APPENDIX F – GEOLOGY AND SOILS

Preliminary Geotechnical Investigation

PRELIMINARY GEOTECHNICAL INVESTIGATION

October 29, 2018

Prepared For:

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Westside Canal Energy Center
Imperial Valley, CA

NV5 PROJECT No.: 1076

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October 29, 2018
NV5 Project No: 1076

Subject: Preliminary Geotechnical Investigation Report

Project: Westside Canal Energy Center
Imperial Valley, California

Dear Mr. Pomillo:

As requested, NV5 is pleased to present the results of the preliminary geotechnical investigation for the subject project. The purpose of the investigation was to evaluate the subsurface conditions at the proposed Westside Canal Energy Center (WCEC) site located in the Imperial Valley area of Imperial County, California. It is understood that the site encompasses approximately 127 acres located on the south side of the Westside Main Canal, and approximately 2,000 feet north of the existing Imperial Valley Substation. It is understood that the project will include the WCEC Project Substation, the T.O. Interconnection Substation, solar photovoltaic arrays, battery storage, an operations and maintenance facility, and a bridge over the Westside Main Canal which will provide primary site access. Per NV5's proposal for geotechnical engineering services dated August 28, 2018, geotechnical design parameters for the proposed was excluded from the scope of this investigation and will be completed at a later date under a separate proposal. The results of the geotechnical field explorations, laboratory tests, and geotechnical engineering recommendations and conclusions are presented herewith.

Based on the subsurface exploration, subsequent testing of the subsurface soils, and engineering analyses, it was concluded that the construction of the proposed project is geotechnically feasible. The geotechnical information presented herein is intended to assist the project design team and construction contractor in their understanding of the geotechnical factors affecting the proposed project, and the preliminary recommendations will be incorporated into the project design and implemented construction.

The forthcoming project specifications, in particular the earthwork/compaction sections, should be reviewed by NV5 for consistency with this report prior to the bid process in order to avoid possible conflicts, misinterpretations, and inadvertent omissions. It should also be noted, that the applicability and final evaluation of the recommendations presented herein, are contingent upon construction phase field monitoring by NV5, in light of the widely acknowledged importance of geotechnical consultant continuity through the various design, planning and construction stages of a project.

NV5 appreciates the opportunity to provide this geotechnical engineering service for this project and looks forward to continuing its role as your geotechnical engineering consultant.

Respectfully submitted,

NV5 West, Inc.

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1.0 INTRODUCTION

This report presents results of NV5's preliminary geotechnical investigation for the proposed Westside Canal Energy Center (WCEC) in Imperial Valley, California. The approximate location of the project area is shown on *Figure 1, Site Location Map*. The purpose of this study was to evaluate the subsurface conditions at the project site and to provide preliminary geotechnical recommendations for the design and construction of the proposed facility. This report summarizes the data collected and presents findings, conclusions, and preliminary recommendations.

This report has been prepared for the exclusive use of the client and their consultants to describe the geotechnical factors at the project site which should be considered in the design and construction of the proposed project. In particular, it should be noted that this report has not been prepared from the perspective of a construction bid preparation instrument and should be considered by prospective bidders only as a source of general information subject to interpretation and refinement by their own expertise and experience, particularly with regard to construction feasibility. Contract requirements as set forth by the project plans and specifications will supersede any general observations and specific recommendations presented in this report.

2.0 SCOPE OF SERVICES

NV5's scope of services for this project included the following tasks:

- Review of readily available background data, published geologic maps, topographic maps, seismic hazard maps and literature relevant to the subject site.
- Review of a preliminary project sketch provided by Sempra Renewables.
- Coordinating with entities having an interest in the field exploration activities including Sempra Renewables, the drilling subcontractor (Pacific Drilling), and Underground Service Alert (USA) for mark-out prior to site exploration.
- Conducting a subsurface investigation, which included the drilling, logging, and sampling of seven (7) exploratory borings located within the project area to a maximum depth of approximately 80 feet below ground surface (bgs). The original proposed scope of work included six (6) borings; however, an additional boring (B-1a) was performed adjacent to boring B-1 which was terminated due to drilling contractor's equipment issues. Soil samples obtained from the borings were transported to NV5's in-house laboratory for observation and testing.
- Performing laboratory testing on selected representative bulk and relatively undisturbed soil samples obtained during the field exploration program to evaluate their pertinent geotechnical engineering properties.
- Site electrical resistivity evaluation using the 4-pin Wenner method.
- Performing an assessment of general seismic conditions and geotechnical hazards affecting the area and potential impacts on the subject project.
- Engineering evaluation of the data collected to develop geotechnical design parameters and recommendations for the design of the proposed construction.

- Preparation of this report including reference maps and graphics, presenting findings, conclusions and geotechnical recommendations for the design and construction of the proposed project.

3.0 SITE AND PROJECT DESCRIPTION

The proposed WCEC site is located in the Imperial Valley area of Imperial County, California. The area in the immediate vicinity of the project limits, as shown on the conceptual site layout provided by Sempra Renewables, is relatively flat with a gentle gradient downward to the northeast. A graded agricultural pad in the south-central portion of the project site rests approximately 8 feet above the northern portion of the site. Elevations at the project site range from approximately 3 to 21 feet below mean sea level. The Westside Main Canal lies to the north of the site (refer to *Figure 2, Field Exploration Plan*). The property is currently undeveloped, was graded for agricultural use in the past, and is sparsely vegetated with weeds. Overhead electrical transmission lines and transmission towers are located immediately to the west and south of the site. The transmissions lines extend from the existing Imperial Valley Substation approximately 0.3 miles south of the WCEC.

Based on preliminary information provided by Sempra Renewables, it is understood that the proposed construction includes the WCEC Project Substation, the T.O. Interconnection Substation, solar photovoltaic arrays, battery storage, an operations and maintenance facility, and a bridge over the Westside Main Canal which will provide primary site access. Detailed site layout and construction plans had not been developed as of the date of this report.

4.0 FIELD EXPLORATION PROGRAM

Before starting NV5's field exploration program, Underground Service Alert was notified of the operations for underground utility marking at the locations of exploration. The subsurface conditions were explored from September 17 through October 2, 2018 by drilling, logging, and sampling of seven exploratory borings (B-1 and B-1a through B-6). The borings were drilled to maximum depths ranging between about 20 to 80 feet bgs by Pacific Drilling using a Unimog M-5 hollow stem auger drill rig and a Diedrich D-50 Turbo hollow stem auger and mud-rotary drill rig.

The borings were logged by an NV5 geologist. Representative samples of the soils encountered were obtained for visual soils classification and laboratory testing. The soil conditions encountered in the borings were visually examined, classified, and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the exploratory test borings are presented in *Appendix A, Exploratory Boring Logs*. The approximate locations of the exploratory borings are presented on *Figure 2, Field Exploration Plan*. Subsequent to logging and sampling, the borings were backfilled.

The bulk and relatively undisturbed drive samples of the soils encountered in the borings were tagged in the field and transported to NV5's laboratory for observation and testing. The drive samples were obtained using the California Modified Split Spoon and Standard Penetration Test (SPT) samplers, as described below.

California Modified Split Spoon Sampler

The split barrel drive sampler was driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1587. The number of blows for the last two of three 6-inch intervals were recorded during sampling and are presented in the logs of borings. The sampler has external and internal diameters of approximately 3.0 and 2.4 inches, respectively, and the inside of the sampler is lined with 1-inch-long brass rings. The relatively undisturbed soil samples within the rings were removed, sealed, and transported to the laboratory for observation and testing.

Standard Penetration Test (SPT) Sampler

A split barrel sampler was driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1586. The numbers of blows for the last two of three 6-inch intervals were recorded during sampling and are presented in the logs of borings (i.e., N-value). The sampler has external and internal diameters of 2.0 and 1.375 inches, respectively. The soil samples obtained in the interior of the barrel were measured, removed, sealed and transported to the laboratory for observation and testing.

5.0 FIELD RESISTIVITY TESTING

On-site resistivity surveys were conducted from September 20 through September 21, 2018, in general accordance with ASTM Method G57. The locations of the aforementioned tests can be found on *Figure 2, Field Exploration Plan*. The surveys were conducted along two perpendicular lines with readings taken with electrode spacings of 2, 4, 6, 8, 12, 20, 30, 50, 100 and 200 feet. The resistivity testing services were provided by Southwest Geophysics, Inc. under subcontract agreement with NV5. Details of the resistivity surveys and test data are presented in *Appendix B, Field Resistivity Test Data*.

6.0 LABORATORY SOIL TESTING

Laboratory testing was performed on selected representative bulk and relatively undisturbed soil samples obtained from the exploratory borings, to aid in the material classifications and to evaluate engineering properties of the materials encountered (see *Appendix C, Laboratory Test Results*). The following tests were performed:

- In-situ density and moisture content (ASTM D2937 and ASTM D2216);
- Particle size analyses (ASTM D6913, ASTM D2487 and ASTM D1140);
- Direct shear (ASTM D3080);
- Expansion index (ASTM D4829);
- Atterberg Limits (ASTM 4318);
- Thermal Resistivity (ASTM D5334 and IEEE 442);
- R-Value (ASTM D2844); and

- Corrosivity test series including sulfate content, chloride content, pH-value, and resistivity (CTM 417, 422 and 532/643, respectively).

Testing was performed in general accordance with applicable ASTM standards, Institute of Electrical and Electronics Engineers (IEEE) standards, and California Test Methods. A summary of the laboratory testing program and the laboratory test results are presented in *Appendix C*.

7.0 GEOLOGY

7.1 GEOLOGIC SETTING

The project site is located in Imperial County in the southern portion of the Salton Trough, a structural depression within the Colorado Desert geomorphic province. This province is generally a low-lying barren desert basin (in part about 230 feet below mean sea level) dominated by the Salton Sea. The province is a depressed block between active branches of the San Andreas fault system. The fault branches are buried by recent alluvial deposits. The dominant structural features related to the San Andreas fault system consist of northwest-trending faults and fault zones. The major northwest-trending fault zones include the San Jacinto fault, Imperial fault, the Superstition Hills fault, the Elsinore fault and the San Andreas fault. The Salton Trough has been inundated during the Quaternary by an ancient freshwater lake (Lake Cahuilla) which resulted in a sequence of lacustrine (lake) deposits consisting of interbedded sand silt and clay. Remnants of the ancient shorelines of the extinct Lake Cahuilla remain prevalent in the Salton Trough.

7.2 SUBSURFACE CONDITIONS

Geologic materials encountered during the subsurface explorations consisted of natural deposits mapped as Quaternary-aged alluvial deposits and Cahuilla Beds (Qa-Qc, undifferentiated) on published geologic maps. *Figure 3, Regional Geologic Map* presents the general distribution of geologic units in the site area. As encountered in the borings, the soils ranged from tan to brown, dry to wet, stiff to hard lean clay and silt, and medium dense to very dense silty sand and poorly-graded sand with silt. Detailed descriptions of the earth materials encountered are presented on the boring logs in *Appendix A*.

7.3 GROUNDWATER

Groundwater was encountered in the exploratory borings at depths between approximately 9 and 19.1 feet bgs, and indicated in the following Table 1.

Table 1 - Depth to Groundwater as Measured in Each Boring

Boring Number	Depth to Groundwater
B-1	9.5 feet
B-1a	9.0 feet
B-2	12.0 feet
B-3	19.1 feet
B-4	Not encountered
B-5	14.0 feet
B-6	18.0 feet

Groundwater levels may vary due to seasonal fluctuations and factors such as a substantial increase in surface water infiltration from landscape irrigation, agricultural activity, storage facility leaks or unusually heavy precipitation. There is uncertainty in the accuracy of short-term groundwater level measurements, particularly in fine-grained soil. The groundwater level, as reported herein, should not be interpreted to represent an accurate or permanent condition. Seasonal variations in the groundwater levels should be anticipated.

7.4 FAULTS

The numerous faults in southern California include active, potentially active, and inactive faults. As used in this report, the definitions of fault terms are based on those developed for the *Alquist-Priolo Special Studies Zones Act of 1972* and published by the California Division of Mines and Geology (Hart and Bryant, 1997). Active faults are defined as those that have experienced surface displacement within Holocene time (approximately the last 11,000 years) and/or have been included within any of the state-designated Earthquake Fault Zones (previously known as *Alquist-Priolo Special Studies Zones*). Faults are considered potentially active if they exhibit evidence of surface displacement since the beginning of Quaternary time (approximately two million years ago) but not since the beginning of Holocene time. Inactive faults are those that have not had surface movement since the beginning of Quaternary time.

Review of geologic maps and literature pertaining to the general site area indicates that the site is not located within a state-designated Earthquake Fault Zone. Review of the *Earthquake Zones of Required Investigation, Mount Signal Quadrangle, California Geologic Survey, Official Map, dated September 12, 2012* indicates that the project site does not lie within an identified earthquake fault zone (see *Figure 5*). In addition, there are no known major or active faults mapped on the project site. Evidence for active faulting at the site was not observed during the subsurface investigation. The relative location of the site to known active faults in the region is depicted on *Figure 4, Regional Fault Map*. The distance from the site to the projection of traces of surface rupture along major active earthquake fault zones, that could affect the site are listed in the following Table 2.

Table 2 - Distance From the Site to Major Active Faults

Fault Name	Distance From the Site
Route 247 fault zone	1.3 miles
Yuha fault	3.7 miles
North Centinela fault	4.4 miles
Yuha Well fault	5.7 miles
Laguna Salada fault	8.4 miles
Superstition Hills fault	9.7 miles
San Jacinto fault	10.9 miles
Imperial fault	14.7 miles
Elsinore fault	17.2 miles
Elmore Ranch fault	22.3 miles
San Andreas fault	42.7 miles
Earthquake Valley fault	46.9 miles
Algodones fault zone	68.8 miles
Newport Inglewood-Rose Canyon fault	83.9 miles
Palos Verdes-Coronado Bank fault	85.8 miles
Burnt Mountain fault	91.9 miles
Eureka Peak Fault	92.4 miles
Pinto Mountain fault	95.9 miles

8.0 SEISMIC AND GEOTECHNICAL HAZARDS

The principal seismic considerations for most facilities in southern California are damage caused by surface rupturing of fault traces, ground shaking, seismically induced ground settlement and liquefaction. Potential impacts to the project due to faulting, seismicity and other geologic hazards are discussed in the following sections.

8.1 FAULT RUPTURE

The project site is not located within an *Earthquake Fault Zone* delineated by the State of California for the hazard of fault surface rupture. The surface traces of known active or potentially active faults are not known to pass directly through the site. The Alquist-Priolo (AP) mapped Route 247 fault zone is located approximately 1.3 miles to the west but does not trend towards the Site. The Alquist-Priolo (AP) mapped Northern Centinela fault zone is located approximately 3.3 miles to the south and trends towards the Site. It should be noted that ground surface rupture due to a seismic event may occur in areas where no evidence of ground rupture had been previously noted. However, based on the distance to the mapped trace of the faults and the distance to other faults in the vicinity of the site, the potential for damage due to surface rupture due to faulting at the project site is considered low.

8.2 SEISMIC SHAKING

The project site is located in southern California, which is considered a seismically active area, and as such, the seismic hazard most likely to impact the site is ground shaking resulting from an earthquake along one of the known active faults in the region. The seismic design of the project may be performed using seismic design recommendations in accordance with the 2016 California Building Code (CBC).

Preliminary seismic parameters were developed for the project site based on the 2016 California Building Code (CBC) and ASCE 7-10 guidance document. Using the USGS Ground Motion Parameter Online Calculator (https://earthquake.usgs.gov/designmaps/us/application.php) based on the following site coordinates: Latitude = 32.729506 degrees, and Longitude = -115.715528 degrees. The earthquake hazard level of the Maximum Considered Earthquake (MCE) is defined in ASCE 7-10 as the ground motion having a probability of exceedance of 2 percent in 50 years. The preliminary seismic design parameters for the project site are presented in Table 3 below.

Table 3 - Recommended 2016 CBC Seismic Design Parameters

Design Parameter	Recommended Value	Reference
Seismic Use Group	III	CBC Table 1604.5
Site Class	D	ASCE 7-10 Section 11.4.2
Mapped Spectral Accelerations for short periods, S_S	1.50g	ASCE 7-10 Section 11.4.3
Mapped Spectral Accelerations for 1-sec period, S_1	0.60g	ASCE 7-10 Section 11.4.3
Short-Period Site Coefficient, F_a	1.0	ASCE 7-10 Section 11.4.3
Long-Period Site Coefficient, F_v	1.5	ASCE 7-10 Section 11.4.3
(1) MCE_R (5% damped) spectral response acceleration for short periods adjusted for site class, S_{MS}	1.50g	ASCE 7-10 Section 11.4.3
(1) MCE_R (5% damped) spectral response acceleration at 1-second period adjusted for site class, S_{M1}	0.90g	ASCE 7-10 Section 11.4.3
Design spectral response acceleration (5% damped) at short periods, S_{DS}	1.00g	ASCE 7-10 Section 11.4.3
Design spectral response acceleration (5% damped) at 1-second period, S_{D1}	0.60g	ASCE 7-10 Section 11.4.3
Seismic Design Category	D	ASCE 7-10 Section 11.6
(2) MCE_G Peak Ground Acceleration adjusted for site class effects, PGA_M	0.50g	ASCE 7-10 Section 11.8.3

(1) MCE_R = Risk-adjusted Maximum Considered Earthquake
(2) MCE_G = Geometric-mean Maximum Considered Earthquake

8.3 LIQUEFACTION AND SEISMICALLY-INDUCED SETTLEMENT

Liquefaction and dynamic settlement of soils can be caused by ground shaking during earthquakes. Dynamic settlement due to earthquake shaking can occur in both dry or unsaturated and saturated sands. Research and historical data indicate that loose, relatively clean granular soils are susceptible to liquefaction and dynamic settlement, whereas the stability of the majority of clayey silts, silty clays and clays is not adversely affected by ground shaking. Liquefaction is generally known to occur in saturated loose cohesionless soils at depths shallower than approximately 50 feet. The potential for liquefaction under the same conditions of ground shaking intensity and duration will decrease for sands that are more well-graded, irregular, gritty, coarser and denser. Also, a pronounced decrease in liquefaction potential will occur with the increase in fine-grained (i.e., silt and clay) content and plasticity of the soil. Idriss and Boulanger (2008) have suggested that soils with plasticity index of greater than 7 may be considered non-liquefiable.

The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures (including pipelines), increased lateral earth pressures on retaining walls, and lateral spreading.

The project site is underlain by poorly to moderately consolidated alluvial materials. The subsurface exploration program encountered poorly to moderately consolidated alluvial silt, clay and silty sand, along with a relatively shallow ground water table. A simplified liquefaction analysis was performed using the liquefaction triggering analysis procedure proposed by Boulanger and Idriss (2014) and the CGS SP-117 procedures using the Standard Penetration Test (SPT) data from borings B-1/B-1A and B-6, and historical high groundwater level of 5 feet below ground surface. A peak ground acceleration (PGA) of 0.5g for geometric-mean MCE (see Table 2) and earthquake moment magnitude of 6.5 based on the results of deaggregation analysis using the USGS online tools were used in liquefaction analysis. The analysis results are presented in *Appendix D, Liquefaction Analysis Results* and summarized in the following paragraphs. The analyses indicate that minor liquefaction effects are expected at the site due to presence of few isolated saturated medium dense sand layers present between depths of 15 and 50 feet bgs. Secondary effects of liquefaction, including seismic settlement and lateral spreading are discussed below.

- <u>Seismic Settlement</u>: Seismically-induced ground settlement can occur with or without liquefaction which results from densification of loose soils as a result of strong seismic ground shaking. Seismic settlement includes both settlement of liquefied soil layers and settlement of non-liquefied, unsaturated, loose sandy sediments. The methods by Ishihara and Yoshimine (1992) to were used estimate liquefaction-induced seismic settlement and Pradel (1998) to estimate dry or unsaturated seismic settlement. The analyses indicate that the site is not susceptible to liquefaction. However, the total seismic settlement expected at the site is on the order of ¼-inch.

- <u>Lateral Spreading</u>: Seismically-induced lateral spreading involves primarily lateral movement of earth materials due to ground shaking in conjunction with liquefaction. Lateral spreading can manifest as near-vertical cracks with predominantly horizontal movement of the soil mass involved towards an adjacent open slope face. Lateral spreading occurs when there is widespread liquefaction and a gentle slope, or a free face toward which lateral spreading may occur. The potential for lateral spreading in the area adjacent to the canal free face was analyzed using data from boring B-1/B-1A and the method proposed by Zhang et al. (2004).

The results indicate low potential for lateral spreading due to absence of widespread liquefaction and relatively shallow depth of the canal compared to the depth of liquefiable soil layers.

8.4 LANDSLIDES AND SLOPE INSTABILITY

There are no high or steep natural slopes on or in close proximity to the project site. Based on the investigation, there appears to be no indications of landslides or deep-seated instability at the site. It is NV5's opinion that the potential damage to the planned facilities due to landsliding or slope instability is considered low.

8.5 SUBSIDENCE

The Imperial Valley is a region generally known for historic ground subsidence. The subsidence has been attributed to regional geologic processes and to fluid withdrawal associated with geothermal production. Most of the subsidence is tectonic in nature and the broad Salton Trough basin has been subsiding for at least the past 35 million years. Historic soil subsidence due to groundwater withdrawal associated with geothermal production has also been documented. The subsidence occurs when groundwater (near the surface or in a deep aquifer) is lowered past its historical level. This occurrence results in an increase of effective stress within a soil layer which typically translates into additional soil consolidation. Due to the depth of the reservoir, subsidence is not localized. Considering the distance to the geothermal production areas to the project site, and that ground subsidence in the Imperial Valley is occurring on a regional and not local level ground subsidence at the site is not expected to create significant differential settlement conditions. Therefore, potential for damaging localized differential settlement from fluid withdrawal subsidence is considered low.

8.6 TSUNAMIS, INUNDATION SEICHES, AND FLOODING

The site and surrounding areas are at an approximate elevation of 3 to 21 feet below mean sea level, the site is approximately 92 miles from the Gulf of California. Therefore, tsunamis (seismic sea waves) are not considered a hazard at the site.

The site is not located near to or downslope of, any large body of water that could affect the site in the event of an earthquake-induced failure or seiche (oscillation in a body of water due to earthquake shaking). The Salton Sea is located approximately 25 miles to the north of the site; therefore, seiches are not considered a hazard at the site.

8.7 EXPANSIVE SOILS

Improvements including foundations and slabs in contact with earth materials with a high potential for expansion can be expected to be subject to distress based on the potential for volume change associated with highly expansive soil. Soils such as these should not be relied upon for foundation bearing.

The project site is underlain predominantly by poorly to moderately consolidated alluvial materials consisting of sandy silt to clay, silty sand and poorly-graded sand with silts. Three tested samples of the near-surface silt and clay soils indicate medium to high expansion potential with an Expansion Index (EI) of 54 to 106. These materials are generally considered unsuitable for use as backfill for structure foundations, retaining walls or pipe bedding. Since site grading will redistribute on-site soils, potential expansive soil properties should be verified at the completion of rough grading.

9.0 CONCLUSIONS AND DESIGN RECOMMENDATIONS

9.1 GENERAL

Based on the available geologic data, known active or potentially active faults with the potential for surface fault rupture are not known to exist beneath the site. Accordingly, the potential for surface rupture at the site due to faulting is considered low during the design life of the proposed structure. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in southern California and the effects of ground shaking can be mitigated if the structure is designed and constructed in conformance with current building codes and engineering practices.

The near-surface soils in the upper 3 to 5 feet were found to be generally desiccated and considered moderately compressible. The near-surface soils have an expansion potential that ranges from medium to high. These soils are considered unsuitable for re-use as compacted fill and backfill. To provide a uniform support for the new structures and surface improvements, it is recommended that these materials be overexcavated and replaced with properly compacted, non-expansive granular fill.

Based on the results of field exploration, laboratory testing, and engineering evaluation and analyses, the proposed construction is considered geotechnically feasible, provided the recommendations contained herein are incorporated into the project plans and specifications and implemented during construction.

9.2 EARTHWORK AND GRADING

Site grading should be performed in accordance with the following recommendations and the *Typical Earthwork Guidelines* provided in *Appendix E*. In the event of conflict, the recommendations presented herein supersede those of *Appendix E*.

- <u>Clearing and Grubbing:</u> Prior to grading, the project area should be cleared of significant surface vegetation, demolition rubble, trash, pavement, debris, etc. Any buried organic debris or other unsuitable contaminated material encountered during subsequent excavation and grading work should also be removed. Removed material and debris should be properly disposed of offsite. Holes resulting from removal of buried obstruction which extend below finished site grades should be filled with properly compacted soils. Any utilities within the footprint of planned structural improvements should be appropriately abandoned.

- <u>Site Grading:</u> Areas to receive surface improvements or fill soils should be treated as follows:
  - <u>Removals Below Proposed New Structures:</u> To provide a uniform bearing condition below the new structures and surface improvements, the existing soils underlying the proposed structures should be completely excavated to a minimum depth of 3 feet below the bottom of foundations. The excavation should extend laterally a distance of at least 5 feet beyond the footprint of the proposed structure. The soils exposed in the bottom of the excavation should be moisture conditioned and uniformly recompacted to at least

90 percent of the soils maximum density (based on ASTM D1557). A cut-fill transition condition should not be allowed underlying proposed structures.

- Excavatability: Based on the subsurface exploration, it is anticipated that the on-site soils can be excavated by modern conventional heavy-duty excavating equipment in good operating condition.

- Structural Fill Placement: Areas to receive fill and/or surface improvements should be scarified to a minimum depth of 6 inches, brought to near-optimum moisture conditions, and compacted to at least 90 percent relative compaction, based on laboratory standard ASTM D1557. Fill soils should be brought to within 2 percent over optimum moisture content and compacted in uniform lifts to at least 90 percent relative compaction (ASTM D1557). Rocks with a maximum dimension greater than 4 inches should not be placed in the upper 3 feet of pad grade. The optimum lift thickness to produce a uniformly compacted fill will depend on the size and type of construction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in loose thickness. Placement and compaction of fill should be observed and tested by the geotechnical consultant.

- Graded Slopes: Graded slopes should be constructed at a gradient of 2:1 (H:V) or flatter. To reduce the potential for surface runoff over slope faces, cut slopes should be provided with brow ditches and berms should be constructed at the top of fill slopes.

- Paved Areas, Flatwork and Trash Enclosures: The soils in proposed paved areas, flatwork, and trash enclosures should be excavated to a minimum depth of one (1) foot below the proposed subgrade elevation, moisture conditioned, and uniformly recompact to at least 90 percent of the soils maximum dry density (based on ASTM D1557). This treatment should extend a horizontal distance of at least one (1) foot beyond the outside perimeter.

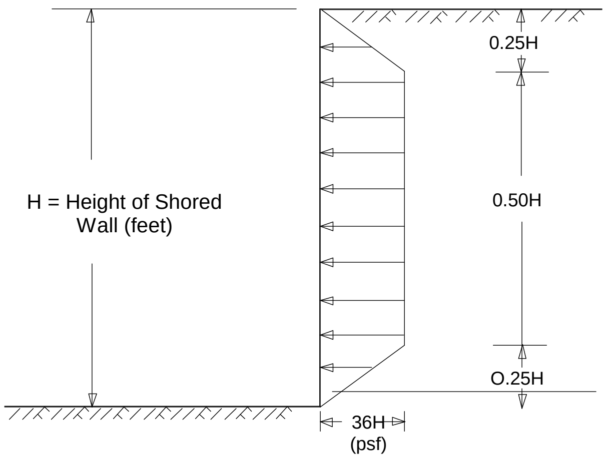
- Import Soils: Import soils should be sampled and tested for suitability by NV5 prior to delivery to the site. Imported fill materials should consist of clean granular soils free from vegetation, debris, or rocks larger than 3 inches in maximum dimension. The Expansion Index value should not exceed a maximum of 20 (i.e., essentially non-expansive).

9.3 TEMPORARY EXCAVATIONS AND SHORING

Temporary, shallow excavations with vertical side slopes less than 4 feet high will generally be stable, although there is a potential for localized sloughing. In these soil types, vertical excavations greater than 4 feet high should not be attempted without proper shoring to prevent local instabilities. Stockpiled (excavated) materials should be placed no closer to the edge of a trench excavation than a distance defined by a line drawn upward from the bottom of the trench at an inclination of 1H:1V, but no closer than 4 feet. All trench excavations should be in accordance with Cal-OSHA regulations. For planning purposes, the native soil materials may be considered as Type B, as defined in the current Cal-OSHA soil classification.

Although not anticipated, in the event of possible applicability, temporary shoring may be accomplished by several methods including: hydraulic shores and trench plates; trench boxes; And soldier piles and lagging. For vertical excavations less than about 15 feet in height, cantilevered shoring may be used. Cantilevered shoring may also be used for deeper excavations; however, the total deflection at the top of the wall should not exceed one-inch. Therefore, shoring of excavations deeper than about 15 feet may need to be accomplished with the aid of tied back earth anchors. The excavation support system should be designed to resist lateral earth pressures of the soil and hydrostatic pressures. Preliminary design of cantilevered temporary shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the subgrade soils, with a level surface behind the cantilevered shoring, will exert an equivalent fluid pressure of 37 pcf.

Tied-back or braced shoring should be designed to resist a trapezoidal distribution of lateral earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in the following diagram with the maximum pressure equal to 36H in psf, where H is the height of the shored wall in feet.



Any surcharge (live, including traffic, or dead load) located within a 1H:1V plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures. The vertical loads imposed by existing structures, if any, should be determined by the structural engineer. The lateral load contribution of a uniform surcharge load located across the 1:1 (H:V) zone behind the excavation may be calculated in accordance with *Figure 5, Lateral Surcharge Loads*. Lateral load contributions of surcharges located at a distance behind the shored wall should be provided by NV5 once the load configurations and layouts are known. As a minimum, a 2-ft equivalent soil surcharge is recommended to account for nominal construction loads.

The actual shoring design should be provided by a registered civil engineer in the State of California experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and the design should be reviewed by NV5 for conformance with the design intent and geotechnical recommendations. The shoring system should further satisfy requirements of Cal-OSHA.

9.4 DEWATERING

Groundwater was encountered at depths between approximately 9 to 19.1 feet below the existing ground surface. The groundwater table is subject to fluctuations in response to a number of factors. If necessary, the actual means and methods of any dewatering scheme should be established by a contractor with local experience. It is important to note that temporary dewatering, if necessary, will require a permit and plan that complies with RWQCB regulations. If excessive water is encountered, NV5 should be contacted to provide additional recommendations for temporary construction dewatering. Any cases of localized seepage or heavy precipitation should be monitored during construction. Based on the subsurface exploration the onsite soils maybe considered to be relatively permeable.

9.5 TRENCH BOTTOM STABILITY

The bottom of onsite excavations will likely expose poorly to moderately consolidated alluvial silt, lean clay, silty sand and poorly-graded sand. As long as excavations do not extend below the water table, these soils should provide a suitable base for construction of pipelines. For the design of flexible conduits, a modulus of soil reaction (E'), of 400 pounds per square inch is recommended. If these soils become wet or saturated they may be prone to settlement due to construction activities such as placement and compaction of backfill soils. Buried improvements underlain by these soils could also be damaged or subjected to unacceptable settlement due to subsidence of these soils. If wet or unusually soft conditions are encountered in the trench bottom, the bottom of the excavations will need to be stabilized. A typical stabilization method includes overexcavation of the soft or saturated soil and replacement with properly compacted fill, gravel or lean concrete to form a "mat" or stable working surface in the bottom of the excavation. There are other acceptable methods that can be implemented to mitigate the presence of compressible soils or unstable trench bottom conditions, and specific recommendations for a particular alternative can be discussed based on the actual construction techniques and conditions encountered.

9.6 BACKFILL PLACEMENT AND COMPACTION

The majority of the on-site soils should generally be suitable for use as trench backfill material if free of deleterious materials and brought to near-optimum moisture conditions (either by wetting or drying as-necessary). Trench backfill should be compacted in uniform lifts (not exceeding 6 inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (ASTM D1557). There should be sufficient clearance along the side of the utility pipe or line to allow for compaction equipment. The pipe bedding shall be compacted under the haunches and alongside the pipe.

