Appendix IS-2

Geotechnical Investigation

PRELIMINARY GEOTECHNICAL INVESTIGATION
ALAMEDA CROSSING DEVELOPMENT

1716 East 7th Street
Los Angeles, California
For
ProLogis

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

October 17, 2024

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

ProLogis
Pier 1, Bay 1
San Francisco, California 94111

Attention: Mr. D.J. Arellano
Vice President

Project No.: **20G243-4R2**

Subject: **Preliminary Geotechnical Investigation (Revised October 2024)**
Alameda Crossing Development
1716 East 7th Street
Los Angeles, California

Mr. Arellano:

In accordance with your request, we have conducted a revised preliminary 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.

Ricardo Frias, RCE 91772
Project Engineer

Gregory K. Mitchell, GE 2364
Principal Engineer

Distribution: (1) Addressee

22885 Savi Ranch Parkway ▾ Suite E ▾ Yorba Linda ▾ California ▾ 92887
voice: (714) 685-1115 ▾ www.socalgeo.com

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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

- The proposed development will consist of three, one-story structures (i.e., Stage Groups A, B, and C) surrounding the main building which will be eight stories (i.e., the Main Building). The Main Building will be 128± feet in height, to the top of the building.
- Undocumented fill soils were encountered at most of the boring locations, extending to depths of 2 to 8± feet below the existing site grades.
- The results of laboratory testing indicate that some of the near-surface soils possess variable strengths and are moderately compressible when loaded. The native soils at depths of 20 to 30± feet possess high strengths and favorable consolidation characteristics.
- The proposed Main Building structure is expected to exert column loads of 800 kips. Based on the presence of low strength alluvium and fill soils at this site, these foundations would cause excessive settlements if supported on the presently existing soils. Based on construction considerations, ground improvement consisting of rammed aggregate columns (RACs) is considered to be the most feasible alternative to support the proposed Main Building structure. RACs consist of pre-augured cavities that are backfilled with compacted aggregate that creates relatively stiff columns of compacted stone surrounded by a stiffened soil matrix. Installation of the RACs will significantly reduce settlements as well as increase the allowable bearing capacity of the soils.
- Stage Groups A, B, and C, as well as accessory structures such as retaining walls, site walls, trash enclosures, etc., may be supported on conventional spread footings underlain by a newly placed layer of compacted structural fill.
- Liquefaction is not a design concern for this project based on the conditions encountered at the boring locations and the mapping performed by the California Geological Survey.

Site Preparation

- Demolition of the structures associated with the existing development, including buildings, associated improvements, and the existing pavements will be required in order to facilitate construction of the new buildings. Demolition should also include all utilities and any other subsurface improvements that will not remain in place for use with the new development. Debris resulting from demolition should be disposed of off-site. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well-mixed with the on-site soils, and incorporated into new structural fills or it may be crushed and made into crushed miscellaneous base (CMB).
- A basement is located below nearly all of the existing multi-level maintenance service building. The existing walls and Portland cement concrete (PCC) floor slab should be demolished and removed in their entirety to allow placement of future ground improvement elements.
- Installation of the RAC system will improve the soils beneath the foundations. However, it will be necessary to improve the soils that will support the new ground level floor slab. The proposed main building structure area should be overexcavated to a depth of at least 5 feet below existing grade and to a depth of 5 feet below proposed building pad subgrade elevation.

The existing undocumented fill soils should also be completely removed. The overexcavation should extend horizontally at least 5 feet beyond the building perimeter.

- Remedial grading is recommended to be performed within the proposed Stage Groups A, B, and C building areas in order to remove the undocumented fill soils in their entirety as well as the upper portion of the near-surface native alluvial soils. The soils within the proposed Stage Group building areas should be overexcavated to a depth of 6 feet below existing grade and to a depth of at least 5 feet below proposed building pad subgrade elevations. The proposed foundation influence zones for the Stage Group buildings should be overexcavated to a depth of at least 4 feet below proposed foundation bearing grade.
- Deeper removals to a depth of 10± feet below the existing site grades may be necessary in the vicinity of Boring Nos. B-2, B-5 and B-9 due to the presence of undocumented fill and loose soils.
- Deeper removals may be required within the area of the existing basement. The extent of undocumented fill soils will need to be determined at the time of remedial grading.
- After the overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be removed. The resulting subgrade should then be scarified to a depth of 12 inches and moisture conditioned to 0 to 4 percent above optimum. The previously excavated soils may then be replaced as compacted structural fill. All structural fill soils placed within the proposed building areas should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density.
- RACs should be installed within the area of the proposed main building foundations. The RACs will be designed and constructed by an independent design-build firm. Installation of the RACs should be monitored by a representative of the geotechnical engineer.
- Conventional shallow foundations used to support accessory structures such as retaining walls, site walls, trash enclosures, etc., should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade. These excavations should then be backfilled with structural fill soils and compacted to at least 90 percent relative compaction.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Main Building Foundations

- The new building foundations can be supported on the RACs that will be installed at the foundation locations.
- 8,000 lbs/ft^2 maximum allowable soil bearing pressure.
- 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.

Stage Group Structure and Accessory Structure Foundations

- The stage group and accessory structures can be supported on conventional shallow foundations supported on new engineered fill soils.
- 2,500 lbs/ft^2 maximum allowable soil bearing pressure.
- 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.

Building Floor Slabs

- Conventional Slabs-on-Grade, minimum thickness: 8 inches.
- Modulus of Subgrade Reaction: k = 150 psi/in.

- Minimum slab reinforcement: Not required for geotechnical considerations. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

Pavement Design Recommendations

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40)				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5	5½	6½
Compacted Subgrade (95% Relative Compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in general accordance with our Proposal No. 22P339R, dated January 19, 2023. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, ground motion hazard analysis, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, 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 at the southeast corner of South Alameda Street and East 7th Street in Los Angeles, California (project site). The project site is also referenced by the street address 1716 East 7th Street. The project site is bounded to the north by East 7th Street, to the west by South Alameda Street, to the south by an existing commercial/industrial building, and to the east by Decatur Street. The general location of the project site is illustrated on the Site Location Map, included as Plate 1 of this report.

The project site consists of several rectangular-shaped parcels which total 8.3± acres in size. The three (3) parcels are transected by two (2) north-south trending streets, identified as Channing Street in the west and Laurence Street in the east. The easternmost parcel is developed with a single-story 30,000± ft² commercial/industrial building, located in the south-central area of the parcel. The building was previously used as the Los Angeles Greyhound Station. The building is of concrete tilt-up construction, assumed to be supported on conventional shallow foundations with a concrete slab-on-grade floor. The building is surrounded by asphaltic concrete (AC) pavements in the drive and parking areas, concrete flatwork, and landscaped planters that include shrubs and medium to large trees. The existing AC pavements and concrete flatwork are in poor condition with moderate to severe cracking throughout. The central parcel is developed with an 87,000± ft² multi-level maintenance service building. The first level of the central portion of the building was previously used as a washing station for buses. Nearly the entire structure is underlain by a large basement. The building is of concrete tilt-up construction, assumed to be supported on conventional shallow foundations with a concrete slab-on-grade floor. The building is generally surrounded by AC pavements and Portland cement concrete (PCC) pavements in the northwestern region. The existing pavements are in poor condition with minor to severe cracking throughout. The eastern and central buildings are vacant but are currently being used by LAPD for training purposes. The remaining parcels are generally developed with AC or PCC pavements with isolated landscaped planters. These pavements are also in poor condition with minor to severe cracking throughout.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth, and visual observations made at the time of the subsurface investigation, the overall site topography slopes downwards to the east at a gradient of less than 1 percent.

3.2 Proposed Development

The project site plan provided to our office on February 6, 2024, and included as Plate 2 in Appendix A of this report, indicates that the project site will be developed with multiple clusters of buildings. The westernmost region of the project site will be developed with two buildings that will share a common wall, identified as Stage Group A, and will be 60,765 ft^2± in size. The south-central region of the project site will be developed with two buildings which will share a common wall. This structure will be identified as Stage Group B and will be 52,980 ft^2± in size. The easternmost region of the project site will be developed with two buildings that will share a common wall, identified as Stage Group C. This structure will be 60,611 ft^2± in size. The north-central region of the project site will be developed with an eight-level multi-purpose structure identified as "Main Building" which includes six levels of integrated automobile parking. The Main Building will be 189,671 ft^2± in size. It is assumed that the Main Building structure will be of reinforced concrete and steel-frame construction. Maximum column loads for this building are expected to be on the order of 1,200 kips to 2,300 kips. The construction of the Stage Group buildings is assumed to be of tilt-up construction. Maximum column and wall loads were given as 150 kips and 10.2 kips/foot, respectively. The buildings will be surrounded by asphaltic concrete and/or PCC pavements and limited areas of concrete flatwork.

No significant basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of 3 to 5± feet are expected to be necessary to achieve the proposed site grades.

3.3 Previous Study

SCG previously conducted a geotechnical investigation at the project site referenced above. The results of this investigation are presented in the report referenced as follows:

> Geotechnical Investigation, Proposed Commercial/Industrial Development, 1716 East 7th Street, Los Angeles, California, prepared by SCG for ProLogis, SCG Project No. 20G243-2, dated September 22, 2022.

As a part of this study, three (3) borings (identified as Boring Nos. B-1 through B-3) were advanced to depths of 50 to 130± feet below the existing site grades. Findings from the prior subsurface exploration and laboratory testing have been incorporated into the analysis and recommendations of this report. Data from the previous study, including the boring logs, along with the results of laboratory testing, are included in this report.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of eleven (11) borings advanced to depths of 30 to 130± feet below the existing site grades. All of the borings were logged during drilling by a member of our staff.

All of the borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil 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. In-situ 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. Additionally, geotechnical cross sections were prepared by SCG illustrating the existing topography, proposed topography, proposed structures, and fill/native contacts. The cross sections are included as Plate 5 in Appendix A of this report.

4.2 Geotechnical Conditions

Pavements

AC pavements were encountered at the ground surface at Boring Nos. B-1, B-3, B-4, B-6, B-7, and B-9. PCC pavements were encountered at the ground surface at Boring Nos. B-2, B-5, B-8, B-10, and B-11. The pavement sections at each boring location are presented below.

- Boring No. B-1: 2± inches of AC pavements with no discernible aggregate base.
- Boring No. B-2: 8± inches of PCC pavements underlain by 6± inches of aggregate base.
- Boring No. B-3: 7± inches of AC pavements with no discernible aggregate base.
- Boring No. B-4: 2± inches of AC pavements underlain by 5± inches of aggregate base underlain by 9± inches of PCC pavements.
- Boring No. B-5: 5± inches of PCC pavements underlain by 9± inches of aggregate base underlain by 7± inches of PCC pavements.

- Boring No. B-6: 4± inches of AC pavements underlain by 6± inches of aggregate base.
- Boring No. B-7: 2± inches of AC pavements underlain by 8± inches of PCC pavements.
- Boring No. B-8: 6½± inches of PCC pavements underlain by 3± inches of aggregate base.
- Boring No. B-9: 7± inches of AC pavements underlain by 5± inches of PCC pavements underlain by 3± inches of aggregate base.
- Boring No. B-10: 9± inches of PCC pavements underlain by 8± inches of PCC pavements.
- Boring No. B-11: 10± inches of PCC pavements underlain by 4± inches of aggregate base.

Artificial Fill

Artificial fill soils were encountered beneath the existing pavements at all of the boring locations, extending to depths of 2 to 8± feet below the existing site grades. The fill soils generally consist of very loose to very dense silty sands, well-graded sands, and gravelly sands with varying silt, clay and gravel content. Boring No. B-5 also encountered a layer of medium stiff clayey silt at depths of 5 to 8± feet. The artificial fill soils possess a disturbed and/or mottled appearance, resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered beneath the fill soils at all of the boring locations, extending to at least the maximum depth explored of 130± feet below the existing site grades. The near-surface alluvium generally consists of loose to medium dense well-graded sands and silty sands, with varying fine gravel content and occasional gravelly sands, extending to depths of 12 to 30± feet. At greater depths and extending to the maximum depth explored of 130± feet, the alluvium generally consists of dense to very dense gravelly sands, silty sands and poorly-graded to well-graded sands with varying fine to coarse gravel content. Boring No. B-1 encountered a soil stratum consisting of medium dense fine sandy silt at depths of 17 to 19½± feet. Boring No. B-3 encountered a soil stratum consisting of hard silty clay at depths of 54½ to 57± feet. Boring No. B-5 encountered a soil stratum consisting of dense silt at depths of 29½ to 30½± feet.

Groundwater

Groundwater was not encountered at any of the boring locations. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 130± feet below existing site grades, at the time of the subsurface investigation.

As part of our research, we reviewed available groundwater data regarding the historic high groundwater level for the project site. The primary reference used to determine the historic groundwater depths in this area is the California Geological Survey (CGS) Open File Report 98-20, the Seismic Hazard Zone Report for the Los Angeles 7.5-Minute Quadrangle, which indicates that the historic high groundwater level for the project site is greater than 150 feet below the ground surface.

As part of our research, we reviewed readily available groundwater data in order to determine regional groundwater depths. The primary reference used to determine the groundwater depths in the project site area is the California State Water Resources Control Board, GeoTracker, website, https://geotracker.waterboards.ca.gov/. Several monitoring wells on record are located 2,300± north of the project site. Water level readings within these monitoring wells indicate a high groundwater level of 96½± feet below the ground surface, in June 2009. The identified wells provide geotechnically meaningful data regarding groundwater and depth, however, have been abandoned as part of environmental cleanup activities.

A report titled Report of Soil investigation Activities, Greyhound lines, Inc., 1614 East 7th Street, Los Angeles, CA 90021 (Strata Environmental Services Inc, 2016) documents the results of soil sampling at the project site, and this report was found on the GeoTracker website. The Strata report indicates that the Los Angeles Regional Water Quality Control Board (LARWQCB) has issued a directive letter indicating that the depth to groundwater at the project site is 95 feet.

4.3 Geologic Conditions

The primary available reference applicable to the project site is the Geologic Map of the Los Angles Quadrangle, Los Angeles County, California, by Thomas W. Dibblee, Jr., 1989. A portion of this map indicating the location of the project site is included herein as Plate 3 in Appendix of this report. The map indicates that the project site is underlain by Alluvial deposits (map symbol Qa) consisting of unconsolidated floodplain deposits of silt, sand and gravel. Areas of older alluvium are also mapped in the area of the project site. The geologic map does not indicate the presence of any faults in the near vicinity of the project site.

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

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. The 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.

Dry 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-17 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

Representative bulk samples were tested for their 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 type or soil mixes may be necessary at a later date. The results of the testing are plotted on Plates C-18 and C-19 in Appendix C of this report.

Direct Shear

Direct shear tests were performed on selected soil samples to determine their shear strength parameters. The tests were 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 tests are presented on Plates C-20 through C-23.

Soluble Sulfates

Representative samples of the near-surface soil 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 (%)	Sulfate Classification
B-1 @ 0 to 5 feet	0.004	Not Applicable (S0)
B-3 @ 0 to 5 feet	0.004	Not Applicable (S0)
B-8 @ 1 to 5 feet	0.018	Not Applicable (S0)
B-9 @ 1 to 5 feet	0.012	Not Applicable (S0)

Expansion Index

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-inch 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-2 @ 0 to 5 feet	3	Very Low

Corrosivity Testing

Representative bulk samples of the near-surface soils were submitted to a subcontracted corrosion engineering laboratory to identify potentially corrosive characteristics with respect to common construction materials. The corrosivity testing included a determination of the electrical resistivity, pH, and chloride and nitrate concentrations of the soils, as well as other tests. The results of the applicable tests are presented below.

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)	Sulfides (mg/kg)	Redox Potential (mV)
B-1 @ 0 to 5 feet	6,000	9.0	7.1	32	--	--
B-3 @ 0 to 5 feet	3,956	8.8	25	31	--	--
B-8 @ 1 to 5 feet	4,623	9.2	89.2	4.9	0.1	153
B-9 @ 1 to 5 feet	8,710	9.3	82.0	4.4	0.3	147

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review of the site plan for the site, 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 all 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

As is the case for most sites in Southern California, the project 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 project site. The structures should be designed to the performance objectives required by the California Building Code.

Faulting and Seismicity

Research of available maps indicates that the project site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, Southern California Geotechnical (SCG) did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the project 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 project site is considered low due to the project site's distance from a significant body of water based on a search of available maps.

2023 LABC Seismic Design Parameters

Based on the standards in place at the time of this report, we expect that the proposed structures will be designed in accordance with the 2023 Edition of the City of Los Angeles Building Code (LABC), which was adopted on January 1, 2023. The 2023 LABC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped S_1 value greater than 0.2.

Section 11.4.8 of ASCE 7-16 states that "it shall be permitted to perform a site response analysis or in Accordance with Section 21.1 and/or a ground motion hazard analysis in accordance with Section 21.2." Therefore, a site-specific ground motion hazard analysis was performed in accordance with Section 21.2 of ASCE 7-16 to determine the seismic design parameters for the new structures at this site.

The site classification was determined using shear wave velocity measurements for the soils present within the upper 100± feet at the project site. The parameter V_{100} is defined as the shear-wave velocity of the soil or bedrock material present within the upper 100 feet at the project site. The shear-wave velocity was determined by a seismic shear wave survey performed by a licensed geophysicist. The results of the shear-wave survey are included in a report prepared by Terra Geosciences, included in Appendix E of this report. Based on the shear-wave survey performed by Terra Geosciences, the V_{100} for the project site is 1,151.2 feet per second. Table 20.3-1 of ASCE 7-16 indicates that an average shear wave velocity ranging between 600 and 1,200 feet per second corresponds to Site Class D.

Details regarding the performance of the ground motion hazard analysis are presented in the report prepared by Terra Geosciences, included in Appendix E of this report. Seismic design parameters computed during this study are tabulated below.

SITE-SPECIFIC SEISMIC DESIGN PARAMETERS BASED ON ASCE 7-16 SECTION 21.2

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.920
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.684
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.885
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.368
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.260
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.910

<u>Liquefaction</u>

Liquefaction is the loss of the 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 ($d<0.005$mm) 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 Earthquake Zones of Required Investigation, Los Angeles Quadrangle map, published by the California Geological Survey (CGS), revised June 15, 2017, indicates that the project site is not located within a designated liquefaction hazard zone. In addition, the subsurface conditions encountered at the project site are not considered to be conducive to liquefaction. Finally, the historic high groundwater table has been mapped at a depth in excess of 100 feet. Based on the conditions encountered at the boring locations, and the mapping performed by the CGS, liquefaction is not considered to be a significant design concern for this project.

6.2 Geotechnical Design Considerations

General

Artificial fill soils were encountered beneath the pavements or at the ground surface at most of the boring locations, extending to depths of 2 to 8± feet below the existing project site grades. These soils consist of variable-strength, very loose to dense silty sands and sands. Additionally, no documentation regarding the placement and compaction of these soils has been provided to our office. The fill soils are therefore considered to be undocumented fill. The fill soils are underlain by native alluvium which possesses variable strengths and composition. The results of laboratory testing indicate that the some of the native alluvial soils within the upper 5 to 12± feet exhibit loose densities and slightly unfavorable compressibility characteristics. Based on these conditions, the artificial fill materials and the upper portion of the near-surface alluvium, in their present condition, are not considered suitable for the support of new foundations and floor slabs of the proposed structures.

Based on our professional experience and our review of the site plan for the project site, the structures may be supported on spread footings or a mat foundation. The use of shallow foundations is typically the most cost-effective method of development. However, due to the presence of the undocumented fill soils and low to moderate strength alluvium within the upper 10 to 20± feet, and the foundation loads of the proposed Main Building structure, shallow foundations (either spread footings or mat foundations) supported on the existing soils would experience significant settlements.

In formulating our recommendations, we have considered several options, including deep overexcavation of the existing soils followed by replacement with compacted structural fill and the use of rammed aggregate columns (RACs). Our analysis indicates that the RACs solution is the most feasible option that will result in acceptable levels of static settlement. The RACs will be installed beneath the proposed Main Building foundations. This report provides recommendations for the use of a RAC foundation system within the Main Building structure area. The benefits of a RAC system include a significant reduction in the extent of remedial grading, an increased bearing capacity of the resulting soils, high coefficient of friction, and reduced static and seismic settlements. Further details regarding the RAC system are presented in a subsequent section of this report.

