TRAUTNER GEOTECH LLC
GEOTECHNICAL ENGINEERING, MATERIAL TESTING
AND ENGINEERING GEOLOGY

GEOTECHNICAL ENGINEERING

RECOMMENDATIONS

BAYFIELD TWIN BRIDGES PROJECT

Prepared For:
Mr. Rich Bechtolt, P.E.
Bechtolt Engineering, Inc.
Project Number: 53371GE
September 4, 2014

1.0 REPORT INTRODUCTION

This report presents our geotechnical engineering recommendations for the proposed Bayfield Twin Bridges Replacement Project site. This report was requested by Mr. Rich Bechtolt, P.E., Bechtolt Engineering, Inc. The field study was completed on March 12, 2014. The laboratory study was completed on April 11, 2014. We performed streambed particle size analysis in the field on August 11, 2014.

The information provided in this report is intended to help develop a design and implementation of construction strategies that are appropriate for the subsurface soil and water conditions at the project site. It is important that we are consulted throughout the design and construction process to verify the implementation of the geotechnical engineering recommendations provided in this report. The recommendations and technical aspects of this report are intended for design and construction personnel who are familiar with construction concepts and techniques, and understand the terminology presented below. We should be contacted if any questions or comments arise as a result of the information presented below.

The following outline provides a synopsis of the various portions of this report;

- Sections 1.0 and 2.0 provide an introduction and an establishment of our scope of service.
- Sections 3.0 and 4.0 of this report present our geotechnical engineering field and laboratory studies
- Sections 5.0 through 6.0 presents our geotechnical engineering design parameters and recommendations which are based on our engineering analysis of the data obtained.
- Section 7.0 provides a brief discussion of construction sequencing and strategies which may influence the geotechnical engineering characteristics of the site.

The discussion and construction recommendations presented in Section 7.0 are intended to help develop site soil conditions that are consistent with the geotechnical engineering recommendations presented previously in the report. The construction considerations section is not intended to address all of the construction planning and needs for the project site, but is intended to provide an overview to aid the owner, design team, and contractor in understanding some construction concepts that may influence some of the geotechnical engineering aspects of the site and proposed development.

The data used to generate our recommendations are presented throughout this report and in the attached figures.

1.1 *Scope of Project*

We understand that the proposed project will consist of removing and replacing the two existing bridge structures located on the Bayfield Parkway road that span over the main Los Piños River channel and the overflow channel. We understand that the proposed bridge structures will likely be girder type structures to allow sufficient hydraulic flow under the bridges.

2.0 GEOTECHNICAL ENGINEERING STUDY

This section of this report presents the scope of our field and laboratory study, and presents the geotechnical engineering recommendations that are provided in detail later in this report.

2.1 *Geotechnical Engineering Study Scope of Service*

The scope of our study which was delineated in our proposal for services, and the order of presentation of the information within this report, is outlined below.

Field Study

- We advanced four (4) test borings at the project within the areas we understand are planned for construction of the proposed bridge support foundation systems. Each test boring was advanced within the Bayfield Parkway road adjacent to each (east and west) existing abutment for each of the two existing bridges.
- Select driven sleeve, bag samples, and rock core were obtained from the test borings and returned to our laboratory for testing.

Laboratory Study

- The laboratory testing and analysis of the samples obtained included;
    - Moisture content and dry density of soil and rock materials encountered in our test borings,
    - Direct shear strength tests performed on select soil samples to help establish a basis for development of lateral earth pressure values, and development of soil strength data for use in computer modelling programs such as LPILE for lateral load design of deep foundation components,
    - Unconfined compressive strength tests on select sections of rock core in order to provide engineering design parameters for the formational materials underlying the project site,

- Swell/consolidation tests to help assess the expansion and consolidation potential of the soil materials and weathered formational claystone materials encountered in our test borings,
- Plastic and liquid limit tests to determine the Plasticity Index of the soil,
- Sieve analysis tests to determine the gradation of the subsurface soil materials, and,
- Soluble Sulfates testing in order to assess the corrosion potential of the site soil and formational materials on Portland cement concrete.

Geotechnical Engineering Recommendations

- This report addresses the geotechnical engineering aspects of the site and provides recommendations including;

Geotechnical Engineering Section(s)

- Subsurface soil and water conditions that may influence the project design and construction considerations
- Estimates of the streambed gravel and cobble soil D_{50} particle size for the main channel and overflow channels,
- Estimates of the R-Value strength characteristics for the existing roadway subgrade soil materials,
- Geotechnical engineering foundation design parameters including;
  - ✓ Geotechnical engineering design parameters for drilled caisson foundation systems and driven pile foundation systems.
  - ✓ A brief discussion of other potential foundation systems that are technically feasible from a geotechnical engineering perspective for the project, but may not be adequate for scour design for the project.
  - ✓ Lateral Earth Pressure values for design of retaining structures.

Construction Consideration Section

- Fill placement considerations including cursory comments regarding site preparation and grubbing operations,
- Considerations for excavation cut slopes,
- Natural soil preparation considerations for use as backfill on the site,
- Compaction recommendations for various types of backfill proposed at the site.

- This report provides design parameters, but does not provide foundation design or design of structure components. The project architect, designer, structural engineer or builder may be contacted to provide a design based on the information presented in this report.

- Our subsurface exploration, laboratory study and engineering analysis do not address environmental or geologic hazard issues

3.0 FIELD STUDY

3.1 *Project location*

The bridge replacement project is located on the Bayfield Parkway, approximately one (1) mile east of the west intersection of Bayfield Parkway and U.S. Highway 160. The project site is located within the Town of Bayfield Limits located in La Plata County, Colorado. The approximate location of the project site (two bridge locations) is shown on Figure 3.1 below. The aerial photograph used for Figure 3.1 was obtained from Google Earth.

Figure 3.1: Project Location

As previously discussed, the proposed project involves the removal and replacement of two (2) existing bridge structures. The east bridge crosses the main channel of the Los Piños River, while the west bridge crosses an overflow channel west of the main channel of the Los Piños River. The western overflow channel rarely has active flowing water. We understand that the existing steel frame bridge structures were constructed in the 1930's. The existing west bridge is about 180 feet in length, and the existing east bridge is about 160 feet in length. We understand that the foundation system that supports these structures is not known at this time however it is suspected that they are supported by a spread footing foundation system. We understand that a gas line, sewer line, and telecommunications line are currently supported above grade by the existing bridges, and a water line and fiber optic line were recently installed under the river bed within the Bayfield Parkway right-of-way.

The portion of Bayfield Parkway between the two bridge structures and east of the east bridge structure is constructed over embankment fill material. The height of the embankment fill material ranges from about six (6) to ten (10) feet, including the embankment fill material immediately adjacent to the existing bridge abutments.

The Los Piños River is a perennial flowing river. The Vallecito Reservoir is located upstream of the project location, therefore the flow rate in the river at the project site is controlled in some part by the reservoir. The west side channel of the river that is located under the west bridge rarely exhibits active flowing water. We understand that flowing water in the side channel only occurs during very high river flow events. We observed relatively stagnant/ponded water within heavy cattail growth in the bottom of the west channel below the west bridge during the preparation of this report.

The subsurface soil and rock materials encountered in the vicinity of the project consists of very dense variable depth alluvial deposits of gravel and cobbles overlying the Animas Formation. The gravel and cobbles primarily consist of very hard quartzite material. It is typically very difficult to advance conventional auger borings through the very dense gravel/cobble deposits in areas adjacent to the Los Piños River corridor of Bayfield. The Animas Formation consists of inter-bedded layers of claystone, shale, and sandstone materials. The Los Piños River has historically scoured to the Animas Formation. Subsurface free water is typically encountered at the approximate water elevation of the Los Piños River due to the relatively permeable nature of the gravel and cobble alluvial deposits in the project area.

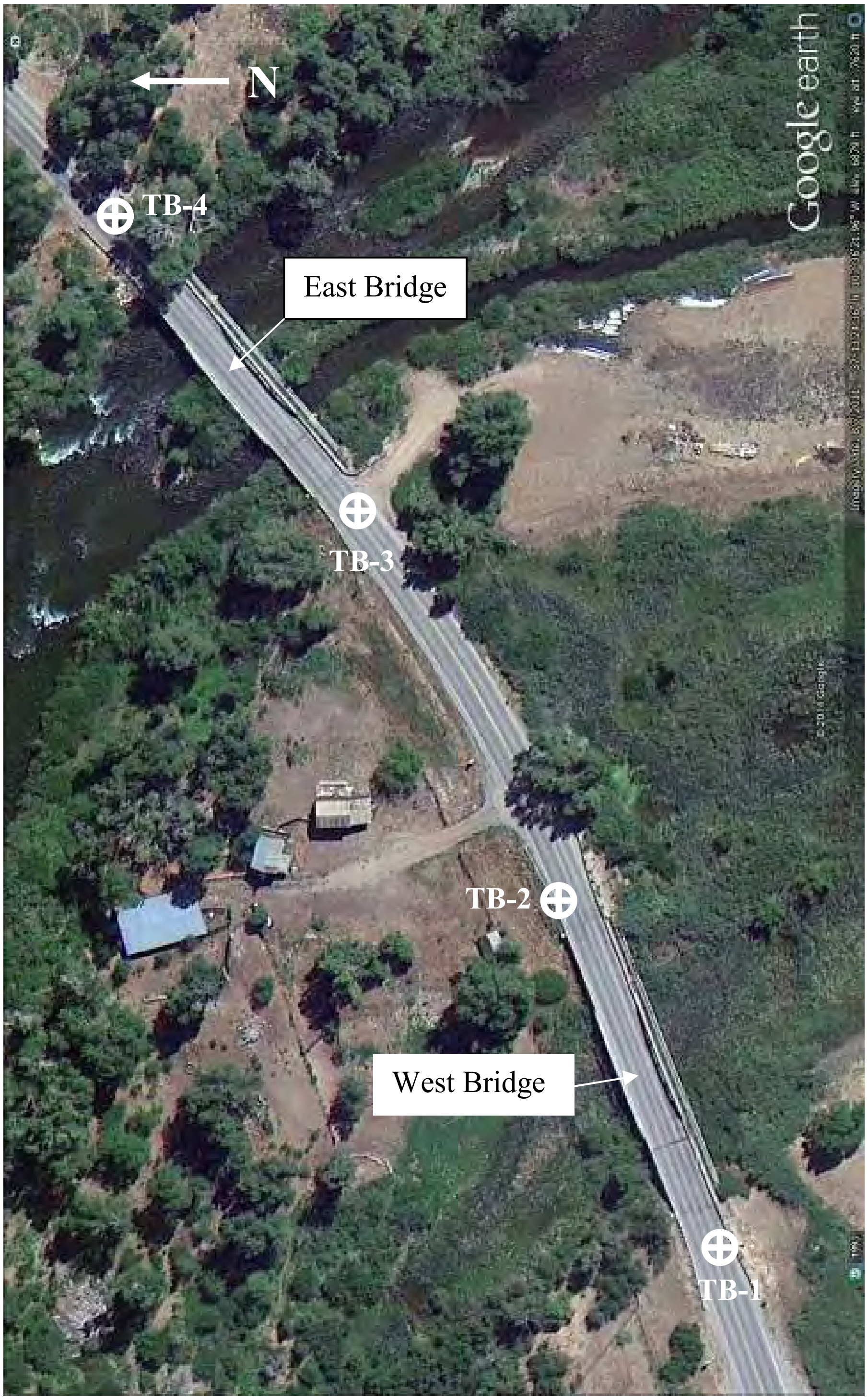
3.3 *Subsurface Soil and Water Conditions*

We advanced four (4) test borings for the project. A test boring was advanced adjacent to each abutment of each of the two existing bridges. The test borings were advanced within the Bayfield Parkway road prism, a distance of approximately twenty (20) feet away from the existing bridge abutments. A lane closure of Bayfield Parkway was required for each of the test borings. Each test boring was completed and backfilled during a day to allow for traffic to return to both lanes of the road for normal traffic flow. It was not feasible to pull our drill/core string midway through a test boring and complete it the following day due to the caving nature of the subsurface soil/rock materials and the fact that the drilling was performed within the traffic lanes of the road.

Each test boring was advanced with conventional hollow stem auger drilling techniques to the very dense cobbles and gravel deposits, at which point we utilized NW wireline core drilling techniques through the hollow stem auger to complete the advancement of the test borings through the cobbles and gravels and into the underlying formational material. The test borings were advanced with our CME-45 track mounted drilling equipment. The drilling was performed by Trautner Geotech personnel consisting of a professional engineer and geologist.

