GEOTECHNICAL INVESTIGATION
WASHOE FOREBAY SPILLWAY CHANNEL REPLACEMENT
MOGUL, WASHOE COUNTY, NEVADA

PREPARED FOR:

TRUCKEE MEADOWS WATER AUTHORITY

AUGUST 2016
FILE: 1860

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

August 17, 2016
Project No: 1860

Mr. Brent Eisert P.E.
TMWA
1355 Capital Boulevard
P.O. Box 30013
Reno, Nevada 89520-3013

RE: Geotechnical Investigation
Washoe Forebay - Spillway Channel Replacement
Mogul, Washoe County, Nevada

Dear Mr. Eisert,

Enclosed is our geotechnical investigation for the proposed TMWA Washoe Forebay - Spillway Channel Replacement project to be located in Mogul, Washoe County, Nevada.

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.

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

Exp. 12-31-17

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

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Mogul, 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 Washoe Forebay Spillway Channel Replacement located in Mogul, 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 Section 14 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 Spillway Channel Location

The project site is located near the southern bank of the Truckee River in Mogul, Nevada. Belli Ranch Subdivision is located to the south and Interstate 80 is located to the north of the project site.

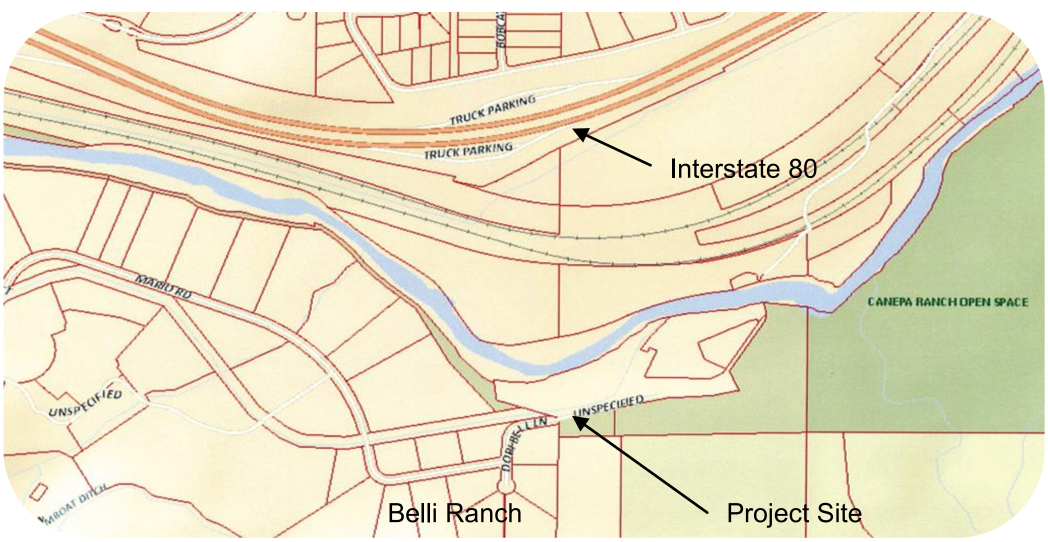

Figure 1: Vicinity Map (N.T.S) Washoe County GIS
(http://wcgisweb.washoecounty.us/QuickMap/)

2.2 Project Description

It is understood that the existing wooden spillway channel will be replaced. The purpose of the spillway channel is to discharge overflow water from the flume to the Truckee River. The spillway is located along the north side and has a perpendicular orientation to the existing concrete flume. The concrete flume has a west to east orientation and flows into the Forebay.

The existing spillway has a length of about 50 feet and is support by wooden posts placed on wooden blocks directly supported by the existing ground. The discharged water flows freely from the end of the spillway to a small pond, which flows into the Truckee River.

The existing wooden spillway channel will be removed and replaced with a steel spillway channel supported on concrete foundations keyed into original ground. To provide additional shear resistance and lateral support, rock anchors will be installed. It is understood that the length and geometry of the new spillway will be similar to the existing spillway.

The foundation system for the spillway channel will consist of 4 separate rows of shallow, spread foundations terraced along the existing slope face. The bottom foundation will be located approximately 8 feet from the end of the spillway. The ascending foundation rows will be spaced from 8 to 15 feet apart starting from the bottom foundation and ending at the top of the slope. Structural loading for each foundation row is approximately 50 kips.

Except for the top spread foundation, three rock anchors per foundation row will be installed. Each rock anchor will be designed for a load of 10 kips.

3.0 SITE CONDITIONS

The project site is located near the base of the northern 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).

The spillway is located overlying the southern bank of the Truckee River. The spillway discharge water has formed a small pond at the base of the bank with a near vertical slope. The total elevation differential between the top of the bank and pond area is about 80 feet. The end of the spillway appears to be about 50 feet above the pond. The slope adjacent to the existing spillway channel is near vertical with estimated gradients of about ¼ H:1.0 V. Figure 1 presents a topographic map from the Washoe County GIS showing the general terrain at the site. This map is shown for reference only. A site-specific ground survey has been completed for design purposes.

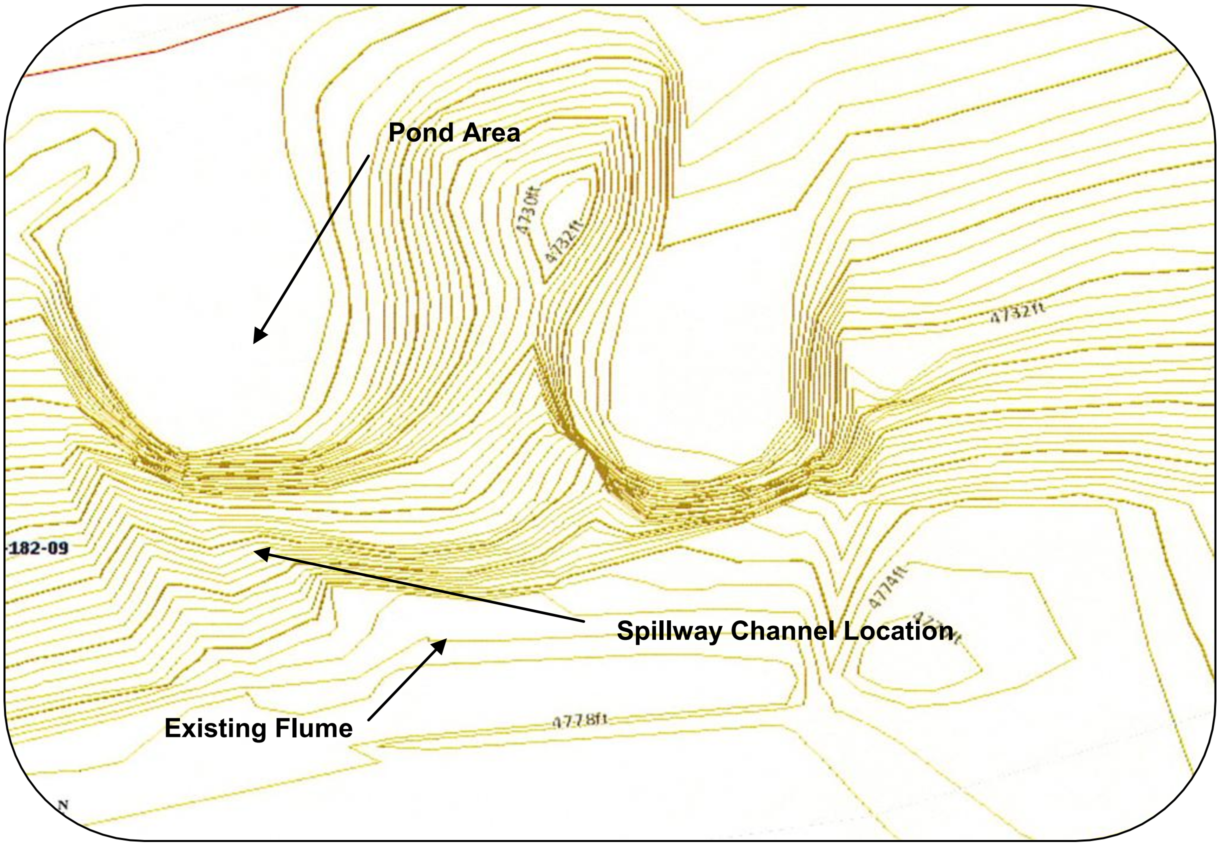

Figure #1: Topographic Map Showing Spillway Channel location
Washoe County GIS
(http://wcgisweb.washoecounty.us/QuickMap/)

4.0 EXPLORATION

Exploration was completed by both exploratory drilling and geologic mapping of the exposed slope adjacent to the channel spillway. A geologic cross-section was developed based on the mapping and exploration (refer to Section 6.0).

4.1 Exploratory Boring

The proposed site was explored in April 2016 by drilling one test boring. The boring was drilled using a truck-mounted GEFCO SS15 sampling drill rig. The upper 4 feet of the soil profile was explored by auger methods until refusal on glacial outwash deposits. At four feet below the existing ground surface (bgs), drilling was switched to ODEX drilling methods and continued to a depth of 10 feet bgs. At 10 feet bgs drilling methodology was switched to coring (refer to 4.1.1) and continued to the maximum depth of exploration at 46 feet bgs. The approximate locations of the test borings are shown on Plate A-1 - Site Plan.

4.1.1 Coring

Rock coring was completed from about 10 to 46 feet bgs. The drill bit used for rock coring was HQ (2.5-inch-diameter core). During coring operations, several different bedrock physical characteristics were recorded, measured, and identified. These bedrock physical characteristics include:

- Drill rate (assists in determining core loss zones);
- Core recovery percentage (identifies areas of highly fractured and/or soft, friable bedrock areas);
- Rock Quality Designation (RQD), which is the ratio of intact pieces of core greater than 4 inches to the length of the recovered cored;
- Discontinuities (spacing and orientation);
- Weathering; and
- Rock type description.

All cores were stored in a core box, photographed, and labeled. Photographs of the cores are presented in Appendix C. Wooden blocks were placed in the box to designate changes in depth and lengths of drilling where rock core was not recovered.

4.2 Exploration Locations and Ground Elevations

Boring location was determined by approximate methods referencing existing site improvements as presented on the Site Plan-Plate A-1 in Appendix A. Ground surface elevations were determined by linear interpolation between ground contour line elevations presented on an existing topographic map and should be considered approximate.

4.3 Material Classification

Soils were examined and classified during exploration in general accordance with ASTM D 2488 (Description and Identification of Soils). During exploration, representative bulk samples were placed in sealed plastic bags and returned to our laboratory for testing. Upon completion of laboratory testing, additional soil classification and verification of the field classifications were subsequently performed in accordance with the Unified Soil Classification System (USCS), as presented in ASTM D 2487. Boring Log (Plate A-2), a USCS chart (Plate A-3 - Graphic Soils Classification Chart), and Physical Criteria for Rock Descriptions (Plate A-4) are presented in Appendix A.

4.4 Geophysical Subsurface Investigation: Refraction Microtremor (ReMi) Shear Wave

Geophysical field measurements using the Refraction Microtremor (ReMi) 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. Shear wave velocity was determine along the south side of the existing concrete flume.

The DAQlink III 24-bit acquisition system (Seismic Source/Optim) utilizing a multichannel geophone cable with twelve geophones, placed at an approximate spacing of 25 feet was used to obtain surface wave data. This 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.

SeisOpt® ReMi^{TM} 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 intregating existing logs or blow counts information.

The resulting S-wave velocity profiles were evaluated to determine the soil Site Classification (refer to Section 7.0 - Seismic Design Parameters) in accordance with 2012 IBC. Results of the ReMi geophysical field measurements are attached as Plate A-5 in Appendix A.

