TECHNICAL MEMORANDUM

Truckee Meadows Water Authority Geotechnical Recommendations for Proposed Improvements to the Old Washoe 4 Well

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Introduction

This technical memorandum (TM) summarizes pertinent findings of geotechnical investigations performed by others and provides geotechnical recommendations for design and construction of proposed improvements to the Old Washoe 4 Well. The well is located adjacent to Interstate Highway 580, midway between Reno and Carson City in Washoe County, Nevada. Proposed improvements include a new well pump control building, asphalt-paved access road, well head improvements, and concrete slabs for appurtenant equipment.

CH2M HILL (CH2M) has been retained by Truckee Meadows Water Authority (TMWA) to provide construction bid documents, which will include this TM as a reference document. The seismic and geotechnical design will be based on the International Building Code (IBC, 2012) standards, which refer to American Society of Civil Engineers (ASCE)-7 (2010) for seismic design requirements.

Available Geotechnical Information

Pertinent geotechnical data from the following reports were reviewed for the current study:

- Geotechnical Investigation, Well House Old Washoe Tank Site, Washoe County, Nevada (Wood Rodgers, 2013)
- Soils Investigation-Old Washoe Tank Access Driveway, Washoe County, Nevada (CME, 2015)

Four test pits were excavated for investigation of subsurface conditions. Wood Rodgers excavated two test pits in the vicinity of the proposed well building in 2013, and CME excavated two test pits in the access road in 2015. The approximate locations of the test pits are shown on Figure 1. Soil materials and conditions encountered are presented in the reports listed above, which are included as Attachments 1 and 2, respectively.

Subsurface Conditions

The existing water tank site was previously graded by cutting the northern portion of the site, and placing this material as fill in the southern portion of the site. According to the current site survey shown on the project drawings, the thickness of fill beneath the proposed well building varies from approximately 2 to 8 feet. The fill material is assumed to be similar in composition to the native soils.

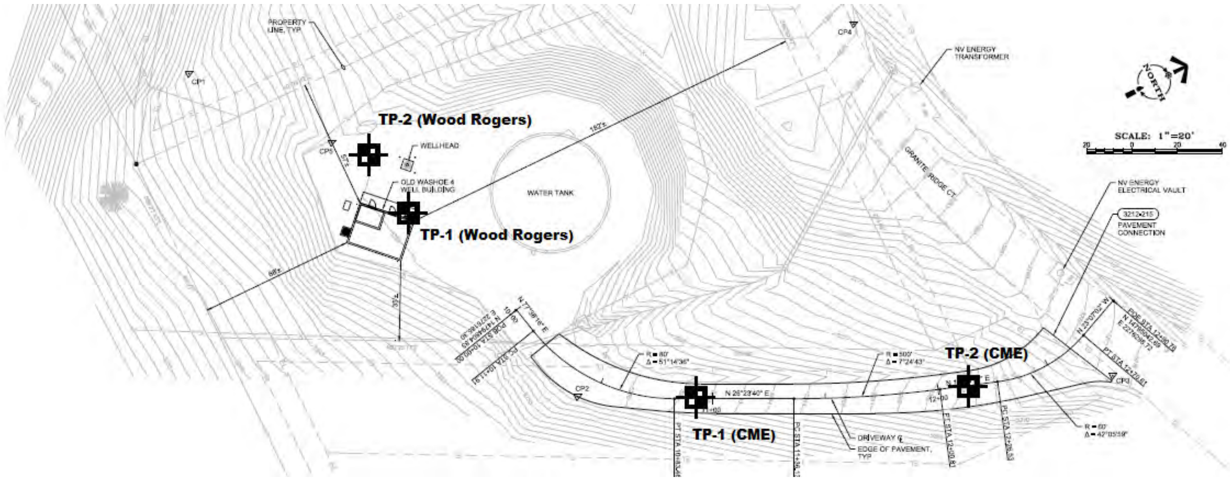

Figure 1. Approximate Locations of Test Pits Excavated for Geotechnical Investigations

The soil at Test Pit TP-1 (Wood Rodgers) was well-graded gravel with silt. The soil material at Test Pits TP-2 (Wood Rodgers), TP-1 (CME), and TP-2 (CME), consisted of clayey sand and gravel, with up to 25 percent fines. From a review of the pictures of the excavated materials in the CME report, the laboratory test results showing 25 percent fines are likely skewed because the large boulders and cobbles were not included in the sample tested for gradation in the laboratory. It is estimated that the material would generally be classified as well-graded gravel with silty clay and sand, with cobbles and boulders up to 24 inches in dimension. Detailed soil information is included in the geotechnical reports in Attachments 1 and 2.

Groundwater was not encountered in subsurface explorations at the site and is not anticipated to be within the zone of proposed excavations for the project.

Recommendations

The geotechnical reports in Attachments 1 and 2 may contain recommendations that conflict with the recommendations presented in this TM. If there are conflicts, the recommendations in this TM should be used for design of the improvements included in the current project.

Design Seismic Parameters

The Wood Rodgers report concluded that the conditions correspond to Site Class C (very dense soil and soft rock), in accordance with the 2006 International Building Code, based on shear wave velocity measurements. However, no data or reference to the shear wave velocity measurements were provided, and the lower-density fill materials are likely to make the average shear wave velocity drop below the minimum required for Site Class C material. The depth to rock beneath the site is unknown, because the rock was not encountered in the test pit excavations. According to the available soil data, the subsurface profile at the project site should be categorized as Site Class D (stiff soil profile), in accordance with ASCE-7 (American Society of Civil Engineers [ASCE], 2010) and the 2012 International Building Code.

The seismic parameters for horizontal ground motion were determined from the ASCE-7 design ground motion maps. The mapped or maximum considered earthquake (MCE_R) peak ground acceleration (PGA), short-period (at 0.2 second) spectral acceleration (S_s), and long-period (at 1 second) spectral acceleration (S_1) for the project site are summarized in Table 1. The site amplification factors (coefficients) for Site Class D are also listed in Table 1.

Table 1. Estimated Seismic Parameters for Proposed Well Building
Truckee Meadows Water Authority Geotechnical Recommendations for Proposed Improvements to the Old Washoe 4 Well

Seismic Parameters	Descriptions	Values
PGA	Mapped peak ground acceleration	0.89 g
S_s	Mapped short-period (at 0.2 seconds) spectral acceleration	2.26 g
S_1	Mapped long-period (at 1 second) spectral acceleration	0.81 g
	Site Class	D
F_{PGA}	PGA site coefficient (Site Class D)	1.0
F_a	Short-period site coefficient (Site Class D)	1.0
F_v	Long-period site coefficient (Site Class D)	1.5
PGA_M	MCE_g PGA adjusted for Site Class D	0.89 g
S_{Ds}	Design short-period (at 0.2 second) spectral acceleration	1.51 g
S_{D1}	Design long-period (at 1 second) spectral acceleration	0.81 g

Notes:
MCE_g = maximum considered earthquake geometric mean.
g = acceleration.

The MCE_R spectral accelerations listed in Table 1 have been adjusted to a risk-targeted value of 1 percent probability of structure collapse in 50 years, as defined in ASCE-7 standards. The S_s and S_1 are the spectral accelerations for the risk-targeted spectrum, and the PGA is defined as the geometric mean of horizontal ground motions.

Over-excavation and Replacement with Structural Fill

The Wood Rodgers report (2013) states that the existing fill appeared to be sound and adequately compacted for support of project structures; however, they recommended that uncompacted fill in the test pits located beneath structures be replaced with structural fill material. It will be very difficult, if not impossible, to discern a difference between the backfill in the test pits and the existing fill material surrounding the test pits.

Large 18-inch boulders were present within the fill material and are likely to have inhibited compaction efforts during placement of the fill, especially in the area immediately surrounding large boulders. The density or degree of compaction cannot be determined through visual methods.

The existing fill material consists of granular soil, with an estimated 10 percent fines. Settlement was estimated to be approximately 0.5 to 1 inch under the maximum net foundation bearing pressure of 2,500 pounds per square foot (psf). The differential settlement could reach a magnitude of 1 inch given that one side of the building will be underlain by a minimal amount of fill, and the other side may have up to 8 feet of fill.

For these reasons, the existing fill materials beneath the proposed well building should be over-excavated to elevation 5,254 feet, which is the approximate elevation of native ground beneath the center of the well building, and replaced with structural fill. The over-excavation should include the entire building footprint and extend at least 2.5 feet beyond the building perimeter. This will provide a more uniform support for the building foundations. The existing fill materials must be screened to remove cobbles and boulders larger than 6 inches prior to use as structural fill.

Because the depth of the test pit was 7.5 feet and the proposed over-excavation depth is 5 feet below the existing ground surface, it is possible that the lower portion of Test Pit TP-1 (Wood Rodgers) is located beneath the recommended over-excavation depth. The exposed subgrade at the base of the over-excavated area should be proof-compacted under the supervision of a geotechnical engineer to verify that there is sufficient over-excavation within the structural area of influence beneath the proposed well building and that the subgrade is firm and unyielding. Proof-compaction should be performed with a vibratory soil compactor weighing at least 9,000 pounds.

