Appendix E: Geotechnical Investigation

GEOTECHNICAL INVESTIGATION
PROPOSED ENERGY STORAGE BUILDING

28636 Matthews Road
Menifee, California
for
PNG Builders

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

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A California Corporation

December 18, 2024

PNG Builders
2392 South Bateman Avenue
Irwindale, California 91010

Attention: Mr. Picasso Bhowmik
Vice President

Project No.: **24G174-1**

Subject: **Geotechnical Investigation**
Proposed Energy Storage Building
28636 Matthews Road
Menifee, California

Mr. Bhowmik:

In accordance with your request, we have conducted a geotechnical investigation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Joseph Lozano Leon
Staff Engineer

Robert G. Trazo, GE 2655
Principal Engineer

Distribution: (1) Addressee

REGISTERED PROFESSIONAL ENGINEER
ROBERT TRAZO
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1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Geotechnical Design Considerations

- Existing undocumented fill soils were encountered at most of the boring locations, extending to depths of 2½ to 4½± feet from the ground surface. The fill soils are underlain by native younger alluvium which possesses varying strengths and densities.
- The results of laboratory testing indicate that the near-surface alluvial soils within the upper 5 to 7± feet possess a moderate potential for collapse when exposed to moisture infiltration as well as moderate consolidation when exposed to load increases in the range of those that will be exerted by the foundations of a new industrial building.
- The Riverside County GIS website indicates that the subject site is located within a zone of low liquefaction susceptibility. In addition, based on the subsurface exploration performed for this project, the project site is generally underlain by dense to very dense older alluvium. Based on the mapping performed by the county of Riverside and the relatively high soil strengths encountered at the boring locations, liquefaction is not considered to be a design concern for this project.
- Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, Southern California Geotechnical, Inc. did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low.
- The potential for other geologic hazards such as seismically induced settlement, lateral spreading, tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.
- Based on the corrosivity testing results, the on-site soils are considered to be mildly corrosive to ferrous pipes. Therefore, corrosion protection may be required for cast iron or ductile iron pipes.
- Based on the site plan for the proposed development, the new building south wall will be constructed approximately 2 feet from an existing Southern California Edison (SCE) easement.

Site Preparation

- Initial site stripping should include removal of any surficial vegetation, as well as any underlying topsoil or other organic materials. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.
- Demolition should include removal of utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of off-site.
- Remedial grading is recommended to be performed within the proposed building area in order to remove all of the undocumented fill soils in their entirety, and the upper portion of the near-surface native alluvial soils. The soils within the proposed building area should be overexcavated to a depth of at least 5 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevations, whichever is greater.

- The depth of overexcavation should also be sufficient to remove any existing fill soils. The proposed foundation influence zones should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade. The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill placed below the foundation bearing grade, whichever is greater.
- The proposed building south wall will be constructed as close as 2± feet to an existing SCE easement. The contractor should take all necessary provisions to protect any improvements on the adjacent properties. Shoring or slot cutting may be required to complete these excavations. Typically, A-B-C slot cuts on 6 to 8-foot centers are suitable to maintain excavation stability. Generally, the most feasible means of completing the slot-cutting process is to make any initial cuts to grade. An excavator or similar equipment can then be used to make 6 to 8-foot-wide slots perpendicular to the easement boundary in shallow trenches. These trenches would extend to the recommended base of overexcavation depth. The slots can then be backfilled to surface grade using the adjacent excavation sidewalls for confinement. These slots should extend onto the subject site a distance equal to or greater than the recommended depth of overexcavation. The mass overexcavation of the building pad area can then be performed, benching into the fill placed during the slot cutting procedures. The geotechnical engineer should observe the conditions and determine the appropriate slot cutting procedures at the time of site grading. A reduced soil bearing pressure of 1,500 lbs/ft^2 should be used in the design of new footings where the full recommended lateral extent of remedial grading cannot be achieved.
- Following completion of the overexcavation, the subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting subgrade should then be scarified to a depth of 12 inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Conventional shallow foundations, supported in structural fill.
- 2,500 lbs/ft^2 maximum allowable soil bearing pressure.
- 1,500 lbs/ft^2 reduced allowable soil bearing pressure if the full recommended lateral extent of remedial grading cannot be achieved.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Substation Foundations

- The foundations for the substation will consist of cast-in-drilled-hole (CIDH) piles.
- The CIDH piles should extend through the existing undocumented fill soils and younger alluvium into the moderate to high strength older alluvial soils below. Based on conditions encountered at the boring locations, we recommended that the top of the pile is embedded at least 18 inches below adjacent exterior grade.
- The minimum pile diameter should be 24 inches with a minimum length of 20 feet.

Building Floor Slab

- Conventional Slab-on-Grade: minimum 6-inch thickness.

- Modulus of Subgrade Reaction: k = 120 psi/in.
- Minimum slab reinforcement: No. 3 bars at 18 inches on center in both directions.
- The actual thickness and reinforcement of the floor slab should be determined by the structural engineer.

Pavements

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	4	6	7	8	10
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 24P329R, dated November 18, 2024. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, H-frame substation foundations and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located on the northeast side of Matthews Avenue at the McLaughlin Road easement intersection in Menifee, California. The site is bounded to the north by the McLaughlin Road easement, to the southwest by Matthews Road, and to the east by an equipment storage yard. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The site consists of a triangular-shaped parcel, 3.7± acres in size. However, due to the existing easements along the southwest and north property lines, the subject site is 3.24± acres in size. The site is presently utilized as an equipment and work vehicle storage yard. Two (2) mobile trailers, 1,440± ft^2 in size each, are present in the central area of the site. Several storage containers are located along the southwest property line. A canopy structure and concrete pad are present in the southern area of the site. The ground surface cover consists of open-graded gravel with isolated areas of exposed soil.

Detailed topographic information was obtained from an ALTA plan prepared by Dewalt Corporation. Based on this plan, the site slopes downward to the west-southwest at gradients of 1 to 3± percent. The maximum site elevation is 1,464± feet mean sea level (msl), in the northeast corner of the site, and the minimum site elevation is 1,460± feet msl, in the northwest corner.

3.2 Proposed Development

Our office was provided with a conceptual site plan, identified as Scheme B and dated October 31, 2024, which was prepared by GAA Architects. Based on this plan, the site will be developed with a new energy storage building, 41,300± ft^2 in size. The building will be located in the eastern area of the site. In addition, an H-frame substation will be constructed in the central area of the site. The proposed structures are expected to be surrounded by asphaltic concrete (AC) or Portland cement concrete (PCC) pavements in the equipment storage areas and parking/drive lane areas, and concrete flatwork. It should be noted that the new building's south wall will be constructed approximately 2 feet from the existing Southern California Edison (SCE) easement.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, supported on a conventional shallow foundation system with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively. Based on information provided by the client, the proposed H-frame substation will be supported on drilled piers, with estimated axial and shear loads of 5 kips and an estimated 300 kips per linear foot for overturning moments. The proposed development is not expected to include any significant amounts of below-grade construction such as basements or crawl spaces. Based on the relatively level topography, cuts and fills of less than 1 to 2± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of six (6) borings (identified as Boring Nos. B-1 through B-6) advanced to depths of 5 to 50± feet below the existing site grades. The boring locations were cleared by a private geophysical testing company prior to drilling. The borings were logged during drilling by a member of our staff.

The borings were advanced with hollow-stem augers, by a truck-mounted drilling rig. Representative bulk and undisturbed soil samples were taken during drilling. Relatively undisturbed samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. Samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

The approximate locations of the borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Pavements

All borings were drilled through a surficial 1± inch-thick layer of open-graded gravel.

Artificial Fill

Artificial fill soils were encountered beneath the existing pavements at all of the boring locations, with the exception of Boring B-6, extending to depths of 2½ to 4½± feet below the existing site grades. The fill soils generally consist of loose to very dense silty sands with varying clay and fine to coarse gravel content. The fill soils possess a disturbed and mottled appearance resulting in their classification as artificial fill.

Younger Alluvium

Native younger alluvium was encountered beneath the artificial fill soils at Boring Nos. B-1 through B-5 and beneath the open-graded gravel at Boring No. B-6, extending to depths of 5 to 12± feet

below the existing site grades. The younger alluvium generally consists of medium dense sandy silts, silty sands and clayey sands, with occasional loose silty sands.

Older Alluvium

Older native alluvium was encountered beneath the younger alluvium at all of the boring locations, with the exception of Boring No. B-5, extending to at least the maximum depth explored of 50± feet below existing site grades. The older alluvium generally consists of dense to very dense sands, silty sands and sandy silts.

Groundwater

Free water was encountered during drilling at Boring No. B-4 at a depth of 32± feet below the ground surface. However, based on the moisture content of the recovered soil samples at greater depths, we believe that perched water was encountered at the boring location at a depth 32± feet. If more detailed ground water information is desired, we recommend that piezometers be installed to help determine the static groundwater table for this site.

As part of our research, we reviewed available groundwater data in order to determine high groundwater level for the site. The primary reference used to determine water levels is the California Department of Water Resources, Water Data Library Station Map, website, https://wdl.water.ca.gov/waterdatalibrary/. One monitoring well is located as close as 2,200± feet southeast of the subject site. Water level readings within this well indicate a high groundwater level of 52± feet below the ground surface in June of 1995.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

Recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. Field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-6 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative soil sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date. The result of this testing is plotted on Plate C-7 in Appendix C of this report.

Direct Shear

A direct shear test was performed on a selected soil sample to determine its shear strength parameters. The test was performed in accordance with ASTM D-3080. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Three samples of the same soil are prepared by remolding them to 90±

percent compaction and near optimum moisture. Each of the three samples are then loaded with different normal loads and the resulting shear strength is determined for that particular normal load. The shearing of the samples is performed at a rate slow enough to permit the dissipation of excess pore water pressure. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The results of the direct shear test are presented on Plate C-8 in Appendix C of this report.

Thermal Resistivity Dry-Out Curve

A representative sample of the near-surface soils was submitted to a subcontracted analytical laboratory for thermal resistivity testing. The results will be obtained using the method for Determination of Thermal Conductivity in general accordance with ASTM D-5334. The laboratory results were not available at the time of this report. The result of this testing will be provided in an addendum report.

In-Situ Soil Resistivity Testing

In-situ soil resistivity testing was performed for this project. The field testing for this procedure provides the average soil electrical resistivity to a depth equal to the spacing between the pins. Measurements for this procedure were in accordance with the "Wenner 4 Pin" Method ASTM G57. The results of the field resistivity testing were not available at the time of this report. The result of this testing will be provided in an addendum report.

Expansion Index (EI)

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829 as required by the California Building Code (CBC). The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The result of the EI testing is as follows:

Sample Identification	**Expansion Index**	**Expansive Potential**
B-1 @ 1 to 5 feet	17	Very Low

Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	**Soluble Sulfates (%)**	**Severity**	**Class**
B-3 @ 1 to 5 feet	0.006	Not Applicable	S0
B-4 @ 1 to 5 feet	0.005	Not Applicable	S0

Corrosivity Testing

Representative samples of the near-surface soils were submitted to a subcontracted corrosion engineering laboratory for determination of electrical resistivity, pH, and chloride concentrations. The resistivity of the soils is a measure of their potential to attack buried metal improvements such as utility lines. The results of some of these tests are presented below.

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)	Sulfides (mg/kg)	Redox Potential (mV)
B-3 @ 1 to 5 feet	2,948	7.1	27.8	92.8	1.4	126
B-4 @ 1 to 5 feet	3,350	6.8	29.6	66.4	0.9	145

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations.

The recommendations are contingent upon grading and foundation construction activities being monitored by the geotechnical engineer of record. The recommendations are provided with the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to verify compliance with these recommendations. Maintaining Southern California Geotechnical, Inc., (SCG) as the geotechnical consultant from the beginning to the end of the project will provide continuity of services. The geotechnical engineering firm providing testing and observation services shall assume the responsibility of Geotechnical Engineer of Record.

The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site-specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structure should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. In addition, our review of the Riverside County RCIT GIS website indicates that the site is not located within a Riverside County fault zone. Furthermore, Southern California Geotechnical, Inc., (SCG) did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low.

The potential for other geologic hazards such as seismically induced settlement, lateral spreading, tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low. Liquefaction is a potential geologic hazard for this site and is discussed below.

Seismic Design Parameters

The 2022 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2022 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2022 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents. The table below was created using data obtained from the application. The output generated from this program is included as Plate E-1 in Appendix E of this report.

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.423
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.530
Site Class	---	C
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.708
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.779
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.138
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.519

Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and grain size characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Clayey (cohesive) soils or soils which possess clay particles (d<0.005mm) in excess of 20 percent (Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The Riverside County GIS website indicates that the subject site is located within a zone of low liquefaction susceptibility. In addition, based on the subsurface exploration performed for this project, the project site is generally underlain by dense to very dense older alluvium. Based on the mapping performed by the county of Riverside and the relatively high soil strengths

encountered at the boring locations, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

Artificial fill soils were encountered at most of the boring locations, extending to depths of 2½ to 4½± feet from the ground surface. The fill soils are generally underlain by native younger alluvium. Native older alluvium is generally present beneath the younger alluvium, as shallow as 6½± feet from the ground surface. The fill soils and near-surface alluvial soils possess varying strengths and densities. The existing fill soils are considered to represent undocumented fill. The results of laboratory testing indicate that the near-surface alluvial soils within the upper 5 to 7± feet possess a moderate potential for collapse when exposed to moisture infiltration as well as moderate consolidation when exposed to load increases in the range of those that will be exerted by the foundations of a new industrial building. Neither the undocumented fill soils nor the near-surface native alluvium, in their present condition, are considered suitable to support the foundations loads of the new buildings Therefore, remedial grading is considered warranted within the proposed building area in order to remove all of the undocumented fill soils in their entirety and the upper portion of the near-surface native alluvial soils, and replace these materials as compacted structural fill soils.

The proposed building will be constructed 2± feet from an existing SCE easement. This configuration will likely require specialized excavation techniques such as slot-cutting or shoring. Specific recommendations are presented in a subsequent section of this report.

Based on email correspondence with the client, the proposed H-frame substation will be supported on concrete foundation elements, consisting of cast-in-drilled-hole (CIDH) piles. Specific CIDH pile design recommendations are presented in a subsequent section of this report.

Settlement

The proposed remedial grading will remove the existing undocumented fill soils and a portion of the near-surface native alluvial soils from within the proposed building area, and replace these materials as compacted structural fill. The native soils that will remain in place beneath the recommended depth of overexcavation will not be subject to significant stress increases from the foundations of the new structure. Therefore, following completion of the recommended remedial grading, post-construction static settlements are expected to be within tolerable limits.

Expansion

The near-surface soils at this site generally consist of silty sands, clayey sands, and sandy silts. The results of laboratory testing indicate that the near-surface soils possess very low expansive potentials (EI = 17). The foundation and floor-slab design recommendations contained within this report are made in consideration of the expansion index test results. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Soluble Sulfates

The results of the soluble sulfate testing, discussed in Section 5.0 of this report, indicate soluble sulfate concentrations less than 0.006 percent. These concentrations are considered to be negligible or "not applicable" with respect to the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the structure area.

Corrosion Potential

The results of laboratory testing indicate that the tested samples of the near-surface soils possess saturated resistivities of 2,948 and 3,350 ohm-cm, and pH values of 6.8 and 7.1. The soils possess redox potentials of 126 and 145 mV and sulfide concentrations of 0.9 and 1.4 mg/kg. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity, pH, sulfide concentration, redox potential, and moisture content are the five factors that enter into the evaluation procedure. Based on these factors, the on-site soils are considered to be mildly corrosive to ferrous pipes. Therefore, corrosion protection may be required for cast iron or ductile iron pipes.

Based on American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary, reinforced concrete that is exposed to external sources of chlorides requires corrosion protection for the steel reinforcement contained within the concrete. ACI 318 defines concrete exposed to moisture and an external source of chlorides as "severe" or exposure category C2. ACI 318 does not clearly define a specific chloride concentration at which contact with the adjacent soil will constitute a "C2" or severe exposure. However, the Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 mg/kg are considered to be corrosive to reinforced concrete. The results of the laboratory testing indicate chloride concentrations ranging from 27.8 to 29.6 mg/kg. Although the soils contain some chlorides, we do not expect that the chloride concentrations of the tested soils are high enough to constitute a "severe" or C2 chloride exposure. Therefore, a chloride exposure category of C1 is considered appropriate for this site.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations ranging from 66.4 to 92.8 mg/kg. **Based on the test results, the on-site soils are considered to be corrosive to copper pipe**.

Since SCG does not practice in the area of corrosion engineering, we recommend that the client contact a corrosion engineer to provide a more thorough evaluation of these test results.

Shrinkage/Subsidence

Removal and recompaction of the near-surface younger alluvium is estimated to result in an average shrinkage of 9 to 19 percent. However, potential shrinkage for individual samples ranged locally between 1 and 23 percent. The potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1 to 0.15± feet.

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

Grading and foundation plans were unavailable at the time of this report. It is therefore recommended that we be provided with copies of the preliminary grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations, and our understanding of the proposed development. We recommend that grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

Existing improvements that will not remain in place for use with the new development should be removed in their entirety. This should include all pavements and utilities, and any other subsurface improvements associated with the existing development. Demolition debris should be disposed of off-site. All applicable federal, state and local specifications and regulations should be followed in demolition, abandonment, and disposal of the resulting debris.

Initial site stripping should include removal of any surficial vegetation from the unpaved areas of the site. This should include any weeds, grasses, shrubs, and trees. Root systems associated with the trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building area in order to remove the existing undocumented fill soils and the upper portion of the near-surface native alluvium. Based on conditions encountered at the boring locations, the fill soils extend to depths of 2½ to 4½± feet below the existing site grades. These fill soils should be removed in their entirety. It is also recommended that the existing soils within the proposed building area be overexcavated to a depth of at least 5 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevations, whichever is greater.

Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade. The overexcavation should extend at least 5 feet beyond the building perimeter, and to an extent equal to the depth of new fill below the foundation bearing grade, whichever is greater. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the overexcavation should also encompass these areas.

The remedial grading activities within the proposed building area will require excavation in close proximity to an existing SCE easement. Therefore, shoring or slot cutting techniques may be required in this area of the site to remove the existing undocumented fill soils and the upper near-surface alluvium while providing adequate lateral support for the adjacent properties. It is recommended that copies of the grading and foundation plans be provided to our office for review with regard to the need for specialized grading techniques in these areas of the site.

Following completion of the overexcavation, the subgrade soils within the building area should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, or low-density native soils are encountered at the base of the overexcavation.

Materials suitable to serve as the structural fill subgrade within the building area should consist of native soils which possess an in-situ density equal to at least 85 percent of the ASTM D-1557 maximum dry density. After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture treated to 0 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The building pad area may then be raised to grade with previously excavated soils or imported, structural fill.

Treatment of Existing Soils: H-Frame Substation Area

Based on email correspondence with the client, and the results of our subsurface exploration, it is expected that the foundations for the substation will consist of at least 24-inch diameter 20-foot-long cast-in-drilled-hole (CIDH) piles. In addition, the top of the pier is recommended to be embedded at least 18 inches below adjacent exterior grade. Since the substation footings are typically subjected to relatively low axial loads, overexcavation in these foundation areas is not considered warranted. Therefore, once the excavations have been made to reach the nominal foundation bearing grade, the exposed subgrade soils should be evaluated by the geotechnical

engineer. Assuming that these excavations are founded in dense to very dense older alluvium, no additional excavation or embedment will be required.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 3 feet below foundation bearing grade and replaced as compacted structural fill. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 3 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill. These grading recommendations apply to small (less than 5 feet in height) retaining walls. Otherwise, the geotechnical engineer of record should be contacted for supplementary recommendations.

