Geotechnical Investigation

State Farm TMWA Well House
Reno, Nevada

Prepared for:

Stantec

May 2015
File: 1670

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May 18, 2015
Project No: 1670

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RE: Geotechnical Investigation
State Farm TMWA Well House
Reno, Nevada

Dear Mr. Angst:

Construction Materials Engineers, Inc. (CME) is pleased to submit the following Geotechnical Investigation Report for the proposed State Farm TMWA Well House located along the west side of Longley Lane, near the intersection with Innovation Drive.

1.0 INTRODUCTION

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

The proposed site is located on Washoe County Assessor Parcel Number 025-480-40, contained in Section 31, T19N, R20E, M.D.M. The area covered by this report as well as some of the existing site improvements are presented on Plate A-1.

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

2.0 SITE DESCRIPTION

The subject site consists of an approximate 7,000 square foot, rectangular shaped parcel adjoining Longley Lane. The parcel is bordered by Longley Lane to the east, commercial/office development to the west and south, and undeveloped property to the north. Site access is from Longley Lane.

Existing site improvements include:

- Fencing along the southern and western property lines;
- Landscaping including several trees and numerous small bushes;
- An approximate 4 to 5 foot deep drainage ditch located adjacent to Longley Lane; and

- Sidewalk and curb and gutter located along Longley Lane.

Toward the central to western side of the parcel, it appears that the existing terrain is 4 to 5 feet higher than the adjacent property to the north and likely represents an elevated fill pad. A drainage ditch is located along the eastside of the parcel adjacent to Longley Lane. The drainage ditch side slopes and fill slope along the north side of the parcel appear to have a gradient of about 3H:1V or flatter.

Photo #1: Looking west from Longley Lane showing existing landscaping on elevated fill pad.

Photo #2: Looking south along existing drainage ditch.

3.0 PROJECT DESCRIPTION

It is understood that a TMWA well house with a paved parking area and access road will be constructed on the parcel. The footprint of the building will be approximately 600 square feet. The finished floor elevation has not been determined but is assumed to be above the adjacent street grade. Building foundation grade is anticipated at about 2 feet below the top of the elevated fill pad. It is assumed the building will have masonry walls with a concrete floor slab-on-grade. Structural loading is assumed to be light to moderate.

The well house will also have a paved access road from Longley Lane and paved parking area. It is assumed that storm drain piping will be placed in the existing drainage ditch.

4.0 FIELD EXPLORATION

The subsurface field exploration, completed in August 2014, consisted of excavating three test pits to maximum depths of 11 feet below the existing ground surface (bsg). Test pits were excavated using a John Deere 310SG rubber-tired backhoe equipped with a 36-inch bucket. Test pits were located in the field by visual sighting and/or measuring from existing features at the site. Approximate test pit locations are presented on Plate A-1.

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 laboratory.

The test pit logs are included as Plate A-2. The elevations shown on the test pit logs were obtained by interpolation between contour lines shown on the attached Field Exploration Location Map (Plate A-2). Topographic information is obtained from Washoe County GIS. The 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 Plate A-3.

Wildcat Dynamic Cone Penetrometer (DCP) testing was completed in Test Pit TP-1. DCP is a portable, manually operated device used to continually measure the consistency and relative strength of loose to medium dense sandy soils and very soft to very stiff fine grained silts and clays. The DCP assembly consists of a 35 lb safety hammer with a 15 inch drop, 1 meter hollow drive rods (sounding rods), and cone tip with a nominal area of 10 cm^2. The DCP probe uses a fluid (cellulose slurry) injection system to reduce friction along the drive rods to allow the drive energy to reach the cone tip.

The dynamic cone resistance (i.e. blow counts) is recorded in the field at 10 cm intervals. Dynamic cone resistances are converted to Standard Penetration Test (SPT) N-value using the Wildcat Dynamic Cone Log software. The N-value is a measure of the standard penetration resistance of the soil and provides an indication of the relative density of the underlying soil strata.

5.0 LABORATORY TESTING

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

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

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

Results from our laboratory test program can be found on the test pit logs (Plates A-2) and in Appendix B.

6.0 SUBSURFACE AND GROUNDWATER CONDITIONS

Sedimentation in the Truckee Meadows has been in progress at varying rates since the formation of the block faulted basin. Most of the sediments, including the coarse grained, gravelly sands that underlie the majority of the Truckee Meadows, were deposited quite abruptly in the post-glacial period during torrential flooding. With the advent of a warm, drier climate, the volume and size distribution of sediment transported was greatly reduced and the sedimentation process became largely limited to the reworking of earlier deposits.

A review of the Geologic map of the Mount Rose NE Quadrangle (Bonham and Rogers 1983), indicates that the project lies in alluvial bajada deposits. These deposits typically consist of thin sheet-like aprons of fine to medium grained clayey sand, which are intercalated with muddy, medium pebble gravel, deposits of low gradient streams that reworked older gravelly outwash and alluvial fan deposits.

The uppermost soil horizon encountered in Test Pits TP-2 and TP-3 to depths ranging from 5 to 5½ feet below the existing ground surface (bgs) consisted of undocumented fill classified as clayey sand **(SC).** Fill soils contained some organics and other debris including PVC piping. The uppermost soil horizon was underlain by silty sand **(SM)** with a thickness of about 5 feet. This soil layer was weakly cemented with calcite stringers. The lowermost soil horizon encountered was poorly graded sand with silt **(SP-SM)** encountered to the depth explored.

Groundwater was encountered in TP-1 at depths of about 7½ feet bgs. Seasonal fluctuations in groundwater level may occur during periods of increased precipitation or recurrent irrigation. The groundwater level encountered with this investigation may change in the future.

7.0 SEISMIC CONDITIONS AND GEOLOGIC HAZARDS

The subject property is located in a moderate to intense seismically active area of the Western United States. The western region is subject to seismicity related to movement of the crustal masses (plate tectonics). The Wasatch Front in Salt Lake City, Utah, forms the eastern boundary of the Basin and Range physiographic province, and the eastern front of the Sierra Nevada Mountains, which is the western margin of the province. The project site lies near the eastern base of the Sierra Nevada, within the western extreme of the Basin and Range.