The test pit locations in the access road, TP-1 (CME) and TP-2 (CME), should be located and re-excavated to a depth of 3 feet. The exposed subgrade should be scarified, moisture conditioned, and compacted to the requirements of structural fill. The excavations should then be backfilled with structural fill up to the required subgrade of the pavement section.

Foundations

Perimeter strip and spread footings may be designed using a net allowable bearing pressure of 2,500 psf for dead load plus live loads. The minimum embedment depth of the footings should be 24 inches, with a minimum width of 18 inches. An increase of up to one-third of the allowable bearing pressure may be used for short-term loading, such as for wind, seismic, or equipment loads.

Total post-construction settlement of shallow-founded facilities is estimated to be less than 0.75 inch, for a maximum bearing pressure of 2,500 psf. The differential settlement of building foundations is estimated to be less than 0.5 inch, if the existing fill is over-excavated and replaced with structural fill as described. The majority of settlement is anticipated to occur during or soon after construction.

Concrete Slabs On-grade

Concrete slabs on-grade may be used to support transformers or other equipment appurtenant to the well building. The soil material below the concrete slabs should be over-excavated to a depth of 2 feet below the final ground surface and replaced with frost-resistant material consisting of granular soil with no more than 10 percent passing the No. 200 sieve.

A layer of aggregate base rock is recommended beneath all concrete floor slabs to create a uniform base for constructing the floor slab that will remain firm under construction activity, especially in the event of rainfall during construction.

Fill and Backfill

Structural fill materials will be required to replace over-excavated areas and to backfill pipe trenches. Structural fill materials should have at least 5 percent but no more than 25 percent fines content (passing the No. 200 sieve), no particles larger than 6 inches in dimension, and no more than 5 percent larger than 4 inches. The native soil may be used as structural fill if it is screened to remove oversize materials. All fill materials should be free of debris, trash, and organic materials.

Structural fill should be compacted to at least 95 percent relative compaction, and it should be moisture conditioned to within 2 percent of the optimum moisture content, in accordance with ASTM International (ASTM) Standard Test Method D1557. Relative compaction is defined as “the ratio, expressed as a percentage, of the field-compacted dry unit weight to the maximum dry unit weight determined in the laboratory using the procedures of ASTM D 1557.” Structural fill material should be placed in thin lifts with a maximum loose thickness of 8 inches.

Perimeter fill slopes should be no steeper than 2 horizontal to 1 vertical. The slopes should be overbuilt horizontally by at least 1 foot and cut back to the final grade to result in a well compacted slope face.

For areas that are difficult to compact, controlled low-strength material (CLSM) may be used as backfill. The 7-day compressive strength should be between 50 to 150 pounds per square inch, and the material must be flowable. Non-plastic native sand may be used to prepare the CLSM, provided that the mix design is submitted for strength testing and the results are acceptable.

For backfill around pipes and for pipe bedding, clean or gravelly sand with less than 5 percent passing No. 200 sieve is recommended so that it can be easily tamped in around the pipe. CLSM may also be used as pipe zone and bedding material.

Trench backfill material is to be placed from the top of the bedding zone (a minimum of 12 inches above the top of the pipe) to the ground surface or pavement subgrade, as required. Onsite material that meets the recommended gradation requirement for structural fill is acceptable as trench backfill. Trench backfill should be compacted to a minimum relative compaction of 90 percent, at a moisture content within 2 percent of the optimum moisture content.

Asphalt Concrete Pavement

Asphalt concrete pavement and aggregate base rock should conform to the requirements of the Standard Specifications for Public Work Construction, in accordance with Washoe County standards. The recommended pavement section for the project paving is based on R-Value design methods, and a traffic index representing a 3 to 4 trucks per week over 15 years. Traffic is anticipated to be utility trucks rather than multiaxle tractor-trailers.

A pavement section of 3 inches of asphalt concrete over 6 inches of Type 2 Class B aggregate base rock will meet requirements for an R value of 14 and a traffic index of 4.5. Pavement section recommendations for other traffic indices will be provided for different traffic indices, if required.

The width of the access road is limited, and the existing access road grade should be cut down 9 inches to allow placement of the required pavement section where needed to achieve the minimum roadway width. The subgrade soils beneath pavement sections should be scarified to a minimum depth of 6 inches, moisture conditioned as necessary to near-optimum moisture condition, and compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D1557. The subgrade should be smooth and unyielding prior to the placement of aggregate base rock.

The aggregate base rock should be spread in thin lifts restricted to 8 inches in loose thickness or less, moisture conditioned as necessary to within 2 percent of optimum moisture content, and compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D-1557 (or American Association of State Highway and Transportation Officials [AASHTO] T-180). Density testing should be performed prior to placement of the asphalt paving.

Construction Observation and Testing

The subgrade in over-excavated areas should be proof-compacted under the supervision of a geotechnical engineer to verify that the subgrade is firm and unyielding.

Compaction testing and continuous supervision of fill placement should be performed by a testing firm retained by TMWA. A minimum frequency of one test per 75 cubic yards of fill is recommended, or a minimum of one test per every 2 vertical feet of fill placed within areas that are worked on a single day.

Limitations

This TM has been prepared for the exclusive use of CH2M design team members and TMWA for specific applications to the design and evaluations of the Old Washoe 4 Well improvements in Washoe County, Nevada. This TM has been prepared in accordance with generally accepted geotechnical engineering practices. No other warranty expressed or implied is made.

The recommendations contained in this TM are based on the data obtained from field investigations conducted by others and a review of available geotechnical reports and documents prepared for the general TMWA site. Soil boring logs indicate subsurface conditions only at specific locations and times and only to the depths penetrated; they do not necessarily reflect variations that may exist between locations or possible changes that may take place with time and depth.

If changes in the nature, design, or purpose of the improvements occur, the conclusions and recommendations of this TM should not be considered valid unless those changes are reviewed and the conclusions of this TM are modified or verified in writing by a CH2M geotechnical engineer. CH2M is not responsible for any claims, damages, or liability associated with the reinterpretation or reuse of the subsurface data in this TM by others.

Works Cited

Wood Rodgers. 2013. *Geotechnical Investigation, Well House Old Washoe Tank Site, Washoe County, Nevada*. Report prepared for Washoe County. June 18.

CME. 2015. Soils Investigation-Old Washoe Tank Access Driveway, Washoe County, Nevada. Report prepared for TMWA. August 14.

Attachment 1
Wood Rodgers Geotechnical Investigation Report

Geotechnical Investigation

Well House
Old Washoe Tank Site
Washoe County, Nevada

Mr. Alan Jones, P.E
WASHOE COUNTY
Community Services Department – Water Resources
4930 Energy Way
Reno, NV 89502

Project No.: 8051.034

June 18, 2013

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EXECUTIVE SUMMARY

The existing Well House site was originally graded and developed in 1997 as part of the improvements associated with the Old Washoe Tank Site. The proposed Well House area is currently graded and has been capped by a layer of aggregate base. Little to no vegetation is present except along the fence line.

The property is mapped in an area of andesite breccia that presents a competent unit of sand and gravels. This unit will provide adequate support for the planned improvements both insitu and as a source of structural fill. Although the fill material that was placed during 1997 presents too much rock to allow density testing, the fill appeared competent and adequately compacted. No significant voids or nesting of rock was noted. Some oversize rock (i.e. > 6-inches) was encountered in our test pits, and may require screening if intended for reuse as trench backfill or footing backfill. The property is well suited for the planned improvements.

1.0 INTRODUCTION

Presented herein are the results of Wood Rodgers' geotechnical exploration, laboratory testing, and associated geotechnical design recommendations for the proposed Well House located at the Old Washoe Tank No 1 located in north Washoe Valley, Washoe County, Nevada. These recommendations are based on surface and subsurface conditions encountered in our explorations, and on details of the proposed project as described in this report. The objectives of this study were to:

1. Determine general soil and ground water conditions pertaining to design and construction of the proposed subdivision.
2. Provide recommendations for design and construction of the project, as related to these geotechnical conditions.

The area covered by this report is shown in Figure 1 and on Plate A-1 (Site Plan & Approximate Test Pit Locations) in Appendix A. Our study included field exploration, laboratory testing, and engineering analyses to identify the physical and mechanical properties of the various on-site materials. Results of our field exploration and testing programs are included in this report and form the basis for all conclusions and recommendations.

2.0 PROJECT DESCRIPTION & SITE CONDITIONS

The project is located in Section 23, Township 17N, Range 19E, M.D.M. The existing Washoe County facility is located off of Granite Ridge Road in the northwestern part of Washoe Valley. The property is surrounded by a perimeter fence and encompasses an area of approximately 0.25 acres. An existing tank and well are located on site with gravel capping the existing pad/drive area.

The proposed pump house is anticipated to be concrete masonry with concrete slab on grade flooring.

FIGURE 1 – Aerial View of Property

The building pad was prepared during 1997 with the development of the tank site. Vegetation is light, consisting mostly of weeds growing along the fence line. The overall site grades toward the southeast at less than one percent.