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 Unconfined Compression Testing

Selected bedrock cores from both the sandy conglomerate and basaltic lahar strata (refer to Section 6.0) were tested for unconfined compression strength (ASTM D2166). The results of these tests are presented on Plate B-1: Unconfined Compression Testing.

5.2 Direct Shear Testing

A direct shear test (ASTM D 3080) was performed on a selected bedrock sample of basaltic lahar stratum. Tests were run on in-situ bedrock samples, tested at three different normal pressures to derive a plot of Mohr's Circle Failure Envelope. Results of these tests are shown on Plate B-2 : Direct Shear Test.

6.0 GEOLOGIC AND GENERAL SOIL CONDITIONS

Based on the Geologic Map for the Verdi Quadrangle (Bell and Garside, 1987), the project site is located among several different geologic units. The uppermost geologic unit is mapped as terrace and glacial outwash deposits of the Truckee River, which is part of the Tahoe Outwash Formation. This Formation is a glacial outwash deposit of Pleistocene age that occurred during periods of catastrophic flooding and is 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).

Underlying the Tahoe Outwash Formation is the Sandstone of Hunter Creek Formation. This formation is comprised of siltstones, sandstones, conglomerates, and diatomaceous siltstone. Based on slope mapping, north of the flume, and an exploratory boring, located south of the flume, several different bedrock types were encountered, as follows:

- The uppermost bedrock was classified as a siltstone/claystone having the following encountered physical and structural properties: intensely to closely fractured; moderately soft; weak; and moderately to deeply weathered. When excavated this material has similar soil properties, determined by visual classification, as either an elastic silt **(MH)** or fat clay **(CH).**

- Underlying the claystone/siltstone is a stratified sandstone consisting of three different beds. In general, the sandstone becomes coarser grained with depth as follows:

1) The uppermost sandstone bed is primarily fine-grained and is classified as a graywacke. A graywacke is characterized as having a darker color; generally tough and well-indurated; and consists predominantly of quartz and feldspars. The following physical and structural properties were encountered: moderately soft to hard; moderately strong; moderately weathered; flaggy (very thinly bedded); and gray/brown.

2) The middle sandstone bed predominantly consists of coarse grained sands and is classified as an arenite with well-sorted grains. The following physical and structural properties were encountered: flaggy (very thinly bedded); weak to moderately strong; moderately weathered; moderately soft; and yellow/brown.

3) The lowermost sandstone bed is classified as a sandy conglomerate containing some sub-rounded to rounded pebbles having predominant diameters of 1-2 inches. The following physical and structural properties were encountered: poorly-sorted and coarse-grained; generally blocky (thickly bedded); moderately hard; moderately strong; moderately to slightly weathered; grey/brown.

- The lowermost geologic unit encountered is a volcanic bedrock classified as a volcanic mudflow deposit (Lahars). Lahars are formed by the rapid mixing of loose pyroclastic debris and water as volcanic flows move down river systems to form thick linear deposits. They are characterized as having a chaotic mix of small and large rock fragments in a fine-grained matrix. In general, the bedrock was encountered with the following physical and structural properties: intensely to closely fractured; moderately hard; moderately strong to weak; moderately weathered; and bluish grey. The prominent joint orientation is nearly north of south with a dip of 35 to 40 degrees.

Photos 1 to 3 present different images of the geologic units encountered.

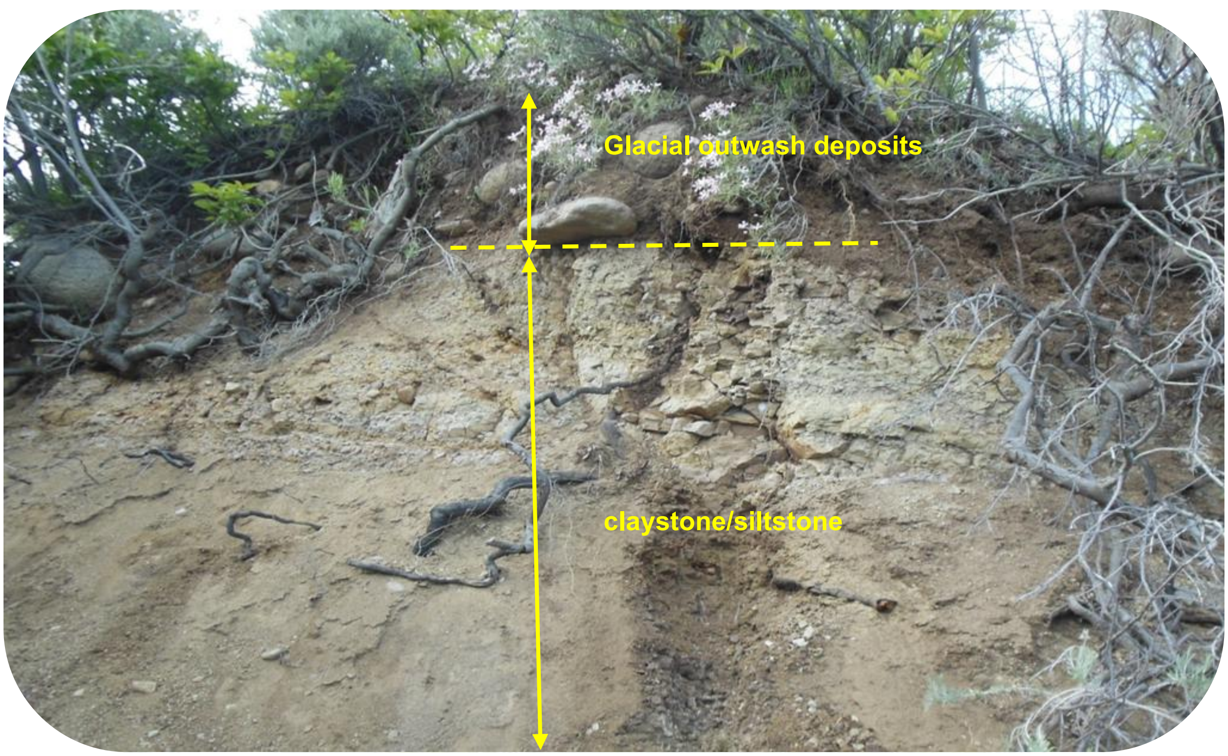

Photo #1: Looking at upper portion of the slope showing glacial outwash deposits and claystone/siltstone

Photo #2: Showing uppermost exposed volcanic bedrock directly below end of existing spillway channel

Photo #3: Showing lowermost exposed basaltic bedrock directly below end of existing spillway channel. Note angular rock fragments in the bedrock indicative of a Lahar deposit.

Using the geologic profile determined from the boring and geologic profile from the cut slope, a geologic cross section was drafted, as presented in Appendix D. This cross section shows three prominent geologic units as referenced in Section 6.0. The geologic cross section is presented for reference only and actual depths to these geologic units 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 8 miles southeast of the project site.

7.2 Faults

Based on a review of the Quaternary Fault Map of Nevada-Reno (Bell, 1984), USGS Quaternary Faults on Google Earth Map, and referenced geologic map, no mapped faults trend through the project site. The closest mapped active faults are part of the Dog Valley Fault Zone, located about 8 miles west of the site and the referenced Eastern Sierra Nevada Frontal Fault Zone. 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, both the Dog Valley Fault Zone and Eastern Sierra Nevada Frontal Fault Zone are considered Holocene Active Faults. 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 velocities of 1450 ft/s was measured (refer to Plate A-5), which corresponds to a Site Classification of C (very dense soil and soft rock).

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 E.

Table 2 – Seismic Design Parameters	
PARAMETER DESCRIPTION	**SPILLWAY CHANNEL LOCATION**
Approximate Latitude of Site	39.505^{0}
Approximate Longitude of Site	119.936^{0}
Peak Ground Acceleration-MCE$_R$ PGA (ASCE 7-10 Standard)	0.50 g
Design Peak Ground Acceleration-DPGA (ASCE 7-10 Standard)	0.40 g
Spectral Response Acceleration at Short period (0.2 sec.) S_s (for Site Class B)	1.500 g
Spectral Response Acceleration at 1-second Period, S_1 (for Site Class B)	0.528 g
Site Class Selected for this Site	C
Site Coefficient F_a, decimal	1.0
Site Coefficient Fv, decimal	1.3
Design Spectral Response Acceleration at Short period, S_{Ds} (Adjusted to Site Class B, SDs= 2/3 SMs)	1.00 g
Design Spectral Response Acceleration at 1-second Period, S_{D1} (Adjusted to Site Class B, SD1=2/3 SM1)	0.457 g

1) MCE$_R$ PGA- Maximum credible earthquake geometric mean peak ground acceleration.

9.0 DISCUSSION AND RECOMMENDATIONS

The proposed Washoe Spillway Channel Replacement Project is located within the upper portion of a steep bank along the south side of the Truckee River near the base of the northern flank of the Carson Range in the Truckee River Corridor. Three prominent geologic units are exposed in the bank face: Tahoe Glacial Outwash Deposits, Sandstone of Hunter Creek Formation, and volcanic bedrock.

The volcanic bedrock and sandstone beds encountered in the Sandstone of Hunter Creek Formation will provide an adequate bearing layer for the spillway foundations. The depth to these bedrock layers will be variable across the spillway alignment and shall be determined during construction. Foundations shall not bear directly on the siltsone/claystone unit of the Sandstone of Hunter Creek Formation.

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.

9.1 Spillway Channel Foundation Support Recommendations

Foundations will consist of both spread foundations and rock anchors.

9.1.1 Bedrock Strength Properties

Elastic modulus was determined by two different methods: Elastic modulus of intact rock (E_i) and elastic modulus (E_m) determined by rock mass rating (RMR). E_i was determined by the unconfined compression test results and E_m was determined by RMR, which is derived from AASHTO (2012). The design elastic modulus is the lesser of these two values. Elastic modulus results are used to determine anticipated foundation settlements. Elastic modulus results are presented in Table 3.

* 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.

Table 3 – Measured Bedrock Elastic Modulus Results				
Borehole and Depth	**Bedrock Type**	**Rock Mass Rating**	**Elastic Modulus (ksf)**	
			E_i	**E_m**
B-1 (33-38)	sandstone	RMR = 43	7,200	140,000
B-1 (38-46)	Basaltic Lahar	RMR = 43	15,480	140,000

The E_m value is higher than the E_i value. Consequently, E_i values will be used for design.

Table 4 presents elastic modulus design recommendations based on Table 3 elastic modulus results.

Table 4 – Design Bedrock Elastic Modulus Recommendations		
Borehole and Depth	**Bedrock Type**	**Design Elastic Modulus (ksf)**
B-1 (33-38)	sandstone	7,200
B-1 (38-46)	Basaltic Lahar	15,480

Strength parameters for the different geotechnical units were evaluated by using methodologies developed by Hoek and Brown (1988, 1997) and direct shear test results. The method developed by Hoek and Brown is based on the rock quality or RMR rating and the unconfined compression strength. Bedrock shear strength used in our analysis is summarized in Table 5.

Table 5 – Assumed Bedrock Strength Properties Summary				
Geologic Unit	**Internal strength angle (Φ)**	**Cohesion (ksf)**	**Uniaxial Compression Strength (ksf)**	**RMR**
Sandstone	35^0	0.9	43.2	43
Basaltic Lahar Deposits	35^0	2.0	61.9	43

9.2 Analysis

9.2.1 Bearing Pressure

Because foundations are located near the edge of the cut slope, allowable bearing pressure was determined by stability analyses using the computer program RESSA (Version 3 - Adama Engineering Inc., 2011). This program performs a two dimensional limit equilibrium analysis to compute the factor of safety (FOS). The limit equilibrium analysis was performed using the comprehensive Bishop method for the rotational analysis. This method satisfies vertical force equilibrium for each slice and overall moment equilibrium about the center of the circular trial forces. Based on Spencer's methodology, three part wedge analysis consisting of a passive, central, and active wedges was also completed.

