Geotechnical Investigation

Mount Rose Water Treatment Plant

Reno, Washoe County, Nevada

Prepared for:

Stantec

April 2017
File: 1875

CONSTRUCTION MATERIALS ENGINEERS, INC.
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April 21, 2017
File: 1875

John Buzzone, PE
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RE: Geotechnical Investigation
Mount Rose Water Treatment Plant
Reno, Washoe County, Nevada

Dear Mr. Buzzone:

Construction Materials Engineers, Inc. (CME) is pleased to submit our geotechnical investigation report for the proposed Mount Rose Water Treatment Plant to be located on the south side of Mountain Ranch Road, in Reno area, 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 appreciate the opportunity to work with you on this project.

Please contact the undersigned if you have any questions.

Sincerely,

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GEOTECHNICAL INVESTIGATION

TMWA Mount Rose Water Treatment Plant
Washoe County, Nevada

1.0 INTRODUCTION

Presented herein are the results of Construction Materials Engineers, Inc. (CME) geotechnical exploration, laboratory testing, and associated geotechnical recommendations for the proposed TMWA Mount Rose Water Treatment Plant to be located on Mountain Ranch Road, in 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 proposed project is located in Sections 35 Township 18N, Range 19E (MDM). Our study included field exploration, laboratory testing and engineering analyses to identify the physical and mechanical properties of the soil types encountered during this investigation. Results of our field exploration and testing programs are included in this report and form the basis for all conclusions and recommendations.

A vicinity map showing the approximate facility location and Diversion Features #1 and #2 are included as Figure 1 (General Project Vicinity).

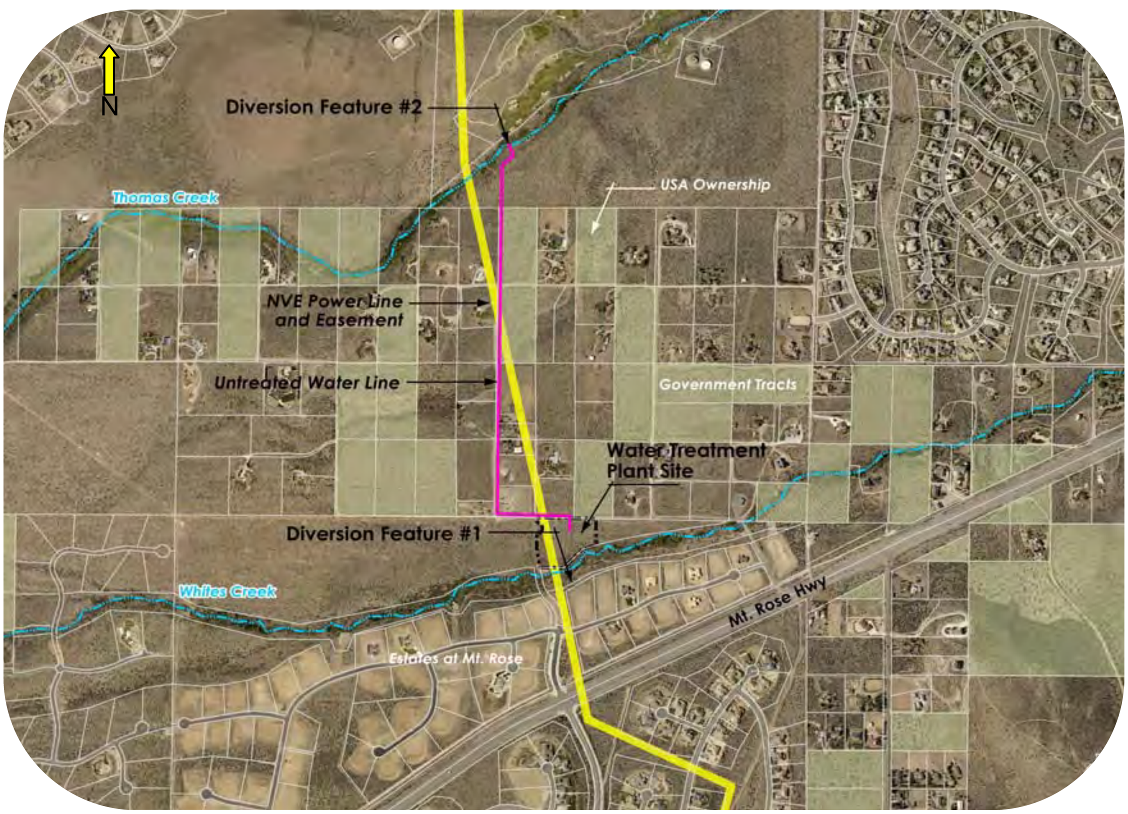


Figure 1: General Project Vicinity[1]

N.T.S

[1] Reference : Figure 1 is a Location Map obtained from Stantec Consulting, May 2016

2.0 PROJECT DESCRIPTION

The project is divided into three different components: Water Treatment Plant, Diversion Feature #1, and Diversion Feature #2.

Diversion Feature #1 is located at the southern end of the project site and will divert water from Whites Creek to a collection point (i.e. pump station/vault) north of the creek bed. Diversion feature #2 is located at the southern bank of Thomas Creek near the Thomas Creek pedestrian/mountain bike trail in the northern end of the project site. Water from both of the diversion features will be pumped to the proposed Water Treatment Plant to be located on the south side of Mountain Ranch Road (refer to Figure 1-Vicinity Map).

2.1 Water Treatment Plant

The Water Treatment Plant will consist of a single story structure supported by shallow spread footings. A water tank will be located on the northeast side of the structure. A generator will be located on the northwest side of structure. A storm water detention basin will be constructed east of the proposed water treatment plant.

Grading will consist of cuts on the order of 12 to 14 feet the north side of the pad and fills on the order of 6 to 8 feet on the south side of the pad.

The site will be accessed from Callahan Ranch Road, a future road to be located west of the site. A tentative site layout is included as Figure 2 (Tentative Site Layout-Mt. Rose Water Treatment Plant).

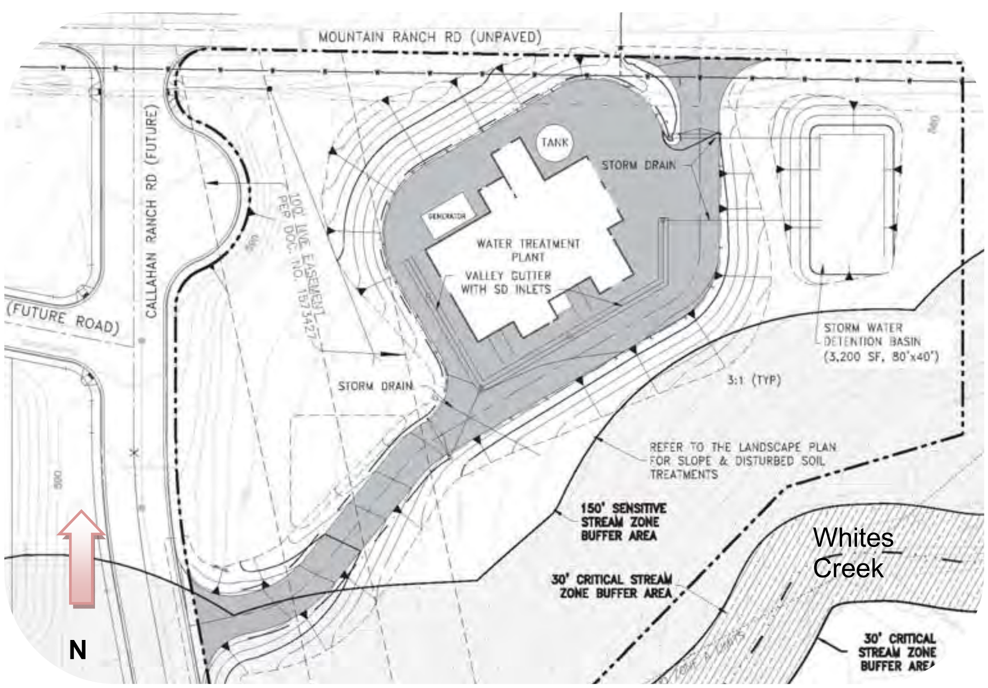


Figure 2: Tentative Site Layout-Mt. Rose Water Treatment Plant[2]
N.T.S

[2] Reference : Figures 2 is a tentative grading map obtained from Stantec Consulting, May 2016

2.2 Diversion Feature #1

This diversion spans approximately 200 to 300 lineal feet of Whites Creek (located south of the proposed Water Treatment Plant). The diversion may include construction of an infiltration gallery, an 8 foot diameter in-ground pump station extending approximately 12 to 14 feet below existing grade, and a 12 inch diameter water line leading from the pump station to the water treatment plant. Appurtenant construction may include subsurface electrical feed to the booster pump station with transformer.

A tentative site layout is included as Figure 3 (Diversion Feature #1-Tentative Site Layout).

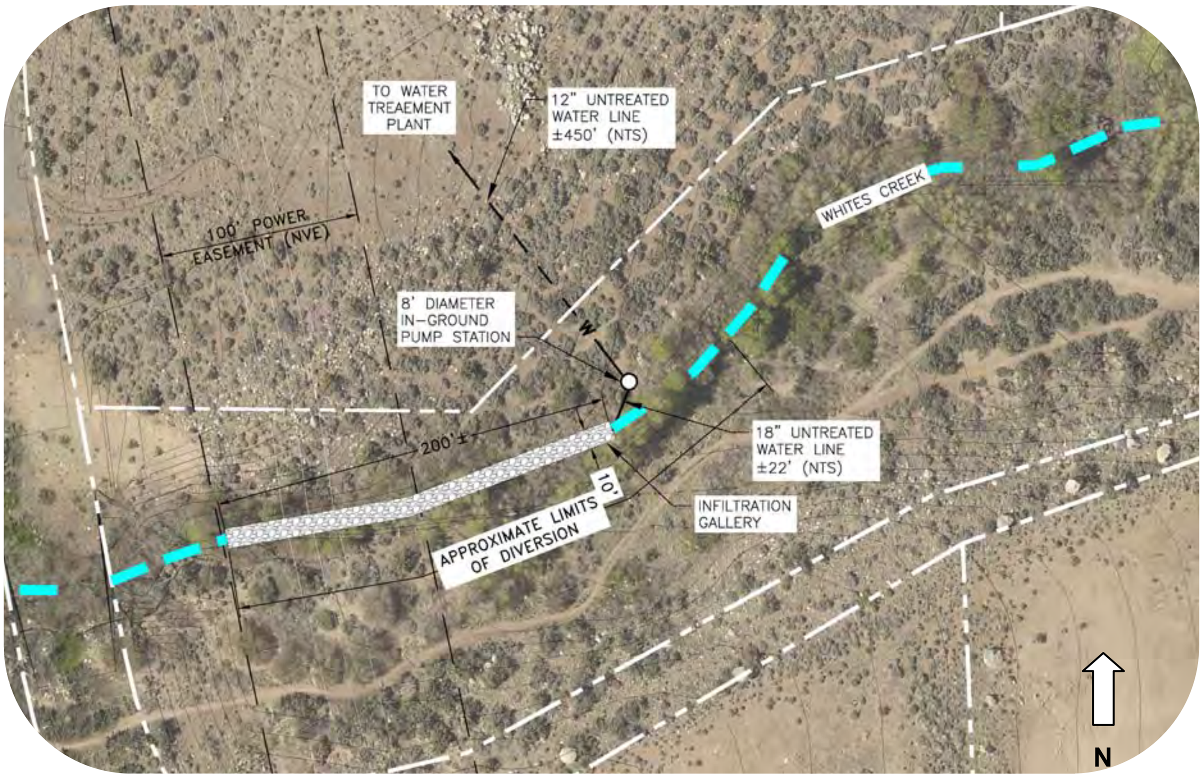


Figure 3: Tentative Site Layout-Diversion Feature #1[3]
N.T.S

[3] Reference : Figures 3 is the tentative site layout for Whites Creek Diversion obtained from Stantec Consulting, May 2016

2.3 Diversion Feature #2

This diversion has a width of 10 feet and spans approximately 200 lineal feet of Thomas Creek (located approximately 3,300 feet north of the proposed Water Treatment Plant). The diversion will include an 8 foot diameter in-ground pump station extending approximately 12 to 14 feet below existing grade, and a 12 inch diameter water line leading from the pump station, south, to the water treatment plant. Appurtenant construction may include subsurface electrical feed to the booster pump station, transformer, and pump station control panel.

A tentative site layout is included as Figure 3 (Diversion Feature #2-Tentative Site Layout).

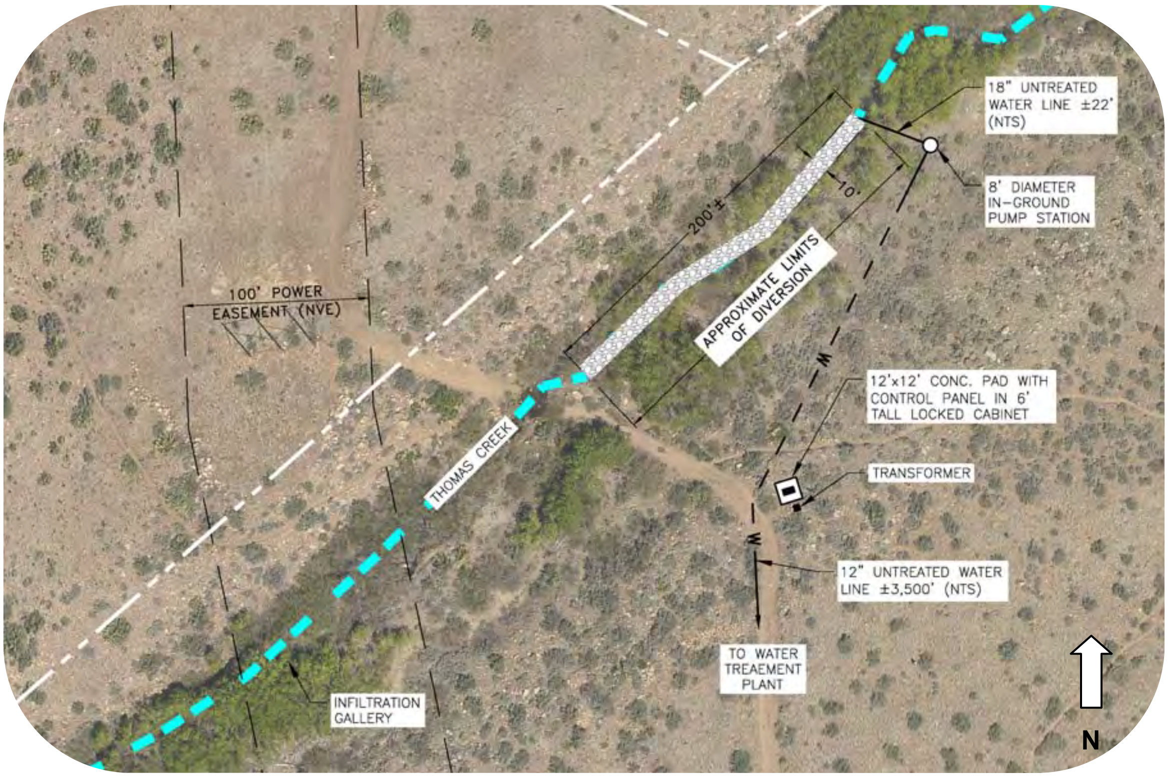


Figure 4: Tentative Site Layout-Diversion Feature #2[4]
N.T.S

[4] Reference : Figures 4 is the tentative site layout for Thomas Creek Diversion obtained from Stantec Consulting, May 2016

3.0 SITE CONDITIONS

3.1 Water Treatment Plant

The proposed water treatment plant site is currently undeveloped. Existing site improvements include overhead electrical lines, roughly graded dirt roads/trails, and site grading that includes the placement of undocumented fills. Based on a review of Google Earth historic imagery, it appears the project site was graded sometime between 2004 and 2006. Several feet of undocumented fill appear to have been placed. Figure 5 (Approximate Limits of Undocumented Fill) shows the approximate limits of the undocumented fill and site grading based on our field observations and imagery review. A large pile of cobbles and boulders (up to 48 inches nominal diameter) is located near the southern end of the graded area.

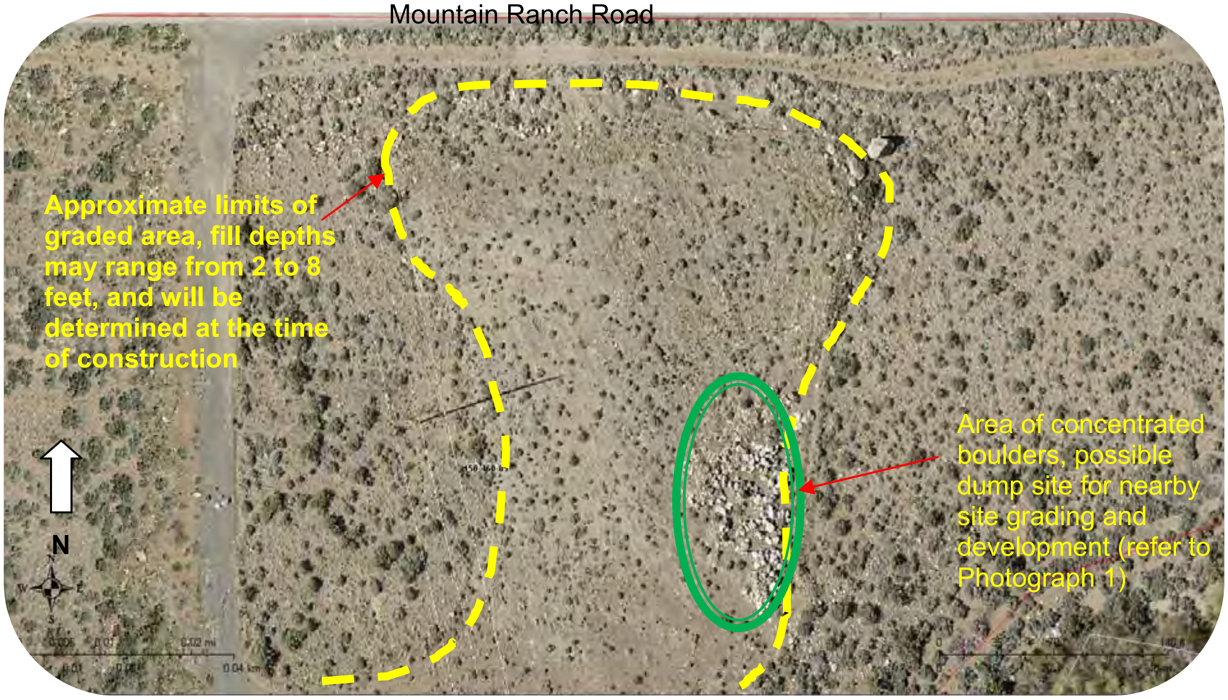


Figure 5: Approximate Limits of Undocumented Fill[5]
N.T.S
(Reference Washoe County GIS Quick Map available at http://wcgisweb.washoecounty.us/quickmap/, accessed June 2016)

5 Undocumented fill may be present in other areas of the project, the limits presented on Figure 5 should be considered approximate and will need to be evaluated during construction.

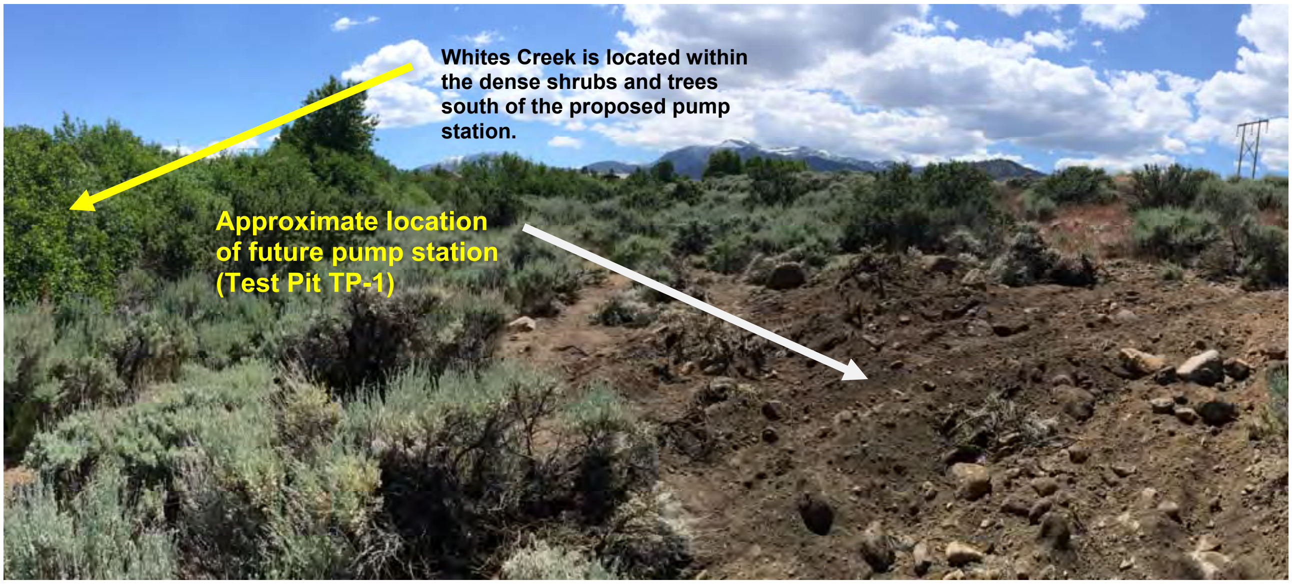
Photograph 1: Note abundant boulders up to 48 inches nominal diameter visible near the south side of the future water treatment plant building pad.

The project site is sloping in a southeasterly direction at an approximate gradient of 8 percent. Vegetation at the site consists of moderately dense to dense shrubs, brush, wheat grass, and other short grasses.

Photograph 2: Taken looking northeast to south near the intersection of Mountain Ranch Road and the future Callahan Ranch Road.

3.2 Diversion Feature #1

Diversion Feature #1 is located down-slope of the proposed water treatment plan. The proposed in-ground pump station is located at the base of an 8 to 10 foot slope. Vegetation at the in-ground booster pump station consists of moderately dense tall brush and shrubs that becomes dense near the proposed Diversion Feature #1 located along Whites Creek.

