SLOPE STABILITY INVESTIGATION AND REPAIR RECOMMENDATIONS

CHALK BLUFFS WATER TREATMENT PLANT HIGHLAND DITCH CANAL RENO, NEVADA

PREPARED FOR:

TMWA

JUNE 2018
FILE: 2056

CONSTRUCTION MATERIALS ENGINEERS, INC.

6980 Sierra Center Parkway, Suite 90
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June 26, 2018

Mr. Chris Struffert P.E.
TMWA
1355 Capital Boulevard
P.O. Box 30013
Reno, Nevada 89520-3013

RE: Slope Stability Investigation and Repair Recommendations
Chalk Bluffs Water Treatment Plant
Highland Ditch Canal
Reno, Nevada

Dear Mr. Struffert,

Construction Materials Engineers Inc. (CME) is pleased to submit the results of our slope stability investigation and repair recommendations for the Highland Ditch Canal located at the Chalk Bluffs Water Treatment Plant in Reno, Nevada.

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

Please feel free to call us should you have any questions or require additional information.

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

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1.0 INTRODUCTION

Presented herein are the results of Construction Materials Engineers Inc. (CME) geotechnical exploration, laboratory testing, associated slope stability investigation, and repair recommendations for the TMWA Chalk Bluffs Water Treatment Plant Highland Ditch Canal Improvements. These recommendations are based on surface and subsurface conditions encountered during our field exploration, and on details of the proposed project as described in this report. The objectives of this study were to:

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

2. Provide recommendations for design and construction of the project, including a long-term slope stabilization solution, as related to these geotechnical and ground water conditions.

A subconsultant, Kane Geotech, will provide design plans for a temporary soil nail wall, as discussed in this report.

Construction plans (CME 2018), for the slope stabilization repair, shall be included as part of this report.

The area covered by this report is shown on Plate A-1 (Exploration Location Map) in Appendix A. Our study included field exploration, laboratory testing, and engineering analysis 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 SITE CONDITIONS AND PROJECT DESCRIPTION

The subject segment of the Highland Ditch Canal is located approximately in the middle of a steep slope along the east side of a ravine. It appears that a cut bench was excavated into the slope face to allow construction of the canal. Water discharged into the canal originates from a siphon that extents from the west to east side of the ravine.

Based on Washoe County GIS topography, the horizontal cut bench has a width that ranges from about 34 to 37 feet. Based on field measurements, the canal consists of a trapezoidal concrete lined ditch with a width of about 23 feet and depth of 6 feet. The canal is located along the east side of the cut bench directly below a steep cut slope. An approximate 11 to 14-foot-wide access road is located along the west side of the ditch adjacent to a steep fill slope. The fill slope appears to have a slope gradient ranging from about 1H:1V to 1. 5H:1V. The total elevation difference between the top of slope and the bottom of the ravine is about 84 feet. The access road along the canal has a length of about 300 feet.

It is understood that the Chalk Bluffs Water Treatment Plant was constructed about 25 years ago. Originally, the canal was lined with a grouted rip-rap. In 2005, the grouted rip-rap was removed and a concrete liner was constructed.

It is further understood that since the original construction, slope stability issues in this segment of the canal have not occurred. However, after the winter of 2016 to 2017, tension cracks, paralleling the edge of the slope, were noticed within the access road. These tension cracks indicate that the slope is unstable and further widening of the cracks could indicate impending slope failure. The primary concern is that if the slope fails, the canal will be breached.

The canal flows year-round and is critical to plant operations. Consequently, stoppage of water flow or diversion during the construction activities is not achievable. Construction recommendations and activities should be planned to protect the existing canal.

3.0 FIELD EXPLORATION

The intent of the field exploration is fivefold:

- Determine the geotechnical profile in the access road including the depth of the existing fill soils;
- Classify the fill soils and determine approximate in-place densities;
- Determine the location of the tension cracks;
- Determine the material type and structural fill available in an existing stockpile located on TMWA property near the canal;
- Sample soils for laboratory testing.

3.1 Test Pits

A total of 12 exploratory test pits were excavated with a backhoe at the following locations:

- Canal access road was explored in August 2017 by excavating 5 test pits;
- The soil stockpile was explored in October, 2017 by excavating 4 test pits;
- An additional 3 test pits were excavated in October, 2017 near the existing paved access road located at the base of the slope.

The maximum depth of exploration was 9 feet below the existing ground surface. Bulk soil samples for laboratory testing were collected at designated depths in representative soil horizons.

3.2 Exploration Locations and Ground Elevations

Test pit locations were determined by approximate methods referencing existing site improvements as presented on the Site Plan-Plate A-1 in Appendix A. Ground surface elevations were determined by linear interpolation between ground contour line elevations presented on an existing topographic map and should be considered approximate.

3.3 Material Classification

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

4.0 LABORATORY TESTING

All soil testing performed in the CME's 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. Test results are presented in Appendix B.

4.1 Index Testing

Samples of representative soil types were analyzed to determine their *insitu* moisture content (ASTM D 2216), grain size distribution (ASTM D 422), and plasticity index (ASTM D 4318). Results of these tests were used to classify the soils according to ASTM D 2487. Based on the index test results, field logs were reviewed and updated as appropriate. Test results are presented on Plate B-1.

4.2 Laboratory Moisture-Density Relationship Test

Moisture density relationship tests (ASTM D 1557) were completed on selected samples of fill soils and bedrock. This test provides a maximum dry density used to compare with the in-situ dry density of the soil to determine relative compaction. Optimum moisture content is also obtained from this test, which represents the moisture content of the soils at its maximum dry density. The test results were used to remold test samples for the direct shear test. Results of these tests are shown on Plate B-2.

4.3 Direct Shear Test

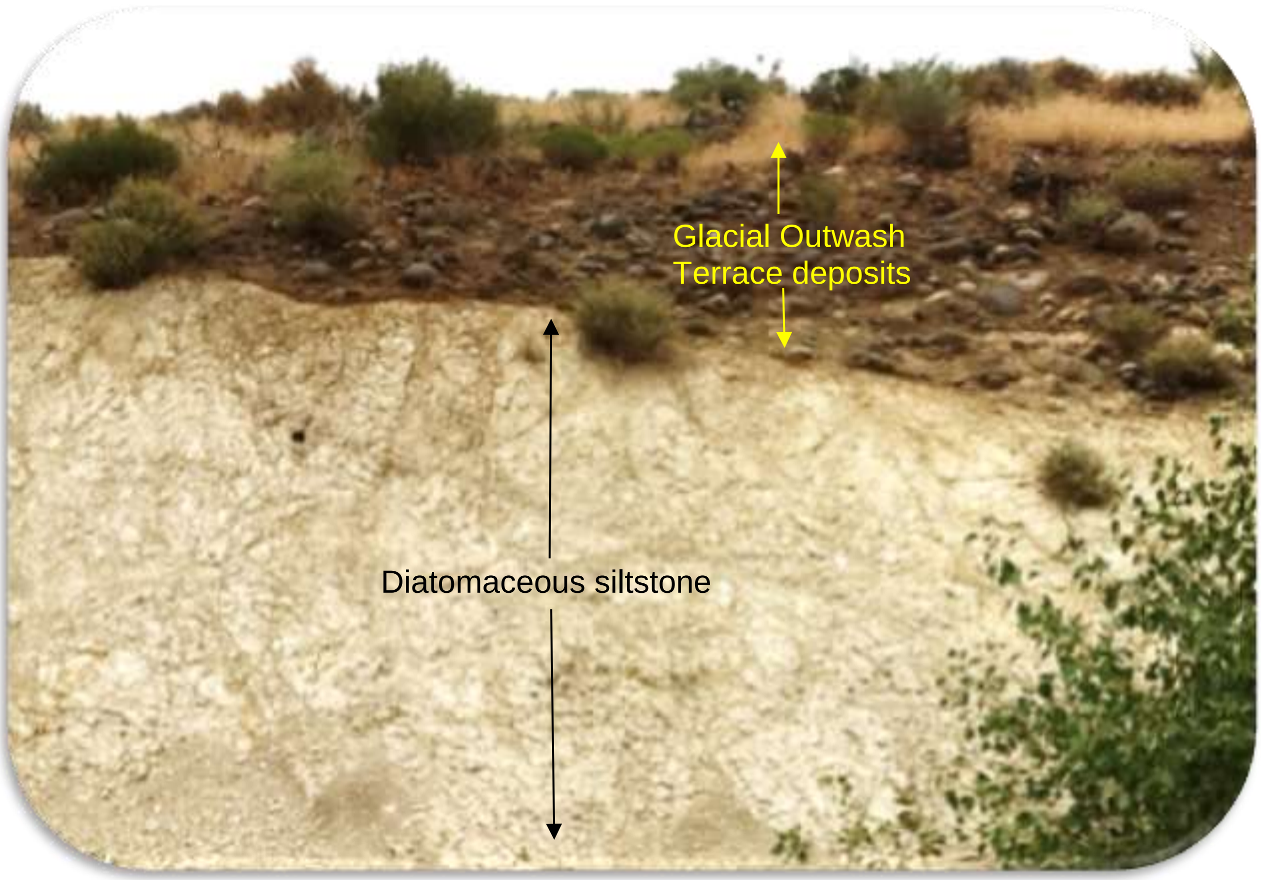
Direct shear tests (ASTM D 3080) were performed on selected samples of the bedrock and existing fill soils, screened to remove particles larger than the number 4 sieve. Tests were run on (in-situ or remolded) soil samples, saturated, and tested at three different normal pressures to derive a plot of Mohr's Circle Failure Envelope. Results of these tests are shown on Plate B-3.

5.0 GEOLOGIC AND GENERAL SOIL PROFILE DESCRIPTIONS

5.1 Regional Geologic Profile

Based on the Geologic Map for the Reno Quadrangle (Bonham and Bingler, 1973), the project site is located among several different geologic units. The uppermost geologic unit is mapped as terrace and glacial outwash deposits of the Truckee River, which is part of the Tahoe Outwash Formation. This Formation is a glacial outwash deposit of Pleistocene age that occurred during periods of catastrophic flooding and is characterized as a heterogeneous mixture of sands, gravels, cobbles and boulders. Boulder-sized particles up to 16 feet in diameter have been encountered in this deposit (Bingler, 1975).

Underlying the Tahoe Outwash Formation is the Sandstone of Hunter Creek Formation. This formation is comprised of siltstones, sandstones, conglomerates, and diatomaceous siltstone. The predominant bedrock type encountered is a diatomaceous siltstone having the following encountered physical and structural properties: intensely to closely fractured; moderately soft; weak; and moderately to deeply weathered. When excavated this material has similar soil properties, determined by visual classification, as an elastic silt **(MH).**



Photograph 1: Cut slope behind Highland Ditch Canal showing the different geologic units

Diatomaceous siltstone is a unique material because even though it's fine-grained with a high plastic index, it still possesses a high internal strength as related to resilient modulus and shear resistance. This siltstone is comprised of diatoms, which are microscopic, single celled plants that secrete siliceous frustules. These siliceous particles are hard, very porous, and angular, which gives the material its high frictional strength. Also, because the material is porous, it has a high absorption characteristic and typically has a high in-place and optimum moisture contents.

5.2 Canal Access Road

The geologic profile encountered in the access roadway consisted of a fill soil layer directly overlying the diatomaceous siltstone.

The existing fill soils encountered in the access roadway had a variable thickness, ranging from 5 to 8.5 feet. Fill soils classified as either a silty sand with gravel, cobbles, and boulders **(SM)** or clayey sand with gravel, cobbles and boulders **(SC).** Boulders up to 3 feet in diameter were encountered. Fills soils overlaid the previously described diatomaceous siltstone formation.

5.3 Stockpile Area

Soils encountered in the stockpile area had a variable soil classification. Three predominant soil types were encountered: silty sand with gravel **(SM)**, clayey sand with gravel, cobbles, and boulders **(SC),** and poorly graded gravel with silt, sand, cobble, and boulders **(GP-GM).** These soils are heterogeneously located in the stockpile. Boulders up to 2 feet in diameter were also encountered. Soils encountered appeared granular and exhibit low to moderate plasticity characteristics.

5.4 Soil Moisture and Groundwater Conditions

Generally, soils were encountered in a slightly moist to moist soil condition. Ground water was not encountered during exploration and is expected to lie at a depth well below that which would affect construction.

6.0 SEISMIC HAZARDS

6.1 Seismicity

Much of the Western United States is a region of moderate to intense seismicity related to movement of the crustal masses (plate tectonics). By far, the most active regions, outside of Alaska, are along the San Andreas Fault zone of western California. Other seismically active areas include the Wasatch Front in Salt Lake City, Utah, which 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.

It is generally accepted that the maximum credible earthquake for this area would have a magnitude in the range of magnitude 7 to 7.5 and likely originate from the frontal fault system of the Eastern Sierra Nevada (Carson Range). The most active segment of this fault system that is closest to the Reno-Stead area is located at the base of the eastern flank of the Carson Range near Thomas Creek, Whites Creek and Mt. Rose Highway, about 10 miles south of the project site.

6.2 Faults

Based on a review of the Reno Folio Earthquake Hazards Map, Bingler, 1974, the map shows a fault trending in a northeasterly direction through the east side of the project site. Several other faults are located within a ½ mile radius of the canal northeast of the site.

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

Based on the referenced fault map, faults in the vicinity of the project are considered Quaternary Active Faults.

6.3 Liquefaction

Liquefaction is defined as a nearly complete loss of soil shear strength occurring during an earthquake, as cyclic shear stresses generate excessive pore water pressure between the soil grains. Soil liquefaction susceptibility depends on several factors including subsurface soil profile, ground water table, relative density, ground acceleration, and duration of shaking.

Soil types most susceptible to liquefaction include loose to medium dense cohesionless sands, soft to stiff non-plastic to low plastic silts, or any combination of silt-sand mixtures lying below the groundwater table. Liquefaction is generally limited to depths of 50 feet or less below the existing ground surface. Because of the depth of the groundwater and near surface bedrock, soil liquefaction potential, in our opinion, is negligible.

7.0 SEISMIC DESIGN PARAMETERS

Seismic design parameters are based on site-specific estimates of spectral response ground acceleration as designated in the 2012 IBC. This approach allows the development of a response spectrum; and based on the period of the structure, a spectral acceleration for that structure can be determined. Seismic design parameters can be determined from the site classification and location (latitude and longitude). Site classification is based on the substrata soil profile type, as presented in Table 1 (Site Classification Definitions).

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

A 100-foot deep boring or geophysical method are required to characterize the soil profile in sufficient detail to determine the site classification. If neither of these field exploration methods are performed, the IBC allows the use of a default site classification of D depending on if other geologic conditions do not exist that would justify a lower site classification (E or F). Based on our field exploration and knowledge of the site geologic conditions, it is our opinion that a default Site Classification of C is appropriate to use in the design of the structures.

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 and site classification other than B. Therefore, IBC provides correction factors (F_a & F_v) to modify the acceleration values depending on the subsurface geologic conditions (site classification).
Spectral response acceleration values were determined from the USGS website: *U.S. Seismic Design Maps* Table 2 (Seismic Design Parameters) 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 design information including spectral response acceleration values is provided in Appendix C.

Table 2 – Seismic Design Parameters	
Approximate Latitude of Site	39.51457
Approximate Longitude of Site	-119.86949
Peak Ground Acceleration-MCE_R PGA (ASCE 7-10 Standard)	0.604 g
Spectral Response Acceleration at Short period (0.2 sec.) S_s (for Site Class B)	1.685 g
Spectral Response Acceleration at 1-second Period, S_1 (for Site Class B)	0.600 g
Site Class Selected for this Site	C
Site Coefficient F_a, decimal	1.0
Site Coefficient Fv, decimal	1.3
Design Spectral Response Acceleration at Short period, S_{Ds} (Adjusted to Site Class B, SDs= 2/3 SMs)	1.124 g
Design Spectral Response Acceleration at 1-second Period, S_{D1} (Adjusted to Site Class B, SD1=2/3 SM1)	0.520 g
1) MCE_R PGA- Maximum credible earthquake geometric mean peak ground acceleration.	

8.0 DISCUSSION AND RECOMMENDATIONS

The existing slope adjacent to the canal and access road is showing signs of instability with the presence of tension cracks. Based on our field exploration, it appears that the access road was constructed with fill soils having thicknesses ranging from about 5 to 8½ feet. These fill soils have a heterogenous composition consisting of a mixture of soils, cobbles, boulders (up to 3 feet in diameter), and some concrete debris. Based on density testing, these fill soils appear to have been loosely placed, which may have contributed to the slope instability. It is recommended that these fill soils are completely removed from the embankment area.

8.1 General Information

The recommendations provided herein, and particularly under **Construction Recommendations** and **Construction Observation and Testing** 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 structural integrity/performance of the planned structure and related improvements could be affected. Sufficient construction observation and testing should be performed to document that the recommendations presented in this report are followed.

Structural areas referred to in this report include all areas of the repaired slope. All compaction requirements presented in this report are relative to ASTM D 1557[1]. Unless otherwise stated in this report, all related construction should be in accordance with the Standard Specifications for Public Works Construction (SSPWC), dated 2016.

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 Conceptual Slope Repair Design Assumptions

It is understood that tension cracks, paralleling the edge of the slope, are present within the access road. Tension cracks were likely formed due to lateral movement in the uppermost fill soil layer. The concern is that if the slope fails, the canal will be breached.

To provide slope repair options, the following conceptual design parameters were determined:

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

- The canal flows year-round, so stoppage of water flow or diversion during the construction activities is not considered a viable option during construction;
- The canal cannot be disturbed during construction;
- Existing embankment fill soils shall be removed;
- To prevent canal disturbance during construction, a temporary support wall such as a soil nail wall to provide lateral support for the canal, while the permanent slope repair is being constructed, is required. The challenge with the soil nail wall is the location and depth of the soil nails, so the canal is not disturbed. The soil nail wall or other temporary support systems will be evaluated and designed to prevent canal disturbance;
- Disturbance of the existing slope, outside the repair footprint, shall be kept to a minimum.

8.3 Slope Repair Construction Options

Based on the conceptual design parameters, several construction options were considered including a slope buttress, reinforced slope, and retaining wall.

8.3.1 Slope Buttress

The slope buttress repair concept consists of constructing a fill wedge starting from the base of the slope. The fill wedge will have a 2H:1V slope gradient and will be benched into the hillside. The following design and construction considerations are required:

- Due to the substantial quantity of fill of material required, importing fill material to the site would be uneconomical. However, a large stockpile of fill material is located within the southeast corner of the Chalk Bluffs site of which a portion is usable for the construction of the fill slope;
- An existing paved access roadway is located at the base of the slope. The fill slope may impact/encroach this roadway requiring modifications/relocation to the existing roadway alignment. Alternate options are to either construct a retaining wall or a reinforced steepened slope along the uphill side of the road;
- Fill material located in the access road is recommended to be removed and replaced with a densified structural fill material. Removal of the existing fill will require constructing a temporary support wall, such as a soil nail wall, to assure that the canal will not be disturbed.

8.3.2 Retaining Wall

A retaining wall could be constructed to support both the access road and canal. The retaining wall would be placed directly at the edge of the access road and have a height that ranged from 4 to 12 feet. The Tensar Sierra Slope retaining wall system is recommended for ease of construction. This retaining wall has a staggered front face (6-inch offset for every 18 inches in vertical height) and consists of a welded wire structural face that is support by geogrid backfill soil reinforcements. This system allows a fully landscaped vegetative surface on the front face of the retaining wall.

Retaining wall construction consists of the following steps:

- Remove the existing fill soils and underlying native bedrock to a sufficient depth to allow construction of the retaining wall. Removal would require temporarily supporting the canal with a soil nail wall;
- The soil nail wall would be constructed incrementally, as the existing soils are removed;
- To reduce the height and to provide base support of the wall, structural fill shall be placed below the wall;
- The final step is the construction of the retaining wall.

8.3.3 Construction of a Steepened Reinforced Slope

A reinforced slope will allow the construction of a steepened slope with an approximate gradient of about 1.6H:1V. Reinforcement would consist of placing geogrid within the embankment fill material. A steepened slope face would have the advantage of a reduced fill quantity and limiting the area of slope disturbance.

8.4 Recommended Slope Repair Construction Option

The recommended slope repair option is the steepened reinforced slope. All three construction options were reviewed by a local contractor (Q&D Construction) to provide construction costs. The steepened reinforced slope option had the lowest construction costs. This construction option will require less structural fill than the slope buttress option and will have less site disturbance. A disadvantage of the retaining wall option was the steep side slope and the need for K-rails or other railing along the access road.

8.5 Slope Stability Analysis

Geotechnical modeling for the slope stability analysis was characterized as having two geologic units: structural slope fill overlying diatomaceous siltstone bedrock. The analysis assumes that slope backfill soils will be keyed directly into the underlying diatomaceous siltstone bedrock and all existing overburden fill soils will be removed. Several sequential analytical steps are required to complete the slope stability analysis, as follows:

1. **Determine the geometry of the slope (both finished and underlying bedrock slope gradients).**

   The finished grade of the slope is designed at 1.6H:1V. Based on the elevation of the underlying diatomaceous siltstone encountered beneath the access roadway and exposures in the existing slope, the assumed slope gradient of the diatomaceous siltstone is similar, at about 1.6H:1V.

2. **Surcharge Loading**

   The slope will experience surcharge loading from vehicle loading traveling on the access road. However, the heaviest loading on the slope will occur during construction. Construction loading consisting of a 20-kip axle loading was assumed in our analysis.

3. Strength parameters and unit weights of fill soils and underlying bedrock

Strength parameters are provided in Table 3 (Soil and Bedrock Design Parameters).

Table 3 – Soil and Bedrock Design Parameters

Soil and bedrock Type	Phi Angle (Ø)	Cohesion (psf)	Unit Weight (pcf)
Structural fill	28	250	120
Diatomaceous Siltstone	37	165	75

4. Seismic Parameters

The peak ground acceleration determined for this area (USGS-ASCE 7-10) is 0.60g. However, because of the height of the slope, a reduction in the peak ground acceleration is recommended (NCHRP, 2008). Analytical studies completed, as presented in the referenced report by the NCHRP, included seismic wave scattering to determine the average ground acceleration within the slope, as a function of slope height. These studies evaluated the changes in ground motion within the soil mass behind the slope face. The consequence of the variation in ground motion is that the average ground motion within the slope is less than the instantaneous acceleration peak value within the slope. Based on the results of this study, as recommended by NCHRP (2008), the adjusted peak ground acceleration used for the slope stability analysis is 0.50g.

5. Complete a slope stability analysis of the un-reinforced slope to determine if reinforcement is required.

The computer program ReSSA 3.0 (Adama Engineering Inc., 2001 to 2011) was utilized to perform slope stability analyses. This program performs a two-dimensional limit equilibrium analysis to compute the factor of safety (FOS) for a layered slope. The limit equilibrium analysis was performed using the simplified Bishop method. This method satisfies vertical force equilibrium for each slice and overall moment equilibrium about the center of the circular trial forces. The slope stability analysis was performed for both static conditions and pseudostatic conditions. The minimum factor of safety values used for this analysis is 1.5 for static conditions and 1.1 for pseudo-static conditions.

