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April 14, 2016
Project No: 1824

Kelly McGlynn, PE
TRUCKEE MEADOWS WATER AUTHORITY
1355 Capital Boulevard
Reno, NV 89502

RE: Geotechnical Investigation
Satellite Hills Booster Pump Station
Sparks, Nevada

Dear Mr. McGlynn:

Construction Materials Engineers, Inc. (CME) is pleased to submit the Geotechnical Investigation Report for the proposed Satellite Hills Booster Pump Station located along the east side of Sparks Boulevard, near the intersection with Satellite Drive in Sparks, Nevada.

1.0 INTRODUCTION

Recommendations presented in this report are based on surface and subsurface conditions encountered during our field exploration and our understanding of the proposed project as described in this report. The purpose of this geotechnical investigation was to explore the general soil, bedrock and groundwater conditions at the subject site and provide geotechnical recommendations for project design and construction.

The proposed site is contained in Section 27, T20N, R20E, M.D.M. The area covered by this report as well as some of the existing site improvements are presented on Plate A-1.

Our geotechnical study included field exploration, laboratory testing, and engineering analysis to identify the physical and mechanical properties of the subsurface soil and bedrock profile. The results of subsurface exploration and laboratory testing are included in this report and serve as the basis for the conclusions and recommendations contained herein.

2.0 SITE DESCRIPTION

The subject property easement encompasses approximately 15,400 square feet with a dimension of about 110 feet by 140 feet. The site is partial developed with an existing generator and transformer placed on a concrete pad that is providing emergency power to an existing below grade booster pump station facility located at the intersection of Satellite Drive and Sparks Boulevard. A fence is located along the perimeter of the developed pad area. A paved driveway from Sparks Boulevard provides access to the generator. The existing improvements will be removed to accommodate the new booster pump station.

The booster pump station will be constructed on a near level pad excavated into the existing hillside having a moderate slope gradient of about 20 percent. Except for several trees located adjacent to the existing site facility, vegetation is generally sparse at the site. Photo #1 shows site topography and enclosure area.

Photo #1: Looking southwest from east side of property. Note existing generator enclosure to the west.

3.0 PROJECT DESCRIPTION

It is understood that a booster pump station is planned to be constructed along the east side of Sparks Boulevard near the intersection with Satellite Drive.

The footprint of the new building will be approximately 600 square feet with a dimension of about 24.7 feet by 24.7 feet. The planned finished floor elevation is 4477.40 feet. It is assumed the building will have masonry walls with concrete floor slab-on-grade. Structural loading is assumed to be light to moderate.

Based on site and grading plans by Lumos and Associates, the building is located about 80 feet from Sparks Boulevard into an existing hillside. Based on the building finished floor elevation, grading will require cuts of about 15 feet. A retaining wall will be constructed along the perimeter of the building pad having a maximum height of about 13 feet. The planned building will be separate from the retaining wall.

A generator and transformer will also be constructed along the south side of the building, placed on a concrete slab-on-grade. Appurtenant construction consists of a paved driveway and underground piping.

4.0 FIELD EXPLORATION

4.1 Test Pits

The proposed site was explored on November 16, 2015 by excavating 3 test pits to maximum depths of 16 feet below the existing ground surface (bgs).

4.2 Exploration Location and Ground Elevation

The test pit locations was determined by approximate methods, referencing existing site improvements, and is presented on the Site Plan (Plate A-1 in Appendix A). The elevation shown on the test pit logs was obtained by interpolation between contour lines obtained from the existing topographic map. The elevations and locations included in this report should be considered accurate only to the degree implied by the methods used.

4.3 Material Classification

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

5.0 LABORATORY TESTING

All soil testing performed in the CME soils laboratory is conducted in accordance with the standards and methodologies described in Volume 4.08 (Soil and Rock; Dimension Stone; Geosynthetics) of the ASTM Standards.

Samples of significant soil types were analyzed to determine their in situ moisture content (ASTM D 2216), grain size distribution (ASTM D 422), and Atterberg Limits (ASTM D4318). Results of these tests were used to classify the soils according to ASTM D 2487

Corrosion testing including soluble sulfates, pH, and resistivity was completed by an outside laboratory.

Results from our laboratory test program are included in Appendix B. Index test results are also presented on the Test Pit logs (Plate A-2).

6.0 GEOTECHNICAL AND GROUNDWATER CONDITIONS

Based on a review of the *Geologic Map of the Vista Quadrangle* (Bell and Bonham, 1987), the project site lies within volcanic bedrock typically consisting of hydrothermally altered andesitic or pyroclastic flows. This volcanic bedrock is characterized as *typically argillized and consists predominantly of montmorillonite and/or kaolinite* (Bell and Bonham, 1987). The severe bleaching, iron staining and development of montmorillonite and kaolinite within the argillized zone is due to the chemical reactions caused by warm to hot acidic water leaching into the rock (hydrothermal alteration).

As exposed within the adjacent bedrock cut slopes, the physical characteristic of the bedrock due to hydrothermal alteration is not uniform. Isolated areas containing clay seams and pockets are exposed that represents intensely propylitized and heavily argillized zones, which typically have the highest expansion potential within the altered bedrock. These clay seams are typically characterized as having a greenish color with a soft to very soft consistency and deeply weathered.

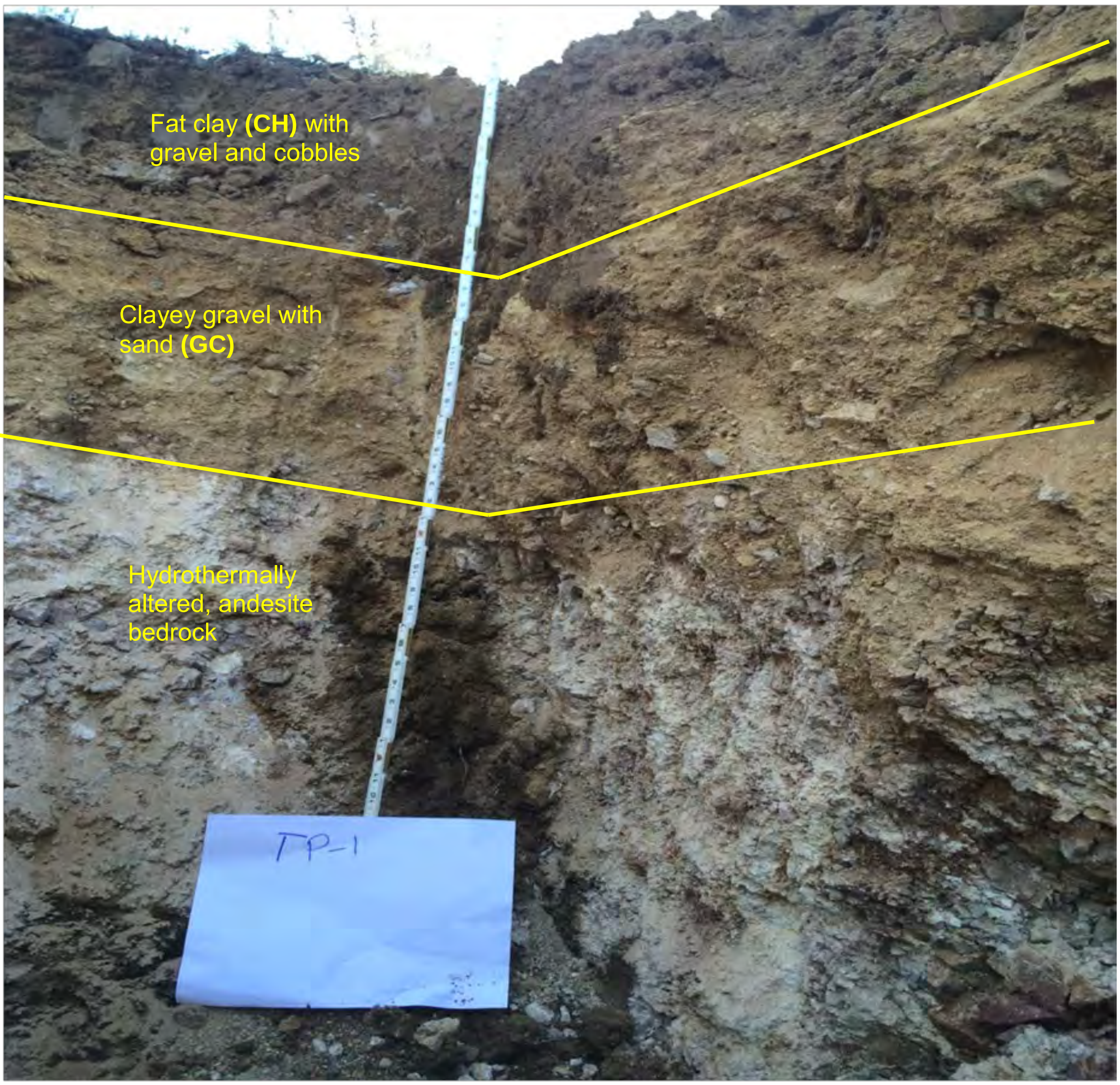

Photo #2: Showing the geotechnical profile encountered in Test Pit TP-1. Note the three different geotechnical layers encountered.