The approximate location of the test borings are shown on Figure 3.3 below. The aerial photograph used for Figure 3.3 was obtained from Google Earth. The logs of the soils encountered in our test borings are presented in Appendix A, and briefly discussed below:

Figure 3.3: Test Boring Locations



The approximate test boring locations shown above were prepared using notes taken during the field work and are intended to show the approximate test boring locations for reference purposes only. We have provided a general description of the subsurface conditions encountered in our test borings in the text below. The logs of the subsurface conditions encountered are presented in Appendix A.

Test Boring One General Subsurface Conditions

Test Boring One was advanced approximately twenty feet west of the west existing bridge abutment for the west bridge structure. The test boring was advanced in the east bound traffic lane of Bayfield Parkway. The existing asphalt pavement section encountered at the road surface consisted of approximately nine (9) inches of asphalt pavement supported over approximately nine (9) inches of aggregate base course material. Below the asphalt pavement section, at a depth of about one and one-half (1½) below the road surface, we encountered man placed embankment fill material consisting of medium stiff sandy clay soil material to a depth of about five (5) feet below the road elevation where we encountered the Animas Formation. The upper one (1) foot of the formational material consisted of hard claystone material, becoming very hard sandstone at a depth of six (6) feet below the road elevation where we initiated drilling with our NW wireline core drilling equipment.

The formational material encountered between about six (6) and twelve (12) feet below the road surface elevation consisted of moderately to highly fractured relatively coarse grained sandstone material. An unconfined compressive strength test performed on a section of rock core obtained from this layer of material exhibited a compressive strength of about 3,400 pounds per square inch (psi). A Rock Quality Designation (RQD) in the range of about fifty (50) percent was obtained for this layer of the formational material.

At depths ranging from about twelve (12) to twenty (20) feet below the road surface elevation we encountered highly fractured inter-bedded layers of sandstone and claystone material of the Animas formation. Some layers within this zone of material were highly weathered. We obtained an unconfined compressive strength of about 30 psi for a section of core obtained from this layer of formational material. The RQD of this layer of material was very low, in the range of about zero (0) to at most twenty (20) percent.

At depths ranging from about twenty (20) feet to the bottom of the test boring at thirty-two (32) feet below the road surface elevation we encountered a drastic change in the characteristics of the formational material. We encountered very hard, gray colored, fine grained sandstone material. The RQD of this material was in the range of about ninety (90) to nearly one-hundred (100) percent, with an unconfined compressive strength of about 11,670 psi for a selection section of rock core obtained from this layer of material.

Due to the relatively impermeable nature of the formational material encountered, the water used during the core drilling operation did not dissipate, therefore we were not able to accurately measure the free water elevation in the test boring. We anticipate that free water may be encountered within fractures in the formational material at the approximate water elevation of the adjacent Los Piños River.

Test Boring Two General Subsurface Conditions

Test Boring Two was advanced approximately twenty feet east of the east existing bridge abutment for the west bridge structure. The test boring was advanced in the west bound traffic lane of Bayfield Parkway. The existing asphalt pavement section encountered at the road surface consisted of approximately six (6) inches of asphalt pavement supported over approximately ten (10) inches of aggregate base course material. Below the asphalt pavement section we encountered man placed embankment fill material consisting of medium dense to loose silty sand material to a depth of about six (6) feet below the road surface elevation where we encountered dense gravel with a silty sand soil matrix. The gravels became progressively dense to the point of refusal with our hollow stem auger at a depth of seven (7) feet below the road surface elevation, at which point we initiated core drilling techniques.

We encountered very dense gravel and cobbles to a depth of about twenty (20) feet below the road surface elevation. Based on recovery measurements, the individual particle size of the gravel and cobbles we encountered ranged from about three-quarter (3/4) to five (5) inches in diameter, however we anticipate that much larger cobbles/boulders may be encountered. Unconfined compressive strengths on sections of core obtained from the cobbles ranged from about 40,000 to 45,000 psi.

We encountered medium to coarse grained very hard formational sandstone material at a depth of about twenty (20) feet below the road surface elevation. A section of core obtained from the surface elevation of the formational sandstone material exhibited an unconfined compressive strength of about 5,700 psi.

Core drilling was terminated at about one and one-half (1½) feet into the formational material due to severe damage to the core bit associated with heavy vibrations and shifting of the drill string during advancement of the boring through the very hard overlying gravel/cobble materials. The fine-grained cohesive matrix of the gravel and cobble material was such that it prevented an accurate measurement of the subsurface free water elevation during the time of advancement of the boring. We anticipate that free water was encountered at a depth of about fifteen (15) feet below the road surface elevation based on the elevation of the ponded water below the west bridge relative to our test boring elevation.

Test Boring Three General Subsurface Conditions

Test Boring Three was advanced approximately twenty (20) feet west of the west existing bridge abutment for the east bridge structure. The test boring was advanced in the east bound traffic lane of Bayfield Parkway. The existing asphalt pavement section encountered at the road surface consisted of approximately eight (8) inches of asphalt pavement supported over approximately six (6) inches of aggregate base course material. Below the asphalt pavement section we encountered man placed embankment fill material consisting of medium dense to loose silty sand material to a depth of about eight (8) feet below the road surface elevation. Below the man placed embankment fill material we encountered loose silty sand material to a depth of about twelve (12) feet below the road surface elevation where we encountered auger refusal on very dense gravel and cobbles where we initiated core drilling techniques. Subsurface free water was encountered at a depth of about twelve (12) feet below the road surface elevation.

We encountered very dense gravel and cobbles with sand to a depth of about twenty (20) feet below the road surface elevation. Based on recovery measurements, the individual particle size of the gravel and cobbles we encountered ranged from about one (1) to twelve (12) inches in diameter. We anticipate that larger cobbles/boulders may be encountered in adjacent locations. Unconfined compressive strengths on sections of core obtained from the cobbles ranged from about 40,000 to 45,000 psi.

We encountered moderately to highly fractured shale material from a depth of about twenty (20) to twenty-four (24) feet below the road surface elevation. Recovery in the shale strata was approximately seventy-five (75) percent indicating that some of the formational material was sufficiently weathered to wash out during the coring operation. We obtained an unconfined compressive strength ranging from about 1,650 to 2,800 psi for the more competent shale material in this soil layer.

At depths ranging from about twenty-four (24) feet to the bottom of core run at about thirty-two (32) feet below the road surface elevation we encountered higher quality shale and sandstone material. The RQD for this layer of formational material was in the range of about eighty (80) percent, with an unconfined compressive strength of about 2,800 psi for a select section of rock core.

Test Boring Four General Subsurface Conditions

Test Boring Four was advanced approximately twenty (20) feet east of the east existing bridge abutment for the east bridge. The test boring was advanced in the east bound traffic lane of Bayfield Parkway. The existing asphalt pavement section encountered at the road surface consisted of approximately six (6) inches of asphalt pavement supported over approximately eight (8) inches of aggregate base course material. Below the asphalt pavement section we encountered man placed embankment fill material consisting of medium dense to very dense gravel and cobbles with a silty sand matrix to a depth of about ten (10) feet below the road surface elevation. We encountered auger refusal at a depth of about six (6) feet below the road surface elevation where we initiated core drilling techniques.

We encountered very dense gravel and cobbles with sand to a depth of about twenty-seven (27) feet below the road surface elevation. Based on recovery measurements, the individual particle size of the gravel and cobbles we encountered ranged from about one (1) to twelve (12) inches in diameter. We anticipate that larger cobbles/boulders may be encountered at adjacent locations. Unconfined compressive strengths on sections of core obtained from the cobbles ranged from about 40,000 to 45,000 psi. We estimate that subsurface free water was encountered at about twelve (12) feet below the road surface elevation.

We encountered low to moderately fractured gray sandstone material from a depth of about twenty-seven (27) feet below the road surface elevation to the bottom of the core run advanced to a depth of about thirty-two (32) feet below the road surface elevation. The RQD for this layer of formational material was in the range of about eighty (80), with an unconfined compressive strength of about 1,460 psi for a select section of rock core. The core drilling was stopped at a depth of thirty-two (32) feet below the road surface elevation due to the need to reopen both lanes of the roadway for the night.

The formational material encountered in our test borings generally classifies as a Class III Rock Mass Rating or “fair quality rock” as determined in Section 10.4.6.4 of the 2010 AASHTO LRFD Bridge Design Specifications.

As discussed in the general description of the subsurface conditions encountered, the maximum cobble/boulder diameter encountered in our test borings was approximately about twelve (12) inches in diameter. We anticipate that much larger boulders, possibly in the range of about three (3) to as much as four (4) feet in diameter may be encountered during the project construction.

The logs of the subsurface soil conditions encountered in our test borings are presented in Appendix A. The logs present our interpretation of the subsurface conditions encountered exposed in the test borings at the time of our field work. Subsurface soil and water conditions are often variable across relatively short distances. It is likely that variable subsurface soil and water conditions will be encountered during construction. Laboratory soil classifications of samples obtained may differ from field classifications.

3.4 *Site Seismic Classification*

The seismic site class as defined by the AASHTO Bridge Design Specifications, Fifth Edition, 2010, is based on some average values of select soil characteristics such as shear wave velocity, standard penetration test result values, undrained shear strength, and plasticity index.

We utilized standard penetration test results obtained during the auger drilling portion of our test borings, and estimates of standard penetration test results for portions of the borings that were advanced with our wireline core where obtaining standard penetration test results was not possible. We estimated that dense to very dense gravel and cobble deposits would yield an average standard penetration (blow count) of about N=50, and the more competent formational materials would yield an average standard penetration of about N=100 (the maximum allowable standard penetration blow count for seismic classification). Based on this information we calculated the Average Standard Penetration Test ($\bar{N}$) using Method B as detailed in Section 3.10.3.1 of the AASHTO Bridge Design Specifications.

Based on our calculations we obtained an $\bar{N}$ value equal to about 60 for Test Boring One, $\bar{N}$ value equal to about 50 for Test Boring Two, $\bar{N}$ value equal to about 20 for Test Boring Three, and $\bar{N}$ value equal to about 70 for Test Boring Four. An average $\bar{N}$ value of about 50 (based on the average $\bar{N}$ value of each test boring) was then calculated. Based on an average $\bar{N}$ value equal to 50 for all four of the test borings, we recommend that the project site be considered a **Site Class C**.

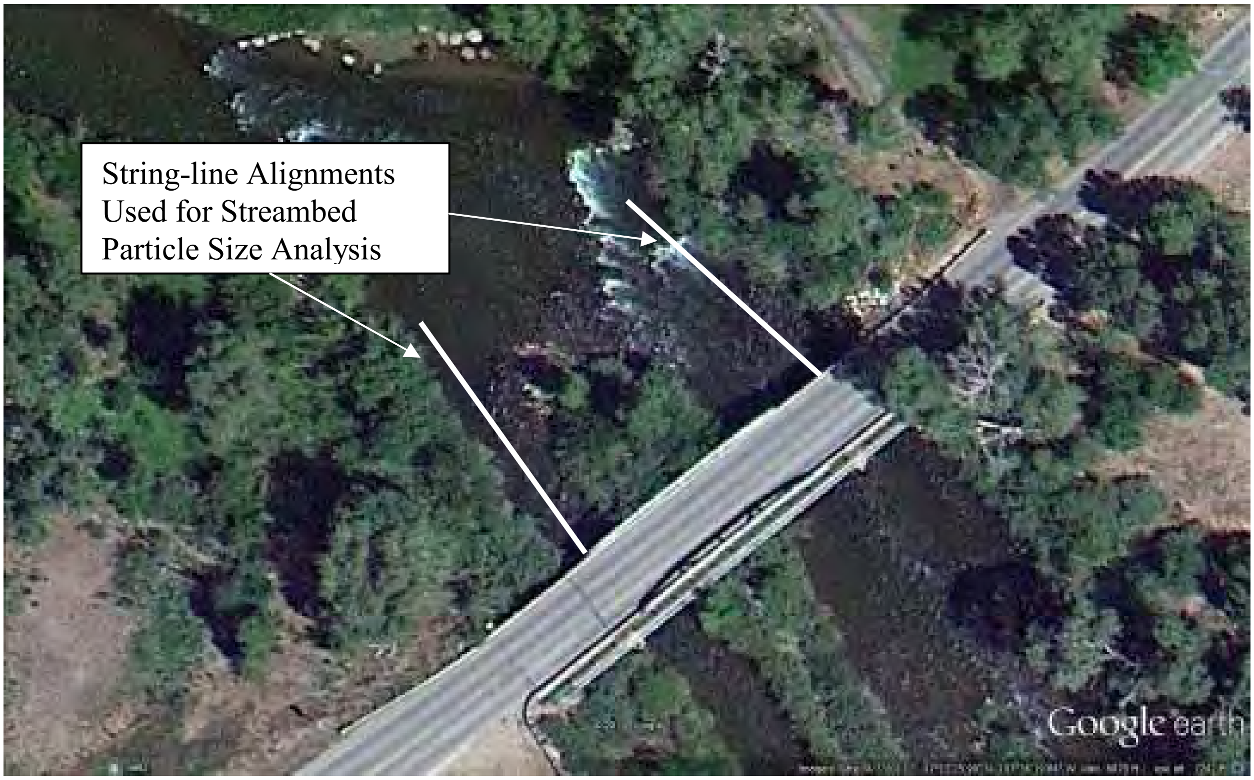
The table below present's seismic site coefficients for the project site based on a Site Class C designation in conjunction with the mapped zero period acceleration, short period acceleration, and long period acceleration. The spectral response maps and subsequent seismic site coefficients were obtained from the 2010 AASHTO Bridge Design Specifications.

