GEOTECHNICAL INVESTIGATION
FLEISH PENSTOCK REPLACEMENT
WASHOE COUNTY, NEVADA

PREPARED FOR:

SHAW ENGINEERING

MAY 2016
FILE: 1858

CME CONSTRUCTION MATERIALS ENGINEERS, INC.
6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

May 31, 2016
Project No: 1858

Mr. Paul Winkelman, P.E.
SHAW ENGINEERING
Reno, Nevada 89503

RE: Geotechnical Investigation
TMWA Fleish Penstock Waterline Replacement
Fleish, Washoe County, Nevada

Dear Mr. Winkelman,

Enclosed is our geotechnical investigation for the proposed TMWA Fleish Penstock Replacement project to be located near the east bank of the Truckee River, in Fleish, Washoe County, Nevada. Please review the enclosed report and provide comments.

The following report includes the results of our field and laboratory investigations and presents our recommendations for the design and construction of the project. We wish to thank you for the opportunity to provide our services and look forward to working on future endeavors together.

If you have any questions or require further information, please contact the undersigned.

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

REGISTERED PROFESSIONAL ENGINEER STATE OF NEVADA
5/31/16
RANDAL A. REYNOLDS
CIVIL
No. 8028
Exp. 12-31-17

Randal A. Reynolds, PE
Senior Geotechnical Engineer
rreynolds@cme-corp.com
Direct: 775-737-7576
Cell: 775-527-3264

Stella Montalvo, PE
Geotechnical Project Manager
smontalvo@cmenv.com
Direct: 775-737-7576

SAM:RAR:sam
V:\Active\1858\report\final report\final cv'r ltr.docx

TABLE OF CONTENTS

1.0 INTRODUCTION **1**

2.0 PROJECT DESCRIPTION **1**

2.1 PENSTOCK LOCATION 1
2.2 PENSTOCK DESCRIPTION 2

3.0 SITE CONDITIONS **3**

4.0 EXPLORATION **4**

4.1 EXPLORATORY TEST PITS 5
4.2 EXPLORATION LOCATIONS AND GROUND ELEVATIONS 6
4.3 MATERIAL CLASSIFICATION 6
4.4 WILDCAT CYNAMIC CONE PENETROMETER 6
4.5 REFRACTION MICROTREMOR (REMI) 6
4.6 REFRACTION COMPRESSIONAL WAVE MEASUREMENTS 7
4.6.1 Refraction Compressional Wave Results 7
4.7 OFF-SITE SOIL SAMPLING 8

5.0 LABORATORY TESTING **9**

5.1 INDEX TESTING 9
5.2 LABORATORY MOISTURE-DENSITY RELATIONSHSIP TEST 9
5.3 DIRECT SHEAR TEST 9
5.4 SOIL CORROSION TESTING 9

6.0 GEOLOGIC AND GENERAL SOIL CONDITIONS **9**

6.1 GENERAL SOIL AND BEDROCK CONDITIONS 9
6.1.1 West Side of Penstock 10
6.1.2 Central to East Side of Penstock 12
6.2 SOIL MOISTURE AND GROUNDWATER CONDITIIONS 13

7.0 SEISMIC HAZARDS 13

7.1 SEISMICITY 13
7.2 FAULTS 13
7.3 LIQUEFACTION 14

8.0 SEISMIC DESIGN PARAMETERS 14

9.0 DISCUSSION AND RECOMMENDATIONS 16

9.1 GENERAL INFORMATION 16
9.2 SITE PREPARATION 17
9.3 PIPELINE TRENCHING 17
9.3.1 Excavation 17
9.3.2 Trench Sidewall Stability 18
9.4 GRADING AND FILLING 18
9.5 BOTTOM OF TRENCH PREPRATION 20
9.6 PIPELINE BEDDING AND BACKFILL 20
9.6.1 Densification and Maximum Lift Thickness Requirements 21
9.7 PENSTOCK SUPPORT RECOMMENDATIONS 21
9.7.1 Modulus of Soil Reaction 21
9.8 FOUNDATIONS 24
9.8.1 Foundation Grade Soils Preparation 24

TABLE OF CONTENTS (CONTINUED)

9.8.2 Foundation Design 24
9.9 RETAINING WALL 25
9.9.1 Static Lateral Earth Pressures 25
9.9.2 Retaining Wall Drainage Recommendations 26
9.9.3 Seismically Induced Loading 27
9.10 SLOPE STABILITY AND EROSION CONTROL 29
9.10.1 Slope Stability Analysis 29
9.10.2 Erosion Potential 30
9.11 SOIL CORROSION TEST RESULTS 30
9.12 CONCRETE 31

10.0 CONSTRUCTION OBSERVATION AND TESTING SERVICES 31

11.0 STANDARD LIMITATION CLAUSE 31

REFERENCES 32

TABLES

1 - Site Classification Definitions
2 - Seismic Design Parameters
3 - Maximum Allowable Temporary Slopes
4 - Guideline Specification for General Structural Fill
5 - Guideline Specification for Pipe Bedding
6 - Modulus of Soil Reaction (E') Material Type 2 Trench Sidewall Soils (1000 psi)
7 - Modulus of Soil Reaction (E') Material Type 2 Trench Sidewall Soils (2000 psi)
8 - Modulus of Soil Reaction (E') Material Type 1 Trench Sidewall Soils
9 - Foundation Allowable Bearing Pressures
10 - Lateral Earth Pressure Values
11 - Drainage Geotextile Minimum Strength and Hydraulic Properties
12 - Pseudo Station Lateral Earth Pressure Values
13 - Soil Corrosion Test Results

Appendix A

A-1: Site Plan Showing Exploration Locations
A-2: Log of Test Pits
A-3: USCS Chart
A-4: Geologic Cross-Section

Appendix B

B-1: Particle Size Distribution and Liquid and Plastic Limits Test Report
B-2: Moisture Density Relationship Test Results
B-3: Direct Shear Test Results
B-4: Soil Corrosion Test Results

Appendix C

C-1: ReMi Geophysical Field Measurements
C-2: Refraction Geophysical Field Measurements

Appendix D

Printout USGS Seismic Design Parameters

Appendix E

Slope Stability Results

GEOTECHNICAL INVESTIGATION
TMWA Fleish Penstock Replacement
Fleish, Washoe County, Nevada

1.0 INTRODUCTION

Presented herein are the results of Construction Materials Engineers, Inc. (CME) geotechnical exploration, laboratory testing, and associated geotechnical design recommendations for the proposed TMWA Fleish Penstock Replacement located in Fleish, Washoe County, Nevada. These recommendations are based on subsurface conditions encountered in our explorations, and on details of the proposed project as described in this report. The objectives of this study were to:

- Determine general soil, bedrock and ground water conditions pertaining to design and construction of the proposed improvements.

- Provide recommendations for the design and construction of the project, as related to these geotechnical conditions.

The proposed project is contained in Sections 20 Township 19N, Range 18E MDBM. The area covered by this report is shown on Plate A-1 (Vicinity Plan) in Appendix A. Our study included field exploration, laboratory testing and engineering analyses to identify the physical and mechanical properties of the various on-site materials. Results of our field exploration and testing programs are included in this report and form the basis for all conclusions and recommendations.

2.0 PROJECT DESCRIPTION

2.1 PENSTOCK LOCATION

Fleish is located approximately 2 miles south of Verdi, Nevada, near the east bank of the Truckee River. US Interstate 80 and the UPRR tracks are located less than 500 feet west of Fleish. The penstock has a length of about 350 feet and is aligned in a near west to east direction (refer to Photo #1).

Photo #1: Looking south at existing penstock from access road.

2.2 PENSTOCK DESCRIPTION

Based on our discussion with TMWA and Shaw Engineering, it is understood that the existing 96 inch diameter steel penstock is deteriorating and will require replacement with a new steel penstock. The penstock begins at the Forebay and slopes downhill before entering the east side of the hydro-electric building.

The existing penstock crosses over the Steamboat Ditch. Currently, the penstock is supported by concrete foundations located on either side of Steamboat Ditch (refer to Photo #2). The base of these foundations are exhibiting extreme deterioration with the presence of a spalled concrete surface. Depending on the penstock alignment option chosen, these foundations may be replaced with the construction of the new penstock.

Photo #2: Looking north at existing penstock from bottom of Steamboat Ditch

An existing access road crosses over the penstock, which is located about 150 feet east of the hydro-electric building (refer to Photo #3). A mortared rubble retaining wall is located along the west side of the access road.

Photo #3: Looking north showing existing penstock descending below the access road

Two potential alignment and grade options and combination thereof are currently being considered for the penstock replacement. All options will remove the existing penstock between the forebay and turbine.

Option 1: This alignment alternative would lower the elevation of the penstock such that the 96-inch diameter steel replacement penstock is located primarily underground. This alignment will place the new penstock beneath the existing Steamboat Ditch and access road. Cuts associated with this option will be on the order of 20 feet and are dependent on the required separation distance between the top of the penstock and the bottom of Steamboat Ditch.

Option 2: Except for the segment of penstock located near the eastside of the existing access road extending to the connection point at the hydro-electric building, the new penstock will follow the existing grade and alignment of the existing penstock. The intent of this new alignment and grade is to move the penstock further east before descending beneath the existing access road. This new penstock alignment would allow the access road to be widened and reduce the slope gradient between the west edge of the access road and the hydro-electric building. Cuts and fills associated with this option will be on the order of 15 feet.

3.0 SITE CONDITIONS

The proposed penstock is located near the base of the west flank of the Carson Range within the Truckee River Corridor. This corridor area has experienced many sequences of flooding with catastrophic flooding in the late Pleistocene period. Remnants of this catastrophic flooding include glacial outwash deposits (refer to Section 6.0).

It appears that the existing penstock has been elevated in several locations by a fill embankment. The thickest portion of the fill embankment appears to exist within the eastern portion of the penstock near the Forebay (refer to photo#4).

Photo #4: Looking south showing fill embankment near penstock connection to the Forebay

Based on visual observation and reviewing the Washoe County GIS topographic survey with contour intervals of 2 feet, the terrain along the penstock is variable with gradients ranging from about 20 to 40 percent (refer to Figure #1 and photo #5).