To create the geotechnical model for the slope stability analysis, the following site conditions were assumed:

- Foundation surcharge loading of 50 kips. Foundation footprint was used for the static and seismic conditions, respectively.
- A series of three spread foundations, spaced at 10 feet, starting from about 5 feet from the edge of the cut slope were assumed. An uppermost continuous spread foundation was also assumed at the top of the spillway.
- The cut slope below the bottom foundation has an overall slope gradient of about ¼H:1V;

Stability analyses were performed for both static conditions and seismic conditions by imposing surcharge loading at foundation grade. The minimum factor of safety (FOS) values used for this analysis is about 1.5 for static conditions and 1.1 for seismic conditions. Factor of safety values exceeded these minimum values (refer to section 9.3 for additional information).

9.2.2 Foundation Grade Soils Preparation

The spillway will be supported by spread foundations, spaced from 10 to 15 feet, terraced on the existing slope face, and oriented parallel to the existing slope face. A continuous, spread foundation will be constructed at the top of the spillway. Based on the geologic profile, it is anticipated that foundations will be placed on either sedimentary units of the Sandstone of Hunter Creek Formation or volcanic bedrock consisting of basaltic lahar deposits.

Foundations can bear directly on sandstone beds of the Sandstone of Hunter Creek Formation and basaltic lahar deposits. However, foundations cannot bear directly on siltstone/claystone beds of the Sandstone of Hunter Creek Formation, which will be encountered in the uppermost portion of the spillway channel. A structural fill layer is recommended between the foundations and siltstone/claystone bedrock as well as a reduced bearing pressure. The other option is to extend the foundation below the claystone/siltstone layer to bear directly on the sandstone beds.

Where siltstone/claystone is encountered below foundations, the minimum thickness of the structural fill shall be 3 feet and placed in accordance with recommendation given in Section 10.2 Grading and Filling. The structural fill shall extend laterally from the edge of the foundation at least 3 feet.

After the old spillway has been removed, additional exploration will be required to determine the depth to bedrock at each foundation location. Bedrock will have to be excavated to provide a level surface for the foundation. Excavation may require the use of chipping hammers or hoe-rams placed at the end of a backhoe or other similar construction equipment. The foundation can be poured directly on the bedrock provided all loosened rock is removed from the surface.

9.2.3 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 9.2.2, the allowable bearing pressures presented in Tables 6 and 7 are recommended for the design of individual column footings.

Table 6 – Foundation Allowable Bearing Pressures on Competent Bedrock[1]	
Loading Conditions	**Maximum Soil Net Allowable Bearing Pressures[(2)] (pounds per square foot)**
Dead Loads plus full time live loads	4,000
Dead Loads plus live loads, plus transient wind, or seismic loads.	5,300

NOTES:
1) Bedrock consists of either sandstone or basaltic lahar deposits
2) The net allowable bearing pressure is that pressure at the base of the footing in excess of the adjacent overburden pressure.

Table 7 – Foundation Allowable Bearing Pressures on Structural Fill	
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.	3,300

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. Regardless of loading, individual column 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 0.50 and 0.40 is recommended for sliding resistance at the base of the spread footing bearing directly on bedrock and structural fill, respectively. A design value of 2,000 and 200 pounds per square foot per foot of depth (psf/ft) is recommended for passive pressures bearing directly on bedrock and structural fill, respectively.

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. Additionally, passive pressure values were reduced to account for the edge of the foundation being at least 5 feet away from the edge of the cut slope.

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. Foundation design values are based on spread footings bearing on either structural fill or bedrock. It is assumed that foundations will be constructed at least 2 feet below exterior grade with foundation sidewalls in direct contact with undisturbed bedrock.

9.2.3.1 Settlement

Since foundations will be placed on bedrock or granular fill soils, an elastic settlement response is expected and the majority of the settlement will occur rapidly, generally during the construction time frame for the structure.

Total settlements are anticipated to be on the order of less than ½ inches. Differential settlement between foundations with similar loads and sizes is anticipated to be ½ of the total settlement.

Estimated settlements are based on the foundation grade soils preparation recommendations followed during construction. Structural fill moisture contents are critical. Failure to adequately moisture condition fills during placement will delay consolidation and may result in greater settlement being experienced by the structures and improvements.

9.2.4 Rock Anchors

Rock anchors will be installed to resist foundation lateral loads. Rock anchors will be drilled into underlying bedrock and grouted in place.

Rock anchors have 5 different primary components (refer to Figure #2):

1) **An anchor nut or head on a bearing plate:** The nut threads onto the bearing plate to pre-tension the rock anchor. The rock anchor shall be stressed with a torque wrench or other methods.

2) **Rock Anchor:** Epoxy coated rock anchor for this application. The rock anchor shall be all thread with a diameter of at least 1-inch.

3) **Bonding medium:** The bonding medium is both the grout and bedrock**.** A high-strength pressurized grout will be injected between the rock anchor and the inside wall of the drill hole.

4) **Free stressing length or unbounded length:** In this segment of the rock anchor a sleeve is placed over the rock anchor to prevent grout from attaching to the rock anchor.

5) **Tendon Bonding length:** The bonding length of the rock anchor transmits the applied tensile load to the surrounding rock.

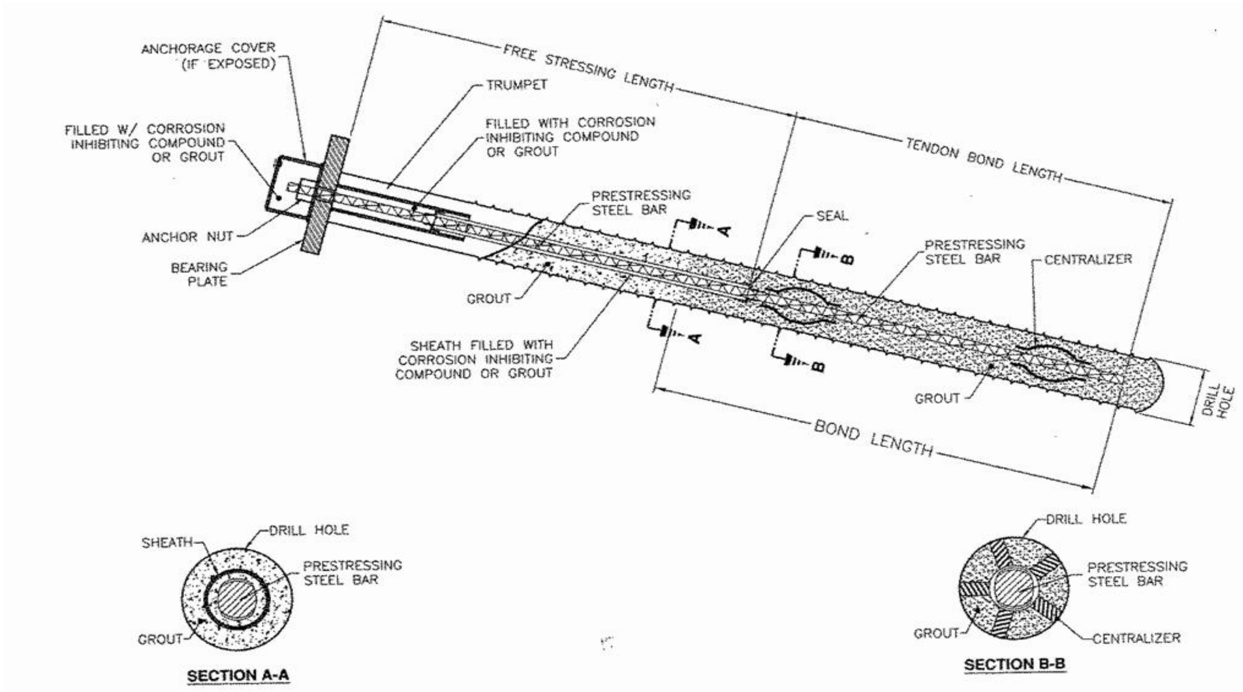

Figure 2: Typical Cross-Section of Rock Anchor with Type 2 Corrosion Protection

The ultimate bonding strength of the bedrock depends on several factors including the unconfined compression strength. The bonded segment of the anchor is designed for two different bedrock types: basaltic lahar deposits or sandstone. Table 8 provides the rock anchor design load equations.

Table 8 – Rock Anchors Design Loads	
Rock Anchors Bonding Medium	**Anchor Design Load (P) (kips)**
basaltic bedrock	$P = (L_b)(d)(\pi)(0.033)$
sandstone	$P = (L_b)(d)(\pi)(0.023)$
Note: 1) L_b = length of bonded anchor (inches) 2) d = hole diameter (inches) 3) π = 3.14	

Equations in Table 8 have the following design/construction assumptions:

- A F.O.S. of 2 has been incorporated into the equations provided in Table 8. It is recommended that rock anchors are field tested to verify design loads;

- The recommended minimum hole diameter is 3 inches with a maximum 1 inch diameter rock anchor;

- Pressurized grouting has been assumed;
- Anchors shall have a minimum spacing of 4 feet;
- The free stressing length shall be at least 6 feet.

Class II corrosion protection is recommended for permanent rock anchors. This protection level is known as a multiple corrosion protection anchor (MCP) and includes a trumpet or additional sheath between the bearing plate and free stressing sleeve, epoxy coated rock anchor, and grout seal. Rock anchors such as the Williams® MCP-1, Williams Engineering Corporation, or equal meets this requirement.

9.3 Slope Stability and Erosion Control

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

9.3.1 Erosion Potential

Water discharged from the end of the spillway will splash on the existing bedrock surface, which has been exposed to water discharge over many years. Based on a visual inspection of this bedrock surface, minimal erosion is evident. As shown on Photo #2, the water cascades onto a rock ledge that protrudes from the face of the bedrock. If significant erosion to the bedrock surface was to occur, the rock ledge would have recessed into the bedrock surface. Significant erosion has not occurred and, it is our opinion that armoring of the bedrock surface is not required.

9.3.2 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 pseudostatic 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 pseudostatic method for evaluating the stability of the slope for seismic conditions. The pseudostatic 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. 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.

10.0 CONSTRUCTION RECOMMENDATIONS

10. 1 Trenching

10.1.1 Trench Excavation

After the removal of the existing spillway, the existing slope will be terraced for the foundation locations. In competent bedrock areas (sandy conglomerate, basaltic lahar deposits) excavation difficulties are possible. Based on the fracturing patterns observed in the existing bedrock outcrop, it is anticipated that the bulk of the bedrock can the excavated with a trackhoe in mass grading; however, specialized construction equipment such as a chipping hammer or hoe-ram may be required for localized areas. A jack hammer may be considered for foundation excavations or tight access areas. Bedrock excavation may cause an enlargement of the trench width due to the removal of larger rock particles.

10.1.2 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 9.

Table 9 - 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, the majority of the excavation will be in bedrock and a near vertical slope angle could be used. However, because of differences in fracturing patterns, it is recommended that a maximum slope gradient of 3H:4V be maintained. 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.

10.2 Grading and Filling

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. Structural fill should be free of vegetation, organic matter, and other deleterious material and shall comply with the material specifications presented in Table 10.

Table 10 - Guideline Specification for Structural Fill

Sieve Size	**Percent by Weight Passing**
4 Inch	100
¾ Inch	70 – 100
No. 40	15 – 60
No. 200	5 – 25
Maximum Liquid Limit	**Maximum Plastic Index**
40	10

Soluble sulfates:< 0.10 percent by weight of soil

It is anticipated that structural fill will have to imported to the site, unless excavated cut from the Tahoe Outwash Formation is available as backfill soil. Native granular soils may have to be screened to remove 4-inch or larger particles and shall meet the requirements given in Table 10.

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.

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.