Conventional foundations and grading techniques are recommended for the Stage Group buildings and accessory structures located outside the Main Building area, such as retaining walls, trash enclosures, property line walls, etc.

The City of Los Angeles does not allow structures to be supported on undocumented fill soils. In addition, the RACs are not permitted in lieu of removal and replacement of undocumented fill. Therefore, remedial grading will be necessary within the proposed building areas to remove the artificial fill soils in their entirety as well as a portion of the near-surface alluvium, and to replace these soils as compacted structural fill.

Demolition of the existing buildings, pavements and associated improvements is expected to cause extensive disturbance to the near-surface soils. Any soils disturbed during demolition of the existing structures and site improvements should also be removed and recompacted as structural fill.

Settlement

Installation of the proposed RAC system will result in a significant decrease in the static and seismic settlements. The RAC system should be designed to reduce the static total settlements to approximately 1 inch and differential static settlements will be less than 0.5 inch over a 30-foot span. These settlements are considered to be within the settlement tolerances of the proposed structures, but this assumption should be verified by the project structural engineer.

On-site Stormwater Infiltration

Based on soil profile of the project site consisting mainly of sands, with fine grained material being encountered at depths greater than 50± feet below the existing site grades, the risk of a perched groundwater condition is considered to be relatively low.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the tested soil samples possess a level of soluble sulfates that is considered to be "not applicable" (S0) with respect to the American Concrete Institute (ACI) Publication 318-14 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 areas.

Corrosion Potential

The results of laboratory testing indicate that the tested samples of the on-site soils possess saturated resistivity values ranging from 3,956 to 8,710 ohm-cm, and pH values ranging from 8.8 to 9.3. 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 project 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.

Relatively low concentrations of chlorides (7.1 to 89.2 mg/kg) were detected in the samples submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested sample, the project site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations of 4.4 to 32 mg/kg. Based on these test results, the on-site soils are not considered to be corrosive to copper pipe with respect to their nitrate concentrations.

SCG does not practice in the area of corrosion engineering. Therefore, the client may wish to contact a corrosion engineer to provide additional evaluation of the corrosion test results.

Expansion

Laboratory testing performed on a representative sample of the near surface soils indicates that these materials are very low expansive (EI = 3). Based on this test result, no special design considerations for expansive soils are considered warranted. However, 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 pads.

Shrinkage/Subsidence

Removal and recompaction of the near surface fill and alluvial soils is estimated to result in an average shrinkage of 7 to 17 percent. However, potential shrinkage for individual samples ranged locally between 1 and 20 percent. The potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations.

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 0.15 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. These estimates should be reviewed and revised as necessary based on the additional subsurface exploration that is expected to occur after the project plans have been finalized.

Grading and Foundation Plan Review

Grading and foundation plans were not available 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 during the project's construction design phase, to confirm that the grading and foundation plans are consistent with the conclusions, recommendations, and assumptions contained within this report. Some minor report revisions to this report may be necessary once the grading and foundation plans are finalized.

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 all 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

The proposed development will require demolition of the existing buildings and other improvements including pavements. Any existing improvements that will not remain in place for use with the new development should be removed in their entirety. This should include all foundations, floor slabs, utilities, trees and associated root masses, and any other above-ground and subsurface improvements associated with the existing structures. The existing pavements are not expected to be reused with the new development. Debris resultant from demolition should be disposed of off-site. Asphaltic concrete and Portland cement concrete debris may be crushed and made into miscellaneous base for use in the proposed pavement areas or crushed to a particle size less than 2 inches and blended with the on-site soils for use in structural fills. All organic materials should be disposed of off-site.

A basement is also located below nearly the entire footprint of the existing multi-level maintenance service building. The existing walls and PCC floor slab should be demolished and removed in their entirety. Debris resultant from demolition should be disposed of off-site. AC concrete and PCC concrete debris may be crushed and made into miscellaneous base for use in the proposed pavement areas or crushed to a particle size less than 2 inches and blended with the on-site soils for use in structural fills.

Detailed structural information regarding the existing buildings has not been provided to SCG. Therefore, the foundation systems supporting the existing buildings are presently unknown by SCG. If any of the existing buildings are supported on deep foundation systems, the deep foundation elements located within the proposed structure areas should be cut off at a depth of at least 3 feet below the bottom of the planned overexcavation. Where deep foundations are encountered within proposed pavement areas, they should be cut off at a depth of at least 2 feet below the proposed pavement subgrade or at a depth of at least 1 foot below the bottom of any planned utilities.

Treatment of Existing Soils: Main Building Pad

Remedial grading will be necessary within the proposed Main Building structure areas to remove all of the undocumented fill soils and a portion of the existing variable strength and variable density near-surface alluvial soils and to provide a uniform blanket of compacted fill upon which to support the proposed structures. Based on conditions encountered at the boring locations, undocumented fill soils extend to depths of 2½ to 8± feet below existing grade.

In addition to removing all of the undocumented fill soils, it is recommended that the overexcavation extend to a depth of at least 5 feet below existing grade and to a depth of at least 5 feet below proposed grade, whichever is greater. The overexcavation areas should extend at least 5 feet beyond the building perimeters. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

Slightly deeper areas of overexcavation to a depth of 10± feet may also be required in the vicinity of Boring Nos. B-2, B-5, and B-9, where artificial fill and loose soils extend. Additional evaluation of the exposed overexcavation subgrade soils by the geotechnical engineer will be required in this area of the project site to verify that the full extent of loose soils, as encountered at the previously mentioned boring locations, are removed. Deeper removals may be required within the area of the basement. The depth of undocumented fill soils will need to be evaluated at the time of remedial grading.

Following completion of the overexcavation, the subgrade soils within the overexcavation areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade. 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, overly moist, or low density native soils are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture conditioned or air dried to achieve a moisture content of 0 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 95 percent of the ASTM D-1557 maximum dry density. The building pad areas may then be raised to grade with previously excavated soils or imported, structural fill. **All structural fill soils within the proposed building areas should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density.**

Treatment of Existing Soils: New Foundation Areas

It is recommended that the existing soils in the area of the Main Building foundations be improved through the installation of RACs. The RACs will be installed throughout the foundation areas. Based on the existing conditions, the RACs will extend to a depth of approximately 15 feet below foundation bearing grade.

The RAC construction process consists of utilizing pre-augured holes that are backfilled with aggregate that is compacted in place using static crowd pressure augmented with a high frequency, low amplitude, vibratory hammer. The impact hammer densifies the aggregate vertically while the tamper foot forces aggregate laterally into the cavity sidewalls, resulting in stiff RAC elements and a stiffened matrix soil between the RACs. The actual diameter of the RACs will be determined by WGI, but typically range from 18 to 24 inches. The RAC design and installation should be in accordance with City of Los Angeles Research Report RR 26139.

The RAC installation process should be observed and documented by a representative of the geotechnical engineer. This documentation should include RAC spacing, diameter, and depth.

Treatment of Existing Soils: Stage Group Building Pads

Remedial grading will be necessary within the proposed Stage Group structure areas to remove all of the undocumented fill soils and a portion of the existing variable strength and variable density near-surface alluvial soils and to provide a uniform blanket of compacted fill upon which to support the proposed structures. Based on conditions encountered at the boring locations, undocumented fill soils extend to depths of 2½ to 8± feet below existing grade within these building areas. It is recommended that the overexcavation also extend to a depth of at least 6 feet below existing grade, and to a depth of at least 5 feet below proposed grade, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 4 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeters, and to an extent equal to the depth of fill below the new foundations. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the overexcavation areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade. 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, overly moist, or low density native soils are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture conditioned or air dried to achieve a moisture content of 0 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 95 percent of the ASTM D-1557 maximum dry density. The building pad areas may then be raised to grade with previously excavated soils or imported, structural fill. **All structural fill soils within the proposed building areas should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density.**

Treatment of Existing Soils: Retaining Walls and Site Walls

It is expected that shallow foundations will also be required for support of appurtenances located outside the building areas, such as retaining walls, site walls, trash enclosures, etc. The existing soils within the foundation areas of these accessory structures 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 should also be removed from the retaining wall areas. 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. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral extent of overexcavation cannot be completed during grading of the proposed retaining walls and site walls located along property lines, the foundations for those walls should be designed using a reduced allowable bearing pressure. Furthermore, the contractor should take necessary precautions to protect the adjacent improvements during rough grading. Specialized grading techniques, such as A-B-C slot cuts, will likely be required during remedial grading. The geotechnical engineer of record should be contacted if additional recommendations, such as shoring design recommendations, are required during grading.

Treatment of Existing Soils: Parking Areas

Based on City of Los Angeles standards, overexcavation and replacement of the existing undocumented fill soils, ranging between 2 to 8± feet, within the new parking areas will be required.

Subgrade preparation in the new parking areas should initially consist of removal of all soils disturbed during stripping operations. All existing undocumented fill soils should also be removed. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils 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. Based on the presence of variable strength alluvial soils throughout the project site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

Treatment of Existing Soils: Flatwork Areas

Subgrade preparation in the new flatwork areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils 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. Based on the presence of variable strength alluvial soils throughout the project site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

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. Drying of the on-site soils may be required before placement and compaction of structural fill.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2023 LABC and the Grading Code of the City of Los Angeles.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. We recommend that fill soils placed within the foundation influence zones and beneath new building areas be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. **In accordance with City of Los Angeles requirements, if soils possessing less than 15 percent clay (finer than 0.005 mm) are used for**

fill, they must be compacted to at least 95 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

All 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, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the City of Los Angeles. All 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 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.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consisted of sands, silty sands and sandy silts. These materials may be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes consisting of sands, silty sands and sandy silts should be made no steeper than 2h:1v. The contractor should take all necessary precautions during grading and foundation construction to prevent damage to structures and improvements which are adjacent to the proposed development. 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. All excavation activities on this project site should be conducted in accordance with Cal-OSHA regulations.

Based on the recommended remedial grading, excavation to depths of 5± feet may be necessary near the north, west and east property lines, where the proposed structures will border Alameda Street, East 7th Street, and Decatur Street. Temporary shoring may be necessary in these areas to complete the recommended remedial grading. Recommendations for temporary shoring parameters can be found in Section 6.9 of this report.

Moisture Sensitive Subgrade Soils

The near-surface soils generally consist of moist silty sands, and sandy silts, 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 project 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 areas as well as the need for subgrade stabilization.

Groundwater

The static groundwater table at this project site is considered to exist at a depth greater than 130± feet. The depth of excavation for utilities and foundations at this project site is expected to be less than 20 feet. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new Main Building foundations will be underlain by existing soils which have been improved by the placement of a system of RACs. Furthermore, it is expected that any new foundations for the Stage Group structures and appurtenances, such as retaining walls, site walls, trash enclosures, etc., will be underlain by newly placed structural fill soils, extending to depths of at least 4 feet below proposed foundation bearing grade. Based on this subsurface profile, the proposed structures may be designed as follows:

Foundation Design Parameters (New Main Building Foundations)

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 8,000 lbs/ft^2.
- 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 24 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab. The building foundations should be directly supported on the RACs.

- 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 standard geotechnical practice. Additional rigidity may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer. The maximum allowable bearing pressure presented above is preliminary; the actual bearing pressure should be determined by the RAC designer based on both bearing capacity and settlement considerations. The bearing pressure is also contingent upon our review of the final site plan and completion of any necessary supplemental geotechnical investigation at the project site.

<u>Foundation Design Parameters (Stage Group Buildings and Non-Building Foundations)</u>

New square and rectangular footings used to support the Stage Groups structures and accessory structures such as retaining walls, screen walls, and trash enclosures 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, typically for new footings along the property lines.

- 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 standard geotechnical practice. Additional rigidity may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

<u>Foundation Construction</u>

The foundation subgrade soils should be evaluated at the time of site grading, 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 support of the new Main Building foundations should consist of existing soils that have been improved through the placement of the RACs. Soils suitable for direct foundation support in the Stage Group and accessory structure areas should consist of newly placed structural fill, compacted to at least 95 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, 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 static 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 30-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: 300 lbs/ft^3
- Friction Coefficient: 0.30
- Friction Coefficient: 0.45 for foundations supported directly on RACs.

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 soils. The maximum allowable passive pressure is 3,000 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs 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 project site, the floors of the proposed structures may be constructed as conventional slabs-on-grade supported on newly placed structural fill (95% compaction), extending to a depth of at least 5 feet below finished pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness: 8 inches.

- Modulus of Subgrade Reaction: 150 psi/in.

- Minimum slab reinforcement: Not required for geotechnical considerations. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed 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 slab area where such moisture sensitive floor coverings are expected. 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 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 actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

6.7 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for sidewalks, patios, and other concrete flatwork, should be prepared in accordance with the recommendations contained in the ***Grading Recommendations*** section of this report. Based on geotechnical considerations, exterior slabs on grade may be designed as follows:

- Minimum slab thickness: 4½ inches.

- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.

- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.

- Moisture condition the slab subgrade soils to at least 0 to 4 percent of optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 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.

- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.

Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

6.8 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 6 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 for preliminary design of new retaining walls for this project site. The following parameters assume that only the on-site sands, silty sands and sandy silts should be utilized for retaining wall backfill. Based on the results of our direct shear testing, the on-site soils consisting of sands, silty sands and sandy silts have been preliminarily assigned a conservative friction angle of 30 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

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.

<u>**RETAINING WALL DESIGN PARAMETERS**</u>

<table>
<tr><th colspan="2" rowspan="2">Design Parameter</th><th>Soil Type</th></tr>
<tr><th>On-Site Sands, Silty Sands and Sandy Silts</th></tr>
<tr><td colspan="2">Internal Friction Angle (ϕ)</td><td>30°</td></tr>
<tr><td colspan="2">Unit Weight</td><td>130 lbs/ft³</td></tr>
<tr><td rowspan="3">Equivalent Fluid Pressure:</td><td>Active Condition (level backfill)</td><td>43 lbs/ft³</td></tr>
<tr><td>Active Condition (2h:1v backfill)</td><td>70 lbs/ft³</td></tr>
<tr><td>At-Rest Condition (level backfill)</td><td>65 lbs/ft³</td></tr>
</table>

The walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft³. 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.

<u>Seismic Lateral Earth Pressures</u>

In addition to the lateral earth pressures presented in the previous section, retaining walls which are more than 6 feet in height should be designed for a seismic lateral earth pressure, in accordance with the 2023 LABC. Based on the current site plan, it is not expected that any walls in excess of 6 feet in height will be required for this project. If any such walls are proposed, our office should be contacted for supplementary design recommendations.

<u>Retaining Wall Foundation Design</u>

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 3 feet below proposed foundation bearing grade. 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

With the exception of fine sandy clays and silty clays, the on-site soils may be used to backfill the retaining walls. However, 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.

Weep holes or a footing drain will not be required on the inside of building stem walls.

6.9 Temporary Shoring Recommendations

Temporary shoring may be required during grading and/or foundation construction activities. The following recommendations assume that the retained soil heights will not exceed 10± feet and any surcharge loads will be setback at least 5 feet from the face of the shoring. If surcharge loads are located within this zone, the effect of these loads upon the shoring system must be considered by the shoring engineer.

Lateral Earth Pressures

It is assumed that the soil behind the shoring system will be relatively level. If sloping backfill is anticipated, the geotechnical engineer should be contacted to provide additional loading information to adequately address these loads. It is assumed that the shoring will consist of either sheet piles or soldier piles and lagging. The shoring may be a braced design or a cantilever design. Plate 3, enclosed in Appendix A of this report, illustrates the lateral earth pressure distributions for both cantilevered shoring and restrained (braced) shoring. The earth pressures shown on Plate 3 are based on static conditions. As discussed previously, if surcharge loads are imposed upon the shoring, they must be considered by the shoring engineer. This should include surcharges related to automobile traffic, as well as surcharges imposed by adjacent building foundations, floor slabs or backslopes. The passive resistance value of the soil below the level of excavation may be assumed to be 300 lbs/ft^2, per foot of depth. Isolated soldier piles may be designed using a passive earth pressure of 430 lbs/ft^3. This assumes that the soldier piles are spaced at least four pile diameters apart.

Shoring Construction

If soldier piles are utilized, they should be spaced no closer than 4 times the nominal soldier pile diameter. The contractor should take all necessary provisions to assure firm contact between the retained soils and the shoring system. A 2-sack cement slurry may be used to fill voids where inadequate contact between the shoring system and the retained soils are observed.

If the shoring system will be designed as a cantilever wall, some deflection will occur. In order to develop the full active pressure, a deflection of 1± inch is expected to occur at the top of the shoring system. The design of the shoring system as well as the protection of adjacent improvements should take this deflection into consideration.

6.10 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 primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of sands, silty sands, and sandy silts. These soils are expected to provide good pavement support characteristics. R-value testing was outside the scope of work for this project. Based on their classification, these soils are assumed to possess an R-value of at least 40. Any fill material imported to the project site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering-controlled conditions. It is recommended that R-value testing be performed as part of a supplementary geotechnical investigation or after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the project site.

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

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 (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic (TI = 6.0)	Truck Traffic (TI = 7.0)	Truck Traffic (TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	4	6	7	8
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)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5	5½	6½
Compacted Subgrade (95% Relative Compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. Reinforcing within all pavements should be designed by the structural engineer. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness. The actual joint spacing and reinforcing of the Portland cement concrete pavements should be determined by the structural engineer.

