AASHTO T291-91/ ASTM D4327
pH	**6.6 units**	AASHTO T289-91
Redox Potential	**389.4 mv**	ASTM D1498
Resistivity	**741 ohm.cm**	AASHTO T288-91
Sulfate - Water Soluble	**0.002 %**	CDOT CP-L 2103 / ASTM D4327
Sulfide	**Trace**	AWWA C105

Abbreviations/ References:

AASHTO - American Association of State Highway and Transportation Officials.
ASTM - American Society for Testing and Materials.
ASA - American Society of Agronomy.
DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.

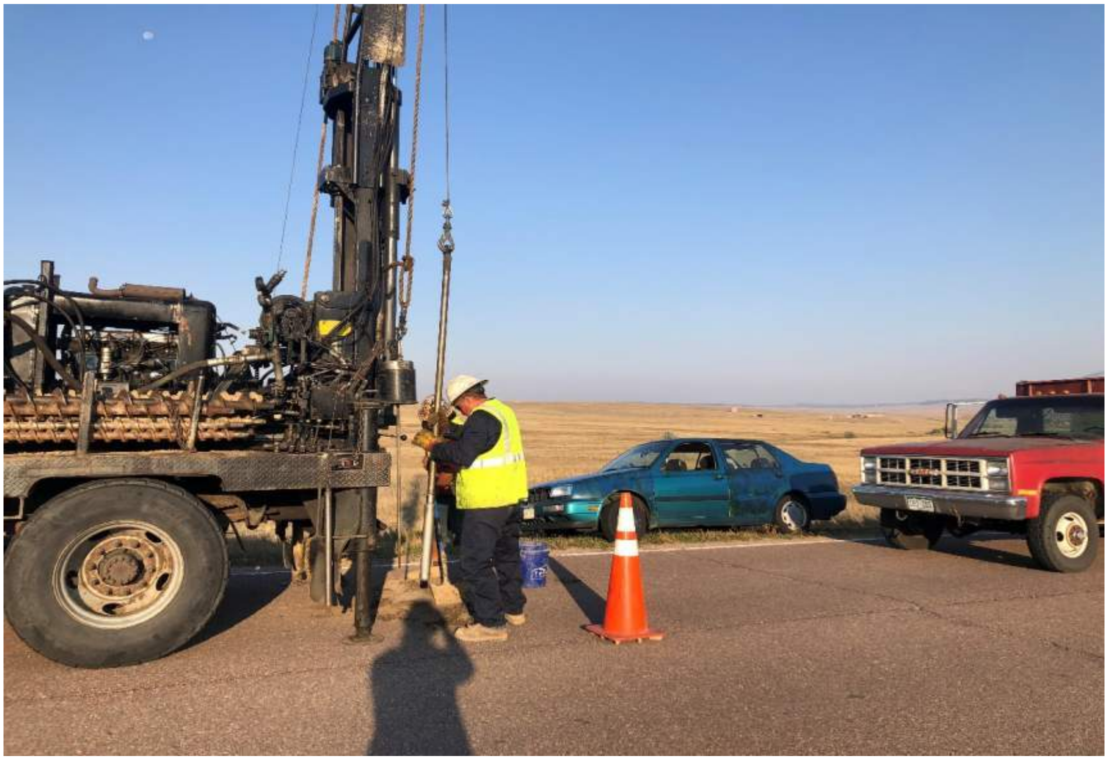
Steve Nielson

DATA APPROVED FOR RELEASE BY

10411 Heinz Way / Commerce City, CO 80640 / 303-659-2313
Mailing Address: P.O. Box 507 / Brighton, CO 80601-0507

Appendix D

Site and Rock Core Photos



DRILLING BORING P-1 - LOOKING WEST

DRILLING BORING B-1 - LOOKING NORTHEAST

VIVID Engineering Group	Project No: D20-2-333	SITE PHOTOS	FIGURE
	Date: 9/16/2020	Proposed Calhan Highway Bridge Replacement	D-1
	Drawn by: BTM	South of Paint Mine Road	
	Reviewed by: WJB	El Paso County, Colorado	

EXISTING BRIDGE ABUTMENT NEAR BORING B-2 - LOOKING NORTHWEST

EXISTING BRIDGE FOUNDATION ELEMENT IN DRAINAGE CHANNEL

VIVID Engineering Group	Project No: D20-2-333	SITE PHOTOS	FIGURE
	Date: 9/16/2020	Proposed Calhan Highway Bridge Replacement	D-2
	Drawn by: BTM	South of Paint Mine Road	
	Reviewed by: WJB	El Paso County, Colorado	

BORING B-1 - 25' TO 30'

BORING B-1 - 30' TO 35'

VIVID Engineering Group	Project No:	D20-2-333	**ROCK CORE PHOTOS**	FIGURE
	Date:	9/16/2020	Proposed Calhan Highway Bridge Replacement	**D-3**
	Drawn by:	BTM	South of Paint Mine Road	
	Reviewed by:	WJB	El Paso County, Colorado	

BORING B-1 - 35' TO 40'

Vivid Engineering Group	Project No: D20-2-333	**ROCK CORE PHOTOS**	FIGURE
	Date: 9/16/2020	Proposed Calhan Highway Bridge Replacement	**D-4**
	Drawn by: BTM	South of Paint Mine Road	
	Reviewed by: WJB	El Paso County, Colorado	

Appendix E

Pavement Thickness Design Calculations

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Vivid Engineering Group, Inc.
1053 Elkton Drive
Colorado Springs, Colorado 80907
USA

Flexible Structural Design Module

Calhan Highway
R-value = 9
Mr = 3,448
Rural Minor Arterial
ESAL = 689,850

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	689,850
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	80 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	3,448 psi
Stage Construction	1
Calculated Design Structural Number	3.90 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	New HMA	0.44	1	7	-	3.08
2	New ABC	0.11	0.9	8.5	-	0.84
Total	-	-	-	15.50	-	3.92

Appendix F

Important Information About This Geotechnical Engineering Report

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org