Geotechnical Investigation

TMWA Arrow Creek Drought Project

Arrow Creek Well 1, Copper Cloud and STMGID Well 11

Reno, Washoe County, Nevada

Prepared for:

Stantec

July 2015
File: 1739

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July 24, 2015
File: 1738

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**RE: Geotechnical Investigation -TMWA Arrow Creek Drought Project
Arrow Creek Well 1, Copper Cloud, and STMGID Well 11
Reno, Washoe County, Nevada**

Dear Mr. Diegle:

Construction Materials Engineers, Inc. (CME) is pleased to submit our geotechnical investigation report for the proposed Arrow Creek Drought Project, in Reno area, Washoe County, Nevada.

The following report includes the results of our field and laboratory investigations and presents our recommendations for the design and construction of the project.

We hope this report provides you with the information you require at this time, please contact the undersigned if you have any questions.

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

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GEOTECHNICAL INVESTIGATION
TMWA Arrow Creek Drought Project
Washoe County, Nevada

1.0 INTRODUCTION

The Arrow Creek Drought project includes booster pump improvements to three existing facilities:

1) **Arrow Creek Well #1**: This well is located on the northeast side of the private Arrow Creek Golf Community;

2) **Copper Cloud Booster Pump Station:** This pump station is located on the southwest side of the private Arrow Creek Golf Community; and

3) **STIMGID Well #11**: This well is located on the south side of West Zolezzi Lane, west of the Oak Glen Drive intersection.

A vicinity map showing the approximate facility locations is included as Figure 1.

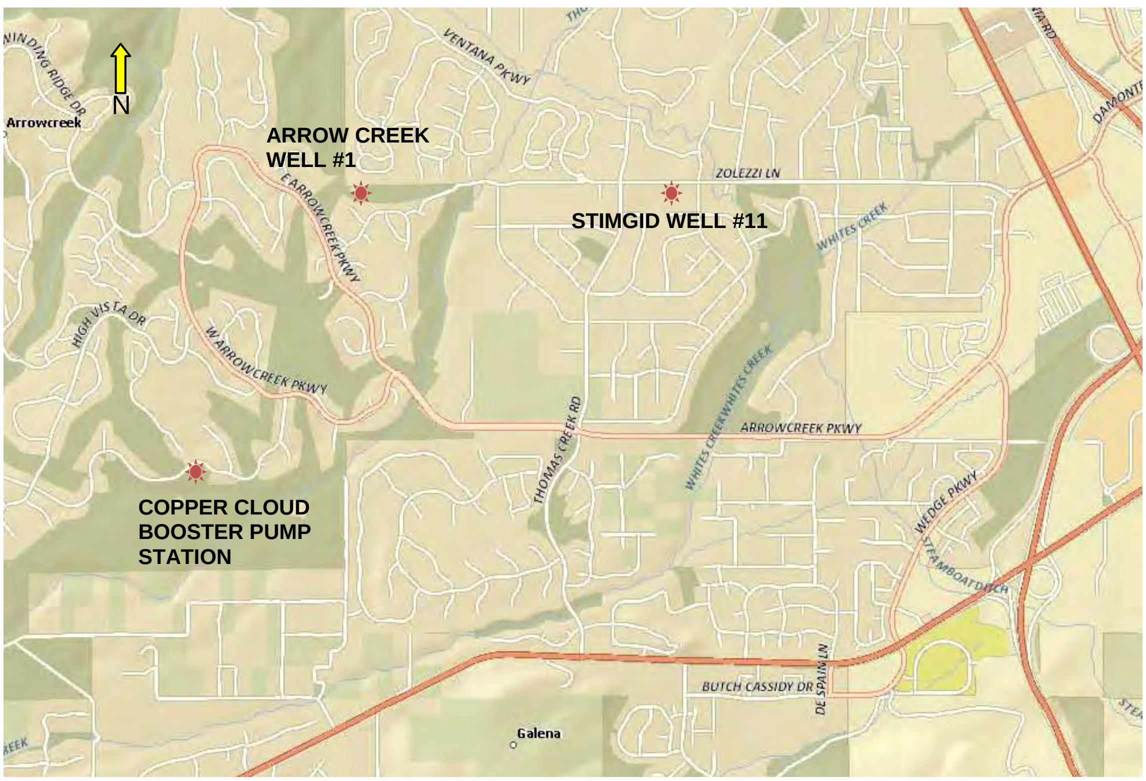


Figure 1: General Project Vicinity
N.T.S

The purpose of this investigation is to characterize the subsurface soils and provide site preparation and grading recommendations for project design. The geotechnical investigation included a literature review, subsurface exploration utilizing backhoe test pits, laboratory testing, and engineering analysis to develop geotechnical recommendations for project design.

The following definitions shall apply unless otherwise specified in this report:

- Structural areas referred to in this report include all areas that will be used for the support of pavements, slabs, vaults, and foundations;
- All compaction requirements presented in this report are relative to ASTM D1557[1]; and
- All related construction should be in general accordance with the Standard Specifications for Public Works Construction (SSPWC), dated 2012.
- Fine-grained soil is defined as soil with more than 40 percent by weight passing the number 200 sieve and a plasticity index lower 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 equal to or greater than 15.
- Granular soil is defined as soil not meeting the above criteria with a particle sizing of less than 6-inches.

[1] 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.

2.0 SITE CONDITIONS

2.1 Arrow Creek Well #1

The Arrow Creek Well #1 site is located approximately 760 feet northeast of the intersection of Flint Ridge Court and Granite Pointe Drive within the limits of the Arrow Creek Private Golf Community. The site is currently occupied by a fenced well house, existing pump station, and supporting utilities.

The site is accessed via a roughly graded dirt road which intersects with Granite Pointe Drive east of the Flint Ridge Court intersection. Arrow Creek Well #1 is masked by an embankment located south of the facility. An existing golf path, consisting of a concrete slab-on-grade, passes the facility to the south.

Photograph 1: Taken looking north toward Test Pit TP-1, note embankment masking existing well house facility.

Vegetation at the site consists of sparse brush and low grasses with moderately dense landscape tree groupings located around the well house fence perimeter to mask the facility from the surrounding fairways.

2.2 Copper Cloud Booster Pump Station

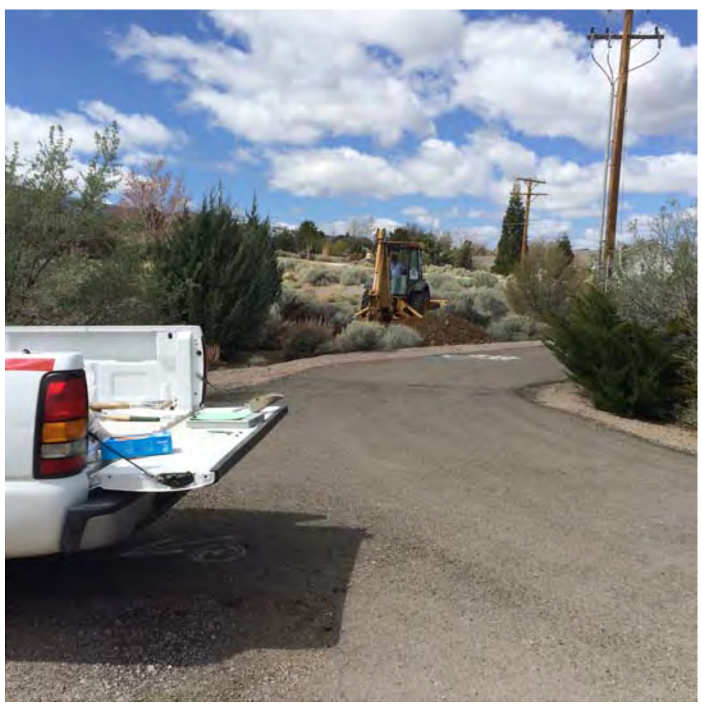
Copper Cloud Booster Pump Station is located on the southwest side of Copper Cloud Drive, approximately 430 feet northwest of the Eagle Vista Court intersection. The existing booster pump station facility consists of fully fenced pad, approximately 40 feet by 30 feet, with existing booster pumps and supporting equipment. The booster pump station is bounded to the west and south by an embankment with a height that is approximately 5 to 6 feet above the booster pump station pad; to the north by Copper Cloud Drive; and to the east by a dirt access road.

Vegetation at the site includes sparse brush present on and near the top of the adjacent embankment (Photograph 2).