The program utilizes the pseudo static method for evaluating the stability of the slope for seismic conditions. The pseudo static method simulates potential inertial forces due to ground accelerations during an earthquake by including horizontal and vertical static seismic forces. These seismic forces are assumed to be proportional to the weight of the potential sliding mass times a seismic coefficient (k_h – horizontal seismic coefficient), expressed in terms of the accelerations of the underlying earth.

The vertical acceleration component was not used in our slope stability analysis. When the vertical acceleration is less than the horizontal component (vertical acceleration typically used in slope stability analyses is ⅔ of the horizontal component), studies have shown that the application of a vertical acceleration in the limit equilibrium analysis will change the horizontal yield acceleration by no more than 10 percent (Munfakh et al). The reason for this low percentage is that the vertical ground motions are generally out of phase with, and of different frequency than the horizontal ground motions. It is therefore a reasonable assumption to ignore the vertical acceleration.

Pseudo-static slope stability analysis using peak ground acceleration in conjunction with a factor of safety of 1.0 provides *excessively conservative assessments* of slope stability (FHWA 1997). Consequently, the seismic coefficient used in slope stability analysis is less than the peak ground acceleration and typically ½ of the peak ground acceleration. The reason is that the alternating inertia forces are of short duration and change direction many times during the seismic event. Because of the change in direction, the factor of safety may fall below 1.0 for a short duration, but during the reverse direction will be above 1.0. Slope deformations will occur when the factor of safety falls below 1.0, but the cumulative deformations during the earthquake are usually tolerable with some repair to the slope face after the earthquake event. The peak ground acceleration determined for this area is 0.50g and a horizontal coefficient of ½ the maximum PGA, or 0.25 g was used in our pseudo-static slope stability analysis.

The unreinforced slope analysis indicates a factor of safety of less than 1.5 for the static condition and 1.1 for the seismic condition. Consequently, slope reinforcement is required.

6. Determine design parameters for the repaired slope reinforcement

The first step for a reinforced slope design is to evaluate design parameters for the reinforcement. Because of its strength and ease of construction, it is recommended to use a structural geogrid reinforcement consisting of a high-density polyethylene. The primary reinforcement strength parameter is the ultimate tension strength (T_{ult}), based on the minimum average roll values (MARV) of the material. The value is reduced to account for creep, installation damage, and durability.

The creep reduction factor is determined by comparing the long-term creep strength to the average ultimate tensile strength. The installation damage factor reduces the long-term strength to account for the effect of installation damage on the geogrid reinforcement. The durability reduction factor is dependent on the susceptibility of the geogrid to attack by microorganism, oxidation, hydrolysis, etc. The recommended reinforcement is a Tensar UX 1600HS uniaxial structural geogrid or equivalent product (Tensar product information is included in Appendix D). Recommended reduction factors used for this geogrid are as follows:

- Creep reduction factor (RFcr) = 2.6
- Installation Damage Reduction Factor (RF_{ID}) = 1.4
- Durability Reduction Factor (RF_D) = 1.0

Reduction factors are also required for the soil reinforcement interaction coefficients consisting of pullout resistance and interface shear strength. The pullout resistance is mobilized by the interface friction and cohesion between the soil backfill and the geogrid. Strength parameters have previously been given for the structural fill. FHWA recommends that a reduction factor be applied to these strength parameters to determine the frictional resistance between the geogrid and structural fill. A reduction factor of 0.80 was used in our analysis.

7. Reinforced Slope Stability Analysis

Two different slope stability analysis were completed consisting of a rotational and translational evaluation. The translational analysis is a two wedge analysis that considers horizontal movement along the reinforced layer interface. Using a 5-foot placement interval for the geogrid reinforcement, the FOS exceeded 1.5 for that static and was about 1.1 for the pseudo static analysis. Slope stability analyses are included in Appendix E.

9.0 CONSTRUCTION RECOMMENDATIONS

9.1 Site Preparation

Prior to constructing the new embankment reinforced fill slope, it is recommended to entirely remove the existing fill slope soils and prepare the slope face for the construction of the reinforced slope. After processing, existing fill soils may be reused as embankment fill if they meet the requirements provided in Section 1.3. Removal of the existing fill soils shall be coordinated with the construction of the temporary soil nail support wall. It is recommended that the soil nail support wall is constructed in two stages:

- The first construction stage is to complete a 3 to 4 foot deep excavation in the canal access road and install the upper half of the soil nail wall;

- Completion of the remaining soil nail wall.

Following installation of the soil nails, a 1.5H:1.0V or flatter cut slope shall be constructed below the base of the soil nail support wall terminating at the uppermost construction bench. Horizontal benches shall be constructed starting at the toe of new embankment slope (refer to construction plans).

As the existing fill soils are removed, all vegetation and topsoil should be stripped and grubbed from structural areas and removed from the site. A stripping depth of 0.5 feet is anticipated. Deeper areas of localized stripping and grubbing to remove organic zones may be required and will be determined during construction. All stripping and grubbing material shall be removed off site. Existing fill shall be completely removed from the access road area, adjacent to the canal, and from the improved slope footprint. It is anticipated that diatomaceous siltstone will be encountered below the existing fill soil layer.

Except for diatomaceous siltstone areas, 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 including uniform mixing of the site soils to achieve required soil moisture content recommendations may be required. It is recommended that the moisture content of the in-situ soils be determined during construction to evaluate if moisture conditioning is required. After the densification process, a firm, stable surface should be produced. Unstable soils, where encountered, should be removed and replaced with structural fill.

Where diatomaceous siltstone is encountered, the bedrock surface shall be cleaned of all loose particles and structural fill can be placed directly on this surface.

9.2 Trenching and Excavation

Excavations will require sidewalls to 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) require that the temporary sidewall slopes be no greater than those presented in Table 1.

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).

These regulations, including the classification system and the maximum slopes, have been adopted and are strictly enforced by the State of Nevada, Department of Industrial Relations, Division of Occupational Safety and Health. 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 bedrock (diatomaceous siltstone) appear to be predominately Type B, with overburden soils being Type C, although variations will exist. Any area in question should be considered Type C unless specifically examined by the geological engineer during construction. All trenching and excavated slopes should be performed and stabilized in accordance with local, state, and OSHA standards. In any case bank stability will remain the responsibility of the contractor, who is present at the site, able to observe changes in ground conditions, and has control over personnel and equipment.

9.3 Grading and Filling

Structural fill is defined as supporting soil placed within the slope and below the access road. Embankment fill should be free of vegetation, organic matter, and other deleterious material.

It is assumed that the existing fill soils to be removed from the slope area and available material from an existing soil stockpile on-site, after processing, will be used as embankment structural fill. Based on our field exploration, existing slope fill and soil stockpile material contains abundant cobbles and some boulder sized particles with diameters of up to 36 inches. To reduce potential damage to the geogrid, structural fill should not contain any particles greater than 4 inches. Based on the material encountered, fill removed from the slope area and stockpile will be required to be screened through a grizzly to remove plus 4-inch particles. Diatomaceous siltstone material can be used as structural fill if placed between the geogrid layers. Screened cobbles and boulders shall also be placed in the existing stockpile area.

Table 5 provides guideline specifications for embankment fill.

Table 5 - Guideline Specification for Embankment Fill	
Sieve Size	**Percent by Weight Passing**
4 Inch	100
¾ Inch	70 – 100
No. 40	15 – 70
No. 200	5 – 40
Maximum Liquid Limit	**Maximum Plastic Index**
40	20

Based on the index test results of the existing embankment material, the bulk of this material, when properly screened, should meet the requirements given in Table 2, although additional laboratory testing during construction will be required. Similarly, it is anticipated that the majority of the existing stockpile material, except material obtained from Test Pit TP-9, after screening, should meet the requirements of Table 2. Consequently, additional exploratory test pits and laboratory testing will be required to locate acceptable fill soils in the existing stockpile.

Structural fill should be placed in 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, 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.

It is recommended that heavy construction equipment, such as large vibratory roller, not be used to densify fill soils near the edge of the slope. A smaller compactor should be used near the slope face. It is recommended that a test section be completed to determine if damage is occurring to the geogrid from the compaction equipment. After completion of the fill densification in the test section, the material should be removed at random locations to observe if damage to the geogrid has occurred. Regardless of compaction equipment, the thickness of the structural fill layer should not be less than recommended.

All fill soils placed on native soils/bedrock with slope gradients steeper than 5H:1V (horizontal to vertical) should be placed on horizontal benches excavated into the existing slope face, at least 8 feet in width, beginning at the toe of the slope. Additionally, a trench key should be constructed at the toe of the slope. A drain shall be placed in the trench key, embedded in drain rock, and sloped to drain to a suitable non-erodible discharge point (refer to referenced construction drawings).

Grading should not be performed with frozen soils or on frozen soils.

9.4 Geogrid Placement and Construction Handling

Geogrid shall be placed on a prepared surface. The surface shall be cleared of all obstacles and should be smooth and level. The intent of the surface preparation is to provide a surface within depressions or voids to allow adequate bonding of the geogrid with the backfill soils.

Before unrolling the geogrid, verify the roll indentation, length, and installation locations with the grading plans. While unrolling the geogrid, inspect for damage and defects.

Orientation of the geogrid is of extreme importance since the geogrids may vary in strength and direction. The geogrid panel length should be measured in the field prior to being rolled out and cut to length. The geogrid panel length should be measured in the field prior to being rolled out and cut to length.

After geogrid has been laid in place, tension by hand until taut, free of wrinkles and lying flat. Geogrids shall be placed perpendicular to the slope face and rolled back to the underlying exposed slope face. Adjacent geogrid panels shall be butted against each other. Some overlay maybe required to assure 100 percent surface coverage. Geogrid panels may be secured in-place with staples, pins, sand bags, or backfill as required by fill properties, fill placement procedures, weather conditions, or as directed by the engineer.

The geogrid may not be spliced in the principal strength direction (perpendicular to the slope face) through overlap. A mechanical connection is available through the manufacturer if required. The geogrid should be installed on one continuous piece with the principal strength direction extending the full length of the reinforced area.

Place only that amount of the geogrid required for immediately pending work to prevent undue damage. After a layer of geogrid has been placed, the succeeding layer of soil shall be placed, compacted and prepared as appropriate. After the specified soil layer has been placed, the next geogrid layer shall be installed. This process shall be repeated for each subsequent layer of geogrid and soil.

9.5 Temporary Protective Measures

It is recommended to cover this access road and upper portion of the slope, until permanent repairs can be constructed, with a minimum 15 mil thick visqueen or tarps. The visqueen or tarp should be covered with sufficient dirt for protection from damage due to vehicle travel, if required. The purpose of the covering is to reduce the moisture penetration into the fill soils, which could promote further slope deformation.

9.6 Erosion Control

Erosion potential depends on numerous factors involving grain size distribution, cohesion, moisture content, slope angle and the velocity of the water or wind on the ground surface. Erosion control is recommended for all cut and fill slopes 5H:1V or steeper. Slopes between 3H:1V and 5H:1V can be stabilized by hydroseeding. Slopes steeper than 3H:1V require mechanical stabilization consisting of rock rip-rap with a minimum of 75 percent of the rock rip-rap 8-inches or greater in diameter. It is recommended that erosion control consists of a rock rip rap meeting the specifications of a Class 150 rock (SSPWC, 2012).

9.7 Recommended Construction Sequencing and Anticipated Construction Difficulties

A soil nail temporary support wall with a height of 7 feet is recommended adjacent to the canal. It is recommended that the existing embankment fill soils are removed in a downward direction starting from the top of the existing slope.

Existing grades including bench cuts shall be verified during construction. The toe of the embankment slope shall be keyed into the existing sedimentary bedrock, which shall be verified during construction. Toe of slope locations may vary from locations presented on the grading plans.

Because of the limited construction area and overall site constraints, the contractor shall carefully coordinate all phases of the project to minimize site disturbance. Coordination with TMWA plant personnel shall also be established.

10.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations presented in this report are based on the assumption that the owner/project manager provides sufficient field testing and construction review during all phases of construction. These construction observation and testing services should include by not be limited to site preparation and grading, foundation grade soil preparation and observation, concrete placement, and asphalt paving.

It is recommended that since our firm prepared this report and have knowledge of the subsurface and surface conditions at the site, CME should be retained to provide these services. Additionally, all plans and specifications should be reviewed by the engineer responsible for this geotechnical report to determine if they have been completed in accordance with the recommendations contained herein. It is the owner's/project manager responsibility to provide the plans and specifications to the engineer.

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

11.0 LIMITATIONS

This report has been prepared in accordance with generally accepted local geotechnical practices. The analyses and recommendations submitted are based upon field exploration performed at the locations shown on Plates A-1 to A-3 in Appendix A of this report. This report does not reflect soils variations that may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary. Sufficient construction observation should be completed in all phases of the project related to geotechnical factors to document compliance with our recommendations.

This report has been prepared to provide information allowing the engineer to design the project. The owner/project manager is responsible for distribution of this report to all designers and contractors whose work is affected by 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[2]. 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[3].

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[2] 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.

[3] 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.

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

REFERENCES

Bingler, E. C., 1974, *Earthquake Hazards Map, Reno Quadrangle*: Nevada Bureau of Mines and Geology, Map 4Ai.

Bonham, H. F. and E. C. Bingler, 1973, *Geologic Map, Reno Quadrangle*: Nevada Bureau of Mines and Geology, Map 4Ag.

Munfakh, G. et al (1998), *Geotechnical Earthquake Engineering Reference Manual* (Publication Number FHWA-HI-99-012), US Department of Transportation, Federal Highway Administration, National Highway Institute, Arlington, Virginia.

National Cooperative Highway Research Program (NCHRP), 2008, *Seismic Analysis and design of retaining Walls, buried Structures, Slopes, and Embankments,* Report 611.

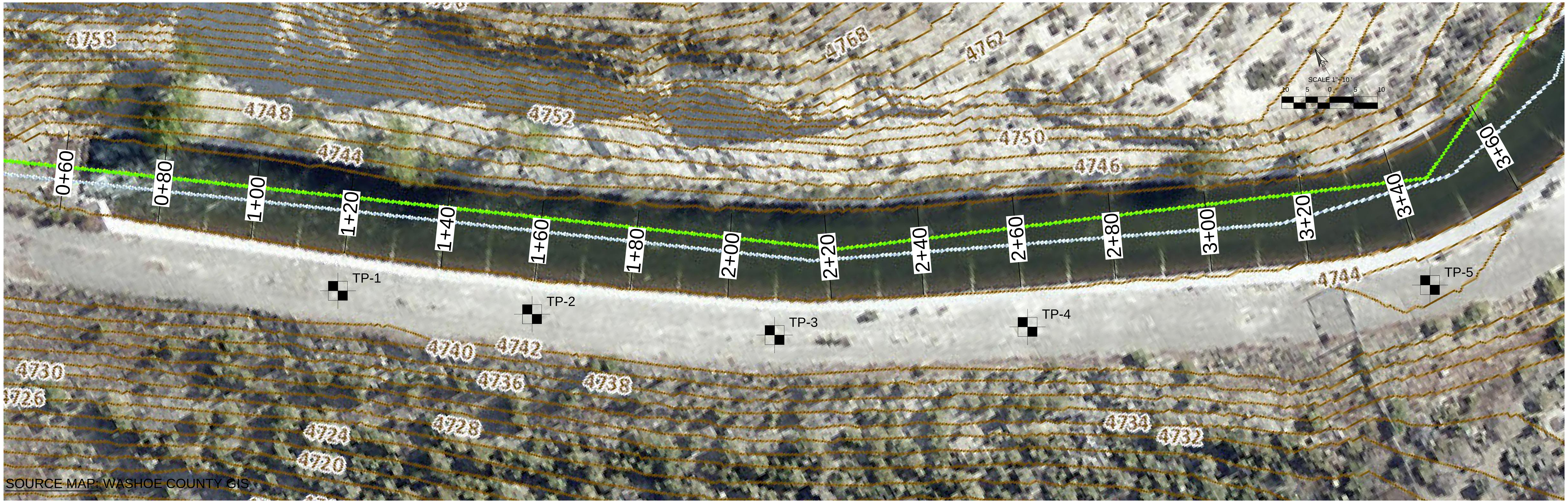
Nevada Earthquake Safety Council, 2006, *Guidelines for Evaluating Potential Surface fault Rupture /land Subsidence Hazards in Nevada.*

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

USGS website: *Earthquake Hazards program to determine Seismic Design Values for Buildings.* (http://earthquake.usgs.gov/research/hazmaps/design/)

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX A



LOG OF TEST PIT NO. TP-1

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description
0 – 7.0	SC			B	1A		MOIST	0.0'-7.0': CLAYEY SAND WITH GRAVEL, COBBLES, AND BOULDERS (FILL), mostly very fine to medium sand, some fine to coarse rounded to subrounded gravel and cobble, few boulders up to 24" in diameter, concrete debris, organics heavy from 0-6" less dense with depth, low plasticity, light brown. NOTE: Density test taken on sidewall of hillside northeast of TP-1 in Diatomaceous Siltstone: -Dry Density of 50.5pcf -Moisture Content of 45.0%
7.0 – 7.5	ROCK						MOIST	7.0'-7.5': SANDSTONE OF HUNTER CREEK FORMATION - DIATOMACEOUS SILTSTONE. NOTE: When remolded has similar properties as an Elastic Silt with Sand. Test pit terminated at 7.5 feet.

LOG OF TEST PIT NO. TP-2

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description
0 – 8.5	SC			B	2A		MOIST	0.0'-8.5': CLAYEY SAND WITH GRAVEL, COBBLE, AND BOULDERS (FILL), mostly very fine to medium sand, little rounded to subrounded gravel and cobble, concrete debris, low plasticity, light brown. NOTE: Dry Density at 3.5' - 76.4pcf MOISTURE CONTENT - 20.9% NOTE: Boulders up to 36" in diameter at 4.5' bgs.
8.5 – 9.0	ROCK						MOIST	8.5'-9.0': SANDSTONE OF HUNTER CREEK FORMATION - DIATOMACEOUS SILTSTONE. NOTE: When remolded has similar properties as an Elastic Silt with Sand. Test pit terminated at 9.0 feet.

LOG OF TEST PIT NO. TP-3

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description
0 – 7.0	SM			B	3A		MOIST	0.0'-7.0': SILTY SAND, WITH GRAVEL, COBBLE, AND BOULDERS (FILL), mostly very fine to medium sand, little fine to coarse rounded to subrounded gravel and cobble, boulders up to 24" in diameter, low plasticity, light brown. NOTE: Dry density at 2.5': 75.8pcf Moisture Content: 21.3%
7.0 – 8.0	ROCK						MOIST	7.0'-8.0': SANDSTONE OF HUNTER CREEK FORMATION - DIATOMACEOUS SILTSTONE. NOTE: When remolded has similar properties as an Elastic Silt with Sand. Test pit terminated at 8.0 feet.

LOG OF TEST PIT NO. TP-4

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description
0 – 5.0	SM			B	4A		MOIST	0.0'-5.0': SILTY SAND WITH GRAVEL, COBBLES, AND BOULDERS (FILL), mostly very fine to medium sand, little fine to coarse rounded to subrounded gravel and cobbles, boulders up to 24" in diameter, low plasticity, light brown. NOTE: Dry Density at 2.5' - 70.1pcf MOISTURE Content at 2.5' - 20.1%
5.0 – 6.0	ROCK						MOIST	NOTE: Dry Density at 4.5' - 72.4pcf Moisture Content at 4.5' - 24.1% 5.0'-6.0': SANDSTONE OF HUNTER CREEK FORMATION - DIATOMACEOUS SILTSTONE. NOTE: When remolded has similar properties as an Elastic Silt with Sand. Test pit terminated at 6.0'.

LOG OF TEST PIT NO. TP-5

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description
0 – 5.0	SM			B	5A		MOIST	0.0'-5.0': SILTY SAND WITH GRAVEL, COBBLES, AND BOULDERS (FILL), mostly very fine to medium sand, few fine to coarse rounded to subrounded gravel and cobble, boulders up to 24" in diameter, low plasticity, light brown. NOTE: Dry Density at 3.0' - 74.1pcf Moisture content at 3.0' -20.1%
5.0 – 10.0	ROCK						MOIST	5.0'-10.0': SANDSTONE OF HUNTER CREEK FORMATION - DIATOMACEOUS SILTSTONE. NOTE: When remolded has similar properties as an Elastic Silt with Sand. Test pit terminated at 10.0 feet.

REVISION NO. | DATE | COMMENT

CME CONSTRUCTION MATERIALS ENGINEERS, INC.
6980 SIERRA CENTER PARKWAY, SUITE 90
RENO, NEVADA 89511

TRUCKEE MEADOWS WATER AUTHORITY
CHALK BLUFFS WATER TREATMENT PLANT
HIGHLAND DITCH CANAL IMPROVEMENTS
TEST PIT LOCATION AND LOGS
RENO, NEVADA
PROJECT NO.: 2056 DATE: 1/30/2018

PLATE
A-1a

VICINITY MAP SHOWING AVAILABLE STOCKPILE AREA

LOG OF TEST PIT NO. TP-6

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description
0	SM			B	6A		SL. MOIST	0.0'-6.0': SILTY SAND WITH GRAVEL FILL, mostly very fine to medium sand, litte fine to coarse subangular to subrounded gravel, trace boulders up to 18" in diameter, low plastic, light gray.
	GC			B	6B		SL. MOIST	6.0'-8.0': CLAYEY GRAVEL WITH SAND, COBBLES, AND BOULDERS, mostly fine to coarse subrounded to rounded gravel and cobbles, some fine to coarse sand, few boulders up to 14" in diameter, low plastic, brown.
								NOTE: Harder excavation at 6.0 feet. Test pit terminated at 8.0 feet.

