Geotechnical Engineering ● Engineering Geology

Geotechnical Design Report

Mountain View Power Partners Wind Repower Project

Riverside County, California

Prepared for:

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October 30, 2020
Project No. GEN 20-33E

TETRA TECH

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Mr. Michael Hughes
AES North America Development
690 North Studebaker Road
Long Beach, California 90803

Subject: **GEOTECHNICAL ENGINEERING REPORT**
MOUNTAIN VIEW POWER PARTNERS WIND REPOWER PROJECT
Palm Springs, California

Dear Mr. Hughes:

Tetra Tech BAS, Inc. (Tetra Tech) is pleased to submit the results of the geotechnical investigation for the Mountain View Power Partners Wind Repower Project located near North Palm Springs in unincorporated Riverside County, California. The purpose of this study was to evaluate the subsurface conditions and to provide geotechnical recommendations for the design and construction of 16 new wind energy turbines and related utility trenches for electrical conveyance systems, access roads, as well as additional recommendations for the foundation design for a new meteorological tower. This report includes a brief description of site conditions and the proposed development, discussions regarding field and laboratory investigation, subsurface conditions, engineering seismology, and geotechnical conclusions and recommendations for design and construction of the proposed wind turbines. The appendices to the report include logs of borings, results of geophysical surveys, results of laboratory tests, seismic demand, and evaluation of seismically induced settlement.

We appreciate the opportunity to provide our professional services on this Project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
Tetra Tech BAS, Inc.

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1. INTRODUCTION

This report presents the results of the geotechnical investigation for the Mountain View Power Partners Wind Repower Project (Project) located near North Palm Springs in unincorporated Riverside County, California (see Figure 1 – Project Location Map). The purpose of this study was to evaluate the surface and subsurface conditions and to provide geotechnical recommendations for the design and construction of 16 new wind energy turbines and related utility trenches for electrical conveyance systems, access roads, as well as additional recommendations for the foundation design for a new meteorological tower. This report summarizes the data collected and presents our findings, conclusions, and recommendations and is intended to be suitable for preparation of the project design plans and specification as well as for the submittal for issuance of grading and construction permits to the County of Riverside.

2. SCOPE OF WORK

Tetra Tech's scope of services for this project consisted of the following tasks:

- Review of available background data, including:
    - geotechnical literature;
    - geologic maps/publications;
    - historical aerial photographs; and
    - seismic hazard maps and technical documents relevant to the subject site.

- Perform a site reconnaissance to observe surface conditions in the vicinity of the planned improvements and to confirm the accessibility of the intended boring locations.

- Mark boring locations, contact Underground Service Alert and conduct a utility survey using ground penetrating radar (GPR) at each of the marked boring locations.

- Conduct geophysical surveys consisting of:
    - 8 active surface wave surveys (MASW);
    - Two 4-electrode Wenner resistivity array tests

- Perform a subsurface evaluation, including the excavating, logging, and sampling of:
    - 16 deep exploratory borings to depths ranging from about 27 to 61.5 feet; and
    - 8 shallow exploratory borings to depths ranging from about 1 to 16.5 feet.

- Collect soil samples from the borings and transport samples to a geotechnical laboratory for supplemental visual classification and additional testing.

- Perform laboratory testing on selected samples retrieved from the borings to evaluate geotechnical engineering properties of the on-site soils.

- Perform engineering evaluation of the collected geotechnical data to develop geotechnical recommendations for the design and construction of the proposed development, including consideration for the following:
    - An evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
    - An evaluation of the suitability of on-site soils for support of wind turbine structures.
    - A determination of the seismic Site Class per ASCE 7-16 using Multichannel Analysis of Surface Waves (MASW).
    - An evaluation of the electrical resistivity of the native surficial soils.
    - An evaluation of the thermal resistivity of the native surficial and recompacted soils.
    - Evaluation of geologic hazards at the site.
    - Recommendations for site grading for the planned improvements, including site preparation, subgrade preparation, and fill and backfill conditioning and placement.
    - Determination of seismic design parameters in accordance with the 2019 California Building Code.

  - Recommendations for design of foundation systems including allowable bearing capacity, lateral resistance, settlement estimates, and subgrade modulus.
  - An evaluation of the liquefaction potential and dynamic settlement of the on-site soils.
  - Recommendations for dynamic shear modulus and Poisson's ratio.
  - Recommendations for trench excavations for buried utilities.
  - Recommendations for stabilization of access roads.
  - An evaluation of the corrosion potential of the on-site soils to buried concrete and steel.

- Preparation of this report, including the provision of reference maps and illustrations, a summary of the collected data, and geotechnical conclusions and recommendations for the design and construction of the proposed project.

3. PROJECT DESCRIPTION

The site is located on a gently sloping, south to southeast trending, alluvial fan surface inclined at about a 3 percent gradient. The site is approximately 3 miles long by 1 mile wide. The ground surface at the northwest corner of the site is approximately at elevation 1,260 feet and drops to elevation 975 feet at the southeast corner of the site. The alluvial fan surface materials consist of cobbles with gravel and sand with locally abundant boulder size particles, particularly on the western part of the site, and it is generally sparsely vegetated.

The proposed project is located on private land, and a portion the site is located in the Whitewater Floodplain Conservation Area, as defined in the Coachella Valley Multiple Species Habitat Conservation Plan (CVMSHCP) (see Plate 1 – Proposed Turbine Layout and Parcel Map). Assessor Parcel Numbers (APNs) are also included in Plate 1.

AES is planning to erect new Vestas V117-3.6MW and V117-4.3MW wind turbines, both with a rotor diameter of approximately 375 feet, and a meteorological tower at the locations indicated in Figure 2 – Site Layout and Subsurface Exploration Locations Map. AES is targeting a 2022 commercial operation date. The new turbines will replace existing, currently in operation, Mitsubishi 600 kW wind turbines, which were installed in 2001. It is proposed that the concrete from the demolition of the existing foundations be crushed and used for miscellaneous base material for the access roads within the project area.

Each of the new wind turbines will be supported on a spread-foot type foundation in the shape of a truncated cone, with a base diameter of about 70 to 75 feet, a top diameter of about 25 feet and a thickness of about 7 feet. The bottom of the conical mat foundation will be embedded about 10 to 14 feet below the ground surface. Scour protection of the foundations could be necessary depending on the results of a hydrologic and hydraulic flow analysis currently being conducted for the Project. A cylindrical pedestal about 20 feet in diameter and about 6.5 feet tall will be built on top of the foundation mat to support the wind turbine tower. The wind turbine nacelles will be located on top of the towers. The hub will be at a height of 300 feet. The schematic configuration of the proposed foundation is shown on the Illustration 1 below.

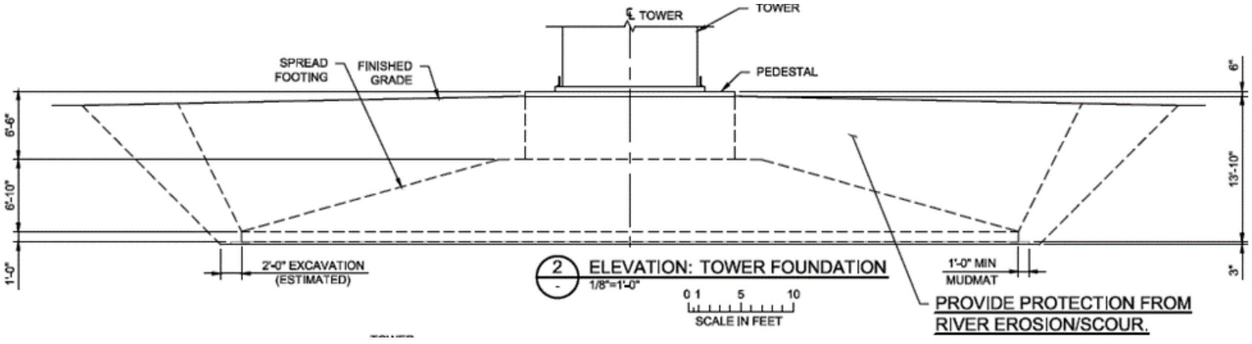

Illustration 1. Schematic configuration of the proposed wind tower mat foundation

The meteorological tower will be supported on a square mat foundation about 23 feet by 23 feet in footprint and 6 feet deep. It will have a pedestal 8 feet in diameter rising from the top of the foundation up to 6 inches above finished grade. A meteorological tower will be placed on top of the column at a height of 300 feet. The tower will provide support for meteorological equipment.

The western transformer of the two transformers at the existing substation located at the east limit of the project area (see Plate 1 – Proposed Turbine Layout and Parcel Map) will be replaced. The existing transformer weighs approximately 130,000 pounds and is founded on 11 feet by 16 feet mat, 3 feet thick. The replacement transformer weighs approximately 210,000 pounds and is planned to be installed on the existing mat.

4. FIELD INVESTIGATIONS

The field investigation effort entailed performing a geotechnical exploration consisting of exploratory borings, seismic geophysical surveys, and electrical resistivity geophysical surveys.

4.1 Exploratory Borings

The subsurface soil and groundwater conditions beneath the site were explored in 3 different phases that allowed optimization in the use of different drilling methods to achieve the target depths for the exploration boreholes. Initially, a hollow stem auger (HSA) truck-mounted drill rig, equipped with 8-inch diameter augers, was used. It was observed that the HSA drill rig could not reach the required target depths for the deep borings, particularly in the eastern part of the site, because of the abundant presence of cobbles and large boulders in that area. Therefore, a Becker Hammer truck-mounted drill rig equipped with 8-inch diameter hollow pipes was subsequently mobilized to penetrate through the cobbles and boulders and achieve the target depths.

Prior to starting the field exploration programs, a field reconnaissance was conducted in coordination with AES site personnel to observe site conditions and to mark the borehole locations. Underground Service Alert was notified of the drilling schedule at least 48 hours prior to drilling and a GPR was used to clear all drilling locations.

On the first day of each phase of drilling all personnel involved attended a mandatory Health and Safety meeting at the local AES office before any field work was conducted. A total of 16 deep borings B-1 through B-16 were advanced to a maximum depth of 61.5 feet in the area of the proposed turbine locations, and 7 additional shallow borings B-91 through B-97 were drilled in the general project areas to a maximum depth of 16.5 feet. The borings were drilled at the locations indicated on Figure 2, and in Plate 1.

The exploratory borings were drilled in 3 phases as follows:

Phase 1:

- Borings B-2, B-9, B-12, B-16 were drilled between August 12th and 13th, 2020 in the area of the proposed turbine locations using an HSA drill rig; multiple attempts were made to drill boreholes B-4, B-5 and B-8, but refusal was encountered at depths shallower than 12.5 feet. Therefore, drilling of these 3 borings was transferred to Phase 3.

- Shallow Borings B-96 and B-97 were drilled between August 12th and 14th, 2020 in the general project area using an HSA drill rig.

Phase 2:

- Borings B-13, B-14, and B-15 were drilled between August 25th and 26th, 2020 in the area of the proposed turbine locations using an HSA drill rig.

- Shallow Borings B-91 through B-94 and B-98 were drilled between August 25th and 26th, 2020 in the general project area using an HSA drill rig.

Phase 3:

- Borings B-1, B-3, B-4, B-5, B-6, B7, B-8, B-10 and B-11 were drilled between September 14th and 17th, 2020 in the area of the proposed turbine locations using a Becker Hammer drill rig.

The longitude and latitude of the as-drilled boring locations were obtained with a handheld NAD 83 Coordinate System Global Positioning System (GPS) unit and are summarized in Table 1.

Both driven ring-type and bulk samples of the subsurface materials were retrieved at selected depths during drilling. The driven samples were collected utilizing both a California-type split spoon sampler and a Standard Penetration Test (SPT) split spoon sampler. Blowcounts were recorded during testing. SPT sampling was performed in general accordance with ASTM D1586. The hammer calibration records indicated an energy transfer ratio of about 80 percent for the Becker Hammer drill rig and about 88 percent for the HSA drill rig.

An attempt was made to retrieve soil samples every 2.5 feet between a depth of 2.5 and 15 feet, and every 5 feet from a depth of 15 feet to the maximum explored depth. However, due to either very dense soil conditions and/or the presence of cobbles and boulders it was not possible to physically retrieve samples at various depths. Following the drilling, all boreholes were checked for groundwater and backfilled with tamped soil cuttings.

The soil borings were logged by a geologist under supervision of a Certified Engineering Geologist in general accordance with the visual-manual procedure for description and identification of soils, ASTM D2488. The geologist prepared the recovered samples for subsequent reference and laboratory testing. Additional details pertinent to material types encountered in the borings, groundwater conditions, sampling depths and types, and abandonment methods and depths are presented in Appendix A -- Boring Logs.

Table 1
Summary of Exploratory Borings

Boring No.	Latitude (°)	Longitude (°)	Total Depth (ft)	Approximate Surface Elevation (ft)	Date Drilled
B-1	33.915977	-116.628427	60.3	1239	9/14/2020
B-2	33.914068	-116.629033	35.5	1214	8/13/2020
B-3	33.911413	-116.629021	40.0	1190	9/15/2020
B-4	33.908737	-116.627425	30.6	1151	9/15/2020
B-5	33.915028	-116.62354	40.3	1205	9/18/2020
B-6	33.913072	-116.624218	40.8	1184	9/18/2020
B-7	33.91083	-116.624264	40.9	1163	9/16/2020
B-8	33.908334	-116.621828	40.8	1125	9/16/2020
B-9	33.913714	-116.617004	27.0	1137	8/13/2020
B-10	33.911451	-116.615586	30.6	1106	9/17/2020
B-11	33.907411	-116.617151	41.3	1088	9/17/2020
B-12	33.910926	-116.603786	60.4	1003	8/12/2020
B-13	33.908735	-116.60459	40.5	1002	8/26/2020
B-14	33.906653	-116.60469	40.4	994	8/25/2020
B-15	33.904733	-116.604768	40.4	991	8/26/2020
B-16	33.902573	-116.604496	60.9	979	8/12/2020
B-91	33.908525	-116.625118	3.0	1141	8/25/2020
B-92	33.914409	-116.620391	1.0	1171	8/25/2020
B-93	33.914366	-116.610962	1.0	1079	8/26/2020
B-94	33.909305	-116.611672	16.5	1060	8/25/2020
B-95	33.902139	-116.608119	15.9	996	8/17/2020
B-96	33.911686	-116.608022	15.7	1041	8/12/2020
B-97	33.901202	-116.600137	16.5	946	8/14/2020
B-98	33.901809	-116.57969	16.5	828	8/26/2020

4.2 MASW Geophysical Surveys

Under subcontract to Tetra Tech, GeoVision Geophysical Services (GeoVision) performed active surface wave measurements at the project site on August 18 to August 25, 2020. The active surface wave technique utilized during this investigation consisted of multi-channel analysis of surface waves (MASW). The purpose of this geophysical survey was to obtain a shear wave velocity profile of the upper 30 meters (i.e., about 100 feet) to be used to provide estimates of small strain stiffness for the subsurface materials and a seismic Site Classification in accordance with 2019 California Building Code (CBC).

A total of 8 MASW field arrays, MASW-1 through MASW-8, were deployed at the project site at the locations indicated in Figure 2. Ground motions were recorded by at least 24 geophones, typically spaced 1 to 3 meters apart, along a linear array and connected to a seismograph. Energy sources for shallow investigations included various sized hammers. The procedures and results of the testing are presented in Appendix B -- MASW Geophysical Survey. A discussion of the results is also included in the Section "Seismic Design Parameters" of this report.

4.3 Electrical Resistivity Geophysical Surveys

Under subcontract to Tetra Tech, GeoVision also performed electrical resistivity measurements on August 26 and September 25, 2020. Two electrical resistivity geophysical soundings, ER-1 and ER-2, were performed by laying out a 4-electrode, Wenner resistivity array along two orthogonal profiles, one oriented approximately in a south to north (SN) direction and the other oriented approximately in a west to east (WE) direction, at the approximate locations indicated in Figure 2. Resistivity field measurements are typically used to evaluate the corrosion potential and grounding characteristics of the foundation soils. The procedures and results of the testing are presented in Appendix C -- Electrical Resistivity Geophysical Survey. A discussion of the results is also included in the "Soil Corrosion Potential" section of this report.

5. LABORATORY TESTING

Laboratory tests were performed on selected samples recovered from the borings to aid in the classification of soils and to evaluate pertinent engineering properties of the foundation soils. The following tests were performed:

- In-situ Moisture Content and Dry Density, ASTM D2937;
- Grain Size Distribution, ASTM D6913 and D7928;
- Direct Shear ASTM D3080;
- Consolidation ASTM D2435;
- Compaction Test using Modified Effort ASTM D1557;
- R-value test ASTM D2844;
- Corrosion Testing: Minimum electrical resistivity and pH CTM 643; Sulphate Content CTM 417; Chloride Content CTM 422;
- Thermal Resistivity using the 3-point dry-out curve method ASTM D5334.

Testing was performed in general accordance with applicable indicated ASTM Standards and California Test Methods. Results of all laboratory tests are presented in Appendix D -- Laboratory Testing. For ease of referral to the soil profile, selected laboratory results, including moisture and density determinations, have also been included in the boring logs in Appendix A – Boring Logs.

6. GEOLOGIC AND SUBSURFACE CONDITIONS

6.1 Regional Geology

The Project is situated along the western margin of the Coachella Valley at the eastern reaches of the San Gorgonio Pass. The relatively narrow San Gorgonio Pass and much wider Coachella Valley form the boundary between the Transverse Ranges geomorphic province to the north and the Peninsular Ranges geomorphic province to the south. The Transverse Ranges are generally characterized by east-west trending mountains that include the Little San Bernardino Mountains to the northeast and the San Bernardino Mountains to the north and northwest of the Project limits. The Peninsular Ranges are characterized by northwest to southeast trending mountain ranges that include the San Jacinto and Santa Rosa Mountains to the south and southeast of the Project limits. The Coachella Valley extends approximately 45 miles southeast from San Gorgonio Pass to the northern shores of the Salton Sea.

The Coachella Valley is part of the tectonically active Salton Trough that comprises a complex transition zone between the right-lateral strike-slip San Andreas fault system and the northwestward progressing spreading ridge complex associated with the Gulf of California segment of the Eastern Pacific Rise. The San Andreas Fault Zone, in proximity to the site, consists of fault strands referred to locally as the Mission Creek fault, the Banning fault, and the Garnet Hill fault.

6.2 Local Geology

The Project is underlain by Quaternary age alluvial fan and stream channel deposits associated with outflow from the San Gorgonio River and Whitewater River to the west and northwest, respectively (see Figure 3 – Geologic Map). These deposits generally consist of sandy gravel with abundant rounded cobbles and boulders of granitic and metamorphic compositions. The concentration of boulders is more prevalent within the westerly portions of the site where the apex of the fan is more aligned with the mouth of Whitewater River. The concentration of boulders gradually decreases to the east along the distal reaches of the alluvial fan apron.

6.3 Surficial Units

Minor accumulations of undocumented artificial fill mantle the alluvial deposits within the site. Descriptions of the surficial soils encountered during site exploration are summarized below.

6.3.1 Undocumented Artificial Fill (not shown on map)

Minor accumulations of undocumented artificial fill, primarily associated with improved and unimproved access roads, erosion control measures and buried utilities, exists locally throughout the site. The fill materials are comprised of locally derived earth materials that are generally only a few feet thick.

6.3.2 Alluvial Deposits (Qf, Qg, Qa)

Surficial soils encountered across the site generally consist of sand and gravel deposits of Quaternary age. Per Dibblee and Minch (2004), these soils are non-indurated alluvial sediments differentiated locally by source. Granular alluvial fan deposits (Qf) transported from Whitewater Canyon are the primary unit found extensively across the westerly reaches of the site. These deposits consist of sands and gravels with abundant cobbles and boulders. Recent stream channel wash deposits (Qg) comprised of sand and gravelly sand were encountered on the west edge of the site nearest the active Whitewater River. Alluvial sand and gravel of valley areas (Qa) with fewer boulder-size clasts were encountered along the eastern portion of the site.

More detailed descriptions of the soils encountered during the investigation can be found on the boring logs in Appendix A.

6.4 Groundwater

The Riverside County hazards map for liquefaction, https://koordinates.com/layer/96846-riverside-county-ca-liquefaction/, indicates that the regional groundwater at the site is expected to be at a depth greater than 100 feet. Data from the California Department of Water Resources under the Sustainable Groundwater Management Act (SGMA) for groundwater wells near the site was obtained from https://sgma.water.ca.gov/webgis/?appid=SGMADataViewer#gwlevels, and the data is summarized in Table 2. The data from the SGMA indicates that the regional groundwater in the vicinity of the site has in fact been at a depth of at least 150 feet.

Table 2
Summary of Groundwater Data from Nearby Wells

Site Well or State Well	Ground Surface Elevation (feet)	Highest Groundwater Elevation (feet)	Historical Shallowest Depth (feet)	Date of Measurement	Monitoring Period	Location of Well in relation to the project site
03S04E20D001S	912.5	761.56	150.9	4/28/1987	2/15/1972 to 4/20/1992	Within the Project area
03S04E20F003S	892.5	738.8	153.7	4/29/1987	9/25/1979 to 4/19/2010	0.1 miles to the south
33533911634530	840.0	423.0	417.0	7/26/1980	7/16/980 to 6/3/2020	0.2 miles to the southeast

Fluctuations of the groundwater level, localized zones of perched water, and increased soil moisture content should be anticipated during and following the rainy season. It is noted that the site is within the Whitewater River Floodplain Conservation Area, indicating that temporary conditions of flooding at the site should be expected. Irrigation of landscaped areas on or adjacent to the site can also cause a fluctuation of local groundwater levels. Evaluation of such factors is beyond the scope of our services. Based on the reviewed and herein presented data, groundwater is not expected to be a concern for the proposed construction.

7. ENGINEERING SEISMOLOGY AND OTHER GEOLOGIC HAZARDS

7.1 General Seismic Setting

The Southern California region is known to be seismically active. Earthquakes occurring within approximately 60 miles of the site are generally capable of generating ground shaking of engineering significance. The project area is located in the general proximity of several Holocene-active faults, as shown on Figure 4 – Regional Faults and Seismicity Map. Holocene-active faults are defined as those that have experienced surface displacement during the Holocene epoch (approximately the last 11,700 years).

Table 3 summarizes known active faults within approximately 45 miles of the project site and lists the type of fault and its maximum moment magnitude (M_{max}), as published by Jennings (1994). The approximate distance of each fault, as measured from the closest surface trace to the site, was calculated from Jennings (2010).

Table 3
Summary of Active Faults

Fault Zone/Fault Name	Approximate Fault Distance to Site[1] (miles)	Direction Relative to the Site	Type of Fault	Maximum Moment Magnitude[2] (Mmax)
Garnet Hill	0.2	North	Oblique right-reverse / right-lateral strike-slip	7.0
Banning	1.8	North	Right-lateral strike-slip	7.2
San Gorgonio Pass	2.1	West	Thrust	7.0
Mission Creek	6.7	Northeast	Right-lateral strike-slip/ thrust	--
Pinto Mountain	7.5	North	Left-lateral strike-slip	7.5
Burnt Mountain	13.3	East	Right-lateral strike-slip	6.5
Eureka Peak	16.2	East	Right-lateral strike-slip	6.8
San Jacinto	16.9	Southwest	Right-lateral strike-slip / minor right reverse	7.5
Johnson Valley	20.3	Northeast	Right-lateral strike-slip	7.3
Copper Mountain	29.6	Northeast	Right-lateral strike-slip	6.5
Hidalgo	34.5	Northeast	Right-lateral strike-slip	7.1
Mesquite Lake	38.0	Northeast	Right-lateral strike-slip	7.0
Elsinore	41.7	Southwest	Right-lateral strike-slip	7.5
Cucamonga Fault	50.8	Northwest	Thrust	7.0
Chino	56.0	West	Right reverse	7.0

Notes:
[1] per Jennings, 2010
[2] per Jennings, 1994

7.2 Primary Active Faults

The San Andreas Fault Zone is the principal boundary between the Pacific and North American tectonic plates. It is a complex strike-slip fault system that represents a continuous zone of faulting from the San Francisco area to the Salton Sea. Motion accommodated by the fault zone is distributed along a complex system of interrelated faults. In southern California, the San Andreas fault consists of three segments: 1) Mojave Desert segment, 2) San Bernardino Mountains segment, and 3) Coachella Valley segment. The northern Coachella Valley segment is located northwest and north of the Project area in the eastern San Gorgonio Pass and the upper portion of the Coachella Valley. This Coachella Valley segment of the San Andreas Fault Zone is comprised of the Garnet Hill, the Banning, and the Mission Creek fault strands that all converge at depth.

The Garnet Hill fault (GHF) is the southern-most segment of the San Andreas Fault Zone in proximity of the subject site. The westernmost extent of the GHF displays distinct geomorphic expression where its surface trace along the eastern flank of Whitewater Canyon forms a significant linear scarp in Holocene age alluvium that projects east-southeast towards the base of Whitewater Hill. However, continuous photo-lineament expression of the GHF diminishes rapidly toward the southeast. Published geologic mapping by Proctor (1968) and Rogers (1965) infers the trace of the GHF as a straight line extending from the southwest flank of Whitewater Hill along the south side of Interstate 10 to the southern base of Garnet Hill, to the southeast. The continuation of the GHF from the prominent alluvial scarp near the Whitewater River toward the southeast was based largely by the linear projection of Whitewater Hill and Garnet Hill and a prominent groundwater barrier that is coincidental with the inferred trace of the GHF (Proctor, 1968).

Later mapping by Dibblee (1982 & 2014) depicts the surface trace of the GHF as non-linear, extending north of Interstate 10 along the base of Whitewater Hill and Hugo Hill to the southeast toward the southern flank of Garnet Hill. More recent structural interpretation by Yule and Sieh (2003) and Cordona (2013) generally agree with Dibblee’s mapped interpretation and indicate that the GHF consists of a series of left-stepping, northwest-trending right-lateral faults with active anticlinal folds at each stepover, as shown on Illustration 2. The eastern and western ends of the fault are marked by pressure ridges referred to locally as Edom Hill and West Whitewater Hill. These larger folds at the end of the fault are interpreted as indicators of the transfer of slip from the Coachella Valley strand of the Banning fault onto the GHF. Less prominent folds at stepovers along the GHF are expressed by Garnet Hill, Hugo Hill, and East Whitewater Hill (see Illustration 2 below). These smaller folds are believed to control the discontinuous geometry of the GHF and are a result of contractional, en echelon stepovers in the fault trace.

As presented by Dibblee (1982 & 2014), Yule and Sieh (2003), and as adopted by the California Geological Survey, the surface trace of the Garnet Hill fault in proximity of the Project is believed to be positioned north of or roughly parallel to and beneath Interstate 10 freeway. The GHF is closest to the subject site along the base of East Whitewater Hill, approximately 0.25 miles north of the project site.

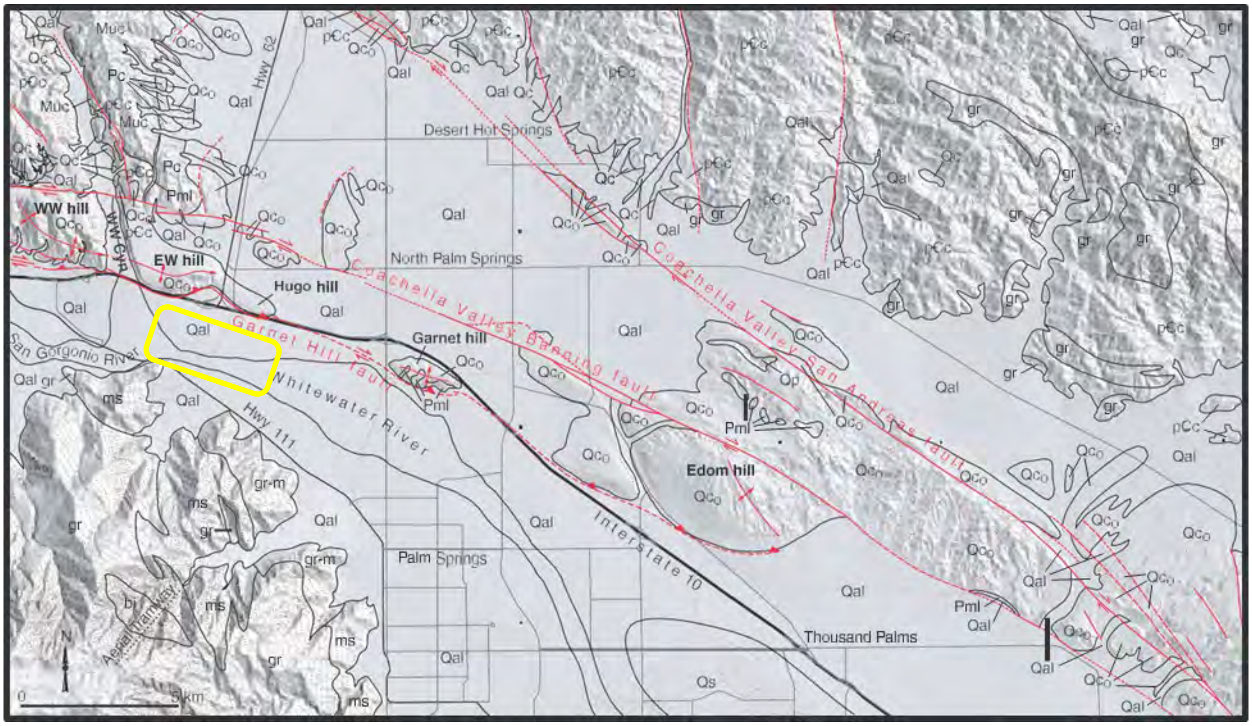

Illustration 2. Generalized geologic map of the northern Coachella Valley emphasizing local segments of the San Andreas Fault Zone. Approximate site boundary indicated by yellow border. The surface trace of Holocene structural features is shown red and includes right-lateral, reverse, and normal-dextral faults and folds (Yule & Sieh, 2003).

The Banning and the Mission Creek fault segments of the San Andreas Fault Zone both exhibit evidence of Holocene ground rupture and are believed to be capable of generating magnitude 7.2 and 7.1 earthquakes, respectively. These fault segments also have the potential to rupture simultaneously and have been included within Alquist-Priolo Earthquake Fault Zones.

The Banning fault forms the southern margin of the San Bernardino Mountains and enters the Coachella Valley from the west through the San Gorgonio Pass. It crosses through southern Desert Hot Springs and the Seven Palms Valley area and continues east along the southern portion of the Indio Hills. In the vicinity of Edom Hill, this fault consists of one primary fault and at least three secondary splays. The closest surface trace of the Banning fault is located approximately 1.7 miles northeast of the project site.

The Mission Creek fault strand enters the Coachella Valley near the convergence of the San Bernardino and Little San Bernardino Mountains, near the northwestern city limits of Desert Hot Springs. It extends roughly along the easterly edge of the valley floor, bisecting the most populated portion of Desert Hot Springs and the Thousand Palms Oasis in the Indio Hills. The closest surface trace of the Mission Creek fault is located approximately 6.5 miles northeast of project site.

7.3 Historical Earthquakes

A large amount of seismic activity has been recorded in southern California. However, only relatively few historic earthquake events with magnitudes greater than 5.5 have been recorded in relatively close proximity of the site since the early 1900's. Superimposed on the area map in Figure 4 are earthquake epicenters recorded by the USGS between 1900 to present day. Notable historic earthquakes in Southern California of significance to the Project are listed in Table 4.

Table 4
Historic Earthquakes in Southern California

Earthquake Name	Date	Fault and Fault Type	Earthquake Magnitude*	Epicenter	
				Latitude	Longitude
Ridgecrest	July 4, 2019	Airport Lake Fault Zone (right-lateral strike-slip)	6.4 M_w	35.71°	-117.51°
Ridgecrest	July 5, 2019	Airport Lake Fault Zone (right-lateral strike-slip)	7.1 M_w	35.77°	-117.60°
Hector Mine	October 16, 1999big bear	Lavic Lake and Bullion Mountain faults (right-lateral strike-slip)	7.1 M_w	34.45°	-116.14°
Landers	June 28, 1992	Johnson Valley, Kickapoo/Landers, Homestead Valley, Emerson Valley and Camp Rock faults (right-lateral strike-slip)	7.3 M_w	34.20°	-116.44°
Big Bear	June 28, 1992	Fault unknown No surface rupture (left-lateral strike-slip)	6.4 M_s	34.2°	-116.83°
Joshua Tree	April 22, 1992	Eureka Peak fault (right-lateral strike-slip)	6.1 M_w	33.96°	-116.32°
Palm Springs**	July 8, 1986	Banning or Garnet Hill fault (right-lateral strike-slip)	5.6 M_L	34.0°	-116.61°
San Jacinto fault or Arroyo Salada	March 19, 1954	San Jacinto fault (right-lateral strike-slip)	6.4 M_w	33.28°	-116.18°
Desert Hot Spring	December 4, 1948	South Branch of San Andreas or Banning fault (right-lateral strike-slip)	6.0 M_w	33.93°	-116.38°

* M_W refers to Moment Magnitude scale
M_L refers to Local Magnitude scale
M_s refers to Surface Wave scale

** The July 8, 1986 earthquake event was strongly felt in the Palm Springs area and caused significant structural damages, as well as injuries. Both extensional and compressional fractures were identified in alluvium and asphalt along the surface trace of the Garnet Hill fault between Whitewater River and Hwy 62 following the seismic event. However, these fractures were concluded to be due to strong shaking and not associated with surface rupture (Person, Waverly, J. et al, 1986).

7.4 Potential for Surface Fault Rupture

7.4.1 Alquist-Priolo Earthquake Fault Zones

Official Maps of Earthquake Fault Zones were reviewed to evaluate the location of the project site relative to active fault zones. Earthquake Fault Zones (known as Special Studies Zones prior to 1994) have been established the State of California in accordance with the Alquist-Priolo Special Studies Zones Act enacted in 1972. The Act directs the State Geologist to delineate the regulatory zones that encompass surface traces of active faults that have a potential for future surface fault rupture. The purpose of the Alquist-Priolo Act is to regulate development near active faults in order to mitigate the hazard of surface fault rupture.

Based on a review of the Earthquake Fault Zone Map for the Desert Hot Springs Quadrangle, the subject project is not located within a State designated Earthquake Fault Zone for fault surface rupture hazard. The closest faults to the site that have been zoned as “Holocene active” by the State of California include the Banning and Mission Creek strands of the San Andres Fault zone, located approximately 1.7 and 6.5 miles northeast of the subject site. Other nearby faults meeting the State of California definition as “Holocene active” include the northwest-trending Jan Jacinto fault, located approximately 17 miles southwest of the site.

7.4.2 County of Riverside Earthquake Fault Zones

As an additional precaution, the County of Riverside has established supplemental fault zones that also require geologic evaluation prior to development of structures intended for human occupancy or other critical structures that can cause harm if damaged by an earthquake. The County of Riverside Fault Zone Maps obtained from the web application https://koordinates.com/layer/96848-riverside-county-ca-fault-zones/ indicate that the proposed location of Turbine #12 (near B-12 location) lies within a Riverside County Fault Zone established for the Garnet Hill fault, as shown on Plate 1 – Proposed Turbine Layout and Parcel Map.

To address the County of Riverside Fault Zone geologic evaluation requirements, Tetra Tech initially performed a stereographic photo-lineament evaluation for the subject project utilizing various historical aerial photographs dating from 1936 through 1996. As observed on the referenced aerial photographs, the westernmost extent of the GHF, to the northwest of the project site, is identified as a distinct geomorphic feature where its surface trace along the eastern flank of Whitewater Canyon forms a significant linear scarp in Holocene age alluvium. The linear scarp projects east-southeast towards the base of Whitewater Hill. However, continuous photo-lineament expression of the GHF diminishes rapidly toward the southeast where thicker accumulations of Holocene age alluvial deposits blanket the low-lying Whitewater River valley. No other photo-lineament expressions of the GHF were identified other than isolated hills, referred to locally as Hugo Hill and Garnet Hill, that are expressed as local bedrock highs projecting southeasterly of the fault scarp identified at the base of Whitewater Hill. Though geomorphic features presented on aerial photographs are better identified with the use of 3-D stereoscopic viewers, an illustrative 2-D summary of Tetra Tech’s photo-lineament evaluation for the subject project is presented on Plate 2 - Annotated 1953 Aerial Photograph Mosaic.

A comprehensive literature review of technical documents pertaining to the surface trace of the Garnet Hill fault was also performed. The findings of this review were summarized in Section 7.2, herein. From this review, it is clear that the current County Fault Zone that encompasses the Garnet Hill fault from Whitewater Hill to Garnet Hill was based primarily on geologic mapping and subsurface interpretations by Proctor and others during the 1960's. More recent mapping and structural interpretation by Dibblee (1982 & 2014), Yule and Sieh (2003) and Cordona (2013) provides compelling evidence that places the surface trace of the Garnet Hill fault on the north side and/or roughly parallel and beneath Interstate 10 freeway in the vicinity of the project site (see Illustration 2 and Figure 3). Additionally, the California Geological Survey has adopted a similar surface trace for the Garnet Hill fault but has not included this segment of the San Andreas Fault Zone within a State defined Earthquake Fault Zone due to lack of compelling youthful geomorphic expression.

Tetra Tech also contacted the Riverside County reviewing geologist via telephone and e-mail communications during July and August of 2020. The preliminary findings of our literature review and photo-lineament evaluation were shared and discussed to determine if supplemental subsurface exploration would be required to satisfy County requirements related to faulting in the vicinity of the project site. According to the County reviewing geologist, a subsurface fault investigation would not be required, unless a thorough review of historic photos indicated evidence of faulting through a proposed wind turbine location.

Based on the information summarized herein, Tetra Tech agrees with the interpreted surface trace of the GHF as interpreted by Dibblee (1982 & 2014) and Yule and Sieh (2003). Consequently, the surface trace of any Holocene-active fault is not known to pass beneath a proposed wind turbine location within the project site. Neither our field exploration nor literature review disclosed an active fault trace projecting to the ground surface in the project area. Therefore, the potential for surface rupture due to faulting during the design life of the proposed development is considered low.

7.5 Seismic Hazard Zones

Maps of seismic hazard zones are issued by the California Geological Survey (CGS, formerly California Department of Conservation, Division of Mines and Geology (CDMG)) in accordance with the Seismic Hazards Mapping Act enacted in April 1997. The intent of the Seismic Hazards Mapping Act is to provide for a statewide seismic hazard mapping and technical advisory program to assist cities and counties in developing compliance requirements to protect the public health and safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure and other seismic hazards caused by earthquakes.

According to the website https://maps.conservation.ca.gov/cgs/EQZApp/app/, liquefaction and landslide hazards have not yet been evaluated by the CGS in the project area of the date of this report. However, Tetra Tech has addressed relevant seismic hazards for the project as presented in the following sections of this report.

7.6 Liquefaction Hazard

Liquefaction of soils can be caused by ground shaking during earthquakes. Research and historical data indicate that loose, relatively clean granular soils and low plasticity silts are susceptible to liquefaction and dynamic settlement, whereas the stability of the majority of clayey silts, silty clays and clays are not typically adversely affected by ground shaking. Liquefaction is generally known to occur in saturated or near-saturated cohesionless soils at depths shallower than about 50 feet. Materials that are above the groundwater table are not considered susceptible to liquefaction, although they may undergo settlement due to seismic shaking.

The site is mapped within the Riverside County hazard maps https://koordinates.com/data/global/north-america/united-states/california/riverside/ as being in an area with moderate susceptibility of liquefaction (see Figure 5 -- Seismic Hazard Zones Map). Since the groundwater is found at depths greater than 150 feet and since the subsurface materials are in a very dense state, the potential for liquefaction is considered minimal.

7.7 Seismically Induced Settlement of Unsaturated Materials

Although the liquefaction hazard is considered minimal, an evaluation of the settlement of the unsaturated soils due to dynamic shaking was conducted at the site. The alluvial fan surface materials consist of cobbles with gravel and sand with locally abundant boulder size particles which are in general not prone to significant compression due to seismic shaking.

7.7.1 Seismic Demand

The ground motion parameters were obtained for the subject site at coordinates 33.910926 N, -116.603786 W, for Site Class C, corresponding to a very dense soil. The Office of Statewide Health Planning and Development (OSHPD) website application (https://seismicmaps.org/) and the USGS Seismic Hazard Interactive Deaggregation website (https://earthquake.usgs.gov/hazards/interactive/) utilizing the 2014 Dynamic Conterminous U.S. v.4.2.0 edition fault model were used to obtain the seismic demand parameters (see Appendix E – Seismic Demand).

Based on the 2019 CBC, a ground motion equivalent to the Maximum Considered Earthquake (MCE) was selected. The mapped geometric mean Peak ground acceleration (PGA_M) was estimated to be approximately 1.228g. From the deaggregation analysis for a return period of 2,475 years (2% in 50 years), an earthquake with a mean magnitude Mw 7.56 and located at a mean distance of approximately 3.05 km (approximately 1.9 miles) was determined to be the most significant ground motion contributing to the seismic hazard at the site. The San Andreas (San Gorgonio Pass-Garnet Hill) Fault was the largest contributor to the seismic hazard at the site (75.5 percent). The ground motion parameters cited above were used in the dynamic settlement analyses.

7.7.2 Seismically Induced Settlement

Seismically induced settlement can occur in saturated sands due to liquefaction or in unsaturated sands due to densification of the soil matrix. The potential for seismically induced settlement of unsaturated materials was estimated utilizing the procedure outlined in Pradel (1998a and 1998b).

Table 5 presents the summary of estimates of seismically induced settlement of materials encountered in the investigative borings generally within the upper 50 feet and below the foundation depth of 12 feet. Based on this table it can be observed that the anticipated magnitudes of the seismically induced settlements are minor and can be readily accommodated by the turbine foundations. Differential settlements can be assumed to be about half of the total settlements. The calculation of the settlement analyses are presented in Appendix F – Seismic Settlement.

Table 5
Summary of Settlement of Unsaturated Materials Analyses
Seismic demand: M_w= 7.56, PGA_M = 1.228g

Boring No.	Boring Depth (feet)	Settlement of Dry Sands below 12 feet (inches)
B-1	60.3	< 0.1
B-2	35.5	< 0.1
B-3	40	< 0.1
B-4	30.6	< 0.1
B-5	40.3	< 0.1
B-6	40.8	< 0.1
B-7	40.9	< 0.1
B-8	40.8	< 0.1
B-9	27	< 0.1
B-10	30.6	< 0.1
B-11	41.3	0.1
B-12	60.4	0.4
B-13	40.5	< 0.1
B-14	40.4	0.3
B-15	40.4	< 0.1
B-16	60.9	0.8

7.8 Lateral Spreading

Due to the low risk of liquefaction, lateral spreading is not considered to be a hazard to the site.

7.9 Landslide/Rockfall Hazard

Due to the relatively flat topography and the absence of significant slopes the potential for landslides or rockfalls is not considered a hazard for the site.

7.10 Collapsible Soils

The phenomenon of hydro-consolidation is typically exhibited in geologically young, unconsolidated, low-density, loose, dry soils commonly present in arid to semi-arid regions. Collapsible soils are usually composed of granular particles that are supported by a clay or silt matrix that can be chemically cemented in place creating a porous structure. The bonds supporting this porous structure have enough shear strength to support loads at low moisture contents, however, once water is introduced the cemented bond structure breaks down and the granular particles are re-arranged causing significant volume loss.

The soil samples tested as part of this investigation exhibited hydro-consolidation potential on the order of 0.3 to 1.8 percent at a vertical stress of 400 psf. However, these samples were devoid of the strengthening influence of gravel, cobbles and boulders and therefore the results cannot be extrapolated for the response of the overall site stratigraphy. Also, unlike with typical new developments with significantly modified drainage patterns and introduction of irrigation, installation of wind turbines is not deemed to change the infiltration regime of the site. Therefore, the potential for hydrocollapse of engineering significance at the site is considered low.

7.11 Subsidence

Land subsidence is the lowering of the ground surface due to extraction or lowering of groundwater water levels or other fluids within the subsurface soil pores. The fluid withdrawal causes the alluvial sediments in the basin to compact. Damage caused by subsidence can be visible soil cracks, fissures, or surface depression.

The demand for water in the Coachella Valley has exceeded the deliveries of imported surface water, and groundwater levels have been declining as a result of increased pumping. The site is not located in an area mapped by the United States Geological Survey (USGS) where either historical or current subsidence has been recorded (https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html). However, significant land subsidence has been recorded by the USGS south of the site in the southern sections of the Coachella Valley. The County of Riverside web application https://gis.countyofriverside.us/Html5Viewer/?viewer=MMC_Public also includes much of the southern Coachella Valley in a zone designated as undergoing “active” subsidence, while much of the northern Coachella Valley that includes the project site is designated in a zone as being “susceptible” to subsidence.

Recent studies conducted by the USGS have determined that portions of the northern part of the Coachella Valley have been undergoing ground surface uplift in areas associated with groundwater replenishment facilities (Sneed, M., and Brandt, J.T., 2020). Most notable uplift was found at the Whitewater Groundwater Replenishment Facility established in the 1970's, which is located along the southern perimeter of the project site. Groundwater replenishment facilities, such as the Whitewater facility and newer replenishment facilities recently completed in the southern Coachella Valley, are primary contributors to the reversal of long-term groundwater level declines throughout the valley (https://piahs.copernicus.org/articles/382/809/2020/). Therefore, the potential for ground subsidence of engineering significance at the site is considered low.

7.12 Scour

Turbines 1 through 4, and Turbines 7 and 8, and the meteorological tower are located within the Whitewater River Floodplain Conservation Area (FEMA Flood Zone A, i.e., area with a 1% annual chance of flooding). Therefore, flooding of the area should be expected during the rainy months, typically between October and March. As a result, there is potential for scour at these locations. Evaluation of scour or scour potential was not part of the present scope of work, but scour should be evaluated to assess possible exposure of the turbine foundations and provide appropriate protection measures.

7.13 Debris Flows

Debris flows are geological phenomena in which water-laden masses of soil and rock fragments rush down mountainous terrain, funnel into stream channels, entrain objects in their paths, and form thick, muddy deposits on valley floors. Debris flows can make up significant percentages of many alluvial fans that emanate from steep mountain fronts.

Whitewater Canyon was a significant source of damaging debris flows prior to relatively recent stormwater protection and flood control improvements implemented by the Coachella Valley Water District (CVWD) and its predecessor agencies. Within CVWD's current boundaries there are 16 stormwater protection channels. The entire system includes approximately 135 miles of channels built along the natural alignment of dry creeks that naturally flow from the surrounding mountains into the Whitewater River located northwest, west and south of the Project. Along with the channels, a number of dikes and levees have been designed and built to collect rapidly flowing flood water as it pours from the adjacent mountains onto the valley floor (source: https://www.cvwd.org/165/Stormwater-Protection-Flood-Control).

The proposed turbines will be located in FEMA designated Flood Zones A and X (area determined to be outside the 500-year flood zone and protected by levee from 100-year flood). The turbines will not be located within the primary drainage course of the Whitewater River, where adverse impacts from future debris flows are most likely. Upgradient and west of the Project, the primary drainage course of the Whitewater River is confined by a manmade earthen levee system that should decrease the potential for debris flows across wind turbine improvements. Therefore, likelihood of hazards associated with debris flows impacting the turbines at the site is considered low.

7.14 Wind Erosion

The Coachella Valley is subjected to frequent wind events throughout each year. One of the windiest locations coincides with the confluence of the San Gorgonio Pass and Whitewater River Valley directly affecting the project area. Much of the Project positioned within this windy zone is covered by thick accumulations of coarse-grained alluvial fan deposits that are generally comprised of dense, sandy gravel with abundant cobbles and boulders. This coarse-grained mantle provides an effective surface armor for much of the site to naturally mitigate significant wind erosion. However, seasonal rainfall events fill intermittent stream channels and gullies bring in finer-grained sediments that are subject to wind erosion upon drying. Since the proposed turbines will be supported by embedded concrete foundations and positioned on elevated pedestals, adverse impacts related to wind erosion are considered low provided the facility ownership provides routine site maintenance for minor soil loss or build up.

7.15 Seiches

The site is not located near a reservoir or a lake, therefore there is no hazard of seiches.

8. DESIGN RECOMMENDATIONS

8.1 General

Based on the results of the field explorations and engineering analyses, Tetra Tech considers the proposed wind turbine and meteorological tower construction feasible from a geotechnical standpoint, provided that the recommendations contained in this report are incorporated into the design plans and implemented during construction.

Recommendations are also provided for assessment of the existing mat foundation for the transformer to be used for a replacement, larger transformer.

Observations and laboratory tests indicate that site soils have high shear strengths, relatively low compressibility with only minor hydro-consolidation potential, and very low expansion potential. Therefore, mat-type foundations are considered to be suitable for the proposed turbine and meteorological tower construction.

Based on the available water-soluble sulfate concentration, the potential of buried concrete to be corroded by on-site soils is considered low. Field and laboratory tests indicate that site soils have high electrical resistivity; therefore, the metallic corrosion potential of these soils is low. Thermal resistivity testing results are provided to assist the designer of the underground utilities/cables.

8.2 Clearing and Grubbing

Prior to any site grading, surface vegetation, trash, and debris should be removed and disposed of offsite. Existing subsurface installations, such as abandoned foundations, pipes, cables, utility collectors, and/or tanks, if present, should be removed or abandoned per the Geotechnical Engineer's recommendations and in accordance with applicable regulations.

8.3 Site Preparation

Site grading is anticipated to encounter some excavation difficulties due to the broad presence of cobbles and boulders throughout the site, especially in the western portion near Turbines 1 through 11 and the proposed location of the meteorological tower. Excavations up to about 14 feet deep are expected to accommodate the wind turbine foundations and about 6 feet deep for the meteorological tower. It is anticipated that heavy duty excavation equipment will be needed to handle the dense soils, cobbles, and boulders.

8.3.1 Subgrade Preparation for Wind Turbine and Meteorological Tower Foundations

A smooth uniform foundation subgrade, devoid of cobbles and boulders, is recommended. The excavated subgrade should be inspected and accepted by the Geotechnical Engineer. The foundation subgrade should be scarified to a depth of 4 inches, moisture-conditioned wet of the optimum moisture content, and compacted to at least 95 percent of the maximum dry density, as evaluated by the latest version of ASTM D1557 (Modified Proctor test).

Any protruding cobbles and boulders should be removed, and any resulting voids should be backfilled with soils without particles greater than 3 inches in the largest dimension. The soil backfill should be placed in 8-inch loose lifts, moisture-conditioned wet of the optimum moisture content and compacted to at least 95 percent of the relative compaction (ASTM D1557). If effective compaction of the soil backfill is not possible or is not practicable, the voids may be backfilled with a 2-sack cement-sand slurry.

Excavated on-site soils may be re-used as compacted fill, provided they are free of organics, deleterious materials, debris, and particles over 3 inches in the largest dimension. Any localized zones of loose and/or unstable soil encountered at the subgrade level should be overexcavated and recompacted as indicated above or as treated as described in the Subgrade Preparation if Loose Soils are Encountered section below.

8.3.2 Subgrade Preparation if Loose Soils are Encountered

Localized zones of loose/soft soils are not likely to be encountered during the grading of the foundation subgrade. However, in such an unlikely event, any unstable soils should be overexcavated and recompacted. If overexcavation and recompaction is not practical, the specific type of remediation and associated area limits will need to be evaluated in the field by the Geotechnical Engineer.

8.4 General Fills and Utility Trench Backfill

All fill placement associated with the replacement of overexcavated soils, fill placed to achieve finish grade level, or utility trench backfill, should be moisture-conditioned at least wet of the optimum moisture content and compacted to at least 90 percent of the relative compaction (ASTM D1557). Fill should be placed in horizontal lifts not more than 8 inches in loose, uncompacted thickness.

Excavated on-site soils may be re-used as compacted fill, provided they are free of organics, deleterious materials, debris, and particles over 3 inches in largest dimension. Locally, particles up to 6 inches in largest dimension may be incorporated in the fill soils during grading based on specific approval and placement recommendations provided by the Geotechnical Engineer.

In the event that any soil materials are imported to the site, such soils should be sampled, tested, and approved by the Geotechnical Engineer of Record prior to arrival on-site. In general, any soils imported to the site for use as fill should be predominantly granular and have an Expansion Index less than 20. Additional recommendations for site grading are provided in the "General Site Grading Recommendations" section of this report.

8.5 Compaction Characteristics and Dry Density of Near Surface Soils

A summary of the compaction characteristics, i.e., maximum dry density and optimum moisture content, obtained for bulk samples of soils taken near the surface throughout the project site is provided in Table 6A. Moisture contents and dry densities of relatively undisturbed ring samples taken at shallow depths are presented in Table 6B. It is noted that due to the cobbly/bouldery nature of the on-site soils and the associated sampling difficulties, only a limited number of relatively undisturbed samples were possible to collect for the testing.

Table 6A
Compaction Characteristics

Boring No.	Sample No.	Depth (feet)	Max Dry Density (pcf)	Optimum Moisture Content (%)
B-3	SK-1	0-5	139.7	4
B-5	SK-A	0-5	129.6	7
B-12	SK-1	0-5	134.0	7
B-94	SK-1	0-5	131.6	6
B-95	SK-1	0-5	132.6	6
B-97	SK-1	0-5	127.7	8

Table 6B
In Situ Moisture Contents and Dry Densities

Boring No.	Sample No.	Depth (feet)	In situ Dry Density (pcf)	In situ Moisture Content (%)
B-12	R-2	2.5-4	115	3.4
B-95	R-3	5-6.5	n/a	0.6

8.6 Temporary Sloped Excavations and Utility Trench Excavations

The on-site soils are expected to pose excavation difficulties, and therefore heavy-duty earth-moving equipment will be needed due to the significant presence of cobbles and boulders. It is anticipated that all excavations to reach subgrade level can be performed as sloped excavations, i.e., without the need for shoring. Where space for sloped sides is not available and shoring will be required, Tetra Tech can provide appropriate shoring recommendations.

All excavations should be performed in accordance with CalOSHA regulations. The on-site soils may be considered a Type C soil to a depth of 20 feet as defined the current CalOSHA soil classification.

Unsurcharged excavations: Temporary short-term unsurcharged excavations in dry conditions should be sloped back at an inclination of 1.5(H):1(V) or flatter.

Surcharge setback recommendations: 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 1(H):1(V), but no closer than 4 feet. A greater setback may be necessary when considering significant surcharge loads such as heavy vehicles, concrete trucks and cranes. Tetra Tech should be advised of such heavy surcharges so that specific setback requirements can be established. Alternatively, a shoring system may be designed to allow reduction in the setback distance.

8.7 Seismic Design Parameters per 2019 CBC

Seismic site class was determined based on the 8 MASW seismic surveys performed by GeoVision. The MASW surveys yielded subsurface shear wave velocities interpreted in about 3 to 30-foot depth intervals that were averaged as V_{s100} over the upper 30 meters, i.e., about 100 feet. The shear wave velocity averages at each MASW location depth-weighted per ASCE 7-16 Chapter 20 are presented in Table 7. Based on the V_{s100} values, the entire project site is Site Class C, i.e., V_{s100} > 1,200 feet/sec.

Table 7
Averaged Shear Wave Velocities over the Upper 100 feet

	MASW Survey Line							
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
V_{s100} (ft/sec)	1,635	1,634	1,593	1,623	1,529	1,452	1,344	1,401
Applicable to Turbine Nos.	1 & 2	3 & 4	5 & 6	7 & 8	9 &10	11	12 & 13	14, 15 &16

The seismic design coefficients provided below in Table 8 are based on Chapter 16 of the 2019 CBC, and on information provided by the SEAOC/OSHPD website application (https://seismicmaps.org/).

Table 8
2019 CBC Seismic Design Parameters
Latitude 33.910926, Longitude -116.603786

Site Class (Table 20.3-1 ASCE 7-16)	**C*** (very dense soil and soft rock)	
Maximum Considered Earthquake MCE_R Spectral Response Acceleration	Short Period (0.2 seconds), S_{MS}	2.847g**
	1 Second Period, S_{M1}	1.347g**
Design Earthquake Spectral Response Accelerations	Short Period (0.2 seconds), S_{DS}	1.898g**
	1 Second Period, SD_1	0.898g**
Site Modified Peak Ground Acceleration PGA_M	1.228g**	

* Site Class based on measured V_{s100}
** Values from SEAOC/OSHPD website https://seismicmaps.org/ based on ASCE 7-16

8.8 Foundations for Turbines and Meteorological Tower

Wind turbines and the meteorological tower may be supported on spread-foot and mat foundations, respectively, supported on native subgrade soils prepared as recommended in this report. It is anticipated that the wind turbine foundations will have a diameter of about 70 to 75 feet and be embedded at a depth of about 10 to 14 feet, whereas the meteorological tower foundation will be a square mat foundation 23 by 23 feet in footprint with an embedment depth of at least 6 feet. The design parameters presented in Tables 9 and 10 should be used for design of the wind turbine and meteorological tower foundations, respectively.

Table 9
Summary of Design Parameters for Wind Turbine Foundations
70 to 75 feet Diameter Mat Foundation Embedded at least 10 feet

Foundation Soil Geotechnical Properties	
Soil Unit Weight	125 pcf
Angle of Internal Friction	40 degrees
Cohesion	100 psf
Geotechnical Design Parameters	
Foundation Anticipated Contact Pressure	- 2,500 psf under normal operating conditions - 3,000 psf under normal extreme wind - 3,500 psf under abnormal extreme - 4,000 psf under seismic conditions
Foundation Settlement for 4,000 psf contact pressure (acceptance differential settlement criteria of 3 mm / meter, i.e., 2.8 inches over 75 feet)	Western Part of the Site (Turbines 1 -- 11): Total: 1.2 inches - Differential: 0.6 inches over a distance of 30 feet or 1.5 inches over 75 feet
	Eastern Part of the Site (Turbines 12 -- 16): Total: 1.7 inches Differential: 0.85 inches over a distance of 30 feet or 2.1 inches over 75 feet
Allowable Coefficient of Friction (includes Factor of Safety of 1.5)	0.40 (concrete on soil)
Allowable Passive Pressure (includes Factor of Safety of 2)	- Neglect the upper 1 foot of embedment - 130 pounds per cubic foot equivalent fluid density (pcf EFD) for submerged conditions - 250 pcf EFD for dry conditions - 1/3 increase for seismic or wind loading

Table 10
Summary of Design Parameters for Meteorological Tower Foundations
23 feet by 23 feet Square Mat Foundation Embedded at least 6 feet

Foundation Soil Geotechnical Properties	
Soil Unit Weight	125 pcf
Angle of Internal Friction	38 degrees
Cohesion	0 psf
Geotechnical Design Parameters	
Mat Foundation Estimated Allowable Pressure	- 3,500 psf for submerged conditions - 1/3 increase for seismic or wind loading
Estimated Mat Settlement	Total: 1 inch - Differential: 0.5 inch over a distance of 30 feet
Allowable Coefficient of Friction (includes Factor of Safety of 1.5)	0.40 (concrete on soil)
Allowable Passive Pressure (includes Factor of Safety of 2)	- Neglect the upper 1 foot of embedment - 130 pounds per cubic foot equivalent fluid density (pcf EFD) for submerged conditions - 250 pcf EFD for dry conditions - 1/3 increase for seismic or wind loading

Lateral loads may be resisted by friction and by the passive resistance of the underlying materials. A one-third increase in the passive value may be used for wind or seismic loads. The passive resistance of the materials may be combined with the frictional resistance without reduction in evaluating the total lateral resistance.

For the on-site coarse-grained soils, a reference modulus of subgrade reaction k_1 for a 1-foot by 1-foot square plate of 290 pci can be used for the foundation design. The design modulus of subgrade reaction k in pci for a circular foundation with diameter greater than 40 feet and for saturated conditions can be determined as:

$$k = k_1 \frac{(\sqrt{\pi R^2} + 1)^2}{4\pi R^2}$$

where R is the radius of the concrete foundation in feet, but no more than 8 times the representative thickness of the concrete foundation. This recommended design subgrade modulus is conservatively provided for saturated condition because Turbines 1 through 4 and 7 and 8 are located within FEMA Flood Zone A with a 1% annual chance of flooding. For unsaturated conditions, e.g., for Turbines 5, 6, and 9 through 16, the subgrade modulus may be increased by a factor of 2.

For the meteorological tower foundation, which is also located within FEMA Flood Zone A, the design modulus of subgrade reaction *k* in pci for a concrete square element for saturated conditions can be determined as can be determined as:

$$k = k_1 \frac{(B\ +1)^2}{8B^2}$$

where B is the side of the concrete foundation in feet, but no more than 14 times the thickness of the concrete foundation. Again, for unsaturated conditions, the subgrade modulus could be increased by a factor of 2.

Alternatively, the spread-foot foundations at locations may be designed using the shear wave velocity and small strain shear modulus profiles presented in Tables 11A through 11H below. A Poisson's ratio of 0.3 should be used for the design of the foundations on on-site soils.