Mapped Spectral Peak Ground Acceleration, PGA (Figure 3.10.2.1-1)	Mapped Spectral Short Period Acceleration S_s (Figure 3.10.2.1-2)	Mapped Spectral 1-second Acceleration S_1 (Figure 3.10.2.1-3)	Zero Period Site Coefficient F_{pga}	Short Term Period Site Coefficient F_a	Long Term Period Site Coefficient F_v
0.06g	0.13g	0.04g	1.2	1.2	1.7

Based on the product of the values obtained for F_v and S_1, the site Seismic Zone obtained from Table 3.10.6-1 is a **Seismic Zone 1**.

3.5 *Estimates of Streambed D_{50} Particle Size*

We calculated estimates of the D_{50} particle size for the streambed area adjacent to the east and west bridge locations. We performed field measurements of the streambed particle size exposed at the streambed surface for the east bridge area. The field measurements were performed by measuring the streambed particle size at two (2) foot intervals for two string-line alignments. The alignments used were located on the upstream side of the east and west existing bridge abutments, extending from these points a distance of approximately eighty (80) feet upstream. We were not able to access the central area of the stream channel due to high water flow rates at the time of our measurements. The figure below indicates the string-line alignments that we used.



Based on our field measurements, a D_{50} particle size of about eight (8) inches was calculated for the main channel streambed surface particles. The smallest particle size measured was about three-quarter (3/4) inch in diameter while the largest particle size measured was about sixteen (16) inches in diameter.

We attempted to perform field measurements of the streambed particle size for the overflow channel below the west bridge structure. This proved impractical due to heavy undergrowth in the overflow channel. Rather, we estimated the D_{50} particle size from the core that we obtained from the gravel and cobbles encountered in Test Boring Two, which was advanced approximately twenty (20) feet east of the overflow stream channel. Based on our analysis of the core obtained from the gravel and cobble materials encountered in this test boring, we estimate that the D_{50} particle size is about two (2) inches.

3.6 *Estimates of R-Value Strength Characteristics for the Existing Roadway Subgrade Materials*

Our scope of services did not include laboratory testing to assess the strength characteristics of the existing subgrade soil materials that support the existing roadway asphalt pavement section. We have provided estimates of the average R-Value for the existing subgrade soil materials encountered in our test borings based on our experience with similar soil types, and the results of sieve analysis and Atterberg Limits testing performed on the subgrade soils encountered in Test Borings Two and Three.

The subgrade soil materials encountered in Test Boring One, advanced to the west of the west bridge consist of sandy clay soil materials and likely exhibit relatively low strength characteristics. We anticipate that the R-Value for the clay soil materials encountered in this test boring is about five (5). We encountered sandy silt soil material at the pavement subgrade elevation in Test Borings Two and Three. This material classifies as USCS type “SM to ML” material or AASHTO type “A-2-4” material. We anticipate that the R-Value for this material, if compacted to at least ninety (90) percent of the maximum dry density as defined by AASHTO T-180, would be in the range of about ten (10) to fifteen (15). The subgrade soil materials encountered in Test Boring Four consist of dense gravel and cobbles that will exhibit a relatively high R-Value.

Based on the average grading characteristics of the subgrade soil materials encountered in all four (4) of our test borings, we feel that an estimated R-Value of about ten (10) may be used for cursory pavement section thickness design calculations. This estimated value is applicable for the existing site subgrade soil materials that support the existing roadway asphalt pavement section. We are available to obtain samples of the subgrade soil materials and perform laboratory testing on these materials to directly quantify the R-Value of the material.

4.0 LABORATORY STUDY

The laboratory study included tests to estimate the strength, swell and consolidation potential of the soils tested. We performed the following tests on select samples obtained from the test borings.

Moisture content and dry density; the moisture content and in-situ dry density of some of the soil samples were assessed in general accordance with ASTM D2216. The density of select sections of rock core obtained from the cobbles/boulders overlying the formational materials, and rock core obtained from the formational materials are tabulated later in this section of the report.

Atterberg Limits; the plastic limit, liquid limit and plasticity index of some of the soil samples was determined in general accordance with ASTM D4318.

Sieve Analysis Tests; We performed sieve analysis tests on select samples of soil in general accordance with ASTM D422 and/or ASTM C136, depending upon the nature of the materials sampled and tested. The primary use of the sieve analysis test, in conjunction with the Atterberg Limits is for classification and characterization of the materials tested.

Based on the results of the sieve analysis and Atterberg Limits tests, the upper portions of the existing man placed embankment fill material encountered in Test Borings Two and Three classify as AASHTO type A-2-4 or USCS type “SM” silty sand material. The results of the sieve analysis and Atterberg Limits tests are presented on Figures 4.1 and 4.2 in Appendix B.

Swell-Consolidation Tests; the one dimensional swell-consolidation potential of some of the soil samples obtained was determined in general accordance with ASTM D2435. The soil sample tested is exposed to varying loads and usually the addition of water. The one-dimensional swell-consolidation response of the soil sample to the loads and/or water is represented graphically on Figures 4.3 and 4.4 in Appendix B.

We selected a sample of the claystone material encountered at a depth of about fourteen and one-half (14½) feet below the roadway elevation in Test Boring One. The claystone material encountered and tested exhibited a measured swell pressure of about 2, 230 pounds per square foot with a measured swell potential of about four and one-half (4½) percent under a 100 pound per square foot surcharge load.

In addition, we performed swell/consolidation testing on a sample obtained from Test Boring Three at a depth of about nine (9) feet below the ground surface. The sample consisted of silty sand soil material. The sample tested did not exhibit any measurable swell potential.

Direct Shear Strength tests; Direct shear strength tests were performed on select soil samples obtained from the existing man placed embankment fill material encountered in Test Borings Two and Three. These samples were selected as we anticipate this type of material may be retained by the bridge structure retaining walls. The tests were performed in general accordance with ASTM D3080. Based on the laboratory test results, we used an angle of internal friction (phi) of 30 degrees to calculate lateral earth pressure values for the silty sand material type. The results of the direct shear tests are presented on Figures 4.5 and 4.6 in Appendix B.

Unconfined Compressive Strength*;* the unconfined compressive strength of select sections of the NWL (NQ diameter) core were tested for strength in our concrete press or unconfined soil press. The results of the unconfined compressive strength tests are presented below.

Unconfined Compressive Strength of Formational Material Core

NQ Core Boring and Depth (feet below the road surface)	Unit Weight (pcf)	Ultimate Compressive Strength (psi)	Material Description
TB-1 @ 7 feet	140.9	3,380	Coarse grained tan sandstone
TB-1 @ 13.5 feet	128.9	30	Weathered Claystone Seam
TB-1 @ 20 feet	159.2	11,670	Fine grained gray sandstone
TB-2 @ 20 feet	136.5	5,700	Coarse grained gray sandstone
TB-3 @ 22 feet	150.8	1,650	Sandy gray claystone
TB-3 @ 28 feet	155.8	2,810	Sandy gray claystone
TB-4 @ 28 feet	147.7	1,460	Gray shale

Unconfined Compressive Strength of Cobble/Boulder Material Core

NQ Core Boring and Depth (feet below the road surface)	Unit Weight (pcf)	Ultimate Compressive Strength (psi)	Material Description
TB-4 @ 8 feet	164.6	42,830	Quartzite
TB-4 @ 12 feet	166.1	45, 230	Quartzite
TB-4 @ 22 feet	164.9	42,100	Quartzite

Soluble Sulfates Tests; Water soluble sulfate concentrations were measured for existing embankment fill materials and formational materials encountered in our test borings. The testing was performed in accordance with CP-L-2103. The results of these tests for each soil strata are tabulated below.

Test Boring and Depth	**Soil Type**	**Soluble Sulfate (percent by weight of dry soil)**
TB-1, 1.5-5 feet	Sandy Clay Embankment Fill (CL)	0.020%
TB-2, 1.5-5 feet	Silty Sand Embankment Fill (SM)	<0.010%
TB-3, 1.5-4 feet	Silty Sand Embankment Fill (SM)	0.020%
TB-1 @ 14.5 feet	Claystone/Shale Formational Material	0.02%
TB-2 @ 20.5 feet	Gray Sandstone Formational Material	0.02%
TB-3 @ 21.5 feet	Sandy Claystone Formational Material	<0.010%
TB-4 @ 30 feet	Clayey Sandstone Material	0.02%

Based on Section 601.04 of the 2011 CDOT Standard Specifications for Road and Bridge Construction, and the results of the soluble sulfate testing we performed, the severity of sulfate exposure is considered as Class 0 constituting a negligible exposure to sulfate attack. However we recommend that the soils at the project sites be considered as moderately corrosive and conform to Class 1 requirements. A maximum water/cement ratio of 0.45 and either an ASTM C150 Type II or V, ASTM C595 IP(MS) or IP(HS), ASTM C 1157 Type MS or HS cement should be used.

5.0 BRIDGE FOUNDATION RECOMMENDATIONS

The proposed bridge structures should be supported by a foundation system that extends to the competent formational shale material that underlies the project site in order to resist damage to the foundation system from potential scour. We anticipate that scour calculations for the project, and CDOT requirements, may necessitate a foundation system that not only extends to the competent formational material, but extends into the competent formational material by some depth. Based on this assumption we anticipate that a deep foundation system that extends into the competent formational material will likely be required to support the bridge structures. Deep foundation systems provided in the 2010 AASHTO LRFD Bridge Design Specifications are, driven piles, drilled shafts (referred to as drilled caissons in this report), and drilled micro-piles.

We feel that a conventional spread footing foundation system that is supported by the competent formational shale material may be successfully used to support the bridge structures if the potential depth of scour is determined to be located at or near the surface of the existing formational material. Incorporating rock anchors with a spread footing foundation may be used to provide further resistance to scour of the formational material underlying the footing(s), and provide lateral resistance to movement of the foundation system in the event that the foundation system is exposed during a severe scour event. Of the various applicable foundation systems for the project, we anticipate that a conventional spread footing foundation system can likely be most easily constructed, however environmental type regulations for excavation, dewatering, and concrete placement may preclude the use of a spread footing foundation system. We have not provided design recommendations for a conventional spread footing foundation system for the project at this time. If it is determined that the potential depth of scour is at or near the surface of the formational material underlying the river channel, and the various environmental regulations for construction of a spread footing can be achieved, then we recommend that a spread footing foundation system be considered for the project. We are available to provide design recommendations for a spread footing foundation system at your request.

As discussed in the beginning of this section of the report, the various deep foundation systems provided in the 2010 AASHTO LRFD Bridge Design Specifications are, driven piles, drilled shafts (referred to as drilled caissons in this report), and drilled micro-piles.

Due to the potential for scour we do not feel that a drilled micro-pile foundation system with the micro-piles extending through the gravel and cobble soil is best suited for the project. Micro-piles generally exhibit little strength to resist lateral or moment loads due to the minimal structural cross section of the micro-pile, especially when loss of adjacent soil support due to scour is possible. Some additional lateral strength may be gained if the micro-piles are cased in relatively heavy wall steel casing, and relatively large diameter micro-piles are installed, however the resistance to lateral forces and moment loads is still typically minimal when compared to the more robust foundation systems such as driven piles or drilled caissons. If micro-piles are considered for support of the bridge structures, then the pile cap would likely need to be located at or near the most probable depth of potential scour, which we anticipate will preclude the use of this type of foundation system.

Of the various deep foundation systems available for the project, we feel that either a driven pile foundation system or drilled caisson foundation system are most applicable. Recommendations for a driven pile foundation system are provided in Section 5.1 below, and recommendations for a drilled caisson foundation system are provided in Section 5.2 below. We understand that a driven pile foundation system is preferred for the project at this time due to construction joints associated with a drilled caisson foundation system.