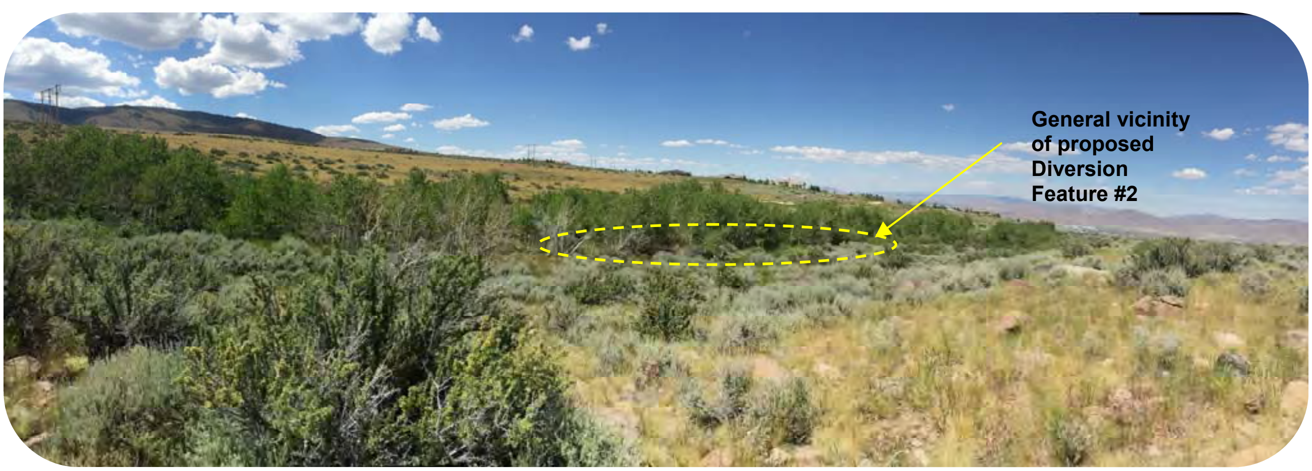


Photograph 3: Diversion Feature #1 site **looking west at Test Pit TP-1 (proposed in-ground pump station site)**

The site is moderately sloping in a south easterly direction at an approximate gradient of 13 percent.

3.3 Diversion Feature #2

Diversion Feature #2 is located north of the Thomas Creek Trail and is not accessible with conventional two wheel drive equipment. The nearest dirt access road is located approximately 150 feet west of the site.



Photograph 4: Taken looking northeast toward Diversion Feature #2

Moderately dense vegetation consisting of tall brush, shrubs, and grasses covers a majority of the site. Dense groupings of mature trees including aspens and willows are present along the banks of Thomas Creek. The diversion is bound to the north by Thomas Creek, the south by Thomas Creek trail, and the east and west by undeveloped land.

4.0 FIELD EXPLORATION

4.1 Test Pit Exploration

Subsurface field exploration, completed in May 2016, consisted of excavating 4 test pits to depths ranging from 9½ to 18½ feet below existing grade (bgs). Three test pits (one at Diversion Feature #1, and two at the Water Treatment Plant) were excavated using a track mounted Case 9020 excavator.

Due to site access restrictions and to limit damage to existing vegetation and trails near the site, Test Pit TP-4 was located approximately 500 feet west of proposed Diversion Feature #2. Test Pit TP-4 was completed using a John Deere 310SG [6]rubber-tired backhoe equipped with a 36-inch bucket.

Test pits were located in the field by visual sighting and/or measuring from existing features at the site. Approximate locations of the test pit excavations are presented on Plate A-1 (Field Exploration Location Map).

Soils encountered within the test pit excavations were visually classified in general accordance with ASTM D 2488 (Description and Identification of Soils). Bulk samples of representative soil strata were collected, placed in sealed plastic bags and returned to our Reno office for laboratory testing.

Test pits were backfilled using the equipment at hand; back-fill was loosely placed and not compacted to the standards typically required for properly placed structural fill[7].

Test pit logs are included as Plates A-2. Elevations shown on the test pit logs were obtained by interpolation between contour lines shown on the attached Field Exploration Location Map (Plate A-1). Elevations and locations included in this report should be considered accurate only to the degree implied by the methods used.

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. A description of the USCS is presented on A-3.

4.2 Creek Bed Sampling

Bulk samples of the existing creek bed materials were collected at a single location on both Whites Creek and Thomas Creek. The sample locations were chosen in the field, by a representative of Stantec.

Bulk samples were collected using a hand shovel and large cobbles and boulders, present within the creek beds, were excluded from sampling.[8]. Sampling at Thomas Creek was attempted at multiple locations along the proposed diversion alignment; however, the presence of large cobbles and boulders made sampling difficult with the tools at hand. Ultimately the sample location was moved upstream within the existing creek crossing where bulk sampling using the hand shovel was achievable.

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[6] The track mounted excavator could not be transported to the site as permission was not obtained to traverse the existing privately owned roads in the area.

[7] **<u>Warning:</u>** Structures and or slabs constructed over loosely placed back-fill may experience significant settlement and/or differential settlement. Removal and densification during replacement of back-fill may be required prior to construction over these areas.

[8] Note: Due to the current flow rates within the creeks at the time of sampling, some of the finer materials may have been washed away before sample was captured and bagged.

Samples were obtained from different sections of the creek bed with the objective of collecting both coarse grained (i.e. sand and gravels) and fine to medium grained materials (i.e. sand/silt).

Laboratory test results from the creek bed samples are included as Appendix C.

4.3 Geophysical Measurements

A ReMi array was performed near the center of the proposed Water Treatment Plant Building Pad. The ReMi array was completed 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 25 feet were used to obtain surface wave data which 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.

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 SPT blow count information.

The resulting 1-dimensional shear wave velocity model (Vs_{100}) is included as Figure 6 (Shear-wave Velocity (Vs) Model). The approximate ReMi line location is shown on Plate A-1 (Field Exploration Location Map)

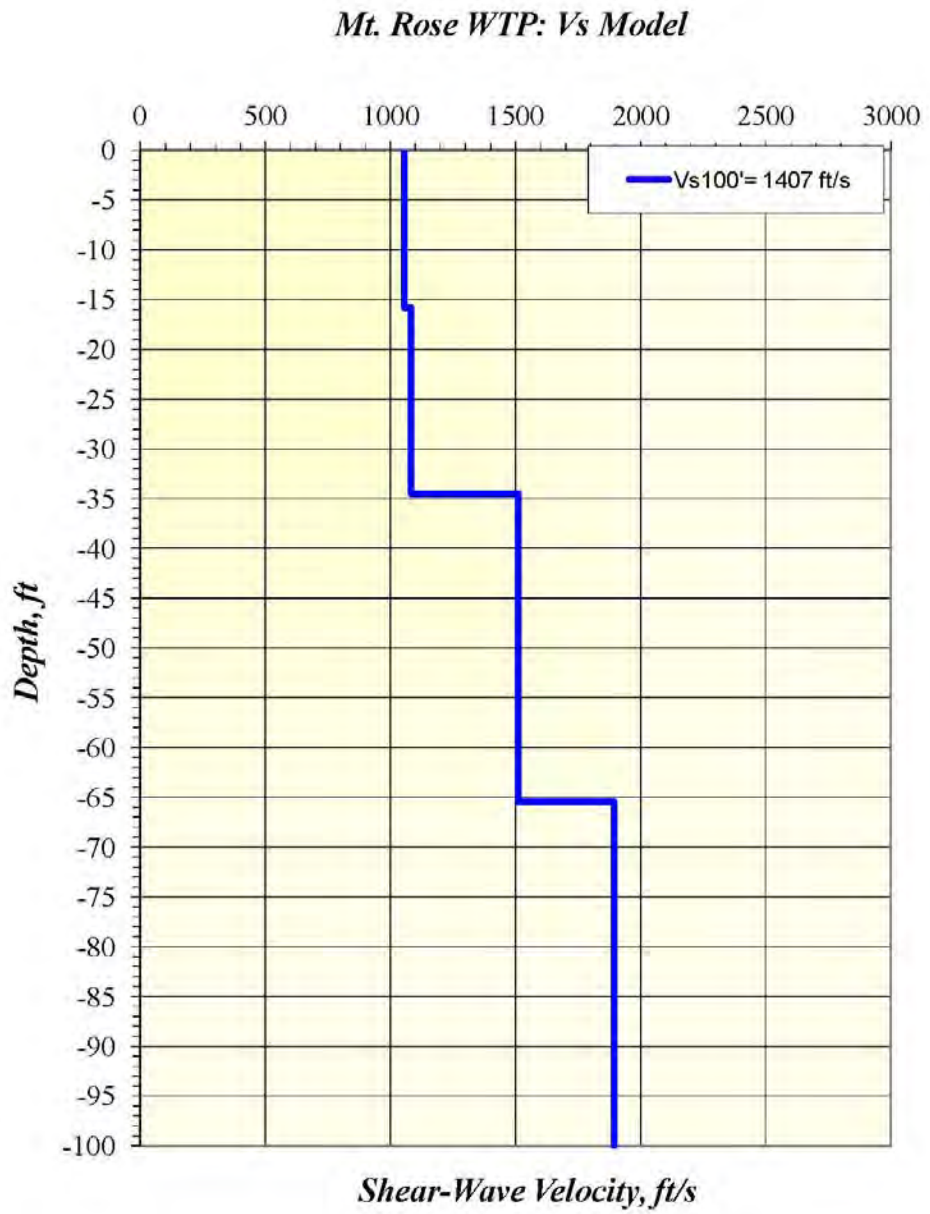

Figure 6: Shear Wave Velocity (Vs) Model

N.T.S

5.0 LABORATORY TESTING

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

Significant soil types collected during test pit exploration were selected and analyzed to determine index properties. The following laboratory tests were completed as part of this investigation:

- *Insitu* moisture content (ASTM D 2216);
- Grain size distribution (ASTM D 422); and
- Atterberg Limits (ASTM D 4318); and
- Corrosion testing including resistivity (ASTM G57), soluble sulfates (ASTM 1580C), and pH (SW-846 9045D) was completed by an outside laboratory.

Laboratory test results for the subsurface exploration are presented on the test pit logs and included as Appendix B. Test results for the creek bed sampling are included as Appendix C.

6.0 SUBSURFACE SOILS AND GROUNDWATER CONDITION

The *Mt. Rose NE Quadrangle Geologic Map,* (Bonham, et. al., 1983), maps the geologic profile at all of the three different project components as Quaternary aged Donner Lake Outwash-Mount Rose Fan Complex. Donner Lake Outwash-Mount Rose Fan Complex is described as pediment and thin fan deposits from major streams draining alpine glaciers on Mount Rose. Soils are described as brown to brownish-grey sandy, muddy, poorly sorted large pebble gravel, cobbles and small boulders. Argillic soil horizons are typically present within the glacial outwash deposits.

More recent geologic mapping (Ramelli et al., 2011) indicates the water treatment plant site is underlain by Quaternary alluvial fan deposits consisting of glacial outwash on the north half of the site transitioning to quaternary aged stream deposits near Diversion Feature #1.

Native soils encountered during the subsurface exploration appear to be similar to the mapped soil types. A general soils profile description for each site is included as Sections 6.1 to 6.3.

6.1 General Soils Profile Water Treatment Plant

Test Pits TP-2 and TP-3 were excavated at the proposed water treatment plant pad location. Soils encountered within Test Pit TP-2 consisted of undocumented fill soils extending to a depth of about 8 feet bgs. The upper 3 feet of the undocumented fill soils encountered consisted of silty sand with gravel **(SM)**. A trace amount of cobbles and boulders up to 18 inches were also encountered within this horizon. At a depth of 3 feet, fill soils consisted of clayey sand with gravel **(SC)**. Below a depth of 8 feet, native soils were encountered and consisted of silty sand with gravel **(SM)** underlain by poorly graded gravel, cobbles and boulders with sand **(GP)** to the depth of exploration.

The uppermost soil horizon encountered in Test Pit TP-3 consisted of poorly graded gravel with cobbles, boulders, clay and sand **(GP)**. Boulders up to 48 inches nominal diameter were present on the ground surface. Below a depth of 1½ feet to 6 feet bgs, soils encountered consisted of clayey sand with gravel **(SC)** exhibiting moderate plasticity characteristics. Silty sand with gravel **(SM)** was encountered from a depth of about 6 to 12 feet bgs, underlain by poorly graded gravel, cobbles and boulders with sand. Excavation refusal was encountered at a depth of 13 feet on nested boulders.

The clayey sand with gravel fill **(SC)** and clayey sand with gravel **(SC)** encountered within the test pit explorations is considered *potentially expansive* with a plasticity index ranging from 15 to 25 and a percent finer than the #200 sieve greater than 20 percent.

6.2 General Soils Profile Diversion Feature #1

Test Pit TP-1 was excavated in the general vicinity of the proposed 8 foot diameter in-ground pump station. Soils encountered were classified as either silty sand **(SM)** or silty sand with gravel and cobbles **(SM)** to a depth of approximately 7 feet bgs. Below a depth of 7 feet bgs, soils encountered were classified as silty gravel with sand **(GM)**. At a depth ranging from 10 to 13 feet, soils become indurated (hardpan layer), but are easily broken using the excavator bucket. Below a depth of 13 feet, soils encountered were classified as either poorly graded gravel with silt **(GP-GM)** or poorly graded sand with gravel **(SP)**. The lower most soil layer encountered at 18 feet was clayey sand **(SC)** to the maximum depth of exploration.

6.3 General Soils Profile Diversion Feature #2

Diversion Feature #2 was not accessible at the time of our field exploration. However, Test Pit TP-4 is located approximately 500 feet northwest of the proposed pump station on the west side of the dirt access road and north of the Thomas Creek Trail (Plate A-1).

Soils encountered in Test Pit TP-4 were classified as either silty clayey sand **(SC-SM)** or silty clayey sand with gravel **(SC-SM)** exhibiting low plasticity characteristics to a depth of 7 feet bgs. The lowermost soil horizon encountered to the depth of refusal (9 ½ feet) consisted of moderately cemented soils classified as silty, clayey gravel with sand **(GC-GM).**

6.4 Groundwater Conditions

Groundwater was not encountered during the subsurface exploration. In general soils were encountered in a moist condition. A very moist to wet lense of poorly graded sand with gravel **(SP)** was encountered at a depth of 17 feet in Test Pit TP-1. The test pit was left open for approximately 1 hour and no significant seepage was observed. However, based on the presence of the clayey sand layer encountered at a depth of 18 feet, it is possible that groundwater may temporarily perch above this depth.

Although groundwater was not encountered during the subsurface exploration, subsurface seepage or perched water may be encountered during construction, especially in areas of deep cuts. Fluctuations in groundwater levels may occur due to increased precipitation, snow melt, irrigation; and proximity to Whites Creek and Thomas Creek.

7.0 SEISMIC CONDITIONS AND GEOLOGIC HAZARDS

7.1 Seismicity

The Western United States is a region of moderate to intense seismicity related movement of the crustal masses (plate tectonics). The most active regions outside of Alaska are along the San Andres Fault zone of western California and the Wasatch Front in Salt Lake City.

Figure 7: Overview Map Showing the Great Basin (N.T.S)
(Image obtained from https://upload.wikimedia.org/wikipedia/commons/5/56/Greatbasinmap.png)

The Wasatch Front in Salt Lake City, Utah, 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 subject site is located in the Eastern Sierra Nevada Mountains, a seismically active zone within the western extreme of the Basin and Range.

7.2 Faults

The project site is situated within the northwestern margin of the Carson Range fault complex. Numerous fault traces associated with the Mount Rose fault zone are mapped near the proposed project boundaries which extend from Thomas Creek south to Whites Creek. Holocene[9] active fault traces have been mapped near the proposed Water Treatment Plant and Diversion Feature #1.

[9] Quaternary earthquake fault evaluation criteria have 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 later 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

Due to the close proximity of mapped Holocene fault traces, a fault study was completed by Piedmont Geosciences, Inc in May 2016 for the Water Treatment Plant site.

7.2.1 Fault Study Water Treatment Plant and Diversion Feature #1

The fault study completed by Piedmont Geosciecnes, Inc. included a review of published geologic maps and technical data pertaining to surface rupture hazards, a stereoscopic review of aerial photography, and field reconnaissance to review existing features present at and around the project site.

The results of this fault study determined that the nearest fault trace is located approximately 250 to 500 feet west of the westerly site boundary. No evidence of faulting at the site was observed during the field reconnaissance. The results from the fault study are attached as Appendix D.

7.2.2 Diversion Feature #2 Nearby Faults

Based on a review of the reference geologic map (Ramelli et. al, 2011), no mapped faults traverse the proposed diversion and in-ground pump station sites. The nearest mapped fault trace is located over 500 feet west of the proposed diversion and in-ground pump station. This fault trace is associated with the Mount Rose Fault zone and is mapped as Quaternary aged (<130,000 years).

An older fault trace (<750,000 years) was mapped trending in a north-south direction near the proposed diversion on previous fault maps, but has since been removed based on the most recent geologic map of the area (Ramelli et. al, 2011). An excerpt from the NBMG Interactive Fault Map showing the general vicinity of Diversion Feature #2 is shown on Figure 8 (Excerpt from NBMG Interactive Fault Map).

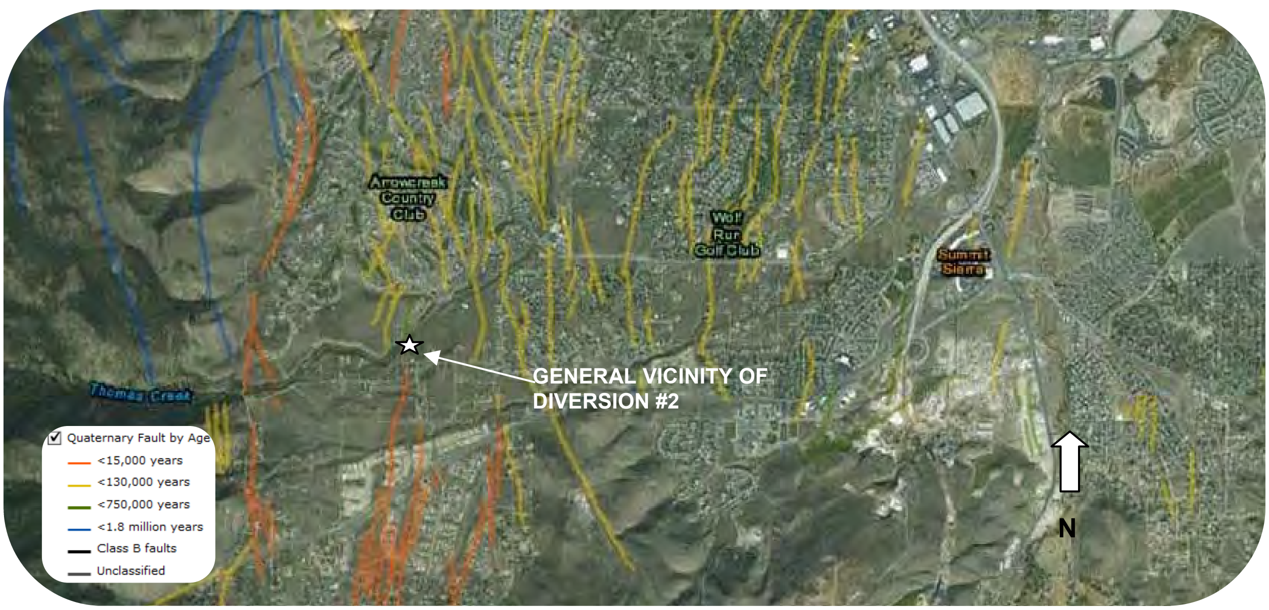

Figure 8: Excerpt from NBMG Interactive Fault Map
(NBMG Quaternary Faults in Nevada, available at https://gisweb.unr.edu/flexviewers/quaternary_faults/)

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potential for activity compared to the Holocene Active Fault. An inactive fault is considered as a fault that does not comply with these age groups.

7.3 Seismic Design Parameters

Seismic design parameters are based on site-specific estimates of spectral response ground acceleration as designated in the 2012 IBC. 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.

Table 1 – Site Classification Definition

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 hardness (based on shear wave velocity primarily for bedrock sites). These two criteria have to be determined to a depth of 100 feet below the ground surface.

SPT testing was not an applicable method of determination for the project site due to the presence of coarse gravels, cobbles and boulders; therefore, geophysical testing at the Water Treatment Plant was completed (results presented as Figure 6, Section 4.3 (Geophysical Measurements)).

Results of the ReMi indicate that the S-wave weighted average (Vs) of the upper 100 feet of the subsurface profile is 1,407 feet per second (fps). Based on this shear wave velocity, a Site Class C is recommended for project design. Table 2 (Seismic Design Parameters (2012 IBC)) provides a summary of seismic design parameters including correction factors F_a & F_v for a Site Classification of C. Copies of the USGS Design Map Summary Reports are included as Appendix E.

Table 2-Seismic Design Parameters (2012 IBC) Water Treatment Plant	
Parameter Description	**Parameter**
Approximate Latitude of Site	39.38881
Approximate Longitude of Site	119.83219
Spectral Response Acceleration at short period (0.2 sec.),S_s (for Site Class B)	2.101
Spectral Response Acceleration at 1-second Period, S_1 (for Site Class B)	0.760
Site Class Selected for this Site	C
Site Coefficient F_a, decimal	1.0
Site Coefficient Fv, decimal	1.3
Peak Ground Acceleration-MCE_R PGA_M (ASCE 7-10 Standard)[1]	0.827g
Design Spectral Response Acceleration at Short period, S_{Ds} (Adjusted to Site Class D, SDs= 2/3 SMs)	1.400g
Approximate Design Earthquake Ground Motion (S_{DS}/2.5-(FEMA/NEHRP,2003)	0.56g
Design Spectral Response Acceleration at 1-second Period, S_{D1} (Adjusted to Site Class D, SD1=2/3 SM1)	0.658g

Notes:
1) MCE_R PGA_M- Maximum credible earthquake geometric mean peak ground acceleration corrected for Site Class.

8.0 DISCUSSION AND RECOMMENDATIONS

8.1 Construction Concerns

Based on the results of our field observations, subsurface exploration and laboratory test program, the project site may be developed as currently proposed provided recommendations of this report are implemented during design and construction of the project.

Primary construction concerns are as follows:

- The presence of undocumented fill soils encountered at the proposed Water Treatment Plant building pad. A limited evaluation of the undocumented fill soils was completed and consisted of excavating one test pit (TP-2) in the existing undocumented fill area. It is unclear if the soil types encountered within Test Pit TP-2 are uniform across the fill area. The thickest fills soils were encountered on the northwest side of the building pad. Undocumented fill shall be completely removed prior to the placement of structural elements. It is anticipated that the majority of these fill soils will be removed during site grading as cuts along the north side of the building pad may range from 6 to 14 feet.

