Geotechnical Engineering Report

Saint Vrain Bridge Replacement

Approximately 2 miles Southwest of the Intersection of Highway 36 and Highway 7

Lyons, Colorado

January 20, 2016

Terracon Project No. 21155048

PRELIMINARY DRAFT
NOT FOR CONSTRUCTION

Prepared for:

J-U-B Engineers, Inc.
Fort Collins, Colorado

Prepared by:

Terracon Consultants, Inc.
Fort Collins, Colorado

January 20, 2016

J-U-B Engineers, Inc.
4745 Boardwalk Drive, Building D
Fort Collins, Colorado 80525

Attn: Mr. Troy Campbell
Project Manager
P: (970) 377-3602
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Re: Geotechnical Engineering Report
Saint Vrain Bridge Replacement
Approximately 2 Miles Southwest of the Intersection of Highway 36 and Highway 7
Lyons, Colorado
Terracon Project No. 21155048

Dear Mr. Campbell:

Terracon Consultants, Inc. (Terracon) has completed the geotechnical engineering services for the project referenced above. These services were performed in general accordance with our Proposal No. P20150069 and signed Agreement for Subconsultant Services dated September 25, 2015. This geotechnical engineering report presents the results of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations and pavements for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

Bryce C. Reeves, E.I.
Geotechnical Engineer

Eric D. Bernhardt, P.E.
Geotechnical Department Manager

Reviewed by: Mathew B. Fielding, P.E., Denver Office Manager

Enclosures

Copies to: Addressee (via e-mail)

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

EXECUTIVE SUMMARY

A geotechnical exploration has been performed for the proposed Saint Vrain Bridge Replacement to be constructed approximately 2 miles southwest of the intersection of Highway 36 and Highway 7 in Lyons, Colorado. Five borings, with boring logs presented as Exhibits A-5 through A-9 and designated as Boring No. 1 through Boring No. 5, were performed to depths of approximately 5 to 85 feet below existing site grades. This report specifically addresses the recommendations for the proposed replacement bridge and pavements.

Based on the information obtained from our subsurface exploration, the proposed bridge can be supported on deep foundations. However, the following geotechnical considerations were identified and will need to be considered:

- The proposed bridge may be supported on a drilled pier foundation system bottomed in bedrock. As an alternative, the proposed bridge may be supported by pre-drilled H-piles driven to practical refusal. The presence of groundwater and granular soils (including cobbles and boulders) overlying the bedrock will require special considerations during construction.

- Based on the AASHTO LRFD Bridge Design Specifications Manual, Version 7 (2014) (hereafter referred to as AASHTO), the seismic site classification for this site is C.

- Existing fill was encountered in the borings performed on this site to depths of about 3 feet below existing site grades. We believe the existing fill below pavements was placed as aggregate base course and the fill in the area of the roadway reconstruction was likely placed during the construction of the temporary roadway. We recommend the proposed pavements do not bear directly on the existing fill materials unless such material was placed and compacted per CDOT requirements. If the owner elects to not replace the fill, at a minimum we recommend performing a proofroll test on the existing fill materials to identify areas for potentially inadequate pavement support before placing additional fill or aggregate base course. Any areas identified as soft and/or unstable will need to be removed and replaced with engineered fill.

- Close monitoring of the construction operations discussed herein will be critical in achieving the design subgrade support. We therefore recommend that Terracon be retained to monitor this portion of the work.

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled **GENERAL COMMENTS** should be read for an understanding of the report limitations.

GEOTECHNICAL ENGINEERING REPORT
Saint Vrain Bridge Replacement
Approximately 2 Miles Southwest of the Intersection of Highway 36 and Highway 7
Lyons, Colorado
Terracon Project No. 21155048
January 20, 2016

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering services performed for the proposed Saint Vrain Bridge replacement to be located approximately 2 miles southwest of the intersection of Highway 36 and Highway 7 in Lyons, Colorado (Exhibit A-1). The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- subsurface soil and bedrock conditions
- groundwater conditions
- grading and drainage
- lateral earth pressures
- seismic considerations
- foundation design and construction
- pavement construction
- earthwork

Our geotechnical engineering scope of work for this project included the initial site visit, the advancement of five test borings to depths ranging from approximately 5 to 85 feet below existing site grades, laboratory testing for soil engineering properties and engineering analyses to provide foundation and pavement design and construction recommendations.

Logs of the borings along with Exploration Plans (Exhibits A-2 and A-3) are included in Appendix A. The results of the laboratory testing performed on soil and bedrock samples obtained from the site during the field exploration are included in Appendix B.

2.0 PROJECT INFORMATION

2.1 Project Description

Item	Description
Site layout	Refer to the Exploration Plans (Exhibits A-2 and A-3 in Appendix A)

Item	Description
Proposed construction	We understand the bridge replacement will include a single or double-span bridge with associated abutments and short wing walls, estimated to be about 20 feet or less in length. We understand new bridge approach alignments with new pavements will also be a part of the proposed construction. Approximately 300 feet of asphalt roadway will also be reconstructed along Old Saint Vrain Road. The proposed bridge will span the South Saint Vrain Creek and connect Old Saint Vrain Road with South Saint Vrain Drive.
Anticipated maximum factored loads (provided by J-U-B)	Option 1 double-span (precast box girders): Abutment 1: 3,534 kips Pier 2: 6,436 kips Abutment 3: 3,534 kips Option 2 double-span (wide flange steel girders) Abutment 1: 1,271 kips Pier 2: 1,911 kips Abutment 3: 1,271 kips
Grading	We anticipate minor grading of about 2 feet or less will be required for the bridge approaches and the roadway reconstruction. Deeper cuts and fills on the order of 10 feet may be required for the construction of the proposed retaining structures at the abutments.
Traffic loading (provided by J-U-B)	Design equivalent single-axle loads (ESAL's): Old Saint Vrain Road: 23,300 Proposed bridge approaches: 852,000

2.2 Site Location and Description

Item	Description
Location	The project site is located approximately 2 miles southwest of the intersection of Highway 36 and Highway 7 in Lyons, Colorado. (40.206929°, -105.293967°)
Existing site features	The previous bridge was destroyed by a flood in 2013. The majority of the bridge approaches are in place. An approximately 300 foot long section of Old Saint Vrain Road was also destroyed in a flood in 2013 and is bordered by asphalt sections on the east and west sides of the alignment. South Saint Vrain Drive (Highway 7) is located on the northern side of the bridge and Old Saint Vrain Road is located on the south side of the bridge.
Surrounding developments	An existing aggregate mine is located southwest of the proposed bridge, it is our understanding the aggregate mine may be closed in the near future. Private residences are located to the north, south, east, and west of the proposed bridge replacement.

Item	Description
Current ground cover	The ground is covered with asphalt pavement, native grasses and weeds, bare ground, and aggregate roadway surfacing.
Existing topography	Outside of the streambed the site is relatively flat. The streambed is approximately 6 feet lower than the pavement elevation.

3.0 SUBSURFACE CONDITIONS

3.1 Typical Subsurface Profile

Specific conditions encountered at each boring location are indicated on the individual boring logs included in Appendix A. Stratification boundaries on the boring logs represent the approximate location of changes in soil types; in-situ, the transition between materials may be gradual. Based on the results of the borings, subsurface conditions on the project site can be generalized as follows:

Material Description	Approximate Depth to Bottom of Stratum (feet)	Consistency/Density/Hardness
Asphalt pavement	About 6 inches thick.	--
Fill materials consisting of sand and crushed gravel (below existing pavements)	About 6 to 8 inches thick.	--
Fill materials consisting of silt, sand, gravel, and cobbles	About 3 feet below existing site grades in Boring Nos. 4, and 5 only.	--
Poorly graded gravel with silt, sand, cobbles, and boulders	About 23 to 53 feet below existing site grades.	Medium dense to very dense
Sandstone bedrock	To the maximum depth of exploration of about 85 feet.	Moderately hard to hard

3.2 Laboratory Testing

Selected soil and rock samples were tested to evaluate physical and engineering properties. Laboratory test results are presented in Appendix B.

3.3 Corrosion Protection (Water-Soluble Sulfates)

Results of water-soluble sulfate testing indicate that ASTM Type I portland cement can be specified for all project concrete on and below grade. Foundation concrete can be designed for Class 0 sulfate exposure in accordance with the provisions of the ACI Design Manual Section 318, Chapter 4 and ASTM C 150.

3.4 Groundwater

The boreholes were observed while drilling and after completion for the presence and level of groundwater. The water levels observed in the boreholes are noted on the attached boring logs, and are summarized below:

Boring Number	Depth to groundwater while drilling, ft.
1	10
2	10
3	10
4	Not encountered
5	Not encountered

These observations represent groundwater conditions at the time of the field exploration, and may not be indicative of other times or at other locations. Groundwater level fluctuations occur due to seasonal variations in the water levels present in the South Saint Vrain Creek, amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the bridge and pavements may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

4.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

4.1 Geotechnical Considerations

Based on subsurface conditions encountered in the borings, the site appears suitable for the proposed construction from a geotechnical point of view provided certain precautions and design and construction recommendations described in this report are followed. We have identified geotechnical conditions that could impact design and construction of the proposed bridge, pavements, and other site improvements.

4.1.1 Foundation Recommendations

The proposed bridge may be supported on a drilled pier foundation system bottomed in bedrock. As an alternative, the proposed bridge may be supported on a pre-drilled H-pile foundation system driven to practical refusal.