LOG OF TEST PIT NO. TP-7

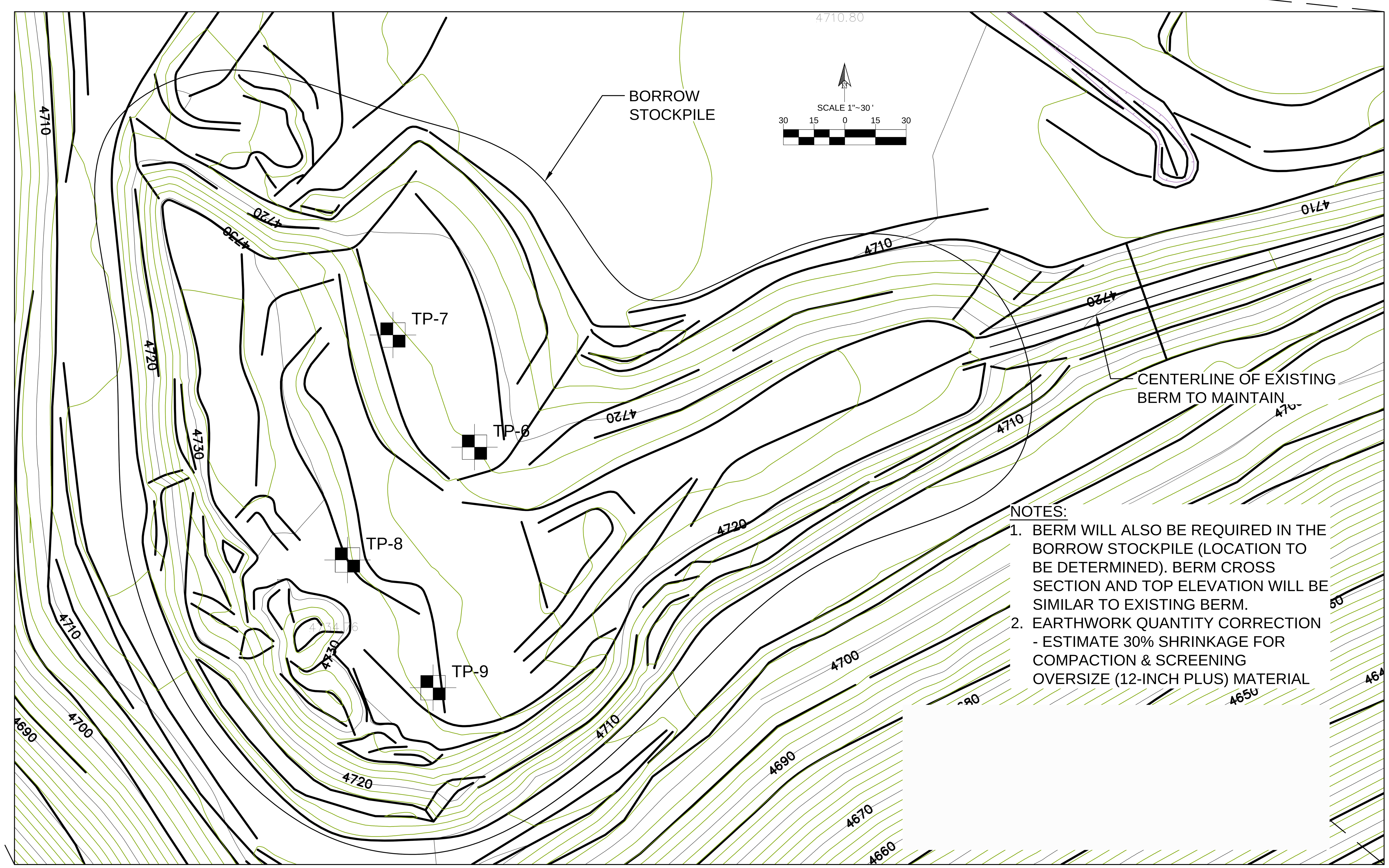
Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description
0	SM			B	7A		SL. MOIST	0.0'-1.0': SITLY SAND WITH GRAVEL FILL, mostly very fine to medium sand, some fine to coarse subangular gravel, low plastic, light gray.
	SC			B	7B		SL. MOIST	1.0'-8.0': CLAYEY SAND WITH GRAVEL, COBBLES, AND BOULDERS, mostly fine to coarse sand, some fine to coarse rounded to subrounded gravel and cobble, boulders up to 24" in diameter, low plastic, dark brown. NOTE: Increased boulder content with depth.
								Test pit terminated at 8.0 feet.

LOG OF TEST PIT NO. TP-8

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description
0	GM			B	8A		SL. MOIST	0.0'-1.5': SITLY GRAVEL WITH WITH SAND AND COBBLES, mostly fine to coarse rounded to subrounded gravel and cobble, little fine to coarse sand, few boulders at the surface, non plastic, brown.
	GP						SL. MOIST	1.5'-5.0': POORLY GRADED GRAVEL WITH SILT, SAND, COBBLE AND BOULDERS, mostly fine to coarse gravel and cobble, few fine to coarse sands, boulders up to 18" in diameter, non plastic, brown.
5	SC			B	8B		MOIST	5.0'-8.5': CLAYEY SAND WITH GRAVEL AND COBBLE, mostly very fine to medium sand, some fine to coarse gravel, little rounded to subrounded cobbles, low plasticity, dark brown.
								Test pit terminated at 8.5 feet.

LOG OF TEST PIT NO. TP-9

Depth in Feet	Unified Soil Classification	Graphic Log	Sample	Sample Type	Sample No.	Consistency/ Density	Moisture	Visual Description
0	SC			B	9A		MOIST	0.0'-7.0': CLAYEY SAND WITH GRAVEL AND COBBLE, mostly very fine to medium sand, some fine to coarse gravel, few rounded cobbles, trace boulders up to 18" in diameter, low plastic, dark brown.
	GC						MOIST	7.0'-8.0': CLAYEY GRAVEL WITH SAND, COBBLE, AND BOULDERS, mostly fine to coarse rounded gravel and cobbles, some fine to coarse sand, low plasticity, dark brown.
								Test pit terminated at 8.0 feet.



NOTES:
1. BERM WILL ALSO BE REQUIRED IN THE BORROW STOCKPILE (LOCATION TO BE DETERMINED). BERM CROSS SECTION AND TOP ELEVATION WILL BE SIMILAR TO EXISTING BERM.
2. EARTHWORK QUANTITY CORRECTION - ESTIMATE 30% SHRINKAGE FOR COMPACTION & SCREENING OVERSIZE (12-INCH PLUS) MATERIAL

COMMENT	DATE	REVISION NO.

CME CONSTRUCTION MATERIALS ENGINEERS, INC.
6980 SIERRA CENTER PARKWAY, SUITE 90
RENO, NEVADA 89511

TRUCKEE MEADOWS WATER AUTHORITY
CHALK BLUFFS WATER TREATMENT PLANT
HIGHLAND DITCH CANAL IMPROVEMENTS
BORROW STOCKPILE PLAN
RENO, NEVADA
PROJECT NO.: 2056 DATE: 1/30/2018

PLATE
A-1b

APPENDIX B

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%

CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY

SAND AND GRAVEL		SILT AND CLAY	
NO. OF BLOWS	RELATIVE DENSITY	NO. OF BLOWS	CONSISTENCY
0-4	VERY LOOSE	0-1	VERY SOFT
5-10	LOOSE	2-4	SOFT
11-30	MEDIUM DENSE	5-8	MEDIUM STIFF
31-50	DENSE	9-15	STIFF
OVER 50	VERY DENSE	16-30	VERY STIFF
		OVER 31	HARD

V:\Active\2056\CAD\soil classification chart and rock description.dwg

CME CONSTRUCTION MATERIALS ENGINEERS INC.
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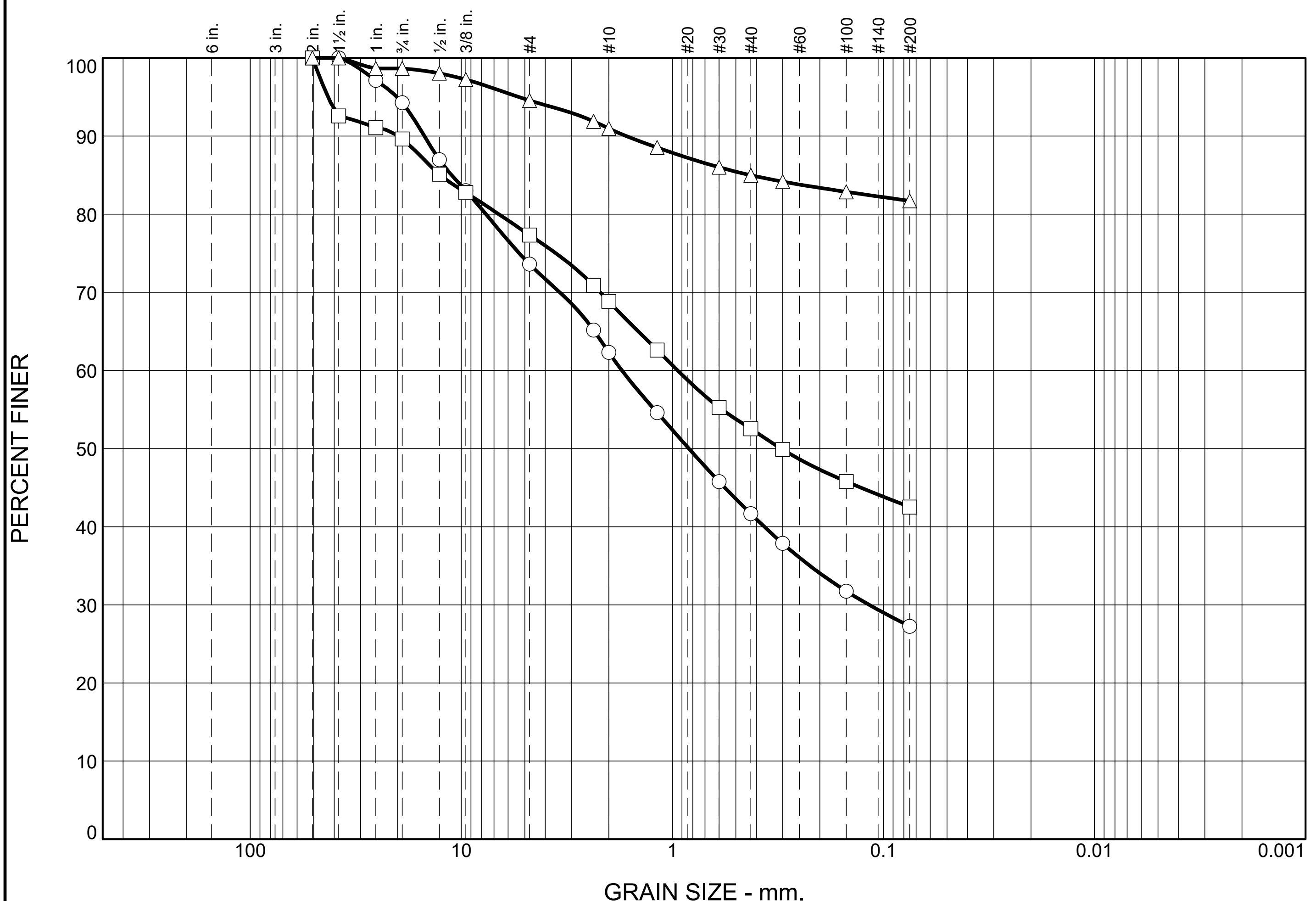
Reno, NV 89511

TMWA
CHALK BLUFFS WATER TREATMENT PLAN
SOIL CLASSIFICATION CHART
RENO, NEVADA

PROJECT NO.:2056 DATE: 01/25/2018

PLATE
A-2

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	5.7	20.7	11.3	20.6	14.5	27.2	
□	0.0	10.4	12.3	8.5	16.3	10.0	42.5	
△	0.0	1.4	4.0	3.6	6.0	3.3	81.7	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	46	27	11.1110	1.7349	0.8350	0.1168				
□	64	47	12.5566	0.9430	0.3052					
△	96	59	0.4270							

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ clayey sand with gravel		SC	
□ silty sand with gravel		SM	
△ elastic silt with sand		MH	

Project No. 2056 **Client:** TMWA

Project: HIGHLAND DITCH CANAL SLOPE STABILITY

○ **Loc.:** TP-1 & TP-2 (SAMPLES 1A & 2A COMBINED) **Sample No.:** 31551

□ **Loc.:** TP-3 & TP-4 (SAMPLES 3A & 4A COMBINED) **Sample No.:** 31551

△ **Location:** SIDEWALL SAMPLE **Sample Number:** 31551

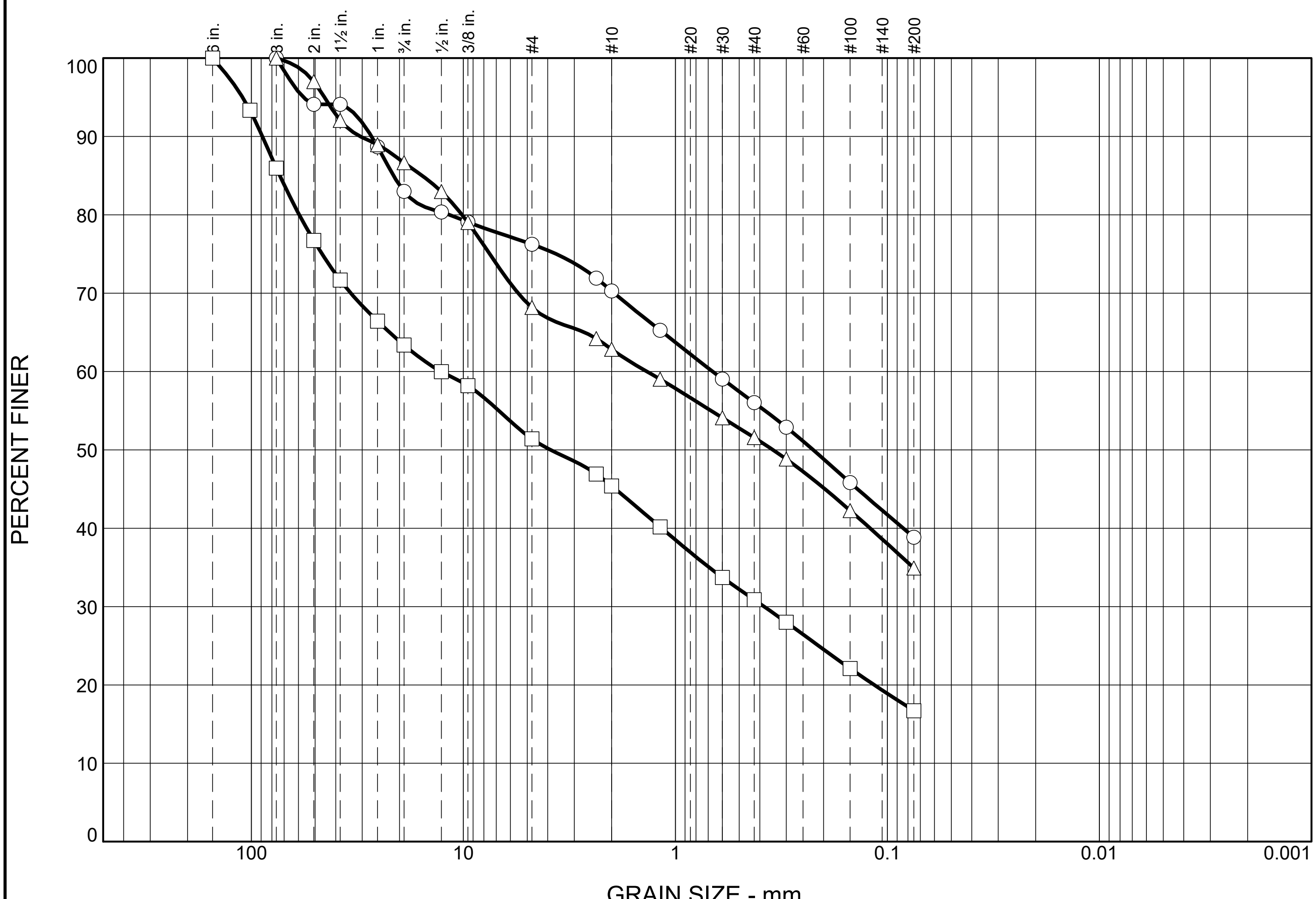
CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Remarks:

PLATE B-1a

Tested By: ○ J. WALTZ □ S. SCHWEITZER △ S. SCHWEITZER **Checked By:** S. HEIN

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	17.0	6.8	5.9	14.3	17.1	38.9	
□	14.0	22.6	12.0	6.0	14.5	14.2	16.7	
△	0.0	13.3	18.5	5.4	11.2	16.7	34.9	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	57	40	21.3549	0.6673	0.2239					
□	39	18	73.4186	12.7559	3.8810	0.3814				
△	47	19	15.6493	1.3634	0.3463					

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

Project No. 2056 **Client:** TMWA

Project: HIGHLAND DITCH CANAL SLOPE STABILITY

○ **Location:** TP-6, SAMPLE 6A **Depth:** 0.0'-6.0' **Sample Number:** 31813

□ **Location:** TP-7, SAMPLE 7B **Depth:** 1.0'-8.0' **Sample Number:** 31813

△ **Location:** TP-9, SAMPLE 9A **Depth:** 0.0'-7.0' **Sample Number:** 31813

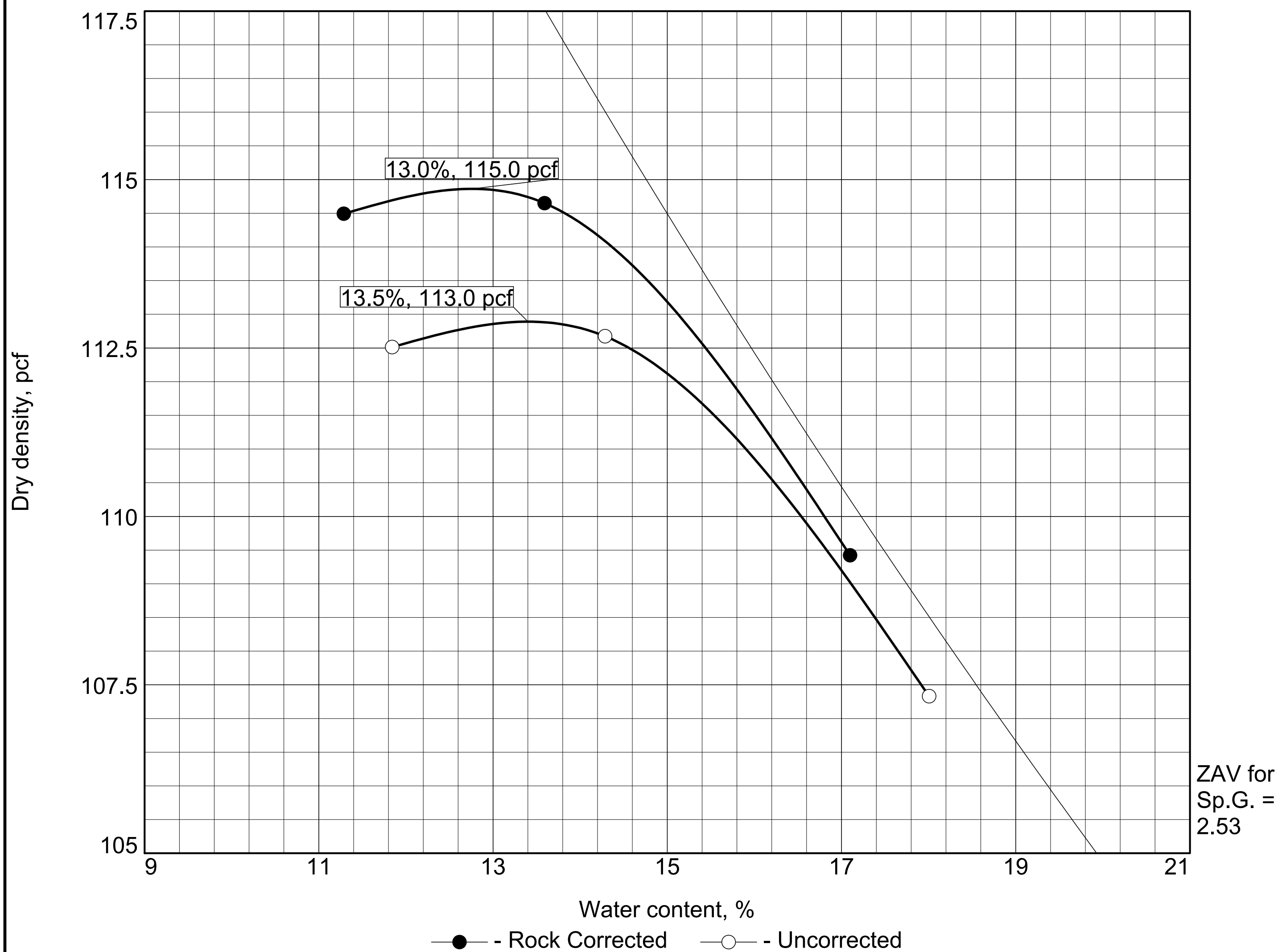
CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Remarks:

PLATE B-1b

Tested By: ○ J. WALTZ □ M. PONTONI △ M. PONTONI **Checked By:** S. HEIN

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method C Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	USCS	AASHTO						
	SC	A-2-7(1)			46	19	5.7	27.2

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 115.0 pcf	113.0 pcf	clayey sand with gravel
Optimum moisture = 13.0 %	13.5 %	

Project No. 2056 **Client:** TMWA
Project: HIGHLAND DITCH CANAL SLOPE STABILITY
Date:
○ **Loc.:** TP-1 & TP-2 (SAMPLES 1A & 2A COMBINED) **Sample No.:** 31551

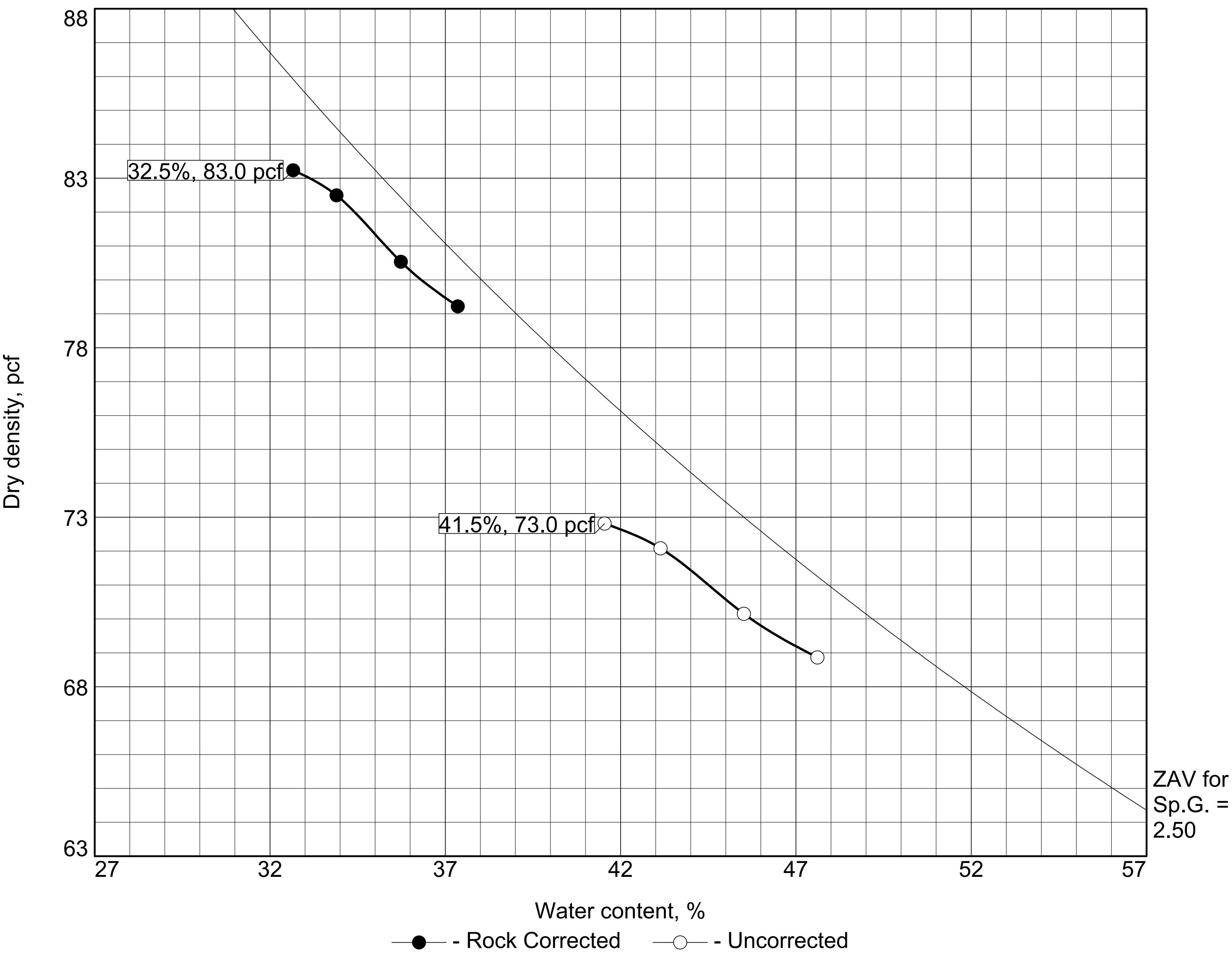
CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Remarks:

