Appendix I

Report of Geotechnical Investigation

Preliminary
Report of Geotechnical Investigation

Proposed Compressor Station Upgrade

Ventura Compressor Station
1555 North Olive Street
Ventura, California

Prepared for:

Southern California Gas Company
Los Angeles, California

Project 4953-16-1091

April 17, 2019

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April 17, 2019
Wood Project 4953-16-1091

Dr. Mehrshad M. Ketabdar
Principal (Team Lead) Civil/Structural Engineer
Southern California Gas Company
555 West 5th Street
Los Angeles, California 90013

Subject: Letter of Transmittal
Report of Geotechnical Investigation
Proposed compressor Station Upgrade
Ventura Compressor Station
1555 North Olive Street
Ventura, California

Dear Dr. Mehrshad:

We are pleased to submit the results of our geotechnical investigation in support of the proposed compressor station upgrade at the Ventura Compressor Station located at 1555 North Olive Street in Ventura, California. Our services were conducted in general accordance with our proposal dated February 27, 2019 and the Standard Services Agreement (Agreement 5660055165) between Southern California Gas Company and our firm effective as of March 15, 2019. The investigation of the subject project was originally conducted under a contract with Worley Parsons (WorleyParsons Project No. 408007-00267), for which we presented our results in a report dated December 23, 2016 This report supersedes our previous report.

The scope of our exploration was originally planned with the WorleyParsons design team and based on the Geotechnical Investigation Scope of Work document, dated July 19, 2016, issued by WorlyParsons, In addition, we were provided with the Removal Action Completion Report, dated October 2011, prepared by Tetra Tech, Inc. for Parcel A of the Former Ventura MGP Site, which is located in the northern half of the subject project site. The Tetra Tech report includes a compaction report for Parcel A, dated September 7, 2011, prepared by Geotechnical Solutions. It our understanding that recent design changes have been made subsequent to the issuance of our 2016 report, which requires revisions to our prior recommendations. We were provided with an updated site plan as well as the latest equipment loads by Mr. Mounssef Asri of your office.

The results of our investigation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the appropriate governmental agencies for their review and approval prior to obtaining a permit.

It has been a pleasure to be of professional service to you. Please contact us if you have any questions or if we can be of further assistance.

Sincerely,

Wood Environment & Infrastructure Solutions, Inc.

Larry Hong
Senior Engineer

Reviewed by:

Mark A. Murphy
Principal Geotechnical Engineer
Project Manager

Rosalind Munro
Principal Engineering Geologist

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(Electronic copies submitted)

Note: While the Project Description described in this report is outdated, the geologic conditions and soils beneath the site are unchanged from the analysis herein. This report will be revised once further engineering is undertaken for the current Project.

Preliminary

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Proposed Compressor Station Upgrade

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1.0 Scope

This report provides foundation design information for the proposed compressor station upgrade located at 1555 North Olive Street in Ventura, California. The location of the project site is illustrated on Figure 1, Vicinity Map. The locations of the proposed station equipment, buildings, and our exploration borings are shown on Figure 2, Boring Plot Plan.

This investigation was authorized to determine the physical characteristics of the soils at the site of the proposed compression station upgrade, and to provide recommendations for foundation design, floor slab and pavement support, and for grading for the project. In addition, dynamic soil properties were to be provided for use in dynamic structural analysis of the compressor foundations. More specifically, the scope of this investigation included the following:

- Review of subsurface explorations and laboratory tests and provide a description of the soil and groundwater conditions encountered.
- Perform a limited geologic-seismic hazards evaluation.
- Provide recommendations for appropriate foundation systems together with the necessary design parameters, including frictional resistance, passive resistance, and the anticipated total and differential settlements.
- Provide recommendations for type of cement and /or coating requirement for foundations and underground utilities.
- Provide recommendations for subgrade preparation and floor slab support.
- Provide recommendations for subgrade modulus.
- Provide recommended parameters for design of retaining walls, including passive, at-rest, and active soil pressures.
- Provide seismic design parameters based on the current California Building Code, including a site-specific ground motion hazard analysis.
- Provide dynamic soil properties for use in dynamic structural analysis of the compressor foundations.
- Provide recommendations for design of asphalt and portland cement concrete paving.
- Provide recommendations for grading, including site preparation, excavation and slopes, the placing of compacted fill, and quality control measures relating to earthwork.
- Provide recommended soil infiltration rate for the design of the proposed waste water sump.

The assessment of general site environmental conditions for the presence of contaminants in the soils and groundwater of the site was beyond the scope of this investigation.

Our recommendations are based on the results of our field explorations, laboratory tests, and appropriate engineering analyses. The results of our field explorations and laboratory tests, which form the basis of our recommendations, are presented in Appendix A.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report. This report has been prepared for Southern California Gas Company and their design consultants to be used solely in the design of the proposed compressor station upgrade in Ventura, California. This report has not been prepared for use by other parties and may not contain sufficient information for purpose of other parties or other uses.

2.0 Site Conditions and Project Description

The project site is an active compressor station currently owned and operated by Southern California Gas Company (SoCalGas). The site is located to the northwest of Downtown Ventura near the interchange of the Ojai Freeway (State Route 33) and the Ventura Freeway (Highway 101) and approximately 2 miles inland from the Pacific Ocean. Existing structures at the site include two compressor buildings, a building containing compressor after coolers, several storage tanks, and various operating equipment in the northern portion of the site; an office building, a warehouse, and a parking lot in the central portion; and a waste spill-containment storage yard and equipment/tool laydown area in the southern portion of the site.

The site is relatively level and clear of vegetation. The northern half of the site is an unpaved lot covered with gravel, whereas the southern half is paved with asphalt. Almost the entire northern half of the project site is within the limits of the former Ventura Manufactured Gas Plants (MGP) Site. The Ventura MGP site occupies an approximately 8.5-acre area from beyond State Highway 33 to the west (near the Ventura River channel) to Olive Street to the east. The portion where the project site coincides with the MGP site has been identified as Parcel A and the site soils were remediated in 2011. The remediation operation was managed by Tetra Tech, Inc., which they summarized in a Removal Action Completion Report, dated October, 2011. In addition, a compaction report dated September 7, 2011, prepared by Geotechnical Soilutions was included as part of the Tetra Tech report (Appendix F).

Based on the two reports previously referenced (Tetra Tech, 2011, and Geotechnical Soilutions, 2011), the excavations to remove the impacted soils within Parcel A of the MGP site have been replaced with compacted import soils. Cement slurry and rock segregated from the excavation soils, after being washed, were also used as backfill in some deeper excavations. The depths of excavations within Parcel A as part of the MGP cleanup range from 5 to 40 feet below surface grade.

Based on the available data, the approximate locations of the deeper excavation pits backfilled with rock and cement slurry are shown on Figure 2. Based on the records, segregated rocks were placed at the bottom of the two excavations, indicated as “Rock Backfill” on Figure 2, and compacted import granular soil was placed over the rock. Filter fabric was placed in between the rocks and import soil. Sand-cement slurry consisting of two-sacks of cement per cubic yard was placed in the trenches located to the immediate north of the existing office building to a depth of approximately 4 feet below surface grade. Compacted import soil was placed above the slurry. The locations of these trenches are shown as “slurry backfill” on Figure 2. An abandoned subsurface structure support on rock foundation was demolished during the MGP cleanup operation; the bottom of the excavation in that area was backfilled with debris generated from the demolition of the structure. Cement slurry was poured to fill the gaps between debris and compacted import soil was used to backfill the remainder of the pit. This location is indicated as “”Rock Slurry” on Figure 2. The fill soils were compacted to 90% of its maximum density and are considered as secondary structural fill since existing undocumented fill soils were left in place beneath the new fill.

In addition, an area along the west wall in the northern half of the site was excavated and backfilled as part of MGP Parcel B cleanup; however, documentation of the placement and compaction of the backfill in this area was not available.

The proposed upgrade project consists of constructing several new equipment, including a new compressor building, and the demolition and relocation of several existing buildings and facilities. Based on the updated site plan and new equipment loads we are provided, the new compressor building will be constructed in the western-central portion of the site, which will contain 4 new reciprocating compressors, each weighing approximately 107 tons and a new air cooler weighing 120 tons. The two existing warehouse/office buildings and the existing waste spill-containment storage yard, which are located at the proposed location of the new compressor building, will be demolished. To replace these demolished buildings, a new office building and warehouse building will be constructed in the eastern-central and southern-central portion of the site, respectively. A new sump pit is planned to be constructed in the southwest corner of the site. Other new construction will include a new MCC room building and several equipment pads in the northwestern portion of the site, which is within the MGP cleanup site. A list of the proposed equipment as well as the loads of each equipment are presented in Table 1. The locations of the proposed new equipment are shown on Figure 2.

3.0 Field Explorations and Laboratory Tests

The soil conditions beneath the site were explored, as requested, by drilling nine borings at the locations of the proposed structures to depths between 20 and 75½ feet below the existing grade. The boring locations from our investigation are presented on Figure 2. Details of our explorations and the logs of the borings are presented in Appendix A.

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

- Moisture content and dry density determinations.
- Direct shear.
- Consolidation.
- Particle-Size Distribution Analyses.
- Fines content.
- Atterberg Limits.
- Soil Permeability.
- Corrosivity.

All testing was performed in general accordance with applicable ASTM specifications. Details of the laboratory testing program and test results are presented in Appendix A. Near surface soil samples were tested for corrosivity and evaluated by HDR. The results of their study and recommendations, including recommendations for type of cement and /or coating requirement for underground structures and utilities, are summarized in a report dated November 7, 2016, which is presented in Appendix B.

Suspension logging was performed by *Geo*Vision in two of the borings located within the proposed footprint of the new compressor building. The results of the suspension logging are presented in a report prepared by *Geo*Vision, which is presented in Appendix C.

4.0 Soil Conditions

The placement and compaction of the fill soils within the former MGP site was documented by Geotechnical Soilutions, Inc. According to their report, the MGP remedial zone in the northern portion of the site encompass an area measured approximately 250 feet by 540 feet in plan area. The excavations to remove the impacted soils, reportedly extending to depths ranging from 5 to 32 feet below the surrounding ground surface, had occurred in various locations within the remedial zone. The excavations were backfilled with fill materials consisting of imported sandy soils compacted to at least 90 percent of maximum dry density obtainable by the ASTM D1557 method of compaction. Cement slurry and rock were also used as backfill in some deeper excavations. As indicated by the Geotechnical Soilutions compaction report, the compacted fill soil was considered to be secondary structural fill. Our borings within the limits of the MGP cleanup site encountered fill soils between 5 and 11 feet in thickness. Outside of the limits of the MGP cleanup site, our borings encountered fill soils, typically ranging from 3 to 7 feet in thickness, with one location (VCU 8) encountering 12½ feet of existing fill, possibly the results of a nearby utility trench.

The fill soils encountered consist predominantly of a layer of sandy clay overlying poorly graded sand and poorly graded gravel. Deeper and/or poorer quality fill could occur between our borings and in other unexplored areas, particularly in areas where existing structures and underground utilities are present.

The natural soils generally consist predominantly of sand and gravel with occasional thin clay interbeds to the maximum depth explored. Based on the Tetra Tech report, large cobbles and boulders as large as 6 feet in diameter were encountered during the excavation to remove the impacted soils in the northern portion of the site.

Groundwater was encountered at depths ranging from 40½ to 45 feet below adjacent grade in our deeper borings. According the California Geological Survey (CGS), the historic-high groundwater level is approximately 20 feet below the existing grade.

The corrosion studies indicate that the on-site soils are moderately corrosive to ferrous metals and that the potential for sulfate attack on portland cement concrete is considered negligible. The report of corrosion studies presented in Appendix B should be referred to for a discussion of the corrosion potential of the soils, and for potential mitigation measures.

5.0 Geology

5.1 Geologic Setting

The site is located in the northern portion of the Ventura-Oxnard basin in southern Ventura County. This sedimentary basin was formed primarily by subsidence of marine sediments and subsequent non-marine deposition along the Santa Clara River and Ventura River watersheds. Regionally, the Ventura-Oxnard basin is part of the Transverse Ranges geomorphic province, which consists of east-west trending mountain ranges and associated valleys, such as Santa Ynez Mountains and Santa Clara River Valley. The predominant structural fabric is formed by east-west trending fault traces and major fold axes, such as the Ventura fault and the Ventura Avenue anticline. The Ventura River and floodplain drain to the south to the Pacific Ocean coastline.

Locally, the site is located in the eastern Santa Ynez Mountains within the Ventura River floodplain. The site is at an approximate elevation of 65 feet (NGVD 29). The relation to the site and topographic features is shown on Figure 1, Vicinity Map.

5.2 Geologic Materials

Fill soils, up to 12½ feet thick were encountered in our 2016 borings. The existing fills throughout the site is described in Sections 2.0 and 4.0. The fill is underlain by Holocene-age alluvium, consisting primarily of sand and gravel, to the maximum boring depth of 75½ feet.

5.3 Groundwater

The site is located within the lower Ventura River Watershed. According to the California Geological Survey (CGS, 2003), the historic high groundwater level at the site is approximately 20 feet below ground surface (bgs). Groundwater was encountered at depths ranging from 40½ to 45 feet bgs in our borings, which were drilled to a maximum depth of 75½ feet bgs.

5.4 Faults

.Numerous faults in Southern California have been previously characterized as active or potentially active. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS), for the Alquist-Priolo Earthquake Fault Zoning Program (Bryant and Hart, 2007). According to Bryant and Hart, an active fault is one with surface displacement within Holocene time (about the last 11,700 years); and a potentially active fault is a fault that has demonstrated surface displacement of Quaternary age deposits (last 1.6 million years) (Jennings and Bryant, 2010, Bryant and Hart, 2007). More recently the CGS has revised fault activity designations for the purpose of the Alquist-Priolo (A-P) Earthquake Fault Zoning Program (CGS, 2018). A Holocene-active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A pre-Holocene fault is a fault that has been demonstrated to not have Holocene surface displacement. An age-undetermined fault is one where the recency of fault movement has not been determined. The closest active fault is the Ventura fault, which is located approximately 1.0 mile south of the site (Jennings and Bryant, 2010; USGS-CGS, 2006).

5.5 Geologic-Seismic Hazards

Surface Fault Rupture

The site is not located within a currently established Alquist-Priolo Earthquake Fault Zone (AP-Zone) for surface fault rupture hazard. The closest AP-Zone, established for the Ventura fault, is located approximately 1.0 mile south of the site (CGS, 2002, 2019).

Based on the available geologic data, active faults, and associated fault splays, with the potential for surface fault rupture are not known to be located directly beneath or projecting toward the site (Hubbard et al., 2014; Jennings and Bryant, 2010; USGS-CGS, 2006). Therefore, the potential for surface rupture due to fault plane displacement propagating to the surface at the site during the design life of the proposed building is considered to be low.

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structure is designed and constructed in conformance with current building codes and engineering practices.

Liquefaction

Liquefaction potential is greatest where the groundwater level is shallow, and submerged loose, fine sands occur within a depth of about 50 feet or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases. According to the County of Ventura General Plan (2013), the City of Ventura General Plan (2005), and the CGS (2003, 2019), the site is within an area identified as having a potential for liquefaction. However, based on the density of the materials encountered beneath the site, the potential for liquefaction adversely impacting the project site in the event of the design earthquake is considered low.

Slope Stability

The general topography of the site is relatively flat which precludes both slope instability and the potential for lurching (earth movement at right angles to a cliff or steep slope during ground shaking). According to the CGS (2003), the Ventura County General Plan (2013), and the City of Ventura General Plan (2005), there are no known landslides near the site, nor is the site in the path of any known or potential landslides.

Tsunamis, Inundation, and Seiches

The site is located 1.5 miles north of the coast at an elevation of about 65 feet above sea level. According to the CGS (2009) and the Ventura County General Plan (2013), the site is not within a Tsunami (seismic sea wave) Inundation Area.

According to the Ventura County General Plan (2013), the site is located within a potential inundation area for an earthquake-induced dam failure by Lake Casitas Dam, which is located approximately 5.5 miles to the north-northwest. The Project Site could be adversely affected in the event of an earthquake-induced dam failure or seiches (wave oscillations in an enclosed or semi-enclosed body of water). However, this dam, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

Flooding

According to the Federal Emergency Management Agency (FEMA), the site is within an area of moderate flooding potential, Zone X (FEMA, 2010). Zone X, as defined by FEMA, is an area between the limits of the 100-year and 500-year floods. The County of Ventura further establishes the site as being within an area of “reduced risk due to levee” (County of Ventura, 2016). Although, based on the flood designation by the County of Ventura, the potential for flooding at the site is considered to be low, a site-specific hydraulic/hydrological study may be required to further assess the flood hazards and base flood elevations.

Oil Wells and Methane Gas

According to the California Division of Oil, Gas and Geothermal Resources (DOGGR, 2016), there are no oil wells within the project boundary, however, the site is located 0.5 miles south of the Ventura oil and gas field, which has numerous documented active, plugged and abandoned wells.

Additional plugged and abandoned oil exploration holes are not known to be located at the site. However, because the site is within close proximity to the limits an oil field, there is a possibility that undocumented wells could be encountered during construction. Any well encountered would need to be properly abandoned in accordance with the current requirements of DOGGR.

According to DOGGR, the site is located adjacent to the active Ventura oil field (DOGGR, 2016). Since the site is located adjacent to an active oil field, there is a potential for methane and other volatile gases to occur beneath the site. If testing indicates that methane is present at the site, a permanent methane gas control system may be necessary beneath the proposed buildings at the site. If necessary, a methane gas specialist should be retained for the design of such a system.

Subsidence

According to Ventura County General Plan Hazards Appendix (2013), the site is not within an area of known subsidence associated with fluid withdrawal (groundwater or petroleum), peat oxidation, or hydrocompaction.

5.6 Geologic Conclusions

Based on the available geologic data, active faults, including associated fault splays, with the potential for surface fault rupture are not known to be located beneath or projecting toward the site. In our opinion, the potential for surface rupture at the site due to fault plane displacement propagating to the ground surface during the design life of the proposed development is considered low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the buildings are designed and constructed in conformance with current building codes and engineering practices.

The site is within an area identified as having potential for inundation and seiches as a result from a Lake Casitas Dam failure. However, this dam, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

The site is located within an area classified as being between the 100-year and 500-year flood according to FEMA. The County of Ventura further designates the area as being within an area of "reduced risk due to levee." Therefore, the potential for flooding at the site is considered low. Further study may be required for a site-specific flood hazard analysis.

Although the site is not within an oil field, it is however, adjacent to the Ventura Oil Field, therefore, the potential exists for the presence of methane and other volatile gases. The absence of significant slopes at the site precludes both slope instability and the potential for lurching (earth movement at right angles to a cliff or steep slope during ground shaking). The potential for other geologic hazards such as liquefaction, tsunamis, and subsidence affecting the site is also considered low.

6.0 Recommendations

6.1 General

As previously stated, the fill soils placed in Parcel A of the former MGP area are considered secondary structural fill. Moreover, records are not available for the existing fill soils outside the MGP site limits in the southern half of the project site and the compacted fill placed during the site cleanup of Parcel B along the northern half of the west wall. Therefore, the existing fill soils are not considered suitable for support of shallow foundations, floor slabs, pavement, or other exterior concrete walks and slabs on grade. All existing fill soils should be removed and replaced as properly compacted fill beneath new structures, floor slabs, and exterior concrete walks and slabs on grade. If this is done, the proposed structures may be supported on conventional spread/continuous footings or pad-type foundations established in the stiff and dense undisturbed natural soils and/or overlying new properly compacted fill. If the recommendations on grading contained herein are implemented, floor slabs may be supported on grade.

As an alternative to the removal and recompaction of existing fill soils beneath foundations, such as in areas where space for construction of proposed equipment is limited, where existing fill soils are particularly deep, or to limit settlements, the proposed structures may be supported on drilled cast-in-place concrete pile foundations and floor slabs structurally supported. Note that difficulty should be anticipated during installation of drilled pile foundations due to the abundant gravel, cobbles, and boulders beneath the site.

If the potential for some settlement and greater than normal maintenance is acceptable, only the upper 2 feet of existing fill soils need be removed and replaced as properly compacted fill beneath floor slabs, pavement, and exterior concrete walks and slabs on grade. Furthermore, project elements that are not particularly sensitive to settlement, such as pavement and exterior concrete walks and slabs on grade and possibly floor slabs, may be supported on the existing secondary structural fill soils within Parcel A of the MGP cleanup site if the risk of excessive settlement is considered acceptable.

The recommended removal and recompaction of existing fill soils should extend beyond all foundations in plan view a distance equal to the depth of removal beneath the foundation. Fill soils need not be removed beyond floor slabs and exterior concrete walks and slabs on grade in plan view. The on-site soils, including those generated from footing excavations and grading operations, less any debris or organic matter, may be used in required fills. Cobbles larger than 4 inches in diameter should not be used in the fill.

6.2 Foundations

General

As previously stated, the proposed structures may be supported on conventional spread/continuous footings or pad-type foundations underlain by the stiff and dense undisturbed natural soils and/or overlying new properly compacted fill. As an alternative to the removal and recompaction of existing fill soils beneath foundations, such as in areas where space for construction of proposed equipment is limited, where existing fill soils are particularly deep, or to limit settlements, the proposed structures may be supported on drilled cast-in-place concrete pile foundations and the floor slab structurally supported. Our recommendations for both shallow and drilled cast-in-place concrete pile foundations are provided in the following subsections. The recommended bearing values, pile capacities, and lateral load design values presented in the following subsections were determined based on a conventional working stress design or Allowable Stress Design (ASD), When considering an ultimate design, approach, the recommended design values may be multiplied by the appropriate ultimate design factors presented in last subsection (Ultimate Design Factors) of this section.

Shallow Foundations

Bearing Values and Settlement

Due to the various types of buildings and equipment being proposed, we have estimated the static settlement based on structural loads of up to 300 kips using different allowable bearing pressures. The results of our computations to estimate the foundation settlement for spread and continuous footings are presented on Figures 3.1 and 3.2, respectively. The allowable bearing value used in design should be obtained from Figures 3.1 and/or 3.2 based on the tolerable settlement for each particular structure. Shallow foundations should have a minimum width of 12 inches and extend at least 2 feet below the lowest adjacent grade or floor level. Our foundation recommendations are summarized in Table 1, Foundation Design Table, for each proposed structure.

A one-third increase in the bearing values on Figures 3.1 and 3.2 and Table 1 may be used for wind or seismic loads. The recommended bearing values are net values, and the weight of concrete in the foundations may be taken as 50 pounds per cubic foot; the weight of any soil backfill may be neglected when determining the downward loads.

Lateral Resistance

Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 may be used between the foundations and the floor slabs and the supporting soils based on a soil friction angle of 32 degrees and a factor of safety of 1.5. The passive resistance of natural soils or properly compacted fill soils may be computed using the following parameters:

Parameters for Computing Lateral Earth Pressures

Friction Angle (degrees)	32
Cohesion (psf)	250
Soil Unit Weight (pcf)	122
Passive Lateral Earth Pressure Coefficient (Kp)	3.25

If a factor of safety of at least 1.5 is used in determining the allowable passive pressure using the parameters above, a one-third increase in the passive value may be used for wind or seismic loads and the frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

Foundation Observation

To verify the presence of satisfactory soils at the design elevations, the bottoms of the foundations should be observed by a soil inspector approved by SoCalGas. Foundations should be deepened as necessary to reach satisfactory supporting soils.

Inspection of the foundation excavations may also be required by the reviewing governmental agencies. The contractor should be familiar with the inspection requirements of the reviewing agencies.

Modulus of Subgrade Reaction

A modulus of subgrade reaction, k, of 115 pounds per cubic inch may be assumed for the on-site undisturbed natural or properly compacted fill soils at-grade. The value is unit values for use with a 1-foot-square area. The modulus should be reduced in accordance with the following equation when used with the larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where: K = unit subgrade modulus

K_R = reduced subgrade modulus

B = foundation width

Deep Foundations

Axial Capacities

We have analyzed the axial capacities for the drilled cast-in-place concrete piles based on the methodology presented in Chapter 5 of Naval Facilities Engineering Command (NAVFAC) Publication DM-7.02, Foundation and Earth Structures, dated September 1986. The estimated allowable downward and upward capacities of 18-, 24-, and 30-inch-diameter drilled cast-in-place concrete piles are presented on Figure 4, Drilled Pile Capacities. The pile capacities shown are dead-plus-live load capacities; a one-third increase may be used for wind or seismic loads. The capacities presented are based on the strength of the soils; the compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

Piles in groups should be spaced at least 3 diameters on centers. If the piles are so spaced, no reduction in the axial capacities need be considered due to group action. However, if the piles are spaced less than 3 diameters on center, a reduction factor will need to be applied. The reduction factor may be computed as the perimeter of the pile group divided by the total of the perimeters of each individual pile with the group, with a maximum value of 1.0.

We have computed the axial stiffnesses for 40-foot long 18-, 24-, and, 30-inch-diameter drilled concrete piles using the computer program SHAFT by ENSOFT, Inc. The results of our computations, in the form of load-settlement curves, are presented on Figure 5, Drilled Pile Stiffness Plot.

Settlement

We estimate the settlement of the proposed structures supported on deep foundations in the manner recommended to be about ½ inch or less with differential settlement of less than ¼ inch.

Lateral Resistance

Lateral loads may be resisted by the piles, by friction between the floor slabs and the supporting soils, and by the passive resistance of the soils against pile caps and grade beams. We have computed the lateral capacities of the piles using the computer program LPILE by ENSOFT, Inc. Resistance of the soils adjacent to 18-, 24-, and 30-inch-diameter drilled piles are shown in the following tables for top of pile deflection of ¼ and ½ inch. These resistances have been calculated assuming both free and fixed-head pile conditions. The piles should be long enough to reach the depths to zero moment presented in the following tables; the depths given in the following tables are with reference to the bottom of pile cap.

Lateral Load Design Data
18-inch Drilled Concrete Pile

	Pile Head Deflection (inches)			
	¼		½	
Pile Head Condition	**Free**	**Fixed**	**Free**	**Fixed**
Lateral Load (kips)	16	48	28	80
Maximum Moment (foot-kips)	65	191	126	344
Depth to Maximum Moment (feet)	5½	0	6	0
Depth to Zero Deflection (feet)	9	11	9½	11
Depth to Zero Moment (feet)	14	16	14½	16

Lateral Load Design Data
24-inch Drilled Concrete Pile

	Pile Head Deflection (inches)			
	¼		½	
Pile Head Condition	**Free**	**Fixed**	**Free**	**Fixed**
Lateral Load (kips)	30	89	54	149
Maximum Moment (foot-kips)	142	421	273	757
Depth to Maximum Moment (feet)	6½	0	6½	0
Depth to Zero Deflection (feet)	11	13	11	13½
Depth to Zero Moment (feet)	17½	19½	17½	20

Lateral Load Design Data
30-inch Drilled Concrete Pile

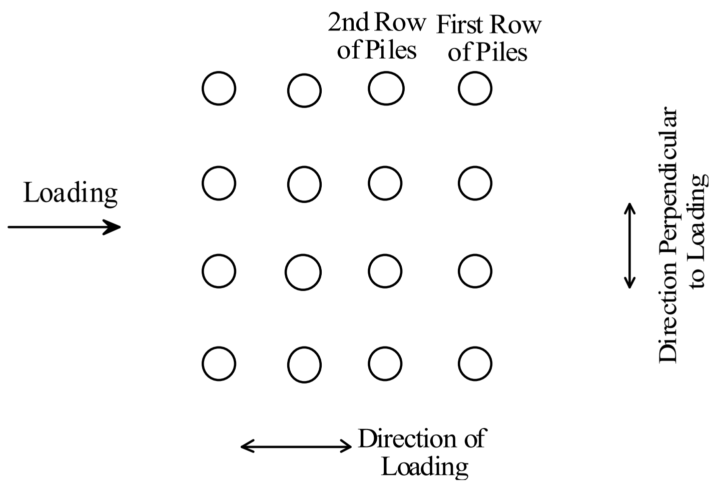
	Pile Head Deflection (inches)			
	¼		½	
Pile Head Condition	**Free**	**Fixed**	**Free**	**Fixed**
Lateral Load (kips)	48	141	87	236
Maximum Moment (foot-kips)	253	760	488	1,375
Depth to Maximum Moment (feet)	7½	0	7½	0
Depth to Zero Deflection (feet)	12½	15½	13	16
Depth to Zero Moment (feet)	20½	23	20½	23½

By: LH 11/7/2016 Checked: MM

The lateral load-deflection relationships between the pile foundation and the soil are nonlinear relationships and can be represented by p-y curves in which "p" is the force acting along the tributary length of the pile and "y", is the lateral deflection. The tabulated p-y curves for the 18-, 24-, and 30-inch diameter drilled piles at 5-foot intervals along the length of the drilled piles are presented in Table 2.