  Undocumented fill soils will require evaluation during construction to determine reuse as structural fill.

- Based on index test results of fill materials encountered in Test Pit TP-2, the clayey sand with gravel and cobbles **(SC)** encountered from 3 to 8 feet bgs, may experience volumetric changes (shrink/swell) due to changes in moisture content. It is recommended that these soils are ***not*** used as structural fill within 3 feet of the proposed bottom of foundation elevation for structures and 2 feet below proposed flat-work or asphalt concrete pavement. Recommendations for site grading and filling are included as Section 8.4 (Grading and Filling).

- Due to inaccessibility, field exploration for the proposed Diversion Feature #2 in-ground pump station was not completed. It is anticipated that soils will likely consist predominately of medium dense coarse-granular materials with cobbles and/or boulders.

- Based on the site geology, an argillic (potentially expansive clay) soils layer may be encountered during construction. It is anticipated that argillic soils, if encountered, will be relatively thin. These soils, if encountered, should be removed and disposed of in an approved location or used in landscape areas. Soils should be reviewed by the project geotechnical engineer during construction.

8.2 General Information

The recommendations provided herein, and particularly under **Site Clearing** (Section 8.3)**, Grading and Filling** (Section 8.4)**,** and **Construction Observation and Testing** (Section 9.0-Limitations) 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 proposed structure(s) 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.

The following definitions and recommendations shall apply for this project:

- Structural areas referred to in this report include all areas that will be used for the support of foundations, concrete slabs, retaining walls, flat work, and asphalt pavements;

- All compaction requirements presented in this report are relative to ASTM D1557[10];

- Unless otherwise stated in this report, all related construction should be in general accordance with the Standard Specifications for Public Works Construction (SSPWC), dated 2016.

- Fine-grained soil is defined as a soil with more than 40 percent by weight passing the number 200 sieve and a plasticity index less than 15.

- Clay soil is defined as a soil with more than 20 percent by weight passing the number 200 sieve and a plasticity index more than or equal to 15.

- Granular soil is defined as a soil not meeting the requirement for a fine-grained or clay soil and having a particle size of 4-inches or less.

- Subgrade is defined as the elevation directly below the aggregate base layer for both concrete slabs-on-grade and pavements.

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/backhoe at the approximate locations shown on the site plan. Test pits were backfilled upon completion of the field portion of our study. Backfill placed during this current exploration was compacted to the extent possible with the equipment on hand. It should be noted that 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 compacted 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.

8.3 Site Clearing

Surface vegetation and organic soils should be stripped and disposed of outside the construction limits or stockpiled for use in non-structural areas. Stripping depths of 6 to 10 inches are anticipated over a majority of the project areas. In areas where established brush and shrubs are present, grubbing depths of up to 12 inches may be required to remove the concentrated root zone.

Stripped topsoil (less any debris) may be stockpiled and reused for landscape purposes; however, this material ***<u>should not</u>*** be incorporated into structural fill.

Depending on the final site layout, deeper areas of localized grubbing to remove tree root balls may be required near Diversion Features #1 and #2, as large grouping of Aspens and Willows are present near the existing creek bank.

[10] 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 D1557 laboratory test procedure. Optimum moisture content is the corresponding moisture content of the same soil at it maximum dry density.

The entire root ball should be removed as part of any tree removal. Large roots (greater than 2 inches in diameter) radiating from the tree root ball area, located within one foot of the final subgrade should be completely removed. Excavations resulting from removal operations should be cleaned of all loose material and widened as necessary to permit access to compaction equipment. Resulting excavations should be backfilled with densified structural fill placed in accordance with Section 8.4 (Grading and Filling) of this report.

8.4 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 the underlying sub-soils.

Structural fill free of debris, vegetation, and organics shall meet the requirements given in Table 3 (Guideline Specifications for Structural Fill).

Table 3- Guideline Specifications for Structural Fill

Sieve Size	Percent by Weight Passing
6 Inch	100
¾ Inch	70 – 100
No. 40	15 – 60
No. 200	5 – 30
Maximum Liquid Limit	**Maximum Plastic Index**
35	12

In general soils encountered, except for argillic zones and fills soils classified as clayey sand **(SC),** below anticipated stripped and grubbed zones, free of debris or other deleterious materials appear to meet the requirements for structural fill, provided particles greater than 6 inches in diameter (where encountered) are removed. Screening may be required to remove particles greater than 6 inches nominal diameter.

Soils used for structural fill shall be uniformly moisture conditioned within three percent of optimum moisture content, placed in layers of 8 inches or less in loose thickness, and densified to at least 90 percent relative compaction. Areas to receive structural fill should be thoroughly cleaned of loose material and proof-rolled to uniform stability. The resulting prepared surface should be firm and non-yielding.

Thicker structural fill lifts, up to 12-inches, could be used if the contractor can demonstrate achieving required density. 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 lifts.

8.4.1 Rock Fill

It is anticipated that a portion of the onsite soils encountered at the Water Treatment Plant may meet the requirements for a rock fill. 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.**

If the use of rock fill is anticipated, the following construction recommendations shall be followed during the placement:

- 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 elevation, shall consist of structural fill containing no particles greater than 6-inches in diameter.

- A moisture-density relationship (ASTM D1557) shall be determined on the portion of the material passing the ¾-inch sieve. This data shall be used in the documentation of the in-place moisture content of the fill and subgrade soil as it relates to optimum.

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

- Rock fill should be placed in such a manner such that nesting of the particles does not occur. In other words, the voids between the rock particles should be 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.

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 cutting back 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.

8.4.2 Undocumented Fill Water Treatment Plant Site

Undocumented fill is classified as fill not monitored or tested by a licensed construction materials testing consultant or firm. Fill was placed at the approximate locations shown on Figure 5 (Approximate Limits of Undocumented Fill) during previous site grading. Undocumented fill thicknesses may range from 2 to 8 feet.

It should be noted that undocumented fill across the site may not be uniform in both thicknesses and classification. Our subsurface exploration included one test pit within the undocumented fill soils. Further evaluation during construction will be required to determine if undocumented fill soils meet the requirements of structural fill or rock fill. *Undocumented fill should be completely removed and replaced with properly densified structural fill prior to placement of structural elements. The removal should extend a minimum lateral distance of 3 feet or greater beyond the outside edge of the structural element and depends on the thickness of the undocumented adjacent to the structural element.* Structural fill placement should be completed in accordance with the recommendations presented in Sections 8.4 (Grading and Filling) of this report.

It is anticipated that the existing silty sand with gravel **(SM)** fill soils encountered in Test Pit TP-2 to a depth of 3 feet will meet the requirements for a structural fill[11], provided oversized materials are removed. However, fill material consisting of clayey sand with gravel and cobbles **(SC)** does not meet the requirements for a structural fill. Clayey sand with gravel and cobbles **(SC)** may be placed in lower lifts of the proposed fill provided a vertical and horizontal separation of at least 3 feet is maintained for foundations and at least 2 feet for flat work and pavements. Select grading may be required during construction to prevent mixing of potentially expansive material similar to those encountered in Test Pit TP-2 from a depth of 3 to 8 feet with non-expansive structural fill.

When undocumented fill is present below structural elements the risk of settlement and differential settlement are increased due to potential variability of densification. Slabs are especially susceptible to damage associated with differential settlement of undocumented fills. Due to the increased risk for settlement, it is recommended that a representative from CME be present to completed **full-time** observations and density testing during removal and placement.

8.4.3 Reuse of Onsite Materials

It is expected that a significant amount of the onsite materials can be stockpiled for reuse in non-structural landscape areas, as structural fill (provided they meet the requirements of Table 3-Guideline Specifications for Structural Fill), rip rap for erosion control, and for rock lined drainage features.

1) **Non-Structural Fill:** Stripped topsoil and grubbed material should be carefully processed to remove oversized material and stockpiled onsite for future use in non-structural landscape areas to promote revegetation of disturbed areas. Care should be taken not to mix topsoil with the onsite granular fill material.

2) **Structural Fill:** Granular soils similar to those encountered during our subsurface exploration, free of deleterious and oversized materials, meeting the requirements for structural fill, should be stockpiled onsite. In general it is expected that a majority of the site soils will meet the requirements for structural fill.

3) **Erosion Control:** If excavated material contains cobble or boulder sized particles, not meeting the minimum requirements for structural fill, this material could be screened to remove finer particles, and stockpiled onsite for future use in erosion control areas[12] designated by the project Civil Engineer or for landscape enhancement. Larger boulders may require additional splitting to accommodate size requirements for the project design.

Stock pile areas should be protected from erosion and runoff. Temporary erosion control measures should be implemented during project construction.

8.5 Permanent Slope Gradients, 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 *en masse* downward movement of soil or rock. Stability of cut and fill slopes depends upon shear strength, unit weight, moisture content, and slope angle.

[11] It should be noted that fill soils may vary from those encountered during the subsurface exploration.
[12] Rock used for erosion control purposes should meet the requirements of ASTM D4992-07 (Standards for Evaluations of Rock to be used for Erosion Control.

Cut and fill slopes with gradients up to 2H:1V (horizontal to vertical) are acceptable for this project. However, to reduce erosion potential, it is recommended that cut and fill slopes, where possible, are designed with gradients of 3H:1V or less. Steeper fill slopes, if required, may be constructed using reinforced earth techniques. The project geotechnical engineer should be consulted for site specific recommendations.

Slope vegetation is an excellent and common means of erosion control. In general, 70 percent groundcover is recommended for soil erosion prevention. If vegetation is the proposed means of stabilization, a licensed professional should be consulted to determine the appropriate type of Tahoe Basin approved plant species are compatible and can be established in the onsite granular material.

Performance of all slopes will be primarily affected by surface runoff. Care must be taken that drainage is not directed to flow over slope faces. Slope faces should be protected against erosion resulting from direct rain impact and melting snow. Brow ditches should be constructed at the top of the slope to collect potential flows and direct them around the slope face.

Dust control during grading and site preparation will be the responsibility of the contractor.

8.6 Trenching and Confined Excavations

All excavations regardless of depth should be evaluated for stability including scaling trench sidewalls to remove loose material prior to occupation by construction personnel. Shoring or sloping of trench walls may be required to protect construction personnel and provide temporary stability. In areas where temporary confined excavations may be unstable, trench boxes may be used to provide safe ingress and egress for construction personnel.

Excavations should comply with current OSHA safety requirements (Federal Register 29 CFR, Part 1926). Soils or bedrock are classified as Type A, B or C, which requires different temporary excavation, cut slope gradients (Table 4-Maximum Allowable Temporary Slopes).

Table 4 - Maximum Allowable Temporary Slopes

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

NOTES:
1. Angles expressed in degrees from the horizontal and have been rounded off.
2. Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.
3. For detailed description of the soil types outlined above visit the US Department of Labor Safety and Health Topics website at: https://www.osha.gov/SLTC/trenchingexcavation/construction.html

Soils encountered during our field exploration consisted predominately of granular clayey sand and clayey sand with gravel, cobbles and boulders. Therefore, it is our opinion that excavations will need to comply with current OSHA safety requirements for a *Type C* soil (Federal Register 29 CFR, Part 1926) and should be adjusted as needed for compliance during construction.

Heavy loads near trench excavations should be avoided as they may cause bank stability issues. Bank stability will remain the responsibility of the contractor present at the site.

Trench excavations should be protected from surface water/runoff. If warranted, dewatering of pipe trench excavations can be accomplished by use of a temporary dewatering system.

If subsurface water conditions differ from those encountered during our subsurface exploration, the geotechnical engineer should be notified immediately to determine if alternative dewatering recommendations are warranted.

8.6.1 Excavations for Underground Utilities or Vaults

Excavations for underground utilities or shallow foundations may be completed using conventional excavation equipment such as a trackhoe.

Test Pit TP-1 (located near the in-ground pump for Diversion Feature #1) was excavated to a depth of 18 ½ feet using the track mounted excavator. Excavation refusal was encountered in Test Pit TP-3 at a depth of 13 feet (using the track mounted excavator) and at 9 ½ feet in Test Pit TP-4 (using the rubber tired backhoe). Excavations for utilities or vaults deeper than 13 feet bgs at Diversion Feature #2 site may require the use of larger equipment such as a Caterpillar D8 with single shank to loosen cemented soils, large trackhoe, or hoe-ram. The size/type of equipment for Diversion Feature #2 will need to be assessed during construction and will be dependent on the type of soils present.

Trench preparation shall consist of removing all loose soil particles from the bottom of the trench created during excavation to expose a firm non-yielding soils surface. Utility vaults should be bottomed on properly densified structural fill or the native medium dense to dense granular soils. If boulders or cobbles are present at the bottom of excavation, removal may be required to maintain trench grade. Resulting voids should be widened to allow access for construction equipment and backfilled with densified structural fill or lean concrete.

Significant undulations (+3 inches) in the bottom of excavation caused by existing cobbles and boulders can be remediated by overexcavating to a depth of 1 foot below the bottom of vault elevation and re-placing with 1 foot of densified structural fill, or lean concrete slurry prior to placement of the subsurface vault.

8.6.2 Trench Bedding and Backfill

Any material used as pipe bedding or trench backfill should meet the minimum requirements of the TMWA Design and Construction Standards (TMWADCS) and the SPPWC (2016).

Pipe zone bedding is the trench backfill located immediately above and below the pipe. It is recommended that pipe zone bedding be placed in (loose) lifts not exceeding 4-inches thick. Pipe zone bedding should be densified to a minimum of 90 percent relative compaction. Compaction equipment should be carefully selected to avoid damage to the pipeline.

Intermediate trench backfill[13] should be placed in (loose) lifts not exceeding 8-inches thick, and densified to at least 90 percent relative compaction.

[13] Material located directly above the pipe zone bedding extending to the proposed finished subgrade or ground surface.

8.6.3 Pipe Zone Bedding

Pipe zone bedding should consist of Class A backfill (Section 200.03.02, SPPWC), or other TMWADCS approved alternate. Class A backfill can be used in trenches which are bottomed above the existing groundwater elevation (assumed to be the predominate trench condition encountered). If groundwater conditions differ from those encountered during the subsurface exploration, alternate recommendations for pipe zone backfill can be provided.

8.6.4 Intermediate Trench Backfill

Trench backfill may consist of native granular soils meeting the following requirements:

- Free of debris, organic matter or other deleterious materials;
- A maximum particle size less than 6 inches; and
- A sand equivalent value of not less than 25.

8.7 Foundation Recommendations

8.7.1 Foundation Grade Soils Preparation

Foundation grade soils preparation depends on the final location of the proposed structure, structure type, foundation grade soils conditions, and anticipated structural loads. Field density testing at foundation grade shall be completed for foundations bottomed in densified structural fill or native granular materials meeting the specifications for a structural fill.

The upper 12 inches of the foundation grade soils shall be scarified and moisture conditioned, as required, and densified to at least 90 percent relative compaction. Large boulders or cobbles protruding into the foundation excavation should be removed and the resulting void backfilled with densified structural fill or lean concrete.

If clayey sands or any material meeting the specifications of a clay soil are encountered at foundation grade, these soils shall be removed and replaced with structural fill. Removal shall be at least 2 feet below foundation grade and extent laterally from the edge of the foundation at least 3 feet.

8.7.2 Foundation Design

Foundation loads were not available. However, structural loads are anticipated to be light to moderate for the proposed structures. When structural loads and foundation depths are known, foundation design recommendations can be finalized.

Provided that the foundation soils preparation has been performed in accordance with the recommendation given in Section 8.7.1 (Foundation Grade Soils Preparation), foundation design parameters presented in Table 5 (Preliminary Foundation Design Parameters) can be utilized for the design of individual column footing and continuous wall footings.

Table 5 – Preliminary Foundation Design Parameters	
Allowable Bearing Pressures (psf)[1,2]:	
Footings bottomed at least 2 feet[3] below the proposed finished grade on properly compacted structural fill or on a suitable native bearing strata.	3,000
Allowable Friction Coefficient:	
Between foundation bottom and supporting soil consisting of properly compacted structural fill or native granular soils	0.40
Allowable Passive Soil Pressure (psf)[1]	
Backfill soils consisting of properly compacted structural fill	350[4]

(1) (psf)-Pounds per square foot

(2) The allowable bearing pressure may be increased by one-third for total loading conditions including wind and seismic forces (2012 IBC). The allowable bearing pressure is a net value; therefore, the weight of the foundation which extends below grade and backfill may be neglected when computing dead loads. The allowable bearing pressure includes a FOS of 3.0 against bearing failure.

(3) Allowable bearing pressures may be increased for foundations bottomed at greater depths. Once the final loads and footing elevations have been determined, the project geotechnical engineer should be contacted to evaluate the net allowable bearing pressure.

(4) The upper one-foot of the soils profile should be neglected when designing for passive pressure, unless confined by a concrete slab or pavement. Design values are based on footings backfilled with properly compacted structural fill.

Lateral loads (such as wind or seismic) may be resisted by passive soil pressure and friction at the bottom of the footing. A design value for passive soil pressure of 350 psf per foot of depth and a friction factor of 0.40 may be utilized for sliding resistance at the base of the footing. The friction coefficient of 0.40 assumes that structural elements will be bottomed on at least 1 foot of properly compacted structural fill on native granular material.

Overturning moments and uplift loading can be resisted by the weight of the foundation, weight of the structure, and any soil overlying the foundation. A unit weight of 120 pounds per cubic foot may be assumed for backfill soils consisting of properly densified structural fill.

It is recommended that footing excavations be observed by the project soils engineer prior to placing concrete reinforcing steel to confirm the subsurface conditions are similar to those described in this report.

8.7.3 Static Settlement

An elastic settlement response is expected for foundations bottomed on properly compacted structural fill or medium dense native granular material. The majority of the settlement is expected to occur rapidly, generally during the construction timeframe.

Once loading is determined, settlement can be estimated. However, based on the loading assumptions of this report and the anticipated foundation grade material, settlement on the order of ¾-inch or less is anticipated.

Differential settlement for foundations with similar loads is anticipated to be about ½ of the total settlement provided the foundations are all bottomed on similar material (e.g. all on suitable native material or properly compacted structural fill).

8.8 Retaining Walls

8.8.1 Static

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. The lateral earth pressure is strongly dependent on the lateral deformations which occur in the soil.

A restrained retaining wall will experience higher lateral earth pressures than a retaining wall that is free to move (cantilever conditions). It is assumed that the pump station vaults or other proposed retaining structures at each of the sites will be designed for active soil pressure conditions (K_a[14]). Lateral earth pressure values are presented in Table 6 (Lateral Earth Pressure Values)

Table 6 –Lateral Earth Pressure Values

STATIC LATERAL EARTH PRESSURE		
Earth Pressure Condition	**Earth Pressure Coefficient**	**Equivalent Fluid Density (psf)** (1,2)
Active (P_a)	0.29 (K_a)	36
Passive (P_p)	-	350(3)

(1) Pounds per square foot per foot of depth
(2) Lateral pressures for level backfill calculated using an average of the Rankine and Coulomb Equations for active/passive earth pressure. Assuming maximum unit weight of 125 pcf and a friction angle of at least 34 degrees.
(3) Assumes a factor of safety of 1.5.
(4) The lateral pressures presented in Table 6 assume 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 shall be constructed as an integral part of the retaining wall.

Subterranean structures and short retaining walls, including foundations, should be designed to resist the lateral earth pressure exerted by the retained soil plus any additional lateral force that will be applied to the wall due to surface loads placed at or near the wall.

Table 6 (Lateral Earth Pressures) provides lateral earth pressures based on the assumption that granular soils are used as backfill. Retained soils should consist of non-expansive granular soils with a minimum friction angle of 34 degrees and a maximum unit weight of 125 pounds per cubic foot.

[14] The active earth pressure coefficient assumes a wall deflection equal to 0.5 percent of the total wall height (e.g. free to rotate with the ability to deflect at the top (wall movement greater than 0.001H for cohesion less soils and greater than 0.01H for cohesive soils).

8.8.2 Seismic

The 2012 IBC requires retaining walls over 6 feet in height to be designed for seismic pressures. 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 seismic ground acceleration component. This component is derived from the design spectral response acceleration parameter at short period **(S_{Ds})**, as described in this section.

- **K_{ae}:** Seismic active earth pressure coefficient.

- **P_{AE}:** Dynamic lateral earth pressure force: $P_{AE}=0.5\gamma H^2K_{AE}$, 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 a function of the short period design spectral response acceleration (**S_{DS}**). 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 earthquake ground motion assuming some outward movement of the retaining wall is acceptable during an earthquake event (IBC, 2012).

- The horizontal ground acceleration for a restrained retaining wall is equal to the design earthquake ground motion with no reduction.

The design earthquake ground motion can be approximated by dividing the short period design spectral response acceleration (**S_{DS}**) by 2.5 (FEMA/NEHRP, 2003). Based on the site grading, it is assumed that retaining walls (if used) will be constructed to retain the proposed cut/fill slopes at the site. Therefore, future retaining walls are assumed to be yielding, a horizontal ground acceleration of 0.28g[15] was used to determine the seismic active earth pressure coefficient. Table 7 (Seismically Induced Lateral Earth Pressure Values) provides seismically induced earth pressure values.

[15] Kh= (0.5*(SDS/2.5))

Table 7 – 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)** ($P_{ae}=\Delta P_{ae}+P_a$)	
	Slope	K_{ae} [(2,3)]	$P_{ae} = (\gamma_{soil} * K_{ae})$ [(3,4)]	Seismic (ΔP_{ae})	Static (P_a)
Assumes lateral wall displacement-yielding conditions	Level	0.48	60	24	36

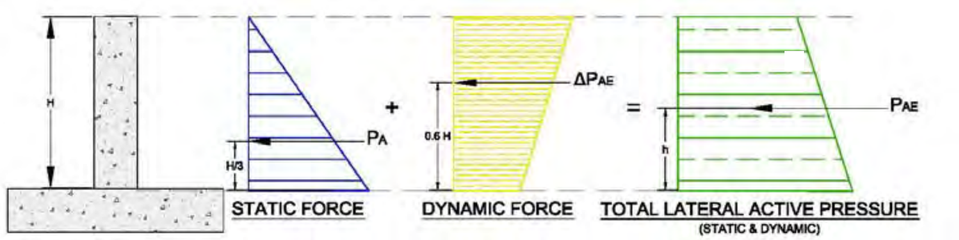
(1) Pounds per square foot per foot of depth. P_{ae} is the **total wall pressure** for pseudo static loading and includes both the static and seismic lateral earth pressure components. Assumes a Ø of 34^0 and γ of 125 pcf. Assumes no hydrostatic forces and no surcharge loading.