Large cobbles and boulders were encountered during our subsurface investigation. The large cobbles and boulders will create difficult drilling and pile driving conditions. The contractor should be prepared to install foundations in these conditions and contract documents should clearly identify the presence of cobbles and boulders.

4.1.2 Existing, Undocumented Fill

As previously noted, existing undocumented fill was encountered in the borings performed on this site to depths of about 3 feet below existing site grades. We do not possess any information regarding whether the fill was placed under the observation of a geotechnical engineer or if it met CDOT requirements. However, we believe the fill encountered directly below the existing asphalt pavements was placed as aggregate base and the fill encountered in Boring Nos. 4 and 5 was placed as a temporary roadway section for Old Saint Vrain Road that was destroyed in the flood.

Support of pavements on or above existing fill soils is discussed in this report. There is an inherent risk for the owner that compressible fill or unsuitable material within or buried by the fill will not be discovered. This risk of unforeseen conditions cannot be eliminated without completely removing the existing fill, but can be reduced by performing additional testing and evaluation. If the owner elects not to remove the fill, at a minimum we recommend performing a proof roll test on the existing fill materials to identify any areas for potentially inadequate pavement support.

4.2 Earthwork

The following presents recommendations for site preparation, excavation, subgrade preparation and placement of engineered fills on the project. All earthwork on the project should be observed and evaluated by Terracon on a full-time basis. The evaluation of earthwork should include observation of over-excavation operations, testing of engineered fills, subgrade preparation, subgrade stabilization, and other geotechnical conditions exposed during the construction of the project.

4.2.1 Site Preparation

Prior to placing any fill, strip and remove existing vegetation and any other deleterious materials from the proposed construction areas.

Stripped organic materials should be wasted from the site or used to re-vegetate exposed slopes (if any) after completion of grading operations. Prior to the placement of fills, the site should be graded to create a relatively level surface to receive fill, and to provide for a relatively uniform thickness of fill beneath proposed pavements.

If fill is placed in areas of the site where existing slopes are steeper than 4:1 (horizontal:vertical), the area should be benched to reduce the potential for slippage between existing slopes and fills. Benches should be wide enough to accommodate compaction and earth moving equipment (preferably 8 feet wide), and to allow placement of horizontal lifts of fill.

4.2.2 Demolition

Demolition of the existing bridge foundations should include complete removal of all foundation systems, below-grade structural elements, and pavements within the proposed construction areas. This should include removal of any utilities to be abandoned along with any loose utility trench backfill or loose backfill found adjacent to existing foundations. All materials derived from the demolition of existing foundations and pavements should be removed from the site. The types of foundation systems supporting the previously existing bridge are not known.

4.2.3 Excavation

Large cobbles and boulders will be encountered during excavation activities. Excavations into the on-site soils will encounter possible caving conditions.

The soils to be excavated can vary significantly across the site as their classifications are based solely on the materials encountered in widely-spaced exploratory test borings. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, the actual conditions should be evaluated to determine any excavation modifications necessary to maintain safe conditions.

Although evidence of underground facilities such as septic tanks, vaults, and basements were not observed during the site reconnaissance, such features could be encountered during construction. If unexpected fills or underground facilities are encountered, such features should be removed and the excavation thoroughly cleaned prior to backfill placement and/or construction.

Depending upon depth of excavation and seasonal conditions, surface water infiltration and/or groundwater may be encountered in excavations on the site. The contractor should be prepared to dewater excavations to maintain stability of all excavations.

The subgrade soil conditions should be evaluated during the excavation process and the stability of the soils determined at that time by the contractors' Competent Person. Slope inclinations flatter than the OSHA maximum values may have to be used. The individual contractor(s) should be made responsible for designing and constructing stable, temporary excavations as required to maintain stability of both the excavation sides and bottom. All excavations should be sloped or shored in the interest of safety following local, and federal regulations, including current OSHA excavation and trench safety standards. The exposed slope face should be protected against the elements

4.2.4 Subgrade Preparation

After the deleterious materials have been removed from the construction areas, the top 8 inches of the exposed ground surface should be scarified, moisture conditioned, and recompacted to at least 95 percent of the maximum dry unit weight as determined by AASHTO T180 before any new fill, foundation, or pavement is placed.

If pockets of soft, loose, or otherwise unsuitable materials are encountered at the bottom of the excavations, the proposed elevations may be reestablished by over-excavating the unsuitable soils and backfilling with compacted engineered fill.

After the bottom of the excavation has been compacted, engineered fill can be placed to bring the pavement subgrade to the desired grade. Engineered fill should be placed in accordance with the recommendations presented in subsequent sections of this report.

The stability of the subgrade may be affected by precipitation, repetitive construction traffic or other factors. If unstable conditions develop, workability may be improved by scarifying and drying. Alternatively, over-excavation of wet zones and replacement with granular materials may be used, or crushed gravel and/or rock can be tracked or "crowded" into the unstable surface soil until a stable working surface is attained. Lightweight excavation equipment may also be used to reduce subgrade pumping.

4.2.5 Fill Materials and Placement

Abutment and wing wall backfill should consist of granular materials meeting the specifications for CDOT Class I structure backfill.

Fill materials below pavements should meet the material property requirements of CDOT Class 6 Aggregate Base Course.

4.2.6 Compaction Requirements

Engineered fill should be placed and compacted in horizontal lifts, using equipment and procedures that will produce recommended moisture contents and densities throughout the lift.

Item	Description
Fill lift thickness	9 inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used
Minimum compaction requirements	95 percent of the maximum dry unit weight as determined by AASHTO T180
Moisture content cohesionless soil (sand)	As required in Section 203.07 of the Standard Specifications.

1. We recommend engineered fill be tested for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified moisture and compaction requirements are achieved.
2. Specifically, moisture levels should be maintained low enough to allow for satisfactory compaction to be achieved without the fill material pumping when proofrolled.

4.2.7 Grading and Drainage

Erosion protection should be provided at the upstream and downstream ends of the structure, as needed, to prevent erosion of the stream bank and to protect the soils surrounding the foundations from erosion. Surface water collected from road and bridge surfaces should be directed to collection points and discharged beyond the toe of the approach fill slopes to reduce the potential of erosion of the fill slopes. The finished embankment slopes should be properly treated to protect the slopes from the effects of precipitation and rainfall surface flows.

All grades must be adjusted to provide effective drainage away from the proposed bridge and pavements during construction and maintained throughout the life of the proposed project. Infiltration of water into excavations must be prevented during construction. Water permitted to pond near or adjacent to the bridge abutments or pavements (either during or post-construction) can result in significantly higher soil movements than those discussed in this report. As a result, any estimations of potential movement described in this report cannot be relied upon if positive drainage is not obtained and maintained, and water is allowed to infiltrate the fill and/or subgrade.

Exposed ground (if any) should be sloped at a minimum of 10 percent grade for at least 10 feet beyond the perimeter of the proposed bridge wing walls, where possible. The use of swales, chases and/or area drains may be required to facilitate drainage in unpaved areas around the perimeter of the bridge. Backfill against abutments and wing walls should be properly compacted and free of all construction debris to reduce the possibility of moisture infiltration. After construction of the proposed bridge and pavements and prior to project completion, we recommend verification of final grading be performed to document positive drainage, as described above, has been achieved.

Flatwork (if any) and pavements will be subject to post-construction movement. Maximum grades practical should be used for paving and flatwork to prevent areas where water can pond. In addition, allowances in final grades should take into consideration post-construction movement of flatwork, particularly if such movement would be critical. Where paving or flatwork abuts the bridge, care should be taken that joints are properly sealed and maintained to prevent the infiltration of surface water.

4.3 Foundations

The proposed bridge and associated wing walls can be supported by a drilled pier foundation system bottomed in bedrock. As an alternative, the proposed bridge and wing walls may be supported on a pre-drilled H-pile foundation system driven to practical refusal. Design recommendations for foundations for the proposed bridge and related structural elements are presented in the following paragraphs.

4.3.1 Drilled Piers Bottomed in Bedrock - Design Recommendations

Axial loads should be resisted by skin friction on the walls of the rock sockets. The ultimate axial resistance for use in design of the diameter and depths of the drilled shafts is presented in the following tables.

Description	Nominal Resistance	
Nominal Axial Side Resistance[1]	North side abutment:	10 ksf
	South side abutment and center pier:	20 ksf
Minimum bedrock embedment[2]	10 feet	

1. Resistance from any portion of the drilled shaft passing through soils should be neglected. In addition, any portion of embedment into rock where casing has been advanced during construction should be neglected.
2. Drilled shafts should be embedded into firm or harder bedrock materials. This embedment depth is for axial resistance. Shafts may need to be embedded deeper into bedrock for lateral resistance.

Resistance factors for use in design of drilled shafts socketed into rock are presented in the following table.

Method/Soil/Condition	Resistance Factor
Side resistance factor – Strength	0.55
Uplift resistance factor – Strength	0.40
Side resistance factor – Extreme	1.00
Uplift resistance factor – Extreme	0.80

The resistance factor presented for the strength limit state is based on foundation redundancy. If a single shaft is used to support an abutment or pier, the resistance factor presented for the strength limit state should be reduced by 20 percent as required in Section 10.5.5.2.4 of AASHTO.

Drilled piers designed using the parameters above are expected to settle about ½ to 1 inch at the service limit state.