The lateral load design data provided above was obtained using an elastic piles model in LPILE. As the pile design progresses and reinforcing details become available, additional analyses can be performed to study the interaction between axial and lateral loading and the impact on the pile behavior.

For piles in groups spaced as shown below and at least 3 pile diameters on centers, no reduction in the lateral capacities need be considered for the first (leading) row of piles in the direction perpendicular to loading. For subsequent rows in the direction of loading, piles in groups spaced closer than 8 pile diameters on centers will have a reduction in lateral capacity due to group effects. Therefore, the lateral capacity of piles in groups, except for the first row of piles, spaced at 3 pile diameters on centers, may be assumed to be reduced by half. The reduction of lateral capacity in the direction of loading for other pile spacing may be interpolated.



The passive resistance of soils against pile caps and grade beams and the frictional resistance between the floor slabs and the supporting soils may be taken as presented in the preceding subsection on Shallow Foundations.

The resistance of the piles, the frictional resistance, and the allowable passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

Pile Installation

Depending on the type of drilling equipment used by the contractor, caving could occur within the pile shafts during drilling for the piles. In addition, difficulty should be anticipated during installation of drilled pile foundations due to the abundant gravel, cobbles, and boulders beneath the site. Precautions should be taken during the installation of the pile to reduce caving and raveling. Among other precautions, the drilling speed should be reduced as necessary to minimize vibration and sloughing of the sand deposits. Drilling mud and/or casing could also be considered. Because of the anticipated caving and raveling, a greater than normal volume of concrete may be required in the piles. It may be desirable for the drilling contractor to consider special techniques to minimize caving and raveling, and hence minimize the required volume of concrete used.

Piles spaced less than 8 feet on centers should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight.

Concrete should be pumped from the bottom up through a rigid pipe extending to the bottom of the drilled excavation, with the pipe being slowly withdrawn as the concrete level rises. The discharge end of the pipe should be at least 5 feet below the surface of the concrete at all times during placement. The discharge pipe should be kept full of concrete during the entire placing operation and should not be removed from the concrete until all of the concrete is placed and fresh concrete appears at the top of the pile. The volume of concrete pumped into the hole should be recorded and compared to design volume.

Only competent drilling contractors with experience in the installation of drilled cast-in-place piles in similar soil conditions should be considered for the pile construction. We suggest requesting the piling contractor to submit a list of similar projects along with references for each project.

The drilling of the pile excavations and the placing of the concrete should be observed continuously by a soils inspector approved by SoCalGas to verify that the desired diameter and depth of piles are achieved.

Ultimate Design Factors

The recommended bearing values, pile capacities, and lateral load design values above are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values provided in the previous sections may be multiplied by the factors shown:

Design Item	Ultimate Design Factor
Bearing Value	3.0
Axial Pile Capacity	2.0
Lateral Pile Capacity	1.0
Passive Pressure	1.5
Coefficient of Friction	1.5

In no event, however, shall foundation sizes be less than those required for dead-plus-live loads when using the working stress design values.

6.3 Seismic Design Parameters

Mapped Seismic Design Parameters

We have determined the mapped seismic design parameters for the site with the latitude of 34.2976 and a longitude of -119.3006. The mapped seismic design parameters were obtained in accordance with the 2016 California Building Code (CBC) and ASCE 7-10 Standard (ASCE, 2013) using the United States Geological Survey (USGS) Seismic Design Maps Web Application. The CBC Site Class was determined to be Site Class “C” based on the results of the shear wave velocity measurements and a review of the local soil and geologic conditions. The mapped seismic parameters may be taken as presented in the following table:

Parameter	Mapped Value
S_S (0.2 second period)	2.38g
S_1 (1.0 second period)	0.90g
Site Class	C
Fa	1.0
Fv	1.3
S_{MS} = FaSS (0.2 second period)	2.38
S_{M1} = FvS1 (1.0 second period)	1.17g
S_{DS} = 2/3 x SMS (0.2 second period)	1.59g
S_{D1} = 2/3 x SM1 (1.0 second period)	0.78g

By: LH 11/7/16
Chkd: MM 11/8/16

6.4 Site-Specific Ground Motion Hazzard Analysis

We have performed a Probabilistic Seismic Hazard Analyses (PSHA) and a Deterministic Seismic Hazard Analyses (DSHA) using the computer program EZ-FRISK (Risk Engineering, 2014) in order to develop site-specific response spectra in accordance with the 2013 CBC and Chapter 21 of ASCE 7-10. For the DSHA, a composite deterministic response spectrum was compiled from the maximum of the 84th percentile spectral ordinates computed for known nearby faults. In addition to known fault sources, background seismicity was also included in the PSHA. The computed PSHA and DSHA ground motions were converted to maximum direction ground motions using the multiplication factors recommended in Shahi and Baker (2013).

The site-specific probabilistic and deterministic response spectra were developed using the average ground motions obtained from the Next Generation Attenuation (NGA) West 2 relationships of Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), Chiou and Youngs (2014). For all four NGA relationships, we have used an average shear wave velocity in the upper 30 meters equal to 440 meters per second based on the average results of the suspension logging performed at the site. We have used a depth to a shear wave velocity of 1,000 meters per second beneath the site ($Z_{1.0}$) and a depth to a shear wave velocity of 2,500 meters per second ($Z_{2.5}$) based on the equations provided by the NGA West2 authors.

In accordance with Chapter 21 of ASCE 7-10, the probabilistic Risk-Targeted Maximum Considered Earthquake (MCE_R) response spectrum was taken as the maximum direction response spectrum with a 2% probability of being exceeded in 50 years multiplied by the risk coefficients C_{RS} and C_{R1}. The risk-targeted coefficients, C_{RS} and C_{R1} were taken from Figures 22-17 and 22-18 in ASCE 7-10. The value of C_{RS} was applied for periods less than or equal to 0.2 second, the value of C_{R1} was applied for periods greater than or equal to 1.0 second, and linear interpolation was used to determine the risk coefficient between 0.2 second and 1.0 second. The C_{RS} and C_{R1} values for this project were determined to be 0.937 and 0.931, respectively.

ASCE 7-10 defines the deterministic MCE_R response spectrum as the maximum of the composite deterministic response spectrum and the deterministic lower limit, as defined on Figure 21.2-1 of ASCE 7-10. The site-specific MCE_R response spectrum was then taken as a composite of the probabilistic and deterministic MCE_R response spectra, determined as described above, which consisted of the lesser of the spectral ordinates between the two spectra. The 5% damped site-specific MCE_R response spectrum and its components are shown on Figure 4. The site-specific design response spectrum was computed by multiplying the ordinates of the site-specific MCE_R response spectrum by two-thirds, with a lower limit at all periods of 80% of the spectral ordinates of the general design response spectrum determined in accordance with Section 11.4.5 of ASCE 7-10. The 5% damped site-specific design response spectrum and its components are shown on Figure 5. The site-specific MCE_R and design response spectra are presented in digitized form for 5% and 10% of critical structural damping in Table 3.

Based on the results of our analyses, the site-specific design acceleration parameters, as defined in Section 21.4 of ASCE 7-10, S_{DS} and S_{D1}, may be taken as 1.79g and 0.89g, respectively, and the site-specific MCE_R acceleration parameters, S_{MS} and S_{M1}, may be taken as 2.68g and 1.33g, respectively.

Dynamic Soil Properties

Based on the results of our explorations, laboratory testing, and suspension logging, selected dynamic properties of the soils were determined for use in the dynamic analysis of the proposed compressors. The dynamic soil properties are summarized in the table below.

Depth to Top of Layer (feet)	Depth to Bottom of Layer (feet)	Average Shear Wave Velocity (feet/second)	Poisson's Ratio	Total Unit Weight (pcf)	Shear Modulus (psf)	Elastic Modulus (psf)
0	10	850	0.32	122	2.74E+06	7.23E+06
10	15	1,400	0.35	125	7.61E+06	2.05E+07
15	20	1,050	0.30	145	4.96E+06	1.29E+07
20	31	1,350	0.34	138	7.81E+06	2.09E+07
31	36	1,800	0.32	139	1.40E+07	3.69E+07
36	40	1,400	0.33	135	8.22E+06	2.19E+07
40	50	1,700	0.40	145	1.30E+07	3.64E+07
50	60	1,850	0.47	145	1.54E+07	4.53E+07

In addition, based on the properties of the materials encountered in our borings, an internal/material damping ratio of 2.5% may be used for the soil column. This damping ratio is based on an assumed induced cyclic shear strain value of approximately 10^{-2} to 10^{-3} percent, which is typical for machine foundations. In addition, based on this level of induced cyclic shear strain and the materials underlying the site, a shear modulus reduction value of 0.9 may be applied to values in the table above for the soil column.

6.5 Floor Slab Support

The subgrade beneath floor slabs on grade should be prepared as recommended in the following section on grading.

Construction activities and exposure to the environment can cause deterioration of the prepared subgrade. Therefore, we recommend our that our field representative observe the condition of the final subgrade soils immediately prior to slab on grade construction, and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

If vinyl or other moisture-sensitive floor covering is planned, we recommend that the floor slab in those areas be underlain by a capillary break consisting of a vapor-retarding membrane over a 4 inch-thick layer of gravel. A 2-inch-thick layer of sand should be placed between the gravel and the membrane to decrease the possibility of damage to the membrane. We suggest the following gradation for the gravel:

Sieve Size	Percent Passing
¾″	90 - 100
No. 4	0 - 10
No. 100	0 - 3

A low-slump concrete should be used to minimize possible curling of the slab. A 2-inch-thick layer of coarse sand can be placed over the vapor retarding membrane to reduce slab curling. If this sand bedding is used, care should be taken during the placement of the concrete to prevent displacement of the sand. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

6.6 Temporary Shoring

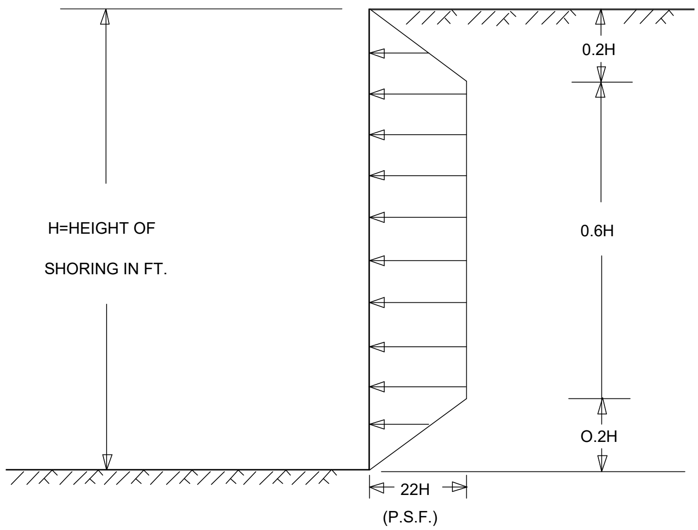
General

Where there is not sufficient space for sloped embankments, shoring will be required. Temporary excavations, such as those for new construction of sump pit, may be supported using conventional soldier beams with wood lagging. Based on our investigation, it should be noted that some hard layers contain cobble and boulders starting at shallow depth may be encountered during excavation, and the installation of soldier piles could be difficult.

Lateral Pressures

For design of cantilevered shoring, a triangular distribution of earth pressure may be used. It may be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot.

For the design of braced shoring, we recommend the use of a trapezoidal distribution of earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in the following diagram with the maximum pressure equal to 22H in pounds per square foot, where H is the height of the shoring in feet. Where a combination of sloped embankment and shoring is used, the pressure would be greater and must be determined for each combination.



In addition to the recommended earth pressures, the upper 10 feet of shoring adjacent to normal vehicular traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. Furthermore, the shoring should be designed to resist any lateral surcharge pressure imposed by existing foundations, heavy equipment, or storage loads.

Design of Solider Piles

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 600 pounds per square foot per foot of depth, up to a maximum of 6,000 pounds per square foot. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed siltstone. The concrete placed in the soldier pile excavations may be a lean mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

Lagging

Continuous lagging will be required between the soldier piles. The soldier piles and lagging should be designed for the full anticipated lateral pressure.

Deflection

The deflection of a cantilevered shoring system may be estimated by the shoring engineer. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement adjacent to the excavation. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

Monitoring

Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system has been finalized.

6.7 Sump Pit Walls and Minor Retaining Walls

Lateral Earth Pressure

The parameters presented in the following table may be used to design the sump pit walls and any minor retaining walls:

Parameters for Computing Lateral Earth Pressures

Friction Angle (degrees)	32
Cohesion (psf)	250
Soil Unit Weight (pcf)	122
Active Lateral Earth Pressure Coefficient (Ka)	0.31
At-Rest Earth Pressure Coefficient (Ko)	0.47

In addition to the computed earth pressure, walls adjacent to areas subject to vehicular traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal vehicular traffic. If the traffic is kept back at least 10 feet (or a distance equal to the height of the wall, whichever is less) from the walls, the traffic surcharge may be neglected. The walls should also be designed to resist any applicable surcharges due to foundation or storage loads. We can provide detailed surcharge pressure recommendations when any adjacent foundation or storage loads and their geometries in relation to walls are provided to us.

Seismic Lateral Earth Pressures

It is anticipated that any minor retaining walls planned for the project will be less than 6 feet in height. Based on the anticipated wall height, the strength characteristics of the on-site soils, and the design level of ground shaking, it is our opinion that the seismic lateral earth pressures will be negligible.

For the design of the proposed sump pit walls, we recommend that a total static-plus-seismic earth pressure equivalent to that developed by a fluid having a unit weight of 73 pounds per cubic foot be used. The static active earth pressure computed using the parameters given in the preceding subsection should be subtracted from this value to obtain the seismic increment of earth pressure. The seismic increment should be combined with the static active earth pressure (not the at-rest pressure).

Drainage

Retaining walls should be designed to resist hydrostatic pressures or be provided with a drain pipe or weepholes. The drain could consist of a 4-inch-diameter perforated pipe placed with perforations down at the base of the wall. The pipe should be sloped at least 2 inches in 100 feet and surrounded by ¾-inch crushed rock or gravel separated from the on-site soils by an appropriate filter fabric. The crushed rock or gravel should have less than 5% passing a No. 200 sieve.

6.8 Soil Permeability

Two samples were selected from borings at and near the planned new waste spill-containment area for constant head permeability tests. The boring locations and the depths at which the samples were taken, soil classifications, and the permeability test results are presented in the following table.

Boring No.	**Sample Depth (ft)**	**Soil Type**	**Soil Permeability**
VCU 5	5	Poorly Graded Sand with Gravel (SP)	2.45E-03 (cm/sec) 9.65E-04 (in/sec)
VCU 6	15	Poorly Graded Gravel with Sand (GP)	1.79E-02 (cm/sec) 7.05E-03 (in/sec)

6.9 Paving

To provide support for paving, the subgrade soils should be prepared as recommended in the following section on grading. Compaction of the subgrade, including trench backfills, to at least 90%, and achieving a firm, hard, and unyielding surface will be important for paving support. The preparation of the paving area subgrade should be performed immediately prior to placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

To provide data for design of paving sections, an R-value of 40 was assumed for on-site soils for the purposes of estimating the pavement thickness. The R-value should be confirmed during grading.

Asphalt Concrete Paving

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of the on-site or comparable soils compacted to at least 90% as recommended, the minimum recommended paving thicknesses are presented in the following table.

Assumed Traffic Index	Asphalt Concrete (Inches)	Base Course (Inches)
4 (Automobile Parking)	3	4
5 (Driveways with Light Truck Traffic)	3	4
6 (Driveways with Heavy Truck Traffic)	4	5

The asphalt paving sections were determined using the Caltrans design method. We can determine the recommended paving and base course thicknesses for other Traffic Indices if required. Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved, and that proper construction procedures are followed.

Portland Cement Concrete Paving

Portland cement concrete paving sections were determined in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for a range of Traffic Indices are presented in the following table. We have assumed that the portland cement concrete will have a compressive strength of at least 3,000 pounds per square inch.

Assumed Traffic Index	Concrete Paving (Inches)	Base Course (Inches)
4 (Automobile Parking)	6½	4
5 (Driveways with Light Truck Traffic)	7	4
6 (Driveways with Heavy Truck Traffic)	7	4

The paving should be provided with joints at regular intervals no more than 15 feet in each direction. Load transfer devices, such as dowels or keys, are recommended at joints in the paving to reduce possible offsets. The paving sections in the above table have been developed based on the strength of unreinforced concrete. Steel reinforcing may be added to the paving to reduce cracking and to prolong the life of the paving.

Base Course

The base course for both asphaltic and concrete paving should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the latest edition of the State of California, Department of Transportation, Standard Specifications. Alternatively, the base course could meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction. The base course should be compacted to at least 95%.

6.10 Grading

As previously stated, the fill soils placed in Parcel A of the former MGP area are considered secondary structural fill. Moreover, records are not available for the existing fill soils outside the MGP site limits in the southern half of the project site and the compacted fill placed during the site cleanup of Parcel B along the northern half of the west wall. Therefore, the existing fill soils are not considered suitable for support of shallow foundations, floor slabs, pavement, or other exterior concrete walks and slabs on grade. All existing fill soils should be removed and replaced as properly compacted fill beneath new structures, floor slabs, and exterior concrete walks and slabs on grade.

If the potential for some settlement and greater than normal maintenance is acceptable, only the upper 2 feet of existing fill soils need be removed and replaced as properly compacted fill beneath floor slabs exterior concrete walks and slabs on grade. Furthermore, project elements that are not particularly sensitive to settlement, such as pavement and exterior concrete walks and slabs on grade and possibly floor slabs, may be supported on the existing secondary structural fill soils within Parcel A of the MGP cleanup site if the risk of some excessive settlement is considered acceptable.

The recommended removal and recompaction of existing fill soils should extend beyond all foundations in plan view a distance equal to the depth of removal beneath the foundation. Fill soils need not be removed beyond floor slabs and exterior concrete walks and slabs on grade in plan view. In addition, the existing fill soils need not be removed and recompacted where the alternative for a structurally supported floor slab is chosen.

All required fill should be uniformly well compacted and observed and tested during placement. The on-site soils (excluding oversized cobbles and boulders) may be used in any required fill.

Site Preparation

After the site is cleared and existing fill soils and soils disturbed due to demolition activities are excavated as recommended, the exposed soils should be carefully observed for the removal of all unsuitable deposits. Next, where fill is to be placed, the exposed soils should be scarified to a depth of 6 inches, brought to near-optimum moisture content, and rolled with heavy compaction equipment. At least the upper 6 inches of the exposed soils should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction.

Good drainage of surface water should be provided by adequately sloping all surfaces. Such drainage will be important to minimize infiltration of water beneath floor slabs and pavement.

Excavation and Temporary Support

Where excavations are deeper than about 4 feet, the sides of the excavations should be sloped back at 1:1 (horizontal to vertical) or shored for safety. Unshored excavations should not extend below a plane drawn at 1½:1 (horizontal to vertical) extending downward from adjacent existing footings. Data for design of shoring is provided in Section 6.5 of this report.

Excavations should be observed by a soils inspector approved by SoCalGas so that any necessary modifications based on variations in the soil conditions can be made. All applicable safety requirements and regulations, including OSHA regulations, should be met.

Compaction

Required fill should be placed in loose lifts not more than 8-inches-thick and compacted. The fill should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557 method of compaction. The moisture content of the on-site sandy soils at the time of compaction should vary no more than 2% below or above optimum moisture content.

Material for Fill

The on-site soils, including which generated from footing excavations and grading operations, less any debris or organic matter, may be used in required fills. Cobbles larger than 4 inches in diameter should not be used in the fill. Any required import material should consist of relatively non-expansive soils with an expansion index of less than 35. The imported materials should contain sufficient fines (at least 15% passing the No. 200 sieve) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be approved by geotechnical engineer of record approved by SoCal Gas prior to being placed at the site.

6.11 Geotechnical Observation

The reworking of the upper soils and the compaction of all required fill should be observed and tested during placement by a representative of the geotechnical engineer of record. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proofrolling and delineation of areas requiring overexcavation.
- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.
- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.
- Observe the installation of pile foundations, if used, to confirm that the desired depths and diameters are achieved.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

7.0 Basis for Recommendations

The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during our subsurface explorations. We have made our recommendations based upon experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the structure configuration, loads, location, or the site grades should be provided to us so that we can review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based upon the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to verify that the actual soil conditions are as expected. This also provides for the procedure whereby the client can be advised of unexpected or changed conditions that would require modifications of our original recommendations. In addition, the presence of our representative at the site provides the client with an independent professional opinion regarding the geotechnically-related construction procedures. If another firm is retained for the geotechnical observation services, our professional responsibility and liability for implementation of the recommendations provided in this report.

Project labor agreements are often written in such a manner to preclude non-union firms from providing inspection and testing services during construction. If your project is considering being signatory to a project labor agreement or other union labor agreement, it would be beneficial for the labor agreement to include language that specifically excludes construction soils and materials inspection. Failure to exclude construction inspection from the project labor agreement would likely preclude the geotechnical engineer of record from continuing services during construction and limit construction inspection and testing to union firms. We would be pleased to meet with you to discuss the implications associated with project labor agreements.

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TABLES

Table 1

Foundation Design Table

Table 1 - Foundation Design Table

	FEED (From SCG-GPE)				Footing Recommendations					
					Recommended Foundation System/ Design Allowable Bearing Values (psf)			Minimum Footing Embedment (ft)	Minimum Footing Width (ft)	Footing Subgrade
Ventura Compressor Station	Equipment Orientation/Type	Equipment Size (ft)	Equipment Weight (tons)/ ea	Quantity	½ inch Total Settlement; ¼ inch Differential Settlement	¾ inch Total Settlement; ¼ inch Differential Settlement	1 inch Total Settlement; ¼ inch Differential Settlement			
Blowdown Stack	Vert	8' dia x 60' ht	16.00	1	Spread Footing/1,600 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Instrument Air Compressor	Skid	8' W x 15' L x 9' ht	14.00	2	Spread Footing/1,800 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Instrument Air Receiver	Vert	6.5' dia x 18' ht	20.00	1	Spread Footing/1,400 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Discharge Scrubber	Vert	4' dia x 10' ht	12.00	1	Spread Footing/2,000 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Starting Air Compressor	Skid	8' W x 15' L x 9' ht	8.00	2	Spread Footing/5,000 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Starting Air Receiver	Vert	6.5' dia x 18' ht	20.00	1	Spread Footing/1,400 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Recip Compressors	Skid	24.5' W x 40' L x 21' ht	107	4	Spread Footing/500 psf	Spread Footing/1,000 psf	Spread Footing/1,800 psf	2	1	Remove all existing fill if encountered
Air Cooler	Unit	19' W x 30' L x 16' ht	120	1	Spread Footing/500 psf	Spread Footing/800 psf	Spread Footing/1,600 psf	2	1	Remove all existing fill if encountered
Silencer	Unit	3.5' W x 4.5' L x 15.5' ht	2	4	Spread Footing/5,500 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Generator (Electrical)	Skid	12' W x 44' L x 13' ht	50.00	1	Spread Footing/750 psf	Spread Footing/1,700 psf	Spread Footing/5,000 psf	2	1	Remove all existing fill if encountered
E & I Building	Building	16' W x 72' L x 11' ht	37.50	1	Spread Footing/800 psf	Spread Footing/2,200 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Transformer	Unit	7' W x 7' L x 7' ht	5.00	1	Spread Footing/5,500 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
NSCR	Vert	6' Dia x 25' ht	15	4	Spread Footing/1,750 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Waste Oil Storage	Horiz	5' Dia x 10' L	10.00	1	Spread Footing/3,000 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Inlet Filter Separator	Horiz	48" x 18' Top Barrel 24" x 18' Bottom	15	1	Spread Footing/1,750 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Engine Oil Storage	Horiz	5' Dia x 10' L	10.00	1	Spread Footing/3,000 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered

Table 1 - Foundation Design Table										
Ventura Compressor Station	FEED (From SCG-GPE)				Footing Recommendations					
					Recommended Foundation System/ Design Allowable Bearing Values (psf)			Minimum Footing Embedment (ft)	Minimum Footing Width (ft)	Footing Subgrade
	Equipment Orientation/Type	Equipment Size (ft)	Equipment Weight (tons)/ ea	Quantity	½ inch Total Settlement; ¼ inch Differential Settlement	¾ inch Total Settlement; ¼ inch Differential Settlement	1 inch Total Settlement; ¼ inch Differential Settlement			
Oily water waste Storage	Horiz	5' Dia x 10' L	10.00	1	Spread Footing/3,000 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Oily water Waste Tank	Horiz	4'W x 4'L x 3'ht	6.00	1	Spread Footing/5,500 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Waste Oil Tank	Horiz	4'W x 4'L x 3'ht	6.00	1	Spread Footing/5,500 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Auxiliary/JW Cooler	Skid	12'x15'	20.00	4	Spread Footing/1,400 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Warehouse Building			10.00	1	Spread Footing/3,000 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered
Office Building			10.00	1	Spread Footing/3,000 psf	Spread Footing/5,500 psf	Spread Footing/5,500 psf	2	1	Remove all existing fill if encountered

Table 2

Lateral Spring Data

Table 2.1 - Lateral Spring Data (18-inch CIDH Piles)

Project Name: SoCalGas Ventura
Amecfw Job No.: 4953-16-1091
Date: 11/29/2016

Depth = 5 ft. below bottom of pile cap

y, in	p, lb/in
0	0
0.025	282.8446
0.05	565.6892
0.075	848.5338
0.1	1131.378
0.125	1218.232
0.15	1172.733
0.175	1135.592
0.2	1104.371
0.225	1077.546
0.25	1054.102
0.275	1033.335
0.3	1014.733
0.675	749.9203
0.70875	749.9203
0.7425	749.9203
0.77625	749.9203

Depth = 10 ft. below bottom of pile cap

y, in	p, lb/in
0	0
0.025	620.3446
0.05	1240.689
0.075	1861.034
0.1	2481.378
0.125	3101.723
0.15	3722.068
0.175	4342.412
0.2	4619.94
0.225	4444.864
0.25	4293.883
0.275	4161.726
0.3	4044.632
0.675	2386.351
0.70875	2386.351
0.7425	2386.351
0.77625	2386.351

Depth = 15 ft. below bottom of pile cap

y, in	p, lb/in
0.0000	0
0.0250	957.8446
0.0500	1915.689
0.0750	2873.534
0.1000	3831.378
0.1250	4789.223
0.1500	5747.068
0.1750	6704.912
0.2000	7662.757
0.2250	8620.601
0.2500	9471.286
0.2750	9178.504
0.3000	8919.125
0.6750	5246.053
0.7088	5246.053
0.7425	5246.053
0.7763	5246.053

Depth = 20 ft. below bottom of pile cap

y, in	p, lb/in
0	0.0
0.025	1295.3
0.05	2590.7
0.075	3886.0
0.1	5181.4
0.125	6476.7
0.15	7772.1
0.175	9067.4
0.2	10363.0
0.225	11658.0
0.25	12953.0
0.275	14249.0
0.3	15544.0
0.675	9247.7
0.70875	9247.7
0.7425	9247.7
0.77625	9247.7

Table 2.2 - Lateral Spring Data (24-inch CIDH Piles)

Project Name: SoCalGas Ventura
Amecfw Job No.: 4953-16-1091
Date: 11/29/2016

Depth = 5 ft. below bottom of pile cap

y, in	p, lb/in
0.0000	0
0.0333	380.9956
0.0667	761.9913
0.1000	1142.987
0.1333	1291.763
0.1667	1267.559
0.2000	1248.119
0.2333	1231.916
0.2667	1218.051
0.3000	1205.95
0.3333	1195.228
0.3667	1185.61
0.4000	1176.898
0.9000	1052.195
0.9450	1052.195
0.9900	1052.195
1.0350	1052.195

Depth = 10 ft. below bottom of pile cap

y, in	p, lb/in
0.0000	0
0.0333	830.9956
0.0667	1661.991
0.1000	2492.987
0.1333	3323.983
0.1667	4154.978
0.2000	4985.974
0.2333	5182.289
0.2667	4963.206
0.3000	4777.662
0.3333	4617.573
0.3667	4477.381
0.4000	4353.115
0.9000	2592.928
0.9450	2592.928
0.9900	2592.928
1.0350	2592.928

Depth = 15 ft. below bottom of pile cap

y, in	p, lb/in
0.0000	0
0.0333	1280.996
0.0667	2561.991
0.1000	3842.987
0.1333	5123.983
0.1667	6404.978
0.2000	7685.974
0.2333	8966.969
0.2667	10248
0.3000	10295
0.3333	9944.398
0.3667	9637.291
0.4000	9365.213
0.9000	5512.258
0.9450	5512.258
0.9900	5512.258
1.0350	5512.258