If the full lateral extent of overexcavation is not achievable for the proposed walls, foundation elements must be redesigned using a lower bearing pressure. The geotechnical engineer of record should be contacted for recommendations pertaining to this type of condition.

Treatment of Existing Soils: Flatwork, Parking and Drive Areas

Based on economic considerations, overexcavation of the existing near-surface existing soils in the new flatwork, parking and drive areas is not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading. Subgrade preparation in the new flatwork, parking and drive areas should initially consist of removal of soils disturbed during stripping and demolition operations.

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed flatwork, parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within these areas. The grading recommendations presented above do not mitigate the extent of undocumented fill soils or potentially compressible/collapsible alluvium in the flatwork, parking and drive areas. As such, some settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the flatwork, parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- Grading and fill placement activities should be completed in accordance with the requirements of the 2022 CBC and the grading code of the city of Menifee.
- Fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

Imported structural fill should consist of very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Menifee. Utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v (horizontal to vertical) plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

Soils used to backfill voids around subsurface utility structures, such as manholes or vaults, should be placed as compacted structural fill. If it is not practical to place compacted fill in these areas, then such void spaces may be backfilled with lean concrete slurry. Uncompacted pea gravel or sand is not recommended for backfilling these voids since these materials have a potential to settle and thereby cause distress of pavements placed around these subterranean structures.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of sands, silty sands and sandy silts, with occasional clayey sands. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes should be made no steeper than 2h:1v. Within clayey sands, temporary excavation slopes of 1½h:1v may be feasible. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. Excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Remedial grading for the proposed building south wall will require excavation in close proximity to an existing SCE easement. In general, the contractor should take necessary provisions to protect any improvements on the adjacent properties. Slot cutting may be required to complete these excavations. Typically, A-B-C slot cuts on 6 to 8-foot centers are suitable to maintain excavation stability. Generally, the most feasible means of completing the slot-cutting process is to make any initial cuts to grade. An excavator or similar equipment can then be used to make 6 to 8-foot-wide slots perpendicular to the easement boundary in shallow trenches. These trenches would extend to the recommended base of overexcavation depth. The slots can then be backfilled to surface grade using the adjacent excavation sidewalls for confinement. These slots should extend onto the subject site a distance equal to or greater than the recommended depth of overexcavation. The mass overexcavation of the building area can then be performed, benching into the fill placed during the slot cutting procedures. The geotechnical engineer should observe the conditions and determine the appropriate slot cutting procedures at the time of site grading.

Moisture Sensitive Subgrade Soils

The near-surface soils possess appreciable silt and clay content and will become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a drier, less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad area as well as the need for a stabilization layer.

Groundwater

Perched water is present at a depth of 32± feet below the existing grades at the time of the subsurface exploration. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Shallow Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by newly placed structural fill soils used to replace all of the existing undocumented fill soils and a portion of the near-surface alluvial soils. These new structural fill soils are expected to extend to a depth of at least 3 feet below proposed foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed structure may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Maximum, net allowable soil bearing pressure: 1,500 lbs/ft^2 if the full recommended lateral extent of remedial grading cannot be achieved.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: four (4) No. 5 rebars (2 top and 2 bottom).
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on geotechnical considerations; additional reinforcement may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The building foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Unsuitable materials should be removed to a depth of suitable bearing compacted structural fill or suitable native alluvium (where reduced bearing pressures are utilized), with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 0 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 50-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 275 lbs/ft^3
- Friction Coefficient: 0.28

If the full recommended lateral extent of remedial grading cannot be achieved, typically for new footings along the property lines, the allowable passive pressure should be reduced to 250 lbs/ft^3.

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2,500 lbs/ft^2.

6.6 CIDH Pile Foundation Design and Construction

Based on email correspondence with the client, and the results of our subsurface exploration, it is expected that the foundations for the substation will consist of cast-in-drilled-hole (CIDH) piles. It is recommended that the CIDH pile foundations extend through the existing undocumented fill soils and younger alluvium into the moderate to high strength older alluvial soils below.

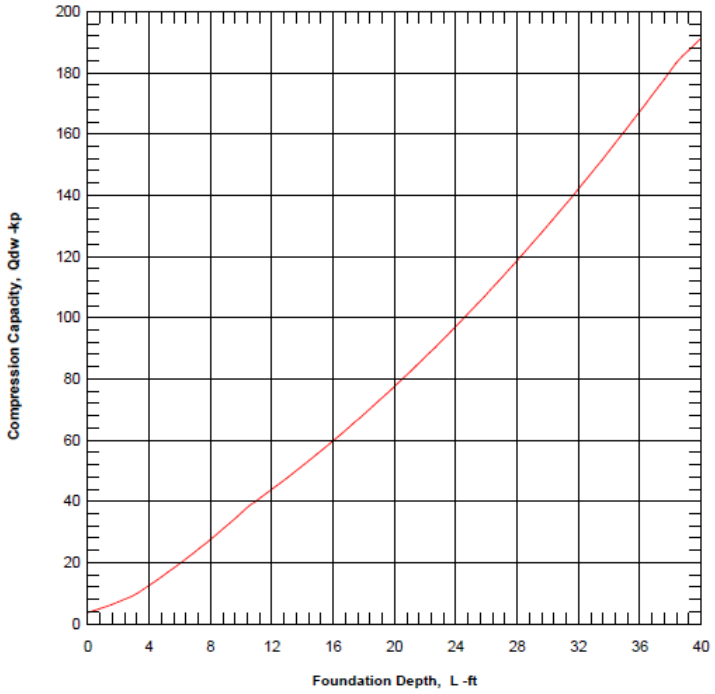
The CIDH pile capacities were obtained using All-Pile, a computer program developed by Civil Tech Software of Bellevue, Washington. **The CIDH pile capacities presented below neglect any skin friction contribution within the zone of the fill soils, which extend to depths of 2½± feet below the existing site grades. The top of the pier is recommended to be embedded at least 18 inches below adjacent exterior grade.**

Bearing Horizon

It is recommended that the CIDH piles be at least 20 feet in length. This will allow the CIDH piles to be embedded into moderate to higher strength older native materials.

Compression Capacity

The graphs below present the allowable capacities for a 24-inch-diameter CIDH piles embedded to depths of up to 40± feet. **As discussed above, it is recommended that all CIDH piles are embedded 18 inches below adjacent exterior grade, and are at least 20 feet long.**



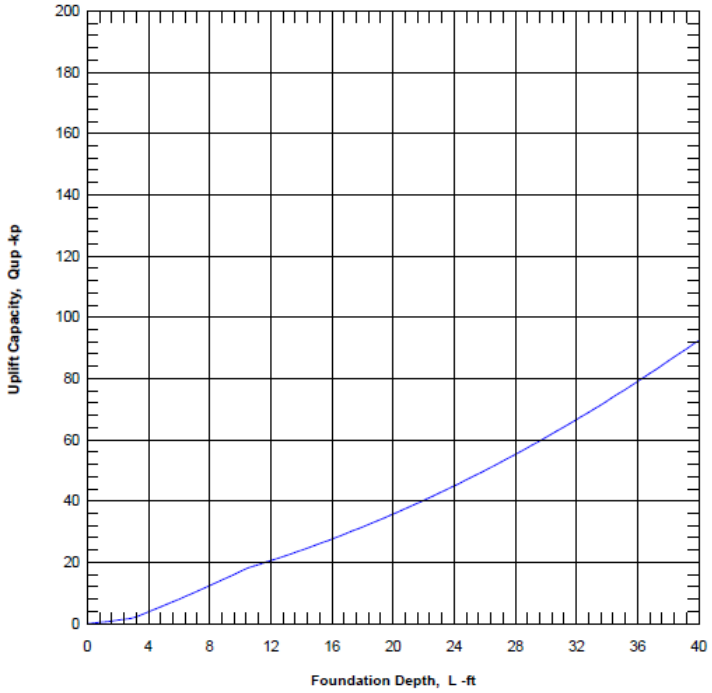
24-inch diameter CIDH Pile - Compression

The allowable compression capacities presented above may be increased by one-third when considering short duration wind or seismic loads. The allowable capacities presented above include a factor of safety of 3.0 for end bearing and 2.0 for skin friction. These capacities are designed to limit the CIDH pile settlement to less than 0.50 inches, under static conditions. The capacities presented above are net capacities, and as such the weight of the concrete within the CIDH pile may be neglected.

More detailed information regarding the capacities of the CIDH piles, including anticipated settlements, are presented on the ALLPILE output forms included in Appendix F of this report.

Uplift Capacity

The allowable uplift capacities for a 24-inch-diameter CIDH pile are presented graphically below.



24-inch diameter CIDH Pile – Uplift Capacity

The allowable uplift capacities presented above include a factor-of-safety of at least 2 with regard to skin friction and a factor-of-safety of 1 with regard to the weight of the CIDH pile. Therefore, they may not be increased by one-third when considering short duration wind or seismic loads.

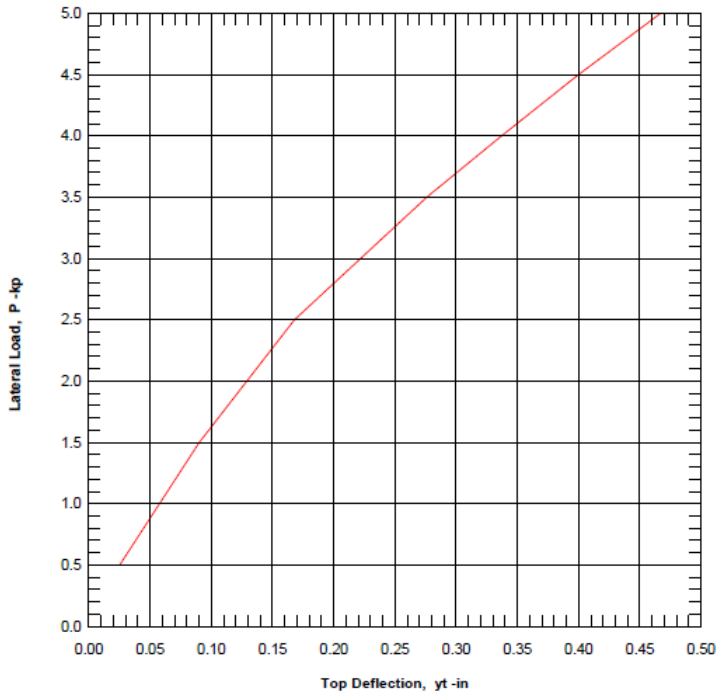
More detailed information regarding the capacities of the CIDH piles are presented on the ALLPILE output forms included in Appendix F of this report.

Settlement

Post-construction total settlements of a CIDH piles designed and constructed in accordance with the above recommendations are expected to be less than 1.0 inch, but will vary with the CIDH pile size. The differential settlement between adjacent CIDH piles is estimated to be less than 0.5 inches. Where CIDH piles are designed for less than the full allowable capacity, a corresponding reduction in total settlement should be expected. Where grade beams are used to connect the CIDH piles, the differential settlements between structurally connected CIDH piles will be less.

Lateral Capacity

The graph below presents the lateral deflections calculated for lateral loads ranging between 0.5 and 5 kips for 24-inch diameter 20-foot-long CIDH piles. Currently, no information has been provided regarding the allowable CIDH pile top-deflections. The calculated deflections are based on a free-head condition.



24-inch diameter CIDH Pile – Lateral Capacity

CIDH Pile Construction

Minimum CIDH pile shaft diameters should be 24 inches to help eliminate arching of concrete and possible void formation within the CIDH piles. On-center CIDH pile spacing should be at least four (4) times the CIDH pile diameter at the bearing surface to eliminate an overlapping axial stress influence. At a minimum, a CIDH pile spacing equivalent to three (3) times the CIDH pile diameter could be utilized, with an associated 20 percent reduction in allowable axial capacities. Based on the conditions encountered at the boring locations, perched water intrusion is not expected to occur. However, if caving or groundwater intrusion does occur during drilling, casing or liners will be required.

Prior to the placement of concrete, a clean-out bucket should be used to ensure that excess materials in the bottom of the CIDH pile have been sufficiently removed and that the dimensions of the CIDH pile are correct. Concrete should be placed using a tremie pipe whenever the distance of fall is greater than five (5) feet. Concrete should be placed at about a 6-inch slump when long reinforcing steel is used. It is recommended that the CIDH pile construction be performed in accordance with American Concrete Institute documents (ACI 336, I-79 and ACI 336-3R-72, both revised 1985). In the event that casing is required, a sufficient head of concrete (minimum of 5 feet) should be maintained in the casing as the casing is being removed to prevent the intrusion of caving soils in the CIDH pile.

It is recommended that the bearing materials at each CIDH pile location be evaluated by the geotechnical engineer prior to placing steel or concrete.

6.7 Floor Slab Design and Construction

Subgrades which will support the new floor slab should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 3 feet below finished pad grades. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: k = 120 psi/in.

- Minimum slab reinforcement: No. 3 bars at 18 inches on-center, in both directions. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab where such moisture sensitive floor coverings are anticipated. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as a 15-mil Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 0 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

- The floor slab should be structurally connected to the foundations as detailed by the structural engineer.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

6.8 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 3 feet in height) retaining walls may be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near-surface soils generally consist of silty sands and sandy silts. Based on the direct shear testing performed on a selected soil sample, these materials are expected to possess a friction angle of about 29 degrees.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type
		On-site Silty Sands and Sandy Silts
Internal Friction Angle (ϕ)		29°
Unit Weight		136 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	47 lbs/ft^3
	Active Condition (2h:1v backfill)	79 lbs/ft^3
	At-Rest Condition (level backfill)	70 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 275 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive

resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In accordance with the 2022 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be underlain by at least 2 feet of newly placed structural fill. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils may be used to backfill the retaining walls, provided that they are very low expansive (EI < 20). All backfill material placed within 3 feet of the back wall-face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a minimum 1-foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) be placed against the face of the retaining walls. This material should extend from the top of the retaining wall footing to within 1 foot of the ground surface on the back side of the retaining wall. This material should be approved by the geotechnical engineer. In lieu of the 1-foot thick layer of free-draining material, a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, may be used. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The layer of free draining granular material should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 2-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the

wall and at an approximate 10-foot on-center spacing. Alternatively, 4-inch diameter holes at an approximate 20-foot on-center spacing can be used for this type of drainage system. In addition, the weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system. The actual design of this type of system should be determined by the civil engineer to verify that the drainage system possesses the adequate capacity and slope for its intended use.

Weep holes or a footing drain will not be required for building stem walls.

6.9 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be supported on the existing fill soils that have been scarified, moisture conditioned, and recompacted. The near-surface soils generally consist of silty sands. Based on their classification, these materials are expected to possess good to excellent pavement support characteristics, with R-values in the range of 40 to 60. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 40. Fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering observed and tested conditions. It is recommended that R-value testing be performed after completion of rough grading to verify that the pavement design recommendations presented herein are valid.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20-year design life, assuming six operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.0	0
5.0	1
6.0	3
7.0	11
8.0	35
9.0	93

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	4	6	7	8	10
Compacted Subgrade	12	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the batch plant-reported maximum density. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. Reinforcement within the PCC pavements should be evaluated by the project structural engineer. The maximum joint spacing within the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

7.0 GENERAL COMMENTS

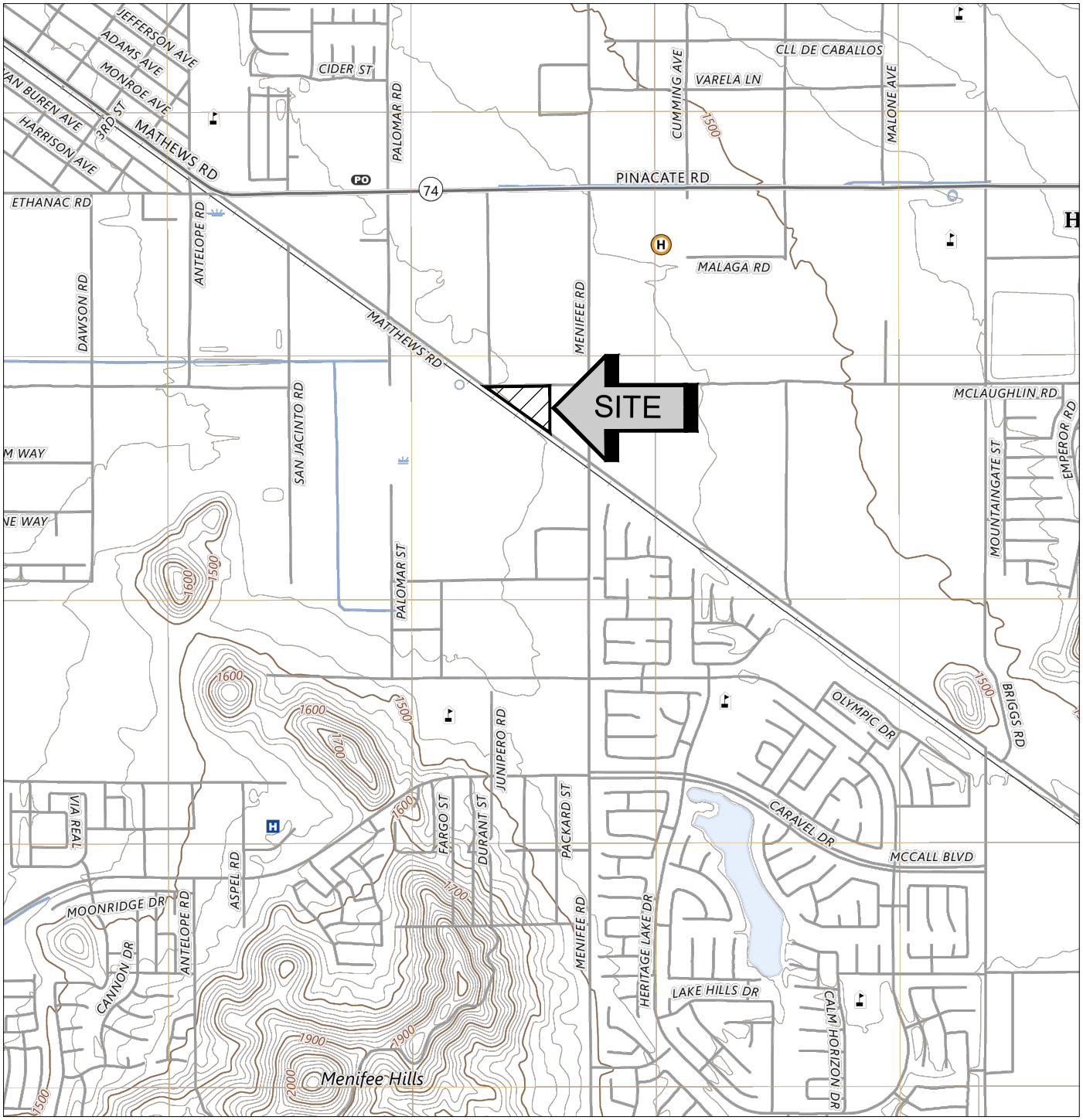
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE ROMOLAND QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA, 2023.