If the center-to-center spacing of drilled shafts is less than 4 diameters, the interaction effects between adjacent shafts shall be evaluated. Adjacent shafts should bear at the same elevation. The capacity of individual piers must be reduced when considering the effects of group action. Capacity reduction is a function of pier spacing and the number of piers within a group. We should be contacted for additional recommendations once the shaft spacing is determined.

If the center-to-center spacing of drilled shafts is less than 6 diameters, the sequence of construction should be specified in the contract documents. Larger spacing may be required to preserve shaft excavation stability or to prevent communication between shafts during excavation and concrete placement.

To satisfy forces in the horizontal direction using LPILE, piers may be designed for the following lateral load criteria:

Parameters	Sand and Gravel	Sand and Gravel	Sandstone Bedrock
LPILE soil type[1]	Sand (above water table)	Sand (submerged)	Stiff clay without free water
Unit weight (pcf)	125	62	130
Average undrained shear strength (psf)	N/A	N/A	9,000
Average angle of internal friction, Φ (degrees)	35	35	N/A
Coefficient of subgrade reaction, k (pci)*	90	60	2,000- static 800 – cyclic
Strain, ε_{50} (%)	N/A	N/A	0.004

Lateral analysis should account for the center to center spacing and P-Y multiplier values per Article 10.7.2.4 of *AASHTO*. A resistance factor of 1.0 should be applied to soil parameters. Lateral resistance analysis using L-Pile© was not included in our original scope for the project. Terracon should be contacted if such an analysis is desired.

4.3.2 Drilled Piers Bottomed in Bedrock - Construction Considerations

Specialized drilling equipment will likely be required for large cobbles/boulders and very hard bedrock layers. In addition, due to caving soils and groundwater steel casing will be required to properly drill the piers prior to concrete placement.

Groundwater should be removed from each pier hole prior to concrete placement. Pier concrete should be placed immediately after completion of drilling and cleaning. A tremie should be used for concrete placement. Free-fall concrete placement in piers will only be acceptable if the concrete is placed in a dry hole and provisions are taken to avoid striking the concrete on the sides of the hole or reinforcing steel, as required in Section 503.07 of the Standard Specifications. The use of a bottom-dump hopper, or an elephant's trunk discharging near the bottom of the hole where concrete segregation will be minimized, is recommended. Due to potential sloughing and raveling, foundation concrete quantities may exceed calculated geometric volumes.

Casing should be withdrawn in a slow continuous manner maintaining a sufficient head of concrete to prevent infiltration of water or caving soils or the creation of voids in pier concrete. Pier concrete should have a relatively high fluidity when placed in cased pier holes or through a tremie. Pier concrete with slump in the range of 5 to 7 inches is recommended.

A representative of Terracon should observe the bearing surface and shaft configuration.

4.3.3 Driven Piles - Design Recommendations

Driven HP 14X117 steel H-piles may designed using a nominal resistance of 550 kips when driven to practical refusal. However, as indicated in AASHTO, the nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions (i.e., ϕ_c = 0.5). We recommend dynamic wave analysis such as CAse Pile Wave Analysis Program (CAPWAP) be used to assess pile capacity on actual test or production piles. If CAPWAP is used for pile load testing, the resistance factor of 0.65 may be used for design of piles. A minimum of 2 piles at the southern abutment, a minimum of 2 piles combined for the center bridge support and northern abutment, and no less than 2 percent of the total production piles must be analyzed using a dynamic wave analysis to use 0.65 as the resistance factor. These piles must be driven and analyzed before production piles are driven.

A cased pilot hole should be used to embed the H-piles the minimum design depth (accounting for both scour and lateral resistance requirements). The minimum diameter pilot hole for a HP 14X117 is 24 inches, a larger diameter pilot hole may be used if needed. Once the H-pile is placed in the cased hole, the annulus should be filled with smooth, rounded, and non-crushed gravel meeting the AASHTO gradation 57. The casing may then be removed and the pile driven to practical refusal as determined by the CAPWAP analyses.

Individual pile settlement should be on the order of 1/2-inch when designed according to the criteria presented in this report.

Piles should be designed to resist lateral loads. To satisfy forces in the horizontal direction using L-PILE, piles may be designed for the lateral load criteria presented below:

Parameters	AASHTO #57	AASHTO #57	Native Sand and Gravel	Sandstone Bedrock
LPILE soil type	Sand (above water table)	Sand (submerged)	Sand (submerged)	Stiff clay without free water
Unit weight (pcf)	90	28	63	130
Average undrained shear strength (psf)	N/A	N/A	N/A	9,000
Average angle of internal friction, Φ (degrees)	32	32	35	N/A
Coefficient of subgrade reaction, k (pci)	25	20	60	2,000- static 800 – cyclic
Strain, ε_{50} (%)	N/A	N/A	N/A	0.004

We understand battered piles will not be used on this project.

Groups of piles required to support concentrated loads will require appropriate reductions of the axial and lateral capacities based on the effective envelope of the pile group. Piles should be spaced at least 30 inches or 2.5 pile diameters center-to-center whichever is greater. The capacity of individual piles must be reduced when considering the effects of group action. Capacity reduction is a function of pile spacing and the number of piles within a group.

The pile driving system should be analyzed using the wave equation to evaluate the potential for overstressing the pile materials during driving. Difficult/severe driving conditions are likely to be encountered due to large cobbles, boulders, and bedrock.

4.3.4 Driven Piles - Construction Considerations

The contractor should select a driving hammer and cushion combination which is capable of installing the selected piling without overstressing the pile material. The contractor should submit the pile driving plan and the pile hammer-cushion combination to the engineer for evaluation of the driving stresses in advance of pile installation.

Some ground heave may be experienced as a result of pile driving at each site. Therefore, it is recommended that the top elevations of the initial piles driven be surveyed. If any heave is noted after the driving of subsequent piles, the piles should be re-driven to their original top elevation. This problem can be particularly acute in pile groups.

The pile hammer should be operated at the manufacturer's recommended stroke when measuring penetration resistance. All piles should be provided with driving shoes to protect the pile tip from damage when penetrating the dense granular soils and seating into bedrock. Terracon should be retained to observe pile driving operations on a full-time basis. Each pile should be observed and checked for buckling, crimping and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations.

4.4 Seismic Considerations

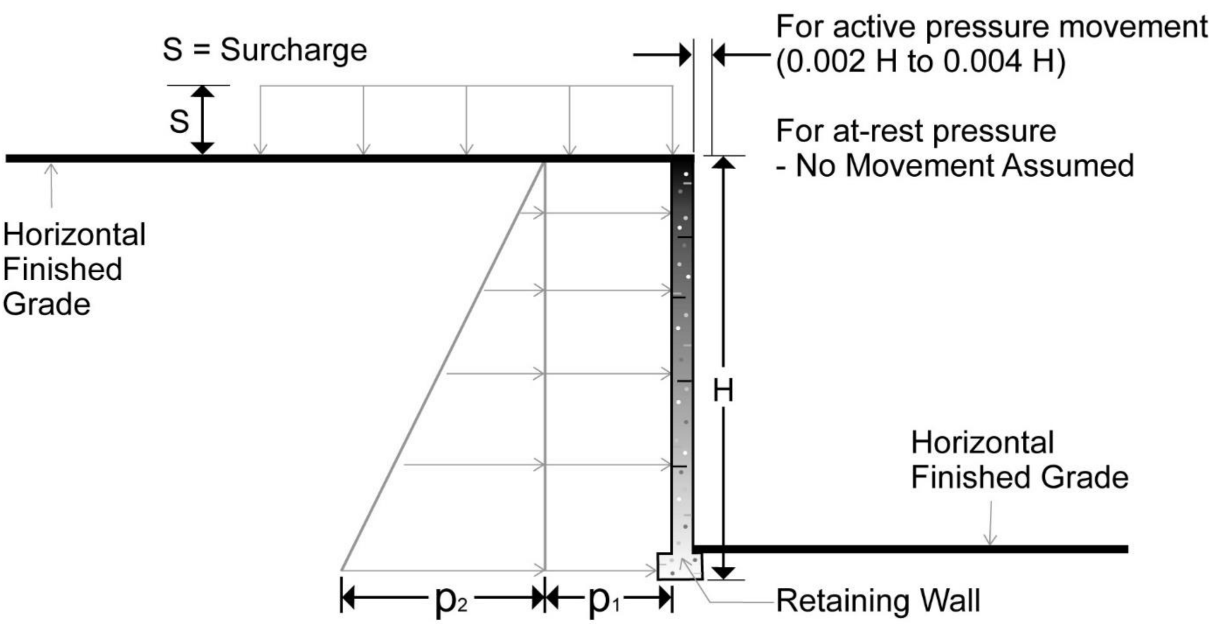
The seismic site class for this project is based on Section 3.10 of *AASHTO*. Site Class C should be used for the design of the proposed structure. The following table presents the interpolations of mapped spectral accelerations for the project site. Based on the high N-values obtained during drilling we anticipate the probability for liquefaction at this site is low.

Acceleration Type	Site Class B Value	Site Factor	Amplified Value for Site Class C
Zero-Period – PGA	0.06g	1.2	0.072g
Short-Period – S_S	0.13g	1.2	0.156g
Long-Period – S_1	0.033g	1.7	0.056g

4.5 Lateral Earth Pressures

Reinforced concrete walls with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to those indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement. The recommended design lateral earth pressures are ultimate values and do not provide for possible hydrostatic pressure on the walls.