7.0 GENERAL COMMENTS

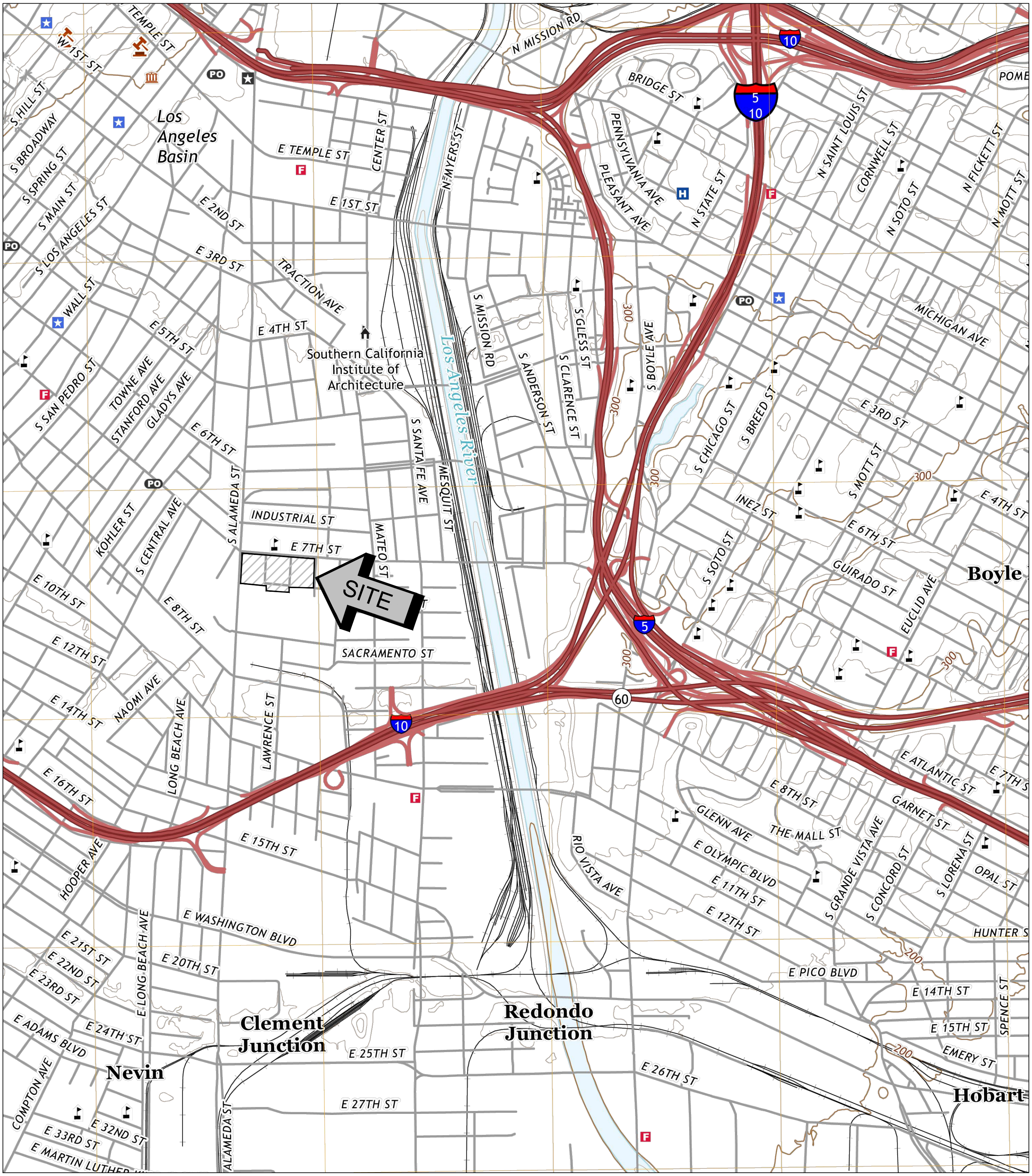
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other 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 project 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



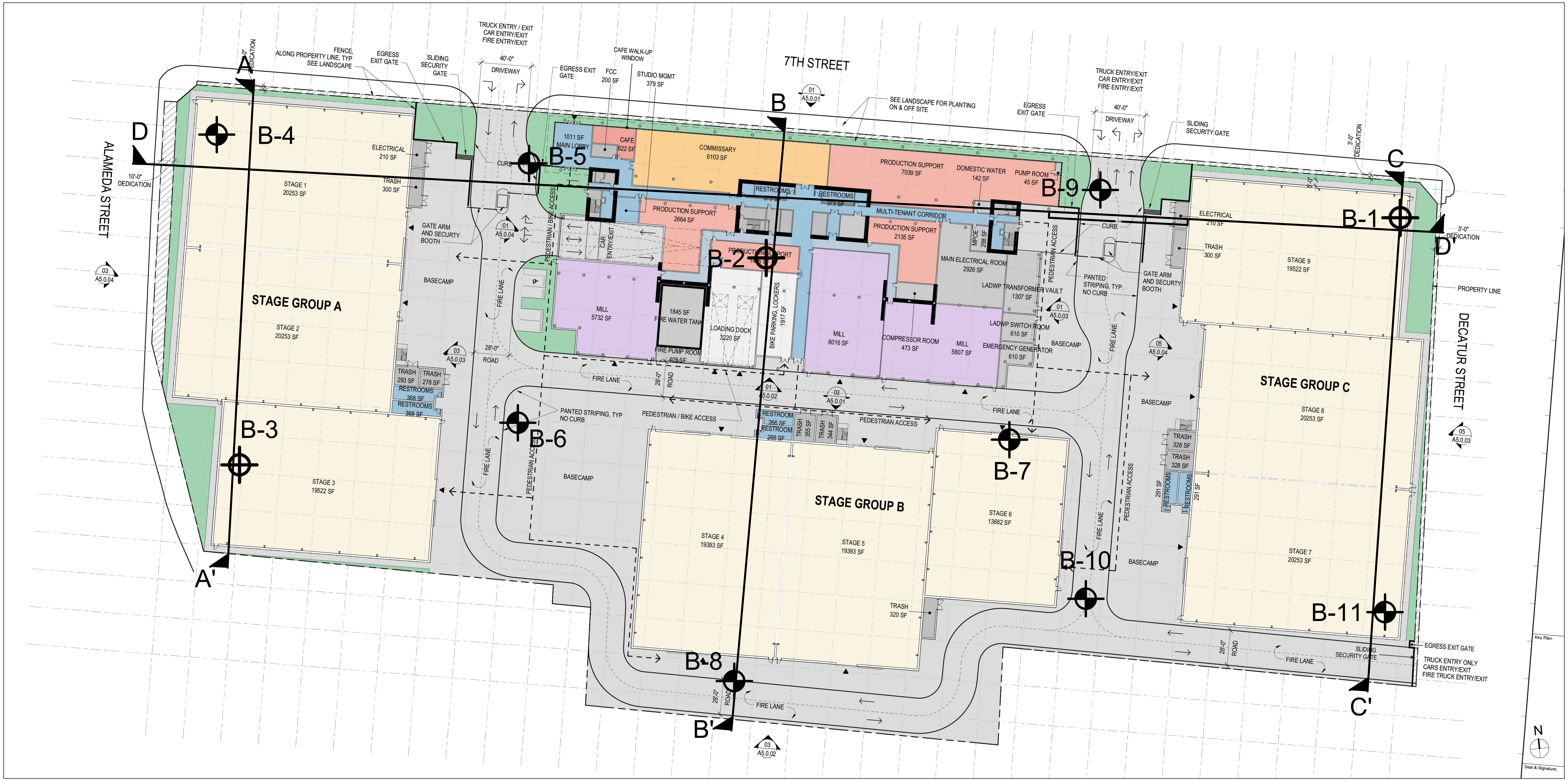
SOURCE: USGS TOPOGRAPHIC MAP OF THE LOS ANGELES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA, 2022.

SITE LOCATION MAP

PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT

LOS ANGELES, CALIFORNIA

SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: MK CHKD: GKM	
SCG PROJECT 20G243-4	
PLATE 1	



GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION (SCG PROJECT NO. 20G243-4)
- APPROXIMATE BORING LOCATION (SCG PROJECT NO. 20G243-1)
- A — A' APPROXIMATE CROSS SECTION LOCATION

NORTH

NOTE: CONCEPTUAL SITE PLAN PROVIDED BY RELATIVITY ARCHITECTS.

BORING AND CROSS SECTION LOCATION PLAN

ALAMEDA CROSSING DEVELOPMENT

LOS ANGELES, CALIFORNIA

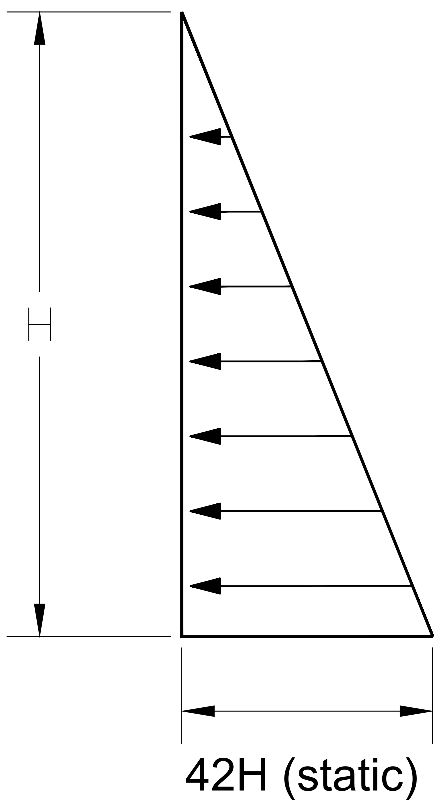
SCALE: 1" = 80'

DRAWN: DE
CHKD: RF

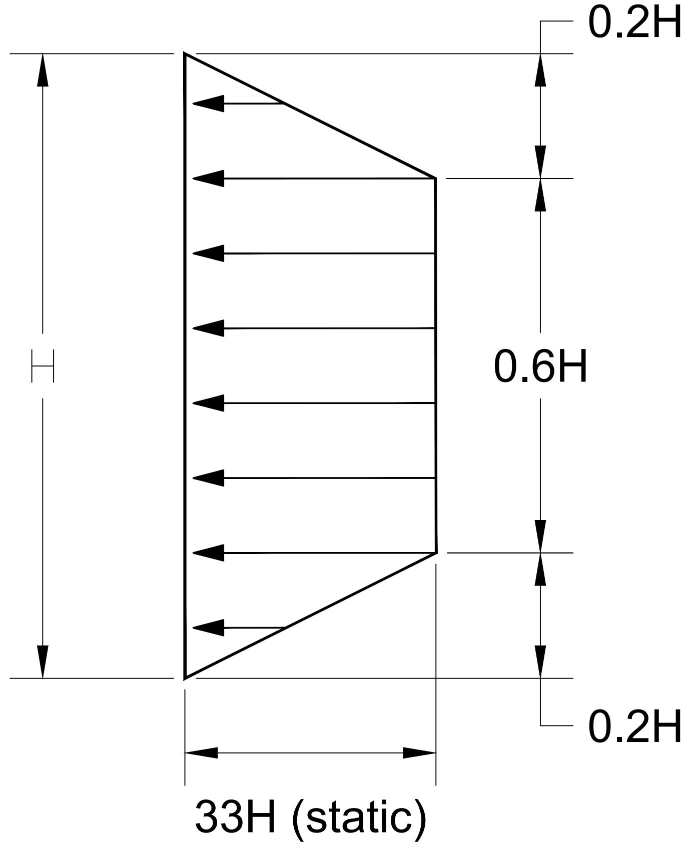
SCG PROJECT
20G243-4R2

PLATE 2

SOUTHERN CALIFORNIA GEOTECHNICAL



CANTILEVERED SHORING



BRACED SHORING

NOTES: THE LATERAL EARTH PRESSURES DEPICTED IN THIS DIAGRAM DO NOT INCLUDE ANY SURCHARGE LOADS.

IN ADDITION TO THE STATIC LOAD, A SEISMIC LOAD SHOULD BE APPLIED FOR PERMANENT SHORING.

ACTIVE PRESSURE DIAGRAM FOR SHORING

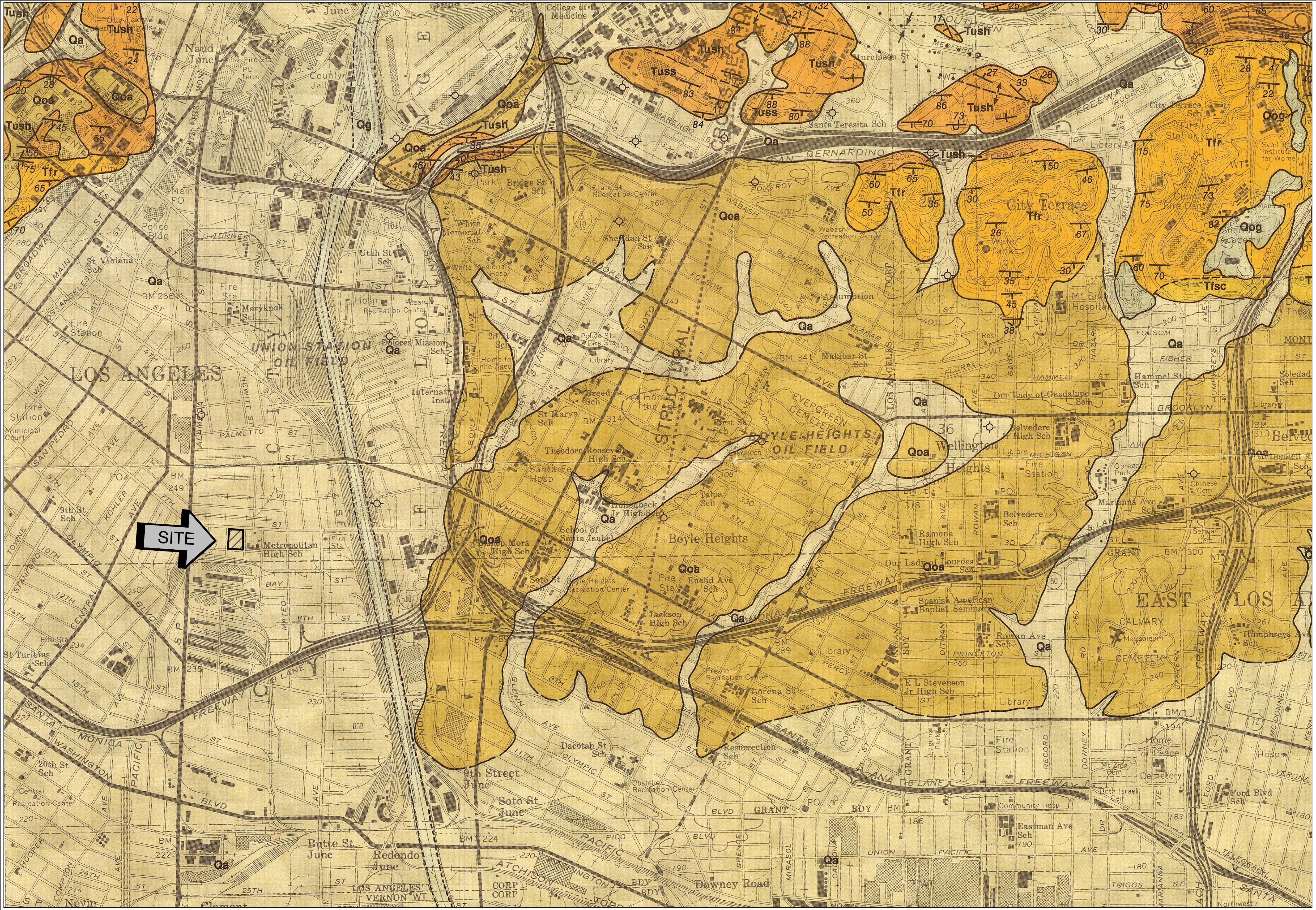
NOT TO SCALE

DRAWN: OS
CHKD: GKM

SCG PROJECT
20G243-4

PLATE 3

SOUTHERN CALIFORNIA GEOTECHNICAL



LOS ANGELES QUADRANGLE
LEGEND

GEOLOGY—LOS ANGELES QUADRANGLE

QUATERNARY

Holocene

Pleistocene

af / Qg / Qa

SURFICIAL SEDIMENTS
- **af** artificial fill
- **Qg** stream channel deposits of gravel, sand and silt
- **Qa** alluvium: unconsolidated floodplain deposits of silt, sand and gravel

Qls

LANDSLIDE DEBRIS
- **Qls** landslide of Elysian Park

Qoa / Qof / Qog

OLDER DISSECTED SURFICIAL SEDIMENTS
- **Qoa** remnants of older weakly consolidated alluvial deposits of gravel, sand and silt
- **Qof** alluvial fan gravel and sand
- **Qog** elevated remnants of older alluvial gravel and fanglomerate deposits, weakly indurated

—UNCONFORMITY—

Pliocene

Tfsc / Tfr

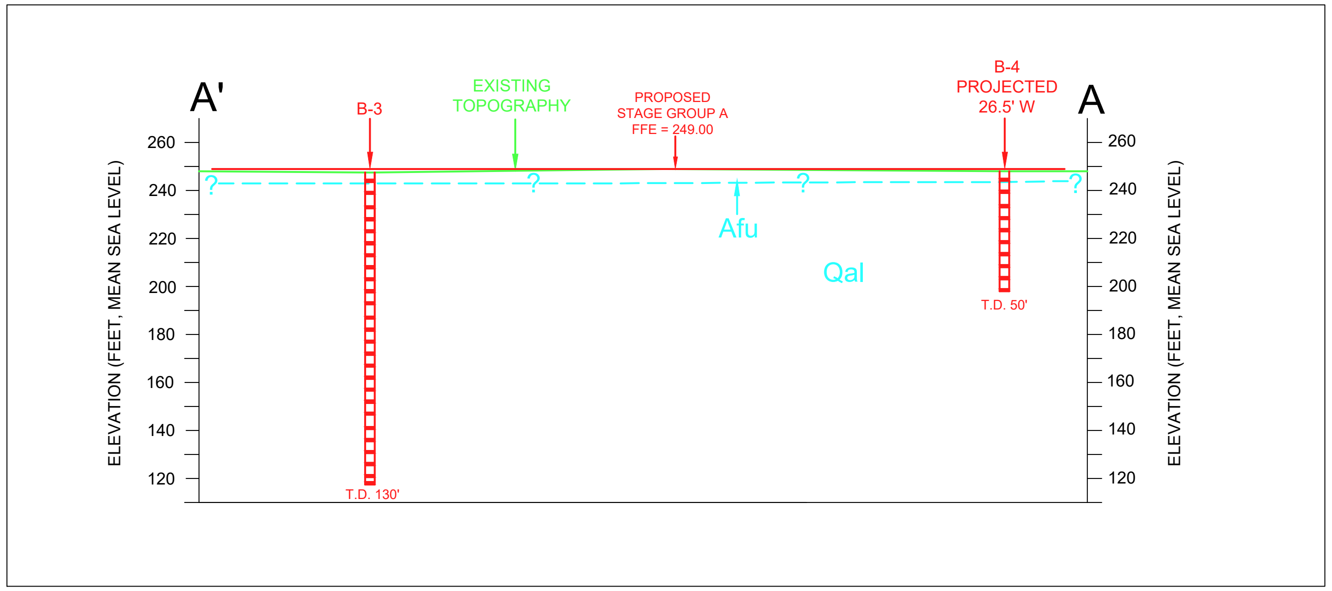
FERNANDO FORMATION
marine and nonmarine; early to late (?) Pliocene age
- **Tfsc** nonmarine sandstone and conglomerate: tan, semi-friable sandstone and bedded arkosic conglomerate of mostly granitic pebbles and cobbles in sandstone matrix; late (?) Pliocene age
- **Tfr** marine claystone (Repetto Member): gray, soft, vaguely bedded claystone, early Pliocene age (Repettian Stage) in lower part, middle and late (?) Pliocene age (Wheelerian Stage) in upper third