PLATE B-2a

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

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	SM	A-7-5(4)			64	17	22.7	42.5

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 83.0 pcf	73.0 pcf	silty sand with gravel
Optimum moisture = 32.5 %	41.5 %	

Project No. 2056 **Client:** TMWA
Project: HIGHLAND DITCH CANAL SLOPE STABILITY
Date:
○ **Loc.:** TP-3 & TP-4 (SAMPLES 3A & 4A COMBINED) **Sample No.:** 31551

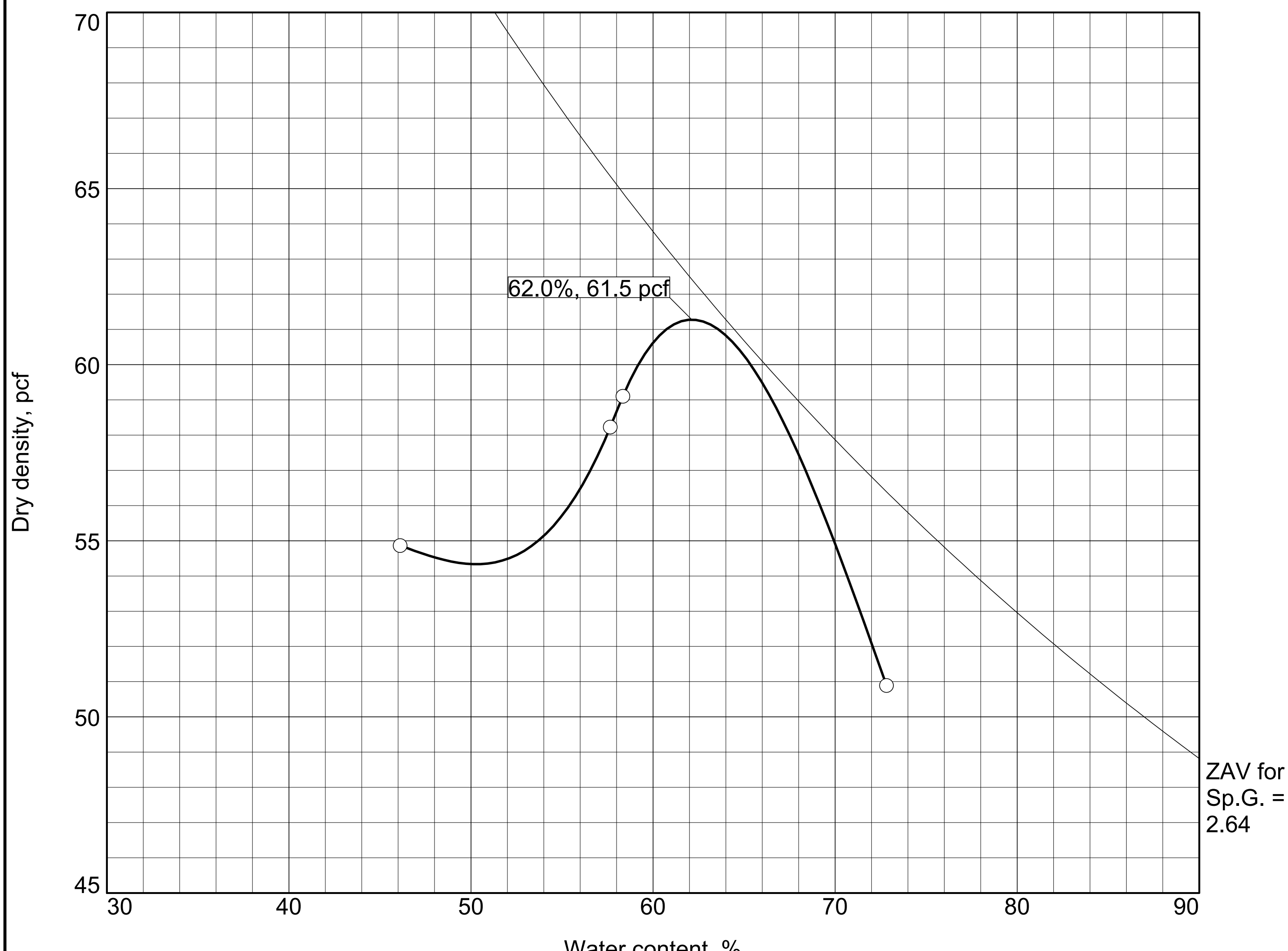
CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Remarks:

PLATE B-2b

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

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	MH	A-7-5(41)			96	37	5.4	81.7

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 61.5 pcf Optimum moisture = 62.0 %	elastic silt with sand

Project No. 2056 **Client:** TMWA

Project: HIGHLAND DITCH CANAL SLOPE STABILITY

Date:

○ **Location:** SIDEWALL SAMPLE **Sample Number:** 31551

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

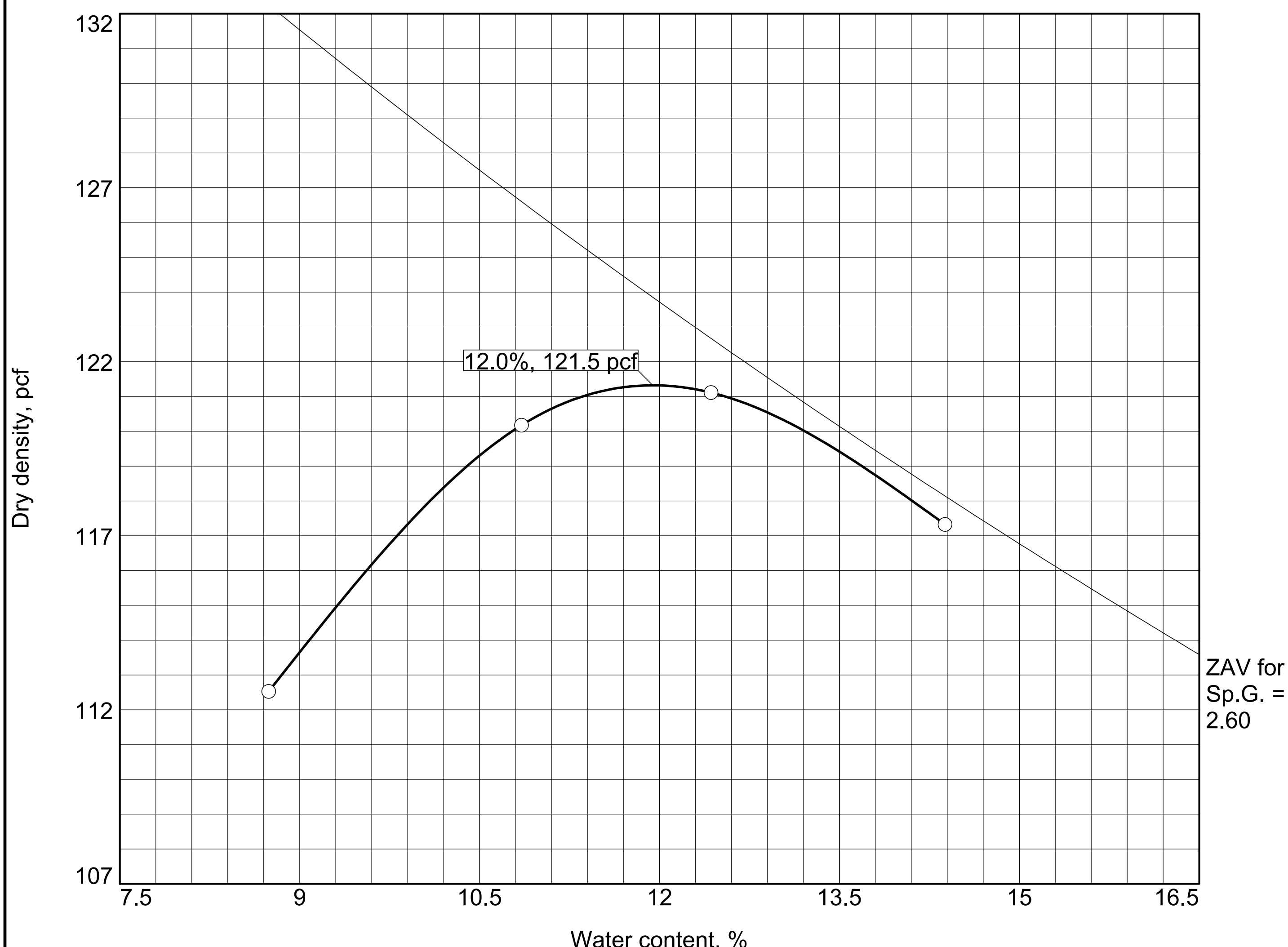
Remarks:

RECEIVED 8/14/2017

PLATE B-2c

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

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
1.0'-8.0'	SC	A-2-6(0)			39	21	48.6	16.7

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 121.5 pcf Optimum moisture = 12.0 %	clayey sand with gravel

Project No. 2056 **Client:** TMWA
Project: HIGHLAND DITCH CANAL SLOPE STABILITY
Date:
○ **Location:** TP-7, SAMPLE 7B **Sample Number:** 31813

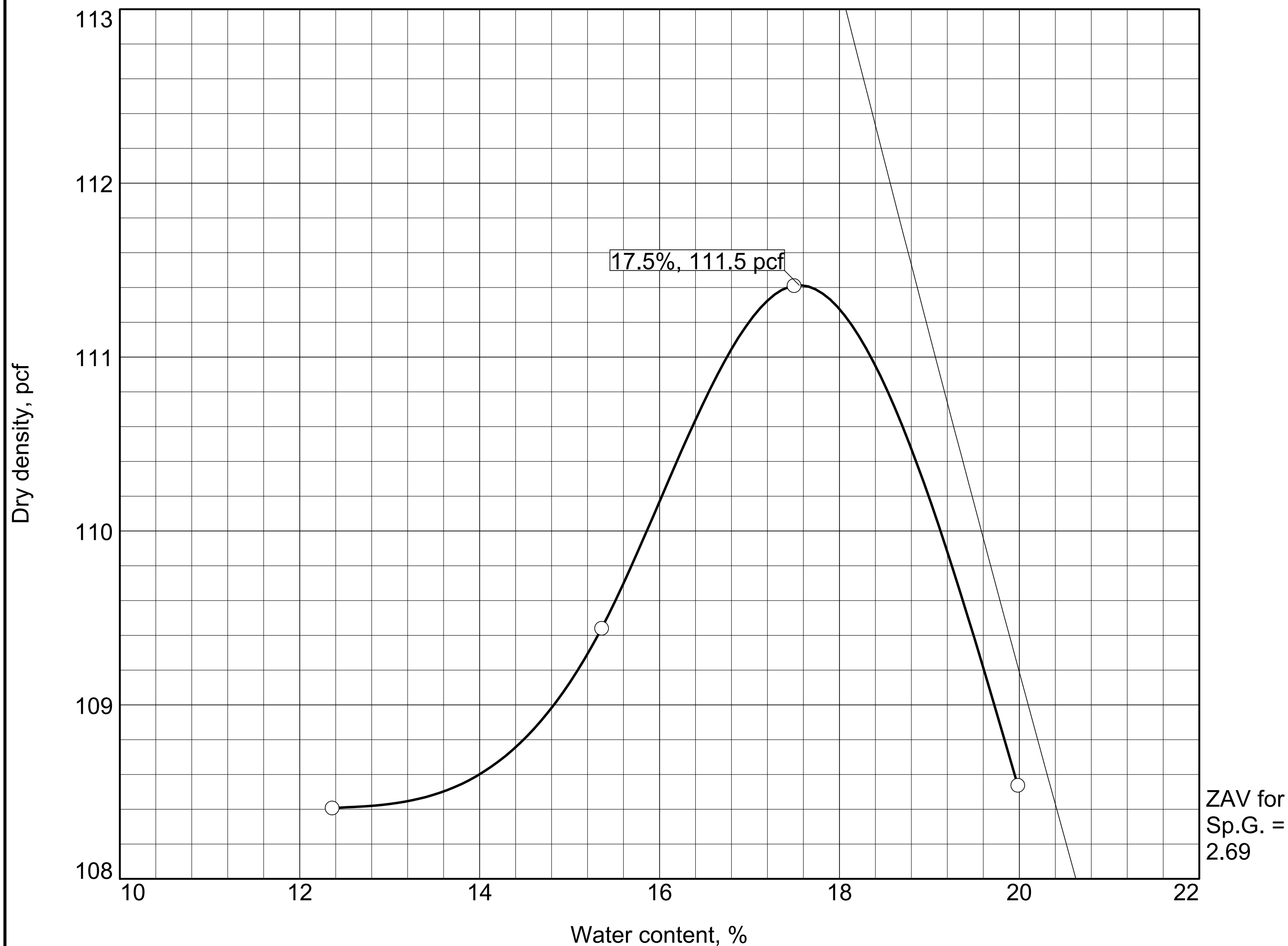
CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Remarks:
RECEIVED 10/17/2017

PLATE B-2d

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

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
0.0'-7.0'	SC	A-2-7(4)			47	28	31.8	34.9

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 111.5 pcf Optimum moisture = 17.5 %	clayey sand with gravel

Project No. 2056 **Client:** TMWA

Project: HIGHLAND DITCH CANAL SLOPE STABILITY

Date:

○ **Location:** TP-9, SAMPLE 9A **Sample Number:** 31813

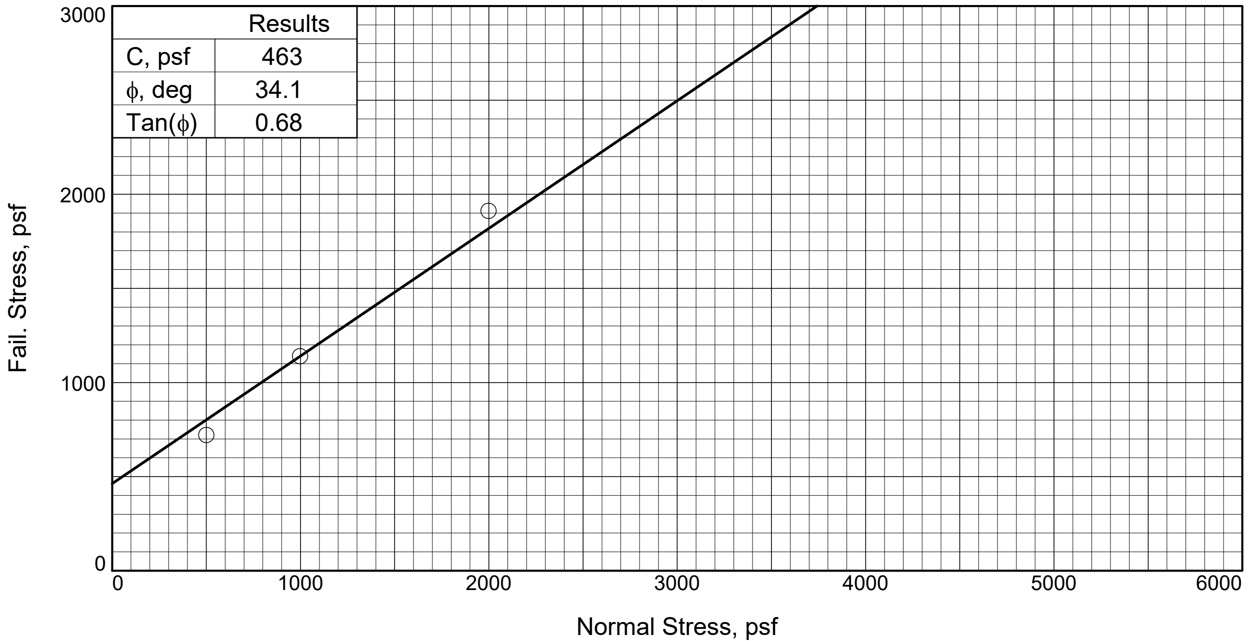
CONSTRUCTION MATERIALS ENGINEERS, INC.

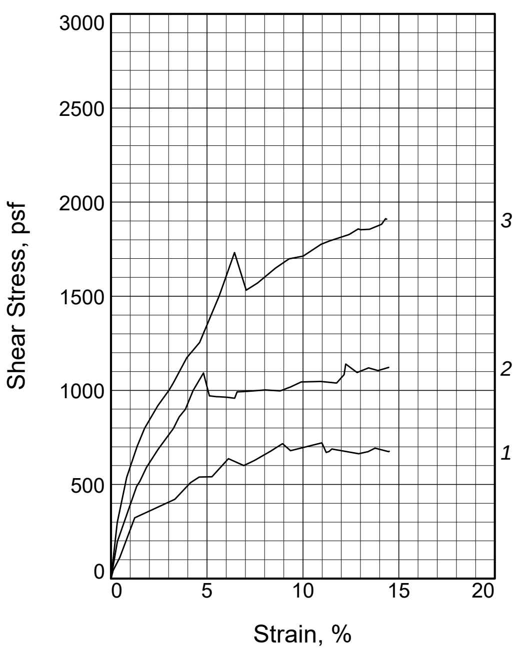
Remarks:

RECEIVED 10/17/2017

PLATE B-2e

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





	Sample No.	1	2	3
Initial	Water Content, %	19.3	19.3	19.3
	Dry Density, pcf	92.3	90.4	90.4
	Saturation, %	66.3	63.3	63.2
	Void Ratio	0.7590	0.7949	0.7963
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	1.00	1.00
At Test	Water Content, %	27.9	29.6	29.7
	Dry Density, pcf	93.8	91.7	91.4
	Saturation, %	99.1	99.8	99.6
	Void Ratio	0.7310	0.7705	0.7755
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.96	0.99	0.99
	Normal Stress, psf	500	997	1998
	Fail. Stress, psf	721	1140	1911
	Strain, %	11.0	12.2	14.3
	Ult. Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.001	0.001	0.001

Sample Type: REMOLDED

Description: clayey sand with gravel

LL= 46 **PL=** 27 **PI=** 19

Assumed Specific Gravity= 2.6

Remarks: Sample remolded to 90% of Maximum Dry Density & target Optimum Moisture Content of 17.6%

PLATE B-3a

Client: TMWA

Project: HIGHLAND DITCH CANAL SLOPE STABILITY

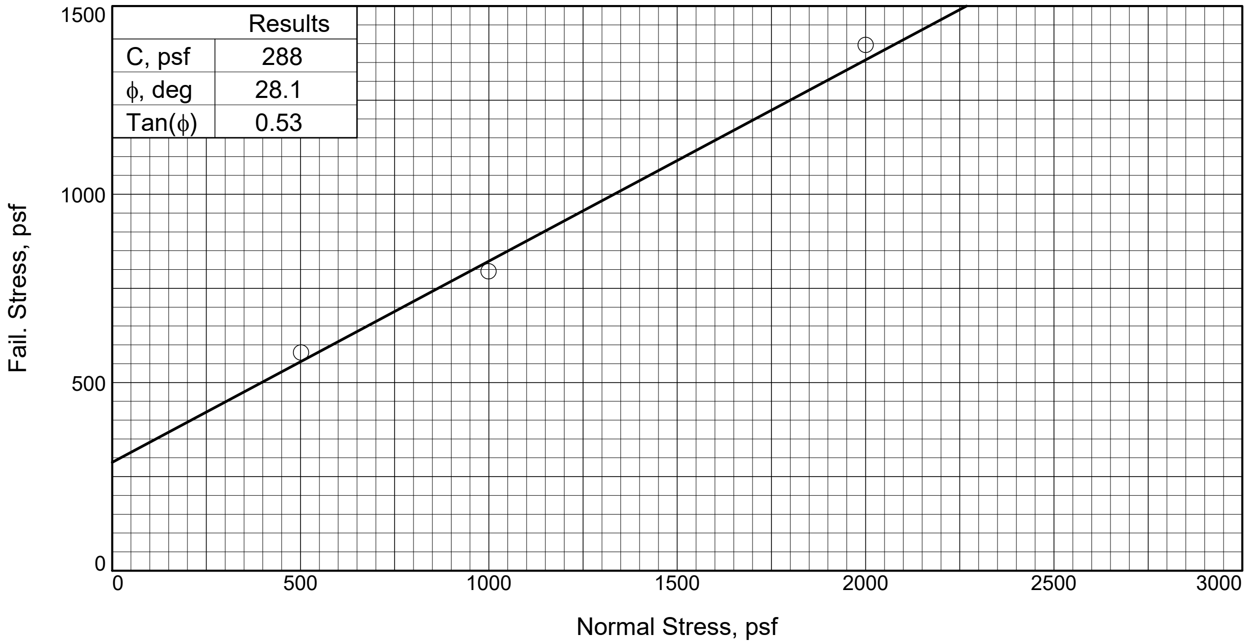
Location: TP-1 & TP-2 (SAMPLES 1A & 2A COMBINED)

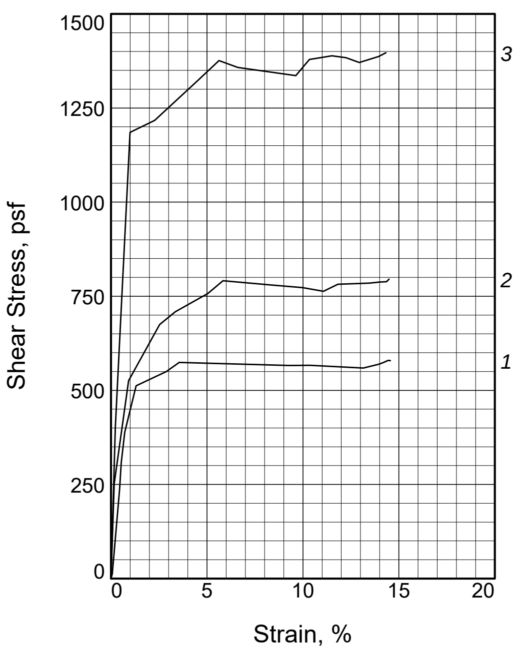
Sample Number: 31551

Proj. No.: 2056 **Date Sampled:** 8/14/2017

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Tested By: A. KASOZI **Checked By:** R. REYNOLDS





Sample No.		1	2	3
Initial	Water Content, %	12.3	12.3	12.3
	Dry Density, pcf	105.3	105.3	105.3
	Saturation, %	57.2	57.4	57.2
	Void Ratio	0.5716	0.5705	0.5716
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.04	1.04	1.04
At Test	Water Content, %	21.1	20.7	19.9
	Dry Density, pcf	105.9	106.6	108.1
	Saturation, %	99.5	99.3	99.4
	Void Ratio	0.5625	0.5526	0.5300
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.03	1.03	1.01
Normal Stress, psf		501	999	2000
Fail. Stress, psf		579	795	1397
Strain, %		14.4	14.5	14.3
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.003	0.003	0.003

Sample Type: REMOLDED

Description: Clayey Sand with Gravel, Cobbles, and Boulders

Assumed Specific Gravity= 2.65

Remarks: TESTED 10/20/17: Specimens remolded to Target Dry Density of 109.7 pcf (i.e. 90% of Proctor Maximum Value) and Optimum Moisture Content of 11.9%.