Photograph 2: Taken looking north toward test pit TP-4

2.3 STMGID Well #11

STMGID Well #11 is located on the south side of West Zolezzi Lane approximately 650 feet west of the Fieldcreek Lane intersection. The site is accessed via a paved access loop that intersects West Zolezzi at two locations.



Photograph 3: Taken looking northwest toward Test Pit TP-6, located on the west side of the entrance loop.

An existing fenced booster pump is located between the entrance points of the access loop. South of the existing booster pump station and access loop is a fenced well house with supporting utilities. Access to the well house is via a dirt road located off the paved access loop.

An overhead high voltage electrical line is located traversing the northeast side of the site in a northwest to southeast direction. A cobble lined swale is located in the west side of the site, beginning on the south side of the access loop and trending northwest approximately 180 feet before terminating near West Zolezzi Lane.

Vegetation at the site consists of concentrated shrubs near the booster pump station, sparse to moderately dense tree groupings near the existing well house and west of the access loop, and sparse brush and low grasses within the remainder of the site.

3.0 FIELD EXPLORATION

Subsurface field exploration, completed on April 2, 2015, consisted of excavating 3 test pits at the Arrow Creek Well #1 site, 1 test pit at the Copper Cloud Booster Pump Station, and 3 test pits at the STMGID Well #11 site.

Test pits were excavated using a John Deere 310SG rubber-tired backhoe equipped with a 36-inch bucket.

Test pits ranged in depth from 4 to 12 feet below the existing ground surface (bgs). Test pits were located in the field by visual sighting and/or measuring from existing features at the site. Approximate locations of the test pit excavations are presented on Plate A-1 (Field Exploration Location Map).

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

Test pits were backfilled using the equipment at hand; back-fill was loosely placed and not compacted to the standards typically required for properly placed structural fill[2].

Test pit logs are included as Plates A-2, A-3, and A-4. Elevations shown on the test pit logs were obtained by interpolation between contour lines shown on the attached Field Exploration Location Map (Plate A-1). Elevations and locations included in this report should be considered accurate only to the degree implied by the methods used.

Upon completion of laboratory testing, additional soil classification and verification of the field classifications were subsequently performed in accordance with the Unified Soil Classification System (USCS), as presented in ASTM D 2487. A description of the USCS is presented on A-5.

[2] **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.

4.0 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 the pavement survey coring and test pit exploration were selected and analyzed to determine index properties. The following laboratory tests were completed as part of this investigation:

- *Insitu* moisture content (ASTM D 2216);
- Grain size distribution (ASTM D 422); and
- Atterberg Limits (ASTM D 4318); and
- Corrosion testing including soluble sulfates was completed by an outside laboratory.

Laboratory test results are presented on the test pit logs and included as Appendix B.

5.0 SUBSURFACE SOILS AND GROUNDWATER CONDITION

The *Mt. Rose NE Quadrangle Geologic Map,* (Bonham, et. al., 1983), maps the geologic profile at the project sites as Quaternary aged Donner Lake Outwash deposits. Donner Lake Outwash is described as bouldery outwash forming strath terraces on bedrock, includes unconsolidated small cobble gravel and interbedded coarse sand. The Donner Lake Outwash Formation is a glacial outwash deposit of Pleistocene age characterized as a heterogeneous mixture of sands, gravels, cobbles and boulders. Boulder-sized particles up to 16 feet in diameter have been encountered in this deposit (Bingler, 1975). Typically, this formation has a well-developed argillic horizon consisting of clayey sands and gravels with moderate to high plasticity soil characteristics.

Native soils encountered during the subsurface exploration appear to be similar to the Donner Lake Outwash deposits. A general soils profile description for each site is included as Sections 5.1 to 5.3.

5.1 General Soils Profile Arrow Creek Well #1

In general soils encountered during the subsurface test pit exploration at Arrow Creek Well #1 consisted predominately of granular material classified as either clayey sand with gravel **(SC)** or clayey gravel with sand **(GC).**

Test Pit TP-1 was excavated at the toe of the existing embankment located on the south side of the project area. The existing embankment soils encountered (the upper ½ foot of the soil profile) consisted of sandy fat clay **(CH)** underlain by native clayey sand with gravel **(SC)** to the depth of refusal encountered at 7 feet below the existing ground surface (bgs). Soils encountered in Test Pit TP-2 consisted of upper clayey sand **(SC)** soil horizon underlain by an argillic zone consisting of clayey sand with gravel **(SC)** to the depth of refusal at 5.25 feet.

Test pit TP-3 was excavated on the northwest side of the project area. The soil profile consisted of clayey sand with gravel **(SC)** to a depth of 2 feet underlain by an argillic zone consisting of clayey gravel with sand, cobbles and boulders **(GC)**. Excavation refusal occurred at a depth of 4 feet bgs.

5.2 General Soils Profile Copper Cloud Booster Pump Station

Due to the presence of existing site utilities and facility fencing, excavation exploration was limited to one test pit. Test pit TP-4 was excavated near the southeast corner of the existing booster pump station approximately

10 feet south of the fencing located at the toe of the existing slope. Fill soils consisting of silty sand with gravel **(SM)** were encountered starting from the ground surface to a depth of approximately 3½ feet bgs. Fill soils were underlain by native silty sand with gravel **(SM)** to silty sand with gravel and cobbles **(SM)** extending to the termination depth of the exploration.

5.3 General Soils Profile STMGID Well #11

Soils encountered at the STMGID Well #11 site consisted predominately of an uppermost clayey sand **(SC)** fill horizon with a thickness of approximately 2 to 4 feet. Fill soils in Test Pit TP-5 consisted of clayey gravel with sand **(GC)** at a depth ranging from 2 to 4 feet bgs. Fill soils across the site, except encountered in Test Pit TP-5 were underlain by native clayey sand **(SC)** to clayey sand with gravel **(SC)** extending to the test pit termination depth. Silty sand with gravel and cobbles **(SM)** was encountered in Test Pit TP-5 at a depth of 5 feet extending to the depth of exploration refusal (8 feet).

5.4 Groundwater Conditions

Groundwater was not encountered during the subsurface exploration at any of the project sites. In general soils were encountered in a moist condition. Current water levels for the existing wells are as follows:

- STMGID Well #11: 160 feet bgs;
- Arrow Creek Well #1: 290 feet bgs; and
- Arrow Creek Well #3 (located approximately 2,500 feet west of Copper Cloud Booster Pumps Station): 230 feet bgs.

Truckee Meadows Water Authority (TMWA) has provided nearby monitoring port water level measurements which indicate that the water table elevations in the project areas are greater than 100 feet bgs.

Groundwater should not affect construction; however, it should be noted that fluctuations in groundwater elevations may occur due to increased precipitation, irrigation or future development.

6.0 SEISMIC CONDITIONS AND GEOLOGIC HAZARDS

The subject site 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 base of the Sierra Nevada, within the western extreme of the Basin and Range.

6.1 Faulting

Based on a review of the USGS website (http://earthquake.usus.gov/hazards/qfaults/map/) and referenced geologic map, several fault traces are located in the vicinity of each project area; with multiple potential fault traces trending toward or through each site.

The subject sites are located in the Mount Rose Fault Zone Complex, which consists of a series of en echelon, discontinuous, short faults, generally trending in a north-south direction.