10.3 Rock Anchors

10.3.1 Installation

Drilled holes shall be cleaned of all drill cuttings, sludge and debris before any anchor is inserted into the hole. Dewatering or pre-grouting may be required for proper grouting of the anchor if groundwater is encountered. Anchors shall be inserted in the hole with the anchor assembly positioned not less than 12 inches (300 mm) from the bottom of the hole in rock, and as shown in the manufacturer's installation manual.

Contractor shall submit proposed grout mix design to the Owner for approval at least 7 days prior to commencing grouting. Grouting of the annular space around the anchor shall be accomplished by pressure grouting with a portable grout pump. Grout pump shall provide 90 to 120 pounds per square inch (621 to 827 kN per square meter) capacity. Grout shall be Wil-X-Cement or approved equal non-shrink grout mixed with 2½ gallons of water per 55 lb. pail, as recommended by the manufacturer.

All grout tremie pipes, tubes and fittings shall be clean and free from dirt particles, grease, hardened grout or other contamination before grouting is commenced for any anchor. All surplus water and diluted grout shall be flushed or blown from all lines before commencing injections. The grout line shall be attached to the tremie pipe with suitable fittings, as recommended by the manufacturer, such that leakage is entirely prevented. If necessary, the grout shall be injected at a pressure to overcome hydrostatic head or as directed by the Owner.

Anchor Group #3 will penetrate through the claystone/diatomaceous siltstone deposit. These deposits are intensely to closely fractured and may caused caving of the corehole. Consequently, the portion of the corehole through these deposits may have to be cased. Additionally, the bonded portion of the rock anchors shall penetrate into the underlying sandstone deposit. The length of the corehole will be determined during construction, but is anticipated to range from 13 to 16 feet.

10.3.2 Testing

Before installing production anchors, the Contractor shall satisfactorily install, grout, and test one anchor in the presence of the Owner's representative. The test anchor can be a production anchor.

The Contractor is responsible to provide equipment as approved by the Owner to certify the pull-out capacity of the anchors. Applied test loads shall be measured with either a calibrated pressure gage or a load cell. The test jack and gage unit for the test shall have been calibrated within one year prior to use on the project. Current calibration certificates for all test equipment shall be submitted to the Owner prior to commencement of the testing.

Anchors shall be tested to 110% of design load or as shown on the plans. Three groups of 3 anchors are planned and at least 1 anchor from each group shall be tested. The cost to provide testing shall be considered as included in the contract unit price and no additional payment shall be made therefore.

The anchor test procedure will be as follows:

1. Anchor grout has cured for a minimum of 72-hours, unless the contractor can demonstrate that the grout has gained sufficient strength to justify an earlier test time.

2. Cribbing used for support of the test jack must be founded a minimum of 1-foot from any part of the anchor hole.

3. Prior to pull testing, the bearing plate and nut shall be placed on the anchor to allow tightening and torque assessment after the pull test has been completed.

4. The anchor shall be loaded to 110% of the design load. Upon reaching the test load, the load shall be locked off the maintained with no creep, deformation, or bleed-off of hydraulic pressure for a period of 3 minutes.

5. The Contractor shall replace anchors not meeting the acceptance criteria. Deficient anchors shall be reinstalled and tested at the Contractor's expense.

6. The cost to provide testing shall be considered as included in the contract unit price and no additional payment shall be made therefore.

7. The Owner will provide an inspection consultant to witness and record all tests in accordance with this contract. No test is accepted without the presence of the Owner's inspection consultant. The Contractor shall notify the Owner not less than 48-hours prior to scheduling pull-out testing.

10.4 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.

10.5 Anticipated Construction Problems

Some difficulty will be encountered in trenching due to the presence of bedrock. Neatline excavations may be difficult in bedrock zones and over break of bedrock shall be anticipated.

11.0 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.

12.0 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 Truckee Meadows Water Authority. 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.

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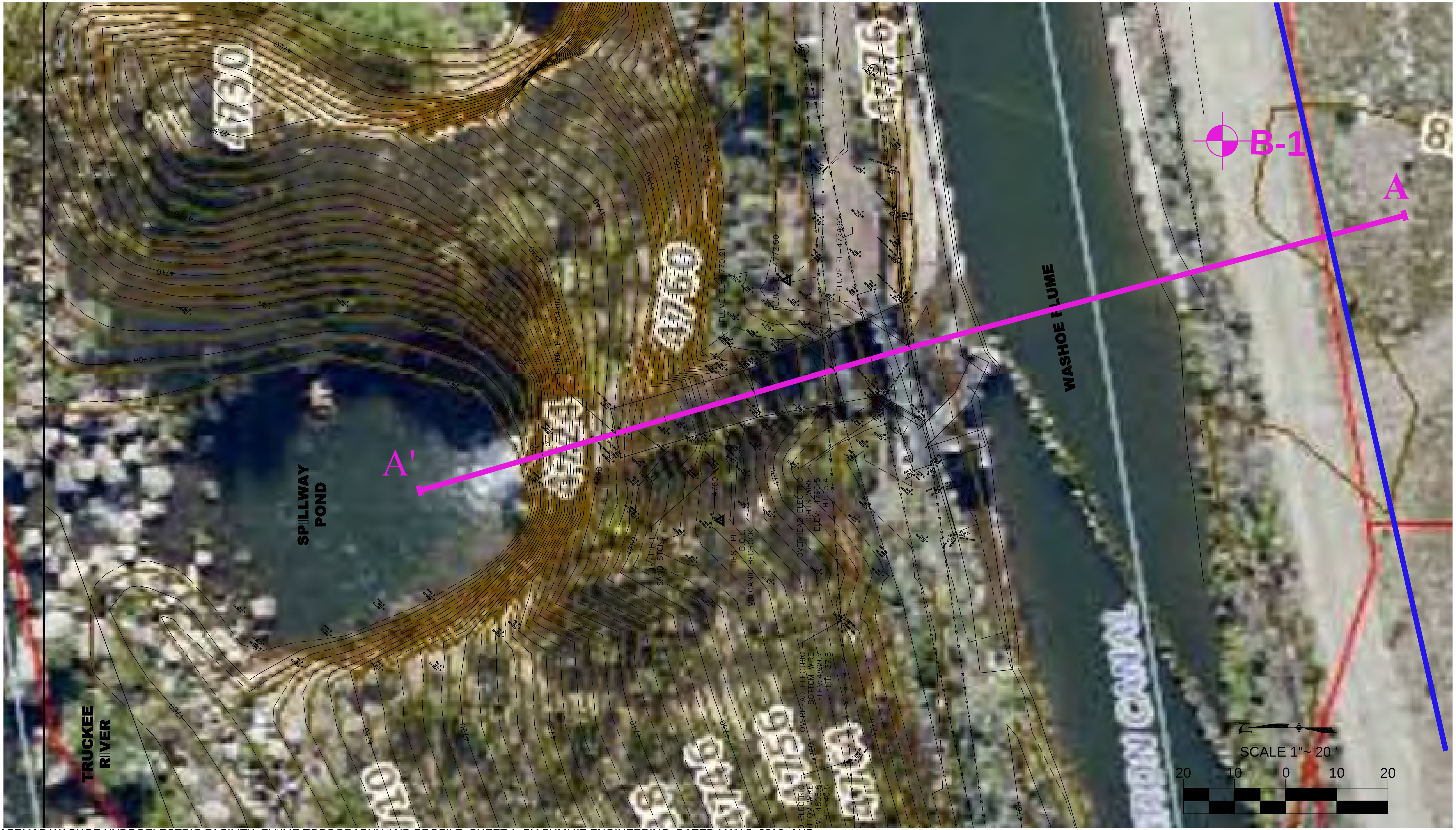
APPENDIX A



REFERENCE: BASEMAP WASHOE HYDROELECTRIC FACILITY, FLUME TOPOGRAPHY AND PROFILE, SHEET 1, BY SUMMIT ENGINEERING, DATED MAY 5, 2016; AND WASHOE COUNTY GIS IMAGE FILE, OBTAINED AT http://wcgisweb.washoecounty.us/quickmap/, MAY 2016

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

TRUCKEE MEADOWS WATER AUTHORITY
WASHOE FOREBAY
FIELD EXPLORATION LOCATION MAP
WASHOE COUNTY, NEVADA

PROJECT NO.: 1860 DATE: 5/18/2016

LEGEND

B-1 APPROXIMATE BORING LOCATION

APPROXIMATE REMI LINE LOCATION (REFER TO PLATE A-5)

A A' GEOLOGIC CROSS SECTION LOCATION (REFER TO APPENDIX D)

PLATE

A-1

V:\Active\1860\drawings\plates.dwg

LOG OF TEST BORING NO. B-1

PROJECT SPILLWAY CHANNEL REPLACEMENT **RIG & BORING TYPE** GEFCO SS15
LOCATION THE EXISTING ACCESS ROAD 10' S. OF THE FENCE
CLIENT: TRUCKEE MEADOWS WATER AUTHORITY **DATE** 04/25/16
PROJECT NO. 1860 **LOGGED BY:** SAM **SURFACE ELEVATION**

BLOW COUNTS: Corrected NO **HAMMER TYP.:** AUTOMATIC

Depth (ft)	Drill Method	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Blow Counts (SPTs)	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0		SM							MOIST	0'-2': SILTY SAND WITH GRAVEL FILL, mostly fine to coarse sand, little fine to coarse rounded gravel and cobbles, non-plastic, brown							
2.5	SSA	GP-GM			S	1A	78	VERY DENSE	MOIST	2'-4': POORLY GRADED GRAVEL WITH SILT, SAND AND COBBLES, mostly coarse subangular to subrounded gravel, little fine to medium sand, non-plastic, grey brown							
5	ODEX	GP			A	1B				4'-8': TAHOE OUTWASH FORMATION-POORLY GRADED GRAVEL WITH COBBLES, AND SAND, mostly large sized dark grey to black basaltic gravel chips visible in cuttings, little fine to medium sand, non-plastic, grey-brown							
7.5		GM/ SM			A	1C				8'-10': TAHOE OUTWASH FORMATION-SOIL MATRIX INCLUDES GRAVEL, COBBLES,& BOULDERS WITH SILT AND SAND, odex drilling slowed significantly at a depth of 8 feet, a sample of the cuttings notes large basaltic rock fragments visible in sample.							
10										ODEX DRILLING TERMINATED AT 10 FEET, SWITCHED TO NX CORE DRILLING DUE TO COMPETENCY OF UNDERLYING BOULDERS. NOTE: Refer to Core Log for additional boring information							
12.5																	
15																	

GROUNDWATER

DEPTH	HOUR	DATE
N.E.		