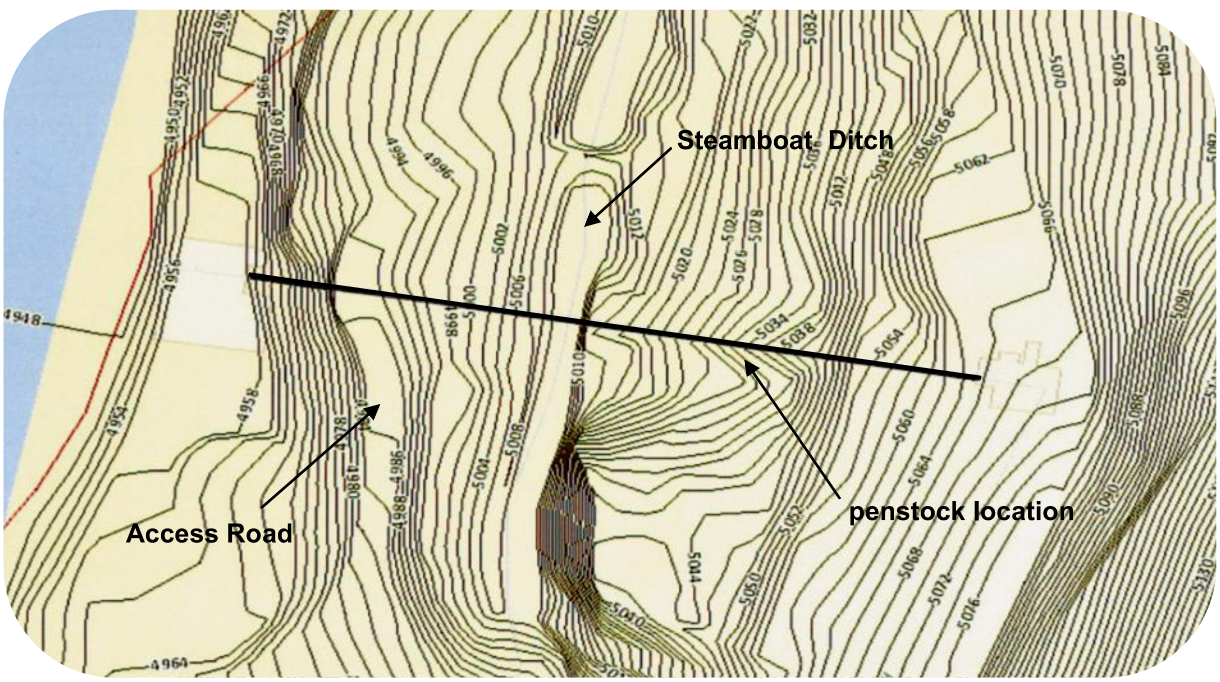


Figure #1: Washoe County GIS topographic survey showing existing penstock location

In general, the terrain between the Forebay to the access road is variable, but has an overall gradient of about 20 percent. The slope gradient between the access road to the hydro-electric building is about 40 percent. The total elevation differential between the top and bottom of the penstock is about 110 feet.

The differential elevation between the top of the access road and the finished floor elevation of the hydro-electric building is about 25 feet. The existing retaining wall along the west side of the access road has a height of about 7 feet.

Photo #5: Looking north showing existing terrain adjacent to the existing penstock east of Steamboat Ditch

Existing vegetation is heavy consisting mostly of sagebrush and grasses, other small bushes, and scattered pine trees. An overhead power line parallels the penstock alignment.

The Steamboat Ditch alignment has a near northerly direction. Below the penstock, the ditch has an approximate width of 10 feet with sidewall heights of about 5 to 6 feet.

The existing access road alignment is also in a near northerly direction above the penstock. The roadway has a width of about 15 feet.

4.0 EXPLORATION

Field exploration consisted of excavating test pits (refer to Section 4.1), Wildcat Dynamic Cone Penetrometer (DCP) (refer to Section 4.4), and geophysical testing consisting of Refraction Microtremor (ReMi) (refer to Section 4.5) and seismic compressional wave refraction studies (refer to Section 4.6).

4.1 EXPLORATORY TEST PITS

Four test pits were excavated near the penstock with a trackhoe. Two test pits were excavated on either side of the penstock immediately east of the existing access road and two test pits were excavated east of Steamboat Ditch along the north side of the penstock. Exploratory test pits were excavated to maximum depths of 18 feet below the existing ground surface.

Our geotechnical personnel logged material encountered during exploration. Representative soil samples were returned to our laboratory for testing.

4.2 EXPLORATION LOCATIONS AND GROUND ELEVATIONS

Exploration locations were determined by referencing to existing improvements and are presented on the Field Exploration Location Map: Plate A-1 in Appendix A. Ground surface elevations were determined by linear interpolation between ground contour line elevations presented on a topographic map completed with this project (Shaw Engineering) and should be considered approximate.

4.3 MATERIAL CLASSIFICATION

CME personnel examined and classified all soils in the field in general accordance with ASTM D 2488. During drilling, representative bulk samples were placed in sealed plastic bags and returned to our Reno, Nevada laboratory for testing. After the completion of laboratory testing as described in the **Laboratory Testing** section, the test pit logs were checked and corrected in accordance with ASTM 2487 (Unified Soil Classification System). Logs of the test pits are presented as Plate A-2 and a USCS chart has been included as Plate A-3 in Appendix A.

4.4 WILDCAT DYNAMIC CONE PENETROMETER

The Wildcat Dynamic Cone Penetrometer (DCP) is a portable, manually operated device used to continually measure the consistency and relative strength of loose to medium dense sandy soils and very soft to very stiff fine grained silts and clays. The DCP assembly consists of a 35 lb safety hammer with a 15 inch drop, 1 meter hollow drive rods (sounding rods), and cone tip with a nominal area of 10 cm^2. The DCP probe uses a fluid (cellulose slurry) injection system to reduce friction along the drive rods to allow the drive energy to reach the cone tip.

The dynamic cone resistance (i.e. blow counts) is recorded in the field at 10 cm intervals. The dynamic cone resistances are converted to Standard Penetration Test (SPT) N-value using the Wildcat Dynamic Cone Log software. The N-value is a measure of the standard penetration resistance of the soil and provides an indication of the relative density of the underlying soil strata.

The DCP is especially useful at remote sites not accessible to standard drilling equipment, or where test pit excavations are the proposed method of exploration.

4.5 REFRACTION MICROTREMOR (REMI)

ReMi field measurements were performed in general accordance with the method described by Louie (2001). The ReMi method provides an effective and efficient means to obtain basic subsurface profile information on an essentially continuous basis across the explored location.

The DAQlink III 24-bit acquisition system (Seismic Source/Optim) utilizing a multichannel geophone cable with twelve geophones, placed at an approximate spacing of 5 ft and 25 feet, was used to obtain surface wave data. The subsurface data was then analyzed to obtain a S-wave vertical profile. Vertical geophones with resonant frequencies of 10 Hz measure surface wave energy from broad band ambient site noise across the geophone array (i.e. ReMi setup location) for multiple 30-second iterations.

Relative elevations of the geophones were recorded in the field using a Technidea Pro-2000 ZipLevel[1]. The resulting data files were sent to Optim, Inc. for processing and analysis. SeisOpt® ReMi™ Version 4.0 software (© Optim, 2013) was used to analyze data files collected in the field. Dispersion curve picks can either be interactively modeled using trial-and-error adjustments or using an automatic inversion code to obtain a one-dimensional shear-wave (S-wave) velocity versus depth profiles. The shear-wave profile can further be calibrated and fine-tuned using any existing logs or blow counts information.

[1] It should be noted that elevation measurements taken in the field are accurate only to the degree implied by the methods used.

The results of this investigation are included as Plate C-1 (Appendix C).

4.6 REFRACTION COMPRESSIONAL WAVE MEASUREMENTS

Geophysical field measurements using the Refraction Compressional Wave (P-wave) technique was performed in general accordance with the method described in ASTM D5777. Seismic compressional wave methods provide a general shallow subsurface profile characterization.

Like the ReMi, the DAQlink III 24-bit acquisition system (Seismic Source/Optim) was used. The multichannel geophone cables with twelve geophones, placed at an approximate spacing of 5 feet, was used to obtain compressional wave data. The compressional wave data was then analyzed to obtain a vertical P-wave velocity profile. Vertical geophones with resonant frequencies of 10 Hz measure P-wave energy produced from sledge hammer blows on a strike plate completed at multiple strike locations across the geophone array. Measurements of the waves, triggered by the hammer hits on a strike plate are recorded for 0.5 seconds at 0.125 millisecond sample intervals.

Relative elevations of the geophones were recorded in the field using a Technidea Pro-2000 ZipLevel. The resulting data files were sent to Optim, Inc. for processing and analysis. SeisOpt® @2D™ Version 6.0 software (© Optim, 2013) was used to analyze data collected in the field. The program uses first-arrival travel time (the time it takes for the seismic P-wave to travel from the seismic energy source to the geophone) and survey geometry along with nonlinear optimization technique call adaptive simulated annealing (© Optim, 2013). This process tests several thousand models before converging on a model that has a distribution best matching the observed travel times (i.e. picks). The resulting model represents a 2-dimensional P-wave velocity model of the subsurface taken along the geophone alignment.

4.6.1 Refraction Compressional Wave Results

The approximate Refraction/ReMi array locations are presented on Plate A-1 (Field Exploration Location Map). A two dimensional P-wave velocity profile of the Refraction Compressional Wave is included as Plate C-2 (Appendix C). Elevations presented on the profile are relative to the elevation of geophone 1 (elevation 0 ft), located near Test Pit TP-4.

Seismic refraction measurements (P-wave) were completed on the north side of the penstock alignment between Test Pits TP-3 and TP-4. The refraction line was located approximately 10 to 15 feet north of the existing penstock.

The refraction compressional wave measurements (P-wave) are commonly used method for estimating soil/rock rippability. The average P-wave velocity is influenced by geological factors like rock hardness, stratification, fracturing, and weathering.