(2) Based on short period design spectral response acceleration (S_{DS}) of 1.4 and assuming yielding conditions. For walls that can with stand movement to mobilize active earth pressure conditions during the design earthquake event, ½ the design earthquake ground motion is applicable.

(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 6 foot tall wall with a P_a=35 psf/ft and a ΔP_{ae}=48psf/ft would have a resultant force (P_{ae}=61 psf/ft) acting at a height (h) equal to about 2.9 feet.

Existing fill soils or native granular soils meeting the requirements for an imported structural fill may be used as backfill provided they are screened to removed oversized material (i.e. >4 inches nominal diameter). The backfill shall extend laterally behind the retaining wall at least the height of the retaining wall.

Backfill placed behind the retaining wall should be compacted to at least 90 percent. Over-compaction should be avoided as it will result in increased 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 a 1H:1V slope away from the base of the wall, whichever is greater.

8.8.3 *Drainage*

The lateral pressures presented in Table 7 (Pseudo State Lateral Earth Pressures) assume back-of-wall drainage is provided to prevent the build-up of hydrostatic pressures and finished grades are planned such that site drainage is directed runoff away from retaining walls. To minimize hydrostatic pressures, retaining wall drainage shall be designed and constructed as an integral part of the retaining wall.

Design options for retaining wall drainage are presented below:

- If drainage can be obtained through the front of the retaining wall (e.g. exterior retaining structures), 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, as presented in this report.

- A back-of-wall drain shall be installed at the base of the foundation behind the retaining wall. The back-of-wall drain shall consist of a 4-inch diameter (minimum) perforated drain pipe bedded in drain rock. The drain pipe shall slope at least 1 percent and daylight away from the retaining wall or other sensitive structures. The discharge location shall be protected from clogging by appropriate means.

- Drain rock shall extend upward behind the retaining wall to about 12-inches below the proposed finished grade, and have a thickness of at least 12 inches. Drain rock shall meet the specifications for Class D Backfill (SSPWC). To prevent fines migration into the drainage layer, drain rock shall be encapsulated (wrapped) with a non-woven geotextile such as Mirafi 180N or equivalent meeting the minimum material properties presented in Table 8 (Drainage Geotextile Minimum Strength and Hydraulic Properties).

TABLE 8 – 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)	$\geq 0.2\ sec^{-1}$
AOS (ASTM D4751)	≤ 0.25 mm

1) The strength properties presented above, are based on Class 1 survivability rating (AASHTO M288)

8.9 Concrete Slabs

All concrete slabs should be directly underlain by aggregate base material. Type 2 aggregate base is the preferred alternate, although other materials may be acceptable. The thickness of base material should be at least 6 inches. Aggregate base courses should be densified to at least 95 percent relative compaction.

Subgrade soils shall be prepared in accordance with recommendations presented in the Grading and Filling section of this report (Section 8.4-Grading and Filling). Prior to construction, the upper six inches of the slab subgrade soils should be scarified to a minimum depth of 6 inches, uniformly moisture conditioned to within 3 percent of optimum moisture content and densified to at least 90 percent relative compaction. The subgrade should be protected against drying until the concrete slab is placed.

Type II cement is recommended for project design. Due to the potential exposure to freeze/thaw conditions the project design engineer should consider air entrainment for the project mix design.

The design engineer should determine the slab thickness and structural reinforcing requirements. Placement and curing should be performed in accordance with procedures outlined by the American Concrete Institute (ACI). Special considerations should be given to concrete placed and cured during hot or cold weather conditions. Proper control joints and reinforcing should be provided to minimize any damage resulting from shrinkage.

8.10 Corrosion Potential

Corrosion testing was completed on two samples from Test Pit TP-2. Silver State Analytical Laboratories completed testing for soluble sulfate, resistivity, and pH. These tests were completed to determine the potential corrosiveness of the soils to concrete and metallic underground utilities. A brief summary of the results is presented below.

- **Soluble Sulfates (ASTM D1580C):** Soluble sulfate were generally not detected (<0.02%). This indicates that the native onsite soils have a negligible sulfate exposure to concrete.

- **pH (EPA 9045D):** The paste pH test results are ranged from 6.84 to 7.05 indicating a moderate potential of corrosion for soils in direct contact with ferrous metals (Baboian et. al, 2006).

- **Resistivity (ASTM G57):** Resistivity results ranged from 5,980 to 7,220 ohms.cm indicating that the site soils are have a moderate potential for corrosion for ferrous metal in direct contact with theses soils.

8.11 Site Drainage Considerations

Final grades should be planned such that surface drainage is constructed and maintained to fall away from the proposed foundations and slabs. A permanent finished slope grade of at least 5 percent for a minimum distance of 10 feet away from proposed pump stations, water tanks, transformer pads, and the water treatment plant structure is recommended. The slope gradient can be reduced to 2 percent for impervious surfaces, such as concrete slabs-on-grade and pavement.

9.0 LIMITATIONS

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 Plates A-1 in Appendix A 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 the recommendations contained herein. 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[16]. 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[17].

This report was prepared by CME for Stantec Consulting. 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. CME accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

The recommendations presented in this report are based on the assumption that the owner/project manager provide adequate field testing and construction review during all phases of construction. These tests and observations should include, but not be limited to:

- Earthwork observation and materials testing;
- QA/QC during placement Portland Cement Concrete or Asphaltic Concrete Pavement;

[16] If the geotechnical engineer is not accorded the privilege of making this recommended review, they can assume no responsibility for misinterpretation or misapplication of the recommendations contained herein or their validity in the event changes have been made to the original design concept.

[17] All structures are subjected to deterioration from environmental and manmade exposures. As a result, all structures require regular and frequent monitoring and maintenance to prevent damage and deterioration. Such monitoring and maintenance is the sole responsibility of the Owner. CME Inc. shall have no responsibility for such issues or resulting damages.

REFERENCES

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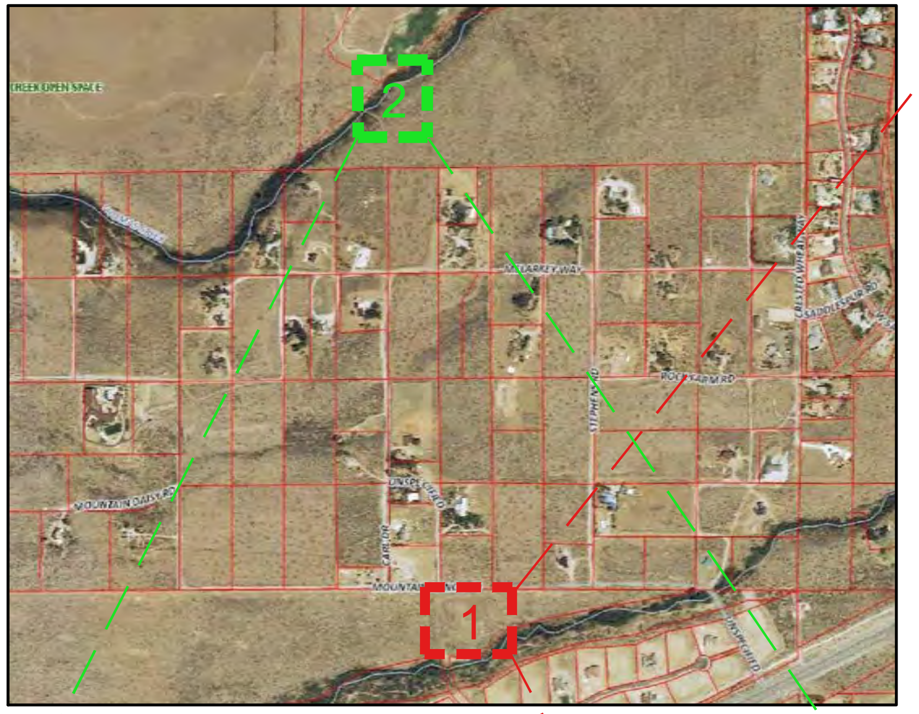
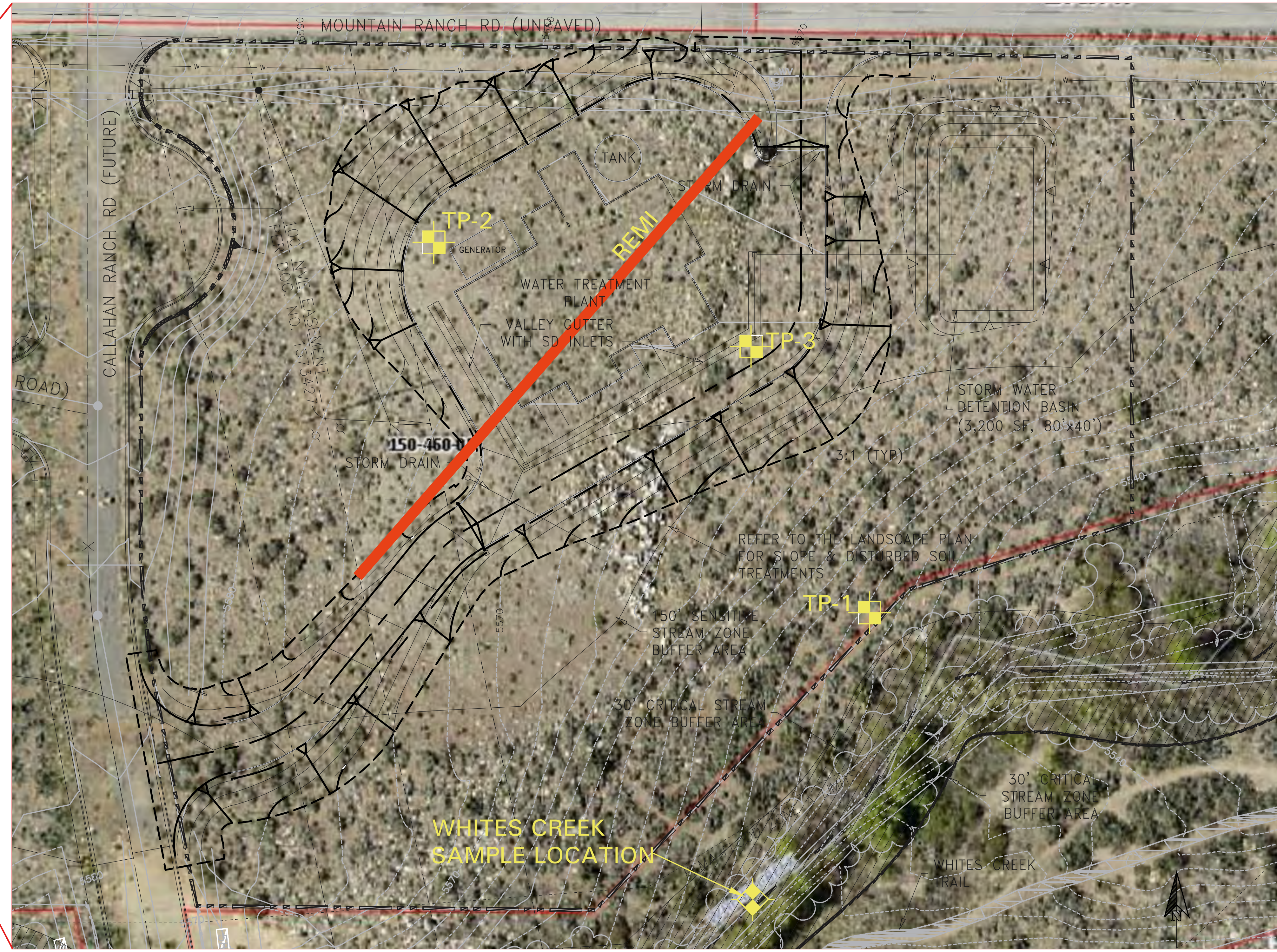
APPENDIX A



SITE VICINITY MAP
N.T.S

FIELD EXPLORATION LOCATION MAP 2
SHOWING TEST PIT TP-4

200 100 0 100 200



SCALE 1"~60 '
60 30 0 30 60

FIELD EXPLORATION LOCATION MAP 1
SHOWING TEST PIT LOCATION TP-1, TP-2, & TP-3

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6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

STANTEC
MT. ROSE WTP
FIELD EXPLORATION LOCATION MAP
WASHOE COUNTY, NEVADA

PROJECT NO.: 1875 DATE: 6/1/2016

LEGEND

TP-2	APPROXIMATE TEST PIT LOCATION
	APPROXIMATE CREEK BOTTOM SAMPLE LOCATION
	APPROXIMATE REMI LINE LOCATION

REFERENCE: BASE MAP FUTURE SITE GRADING PLAN BY STANTEC CONSULTING, IMAGE FROM WASHOE COUNTY GIS, ACCESSED JUNE 2016

PLATE
A-1

V:\Active\1875\cad\2-Grading_Plan.dwg

LOG OF TEST PIT NO. TP-1

PROJECT MT. ROSE WTP **EQUIPMENT TYPE** CASE 9020
CLIENT STANTEC
LOCATION N 39°23.349', W 119°48.822' (NAD27 CO NUS, CLARKE 1866) GARMIN ETREX30 GPS
PROJECT NO. 1875 **DATE** 05/24/16 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)**

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SM			B	1A		SL. MOIST	0'-2': SILTY SAND, mostly fine to medium sand, few subangular gravels, non-plastic,brown to strong brown							
4	SM			B	1B		SL. MOIST	2'-7': SILTY SAND WITH GRAVEL AND COBBLES, some fine to medium sand, some fine to coarse subrounded gravels and cobbles, few subrounded boulders up to 36" nominal diameter, strong brown	10.1	NV	NP			3.8	A, G
8	GM						MOIST	7'-10': SILTY GRAVEL WITH SAND, some fine to coarse subrounded gravels, cobbles and boulders up to 36" nominal diameter, little fine to medium sand, non-plastic, strong brown							
12	GM						MOIST	10'-13': INDURATED-SILTY GRAVEL WITH SAND, possibly older alluvial deposit, some fine to coarse subrounded grave, cobbles and trace boulders, few subangular cobbles, little sand, low to non-plastic, yellowish brown							
16	GP-GM							13'-17': POORLY GRADED GRAVEL WITH SILT, mostly fine to couarse subrounded to subangular gravel, few fine to medium sands, non-plastic, yellow brown							
	SP-SM						VERY MOIST	17'-18': POOLY GRADED SAND WITH SILT AND GRAVEL, mostly fine to coarse sand, little subrounded gravel, non-plastic, brown Note: Minor seepage visible in this layer at SE corner of test pit.							
	SC						VERY MOIST	18-18½': CLAYEY SAND, some fine to medium sand, moderately plastic, greyish brown							
20								TERMINATED AT 18½-FEET, NO FREE WATER ENCOUNTERED							
24															

GROUNDWATER

DEPTH	HOUR	DATE
N.E.		

SAMPLE TYPE
B - Bulk Sample

LABORATORY TESTS
SG - Bulk Specific Gravity
A - Atterberg Limits
G - Grain Size
C - Consolidation
MD - Moisture/Density
DS - Direct Shear

PLATE NO.: A-2a

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-2

PROJECT MT. ROSE WTP **EQUIPMENT TYPE** CASE 9020
CLIENT STANTEC
LOCATION N 39°23.356', W 119°48.865' (NAD27 CO NUS, CLARKE 1866) GARMIN ETREX30 GPS
PROJECT NO. 1875 **DATE** 05/24/16 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)**

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SM			B	2A		SL. MOIST	0'-3': FILL-SILTY SAND WITH GRAVEL, mostly fine to medium sand, non-plastic, little rounded gravel, trace boulders up to 18" nominal diameter, , non-plastic, brown							
4	SC			B	2B		MOIST	3'-8': FILL-CLAYEY SAND WITH GRAVEL AND COBBLES, mostly fine to medium sand, little fine to coarse rounded gravel, moderate plasticity, strong brown	26.8	45	25			11.9	A, G
8	SP-SM			B	2C		MOIST	8'-10': POORLY GRADED SAND WITH SILT AND GRAVEL, mostly fine to coarse sand, some fine to coarse gravel and cobbles, non-plastic, brown							
12	SM						MOIST	10'-17': SILTY SAND WITH GRAVEL, mostly fine to medium sand, little subrounded gravel, few rounded cobbles, non-plastic, light brown							
16															
								TERMINATED AT 17 FEET, NO FREE WATER ENCOUNTERED							
20															
24															

GROUNDWATER

	DEPTH	HOUR	DATE
∇	N.E.		
▼			

SAMPLE TYPE
B - Bulk Sample

LABORATORY TESTS
SG - Bulk Specific Gravity
A - Atterberg Limits
G - Grain Size
C - Consolidation
MD - Moisture/Density
DS - Direct Shear

PLATE NO.: A-2b

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-3

PROJECT MT. ROSE WTP **EQUIPMENT TYPE** CASE 9020
CLIENT STANTEC
LOCATION N 39°23.349', W 119°48.831' (NAD27 CO NUS, CLARKE 1866) GARMIN ETREX30 GPS
PROJECT NO. 1875 **DATE** 05/24/16 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)**

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	GP-GC						SL. MOIST	0-1½': POORLY GRADED GRAVEL WITH COBBLES, BOULDERS, CLAY AND SAND, mostly fine to coarse subrounded gravel, cobbles, few boulders up to 48" nominal diameter, brown							
	SC			B	3A		SL. MOIST	1½'-6': CLAYEY SAND WITH GRAVEL, mostly fine to medium sand, little subangular to subrounded gravel, few cobbles, moderate plasticity, strong brown	19.6	34	16			9.1	A, G
4															
	SM						SL. MOIST	6'-12': SILTY SAND WITH GRAVEL, mostly fine to medium sand, some fine to coarse subangular and subrounded gravel, non-plastic, trace boulders up to 36" nominal diameter, brown to light brown							
8															
12	GP						MOIST	12'-13': POORLY GRADED GRAVEL, COBBLES AND BOULDERS WITH SAND, mostly fine to coarse subrounded gravel and cobbles, few nested boulders up to 48" nominal diameter, little fine to medium sand, brown							
								PRATICAL REFUSAL AT 13 FEET ON NESTED BOULDERS, NO FREE WATER ENCOUNTERED							
16															
20															
24															

GROUNDWATER

	DEPTH	HOUR	DATE
▽	N.E.		
▼			

SAMPLE TYPE

B - Bulk Sample

LABORATORY TESTS

SG - Bulk Specific Gravity
A - Atterberg Limits
G - Grain Size
C - Consolidation
MD - Moisture/Density
DS - Direct Shear

PLATE NO.: A-2c

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-4

PROJECT MT. ROSE WTP **EQUIPMENT TYPE** JOHN DEERE 310SG
CLIENT STANTEC
LOCATION N 39°23.846', W 119°49.047' (NAD27 CO NUS, CLARKE 1866) GARMIN ETREX30 GPS
PROJECT NO. 1875 **DATE** 05/24/16 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)**

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SC-SM			B	4A		SL. MOIST	0'-6': SILTY, CLAYEY SAND WITH GRAVEL, COBBLES AND BOULDERS, some fine to medium sand, little fine to coarse gravels, cobbles and boulders up to 36" nominal diameter, brown to reddish brown	22.8	24	7			10.8	A, G
4															
	SC-SM						SL. MOIST	6'-7': SILTY,CLAYEY SAND WITH GRAVEL AND COBBLES, some fine to medium sand, some fine to coarse subrounded gravel and cobbles up to 10" nominal diameter, low plasticity, reddish brown							
8	GC-GM						SL. MOIST	7'-9½': MODERATELY CEMENTED-SILTY, CLAYEY GRAVEL WITH SAND, mostly fine to coarse subrounded to subangular gravel and cobbles, few boulders up to 24" nominal diameter, little fine to medium sand, low plasticity, red brown to light brown							
								REFUSAL AT 9½ FEET, NO FREE WATER ENCOUNTERED							
12															
16															
20															
24															

GROUNDWATER

DEPTH	HOUR	DATE
N.E.		