SITE LOCATION MAP

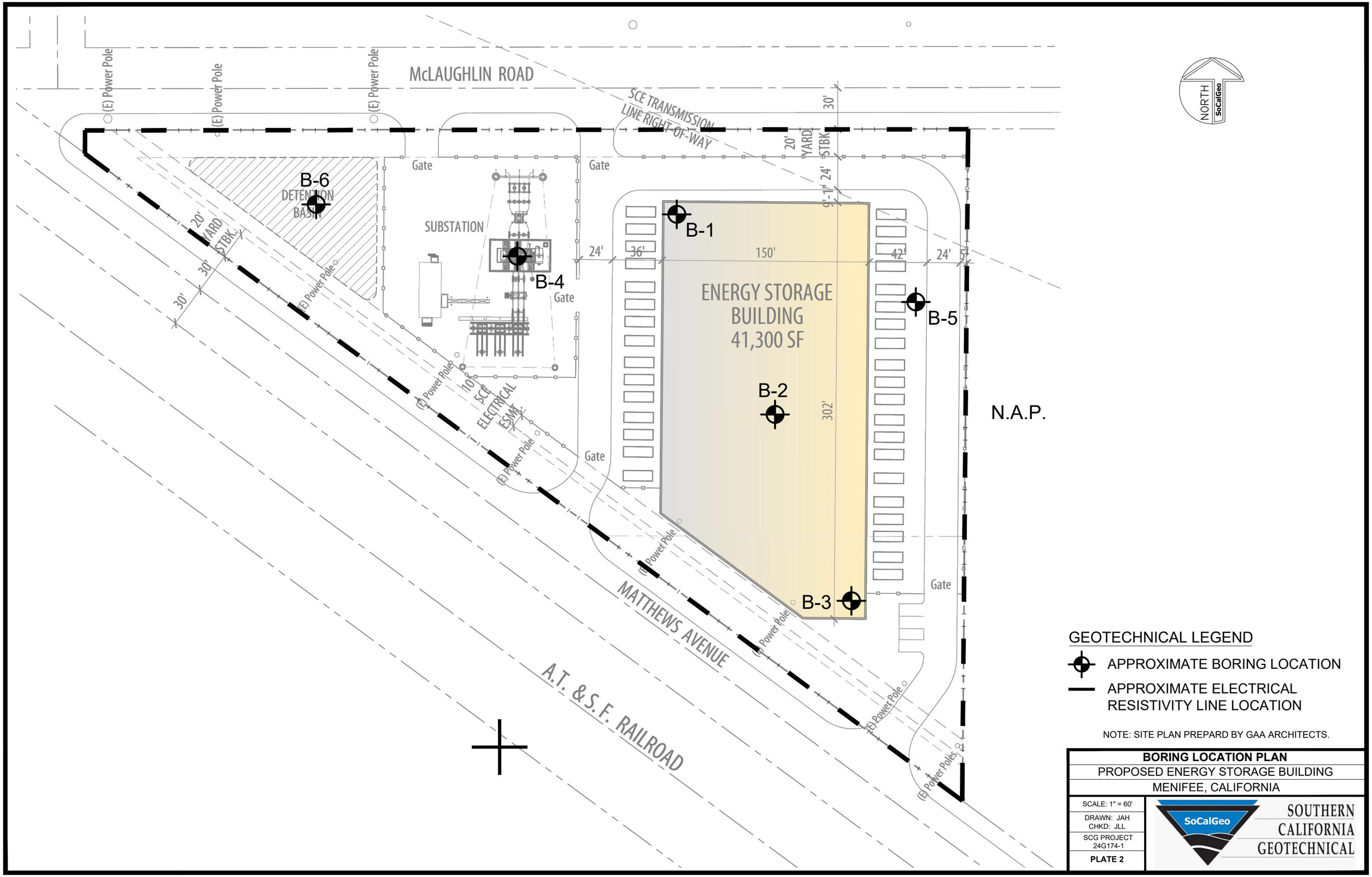
PROPOSED ENERGY STORAGE BUILDING

MENIFEE, CALIFORNIA

SCALE: 1" = 2000'	
DRAWN: JAH CHKD: JLL	
SCG PROJECT 24G174-1	
PLATE 1	

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- APPROXIMATE ELECTRICAL RESISTIVITY LINE LOCATION

NOTE: SITE PLAN PREPARD BY GAA ARCHITECTS.

BORING LOCATION PLAN

PROPOSED ENERGY STORAGE BUILDING

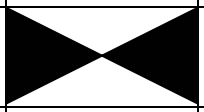
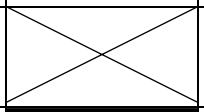
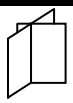
MENIFEE, CALIFORNIA

SCALE: 1" = 60'	
DRAWN: JAH CHKD: JLL	
SCG PROJECT 24G174-1	
PLATE 2	

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

SAMPLE TYPE	GRAPHICAL SYMBOL	SAMPLE DESCRIPTION
AUGER		SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED)
CORE		ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK.
GRAB		SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED)
CS		CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED)
NSR		NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL.
SPT		STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED)
SH		SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED)
VANE		VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED.

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.

MOISTURE CONTENT: Moisture content of a soil sample, expressed as a percentage of the dry weight.

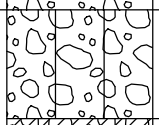
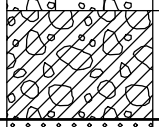
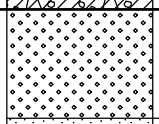
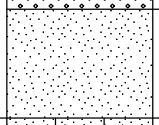
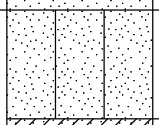
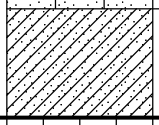
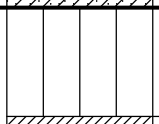
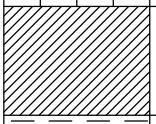
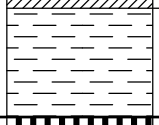
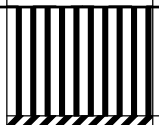
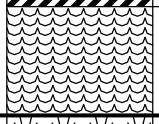
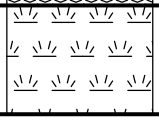
LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE: The percentage of the sample finer than the #200 standard sieve.

UNCONFINED SHEAR: The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, 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 OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 24G174-1
PROJECT: Proposed Energy Storage Building
LOCATION: Menifee, California

DRILLING DATE: 12/3/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 13 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1463.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					1± inch Open-graded gravel							
		53			FILL: Gray Brown Silty fine to coarse Sand, little fine to coarse Gravel, little Clay, dense-damp		5					EI = 17 @ 1 to 5 feet
		14			FILL: Brown Silty fine Sand, little Clay, loose-damp	108	6					
5		25			YOUNGER ALLUVIUM: Light Brown fine Sandy Silt, trace medium Sand, trace Calcareous nodules, medium dense-damp	104	6					
		71/11"			OLDER ALLUVIUM: Gray Brown fine to medium Sand, trace coarse Sand, little Silt, dense to very dense-damp to moist	111	9					
		67				122	7					
10												
		57					6					
15												
					Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, dense-moist to very moist							
		45					16					
20					Boring Terminated at 20 feet							

TBL 24G174-1.GPJ SOCALGEO.GDT 12/18/24

TEST BORING LOG **PLATE B-1**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 24G174-1
PROJECT: Proposed Energy Storage Building
LOCATION: Menifee, California

DRILLING DATE: 12/3/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1463 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					1± inch Open-graded gravel							
		14			FILL: Brown Silty fine to coarse Sand, trace fine Gravel, medium dense-damp		5					
		17			YOUNGER ALLUVIUM: Light Brown to Brown Silty fine Sand, trace Calcareous nodules, slightly porous, medium dense-damp to moist		8					
5		21					7					
		54			OLDER ALLUVIUM: Gray Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, very dense-moist to very moist		15					
10												
					Red Brown Silty fine Sand, trace medium Sand, very dense-moist to very moist							
		55					13					
15					Boring Terminated at 15 feet							

TBL 24G174-1.GPJ SOCALGEO.GDT 12/18/24

TEST BORING LOG

PLATE B-2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 24G174-1
PROJECT: Proposed Energy Storage Building
LOCATION: Menifee, California

DRILLING DATE: 12/3/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 22 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1462 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					1± inch Open-graded gravel							
		24			FILL: Gray Brown Silty fine Sand, little medium to coarse Sand, little fine Gravel, mottled, medium dense-damp to moist	110	7					
		31			YOUNGER ALLUVIUM: Brown Clayey fine Sand, little medium Sand, little Silt, little to some Calcareous nodules, medium dense-moist	97	15					
5		18				94	18					
		37			YOUNGER ALLUVIUM: Brown Silty fine Sand, trace medium Sand, medium dense-moist	97	10					
		71/10"			OLDER ALLUVIUM: Brown Silty fine to medium Sand, trace coarse Sand, very dense-damp to moist	128	7					
10												
					Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, dense-moist to very moist							
		40					12					
15												
		37					15					
20												
					Gray Brown fine to coarse Sand, little Silt, dense-very moist							
		46					12					
					Brown fine Sandy Silt, trace medium Sand, dense-very moist		18					
25												
					Boring Terminated at 25 feet							

TBL 24G174-1.GPJ SOCALGEO.GDT 12/18/24

TEST BORING LOG

PLATE B-3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 24G174-1
PROJECT: Proposed Energy Storage Building
LOCATION: Menifee, California

DRILLING DATE: 12/3/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: 32 feet
CAVE DEPTH: 28.5 feet
READING TAKEN: 5 hrs After Completion

SURFACE ELEVATION: 1462 feet MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					1± inch Open-graded gravel							
		34			FILL: Gray Brown Silty fine to coarse Sand, little fine Gravel, medium dense-damp	119	4					
		12			YOUNGER ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, loose-damp	109	6					
5		55			YOUNGER ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, little Calcareous nodules, medium dense to dense-damp to moist	109	8					
		48				116	6					
		30				110	7					
10												
					OLDER ALLUVIUM: Red Brown Silty fine Sand, trace medium to coarse Sand, very dense-damp to moist							
		72					8					
15												
					Red Brown fine Sandy Silt, trace medium to coarse Sand, dense to very dense-damp to very moist							
		46					15					
20												
		73/11"					16					
25												
		61					7					
30												
					Red Brown fine to coarse Sand, little Silt, very dense-wet							Perched Water Encountered at 32± feet
		70/11"					13					
					Red Brown Silty fine Sand to fine Sandy Silt, trace medium Sand,		20					

TBL 24G174-1.GPJ SOCALGEO.GDT 12/18/24

TEST BORING LOG

PLATE B-4a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 24G174-1
PROJECT: Proposed Energy Storage Building
LOCATION: Menifee, California

DRILLING DATE: 12/3/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: 32 feet
CAVE DEPTH: 28.5 feet
READING TAKEN: 5 hrs After Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					very dense-very moist							
					Red Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, very dense-very moist							
40		78/10"					14					
45		76/11"					15					
50		57					20					
					Boring Terminated at 50 feet							

TBL 24G174-1.GPJ SOCALGEO.GDT 12/18/24

TEST BORING LOG

PLATE B-4b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 24G174-1
PROJECT: Proposed Energy Storage Building
LOCATION: Menifee, California

DRILLING DATE: 12/3/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 3 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1463.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					1± inch Open-graded gravel							
		51			FILL: Gray Brown Silty fine to coarse Sand, little fine Gravel, mottled, very dense-damp		5					
		19			YOUNGER ALLUVIUM: Light Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, some Calcareous nodules, medium dense-damp		10					
5					Boring Terminated at 5 feet							

TBL 24G174-1.GPJ SOCALGEO.GDT 12/18/24

TEST BORING LOG

PLATE B-5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 24G174-1
PROJECT: Proposed Energy Storage Building
LOCATION: Menifee, California

DRILLING DATE: 12/3/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1461 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					1± inch Open-graded gravel							
					YOUNGER ALLUVIUM: Brown Silty fine to coarse Sand, medium dense to very dense-damp							
		25					6					
5												
		50					6					
					OLDER ALLUVIUM: Brown to Gray Brown Silty fine Sand, trace to little medium Sand, very dense-damp to moist		10					
10												
		78					9					
15												
					Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, very dense-moist							
		80/10"					10					
20												
					Boring Terminated at 20 feet							

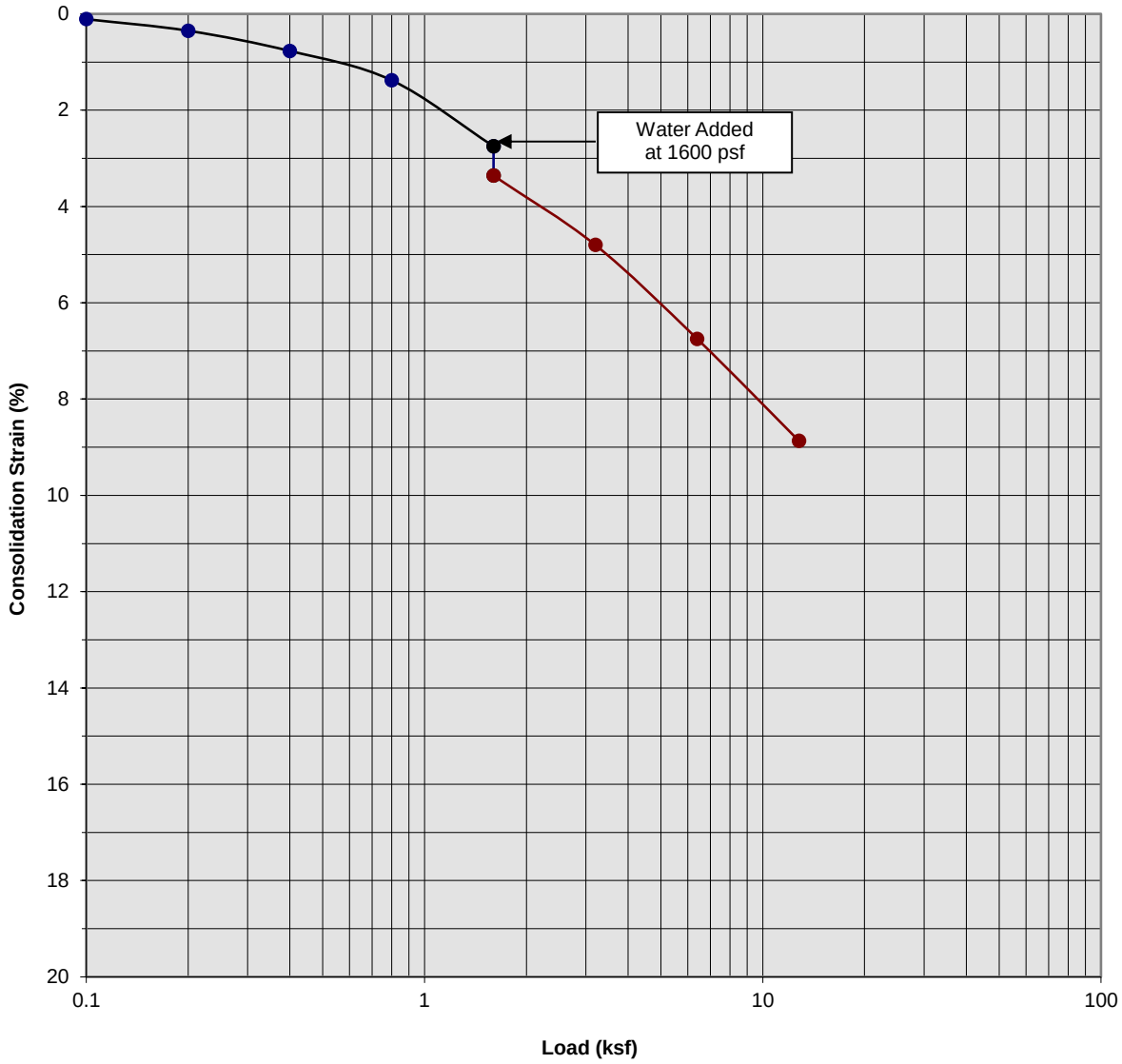
TBL 24G174-1.GPJ SOCALGEO.GDT 12/18/24

TEST BORING LOG

PLATE B-6

APPENDIX C

Consolidation/Collapse Test Results



Classification: Brown Clayey fine Sand, little medium Sand, little Silt

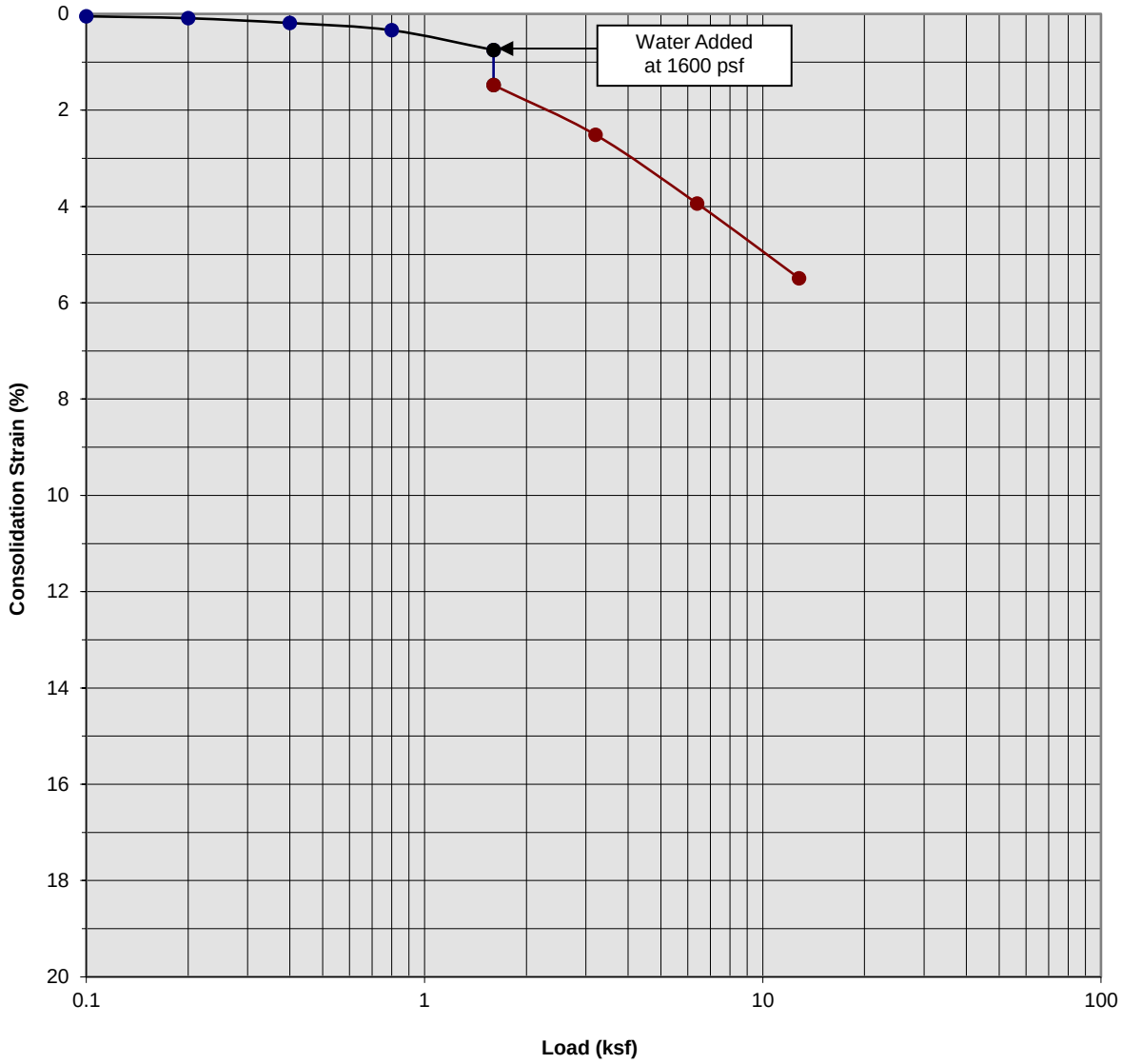
Boring Number:	B-3	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	5 to 6	Initial Dry Density (pcf)	93.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	101.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.61