Compressional velocities have a direct correlation to a materials bulk density. High velocities indicate a higher bulk density. Consequently, the bedrock is more rippable as the average P-wave velocity decelerates. Caterpillar has published charts for various rock types that compare the P-wave velocities recorded with ripper performance encountered in similar materials. Caterpillar indicates that P-wave velocities on the order of 9,000 feet per second are non-rippable for granitic bedrock.

The resulting two dimensional P-wave velocity profile was analyzed in conjunction with the soil conditions encountered during our test pit exploration. P-wave velocities results on the order of 1,200 to 2,800 feet per second (ft/sec) were measured within the investigated limits of the test pits. The lower range of P-wave velocities (1,200 to 2,000 ft/sec or less) may be indicative of residual soils (near the east side of the alignment) or colluvial deposits. P-wave velocities greater than 2,000 ft/sec may be indicative of decomposed to moderately weathered granodiorite.

4.7 OFF-SITE SOIL SAMPLING

A large stockpile of excess material was generated with the excavation of the Fleish Tunnel located about 1½ miles southwest of the project site along the south side of the Truckee River. The stockpile was sampled for laboratory testing including a particle size analysis with results presented in Appendix B. Photos #6 and #7 show the stockpile.

Photo #6: Looking east at west end of soil stockpile

Photo #7: Looking at north side of stockpile area

5.0 LABORATORY TESTING

All soils testing performed in CME's soils laboratory was conducted in accordance with the standards and methodologies described in Volume 4.08 of the ASTM Standards.

5.1 INDEX TESTING

Samples of significant soil types were analyzed to determine their in-situ moisture content (ASTM D 2216), grain size distribution (ASTM D 422), and plasticity index (ASTM D 4318). The results of these tests are presented on Plate B-1 and also on the boring and test pit logs.

Results of these tests were used to classify the soils according to ASTM D 2487 and to check the field logs, which were then updated as appropriate.

5.2 LABORATORY MOISTURE-DENSITY RELATIONSHIP TEST

Moisture density relationship tests (ASTM D 1557) were performed on selected samples of native soils. This test provides a maximum dry density used to compare with the in-situ dry density of the soil to determine relative compaction. Optimum moisture content is also obtained from this test, which represents the moisture content of the soils at its maximum dry density. Results of this test were used to provide remolding dry densities and moisture contents for direct shear testing and presented on Plate B-2 - Moisture density relationship test.

5.3 DIRECT SHEAR TEST

Direct shear tests (ASTM D 3080) were performed on selected samples of native soils, screened to remove particles larger than the number 4 sieve. Tests were run on (in-situ or remolded) soil samples, saturated, and tested at three different normal pressures to derive a plot of Mohr's Circle Failure Envelope. Results of this test are shown on Plate B-3 - Direct Shear Test.

5.4 SOIL CORROSION TESTING

Silver State Analytical Laboratory completed soil corrosion testing including soluble sulfates, resistivity, pH, redox potential, and chlorides on selected samples of native soils. Test results are reported on Plate B-4 – Corrosive Soil Analysis in Appendix B of this report and are discussed in Section 9.9.

6.0 GEOLOGIC AND GENERAL SOIL CONDITIONS

Geologically, several different geologic units were encountered consisting of alluvium, colluvium, glacial outwash deposits overlying granitic bedrock with depth. Several prominent granitic outcrops are located near the penstock alignment. Based on the existing topography, it appears that portions of the existing penstock are supported by fill soils.

6.1 GENERAL SOIL AND BEDROCK CONDITIONS

The geologic profile encountered is differentiated by location along the penstock: west side and central to east side. Sections 6.1.1 and 6.1.2 present a summary of geologic conditions encountered.

6.1.1 West Side of Penstock

The west side of the penstock, adjacent to the access road, three prominent geologic units were encountered:

1) The uppermost geologic unit encountered is a fine-grained alluvium classified as a silty sand (**SM).** Few fine to coarse gravels and trace cobbles were also encountered in this soil deposit. The geologic unit was encountered to a depth of 8 to 9 feet below the existing ground surface.

2) Underlying this uppermost soil layer is a silty sand with gravel, cobbles, and boulders **(SM)** encountered to a depth of 11 to 13 feet below the existing ground surface.

3) The lowermost geologic unit encountered to the depth explored are coarse grained deposits of the Tahoe Outwash Formation. These deposits are classified as poorly graded gravels with cobbles, boulder and sand **(GP)**. Boulders encountered had a diameter of up to 3 feet. However, larger boulders may be encountered. The Tahoe Outwash Formation is a glacial outwash deposit of Pleistocene age occurring during periods of catastrophic flooding. This formation characterized as a heterogeneous mixture of sands, gravels, cobbles and boulders. Boulder-sized particles up to 16 feet in diameter have been encountered in this deposit (Bingler, 1975).

Refer to Photo #8 for trench sidewall geologic profile of Test Pit TP-1 and Photo #9 for trench sidewall geologic profile of Test Pit TP-2.

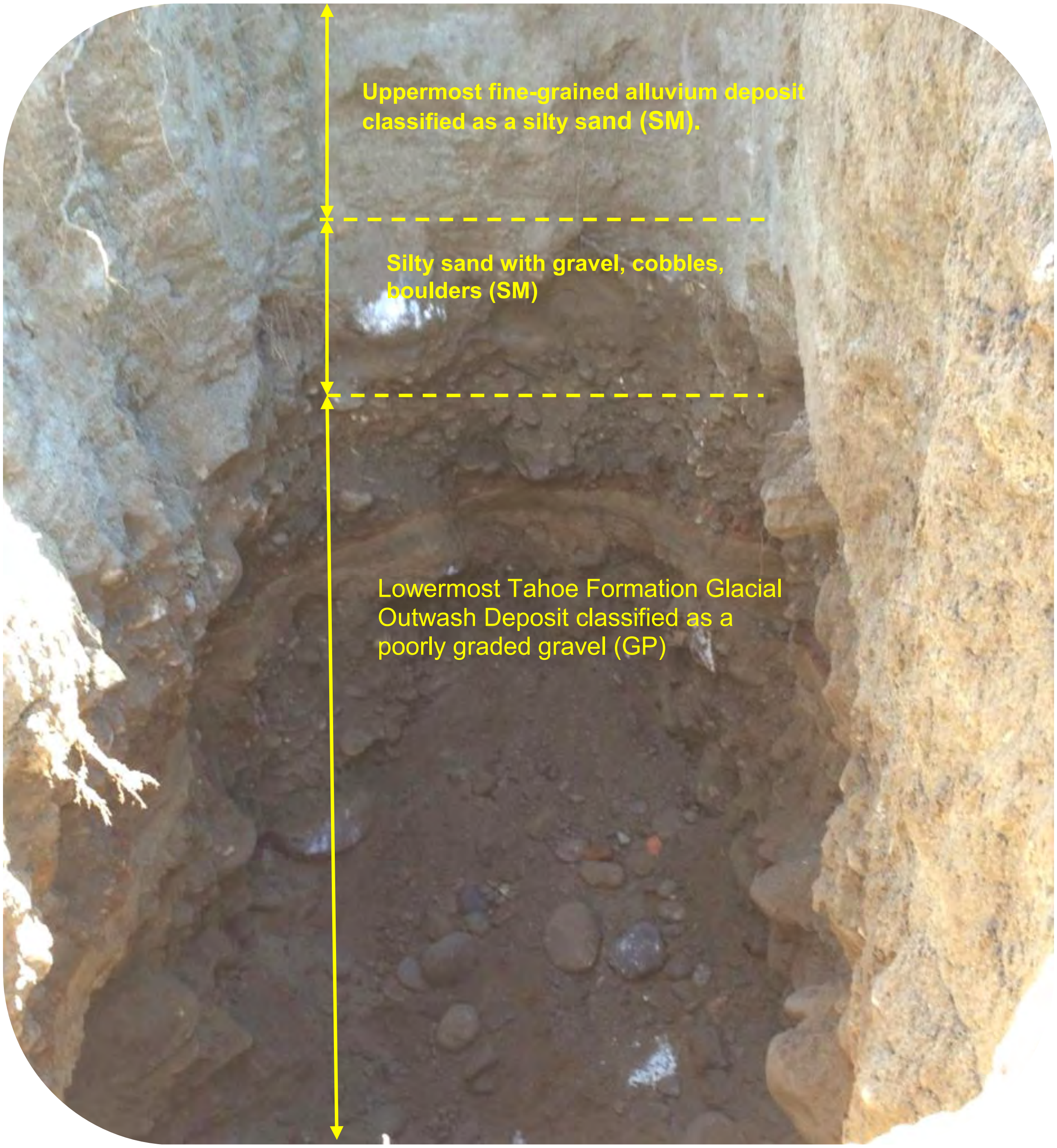


Photo # 8: Test Pit TP-1 Trench Sidewall Showing Geologic Soil Profile

Photo # 9: Test Pit TP-2 Trench Sidewall Showing Geologic Soil Profile

6.1.2 Central to East side of Penstock

The central to east side of the penstock, located between Steamboat Ditch to the Forebay, three prominent geologic units were encountered:

1) The uppermost geologic unit encountered is classified as a fine-grained alluvium classified as a silty sand **(SM).** Few fine to coarse gravels and cobbles were also encountered in this soil horizon. The geologic unit was encountered to a depth of about 7 feet below the existing ground surface.

2) Underlying this uppermost soil layer, colluvium deposits were encountered classified as a silty gravel with cobbles, boulders and sand (**GM**). These deposits were encountered to a depth ranging from 11 to 12 feet below the existing ground surface.

3) The lowermost geologic unit encountered was a granitic bedrock. In Test Pit TP-4, the granitic bedrock was deeply weathered and when excavated this bedrock has a similar soil

classification as a poorly sand with silt **(SP-SM).** In Test Pit TP-3, the granitic bedrock was moderately weathered, moderately hard to hard, and moderately strong. The trackhoe met practical refusal at a depth of about 13 feet below the existing ground surface.