Depth = 20 ft. below bottom of pile cap

y, in	p, lb/in
0	0.0
0.03333	1731.0
0.06667	3462.0
0.1	5193.0
0.13333	6924.0
0.16667	8655.0
0.2	10386.0
0.23333	12117.0
0.26667	13848.0
0.3	15579.0
0.33333	17310.0
0.36667	16799.0
0.4	16324.0
0.9	9594.6
0.945	9594.6
0.99	9594.6
1.035	9594.6

Table 2.3a - Lateral Spring Data (30-inch CIDH Piles)

Project Name: SoCalGas Ventura
Amecfw Job No.: 4953-16-1091
Date: 11/29/2016

Depth = 5 ft. below bottom of pile cap

y, in	p, lb/in
0.0000	0
0.0417	478.9489
0.0833	957.8978
0.1250	1322.121
0.1667	1324.259
0.2083	1325.92
0.2500	1327.278
0.2917	1328.428
0.3333	1329.425
0.3750	1330.304
0.4167	1331.092
0.4583	1331.804
0.5000	1332.455
1.1250	1341.81
1.1813	1341.81
1.2375	1341.81
1.2938	1341.81

Depth = 10 ft. below bottom of pile cap

y, in	p, lb/in
0.0000	0
0.0417	1041.449
0.0833	2082.898
0.1250	3124.347
0.1667	4165.796
0.2083	5207.244
0.2500	5694.648
0.2917	5443.201
0.3333	5234.38
0.3750	5056.847
0.4167	4903.146
0.4583	4768.135
0.5000	4648.129
1.1250	2946.003
1.1813	2946.003
1.2375	2946.003
1.2938	2946.003

Depth = 15 ft. below bottom of pile cap

y, in	p, lb/in
0.0000	0
0.0417	1603.949
0.0833	3207.898
0.1250	4811.847
0.1667	6415.796
0.2083	8019.744
0.2500	9623.693
0.2917	11228
0.3333	11187
0.3750	10762
0.4167	10395
0.4583	10075
0.5000	9790.459
1.1250	5766.236
1.1813	5766.236
1.2375	5766.236
1.2938	5766.236

Depth = 20 ft. below bottom of pile cap

y, in	p, lb/in
0	0.0
0.04167	2166.4
0.08333	4332.9
0.125	6499.3
0.16667	8665.8
0.20833	10832.0
0.25	12999.0
0.29167	15165.0
0.33333	17332.0
0.375	18558.0
0.41667	17925.0
0.45833	17370.0
0.5	16879.0
1.125	9925.1
1.18125	9925.1
1.2375	9925.1
1.29375	9925.1

Table 2.3b - Lateral Spring Data (30-inch CIDH Piles)

Project Name: SoCalGas Ventura
Amecfw Job No.: 4953-16-1091
Date: 11/29/2016

Depth = 25 ft. below bottom of pile cap

y, in	p, lb/in
0	0.0
0.04167	2728.9
0.08333	5457.9
0.125	8186.8
0.16667	10916.0
0.20833	13645.0
0.25	16374.0
0.29167	19103.0
0.33333	21832.0
0.375	24561.0
0.41667	27289.0
0.45833	26670.0
0.5	25915.0
1.125	15226.0
1.18125	15226.0
1.2375	15226.0
1.29375	15226.0

Table 3

Horizontal Response Spectra Pseudospectral Acceleration in g

Table 3. Horizontal Response Spectra
Pseudospectral Acceleration in g

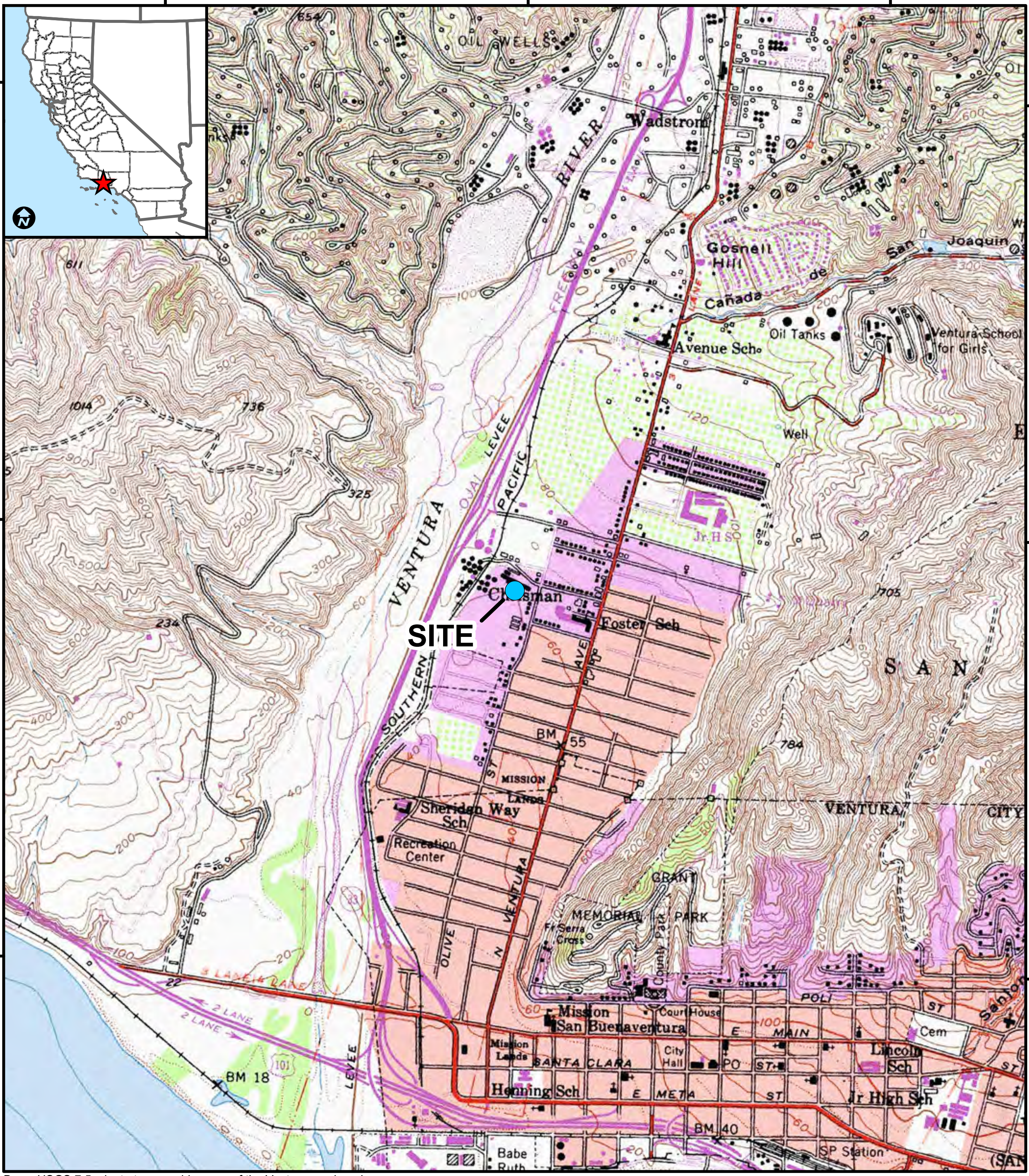
	5% Damping		10% Damping	
Period in Seconds	Maximum Considered Earthquake	Design	Maximum Considered Earthquake	Design
0.010	1.09	0.73	1.09	0.73
0.020	1.11	0.74	1.10	0.74
0.030	1.17	0.78	1.14	0.76
0.050	1.38	0.92	1.29	0.86
0.075	1.74	1.16	1.55	1.03
0.100	2.05	1.37	1.77	1.18
0.150	2.46	1.64	2.03	1.35
0.200	2.68	1.79	2.16	1.43
0.250	2.80	1.87	2.23	1.48
0.300	2.80	1.87	2.22	1.48
0.400	2.55	1.70	2.02	1.34
0.500	2.32	1.55	1.84	1.22
0.750	1.72	1.14	1.35	0.90
1.000	1.33	0.89	1.06	0.70
1.500	0.85	0.57	0.69	0.45
2.000	0.60	0.40	0.48	0.32
3.000	0.36	0.24	0.30	0.20
4.000	0.24	0.16	0.20	0.13

By: LH 11/30/16
Chkd: MM 12/5/16

FIGURES

Figure 1

Site Vicinity Map



Base: USGS 7.5 minute topographic maps of the Ventura quadrangle

G:\4953_Geotech\2016\161091 Ventura Compressor Station Upgrade\GIS\161091_Figure_1_Site_Vicinity_Map_Wood.mxd

0 1,000 2,000 4,000 Feet

wood.

Wood
Environment & Infrastructure Solutions, Inc.
6001 Rickenbacker Road
Los Angeles, California 90040
Tel: 323.889.5300
Fax: 323.721.6700

Proposed Compressor Station Upgrade
Southern California Gas Company
1555 North Olive Street
Ventura, California

LAT: LON:	34.2978 -119.3000
SCALE:	1:24,000
DRAWN:	MAE
CHECK:	PJE
DATE:	11/7/2016

SITE VICINITY MAP

FIGURE: 1

PROJECT: 4953-16-1091

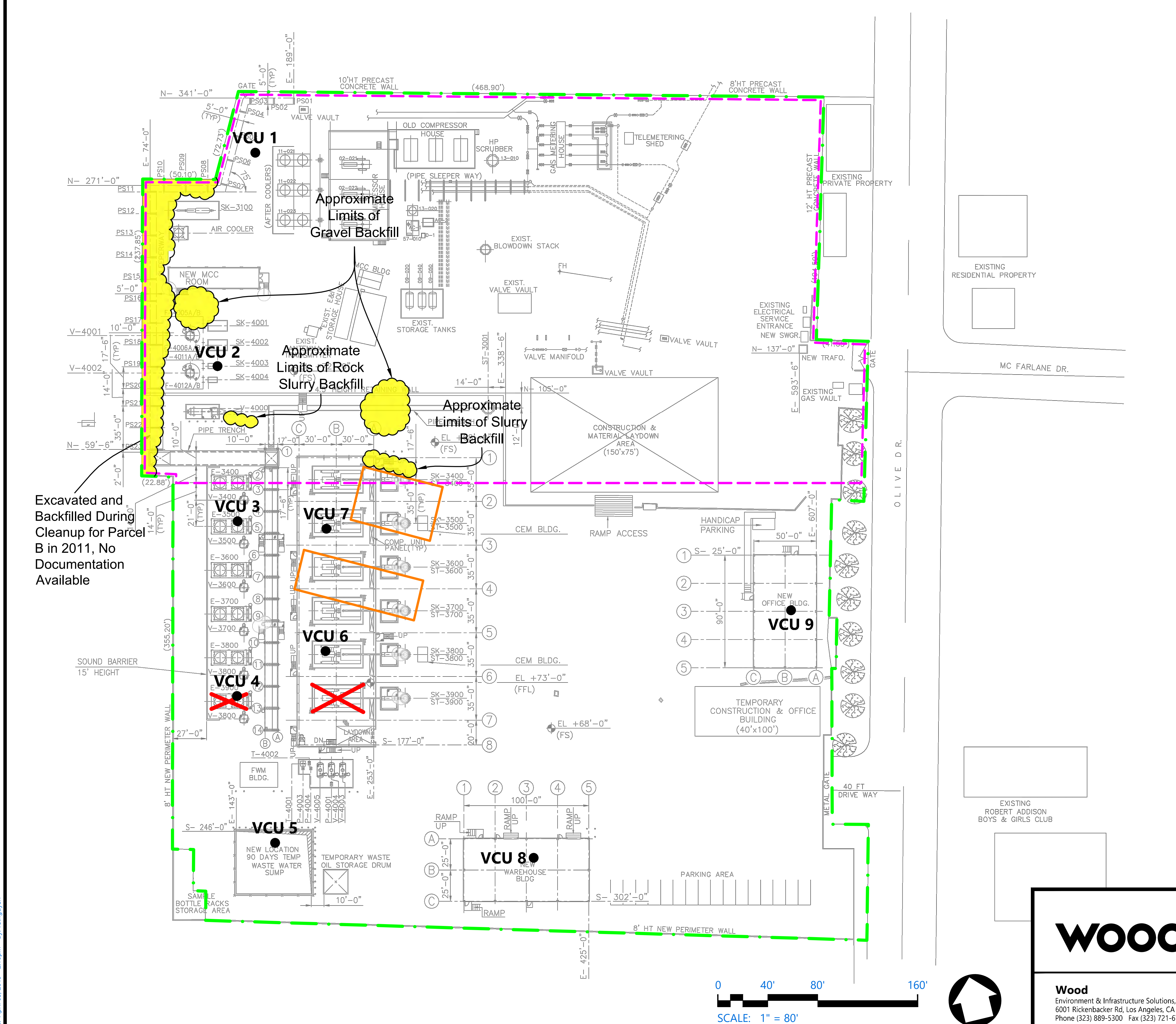
Figure 2

Boring Plot Plan

DRAWING NO.	REFERENCE DRAWING DESCRIPTION
33115-3002-D-PIP	OVERALL PLOT PLAN
33115-3006-D-PIP	EQUIPMENT LAYOUT

EQUIPMENT LIST (NEW)

PACKAGE EQUIPMENT	
SK-3400	GAS COMPRESSOR SKID
SK-3500	GAS COMPRESSOR SKID
SK-3600	GAS COMPRESSOR SKID
SK-3700	GAS COMPRESSOR SKID
SK-3800	GAS COMPRESSOR SKID
SK-3900	GAS COMPRESSOR SKID
SK-4001	INSTRUMENT AIR COMPRESSOR
SK-4002	INSTRUMENT AIR COMPRESSOR
SK-4003	STARTING AIR COMPRESSOR
SK-4004	STARTING AIR COMPRESSOR
SK-3100	EMERGENCY GENERATOR SKID
ST-3001	BLOWDOWN STACK
ST-3400	ENGINE EXHAUST STACK
ST-3500	ENGINE EXHAUST STACK
ST-3600	ENGINE EXHAUST STACK
ST-3700	ENGINE EXHAUST STACK
ST-3800	ENGINE EXHAUST STACK
ST-3900	ENGINE EXHAUST STACK
EXCHANGERS	
E-3400	DISCHARGE COOLER
E-3500	DISCHARGE COOLER
E-3600	DISCHARGE COOLER
E-3700	DISCHARGE COOLER
E-3800	DISCHARGE COOLER
E-3900	DISCHARGE COOLER
F-4005A/B	OUTLET FILTER
F-4006A/B	OUTLET FILTER
F-4011A/B	OUTLET FILTER
F-4012A/B	OUTLET FILTER
VESSELS	
V-3400	SUCTION SCRUBBER
V-3500	SUCTION SCRUBBER
V-3600	SUCTION SCRUBBER
V-3700	SUCTION SCRUBBER
V-3800	SUCTION SCRUBBER
V-3900	SUCTION SCRUBBER
V-4000	FILTER/SEPRATOR
V-4001	INSTRUMENT AIR RECEIVER
V-4002	STARTING AIR RECEIVER
V-4003	OILY WASTE STORAGE DRUM
V-4004	ENGINE OIL STORAGE DRUM
V-4005	WASTE OIL STORAGE DRUM
T-4001	COMP AREA OILY WASTE DRUM
T-4002	COMP AREA WASTE OIL DRUM
PUMPS	
P-4001	ENGINE OIL CHARGE PUMP
P-4003	COMPRESSOR AREA SUMP PUMP
P-4004	WASTE OIL DRUM PUMP



LEGEND:

- VCU 9 ● Boring Locations
- Limits of Former Manufactured Gas Plants (MGP) Site
- Limits of Project Site
- Existing Office / Warehouse Building to be Bemolished

wood.

Wood
Environment & Infrastructure Solutions, Inc.,
6001 Rickenbacker Rd, Los Angeles, CA 90040
Phone (323) 889-5300 Fax (323) 721-6700

Proposed Compressor Station Upgrade
Southern California Gas Company
1555 North Olive Street, Ventura, California

LT,LNG:	
PREPARED BY:	VMN
SCALE:	1" = 80'
BY:	WI
CHKD:	LH
DATE:	04/03/2019

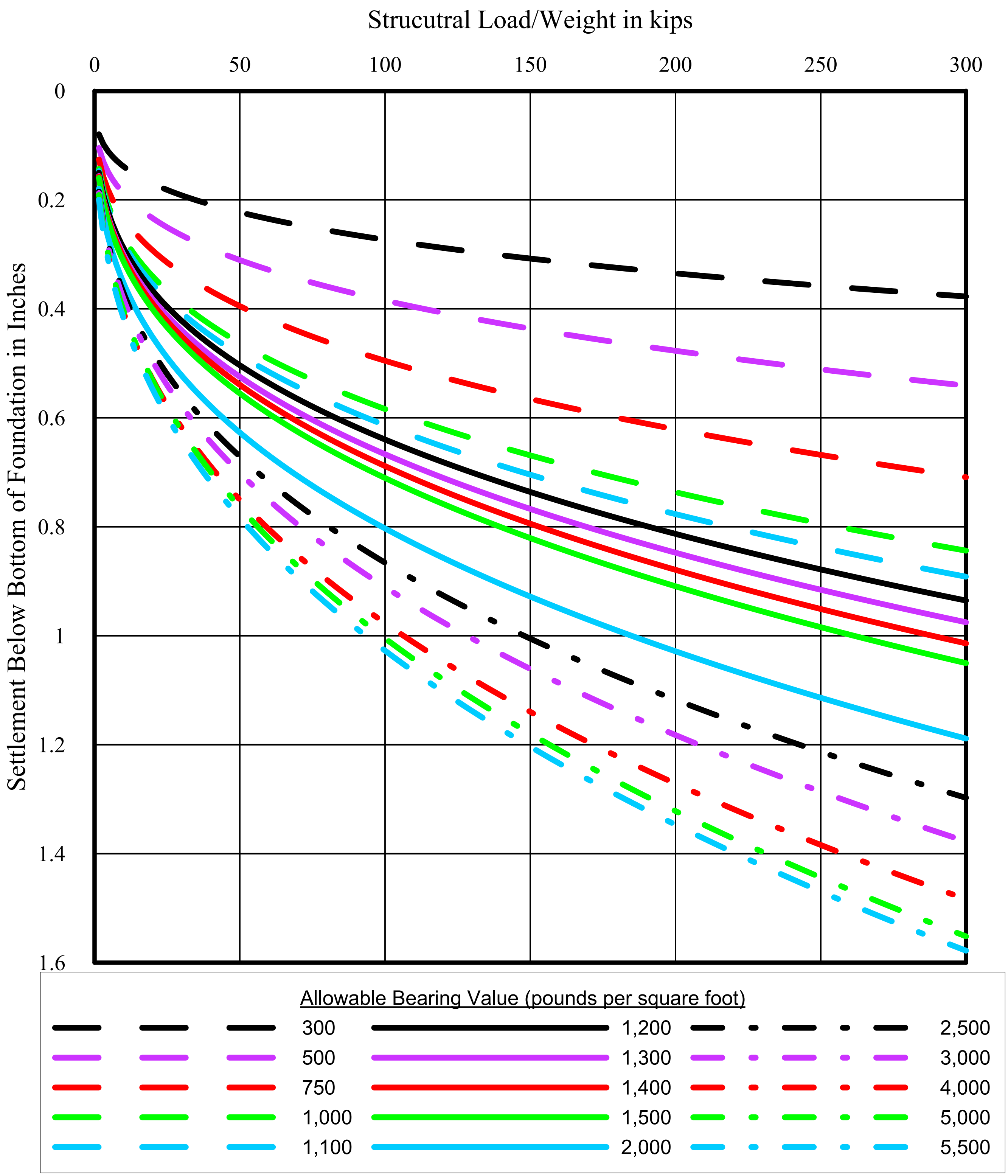
Plot Plan

FIGURE NO. 2

PROJECT NO. 4953-16-1091

Figure 3.1

Estimated Settlement for Square Foundations



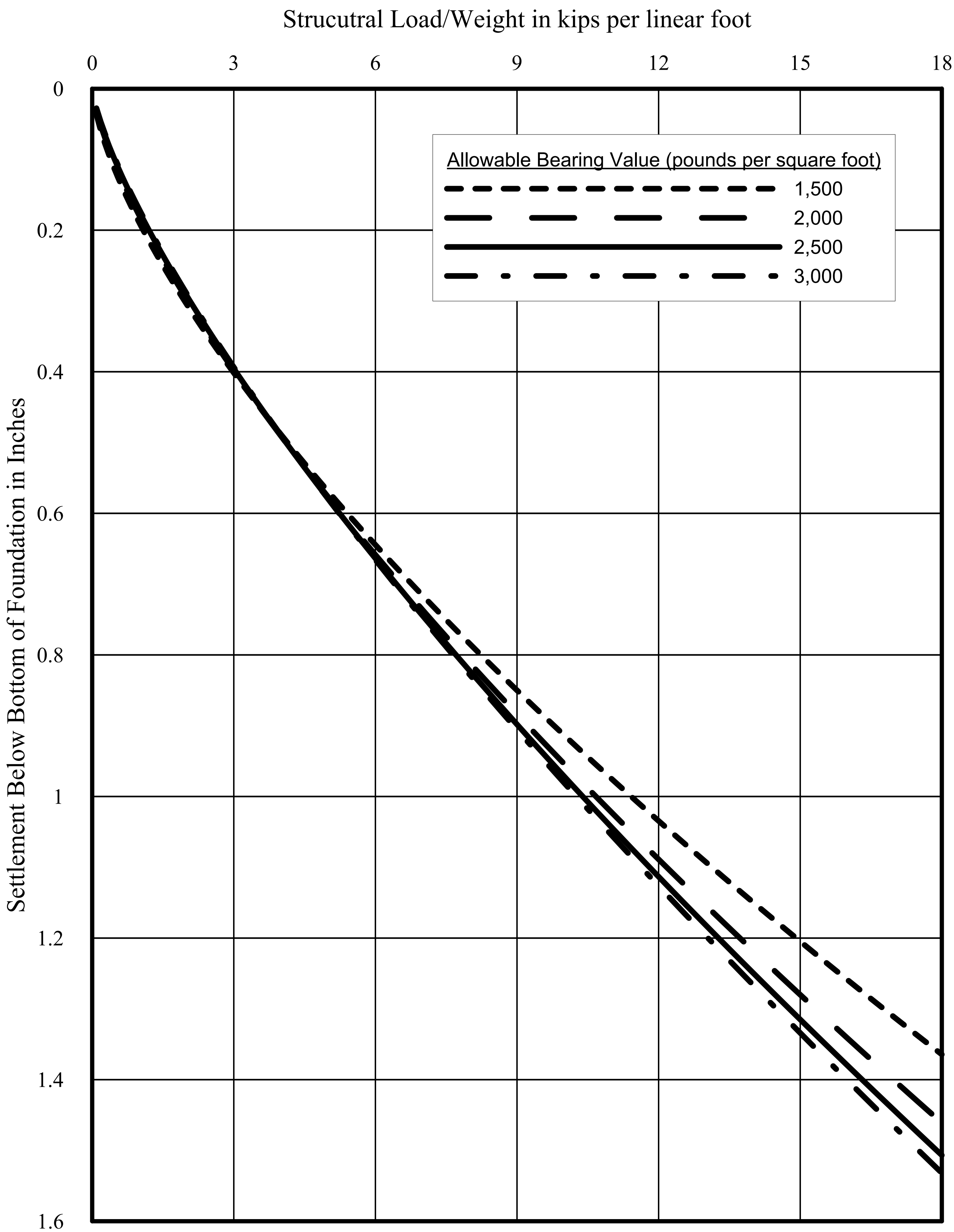
SQUARE FOUNDATIONS
(Length less than 5 times the width)

Prepared/Date: WL 12/23/2016
Checked/Date: LH 12/23/2016

Proposed Compressor Station Upgrade Ventura Compressor Station 1555 North Olive Street Ventura, California	wood.	ESTIMATED SETTLEMENT Project No. 4953-16-1091 Figure 3.1

Figure 3.2

Estimated Settlement for Continuous Foundations



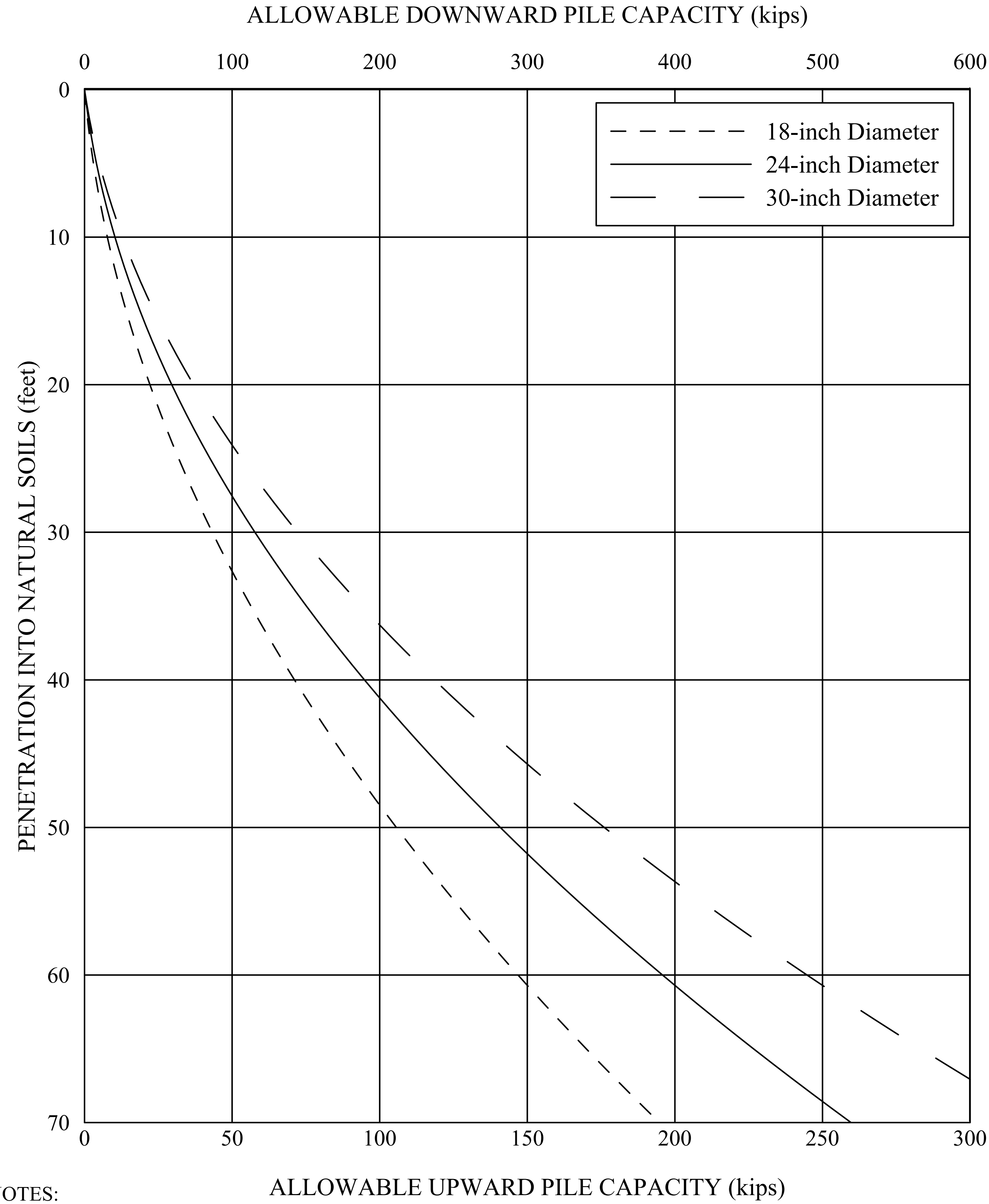
CONTINUOUS FOUNDATIONS
(Length is greater than or equal to 5 times the width)

Prepared/Date: WL 12/5/2016
Checked/Date: MM 12/5/2016

Proposed Compressor Station Upgrade Ventura Compressor Station 1555 North Olive Street Ventura, California	wood.	ESTIMATED SETTLEMENT Project No. 4953-16-1091 Figure 3.2

Figure 4

Drilled Pile Capacities



NOTES:

(1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 3 pile diameters on centers. Piles should be drilled and filled alternately with the concrete permitted to set at least 8 hours before drilling an adjacent hole.