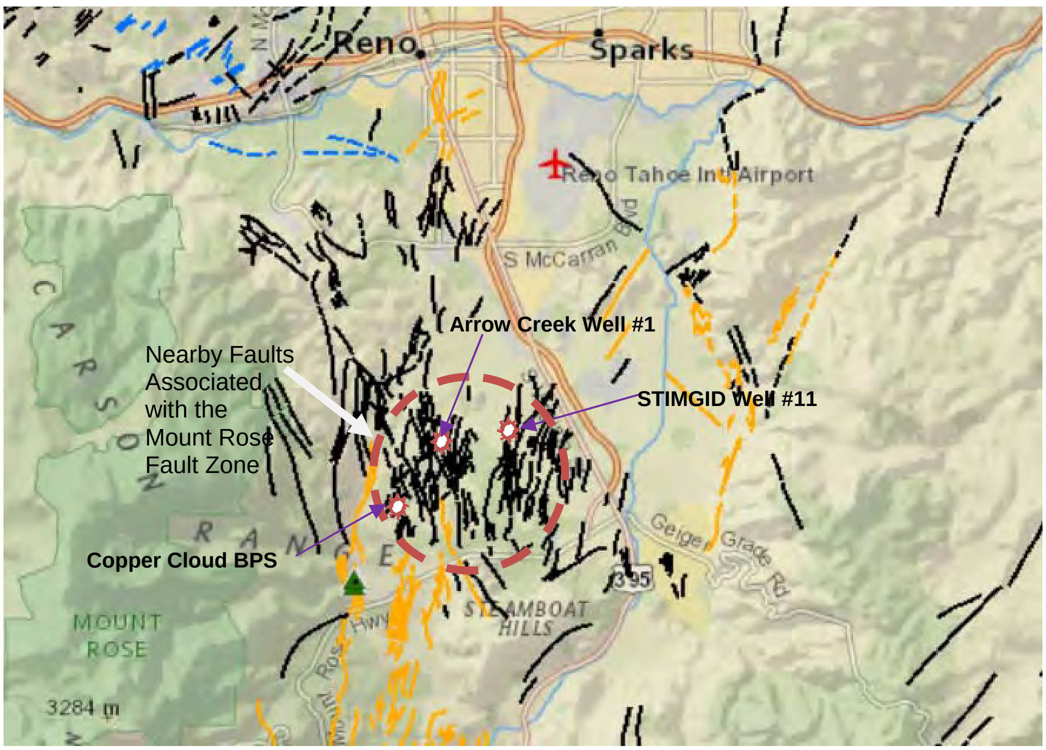


Figure 2: USGS Quaternary Fault Hazards Map

(earthquake.usus.gov/hazards/qfaults/map/, accessed February 2015**)**

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 faults located near the subject sites are classified as Latest Quaternary Active Faults (<15,000 years old (Sawyer, Fault 1647)). Additional exploration, including fault trenches, would be required to provide a definitive fault location.

6.2 Seismic Design Parameters

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

Table 1 – Site Classification Definition

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

The soil/bedrock profile classification is based on two criteria: density (primarily for soils based on SPT blow count data) or hardness (based on shear wave velocity primarily for bedrock sites). These two criteria have to be determined to a depth of 100 feet below the ground surface.

Geophysical studies and SPT testing were not included in the scope of services for this project. Based on our experience in the project area, it is our opinion that a Site Class D be used for project design. Table 1 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 (2012 IBC)			
	Arrow Creek Well #1	**Copper Cloud BPS**	**STMGID Well#11**
Approximate Latitude of Site	39.41838	39.40246	39.4185
Approximate Longitude of Site	119.8075	119.8180	119.7846
Spectral Response Acceleration at short period (0.2 sec.),S_s (for Site Class B)	2.230	2.182	2.241
Spectral Response Acceleration at 1-second Period, S_1 (for Site Class B)	0.782	0.775	0.766
Site Class Selected for this Site	D	D	D
Site Coefficient F_a, decimal	1.0	1.0	1.0
Site Coefficient Fv, decimal	1.5	1.5	1.5
Peak Ground Acceleration-MCE_R PGA (ASCE 7-10 Standard)[1]	0.892 g	0.867 g	0.889 g
Design Spectral Response Acceleration at Short period, S_{Ds} (Adjusted to Site Class D, SDs= 2/3 SMs)	1.487	1.455	1.494
Design Spectral Response Acceleration at 1-second Period, S_{D1} (Adjusted to Site Class D, SD1=2/3 SM1)	0.782	0.775	0.766
Notes: 1) MCE_R PGA- Maximum credible earthquake geometric mean peak ground acceleration.			

7.0 DISCUSSION AND RECOMMENDATIONS

7.1 Construction Concerns

Grading plans were not available at the time this report was prepared. The exact location of the proposed improvements has not been determined for each of the subject sites. Based on the existing site topography, it is anticipated that a majority of the site grading at each location will include cuts on the order of 2 to 5 feet. Deeper cuts may be required for vault/booster pump station installation. Fill berms to mask proposed improvements may be required at the Arrow Creek Well #1 and Copper Cloud BPS sites, and are anticipated to be less than 4 feet.

The primary construction concerns are as follows:

Arrow Creek Well #1: During the subsurface exploration, excavation refusal was encountered at depths ranging from about 4 to 7 feet. Depending on the proposed installation depth of the booster pump vault(s), the presence of strongly cemented soils may require larger excavation equipment, such as a trackhoe, to penetrate below these depths. The presence of large cobbles and boulders may make confined trench excavations difficult. Screening of excavated material will likely be required during backfilling of proposed vaults and in areas where existing granular soils will be used as structural fill. Based on soil conditions encountered in Test Pit TP-5, the existing embankment that masks the well house may consist of fine-grained clay soils. Additional testing and exploration will be required if the existing embankment soils are considered for use as structural fill.

Copper Cloud BPS: It is expected that excavations for the proposed booster pump vaults can be completed using conventional excavation equipment such as a rubber tired backhoe or track-mounted excavator. The presence coarse material (i.e. cobbles and boulders) may make confined trench excavations difficult. Screening of the excavated material may be required during backfilling of proposed vaults and in areas where existing soils will be used as structural fill. Fill soils were encountered to a depth of 3 ½ feet bgs. These fill soils may not have a consistent thickness across the site. Removal and densification of existing fill soils will be dependent on the final location of proposed structures.

STMGID Well #11: Fill soils were encountered across the exploration area. The depth of fill ranged from 2 feet southwest of the existing well house and increased to 4 feet north of the existing well house. At the time of our exploration, documentation regarding the placement and compaction of these soils was not available. For the purposes of this report, it is assumed that these soils are undocumented and may require removal, replacement with structural fill, and densification in accordance with the recommendations of this report. If documentation during fill placement is provided, an addendum to this report can be prepared.

7.2 Discussion

The recommendations provided herein, and particularly under **Site Clearing** (Section 7.3)**, Grading and Filling** (Section 7.4)**,** and **Construction Observation and Testing** (Section 8.0) 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 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.

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

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

7.3 Site Clearing

Prior to construction, surface vegetation and organic soils should be stripped and disposed of outside the construction limits or stockpiled for use in non-structural areas. Stripping depths of 0.2 to 0.33 feet are anticipated over a majority of the project areas. In areas where established brush and shrubs are present, grubbing depths of up to 12 inches may be required to remove the concentrated root zone.

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

Depending on the final site layout, deeper areas of localized grubbing to remove tree root balls may be required at the Arrow Creek Well #1 and STMGID Well #11 project sites. The entire root ball 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 root ball area, located within one foot of the final subgrade should be completely removed. Excavations resulting from removal operations should be cleaned of all loose material and widened as necessary to permit access to compaction equipment. Resulting excavations should be backfilled with Class 1 structural fill placed in accordance with Section 7.4 (Grading and Filling) of this report.

7.4 Grading and Filling

Structural fill is defined as supporting soil placed below foundations, concrete slabs-on-grade, pavements, or any structural element that derives support from the underlying sub-soils. Structural fill shall meet the requirements for a granular soil. In general soils encountered during our field exploration, below the anticipated stripped and grubbed zones, free of debris or other deleterious materials appear to meet the requirements for structural fill, provided particles greater than 6 inches in diameter , where encountered, are removed.

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

Table 3- Guideline Specifications for Structural Fill	
Sieve Size	**Percent by Weight Passing**
6 Inch	100
¾ Inch	70 – 100
No. 40	15 – 70
No. 200	5 – 25
Maximum Liquid Limit	**Maximum Plastic Index**
35	10

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

Thicker structural fill lifts, up to 12-inches, could be used if the contractor can demonstrate achieving required density. Moisture contents greater than 3 percent of optimum moisture are acceptable if the soil lift is stable and required relative compaction can be attained in the soil lift and succeeding lifts.

7.4.1 Rock Fill

It is anticipated that a portion of the onsite soils encountered at the Arrow Creek Well #1 may meet the requirements for a rock fill. Where less than 70 percent passes the ¾-inch sieve, soils are too coarse for standard density testing techniques, and shall be referred to as a **rock fill.** If the use of rock fill is anticipated, the following construction recommendations shall be followed during the placement:

- Particles up to 12-inches in diameter can be incorporated in fill areas, provided they are placed at least 1-foot below subgrade elevations. Material placed in the upper 1 foot of subgrade elevation, shall consist of structural fill containing no particles greater than 6-inches in diameter.