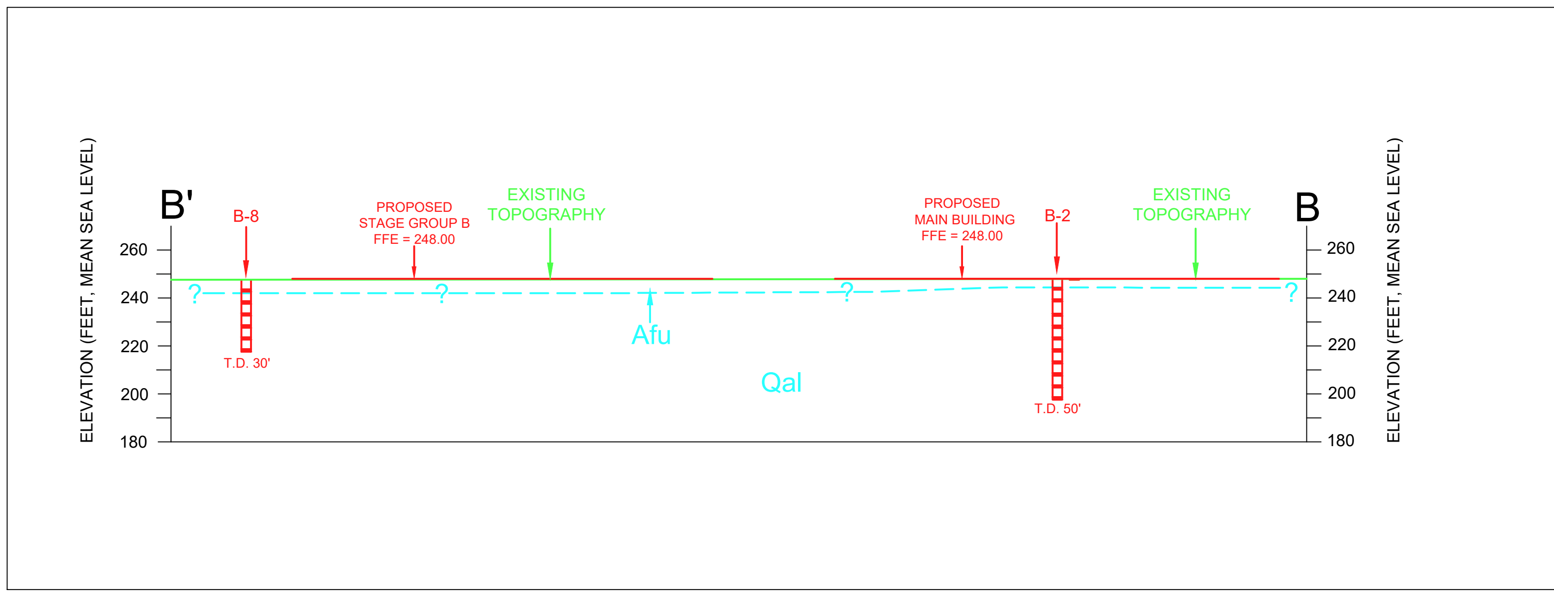
Tuss / Tush

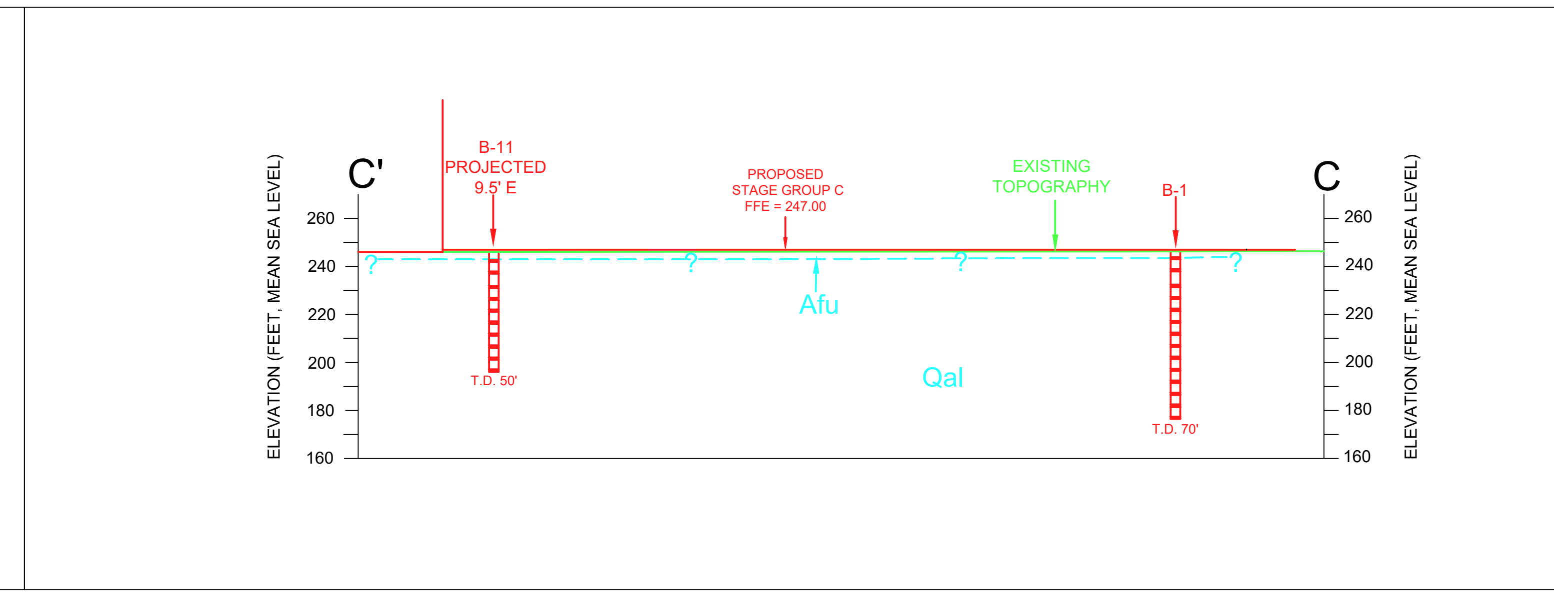
UNNAMED SHALE
(Yorba Member of Puente Formation of Schoellhamer, et al., 1965; included in Puente Formation by Lamar, 1970, and Weber, 1980; equivalent to Upper Modelo Formation of Hoots, 1931, in Santa Monica Mtns., and to Sisquoc Shale of Ventura Basin)
marine; late Miocene age ("Delmontian" and late Mohnian Stages)
- **Tush** gray to light brown, thin bedded, silty clay shale, locally contains scattered large calcareous nodules; in places contains thin interbeds of fine-grained sandstone; lower part locally contains thin lenses of light tan, platy, semi-siliceous or diatomaceous shale
- **Tuss** light gray to tan semi-friable sandstone, with thin interbeds of silty shale

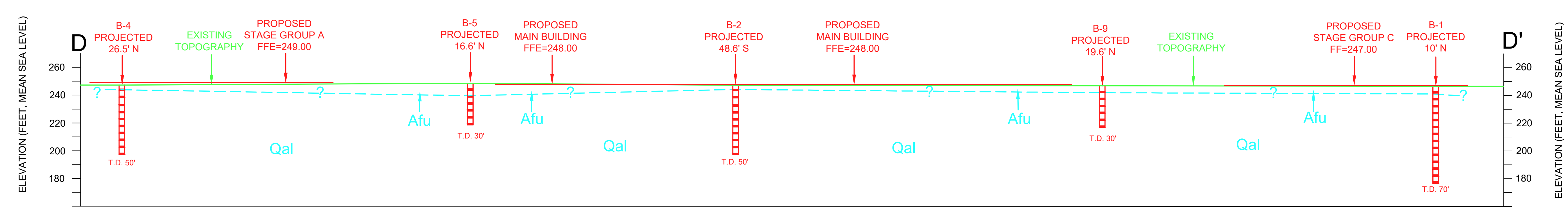
SOURCE: "GEOLOGIC MAP OF THE LOS ANGELES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA, BY THOMAS W. DIBBLEE, JR., 1989."

GEOLOGIC MAP	
PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT	
LOS ANGELES, CALIFORNIA	
SCALE: 1" = 2000'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: RB CHKD: DK	
SCG PROJECT 20G243-4	
PLATE 4	









GEOLOGICAL UNITS

Afu - UNDOCUMENTED ARTIFICIAL FILL

Qal - ALLUVIUM

GEOTECHNICAL LEGEND

CONTACT QUERIED WHERE UNCERTAIN

PROPOSED BUILDING ELEVATION

EXISTING ELEVATION

GEOTECHNICAL CROSS SECTIONS

ALAMEDA CROSSING DEVELOPMENT

LOS ANGELES, CALIFORNIA

SCALE: 1" = 80'

DRAWN: DE
CHKD: RF

SCG PROJECT
20G243-4R2

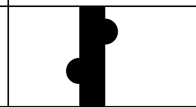
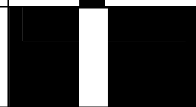
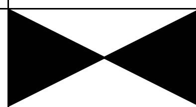
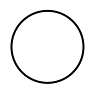
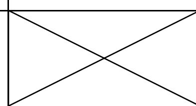
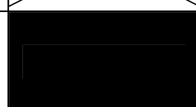
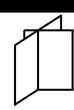
PLATE 5

SoCalGeo

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

BORING NO. B-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 12/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 45 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
			ASPHALT: 2± inches Asphaltic Concrete with no discernible Aggregate Base							
			FILL: Gray Brown to Dark Gray Brown Silty fine to coarse Sand, little fine to coarse Gravel, mottled, very loose-moist		9					
	4		ALLUVIUM: Gray Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, trace Silt, very loose to loose-damp to moist	95	8					
5	10		Light Gray Brown to Gray Brown fine to coarse Sand, trace fine Gravel, trace Silt, loose to medium dense-dry to damp	94	3					
	24			108	2					
10	35			107	2					
15	35			102	2					
			Dark Brown fine Sandy Silt, little Iron Oxide staining, medium dense-very moist							
20	20			93	29					
			Gray Brown Silty fine Sand, trace to little medium to coarse Sand, medium dense-very moist							
			Gray Brown fine to coarse Sand, trace fine Gravel, trace Silt, very dense-damp							
25	50/4"			111	3					
			Gray Brown fine Sand, trace medium Sand, little Silt, dense-damp to moist							
30	45			98	7					
			Gray to Gray Brown fine to coarse Sand, little fine to coarse Gravel, trace Silt, very dense-dry to damp							
35	50/5"				2					
	50/5"				3					

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 12/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 45 feet
READING TAKEN: At Completion

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
					Gray to Gray Brown fine to coarse Sand, little fine to coarse Gravel, trace Silt, very dense-dry to damp							
45	X	50/5"					3					
50	X	50/5"					3					
55	X	69					4					
60	X	50/5"					3					
65	X	50/5"					4					
					Gray fine Sand, little Silt nodules, very dense-moist							
70	X	57					14					
					Boring Terminated at 70'							

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 12/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 28 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					CONCRETE: 8± inches Portland Cement Concrete with 6± inches of Aggregate Base							
		22			FILL: Brown Silty fine Sand, trace medium to coarse Sand, mottled, medium dense-damp	111	5					EI = 3 @ 0 to 5 feet
		26			ALLUVIUM: Gray fine Sand, trace to little Silt, medium dense-damp	88	2					
5		5			Light Gray Brown fine to coarse Sand, trace Silt, trace to little fine Gravel, medium dense-dry to damp	106	12					
		3			Dark Brown Silty fine to coarse Sand, trace fine Gravel, very loose-moist to very moist	90	12					
		6			Light Gray Brown fine to medium Sand, trace coarse Sand, trace Silt, loose-dry to damp	103	2					
10					Gray Brown to Dark Gray Brown Silty fine Sand, trace Iron Oxide staining, loose-dry							
15		20			Light Gray Brown to Gray Brown fine to coarse Sand, trace fine Gravel, medium dense-dry to damp	106	2					
20		42			Gray Brown fine to medium Sand, trace coarse Sand, trace Silt, medium dense-damp to moist	102	6					
25		50/5"			Gray Brown fine to coarse Sand, trace to little fine Gravel, trace Silt, very dense-dry to damp	119	2					
30		50/5"				103	2					
35		58					3					
		50/3"					2					

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 12/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 28 feet
READING TAKEN: At 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
					Gray Brown fine to coarse Sand, trace to little fine Gravel, trace Silt, very dense-dry to damp							
	X	50/5"					2					
45												
	X	50/5"					2					
50												
					Boring Terminated at 50'							

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 12/10/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 125 feet
READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					ASPHALT: 7± inches Asphaltic Concrete with no discernible Aggregate Base							
		50/5"			FILL: Brown Silty fine Sand, trace to little medium to coarse Sand, little fine Gravel, mottled, dense to very dense-moist	113	8					
		50/5"			@ 3', little coarse Gravel	119	8					
5		22			ALLUVIUM: Gray Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, medium dense-damp	106	5					
		12			Light Gray fine Sand, trace Silt, trace medium to coarse Sand, loose to medium dense-dry to damp	96	2					
10		22			@ 9', little medium to coarse Sand, trace fine Gravel	100	2					
15		57			Light Gray to Gray fine to coarse Sand, little fine to coarse Gravel, dense-dry to damp	119	2					
20		46			Gray Brown fine to coarse Sand, trace to little Silt, trace fine Gravel, dense-damp	115	3					
25		20			Gray Brown fine Sand, trace medium to coarse Sand, trace to little Silt, medium dense-damp		3					
30		37			Light Gray fine to coarse Sand, trace to little fine Gravel, trace Silt, dense-dry to damp		2					
35		38			Gray fine Sand, trace medium to coarse Sand, trace fine Gravel, trace to little Silt, dense-dry to damp		2					
		50/5"			Gray to Gray Brown fine to coarse Sand, trace to little fin to coarse Gravel, trace to little Silt, very dense-dry to damp		2					

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

TEST BORING LOG

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 12/10/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 125 feet
READING TAKEN: At 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
					Gray to Gray Brown fine to coarse Sand, trace to little fin to coarse Gravel, trace to little Silt, very dense-dry to damp							
	X	50/5"					3					
45												
	X	50/5"					1					
50												
	X	50/3"					3					
					Gray Brown Silty Clay, trace fine Sand, trace Calcareous veining, hard-very moist							
55			2.5				35					
					Light Gray to Gray fine to medium Sand, trace coarse Sand, trace Silt, very dense-dry to damp							
	X	63					3					
60												
					Light Gray to Gray fine Sand, trace medium Sand, trace to little Silt, very dense-damp							
	X	69					4					
65												
					Gray fine to coarse Sand, little fine Gravel, very dense-damp							
	X	50/5"					4					
70												
					Light Gray fine to medium Sand, trace coarse Sand, trace fine Gravel, very dense-damp							
	X	50/5"					3					
75					Gray Brown fine Sandy Silt, trace Clay, trace Iron Oxide staining with 3" Silt nodules, dense-very moist							
	X	38					23					

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 12/10/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 125 feet
READING TAKEN: At Completion

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
					Gray Brown fine Sandy Silt, trace Clay, trace Iron Oxide staining with 3" Silt nodules, dense-very moist							
					Gray Brown Silty fine Sand, trace Calcareous nodules, very dense-moist to very moist							
85	X	50/5"					12					
					Gray fine to coarse Sand, trace to little Silt, little fine to coarse Gravel, very dense-dry to damp							
90	X	50/3"					2					
95					Gray to Dark Gray Silt, trace fine Sand, very dense-moist to very moist							
	X	50/3"					21					
100					Light Gray fine Sand, little Silt, very dense-damp		6					
105					Gray Silty fine to coarse Sand, little fine to coarse Gravel, very dense-damp to moist							
110	X	50/5"					5					
115												
	X	50/5"			@ 118.5', trace Silt nodules		12					

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California
DRILLING DATE: 12/10/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon
WATER DEPTH: Dry
CAVE DEPTH: 125 feet
READING TAKEN: At 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
					Gray Silty fine to coarse Sand, little fine to coarse Gravel, very dense-damp to moist							
125												
	X	50/5"			@ 128.5', trace Clay		10					
130												
					Boring Terminated at 130'							

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ASPHALT: 2± inches Ashaltic Concrete with 5 inches of Aggregate Base over 9 inches of Portland Cement Concrete							
		9			FILL: Gray Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, loose-damp	106	4					
		13			FILL: Gray Brown fine to medium Sand, trace coarse Sand, trace Silt, loose-damp	109	4					
5		22			ALLUVIUM: Light Brown fine to coarse Sand, trace fine Gravel, medium dense-dry to damp	103	2					
		16			@ 7 feet, trace to little fine to coarse Gravel	108	3					
10		20			@ 9 feet, trace Silt	119	2					
15		40				107	3					
20		24				106	3					
					Dark Brown Silty fine to medium Sand, trace Clay, trace coarse Sand, trace fine Gravel, medium dense-moist							
25		38				115	15					
					Brown fine to coarse Sand, trace to little Silt, trace fine Gravel, medium dense to dense-damp							
30		45				119	6					
					Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, dense-damp							
		32					5					

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	(Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, dense-damp							
					Light Gray Brown fine to coarse Sand, trace Silt, dense-dry to damp							
	X	33					2					
40												
	X	46					3					
45												
	X	50/3"			@ 48½ feet, occasional Cobbles, very dense		3					
50												
					Boring Terminated at 50' and grouted at completion							

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

TEST BORING LOG **PLATE B-4b**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					CONCRETE: 5± inches Portland Cement Concrete with 9 inches of Aggregate Base over 7 inches of Portland Cement Concrete							
	Bulk						5					
					FILL: Brown Silty fine to medium Sand, little coarse Sand, trace fine Gravel, loose-damp							
	Bulk						6					
5					FILL: Dark Brown Clayey Silt, trace iron oxide staining, medium stiff-very moist							
	X	9	1.0				40					
					ALLUVIUM: Light Brown fine to coarse Sand, trace to little fine to coarse Gravel, trace Silt, medium dense-dry to damp							
	X	15					8					
10												
	X	20					2					
15												
	X	20					3					
20												
	X	48			@ 23½ feet, little fine to coarse Gravel, dense		3					
25												
	X	46					3					
30					Gray Brown Silt, little fine Sand, little iron oxide staining, dense-very moist		32					
					Brown Silty fine to medium Sand, little coarse Sand, trace fine to coarse Gravel, dense-damp							
	X	41					3					
					Boring Terminated at 32½' and grouted at completion							

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

TEST BORING LOG **PLATE B-5**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ASPHALT: 4± inches Ashaltic Concrete with 6 inches of Aggregate Base							
	X	24			FILL: Dark Gray Brown Gravelly fine to coarse Sand, little Silt, medium dense-damp		4					
	X	26			ALLUVIUM: Brown Silty fine to coarse Sand, trace fine to coarse Gravel, medium dense to dense-damp to moist		12			24		
5												
	X	39			@ 6 feet, little fine to coarse Gravel		7					
	X	39					8			17		
10												
	X	50/2"			@ 13½ feet, very dense-damp		3					
15												
					Light Brown fine to medium Sand, trace fine Gravel, trace coarse Sand, trace Silt, medium dense to dense-damp							
	X	24					3			4		
20												
	X	30			@ 23½ feet, trace fine to coarse Gravel, trace Silt		4					
25												
	X	24			@ 28½ feet, medium dense		3					
30												
					Boring Terminated at 30' and grouted at completion							

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

TEST BORING LOG **PLATE B-6**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ASPHALT: 2± inches Ashaltic Concrete with 8 inches of Portland Cement Concrete							
	Bulk				FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, loose-damp		3					
	Bulk				ALLUVIUM: Brown fine to medium Sand, little Silt, trace coarse Sand, trace fine Gravel, medium dense-damp		5					
5	X	15			Brown Silty fine to medium Sand, little coarse Sand, little fine to coarse Gravel, loose to medium dense-dry to damp		3					Disturbed Sample
	X	12					2					Disturbed Sample
10	X	27										No Sample Recovery
					Light Brown fine to coarse Sand, little fine to coarse Gravel, trace Silt, dense to very dense-dry to damp							
15	X	65/10"				116	2					
20	X	29			@ 19 feet, little Silt, medium dense	111	2					
25	X	47			@ 24 feet, dense	114	2					
					Brown Gravelly fine to coarse Sand, trace to little Silt, occasional Cobbles, dense-dry to damp							
30	X	51				107	2					
					Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, dense-dry to damp							
	X	39					2					

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At 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
					Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, dense-dry to damp							
	X	30			@ 38½ feet, little fine to coarse Gravel, occasional Cobbles, medium dense to dense		4					
40												
	X	50/3"			@ 43½ feet, occasional Cobbles, very dense		2					
45												
	X	50/5"			@ 48½ feet, occasional Cobbles, very dense		2					
50												

Boring Terminated at 50' and grouted at completion

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/10/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					CONCRETE: 6½± inches Portland Cement Concrete with 3 inches of Aggregate Base							
	Bulk				FILL: Dark Gray Brown Silty fine to medium Sand, trace to little coarse Sand, trace fine Gravel, clay brick, loose-damp		8					
	Bulk				@ 3 feet, trace Clay, trace fine to coarse Gravel		8					
5	Ring	13				108	8					
					ALLUVIUM: Light Brown fine to medium Sand, trace Silt, loose to medium dense-damp to moist							
	Ring	15				106	7					
	Ring	18			@ 9 feet, litte coarse Sand, little fine to coarse Gravel	107	6					
10												
	SPT	23					4					
15												
					Brown Gravelly fine to coarse Sand, trace Silt, occasional Cobbles, dense to very dense-dry to damp							
	SPT	31					2					
20												
	SPT	74					4					
25												
	SPT	80					3					
30					Boring Terminated at 30'							

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/8/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ASPHALT: 7± inches Ashaltic Concrete with 5 inches of Portland Cement Concrete over 3 inches of Aggregate Base							
		18			FILL: Dark Brown Silty fine to medium Sand, trace coarse Sand, mottled, medium dense-moist	105	11					
					FILL: Gray Brown Silty fine Sand, loose-moist							
		11			ALLUVIUM: Light Brown fine to medium Sand, trace fine to coarse Gravel, loose-damp to moist	95	11					
5		12				92	3			1		
		14				95	3					
		20			Gray Brown fine to coarse Sand, some Silt, trace fine Gravel, medium dense-damp		4			11		Disturbed Sample
10												
					Light Brown Silty fine Sand, little medium Sand, dense-moist							
		39				111	12			25		
15					Light Brown Gravelly fine to coarse Sand, little Silt, medium dense to dense-dry to damp							
		18					2			4		
20												
		24					3					
25												
		26					4					
30					Boring Terminated at 30'							

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

BORING NO. B-10

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California
DRILLING DATE: 2/13/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek
WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION (SURFACE ELEVATION: --- MSL)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
				CONCRETE: 8± inches Portland Cement Concrete with 8 inches of Portland Cement Concrete							
	Grab			FILL: Dark Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, loose-moist		11					
	Grab			ALLUVIUM: Brown fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, medium dense-dry to damp		5					
5	Ring	19			99	2					
	Ring	15		Gray Brown Silty fine Sand, loose to medium dense-moist	98	8					
	Ring	22		Brown Gravelly fine to coarse Sand, medium dense-dry to damp	110	2					
10											
				Brown fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, medium dense-damp							
	SPT	18				3					
15											
				Dark Brown Silty fine to medium Sand, with interbeded layers of Clayey Silt, medium dense-moist to very moist							
	SPT	12				16					
20											
				Light Brown Gravelly fine to coarse Sand, trace Silt, occasional Cobbles, medium dense to dense-damp							
	SPT	28				3					
25											
	SPT	39				4					
30				Boring Terminated at 30'							