PLATE B-3b

Client: TMWA

Project: CHALK BLUFFS - HIGHLAND DITCH CANAL

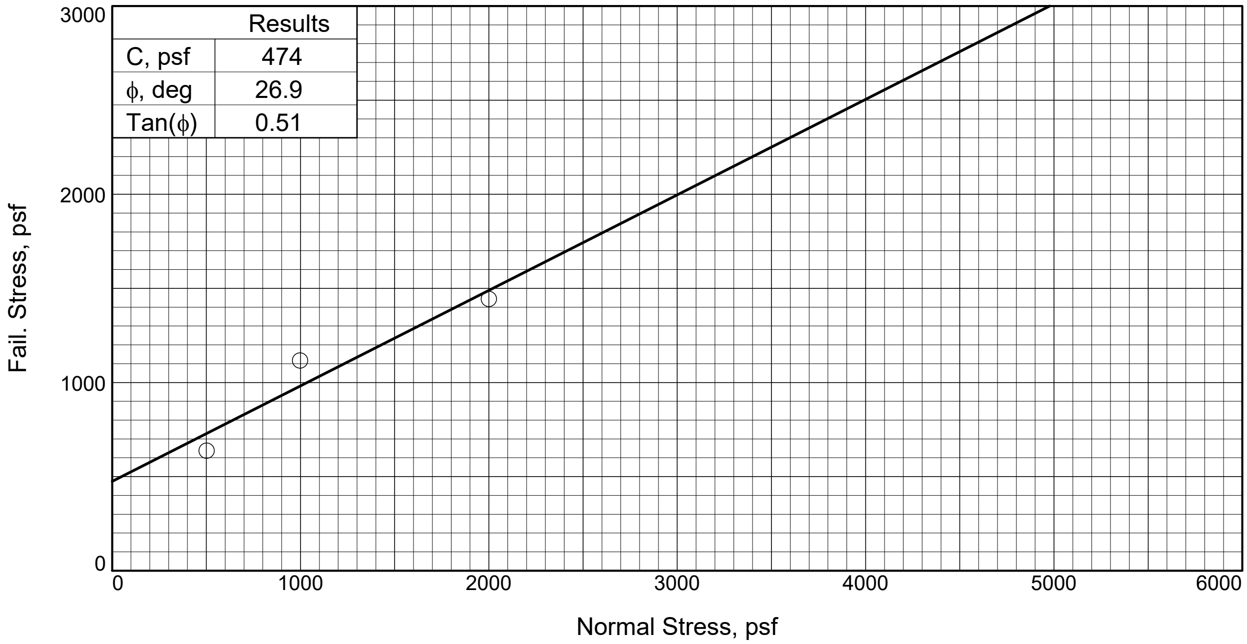
Location: TP-7, SAMPLE 7B

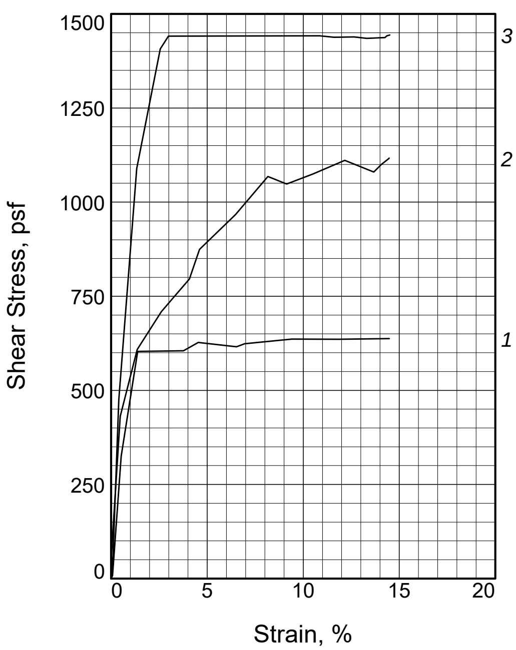
Sample Number: 31813 **Depth:** 1.0' - 8.0'

Proj. No.: 2056 **Date Sampled:** 10/17/17

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Tested By: A. KASOZI **Checked By:** R. REYNOLDS





Sample No.		1	2	3
Initial	Water Content, %	18.4	18.4	18.4
	Dry Density, pcf	98.5	98.6	98.4
	Saturation, %	70.2	70.3	70.1
	Void Ratio	0.7051	0.7039	0.7063
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.01	1.01	1.01
At Test	Water Content, %	26.3	25.6	24.5
	Dry Density, pcf	98.0	99.3	101.0
	Saturation, %	99.3	99.6	99.6
	Void Ratio	0.7131	0.6906	0.6626
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.01	1.00	0.98
Normal Stress, psf		501	997	2000
Fail. Stress, psf		637	1117	1444
Strain, %		14.4	14.5	14.5
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.003	0.003	0.003

Sample Type: REMOLDED

Description: Clayey Sand with Gravel and Cobbles

Assumed Specific Gravity= 2.69

Remarks: TESTED 10/25/17: Specimens remolded to Target Dry Density of 100.4 pcf (i.e. 90% of Proctor Maximum Value) and Optimum Moisture Content of 17.5%.

PLATE B-3c

Client: TMWA

Project: CHALK BLUFFS - HIGHLAND DITCH CANAL

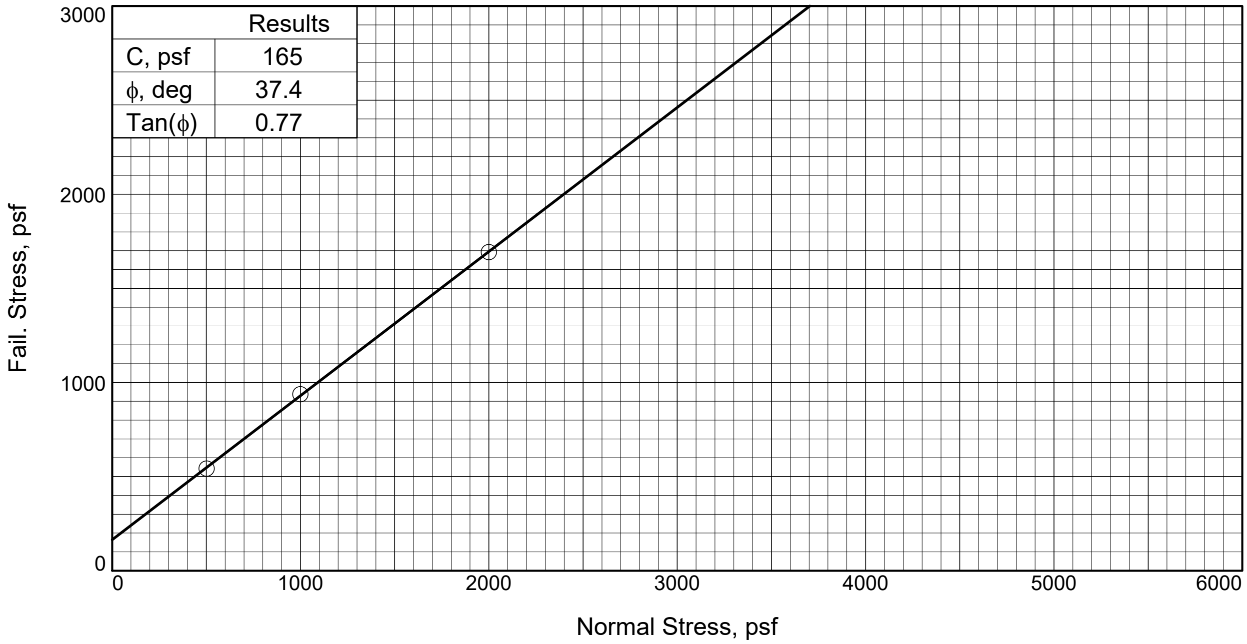
Location: TP-9, SAMPLE 9A

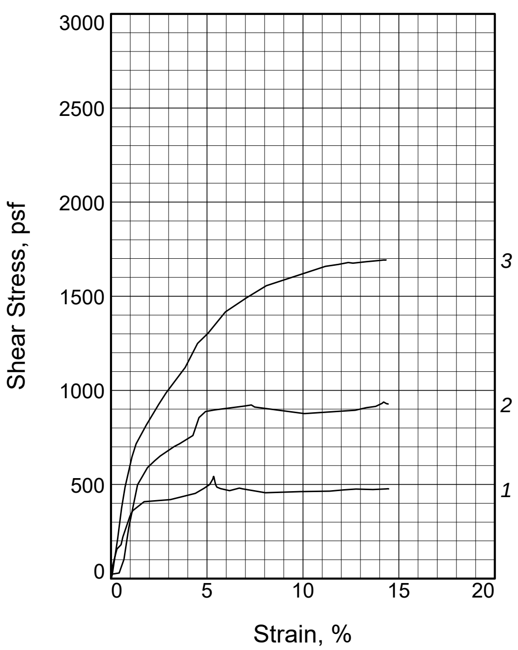
Sample Number: 31813 **Depth:** 0.0' - 7.0'

Proj. No.: 2056 **Date Sampled:** 10/17/17

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

Tested By: A. KASOZI **Checked By:** R. REYNOLDS





Sample No.		1	2	3
Initial	Water Content, %	46.2	46.2	46.2
	Dry Density, pcf	49.1	49.1	49.1
	Saturation, %	53.0	53.0	53.0
	Void Ratio	2.1778	2.1778	2.1778
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.02	1.02	1.02
At Test	Water Content, %	82.9	82.8	83.0
	Dry Density, pcf	50.6	50.8	50.6
	Saturation, %	99.5	99.8	99.6
	Void Ratio	2.0839	2.0728	2.0823
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.99	0.99
Normal Stress, psf		501	999	2000
Fail. Stress, psf		543	938	1693
Strain, %		5.4	14.2	14.2
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.000	0.000	0.000

Sample Type: REMOLDED
Description: NATIVE BEDROCK - DIATOMACEOUS SILTSTONE

Assumed Specific Gravity= 2.5
Remarks: TESTED 08/29/2017

Client: TMWA

Project: CHALK BLUFFS - HIGHLAND DITCH CANAL

Location: EXISTING SLOPE; NW OF TP-1
Sample Number: 31613
Proj. No.: 2056 **Date Sampled:**

CME CONSTRUCTION MATERIALS ENGINEERS, INC.

PLATE B-3d

Tested By: A. KASOZI **Checked By:** R. REYNOLDS

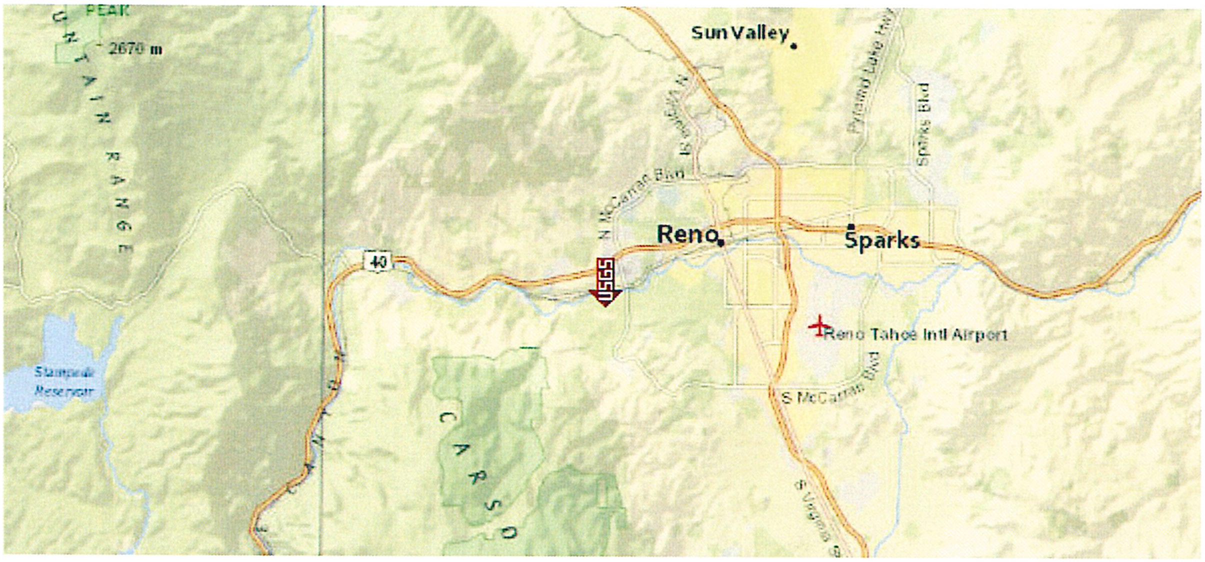
CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX C

USGS Design Maps Summary Report

User–Specified Input

Report Title	Chalk Bluffs-highland ditch Wed January 3, 2018 23:43:42 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	39.51457°N, 119.86949°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	IV (e.g. essential facilities)



USGS–Provided Output

S_S = 1.685 g	S_{MS} = 1.685 g	S_{DS} = 1.124 g
S_1 = 0.600 g	S_{M1} = 0.780 g	S_{D1} = 0.520 g

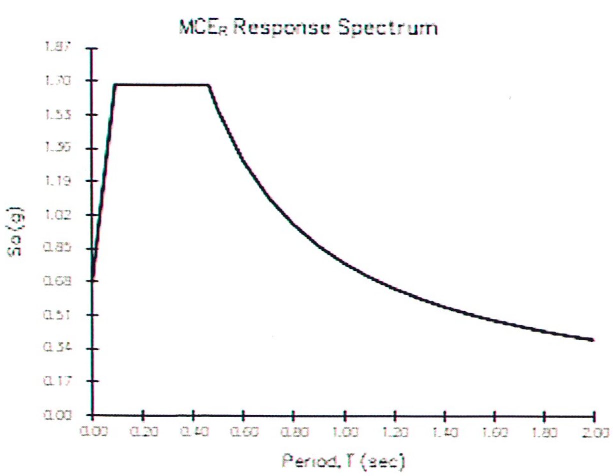
$F_V = 1.3$ $\beta = \frac{(1.3)(0.6)}{0.6}$

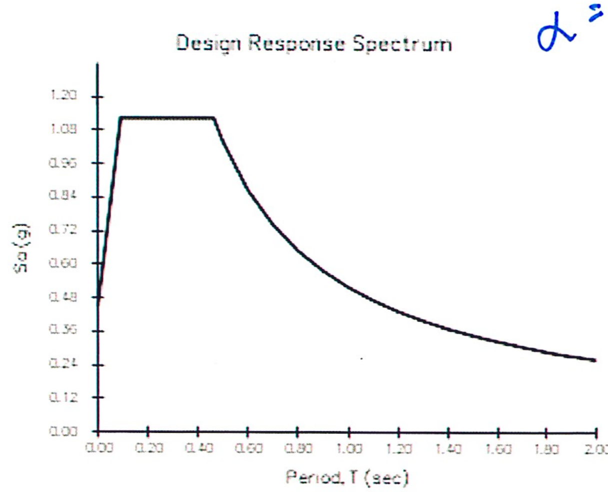
$S_1 = 0.6$

$k_{max} = .6$ $B = 1.3$

$\alpha = .84$

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





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

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

USGS Design Maps Detailed Report

ASCE 7-10 Standard (39.51457°N, 119.86949°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category IV (e.g. essential facilities)

Section 11.4.1 — Mapped Acceleration Parameters

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

From Figure 22-1 [1] $S_S = 1.685$ g

From Figure 22-2 [2] $S_1 = 0.600$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

Table 11.4–1: Site Coefficient F_a

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

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

For Site Class = C and S_S = 1.685 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

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

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

For Site Class = C and S_1 = 0.600 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.685 = 1.685$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 0.600 = 0.780$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

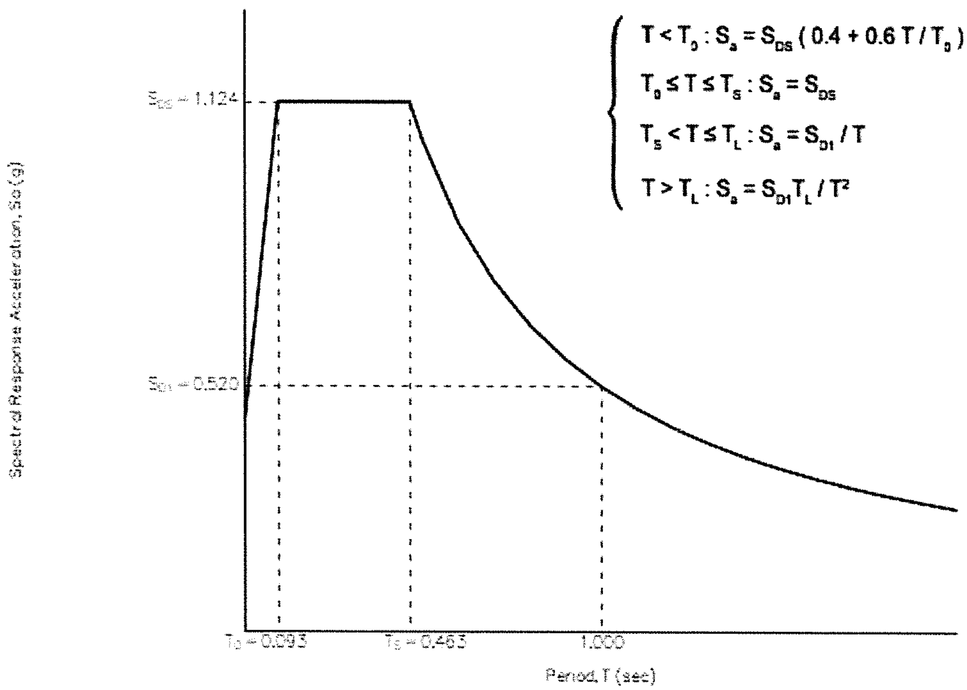
Equation (11.4–3): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.685 = 1.124$ g

Equation (11.4–4): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.780 = 0.520$ g

Section 11.4.5 — Design Response Spectrum

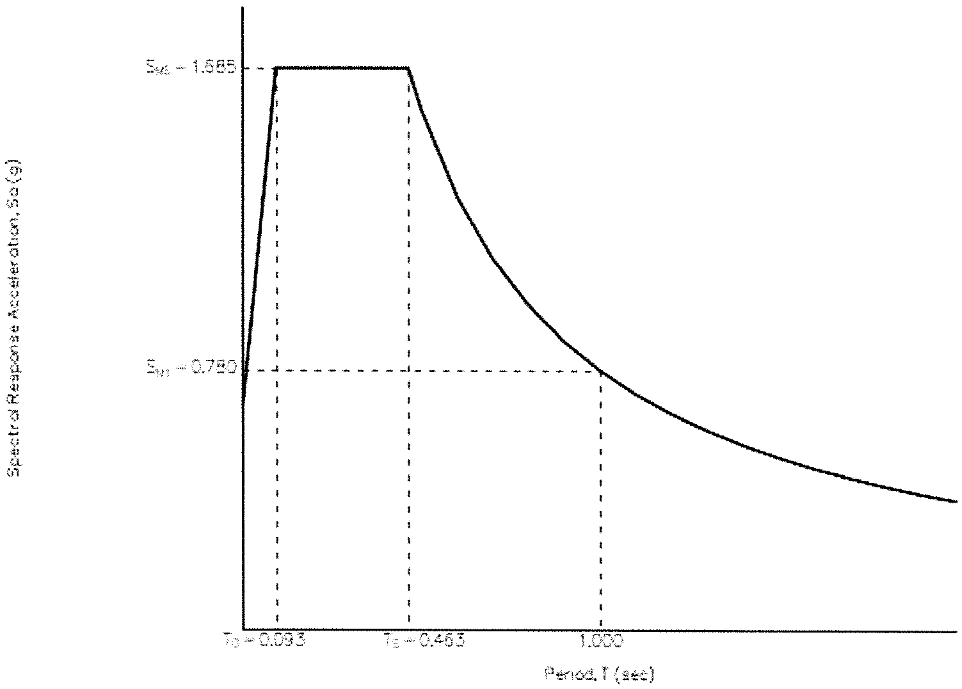
From Figure 22-12[3] $T_L = 6$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.604

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.604 = 0.604$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	$\boxed{1.0}$
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

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

For Site Class = C and PGA = 0.604 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} = 0.930

From Figure 22-18 [6] C_{R1} = 0.906

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 = IV and S_{DS} = 1.124 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 = IV and S_{D1} = 0.520 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*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX D

Tensar International Corporation
5883 Glenridge Drive, Suite 200
Atlanta, Georgia 30328-5363
Phone: 800-TENSAR-1
www.tensar-international.com

Product Specification - Structural Geogrid UX1600HS

Tensar International Corporation reserves the right to change its product specifications at any time. It is the responsibility of the specifier and purchaser to ensure that product specifications used for design and procurement purposes are current and consistent with the products used in each instance.

Product Type:	**Integrally Formed Structural Geogrid**
Polymer:	**High Density Polyethylene**
Load Transfer Mechanism:	**Positive Mechanical Interlock**
Recommended Applications:	**Sierra System (Reinforced Slopes), Prism System (Embankments), Temporary Walls**

Product Properties

Index Properties	Units	MD Values[1]
▪ Tensile Strength @ 5% Strain[2]	kN/m (lb/ft)	58 (3,980)
▪ Ultimate Tensile Strength[2]	kN/m (lb/ft)	144 (9,870)
▪ Junction Strength[3]	kN/m (lb/ft)	135 (9,250)
▪ Flexural Stiffness[4]	mg-cm	6,000,000
Durability		
▪ Resistance to Long Term Degradation[5]	%	100
▪ Resistance to UV Degradation[6]	%	95
Load Capacity		
▪ Maximum Allowable (Design) Strength for 120-year Design Life[7]	kN/m (lb/ft)	52.7 (3,620)
Recommended Allowable Strength Reduction Factors[7]		
▪ Minimum Reduction Factor for Installation Damage (RF_{ID})[8]		1.05
▪ Reduction Factor for Creep for 120-year Design Life (RF_{CR})[9]		2.60
▪ Minimum Reduction Factor for Durability (RF_D)		1.00

Dimensions and Delivery

The structural geogrid shall be delivered to the jobsite in roll form with each roll individually identified and nominally measuring 1.33 meters (4.36 feet) in width and 61.0 meters (200.0 feet) in length. A typical truckload quantity is 216 rolls.