SAMPLE TYPE
A - Drill Cuttings B - Bulk Sample
R - 3" O.D. 2.42" I.D. Ring Sample
S - 2" O.D. 1.38" I.D. Sampler
U - 3" O.D. 2.42" I.D. Tube Sample
T - 3" O.D. Thin-Walled Shelby Tube

LABORATORY TESTS
A - Atterberg Limits
G - Grain Size
C - Consolidation
MD - Moisture/Density
DS - Direct Shear
TX - Triaxial

PLATE NO.: A-2

EXPLORATION METHODOLOGY
HSA: HOLLOW-STEM AUGER, SSA: SOLID-STEM AUGER
ODEX:ODEX SYSTEM, AR: AIR ROTARY, MR: MUD ROTARY

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

ROCK LOG OF TEST BORING NO. B-1

PROJECT SPILLWAY CHANNEL REPLACEMENT
TRUCKEE MEADOWS WATER AUTHORITY
RIG & BORING TYPE GEFCO SS15
LOCATION THE EXISTING ACCESS ROAD 10' S. OF THE FENCE
PROJECT NO. 1860 **DATE** 04/25/16 **LOGGED BY:** SAM **SURFACE ELEVATION**

Depth in Feet	Boring Type Drill Rate (Ft/Min)	Graphical Log	Sample	Sample Type	% Core Recovery	Rock Quality Designation (RQD)	Discontinuities (Spacing)	Discontinuities (Orientation)	Visual Description	Weathering	RMR	Laboratory Tests
10 – 12.5				C	44	0			10'-15': <u>TAHOE OUTWASH FORMATION</u>- <u>GRAVEL WITH COBBLES, BOULDERS, AND SAND</u>, mostly fine to coarse gravel and cobbles recovered, black basaltic cobble core approximately 10" in length recovered at 10'-13', granitic cobble core approximately 12 recovered at a depth of 13'-15'			
12.5 – 15	3			C	54	0						
15 – 17.5	3			C	32	0			15'-18': <u>SANDSTONE OF HUNTER CREEK FORMATION</u>- CONGLOMERATE CONSISTING OF: <u>GRAVEL, SAND, AND CLAY</u>, mostly coarse subangular gravel and cobbles, little to some fine to coarse sand, non-plastic, yellow brown			
17.5 – 20									18'-28': <u>SANDSTONE OF THE HUNTER CREEK FORMATION</u>- <u>SANDY CONGLOMERATE WITH SUBANGULAR TO SUBROUNDED GRAVEL</u>, sand and finer grained soil material visible in circulation basin, yellow brown to brown			
20 – 22.5	3			C	8.3	0						
22.5 – 27.5	3			C	10	0						
27.5									28'-33': <u>SANDSTONE OF HUNTER CREEK</u>			

GROUNDWATER

DEPTH	HOUR	DATE
N.E.		

SAMPLE TYPE

A - Drill Cuttings. B. Bag sample.
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
C - Core. R - Rotary Cuttings.

LABORATORY TESTS

A - Atterberg Limits
G - Grain Size
C - Consolidation
DS - Direct Shear
U - Unconfined Compression

PLATE NO.: A-2

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

ROCK LOG OF TEST BORING NO. B-1

PROJECT SPILLWAY CHANNEL REPLACEMENT
TRUCKEE MEADOWS WATER AUTHORITY
RIG & BORING TYPE GEFCO SS15
LOCATION THE EXISTING ACCESS ROAD 10' S. OF THE FENCE
PROJECT NO. 1860 **DATE** 04/25/16 **LOGGED BY:** SAM **SURFACE ELEVATION**

Depth in Feet	Boring Type Drill Rate (Ft/Min)	Graphical Log	Sample	Sample Type	% Core Recovery	Rock Quality Designation (RQD)	Discontinuities (Spacing)	Discontinuities (Orientation)	Visual Description	Weathering	RMR	Laboratory Tests
30 32.5	4			C	27	0			<u>FORMATION</u>- <u>SANDSTONE OR CEMENTED SANDY CONGLOMERANT</u>, includes lenses of subrounded to subangular gravels, brown Note: Faster drilling rate from 28 to 30 feet indicating less dense material			
35 37.5	4.5			C	100	40			33'-38': <u>SANDSTONE OF HUNTERCREEK FORMATION</u>- <u>SANDY CONGLOMERANT OF CLAYEY GRAVEL WITH SAND</u>, mostly fine to coarse subangular gravel, some fine to coarse sand, moderately cemented, yellow brown to brown		43	
40	4			C	100	33	4	RANDOM	38'-41': <u>BASALTIC LAHAR DEPOSISTES</u>, closely fractured, moderately soft to soft, plastic to friable,bluish grey Note: Coarse embedded angular rock fragments yellow black and orange,in a fine grained matrix	Deep	43	
42.5 45	4			C	100	42	4 to 5	RANDOM	41'-46': <u>BASALTIC LAHAR DEPOSISTES</u>, intensely to closely fractured, moderately hard, friable to weak,bluish grey	Mod.	43	
									CORING TERMINATED AT 46 FEET			

GROUNDWATER

	DEPTH	HOUR	DATE
▽	N.E.		
▼			

SAMPLE TYPE

A - Drill Cuttings. B. Bag sample.
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
C - Core. R - Rotary Cuttings.

LABORATORY TESTS

A - Atterberg Limits
G - Grain Size
C - Consolidation
DS - Direct Shear
U - Unconfined Compression

PLATE NO.: A-2

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

UNIFIED SOIL CLASSIFICATION CHART

COARSE-GRAINED SOILS
(more than 50% of material is larger than No. 200 sieve size.)

		Clean Gravels (Less than 5% fines)
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (More than 12% fines)
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
		Clean Sands (Less than 5% fines)
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
		Sands with fines (More than 12% fines)
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures

FINE-GRAINED SOILS
(50% or more of material is smaller than No. 200 sieve size.)

SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	CH	Inorganic clays of high plasticity, fat clays
	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

ESTIMATED PERCENTAGES OF GRAVEL, SAND, AND FINES BASED ON VISUAL DESCRIPTION

TRACE	<5%
FEW	5%-15%
LITTLE	15%-30%
SOME	30%-50%
MOSTLY	>50%

SOIL STRUCTURE COMMON DESCRIPTIVE TERMS

FISSURED: SHRINKAGE OR RELIEF CRACKS OFTEN FILLED WITH SILT OR SAND

POCKET: INCLUSION OF MATERIAL WITH EITHER A DIFFERENT TEXTURE OR CLASSIFICATION FROM THE MAIN SOIL LAYER

LAMINATED: THIN ALTERNATING SOIL LAYERS WITH EITHER A DIFFERENT TEXTURE OR CLASSIFICATION.

SEAM: THIN LAYER OF MATERIAL WITH EITHER A DIFFERENT TEXTURE OR CLASSIFICATION FROM MAIN SOIL LAYER.

MOTTLED: SOILS WITH IRREGULAR MARKS OR SPOTS OF DIFFERENT COLORS. USUALLY INDICATES POOR AERATION AND LACK OF GOOD DRAINAGE. MAY INDICATE A MARKER HORIZON OF A PREVIOUS GROUNDWATER LEVEL.

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

TMWA
WASHOE FOREBAY
SOIL CLASSIFICATION CHART
WASHOE COUNTY, NEVADA

PROJECT NO.:1860 DATE: 05/19/16

PLATE
A-3

PHYSICAL PROPERTIES CRITERIA FOR ROCK DESCRIPTIONS

BEDDING OF SEDIMENTARY ROCKS

SPLITTING PROPERTY	THICKNESS	STRATIFICATION
MASSIVE	GREATER THAN 4.0 FEET	VERY THICK-BEDDED
BLOCKY	2.0 TO 4.0 FEET	THICK-BEDDED
SLABBY	0.2 TO 2.0 FEET	THIN-BEDDED
FLAGGY	0.05 TO 0.2 FEET	VERY THIN-BEDDED
SHALY OR PLATY	0.01 TO 0.05 FEET	LAMINATED
PAPERY	LESS THAN 0.1 FEET	THINLY LAMINATED

FRACTURING

INTENSITY	FRACTURE SPACING (FT) AND CORRESPONDING SPACING DESIGNATION [#] SHOWN ON BORING LOGS	ORIENTATION
VERY LITTLE FRACTURED	GREATER THAN 4.0 [1]	**DIAGONAL:** PREDOMINATE ANGLE IS NEAR 45°
OCCASIONALLY FRACTURED	1.0 TO 4.0 [2]	**HORIZONTAL:** PREDOMINATE ANGLE IS NEAR 0°
MODERATELY FRACTURED	0.5 TO 1.0 [3]	**VERTICAL:** PREDOMINANT ANGLE IS NEAR 90°
CLOSELY FRACTURED	0.1 TO 0.5 [4]	**RANDOM:** PREDOMINANT ANGLE IS NOT CLEARLY DEFINED
INTENSELY FRACTURED	0.005 TO 0.1 [5]	
CRUSHED	LESS THAN 0.005 [6]	

HARDNESS

SOFT= RESERVED FOR PLASTIC MATERIAL ALONE	**MODERATELY SOFT=** CAN BE GOUGED DEEPLY OR CARVED EASILY WITH A KNIFE
MODERATELY HARD= CAN BE READILY SCRATCHED BY A KNIFE BLADE; SCRATCH LEAVES A HEAVY TRACE OF DUST AND IS READILY VISIBLE AFTER THE POWDER HAS BEEN BLOWN AWAY.	**HARD=** CAN BE SCRATCHED WITH DIFFICULTY; SCRATCH PRODUCES LITTLE POWDER AND IS OFTEN FAINTLY VISIBLE.
VERY HARD= CANNOT BE SCRATCHED WITH KNIFE BLADE; LEAVES A METALLIC STREAK.	

STRENGTH

PLASTIC OR VERY LOW STRENGTH	**FRIABLE=** CRUBMLES EASILY BY RUBBING WITH FINGERS
WEAK= AN UNFRACTURED SPECIMEN WILL CRUMBLE UNDER LIGHT HAMMER BLOWS.	**MODERATELY STRONG=** SPECIMEN WILL SITHSTAND A FEW HEAVY HAMMER BLOWS BEFORE BREAKING.
STRONG= SPECIMEN WILL WITHSTAND A FEW HEAVY RINGING HAMMER BLOWS AND WILL YIELD WITH DIFFICULTY ONLY DUST AND SMALL FLYING PIECES	**VERY STRONG=** SPECIMEN WILL RESIST HEAVY RINGING HAMMER BLOWS AND WILL YEILD WITH DIFFICULTY ONLY DUST AND SMALL FLYING FRAGMENTS

WEATHERING

D. DEEP= MODERATE TO COMPLETE MINERAL DECOMPOSITION; EXTENSIVE DISINTEGRATION; DEEP AND THOROUGH DISCOLORATION, MANY FRACTURES, ALL EXTENSIVELY COATED OR FILLED WITH OXIDES, CARBONATES AND/OR CLAY SILT.	**M. MODERATE=** SLIGHT CHANGE OR PARTIAL DECOMPOSITION OF MINERALS; LITTLE DISINTEGRATION; CEMENTATION LITTLE TO UNAFFECTED. MODERATE TO OCCASIONALLY INTENSE DISCOLORATION. MODERATELY COATED FEATURES.
S. SLIGHTLY= NO MEGASCOPIC DECOMPOSITION OF MINERALS; LITTLE OR NO EFFECT ON NORMAL CEMETATION. SLIGHT AND INTERMITTENT, OR LOCALIZED DISCOLORATION. FEW STAINS ON FRACTURED SURFACES.	**F. FRESH=** UNAFFECTED BY WEATHERING AGENTS. NO DISINTEGRATION OR DISCOLORATION. FRACTURES USUALLY LESS NUMEROUS THAN JOINTS.