3.0 EXPLORATION

The project was explored in May 2013 by excavating 2 test pits using a Deere 310J rubber tire backhoe. The approximate locations of the test pits are shown on Plate A-1 – Site Map and Approximate Test Pit Locations and in Figure 1. The maximum depth of test pit advance was 9 feet below the existing ground surface. Bulk samples for index testing were collected from the trench walls at specific depths in each soil horizon.

Wood Rodgers' personnel examined and classified all soils in the field in general accordance with ASTM D 2488 (Description and Identification of Soils). During exploration, representative bulk samples were placed in sealed plastic bags and returned to our Reno, Nevada laboratory for testing. Additional soil classifications, as well as verification of the field classifications, were subsequently performed in accordance with ASTM 2487 (Unified Soil Classification System [USCS]) upon completion of laboratory testing as described below in the Laboratory Testing section. Logs of the test boring and test pits are presented as Plate A-2. A USCS chart has been included as Plate A-3 - Graphic Soils Classification Chart.

4.0 LABORATORY TESTING

All soil testing performed in the Wood Rodgers' laboratory is conducted in accordance with the standards and methods 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 contents (ASTM D 2216), grain size distribution (ASTM D 6913), and plasticity indices (ASTM D 4318). Results of these tests are shown on Plates A-4a and b – Summaries of Test Data. The test results were used to classify the soils according the USCS (ASTM D 2487) and to verify the field logs, which were then updated. A bulk sample of the rock fill was collected and tested in the laboratory for gradation and plasticity. The sample presented 64 percent gravel, 28 percent sand, and 8.4 percent non-plastic fines.

5.0 GEOLOGIC AND GENERAL SOIL AND GROUNDWATER CONDITIONS

Based on the Nevada Bureau of Mines and Geology, Washoe City Geologic Map (Tabor & Ellen, 1975) the site is mapped in an area of Tkb: andesite breccia with minor flows. This unit would typically appear to be competent and dense cemented sands and gravels.

The units encountered in our explorations typically consisted of reworked clayey sand and poorly graded gravel fill overlying the bedrock. The fill was placed during grading for the existing tank.

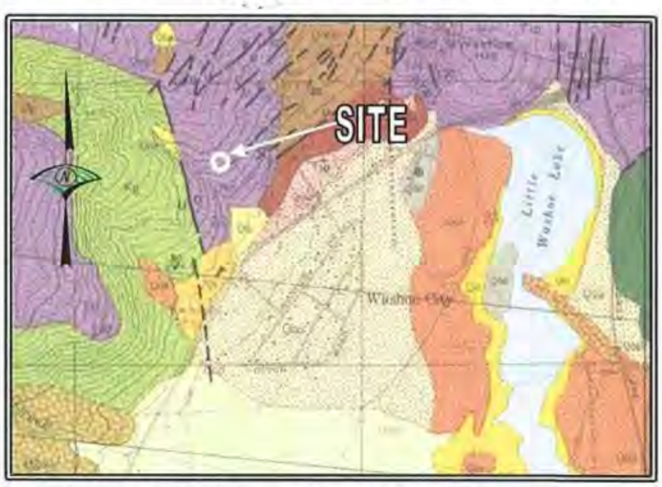


FIGURE 2 - Geologic Map of Project Area

Because the fill material presented more than 30% retained on the ¾" sieve, density testing of the existing fill could not be performed in accordance with ASTM standards. However, based on our observations and excavation characteristics of the material, the existing fill appeared to be sound and adequately compacted.

Groundwater was not encountered in any of our explorations and is expected to lie at a depth that would not affect construction or impact the design of the planned improvements.

6.0 SEISMIC HAZARDS

FIGURE 3 – USGS Quaternary Fault Map

The Truckee Meadows lies within the western extreme of the Basin and Range physiographic province sandwiched between the Virginia Range to the east and the Carson Range to the west. The Basin and Range province is characterized by a series of valleys bounded by north/south trending mountain ranges, byproducts of the seismically active zones of the Wasatch Front in Utah and the Sierra Nevada Mountains along the California/Nevada border. Faulting and seismic activity are integral to the formation of this series of alternating valleys and mountain ranges. As a consequence the presence of faults, active and inactive, is common in western Nevada.

A criteria for evaluating earthquake faults has been formulated by a professional committee for the State of Nevada Seismic Safety Council, but has not yet been adopted by the State or Counties. The guidelines present that faults with evidence of movement within the past 10,000 years (Holocene time) are considered Holocene Active. Faults with evidence of displacement within the last 130,000 years are considered Late Quaternary Active and faults with movement within the last 1.6 million years are considered Quaternary Active. The USGS Earthquake Hazards Program, was accessed to review the proximity of any active faults as previously characterized. The closest mapped fault is located approximately 1/4 mile to the northeast the site; this fault would be considered Quaternary Active. This fault is sufficiently distant that offsets or additional considerations would not be required. In addition, because the project site is developed further investigations or assessments of potential faulting would not be feasible.

Liquefaction is a loss of soil shear strength that can occur during a seismic event, as excessive pore water pressure, between the soil grains, is induced by cyclic shear stresses. This phenomenon is limited to unconsolidated, clean to silty sand (up to 35 percent non-plastic fines) lying below the ground water table (typically less than 40 feet deep). Based on the information obtained during our exploration and research programs, no liquefaction potential exists at the site due to the competent nature of the subgrade soil and bedrock and depth to groundwater.

7.0 DISCUSSION AND RECOMMENDATIONS

7.1 General Information

The following definitions characterize terms utilized in this report:

- Fine-grained soil possesses more than 40 percent by weight passing the number 200 sieve and exhibits a plasticity index lower than 15.
- Clay soil possesses more than 30 percent passing the number 200 sieve and exhibits a plasticity index greater than 15.
- Granular soil does not meeting the above criteria and has a maximum particle size less than 6-inches.

The recommendations provided herein, particularly under Site Preparation, Grading and Filling, Foundation Design, Site Drainage and Quality Control 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. 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 holes were advanced at the approximate locations shown on the site plan. All test holes 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 re-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.

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[1].

7.2 Soil Profile Type Amplification Factors

Because the average 100-feet measured shear wave velocity exceeded 1,200 feet per second, the site can be classified as a Site Class C (very dense soil/soft rock profile) as listed in Table

[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 O 1557 laboratory test procedure. Optimum moisture content is the corresponding moisture content of the same soil at its maximum dry density.

1613.5.2 of the 2006 International Building Code. Based on the average latitude and longitude of the site (39.326 N, -119.817 W) the mapped spectral response accelerations for the 0.2 seconds (S_s) and 1 second (S_1) periods are 1.799 and 0.727, respectively (USGS Earthquake Hazards Program). Based on these mapped spectral response accelerations, the Site Coefficients F_a and F_v, as a function of site class, are 1.0 and 1.30, respectively.

7.3 Site Preparation

Little to no vegetation is present in the proposed improvement area. Therefore stripping is not required. Simply remove what limited vegetation is present. All areas to receive structural fill or structural loading should be densified to a minimum depth of 8-inches to at least 90 percent relative compaction in accordance with ASTM D 1557. It is recommended that soils have moisture contents of plus or minus 3 percent of optimum moisture (ASTM D1557) prior to densification. Higher moisture contents will be acceptable if the soil horizon is stable and density can be achieved in subsequent structural fill lifts. Scarification and moisture conditioning may be required to achieve the required soil moisture content recommendations.

7.4 Grading and Filling

According to the "Old Washoe Water System Tank No. 1 and Transmission Main" plans the fill soils encountered were placed as part of the initial construction in 1997. This fill consisted of poorly graded gravel and appeared to be adequately compacted; overexcavation and replacement to prepare the building pad is therefore not required. Oversize (i.e. > 6-inches) particles were observed in the fill soils. Screening of particles larger than 3-inches will facilitate fine grading and reuse for trench or foundation backfill if needed.

Structural fill is defined as any material placed below structural elements, including; foundations, concrete slabs-on-grade, pavements, or any structure that derives support from the underlying soil. Granular and fine-grained soil generated on-site and free of vegetation, organic matter, and other deleterious material can be used as structural fill. If any imported structural fill is required, it should be free of vegetation, organic matter, and other deleterious material and meet the requirements of Table 1.

TABLE 1 - Guideline Specification for Imported Structural Fill

Sieve Size	Percent by Weight Passing	
3 inch	90 - 100	
¾ Inch	70 – 100	
No. 40	15 – 70	
No. 200	10 – 30	
Percent Passing No. 200 Sieve	**Maximum Liquid Limit**	**Maximum Plastic Index**
10 – 30	35	15

Structural fill should be placed in maximum 12-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 depends on the soil type and the moisture-density relationship test results (ASTM D1557). However, soils should have moisture contents of at least plus or minus 3 percent of optimum moisture (ASTM D1557). Higher moisture contents are acceptable if the soil lifts are stable and required relative compaction can be attained in the soil lift and subsequent soil lifts.

7.5 Trenching and Excavation

Regulations amended in Part 1926, Volume 54, Number 209 of the Federal Register (Table B-1, October 31, 1989) require that the temporary sidewall slopes be no greater than those presented in Table 2. Temporary trenches with near vertical sidewalls should be relatively stable to a depth of approximately five feet. Excavations to greater depths will require shoring or laying back of sidewalls to maintain adequate stability.