7.1 Faulting

To determine the location of mapped earthquake faulting trending through or near the project site, a review of the Mount Rose NE Quadrangle Earthquake Hazards Map (Szecsody, G. C., 1983) and the USGS Website: *Earthquake Hazards Program Quaternary Faults in Google Earth* was completed. These maps indicate that no mapped faults trend through the project site. The closest mapped fault trends in a northeasterly direction located approximately 1-mile east of the site.

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

7.2 Liquefaction

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

The referenced Earthquake Hazards Map indicates that the project site is located in a soil liquefaction susceptible area. Based on the soil types encountered with this investigation and reviewing another geotechnical investigation (SEA, 1997) completed near the project site, it is our opinion that the site has a moderate liquefaction potential. Based on the material properties, the poorly graded sand with silt **(SP-SM)** soil horizon encountered below the water table in Test Pit TP-1 may be susceptible to soil liquefaction during a major earthquake event. It is our opinion that ground settlement is possible if this soil horizon completely liquefied (Refer to Section 7.2.1).

In Nevada there is no specific policy that requires structures to be designed to mitigate for soil liquefaction. Such designs tend to be very costly and are usually limited to those structures with a public safety function including; fire and police facilities, hospitals or buildings with high occupancy. These buildings may include large commercial, retail, office and manufacturing facilities, schools, municipal or major governmental buildings. These types of structures present a significant potential for loss of life and/or are important enough, from a public safety standpoint, such that a design to mitigate for soil liquefaction may be warranted.

7.2.1 Seismic Settlement

During a seismic event, liquefaction is caused by a gradual build-up of excess pore water pressure. After the seismic event, this pore water pressure dissipates causing the soil layer to settle. To determine an approximate settlement quantity within the liquefiable soil layer, procedures by Tokimatsu and Seed (1987) were utilized. For the potentially liquefiable soil layer identified in the soil profile encountered within Test Pit TP-1, it is estimated that settlements on the order of 1 inch or more are possible. This estimated settlement is based on the assumption that the DCP blow counts obtained below a depth of 12.5 feet pertain to the poorly graded sand with silt **(SP-SM)** layer encountered above this depth. However, SPT blow counts encountered in the granular soil layers below the groundwater table from the referenced geotechnical investigation completed near this site indicated similar blow counts and, consequently, this is a reasonable assumption. Estimated settlements do not include potential settlements from deeper soil layers and additional field exploration would be required to determine total seismically induced settlements. It is advised that Tokimatsu and Seed indicate an error magnitude of up to 50 percent with their procedures, settlement estimates given by their procedure should be considered approximate.

7.2.2 Lateral Spread

Lateral displacement, or lateral spread, is the horizontal movement of soil layers as a consequence of soil liquefaction. Horizontal soil movement is due to the effect of dynamic earthquake-generated inertial forces and static gravitational forces. Lateral spread occurs on sloped terrain or as movement towards a nearby free face, such as a steep embankment or a stream bank. In general, the regional terrain is relatively flat and lateral spread potential due to sloping terrain is anticipated to be minor.

7.3 Slope Stability

Slope instability hazards occur in areas of active and/or relict mass wasting features (e.g. landslides, debris flows, rock falls). Site topography is gently sloping to the northeast at a gradient of less than 6 percent. No significant or major manufactured slopes are proposed. Therefore, the potential for slope instability across the subject site is judged to be 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 Definitions

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

The soil/bedrock profile classification is based on two criteria: density (primarily for soils based on SPT blow count data) or hardness (based on shear wave velocity primarily for bedrock sites). These two criteria have to be determined to a depth of 100 feet below the ground surface. A 100-foot deep boring or geophysical studies such as ReMi is required to define the soil profile in sufficient detail to determine the site classification. A 100-foot boring or geophysical studies was not part of our scope of services for this project. However, the IBC allows the use of a default site classification of D if the soil profile to a depth of 100-feet is not characterized and other geologic conditions do not exist that would justify a site classification of E or F.

Even though the subsurface soils maybe prone to liquefaction during a seismic event, a site-specific evaluation (Site Classification F) is not required because the structure is a one-story building and, therefore, the building period is less than 0.5 second (NEHRP, 1997). In accordance with ASCE 7-10, a site classification of D can be utilized for the building design if the period of the building is less than 0.5. Studies have shown that short-period ground motions, which would amplify movement for buildings with short response periods, are generally attenuated due to liquefaction whereas long-period ground motions may be amplified. Based on existing soils information data, a default Site Classification of D is deemed appropriate to use in the design of the structure.

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

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

Table 2 – Seismic Design Parameters	
PARAMETER DESCRIPTION	**Design Values**
Approximate Latitude of Site	39.540^0
Approximate Longitude of Site	119.815^0
Peak Ground Acceleration-MCE_R PGA (ASCE 7-10 Standard)	0.82 g
Spectral Response Acceleration at Short period (0.2 sec.) S_s (for Site Class B)	2.10 g
Spectral Response Acceleration at 1-second Period, S_1 (for Site Class B)	0.70 g
Site Class Selected for this Site	D
Site Coefficient F_a, decimal	1.0
Site Coefficient Fv, decimal	1.5
1) MCE_R PGA- Maximum credible earthquake geometric mean peak ground acceleration.	

8.0 RECOMMENDATIONS AND DISCUSSION

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

The primary design considerations are the presence of undocumented fill and the potential for soil liquefaction occurring during a major seismic event. Because the fill is undocumented (no placement records or density tests), it is recommended to be removed and replaced with structural fill.

8.1 General Information

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

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

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

The test pits were excavated by backhoe at the approximate locations shown on the site plan. Locations were determined in the field by approximate means. All test pits were backfilled upon completion of the field portion of our study. The backfill was compacted to the extent possible with the equipment on hand. However, 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 replaced with structural fill 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

All vegetation and topsoil should be stripped and grubbed from structural areas and removed from the site or used as topsoil in non-structural areas. Fill soils were encountered in the uppermost soil strata and are about 5 feet thick. It is recommended that these soils are completely removed below structural areas to native alluvium and replaced with structural fill. Existing fill soils shall be removed at least 3 feet laterally from any structural area or to a distance that equals the thickness of the fill soils below the structural area, whichever is greater. Native alluvium and structural fill shall be densified to the recommendations given in this report.