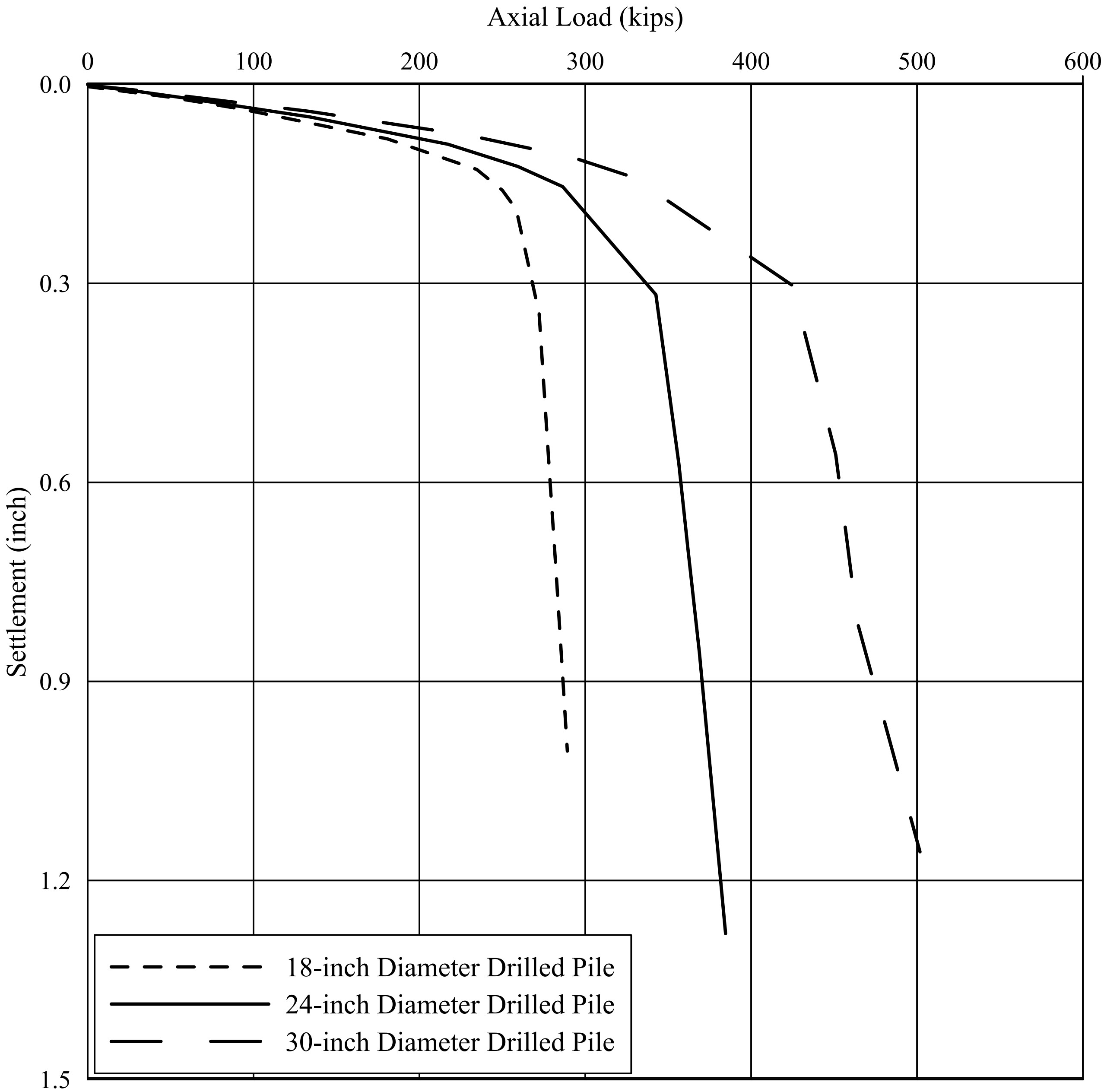
(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.

Prepared/Date: LH 11/7/16
Checked/Date: MM 11/7/16

Proposed Compressor Station Upgrade Ventura Compressor Station 1555 North Olive Street Ventura, California	wood.	DRILLED PILE CAPACITIES Project No. 4953-16-1091 Figure 4

Figure 5

Drilled Pile Stiffness Plot

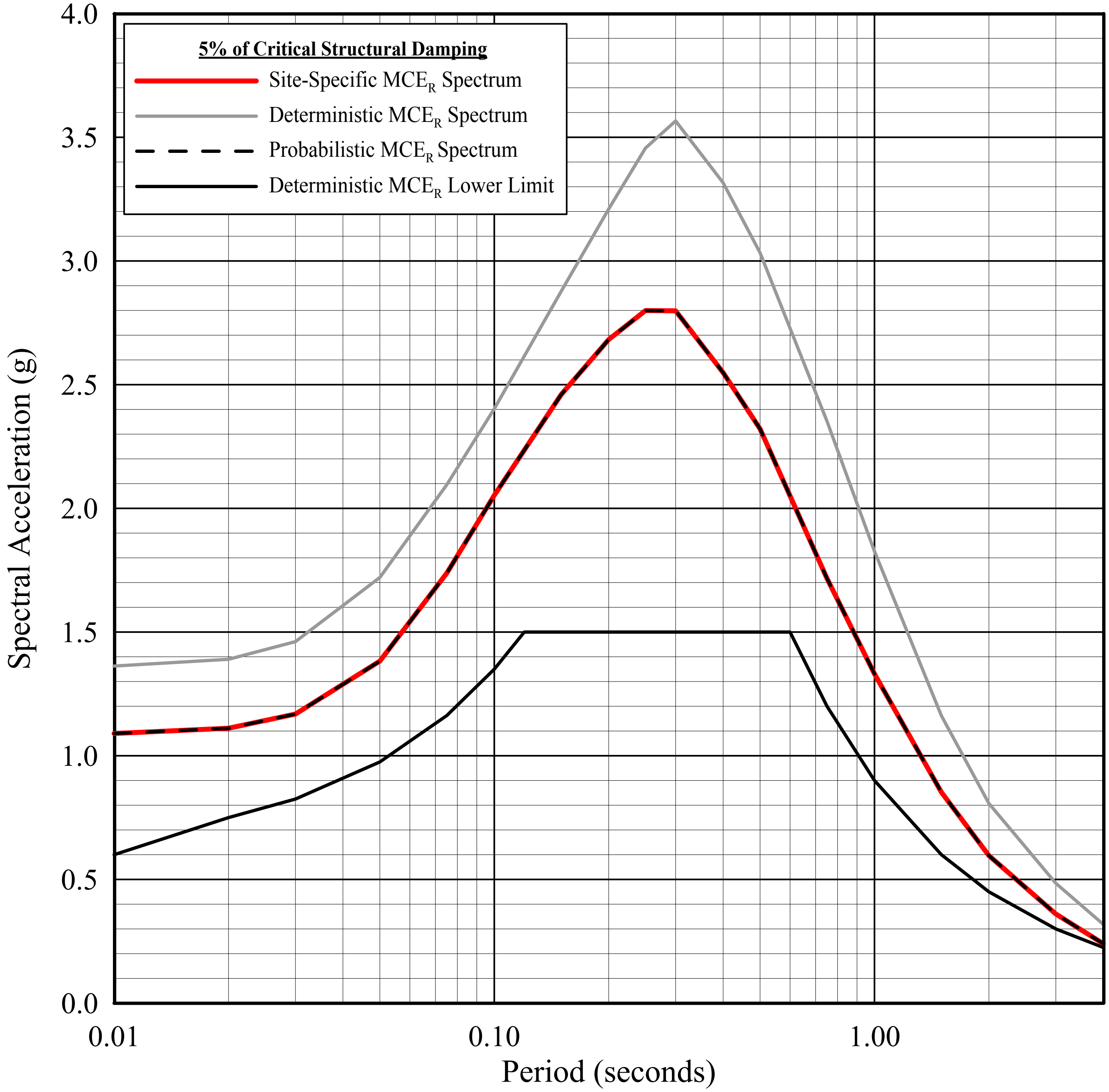


Prepared/Date: WL 12/5/16
Checked/Date: MM 12/5/16

Proposed Compressor Station Upgrade Ventura Compressor Station 1555 North Olive Street Ventura, California	wood.	DRILLED PILE STIFFNESS PLOT Project No. 4953-16-1091 Figure 5

Figure 6

Horizontal Response Spectra Components of the Risk-Targeted Mazimum Considered Earthquake (MCE_R)



NOTES: Probabilistic MCE$_R$ spectrum was computed for a ground motion level expected to achieve a 1% probability of collapse within a 50 year period.

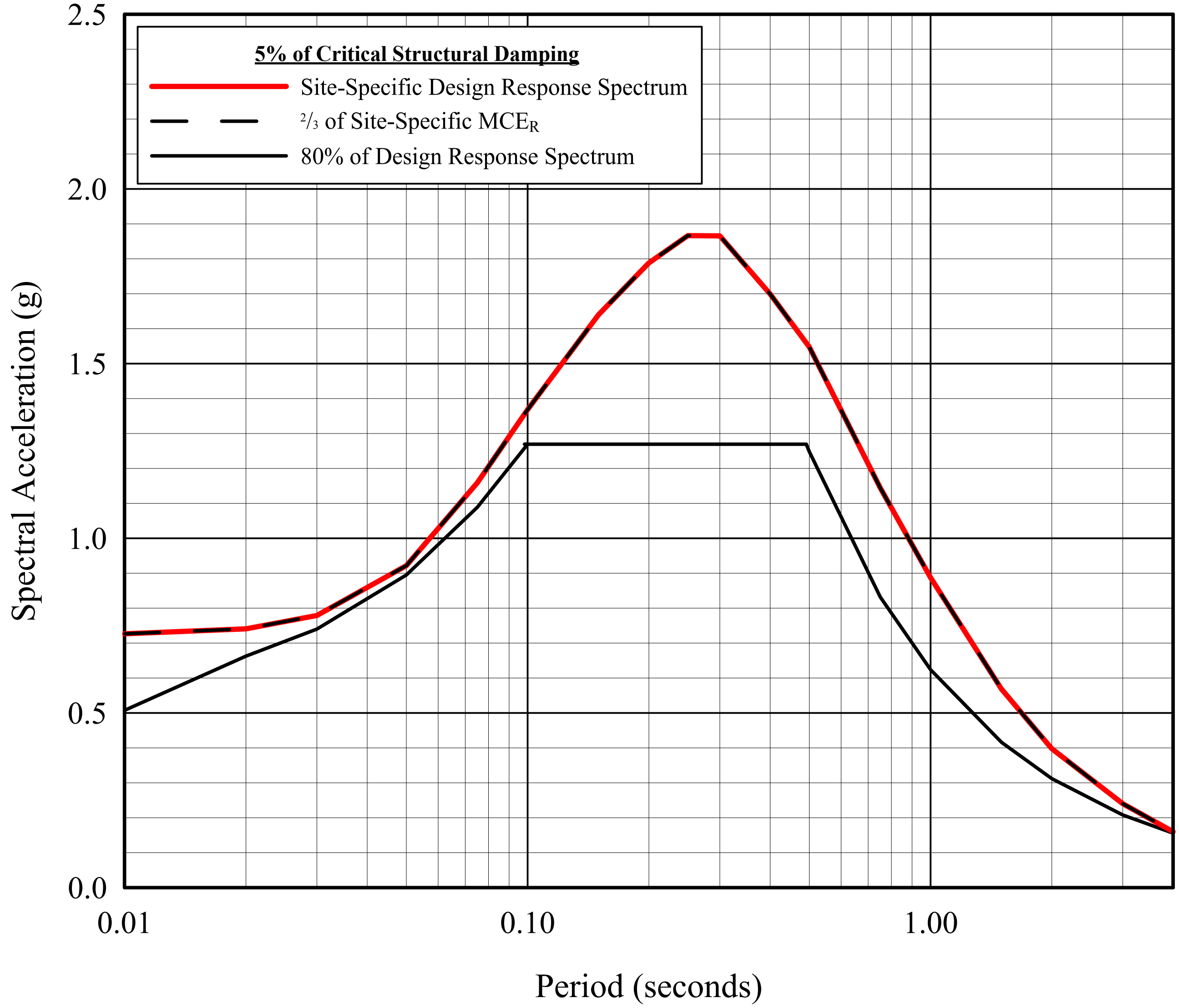
Deterministic MCE$_R$ spectrum is governed by a Magnitude 7.3 earthquake on the Pitas Point (Connected) fault.

Prepared/Date: LH 11/8/2016
Checked/Date: MM 11/8/2016

Proposed Compressor Station Upgrade Ventura Compressor Station 1555 North Olive Street Ventura, California	wood.	Horizontal Response Spectra Components of the Risk-Targeted Maximum Considered Earthquake (MCE$_R$) Project No. 4953-16-1091 Figure 6

Figure 7

Horizontal Response Spectra Components of the Design Response Spectrum



Prepared/Date: WL 11/8/2016
Checked/Date: MM 11/8/2016

Proposed Compressor Station Upgrade Ventura Compressor Station 1555 North Olive Street Ventura, California	wood.	Horizontal Response Spectra Components of the Design Response Spectrum Project No. 4953-16-1091 Figure 7

APPENDICES

Appendix A

FIELD EXPLORATIONS AND LABORATORY TEST RESULTS

Appendix A

Field Explorations and Laboratory Test Results

Current Field Explorations

The soil conditions beneath the site were explored by drilling 9 borings at the locations shown on Figure 2. The borings were drilled to depths of between 20 and 75½ feet below the existing grade using a 5-inch-diameter truck mounted mud rotary-wash drilling equipment.

The soils encountered were logged by our field technician, and undisturbed and bulk samples were obtained for laboratory inspection and testing. The logs of the borings are presented on Figures A-1.1 and A-1.9; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The number of blows required to drive the Modified California sampler 18 inches using a 140 pound hammer falling 30 inches is indicated on the logs. In addition to obtaining undisturbed samples, standard penetration tests (SPT) were also performed; the results of the tests are indicated on the logs. The soils are classified in the accordance with the Unified Soil Classification System described on Figure A-2.

Suspension logging was performed by *Geo*Vision in two of the borings (VCU6 and VCU7) located within the proposed footprint of the new compressor buildings. The results of the suspension logging are presented in a report prepared by *Geo*Vision, which is presented in Appendix C.

Current Laboratory Tests

Laboratory tests were performed by AP Engineering on selected samples obtained from the borings to aid in the classification of the soils and to evaluate their engineering properties.

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are presented to the left of the boring logs.

To aid in classification of the soils and to define the plasticity characteristics of the materials, Atterberg Limits tests were performed to determine the liquid limit and plastic limit of several of the samples. The testing procedure was in general accordance with ASTM Designation D4318. The results of the tests are shown on the boring logs.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field moisture content and after soaking to near-saturated moisture content and at various surcharge pressures. The results of the tests are presented on Figure A-3, Direct Shear Test Data.

Confined consolidation tests were performed on three selected undisturbed sample to determine the compressibility of the soils. Water was added to the sample during the tests to illustrate the effect of moisture on the compressibility. The results of the test are presented on Figure A-4, Consolidation Test Data.

To determine the particle size distribution of the soils and to aid in classifying the soils, mechanical analyses were performed on five samples in accordance with ASTM D6913. The results of the test are presented on Figure A-5, Particle-Size Distribution.

Laboratory soil permeability testing were performed on two selected sample to support the design of the waste spill-containment yard. The tests were performed in accordance with ASTM D2434. The results of the test are presented on Figure A-6, Constant Head Permeability Test

The corrosion studies of the near surface onsite soil samples were conducted by HDR. The results of the tests and the recommendations are summarized in the report presented on Appendix B.

BORING VCU1

DATE DRILLED: October 10, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 75 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
	0						SM	1½-inch thick gravel at the surface FILL - SILTY SAND - moist, light yellow, fine grained
								Some coarse grained, pieces of black plastic, some cobbles
							CL	FILL - SANDY LEAN CLAY - moist, olive brown, fine sand, some fine subrounded gravel (up to ½ inch in size)
70	5		11.5	122	50/2"		SP-SM	POORLY GRADED SAND with SILT and GRAVEL - very dense, moist, olive brown, fine grained, some medium, trace coarse, fine to coarse subrounded to subangular gravel (up to ¾ inch in size)
		28	8.1					Become medium dense, less gravel, more silt, some fine rounded to subrounded gravel (up to ½ inch in size)
65	10							
					21			Thin olive brown Clay interbed
							GP-GM	POORLY GRADED GRAVEL with SILT and SAND - very dense, moist, fine to coarse subrounded to subangular gravel (up to 3 inches in size), occasional cobbles and boulders, fine to coarse sand
60	15							(61% gravel; 11% Passing No. 200 Sieve)
		75/6"	9.0					
							SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive brown, fine to medium grained, fine to coarse subrounded to subangular gravel (up to ¾ inch in size)
55	20							Less gravel, fine rounded to subrounded gravel (up to ½ inch in size)
					98/10"			
							GP	POORLY GRADED GRAVEL with SAND - very dense, moist, dark gray, fine to coarse subrounded to subangular gravel (up to 3 inches in size), occasional cobbles
50	25	85	9.1					
							SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive brown, fine grained, trace to some medium, fine subrounded gravel (up to ½ inch in size)
								Less gravel
			13.8	115	72/8"			
45	30							END OF BORING AT 30 FEET
40	35							
	40							

NOTES:

Hand augered upper 3 feet to avoid damage to utilities. Groundwater was not encountered. Boring was backfilled with bentonite cement grout.

* Number of blows required to drive the last 12 of a total of 18 inches of the Modified California sampler using a 140-pound automatic hammer falling 30 inches.

** Elevations were obtained from topographic map provided by WorleyParsons.

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Field Tech: LH
Prepared By: JO
Checked By: JF

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BORING VCU2

DATE DRILLED: October 10, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 71 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
70							SM	1½-inch thick gravel at the surface FILL - SILTY SAND - moist, olive yellow, fine grained, trace to some fine to coarse subrounded gravel (up to ¾ inch in size)
							SP-SM	FILL - POORLY GRADED SAND with SILT - moist, olive, fine grained, some medium, some fine rounded gravel (up to ¼ inch in size)
	5							
65		69/11"	7.7					Some fine to coarse subrounded to subangular gravel (up to 1 inch in size)
								Less gravel
			12.3	123	83/11"			
	10						SM	FILL - SILTY SAND - moist, dark gray, fine grained
60							SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive, fine to medium grained, some coarse, fine to coarse subrounded gravel (up to ¾ inch in size)
		50/6"	19.5					Increase in gravel content
	15							
55					88			Becomes dark olive, more fine gravel, some silt
								Some pockets of gravel
		50/3"	14.7				GP	POORLY GRADED GRAVEL with SAND - very dense, moist, pale olive gray, fine rounded to subrounded gravel (up to ½ inch in size) , fine sand
	20							END OF BORING AT 20 FEET
50								
	25							
45								
	30							
40								
	35							
35								
	40							

Hand augered upper 5 feet to avoid damage to utilities. Groundwater was not encountered. Boring was backfilled with bentonite cement grout.

Field Tech: LH
Prepared By: JO
Checked By: JF

BORING VCU3

DATE DRILLED: October 7, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 69 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
	5		7.6	139	50/5"	
60		24	17.1			
	10				50/1"	
55	15	60	13.5			
50	20				100	
45	25	100/8"	10.7			
40	30				50/2"	
35	35	42	16.0			
30	40					

USCS	DESCRIPTION
	2½-inch thick Asphalt Concrete over 2½-inch thick Base Course
CL	FILL - SANDY LEAN CLAY - moist, olive brown, fine sand, trace fine subrounded gravel (up to ½ inch in size)
SP	FILL - POORLY GRADED SAND with GRAVEL - moist, dark gray, fine grained, some medium, fine to coarse subrounded gravel (up to 1 inch in size)
	Boulder to size of 1 foot in diameter
SP	POORLY GRADED SAND - medium dense, moist, fine grained, trace medium, some fine rounded to subrounded gravel (up to 1/2 inch in size)
GP	POORLY GRADED GRAVEL with SAND - very dense, moist, olive with mottled white, gray and green, fine to coarse subrounded to subangular gravel (up to 2 inches in size), fine to medium sand, some coarse, (sample not recovered)
	More gravel, some pockets of fine sand, some medium sand, occasional cobble
	More cobbles and boulders
	Some coarse sand, subrounded to subangular cobble (up to 3¼ inches in size), thin interbedded sand layer
	Boulder up to 8 inches in diameter
SP	POORLY GRADED SAND with GRAVEL - very dense, moist, dark olive gray, fine to medium grained, some coarse, fine to coarse rounded to subrounded gravel (up to 1½ inches in size), some small clay nodules
	Some clay interbeds/nodules
	(Sample not recovered), slightly less gravel, olive brown, fine grained, some medium, fine rounded gravel (up to ½ inch in size)
SW	WELL-GRADED SAND with GRAVEL - dense, moist, dark olive gray, fine to coarse grained, fine rounded to subrounded gravel (up to ½ inch in size), strong petroleum odor, some iron oxide stains
SP	POORLY GRADED SAND with GRAVEL - very dense, moist to wet, dark gray, fine to medium grained, some coarse, fine to coarse subrounded

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Field Tech: LH
Prepared By: JO
Checked By: JF

(CONTINUED ON FOLLOWING FIGURE)

BORING VCU3 (Continued)

DATE DRILLED: October 7, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 69 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
			14.9	114	50/4"			to subangular gravel (up to ¾ inch in size), strong petroleum odor
25	45	31	23.4				CL-ML	SANDY SILTY CLAY - hard, wet, dark greenish gray, fine sand, (57% Passing No. 200 Sieve, LL=23, PI=5)
							SP	POORLY GRADED SAND with GRAVEL - very dense, wet, light gray with mottled red, fine grained, some medium to coarse, fine to coarse subrounded gravel (up to ¾ inch in size)
20	50				50/4"			Increase in gravel content
								END OF BORING AT 50½ FEET
15	55							
10	60							
5	65							
0	70							
-5	75							
-10	80							

NOTES:

Hand augered upper 2½ feet to avoid damage to utilities. Groundwater not measured. Boring was backfilled with bentonite cement grout.

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Field Tech: LH
Prepared By: JO
Checked By: JF

BORING VCU4

DATE DRILLED: September 29-30, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 67 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
65						
	5					
		89/11"	10.2			
60						
			7.4		50/3"	
	10					
		46	6.3			
55						
					50/3"	
	15					
		69/11"	12.3			
50						
	20					
			9.8	133	25/0.5"	
45						
	25					
		147/11"	10.3			
40						
	30					
			7.5	115	85/11"	
35						
	35					
		92/7"	8.3			
30						
	40					

USCS	DESCRIPTION
	3½-inch thick Asphalt Concrete over 1-inch thick Base Course
CL	FILL - LEAN CLAY - moist, olive brown, fine sand, some fine to coarse subrounded gravel (up to 2 inches in size)
GP	FILL - POORLY GRADED GRAVEL with SAND - moist, dark gray, subrounded to subangular gravel, 2-inch diameter cobble, fine to medium sand
	Boulder to the size of 2 feet
SP	POORLY GRADED SAND with GRAVEL - very dense, moist, gray, fine to medium grained, some coarse, fine to coarse subrounded to rounded gravel (up to 1¼ inch in size), some petroleum odor, cobbles encountered
GP-GM	POORLY GRADED GRAVEL with SILT and SAND - very dense to dense, moist, gray, fine to coarse subrounded gravel (up to 3 inches in size), fine to medium sand, some coarse, occasional cobble
	(64% gravel; 9% Passing No. 200 Sieve)
	More cobbles
	(Sample not recovered), cobbles from 14 to 15 feet
	Becomes pale gray to dark bluish gray, pockets of fine to coarse sand
SP	POORLY GRADED SAND with GRAVEL - very dense, moist, gray, fine to medium grained, some coarse, fine to coarse rounded to subrounded gravel (up to ¾ inch in size)
	Cobble at 23 feet
GP	POORLY GRADED GRAVEL with SAND - very dense, moist, gray, fine to coarse subrounded to subangular gravel (up to 3 inches in size), some cobbles, fine to medium sand, some pockets of pale green fine sand, some clay nodules
SP	POORLY GRADED SAND - very dense, moist, olive brown, fine grained, trace to some medium
	Thin layer of Well-Graded Sand, moist, dark gray, fine to coarse grained, trace to some fine subrounded gravel (up to ½ inch in size)
SP	POORLY GRADED SAND with GRAVEL - very dense, moist, dark gray, some petroleum odor
	Some clay nodules, strong petroleum odor

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(CONTINUED ON FOLLOWING FIGURE)

Field Tech: LH
Prepared By: JO
Checked By: JF

BORING VCU4 (Continued)

DATE DRILLED: September 29-30, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 67 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
25		52/4"	10.0				SW	WELL-GRADED SAND with GRAVEL - very dense, moist, dark gray, fine to coarse grained, fine to coarse rounded to subrounded gravel (up to ¾ inch in size), small clay nodules
	45				50/3"			(Sample not recovered)
20		50/1"						(Sample not recovered)
					50/0.5"			(Sample not recovered)
15	50	53/6"	6.8					
								END OF BORING AT 51½ FEET
10	55							
5	60							
0	65							
-5	70							
-10	75							
	80							

NOTES:

Hand augered upper 3 feet to avoid damage to utilities. Groundwater was measured at 40.7 feet below the ground surface 15 minutes after drilling. Severe caving due to gravel. Boring was backfilled with bentonite cement grout.

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Field Tech: LH
Prepared By: JO
Checked By: JF

Proposed Southern California Gas Compressor Station
1555 North Olive Street
Ventura, California

wood.

LOG OF BORING
Project: 4953-16-1091 Figure: A-1.4b

BORING VCU5

DATE DRILLED: October 5-6, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 64 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
	0							2½-inch thick Asphalt Concrete over 1½-inch thick Base Course
							CL	FILL - SANDY CLAY - moist, olive to olive brown, fine sand, some medium, some fine subrounded gravel (up to ½ inch in size)
							GP	FILL - POORLY GRADED GRAVEL with SAND - moist, dark gray, fine to coarse subrounded to subangular gravel (up to 3 inches in size), occasional cobble up to 8 inches in diameter at 3 ft
60							GP	POORLY GRADED GRAVEL with SAND - very dense, moist, dark gray, fine to coarse subrounded to subangular gravel (up to 3 inches in size), occasional cobbles
	5		15.5	111	50/3"		SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive brown to reddish brown, fine grained, some medium, fine to coarse subrounded gravel (up to 2½ inches in size)
55		102	7.4				GW-GM	WELL-GRADED GRAVEL with SILT and SAND - very dense, moist, fine to coarse subrounded to subangular gravel (up to 3 inches in size), with interbedded olive to brown fine-grained Poorly Graded Sand layers, some cobbles up to 4 inches in diameter, (63% gravel; 12% Passing No. 200 Sieve)
	10		16.9		73			
50							SP	POORLY GRADED SAND with GRAVEL - dense, moist, olive yellow to light olive brown, fine to medium grained, fine to coarse subrounded gravel (up to 3 inches in size), occasional cobbles
	15	115/10"	8.0				GP-GM	POORLY GRADED GRAVEL with SILT and SAND - very dense, moist, gray, fine to coarse subrounded gravel (up to 3 inches in size), pockets and thin interbeds of olive yellow fine-grained Poorly Graded Sand, (71% gravel; 8% Passing No. 200 Sieve)
45								Encounter 1¼-foot boulder
	20		10.1	124	50/3"		SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive yellow, fine to medium grained, some coarse, fine to coarse subrounded gravel (up to 3 inches in size), occasional cobble up to 3½ inches in diameter
40							GP	POORLY GRADED GRAVEL with SAND - very dense, moist, gray, fine to coarse rounded to subrounded gravel (up to 2 inches in size), fine sand
	25	68/8.5"	19.6					
35								Becomes bluish green, fine to medium sand
	30		8.5		50/5"			
								END OF BORING AT 31 FEET
30								
	35							
25								
	40							

NOTES:

Hand augered upper 2½ feet to avoid damage to utilities. Groundwater was not encountered. Boring was backfilled with bentonite cement grout.