Imported backfill should consist of granular, non-expansive soil with an Expansion Index (EI) of 20 or less and should not contain any contaminated soil, expansive soil, debris, organic matter, or other deleterious materials. The Sand Equivalent (SE) of the imported material shall be 20 or greater. Import material should be evaluated for suitability by the geotechnical consultant prior to transport to the site.

The upper 12 inches of subgrade soil and all rock base should be compacted to at least 95 percent. The moisture content of the backfill should be maintained within 2 percent of optimum moisture content during compaction. All backfill should be mechanically compacted. Flooding or jetting is not recommended and should not be allowed.

9.7 BUILDING AND SUBSTATION FOUNDATIONS

Foundations for proposed building and substation structures should be founded entirely on at least 3 feet of compacted essentially non-expansive granular fill prepared in accordance with Section 8.2. Recommendations for the design and construction of foundation system are presented below.

9.7.1 Design Parameters

Recommended shallow foundation design parameters are presented in Table 4. Footings should be designed and reinforced in accordance with the recommendations of the structural engineer and should conform to the latest edition of the California Building Code.

Table 4
Geotechnical Design Parameters For Shallow Foundations

Foundation Dimensions	Continuous or spread foundations at least 12 inches in width and embedded at least 18 inches below the lowest adjacent grade. Concrete mat slabs with a minimum thickness of 12 inches should be founded a minimum of 24 inches below the lowest adjacent grade.
Allowable Bearing Capacity (dead-plus-live load)	2,000 pounds per square foot (psf), which may be increased by 300 psf for each additional foot of width and by 100 psf for each additional foot of depth to a maximum of 4,000 psf. This assumes that foundations are founded on at least 3 feet of essentially non-expansive granular fill. A one-third (1/3) increase is allowed for wind or seismic loads.
Reinforcement	Reinforce in accordance with requirements as provided by the project Structural Engineer.
Allowable Coefficient of Friction	0.30 0.10 in the event a vapor barrier is used.
Allowable Lateral Passive Resistance (Equivalent Fluid Pressure)	250 pounds per cubic foot (pcf) per foot of depth. A one-third (1/3) increase in passive resistance value may be used for wind and seismic loads. The total allowable lateral resistance may be taken as the sum of the frictional resistance and the passive resistance, provided that the passive bearing resistance does not exceed one-half (1/2) of the total allowable lateral passive resistance.

Note: The above parameters assume level ground or sloping no steeper than 5H:1V.

9.7.2 Settlement

Estimated settlements will depend on the foundation size and depth, and the loads imposed and the allowable bearing values used for design. For preliminary design purposes, the total static settlement for foundations loaded to accordance with the allowable bearing capacities recommended above is estimated to be less than 1 inch. Differential static settlements are anticipated to be 0.5 inch or less.

9.7.3 Foundation Observation

To verify the presence of satisfactory materials at design elevations, footing excavations should be observed to be clean of loosened soil and debris before placing steel or concrete and probed for soft areas. If soft or loose soils or unsatisfactory materials are encountered, these materials should be removed and may be replaced with a two-sack, sand-cement slurry or structural concrete. Footing excavations should be deepened as necessary to extend into satisfactory bearing materials; however, NV5 should be notified to approve the proposed change.

9.7.4 Interior Concrete Slabs-on-Grade

Interior concrete slabs-on-grade may be supported at grade on compacted fill with very low to low expansion potential. For design of these concrete slabs, a modulus of subgrade reaction (k) of 150 pci may be used. Floor slabs should be designed and reinforced in accordance with the structural engineer's recommendations. NV5 recommends that interior floor slabs be at least 4 inches thick with a water cement ratio of 0.50 or less. Near surface groundwater is not expected and groundwater is not anticipated to adversely impact the structural performance of the floor slabs. However, in areas where slabs will be covered with moisture-sensitive flooring, it is common practice to place a capillary break consisting of at least 4 inches of free draining crushed gravel on the finished subgrade soil that, in turn, is overlain by a flexible sheet membrane, such as Stego Wrap™, Moistop Plus™, or an equivalent meeting the requirements of ASTM E1745-09, that serves as a water and/or moisture vapor retarder. The crushed gravel should be graded so that 100 percent passes the 1-inch sieve and less than 5 percent passes the No. 4 sieve. Care should be taken to properly place, lap, and seal the membrane in accordance with the manufacturer's recommendations to provide a vapor tight barrier. Tears and punctures in the membrane should be completely repaired prior to placement of concrete. The upper 12 inches of subgrade soil located below the vapor retarder should be moisture-conditioned within 2 percent over the optimum moisture content, and compacted to a minimum of 90 percent relative compaction (ASTM D1557).

At a minimum, slabs should be reinforced with No. 4 reinforcing bars spaced at 18 inches on-center, each way, placed in the middle one-third of the section, to help control shrinkage cracking of concrete. Reinforcement should be properly placed and supported on "chairs". Welded wire mesh is not recommended. The concrete reinforcement and joint spacing should conform to the minimum requirements of the American Concrete Institute (ACI) section 302.1R and established by the project structural engineer.

9.7.5 Exterior Concrete Slabs-on-Grade

Exterior concrete flatwork should have a minimum concrete thickness of 4 inches. Concrete slabs should be supported on at least 4 inches of Class 2 aggregate base compacted to at least 95 percent of the maximum dry density. The upper 12 inches of subgrade soil located below the aggregate base should be moisture-conditioned within 2 percent over the optimum moisture content, and recompacted to a minimum of 90 percent relative compaction (ASTM D1557).

The driveway slab areas and connecting sidewalks should have a minimum concrete thickness of 6 inches. The driveway concrete slab should be underlain by at least 6 inches of Class 2 aggregate base compacted to at least 95 percent of the maximum dry density. The upper 12 inches of subgrade soil located below the aggregate base should be reconditioned to achieve a moisture content within 2 percent over the optimum moisture content, and recompacted to a minimum of 95 percent relative compaction (ASTM D1557).

For exterior concrete flatwork, it is recommended that narrow strip concrete slabs, such as sidewalks, be reinforced with at least No. 3 reinforcing bars placed longitudinally at 36 inches on-center. Wide exterior slabs should be reinforced with at least No. 3 reinforcing bars placed 36 inches on-center, each way. The reinforcement should be extended through the control joints to reduce the potential for differential movement. Control joints should be constructed in accordance with recommendations from the structural engineer or architect.

9.8 SOLAR ARRAY FOUNDATIONS

Solar array panels and attached devices may be supported on short driven steel posts or drilled concrete piers. Preliminary design parameters and recommendations for solar array foundations provided in the following sections.

9.8.1 Driven Steel Posts

Preliminary axial and lateral pile capacities of W6x9 and W6x15 driven steel posts embedded at depths of 6, 8 and 10 feet below ground surface are presented in Table 5. Due to corrosive nature of native soils, special provisions for corrosion protection of the steel posts will be required.

Table 5 – Preliminary Axial and Lateral Capacities of Driven Steel Posts

Parameter	W6x9 Driven Steel Post			W6x15 Driven Steel Post		
Specified Embedment Depth (ft)	6	8	10	6	8	10
Height Above Ground (ft)	4	4	4	4	4	4
Total Length (ft)	10	12	14	10	12	14
Allowable Axial Capacity (kips) for Factor of Safety, FS = 2.5	4.0	5.3	6.6	4.9	6.5	8.1
Allowable Uplift Capacity (kips) for Factor of Safety, FS = 2.5	2.8	3.8	4.7	3.4	4.6	5.7
Lateral Capacity for ½-inch Free-Head Deflection (kips)	1.4	1.4	1.4	2.2	2.2	2.2
Maximum Bending Moment (ft-kips)29.1	6.6	6.6	6.6	10.3	10.3	10.3
Depth to Maximum Bending Moment from Top of Post (ft)	5.0	5.0	5.0	5.15	5.15	5.15

9.8.2 Drilled Concrete Piers

Equation 18-1 in Section 1807.3.2.1 of the 2016 California Building Code provides the formula for minimum embedment depth of a drilled concrete post required to resist lateral loads where no lateral constraint is present at or above the ground surface. The formula for the minimum embedment depth is as follows:

$$d = 0.5 A \{1 + [1 + (4.36 h/A)]^{1/2}\}$$

where:

d = Embedment depth in *feet* but not over 12 feet for purpose of computing lateral pressure.

$A = 2.34 P/(S_1 b)$

P = Applied lateral force in *pounds.*

S_1 = Allowable lateral soil bearing pressure as given in Section1806.2 based on a depth of one-third the depth of embedment in *pounds per square foot (psf)*.

b = Diameter of concrete pier in *feet*.

h = Vertical distance in *feet* from ground surface to point of application of “P”.

The short pier foundation may be designed using an allowable soil bearing pressure of 2,000 psf when embedded in the native soils.

9.9 RETAINING WALLS

Retaining walls should be designed in accordance with the following recommendations and design parameters presented herein.

- Bearing Capacity - The proposed wall may be supported on continuous footings bearing on dense natural soils or properly compacted fill soils at a minimum depth of 18 inches beneath the lowest adjacent grade. At this depth, footings may be designed for an allowable soil-bearing pressure of 2,000 psf. This value may be increased by one-third for loads of short duration, such as wind or seismic forces.

- Lateral Earth Pressures - Based on laboratory test results and encountered soil conditions, the recommended lateral earth pressures for preliminary design of flexible retaining walls supported on shallow foundations are summarized in the following Table 6.

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Table 6 - Recommended Lateral Earth Pressures

Parameter	Recommended Values				
	Level Backfill	5H:1V Slope	4H:1V Slope	3H:1V Slope	2H:1V Slope
Static Active Earth Pressure (P_a)	37H	43H	45H	49H	62H
Static At-Rest Earth Pressure (P_o)	60H	72H	75H	79H	87H
Seismic Earth Pressure (P_e)	23H	26H	27H	30H	38H
Coefficient of Friction (µ) for Lateral Resistance of Footing	0.35	N/A	N/A	N/A	N/A
Passive Earth Pressure (P_p) for Lateral Resistance of Footing	250H	N/A	N/A	N/A	N/A

Notes:

1. All values of height (H) are in feet (ft) and pressure (P) in pounds per square feet (psf).
2. Seismic earth pressure (P_e) is in addition to the static active or at-rest pressure, P_a and P_o which should be distributed as an inverted triangle along the wall height and the resultant of this pressure is an increment of force which should be applied to the back of the wall in the upper one-third (1/3) of the wall height and may also be applied as a reduction of force to the front of the wall in the upper one-third (1/3) of the footing depth.
3. The above pressure values do not include hydrostatic pressures that might be caused by groundwater or water trapped behind the structure.
4. The pressures listed in the table were based on the assumption that backfill soils will be compacted to 90 percent of maximum dry density (per ASTM D1557).
5. The coefficient of friction (µ) should be applied to dead normal (buoyant) loads when evaluating the sliding frictional resistance.
6. A resistance factor of 0.5 has been applied to the passive earth pressure and may be combined with the sliding frictional resistance using a resistance factor of 0.80. Neglect the upper 6 inches for passive pressure unless the surface is contained by a pavement or a slab. The passive earth pressure should not exceed a maximum value of 3,000 psf.
7. In addition to the above-mentioned pressures, retaining walls must be designed to resist horizontal pressures that may be generated by surcharge loads applied at the ground surface such as from uniform loads or vehicle loads. Figure 5 may be used to evaluate these surcharge loads.

- Drainage and Waterproofing - Retaining walls should be properly drained, and if desired, appropriately waterproofed. Adequate backfill drainage is essential to provide a free-drained backfill condition and to reduce the potential for the development of hydrostatic pressure buildup behind walls. Drainage behind the retaining walls may be provided with geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, placed continuously along the back of the wall and connected to a 4-inch-diameter perforated pipe. The pipe should be sloped at least 2 percent and surrounded by 3 cubic feet per foot of ¾-inch crushed rock wrapped in suitable non-woven filter fabric (Mirafi 140N or equivalent) or Caltrans Class 2 permeable granular filter materials without filter fabric. The crushed rock should meet the requirements defined in Section 200-1.2 of the latest edition of the Standard Specification for

Public Works Construction (Greenbook). These drains should be connected to an adequate discharge system.

In lieu of a perforated drainage pipe and connection to an existing drainage system, weep holes or open vertical masonry joints may be provided in the lowest row of block exposed to the air to reduce the buildup of hydrostatic pressure behind the wall. Weep holes should be a minimum of three inches in diameter and provided at intervals of at least every six feet along the wall. Open vertical masonry joints should be provided at a minimum of 32-inch intervals. A continuous gravel fill, a minimum of one cubic foot per foot should be placed behind the weep holes or open masonry joints. The gravel should be wrapped in filter fabric (Mirafi 140N or equivalent). To prevent efflorescence at the face of the wall, the wall may also be appropriately waterproofed. Waterproofing treatments and alternative, suitable wall drainage products are available commercially. Design of waterproofing and its protection during construction should be addressed by the project design professional.

- Retaining Wall Backfill Compaction - Retaining wall backfill material should be non-expansive (E.I. of 20 or less) and free draining. Backfill should be brought to near-optimum moisture conditions and compacted by mechanical means to at least 90 percent relative compaction (ASTM D1557). Care should be taken when using compaction equipment in close proximity to retaining walls so that the walls are not damaged by excessive loading.

9.10 PAVEMENTS

Design of asphalt concrete pavement sections depends primarily on support characteristics (strength) of soil beneath the pavement section and on cumulative traffic loads within the service life of the pavement. Strength of the pavement subgrade is represented by R-value test data. R-value tests were performed on representative samples of the near-surface soil. The results yielded R-values ranging from 5 (lean clay) and 57 (silty sand). A summary of the test is included in Appendix C.

Traffic loads within service life of a pavement are represented by a Traffic Index (TI), which is calculated based on anticipated traffic loads and on the projected number of load repetitions during the design life of the pavement. The design TI value should be verified by the project Civil/Traffic Engineer prior to construction.

Preliminary pavement section recommendations were developed using a design R-value of 5 and maximum Traffic Index (TI) = 6 assumed for light auto parking and drive lanes and TI = 8 for fire lanes. Based on these design parameters, analysis in accordance with California Department of Transportation (Caltrans) Highway Design Manual, and assuming compliance with site preparation recommendations, NV5 recommends the flexible and rigid structural pavement sections presented in the following Table 7.

Table 7 - Recommended Pavement Sections (Design R-value = 5)

Location	Alternative 1		Alternative 2	
	Hot-Mix Asphalt (HMA)	Aggregate Base (AB)	Jointed Plain Portland Cement Concrete (JPCP)	Aggregate Base (AB)
Light Auto Parking and Drive Lanes	4.0	12.0	5.0	4.0
Fire Lanes	8.0	12.0	6.0	4.0

Assuming that the near-surface on-site soils will be thoroughly mixed and compacted during grading operations, it is recommended that R-value testing be performed on representative soil samples after rough grading operations on the upper 2 feet to confirm applicability of the above pavement sections. If the paved areas are to be used during construction, or if the type and frequency of traffic is greater than assumed in the design, the pavement section should be re-evaluated for the anticipated traffic.

The upper 12 inches of subgrade soils should be compacted to a minimum dry density of 95 percent of the material's maximum dry density as determined by the ASTM D1557 test procedure. The aggregate base should conform to Class II aggregate base in accordance with Section 400.2.3 of the 2009 Regional Supplement to Greenbook Standard Specifications for Public Works Construction. The base course should also be compacted to a minimum dry density of 95 percent. Field and laboratory testing should be used to check compaction, aggregate gradation, and compacted thickness.

The asphalt pavement should be compacted to 95 percent of the unit weight as tested in accordance with the Hveem procedure (ASTM D1560). The maximum lift thickness should be 4.0 inches. The asphalt material shall conform to Type III, Class B2 or B3 of the Standard Specifications for Public Works Construction and the supplement. An approved mix design should be submitted 30 days prior to placement. The mix design should include proportions of materials, maximum density and required lay-down temperature range. Field and lab testing should be used to verify oil content, aggregate gradation, compaction, compacted thickness, and lay-down temperature.

Control joints are required for the Portland cement concrete pavement (rigid) at a maximum of 15 feet spacing each way and should be constructed immediately after concrete finishing.

The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of the pavement. The ponding of water on or adjacent to pavement areas will likely cause failure of the subgrade and resultant pavement distress. Where planters are proposed, the perimeter curb should extend at least 6 inches below the subgrade elevation of the adjacent pavement. In addition, experience indicates that even with these provisions, a saturated subgrade condition can develop as a result of increased irrigation, landscaping and surface runoff. A subdrain system should be considered along the perimeter of pavement subgrade areas to reduce the potential of this condition developing. The subdrain system should be designed to intercept irrigation water and surface runoff prior to entry into the pavement subgrade and carry the water to a suitable outlet.

9.11 SOIL CORROSION

The corrosion potential of the on-site materials to steel and buried concrete was evaluated. Laboratory testing was performed on selected soil samples to evaluate pH, minimum resistivity, and chloride and soluble sulfate content. Table 8 below, presents the results of the corrosivity testing.

Table 8 - Corrosivity Test Results

Test Location	Depth (feet)	Material Type	Percent Finer Than No. 200	pH	Minimum Resistivity (ohm-cm)	Water Soluble Sulfate Content (ppm)	Water Soluble Chloride Content (ppm)
B-3	3 - 5	Silty Sand	40.4	9.3	820	420	130
B-6	1 - 3	Fat Clay	Not Tested	8.5	120	2310	2140

General recommendations to address the corrosion potential of the on-site soils are provided below. If additional recommendations are desired, it is recommended that a corrosion specialist be consulted.

Caltrans Corrosion Guidelines dated March 2018 considers a site to be corrosive if one or more of the following conditions exist for the representative soil samples taken at the site:

> *Chloride concentration is 500 ppm or greater, sulfate concentration is 1500 ppm or greater, or the pH is 5.5 or less*

Based on experience and the Caltrans Corrosion Guidelines, some of the site soils are considered corrosive to steel and concrete foundation elements based on sulfate and chloride test results.

As indicated in the 2006 edition (second edition) of “Corrosion Basics - An Introduction”, a general guideline for soil resistivity and corrosion-severity ratings is presented in Table 9 below.

Table 9 - Corrosivity Test Results

Soil Resistivity	Corrosivity
<1,000 ohm-cm	Extremely Corrosive
1,000 to 3,000 ohm-cm	Highly Corrosive
3,000 to 5,000 ohm-cm	Corrosive
5,000 to 10,000 ohm-cm	Moderately Corrosive
10,000 to 20,000 ohm-cm	Mildly Corrosive
>20,000 ohm-cm	Essentially Noncorrosive

Soil resistivity is not the only parameter affecting the risk of corrosion damage; and a high soil resistivity will not guarantee the absence of serious corrosion. For example, the American Water Works Association (AWWA) has developed a numerical soil-corrosivity scale, applicable to cast-iron alloys. The soil resistivity test results suggest the potential for soils to be extremely corrosive to ferrous pipes.

Any imported soils should be evaluated for corrosion characteristics if they will be in contact with buried or at-grade structures and appropriate mitigation measures should be included in the structure design. It is recommended that a corrosion specialist be contacted to determine if mitigation measures are necessary.

10.0 DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many pipelines has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the backfill, subgrade and base will be important to the performance of the proposed improvements. The following sections present NV5's recommendations relative to the review of construction documents and the monitoring of construction activities.

10.1 PLANS AND SPECIFICATIONS

The design plans and specifications will be reviewed and approved by NV5 prior to construction, as the geotechnical recommendations may need to be re-evaluated in the light of the actual design configuration. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications.

10.2 CONSTRUCTION MONITORING

Site preparation, removal of unsuitable soils, assessment of imported fill materials, backfill placement, and other earthwork operations should be observed and tested. The substrata exposed during the construction may differ from that encountered in the test borings. Continuous observation by a representative of NV5 during construction allows for evaluation of the soil/rock conditions as they are encountered and allows the opportunity to recommend appropriate revisions where necessary.

11.0 LIMITATIONS

The recommendations and opinions expressed in this report are based on NV5's review of background documents and on information developed during this study. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site. More detailed limitations of this geotechnical study are presented in the GBC's information bulletin in *Appendix F*.

Due to the limited nature of the field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during the proposed structure construction operations.

Site conditions, including ground-water level, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which NV5 has no control.

NV5's recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill/backfill placement, etc. Accordingly, the recommendations are made contingent upon the opportunity for NV5 to observe grading operations and foundation excavations for the proposed construction. If parties other than NV5 are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. NV5 should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

NV5 has endeavored to perform this study using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil/rock conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this study.

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FIGURES

Reference: Google Earth 2018

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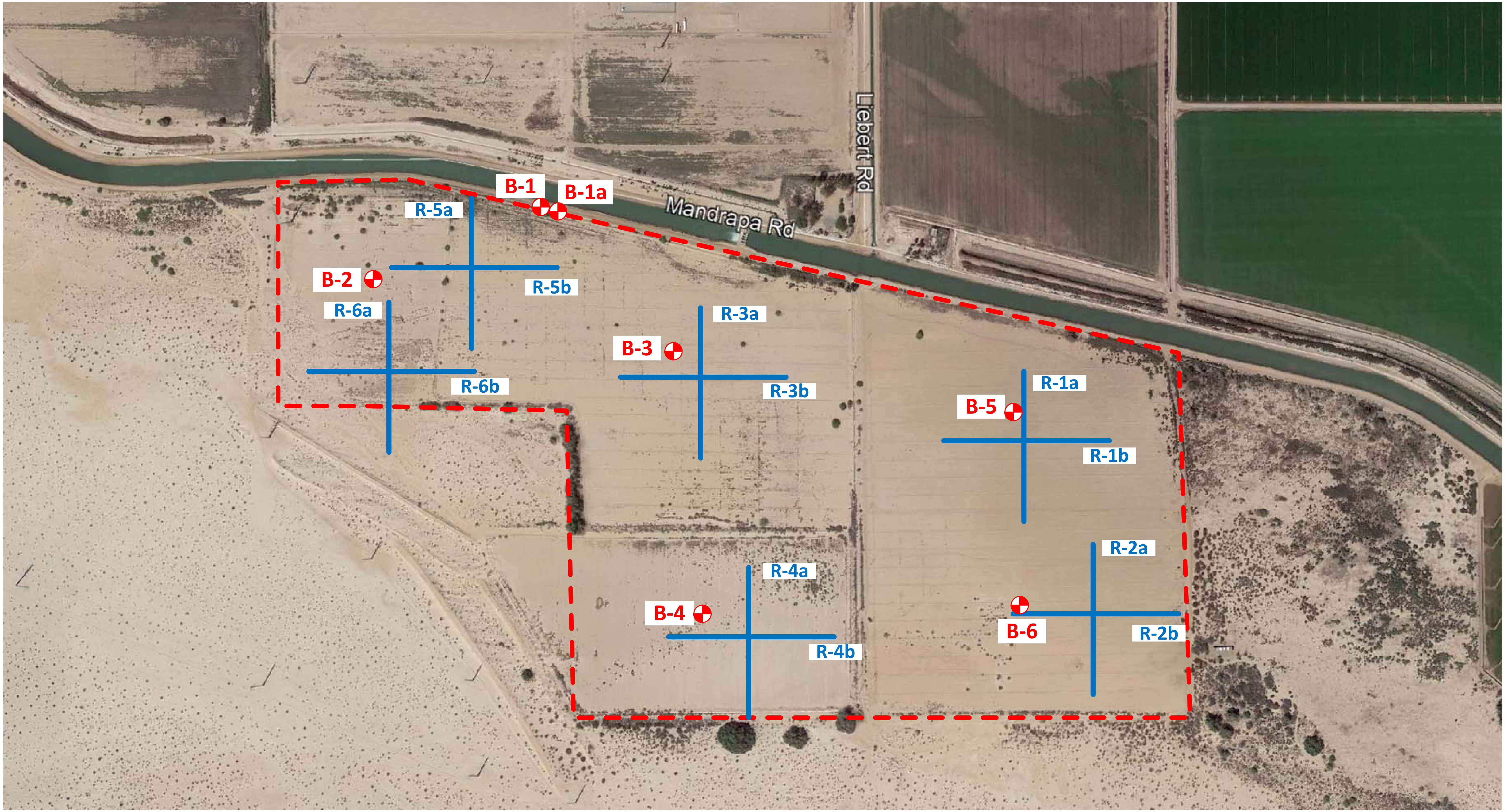
15092 Avenue of Science, Suite 200
San Diego, CA
Tel: (858) 385-0500, Fax: (858) 385-0400

Project No: **1076**
Drawn: **SB**
Date: **September 2018**

Site Location Map
Sempra Renewables
Westside Canal Energy Center
Imperial Valley, California

Figure No. 1



Legend:

B-6 Approximate locations of geotechnical borings

R-6b Approximate locations of field resistivity tests

Approximate limits of project site

N

0 700 ft 1400 ft

Reference: Google Earth 2018

Not a surveyed map

Not a construction drawing

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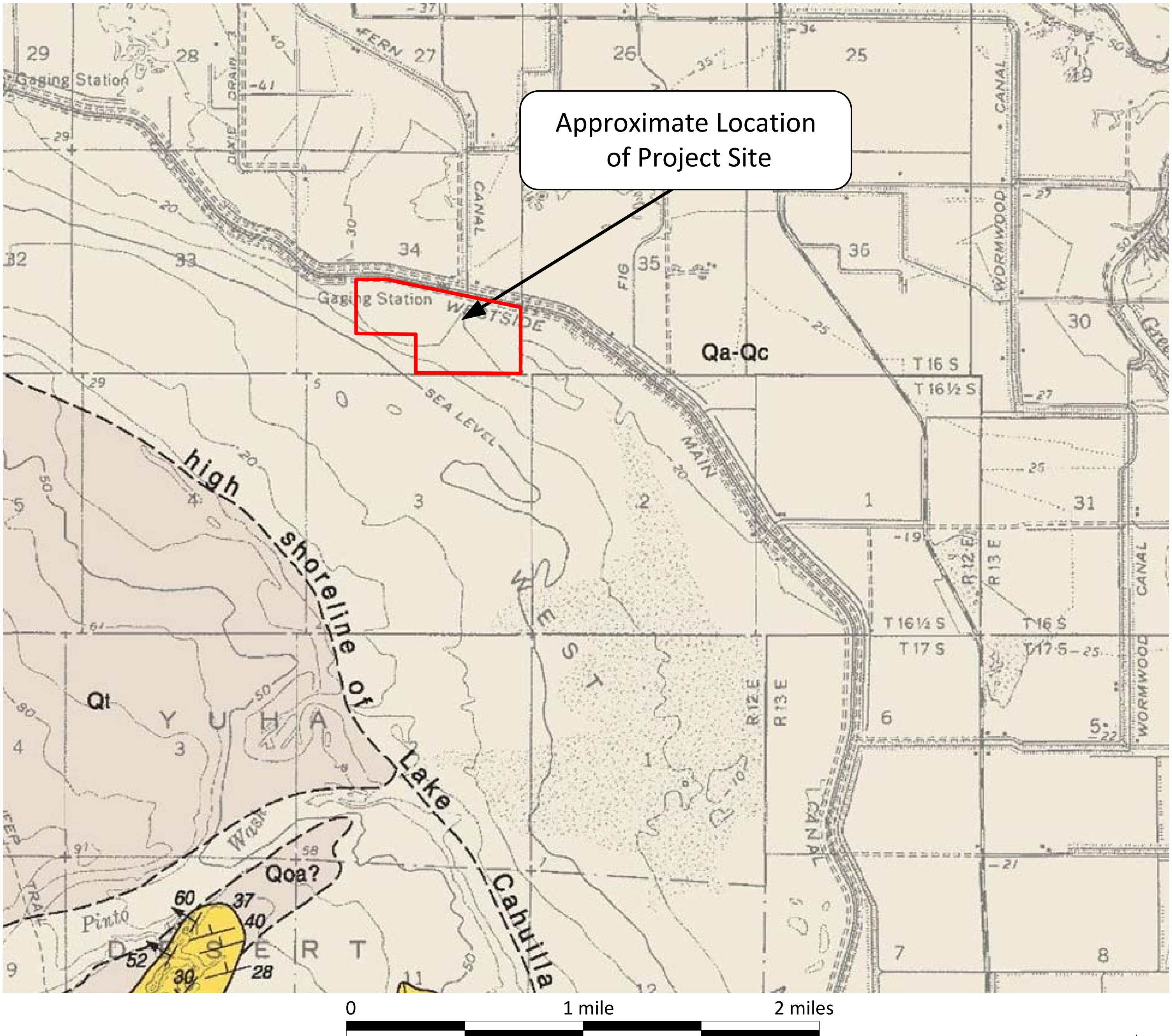
Field Exploration Plan

Sempra Renewables

Westside Canal Energy Center

Imperial Valley, California

Figure No. 2



DESCRIPTION OF MAP UNITS

Qa	Qa-Qc	Qc	Qt	Qt1 / Qt2

ALLUVIUM

Unconsolidated and undissected surficial sediments of valley fill and floodplains; age, Recent

Qa *Alluvial clay and silt, grading to sandy gravel near mountains*

Qc *Cahuilla Beds, thin series of tan-gray claystones, sands, and gravels deposited in former Lake Cahuilla, fossiliferous,* **Qa-Qc** *where locally undifferentiated from* **Qa** *or* **Qc**

Qt *Terrace deposits, includes* **Qt1**, *younger gravel and sand, locally undifferentiated from* **Qt**, *and* **Qt2**, *boulder to pebble gravel and sand, locally folded and faulted*

N

Not a surveyed map
Not a construction drawing

Reference: Dibblee, Jr., T.W., Division of Mines and Geology, *Geologic Map of The Coyote Wells & Heber 15 Minute Quadrangles, Imperial County, California, 2008. Map #DF-405, Scale 1:62,500.*

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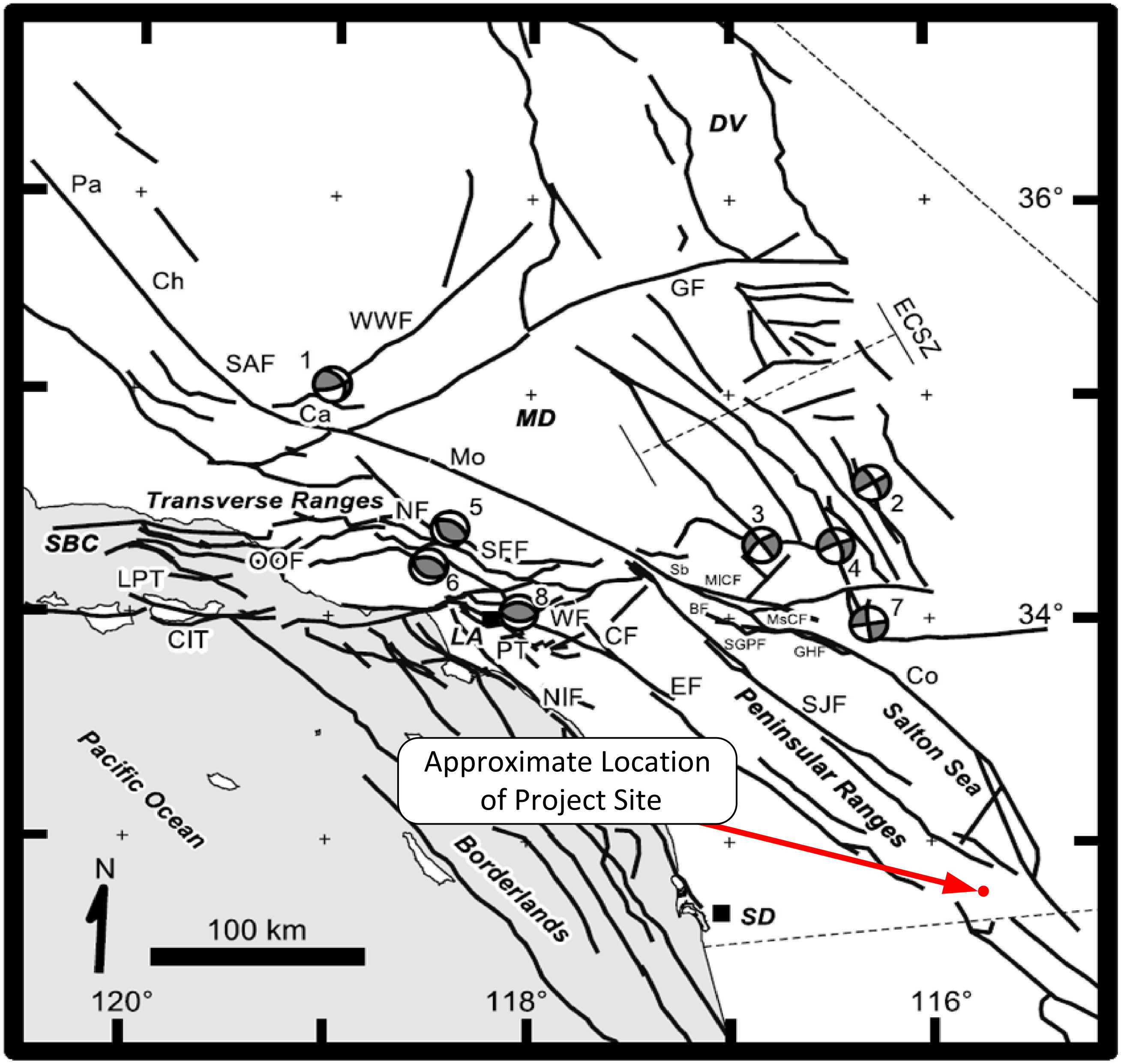
Regional Geologic Map
Sempra Renewables
Westside Canal Energy Center
Imperial Valley, California

Figure No. 3



Map of southern California showing the geographic regions, faults and focal mechanisms of the more significant earthquakes. **Regions:** Death Valley, DV; Mojave Desert MD; Los Angeles, LA; Santa Barbara Channel, SBC; and San Diego, SD. **Indicated Faults:** Banning fault, BF; Channel Island thrust, CIT; Chino fault, CF; Eastern California Shear Zone, ECSZ; Elsinore fault, EF; Garlock fault, GF; Garnet Hill fault, GHF; Lower Pitas Point thrust, LPT; Mill Creek fault, MICF; Mission Creek fault, MsCF; Northridge fault, NF; Newport Inglewood fault, NIF; offshore Oak Ridge fault, OOF; Puente Hills thrust, PT; San Andreas fault (sections: Parkfield, Pa; Cholame, Ch; Carrizo; Ca; Mojave, Mo; San Bernardino, Sb; and Coachella, Co); San Fernando fault, SFF; San Gorgonio Pass fault, SGPF; San Jacinto fault, SJF; Whittier fault, WF; and White Wolf fault, WWF. **Earthquake Focal Mechanisms:** 1952 Kern County, 1; 1999 Hector Mine, 2; 1992 Big Bear, 3; 1992 Landers, 4; 1971 San Fernando, 5; 1994 Northridge, 6; 1992 Joshua Tree, 7; and 1987 Whittier Narrows, 8.