5.1 *Driven Piles*

We understand that a driven pile foundation system is the primary foundation type being considered for the project and that this is due to issues associated with construction joints associated with a drilled caisson foundation system. We feel that a driven pile foundation system can be successfully used for the project, however there are significant obstacles regarding the potential difficulty of successfully installing driven piles that must be considered by the project design team. These anticipated issues are listed below;

- The cobble and gravel deposits overlying the formational material were very dense in Test Borings Two through Four. The gravel and cobble soils are a clast-supported alluvial deposit, meaning that the cobbles and gravels (clasts) are primarily in direct contact with each other, as opposed to a matrix-supported deposit where finer grained soil exist between the gravel and cobbles. We anticipate that successful installation of driven piles through the cobble/gravel and potential boulder deposits will require predrilling and possibly casing of the pre-drill boring. We were not able to perform standard penetration testing within the gravel/cobble deposit due to the method of core drilling needed to advance our test borings through the very dense and very hard gravel/cobbles. An average standard penetration value of about 50 was assumed as a conservative value for seismic design criteria. However, we feel that a more accurate estimate of the standard penetration blow counts for this material could easily be 100 or greater at some depths.

- We anticipate that the chosen foundation system will likely need to be extended some depth into the formational material for scour protection. It has been our experience that it is typically only possible to drive H-Piles with reinforced tips about one (1) to two (2) feet or less into the surface of the Animas Formation. For this reason we anticipate that pre-drilling each of the pile locations may be required not only to penetrate the clast-supported gravel and cobble soils, but also to achieve the needed embedment depth into the formational material. Casing will likely be necessary for any borings that are advanced into the gravel/cobbles soils. We anticipate that these soils will likely cave into the predrill borings and accumulate within the formational material, therefore preventing driving the pile to the intended embedment depth. If it is decided that the foundation system does not need to extend into the formational material, then predrilling some of the pile locations may not be necessary.

- The casing that will likely be needed must be carefully selected in order to maintain adequate lateral support from the soil and rock materials adjacent to the pile. We anticipate that it may be necessary to grout the void between the pile and surrounding soils at some locations in order to gain sufficient lateral support of the pile. Alternatively the casing could be pulled simultaneously with the pile encasement concrete to promote an intimate contact between the pile-concrete and the adjacent gravel/cobble soils

We understand that HP12x53, HP12x74, and HP14x89 piles are being considered for the project at this time. As discussed above, we anticipate that the pile locations will require pre-drilling and casing in order to achieve adequate tip penetration of the pile into the formational material. Therefore, since the pile will likely be cased, the driven piles become considered as being point bearing on hard rock as discussed in Section 10.7.3.2.3 of the AASHTO LRFD Bridge Design Manual, and the nominal resistance of the pile is controlled by the structural limit state.

Based on the dynamic formula presented in Section 10.7.3.8.5 of the AASHTO LRFD Bridge Design Manual, an unfactored nominal axial pile resistance in excess of 500 kips per pile could be obtained assuming the pile is driven to refusal (10 blows/inch) with a pile driving hammer developing 40,000 foot-pounds per stroke. We anticipate that relatively immediate refusal will occur once the tip of the pile reaches the undisturbed formational material. We anticipate that damage to the pile could easily and rapidly occur if the potential energy of the hammer is greater than the yield stress of the pile.

We recommend that the piles be driven with an appropriately sized hammer to avoid damage to the pile. Once the tip elevation reaches the bottom of the predrilled pile boring then we recommend a maximum number of blows of five (5) blows per one-half (1/2) inch to minimize the potential for damage to the pile to occur. The bottom elevation of the predrilled pile boring must be accurately measured and carefully correlated to the tip elevation of the pile during the driving operation. The piles should be driven with high strength tip protection. We recommend that at least one (1) pile per bridge abutment be monitored with pile driving analyzer (PDA) equipment in order to verify that the needed pile capacity is achieved, and that damage to the pile does not occur at the set criteria discussed above based on the actual energy imparted to the pile by the selected hammer.

The calculated bearing resistance of piles will be dependent on the selected pile driving hammer for the project. We recommend that a resistance factor of 0.4 be applied to the unfactored nominal bearing resistance of piles if less than two (2) piles per bridge abutment are tested with PDA equipment. As presented in Table 10.5.5.2.3-1 of the AASHTO LRFD Bridge Design Manual, a resistance factor of 0.65 may be used if two (2) or more piles are tested with PDA equipment during the pile installation process. Additional reduction factors will be required for pile groups where the individual piles will be closer than three (3) pile diameters from one another.

We have provided parameters to assist in the calculation of lateral resistance of piles in Section 5.3 below. It must be understood that these parameters are only applicable if the pile can achieve full lateral contact with the surrounding soil and formational material. Since casing will likely be required for successful installation of the piles, then it may be necessary to grout the void between the pile and the casing in order to achieve full lateral contact of the pile with the surrounding soil materials. Proper selection of the casing material and diameter will improve the lateral contact that the pile makes with the surrounding soil and formational materials. We anticipate that adequate and full depth grouting of the driven piles may be very difficult to achieve. If lateral forces must be resolved by the pile foundation system then it may be necessary to achieve the needed lateral support with battered piles or to construct the piles with the casing removed simultaneously with the concrete-grout placement. Any resistance to uplift of the piles should not be considered unless full contact of the pile with the surrounding soil materials can be established.

5.2 *Drilled Caissons*

Successful installation of drilled caissons will be difficult for this project site due to the very dense and very hard nature of gravel/cobble deposits overlying the formational material. In addition, the formational material that will be encountered is very hard, with some layers exhibiting unconfined compressive strengths of up to about 11,500 psi. We anticipate that successful installation of drilled caissons on this project site will require a relatively large caisson drilling rig and a very experienced caisson drilling contractor.

As previously discussed, the largest cobble size encountered in our test borings was in the range of about twelve (12) inches, however we anticipate that much larger boulders, possibly in the range of about three (3) to four (4) feet in diameter may be encountered in some locations. The unconfined compressive strength of the quartzite cobbles/boulders encountered in our test borings was in the range of up to about 45,000 pounds per square inch. If the foundation location for the west abutment of the west bridge is placed near our test boring location (Test Boring One), then we anticipate that formational material will be encountered at relatively shallow depths and will facilitate installation of caissons at this location as the gravel and cobble materials would not be encountered.

It may be very difficult and time consuming to attempt to break down the individual cobbles and boulders during the drilling operations. Specialty equipment, such as a large down-hole hammer may be needed if the selected design includes in-placed degradation of the cobbles and boulders immediately prior to driving the piles. It will be necessary to select an adequate size auger to pick up the majority of this material on the auger flights and remove it from the boring rather than attempt to drill/break down the individual cobbles/boulders. We feel that at minimum a three (3) to five (5) foot diameter auger should be used. We anticipate that some isolated larger boulders may be encountered that will need to be broken up down-hole. It may be possible to facilitate drilling the larger boulder sized materials by predrilling some the potential boulders with smaller diameter percussion rock drilling techniques and/or incorporate small-scale down-hole blasting techniques to facilitate breaking down the larger boulders. These items should be discussed with the pier drilling contractor.

Installation of casing will be necessary during advancement of the drilled caisson borings within the gravel, cobble, and potential boulder materials overlying the formational material. The selected pier driller must have sufficient experience with installing and or advancing casing during the drilling process. The casing will likely need to extend into the formational material in order to keep the overlying gravel and cobbles from caving into the bottom of the caisson boring and to help facilitate removal of water from the boring prior to placement of the caisson concrete. The pier concrete will need to be placed with a tremmie of pumped to the bottom of the pier boring to displace the water that may accumulate in the boring during the construction operations.

The length of time and difficulty of the pier drilling operations may be significantly reduced if the caisson drilling elevation is lowered as close as possible to the formational material elevation. As previously discussed, if the foundation location for the west abutment of the west bridge is placed near our test boring location (Test Boring One), then we anticipate that formational material will be encountered at relatively shallow depths. If the caisson drilling elevation could be established at the approximate stream elevation (approximate top of subsurface water elevation), then we anticipate that the formational material would be encountered at a depth of about four (4) to five (5) feet below the caisson drilling grade in the area of the east abutment of the west bridge location, at a depth of about seven (7) to eight (8) feet below the west abutment area of the east bridge, and at a depth of about fifteen (15) feet below the east abutment of the east bridge. These depths are based on the subsurface conditions encountered at our test boring locations relative to the stream elevation as measured during the preparation of this report.

The drilled caisson borings should be advanced a minimum of two (2) caisson diameters or five (5) feet into the competent formational material. Additional embedment may be required depending on potential scour depth, lateral resistance, and uplift resistance requirements. Based on our field exploration, we feel that the upper one (1) to two (2) feet of the formational material may be weathered in some locations. The weathered formational material should not be relied on for end bearing support capacity.

The following design parameters may be used for individual caissons with a distance of at least four (4) caisson diameters center to center between adjacent caissons;

- An ultimate end bearing (tip) capacity (q_p) of 52 kips per square foot may be used provided the end bearing tip elevation of the caisson extends a minimum of at least two (2) caisson diameters or five (5) feet into the competent formational material. This ultimate end bearing capacity is based on formula 10.8.3.5.2c-2 of the 2010 AASHTO LRFD Bridge Design Specifications. An N_{60} (SPT value) of 100 was assumed for these calculations.
- Based on our review of Table 10.5.5.2.4-1 of the 2010 AASHTO LRFD Bridge Design Specifications, a resistance factor of 0.55 should be used for tip resistance based on an Intermediate Geo Material (IGM) designation for the formational shale material for LRFD design procedures.
- An ultimate side friction value (q_s) of 5 kips per square foot may be used for the portion of the caisson that extends into the competent formational material for compression and tensional forces.
- A side resistance factor of 0.6 should be used for the IGM designation of the formational shale material for LRFD design procedures.
- We anticipate that about one-quarter inch of deflection will occur prior to fully initiating both of the ultimate end bearing and side friction resistance forces.
- The granular soil materials overlying the formational material will contribute in some part to side friction capacity depending on the depth of these materials adjacent to the caisson and the method of caisson construction. We may be contacted to provide additional side friction capacity for the granular native soil materials once the caisson drilling elevation is known and the method of caisson construction (casing method used) is known.

The above design parameters are based on a minimum spacing distance of at least 4.0 caisson diameters center to center. The sum of the factored end bearing and side friction capacity should be reduced by a factor of 0.65 for a center to center spacing of 2.5 caisson diameters. The caisson spacing reduction factor may be interpolated between 0.65 and 1.0 for caisson spacing between 2.5 and 4.0 caisson diameters center to center. Caissons should not be placed closer than 2.5 caisson diameters center to center. The caisson spacing reduction factors are based on Section 10.8.3.6.3 of the 2010 AASHTO LRFD Bridge Design Specifications.

We have provided input parameters for use with LPILE computer analysis software for the various soil layers encountered in our test borings. These parameters provided in Section 5.3 below are applicable for drilled caisson foundation systems provided the foundation system makes sufficient lateral contact with the various soil material types.

The caissons should be installed using drilling equipment which is good working order and intended for advancing large diameter borings. Proper performance of the drilled caissons requires appropriate drilling and installation techniques. All drilled caissons must be installed by a contractor who is familiar with caisson construction, including casing and dewatering procedures.

Proper performance of the drilled caissons is partially influenced by the character and quality of the concrete used to construct the caisson. The caisson concrete should not be too stiff, which may prevent proper consolidation of the concrete, or too fluid, which may adversely affect the strength of the concrete. Generally the concrete should have a slump between about (3) to six (6) inches. If it is decided to use a Colorado Department of Transportation CDOT Class BZ concrete, then the CDOT specifications for slump and other specification should be followed.

It may be necessary to use mid- or high-range water reducing concrete admixtures to obtain concrete with both a suitable slump and acceptable concrete compressive strength characteristics. Use of a tremmie and/or pumping equipment should be used to place concrete in drilled caisson borings deeper than about ten (10) feet.

Water will be encountered during the caisson drilling operation, and will require dewatering prior to placement of the caisson concrete. Water that accumulates in the bottom of the caisson boring should be pumped prior to placement of the caisson concrete to within a few inches of the bottom of the caisson boring. If the water is accessing the caisson boring too rapidly to feasibly pump, then the water may be displaced by pumping concrete at the bottom of the caisson boring. When the concrete level is placed to the bottom of the caisson boring casing, sealing additional water flow from entering the boring, then the remaining excess water should be pumped from the boring.