Using the geologic profile determined from the exploratory test pits, a geologic cross section was drafted and is presented as Plate A-4 in Appendix A. This cross section shows prominent geologic units encountered. The geologic cross section is shown for reference only and actual depths to these geologic units between test pit locations will be determined during construction.

6.2 SOIL MOISTURE AND GROUNDWATER CONDITIONS

Groundwater was not encountered to the exploration depths completed with this investigation. Soils were generally encountered in a moist condition. Although not encountered, perched water tables are common in mountainous terrain and can arise due to changes in precipitation, seasonal variations, or other conditions not noted at the time of our investigation.

7.0 SEISMIC HAZARDS

7.1 SEISMICITY

Much of the Western United States is a region of moderate to intense seismicity related to movement of the crustal masses (plate tectonics). By far, the most active regions outside of Alaska are along the San Andreas Fault zone of western California. Other seismically active areas include the Wasatch Front in Salt Lake City, Utah, which forms the eastern boundary of the Basin and Range physiographic province, and the eastern front of the Sierra Nevada Mountains, which is the western margin of the province. The project site lies near the eastern base of the Sierra Nevada, within the western extreme of the Basin and Range.

It is generally accepted that the maximum credible earthquake in this area would be in the range of magnitude 7 to 7.5 originating from the frontal fault system of the Eastern Sierra Nevada. The most active segment of this fault system that is closest to the Reno-Stead area is located at the base of the eastern flank of the Carson Range near Thomas Creek, Whites Creek and Mt. Rose Highway, some 10 miles southeast of the project site.

7.2 FAULTS

Based on a review of the Quaternary Fault Map of Nevada-Reno (Bell, 1984) and USGS Quaternary Faults on Google Earth Map, no mapped faults trend through the project site. The closest mapped active fault is part of the Dog Valley Fault Zone located about 5 miles west of the site. The Dog Valley Fault Zone is a northeast trending, generally concealed, strike-slip fault extending from Dog Valley to Donner Lake. In 1966 a magnitude 6.0 earthquake was reported to have originated from this fault zone. Other mapped faults are located within 2 miles of the site consisting of older bedrock faults.

Quaternary earthquake fault evaluation criterion has been formulated by a professional committee for the State of Nevada Seismic Safety Council. These guidelines are consistent with the State of California Alquist-Priolo Act of 1972, which defines Holocene Active Faults as those with evidence of displacement within the past 10,000 years (Holocene time). Those faults with evidence of displacement during Pleistocene time (10,000 to 1,600,000 years before present) are classified as either late Quaternary Active Fault (10,000 to 130,000 years) or Quaternary Active Fault (> 130,000 years). Both of the latter fault designations are considered to have a decreased potential for activity compared to the Holocene Active Fault. An inactive fault is considered is a fault that does not comply with these age groups.

Based on the referenced earthquake hazard maps, the Dog Valley Fault Zone is considered a Holocene Active Fault. The other faults located near the project site are considered either Quaternary Active or inactive faults.

7.3 LIQUEFACTION

Liquefaction is a nearly a complete loss of soil shear strength that can occur during an earthquake, as cyclic shear stresses generate excessive pore water pressure between the soil grains. The higher the ground acceleration caused by a seismic event or the longer the duration of shaking, the more likely liquefaction will occur. The soil types most susceptible to liquefaction are loose to medium dense cohesionless sands, soft to stiff non-plastic to low plastic silts, or any combination of silt-sand mixtures lying below the groundwater table. Liquefaction is generally limited to depths of 50 feet or less below the existing ground surface. Based on the soil types encountered, depth of the groundwater table, and presence of shallow bedrock liquefaction potential at the site is considered minimal.

8.0 SEISMIC DESIGN PARAMETERS

Seismic design parameters are based on site-specific estimates of spectral response ground acceleration as designated in the International Building Code (IBC, 2012). The benefit of this approach is that a response spectrum can be developed from this data and, based on the period of the structure, a spectral acceleration for that structure can be determined. These values are based on two criteria: site classification and site location (latitude and longitude). Site classification is based on the substrata soil profile type, as presented in Table 1 (Site Classification Definitions).

Table 1 – Site Classification Definitions

Site Classification	Soil Profile Type Description
A	Hard Rock
B	Rock
C	Very Dense Soil and Soft Rock
D	Stiff Soil Profile
E	Soft Soil Profile
F	Soil Type Requiring Site-Specific Evaluation

The soil/bedrock profile classification is based on two criteria: density (primarily for soils based on SPT blow count data or shear wave velocity) or hardness (based on shear wave velocity primarily for bedrock sites). These two criteria have to be determined to a depth of 100 feet bgs. A shear wave velocity of 2297 ft/sec was determine with our refraction study. Based on this shear wave velocity, it is recommended that a Site Classification of C (very dense soil and soft rock) be used for structure design.

Spectral response acceleration values (S_s & S_1) are based on structures underlain by bedrock with a site classification of B. Acceleration values may amplify or attenuate depending on the subsurface geologic conditions. Therefore, the IBC provides correction factors (F_a & F_v) to modify the acceleration values if the site is located overlying subsurface geologic conditions with a site classification other than B.

Spectral response acceleration values were determined from the USGS website: *U.S. Seismic Design Maps* Table 2 provides a summary of seismic design parameters, based on 2010 ASCE 7, as referenced by IBC, including correction factors F_a & F_v. A printout of the design information including spectral response acceleration values is provided in Appendix D.

Table 2 – Seismic Design Parameters	
PARAMETER DESCRIPTION	**PENSTOCK LOCATION**
Approximate Latitude of Site	39.4810
Approximate Longitude of Site	119.9923
Peak Ground Acceleration-MCE_R PGA (ASCE 7-10 Standard)	0.500 g
Design Peak Ground Acceleration-DPGA (ASCE 7-10 Standard)	0.370 g
Spectral Response Acceleration at Short period (0.2 sec.) S_s (for Site Class B)	1.401 g
Spectral Response Acceleration at 1-second Period, S_1 (for Site Class B)	0.467 g
Site Class Selected for this Site	C
Site Coefficient F_a, decimal	1.0
Site Coefficient Fv, decimal	1.333
Design Spectral Response Acceleration at Short period, S_{Ds} (Adjusted to Site Class B, SDs= 2/3 SMs)	0.934 g
Design Spectral Response Acceleration at 1-second Period, S_{D1} (Adjusted to Site Class B, SD1=2/3 SM1)	0.415 g
1) MCE_R PGA- Maximum credible earthquake geometric mean peak ground acceleration.	

9.0 DISCUSSION AND RECOMMENDATIONS

The proposed penstock replacement project is located near the base of the western flank of the Carson Range in the Truckee River Corridor. A variety of geologic units were encountered in the penstock alignment. The two prominent uppermost geologic units consist of a fine-grained alluvium grading into either a coarse grained glacial outwash deposit or colluvium. The lowermost geologic unit encountered is granitic bedrock. The geologic units encountered will provide adequate bearing stratum for the penstock. The uppermost soil deposits could be used as either bedding or trench backfill, depending on required soil modulus, but will require processing to remove large gravels, cobbles and boulders.

The existing penstock will be removed and replaced with the new penstock pipeline having a similar alignment. However, the depth of the new penstock will be different, especially near the west end of the penstock alignment. The new penstock will be constructed by open trench methodology.

It is anticipated that the majority of the penstock alignment can be excavated with standard construction equipment consisting of a track-mounted excavator. It is anticipated that granitic bedrock, where encountered, could be excavated to design depths with a trackhoe, although the excavation may be slow and difficult in localized areas. Other specialized hydraulic equipment, such as a chipping hammer (hoe-ram) may be required.

9.1 GENERAL INFORMATION

The recommendations provided herein, and particularly under **Site Preparation, Grading and Filling,** and **Construction Observation and Testing** are intended to reduce risks of structural distress related to consolidation or expansion of native soils and/or structural fills. These recommendations, along with proper design and construction of the planned structures and associated improvements, work together as a system to improve overall performance. If any aspect of this system is ignored or poorly implemented, the performance of the project will suffer. Sufficient construction observation and testing should be performed to document that the recommendations presented in this report are followed.

Structural areas referred to in this report include all areas of concrete slabs, asphalt pavements, as well as pads for any minor structures. All compaction requirements presented in this report are relative to ASTM D 1557*. Unless otherwise stated in this report, all related construction should be in accordance with the Standard Specifications for Public Works Construction, dated 2012.

Any evaluation of the site for the presence of surface or subsurface hazardous substances is beyond the scope of this study. When suspected hazardous substances are encountered during routine geotechnical investigations, they are noted in the exploration logs and reported to the client. No such substances were identified during our exploration.

The test pits were excavated by trackhoe at the approximate locations shown on the site plan. All test pits were backfilled upon completion of the field portion of our study. The backfill was compacted to the extent possible with the equipment on hand. However, the backfill was not compacted to the requirements presented herein under **Grading and Filling**. If structures, concrete flatwork, pavement, utilities or other improvements are to be located in the vicinity of any of the test pits, the backfill should be removed and recompacted in accordance with the requirements contained in the soils report. Failure to properly compact backfill could result in excessive settlement of improvements located over test pits.

---

* Relative compaction refers to the ratio (percentage of the in-place density of a soil divided by the same soil's maximum dry density as determined by the ASTM D 1557 laboratory test procedure. Optimum moisture content is the corresponding moisture content of the same soil at its maximum dry density.

9.2 SITE PREPARATION

The existing penstock will be removed and the new penstock alignment will be graded. Excavation depths of up to 20 feet are anticipated. Refer to Trenching (Section 9.3) for excavation sidewall gradients.

All vegetation, topsoil and existing fill, should be stripped and grubbed from structural areas and removed from the site. Localized deeper stripping and grubbing to remove organic zones may be required and will be determined during construction.

The entire root bulb should be removed as part of any tree removal. Large roots (greater than 2 inches in diameter) radiating from the tree bulb area should be completely removed. Resulting excavations should be backfilled with structural fill.