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-11

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/8/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION — SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					CONCRETE: 10± inches Portland Cement Concrete with 4 inches of Aggregate Base							
	X	24			FILL: Dark Brown Silty fine to medium Sand, little coarse Sand, trace to little fine to coarse Gravel, clay brick fragments, mottled, medium dense-moist		8					Disturbed Sample
	X	18			ALLUVIUM: Light Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, trace Silt, medium dense-damp	101	4					
5	X	13			Gray Brown Silty fine Sand, trace medium to coarse Sand, trace fine root fibers, loose-dry to damp	98	2					
	X	17			Light Brown fine to medium Sand, trace coarse Sand, trace fine root fibers, medium dense-dry	90	1					
10	X	15			Gray Brown fine to medium Sand, loose to medium dense-damp	92	4					
15	X	44			Light Gray Brown Gravelly fine to coarse Sand, medium dense-dry	120	1					
20	X	41			Gray Brown Silty fine to coarse Sand, little fine to coarse Gravel, medium dense to dense-dry to damp	92	5					
25	X	48				113	2					
30	X	48			Light Brown fine to coarse Sand, little fine to coarse Gravel, little Silt, dense-very moist	99	16					
	X	50/4"			Gray Brown Gravelly fine to coarse Sand, little Silt, occasional Cobbles, very dense-damp		4					

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-11

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/8/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

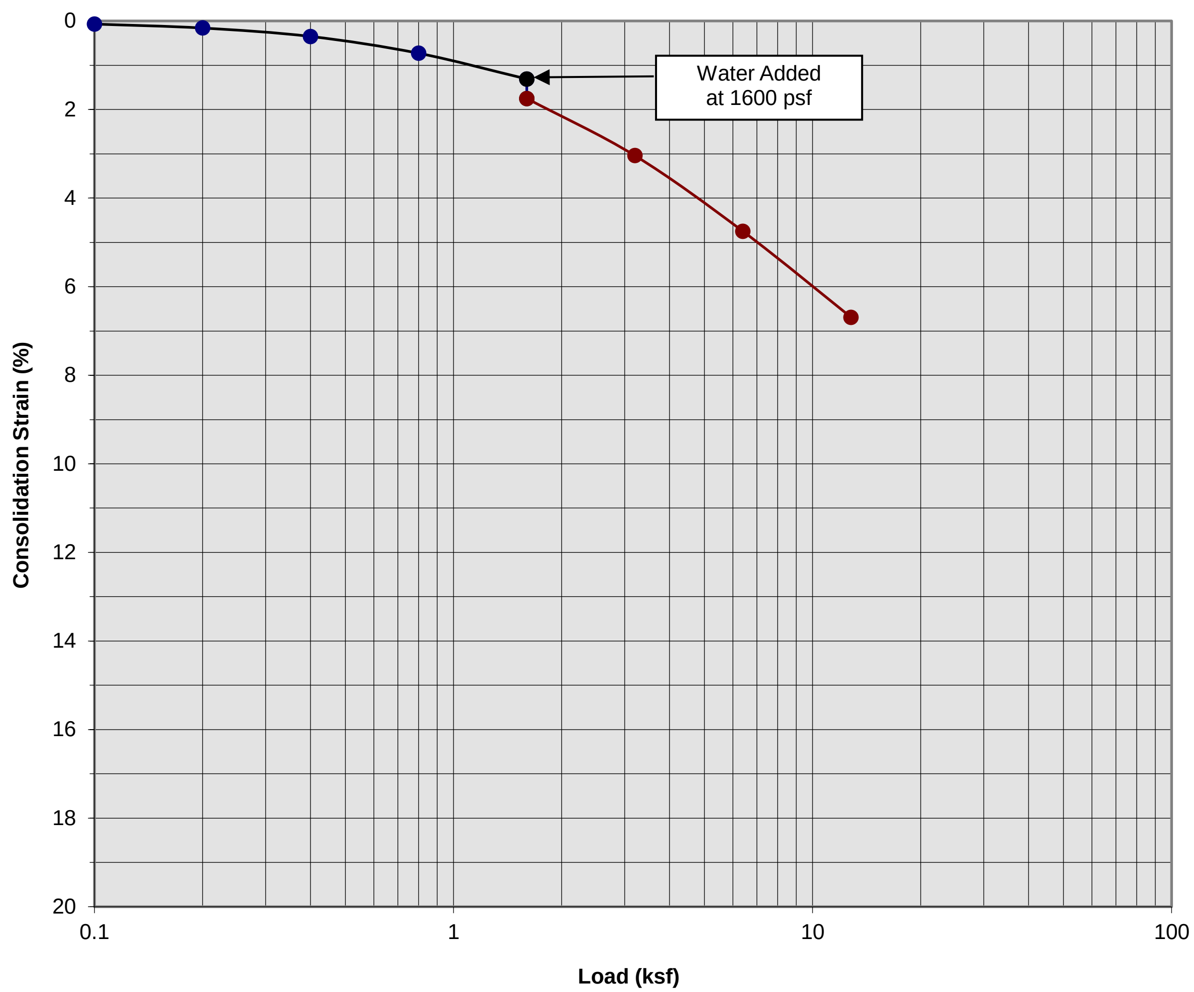
WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

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
					Gray Brown Gravelly fine to coarse Sand, little Silt, occasional Cobbles, very dense-damp							
	X	50/5"					3					
40												
					Dark Gray Brown Silty fine to coarse Sand, little fine to coarse Gravel, occasional Cobbles, very dense-damp							
	X	52					4					
45												
	X	57					3					
50												
					Boring Terminated at 50'							

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

APPENDIX C

Consolidation/Collapse Test Results

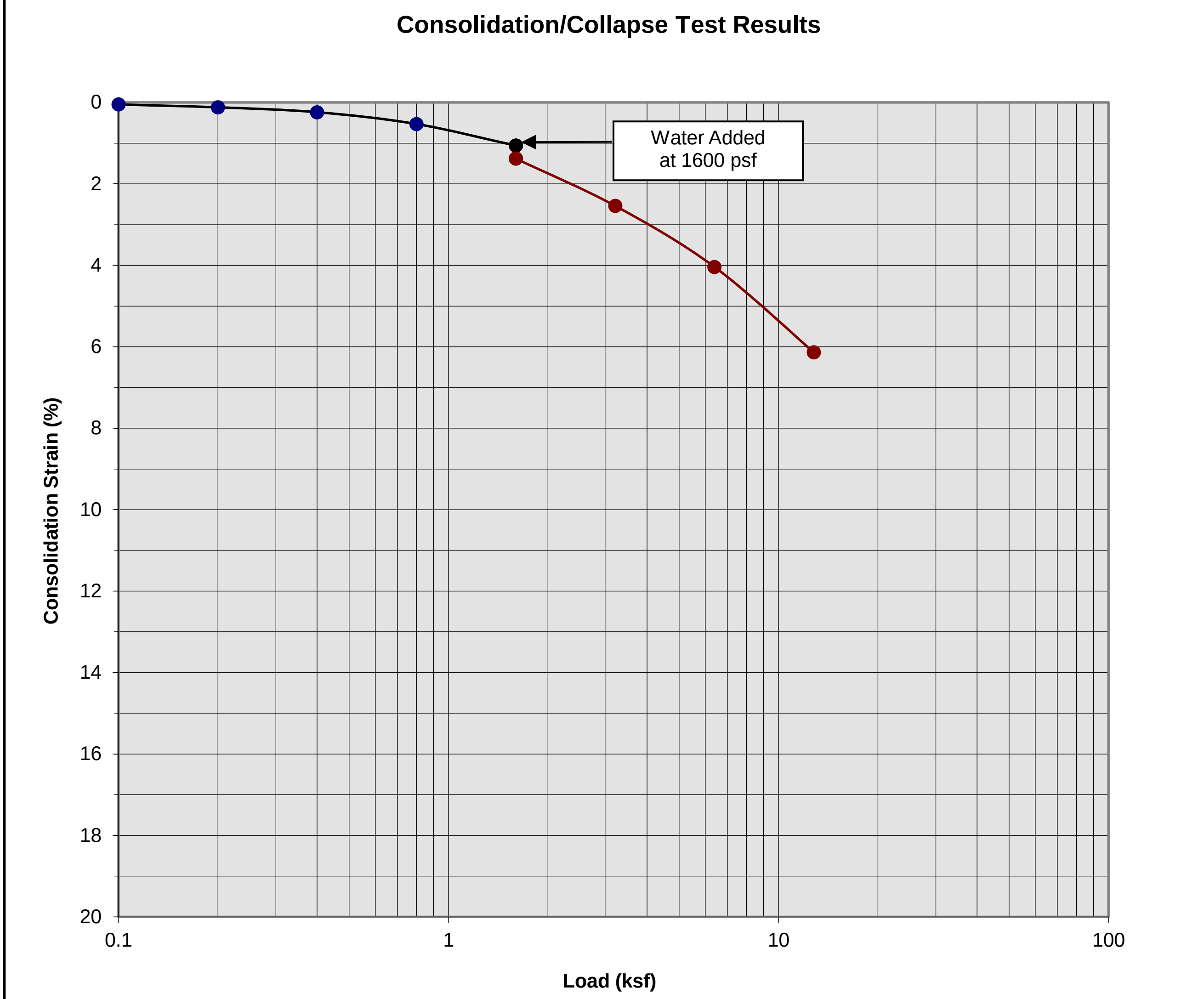


Classification: Dark Brown Silty fine to coarse Sand, trace fine Gravel

Boring Number:	B-2	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	5 to 6	Initial Dry Density (pcf)	106.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.44

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



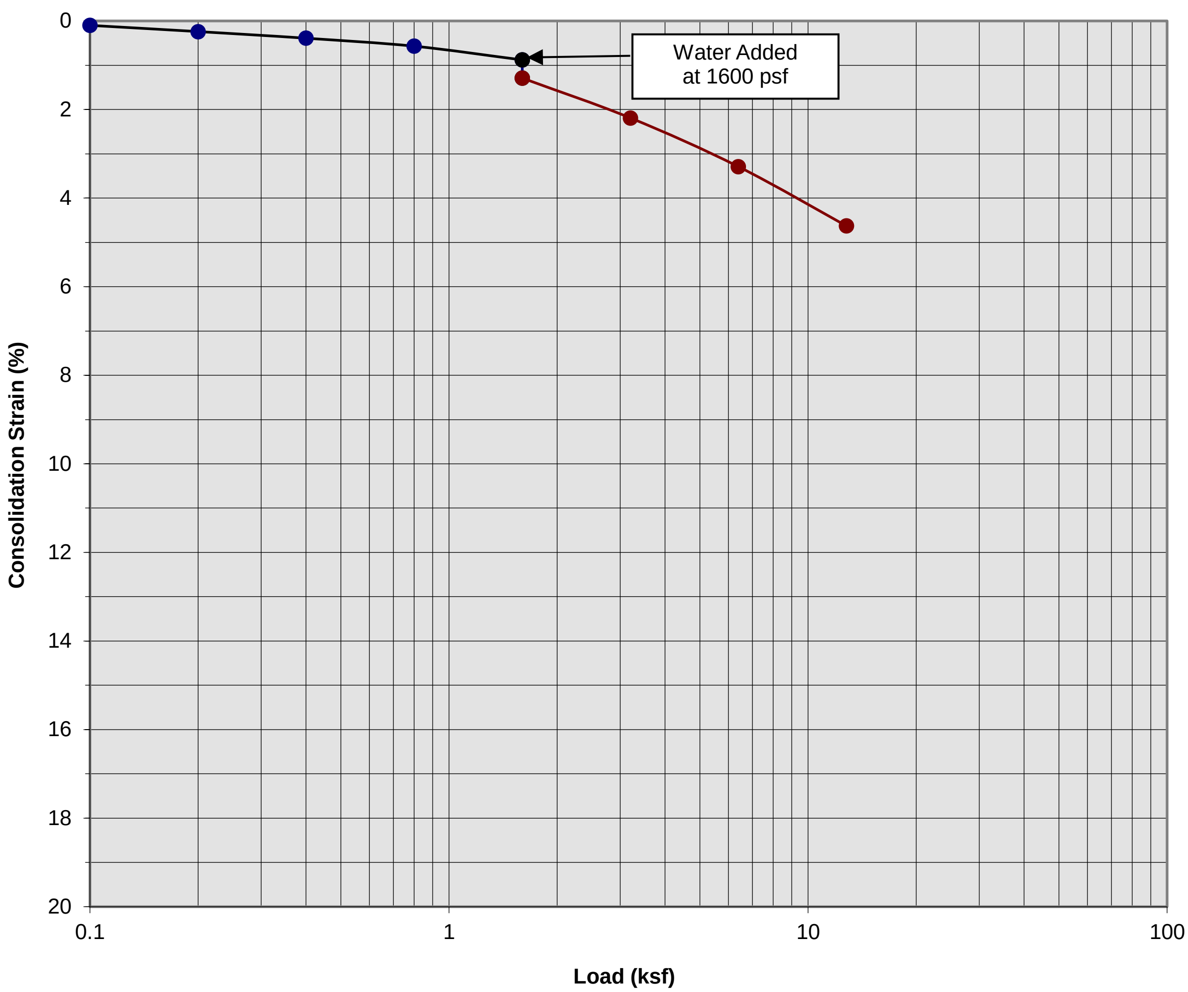
Classification: Dark Brown Silty fine to coarse Sand, trace fine Gravel

Boring Number:	B-2	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	7 to 8	Initial Dry Density (pcf)	89.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.32

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



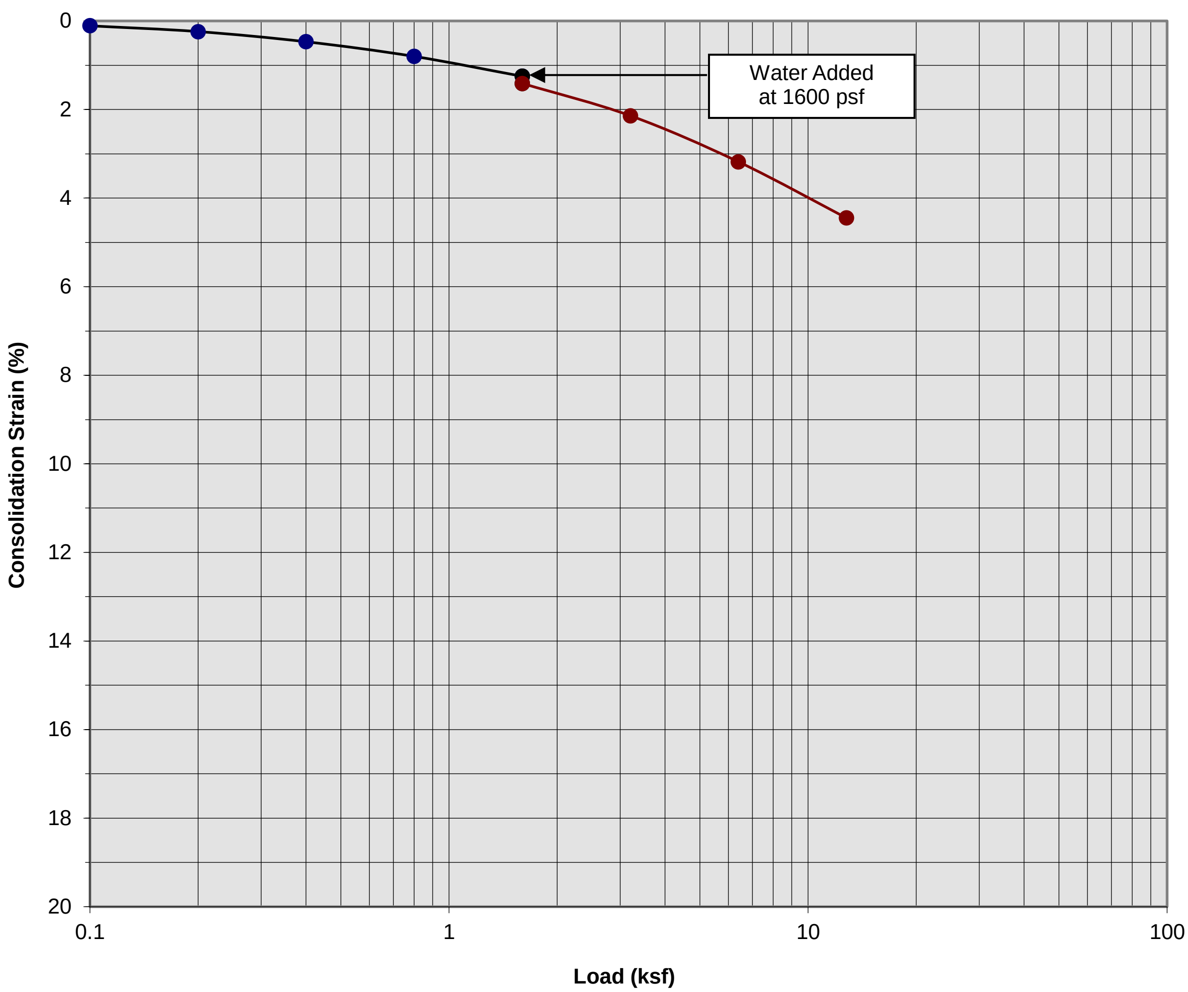
Classification: Gray Brown to Dark Gray Brown Silty fine Sand

Boring Number:	B-2	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	9 to 10	Initial Dry Density (pcf)	102.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	106.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.41

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

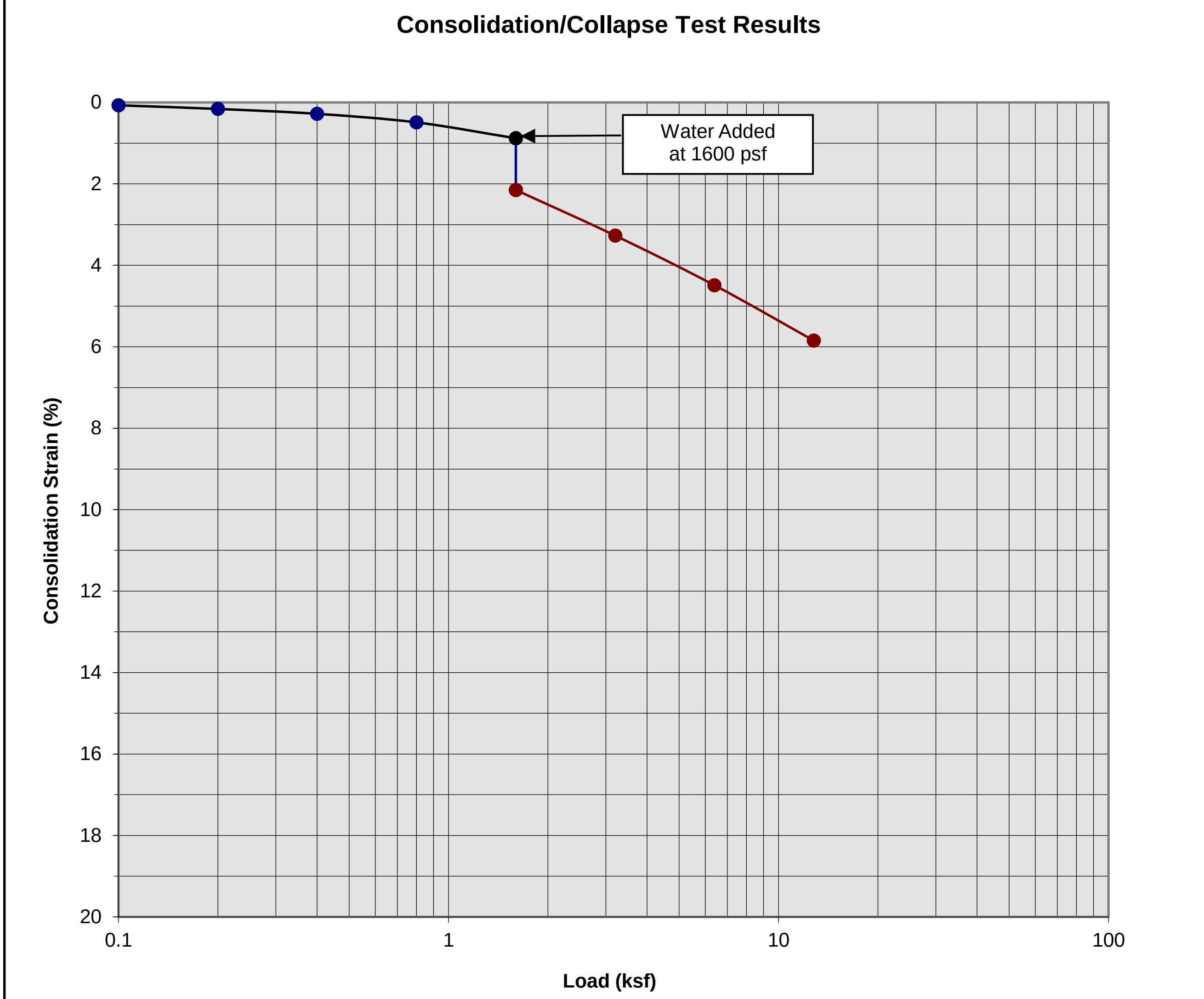


Classification: Light Gray Brown to Gray Brown fine to coarse Sand, trace Silt

Boring Number:	B-2	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	14 to 15	Initial Dry Density (pcf)	105.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.16