Table 11A
Wind Turbines 1 and 2
V_{s100} = 1,635 m/sec, Site Class C

Depth to Top of Layer (ft)	Layer Thickness (ft)	Total Unit Weight (pcf)	Shear Wave Velocity V_s (ft/s)	Estimated Small Strain Shear Modulus G (psf)
0	3.3	116	712	1.83E+06
3.3	6.6	122	1,160	5.10E+06
9.8	9.8	125	1,487	8.58E+06
19.7	13.1	126	1,684	1.11E+07
32.8	16.4	127	1,769	1.23E+07
49.2	23	127	1,841	1.34E+07
72.2	29.5	128	1,890	1.42E+07
101.7	Rest of the profile	130	2,249	2.04E+07

Table 11B
Wind Turbines 3 and 4 and the Meteorological Tower
V_{s100} = 1,634 m/sec, Site Class C

Depth to Top of Layer (ft)	Layer Thickness (ft)	Total Unit Weight (pcf)	Shear Wave Velocity V_s (ft/s)	Estimated Small Strain Shear Modulus G (psf)
0	3.3	116	739	1.97E+06
3.3	6.6	125	1,527	9.05E+06
9.8	9.8	126	1,666	1.09E+07
19.7	13.1	125	1,532	9.11E+06
32.8	16.4	125	1,586	9.76E+06
49.2	23	127	1,760	1.22E+07
72.2	29.5	128	1,907	1.45E+07
101.7	Rest of the profile	130	2,250	2.04E+07

Table 11C
Wind Turbines 5 and 6

V_{s100} = 1,592 m/sec, Site Class C

Depth to Top of Layer (ft)	Layer Thickness (ft)	Total Unit Weight (pcf)	Shear Wave Velocity V_s (ft/s)	Estimated Small Strain Shear Modulus G (psf)
0	3.3	111	572	1.13E+06
3.3	4.9	122	1,099	4.58E+06
8.2	8.2	127	1,791	1.27E+07
16.4	13.1	127	1,827	1.32E+07
29.5	16.4	127	1,683	1.12E+07
45.9	23	127	1,710	1.15E+07
68.9	29.5	127	1,764	1.23E+07
98.4	Rest of the profile	130	2,227	2.00E+07

Table 11D
Wind Turbines 7 and 8

V_{s100} = 1,623 m/sec, Site Class C

Depth to Top of Layer (ft)	Layer Thickness (ft)	Total Unit Weight (pcf)	Shear Wave Velocity V_s (ft/s)	Estimated Small Strain Shear Modulus G (psf)
0	3.3	111	577	1.15E+06
3.3	4.1	119	890	2.93E+06
7.4	9	128	1,860	1.38E+07
16.4	13.1	128	1,837	1.34E+07
29.5	19.7	127	1,728	1.18E+07
49.2	26.2	127	1,799	1.28E+07
75.5	32.8	128	1,842	1.35E+07
108.3	Rest of the profile	129	1,987	1.58E+07

Table 11E
Wind Turbines 9 and 10

V_{s100} = 1,517 m/sec, Site Class C

Depth to Top of Layer (ft)	Layer Thickness (ft)	Total Unit Weight (pcf)	Shear Wave Velocity V_s (ft/s)	Estimated Small Strain Shear Modulus G (psf)
0	3.3	120	947	3.34E+06
3.3	4.9	124	1,323	6.74E+06
8.2	8.2	126	1,655	1.07E+07
16.4	13.1	125	1,608	1.00E+07
29.5	16.4	125	1,547	9.29E+06
45.9	23	125	1,513	8.89E+06
68.9	23	126	1,568	9.62E+06
91.9	Rest of the profile	127	1,736	1.19E+07

Table 11F
Wind Turbine 11

V_{s100} = 1,452 m/sec, Site Class C

Depth to Top of Layer (ft)	Layer Thickness (ft)	Total Unit Weight (pcf)	Shear Wave Velocity V_s (ft/s)	Estimated Small Strain Shear Modulus G (psf)
0	3.3	119	904	3.02E+06
3.3	4.9	122	1,179	5.27E+06
8.2	8.2	124	1,423	7.80E+06
16.4	13.1	124	1,436	7.94E+06
29.5	19.7	124	1,441	8.00E+06
49.2	26.2	125	1,459	8.26E+06
75.5	32.8	126	1,692	1.12E+07
108.3	Rest of the profile	129	1,970	1.55E+07

Table 11G
Wind Turbines 12 and 13

V_{s100} = 1,344 m/sec, Site Class C

Depth to Top of Layer (ft)	Layer Thickness (ft)	Total Unit Weight (pcf)	Shear Wave Velocity V_s (ft/s)	Estimated Small Strain Shear Modulus G (psf)
0	2.3	112	591	1.21E+06
2.3	4.3	124	1,305	6.56E+06
6.6	8.2	124	1,458	8.19E+06
14.8	13.1	124	1,291	6.42E+06
27.9	19.7	123	1,272	6.18E+06
47.6	26.2	124	1,371	7.24E+06
73.8	32.8	125	1,554	9.37E+06
106.6	Rest of the profile	127	1,769	1.23E+07

Table 11H
Wind Turbines 14, 15 and 16

V_{s100} = 1,402 m/sec, Site Class C

Depth to Top of Layer (ft)	Layer Thickness (ft)	Total Unit Weight (pcf)	Shear Wave Velocity V_s (ft/s)	Estimated Small Strain Shear Modulus G (psf)
0	3.3	119	869	2.79E+06
3.3	13.1	124	1,319	6.70E+06
16.4	16.4	124	1,345	6.97E+06
32.8	19.7	124	1,443	8.02E+06
52.5	23	125	1,470	8.39E+06
75.5	26.2	125	1,515	8.91E+06
101.7	Rest of the profile	128	1,906	1.44E+07

8.9 Foundations for Replacement Transformer

The western transformer of the two transformers at the existing substation located at the east limit of the project area (see Plate 1 – Proposed Turbine Layout and Parcel Map) will be replaced. The existing transformer weighs approximately 130,000 pounds and is founded on 11 feet by 16 feet mat, 3 feet thick. The replacement transformer weighs approximately 210,000 pounds and is planned to be installed on the existing mat. Whereas the existing transformer produces a contact bearing pressure of about 750 psf, the replacement transformer will result in a contact pressure of about 1200 psf. These contact pressures are well within the allowable bearing capacity of the mat foundation estimated to be about 4,000 psf for an associated settlement of less than 1 inch. Given that the existing transformer will be first removed, i.e., the mat will be unloaded, and then the new, heavier, transformer will be installed, which will increase the contact pressure by about 450 psf, or 60 percent, the associated settlement of the mat due to this unload/reload cycle and the additional surcharge will be nominal and less than about 0.5 inches.

The existing transformer mat is surrounded by an oil retention trough about 7 feet wide and 2.5 feet deep with 6 inches thick bottom. The trough bottom is doweled into the transformer mat foundation by #4 rebar 18 inches o.c. as shown on Illustration 3 below.

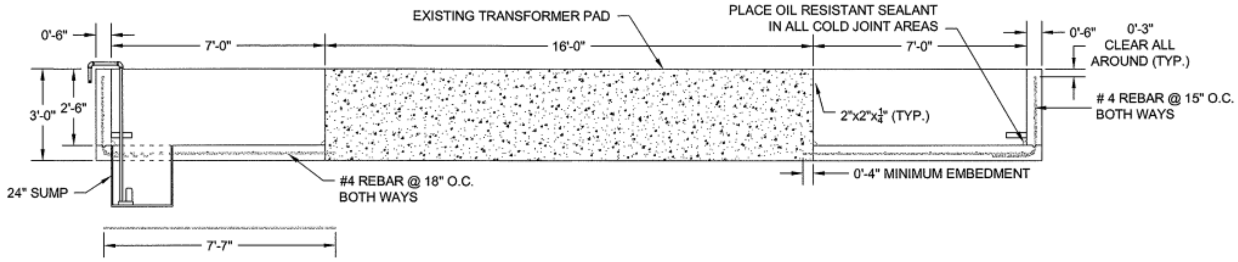

Illustration 3. **S**chematic of the existing mat foundation and the surrounding oil retaining trough.

This connection between the oil retention trough bottom and the mat foundation presents a weak joint that is sensitive even to minimal differential settlement. Although the surcharge on the existing foundation due to the replacement transformer and the associated settlement are minor, there will be virtually no load transfer onto the bottom of the oil retention trough and consequently the oil retention trough will settle only very minimally. This will create an abrupt differential settlement across the connection between the trough bottom and the existing mat and will likely result in some cracking. The cracking is expected to be relatively minor and may be possible to be repaired by epoxy infilling or similar sealant or the through bottom may be rebuilt.

Provided below in Table 12 are the design parameters for the assessment of the existing mat foundation.

Table 12
Summary of Design Parameters for Existing Transformer Mat Foundation
11 feet by 16 feet Square Mat Foundation Embedded at least 24 inches

Foundation Soil Geotechnical Properties	
Soil Unit Weight	125 pcf
Angle of Internal Friction	38 degrees
Cohesion	0 psf
Geotechnical Design Parameters	
Mat Foundation Estimated Allowable Pressure	- 4,000 psf - 1/3 increase for seismic or wind loading
Estimated Mat Settlement due to unloading by removal of the existing about 130,000-pound transformer and reloading due to installation of a replacement about 210,000-pind transformer	Total: 0.5 inch - Differential: 0.3 inch over a distance of 30 feet
Allowable Coefficient of Friction (includes Factor of Safety of 1.5)	0.40 (concrete on soil)
Allowable Passive Pressure (includes Factor of Safety of 2)	- Neglect the upper 1 foot of embedment - 250 pcf EFD for dry conditions - 1/3 increase for seismic or wind loading

Lateral loads may be resisted by friction and by the passive resistance of the underlying materials. A one-third increase in the passive value may be used for wind or seismic loads. The passive resistance of the materials may be combined with the frictional resistance without reduction in evaluating the total lateral resistance.

The substation is located within FEMA designated Flood Zone X, i.e., outside the 500-year flood zone and protected by levee from 100-year flood. For the on-site coarse-grained soils, the reference modulus of subgrade reaction k_1 for a 1-foot by 1-foot square plate of 290 pci can be used for the foundation design. The design modulus of subgrade reaction k in pci for a rectangular foundation in unsaturated conditions can be determined as:

$$k = k_1 \frac{(B+1)^2}{4B^2} \frac{1+0.5\frac{B}{L}}{1.5}$$

where B is the side of the concrete foundation in feet, but no more than 14 times the thickness of the concrete foundation. For saturated conditions, the subgrade modulus should be decreased by a factor of 2.

8.10 Exterior Concrete Slabs-On-Grade

Exterior concrete slabs should be placed on subgrade prepared in accordance with the recommendations provided in the “Site Preparation” and section of this report. A Structural Engineer or an Engineer specialized in concrete design should be consulted if cracking of the exterior slabs is to be minimized. As a minimum for exterior walkways, it is recommended that narrow strip concrete slabs, such as sidewalks, be reinforced with at least No. 4 reinforcing bars placed longitudinally at 18 inches on center. Wide exterior slabs should be reinforced with at least No. 4 reinforcing bars placed 18 inches on center, each way. Welded wire mesh reinforcement is not recommended. Reinforcement should extend through the control joints to reduce the potential for differential movement. Control joints should be provided as recommended by American Concrete Institute (ACI) guidelines and at a maximum spacing (in feet) of 2 to 3 times of the slab thickness (in inches), but generally no more than 10 feet. All joints should form approximately square patterns to reduce potential for randomly oriented shrinkage cracks. The control joints should be tooled at the time of the pour or sawcut to ¼ of slab depth within 6 to 8 hours of concrete placement. Precautions should be taken to prevent curling of slabs (refer to ACI guidelines).

8.11 Utility Trenches

Utility trench excavations should follow the recommendations provided in the “Temporary Slopes and Trench Excavations” section of this report. Construction recommendations for the trench bottom preparation and trench backfill are provided below.

8.11.1 Foundations Adjacent to Utility Trenches

The bottom of trenches that are required for any buried utilities should be kept outside a zone defined by a 1(H):1(V) plane projected downward from the outside bottom edge of any existing or proposed foundation. Backfill materials and procedures shall conform to the recommendations provided in the Site Preparation and General Site Grading Recommendations sections of this report. If any utilities need to be placed within the zone of influence, the utility conduit (pipes, cables) should be designed to account for the increased surcharge from the foundation pressures and to withstand potential differential settlement between the surcharged and unsurcharged segments of the pipe/cable. Generally, the utility conduits within the impacted zone should be protected by concrete encasement or utilidors.

For utility conduits that will cross beneath foundations, the piping and encasement should be designed to withstand differential settlements of up to 1 inch over a distance equal to half of the depth of the pipe/cable crown below the bottom of the foundation element. Geotechnical Engineer should be contacted to review any specific utility interaction configurations and their proposed mitigation.

8.11.2 Utility Trench Bottom Preparation

Trench bottom preparation should produce a uniform, firm, and unyielding subgrade. The exposed trench bottom should be probed and accepted by the Geotechnical Engineer. Any particle size greater than 3 inches should be removed. Surface water should be controlled so that the trench subgrade is protected even during periods of heavy rainfall.

8.11.3 Utility Trench Backfill

Selection of electrical conduits (cable) trench backfill materials is of particular importance for wind turbine projects as the backfill needs to meet the thermal resistivity properties assumed in the design. The electrical conduit trench backfill material is assumed to be on-site soils processed by removing coarse gravel sizes (clasts greater than ¾ inch). If different or imported soils are to be used, the thermal resistance and gradation of such materials should be evaluated and accepted by the electrical engineer and the Geotechnical Engineer, respectively.

Although the recommendations provided below are intended for electrical and fiberoptic cables, the conventional trench terminology utilizes reference to a "pipe", rather than to a "cable", e.g., pipe zone, pipe bedding. Therefore, for consistency, the conventional terminology is utilized, although the word cable and pipe may be inserted alternatively.

<u>Bedding and pipe zone backfill</u>. The bedding is the material placed in the bottom of the trench on which the cable is laid. Bedding material should extend at least 6 inches below the bottom of cable and up to the cable springline level. The pipe zone is defined as the area above the bedding, around the cable, and up to typically at least 12 inches over the cable.

Bedding and pipe zone backfill material should consist of clean sand, i.e., sand with less than about 30 percent fines and no more than 20 percent of fine gravel. The excavated sandy soils are likely suitable bedding and pipe zone backfill material as long as gravel sizes greater than ¾ inch are removed. The pipe bedding material should be placed over the full width of the trench on firm and unyielding subgrade soils approved by the Geotechnical Engineer. After placement of the cable, the pipe zone backfill should be brought up uniformly on both sides of the cable to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the cable haunches.

The bedding and pipe zone backfill should be moisture-conditioned to at least wet of optimum moisture and compacted to achieve relative compaction of at least 95 percent per ASTM D1557 in horizontal lifts no greater than 6 inches. The use of mechanized compaction equipment within the pipe zone should be carefully controlled to avoid overstressing or damaging the cable.

<u>General trench backfill.</u> This zone extends from the top of the pipe zone to the finished grade. The backfill may consist of approved excavated soil devoid of particles greater than 3 inches in the largest dimension, moisture-conditioned wet of optimum moisture content and compacted to at least 90 percent of relative compaction per the latest version of ASTM D1557. Lift thickness for backfill will be dependent on the type of compaction equipment utilized but should not generally exceed 8 inches in loose thickness. Care should be exercised to avoid damaging the cable during compaction of the trench backfill.

8.12 Access Roads

Unpaved access roads are planned within the Project. It is expected that the roads will be nominally graded to provide essential drainage control per Civil Engineer's recommendations. The road wearing surface may be stabilized by gravel or improved with a driving surface course.

It is understood that Crushed Miscellaneous Base (CMB) manufactured from demolished concrete foundations is preferred for the driving surface course. Such CMB should conform to the gradation and quality specifications of the fine CMB per Specifications for Public Works Construction (Greenbook) Sections 203-6 and 200-2, respectively.

The access road subgrade should be scarified to at least 3 inches, be moisture-conditioned to near optimum moisture content, and compacted just prior to placement of the base course to 95 percent of relative compaction. Positive drainage of the subgrade to the road shoulders should be provided. The driving course surface should consist of at least 6 inches of CMB moisture-conditioned within 2 percent of the optimum moisture content and compacted to at least 95 percent of the relative compaction (ASTM D1557). The roadway camber should be installed sloping at least 2 percent towards the road shoulders into suitable perimeter drainage collection system.

8.13 Soil Corrosion Potential

The corrosion potential of the on-site soils to buried steel and concrete was evaluated based on laboratory testing of 3 near-surface soil samples. Table 13 below presents the results of the corrosivity testing.

Table 13
Corrosivity Test Results

Boring	Sample ID	Soil type	Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Chlorides (%)	Soluble Sulfate Content in Soil (%)
B-1	SK-1	Poorly graded gravel (GP)	0 - 5	8.1	4,800	0.00085	0.0055 Category S0 per 2019 CBC
B-5	SK-A	Poorly graded gravel (GP)	0 - 5	8.7	11,600	0.00069	0.0011 Category S0 per 2019 CBC
B-13	SK-1	Poorly Graded Sand with Gravel (SP)	0 - 5	8.0	3,980	0.00037	0.0017 Category S0 per 2019 CBC

Per 2019 CBC based on 2018 IBC, Section 1904.1, concrete subject to exposure to sulfates shall comply with the durability requirements set forth in ACI 318 Section 19.3. Based on the measured water-soluble sulfate results, the exposure of buried concrete to sulfate attack is not a concern, i.e., exposure class S0 per ACI 318-14, Table 19.3.1.1. Consequently, per Table 19.3.2.1, injurious sulfate attack would not be anticipated for concrete with a minimum 28-day compressive strength of 2,500 psi.

Per 2019 CBC, Section 1904.1, concrete, including protection for corrosion and exposure to chlorides, shall conform to durability requirements established in ACI 318. Reinforcement should be protected from corrosion in accordance with ACI 318-14 Chapter 19.

The results of the 4-electrode field electrical resistivity tests yielded resistivity values ranging from 102,000 to 784,000 ohm-cm, with most of the values greater than 200,000 ohm-cm. It is noted that the field resistivity indicated that at the 2 locations where the testing was performed, the materials were very similar in terms of their electrical properties supporting the general observation of a relatively uniform near surface material throughout the site. Details on the testing procedures, and complete test results are presented in Appendix C – Electrical Resistivity Geophysical Survey.

The results of both the field and laboratory resistivity testing indicate that metallic corrosion potential of the on-site soil is relatively low. The corrosion potential of buried metals was evaluated based on the minimum resistivity and our experience with similar soils. The on-site soils are anticipated to be "mildly corrosive" to buried metals as defined by the NACE (1984).

A Corrosion Specialist should be consulted regarding suitable types of piping and necessary protection for underground metal conduits. The corrosion potential of the on-site soils should be verified during construction for each encountered soil type. Imported fill materials should be tested prior to placement to confirm that their corrosion potential is not more severe than the one assumed for the Project.

8.14 Soil Thermal Resistivity

Soil thermal resistivity testing measures the capacity of the ground to conduct or dissipate heat. The thermal properties of a soil layer or compacted fill material is required for the design and installation of underground pipelines and electrical transmission cables (especially high voltage cables). The heat produced by current flowing through an underground power cable must be properly dissipated to avoid premature heat-related failures. Specific recommendations for thermal protection of underground utility lines and pipelines should be provided by a specialist practicing in the subject area of expertise.

An attempt was made during the field exploration program to retrieve undisturbed samples from near surface materials, however due to the abundant presence of cobbles and boulders throughout most of the site it was not possible to retrieve more than a couple of relatively undisturbed samples. Bulk samples retrieved at several locations were also used to produce remolded samples to assess the thermal resistivity properties of the near surface materials, typically within the upper 5 feet of the ground surface. Laboratory thermal resistivity testing was performed on the following soil samples:

- Two undisturbed soil samples, B-12 R-2 and B-95 R-3. It is noted that the natural moisture content for B-12 R-2 was about 3 percent and for B-95 R-3 was very low (close to 0.6percent), which represents typical soil conditions in a desert environment.

- Seven remolded soil samples, B-12 SK-1, B-94 SK-1, B-95 SK-1, B-97 SK-1, B-5 SK-A, B-3 SK-1, B-5 SK-1.

Thermal resistivity testing was performed in general accordance with ASTM D5334 using the 3-point method. The remolded soil samples were prepared by moisture-conditioning to about 2 percent of optimum moisture content per ASTM D1557 compacting to approximately 90 percent of maximum dry density per ASTM D1557.

For a moisture content of 0 percent, the results of thermal resistivity both on the undisturbed samples and the remolded samples yield thermal resistivity values ranging between 140 and 338 ºC-cm/W with most values on the order of 200 ºC-cm/W. For the remolded samples at a moisture content of about 4 percent and higher, the curves seem to indicate a constant thermal resistivity with values ranging between 50 and 85 ºC-cm/W. The laboratory thermal resistivity data indicate that the near-surface materials are very similar throughout the project site, supporting the general observation of overall uniform near-surface materials. The laboratory thermal resistivity data is included in Appendix D – Laboratory Testing.

8.15 Drainage Control

The intent of this section is to provide general information regarding the control of surface water. The control of surface water is essential to the satisfactory performance of the planned construction and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath and adjacent to the improvements, even during periods of heavy rainfall. The following recommendations should be considered as minimal and implemented as applicable.

- Ponding and areas of low flow gradients should be avoided.

- Paved surfaces within 10 feet from the turbine foundation above-ground pedestal should be provided with a gradient of at least 2 percent sloping away from improvement.

- Unpaved areas should be designed with a drainage gradient of at least 5 percent away from structures.

- Positive drainage of the subgrade and pavement surface areas should be provided to reduce water infiltration into the pavement subgrade.

- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and convey water to appropriate discharge points.

- No site improvements or grading should obstruct the free flow of surface water.

9. GENERAL SITE GRADING RECOMMENDATIONS

The intent of this section is to provide general information regarding the grading of the site. Site grading operations should conform with applicable local building and safety codes and to the rules and regulations of those governmental agencies having jurisdiction over the subject construction.

The grading contractor is responsible for notifying governmental agencies, as required, and the Geotechnical Engineer at the start of site cleanup, at the initiation of grading, and any time that grading operations are resumed after an interruption. Each step of the grading should be accepted in a specific area by the Geotechnical Engineer, and where required, should be approved by the applicable governmental agencies prior to proceeding with subsequent work.

The following site grading recommendations should be regarded as minimal. The site grading recommendations should be incorporated into the Project plans and specifications.

- Prior to grading, existing vegetation, trash, surface structures and debris should be removed and disposed off-site at a legal dumpsite. Any existing utility lines, or other subsurface structures which are not to be utilized, should be removed, destroyed, or abandoned in compliance with current governmental regulations.

- Subsequent to cleanup operations, and prior to initial grading, a reasonable search should be made for subsurface obstructions and/or possible loose fill or detrimental soil types. This search should be conducted by the contractor, with advice from and under the observation of the Geotechnical Engineer.

- Prior to the placement of fill or foundations, the site should be prepared in accordance with the recommendations presented in the Site Preparation section of this report or the turbine manufacturer's specifications, whichever is more stringent. Undocumented fill or disturbed soils within foundation footprint should be removed and processed as recommended by the Geotechnical Engineer.

- The exposed subgrade and/or excavation bottom should be observed and approved by the Geotechnical Engineer for conformance with the intent of the recommendations presented in this report and prior to any further processing or fill placement. Proof rolling may be considered to provide a general guidance about the quality of the fill. It should be understood that the actual encountered conditions may warrant excavation and/or subgrade preparation beyond the extent recommended and/or anticipated in this report.

- On-site inorganic granular soils that are free of debris or contamination are considered suitable for placement as compacted fill. Any rock or other soil fragments greater than 3 inches in size should not be placed within 5 feet of a foundation subgrade.

- Any imported fill material required for backfill or grading should be tested and approved prior to delivery to the site.

- Visual observations and field tests should be performed during grading by the Geotechnical Engineer. This is necessary to assist the contractor in obtaining the proper moisture content and required degree of compaction. In general field compaction testing should be performed as recommended in Table 14.

Table 14
Field and Laboratory Testing

Test or Observation	**Test Method**	**Minimum Testing Frequency** (per Irwindale Backfilling Committee, 2005)
Field Density	Maximum dry density and optimum moisture content by ASTM D1557	As deemed necessary or the most frequent of: 1 per major material type 1 per 4 weeks of construction
Field Compaction	Field Density by Sand cone (ASTM D1556) or Nuclear gauge (ASTM D2922) Minimum 10 percent of tests performed using the Sand cone method.	As deemed necessary or the most frequent of: 1 per 1,000 CYs, or 1 per each 2 compaction lifts, or 2 per full day of placement 1 per 250 LF of roadway

- Wherever, in the opinion of the Geotechnical Engineer, an unsatisfactory condition is being created in any area, whether by cutting or filling, the work should not proceed in that area until the condition has been corrected.

10. DESIGN REVIEW CONSTRUCTION SERVICES

Geotechnical review of plans and specifications and participation during the construction are an integral part of the design practice. The following paragraphs present our recommendations relative to the review of construction documents and the monitoring of construction activities.

The herein presented recommendations are in addition to any requirements specified by the turbine manufacturer

10.1 Plans and Specifications

Upon completion, the civil and structural design plans and specifications should be reviewed and approved by Tetra Tech prior to submittal for issuance of grading, building, and/or construction permits as the geotechnical recommendations may need to be re-evaluated based on the actual design configuration and loads. This review is intended to evaluate whether the recommendations contained in this report have been incorporated into the Project plans and specifications as intended.

10.2 Construction Monitoring

The objective of the construction quality assurance (CQA) is to assist in the construction of the soils and soils-structure interaction components of the Project. Observation of site overexcavation, processing and assessment of fill materials, fill placement, and other site grading operations by the Geotechnical Engineer should be implemented during construction to allow for evaluation of the geotechnical-related conditions as they are encountered. This process provides the Geotechnical Engineer with the opportunity to recommend appropriate revisions as needed.

10.2.1 Grading Observations

Continuous observations by the Geotechnical Engineer should be provided to assess all encountered soil types, verify the extent of removals and overexcavation, suitability of import materials, lift thicknesses and densities during placement and compaction of fill materials. The Geotechnical Engineer should observe all temporary excavations and construction slopes, as well as the backfill operations so that appropriate modifications to the design criteria presented herein may be recommended, if necessary, due to encountered conditions differing from the design assumptions.

10.2.2 Foundation Construction Observations

The Geotechnical Engineer should observe and evaluate the presence of satisfactory materials at the foundation subgrade. Foundation excavations should be clean of loosened soil and debris before placing steel or concrete. If soft or loose soils or other unsatisfactory materials are encountered, such materials should be removed and replaced with compacted fill prior to pouring of concrete.

10.2.3 Pavement Construction Observations

Preparation of the pavement subgrade and the placement of base course and pavement sections should be observed by the Geotechnical Engineer. Careful observation is recommended to evaluate that the pavement subgrade is uniformly compacted, and the recommended pavement and base course thicknesses are achieved. Paved areas should be properly sloped, and surface drainage established to reduce water infiltration into the pavement subgrade. Curbing located adjacent to paved areas should be founded in the soil subgrade in order to provide a cutoff to reduce water infiltration into the base course.

10.2.4 Construction Quality Assurance Reporting

The following list is intended to provide basic minimum guidelines for the reporting during the excavation and backfilling operations:

- A Daily Field Report should be generated each time a representative of the Geotechnical Engineer is performing QA work at the site.

- The Daily Field Reports should contain, at a minimum, a detailed description of the field activities, utilized equipment, areas of work, date, time, weather, and locations and results of all observations and performed tests.

- Provisions should be made for vertical and horizontal control for recording observations and test locations.

- A complete set of Daily Field Reports should be submitted as a part of formal final reporting.

11. LIMITATIONS

The recommendations and opinions expressed in this report are based on Tetra Tech's analyses based on review of background documents, and on information obtained from field explorations and associated laboratory testing. It should be noted that this study did not evaluate the presence of hazardous materials on any portion of the site. Furthermore, it is anticipated that a good portion of the site will be subject to the risks of flooding, scour, and even flows. Such hazards are recognized but their impact and countermeasures were not addressed in this report.

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 grading operations, for example the extent of unsuitable soils and overexcavation, or presence of large boulders, which may result in an additional mitigation effort.

Site conditions can change with time as a result of natural processes or the activities of man. 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 Tetra Tech has no control. Therefore, this report should be reviewed and recertified by Tetra Tech if it were to be used for a project design commencing more than one year after the date of issuance of this report.

Tetra Tech's recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction and on verification of the foundation conditions. Accordingly, the recommendations are made contingent upon the opportunity for Tetra Tech to observe all aspects of grading operations and foundation excavations for the proposed construction. If parties other than Tetra Tech are engaged to provide such services, such parties are assuming complete responsibility as the Geotechnical Engineer of Record for the Project and implicitly concur with the recommendations provided in this report or may provide 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. Tetra Tech should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. Reliance by others on the data presented herein or for purposes other than those stated in the text is authorized only if so permitted in writing by Tetra Tech. Such an authorization may incur additional expenses and charges.

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

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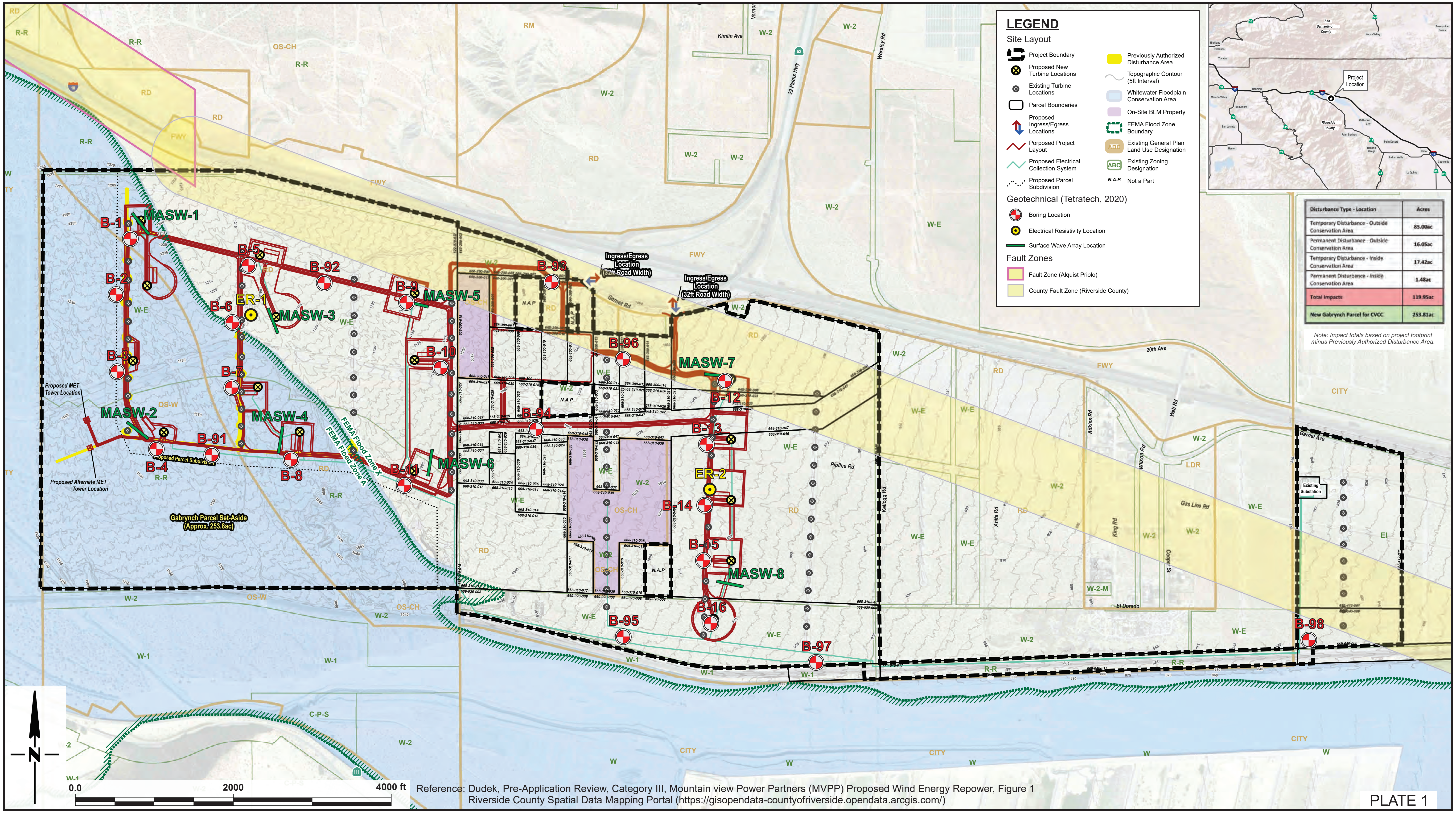
Reviewed Aerial Photographs

Flight Date	Film / Flight ID	Frames	Scale	Source
6/8/1936	C_4058	48,49,50	1:170,00	Fairchild Aerial Surveys
1/1/1953	AXM-1953B	12K-145, 13K-171, 10K-12, 1K-38	1:20,000	USDA
8/14/1980	USDA_FIRESCOPE	280-16	1:40,000	USDA
6/4/1996	NAPP-2C	9553-162	1:40,000	USGS
9/27/1996	NAPP-2C	9572-92	1:40,000	USGS

Plates



LEGEND

Site Layout

- Project Boundary
- ⊗ Proposed New Turbine Locations
- ⊗ Existing Turbine Locations
- ▢ Parcel Boundaries
- Proposed Ingress/Egress Locations
- Porposed Project Layout
- Proposed Electrical Collection System
- Proposed Parcel Subdivision
- 🟨 Previously Authorized Disturbance Area
- Topographic Contour (5ft Interval)
- Whitewater Floodplain Conservation Area
- On-Site BLM Property
- FEMA Flood Zone Boundary
- ABC Existing General Plan Land Use Designation
- ABC Existing Zoning Designation
- N.A.P. Not a Part

Geotechnical (Tetratech, 2020)

- Boring Location
- ● Electrical Resistivity Location
- Surface Wave Array Location

Fault Zones

- Fault Zone (Alquist Priolo)
- County Fault Zone (Riverside County)

Disturbance Type - Location	Acres
Temporary Disturbance - Outside Conservation Area	85.00ac
Permanent Disturbance - Outside Conservation Area	16.05ac
Temporary Disturbance - Inside Conservation Area	17.42ac
Permanent Disturbance - Inside Conservation Area	1.48ac
Total Impacts	119.95ac
New Gabrynch Parcel for CVCC	253.81ac

Note: Impact totals based on project footprint minus Previously Authorized Disturbance Area.

Reference: Dudek, Pre-Application Review, Category III, Mountain view Power Partners (MVPP) Proposed Wind Energy Repower, Figure 1
Riverside County Spatial Data Mapping Portal (https://gisopendata-countyofriverside.opendata.arcgis.com/)

PLATE 1

TETRA TECH

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MOUNTAIN VIEW I & II WIND ENERGY PROJECT - RIVERSIDE COUNTY, CA

PROPOSED TURBINE LAYOUT AND PARCEL MAP

JOB NO. GEN 20-33E
DATE OCT 2020
DRAWN BY SCM
CHECKED BY FC

USDA Flight ID- AXM 1953 B, Frames- AXK-1K-38, AXK-10K-12, AXK-12K-145, AXK-13K-171

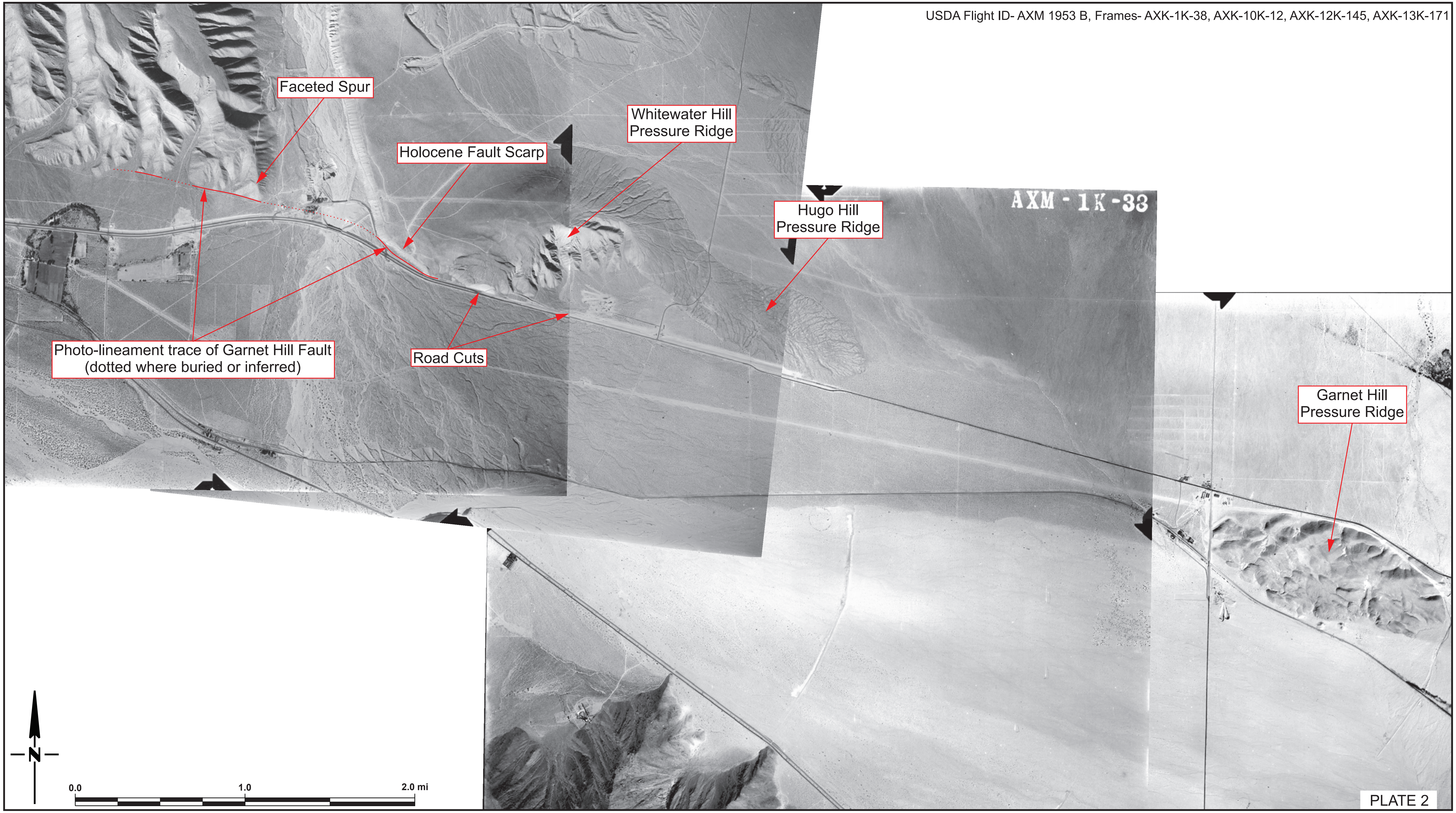

PLATE 2

TETRA TECH
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MOUNTAIN VIEW I & II WIND ENERGY PROJECT - RIVERSIDE COUNTY, CA

ANNOTATED 1953 AERIAL PHOTOGRAPH MOSAIC

JOB NO. GEN 20-33E
DATE OCT 2020
DRAWN BY SCM
CHECKED BY FC

Figures

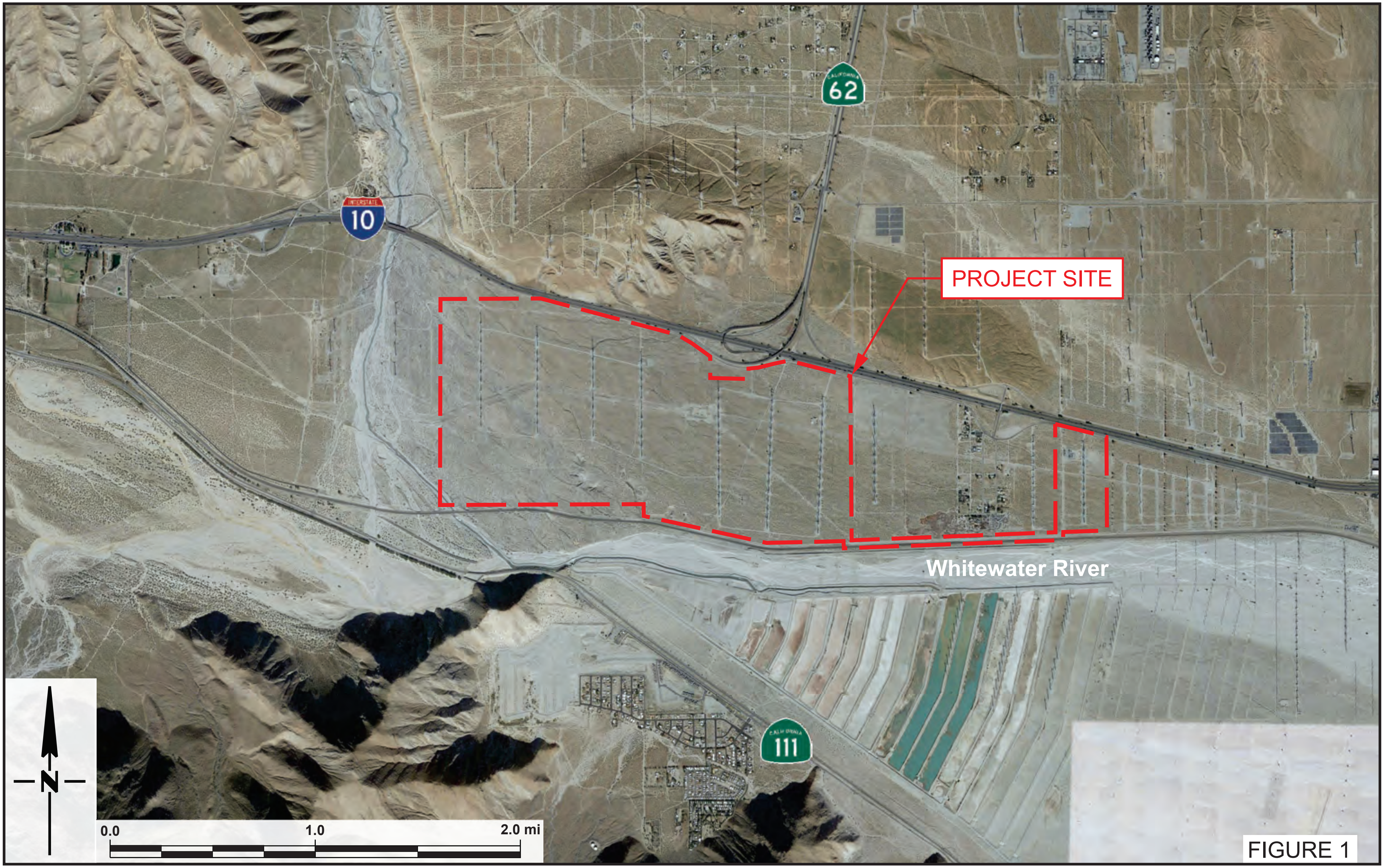

FIGURE 1

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MOUNTAIN VIEW I & II WIND ENERGY PROJECT - RIVERSIDE COUNTY, CA

PROJECT LOCATION MAP

JOB NO. GEN 20-33E

DATE OCT 2020

DRAWN BY SCM

CHECKED BY FC

FIGURE 2

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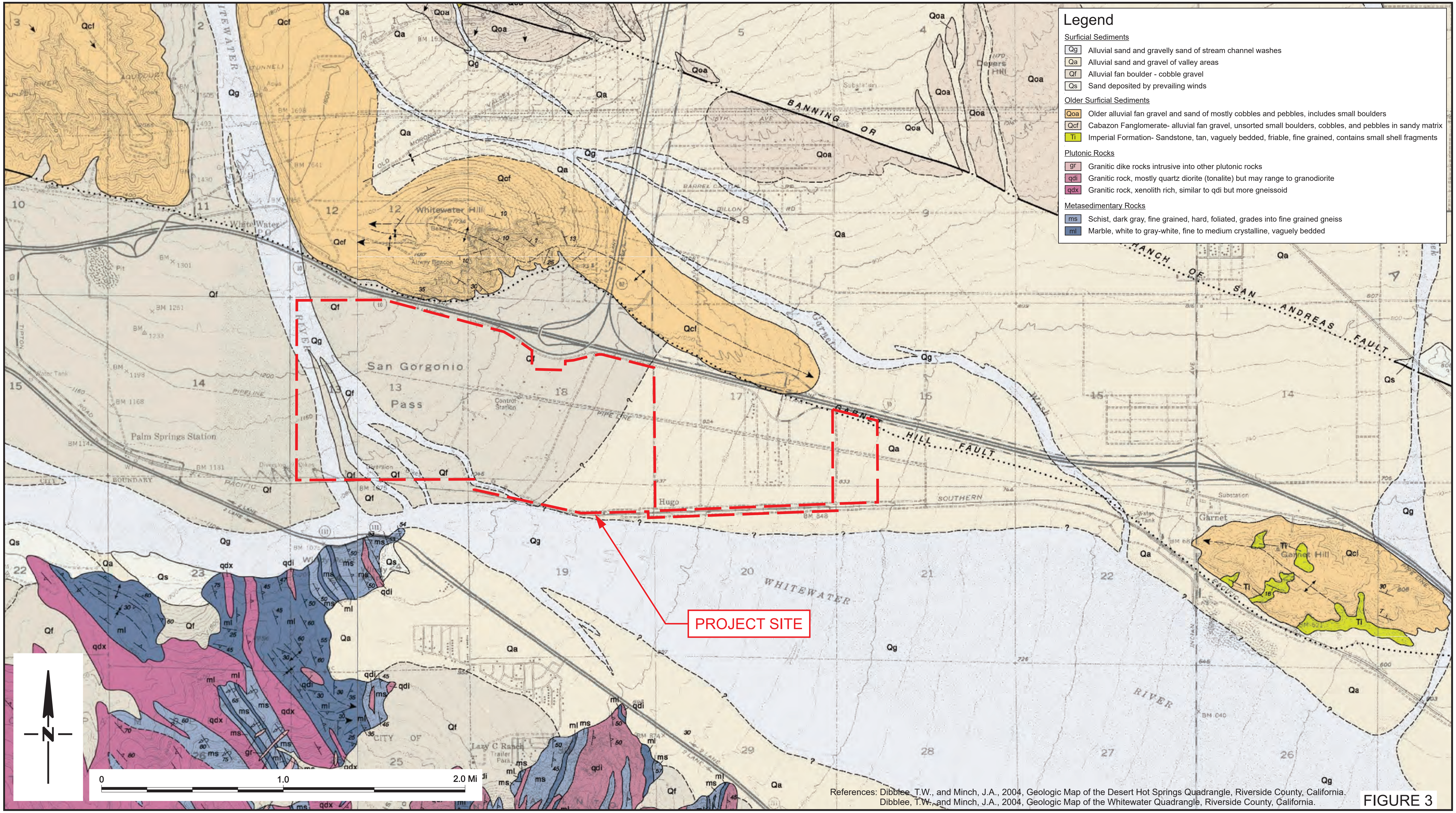
MOUNTAIN VIEW I & II WIND ENERGY PROJECT - RIVERSIDE COUNTY, CA

SITE LAYOUT AND SUBSURFACE EXPLORATION LOCATIONS MAP

JOB NO.	GEN 20-33E
DATE	OCT 2020
DRAWN BY	SCM
CHECKED BY	FC



Legend

Surficial Sediments

Qg	Alluvial sand and gravelly sand of stream channel washes
Qa	Alluvial sand and gravel of valley areas
Qf	Alluvial fan boulder - cobble gravel
Qs	Sand deposited by prevailing winds

Older Surficial Sediments

Qoa	Older alluvial fan gravel and sand of mostly cobbles and pebbles, includes small boulders
Qcf	Cabazon Fanglomerate- alluvial fan gravel, unsorted small boulders, cobbles, and pebbles in sandy matrix
Ti	Imperial Formation- Sandstone, tan, vaguely bedded, friable, fine grained, contains small shell fragments

Plutonic Rocks

gr	Granitic dike rocks intrusive into other plutonic rocks
qdi	Granitic rock, mostly quartz diorite (tonalite) but may range to granodiorite
qdx	Granitic rock, xenolith rich, similar to qdi but more gneissoid

Metasedimentary Rocks

ms	Schist, dark gray, fine grained, hard, foliated, grades into fine grained gneiss
ml	Marble, white to gray-white, fine to medium crystalline, vaguely bedded

References: Dibblee, T.W., and Minch, J.A., 2004, Geologic Map of the Desert Hot Springs Quadrangle, Riverside County, California.
Dibblee, T.W., and Minch, J.A., 2004, Geologic Map of the Whitewater Quadrangle, Riverside County, California.

FIGURE 3

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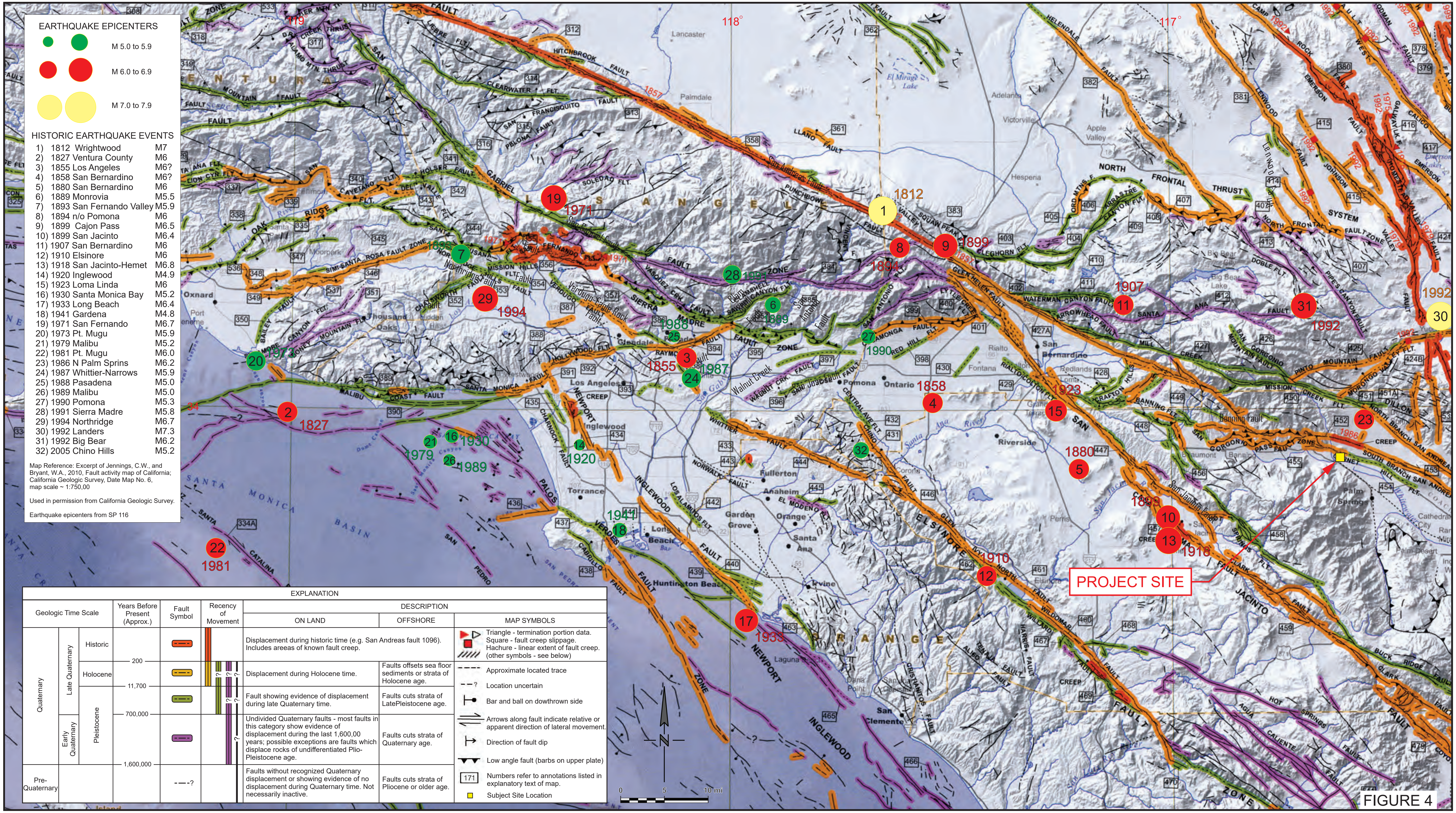
MOUNTAIN VIEW I & II WIND ENERGY PROJECT - RIVERSIDE COUNTY, CA

GEOLOGIC MAP

JOB NO.	GEN 20-33E
DATE	OCT 2020
DRAWN BY	SCM
CHECKED BY	FC



EARTHQUAKE EPICENTERS

● ●	M 5.0 to 5.9
● ●	M 6.0 to 6.9
● ●	M 7.0 to 7.9

HISTORIC EARTHQUAKE EVENTS

1)	1812 Wrightwood	M7
2)	1827 Ventura County	M6
3)	1855 Los Angeles	M6?
4)	1858 San Bernardino	M6?
5)	1880 San Bernardino	M6
6)	1889 Monrovia	M5.5
7)	1893 San Fernando Valley	M5.9
8)	1894 n/o Pomona	M6
9)	1899 Cajon Pass	M6.5
10)	1899 San Jacinto	M6.4
11)	1907 San Bernardino	M6
12)	1910 Elsinore	M6
13)	1918 San Jacinto-Hemet	M6.8
14)	1920 Inglewood	M4.9
15)	1923 Loma Linda	M6
16)	1930 Santa Monica Bay	M5.2
17)	1933 Long Beach	M6.4
18)	1941 Gardena	M4.8
19)	1971 San Fernando	M6.7
20)	1973 Pt. Mugu	M5.9
21)	1979 Malibu	M5.2
22)	1981 Pt. Mugu	M6.0
23)	1986 N Palm Sprins	M6.2
24)	1987 Whittier-Narrows	M5.9
25)	1988 Pasadena	M5.0
26)	1989 Malibu	M5.0
27)	1990 Pomona	M5.3
28)	1991 Sierra Madre	M5.8
29)	1994 Northridge	M6.7
30)	1992 Landers	M7.3
31)	1992 Big Bear	M6.2
32)	2005 Chino Hills	M5.2

Map Reference: Excerpt of Jennings, C.W., and Bryant, W.A., 2010, Fault activity map of California; California Geologic Survey, Date Map No. 6, map scale ~ 1:750,00

Used in permission from California Geologic Survey.