Photo #2 shows the geotechnical profile encountered in Test Pit TP-1. Two distinctive soil layers were encountered above the bedrock layer consisting of an uppermost fat clay with gravel and cobble **(CH)** layer transitioning to a clayey gravel with sand **(GC)** layer at a depth of about 1.5 feet below the existing ground surface(bgs). Bedrock was encountered at a depth of about 3 feet bgs. The clayey gravel with sand **(GC)** soil horizon was not encountered in Test Pit TP-3 located in the northeast corner of the easement area.

The hydrothermally altered andesite bedrock is generally deeply weathered, crushed to closely fractured, and soft to moderately hard. The bedrock hardness and weathering is extremely variable. In Test Pit TP-1 at a depth of about 7½ feet, the trackhoe met refusal on a silicified bedrock zone. However, directly adjacent to this silicified bedrock zone, the bedrock was deeply weathered and readily excavated. Typically, isolated hard bedrock zones will be encountered in this bedrock.

Bedrock zones consisting of soft and plastic material that have been bleached to a whitish yellow coloration will also be encountered. When excavated, these bedrock zones have a similar soil classification as a fat clay with sand **(CH)** or sandy elastic silt **(MH)**, which can experience volume changes with changes in moisture. Bedrock fractures were typically infilled with gypsum crystals. Gypsum is a sulfate mineral, which can cause deterioration to concrete.

7.0 SEISMIC CONDITIONS AND GEOLOGIC HAZARDS

The subject property is located in a moderate to intense seismically active area of the Western United States. The western region is subject to seismicity related to movement of the crustal masses (plate tectonics). 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 project site lies near the eastern base of the Sierra Nevada, within the western extreme of the Basin and Range.

7.1 Faulting

To determine the location of mapped earthquake faulting trending through or near the project site, a review of the USGS Website: *Earthquake Hazards Program Quaternary Faults in Google Earth* was completed. These maps indicate that no mapped faults trend through the project site. Several faults are mapped north of the site. The closest mapped fault trends in a near northerly direction and the southern end of the fault is located less than ½-mile north of the site near the intersection of Disc Drive and Sparks Boulevard.

Quaternary earthquake fault evaluation criterion has been formulated by a professional committee for the State of Nevada Seismic Safety Council (2006), which defines Holocene Active Faults as those with evidence of displacement within the past 10,000 years (Holocene time). Those faults with evidence of displacement during Pleistocene time (10,000 to 1,600,000 years before present) are classified as either Late Quaternary Active Fault (10,000 to 130,000 years) or Quaternary Active Fault (>130,000 years). Both of the latter fault designations are considered to have a decreased potential for activity compared to the Holocene Active Fault. An inactive fault is considered to be a fault that does not comply with these age groups. The fault closest to the site is classified as a Late Quaternary Active Fault.

7.2 Liquefaction

Liquefaction is a loss of soil shear strength that can occur during a seismic event, as cyclic shear stresses cause excessive pore water pressure between the soil grains. This phenomenon is generally limited to unconsolidated, clean to silty sand (up to 35 percent non-plastic fines) lying below the ground water table to depths up to 50 feet below the existing ground surface. The higher the ground acceleration and the longer that shaking caused by a seismic event occurs, the more likely liquefaction will take place. Severe liquefaction can result in catastrophic settlements of large civil structures.

Because the presence of bedrock, soil liquefaction potential is negligible.

8.0 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 Definitions

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

The soil/bedrock profile classification is based on two criteria: density (primarily for soils based on SPT blow count data) or 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. A 100-foot deep boring or geophysical studies such as ReMi is required to define the soil profile in sufficient detail to determine the site classification. A 100-foot boring or geophysical studies was not part of our scope of services for this project. However, because the site is in bedrock, it is our opinion that a Site Classification of C can be used for the design of this project.

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

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

Table 2 – Seismic Design Parameters

PARAMETER DESCRIPTION	Satellite Hills Booster Pump Station
Approximate Latitude of Site	39.5706
Approximate Longitude of Site	119.7239
Peak Ground Acceleration-MCE_R PGA (ASCE 7-10 Standard)	0.556 g
Design Peak Ground Acceleration-DPGA (ASCE 7-10 Standard)	0.400 g
Spectral Response Acceleration at Short period (0.2 sec.) S_s (for Site Class B)	1.503 g
Spectral Response Acceleration at 1-second Period, S_1 (for Site Class B)	0.503 g
Site Class Selected for this Site	C
Site Coefficient F_a, decimal	1.0
Site Coefficient Fv, decimal	1.3
Design Spectral Response Acceleration at Short period, S_{Ds} (Adjusted to Site Class B, SDs= 2/3 SMs)	1.002 g
Design Spectral Response Acceleration at 1-second Period, S_{D1} (Adjusted to Site Class B, SD1=2/3 SM1)	0.436 g

1) MCE_R PGA- Maximum credible earthquake geometric mean peak ground acceleration.

9.0 RECOMMENDATIONS AND DISCUSSION

Based on the results of our field investigation, laboratory testing, literature review and analysis, it is our opinion that the proposed booster pump station may be developed as planned. The following geotechnical conclusions and recommendations are provided for project design. These recommendations and conclusions may change if additional information becomes available or if the subsurface conditions vary from those encountered within the explored location shown on Plate A-2.

Because of the presence of hydrothermally altered bedrock, construction alternatives to reduce potential future structural movement will be presented. Inspection of foundation soils by our geotechnical engineer during construction is recommended to verify geotechnical conditions below the building area.

9.1 General Information

Clay soil can potentially shrink or swell (volume changes) in response to changes in the moisture content of the soil. The existing bedrock is considered a clay soil. Moisture changes in these soils can occur as a result of seasonal variations in precipitation, poor site drainage, landscape irrigation, leaking underground pipes, capillary action, or from other sources. Volume changes in clay soils can cause differential movements in structural elements constructed in the sphere of influence or bearing on the clay soil.
The construction standard in this area to reduce differential movements due to clay soil volume changes is to separate structural elements from the clay soil with a structural fill layer. The structural fill layer provides a surcharge on the clay soil, distributes any movement in the underlying clay soil over a wider area, and can reduce shrinkage (moisture loss) within the clay soil. This construction standard is generally economical and has proven to provide largely satisfactory results. However, it should be understood that constructing over clay soils is an inherent risk, and the potential for differential movement because of volume changes in the clay soils, although reduced utilizing the construction recommendations given in this report, still exists. These differential movements may necessitate future maintenance or repair of structural elements.

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

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

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

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

9.2 Site Preparation

The existing site improvements will be removed including the exterior fence and trees. Any underground lines located below structural areas should also be removed and backfilled with structural fill.

All vegetation, topsoil and existing fill, if encountered, should be stripped and grubbed from structural areas and removed from the site. The entire root bulb should be removed as part of any tree removal at the project site. Large roots (greater than 2 inches in diameter) radiating from the tree bulb area, located within one foot of the final subgrade or foundation grade elevation, should be completely removed. Resulting excavations should be backfilled with structural fill.

Because the project site is located in potentially expansive bedrock, it is recommended that prior to the placement of structural fill, the bedrock is scarified to a depth of 12 inches, thoroughly mixed, moisture conditioned, if required, to over optimum moisture content, and densified to at least 90 percent relative compaction in accordance with ASTM D 1557. The intent of this soil preparation is to mix clay seams with more granular sections of the bedrock to reduce the overall expansion potential.

It is recommended that prior to densification, the moisture content of the soils be determined to evaluate the need for moisture conditioning. After the densification process, a firm, stable surface should be produced. If unstable native soils due to excessive moisture content are encountered they should be removed and replaced with structural fill.

9.3 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 shall be free of vegetation, organic matter, and other deleterious material. Three types of fill are recommended:

- Class 1 structural fill shall be placed below foundations, concrete slabs-on-grade, and pavement areas (refer to Table 3);
- Class 2 structural fill shall be used for retaining wall backfill (refer to Table 4);
- Clay layer backfill shall be used behind the retaining wall foundation (refer to Table 5).

Class 1 and 2 structural fill shall be imported and meet the requirements provided in Tables 3 and 4. The location of all structural fills are presented in Figure 1 located on page 11.