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TMWA
WASHOE FOREBAY
ROCK DESCRIPTION
WASHOE COUNTY, NEVADA

PROJECT NO.:1860 DATE: 05/19/16

PLATE

A-4

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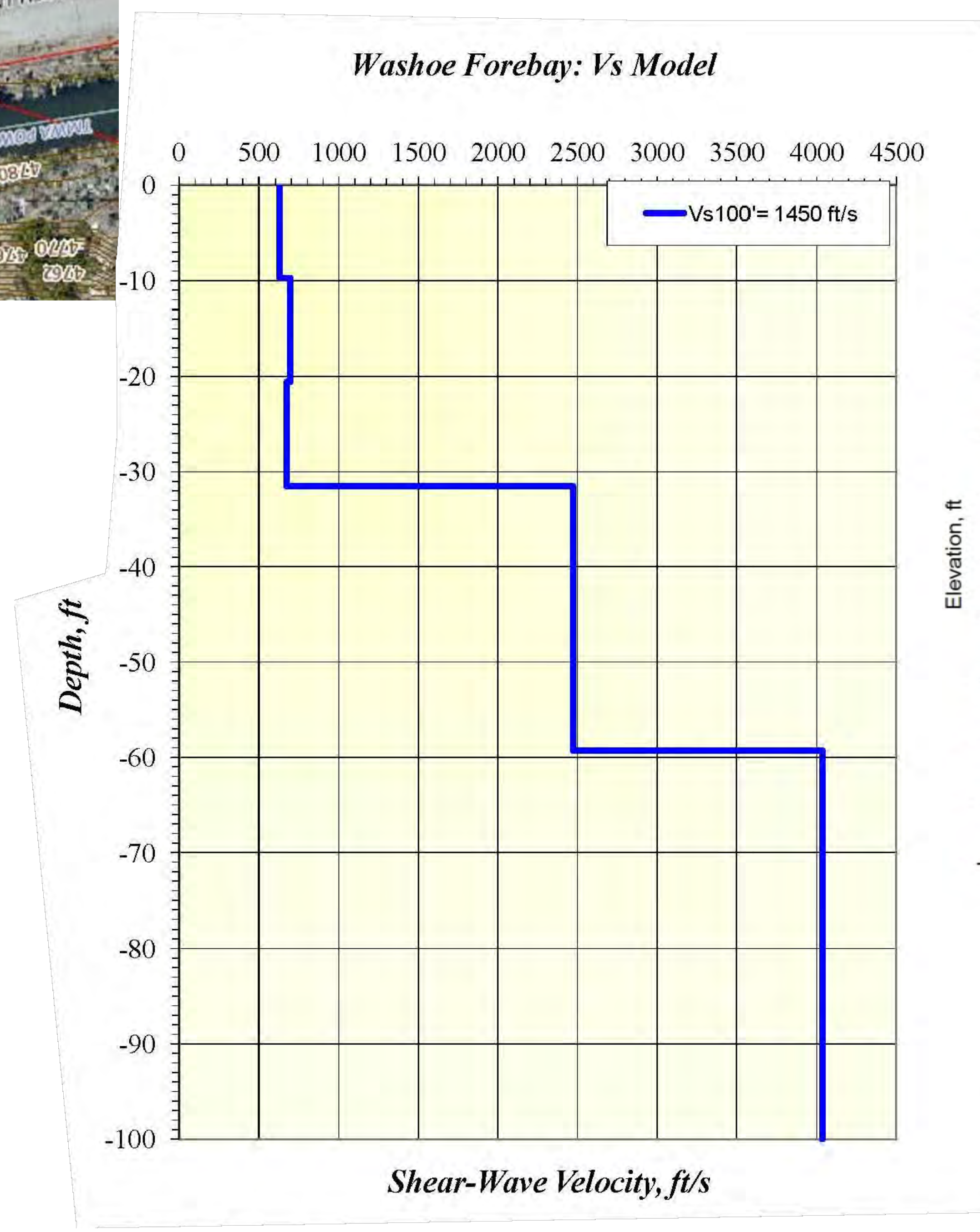
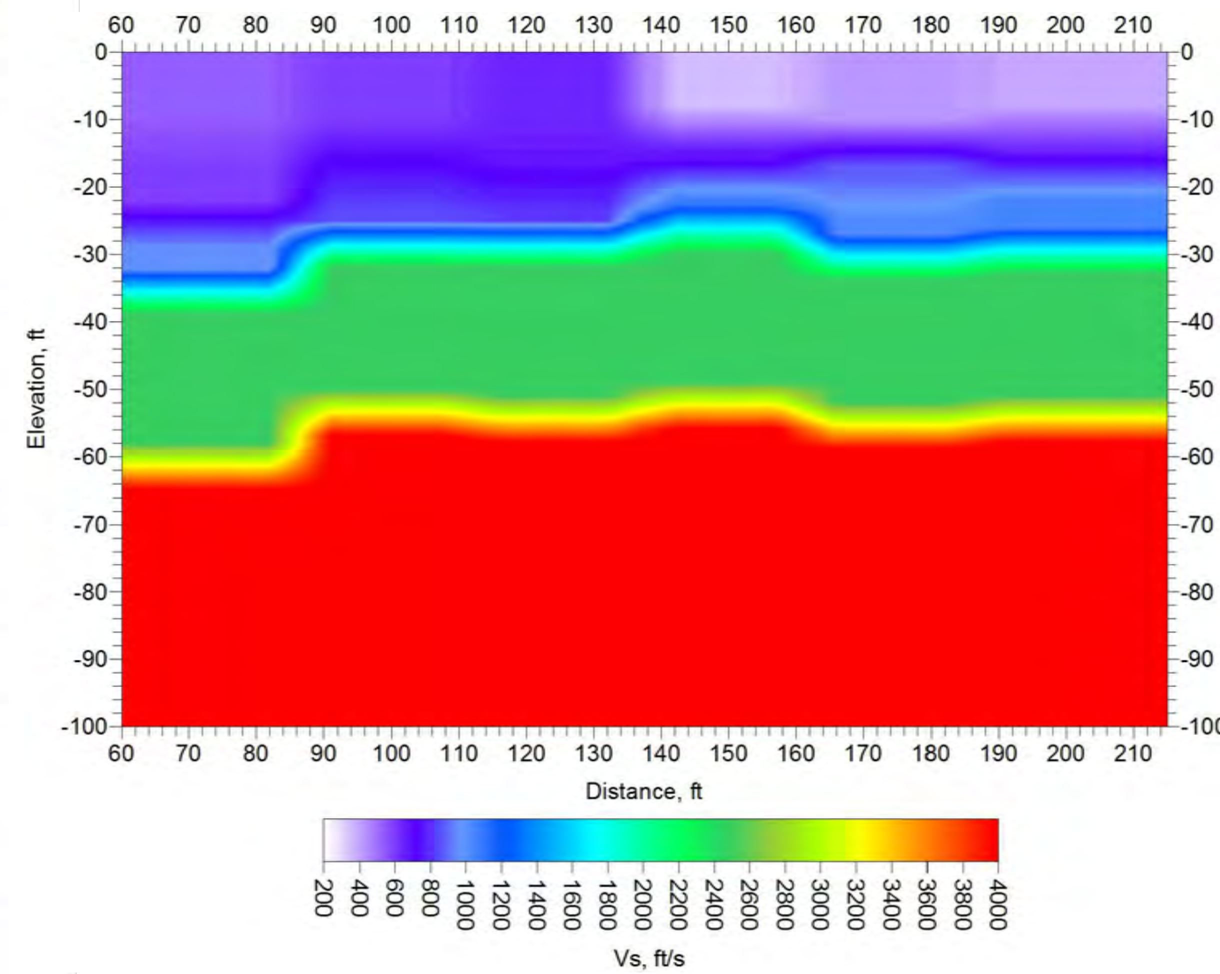
REMI LINE VICINITY MAP
SCALE 1"~ 80 '



REMI LINE -1 DIMENSIONAL PROFILE
N.T.S



REMI LINE B/B' -2 DIMENSIONAL PROFILE
N.T.S

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TRUCKEE MEADOWS WATER AUTHORITY
WASHOE FOREBAY
GEOPHYSICAL FIELD MEASURMENT RESULTS
WASHOE COUNTY, NEVADA

PROJECT NO.: 1860 DATE: 5/18/2016

LEGEND

B-1 APPROXIMATE BORING LOCATION

APPROXIMATE REMI LINE LOCATION

A A' GEOLOGIC CROSS SECTION LOCATION, PLATE A-4

PLATE
A-5

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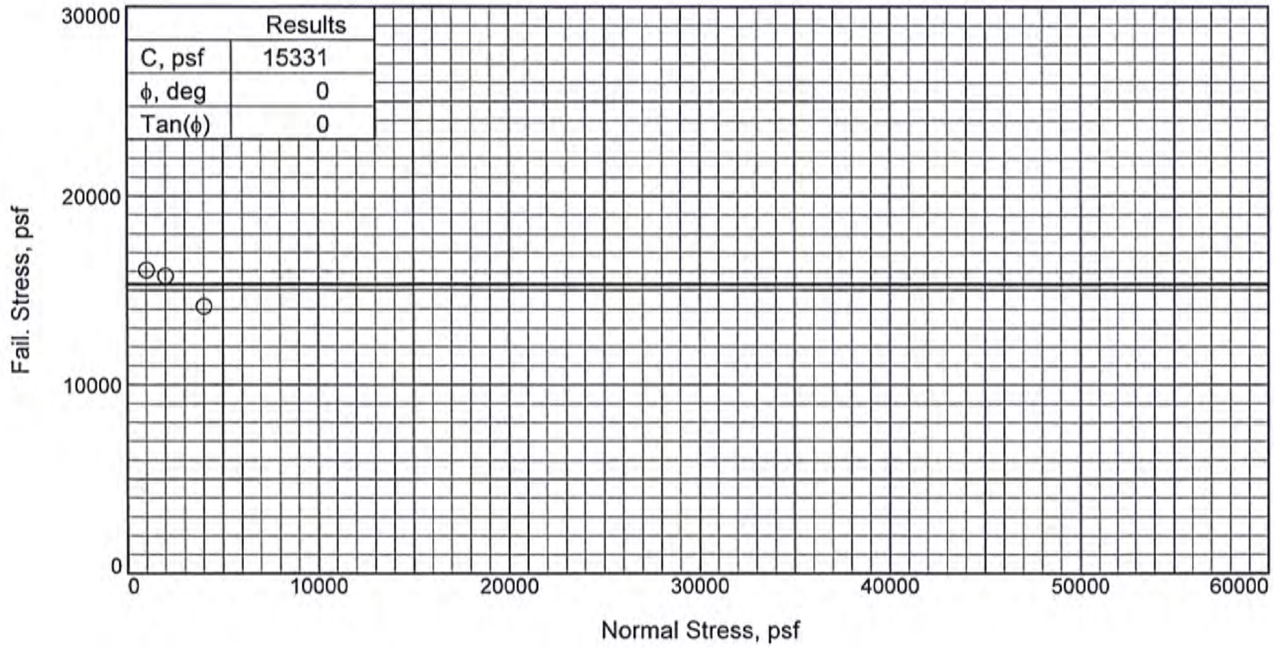
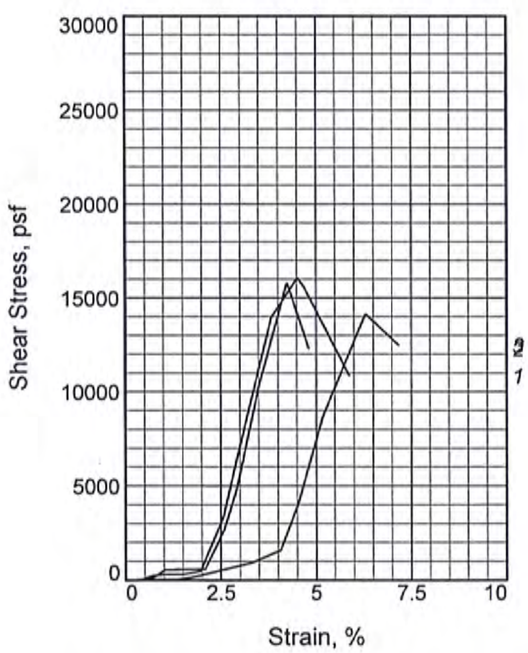
APPENDIX B





Sample No.		1	2	3
Initial	Water Content, %	19.3	26.3	23.0
	Dry Density, pcf	95.7	83.0	91.6
	Saturation, %	70.0	70.1	75.9
	Void Ratio	0.7295	0.9932	0.8051
	Diameter, in.	2.350	2.350	2.350
	Height, in.	0.970	1.030	1.060
At Test	Water Content, %	19.3	26.3	23.0
	Dry Density, pcf	99.4	83.8	92.5
	Saturation, %	76.8	71.5	77.5
	Void Ratio	0.6651	0.9738	0.7881
	Diameter, in.	2.350	2.350	2.350
	Height, in.	0.934	1.020	1.050
Normal Stress, psf		1009	2003	4005
Fail. Stress, psf		16069	15773	14152
Strain, %		4.5	4.3	6.3
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.001	0.001	0.001

Sample Type: INSITU

Description:

Assumed Specific Gravity= 2.65

Remarks: Sample was consolidated prior to performing direct shear.