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

- Prior to densification, the moisture content of the fraction of the rock fill passing the ¾-inch sieve should be plus or minus 3 percent of optimum. Higher moisture contents are acceptable if the soil lift is stable and required compaction can be obtained in succeeding fill lifts.

  Density shall be established by a proof rolling program consisting of at least five complete passes over the fill layer with a minimum 20-ton roller (825 Caterpillar Sheepsfoot compactor, or equivalent). Monitoring of the proof-rolling program should be provided to establish that no significant increase in measured density is occurring with subsequent passes prior to terminating compaction efforts. The rolling pattern established shall be reported and shall include: number of passes (each way), equipment used, and thickness of fill lift. Moisture contents should be reported as part of the construction observation and testing program. The final surface should be smooth, firm and exhibit no signs of deflection. Granular soils with particles up to 12-inches in diameter can be placed in maximum 18-inch lifts.

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

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

7.4.2 Undocumented Fill STMGID Well #11 and Copper Cloud BPS

Undocumented fill is classified as fill not monitored or tested by a licensed construction materials testing consultant or firm. As part of the previous development grading, fill was placed at the STMGID Well #11 and Copper Cloud BPS. Fill thicknesses range from 2 to 4 feet. If documentation regarding the placement of this material is not available, remedial earthwork will be required in structural areas where undocumented fill soils are present.

Determining the in-place relative density of the existing fill was not completed as part of this exploration. Undocumented fill and should be completely removed and replaced with properly densified structural fill prior to placement of structural elements. The removal should extend at least 3 feet laterally from the outside edge of the structural element. Structural fill placement should be completed in accordance with the recommendations presented in Sections 7.4 (Grading and Filling) of this report. It is anticipated that the existing fill soils encountered during the subsurface exploration will meet the requirements for a structural fill[3].

When undocumented fill is present below structural elements the risk of settlement and differential settlement are increased due to potential variability of densification. Slabs are especially susceptible to damage associated with differential settlement of undocumented fills. Due to the increased risk for settlement, it is

[3] It should be noted that fill soils may vary from those encountered during the subsurface exploration.

recommended that a representative from CME be present to completed **full-time** observations and density testing during removal and placement.

7.4.3 Reuse of Onsite Materials

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

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

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

3) **Erosion Control:** If excavated material contains cobble or boulder sized particles, not meeting the minimum requirements for structural fill, this material could be screened to remove finer particles, and stockpiled onsite for future use in erosion control areas[4] designated by the project Civil Engineer. Larger boulders may require additional splitting to accommodate size requirements for the project design.

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

7.4.4 Trenching and Confined Excavations

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

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

[4] Rock used for erosion control purposes should meet the requirements of ASTM D4992-07 (Standards for Evaluations of Rock to be used for Erosion Control.

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

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

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

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

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

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

7.4.5 Excavations for Underground Utilities or Vaults

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

Excavation refusal was encountered at depths ranging from 4 to 7 feet at the Arrow Creek Well #1 site. Excavations for utilities or vaults deeper than 4 feet bgs at the Arrow Creek Well 1 site may require the use of a Caterpillar D8 with single shank to loosen cemented soils, large trackhoe, or hoe-ram.

Trench preparation shall consist of removing all loose soil particles from the bottom of the trench created during excavation to expose a firm non-yielding soils surface.

7.4.6 Trench Bedding and Backfill

Any material used as pipe bedding or trench backfill should meet the minimum requirements of the TMWA Design and Construction Standards (TMWADCS) and the SPPWC (2012).

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

Intermediate trench backfill[5] should be placed in (loose) lifts not exceeding 8-inches thick, and densified to at least 90 percent relative compaction.

7.4.7 Pipe Zone Bedding

Pipe zone bedding should consist of Class A backfill (Table 200.03.02-1, SPPWC), or other TMWADCS approved alternate. Class A backfill can be used in trenches which are bottomed above the existing groundwater elevation (assumed to be the predominate trench condition encountered). If groundwater conditions differ from those encountered during the subsurface exploration, alternate recommendations for pipe zone backfill can be provided.

7.4.8 Intermediate Trench Backfill

Trench backfill may consist of native granular soils which are free of debris and organic matter, with a maximum particle size less than 6 inches and a sand equivalent value of not less than 25.

7.5 Foundation Recommendations

7.5.1 Foundation Grade Soils Preparation

Foundation grade soils preparation will be dependent on the final location of the proposed structure, structure type, foundation grade soils conditions, and anticipated structural loads. Field density testing at foundation grade shall be completed for foundations bottomed in properly placed ***documented fill soils*** or native subgrade soils.

The upper 6 inches of the foundation grade soils shall be scarified and densified to at least 90 percent relative compaction.

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

7.5.2 Foundation Design

Foundation loads were not available at the time this report was prepared and are anticipated to be light to moderate for this structure. Once structural loads and foundation depths are known, finalization of the foundation design recommendations can be completed.

Provided that the foundation soils preparation has been performed in accordance with the recommendation given in Section 7.5.1 (Foundation Grade Soils Preparation), the foundation design parameters presented in Table 5 (Preliminary Foundation Design Parameters) can be utilized for the design of individual column footing and continuous wall footings.

Table 5 – Preliminary Foundation Design Parameters	
Allowable Bearing Pressures (psf)(1,2):	
Footings bottomed at least 2 feet(3) below the finished grade on properly compacted structural fill or on a suitable native bearing strata.	2,500 to 3,000
Allowable Friction Coefficient:	
Between foundation bottom and supporting soil consisting of properly compacted structural fill or native granular soils	0.40
Allowable Passive Soil Pressure (psf)(1)	
Backfill soils consisting of properly compacted structural fill	350(4)

(1) (psf)-Pounds per square foot

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

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

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

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

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

<u>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.</u>

7.5.3 Static Settlement

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

Once loading is determined, settlement can be estimated. However, based on the loading assumptions of this report and the anticipated foundation grade material, settlement on the order of ¾-inch or less is anticipated. Differential settlement for foundations with similar loads is anticipated to be about ½ of the total settlement provided the foundations are all bottomed on similar material (e.g. all on suitable native material or properly compacted structural fill).

7.6 Lateral Earth Pressure

Static lateral earth pressures are dependent on the relative rigidity and allowable movement of the retaining structure as well as the strength properties of the backfill soil and drainage conditions behind the retaining wall. The lateral earth pressure is strongly dependent on the lateral deformations which occur in the soil.

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

Table 6 – Preliminary Lateral Earth Pressure Values

STATIC LATERAL EARTH PRESSURE		
Earth Pressure Condition	**Earth Pressure Coefficient**	**Equivalent Fluid Density (psf) (1,2)**
Active (P_a)	0.29 (K_a)	35
Passive (P_p)	-	350(3)

(1) Pounds per square foot per foot of depth
(2) Lateral pressures for level backfill calculated using an average of the Rankine and Coulomb Equations for active/passive earth pressure. Assuming maximum unit weight of 125 pcf and a friction angle of at least 34 degrees.
(3) Assumes a factor of safety of 1.5.

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

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

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

Existing fill soils or native granular soils meeting the requirements for an imported structural fill may be used as backfill. The backfill shall extend laterally behind the retaining wall at least the height of the retaining wall.

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

7.7 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 7.4). Prior to construction, the upper six inches of the slab subgrade soils should be scarified to a minimum depth of 6 inches, uniformly moisture conditioned to within 3 percent of optimum moisture content and densified to at least 90 percent relative compaction. The subgrade should be protected against drying until the concrete slab is placed.

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

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

7.8 Corrosion Potential

Corrosion testing was completed for three samples (one sample from each site). Silver State Analytical Laboratories completed testing for soluble sulfate, resistivity, and pH on selected samples of native soils. 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 D2791A/SM4500E):** Soluble sulfate were generally not detected. This indicates that the native onsite soils have a negligible sulfate exposure to concrete.

- **pH (EPA 9045D):** The paste pH test results are ranged from 6.61 to 7.91 indicating a moderate potential of corrosion for soils in direct contact with ferrous metals (Baboian et. al, 2006).

- **Resistivity (ASTM G57):** Resistivity results ranged from 2,490 to 6,610 ohms.cm indicate that the site soils are have a moderate to high potential for corrosion for ferrous metal in direct contact with theses soils.

7.9 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 proposed pump stations is recommended. The slope gradient can be reduced to

2 percent for impervious surfaces, such as concrete slabs-on-grade and pavement constructed adjacent to the pump station.