Earthquake epicenters from SP 116

EXPLANATION

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE	MAP SYMBOLS
Quaternary	Late Quaternary	Historic	200	🟥		Displacement during historic time (e.g. San Andreas fault 1096). Includes areeas of known fault creep.		▶ ▷ ■ ////// Triangle - termination portion data. Square - fault creep slippage. Hachure - linear extent of fault creep. (other symbols - see below)
		Holocene	11,700	🟨		Displacement during Holocene time.	Faults offsets sea floor sediments or strata of Holocene age.	- - - - Approximate located trace
		Pleistocene	700,000	🟩		Fault showing evidence of displacement during late Quaternary time.	Faults cuts strata of LatePleistocene age.	- - ? Location uncertain ├● Bar and ball on dowthrown side
	Early Quaternary		1,600,000	🟪		Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,00 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Faults cuts strata of Quaternary age.	⇌ Arrows along fault indicate relative or apparent direction of lateral movement. ↦ Direction of fault dip ▼▼ Low angle fault (barbs on upper plate)
Pre-Quaternary				- — -?		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Faults cuts strata of Pliocene or older age.	171 Numbers refer to annotations listed in explanatory text of map. 🟨 Subject Site Location

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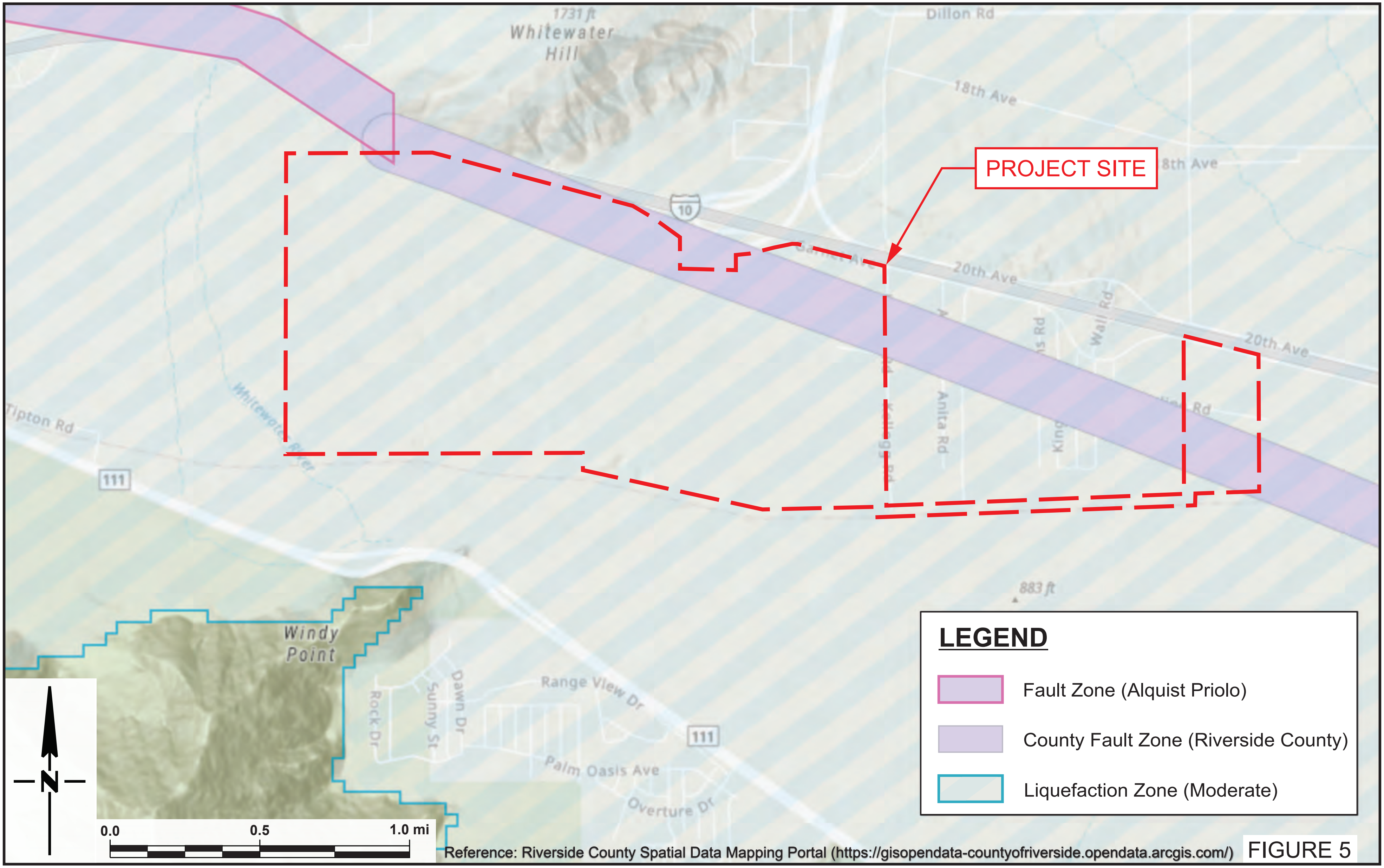
MOUNTAIN VIEW I & II WIND ENERGY PROJECT - RIVERSIDE COUNTY, CA

REGIONAL FAULTS AND SEISMICITY MAP

JOB NO. GEN 20-33E
DATE OCT 2020
DRAWN BY SCM
CHECKED BY FC



Reference: Riverside County Spatial Data Mapping Portal (https://gisopendata-countyofriverside.opendata.arcgis.com/)

FIGURE 5

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MOUNTAIN VIEW I & II WIND ENERGY PROJECT - RIVERSIDE COUNTY, CA

SEISMIC HAZARD ZONES MAP

JOB NO.	GEN 20-33E
DATE	OCT 2020
DRAWN BY	SCM
CHECKED BY	FC

Appendix A

Boring Logs

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BORING B-1

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1239 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 9/14/2020
DRILLING CONTRACTOR Great West Drilling
BOREHOLE DEPTH 60.3 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Becker Hammer
BOREHOLE DIAMETER 8 in
COORDINATES 33.915977; -116.628427
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 150 ft south of proposed turbine 1 location

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧗ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮	SK-1			GP	[NATIVE] Alluvium Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2.5', clast supported			CORR	
	⧗	R-2	50/5"							
										1235
5	⧗	R-3	50/4"							
										1230
10	◩	SPT-4	50/6"							
	⧗	R-5	50/3"							
										1225
15	◩	SPT-6	50/5"							
										1220
20	⧗	R-7	50/5"							
										1215
25	◩	SPT-8	50/5"							
										1210
30	⧗	R-9	47-50/3"							
										1205
35										

Continued on next page

BORING B-1

Sheet 2 of 2

PROJECT NAME Mountain View **CLIENT** AES **GROUND ELEVATION** 1239 ft
PROJECT NUMBER GEN 20-33 **PROJECT LOCATION** Whitewater, CA **GROUNDWATER DEPTH** Not encountered

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	SPT-10	50/4"		Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2.5', clast supported *(continued)*				
								1200
40	R-11	50/4"						
								1195
45								
								1190
50	SPT-12	23-50/2"						
								1185
55	R-13	50/5"						
								1180
60	SPT-14	50/3"						

Notes:
- Total depth of boring 60.3'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-2

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1214 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/13/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 35.5 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.914068; -116.629033
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 250 ft west of proposed turbine 2 location

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

MATERIAL DESCRIPTION

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0			SP	[NATIVE] Alluvium Poorly graded SAND with Gravel, very dense, dry, light olive gray (5Y 6/2), medium to coarse sand, medium to coarse gravel, trace of cobbles and boulders				
								1210
5	◩ SPT-1	16-50/0"					G/S/F = 33/64/3	
								1205
10	⧓ R-2	50/6"						
	◩ SPT-3	24-28-38 (66)						
								1200
15	⧓ R-4	11-50/3"			109.4	0.6	DS, CONSOL	
								1195
20	◩ SPT-5	8-50/5"	GP-GM	Poorly Graded GRAVEL with Silt and Sand, very dense, dry, olive gray (5YR 4/2), medium to coarse sand, medium to coarse gravel, trace of cobbles and boulders			G/S/F = 50/44/6	
	⧓ R-6	50/5"	SP	Poorly graded SAND with Gravel, very dense, dry, light olive gray (5Y 6/2), medium to coarse sand, medium to coarse gravel, trace of cobbles and boulders				
								1190
25	◩ SPT-7	16-50/4"						
								1185
30	⧓ R-8	50/5"						
								1180
35								

Continued on next page

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BORING B-2

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1214 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧗ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
	◩	SPT-7	50/6"							

Notes:
- Total depth of boring 35.5'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-3

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1190 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 9/15/2020
DRILLING CONTRACTOR Great West Drilling
BOREHOLE DEPTH 40 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Becker Hammer
BOREHOLE DIAMETER 8 in
COORDINATES 33.911413; -116.629021
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 85 ft west of proposed turbine 3 location

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧗ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0			GP	[NATIVE] Alluvium Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported				1190
	▮ SK-1							
	⧗ R-2	50/5"			135.0	3.3	G/S/F = 58/42/0, RESISTIVITY	
5	⧗ R-3	50/5"		...(5 ft) boulders up to 4' diameter				1185
10	◩ SPT-4	45-50/3"						1180
	▮ SK-5							
15	⧗ R-6	50/5"						1175
20	◩ SPT-7	32-50/5"						1170
25	⧗ R-8	50/4"						1165
30	◩ SPT-9	21-40-50/3"						1160
	▮ SK-10		SP	Poorly Graded SAND with Gravel, very dense, dry, olive gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2'			G/S/F = 38/61/1	
35								1155

Continued on next page

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BORING B-3

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1190 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	⧓	R-11	45-50/4"			Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported	113.5	2.8	DS	1155
40										1150

Notes:
- Total depth of boring 40'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-4

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1151 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 9/15/2020
DRILLING CONTRACTOR Great West Drilling
BOREHOLE DEPTH 30.6 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Becker Hammer
BOREHOLE DIAMETER 8 in
COORDINATES 33.908737; -116.627425
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 105 ft south of proposed turbine 4 location

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	SK-1		GP	[NATIVE] Alluvium Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported			R-VALUE	1150
5	R-2	50/4"						1145
10	SPT-3	45-40-50 (90)						1140
15	R-4	50/4"						1135
	SK-5							
20	SPT-6	31-50/6"						1130
25	R-7	40-50/3"			107.5	4.7		1125
30	SPT-8	17-50/1"						

Notes:
- Total depth of boring 30.6'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-5

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1205 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 9/18/2020
DRILLING CONTRACTOR Great West Drilling
BOREHOLE DEPTH 40.3 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Becker Hammer
BOREHOLE DIAMETER 8 in
COORDINATES 33.915028; -116.62354
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 65 ft south of proposed turbine 5 location

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧗ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	SK-1		GP	[NATIVE] Alluvium Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported	125.5	5.7	G/S/F = 54/44/2, CORR, RESISTIVITY	1205
5	R-2	50/3"						1200
10	R-3	50/2"						1195
15	SPT-4	50/4"						1190
20	R-5	50/5"						1185
	SK-6							
25	SPT-7	50/4"						1180
30	R-8	17-50/3"			114.1	1.5	CONSOL	1175
35								1170

Continued on next page

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BORING B-5

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1205 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	◩ SPT-9	50/3"			Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported *(continued)*				1170
	▮ SK-10								
40	⧓ R-11	50/4"							1165

Notes:
- Total depth of boring 40.3'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-6

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1184 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

DATES DRILLED 9/18/2020
DRILLING CONTRACTOR Great West Drilling
BOREHOLE DEPTH 40.8 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Becker Hammer
BOREHOLE DIAMETER 8 in
COORDINATES 33.913072; -116.624218
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 425 ft west of proposed turbine 6 location

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	SK-1			GP	[NATIVE] Alluvium Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported				
									1180
5	R-2	50/5"							
									1175
10	R-3	50/5"							
									1170
15	SPT-4	50/4"							
	SK-5								
									1165
20	R-6	50/3"							
									1160
25	SPT-7	29-40-46 (86)							
									1155
30	R-8	50/5"							
									1150
35									

Continued on next page

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BORING B-6

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1184 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧗ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	◩	SPT-9	14-35-38 (73)			Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported *(continued)*				
										1145
40	⧗	R-10	33-50/4"				108.9	1.8	DS	

Notes:
- Total depth of boring 40.8'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-7

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1163 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 9/16/2020
DRILLING CONTRACTOR Great West Drilling
BOREHOLE DEPTH 40.9 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Becker Hammer
BOREHOLE DIAMETER 8 in
COORDINATES 33.91083; -116.624264
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 230 ft west of proposed turbine 7 location

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧗ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮	SK-1			GP	[NATIVE] Alluvium Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported				
										1160
5										
	⧗	R-2	50/4"							1155
10	⧗	R-3	50/3"							
	⧗	R-4	50/3"							
										1150
15	◩	SPT-5	50/3"							
										1145
20	⧗	R-6	50/3"							
										1140
25										
										1135
30	◩	SPT-7	33-50/3"							
	▮	SK-8								1130
35										

Continued on next page

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BORING B-7

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1163 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▯ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	⧓	R-9	50/5"			Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported *(continued)*				
										1125
40	◩	SPT-10	9-50/5"							

Notes:
- Total depth of boring 40.9'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-8

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1125 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

DATES DRILLED 9/16/2020
DRILLING CONTRACTOR Great West Drilling
BOREHOLE DEPTH 40.8 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Becker Hammer
BOREHOLE DIAMETER 8 in
COORDINATES 33.908334; -116.621828
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 260 ft south of proposed turbine 8 location

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮	SK-1			SP-SM	[NATIVE] Alluvium Poorly Graded SAND with Silt and Gravel, very dense, dry, olive gray (5Y 4/2), medium to coarse sand, fine to coarse gravel with cobbles and boulders up to ~2', clast supported				1125
5					GP	Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, cobbles and boulders, clast supported				1120
	⧓	R-2	50/5"						G/S/F = 39/55/6	
	⧓	R-3	50/3"							
10										1115
15										1110
	◩	SPT-4	50/3"							
	▮	SK-5								
20	⧓	R-6	50/5"							1105
25										1100
	◩	SPT-7	50/1"							
	▮	SK-8								
30	⧓	R-9	50/5"							1095
35										1090

Continued on next page

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BORING B-8

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1125 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▯ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35										1090
	◩	SPT-10	16-35-37 (72)			Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, cobbles and boulders, clast supported *(continued)*				
40										1085
	⧓	R-11	30-50/4"				109.7	1.1	CONSOL	

Notes:
- Total depth of boring 40.8'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-9

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1137 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/13/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 27 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.913714; -116.617004
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 67 ft south of proposed turbine 9 location

◪	Standard Penetration Test (SPT)	○	No Recovery	■	Shelby Tube
⧓	California-Type Ring Sample	☒	No Recovery	▮	Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0			░	SP	[NATIVE] Alluvium Poorly graded SAND with Gravel, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, cobbles				
									1135
5	◪ SPT-1	7-50/4"							
									1130
10	⧓ R-2	50/3"							
									1125
	◪ SPT-3	50/3"							
15	⧓ R-4	50/6"							
									1120
20	◪ SPT-5	50-50/4"							
									1115
	⧓ R-6	50/5"							
25	◪ SPT-7	50/6"						G/S/F = 47/49/4	
									1110

Notes:
- Total depth of boring 27'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-10

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1106 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

DATES DRILLED 9/17/2020
DRILLING CONTRACTOR Great West Drilling
BOREHOLE DEPTH 30.6 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Becker Hammer
BOREHOLE DIAMETER 8 in
COORDINATES 33.911451; -116.615586
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 440 ft east of proposed turbine 10 location

◩	Standard Penetration Test (SPT)	○	No Recovery	■	Shelby Tube
⧗	California-Type Ring Sample	☒	No Recovery	▮	Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0					GP	[NATIVE] Alluvium Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, cobbles and boulders, clast supported				
	▮	SK-1								1105
5	⧗	R-2	50/3"							
										1100
	▮	SK-3								
10	⧗	R-4	31-50/2"							
										1095
15	◩	SPT-5	50/5"							
										1090
20	⧗	R-6	50/5"							
										1085
25	◩	SPT-7	28-50/4"							
										1080
	▮	SK-8								
30	◩	SPT-9	37-50/1"							

Notes:
- Total depth of boring 30.6'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-11

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1088 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

DATES DRILLED 9/17/2020
DRILLING CONTRACTOR Great West Drilling
BOREHOLE DEPTH 41.3 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Becker Hammer
BOREHOLE DIAMETER 8 in
COORDINATES 33.907411; -116.617151
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 115 ft south of proposed turbine 11 location

◩	Standard Penetration Test (SPT)	○	No Recovery	■	Shelby Tube
⧗	California-Type Ring Sample	☒	No Recovery	▮	Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮ SK-1			SP	[NATIVE] Alluvium Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, cobbles and boulders, clast supported				
									1085
5									
	⧗ R-2	50/3"							
									1080
10	⧗ R-3	29-50/4"							
									1075
15	◩ SPT-4	36-29-45 (74)							
									1070
20	⧗ R-5	33-50/6"							
									1065
25	◩ SPT-6	14-24-29 (53)							
									1060
30	⧗ R-7	50/2"							
									1055
35									

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BORING B-11

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1088 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	◩	SPT-8	15-31-41 (72)			Poorly graded GRAVEL with Sand, very dense, dry, light brownish gray (2.5Y 6/2), medium to coarse sand, cobbles and boulders, clast supported *(continued)*				
										1050
40	⧓	R-9	14-35-50/4"				101.0	1.9		

Notes:
- Total depth of boring 41.3'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-12

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1003 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

DATES DRILLED 8/12/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 60.4 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.910926; -116.603786
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately at proposed turbine 12 location

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0					SP	[NATIVE] Alluvium Poorly Graded SAND with GRAVEL, very dense, dry, light olive gray (5Y 6/2), fine to coarse sand, gravels up to ~4"				
	▮	SK-1					128.5	2.9	RESISTIVITY	
	⧓	R-2	11-16-50/6" (66)				115.3	1.1	RESISTIVITY	1000
5	◩	SPT-3	7-50/5"							
	⧓	R-4	13-50/5"				112.3	1.2	DS	995
10	◩	SPT-5	11-17-19 (36)			...(10 ft) dense				
	⧓	R-6	50/5"			...(12.5 ft) very dense				990
15	◩	SPT-7	11-19-22 (41)			...(15 ft) dense				
										985
20	⧓	R-8	11-50/3"			...(20 ft) very dense				
										980
25	◩	SPT-9	14-40-50/6"							
										975
30	⧓	R-10	16-37-50/5"				107.5	1.5	DS, CONSOL	
										970
35										

Continued on next page

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BORING B-12

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1003 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧗ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

MATERIAL DESCRIPTION

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	◩ SPT-11	14-24-32 (56)			Poorly Graded SAND with GRAVEL, very dense, dry, light olive gray (5Y 6/2), fine to coarse sand, gravels up to ~4" *(continued)*				
									965
40	⧗ R-12	50/6"							
									960
45	◩ SPT-13	10-19-23 (42)		SW-SM	Well GRADED SAND with Silt, dense, dry, olivee gray (5Y 4/2), fine to coarse sand			G/S/F = 11/83/6	
									955
50	⧗ R-14	50/4"		SP	Poorly Graded SAND with GRAVEL, very dense, light olive gray (5Y 6/2), dry, fine to coarse sand, gravels up to ~4"				
									950
55	◩ SPT-15	33-50/5"							
									945
60	⧗ R-16	50/5"							

Notes:
- Total depth of boring 60.4'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-13

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1002 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

DATES DRILLED 8/26/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 40.5 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.908735; -116.60459
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 240 ft west of proposed turbine 13 location

◪ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧗ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

MATERIAL DESCRIPTION

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮	SK-1			SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, very dense, dry, light olive gray (5Y 6/2), fine to coarse sand, trace of cobbles			CORR	1000
5	◪	SPT-2	20-40-48 (88)			...(5 ft) damp				995
10	⧗	R-3	27-47-50/5"				115.5	1.3		990
	◪	SPT-4	20-25-22 (47)		SP-SM	Poorly Graded SAND with Silt and Gravel, dense, dry, olive gray (5Y 4/2), fine to coarse sand, coarse gravel			G/S/F = 38/57/5	
15	⧗	R-5	25-50/5"		SP	Poorly Graded SAND with Gravel, very dense, dry, light olive gray (5Y 6/2), fine to coarse sand, trace of cobbles	98.2	3.0	CONSOL	985
20	◪	SPT-6	24-50/5"							980
	⧗	R-7	42-50/5"				119.5	1.6		
25	◪	SPT-8	30-50/5"							975
30	⧗	R-9	50/5"				114.2	2.2		970
35										

Continued on next page

BORING B-13

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1002 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	◩ SPT-10	14-27-41 (68)			Poorly Graded SAND with Silt and Gravel, dense, dry, olive gray (5Y 4/2), fine to coarse sand, coarse gravel *(continued)*				965
40	⧓ R-11	50/6"				113.3	2.6		

Notes:
- Total depth of boring 40.5'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-14

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 994 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

DATES DRILLED 8/25/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 40.4 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.906653; -116.60469
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 270 ft west of proposed turbine 14 location

◪ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮			SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, very dense, light olive gray (5Y 6/2), dry, fine to coarse, trace of cobbles				
	▮ SK-1								
	☒ R-2	50/3"							
									990
5	◪ SPT-3	7-12-14 (26)			...(5 ft) medium dense				
	⧓ R-4	40-50/5"			...(7.5 ft) very dense	123.8	1.9		
									985
10	◪ SPT-5	12-14-17 (31)			...(10 ft) dense				
	⧓ R-6	21-26-37 (63)			...(12.5 ft) very dense	113.0	3.1		
									980
15	◪ SPT-7	22-38-48 (86)							
									975
20	⧓ R-8	16-27-46 (73)				103.2	3.1	CONSOL	
									970
25	◪ SPT-9	16-16-20 (36)			...(25 ft) dense				
									965
30	⧓ R-10	33-50/6"			...(30 ft) very dense	115.9	2.2		
									960
35									

Continued on next page

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BORING B-14

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 994 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◪ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▯ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	◪	SPT-11	18-35-50/5"	▨		Poorly Graded SAND with Gravel, very dense, light olive gray (5Y 6/2), dry, fine to coarse, trace of cobbles *(continued)*				
										955
40	⧓	R-12	50/5"							

Notes:
- Total depth of boring 40.4'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-15

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 991 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/26/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 40.4 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.904733; -116.604768
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately 300 ft west of proposed turbine 13 location

◪ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮	SK-1			SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, very dense, dry, light olive gray (5Y 6/2), fine to coarse sand, trace of gravel and cobbles				990
5	◪	SPT-2	8-23-50/5"							985
10	⧓	R-3	35-23-28 (51)				127.0	1.6		980
	◪	SPT-4	5-8-11 (19)			...(12.5) medium dense, damp				
15	⧓	R-5	50/2"			...(15) very dense				975
20	◪	SPT-6	27-24-25 (49)							970
	⧓	R-7	17-50/5"				115.2	1.5		
25	◪	SPT-8	10-50/4"		SP-SM	Poorly Graded SAND with Silt and Gravel, very dense, dry, olive gray (5Y 4/2), fine to coarse sand, trace of gravel and cobbles			G/S/F = 29/66/5	965
30	⧓	R-9	14-30-50/5"		SP	Poorly Graded SAND with Gravel, very dense, dry, light olive gray (5Y 6/2), fine to coarse sand, trace of gravel and cobbles	108.4	2.6	CONSOL	960
35										

Continued on next page

BORING B-15

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 991 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	◩ SPT-10	23-30-40 (70)			Poorly Graded SAND with Gravel, very dense, dry, light olive gray (5Y 6/2), fine to coarse sand, trace of gravel and cobbles *(continued)*				955
40	⧓ R-11	50/5"							

Notes:
- Total depth of boring 40.4'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-16

Sheet 1 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 979 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/12/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 60.9 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.902573; -116.604496
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION Approximately at proposed turbine 16 location

◤ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

MATERIAL DESCRIPTION

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮	SK-1		⁘	SP	[NATIVE] Alluvium Poorly Graded SAND with GRAVEL, medium dense, light olive gray (5Y 6/2), dry, fine to coarse, gravels up to ~4"				
	⧓	R-2	4-6-6 (12)	⁘						975
5	◤	SPT-3	7-9-9 (18)	⁘						
	⧓	R-4	25-14-11 (25)	⁘						970
10	◤	SPT-5	17-15-12 (27)	⁘						
	⧓	R-6	36-50	⁘		...(12.5 ft) very dense				965
15	◤	SPT-7	13-18-21 (39)	⁘		...(15 ft) dense				960
20	⧓	R-8	50/5"	⁘		...(20 ft) very dense				
	◤	SPT-9	13-22-22 (44)	⁘		...(22.5 ft) dense				955
25	⧓	R-10	21-50/5"	⁘		...(25 ft) very dense				950
30	◤	SPT-11	6-14-18 (32)	⁘		...(30 ft) dense				945
35										

Continued on next page

BORING B-16

Sheet 2 of 2

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 979 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	⧓	R-12	12-37-45 (82)			Poorly Graded SAND with GRAVEL, medium dense, light olive gray (5Y 6/2), dry, fine to coarse, gravels up to ~4" *(continued)* ..(35 ft) very dense	110.8	0.6	DS	
										940
40	◩	SPT-13	9-12-19 (31)			...(40 ft) medium dense				
										935
45	⧓	R-14	27-40-50/5"			...(45 ft) very dense	111.5	0.6		
										930
50	◩	SPT-15	24-24-35 (59)			...(50 ft) dense				
										925
55	⧓	R-16	7-32-50/5"			...(55 ft) very dense				
										920
60	◩	SPT-17	21-50/5"							

Notes:
- Total depth of boring 60.9'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-91

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1141 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/25/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 3 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.908525; -116.625118
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮ SK-1			SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, very dense, light olive gray (5Y 6/2), dry, fine to coarse gravel, cobbles and boulders up to ~3'				1140
	◩ SPT-2	50/3"							

Notes:
- Total depth of boring 3'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.
- Refusal at 3'.

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BORING B-92

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1171 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

DATES DRILLED 8/25/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 1 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.914409; -116.620391
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧗ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0					GP	[NATIVE] Alluvium Poorly Graded GRAVEL with Sand, very dense, light olive gray (5Y 6/2), dry, cobble to boulder up to ~2', clast supported				1170

Notes:
- Total depth of boring 1'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.
- Refusal at 1'.

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BORING B-93

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1079 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered

DATES DRILLED 8/26/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 1 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.914366; -116.610962
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0				░	SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, very dense, light olive gray (5Y 6/2), dry, fine to coarse sand				

Notes:
- Total depth of boring 1'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.
- Refusal at 1'.

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BORING B-94

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1060 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/25/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 16.5 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.909305; -116.611672
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0					SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, very dense, light olive gray (5Y 6/2), dry, fine to coarse, cobbles and boulders				1060
	▮	SK-1								
	⧓	R-2	50/3"				127.5	4.8	RESISTIVITY	
5	◩	SPT-3	8-50/1"							1055
	⧓	R-4	50/5"							
10	◩	SPT-5	20-44-24 (68)							1050
15	⧓	R-6	19-25-35 (60)				111.8	2.3		1045

Notes:
- Total depth of boring 16.5'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-95

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 996 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/17/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 15.9 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.902139; -116.608119
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮	SK-1		▨	SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, dense, light olive gray (5Y 6/2), dry, medium to coarse, trace of cobbles	127.5	5.0	RESISTIVITY	995
	◩	SPT-2	27-22-10 (32)	▨						
5	⧓	R-3	9-12-18 (30)	▨		...(5 ft) medium dense			RESISTIVITY	990
	◩	SPT-4	12-25-26 (51)	▨		...(7.5 ft) very dense				
10	⧓	R-5	28-36-50/1"	▨			126.8	0.4		985
15	◩	SPT-6	26-50/5"	▨						

Notes:
- Total depth of boring 15.9'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-96

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 1041 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/12/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 15.7 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.911686; -116.608022
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION

◩ Standard Penetration Test (SPT)	○ No Recovery	■ Shelby Tube
⧓ California-Type Ring Sample	☒ No Recovery	▮ Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮			SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, very dense, light olive gray (5Y 6/2), dry, fine to coarse, trace of cobbles			R-VALUE	1040
	▮ SK-1 ⧓ R-2	30-50/4"							
5	◩ SPT-3	50/3"							1035
	⧓ R-4	50/5"							
10	◩ SPT-5	14-15-26 (41)			...(10 ft) dense				1030
15	⧓ R-6	21-50/2"			...(15 ft) very dense	125.7	1.5		

Notes:
- Total depth of boring 15.7'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

Tetra Tech BAS
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Diamond Bar, CA 91765
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BORING B-97

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 946 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/14/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 16.5 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.901202; -116.600137
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION

◩ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▐ Grab/Bulk Sample

MATERIAL DESCRIPTION

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▐				SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, medium dense, light olive gray (5Y 6/2), dry, fine to coarse, trace of cobbles				945
	▐	SK-1					125.7	5.6	RESISTIVITY	
	◩	SPT-2	10-8-11 (19)							
5	⧓	R-3	17-24-33 (57)			...(5 ft) dense	110.9	1.9		940
	◩	SPT-4	17-25-29 (54)			...(7 ft) very dense				
10	⧓	R-5	50/4"							935
15	◩	SPT-6	11-12-33 (45)			...(15 ft) dense				930

Notes:
- Total depth of boring 16.5'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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BORING B-98

Sheet 1 of 1

PROJECT NAME Mountain View
CLIENT AES
GROUND ELEVATION 828 ft
PROJECT NUMBER GEN 20-33
PROJECT LOCATION Whitewater, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 8/26/2020
DRILLING CONTRACTOR 2R Drilling
BOREHOLE DEPTH 16.5 ft
LOGGED/CHECKED BY SCM/FC
DRILLING METHOD Hollow-Stem Auger
BOREHOLE DIAMETER 8 in
COORDINATES 33.901809; -116.57969
HAMMER DATA 140 lb Autohammer, 30" drop
BACKFILL Cuttings
LOCATION DESCRIPTION

◩	Standard Penetration Test (SPT)	○	No Recovery	■	Shelby Tube
⧓	California-Type Ring Sample	☒	No Recovery	▮	Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE	NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	▮				SP	[NATIVE] Alluvium Poorly Graded SAND with Gravel, very dense, light olive gray (5Y 6/2), dry, fine to coarse sand				
	▮ ⧓	SK-1 R-2	50/5"						R-VALUE	825
5	◩	SPT-3	18-18-17 (35)			...(5 ft) dense			G/S/F = 9/88/3	
	⧓	R-4	42-38-42 (80)			...(7.5 ft) very dense	119.4	1.4		820
10	◩	SPT-5	6-9-18 (27)			...(10 ft) medium dense				
										815
15	⧓	R-6	5-12-10 (22)							

Notes:
- Total depth of boring 16.5'.
- No groundwater seepage encountered.
- Boring backfilled with cuttings.

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Appendix B

MASW Geophysical Survey

REPORT

GEOPHYSICAL INVESTIGATION

AES WINDMILL FARM
WHITEWATER, CA

GEO*Vision* Project No. 20260

Prepared for

Tetra Tech BAS
21700 Copley Drive, #200
Diamond Bar, California 91765

Prepared by

GEO*Vision*, Inc.
1124 Olympic Drive
Corona, California 92881
(951) 549-1234

Report 20260-01 Rev 1

September 21, 2020

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1 INTRODUCTION

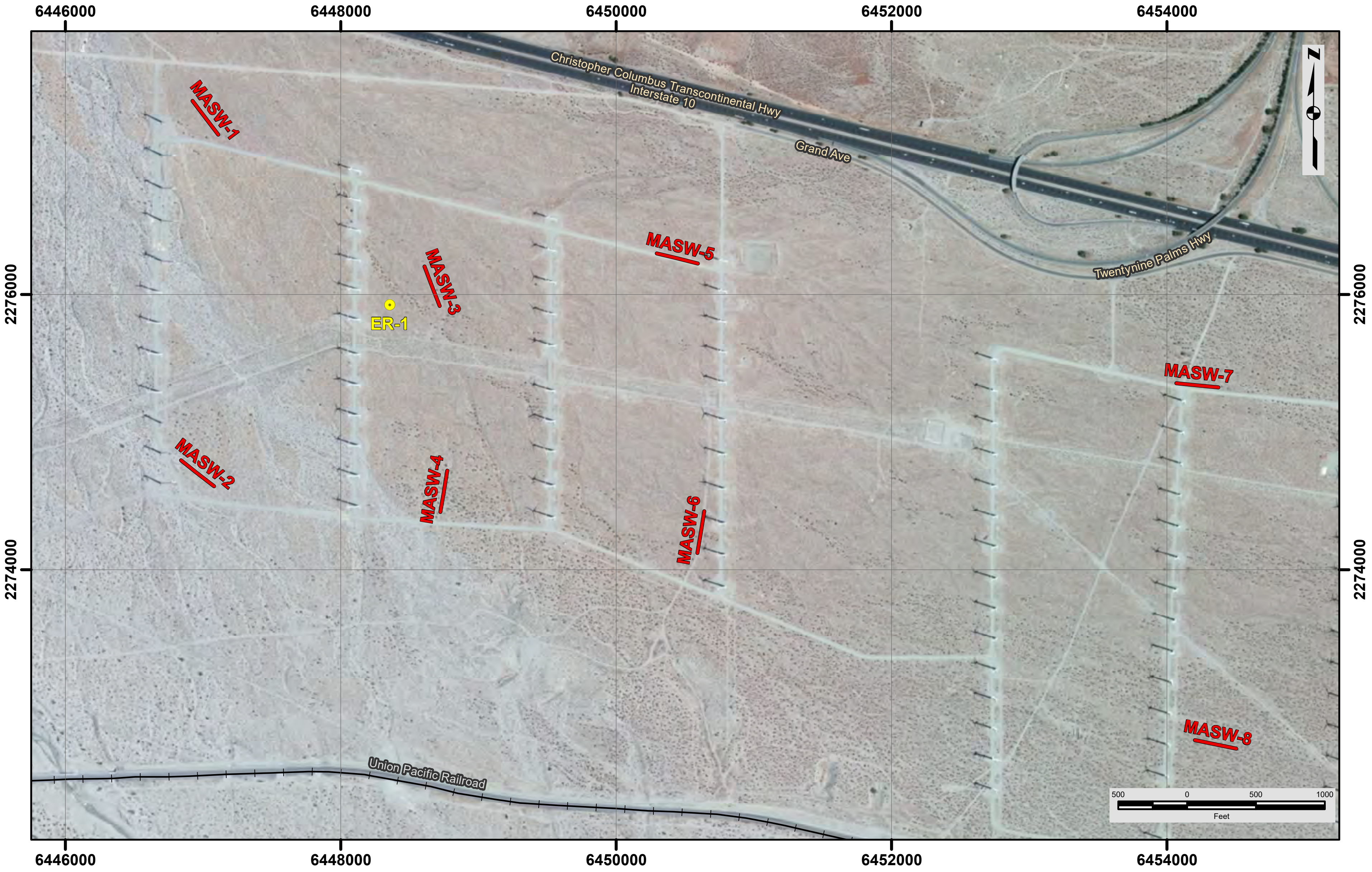
In-situ seismic measurements using surface wave techniques were performed at eight sites at the AES Windmill Farm in Whitewater, California on August 18-25, 2020. The purpose of this investigation was to provide a shear (S) wave velocity profile to a depth of 30 m (100 ft), or greater, and estimate the average S-wave velocity of the upper 30 m (V_{S30}) or 100 ft (V_{S100ft}). The surface wave technique utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) method. The locations of the surface wave testing locations are shown on Figure 1.

V_{S30} is used in the NEHRP provisions and the Uniform Building Code (UBC) to separate sites into classes for earthquake engineering design (BSSC, 2009). V_{S100ft} is used in the International Building Code (IBC) for site classification. These site classes are as follows:

Class A – hard rock – $V_{S30} > 1500$ m/s (UBC) or $V_{S100ft} > 5{,}000$ ft/s (IBC)
Class B – rock – $760 < V_{S30} \leq 1500$ m/s (UBC) or $2{,}500 < V_{S100ft} \leq 5{,}000$ ft/s (IBC)
Class C – very dense soil and soft rock – $360 < V_{S30} \leq 760$ m/s (UBC)
or $1{,}200 < V_{S100ft} \leq 2{,}500$ ft/s (IBC)
Class D – stiff soil – $180 < V_{S30} \leq 360$ m/s (UBC) or $600 < V_{S100ft} \leq 1{,}200$ ft/s (IBC)
Class E – soft soil – $V_{S30} < 180$ m/s (UBC) or $V_{S100ft} < 600$ ft/s (IBC)
Class F – soils requiring site-specific evaluation

At many sites, active surface wave techniques (MASW) with the utilization of portable energy sources, such as hammers and weight drops, are sufficient to obtain S-wave velocity sounding to 30 m (100 ft) depth. At sites with high ambient noise levels and/or very soft soils, these energy sources may not be sufficient to image to this depth and a larger energy source, such as a bulldozer, is necessary. Alternatively, passive surface wave techniques, such as the array microtremor technique can be used to extend the depth of investigation at sites that have adequate ambient noise conditions. It should be noted that two-dimensional passive-source surface wave arrays (e.g. triangular, circular, or L-shaped arrays) are expected to perform better than linear arrays.

This report contains the results of the surface wave measurements conducted at the site. An overview of the surface wave methods is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Data modeling is presented in Section 5 and interpretation and results are presented in Section 6. References and our professional certification are presented in Sections 7 and 8, respectively.



▬ Surface Wave Array (MASW)

⊙ Electrical Resistivity Sounding

NOTES:
1. Coordinate System: California State Plane, NAD83, Zone VI (0406), US Survey Feet
2. Base map source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

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Date:	9/1/2020
GV Project:	20260
Developed by:	C Martinez
Drawn by:	T Rodriguez
Approved by:	A Martin
File Name:	20260-1.MXD

FIGURE 1
SITE MAP

SITE LOCATED AT
WINDMILL FARM
WHITEWATER, CALIFORNIA

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2 OVERVIEW OF SURFACE WAVE TECHNIQUES

2.1 Introduction

Active- and passive-source (ambient vibration) surface wave techniques are routinely utilized for site characterization. Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the horizontal over vertical spectral ratio (HVSR) technique and the array and refraction microtremor methods.

The basis of surface wave methods is the dispersive characteristic of Rayleigh and Love waves when propagating in a layered medium. Surface waves of different wavelengths (λ) or frequencies (f) sample different depth. As a result of the variance in the shear stiffness of the distinct layers, waves with different wavelengths propagate at different phase velocities; hence, dispersion. A surface wave dispersion curve is the variation of V_R or V_L with λ or f. The Rayleigh wave phase velocity (V_R) depends primarily on the material properties (V_S, mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. The Love wave phase velocity (V_L) depends primarily on V_S and mass density. Rayleigh and Love wave propagation are also affected by damping or seismic quality factor (Q). Rayleigh wave techniques are utilized to measure vertically polarized S-waves (S_V-wave); whereas, Love wave techniques are utilized to measure horizontally polarized S-waves (S_H-wave).

2.2 Surface Wave Techniques

The MASW technique were utilized during this investigation and is discussed below.

2.2.1 MASW Technique

A description of the MASW method is given by Park, 1999a and 1999b and Foti, 2000. Ground motions are typically recorded by 24, or more, geophones typically spaced 1 to 3 m apart along a linear array and connected to a seismograph. Energy sources for shallow investigations include various sized hammers and vehicle mounted weight drops. When applying the MASW technique to develop a one-dimensional (1-D) V_S model, it is preferable to use multiple-source offsets from both ends of the array. The most commonly applied MASW technique is the Rayleigh-wave based MASW method, which we refer to as MAS_RW to distinguish from Love-wave based MASW (MAS_LW). MAS_RW and MAS_LW acquisition can easily be combined with P- and S-wave seismic refraction acquisition, respectively. MAS_RW data are generally recorded using a vertical source and vertical geophone but may also be recorded using a horizontal geophone with radial (in-line) orientation. MAS_LW data are recorded using transversely orientated horizontal source and transverse horizontal geophone.

A wavefield transform is applied to the time-history data to convert the seismic record from time-offset space to frequency-wavenumber (f-k) space in which the fundamental or higher surface-wave modes can be easily identified as energy maxima and picked. Frequency and/or wavenumber can easily be mapped to phase velocity, slowness, or wavelength using the following properties: $k = 2\pi/\lambda$, $\lambda = v/f$. Common wave-field transforms include: the f-k transform (a 2D fast Fourier transform), slant-stack transform (also referred to as intercept-slowness or τ-p transform and equivalent to linear Radon transform), frequency domain beamformer, and phase-shift transform. The minimum wavelength that can be recovered from MASW data set without spatial aliasing is equal to the minimum receiver spacing. Occasionally, SASW analysis procedures are used to extract surface wave dispersion data, from fixed receiver pairs, at smaller wavelengths than can be recovered by wavefield transformation. Construction of a dispersion curve over the wide frequency/wavelength range necessary to develop a robust V_S model while also limiting the maximum wavelength based on an established near-field criterion (e.g. Yoon and Rix, 2009; Li and Rosenblad, 2011), generally requires multiple source offsets.

Although the clear majority of MASW surveys record Rayleigh waves, it has been shown that Love wave techniques can be more effective in some environments, particularly shallow rock sites and sites with a highly attenuative, low velocity surface layer (Xia, et al., 2012; **GEO***Vision*, 2012; Yong, et al., 2013; Martin, et al., 2014). Rayleigh wave techniques, however, are generally more effective at sites where velocity gradually increases with depth because larger energy sources are readily available for the generation of Rayleigh waves. Rayleigh wave techniques are also more applicable to sites with high velocity layers and/or velocity inversions because the presence of such structures is more apparent in the Rayleigh wave dispersion curves than in Love wave dispersion curves. Rayleigh wave techniques are preferable at sites with a high velocity surface layer because Love waves do not theoretically exist in such environments. Occasionally, the horizontal radial component of a Rayleigh wave may yield higher quality dispersion data than the vertical component because different modes of propagation may have more energy in one component than the other. Recording both the vertical and horizontal components of the Rayleigh wave is particularly useful at sites with complex modes of propagation or when attempting to recover multiple Rayleigh wave modes for multi-mode modeling as demonstrated in Dal Moro, et al, 2015. Joint inversion of Rayleigh and Love wave data may yield more accurate V_S models and also offer a means to investigate anisotropy, where S_V- and S_H-wave velocity are not equal, as shown in Dal Moro and Ferigo, 2011.

2.3 Surface Wave Dispersion Curve Modeling

The dispersion curves generated from the active surface wave soundings are generally combined and modeled using iterative forward and inverse modeling routines. The final model profile is assumed to represent actual site conditions. The theoretical model used to interpret the dispersion curve assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of active surface wave testing provide a good “global” estimate of the material properties along the array. The results may be more representative of the site than a borehole “point” estimate.

The surface wave forward problem is typically solved using the Thomson-Haskell transfer-matrix (Thomson, 1950; Haskell, 1953) later modified by Dunkin (1965) and Knopoff (1964), dynamic stiffness matrix (Kausel and Roësset, 1981), or reflection and transmission coefficient (Kennett, 1974) methods. All of these methods can determine fundamental- and higher-mode phase velocities, which correspond to plane waves in 2-D space. The transfer-matrix method is often used in MASW surface-wave software packages, whereas the dynamic stiffness matrix is utilized in many SASW software packages. MAS_RW surface-wave modeling may involve modeling of the fundamental mode, some form of effective mode, or multiple individual modes (multi-mode). As outlined in Roësset et al. (1991), several options exist for forward modeling of Rayleigh wave SASW data. One formulation takes into account only fundamental mode plane

Rayleigh-wave motion (called the 2-D solution), whereas another includes all stress waves (e.g. body, fundamental, and higher mode surface waves) and incorporates a generalized receiver geometry (3-D global solution) or actual receiver geometry (3-D array solution).

The fundamental mode assumption is generally applicable to modeling Rayleigh-wave dispersion data collected at normally dispersive sites, providing there are not abrupt increases in velocity or steep velocity gradients. Effective-mode or multi-mode approaches are often required for irregularly dispersive sites and sites with steep velocity gradients at shallow depth. If active surface wave data are combined or MAS$_R$W data are combined from multiple seismic records with different source offsets and receiver gathers, then effective-mode computations are limited to algorithms that assume far-field plane Rayleigh wave propagation. Local search (e.g. linearized matrix inversion methods) or global search methods (e.g., Monte Carlo approaches such as simulated annealing, generic algorithms and neighborhood algorithm) are typically used to solve the inverse problem.

The maximum wavelength (λ_{max}) recovered from a surface wave data set is typically used to estimate depth of investigation although a sensitivity analysis of the V_S models would be a more robust means to estimate depth of investigation. For normally dispersive velocity profiles with a gradual increase in V_S with depth, the maximum depth of investigation is on the order of $\lambda_{max}/2$ for both Rayleigh and Love wave dispersion data. For velocity profiles with an abrupt increase in V_S at depth, the maximum depth of investigation is on the order of $\lambda_{max}/3$ for Rayleigh wave dispersion data but less than $\lambda_{max}/3$ for Love wave dispersion data. The depth of investigation can be highly variable for sites with complex velocity structure (e.g. high velocity layers).

As with all surface geophysical methods, the inversion of surface wave dispersion data does not yield a unique V_S model and multiple possible solutions may equally fit the experimental data. Based on experience at other sites, the shear wave velocity models (V_S and layer thicknesses) determined by surface wave testing are within 20% of the velocities and layer thicknesses that would be determined by other seismic methods (Brown, 1998). The average velocity of the upper 30 m, however, is much more accurate, often to better than 5%, because it is not sensitive to the layering in the model. V_{S30} does not appear to suffer from the non-uniqueness inherent in V_S models derived from surface wave dispersion curves (Martin et al., 2006, Comina et al., 2011). Therefore, V_{S30} is more accurately estimated from the inversion of surface wave dispersion data than the resulting V_S models.

It may not always be possible to develop a coherent, fundamental mode dispersion curve over sufficient frequency range for modeling due to dominant higher modes with the higher modes not clearly identifiable for multi-mode modeling. It may, however, be possible to identify the Rayleigh wave phase velocity of the fundamental mode at 40 m wavelength (V_{R40}) in which case V_{S30} can at least be estimated using the Brown et al., 2000 relationship:

$$V_{S30} = 1.045V_{R40}$$

This relationship was established based on a statistical analysis of a large number of surface wave data sets from sites with control by velocities measured in nearby boreholes and has been further evaluated by Martin and Diehl, 2004, and Albarello and Gargani, 2010. Further investigation of this approach has revealed that V_{S30} is generally between V_{R40} and V_{R45} with V_{R40} often being most appropriate for shallow groundwater sites and V_{R45} for deep ground water sites. A detailed study of such an approach for Love wave dispersion data has not been conducted; however, preliminary analysis demonstrates that V_{S30} is generally between V_{L50} and V_{L55}. Although we do not recommend that these empirical V_{S30} estimates replace modeling of surface wave dispersion data, they do offer a means of cost effectively evaluating V_{S30} over a large area. V_{R40} or V_{L55} can also be used to quantify error in V_{S30} by evaluating the scatter in the dispersion data at these wavelengths.

3 FIELD PROCEDURES

The surface wave sounding locations at the site were established by Tetra Tech and **GEO***Vision* personnel and are shown in Figure 1. Surface wave data were acquired along all arrays using the MASW technique.

MASW equipment used during this investigation consisted of two Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, seismic cable, a 4 lb. hammer, a 12 and 20 lb. sledgehammer, and an accelerated weight drop (AWD). MASW data were acquired along a linear array of 48 geophones spaced 2 m (6.56 ft) apart. Shot points were located between 2 and 30 m (6.56 and 98.4 ft) from the end geophone locations and at 16 m (52.5 ft) intervals in the interior of the array. The 4 lb. hammer and/or 12 lb. sledgehammer were used for the near offset source locations and interior source locations. The AWD and 20 lb. sledgehammer were used for source locations offset from the ends of the array. Data from the transient impacts (hammers) were generally averaged 6 – 10 times to improve the signal-to-noise ratio. All field data were saved to hard disk and documented on field data acquisition forms.

4 DATA REDUCTION

The MASW data were reduced using the software Seismic Pro Surface V9.0 developed by Geogiga and multiple in-house scripts for various data extraction and formatting tasks, with all data reduction documented in a Microsoft Excel spreadsheet.

The following steps were used for data reduction:

- Input seismic records to be used for analysis into software package.
- Check and correct source and receiver geometry as necessary.
- Select offset range used for analysis (multiple offset ranges utilized for each seismic record as discussed below) and document in spreadsheet.
- Apply phase shift transform to seismic record to convert the data from time – offset to frequency – phase velocity space.
- Identify, pick, save, and document dispersion curve.
- Change the receiver offset range and repeat process.
- Repeat process for all seismic records.
- Use in-house script to apply near-field criteria with maximum wavelength set equal to 1.0 times the source to midpoint of receiver array distance.
- Use in-house script to merge multiple dispersion curves extracted from the MASW data collected along each seismic line for a specific source type (different source locations, different receiver offset ranges, etc.).
- Edit dispersion data, as necessary (e.g. delete poor quality curves and outliers).
- Calculate a representative dispersion curve at equal log-frequency or log-wavelength spacing for the MASW dispersion data using a moving average, polynomial curve fitting routine.

This unique data reduction strategy, which can involve combination of over 100 dispersion curves for a 1D sounding, is designed for characterizing sites with complex velocity structure that do not yield surface wave dispersion data over a wide frequency range from a single source type or source location. The data reduction strategy ensures that the dispersion curve selected for modeling is representative of average conditions beneath the array and spans as broad a frequency/wavelength range as possible while considering near field effects.

The representative dispersion curves from the surface wave data were combined and the moving average polynomial curve fitting routine in WinSASW V3 was used to generate a composite representative dispersion curve for modeling. An equal logarithm wavelength sample rate was used for the representative dispersion curve to reflect the gradual loss in model resolution with depth.

5 DATA MODELING

Surface wave data were modeled using the fundamental mode routine in WinSASW V3 software package. During this process an initial velocity model was generated based on general characteristics of the dispersion curve and the inverse modeling routine utilized to adjust the layer V_S until an acceptable agreement with the observed data was obtained. Layer thicknesses were adjusted, and the inversion process repeated until a V_S model was developed with low RMS error between the observed and calculated dispersion curves. In many cases, once an acceptable V_S model is developed, layer thicknesses are again adjusted, and the inversion process repeated to develop an ensemble of V_S models with similar RMS error to quantify non-uniqueness. The primary purpose of this investigation was to estimate V_{S30} and, therefore, it was not considered necessary to develop multiple V_S models. Data inputs into the modeling software include layer thickness, S-wave velocity, P-wave velocity or Poisson's ratio, and mass density. P-wave velocity and mass density only have a very small influence (i.e. less than 10%) on the S-wave velocity model generated from a surface wave dispersion curve. However, realistic assumptions for P-wave velocity, which is significantly impacted by the location of the saturated zone, and mass density will slightly improve the accuracy of the S-wave velocity model.

Constant mass density values of 1.78 to 2.09 gm/cm^3 (111 to 130 lb./ft^3) were used in the velocity profiles for subsurface soils/rock depending on P- and S-wave velocity. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible (±2%) effect on the estimated V_S from surface wave dispersion data. During modeling of Rayleigh wave dispersion data, the compression wave velocity, V_P, for unsaturated sediments was estimated using a Poisson's ratio, v, of 0.3 and the relationship:

$$V_P = V_S\,[(2(1\text{-}v))/(1\text{-}2v)]^{0.5}$$

Poisson's ratio has a larger effect than density on the estimated V_S from Rayleigh wave dispersion data. Achenbach (1973) provides approximate relationship between Rayleigh wave velocity (V_R), V_S and v:

$$V_R = V_S\,[(0.862 + 1.14\,v)/(1+ v)]$$

Using this relationship, it can be shown that V_S derived from V_R only varies by about 10% over possible 0 to 0.5 range for Poisson's ratio where:

$$V_S = 1.16V_R \text{ for } v = 0$$
$$V_S = 1.05V_R \text{ for } v = 0.5$$

The realistic range of the Poisson's ratio for typical unsaturated sediments is about 0.25 to 0.35. Over this range, V_S derived from modeling of Rayleigh wave dispersion data will vary by about 5%. There is no evidence of shallow, saturated sediments in the seismic data, which would have a high Poisson's ratio.

6 INTERPRETATION AND RESULTS

The fit of the calculated fundamental mode dispersion curve to the experimental data collected along all eight arrays and the modeled V_S profile for the surface wave soundings are presented as Figures 2 – 9, respectively. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in V_S at greater depth. The V_S profile used to match the field data is provided in tabular form in both metric and Imperial units as Tables 1 – 16, respectively.

6.1 MASW-1

The V_S models were developed from the surface wave dispersion data derived from MASW data acquired along MASW-1, respectively (Figure 2 and Tables 1 and 2).

The estimated depth of investigation for the surface wave sounding is about 40 m (131 ft). The V_S model indicates that V_S gradually increases with depth from about 217 m/s (712 ft/s) immediately below the surface to about 686 m/s (2,251 ft/s) at a depth of about 31 m (102 ft).

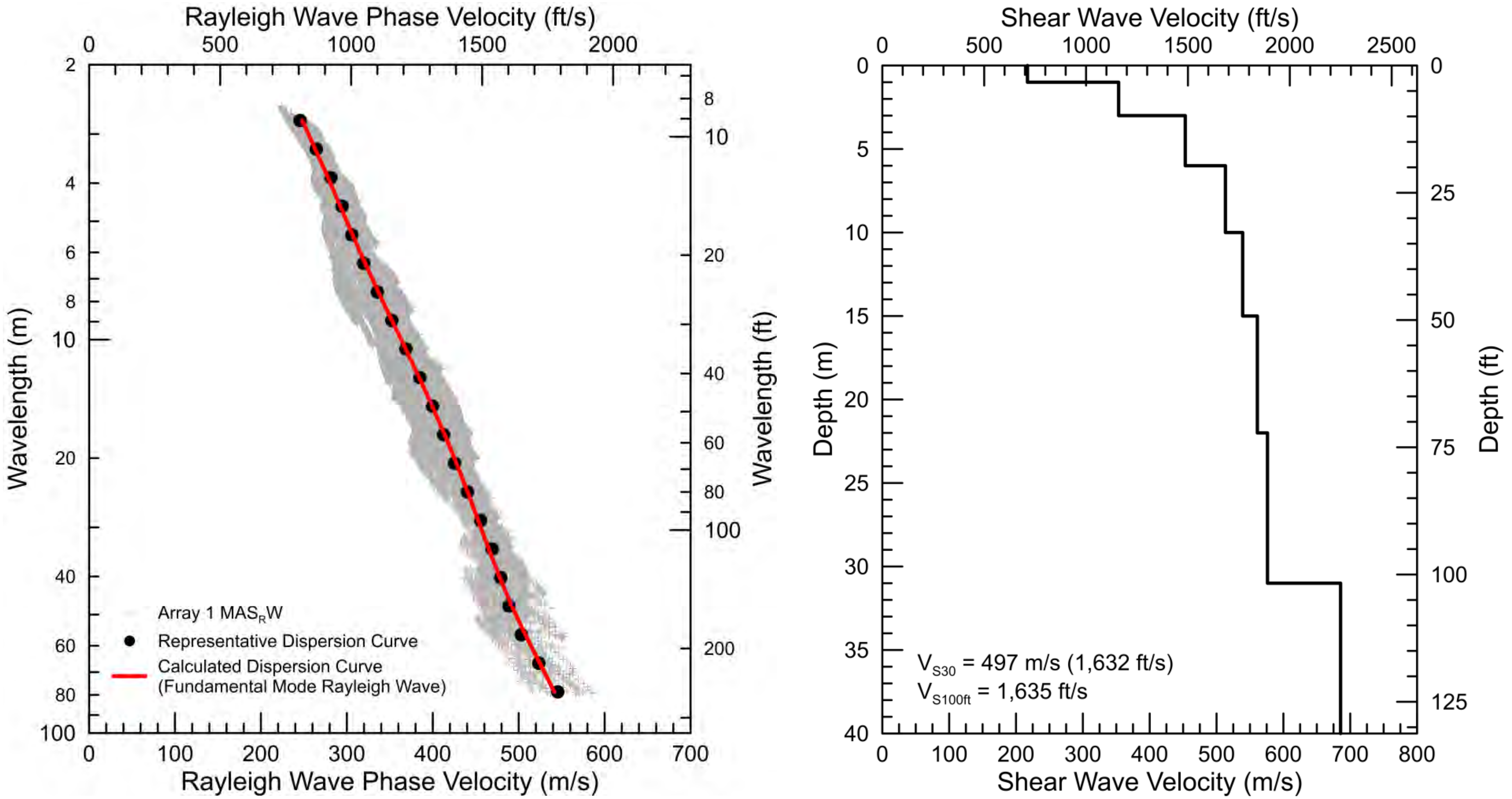
The average shear wave velocity to a depth of 30 m (V_{S30}) is 497 m/s for the MASW-1 V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,635 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of MASW-1 is classified as Site Class C, very dense soil and soft rock.

Table 1 MASW-1 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	1	217	406	0.300	1.86
1	2	354	661	0.300	1.95
3	3	453	848	0.300	2.00
6	4	513	960	0.300	2.02
10	5	539	1009	0.300	2.03
15	7	561	1050	0.300	2.04
22	9	576	1078	0.300	2.05
31	Half Space	686	1283	0.300	2.08

Table 2 MASW-1 Vs Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb./ft^3)
0.0	3.3	712	1333	0.300	116
3.3	6.6	1160	2170	0.300	122
9.8	9.8	1487	2781	0.300	125
19.7	13.1	1684	3150	0.300	126
32.8	16.4	1769	3309	0.300	127
49.2	23.0	1841	3444	0.300	127
72.2	29.5	1890	3537	0.300	128
101.7	Half Space	2249	4208	0.300	130



V_{S30} = 497 m/s (1,632 ft/s)

V_{S100ft} = 1,635 ft/s

MASW 1 - Field, representative, and calculated surface wave dispersion data (left) and associated V_S model (right)

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Project No.: 20260
Date: September 11, 2020
Drawn By: C. Martinez
Approved By: A. Martin
File P:\Project Files\2020\20260 - Tetra Tech...\Report\Figure2.cdr

FIGURE 2
MASW 1: SURFACE WAVE MODEL

AES WINDMILL FARM
WHITEWATER, CALIFORNIA

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6.2 MASW-2

The V_S models were developed from the surface wave dispersion data derived from MASW data acquired along MASW-2, respectively (Figure 3 and Tables 3 and 4).

The estimated depth of investigation for the surface wave sounding is about 40 m (131 ft). The V_S model indicates that V_S gradually increases with depth from about 225 m/s (739 ft/s) immediately below the surface to about 686 m/s (2,251 ft/s) at a depth of about 31 m (102 ft).

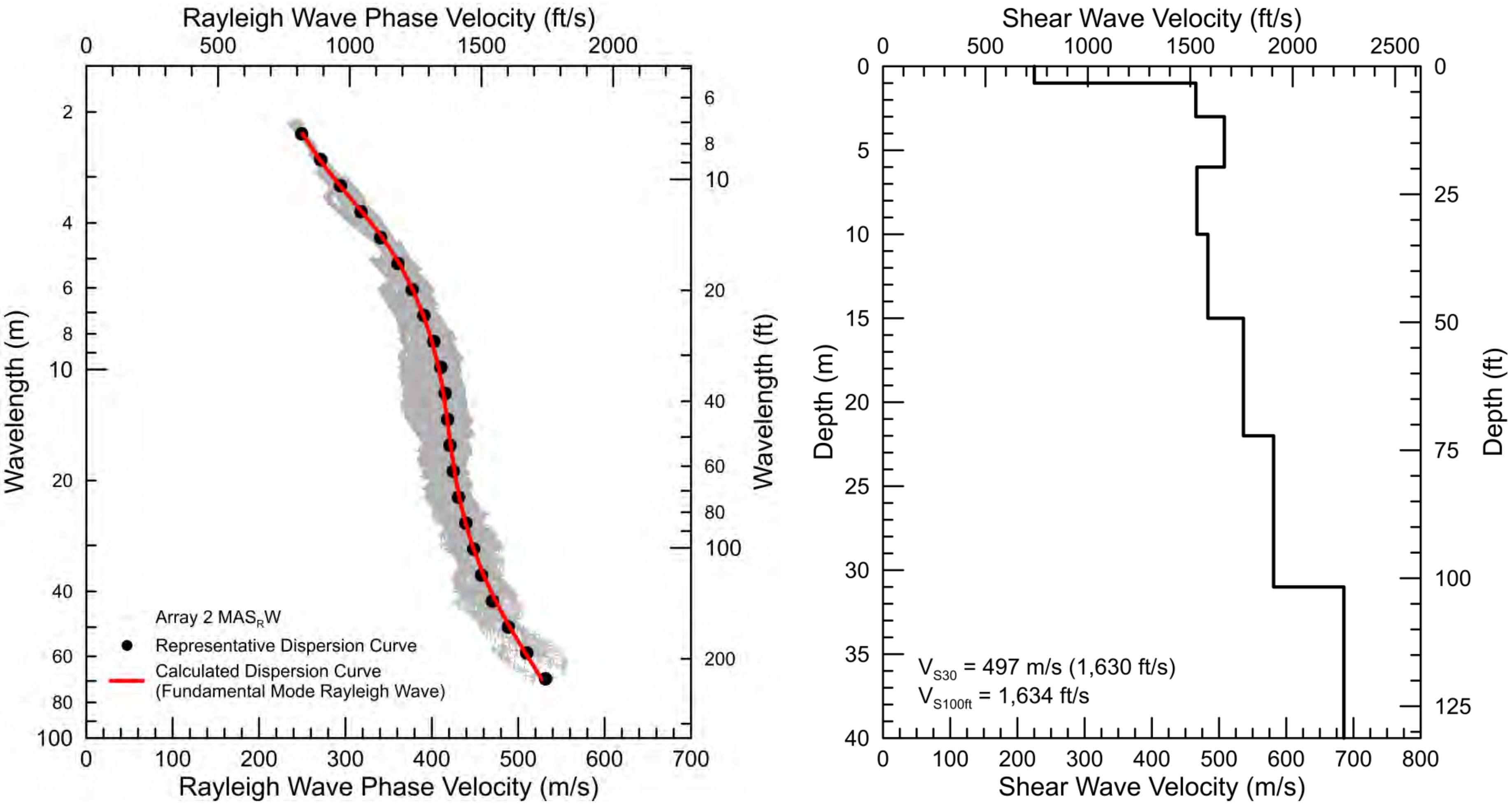
The average shear wave velocity to a depth of 30 m (V_{S30}) is 497 m/s for the MASW-2 V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,634 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of MASW-2 is classified as Site Class C, very dense soil and soft rock.

Table 3 MASW-2 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	1	225	421	0.300	1.86
1	2	466	871	0.300	2.01
3	3	508	950	0.300	2.02
6	4	467	874	0.300	2.01
10	5	483	904	0.300	2.01
15	7	536	1003	0.300	2.03
22	9	581	1087	0.300	2.05
31	Half Space	686	1283	0.300	2.09

Table 4 MASW-2 V_S Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb./ft^3)
0.0	3.3	739	1382	0.300	116
3.3	6.6	1527	2857	0.300	125
9.8	9.8	1666	3117	0.300	126
19.7	13.1	1532	2867	0.300	125
32.8	16.4	1586	2967	0.300	125
49.2	23.0	1760	3292	0.300	127
72.2	29.5	1907	3567	0.300	128
101.7	Half Space	2250	4209	0.300	130



MASW 2 - Field, representative, and calculated surface wave dispersion data (left) and associated V_S model (right)

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FIGURE 3
MASW 2: SURFACE WAVE MODEL

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6.3 MASW-3

The V_S models were developed from the surface wave dispersion data derived from MASW data acquired along MASW-3, respectively (Figure 4 and Tables 5 and 6).

The estimated depth of investigation for the surface wave sounding is about 40 m (131 ft). The V_S model indicates that V_S gradually increases with depth from about 174 m/s (571 ft/s) immediately below the surface to about 679 m/s (2,228 ft/s) at a depth of about 30 m (98 ft).

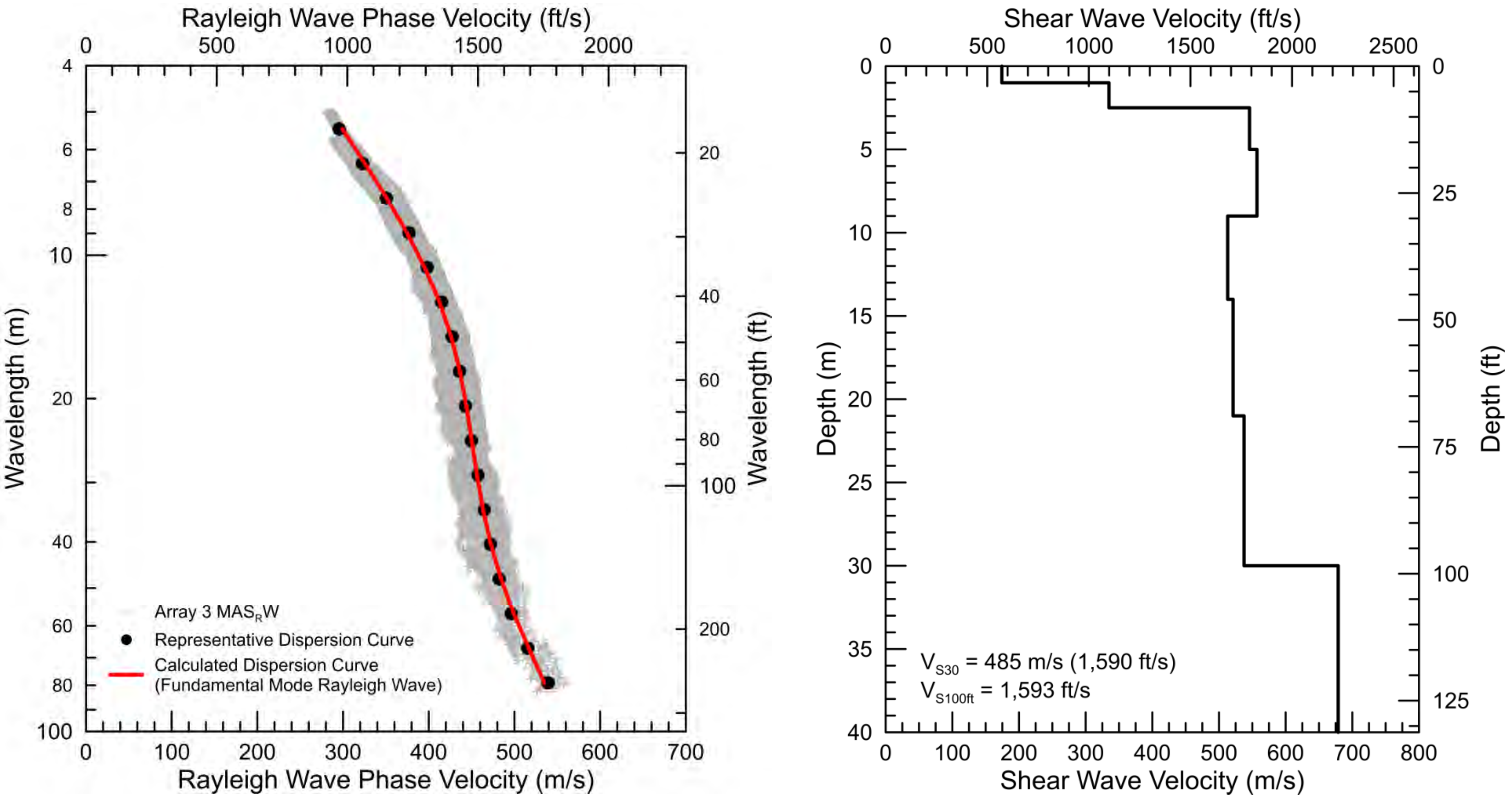
The average shear wave velocity to a depth of 30 m (V_{S30}) is 485 m/s for the MASW-3 V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,593 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of MASW-3 is classified as Site Class C, very dense soil and soft rock.