N

Reference: Plesch, Anndreas et. al., 2007, Community Fault Model (CFM) for Southern California; in the *Bulletin of the Seismological Society of America*, Vol. 97, No. 6. pp. 1793-1802, dated December.

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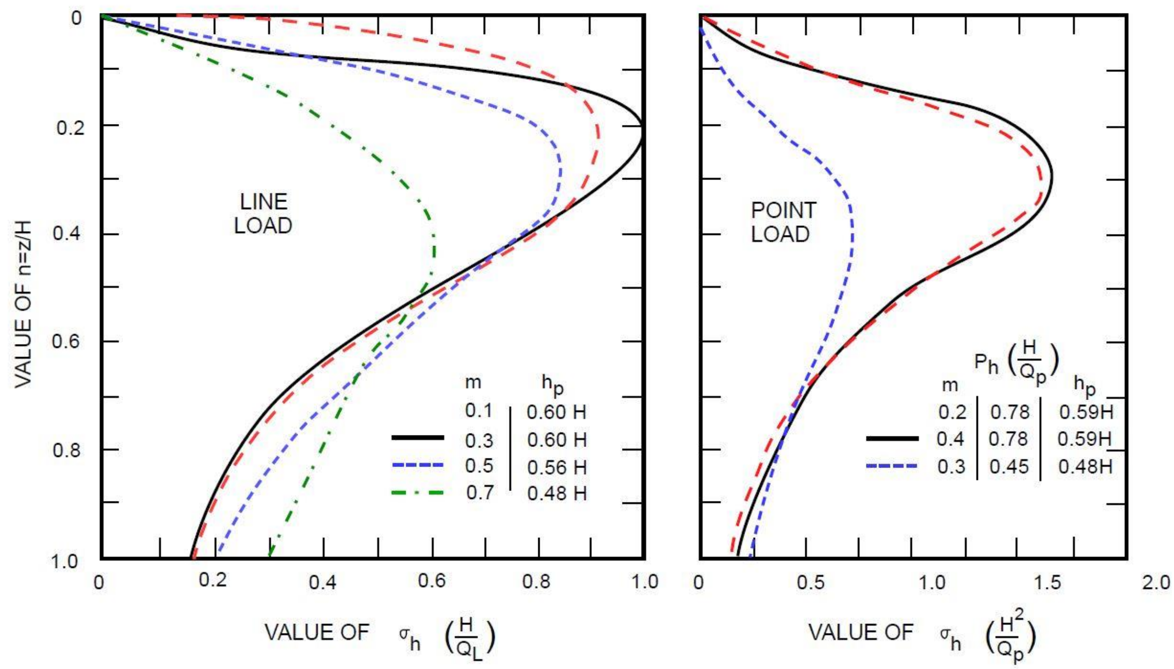
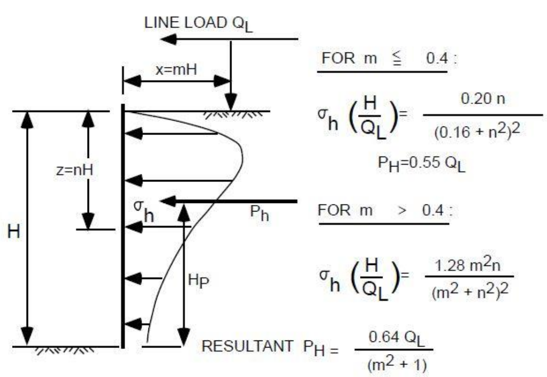
Drawn: **SB**
Date: **September 2018**

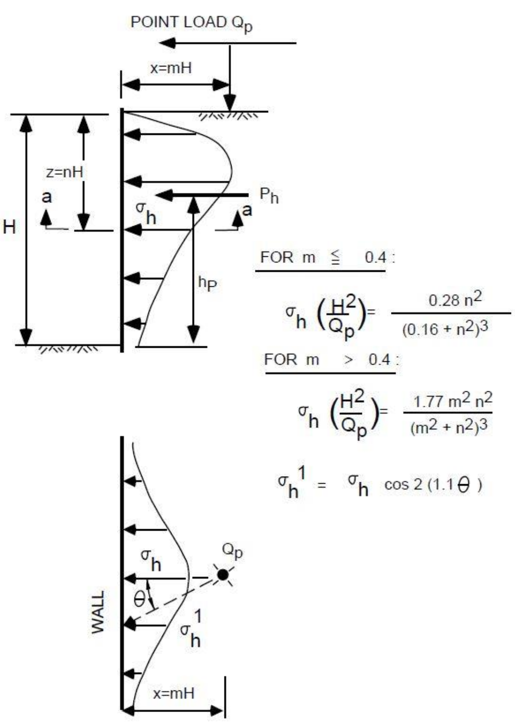
Regional Fault Map
Sempra Renewables
Westside Canal Energy Center
Imperial Valley, California

Figure No. 4





PRESSURE FROM LINE LOAD Q_L
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



SECTION a ~a
PRESSURE FROM POINT LOAD Q_p
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)

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Project No: **1076**
Drawn: **SB**
Date: **October 2018**

Lateral Surcharge Loads
Sempra Renewables
Westside Canal Energy Center
Imperial Valley, California

Figure No. 5

APPENDIX A

Exploratory Boring Logs

Logs of Exploratory Borings

Bulk and relatively undisturbed drive samples were obtained in the field during our subsurface evaluation. The samples were tagged in the field and transported to our laboratory for observation and testing. The drive samples were obtained using the Modified California Sampler (CAL) and Standard Penetration Test (SPT) samplers as described below.

Modified California Split Spoon Sampler

The split barrel drive sampler is driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1587. The number of blows per foot recorded during sampling is presented in the logs of exploratory borings. The sampler has external and internal diameters of approximately 3.0 and 2.4 inches, respectively, and the inside of the sampler is lined with 1-inch-long brass rings. The relatively undisturbed soil sample within the rings is removed, sealed, and transported to the laboratory for observation and testing.

Standard Penetration Test (SPT) Sampler

The split barrel sampler is driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1586. The number of blows per foot recorded during sampling is presented in the logs of exploratory borings. The sampler has external and internal diameters of 2.0 and 1.4 inches, respectively. The soil sample obtained in the interior of the barrel is measured, removed, sealed and transported to the laboratory for observation and testing.

SAMPLE/SAMPLER TYPE GRAPHICS

- AUGER SAMPLE
- STANDARD PENETRATION SPLIT SPOON SAMPLER
- BULK / GRAB SAMPLE
- MODIFIED CALIFORNIA SAMPLER
- SHELBY TUBE SAMPLER
- HQ ROCK CORE SAMPLE
- NQ ROCK CORE SAMPLE

GROUNDWATER LEVEL GRAPHICS

- WATER LEVEL (during drilling operations)
- WATER LEVEL (immediately after drilling completion)
- WATER LEVEL (additional levels after drilling completion)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an intergral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions my be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System (USCS) designationspresented on the logs were based on visual classification in thefield and were modified where appropriate based on gradation andindex property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5 and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then Y/X indicates Y number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of materials is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu≥4 and 1≤Cc≤3	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu<4 and/or 1>Cc>3	GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5 TO 12% FINES	Cu≥4 and 1≤Cc≤3	GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
			Cu<4 and/or 1>Cc>3	GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu<4 and/or 1>Cc>3	GP-GM	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH >12% FINES		GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu≥6 and 1≤Cc≤3	SW	WELL GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu<6 and/or 1>Cc>3	SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SAND WITH 5 TO 12% FINES	Cu≥6 and 1≤Cc≤3	SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu>6 and/or 1<Cc>3	SP-SM	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH >12% FINES		SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SITLY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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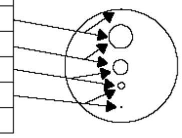
Project No: **1076**
Drawn: **SB**
Date: **September 2018**

Title: **Boring Log Legend**
Project: **Sempra Renewables Westside Canal Energy Center Imperial Valley, California**

Chart 1

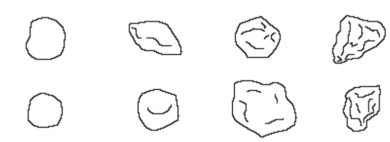
GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in.	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in.	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 - 3 in.	3/4 - 3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in.	0.19 - 0.75 in. (4.75 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.75 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.074 - 0.43 mm.)	Four-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (0.074 mm.)	Flour-sized and smaller



ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded edges
Rounded	Particles have smoothly curved sides and no edges


Rounded Subrounded Subangular Angular

MUNSELL COLOR

NAME	ABBR
Red	R
Yellow Red	YR
Yellow	Y
Green Yellow	GY
Green	G
Blue Green	BG
Blue	B
Purple Blue	PB
Purple	P
Red Purple	RP
Black	N

PLASTICITY

DESCRIPTION	CRITERIA
Non-plastic	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

MOISTURE CONTENT

DESCRIPTION	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below groundwater table

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	CRITERIA
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violet reaction, with bubbles forming immediately

APPARENT/RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (#blows/ft)	MODIFIED CALIFORNIA SAMPLER (#blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<5	0 - 15
Loose	4 - 10	6 - 15	15 - 35
Medium Dense	11 - 30	16 - 40	35 - 65
Dense	31 - 50	41 - 70	65 - 85
Very Dense	>50	>71	85 - 100

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT-N_{60} (#blows/0.3m)	CRITERIA
Very Soft	<2	Thumb will penetrate soil more than 1 in. (25 mm.)
Soft	2 - 4	Thumb will penetrate soil about 1 in. (25 mm.)
Medium Stiff	5 - 8	Thumb will indent soil about 1/4-in. (6 mm.)
Stiff	8 - 15	Can be imprinted with considerable thumbnail pres.
Very Stiff	15 - 30	Thumb will not indent soil but readily indented with thumbnail
Hard	>30	Thumbnail will not indent soil

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. (6 mm.) thick, note thickness
Laminated	Alternating layers of varying material or color with layers less than 1/4-in. (6 mm.) thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into smaller angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

CEMENTATION

DESCRIPTION	CRITERIA
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

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Project No: **1076**
Drawn: **SB**
Date: **September 2018**

Title: **Soil Classification**
Project: **Sempra Renewables**
Westside Canal Energy Center
Imperial Valley, California

NV5

Boring Log

Date		Project Number	Project	Boring No.
Started: 9/17/18	Completed: 9/18/18	1076	Westside Canal Energy Center	B-1
Hammer Efficiency: 93 %		Rig Type: Unimog M-5 (Pacific)	Surface Elevation: -21.0'	Logged By: S. Burford
Latitude: 32.731764		Longitude: -115.718873	Location: Near canal	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
9.5	8:40am	9/17/2018

Groundwater Depth (ft.)	Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0							[ALLUVIUM (Qa-Qc)] Lean CLAY with Sand (CL): Tan, dry to moist, stiff
		G- 1		20 1		Moisture Content Sieve Analysis Expansion Index		
	5	MC- 1	3 8 10	26 1	97 7	Moisture / Density	CL	Stiff, Pocket Penetrometer: 1.75 ton/ft^2
		G- 2						
▼ 9.5	10	SPT- 1	3 11 14	25 8		Moisture Content		Very stiff
	13.0'							El. -34.0'
								Sandy Lean CLAY (CL): moist, hard
		G- 2				Sieve Analysis		
	15	SPT- 2	6 13 23	22 1		Moisture Content	CL	
	18.0'							El. -39.0'
								Silty SAND (SM): Tan, moist, medium dense
	20	SPT- 3	2 5 8	21 8		Moisture Content	SM	
	21.5'							El. -42.5'

Notes: Drilled using 6" O.D. Hollow Stem Auger. Boring terminated at depth of (21.5'). Backfilled with neat cement. Groundwater measured at 9.5' bgs.

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Boring Log

Date: Started: 10/2/18; Completed: 10/2/18	Project Number 1076	Project Westside Canal Energy Center	Boring No. B-1a
Hammer Efficiency: 80 %	Rig Type: Diedrich D50 (Pacific)	Surface Elevation: -21.0'	Logged By: S. Burford
Latitude: 32.731760	Longitude: -115.718833	Location: Near canal	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
9	8:20am	10/2/2018

Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0									[ALLUVIUM (Qa-Qc)] Lean CLAY with Sand and Sandy Lean CLAY (CL): Tan, dry to moist
	5									Free drilled down to 15' BGS for first sample
▼									CL	
	10									
	15			SPT- 1	3 5 7	24 3		Moisture Content		Stiff
	18.0'									El. -39.0'
										Silty SAND (SM): Tan, moist
	20			SPT- 2	3 7 11	24 8		Moisture Content Sieve (20-26 5)		Medium dense. Switched from Hollow Stem to Mud-Rotary at 20' BGS
									SM	
	25			SPT- 3	16 17 20	22 5		Moisture Content		Dense
	30									30.0' El. -51.0'

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Boring Log

Date		Project Number	Project	Boring No.
	Started: 10/2/18	1076	Westside Canal Energy Center	B-1a
	Completed: 10/2/18			
	Hammer Efficiency: 80 %	Rig Type: Diedrich D50 (Pacific)	Surface Elevation: -21.0'	Logged By: S. Burford
Latitude: 32.731760		Longitude: -115.718833	Location: Near canal	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
9	8:20am	10/2/2018

Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	30			SPT- 4	15 21 23	22 1		Moisture Content Sieve (30-51 5)		Poorly-graded SAND with Silt (SP-SM): Tan, moist to wet, dense to very dense
	35			SPT- 5	13 16 22	22 7		Moisture Content		
	40			SPT- 6	13 19 28	22 4		Moisture Content		
	45			SPT- 7	18 33 50/5 5	21 4		Moisture Content	SP-SM	
	50			SPT- 8	16 21 25	22 4		Moisture Content		
	55			SPT- 9	20 33 50/6"	22 0		Moisture Content Sieve (55-76 5)		
	60									

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Boring Log

Date		Project Number	Project	Boring No.
	Started: 10/2/18	1076	Westside Canal Energy Center	B-1a
	Completed: 10/2/18			
	Hammer Efficiency: 80 %	Rig Type: Diedrich D50 (Pacific)	Surface Elevation: -21.0'	Logged By: S. Burford
Latitude: 32.731760		Longitude: -115.718833	Location: Near canal	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
9	8:20am	10/2/2018

Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	60			SPT- 10	13 20 26	23 1		Moisture Content	SP-SM	Poorly-graded SAND with Silt (SP-SM): Tan, moist to wet, dense to very dense
	65			SPT- 11	17 30 38	22 0		Moisture Content		
	70			SPT- 12	18 30 46	21 3		Moisture Content		Traces of gravel encountered from 72-75' BGS
	75			SPT- 13	22 32 39	21 2		Moisture Content		79.0' El. -100.0'
				SPT- 14	16 20 22				CL	Lean CLAY (CL): Brown, moist, hard 80.0' El. -101.0'
	80									

Notes: Drilled using 6" O.D. Hollow Stem Auger. Boring terminated at depth of (80.0'). Switched to Mud-Rotary drilling at 20' BGS. Backfilled with neat cement. Groundwater measured at 9.0' bgs.

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Boring Log

Date — Started: 9/17/18	Project Number 1076	Project: Westside Canal Energy Center	Boring No. B-2
Completed: 9/17/18			
Hammer Efficiency: 93 %	Rig Type: Unimog M-5 (Pacific)	Surface Elevation: -21.0'	Logged By: S. Burford
Latitude: 32.730861	Longitude: -115.721389	Location: Northwest corner	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
12	2:00pm	9/17/2018

Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
0–3.0						CL	[ALLUVIUM (Qa-Qc)] Sandy Lean CLAY (CL) to Clayey Sand (SC): Tan, dry to moist (3.0', El. -24.0')
3.0–8.0	G- 1				Expansion Index Thermal Resistivity	CH	Fat CLAY (CH): Brown, dry to moist, very stiff (8.0', El. -29.0')
	MC- 1	11 19 32	5 1	102 1	Moisture / Density		
8.0–15.0	G- 2					CL	Lean CLAY (CL): Brown, moist to wet, orange-brown laminations, thinly bedded, stiff (15.0', El. -36.0')
	SPT- 1	4 5 7	27 2		Moisture Content		
	G- 3						
15.0–20.0	SPT- 2	5 6 8	27 0		Moisture Content	ML	Sandy SILT (ML): Tan, moist to wet, stiff to hard (20.0', El. -41.0')
	SPT- 3	11 21 28	21 5		Moisture Content		

Groundwater Depth (ft.): ▼ 12

Notes: Drilled using 6" O.D. Hollow Stem Auger. Boring terminated at depth of (20.0'). Backfilled with neat cement. Groundwater measured at 12.0' bgs.

Boring Log

Date		Project Number	Project	Boring No.
Started: 9/18/18	Completed: 9/18/18	1076	Westside Canal Energy Center	B-3
Hammer Efficiency: 93 %		Rig Type: Unimog M-5 (Pacific)	Surface Elevation: -18.0'	Logged By: S. Burford
Latitude: 32.729953		Longitude: -115.717017	Location: North center	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
19 1	1:30pm	9/18/2018

Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
0						SM	[ALLUVIUM (Qa-Qc)] Silty SAND (SM): Tan, dry to moist
3–4.5	G- 1				Expansion Index No 200 Sieve R Value Thermal Resistivity Corrosivity		4.5' El. -22.5'
5	SPT- 1	15 9 11	8 4		Moisture Content	CL	Lean CLAY (CL): Brown, dry to moist, very stiff 7.0' El. -25.0'
7–10	G- 2					ML	Clayey SILT (ML): Tan, moist
10	MC- 1	19 8 12	20 8	104 2	Moisture / Density		Stiff, Pocket Penetrometer: 1.75 ton/ft^2 12.0' El. -30.0'
13–15	G- 3						Lean CLAY (CL): Brown, moist, stiff
15	SPT- 2	3 5 8	28 8		Moisture Content	CL	
18.5	SPT- 3	3 3 6	26 0		Moisture Content		
20							20.0' El. -38.0'

Notes: Drilled using 6" O.D. Hollow Stem Auger. Boring terminated at depth of (20.0'). Backfilled with neat cement. Groundwater measured at 19.1' bgs.

Boring Log

Sheet 1 of 1

Date		Project Number	Project	Boring No.
	Started: 9/18/18	1076	Westside Canal Energy Center	B-4
	Completed: 9/18/18			
	Hammer Efficiency: 93 %	Rig Type: Unimog M-5 (Pacific)	Surface Elevation: -9.0'	Logged By: S. Burford
	Latitude: 32.726831	Longitude: -115.716616	Location: South center	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date

Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0									[ALLUVIUM (Qa-Qc)] Clayey SAND (SC): Tan, moist
				G- 1				Expansion Index Thermal Resistivity	SC	
	5			MC- 1	8 13 20	22 3	96 4	Moisture / Density		6.5' El. -15.5'
										Lean CLAY (CL): Brown, moist
				G- 2						
	10			SPT- 1	5 7 9	26 3		Moisture Content		Stiff
				G- 3					CL	
	15			MC- 2	15 15 16	16 6	104 8	Moisture / Density		Very stiff, Pocket Penetrometer: 3.25 ton/ft^2
				SPT- 2	4 4 7	22 9		Moisture Content		Stiff
	20									20.0' El. -29.0'

Notes: Drilled using 6" O.D. Hollow Stem Auger. Boring terminated at depth of (20.0'). Backfilled with neat cement. Groundwater not encountered.

Boring Log

Date		Project Number	Project	Boring No.
Started: 10/1/18	Completed: 10/1/18	1076	Westside Canal Energy Center	B-5
Hammer Efficiency: 80 %		Rig Type: Diedrich D50 (Pacific)	Surface Elevation: -20.0'	Logged By: S. Burford
Latitude: 32.729244		Longitude: -115.712156	Location: Northeast corner	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
14	12:40pm	10/1/2018

Groundwater Depth (ft.) / Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
0						SM	[ALLUVIUM (Qa-Qc)] Silty SAND (SM): Tan, dry
	G- 1		4 6		Moisture Content Expansion Index No 200 Sieve		
5							5.0' El. -25.0'
	MC- 1	1 8 15	11 2	107 9	Moisture / Density	CL	Lean CLAY (CL): Brown, moist
	G- 2						
10	SPT- 1	4 5 8	22 2		Moisture Content		Stiff
▼							14.0' El. -34.0'
15	MC- 2	3 5 7			No Recovery	ML	Sandy SILT (ML): Tan, wet
	G- 3						
	SPT- 2	11 16 25	22 6		Moisture Content		Hard
20							20.0' El. -40.0'

Notes: Drilled using 6" O.D. Hollow Stem Auger. Boring terminated at depth of (20.0'). Backfilled with neat cement. Groundwater measured at 14.0' bgs.

NV5

Boring Log

Date		Project Number	Project	Boring No.
	Started: 10/1/18	1076	Westside Canal Energy Center	B-6
	Completed: 10/1/18			
	Hammer Efficiency: 80 %	Rig Type: Diedrich D50 (Pacific)	Surface Elevation: -17.0'	Logged By: S. Burford
	Latitude: 32.726936	Longitude: -115.712139	Location: Southeast corner	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
18	9:55am	10/1/2018

Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0								CL	[ALLUVIUM (Qa-Qc)] Sandy Lean CLAY (CL): Tan, dry
				G- 1		8 8		Expansion Index Thermal Resistivity R-Value Corrosivity Moisture Content		2.0' El. -19.0'
										Fat CLAY (CH): Brown, moist to wet
	5			MC- 1	7 11 17	24 1	99 5	Moisture / Density Direct Shear		
				G- 2						
	10			SPT- 1	3 6 8	25 4		Moisture Content Atterberg Limit		Stiff
				G- 3						
	15			MC- 2	2 4 12	29 1	94 3	Moisture / Density	CH	
▼										
	20			SPT- 2	6 8 9	29 3		Moisture Content Atterberg Limit		Very stiff
				G- 4						
	25			MC- 3	2 2 6	28 1		CAL-3 Disturbed Moisture Content		
				G- 5						29.5' El. -46.5'
	30								SM	

NV5

Boring Log

Date		Project Number	Project	Boring No.
Started: 10/1/18	Completed: 10/1/18	1076	Westside Canal Energy Center	B-6
Hammer Efficiency: 80 %		Rig Type: Diedrich D50 (Pacific)	Surface Elevation: -17.0'	Logged By: S. Burford
Latitude: 32.726936		Longitude: -115.712139	Location: Southeast corner	

Sample Type
G - Bulk / Grab Sample
SPT - 2" O D. 1.5" I.D. Tube Sample
MC - 3 " O.D. 2.4" I D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
18	9:55am	10/1/2018

Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
30	SPT- 3	8 14 24	16 8		Moisture Content	SM	Silty SAND (SM): Tan, moist, dense. Water added to borehole at 30' to maintain stability
35	SPT- 4	3 6 7	24 7		Moisture Content Atterberg Limit		36.0' El. -53.0'
						CL	Lean CLAY with Sand (CL): Brown, moist to wet, stiff
							39.0' El. -56.0'
40	SPT- 5	5 9 9	33 1		Moisture Content	ML	Sandy SILT (ML): Tan, wet, very stiff
							41.0' El. -58.0'
						CL	Lean CLAY (CL): Brown, moist to wet
							43.0' El. -60.0'
						CL	Sandy Lean CLAY (CL): Brown, moist to wet
45	SPT- 6	6 10 11	26 7		Moisture Content Atterberg Limit		Very stiff
50	SPT- 7	9 18 31	25 2		Moisture Content		Hard
							51.5' El. -68.5'

Notes: Drilled using 6" O.D. Hollow Stem Auger. Boring terminated at depth of (51.5'). Backfilled with neat cement. Groundwater measured at 18.0' bgs.

APPENDIX B

Field Resistivity Test Data

October 5, 2018
Project No. 118487

Mr. Sean Roy
NV5
15092 Avenue of Science, Suite 200
San Diego, CA 92128

Subject: Geophysical Evaluation
Westside Canal Project
El Centro, California

Dear Mr. Sean Roy:

In accordance with your authorization, we have performed geophysical survey services pertaining to the proposed Westside Canal Project located south of the intersection of Liebert Road and Mandrapa Road in El Centro, California (Figure 1). The purpose of our services was to collect in-situ electrical resistivity measurements for use in the design and construction of the proposed project. Our services were conducted on September 20 and September 21, 2018. This report presents the survey methodology, equipment used, analysis, and results.

Our scope of services for the project included collection of electrical resistivity data at the site, compilation of the data collected, and preparation of this data report. Specifically, we conducted two crossing, nearly orthogonal resistivity soundings at six locations (R-1 through R-6) onsite for a total of twelve. The roughly north-south trending lines are given an “a” designation (e.g., R-1a) and the roughly west-east lines are given a “b” designation (e.g., R-1b). Figures 1 and 2 illustrate the approximate sounding locations, and Figures 3a and 3b illustrate the conditions in the study area as viewed from the south and west.

The data were collected in general accordance with ASTM G57 using an Advanced Geosciences, Inc. (AGI) MiniSting earth resistivity meter and four steel electrodes in a Wenner configuration. For each of the locations, soil resistance measurements were collected at several electrode spacings, which were designated by your office, along the two lines with the middle of each sounding generally located at a common center point. The stainless-steel electrodes were hammered into place and the soils surrounding the electrodes were moistened with saline water where necessary.

The results of the electrical resistivity survey are presented in Figures 4a through 4c. The measurements collected along each of the soundings are generally consistent (with slight variations) indicating that the subsurface conditions are fairly uniform with respect to apparent resistivity.