8.0 LIMITATIONS

This report has been prepared in accordance with generally accepted local geotechnical practices. The analyses and recommendations submitted are based upon field exploration performed at the locations shown on Plates A-1 in Appendix A of this report. This report does not reflect soils variations that may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary. Sufficient construction observation should be completed in all phases of the project related to geotechnical factors to document compliance with our recommendations.

This report has been prepared to provide information allowing the engineer to design the project. The owner/project manager is responsible for distribution of this report to all designers and contractors whose work is affected by the recommendations contained herein. In the event of changes in the design, location, or ownership of the project after presentation of this report, our recommendations should be reviewed and possibly modified by the geotechnical engineer[7]. 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[8].

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

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

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

[7] 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.

[8] 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.

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APPENDIX A

REFERENCE: WASHOE COUNTY GIS (https://www.washoecounty.us/gis/), ACCESSED JULY 24, 215

CME CONSTRUCTION MATERIALS ENGINEERS INC.

6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

STANTEC CONSULTING
ARROWCREEK DROUGHT PROJECT
FIELD EXPLORATION LOCATION MAP-ARROWCREEK WELL #11
WASHOE COUNTY, NEVADA

PROJECT NO.: 1738

DATE: 7/24/2015

PLATE

A-1a

V:\Active\1738\autocad\soil classification chart and rock description.dwg

LEGEND

APPROXIMATE TEST PIT LOCATION

TP-1

REFERENCE: WASHOE COUNTY GIS (https://www.washoecounty.us/gis/), ACCESSED JULY 24, 215

CME CONSTRUCTION MATERIALS ENGINEERS INC.

6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

STANTEC CONSULTING
ARROWCREEK DROUGHT PROJECT
FIELD EXPLORATION LOCATION MAP-COPPER CLOUD BPS
WASHOE COUNTY, NEVADA

PROJECT NO.: 1738

DATE: 7/24/2015

PLATE

A-1b

V:\Active\1738\autocad\soil classification chart and rock description.dwg

REFERENCE: WASHOE COUNTY GIS (https://www.washoecounty.us/gis/), ACCESSED JULY 24, 215

V:\Active\1738\autocad\soil classification chart and rock description.dwg

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

STANTEC CONSULTING
ARROWCREEK DROUGHT PROJECT
FIELD EXPLORATION LOCATION MAP-STMGID WELL#11
WASHOE COUNTY, NEVADA

PROJECT NO.: 1738

DATE: 7/24/2015

PLATE

A-1c

LOG OF TEST PIT NO. TP-1 (ARROWCREEK WELL #1)

PROJECT ARROWCREEK DROUGHT PROJECT **EQUIPMENT TYPE** JOHN DEERE 310 SG RUBBER-TIRE
CLIENT STANTEC CONSULTING
LOCATION SOUTHWEST SIDE OF EXISTING DRIVEWAY NEAR PAVED CART PATH
PROJECT NO. 1738 **DATE** 04/02/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** ≅5,155' (PLATE A-1a)

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'-½': SANDY FAT CLAY, little fine to medium sand, few angular fine gravels, moderately plastic, strong brown							
	SC						MOIST	½'-7': CLAYEY SAND WITH GRAVEL, sine fine to coarse sand, little fine to coarse subangular gravel, few subrounded cobbles and boulders up to 18-inches nominal diameter, yellow brown							
2				B	1B				17.8	32	12			8.6	A,G
4															
6															
7								REFUSAL AT 7 FEET, NO FREE WATER ENCOUNTERED Note: Large diamter cobbles and boulders may not have been collected during sampling.							
8															
10															
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-2a

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-2 (ARROWCREEK WELL #1)

PROJECT ARROWCREEK DROUGHT PROJECT **EQUIPMENT TYPE** JOHN DEERE 310 SG RUBBER-TIRE
CLIENT STANTEC CONSULTING
LOCATION EASTSIDE OF EXISTING DRIVEWAY NORTH OF EX. WELL HOUSE
PROJECT NO. 1738 **DATE** 04/02/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** ≅5,109' (PLATE A-1a)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SC		X	B	2A		MOIST	0'-1': CLAYEY SAND, some fine to coarse sand, few rounded to subrounded gravels, moderately plastic, strong brown	47.7	32	12			16.5	A, G
	SC						MOIST	Note: Redox staining at 1 foot							
2			X	B	2B			1'-5.25': CLAYEY SAND WITH GRAVEL, some fine to coarse sand, little fine to coarse angular to subangular gravel and cobboles, few boulders up to 16 inches nominal diameter, yellow brown Note: Possible argillic zone, cemented clasts of fine gravel, sand and clay visible at 2 feet, at 5 feet becomes increasingly difficult to excavate							
4															
								REFUSAL AT 5.25 FEET, NO FREE WATER ENCOUNTERED Note: Large diamter cobbles and boulders may not have been collected during sampling.							
6															
8															
10															
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-2b

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-3 (ARROWCREEK WELL #1)

PROJECT ARROWCREEK DROUGHT PROJECT **EQUIPMENT TYPE** JOHN DEERE 310 SG RUBBER-TIRE
CLIENT STANTEC CONSULTING
LOCATION NORTHWEST OF EXISTING DRIVEWAY WEST OF EX. WELL HOUSE
PROJECT NO. 1738 **DATE** 04/02/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** ≅5,112' (PLATE A-1b)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SC			B	3A		MOIST	0-2': CLAYEY SAND WITH GRAVEL, mostly ine to medium sand, little subangular gravel, few cobbles, trace boulders up to 18-inches nominal diamter, strong brown							
2	GC						moist	2'-4': COBBLES AND BOULDERS, CLAYEY GRAVEL WITH SAND, mostly fine to coarse subrounded gravel, few cobbles and boulders up to 36 inches nominal diamter, little fine to coarse sand, moderately cemented, yellowish brown Note: Argillic zone encoutered at 2 feet, redox staining visible							
4								REFUSAL AT 4 FEET, NO FREE WATER ENCOUNTERED Note: Large diamter cobbles and boulders may not have been collected during sampling.							
6															
8															
10															
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-4 (COPPER CLOUD BOOSTER PUMP)

PROJECT ARROWCREEK DROUGHT PROJECT **EQUIPMENT TYPE** JOHN DEERE 310 SG RUBBER-TIRE
CLIENT STANTEC CONSULTING
LOCATION NEAR THE SE CORNER 10 FEET S. OF THE FENCE LINE
PROJECT NO. 1738 **DATE** 04/02/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** ≅5,570' (PLATE A-1b)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SM						MOIST	0'-3½': <u>SILTY SAND WITH GRAVEL FILL</u>, mostly fine to medium sand, litthe subangular gravel, brown							
3½	SM						MOIST	3½'-9': <u>SILTY SAND WITH GRAVEL</u>, some fine to coarse sand, some fine to coarse subangular gravel,few cobbles, trace boulders upt to 16 inches nominal diameter, brown							
				B	4A				13.5	26	1			11.2	A, G
8								Note: Difficult to excavate at 8 feet, broke bucket tooth on existing embeded boulder							
9	SM						MOIST	9'-12': <u>SILT SAND WITH GRAVEL AND COBBLES</u>, mostly fine to medium sand, little subangular gravel and cobbles up to 6-inches nominal diamter, brown							
12								TERMINATED AT 12 FEET, NO FREE WATER ENCOUNTERED							

GROUNDWATER

DEPTH	HOUR	DATE
N.E.		