Proposed Energy Storage Building
Menifee, California
Project No. 24G174-1

PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty fine to medium Sand, trace coarse Sand

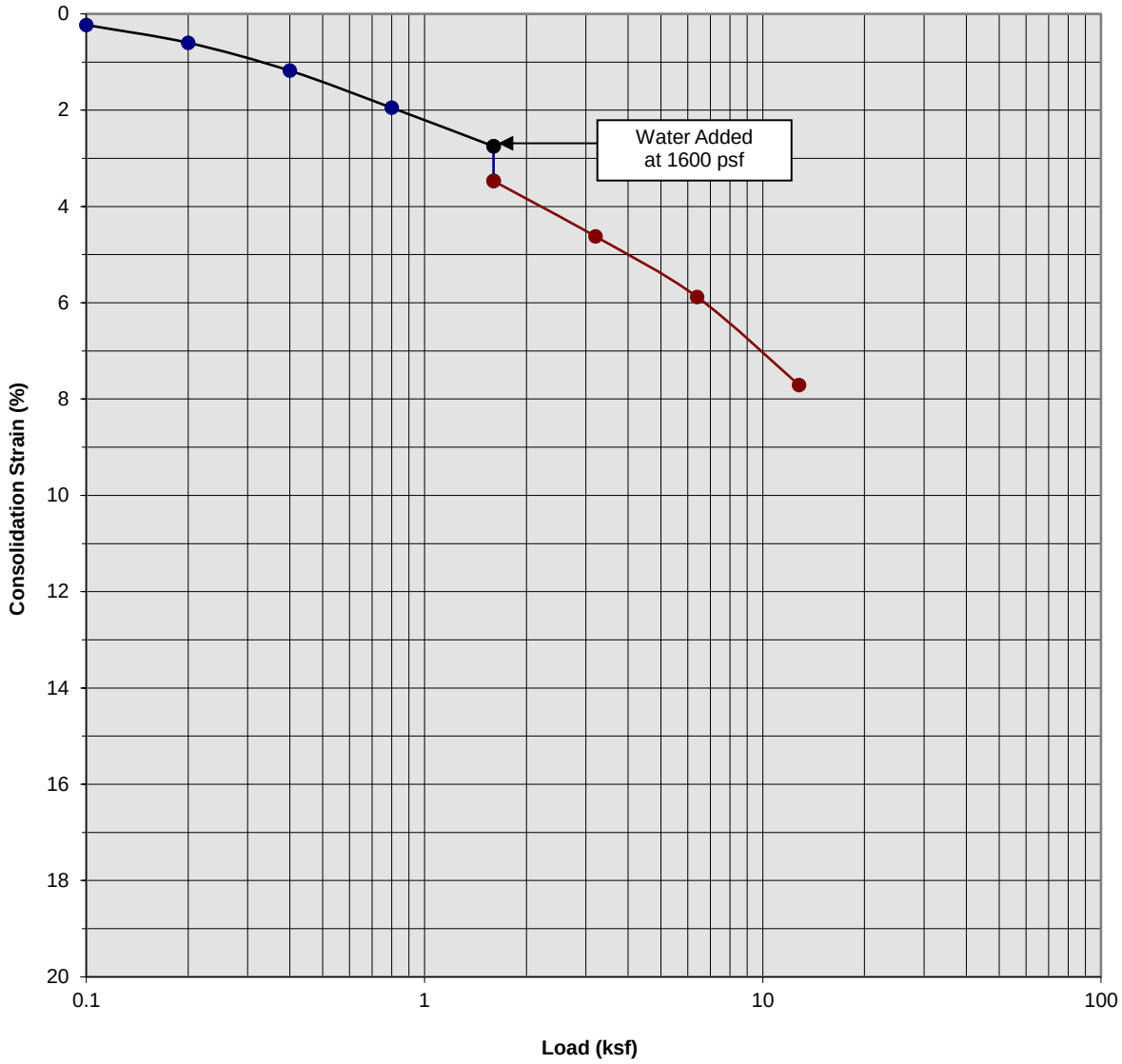
Boring Number:	B-3	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	10
Depth (ft)	9 to 10	Initial Dry Density (pcf)	127.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	134.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.73

Proposed Energy Storage Building
Menifee, California
Project No. 24G174-1

PLATE C- 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty fine Sand, trace medium to coarse Sand

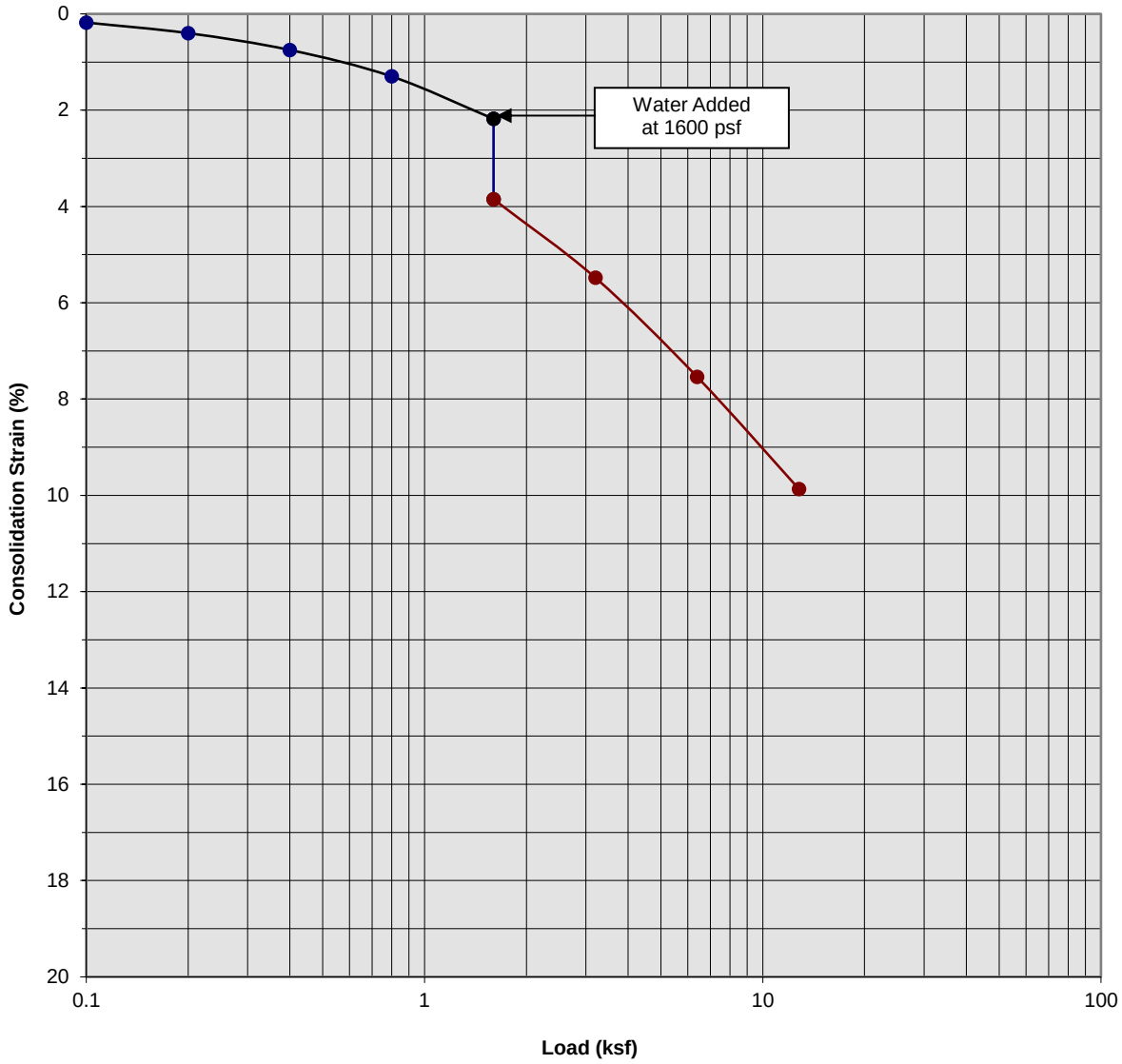
Boring Number:	B-4	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	3 to 4	Initial Dry Density (pcf)	109.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	118.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.72

Proposed Energy Storage Building
Menifee, California
Project No. 24G174-1

PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty fine Sand, trace medium to coarse Sand

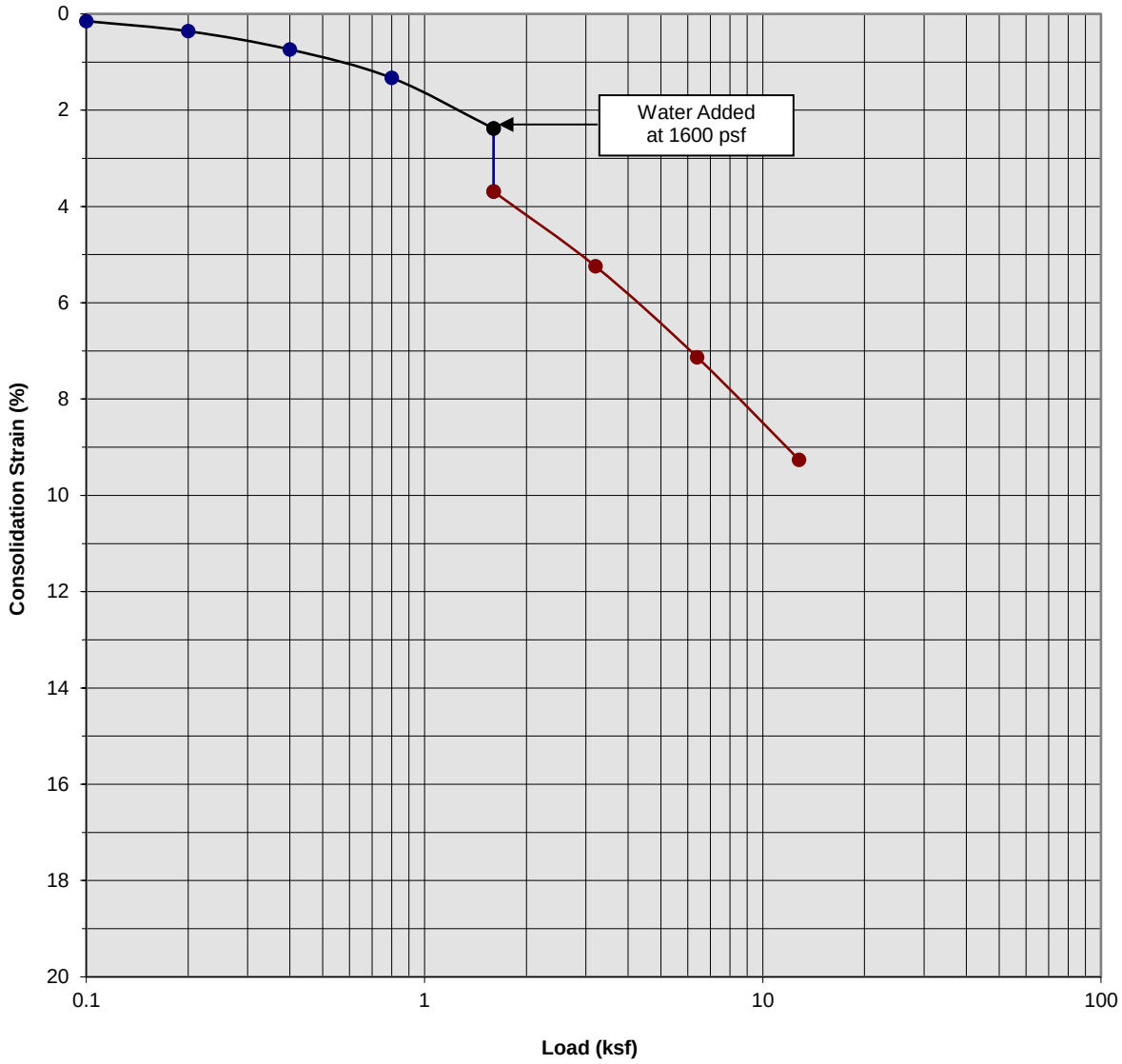
Boring Number:	B-4	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	5 to 6	Initial Dry Density (pcf)	108.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.67

Proposed Energy Storage Building
Menifee, California
Project No. 24G174-1

PLATE C- 4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty fine Sand, trace medium to coarse Sand

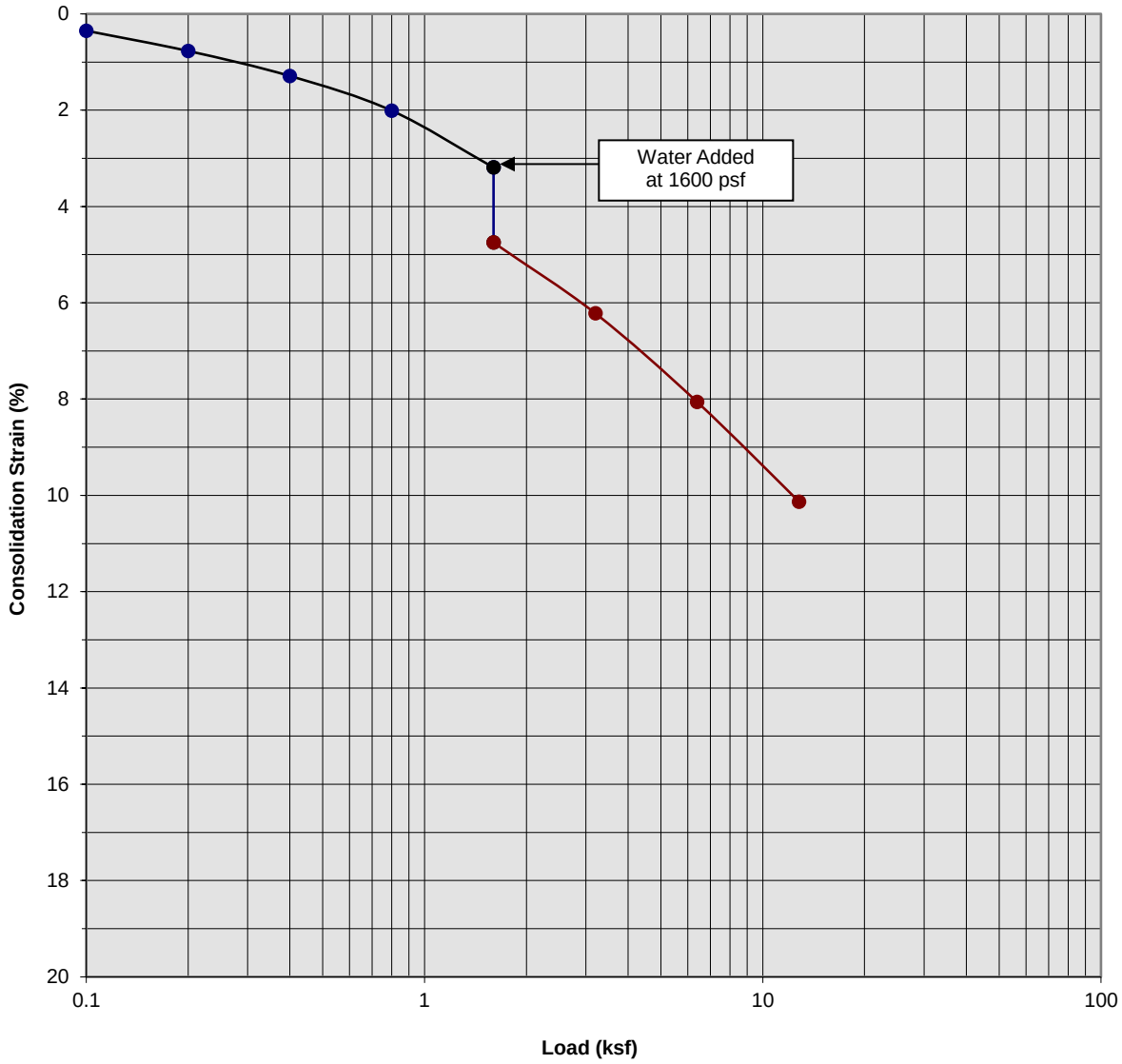
Boring Number:	B-4	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	7 to 8	Initial Dry Density (pcf)	116.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	127.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.31

Proposed Energy Storage Building
Menifee, California
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PLATE C- 5

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty fine Sand, trace medium to coarse Sand

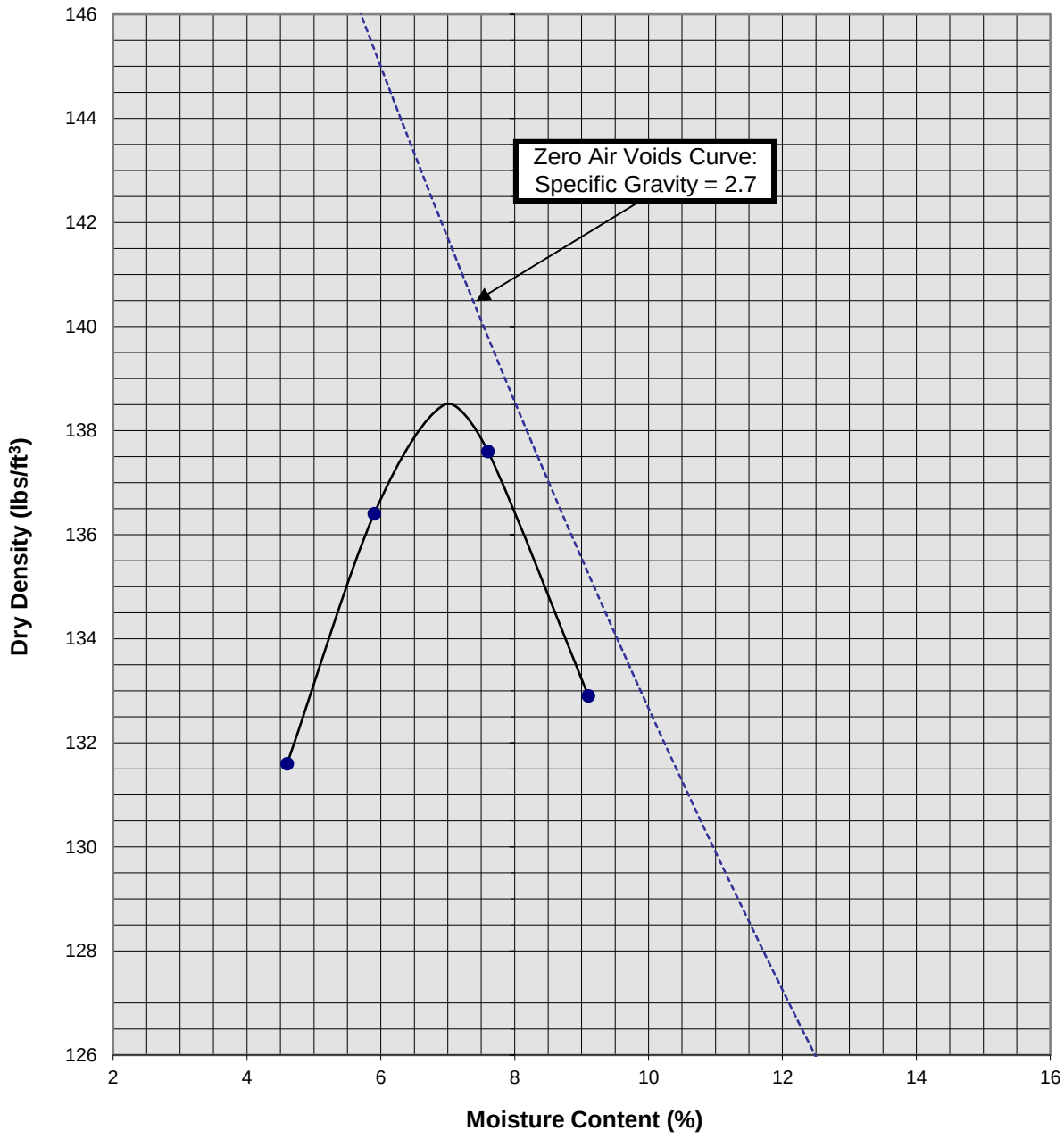
Boring Number:	B-4	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	9 to 10	Initial Dry Density (pcf)	111.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.56