Notes:

1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02. Brief descriptions of test procedures are given in the following notes.
2. True resistance to elongation when initially subjected to a load measured via ASTM D6637-01 without deforming test materials under load before measuring such resistance or employing "secant" or "offset" tangent methods of measurement so as to overstate tensile properties.
3. Load transfer capability determined in accordance with GRI-GG2-05.
4. Resistance to bending force determined in accordance with ASTM D5732-01, using specimen dimensions of 864 millimeters in length by one aperture in width.
5. Resistance to loss of load capacity or structural integrity when subjected to chemically aggressive environments in accordance with EPA 9090 immersion testing.
6. Resistance to loss of load capacity or structural integrity when subjected to 500 hours of ultraviolet light and aggressive weathering in accordance with ASTM D4355-05.
7. Reduction factors are used to calculate the geogrid strength available for resisting force in long-term load bearing applications. Allowable Strength (T_{allow}) is determined by reducing the ultimate tensile strength (T_{ult}) by reduction factors for installation damage (RF_{ID}), creep (RF_{CR}) and chemical/biological durability ($RF_D = RF_{CD} \cdot RF_{BD}$) per GRI-GG4-05 [$T_{allow} = T_{ult}/(RF_{ID} \cdot RF_{CR} \cdot RF_D)$]. Recommended minimum reduction factors are based on product-specific testing. Project specifications, standard public agency specifications and/or design code requirements may require higher reduction factors. Design of the structure in which the geogrid is used, including the selection of appropriate reduction factors and design life, is the responsibility of the outside licensed professional engineer providing the sealed drawings for the project.
8. Minimum value is based on Installation Damage Testing in Sand, Silt, and Clay soils. Coarser soils require increased RF_{ID} values.
9. Reduction Factor for Creep determined for 120-year design life and in-soil temperature of 20°C using standard extrapolation techniques to creep rupture data obtained following the test procedure in ASTM D5262-04. Actual design life of the completed structure may differ

This product specification supersedes all prior specifications for the product described above and is not applicable to any products shipped prior to June 1, 2007

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX E

Chalk Bluffs water treatment Plant

Report created by ReSSA(3.0): Copyright (c) 2001-2011, ADAMA Engineering, Inc.

PROJECT IDENTIFICATION

Title:	Chalk Bluffs water treatment Plant
Project Number:	2056 -
Client:	TMWA
Designer:	Randy Reynolds
Station Number:	0+90

Description:
Static conditon with no reinforcement and surcharge

Company's information:

Name:	CME
Street:	
	,
Telephone #:	
Fax #:	
E-Mail:	

Original file path and name: V:\Active\ ..... Analysis\slope 1.5H1.0V w reinforcement static.MSE
Original date and time of creating this file: Mon Jan 08 16:21:39 2018

PROGRAM MODE: Analysis of a General Slope using NO reinforcement material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ^{3}]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ^{2}]
....1.......Structural fill material...........................	120.0	28.0	250.0
....2.......diatomaceous siltstone..........................	75.0	37.0	165.0

REINFORCEMENT

Analysis of slope WITHOUT reinforcement.

WATER

Water is not present

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - GENERAL - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.

GEOMETRY

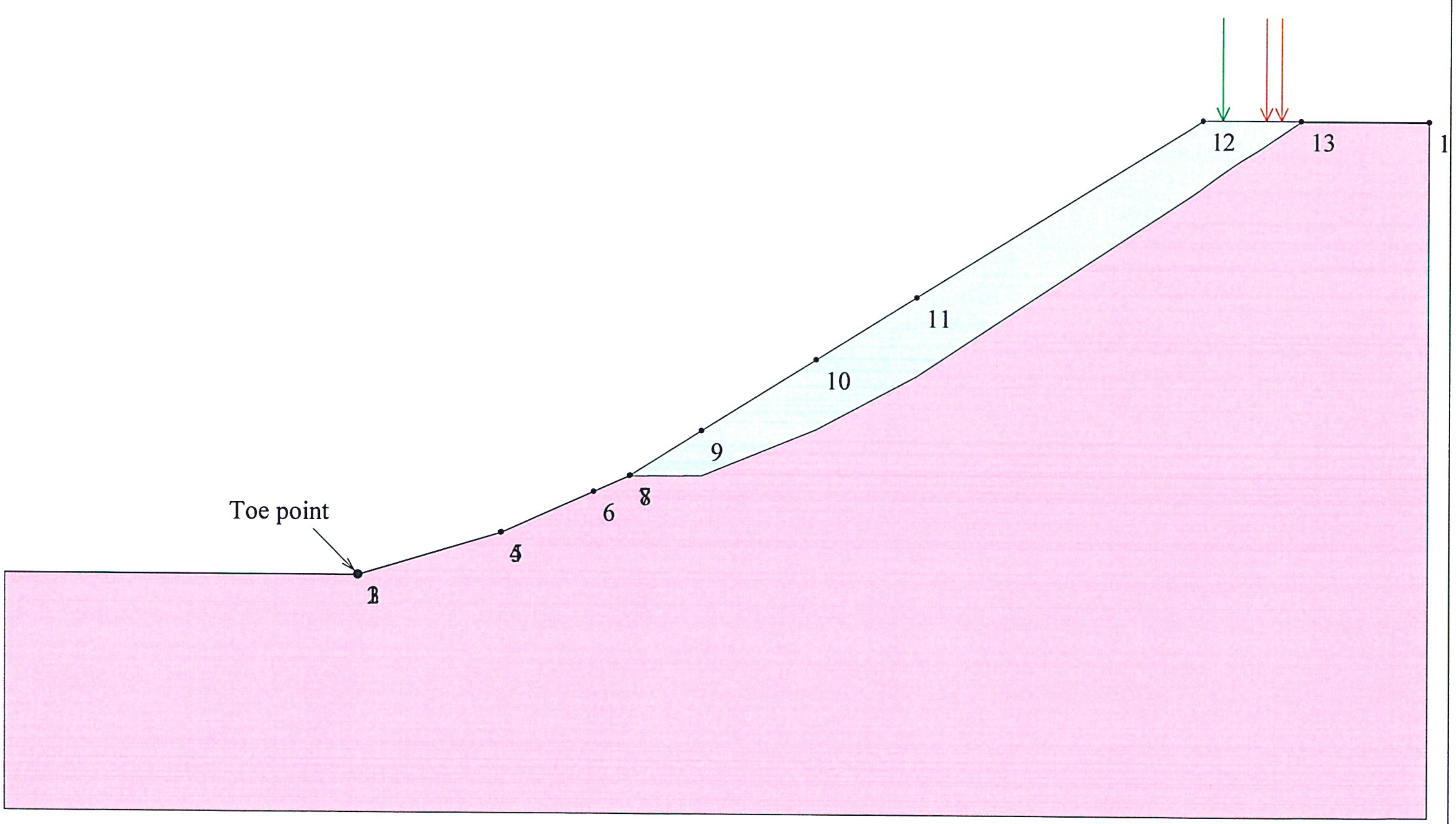
Soil profile contains 2 layers (see details in next page)

UNIFORM SURCHARGE

Load Q1 = 4000.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 449.00 and ends at X1e = 451.50 [ft].
Load Q2 = 4000.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X2s = 455.00 and ends at X2e = 457.50 [ft].
Surcharge load, Q3 ....None

STRIP LOAD

............................None...................................



SCALE:

0246[ft]

TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]

	#	Xi	Yi
Top of Layer 1	1	328.00	394.00
	2	348.00	400.00
	3	366.00	408.00
	4	446.00	458.00
	5	478.00	458.00
Top of Layer 2	6	327.90	393.90
	7	347.90	399.90
	8	365.90	407.90
	9	376.00	408.00
	10	392.00	414.50
	11	406.00	422.00
	12	460.00	457.90

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]

#	X	Y1	Y2
1	327.90	394.00	393.90
2	328.00	394.00	393.93
3	328.08	394.03	393.96
4	347.90	399.97	399.90
5	348.00	400.00	399.94
6	360.89	405.73	405.67
7	365.90	407.96	407.90
8	366.00	408.00	407.90
9	376.00	414.25	408.00
10	392.00	424.25	414.50
11	406.00	433.00	422.00
12	446.00	458.00	448.59
13	460.00	458.00	457.90
14	478.00	458.00	457.90

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	430.50	448.31	362.20	406.36	352.66	498.47	92.60	1.67	
2	433.07	449.92	362.25	406.36	350.19	505.33	99.70	1.65	
3	435.64	451.52	362.25	406.36	347.39	512.71	107.39	1.64	
4	438.21	453.13	361.46	406.25	346.87	516.40	111.11	1.62	
5	440.78	454.73	361.49	406.24	343.63	524.51	119.61	1.61	
6	443.34	456.34	366.60	408.72	349.39	522.11	114.69	1.61	
7	445.91	457.95	361.54	406.23	339.25	537.28	132.93	1.59	
8	448.48	458.00	351.41	401.76	320.16	567.58	168.74	1.58	
9	451.05	458.00	366.80	408.71	330.86	566.78	162.11	1.42	
10	453.62	458.00	351.48	401.72	298.16	619.30	224.02	1.43	
11	456.19	458.00	347.10	399.74	266.47	681.94	293.50	1.40 .	OK
12	458.75	458.00	402.23	430.65	374.94	559.14	131.36	1.43	
13	461.32	458.00	351.65	401.75	277.48	681.40	289.32	1.55	
14	463.89	458.00	341.00	398.22	329.86	577.33	179.45	1.59	
15	466.46	458.00	347.10	399.79	335.05	575.98	176.60	1.62	
16	469.03	458.00	352.06	401.92	343.57	569.66	167.95	1.68	
17	471.60	458.00	356.87	404.14	351.76	564.13	160.08	1.74	
18	474.16	458.00	341.05	398.06	282.67	705.47	312.91	1.81	
19	476.73	458.00	331.07	395.28	324.90	610.11	214.92	1.84	
20	479.30	458.00	347.10	399.78	316.97	647.39	249.44	1.86	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	331.60	395.21	456.19	458.00	302.53	607.89	214.66	1.50	
2	336.08	396.54	456.19	458.00	284.58	645.28	254.02	1.45	
3	342.34	398.30	456.19	458.00	266.59	681.17	292.84	1.41	
. 4	347.10	399.74	456.19	458.00	266.47	681.94	293.50	1.40 .	OK
5	351.48	401.72	453.62	458.00	298.16	619.30	224.02	1.43	
6	356.65	403.97	453.62	458.00	299.23	621.04	224.54	1.43	
7	361.58	406.17	453.62	458.00	308.11	608.76	209.53	1.43	
8	366.88	408.69	456.19	458.00	295.67	643.19	245.07	1.41	
9	371.40	411.69	456.19	458.00	312.66	619.98	216.42	1.42	
10	376.88	414.95	456.19	458.00	327.73	600.07	191.53	1.43	
11	381.52	417.99	456.19	458.00	341.24	582.85	169.70	1.44	
12	387.23	421.29	456.19	458.00	353.45	567.87	150.42	1.46	
13	391.76	424.31	456.19	458.00	364.57	554.77	133.26	1.42	
14	396.69	427.50	451.05	458.00	392.26	499.09	71.73	1.45	
15	402.23	430.65	458.75	458.00	374.94	559.14	131.36	1.43	
16	406.82	433.73	456.19	458.00	399.03	511.93	78.58	1.43	
17	411.99	436.87	456.19	458.00	406.21	505.77	69.13	1.42	
18	416.90	440.00	456.19	458.00	415.29	495.40	55.42	1.46	
19	422.17	443.14	451.05	458.00	425.72	471.72	28.81	1.46	
20	427.09	446.27	451.05	458.00	430.81	469.01	23.05	1.44	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.40

Critical Circle: Xc = 266.47[ft], Yc = 681.94[ft], R = 293.50[ft]. (Number of slices used = 59)

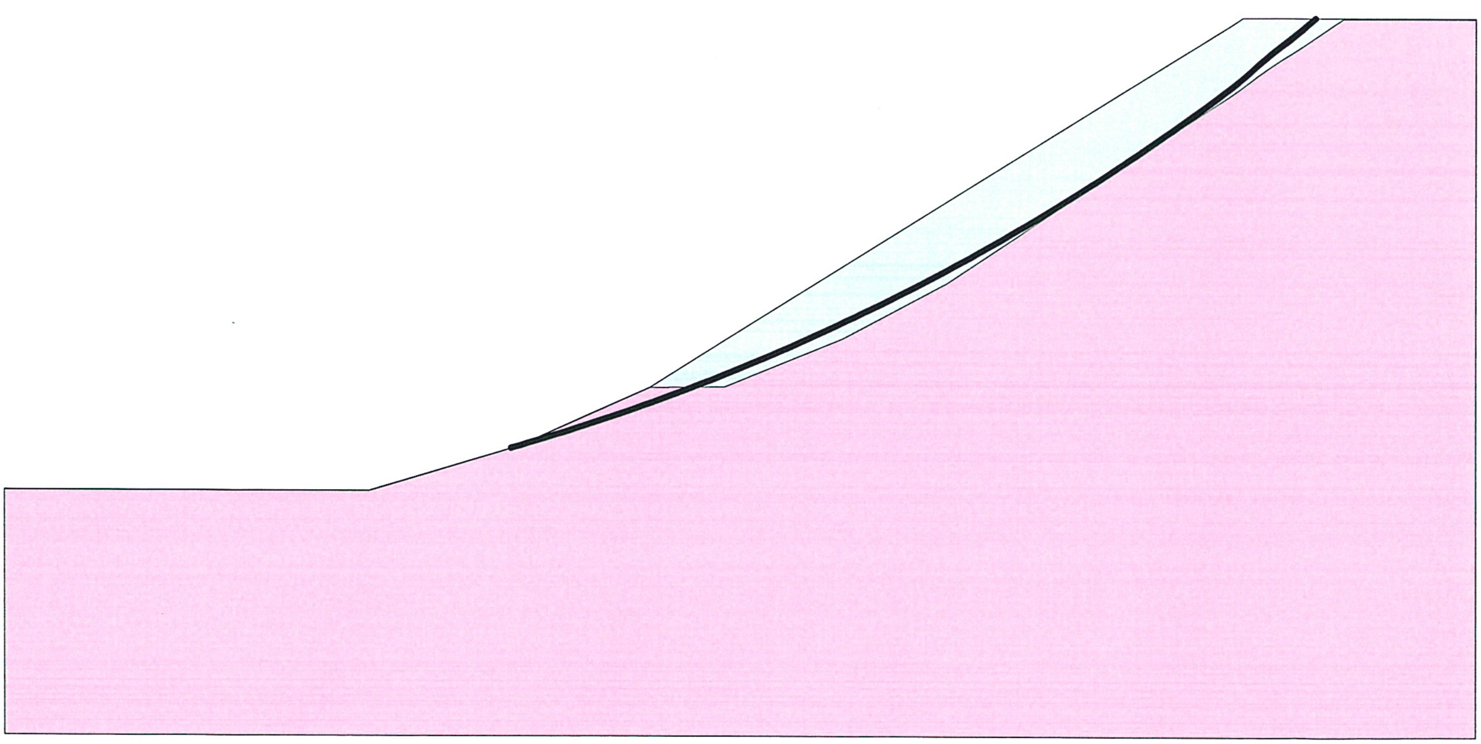
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

N O T C O N D U C T E D

Three-Part Wedge Stability Analysis

N O T C O N D U C T E D

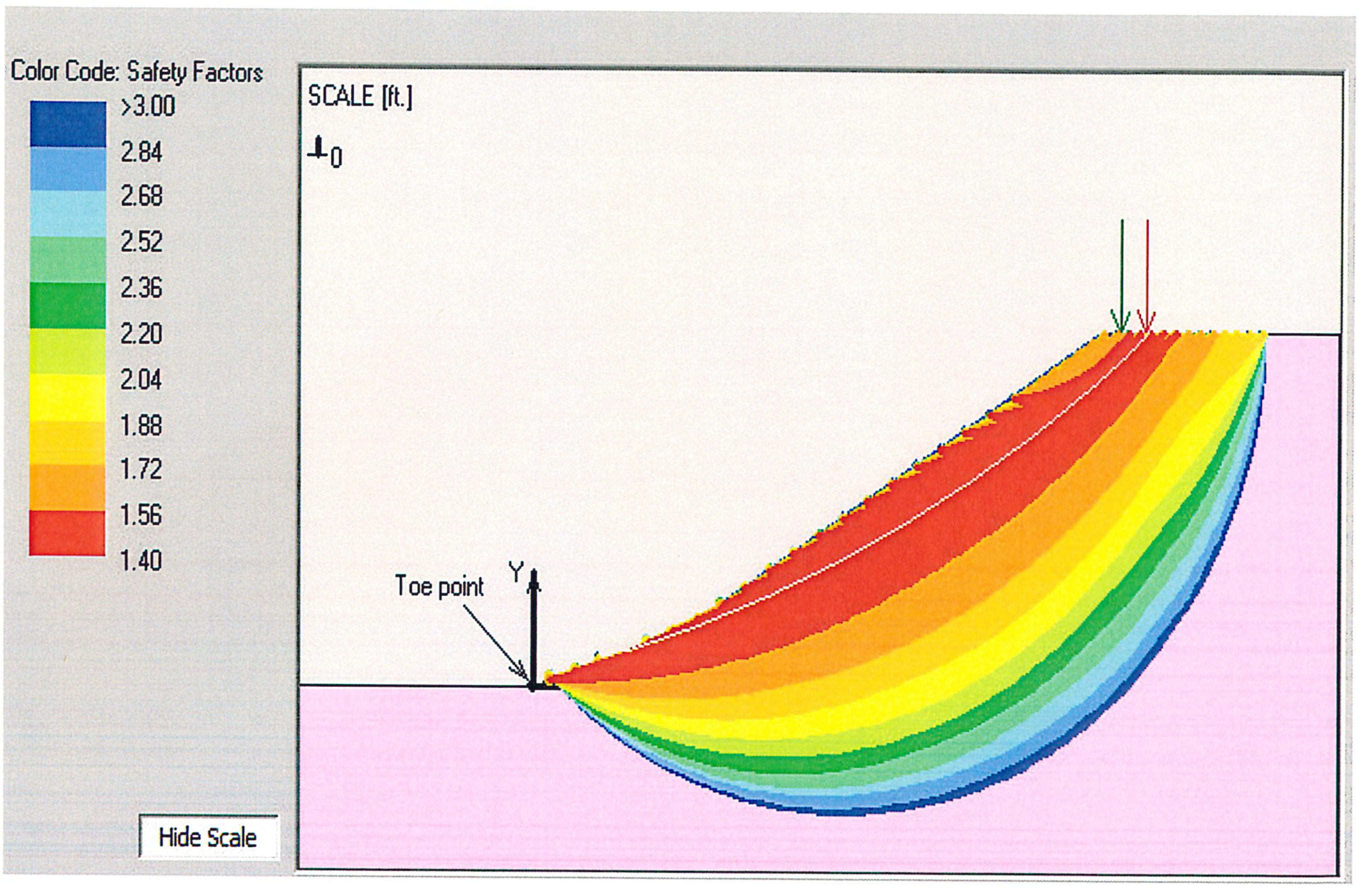
REINFORCEMENT LAYOUT: DRAWING



SCALE:

0246[ft]

SAFETY MAP: BISHOP ROTATIONAL ANALYSIS MODE



Chalk Bluffs water treatment Plant

PROJECT IDENTIFICATION

Title: Chalk Bluffs water treatment Plant
Project Number: 2056 -
Client: TMWA
Designer: Randy Reynolds
Station Number: 0+90

Description:
Siesmic conditon with no reinforcement

Company's information:

Name: CME
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name: V:\Active\ ..... Analysis\slope 1.5H1.0V w reinforcement static.MSE
Original date and time of creating this file: Mon Jan 08 16:21:39 2018

PROGRAM MODE: Analysis of a General Slope using NO reinforcement material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ^{3}]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ^{2}]
....1.......Structural fill material...........................	120.0	28.0	250.0
....2.......diatomaceous siltstone..........................	75.0	37.0	165.0

REINFORCEMENT

Analysis of slope WITHOUT reinforcement.

WATER

Water is not present

SEISMICITY

Horizontal peak ground acceleration coefficient, Ao = 0.500
Design horizontal seismic coefficient, kh = Am = 0.50 x Ao = 0.250 & design vertical seismic coefficient, kv (down) = 0.000 x kh = 0.000

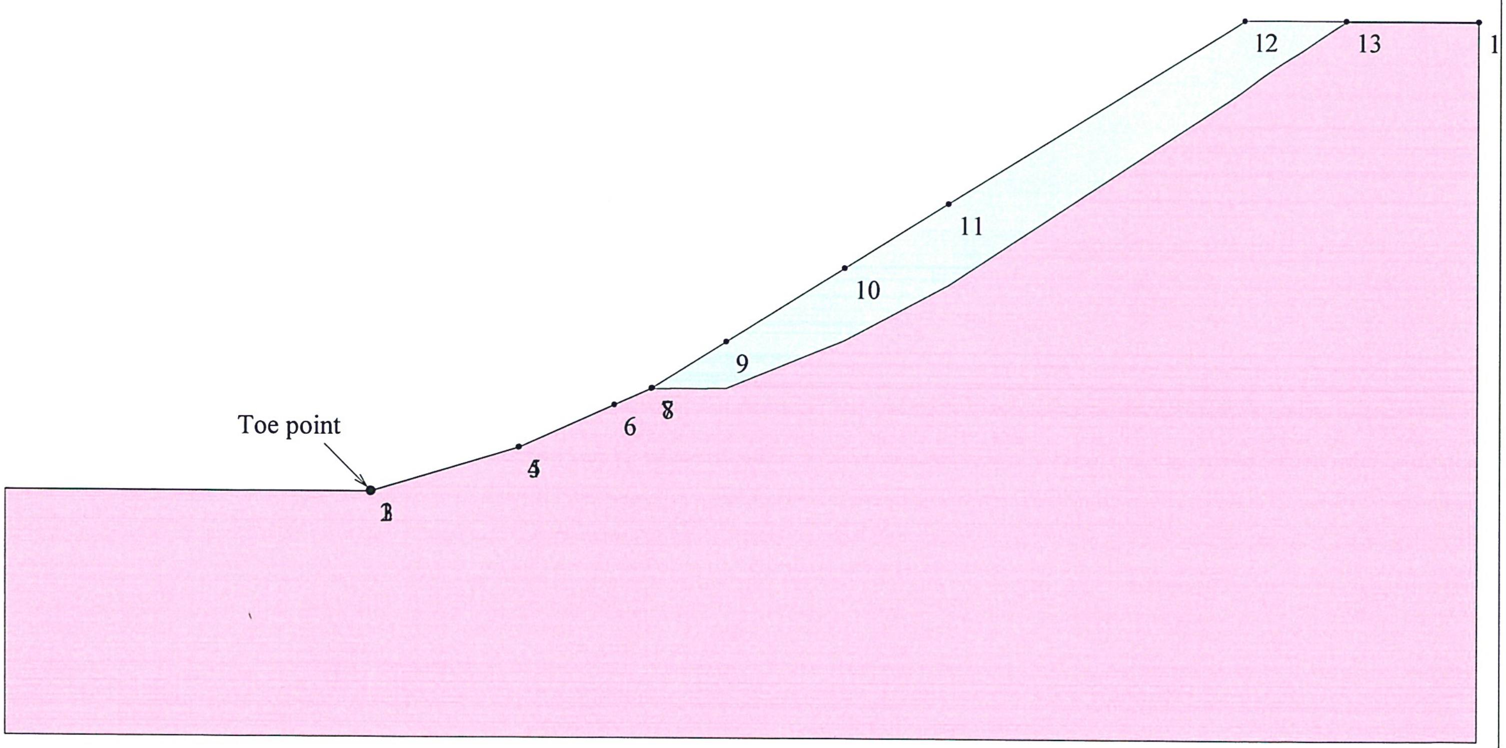
DRAWING OF SPECIFIED GEOMETRY - GENERAL - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.