SAMPLE TYPE
B - Bulk Sample

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

PLATE NO.: A-2d

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-5 (STMGID WELL #11)

PROJECT ARROWCREEK DROUGHT PROJECT **EQUIPMENT TYPE** JOHN DEERE 310 SG RUBBER-TIRE
CLIENT STANTEC CONSULTING
LOCATION NORTH EAST OF THE EXISTING WELL HOUSE
PROJECT NO. 1738 **DATE** 04/02/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** ≅4,825' (PLATE A-1c)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SC						MOIST	0'-2': <u>CLAYEY SAND FILL</u>, some fine to coarse sand, few fine angular gravels, greyish brown Note: Trace construction debris encountered at 2 feet, including warning tape.	22.9	28	9			11.0	A, G
2	GC			B	5B		MOIST	2'-4': <u>CLAYEY GRAVEL WITH SAND FILL</u>, some fine to coarse subrounded gravel and cobbles, some fine to coarse sand, low plasticity, brown Note: Survey nail encountered at 4 feet.							
4	SC			B	5C		MOIST	4'-5': <u>CLAYEY SAND WITH GRAVEL</u>, some fine to medium sand, some angular gravel, weakly cemented, yellow brown							
5	GM						MOIST	5'-8': <u>SILTY GRAVEL WITH SAND AND COBBLES</u>, mostly fine to coarse angular gravel, some fne to coarse sand, few subrounded cobbles up to 6 inches nominal diamter, trace boulders up to 38" nominal dimater, non-plastic, brown							
8								REFUSAL AT 8 FEET, NO FREE WATER ENCOUNTERED Note: Large diamter cobbles and boulders may not have been collected during sampling.							
10															
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-2e

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-6 (STMGID WELL #11)

PROJECT ARROWCREEK DROUGHT PROJECT **EQUIPMENT TYPE** JOHN DEERE 310 SG RUBBER-TIRE
CLIENT STANTEC CONSULTING
LOCATION NW OF EXISTING WELL HOUSE AND SW OF BOOSTER PUMPS
PROJECT NO. 1738 **DATE** 04/02/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** ≅4,828' (PLATE A-1c)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SC						MOIST	0'-4': CLAYEY SAND FILL, some fine to coarse sand, few subrounded to rounded gravels and cobbles, low plasticity, strong brown							
2				B	6A				27.6	26	10			10.3	A, G
4	SC						MOIST	4'-10': CLAYEY SAND WITH GRAVEL AND COBBLES, some fine to medium snad, some subangular gravewl and cobbles, few boulders up to 12 inches nominal diamter, low plasticity, brown							
6															
							MOIST								
				B	6B				21.5	26	10			5.4	A, G
8															
10								TERMINATED AT 10 FEET, NO FREE WATER ENCOUNTERED							
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-2f

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-7 (STMGID WELL #11)

PROJECT ARROWCREEK DROUGHT PROJECT **EQUIPMENT TYPE** JOHN DEERE 310 SG RUBBER-TIRE
CLIENT STANTEC CONSULTING
LOCATION W OF EXISTING WELL HOUSE AND SW OF BOOSTER PUMPS
PROJECT NO. 1738 **DATE** 04/02/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** ≅4,830' (PLATE A-1c)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0	SC						MOIST	0'-2': CLAYEY SAND FILL, some fine to medium sand, few subrounded to rounded gravels, few cobbles, low plasticity, strong brown							
2	SC						MOIST	2'-8½': CLAYEY SAND WITH GRAVEL AND COBBLES, some fine to medium sand, some subangular gravel and cobbels, non-plastic, brown							
4															
6															
8															
	SC						MOIST	8½'-10': CLAYEY SAND, mostly fine to coarse sand, little fine angular gravel, low plasticity, yellow brown							
10								TERMINATED AT 10 FEET, NO FREE WATER ENCOUNTERED							
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-2g

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.

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

STANTEC
ARROW CREEK DROUGHT PROJECT
SOIL CLASSIFICATION CHART
WASHOE COUNTY , NEVADA

PROJECT NO.:1738 DATE: 04/24/15

PLATE A-3

APPENDIX B

Particle Size Distribution Report

	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○		10.8	14.1	11.6	19.7	18.6	17.8	
□	0.0	11.0	3.7	5.1	12.1	20.4	47.7	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	32	20	24.0342	2.5492	1.3055	0.2475				
□	43	17	4.3515	0.1873	0.0882					

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ clayey sand with gravel	4/8/2015	SC	
□ clayey sand	4/8/2015	SC	

Project No. 1738 **Client:** STANTEC CONSULTING

Project: TMWA ARROWCREEK DROUGHT PROJECT

○ **Location:** Arrowcreek Well #1 **Depth:** 0.5-4' **Sample Number:** TP-1 1B

□ **Location:** Arrowcreek Well #1 **Depth:** 0-1' **Sample Number:** TP-2 2A

Remarks:

○ MOISTURE CONTENT - 8.6%

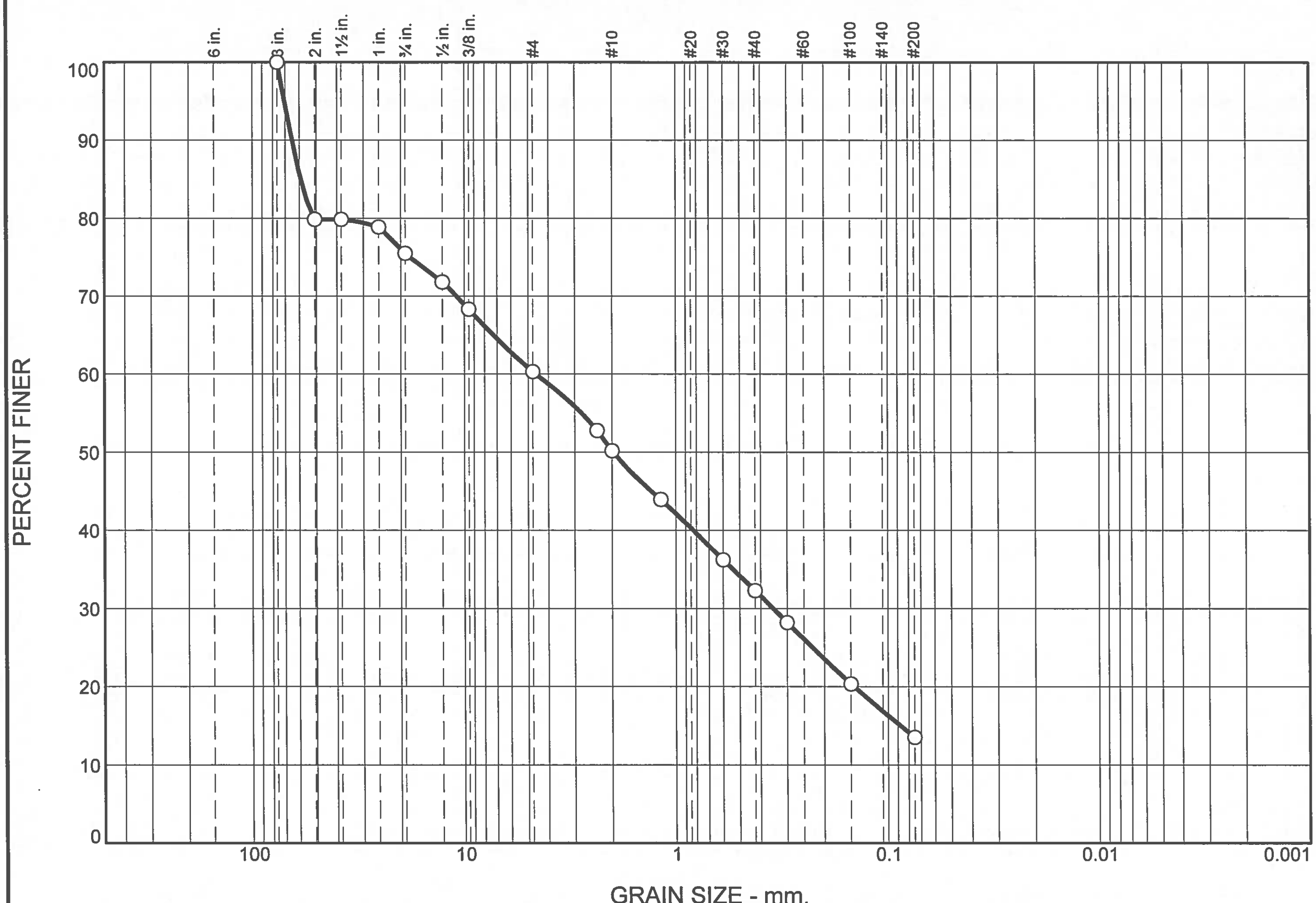
□ MOISTURE CONTENT - 16.5%

CME

Figure B-1

Tested By: A. HAMPEL **Checked By:** S. HEIN

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
o	0.0	24.5	15.2	10.1	17.9	18.8	13.5	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o	26	25	59.0823	4.5900	1.9760	0.3495	0.0877			

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
o silty sand with gravel	4/8/2015	SM	

Project No. 1738 **Client:** STANTEC CONSULTING

Project: TMWA ARROWCREEK DROUGHT PROJECT

o **Loc.:** Copper Cloud Booster Pump Station **Depth:** 3.5'-6' **Sample No.:** TP-4 4A