Proposed Energy Storage Building
Menifee, California
Project No. 24G174-1

PLATE C- 6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Moisture/Density Relationship
ASTM D-1557



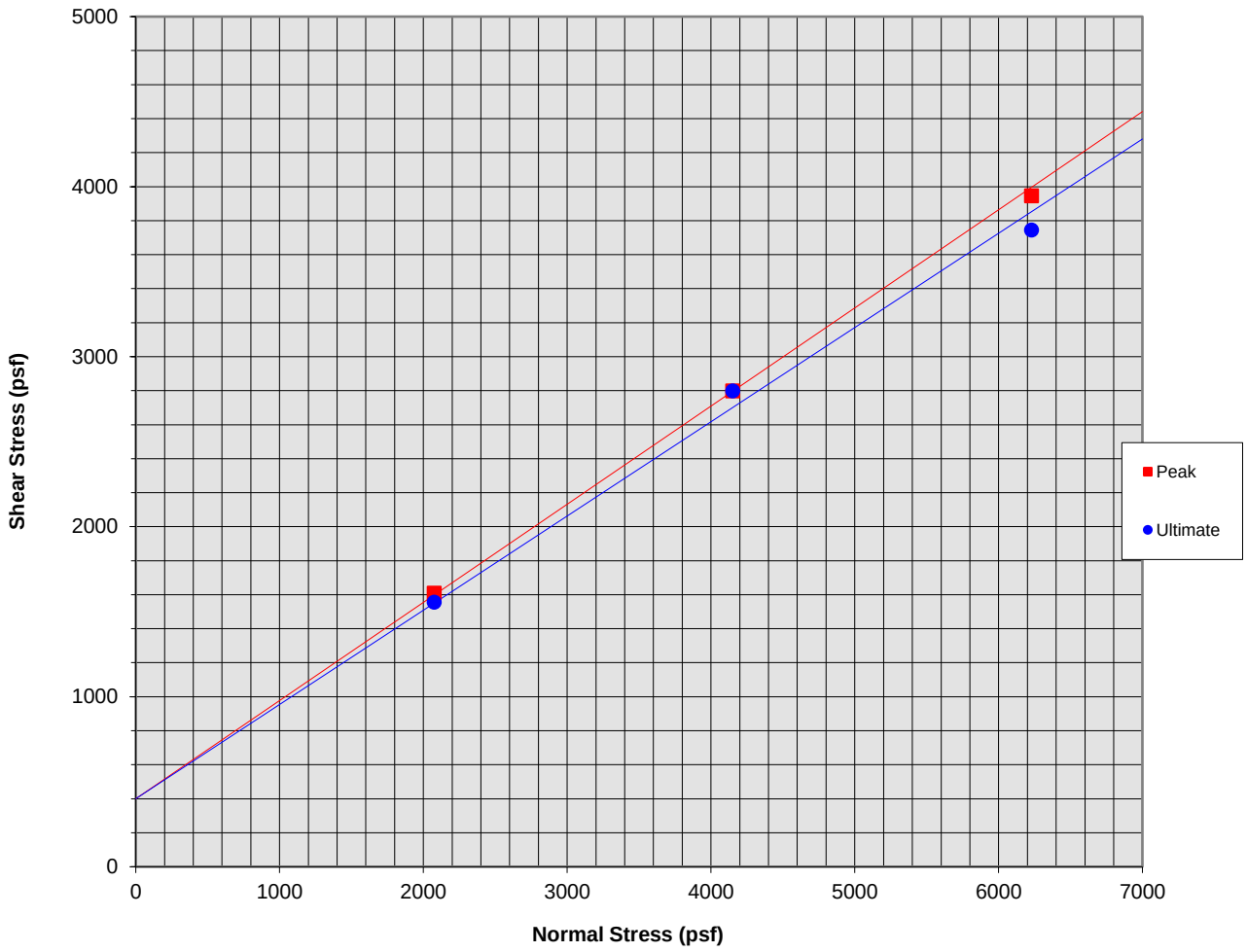
Soil ID Number	B-1 @ 1-5'
Optimum Moisture (%)	7
Maximum Dry Density (pcf)	138.5
Soil Classification	Brown Silty fine to medium Sand, little Clay, trace coarse Sand trace fine to coarse Gravel

Proposed Energy Storage Building
Menifee, California
Project No. 24G174-1

PLATE C- 7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Direct Shear Test Results
(Undisturbed)



Sample Description: B-1 @ 5 to 6 feet

Classification: Light Brown fine Sandy Silt, trace medium Sand

Sample Data		Test Results		
Initial Moisture Content	6.0			
Final Moisture Content	19.0		Peak	Ultimate
Initial Dry Density	104.0	ϕ (°)	30.0	29.0
Final Dry Density	--	C (psf)	400	400
Specimen Diameter (in)	2.4			
Specimen Thickness (in)	1.0			

Proposed Energy Storage Building
Menifee, California
Project No. 24G174-1

PLATE C- 8

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.

- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

Compacted Fills

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 20. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

    - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

    - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or

concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

- Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

- To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

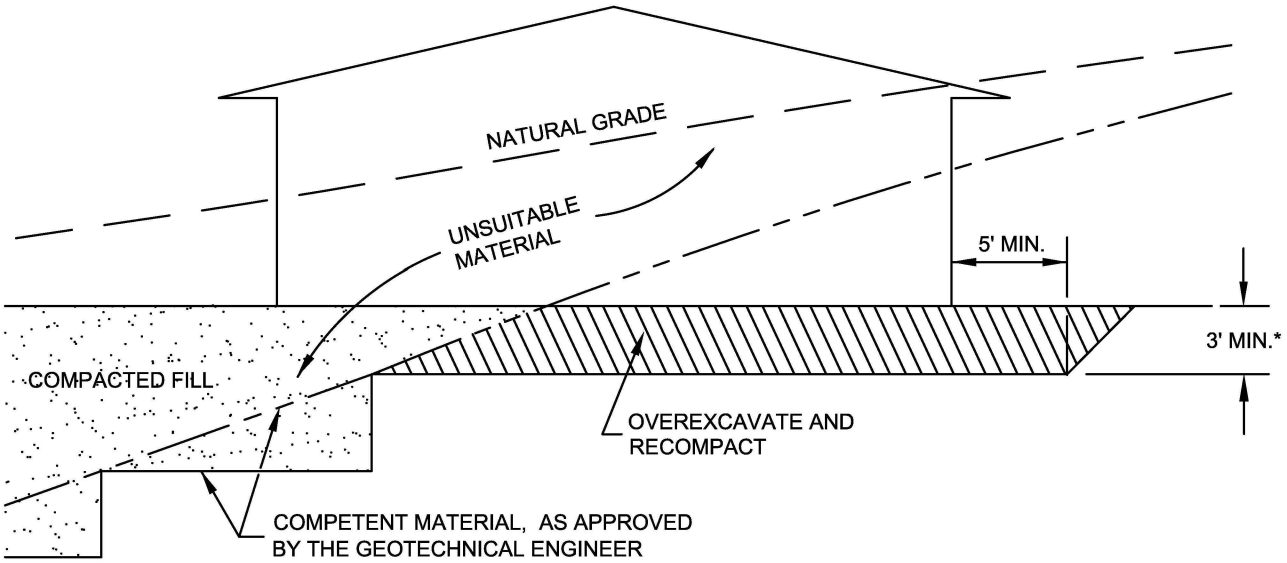
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

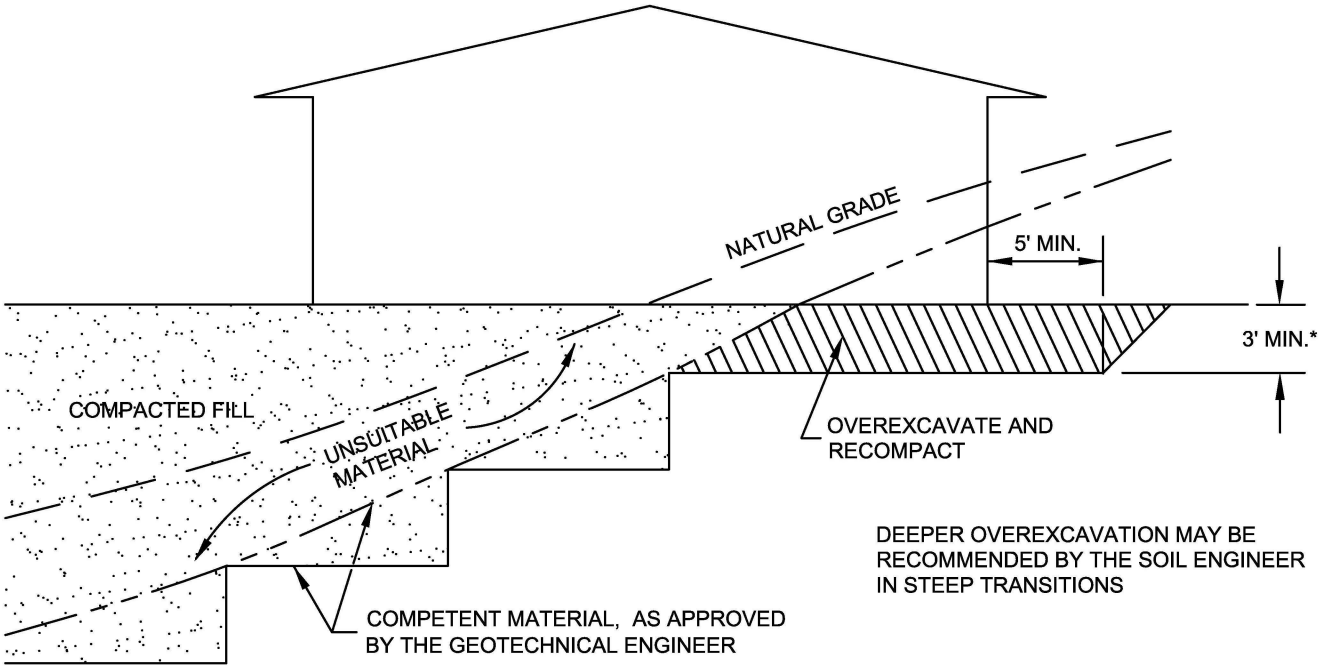
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT



CUT/FILL LOT (TRANSITION)



*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION. ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

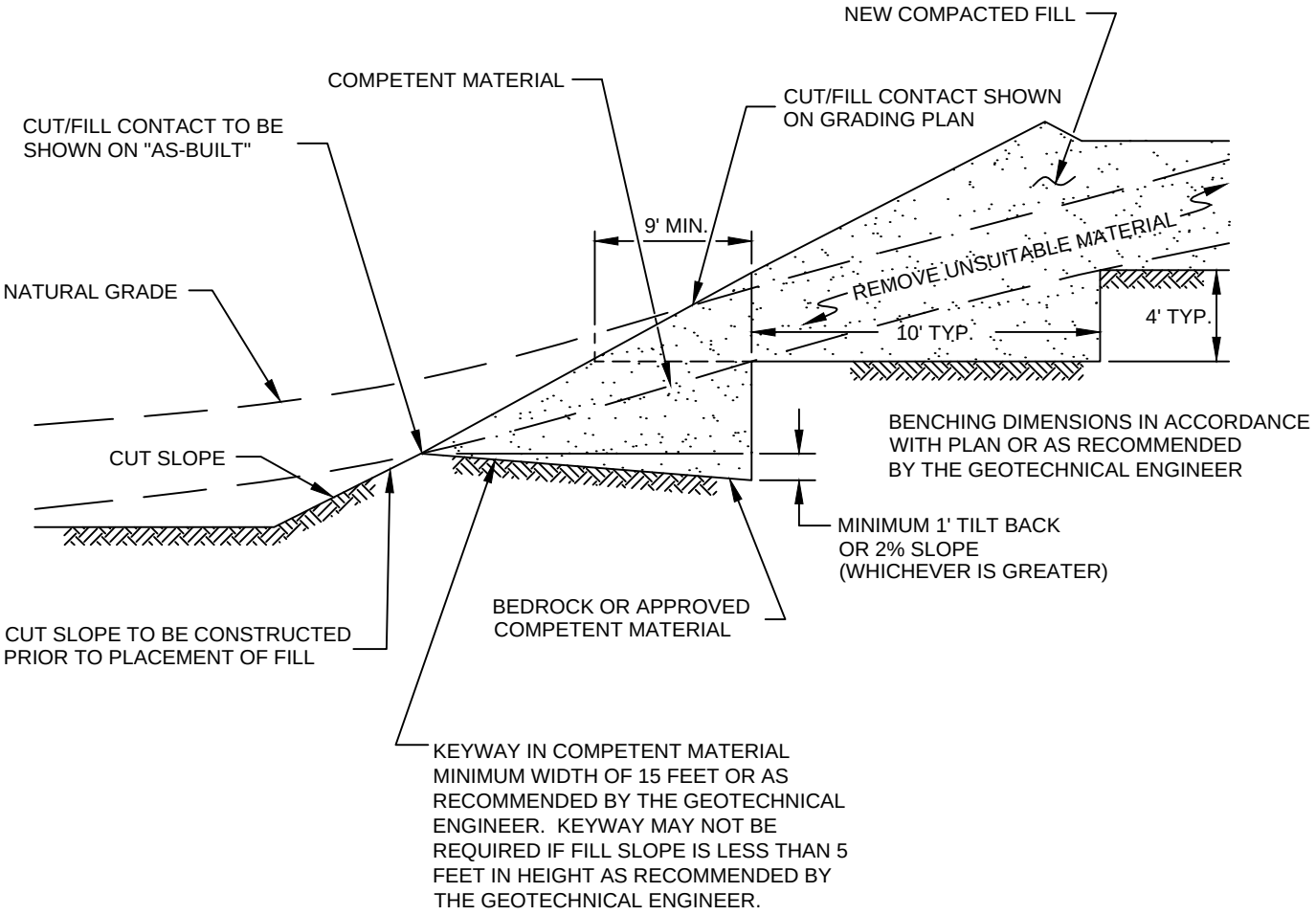
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

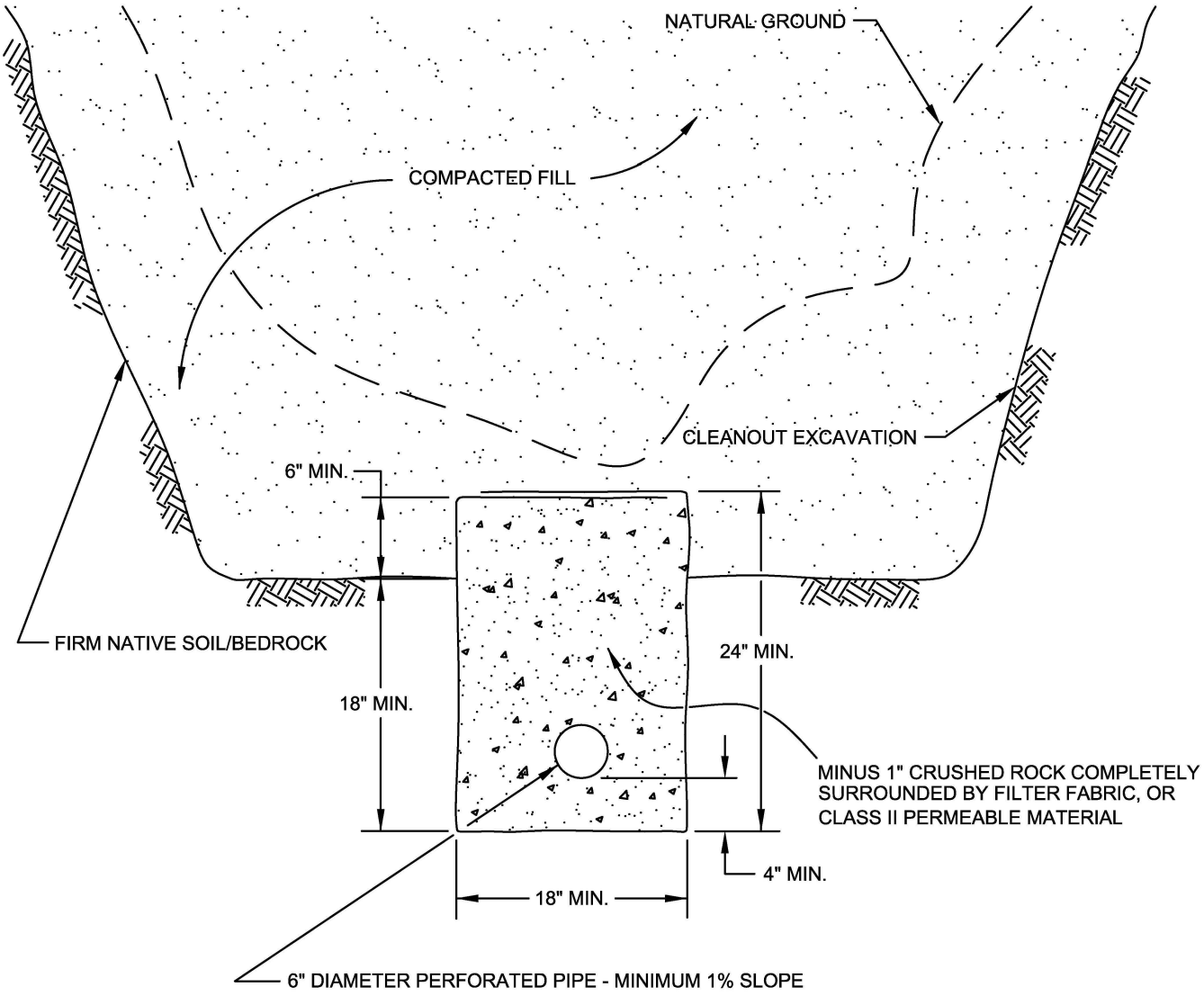
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