The support elevation of the caisson must be thoroughly cleaned prior to placement of the concrete. Loose material in the bottom of the borings will cause settlement of the caisson. The caisson support elevation may be cleaned using clean-out tools attached to the drill rig, hand equipment, excavation suction equipment, or a combination of these. Under no circumstances should the caisson foundation concrete be placed when substantial loose material exists in the bottom of the borings. We recommend placing the pier steel reinforcement and concrete as soon as possible after the caisson boring has been completed to prevent soil material from caving into the caisson boring.

We anticipate that the caisson borings will need to be cased to the elevation of the formational material due to the high potential that the upper portion of the borings will cave. Additional casing of the boring into the formational material may be prudent to help seal the boring from accumulating subsurface water. The selected caisson drilling contractor must have thorough experience with boring casing procedures. It may be necessary to advance casing during penetration of the rig cutting head.

We do not feel that it is necessary to perform load testing of the caissons to substantiate the capacities provided above. However, we should be contacted during the caisson installation to;

- evaluate the drill rig specifications proposed for use in the caisson installation,
- review the concrete mix design proposed for use in the caissons,
- measure the depth of the caisson borings,
- verify the competency of the end bearing support materials,
- insure that the bottom of the caisson borings are clean prior to placement of the concrete, and,
- check the plumbness of the caisson borings.

5.3 LPILE Lateral Capacity Design Information for Drilled Caisson and Driven Pile Foundations

The LPILE parameters provided below may be utilized for lateral design of the caissons. The tables provided below present a summary of soil parameters for use with LPILE computer analysis program for the different soil strata encountered in each of our test borings.

- The referenced soil layer depths are based on the road surface elevation adjacent to our test boring at the time of the field study. The parameters should be adjusted based on the actual caisson elevation.
- LPILE soil types were obtained from LPILE version 2013 computer software.
- The undrained shear strength values are based on direct shear strength testing and unconfined compressive strength tests.
- The effective unit weight values are based on laboratory determined densities of select soil samples that we obtained during our field study.
- The LPILE “k” value, or soil modulus value was obtained from empirical data provided with the LPILE software. Obtaining project specific “k” values would require full-scale load testing of drilled piers placed on the project site.
- The LPILE values for E_{50} and k_rm were obtained from empirical data and data that we have acquired from testing of similar material types.
- We utilized empirical data for estimating Young’s Modulus.
- The rock quality designation (RQD) values are based on actual RQD measurements performed on rock core that we obtained from the test borings.
- The uniaxial compressive strength values were obtained from actual unconfined compressive strength tests that we performed.

LPILE Parameters for Test Boring One

Top of Layer (ft)	Bottom of Layer (ft)	LPILE Soil Type	Undrained Shear Strength (psf)	Angle of Internal Friction (deg)	Unit Weight (pcf)	p-y Modulus (k) (pci)	E50 (clays) K_rm (weak rock)	Young's Modulus (psi)	RQD	Uniaxial Compressive Strength (psi)
0	6	Soft Clay	500	--	125.0	--	0.01	--	--	--
6	12	Weak Rock	--	--	140.0	--	0.0005	50,000	50	3,000
12	20	Stiff Clay	2,000	--	130.0	--	0.01	--	--	--
20	32	Weak Rock	--	--	160.0	--	0.0005	50,000	80	10,000

LPILE Parameters for Test Boring Two

Top of Layer (ft)	Bottom of Layer (ft)	LPILE Soil Type	Undrained Shear Strength (psf)	Angle of Internal Friction (deg)	Unit Weight (pcf)	p-y Modulus (k) (pci)	E50 (clays) K_rm (weak rock)	Young's Modulus (psi)	RQD estimated	Uniaxial Compressive Strength (psi)
0	7	Sand (Reese)	--	30	125.0	25	--	--	--	--
7	15	Sand (Reese) (Above free water)	--	35	130.0	225	--	--	--	--
15	20	Sand (Reese) (Below free water)	--	35	130.0	125	--	--	--	--
20	21.5	Weak Rock	--	--	135.0	--	0.0005	50,000	75	5,500

LPILE Parameters for Test Boring Three

Top of Layer (ft)	Bottom of Layer (ft)	LPILE Soil Type	Undrained Shear Strength (psf)	Angle of Internal Friction (deg)	Unit Weight (pcf)	p-y Modulus (k) (pci)	E50 (clays) K_rm (weak rock)	Young's Modulus (psi)	RQD estimated	Uniaxial Compressive Strength (psi)
0	12	Sand (Reese)	--	30	125.0	25	--	--	--	--
12	20	Sand (Reese) (Below free water)	--	35	130.0	125	--	--	--	--
20	31.5	Weak Rock	--	--	135.0	--	0.0005	50,000	75	2,000

LPILE Parameters for Test Boring Four

Top of Layer (ft)	Bottom of Layer (ft)	LPILE Soil Type	Undrained Shear Strength (psf)	Angle of Internal Friction (deg)	Unit Weight (pcf)	p-y Modulus (k) (pci)	E50 (clays) K_rm (weak rock)	Young's Modulus (psi)	RQD estimated	Uniaxial Compressive Strength (psi)
0	12	Sand (Reese) (Above free water)	--	35	130.0	225	--	--	--	--
12	27.5	Sand (Reese) (Below free water)	--	35	130.0	125	--	--	--	--
27.5	32	Weak Rock	--	--	135.0	--	0.0005	50,000	75	1,500

6.0 RETAINING STRUCTURES

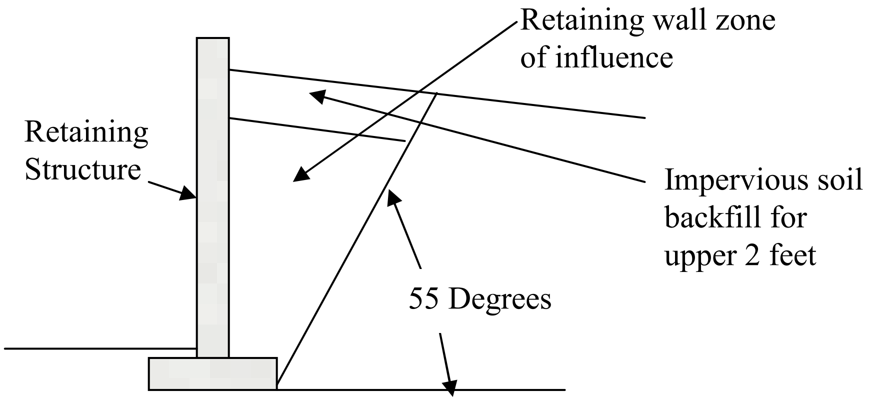
The man placed embankment fill material retained by the existing bridge abutment structure consists of a mixture of sand and gravel with a silt soil matrix. The lateral earth pressure values provided below for the “Level Existing Soil Backfill” are applicable for these soils. The lateral earth pressure values provided in the table below for “Level Imported Granular Backfill” are applicable for an imported granular material with an angle of internal friction of thirty-five (35) degrees of greater.

Lateral Earth Pressure Values

Type of Lateral Earth Pressure	Level Existing Soil Backfill (pounds per cubic foot/foot)*	Level Imported Granular Backfill (pounds per cubic foot/foot)
Active	45	35
At-rest	70	55
Passive	415	460
Allowable Coefficient of Friction	0.38	0.45

The values tabulated above are for well drained backfill soils. The values provided above do not include any forces due to adjacent surcharge loads or sloped soils. If the backfill soils become saturated the imposed lateral earth pressures will be significantly higher than those tabulated above. Exterior retaining structures may be constructed with weep holes to allow subsurface water migration through the retaining structures.

The granular imported soil backfill values tabulated above are appropriate for material with an angle of internal friction of thirty-five (35) degrees, or greater. The granular backfill must be placed within the retaining structure zone of influence as shown below in order for the lateral earth pressure values tabulated above for the granular material to be appropriate.



Retaining Structure Zone of Influence Concept, No Scale

Backfill should not be placed and compacted behind the retaining structure unless approved by the project structural engineer. Backfill placed prior to construction of all appropriate structural members such as floors, or prior to appropriate curing of the retaining wall concrete (if used) may result in severe damage and/or failure of the retaining structure.

6.1 *Considerations for Settlement of the Abutment Backfill Materials*

We understand that the bridge abutments for the west bridge structure may be placed nearer to one-another than the existing bridge configuration. Based on our review of the FIR level plans for the project, up to about twelve (12) to fifteen (15) feet of fill material will be required to establish the roadway elevation adjacent to the new abutment locations.

Settlement of the backfill material will occur regardless of the backfill material characteristics and regardless of the compaction level of the material. At minimum, we recommend that the backfill material consist of granular fill material such as a CDOT Class 2 aggregate sub-base course material with less than about fifteen (15) percent passing the #200 sieve screen. We anticipate that up to about two (2) percent post construction consolidation may occur with this type of material, even if uniformly compacted to at least ninety (90) percent of the maximum dry density as defined by AASHTO T-180. Based on a backfill depth ranging from twelve (12) to fifteen (15) feet, we anticipate that the total post construction settlement of the abutment backfill material could be in the range of about three (3) to four (4) inches. We anticipate that the post construction settlement of the abutment backfill material can be significantly reduced if the material is treated with about three (3) to five (5) percent by weight with Portland cement prior to compaction of the material. We should be contacted to provide a Portland cement treated soil mix design if this concept will be considered for the project.

Even if the backfill soil materials are treated with a Portland cement additive, we anticipate that at least one (1) to two (2) inches of settlement may occur in the abutment backfill materials and soil materials supporting the new backfill material. The roadway/bridge design should accommodate this potential for future settlement of the abutment backfill (and supported roadway) relative to the bridge abutments. We anticipate that additional asphalt cement pavement will need to be placed periodically at the interface between the bridge abutments and adjacent roadway for some time after construction of the project.

The streamside abutment backfill material, placed between the new bridge abutments and stream, should consist of a granular fill material with a maximum slope inclination of about three to one (3:1, horizontal to vertical) in order to provide adequate stability of the stream side fill materials, abutment/piles, and exterior abutment backfill material. The fill material should be compacted to at least ninety (90) percent of the maximum dry density as defined by AASHTO T-180.

7.0 CONSTRUCTION CONSIDERATIONS

This section of the report provides comments, considerations and recommendations for aspects of the site construction which may influence, or be influenced by the geotechnical engineering considerations discussed above. The information presented below is not intended to discuss all aspects of the site construction conditions and considerations that may be encountered as the project progresses. If any questions arise as a result of our recommendations presented above, or if unexpected subsurface conditions are encountered during construction we should be contacted immediately.

7.1 *Fill Placement Recommendations*

There are several references throughout this report regarding both natural soil and compacted structural fill recommendations. The recommendations presented below are appropriate for the fill placement considerations discussed throughout the report above.

All areas to receive fill, structural components, or other site improvements should be properly prepared and grubbed at the initiation of the project construction. The grubbing operations should include scarification and removal of organic material and soil. No fill material or concrete should be placed in areas where existing vegetation or fill material exist.

7.1.1 *Natural Soil Fill*

Any natural soil used for any fill purpose should be free of all deleterious material, such as organic material and construction debris. Natural soil fill includes excavated and replaced material or in-place scarified material.

The natural soils should be moisture conditioned, either by addition of water to dry soils, or by processing to allow drying of wet soils. The proposed fill materials should be moisture conditioned to between about optimum and about two (2) percent above optimum soil moisture content. This moisture content can be estimated in the field by squeezing a sample of the soil in the palm of the hand. If the material easily makes a cast of soil which remains in-tact, and a minor amount of surface moisture develops on the cast, the material is close to the desired moisture content. Material testing during construction is the best means to assess the soil moisture content.

Moisture conditioning of clay or silt soils may require many hours of processing. If possible, water should be added and thoroughly mixed into fine grained soil such as clay or silt the day prior to use of the material. This technique will allow for development of a more uniform moisture content and will allow for better compaction of the moisture conditioned materials.

The moisture conditioned soil should be placed in lifts that do not exceed the capabilities of the compaction equipment used and compacted to at least ninety (90) percent of maximum dry density as defined by ASTM D1557, modified Proctor test. We typically recommend a maximum fill lift thickness of six (6) inches for hand operated equipment and eight (8) to ten (10) inches for larger equipment. Care should be exercised in placement of utility trench backfill so that the compaction operations do not damage the underlying utilities.

Typically the maximum lift thickness is about six (6) to eight (8) inches, therefore the maximum allowable rock size for natural soil fill is about six (6) inches. If smaller compaction equipment is being used, such as walk behind compactors in trenches, the maximum rock size should be less than about three (3) inches.