The entire root bulb should be removed as part of any tree removal. Large roots (greater than 2 inches in diameter) radiating from the tree bulb area, located within one foot of the final subgrade or foundation grade elevation, should be completely removed. Resulting excavations should be backfilled with structural fill.

All areas to receive structural fill or structural loading should be densified to at least 90 percent relative compaction in accordance with ASTM D 1557 for a minimum depth of 8 inches. It is recommended that soils have moisture contents of plus or minus 3 percent of optimum moisture (ASTM D1557) prior to densification. Moisture contents above 3 percent of optimum moisture will be acceptable if the soil horizon maintains its stability when subjected to construction equipment loads 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 prior to densification, the moisture content of the soils be determined to evaluate the need for moisture conditioning. After the densification process, a firm, stable surface should be produced. If unstable native soils due to excessive moisture content are encountered they should be removed and replaced with structural fill.

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

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 shall be free of vegetation, organic matter, and other deleterious material. Structural fill shall meet the requirements provided in Table 3.

Table 3 - Guideline Specification for Structural Fill	
Sieve Size	**Percent by Dry Weight Passing**
4 inch	100
¾ inch	70 -100
No. 40	15 - 60
No. 200	5 - 30
Maximum Liquid Limit	**Maximum Plastic Index**
40	10
Soluble sulfates:< 0.10 percent by weight of soil	

Other material types not meeting the specifications given in Table 3 may be acceptable as structural fill, as approved by the geotechnical engineer. Based on our laboratory test results, existing fill soils do not meet the requirements presented in Table 3 and can only be used as fill in non-structural areas. Based on our laboratory test results, native granular alluvium does meet the requirements presented in Table 3 and can be used as structural fill. The soil properties of all material anticipated to be used as structural fill shall be verified during construction with additional laboratory testing.

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

8.4 Trenching and Excavation

Excavations will require shoring or the excavation sidewalls shall be sloped to maintain adequate stability. Regulations amended in Part 1926, Volume 54, Number 209 of the Federal Register (Table B-1, October 31, 1989) requires that the temporary sidewall slopes be no greater than those presented in Table 4.

Table 4 - Maximum Allowable Temporary Slopes

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

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

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

On the basis of our exploration, it is our opinion that the bulk of the site soils above the water table appear to be predominately Type B, although variations will exist. Trenching below teh water table will be difficult in the poorly graded sands **(SP-SM)**. Sloughing of these soils should be anticipated and shoring as well as dewatering should be anticipated.

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

8.5 Foundation Grade Soil Recommendations

As recommended in Section 8.2 - Site preparation, existing fill soils encountered below the entire building pad including foundations shall be removed and replaced with structural fill. Based our field exploration, existing fill soils have a thickness of about 5 to 5 ½ feet. Two foundation grade soils preparation options are presented. The first option employs a reinforced foundation grade soil improvement approach to reduce the effects of potential differential settlements caused by soil liquefaction as presented in Section 8.5.1. If reducing the effects of potential differential settlements due to potential soil liquefaction is not chosen, than the foundation grade soil preparation presented in Section 8.5.2 is recommended. Bearing capacity failure due to soil liquefaction is not anticipated because of the depth of the anticipated liquefiable soil layers, anticipated foundation grade, and foundation width.

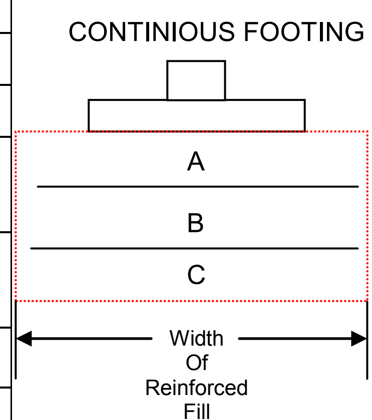
8.5.1 Reinforced Foundation Grade Soil Improvement Approach

Because of potential differential settlements across the building footprint caused by soil liquefaction during the design seismic event, a Mechanically Stabilized Earth Composite Raft Foundation (MSECRF) is recommended to be constructed below building foundations. It is difficult to predict and quantify the amount of ground surface deformation caused by liquefaction-induced differential settlement. Soil liquefaction mitigation would be difficult to implement and costly. The most destructive consequence of soil liquefaction is potential differential settlements across the building pad. The MSECRF is a method for improving the foundation grade soils by creating a stiffened platform that bridges the underlying weak soil layer. The intent of the MSECRF foundation support system is to provide a continuous uniform pad below the proposed structure footings and reduce the seismically induced differential settlement across the building footprint. MSECRF systems are comprised of geogrid layered with properly compacted structural fill. Table 5 provides a design summary of the geogrid locations and dimensions.

TABLE 5 – MSECRF Design Summary

Applied Bearing Pressure	Refer to Section 8.6
Continuous Foundation Width	1.5 feet (assumed)
Foundation Length	20 to 30 feet
Total Number of Geogrid Layers	3
Geogrid Type	Refer to Section 8.5.1
A: Distance from Footing to Uppermost Grid	12 to18 inches
B: Spacing between Geogrid Layers	12 inches
Width of Reinforced fill	7.5 feet [2]
C: Distance from Lowest Grid to bottom of Reinforced Fill	12 inches

MSECRF DIAGRAM [1]



1. The diagram provided in the table does not reflect actual project design conditions. It should only be used for clarification of the distance values provided in the table.
2. Assumes that the geogrid will be extended at least 3 feet past each edge of the slab/foundation

8.5.1.1 Construction Recommendations

Structural fill meeting the requirements in Table 6, shall be placed above, below and between each layer of geogrid. Structural fill shall be densified to at least 95 percent relative compaction. The thickness of the top layer of structural fill located directly below the foundation varies between 12 to 18 depending on underground utility locations. The best case construction scenario is not to cut the geogrid. However, if cutting is unavoidable an additional geogrid section shall be placed over the spliced area with a minimum overlap of 2 feet. The corners of the building should also be overlaid.