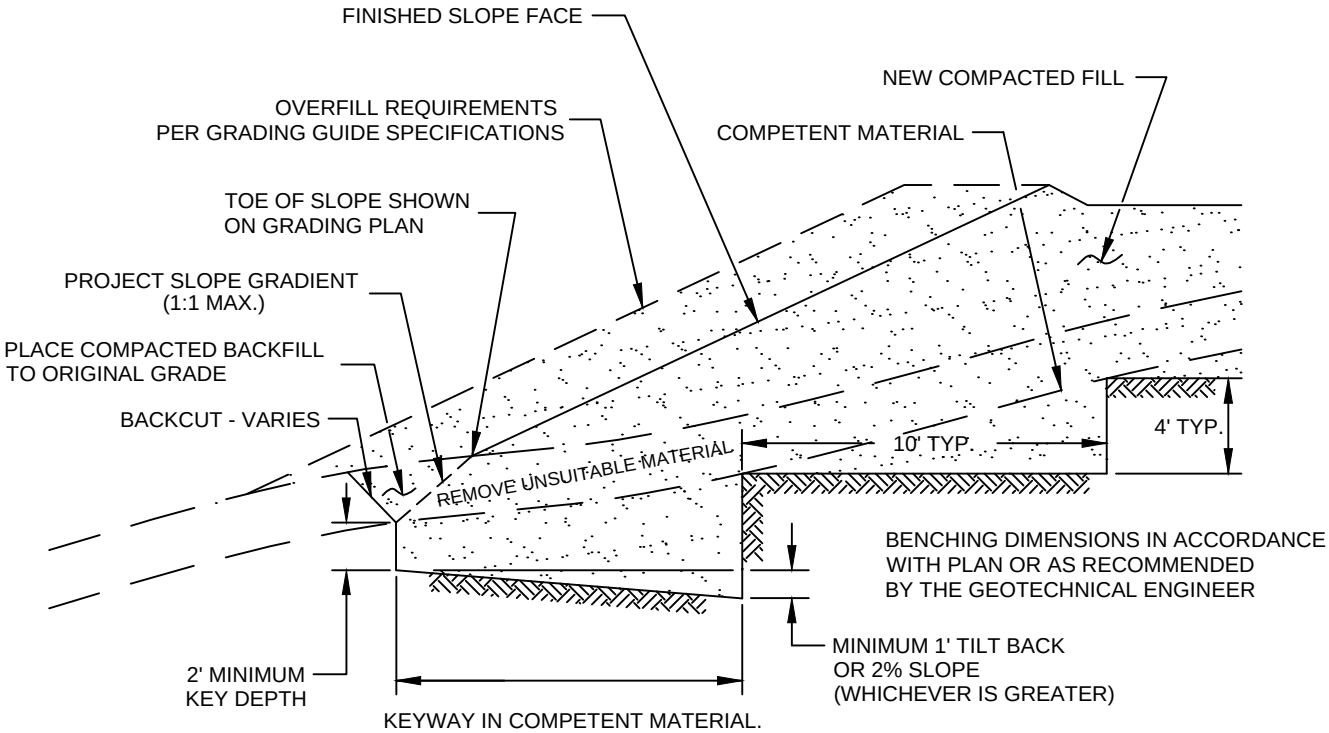
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



NOTE:
BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL

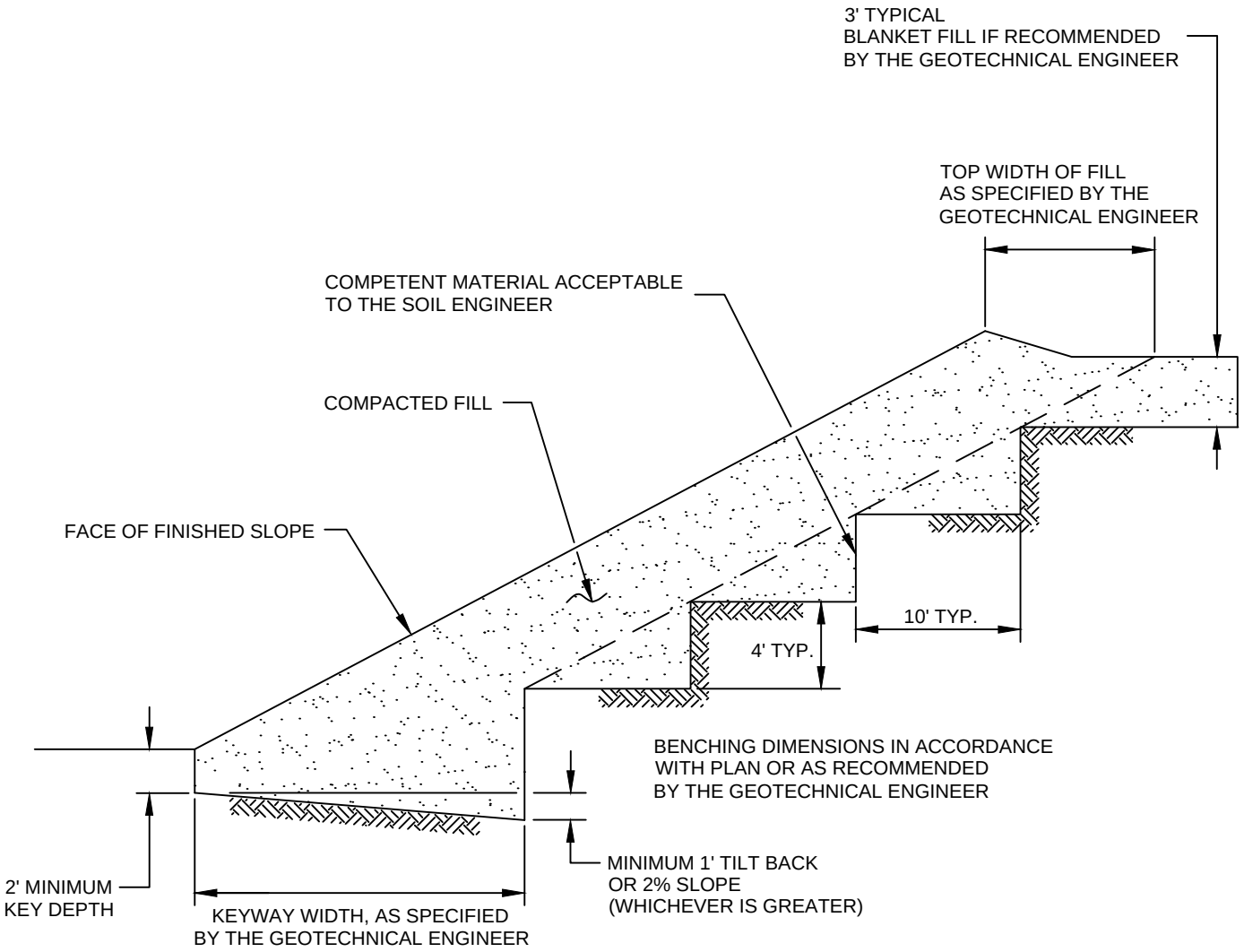
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

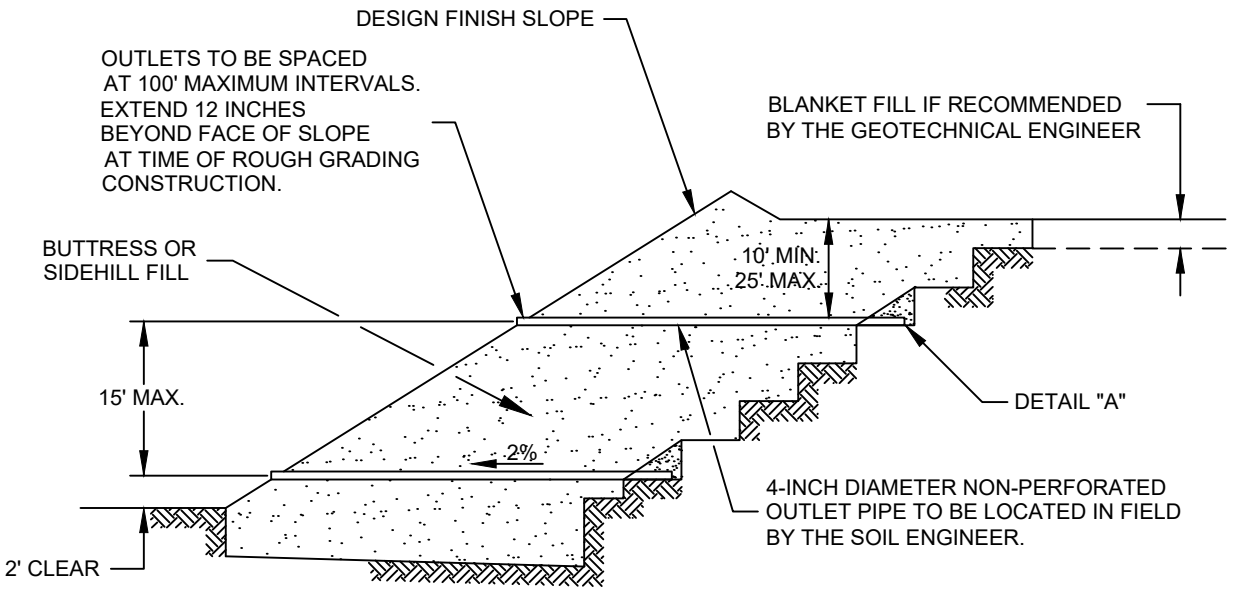
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



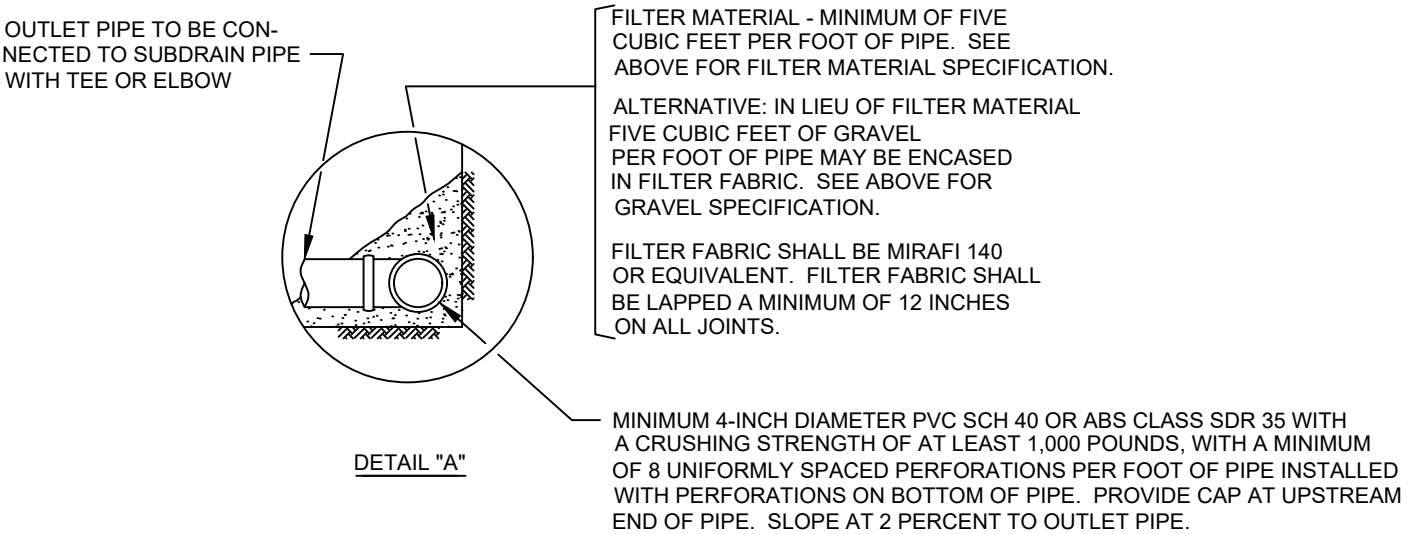
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

SLOPE FILL SUBDRAINS

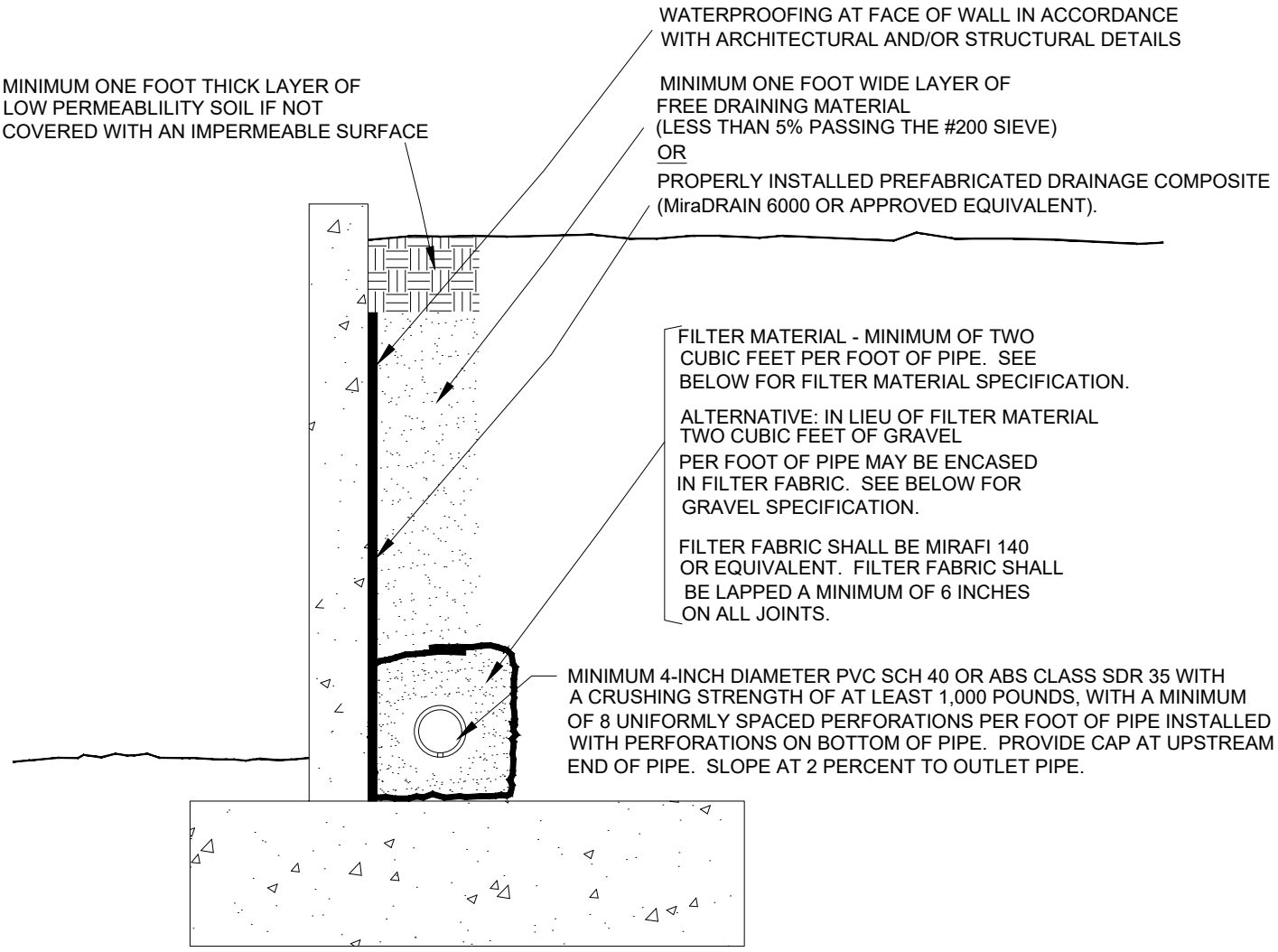
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

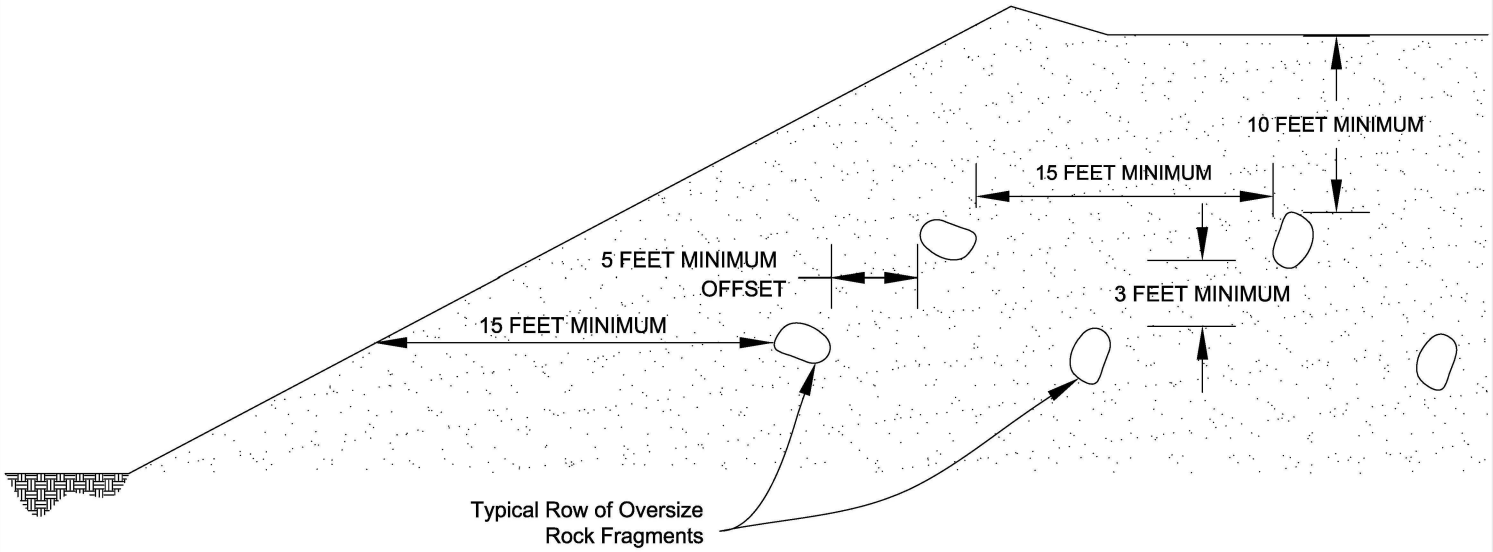
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

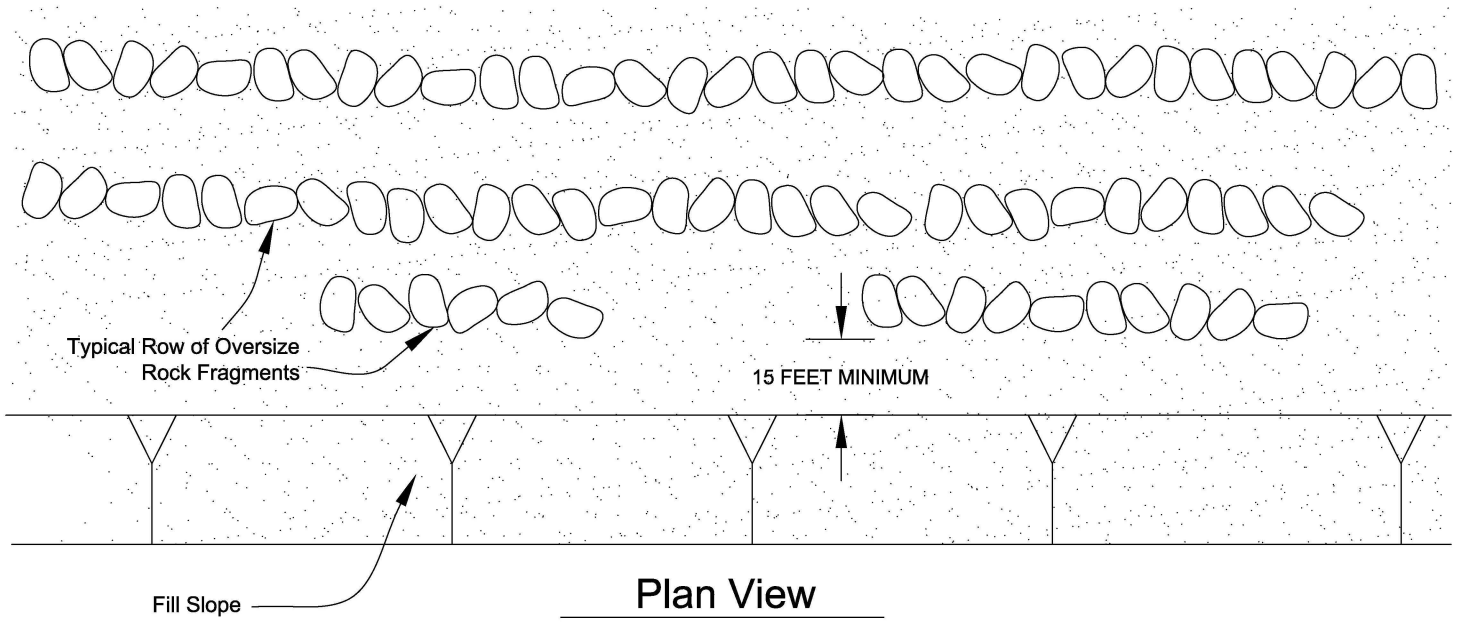
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