7.1.2 *Granular Compacted Structural Fill*

Granular compacted structural fill is referenced in numerous locations throughout the text of this report. Granular compacted structural fill should be constructed using an imported commercially produced rock product such as aggregate road base. Many products other than road base, such as clean aggregate or select crusher fines may be suitable, depending on the intended use. If a specification is needed by the design professional for development of project specifications, a material conforming to the Colorado Department of Transportation (CDOT) “Class 6” aggregate road base material can be specified. This specification can include an option for testing and approval in the event the contractor's desired material does not conform to the Class 6 aggregate specifications. We have provided the CDOT Specifications for Class 6 material below

Grading of CDOT Class 6 Aggregate Base-Course Material	
Sieve Size	Percent Passing Each Sieve
¾ inch	100
#4	30 – 65
#8	25 – 55
#200	3 – 12

Liquid Limit less than 30

All compacted structural fill should be moisture conditioned and compacted to at least ninety (90) percent of maximum dry density as defined by ASTM D1557, modified Proctor test. Areas where the structural fill will support traffic loads under concrete slabs or asphalt concrete should be compacted to at least ninety-five (95) percent of maximum dry density as defined by ASTM D1557, modified Proctor test.

Clean aggregate fill, if appropriate for the site soil conditions, must not be placed in lifts exceeding eight (8) inches and each lift should be thoroughly vibrated, preferably with a plate-type vibratory compactor prior to placing overlying lifts of material or structural components. We should be contacted prior to the use of clean aggregate fill materials to evaluate their suitability for use on this project.

7.2 *Excavation Considerations*

Unless a specific classification is performed, the site soils should be considered as an Occupational Safety and Health Administration (OSHA) Type C soil and should be sloped and/or benched according to the current OSHA regulations. Excavations should be sloped and benched to prevent wall collapse. Any soil can release suddenly and cave unexpectedly from excavation walls, particularly if the soils are very moist, or if fractures within the soil are present. Daily observations of the excavations should be conducted by OSHA competent site personnel to assess safety considerations.

If possible excavations should be constructed to allow for water flow from the excavation the event of precipitation during construction. If this is not possible it may be necessary to remove water from snowmelt or precipitation from the foundation excavations to help reduce the influence of this water on the soil support conditions and the site construction characteristics.

7.2.1 *Excavation Cut Slopes*

We anticipate that some permanent excavation or embankment fill slopes will be included in the site development. Temporary cut slopes should not exceed five (5) feet in height and should not be steeper than about one to one (1:1, horizontal to vertical) for most soils. Permanent excavation or embankment fill slopes of greater than five (5) feet or steeper than two and one-half to one (2½:1, h:v) must be analyzed on a site specific basis.

We did not observe evidence of existing unstable slope areas influencing the site, but due to the steepness and extent of the slopes in the area we suggest that the magnitude of the proposed excavation slopes be minimized and/or supported by retaining structures.

8.0 CONSTRUCTION MONITORING AND TESTING

Construction monitoring including engineering observations and materials testing during construction is a critical aspect of the geotechnical engineering contribution to any project. Unexpected subsurface conditions are often encountered during construction. The site foundation excavation should be observed by the geotechnical engineer or a representative during the early stages of the site construction to verify that the actual subsurface soil and water conditions were properly characterized as part of field exploration, laboratory testing and engineering analysis. If the subsurface conditions encountered during construction are different than those that were the basis of the geotechnical engineering report then modifications to the design may be implemented prior to placement of fill materials or foundation concrete.

Compaction testing of fill material should be performed throughout the project construction so that the engineer and contractor may monitor the quality of the fill placement techniques being used at the site. Generally we recommend that compaction testing be performed for any fill material that is placed as part of the site development. Compaction tests should be performed on each lift of material placed in areas proposed for support of structural components. In addition to compaction testing we recommend that the grain size distribution, clay content and swell potential be evaluated for any imported materials that are planned for use on the site. Concrete tests should be performed on foundation concrete and flatwork. If asphaltic concrete is placed for driveways or aprons near the structure we are available to provide testing of these materials during placement. We are available to develop a testing program for soil, aggregate materials, concrete and asphaltic concrete for this project.

PN: 53371GE
September 4, 2014

9.0 CONCLUSIONS AND CONSIDERATIONS

The information presented in this report is based on our understanding of the proposed construction that was provided to us and on the data obtained from our field and laboratory studies. We recommend that we be contacted during the design and construction phase of this project to aid in the implementation of our recommendations. Please contact us immediately if you have any questions, or if any of the information presented above is not appropriate for the proposed site construction.

The recommendations presented above are intended to be used only for this project site and the proposed construction which was provided to us. The recommendations presented above are not suitable for adjacent project sites, or for proposed construction that is different than that outlined for this study.

Our recommendations are based on limited field and laboratory sampling and testing. Unexpected subsurface conditions encountered during construction may alter our recommendations. We should be contacted during construction to observe the exposed subsurface soil conditions to provide comments and verification of our recommendations.
We are available to review and tailor our recommendations as the project progresses and additional information which may influence our recommendations becomes available.

Please contact us if you have any questions, or if we may be of additional service.

Respectfully,
TRAUTNER GEOTECH LLC

Reviewed

JONATHAN P. BUTLER

DAVID LEE TRAUTNER

Jonathan P. Butler, P.E.
Geotechnical Engineer

David L. Trautner, P.E.
Principal Geotechnical Engineer

APPENDIX A

Logs of Test Borings

TRAUTNER GEOTECH LLC

LOG OF BORING TB-1

Twin Bridges Project
Town of Bayfield
Bechtolt Engineering, Inc.

PN: 53371GE

Field Engineer : J. Butler
Hole Diameter : 3.25 Hollow Stem/NW core
Drilling Method : Hollow Stem/NW wireline
Sampling Method : Auger/Core
Date Drilled : March 10 ,2014
Total Depth : 32 feet
Location : West Bridge
: 20' west of West abutment
: Eastbound lane
Elevation : Top of pavement

☒ Bag Sample
▨ Core Run
■ Standard Split Spoon

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Run depth	Blow Count	RECOVERY, R.Q.D., SUBSURFACE WATER
0	ASPHALT					SUBSURFACE WATER: Sixteen (16) feet estimated
1	MAN-PLACED FILL MATERIAL, AGGREGATE BASE COURSE, GRAVEL, COBBLES, sandy, clayey, dense, moist, brown	GP		☒		
3	MAN-PLACED FILL MATERIAL, CLAY, sandy, medium stiff, moist, brown	CL	▨	☒		
5	WEATHERED FORMATIONAL MATERIAL at five (5) feet, claystone, hard, tan, Animas Formation			■	10/6 20/6 20/2	
6				▨		Top of First Run at six (6) feet Recovery= 100% R.Q.D= 0%
7	FORMATIONAL MATERIAL, sandstone, moderately to highly fractured, hard, tan, Animas Formation			▨		Bottom of First Run at seven (7) feet Top of Second Run at seven (7) feet Recovery= 100% R.Q.D.= 49%
12	Interbedded sandstone and claystone, highly fractured, hard, tan and gray, Animas Formation			▨		Bottom of Second Run at twelve (12) feet Top of Third Run at twelve (12) feet Recovery= 100% R.Q.D.= 0%
17				▨		Bottom of Third Run at seventeen (17) feet Top of Fourth Run at seventeen (17) feet Recovery=100% R.Q.D.= 53%
18	Claystone, highly fractured/weathered, hard, tan, Animas Formation			▨		
20	Fine grained sandstone, moderately fractured, very hard, gray, Animas Formation			▨		
22	Fine grained sandstone, competent, very hard, gray, Animas Formation			▨		Bottom of Fourth Run at twenty-two (22) feet Top of Fifth Run at twenty-two (22) feet Recovery=100% R.Q.D.= 97%
27				▨		Bottom of Fifth Run at twenty-seven (27) feet Top of Sixth Run at twenty-seven (27) feet Recovery= 100% R.Q.D.= 100%
32	Bottom of test boring at thirty-two (32) feet					Bottom of Sixth Run at thirty-two (32) feet
33						
34						
35						

04-29-2014 T:\Current GE\53371GE, Bayfield Twin Bridges\Bayfield Twin Bridges\Logs of Test Borings\Bayfield Twin Bridges TB-1.bor

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : 3.25 Hollow Stem/NW core
Drilling Method : Hollow Stem/NW wireline
Sampling Method : Auger/Core
Date Drilled : March 11 ,2014
Total Depth : 21.5 feet
Location : West Bridge
: 20' east of East abutment
: Westbound lane
Elevation : Top of pavement

LOG OF BORING TB-2

Twin Bridges Project
Town of Bayfield
Bechtolt Engineering, Inc.

PN: 53371GE

☒ Bag Sample
▨ Core Run
■ Standard Split Spoon

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Run depth	Blow Count	RECOVERY, R.Q.D., SUBSURFACE WATER
0	ASPHALT					SUBSURFACE WATER: Fifteen (15) feet estimated
1	MAN-PLACED FILL MATERIAL, AGGREGATE BASE COURSE (four inches of asphalt millings over six inches of ABC, GRAVEL, COBBLES, sandy, clayey, dense, moist, brown	GP		☒		
2				☒		
3	MAN-PLACED FILL MATERIAL, SAND, silty, loose to medium dense, moist, dark brown	SM		☒		
4				☒		
5				■	4/6 4/6	
6	MAN-PLACED FILL MATERIAL, GRAVEL, SILT, sandy, few cobbles, medium dense, moist, brown	GM		■	3/6	
7	Auger refusal at seven (7) feet, GRAVEL, SILT, sandy, few cobbles, dense, moist, brown, generally three-quarter (3/4) to three (3) inch minus gravel recovered in core run	GM		▨		Top of First Run at six (6) feet
8				▨		
9				▨		Recovery= 40%
10	GRAVEL, COBBLES, sandy, very dense, very moist, brown, generally one and one-half (1 1/2) inch to five (5) inch minus gravel/cobble recovered in core run	GP		▨		
11				▨		
12				▨		Bottom of First Run at twelve (12) feet Top of Second Run at twelve (12) feet
13				▨		
14				▨		Recovery= 60%
15				▨		
16				▨		
17				▨		Bottom of Second Run at seventeen (17) feet Top of Third Run at seventeen (17) feet
18				▨		
19				▨		Recovery= 100%
20	FORMATIONAL MATERIAL at twenty (20) feet, sandstone, gray, Animas Formation			▨		
21				▨		Bottom of Third Run at twenty-one and one-half (21.5) feet
22	NW core refusal at twenty-one and one-half (21.5) feet (burned out bit)					
23						
24						
25						

TRAUTNER GEOTECH LLC

LOG OF BORING TB-3

Field Engineer : J. Butler
Hole Diameter : 3.25 Hollow Stem/NW core
Drilling Method : Hollow Stem/NW wireline
Sampling Method : Auger/Core
Date Drilled : March 12 ,2014
Total Depth : 31.5 feet
Location : East Bridge
: 20' west of West abutment
: Eastbound lane
Elevation : Top of pavement

Twin Bridges Project
Town of Bayfield
Bechtolt Engineering, Inc.

PN: 53371GE

☒ Bag Sample
▨ Core Run
■ Standard Split Spoon

Depth in feet	DESCRIPTION	USCS	Run depth	Blow Count	RECOVERY, R.Q.D., SUBSURFACE WATER
0	ASPHALT				SUBSURFACE WATER: Twelve (12) feet measured
1	MAN-PLACED FILL MATERIAL, AGGREGATE BASE COURSE, GRAVEL, COBBLES, sandy, clayey, dense, moist, brown	GP			
3	MAN-PLACED FILL MATERIAL, SAND, SILT, gravels, loose to medium dense, moist, brown	SM			
4			■	4/6 4/6 2/6	
9	SAND, SILT, loose, very moist, brown	SM	■	2/12	
12	GRAVEL, COBBLES, sandy, very dense, wet, brown, generally three (3) inch to eight (8) inch minus cobble in core run	GP	▨		Top of First Run at twelve (12) feet
14.5					Recovery= 67%
16.5			▨		Bottom of First Run at sixteen and one-half (16.5) feet Top of Second Run at sixteen and one-half (16.5) feet
19					Recovery= 60%
20	FORMATIONAL MATERIAL at twenty (20) feet, shale, moderately to highly fractured, hard, gray, Animas Formation				
21.5			▨		Bottom of Second Run at twenty-one and one-half (21.5) feet Top of Third Run at twenty-one and one-half (21.5) feet
24	Sandstone, low to moderately fractured, hard, gray, Animas Formation				Recovery= 73% R.Q.D.= 63%
26.5			▨		Bottom of Third Run at twenty-six and one-half (26.5) feet Top of Fourth Run at twenty-six and one-half (26.5) feet
29					Recovery=97% R.Q.D.= 79%
31.5	Bottom at thirty-one and one-half (31.5) feet				Bottom of Fourth Run at thirty-one and one-half (31.5) feet

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : 3.25 Hollow Stem/NW core
Drilling Method : Hollow Stem/NW wireline
Sampling Method : Auger/Core
Date Drilled : March 12 ,2014
Total Depth : 32 feet
Location : East Bridge
: 20' east of East abutment
: Eastbound lane
Elevation : Top of pavement

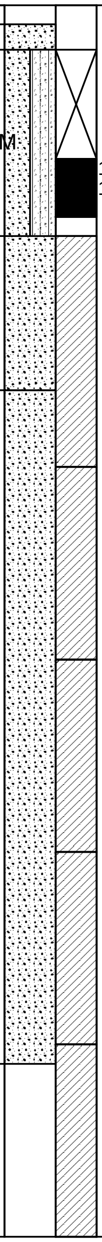
LOG OF BORING TB-4

Twin Bridges Project
Town of Bayfield
Bechtolt Engineering, Inc.