Geogrid can either have a triaxial or biaxial geometric shape and shall be composed of polymer. Tensar® TX5 and TX7 geogrid or a TerraGrid RX1200 (Hanes® Geo Components) could be used for this project. Any substitute geogrid shall meet the material properties of either one of these geogrids.

If foundation grade or subgrade soils are allowed to be exposed to inclement or freezing weather conditions, they may need to be scarified and recompacted or removed to expose suitable foundation or subgrade soils, and the resulting over-excavation backfilled with compacted structural fill. The bottom of all excavations should be dry and free of loose materials at the time of concrete placement.

Table 6 - Guideline Specification for MSECRF Structural Fill

Sieve Size	Percent by Dry Weight Passing
4 inch	100
¾ inch	70 -100
No. 4	35 - 70
No. 40	15 - 40
No. 200	2 - 20
Maximum Liquid Limit	**Maximum Plastic Index**
40	5
Soluble sulfates:< 0.10 percent by weight of soil	

8.5.2 Unreinforced Foundation Grade Soils Improvement Option

After the existing fill soils have been removed, structural fill shall be placed below foundation grade. Structural fill placement and material requirements shall follow the recommendations provided in Section 8.3.

8.6 Foundation Design

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

Table 7 – Foundation Allowable Bearing Pressures	
Loading Conditions	**Maximum Soil Net Allowable Bearing Pressures(1) (pounds per square foot)**
Dead Loads plus full time live loads	2,500
Dead Loads plus live loads, plus transient wind, or seismic loads.	2,700

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

Footings shall be set at least two feet below adjacent finished grade elevation for frost protection and confinement. Regardless of loading, individual column foundations and continuous spread foundations should be at least 18 inches wide, or as required by code.

Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. A friction factor of 0.40 may be utilized for sliding resistance at the base of the spread footing and a design value of 300 pounds per square foot per foot of depth is recommended for passive soil pressures. It should be understood that some lateral deformation on the order of 2 to 4 percent of the depth of embedment (Tomlinson, 1986) for a properly compacted backfill is required to mobilize the ultimate passive resistance. To reduce the amount of displacement required to develop the design passive pressure, a factor of safety of 1.5 was applied to the passive pressure and sliding resistance from their ultimate values.

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

8.7 Settlement

Due to the material characteristics of native soils, an elastic settlement response is expected and the majority of the settlement will occur rapidly, generally during the construction time frame for the building. Total static settlements are anticipated to be on the order of ¾ inch, or less. Differential settlement between foundations with similar loads and sizes is anticipated to be on the order of ½ of the total settlement.

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

8.8 Site Drainage

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

Stemwall backfill shall be densified to the requirements given in Section 8.3 to decrease permeability and reduce the potential for irrigation and storm water to enter under floor areas. This will also reduce the potential for settling of backfill soils causing a reduction in the slope gradient away from the structure.

8.9 Concrete Slabs

All concrete slabs should be directly underlain by at least 6-inches of Type II, Class B aggregate base complying with the specifications provided in SSPWC, 2012.

Type II cement should be used for all concrete work. A concrete mix with a maximum water/cementitious ratio of 0.5 should be utilized for all concrete work in contact with native soils, including foundations. In accordance with ACI 318, concrete exposed to freezing and thawing in a moist condition or to deicing chemicals should consist of a mix with a maximum of 0.45 water/cementitious ratio and minimum 28-day compression strength of 4500 psi. The contractor should submit a concrete mix design to the owner at least 10 working days prior to construction for approval.

The northern Nevada area is a region with low relative humidity. As a consequence, concrete flatwork is prone to excessive shrinking and curling. Concrete mix proportions and construction techniques, including the addition of excess water and improper curing, can adversely affect the finished quality of the concrete resulting in cracking, curling and spalling of slabs. We recommend that all placement and curing be performed in accordance with procedures outlined by the American Concrete Institute. Special considerations should be given to concrete placed and cured during hot or cold weather conditions. Proper control joints and reinforcing should be provided to minimize any damage resulting from shrinkage.

8.10 Soil Corrosion Testing

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

- **Soluble sulfates:** Soluble sulfate test results vary from a non-detect to 23 ppm. These results indicate a negligible sulfate exposure to concrete.

- **pH:** The pH test results ranged from 7.2 to 8.2, which indicates a generally neutral soil condition.

- **Resistivity:** Resistivity test results ranged from 810 to 6400 ohms x cm. The lower resistivity results are for the existing fill soils, which will be removed from below structural areas. Resistivity results for native granular soils were 6400 ohms x com. In general, soils with resistivities below 3,000 are corrosive to metal pipes. Overall, the native granular soils have a low to moderate corrosion potential to metal pipes.

8.11 Structural Section Construction

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

The following presents construction recommendations for the structural section:

- Subgrade soil should be prepared in accordance with the recommendations of this report. Base material should be densified to at least 95 percent relative compaction;

- Type 2 Plantmix aggregate in accordance with Section 200.02 of the referenced standard specifications for public works improvement should be utilized for the pavement. All pavement construction shall conform to the referenced standard specifications; and

- The contractor should submit a pavement mix design to the owner at least 10 working days prior to construction for approval. It is recommended that when pavement is placed adjacent to concrete flatwork, the finish compacted grade of the pavement be at least ¼ to ½ of an inch higher than the edge of adjacent concrete surface. This is to allow adequate compaction of the pavement without damaging the concrete.

8.12 Pavement Maintenance

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

9.0 ADDITIONAL GEOTECHNICAL SERVICES

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

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

10.0 LIMITATIONS

This report has been prepared in accordance with generally accepted local geotechnical practices. The analyses and recommendations submitted are based upon field exploration performed at the locations shown on Plate 2 of this report.