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

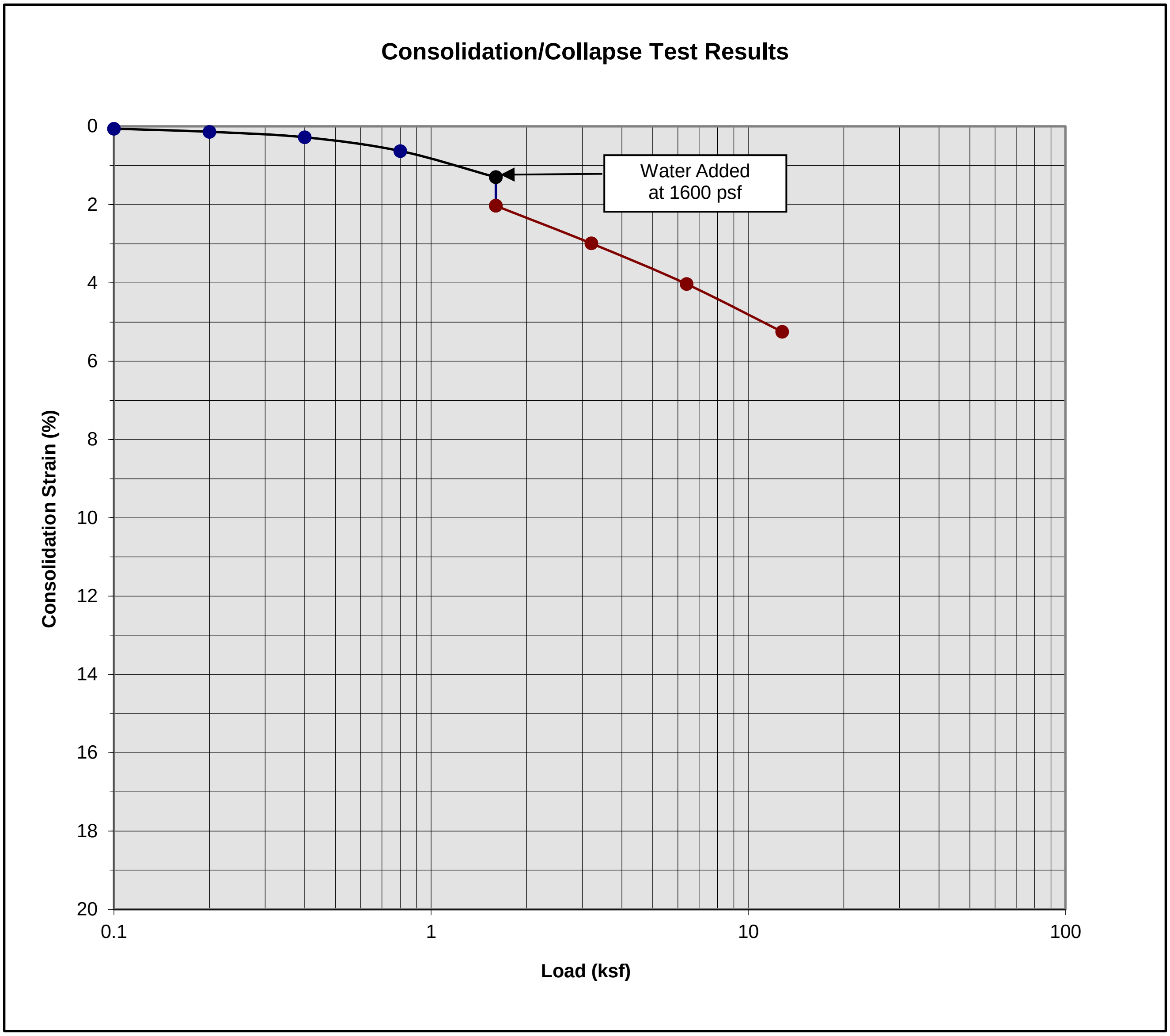


Classification: Gray Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel

Boring Number:	B-3	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	105.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.27

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

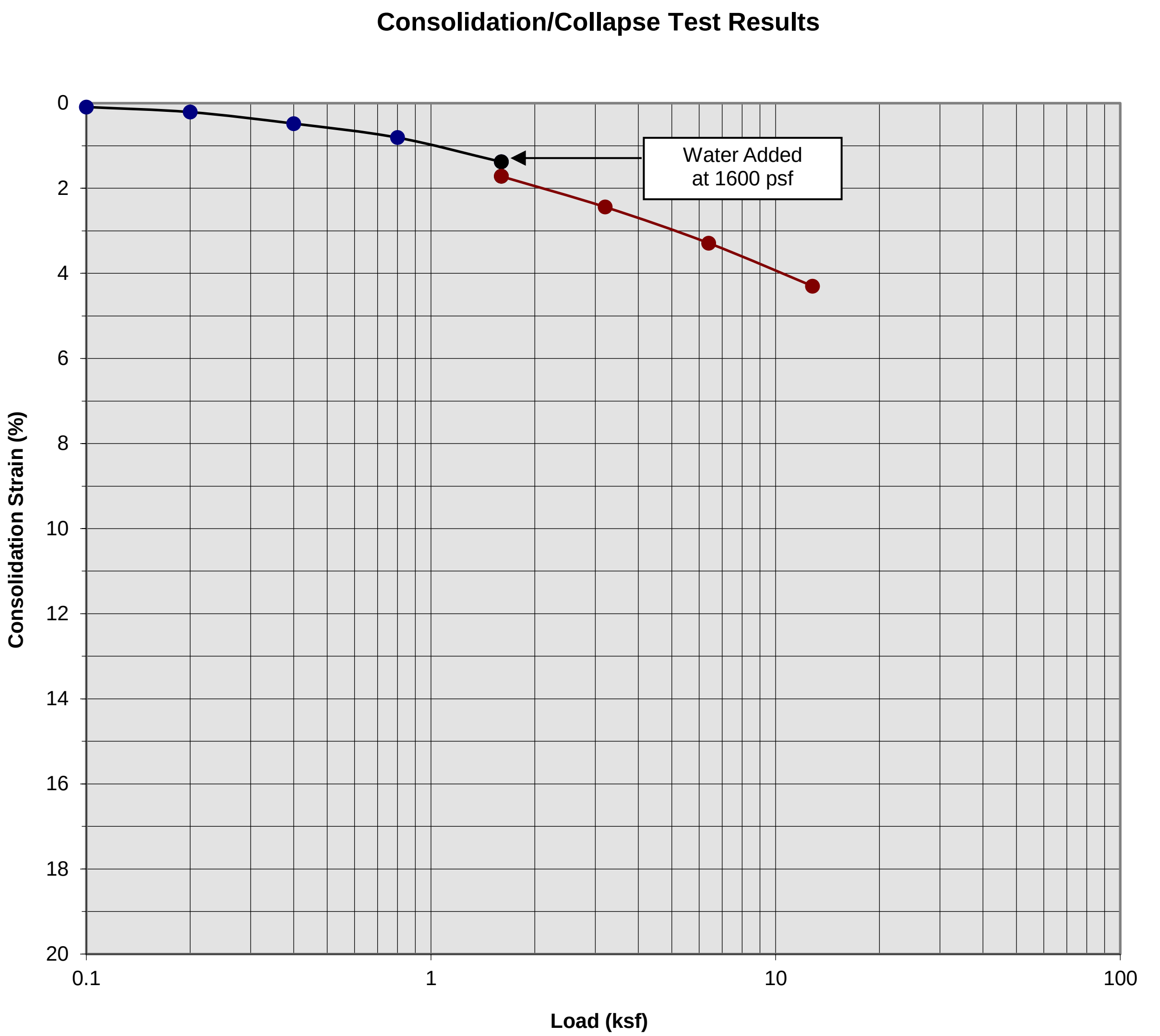


Classification: Light Gray fine Sand, trace medium to coarse Sand, trace Silt

Boring Number:	B-3	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	7 to 8	Initial Dry Density (pcf)	95.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	100.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.73

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

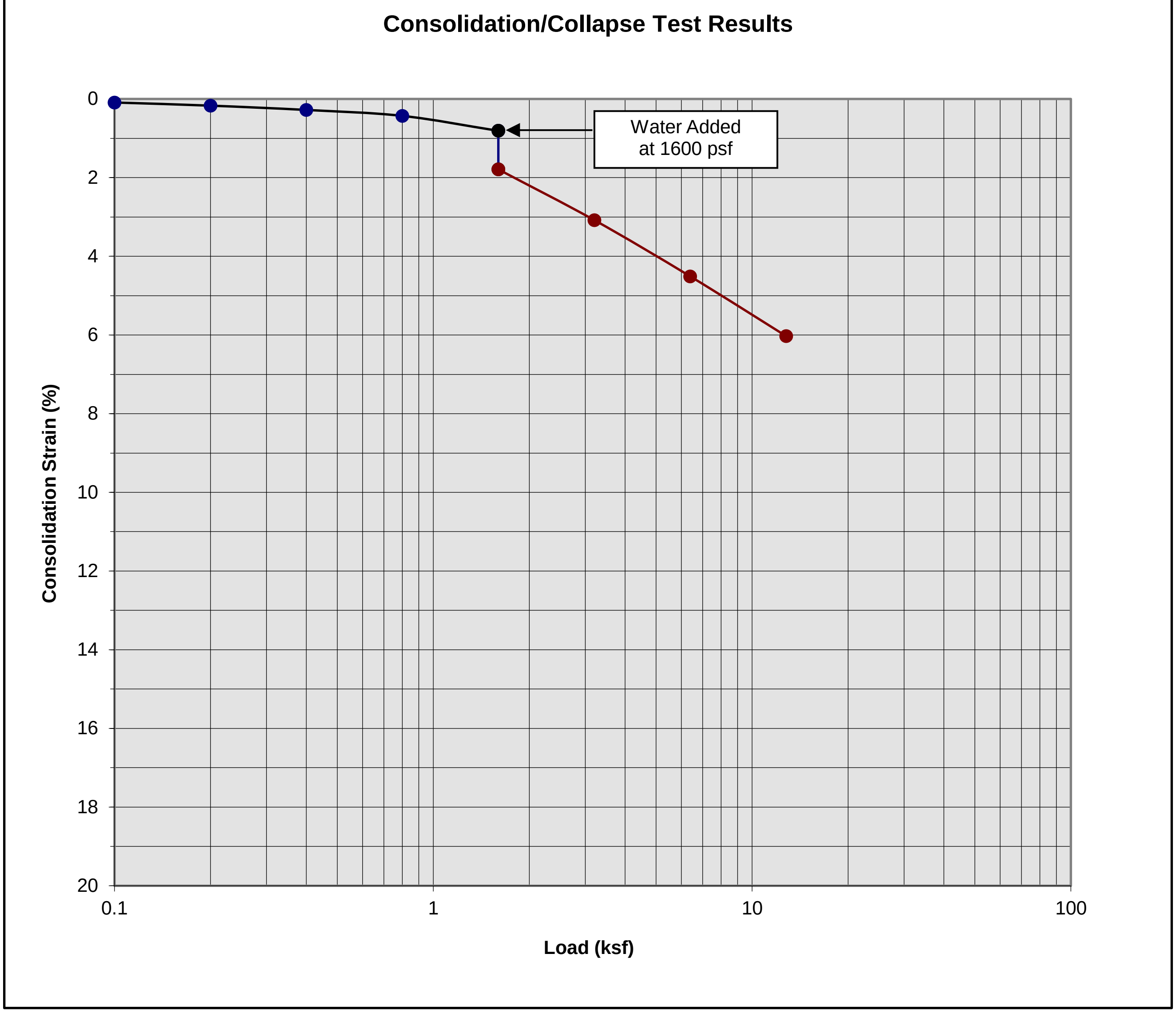


Classification: Light Gray fine Sand, little medium to coarse Sand, trace Silt, trace fine Gravel

Boring Number:	B-3	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	9 to 10	Initial Dry Density (pcf)	99.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.34

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

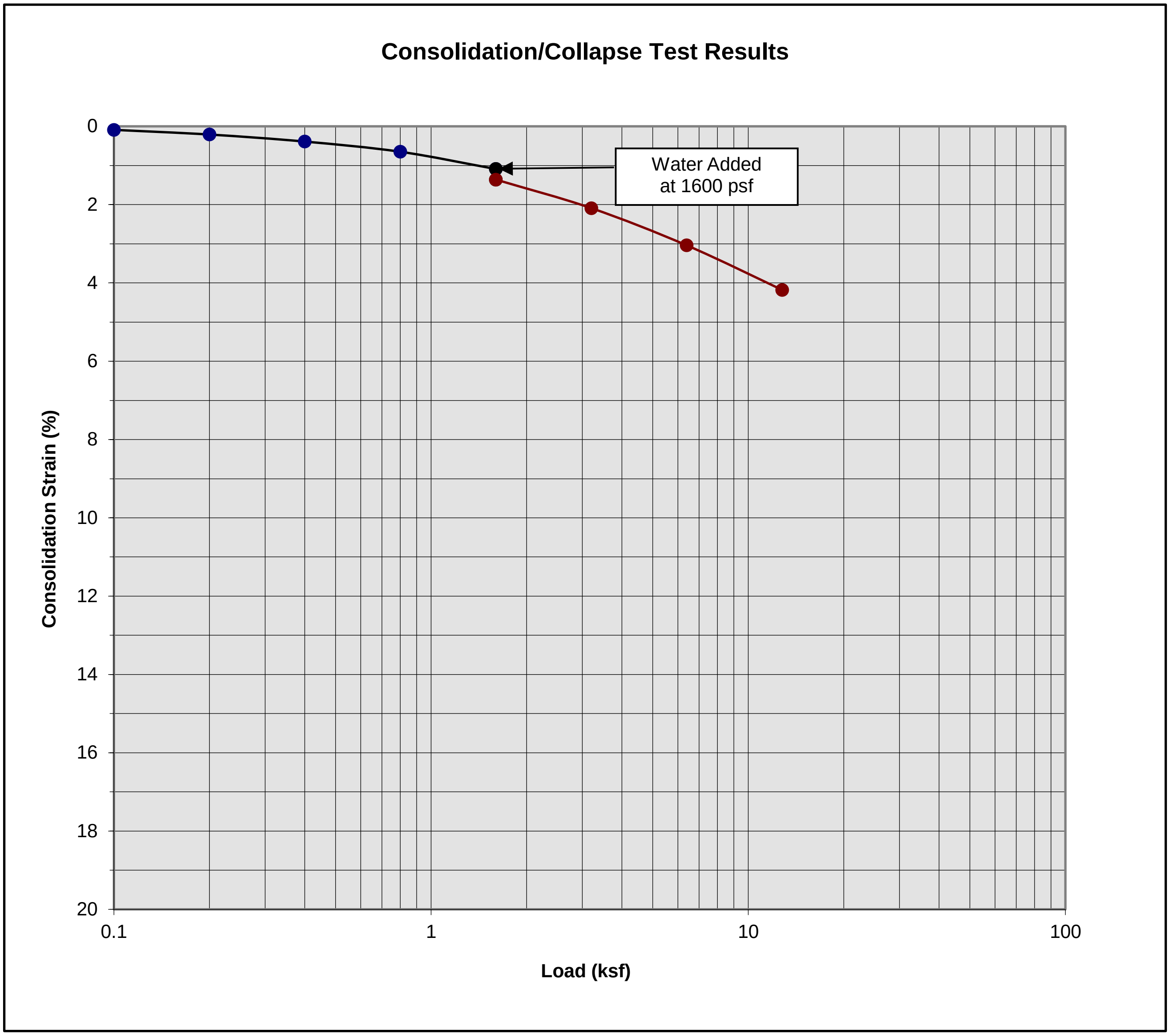


Classification: Light Gray to Gray fine to coarse Sand, little fine to coarse Gravel

Boring Number:	B-3	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	14 to 15	Initial Dry Density (pcf)	118.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	125.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.98

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-8

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



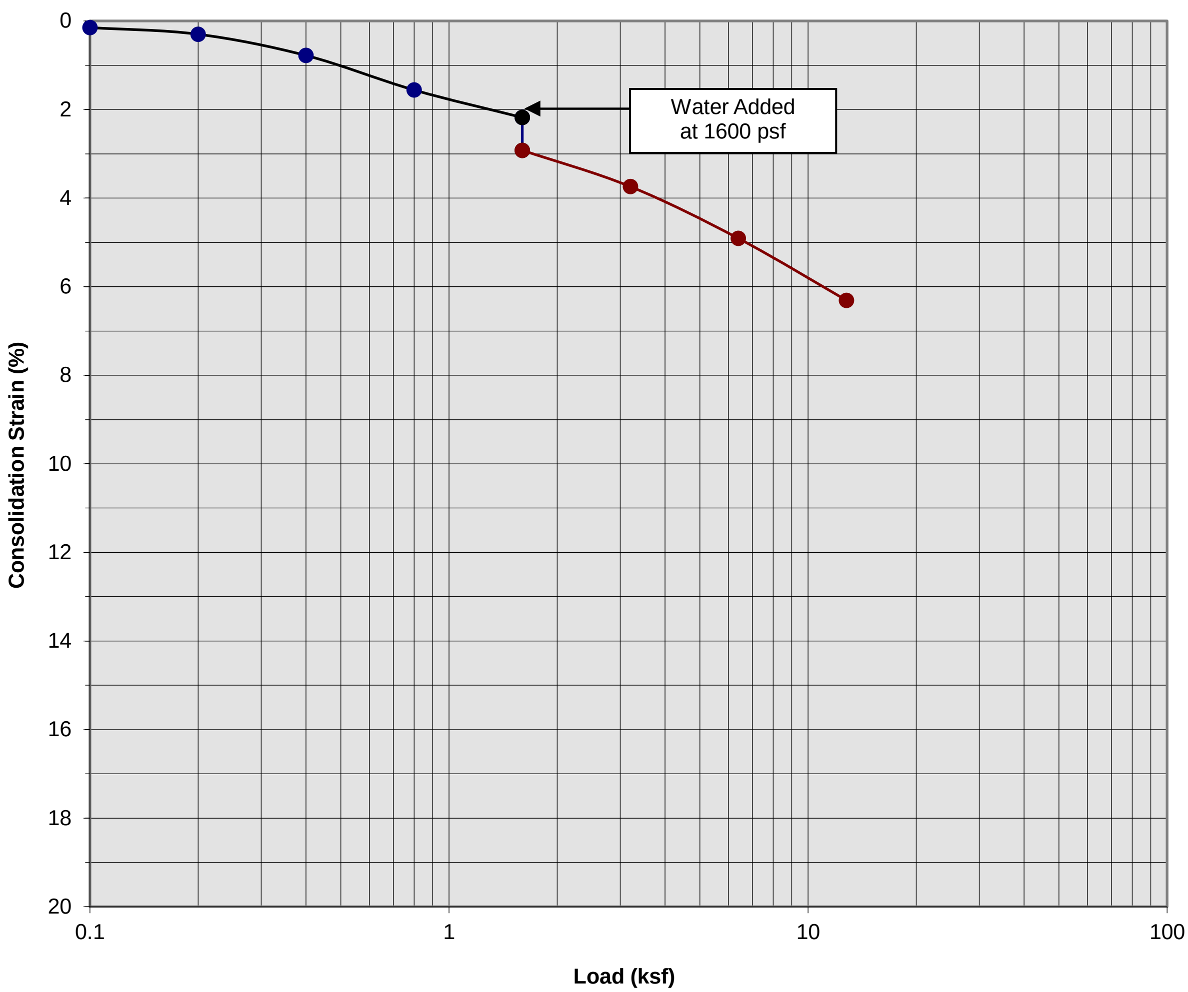
Classification: Gray Brown fine to coarse Sand, trace to little Silt, trace fine Gravel