If walls will extend below the expected high water level of the river, the walls should either be designed to resist hydrostatic pressures or should include a drainage layer extending to appropriate outlet locations to reduce the potential for hydrostatic pressure walls. Weep holes may also be used in conjunction with the drainage layer to reduce the potential for hydrostatic pressures.



EARTH PRESSURE COEFFICIENTS

Earth Pressure Conditions	Coefficient for Backfill Type	Equivalent Fluid Density (pcf)	Surcharge Pressure, p_1 (psf)	Earth Pressure, p_2 (psf)
Active (Ka)	Granular soil - 0.27	35	(0.27)S	(35)H
At-Rest (Ko)	Granular soil - 0.43	56	(0.43)S	(56)H
Passive (Kp)	Granular soil - 3.69	480	---	---

Applicable conditions to the above include:

- For active earth pressure, wall must rotate about base, with top lateral movements of about 0.002 **H** to 0.004 **H**, where **H** is wall height;
- For passive earth pressure to develop, wall must move horizontally to mobilize resistance;
- Uniform surcharge, where S is surcharge pressure;

- In-situ soil backfill weight a maximum of 130 pcf;
- Horizontal backfill, compacted to 95 percent of maximum dry unit weight as determined by AASHTO T180;
- Loading from heavy compaction equipment not included;
- No hydrostatic pressures acting on wall;
- No dynamic loading;
- A load factor has not been included in the soil parameters; and
- Ignore passive pressure in frost zone.

4.6 Pavements

4.6.1 Pavements – Subgrade Preparation

On most project sites, the site grading is accomplished relatively early in the construction phase. Fills are typically placed and compacted in a uniform manner. However as construction proceeds, the subgrade may be disturbed due to utility excavations, construction traffic, desiccation, or rainfall/snow melt. As a result, the pavement subgrade may not be suitable for pavement construction and corrective action will be required. The subgrade should be carefully evaluated at the time of pavement construction for signs of disturbance or instability. We recommend the pavement subgrade be thoroughly proofrolled with a loaded tandem-axle dump truck prior to final grading and paving. All pavement areas should be moisture conditioned and properly compacted to the recommendations in this report immediately prior to paving.

4.6.2 Pavements – Design Recommendations

Design of pavements for the project have been based on the procedures outlined in the 1993 *Guideline for Design of Pavement Structures* prepared by the American Association of State Highway and Transportation Officials (AASHTO).

We were provided with traffic count data from 2012 by J-U-B Engineers, Inc. We calculated 18-kip equivalent single-axle load (ESAL) from the provided average annual daily traffic (AADT) numbers provided to us. Old Saint Vrain Road was reported to have an AADT of 70 with 1 percent truck traffic, the proposed bridge approach was reported to have and AADT of 49 with 75 percent truck traffic.

For our pavement thicknesses design recommendations, we calculated design ESALs of 23,300 for Old Saint Vrain Road and 852,000 for the proposed bridge. These calculated traffic design values should be verified by the civil engineer or owner prior to final design and construction. If the actual traffic values vary from the calculated values, the pavement thickness recommendations may not be applicable. When the actual traffic design information is available Terracon should be contacted so that the design recommendations can be reviewed and revised if necessary.

Design parameters or other data used for determining the pavement thickness for this project are summarized in the following table:

Design Parameter/Data		Value
Calculated R-value		72
Correlated soil/subgrade resilient modulus (M_R)		27,000 psi
Reliability		85%
Overall standard deviation (S_o)		0.44
Design serviceability loss (ΔPSI)		2.0
Required structural number (SN_R) (Old Saint Vrain Road) 20-year design period		1.00
Required structural number (SN_R) (bridge approaches) 20-year design period		1.96
Pavement layer coefficient	Asphalt concrete (AC)	0.44
	Aggregate base course (ABC)	0.12

Using the design values above, appropriate ESAL, environmental criteria and other factors, the structural numbers (SN) of the pavement sections were determined on the basis of the 1993 AASHTO design equation.

Recommended minimum pavement sections are provided in the table below.

Traffic Area	Alternative	Recommended Pavement Thicknesses (Inches)		
		Asphaltic Concrete Surface	Aggregate Base Course	Total
Old Saint Vrain Road	A	3	6	9
	B[1]	4	--	4
Proposed Bridge Approaches	A	4	8	12

1. Full depth asphalt recommendations are only valid if fill placed below asphalt pavement (if any) has a minimum R-value of 72.

Aggregate base course should consist of a blend of sand and gravel which meets strict specifications for quality and gradation. Use of materials meeting Colorado Department of Transportation (CDOT) 6 specifications is recommended for aggregate base course. Aggregate base course should be placed in lifts not exceeding 6 inches and compacted to a minimum of 95 percent of the maximum dry unit weight as determined by AASHTO T180.

Asphaltic concrete should be composed of a mixture of aggregate, filler and additives (if required) and approved bituminous material. The asphalt concrete should conform to approved mix designs stating the Superpave properties, optimum asphalt content, job mix formula and recommended mixing and placing temperatures. Asphaltic cement bituminous material should meet the Superpave Performance specifications of PG 64-22. Aggregate used in asphalt concrete should meet CDOT Grading S specifications or equivalent. Mix designs should be submitted prior to construction to verify their adequacy. Asphalt material should be placed in maximum 3-inch lifts and compacted within a range of 92 to 96 percent of the theoretical maximum specific gravity according to Colorado Procedure 51-14.

Pavement performance is affected by its surroundings. In addition to providing preventive maintenance, the civil engineer should consider the following recommendations in the design and layout of pavements:

- Site grades should slope a minimum of 2 percent away from the pavements;
- The subgrade and the pavement surface have a minimum 2 percent slope to promote proper surface drainage;
- Consider appropriate edge drainage and pavement under drain systems;
- Install pavement drainage surrounding areas anticipated for frequent wetting;
- Install joint sealant and seal cracks immediately; and
- Placing compacted, low permeability backfill against the exterior side of pavements.

4.6.3 Pavements – Construction Considerations

Openings in pavement are sources for water infiltration into surrounding pavements. Water collects migrates into the surrounding subgrade soils thereby degrading support of the pavement. This is especially applicable for low permeability near-surface soils. The civil design for the pavements with these conditions should include features to restrict or to collect and discharge excess water. Examples of features are edge drains connected to the storm water collection system or other suitable outlet and impermeable barriers preventing lateral migration of water such as a cutoff wall installed to a depth below the pavement structure.

4.6.4 Pavements – Maintenance

Preventative maintenance should be planned and provided for an ongoing pavement management program in order to enhance future pavement performance. Preventive maintenance consists of both localized maintenance (e.g. crack and joint sealing and patching) and global maintenance (e.g. surface sealing). Preventative maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements.

5.0 GENERAL COMMENTS

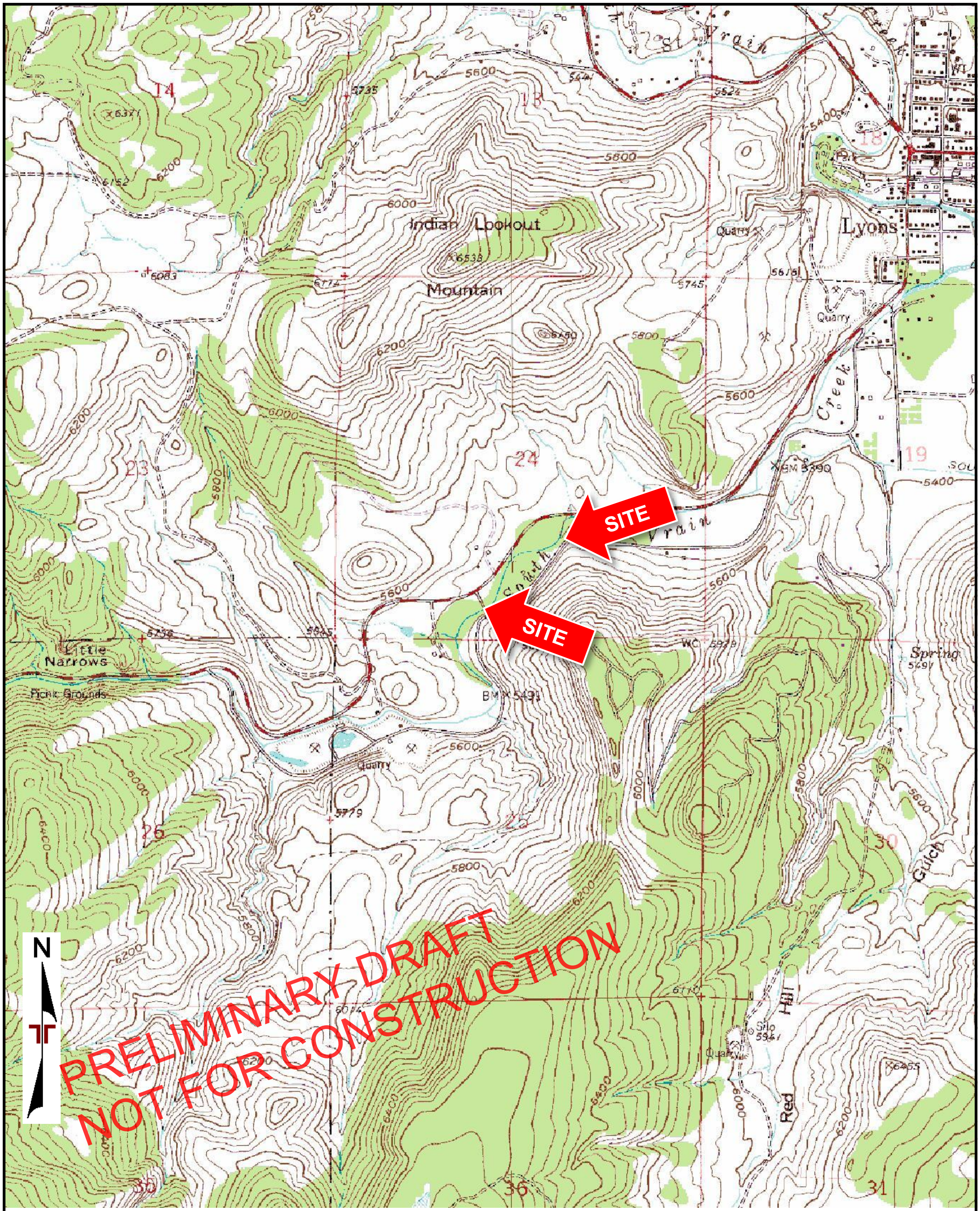
Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Terracon also should be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, and bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as described in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX A
FIELD EXPLORATION



TOPOGRAPHIC MAP IMAGE COURTESY OF THE U.S. GEOLOGICAL SURVEY
QUADRANGLES INCLUDE: LYONS, CO (1/1/1978).