The field services and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions presented in this report. Please also note that our evaluation was limited to measuring in-situ apparent soil resistivity at six locations selected by your office. Southwest Geophysics, Inc. should be contacted if the reader has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of this report by parties other than the client is undertaken at said parties' sole risk.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

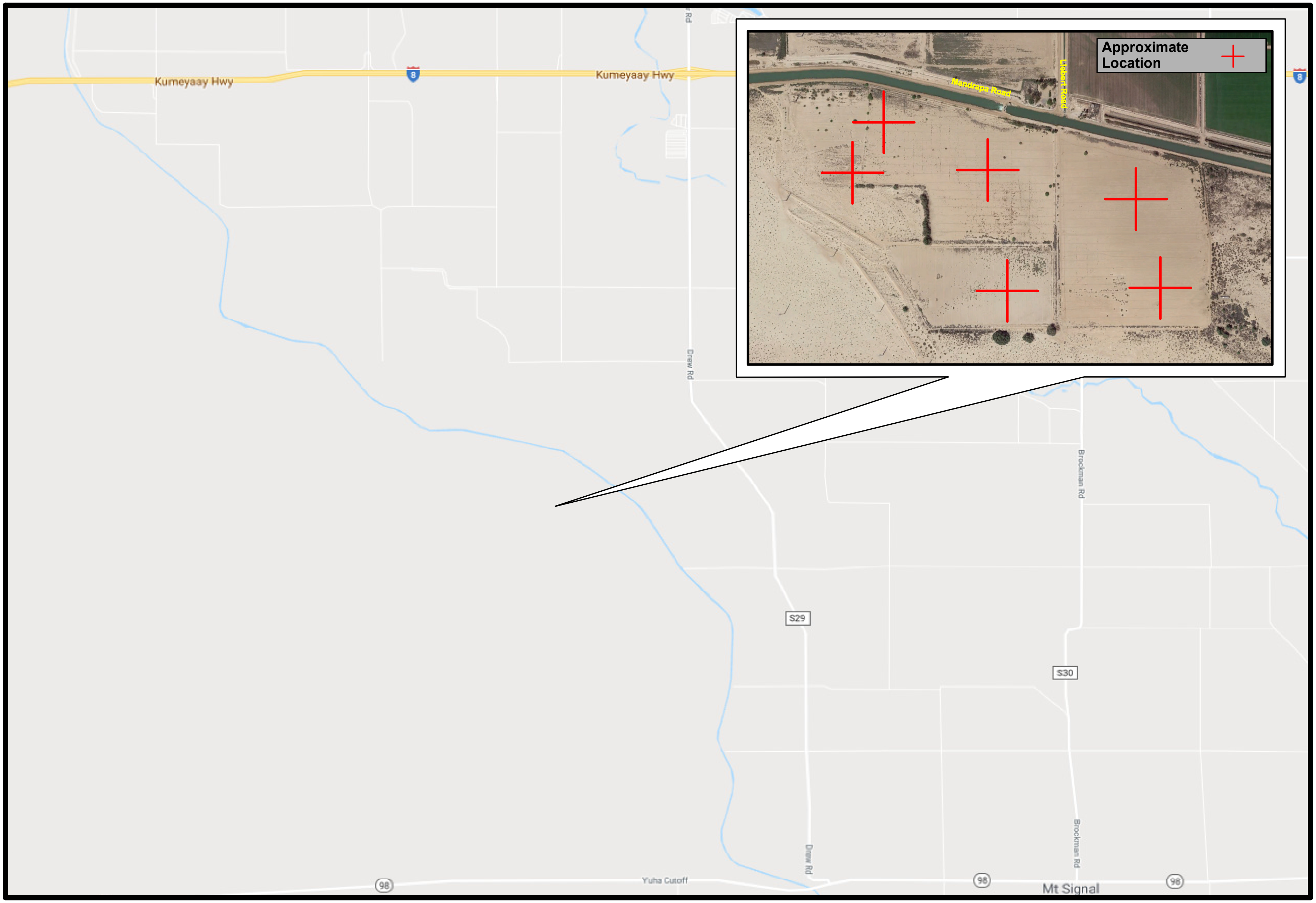
Afrildo Iko Syahrial
Project Geophysicist

Patrick F. Lehrmann, P.G., P.Gp.
Principal Geologist/Geophysicist

ASB/AIS/PFL/pfl

Attachments: Figure 1 – Site Location Map
Figure 2 – Line Location Map
Figure 3a – Site Photographs (R-1 through R-3)
Figure 3b – Site Photographs (R-4 through R-6)
Figure 4a – Electrical Resistivity Results (R-1 and R-2)
Figure 4b – Electrical Resistivity Results (R-3 and R-4)
Figure 4c – Electrical Resistivity Results (R-5 and R-6)

Distribution: Addressee (electronic)



SITE LOCATION MAP

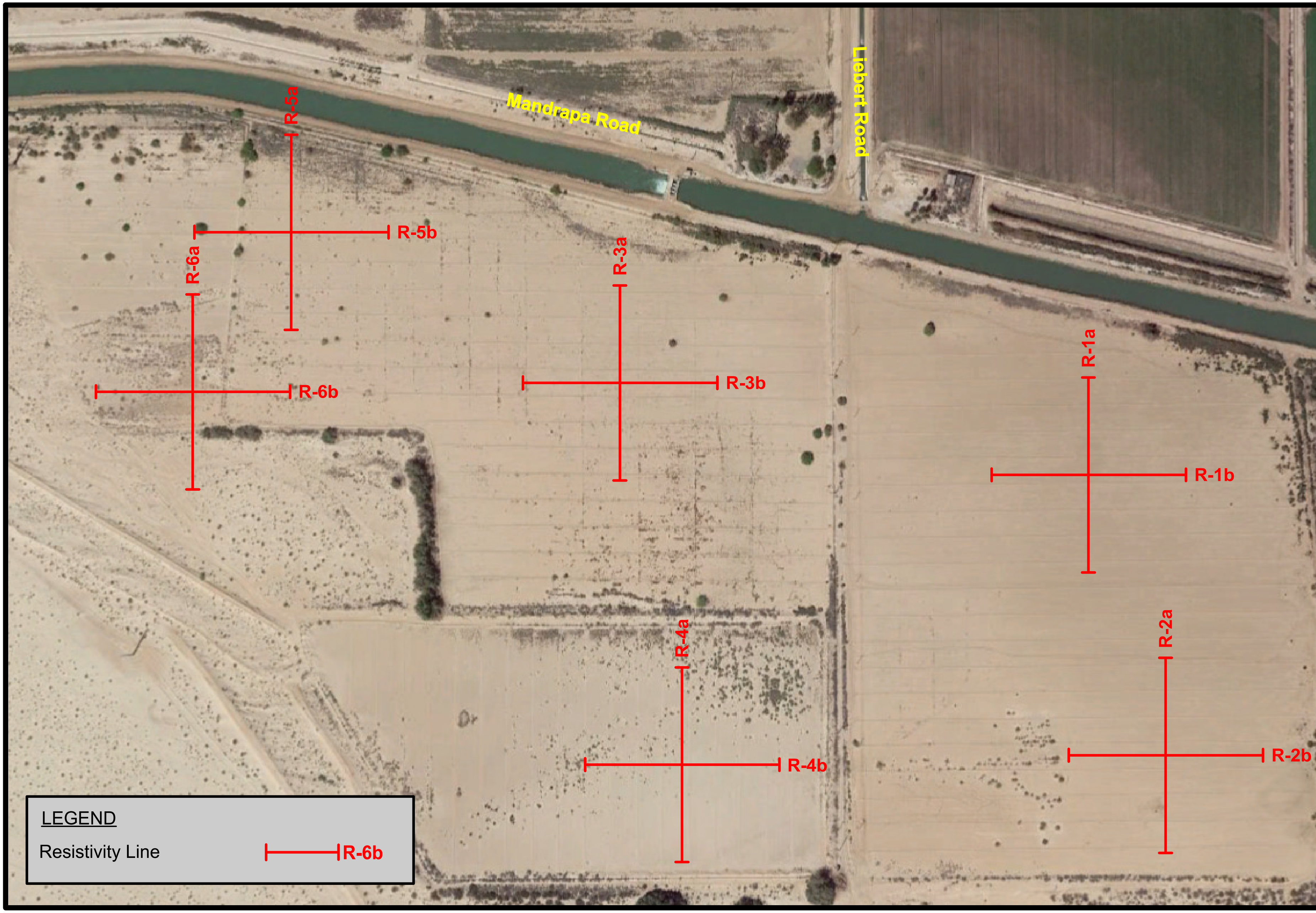
N

Westside Canal
El Centro, California

Project No.: 118487 | Date: 10/18

SOUTHWEST GEOPHYSICS INC

Figure 1



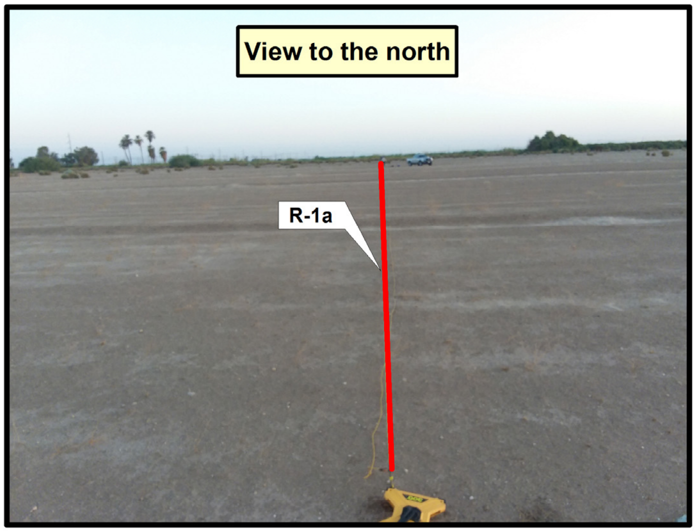
LINE LOCATION MAP

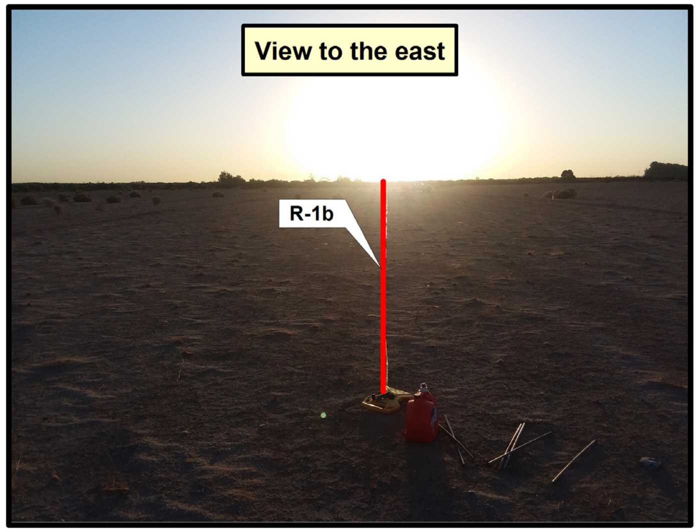
Westside Canal
El Centro, California

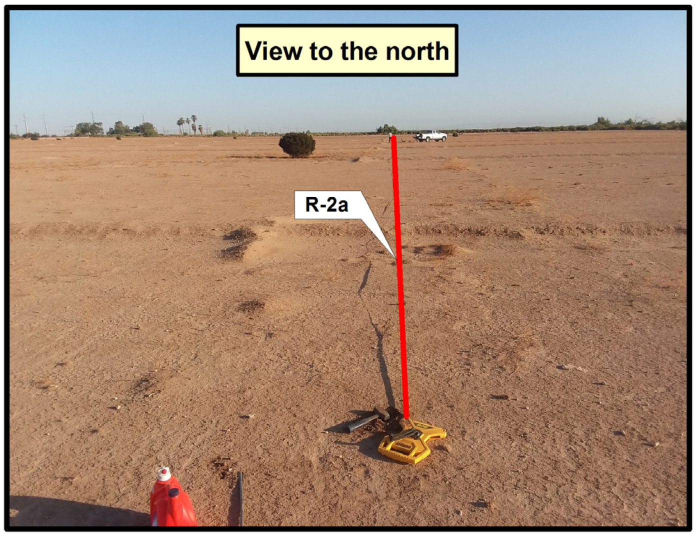
Project No.: 118487 | Date: 10/18

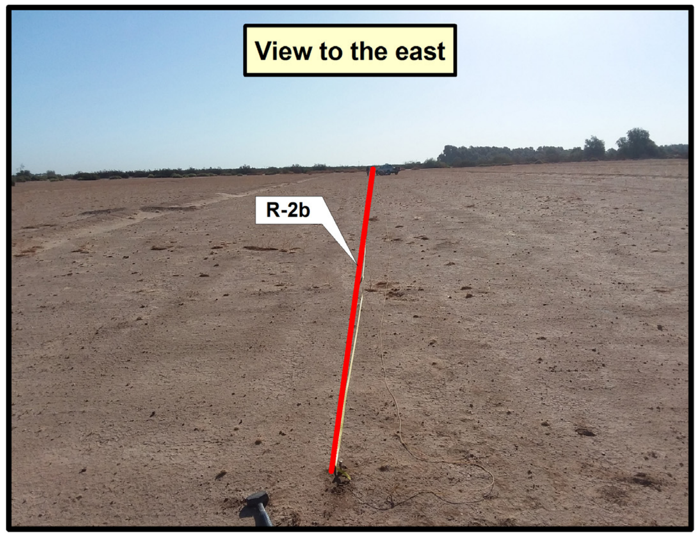
SOUTHWEST GEOPHYSICS INC

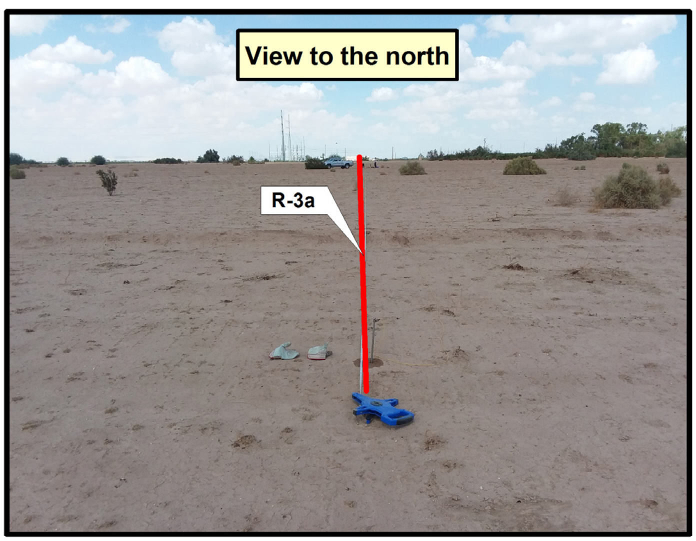
Figure 2

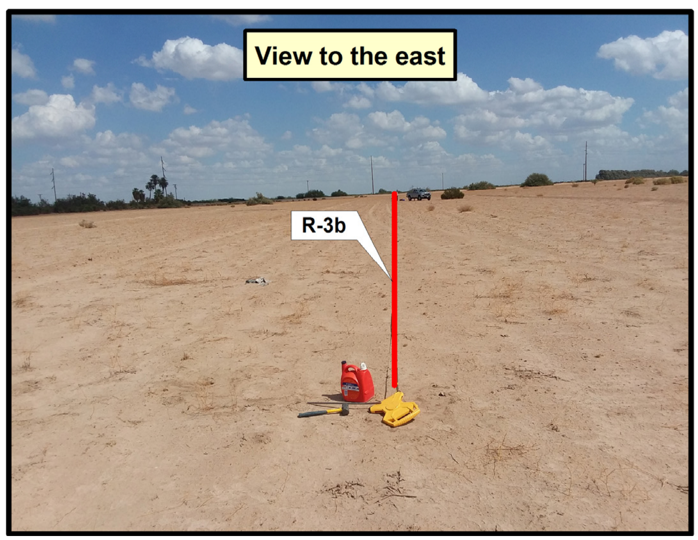




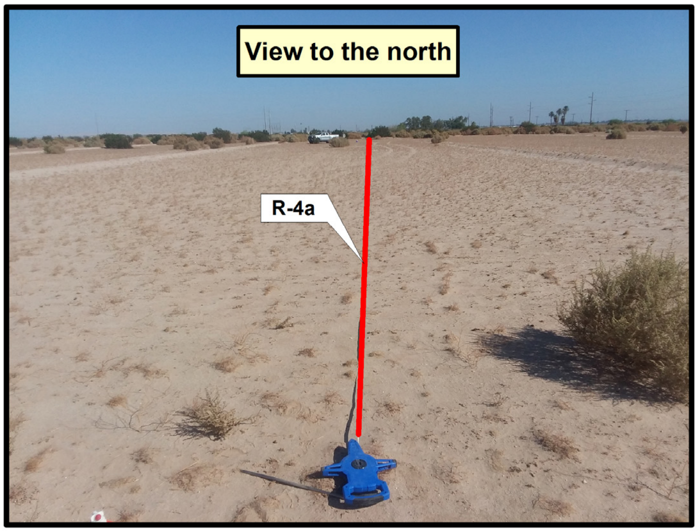


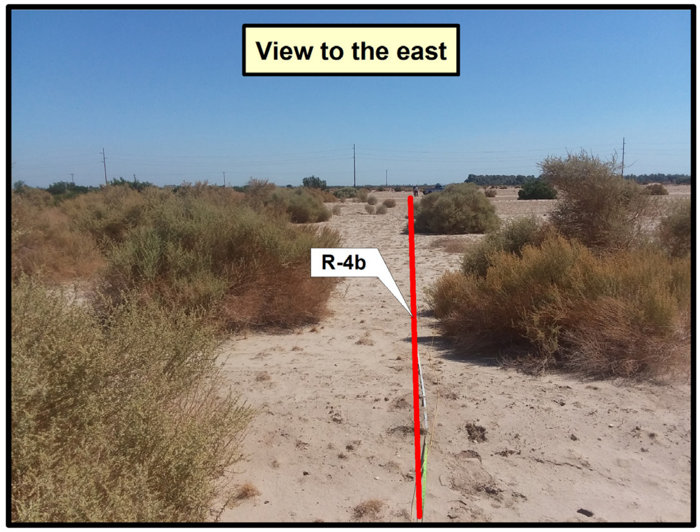


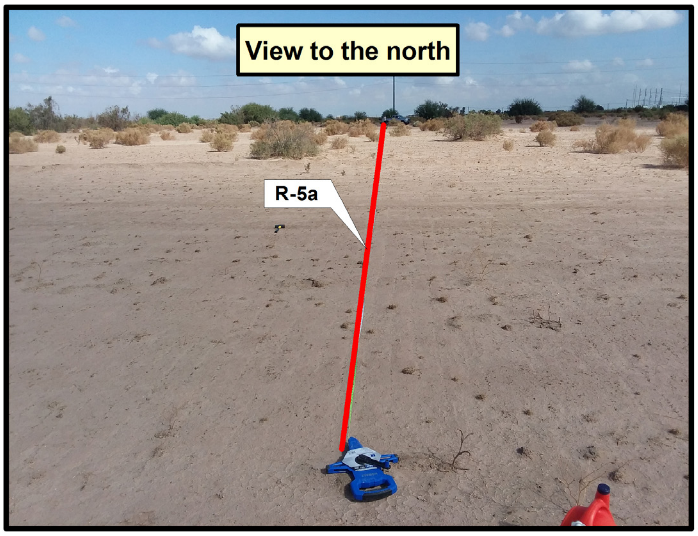


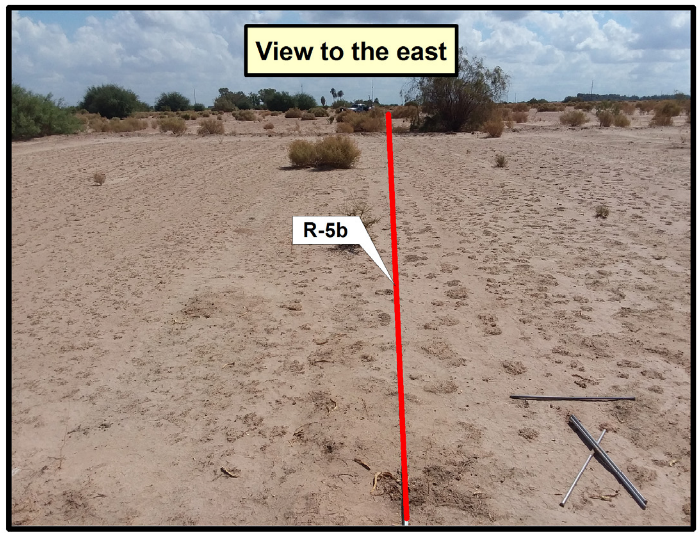


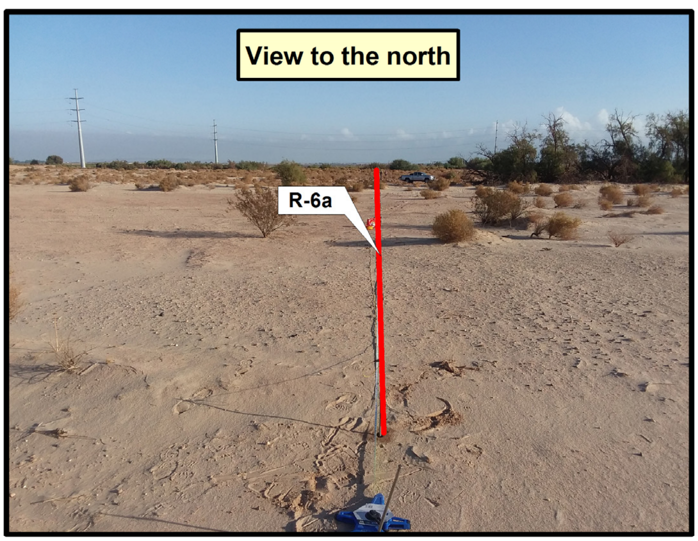
SITE PHOTOGRAPHS	Westside Canal El Centro, California Project No.: 118487 — Date: 10/18	SOUTHWEST GEOPHYSICS INC. Figure 3a

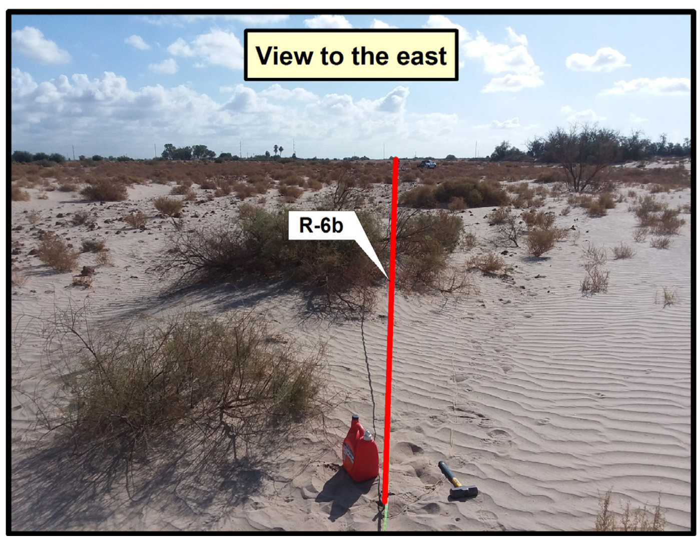












SITE PHOTOGRAPHS

Westside Canal
El Centro, California

Project No.: 118487 | Date: 10/18

SOUTHWEST GEOPHYSICS INC.

Figure 3b

Line No. Orientation	Spacing (ft)	Current (mA)	Resistance (Ohms)	Error (%)	Apparent Resistivity (ohm-cm)	Apparent Resistivity (ohm-ft)
R-1a	2	10	46.62	0.1	17857	586
(N-S)	4	5	23.45	0.0	17964	589
	6	5	16.09	0.0	18489	607
	8	2	20.07	0.1	30749	1009
	12	10	9.27	0.0	21297	699
	20	10	4.19	0.1	16041	526
	30	5	1.98	0.1	11353	372
	50	10	0.41	0.3	3937	129
	100	20	0.10	0.3	1992	65
	200	20	0.04	0.2	1462	48
R-1b	2	20	60.89	0.0	23322	765
(E-W)	4	20	21.69	0.0	16616	545
	6	20	14.32	0.0	16455	540
	8	20	13.32	0.2	20407	670
	12	20	12.12	0.0	27853	914
	20	10	5.77	0.2	22108	725
	30	10	2.11	0.2	12123	398
	50	10	0.38	0.2	3631	119
	100	10	0.13	0.3	2432	80
	200	20	0.05	0.0	1848	61
R-2a	2	5	623.80	0.0	238930	7839
(N-S)	4	10	137.80	0.0	105561	3463
	6	5	105.00	0.0	120652	3958
	8	5	72.01	0.0	110326	3620
	12	5	31.56	0.1	72529	2380
	20	10	4.28	0.0	16382	537
	30	5	1.01	0.1	5809	191
	50	5	0.18	0.1	1696	56
	100	20	0.04	0.3	778	26
	200	20	0.02	0.1	954	31
R-2b	2	5	286.00	0.0	109544	3594
(E-W)	4	5	166.20	0.1	127317	4177
	6	5	87.12	0.0	100107	3284
	8	5	56.30	0.0	86257	2830
	12	10	24.62	0.1	56580	1856
	20	5	7.41	0.0	28374	931
	30	2	1.15	0.3	6630	218
	50	10	0.10	0.1	985	32
	100	10	0.04	0.3	858	28
	200	20	0.03	0.0	1002	33

ELECTRICAL RESISTIVITY RESULTS

Westside Canal
El Centro, California

Project No.: 118487

Date: 10/18

SOUTHWEST GEOPHYSICS INC.

Figure 4a

Line No. Orientation	Spacing (ft)	Current (mA)	Resistance (Ohms)	Error (%)	Apparent Resistivity (ohm-cm)	Apparent Resistivity (ohm-ft)
R-3a	2	5	30.78	0.2	11789	387
(N-S)	4	5	3.04	0.1	2326	76
	6	10	1.28	0.1	1465	48
	8	10	0.65	0.1	994	33
	12	5	0.34	0.2	778	26
	20	10	0.15	0.1	593	19
	30	10	0.11	0.2	633	21
	50	5	0.07	0.2	715	23
	100	5	0.05	0.2	936	31
	200	5	0.02	0.2	781	26
R-3b	2	20	23.69	0.0	9074	298
(E-W)	4	10	3.25	0.2	2488	82
	6	5	1.13	0.0	1294	42
	8	10	0.67	0.2	1022	34
	12	20	0.35	0.0	801	26
	20	5	0.15	0.3	570	19
	30	10	0.10	0.1	592	19
	50	10	0.08	0.0	765	25
	100	10	0.06	0.2	1109	36
	200	20	0.04	0.1	1476	48
R-4a	2	10	441.00	0.0	168913	5542
(N-S)	4	10	35.51	0.0	27202	892
	6	10	7.00	0.0	8042	264
	8	10	3.64	0.0	5575	183
	12	10	1.50	0.1	3436	113
	20	5	0.57	0.1	2191	72
	30	10	0.27	0.1	1524	50
	50	20	0.09	0.3	859	28
	100	20	0.03	0.1	497	16
	200	50	0.02	0.0	612	20
R-4b	2	5	354.40	0.0	135743	4454
(E-W)	4	5	107.10	0.2	82043	2692
	6	5	15.09	0.0	17339	569
	8	10	5.43	0.0	8322	273
	12	10	1.63	0.1	3748	123
	20	2	0.66	0.1	2514	82
	30	10	0.25	0.0	1444	47
	50	5	0.09	0.1	902	30
	100	20	0.02	0.3	356	12
	200	20	0.02	0.1	582	19

ELECTRICAL RESISTIVITY RESULTS

Westside Canal
El Centro, California

Project No.: 118487

Date: 10/18

SOUTHWEST GEOPHYSICS INC.

Figure 4b

Line No. Orientation	Spacing (ft)	Current (mA)	Resistance (Ohms)	Error (%)	Apparent Resistivity (ohm-cm)	(ohm-ft)
R-5a	2	20	9.78	0.2	3746	123
(N-S)	4	50	1.57	0.0	1203	39
	6	50	0.49	0.1	559	18
	8	100	0.30	0.1	458	15
	12	50	0.18	0.0	406	13
	20	100	0.13	0.1	508	17
	30	200	0.10	0.1	561	18
	50	200	0.06	0.1	595	20
	100	100	0.03	0.0	656	22
	200	200	0.02	0.0	742	24
R-5b	2	20	8.27	0.0	3166	104
(E-W)	4	20	1.11	0.1	848	28
	6	20	0.57	0.1	650	21
	8	50	0.33	0.1	499	16
	12	50	0.20	0.0	467	15
	20	50	0.13	0.1	483	16
	30	100	0.10	0.1	561	18
	50	50	0.06	0.2	580	19
	100	100	0.03	0.1	629	21
	200	20	0.01	0.1	561	18
R-6a	2	100	2.77	0.0	1062	35
(N-S)	4	200	0.38	0.1	289	9
	6	200	0.22	0.0	247	8
	8	200	0.19	0.1	293	10
	12	500	0.09	0.0	216	7
	20	500	0.05	0.0	204	7
	30	200	0.04	0.1	219	7
	50	500	0.03	0.0	247	8
	100	500	0.02	0.0	329	11
	200	20	0.02	0.0	630	21
R-6b	2	100	2.53	0.1	970	32
(E-W)	4	200	0.68	0.0	522	17
	6	100	0.27	0.0	307	10
	8	200	0.13	0.0	205	7
	12	200	0.08	0.0	182	6
	20	200	0.06	0.1	221	7
	30	500	0.04	0.0	234	8
	50	200	0.03	0.0	257	8
	100	200	0.02	0.1	341	11
	200	200	0.01	0.1	506	17

ELECTRICAL RESISTIVITY RESULTS

Westside Canal
El Centro, California

Project No.: 118487

Date: 10/18

SOUTHWEST GEOPHYSICS INC.

Figure 4c

APPENDIX C

Laboratory Test Results

SUMMARY OF LABORATORY TEST RESULTS

In-situ Moisture and Density Tests

The in-situ moisture contents and dry densities of selected samples obtained from the test borings were evaluated in general accordance with the latest version of D2216 and D2937 laboratory test methods. The method involves obtaining the moist weight of the sample and then drying the sample to obtain it's dry weight. The moisture content is calculated by taking the difference between the wet and dry weights, dividing it by the dry weight of the sample and expressing the result as a percentage. The results of the in-situ moisture content and density tests are presented in the following table and on the logs of exploratory borings in Appendix A.

RESULTS OF MOISTURE CONTENT AND DENSITY TESTS
(ASTM D2216 and ASTM D2937)

Sample Location	Moisture Content (percent)	Dry Density (pounds per cubic foot)
Boring 1 @ 3 - 5 feet	20.1	Not Tested
Boring 1 @ 5.5 - 6 feet	26.1	97.7
Boring 1 @ 10 - 11.5 feet	25.8	Not Tested
Boring 1 @ 15 - 16.5 feet	22.1	Not Tested
Boring 1 @ 20 – 21.5 feet	21.8	Not Tested
Boring 1a @ 15 – 16.5 feet	24.3	Not Tested
Boring 1a @ 20 - 21.5 feet	24.8	Not Tested
Boring 1a @ 25 - 26.5 feet	22.5	Not Tested
Boring 1a @ 30 - 31.5 feet	22.1	Not Tested
Boring 1a @ 35 - 36.5 feet	22.7	Not Tested
Boring 1a @ 40 - 41.5 feet	22.4	Not Tested
Boring 1a @ 45 - 46.5 feet	21.4	Not Tested
Boring 1a @ 50 - 51.5 feet	22.4	Not Tested
Boring 1a @ 55 - 56.5 feet	22.0	Not Tested
Boring 1a @ 60 - 61.5 feet	23.1	Not Tested
Boring 1a @ 65 - 66.5 feet	22.0	Not Tested
Boring 1a @ 70 - 71.5 feet	21.3	Not Tested
Boring 1a @ 75 - 76.5 feet	21.2	Not Tested

Sample Location	Moisture Content (percent)	Dry Density (pounds per cubic foot)
Boring 2 @ 6 – 6.5 feet	5.1	102.1
Boring 2 @ 10 – 11.5 feet	27.2	Not Tested
Boring 2 @ 15 - 16.5 feet	27.0	Not Tested
Boring 2 @ 18.5 – 20 feet	21.5	Not Tested
Boring 3 @ 5 – 6.5 feet	8.4	Not Tested
Boring 3 @ 11 – 11.5 feet	20.8	104.2
Boring 3 @ 15 - 16.5 feet	28.8	Not Tested
Boring 3 @ 18.5 - 20 feet	26.0	Not Tested
Boring 4 @ 6 - 6.5 feet	22.3	96.4
Boring 4 @ 10 – 11.5 feet	26.3	Not Tested
Boring 4 @ 16 – 16.5 feet	16.6	104.8
Boring 4 @ 18.5 - 20 feet	22.9	Not Tested
Boring 5 @ 3 – 5 feet	4.6	Not Tested
Boring 5 @ 6 - 6.5 feet	11.2	107.9
Boring 5 @ 10 – 11.5 feet	22.2	Not Tested
Boring 5 @ 18.5 – 20 feet	22.6	Not Tested
Boring 6 @ 1 – 3 feet	8.8	Not Tested
Boring 6 @ 6 – 6.5 feet	24.1	99.5
Boring 6 @ 10 – 11.5 feet	25.4	Not Tested
Boring 6 @ 16 – 16.5 feet	29.1	94.3
Boring 6 @ 20 - 21.5 feet	29.3	Not Tested
Boring 6 @ 26 - 26.5 feet	28.1	Not Tested
Boring 6 @ 30 - 31.5 feet	16.8	Not Tested
Boring 6 @ 35 - 36.5 feet	24.7	Not Tested
Boring 6 @ 40 - 41.5 feet	33.1	Not Tested
Boring 6 @ 45 – 46.5 feet	26.7	Not Tested
Boring 6 @ 50 – 51.5 feet	25.2	Not Tested

Classification

Soils were visually and texturally classified in general accordance with the Unified Soil Classification System (ASTM D2487). Soil classifications are indicated on the logs of the exploratory borings presented in Appendix A.

Particle-size Distribution Tests

An evaluation of the grain-size distribution of selected soil samples was performed in general accordance with the latest versions of ASTM D1140 and ASTM D6913 (excluding hydrometer). These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System. Particle size distribution test results are presented on the laboratory test sheets attached in this appendix.

Expansion Index Tests

Expansion index tests were performed on samples of the on-site soils. The tests were performed in general accordance with ASTM D4829. The results of the tests are presented below and attached in this appendix.

Location	B-1 @ 3' - 5'	B-2 @ 3' - 5'	B-3/B-5 @ 3' - 5'	B-4 @ 3' - 5'	B-6 @ 1' - 3'
Material Type	Tan Lean CLAY with Sand (CL)	Brown Fat CLAY (CH)	Tan Silty SAND (SM)	Tan Clayey SAND (SC)	Brown Fat CLAY (CH)
Source	Native	Native	Native	Native	Native
Initial Moisture Content, %	10.2	10.2	8.3	7.6	11.6
Final Moisture Content, %	20.5	25.9	16.1	17.3	27.8
Dry Density, pcf	109.7	108.4	116.3	118.6	104.5
Initial Saturation, %	51.3	49.7	49.8	48.8	51.1
Expansion Index	**50**	**106**	**14**	**54**	**106**
Potential Expansion	**LOW**	**HIGH**	**VERY LOW**	**MEDIUM**	**HIGH**

Atterberg Limits

Atterberg limits tests were performed in general accordance with ASTM D4318 on selected soil samples. These tests were useful in classification of the soils. Test results are attached in this appendix and summarized below.