Boring Number:	B-3	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	19 to 20	Initial Dry Density (pcf)	114.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.27

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-9

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

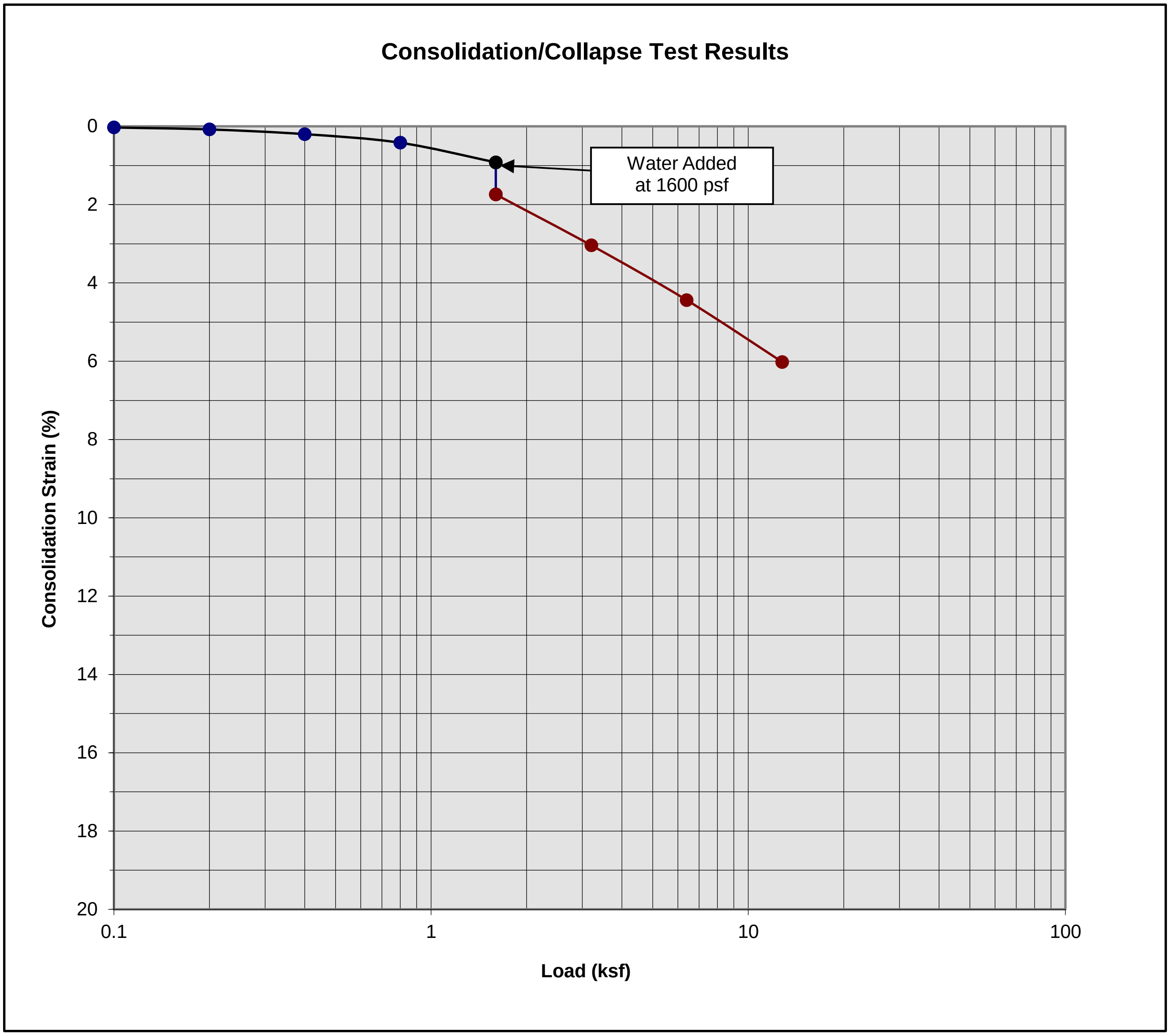


Classification: Light Brown fine to coarse Sand, trace fine Gravel

Boring Number:	B-4	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	14 to 15	Initial Dry Density (pcf)	107.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.74

Proposed Commercial/Industrial Development
Los Angeles, CA
Project No. 20G243-4
PLATE C- 10

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

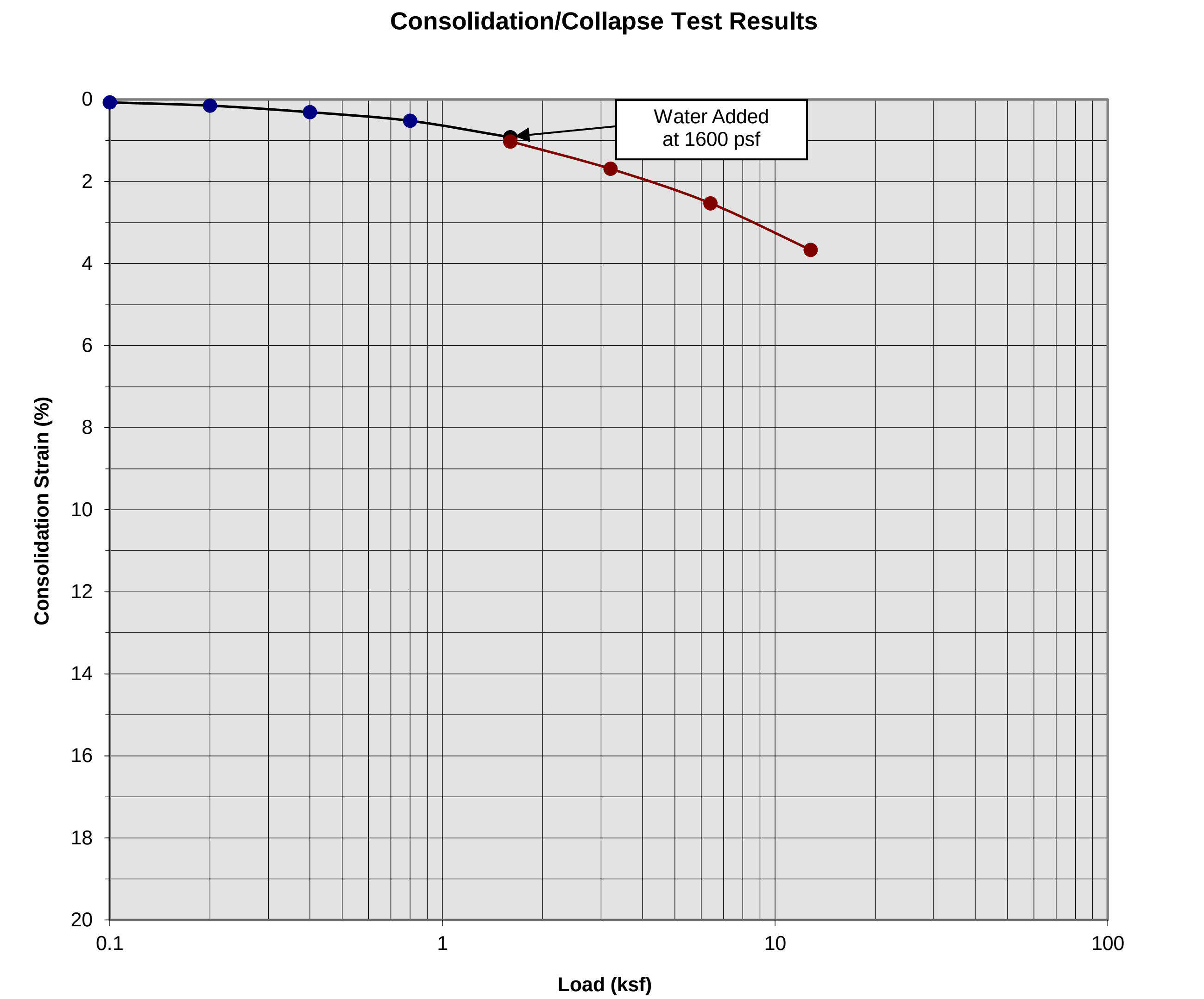


Classification: Light Brown fine to coarse Sand, trace fine Gravel

Boring Number:	B-4	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	19 to 20	Initial Dry Density (pcf)	106.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.82

Proposed Commercial/Industrial Development
Los Angeles, CA
Project No. 20G243-4
PLATE C- 11

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

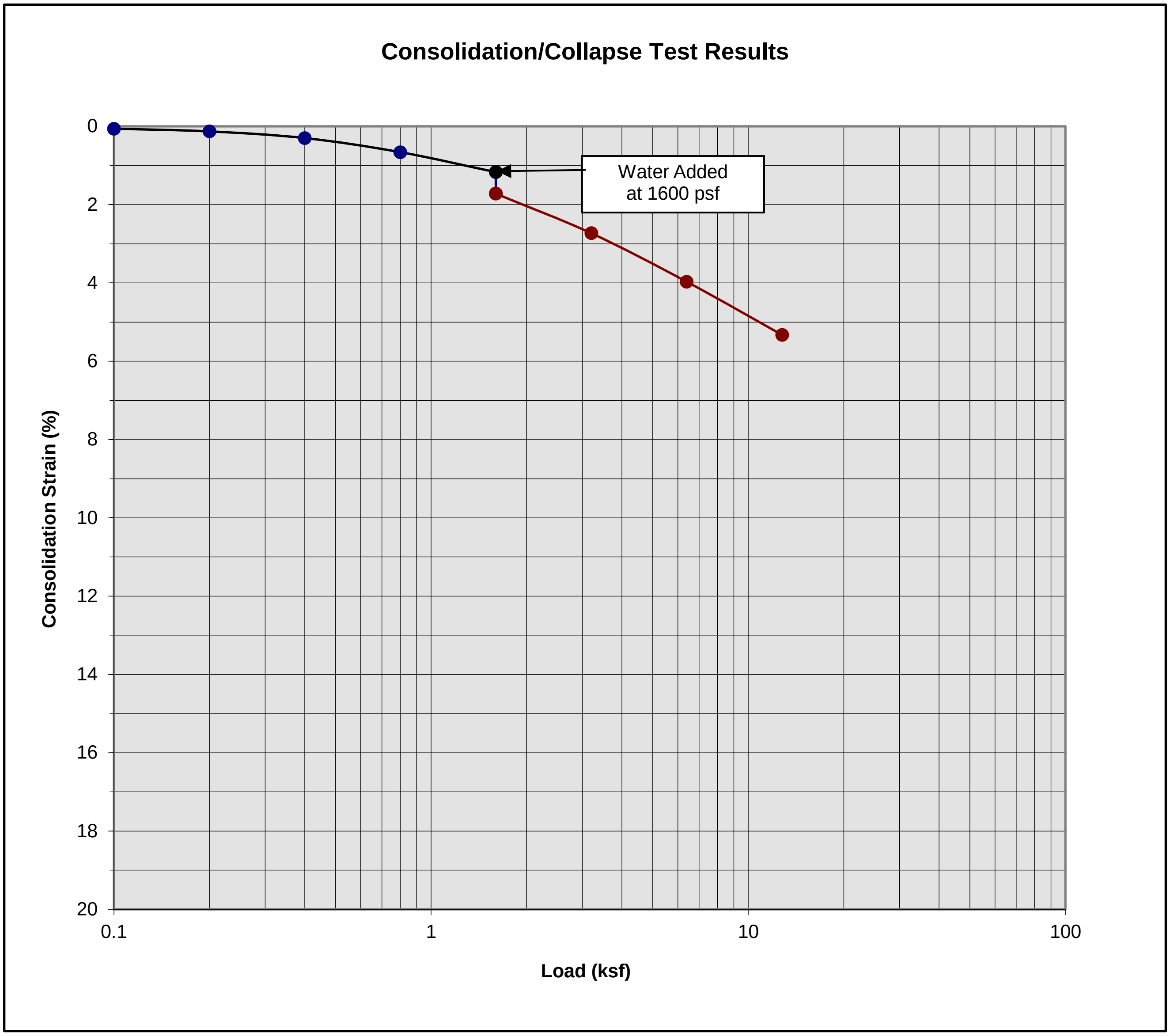


Classification: Dark Brown Silty fine to course Sand, trace Clay

Boring Number:	B-4	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	24 to 25	Initial Dry Density (pcf)	115.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.10

Proposed Commercial/Industrial Development
Los Angeles, CA
Project No. 20G243-4
PLATE C- 12

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



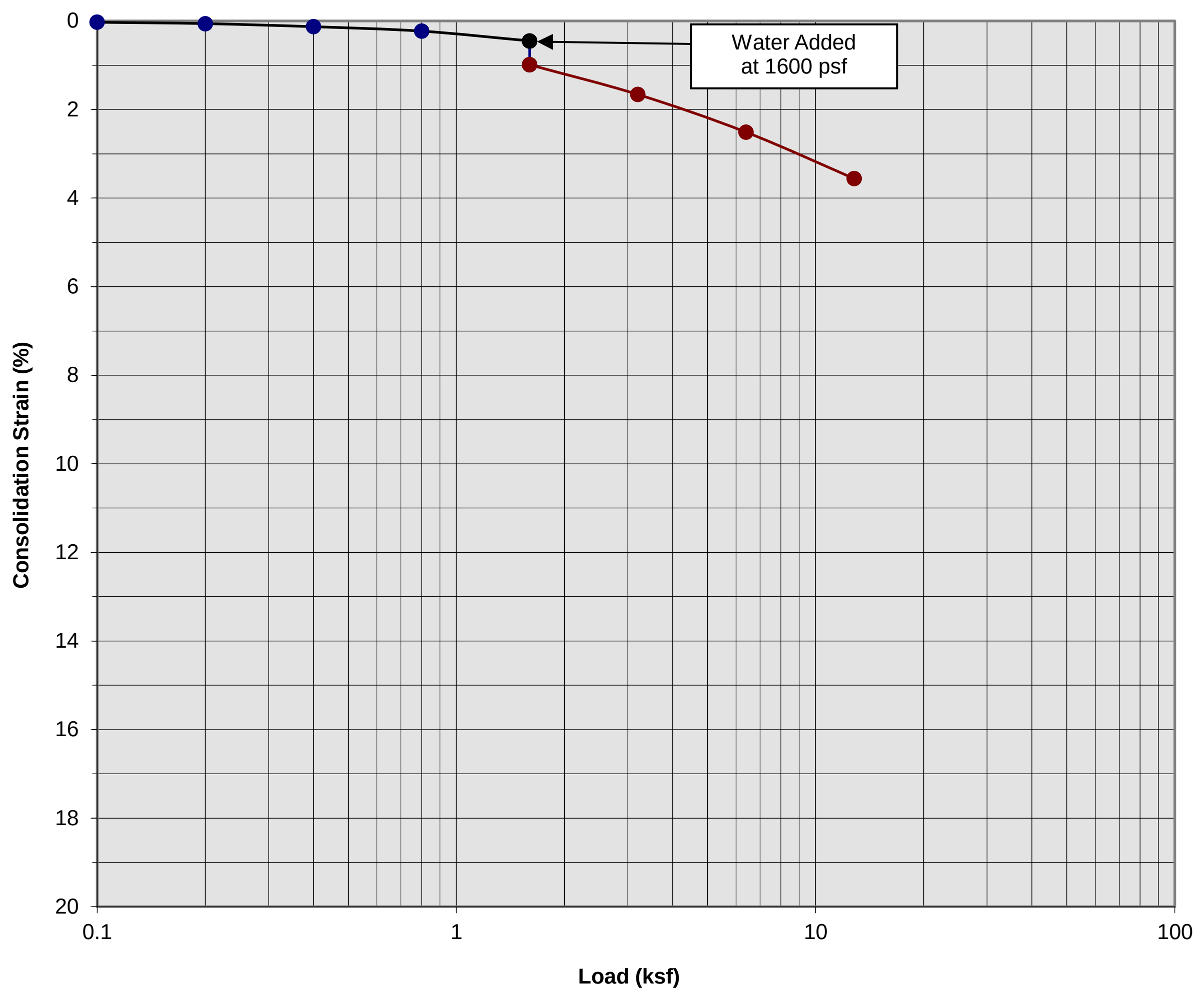
Classification: Brown fine to coarse Sand, trace to little Silt, trace fine Gravel

Boring Number:	B-4	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	29 to 30	Initial Dry Density (pcf)	119.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	125.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.55

Proposed Commercial/Industrial Development
Los Angeles, CA
Project No. 20G243-4
PLATE C- 13

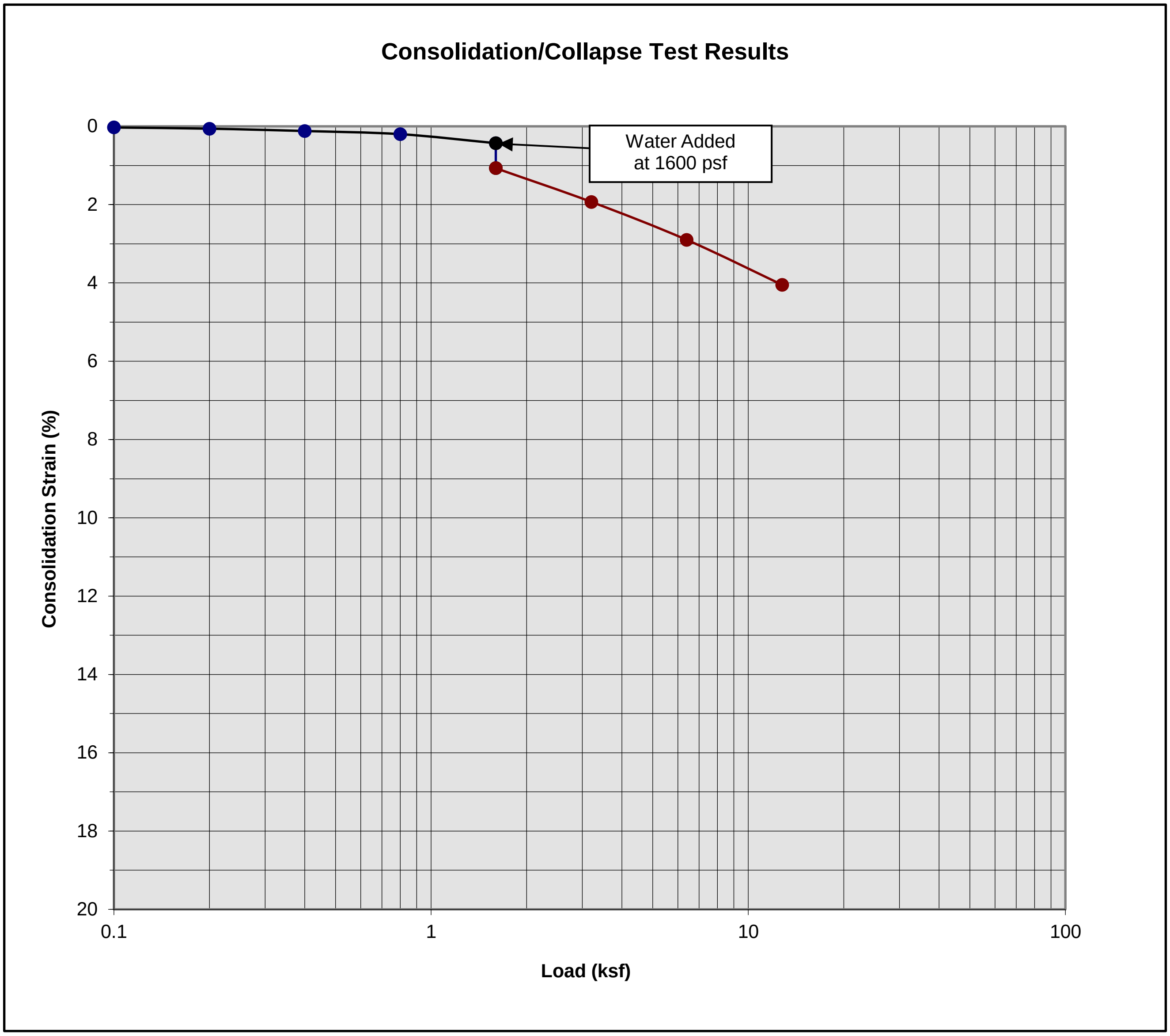
SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Light Gray Brown Gravelly fine to coarse Sand