This report has been prepared to provide information allowing the engineer to design the project. The owner/project manager is responsible for distribution of this report to all designers and contractors whose work is affected by geotechnical recommendations. In the event of changes in the design, location, or ownership of the project after presentation of this report, our recommendations should be reviewed and possibly modified by the geotechnical engineer[1]. The engineer makes no other warranties, either expressed or implied, as to the professional advice provided under the terms of this agreement and included in this report[2].

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

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

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

The following appendices are included and complete this report.

Appendix A - Field Exploration

Appendix B - Laboratory Test Results

Appendix C- USGS Seismic Design Parameters Summary Report

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

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

PROFESSIONAL ENGINEER - STATE OF NEVADA
RANDAL A. REYNOLDS
5/18/15
CIVIL
No. 1618
REGISTERED

Expires 12-31-15

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

RAR:jly
Enclosures
V:\Active\1670\geo rpt 5-18-15.docx

References

American Society for Testing and Materials (ASTM), 2004, *Soil and Rock; Dimension Stone; Geosynthetics,* Volume 4.08.

Bonham, H. F. and D. K. Rogers, 1983, *Geologic Map, Mt. Rose Quadrangle*: Nevada Bureau of Mines and Geology, Map 4Bg.

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

NEHRP, 1997, *Recommended Provisions for Seismic Regulations for New Buildings and other Structures*, Building Seismic Safety Council for the Federal Emergency Management Agency.

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

SEA Consulting Inc., 1997*, Geotechnical Investigation Riggins Court Office Building, Reno, Nevada, project No: 3030-01-1.*

Standard Specifications for Public Works Construction, 2012 (Washoe County, Sparks-Reno, Carson City, Yerington, Nevada).

Szecsody, G. C., 1983, *Earthquake Hazards Map, NE Mt. Rose Quadrangle*: Nevada Bureau of Mines and Geology, Map 4Bi.

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

Tokimatsu, A. M. and H. B. Seed, 1987, *Evaluation of Settlements in Sands Due to Earthquake Shaking;* American Society of Civil Engineering, Geotechnical Engineering Journal, Vol 113, No. 8.

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

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

CME

APPENDIX A

VICINITY MAP

N.T.S

REFERENCE: BASE MAP WASHOE COUNTY TECHNOLOGY-REGIONAL SERVICES DIVISION (GIS), WWW.WASHOECOUNTY.US/GIS, ACCESSED AUGUST 28, 2014

FIELD EXPLORATION LOCATION MAP

SCALE: 1"~60'

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Reno, NV 89511

STANTEC CONSULTING, INC.
TMWA STATE FARM WELL HOUSE
FIELD EXPLORATION LOCATION MAP
LONGLY LANE, RENO NEVADA

PROJECT NO.: 1670

DATE: 9/9/2014

LEGEND

APPROXIMATE TEST PIT LOCATION
TP-1

DCP-1
APPROXIMATE LOCATION OF DYNAMIC
CONE PENETROMETER (DCP) TEST

PLATE

A-1

V:\Active\1670\autocad\TP-1.dwg

LOG OF TEST PIT NO. TP-1

PROJECT TMWA STATE FARM WELL HOUSE **RIG & BORING TYPE** DEERE 310SG
CLIENT STANTEC CONSULTING, INC. **LOCATION** NE CORNER OF PROPERTY
PROJECT NO. 1670 **DATE** 08/26/2014 **LOGGED BY:** SAM **SURFACE ELEVATION** 4,444' (TOPO PLATE A-1)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	DCP (Blow/10cm)	DCP "N-value"	Moisture Content %	Laboratory Tests
0							MOIST	0'-½': TOPSOIL AND SEDIMENT, mostly fine to medium sand, non-plastic, minor fiberous roots, brown				3 5 5 12 50/ 5CM	3 6 6 15 25+		
	SM			B	1A	LOOSE TO VERY DENSE	MOIST	Note: Test Pit completed at bottom of existing drainage ditch.							
2.5	SM						MOIST	½'-2': SILTY SAND, mostly fine to medium sand, non-plastic, brown							
				B	1B		TO	2'-7½': SILTY SAND, some fine to medium sand, moderately cemented, non- plastic, yellow brown Note: Platy when excavated, light yellow veins visible in sample	16	NV	NP			22.1	A, G
5								Redox staining visible at 5'							
							WET								
7.5	SP-SM			B	1C		WET	7½'-11': POORLY GRADED SAND WITH SILT, mostly fine to medium sand, non- plastic, grey brown	11	NV	NP			18.8	A, G
10								Note: Left test pit open for 2 hours , sidewall sloughed from bottom of trench to 5 feet on west side of trench and to 7 feet on east side							
								TERMINATED AT 11 FEET, WATER ENCOUNTERED AT A DEPTH OF 7.75 FEET Note: DCP refusal on cemented soil layer at a depth of about 1.75 feet bgs , continued DCP at a depth of 12.8'bgs. DCP refusal at 15.75' bgs.							
12.5 – 15						MED. DENSE TO DENSE	WET					9 13 22 17 16 18 17 20 31 50/ 8CM	7 11 17 13 12 14 13 15 24 25+		

GROUNDWATER

	DEPTH	HOUR	DATE
▽	7'9"		8/26/2014
▼			

SAMPLE TYPE
B - Bulk Sample
U: Hand Sampled, 3" OD, 2.42" ID, Tube

LABORATORY TESTS
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 TMWA STATE FARM WELL HOUSE **EQUIPMENT TYPE** DEERE 310SG
CLIENT STANTEC CONSULTING, INC. **LOCATION** W SIDE OF PROPERTY
PROJECT NO. 1670 **DATE** 08/26/2014 **LOGGED BY:** SAM **SURFACE ELEVATION** ≅4,446 (TOPO PLATE A-1)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0								0'-½': MULCH AND BARK Note: Geotextile fabric between soil and organic material							
	SC						MOIST	½'-4.9': FILL-CLAYEY SAND, some very fine to fine sand, low plasticity, dark brown							
2.5				B	2A			Note: Higher fines content than native silty sand encountered in TP-1. Broken piece of PVC pipe approximatly 1 foot long encountered at a depth of 3 feet.							
5	SM						MOIST	4.9'-7': SILTY SAND, some fine to medium sand, plastic, moderately cemented, brown to light yellow brown Note: Platy when excavated							
7.5								TERMINATED AT 7 FEET, NO FREE WATER ENCOUNTERED							
10															
12.5															
15															