Based on the results of our exploration, it is our opinion that the bulk of the site soils appear to be predominately Type C, although variations exist. All trenching should be performed and stabilized in accordance with local, state, and OSHA standards. Bank stability is 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.

TABLE 2 - 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] - cohesive, non-fissured soils, with an unconfined compressive strength of 1.5 tons per square foot (tsf) or greater	3H:4V	(53 degrees)
Type B - cohesive soils with an unconfined compressive strength between 0.5 and 1.5 tsf	1H:1V	(45 degrees)
Type C - unconfined compressive strength below 0.5 tsf	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. Numerous additional factors and exclusions are included in the formal definitions.
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).

7.6 Foundations

It is our understanding that spread footings will be utilized for this project. Provided the foundation soils have been prepared in accordance with the recommendations of this report, the bearing pressures presented in Table 3 can be utilized for design.

TABLE 3 – Allowable Foundation 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.	3,000

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

For frost protection, footings should all be set at least two feet below adjacent outside or unheated interior finish grades. Footings not located within frost prone areas should be placed at least 12 inches below surrounding ground or slab level for confinement. Regardless of loading, individual pad foundations and continuous spread foundations should be at least 18 and 12 inches wide, respectively, 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. The recommended coefficient of base friction is 0.35, and has been reduced by a factor of 1.5 on the ultimate soil strength. Design values for active and passive equivalent fluid pressures are 35 and 350 pounds per square foot per foot of depth, respectively. 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. These design values are based on spread footings bearing on native granular soils, native fine-grained soils, or structural fill and backfilled with structural fill.

If loose, soft, wet, or disturbed soils are encountered at the foundation subgrade, these soils should be removed to expose suitable foundation soils, and the resulting over-excavation backfilled with compacted structural fill. The base of all excavations should be dry and free of loose materials at the time of concrete placement.

Total settlement for the structures is anticipated to be on the order of ½ inch, or less. Differential settlement between foundations with similar loads and sizes is anticipated to be ½ of the total settlement.

7.7 Site Drainage

Adequate surface drainage must be constructed and maintained away from the structures. The permanent finish slopes away from the structure should be sufficient to allow water to drain away quickly from and prevent any ponding of water adjacent to the structure. All runoff should be collected within permanent drainage paths that can convey water off the property.

7.8 Concrete Slabs

Any planned concrete slabs-on-grade, site work, and civil improvements should be underlain by a 6" layer of compacted Type 2, Class B aggregate base. Aggregate base shall be compacted to not less than 95 percent of the soils maximum dry density.

We recommend that all concrete 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.

Soluble sulfate levels within the existing pad were in the negligible range and therefore special concrete considerations for sulfate attack will not be required. The project site is however in a severe weather zone and therefore concrete exposed to inclement should be air entrained and meet the minimum concrete requirements presented in the Standard Specifications for Public Works Construction.

8.0 CONSTRUCTION OBSERVATION AND TESTING SERVICES

The recommendations presented in this report are based on the assumption that the contractors perform their work as required by the project documents and that owner/project manager provides sufficient field-testing and construction review during all phases of construction. Prior to construction, the owner/project manager should schedule a pre-job conference including, but not limited to, the owner, architect, civil engineer, the general contractor, earthwork and materials subcontractors, building official, and geotechnical engineer. It is the owner's/project manager 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 manger for review and distributed to the appropriate parties.

During construction, Wood Rodgers Incorporated should have the opportunity to provide sufficient on-site observation of site preparation and grading, over-excavation, fill placement, foundation installation, and paving. These observations would allow us to document that the geotechnical conditions are as anticipated and that the contractor's work meets with the criteria in the approved plans and specifications. Verification of horizontal and vertical control must be provided by whoever was responsible for establishing those boundaries and constructing associated improvements.

9.0 STANDARD LIMITATION CLAUSE

This report has been prepared in accordance with generally accepted local geotechnical practices. The analyses and recommendations submitted are based upon field exploration performed and the conditions encountered as discussed in our 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. We recommend our firm be retained to perform construction observation in all phases of the project related to geotechnical factors to document compliance with our recommendations. The owner/project manger is responsible for distribution of this geotechnical report to all designers and contractors whose work is related to geotechnical factors.

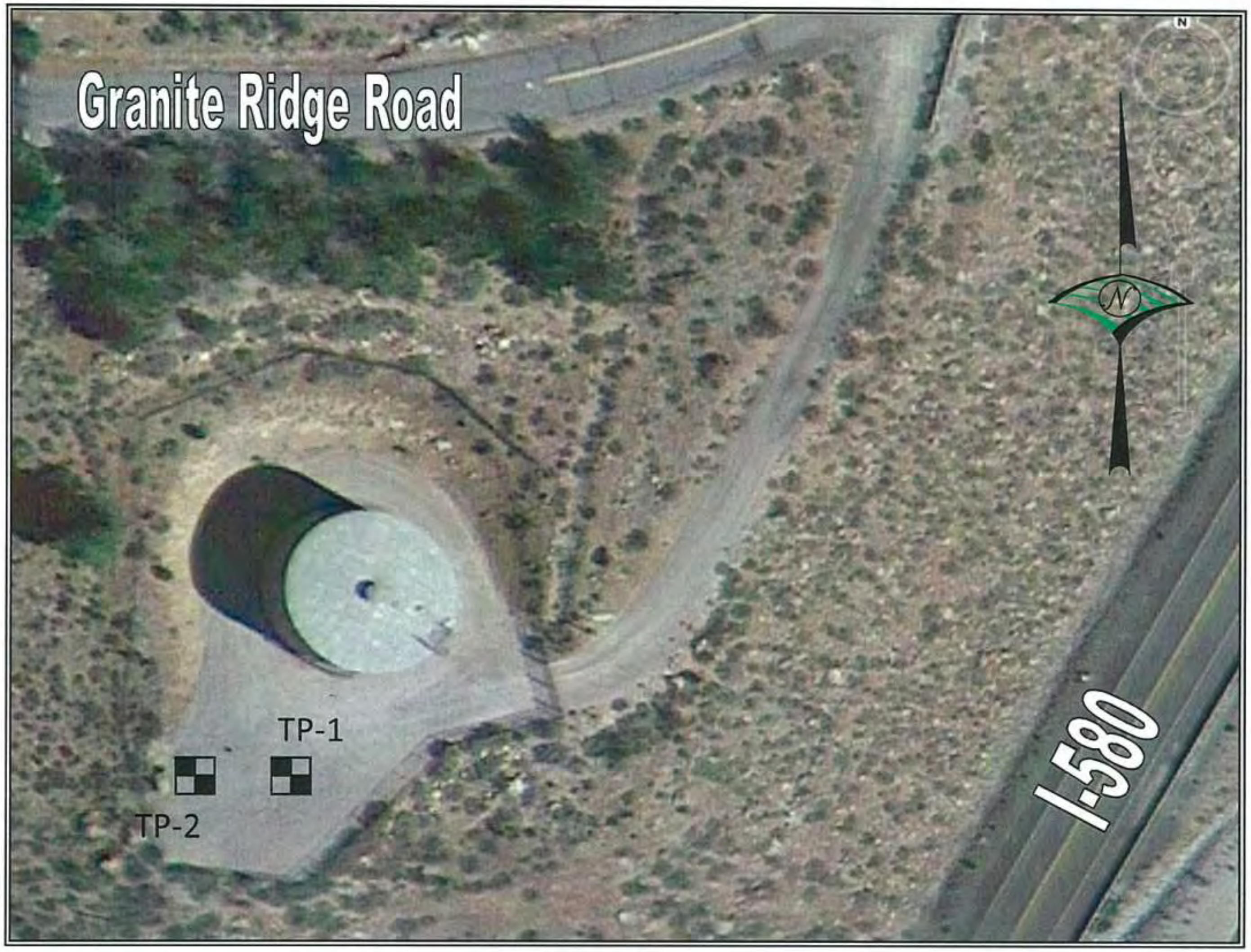
It is the contractor's responsibility for the grading and construction of the designed improvements. This responsibility includes the means, methods, techniques, sequence, and procedures of construction and safety of construction at the site. All construction shall conform to the requirements of the most recently adopted version of the Standard Specifications for Public Works Construction and the requirements of Washoe County, Nevada. Failure to inspect the work shall not relieve the contractor from his obligation to perform sound and reliable work as described herein and as described in the Standard Specifications for Public Works Construction.

All plans and specifications should be reviewed by the design engineer responsible for this geotechnical report, to determine if they have been prepared in accordance with the recommendations contained in this report, prior to submitting to the building department for review. It is the owner's/project manager responsibility to provide the plans and specifications to the engineer.

This report has been prepared to provide information allowing the architect and 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 aspects. 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. If the geotechnical engineer is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation or misapplication of our recommendations or their validity in the event changes have been made in the original design concept without our prior review. 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.