Plate: ________

Client: TMWA

Project: WASHOE FOREBAY SPILL CHANNEL REPLACEMENT

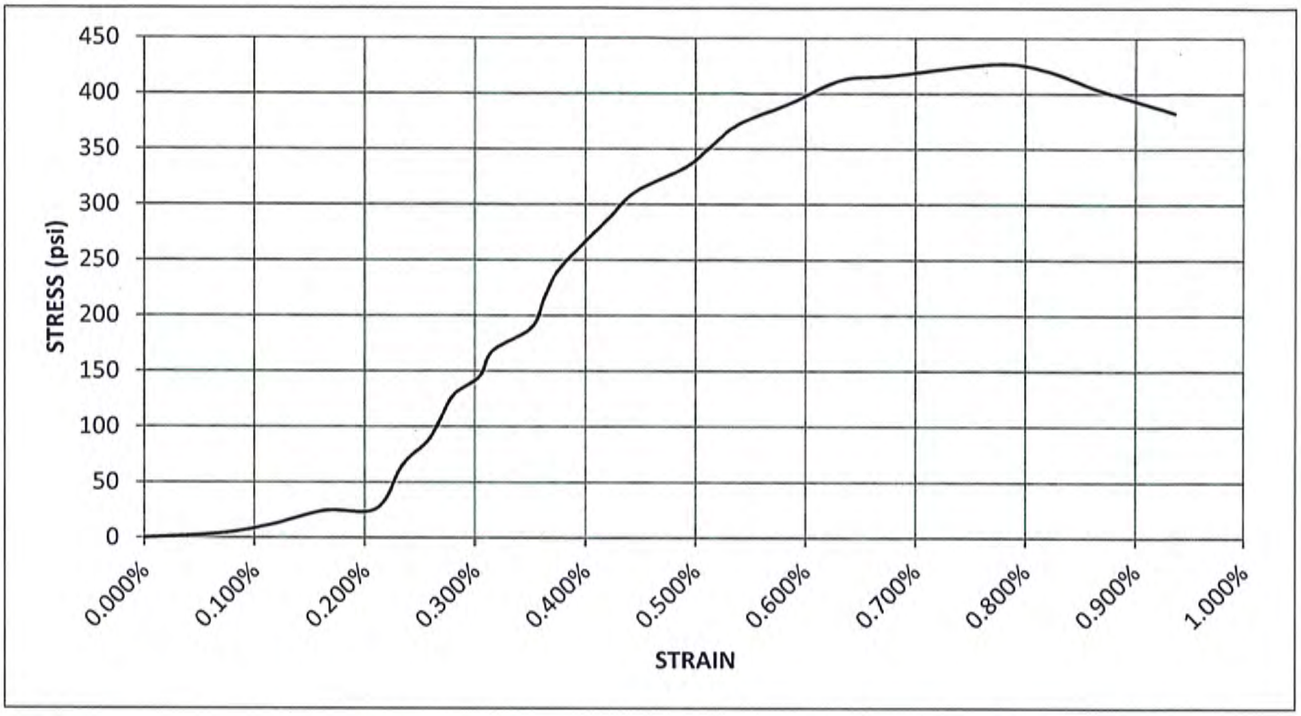
Location: B-1

Proj. No.: 1860 **Date Sampled:**

CME

Tested By: S. Hein **Checked By:** R. Reynolds

Graph of Stress vs. Strain



Sample No.	B-1, 41.0'-41.5'
Water Content, %	28.6
Dry Density, pcf	83.3
Diameter, in	2.39
Height, in	4.27
Strain Rate, in./min	0.025
Height to diameter ratio	1.8
Max Axial Load, lbs	1914
Axial Strain at failure, %	0.77
Max Stress, psi	426.6

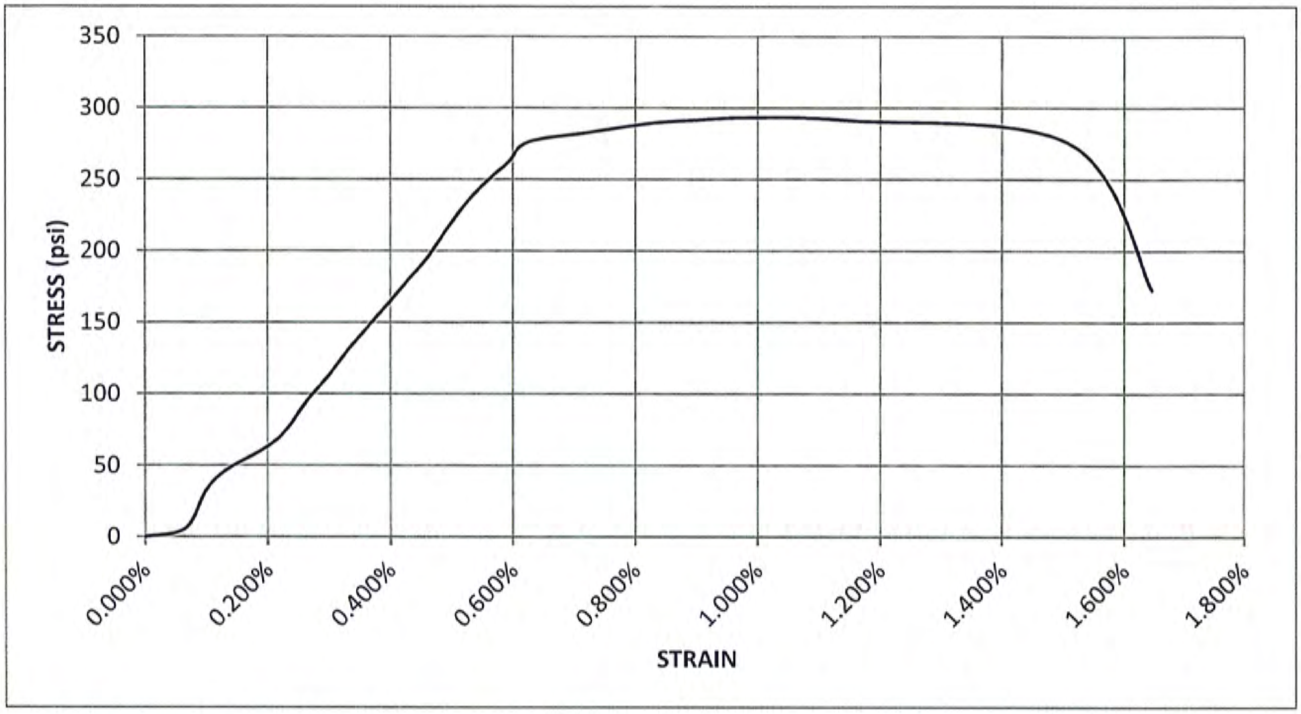
Visual Description:

Remarks:

Tested By: SH Check By: RR

CME CONSTRUCTION MATERIALS ENGINEERS, INC.	**Project:** WASHOE SPILLWAY CHANNEL **Client:** TMWA **Test:** UNCONFINED COMPRESSIVE STRENGTH	**Project NO.** 1863

Graph of Stress vs. Strain



Sample No.	B-1
Water Content, %	
Dry Density, pcf	
Diameter, in	2.401
Height, in	3.039
Strain Rate, in./min	0.025
Height to diameter ratio	1.3
Max Axial Load, lbs	1328
Axial Strain at failure, %	0.99
Max Stress, psi	293.1

Visual Description:

Remarks:

Tested By: SH Check By: RR

CME CONSTRUCTION MATERIALS ENGINEERS, INC.	**Project:** WASHOE SPILLWAY CHANNEL **Client:** TMWA **Test:** Uncofined Compressive Strength	**Project NO.** 1863

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX C

PHOTO #1: CORE DEPTH 10'-13' , NOTE BASALT BOULDER IN SAMPLE, GLACIAL OUTWASH

PHOTO #2: CORE DEPTH 13'-18', NOTE GRANITIC BOULDER VISIBLE IN SAMPLE, GLACIAL OUTWASH

PHOTO #3: CORE DEPTH 33'-38' , HUNTERCREEK SANDSTONE FORMATION

PHOTO #4: CORE DEPTH 36'-43' , BASALTIC LAHAR BEDROCK

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TRUCKEE MEADOWS WATER AUTHORITY
WASHOE FOREBAY
ROCK CORE PHOTOGRAPHS
WASHOE COUNTY, NEVADA