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B12SOIL_CRANDALL (NO DECIMAL) C:\USERS\WYATT.IWANAGA\DESKTOP\WYATT_GINT.GLB
P:\4953 GEOTECH\2016-PROJ\161091 VENTURA COMPRESSOR STATION UPGRADE\3.2 ALL FIELD NOTES\4953-16-1091.GPJ 4/3/19

Field Tech: LH
Prepared By: JO
Checked By: JF

BORING VCU6

DATE DRILLED: October 3, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 67 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
								2½-inch thick Asphalt Concrete over 1-inch thick Base Course
							CL	FILL - SANDY CLAY - moist, olive brown, fine sand, some medium
65							GP	FILL - POORLY GRADED GRAVEL with SAND - moist, olive to olive brown, fine to coarse subrounded gravel (up to 3 inches in size), fine sand, occasional cobbles
	5						GP	POORLY GRADED GRAVEL with SAND - very dense, moist, olive, fine to coarse subrounded to subangular gravel (up to 3 inches in size), fine sand, some medium, trace coarse, occasional cobbles
			19.6		58/3"			
60							SP	POORLY GRADED SAND with GRAVEL - dense, moist, olive brown, fine grained, some medium, fine to coarse rounded to subrounded gravel (up to 3 inches in size), occasional cobble
		42	10.0					
								8-inch diameter cobble at 9.5 feet, increase in gravel content
	10							
			5.9	141	79			
55								
		50/4"	4.8					Becomes very dense, fine to medium grained, some coarse, fine to coarse subrounded gravel up to 1½ inches in diameter
	15						GP	POORLY GRADED GRAVEL with SAND - very dense, moist, olive brown, fine to coarse subrounded to subangular gravel (up to 3 inches in size), occasional cobbles, fine sand, some medium to coarse, some iron oxide stains
			7.4	135	91			
50								
								Boulder up to 1½ feet in diameter
	20							
		80	9.9					Becomes dark gray and olive, fine to coarse subrounded gravel (up to 2½ inches in size), some fine to medium sand pockets, trace coarse sand, some iron oxide stains
45								
								Cobble up to 5 inches in diameter
							CL	SANDY LEAN CLAY - hard, moist, olive, fine sand, some medium, some fine to coarse rounded to subrounded gravel (up to 2 inches in size)
					92/8"			
	25							
40								
								Becomes olive with mottled green, olive brown, and red (50% Passing No. 200 Sieve)
			9.2	126	68			
	30						SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive brown to reddish brown, fine to medium grained, some coarse, fine subrounded gravel (up to to ½ inch in size)
35								
		59	7.8					Becomes fine grained, some medium to coarse, fine to coarse rounded to subrounded gravel (up to ½ inch in size)
	35						SP	POORLY GRADED SAND - very dense, moist, olive brown to reddish brown, fine to medium grained, some coarse, some fine subrounded to subangular gravel (up to ½ inch in size)
			8.0	129	93/10"			
30							SP	POORLY GRADED SAND with GRAVEL - very dense, moist, dark gray with mottled greenish blue, yellow and red, fine to medium grained, some coarse, fine to coarse subrounded to subangular gravel (up to 2 inches in size)
	40							

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(CONTINUED ON FOLLOWING FIGURE)

Field Tech: LH
Prepared By: JO
Checked By: JF

BORING VCU6 (Continued)

DATE DRILLED: October 3, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 67 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	USCS	DESCRIPTION
		92	17.4				Some pockets of yellow silt, portion of silt is extremely weakly cemented, strong petroleum odor
25							6-inch diameter cobble
	45	86	12.9			SP	POORLY GRADED SAND - very dense, moist, olive gray, fine to medium grained, some coarse, some fine to coarse subrounded to rounded gravel (up to ¾ inch in size), strong petroleum odor
20						SP	POORLY GRADED SAND with GRAVEL - very dense, wet, gray and green, fine to medium grained, some coarse, fine to coarse subrounded to subangular gravel (up to 2½ inches in size)
	50	50/4"					
15							
	55				50/2"	GP	POORLY GRADED GRAVEL with SAND - very dense, wet, light gray, fine to coarse subrounded to rounded gravel (up to 3 inches in size), occasional cobbles, (sample not recovered)
10							5-inch diameter cobble
	60	50/2"					(Sample not recovered)
5							Alternating layers of Poorly Graded Gravel and Poorly Graded Sand
	65	50/4"					(Sample not recovered)
0							5-inch diameter cobble
	70	50/5"	8.6				Subrounded gravel (up to 2 inches in size), fine sand
-5							Layer of Lean Clay
	75	62/3.5"					Fine to coarse gravel (up to 1 inch in size), some clay, (sample not recovered)
-10							
	80						

END OF BORING AT 75½ FEET
NOTES:
Hand augered upper 2 feet to avoid damage to utilities. Suspension logging performed from 0 to 59½ feet. Drilling mud bailed on 10/4/2016. Groundwater level was measured at 45 feet below the ground surface on 10/5/2016. Boring was backfilled with bentonite cement grout.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: LH
Prepared By: JO
Checked By: JF

BORING VCU7

DATE DRILLED: September 27-28, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 69 **

DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)
5.5	45	17.8		
8		30.1	94	17
10.5	30			
13		8.8		34
15.5	62	10.9		
20.5		8.6	133	96
25.5	51	9.1		
30.5		11.0	122	74
35.5	50/2"	9.8		

USCS	DESCRIPTION
	3-7/8-inch thick Asphalt Concrete over 1-inch thick Base Course
CL	FILL - SANDY CLAY - moist, light olive gray to gray, fine sand, some fine to coarse rounded to subrounded gravel (up to 3 inches in size)
GP	FILL - POORLY GRADED GRAVEL with SAND - moist, dark brown gray, fine to coarse subrounded to subangular gravel (up to 3 inches in size), cobbles up to 5 inches in diameter, fine sand, trace to some medium
SP	FILL - POORLY GRADED SAND - moist, olive brown, fine grained, some medium
CL	LEAN CLAY - stiff to hard, moist, olive brown to dark brown, some fine sand, (LL=31; PI=10)
SP	POORLY GRADED SAND - medium dense, moist, olive brown, fine grained, some medium, some fine to coarse subrounded gravel (up to 1½ inches in size)
SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive brown ,fine to medium grained, some coarse, fine to coarse subrounded to subangular gravel (up to 1¾ inches in size), occasional cobble
	Fine to coarse rounded to subrounded gravel (up to 1½ inches in size)
	Becomes pale greenish gray, fine to medium grained, some coarse, fine to coarse subrounded to subangular gravel (up to 1 inch in size), some clay nodules
CL	LEAN CLAY - moist, reddish brown, some fine sand
SW	WELL-GRADED SAND with GRAVEL - very dense, moist, bluish gray, fine to coarse grained, fine to coarse subrounded to subangular gravel (up to 1 inch in size), occasional cobbles
CL	LEAN CLAY - hard, moist, reddish to olive brown, trace fine sand
GP	POORLY GRADED GRAVEL with SAND - very dense, moist, fine to coarse subrounded to subangular gravel (up to 2 inches in size), occasional cobble, fine to medium sand, some coarse, trace iron oxide stains

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: LH
Prepared By: JO
Checked By: JF

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

BORING VCU7 (Continued)

DATE DRILLED: September 27-28, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 69 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
			12.1	111	94/11"		SP-SM	POORLY GRADED SAND with SILT - very dense, wet, bluish gray, fine grained, trace to some medium, no gravel, strong petroleum odor, (10% Passing No. 200 Sieve)
25	45	50/3"						(Sample not recovered)
							SP	POORLY GRADED SAND with GRAVEL - very dense, wet, brownish gray, fine to medium grained, some coarse, fine to coarse subrounded gravel (up to 2 inches in size)
20	50	50/1"						(Sample not recovered)
15	55	50/3"						(Sample not recovered)
10	60	50/2"						(Sample not recovered)
5	65	50/2"						Becomes light olive brown to gray, fine to coarse rounded to subrounded gravel (up to 2 inches in size)
								END OF BORING AT 65.2 FEET
0	70							
-5	75							
-10	80							

NOTES:

Hand augered upper 2 feet to avoid damage to utilities. Suspension logging performed from 0 to 50 feet. Drilling mud bailed on 9/28/2016. Groundwater level measured at 42 feet below the ground surface on 9/29/2016. Boring was backfilled with bentonite cement grout.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

B12SOIL_CRANDALL (NO DECIMAL) C:\USERS\WYATT.IWANAGA\DESKTOP\WYATT_GINT.GLB
P:\4953 GEOTECH\2016-PROJ\161091 VENTURA COMPRESSOR STATION UPGRADE\3.2 ALL FIELD NOTES\4953-16-1091.GPJ 4/3/19

Field Tech: LH
Prepared By: JO
Checked By: JF

BORING VCU8

DATE DRILLED: October 6, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 63 **

"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)
	12.9	118	14
12	27.8		
	12.0	130	50/3"
98/8"	10.2		
			96
59	10.7		
	12.2	130	50/3"

USCS	Description
CL	2-7/8-inch thick Asphalt Concrete FILL - SANDY CLAY - moist, olive brown, fine sand, trace fine subrounded gravel (up to ½ inch in size)
SM	FILL - SILTY SAND - moist, olive brown, fine grained, some medium to coarse, trace to some fine subrounded gravel (up to ½ inch in size)
SC	FILL - CLAYEY SAND - moist, olive, fine grained, some medium, some fine to coarse subrounded gravel (up to ¾ inch in size) (44% Passing No. 200 Sieve)
SP-SM	FILL - POORLY GRADED SAND with SILT and GRAVEL - moist, olive brown, fine grained, some medium to coarse, some fine to coarse rounded to subrounded gravel (up to ½ inch in size)
GP	POORLY GRADED GRAVEL with SAND - very dense, moist, gray, fine to coarse rounded to subrounded gravel (up to 2½ inches in size), fine to medium sand, trace to some coarse
SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive brown, fine grained, some medium, trace coarse, fine to coarse rounded to subrounded gravel (up to 1½ inches in size), some interbedded thin Poorly Graded Gravel layers Slightly more gravel More medium sand

END OF BORING AT 31½ FEET

NOTES:

Hand augered upper 3 feet to avoid damage to utilities. Groundwater was not encountered. Boring was backfilled with bentonite cement grout.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

B12SOIL_CRANDALL (NO DECIMAL) C:\USERS\WYATT.IWANAGA\DESKTOP\WYATT_GINT.GLB
P:\4953 GEOTECH\2016-PROJ\161091 VENTURA COMPRESSOR STATION UPGRADE\3.2 ALL FIELD NOTES\4953-16-1091.GPJ 4/3/19

Field Tech: LH
Prepared By: JO
Checked By: JF

BORING VCU9

DATE DRILLED: September 27, 2016
EQUIPMENT USED: Mud Rotary Wash (Truck-Mounted Mayhew 1000)
HOLE DIAMETER (in.): 4-7/8
ELEVATION (ft.): 62 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
62	0							3-7/8-inch thick Asphalt Concrete
60							CL	FILL - SANDY CLAY - moist, olive to olive brown, fine sand, trace medium, trace fine gravel (up to ½ inch in size)
							GP	FILL - POORLY GRADED GRAVEL with SAND - moist, dark gray to brownish gray, fine gravel, cobbles up to 1 foot in diameter, fine to medium sand, some roots
	5	67	6.8				SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive brown, fine grained, some medium, fine to coarse subrounded to subangular gravel (up to 2 inches in size), occasional cobble
55					50/2"		GM	SILTY GRAVEL with SAND - very dense, moist, dark gray, fine to coarse subrounded gravel (up to 3 inches in size), occasional cobbles (up to 5 inches in diameter), fine sand, some pockets of Silty Sand
	10	50/3"						(Sample not recovered) Encountered cobble and boulder
50								Encountered cobble and boulder
	15	67/9"	10.9					Thin interbeds of Poorly Graded Sand, pockets of clay, fine to coarse rounded to subangular gravel (up to 3 inches in size), occasional cobbles (up to 5 inches in diameter), (39% gravel; 25% Passing No. 200 Sieve)
45								6-inch diameter cobble
							SP	POORLY GRADED SAND with GRAVEL - very dense, moist, olive brown, fine grained, trace to some medium, fine to coarse subrounded to subangular gravel (up to 2½ inches in size)
	20				50/4"			
40								Slightly more sand, small calcium carbonate nodules
	25	50/4"						
35								
	30		11.7		50/3"			Fine to medium grained, fine to coarse rounded to subrounded gravel (up to 2 inches in size)
30								END OF BORING AT 31½ FEET
	35							
25								
	40							

NOTES:

Hand augered upper 3 feet to avoid damage to utilities. Groundwater was not encountered. Severe caving during drilling due to gravel. Boring was backfilled with bentonite cement grout.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

B12SOIL_CRANDALL (NO DECIMAL) C:\USERS\WYATT.IWANAGA\DESKTOP\WYATT_GINT.GLB
P:\4953 GEOTECH\2016-PROJ\161091 VENTURA COMPRESSOR STATION UPGRADE\3.2 ALL FIELD NOTES\4953-16-1091.GPJ 4/3/19

Field Tech: LH
Prepared By: JO
Checked By: JF

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel - sand - silt mixtures.
			GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand - silt mixtures
			SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
			CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays
BEDROCK				SANDSTONE
				SILTSTONE
				GRANITE

Undisturbed Sample	Auger Cuttings
Split Spoon Sample	Bulk Sample
Rock Core	Crandall Sampler
Dilatometer	Modified California Sampler
Packer	No Recovery
Water Table at time of drilling	Water Table after drilling

Correlation of Penetration Resistance with Relative Density and Consistency

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 30	Medium Dense	5 - 8	Medium Stiff
31 - 50	Dense	9 - 15	Stiff
Over 50	Very Dense	16 - 30	Very Stiff
		Over 30	Hard

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

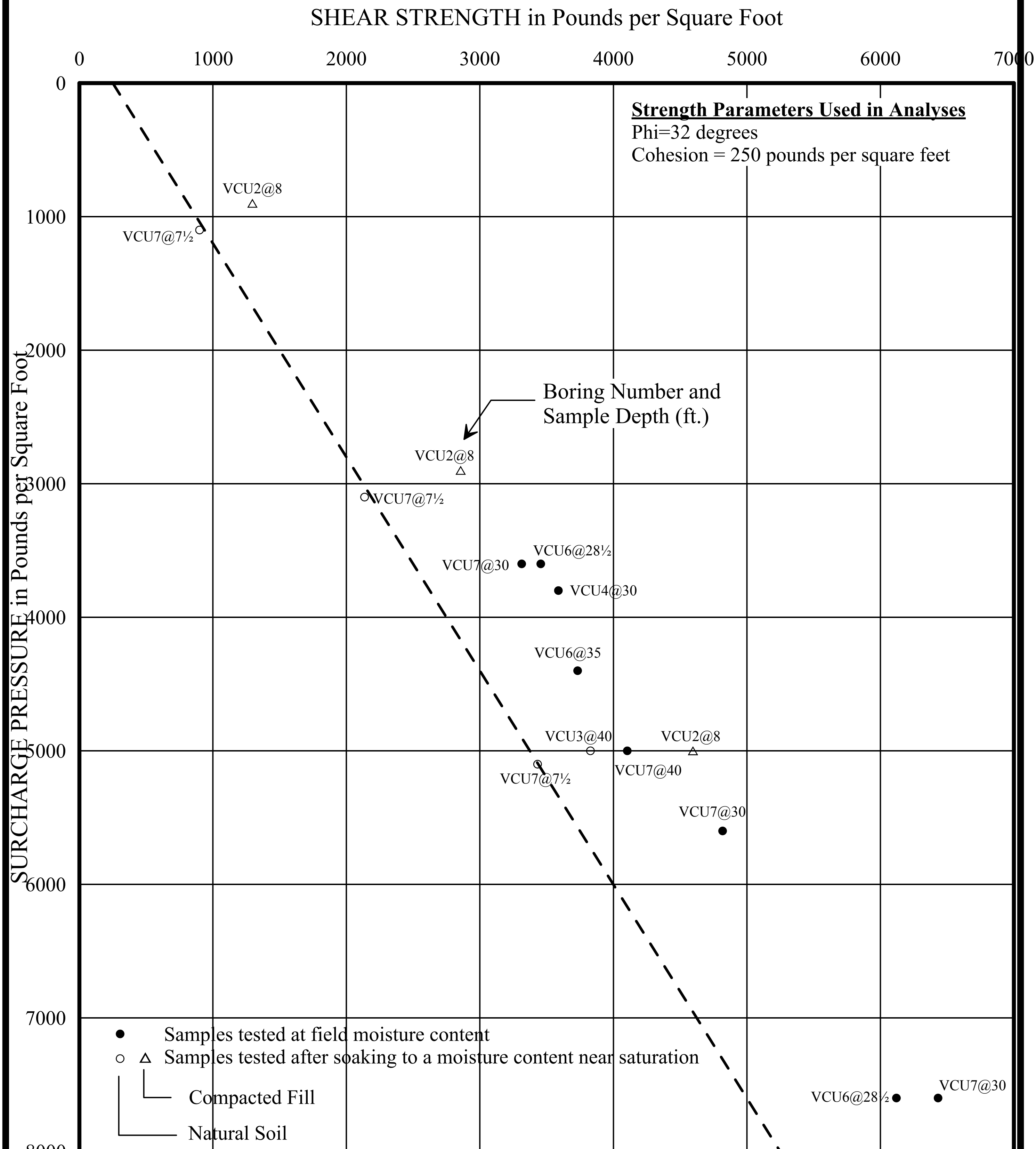
SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

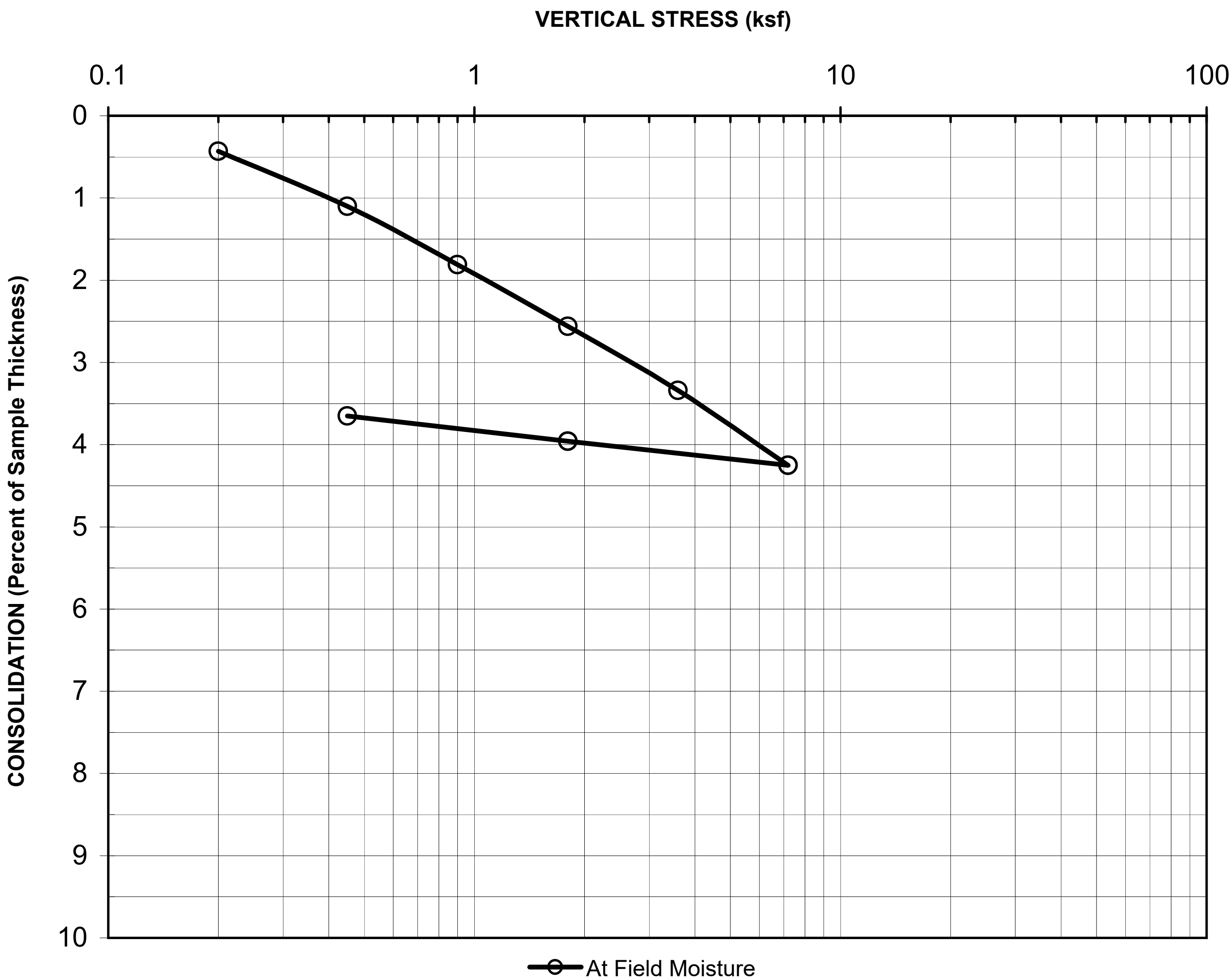
KEY TO SYMBOLS AND DESCRIPTIONS

Figure A-2



Prepared/Date: LH 11/3/2016
Checked/Date: JF 11/8/2016

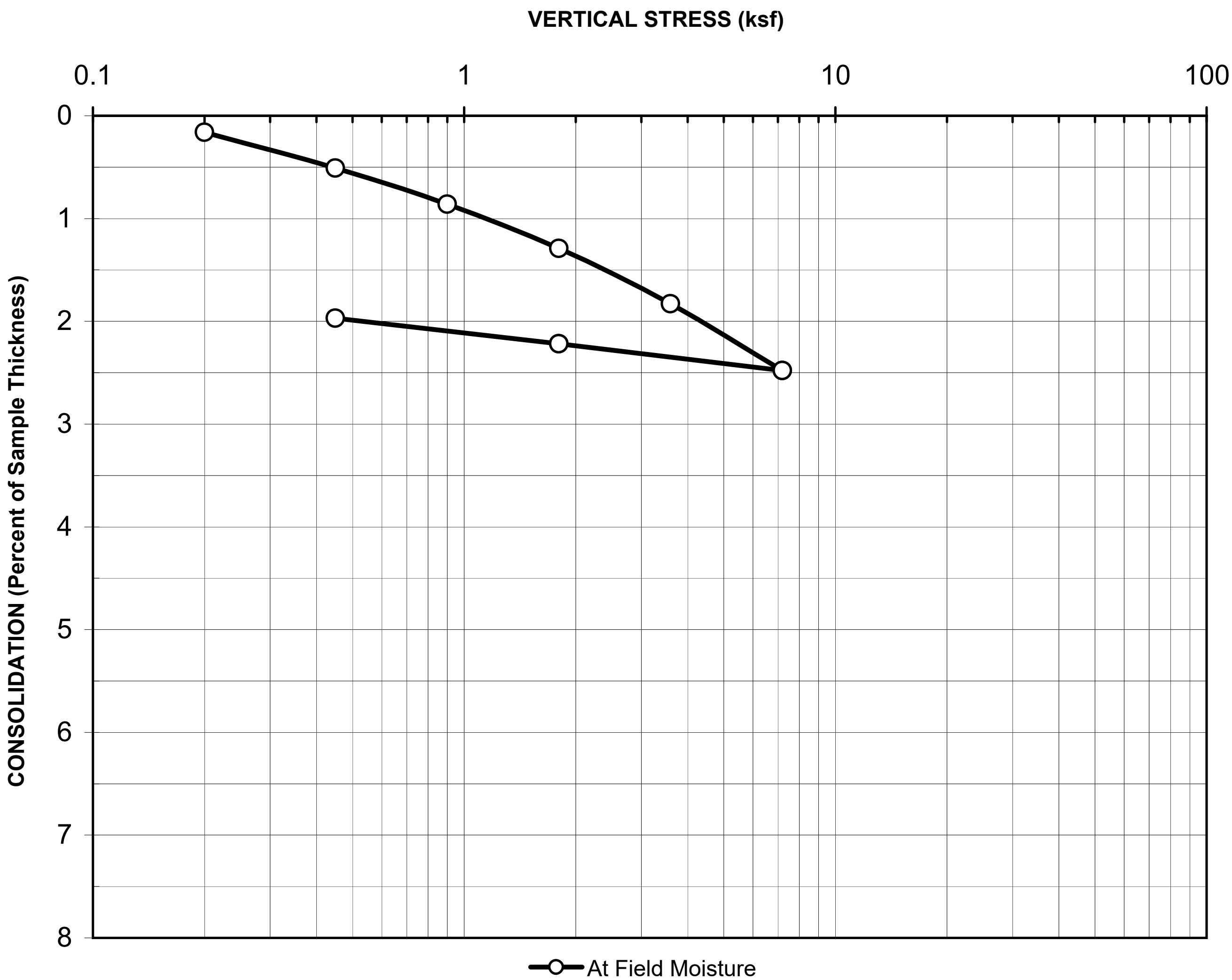
Proposed Compressor Station Upgrade Southern California Gas Company 1555 North Olive Street Ventura, California	wood.	DIRECT SHEAR TEST DATA Project No. 4953-16-1091 Figure A-3



Boring No. :	VCU4	Initial Dry Unit Weight (pcf):	114.4
Sample No.:	9	Initial Moisture Content (%):	7.5
Depth (feet):	30-31.5	Final Moisture Content (%):	7.4
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Well-Graded Sand	Initial Void Ratio:	0.47
Remarks:	Sample was not inundated.		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	SCG - Ventura
Project No.:	4953-16-1091
Date:	10/24/2016
AP No:	16-1023



Boring No. :	VCU6	Initial Dry Unit Weight (pcf):	127.3
Sample No.:	9	Initial Moisture Content (%):	9.2
Depth (feet):	28.5-30	Final Moisture Content (%):	8.4
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Clay	Initial Void Ratio:	0.32
Remarks:	Sample was not inundated.		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	SCG - Ventura
Project No.:	4953-16-1091
Date:	11/1/2016
AP No:	16-1023

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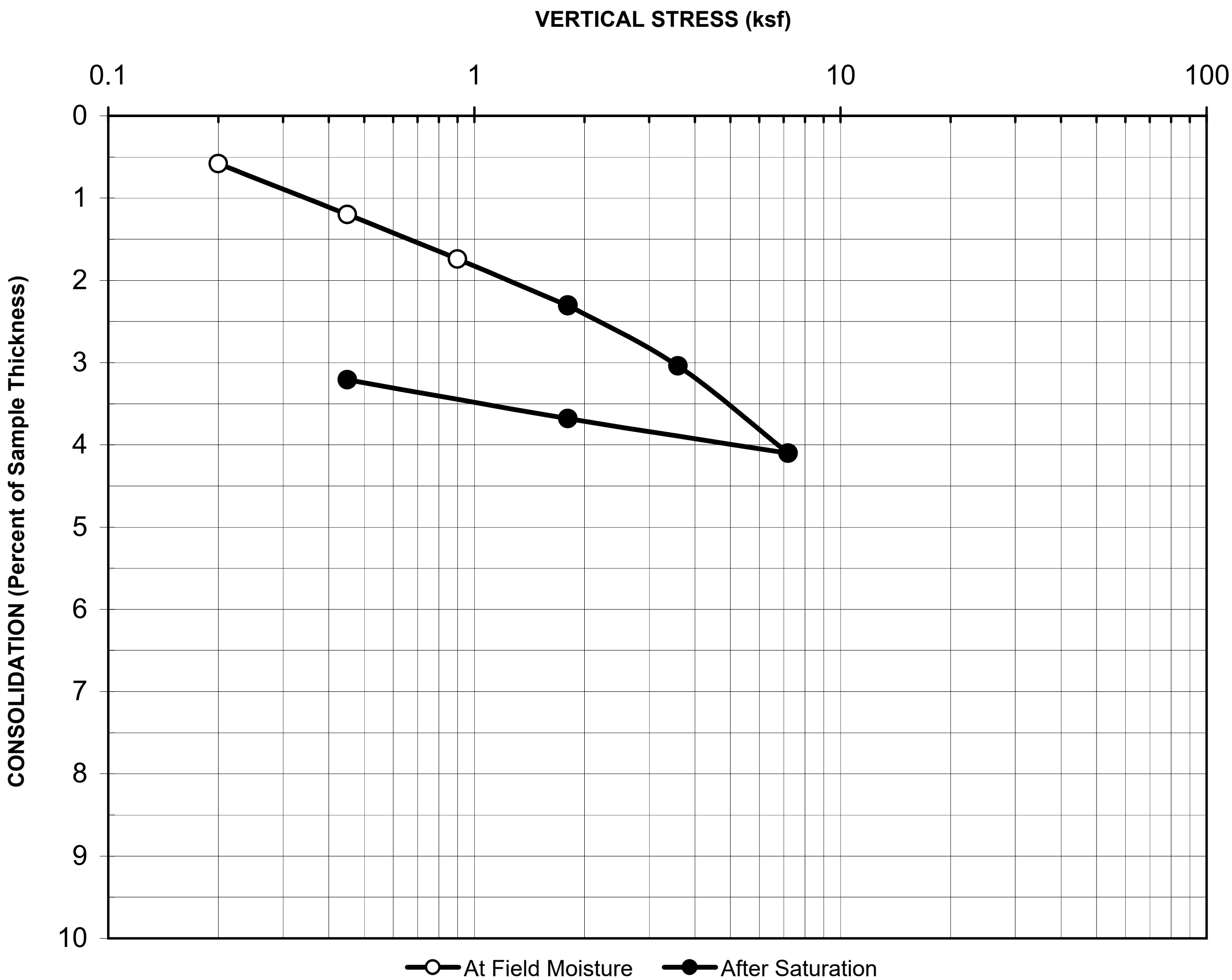