Table 3 - Guideline Specification for Class 1 Structural Fill

Sieve Size	**Percent by Dry Weight Passing**
4 inch	100
¾ inch	70 - 100
No. 40	30 - 70
No. 200	10 - 30
Maximum Liquid Limit	**Maximum Plastic Index**
35	10
Soluble sulfates:< 0.10 percent by weight of soil	

Table 4 - Guideline Specification for Class 2 Structural Fill[1,2]

Sieve Size	Percent by Dry Weight Passing
4 inch	100
¾ inch	70 - 100
No. 40	30 - 70
No. 200	5 - 30
Maximum Liquid Limit	**Maximum Plastic Index**
35	10

Notes:
1. Minimum strength properties: Ø = 35^0 and cohesion = 100 psf (ASTM D3080).
2. Soluble sulfates:< 0.10 percent by weight of soil

Other material types not meeting the specifications given in Table 3 and 4 may be acceptable as structural fill including other combinations of strength parameters (Ø and cohesion), as presented in Table 4. These strength combinations could include higher cohesion values with lower phi angles or, inversely, higher phi angles with lower cohesion values. Combined strength parameters, different from Table 4, shall be approved by the geotechnical engineer prior to fill placement. The soil properties of all material anticipated to be used as structural fill shall be verified prior to fill placement with additional laboratory testing.

Table 5 - Guideline Specification for Clay Layer Backfill

Sieve Size	Percent by Dry Weight Passing
4 inch	100
¾ inch	70 -100
No. 40	40 - 100
No. 200	40 - 100
Minimum Plastic Index	
20	
Soluble sulfates:< 0.10 percent by weight of soil	

It is anticipated that native clay soils will meet the gradation and plasticity requirements presented in Table 5. Clay layer backfill should be tested and stockpiled.

Structural fill should be placed in maximum 8-inch thick (loose) level lifts or layers and densified to at least 90 percent relative compaction. The required moisture content of the soils, prior to densification, shall range between plus or minus 3 percent of optimum moisture, as determined by moisture-density relationship test results (ASTM D1557). Moisture contents greater than 3 percent of optimum moisture are acceptable if the soil lift is stable and required relative compaction can be attained in the soil lift and succeeding soil lifts. Grading should not be performed with frozen soils or on frozen soils.

9.3.1 Foundation Grade Soil Recommendations

Foundation grade elevation is anticipated to be 2 feet below finished floor elevation or about a maximum depth of 15 feet below existing ground elevation. Because of potential expansive bedrock, it is recommended that foundations are placed on at least 4 feet of structural fill, as presented in Figure 1. Structural fill should extend laterally from the edge of the foundation at least 3 feet.

Foundation grade soils preparation shall follow the recommendations of Section 9.2 (Site Preparation) and Section 9.3 (Grading and Filling).

9.4 Trenching and Excavation

Based on the excavation performance of the trackhoe used for the field exploration, excavations into the bedrock can be completed with conventional construction equipment, such as a large trackhoe, Caterpillar D-9 dozer or equivalent. However, more resistant, silicified zones may be encountered that may require the use of a hoe-ram or equivalent type equipment to break-up the rock.

Excavations will require shoring or the excavation sidewalls shall be sloped to maintain adequate stability. Regulations amended in Part 1926, Volume 54, Number 209 of the Federal Register (Table B-1, October 31, 1989) requires that the temporary sidewall slopes be no greater than those presented in Table 6.

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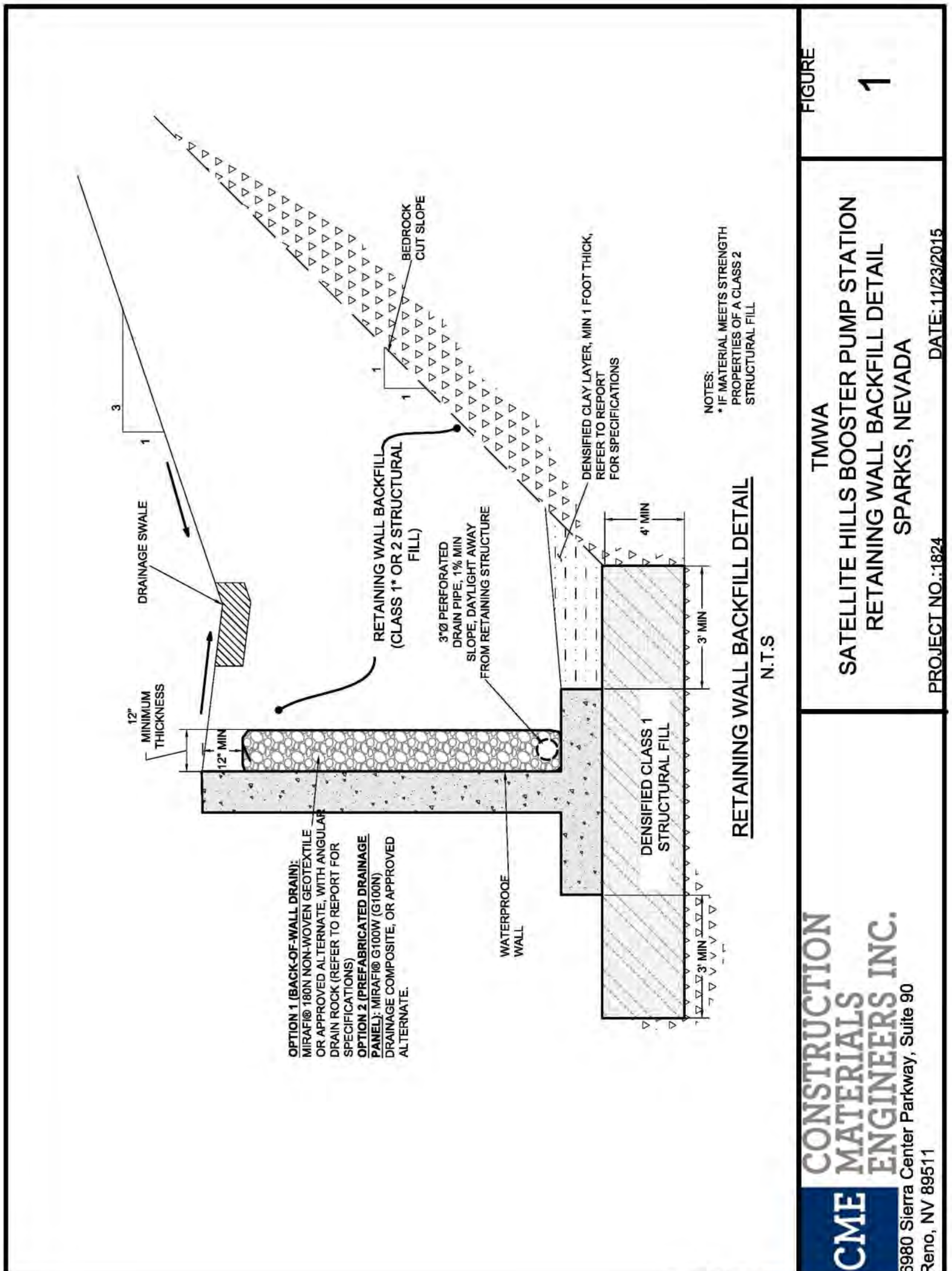


Table 6 - Maximum Allowable Temporary Slopes

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

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

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

On the basis of our exploration, it is our opinion that the majority of the geotechnical profile encountered can be classified as either a Type A or Type B soil classification, although silicified bedrock areas will classify as Stable Rock. Consequently, material types anticipated could have temporary cut slope gradients ranging from near vertical to a 1H:1V slope gradient with a maximum height of 20 feet, as determined during construction.

All trenching should be performed and stabilized in accordance with local, state, and OSHA standards. In any case bank stability will remain the responsibility of the contractor, who is present at the site, able to observe changes in ground conditions, and has control over personnel and equipment.

9.5 Foundation Design

It is recommended that shallow, spread footings be used for foundation support and is the basis for our design recommendations. Provided that foundation grade soils preparation has been performed in accordance with the recommendations contained in this report, the bearing pressures presented in Table 7 can be utilized for the design of individual column footings and continuous wall footings.

Table 7 – Foundation Allowable Bearing Pressures

Loading Conditions	Maximum Soil Net Allowable Bearing Pressures[1] (pounds per square foot)
Dead Loads plus full time live loads	3,000
Dead Loads plus live loads, plus transient wind, or seismic loads.	4,000

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

Footings shall be set at least two feet below adjacent finished grade elevation for frost protection and confinement.

Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. A friction factor of 0.40 may be utilized for sliding resistance at the base of the spread footing. A design value of 400 pounds per square foot per foot of depth is recommended for static passive soil pressures. Under seismic loading, a reduction in passive pressure will occur and a design value of 350 pounds per square foot per foot of depth is recommended.

It should be understood that some lateral deformation on the order of 2 to 4 percent of the depth of embedment (Tomlinson, 1986) for a properly compacted backfill is required to mobilize the ultimate passive resistance. To reduce the amount of displacement required to develop the design passive pressure, a factor of safety of 1.5 was applied to the passive pressure and sliding resistance from their calculated ultimate values.

In designing for passive pressure, the upper one-foot of the soil profile should not be included unless confined by a concrete slab, or pavement. Design values are based on spread footings bearing on structural fill and backfilled with structural fill.

9.5.1 Settlement

Due to the material characteristics of the structural fill and underlying bedrock, an elastic settlement response is expected, and the majority of the settlement will occur rapidly, generally during the construction time frame for the building. Settlements are based on an assumed load of 5.0 klf (kips per lineal foot) for building walls. Total static settlements are anticipated to be on the order of ½ inch, or less. Differential settlement between foundations with similar loads and sizes is anticipated to be on the order of ½ of the total settlement.

9.6 Retaining Walls

9.6.1 Design Concepts

Both static and seismic lateral earth pressures imposed on the retaining wall will be evaluated. These lateral earth pressures will be based on the slope geometry behind the wall, which will be variable. The slope geometry behind the northern and southern retaining walls (perpendicular to the retaining wall) will be relatively flat. The finished grade behind the eastern retaining wall is initially flat for a distance of 3 feet to accommodate a rock rip-rapped drainage swale before transitioning to a 3H:1V slope gradient.

Generally, cohesionless retaining wall backfill soils are specified because typical lateral soil pressure evaluation equations such as Coulomb or Rankine are based on cohesionless soil properties. However, soils with cohesive properties will dramatically reduce soil lateral pressures on the retaining wall, especially when evaluating for seismic lateral loading. Generally, most soils with some fines possess cohesive soil properties. In order to take advantage of reduced lateral soil pressures with soils possessing cohesive properties, recommended lateral soil pressures are based on wall backfill soils having soil strength properties combining both an internal friction angle (Ø) and cohesion.

9.6.2 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). Restrained retaining wall lateral earth pressure is based on the at-rest soil condition (K_o). Lateral earth pressure values for the retaining wall that is free to rotate and deflect at the top of the wall (wall movement greater than 0.001H for cohesion less soils and greater than 0.01H for cohesive soils) are based on active soil conditions (K_a). Since retaining walls will be separate from the building, it is assumed all the walls will be yielding and, therefore, designed for active conditions.

Table 8 provides lateral earth pressures for static lateral earth pressure conditions based on the assumption that the retaining wall is backfilled with granular, non-expansive soils in accordance with the recommendations presented Section 9.3 (Grading and Filling). The backfill should extend laterally behind the retaining wall at least the height of the retaining wall.

Table 8 – Static Lateral Earth Pressure Values [1,2,3]

Wall Type	Static Lateral Earth Pressures Based On Back Slope Gradients			
	level		3H:1V	
Assumes movement of wall face to allow full development of active pressures (K_a)	26	0.21[4]	32	0.25[4]

NOTES:
1) Pounds per square foot per foot of depth.
2) Surcharge loads will increase lateral earth pressure and can be given upon request.
3) Assumes backfill soils are granular with a minimum phi angle of 35^0, cohesion = 100 psf, and unit weight of 125 pcf complying with specifications provided in Section 9.3 (Grading and Filling)
4) Coefficient of Lateral Earth Pressure.

The lateral pressures presented in Table 8 assumes positive foundation drainage is provided to prevent the build-up of hydrostatic pressures and finished site drainage is provided to direct runoff away from retaining walls. To minimize hydrostatic pressures, retaining wall drainage should be constructed as an integral part of the retaining wall.

9.6.3 Retaining Wall Drainage Recommendations

Design options for retaining wall drainage are presented below:

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

- Sub-drainage can be installed at the base of the foundation behind the retaining wall. The sub drain is comprised of a slotted non-corrosive piping system bedded in drain rock. Drain rock should be encapsulated with non-woven geotextile drainage fabric (refer to Table 9), have a thickness of at least 12 inches behind the back face of the retaining wall, and extend upward behind the retaining wall to 1 foot below finish grade. Drain rock shall meet the requirements of Section 200.03 (SSPWC, 2012) for a Class C backfill. The drain pipe should be sloped to allow the gravity flow of subsurface water to discharge locations away from the retaining wall. The discharge location should be protected from clogging by appropriate means.

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

Table 9 – Drainage Geotextile Minimum Strength and Hydraulic Properties

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

Based on the required use of this geotextile, strength properties are based on Class 1 survivability rating (AASTHO M288). Products such as a Mirafi 180N, or approved equal can be utilized for this project.
Behind the edge of the foundation and extending to the native bedrock cut slope, a minimum one foot thick clay layer shall be placed overlying the foundation grade structural fill layer (refer to Figure 1). The intent of this clay layer is to minimize any infiltration of water into the underlying structural fill layer.

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

9.6.4 Seismically Induced Loading

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

- **PGA**: Design peak ground acceleration (PGA) is based on the design earthquake ground motions (2% probability in 50 years, IBC 2012).

- **k_h**: Horizontal ground acceleration component. This component is derived from the PGA, as described in this section.

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

- **P_{AE}:** Dynamic lateral earth pressure force: $P_{AE}=0.5\gamma H^2 K_{AE}$, where γ=soil unit weight and H=height of the wall. This pressure is a combination of both static and dynamic loads such that $P_{AE} = P_a + \Delta P_{ae}$, where P_a is the static lateral pressure and ΔP_{ae} is the dynamic lateral component.

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

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

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

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

The design peak ground acceleration is 0.4g, as given in Section 8.0. Since site retaining walls are assumed to be yielding, a horizontal ground acceleration values of 0.2g (yielding) was used.

Cohesion in the soil backfill can sufficiently reduce the soil pressure lateral loads. Due to limitations in the M-O Method using cohesive soils and sloped backfill geometry, earth pressure computations were completed by back calculating an applied boundary force to the wall face in general accordance with the Generalized Limitation Equilibrium (GLE) approach for determining seismic active pressures as outlined in

National Cooperative Highway Research Program (NCHRP) Report 611 (2008). The computer program Slide v6.0 (Rocscience Inc., 2014) was utilized to perform these analyses. This program performs a two dimensional limit equilibrium analysis to compute the factor of safety (FOS) using the simplified Spencer method. Table 10 (Seismically Induced Lateral Earth Pressure Values) provides seismically induced earth pressure values for both flexible and rigid walls.

Table 10– Pseudo Static Lateral Earth Pressure Values

Earth Pressure Condition	Pseudo Static Earth Pressure Coefficient		Total Wall Pressure (Seismic and Static) Equivalent Fluid Pressure[1] (psf/ft)	Component Earth Pressures[1] (psf/ft)	
	Slope	K_{ae} [2,3]	($\gamma_{soil} * K_{ae}$) [3,4]	Seismic	Static
Pseudo Static (assumes lateral wall displacement-active conditions)	Level	0.25	31	6	25
	3H:1V	0.34	43	11	32

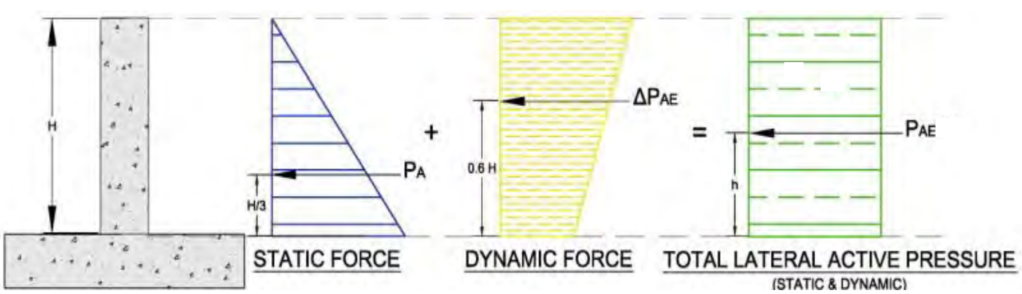
(1) Pounds per square foot per foot of depth. **<u>Total wall pressure</u>** includes both the static and seismic lateral earth pressure components. Assumed soil properties: Ø of 35^0 , soil unit weight (Y) of 125 pcf, and cohesion = 100 psf. Assumes no hydrostatic forces and no surcharge loading.

(2) Based on a design ground acceleration (PGA) of 0.4g. Walls that can mobilize active earth pressure conditions, ½ the design ground acceleration (horizontal seismic coefficient of 0.2g) is used for design. .

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

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

$$h = \frac{P_{a} * ({}^{H}/_{3}) + \Delta P_{ae} * (0.6H)}{P_{ae}}$$



For example a 13 foot tall wall with a P_a=32 psf/ft and a ΔP_{ae}=11psf/ft would have a resultant force (P_{ae}=43 psf/ft) acting at a height (h) equal to about 4.8 feet.

9.7 Site Drainage

Adequate surface drainage shall be constructed and maintained to fall away from the structure. The permanent finished slope grade away from the structure should be at least 5 percent for a minimum distance of 10 feet away from the building. The slope gradient can be reduced to 2 percent for impervious surfaces, such as concrete slabs-on-grade and pavement, constructed adjacent to the building. It is recommended that all runoff be collected within permanent drainage paths away from the structure that can convey water off the property.

Stemwall backfill shall be densified to the requirements given in Section 9.3 (Grading and Filling) to decrease permeability and reduce the potential for irrigation and storm water to enter under floor areas. This will also reduce the potential for settling of backfill soils causing a reduction in the slope gradient away from the structure.

9.8 Concrete Slabs

All concrete slabs should be directly underlain by at least 6-inches of aggregate base material consisting of a Type 2, Class B aggregate base. Base material should be densified to at least 95 percent relative compaction. It is recommended that concrete slabs-on-grade are placed on at least 2 feet structural fill not including the base course. Structural fill shall extend at least 2 feet beyond the edge of the concrete slab. Prior to placement of structural fill, native bedrock cut should be prepared in accordance with the site preparation recommendations.

The northern Nevada area is a region with low relative humidity. As a consequence, concrete flatwork is prone to excessive shrinking and curling. Concrete mix proportions and construction techniques, including the addition of excess water and improper curing, can adversely affect the finished quality of the concrete resulting in cracking, curling and spalling of slabs. We recommend that all placement and curing be performed in accordance with procedures outlined by the American Concrete Institute. 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.

9.9 Slope Stability and Erosion Control

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

Slope stability is related to mass wasting, landslides or the enmasse downward movement of soil or rock. Stability of cut and fill slopes depends upon shear strength, unit weight, moisture content, and slope angle. It our opinion that slopes (cut or fill) can be designed with a 2H:1V (horizontal to vertical) or flatter slope gradient. Temporary cut slope gradients shall be designed in accordance with recommendations provided in Section 9.4.

Erosion potential depends on numerous factors involving grain size distribution, cohesion, moisture content, slope angle and the velocity of the water or wind on the ground surface. Existing cut slopes adjacent to the project site appear to have a 2H:1V cut slope gradient and do not have erosion control or vegetation, which would likely be difficult to grow in the bedrock. These slopes appear to be stable with some surface erosion. Bedrock cut slopes for this project will likely have a similar appearance if not protected and if acceptable, erosion control is not required.

Fill slopes should be protected with erosion control and the recommendations presented in Section 9.11.1 shall be followed.

9.9.1 Erosion Control Recommendations

Slopes steeper than 3H:1V require mechanical stabilization consisting of rock rip-rap with a minimum of 75 percent of the rock rip-rap 8-inches or greater in diameter. Other methods of stabilization on slopes steeper than 3H:1V can be used if demonstrated to be as effective as mechanical stabilization. Landscape slope stabilization designed by a registered Landscape Architect may also be used on slopes steeper than 3H:1V. However, a growth medium maybe difficult to establish in the site bedrock unless topsoil is imported to the site. Slopes between 3H:1V and 5H:1V can be stabilized by hydroseeding or rock rip-rap.

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

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

9.10 Soil Corrosion Testing

Soil corrosion tests included pH, soluble sulfates, and resistivity. Except for soluble sulfates, it is recommended that these test results be reviewed by a corrosion engineer to determine soil corrosion potential. A brief summary of corrosion potential is presented below:

- **Soluble sulfates:** Soluble sulfate test results ranged from 0.16 to 1.22 percent of soil weight. These results indicate a moderate to severe sulfate exposure to concrete. Based on ACI, it is recommended that concrete have a maximum water cement ratio of 0.45 and a minimum 28 day compressive strength of 4500 psi. A Type V cement is also recommended. In accordance with ACI 318, concrete exposed to freezing and thawing in a moist condition or to deicing chemicals should consist of a mix with a maximum of 0.45 water/cementitious ratio and minimum 28-day compression strength of 4500 psi. The contractor should submit a concrete mix design to the owner at least 10 working days prior to construction for approval.

- **pH:** The pH test results range from 3.8 to 6.9, which indicates an acidic soil condition.

- **Resistivity:** Resistivity test results range from 383 to 400 ohms x cm. In general, soils with resistivity values below 3,000 are corrosive to metal pipes.

9.11 Structural Section Construction

The recommended minimum structural section is 3 inches of AC overlying 6 inches of aggregate base. This structural section is based on occasional truck traffic (1 to 2 times a week).

The following presents construction recommendations for the structural section:

- Aggregate base course shall be placed on at least 2 feet of Class 1 structural fill. Structural fill shall extend at least 2 feet beyond the edge of the pavement. Prior to placement of structural fill, native bedrock cut should be prepared in accordance with the site preparation recommendations.
- Base material should be densified to at least 95 percent relative compaction;
- Type 2 Plantmix aggregate in accordance with Section 200.02 of the referenced standard specifications for public works improvement should be utilized for the pavement. All pavement construction shall conform to the referenced standard specifications; and
- The contractor should submit a pavement mix design to the owner at least 10 working days prior to construction for approval. It is recommended that when pavement is placed adjacent to concrete flatwork, the finish compacted grade of the pavement be at least ¼ to ½ of an inch higher than the edge of adjacent concrete surface. This is to allow adequate compaction of the pavement without damaging the concrete.

9.12 Pavement Maintenance

Maintenance is **mandatory** to long-term pavement performance. Maintenance refers to any activity performed on the pavement that is intended to preserve its original service life or load-carrying capacity. Examples of maintenance activities include patching, crack or joint sealing, and seal coats. If these maintenance activities are ignored or deferred, premature failure of the pavement **will occur**.

10.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations presented in this report are based on the assumption that the owner/project manager provides sufficient field testing and construction observation by a qualified firm during all phases of construction. These construction observation and testing services should include but not be limited to site preparation and grading, concrete placement, and asphalt paving. It is recommended that since we prepared this report and have knowledge of the subsurface and surface conditions at the site, CME should be retained to provide these services. Additionally, all plans and specifications should be reviewed by the engineer responsible for this geotechnical report to determine if they have been completed in accordance with the recommendations contained herein. It is the owner's/project manager responsibility to provide the plans and specifications to the engineer.

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

11.0 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 Plate A-1 of this report.

This report has been prepared to provide information allowing the engineer to design the project. The owner/project manager is responsible for distribution of this report to all designers and contractors whose work is affected by geotechnical recommendations. In the event of changes in the design, location, or ownership of the project after presentation of this report, our recommendations should be reviewed and possibly modified by the geotechnical engineer[1]. 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[2].

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

[1] If the geotechnical engineer is not accorded the privilege of making this recommended review, he can assume no responsibility for misinterpretation or misapplication of his recommendations or their validity in the event changes have been made in the original design concept without his prior review.

[2] 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.

The following appendices are included and complete this report.

Appendix A: Field Exploration

Appendix B: Laboratory Test Results

Appendix C: USGS Seismic Design Parameters Summary Report

We trust that this report provides you with the information you require at this time. If there are any questions regarding the recommendations presented in this report, please contact our office

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

REGISTERED PROFESSIONAL ENGINEER STATE OF NEVADA
RANDAL A. REYNOLDS
4/14/16
CIVIL
No. 8028
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Enclosures
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References

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American Concrete Institute (ACI), 2012, *Concrete Quality and Field Practices.*

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

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Nevada Earthquake Safety Council, 2006, *Guidelines for Evaluating Potential Surface fault Rupture /land Subsidence Hazards in Nevada.*

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

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

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

CME

CONSTRUCTION MATERIALS ENGINEERS, INC.