Location	B-6 @ 10 – 11.5 ft	B-6 @ 20 – 21.5 ft	B-6 @ 35 – 36.5 ft	B-6 @ 45 – 46.5
Material Type	Fat CLAY (CH)	Fat CLAY (CH)	Lean CLAY with Sand (CL)	Sandy Lean CLAY (CL)
Liquid Limit	75	66	32	34
Plastic Limit	20	19	14	18
Plasticity Index	55	47	18	16

Thermal Resistivity Tests

Various bulk soil samples were packaged and returned to NV5's in house laboratory for thermal resistivity analysis. The bulk soil samples were placed, remolded and compacted within a 2.4 inch diameter by 6 inch long mold. Testing for thermal resistivity (rho) was completed in general accordance with test methods IEEE 442 and ASTM D5334. The results of the laboratory testing are summarized below and included in this appendix and summarized in the table below.

Sample # and Depth	Soil Description	Remolded & Compacted Dry Density (pcf)	Expansion Index	% Passing the No. 200 Sieve	Thermal Resistivity @ 0% Moisture (Dry) (°C-cm/W	Thermal Resistivity @ 4% Critical Moisture (Wet) (°C-cm/W)	Thermal Resistivity @ Wet Point (°C-cm/W)	Moisture Content @ Wet Point (%)
B2 @ 3-5'	Fay CLAY (CH)	108	106	Not Tested	136	84	71	10.7
B3 @ 3-5'	Silty SAND (SM)	111	14	40.4	145	70	65	5.7
B4 @ 3-5'	Clayey SAND (SC)	110	54	Not Tested	131	77	66	7.2
B6 @ 1-3'	Fat CLAY (CH)	104	106	Not Tested	140	104	75	13.4

Resistance "R" values test

R-Value tests were performed on samples of the on-site soils. The tests were performed in general accordance with California Test Method 301/ ASTM D2844. The result of the tests are presented below and attached in this appendix.

Location	B-3 @ 3 – 5 ft	B-6 @ 1 – 3 ft
"R" Value	57	5
Material Type	Silty SAND (SM)	Fat CLAY (CH)

Direct Shear

A direct shear test was performed on a representative relatively undisturbed sample in general accordance with ASTM D3080 to evaluate the shear strength characteristics of the on-site materials. The test method consists of placing the soil sample in the direct shear device, applying a series of normal stresses, and then shearing the sample at the constant rate of shearing deformation. The shearing force and horizontal displacements are measured and recorded as the soil specimen is sheared. The shearing is continued well beyond the point of maximum stress until the stress reaches a constant or residual value. The results of the tests are presented in the following table and attached in this appendix.

RESULTS OF DIRECT SHEAR TEST
(ASTM D3080)

Location	USCS Classification	Peak Friction (degrees)	Ultimate Friction (degrees)	Peak Cohesion (psf)	Ultimate Cohesion (psf)	Notes
Boring 6 @ 6 - 6.5 ft.	CH	32	29	933	341	Relatively undisturbed

Soil Corrosivity Tests

Water soluble sulfate, chloride, resistivity and pH tests were performed by Clarkson Laboratory and Supply Inc., in general accordance with California Test Methods 643, 417 and 422 to provide an indication of the degree of corrosivity of the subgrade soils at locations tested with regard to concrete and normal grade steel. The results of the tests are presented in the following table and on the laboratory test sheets attached in this appendix.

RESULTS OF CORROSIVITY TESTS
(CTM 417, CTM 422 and CTM 643)

Sample Location	B-3 @3-5 ft	B-6 @1-3 ft
pH	9.3	8.5
Minimum Resistivity (Ohm-cm)	820	120
Water Soluble Sulfates (ppm)	420	2,310
Water Soluble Chlorides (ppm)	130	2,140
Material Type	Silty SAND (SM)	Fat CLAY (CH)
Percent Finer Than No. 200 Sieve	40.4%	Not Tested

Natural Moisture Report

(ASTM D2216)

Date:	October 10, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6918
Address:	488 8th Avenue	Lab Number:	116882-116894
	San Diego, CA 92101		
Project:	Westside Canal Energy Center		
Project Add:	Imperial Valley, CA		
Sampled By:	Sean Burford		
Date Sampled	10/2/2018		
Date Rcvd:	10/2/2018		

Lab Number	116882	116883	116884	116885	116886
Exploration No.	B-1A	B-1A	B-1A	B-1A	B-1A
Depth, ft.	15-16.5	20-21.5	25-26.5	30-31.5	35-36.5
Moisture Content, %	24.3	24.8	22.5	22.1	22.7

Lab Number	116887	116888	116889	116890	116891
Exploration No.	B-1A	B-1A	B-1A	B-1A	B-1A
Depth, ft.	40-41.5	45-46.5	50-51.5	55-56.5	60-61.5
Moisture Content, %	22.4	21.4	22.4	22.0	23.1

Lab Number	116892	116893	116894		
Exploration No.	B-1A	B-1A	B-1A		
Depth, ft.	65-66.5	70-71.5	75-76.5		
Moisture Content, %	22.0	21.3	21.2		

Respectfully Submitted,
NV5 West, Inc.

Reviewed by:

Carl Henderson, PhD, PE, GE
CQA Group Director

Natural Moisture & Density Report

(ASTM D2216 & ASTM D2937)

Date:	October 11, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6881
Address:	488 8th Avenue	Lab Number:	116792-116810
	San Diego, CA 92101		
Project:	Westside Canal Energy Center		
Project Add:	Imperial Valley, CA		

Sampled By:	Sean Burford
Date Sampled	9/17-18/2018
Date Rcvd:	9/19/2018

Lab Number	116880	116792	116793	116794	116795
Exploration No.	B1	B1	B1	B1	B1
Depth, ft.	3-5	5.5-6	10-11.5	15-16.5	20-21.5
Moisture Content, %	20.1	26.1	25.8	22.1	21.8
Dry Density, pcf	-	97.7	-	-	-

Lab Number	116797	116798	116799	116800	116802
Exploration No.	B2	B2	B2	B2	B3
Depth, ft.	6-6.5	10-11.5	15-16.5	18.5-20	5-6.5
Moisture Content, %	5.1	27.2	27.0	21.5	8.4
Dry Density, pcf	102.1	-	-	-	-

Lab Number	116803	116804	116805	116807	116808
Exploration No.	B3	B3	B3	B4	B4
Depth, ft.	11-11.5	15-16.5	18.5-20	6-6.5	10-11.5
Moisture Content, %	20.8	28.8	26.0	22.3	26.3
Dry Density, pcf	104.2	-	-	96.4	-

Natural Moisture & Density Report

(ASTM D2216 & D2937)

Date:	October 12, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6919
Address:	488 8th Avenue	Lab Number:	116895-116909
	San Diego, CA 92101		
Project:	Westside Canal Energy Center		
Project Add:	Imperial Valley, CA		

Sampled By:	Sean Burford
Date Sampled	10/1/2018
Date Rcvd:	10/2/2018

Lab Number	116895	116896	116897	116898	116899
Exploration No.	B5	B5	B5	B5	B6
Depth, ft.	3-5	6-6.5	10-11.5	18.5-20	1-3
Moisture Content, %	4.6	11.2	22.2	22.6	8.8
Dry Density, pcf.	-	107.9	-	-	-

Lab Number	116900	116901	116902	116903	116904
Exploration No.	B6	B6	B6	B6	B6
Depth, ft.	6-6.5	10-11.5	16-16.5	20-21.5	26-26.5
Moisture Content, %	24.1	25.4	29.1	29.3	28.1
Dry Density, pcf.	99.5	-	94.3	-	-

Lab Number	116905	116906	116907	116908	116909
Exploration No.	B6	B6	B6	B6	B6
Depth, ft.	30-31.5	35-36.5	40-41.5	45-46.5	50-51.5
Moisture Content, %	16.8	24.7	33.1	26.7	25.2
Dry Density, pcf.	-	-	-	-	-

Respectfully Submitted,
NV5 West, Inc.

Reviewed by: Carl Henderson

Carl Henderson, PhD, PE, GE
CQA Group Director

Lab Number	116809	116810			
Exploration No.	B4	B4			
Depth, ft.	16-16.5	18.5-20			
Moisture Content, %	16.6	22.9			
Dry Density, pcf	104.8	-			

Respectfully Submitted,
NV5 West, Inc.

Reviewed by:

Carl Henderson, PhD, PE, GE
CQA Group Director

N|V|5

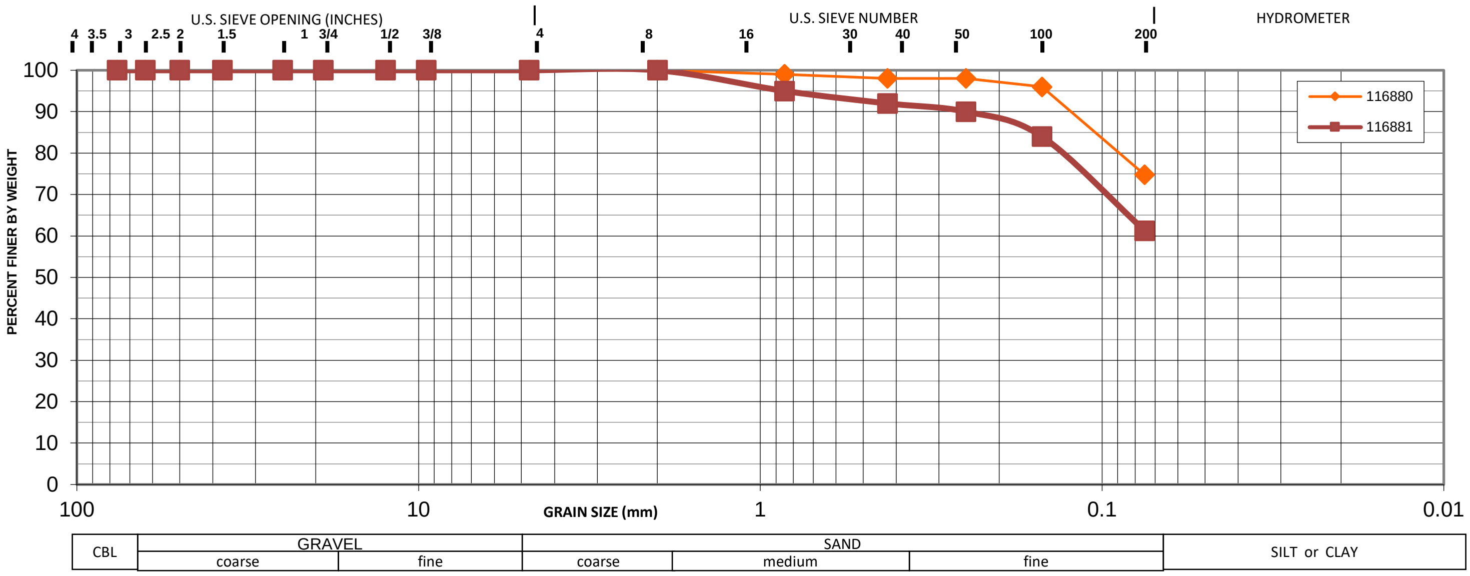
REPORT OF SIEVE ANALYSIS TEST

ASTM D6913 - Soil

Date: October 11, 2018
Client: Sempra Renewables
Address: 488 8th Avenue
San Diego, CA 92101
Project : Westside Canal Energy Center
Project Address: Imperial Valley, CA

Job Number: 1076
Report Number: 6881
Lab Number: 116880-116881

	116880	116881				
Material	Lean CLAY with Sand (CL)	Sandy Lean CLAY (CL)				
Color	Tan	Tan				
Material Source	Native	Native				
Sample Location	B1 @ 3'-5'	B1 @ 13'-15'				
Date Sampled	9/17-18/2018	9/17-18/2018				
Date Submitted	9/19/2018	9/19/2018				
Sampled By	Sean Burford	Sean Burford				
Date Tested	10/3/2018	10/3/2018				
Tested By	Edwin Ocampo	Edwin Ocampo				



Sample ID:	116880	116881				
Sieve Size	% Passing					
76.2mm (3")	100	100				
63mm (2 1/2")	100	100				
50mm (2")	100	100				
37.5mm (1 1/2")	100	100				
25mm (1")	100	100				
19mm (3/4")	100	100				
12.5mm (1/2")	100	100				
9.5mm (3/8")	100	100				
4.75mm (#4)	100	100				
2mm (#10)	100	100				
850µm (#20)	99	95				
425µm (#40)	98	92				
250µm (#60)	98	90				
150 µm (#100)	96	84				
75 um (#200) washµ	74.8	61.3				
Fineness Modulus	0.1	0.3				
Shape (sand & gravel)	N.R.	N.R.				
Hardness (sand & gravel)	N.R.	H&D				
Specific Gravity	2.65	2.65				
Coef. of Curvature (C_C)	N.R.	N.R.				
Coef. of Uniformity (C_U)	N.R.	N.R.				
% Gravel	0	0				
% Sand	25	39				
% Fines	74.8	61.3				
USCS Class:	CL	CL				

Notes: *Hardness: H&D = Hard & Durable; W&F = Weathered & Friable*
N.R.: Not Recorded; N/A: Not Available.

Respectfully Submitted,
NV5 West, Inc.

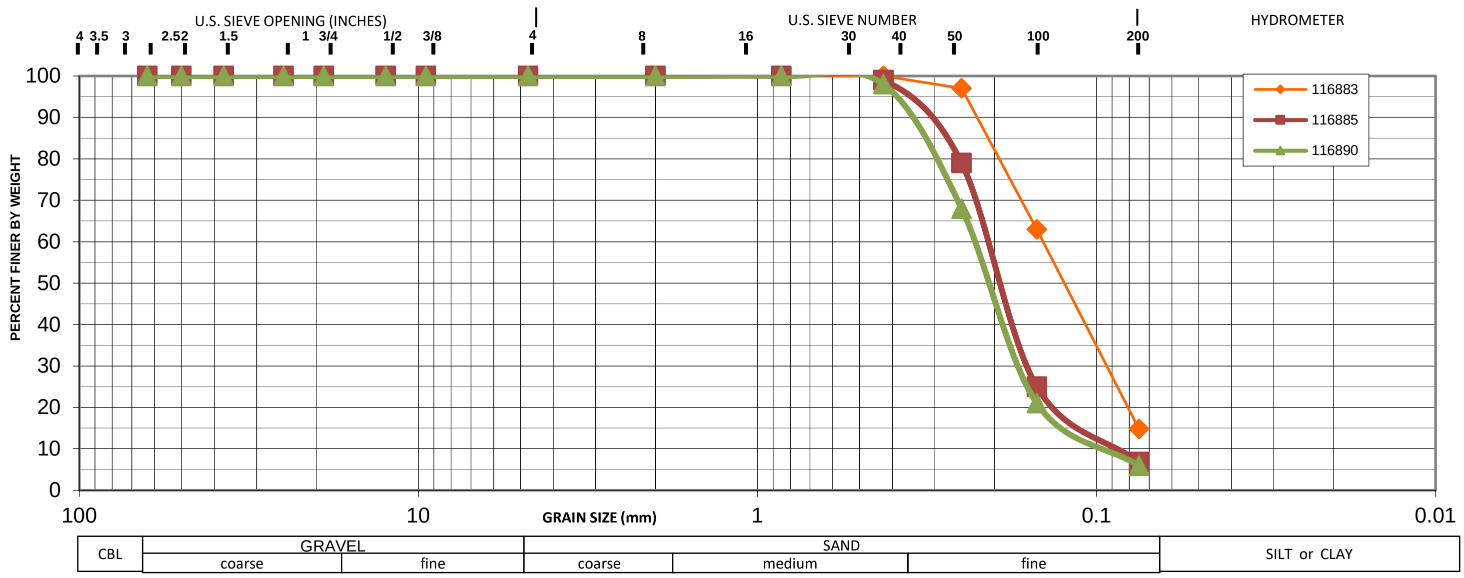
Carl Henderson, PhD, PE, GE
CQA Group Director

REPORT OF SIEVE ANALYSIS TEST

ASTM D6913 - Soil

Date:	October 10, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6918
Address:	488 8th Avenue San Diego, CA 92101	Lab Number:	116883, 116885, 116890
Project :	Westside Canal Energy Center		
Project Address:	Imperial Valley, CA		

	116883	116885	116890			
Material	Silty SAND (SM)	Poorly-graded SAND with Silt (SP-SM)	Poorly-graded SAND with Silt (SP-SM)			
Color	Tan	Brown	Tan			
Sample Source	Native	Native	Native			
Sample Location	B-1A @ 20'-21.5' & 25'-26.5'	B-1A @ 30'-31.5' to 50'-51.5'	B-1A @ 55'-56.5' to 75'-76.5'			
Date Sampled	10/2/2018	10/2/2018	10/2/2018			
Date Submitted	10/2/2018	6/29/2018	6/29/2018			
Sampled By	Sean Burford	Sean Burford	Sean Burford			
Date Tested	10/4/2018	10/8/2018	10/8/2018			
Tested By	Edwin Ocampo	Edwin Ocampo	Edwin Ocampo			



Sample ID:	116883	116885	116890			
Sieve Size	% Passing					
63mm (2 1/2")	100	100	100			
50mm (2")	100	100	100			
37.5mm (1 1/2")	100	100	100			
25mm (1")	100	100	100			
19mm (3/4")	100	100	100			
12.5mm (1/2")	100	100	100			
9.5mm (3/8")	100	100	100			
4.75mm (#4)	100	100	100			
2mm (#10)	100	100	100			
850µm (#20)	100	100	100			
425µm (#40)	100	99	98			
250µm (#60)	97	79	68			
150 µm (#100)	63	25	21			
75 um (#200) washµ	14.8	6.8	6.0			
Fineness Modulus	0.4	0.8	0.8			
Shape (sand & gravel)	N.R.	N.R.	Round			
Hardness (sand & gravel)	N.R.	H&D	N.R.			
Specific Gravity	2.65	2.65	2.65			
Coef. of Curvature (C_C)	N.R.	N.R.	N.R.			
Coef. of Uniformity (C_U)	N.R.	N.R.	N.R.			
% Gravel	0	0	0			
% Sand	85	93	94			
% Fines	14.8	6.8	6.0			
USCS Class:	SM	SP-SM	SP-SM			

Notes: *Hardness: H&D = Hard & Durable; W&F = Weathered & Friable*
N.R.: Not Recorded; N/A: Not Available.

Respectfully Submitted,
NV5 West, Inc.

Carl Henderson

Carl Henderson, PhD, PE, GE
CQA Group Director

Material Finer Than 75-µm (No.200) Sieve in Soils by Washing

(ASTM D1140)

Date:	October 18, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6948
Address:	488 8th Avenue	Lab Number:	117009
	San Diego, CA 92101		
Project:	Westside Canal Energy Center		
Project Add:	Imperial Valley, CA		

Sampled By:	Sean Burford
Date Sampled	10/17/2018
Date Rcvd:	10/17/2018

Lab Number	117009
Sample No.	B3 & B5
Depth, ft.	3'-5'
Source	Native
Material Type	Brown Silty SAND (SM)
% Finer Than 75-µm	40.4

Respectfully Submitted,
NV5 West, Inc.

Reviewed by:

Carl Henderson, PhD, PE, GE
CQA Group Director

Expansion Index Test Report

(ASTM D4829)

Date:	October 11, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6881
Address:	488 8th Avenue	Lab Number:	116796-11806
	San Diego, CA 92101		
Project:	Westside Canal Energy Center		
Project Add:	Imperial Valley, CA		
Sampled By:	Sean Burford		
Date Sampled	9/17-18/2018		
Date Rcvd:	9/19/2018		

Lab Number	116796	116806
Location	B2 @ 3'-5'	B4 @ 3'-5'
Material Type	Brown Fat CLAY (CH)	Tan Clayey SAND (SC)
Source	Native	Native
Initial Moisture Content, %	10.2	7.6
Final Moisture Content, %	25.9	17.3
Dry Density, pcf	108.4	118.6
Initial Saturation, %	49.7	48.8
Expansion Index	**106**	**54**
Potential Expansion	**HIGH**	**MEDIUM**

Respectfully Submitted,
NV5 West, Inc.

Carl Henderson, PhD, PE, GE
CQA Group Director

Expansion Index Test Report

(ASTM D4829)

Date:	October 12, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6919
Address:	488 8th Avenue	Lab Number:	116899
	San Diego, CA 92101		
Project:	Westside Canal Energy Center		
Project Add:	Imperial Valley, CA		

Sampled By:	Sean Burford
Date Sampled	10/1/2018
Date Rcvd:	10/2/2018

Lab Number	116899
Location	B6 @ 1’-3’
Material Type	Brown Fat CLAY (CH)
Source	Native
Initial Moisture Content, %	11.6
Final Moisture Content, %	27.8
Dry Density, pcf	104.5
Initial Saturation, %	51.1
Expansion Index	**106**
Potential Expansion	**HIGH**

Respectfully Submitted,
NV5 West, Inc.

Carl Henderson, PhD, PE, GE
CQA Group Director

Expansion Index Test Report

(ASTM D4829)

Date:	October 18, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6948
Address:	488 8th Avenue	Lab Number:	117008-117009
	San Diego, CA 92101		
Project:	Westside Canal Energy Center		
Project Add:	Imperial Valley, CA		
Sampled By:	Sean Burford		
Date Sampled	10/17/2018		
Date Rcvd:	10/17/2018		

Lab Number	117008	117009
Location	B1 @ 3'-5'	B3/B5 @ 3'-5' Mixture
Material Type	Tan Lean CLAY with Sand (CL)	Tan Silty SAND (SM)
Source	Native	Native
Initial Moisture Content, %	10.2	8.3
Final Moisture Content, %	20.5	16.1
Dry Density, pcf	109.7	116.3
Initial Saturation, %	51.3	49.8
Expansion Index	**50**	**14**
Potential Expansion	**LOW**	**VERY LOW**

Respectfully Submitted,
NV5 West, Inc.

Carl Henderson

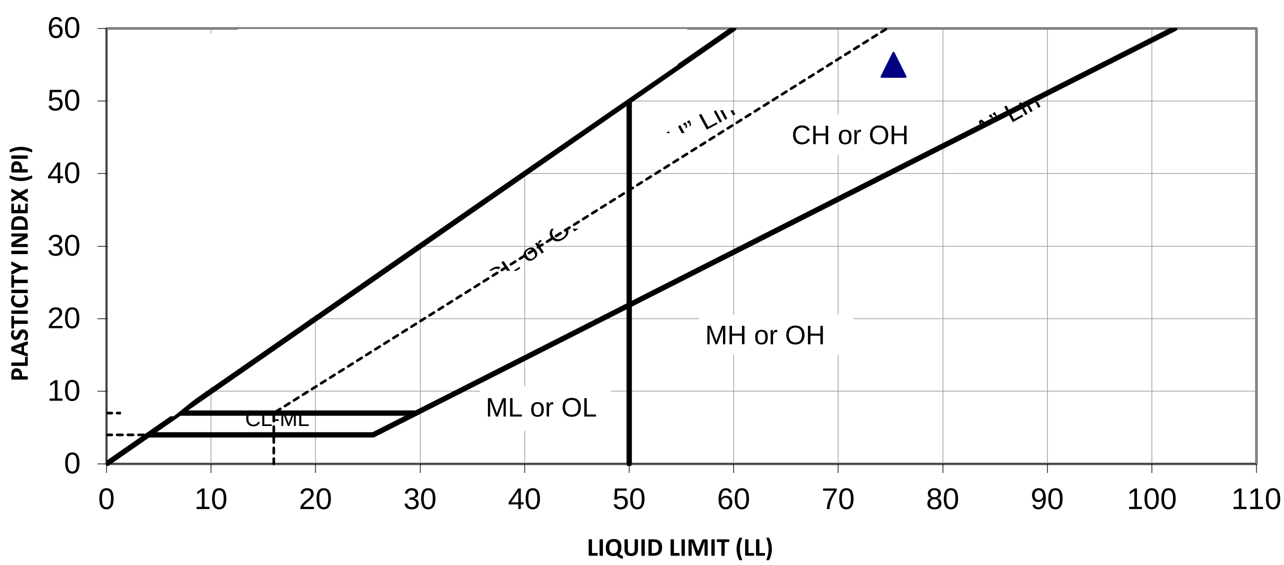
Carl Henderson, PhD, PE, GE
CQA Group Director

REPORT OF LIQUID LIMIT, PLASTIC LIMIT & PLASTICITY INDEX TESTS

(ASTM 4318)

Date:	October 12, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6919
Address:	488 8th Avenue	Lab Number:	116901
	San Diego, CA 92101		

Project:	Westside Canal Energy Center
Project Address:	Imperial Valley, CA
Material:	Brown Fat CLAY (CH)
Location:	B6 @ 10'-11.5'
Date Sampled:	10/1/2018
Date Submitted:	10/2/2018
Sampled By:	Sean Burford
Date Tested:	10/8/2018



SUMMARY OF TEST RESULTS

SAMPLE ID	SOURCE /LOCATION DEPTH	%>#40	TEST RESULT			USCS	
			LL	PL	PI	Class	Group Name
116901	B6 @ 10'-11.5'	NR	**75**	**20**	**55**	CH	Fat CLAY

Reviewed By:

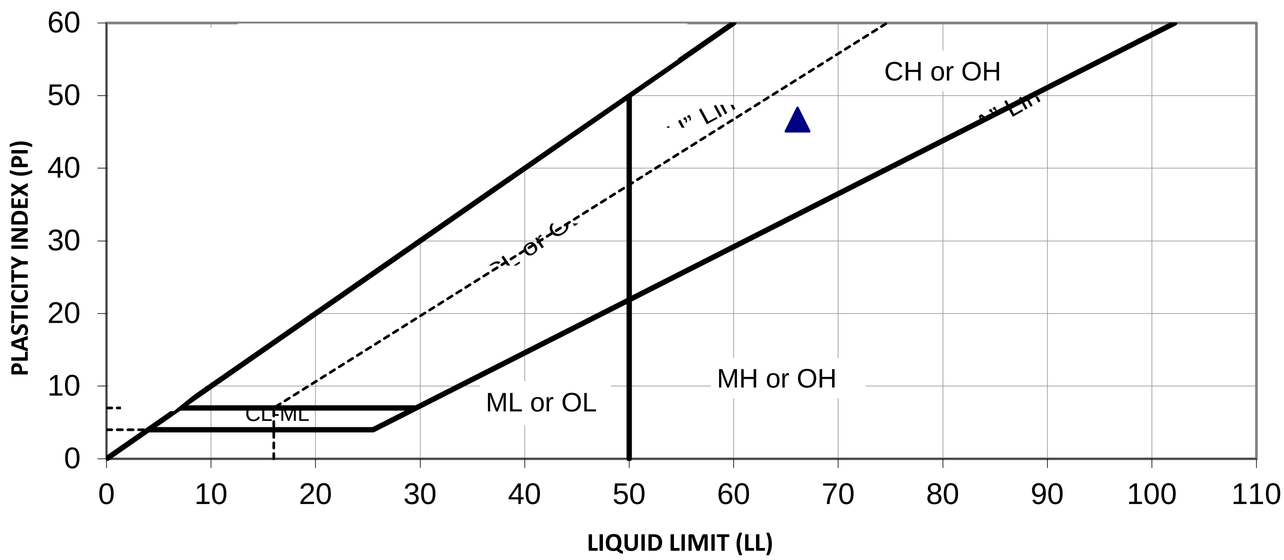
Carl Henderson, PhD, PE, GE
CQA Group Director

REPORT OF LIQUID LIMIT, PLASTIC LIMIT & PLASTICITY INDEX TESTS

(ASTM 4318)

Date:	October 12, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6919
Address:	488 8th Avenue	Lab Number:	116903
	San Diego, CA 92101		

Project:	Westside Canal Energy Center
Project Address:	Imperial Valley, CA
Material:	Brown Fat CLAY (CH)
Location:	B6 @ 20'-21.5'
Date Sampled:	10/1/2018
Date Submitted:	10/2/2018
Sampled By:	Sean Burford
Date Tested:	10/9/2018



SUMMARY OF TEST RESULTS

SAMPLE ID	SOURCE /LOCATION DEPTH	%>#40	TEST RESULT			USCS	
			LL	PL	PI	Class	Group Name
116903	B6 @ 20'-21.5'	NR	**66**	**19**	**47**	CH	Fat CLAY

Reviewed By: Carl Henderson

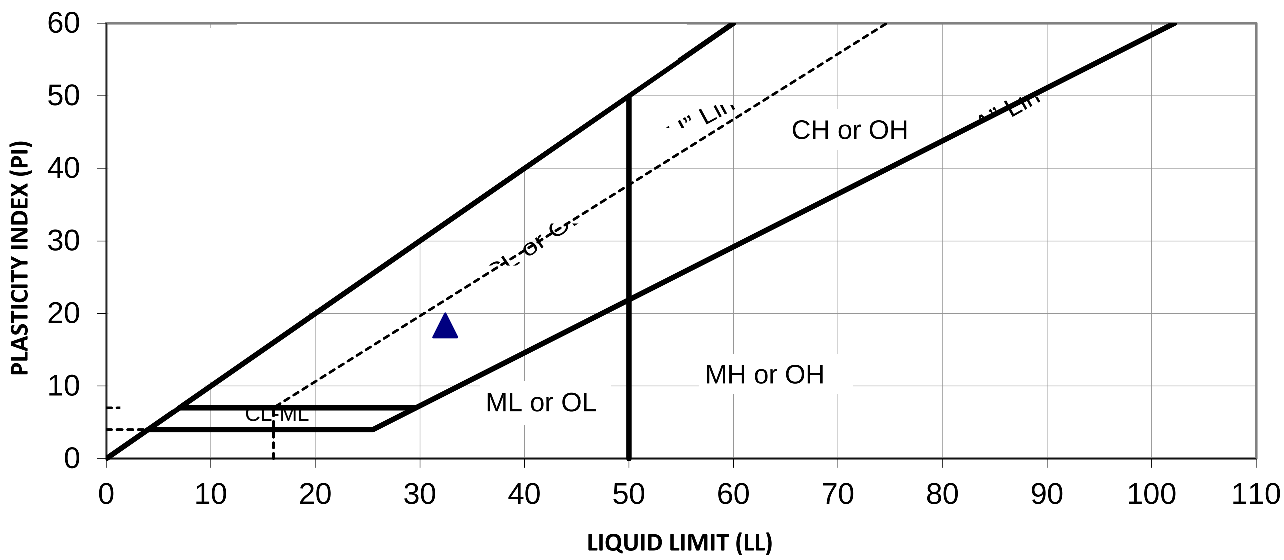
Carl Henderson, PhD, PE, GE
CQA Group Director

REPORT OF LIQUID LIMIT, PLASTIC LIMIT & PLASTICITY INDEX TESTS

(ASTM 4318)

Date:	October 12, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6919
Address:	488 8th Avenue	Lab Number:	116906
	San Diego, CA 92101		

Project:	Westside Canal Energy Center
Project Address:	Imperial Valley, CA
Material:	Brown Lean CLAY with Sand (CL)
Location:	B6 @ 35'-36.5'
Date Sampled:	10/1/2018
Date Submitted:	10/2/2018
Sampled By:	Sean Burford
Date Tested:	10/9/2018



SUMMARY OF TEST RESULTS

SAMPLE ID	SOURCE /LOCATION DEPTH	%>#40	TEST RESULT			USCS	
			LL	PL	PI	Class	Group Name
116906	B6 @ 35'-36.5'	NR	**32**	**14**	**18**	CL	Lean CLAY with Sand