All areas to receive structural fill or structural loading should be densified to at least 90 percent relative compaction in accordance with ASTM D 1557 for a minimum depth of 8 inches. It is recommended that soils have moisture contents of plus or minus 3 percent of optimum moisture (ASTM D1557) prior to densification. Moisture contents above 3 percent of optimum moisture will be acceptable if the soil horizon maintains its stability when subjected to construction equipment loads and density can be achieved in subsequent structural fill lifts. Scarification and moisture conditioning including uniform mixing of the site soils to achieve required soil moisture content recommendations may be required. It is recommended that the moisture content of the in-situ soils be determined during construction to evaluate if moisture conditioning is required. After the densification process, a firm, stable surface should be produced. Unstable soils, where encountered, should be removed and replaced with either structural fill.

The densification requirement on soils that are firm and unyielding, as determined by a representative of the geotechnical engineer, may be waived providing that all loosened material is removed to undisturbed ground.

Where less than 70 percent passes the ¾ - inch sieve, soils are too coarse for standard density testing techniques. Consequently, density is established by a proof rolling program consisting of at least five single passes with a minimum 10-ton roller in mass grading is recommended. The final surface should be smooth, firm and exhibit no signs of deflection. This alternate has proven adequate provided all other geotechnical recommendations are closely followed.

9.3 PIPELINE TRENCHING

9.3.1 Excavation

Based on the geophysical refraction study results and anticipated depth of excavation, approximately 20 feet, the bedrock within the vicinity of the planned excavation depths is considered excavatable using conventional to heavy earthwork equipment such as a track mounted excavator equipped with rock tooth bucket. However, the refraction geophysical results should be considered an overall assessment and does not preclude localized variations along the alignment including the presence of granitic corestones below the overburden soils. Corestones represent less weathered granitic bedrock inclusions, which will be more difficult to excavate. Provisions for excavation into localized areas of harder, less weathered bedrock should be established with the contractor. These excavation provisions may require the use of specialized construction equipment such as a chipping hammer (hoe-ram) placed on the end of a backhoe or trackhoe. A dozer (Caterpillar D-10 or larger) and single tooth rippers could be considered to expedite the excavation and readily allow a deeper penetration. Bedrock excavation may cause an enlargement of the trench width due to the removal of larger rock particles.

Overburden soils may contain large boulders, which may make confined excavations difficult, splitting of boulders for removal may be required.

9.3.2 Trench Sidewall Stability

Regulations amended in Part 1926, Volume 54, Number 209 of the Federal Register (Table B-1, October 31, 1989) require that the temporary sidewall slopes be no greater than those presented in Table 3.

Table 3 - Maximum Allowable Temporary Slopes

Soil or Rock Type	Maximum Allowable Slopes[1] For Excavations Less Than 20 Feet Deep[2]	
Stable Rock	Vertical	(90 degrees)
Type A[3]	3H:4V	(53 degrees)
Type B	1H:1V	(45 degrees)
Type C	3H:2V	(34 degrees)

NOTES:
1. Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.
2. Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.
3. A short-term (open 24 hours or less) maximum allowable slope of 1H:2V (63 degrees) is allowed in excavations in Type A soil that are 12 feet or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet in depth shall be 3H:4V (53 degrees).

In general, Type A soils are cohesive, non-fissured soils, with an unconfined compressive strength of 1.5 tons per square foot (tsf) or greater. Type B are cohesive soils with an unconfined compressive strength between 0.5 and 1.5 tsf, while those designated as Type C have an unconfined compressive strength below 0.5 tsf. Numerous additional factors and exclusions are included in the formal definitions. Complete definitions and requirements on sloping and benching of trench sidewalls can be found in Appendix A and B of Subpart P of the previously referenced Federal Register. Appendices C through F of Subpart P apply to requirements and methodologies for shoring.

On the basis of our exploration, it is our opinion that the bulk of the site soils appear to be predominately Type C, although variations will exist. Any area in question should be considered Type C unless specifically examined by the geological engineer during construction. All trenching should be performed and stabilized in accordance with local, state, and OSHA standards. In any case bank stability will remain the responsibility of the contractor, who is present at the site, able to observe changes in ground conditions, and has control over personnel and equipment.

9.4 GRADING AND FILLING

Three types of structural fill are anticipated for this project: pipe bedding, pipe backfill, and general structural fill.

Structural fill is defined as supporting soil placed below foundations, concrete slabs-on-grade, pavements, or any structural element that derives support from underlying soils. General Structural fill should be free of vegetation, organic matter, and other deleterious material and shall comply with the material specifications presented in Table 4.

Table 4 - Guideline Specification for General Structural Fill	
Sieve Size	**Percent by Weight Passing**
6 Inch	100
¾ Inch	70 – 100
No. 40	15 – 60
No. 200	5 – 35
Maximum Liquid Limit	**Maximum Plastic Index**
40	10
Soluble sulfates:< 0.10 percent by weight of soil	

Except for particles 6-inches or larger, native granular soils free of vegetation, organic matter, and other deleterious material will be suitable as structural fill. Native granular soils may have to be screened to remove 6-inch or larger particles. Screened cobbles and boulders could be used as rock rip-rap for slope erosion control. Particles up to 12 inches in diameter can be incorporated in fill areas, provided they are placed at least 1 foot below subgrade elevations. Material placed in the upper 1 foot of subgrade or foundation grade elevation, shall consist of structural fill containing no particles greater than 6 inches in diameter.

Structural fill should be placed in maximum 8-inch thick (loose) level lifts or layers and densified to at least 90 percent relative compaction. The required moisture content of the soils, prior to densification, shall range between plus or minus 3 percent of optimum moisture, as determined by moisture-density relationship test results (ASTM D1557). Moisture contents greater than 3 percent of optimum moisture are acceptable if the soil lift is stable and required relative compaction can be attained in the soil lift and succeeding soil lifts.

Where less than 70 percent passes the ¾-inch sieve, soils are too coarse for standard density testing techniques, and shall be referred to as a rock fill. It is anticipated that some of native granular soils contains material that meets this classification. The following construction recommendations shall be followed during the placement of rock fill material.

- A moisture-density relationship (ASTM D1557) test shall be determined on the portion of the material passing the ¾-inch sieve. Optimum moisture content determined by this test shall be used in the documentation of the in-place moisture content of the fill soils during construction. Prior to densification, the moisture content of the fraction of the rock fill passing the ¾-inch sieve should be plus or minus 3 percent of optimum. Higher moisture contents are acceptable if the soil lift is stable and required compaction can be obtained in succeeding fill lifts.

- Density shall be established by a proof rolling program consisting of at least five complete passes over the fill layer with a minimum 20 ton roller (825 Caterpillar Sheepsfoot compactor, or equivalent). Monitoring of the proof-rolling program should be provided to establish that no significant increase in measured density is occurring with subsequent passes prior to terminating compaction efforts. The rolling pattern established shall be reported and include: number of passes (each way), equipment used, and thickness of fill lift. Moisture contents should be reported as part of the construction observation and testing program. The final surface should be smooth, firm and exhibit no signs of deflection.

- Rock fill shall be placed such that nesting of the particles does not occur and voids between the rock particles are filled with a finer grained material to create a dense, homogenous mixture. Compliance with this requirement will be based on full-time observation of the grading contractor during fill placement.

- Granular soils with particles up to 12-inches in diameter can be placed in maximum 18-inch lifts.

Fill slope surfaces should be densified to the same percent compaction as the body of the fill. This may be accomplished by densifying the surface of the embankment as it is constructed or by overbuilding the fill and then excavating the slope face to its compacted core. The cut away material should be placed and compacted as outlined above rather than left at the base of the slope.

All fill soils placed on native soils/bedrock with slope gradients steeper than 5H:1V (horizontal to vertical) should be placed on horizontal benches excavated into the existing slope face, at least 8 feet in width, beginning at the toe of the slope.

Grading should not be performed with frozen soils or on frozen soils.

9.5 BOTTOM OF TRENCH PREPARATION

Native soils encountered in the bottom of the trench shall be densified in accordance with the recommendations given in Section 9.2. Bottom of trench preparation in areas with firm, unyielding soils or bedrock, as determined during construction, shall consist of removing all loose soil particles from the bottom of the trench. If soils become wet and unstable, they should be removed and replaced with structural fill.

9.6 PIPELINE BEDDING AND BACKFILL

Bedding shall directly support the pipeline and be placed along the entire circumference of the pipe. Bedding shall have a minimum thickness of 2 feet at the pipe springline and 1.5 feet at the invert and top of pipe. Bedding thickness at the bottom of the pipe could be reduced to 1 foot if overlying competent bedrock. Bedding shall be free of vegetation, organic matter, and other deleterious material and shall comply with the material specifications presented in Table 5.

Table 5 - Guideline Specification for Pipe Bedding

Sieve Size	**Percent by Weight Passing**
1 Inch	100
No. 4	85 – 100
No. 40	10 – 50
No. 200	3 – 25
Maximum Liquid Limit	**Maximum Plastic Index**
40	10
Soluble sulfates:< 0.10 percent by weight of soil	

It is intended to use native soils as bedding. Based on our particle sizing results, native soils with the exception of large gravel, cobble, and boulders sized particles complies with the requirement for a bedding sand. Native soils will be required to be processed by screening to remove particles greater than 1-inch in diameter.

The stockpile soils consisting of excess excavated material originating from the Fleish Tunnel, as described in Section 4.7, is also a source for bedding material. This material will also have to processed by screening to remove particles greater than 1-inch in diameter.

Backfill is considered material placed above the bedding layer. Native granular soils with a particle sizing of less than 12-inches and free of debris, organics, or other deleterious material can be used as backfill material. The upper 1 foot below roadway subgrade elevation shall consist of granular material with a maximum particle size of 6-inches. All backfill soils shall be tested for conformance with project specifications prior to use as a trench backfill soil.

9.6.1 Densification and Maximum Lift Thickness Requirements

Bedding shall be placed in maximum 8-inch thick (loose) lifts and densified to a minimum of 90 percent relative compaction. Backfill shall be placed in accordance with recommendations provided in Section 9.4 Grading and Filling.

It is recommended that soils have moisture contents of at least plus or minus 3 percent of optimum moisture (ASTM D1557). Higher moisture contents are acceptable if the soil lift is stable and required relative compaction can be attained in the soil lift and succeeding soil lifts.