Boring No. :	VCU7	Initial Dry Unit Weight (pcf):	103.2
Sample No.:	2	Initial Moisture Content (%):	23.4
Depth (feet):	7.5-9	Final Moisture Content (%):	22.3
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Lean Clay	Initial Void Ratio:	0.63
Remarks:	Swell= 0.01% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	SCG - Ventura
Project No.:	4953-16-1091
Date:	10/24/2016
AP No:	16-1023

Figure A-4.3

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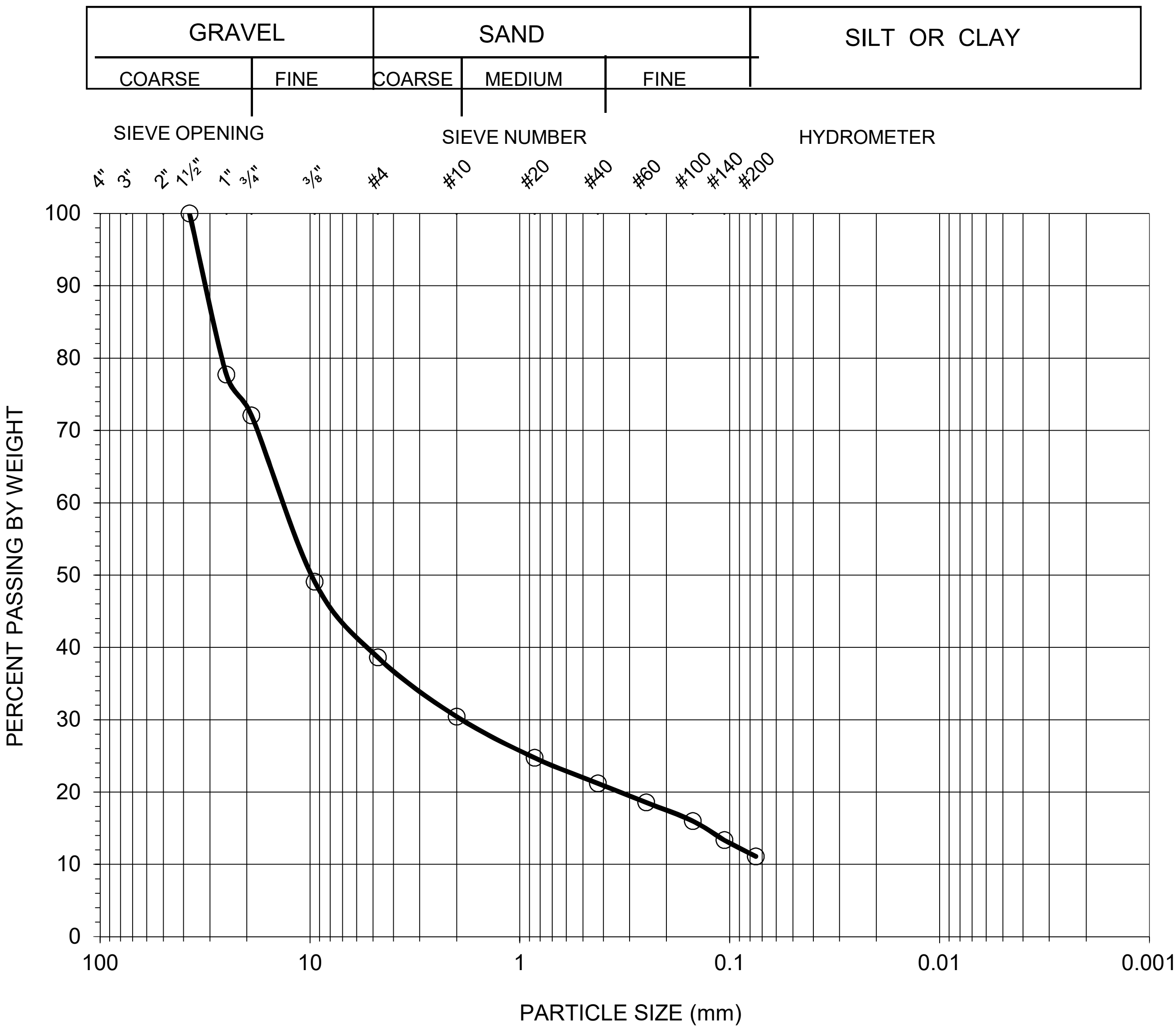
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AMEC Foster Wheeler	Tested by:	ST	Date:	10/26/16
Project Name:	SCG - Ventura	Computed by:	JP	Date:	10/31/16
Project Number:	4953-16-1091	Checked by:	AP	Date:	10/31/16



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	VCU1	5	15.75-17.25	61	28	11	N/A	GP-GM

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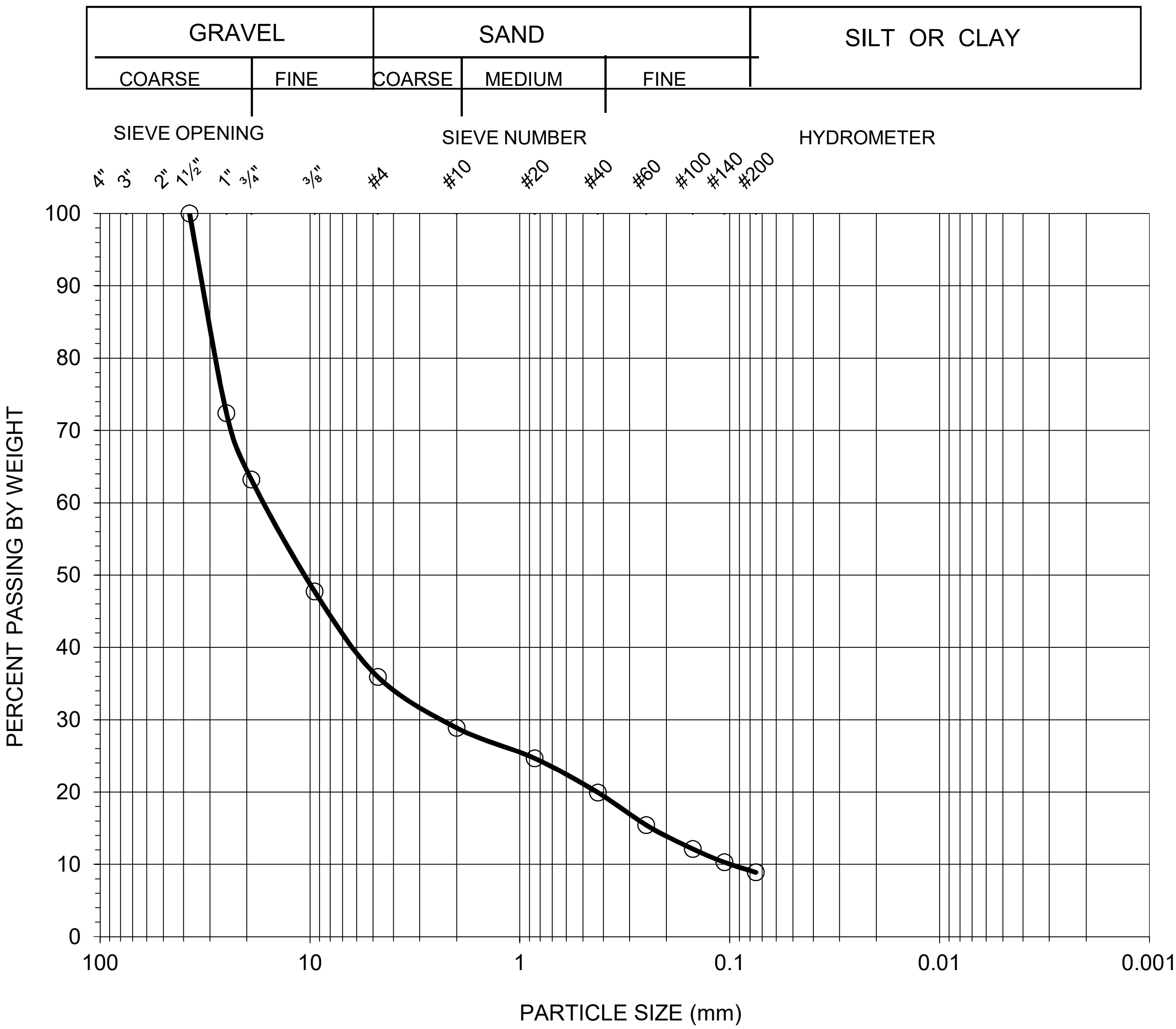
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AMEC Foster Wheeler	Tested by:	ST	Date:	10/26/16
Project Name:	SCG - Ventura	Computed by:	JP	Date:	10/31/16
Project Number:	4953-16-1091	Checked by:	AP	Date:	10/31/16



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	VCU4	4	10-11.5	64	27	9	N/A	GP-GM

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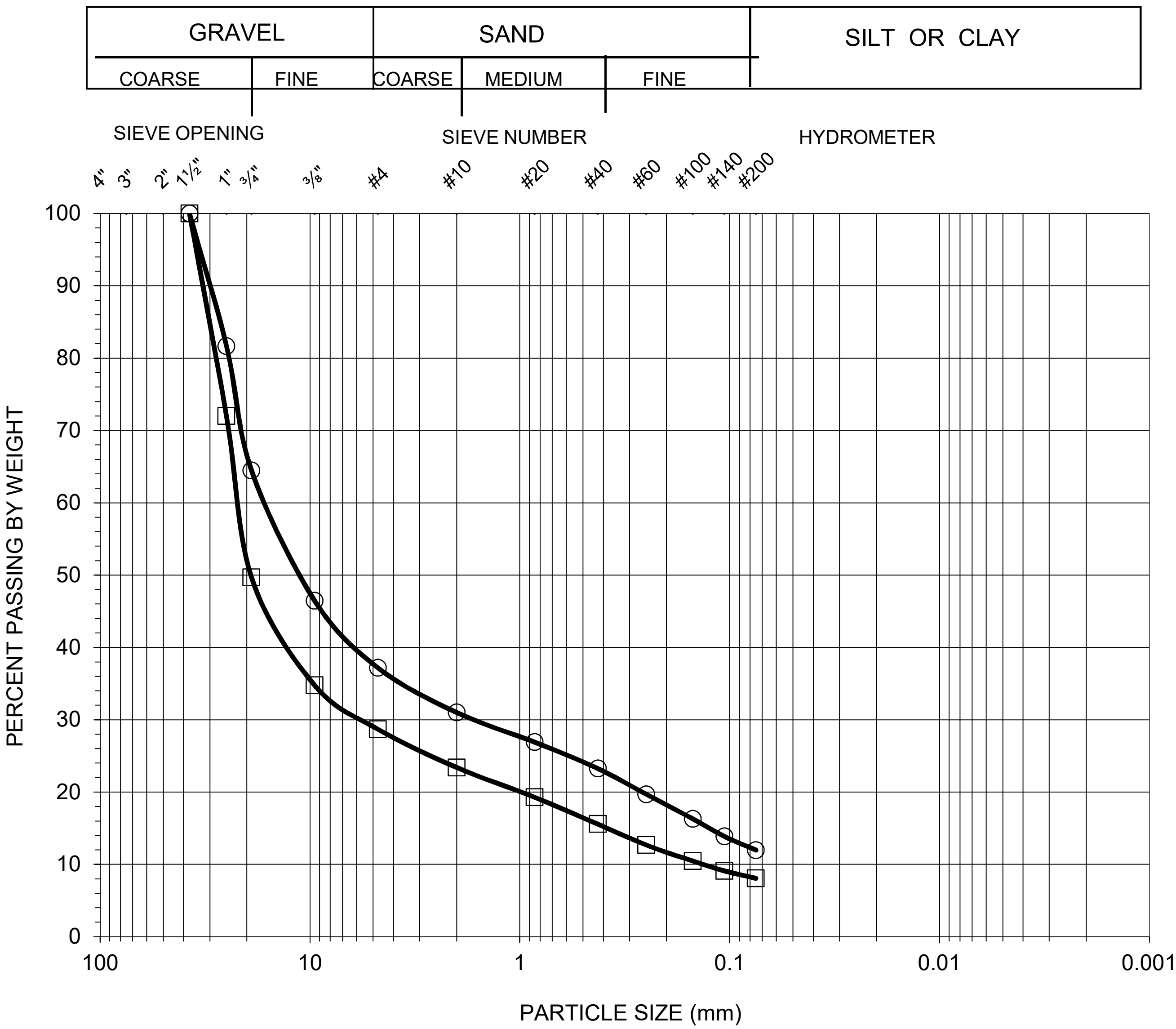
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AMEC Foster Wheeler	Tested by:	ST	Date:	10/26/16
Project Name:	SCG - Ventura	Computed by:	JP	Date:	10/31/16
Project Number:	4953-16-1091	Checked by:	AP	Date:	10/31/16



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	VCU5	3	8-9.5	63	25	12	N/A	GW-GM
□	VCU5	5	14-15.5	71	21	8	N/A	GP-GM

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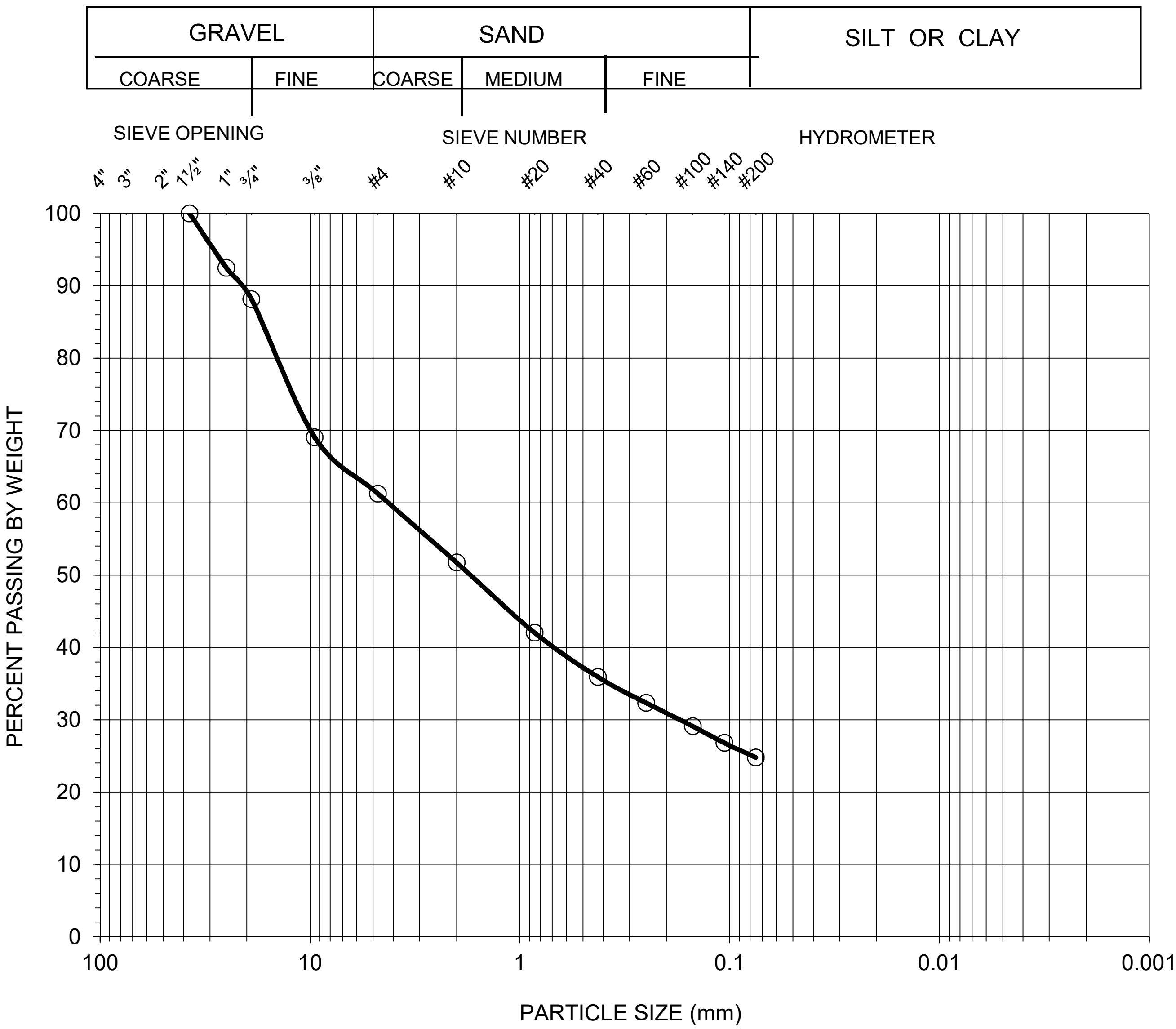
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AMEC Foster Wheeler	Tested by:	ST	Date:	10/26/16
Project Name:	SCG - Ventura	Computed by:	JP	Date:	10/31/16
Project Number:	4953-16-1091	Checked by:	AP	Date:	10/31/16



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	VCU9	5	15-16.5	39	36	25	N/A	GM

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CONSTANT HEAD PERMEABILITY TEST

ASTM D2434

Project	SCG - Ventura		Tested by	LS	Date	11/04/16
Project No.	4953-16-1091		Calculated by	JP	Date	11/04/16
Boring No.	VCU5		Checked by	AP	Date	11/07/16
Sample No.	2	Depth (ft): 5-6.5				
Soil Description	Sand w/clay & gravel	Max. Size: N/A				

CONDITION OF SPECIMEN

				Before	After
Diameter	6.13	cm	Container No.		
Sample Area	29.55	cm^2	Wt. Wet Soil+Container(gms)	185.07	614.39
Length	7.62	cm	Wt. Dry Soil+Container(gms)	167.12	540.09
Weight Before	463.58	g	Wt. Container (gms)	50.96	133.99
Wet Density	128.50	pcf	Moisture, (%)	15.45	18.30
Dry Density	111.30	pcf			

Max. Dry Density (pcf)	N/A
Optimum Moisture (%)	N/A
Relative Compaction (%)	N/A

Sample Type: Rings
Mod. Cal.

PERMEABILITY DATA

Trial No.	Manometers		Head, h	Outflow	Time	Q/At	h/L	Temp.	k
	H1	H2	cm	Q, ml	sec			°c	cm/sec
1	34.9	12.7	22.2	25	114.8	0.0074	2.9134	21.4	2.53E-03
2	34.9	12.7	22.2	25	115.7	0.0073	2.9134	21.4	2.51E-03
3	34.9	12.7	22.2	25	116.8	0.0072	2.9134	21.4	2.49E-03
4	34.9	12.7	22.2	25	116.5	0.0073	2.9134	21.4	2.49E-03
5	34.9	12.7	22.2	25	116.1	0.0073	2.9134	21.4	2.50E-03
6	34.9	9.0	25.9	25	96.9	0.0087	3.3990	21.4	2.57E-03
7	34.9	9.0	25.9	25	96.7	0.0088	3.3990	21.4	2.57E-03
8	34.9	9.0	25.9	25	97.5	0.0087	3.3990	21.4	2.55E-03
9	34.9	9.0	25.9	25	96.4	0.0088	3.3990	21.4	2.58E-03
10	34.9	9.0	25.9	25	96.6	0.0088	3.3990	21.4	2.58E-03
Corrected k_{20} (cm/sec) :									**2.45E-03**

Figure A-6.1

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CONSTANT HEAD PERMEABILITY TEST

ASTM D2434

Project	SCG - Ventura	Tested by	LS	Date	10/26/16
Project No.	4953-16-1091	Calculated by	JP	Date	10/31/16
Boring No.	VCU6	Checked by	AP	Date	10/31/16

Sample No. 5 Depth (ft): 15-16.5

Soil Description Poorly graded gravel with sand Max. Size: 3/4"

CONDITION OF SPECIMEN

				Before	After
Diameter	6.13	cm	Container No.		
Sample Area	29.55	cm^2	Wt. Wet Soil+Container(gms)	318.41	666.76
Length	7.62	cm	Wt. Dry Soil+Container(gms)	299.98	615.78
Weight Before	512.05	g	Wt. Container (gms)	50.18	150.06
Wet Density	141.93	pcf	Moisture, (%)	7.38	10.95
Dry Density	132.18	pcf			

Max. Dry Density (pcf)	N/A
Optimum Moisture (%)	N/A
Relative Compaction (%)	N/A

Sample Type Rings
Mod. Cal.

PERMEABILITY DATA

Trial No.	Manometers		Head, h	Outflow	Time	Q/At	h/L	Temp.	k
	H1	H2	cm	Q, ml	sec			°c	cm/sec
1	19.3	17.4	1.9	25	159.8	0.0053	0.2493	24.6	2.12E-02
2	19.3	17.4	1.9	25	159.5	0.0053	0.2493	24.6	2.13E-02
3	19.3	17.4	1.9	25	159.4	0.0053	0.2493	24.6	2.13E-02
4	19.3	17.4	1.9	25	160.1	0.0053	0.2493	24.6	2.12E-02
5	19.3	17.4	1.9	25	159.8	0.0053	0.2493	24.6	2.12E-02
6	19.3	13.4	5.9	25	58.4	0.0145	0.7743	24.6	1.87E-02
7	19.3	13.4	5.9	25	58.6	0.0144	0.7743	24.6	1.86E-02
8	19.3	13.4	5.9	25	58.9	0.0144	0.7743	24.6	1.86E-02
9	19.3	13.4	5.9	25	58.8	0.0144	0.7743	24.6	1.86E-02
10	19.3	13.4	5.9	25	59.0	0.0143	0.7743	24.6	1.85E-02
Corrected k_{20} (cm/sec) :									**1.79E-02**

Figure A-6.2

Appendix B

Soil Corrosivity Report

Monday, November 07, 2016 via email: walter.lopez@amecfw.com

AMEC FOSTER WHEELER
6001 Rickenbacker Road
Los Angeles, CA 90040

Attention: Mr. Walter Lopez

Re: Soil Corrosivity Study
So Cal Gas - Ventura Compression Station Upgrade
Ventura, California
HDR #16-0821
AMEC #4953-16-1091

Introduction

Laboratory tests have been completed on four soil samples provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. HDR Engineering, Inc. (HDR) assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed project will consist of multiple single-story buildings with no subterranean levels. The site is located at 1555 North Olive Street in Ventura, California, and the water table is reportedly 42.5 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

431 W. Baseline Road, Claremont, CA 91711-1608
(909) 626-0967

Laboratory Soil Corrosivity Tests

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per CTM 643. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and Standard Method 2320-B[1]. Laboratory test results are shown in the attached Table 1.

Soil Corrosivity

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[2]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

---

[1] American Public Health Association (APHA). 2012. Standard Methods of Water and Wastewater. 22nd ed. American Public Health Association, American Water Works Association, Water Environment Federation publication. APHA, Washington D.C.

[2] Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.

Electrical resistivities were in the mildly and moderately corrosive categories with as-received moisture. When saturated, the resistivities were all in the moderately corrosive category. Some of the resistivities dropped considerably with added moisture because those samples were dry as-received.

Soil pH values varied from 7.2 to 7.7. This range is neutral to mildly alkaline.[3] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples ranged from low to moderate.

Nitrate was detected in low concentrations.

Tests were not made for sulfide and oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as moderately corrosive to ferrous metals.

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement *all* the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and the possible future application of cathodic protection.

---

[3] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:

    a. At each end of the pipeline.

    b. At each end of all casings.

    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically isolate each buried steel pipeline per NACE SP0286 from:

    a. Dissimilar metals.

    b. Dissimilarly coated piping (cement-mortar vs. dielectric).

    c. Above ground steel pipe.

    d. All existing piping.

4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable dielectric coating intended for underground use such as:

        i. Polyurethane per AWWA C222 *or*

        ii. Extruded polyethylene per AWWA C215 *or*

        iii. A tape coating system per AWWA C214 *or*

        iv. Hot applied coal tar enamel per AWWA C203 *or*

        v. Fusion bonded epoxy per AWWA C213.

    b. Although it is customary to cathodically protect bonded dielectrically coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

a. As an alternative to dielectric coating and possible future cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of ASTM C150 cement. Joint bonds, test stations, and insulated joints are still recommended for these alternatives.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Iron Pipe

Implement *all* the following measures:

1. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and possible future application of cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:

    a. At each end of the pipeline.

    b. At each end of any casings.

    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable coating intended for underground use such as:

        i. Polyethylene encasement per AWWA C105; *or*

ii. Epoxy coating; *or*

iii. Polyurethane; *or*

iv. Wax tape.

NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Although it is customary to cathodically protect coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

a. As an alternative to coating systems described in Option 1 and possible future cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement.

Copper Tubing

Implement *all* the following measures:

1. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.

2. Electrically insulate cold water piping from hot water piping systems.

3. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves and bed and backfill with clean sand at least 2 inches thick surrounding the tubing. Clean sand should have a minimum resistivity of no less than 3,000 ohm-cm, and a pH of 6.0–8.0. Copper tubing for cold water can also be treated the same as for hot water.

4. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing by one of the following measures:

a. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. *or*

b. Applying cathodic protection per NACE SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.

2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.10 percent.[4,5,6]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentration[7] found onsite.

---

[4] *2015 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318 Table 19.3.2.1*

[5] *2012 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318 Table 19.3.2.1*

[6] *2013 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318 Table 19.3.2.1*

[7] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

James Keegan

Greg Frost, PE

Enc: Table 1

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Table 1 - Laboratory Tests on Soil Samples

AMEC Foster Wheeler
So Cal Gas - Ventura Compression Station Upgrade
Your #4953-16-1091, HDR Lab #16-0821SCS
7-Nov-16

Sample ID			VCU-1 @ 1-5'	VCU-2 @ 1-5'	VCU-4 SPT-1 @ 5-6.5'	VCU-9 SPT-1 @ 5'
Resistivity		**Units**				
as-received		ohm-cm	32,800	84,000	6,400	34,000
saturated		ohm-cm	2,080	3,680	4,800	7,200
pH			7.2	7.5	7.6	7.7
Electrical Conductivity		mS/cm	0.25	0.15	0.13	0.10
Chemical Analyses						
Cations						
calcium	Ca^{2+}	mg/kg	79	58	64	65
magnesium	Mg^{2+}	mg/kg	61	11	8.1	4.7
sodium	Na^{1+}	mg/kg	59	76	73	51
potassium	K^{1+}	mg/kg	9.4	14	15	12
Anions						
carbonate	CO_3^{2-}	mg/kg	ND	ND	12	17
bicarbonate	HCO_3^{1-}	mg/kg	55	159	195	192
fluoride	F^{1-}	mg/kg	3.1	3.2	6.7	5.1
chloride	Cl^{1-}	mg/kg	12	30	7.9	5.7
sulfate	SO_4^{2-}	mg/kg	468	183	103	59
phosphate	PO_4^{3-}	mg/kg	ND	3.3	3.3	3.0
Other Tests						
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND	ND
nitrate	NO_3^{1-}	mg/kg	38	28	ND	ND
sulfide	S^{2-}	qual	na	na	na	na
Redox		mV	na	na	na	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
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

Appendix C

Results of Suspension Logging

VENTURA COMPRESSOR STATION

SUSPENSION PS VELOCITIES

BOREHOLES VCU-6 & VCU-7

VENTURA, CA

October 27, 2016

Report 16372-01 rev 0

VENTURA COMPRESSOR STATION
SUSPENSION PS VELOCITIES
BOREHOLES VCU-6 & VCU-7
VENTURA, CA

Prepared for

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October 27, 2016

Report 16372-01 rev 0

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APPENDICES

INTRODUCTION

GEO*Vision* acquired borehole geophysical data in two boreholes in Ventura, CA. The work was performed for AMEC Foster & Wheeler, Inc. Fieldwork was performed by Victor Gonzalez. Analysis was completed by Emily Feldman, and reviewed by John Diehl. The report was prepared by Jonathan Jordon and reviewed by John Diehl, Professional Engineer.

SCOPE OF WORK

This report presents results of Suspension PS velocity data acquired in two boreholes on September 28 and October 5, 2016, as detailed in Table 1. The purpose of these measurements was to supplement stratigraphic information by acquiring shear wave and compressional wave velocities as a function of depth.

The OYO Suspension PS Logging System (Suspension System) was used to obtain in-situ horizontal shear (S_H) and compressional (P) wave velocity measurements in one uncased borehole at 1.6 foot intervals. Measurements followed **GEO***Vision* Procedure for P-S Suspension Seismic Velocity Logging, revision 1.5. Acquired data were analyzed and a profile of velocity versus depth was produced for both S_H and P waves.

A detailed reference for the suspension PS velocity measurement techniques used in this study is:

> Guidelines for Determining Design Basis Ground Motions, Report TR-102293, Electric Power Research Institute, Palo Alto, California, November 1993, Sections 7 and 8.

INSTRUMENTATION

Suspension Velocity Instrumentation

Suspension velocity measurements were performed using the suspension PS logging system, manufactured by OYO Corporation, and their subsidiary, Robertson Geologging. This system directly determines the average velocity of a 3.3-foot high segment of the soil column surrounding the borehole of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the borehole producing relatively constant amplitude signals at all depths.