Table 5 MASW-3 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	1	174	326	0.300	1.78
1	1.5	335	627	0.300	1.95
2.5	2.5	546	1021	0.300	2.03
5	4	557	1042	0.300	2.04
9	5	513	960	0.300	2.03
14	7	521	975	0.300	2.03
21	9	538	1006	0.300	2.04
30	Half Space	679	1270	0.300	2.08

Table 6 MASW-3 V_S Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb./ft^3)
0.0	3.3	572	1070	0.300	111
3.3	4.9	1099	2056	0.300	122
8.2	8.2	1791	3350	0.300	127
16.4	13.1	1827	3419	0.300	127
29.5	16.4	1683	3149	0.300	127
45.9	23.0	1710	3199	0.300	127
68.9	29.5	1764	3300	0.300	127
98.4	Half Space	2227	4166	0.300	130



MASW 3 - Field, representative, and calculated surface wave dispersion data (left) and associated V_S model (right)

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FIGURE 4
MASW 3: SURFACE WAVE MODEL

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6.4 MASW-4

The V_S models were developed from the surface wave dispersion data derived from MASW data acquired along MASW-4, respectively (Figure 5 and Tables 6 and 7).

The estimated depth of investigation for the surface wave sounding is about 40 m (131 ft). The V_S model indicates that V_S gradually increases with depth from about 176 m/s (577 ft/s) immediately below the surface to about 606 m/s (1,988 ft/s) at a depth of about 33 m (108 ft).

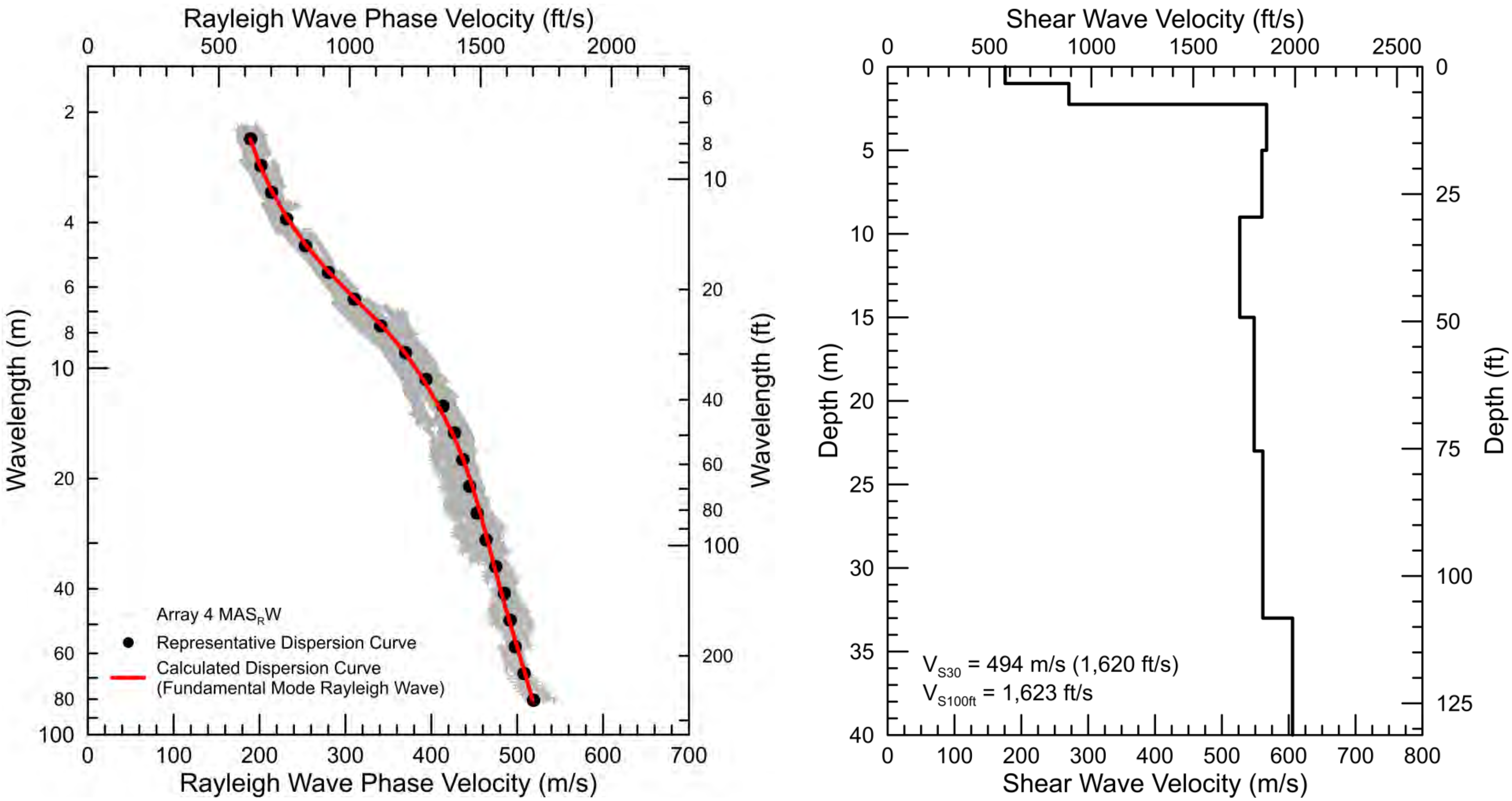
The average shear wave velocity to a depth of 30 m (V_{S30}) is 494 m/s for the MASW-4 V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,623 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of MASW-4 is classified as Site Class C, very dense soil and soft rock.

Table 7 MASW-4 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	1	176	329	0.300	1.78
1	1.25	271	507	0.300	1.91
2.25	2.75	567	1061	0.300	2.05
5	4	560	1048	0.300	2.05
9	6	527	986	0.300	2.03
15	8	548	1026	0.300	2.04
23	10	561	1050	0.300	2.05
33	Half Space	606	1133	0.300	2.06

Table 8 MASW-4 V_S Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb./ft^3)
0.0	3.3	577	1079	0.300	111
3.3	4.1	890	1664	0.300	119
7.4	9.0	1860	3480	0.300	128
16.4	13.1	1837	3437	0.300	128
29.5	19.7	1728	3233	0.300	127
49.2	26.2	1799	3365	0.300	127
75.5	32.8	1842	3445	0.300	128
108.3	Half Space	1987	3718	0.300	129



MASW 4 - Field, representative, and calculated surface wave dispersion data (left) and associated V_S model (right)

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FIGURE 5
MASW 4: SURFACE WAVE MODEL

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6.5 MASW-5

The V_S models were developed from the surface wave dispersion data derived from MASW data acquired along MASW-5, respectively (Figure 6 and Tables 7 and 8).

The estimated depth of investigation for the surface wave sounding is about 40 m (131 ft). The V_S model indicates that V_S gradually increases with depth from about 289 m/s (948 ft/s) immediately below the surface to about 529 m/s (1,736 ft/s) at a depth of about 28 m (92 ft).

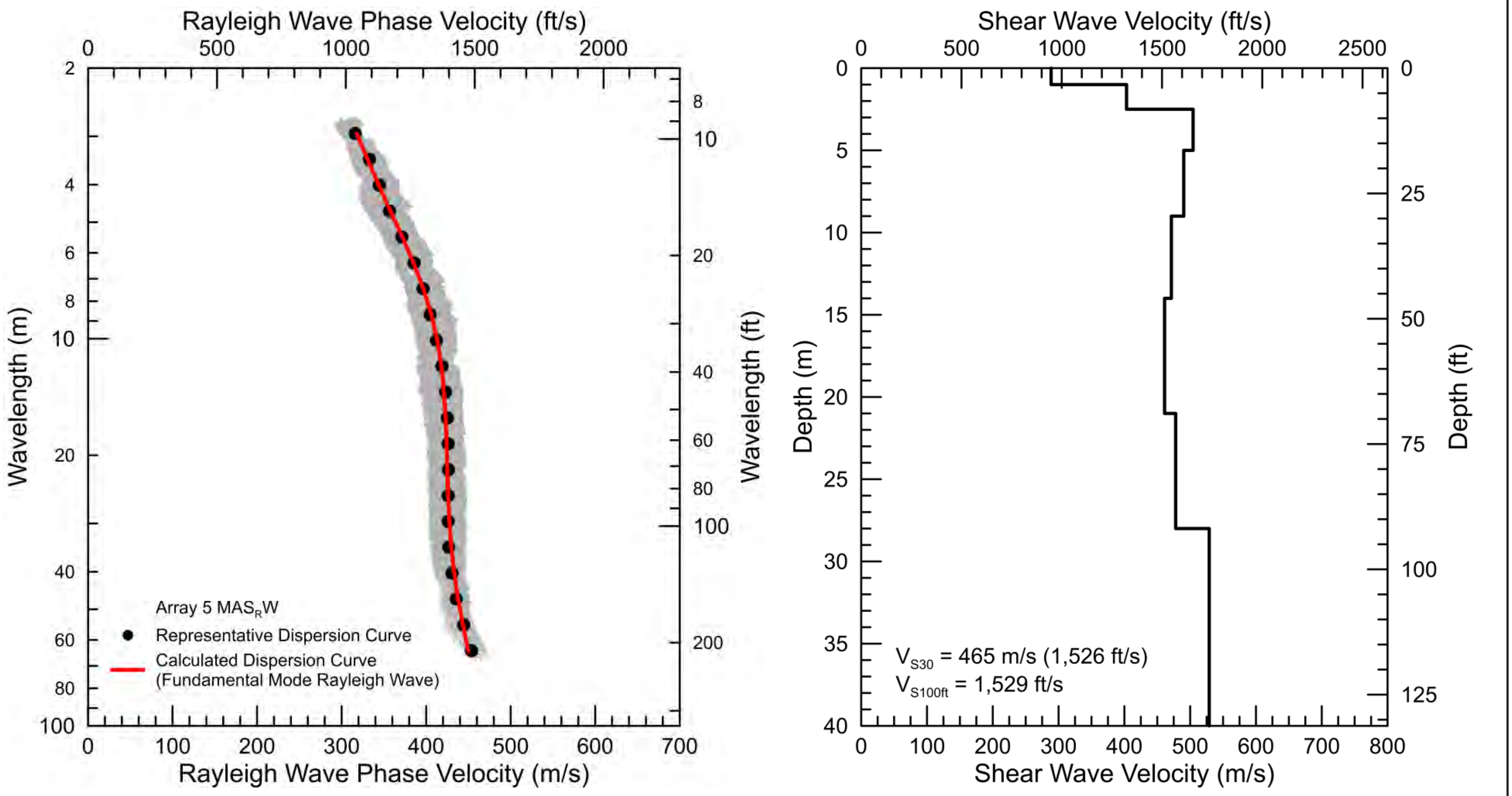
The average shear wave velocity to a depth of 30 m (V_{S30}) is 465 m/s for the MASW-5 V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,529 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of MASW-5 is classified as Site Class C, very dense soil and soft rock.

Table 9 MASW-5 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	1	289	540	0.300	1.93
1	1.5	403	755	0.300	1.98
2.5	2.5	505	944	0.300	2.02
5	4	490	917	0.300	2.01
9	5	471	882	0.300	2.01
14	7	461	863	0.300	2.01
21	7	478	894	0.300	2.02
28	Half Space	529	990	0.300	2.04

Table 10 MASW-5 V_S Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb./ft^3)
0.0	3.3	947	1772	0.300	120
3.3	4.9	1323	2475	0.300	124
8.2	8.2	1655	3096	0.300	126
16.4	13.1	1608	3009	0.300	125
29.5	16.4	1547	2894	0.300	125
45.9	23.0	1513	2830	0.300	125
68.9	23.0	1568	2933	0.300	126
91.9	Half Space	1736	3247	0.300	127



MASW 5 - Field, representative, and calculated surface wave dispersion data (left) and associated V_S model (right)

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FIGURE 6
MASW 5: SURFACE WAVE MODEL

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6.6 MASW-6

The V_S models were developed from the surface wave dispersion data derived from MASW data acquired along MASW-6, respectively (Figure 7 and Tables 8 and 9).

The estimated depth of investigation for the surface wave sounding is about 40 m (131 ft). The V_S model indicates that V_S gradually increases with depth from about 276 m/s (905 ft/s) immediately below the surface to about 600 m/s (1,968 ft/s) at a depth of about 33 m (108 ft).

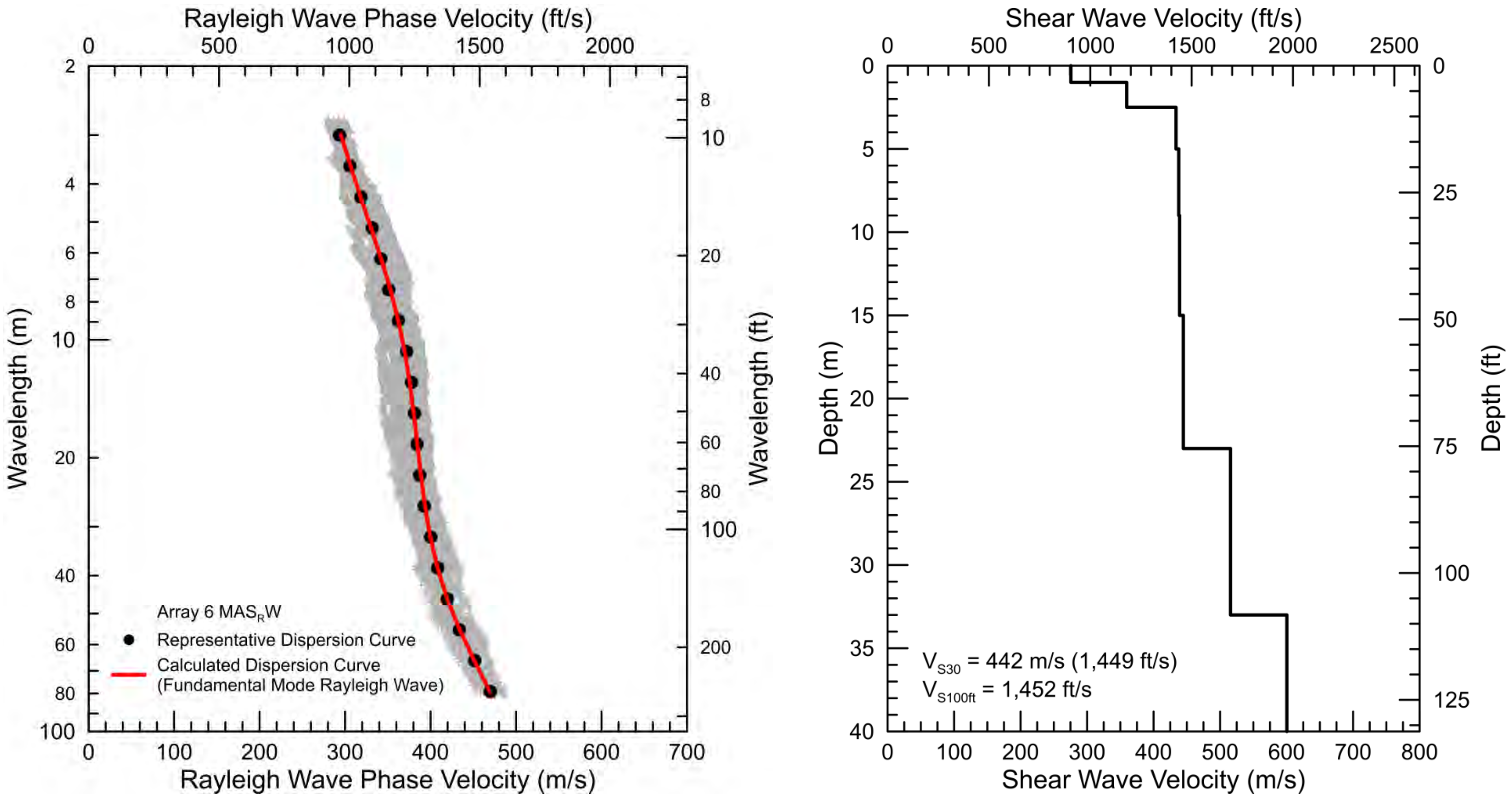
The average shear wave velocity to a depth of 30 m (V_{S30}) is 442 m/s for the MASW-6 V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,452 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of MASW-6 is classified as Site Class C, very dense soil and soft rock.

Table 11 MASW-6 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	1	276	516	0.300	1.91
1	1.5	360	673	0.300	1.95
2.5	2.5	434	811	0.300	1.99
5	4	438	819	0.300	1.99
9	6	439	821	0.300	1.99
15	8	445	832	0.300	2.00
23	10	516	965	0.300	2.02
33	Half Space	600	1123	0.300	2.06

Table 12 MASW-6 V_S Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb./ft^3)
0.0	3.3	904	1692	0.300	119
3.3	4.9	1179	2207	0.300	122
8.2	8.2	1423	2662	0.300	124
16.4	13.1	1436	2687	0.300	124
29.5	19.7	1441	2695	0.300	124
49.2	26.2	1459	2730	0.300	125
75.5	32.8	1692	3165	0.300	126
108.3	Half Space	1970	3685	0.300	129



MASW 6 - Field, representative, and calculated surface wave dispersion data (left) and associated V_S model (right)

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FIGURE 7
MASW 6: SURFACE WAVE MODEL

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6.7 MASW-7

The V_S models were developed from the surface wave dispersion data derived from MASW data acquired along MASW-7, respectively (Figure 8 and Tables 9 and 10).

The estimated depth of investigation for the surface wave sounding is about 40 m (131 ft). The V_S model indicates that V_S gradually increases with depth from about 180 m/s (591 ft/s) immediately below the surface to about 539 m/s (1,768 ft/s) at a depth of about 32.5 m (107 ft).

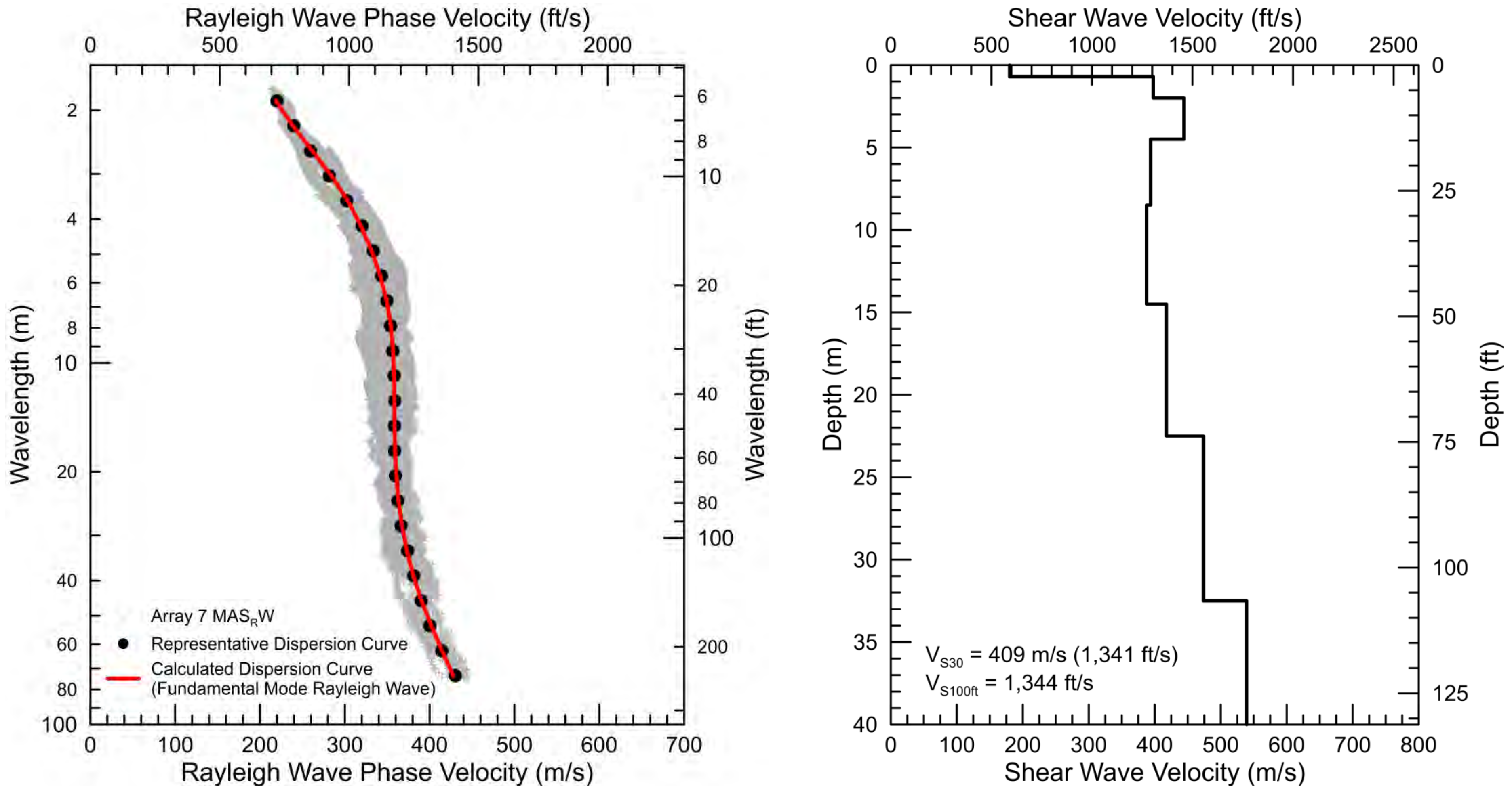
The average shear wave velocity to a depth of 30 m (V_{S30}) is 409 m/s for the MASW-7 V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,344 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of MASW-7 is classified as Site Class C, very dense soil and soft rock.

Table 13 MASW-7 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	0.7	180	337	0.300	1.79
0.7	1.3	398	744	0.300	1.98
2	2.5	444	831	0.300	1.99
4.5	4	394	736	0.300	1.98
8.5	6	388	725	0.300	1.97
14.5	8	418	782	0.300	1.99
22.5	10	474	886	0.300	2.01
32.5	Half Space	539	1009	0.300	2.03

Table 14 MASW-7 V_S Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb./ft^3)
0.0	2.3	591	1105	0.300	112
2.3	4.3	1305	2442	0.300	124
6.6	8.2	1458	2727	0.300	124
14.8	13.1	1291	2416	0.300	124
27.9	19.7	1272	2379	0.300	123
47.6	26.2	1371	2564	0.300	124
73.8	32.8	1554	2908	0.300	125
106.6	Half Space	1769	3310	0.300	127



MASW 7 - Field, representative, and calculated surface wave dispersion data (left) and associated V_S model (right)

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FIGURE 8
MASW 7: SURFACE WAVE MODEL

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6.8 MASW-8

The V_S models were developed from the surface wave dispersion data derived from MASW data acquired along MASW-8, respectively (Figure 9 and Tables 9 and 10).

The estimated depth of investigation for the surface wave sounding is about 40 m (131 ft). The V_S model indicates that V_S gradually increases with depth from about 265 m/s (869 ft/s) immediately below the surface to about 581 m/s (1,906 ft/s) at a depth of about 31 m (102 ft).

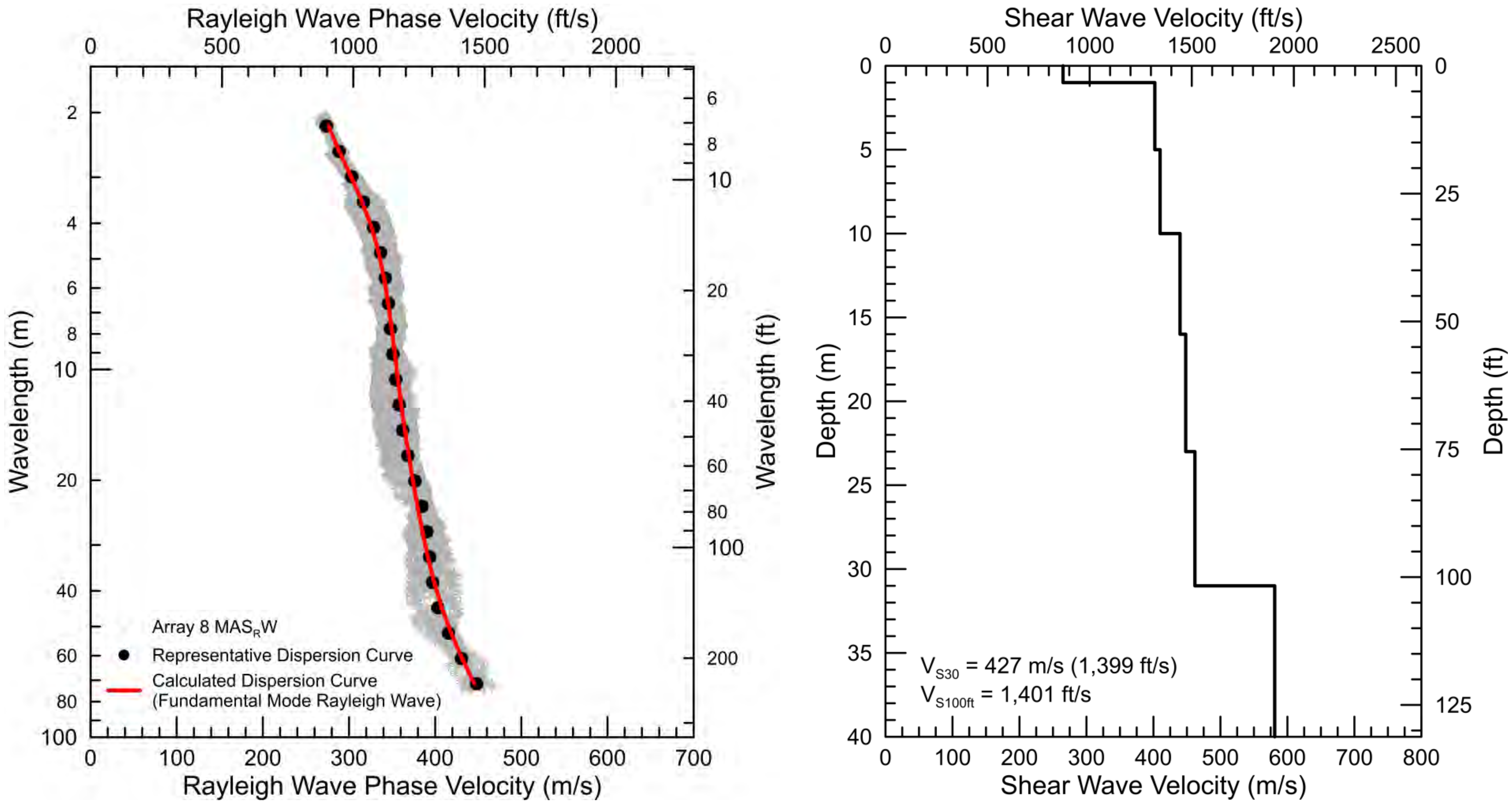
The average shear wave velocity to a depth of 30 m (V_{S30}) is 427 m/s for the MASW-8 V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,401 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of MASW-8 is classified as Site Class C, very dense soil and soft rock.

Table 15 MASW-8 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	1	265	496	0.300	1.90
1	4	402	752	0.300	1.99
5	5	410	767	0.300	1.99
10	6	440	823	0.300	1.99
16	7	448	839	0.300	2.00
23	8	462	864	0.300	2.01
31	Half Space	581	1087	0.300	2.05

Table 16 MASW-8 V_S Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb./ft^3)
0.0	3.3	869	1627	0.300	119
3.3	13.1	1319	2468	0.300	124
16.4	16.4	1345	2516	0.300	124
32.8	19.7	1443	2699	0.300	124
52.5	23.0	1470	2751	0.300	125
75.5	26.2	1515	2835	0.300	125
101.7	Half Space	1906	3566	0.300	128



MASW 8 - Field, representative, and calculated surface wave dispersion data (left) and associated V_S model (right)

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FIGURE 9
MASW 8: SURFACE WAVE MODEL

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8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

Prepared by,

09/21/2020

Christopher Martinez Date
Staff Geophysicist
GEO*Vision* Geophysical Services

Reviewed and approved by,

Antony J Martin

09/21/2020

Antony J. Martin Date
California Professional Geophysicist, P. Gp.
GEO*Vision* Geophysical Services

ANTONY JEFFREY MARTIN
GP 989

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

Appendix C

Electrical Resistivity Geophysical Survey

GEOVision
geophysical services

October 2, 2020

Project Number 20260

Peter Skopek, PhD
Tetra Tech BAS GeoScience
21700 Copley Drive #200
Diamond Bar, CA 91765
(909) 860 – 7777 x3235

Subject: **Four-Electrode Resistivity Survey**
Whitewater, California

Dr Skopek:

Geophysical surveys were conducted at two locations on August 26, 2020, and September 25, 2020 at the project site in Whitewater, California. The purpose of the geophysical survey was to map the subsurface electrical structure at the preselected locations, per ASTM Standard G 57. The site was located in an existing windmill farm in open desert terrain consisting of mostly loose, well bioturbated sand surface with abundant cobbles/boulders and sparse brush.

Resistivity data were collected at two preselected locations, ER-1 and ER-2. At each location two orthogonal profiles were deployed, one oriented approximately S-N, the other approximately W-E. Soundings at each location were concentric, the center point being the GPS point provided on Table 1. Tetra Tech preselected the approximate sounding locations which allowed the use of conventional a-spacings to a maximum of 150 ft. Sounding center and profile end point locations were obtained using a submeter GPS system with differential corrections (Table 1).

Table 1: Resistivity Soundings Location (Center point)

Location	Northing (Feet)	Easting (Feet)
ER-1	2275925.711	6448355.343
ER-2	22743687.946	6454135.762

Note: Coordinates in California State Plane, Zone VI (0406)
NAD83, in US Survey Feet.

EQUIPMENT AND METHODOLOGY

Resistivity equipment used during this investigation included an Advanced Geosciences, Inc. SuperSting R8 Resistivity/IP meter coupled to 3/8-inch stainless steel electrode stakes with 18-gauge insulated copper wire. A test resistor, rated at 4 ohms at 25 degrees Celsius, was used to verify that the SuperSting meter was operating within manufacturer specifications. The SuperSting is rated up to 200 W and is capable of continuous output current between 1 mA to 2,000 mA with an output voltage of 800 V peak to peak. The operator may select a maximum output current, which the instrument will automatically reduce as needed depending on soil conditions and ground impedance. The transmitter then maintains a steady current through the measurement cycle, recording input voltage and writing V/I to internal memory.

Resistivity data were acquired using a four-electrode array, specifically the Wenner Array. The generalized form of the four-electrode array is shown in Figure 1.

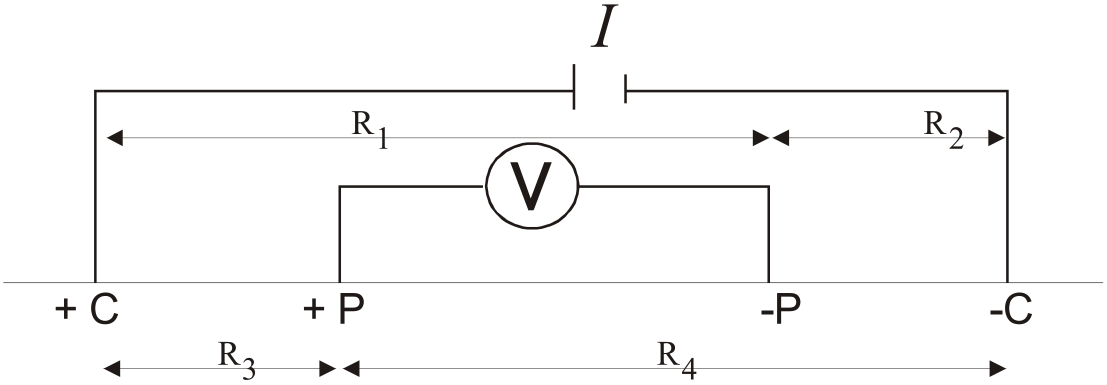


Figure 1: The generalized form of the four-electrode array (Wenner Array – R2=R3, R1=2R3, R4=2R2).

When the material upon which the current is induced is uniform, the resistivity calculated will be constant independent of electrode configuration. However, in a field investigation where subsurface heterogeneities exist, the calculated resistivity values will vary with electrode array. This calculated resistivity is referred to as apparent resistivity (ρ_a), and can be calculated using the relationship:

$$\rho_a = \frac{2\pi V}{I\left\{(\frac{1}{R_3} - \frac{1}{R_4}) - (\frac{1}{R_1} - \frac{1}{R_2})\right\}}$$

For the Wenner array, which was used during this investigation, where R1 = R4; R3 = R2 and R1 = 2R2 = $2a$, it can be shown that the formula for calculated apparent resistivity can be reduced to the following form:

$$\rho_a = 2\pi a\left(\frac{V}{I}\right).$$

FIELD PROCEDURES

Before conducting the geophysical survey, battery levels were checked on the resistivity meter and general site conditions were recorded on the field log.

A test resistor, rated at 4 ohms at 25 degrees Celsius, was connected to the positive and negative current and potential leads on the SuperSting meter immediately before each sounding. The resistance value across the test resistor, time of the test measurement, and ambient temperature were recorded on the field log.

Resistivity measurements were made using the Wenner Array (Figure 1). The Wenner Array uses a constant spacing between the four electrodes (a-spacing). Electrode locations were established using 300-ft fiberglass survey tapes. Measurements at typical electrode a-spacings between 3 to 150 ft were attempted along each profile. Typical a-spacing attempted along each profile were 3, 5, 7.5, 10, 15, 30, 50, 75, 100, and 150 ft. However, at some a-spacings data could not be collected due to very high contact resistance preventing electrical coupling between the electrodes and the subsurface. High contact resistance, at this site, was caused by the very dry, loose sand at the surface or rocks/boulders impeding electrode advancement. If after several attempts data could still not be acquired, the a-spacing was skipped and electrodes were moved to the next a-spacing, as summarized in the results section and associated tables.

For each resistivity measurement, four stainless steel electrodes were deployed along a linear array at the specified distances (a-spacing). A current was applied to the outer electrodes, and the potential difference (voltage) was measured across the inner electrodes. The SuperSting meter displayed the resistance value equal to V/I. This value was recorded, along with the a-spacing, on a field data sheet. At least two measurements were recorded at each station for quality control. If there was significant variation between the first and second measurements, the control leads, electrode cable and electrode coupling were field verified to ensure proper coupling then the measurement was repeated. After each measurement, the electrodes were moved to the next a-spacing and another set of measurements was taken.

DATA REDUCTION

Resistivity data were reduced using a spreadsheet. Electrode spacing (a-spacing) and resistance reading (V/I) were entered into the spreadsheet for each measurement and apparent resistivity was calculated using the aforementioned formula. Apparent resistivity values were also calculated for the repeat measurements and presented in units of ohm-feet, ohm-meters and ohm-centimeters.

RESULTS

Resistivity data for location ER-1 are presented in Table 2 (S-N) and Table 3 (W-E) and for location ER-2 in Table 4 (S-N) and Table 5 (W-E). All calculations were conducted using known geometry and measured resistance (V/I) values which were recorded in the field logs. Apparent resistivity values are presented in ohm-ft, ohm-m and ohm-cm. The ASTM Standard G57 specifies that apparent resistivity be presented in ohm-cm. All completed data processing forms are retained in project files.

Site conditions were not ideal for resistivity measurements. In addition, ambient temperature exceeded 100F degree by late morning. The very dry, loose sand and boulders/cobbles made it very difficult or impossible to inject current and complete the electrical circuit to take measurements, i.e., ground contract resistance was very high. In fact, data could not be acquired at a-spacing less than 3 ft on the ER-1 W-E profile nor at less than 5 ft on the ER-1 S-N profile and neither ER-2 profiles. In order to complete the circuit and successfully inject current, an approximately 1 ft deep divot was dug and filled with saline solution before planting each electrode. Even so, at some locations, e.g., at 100 ft on ER-1 S-N profile and 30 ft on ER-2 W-E profile, rocks and boulders at or right below the surface impeded electrode advancement or digging, thus data could not be collected at that spacing. For the ER-1 W-E profile, data could not be acquired beyond 50ft a-spacing, similarly along ER-2 W-E. Notably ER-2 is near existing windmills. The west end of ER-2 W-E profile crossed over a buried electrical line between existing windmills. Also ER-2 S-N crossed a buried gas line at the southern end of the profile (Figure 1).

As presented in Tables 2 through 5, successful measurements were mostly consistent and repeatable. At larger a-spacings lower signal to noise ratio was evident as slight divergence of repeat measurements, e.g., ER-1 S-N 150ft.

Calculated resistivity values for the each soundings pair, were fairly similar, exhibiting very high near surface resistivity, decreasing slightly to at least 50ft. On the longer S-N profiles, data reveal an increase below 50ft. Overall, resistivity values were relatively high, ranging from greater than 1,000 ohm-m up to approximately 7,500 ohm-m. The difference between the W-E and S-N profiles could be due to electrical anisotropy in the perpendicular orientations caused by variability in subsurface geology.

The extended effort to acquire data at location ER-1 took a full field day and thus the second proposed location, ER-2, could not be occupied on the same day. GEOVision remobilized to acquire data at ER-2 several weeks later. Site conditions were similar at both locations.

SUMMARY

Soil resistivity soundings were made at a client specified location, ER-1 and ER-2, in Whitewater, California. The soundings were acquired in accordance with ASTM standard G57, as much as possible. Some acquisition modifications were necessary in order to collect data in the desert terrain. Electrical resistivity data were acquired at up to eight electrode spacings for each sounding. Successful field measurements and calculated values were reasonably consistent and repeatable, as summarized in Tables 2 through 5.

If you have any questions concerning this investigation, please call us at 951-549-1234.

Sincerely,

GEO*Vision* Geophysical Services

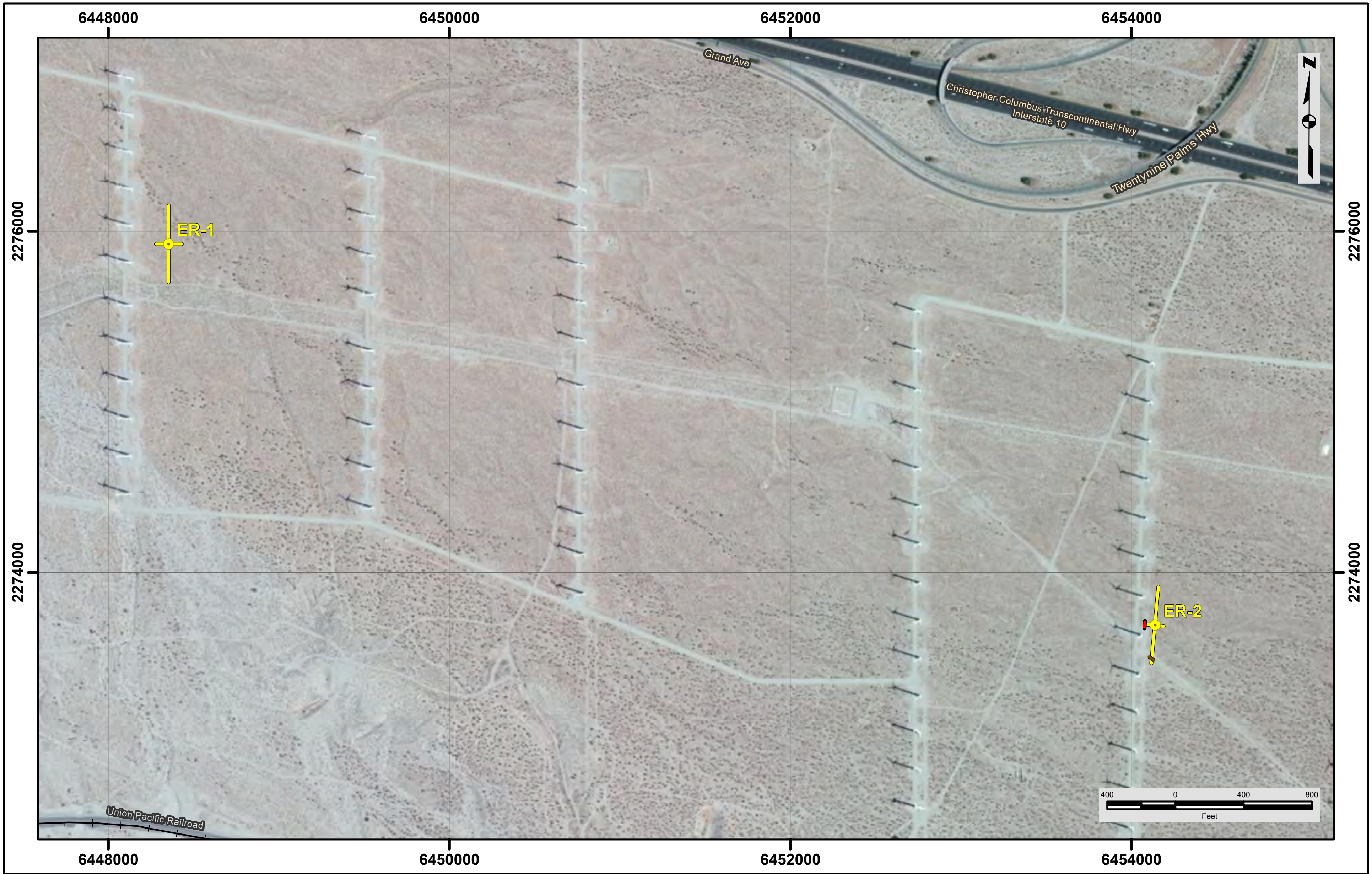
PROFESSIONAL GEOPHYSICIST
VICTOR GONZALEZ
PGp 1074
STATE OF CALIFORNIA

Submitted by:
Christopher Martinez
Staff Geophysicist

Reviewed and Approved by:
Victor Gonzalez
Senior Geophysicist, PG, PGp

Attachments:

Table 2: Resistivity Sounding ER-1 S-N
Table 3: Resistivity Sounding ER-1 W-E
Table 2: Resistivity Sounding ER-2 S-N
Table 3: Resistivity Sounding ER-2 W-E



- Electrical Resistivity Sounding
- High Voltage Electric Line
- High Pressure Gas Line

NOTES:

1. Coordinate System: California State Plane, NAD83, Zone VI (0406), US Survey Feet
2. Base map source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

GEOVision geophysical services

Date:	10/1/2020
GV Project:	20260
Developed by:	C Martinez
Drawn by:	T Rodriguez
Approved by:	V Gonzalez
File Name:	20260-1.MXD

FIGURE 1
SITE MAP

SITE LOCATED AT
WINDMILL FARM
WHITEWATER, CALIFORNIA

PREPARED FOR
TETRA TECH BAS GEOSCIENCE

TABLE 2: RESISTIVITY SOUNDING ER-1 S-N

Job Number: 20260 Date: 8/26/2020 Temperature: Approx 72 ^{o}F

4 ohm Test Resistor Reading Rated at 83 $_{o}$F (25 $_{o}$C)

Test Resistance	Repeat	Time
3.937 ohm	3.937 ohm	08:22

A-Spacing [ft]	Geometric Multiplier [2(pi)A]	Resistance Measurement [Ohm]	Apparent Resistivity [Ohm-ft]	Apparent Resistivity [Ohm-m]	Apparent Resistivity [Ohm-cm]	Repeat Resistance Measurement [Ohm]	Repeat Apparent Resistivity [Ohm-ft]	Repeat Apparent Resistivity [Ohm-m]	Repeat Apparent Resistivity [Ohm-cm]
5.0	31.4	482.30	15151.9	4618.3	461830.0	481.20	15117.3	4607.8	460776.6
7.5	47.1	170.50	8034.6	2449.0	244895.3	169.60	7992.2	2436.0	243602.6
10.0	62.8	117.60	7389.0	2252.2	225217.5	124.50	7822.6	2384.3	238431.8
15.0	94.2	108.20	10197.6	3108.2	310823.1	108.20	10197.6	3108.2	310823.1
30.0	188.5	34.72	6544.6	1994.8	199478.4	35.12	6620.0	2017.8	201776.5
50.0	314.2	30.38	9544.2	2909.1	290906.0	29.66	9318.0	2840.1	284011.5
75.0	471.2	43.26	20385.8	6213.6	621359.0	42.33	19947.5	6080.0	608001.1
150.0	942.5	5.71	5384.4	1641.2	164115.8	6.66	6272.2	1911.8	191176.3

TABLE 3: RESISTIVITY SOUNDING ER-1 W-E

Job Number: 20260 Date: 8/26/2020 Temperature: Approx 102 ^{o}F

4 ohm Test Resistor Reading Rated at 83 $_{o}$F (25 $_{o}$C)

Test Resistance	Repeat	Time
3.935 ohm	3.938 ohm	10:40

A-Spacing [ft]	Geometric Multiplier [2(pi)A]	Resistance Reading [Ohm]	Apparent Resistivity [Ohm-ft]	Apparent Resistivity [Ohm-m]	Apparent Resistivity [Ohm-cm]	Repeat Resistance Reading [Ohm]	Repeat Apparent Resistivity [Ohm-ft]	Repeat Apparent Resistivity [Ohm-m]	Repeat Apparent Resistivity [Ohm-cm]
3.0	18.8	1365.00	25729.6	7842.4	784239.5	1359.00	25616.5	7807.9	780792.3
5.0	31.4	406.60	12773.7	3893.4	389342.9	405.60	12742.3	3883.9	388385.3
7.5	47.1	351.90	16582.9	5054.5	505446.7	353.50	16658.3	5077.4	507744.8
10.0	62.8	247.50	15550.9	4739.9	473990.9	248.40	15607.4	4757.1	475714.5
15.0	94.2	107.20	10103.4	3079.5	307950.5	108.70	10244.7	3122.6	312259.5
30.0	188.5	20.42	3849.1	1173.2	117319.9	23.21	4375.0	1333.5	133349.4
50.0	314.2	10.64	3342.7	1018.8	101884.1	6.18	1941.8	591.9	59186.6

TABLE 4: RESISTIVITY SOUNDING ER-2 S-N

Job Number: 20260 Date: 9/25/2020 Temperature: Approx 78 ^{o}F

4 ohm Test Resistor Reading Rated at 83 $_{o}$F (25 $_{o}$C)

Test Resistance	Repeat	Time
3.933	3.934	07:09

A-Spacing [ft]	Geometric Multiplier [2(pi)A]	Resistance Reading [Ohm]	Apparent Resistivity [Ohm-ft]	Apparent Resistivity [Ohm-m]	Apparent Resistivity [Ohm-cm]	Repeat Resistance Reading [Ohm]	Repeat Apparent Resistivity [Ohm-ft]	Repeat Apparent Resistivity [Ohm-m]	Repeat Apparent Resistivity [Ohm-cm]
5.0	31.4	487.50	15315.3	4668.1	466809.3	491.60	15444.1	4707.4	470735.2
7.5	47.1	457.60	21563.9	6572.7	657267.4	455.20	21450.8	6538.2	653820.2
10.0	62.8	153.90	9669.8	2947.4	294736.2	153.30	9632.1	2935.9	293587.1
30.0	188.5	32.35	6097.8	1858.6	185861.9	32.40	6107.3	1861.5	186149.2
50.0	314.2	16.33	5130.2	1563.7	156369.1	16.19	5086.2	1550.3	155028.5
75.0	471.2	12.84	6050.7	1844.3	184425.6	12.88	6069.6	1850.0	185000.1
100.0	628.3	7.77	4884.5	1488.8	148881.0	7.86	4939.8	1505.7	150566.3
150.0	942.5	3.76	3542.8	1079.8	107983.8	3.73	3510.7	1070.1	107007.0

TABLE 5: RESISTIVITY SOUNDING ER-2 W-E

Job Number: 20260 Date: 9/25/2020 Temperature: Approx 87 °F

4 ohm Test Resistor Reading Rated at 83 °F (25 °C)

Test Resistance	Repeat	Time
3.936	3.936	09:25

A-Spacing [ft]	Geometric Multiplier [2(pi)A]	Resistance Reading [Ohm]	Apparent Resistivity [Ohm-ft]	Apparent Resistivity [Ohm-m]	Apparent Resistivity [Ohm-cm]	Repeat Resistance Reading [Ohm]	Repeat Apparent Resistivity [Ohm-ft]	Repeat Apparent Resistivity [Ohm-m]	Repeat Apparent Resistivity [Ohm-cm]
5.0	31.4	211.70	6650.8	2027.1	202714.9	213.00	6691.6	2039.6	203959.7
7.5	47.1	126.40	5956.5	1815.5	181552.9	126.40	5956.5	1815.5	181552.9
10.0	62.8	116.90	7345.0	2238.8	223876.9	116.80	7338.8	2236.9	223685.4
50.0	314.2	15.08	4737.5	1444.0	144399.7	15.09	4740.7	1445.0	144495.4

Appendix D

Laboratory Testing

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MOISTURE CONTENT AND DENSITY

ASTM D2937

Job Name:	Mountain View I&II Wind Turbine Repowering	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	10/2/2020
Tested By:	MG	**Note:**	Page 1 of 1

Boring / Test Pit / Trench		**B-4**	**B-11**							
Sample Number		**R-7**	**R-9**							
Sample Depth	*feet*	**25-26.5**	**40-41.5**							
USCS Soil Description		Gray Native SP	Gray Native SP							
Number of Rings		5	6							
Total Weight Rings + Soil	*grams*	891.90	1001.40							
* Volume of Rings	*ft^3*	0.0133	0.0159							
* Weight of Rings	*grams*	214.00	256.80							
* Weight of Soil	*grams*	677.90	744.60							
* Wet Density	*pcf*	112.48	102.95							
	Container ID	P27	P20							
Tare	*grams*	9.3	8.8							
Wet Soil + Tare	*grams*	435.8	287.8							
Dry Soil + Tare	*grams*	416.8	282.6							
* Weight of Water	*grams*	19	5.2							

*** Dry Density**	***pcf***	**107.5**	**101.0**							
*** Moisture Content**	***%***	**4.7**	**1.9**							

TETRA TECH BAS
GEOSCIENCE

MOISTURE CONTENT AND DENSITY

ASTM D2937

Job Name:	Mountain View I&II Wind Turbine Repowering	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	9/7/2020
Tested By:	MG	**Note:**	Page 1 of 3

Boring / Test Pit / Trench		**B-12**	**B-13**		**B-13**	**B-13**	**B-13**	**B-14**	**B-14**	**B-14**
Sample Number		**R-2**	**R-3**		**R-7**	**R-9**	**R-11**	**R-4**	**R-6**	**R-10**
Sample Depth	*feet*	**2.5-4**	**10-11.5**		**22.5-24**	**30-31.5**	**40-41.5**	**7.5-9**	**12.5-14**	**30-31.5**
USCS Soil Description		Olive Gray Native SP	Olive Gray Native SP		Olive Gray Native SP	Olive Gray Native SP	Olive Gray Native SP	Olive Gray Native SP	Olive Gray Native SP	Olive Gray Native SP
Number of Rings		3	6		6	5	5	6	6	6
Total Weight Rings + Soil	*grams*	556.40	1115.70		1148.00	927.90	925.60	1182.30	1112.30	1125.90
* Volume of Rings	*ft^3*	0.0080	0.0159		0.0159	0.0133	0.0133	0.0159	0.0159	0.0159
* Weight of Rings	*grams*	134.76	269.52		269.52	224.60	224.60	269.52	269.52	269.52
* Weight of Soil	*grams*	421.64	846.18		878.48	703.30	701.00	912.78	842.78	856.38
* Wet Density	*pcf*	116.60	117.00		121.46	116.69	116.31	126.21	116.53	118.41
Container ID		X55	X22		X4	X25	X18	X48	X35	X42
Tare	*grams*	10.5	10.5		10.5	10.5	10.5	10.5	10.5	10.5
Wet Soil + Tare	*grams*	207.5	270.4		294.9	275.1	270.2	364	302.5	270.9
Dry Soil + Tare	*grams*	205.3	267		290.4	269.4	263.5	357.3	293.6	265.3
* Weight of Water	*grams*	2.2	3.4		4.5	5.7	6.7	6.7	8.9	5.6

*** Dry Density**	***pcf***	**115.3**	**115.5**		**119.5**	**114.2**	**113.3**	**123.8**	**113.0**	**115.9**
*** Moisture Content**	***%***	**1.1**	**1.3**		**1.6**	**2.2**	**2.6**	**1.9**	**3.1**	**2.2**

TETRA TECH BAS
GEOSCIENCE

MOISTURE CONTENT AND DENSITY

ASTM D2937

Job Name:	Mountain View I&II Wind Turbine Repowering	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	9/7/2020
Tested By:	MG	**Note:**	Page 2 of 3

Boring / Test Pit / Trench		**B-15**	**B-15**		**B-16**	**B-94**		**B-95**	**B-96**	**B-97**
Sample Number		**R-3**	**R-7**		**R-14**	**R-6**		**R-5**	**R-6**	**R-3**
Sample Depth	*feet*	**10-11.5**	**22.5-24**		**45-46.5**	**15-16.5**		**10-11.5**	**15-16.5**	**5-6.5**
USCS Soil Description		Olive Gray Native SP	Olive Gray Native SP		Olive Gray Native SP	Olive Gray Native SP		Olive Gray Native SP	Olive Gray Native SP	Olive Gray Native SP
Number of Rings		6	6		6	6		6	6	6
Total Weight Rings + Soil	*grams*	1202.50	1115.30		1080.50	1096.80		1190.50	1192.70	1086.50
* Volume of Rings	*ft^3*	0.0159	0.0159		0.0159	0.0159		0.0159	0.0159	0.0159
* Weight of Rings	*grams*	269.52	269.52		269.52	269.52		269.52	269.52	269.52
* Weight of Soil	*grams*	932.98	845.78		810.98	827.28		920.98	923.18	816.98
* Wet Density	*pcf*	129.00	116.94		112.13	114.39		127.34	127.64	112.96
Container ID		X33	X8		X56	X10		X13	X46	X44
Tare	*grams*	10.5	10.5		10.5	10.5		10.5	10.5	10.5
Wet Soil + Tare	*grams*	340.3	333.9		341.3	274		334.6	341.2	260
Dry Soil + Tare	*grams*	335.2	329.2		339.3	268		333.2	336.2	255.4
* Weight of Water	*grams*	5.1	4.7		2	6		1.4	5	4.6

*** Dry Density**	***pcf***	**127.0**	**115.2**		**111.5**	**111.8**		**126.8**	**125.7**	**110.9**
*** Moisture Content**	***%***	**1.6**	**1.5**		**0.6**	**2.3**		**0.4**	**1.5**	**1.9**

TETRA TECH BAS
GEOSCIENCE

MOISTURE CONTENT AND DENSITY

ASTM D2937

Job Name:	Mountain View I&II Wind Turbine Repowering	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	9/7/2020
Tested By:	MG	**Note:**	Page 3 of 3

Boring / Test Pit / Trench			**B-98**							
Sample Number			**R-4**							
Sample Depth	*feet*		**7.5-9**							
USCS Soil Description			Olive Gray Native SP							
Number of Rings			3							
Total Weight Rings + Soil	*grams*		572.80							
* Volume of Rings	*ft ^{3}*		0.0080							
* Weight of Rings	*grams*		134.76							
* Weight of Soil	*grams*		438.04							
* Wet Density	*pcf*		121.13							
Container ID			X47							
Tare	*grams*		10.5							
Wet Soil + Tare	*grams*		263.2							
Dry Soil + Tare	*grams*		259.6							
* Weight of Water	*grams*		3.6							

*** Dry Density**	***pcf***		**119.4**							
*** Moisture Content**	***%***		**1.4**							

TETRA TECH BAS
GEOSCIENCE

MODIFIED PROCTOR

Maximum Dry Density / Optimum Moisture Content

ASTM D1557

Job Name:	Mountain View I &II Wind	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	9/29/2020
Tested By:	MG	**Sample Identification:**	B-3, SK-1
Note:		**Sample Depth:**	0-5ft
Sample Description:	Native Olive Gray Poorly Graded GRAVEL with Sand, GP		

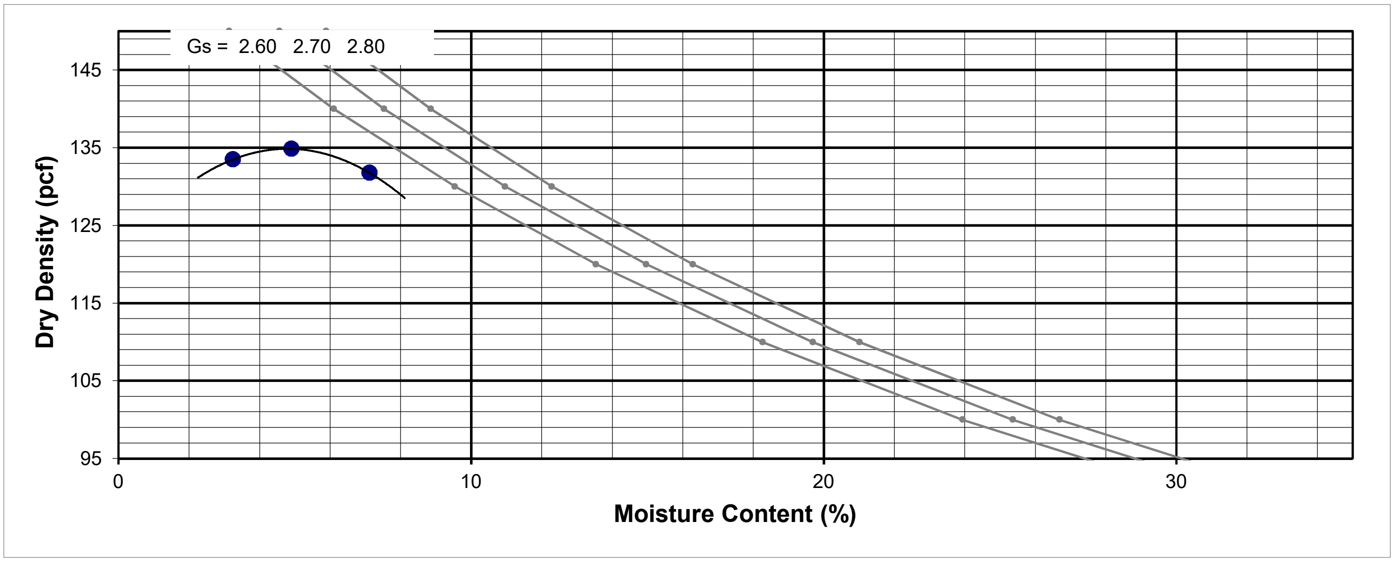
Trial Number		1	2	3	4	5
Water Added	*ml*	100	200	300		
Weight of Soil + Mold	*grams*	7473.8	7597.4	7585.9		
Weight of Mold	*grams*	2784.6	2784.6	2784.6		
Weight of Wet Soil	*grams*	4689.2	4812.8	4801.3		
Wet Density	*pcf*	137.84	141.47	141.13		
Container ID		X4	O1	X6		
Wet Soil + Tare	*grams*	439.2	540.6	504	0	
Dry Soil + Tare	*grams*	425.7	515.8	471.2	0	
Tare	*grams*	10.5	10.5	10.5	0	
Dry Soil	*grams*	415.2	505.3	460.7		
Water	*grams*	13.5	24.8	32.8		
Moisture Content	***%***	**3.25**	**4.91**	**7.12**		
Dry Density	***pcf***	**133.5**	**134.9**	**131.8**		

Compaction Method

Method A, B, or C	C
% retained 3/4	20%
Mold Size	6
Mold Volume	0.075

Preparation Method

Moist	X
Dry	



Maximum Dry Density	**135.0**	pcf	**Optimum Moisture Content**	**5.0**	%
w/ Rock Correction	**139.7**	pcf	w/ Rock Correction	**4.0**	%

TETRA TECH BAS
GEOSCIENCE

MODIFIED PROCTOR

Maximum Dry Density / Optimum Moisture Content

ASTM D1557

Job Name:	Mountain View I &II Wind	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	9/16/2020
Tested By:	MG	**Sample Identification:**	B-5
Note:		**Sample Depth:**	0-5ft
Sample Description:	Native Olive Gray Poorly Graded GRAVEL with Sand, GP		