Bedding and backfill shall not consist of frozen soils or be placed on frozen soils.

9.7 PENSTOCK SUPPORT RECOMMENDATIONS

The penstock will be supported by a bedding layer overlying native soils. It is recommended that the bedding layer conform to the material specifications and density requirements presented in Section 9.6. A short section of the penstock in the uphill side, near the Forebay, will likely be supported on fill soils.

All existing fill soils shall be removed from below the Penstock and where needed replaced with structural fill placed in accordance with recommendations provided in Section 9.4.

9.7.1 Modulus of Soil Reaction

The modulus of soil reaction **(E')** is defined as an empirical value used to express the stiffness of the embedment soils (bedding and trench sidewall soils) in predicting flexible pipe deflection. This value represents the resistance of the embedment soils to the outward movement of the pipe wall. The **E'** value is a combined modulus of soil reaction considering both the bedding soils **(E'_b)** and trench sidewall soils **(E'_n)** such that **E'** is determined by the following equation (U.S. Department of the Interior Bureau of Reclamation, 2015):

$$\mathbf{E' = S_c E'_b}$$

S_c equals a correction factor based on the ratio of E'_b to E'_n as well as the ratio between the trench width **(B)** at the pipe springline and the pipe diameter **(D)**. The correction factor S_c decreases to 1.0, as the trench width **(B)** increases in relationship to the pipe diameter **(D)** up to a maximum value of B/D=5. Additionally, as E'_n becomes closer to E'_b, the correction factor is closer to 1.0.

Both E'_n and E'_b depends on soil type and density. Higher values of **E'** are generally for granular soils with a dense to very dense relative density.

The Interior Bureau of Reclamation provides values of E'_b based on anticipated percent compaction and material types. Two types of bedding soils were considered consisting of either sands and gravels with 12% or less fines (**GW, GP SW, SP**) or sands and gravels with more than 12% fines (**GM, SM**). All bedding soils will be placed in uniform lifts and densified to at least 90 percent relative compaction (ASTM D1557). Based on this criteria, E'_b is as follows:

- Bedding Type 1 **(GW, GP SW, SP** bedding soils): 4000 psi
- Bedding Type 2 **(GM, SM** bedding soils): 2500 psi

The Interior Bureau of Reclamation provides values of **E'_n** based on anticipated percent compaction and Material Types (1, 2, or 3). Based on the field exploration results, anticipated trench soils including density, a Material Type of either 1 or 2 is recommended. If the trench springline is located in the uppermost fine-grained alluvium soils **(SM, GM** soils), a Material Type of 2 is recommended. However, if the trench springline is bearing in the glacial outwash deposits (**GP** soils) or bedrock, a Material Type of 1 is recommended. Material Type 1 has an **E'_n** of 4000 psi, while Material Type 2 has an **E'_n** of either 1000 or 2000 psi depending on trench depth. Based on our field exploration, an **E'_n** of 2000 psi can be used if the springline of the pipe is at least 6 feet below original ground.

It is assumed that the minimum trench width will be at least 1.5 times the pipe diameter, or approximately 2 feet on either side of the pipe with a D/B ratio of 1.5, but may range from 1.5 to 2.5.

Tables 6, 7, and 8 provides recommended **E'** values based on assumed **E'_n** and **E'_b** values and **D/B** ratios. Although it is anticipated that bedding produced from the native soils will be a Bedding Type 2, all the tables, for comparison, consider both bedding types (2500 or 4000 psi).

Table 6 considers Material Type 2 (**E'_n** of 1000 psi) trench sidewall soils with a pipe springline located in the uppermost soil layer to a depth of less than 6 feet below the existing ground surface.

Table 6 – Modulus of Soil Reaction (E') Material Type 2 Trench Sidewall Soils (1000 psi)[1]

E'_n	E'_b	E'_b / E'_n	D/B ratio	S_c	$E' = S_c E'_b$	$F_d^2(E')$
1000	4000	4.0	1.5	0.32	1280	770
1000	4000	4.0	2.0	0.40	1600	960
1000	4000	4.0	2.5	0.48	1920	1150
1000	2500	2.5	1.5	0.48	1200	720
1000	2500	2.5	2.0	0.56	1400	840
1000	2500	2.5	2.5	0.65	1625	975

Notes:

1. Based on pipe springline less than 6 feet below the existing ground surface.
2. **F_d** represents a correction factor of 0.6, which is applied to the modulus of soil reaction (**E').** This correction factor is recommended by the Interior Bureau of Reclamation when deflection is a critical criterion for pipeline design. If deflection can have a range of 0.5%, then a correction factor of 0.75 can be used.

Table 7 considers Material Type 2 (**E'_n** of 2000 psi) trench sidewall soils with a pipe springline located in the uppermost soil layer to a depth of greater than 6 feet below the existing ground surface.

Table 7 – Modulus of Soil Reaction (E') Material Type 2 Trench Sidewall Soils (2000 psi)[1]

E'_n	E'_b	E'_b / E'_n	D/B ratio	S_c	$E' = S_c E'_b$	F_d[2](E')
2000	4000	2.0	1.5	0.60	2400	1440
2000	4000	2.0	2.0	0.67	2680	1600
2000	4000	2.0	2.5	0.71	2840	1700
2000	2500	1.25	1.5	0.85	2125	1275
2000	2500	1.25	2.0	0.88	2200	1320
2000	2500	1.25	2.5	0.92	2300	1380

Notes:

1. Based on pipe springline at least 6 feet below the existing ground surface.
2. **F_d** represents a correction factor of 0.6, which is applied to the modulus of soil reaction (**E'**). This correction factor is recommended by the Interior Bureau of Reclamation when deflection is a critical criterion for pipeline design. If deflection can have a range of 0.5%, then a correction factor of 0.75 can be used.

Table 8 considers material Type 1 trench soils. This conditions is for the pipe springline located in either bedrock or glacial outwash deposits, which would be localized areas along the pipe alignment.

Table 8 – Modulus of Soil Reaction (E') Material Type 1 Trench Sidewall Soils[1]						
E'_n	E'_b	E'_b / E'_n	D/B ratio	S_c	$E' = S_c E'_b$	$F_d^2(E')$
4000	4000	1.0	1.5	1.0	4000	2800
4000	4000	1.0	2.0	1.0	4000	2800
4000	4000	1.0	2.5	1.0	4000	2800
4000	2500	0.67	1.5	1.32	3300	1980
4000	2500	0.67	2.0	1.21	3025	1815
4000	2500	0.67	2.5	1.14	2850	1710

Notes:

1. Based on a pipe springline below 11 feet below the existing ground surface.
2. F_d represents a correction factor of 0.7, which is applied to the modulus of soil reaction (**E')**. This correction factor is recommended by the Interior Bureau of Reclamation when deflection is a critical criterion for pipeline design. If deflection can have a range of 0.5%, then a correction factor of 1.0 can be used.

It is recommended that after the pipeline alignment has been established, the modulus of soil reaction is evaluated based on design pipeline elevations.

9.8 FOUNDATIONS

Foundations will be required for the penstock, if design option #2 is chosen and the penstock crosses over the Steamboat Ditch.

9.8.1 Foundation Grade Soils Preparation

Foundations shall bear directly on at least two feet of structural fill overlying native granular soils. Structural fill shall extend laterally at least 2 feet beyond the edge of the foundation. Native, granular soils below the structural fill should be prepared in accordance with Section 9.2 – Site Preparation. Structural fill should be prepared in accordance with Section 9.4 – Grading Filling. Foundations shall be placed at least 2 feet below the bottom of the Steamboat Ditch.

If foundation grade soils are allowed to be exposed to inclement or freezing weather conditions, becoming loose, soft, wet, frozen or disturbed, they will need to be scarified and recompacted or removed to expose suitable foundation or subgrade soils, and the resulting over-excavation backfilled with compacted structural fill. The bottom of all excavations should be dry and free of loose materials at the time of concrete placement.

9.8.2 Foundation Design

It is recommended that shallow, spread footings be used for foundation support and is the basis for our design recommendations. Provided that foundation grade soils preparation has been performed in accordance with the recommendations of Section 9.6.1, the allowable bearing pressures presented in Table 9 are recommended for the design of individual column footings.

Table 9 – Foundation Allowable Bearing Pressures	
Loading Conditions	**Maximum Soil Net Allowable Bearing Pressures[1] (pounds per square foot)**
Dead Loads plus full time live loads	2,500
Dead Loads plus live loads, plus transient wind, or seismic loads.	3,325

NOTES:
1. The net allowable bearing pressure is that pressure at the base of the footing in excess of the adjacent overburden pressure.

Foundation grade shall be at least two feet below adjacent outside grades for frost protection. Footings not located within frost prone areas should be placed at least 1 foot below surrounding ground or slab level for confinement. Regardless of loading, continuous spread foundations should be at least 18 inches wide, or as required by code.

Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. A design friction factor of 0.35 is recommended for sliding resistance at the base of the spread footing and a design value of 350 pounds per square foot per foot of depth (psf/ft) is recommended for passive soil pressures. It should be understood that some lateral deformation on the order of 2 to 4 percent of the depth of embedment (Tomlinson, 1986) for a properly compacted backfill is required to mobilize the ultimate passive pressure resistance. To reduce the amount of displacement required to develop passive pressure, a factor of safety of 1.5 was applied to the ultimate passive pressure and sliding resistance to determine their design values.

In designing for passive pressure, the upper one-foot of the soil profile should not be included unless confined by a concrete slab, or pavement. The passive pressure value is based on maintaining a nearly level surface gradient in front of the foundations with a length that is at least 3.5 times the depth of the foundation below exterior finished grade. If this gradient can't be maintained, the passive pressure resistance will be reduced. Foundation design values are based on spread footings bearing on structural fill and backfilled with structural fill.

Seismic passive pressure was determined using charts developed by log spiral procedures (Shamsabadi et al, 2007). Under seismic loading, a reduction in passive pressure will occur and a design value of 300 pounds per square foot per foot of depth is recommended.