Project Manager: BCR	Project No. 21155048	Terracon	SITE LOCATION MAP	Exhibit
Drawn by: BCR	Scale: 1"=24,000 SF	1289 First Ave. Greeley, CO 80631	Saint Vrain Bridge Replacement	A-1
Checked by: EDB	File Name:		**Approximately 2 miles Southwest of the intersection of Highway 36 and Highway 7 Lyons, CO**	
Approved by: EDB	Date: 12/22/2015			

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

AERIAL PHOTOGRAPHY PROVIDED BY MICROSOFT BING MAPS

Project Manager: BCR	Project No. 21155048	Terracon 1289 First Ave. Greeley, CO 80631	EXPLORATION PLAN	Exhibit
Drawn by: BCR	Scale: AS SHOWN		Saint Vrain Bridge Replacement **Approximately 2 miles Southwest of the intersection of Highway 36 and Highway 7** **Lyons, CO**	A-2
Checked by: EDB	File Name:			
Approved by: EDB	Date: 12/22/2015			

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

AERIAL PHOTOGRAPHY PROVIDED BY MICROSOFT BING MAPS

Project Manager: BCR	Project No. 21155048	Terracon 1289 First Ave. Greeley, CO 80631	EXPLORATION PLAN	Exhibit
Drawn by: BCR	Scale: AS SHOWN		Saint Vrain Bridge Replacement Approximately 2 miles Southwest of the intersection of Highway 36 and Highway 7 Lyons, CO	A-3
Checked by: EDB	File Name:			
Approved by: EDB	Date: 12/22/2015			

Field Exploration Description

The locations of borings were based upon the proposed locations of the abutments and center bridge support. The borings were located in the field by measuring from existing site features. The ground surface elevation was surveyed at each boring location by the project surveyor. At the time this report was prepared, the survey information was not provided to us.

The borings were drilled with a CME-75 truck-mounted ODEX system and an NQ size core bit. During the drilling operations, lithologic logs of the borings were recorded by the field engineer. Disturbed samples were obtained at selected intervals utilizing a 2-inch outside diameter split-spoon sampler. Disturbed bulk samples were obtained from auger cuttings and the creek channel. This test consists of driving the sampler into the ground with a 140-pound hammer free-falling through a distance of 30 inches. The number of blows required to advance the split-spoon sampler 18 inches (final 12 inches are recorded) or the interval indicated, is recorded as a standard penetration resistance value (N-value). The blow count values are indicated on the boring logs at the respective sample depths. Rock quality designation (RQD) was also measured on rock core samples. RQD is a rough measurement of the degree of jointing or fracture in a rock mass measured as a percentage of the drill core length of 4 inches or more.

A CME automatic hammer was used to advance the samplers in the borings performed on this site. A greater efficiency is typically achieved with the automatic hammer compared to the conventional safety hammer operated with a cathead and rope. Published correlations between the SPT values and soil properties are based on the lower efficiency cathead and rope method. This higher efficiency affects the standard penetration resistance blow count value by increasing the penetration per hammer blow over what would be obtained using the cathead and rope method. The effect of the automatic hammer's efficiency has been considered in the interpretation and analysis of the subsurface information for this report.

The standard penetration test provides a reasonable indication of the in-place density of sandy type materials, but only provides an indication of the relative stiffness of cohesive materials since the blow count in these soils may be affected by the moisture content of the soil. In addition, considerable care should be exercised in interpreting the N-values in gravelly soils, particularly where the size of the gravel particle exceeds the inside diameter of the sampler.

Groundwater measurements were obtained in the borings at the time of site exploration. After completion of drilling, the borings were backfilled with auger cuttings. Some settlement of the backfill and/or patch may occur and should be repaired as soon as possible.

BORING LOG NO. 1

PROJECT: Saint Vrain Bridge Replacement

CLIENT: J-U-B Engineers, Inc.
Fort Collins, Colorado

SITE: Approximately 2 miles SW of Hwy 36 & Hwy 7
Lyons, Colorado

LOCATION See Exhibit A-2

Latitude: 40.206807° Longitude: -105.293883°

GRAPHIC LOG / DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (%)	FIELD TEST RESULTS	RQD (%)	UNCONFINED COMPRESSIVE STRENGTH (psi)	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI
0.5	**ASPHALT PAVEMENT - 6 inches**									
1.1	**FILL - SAND AND GRAVEL - 7 inches**									
	POORLY GRADED GRAVEL WITH SILT, SAND, COBBLES AND BOULDERS, fine to coarse grained, dark brown to light brown, dense to very dense	5		X		9-25-18 N=43			6	NP
		10	▽	X		15-27-45 N=72			7	
		15		X		34-50/5"				
		20		X		14-39-50/6"				
		25		X		50/6"				
		30		X		50/4"				
		35		X		29-50/6"				
		40		X		35-50/5"				
		45		X		50/6"				
50.5	**SEDIMENTARY BEDROCK - SANDSTONE**, fine to coarse grained, dark red with gray, trace muscovite rich clay layers, trace gray colored sand and gravel with flow structure, fine crossbedding	50		X		11-19-32 N=51				
		55–60		Core	79		29	2,660	16	
		60–65		Core	100		40			
		65–70		Core	100		55		6	NP
	Fine interbedding of gray to green siltstone and sandstone to 85 feet below ground surface. Exhibits fine crossbedding and lamination.	70–75		Core	100		82			
		75–80		Core	100		87	1,113	2	
85.0		80–85		Core	100		78	7,300		
	Boring Terminated at 85 Feet	85								

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Odex and NQ size rock-core-barrel

Abandonment Method:
Borings backfilled with soil cuttings upon completion.

See Exhibit A-4 for description of field procedures

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
1289 First Avenue
Greeley, Colorado

Boring Started: 11/24/2015	Boring Completed: 11/24/2015
Drill Rig: CME-75	Driller: Alex G
Project No.: 21155048	Exhibit: A-5

BORING LOG NO. 2

PROJECT: Saint Vrain Bridge Replacement

CLIENT: J-U-B Engineers, Inc.
Fort Collins, Colorado

SITE: Approximately 2 miles SW of Hwy 36 & Hwy 7
Lyons, Colorado

LOCATION See Exhibit A-2

Latitude: 40.206552° Longitude: -105.293637°

DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (%)	FIELD TEST RESULTS	RQD (%)	UNCONFINED COMPRESSIVE STRENGTH (psi)	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI
0.5	**ASPHALT PAVEMENT - 6 inches**									
1.2	**FILL - SAND AND GRAVEL - 8 inches**									
	POORLY GRADED GRAVEL WITH SILT, SAND, COBBLES AND BOULDERS, fine to coarse grained, brown to reddish-brown, medium dense to very dense	5				9-4-7 N=11			4	
		10	▽			8-24-50/5"			7	
		15				50/5"			9	
		20				40-50			8	NP
		25				50/6"				
		30				26-50/6"				
		35				20-50/4"				
		45				28-50/3"				
		50				28-50/5"				
53.0	**SEDIMENTARY BEDROCK - SANDSTONE**, fine to coarse grained, dark red with gray, trace muscovite rich clay layers, trace gray colored sand and gravel with flow structure, fine crossbedding	55–60			88		79	5,158		
		60–65			99		88	3,062		
		65–70			100		94			
	Interbedding of gray to green siltstone and sandstone to 74 feet below ground surface. Exhibits flow structure.	70–75			100		82	3,696		
		75–80			100		85	3,225		
84.0		80–84			100		28			

Boring Terminated at 84 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Odex and NQ size rock-core-barrel

Abandonment Method:
Borings backfilled with soil cuttings upon completion.