PROJECT NO.: 1860 DATE: 5/19/2016

LEGEND

PLATE
C-1

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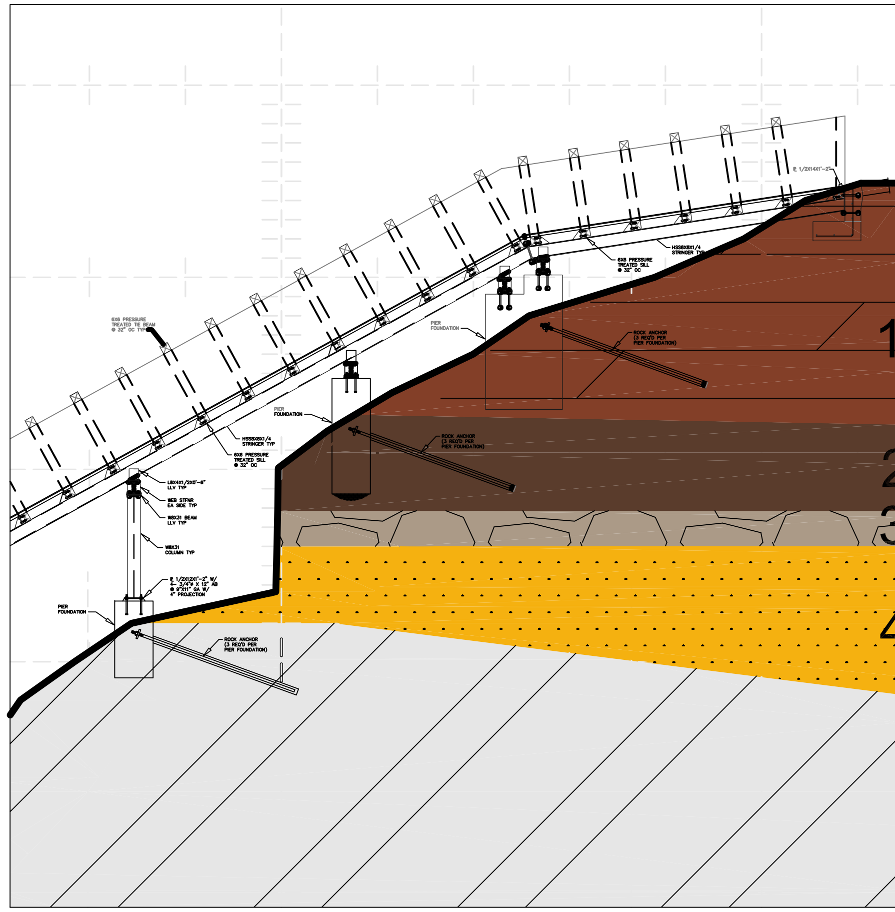
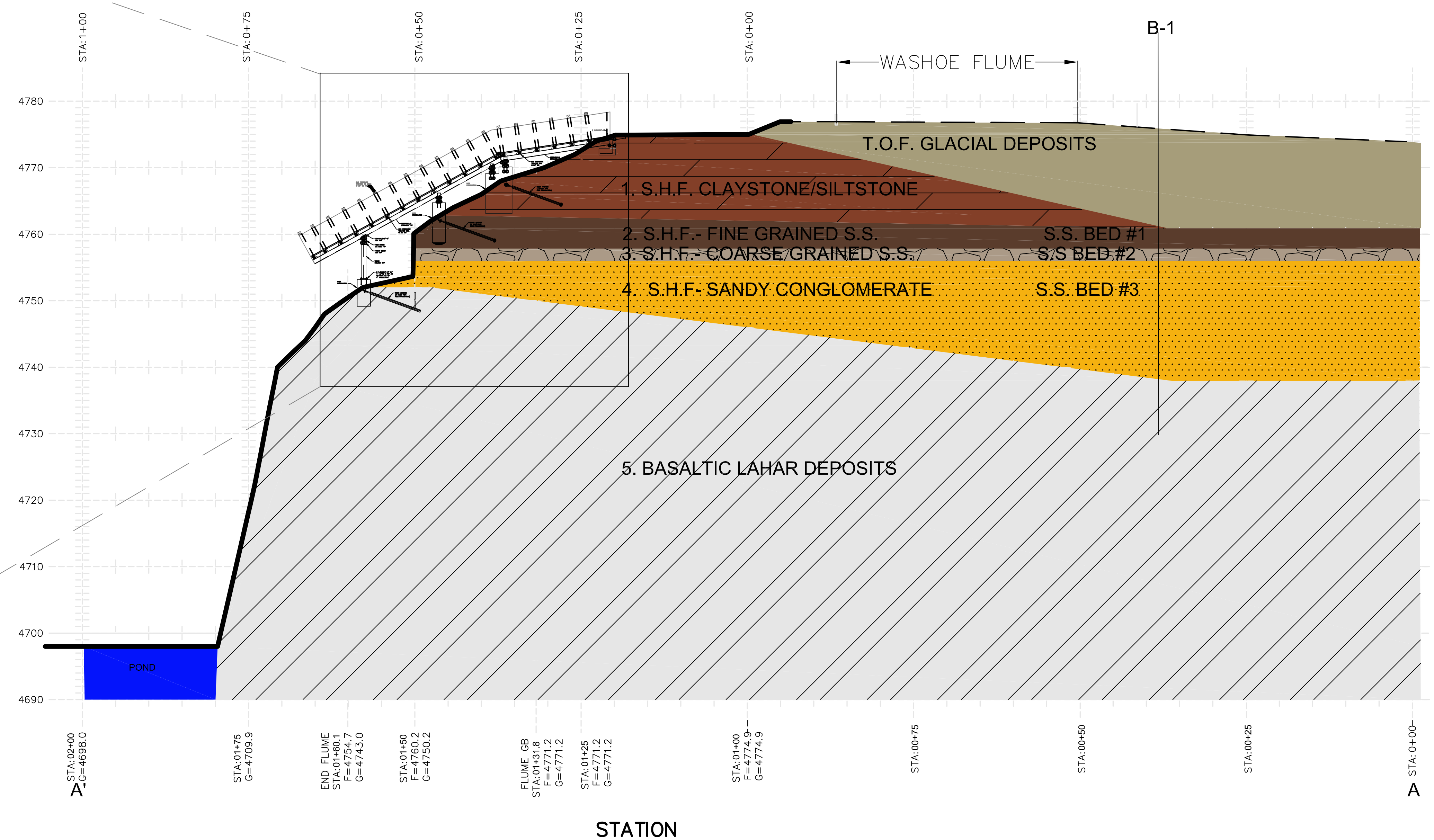
CME CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX D



GEOLOGIC PROFILE NEAR PROPOSED FOUNDATION GRADE

1"=10'



GEOLOGIC CROSS SECTION (A-A')

1"=20'

NOTES:

1) STRATIFICATION LINES SHOWN ON THE GEOLOGIC PROFILE SHOULD BE CONSIDERED APPROXIMATE.

2) SUBSURFACE CONDITIONS ENCOUNTERED DURING CONSTRUCTION MAY VARY FROM THOSE PRESENTED ON THE GEOLOGIC CROSS SECTION.

3) THE TENTATIVE FOREBAY PROFILE AND FOUNDATION LAYOUT PROVIDED BY HARTMAN STRUCTUREAL ENGINEERING, JUNE 2016

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Reno, NV 89511

TRUCKEE MEADOWS WATER AUTHORITY
WASHOE FOREBAY
GEOLOGIC CROSS SECTION
WASHOE COUNTY, NEVADA

PROJECT NO.: 1860 DATE:5/19/2016

LEGEND

PLATE

D-1

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CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX E

USGS Design Maps Summary Report

User–Specified Input

Report Title Washoe Spillway Channel and Weir Wall
Wed April 13, 2016 21:45:00 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 39.505°N, 119.936°W

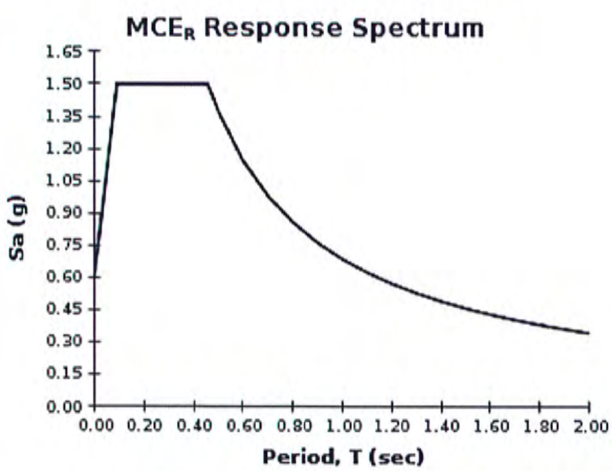
Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

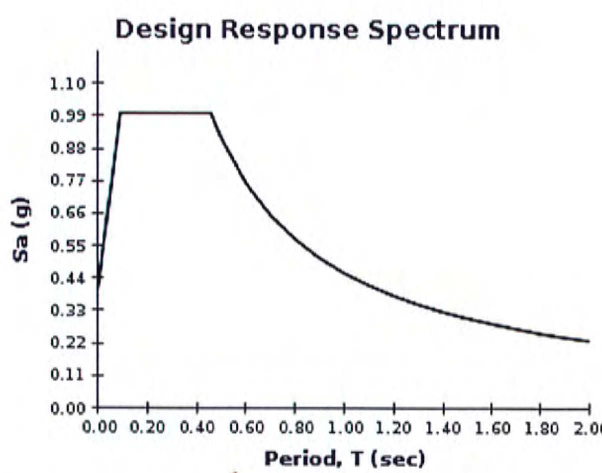
Risk Category I/II/III

USGS–Provided Output

$S_S =$ 1.500 g	$S_{MS} =$ 1.500 g	$S_{DS} =$ 1.000 g
$S_1 =$ 0.528 g	$S_{M1} =$ 0.686 g	$S_{D1} =$ 0.457 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (39.505°N, 119.936°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] S_S = 1.500 g

From Figure 22-2 [2] S_1 = 0.528 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index PI > 20, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = C and S_S = 1.500 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.528 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.500 = 1.500$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 0.528 = 0.686$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

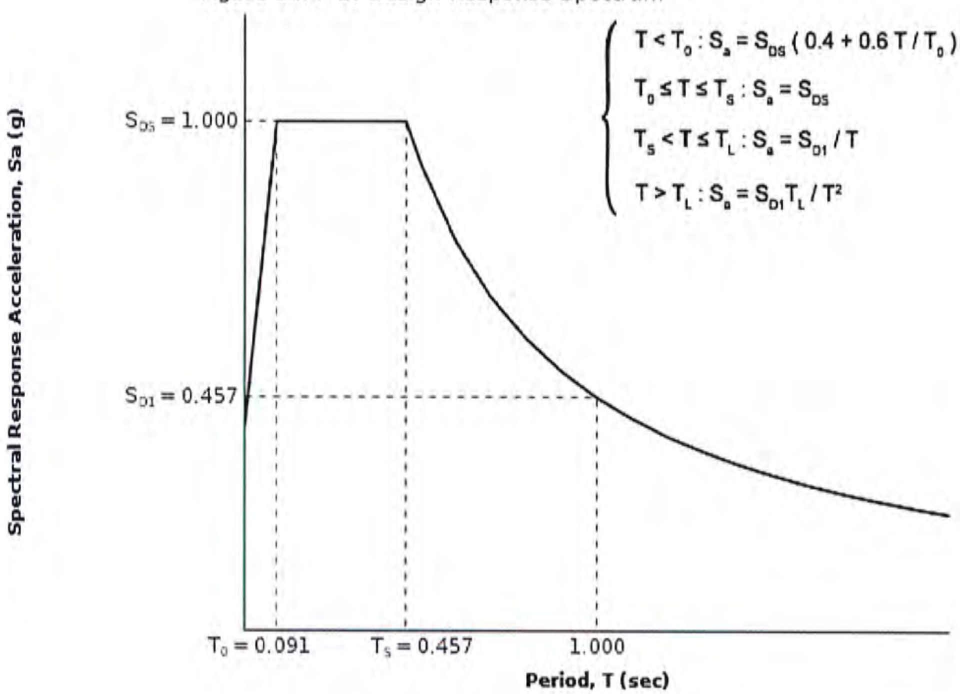
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.500 = 1.000$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.686 = 0.457$ g

Section 11.4.5 — Design Response Spectrum

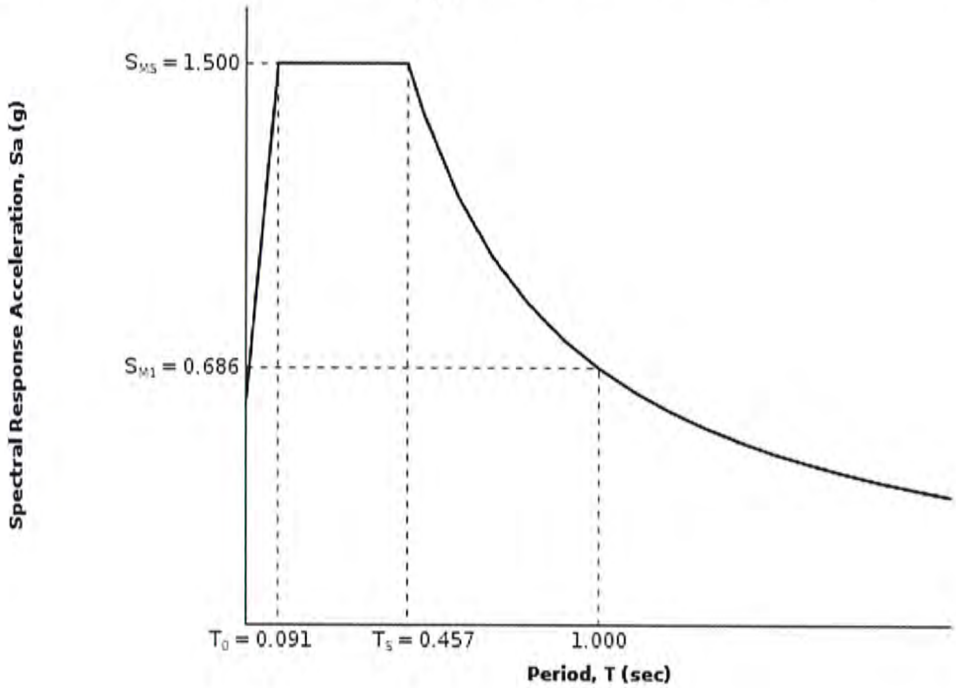
From Figure 22-12 [3] $T_L = 6$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.503

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.503 = 0.503$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = C and PGA = 0.503 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.955

From Figure 22-18 [6] C_{R1} = 0.937

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.000 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.457 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf