ADDENDUM #1
PAH RAH PARK PRE-MANUFACTURED RESTROOM BUILDING
BID #16/17-004
BIDS DUE NO LATER THAN: 1:45 PM ON SEPTEMBER 8, 2016
PUBLIC BID OPENING: 2:00 PM ON SEPTEMBER 8, 2016

This addendum is to notify all potential proposers of clarifications made to the Bid documents as stated below.

The Workscope has been revised as follows to reflect supplier provide utilities to the Point of Connection as outlined in the specifications and coordinate with the City's general site work contractor who will make the final connections.

1. Workscope: The work performed under this contract consist of, but is not limited to: all material, labor, tools, expendable equipment, utility, and transportation for the construction (offsite) and installation of a prefabricated restroom located at Pah Rah Park, 1750 Shadow Ln, Sparks, NV. The vendor will furnish necessary utilities to the point of connection and be required to coordinate with the City's general site work contractor. The site is within the City limits of the City of Sparks, Washoe County, Nevada, and is more specifically designated in the plans for this project.
2. Attached is the Geotechnical Report prepared by Construction Materials Engineering for the supplier to be able to develop the footing design.

Please note and adjust your bid according to the revisions, additions, deletions, clarifications or modifications as presented on this Addendum #1, which are made a part of this bid. NOTE: To avoid disqualification, this Addendum 1 (and any other addenda) must be signed by an authorized representative of the bidding firm in the space provided and must be submitted with your firm's sealed proposal. Failure to return this addendum, duly signed, may be cause for rejection of the bid. ALL ADDENDA SHOULD BE SIGNED AND PLACED IN SEQUENTIAL ORDER AND ATTACHED TO THE FRONT OF THE BID PACKAGE, COMPLETE WITH ALL REQUIRED DOCUMENTS.

____
CONTRACTOR BUSINESS NAME

X________________________
Authorized Signature

____
Printed Name of Person Signing

____
Dan Marran, C.P.M., CPPO
Contracts and Risk Manager

September 1, 2016

GEOTECHNICAL INVESTIGATION
PAH RAH PARK REST ROOM
SPARKS, WASHOE COUNTY, NEVADA

PREPARED FOR:

CITY OF SPARKS

AUGUST 2016
FILE: 1902

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

August 26, 2016
Project No: 1902

Chris Cobb, PE
CITY OF SPARKS
Public Works Department
431 Prater Way
Sparks, NV 89431

RE: Geotechnical Investigation
Pah Rah Park Restroom
Sparks, Nevada

Dear Mr. Cobb:

Enclosed is our geotechnical investigation for the proposed new restroom and storage building at Pah Rah Mountain Park.

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

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

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

RANDAL A. REYNOLDS
8/26/16
CIVIL
No. 8028
REGISTERED PROFESSIONAL ENGINEER - STATE OF NEVADA
Exp. 12-31-17

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

RAR:rar:jwl
V:\Active\1902\report\final\C'vr ltr.docx

TABLE OF CONTENTS

FIGURES

TABLE OF CONTENTS (CONTINUED)

TABLES

PHOTOGRAPHS

APPENDICES

GEOTECHNICAL INVESTIGATION

Pah Rah Mountain Park Restroom
Washoe County, Nevada

1.0 INTRODUCTION

Presented herein are the results of Construction Materials Engineers Inc. (CME) geotechnical exploration, laboratory testing, and associated geotechnical design recommendations for the Pah Rah Park prefabricated restroom and storage building to be located in Sparks, Nevada. These recommendations are based on surface and subsurface conditions encountered during our field exploration, and on details of the proposed project as described in this report. The objectives of this study were to:

1. Investigate general soil and ground water conditions pertaining to design and construction of the proposed project.

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

Our geotechnical study included subsurface field exploration, laboratory testing, and engineering analysis to provide recommendations for project design. The area covered by this report is shown on Plate A-1 (Field Exploration Location Map) in Appendix A. Results of our field exploration and testing programs form the basis for all conclusions and recommendations.

2.0 PROJECT DESCRIPTION

It is understood that a new prefabricated restroom/storage building will replace an older existing restroom on the north side of Pah Rah Mountain Park. The proposed project includes:

- A single-story, 23 feet by 23 feet (529 square feet), masonry block restroom/storage building with slab-on-grade flooring, supported by shallow, spread foundations;
- Appurtenant construction may include improvements to the existing concrete pavement, curb and gutters, and underground utilities.

It is understood that the building finished pad grade elevation will be near the existing restroom pad grade. Building structural loads are assumed to be light.

3.0 SITE CONDITIONS

Pah Rah Park is located west of the intersection of South Los Altos Parkway and Vista Boulevard in Sparks, Nevada. The proposed restroom and storage building will be located along the north side of Pah Rah Park near the west end of the existing parking lot. The site is located in Section 26, Township 20N, Range 20E (M.D.M), in Washoe County, Nevada. The restroom will be located on APN 030-550-07. A general project vicinity map is show below as Figure 1 (Vicinity Map).

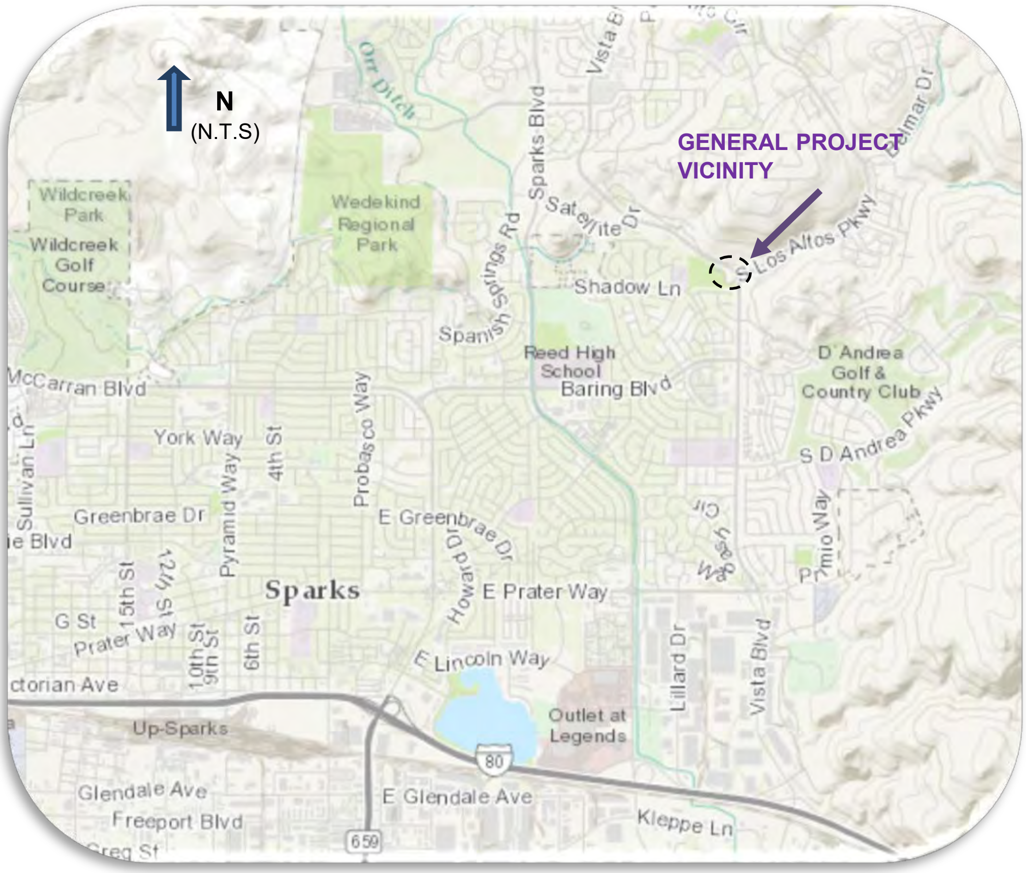


Figure 1: Vicinity Map

(Washoe County Technology Services-Regional Services Division (GIS) www.washoecounty.us/gis)

The project site is currently developed.   Existing site improvements include:

- A single-story masonry block restroom with landscaping to the south and east of the building (refer to Photo #1);
- A playground and picnic and barbeque area to the northeast;
- Sidewalks, curb and gutter; and
- Subsurface utilities.

The project site is bound by Vista Blvd to the north, a round-a-bout and parking lot to the east, an open playground picnic area to the west, and a grass field to the south. The project site is predominately paved with small landscaped areas located on the southeast and northeast boundaries. Site topography is gently sloping to the southwest.

Photo 1: Looking west from parking lot towards existing restroom

4.0 FIELD EXPLORATION

4.1 Test Pit Exploration

Subsurface field exploration, completed in August 2016, consisted of excavating 2 test pits to depths of 2½ and 8 feet below existing grade (bgs). Test pits were excavated using a John Deere 310 SG rubber-tired backhoe equipped with an 18 inch bucket. The test pit excavated to 2½ feet was terminated at a shallow depth due to encountering a water line.

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 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[1].

[1] **Warning:** 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.

Test pit logs are included as Plates A-2. Elevations shown on the test pit logs were obtained from Google Earth. 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.

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);
- Atterberg Limits (ASTM D 4318);
- Moisture-density relationship tests (ASTM D 1557);
- 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.

6.0 SUBSURFACE SOILS AND GROUNDWATER CONDITION

6.1 Subsurface Soils

Based on a review of the Vista Quadrangle Geologic Map (Bell & Bonham, 1987), the site is underlain by Quaternary aged alluvial deposits. These deposits are described as alluvial-fan deposits of the Virginia and Pah Rah Ranges consisting of brown silty sand and pebbly medium sand. Soils encountered during the subsurface exploration appear to be consistent with the mapped soil conditions.

In general, the soil profile encountered in Test Pit TP-1 consisted predominately of clayey sand **(SC)** to the maximum depth of exploration. The soils profile encountered in Test Pit TP-2 consisted of an uppermost horizon of poorly graded sand with silt (**SP-SM)** to a depth of 3½ feet bgs. This soil stratum appeared to have a loose relative density and are likely fill soils.

The uppermost horizon was underlain by poorly graded sand with silt (**SP-SM**) containing interbeds of moderately strong cemented (calcareous materials) with a light grey color to the maximum depth of exploration. In general, this soil horizon appeared to be indurated generally by cementation.

PHOTOGRAPH 2: Sidewall of TP-2. Note vegetation and root depths.

6.2 Groundwater and Soil Moisture Conditions

Groundwater was not encountered during the subsurface exploration. In general, soils were encountered in a moist to very moist condition. The NRCS Web Soil Survey maps the water table at a depth greater than 80 inches. It is anticipated that groundwater will not be encountered during construction.

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.

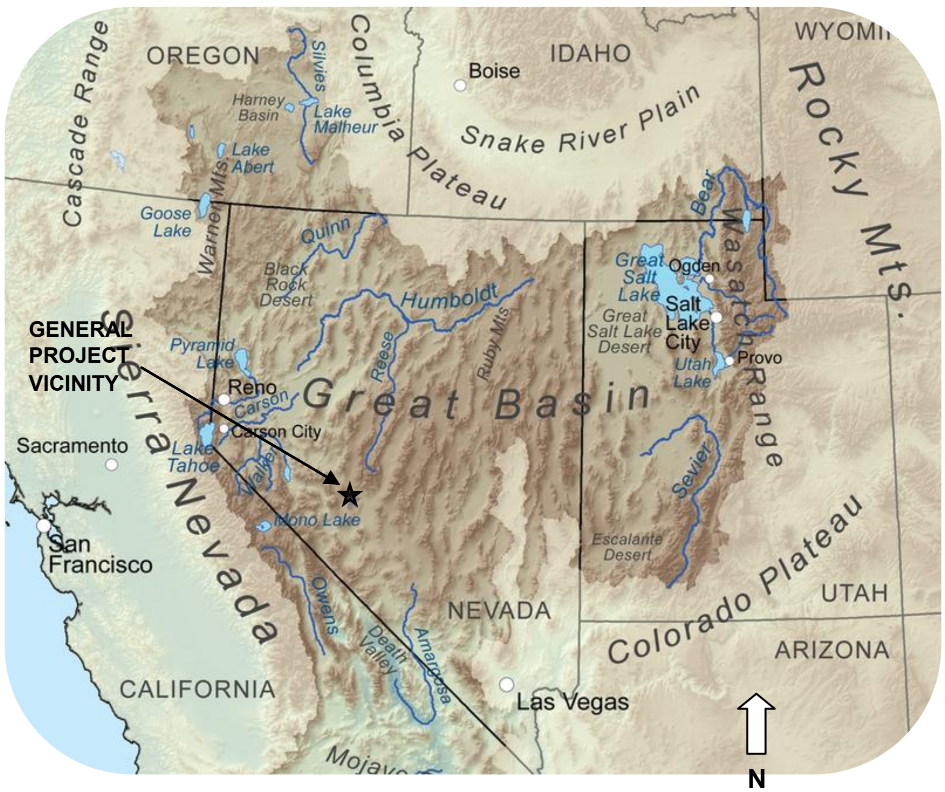


Figure 2: Overview Map Showing the Great Basin (N.T.S)

(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. The eastern front of the Sierra Nevada Mountains forms 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.

7.2 Faults

The Nevada Bureau of Mines and Geology (NBMG) geographical information system was referenced to determine existing fault traces at or near the project site. The NBMG map indicates no published fault traces through the project site. Several fault traces are located within a one-mile radius of the project site. The closest fault trace is located < ½ of a mile south of the project site and is associated with an unnamed group of lineaments east of Reno.

Quaternary earthquake fault evaluation criterion has been formulated by a professional committee for the State of Nevada Earthquake Safety Council (1996 revised 1998). These guidelines are consistent with the State of California Alquist-Priolo Act of 1972, which defines Holocene Active Faults as those with evidence of displacement within the past 10,000 years (Holocene time). Those faults with evidence of displacement during Pleistocene time (10,000 to 1,600,000 years before present) are classified as either late Quaternary Active Fault (10,000 to 130,000 years) or Quaternary Active Fault (>130,000 years). Both of the latter fault designations are considered to have a decreased potential for activity compared to the Holocene Active Fault. An inactive fault is considered a fault that does not comply with these age groups.

Based on these definitions, the fault closest to the site is classified as Quaternary Active.

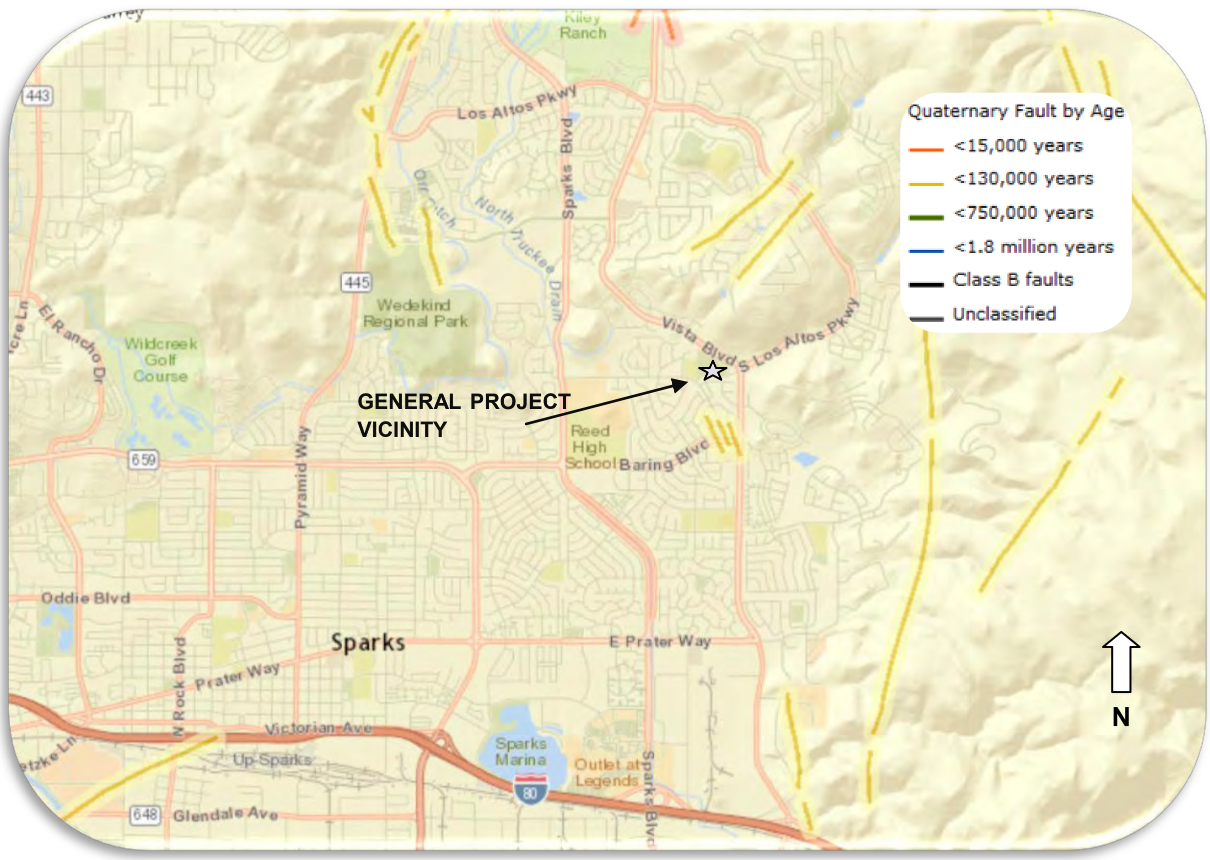


Figure 3: Excerpt from NBMG Interactive Fault Map
(NBMG Quaternary Faults in Nevada, available at https://gisweb.unr.edu/flexviewers/quaternary_faults/)

7.3 Liquefaction and Lateral Displacement

Liquefaction is nearly a complete loss of soil shear strength that can occur during an earthquake, as cyclic shear stresses generate excessive pore water pressure between the soil grains. The higher the ground acceleration caused by a seismic event or the longer the duration of shaking, the more likely liquefaction will occur.

The soil types most susceptible to liquefaction are loose to medium dense cohesionless sands, soft to stiff non-plastic to low plastic silts, or any combination of silt-sand mixtures lying below the groundwater table. Liquefaction is generally limited to depths of 50 feet or less below the existing ground surface.

Based on anticipated ground water depth and our knowledge of soil conditions at the site including densely cemented soils, it is in our opinion that the potential for liquefaction is low.

7.4 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: relative density (primarily for soils based on either SPT blow count data or shear wave velocity) or hardness (based on shear wave velocity primarily for bedrock sites). These two criteria have to be determined to a depth of 100 feet below the ground surface. A 100-foot deep boring or geophysical methods are required to characterize the soil profile in sufficient detail to determine the site classification. If neither of these field exploration methods are performed, the IBC allows the use of a default site classification of D if other geologic conditions do not exist that would justify a lower site classification (E or F). Based on our field exploration and knowledge of the geologic conditions, it is our opinion that a default Site Classification of D is appropriate to use in the design of the structures.

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 D. Copies of the USGS Design Map Summary Reports are included as Appendix C.

Table 2 – Seismic Design Parameters	
Parameter Description	**Parameter**
Approximate Latitude of Site	39.5649
Approximate Longitude of Site	119.7079
Peak Ground Acceleration-MCE_R PGA (ASCE 7-10 Standard)	0.560 g
Design Peak Ground Acceleration-DPGA (ASCE 7-10 Standard)	0.373 g
Spectral Response Acceleration at Short period (0.2 sec.) S_s (for Site Class B)	1.509 g
Spectral Response Acceleration at 1-second Period, S_1 (for Site Class B)	0.504 g
Site Class Selected for this Site	D
Site Coefficient F_a, decimal	1.0
Site Coefficient Fv, decimal	1.5
Design Spectral Response Acceleration at Short period, S_{Ds} (Adjusted to Site Class B, SDs= 2/3 SMs)	1.006 g
Design Spectral Response Acceleration at 1-second Period, S_{D1} (Adjusted to Site Class B, SD1=2/3 SM1)	0.504 g
1) MCE_R PGA- Maximum credible earthquake geometric mean peak ground acceleration.	

8.0 DISCUSSION AND RECOMMENDATIONS

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.

The primary soil constraint encountered is the presence of near surface granular soils, as encountered in Test Pit TP-2, that may be fill soils. These soils appear to have a loose relative density and are recommended to be removed below structural areas and replaced with structural fill. The removed soils, free of organics or other detrimental material, could be used as structural fill.

8.1 General Information

The recommendations provided herein, and particularly under **Site Preparation, Grading and Filling, Foundation Design, Site Drainage** and **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 planned 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 structural integrity/performance of the planned structure and related improvements could be affected. 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[2];
- 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[3].
- 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.

---

[2] 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.

[3] Clay soil is technically defined as a soil, where more than 10 percent of the soil particles are less than 5 micrometers in size (ASTM D422), having a plasticity index equal to or greater than 15. A hydrometer test is required to determine the percentage of soil particles less than 5 micrometers in size, in the absence of hydrometer testing, an alternative classification method for clay soil is based on the percentage of fines passing the number 200 sieve (#200).

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.

Test pits were excavated by a Deere 310 SG rubber-tire backhoe with an 18 inch bucket 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.

8.2 Site Preparation

The existing bathroom structure and other site improvements including concrete slabs-on-grade are located within the footprint of the new bathroom structure and will be removed. Existing foundations, floor slabs, and utilities shall be completely removed below the new building footprint.

All vegetation and topsoil should be stripped and grubbed from structural areas and removed from the site. It is anticipated that stripping and grubbing depths on the order of 8 to 12 inches will be required. Localized areas may require increased stripping and grubbing depths and depends on the depth of organic material.

Several mature landscape trees are located along the south and southwest perimeter of the site. The entire root bulb should be removed during grubbing of existing trees and tree root zones. Larger roots (greater than 2 inches in diameter) radiating from the tree bulb, located within one foot of the final subgrade or foundation grade elevation, should be completely removed. Resulting excavations should be backfilled with structural fill. Any other vegetation encountered during the construction process should be stripped/grubbed and removed from the project site or used as topsoil in non-structural areas.

It is recommended that the uppermost granular soil horizon, as encountered in Test Pit TP-2, be removed below structural areas. This soil horizon had a depth of about 3½ feet below existing grade. The lateral extent of this soil horizon is unknown, but is likely located throughout the existing unimproved area north of the existing bathroom. Additional test pits and site soil observations will be required during construction to identify the limits of this soil horizon recommended to be removed.

Subgrade soils should be densified to at least 90 percent relative compaction for a minimum depth of 12 inches. Soils should have moisture contents of plus or minus 3 percent of optimum moisture (ASTM D1557) prior to densification. Higher moisture contents will be acceptable if the soil horizon is stable and density can be achieved in subsequent structural fill lifts. Scarification and moisture conditioning may be required to achieve the required soil moisture content recommendations. It is recommended that the moisture content of the soils shall be determined prior to densification to evaluate the need for moisture conditioning. After the densification process, a firm, stable surface should be produced

It is recommended that a large vibratory roller is used to densify subgrade soils. The roller shall make at least 3 to 4 passes over the soils.

8.3 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 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
4 Inch	100
¾ Inch	70 – 100
No. 40	15 – 60
No. 200	5 – 30
Maximum Liquid Limit	**Maximum Plastic Index**
30	10

Based on laboratory test results, soils encountered below anticipated stripped and grubbed zones, free of debris or other deleterious materials appear to meet the requirements for structural fill. Additionally, soils derived from strongly cemented zones may contain cemented chunks that are recommended to be broken down by mechanical methods prior to placement as a structural fill.

Structural fills 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 prepared in accordance with Section 8.2- Site Preparation.

Loose structural fill lift thicknesses, up to 12-inches, are acceptable 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 Permanent Slope Gradients, Stability, and Erosion Control

The project site is relatively level such that cut depths and fill thicknesses will be minimal. If required, 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[4]. The project geotechnical engineer should be consulted for site specific recommendations.

[4] Steeper fill slopes, if required, may be constructed using reinforced earth techniques.

8.5 Trenching and Confined Excavations

All excavations regardless of depth should be evaluated to check the stability 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 require different temporary excavation cut slope gradients. Maximum allowable slopes for excavations less than 20 feet deep are presented in Table 4 (Maximum Allowable Temporary Slopes). Based on our observation of site soils, excavations should comply with current OSHA safety requirements for a either Type A or Type C soil. Type A soils consist of strongly cemented soil types. Type C soils are all other granular soils. Soil conditions should be verified during construction to assess required trench slope gradients.

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	90°
Type A	3H:4V	53°
Type B	1H:1V	45°
Type C	3H:2V	34°

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

Trench excavations should be protected from surface water/runoff. Temporary drainage swales should be excavated to divert surface flows into a collection area away from the open excavation. 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.

8.5.1 Excavatability

Based on the conditions encountered, excavations may be completed using conventional excavation equipment such as a track mounted excavator or rubber-tired backhoe.

8.6 Foundation Recommendations

Structural loads were not available at the time this report was prepared. For the purpose of this report, it is assumed that structural loads will be light. Based on the information provided by the City of Sparks, the prefabricated restroom will have concrete slab-on-grade flooring and be supported by shallow, spread foundations. Recommendations for foundation grade soils preparation and foundation design are based on the loading and foundation design assumptions of this report. If alternate foundations are proposed, additional recommendations can be provided upon request.

8.6.1 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. Foundations can bear directly on either densified structural fill or native cemented granular materials as encountered below 3½ feet in depth in Test Pit TP-2.

Foundation grade soils shall be prepared in accordance with recommendations given in Sections 8.2 - Site Preparation and 8.3 Grading and Filling.

8.6.2 Shallow Spread Footing Foundation Design

Provided foundation soils preparation has been performed in accordance with the recommendations given in Section 8.6.1 (Foundation Grade Soils Preparation), foundation design parameters presented in Table 5 (Foundation Design Parameters) can be utilized for the design of individual column footing and continuous wall footings.

Table 5 – 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.	2,500
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.

(5) Structural dead and fulltime live loads on the order of 3.0 kips per lineal foot were assumed..

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 or 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.

8.6.3 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.

Based on the loading assumptions of this report[5] and the anticipated foundation grade material, settlement on the order of ½-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).

8.7 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 8.3-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 structural 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.

8.8 Corrosion Potential

Corrosion testing was completed on a soil sample from Test Pit TP-2. Western Environmental Testing 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 test results detected a level of 27mg/kg indicating that site soils have a negligible potential for sulfate exposure to concrete.
- **pH (EPA 9045D):** The paste pH test result of 7.67 indicates site soils are slightly alkaline and have a moderate potential of corrosion for soils in direct contact with ferrous metals (Baboian et. al, 2006).
- **Resistivity (ASTM G57):** Resistivity test results of 1500 ohms.cm were measured. Results indicate that the site soils have a severe corrosion potential for ferrous metal in direct contact with these soils (Baboian, 2005). A corrosion specialist should be consulted for project design.

[5] Structural loads on the order of 1.5 kips per lineal foot are proposed for continuous spread footings.

8.9 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 the proposed building is recommended. The slope gradient can be reduced to 2 percent for impervious surfaces adjacent to structures, such as concrete slabs-on-grade and pavement.

9.0 STANDARD LIMITATIONS CLAUSE

This report has been prepared in accordance with generally accepted local geotechnical practices. The analyses and recommendations submitted are based upon field exploration performed at the locations shown on 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[6]. 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[7].

This report was prepared by CME for the City of Sparks. 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;
- Field density and materials testing; and
- Special inspection of structural elements.

---

[6]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.

[7]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

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

J. W. Bell and H. F. Bonham Jr., et.al, *Geologic Map* Vista Quadrangle, NBMG, 1987.

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

Robert Baboian, et. al., Corrosion Tests and Standards, Application and Interpretation, 2nd edition, 2006

Craig M. dePolo*, Quaternary Faults in Nevada, Nevada*, *Map 167,* Nevada Bureau of Mines and Geology (NBMG), *2008.*

Google Earth Pro aerial images, Accessed August 2016

International Building Code, 2012; *International Code Council, Inc.*

NRCS Web Soil Survey, *http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm* ,accessed August 2016

Standard Specification for Public Works Construction, Regional Transportation Commission, 2016

USGS, *Seismic Design Maps (software),* *http://earthquake.usgs.gov/designmaps/us/application.php,* accessed August 2016

USGS Quaternary Fault Hazards Map, *http://earthquake.usus.gov/hazards/qfaults/map/,* accessed August 2016

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX A

BASE MAP OBTAINED FROM WASHOE COUNTY GIS MAP APPLICATION (http://washoe.maps.arcgis.com/home/webmap), AUGUST 2016

C:\Users\sheim\Documents\1902 site plan.dwg

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

CITY OF SPARKS
PAR RAH PARK RESTROOM
FIELD EXPLORATION LOCATION MAP
WASHOE COUNTY, NEVADA

PROJECT NO.: 1902 DATE: 8/22/2016

LEGEND

PROPOSED/APPROXIMATE TEST PIT LOCATION

PLATE

A-1

LOG OF TEST PIT NO. TP-1

PROJECT PAH RAH MOUNTAIN PARK RESTROOM **EQUIPMENT TYPE** DEERE 310 SG RUBBER TIRE
CLIENT CITY OF SPARKS
LOCATION PAH RAH MOUNTAIN PARK, APPROXIMATELY 7' E. OF EXISTING RESTROOM
PROJECT NO. 1902 **DATE** 8/12/2016 **LOGGED BY:** SH **SURFACE ELEVATION (ft)** 4482' (GOOGLE EARTH)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Bulk Specific Gravity	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SC			B	1A		VERY MOIST	0-2.5': CLAYEY SAND, mostly fine to medium sand, low plasticity, dark brown. NOTE: Roots to 1 foot below ground surface, roughly 1/2" nominal diameter.							
2.5								Test pit terminated at 2.5 feet due to a ruptured water line.							
5															
7.5															
10															
12.5															
15															

GROUNDWATER

	DEPTH	HOUR	DATE
▽	N.E.		8/12/2016
▼			

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 PAH RAH MOUNTAIN PARK RESTROOM **EQUIPMENT TYPE** DEERE 310SG RUBBER TIRE
CLIENT CITY OF SPARKS
LOCATION PAH RAH MOUNTAIN PARK, APPROXIMATLEY 20' NE OF EXISTING RESTROOM
PROJECT NO. 1902 **DATE** 8/12/2016 **LOGGED BY:** SH **SURFACE ELEVATION (ft)** 4484' (GOOGLE EARTH)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Bulk Specific Gravity	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SP-SM						VERY MOIST	0-3.5': POORLY GRADED SAND WITH SILT, mostly fine to medium sand, non plastic, dark brown.							
				B	2A				11.2	NV	NP		116.0	13.5	A, G, MD
								NOTE: Appears to be loosely placed undocumented fill.							
2.5								NOTE: Nuke test at 1.5 ' D.D. - 69.1pcf M.C. - 13.8%							
								NOTE: Bulk sample (2D) taken from 0-4'.							
	SP-SM			B	2B		VERY MOIST	3.5'-8': POORLY GRADED SAND WITH SILT, mostly fine to medium sand, non plastic, dark brown, heavily cemented.							
5								NOTE: White calcareous streaks from 4- 8 feet.							
								NOTE: Very hard excavating from 3.5'-6'							
								NOTE: Moderately cemented from 6'-8', slightly easier excavating.							
7.5				B	2C				9.0	NV	NP			8.6	A, G
								Test pit terminated at 8 feet.							
								NFWE							
10															
12.5															
15															

GROUNDWATER

	DEPTH	HOUR	DATE
▽	N.E.		8/12/2016
▼			

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.

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

CITY OF SPARKS
PAH RAH PARK RESTROOM
SOIL CLASSIFICATION CHART
WASHOE COUNTY, NEVADA

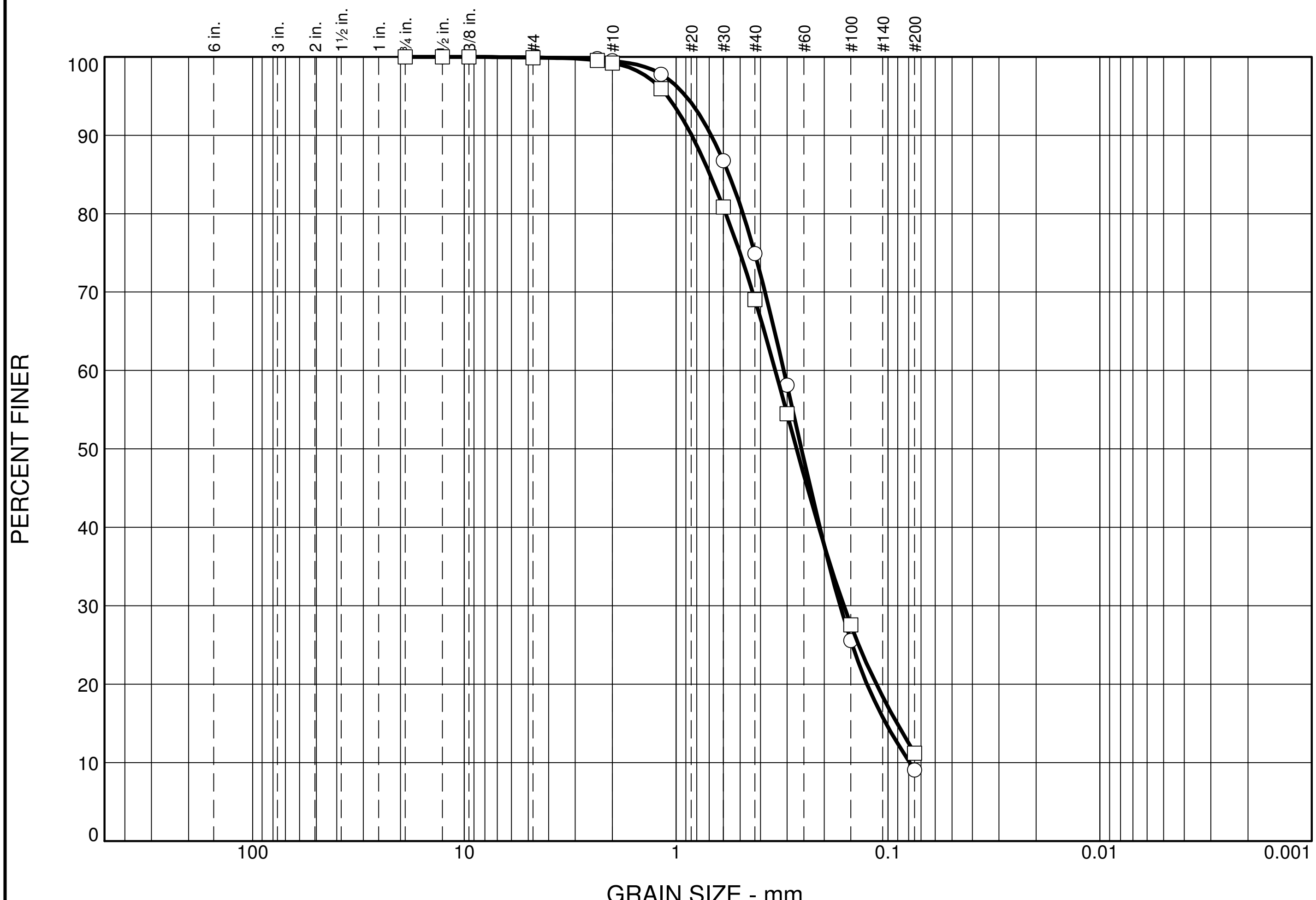
PROJECT NO.:1902 DATE: 08/22/2016

PLATE A-3

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	0.1	0.4	24.6	65.9	9.0	
□	0.0	0.0	0.1	0.7	30.2	57.8	11.2	

	Colloids	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○		NV	NP	0.5641	0.3111	0.2564	0.1683	0.1024	0.0791	1.15	3.93
□		NV	NP	0.6931	0.3410	0.2703	0.1618	0.0906			

Material Description	USCS	AASHTO
○ poorly graded sand with silt	SP-SM	A-3
□ poorly graded sand with silt	SP-SM	A-2-4(0)

Project No. 1902 **Client:** CITY OF SPARKS

Project: PAH RAH MOUNTAIN PARK RESTROOM

○ **Location:** TP-2 **Depth:** 7.0-8.0' **Sample Number:** 2C

□ **Location:** TP-2 **Depth:** 0-4.0' **Sample Number:** 2D

Date: ○ 8/15/2016 □ 8/15/2016

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

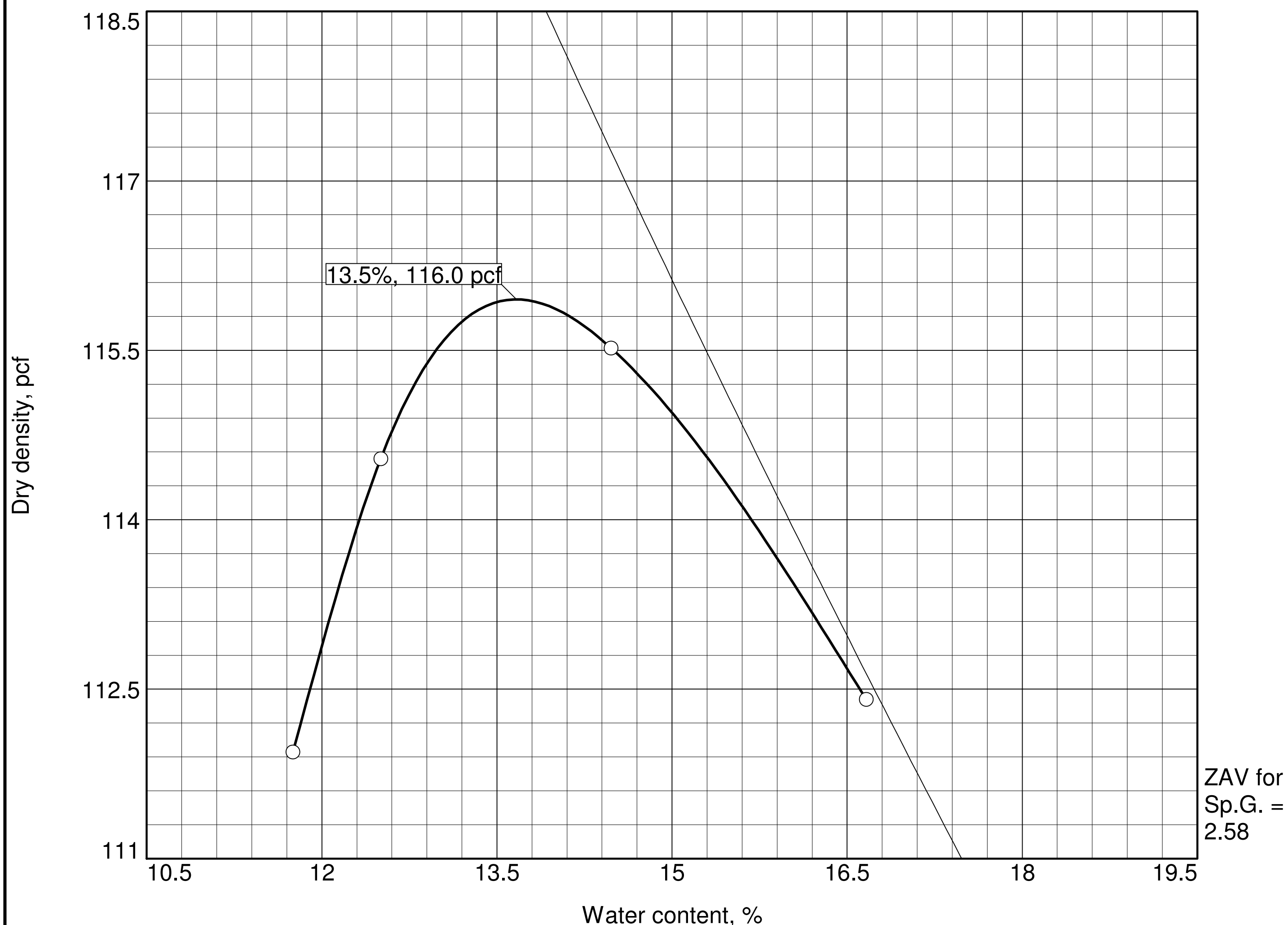
Remarks:

○ MOISTURE CONTENT - 8.6%

□ MOISTURE CONTENT - 11.8%

PLATE B-1

Tested By: T. LEGG **Checked By:** S. HEIN



Test specification: ASTM D 1557-07 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0-4.0'	SP-SM	A-2-4(0)			NV	NP	0.1	11.2

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 116.0 pcf Optimum moisture = 13.5 %	poorly graded sand with silt

Project No. 1902 **Client:** CITY OF SPARKS

Project: PAH RAH MOUNTAIN PARK RESTROOM

Date: 8/16/2016

○ **Location:** TP-2 **Sample Number:** 2D

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Remarks:

PLATE B-2

Tested By: S. BRUKETTA **Checked By:** S. HEIN

Western Environmental Testing Laboratory
Analytical Report

Construction Materials Engineers
6980 Sierra Center Parkway, Suite 90
Reno, NV 89511
Attn: Steve Vineis
Phone: (775) 851-8205 **Fax:** (775) 737-7615
PO\Project: *City of Sparks Pah Rah Bathroom*

Date Printed: 8/22/2016
OrderID: *1608482*

Customer Sample ID: Test Pit 2 2D C 0-3 1/2 Inches
WETLAB Sample ID: 1608482-001

Collect Date/Time: 8/15/2016
Receive Date: 8/15/2016 16:45

Analyte	Method	Results	Units	DF	RL	Analyzed	LabID
General Chemistry							
Paste pH	SW846 9045B	7.67	pH Units	1		8/17/2016	NV00925
Resistivity	SM 2510B	1500	ohms.cm	1	1.0	8/18/2016	NV00925
Anions by Ion Chromatography							
Sulfate	EPA 300.0	27	mg/kg	3	3.0	8/16/2016	NV00925
Sample Preparation							
Saturated Paste Preparation	CSTPM S:1.0	Complete		1		8/17/2016	NV00925
3:1 DI Water Extraction	WL 3.0	Complete		1		8/15/2016	NV00925

V:\Active\1902\APPENDICES\corrosion testing.dwg

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

CITY OF SPARKS
PAH RAH PARK RESTROOM
CORROSION TESTING
WASHOE COUNTY, NEVADA

PROJECT NO.:1902 DATE: 8/22/2016

PLATE
B-3

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX C

Design Maps Summary Report

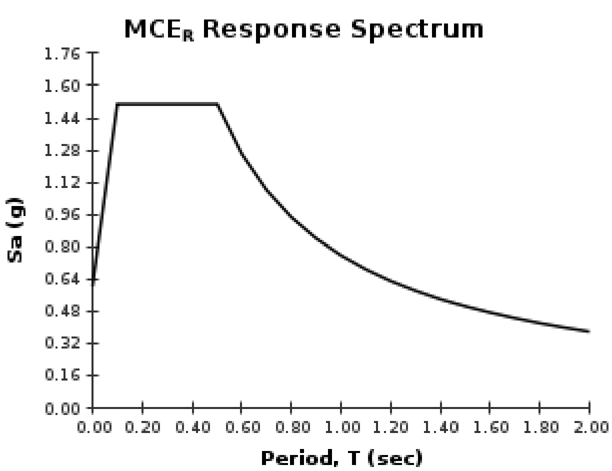
User–Specified Input

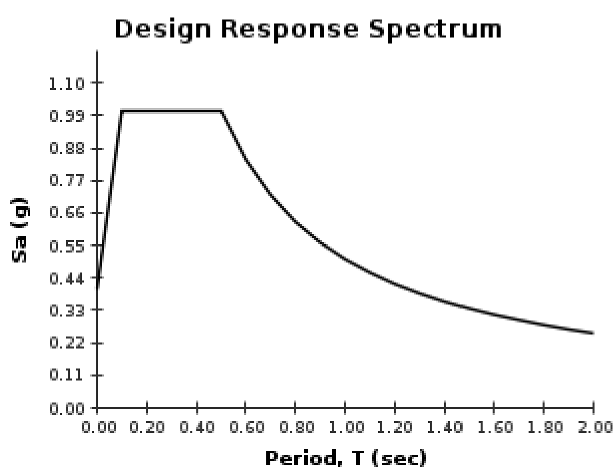
Report Title	Pah Rah Park Restroom Thu August 25, 2016 15:49:41 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	39.56492°N, 119.7078°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	1.509 g	S_{MS} =	1.509 g	S_{DS} =	1.006 g
S_1 =	0.504 g	S_{M1} =	0.757 g	S_{D1} =	0.504 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.





Design Maps Detailed Report

2012/2015 International Building Code (39.56492°N, 119.7078°W)

Site Class D – "Stiff Soil", 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 2012/2015 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 = 1.509 g
From Figure 1613.3.1(2) [2]	S_1 = 0.504 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 D, 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 ≥ 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	$\boxed{1.0}$
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 = D and S_S = 1.509 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	$\boxed{1.5}$
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 = D and S_1 = 0.504 g, F_v = 1.500

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 1.509 = 1.509$ g

Equation (16-38): $S_{M1} = F_vS_1 = 1.500 \times 0.504 = 0.757$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.509 = 1.006$ g

Equation (16-40): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.757 = 0.504$ 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} = 1.006 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} = 0.504 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)" = D

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

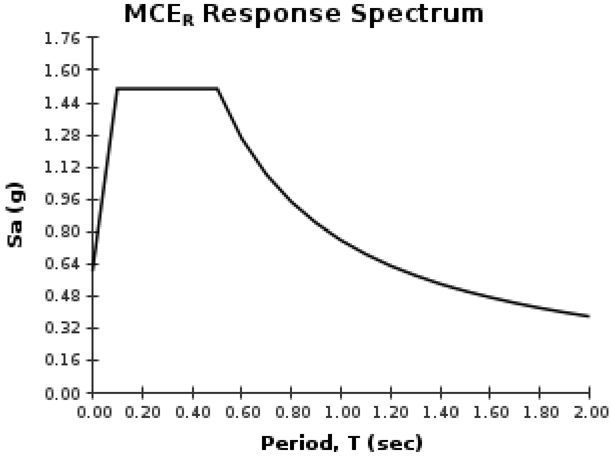
Report Title	Pah Rah Park Restroom Thu August 25, 2016 15:42:51 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	39.56492°N, 119.7078°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

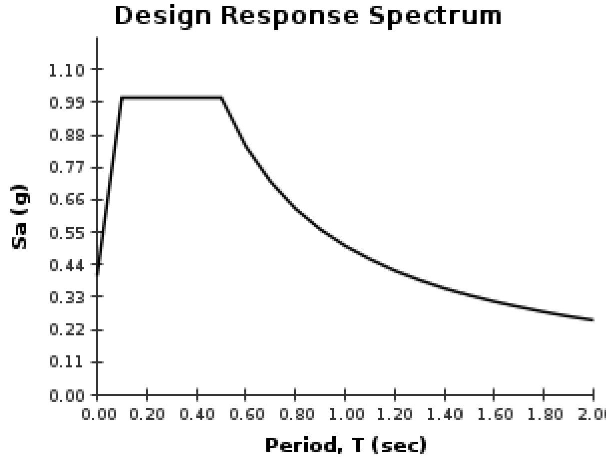
S_S = 1.509 g	S_{MS} = 1.509 g	S_{DS} = 1.006 g
S_1 = 0.504 g	S_{M1} = 0.757 g	S_{D1} = 0.504 g

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.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (39.56492°N, 119.7078°W)

Site Class D – “Stiff Soil”, 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] $S_S = 1.509$ g

From Figure 22-2 [2] $S_1 = 0.504$ 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 D, 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* ≥ 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 = D and S_S = 1.509 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 = D and S_1 = 0.504 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.509 = 1.509$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.504 = 0.757$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

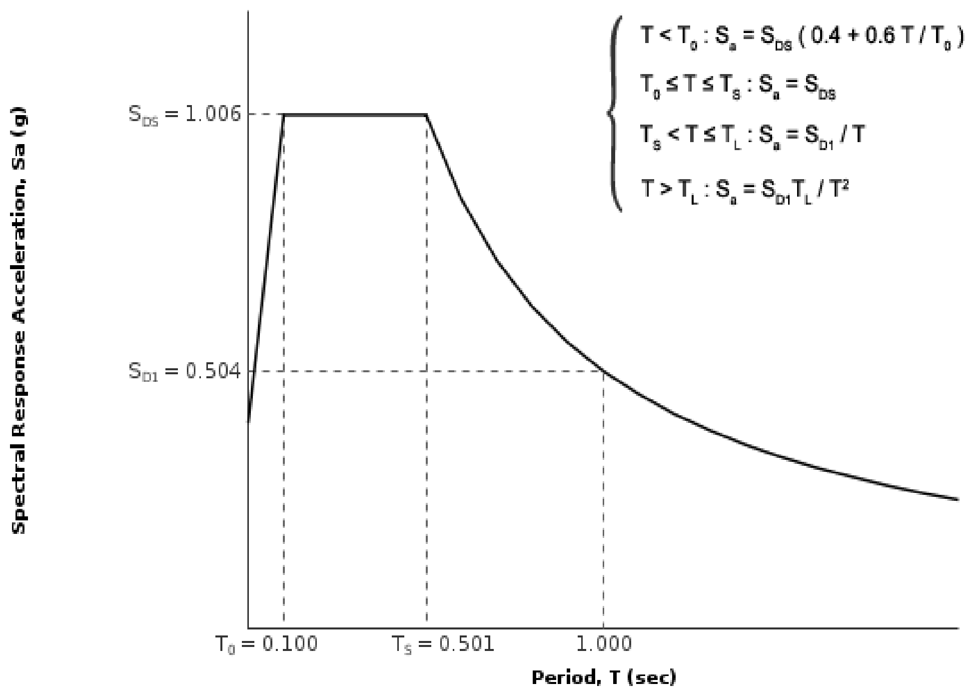
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.509 = 1.006$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.757 = 0.504$ g

Section 11.4.5 — Design Response Spectrum

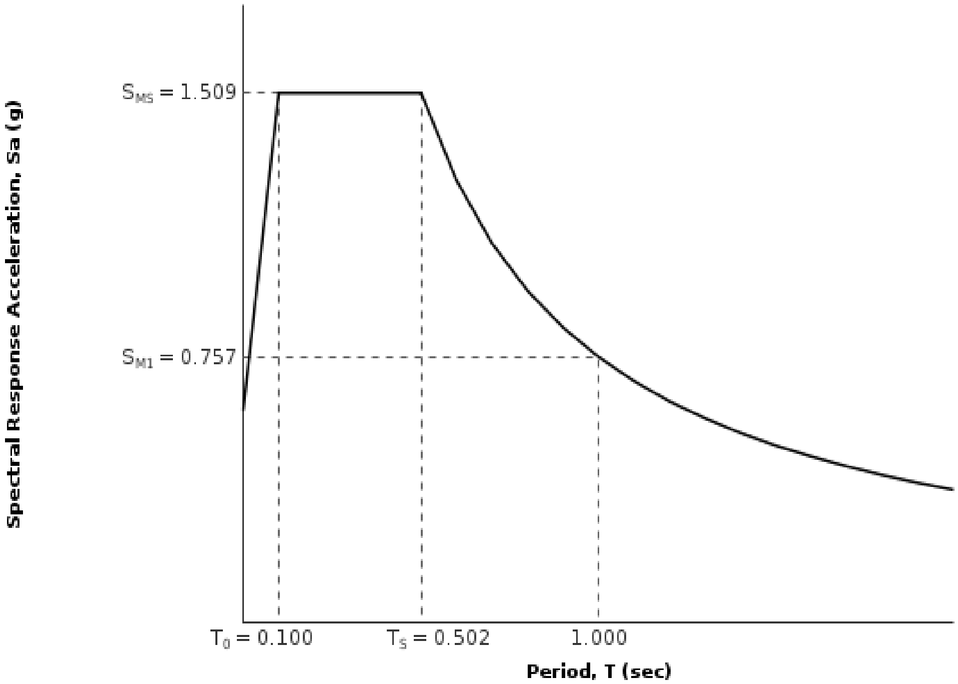
From Figure 22-12 [3] T_L = 6 seconds

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 = 0.560

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.560 = 0.56$ 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 = D and PGA = 0.560 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} = 0.949

From Figure 22-18 [6] C_{R1} = 0.946

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} = 1.006 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} = 0.504 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