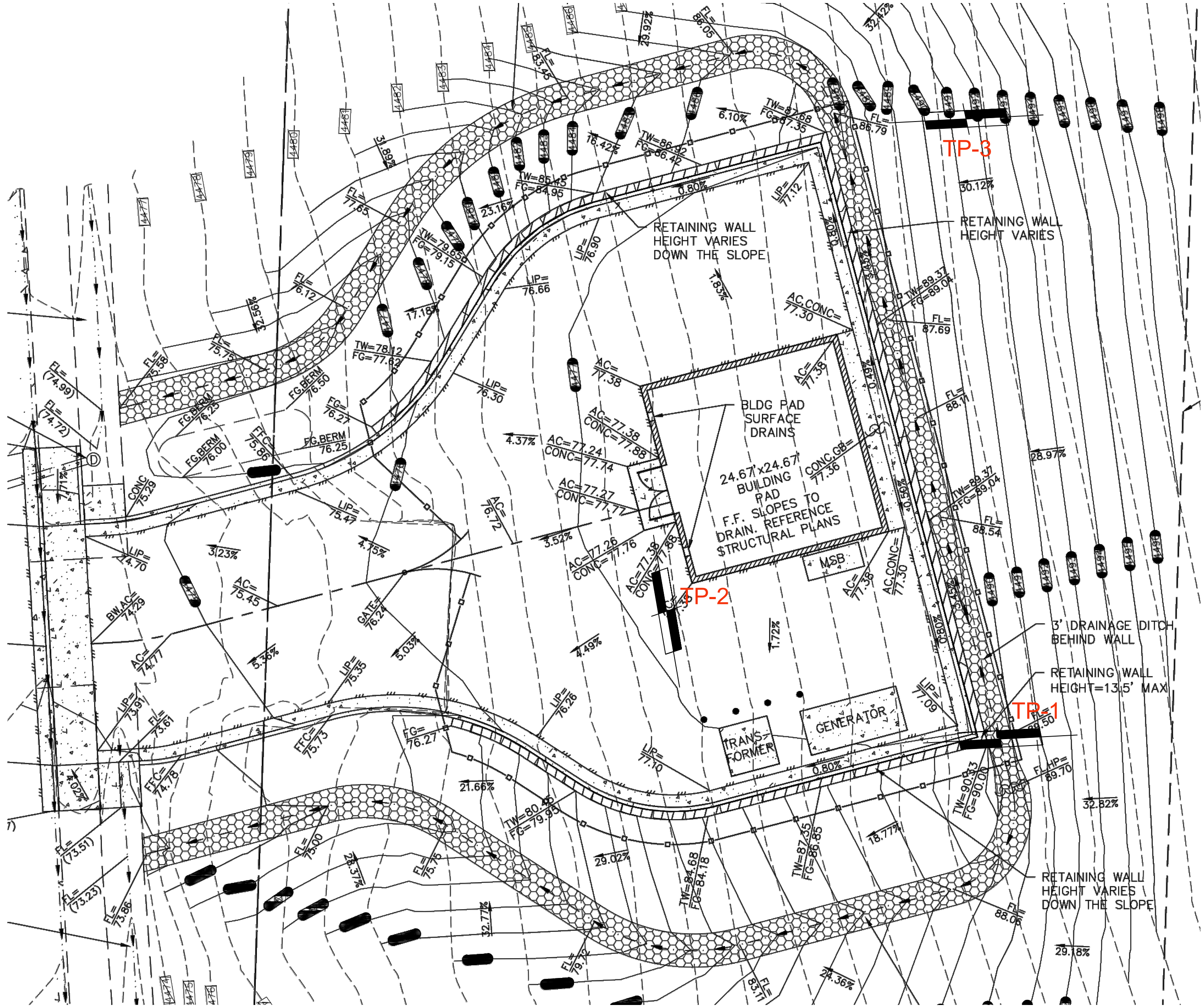
APPENDIX A

SCALE 1"~20 '

20 10 0 10 20

LEGEND

TP-1 APPROXIMATE TEST PIT LOCATION, NOTE ELONGATION OF TEST PIT DUE TO DEPTH OF EXPLORATION. LIMITS OF TEST PIT EXCAVATION SHOULD BE LOCATED PRIOR TO THE PLACEMENT OF STRUCTURAL ELEMENTS



REFERENCE BASE MAP PRELIMINARY GRADING PLAN, SATELLITE HILLS BOOSTER PUMP STATION,. RELOCATION IMPROVEMENTS, SHEET C2, LUMOS, 10-15-15

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

TMWA
SATELLITE HILLS BOOSTER PUMP STATION
FIELD EXPLORATION LOCATION MAP
SPARKS, NEVADA

PROJECT NO.: 1824

DATE: 2/11/16 (REVISED)

PLATE

A-1

LOG OF TEST PIT NO. TP-1

PROJECT SATELLITE HILLS BOOSTER PUMP STATION **EQUIPMENT TYPE** EXCAVATOR CASE 9020
CLIENT TMWA
LOCATION APPROX. 40' SOUTH AND 30' E. OF THE TRANSFORMER ENCLOSURE
PROJECT NO. 1824 **DATE** 11/16/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** 4,491'± (PLATE A-1)

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	CH			B	1A		MOIST	0-1½': FAT CLAY WITH GRAVEL, little subangular gravel, yellow brown							
2	GC			B	1B		MOIST	1½'-3': CLAYEY GRAVEL WITH SAND, little subangular gravel, few subangular cobbles up to 8" nominal diameter, little fine to medium sands, yellow brown							
4	ROCK			B	1C		MOIST	3'-15':WEATHERED ANDESITE (ALTA FFORMATION), intensely fractured, moderately hard, weakly to moderately strong, deep to moderately weathered, pale yellow-brown Note: -Interbeds of gypsiferous crystals							
6															
8				B	1D										
10	MH			B	1E		MOIST	-Interbedded gypsiferous crystal lenses approximately 1 inch thick, hard layer of andesite encountered at 7½' on the east side of the test pit Excavates similar to a Sandy Elastic Silt (MH)	50.8	64	30			13.2	A, G
12								-Significant pocket of gypsiferous crystals visible interbedded with moderately hard andesite bedrock							

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-1

PROJECT SATELLITE HILLS BOOSTER PUMP STATION **EQUIPMENT TYPE** EXCAVATOR CASE 9020
CLIENT TMWA
LOCATION APPROX. 40' SOUTH AND 30' E. OF THE TRANSFORMER ENCLOSURE
PROJECT NO. 1824 **DATE** 11/16/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** 4,491'± (PLATE A-1)

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
14				B	1F			-Less visible gypsiferous materials, mostly intensely fractured, moderately hard andesite bedrock							
								TERMINATED AT 15 FEET, NO FREE WATER ENCOUNTERED							
16															
18															
20															
22															
24															
26															

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 SATELLITE HILLS BOOSTER PUMP STATION **EQUIPMENT TYPE** EXCAVATOR CASE 9020
CLIENT TMWA
LOCATION APPROX. 10' SOUTH AND 26' E. OF THE NE COR. OF THE TRANSFORMER ENCLOSURE
PROJECT NO. 1824 **DATE** 11/16/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** 4,486'± (PLATE A-1)

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	CH						MOIST	0-2': FAT CLAY , few subangular gravels, trace fine sand, yellow brown							
				B	2A										
2	GC						MOIST	2'-3': CLAYEY GRAVEL WITH SAND, some subangular gravel, little fine to medium sands, yellow brown							
	ROCK						MOIST	3'-16':WEATHERED ANDESITE (ALTA FORMATION), crushed, friable to plastic, weak, deeply weathered, pale yellow-brown							
4								Note:							
				B	2B			-Interbedded gypsiferous crystals throughout profile							
6															
8	MH							-Altered andesite-greenish yellow, deeply weathered, plastic, tiny root visible, large gypsiferous lense at 9 feet, approximately ½-inch thick, excavates similar to an Elastic Silt (MH)							
				B	2C				88.8	81	41			22.7	A, G
10															
								-Intensely fractured, moderately weathered, friable							
12															
								-Crushed, plastic, deeply weathered							

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-2

PROJECT SATELLITE HILLS BOOSTER PUMP STATION **EQUIPMENT TYPE** EXCAVATOR CASE 9020
CLIENT TMWA
LOCATION APPROX. 10' SOUTH AND 26' E. OF THE NE COR. OF THE TRANSFORMER ENCLOSURE
PROJECT NO. 1824 **DATE** 11/16/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** 4,486'± (PLATE A-1)

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
14															
16								TERMINATED AT 16 FEET, NO FREE WATER ENCOUNTERED							
18															
20															
22															
24															
26															

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 SATELLITE HILLS BOOSTER PUMP STATION **EQUIPMENT TYPE** EXCAVATOR CASE 9020
CLIENT TMWA
LOCATION APPROX. 40' N. AND 60' E. OF THE NE COR. OF THE TRANSFORMER ENCLOSURE
PROJECT NO. 1824 **DATE** 11/16/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** 4,493'± (PLATE A-1)

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	CH						MOIST	0-3: <u>FAT CLAY</u> ,trace fine to medium sand,yellow brown							
				B	3A										
2															
3	ROCK						MOIST	3'-16':<u>WEATHERED ANDESITE (ALTA FORMATION)</u>, crushed, friable, weak to moderately hard, deeply weathered, pale yellow-brown Note: -Interbedded gypsiferous crystals throughout profile							
4															
				B	3B										
6															
8															
				B	3C										
10															
								-Intensely fractured to crushed, moderately weathered, friable, excavates similar to an Elastic Silt (MH)							
12															

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-3

PROJECT SATELLITE HILLS BOOSTER PUMP STATION **EQUIPMENT TYPE** EXCAVATOR CASE 9020
CLIENT TMWA
LOCATION APPROX. 40' N. AND 60' E. OF THE NE COR. OF THE TRANSFORMER ENCLOSURE
PROJECT NO. 1824 **DATE** 11/16/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** 4,493'± (PLATE A-1)

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
14				B	4D										
								TERMINATED AT 15 FEET, NO FREE WATER ENCOUNTERED							
16															
18															
20															
22															
24															
26															

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.