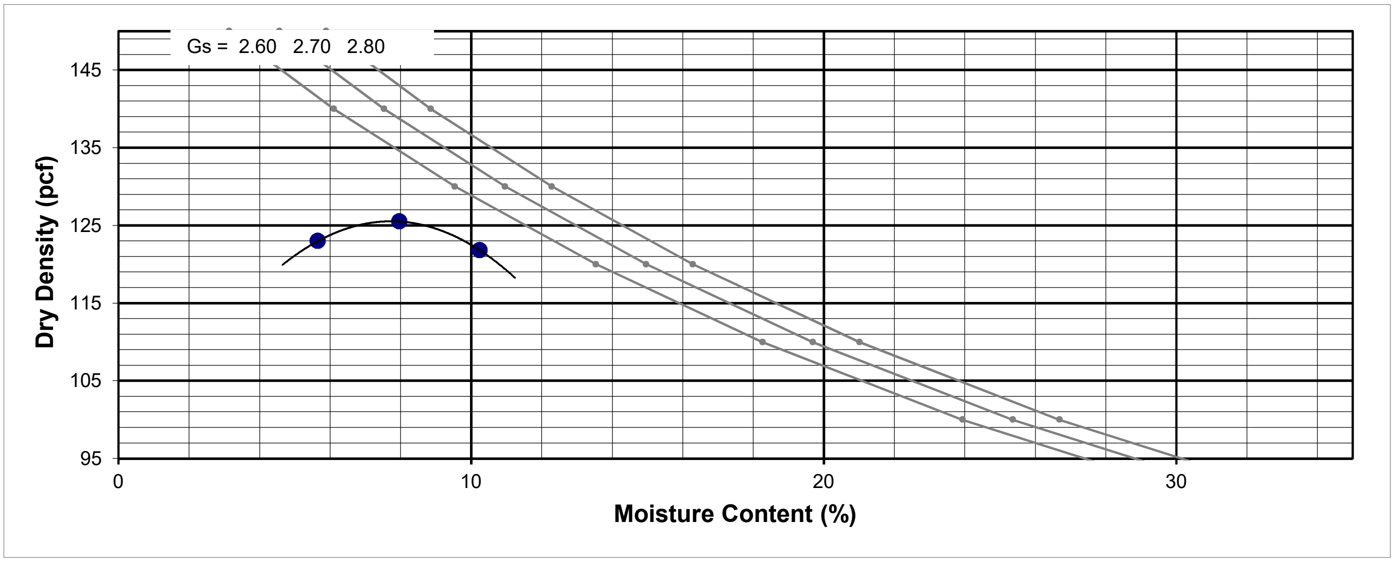
Trial Number		1	2	3	4	5
Water Added	*ml*	100	150	200		
Weight of Soil + Mold	*grams*	3997.6	4081.7	4063.4		
Weight of Mold	*grams*	2033	2033	2033		
Weight of Wet Soil	*grams*	1964.6	2048.7	2030.4		
Wet Density	*pcf*	129.94	135.50	134.29		
Container ID		X34	X52	X21		
Wet Soil + Tare	*grams*	281.5	319.4	311.9	0	
Dry Soil + Tare	*grams*	267	296.6	283.9	0	
Tare	*grams*	10.5	10.5	10.5	0	
Dry Soil	*grams*	256.5	286.1	273.4		
Water	*grams*	14.5	22.8	28		
Moisture Content	***%***	**5.65**	**7.97**	**10.24**		
Dry Density	***pcf***	**123.0**	**125.5**	**121.8**		

Compaction Method

Method A, B, or C	B
%retained in 3/8	14%
Mold Size	4
Mold Volume	0.033333

Preparation Method

Moist	X
Dry	



Maximum Dry Density	**125.5**	pcf	**Optimum Moisture Content**	**8.0**	%
w/ Rock Correction	**129.6**	pcf	w/ Rock Correction	**7.0**	%

MODIFIED PROCTOR

Maximum Dry Density / Optimum Moisture Content

ASTM D1557

Job Name:	Mountain View I &II Wind	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	9/9/2020
Tested By:	MG	**Sample Identification:**	B-12
Note:		**Sample Depth:**	0-5ft
Sample Description:	Native Sandy Gravel Well Graded SAND with Gravel		

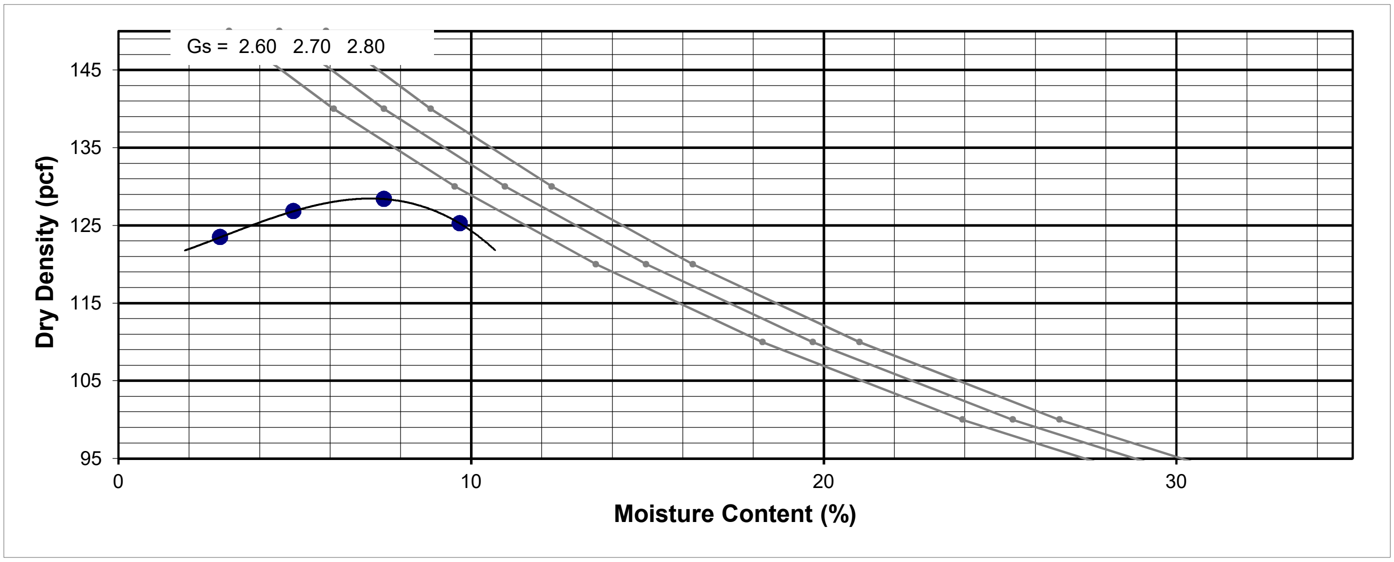
Trial Number		1	2	3	4	5
Water Added	*ml*	50	100	150	200	
Weight of Soil + Mold	*grams*	3953.9	4045.7	4120.5	4110.2	
Weight of Mold	*grams*	2033	2033	2033	2033	
Weight of Wet Soil	*grams*	1920.9	2012.7	2087.5	2077.2	
Wet Density	*pcf*	127.05	133.12	138.06	137.38	
Container ID		Z19	Z29	Z24	Z36	
Wet Soil + Tare	*grams*	271.4	300.2	282.4	296.4	
Dry Soil + Tare	*grams*	263.9	286.2	262.9	270.6	
Tare	*grams*	4	4	4	4	
Dry Soil	*grams*	259.9	282.2	258.9	266.6	
Water	*grams*	7.5	14	19.5	25.8	
Moisture Content	*%*	**2.89**	**4.96**	**7.53**	**9.68**	
Dry Density	***pcf***	**123.5**	**126.8**	**128.4**	**125.3**	

Compaction Method

Method A, B, or C	B
%retained in 3/8	20%
Mold Size	4
Mold Volume	0.033333

Preparation Method

Moist	X
Dry	



Maximum Dry Density	**128.5**	pcf	**Optimum Moisture Content**	**7.5**	%
w/ Rock Correction	**134.0**	pcf	w/ Rock Correction	**7.0**	%

TETRA TECH BAS
GEOSCIENCE

MODIFIED PROCTOR

Maximum Dry Density / Optimum Moisture Content

ASTM D1557

Job Name:	Mountain View I &II Wind	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	9/16/2020
Tested By:	MG	**Sample Identification:**	B-94, SK-1
Note:		**Sample Depth:**	0-5ft
Sample Description:	Native Sandy Gravel Well Graded SAND with Gravel		

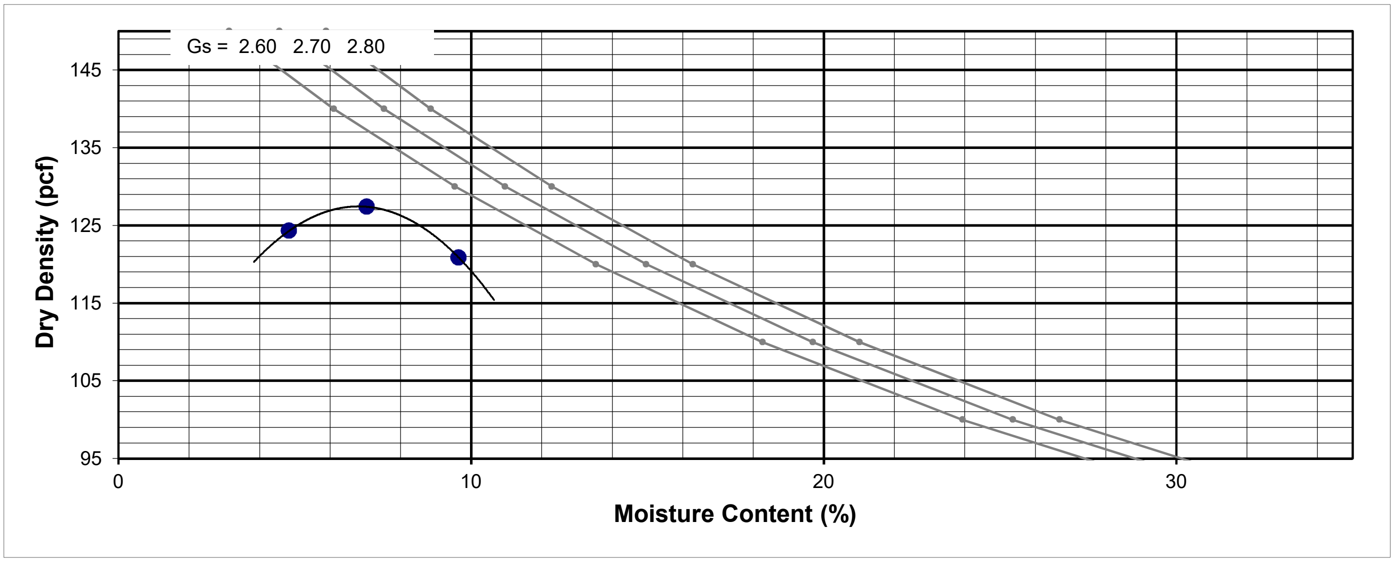
Trial Number		1	2	3	4	5
Water Added	*ml*	100	150	200		
Weight of Soil + Mold	*grams*	4003.5	4094.9	4036.7		
Weight of Mold	*grams*	2033	2033	2033		
Weight of Wet Soil	*grams*	1970.5	2061.9	2003.7		
Wet Density	*pcf*	130.33	136.37	132.52		
Container ID		X25	X16	X17		
Wet Soil + Tare	*grams*	285.6	258.2	302.6	0	
Dry Soil + Tare	*grams*	272.9	241.9	276.9	0	
Tare	*grams*	10.5	10.5	10.5	0	
Dry Soil	*grams*	262.4	231.4	266.4		
Water	*grams*	12.7	16.3	25.7		
Moisture Content	***%***	**4.84**	**7.04**	**9.65**		
Dry Density	***pcf***	**124.3**	**127.4**	**120.9**		

Compaction Method

Method A, B, or C	B
%retained in 3/8	15%
Mold Size	4
Mold Volume	0.033333

Preparation Method

Moist	X
Dry	



Maximum Dry Density	**127.5**	pcf	**Optimum Moisture Content**	**7.0**	%
w/ Rock Correction	**131.6**	pcf	w/ Rock Correction	**6.0**	%

TETRA TECH BAS
GEOSCIENCE

MODIFIED PROCTOR

Maximum Dry Density / Optimum Moisture Content

ASTM D1557

Job Name:	Mountain View I &II Wind	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	9/16/2020
Tested By:	MG	**Sample Identification:**	B-95, SK-1
Note:		**Sample Depth:**	0-5ft
Sample Description:	Native Sandy Gravel Well Graded SAND with Gravel		

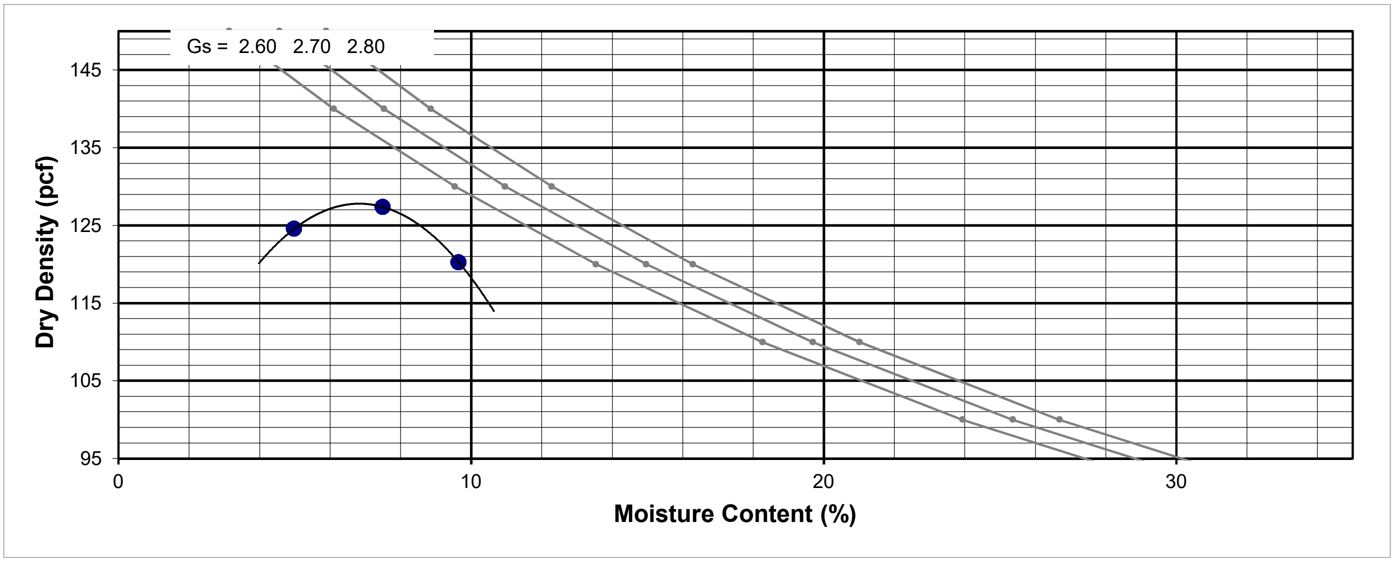
Trial Number		1	2	3	4	5
Water Added	*ml*	100	150	200		
Weight of Soil + Mold	*grams*	4010.2	4103.2	4026.7		
Weight of Mold	*grams*	2033	2033	2033		
Weight of Wet Soil	*grams*	1977.2	2070.2	1993.7		
Wet Density	*pcf*	130.77	136.92	131.86		
Container ID		X16	X40	X26		
Wet Soil + Tare	*grams*	261.3	241.5	302.6	0	
Dry Soil + Tare	*grams*	249.4	225.4	276.9	0	
Tare	*grams*	10.5	10.5	10.5	0	
Dry Soil	*grams*	238.9	214.9	266.4		
Water	*grams*	11.9	16.1	25.7		
Moisture Content	*%*	**4.98**	**7.49**	**9.65**		
Dry Density	*pcf*	**124.6**	**127.4**	**120.3**		

Compaction Method

Method A, B, or C	B
%retained in 3/8	18%
Mold Size	4
Mold Volume	0.033333

Preparation Method

Moist	X
Dry	



Maximum Dry Density	**127.5**	pcf	**Optimum Moisture Content**	**7.5**	%
w/ Rock Correction	**132.6**	pcf	w/ Rock Correction	**6.0**	%

TETRA TECH BAS
GEOSCIENCE

MODIFIED PROCTOR

Maximum Dry Density / Optimum Moisture Content

ASTM D1557

Job Name:	Mountain View I &II Wind	**Date Sampled:**	9/2/2020
Job Number:	GEN-20-33	**Date Completed:**	9/16/2020
Tested By:	MG	**Sample Identification:**	B-97, SK-1
Note:		**Sample Depth:**	0-5ft
Sample Description:	Native Sandy Gravel Well Graded SAND with Gravel		

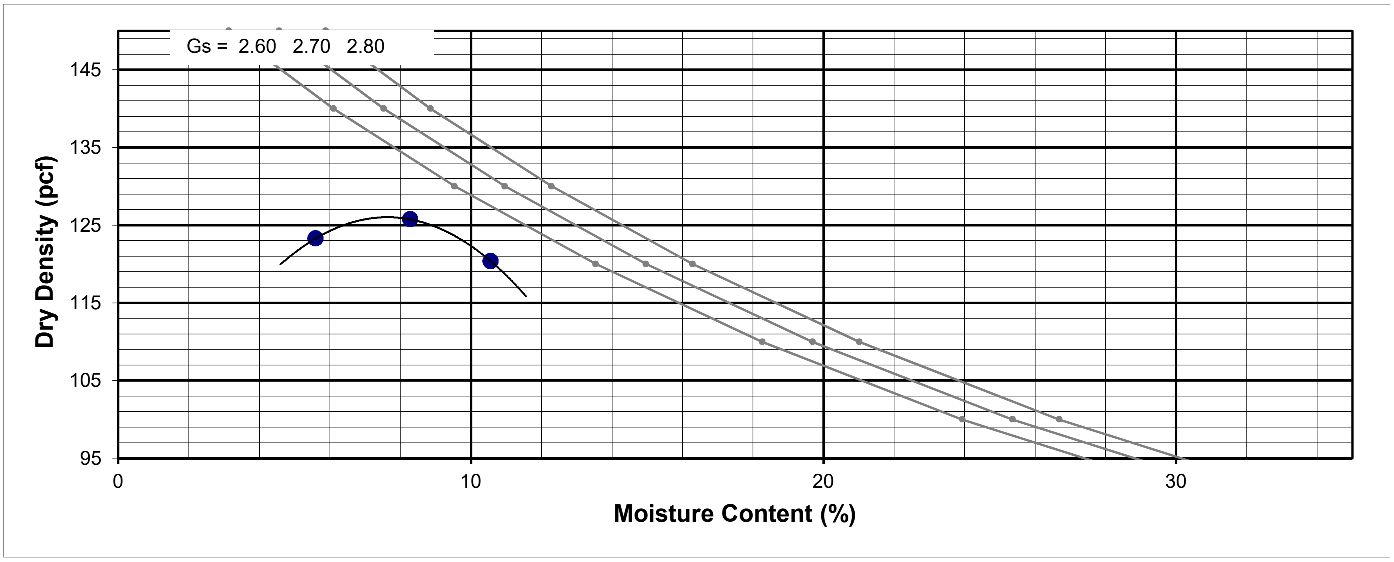
Trial Number		1	2	3	4	5
Water Added	*ml*	100	150	200		
Weight of Soil + Mold	*grams*	4001.6	4091.7	4045.2		
Weight of Mold	*grams*	2033	2033	2033		
Weight of Wet Soil	*grams*	1968.6	2058.7	2012.2		
Wet Density	*pcf*	130.20	136.16	133.08		
Container ID		X5	X26	X8		
Wet Soil + Tare	*grams*	291.5	264	298.4	0	
Dry Soil + Tare	*grams*	276.6	244.6	270.9	0	
Tare	*grams*	10.5	10.5	10.5	0	
Dry Soil	*grams*	266.1	234.1	260.4		
Water	*grams*	14.9	19.4	27.5		
Moisture Content	***%***	**5.60**	**8.29**	**10.56**		
Dry Density	***pcf***	**123.3**	**125.7**	**120.4**		

Compaction Method

Method A, B, or C	B
%retained in 3/8	7%
Mold Size	4
Mold Volume	0.033333

Preparation Method

Moist	X
Dry	



Maximum Dry Density	**125.7**	pcf	**Optimum Moisture Content**	**8.3**	%
w/ Rock Correction	**127.7**	pcf	w/ Rock Correction	**8.0**	%

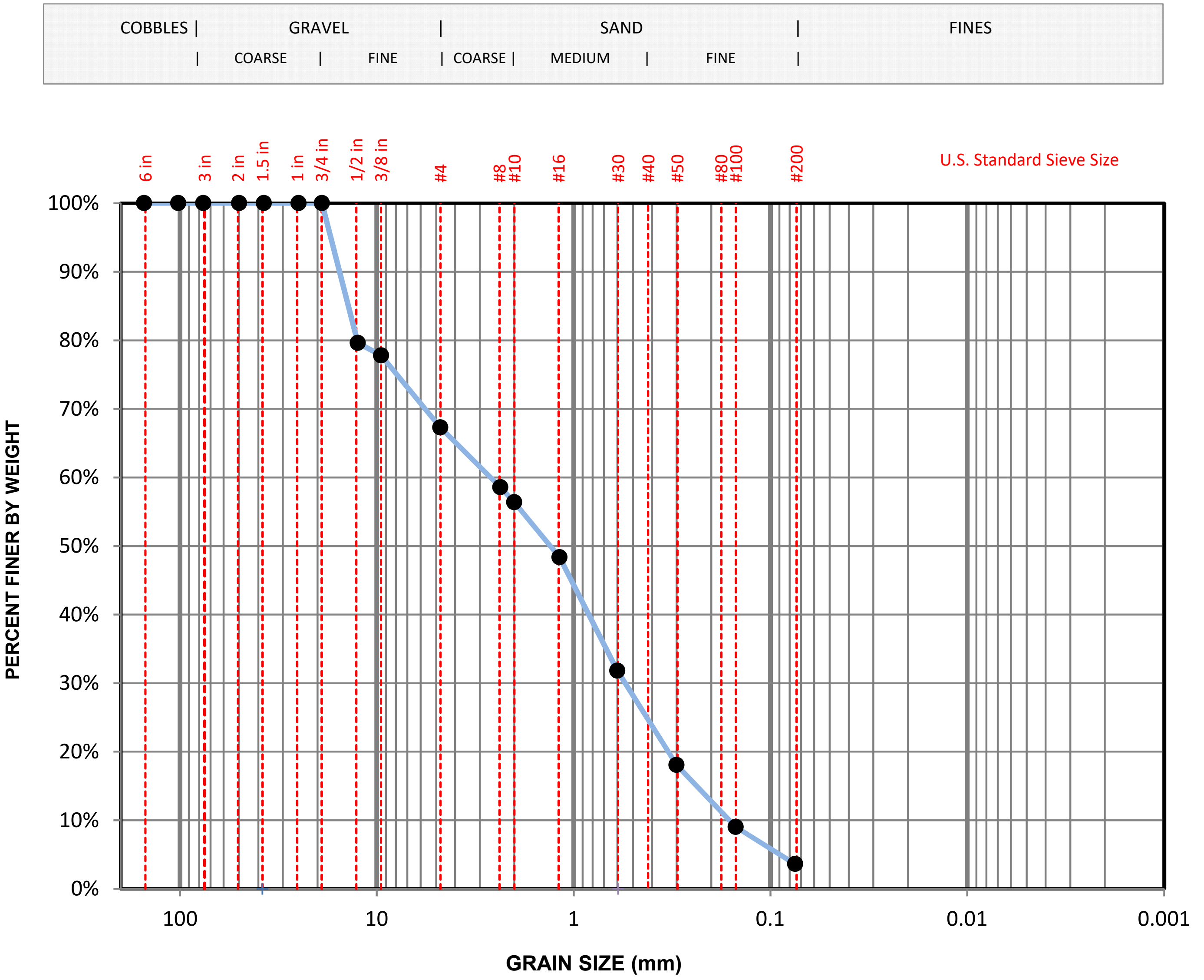
TETRA TECH BAS
GEOSCIENCE

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

2020-08

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	9/11/2020
Note:			
Sample Description:	Olive Gray Poorly Graded SAND with Gravel, SP		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-2	SPT-1	5-6.5	-	-	SP	**0%**	**32.7%**	**63.7%**	**3.6%**	**0%**

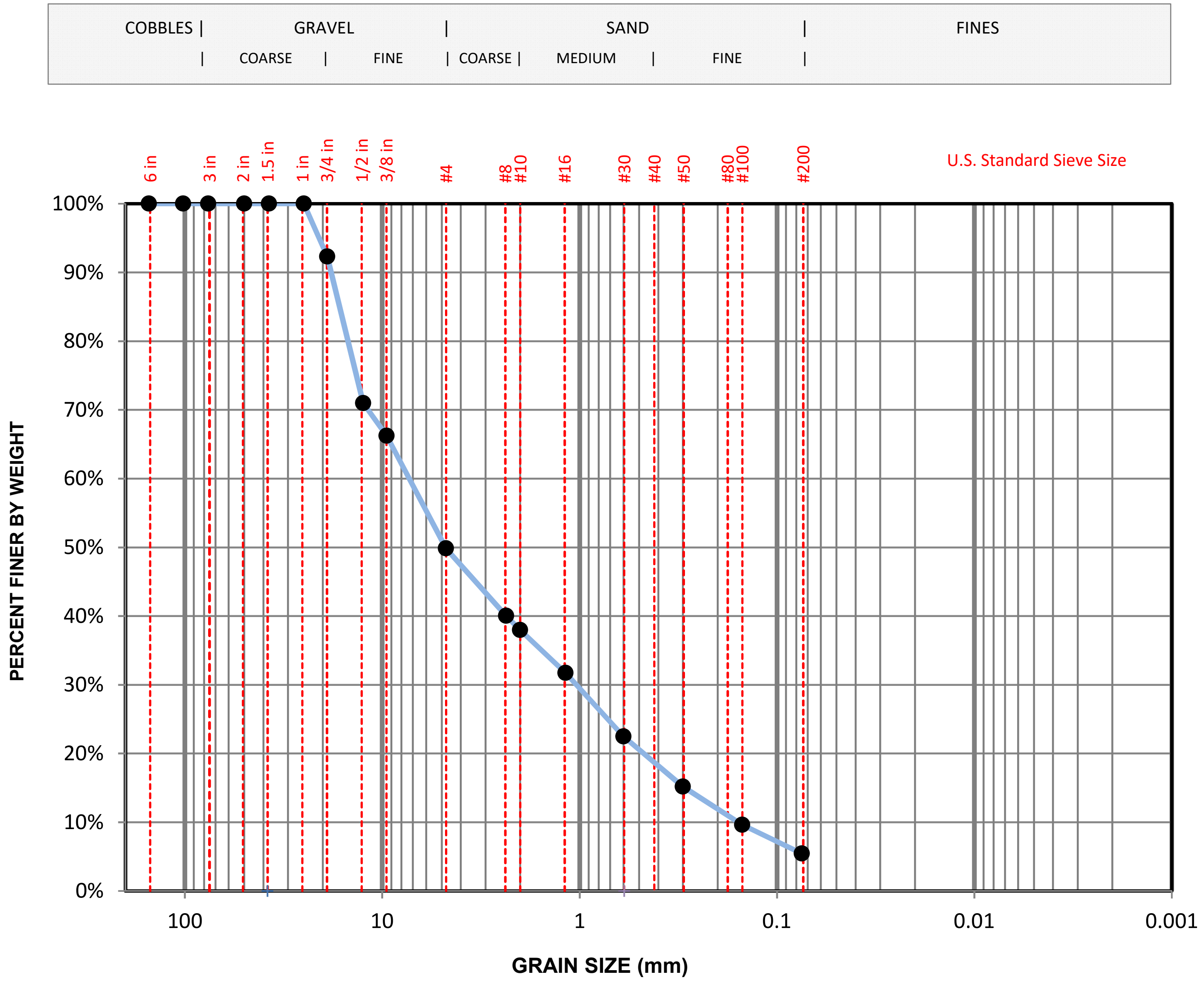
TETRA TECH BAS
GEOSCIENCE

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

2020-08

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	9/11/2020
Note:			
Sample Description:	Olive Gray Poorly Graded GRAVEL with Silt and Sand, GP-GM		



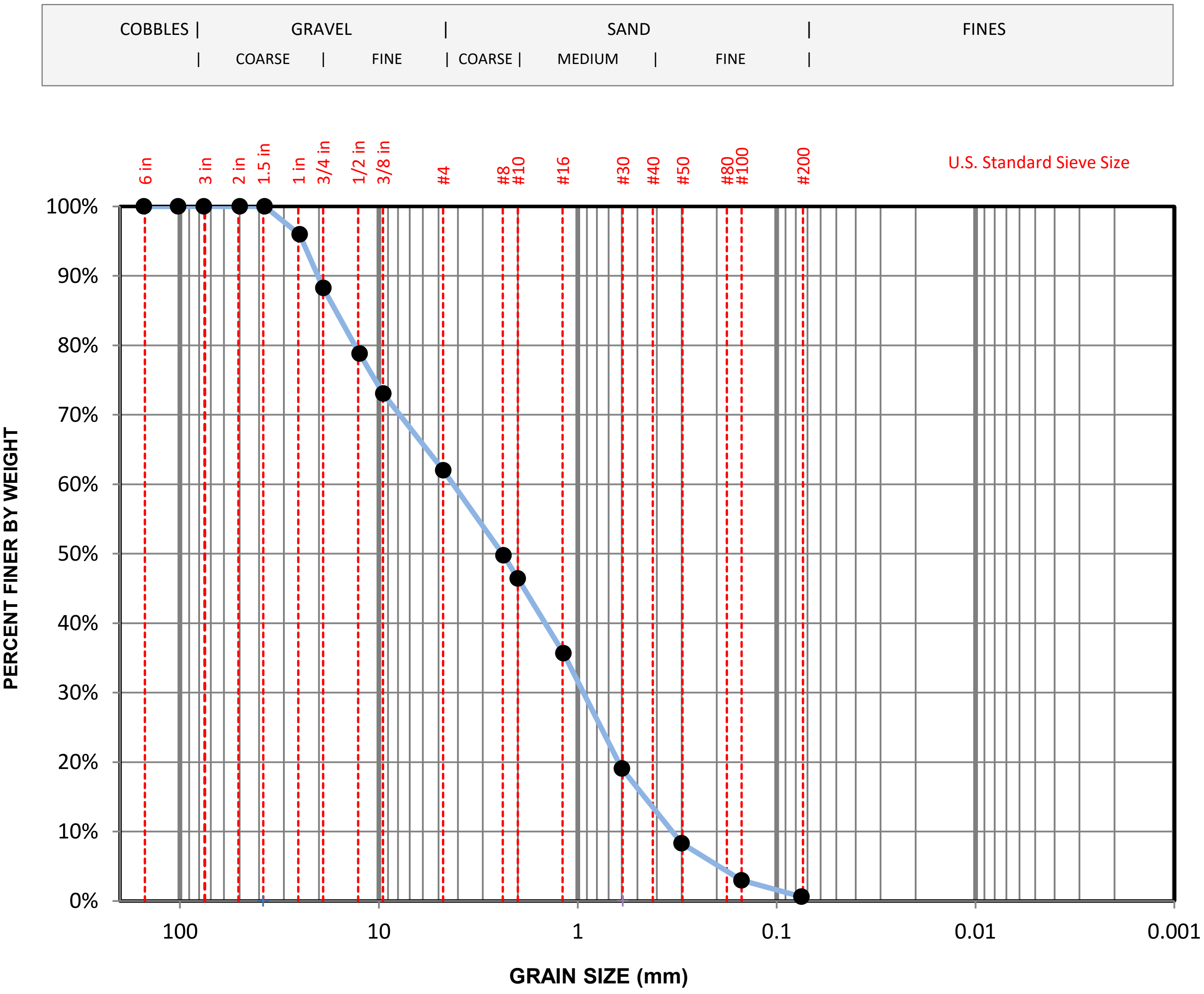
Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-2	SPT-5	20-21.5	-	-	GP-GM	**0%**	**50.1%**	**44.4%**	**5.5%**	**0%**

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

2020-08

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	10/4/2020
Note:			
Sample Description:	Olive Gray Poorly Graded SAND with Gravel , SP		

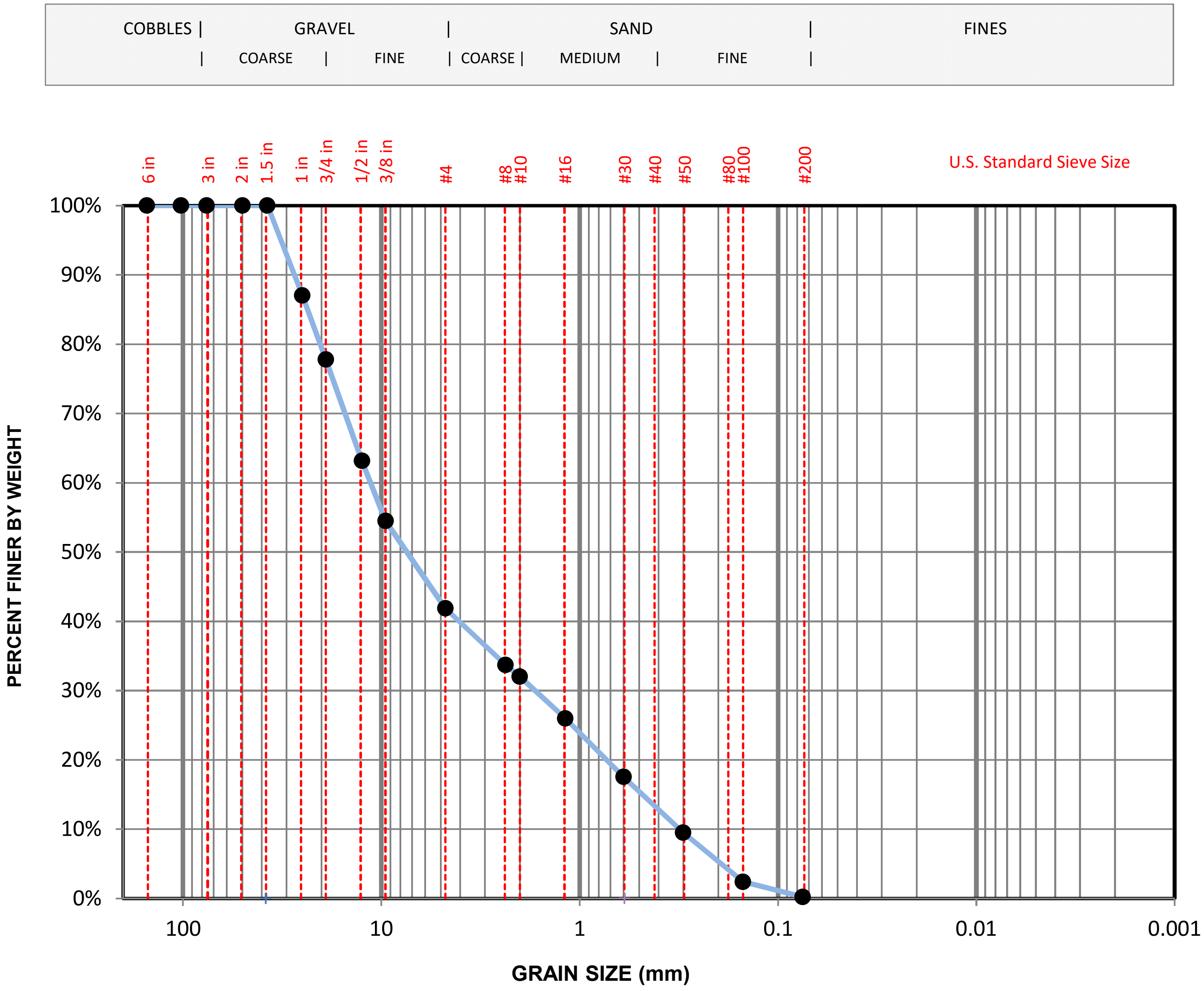


Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2μ
●	B-3	SK-10	33-35	-	-	SP	**0%**	**38%**	**61%**	**1%**	**0%**

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	10/4/2020
Note:			
Sample Description:	Olive Gray Poorly Graded GRAVEL with Sand, GP		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2μ
●	B-3	SK-1	0-5ft	-	-	GP	**0%**	**58%**	**42%**	**0%**	**0%**

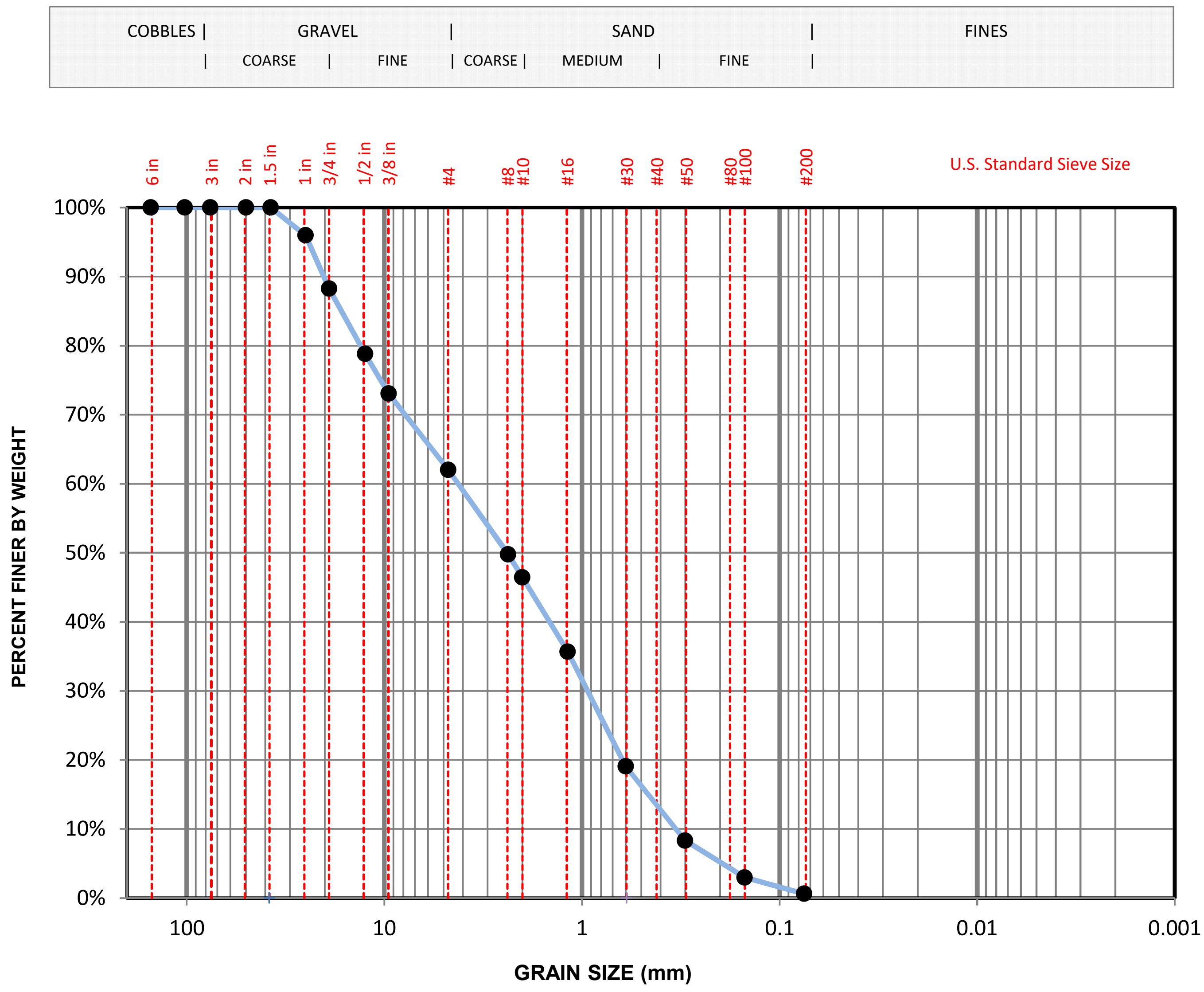
TETRA TECH BAS
GEOSCIENCE

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

2020-08

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	10/4/2020
Note:			
Sample Description:	Olive Gray Poorly Graded SAND with Gravel , SP		

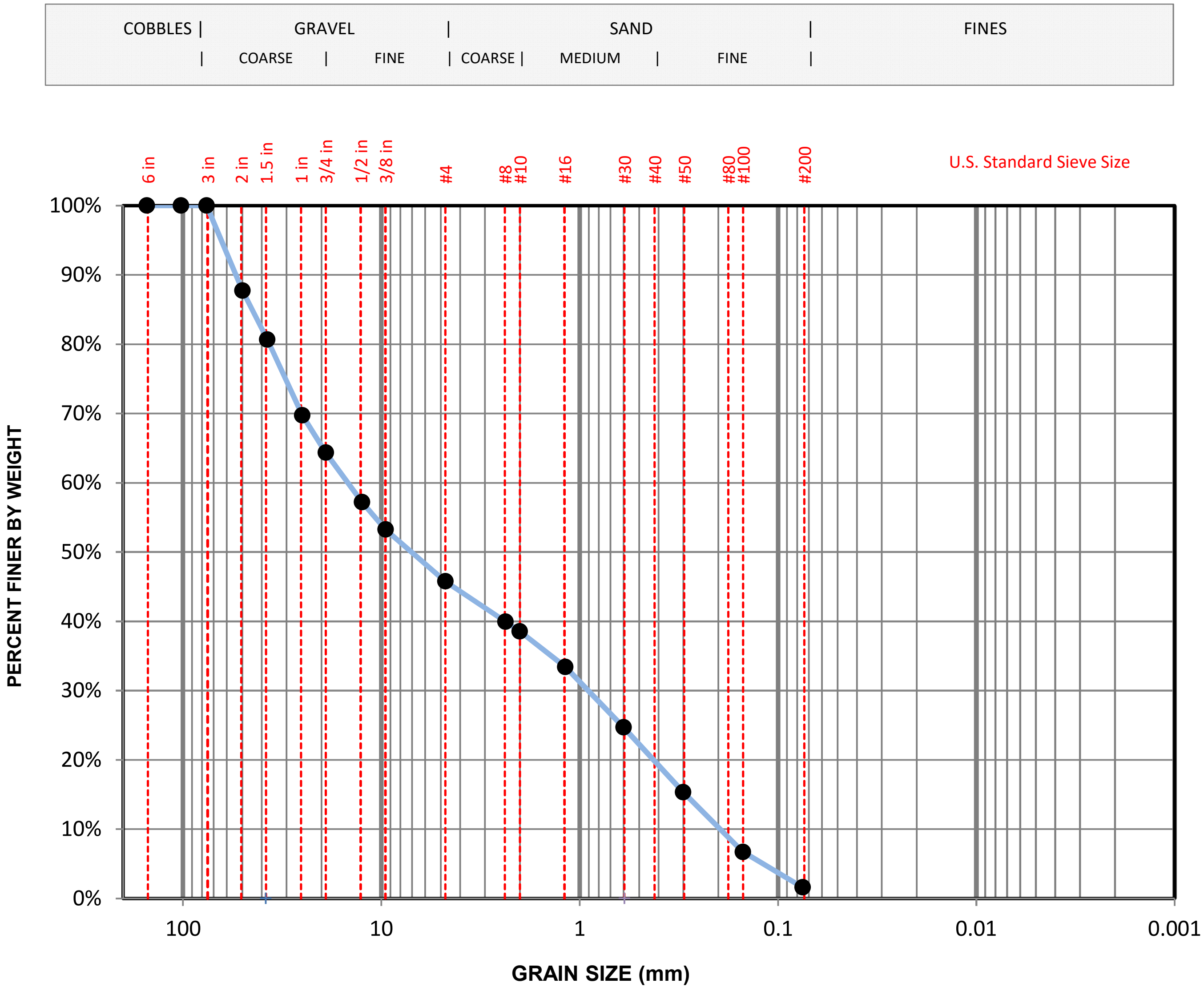


Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-3	SK-10	33-35	-	-	SP	**0%**	**38%**	**61%**	**1%**	**0%**

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	10/4/2020
Note:			
Sample Description:	Olive Gray Poorly Graded GRAVEL with Sand, GP		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2μ
●	B-5	SK-1	0-5ft	-	-	GP	**0%**	**54%**	**44%**	**2%**	**0%**

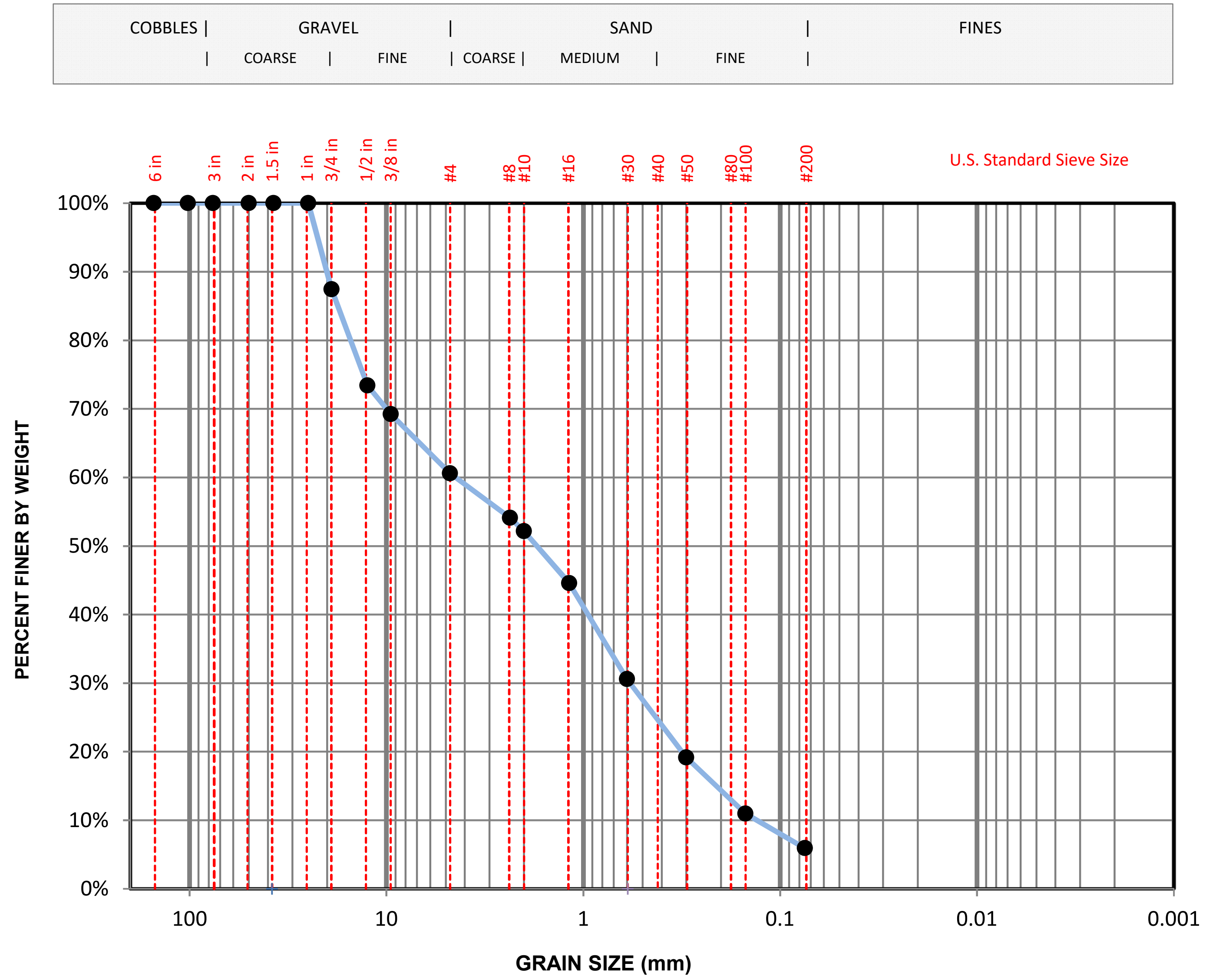
TETRA TECH BAS
GEOSCIENCE

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

2020-08

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	9/11/2020
Note:			
Sample Description:	Olive Gray Poorly Graded SAND with Silt and Gravel, SP-SM		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-8	SPT-2	2.5-4	-	-	SP-SM	**0%**	**39%**	**55%**	**6%**	**0%**

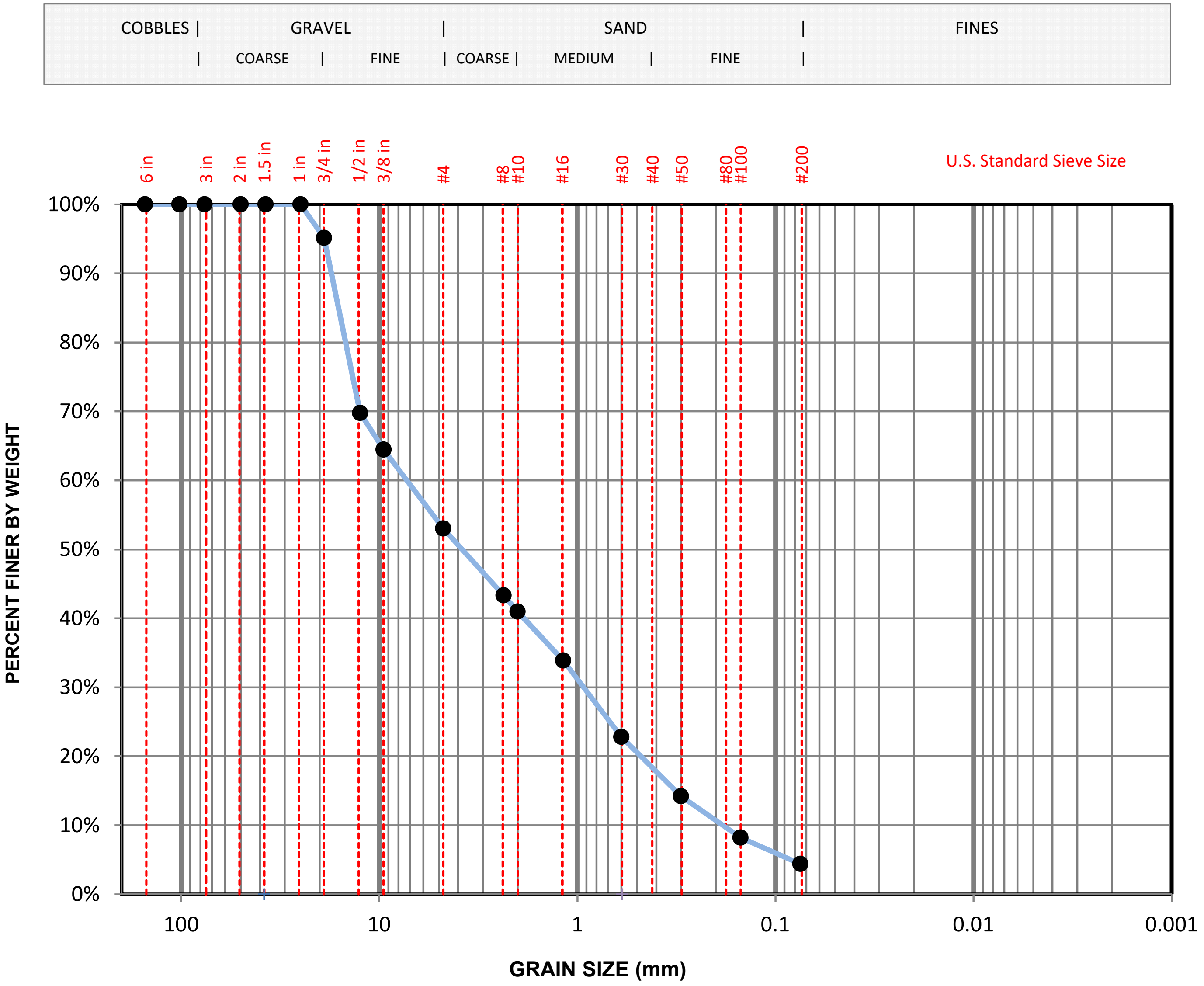
TETRA TECH BAS
GEOSCIENCE

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

2020-08

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	9/11/2020
Note:			
Sample Description:	Olive Gray Poorly GRADED SAND with Gravel, SP		

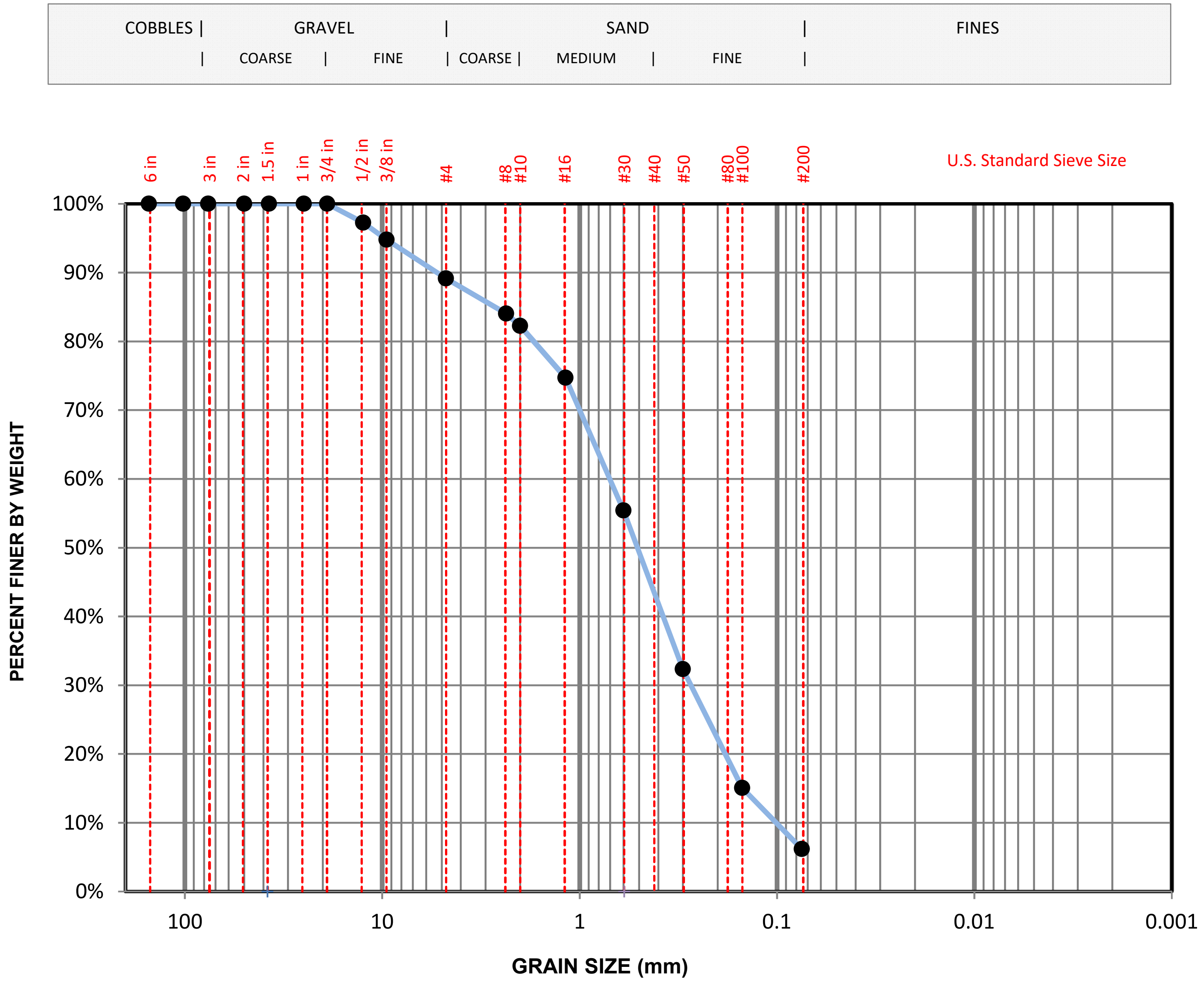


Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-9	SPT-7	25-26.5	-	-	SP	**0%**	**47%**	**49%**	**4%**	**0%**

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	9/11/2020
Note:			
Sample Description:	Olive Gray Well GRADED SAND with Silt, SW-SM		

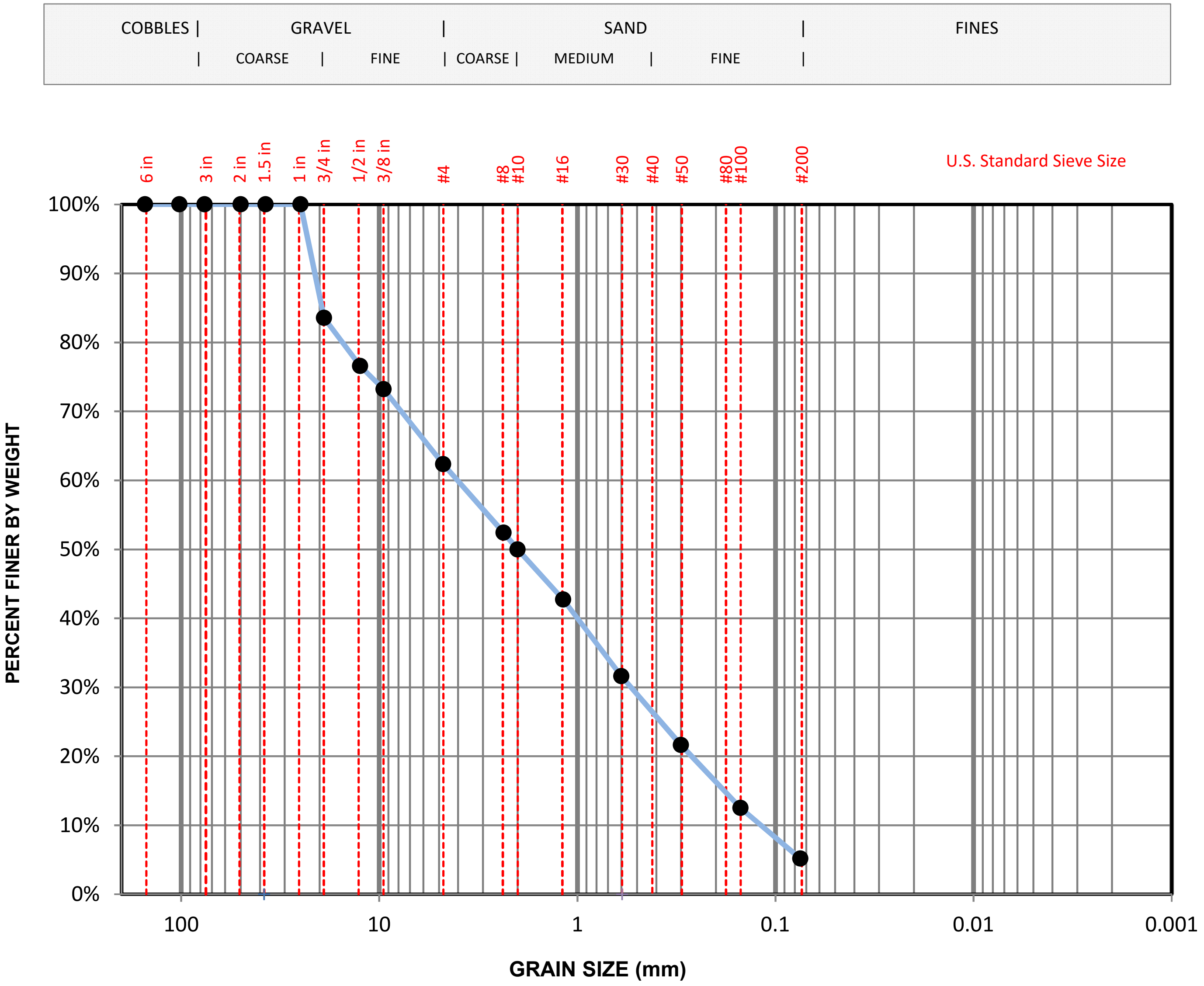


Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-12	SPT-13	45-46.5	-	-	SW-SM	**0%**	**11%**	**83%**	**6%**	**0%**

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	9/11/2020
Note:			
Sample Description:	Olive Gray Poorly Graded SAND with Silt and Gravel, SP-SM		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-13	SPT-4	12.5-14	-	-	SP-SM	**0%**	**38%**	**57%**	**5%**	**0%**