GROUNDWATER

	DEPTH	HOUR	DATE
▽	N.E.		8/26/2014
▼			

SAMPLE TYPE
B - Bulk Sample

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

PLATE NO.: A-2b

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-3

PROJECT TMWA STATE FARM WELL HOUSE **EQUIPMENT TYPE** DEERE 310SG
CLIENT STANTEC CONSULTING, INC. **LOCATION** CENTER SIDE OF PROPERTY
PROJECT NO. 1670 **DATE** 08/26/2014 **LOGGED BY:** SAM **SURFACE ELEVATION** ≅4,446 (TOPO PLATE A-1)

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description	%-200	Liquid Limit	Plasticity Index	Pocket Pen. (tsf)	Dry Density (pcf)	Moisture Content %	Laboratory Tests
0								0'-½': MULCH AND BARK Note: Geotextile fabric between soil and organic material, roots to 1½ feet bgs							
	SC						MOIST	½'-5½': FILL-CLAYEY SAND, mostly fine sand, low plasticity, brown							
2.5				B	2A			Note: Higher fines content than native silty sand encountered in TP-1.	49	44	24			19.5	A, G
5															
	SM						MOIST	4.9'-8': SILTY SAND, some fine to medium sand, plastic, moderately cemented at 6½ feet, brown to light yellow brown Note: Platy when excavated, yellow brown stringers visible in sample							
7.5															
								TERMINATED AT 8 FEET, NO FREE WATER ENCOUNTERED							
10															
12.5															
15															

GROUNDWATER

	DEPTH	HOUR	DATE
▽	N.E.		8/26/2014
▼			

SAMPLE TYPE
B - Bulk Sample

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

PLATE NO.: A-2c

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

UNIFIED SOIL CLASSIFICATION CHART

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

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

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

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

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

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

DUTCH FORMULA FOR CALCULATING DYNAMIC CONE RESISTANCE (DCP)

M=Mass of Hammer=15.98 kg
H= Height of Drop=38.1cm
N=Number of Blows/10cm
Ap=Project Area of Cone=10cm
M'=Mass of Driven Portion of Hammer= 2.49 kg
Pa=Mass of Rod String= 3.26x # rods (each sounding rod is about 1 meter)
Rd= Dynamic Cone Resistance (kg/cm^2)

$$Rd = \frac{M^2 x\ H\ x\ N}{Ap\ (M=M'+Pa)10}$$

Note: Soil consistency/density presented on the test pit logs were calculated using the Wildcat DCP logging spreadsheet

Reference: Triggs Technologies, Inc. *Manual 20 for the Wildcat Dynamic Cone Penetrometer* (Drafted February 12, 2013)