This report was prepared by Wood Rodgers, Inc. for the benefit of Washoe County. The material in it reflects Wood Rodgers' best judgment in light of the information available to it 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 on it, are the responsibility of such third parties. Wood Rodgers' accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

APPENDIX A



WOOD RODGERS
5440 Reno Corporate Drive, Reno, NV 89511
Phone 775.823.4068 Fax 775.823.4066

SITE PLAN AND APPROXIMATE
EXPLORATION LOCATIONS

Geotechnical Investigation

Old Washoe Tank Site
Pump House

Project No.: 8051.034
Date: 6.15.13

PLATE
A-1

Wood Rodgers, Inc.
5440 Reno Corporate Drive
Reno, NV 89511
Telephone: 775-823-4068
Fax: 775-823-4066

TEST PIT NUMBER TP-1

CLIENT Washoe County Water Department

PROJECT NAME Old Washoe Tank Site Well House

PROJECT NUMBER 8051.034

PROJECT LOCATION See Site Plan

DATE STARTED 5/8/13 COMPLETED 5/8/13

GROUND ELEVATION

TEST PIT SIZE inches

EXCAVATION CONTRACTOR Stampede

GROUND WATER LEVELS:

EXCAVATION METHOD Deere 310 SG

AT TIME OF EXCAVATION --- No Free Water Encontered

LOGGED BY Tom Harding CHECKED BY Tom Harding

AT END OF EXCAVATION --- No Free Water Encontered

NOTES:

AFTER EXCAVATION --- No Free Water Encontered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	R-VALUE	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		GRAVEL OVER TOP										
		FILL - POORLY GRADE GRAVEL, (GP) slightly moist, medium dense, with cobbles to 12", boulders to 18"										
2.5			GB 1A					25.1	NP	NP	NP	8.4
5.0												
			GB 1B									
7.5												

Bottom of Test Pit at 8.0 Feet.

Wood Rodgers, Inc.
5440 Reno Corporate Drive
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Telephone: 775-823-4068
Fax: 775-823-4066

TEST PIT NUMBER TP-2

CLIENT Washoe County Water Department

PROJECT NAME Old Washoe Tank Site Well House

PROJECT NUMBER 8051.034

PROJECT LOCATION See Site Plan

DATE STARTED 5/8/13 COMPLETED 5/8/13

GROUND ELEVATION TEST PIT SIZE inches

EXCAVATION CONTRACTOR Stampede

GROUND WATER LEVELS:

EXCAVATION METHOD Deere 310 SG

AT TIME OF EXCAVATION --- No Free Water Encontered

LOGGED BY Tom Harding CHECKED BY Tom Harding

AT END OF EXCAVATION --- No Free Water Encontered

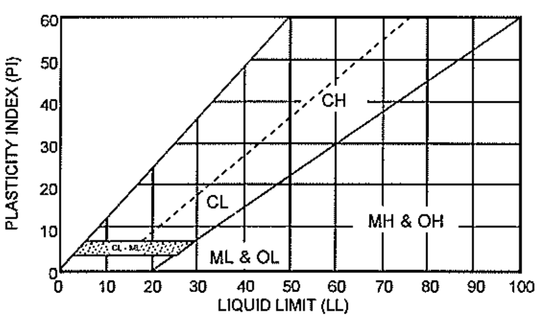
NOTES:

AFTER EXCAVATION --- No Free Water Encontered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	R-VALUE	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0.0		GRAVEL OVER TOP										
		FILL - CLAYEY SAND WITH GRAVEL, (SC) brown tan, slightly moist, medium dense, with cobbles to 12"	GB 2A									
2.5		Red, moist, less gravel and cobles	GB 2B									
5.0												
7.5		Decreasing plastic, trace cobbles to 8"										

Bottom of Test Pit at 9.0 Feet.

MAJOR DIVISION					TYPICAL NAMES
COARSED-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVEL MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES		GW	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
				GP	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES		GM	SILTY GRAVELS, SILTY GRAVELS WITH SAND
				GC	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SAND MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES		SW	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
				SP	POORLY GRADED SAND WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES		SM	SILTY SANDS WITH OR WITHOUT GRAVEL
				SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT 50% OR LESS			ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
				OL	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILT AND CLAY LIQUID LIMIT GREATER THAN 50%			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOLID, ELASTIC SILTS
				CH	INORGANIC CLAYS OR HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC SILTS OR CLAYS MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS



CONSISTENCY		RELATIVE DENSITY	
SILTS & CLAYS	SPT BLOW* COUNTS (N)	SANDS & GRAVELS	SPT BLOW* COUNTS (N)
VERY SOFT	0 - 2	VERY LOOSE	0 - 4
SOFT	3 - 4	LOOSE	5 - 10
MEDIUM STIFF	5 - 8	MEDIUM DENSE	11 - 30
STIFF	9 - 15	DENSE	31 - 50
VERY STIFF	16 - 30	VERY DENSE	50 +
HARD	30 +		

* The Standard Penetration Resistance (N) In blows per foot is obtained by the ASTM D1585 procedure using 2" O.D., 1 3/8" I.D. samplers.

DESCRIPTION OF ESTIMATED PERCENTAGES OF GRAVEL, SAND, AND FINES	
TRACE	Particles are present but est. < 5%
FEW	5% - 10%
LITTLE	15% - 20%
SOME	30% - 45%
MOSTLY	50% - 100%

NOTE: Percentages are presented within soil description for soil horizon with laboratory tested soil samples.

DEFINITIONS OF SOIL FRACTIONS	
SOIL COMPONENT	PARTICLE SIZE RANGE
COBBLES	ABOVE 3 INCHES
GRAVEL	3 IN. TO NO. 4 SIEVE
COARSE GRAVEL	3 IN. TO 3/4 IN.
FINE GRAVEL	3/4 IN. TO NO. 4 SIEVE
SAND	NO. 4 TO NO. 200
COARSE SAND	NO. 4 TO NO. 10
MEDIUM SAND	NO. 10 TO NO. 40
FINE SAND	NO. 40 TO NO. 200
FINES (SILT OR CLAY)	MINUS NO. 200 SIEVE

WOOD RODGERS
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UNIFIED SOIL CLASSIFICATION AND KEY TO SOIL DESCRIPTIONS

Geotechnical Investigation

Old Washoe Tank Site Pump House

Project No.: 8051.034
Date: 6.15.13

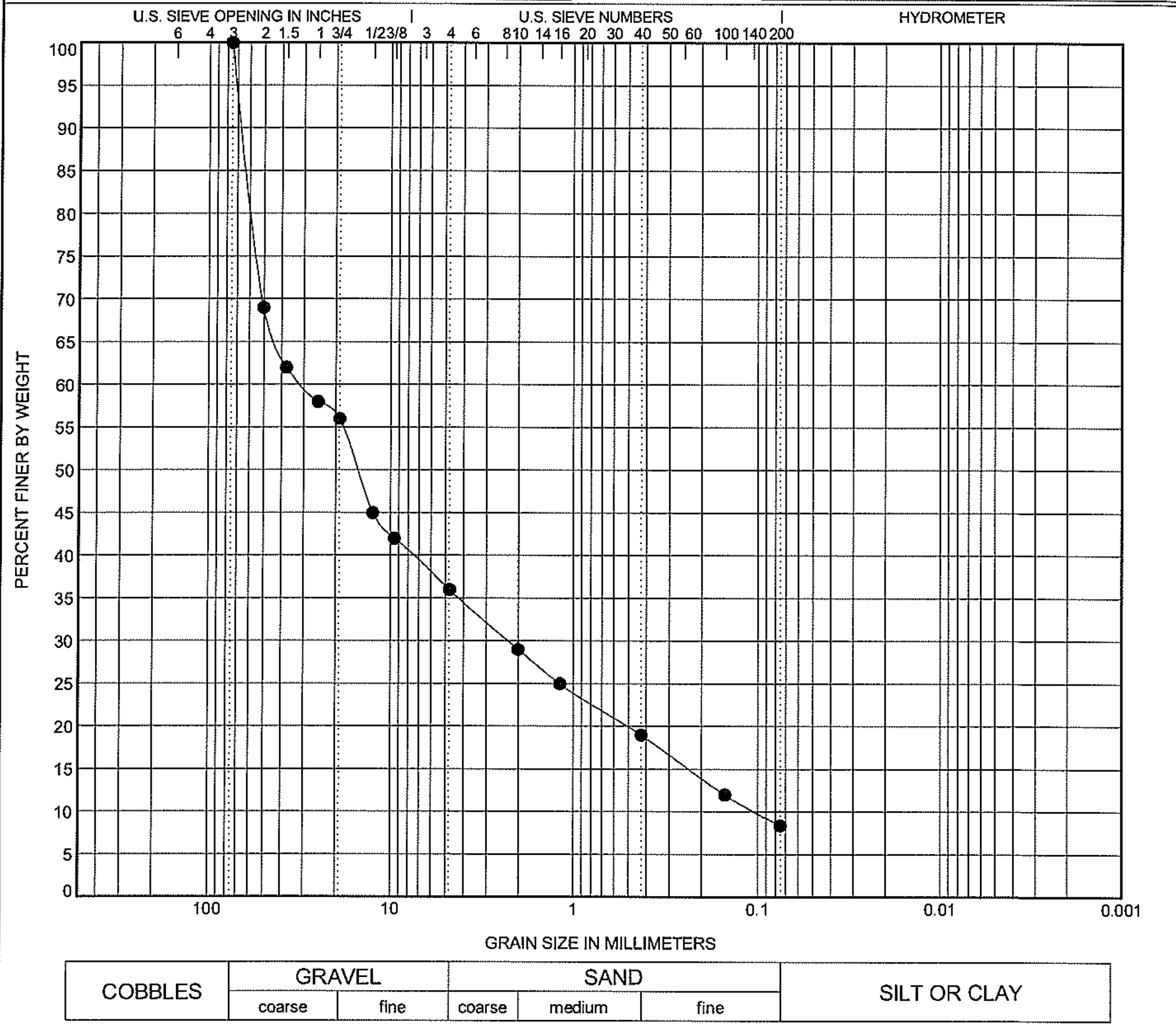
PLATE A-3

Wood Rodgers, Inc.
5440 Reno Corporate Drive
Reno, NV 89511
Telephone: 775-823-4068
Fax: 775-823-4066