9.9 RETAINING WALL

9.9.1 Static Lateral Earth Pressures

Static lateral earth pressures are dependent on the relative rigidity and allowable movement of the retaining structure as well as the strength properties of the backfill soil and drainage conditions behind the retaining wall. A restrained retaining wall will have a higher lateral earth pressure than a retaining wall that is free to move (cantilever conditions). Restrained retaining wall lateral earth pressure is based on the at-rest soil condition (K_o). Lateral earth pressure values for the retaining wall that is free to rotate and deflect at the top of the wall (wall movement greater than 0.001H for cohesion less soils and greater than 0.01H for cohesive soils) are based on active soil conditions (K_a).

Table 10 (Static Lateral Earth Pressure Values) provides lateral earth pressures based on the assumption that the retaining wall is backfilled with granular, non-expansive soils in accordance with the recommendations presented Section 9.4 (Grading and Filling) and conforming to the specifications in Table 4. The backfill should extend laterally behind the retaining wall at least the height of the retaining wall. Slopes behind the retaining wall have not been determined, but are assumed to be a maximum of 3H:1V. Retaining wall are also assumed to yield sufficiently to produce active soil conditions.

Table 10 – Lateral Earth Pressure Values

Wall Type	Static Lateral Earth Pressure (1,2,3)	
	Level backfill	3H:1V backfill
Assumes movement of wall face to allow full development of active pressures (K_a).	35	43

NOTES:
1) Pounds per square foot per foot of depth
2) Surcharge loads will increase lateral earth pressure and can be given upon request.
3) Assumes backfill soils are granular complying with specifications provided in Section 9.4 (Grading and Filling).

The lateral pressures presented in Table 10 assumes positive foundation drainage is provided to prevent the build-up of hydrostatic pressures and finished site drainage is provided to direct runoff away from retaining walls. To minimize hydrostatic pressures, retaining wall drainage should be constructed as an integral part of the retaining wall.

9.9.2 Retaining Wall Drainage Recommendations

Design options for retaining wall drainage are presented below:

1) If drainage can be obtained through the front of the retaining wall, weep holes could be installed near the base of the retaining wall. Weep hole sizing and spacing is dependent on the amount of drainage anticipated behind the retaining wall. A filter cover shall cover the weep holes to prevent piping and loss of backfill material. A pre-manufactured drain such as Mirafi® G100W or G100N, or approved equal is recommended. For this application, it is recommended that drain rock be used as backfill directly against the back face of the retaining wall (refer to Option 2).

2) Sub-drainage can be installed at the base of the foundation behind the retaining wall. The sub drain is comprised of a slotted non-corrosive piping system bedded in drain rock. Drain rock should be encapsulated with non-woven geotextile drainage fabric (refer to Table 11), have a thickness of at least 12 inches behind the back face of the retaining wall, and extend upward behind the retaining wall to 1 foot below finish grade. Drain rock shall meet the requirements of Section 200.03 (SSPWC, 2012) for a Class D backfill. The drain pipe should be sloped to allow the gravity flow of subsurface water to discharge locations away from the retaining wall. The discharge location should be protected from clogging by appropriate means.

3) Alternately, a pre-manufactured drainage composite, such as Mirafi® G100W (G100N), or approved equal may be installed. The drain system should extend to 1 foot below finish grade behind the retaining wall. Specific manufacturer's recommendations should be followed for application and installation of pre-manufactured drainage systems.

Table 11 – Drainage Geotextile Minimum Strength and Hydraulic Properties	
Trapezoid Tear Strength (ASTM D 4533)	80 lbs.
Puncture Strength (ASTM D 4833)	80 lbs.
Grab Strength (ASTM D 4632)	200 lbs.
Burst Strength (ASTM D 3786)	250 psi.
Minimum permittivity (ASTM D 4491)	≥ 0.2 sec^{-1}
AOS (ASTM D4751)	≤ 0.25 mm

Based on the required use of this geotextile, strength properties are based on Class 1 survivability rating (AASTHO M288). Products such as a Mirafi 180N, or approved equal can be utilized for this project.

Backfill behind the retaining wall should be densified to 90 percent relative compaction. Over-compaction should be avoided as it will increase the lateral forces exerted on the wall by the soil. Heavy equipment should not be used for placing and/or compacting backfill adjacent to the retaining wall and should be kept a minimum of three feet or at a distance determined by a1H:1V slope away from the base of the wall whichever is greater. Hand compaction equipment should be used adjacent to the wall.

9.9.3 Seismically Induced Loading

The following definitions shall be used in the analysis of seismically induced loading:

- **PGA**: Design peak ground acceleration (PGA) is based on the design earthquake ground motions (2% probability in 50 years, IBC 2012).
- **k_h**: Horizontal ground acceleration component. This component is derived from the PGA, as described in this section.
- **K_{ae}:** Seismic active earth pressure coefficient.
- **P_{AE}:** Dynamic lateral earth pressure force: $P_{AE}=0.5\gamma H^2 K_{AE}$, where γ=soil unit weight and H=height of the wall. This pressure is a combination of both static and dynamic loads such that $P_{AE}= P_a + \Delta P_{ae}$, where P_a is the static lateral pressure and ΔP_{ae} is the dynamic lateral component.

The dynamic response of most types of retaining walls is complex. Wall movements and pressures depend on the response of the soil underlying the wall; the response of the backfill; the inertial and flexural response of the wall itself; and the nature of the input motions. *Given the complex, interacting phenomena and the inherent variability and uncertainty of soil properties, it is not currently possible to accurately analyze all aspects of the seismic response of the retaining wall. As a result, models that make various simplifications about the soil, structure, and input motions are commonly used for seismic design of retaining walls* (Kramer, 1996). The standardized approach is the use of the Mononobe-Okabe method (M-O Method) that is a direct extension of the static Coulomb theory to pseudostatic conditions. In this analysis, pseudostatic accelerations are applied to a Coulomb active wedge. The pseudostatic soil thrust is then obtained from force equilibrium conditions. Using this method, K_{AE} can be determined.

Determination of k_h is based on the anticipated peak ground acceleration. The difference in determining the seismic induced loading for a yielding or restrained retaining wall is the value of the horizontal ground acceleration component.

- The horizontal ground acceleration for a yielding retaining wall is equal to 50 percent of the design PGA assuming some outward movement of the retaining wall is acceptable during an earthquake event (AASHTO, 2012).

- The horizontal ground acceleration for a restrained retaining wall is equal to the design PGA with no reduction (AASHTO, 2012).

The design peak ground acceleration is 0.37g (S_{DS}/2.5). Since site retaining walls are assumed to be yielding, a horizontal ground acceleration of 0.19 g was used to determine the seismic active earth pressure coefficient. Table 12 (Seismically Induced Lateral Earth Pressure Values) provides seismically induced earth pressure values.

Table 12– Pseudo Static Lateral Earth Pressure Values

Earth Pressure Condition	**Pseudo Static Earth Pressure Coefficient**		**Seismically Induced Equivalent Fluid Pressure[1] (psf/ft)**	**Component Earth Pressures[1] (psf/ft)** ($P_{ae}=\Delta P_{ae}+P_a$)	
Pseudo Static (assumes lateral wall displacement-active conditions)	Slope	K_{ae}[2,3]	$P_{ae} = (\gamma_{soil} * K_{ae})$[3,4]	Seismic (ΔP_{ae})	Static (P_a)
	Level	0.39	49	14	35
	3H:1V	0.61	76	33	43

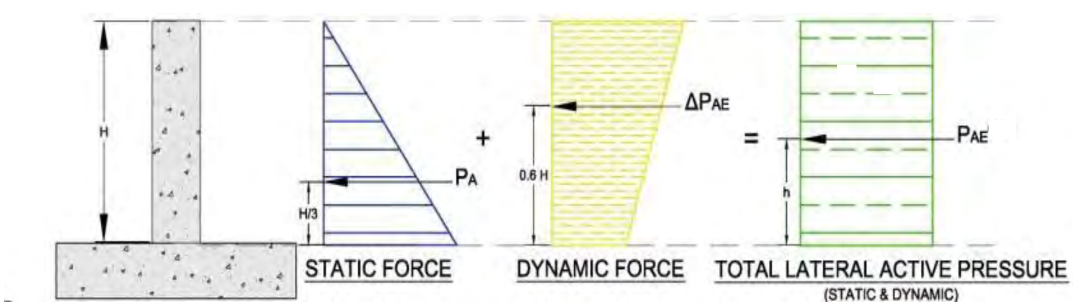
1) Pounds per square foot per lineal feet of wall. P_{ae} is the **total wall pressure** for pseudo static loading and includes both the static and seismic lateral earth pressure components. Assumes a Ø of 34^0 and γ of 125 pcf. Assumes no hydrostatic forces and no surcharge loading.

2) Based on a design peak ground acceleration (PGA) of 0.37g. It is assumed that walls will yield sufficiently to mobilize active earth pressure conditions during the design earthquake event and ½ the design peak ground acceleration is the standard for design. A horizontal seismic coefficient of 0.19 g and a vertical seismic coefficient of 0.0g, was used for design.

3) Assumes rotation of wall face to allow full development of active pressures.

4) The static and seismic resultant forces are assumed to act at heights, ranging from 0.33 H to 0.6 H, respectively, where H is the wall height. The following equation (Kramer, 1996) may be used to calculate the total wall pressure resultant force location:

$$h=\frac{P_a*(^H/_3)+\Delta P_{ae}*(0.6H)}{P_{ae}}$$



For example a 7 foot tall wall with a P_a=35 psf/ft and a ΔP_{ae}=14psf/ft would have a resultant force (P_{ae}=49 psf/ft) acting at a height (h) equal to about 2.9 feet.

9.10 SLOPE STABILITY AND EROSION CONTROL

Overall stability of cut and filled surfaces involves two separate aspects: slope stability and erosion potential.

Slope stability is related to mass wasting, landslides or the enmasse downward movement of soil or rock. Stability of cut and fill slopes depends upon shear strength, unit weight, moisture content, and slope angle. A slope stability analysis was completed for the steepest section of penstock located immediately east of the hydro-electric building. The analysis is for a cross -section of backfill soils adjacent to the pipeline and using the current slope profile with the access road shifted to the east approximately 15 feet.