SAMPLE TYPE

B - Bulk Sample

LABORATORY TESTS

SG - Bulk Specific Gravity
A - Atterberg Limits
G - Grain Size
C - Consolidation
MD - Moisture/Density
DS - Direct Shear

PLATE NO.: A-2d

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

STANTEC
MR. ROSE WTP
SOIL CLASSIFICATION CHART
WASHOE COUNTY, NEVADA

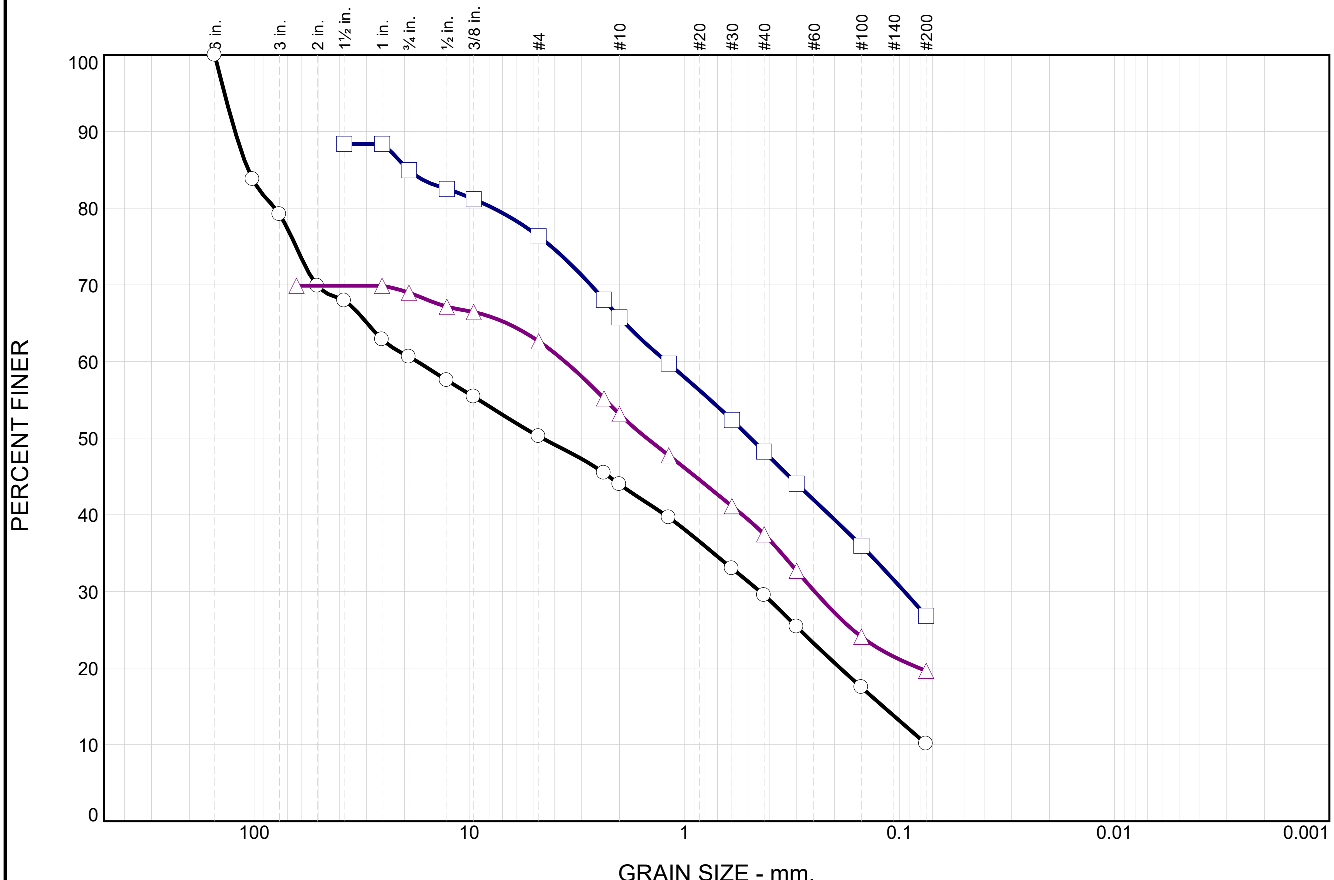
PROJECT NO.:1875 DATE: 06/16/16

PLATE
A-3

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX B

Particle Size Distribution Report



	% +3"	% Gravel Coarse	% Gravel Fine	% Sand Coarse	% Sand Medium	% Sand Fine	% Fines Silt	% Fines Clay
○	20.8	18.6	10.4	6.2	14.5	19.4	10.1	
□			8.7	10.5	17.6	21.4	26.8	
△			6.4	9.5	15.7	17.8	19.6	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	NV	NP	106.4652	17.4673	4.5809	0.4448	0.1190			
□	45	20	19.1300	1.2115	0.4916	0.0948				
△	34	18		3.5893	1.4939	0.2477				

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ SILTY SAND WITH GRAVEL AND COBBLES	06/03/16	SM	3.8
□ CLAYEY SAND WITH GRAVEL	06/02/16	SC	11.9
△ CLAYEY SAND WITH GRAVEL	06/03/16	SC	9.1

Project No. 1875 **Client:** STANTEC

Project: MT. ROSE WTP

○ **Source of Sample:** TP-1 **Depth:** 2 **Sample Number:** 1B

□ **Source of Sample:** TP-2 **Depth:** 3 **Sample Number:** 2B

△ **Source of Sample:** TP-3 **Depth:** 1.5 **Sample Number:** 3A

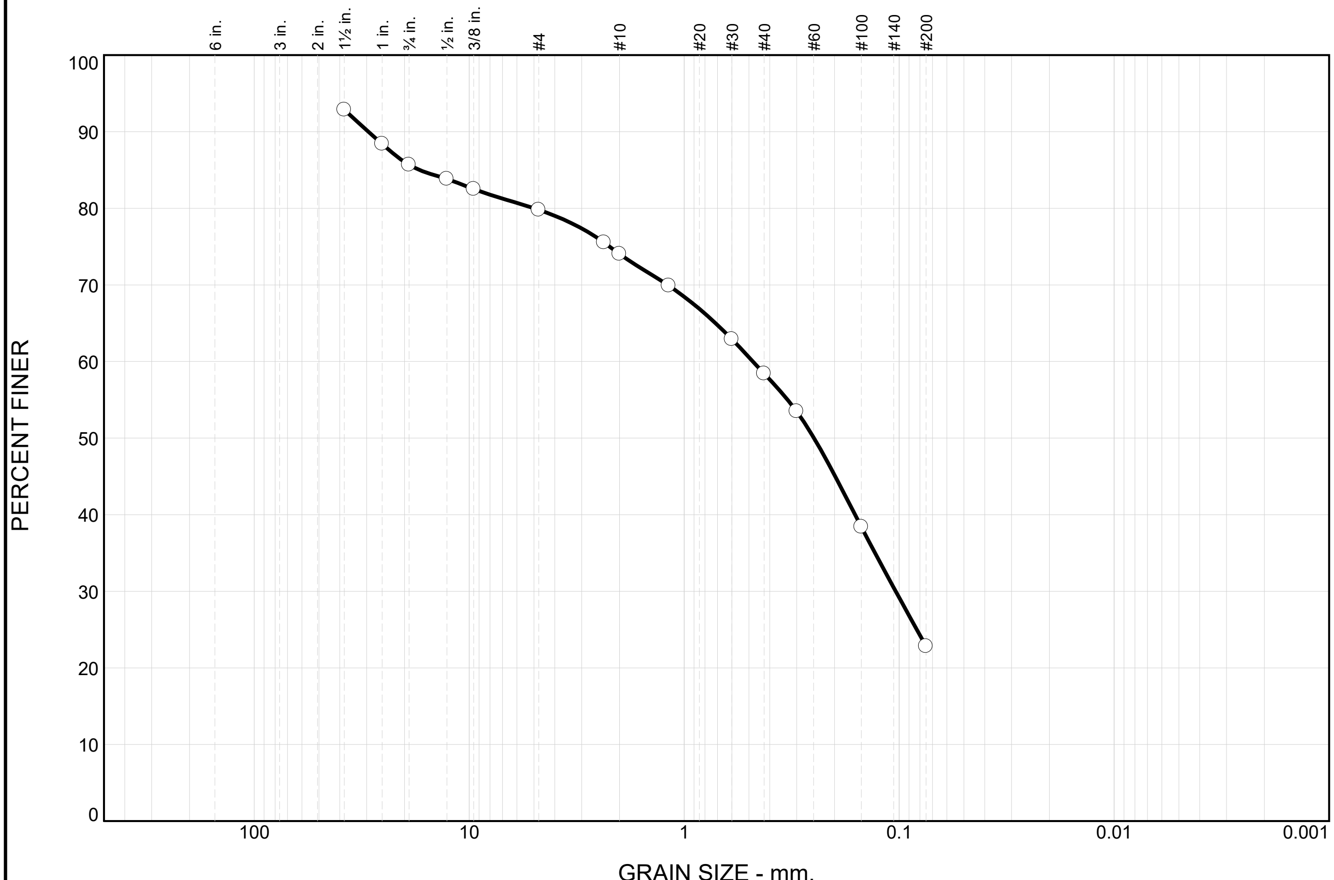
CME

Remarks:

Plate B-1A

Tested By: ○ TL/SB □ TL.DN,SB △ TL,DN **Checked By:** SV

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○			5.9	5.8	15.6	35.6	22.8	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	24	17	17.0086	0.4791	0.2490	0.1038				

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ SILTY CLAYEY SAND WITH GRAVEL	05/31/16	SC-SM	10.8

Project No. 1875 **Client:** STANTEC
Project: MT. ROSE WTP

○ **Source of Sample:** TP-4 **Depth:** 0 **Sample Number:** 4A

CME

Remarks:
○NOTE LARGE COBBLES AND BOULDERS NOT COLLECTED FOR PARTICAL SIZE ANALYSIS

Plate B-1B

Tested By: TL, DL **Checked By:** SV

SilverState
Analytical Laboratories

Sierra Environmental Monitoring

EnviroTech.

Revised Laboratory Report

Report ID: 148565

CME-Construction Materials Engineers, Inc		**Date:**	6/15/2016
Attn: Stella Montalvo		**Client:**	CON-160418
6980 Sierra Center Parkway, Suite 90		**Taken by:**	S. Montalvo
Reno, Nevada 89511		**PO #:**	1875

Analysis Report

Laboratory Accreditation Number: NV-00015

Laboratory Sample ID	**Customer Sample ID**	**Date Sampled**	**Time Sampled**	**Date Received**
S201606-0269	TP-2 2A1	5/24/2016		6/6/2016

Parameter	Method	Result	Units	Reporting Limit	Analyst	Date Analyzed	Data Flag
pH - Saturated Paste	SW-846 9045D	6.84	pH Units		Bergstrom	6/7/2016	
pH - Temperature	SW-846 9045D	22	°C		Bergstrom	6/7/2016	
Resistivity ASTM	ASTM G57	5980	ohm cm		Bergstrom	6/10/2016	
Sulfate ASTM 1580C	ASTM 1580C	<0.02	%	0.02	Bergstrom	6/9/2016	

Laboratory Accreditation Number: NV-00015

Laboratory Sample ID	**Customer Sample ID**	**Date Sampled**	**Time Sampled**	**Date Received**
S201606-0270	TP-2 2C	5/24/2016		6/6/2016

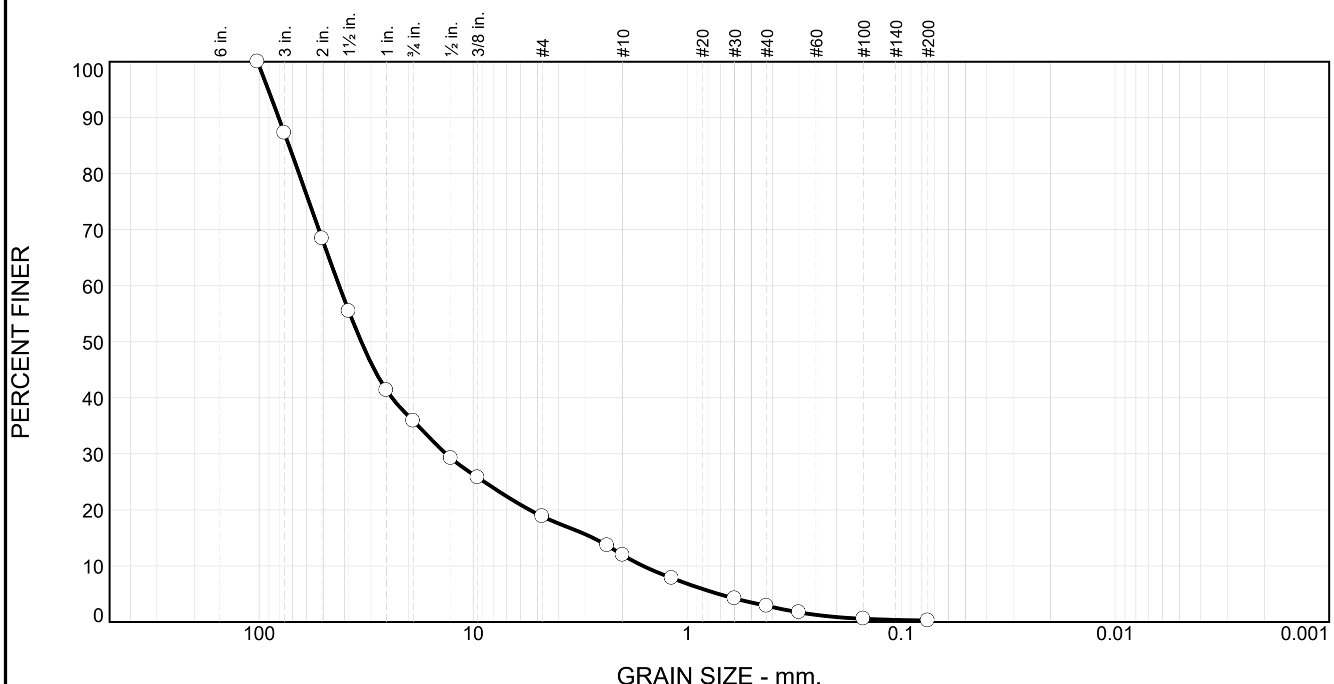
Parameter	Method	Result	Units	Reporting Limit	Analyst	Date Analyzed	Data Flag
pH - Saturated Paste	SW-846 9045D	7.05	pH Units		Bergstrom	6/7/2016	
pH - Temperature	SW-846 9045D	22	°C		Bergstrom	6/7/2016	
Resistivity ASTM	ASTM G57	7220	ohm cm		Bergstrom	6/10/2016	
Sulfate ASTM 1580C	ASTM 1580C	<0.02	%	0.02	Bergstrom	6/9/2016	

Data Flag Legend:

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX C

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
12.7	51.4	17.0	6.9	9.1	2.6	0.3	

Test Results (ASTM D 422 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
4	100.0		
3	87.3		
2	68.5		
1.5	55.5		
1	41.4		
.75	35.9		
.5	29.3		
.375	25.8		
#4	18.9		
#8	13.7		
#10	12.0		
#16	7.9		
#30	4.2		
#40	2.9		
#50	1.7		
#100	0.6		
#200	0.3		

* (no specification provided)

Material Description

POORLY GRADED GRAVEL WITH SAND

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GW/GP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 80.9391 D_{85}= 72.4581 D_{60}= 42.2532
D_{50}= 33.2809 D_{30}= 13.3695 D_{15}= 2.7441
D_{10}= 1.6068 C_u= 26.30 C_c= 2.63

Remarks

Date Received: 05/24/16 **Date Tested:** 05/24/16

Tested By: TL/SB

Checked By: SV

Title:

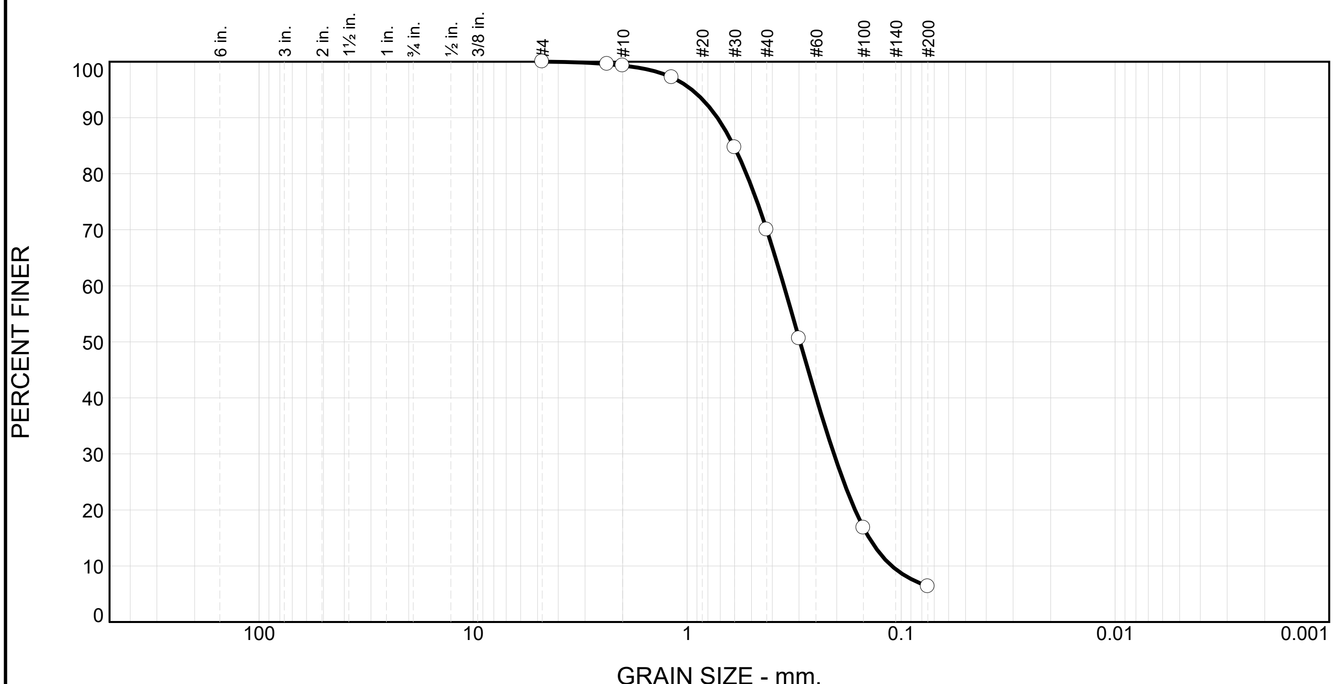
Location: WHITES CREEK- CREEK BED (COARSE SAMPLE)
Sample Number: SB-1
Date Sampled: 05/24/16

CME

Client: STANTEC
Project: MT. ROSE WTP

Project No: 1875 **Plate** C-1A

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.7	29.3	63.7	6.3	

Test Results (ASTM D 422 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#8	99.6		
#10	99.3		
#16	97.2		
#30	84.7		
#40	70.0		
#50	50.6		
#100	16.8		
#200	6.3		

* (no specification provided)

Material Description

POORLY GRADED SAND WITH SILT

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=**

Coefficients

D_{90}= 0.7222 D_{85}= 0.6054 D_{60}= 0.3532
D_{50}= 0.2969 D_{30}= 0.2061 D_{15}= 0.1410
D_{10}= 0.1105 C_u= 3.20 C_c= 1.09

Remarks

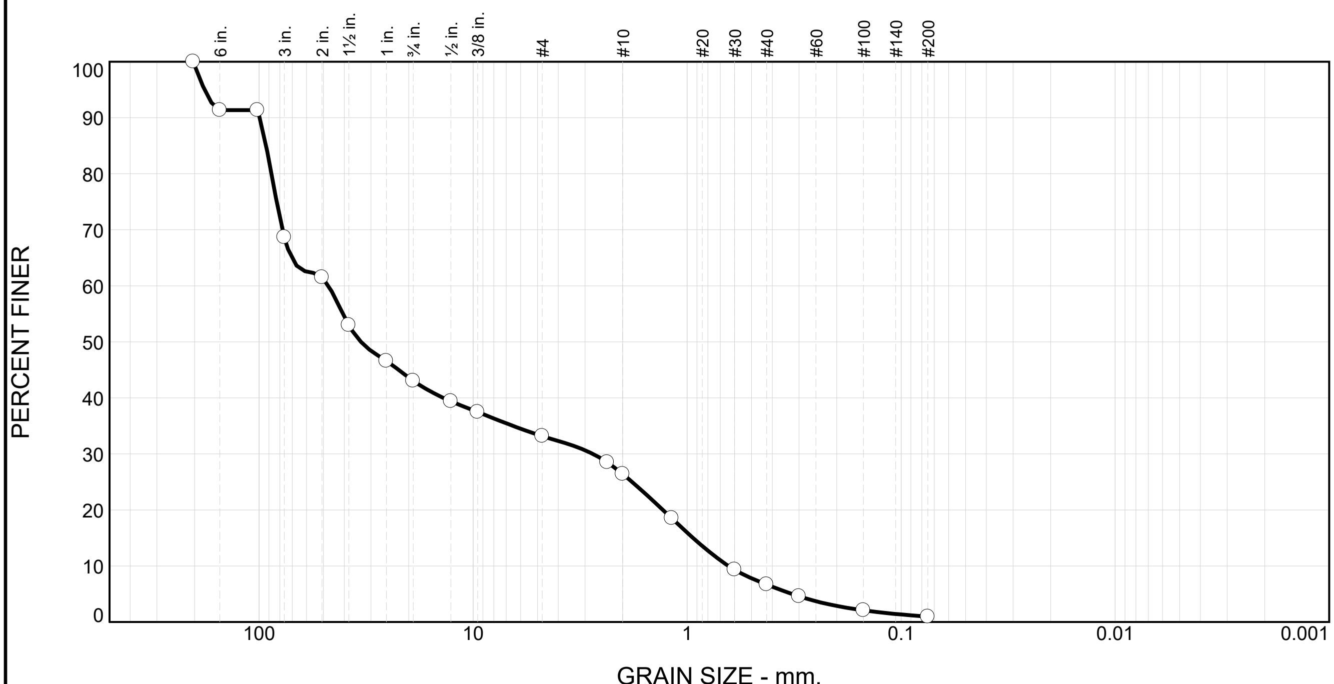
Date Received: 05/24/16 **Date Tested:** 05/27/16
Tested By: TL/SB
Checked By: SV
Title:

Location: WHITES CREEK-CREEK BED (FINE SAMPLE) **Date Sampled:** 05/24/16
Sample Number: SB-2

CME

Client: STANTEC
Project: MT. ROSE WTP
Project No: 1875 **Plate** C-1B

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
31.3	25.7	9.8	6.8	19.7	5.8	0.9	

Test Results (ASTM D 422 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
8	100.0		
6	91.3		
4	91.3		
3	68.7		
2	61.5		
1.5	53.0		
1	46.6		
.75	43.0		
.5	39.4		
.375	37.5		
#4	33.2		
#8	28.5		
#10	26.4		
#16	18.5		
#30	9.3		
#40	6.7		
#50	4.6		
#100	2.1		
#200	0.9		

* (no specification provided)

Material Description

POORLY GRADED GRAVEL WITH COBBLES AND SAND

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 99.1980 D_{85}= 92.7014 D_{60}= 47.3940
D_{50}= 33.4235 D_{30}= 2.7535 D_{15}= 0.9373
D_{10}= 0.6401 C_u= 74.05 C_c= 0.25