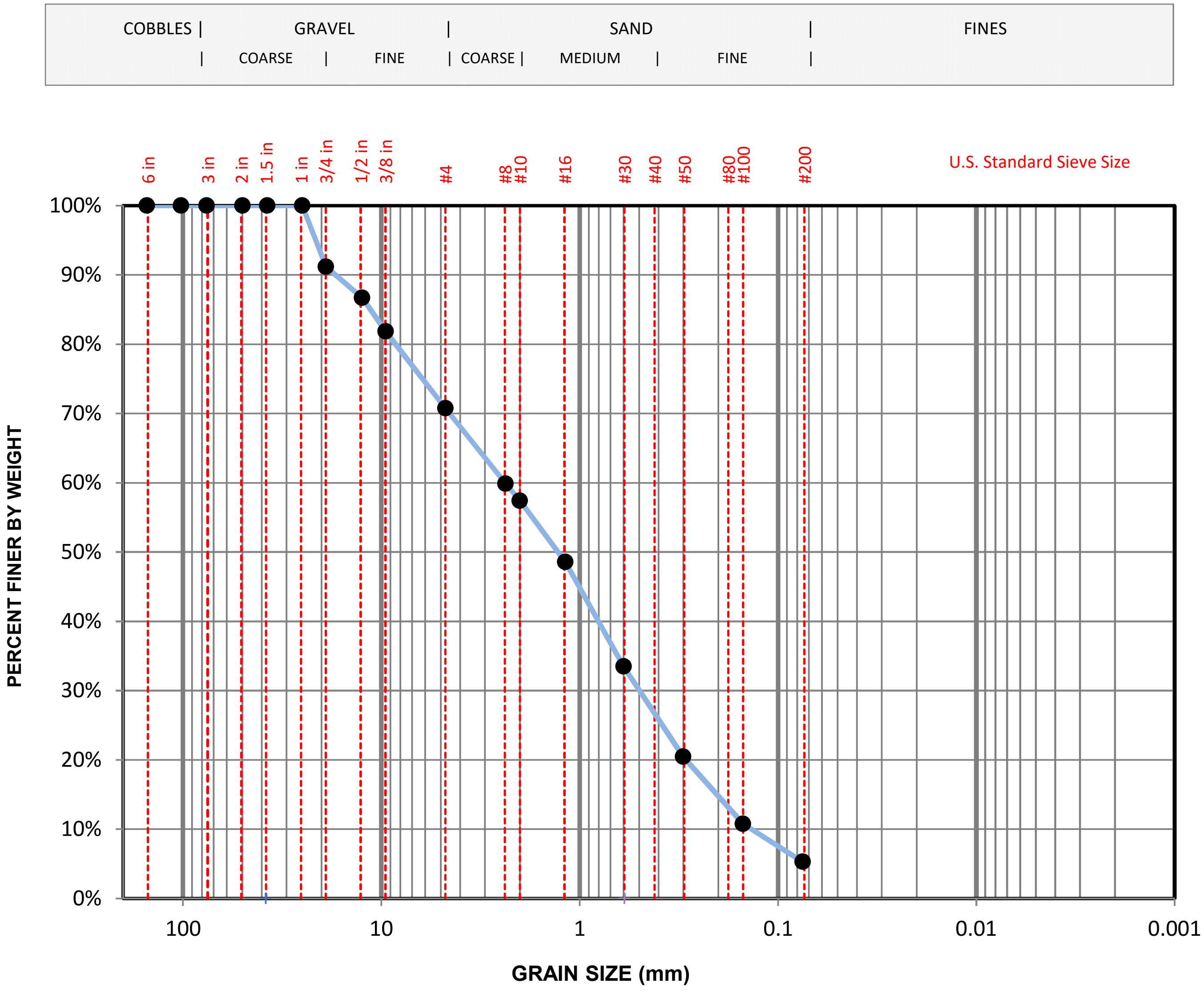
TETRA TECH BAS
GEOSCIENCE

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

2020-08

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	9/11/2020
Note:			
Sample Description:	Olive Gray Poorly Graded SAND with Silt and Gravel, SP-SM		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-15	SPT-8	25-26.5	-	-	SP-SM	**0%**	**29.2%**	**65.5%**	**5.3%**	**0%**

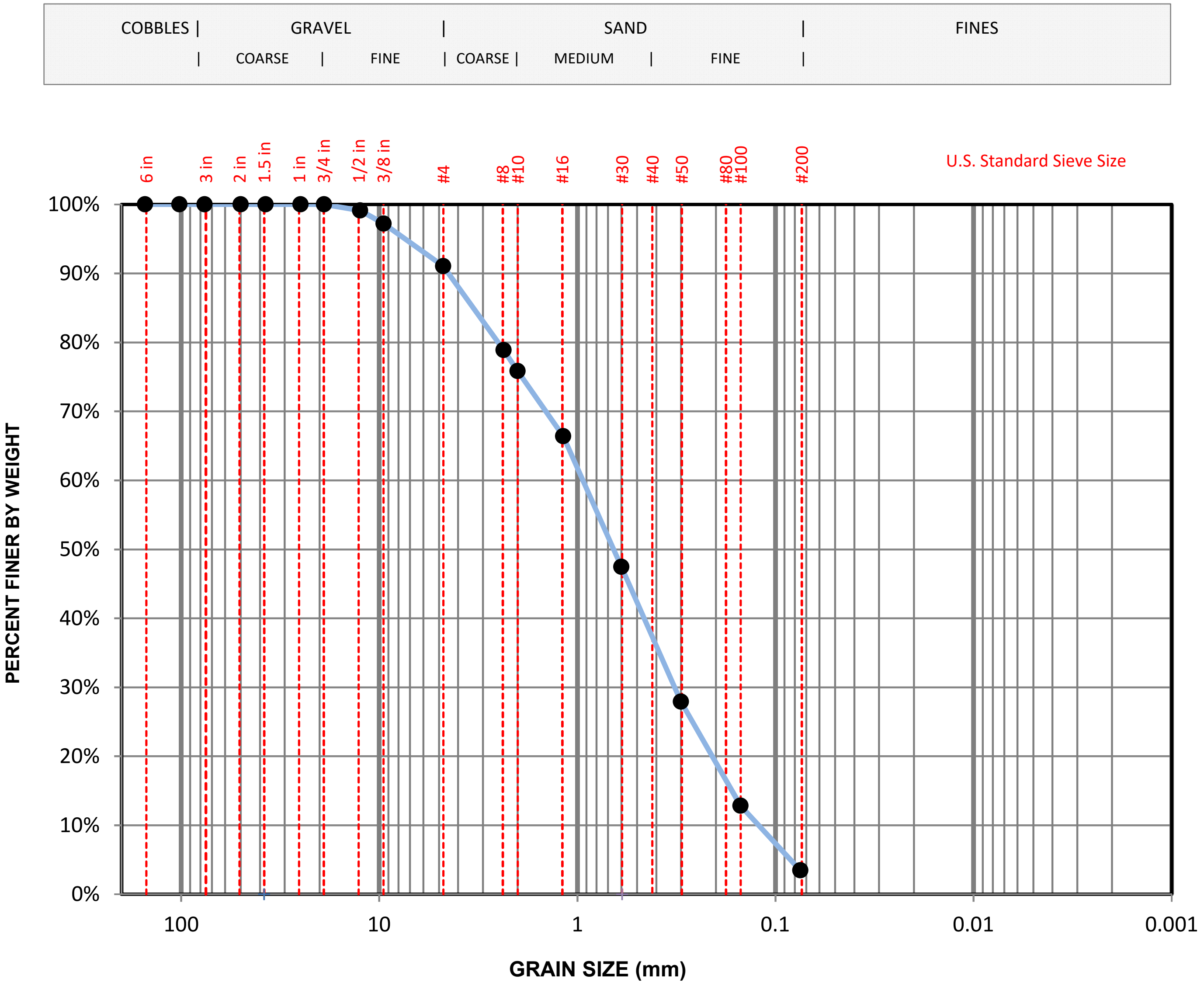
TETRA TECH BAS
GEOSCIENCE

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

2020-08

Job Name:	Mountain View I&II Wind T.		
Job Number:	GEN-20-33	**Date Sampled:**	9/2/2020
Tested By:	MG	**Date Completed:**	9/11/2020
Note:			
Sample Description:	Olive Gray Poorly Graded SAND, SP		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-98	SPT-3	5-6.5	-	-	SP	**0%**	**9%**	**88%**	**3%**	**0%**

AP Engineering and Testing, Inc.
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2607 Pomona Boulevard | Pomona, CA 91768
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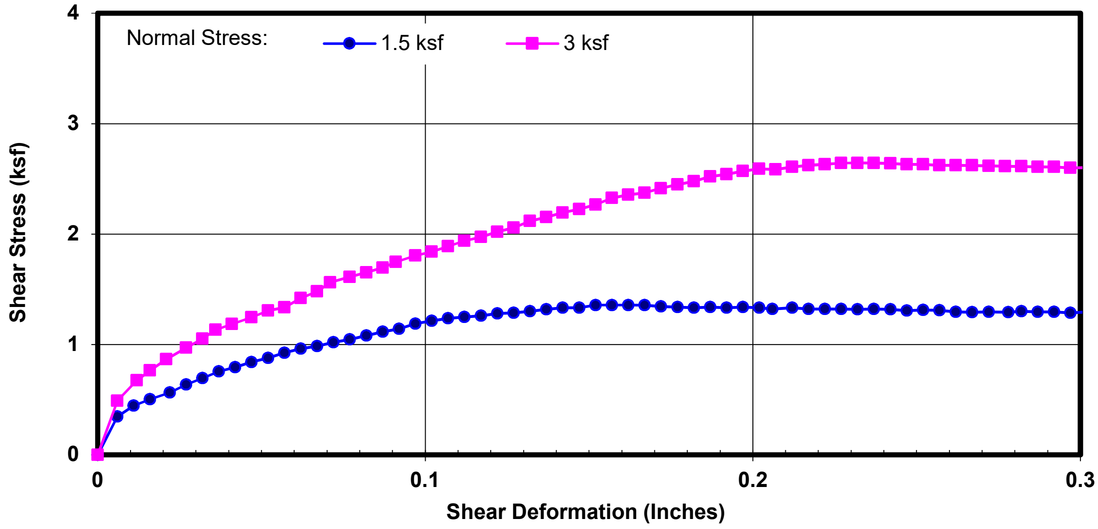
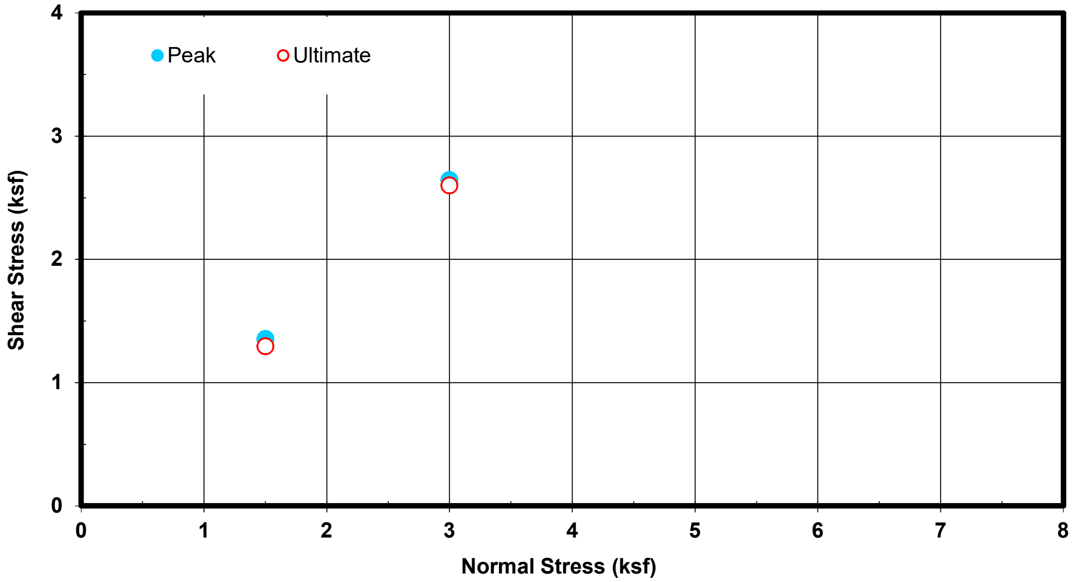
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Tetra Tech
Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Boring No.: B-2
Sample No.: R-4 **Depth (ft):** 15-16.5
Sample Type: Mod. Cal.
Soil Description: Well-Graded Sand w/silt & gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/08/20
Computed By: NR **Date:** 09/10/20
Checked by: AP **Date:** 09/14/20

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
110.0	109.4	0.6	18.4	3	92	1.5	1.356	1.296
						3	2.644	2.601





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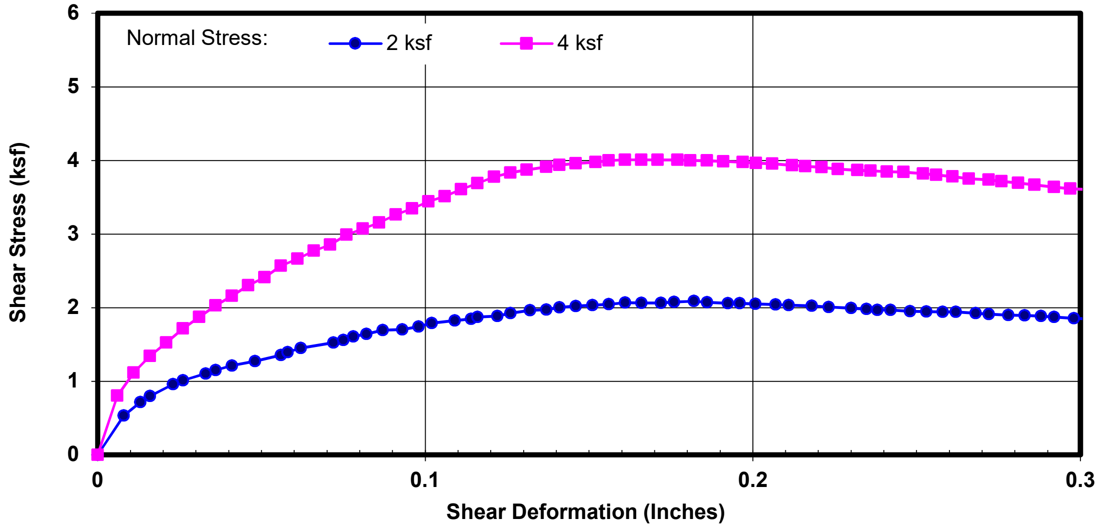
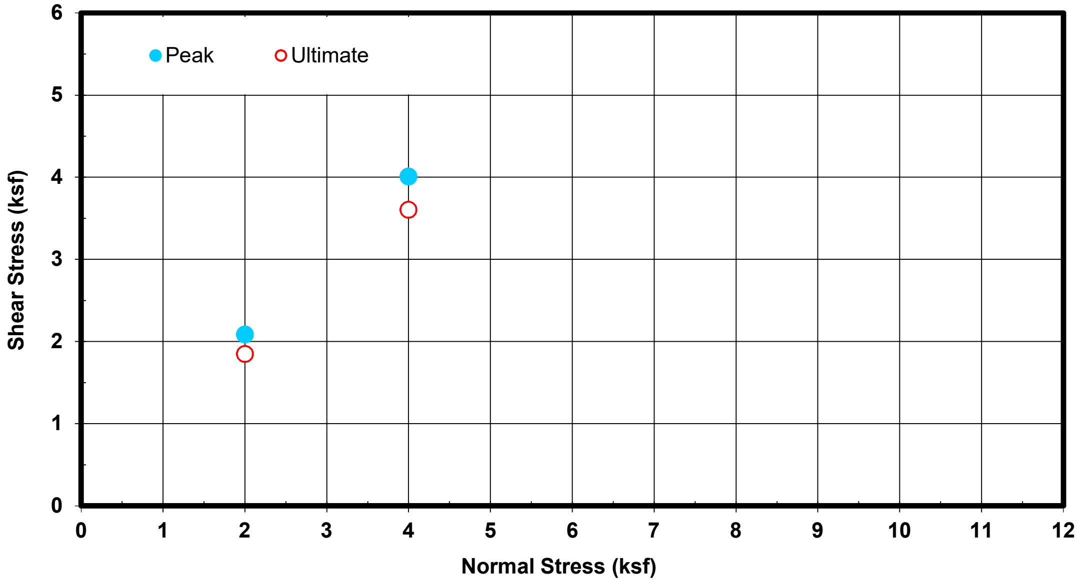
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Tetra Tech
Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Boring No.: B-3
Sample No.: R-11 **Depth (ft):** 35-36.5
Sample Type: Mod. Cal.
Soil Description: Sand w/silt & gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/28/20
Computed By: NR **Date:** 10/05/20
Checked by: AP **Date:** 10/05/20

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
116.7	113.5	2.8	16.4	16	91	2	2.088	1.848
						4	4.008	3.605



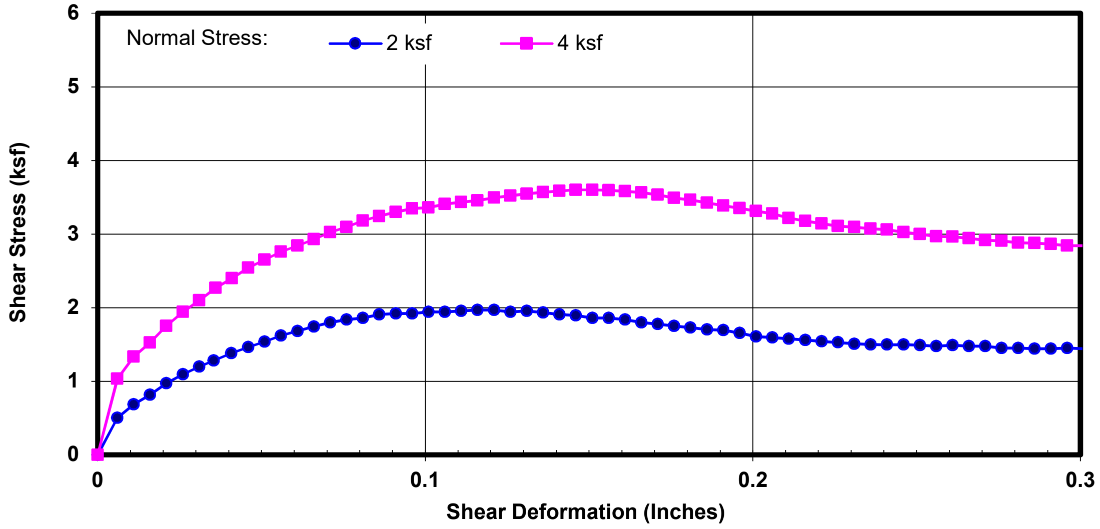


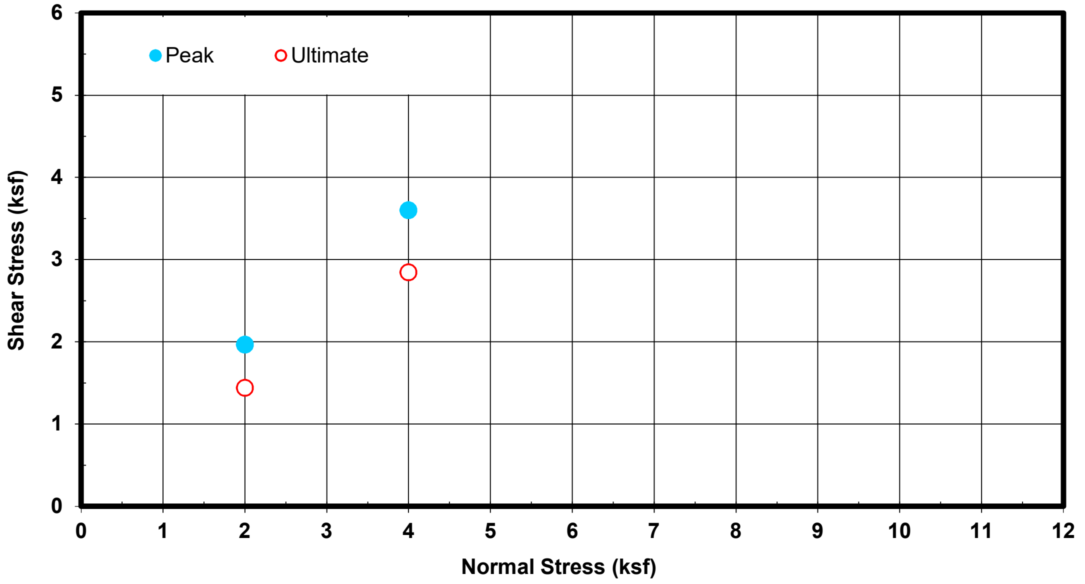
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Tetra Tech
Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Boring No.: B-6
Sample No.: R-10 **Depth (ft):** 40-41.5
Sample Type: Mod. Cal.
Soil Description: Sand w/silt & gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/28/20
Computed By: NR **Date:** 10/05/20
Checked by: AP **Date:** 10/05/20

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
110.8	108.9	1.8	18.4	9	91	2	1.968	1.440
						4	3.600	2.844





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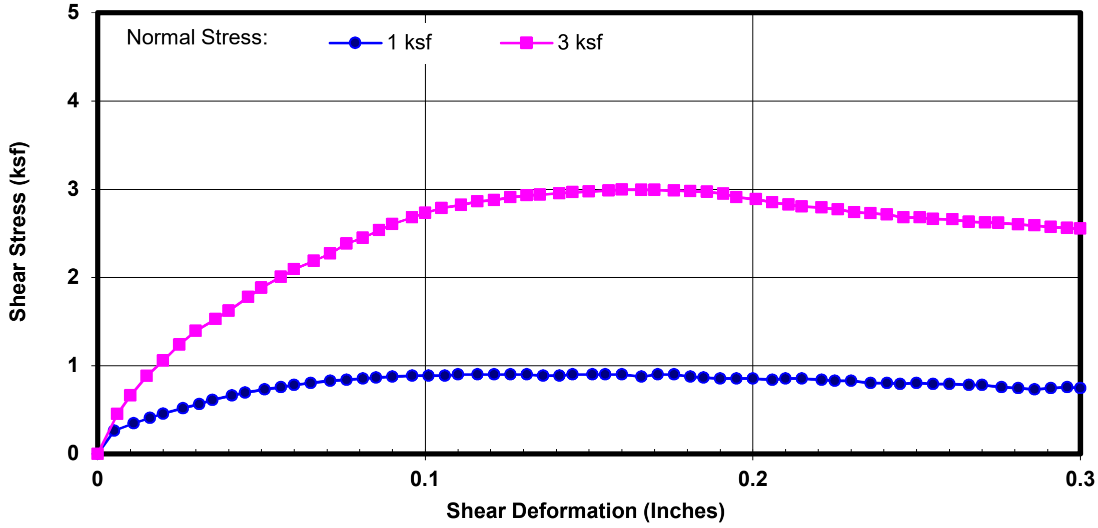
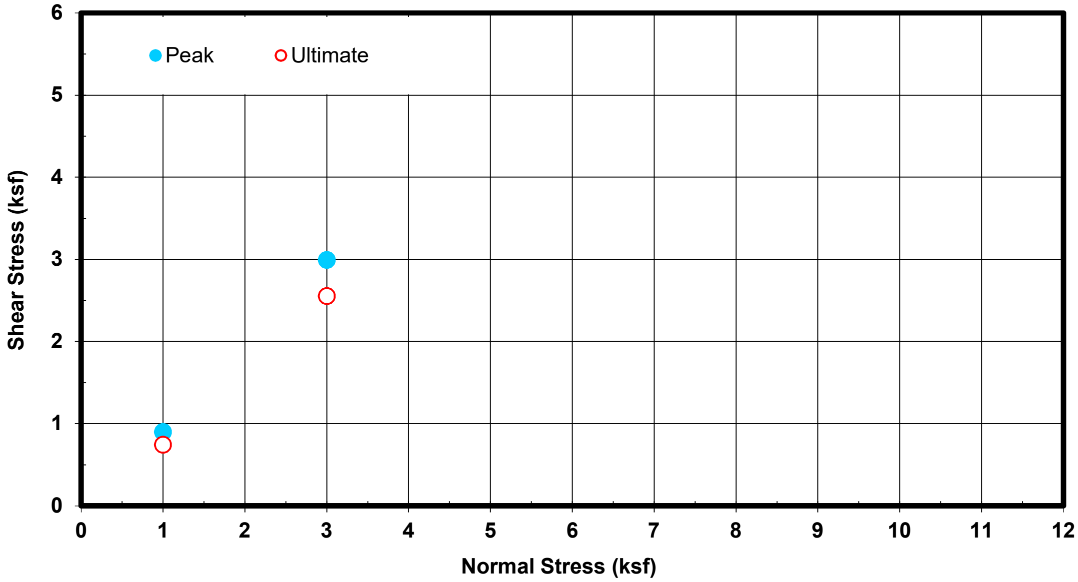
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Tetra Tech
Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Boring No.: B-12
Sample No.: R-4 **Depth (ft):** 7.5-9
Sample Type: Mod. Cal.
Soil Description: Well-Graded Sand w/silt & gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/08/20
Computed By: NR **Date:** 09/10/20
Checked by: AP **Date:** 09/14/20

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
113.6	112.3	1.2	16.5	7	89	1	0.900	0.744
						3	2.995	2.554





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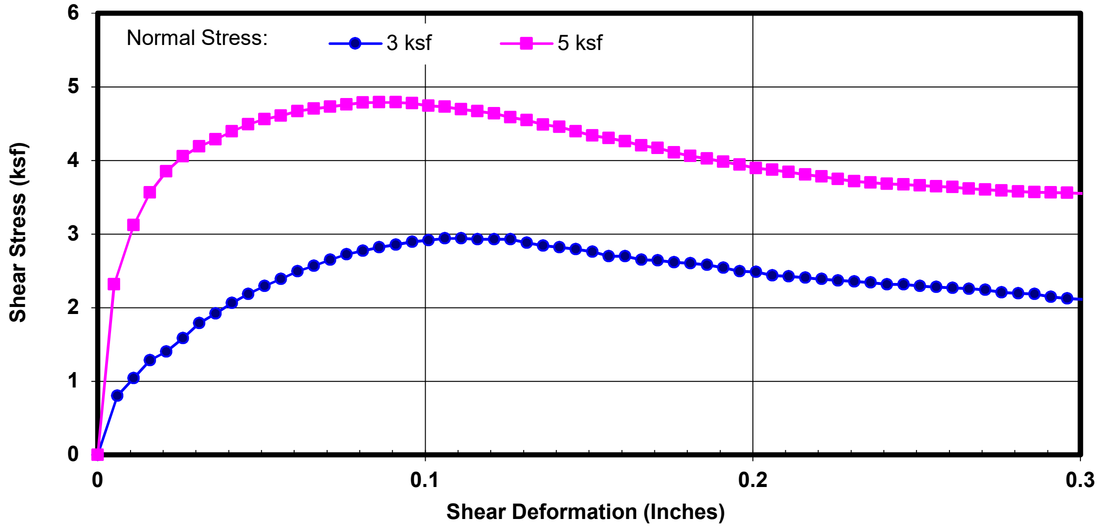
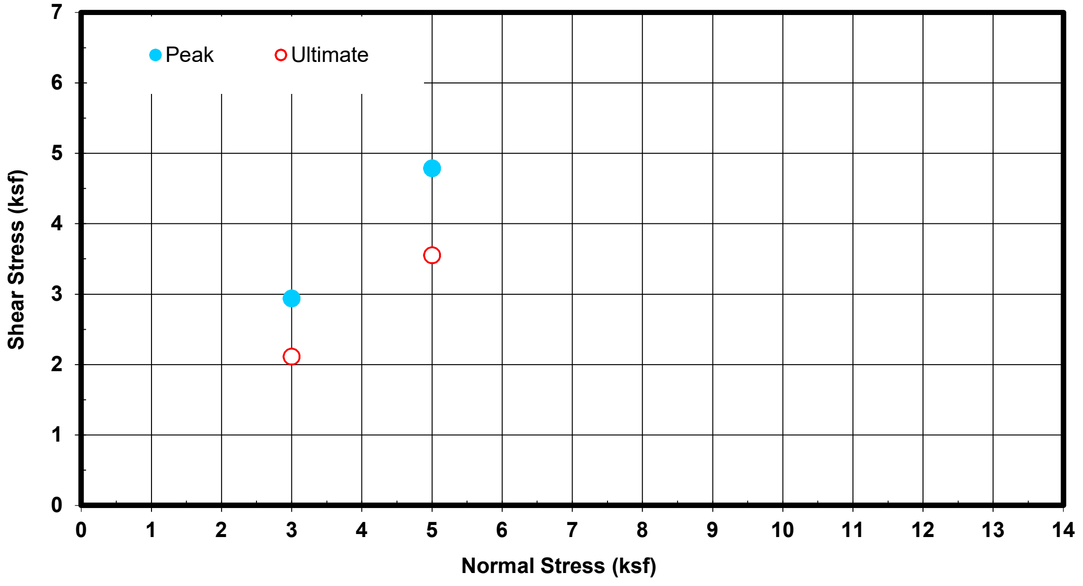
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Tetra Tech
Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Boring No.: B-12
Sample No.: R-10 **Depth (ft):** 30-31.5
Sample Type: Mod. Cal.
Soil Description: Well-Graded Sand w/silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/08/20
Computed By: NR **Date:** 09/10/20
Checked by: AP **Date:** 09/14/20

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
109.1	107.5	1.5	19.1	7	91	3	2.940	2.112
						5	4.788	3.552





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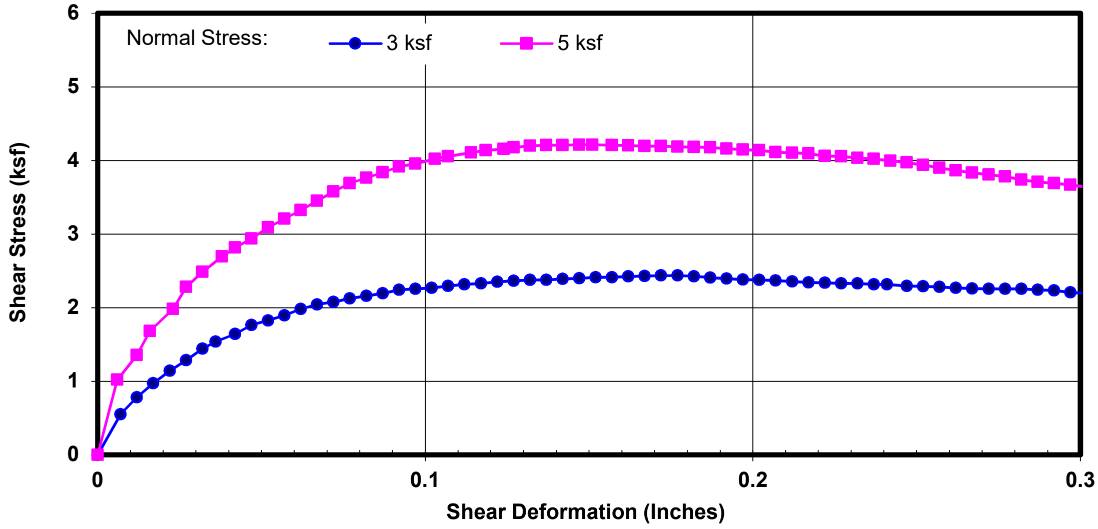
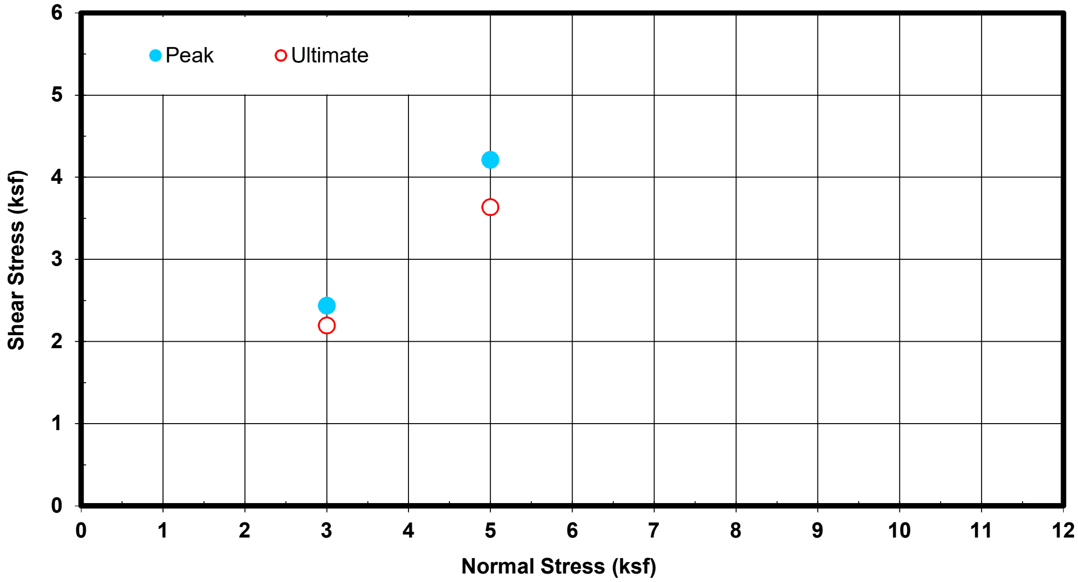
DIRECT SHEAR TEST RESULTS
ASTM D 3080

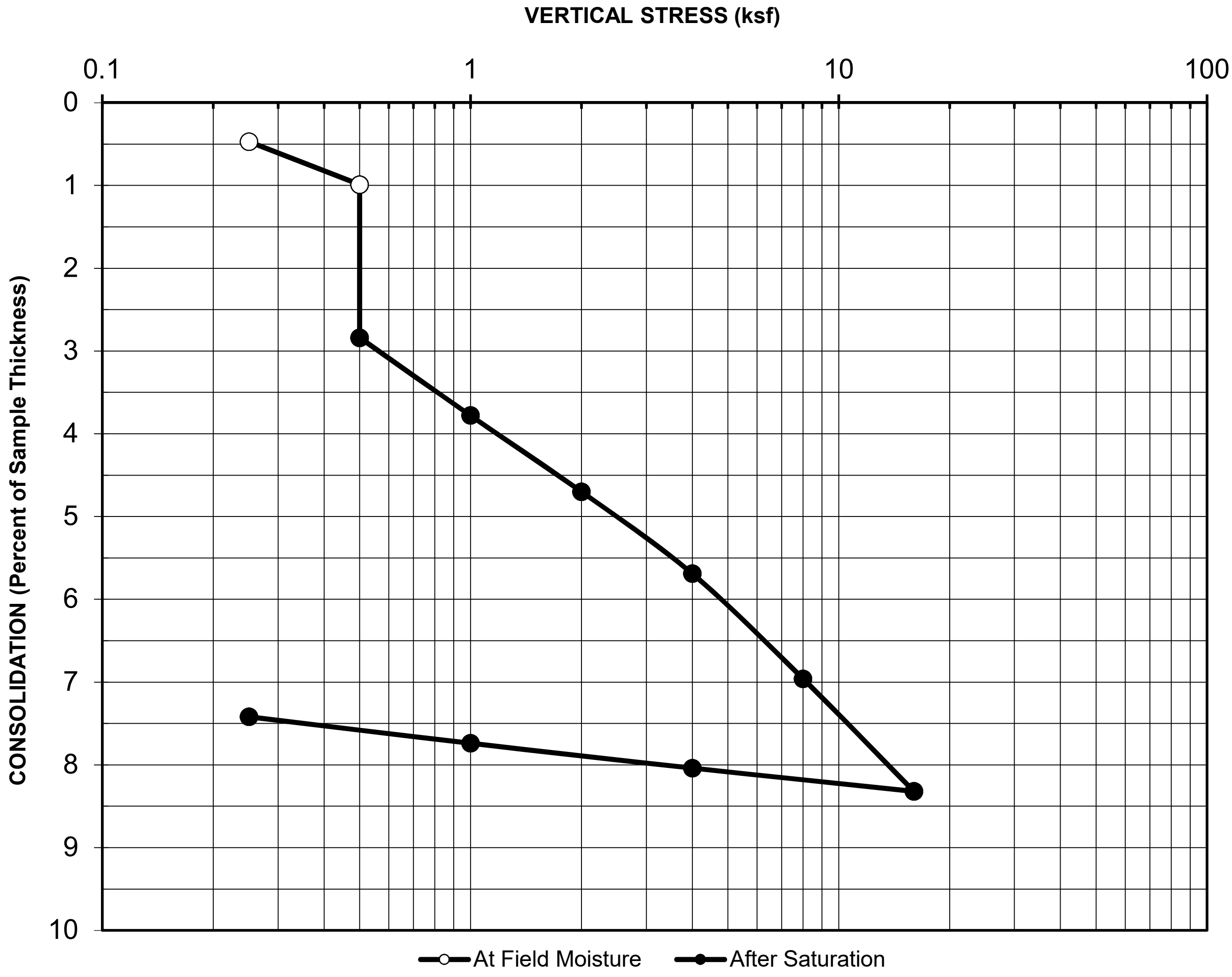
Client: Tetra Tech
Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Boring No.: B-16
Sample No.: R-12 **Depth (ft):** 35-36.5
Sample Type: Mod. Cal.
Soil Description: Well-Graded Sand w/silt & gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/08/20
Computed By: NR **Date:** 09/10/20
Checked by: AP **Date:** 09/14/20

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
111.4	110.8	0.6	17.4	3	90	3	2.436	2.196
						5	4.213	3.636







Boring No. : B-2

Sample No.: R-4

Depth (feet): 15-16.5

Sample Type: Mod Cal

Soil Description: Well-Graded Sand w/silt & gravel

Remarks: Collapse= 1.85% upon inundation

Initial Dry Unit Weight (pcf): 108.8

Initial Moisture Content (%): 0.6

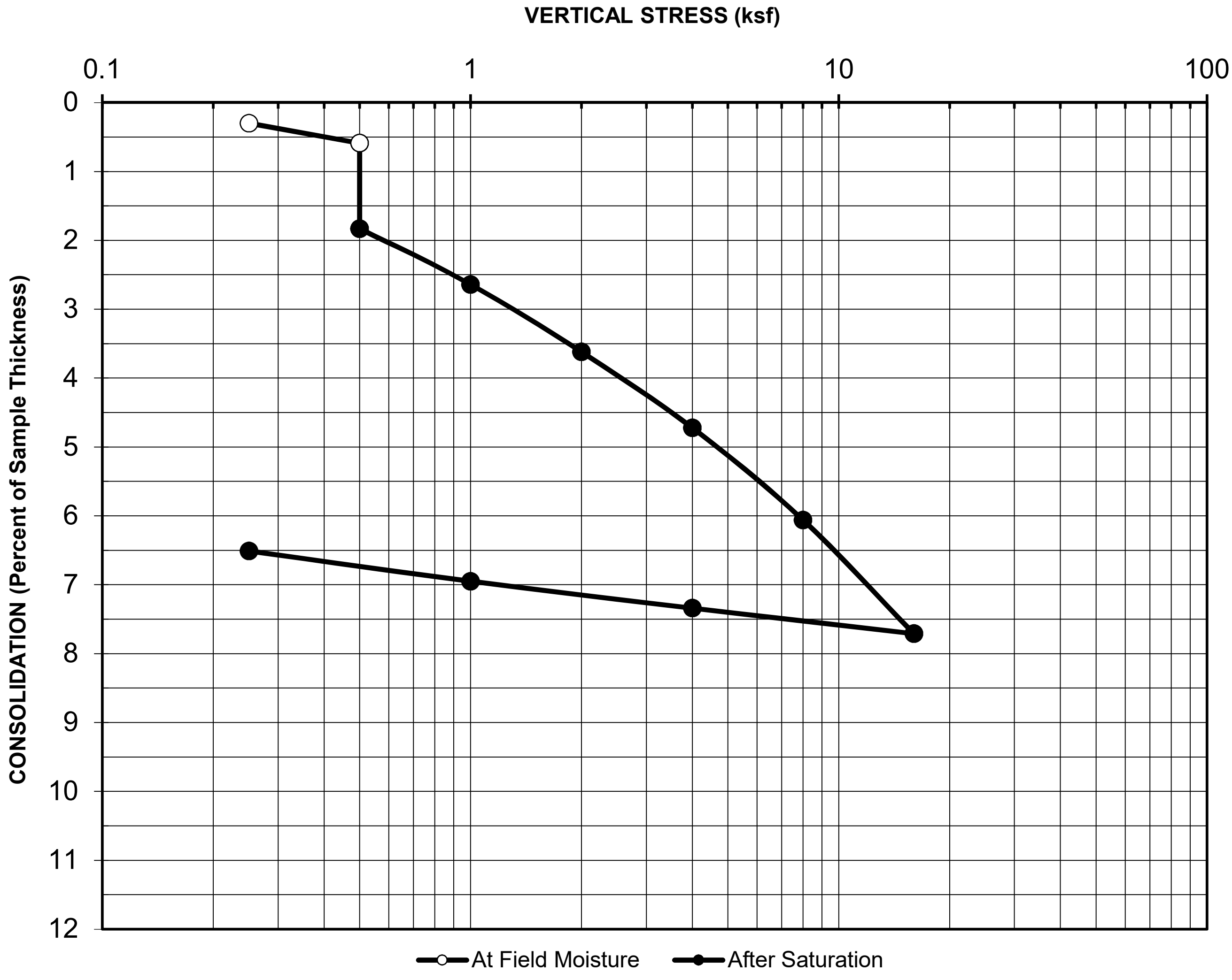
Final Moisture Content (%): 14.9

Assumed Specific Gravity: 2.7

Initial Void Ratio: 0.55

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Date: 9/4/2020
AP No: 20-0907 **Sheet No:** 1



Boring No. :	B-5	Initial Dry Unit Weight (pcf):	114.1
Sample No.:	R-8	Initial Moisture Content (%):	1.5
Depth (feet):	30-31.5	Final Moisture Content (%):	14.0
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sand w/silt & gravel	Initial Void Ratio:	0.48
Remarks:	Collapse= 1.24% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Date: 9/24/2020
AP No: 20-0907 **Sheet No:** 1

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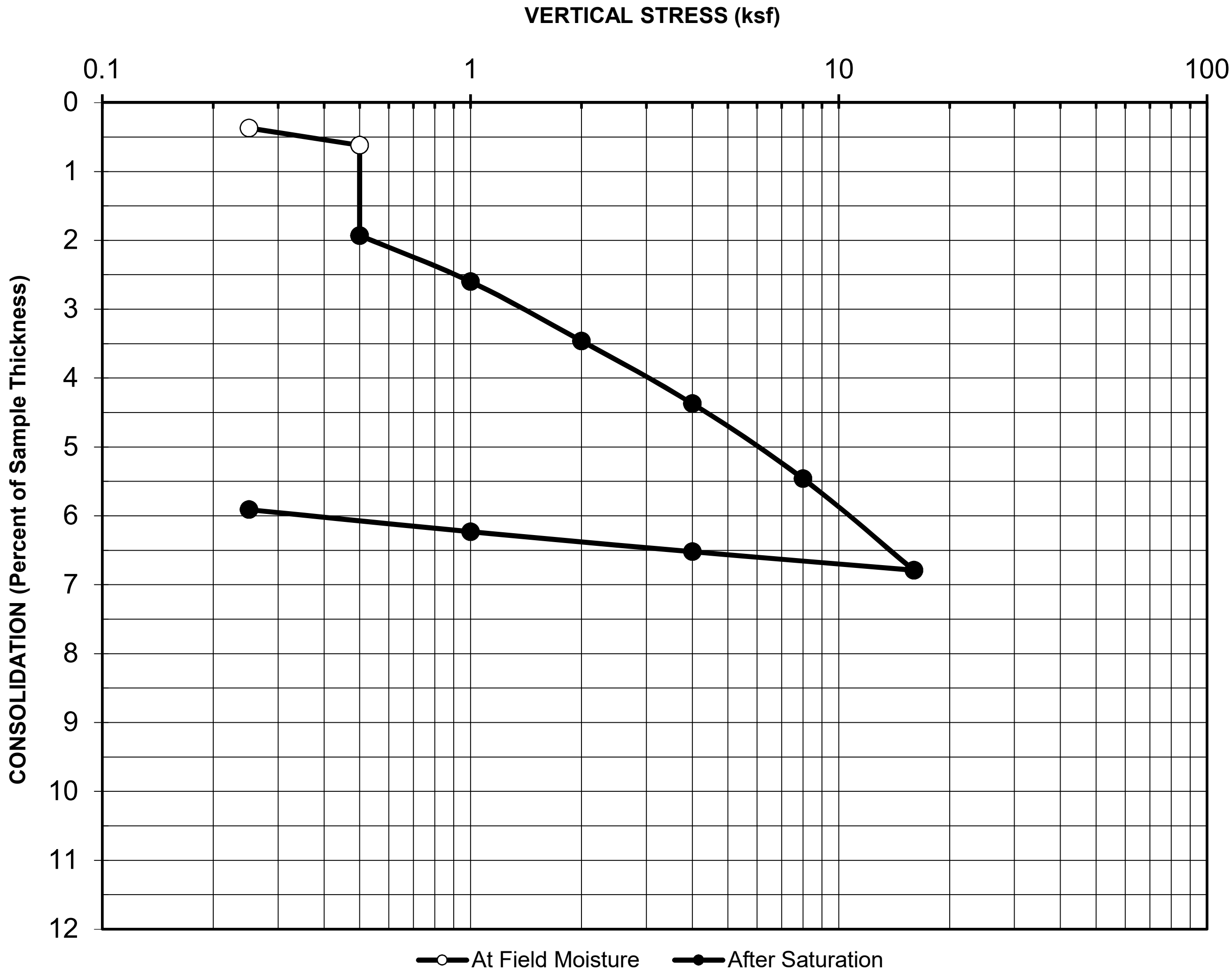
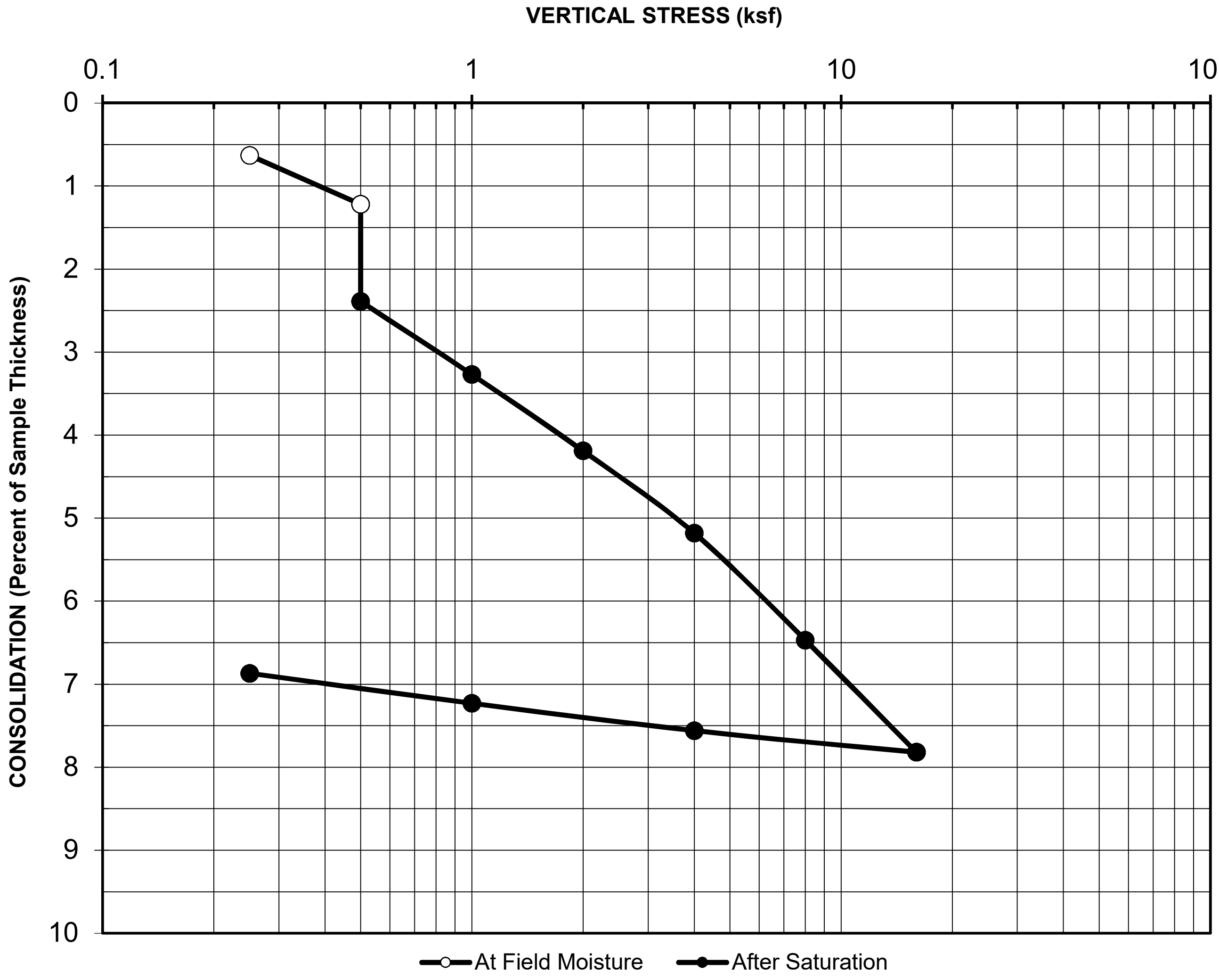


Boring No. :	B-8	Initial Dry Unit Weight (pcf):	109.7
Sample No.:	R-11	Initial Moisture Content (%):	1.1
Depth (feet):	40-41.5	Final Moisture Content (%):	13.6
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sand w/silt & gravel	Initial Void Ratio:	0.54
Remarks:	Collapse= 1.31% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Date: 9/24/2020
AP No: 20-0907 **Sheet No:** 1



Boring No. : B-12

Sample No.: R-10

Depth (feet): 30-31.5

Sample Type: Mod Cal

Soil Description: Well-Graded Sand w/silt

Remarks: Collapse= 1.17% upon inundation

Initial Dry Unit Weight (pcf): 111.8

Initial Moisture Content (%): 1.5

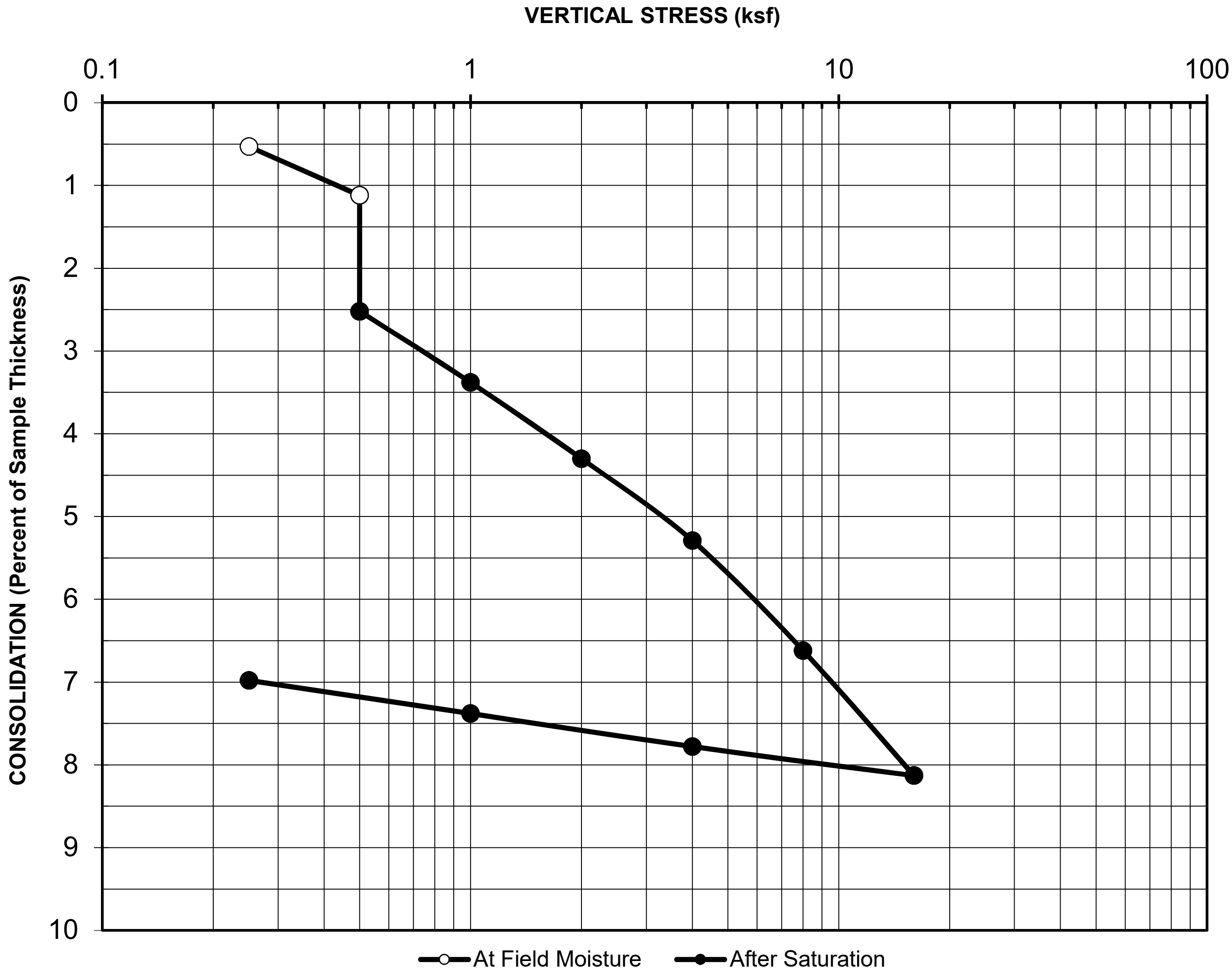
Final Moisture Content (%): 13.7

Assumed Specific Gravity: 2.7

Initial Void Ratio: 0.51

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Date: 9/4/2020
AP No: 20-0907 **Sheet No:** 1



Boring No. : B-13

Sample No.: R-5

Depth (feet): 15-16.5

Sample Type: Mod Cal

Soil Description: Well-Graded Sand w/silt

Remarks: Collapse= 1.40% upon inundation

Initial Dry Unit Weight (pcf): 98.2

Initial Moisture Content (%): 3.0

Final Moisture Content (%): 20.5

Assumed Specific Gravity: 2.7

Initial Void Ratio: 0.72

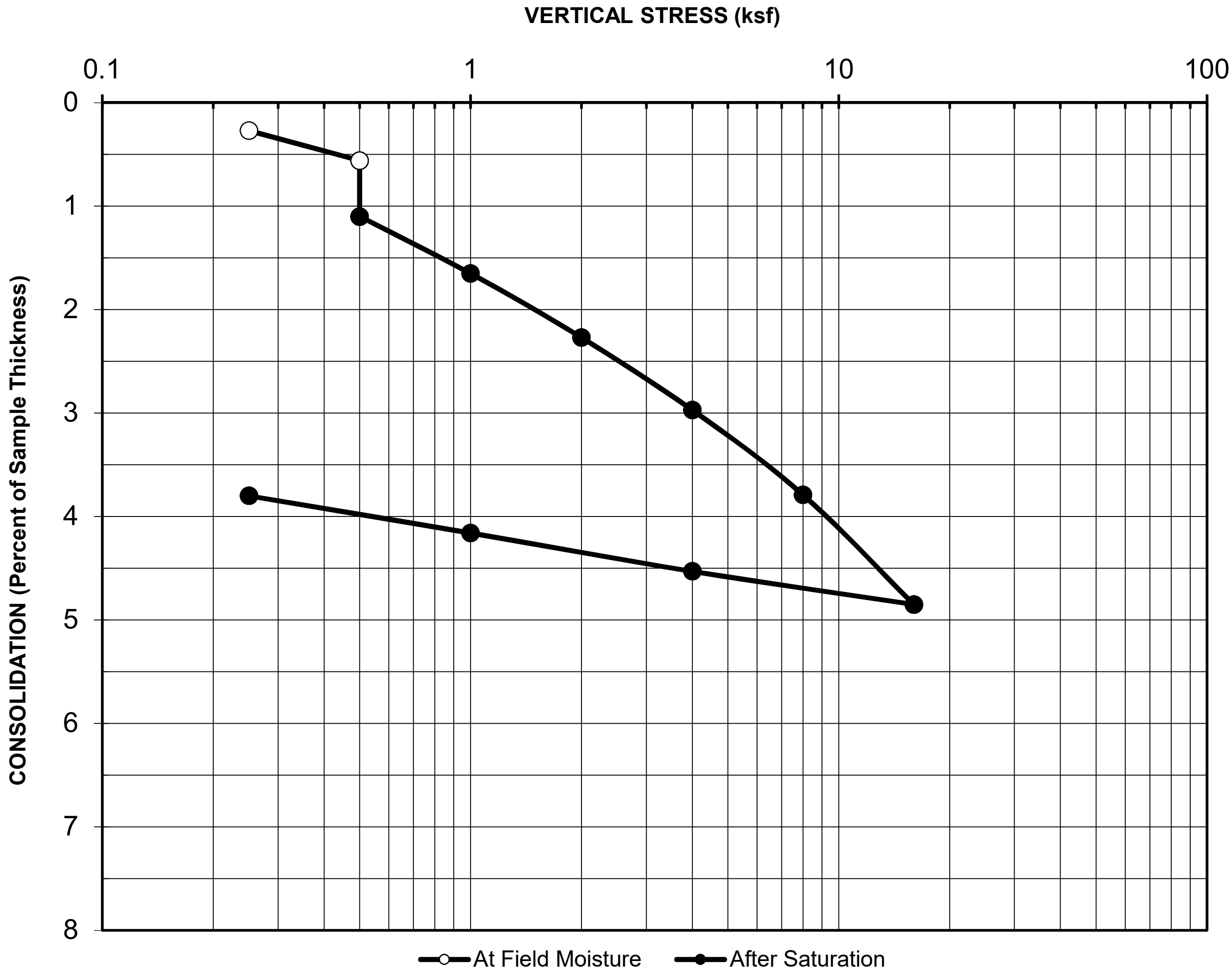
CONSOLIDATION CURVE
ASTM D 2435

Project Name: Mountain View I & II Wind Turbine Repowering

Project No.: GEN 20-33

Date: 9/4/2020

AP No: 20-0907 **Sheet No:** 1



Boring No. : B-14

Sample No.: R-8

Depth (feet): 20-21.5

Sample Type: Mod Cal

Soil Description: Silty Sand

Remarks: Collapse= 0.54% upon inundation

Initial Dry Unit Weight (pcf): 103.2

Initial Moisture Content (%): 3.1

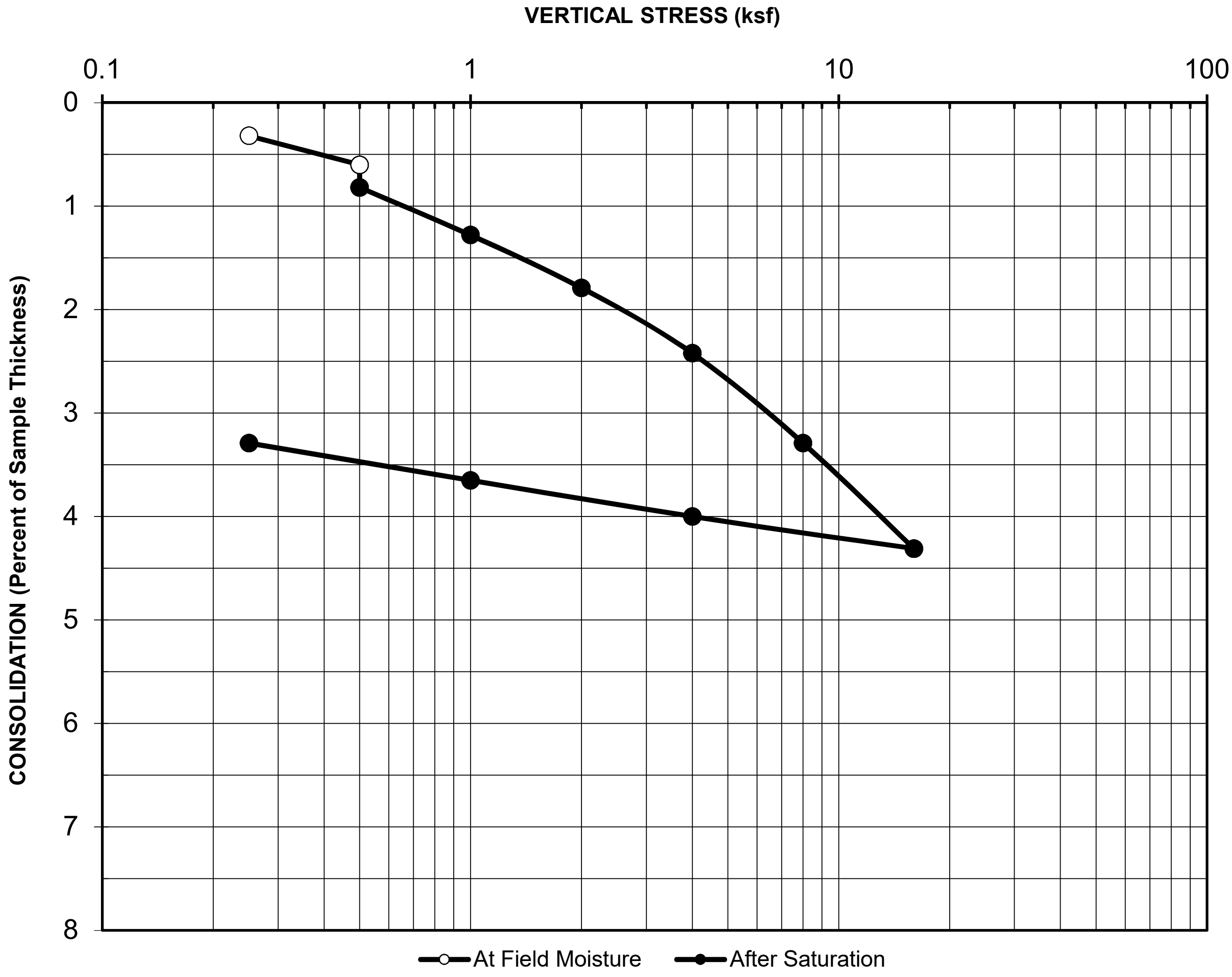
Final Moisture Content (%): 19.1

Assumed Specific Gravity: 2.7

Initial Void Ratio: 0.63

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Mountain View I & II Wind Turbine Repowering
Project No.: GEN 20-33
Date: 9/4/2020
AP No: 20-0907 **Sheet No:** 1