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Reno, NV 89511

CLEAR CREEK RESIDENTIAL, LLC.
TMWA STATEFARM WELL HOUSE
SOIL CLASSIFICATION CHART
LONGLY LANE, RENO NEVADA

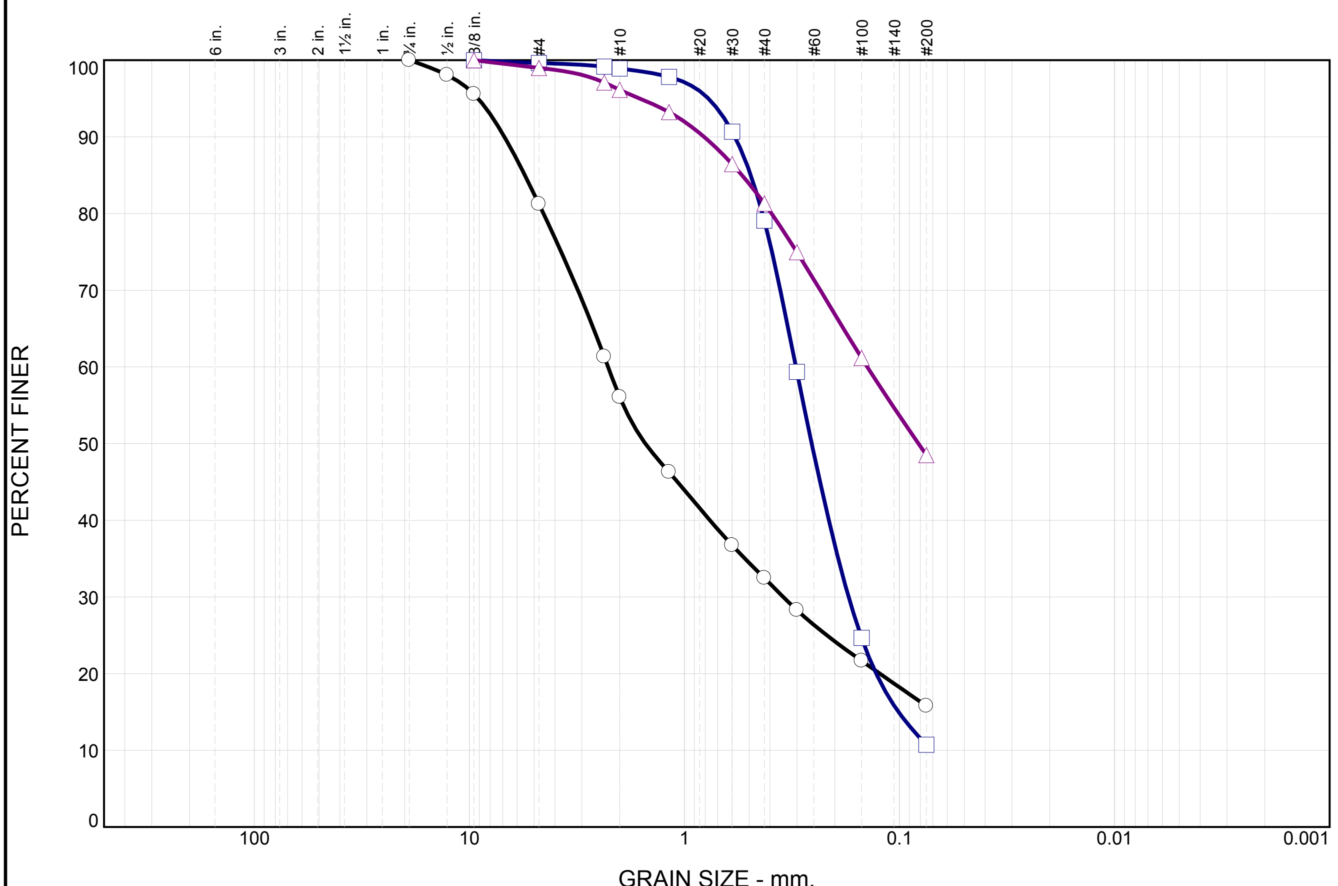
PROJECT NO.:1670 DATE: 09/09/14

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	18.8	25.1	23.6	16.7	15.8	
□	0.0	0.0	0.4	0.7	19.8	68.4	10.7	
△	0.0	0.0	1.0	2.9	14.8	32.8	48.5	

╳	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	NV	NP	5.5205	2.2655	1.5200	0.3472				
□			0.4915	0.3032	0.2560	0.1724	0.1011			
△	44	20	0.5402	0.1412	0.0816					

	MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○	silty sand with gravel (SM)		SM	
□				
△	clayey sand (SC)		SC	

Project No. 1670 **Client:** STANTEC CONSULTING

Project: TMWA- STATE FARM WELL HOUSE

○ **Source of Sample:** TEST PIT 1 **Depth:** 2.0'-4.0' **Sample Number:** 1B

□ **Source of Sample:** TEST PIT 1 **Depth:** 7.5'-8.0' **Sample Number:** 1C

△ **Source of Sample:** TEST PIT 3 **Depth:** 0.0'-4.0' **Sample Number:** 3A

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Remarks:

○ Received Moisture: 22.1%

□ Received Moisture: 18.8%

△ Received Moisture: 19.5%

Figure B-1

Tested By: ○ M. CAVIGLIA □ M. PONTONI △ M. CAVIGLIA **Checked By:** S. VINEIS

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: *Stantec-TMWA State Farm Well House*

Date Printed: 9/5/2014
OrderID: *1408787*

Customer Sample ID: Test Pit 1 1B @ 2' - 4' **Collect Date/Time:** 8/28/2014
WETLAB Sample ID: 1408787-001 **Receive Date:** 8/28/2014 16:50

Analyte	Method	Results	Units	DF	RL	Analyzed	LabID
General Chemistry							
Paste pH	SW846 9045D	8.24	pH Units	1		8/29/2014	NV00925
Resistivity	SM 2510B	6400	ohms.cm	1	1.0	8/29/2014	NV00925
Anions by Ion Chromatography							
Sulfate	EPA 300.0	ND	mg/kg	15	15	8/29/2014	NV00925
Sample Preparation							
Saturated Paste Preparation	CSTPM S:1.0	Complete		1		8/29/2014	NV00925
3:1 DI Water Extraction	WL 3.0	Complete		1		8/29/2014	NV00925

Customer Sample ID: Test Pit 3 3A @ 0' - 4' **Collect Date/Time:** 8/28/2014
WETLAB Sample ID: 1408787-002 **Receive Date:** 8/28/2014 16:50

Analyte	Method	Results	Units	DF	RL	Analyzed	LabID
General Chemistry							
Paste pH	SW846 9045D	7.24	pH Units	1		8/29/2014	NV00925
Resistivity	SM 2510B	810	ohms.cm	1	1.0	8/29/2014	NV00925
Anions by Ion Chromatography							
Sulfate	EPA 300.0	23	mg/kg	15	15	8/29/2014	NV00925
Sample Preparation							
Saturated Paste Preparation	CSTPM S:1.0	Complete		1		8/29/2014	NV00925
3:1 DI Water Extraction	WL 3.0	Complete		1		8/29/2014	NV00925

DF=Dilution Factor, RL=Reporting Limit, ND=Not Detected or <RL

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CME CONSTRUCTION MATERIALS ENGINEERS INC.
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Reno, NV 89511

STANTEC CONSULTING, INC
TMWA STATE FARM WELL HOUSE
CORROSION TEST RESULTS
LONGLY LANE, RENO NEVADA

PROJECT NO.:1670 DATE: 09/09/2014

PLATE
B-2

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX C

USGS Design Maps Summary Report

User–Specified Input

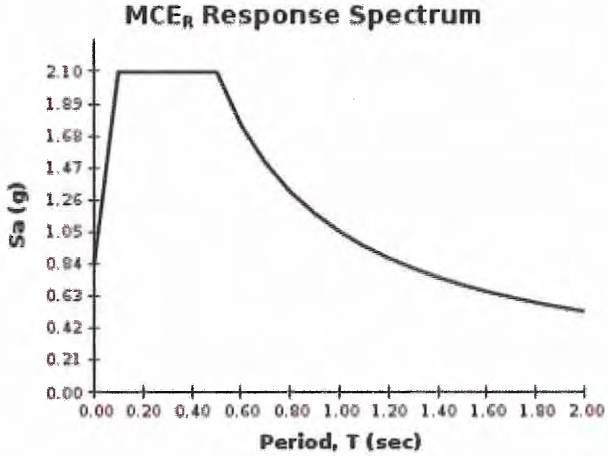
Report Title	TMWA State Farm Wellhouse Wed August 20, 2014 15:50:10 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	39.4645°N, 119.7712°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