See Exhibit A-4 for description of field procedures

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
1289 First Avenue
Greeley, Colorado

Boring Started: 10/15/2015	Boring Completed: 10/15/2015
Drill Rig: CME-75	Driller: Alex G
Project No.: 21155048	Exhibit: A-6

BORING LOG NO. 3

PROJECT: Saint Vrain Bridge Replacement

CLIENT: J-U-B Engineers, Inc. Fort Collins, Colorado

SITE: Approximately 2 miles SW of Hwy 36 & Hwy 7 Lyons, Colorado

LOCATION See Exhibit A-2

Latitude: 40.206212° Longitude: -105.293306°

DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (%)	FIELD TEST RESULTS	RQD (%)	UNCONFINED COMPRESSIVE STRENGTH (psi)	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI
0.5	**ASPHALT PAVEMENT - 6 inches**									
1.2	**FILL - SAND AND GRAVEL - 8 inches**									
	POORLY GRADED GRAVEL WITH SILT, SAND, COBBLES AND BOULDERS, fine to coarse grained, light brown, medium dense to very dense	5				9-8-7 N=15			2	
		10	▽			26-13-16 N=29			3	NP
		15				43-50/3"				
23.0		20				50/5"				
	SEDIMENTARY BEDROCK - SANDSTONE, coarse grained, red to white, granitic source rock, plagioclase and quartz rich	25–30			88		61	1,913		
		30–35			100		70			
	Fine interbedding of dark red, fine grained sandstone to 47.5 feet below ground surface.	35–40			100		67	5,395		
		40–45			100		97			
47.5	**SEDIMENTARY BEDROCK - SANDSTONE**, fine to coarse grained, dark red with gray, trace muscovite rich clay layers, fine crossbedding	45–50			100		97			
55.0	Interbedded light gray sandstone, fine grained, crossbedding and flow structure, about 12 inches in depth.	50–55			100		95	4,024		
	Boring Terminated at 55 Feet									

PRELIMINARY DRAFT
NOT FOR CONSTRUCTION

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Odex and NQ size rock-core-barrel

Abandonment Method:
Borings backfilled with soil cuttings upon completion.

See Exhibit A-4 for description of field procedures

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
1289 First Avenue
Greeley, Colorado

Boring Started: 11/30/2015	Boring Completed: 11/30/2015
Drill Rig: CME-75	Driller: Alex G
Project No.: 21155048	Exhibit: A-7

BORING LOG NO. 4

PROJECT: Saint Vrain Bridge Replacement

**CLIENT: J-U-B Engineers, Inc.
Fort Collins, Colorado**

**SITE: Approximately 2 miles SW of Hwy 36 & Hwy 7
Lyons, Colorado**

LOCATION See Exhibit A-2

Latitude: 40.208873° Longitude: -105.289666°

DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	UNCONFINED COMPRESSIVE STRENGTH (psi)	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI
0.5	**FILL - SAND AND GRAVEL - 6 inches**							
3.0	**FILL - POORLY GRADED SAND WITH SILT AND GRAVEL**, with cobbles, brown, loose				4-3-5 N=8		1	NP
5.3	**POORLY GRADED GRAVEL WITH SILT AND SAND**, with cobbles, fine to coarse grained, brown to reddish-brown, very dense	5			23-24-50/4"		0	

Boring Terminated at 5.3 Feet

PRELIMINARY DRAFT
NOT FOR CONSTRUCTION

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4.25 inch hollow stem auger

Abandonment Method:
Borings backfilled with soil cuttings upon completion.

See Exhibit A-4 for description of field procedures

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

No free water observed

Terracon
1289 First Avenue
Greeley, Colorado

Boring Started: 10/15/2015	Boring Completed: 10/15/2015
Drill Rig: CME-75	Driller: Terracon
Project No.: 21155048	Exhibit: A-8

BORING LOG NO. 5

PROJECT: Saint Vrain Bridge Replacement

**CLIENT: J-U-B Engineers, Inc.
Fort Collins, Colorado**

**SITE: Approximately 2 miles SW of Hwy 36 & Hwy 7
Lyons, Colorado**

LOCATION See Exhibit A-2

Latitude: 40.209253° Longitude: -105.288972°

DEPTH	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	UNCONFINED COMPRESSIVE STRENGTH (psi)	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI
0.5	**FILL - SAND AND GRAVEL - 6 inches**							
3.0	**FILL - POORLY GRADED SAND WITH SILT AND GRAVEL**, fine to coarse grained, brown, loose				7-2-2 N=4		3	NP
5.5	**POORLY GRADED GRAVEL WITH SILT AND SAND**, with cobbles, fine to coarse grained, brown to reddish-brown, medium dense	5			6-6-8 N=14		4	

Boring Terminated at 5.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
4.25 inch hollow stem auger

Abandonment Method:
Borings backfilled with soil cuttings upon completion.

See Exhibit A-4 for description of field procedures

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

No free water observed

Terracon
1289 First Avenue
Greeley, Colorado

Boring Started: 10/15/2015	Boring Completed: 10/15/2015
Drill Rig: CME-75	Driller: Terracon
Project No.: 21155048	Exhibit: A-9

APPENDIX B
LABORATORY TESTING

Laboratory Testing Description

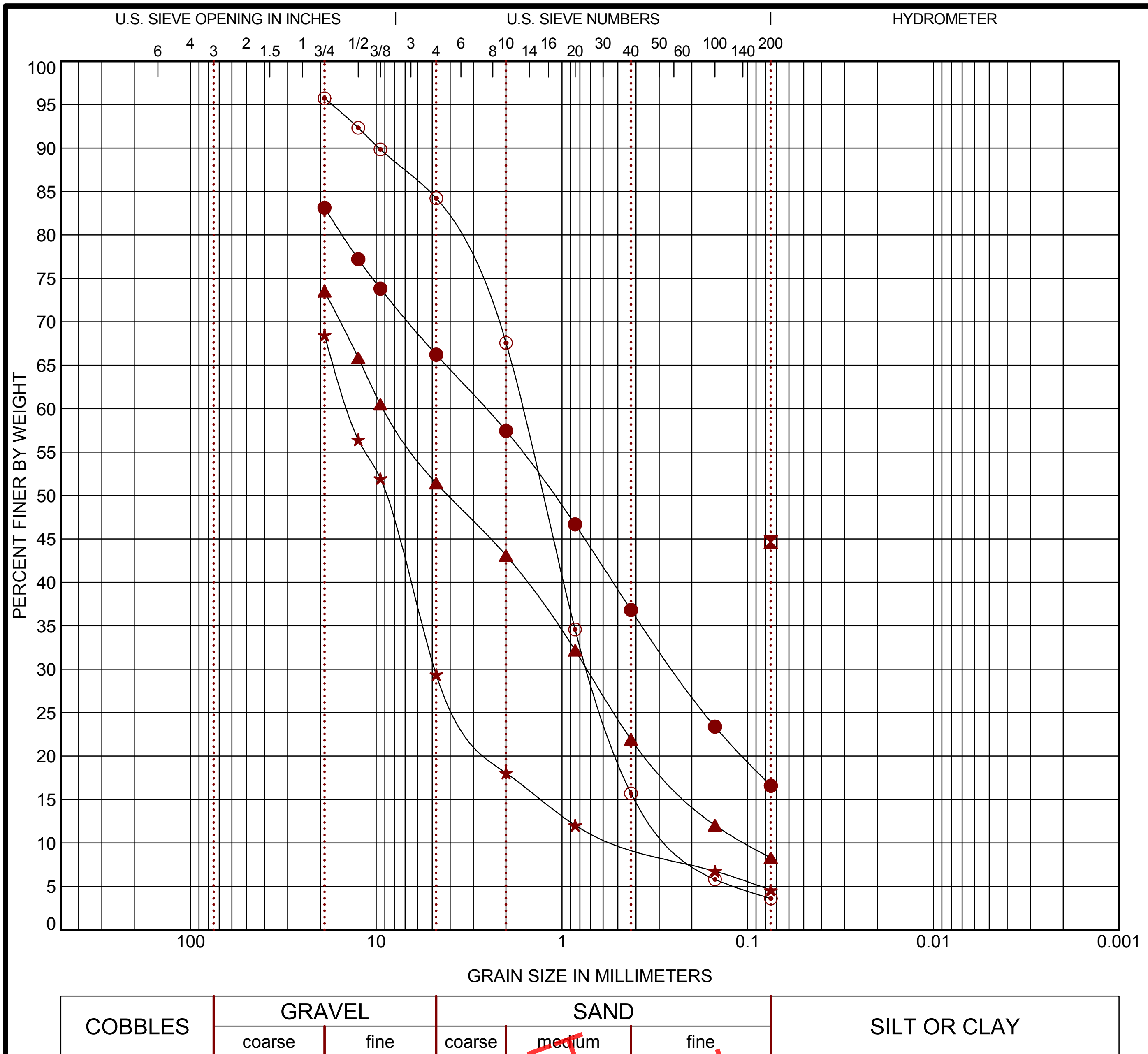
The soil and bedrock samples retrieved during the field exploration were returned to the laboratory for observation by the project geotechnical engineer. At that time, the field descriptions were reviewed and an applicable laboratory testing program was formulated to determine engineering properties of the subsurface materials.

Laboratory tests were conducted on selected soil and bedrock samples. The results of these tests are presented on the boring logs and in this appendix. The test results were used for the geotechnical engineering analyses, and the development of foundation and earthwork recommendations. The laboratory tests were performed in general accordance with applicable locally accepted standards. Soil samples were classified in general accordance with the Unified Soil Classification System described in Appendix C. Recovery of gravels, cobbles, and boulders larger than 1½ inches in diameter is not possible using the standard split-spoon sampler. Grain size sieve analysis test results do not include materials larger than 1½ inches in diameter. Visual classification of subsurface materials are presented on the borings logs and should be used in conjunction with the grain size sieve analysis. Rock samples were visually classified in general accordance with the description of rock properties presented in Appendix C. Procedural standards noted in this report are for reference to methodology in general. In some cases variations to methods are applied as a result of local practice or professional judgment.