CME

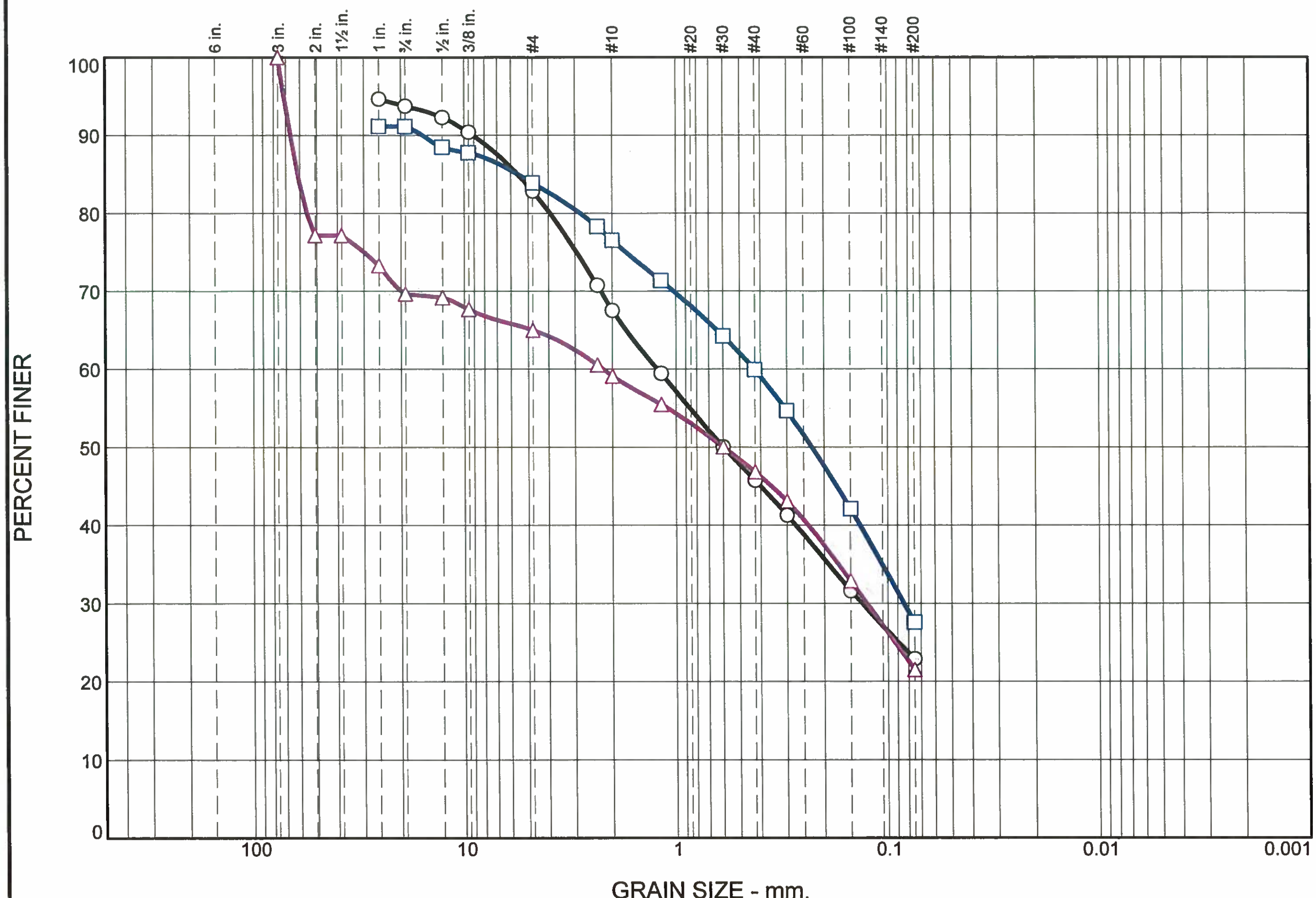
Remarks:

o MOISTURE CONTENT - 11.2%

Figure B-2

Tested By: A. HAMPEL **Checked By:** S. HEIN

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○			11.0	15.3	21.8	22.8	22.9	
□			7.3	7.4	16.6	32.3	27.6	
△	0.0	30.3	4.7	5.9	12.3	25.3	21.5	

╳	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	28	19	5.5797	1.2279	0.5983	0.1329				
□	26	16	5.5527	0.4270	0.2280	0.0839				
△	26	16	61.3223	2.2142	0.5987	0.1257				

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ clayey sand	4/8/2015	SC	
□ clayey sand	4/8/2015	SC	
△ clayey sand with gravel	4/8/2015	SC	

Project No. 1738 **Client:** STANTEC CONSULTING

Project: TMWA ARROWCREEK DROUGHT PROJECT

○ **Location:** STMGID WELL #11 **Depth:** 0-2' **Sample Number:** TP-5 5A

□ **Location:** STMGID WELL #11 **Depth:** 0-4' **Sample Number:** TP-6 6A

△ **Location:** STMGID WELL #11 **Depth:** 7'-8' **Sample Number:** TP-6 6B

CME

Remarks:

○ MOISTURE CONTENT - 11.0%

□ MOISTURE CONTENT - 10.3%

△ MOISTURE CONTENT - 6.4%

Figure B-3

Tested By: A. HAMPEL **Checked By:** S. HEIN

APPENDIX C

USGS Design Maps Summary Report

User–Specified Input

Report Title Copper Cloud
Thu April 23, 2015 21:56:11 UTC

Building Code Reference Document 2012 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 39.40246°N, 119.81801°W

Site Soil Classification Site Class D – "Stiff Soil"

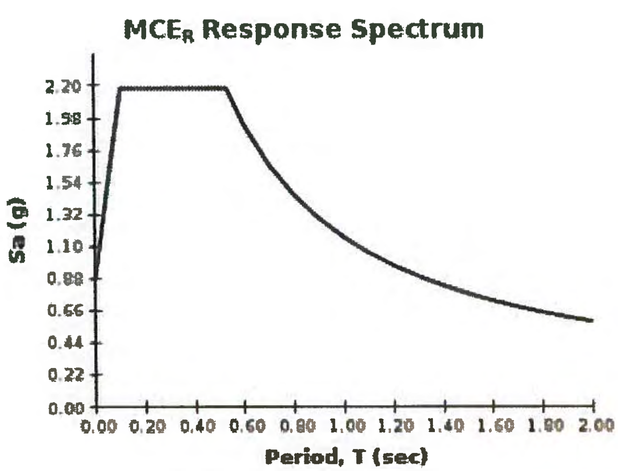
Risk Category I/II/III

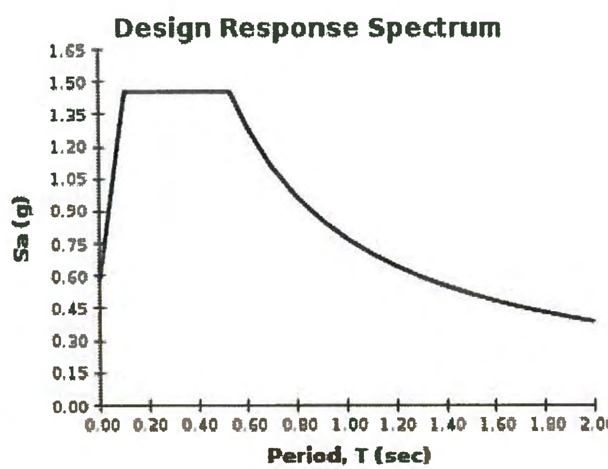
USGS–Provided Output

S_S = 2.182 g S_{MS} = 2.182 g S_{DS} = 1.455 g

S_1 = 0.775 g S_{M1} = 1.163 g S_{D1} = 0.775 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 International Building Code (39.40246°N, 119.81801°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 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1] $S_S = 2.182$ g

From Figure 1613.3.1(2) [2] $S_1 = 0.775$ 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 $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

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

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

For Site Class = D and S_S = 2.182 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

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

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

For Site Class = D and S_1 = 0.775 g, F_v = 1.500

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 2.182 = 2.182$ g

---

Equation (16-38): $S_{M1} = F_vS_1 = 1.500 \times 0.775 = 1.163$ g

---

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 2.182 = 1.455$ g

---

Equation (16-40): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 1.163 = 0.775$ 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} = 1.455 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} = 0.775 g, Seismic Design Category = D