GEOMETRY
Soil profile contains 2 layers (see details in next page)

UNIFORM SURCHARGE
Surcharge load, Q1 ....None
Surcharge load, Q2 ....None
Surcharge load, Q3 ....None

STRIP LOAD
............................None..................................



SCALE:

0246 [ft]

TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]

	#	Xi	Yi
Top of Layer 1	1	328.00	394.00
	2	348.00	400.00
	3	366.00	408.00
	4	446.00	458.00
	5	478.00	458.00
Top of Layer 2	6	327.90	393.90
	7	347.90	399.90
	8	365.90	407.90
	9	376.00	408.00
	10	392.00	414.50
	11	406.00	422.00
	12	460.00	457.90

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]

#	X	Y1	Y2
1	327.90	394.00	393.90
2	328.00	394.00	393.93
3	328.08	394.03	393.96
4	347.90	399.97	399.90
5	348.00	400.00	399.94
6	360.89	405.73	405.67
7	365.90	407.96	407.90
8	366.00	408.00	407.90
9	376.00	414.25	408.00
10	392.00	424.25	414.50
11	406.00	433.00	422.00
12	446.00	458.00	448.59
13	460.00	458.00	457.90
14	478.00	458.00	457.90

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	430.50	448.31	362.20	406.36	352.66	498.47	92.60	1.10	
2	433.07	449.92	362.25	406.36	350.19	505.33	99.70	1.08	
3	435.64	451.52	362.25	406.36	347.39	512.71	107.39	1.08	
4	438.21	453.13	361.46	406.25	346.87	516.40	111.11	1.06	
5	440.78	454.73	361.49	406.24	343.63	524.51	119.61	1.06	
6	443.34	456.34	361.51	406.23	339.96	533.30	128.89	1.06	
7	445.91	457.95	361.54	406.23	339.25	537.28	132.93	1.04	
8	448.48	458.00	351.41	401.76	320.16	567.58	168.74	1.03	
. 9	451.05	458.00	361.57	406.20	323.29	575.49	173.57	1.03 .	OK
10	453.62	458.00	351.48	401.72	298.16	619.30	224.02	1.04	
11	456.19	458.00	351.50	401.69	276.43	666.73	275.46	1.05	
12	458.75	458.00	356.70	403.99	299.81	634.93	237.84	1.08	
13	461.32	458.00	356.71	403.99	286.66	667.96	273.11	1.10	
14	463.89	458.00	356.73	403.99	281.88	685.82	291.60	1.11	
15	466.46	458.00	356.76	404.00	276.75	704.95	311.40	1.13	
16	469.03	458.00	347.10	399.78	321.55	610.05	211.82	1.15	
17	471.60	458.00	347.10	399.78	320.46	618.95	220.79	1.16	
18	474.16	458.00	341.08	398.18	318.38	626.63	229.57	1.17	
19	476.73	458.00	331.03	395.23	315.94	630.74	236.00	1.19	
20	479.30	458.00	331.12	395.23	315.38	638.68	243.96	1.20	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	331.31	395.13	451.05	458.00	294.38	610.92	218.93	1.09	
2	336.98	396.73	453.62	458.00	287.13	633.29	241.76	1.06	
3	341.85	398.19	453.62	458.00	288.28	632.64	240.49	1.05	
4	347.10	399.74	453.62	458.00	289.66	631.29	238.56	1.04	
5	351.41	401.76	448.48	458.00	320.16	567.58	168.74	1.03	
6	356.64	403.99	451.05	458.00	315.49	585.44	186.05	1.03	
. 7	361.57	406.20	451.05	458.00	323.29	575.49	173.57	1.03 .	OK
8	366.80	408.71	451.05	458.00	330.86	566.78	162.11	1.04	
9	372.28	411.94	451.05	458.00	342.16	553.84	145.06	1.05	
10	376.69	414.95	451.05	458.00	352.56	542.37	129.68	1.06	
11	382.19	418.17	451.05	458.00	362.20	532.16	115.73	1.08	
12	387.20	421.29	453.62	458.00	367.02	536.23	116.70	1.10	
13	392.25	424.42	451.05	458.00	383.49	508.03	84.07	1.12	
14	396.69	427.50	451.05	458.00	392.26	499.09	71.73	1.15	
15	402.16	430.65	453.62	458.00	395.77	504.75	74.38	1.18	
16	406.97	433.77	453.62	458.00	405.13	494.33	60.59	1.22	
17	411.80	436.90	453.62	458.00	413.09	486.33	49.45	1.28	
18	416.95	440.04	453.62	458.00	421.19	477.80	37.99	1.36	
19	422.19	443.13	456.19	458.00	426.13	480.43	37.50	1.48	
20	427.09	446.27	453.62	458.00	431.69	471.72	25.87	1.66	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.03

Critical Circle: Xc = 323.29[ft], Yc = 575.49[ft], R = 173.57[ft]. (Number of slices used = 55)

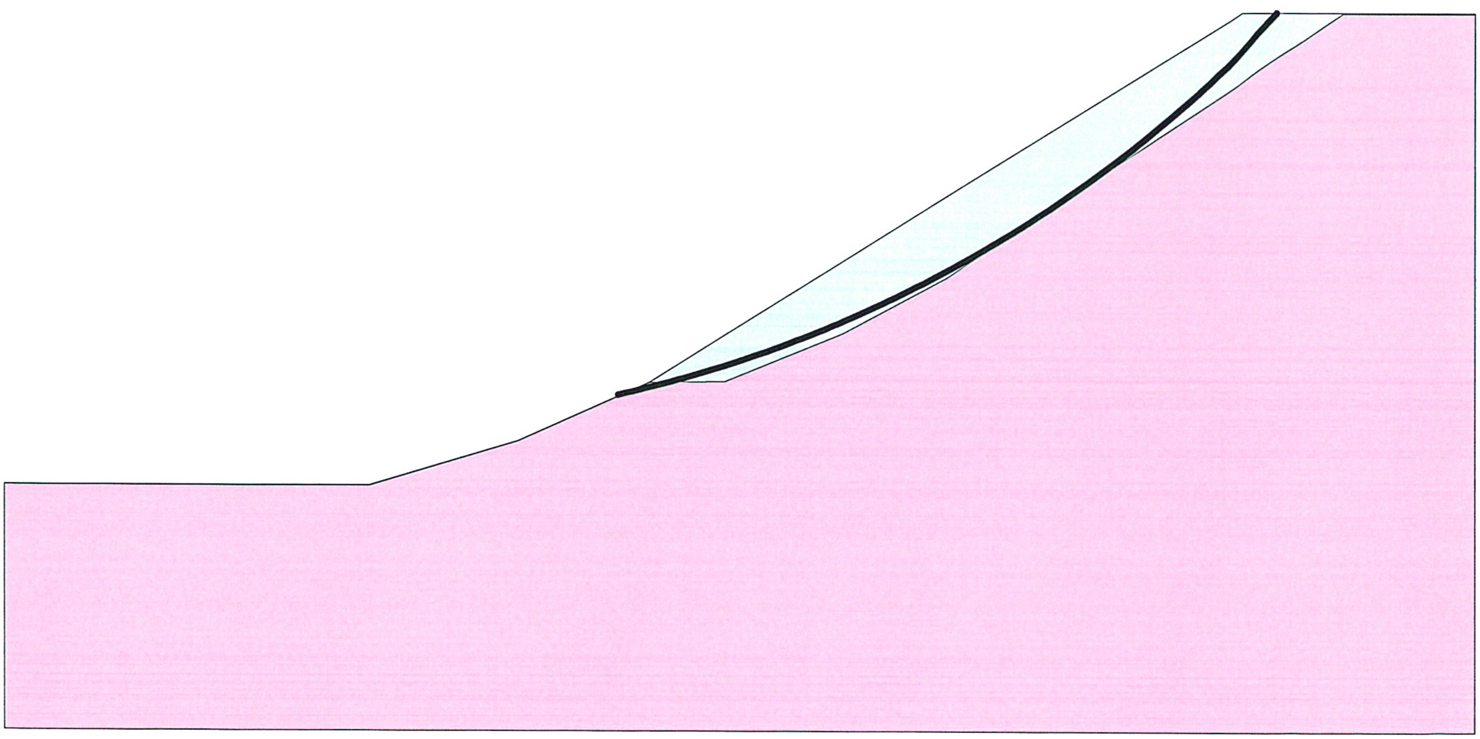
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

NOT CONDUCTED

Three-Part Wedge Stability Analysis

NOT CONDUCTED

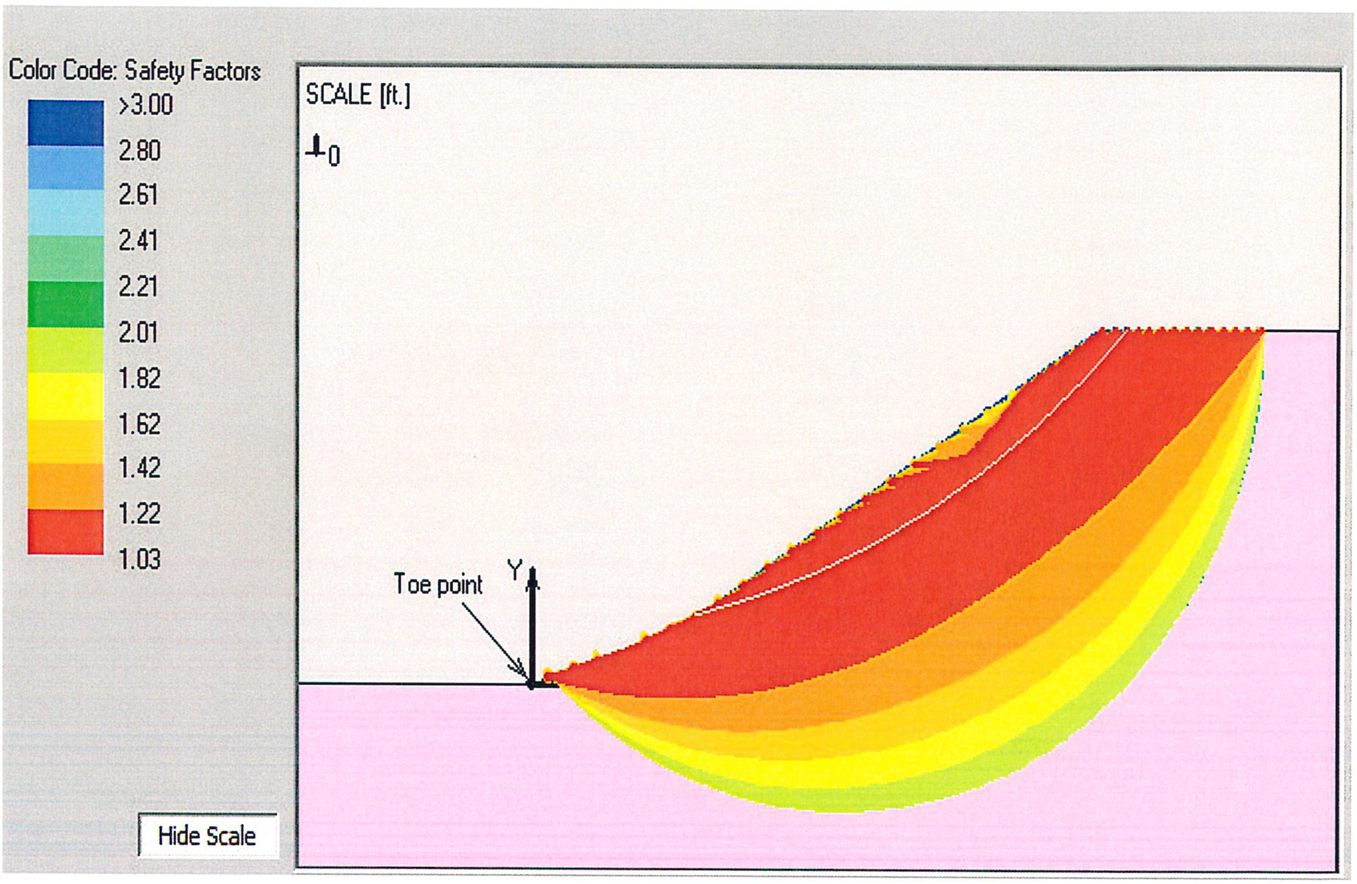
REINFORCEMENT LAYOUT: DRAWING



SCALE:

0246[ft]

SAFETY MAP: BISHOP ROTATIONAL ANALYSIS MODE



Chalk Bluffs water treatment Plant

PROJECT IDENTIFICATION

Title:	Chalk Bluffs water treatment Plant
Project Number:	2056 -
Client:	TMWA
Designer:	Randy Reynolds
Station Number:	0+90

Description:

Seismic conditon with reinforcement at 5 foot spacings

Company's information:

Name:	CME
Street:	
	,
Telephone #:	
Fax #:	
E-Mail:	

Original file path and name: V:\Active\ ..... .5H1.0V w reinforcement seismic 5 foot spacing.MSE
Original date and time of creating this file: Mon Jan 08 16:21:39 2018

PROGRAM MODE: Analysis of a General Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft^3]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft^2]
....1.......Structural fill material...........................	120.0	28.0	250.0
....2.......diatomaceous siltstone..........................	75.0	37.0	165.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Additional Reduction Factor, RFa	Coverage Ratio, Rc
1	Geosynthetic type #1	9870.00	1.40	1.00	2.60	1.00	1.00

Interaction Parameters		== Direct Sliding ==		==== Pullout ====	
Type #	Geosynthetic Designated Name	Cds-phi	Cds-c	Ci	Alpha
1	Geosynthetic type #1	0.80	0.00	0.80	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Water is not present

SEISMICITY

Horizontal peak ground acceleration coefficient, Ao = 0.500
Design horizontal seismic coefficient, kh = Am = 0.50 x Ao = 0.250 & design vertical seismic coefficient, kv (down) = 0.000 x kh = 0.000

DRAWING OF SPECIFIED GEOMETRY - GENERAL - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.

GEOMETRY

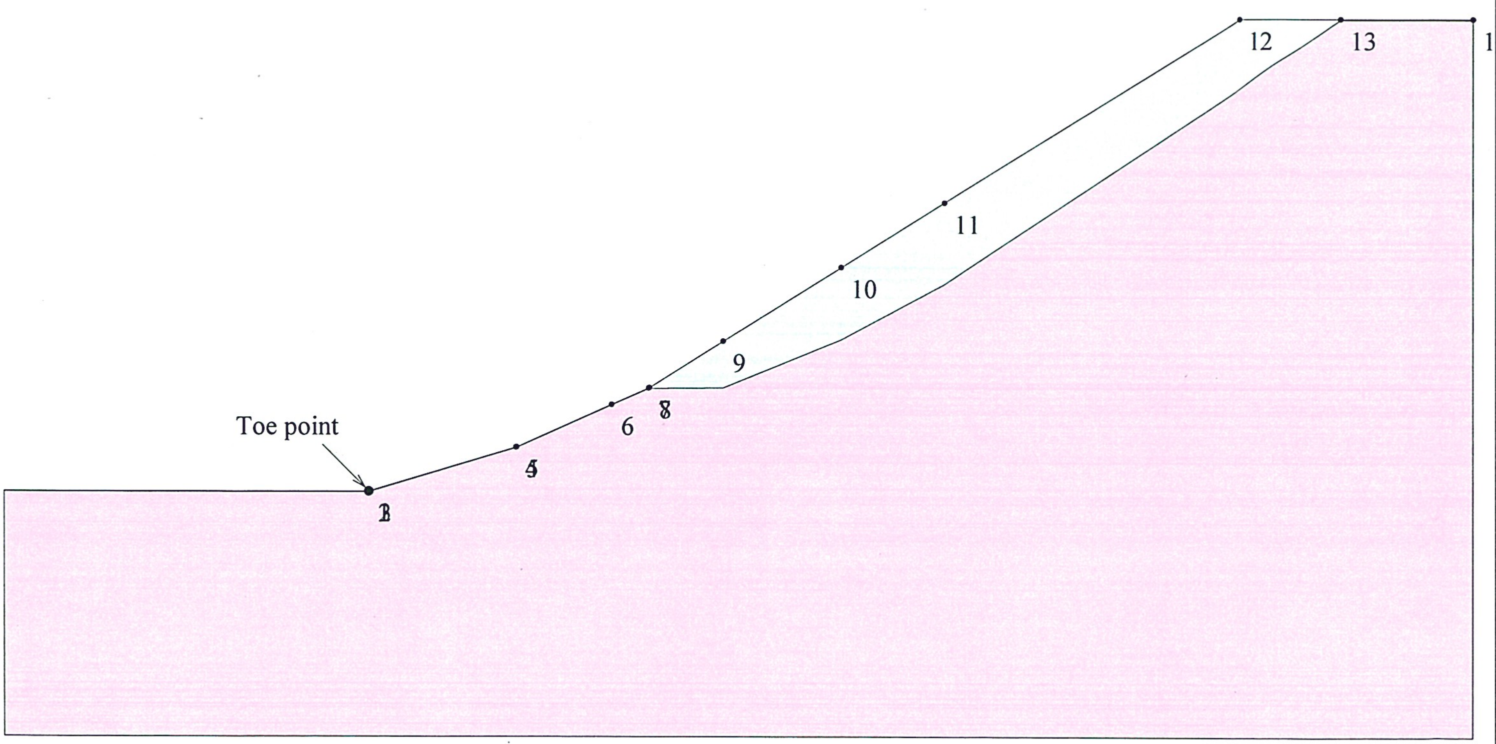
Soil profile contains 2 layers (see details in next page)

UNIFORM SURCHARGE

Surcharge load, Q1 ....None
Surcharge load, Q2 ....None
Surcharge load, Q3 ....None

STRIP LOAD

............................None...................................



SCALE:

0246[ft]

TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]

	#	Xi	Yi
Top of Layer 1	1	328.00	394.00
	2	348.00	400.00
	3	366.00	408.00
	4	446.00	458.00
	5	478.00	458.00
Top of Layer 2	6	327.90	393.90
	7	347.90	399.90
	8	365.90	407.90
	9	376.00	408.00
	10	392.00	414.50
	11	406.00	422.00
	12	460.00	457.90

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]

#	X	Y1	Y2
1	327.90	394.00	393.90
2	328.00	394.00	393.93
3	328.08	394.03	393.96
4	347.90	399.97	399.90
5	348.00	400.00	399.94
6	360.89	405.73	405.67
7	365.90	407.96	407.90
8	366.00	408.00	407.90
9	376.00	414.25	408.00
10	392.00	424.25	414.50
11	406.00	433.00	422.00
12	446.00	458.00	448.59
13	460.00	458.00	457.90
14	478.00	458.00	457.90

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	446.00	458.00	347.66	399.92	349.64	508.87	108.97	1.18	
2	447.76	458.00	347.66	399.92	349.69	511.72	111.82	1.17	
3	449.51	458.00	347.66	399.92	347.74	518.13	118.20	1.16	
4	451.26	458.00	347.66	399.92	343.22	529.27	129.43	1.15	
5	453.01	458.00	347.66	399.92	340.53	537.44	137.70	1.13	
6	454.77	458.00	347.66	399.92	337.49	546.46	146.90	1.12	
7	456.52	458.00	347.66	399.91	330.61	562.92	163.90	1.11	
8	458.27	458.00	347.66	399.91	330.10	567.70	168.70	1.11	
9	460.02	458.00	347.66	399.91	325.59	580.33	181.76	1.11	OK
10	461.78	458.00	347.66	399.91	324.93	585.71	187.18	1.11	
11	463.53	458.00	347.66	399.91	324.25	591.21	192.73	1.12	
12	465.28	458.00	347.66	399.91	323.56	596.84	198.40	1.13	
13	467.03	458.00	347.66	399.91	322.85	602.59	204.19	1.14	
14	468.79	458.00	347.66	399.91	322.13	608.47	210.12	1.15	
15	470.54	458.00	347.66	399.91	321.39	614.49	216.18	1.16	
16	472.29	458.00	347.66	399.91	320.64	620.63	222.37	1.16	
17	474.04	458.00	343.35	398.71	320.32	623.18	225.65	1.17	
18	475.80	458.00	335.05	396.39	320.52	621.14	225.22	1.18	
19	477.55	458.00	331.06	395.23	315.76	633.25	238.51	1.19	
20	479.30	458.00	331.12	395.23	315.38	638.68	243.96	1.20	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	331.92	395.30	460.02	458.00	322.94	575.85	180.78	1.12	
2	336.04	396.46	460.02	458.00	323.60	577.18	181.15	1.11	
3	338.89	397.52	460.02	458.00	324.25	578.38	181.45	1.11	
4	343.23	398.70	460.02	458.00	324.91	579.44	181.67	1.11	
5	347.66	399.91	460.02	458.00	325.59	580.33	181.76	1.11	OK
6	350.60	401.45	460.02	458.00	326.07	583.08	183.27	1.11	
7	355.04	403.24	460.02	458.00	330.93	577.49	175.91	1.11	
8	358.42	404.90	460.02	458.00	339.65	564.56	160.77	1.12	
9	362.77	406.67	458.27	458.00	351.17	542.76	136.58	1.13	
10	366.66	408.56	458.27	458.00	362.84	525.24	116.74	1.15	
11	370.16	410.89	460.02	458.00	353.41	552.11	142.20	1.13	
12	373.87	413.34	458.27	458.00	370.81	521.20	107.90	1.16	
13	378.01	415.72	458.27	458.00	365.74	536.32	121.22	1.16	
14	381.79	418.14	458.27	458.00	378.79	517.20	99.11	1.18	
15	385.64	420.51	460.02	458.00	373.92	536.29	116.38	1.21	
16	389.66	422.94	456.52	458.00	388.46	506.52	83.58	1.22	
17	393.58	425.33	460.02	458.00	382.59	531.58	106.82	1.27	
18	397.57	427.74	458.27	458.00	396.21	506.48	78.75	1.26	
19	401.09	430.17	456.52	458.00	407.37	486.77	56.95	1.34	
20	404.88	432.53	458.27	458.00	404.02	503.01	70.49	1.31	
21	408.70	434.97	456.52	458.00	414.30	484.51	49.85	1.40	
22	412.81	437.33	460.02	458.00	412.92	501.32	63.99	1.40	
23	416.43	439.77	458.27	458.00	422.16	483.73	44.34	1.50	
24	420.55	442.12	460.02	458.00	421.54	496.65	54.54	1.52	
25	424.05	444.57	460.02	458.00	429.94	483.71	39.57	1.68	
26	428.21	446.92	460.02	458.00	431.83	487.72	40.97	1.77	
27	432.02	449.33	460.02	458.00	438.31	478.55	29.90	2.03	
28	435.74	451.75	460.02	458.00	441.97	477.86	26.84	2.44	
29	439.63	454.17	460.02	458.00	447.20	470.05	17.60	3.32	
30	443.49	456.59	460.02	458.00	451.04	465.69	11.82	5.65	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.11