GRAIN SIZE DISTRIBUTION

CLIENT Washoe County Water Department **PROJECT NAME** Old Washoe Tank Site Well House

PROJECT NUMBER 8051.034 **PROJECT LOCATION** See Site Plan



TEST PIT	DEPTH	Classification	LL	PL	PI	Cc	Cu
● TP-1	2.0	WELL-GRADED GRAVEL with SILT and SAND(GW-GM)	NP	NP	NP	1.64	300.01

TEST PIT	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● TP-1	2.0	75	30.619	2.263	0.102	64.0	27.6	8.4	

GRAIN SIZE - GINT STD US LAB.GDT - 5/21/13 08:22 - C:\USERS\PUBLIC\DOCUMENTS\BENTLEY\GINT\PROJECTS\OLD WASHOE TANK SITE\OLD WASHOE TANK SITE.GPJ

Attachment 2
CME Geotechnical Investigation Report

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

August 14, 2015
File: 1789

Mr. Kelly McGlynn, PE
Truckee Meadows Water Authority
1355 Capital Blvd.
Reno, Nevada 89502

**RE: Soils Investigation-Old Washoe Tank Access Driveway
Washoe County, Nevada**

Dear Mr. McGlynn:

Construction Materials Engineers, Inc. (CME) is pleased to submit the following soil investigation report for the Old Washoe Tank Access Road, located on the south side of Granite Ridge Drive in Washoe County, Nevada. This soil investigation report contains the results of our subsurface exploration and laboratory testing.

1.0 INTRODUCTION

The project site is located west of Interstate 580 in Washoe Valley. Site improvements include a municipal water tank, associated service lines, perimeter fencing, and a graded gravel access road. A vicinity map is included as Figure 1 (General Project Vicinity).

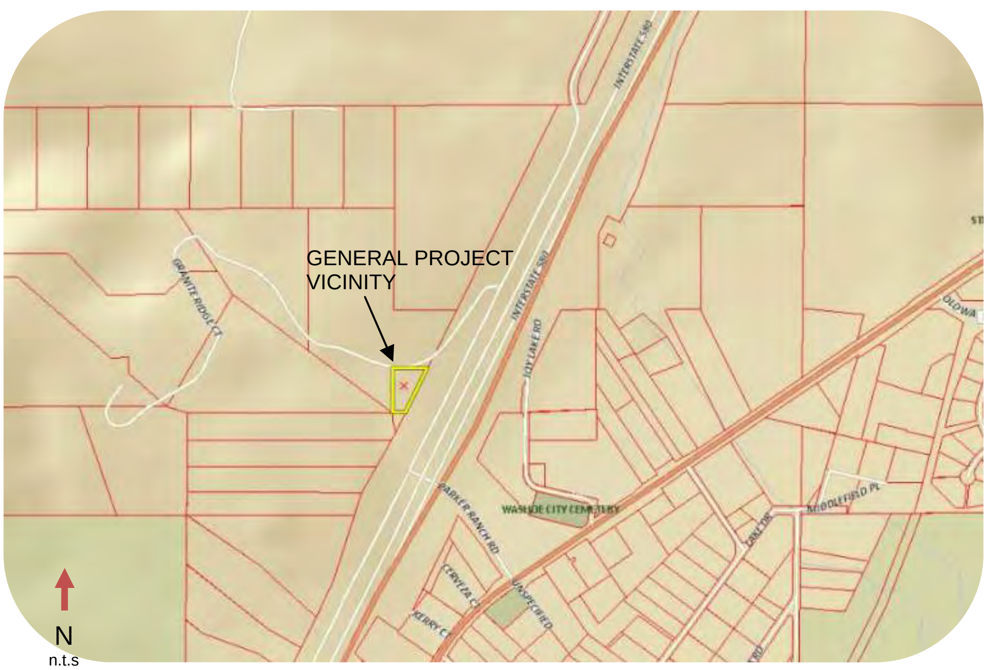


Figure 1: General Project Vicinity
Reference: Washoe County GIS (http://wcgisweb.washoecounty.us/QuickMap/) Accessed 08-13-15

Office 775-851-8205 fax 775-851-8593 www.cme-corp.com

The soils investigation included test pit excavations at two locations along the existing access road alignment, collection of bulk samples for laboratory testing, and preparation of this summary report.

2.0 FIELD EXPLORATION

Subsurface field exploration, completed on August 7, 2015, consisted of excavating two (2) test pits. One test pit was completed at the south end of the existing access road and the other on the north end. The approximate test pit locations are included as Plate A-1 (SAMPLE LOCATION MAP).

Photograph 1: Taken looking south toward Test Pit TP-2.
Test Pit TP-1 is located south of TP-2 near the pictured service vehicle.

Soils encountered within the test pits 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 Pit logs are included as Plates A-2 (Test Pit Logs). 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 (Soils Classification Chart).

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 engineered fill[1].

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

The elevations shown on the test pit logs were obtained by interpolation between contours presented on Plate A-1 (SAMPLE LOCATION MAP). The elevations and locations included in this soils investigation should be considered accurate only to the degree implied by the methods used.

3.0 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.

The following laboratory tests were completed as part of this investigation:

- *Insitu* moisture content (ASTM D 2216);
- Grain size distribution (ASTM D 422);
- Atterberg Limits (ASTM D 4318); and
- R-Value (ASTM D 2844).

Laboratory test results are summarized in Table 1 (Laboratory Soil Index Test Results), and are included as Appendix B.

TABLE 1 – LABORATORY SOIL INDEX TEST RESULTS

Test Pit (TP-#)	Sample Depth	Insitu Moisture	Plasticity Index[(2)]	Liquid Limit	Percent Passing #200 [(3)]	R-value	USCS Soil [(1)] Classification
TP-1	½' - 4'	8.9	16	37	25.2	14	CLAYEY SAND WITH GRAVEL (SC)
TP-2	1 ½'-4'	11.6	17	41	22.0	27	CLAYEY GRAVEL WITH SAND (GC)

Notes:

1) A description of the USCS is presented on Plate A-2 (Soil Classification Chart). Note large cobbles and boulders were not collected as part of the bulk sampling procedure.
2) The Plasticity Index is the numerical difference between the Liquid Limit and Plastic Limit (LL-PL=PI). The Plastic Limit is the percent water content corresponding to an arbitrary limit between the plastic and semisolid states of a soils consistency.
3) Granular material is defined as 50 percent or more retained above the #200 sieve. Fine grained soils are defined as 50 percent or more passing the #200 sieve.

4.0 SUBSURFACE SOILS AND GROUNDWATER CONDITION

4.1 General Soils Profile

In general the subsurface soils profile consisted of a layer of imported silty gravel with sand (GM) fill to depths ranging from 0.25 to 1 ½ feet below the existing ground surface (bgs). Below the upper fill soils horizon, clayey sand with gravel (SC) and clayey gravel with sand (GC) was encountered to the depth of exploration. Cobbles and boulders up to 24 inches nominal diameter were encountered at depth of 1 to 2 ½ feet to the depth of exploration.

Photographs 2 and 3: Taken looking northwesterly toward Test Pit TP-1. Note large cobbles and boulders encountered below a depth of 1 foot to the depth of exploration.

Photographs 4: Taken looking northwesterly toward Test Pit TP-2. Gravel fill soils were encountered to a depth of 1 ½ feet bgs.

4.2 Soil Moisture and Groundwater Conditions

Groundwater was not encountered during the subsurface exploration. In general soils were encountered in a slightly moist to moist conditions.

We hope this letter provides you with the information you require at this time. Please feel free to contact the undersigned if you have any questions or need additional information.