Remarks

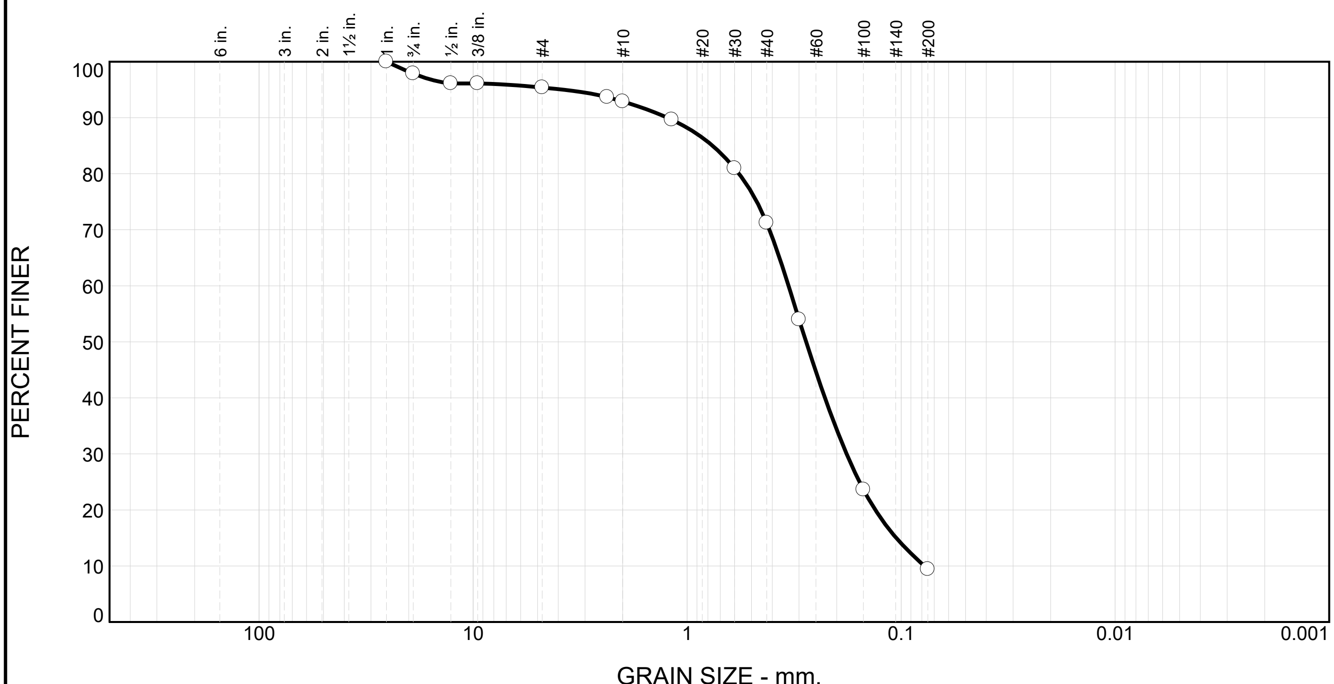
Date Received: 05/24/16 **Date Tested:** 05/27/16
Tested By: TL
Checked By: SV
Title:

Location: THOMAS CREEK-CREEK BED (COARSE SAMPLE)
Sample Number: SB-3
Date Sampled: 05/24/16

CME

Client: STANTEC
Project: MT. ROSE WTP
Project No: 1875 **Plate** C-1C

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	2.1	2.5	2.5	21.7	61.8	9.4	

Test Results (ASTM D 422 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
3/4"	97.9		
1/2"	96.1		
3/8"	96.1		
#4	95.4		
#8	93.7		
#10	92.9		
#16	89.6		
#30	81.0		
#40	71.2		
#50	54.0		
#100	23.6		
#200	9.4		

* (no specification provided)

Material Description

POORLY GRADED SAND WITH SILT

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=**

Coefficients

D_{90}= 1.2359 D_{85}= 0.7607 D_{60}= 0.3360
D_{50}= 0.2780 D_{30}= 0.1798 D_{15}= 0.1054
D_{10}= 0.0780 C_u= 4.31 C_c= 1.23

Remarks

Date Received: 05/24/16 **Date Tested:** 05/25/16
Tested By: GM/TL
Checked By: SV
Title:

Location: THOMAS CREEK-CREEK BED (FINE SAMPLE)
Sample Number: SB-4
Date Sampled: 05/24/16

CME

Client: STANTEC
Project: MT. ROSE WTP
Project No: 1875 **Plate** C-1D

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX D

PIEDMONT GEOSCIENCES, Inc.
10235 Blackhawk Dr.
Reno, Nevada 89508
775-972-3234

May 23, 2016

Construction Materials Engineers, Inc.
6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

ATTN.: Stella Montalvo

Thomas L. Sawyer
Piedmont GeoSciences, Inc.
10235 Blackhawk Drive
Reno, Nevada 89508-8527

SUBJECT: QUATERNARY FAULT RUPTURE HAZARD EVALUATION
Truckee Meadows Water Agency
Mt. Rose Water Treatment Facility
Reno, Washoe County, Nevada

Dear Stella,

This report describes a Quaternary fault evaluation conducted in support of the Truckee Meadows Water Agency's (TMWA) proposed Mt. Rose Water Treatment Facility for Construction Materials Engineers, Inc., Reno, Nevada by Mr. Thomas L. Sawyer of Piedmont GeoSciences, Inc., Reno, Nevada. The evaluation focuses on identifying the potential for surface-fault rupture hazard at or adjacent to the proposed Water Treatment Facility site, in the southwestern Truckee Meadows (Figure 1—herein "the project site").

BACKGROUND

The Quaternary fault evaluation of the proposed Mt. Rose Water Treatment Facility site adapts seismic hazard guidelines developed by the Nevada Earthquake Safety Council and jointly adopted by the Association of Engineering Geologists and the Nevada Bureau of Mines and Geology. The "*Guidelines for Evaluation of Potential Surface Fault Rupture/Land Subsidence Hazards in Nevada*" (AEG and NBMG, 1998) are generally recognized as the standard of practice in the state (Price, 1988).

The AEG-NBMG guidelines specify three general classes of Quaternary fault activity:

- *Holocene active fault* —Any fault with demonstrable movement in the past 10,000 years;
- *Late Quaternary active fault* —Any fault with demonstrable movement in the past 130,000 years;
- *Quaternary active fault* —Any fault with demonstrable movement in the past 1.6 million years.

The guidelines specify the “minimum level of investigation” in areas where Quaternary faults are present that shall include three principal tasks:

Task 1) Review of existing technical data;

Task 2) Interpret aerial photographs and other imagery; and

Task 3) Surface geologic investigation.

A forth task may be necessary “if a Quaternary active fault is mapped or otherwise interpreted to be present on the site”, as follows:

Task 4) Subsurface geologic investigation.

Nevada is ranked as the third most seismic activity state in the United States, after Alaska and California. Earthquake activity largely is concentrated in the western part of the state within the Walker Lane shear zone. About 20% of the relative motion between the Pacific and North American plates occurs along this shear zone (Kreemer and Hammond, 2007; Bormann et al., 2012). In the Reno region the shear zone is distributed from the Lake Tahoe basin on the west to the Fernley-Pyramid Lake area on the east (Figure 2).

The proposed Mt. Rose Water Treatment Facility site lies within the Carson Range fault zone, one of the principal faults of the northern Walker Lane shear zone (Figure 2). The widely distributed fault zone bounds the eastern front of the Carson Range and at approximately 2 to 3 mm/yr is characterized by one of the highest slip rates in Nevada (Ramelli et al., 1999). The most prominent trace of the fault zone bounds the abrupt range front, approximately 1.2 miles west of the project site (Bonham and Rogers, 1983; Szecsody, 1983) (Figure 3). Subsurface geologic studies provide evidence of surface faulting along the range-front trace within the past 1,000 years (Schilling and Szecsody, 1982).

Previous researchers have mapped numerous Quaternary faults in the vicinity of the project site (e.g., Bonham and Rogers, 1983; Szecsody, 1983; Bell, 1984; US Geological Survey National Quaternary fault and fold database [internet resource], 17 May 2016). Recently a group of researchers at the Nevada Bureau of Mines and Geology (Ramelli et al., 2011) revised the previous Quaternary fault and surfacial geologic mapping of the project site area.

SCOPE AND LIMITATIONS

The approach used in this surface rupture hazard evaluation of the project site conforms to the first three tasks as outlined in the AEG and NBMG (1998) guidelines, as follows:

Task 1) Compiled and reviewed existing geologic maps and published technical data pertaining to surface-rupture hazards at and near the project site;

Task 2) Obtained and stereoscopically examined and interpreted aerial photography and other remotely sensed imagery to identify geomorphic features commonly associated with Quaternary surface faults (see Table 1, for partial list of features) at or near the project site. Photogeologic mapping was preformed from interpretation of 1:12,000 (nominal scale), black-and-white aerial photographs of the project site area (Table 2) that were acquired by Dr. D.B. Slemmons under low-sun-angle conditions specifically to highlight the geomorphic expression of Quaternary faults; and

Task 3) Conducted reconnaissance-level Quaternary geologic mapping of the project site and site vicinity involving surfacial geologic and geomorphic mapping of fault-related features and geologic deposits. The Task 2 photogeologic mapping was verified and modified during field reconnaissance.

The forth task specified in the AEG and NBMG (1998) guidelines, a subsurface geologic investigation, is contingent on the findings of the current surface rupture hazard investigation (discussed below).

FINDINGS

The Task 1 data review indicates that the closest previously mapped Quaternary fault passes about 250 feet to nearly 500 feet to the west of the western site boundary (Bonham and Rogers, 1983; Szecsody, 1983; Ramelli et al., 2011) (Figure 3). The fault trace was verified and mapped from detail examination of aerial photographs, mostly 1:12,000, low-sun-angle, black-and-white air photos (Table 2) as part of Task 2, as well as during Task 3 field reconnaissance (Figure 4). The fault generally consists of two subparallel fault traces that exhibit geomorphic evidence for down-to-the-west normal offset of two different ages of alluvial-fan deposits, map units Qpf and Qpfo (younger and older, respectively).

This two-fold subdivision of alluvial-fan deposits agrees with previous workers who attributed their deposition to periods of late Pleistocene glacial outwash. Specifically, the older deposits (unit Qpfo) were considered to be associated with the Donner glacial period and the younger (unit Qpf) with the Tahoe glacial period (e.g., Bonham and Rogers, 1983). However recent revisions indicate that the deposits likely correlate to the Tahoe and younger Tiago glacial periods (Ramelli et al., 2011), respectively. The relationship of the fault traces to latest Pleistocene (possibly Holocene?) terrace deposits (unit Qpt) is unknown because they do not intersect within the map area. However, inset late Holocene stream-terrace deposits (unit Qht) are mapped along Whites Creek as concealing the fault traces (Figure 4).

The next closest fault trace is located about 1,700 feet to the east of the project site. Like the fault trace to the west, this fault also exhibits down-west offsets of the geomorphic surfaces on alluvial-fan units Qpfo and Qpf. Similarly the fault trace is concealed by late Holocene stream-terrace deposits (i.e., unit Qht) along Whites Creek (Figure 4).

The project site is underlain by the older alluvial-fan deposits (unit Qpfo) and, in the southwestern part of the site, by late Pleistocene (to Holocene?) stream-terrace deposits (Qpt). The terrace deposits are inset below the widespread alluvial-fan deposits and are unconformably overlain by Holocene stream-terrace deposits (i.e., unit Qht). Limited available exposure suggests that a cambic or cambic-like soil profile has develop in unit Qpt deposits, possibly indicating a Holocene rather than late Pleistocene age.

No geomorphic evidence was found to indicate or suspect the presence of a Quaternary fault at or adjacent to the project site. Apparently graded geomorphic surfaces extend east-west across the piedmont slope directly north of the project site on alluvial-fan deposits Qpfo and south of the site on deposits Qpf. These unbroken surfaces suggest a lack of through-going north-south-striking faults through the project site.

Although a set of approximately north-south striking fractures in the older alluvium was exposed in the southwest corner of the project site. The fractures have whitish calcium carbonate coatings and cut a paleosol (i.e., buried soil) developed in the older alluvium. The paleosol is characterized by relatively high clay and calcium carbonate content (carbonate engulfed argillic soil?)—consistent with a late Pleistocene soil in this region—and is buried by stream-terrace deposits Qpt. The fractures generally are oriented subparallel to nearby Holocene-active fault traces and, thus, may be related to faulting or secondary effects associated with surface faulting. However, the fractures appear to be restricted to the older deposits and paleosol and do not extend up-dip into overlying Pleistocene (Holocene?) terrace deposits. In addition, there is no obvious geomorphic expression along the projection of the fractures to the north on the geomorphic surface of unit Qpt nor further north on unit Qpfo (Figure 4), although there has been some surface modifications along the transmission line (see green dashed line in Figure 4) in this area.

CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations of this surface rupture hazard evaluations of the TMWA proposed Mt. Rose Water Treatment Facility are as follows:

- The project site lies with the Holocene-active Carson Range fault zone. The regional fault zone is capable of generating large-magnitude earthquakes in the future that are expected to rupture along identified Quaternary fault traces and producing moderately sever levels of ground shaking at the project site.

- The closest Quaternary fault is mapped approximately 250 feet or more to the west of the project site and exhibits geomorphic evidence for latest Pleistocene to Holocene fault activity.

- No geomorphic evidence was found to indicate nor to suspect the presence of a Quaternary-active fault at or adjacent to the project site. Thus, we do not recommend proceeding with a subsurface geologic investigation (i.e., Phase II activities), which was originally proposed as being contingent on the findings of this Phase I surficial geologic investigation.

- The potential for surface-fault rupture hazard at or directly adjacent to (within approximately 50 feet) the Mt. Rose Water Treatment Facility site is judged to be absent or insignificant based on the findings of the present Quaternary fault evaluation.

If you require additional information, please do not hesitate to contact me at your earliest convenience.

Sincerely,
PIEDMONT GEOSCIENCES, INC.

Thomas L. Sawyer

Thomas L. Sawyer
Principal Geologist

REFERENCES CITED

AEG (Association of Engineering Geologists) and NBMG (Nevada Bureau of Mines and Geology), 1998, Guidelines for evaluating potential surface fault rupture/land subsidence hazards in Nevada: Nevada Bureau of Mines and Geology, Revision 1 dated 20 November 1998, 7 p., (http://www.nbmg.unr.edu/nesc/guidelines.html).

Bell, J.W., 1984, Quaternary fault map of Nevada, Reno Sheet: Nevada Bureau of Mines and Geology Map 79, 1:250,000 scale.

Bonham H.F. Jr., and Rogers, D.K., 1983, Mt. Rose NE Quadrangle Geologic Map: Nevada Bureau of Mines and Geology Map 4Bg, 1:24,000 scale.

dePolo, C.M., 2008, Quaternary faults in Nevada: Nevada Bureau of Mines and Geology Map 167, 1:1,000,000.

Kreemer, C., and W.C. Hammond, 2007, Geodetic constraints on areal changes in the Pacific-North America plate boundary zone: What controls Basin and Range extension? *Geology*, Vol. 35, p. 943-946.

Ramelli, A.R., Bell, J.W., dePolo, C.M., and Yount, J.C., 1999, Large-magnitude, late Holocene earthquakes on the Genoa fault, west-central Nevada and eastern California: Bulletin of the Seismological Society of America, v. 89, p. 1458-1472.

Ramelli, A.R., Henry, C.D., and Walker, J.P., 2011, Preliminary revised geologic maps of the Reno urban area, Nevada: Nevada bureau of Mines and Geology Open-File Report 11-7, 3 plates.

Schilling, J. and Szecsody G.C., 1982, Earthquake hazard maps, Mt. Rose NE and Reno NW 7½-minute quadrangles: Finial technical report submitted to the U.S. Geological Survey, 63 p.

Szecsody, G.C., 1983, Earthquake hazards map, Mt. Rose NE 71/2-minute quadrangle, Nevada: Nevada Bureau of Mines and Geology Map 4Bi.

US Geological Survey, 2016, National Quaternary fault and fold database: Internet resource (http://qfaults.cr.usgs.gov) accessed 17 May 2016.

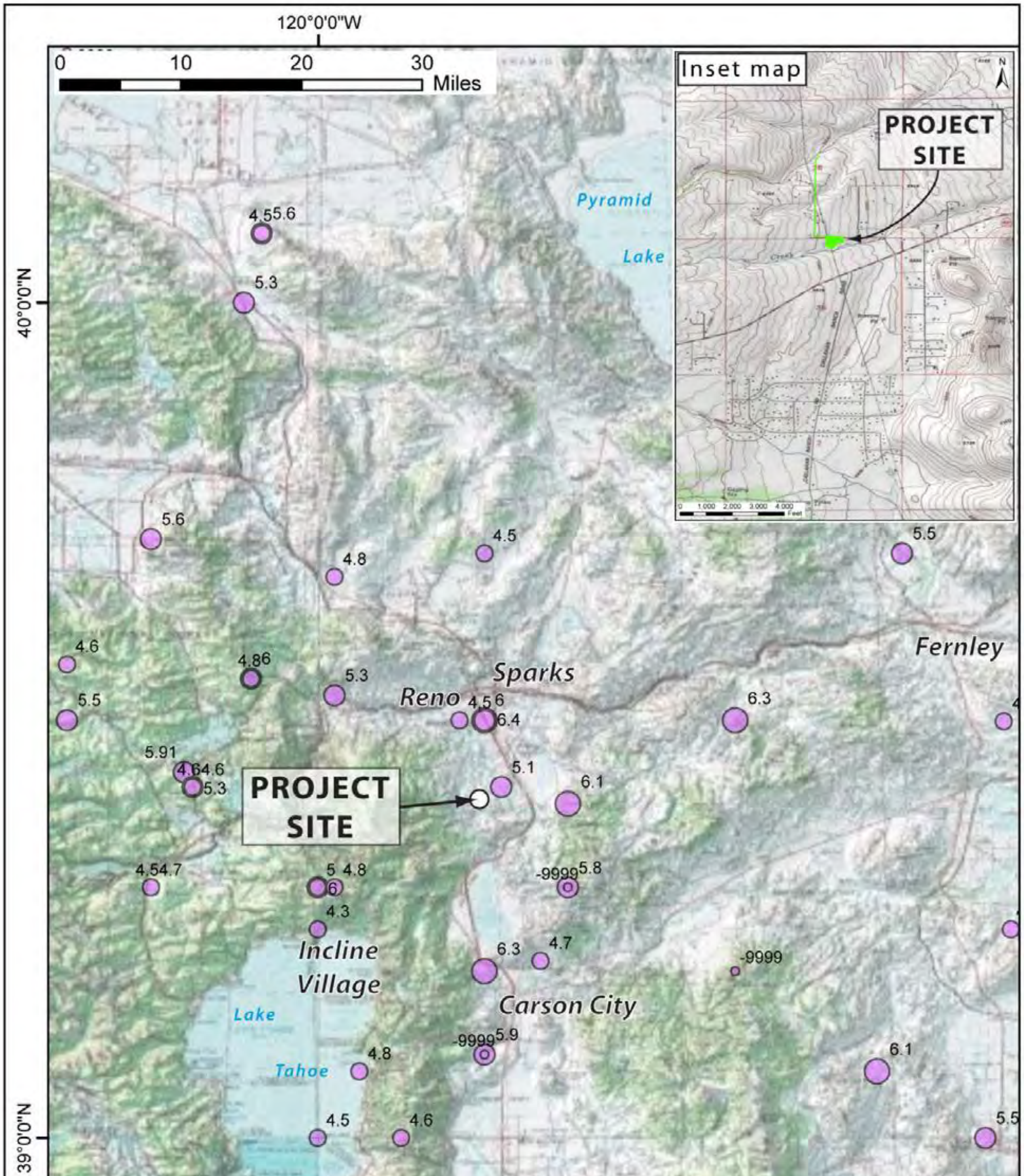
TABLE 1—GEOMORPHIC FEATURES ASSOCIATED WITH QUATERNARY FAULTS.

Normal faults	Strike-Slip Faults
Over-steepened base of mountain fronts	Scarps in Quaternary deposits and landforms
Scarps in Quaternary deposits and landforms, and offsets in Quaternary deposits	Scarps, and laterally offset streams and ridges, and offsets in Quaternary deposits
Grabens in Quaternary deposits	Rift valleys
Faceted spurs	Shutter ridges
Zigzag fault patterns	Pressure ridges
Wine glass canyons	Sag ponds
Fissures, springs, vegetation alignments	Fissures, spring and vegetation alignments

Source: AEG and NBMG, 1998

TABLE 2—AERIAL PHOTOGRAPHS EXAMINED

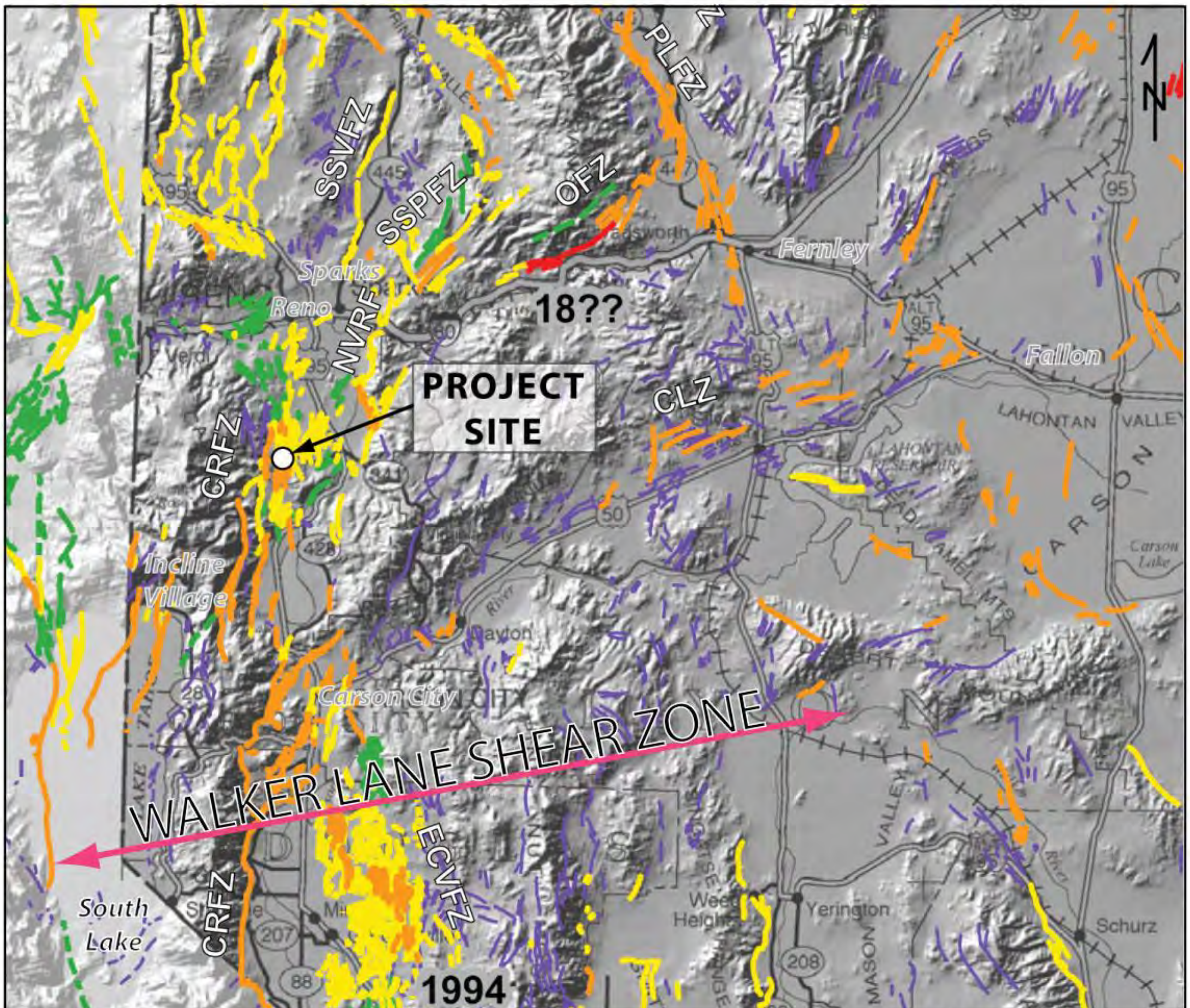
Source	Date	Nominal Scale	Flight Line/Photo #
Nevada Bureau of Mines and Geology / David B. Slemmons collection	ca1970	~1:12,000	312-315
Nevada Bureau of Mines and Geology / David B. Slemmons collection	ca1970	~1:12,000	36-39