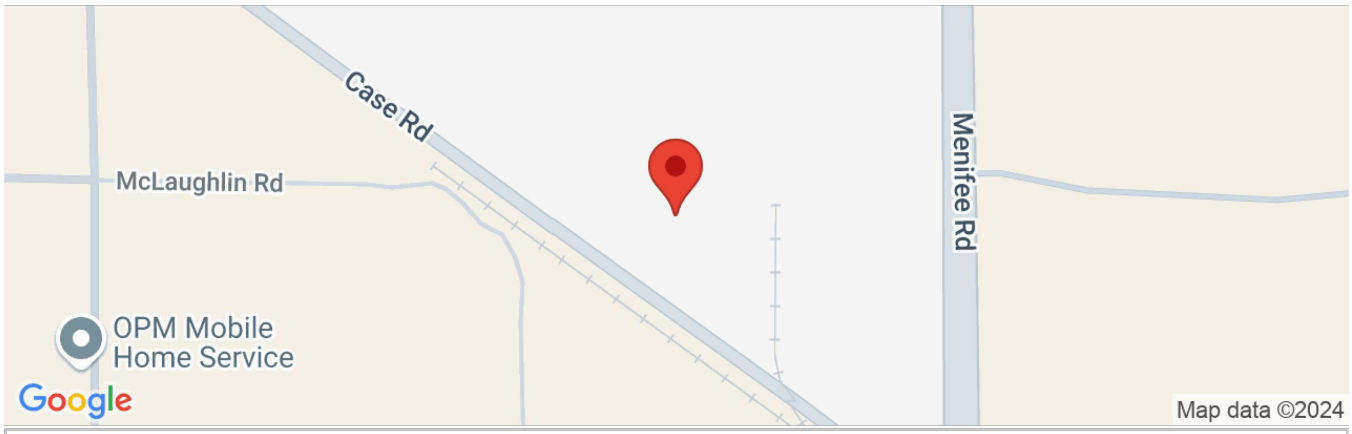
PLATE D-8

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

Latitude, Longitude: 33.735557, -117.156947



Date	12/11/2024, 3:33:41 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.423	MCE_R ground motion. (for 0.2 second period)
S_1	0.53	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.708	Site-modified spectral acceleration value
S_{M1}	0.779	Site-modified spectral acceleration value
S_{DS}	1.138	Numeric seismic design value at 0.2 second SA
S_{D1}	0.519	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.47	Site amplification factor at 1.0 second
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.6	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.423	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.526	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.53	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.579	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.606	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.932	Mapped value of the risk coefficient at short periods
C_{R1}	0.915	Mapped value of the risk coefficient at a period of 1 s
C_V	1.185	Vertical coefficient

SOURCE: SEAOC/OSHPD Seismic Design Maps Tool
<https://seismicmaps.org/>

SEISMIC DESIGN PARAMETERS - 2022 CBC
PROPOSED ENERGY STORAGE BUILDING
MENIFEE, CALIFORNIA

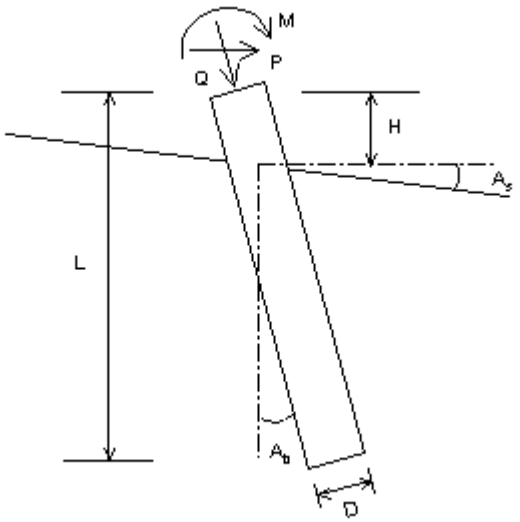
DRAWN: JLL
CHKD: RGT
SCG PROJECT 24G174-1
PLATE E-1

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

VERTICAL ANALYSIS

Figure 1



Drilled Pile (dia <=24 in. or 61 cm)

Loads:
Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

(with Load Factor)
Vertical Load, Q= 5.0 -kp

Profile:
Pile Length, L= 20.0 -ft
Top Height, H= -1.5 -ft
Slope Angle, As= 0
Batter Angle, Ab= 0

* Zero Friction *
Zero Friction Start: 0 -ft End: 2.5 -ft

Soil Data:

Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K -lb/i3	e50 or Dr %	Nspt
0	119	30	0.00	77.5	45.73	15
4.5	120	29.0	0.40	1031.5	0.51	24
12	130	32.0	0.10	2906.0	0.29	60

Pile Data:

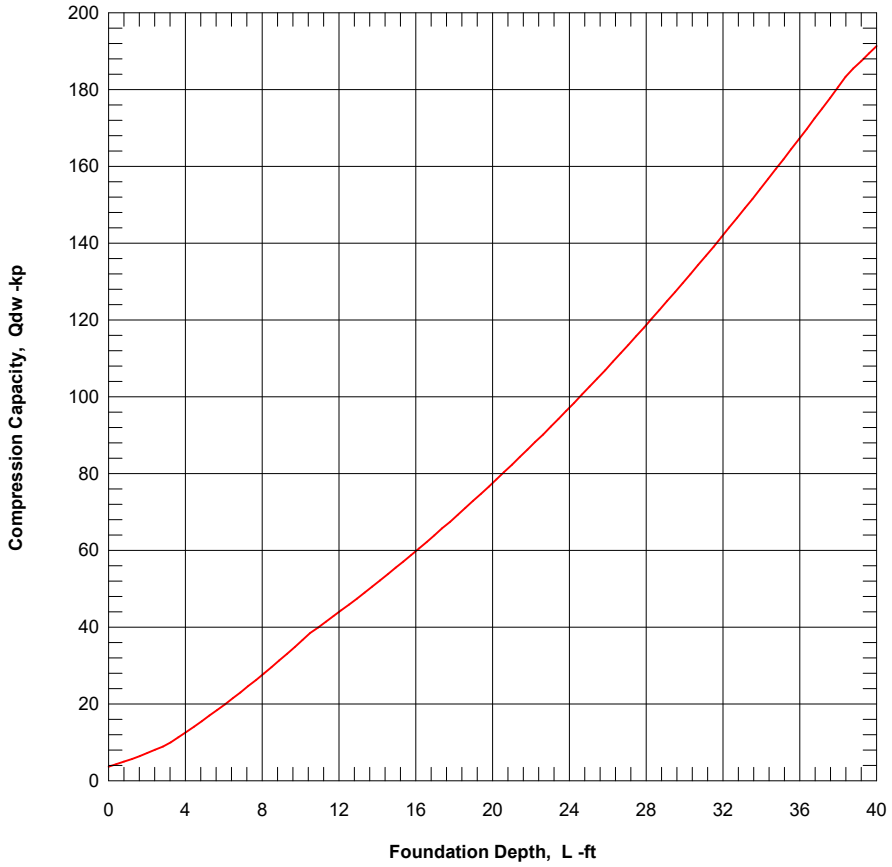
Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	24	452.4	75.4	16286.0	3000	0.471
20.0						

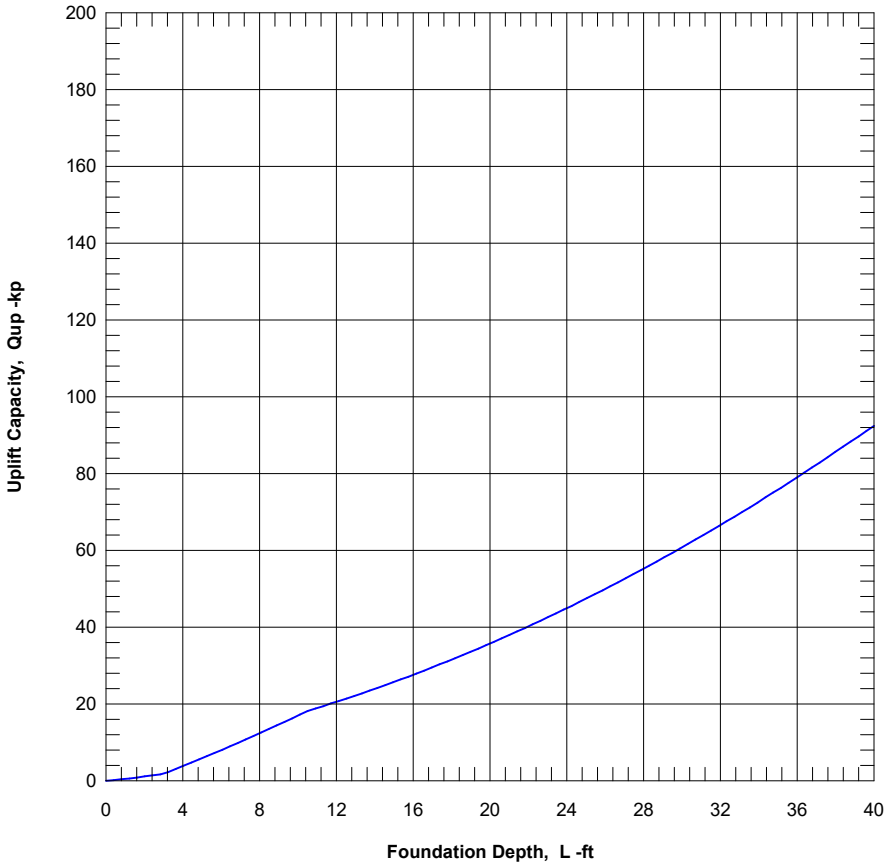
Vertical Capacity:
Weight above Ground= 0.00 Total Weight= 9.42-kp *Soil Weight is not included
Side Resistance (Down)= 73.553-kp Side Resistance (Up)= 52.658-kp
Tip Resistance (Down)= 122.288-kp Tip Resistance (Up)= 0.000-kp
Total Ultimate Capacity (Down) Qult= 195.841-kp Total Ultimate Capacity (Up)= 62.074-kp
Total Allowable Capacity (Down) Qallow= 77.539-kp Total Allowable Capacity (Up) Qallow= 35.745-kp
OK! Qallow > Q

Settlement Calculation:
At Q= 5.00-kp Settlement= 0.00491-in
At Xallow= 0.50-in Q= 146.78932-kp

Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.

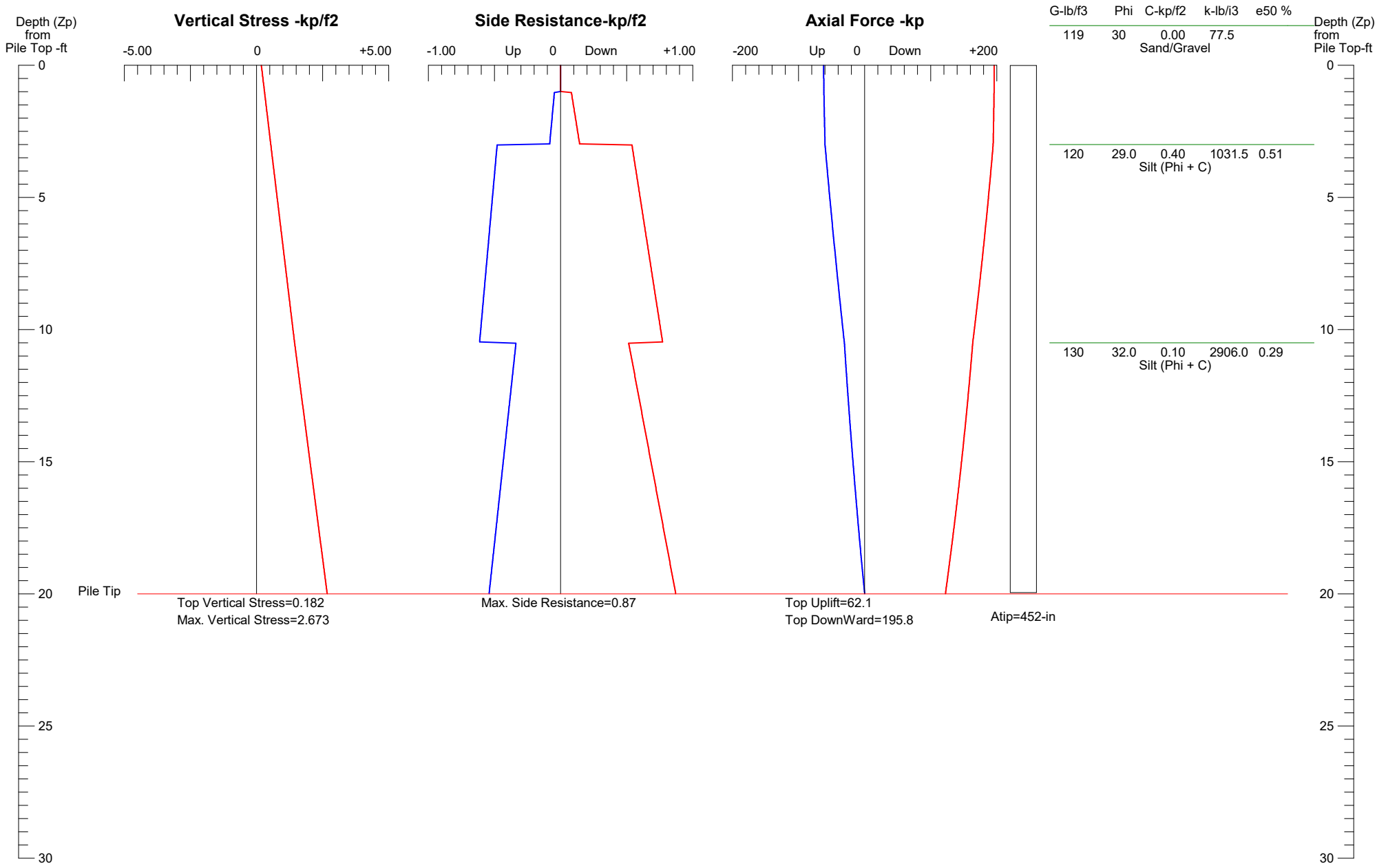
ALLOWABLE CAPACITY vs FOUNDATION DEPTH





SOIL STRESS, SIDE RESISTANCE, & AXIAL FORCE vs DEPTH

Based on Ultimate Load Condition

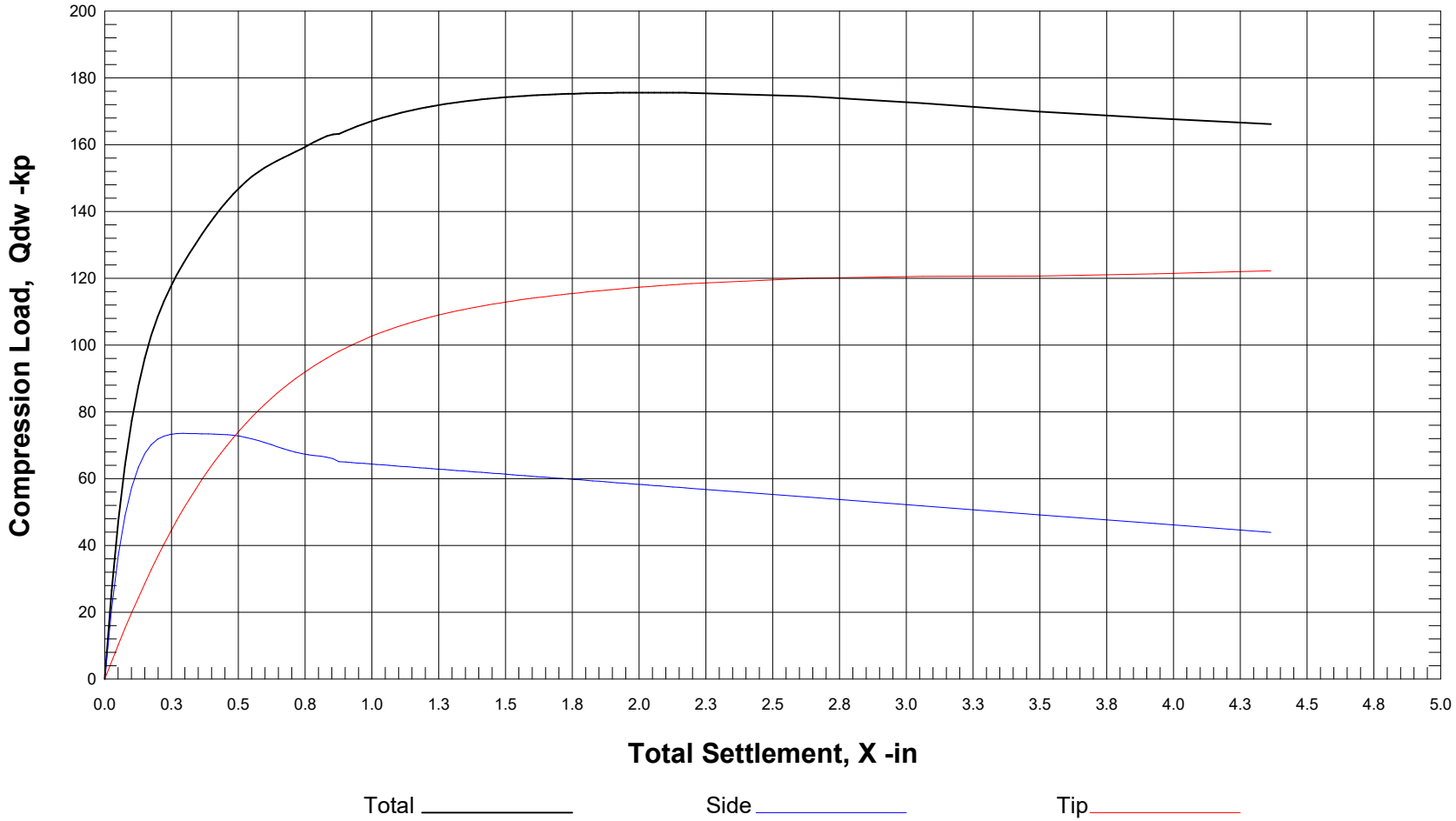


CivilTech Software

24G174-1 PNG Builders - Substation
24 in dia, 20-ft deep

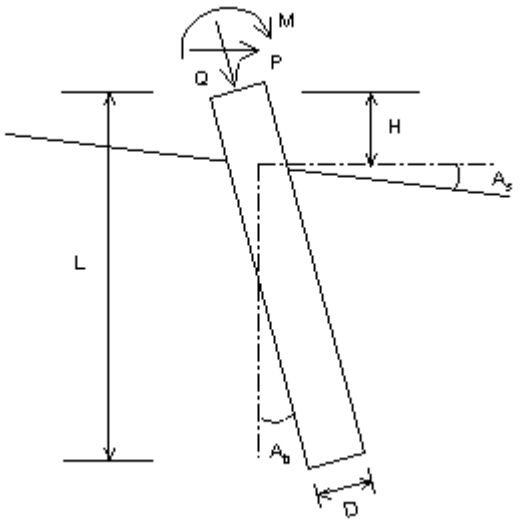
Figure 1

Vertical Load vs. Total Settlement



LATERAL ANALYSIS

Figure 2



Drilled Pile (dia <=24 in. or 61 cm)