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

Jon A. Del Santo, PE
Project Manager
jdelsanto@cmenv.com
Direct: 775-737-7564
Cell: 775-846-4399

Stella A. Montalvo, PE
Geotechnical Project Manager
RE Number 21801
Expiration Date: 12-31-15
smontalvo@cmenv.com
Direct: 775-737-7569

PROFESSIONAL ENGINEER - STATE OF NEVADA
STELLA A. MONTALVO
Exp. 12-31-2015
CIVIL
No. 021801
08-14-15

SAM:jad:jy
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Attachments:

Appendix A

Plate A-1	SAMPLE LOCATION MAP
Plate A-2	Test Pit Logs (2 Pages)
Plate A-3	Soil Classification Chart (1 Page)

Appendix B

Plate B-1	Grain Size Analysis Results (1 pages)
Plate B-2	R-value (Test Pit TP-1)
Plate B-3	R-value (Test Pit TP-2)

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX A

NOT TO SCALE

LEGEND

TP-1 APPROXIMATE TEST PIT LOCATION

REFERENCE: BASE MAP OBTAINED FROM WASHOE COUNTY GIS (https://www.washoecounty.us/gis/mapping_apps.php), ACCESSED 08-12-15

V:\Active\1789\DRAWINGS\OLD WASHOE TANK FIELD EX LOCATION MAP.dwg

CME CONSTRUCTION MATERIALS ENGINEERS INC.

6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

TMWA
OLD WASHOE TANK ACCESS ROAD MATERIALS TESTING
SAMPLE LOCATION MAP
WASHOE COUNTY, NEVADA

PROJECT NO.: 1789

DATE: 8/12/2015

PLATE

A-1

LOG OF TEST PIT NO. TP-1

PROJECT TMWA OLD WASHOE TANK **EQUIPMENT TYPE** DEERE 310 SG
CLIENT TMWA
LOCATION SOUTH END OF THE EXISTING TANK ACCESS ROAD (PLATE A-1)
PROJECT NO. 1789 **DATE** 08/07/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** ≅5,261' (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	GM						SL. MOIST	0-3": SILTY GRAVEL WITH SAND FILL, mostly fine subangular gravel, some fine to medium sand, grey Note: Appears to be gravel road topping imported to site							
	SC						SL. MOIST	3"-1': CLAYEY SAND WITH GRAVEL (POSSIBLE FILL?), mostly fine to medium sand, little subangular fine gravel, yellow brown Note: Appears to have been previously worked/ densified native soils							
1	SC						SL. MOIST	1'-4': CLAYEY SAND WITH GRAVEL, COBBLES AND BOULDERS, some fine to medium sand, some fine to coarse subangular gravels, cobbles and boulders up to 24 inches nominal diameter, moderately plastic, strong brown							
2				B	1A				25.2	37	16			8.9	A, G
3															
4								TERMINATED AT 4 FEET, NO FREE WATER ENCOUNTERED							
5															
6															

GROUNDWATER

	DEPTH	HOUR	DATE
▽	N.E.		
▼			

SAMPLE TYPE
B - Bulk Sample

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

PLATE NO.: A-2a

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

LOG OF TEST PIT NO. TP-2

PROJECT TMWA OLD WASHOE TANK **EQUIPMENT TYPE** DEERE 310 SG
CLIENT TMWA
LOCATION NORTH END OF THE EXISTING TANK ACCESS ROAD (PLATE A-1)
PROJECT NO. 1789 **DATE** 08/07/15 **LOGGED BY:** SAM **SURFACE ELEVATION (ft)** ≅5,271' (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	GM			B	2B		SL. MOIST	0-1½': SILTY GRAVEL WITH SAND FILL, mostly fine subangular gravel, little fine to medium sand, grey Note: Appears to be import to the site.							
1															
2	SC						MOIST	1½'-2½': CLAYEY SAND WITH GRAVEL (POSSIBLE FILL), mostly fine to medium sand, some fine subangular gravel, moderately plastic, yellow brown Note: Appears to have been previously worked/ densified native soils.							
3	GC			B	2A		MOIST	2½'-4': CLAYEY GRAVEL WITH SAND, mostly fine to coarse subangular gravels, cobbles and boulders up to 18 inches nominal diameter, little fine to medium sand, yellow brown	22	41	17			11.6	A, G
4								TERMINATED AT 4 FEET, NO FREE WATER ENCOUNTERED							
5															
6															

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.

UNIFIED SOIL CLASSIFICATION CHART

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

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

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

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

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

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

SOIL STRUCTURE COMMON DESCRIPTIVE TERMS

FISSURED: SHRINKAGE OR RELIEF CRACKS OFTEN FILLED WITH SILT OR SAND

POCKET: INCLUSION OF MATERIAL WITH EITHER A DIFFERENT TEXTURE OR CLASSIFICATION FROM THE MAIN SOIL LAYER

LAMINATED: THIN ALTERNATING SOIL LAYERS WITH EITHER A DIFFERENT TEXTURE OR CLASSIFICATION.

SEAM: THIN LAYER OF MATERIAL WITH EITHER A DIFFERENT TEXTURE OR CLASSIFICATION FROM MAIN SOIL LAYER.

MOTTLED: SOILS WITH IRREGULAR MARKS OR SPOTS OF DIFFERENT COLORS. USUALLY INDICATES POOR AERATION AND LACK OF GOOD DRAINAGE. MAY INDICATE A MARKER HORIZON OF A PREVIOUS GROUNDWATER LEVEL.

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

TMWA
OLD WASHOE TANK ACCESS ROAD MATERIALS TESTING
SOIL CLASSIFICATION CHART
WASHOE COUNTY, NEVADA

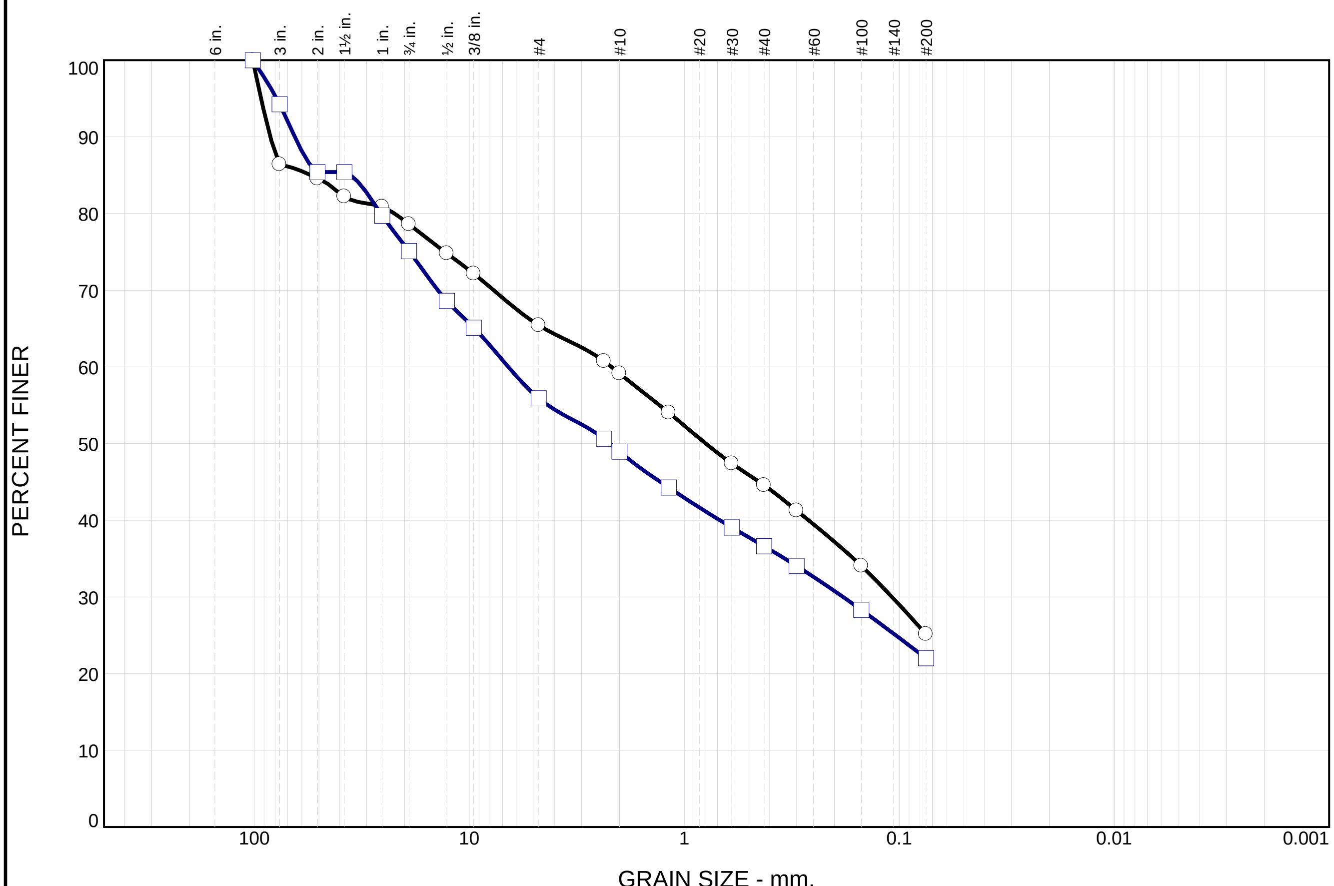
PROJECT NO.:1789 DATE: 08/13/15

PLATE
A-3

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX B

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	13.6	7.8	13.2	6.3	14.5	19.4	25.2	
□	5.7	19.2	19.2	6.9	12.4	14.6	22.0	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	37	21	54.5725	2.1787	0.7927	0.1080				
□	41	24	35.5499	6.5772	2.2126	0.1822				