PN: 53371GE

☒ Bag Sample
▨ Core Run
■ Standard Split Spoon

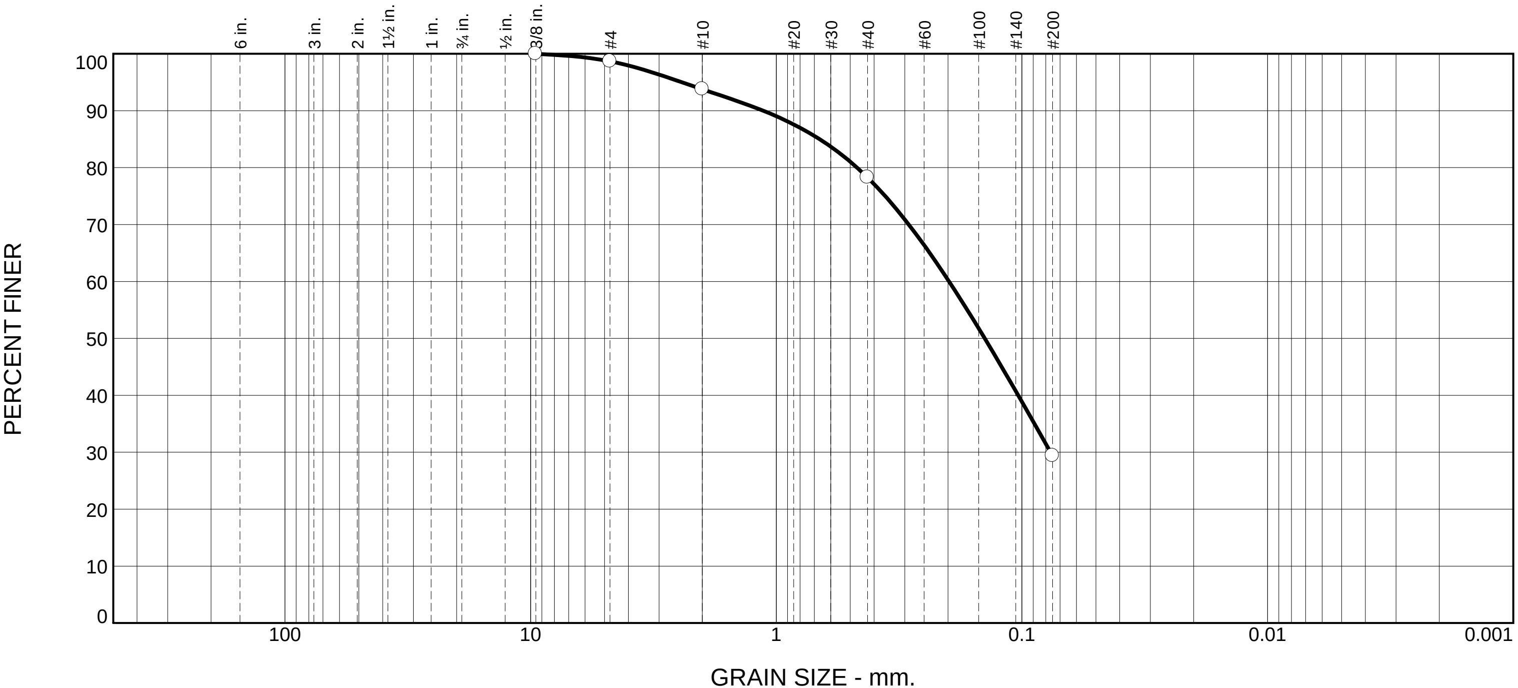
Depth in feet	DESCRIPTION	USCS	Blow Count	RECOVERY, R.Q.D., SUBSURFACE WATER
0	ASPHALT			SUBSURFACE WATER: Twelve (12) feet estimated
1	MAN-PLACED FILL MATERIAL, AGGREGATE BASE COURSE, GRAVEL, COBBLES, sandy, clayey, dense, moist, brown	GP		
3	MAN-PLACED FILL MATERIAL, GRAVEL, COBBLES, sandy, slightly siltydense, moist, brown	GP/GM	10/6 10/6 9/6	
6	Auger refusal at six (6) feet, MAN-PLACED FILL MATERIAL, GRAVEL, COBBLES, sandy, very dense, moist, brown, generally four (4) to eight (8) inch minus cobble in core runs	GP		Top of First Run at six (6) feet
				Recovery= 38%
10	GRAVEL, COBBLES, sandy, very dense, moist, brown	GP		
12				Bottom of First Run at twelve (12) feet Top of Second Run at twelve (12) feet
				Recovery= 65%
17				Bottom of Second Run at seventeen (17) feet Top of Third Run at seventeen (17) feet
				Recovery= 65%
22				Bottom of Third Run at twenty-two (22) feet Top of Fourth Run at twenty-two (22) feet
				Recovery=43%
27				Bottom of Fourth Run at twenty-seven (27) feet Top of Fifth Run at twenty-seven (27) feet
27.5	FORMATIONAL MATERIAL at twenty-seven and one-half (27.5) feet, sandstone, low to moderately fractured, hard, gray, Animas Formation			
				Recovery=92% R.Q.D.= 80%
32	Bottom of test boring at thirty-two (32) feet			Bottom of Fifth Run at thirty-two (32) feet



APPENDIX B

Laboratory Test Results

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	1	5	16	49	29	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.375	100		
#4	99		
#10	94		
#40	78		
#200	29		

* (no specification provided)

Material Description

SM - Silty sand

Atterberg Limits (ASTM D 4318)

PL= 21 **LL=** 22 **PI=** 1

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-2-4(0)

Coefficients

D_{90}=	1.1305	D_{85}=	0.6662	D_{60}=	0.1980
D_{50}=	0.1417	D_{30}=	0.0764	D_{15}=	
D_{10}=		C_u=		C_c=	

Remarks

Date Received: 3-12-14 **Date Tested:** 3-21-14

Tested By: J. Townsend

Checked By: R. Barrett

Title: Engineer Tech.

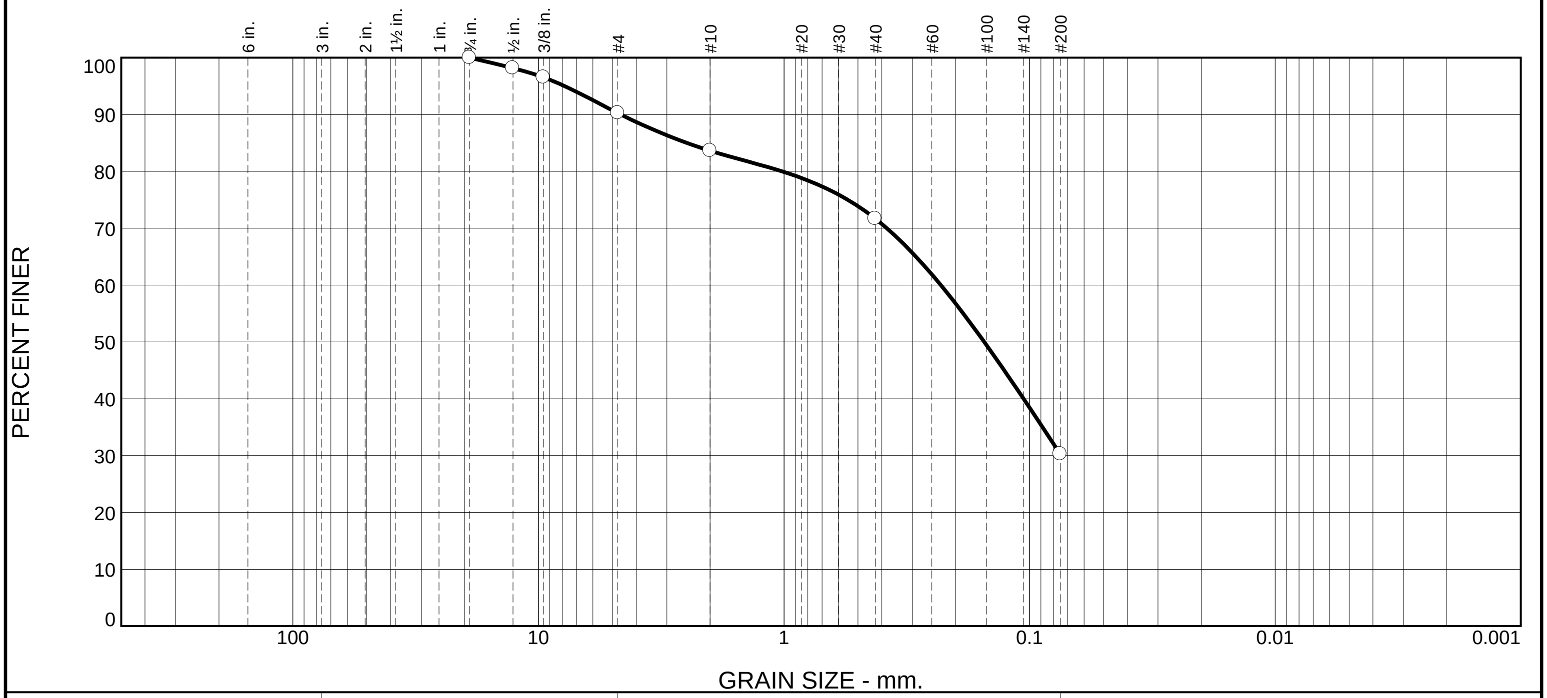
Location: Test Boring 2
Sample Number: 11404-E **Depth:** 16"-5'
Date Sampled: 3-12-14

TRAUTNER GEOTECH LLC

Client: Bechtolt Engineer-Rich Bechtolt
Project: Twin Bridges Project-Bayfield

Project No: 53371GE **Figure** 4.1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	10	6	12	42	30	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100		
.50	98		
.375	97		
#4	90		
#10	84		
#40	72		
#200	30		

* (no specification provided)

Material Description

SC-SM - Silty clayey sand

Atterberg Limits (ASTM D 4318)

PL= 20 **LL=** 24 **PI=** 4

Classification

USCS (D 2487)= SC-SM **AASHTO (M 145)=** A-2-4(0)

Coefficients

D_{90}=	4.6229	D_{85}=	2.4833	D_{60}=	0.2300
D_{50}=	0.1530	D_{30}=		D_{15}=	
D_{10}=		C_u=		C_c=	

Remarks

Date Received: 3-12-14 **Date Tested:** 3-21-14

Tested By: J. Townsend

Checked By: R. Barrett

Title: Engineer Tech.