9.10.1 SLOPE STABILITY ANALYSIS

The computer program ReSSA 3.0 (Adama Engineering Inc., 2001 to 2011) was utilized to perform slope stability analyses. This program performs a two dimensional limit equilibrium analysis to compute the factor of safety (FOS) for a layered slope. The limit equilibrium analysis was performed using the simplified Bishop method. This method satisfies vertical force equilibrium for each slice and overall moment equilibrium about the center of the circular trial forces. The slope stability analysis was performed for both static conditions and pseudo static conditions. The minimum factor of safety values used for this analysis is 1.5 for static conditions and 1.1 for pseudo static conditions.

The program utilizes the pseudo static method for evaluating the stability of the slope for seismic conditions. The pseudo static method simulates potential inertial forces due to ground accelerations during an earthquake by including horizontal and vertical static seismic forces. These seismic forces are assumed to be proportional to the weight of the potential sliding mass times a seismic coefficient (k_h – horizontal seismic coefficient), expressed in terms of the accelerations of the underlying earth.

The vertical acceleration component was not used in our slope stability analysis. As long as the vertical acceleration is less than the horizontal component (vertical acceleration typically used in slope stability analyses is ⅔ of the horizontal component), studies have shown that the application of a vertical acceleration in the limit equilibrium analysis will change the horizontal yield acceleration by no more than 10 percent (Munfakh et al). The reason for this low percentage is that the vertical ground motions are generally out of phase with, and of different frequency than the horizontal ground motions. It is therefore a reasonable assumption to ignore the vertical acceleration.

Pseudo-static slope stability analysis using peak ground acceleration in conjunction with a factor of safety of 1.0 provides *excessively conservative assessments* of slope stability (FHWA 1997). Consequently, the seismic coefficient used in slope stability analysis is typically less than the peak ground acceleration. The reason is that the alternating inertia forces are of short duration and change direction many times during the seismic event. Because of the change in direction, the factor of safety may fall below 1.0 for a short duration, but during the reverse direction will be above 1.0. Slope deformations will occur when the factor of safety falls below 1.0, but the cumulative deformations during the earthquake are usually tolerable with some repair to the slope face after the earthquake event. Hynes and Franklin (1984), as referenced by FHWA (1997), investigated deformations after a seismic event. They concluded that by using ½ of the peak ground acceleration, the permanent seismic deformation would be less than 0.3 meters. Based on this reference and past studies, a horizontal coefficient of ½ the maximum PGA, or 0.25g was used in our pseudo-static slope stability analysis. The following assumptions were made in our analysis:

- The geotechnical model for the slope stability analysis was characterized as having three predominant geologic units: fine-grained alluvium classified as a silty sand **(SM)**; glacial outwash deposits classified as poorly graded gravels, cobbles, and boulders with sand **(GP-GM);** and granitic bedrock with depth.

- Based on direct shear test results, an internal friction angle of 34^0 and cohesion of 400 psf was used for the fine-grained alluvium and an internal friction angle of 38^0 and cohesion of 100 psf was used for the glacial outwash deposits.

- A truck surcharge load was also assumed on the access road with a 20 kip axle load.

Based on our analysis and assumptions, the factor of safety calculated is 1.57 for static conditions and 1.19 for pseudo-static conditions. These factor of safeties are acceptable for design. Slope stability results are presented in Appendix E.

9.10.2 Erosion Potential

Erosion potential depends on numerous factors involving grain size distribution, cohesion, moisture content, slope angle and the velocity of the water or wind on the ground surface. Erosion control is recommended for all cut and fill slopes 5H:1V or steeper. Slopes between 3H:1V and 5H:1V can be stabilized by hydroseeding and planting of native shrubs and trees. Slopes steeper than 3H:1V require mechanical stabilization consisting of rock rip-rap with a minimum of 75 percent of the rock rip-rap 8-inches or greater in diameter. Other methods of stabilization on slopes steeper than 3H:1V can be used if demonstrated to be as effective as mechanical stabilization. Landscape slope stabilization designed by a registered Landscape Architect may also be used on slopes steeper than 3H:1V.

If vegetation is the proposed means of stabilization, a licensed professional should be consulted to provide a durable seed mix that will establish a firm root system in the semiarid environment of Northern Nevada. Vegetation stabilization may take several months or up to a year to establish. Temporary erosion control blankets (ECB) may be considered to provide erosion control until vegetation is established. The service life of these blankets will vary based on blanket type. In general, straw blankets have service lives of about 18 to 24 months, while coconut blankets has a service live of about 36 months.

Cut and fill slopes, even when stabilized or vegetated as described, may be subject to gully development and erosion. Therefore, the crest of each slope should be protected by a drainage berm capable of redirecting runoff away from the slope face.

9.11 SOIL CORROSION TEST RESULTS

Soil corrosion tests include pH, redox potential, chlorides, soluble sulfates, and resistivity. A listing of all test results by sample location is presented in Table 13. A brief summary of the soil corrosion tests is presented below:

- **Soluble sulfates:** Soluble sulfate test results are less than 0.02% indicating a negligible sulfate exposure to concrete.

- **pH:** The pH test results ranged from 7.1 to 9.1, which indicates a slightly alkaline soil condition.

- **Resistivity:** Resistivity test results were high and ranged from 6,150 to 10,250 ohms x cm. In general, soils with a resistivity below 3,000 ohms x cm are corrosive to metal pipes. The resistivity results indicate that soil corrosion potential is low.

- **Chlorides:** Chlorides are less than 10 ppm**,** indicating non-corrosion soil conditions.

- **Redox potential:** The redox potential indicates the degree of aeration in the soil. The redox potential was relatively uniform ranging from 522 to 563 mv, which indicates an aerobic soil condition and is generally non-corrosive to metal pipes.

Table 13 - Soil Corrosion Test Results

Exploration location[1]	Laboratory Tests				
	Resistivity (ohmxcm)	Redox potential (mv)	Chlorides (PPM)	pH	Sulfates (%)
TP-1 (1B)	10250	522	<10	7.9	<0.02
TP-2 (2A)	6780	541	<10	6.9	<0.02
TP-3 (3A)	6470	563	<10	7.1	<0.02
TP-4 (4A)	6150	555	<10	7.3	<0.02

Notes:
1) Refer to site plan and logs for soil sample location.

9.12 CONCRETE

A concrete mix with a maximum water/cementitious ratio of 0.5 should be utilized for all concrete work in contact with native soils, including foundations. Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals should consist of a mix with a maximum of 0.45 water/cementitious ratio and have a compressive strength of 4,500 psi in 28 days.

10.0 CONSTRUCTION OBSERVATION AND TESTING SERVICES

The recommendations presented in this report are based on the assumption that the owner/project manager provides sufficient field testing and construction review during all phases of construction. Prior to construction, the owner/project manager should schedule a pre-job conference to include, but not be limited to: owner/project manager, project engineer, general contractor, earthwork and materials subcontractors, and geotechnical engineer. It is the owner's/project manager’s responsibility to set-up this meeting and contact all responsible parties. The conference will allow parties to review the project plans, specifications, and recommendations presented in this report, and discuss applicable material quality and mix design requirements. All quality control reports should be submitted to the owner/project manager for review and distributed to the appropriate parties.

11.0 STANDARD LIMITATION CLAUSE

This report has been prepared in accordance with generally accepted local geotechnical practices. The analyses and recommendations submitted are based upon field exploration performed at the locations shown on Plate A-2 – Field Exploration Location Maps of this report. This report does not reflect soils variations that may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary. Sufficient construction observation should be completed in all phases of the project related to geotechnical factors to document compliance with our recommendations.

This report has been prepared to provide information allowing the engineer to design the project. The owner/project manager is responsible for distribution of this report to all designers and contractors whose work is affected by geotechnical recommendations. In the event of changes in the design, location, or ownership of the project after presentation of this report, our recommendations should be reviewed and possibly modified by the geotechnical engineer. If the geotechnical engineer is not accorded the privilege of making this recommended review, he can assume no responsibility for misinterpretation or misapplication of his recommendations or their validity in the event changes have been made in the original design concept without his prior review. The engineer makes no other warranties, either expressed or implied, as to the professional advice provided under the terms of this agreement and included in this report.

This report was prepared by CME for the account of the Shaw Engineering. The material in it reflects our best judgment in light of the information available to us at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions to be made based upon it, are the responsibility of such third parties. Construction Materials Engineers Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

REFERENCES

ADAMA Engineering Inc., 2001-2011, Reinforced and Unreinforced Slope Stability Analysis ReSSA© (3.0).

American Society for Testing and Materials (ASTM), 1993, *Soil and Rock; Dimension Stone; Geosynthetics,* Volume 4.08.

Bowles, J. E., 1996, *Foundation Analysis and Design,* McGraw Hill.

Federal Highway Administration, 1997 (May), Geotechnical Earthquake Engineering For Highways Volumes 1 & 2 , Report No. FHWA-Sa-97-076.

Kramer S.L., 1996, *Geotechnical Earthquake Engineering,* Prentice Hall.

International Building Code (2012), 2011, International Code Council, Inc..

Munfakh, G. et al (1998), *Geotechncial Earthquake Engineering Reference Manual* (Publication Number FHWA-HI-99-012), US Department of Transportation , Federal Highway Administration, National Highway Institute, Arlington, Virginia.

Standard Specifications for Public Works Construction, 2012 (Washoe County, Sparks-Reno, Carson City, Yerington, Nevada).

Tolimson, M.J.,1986, *Foundation Design and Construction*, John Wiley and Sons, Inc., New York, 5th Edition.

U.S. Department of the Interior Bureau of Reclamation, December 2013, revised August 2015, *Method for Prediction of Flexible Pipe Deflection*, M-25 Second Edition.

USGS website: *Earthquake Hazards Program U.S. Seismic Design Maps* http://earthquake.usgs.gov/designmaps/us/application.php

USGS Website: *Earthquake Hazards Program Quaternary Faults in Google Earth* http://earthquake.usgs.gov/hazards/qfaults/google.php