PROJECT LOCATION MAP
w/ selected Historical Seismicity
Mt. Rose Water Treatment Facility

FIGURE
1

Project No. 1220	SCALE As Reported	DATE 23 May 2016



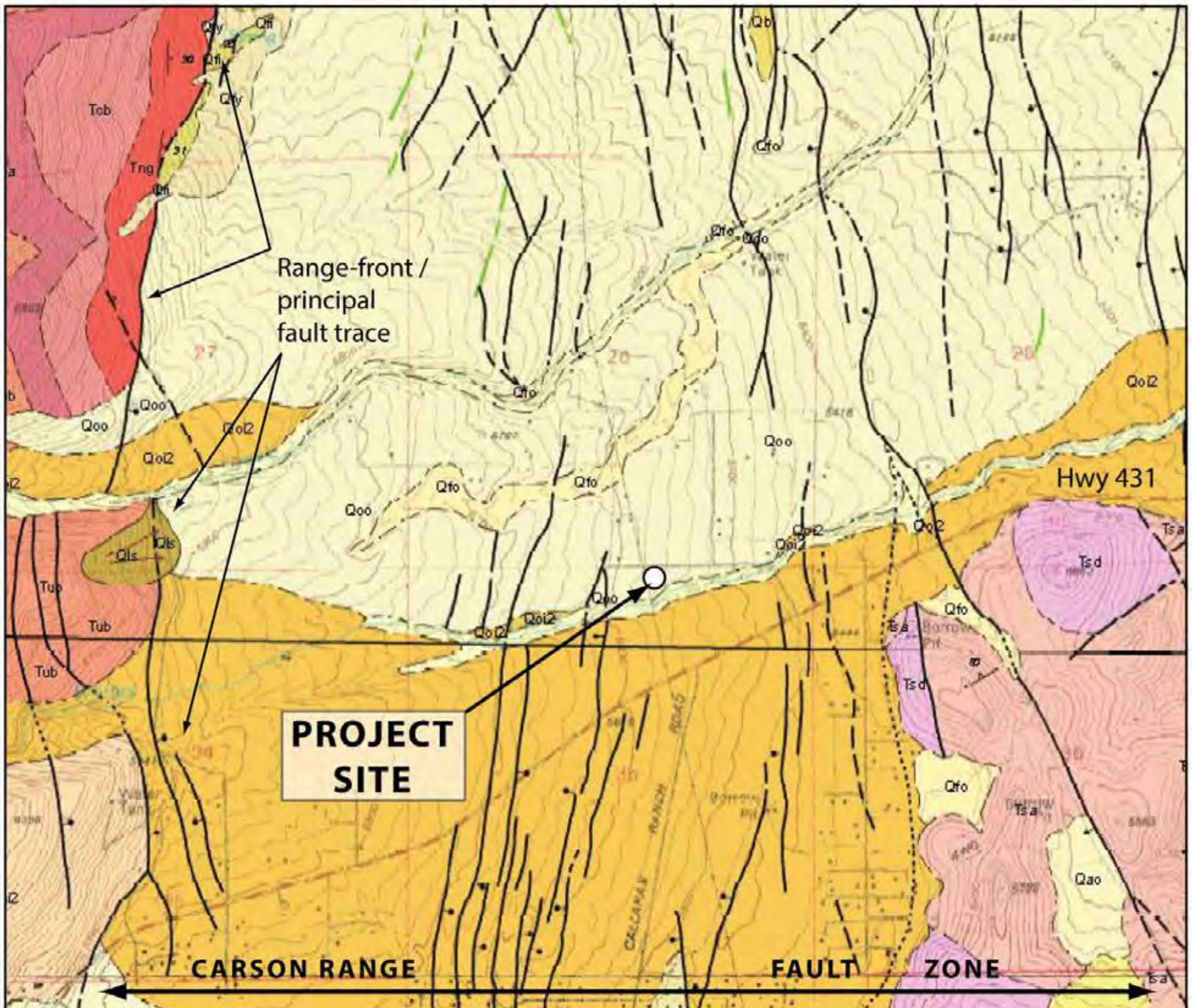
LEGEND OF FAULT ACTIVITY

- Historical(?)
- Holocene and latest Pleistocene
- Quaternary, Late Pleistocene
- Mid Quaternary
- Quaternary

FAULT NAME ABBREVIATIONS

CLZ	Carson lineament zone
CRFZ	Carson Range fault zone
ECVFZ	East Carson Valley fault zone
NVRF	Northern Virgina Range faults
OFZ	Olinghouse fault zone
PLFZ	Pyramid Lake fault zone
SSPFZ	Spanish Springs Peak fault zone
SSVFZ	Spanish Springs Valley fault zone
WVFZ	Warmsprings Valley fault zone

PGS	**REGIONAL QUATERNARY FAULT ACTIVITY MAP** Mt. Rose Water Treatment Facility		FIGURE 2
Proj. No. 1220	SOURCE: dePolo (2008)	SCALE As Reported	DATE 23 May 2016



LEGEND OF SELECTED MAP UNITS

Qoi2	Quaternary glacial outwash, intermediate age
Qfo	Quaternary alluvial-fan deposits, older
Qoo	Quaternary glacial outwash, older
Tx	various Tertiary volcanic rocks and sediment

0 0.5 1.0 mi

0 1.0 2.0 km

N

PGS	**GEOLOGIC MAP OF PROJECT SITE AREA** Mt. Rose Water Treatment Facility	FIGURE 3

Proj. No. 1220	SOURCE: Ramelli, et al. (2011)	SCALE As Reported	DATE 23 May 2016

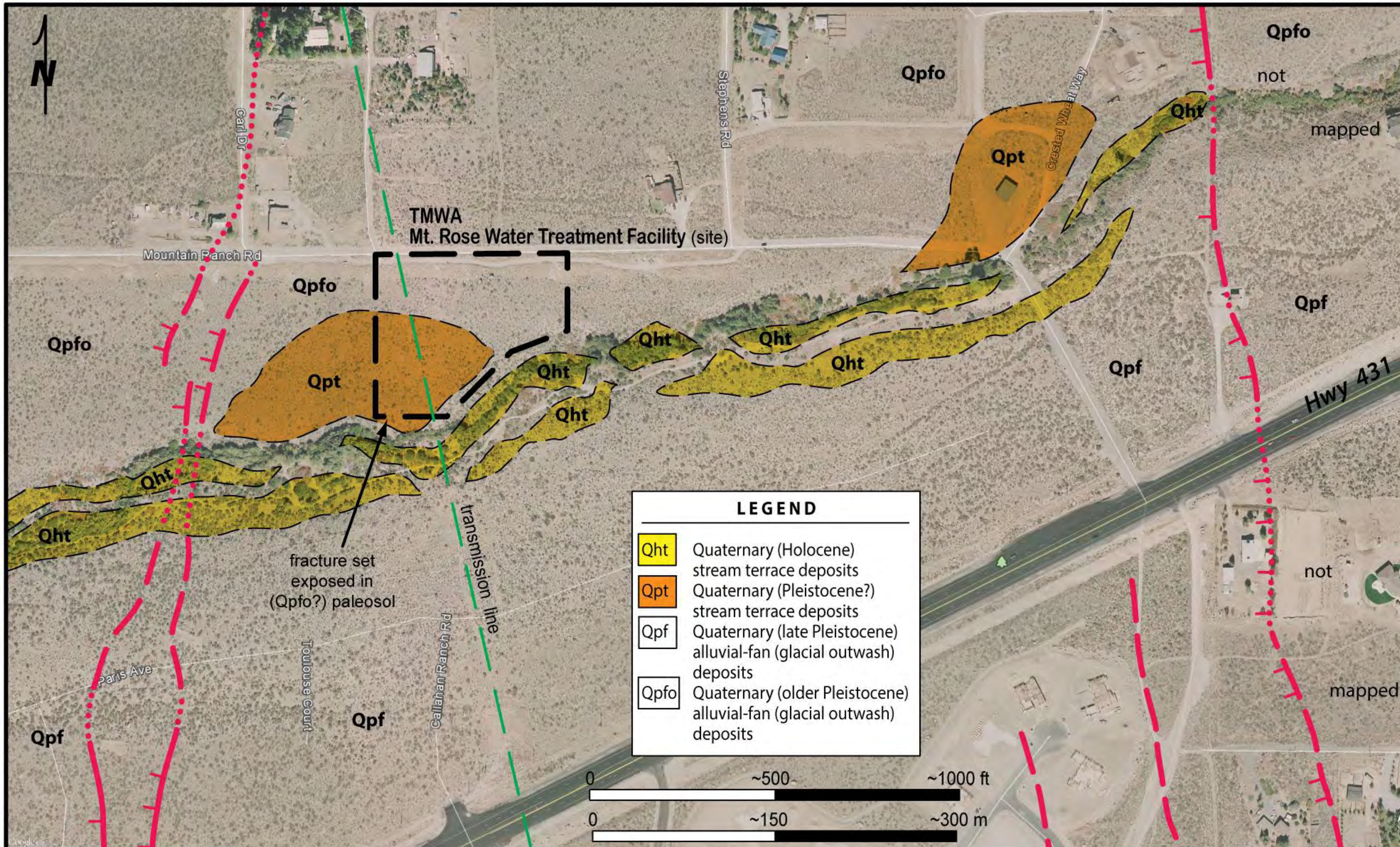


Figure 4. Map of the TMWA Mt. Rose Water Treatment Facility site (black dashed line) and adjacent areas shown in relation to the surface trace of Quaternary faults (red dashed and dotted lines) of the Carson Range fault zone and in relation to Quaternary surfacial deposits (from youngest to oldest, Qht, Qpt, Qpf and Qpfo) on a Google Earth image (acquired Oct., 2006). The project site is underlain by older alluvial-fan deposits (Qpfo) and to a lesser extent by late Pleistocene stream-terrace deposits (Qpt). Note: This map is intended for use in evaluating surface rupture hazards only at the TMWA project site.

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX E

USGS Design Maps Summary Report

User–Specified Input

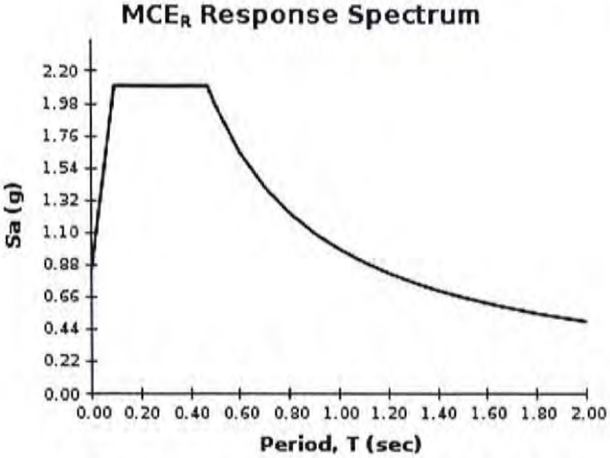
Report Title	Mt. Rose WTP Thu June 16, 2016 18:04:00 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	39.38881°N, 119.83219°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

USGS–Provided Output

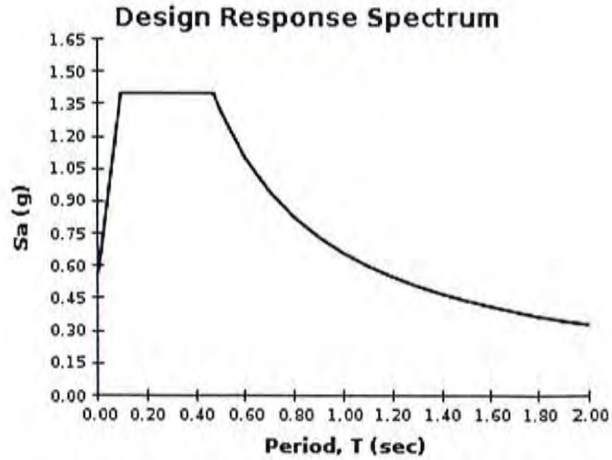
S_S =	2.101 g	S_{MS} =	2.101 g	S_{DS} =	1.400 g
S_1 =	0.760 g	S_{M1} =	0.988 g	S_{D1} =	0.658 g

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.

MCE_R Response Spectrum



Design Response Spectrum



USGS Design Maps Detailed Report

2012 International Building Code (39.38881°N, 119.83219°W)

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

Section 1613.3.1 — 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 2012 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1] S_S = 2.101 g

From Figure 1613.3.1(2) [2] S_1 = 0.760 g

Section 1613.3.2 — Site class definitions

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 Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

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 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \ge 1.25$
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 = 2.101 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
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 = 0.760 g, F_v = 1.300

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 2.101 = 2.101$ g

Equation (16-38): $S_{M1} = F_vS_1 = 1.300 \times 0.760 = 0.988$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 2.101 = 1.400$ g

Equation (16-40): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.988 = 0.658$ g

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

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} = 1.400 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

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} = 0.658 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 1613.3.5(1) or 1613.3.5(2)" = E

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf

USGS Design Maps Summary Report

User–Specified Input

Report Title Mt. Rose WTP
Thu June 16, 2016 18:04:35 UTC

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

Site Coordinates 39.38881°N, 119.83219°W

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

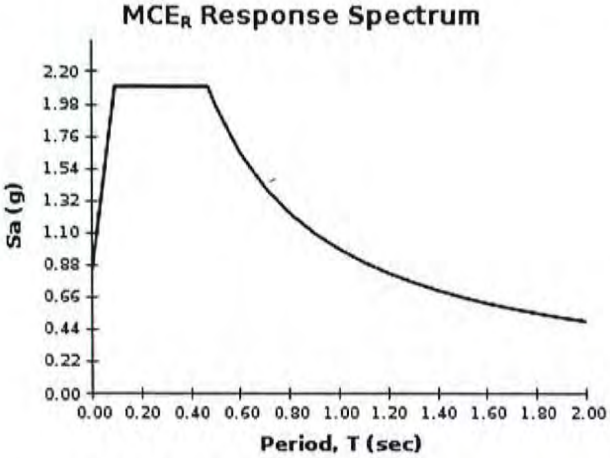
Risk Category I/II/III

USGS–Provided Output

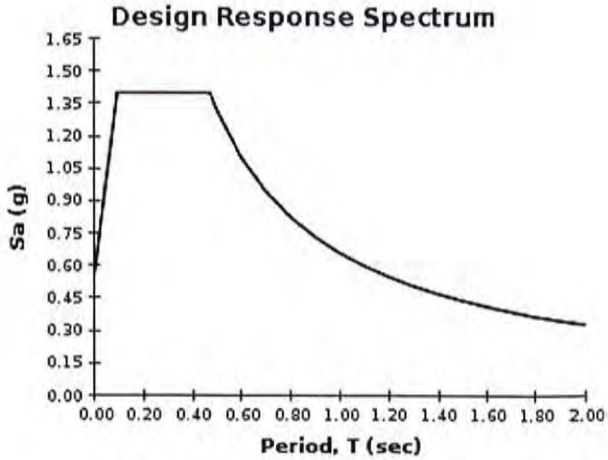
S_S =	2.101 g	S_{MS} =	2.101 g	S_{DS} =	1.400 g
S_1 =	0.760 g	S_{M1} =	0.988 g	S_{D1} =	0.658 g

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.

MCE_R Response Spectrum



Design Response Spectrum



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

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (39.38881°N, 119.83219°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] $S_S = 2.101$ g

From Figure 22-2 [2] $S_1 = 0.760$ 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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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	$\boxed{1.0}$
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 = 2.101 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				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
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	$\boxed{1.3}$
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 = 0.760 g, F_v = 1.300

Equation (11.4-1): $S_{MS} = F_aS_S = 1.000 \times 2.101 = 2.101$ g

Equation (11.4-2): $S_{M1} = F_vS_1 = 1.300 \times 0.760 = 0.988$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

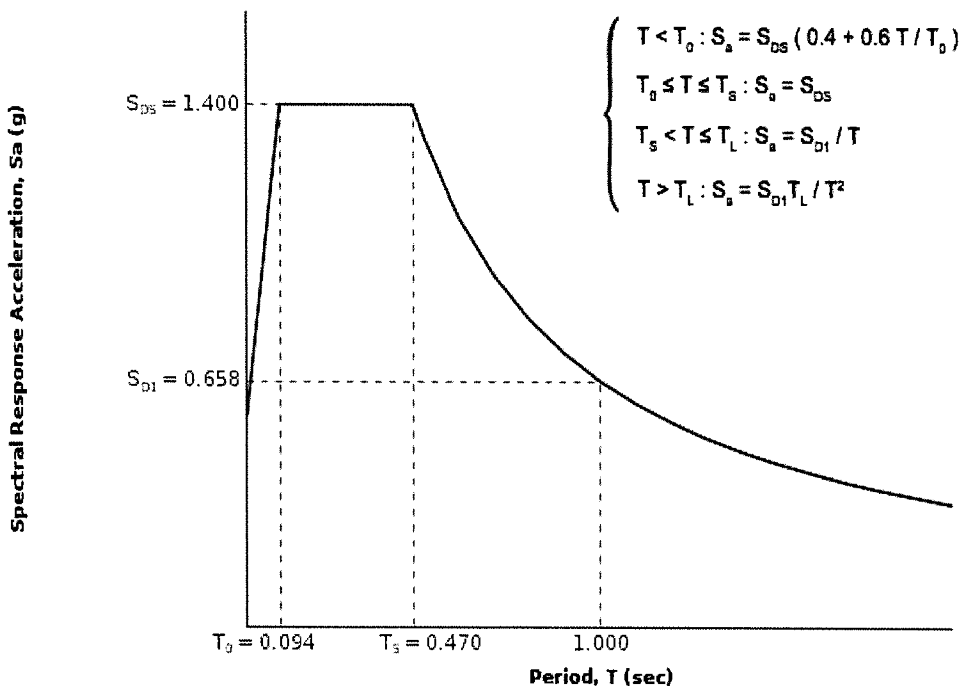
Equation (11.4-3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.101 = 1.400$ g

Equation (11.4-4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.988 = 0.658$ g

Section 11.4.5 — Design Response Spectrum

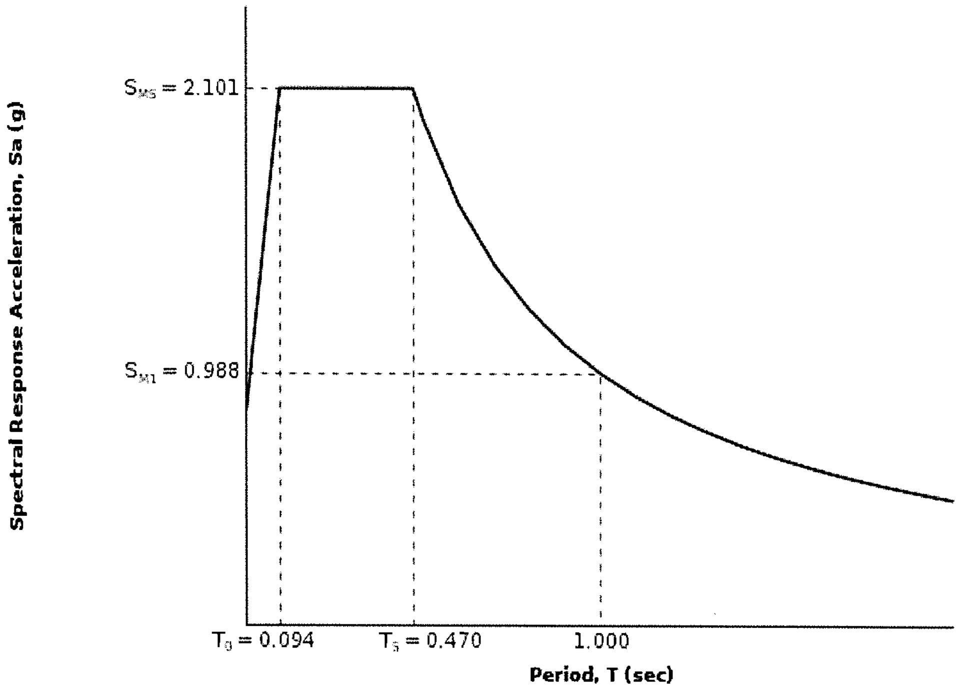
From Figure 22-12 [3] $T_L = 6$ seconds

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 = 0.827

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.827 = 0.827$ 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 = 0.827 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} = 0.890

From Figure 22-18 [6] C_{R1} = 0.861

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} = 1.400 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} = 0.658 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" = E

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

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

January 8, 2018
File: 1875

Addendum No.:1

John Buzzone, PE
STANTEC
6995 Sierra Center Parkway
Reno, NV 89511

RE: Recommendations for Pump Station Vaults and Retaining Walls Lateral Earth Pressures - Proposed Mt. Rose Water Treatment Plant, Reno, Washoe County, Nevada

Reference: *Geotechnical Investigation Mount Rose Water Treatment Plant, Reno, Washoe County, Nevada,* by Construction Materials Engineers, April 21, 2017

Dear Mr. Buzzone:

Construction Materials Engineers, Inc. (CME) received a request from Ms. Kara V. VanValkenburg with BJG Architecture & Engineering to provide alternative lateral earth pressure recommendations for the proposed pump station to be installed as part of the Mt. Rose Water Treatment Plant project. The proposed pump station vault is approximately 31 feet long, 15 feet wide, and 25 feet tall.

Pump station vault will be designed with the assumption that vertical side walls will act as a restrained retaining wall (at-rest soil pressure) . Ms. VanValkenburg has requested lateral earth pressure recommendations for the proposed pump station vault to include:

1) Static lateral earth pressure (active, at-rest, and passive) for “moist” or saturated soil conditions (i.e. increase in soil moisture content of the backfill soils, non-buoyant)

2) Static lateral earth pressure (active, at-rest, and passive) for submerged soil conditions (i.e. buoyant unit weight plus hydrostatic pressure below water table);

3) Pseudostatic lateral earth pressure (seismic) for “moist” or saturated soil conditions (i.e. increase in soil moisture content of the backfill soils, non-buoyant);

Recommendations included in this addendum should be attached to the original geotechnical investigation report and information contained herein included as part of future contract documents.