Reviewed By: Carl Henderson

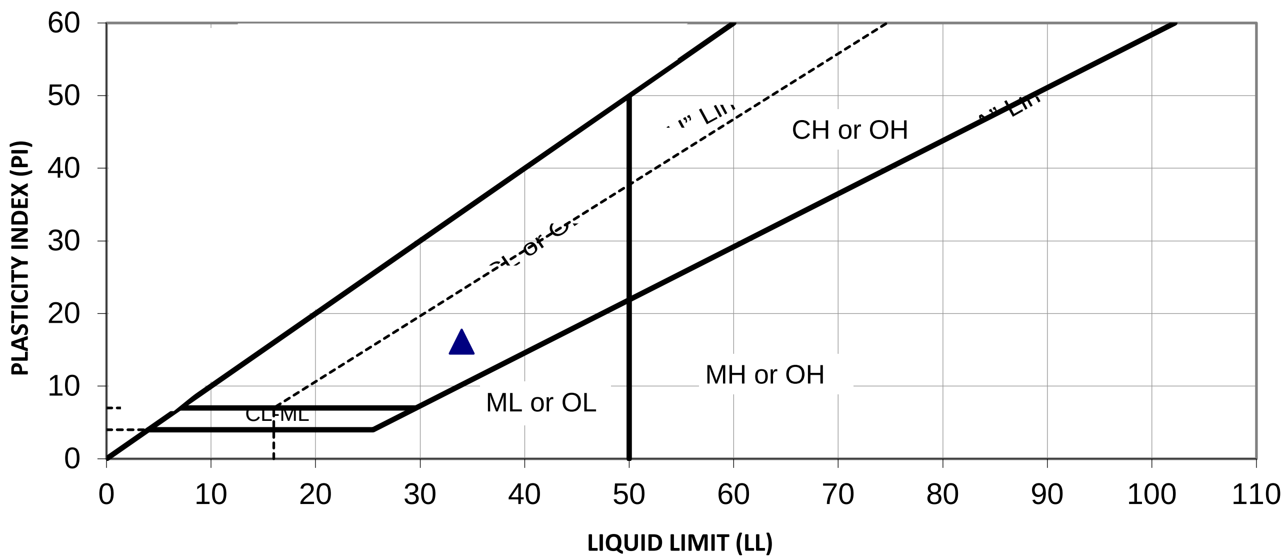
Carl Henderson, PhD, PE, GE
CQA Group Director

REPORT OF LIQUID LIMIT, PLASTIC LIMIT & PLASTICITY INDEX TESTS

(ASTM 4318)

Date:	October 12, 2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6919
Address:	488 8th Avenue	Lab Number:	116908
	San Diego, CA 92101		

Project:	Westside Canal Energy Center
Project Address:	Imperial Valley, CA
Material:	Brown Sandy Lean CLAY (CL)
Location:	B6 @ 45'-46.5'
Date Sampled:	10/1/2018
Date Submitted:	10/2/2018
Sampled By:	Sean Burford
Date Tested:	10/5/2018



SUMMARY OF TEST RESULTS

SAMPLE ID	SOURCE /LOCATION DEPTH	%>#40	TEST RESULT			USCS	
			LL	PL	PI	Class	Group Name
116908	B6 @ 45'-46.5'	NR	**34**	**18**	**16**	CL	Sandy Lean CLAY

Reviewed By: Carl Henderson

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CQA - Infrastructure - Energy - Program Management - Environmental

Client Name: Sempra Renewables

Project: Westside Canal Energy Center

Report Date: 10/11/2018
NV5 Project No.: 1076
Lab Number: 116796
Location: B2 @ 3'-5'

Test Material Description: Soils Thermal Sample #1 (1 of 1), 2.4" x6"
Test Material: Brown Fat CLAY (CH)
Sample Date: 9/17-18/18

Test Description	Test Method	# of Cylinders
Thermal Resistivity Measurement	IEEE 442 / ASTM D5334	1

Probe Type: TR1
Ambient Temperature: 21.6 °C

Results:

Dry Density (pcf)	Tested Max. Thermal Resistivity at 0% Moisture (°C-cm/W)	Max. Thermal Resistivity at 4% Critical Moisture (°C-cm/W)	Tested Thermal Resistivity at Wet Point (°C-cm/W)
108	136	84	71

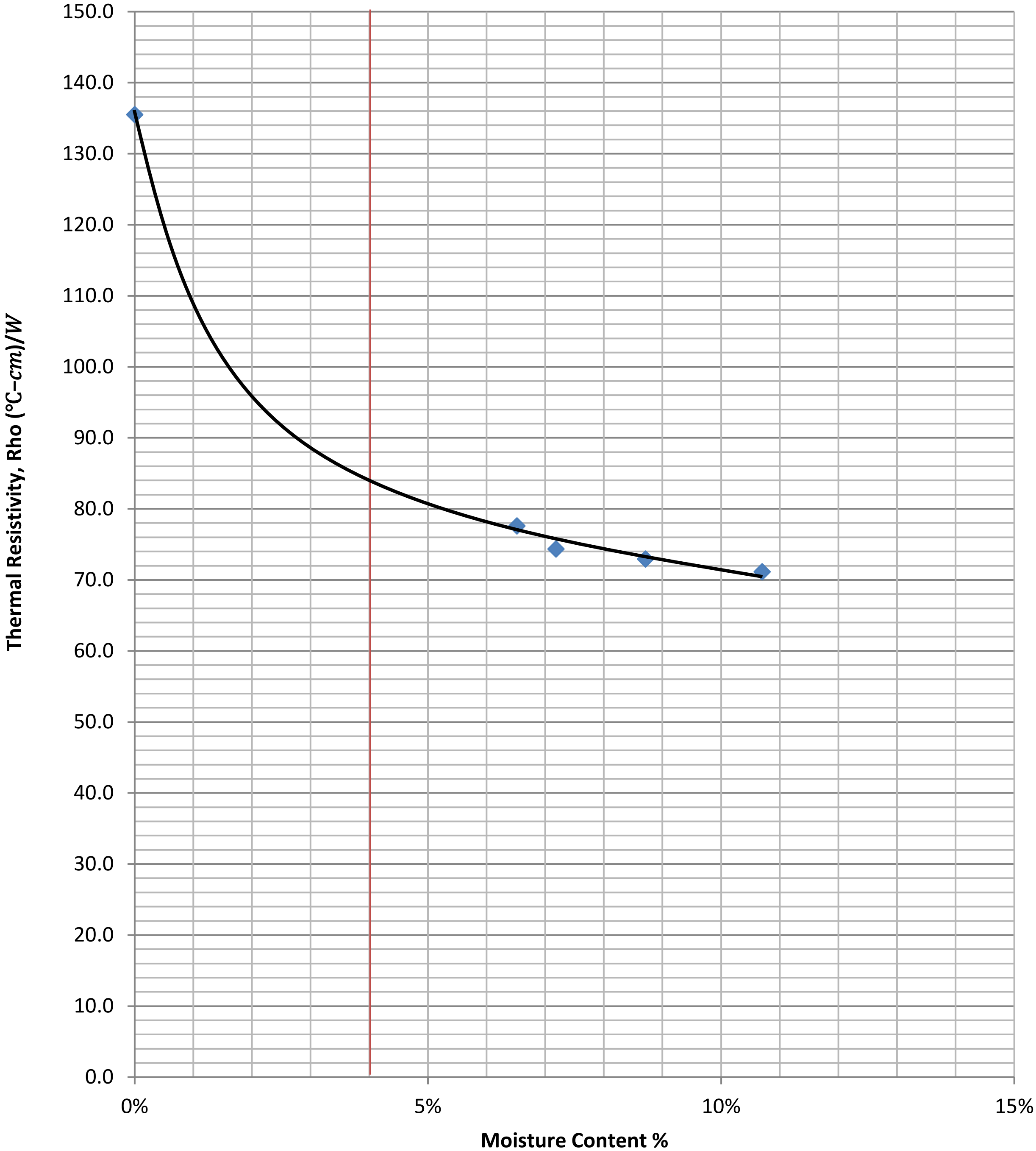
Note: The accuracy of TR-1 Probe is ±10%

Respectfully submitted,
NV5

Carl Henderson

Carl Henderson, PhD,PE,GE
CQA GrouP Director

Thermal Resistivity Dryout Curve



Westside Canal Energy Project

Lab Number: 116796
B2 @ 3'-5'

Client Name: Sempra Renewables

Project: Westside Canal Energy Center

Report Date: 10/11/2018
NV5 Project No.: 1076
Lab Number: 116801
Location: B3 @ 3'-5'

Test Material Description: Soils Thermal Sample #1 (1 of 1), 2.4" x6"
Test Material: Tan Silty SAND (SM)
Sample Date: 9/17-18/18

Test Description	Test Method	# of Cylinders
Thermal Resistivity Measurement	IEEE 442 / ASTM D5334	1

Probe Type: TR1
Ambient Temperature: 21.6 °C

Results:

Dry Density (pcf)	Tested Max. Thermal Resistivity at 0% Moisture (°C-cm/W)	Max. Thermal Resistivity at 4% Critical Moisture (°C-cm/W)	Tested Thermal Resistivity at Wet Point (°C-cm/W)
111	145	70	65

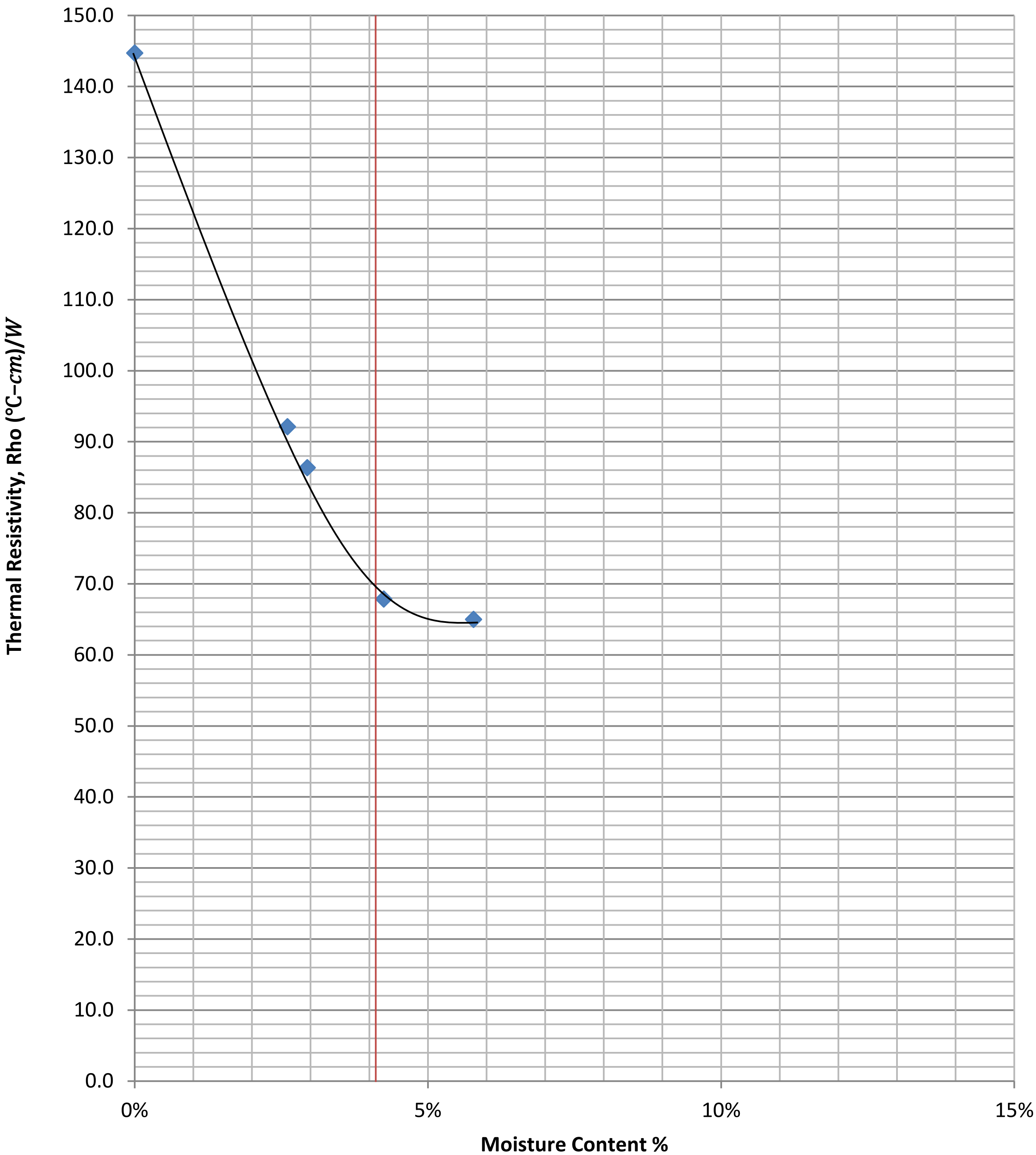
Note: The accuracy of TR-1 Probe is ±10%

Respectfully submitted,
NV5

Carl Henderson

Carl Henderson, PhD,PE,GE
CQA GrouP Director

Thermal Resistivity Dryout Curve



Westside Canal Energy Project

Lab Number: 116801
B3 @ 3'-5'

Client Name: Sempra Renewables

Project: Westside Canal Energy Center

Report Date: 10/11/2018
NV5 Project No.: 1076
Lab Number: 116806
Location: B4 @ 3'-5'

Test Material Description: Soils Thermal Sample #1 (1 of 1), 2.4" x6"
Test Material: Tan Clayey SAND (SC)
Sample Date: 9/17-18/18

Test Description	Test Method	# of Cylinders
Thermal Resistivity Measurement	IEEE 442 / ASTM D5334	1

Probe Type: TR1
Ambient Temperature: 21.6 °C

Results:

Dry Density (pcf)	Tested Max. Thermal Resistivity at 0% Moisture (°C-cm/W)	Max. Thermal Resistivity at 4% Critical Moisture (°C-cm/W)	Tested Thermal Resistivity at Wet Point (°C-cm/W)
110	131	77	66

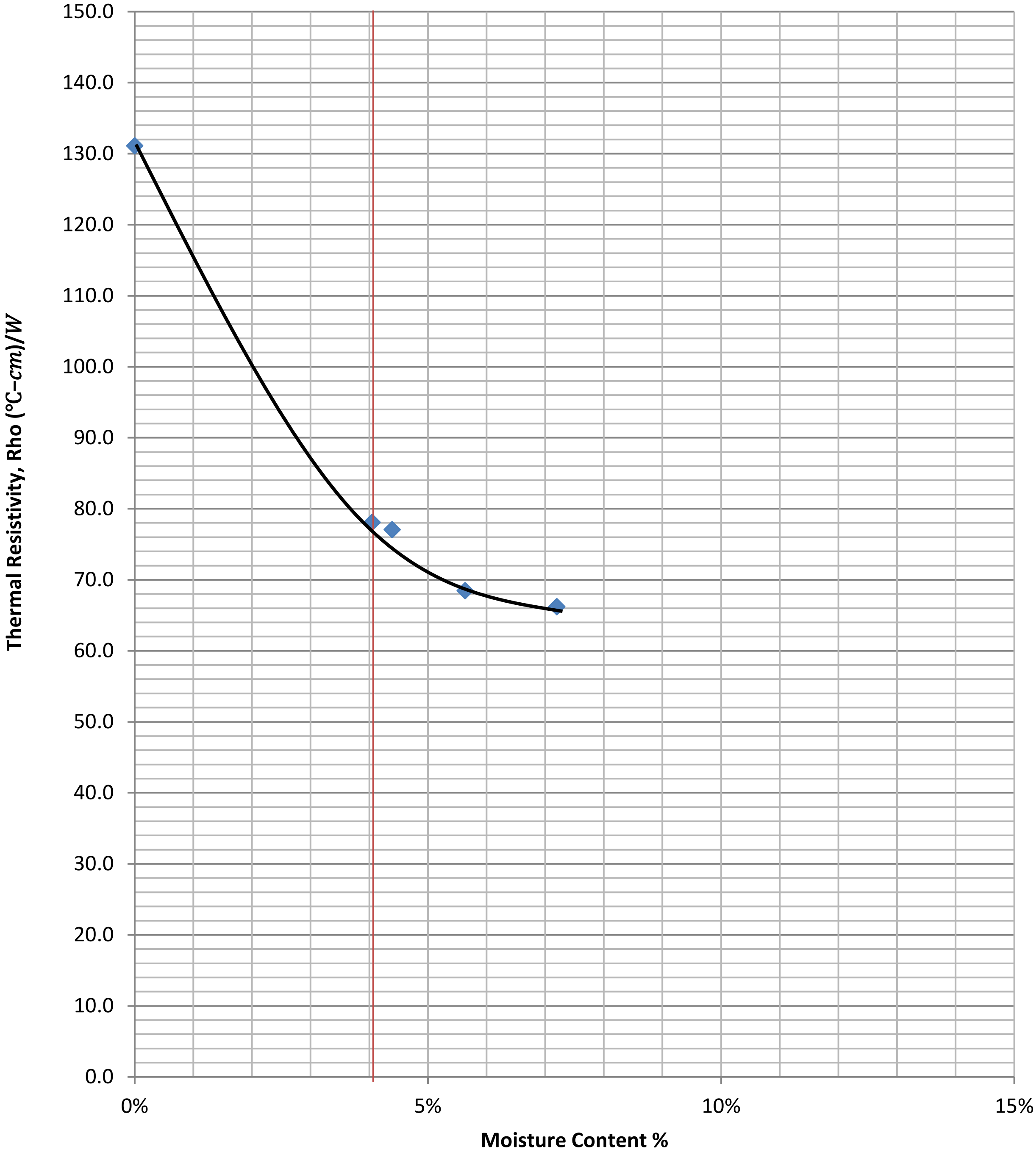
Note: The accuracy of TR-1 Probe is ±10%

Respectfully submitted,
NV5

Carl Henderson

Carl Henderson, PhD,PE,GE
CQA GrouP Director

Thermal Resistivity Dryout Curve



Westside Canal Energy Project

Lab Number: 116806
B4 @ 3'-5'

Client Name: Sempra Renewables

Project: Westside Canal Energy Center

Report Date: 10/18/2018
NV5 Project No.: 1076
Lab Number: 116899
Location: B6 @ 1'-3'

Test Material Description: Soils Thermal Sample #1 (1 of 1), 2.4" x6"
Test Material: Brown Fat CLAY (CH)
Sample Date: 9/17-18/18

Test Description	Test Method	# of Cylinders
Thermal Resistivity Measurement	IEEE 442 / ASTM D5334	1

Probe Type: TR1
Ambient Temperature: 21.6 °C

Results:

Dry Density (pcf)	Tested Max. Thermal Resistivity at 0% Moisture (°C-cm/W)	Max. Thermal Resistivity at 4% Critical Moisture (°C-cm/W)	Tested Thermal Resistivity at Wet Point (°C-cm/W)
104	140	104	75

Note: The accuracy of TR-1 Probe is ±10%

Respectfully submitted,
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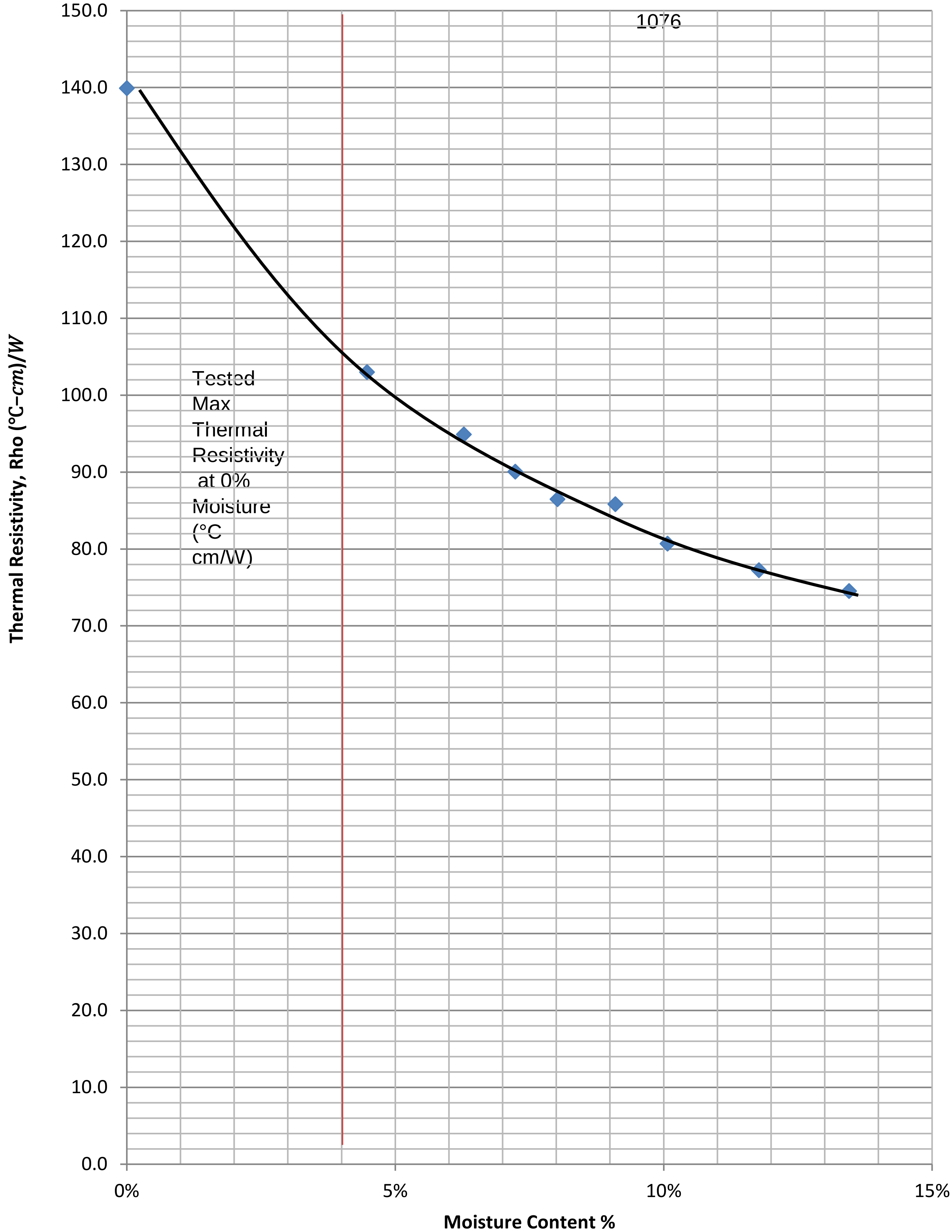
Carl Henderson, PhD,PE,GE
CQA Group Director

Thermal Resistivity Dryout Curve



RESISTANCE "R" VALUE TEST

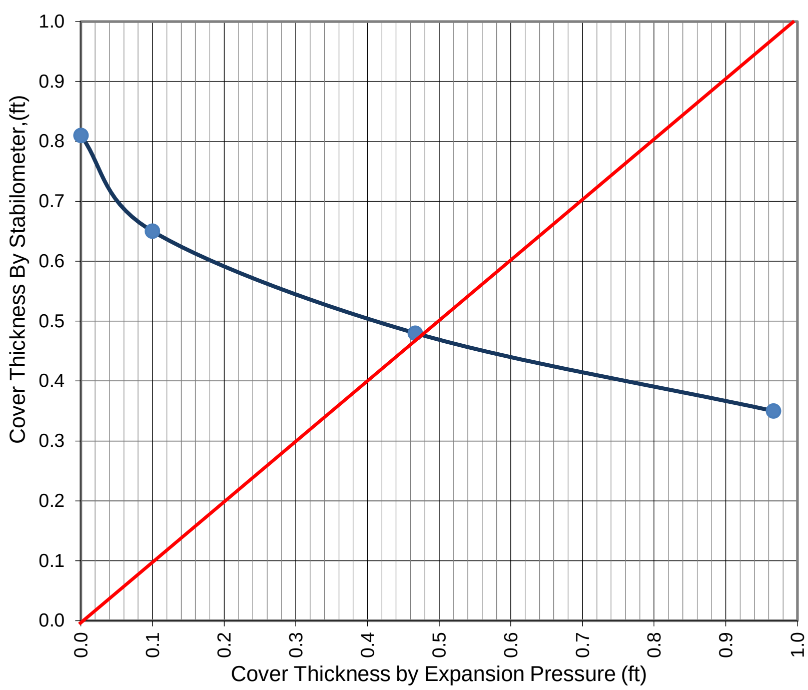
(CTM301 Caltrans / ASTM D2844)

Date:	10/11/2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6881
Address:	488 8th Avenue San Diego, CA 92101	Lab Number:	116801
Project :	Westside Canal Energy Center		
Project Address :	Imperial Valley, CA		

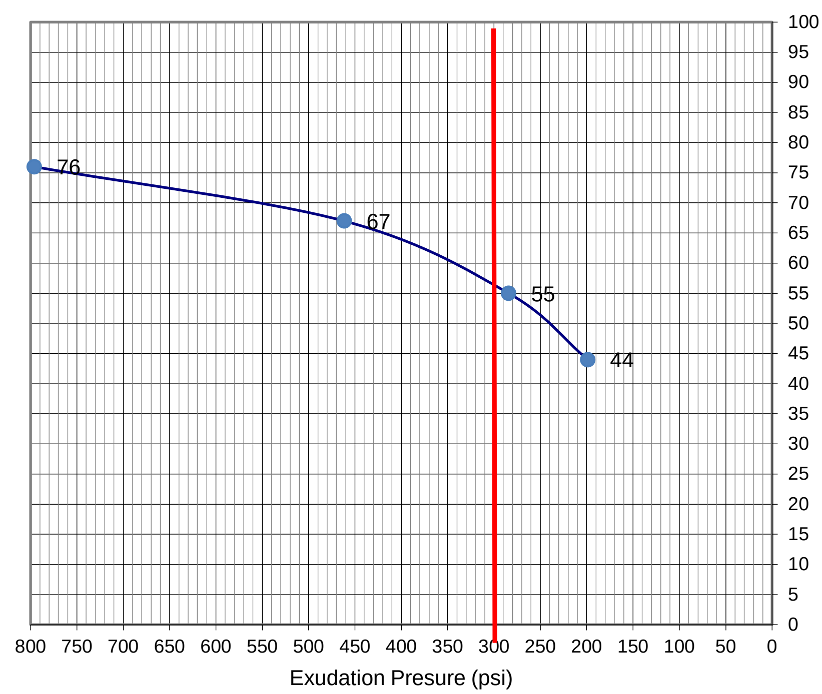
Material:	Tan Silty SAND (SM)
Material Source:	Native
Location:	B3 @ 3'-5'
Sampled By:	Sean Burford
Date Sampled:	9/17-18/2108
Date Received:	9/19/2018

Tested By: Noah Regalado

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



TEST SPECIMEN	A	B	C	D
COMP. FOOT PRESSURE, psi	350	350	350	350
INITIAL MOISTURE %	1.1	1.1	1.1	1.1
MOISTURE @ COMPACTION %	7.8	8.3	8.7	9.1
DRY DENSITY, pcf	128.4	128.8	128.5	128.8
EXUDATION PRESSURE, psi	796	462	284	199
STABILOMETER VALUE 'R'	76	67	55	44

R-VALUE BY EXUDATION	57
R-VALUE BY EXPANSION	67
R-VALUE AT EQUILIBRIUM	**57**

Respectfully Submitted,

NV5 West, Inc.

Reviewed By:

Carl Henderson, PhD, PE, GE
CQA Group Director

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RESISTANCE "R" VALUE TEST

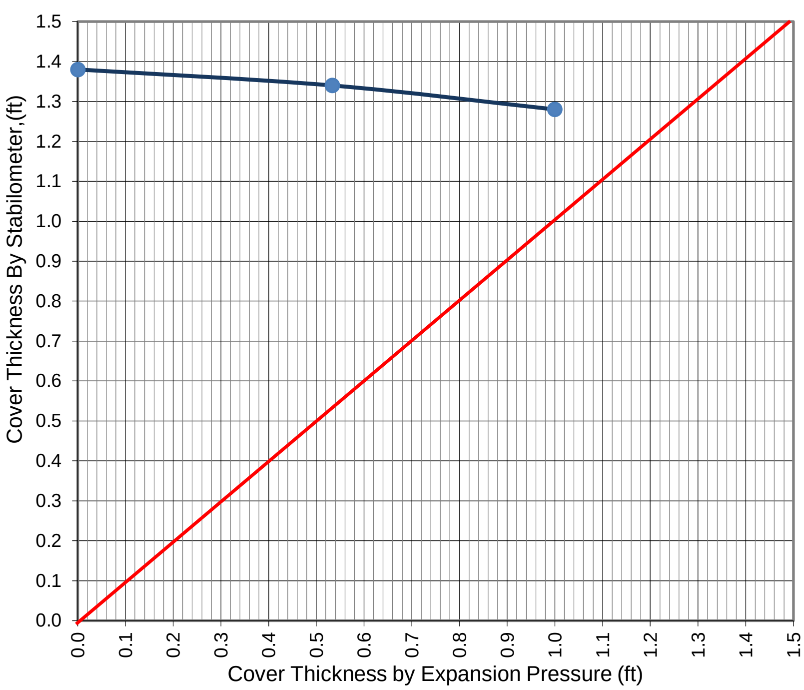
(CTM301 Caltrans / ASTM D2844)

Date:	10/12/2018	Job Number:	1076
Client:	Sempra Renewables	Report Number:	6919
Address:	488 8th Avenue San Diego, CA 92101	Lab Number:	116899
Project :	Westside Canal Energy Center		
Project Address :	Imperial Valley, CA		

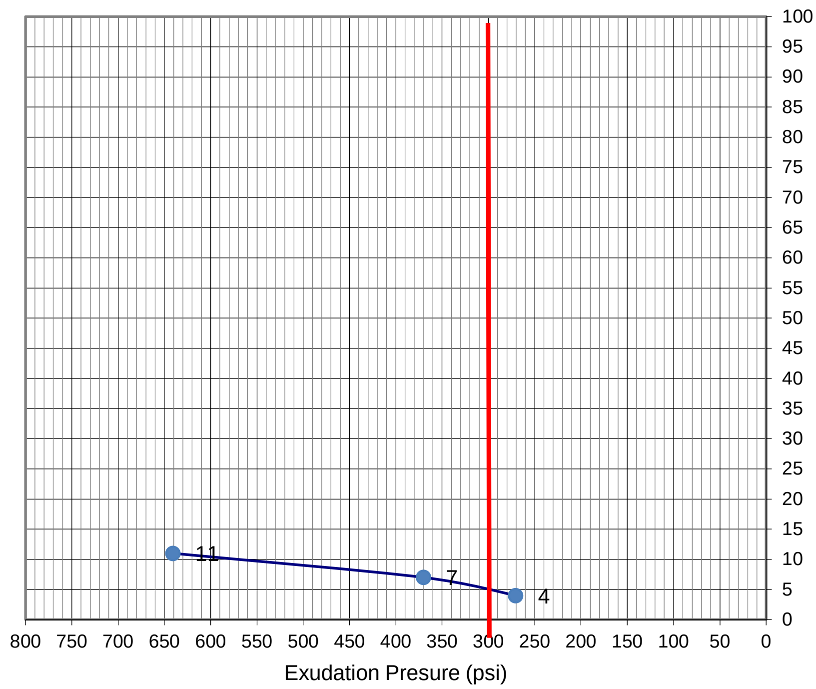
Material:	Brown Fat CLAY (CH)
Material Source:	Native
Location:	B6 @ 1'-3'
Sampled By:	Sean Burford
Date Sampled:	10/1/2018
Date Received:	10/2/2018

Tested By: Noah Regalado

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



TEST SPECIMEN	A	B	C	D
COMP. FOOT PRESSURE, psi	105	90	70	
INITIAL MOISTURE %	7.2	7.2	7.2	
MOISTURE @ COMPACTION %	18.0	21.5	23.3	
DRY DENSITY, pcf	110.3	103.5	100.2	
EXUDATION PRESSURE, psi	641	370	271	
STABILOMETER VALUE 'R'	11	7	4	

R-VALUE BY EXUDATION	5
R-VALUE BY EXPANSION	0
R-VALUE AT EQUILIBRIUM	**5**

Respectfully Submitted,

NV5 West, Inc.