Boring No. : B-15

Sample No.: R-9

Depth (feet): 30-31.5

Sample Type: Mod Cal

Soil Description: Silty Sand

Remarks: Collapse= 0.22% upon inundation

Initial Dry Unit Weight (pcf): 108.4

Initial Moisture Content (%): 2.6

Final Moisture Content (%): 17.5

Assumed Specific Gravity: 2.7

Initial Void Ratio: 0.55

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Mountain View I & II Wind Turbine Repowering

Project No.: GEN 20-33

Date: 9/4/2020

AP No: 20-0907 **Sheet No:** 1

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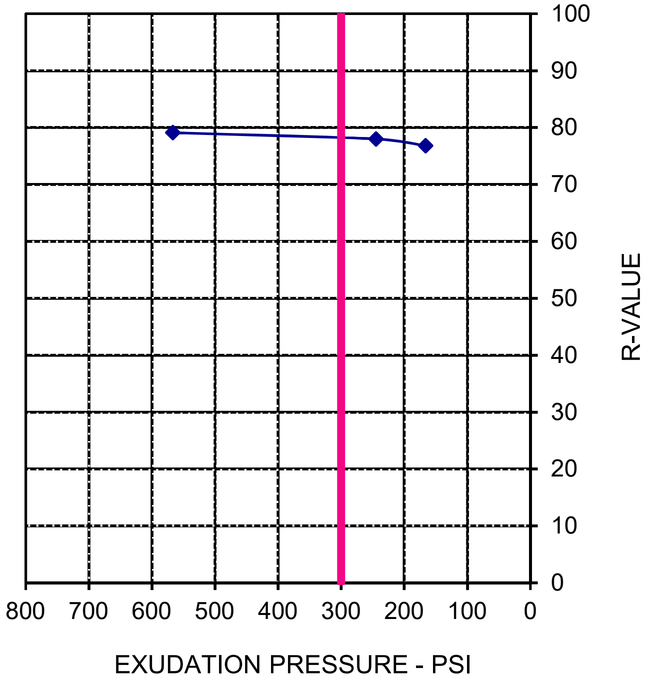
R-VALUE TEST DATA

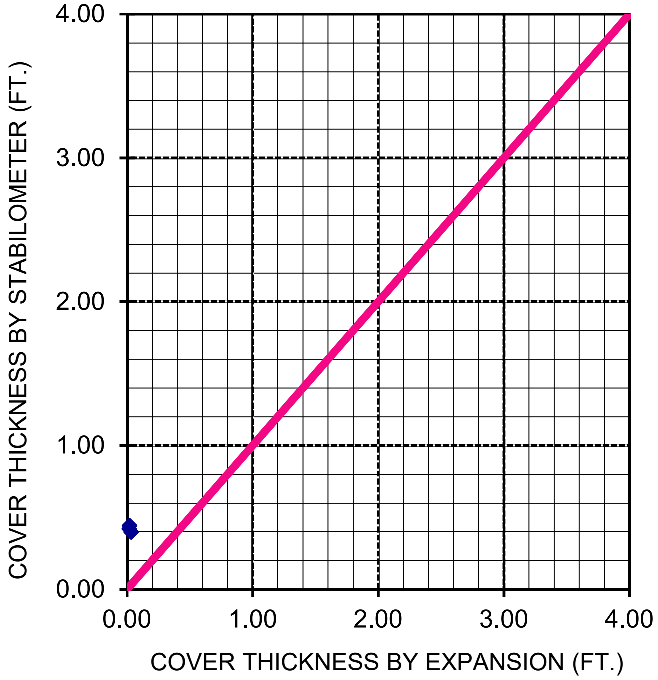
ASTM D2844

Project Name: Mountain View I & II Wind Turbine Repowering
Tested By: ST Date: 09/23/20
Project Number: GEN 20-33
Computed By: KM Date: 09/24/20
Boring No.: B-4
Checked By: AP Date: 10/05/20
Sample No.: SK-1 Depth (ft.): 0-5
Location: N/A
Soil Description: Sand w/silt & gravel

Mold Number	D	E	F	
Water Added, g	30	51	41	
Compact Moisture(%)	6.8	8.8	7.9	
Compaction Gage Pressure, psi	350	350	350	
Exudation Pressure, psi	567	245	166	
Sample Height, Inches	2.3	2.3	2.3	
Gross Weight Mold, g	3032	3036	2948	
Tare Weight Mold, g	1965	1955	1869	
Net Sample Weight, g	1067	1081	1079	
Expansion, inchesx10^{-4}	10	5	6	
Stability 2,000 (160 psi)	11/18	12/18	13/18	
Turns Displacement	4.56	4.90	5.26	
R-Value Uncorrected	81	80	79	
R-Value Corrected	79	78	77	
Dry Density, pcf	131.6	130.8	131.7	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.40	0.42	0.44	
G.E. by Expansion	0.03	0.02	0.02	

R-VALUE	By Exudation:	78
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	78
Remarks	Gf = 1.34, and 36.5 % Retained on the ¾" *Not Applicable	





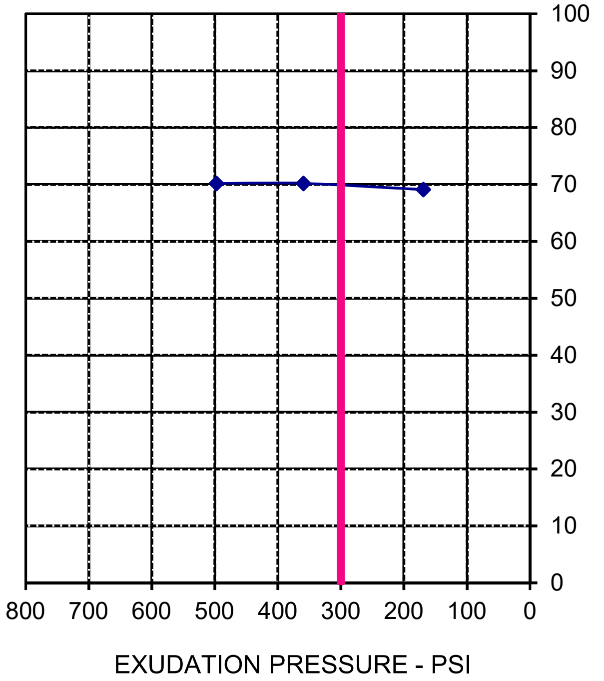
R-VALUE TEST DATA

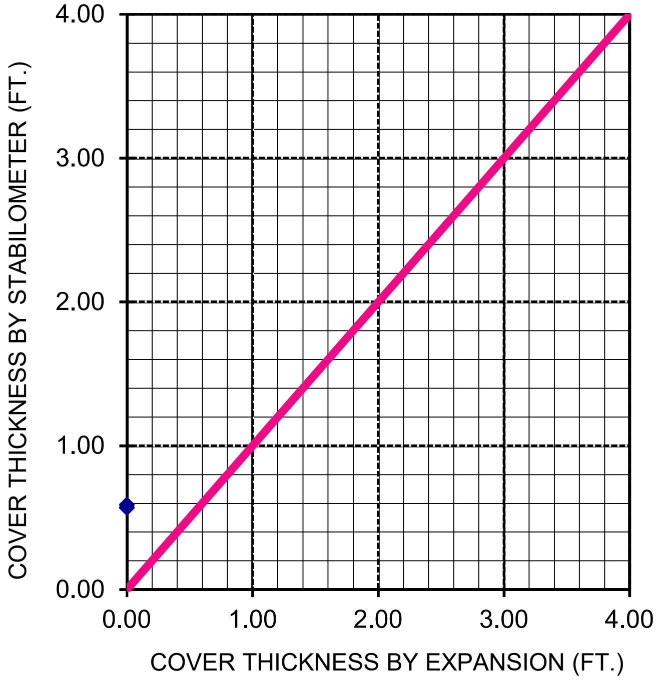
ASTM D2844

Project Name: Mountain View I & II Wind Turbine Repowering
Tested By: ST Date: 09/08/20
Project Number: GEN 20-33
Computed By: KM Date: 09/09/20
Boring No.: B-96
Checked By: AP Date: 09/14/20
Sample No.: SK-1 Depth (ft.): 0-5
Location: N/A
Soil Description: Well-Graded Sand w/silt & gravel

Mold Number	D	E	F	
Water Added, g	26	52	72	
Compact Moisture(%)	3.0	5.6	7.6	
Compaction Gage Pressure, psi	250	250	250	
Exudation Pressure, psi	497	360	169	
Sample Height, Inches	2.4	2.4	2.4	
Gross Weight Mold, g	2994	3005	2940	
Tare Weight Mold, g	1965	1956	1870	
Net Sample Weight, g	1028	1050	1071	
Expansion, inchesx10^{-4}	0	0	0	
Stability 2,000 (160 psi)	12/21	12/22	13/23	
Turns Displacement	6.35	6.22	6.20	
R-Value Uncorrected	72	72	71	
R-Value Corrected	70	70	69	
Dry Density, pcf	126.0	125.5	125.6	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.57	0.57	0.59	
G.E. by Expansion	0.00	0.00	0.00	

R-VALUE	By Exudation:	70
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	70
Remarks	Gf = 1.34, and 25.7 % Retained on the ¾" *Not Applicable	





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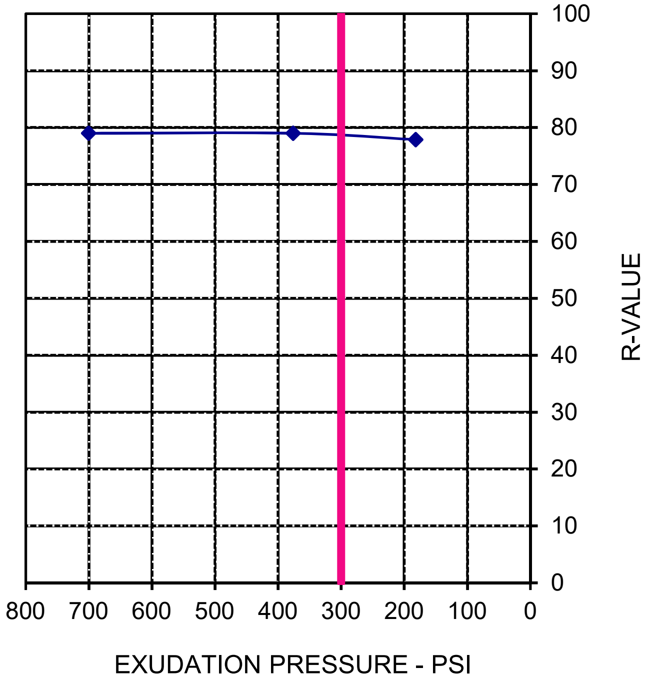
R-VALUE TEST DATA

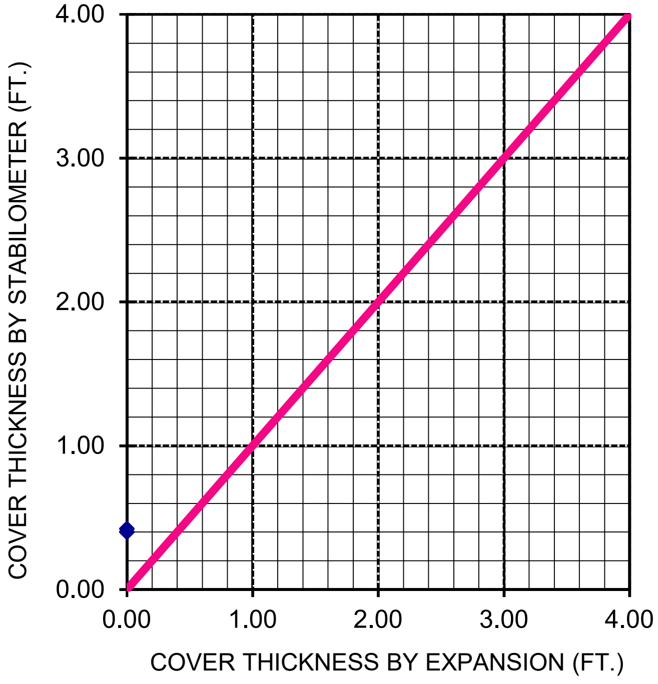
ASTM D2844

Project Name: Mountain View I & II Wind Turbine Repowering
Tested By: ST Date: 09/08/20
Project Number: GEN 20-33
Computed By: KM Date: 09/09/20
Boring No.: B-98
Checked By: AP Date: 09/14/20
Sample No.: SK-1 Depth (ft.): 0-5
Location: N/A
Soil Description: Well-Graded Sand w/silt

Mold Number	G	H	I	
Water Added, g	41	65	84	
Compact Moisture(%)	5.3	7.7	9.6	
Compaction Gage Pressure, psi	250	250	250	
Exudation Pressure, psi	700	376	182	
Sample Height, Inches	2.4	2.4	2.4	
Gross Weight Mold, g	2862	2883	2873	
Tare Weight Mold, g	1828	1837	1819	
Net Sample Weight, g	1034	1045	1054	
Expansion, inchesx10^{-4}	0	0	0	
Stability 2,000 (160 psi)	7/16	8/17	9/18	
Turns Displacement	5.67	5.37	5.30	
R-Value Uncorrected	80	80	79	
R-Value Corrected	79	79	78	
Dry Density, pcf	124.0	122.5	121.3	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.40	0.40	0.42	
G.E. by Expansion	0.00	0.00	0.00	

R-VALUE	By Exudation:	79
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	79
Remarks	Gf = 1.34, and 8.5 % Retained on the ¾" *Not Applicable	





TRANSMITTAL LETTER

DATE: September 24, 2020

ATTENTION: **Fernando Cuenca**

TO: Tetra Tech
1360 Valley Vista Drive
Diamond Bar, CA 91765

SUBJECT: Laboratory Test Data
Mountain View I & II Wind Turbine
Your #GEN 20-33, HDR Lab #20-0608LAB

COMMENTS: Enclosed are the results for the subject project.

James T. Keegan, MD
Corrosion and Lab Services Section Manager

Table 1 - Laboratory Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0608LAB
24-Sep-20

Sample ID			B-5 SK-A @ 0-7.5'	B-13 SK-1
Resistivity		**Units**		
as-received		ohm-cm	760,000	>4,000,000
minimum		ohm-cm	11,600	3,960
pH			8.7	8.0
Electrical Conductivity		mS/cm	0.06	0.07
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	63	58
magnesium	Mg^{2+}	mg/kg	8.3	7.7
sodium	Na^{1+}	mg/kg	9.7	13
potassium	K^{1+}	mg/kg	23	15
Anions				
carbonate	CO_3^{2-}	mg/kg	45	ND
bicarbonate	HCO_3^{1-}	mg/kg	58	113
fluoride	F^{1-}	mg/kg	2.1	1.7
chloride	Cl^{1-}	mg/kg	3.7	6.9
sulfate	SO_4^{2-}	mg/kg	11	17
phosphate	PO_4^{3-}	mg/kg	ND	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	18	143
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Minimum resistivity per CTM 643, Chlorides per CTM 422, Sulfates per CTM 417

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

Table 1 - Laboratory Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0649LAB
9-Oct-20

Sample ID			B-1 SK-1 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	160,000
minimum		ohm-cm	4,800
pH			8.1
Electrical Conductivity		mS/cm	0.09
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	58
magnesium	Mg^{2+}	mg/kg	7.8
sodium	Na^{1+}	mg/kg	23
potassium	K^{1+}	mg/kg	15
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	156
fluoride	F^{1-}	mg/kg	2.1
chloride	Cl^{1-}	mg/kg	8.5
sulfate	SO_4^{2-}	mg/kg	55
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	68
sulfide	S^{2-}	qual	na
Redox		mV	na

Minimum resistivity per CTM 643, Chlorides per CTM 422, Sulfates per CTM 417

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

HDR

TRANSMITTAL LETTER

DATE: September 24, 2020

ATTENTION: **Fernando Cuenca**

TO: Tetra Tech
1360 Valley Vista Drive
Diamond Bar, CA 91765

SUBJECT: Laboratory Test Data
Mountain View I & II Wind Turbine
Your #GEN 20-33, HDR Lab #20-0608LAB

COMMENTS: Enclosed are the results for the subject project.

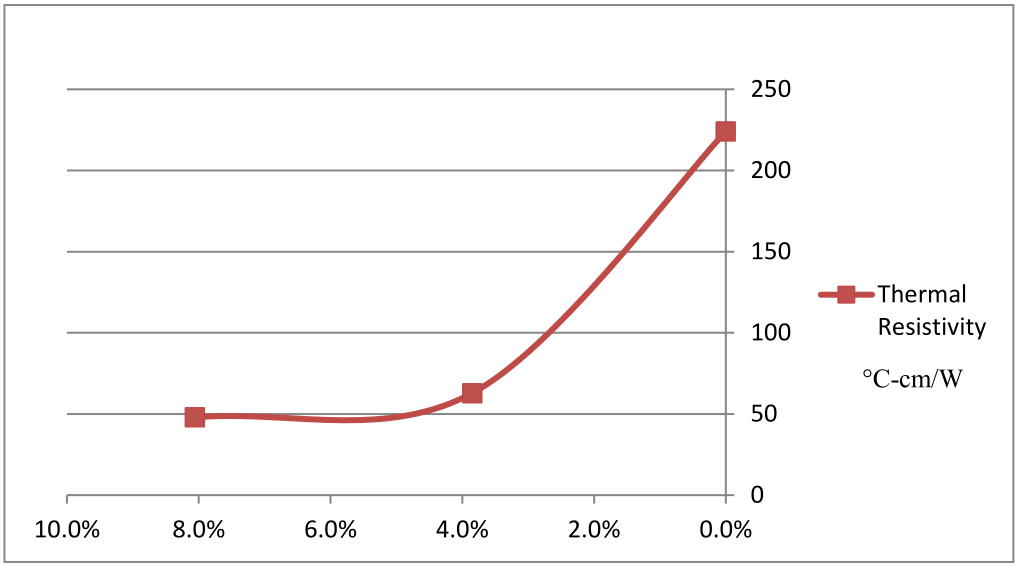
James T. Keegan, MD
Corrosion and Lab Services Section Manager

Table 2 - Thermal Resistivity Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0608LAB
24-Sep-20

Sample ID B-12 SK-1 @ 0-5'

Thermal Resistivity	**Units**	
	°C-cm/W	% Moisture
	47.9	8.1%
	62.8	3.9%
	224	0.0%

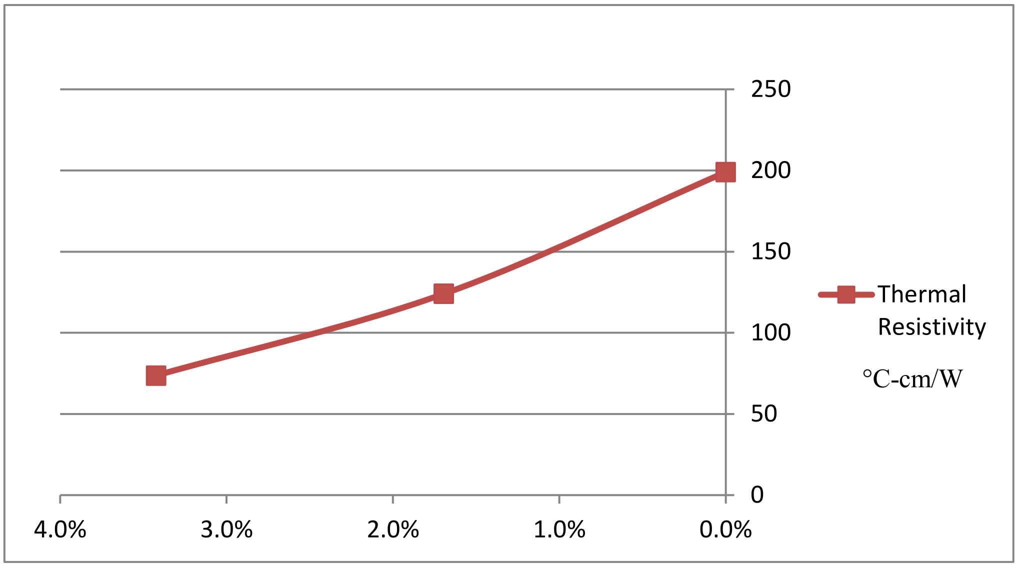


Thermal resistivity determined per ASTM D5334
°C-cm/W = degrees centigrade x centimeters per watt

Table 2 - Thermal Resistivity Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0608LAB
24-Sep-20

Sample ID		B-12 R-2 @ 2.5-4'
Thermal Resistivity	**Units**	
	°C-cm/W	% Moisture
	73.7	3.4%
	124	1.7%
	199	0.0%

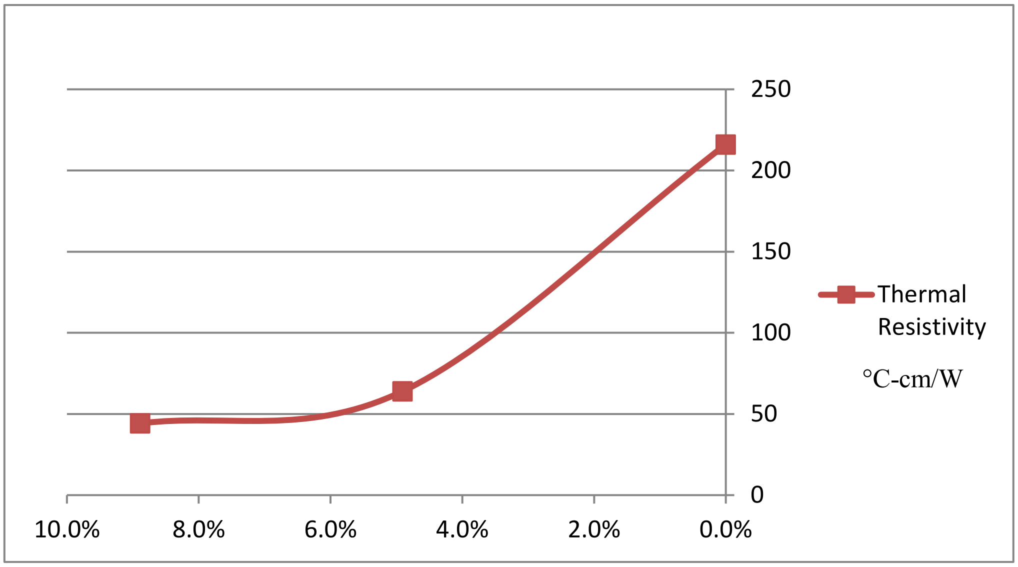


Thermal resistivity determined per ASTM D5334
°C-cm/W = degrees centigrade x centimeters per watt

Table 2 - Thermal Resistivity Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0608LAB
24-Sep-20

Sample ID		B-94 SK-1 @ 0-5'
Thermal Resistivity	**Units**	
	°C-cm/W	% Moisture
	44.2	8.9%
	63.9	4.9%
	216	0.0%

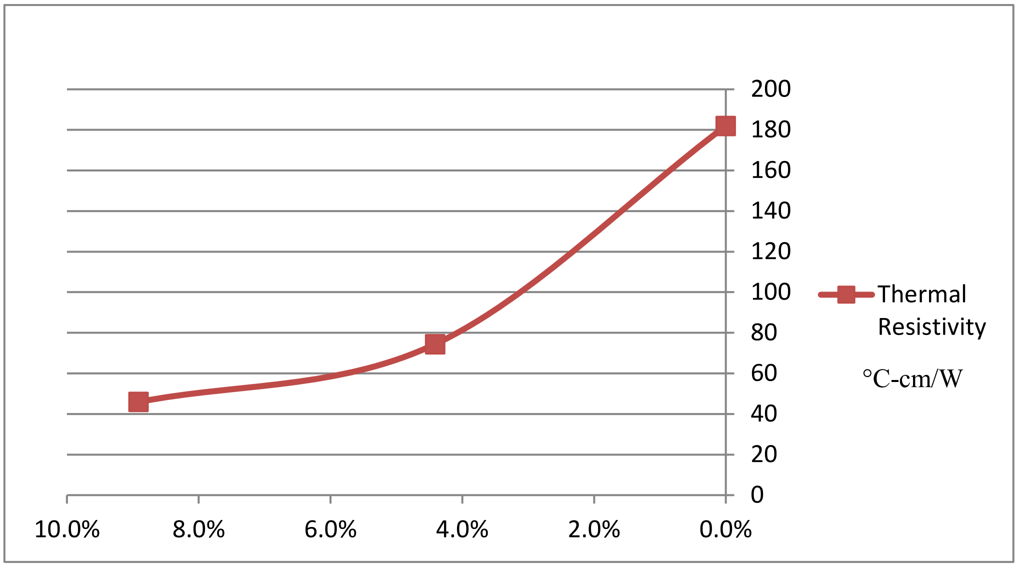


Thermal resistivity determined per ASTM D5334
°C-cm/W = degrees centigrade x centimeters per watt

Table 2 - Thermal Resistivity Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0608LAB
24-Sep-20

Sample ID		B-95 SK-1 @ 0-5'
Thermal Resistivity	**Units**	
	°C-cm/W	% Moisture
	45.9	8.9%
	74.4	4.4%
	182	0.0%

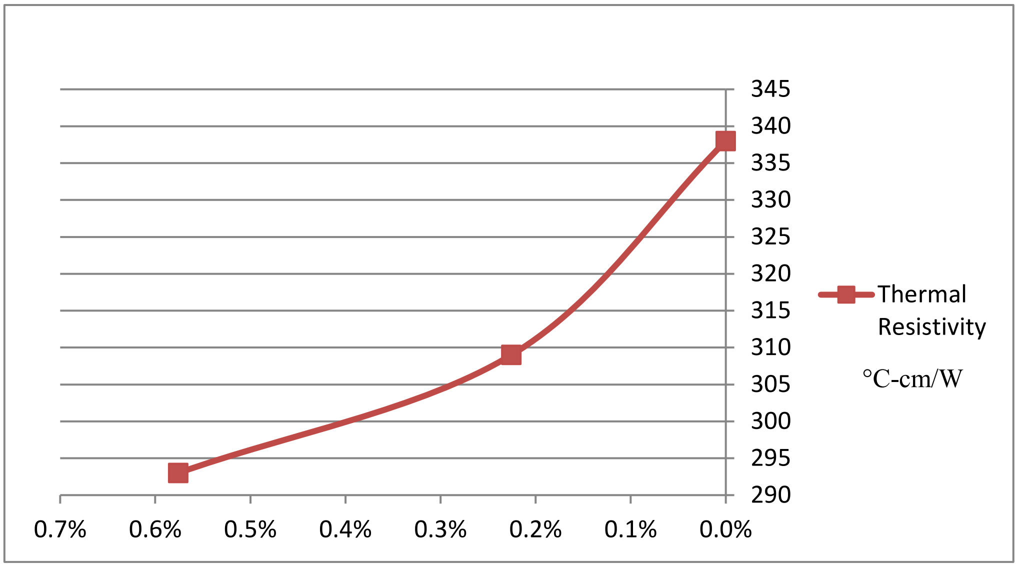


Thermal resistivity determined per ASTM D5334
°C-cm/W = degrees centigrade x centimeters per watt

Table 2 - Thermal Resistivity Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0608LAB
24-Sep-20

Sample ID		B-95 R-3 @ 5-6.5'
Thermal Resistivity	**Units**	
	°C-cm/W	% Moisture
	293	0.6%
	309	0.2%
	338	0.0%



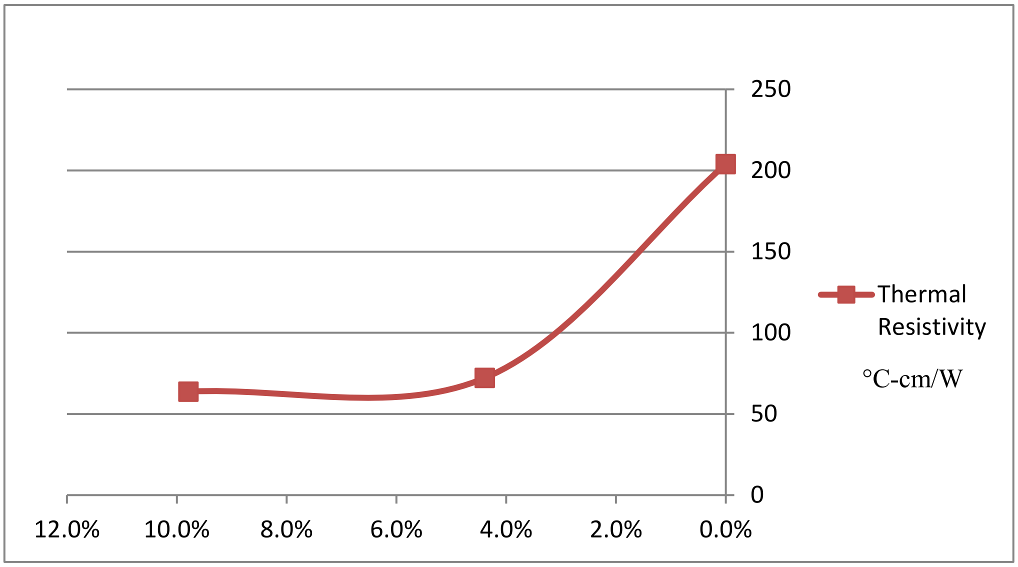
Thermal resistivity determined per ASTM D5334
°C-cm/W = degrees centigrade x centimeters per watt

Table 2 - Thermal Resistivity Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0608LAB
24-Sep-20

Sample ID	B-97 SK-1 @ 0-5'

Thermal Resistivity	**Units**	
	°C-cm/W	% Moisture
	63.8	9.8%
	72.2	4.4%
	204	0.0%



Thermal resistivity determined per ASTM D5334
°C-cm/W = degrees centigrade x centimeters per watt

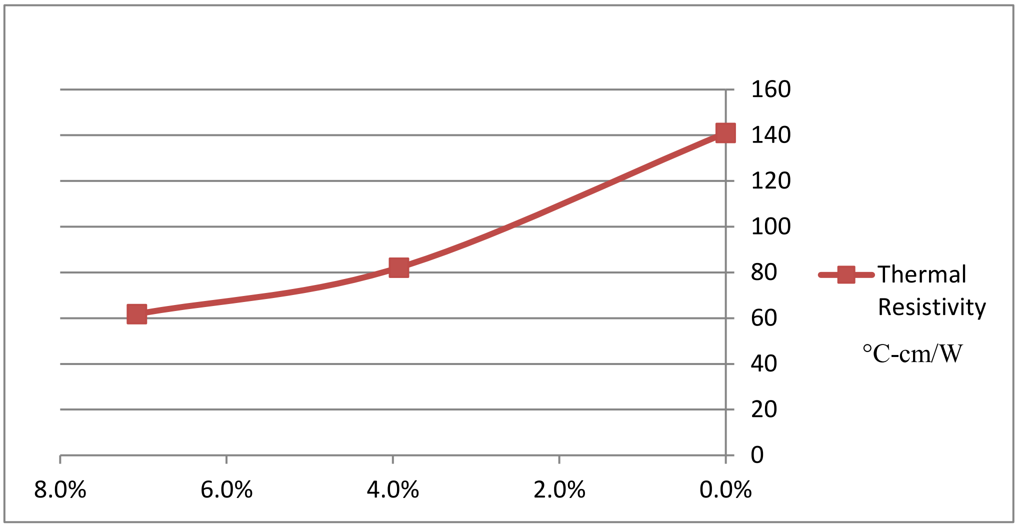
Table 2 - Thermal Resistivity Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0608LAB
24-Sep-20

Sample ID

B-5 SK-A @ 0-5'

Thermal	**Units**	
	°C-cm/W	% Moisture
	61.8	7.1%
	82.1	3.9%
	141	0.0%



Thermal resistivity determined per ASTM D5334
°C-cm/W = degrees centigrade x centimeters per watt

TRANSMITTAL LETTER

DATE: October 9, 2020

ATTENTION: **Fernando Cuenca**

TO:

Tetra Tech
1360 Valley Vista Drive
Diamond Bar, CA 91765

SUBJECT:

Laboratory Test Data
Mountain View I & II Wind Turbine
Your #GEN 20-33, HDR Lab #20-0649LAB

COMMENTS: Enclosed are the results for the subject project.

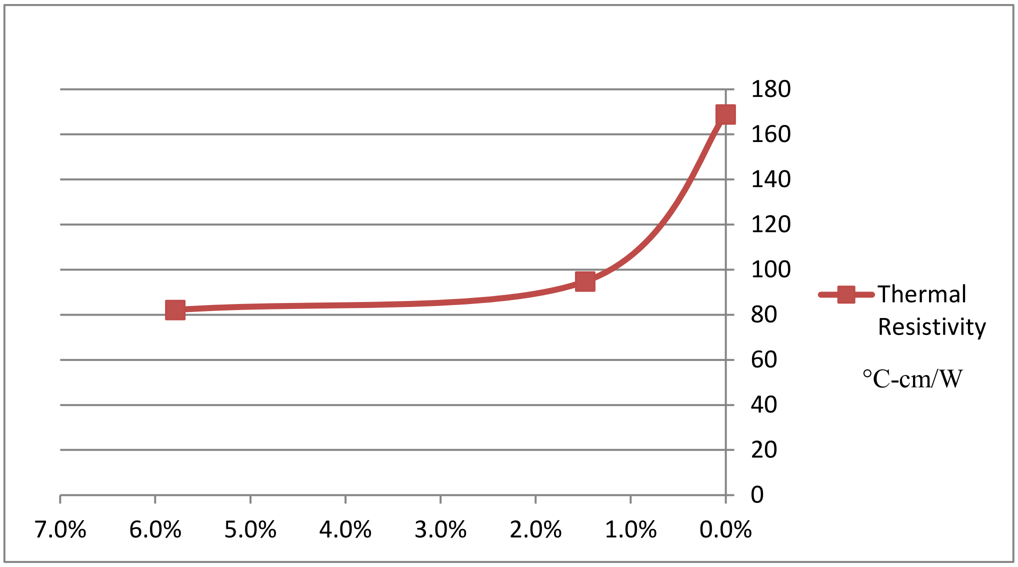
James T. Keegan, MD
Corrosion and Lab Services Section Manager

Table 2 - Thermal Resistivity Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0649LAB
17-Oct-18

Sample ID	B-3 SK-1 @ 0-5'

Thermal Resistivity	**Units**	
	°C-cm/W	% Moisture
	82.14	5.8%
	94.8	1.5%
	168.8	0.0%



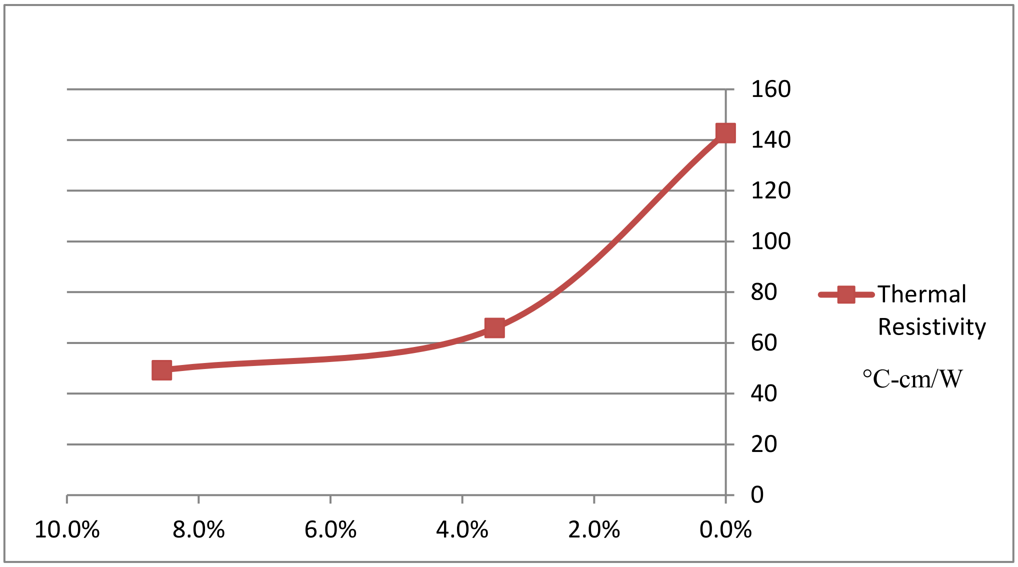
Thermal resistivity determined per ASTM D5334
°C-cm/W = degrees centigrade x centimeters per watt

Table 2 - Thermal Resistivity Tests on Soil Samples

Tetra Tech
Mountain View I & II Wind Turbine Repowering
Your #GEN 20-33, HDR Lab #20-0649LAB
17-Oct-18

Sample ID B-5 SK-1 @ 0-5'

Thermal Resistivity	**Units**	
	°C-cm/W	% Moisture
	49.23	8.6%
	65.89	3.5%
	142.8	0.0%



Thermal resistivity determined per ASTM D5334
°C-cm/W = degrees centigrade x centimeters per watt

Appendix E

Seismic Demand

Mountain View

Latitude, Longitude: 33.910926, -116.603786

Date	9/10/2020, 6:14:14 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	2.373	MCE_R ground motion. (for 0.2 second period)
S_1	0.962	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.847	Site-modified spectral acceleration value
S_{M1}	1.347	Site-modified spectral acceleration value
S_{DS}	1.898	Numeric seismic design value at 0.2 second SA
S_{D1}	0.898	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	E	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.4	Site amplification factor at 1.0 second
PGA	1.023	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.228	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.373	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.652	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.381	Factored deterministic acceleration value. (0.2 second)
S1RT	0.962	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.093	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.014	Factored deterministic acceleration value. (1.0 second)
PGAd	1.023	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.895	Mapped value of the risk coefficient at short periods
C_{R1}	0.88	Mapped value of the risk coefficient at a period of 1 s

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (v4.2.0)

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.910926

Time Horizon

Return period in years

2475

Longitude

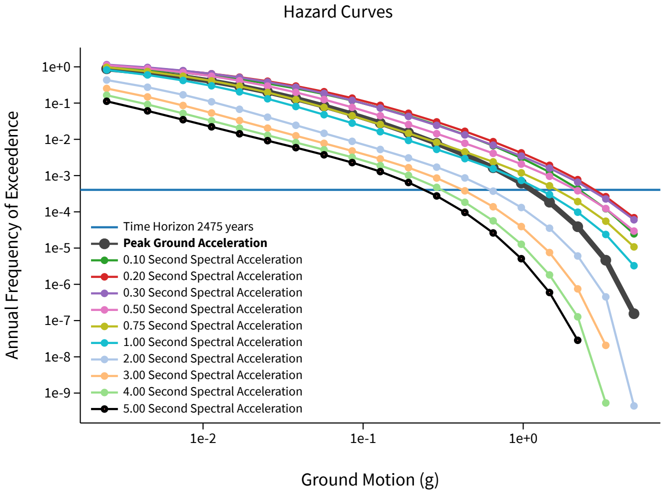
Decimal degrees, negative values for western longitudes

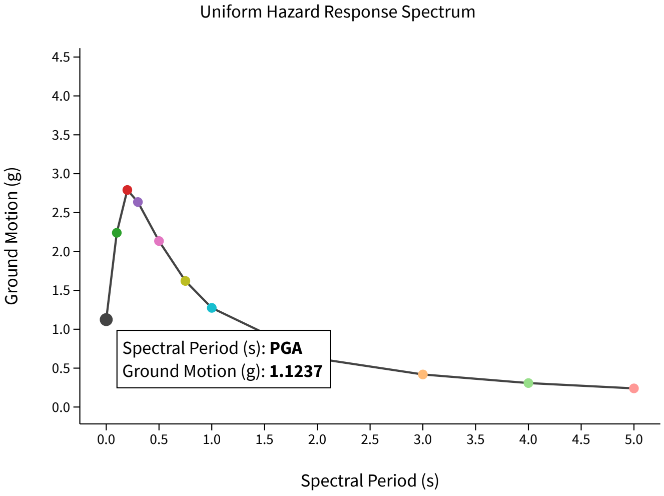
-116.603786

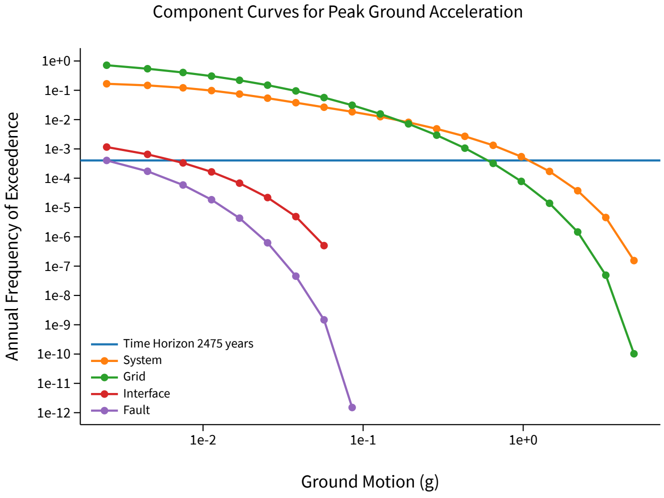
Site Class

537 m/s (Site class C)

^ Hazard Curve





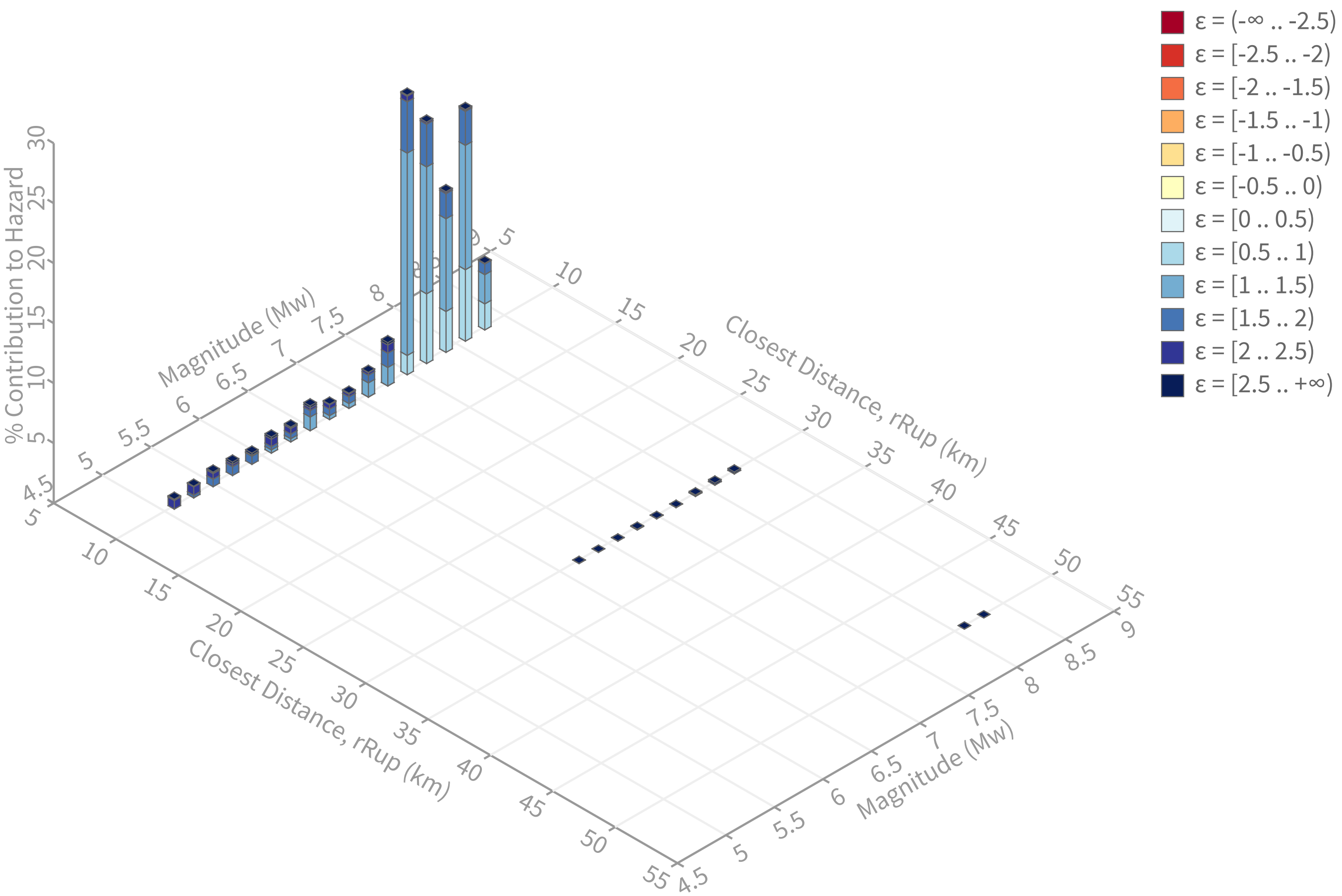


View Raw Data

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 1.1236659 g

Recovered targets

Return period: 2894.4607 yrs
Exceedance rate: 0.0003454875 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.02 %

Mean (over all sources)

m: 7.56
r: 3.05 km
ε_0: 1.3 σ

Mode (largest m-r bin)

m: 7.51
r: 2.37 km
ε_0: 1.23 σ
Contribution: 23.3 %

Mode (largest m-r-ε_0 bin)

m: 7.52
r: 1.67 km
ε_0: 1.11 σ
Contribution: 16.86 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							45.43
San Andreas (San Gorgonio Pass-Garnet HIll) [5]		1.60	7.77	1.13	116.598°W	33.918°N	32.19	37.67
San Andreas (San Gorgonio Pass-Garnet HIll) [4]		2.00	7.40	1.21	116.588°W	33.915°N	71.06	2.08
San Andreas (North Branch Mill Creek) [8]		10.08	7.92	1.67	116.537°W	33.978°N	39.76	1.98
San Andreas (San Gorgonio Pass-Garnet HIll) [6]		6.05	7.33	1.71	116.657°W	33.940°N	303.18	1.49
UC33brAvg_FM31	System							45.19
San Andreas (San Gorgonio Pass-Garnet HIll) [5]		1.60	7.77	1.13	116.598°W	33.918°N	32.19	37.78
San Andreas (North Branch Mill Creek) [8]		10.08	7.93	1.67	116.537°W	33.978°N	39.76	1.96
San Andreas (San Gorgonio Pass-Garnet HIll) [4]		2.00	7.40	1.21	116.588°W	33.915°N	71.06	1.81
San Andreas (San Gorgonio Pass-Garnet HIll) [6]		6.05	7.34	1.71	116.657°W	33.940°N	303.18	1.40
UC33brAvg_FM31 (opt)	Grid							4.69
PointSourceFinite: -116.604, 33.924		5.17	5.73	1.84	116.604°W	33.924°N	0.00	1.55
PointSourceFinite: -116.604, 33.924		5.17	5.73	1.84	116.604°W	33.924°N	0.00	1.55
UC33brAvg_FM32 (opt)	Grid							4.69
PointSourceFinite: -116.604, 33.924		5.17	5.73	1.84	116.604°W	33.924°N	0.00	1.55
PointSourceFinite: -116.604, 33.924		5.17	5.73	1.84	116.604°W	33.924°N	0.00	1.54

Appendix F

Seismic Settlement

Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-1**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method		
Correction for fines content	Idriss & Boulang. 2008, 2014	
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs	
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014	
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014	
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014	
Magnitude scaling factor MSF	Idriss & Boulang. 2014	
Dry settlement	Pradel, 1998a,b	
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration	

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	61.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	14	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.00**	
Dry sand settlement	0.13	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.13**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	5	117.3	117.3	3	n/plastic	99.0	126.4	126.4	–	–	- no groundwater	0.03	0.13
5	5	117.3	117.3	3	n/plastic	99.0	134.6	134.6	–	–	- no groundwater	0.03	0.11
10	2	117.3	117.3	3	n/plastic	99.0	154.2	154.2	–	–	- no groundwater	0.01	0.08
12	3	117.3	117.3	3	n/plastic	99.0	159.0	159.0	–	–	- no groundwater	0.01	0.07
15	5	117.3	117.3	3	n/plastic	99.0	186.2	186.2	–	–	- no groundwater	0.01	0.06
20	5	117.3	117.3	3	n/plastic	99.0	200.9	200.9	–	–	- no groundwater	0.01	0.05
25	5	117.3	117.3	3	n/plastic	99.0	217.6	217.6	–	–	- no groundwater	0.01	0.04
30	5	117.3	117.3	3	n/plastic	99.0	258.1	258.1	–	–	- no groundwater	0.01	0.03
35	5	117.3	117.3	3	n/plastic	99.0	291.6	291.6	–	–	- no groundwater	0.01	0.02
40	10	117.3	117.3	3	n/plastic	99.0	334.7	334.7	–	–	- no groundwater	0.01	0.02
50	5	117.3	117.3	3	n/plastic	99.0	334.6	334.6	–	–	- no groundwater	0.00	0.01
55	5	117.3	117.3	3	n/plastic	99.0	334.7	334.7	–	–	- no groundwater	0.00	0.00
60	1.4	117.3	117.3	3	n/plastic	99.0	334.6	334.6	–	–	- no groundwater	0.00	0.00
61.4	0.1	117.3	117.3	3	n/plastic	99.0	334.6	334.6	2.00	–	- too dense – (N1)60,CS > 32	0.00	0.00
418.40	61.50	1642.20	1642.20	42.00	0.00	1386.18	3401.91	3401.91	2.00	0.00		0.13	0.62

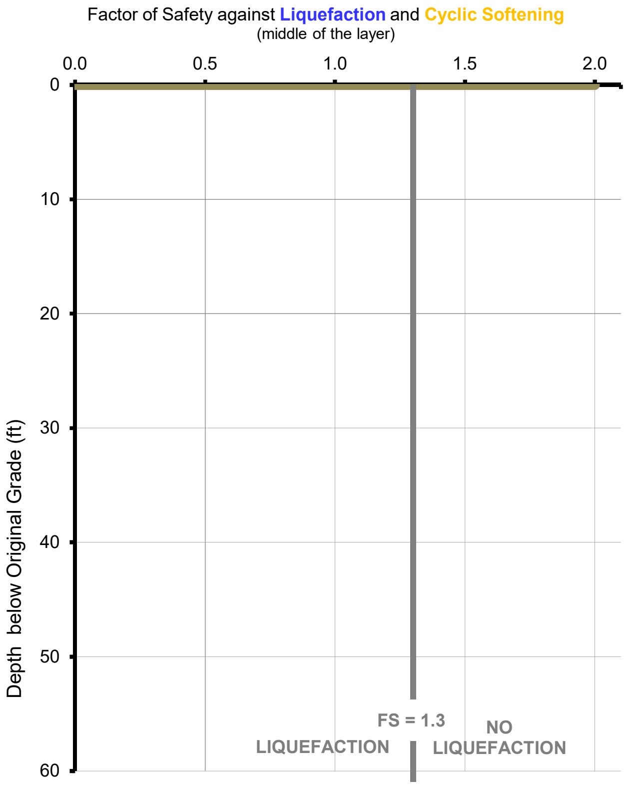
Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	61.40	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

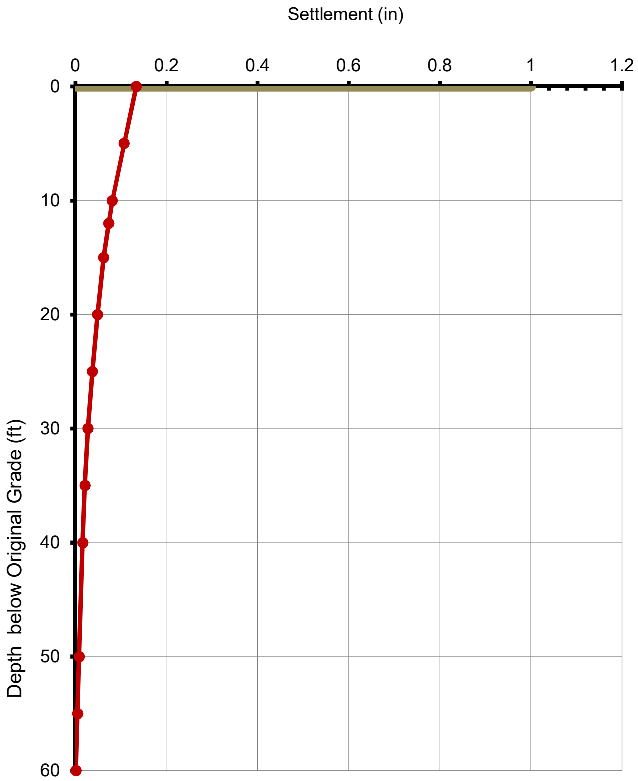
Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

Version v2 2018-07

B-1 Design excavation depth 0.00 feet
Design groundwater depth 61.40 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-2**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	36.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	9	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.17**	
Dry sand settlement	0.10	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.10**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	117.3	117.3	3	n/plastic	99.0	139.5	139.5	–	–	- no groundwater	0.04	0.10
10	2	117.3	117.3	3	n/plastic	99.0	167.3	167.3	–	–	- no groundwater	0.01	0.06
12	3	117.3	117.3	3	n/plastic	66.0	121.2	121.2	–	–	- no groundwater	0.02	0.05
15	5	117.3	117.3	3	n/plastic	99.0	205.6	205.6	–	–	- no groundwater	0.01	0.03
20	2	117.3	117.3	3	n/plastic	99.0	219.1	219.1	–	–	- no groundwater	0.00	0.02
22	3	117.3	117.3	3	n/plastic	99.0	229.7	229.7	–	–	- no groundwater	0.01	0.02
25	5	117.3	117.3	3	n/plastic	99.0	248.9	248.9	–	–	- no groundwater	0.01	0.01
30	5	117.3	117.3	3	n/plastic	99.0	303.9	303.9	–	–	- no groundwater	0.00	0.00
35	1.5	117.3	117.3	3	n/plastic	99.0	336.1	336.1	2.17	–	- too dense – (N1)60,CS > 32	0.00	0.00
169.00	36.50	1055.70	1055.70	27.00	0.00	858.10	1971.36	1971.36	2.17	0.00		0.10	0.30

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	35.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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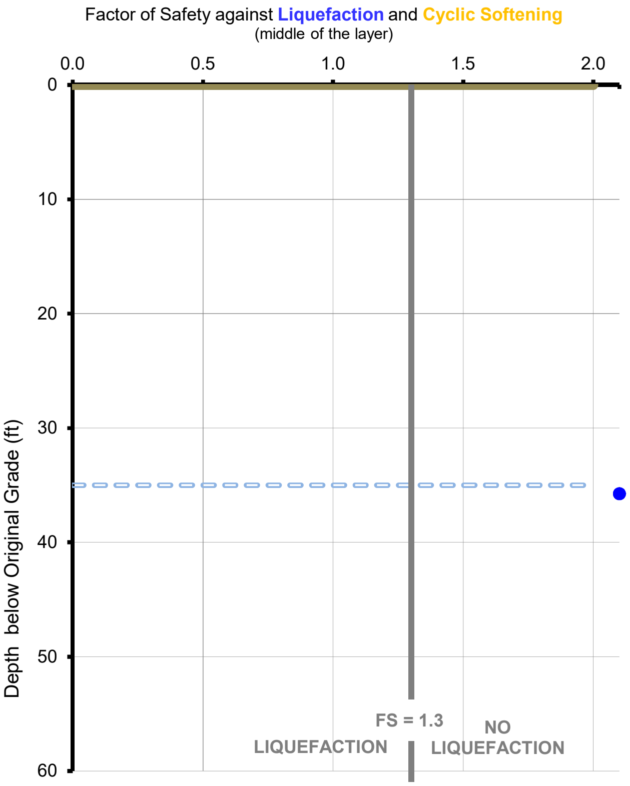
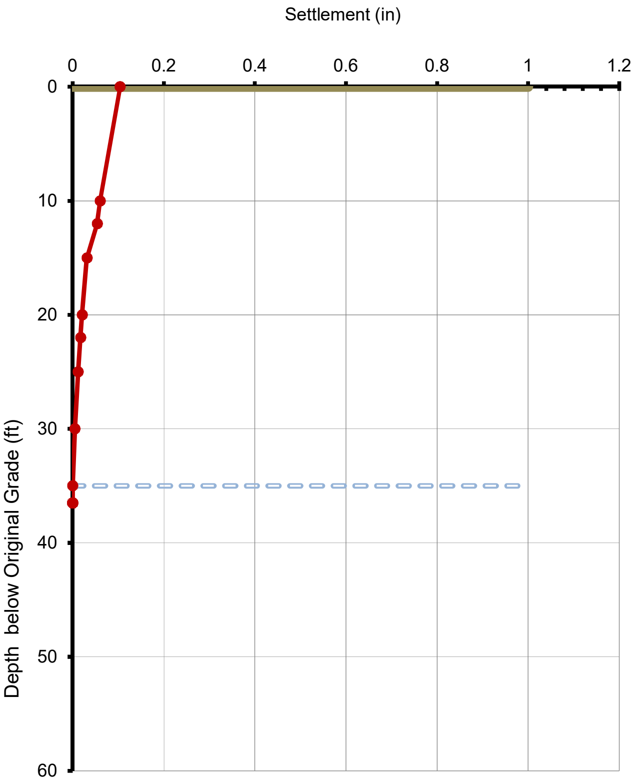
B-2 Design excavation depth 0.00 feet
Design groundwater depth 35.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-3**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	40	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	9	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.14**	
Dry sand settlement	0.12	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.12**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	5	117.3	117.3	3	n/plastic	99.0	126.4	126.4	–	–	- no groundwater	0.03	0.12
5	5	117.3	117.3	3	n/plastic	99.0	134.6	134.6	–	–	- no groundwater	0.03	0.09
10	5	117.3	117.3	3	n/plastic	99.0	157.1	157.1	–	–	- no groundwater	0.02	0.06
15	5	117.3	117.3	3	n/plastic	99.0	186.2	186.2	–	–	- no groundwater	0.01	0.05
20	5	117.3	117.3	3	n/plastic	99.0	200.8	200.8	–	–	- no groundwater	0.01	0.03
25	5	117.3	117.3	3	n/plastic	99.0	217.7	217.7	–	–	- no groundwater	0.01	0.02
30	5	117.3	117.3	3	n/plastic	99.0	258.0	258.0	–	–	- no groundwater	0.01	0.01
35	4.5	117.3	117.3	3	n/plastic	99.0	289.7	289.7	–	–	- no groundwater	0.00	0.00
39.5	0.5	117.3	117.3	3	n/plastic	99.0	311.7	311.7	2.14	–	- too dense – (N1)60,CS > 32	0.00	0.00
179.50	40.00	1055.70	1055.70	27.00	0.00	891.16	1882.19	1882.19	2.14	0.00		0.12	0.39

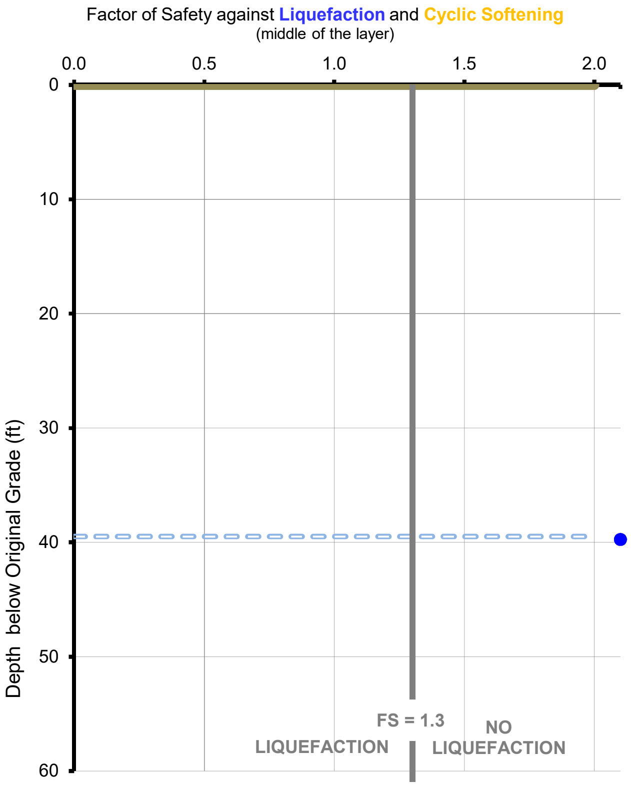
Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	39.50	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

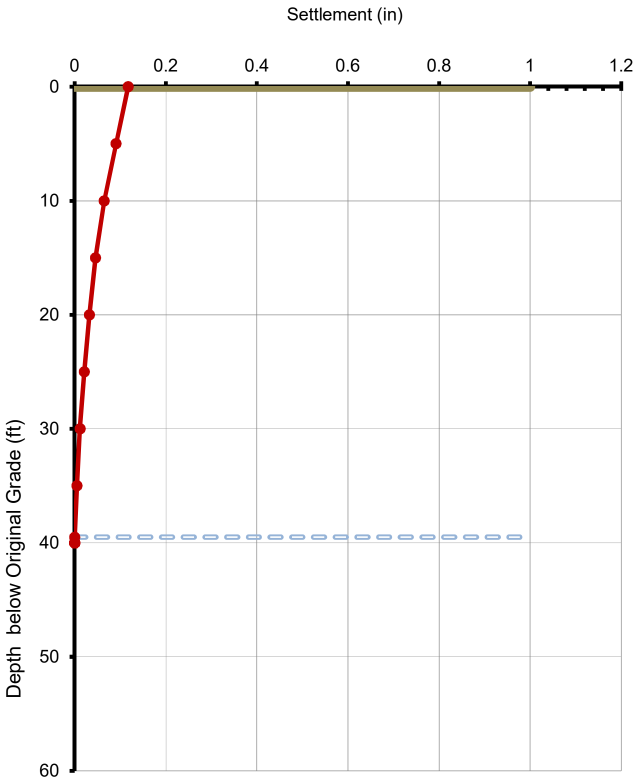
Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