Critical Circle: Xc = 325.59[ft], Yc = 580.33[ft], R = 181.76[ft]. (Number of slices used = 59)

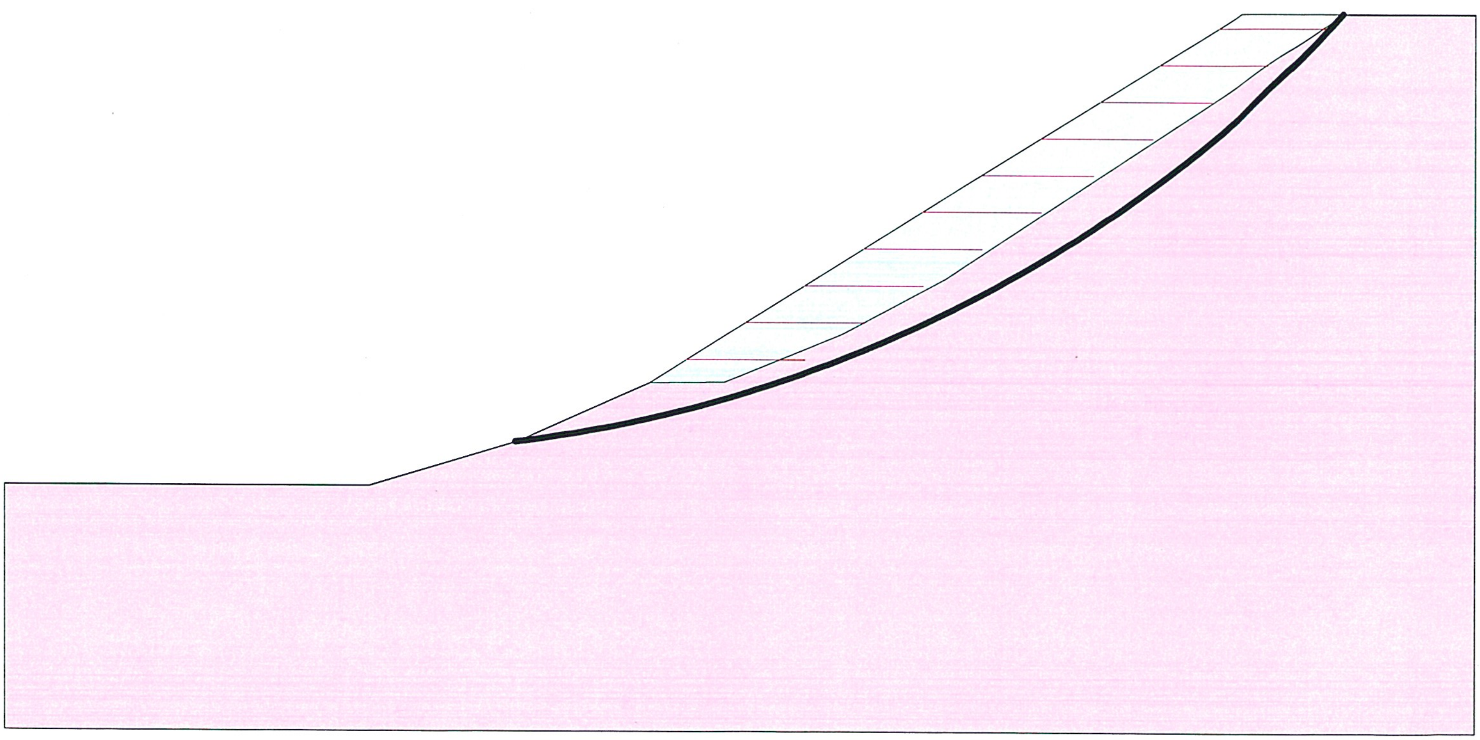
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

N O T C O N D U C T E D

Three-Part Wedge Stability Analysis

N O T C O N D U C T E D

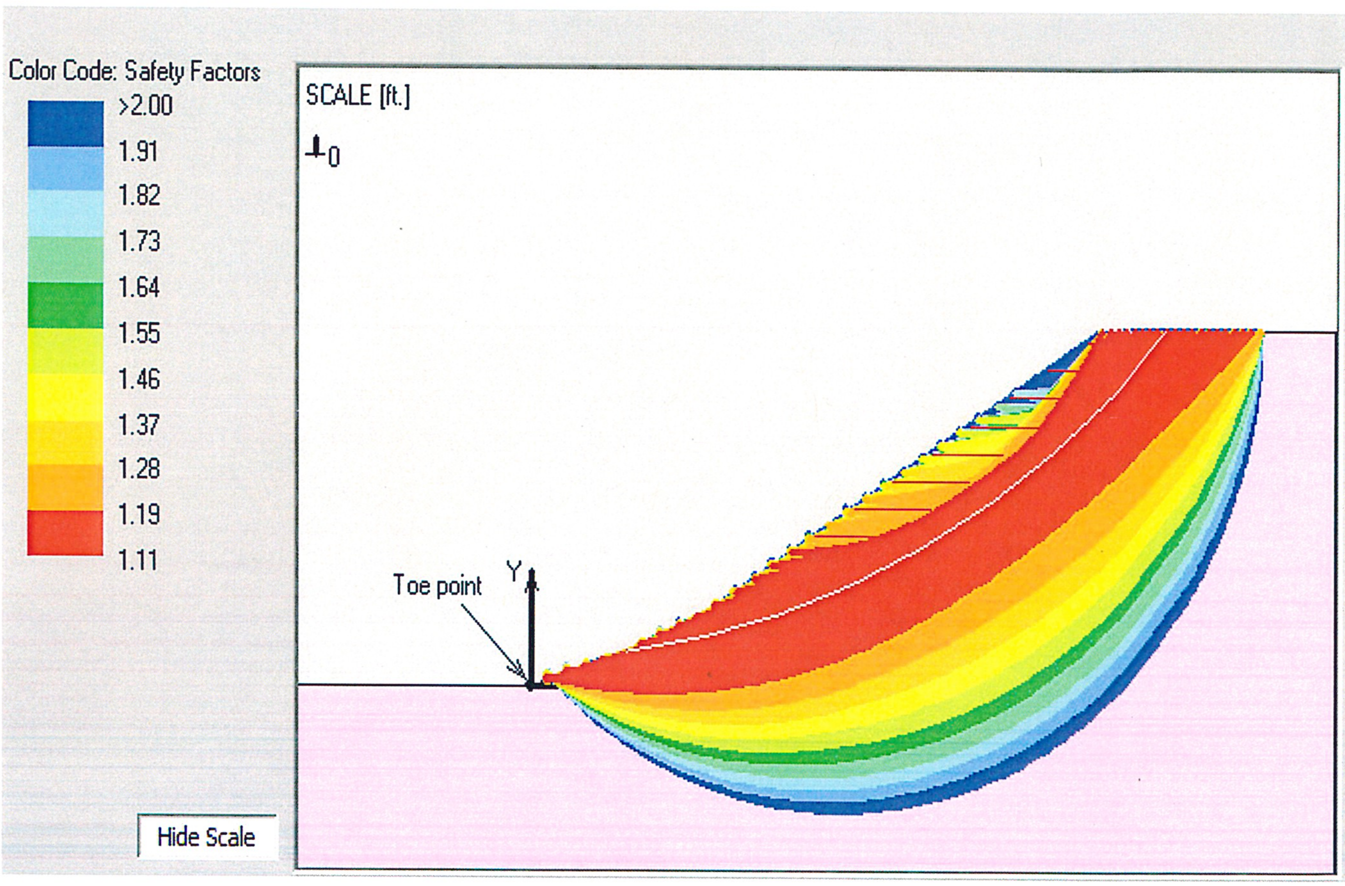
REINFORCEMENT LAYOUT: DRAWING



SCALE:

0246[ft]

SAFETY MAP: BISHOP ROTATIONAL ANALYSIS MODE



Chalk Bluffs water treatment Plant

PROJECT IDENTIFICATION

Title:	Chalk Bluffs water treatment Plant
Project Number:	2056 -
Client:	TMWA
Designer:	Randy Reynolds
Station Number:	0+90

Description:
Static conditon with reinforcement at 5 foot spacings and surcharge.

Company's information:

Name: CME
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name: V:\Active\ ..... 1.5H1.0V w reinforcement static 5 foot spacing.MSE
Original date and time of creating this file: Mon Jan 08 16:21:39 2018

PROGRAM MODE: Analysis of a General Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ^{3}]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ^{2}]
....1.......Structural fill material..........................	120.0	28.0	250.0
....2.......diatomaceous siltstone..........................	75.0	37.0	165.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Additional Reduction Factor, RFa	Coverage Ratio, Rc
1	Geosynthetic type #1	9870.00	1.40	1.00	2.60	1.00	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
1	Geosynthetic type #1	0.80	0.00	0.80	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Water is not present

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - GENERAL - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.

GEOMETRY

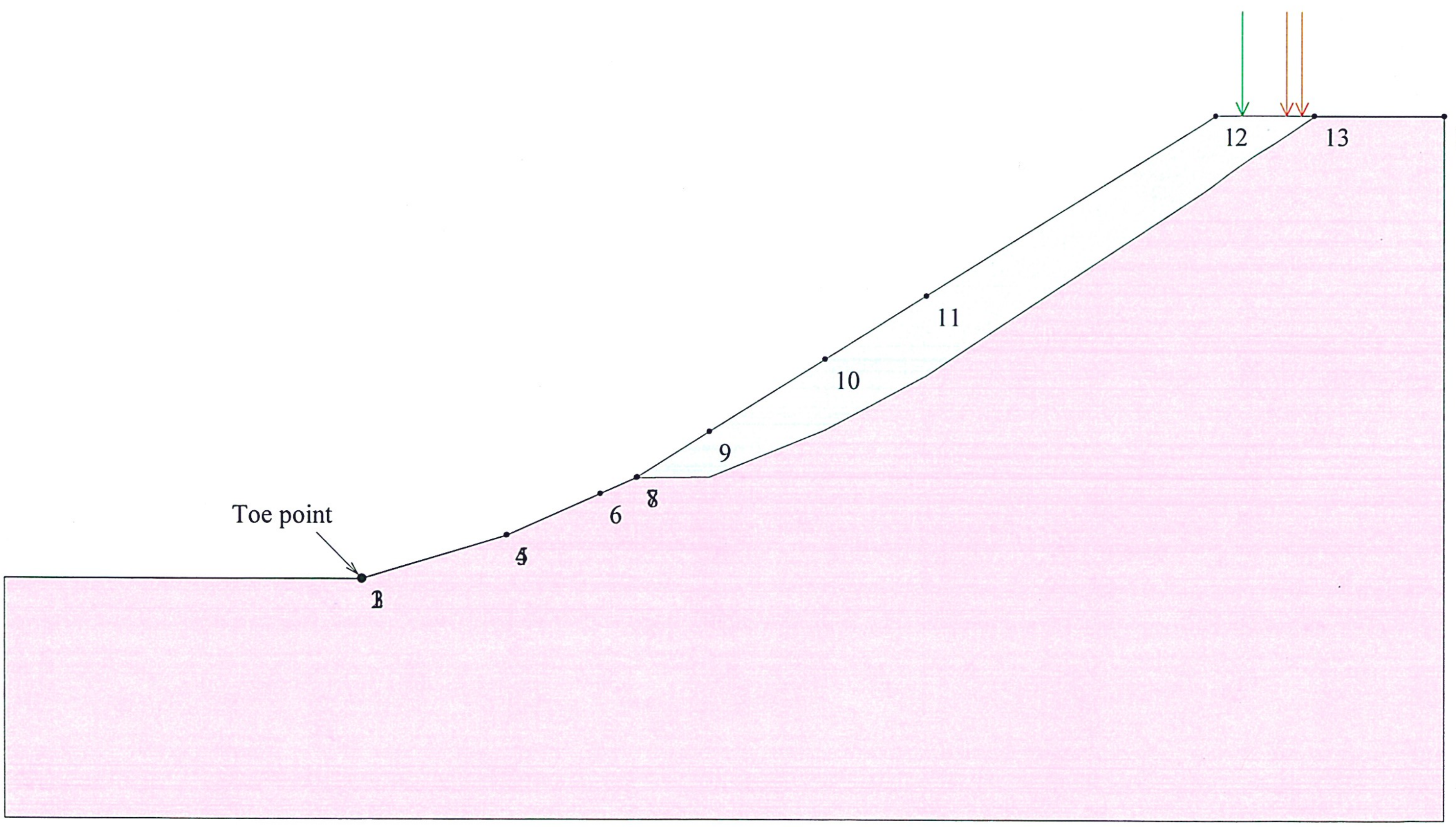
Soil profile contains 2 layers (see details in next page)

UNIFORM SURCHARGE

Load Q1 = 4000.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 450.00 and ends at X1e = 452.50 [ft].
Load Q2 = 4000.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X2s = 456.00 and ends at X2e = 458.50 [ft].
Surcharge load, Q3 ....None

STRIP LOAD

............................None..................................



SCALE:

0 2 4 6 [ft]

TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]

	#	Xi	Yi
Top of Layer 1	1	328.00	394.00
	2	348.00	400.00
	3	366.00	408.00
	4	446.00	458.00
	5	478.00	458.00
Top of Layer 2	6	327.90	393.90
	7	347.90	399.90
	8	365.90	407.90
	9	376.00	408.00
	10	392.00	414.50
	11	406.00	422.00
	12	460.00	457.90

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]

#	X	Y1	Y2
1	327.90	394.00	393.90
2	328.00	394.00	393.93
3	328.08	394.03	393.96
4	347.90	399.97	399.90
5	348.00	400.00	399.94
6	360.89	405.73	405.67
7	365.90	407.96	407.90
8	366.00	408.00	407.90
9	376.00	414.25	408.00
10	392.00	424.25	414.50
11	406.00	433.00	422.00
12	446.00	458.00	448.59
13	460.00	458.00	457.90
14	478.00	458.00	457.90

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	446.00	458.00	343.16	398.83	351.24	503.73	105.22	1.84	
2	447.76	458.00	343.17	398.82	351.42	506.25	107.75	1.82	
3	449.51	458.00	343.18	398.80	347.97	515.28	116.58	1.81	
4	451.26	458.00	443.53	456.53	446.23	463.36	7.35	1.63	
5	453.01	458.00	370.05	410.89	358.45	527.93	117.61	1.57	
6	454.77	458.00	347.66	399.91	334.42	552.11	152.77	1.59	
7	456.52	458.00	347.66	399.91	326.86	569.93	171.28	1.58	
8	458.27	458.00	370.13	410.89	354.04	547.00	137.06	1.52	OK
9	460.02	458.00	362.78	406.68	350.79	547.18	141.02	1.57	
10	461.78	458.00	370.18	410.89	349.20	564.27	154.81	1.60	
11	463.53	458.00	332.01	395.29	314.36	601.57	207.04	1.63	
12	465.28	458.00	336.07	396.46	318.44	599.88	204.19	1.65	
13	467.03	458.00	334.83	396.40	326.22	587.55	191.34	1.67	
14	468.79	458.00	339.06	397.57	330.19	586.06	188.70	1.69	
15	470.54	458.00	343.33	398.73	330.34	592.76	194.47	1.72	
16	472.29	458.00	343.36	398.76	340.84	574.17	175.43	1.75	
17	474.04	458.00	347.66	399.92	341.45	579.97	180.16	1.78	
18	475.80	458.00	351.09	401.56	341.80	588.09	186.76	1.82	
19	477.55	458.00	331.10	395.28	324.82	612.26	217.07	1.85	
20	479.30	458.00	331.17	395.28	324.66	616.94	221.75	1.88	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point (X , Y) [ft]		Entry Point (X , Y) [ft]		Critical Circle (Xc , Yc , R) [ft]			Fs	STATUS
1	331.78	395.27	458.27	458.00	315.66	586.66	192.07	1.56	
2	335.94	396.44	458.27	458.00	316.08	588.23	192.82	1.56	
3	338.84	397.50	458.27	458.00	320.76	581.30	184.69	1.56	
4	343.23	398.75	458.27	458.00	338.66	548.93	150.25	1.55	
5	347.66	399.92	458.27	458.00	336.91	554.75	155.20	1.54	
6	350.60	401.51	458.27	458.00	338.04	556.30	155.30	1.53	
7	355.03	403.25	458.27	458.00	339.16	557.90	155.46	1.53	
8	358.42	404.90	458.27	458.00	340.28	559.44	155.60	1.52	
9	362.77	406.67	458.27	458.00	348.19	548.29	142.37	1.54	
10	366.48	408.49	458.27	458.00	333.80	578.91	173.53	1.57	
11	370.13	410.89	458.27	458.00	354.04	547.00	137.06	1.52	OK
12	373.87	413.36	458.27	458.00	374.58	514.14	100.77	1.58	
13	377.98	415.71	458.27	458.00	362.97	541.57	126.75	1.54	
14	381.81	418.15	458.27	458.00	380.58	513.79	95.65	1.56	
15	385.60	420.50	458.27	458.00	374.44	531.29	111.35	1.55	
16	389.67	422.94	458.27	458.00	386.76	513.27	90.38	1.54	
17	393.46	425.31	458.27	458.00	383.13	526.39	101.60	1.60	
18	397.56	427.74	458.27	458.00	394.73	509.44	81.75	1.53	
19	401.12	430.16	458.27	458.00	406.76	491.17	61.27	1.61	
20	404.88	432.54	458.27	458.00	405.26	500.45	67.92	1.56	
21	409.02	434.94	458.27	458.00	417.29	481.42	47.20	1.61	
22	412.47	437.35	458.27	458.00	414.87	493.12	55.83	1.58	
23	416.62	439.74	458.27	458.00	422.75	482.39	43.08	1.59	
24	420.31	442.12	458.27	458.00	419.63	497.03	54.92	1.58	
25	424.31	444.54	458.27	458.00	430.01	479.73	35.65	1.61	
26	428.03	446.94	458.27	458.00	432.04	482.85	36.13	1.55	
27	431.99	449.33	458.27	458.00	438.06	475.12	26.49	1.62	
28	435.85	451.73	453.01	458.00	440.33	466.10	15.05	1.68	
29	439.71	454.15	453.01	458.00	444.99	460.80	8.49	1.73	
30	443.53	456.53	451.26	458.00	446.23	463.36	7.35	1.63	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.52
Critical Circle: Xc = 354.04[ft], Yc = 547.00[ft], R = 137.06[ft]. (Number of slices used = 53)

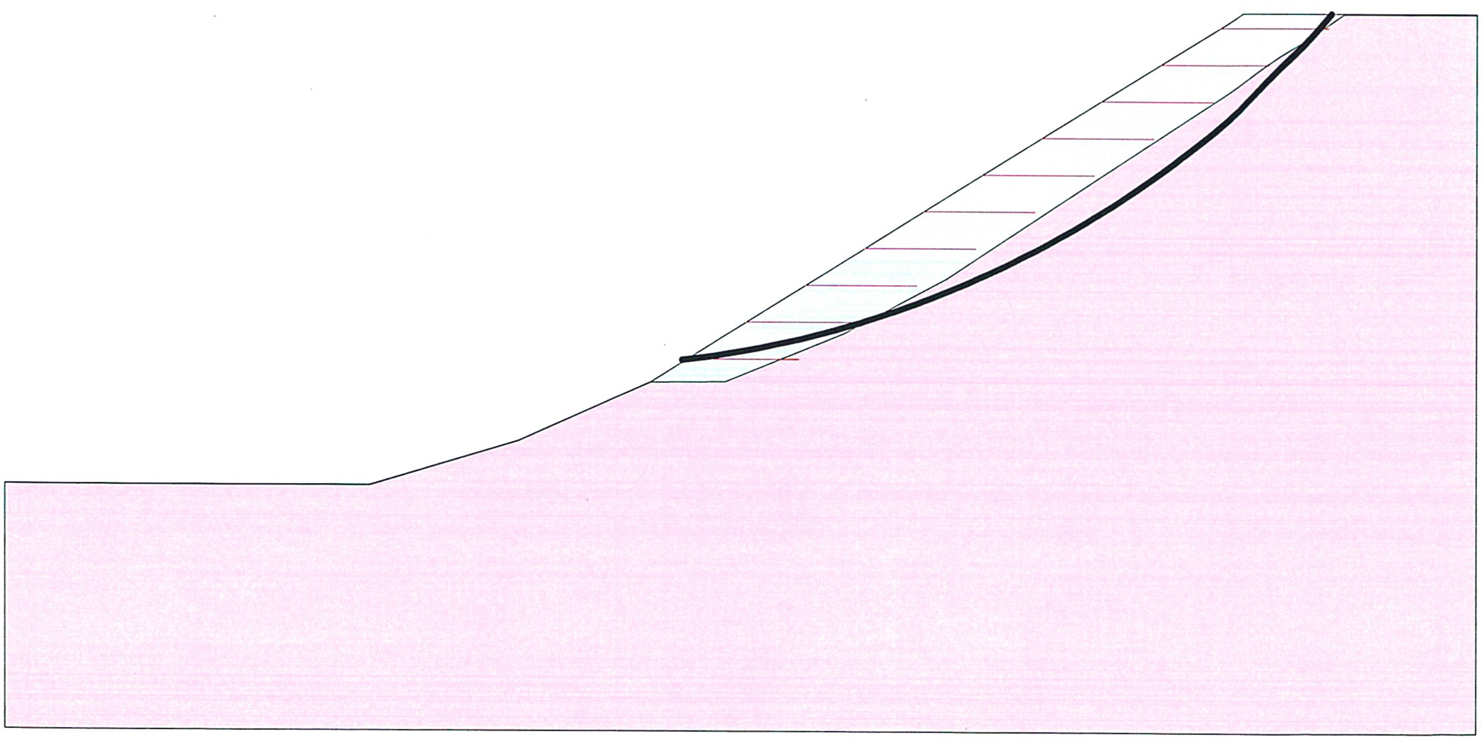
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

NOT CONDUCTED

Three-Part Wedge Stability Analysis

NOT CONDUCTED

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0246[ft]

SAFETY MAP: BISHOP ROTATIONAL ANALYSIS MODE