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

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = E

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

References

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

USGS Design Maps Summary Report

User–Specified Input

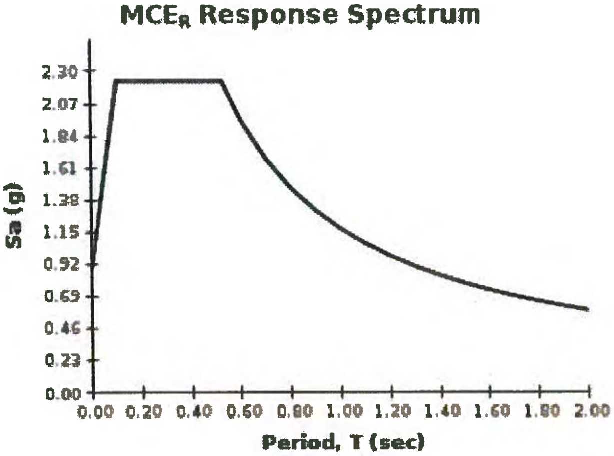
Report Title	arrow creek well 1 Thu April 23, 2015 21:55:08 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	39.41838°N, 119.80756°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

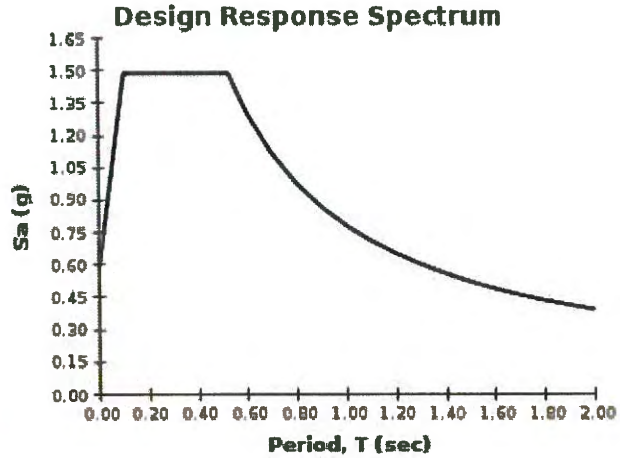
S_S =	2.230 g	**S_{MS} =**	2.230 g	**S_{DS} =**	1.487 g
S_1 =	0.782 g	**S_{M1} =**	1.173 g	**S_{D1} =**	0.782 g

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

MCE_R Response Spectrum



Design Response Spectrum



USGS Design Maps Detailed Report

2012 International Building Code (39.41838°N, 119.80756°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 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1]	$S_S = 2.230$ g
From Figure 1613.3.1(2) [2]	$S_1 = 0.782$ 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 $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
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 = 2.230 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 = 0.782 g, F_v = 1.500

Equation (16-37):	$S_{MS} = F_aS_S = 1.000 \times 2.230 = 2.230$ g
Equation (16-38):	$S_{M1} = F_vS_1 = 1.500 \times 0.782 = 1.173$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):	$S_{DS} = ⅔ S_{MS} = ⅔ \times 2.230 = 1.487$ g
Equation (16-40):	$S_{D1} = ⅔ S_{M1} = ⅔ \times 1.173 = 0.782$ 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} = 1.487 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} = 0.782 g, Seismic Design Category = D

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

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = E

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

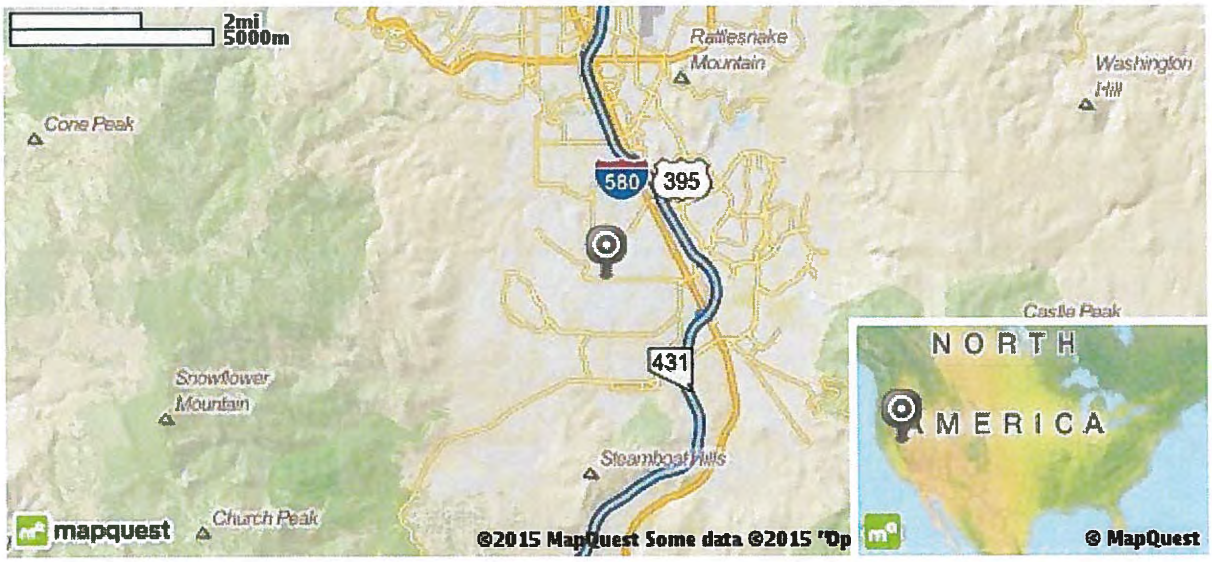
References

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

USGS Design Maps Summary Report

User–Specified Input

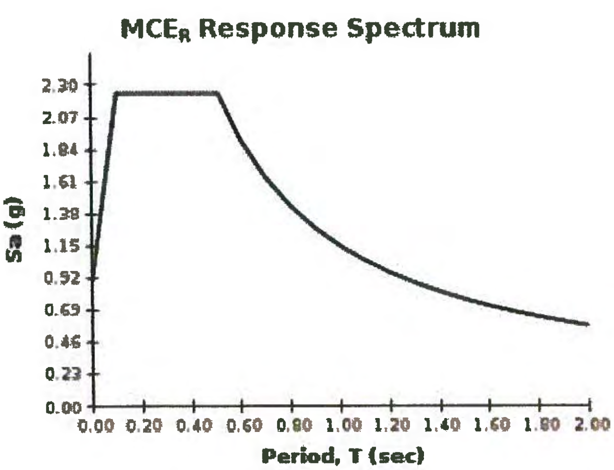
Report Title	stmgid well 11 Thu April 23, 2015 21:53:50 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	39.4185°N, 119.7846°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

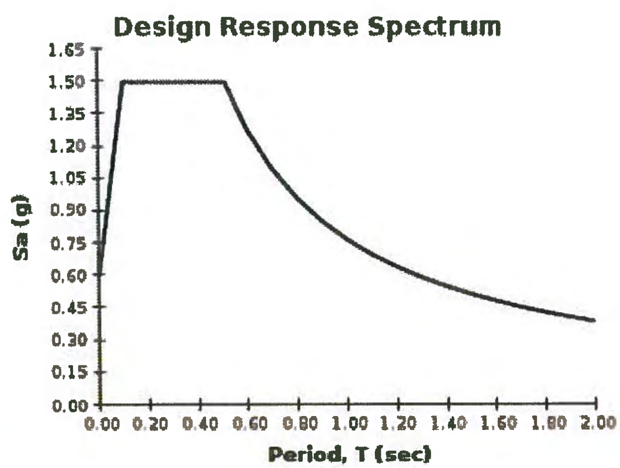


USGS–Provided Output

$S_S =$ 2.241 g	$S_{MS} =$ 2.241 g	$S_{DS} =$ 1.494 g
$S_1 =$ 0.766 g	$S_{M1} =$ 1.149 g	$S_{D1} =$ 0.766 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.





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

USGS Design Maps Detailed Report

2012 International Building Code (39.4185°N, 119.7846°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 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1]	$S_S = 2.241$ g
From Figure 1613.3.1(2) [2]	$S_1 = 0.766$ 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 $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
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 = 2.241 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 = 0.766 g, F_v = 1.500

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 2.241 = 2.241$ g

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Equation (16-38): $S_{M1} = F_vS_1 = 1.500 \times 0.766 = 1.149$ g

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Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.241 = 1.494$ g

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Equation (16-40): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.149 = 0.766$ g

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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} = 1.494 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} = 0.766 g, Seismic Design Category = D

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

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = E

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

References

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