UNIFIED SOIL CLASSIFICATION CHART

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

GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)	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
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)	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.

V:\Active\1824\drawings\SOIL CLASSIFICATION CHART PLATE 3A.dwg

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6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

TMWA
SATELLITE HILLS BOOSTER PUMP STATION
SOIL CLASSIFICATION CHART
SPARKS, NEVADA

PROJECT NO.:1824 DATE: 12/1/15

PLATE
A-3a

PHYSICAL PROPERTIES CRITERIA FOR ROCK DESCRIPTIONS

BEDDING OF SEDIMENTARY ROCKS

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

FRACTURING

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

HARDNESS

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

STRENGTH

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

WEATHERING

D. DEEPLY=MODERATE TO COMPLETE MINERAL DECOMPOSITION; EXTENSIVE DISINTEGRATION; DEEP AND THOROUGH DISCOLORATION, MANY FRACTURES; THE EXTERNAL APPEARANCE IS THAT OF A SOIL, INTERNALLY THE ROCK TEXTURE IS PARTLY PRESERVED, BUT THE GRAINS HAVE BEEN COMPLETELY SEPARATED	**M. MODERATE=**SLIGHT CHANGE OR PARTIAL DECOMPOSITION OF MINERALS; LITTLE DISINTEGRATION; CEMENTATION LITTLE TO UNAFFECTED. MODERATE TO OCCASIONALLY INTENSE DISCOLORATION. MODERATELY COATED FEATURES.
S. SLIGHTLY= NO MEGASCOPIC DECOMPOSITION OF MINERALS; LITTLE OR NO EFFECT ON NORMAL CEMETATION. SLIGHT AND INTERMITTENT, OR LOCALIZED DISCOLORATION. FEW STAINS ON FRACTURED SURFACES.	**F. FRESH=**UNAFFECTED BY WEATHERING AGENTS. NO DISINTEGRATION OR DISCOLORATION. FRACTURES USUALLY LESS NUMEROUS THAN JOINTS.

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TMWA
SATELLITE HILLS BOOSTER PUMP STATION
ROCK DESCRIPTIONS
SPARKS, NEVADA

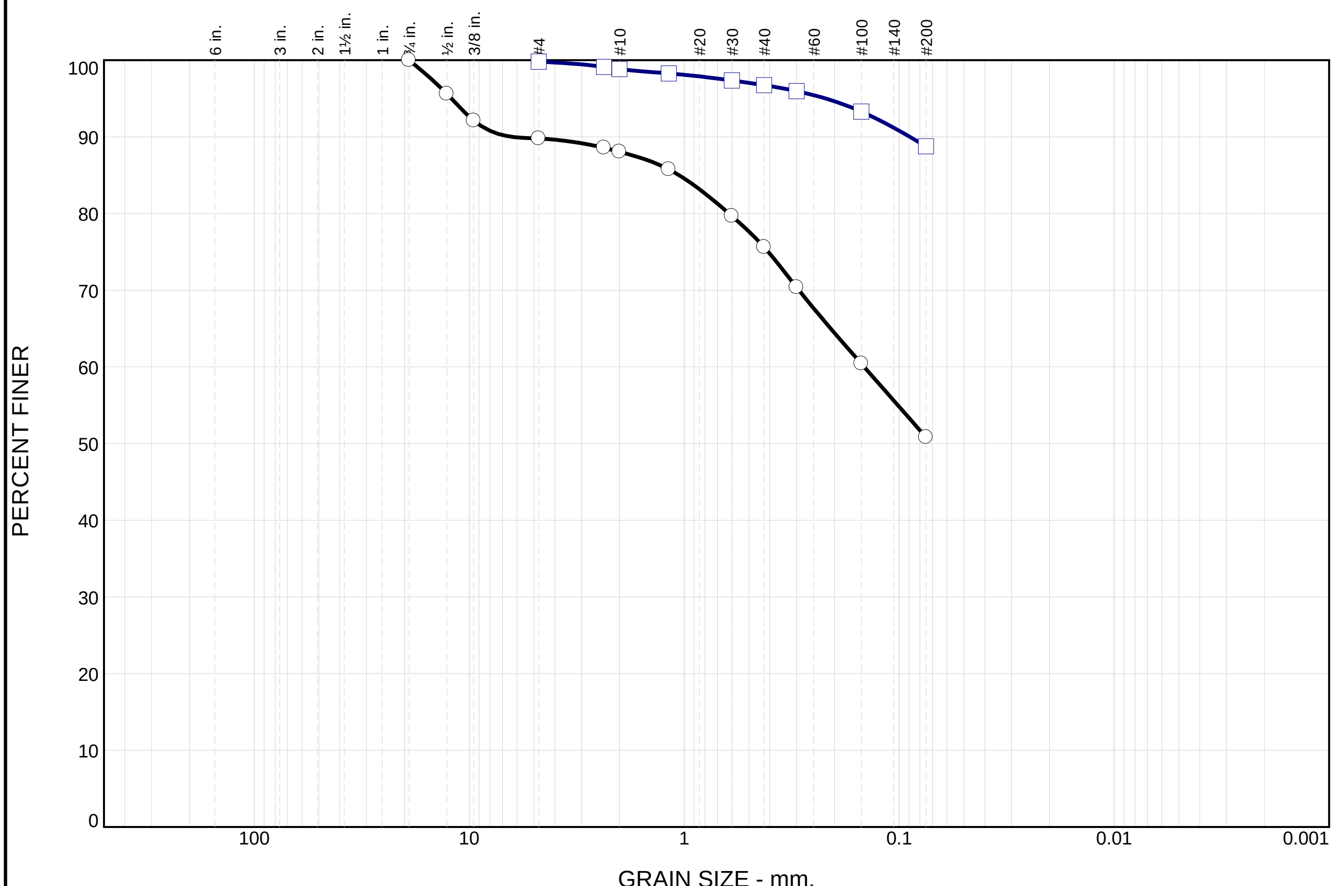
PROJECT NO.:1824 DATE: 12/1/15

PLATE
A-3b

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX B

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	10.2	1.7	12.5	24.8	50.8	
□				1.0	2.1	7.9	88.8	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	64	34	1.0559	0.1455						
□	81	40								

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ SANDY ELASTIC SILT		MH	13.2
□ ELASTIC SILT		MH	22.7

Project No. 1824 **Client:** TMWA

Project: SATELLITE HILLS BOOSTER PUMP STATION

○ **Source of Sample:** TP-1 **Depth:** 9 **Sample Number:** 1E

□ **Source of Sample:** TP-2 **Depth:** 8 **Sample Number:** 2C

CME

Remarks:

Plate B-1

Tested By: MP **Checked By:** MP

SilverState Analytical Laboratories

LABORATORY REPORT

Date: November 20, 2015

LABORATORY NO: R15-0543

CLIENT: Construction Materials Engineers, INC
6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

PAGE: 1 of 1

CLIENT PROJECT: 30002

CLIENT PO #: 1824

Sampled By: Client
Date Sampled: --
Time Sampled: --

Submitted by: Client
Date Received: 11/17/15
Time Received: 1640

Report Attention:

Sample ID	Parameter	Result	Unit	MRL	Method	Date Analyzed	Analyst
TP-1, 1D	Water Soluble Sulfates	1.22	%	0.02	ASTM 1580C	11/18/15	LB
9'-10'	Resistivity	401.4	Ω•cm	---	ASTM G-57	11/19/15	LB
	pH	6.86	S.U.	---	EPA9045D	11/18/15	LB
TP-2, 2C	Water Soluble Sulfates	0.16	%	0.02	ASTM 1580C	11/18/15	LB
8'-9'	Resistivity	384.6	Ω•cm	---	ASTM G-57	11/19/15	LB
	pH	3.80	S.U.	---	EPA9045D	11/18/15	LB