1.0 Retaining Walls and Vaults

1.1 Anticipated Soil Conditions

It is understood that Test Pit TP-1 was excavated in the general vicinity of the proposed in-ground pump station. Soils encountered were classified as either silty sand **(SM)** or silty sand with gravel and cobbles **(SM)** to a depth of approximately 7 feet bgs. Below a depth of 7 feet bgs, soils encountered were classified as silty gravel with sand **(GM)**. At a depth ranging from 10 to 13 feet, soils become indurated (hardpan layer), but are easily broken using the excavator bucket. Below a depth of 13 feet, soils encountered were classified as either poorly graded gravel with silt **(GP-GM)** or poorly graded sand with gravel **(SP)**.

At a depth of 18 feet, soil moisture content increased significantly but ground water was not observed. It is assumed that groundwater levels at the proposed pump station extend below the depth of exploration (18 ½ feet).

1.2 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). The 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 ability to deflect at the top (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[1]).

Lateral pressures have been provided for two conditions:

1) **Saturated-backfill:** Backfill soils may be seasonally saturated. This condition arises from surficial moisture infiltrating into the backfill soils and buoyant forces within the soil matrix develop. This condition is assumed to occur due to the proposed embedment depth of the pump station and lack of foundation drainage;

2) **Submerged-backfill:** This condition is assumed to occur with a rise in the groundwater table, which may be due to flooding. Flooding of the backfill soils may occur due to increased flows and breaching of water from the adjacent creek bank. The effect on wall pressure will be an increased lateral load due to hydrostatic pressures.

[1] Assumes a deflection equal to 0.5 percent of the total wall height.

It is assumed that the pump station is sealed (i.e. no groundwater will enter the vault). For the submerged/buoyant condition it is assumed that groundwater will rise alongside the pump stationvault.

Table 1 –Lateral Earth Pressure Values

STATIC LATERAL EARTH PRESSURE

Soil Moisture Assumption	Earth Pressure Condition	Earth Pressure Coefficient	Equivalent Fluid Pressure (psf)
Saturated (1,2,3)	Active (P_a)	0.29 (K_a)	39
	At Rest (P_o)	0.44 (K_o)	59
Submerged/ Buoyant (1,2,4)	Active (P_a)	0.29 (K_a)	81
	At Rest (P_o)	0.44 (K_o)	90

(1) Pounds per square foot per foot of depth
(2) Lateral pressures for level backfill calculated using an average of the Rankine and Coulomb Equations for lateral earth pressure.
(3) Saturated condition assumes increase in soil moisture content of the surrounding backfill soils for an extended period of time. This increase in moisture content assumes an increase in backfill unit weight. A maximum saturated unit weight of 135 pcf (maximum unsaturated unit weight 125 pcf) and a friction angle of at least 34 degrees.
(4) For vaults located below the groundwater table (i.e. submerged soils) buoyant unit weight for backfill soils 62.6 pcf and a unit weight of water equal to 62.4 pcf.

Subterranean structures and short retaining walls, including foundations, should be designed to resist the lateral earth pressure exerted by the retained soil plus any additional lateral force that will be applied to the wall due to surface loads placed at or near the wall. The lateral earth pressure may vary based on the soil moisture content and groundwater levels. A diagram illustrating static lateral forces for “At Rest” conditions with hydrostatic loading is included as Plate C-1 (Typical Static Lateral Force Diagram “At Rest Condition”).

A passive pressure of 350 psf may be used for saturated conditions. For submerged soil conditions, passive pressure of 190 psf is applicable for design.

1.3 Hydrostatic Uplift

The hydrostatic uplift force is dependent on the groundwater level around the pump station. Resistance to the uplift force can be achieved by using a combination of the shearing resistance of the backfill soil and the weight of the structure. As an option to increase the resistance, the pump station vault base could be extended to incorporate the soil backfill weight resting on the base extension. A typical uplift force diagram is attached as Plate C-2 (Typical Pump Station Uplift Force Diagram).

A saturated unit weight of 135 pcf can be used for the soil located above the static waterline when calculating the vertical restraint. Soils located below the assumed static waterline should utilize a buoyant unit weight of 62.6 pcf (unsaturated unit weight (125 pcf) reduced by the unit weight of water (62.4 pcf))

The structure should be protected from excessive hydrostatic uplift. If adequate dead load is not available to resist the maximum buoyant condition (an empty structure), soil anchors may also be used to resist excessive uplift loads.

1.4 Seismic Lateral Earth Pressures

The 2012 IBC requires retaining walls over 6 feet in height to be designed for seismic pressures. 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).
- $\mathbf{k_h}$: Horizontal seismic ground acceleration component. This component is derived from the design spectral response acceleration parameter at short period **(S_{Ds})**, as described in this section.
- $\mathbf{K_{AE}}$**:** Seismic active earth pressure coefficient.
- $\mathbf{P_{AE}}$: Dynamic lateral earth pressure force: $P_{AE}= ½\gamma H^2 K_{AE}$, 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.
- $\mathbf{P_{AE(sat)}}$**:** Dynamic lateral earth pressure force assuming soils are located below the water table in saturated soils (i.e. buoyant unit weight plus hydrostatic pressure).
- $\mathbf{P_w}$**:** Hydrostatic pressure force due to saturation of backfill soils.

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.

The standardized approach for pseudostatic earth pressure determination is the use of the Mononobe-Okabe method (M-O Method)[2] 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.

For the proposed vault, we have a specified geometry which assumes a 25-foot vertical face vault wall with a bury depth of approximately 22.5 feet. The vault walls will be near-vertical (i.e. no wall batter), and surcharge loading above the proposed vault is not proposed. Based on the assumed geometry and anticipated soil conditions at the site, earth pressure computations were completed by back calculating an applied boundary force to the vertical plane (i.e. vertical soil/vault contact plane) assuming level backfill conditions, in general accordance with the GLE approach for determining seismic active pressures as outlined in National Cooperative

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[2] *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).*

Highway Research Program (NCHRP) Report 611 (2008). Slide v6.0 (RocScience©) was utilized to model the anticipated slope conditions and vault face geometry.

The horizontal seismic ground acceleration component k_h is a function of the short period design spectral response acceleration (**S_{DS}**). For the purposes of this design the earthquake ground motion was approximated by dividing the short period design spectral response acceleration (**S_{DS}**) by 2.5 (FEMA/NEHRP, 2009). A design horizontal seismic coefficient was determined by taking ½ of the design earthquake ground motion (k_h= ½*(S_{DS}/2.5)). A k_h of 0.2g was used for the GLE analysis. Several different failure surfaces were analyzed using Slide v6.0 software and an average K_{AE} was back calculated from the resultant applied boundary load required to achieve a pseudostatic stability factor of safety of at least 1.1. The average K_{AE} was then used to determine the seismically induced equivalent fluid pressure (P_{AE}) for saturated soil conditions.

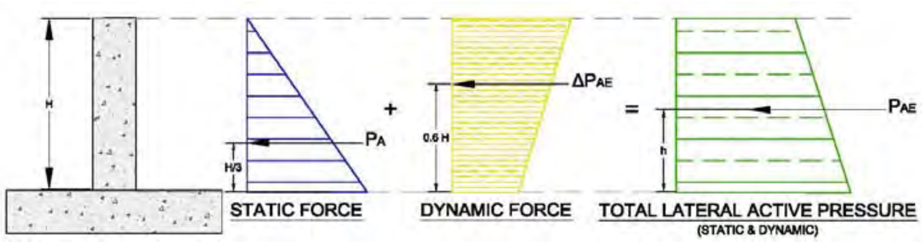
It is assumed a low probability that the pump station will experience both the "submerged" soil condition and earthquake loading forces concurrently. Therefore; the results included as Table 2 (Pseudostatic Lateral Earth Pressure Value-Pump Station) are assumed to exemplify the appropriate design condition for this project.

Table 2 – Pseudostatic Lateral Earth Pressure Values-Pump Station

Earth Pressure Condition	**Pseudo Static Earth Pressure Coefficient**		**Seismically Induced Equivalent Fluid Pressure[1] (psf/ft)**	**Component Earth Pressures[1] (psf/ft)** (P_{AE}=ΔP_{AE}+P_A)	
	Slope	*$K_{AE}^{(2,3)}$*	*$P_{AE} = (\gamma_{soil} * K_{ae})^{(4)}$*	*Seismic (ΔP_{AE})*	*Static (P_A)*
Assumes Saturated ("Moist") Condition	Level	0.64	87	51	36

(1) Pounds per square foot per foot of depth. P_{AE} is the **total wall pressure** for pseudo static loading and includes both the static and seismic lateral earth pressure components. A maximum saturated unit weight of 135 pcf (maximum unsaturated unit weight 125 pcf) and a friction angle of at least 34 degree, with no surcharge loading.
(2) Based on short period design spectral response acceleration (S_{DS}) of 1.4 and assuming yielding conditions. For walls that can with stand movement to mobilize active earth pressure conditions during the design earthquake event, ½ the design earthquake ground motion is applicable.
(3) Seismically induced equivalent fluid pressure assumes soil unit weight under saturated conditions.
(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 6 foot tall wall with a P_A=35 psf/ft and a ΔP_{AE}=48psf/ft would have a resultant force (P_{AE}=61 psf/ft) acting at a height (h) equal to about 2.9 feet. ***NOTE: FORCE MODEL DIAGRAM EXCLUDES HYDROSTATIC PRESSURE (P_w)***

We appreciate the opportunity to provide our services. If you have any questions or require further information, please do not hesitate to call.

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

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Senior Geotechnical Engineer
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PROFESSIONAL ENGINEER - STATE OF NEVADA
STELLA A. MONTALVO
Exp. 12-31-2019
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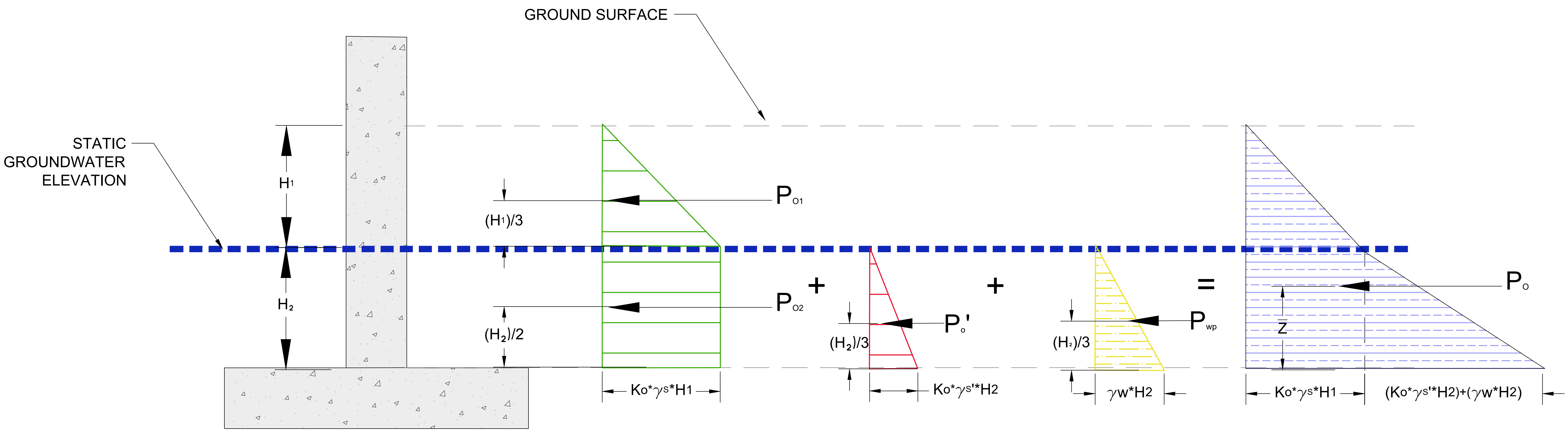
No. 021801

01-8-18



$$P_o(lb/ft)=P_{o1}+P_{o2}+P_o'+P_{wp} \Rightarrow \tfrac{1}{2}K_o\gamma_s H_1^2 + K_o\gamma_s H_1 H_2 + \tfrac{1}{2}(K_o\gamma_s'+\gamma w)H_2^2$$

$$\bar{Z}(ft)= \frac{(P_{o1}((H_1/3)+H_2)) + (P_{o2}*\tfrac{1}{2}H_2) + (P_o'*(H_2/3)) + (Pwp*H_2/3)}{P_o(lb/ft)}$$

ASSUMED GRANULAR BACKFILL SOIL PROPERTIES

γ_s=135 lb/ft^3 K_o=0.44

γ_s'=62.6 lb/ft^3 φ=34°

γ_w=62.4 lb/ft^3

γ_s :UNIT WEIGHT OF BACKFILL SOILS "MOIST CONDITION" (lb/ft^3)

TYPICAL LATERAL FORCE DISTRIBUTION DIAGRAM INCLUDING HYDROSTATIC CONDITIONS

N.T.S

V:\Active\1875\Addendum for Hydrostatic pressures\pump station diagram 9-19-17.dwg

CME CONSTRUCTION MATERIALS ENGINEERS INC.
6980 Sierra Center Parkway, Suite 90
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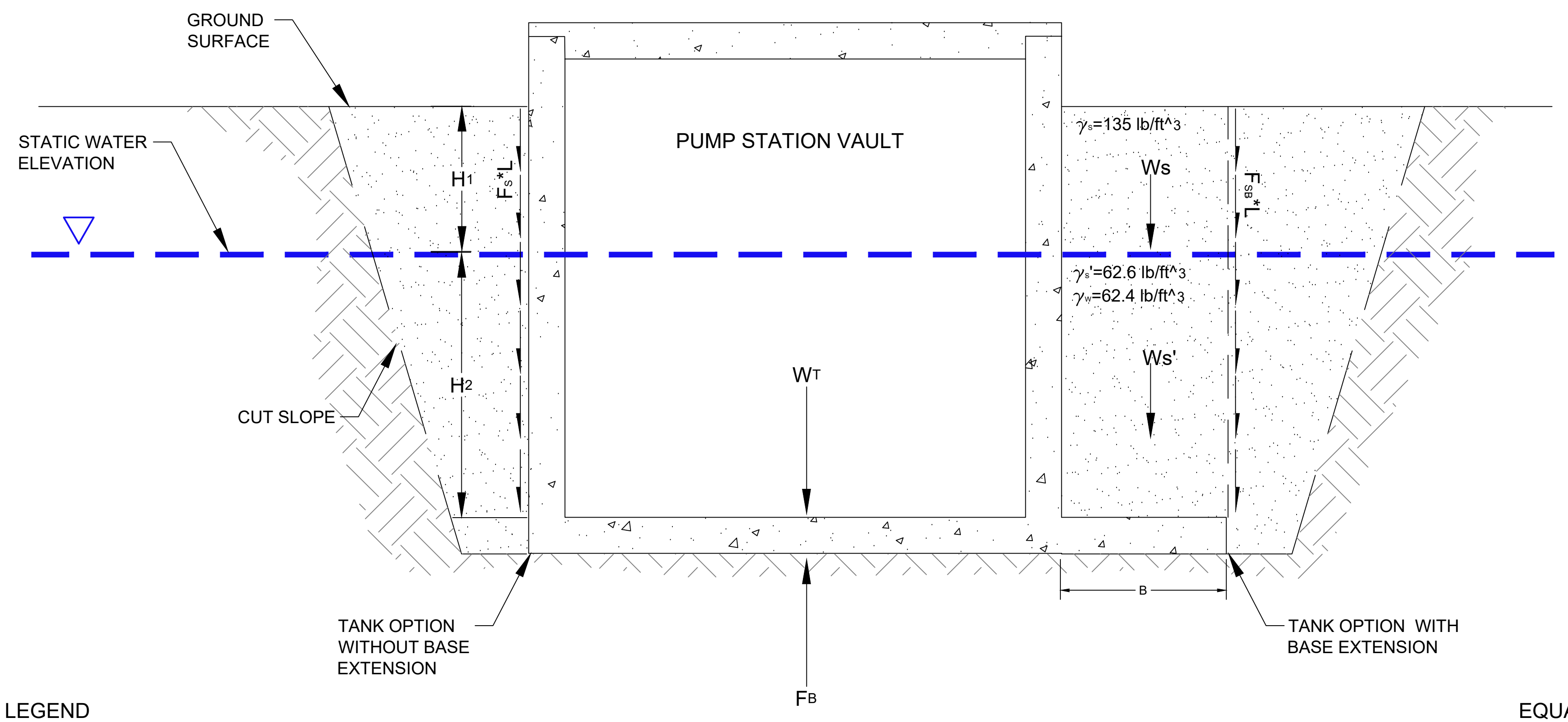
STANTEC
MOUNT ROSE WATER TREATMENT PLANT
TYPICAL STATIC LATERAL FORCE DIAGRAM "AT REST CONDTION"
WASHOE COUNTY, NEVADA
PROJECT NO.: 1875 DATE: 9/19/2017

LEGEND

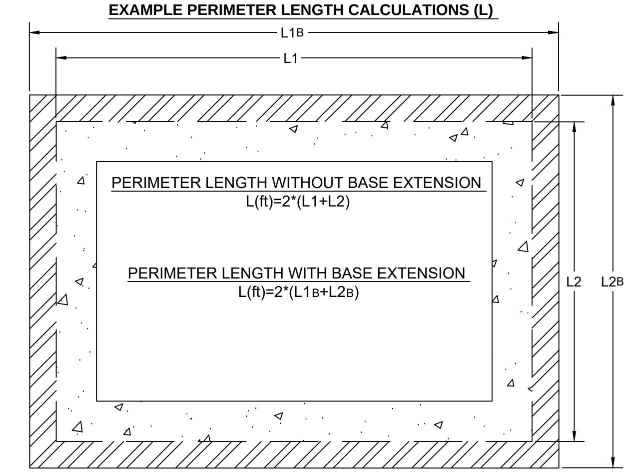
PLATE
C-1

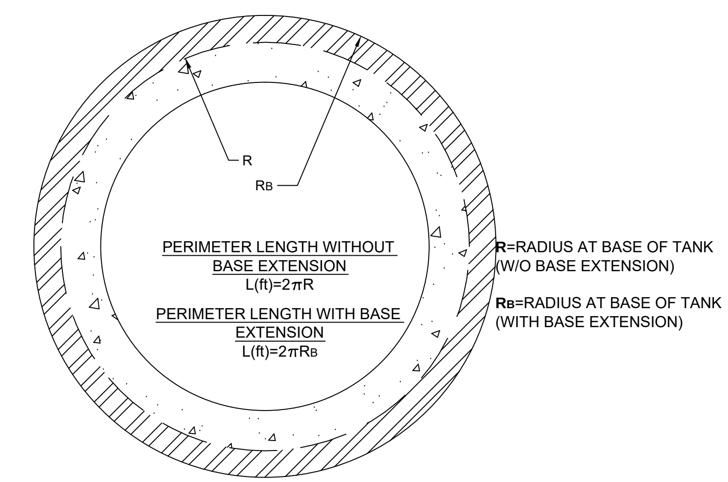
TYPICAL PUMP STATION UPLIFT FORCE DIAGRAM

N.T.S



EXAMPLE PERIMETER LENGTH CALCULATIONS (L)





LEGEND

- BASE EXTENSION DESIGNATION (PERIMETER CALCULATION EXAMPLE)
- GRANULAR BACKFILL (TYPICAL)
- DESIGNATES LIMITS OF EXCAVATION CUT SLOPE (TYPICAL)
- CONCRETE PUMP STATION/VAULT (TYPICAL)

W_T	EFFECTIVE WEIGHT (lb) OF THE PUMP STATION/VAULT
F_B	BUOYANT FORCE (lb)= γw(lb/ft^3)*DISPACED VOLUME OF THE FLUID (ft^3)
Ws	WEIGHT (lb) OF SOIL ABOVE WATER TABLE= γs *H_1*B*L (=> DIVIDE BY 1,000 TO COVERT TO KIPS)
Ws'	EFFECTIVE WEIGHT (lb) OF SOIL (BELOW WATER TABLE)=(γs-γw)*H_2*B*L (=> DIVIDE BY 1,000 TO COVERT TO KIPS)
H_1	HEIGHT (ft) OF SOIL ABOVE WATER TABLE
H_2	HEIGHT (ft) OF SOIL BELOW WATER TABLE
γ_s	UNIT WEIGHT OF BACKFILL SOILS (lb/ft^3)
γ_s'	EFFECTIVE UNIT WEIGHT OF BACKFILL SOILS (lb/ft^3)
F_S	SHEARING RESISTANCE OF SOIL TO TANK WALL (kip/ft)
F_{SB}	SHEARING RESISTANCE OF SOIL TO SOIL (kip/ft)
L	PERIMETER LENGTH AROUND BASE OF TANK (ft)
B	WIDTH OF BASE EXTENSION (ft)
δ	FRICTION ANGLE BETWEEN WALL AND GRANULAR BACKFILL
φ	SOIL FRICTION ANGLE

EQUATIONS

FACTOR OF SAFETY (UPLIFT)= (NO BASE EXTENSION) $\frac{W_T+(F_S*L)}{F_B} \geq 2$

FACTOR OF SAFETY (UPLIFT)= (WITH BASE EXTENSION) $\frac{W_T+W_S+W_S'+(F_{SB}*L)}{F_B} \geq 2$

WHERE:

$W_S\,(\text{KIPS})= \frac{\gamma s *H_1*B*L}{1,000}$ $F_S(\text{KIPS})=(0.016*H_1^2)+(0.032*H_1*H_2)+(0.008*H_2^2)$

$W_S'\,(\text{KIPS})= \frac{\gamma s *H_2*B*L}{1,000}$ $F_{SB}(\text{KIPS})=(0.019*H_1^2)+(0.038*H_1*H_2)+(0.009*H_2^2)$

ASSUMED GRANULAR BACKFILL SOIL PROPERTIES

γ_s=135 lb/ft^3	K_o=0.44
γ_s'=62.6 lb/ft^3	δ=30°
γ_w=62.4 lb/ft^3	φ=34°

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STANTEC
MOUNT ROSE WATER TREATMENT PLANT
TYPICAL PUMP STATION UPLIFT FORCE DIAGRAM
WASHOE COUNTY, NEVADA

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LEGEND

PLATE C-2