Location: TC 3

Sample Number: 11404-G **Depth:** 14"-4'

Date Sampled: 3-12-14

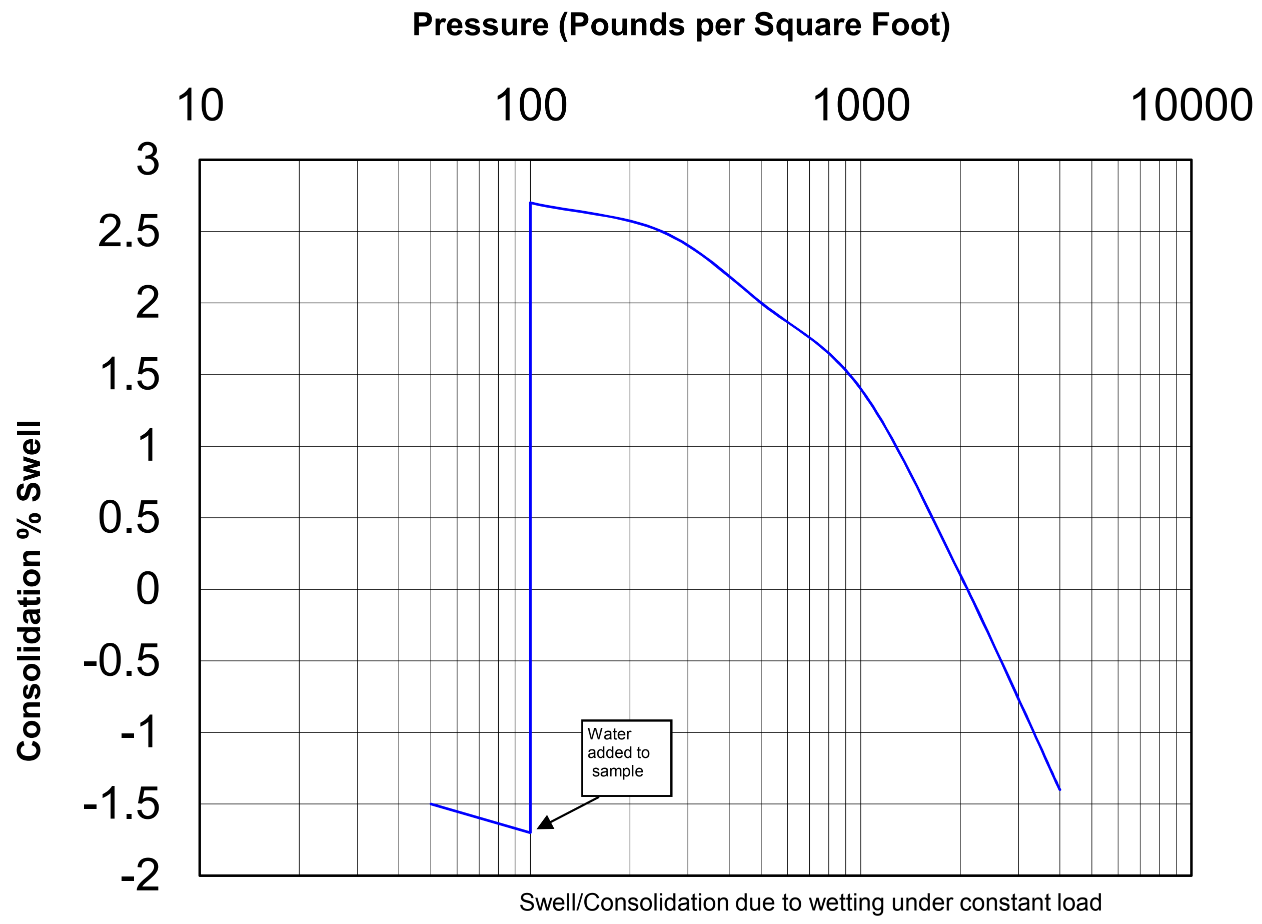
TRAUTNER GEOTECH LLC

Client: Bechtolt Engineer-Rich Bechtolt

Project: Twin Bridges Project-Bayfield

Project No: 53371GE **Figure** 4.2

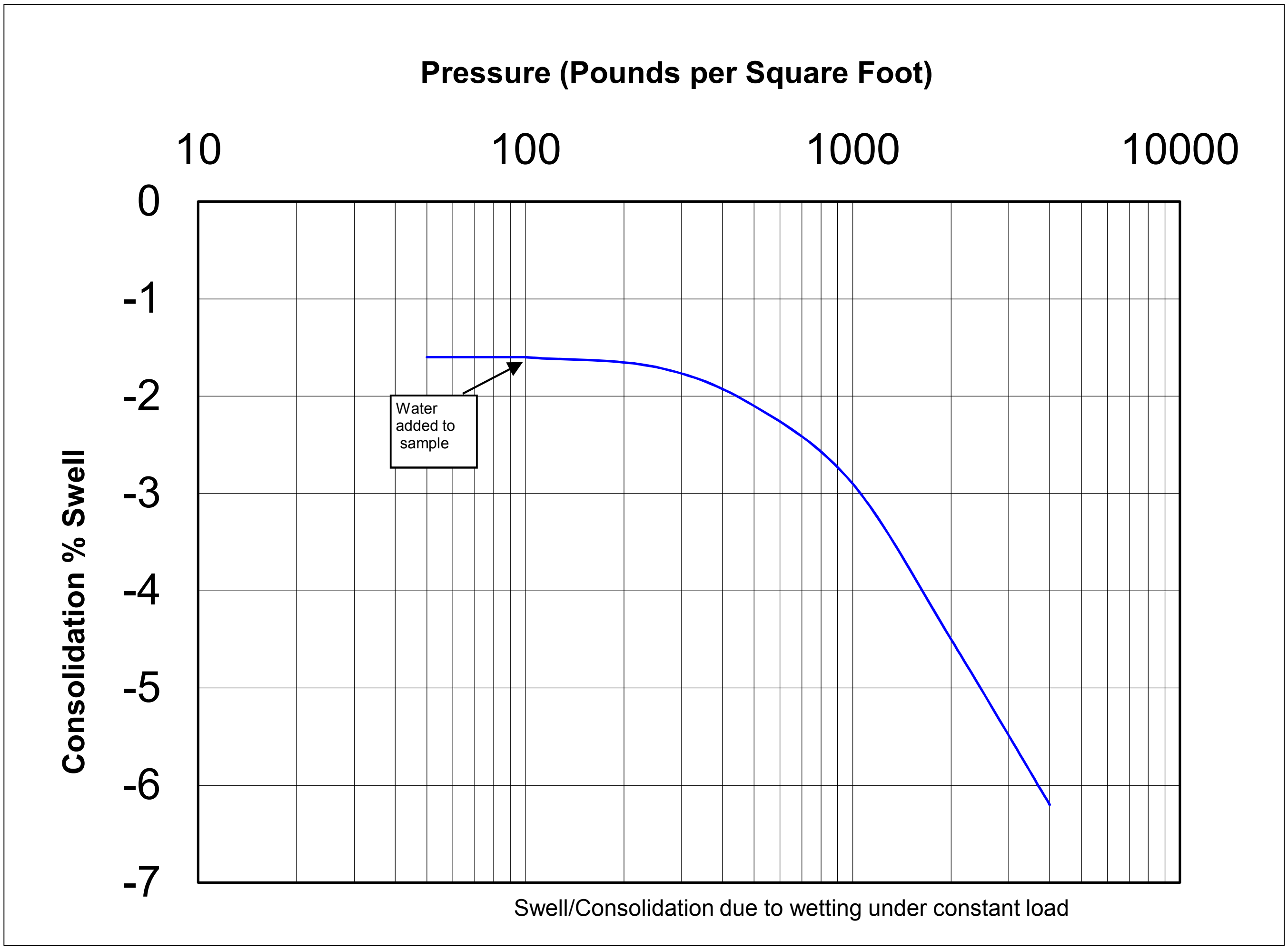
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source	TC-1@14.5'	
Soil Description	Claystone	
Swell Pressure (P.S.F)	2,230	
	Initial	Final
Moisture Content (%)	14.4	15.8
Dry Density (P.C.F)	119.9	122.3
Height (in.)	1.000	0.986
Diameter (in.)	1.94	1.94

Project Number	53371GE
Date	March 19, 2014
Figure	4.3

SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source	TC-3@9'	
Soil Description	Clayey Sand (SC)	
Swell Pressure (P.S.F)	0	
	Initial	Final
Moisture Content (%)	17.9	15.3
Dry Density (P.C.F)	109.8	119.2
Height (in.)	1.000	0.938
Diameter (in.)	1.94	1.94

Project Number	53371GE
Date	March 19, 2014
Figure	4.4

Direct Shear Test Results

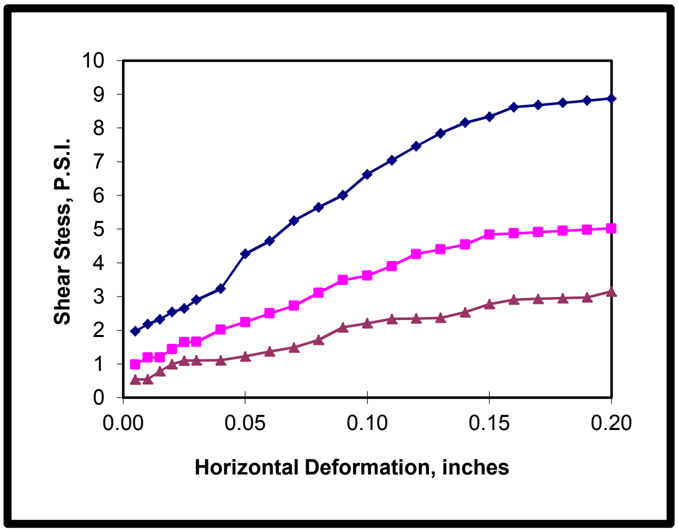
ASTM D3080-90

Project: Bayfield Twin Bridges
Project Number: 53371GE
Laboratory Number: 11404-F
Date: 4/17/2014
Project Technician: RB

Visual Soil Description: Sand (SC)
Type of Specimen: Remolded
Diameter 1.946 in.
Thickness 2.0 in
Sample Source: TB-2@5'

Summary of Sample Data:	
Initial Moisture Content (%)	14
Intial Dry Density (P.C.F)	119.1
Final Moisture Content (%)	14.5
Final Dry Density (P.C.F)	121

Residual Direct Shear Test Results:			
Normal Stress (P.S.I)	2.1	4.3	8.6
Max. Shear Stress (P.S.I)	1.7	3.1	5.6



ESTIMATED STRENGTH PARAMETERS	
Angle of Internal Friction, phi	32
Cohesion, P.S.F.	60

Figure 4.5

Direct Shear Test Results

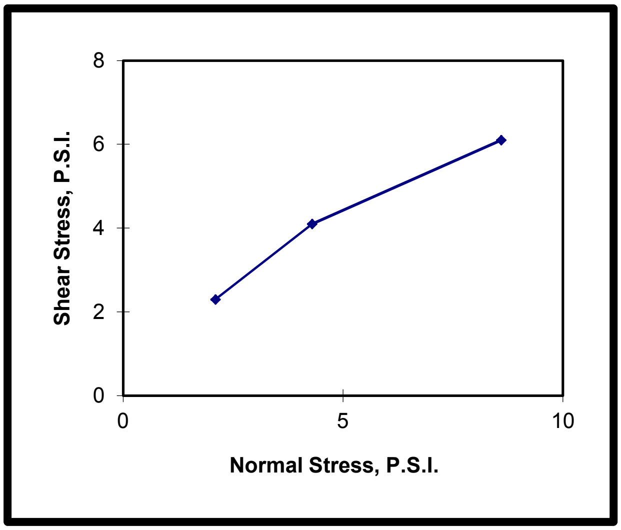
ASTM D3080-90

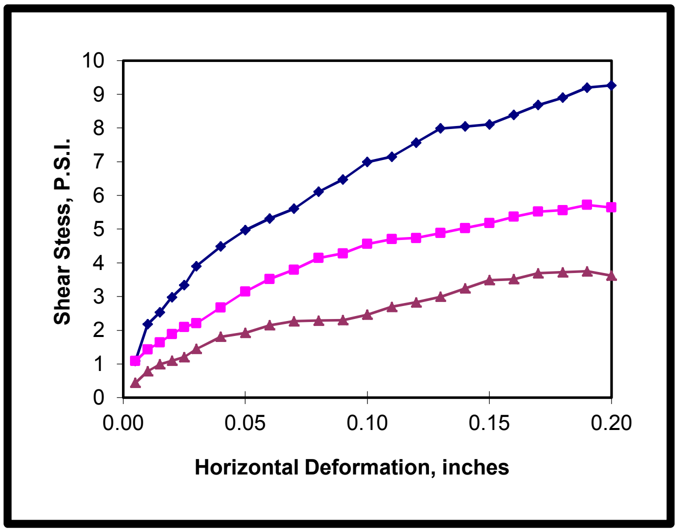
Project: Bayfield Twin Bridges
Project Number: 53371GE
Laboratory Number: 11404-J
Date: 4/17/2014
Project Technician: RB

Visual Soil Description: Silty Sand (SM)
Type of Specimen: Remolded
Diameter 1.946 in.
Thickness 2.0 in
Sample Source: TB-4@1'-4'

Summary of Sample Data:	
Initial Moisture Content (%)	1.7
Intial Dry Density (P.C.F)	117.7
Final Moisture Content (%)	n/m
Final Dry Density (P.C.F)	n/m

Residual Direct Shear Test Results:			
Normal Stress (P.S.I)	2.1	4.3	8.6
Max. Shear Stress (P.S.I)	2.3	4.1	6.1





ESTIMATED STRENGTH PARAMETERS	
Angle of Internal Friction, phi	32
Cohesion, P.S.F.	140

Figure 4.6