Loads:
Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

(with Load Factor)
Vertical Load, Q= 5.0 -kp
Shear Load, P= 5.0 -kp
Moment, M= 300.0 -kp-f

Profile:
Pile Length, L= 20.0 -ft
Top Height, H= -1.5 -ft
Slope Angle, As= 0
Batter Angle, Ab= 0

* Zero Friction *
Zero Friction Start: 0 -ft End: 2.5 -ft

Soil Data:

Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K -lb/i3	e50 or Dr %	Nspt
0	119	30	0.00	77.5	45.73	15
4.5	120	29.0	0.40	1031.5	0.51	24
12	130	32.0	0.10	2906.0	0.29	60

Pile Data:

Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	24	452.4	75.4	16286.0	3000	0.471
20.0						

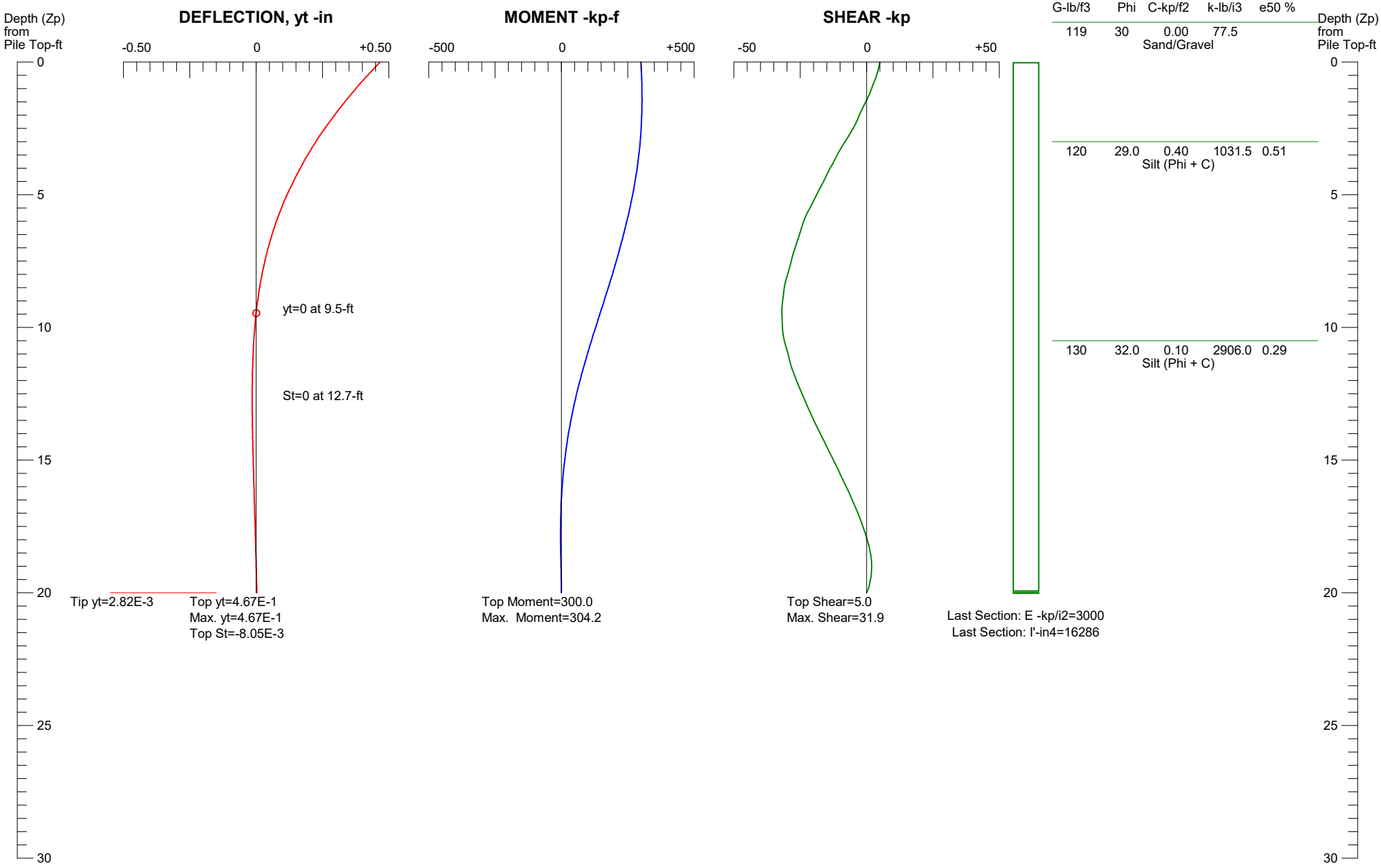
Single Pile Lateral Analysis:
Top Deflection, yt= 0.46700-in
Max. Moment, M= 304.17-kp-f
Top Deflection Slope, St= -0.00805
OK! Top Deflection, 0.4670-in is less than the Allowable Deflection= 0.50-in

Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.
The Max. Moment calculated by program is an internal force from the applied load conditions. Structural engineer has to check whether the pile has enough capacity to resist the moment with adequate factor of safety. If not, the pile may fail under the load conditions.

PILE DEFLECTION & FORCE vs DEPTH

Pile below Ground (not to scale)

Single Pile, Khead=1, Kbc=1

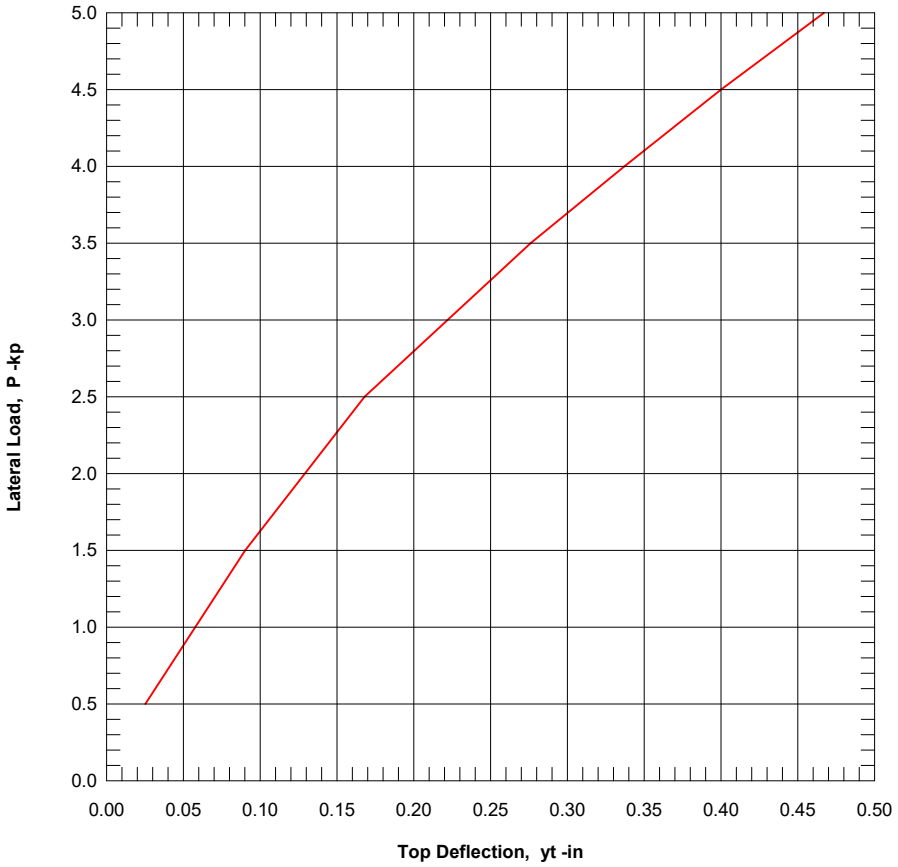


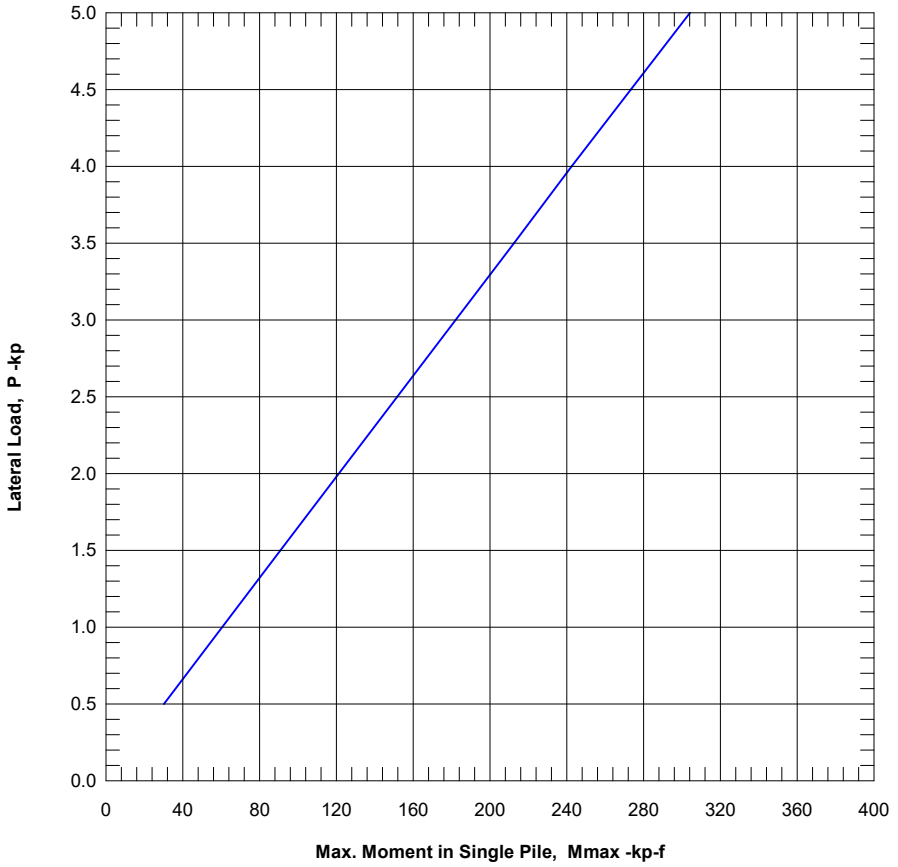
CivilTech Software

24G174-1 PNG Builders - Substation
24 in dia, 20-ft deep

Figure 2

LATERAL LOAD vs DEFLECTION & MAX. MOMENT

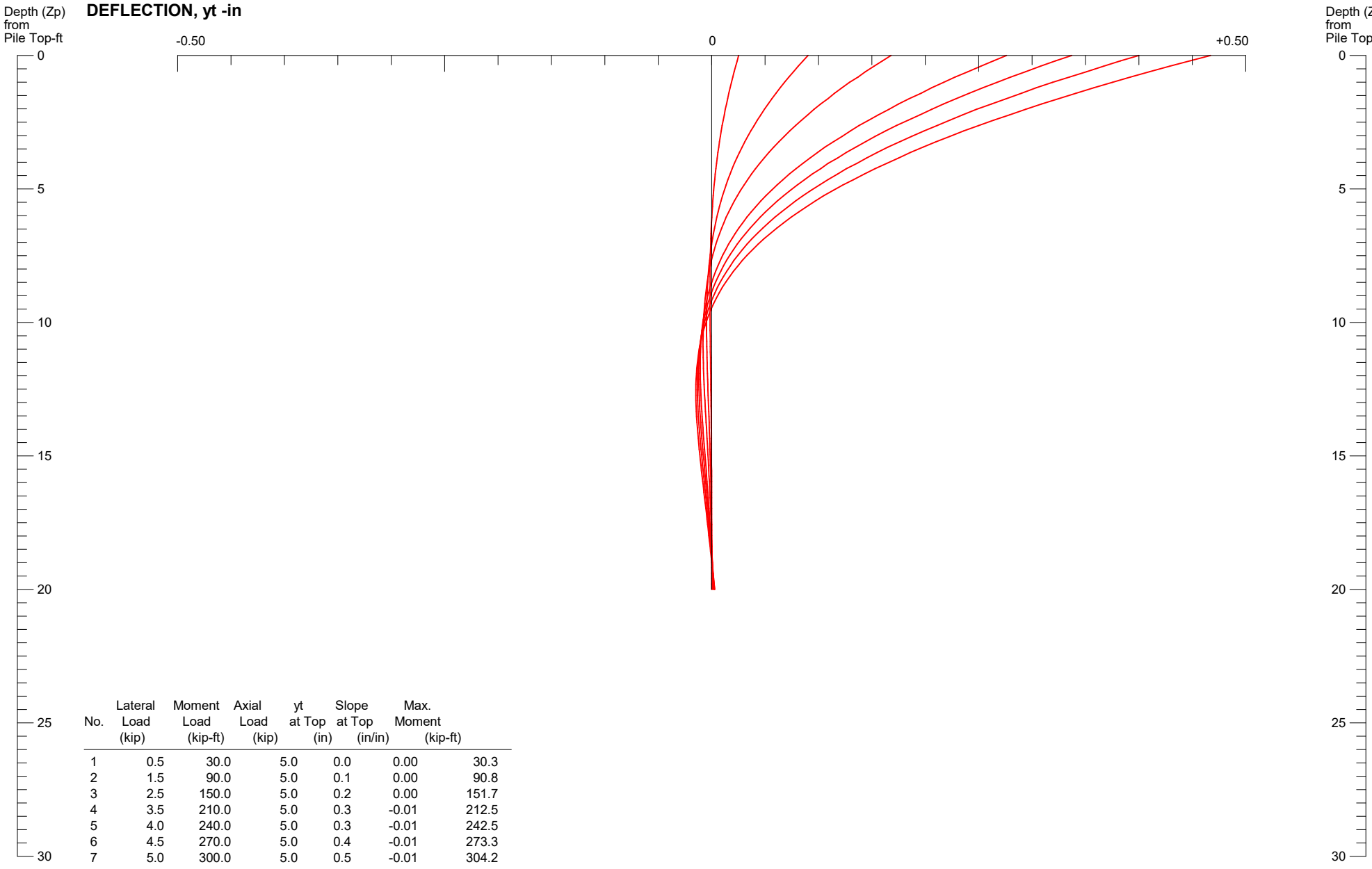




PILE DEFLECTION vs LOADING

Single Pile, Khead=1, Kbc=1

Pile below Ground (not to scale)

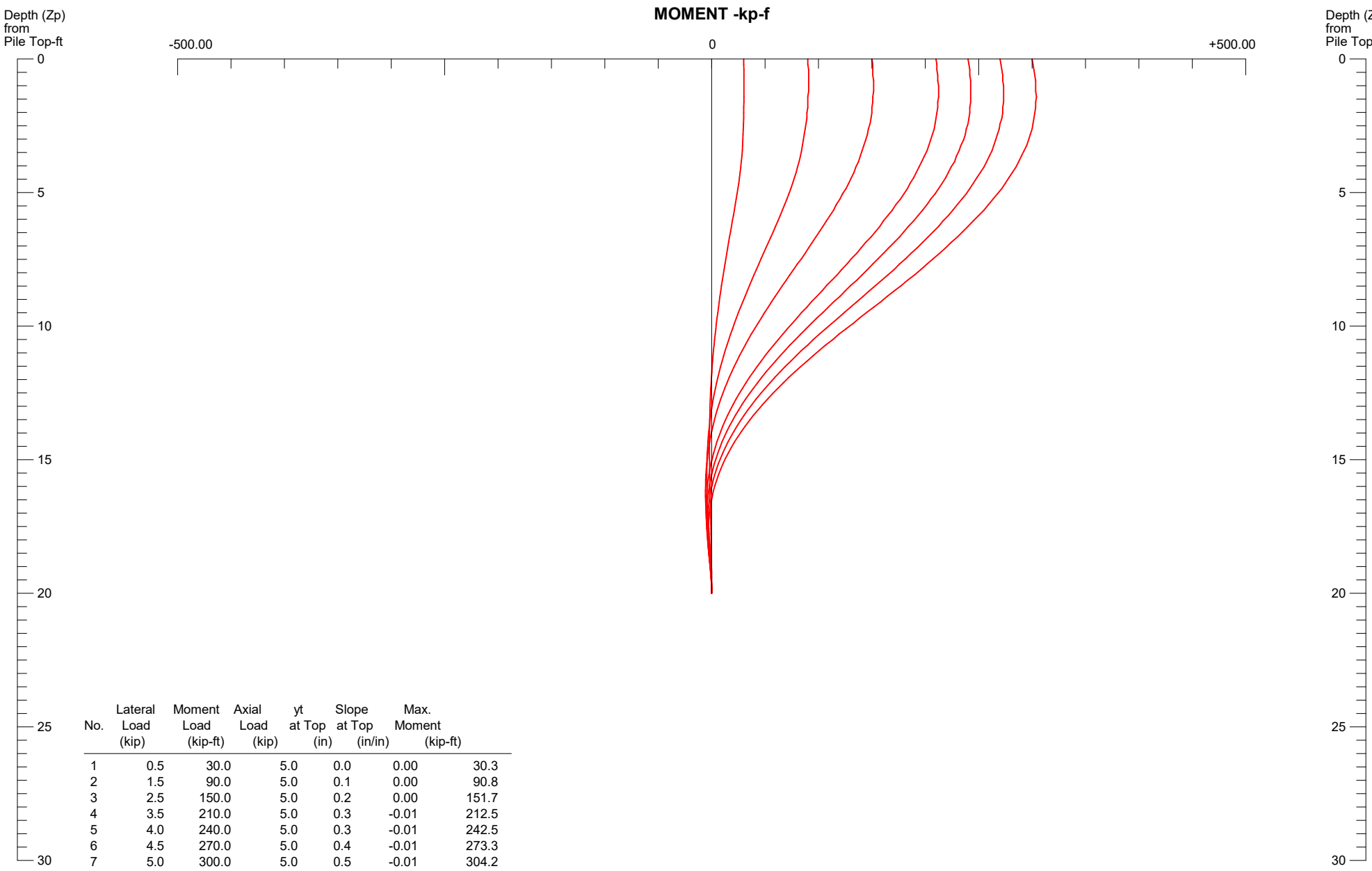


No.	Lateral Load (kip)	Moment Load (kip-ft)	Axial Load (kip)	yt at Top (in)	Slope at Top (in/in)	Max. Moment (kip-ft)
1	0.5	30.0	5.0	0.0	0.00	30.3
2	1.5	90.0	5.0	0.1	0.00	90.8
3	2.5	150.0	5.0	0.2	0.00	151.7
4	3.5	210.0	5.0	0.3	-0.01	212.5
5	4.0	240.0	5.0	0.3	-0.01	242.5
6	4.5	270.0	5.0	0.4	-0.01	273.3
7	5.0	300.0	5.0	0.5	-0.01	304.2

PILE MOMENT vs LOADING

Pile below Ground (not to scale)

Single Pile, Khead=1, Kbc=1



No.	Lateral Load (kip)	Moment Load (kip-ft)	Axial Load (kip)	yt at Top (in)	Slope at Top (in/in)	Max. Moment (kip-ft)
1	0.5	30.0	5.0	0.0	0.00	30.3
2	1.5	90.0	5.0	0.1	0.00	90.8
3	2.5	150.0	5.0	0.2	0.00	151.7
4	3.5	210.0	5.0	0.3	-0.01	212.5
5	4.0	240.0	5.0	0.3	-0.01	242.5
6	4.5	270.0	5.0	0.4	-0.01	273.3
7	5.0	300.0	5.0	0.5	-0.01	304.2