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear-wave source (S_H) and compressional-wave source (P), joined to two biaxial receivers by a flexible isolation cylinder, as shown in Figure 1. The separation of the two receivers is 3.3 feet, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe as used in these surveys is approximately 25 feet, with the center point of the receiver pair 12.5 feet above the bottom end of the probe.

The probe receives control signals from, and sends the digitized receiver signals to, instrumentation on the surface via an armored multi-conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data using a sheave of known circumference fitted with a digital rotary encoder.

The entire probe is suspended in the borehole by the cable, therefore, source motion is not coupled directly to the borehole walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the borehole and surrounding the source. This pressure wave is converted to P and S_H-waves in the surrounding soil and rock as it passes through the casing and grout annulus and impinges upon the wall of the borehole. These waves propagate through the soil and rock surrounding the borehole, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location. Separation of the P and S_H-waves at the receivers is performed using the following steps:

1. Orientation of the horizontal receivers is maintained parallel to the axis of the source, maximizing the amplitude of the recorded S_H -wave signals.
2. At each depth, S_H-wave signals are recorded with the source actuated in opposite directions, producing S_H-wave signals of opposite polarity, providing a characteristic S_H-wave signature distinct from the P-wave signal.
3. The 6.3 foot separation of source and receiver 1 permits the P-wave signal to pass and damp significantly before the slower S_H-wave signal arrives at the receiver. In faster soils or rock, the isolation cylinder is extended to allow greater separation of the P- and S_H-wave signals.
4. In saturated soils, the received P-wave signal is typically of much higher frequency than the received S_H-wave signal, permitting additional separation of the two signals by low pass filtering.
5. Direct arrival of the original pressure pulse in the fluid is not detected at the receivers because the wavelength of the pressure pulse in fluid is significantly greater than the dimension of the fluid annulus surrounding the probe (feet versus inches scale), preventing significant energy transmission through the fluid medium.

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:

1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction and the horizontal receiver signals are recorded.

3. The source is fired again and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and S_H-wave arrivals; reversal of the source changes the polarity of the S_H-wave pattern but not the P-wave pattern.

The data from each receiver during each source activation is recorded as a different channel on the recording system. The Suspension PS system has six channels (two simultaneous recording channels), each with a 1024 sample record. The recorded data are displayed as six channels with a common time scale. Data are stored on disk for further processing.

Review of the displayed data on the recorder or computer screen allows the operator to set the gains, filters, delay time, pulse length (energy), and sample rate to optimize the quality of the data before recording. Verification of the calibration of the Suspension PS digital recorder is performed every twelve months using a NIST traceable frequency source and counter, as presented in Appendix B.

MEASUREMENT PROCEDURES

Suspension Velocity Measurement Procedures

The boreholes were logged uncased and filled with fresh water mud. Measurements followed the **GEO***Vision* Procedure for P-S Suspension Seismic Velocity Logging, revision 1.5. Prior to the logging run, the probe was positioned with the top of the probe even with a stationary reference point. The electronic depth counter was set to the distance between the mid-point of the receiver and the top of the probe, minus the height of the stationary reference point, if any. Measurements were verified with a tape measure, and calculations recorded on a field log.

The probe was lowered to the bottom of the borehole, stopping at 1.6 foot intervals to collect data, as summarized in Table 2. At each measurement depth the measurement sequence of two opposite horizontal records and one vertical record was performed. Gains were adjusted as required. The data from each depth were viewed on the computer display, checked, and saved to disk before moving to the next depth.

Upon completion of the measurements, the probe was returned to the surface and the zero depth indication at the depth reference point was verified prior to removal from the borehole.

DATA ANALYSIS

Suspension Velocity Analysis

Using the proprietary OYO program PSLOG.EXE version 1.0, the recorded digital waveforms were analyzed to locate the most prominent first minima, first maxima, or first break on the vertical axis records, indicating the arrival of P-wave energy. The difference in travel time between receiver 1 and receiver 2 (R1-R2) arrivals was used to calculate the P-wave velocity for that 1.0 meter segment of the soil column. When observable, P-wave arrivals on the horizontal axis records were used to verify the velocities determined from the vertical axis data. The time picks were then transferred into a Microsoft Excel® template to complete the velocity calculations based on the arrival time picks made in PSLOG. The Microsoft Excel® analysis files accompany this report.

The P-wave velocity over the 6.3-foot interval from source to receiver 1 (S-R1) was also picked using PSLOG, and calculated and plotted in Microsoft Excel®, for quality assurance of the velocity derived from the travel time between receivers. In this analysis, the depth values as recorded were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the P-wave signal at receiver 1 and subtracting the calculated and experimentally verified delay, in milliseconds, from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of acceleration of the solenoid before impact.

As with the P-wave records, the recorded digital waveforms were analyzed to locate clear S_H-wave pulses, as indicated by the presence of opposite polarity pulses on each pair of horizontal records. Ideally, the S_H-wave signals from the 'normal' and 'reverse' source pulses are very nearly inverted images of each other. Digital Fast Fourier Transform – Inverse Fast Fourier Transform (FFT – IFFT) lowpass filtering was used to remove the higher frequency P-wave signal from the S_H-wave signal. Different filter cutoffs were used to separate P- and S_H-waves at different depths, ranging from 600 Hz in the slowest zones to 4000 Hz in the regions of highest velocity. At each depth, the filter frequency was selected to be at least twice the fundamental frequency of the S_H-wave signal being filtered.

Generally, the first maxima were picked for the 'normal' signals and the first minima for the 'reverse' signals, although other points on the waveform were used if the first pulse was distorted. The absolute arrival time of the 'normal' and 'reverse' signals may vary by +/- 0.2 milliseconds, due to differences in the actuation time of the solenoid source caused by constant mechanical bias in the source or by borehole inclination. This variation does not affect the R1-R2 velocity determinations, as the differential time is measured between arrivals of waves created by the same source actuation. The final velocity value is the average of the values obtained from the 'normal' and 'reverse' source actuations.

As with the P-wave data, S_H-wave velocity calculated from the travel time over the 6.33-foot interval from source to receiver 1 was calculated and plotted for verification of the velocity derived from the travel time between receivers. In this analysis, the depth values were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the S_H-wave signal at the near receiver and subtracting the calculated and experimentally verified delay, in milliseconds, from the beginning of the record at the source trigger pulse to source impact.

Poisson's Ratio, **ν**, was calculated in the Microsoft Excel® template using the following formula:

$$\boldsymbol{\nu} = \frac{\left(\frac{V_s}{V_p}\right)^2 - 0.5}{\left(\frac{V_s}{V_p}\right)^2 - 1.0}$$

Data and analyses were reviewed by a **GEO***Vision* Professional Geophysicist or Engineer as a component of the in-house data validation program.

Figure 2 shows an example of R1 - R2 measurements on a sample filtered suspension record. In Figure 2, the time difference over the 3.3 foot interval of 1.88 milliseconds for the horizontal signals is equivalent to an S_H-wave velocity of 1745 feet/second. Whenever possible, time differences were determined from several phase points on the S_H-waveform records to verify the data obtained from the first arrival of the S_H-wave pulse. Figure 3 displays the same record before filtering of the S_H-waveform record with a 1400 Hz FFT - IFFT digital lowpass filter, illustrating the presence of higher frequency P-wave energy at the beginning of the record, and distortion of the lower frequency S_H-wave by residual P-wave signal.

RESULTS

Suspension Velocity Results

Suspension R1-R2 P- and S_H-wave velocities for boreholes VCU-6 and VCU-7 are plotted in Figures 4 and 5, respectively. Suspension velocity data are also presented in Tables 3 and 4, respectively. The Microsoft Excel® analysis files accompany this report.

P- and S_H-wave velocity data from R1-R2 analysis and quality assurance analysis of S-R1 data are plotted together in Figure A-1 and A-2 in Appendix A to aid in visual comparison. It should be noted that R1-R2 data are an average velocity over a 3.3-foot segment of the soil column; S-R1 data are an average over 6.3 feet, creating a significant smoothing relative to the R1-R2 plots. The S-R1 velocity data are also presented in Table A-1 and A-2 and included in the Microsoft Excel® analysis files, which also includes Poisson's Ratio calculations, tabulated data and plots.

SUMMARY

Discussion of Suspension Velocity Results

Suspension PS velocity data are ideally collected in uncased fluid filled boreholes drilled with rotary wash methods, as were the boreholes for this project. Overall, Suspension PS velocity data quality is judged on 5 criteria, as summarized below.

	Criteria	VCU-6 and VCU-7
1	Consistent data between receiver to receiver (R1 – R2) and source to receiver (S – R1) data.	Yes.
2	Consistency between data from adjacent depth intervals.	Yes
3	Consistent relationship between P-wave and SH -wave (excluding transition to saturated soils)	Yes Saturation occurs between 45-50ft BGS in both boreholes
4	Clarity of P-wave and SH-wave onset, as well as damping of later oscillations.	The quality is acceptable, though VCU-7 was more difficult to interpret
5	Consistency of profile between adjacent borings, if available.	Very consistent overall, though VCU-7 was not drilled as deep as VCU-6

Quality Assurance

These borehole geophysical measurements were performed using industry-standard or better methods for measurements and analyses. All work was performed under **GEO***Vision* quality assurance procedures, which include:

- Use of NIST-traceable calibrations, where applicable, for field and laboratory instrumentation
- Use of standard field data logs
- Use of independent verification of velocity data by comparison of receiver-to-receiver and source-to-receiver velocities
- Independent review of calculations and results by a registered professional engineer, geologist, or geophysicist.

Suspension Velocity Data Reliability

P- and S_H-wave velocity measurement using the Suspension Method gives average velocities over a 3.3-foot interval of depth. This high resolution results in the scatter of values shown in the graphs. Individual measurements are very reliable with estimated precision of +/- 5%. Depth indications are very reliable with estimated precision of +/- 0.2 feet. Standardized field procedures and quality assurance checks contribute to the reliability of these data.

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

10/27/2016

Jonathan Jordan Date
Staff Geophysicist
GEO*Vision* Geophysical Services

Reviewed and approved by

REGISTERED PROFESSIONAL ENGINEER
J. G. DIEHL
No. 30362
CIVIL
STATE OF CALIFORNIA

10/27/16

John G. Diehl Date
California Professional Engineer 30362
GEO*Vision* Geophysical Services

* 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.

Table 1. Borehole locations and logging dates

BOREHOLE DESIGNATION	DATES LOGGED	COORDINATES (FEET) [1]		ELEVATION (TOP OF WELL CASING) [1] (FEET)
		NORTHING	EASTING	
VCU-6	October 05, 2016			
VCU-7	September 28, 2016			

[1] Survey data not available at time of report issue

Table 2. Logging dates and depth ranges

BOREHOLE NUMBER	TOOL AND RUN NUMBER	DEPTH RANGE (feet)	OPEN HOLE (FEET)	SAMPLE INTERVAL (FEET)
VCU-6	SUSPENSION DOWN 01	1.6 --- 59.6	73	1.6
VCU-7	SUSPENSION DOWN 01	1.6 –- 50.3	63	1.6

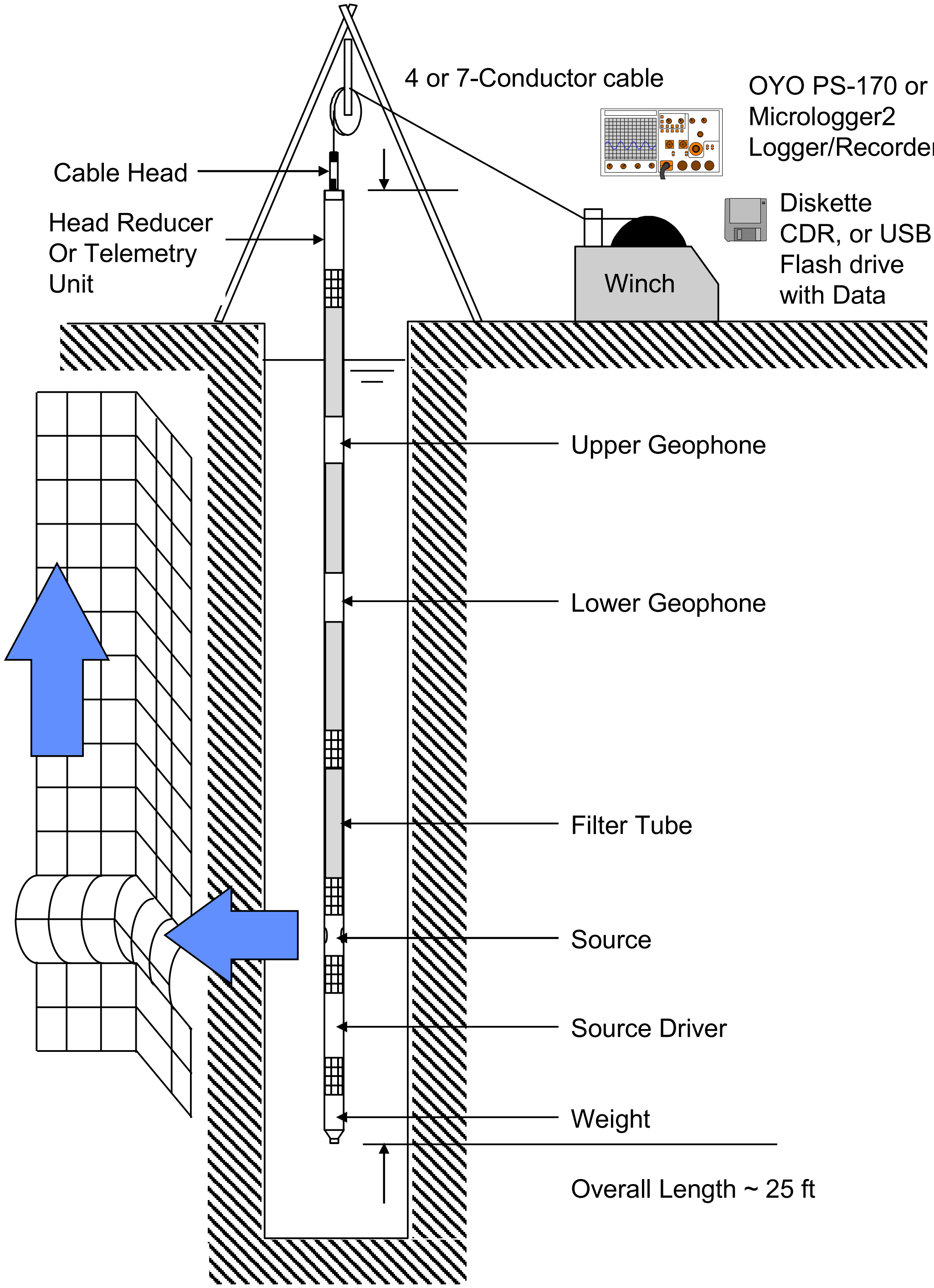

Figure 1: Concept illustration of P-S logging system

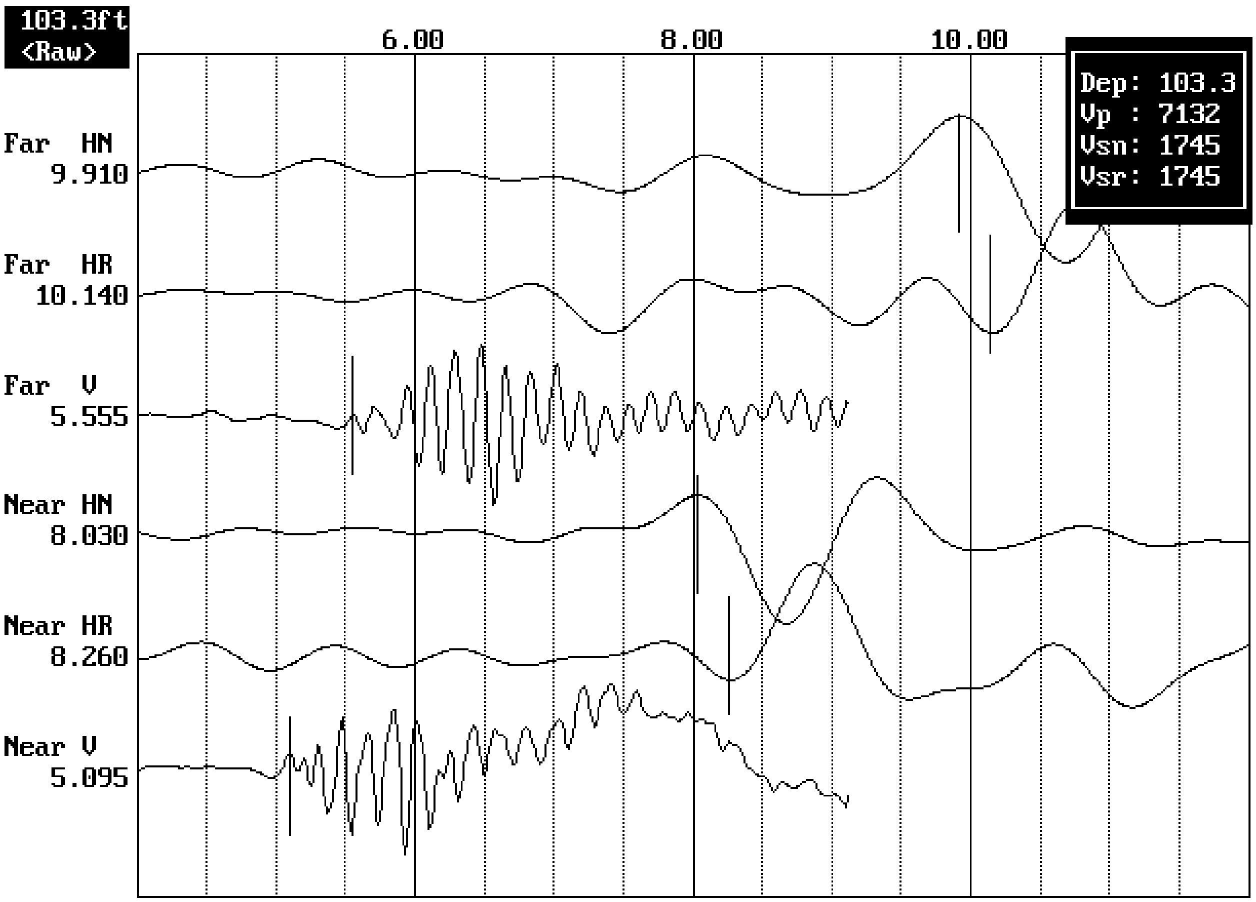

Figure 2: Example of filtered (1400 Hz lowpass) suspension record

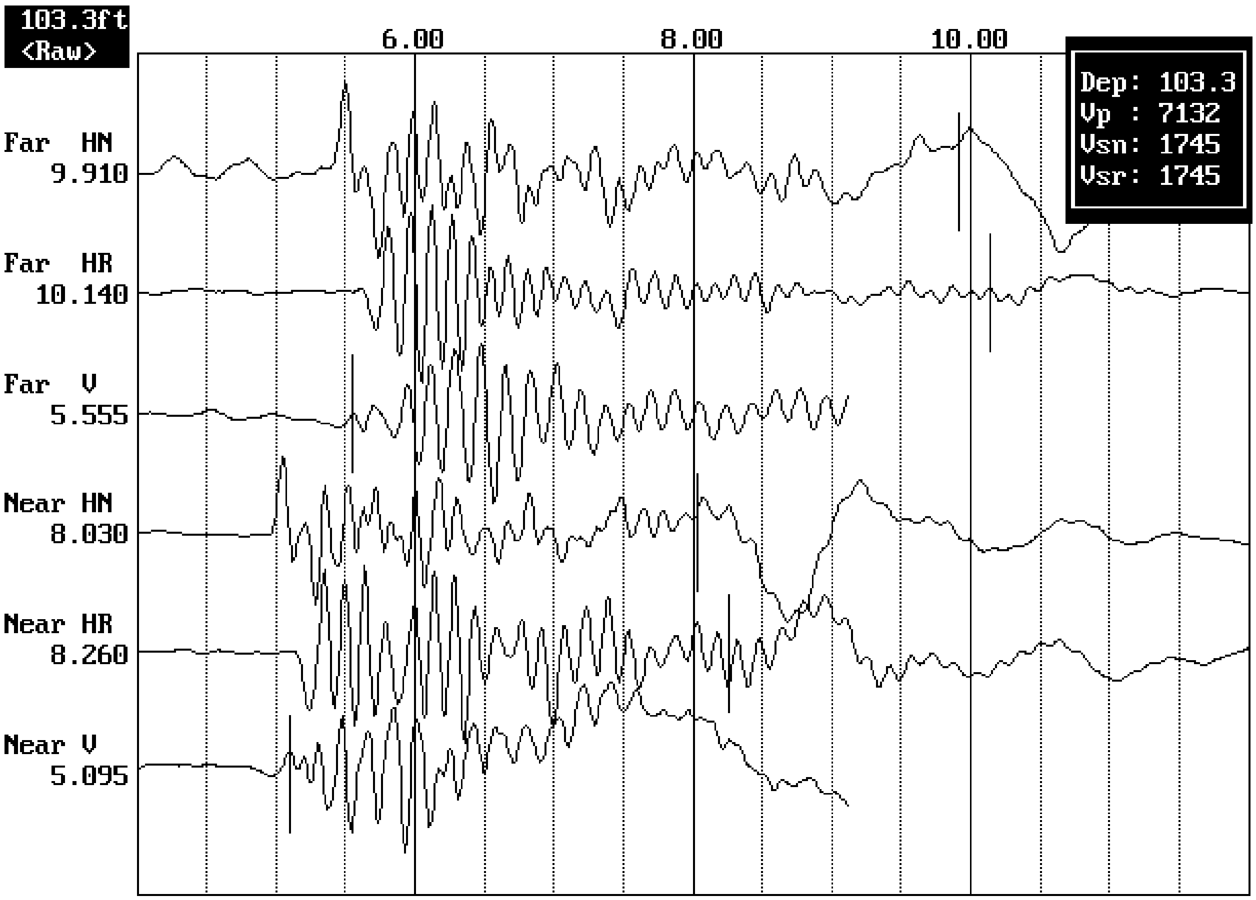

Figure 3. Example of unfiltered suspension record

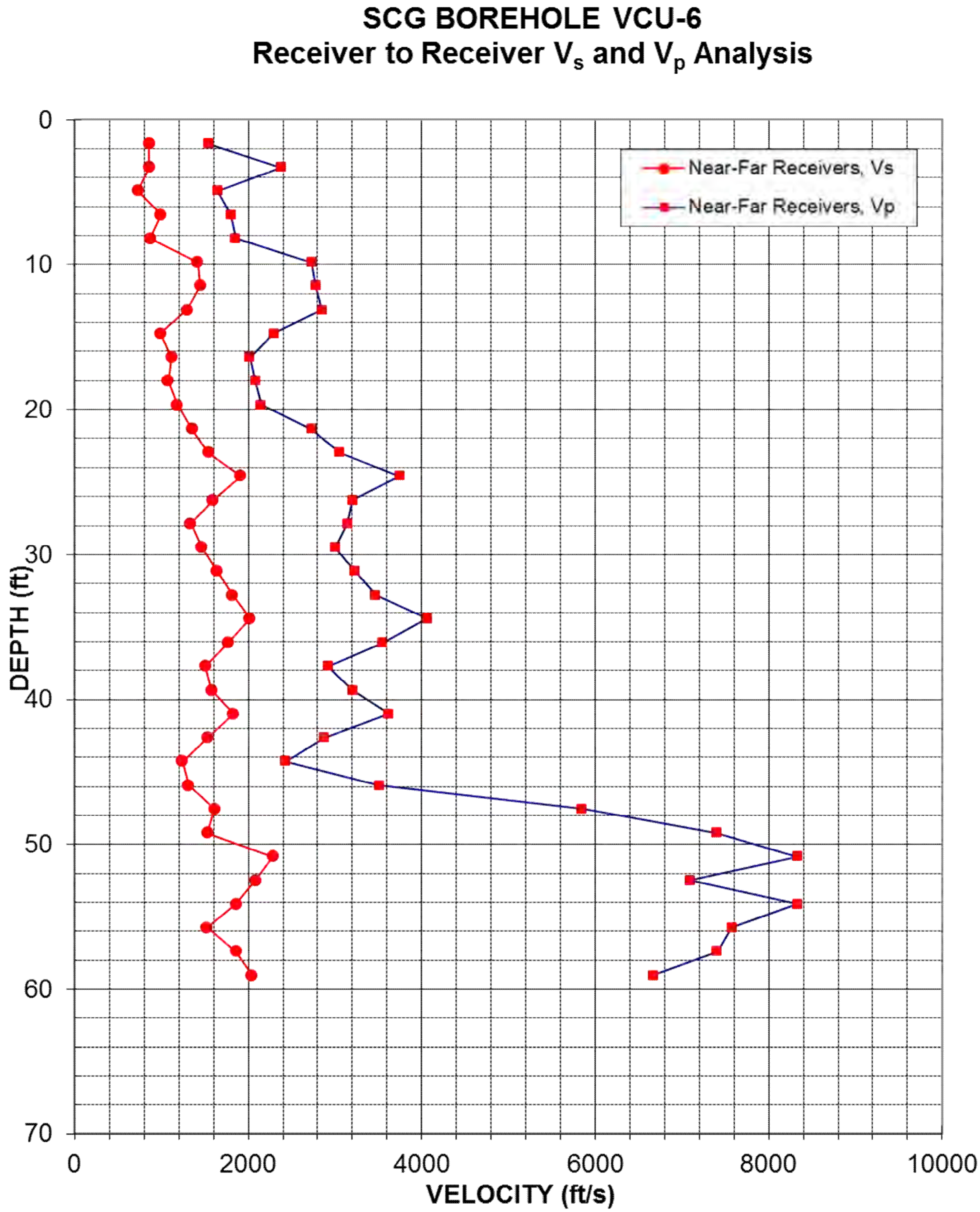

Figure 4: Borehole VCU-6, Suspension R1-R2 P- and S_H-wave velocities

Table 3. Borehole VCU-6, Suspension R1-R2 depths and P- and S_H-wave velocities

Table 3
Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole VCU-6

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
1.6	860	1550	0.28	0.5	260	470	0.28
3.3	860	2380	0.42	1.0	260	730	0.42
4.9	730	1650	0.38	1.5	220	500	0.38
6.6	1000	1800	0.28	2.0	300	550	0.28
8.2	870	1850	0.36	2.5	270	560	0.36
9.8	1420	2730	0.32	3.0	430	830	0.32
11.5	1450	2780	0.31	3.5	440	850	0.31
13.1	1290	2850	0.37	4.0	390	870	0.37
14.8	990	2300	0.39	4.5	300	700	0.39
16.4	1120	2020	0.28	5.0	340	620	0.28
18.0	1080	2080	0.32	5.5	330	640	0.32
19.7	1180	2150	0.28	6.0	360	660	0.28
21.3	1360	2730	0.34	6.5	410	830	0.34
23.0	1540	3060	0.33	7.0	470	930	0.33
24.6	1920	3750	0.32	7.5	580	1140	0.32
26.3	1590	3210	0.34	8.0	490	980	0.34
27.9	1330	3140	0.39	8.5	410	960	0.39
29.5	1470	3000	0.34	9.0	450	920	0.34
31.2	1640	3240	0.33	9.5	500	990	0.33
32.8	1820	3470	0.31	10.0	560	1060	0.31
34.5	2020	4070	0.34	10.5	620	1240	0.34
36.1	1760	3550	0.34	11.0	540	1080	0.34
37.7	1510	2920	0.32	11.5	460	890	0.32
39.4	1580	3210	0.34	12.0	480	980	0.34
41.0	1830	3620	0.33	12.5	560	1100	0.33
42.7	1530	2870	0.30	13.0	470	880	0.30
44.3	1240	2430	0.32	13.5	380	740	0.32
45.9	1310	3510	0.42	14.0	400	1070	0.42
47.6	1610	5850	0.46	14.5	490	1780	0.46
49.2	1530	7410	0.48	15.0	470	2260	0.48
50.9	2280	8330	0.46	15.5	700	2540	0.46
52.5	2080	7090	0.45	16.0	640	2160	0.45
54.1	1860	8330	0.47	16.5	570	2540	0.47
55.8	1520	7580	0.48	17.0	460	2310	0.48
57.4	1860	7410	0.47	17.5	570	2260	0.47

Table 3
Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole VCU-6

American Units			
Depth at Midpoint Between Receivers	Velocity V_s	Velocity V_p	Poisson's Ratio
(ft)	(ft/s)	(ft/s)	
59.1	2040	6670	0.45

Metric Units			
Depth at Midpoint Between Receivers	Velocity V_s	Velocity V_p	Poisson's Ratio
(m)	(m/s)	(m/s)	
18.0	620	2030	0.45

Notes: "-" means no data available at that particular interval of depth.