ND: Non Detect
MRL: Method Reporting Limit
EPA Flags: None

REVIEWED BY: Carlyreel signing for
John Sloan
Laboratory Director
EPA: NV00930 (SSAL-LV)
EPA: NV00931 (SSAL-Reno)

V:\Active\1824\drawings\SOIL CLASSIFICATION CHART PLATE 3A.dwg

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Reno, NV 89511

TMWA
SATELLITE HILLS BOOSTER PUMP STATION
CORROSION TEST RESULTS
SPARKS, NEVADA

PROJECT NO.:1824
DATE: 12/1/15

PLATE
B-2

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX C

USGS Design Maps Summary Report

User–Specified Input

Report Title Satellite Hills Booster Pump Station
Mon February 22, 2016 17:23:25 UTC

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

Site Coordinates 39.5706°N, 119.7239°W

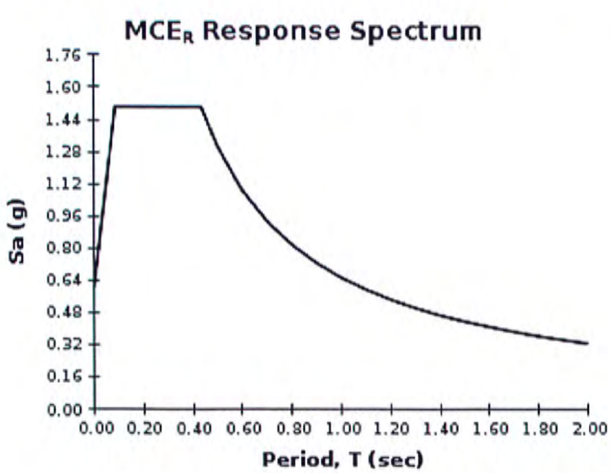
Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

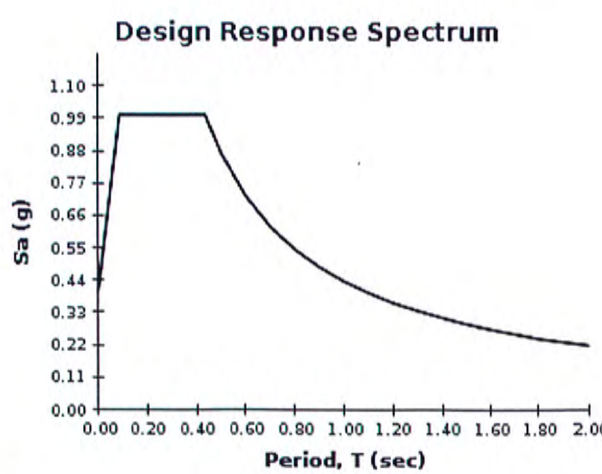
Risk Category I/II/III

USGS–Provided Output

$\mathbf{S_s} =$	1.503 g	$\mathbf{S_{MS}} =$	1.503 g	$\mathbf{S_{DS}} =$	1.002 g
$\mathbf{S_1} =$	0.503 g	$\mathbf{S_{M1}} =$	0.654 g	$\mathbf{S_{D1}} =$	0.436 g

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





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

USGS Design Maps Detailed Report

ASCE 7-10 Standard (39.5706°N, 119.7239°W)

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

Section 11.4.1 — Mapped Acceleration Parameters

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

From Figure 22-1 [1]	$S_S = 1.503$ g
From Figure 22-2 [2]	$S_1 = 0.503$ 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	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

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

For Site Class = C and S_S = 1.503 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	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 = 0.503 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.503 = 1.503$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 0.503 = 0.654$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

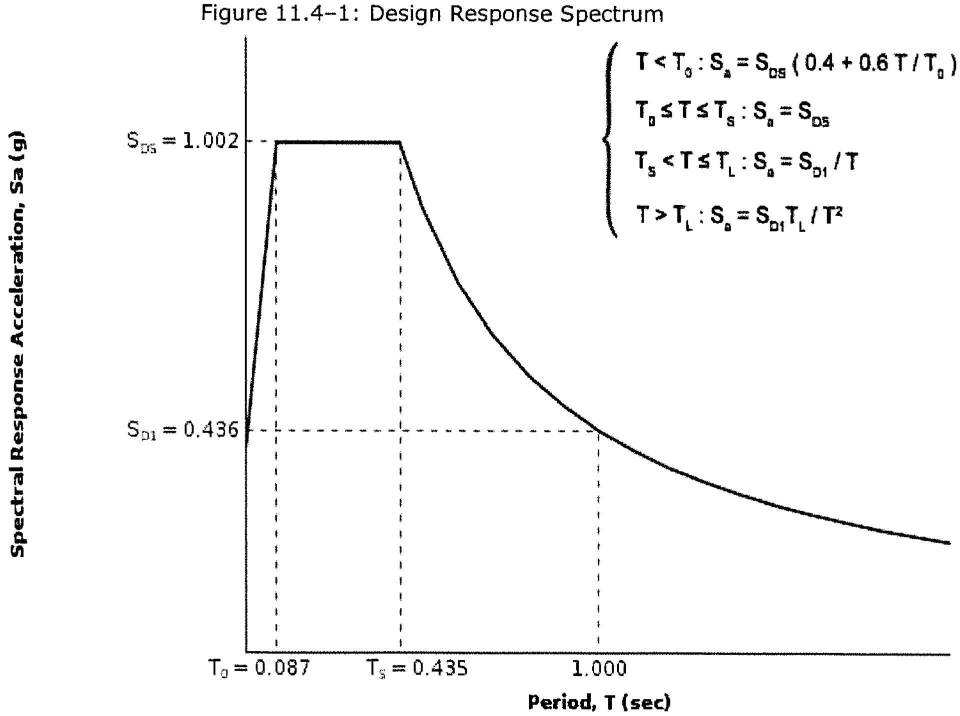
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.503 = 1.002$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.654 = 0.436$ g

Section 11.4.5 — Design Response Spectrum

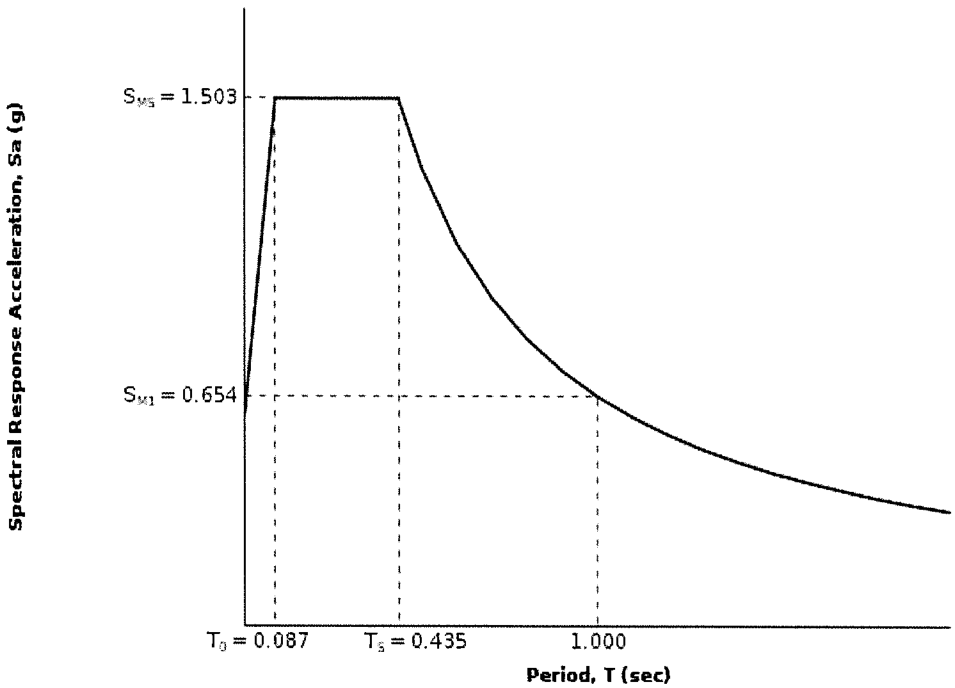
From Figure 22-12 [3] $T_L = 6$ seconds

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 = 0.556

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.556 = 0.556$ 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	$\boxed{1.0}$
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 PGA

For Site Class = C and PGA = 0.556 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} = 0.951

From Figure 22-18 [6] C_{R1} = 0.947

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} = 1.002 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} = 0.436 g, Seismic Design Category = D

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

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

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

References

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