- Water content
- Grain-size distribution
- Compressive strength
- Water-soluble sulfate content
- Plasticity index
- Dry density
- R-value

GRAIN SIZE DISTRIBUTION

ASTM D422



Boring ID		Depth	USCS Classification	LL	PL	PI	Cc	Cu
●	1	4 - 5.5	SILTY SAND with GRAVEL(SM)	NP	NP	NP		
⊠	1	65 - 70	SANDSTONE	NP	NP	NP		
▲	2	19 - 20	POORLY GRADED GRAVEL with SILT and SAND(GP-GM)	NP	NP	NP	0.57	88.75
★	3	9 - 10.5	POORLY GRADED GRAVEL with SAND(GP)	NP	NP	NP	3.79	32.39
⊙	4	2 - 3.5	WELL-GRADED SAND with GRAVEL(SW)	NP	NP	NP	1.35	7.05

Boring ID		Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Fines
●	1	4 - 5.5	19	2.574	0.25		16.9	49.6	16.6
⊠	1	65 - 70	0.075				0.0	0.0	44.6
▲	2	19 - 20	19	9.141	0.734	0.103	22.1	43.1	8.3
★	3	9 - 10.5	19	14.154	4.842	0.437	39.1	24.8	4.6
⊙	4	2 - 3.5	19	1.643	0.718	0.233	11.5	80.6	3.6

PROJECT: Saint Vrain Bridge Replacement

SITE: Approximately 2 miles SW of Hwy 36 & Hwy 7
Lyons, Colorado

Terracon
1289 First Avenue
Greeley, Colorado

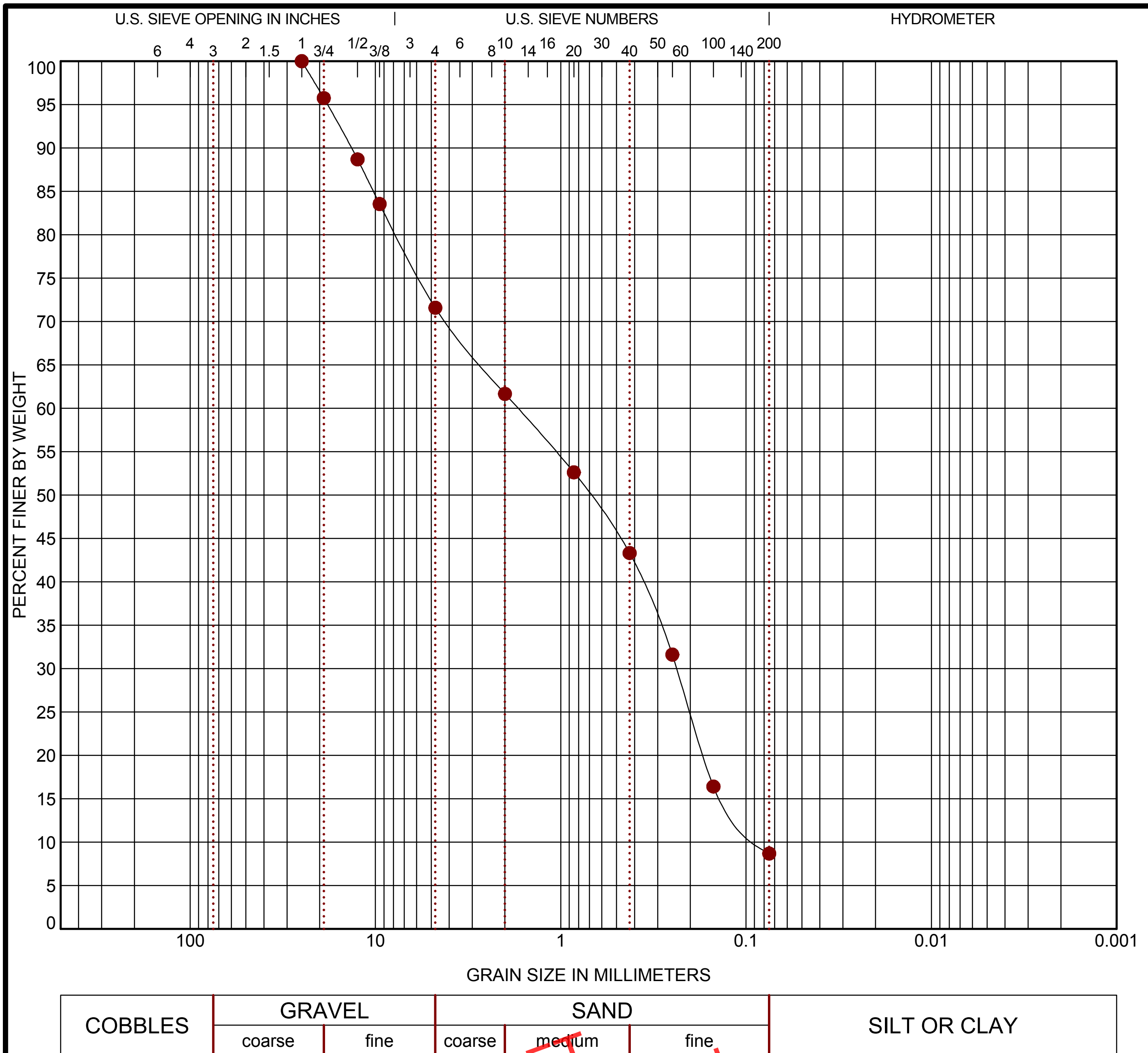
PROJECT NUMBER: 21155048

CLIENT: J-U-B Engineers, Inc.
Fort Collins, Colorado

EXHIBIT: B-2

GRAIN SIZE DISTRIBUTION

ASTM D422



Boring ID	Depth	USCS Classification	LL	PL	PI	Cc	Cu
● 5	2 - 3.5	POORLY GRADED SAND with SILT and GRAVEL(SP-SM)	NP	NP	NP	0.39	20.25

Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Fines
● 5	2 - 3.5	25	1.709	0.237	0.084	28.4	62.9	8.7

PROJECT: Saint Vrain Bridge Replacement

SITE: Approximately 2 miles SW of Hwy 36 & Hwy 7
Lyons, Colorado

Terracon
1289 First Avenue
Greeley, Colorado

PROJECT NUMBER: 21155048

CLIENT: J-U-B Engineers, Inc.
Fort Collins, Colorado

EXHIBIT: B-3

Terracon

1501 Sharp Point Drive, Suite C
Fort Collins, Colorado 80525
(970) 484-0359 FAX (970) 484-0454

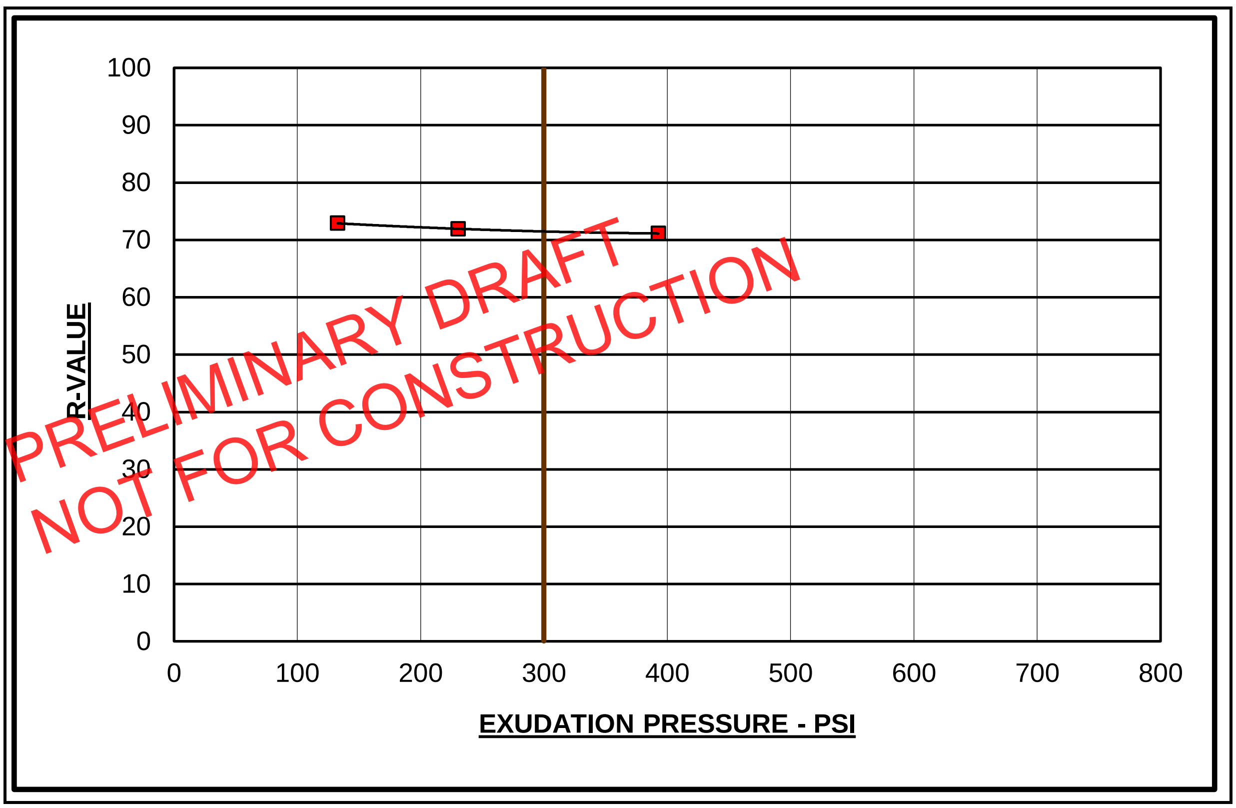
RESISTANCE R-VALUE & EXPANSION PRESSURE OF COMPACTED SOIL AASHTO T190

CLIENT: J-U-B Engineer, Inc. **DATE OF TEST:** 07-Dec-15
PROJECT: Saint Vrain Bridge Replacement
LOCATION: Combined Bulk sample from Boring Nos. 4 and 5 at 0.5 to 4 feet
TERRACON NO. 21155048 **CLASSIFICATION:** Poorly Graded Sand with Silt and Gravel (SP-SM)