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ clayey sand with gravel (SC)		SC	
□ clayey gravel with sand (GC)		GC	

Project No. 1789 **Client:** TRUCKEE MEADOWS WATER AUTHORITY

Project: OLD WASHOE TANK

○ **Source of Sample:** TEST PIT 1 **Depth:** 0.0'-4.0' **Sample Number:** 1A

□ **Source of Sample:** TEST PIT 2 **Depth:** 1.5'-4.0' **Sample Number:** 2A

CME

Remarks:

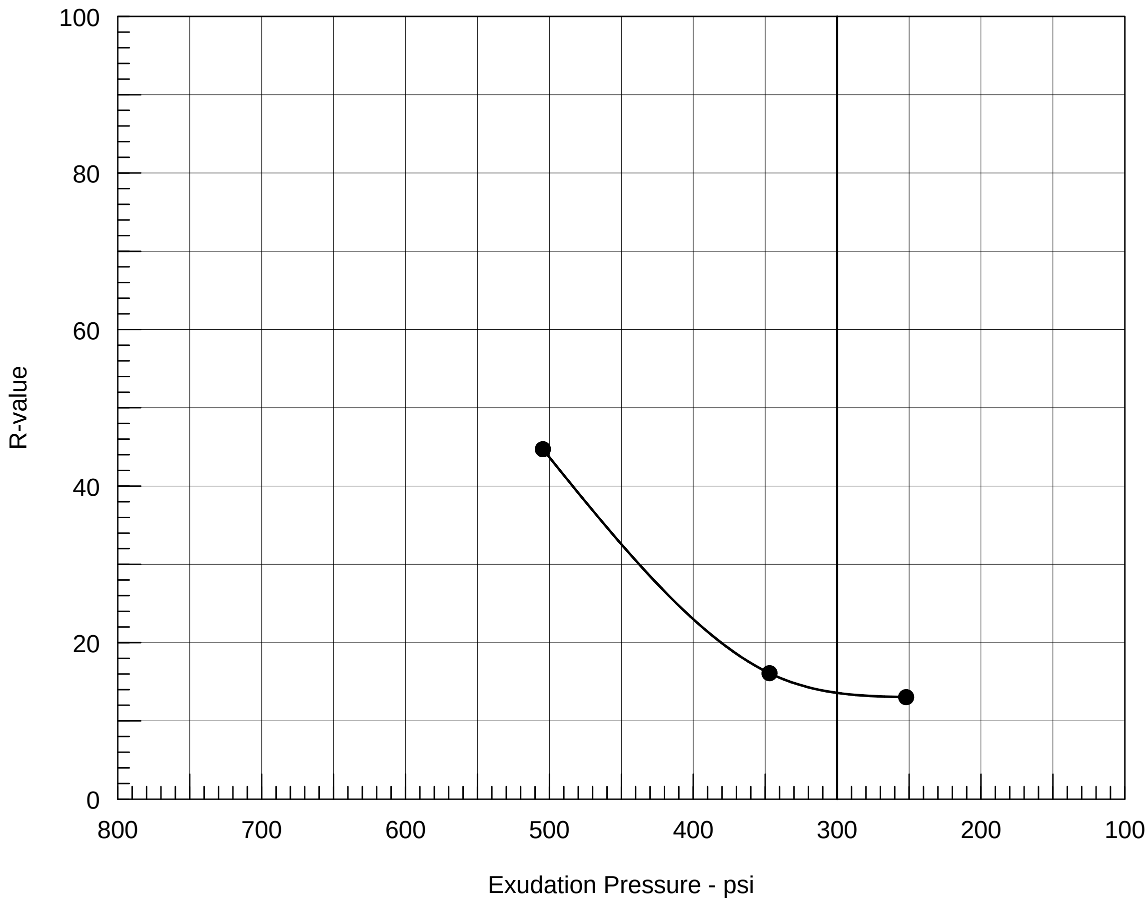
○ RECEIVED 8/7/2015

□ RECEIVED 8/7/2015

Figure B-1

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

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	260	108.5	18.3	0.55	78	2.45	505	45	45
2	120	106.3	20.1	0.21	123	2.45	347	16	16
3	75	104.3	21.1	0.00	130	2.50	252	13	13

Test Results	Material Description
R-value at 300 psi exudation pressure = 14	clayey sand with gravel (SC)

Project No.: 1789

Project: OLD WASHOE TANK

Source of Sample: TEST PIT 1 **Depth:** 0.0'-4.0'

Sample Number: 1A

Date: 8/12/2015

R-VALUE TEST REPORT

Construction Materials Engineers, Inc.

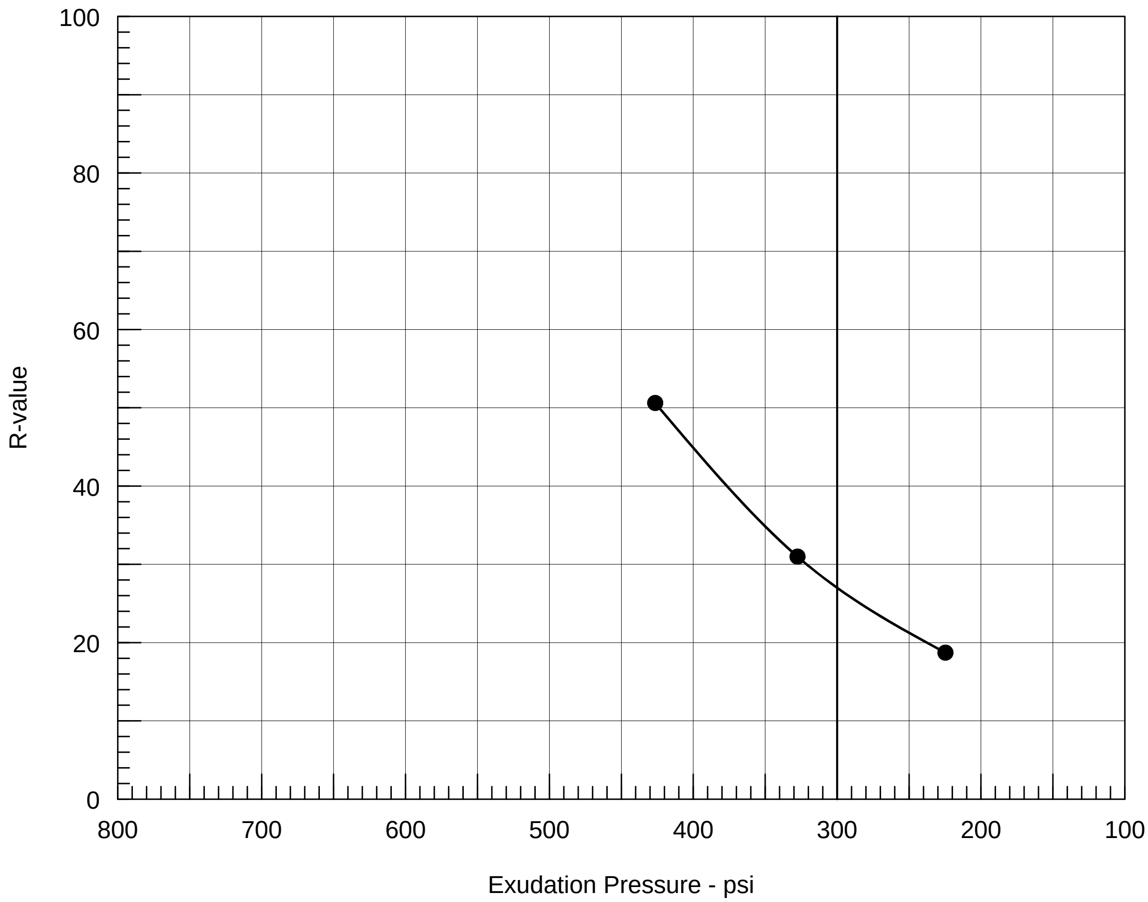
Tested by: G. MORALES

Checked by: S. VINEIS

Remarks:
RECEIVED 8/7/2015

Figure B-2

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	290	105.7	19.0	0.21	68	2.50	426	51	51
2	180	103.4	22.6	0.09	100	2.50	328	31	31
3	100	100.6	23.4	0.03	117	2.55	225	19	19

Test Results	Material Description
R-value at 300 psi exudation pressure = 27	clayey gravel with sand (GC)

Project No.: 1789

Project: OLD WASHOE TANK

Source of Sample: TEST PIT 2 **Depth:** 1.5'-4.0'

Sample Number: 2A

Date: 8/12/2015

Tested by: G. MORALES

Checked by: S. VINEIS

Remarks:
RECEIVED 8/7/2015

R-VALUE TEST REPORT

Construction Materials Engineers, Inc.

Figure B-3