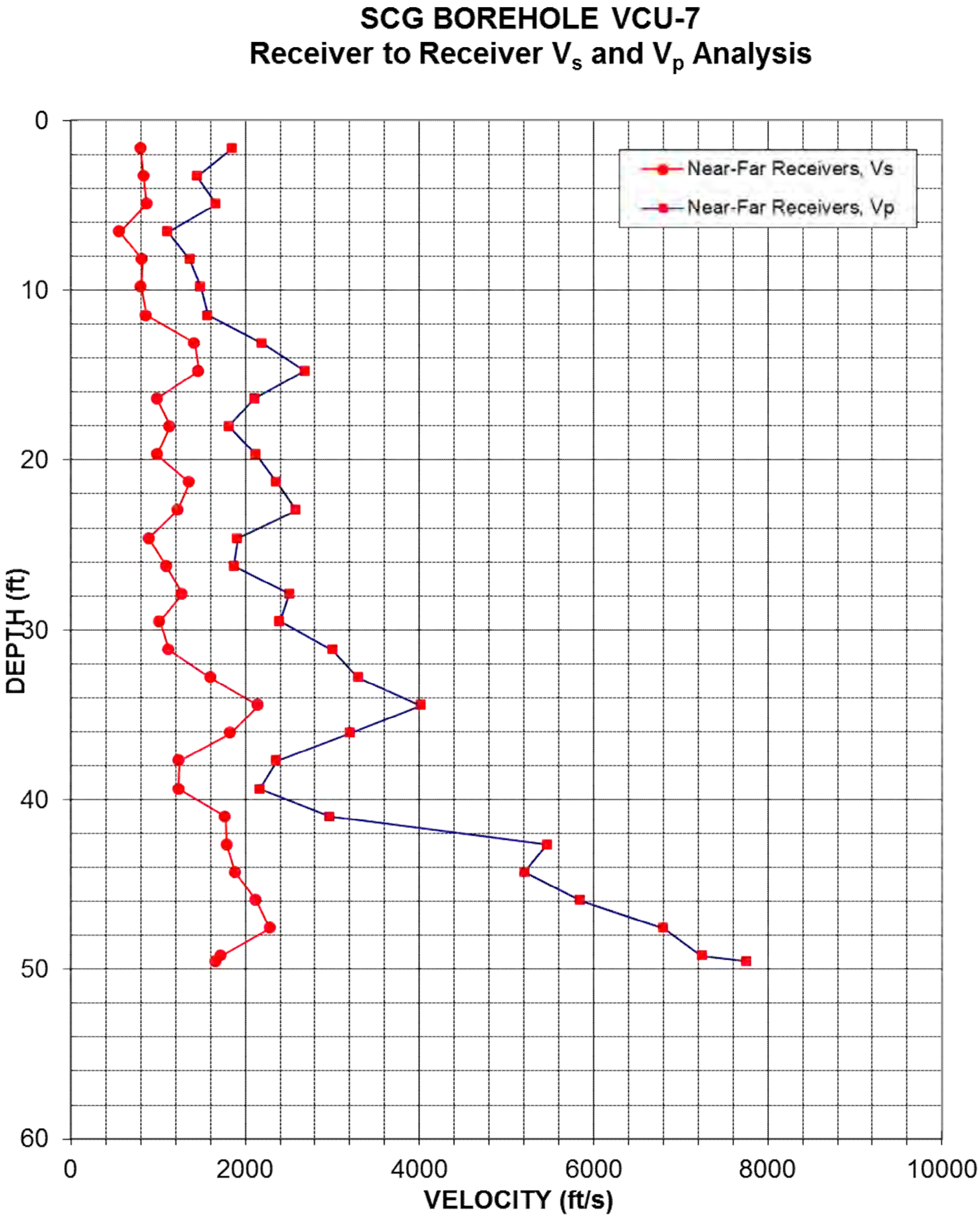

Figure 5: Borehole VCU-7, Suspension R1-R2 P- and S_H-wave velocities

Table 4. Borehole VCU-7, Suspension R1-R2 depths and P- and S_H-wave velocities

Table 4

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio

Based on Receiver-to-Receiver Travel Time Data - Borehole VCU-7

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity V_s	Velocity V_p	Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity V_s	Velocity V_p	Poisson's Ratio
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
1.6	800	1850	0.39	0.5	240	560	0.39
3.3	830	1450	0.25	1.0	250	440	0.25
4.9	870	1670	0.31	1.5	270	510	0.31
6.6	560	1110	0.33	2.0	170	340	0.33
8.2	820	1370	0.22	2.5	250	420	0.22
9.8	800	1490	0.29	3.0	240	450	0.29
11.5	870	1570	0.28	3.5	260	480	0.28
13.1	1420	2190	0.14	4.0	430	670	0.14
14.8	1470	2690	0.29	4.5	450	820	0.29
16.4	990	2110	0.36	5.0	300	640	0.36
18.0	1130	1810	0.18	5.5	350	550	0.18
19.7	990	2120	0.36	6.0	300	650	0.36
21.3	1360	2360	0.25	6.5	410	720	0.25
23.0	1230	2580	0.35	7.0	370	790	0.35
24.6	890	1920	0.36	7.5	270	580	0.36
26.3	1090	1870	0.24	8.0	330	570	0.24
27.9	1280	2510	0.32	8.5	390	760	0.32
29.5	1020	2400	0.39	9.0	310	730	0.39
31.2	1120	3000	0.42	9.5	340	920	0.42
32.8	1600	3300	0.35	10.0	490	1010	0.35
34.5	2150	4020	0.30	10.5	660	1220	0.30
36.1	1830	3210	0.26	11.0	560	980	0.26
37.7	1240	2360	0.31	11.5	380	720	0.31
39.4	1230	2160	0.26	12.0	380	660	0.26
41.0	1770	2980	0.22	12.5	540	910	0.22
42.7	1790	5460	0.44	13.0	550	1670	0.44
44.3	1880	5210	0.42	13.5	570	1590	0.42
45.9	2120	5850	0.42	14.0	650	1780	0.42
47.6	2280	6800	0.44	14.5	700	2070	0.44
49.2	1720	7250	0.47	15.0	520	2210	0.47
49.5	1660	7750	0.48	15.1	510	2360	0.48

Notes: "-" means no data available at that particular interval of depth.

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

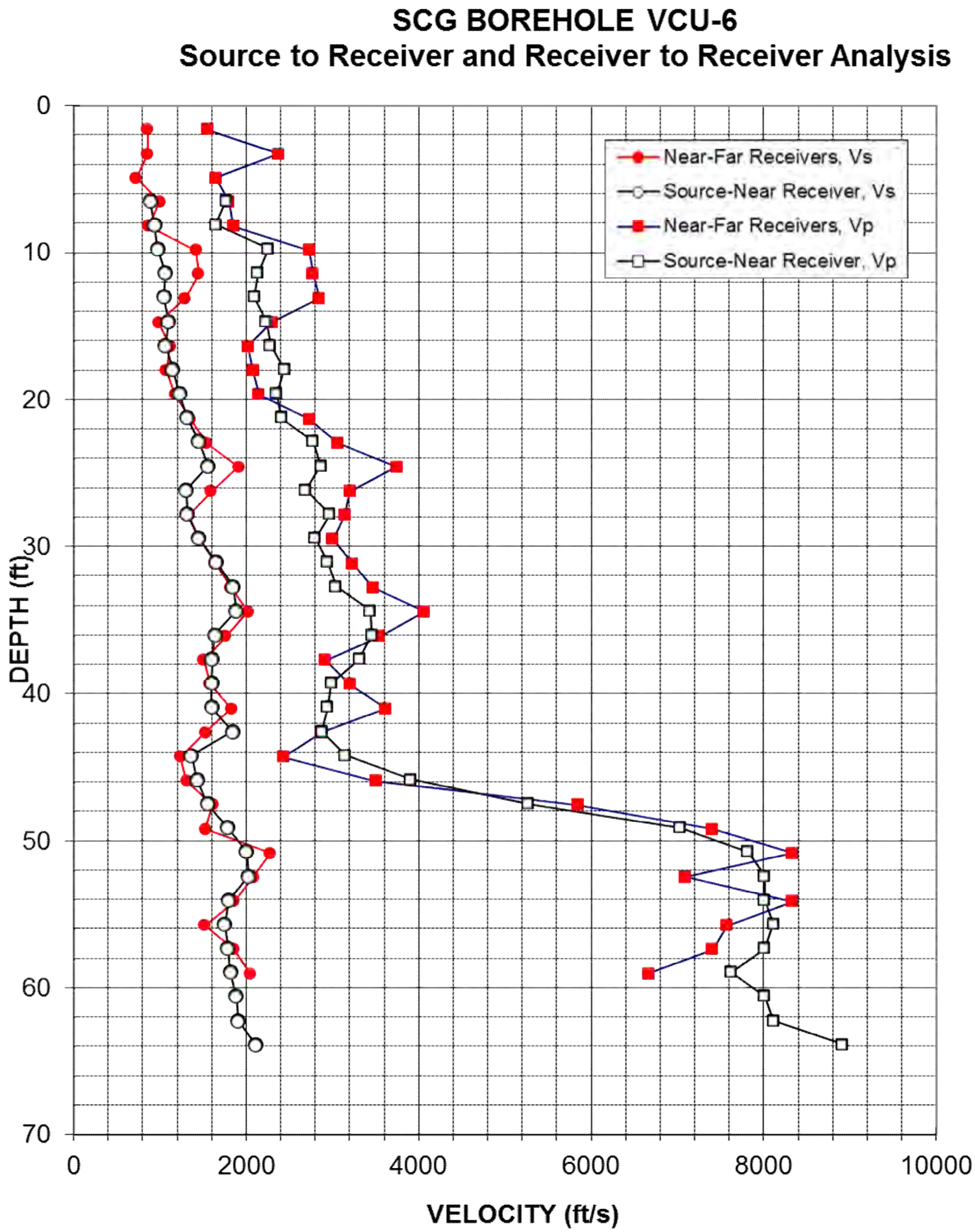

Figure A-1: Borehole VCU-6, Suspension S-R1 P- and S_H-wave velocities

Table A-1. Borehole VCU-6, S - R1 quality assurance analysis P- and S_H-wave data

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole VCU-6

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
6.5	890	1770	0.33
8.1	940	1650	0.26
9.8	970	2260	0.39
11.4	1060	2140	0.34
13.0	1050	2100	0.33
14.7	1100	2240	0.34
16.3	1060	2290	0.36
18.0	1150	2440	0.36
19.6	1230	2350	0.31
21.2	1320	2410	0.29
22.9	1450	2780	0.31
24.5	1560	2860	0.29
26.2	1300	2690	0.35
27.8	1320	2970	0.38
29.4	1450	2800	0.32
31.1	1660	2940	0.27
32.7	1850	3040	0.21
34.4	1880	3440	0.29
36.0	1640	3460	0.36
37.6	1610	3310	0.35
39.3	1600	2990	0.30
40.9	1600	2940	0.29
42.6	1840	2880	0.15
44.2	1360	3150	0.38
45.8	1430	3910	0.42
47.5	1550	5280	0.45
49.1	1790	7030	0.47
50.8	2000	7810	0.46
52.4	2030	8010	0.47
54.0	1800	8010	0.47
55.7	1750	8120	0.48
57.3	1790	8010	0.47
59.0	1820	7630	0.47
60.6	1880	8010	0.47
62.2	1910	8120	0.47

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
2.0	270	540	0.33
2.5	290	500	0.26
3.0	300	690	0.39
3.5	320	650	0.34
4.0	320	640	0.33
4.5	340	680	0.34
5.0	320	700	0.36
5.5	350	740	0.36
6.0	370	720	0.31
6.5	400	730	0.29
7.0	440	850	0.31
7.5	480	870	0.29
8.0	400	820	0.35
8.5	400	910	0.38
9.0	440	850	0.32
9.5	510	900	0.27
10.0	560	930	0.21
10.5	570	1050	0.29
11.0	500	1050	0.36
11.5	490	1010	0.35
12.0	490	910	0.30
12.5	490	900	0.29
13.0	560	880	0.15
13.5	420	960	0.38
14.0	440	1190	0.42
14.5	470	1610	0.45
15.0	550	2140	0.47
15.5	610	2380	0.46
16.0	620	2440	0.47
16.5	550	2440	0.47
17.0	530	2470	0.48
17.5	550	2440	0.47
18.0	550	2320	0.47
18.5	570	2440	0.47
19.0	580	2470	0.47

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole VCU-6

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
63.9	2110	8920	0.47

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
19.5	640	2720	0.47

Notes: "-" means no data available at that particular interval of depth.

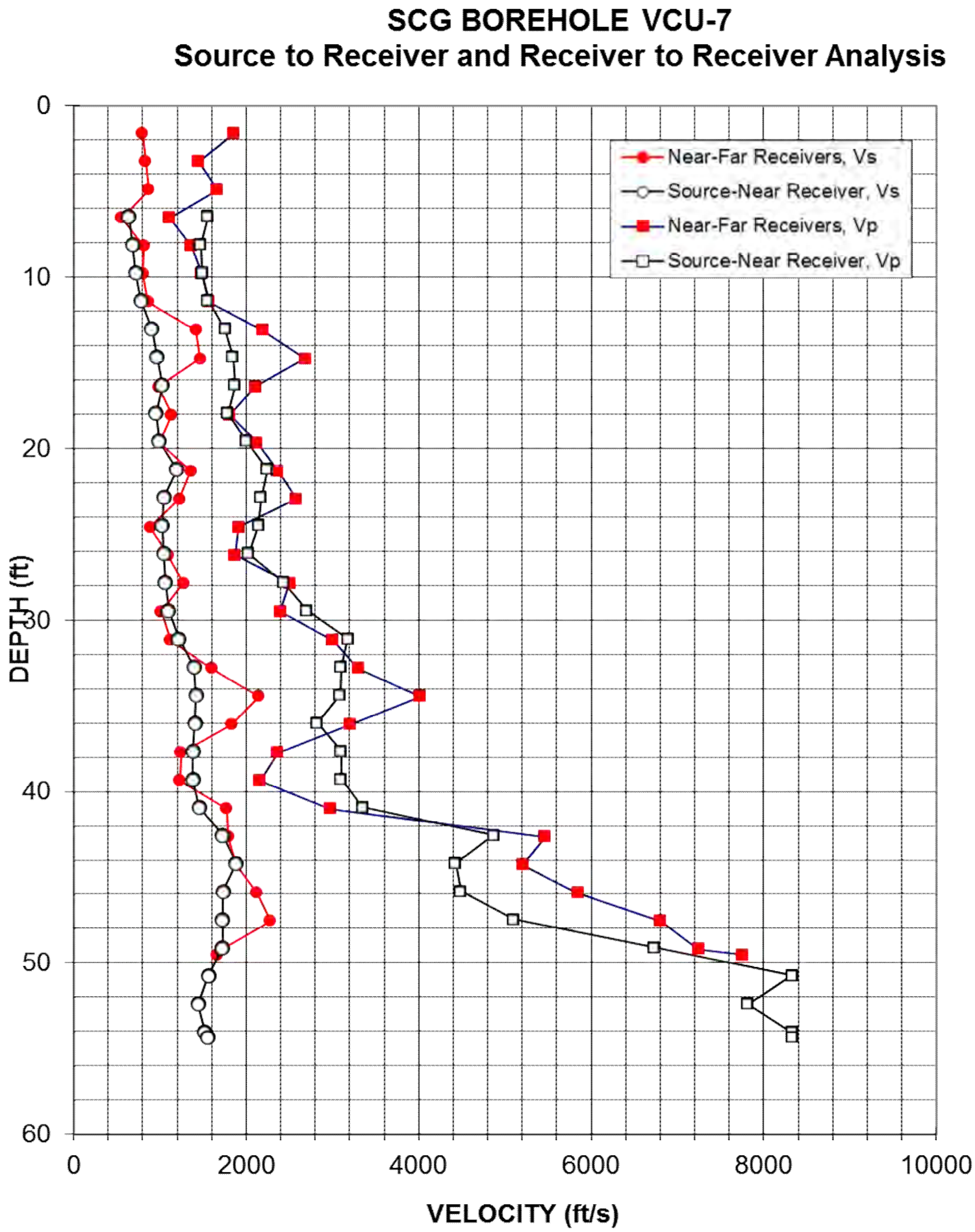

Figure A-2: Borehole VCU-7, Suspension S-R1 P- and S_H-wave velocities

Table A-2. Borehole VCU-7, S - R1 quality assurance analysis P- and S_H-wave data

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole VCU-7

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
6.5	640	1560	0.40
8.1	680	1470	0.36
9.8	720	1490	0.35
11.4	780	1560	0.33
13.0	910	1760	0.32
14.7	970	1850	0.31
16.3	1030	1870	0.28
18.0	950	1790	0.30
19.6	990	2000	0.34
21.2	1200	2240	0.30
22.9	1050	2180	0.35
24.5	1030	2150	0.35
26.2	1050	2030	0.32
27.8	1060	2430	0.38
29.4	1100	2710	0.40
31.1	1220	3180	0.41
32.7	1390	3100	0.37
34.4	1420	3090	0.37
36.0	1410	2830	0.34
37.6	1380	3100	0.38
39.3	1380	3100	0.38
40.9	1460	3350	0.38
42.6	1730	4870	0.43
44.2	1880	4430	0.39
45.8	1730	4490	0.41
47.5	1730	5100	0.44
49.1	1730	6730	0.46
50.8	1570	8330	0.48
52.4	1450	7810	0.48
54.0	1520	8330	0.48
54.4	1550	8330	0.48

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
2.0	200	480	0.40
2.5	210	450	0.36
3.0	220	460	0.35
3.5	240	480	0.33
4.0	280	540	0.32
4.5	300	560	0.31
5.0	310	570	0.28
5.5	290	550	0.30
6.0	300	610	0.34
6.5	360	680	0.30
7.0	320	660	0.35
7.5	310	650	0.35
8.0	320	620	0.32
8.5	320	740	0.38
9.0	340	820	0.40
9.5	370	970	0.41
10.0	420	950	0.37
10.5	430	940	0.37
11.0	430	860	0.34
11.5	420	950	0.38
12.0	420	950	0.38
12.5	440	1020	0.38
13.0	530	1480	0.43
13.5	570	1350	0.39
14.0	530	1370	0.41
14.5	530	1560	0.44
15.0	530	2050	0.46
15.5	480	2540	0.48
16.0	440	2380	0.48
16.5	460	2540	0.48
16.6	470	2540	0.48

Notes: "-" means no data available at that particular interval of depth.

APPENDIX B

BOREHOLE GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS

MICRO PRECISION CALIBRATION INC.

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

ilac-MRA ANAB

Calibration CERT #
AC-1969

Certificate of Calibration

Date: Jul 14, 2016 **Cert No. 222200812421160**

Customer:
GEOVISION
1124 OLYMPIC DRIVE
CORONA CA 92881

		Work Order #:	N/A
MPC Control #:	AM6768	Serial Number:	160024
Asset ID:	160024	Department:	N/A
Gage Type:	LOGGER	Performed By:	TYLER MCKEEN
Manufacturer:	OYO	Received Condition:	IN TOLERANCE
Model Number:	3403	Returned Condition:	IN TOLERANCE
Size:	N/A	Cal. Date:	July 14, 2016
Temp/RH:	72.0°F / 54.0%	Cal. Interval:	12 MONTHS
		Cal. Due Date:	July 14, 2017

Calibration Notes:
See attached data sheet for calculations. (1 Page)

Calibrated IAW customer supplied data form Rev 2.1
Frequency measurement uncertainty = 0.0005 Hz
Unit calibrated with Laptop Panasonic Model CF-29,s/n: 4FKSA41798
Calibrated To 4:1 Accuracy Ratio

This Calibration has been performed in conformance with, and complies to all requirements as set forth in S&ME purchase order SCP-0022, Dated July 13, 2016

Standards Used to Calibrate Equipment

I.D.	Description.	Model	Serial	Manufacturer	Cal. Due Date	Traceability #
DB8748	GPS TIME AND FREQUENCY RECEIVER	58503A	3625A01225	HEWLETT PACKARD	Jun 17, 2017	222008122553843
T1100	UNIVERSAL COUNTER	53131A	3546A09912	HEWLETT PACKARD	Feb 2, 2017	222008122827657
AM4000	WAVEFORM GENERATOR	33250A	MY40000703	AGILENT	Jul 8, 2017	222200812420653

Calibrating Technician: TYLER MCKEEN

QC Approval: Jim Williams

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for normal distribution corresponds to a coverage probability of approximately 95%. The standard uncertainty of measurement has been determined in accordance with EA's Publication and NIST Technical Note 1297, 1994 Edition. Services rendered comply with ISO 17025:2005, ANSI/NCSL Z540-1, MPC Quality Manual, MPC CSD and with customer purchase order instructions.

Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. The information on this report, pertains only to the instrument identified.

All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days.

MICRO PRECISION CALIBRATION INC.

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

ilac-MRA ANAB
Calibration CERT #
AC-1969

Certificate of Calibration

Date: Jul 14, 2016

Cert No. 222200812421160

Procedures Used in this Event

Procedure Name	Description
GEOVISION SEISMIC	Suspension PS Seismic Logger/Recorder Calibration Procedure

Calibrating Technician: TYLER MCKEEN

QC Approval: Jim Williams

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for normal distribution corresponds to a coverage probability of approximately 95%. The standard uncertainty of measurement has been determined in accordance with EA's Publication and NIST Technical Note 1297, 1994 Edition. Services rendered comply with ISO 17025:2005, ANSI/NCSL Z540-1, MPC Quality Manual, MPC CSD and with customer purchase order instructions.

Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. The information on this report, pertains only to the instrument identified.

All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. This report may not be reproduced in part or in a whole without the prior written approval of the issuing MPC lab.

GEOVision
geophysical services

SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.:	OYO/RG	Model no.:	3403
Serial no.:	160024	Calibration date:	7/14/16
By:	Emily Feldman	Due date:	7/14/17
Counter mfg.:	Hewlett Packard	Model no.:	53131A
Serial no.:	3546A09912	Calibration date:	2/2/16
By:	Micro Precision	Due date:	2/2/17
Signal generator mfg.:	Agilent	Model no.:	33250A
Serial no.:	MY40000703	Calibration date:	7/8/16
By:		Due date:	7/8/17
Laptop controller mfg.:	Panasonic	Model no.:	CF-29 Toughbook
Serial no.:	4FKSA41798	Calibration date:	N/A

SYSTEM SETTINGS:

Gain:	2
Filter	open Low pass 10k
Range:	200 to 50 μs
Delay:	0.3 msec
Stack (1 std)	1
System date = correct date and time	7/14/16, 1343 hrs

PROCEDURE:

Set sine wave frequency to target frequency with amplitude of approximately 0.25 volt peak
Note actual frequency on data form.
Set sample period and record data file to disk. Note file name on data form.
Pick duration of 9 cycles using PSLOG.EXE program, note duration on data form, and save as .sps file. Calculate average frequency for each channel pair and note on data form.

Average frequency must be within +/- 1% of actual frequency at all data points.

Maximum error ((AVG-ACT)/ACT*100)% As found 0.12% As left 0.12%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name C002	Time for 9 cycles Hn (msec)	Average Frequency Hn (Hz)	Time for 9 cycles Hr (msec)	Average Frequency Hr (Hz)	Time for 9 cycles V (msec)	Average Frequency V (Hz)
50.00	50.00	200	001	180.0	50.00	180.0	50.00	180.0	50.00
100.0	100.0	100	002	90.1	99.89	90.0	100.0	89.9	99.89
200.0	200.0	50	003	45.05	199.8	45.0	200.0	~~44.75~~ 45.05	~~201.1~~ 199.8 EF 7/14/16
500.0	500.0	20	004	17.98	500.6	18.0	500.0	18.0	500.0
1000	1000	10	005	8.99	1001	8.99	1001	9.0	1000
2000	2000	5	006	4.50	2000	4.505	1998	4.495	2002

Calibrated by: Tyler McKeen 7/14/16 [signature]
Name Date Signature

Witnessed by: Emily Feldman 7/14/16 [signature]
Name Date Signature

Suspension PS Seismic Recorder/Logger Calibration Data Form Rev 2.1 February 7, 2012

MICRO PRECISION CALIBRATION INC.

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

Certificate of Calibration

Date: Jul 14, 2016 **Cert No. 222200812421163**

Customer:
GEOVISION
1124 OLYMPIC DRIVE
CORONA CA 92881

Work Order #: N/A

MPC Control #:	BG9698	Serial Number:	15014
Asset ID:	15014	Department:	N/A
Gage Type:	LOGGER	Performed By:	TYLER MCKEEN
Manufacturer:	OYO	Received Condition:	IN TOLERANCE
Model Number:	03331-0000	Returned Condition:	IN TOLERANCE
Size:	N/A	Cal. Date:	July 14, 2016
Temp/RH:	72.0°F / 54.0%	Cal. Interval:	12 MONTHS
		Cal. Due Date:	July 14, 2017

Calibration Notes:
See attached data sheet for calculations. (1 Page)

Calibrated IAW customer supplied data form Rev 2.1
Frequency measurement uncertainty = 0.0005 Hz
Unit calibrated with Laptop Panasonic Model CF-29,s/n: 4FKSA41798
Calibrated To 4:1 Accuracy Ratio

Standards Used to Calibrate Equipment

I.D.	Description.	Model	Serial	Manufacturer	Cal. Due Date	Traceability #
BD7715	UNIVERSAL COUNTER	53131A	3416A05377	HEWLETT PACKARD	Aug 26, 2016	222008122635161
T1100	UNIVERSAL COUNTER	53131A	3546A09912	HEWLETT PACKARD	Feb 2, 2017	222008122827657
AM4000	WAVEFORM GENERATOR	33250A	MY40000703	AGILENT	Jul 8, 2017	222200812420653

Procedures Used in this Event

Procedure Name	Description
GEOVISION SEISMIC	Suspension PS Seismic Logger/Recorder Calibration Procedure

Calibrating Technician: TYLER MCKEEN

QC Approval: JIM WILLIAMS

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for normal distribution corresponds to a coverage probability of approximately 95%. The standard uncertainty of measurement has been determined in accordance with EA's Publication and NIST Technical Note 1297, 1994 Edition. Services rendered comply with ISO 17025:2005, ANSI/NCSL Z540-1, MPC Quality Manual, MPC CSD and with customer purchase order instructions.

Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. The information on this report, pertains only to the instrument identified.

BG 9698 15014

GEOVision
geophysical services

SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.:	OYO	Model no.:	3331
Serial no.:	15014	Calibration date:	07/14/2016
By:	Emily Feldman	Due date:	07/14/2017
Counter mfg.:	Hewlett Packard	Model no.:	53131A
Serial no.:	3546A09912	Calibration date:	02/02/16
By:	Micro Precision	Due date:	02/02/17
Signal generator mfg.:	Agilent	Model no.:	~~07~~ 33250A
Serial no.:	MY40000703	Calibration date:	07/08/16
By:	Micro Precision	Due date:	07/08/17
Laptop controller mfg.:	Panasonic	RG3 Model no.:	CF-29 Toughbook
Serial no.:	4FKSA41798	Calibration date:	N/A

SYSTEM SETTINGS:

Gain:	x10
Filter	open
Range:	200 - 5 μS
Delay:	4 msec
Stack (1 std)	1
System date = correct date and time	Yes, 12:25, 7/14/16

PROCEDURE:

Set sine wave frequency to target frequency with amplitude of approximately 0.25 volt peak
Note actual frequency on data form.
Set sample period and record data file to disk. Note file name on data form.
Pick duration of 9 cycles using PSLOG.EXE program, note duration on data form, and save as .sps file. Calculate average frequency for each channel pair and note on data form.

Average frequency must be within +/- 1% of actual frequency at all data points.

Maximum error ((AVG-ACT)/ACT*100)% As found 0.22% As left 0.22%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name Cal2-	Time for 9 cycles Hn (msec)	Average Frequency Hn (Hz)	Time for 9 cycles Hr (msec)	Average Frequency Hr (Hz)	Time for 9 cycles V (msec)	Average Frequency V (Hz)
50.00	50.00	200	001	180	50.00	180	50.00	179.8	50.05
100.0	100.0	100	002	89.9	99.9	89.9	99.9	90.1	99.9
200.0	200.0	50	003	44.95	200.2	45	200.0	45	200.0
500.0	500.0	20	004	18.02	499.5	18.0	~~18.0~~ 500.0	17.98	500.6
1000	1000	10	005	9.01	998.9	8.98	1002.2	9	1000
2000	2,000	5	006	4.505	1998	4.50	2000	4.495	2002

Calibrated by: Tyler McKean 7/14/16
Name Date Signature

Witnessed by: Emily Feldman 7/14/16
Name Date Signature

Suspension PS Seismic Recorder/Logger Calibration Data Form Rev 2.1 February 7, 2012