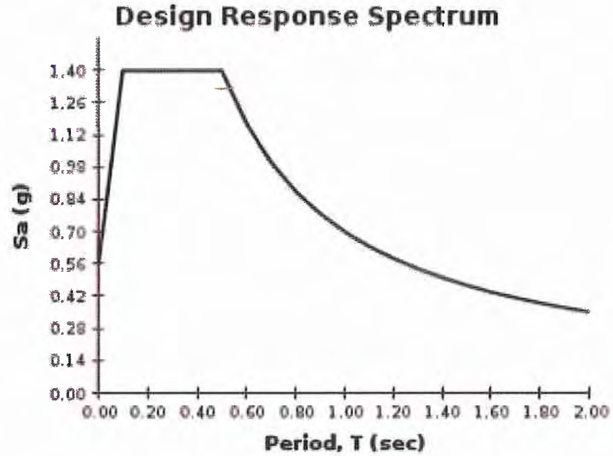
$S_S =$	2.096 g	$S_{MS} =$	2.096 g	$S_{DS} =$	1.397 g
$S_1 =$	0.700 g	$S_{M1} =$	1.050 g	$S_{D1} =$	0.700 g

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 (39.4645°N, 119.7712°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]	$S_S = 2.096$ g
From Figure 22-2 [2]	$S_1 = 0.700$ 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 $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

Table 11.4–1: Site Coefficient F_a

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

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

For Site Class = D and S_s = 2.096 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

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

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

For Site Class = D and S_1 = 0.700 g, F_v = 1.500

Equation (11.4-1): $S_{MS} = F_aS_S = 1.000 \times 2.096 = 2.096$ g

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Equation (11.4-2): $S_{M1} = F_vS_1 = 1.500 \times 0.700 = 1.050$ g

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Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 2.096 = 1.397$ g

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Equation (11.4-4): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 1.050 = 0.700$ g

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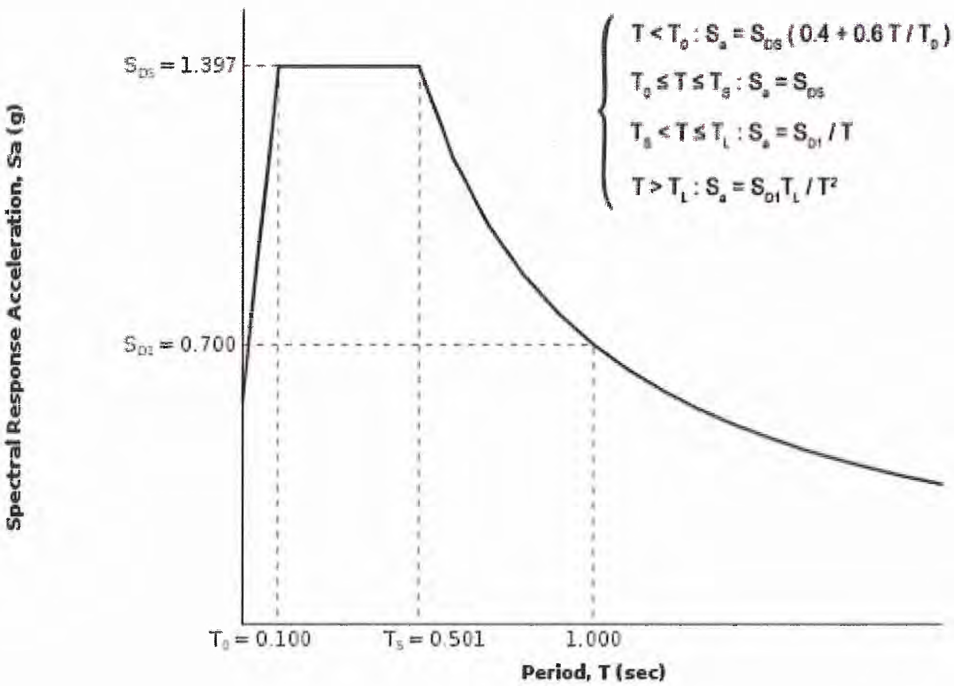
Section 11.4.5 — Design Response Spectrum

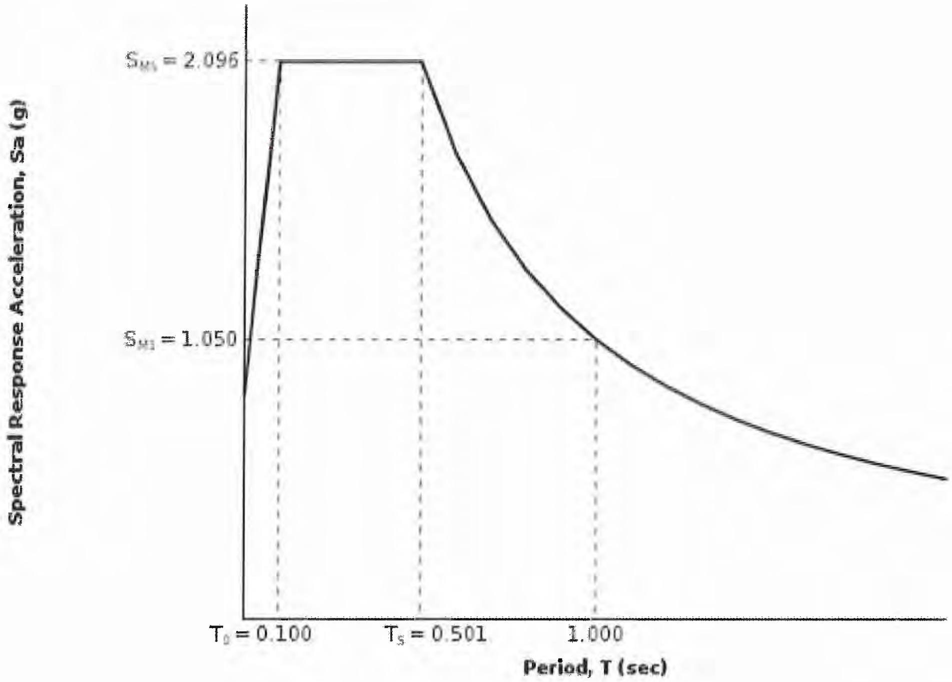
From Figure 22-12 [3] $T_L = 6$ seconds

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 = 0.817

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.817 = 0.817$ 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	$\boxed{1.0}$
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 PGA

For Site Class = D and PGA = 0.817 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} = 0.876

From Figure 22-18 [6] C_{R1} = 0.871

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} = 1.397 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} = 0.700 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