SAMPLE DATA TEST RESULTS

TEST SPECIMEN NO.	1	2	3
COMPACTION PRESSURE (PSI)	350	350	350
DENSITY (PCF)	127.3	125.9	126.3
MOISTURE CONTENT (%)	9.8	9.7	9.6
EXPANSION PRESSURE (PSI)	-0.19	-0.09	-0.16
HORIZONTAL PRESSURE @ 160 PSI	32	32	34
SAMPLE HEIGHT (INCHES)	2.71	2.52	2.52
EXUDATION PRESSURE (PSI)	132.7	230.6	392.8
CORRECTED R-VALUE	72.9	71.9	71.1
UNCORRECTED R-VALUE	67.2	71.9	71.1

R-VALUE @ 300 PSI EXUDATION PRESSURE = 72



Colorado Analytical LABORATORIES, INC.

Analytical Results

TASK NO: 151209081

Report To: Mazie R. Ashe
Company: Terracon, Inc. - Greeley
1289 First Avenue
Greeley CO 80631

Bill To: Accounts Payable
Company: Terracon, Inc. - Lenexa
13910 W. 96th Terrace
Lenexa KS 66215

Task No.: 151209081
Client PO:
Client Project: 21155048

Date Received: 12/9/15
Date Reported: 12/15/15
Matrix: Soil - Geotech

Customer Sample ID 5 @ 2 Ft.
Lab Number: 151209081-01

Test	Result	Method
Sulfate - Water Soluble	**0.009 %**	AASHTO T290-91/ ASTM D4327

Customer Sample ID 1 @ 9 Ft.
Lab Number: 151209081-02

Test	Result	Method
Sulfate - Water Soluble	**< 0.001 %**	AASHTO T290-91/ ASTM D4327

Customer Sample ID 2 @ 14 Ft.
Lab Number: 151209081-03

Test	Result	Method
Sulfate - Water Soluble	**0.001 %**	AASHTO T290-91/ ASTM D4327

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.

DATA APPROVED FOR RELEASE BY

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APPENDIX C
SUPPORTING DOCUMENT

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING

- Auger Cuttings
- Rock Core
- Standard Penetration Test

WATER LEVEL

- **Water Initially Encountered**
- **Water Level After a Specified Period of Time**
- **Water Level After a Specified Period of Time**

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

FIELD TESTS

N	**Standard Penetration Test Resistance (Blows/Ft.)**
(HP)	**Hand Penetrometer**
(T)	**Torvane**
(DCP)	**Dynamic Cone Penetrometer**
(PID)	**Photo-Ionization Detector**
(OVA)	**Organic Vapor Analyzer**

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		**CONSISTENCY OF FINE-GRAINED SOILS** (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance			**BEDROCK**	
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength Qu, (psi)**	**Standard Penetration or N-Value Blows/Ft.**	**Standard Penetration or N-Value Blows/Ft.**	**Descriptive Term (Consistency)**
Very Loose	0 - 3	Very Soft	less than 3.50	0 - 1	< 20	Weathered
Loose	4 - 9	Soft	3.5 to 7.0	2 - 4	20 - 29	Firm
Medium Dense	10 - 29	Medium-Stiff	7.0 to 14.0	4 - 8	30 - 49	Medium Hard
Dense	30 - 50	Stiff	14.0 to 28.0	8 - 15	50 - 79	Hard
Very Dense	> 50	Very Stiff	28.0 to 55.5	15 - 30	>79	Very Hard
		Hard	> 55.5	> 30		

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm
Silt or Clay	Passing #200 sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1 - 10
Medium	11 - 30
High	> 30

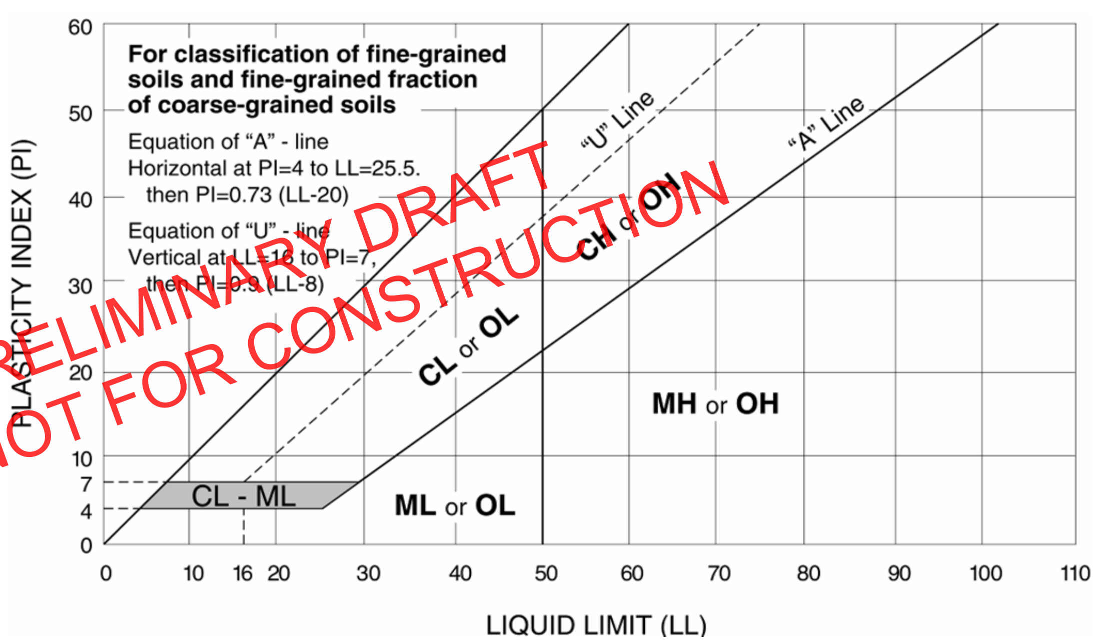
UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	Group Name [B]
Coarse Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu ≥ 4 and 1 ≤ Cc ≤ 3 [E]	GW	Well-graded gravel [F]
			Cu < 4 and/or 1 > Cc > 3 [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel [F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu ≥ 6 and 1 ≤ Cc ≤ 3 [E]	SW	Well-graded sand [I]
			Cu < 6 and/or 1 > Cc > 3 [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G,H,I]
			Fines classify as CL or CH	SC	Clayey sand [G,H,I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above "A" line [J]	CL	Lean clay [K,L,M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [K,L,M,N]
					Organic silt [K,L,M,O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K,L,M]
			PI plots below "A" line	MH	Elastic Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [K,L,M,P]
					Organic silt [K,L,M,Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



DESCRIPTION OF ROCK PROPERTIES

WEATHERING

Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very slight	Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 in. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very severe	All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil". Rock "fabric" not discernible or discernible only in small, scattered locations. Quartz may be present as dikes or stringers.

HARDNESS (for engineering description of rock – not to be confused with Moh's scale for minerals)

Very hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately hard	Can be scratched with knife or pick. Gouges or grooves to ¼ in. deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16 in. deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1-in. maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Joint, Bedding, and Foliation Spacing in Rock [a]

Spacing	Joints	Bedding/Foliation
Less than 2 in.	Very close	Very thin
2 in. – 1 ft.	Close	Thin
1 ft. – 3 ft.	Moderately close	Medium
3 ft. – 10 ft.	Wide	Thick
More than 10 ft.	Very wide	Very thick

a. Spacing refers to the distance normal to the planes, of the described feature, which are parallel to each other or nearly so.

Rock Quality Designator (RQD) a

RQD, as a percentage	Diagnostic description
Exceeding 90	Excellent
90 – 75	Good
75 – 50	Fair
50 – 25	Poor
Less than 25	Very poor

a. RQD (given as a percentage) = length of core in pieces 4 in. and longer/length of run.

Joint Openness Descriptors

Openness	Descriptor
No Visible Separation	Tight
Less than 1/32 in.	Slightly Open
1/32 to 1/8 in.	Moderately Open
1/8 to 3/8 in.	Open
3/8 in. to 0.1 ft.	Moderately Wide
Greater than 0.1 ft.	Wide

References: American Society of Civil Engineers. Manuals and Reports on Engineering Practice - No. 56. Subsurface Investigation for Design and Construction of Foundations of Buildings. New York: American Society of Civil Engineers, 1976. U.S. Department of the Interior, Bureau of Reclamation, Engineering Geology Field Manual.