Reviewed By: Carl Henderson

Carl Henderson, PhD, PE, GE
CQA Group Director

DIRECT SHEAR TEST (ASTM D3080)

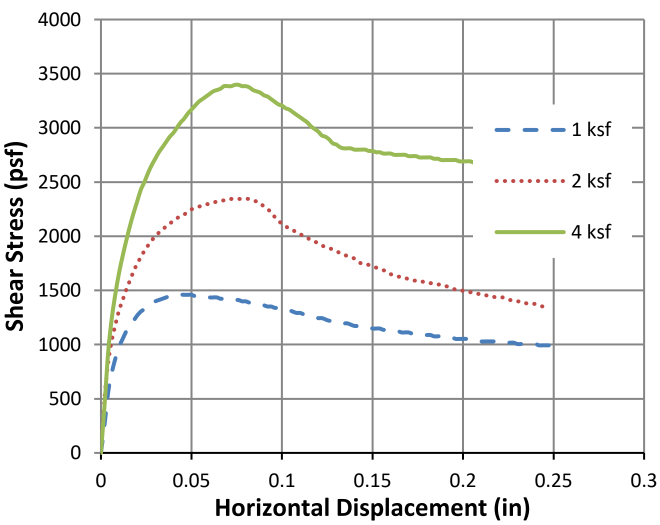
Project No.	**1076**	Date:	**10/12/2018**
Client:	**Sempra Renewables**	Report No.:	**6919**
Proj. Name:	**Westside Canal Energy Center**	Lab No.:	**116900**
Location:	**Imperial Valley, CA**	Date Rcvd:	**10/2/2018**
Sample date:	**10/1/2018**	Test Date:	**10/8/2018**

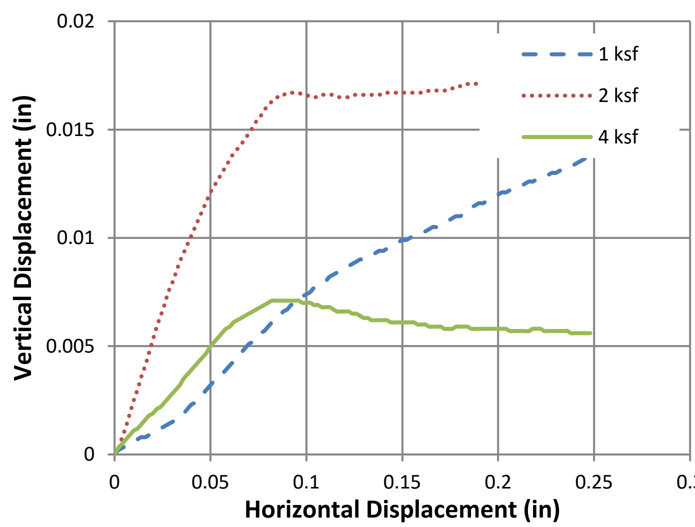
Sample Location: **6'-6.5'** Boring No. **B6**

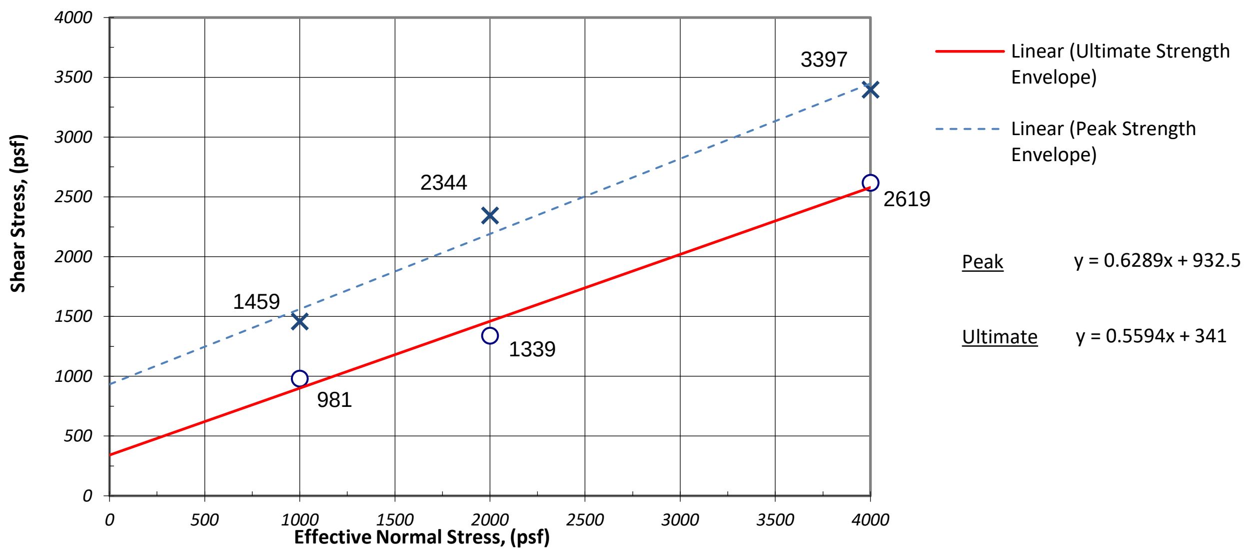
TEST DATA:

		1 ksf	2 ksf	4 ksf
	Sample ID:	1 ksf	2 ksf	4 ksf
Initial	Water Content (%)	24.1	24.1	24.1
Initial	Dry Density	99.8	99.4	100.1
Initial	Saturation (%)	75.9	75.3	76.4
Final	Water Content (%)	33.4	29.9	30.2
Final	Dry Density	96.9	97.9	98.1
Final	Saturation (%)	99.4	90.7	92.0
Normal Stress (psf)		1000	2000	4000
Ultimate Shear Stress (psf)		981	1339	2619
Peak Shear Stress (psf)		1459	2344	3397

Sample Type:	Relatively Undisturbed Sample
Description:	Fat CLAY (CH)
Color:	Brown
Tested By:	Darrel Delgado







Peak Cohesion, C'(psf): **933**	Ultimate Cohesion, C'(psf): **341**
Peak Friction,Φ' (deg): **32**	Ultimate Friction,Φ' (deg): **29**

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15092 Avenue of Science, Ste 200
San Diego CA 92128
p. 858 385 0500 f. 858 715 5810

Respectfully Submitted,
NV5 West, Inc.

Carl Henderson, PhD, PE, GE
CQA Group Director

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Date: October 3, 2018
Purchase Order Number: 18-0476
Sales Order Number: 41787
Account Number: NV5-SD

To:

NV5 West Inc
15092 Avenue of Science #200
San Diego, CA 92128
Attention: Michelle Albrecht

Laboratory Number: SO7038   Customers Phone: 858-715-5800
Fax: 858-715-5810

Sample Designation:

One soil sample received on 10/02/18 at 1:00pm,
taken from Westside Canal Energy Project Lab#116801
Report#6881 marked as B-3,3-5'.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 9.3

Water Added (ml)	Resistivity (ohm-cm)
10	2200
5	1100
5	980
5	820
5	820
5	850
5	850
5	870

28 years to perforation for a 16 gauge metal culvert.
37 years to perforation for a 14 gauge metal culvert.
51 years to perforation for a 12 gauge metal culvert.
65 years to perforation for a 10 gauge metal culvert.
79 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.042% (420ppm)
Water Soluble Chloride	Calif. Test 422	0.013% (130ppm)

Laura Torres

Laura Torres
LT/ilv

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Date: October 10, 2018
Purchase Order Number: 18-0478
Sales Order Number: 41838
Account Number: NV5-SD

To:
--
NV5 West Inc
15092 Avenue of Science #200
San Diego, CA 92128
Attention: Michelle Albrecht

Laboratory Number: SO7049   Customers Phone: 858-715-5800
Fax: 858-715-5810

Sample Designation:
--
One soil sample received on 10/05/18 at 1:00pm,
taken on from Westside Canal Energy Project
marked as Lab#116899 Report#6919 B-6@1-3'.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.5

Water Added (ml)	Resistivity (ohm-cm)
10	1800
5	550
5	170
5	130
5	120
5	120
5	130
5	150

13 years to perforation for a 16 gauge metal culvert.
17 years to perforation for a 14 gauge metal culvert.
23 years to perforation for a 12 gauge metal culvert.
29 years to perforation for a 10 gauge metal culvert.
36 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.231% (2310ppm)

Water Soluble Chloride Calif. Test 422   0.214% (2140ppm)

Laura Torres

Laura Torres
LT/ilv

APPENDIX D

Liquefaction Analysis Results

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Westside Canal Energy Center
Project No.	1076
Project Location	Imperial Valley, California
Analyzed By	Carlos Amante
Reviewed By	Carl Henderson

SELECTED METHODS OF ANALYSIS

Analysis Description	Analysis for Borings B-1/B-1a
Triggering of Liquefaction	Boulanger-Idriss (2014)
Severity of Liquefaction	LPI: Liquefaction Potential Index based on Iwasaki et al. (1978)
Seismic Compression Settlement (Dry/Unsaturated Soil)	Pradel (1998)
Liquefaction-Induced Settlement (Saturated Soil)	Ishihara and Yoshimine (1992)
Liquefaction-Induced Lateral Spreading	Zhang et al. (2004)
Residual Shear Strength of Liquefied Soil	Idriss and Boulanger (2008)

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	6.50
Peak Ground Acceleration, A_{max}	0.50 g
Required Factor of Safety, FS	1.20

BORING DATA AND SITE CONDITIONS

Boring No.	B-1/B-1A
Ground Surface Elevation	-21.0 feet
Proposed Grade Elevation	-21.0 feet
GWL Depth Measured During Test	9.0 feet
GWL Depth Used in Design	5.0 feet
Borehole Diameter	6.0 inches
Hammer Weight	140.0 pounds
Hammer Drop	30.0 inches
Hammer Energy Efficiency Ratio, ER (%)	80.0 %
Hammer Distance to Ground Surface	5.0 feet
Topographic Site Condition:	TSC3 (Level Ground with Nearby Free Face)
- Ground Slope, S (%)	<<= Leave this blank
- Free Face Distance to Height Ratio, (L/H)	1.00 <<= Enter (L/H) Enter H =>> 10.0 feet
Average Total Unit Weight of New Fill	120.0 pcf

INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Screening *Susceptible Soil? (Y, N)*	Total Unit Weight γ_t (pcf)	Field SPT Blow Count N_{field} (blows/ft)	Type of Soil Sampler	Fines Content FC (%)
0.0	10.00	CL	N	120.0			
10.0	15.00	CL	N	120.0			
15.0	18.00	CL	N	120.0			
18.0	25.00	SM	Y	120.0	18.0	SPT1	15.0
25.0	30.00	SM	Y	120.0	37.0	SPT1	15.0
30.0	35.00	SP-SM	Y	120.0	44.0	SPT1	7.0
35.0	40.00	SP-SM	Y	120.0	38.0	SPT1	7.0
40.0	45.00	SP-SM	Y	120.0	47.0	SPT1	7.0
45.0	50.00	SP-SM	Y	120.0	83.0	SPT1	7.0
50.0	55.00	SP-SM	Y	120.0	46.0	SPT1	6.0
55.0	60.00	SP-SM	Y	120.0	83.0	SPT1	6.0
60.0	65.00	SP-SM	Y	120.0	46.0	SPT1	6.0

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Westside Canal Energy Center
Project No.	1076
Project Location	Imperial Valley, California
Analyzed By	Carlos Amante
Reviewed By	Carl Henderson

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	6 50
Peak Ground Acceleration, A_{max}	0 50 g
Required Factor of Safety, FS	1 20

BORING DATA AND SITE CONDITIONS

Boring No.	B-1/B-1A
Ground Surface Elevation	-21 0 feet
Proposed Grade Elevation	-21 0 feet
GWL Depth Measured During Test	9 0 feet
GWL Depth Used in Design	5 0 feet
Borehole Diameter	6 0 inches
Hammer Weight	140 0 pounds
Hammer Drop	30 0 inches
Hammer Energy Efficiency Ratio, ER	80 0 %
Hammer Distance to Ground Surface	5 0 feet
Topographic Site Condition:	TSC3 (Level Ground with Nearby Free Face)
- Ground Slope, S	N/A
- Free Face (L/H) Ratio	1 0 H = 10 feet
Average Total Unit Weight of New Fill	120 0 pcf

SUMMARY OF RESULTS

Severity of Liquefaction:

Total Thickness of Liquefiable Soils, H_{liq}: 7 00 feet (cumulative total thickness in the upper 65 feet)

Liquefaction Potential Index (LPI): 1 50 *** (Low risk, with minor liquefaction effects)

Seismic Ground Settlements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Seismic Compression Settlement:	Pradel (1998)	0 00 inches	0 00 inches	0 00 inches	(Dry/Unsaturated Soils)
Liquefaction-Induced Settlement:	Ishihara and Yoshimine (1992)	0 28 inches	0 28 inches	0 28 inches	(Saturated Soils)
Total Seismic Settlement:		0 28 inches	0 28 inches	0 28 inches	

Seismic Lateral Displacements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0 25 inches	0 25 inches	0 25 inches	(During Ground Shaking)
Lateral Spreading Displacement:	Zhang et al (2004)	0 00 inches	0 00 inches	0 00 inches	(After Ground Shaking)

NOTES AND REFERENCES

\+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs} = f\{(N_1)_{60}, FC\}$ where $(N_1)_{60} = N_{field}\, C_N\, C_E\, C_B\, C_R\, C_S$

++ Liquefaction susceptibility screening is performed to identify soil layers assessed to be non-liquefiable based on laboratory test results using the criteria proposed by Cetin and Seed (2003), Bray and Sancio (2006), or Idriss and Boulanger (2008)

* FS_{liq} = Factor of Safety against liquefaction = (CRR/CSR), where CRR = $CRR_{7\ 5}$ MSF K_σ K_α, MSF = Magnitude Scaling Factor, $K_\sigma = f[(N_1)_{60}, \sigma'_{vo}]$, K_α =1 0, (level ground), CSR = Cyclic Stress Ratio = 0 65 A_{max} $(\sigma_{vo}/\sigma'_{vo})$ r_d, and $CRR_{7\ 5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude M_w of 7 5

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008)

*** Based on Iwasaki et al (1978) and Toprak and Holzer (2003)

\+ Reference: Boulanger, R W and Idriss, I M (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No UCD/CGM-14/01, 1-134

INPUT SOIL PROFILE DATA

Bottom of Soil Layer Elevation (feet)	Soil Depth During Test (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Field SPT Blow Count N_{field} (blows/ft)	Type of Soil Sampler	Fines Content FC (%)
-31 0	5 0	CL	N	120 0			
-36 0	12 5	CL	N	120 0			
-39 0	16 5	CL	N	120 0			
-46 0	21 5	SM	Y	120 0	18 0	SPT1	15 0
-51 0	27 5	SM	Y	120 0	37 0	SPT1	15 0
-56 0	32 5	SP-SM	Y	120 0	44 0	SPT1	7 0
-61 0	37 5	SP-SM	Y	120 0	38 0	SPT1	7 0
-66 0	42 5	SP-SM	Y	120 0	47 0	SPT1	7 0
-71 0	47 5	SP-SM	Y	120 0	83 0	SPT1	7 0
-76 0	52 5	SP-SM	Y	120 0	46 0	SPT1	6 0
-81 0	57 5	SP-SM	Y	120 0	83 0	SPT1	6 0
-86 0	62 5	SP-SM	Y	120 0	46 0	SPT1	6 0

LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +

Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. For Vert. Stress C_N	SPT Corr. For Hammer Energy C_E	SPT Corr. For Borehole Size C_B	SPT Corr. For Rod Length C_R	SPT Corr. For Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)
600 0	444 0									0 989		0 434			NL: Clay rich Soil			0 28	0 25	0 00
1500 0	1032 0									0 953		0 450			NL: Clay rich Soil			0 28	0 25	0 00
1980 0	1262 4									0 932		0 475			NL: Clay rich Soil			0 28	0 25	0 00
2580 0	1550 4	1 043	1 333	1 050	0 950	1 000	23 9	25 0	28 2	0 902	1 017	0 488	0 535	1 095	LIQUEFY	449 1	81 4	0 28	0 25	0 00
3300 0	1896 0	0 983	1 333	1 050	0 950	1 000	49 2	48 4	51 6	0 865	0 979	0 489			NL: Dense Soil			0 00	0 00	0 00
3900 0	2184 0	0 963	1 333	1 050	1 000	1 000	61 6	59 3	59 5	0 832	0 941	0 483			NL: Dense Soil			0 00	0 00	0 00
4500 0	2472 0	0 927	1 333	1 050	1 000	1 000	53 2	49 3	49 5	0 799	0 908	0 473			NL: Dense Soil			0 00	0 00	0 00
5100 0	2760 0	0 928	1 333	1 050	1 000	1 000	65 8	61 0	61 2	0 766	0 877	0 460			NL: Dense Soil			0 00	0 00	0 00
5700 0	3048 0	1 028	1 333	1 050	1 000	1 000	116 2	119 5	119 6	0 734	0 850	0 446			NL: Dense Soil			0 00	0 00	0 00
6300 0	3336 0	0 888	1 333	1 050	1 000	1 000	64 4	57 2	57 2	0 703	0 825	0 432			NL: Dense Soil			0 00	0 00	0 00
6900 0	3624 0	1 041	1 333	1 050	1 000	1 000	116 2	121 0	121 0	0 673	0 802	0 417			NL: Dense Soil			0 00	0 00	0 00
7500 0	3912 0	0 855	1 333	1 050	1 000	1 000	64 4	55 0	55 1	0 646	0 780	0 402			NL: Dense Soil			0 00	0 00	0 00

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Westside Canal Energy Center
Project No.	1076
Project Location	Imperial Valley, California
Analyzed By	Carlos Amante
Reviewed By	Carl Henderson

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	6 50
Peak Ground Acceleration, A_{max}	0 50 g
Required Factor of Safety, FS	1 20

BORING DATA AND SITE CONDITIONS

Boring No.	B-1/B-1A	
Ground Surface Elevation	-21 0 feet	
Proposed Grade Elevation	-21 0 feet	
GWL Depth Measured During Test	9 0 feet	
GWL Depth Used in Design	5 0 feet	
Borehole Diameter	6 0 inches	
Hammer Weight	140 0 pounds	
Hammer Drop	30 0 inches	
Hammer Energy Efficiency Ratio, ER	80 0 %	
Hammer Distance to Ground Surface	5 0 feet	
Topographic Site Condition:	TSC3 (Level Ground with Nearby Free Face)	
- Ground Slope, S	N/A	
- Free Face (L/H) Ratio	1 0	H = 10 feet
Average Total Unit Weight of New Fill	120 0 pcf	

SUMMARY OF RESULTS

Severity of Liquefaction:

Total Thickness of Liquefiable Soils, H_{liq}: 7 00 feet (cumulative total thickness in the upper 65 feet)

Liquefaction Potential Index (LPI): 1 50 *** (Low risk, with minor liquefaction effects)

Seismic Ground Settlements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Seismic Compression Settlement:	Pradel (1998)	0 00 inches	0 00 inches	0 00 inches	(Dry/Unsaturated Soils)
Liquefaction-Induced Settlement:	Ishihara and Yoshimine (1992)	0 28 inches	0 28 inches	0 28 inches	(Saturated Soils)
Total Seismic Settlement:		0 28 inches	0 28 inches	0 28 inches	

Seismic Lateral Displacements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0 25 inches	0 25 inches	0 25 inches	(During Ground Shaking)
Lateral Spreading Displacement:	Zhang et al (2004)	0 00 inches	0 00 inches	0 00 inches	(After Ground Shaking)

NOTES AND REFERENCES

\+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs} = f\{(N_1)_{60}, FC\}$ where $(N_1)_{60} = N_{field}\, C_N\, C_E\, C_B\, C_R\, C_S$

++ Liquefaction susceptibility screening is performed to identify soil layers assessed to be non-liquefiable based on laboratory test results using the criteria proposed by Cetin and Seed (2003), Bray and Sancio (2006), or Idriss and Boulanger (2008)

* FS_{liq} = Factor of Safety against liquefaction = (CRR/CSR), where CRR = $CRR_{7\,5}$ MSF K_σ K_α, MSF = Magnitude Scaling Factor, $K_\sigma = f[(N_1)_{60}, \sigma'_{vo}]$, K_α =1 0, (level ground), CSR = Cyclic Stress Ratio = 0 65 A_{max} $(\sigma_{vo}/\sigma'_{vo})$ r_d, and $CRR_{7\,5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude M_w of 7 5

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008)

*** Based on Iwasaki et al (1978) and Toprak and Holzer (2003)

\+ Reference: Boulanger, R W and Idriss, I M (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No UCD/CGM-14/01, 1-134

INPUT SOIL PROFILE DATA								LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +																				
Bottom of Soil Layer Elevation (feet)	Soil Depth During Test (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Field SPT Blow Count N_{field} (blows/ft)	Type of Soil Sampler	Fines Content FC (%)	Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. For Vert. Stress C_N	SPT Corr. For Hammer Energy C_E	SPT Corr. For Borehole Size C_B	SPT Corr. For Rod Length C_R	SPT Corr. For Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)

REFERENCES:

1 Boulanger, R W and Idriss, I M (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No UCD/CGM-14/01, 1-134
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SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

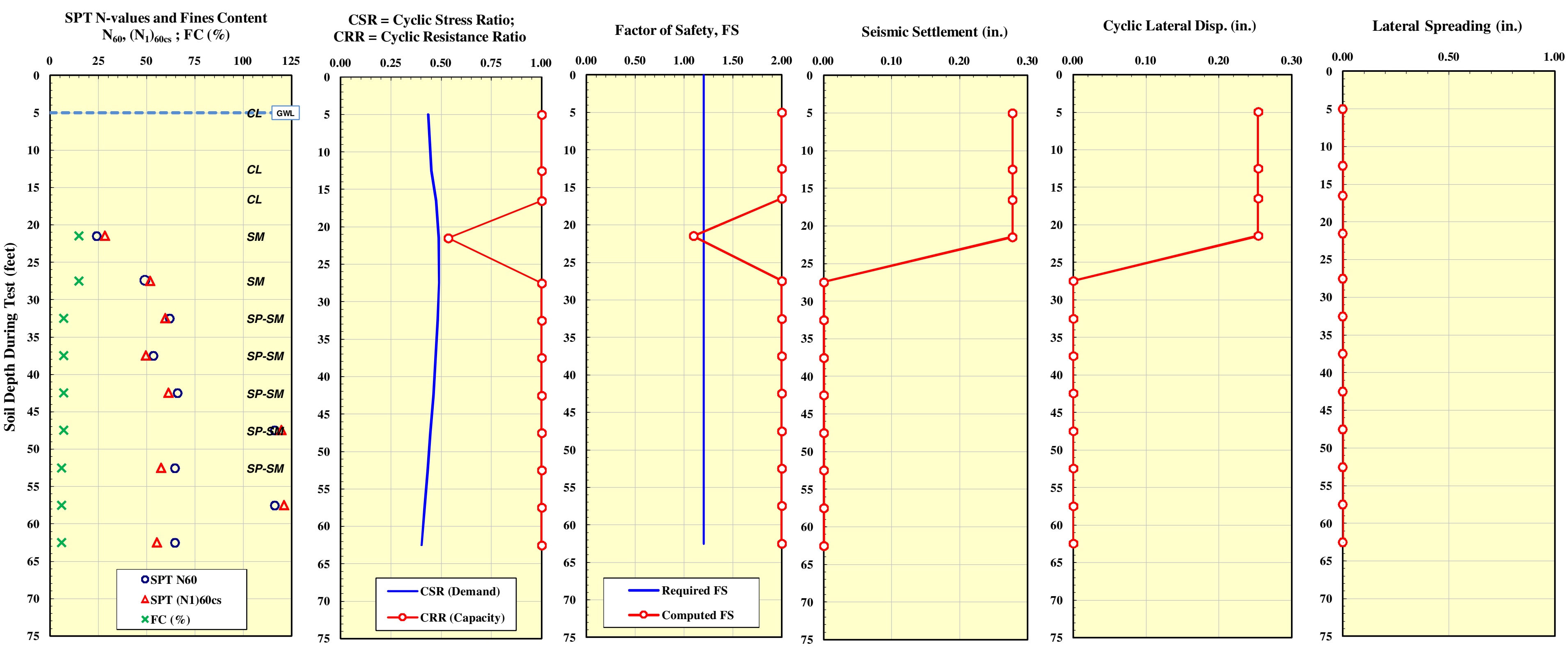
PROJECT INFORMATION	
Project Name	Westside Canal Energy Center
Project No.	1076
Project Location	Imperial Valley, California
Analyzed By	Carlos Amante
Reviewed By	Carl Henderson

TOPOGRAPHIC CONDITIONS	
Ground Slope, S	N/A
Free Face (L/H) Ratio	1.00

GROUNDWATER LEVEL DATA	
GWL Depth Measured During Test	9.00 feet
GWL Depth Used in Design	5.00 feet

BORING DATA	
Boring No.	B-1/B-1A
Ground Surface Elevation	-21.00 feet
Proposed Grade Elevation	-21.00 feet
Borehole Diameter	6.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	80.00 %
Hammer Distance to Ground Surface	5.00 feet

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	6.50
Peak Ground Acceleration, A_{max}	0.50 g
Required Factor of Safety, FS	1.20



Analysis Methods Used ==>>

Liquefaction Triggering:
Boulanger-Idriss (2014)

Seismic Settlements:
Above GWL: Pradel (1998)
Below GWL: Ishihara and Yoshimine (1992)

Cyclic Lateral Displacements:
Above GWL: Pradel (1998)
Below GWL: Tokimatsu and Asaka (1998)

Lateral Spreading:
Zhang et al. (2004)

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION	
Project Name	Westside Canal Energy Center
Project No.	1076
Project Location	Imperial Valley, California
Analyzed By	Carlos Amante
Reviewed By	Carl Henderson

SELECTED METHODS OF ANALYSIS	
Analysis Description	Analysis for Boring B-6
Triggering of Liquefaction	Boulanger-Idriss (2014)
Severity of Liquefaction	LPI: Liquefaction Potential Index based on Iwasaki et al. (1978)
Seismic Compression Settlement (Dry/Unsaturated Soil)	Pradel (1998)
Liquefaction-Induced Settlement (Saturated Soil)	Ishihara and Yoshimine (1992)
Liquefaction-Induced Lateral Spreading	Zhang et al. (2004)
Residual Shear Strength of Liquefied Soil	Idriss and Boulanger (2008)

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	6.50
Peak Ground Acceleration, A_{max}	0.50 g
Required Factor of Safety, FS	1.20

BORING DATA AND SITE CONDITIONS			
Boring No.	B-6		
Ground Surface Elevation	-17.0 feet		
Proposed Grade Elevation	-17.0 feet		
GWL Depth Measured During Test	18.0 feet		
GWL Depth Used in Design	5.0 feet		
Borehole Diameter	6.0 inches		
Hammer Weight	140.0 pounds		
Hammer Drop	30.0 inches		
Hammer Energy Efficiency Ratio, ER (%)	80.0 %		
Hammer Distance to Ground Surface	5.0 feet		
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)		
- Ground Slope, S (%)	<<= Leave this blank		
- Free Face Distance to Height Ratio, (L/H)	<<= Leave this blank	Set H to zero =>>	0.0 feet
Average Total Unit Weight of New Fill	120.0 pcf		

INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Screening *Susceptible Soil? (Y, N)*	Total Unit Weight γ_t (pcf)	Field SPT Blow Count N_{field} (blows/ft)	Type of Soil Sampler	Fines Content FC (%)
0.0	2.00	CL	N	120.0			
2.0	10.00	CH	N	120.0			
10.0	15.00	CH	N	120.0			
15.0	20.00	CH	N	120.0			
20.0	25.00	CH	N	120.0			
25.0	29.50	CH	N	120.0			
29.5	36.00	SM	Y	120.0	38.0	SPT1	15.0
36.0	39.00	CL	N	120.0			
39.0	41.00	ML	N	120.0			
41.0	43.00	CL	N	120.0			
43.0	50.00	CL	N	120.0			
50.0	51.50	CL	N	120.0			

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Westside Canal Energy Center
Project No.	1076
Project Location	Imperial Valley, California
Analyzed By	Carlos Amante
Reviewed By	Carl Henderson

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	6 50
Peak Ground Acceleration, A_{max}	0 50 g
Required Factor of Safety, FS	1 20

BORING DATA AND SITE CONDITIONS

Boring No.	B-6
Ground Surface Elevation	-17 0 feet
Proposed Grade Elevation	-17 0 feet
GWL Depth Measured During Test	18 0 feet
GWL Depth Used in Design	5 0 feet
Borehole Diameter	6 0 inches
Hammer Weight	140 0 pounds
Hammer Drop	30 0 inches
Hammer Energy Efficiency Ratio, ER	80 0 %
Hammer Distance to Ground Surface	5 0 feet
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)
- Ground Slope, S	0 0 %
- Free Face (L/H) Ratio	N/A H = 0 feet
Average Total Unit Weight of New Fill	120 0 pcf

SUMMARY OF RESULTS

Severity of Liquefaction:

Total Thickness of Liquefiable Soils, H_{liq}: 0 00 feet (cumulative total thickness in the upper 65 feet)

Liquefaction Potential Index (LPI): 0 00 *** (Very low risk, with no surface manifestation of liquefaction)

Seismic Ground Settlements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Seismic Compression Settlement:	Pradel (1998)	0 00 inches	0 00 inches	0 00 inches	(Dry/Unsaturated Soils)
Liquefaction-Induced Settlement:	Ishihara and Yoshimine (1992)	0 00 inches	0 00 inches	0 00 inches	(Saturated Soils)
Total Seismic Settlement:		0 00 inches	0 00 inches	0 00 inches	

Seismic Lateral Displacements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0 00 inches	0 00 inches	0 00 inches	(During Ground Shaking)
Lateral Spreading Displacement:	Zhang et al (2004)	0 00 inches	0 00 inches	0 00 inches	(After Ground Shaking)

NOTES AND REFERENCES

\+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs} = f\{(N_1)_{60}, FC\}$ where $(N_1)_{60} = N_{field}\, C_N\, C_E\, C_B\, C_R\, C_S$

++ Liquefaction susceptibility screening is performed to identify soil layers assessed to be non-liquefiable based on laboratory test results using the criteria proposed by Cetin and Seed (2003), Bray and Sancio (2006), or Idriss and Boulanger (2008)

* FS_{liq} = Factor of Safety against liquefaction = (CRR/CSR), where CRR = $CRR_{7\ 5}$ MSF K_σ K_α, MSF = Magnitude Scaling Factor, $K_\sigma = f[(N_1)_{60}, \sigma'_{vo}]$, K_α =1 0, (level ground), CSR = Cyclic Stress Ratio = 0 65 A_{max} $(\sigma_{vo}/\sigma'_{vo})$ r_d, and $CRR_{7\ 5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude M_w of 7 5

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008)

*** Based on Iwasaki et al (1978) and Toprak and Holzer (2003)

\+ Reference: Boulanger, R W and Idriss, I M (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No UCD/CGM-14/01, 1-134

INPUT SOIL PROFILE DATA

Bottom of Soil Layer Elevation (feet)	Soil Depth During Test (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Field SPT Blow Count N_{field} (blows/ft)	Type of Soil Sampler	Fines Content FC (%)
-19 0	1 0	CL	N	120 0			
-27 0	6 0	CH	N	120 0			
-32 0	12 5	CH	N	120 0			
-37 0	17 5	CH	N	120 0			
-42 0	22 5	CH	N	120 0			
-46 5	27 3	CH	N	120 0			
-53 0	32 8	SM	Y	120 0	38 0	SPT1	15 0
-56 0	37 5	CL	N	120 0			
-58 0	40 0	ML	N	120 0			
-60 0	42 0	CL	N	120 0			
-67 0	46 5	CL	N	120 0			
-68 5	50 8	CL	N	120 0			

LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +

Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. For Vert. Stress C_N	SPT Corr. For Hammer Energy C_E	SPT Corr. For Borehole Size C_B	SPT Corr. For Rod Length C_R	SPT Corr. For Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)
120 0	120 0									1 000		0 325			NL: Dry Soil			0 00	0 00	0 00
720 0	564 0									0 985		0 408			NL: Clay rich Soil			0 00	0 00	0 00
1500 0	1032 0									0 953		0 450			NL: Clay rich Soil			0 00	0 00	0 00
2100 0	1320 0									0 926		0 479			NL: Clay rich Soil			0 00	0 00	0 00
2700 0	1608 0									0 896		0 489			NL: Clay rich Soil			0 00	0 00	0 00
3270 0	1881 6									0 866		0 489			NL: Clay rich Soil			0 00	0 00	0 00
3930 0	2198 4	0 902	1 333	1 050	1 000	1 000	53 2	48 0	51 3	0 831	0 877	0 483			NL: Dense Soil			0 00	0 00	0 00
4500 0	2472 0									0 799		0 473			NL: Clay rich Soil			0 00	0 00	0 00
4800 0	2616 0									0 783		0 467			NL: Clay rich Soil			0 00	0 00	0 00
5040 0	2731 2									0 770		0 462			NL: Clay rich Soil			0 00	0 00	0 00
5580 0	2990 4									0 741		0 449			NL: Clay rich Soil			0 00	0 00	0 00
6090 0	3235 2									0 714		0 437			NL: Clay rich Soil			0 00	0 00	0 00

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION	
Project Name	Westside Canal Energy Center
Project No.	1076
Project Location	Imperial Valley, California
Analyzed By	Carlos Amante
Reviewed By	Carl Henderson

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	6 50
Peak Ground Acceleration, A_{max}	0 50 g
Required Factor of Safety, FS	1 20

BORING DATA AND SITE CONDITIONS		
Boring No.	B-6	
Ground Surface Elevation	-17 0 feet	
Proposed Grade Elevation	-17 0 feet	
GWL Depth Measured During Test	18 0 feet	
GWL Depth Used in Design	5 0 feet	
Borehole Diameter	6 0 inches	
Hammer Weight	140 0 pounds	
Hammer Drop	30 0 inches	
Hammer Energy Efficiency Ratio, ER	80 0 %	
Hammer Distance to Ground Surface	5 0 feet	
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)	
- Ground Slope, S	0 0 %	
- Free Face (L/H) Ratio	N/A	H = 0 feet
Average Total Unit Weight of New Fill	120 0 pcf	

SUMMARY OF RESULTS

Severity of Liquefaction:

Total Thickness of Liquefiable Soils, H_{liq}: 0 00 feet (cumulative total thickness in the upper 65 feet)

Liquefaction Potential Index (LPI): 0 00 *** (Very low risk, with no surface manifestation of liquefaction)

Seismic Ground Settlements:	Analysis Method	Upper 30 feet	Upper 50 feet	Upper 65 feet	
Seismic Compression Settlement:	Pradel (1998)	0 00 inches	0 00 inches	0 00 inches	(Dry/Unsaturated Soils)
Liquefaction-Induced Settlement:	Ishihara and Yoshimine (1992)	0 00 inches	0 00 inches	0 00 inches	(Saturated Soils)
Total Seismic Settlement:		0 00 inches	0 00 inches	0 00 inches	

Seismic Lateral Displacements:	Analysis Method	Upper 30 feet	Upper 50 feet	Upper 65 feet	
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0 00 inches	0 00 inches	0 00 inches	(During Ground Shaking)
Lateral Spreading Displacement:	Zhang et al (2004)	0 00 inches	0 00 inches	0 00 inches	(After Ground Shaking)

NOTES AND REFERENCES

+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs} = f\{(N_1)_{60}, FC\}$ where $(N_1)_{60} = N_{field}\ C_N\ C_E\ C_B\ C_R\ C_S$

++ Liquefaction susceptibility screening is performed to identify soil layers assessed to be non-liquefiable based on laboratory test results using the criteria proposed by Cetin and Seed (2003), Bray and Sancio (2006), or Idriss and Boulanger (2008)

* FS_{liq} = Factor of Safety against liquefaction = (CRR/CSR), where CRR = $CRR_{7\ 5}$ MSF K_σ K_α , MSF = Magnitude Scaling Factor, $K_\sigma = f[(N_1)_{60}, \sigma'_{vo}]$, K_α =1 0, (level ground), CSR = Cyclic Stress Ratio = 0 65 A_{max} $(\sigma_{vo}/\sigma'_{vo})$ r_d , and $CRR_{7\ 5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude M_w of 7 5

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008)

*** Based on Iwasaki et al (1978) and Toprak and Holzer (2003)

+ Reference: Boulanger, R W and Idriss, I M (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No UCD/CGM-14/01, 1-134

INPUT SOIL PROFILE DATA							
Bottom of Soil Layer Elevation (feet)	Soil Depth During Test (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Field SPT Blow Count N_{field} (blows/ft)	Type of Soil Sampler	Fines Content FC (%)

LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +																					
Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. For Vert. Stress C_N	SPT Corr. For Hammer Energy C_E	SPT Corr. For Borehole Size C_B	SPT Corr. For Rod Length C_R	SPT Corr. For Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)	

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SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

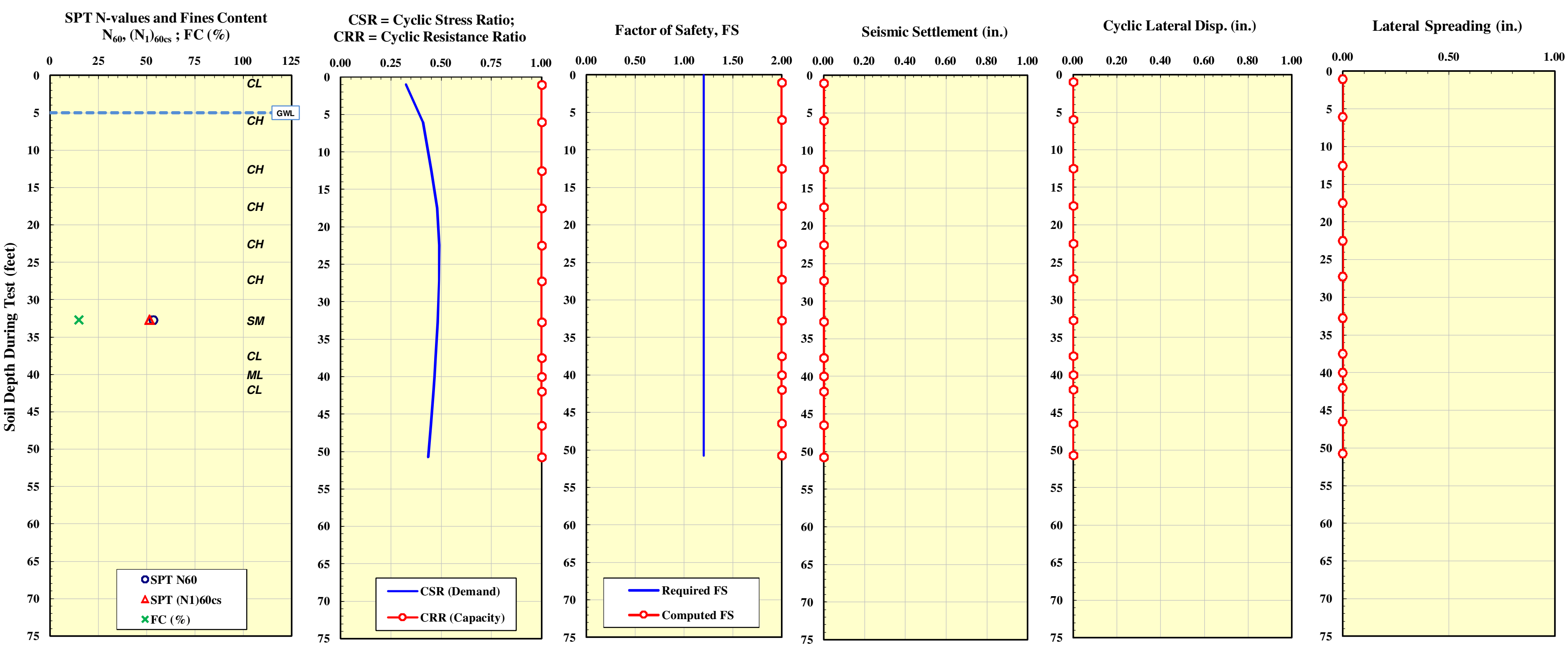
PROJECT INFORMATION	
Project Name	Westside Canal Energy Center
Project No.	1076
Project Location	Imperial Valley, California
Analyzed By	Carlos Amante
Reviewed By	Carl Henderson

TOPOGRAPHIC CONDITIONS	
Ground Slope, S	0.00 %
Free Face (L/H) Ratio	N/A

GROUNDWATER LEVEL DATA	
GWL Depth Measured During Test	18.00 feet
GWL Depth Used in Design	5.00 feet

BORING DATA	
Boring No.	B-6
Ground Surface Elevation	-17.00 feet
Proposed Grade Elevation	-17.00 feet
Borehole Diameter	6.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	80.00 %
Hammer Distance to Ground Surface	5.00 feet

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	6.50
Peak Ground Acceleration, A_{max}	0.50 g
Required Factor of Safety, FS	1.20



Analysis Methods Used ==>>

Liquefaction Triggering:
Boulanger-Idriss (2014)

Seismic Settlements:
Above GWL: Pradel (1998)
Below GWL: Ishihara and Yoshimine (1992)

Cyclic Lateral Displacements:
Above GWL: Pradel (1998)
Below GWL: Tokimatsu and Asaka (1998)

Lateral Spreading:
Zhang et al. (2004)

APPENDIX E

Typical Earthwork Guidelines

TYPICAL EARTHWORK GUIDELINES

1. GENERAL

These guidelines and the standard details attached hereto are presented as general procedures for earthwork construction for sites having slopes less than 10 feet high. They are to be utilized in conjunction with the project grading plans. These guidelines are considered a part of the geotechnical report, but are superseded by recommendations in the geotechnical report in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new recommendations which could supersede these specifications and/or the recommendations of the geotechnical report. It is the responsibility of the contractor to read and understand these guidelines as well as the geotechnical report and project grading plans.

1.1. The contractor shall not vary from these guidelines without prior recommendations by the geotechnical consultant and the approval of the client or the client's authorized representative. Recommendations by the geotechnical consultant and/or client shall not be considered to preclude requirements for approval by the jurisdictional agency prior to the execution of any changes.

1.2. The contractor shall perform the grading operations in accordance with these specifications, and shall be responsible for the quality of the finished product notwithstanding the fact that grading work will be observed and tested by the geotechnical consultant.

1.3. It is the responsibility of the grading contractor to notify the geotechnical consultant and the jurisdictional agencies, as needed, prior to the start of work at the site and at any time that grading resumes after interruption. Each step of the grading operations shall be observed and documented by the geotechnical consultant and, where needed, reviewed by the appropriate jurisdictional agency prior to proceeding with subsequent work.

1.4. If, during the grading operations, geotechnical conditions are encountered which were not anticipated or described in the geotechnical report, the geotechnical consultant shall be notified immediately and additional recommendations, if applicable, may be provided.

1.5. An as-graded report shall be prepared by the geotechnical consultant and signed by a registered engineer and registered engineering geologist. The report documents the geotechnical consultants' observations, and field and laboratory test results, and provides conclusions regarding whether or not earthwork construction was performed in accordance with the geotechnical recommendations and the grading plans. Recommendations for foundation design, pavement design, subgrade treatment, etc., may also be included in the as-graded report.

1.6. For the purpose of evaluating quantities of materials excavated during grading and/or locating the limits of excavations, a licensed land surveyor or civil engineer shall be retained.

2. SITE PREPARATION

Site preparation shall be performed in accordance with the recommendations presented in the following sections.

2.1. The client, prior to any site preparation or grading, shall arrange and attend a pre-grading meeting between the grading contractor, the design engineer, the geotechnical consultant, and representatives of appropriate governing authorities, as well as any other involved parties. The parties shall be given two working days notice.

2.2. Clearing and grubbing shall consist of the substantial removal of vegetation, brush, grass, wood, stumps, trees, tree roots greater than 1/2-inch in diameter, and other deleterious materials from the areas to be graded. Clearing and grubbing shall extend to the outside of the proposed excavation and fill areas.

2.3. Demolition in the areas to be graded shall include removal of building structures, foundations, reservoirs, utilities (including underground pipelines, septic tanks, leach fields, seepage pits, cisterns, etc.), and other manmade surface and subsurface improvements, and the backfilling of mining shafts, tunnels and surface depressions. Demolition of utilities shall include capping or rerouting of pipelines at the project perimeter, and abandonment of wells in accordance with the requirements of the governing authorities and the recommendations of the geotechnical consultant at the time of demolition.

2.4. The debris generated during clearing, grubbing and/or demolition operations shall be removed from areas to be graded and disposed of off site at a legal dump site. Clearing, grubbing, and demolition operations shall be performed under the observation of the geotechnical consultant.

2.5. The ground surface beneath proposed fill areas shall be stripped of loose or unsuitable soil. These soils may be used as compacted fill provided they are generally free of organic or other deleterious materials and evaluated for use by the geotechnical consultant. The resulting surface shall be evaluated by the geotechnical consultant prior to proceeding. The cleared, natural ground surface shall be scarified to a depth of approximately 8 inches, moisture conditioned, and compacted in accordance with the specifications presented in Section 5 of these guidelines.

3. REMOVALS AND EXCAVATIONS

Removals and excavations shall be performed as recommended in the following sections.

3.1. Removals

3.1.1. Materials which are considered unsuitable shall be excavated under the observation of the geotechnical consultant in accordance with the recommendations contained herein. Unsuitable materials include, but may not be limited to, dry, loose, soft, wet, organic, compressible natural soils, fractured, weathered, soft bedrock, and undocumented or otherwise deleterious fill materials.

3.1.2. Materials deemed by the geotechnical consultant to be unsatisfactory due to moisture conditions shall be excavated in accordance with the recommendations of the geotechnical consultant, watered or dried as needed, and mixed to generally uniform moisture content in accordance with the specifications presented in Section 5 of this document.

3.2. Excavations

3.2.1. Temporary excavations no deeper than 4 feet in firm fill or natural materials may be made with vertical side slopes. To satisfy California Occupational Safety and Health Administration (CAL OSHA) requirements, any excavation deeper than 4 feet shall be shored or laid back at a 1:1 inclination or flatter, depending on material type, if construction workers are to enter the excavation.

4. COMPACTED FILL

Fill shall be constructed as specified below or by other methods recommended by the geotec1mical consultant. Unless otherwise specified, fill soils shall be compacted to 90 percent relative compaction, as evaluated in accordance with ASTM Test Method D 1557.

4.1. Prior to placement of compacted fill, the contractor shall request an evaluation of the exposed ground surface by the geotechnical consultant. Unless otherwise recommended, the exposed ground surface shall then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve a generally uniform moisture content at or near the optimum moisture content. The scarified materials shall then be compacted to 90 percent relative compaction. The evaluation of compaction by the geotechnical consultant shall not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify the geotechnical consultant and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

4.2. Excavated on-site materials which are in general compliance with the recommendations of the geotechnical consultant may be utilized as compacted fill provided they are generally free of organic or other deleterious materials and do not contain rock fragments greater than 6 inches in dimension. During grading, the contractor may encounter soil types other than those analyzed during the preliminary geotechnical study. The geotechnical consultant shall be consulted to evaluate the suitability of any such soils for use as compacted fill.

4.3. Where imported materials are to be used on site, the geotechnical consultant shall be notified three working days in advance of importation in order that it may sample and test the materials from the proposed borrow sites. No imported materials shall be delivered for use on site without prior sampling, testing, and evaluation by the geotechnical consultant.

4.4. Soils imported for on-site use shall preferably have very low to low expansion potential (based on UBC Standard 18-2 test procedures). Lots on which expansive soils may be exposed at grade shall be undercut 3 feet or more and capped with very low to low expansion potential fill. In the event expansive soils are present near the ground surface, special design and construction considerations shall be utilized in general accordance with the recommendations of the geotechnical consultant.

4.5. Fill materials shall be moisture conditioned to near optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils shall be generally uniform in the soil mass.

4.6. Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill shall be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

4.7. Compacted fill shall be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift shall be watered or dried as needed to achieve near optimum moisture condition, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other appropriate compacting rollers, to the specified relative compaction. Successive lifts shall be treated in a like manner until the desired finished grades are achieved.

4.8. Fill shall be tested in the field by the geotechnical consultant for evaluation of general compliance with the recommended relative compaction and moisture conditions. Field density testing shall conform to ASTM D 1556-00 (Sand Cone method), D 2937-00 (Drive-Cylinder method), and/or D 2922-96 and D 3017-96 (Nuclear Gauge method). Generally, one test shall be provided for approximately every 2 vertical feet of fill placed, or for approximately every 1000 cubic yards of fill placed. In addition, on slope faces one or more tests shall be taken for approximately every 10,000 square feet of slope face and/or approximately every 10 vertical feet of slope height. Actual test intervals may vary as field conditions dictate. Fill found to be out of conformance with the grading recommendations shall be removed, moisture conditioned, and compacted or otherwise handled to accomplish general compliance with the grading recommendations.

4.9. The contractor shall assist the geotechnical consultant by excavating suitable test pits for removal evaluation and/or for testing of compacted fill.

4.10. At the request of the geotechnical consultant, the contractor shall "shut down" or restrict grading equipment from operating in the area being tested to provide adequate testing time and safety for the field technician.

4.11. The geotechnical consultant shall maintain a map with the approximate locations of field density tests. Unless the client provides for surveying of the test locations, the locations shown by the geotechnical consultant will be estimated. The geotechnical consultant shall not be held responsible for the accuracy of the horizontal or vertical locations or elevations.

4.12. Grading operations shall be performed under the observation of the geotechnical consultant. Testing and evaluation by the geotechnical consultant does not preclude the need for approval by or other requirements of the jurisdictional agencies.

4.13. Fill materials shall not be placed, spread or compacted during unfavorable weather conditions. When work is interrupted by heavy rains, the filling operation shall not be resumed until tests indicate that moisture content and density of the fill meet the project specifications. Regrading of the near-surface soil may be needed to achieve the specified moisture content and density.

4.14. Upon completion of grading and termination of observation by the geotechnical consultant, no further filling or excavating, including that planned for footings, foundations, retaining walls or other features, shall be performed without the involvement of the geotechnical consultant.

4.15. Fill placed in areas not previously viewed and evaluated by the geotechnical consultant may have to be removed and recompacted at the contractor's expense. The depth and extent of removal of the unobserved and undocumented fill will be decided based upon review of the field conditions by the geotechnical consultant.

4.16. Off-site fill shall be treated in the same manner as recommended in these specifications for on-site fills. Off-site fill subdrains temporarily terminated (up gradient) shall be surveyed for future locating and connection.

5. OVERSIZED MATERIAL

Oversized material shall be placed in accordance with the following recommendations.

5.1. During the course of grading operations, rocks or similar irreducible materials greater than 6 inches in dimension (oversized material) may be generated. These materials shall not be placed within the compacted fill unless placed in general accordance with the recommendations of the geotechnical consultant.

5.2. Where oversized rock (greater than 6 inches in dimension) or similar irreducible material is generated during grading, it is recommended, where practical, to waste such material off site, or on site in areas designated as "nonstructural rock disposal areas." Rock designated for disposal areas shall be placed with sufficient sandy soil to generally fill voids. The disposal area shall be capped with a 5-foot thickness of fill which is generally free of oversized material.

5.3. Rocks 6 inches in dimension and smaller may be utilized within the compacted fill, provided they are placed in such a manner that nesting of rock is not permitted. Fill shall be placed and compacted over and around the rock. The amount of rock greater than ¾-inch in dimension shall generally not exceed 40 percent of the total dry weight of the fill mass, unless the fill is specially designed and constructed as a "rock fill."

5.4. Rocks or similar irreducible materials greater than 6 inches but less than 4 feet in dimension generated during grading may be placed in windrows and capped with finer materials in accordance with the recommendations of the geotechnical consultant and the approval of the governing agencies. Selected native or imported granular soil (Sand Equivalent of 30 or higher) shall be placed and flooded over and around the windrowed rock such that voids are filled. Windrows of oversized materials shall be staggered so that successive windrows of oversized materials are not in the same vertical plane. Rocks greater than 4 feet in dimension shall be broken down to 4 feet or smaller before placement, or they shall be disposed of off site.

6. SLOPES

The following sections provide recommendations for cut and fill slopes.

6.1. Cut Slopes

6.1.1. The geotechnical consultant shall observe cut slopes during excavation. The geotechnical consultant shall be notified by the contractor prior to beginning slope excavations.

6.1.2. If, during the course of grading, adverse or potentially adverse geotechnical conditions are encountered in the slope which were not anticipated in the preliminary evaluation report, the geotechnical consultant shall evaluate the conditions and provide appropriate recommendations.

6.2. Fill Slopes

6.2.1. When placing fill on slopes steeper than 5:1 (horizontal:vertical), topsoil, slope wash, colluvium, and other materials deemed unsuitable shall be removed. Near-horizontal keys and near-vertical benches shall be excavated into sound bedrock or fine fill material, in accordance with the recommendation of the geotechnical consultant. Keying and benching shall be accomplished. Compacted fill shall not be placed in an area subsequent to keying and benching until the area has been observed by the geotechnical consultant. Where the natural gradient of a slope is less than 5:1, benching is generally not recommended. However, fill shall not be placed on compressible or otherwise unsuitable materials left on the slope face.

6.2.2. Within a single fill area where grading procedures dictate two or more separate fills, temporary slopes (false slopes) may be created. When placing fill adjacent to a temporary slope, benching shall be conducted in the manner described in Section 7.2. A 3-foot or higher near-vertical bench shall be excavated into the documented fill prior to placement of additional fill.

6.2.3. Unless otherwise recommended by the geotechnical consultant and accepted by the Building Official, permanent fill slopes shall not be steeper than 2:1 (horizontal:vertical). The height of a fill slope shall be evaluated by the geotechnical consultant.

6.2.4. Unless specifically recommended otherwise, compacted fill slopes shall be overbuilt and cut back to grade, exposing firm compacted fill. The actual amount of overbuilding may vary as field conditions dictate. If the desired results are not achieved, the existing slopes shall be overexcavated and reconstructed in accordance with the recommendations of the geotechnical consultant. The degree of overbuilding may be increased until the desired compacted slope face condition is achieved. Care shall be taken by the contractor to provide mechanical compaction as close to the outer edge of the overbuilt slope surface as practical.

6.2.5. If access restrictions, property line location, or other constraints limit overbuilding and cutting back of the slope face, an alternative method for compaction of the slope face may be attempted by conventional construction procedures including backrolling at intervals of 4 feet or less in vertical slope height, or as dictated by the capability of the available equipment, whichever is less. Fill slopes shall be backrolled utilizing a conventional sheepsfoot-type roller. Care shall be taken to maintain the specified moisture conditions and/or reestablish the same, as needed, prior to backrolling.

6.2.6. The placement, moisture conditioning and compaction of fill slope materials shall be done in accordance with the recommendations presented in Section 5 of these guidelines.

6.2.7. The contractor shall be ultimately responsible for placing and compacting the soil out to the slope face to obtain a relative compaction of 90 percent as evaluated by ASTM D 1557 and a moisture content in accordance with Section 5. The geotechnical consultant shall perform field moisture and density tests at intervals of one test for approximately every 10,000 square feet of slope.

6.2.8. Backdrains shall be provided in fill as recommended by the geotechnical consultant.

6.3. Top-of-Slope Drainage

6.3.1. For pad areas above slopes, positive drainage shall be established away from the top of slope. This may be accomplished utilizing a berm and pad gradient of 2 percent or steeper at the top-of-slope areas. Site runoff shall not be permitted to flow over the tops of slopes.

6.3.2. Gunite-lined brow ditches shall be placed at the top of cut slopes to redirect surface runoff away from the slope face where drainage devices are not otherwise provided.

6.4. Slope Maintenance

6.4.1. In order to enhance surficial slope stability, slope planting shall be accomplished at the completion of grading. Slope plants shall consist of deep-rooting, variable root depth, drought-tolerant vegetation. Native vegetation is generally desirable. Plants native to semiarid and mid areas may also be appropriate. Large-leafed ice plant should not be used on slopes. A landscape architect shall be consulted regarding the actual types of plants and planting configuration to be used.

6.4.2. Irrigation pipes shall be anchored to slope faces and not placed in trenches excavated into slope faces. Slope irrigation shall be maintained at a level just sufficient to support plant growth. Property owners shall be made aware that over watering of slopes is detrimental to slope stability. Slopes shall be monitored regularly and broken sprinkler heads and/or pipes shall be repaired immediately.

6.4.3. Periodic observation of landscaped slope areas shall be planned and appropriate measures taken to enhance growth of landscape plants.

6.4.4. Graded swales at the top of slopes and terrace drains shall be installed and the property owners notified that the drains shall be periodically checked so that they may be kept clear. Damage to drainage improvements shall be repaired immediately. To reduce siltation, terrace drains shall be constructed at a gradient of 3 percent or steeper, in accordance with the recommendations of the project civil engineer.

6.4.5. If slope failures occur, the geotechnical consultant shall be contacted immediately for field review of site conditions and development of recommendations for evaluation and repair.

7. TRENCH BACKFILL

The following sections provide recommendations for backfilling of trenches.

7.1. Trench backfill shall consist of granular soils (bedding) extending from the trench bottom to 1 foot or more above the pipe. On-site or imported fill which has been evaluated by the geotechnical consultant may be used above the granular backfill. The cover soils directly in contact with the pipe shall be classified as having a very low expansion potential, in accordance with UBC Standard 18-2, and shall contain no rocks or chunks of hard soil larger than 3/4-inch in diameter.

7.2. Trench backfill shall, unless otherwise recommended, be compacted by mechanical means to 90 percent relative compaction as evaluated by ASTM D 1557. Backfill soils shall be placed in loose lifts 8-inches thick or thinner, moisture conditioned, and compacted in accordance with the recommendations of Section 5 of these guidelines. The backfill shall be tested by the geotechnical consultant at vertical intervals of approximately 2 feet of backfill placed and at spacings along the trench of approximately 100 feet in the same lift.

7.3. Jetting of trench backfill materials is generally not a recommended method of densification, unless the on-site soils are sufficiently free-draining and provisions have been made for adequate dissipation of the water utilized in the jetting process.

7.4. If it is decided that jetting may be utilized, granular material with a sand equivalent greater than 30 shall be used for backfilling in the areas to be jetted. Jetting shall generally be considered for trenches 2 feet or narrower in width and 4 feet or shallower in depth. Following jetting operations, trench backfill shall be mechanically compacted to the specified compaction to finish grade.

7.5. Trench backfill which underlies the zone of influence of foundations shall be mechanically compacted to 90 percent or greater relative compaction, as evaluated by ASTM D 1557-02. The zone of influence of the foundations is generally defined as the roughly triangular area within the limits of a 1:1 (horizontal:vertical) projection from the inner and outer edges of the foundation, projected down and out from both edges.

7.6. Trench backfill within slab areas shall be compacted by mechanical means to a relative compaction of 90 percent, as evaluated by ASTM D 1557. For minor interior trenches, density testing may be omitted or spot testing may be performed, as deemed appropriate by the geotechnical consultant.

7.7. When compacting soil in close proximity to utilities, care shall be taken by the grading contractor so that mechanical methods used to compact the soils do not damage the utilities. If the utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, then the grading contractor may elect to use light mechanical compaction equipment or, with the approval of the geotechnical consultant, cover the conduit with clean granular material. These granular materials shall be jetted in place to the top of the conduit in accordance with the recommendations of Section 8.4 prior to initiating mechanical compaction procedures. Other methods of utility trench compaction may also be appropriate, upon review by the geotechnical consultant and the utility contractor, at the time of construction.

7.8. Clean granular backfill and/or bedding materials are not recommended for use in slope areas unless provisions are made for a drainage system to mitigate the potential for buildup of seepage forces or piping of backfill materials.

7.9. The contractor shall exercise the specified safety precautions, in accordance with OSHA Trench Safety Regulations, while conducting trenching operations. Such precautions include shoring or laying back trench excavations at 1:1 or flatter, depending on material type, for trenches in excess of 5 feet in depth. The geotechnical consultant is not responsible for the safety of trench operations or stability of the trenches.

8. DRAINAGE

The following sections provide recommendations pertaining to site drainage.

8.1. Roof, pad, and slope drainage shall be such that it is away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.).

8.2. Positive drainage adjacent to structures shall be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of the structure at a gradient of 2 percent or steeper for a distance of 5 feet or more outside the building perimeter, further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.

8.3. Surface drainage on the site shall be provided so that water is not permitted to pond. A gradient of 2 percent or steeper shall be maintained over the pad area and drainage patterns shall be established to remove water from the site to an appropriate outlet.

8.4. Care shall be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property. Drainage patterns established at the time of finish grading shall be maintained for the life of the project. Property owners shall be made very clearly aware that altering drainage patterns may be detrimental to slope stability and foundation performance.

9. SITE PROTECTION

The site shall be protected as outlined in the following sections.

9.1. Protection of the site during the period of grading shall be the responsibility of the contractor unless other provisions are made in writing and agreed upon among the concerned parties. Completion of a portion of the project shall not be considered to preclude that portion or adjacent areas from the need for site protection, until such time as the project is finished as agreed upon by the geotechnical consultant, the client, and the regulatory agency.

9.2. The contractor is responsible for the stability of temporary excavations. Recommendations by the geotechnical consultant pertaining to temporary excavations are made in consideration of stability of the finished project and, therefore, shall not be considered to preclude the responsibilities of the contractor. Recommendations by the geotechnical consultant shall also not be considered to preclude more restrictive requirements by the applicable regulatory agencies.

9.3. Precautions shall be taken during the performance of site clearing, excavation, and grading to protect the site from flooding, ponding, or inundation by surface runoff. Temporary provisions shall be made during the rainy season so that surface runoff is away from and off the working site. Where low areas cannot be avoided, pumps shall be provided to remove water as needed during periods of rainfall.

9.4. During periods of rainfall, plastic sheeting shall be used as needed to reduce the potential for unprotected slopes to become saturated. Where needed, the contractor shall install check dams, desilting basins, riprap, sandbags or other appropriate devices or methods to reduce erosion and provide recommended conditions during inclement weather.

9.5. During periods of rainfall, the geotechnical consultant shall be kept informed by the contractor of the nature of remedial or precautionary work being performed on site (e.g., pumping, placement of sandbags or plastic sheeting, other labor, dozing, etc.).

9.6. Following periods of rainfall, the contractor shall contact the geotechnical consultant and arrange a walk-over of the site in order to visually assess rain-related damage. The geotechnical consultant may also recommend excavation and testing in order to aid in the evaluation. At the request of the geotechnical consultant, the contractor shall make excavations in order to aid in evaluation of the extent of rain-related damage.

9.7. Rain or irrigation related damage shall be considered to include, but may not be limited to, erosion, silting, saturation, swelling, structural distress, and other adverse conditions noted by the geotechnical consultant. Soil adversely affected shall be classified as "Unsuitable Material" and shall be subject to overexcavation and replacement with compacted fill or to other remedial grading as recommended by the geotechnical consultant.

9.8. Relatively level areas where saturated soils and/or erosion gullies exist to depths greater than 1 foot shall be overexcavated to competent materials as evaluated by the geotechnical consultant. Where adverse conditions extend to less than 1 foot in depth, saturated and/or eroded materials may be processed in-place. Overexcavated or in-place processed materials shall be moisture conditioned and compacted in accordance with the recommendations provided in Section 5. If the desired results are not achieved, the affected materials shall be overexcavated, moisture conditioned, and compacted until the specifications are met.

9.9. Slope areas where saturated soil and/or erosion gullies exist to depths greater than 1 foot shall be overexcavated and replaced as compacted fill in accordance with the applicable specifications. Where adversely affected materials exist to depths of I foot or less below proposed finished grade, remedial grading by moisture conditioning in-place and compaction in accordance with the appropriate specifications may be attempted. If the desired results are not achieved, the affected materials shall be overexcavated, moisture conditioned, and compacted until the specifications are met. As conditions dictate, other slope repair procedures may also be recommended by the geotechnical consultant.

9.10. During construction, the contractor shall grade the site to provide positive drainage away from structures and to keep water from ponding adjacent to structures. Water shall not be allowed to damage adjacent properties. Positive drainage shall be maintained by the contractor until permanent drainage and erosion reducing devices are installed in accordance with project plans.

APPENDIX F

GBC - Important Information About This Geotechnical-Engineering Report

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

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