Version v2 2018-07

B-3 Design excavation depth 0.00 feet
Design groundwater depth 39.50 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-4**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	31.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	6	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.27**	
Dry sand settlement	0.11	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.11**	**inches**

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	112.5525	112.6	3	n/plastic	99.0	130.6	130.6	–	–	- no groundwater	0.05	0.11
10	5	112.5525	112.6	3	n/plastic	99.0	156.1	156.1	–	–	- no groundwater	0.02	0.05
15	5	112.5525	112.6	3	n/plastic	99.0	184.2	184.2	–	–	- no groundwater	0.01	0.03
20	5	112.5525	112.6	3	n/plastic	99.0	198.0	198.0	–	–	- no groundwater	0.01	0.02
25	5	112.5525	112.6	3	n/plastic	99.0	213.7	213.7	–	–	- no groundwater	0.01	0.01
30	1.5	112.5525	112.6	3	n/plastic	99.0	242.3	242.3	2.27	–	- too dense – (N1)60,CS > 32	0.00	0.00
100.00	31.50	675.32	675.32	18.00	0.00	594.08	1125.00	1125.00	2.27	0.00		0.11	0.23

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	30.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

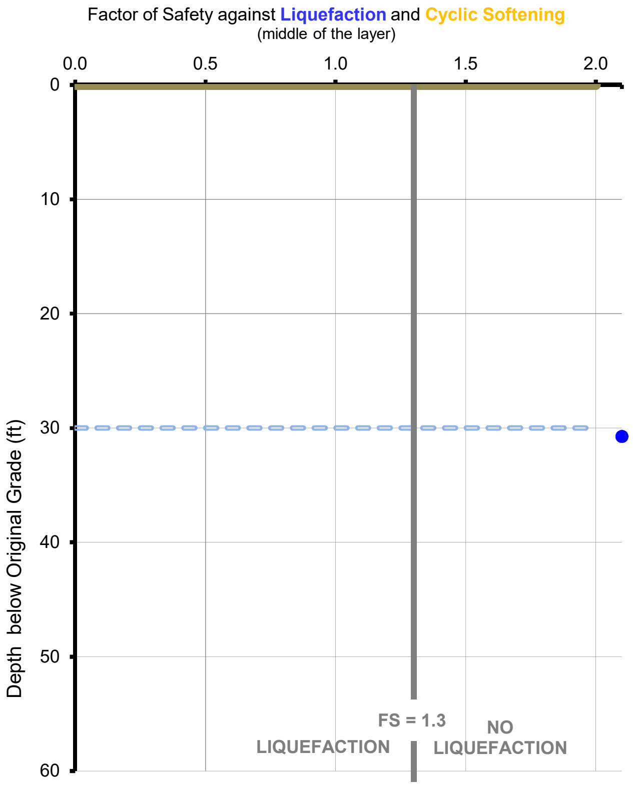
Earthquake loading	
M	7.56
PGA	1.228

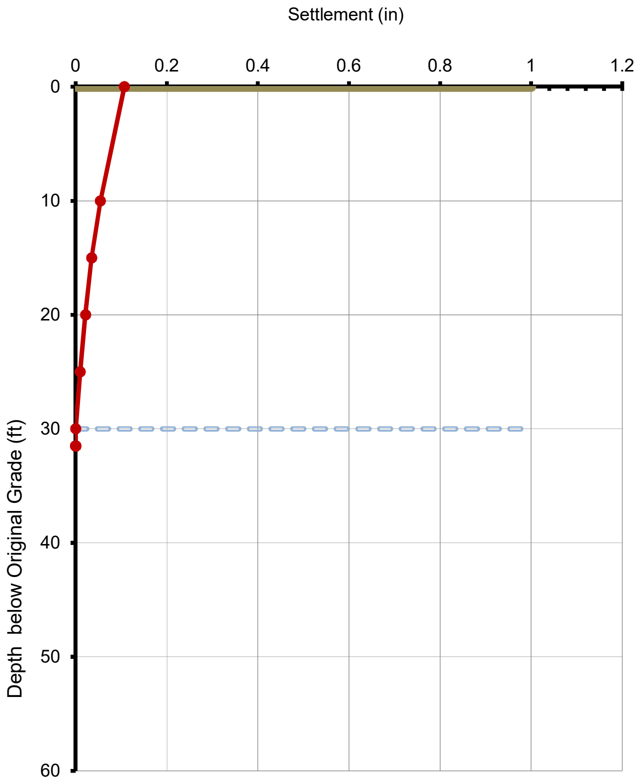
Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

Version v2 2018-07

B-4

Design excavation depth 0.00 feet
Design groundwater depth 30.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-5**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction Analysis Statistics	
Total thickness of evaluated profile	41.5 feet
Profile thickness susceptible to liquefaction	0 feet
Number of evaluated intervals	8
Number of potentially liquefiable intervals	1
Average Factor of Safety of sandy intervals	**2.12**
Dry sand settlement	0.12 inches
Liquefaction settlement	0.00 inches
Total earthquake-induced settlement	**0.12 inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	117.3	117.3	3	n/plastic	99.0	130.9	130.9	–	–	- no groundwater	0.05	0.12
10	5	117.3	117.3	3	n/plastic	99.0	157.1	157.1	–	–	- no groundwater	0.02	0.06
15	5	117.3	117.3	3	n/plastic	99.0	186.2	186.2	–	–	- no groundwater	0.01	0.05
20	5	117.3	117.3	3	n/plastic	99.0	200.9	200.9	–	–	- no groundwater	0.01	0.03
25	5	117.3	117.3	3	n/plastic	99.0	217.6	217.6	–	–	- no groundwater	0.01	0.02
30	5	117.3	117.3	3	n/plastic	99.0	258.1	258.1	–	–	- no groundwater	0.01	0.01
35	5	117.3	117.3	3	n/plastic	99.0	291.6	291.6	–	–	- no groundwater	0.01	0.01
40	1.5	117.3	117.3	3	n/plastic	99.0	322.0	322.0	2.12	–	- too dense – (N1)60,CS > 32	0.00	0.00
175.00	41.50	938.40	938.40	24.00	0.00	792.13	1764.41	1764.41	2.12	0.00		0.12	0.30

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	40.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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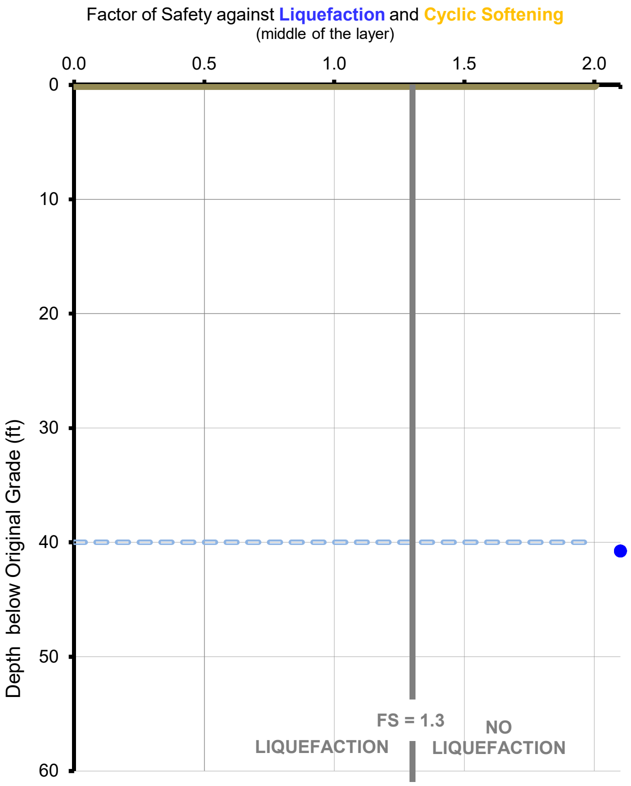
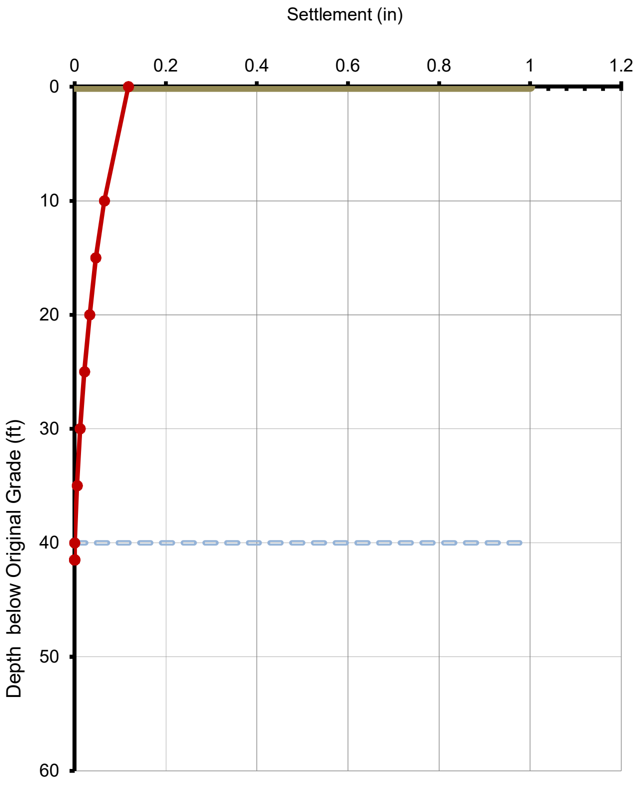
B-5	Design excavation depth	0.00 feet
	Design groundwater depth	40.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-6**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method		
Correction for fines content	Idriss & Boulang. 2008, 2014	
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs	
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014	
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014	
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014	
Magnitude scaling factor MSF	Idriss & Boulang. 2014	
Dry settlement	Pradel, 1998a,b	
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration	

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	41.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	8	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.12**	
Dry sand settlement	0.13	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.13**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	117.3	117.3	3	n/plastic	99.0	130.9	130.9	–	–	- no groundwater	0.05	0.13
10	5	117.3	117.3	3	n/plastic	99.0	157.1	157.1	–	–	- no groundwater	0.02	0.08
15	5	117.3	117.3	3	n/plastic	99.0	186.2	186.2	–	–	- no groundwater	0.01	0.06
20	5	117.3	117.3	3	n/plastic	99.0	200.9	200.9	–	–	- no groundwater	0.01	0.04
25	5	117.3	117.3	3	n/plastic	96.0	209.0	209.0	–	–	- no groundwater	0.01	0.03
30	5	117.3	117.3	3	n/plastic	99.0	258.1	258.1	–	–	- no groundwater	0.01	0.02
35	5	117.3	117.3	3	n/plastic	74.0	174.8	174.8	–	–	- no groundwater	0.02	0.02
40	1.5	117.3	117.3	3	n/plastic	99.0	322.0	322.0	2.12	–	- too dense – (N1)60,CS > 32	0.00	0.00
175.00	41.50	938.40	938.40	24.00	0.00	764.13	1639.00	1639.00	2.12	0.00		0.13	0.37

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	40.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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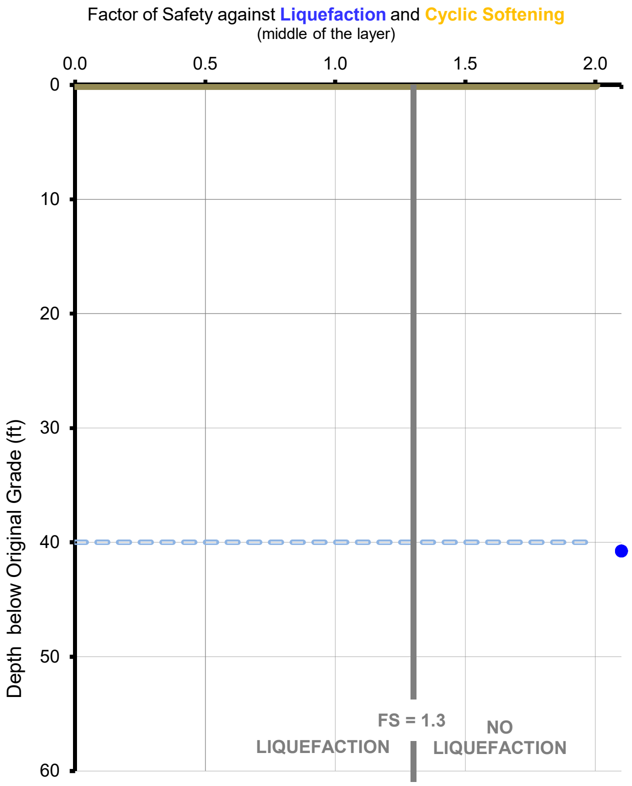
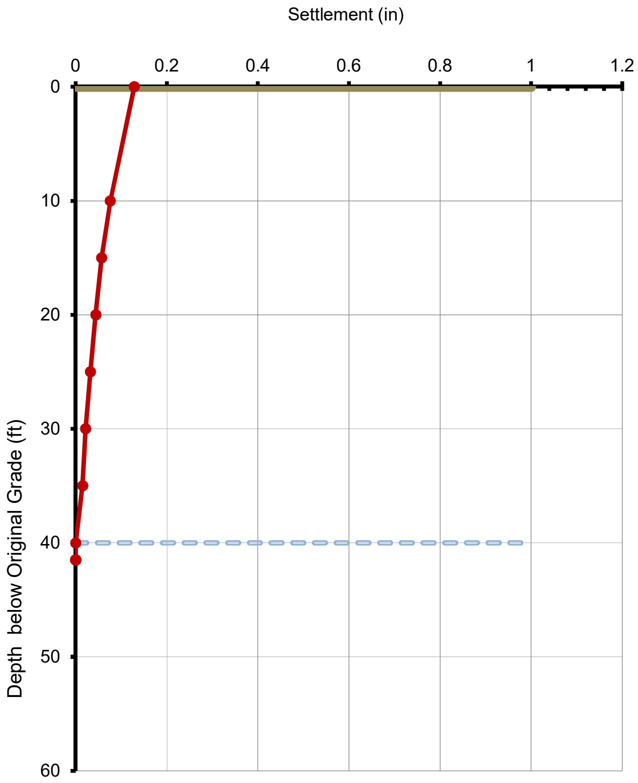
B-6

Design excavation depth 0.00 feet
Design groundwater depth 40.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-7**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method

Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Liquefaction Analysis Statistics

Total thickness of evaluated profile	41.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	7	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.12**	
Dry sand settlement	0.12	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.12**	**inches**

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Total Unit Weight Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	Considered Blowcounts N1,60	Considered Blowcounts N1,60,cs	Factor of Safety Liquefaction	Factor of Safety Cyclic softening	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	117.3	117.3	3	n/plastic	99.0	130.9	130.9	–	–	- no groundwater	0.05	0.12
10	5	117.3	117.3	3	n/plastic	99.0	157.1	157.1	–	–	- no groundwater	0.02	0.06
15	5	117.3	117.3	3	n/plastic	99.0	186.2	186.2	–	–	- no groundwater	0.01	0.05
20	10	117.3	117.3	3	n/plastic	99.0	208.9	208.9	–	–	- no groundwater	0.02	0.03
30	5	117.3	117.3	3	n/plastic	99.0	258.0	258.0	–	–	- no groundwater	0.01	0.01
35	5	117.3	117.3	3	n/plastic	99.0	291.7	291.7	–	–	- no groundwater	0.01	0.01
40	1.5	117.3	117.3	3	n/plastic	99.0	321.8	321.8	2.12	–	- too dense – (N1)60,CS > 32	0.00	0.00
150.00	41.50	821.10	821.10	21.00	0.00	693.10	1554.70	1554.70	2.12	0.00		0.12	0.28

Profile

In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	40.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading

M	7.56
PGA	1.228

Checks

Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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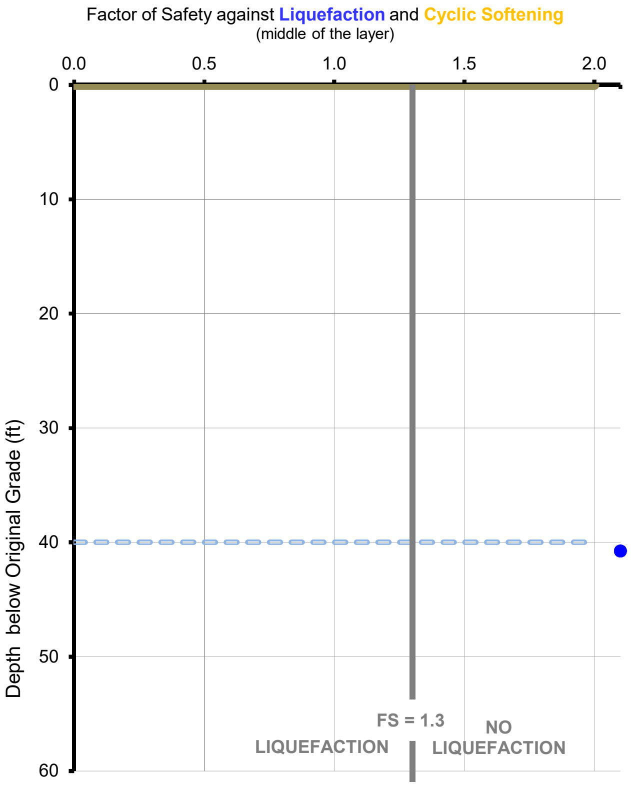
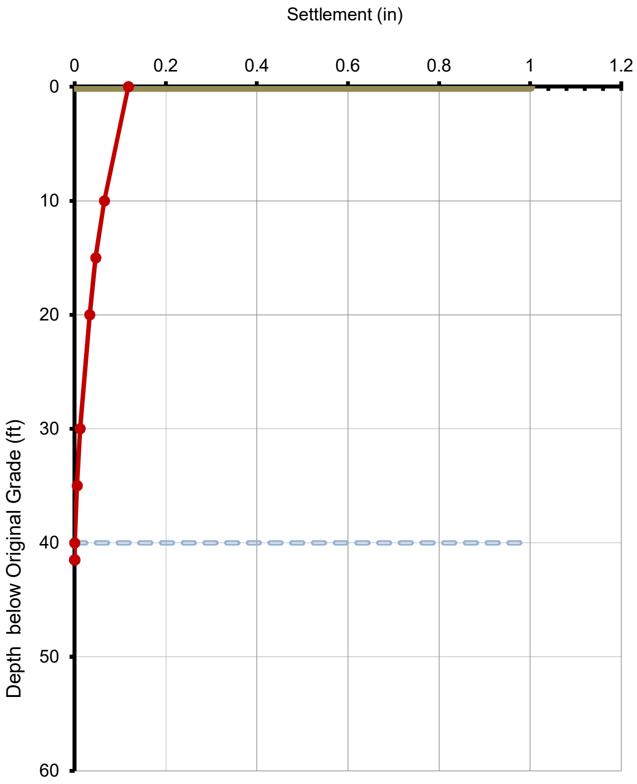
B-7

Design excavation depth	0.00	feet
Design groundwater depth	40.00	feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-8**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	41.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	8	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.12**	
Dry sand settlement	0.13	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.13**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	N1,60	N1,60,cs	Factor of Safety Liquefaction	Cyclic softening	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	8	117.3	117.3	3	n/plastic	99.0	129.3	129.3	–	–	- no groundwater	0.04	0.13
8	7	117.3	117.3	3	n/plastic	99.0	155.2	155.2	–	–	- no groundwater	0.03	0.08
15	5	117.3	117.3	3	n/plastic	99.0	186.2	186.2	–	–	- no groundwater	0.01	0.06
20	5	117.3	117.3	3	n/plastic	99.0	200.9	200.9	–	–	- no groundwater	0.01	0.04
25	5	117.3	117.3	3	n/plastic	99.0	217.6	217.6	–	–	- no groundwater	0.01	0.03
30	5	117.3	117.3	3	n/plastic	99.0	258.1	258.1	–	–	- no groundwater	0.01	0.02
35	5	117.3	117.3	3	n/plastic	72.0	167.3	167.3	–	–	- no groundwater	0.02	0.02
40	1.5	117.3	117.3	3	n/plastic	99.0	322.0	322.0	2.12	–	- too dense – (N1)60,CS > 32	0.00	0.00
173.00	41.50	938.40	938.40	24.00	0.00	765.13	1636.60	1636.60	2.12	0.00		0.13	0.39

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	40.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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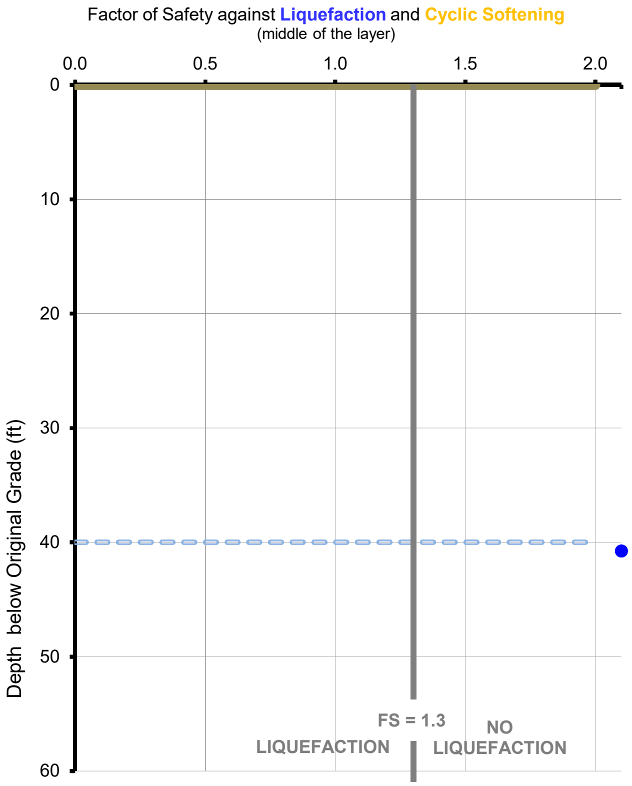
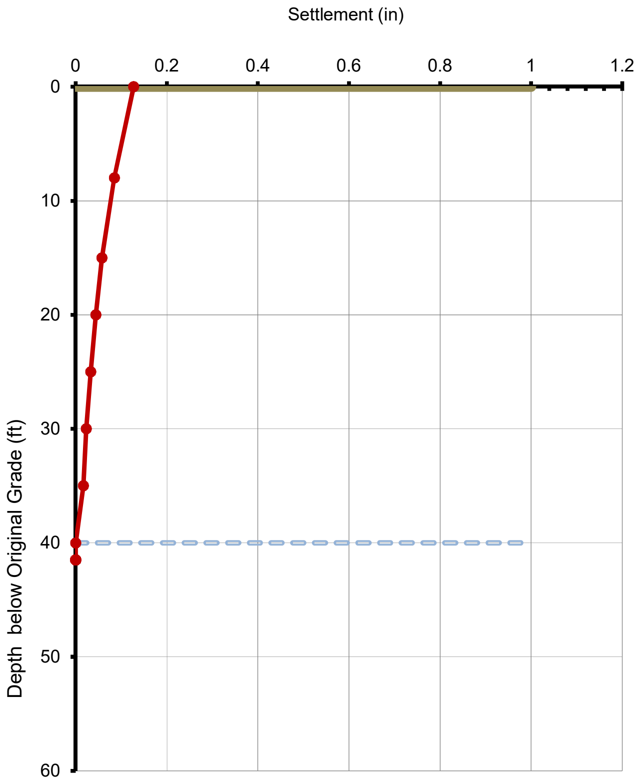
B-8	Design excavation depth	0.00	feet
	Design groundwater depth	40.00	feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-9**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	27	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	7	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.31**	
Dry sand settlement	0.08	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.08**	**inches**

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	N1,60	N1,60,cs	Factor of Safety Liquefaction	Cyclic softening	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	117.3	117.3	3	n/plastic	99.0	139.5	139.5	–	–	- no groundwater	0.04	0.08
10	2	117.3	117.3	3	n/plastic	99.0	167.3	167.3	–	–	- no groundwater	0.01	0.03
12	3	117.3	117.3	3	n/plastic	99.0	173.6	173.6	–	–	- no groundwater	0.01	0.03
15	5	117.3	117.3	3	n/plastic	99.0	205.6	205.6	–	–	- no groundwater	0.01	0.02
20	2	117.3	117.3	3	n/plastic	99.0	219.1	219.1	–	–	- no groundwater	0.00	0.01
22	3	117.3	117.3	3	n/plastic	99.0	229.7	229.7	–	–	- no groundwater	0.01	0.01
25	2	117.3	117.3	3	n/plastic	99.0	241.3	241.3	2.31	–	- too dense – (N1)60,CS > 32	0.00	0.00
104.00	27.00	821.10	821.10	21.00	0.00	693.08	1376.15	1376.15	2.31	0.00		0.08	0.17

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	25.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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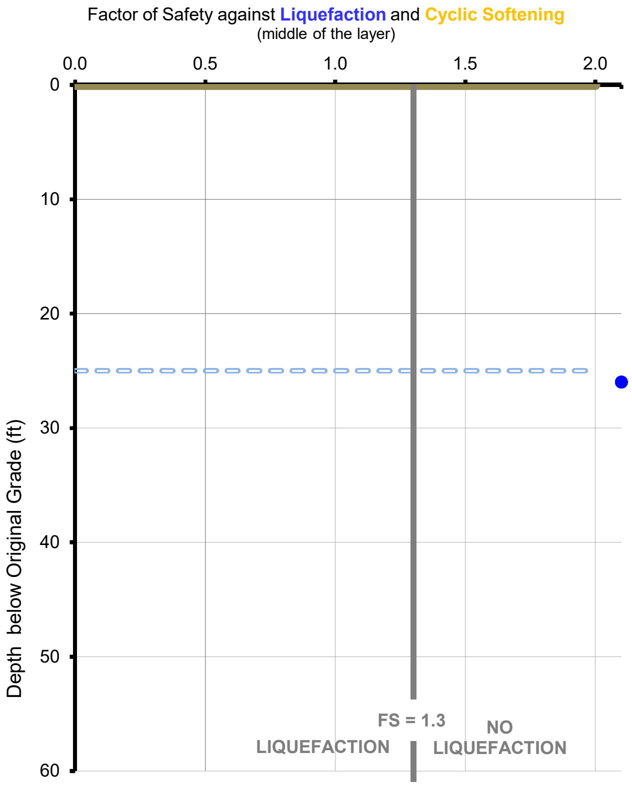
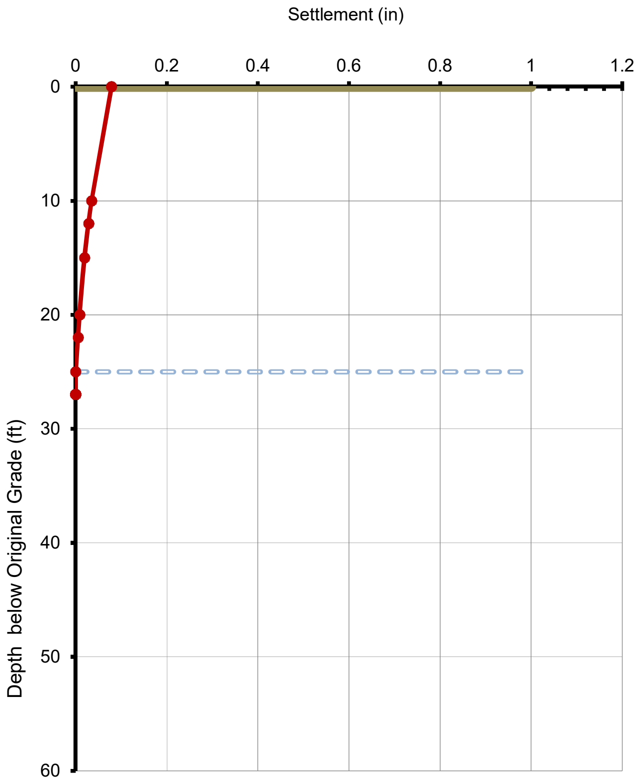
Design excavation depth	0.00	feet
Design groundwater depth	25.00	feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-10**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method		
Correction for fines content	Idriss & Boulang. 2008, 2014	
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs	
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014	
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014	
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014	
Magnitude scaling factor MSF	Idriss & Boulang. 2014	
Dry settlement	Pradel, 1998a,b	
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration	

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	31.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	6	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.24**	
Dry sand settlement	0.11	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.11**	**inches**

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Total Unit Weight Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	Considered Blowcounts N1,60	Considered Blowcounts N1,60,cs	Factor of Safety Liquefaction	Factor of Safety Cyclic softening	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	117.3	117.3	3	n/plastic	99.0	130.9	130.9	–	–	- no groundwater	0.05	0.11
10	5	117.3	117.3	3	n/plastic	99.0	157.1	157.1	–	–	- no groundwater	0.02	0.05
15	5	117.3	117.3	3	n/plastic	99.0	186.2	186.2	–	–	- no groundwater	0.01	0.03
20	5	117.3	117.3	3	n/plastic	99.0	200.9	200.9	–	–	- no groundwater	0.01	0.02
25	5	117.3	117.3	3	n/plastic	99.0	217.6	217.6	–	–	- no groundwater	0.01	0.01
30	1.5	117.3	117.3	3	n/plastic	99.0	248.6	248.6	2.24	–	- too dense – (N1)60,CS > 32	0.00	0.00
100.00	31.50	703.80	703.80	18.00	0.00	594.10	1141.34	1141.34	2.24	0.00		0.11	0.22

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	30.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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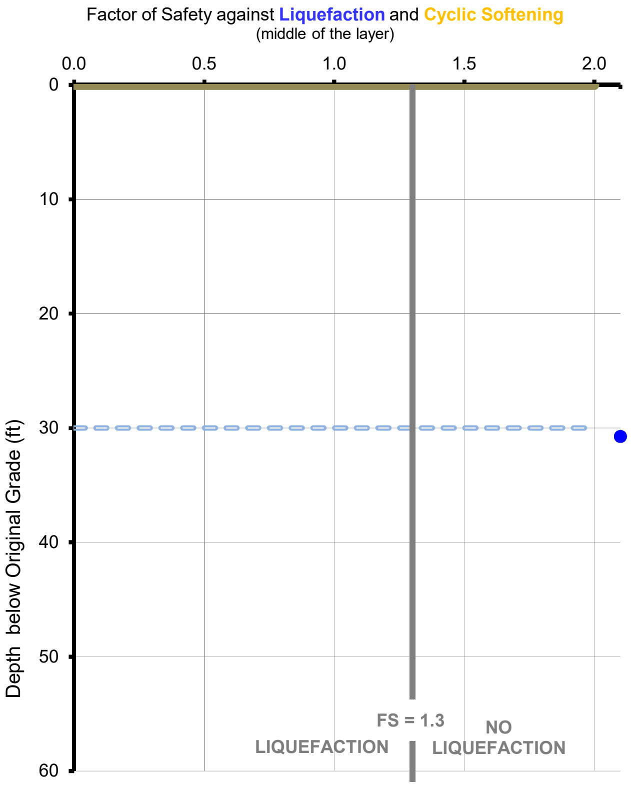
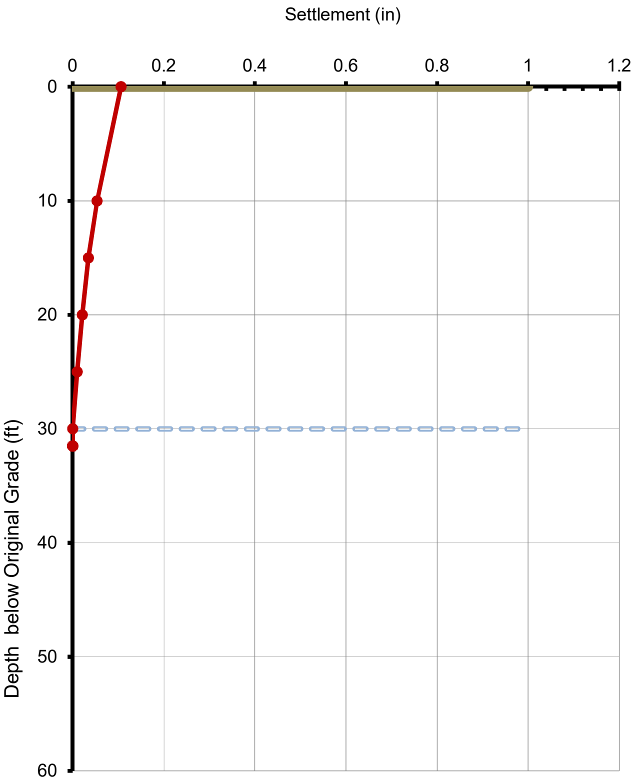
B-10

Design excavation depth 0.00 feet
Design groundwater depth 30.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-11**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method		
Correction for fines content	Idriss & Boulang. 2008, 2014	
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs	
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014	
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014	
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014	
Magnitude scaling factor MSF	Idriss & Boulang. 2014	
Dry settlement	Pradel, 1998a,b	
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration	

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	41.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	8	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.23**	
Dry sand settlement	0.18	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.18**	**inches**

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	102.919	102.9	3	n/plastic	99.0	129.9	129.9	–	–	- no groundwater	0.05	0.18
10	5	102.919	102.9	3	n/plastic	99.0	154.2	154.2	–	–	- no groundwater	0.02	0.13
15	5	102.919	102.9	3	n/plastic	74.0	137.4	137.4	–	–	- no groundwater	0.03	0.11
20	5	102.919	102.9	3	n/plastic	99.0	192.6	192.6	–	–	- no groundwater	0.01	0.08
25	5	102.919	102.9	3	n/plastic	58.0	110.3	110.3	–	–	- no groundwater	0.05	0.07
30	5	102.919	102.9	3	n/plastic	99.0	237.7	237.7	–	–	- no groundwater	0.01	0.02
35	5	102.919	102.9	3	n/plastic	72.0	160.9	160.9	–	–	- no groundwater	0.02	0.02
40	1.5	102.919	102.9	3	n/plastic	99.0	278.6	278.6	2.23	–	- too dense – (N1)60,CS > 32	0.00	0.00
175.00	41.50	823.35	823.35	24.00	0.00	699.13	1401.61	1401.61	2.23	0.00		0.18	0.61

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	40.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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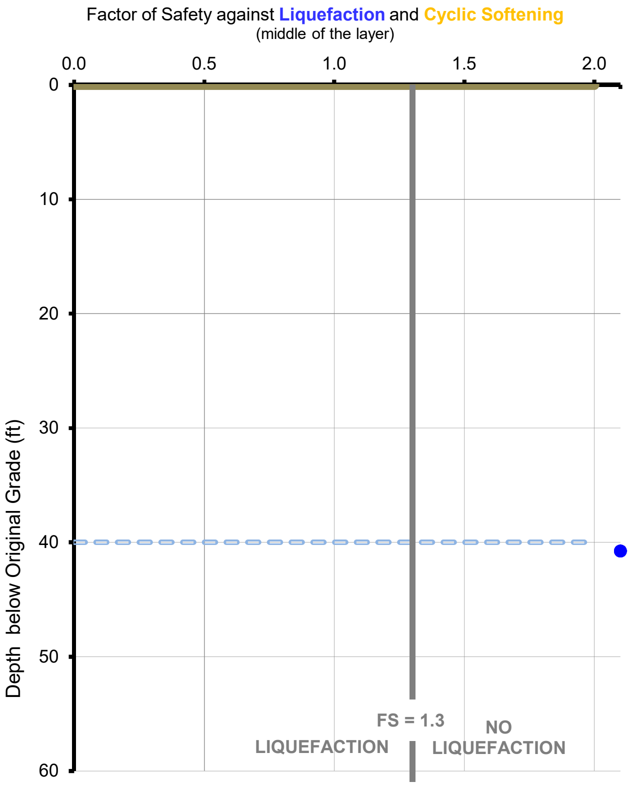
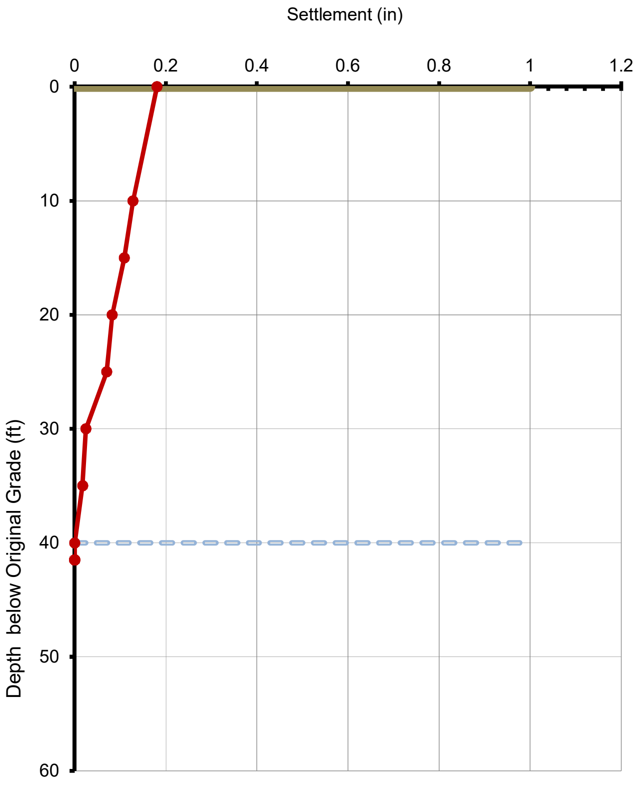
B-11	Design excavation depth	0.00 feet
	Design groundwater depth	40.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-12**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	61.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	15	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.00**	
Dry sand settlement	0.53	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.53**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	5	116.5683	116.6	3	n/plastic	44.2	84.8	84.8	–	–	- no groundwater	0.09	0.53
5	2	116.5683	116.6	3	n/plastic	99.0	141.6	141.6	–	–	- no groundwater	0.01	0.43
7	3	116.5683	116.6	3	n/plastic	99.0	146.4	146.4	–	–	- no groundwater	0.01	0.43
10	2	116.5683	116.6	3	n/plastic	36.0	71.8	71.8	–	–	- no groundwater	0.06	0.41
12	3	116.5683	116.6	3	n/plastic	99.0	173.4	173.4	–	–	- no groundwater	0.01	0.35
15	5	116.5683	116.6	3	n/plastic	31.0	65.0	65.0	–	–	- no groundwater	0.21	0.34
20	5	116.5683	116.6	3	n/plastic	99.0	224.8	224.8	–	–	- no groundwater	0.01	0.13
25	5	116.5683	116.6	3	n/plastic	99.0	248.0	248.0	–	–	- no groundwater	0.01	0.12
30	5	116.5683	116.6	3	n/plastic	99.0	302.1	302.1	–	–	- no groundwater	0.00	0.11
35	5	116.5683	116.6	3	n/plastic	56.0	132.5	132.5	–	–	- no groundwater	0.03	0.10
40	5	116.5683	116.6	3	n/plastic	99.0	370.0	370.0	–	–	- no groundwater	0.00	0.08
45	5	116.5683	116.6	3	n/plastic	42.0	86.1	86.1	–	–	- no groundwater	0.07	0.07
50	5	116.5683	116.6	3	n/plastic	99.0	370.0	370.0	–	–	- no groundwater	0.00	0.01
55	5	116.5683	116.6	3	n/plastic	99.0	369.9	369.9	–	–	- no groundwater	0.00	0.00
60	1.5	116.5683	116.6	3	n/plastic	67.0	246.2	246.2	2.00	–	- too dense – (N1)60,CS > 32	0.00	0.00
409.00	61.50	1748.52	1748.52	45.00	0.00	1167.38	3032.58	3032.58	2.00	0.00		0.53	3.10

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	60.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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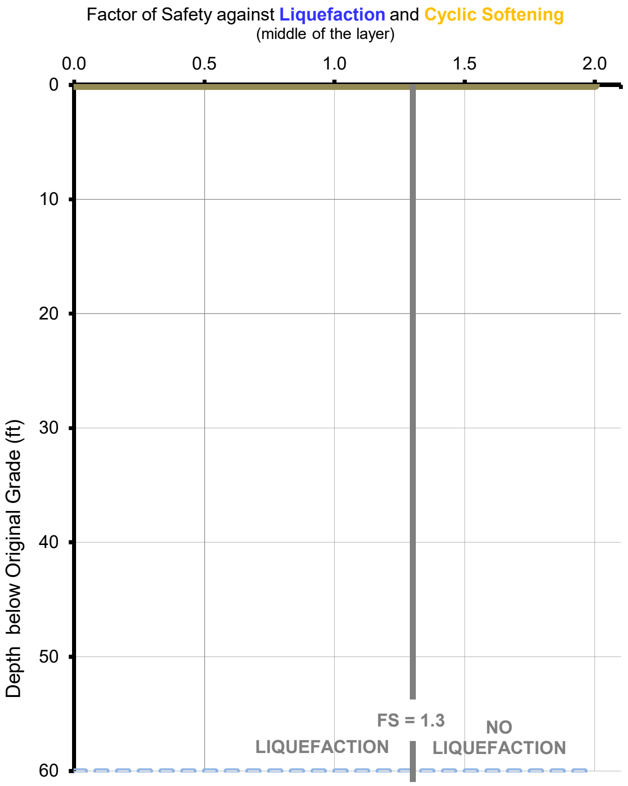
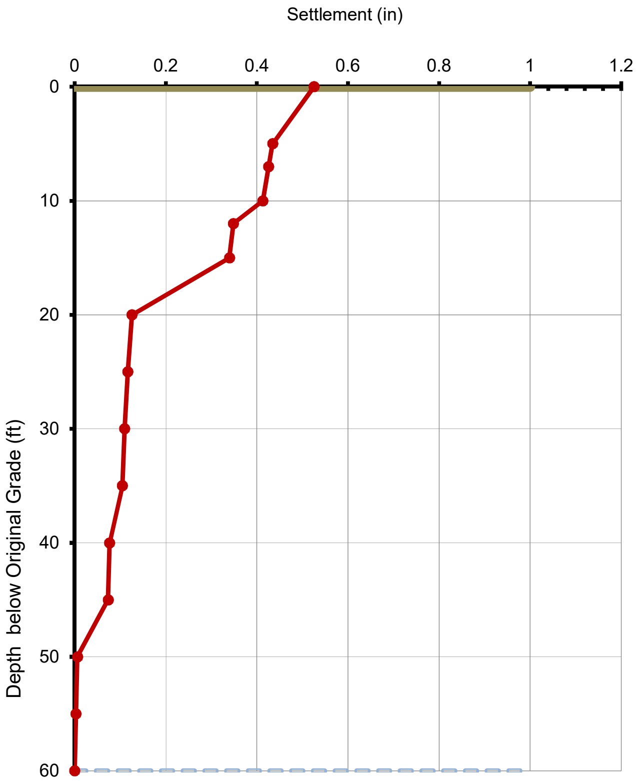
Design excavation depth	0.00	feet
Design groundwater depth	60.00	feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-13**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	41.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	10	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.12**	
Dry sand settlement	0.16	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.16**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	117.0015	117.0	3	n/plastic	88.0	129.4	129.4	–	–	- no groundwater	0.05	0.16
10	2	117.0015	117.0	3	n/plastic	99.0	167.2	167.2	–	–	- no groundwater	0.01	0.10
12	3	117.0015	117.0	3	n/plastic	47.0	89.2	89.2	–	–	- no groundwater	0.05	0.10
15	5	117.0015	117.0	3	n/plastic	99.0	205.4	205.4	–	–	- no groundwater	0.01	0.05
20	2	117.0015	117.0	3	n/plastic	99.0	218.9	218.9	–	–	- no groundwater	0.00	0.03
22	3	121.412	121.4	3	n/plastic	99.0	229.7	229.7	–	–	- no groundwater	0.01	0.03
25	5	121.412	121.4	3	n/plastic	99.0	249.6	249.6	–	–	- no groundwater	0.01	0.03
30	5	116.7124	116.7	3	n/plastic	99.0	305.7	305.7	–	–	- no groundwater	0.00	0.02
35	5	116.7124	116.7	3	n/plastic	68.0	179.6	179.6	–	–	- no groundwater	0.01	0.01
40	1.5	116.2458	116.2	3	n/plastic	26.8	47.7	47.7	2.12	–	- too dense – (N1)60,CS > 32	0.00	0.00
209.00	41.50	1177.50	1177.50	30.00	0.00	823.90	1822.54	1822.54	2.12	0.00		0.16	0.53

Profile			Earthquake loading		Checks	
In-Situ Groundwater depth	no GW	feet	M	7.56	Groundwater depth check	OK
DESIGN Groundwater depth	40.00	feet	PGA	1.228	Design groundwater/excavation depth check	OK
DESIGN Excavation depth	0.00	feet			Fines correction method compatibility	OK
DESIGN Surcharge (fill)	0.00	feet			Idris & Boulanger, 2004 method for C_N	not used
					Cetin 2009 settlement method	not used

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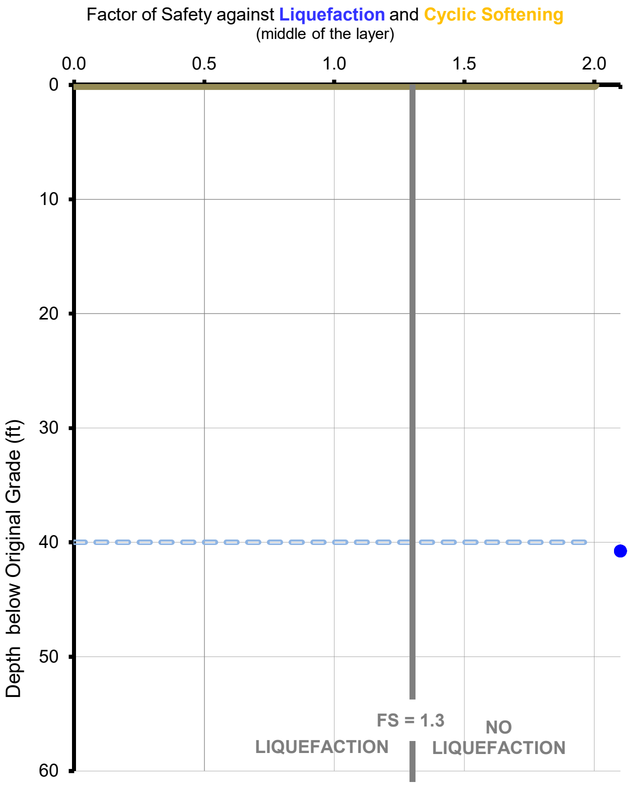
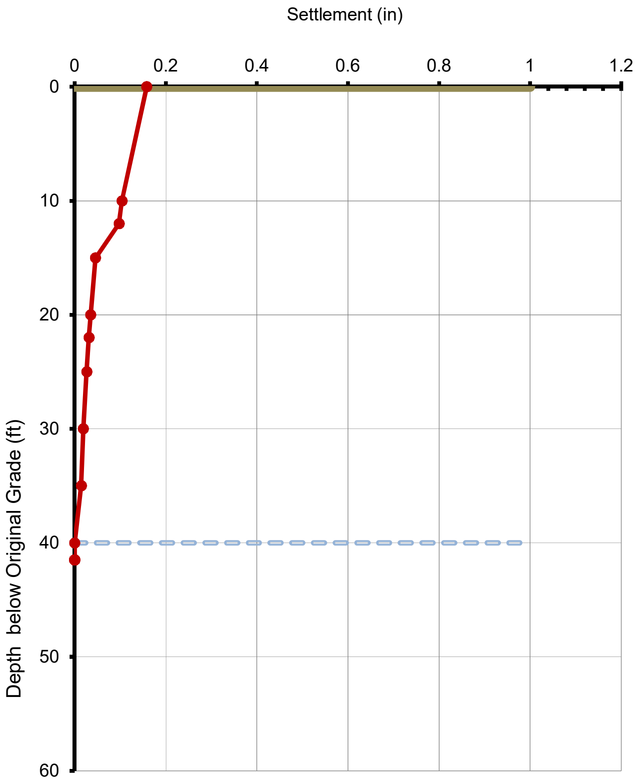
B-13

Design excavation depth 0.00 feet
Design groundwater depth 40.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-14**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	41.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	11	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.10**	
Dry sand settlement	0.62	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.62**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Total Unit Weight Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	Considered Blowcounts N1,60	Considered Blowcounts N1,60,cs	Factor of Safety Liquefaction	Factor of Safety Cyclic softening	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	5	126.1522	126.2	3	n/plastic	99.0	133.9	133.9	–	–	- no groundwater	0.02	0.62
5	2	126.1522	126.2	3	n/plastic	26.0	53.7	53.7	–	–	- no groundwater	0.16	0.59
7	3	126.1522	126.2	3	n/plastic	99.0	147.7	147.7	–	–	- no groundwater	0.01	0.43
10	2	126.1522	126.2	3	n/plastic	31.0	62.3	62.3	–	–	- no groundwater	0.10	0.42
12	3	116.503	116.5	3	n/plastic	42.2	80.5	80.5	–	–	- no groundwater	0.07	0.32
15	5	116.503	116.5	3	n/plastic	86.0	180.9	180.9	–	–	- no groundwater	0.01	0.25
20	5	116.503	116.5	3	n/plastic	46.9	97.2	97.2	–	–	- no groundwater	0.07	0.23
25	5	116.503	116.5	3	n/plastic	36.0	70.5	70.5	–	–	- no groundwater	0.16	0.16
30	5	118.4498	118.4	3	n/plastic	99.0	311.1	311.1	–	–	- no groundwater	0.00	0.01
35	5	118.4498	118.4	3	n/plastic	99.0	369.9	369.9	–	–	- no groundwater	0.00	0.00
40	1.5	118.4498	118.4	3	n/plastic	67.0	182.7	182.7	2.10	–	- too dense – (N1)60,CS > 32	0.00	0.00
199.00	41.50	1325.97	1325.97	33.00	0.00	731.19	1690.36	1690.36	2.10	0.00		0.62	3.03

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	40.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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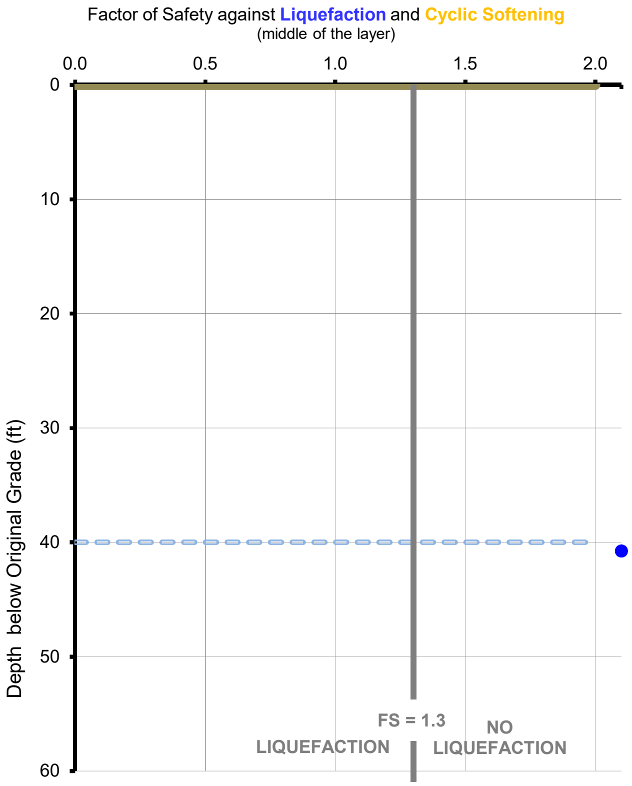
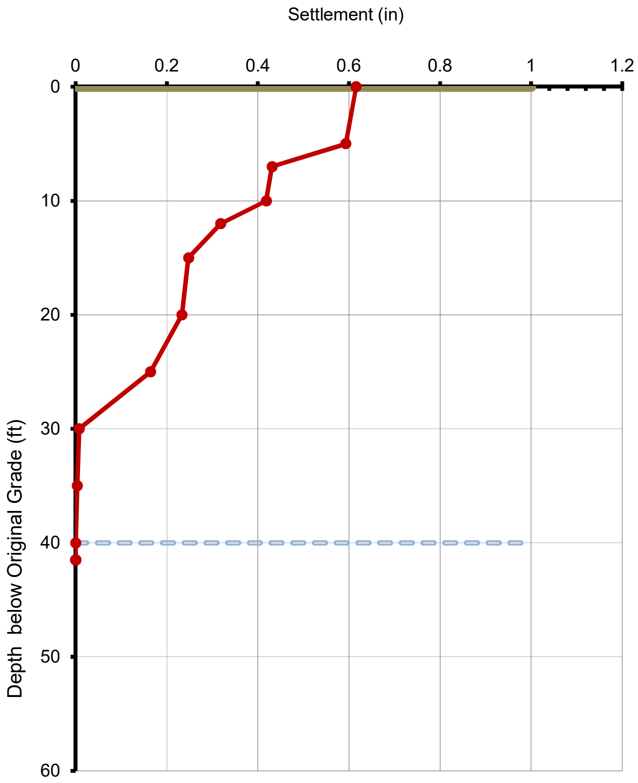
Design excavation depth	0.00 feet
Design groundwater depth	40.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-15**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	41.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	10	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.08**	
Dry sand settlement	0.87	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**0.87**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	10	129.032	129.0	3	n/plastic	99.0	140.6	140.6	–	–	- no groundwater	0.04	0.87
10	2	129.032	129.0	3	n/plastic	99.0	170.1	170.1	–	–	- no groundwater	0.01	0.83
12	3	129.032	129.0	3	n/plastic	19.0	37.6	37.6	–	–	- no groundwater	0.76	0.82
15	5	129.032	129.0	3	n/plastic	99.0	212.2	212.2	–	–	- no groundwater	0.01	0.06
20	2	129.032	129.0	3	n/plastic	49.0	102.1	102.1	–	–	- no groundwater	0.02	0.05
22	3	116.928	116.9	3	n/plastic	99.0	239.9	239.9	–	–	- no groundwater	0.00	0.03
25	5	116.928	116.9	3	n/plastic	99.0	261.0	261.0	–	–	- no groundwater	0.01	0.02
30	5	116.928	116.9	3	n/plastic	99.0	324.4	324.4	–	–	- no groundwater	0.00	0.02
35	5	116.928	116.9	3	n/plastic	70.0	192.9	192.9	–	–	- no groundwater	0.01	0.01
40	1.5	116.928	116.9	3	n/plastic	99.0	370.0	370.0	2.08	–	- too dense – (N1)60,CS > 32	0.00	0.00
209.00	41.50	1229.80	1229.80	30.00	0.00	831.13	2050.83	2050.83	2.08	0.00		0.87	2.71

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	40.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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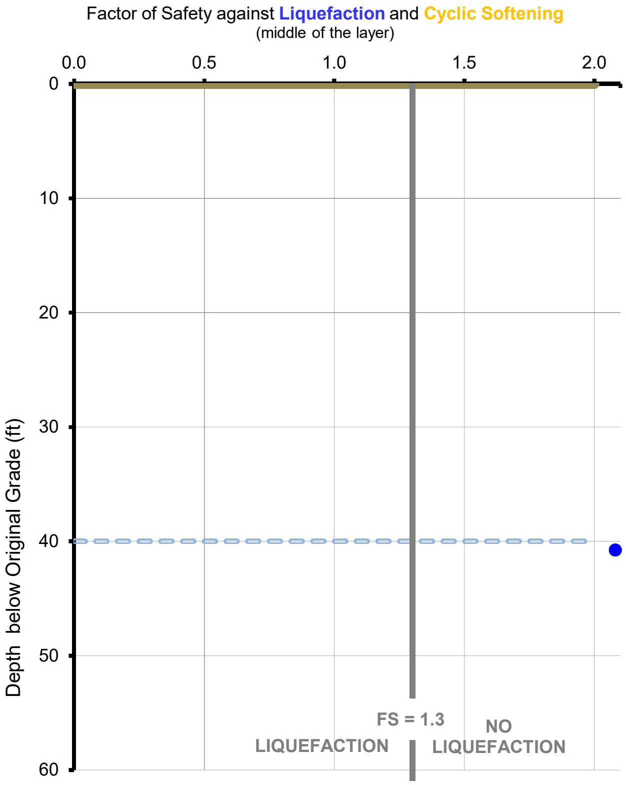
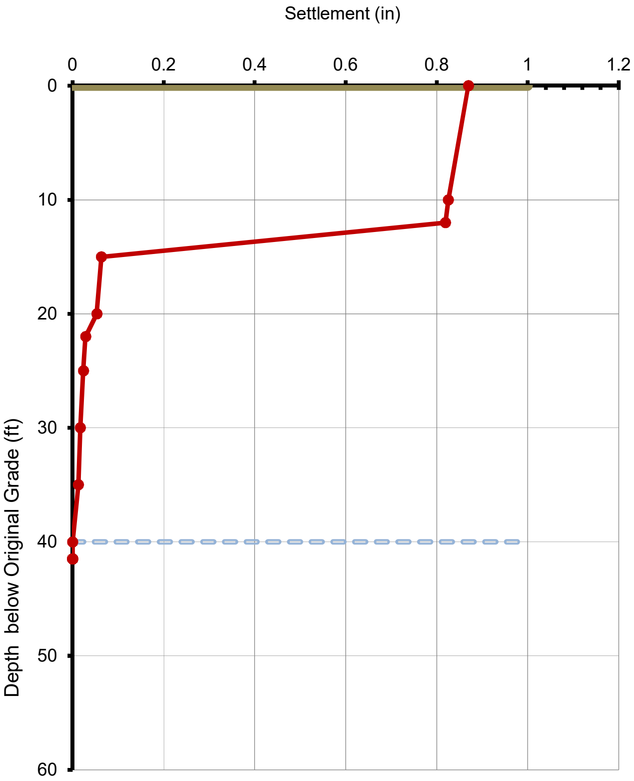
Design excavation depth	0.00 feet
Design groundwater depth	40.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	Mountain View	Boring:	**B-16**	Engineer:	TA	Date:	9/24/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	61.5	feet
Profile thickness susceptible to liquefaction	0	feet
Number of evaluated intervals	17	
Number of potentially liquefiable intervals	1	
Average Factor of Safety of sandy intervals	**2.04**	
Dry sand settlement	7.40	inches
Liquefaction settlement	0.00	inches
Total earthquake-induced settlement	**7.40**	**inches**

Liquefaction behavior Plasticity Index threshold	less or equal to 7
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	5	112.169	112.2	3	n/plastic	8.0	21.0	21.0	–	–	- no groundwater	4.80	7.40
5	2	112.169	112.2	3	n/plastic	18.0	41.2	41.2	–	–	- no groundwater	0.40	2.60
7	3	112.169	112.2	3	n/plastic	14.7	32.0	32.0	–	–	- no groundwater	1.42	2.20
10	2	112.169	112.2	3	n/plastic	27.0	56.3	56.3	–	–	- no groundwater	0.14	0.78
12	3	112.169	112.2	3	n/plastic	99.0	172.1	172.1	–	–	- no groundwater	0.01	0.64
15	5	112.169	112.2	3	n/plastic	39.0	81.9	81.9	–	–	- no groundwater	0.11	0.63
20	2	112.169	112.2	3	n/plastic	99.0	215.5	215.5	–	–	- no groundwater	0.00	0.53
22	3	112.169	112.2	3	n/plastic	44.0	90.7	90.7	–	–	- no groundwater	0.05	0.52
25	5	112.169	112.2	3	n/plastic	99.0	242.9	242.9	–	–	- no groundwater	0.01	0.47
30	5	112.169	112.2	3	n/plastic	32.0	64.0	64.0	–	–	- no groundwater	0.19	0.47
35	5	112.169	112.2	3	n/plastic	54.9	128.2	128.2	–	–	- no groundwater	0.03	0.28
40	5	112.169	112.2	3	n/plastic	31.0	57.7	57.7	–	–	- no groundwater	0.20	0.25
45	5	112.169	112.2	3	n/plastic	59.6	153.1	153.1	–	–	- no groundwater	0.02	0.05
50	5	112.169	112.2	3	n/plastic	59.0	155.0	155.0	–	–	- no groundwater	0.02	0.03
55	5	112.169	112.2	3	n/plastic	99.0	370.0	370.0	–	–	- no groundwater	0.01	0.02
60	1.4	112.169	112.2	3	n/plastic	99.0	369.9	369.9	–	–	- no groundwater	0.00	0.00
61.4	0.1	112.169	112.2	3	n/plastic	99.0	369.9	369.9	2.04	–	- too dense – (N1)60,CS > 32	0.00	0.00
492.40	61.50	1906.87	1906.87	51.00	0.00	981.45	2621.26	2621.26	2.04	0.00		7.40	16.88

Profile		
In-Situ Groundwater depth	no GW	feet
DESIGN Groundwater depth	61.40	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading	
M	7.56
PGA	1.228

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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B-16 Design excavation depth 0.00 feet
Design groundwater depth 61.40 feet