Boring Number:	B-11	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	14 to 15	Initial Dry Density (pcf)	120.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	124.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.54

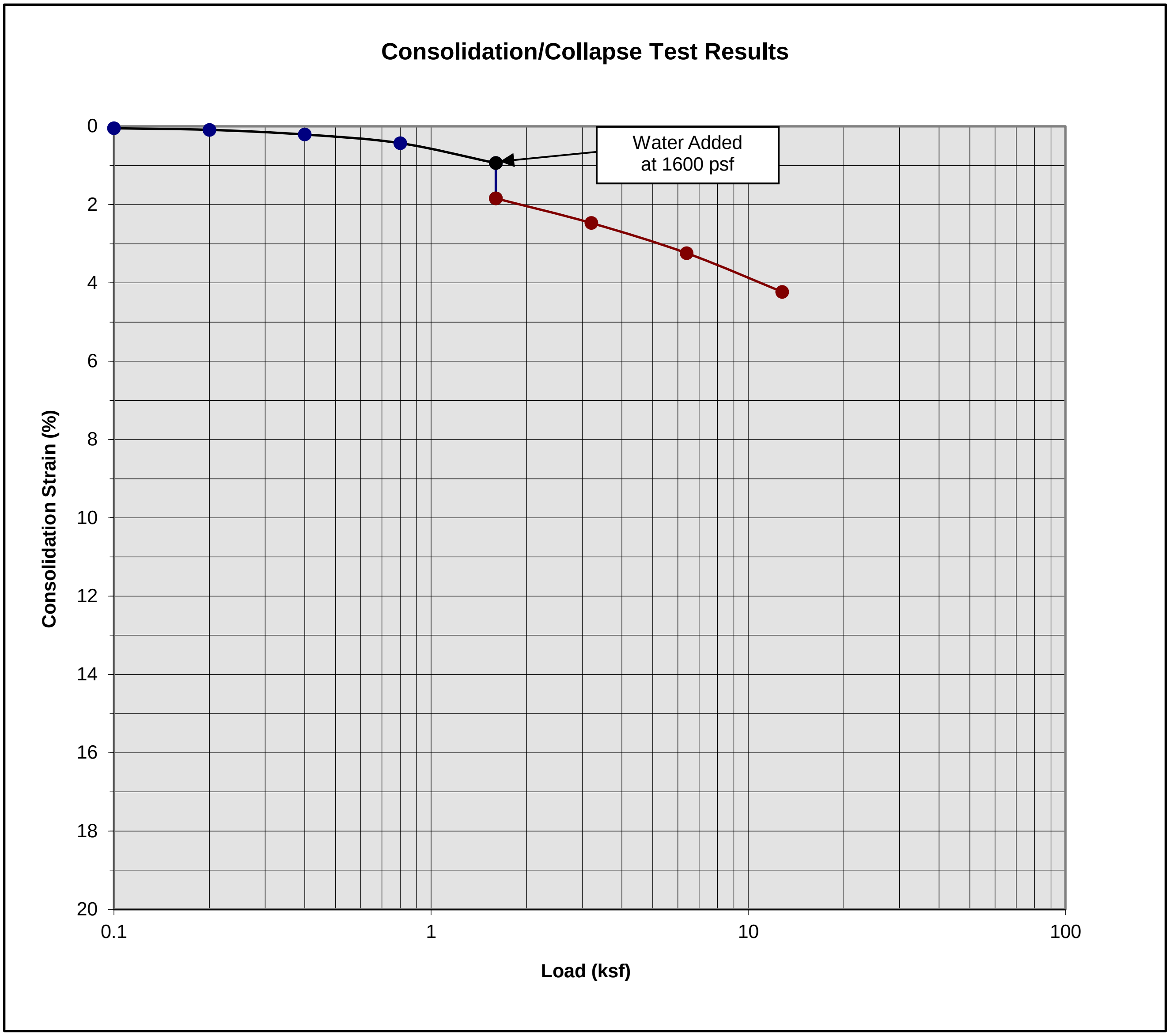


Classification: Gray Brown Silty fine to coarse Sand, little fine to coarse Gravel

Boring Number:	B-11	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	19 to 20	Initial Dry Density (pcf)	92.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.64

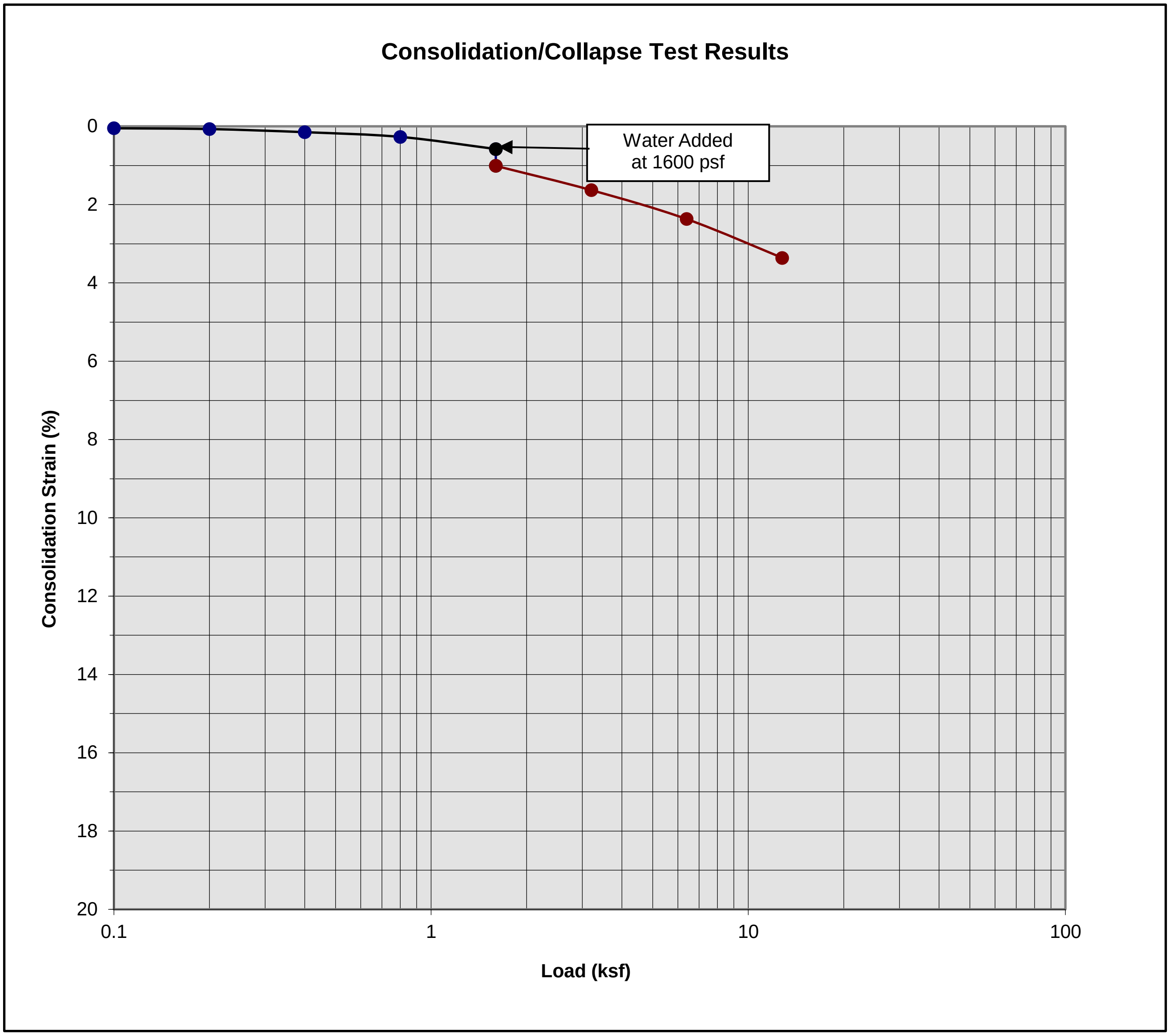
Proposed Commercial/Industrial Development
Los Angeles, CA
Project No. 20G243-4
PLATE C- 15

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Gray Brown Silty fine to coarse Sand, little fine to coarse Gravel

Boring Number:	B-11	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	24 to 25	Initial Dry Density (pcf)	113.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	118.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.90

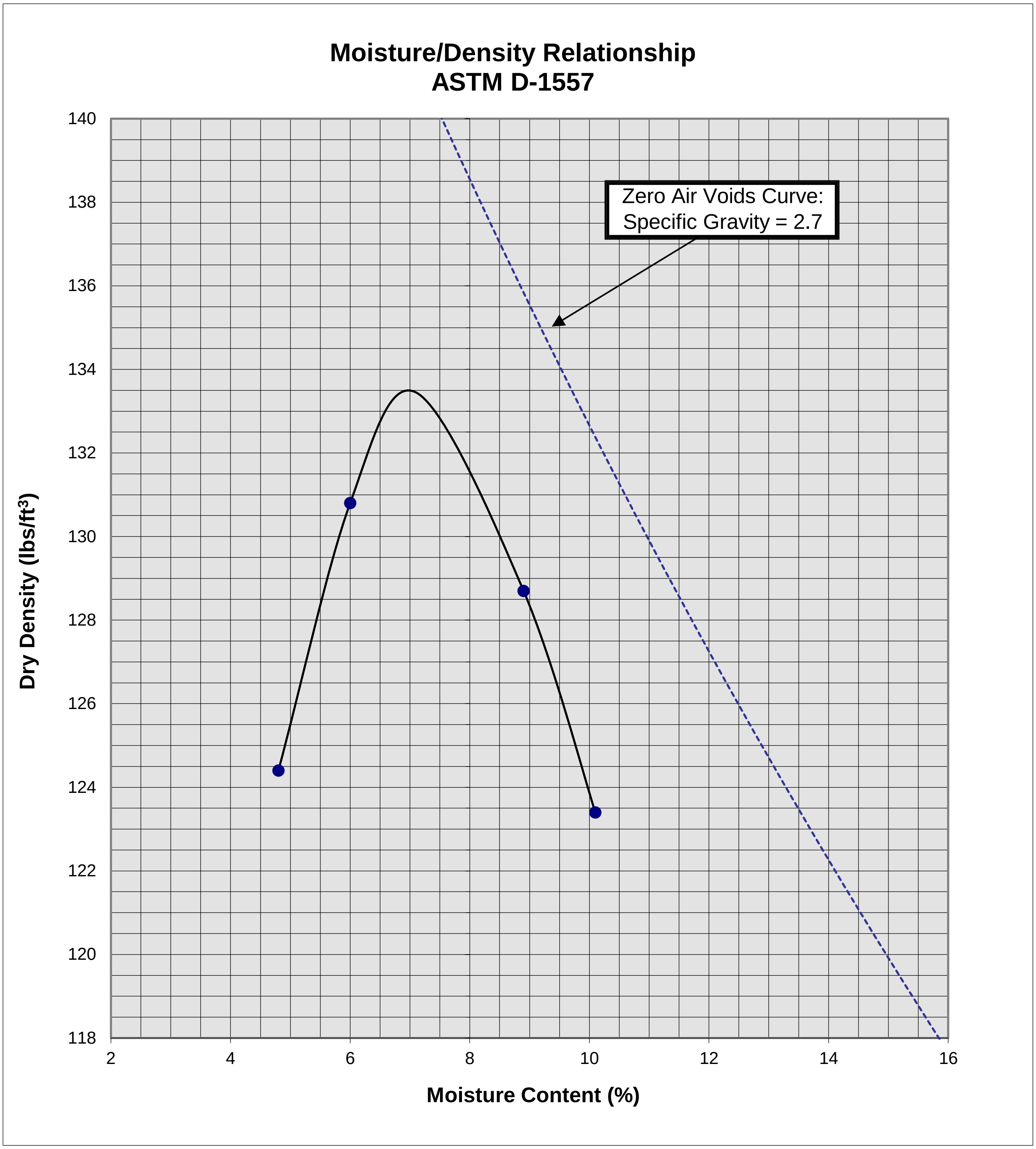


Classification: Light Brown fine to coarse Sand, little fine to coarse Gravel, little Silt

Boring Number:	B-11	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	29 to 30	Initial Dry Density (pcf)	99.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	102.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.43

Proposed Commercial/Industrial Development
Los Angeles, CA
Project No. 20G243-4
PLATE C- 17

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

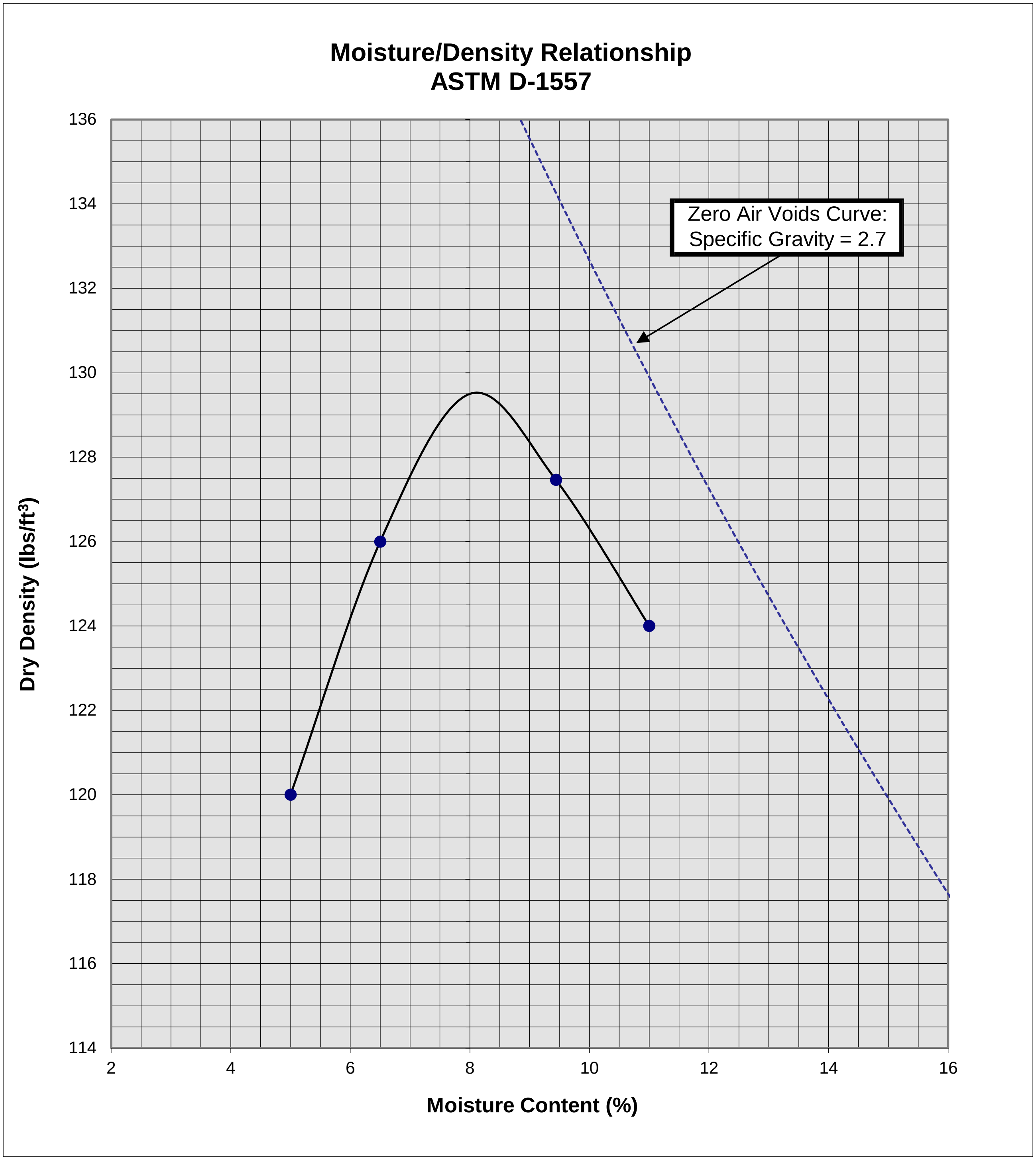


Soil ID Number		B-3 @ 0-5'
Optimum Moisture (%)		7
Maximum Dry Density (pcf)		133.5
Soil Classification	Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel	

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4

PLATE C-18

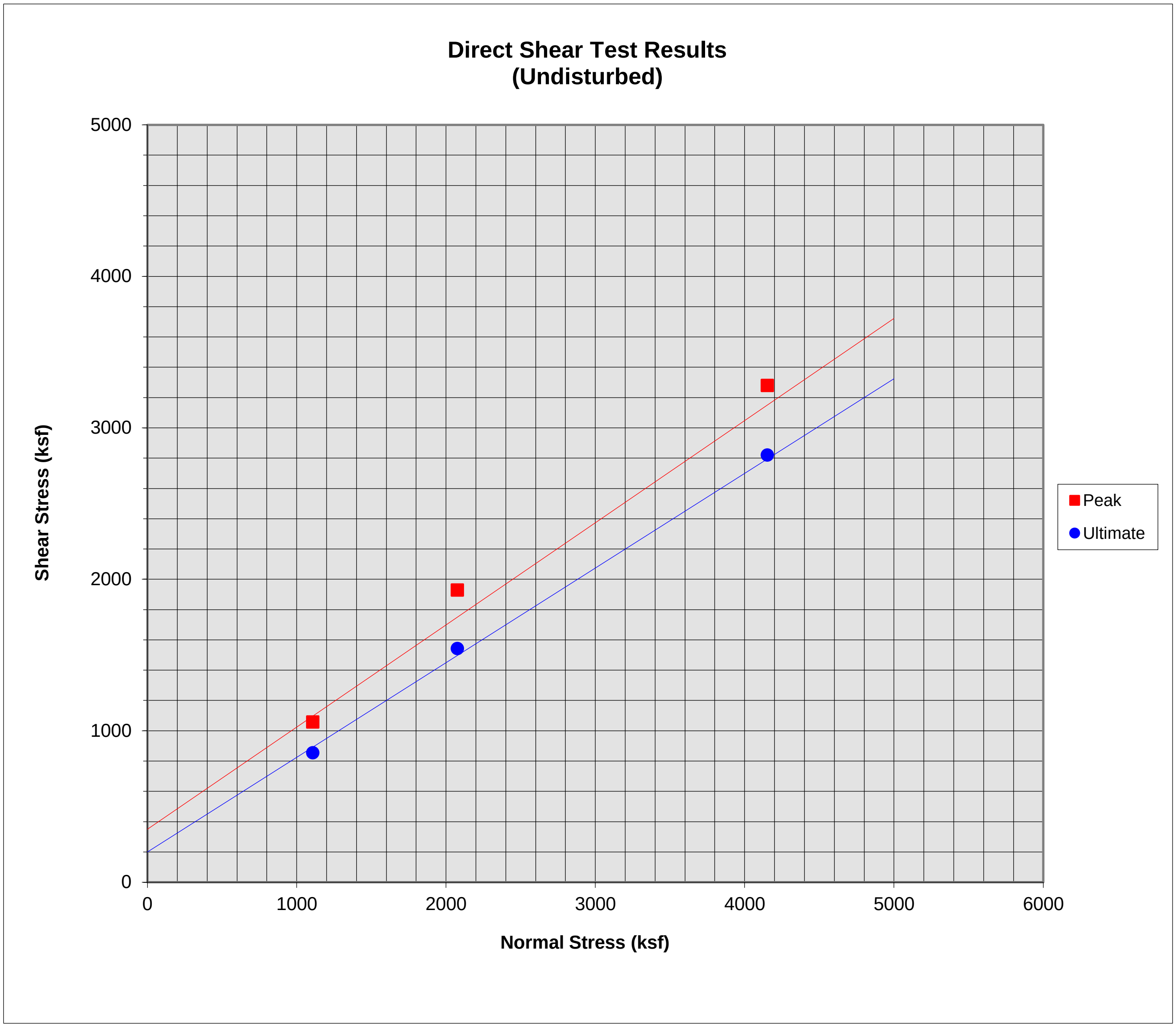
SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-11 @ 1-5'
Optimum Moisture (%)	8
Maximum Dry Density (pcf)	129.5
Soil Classification	Gray Brown Silty fine to medium Sand, little fine Gravel

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C-19

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Sample Description: B-1 @ 9 to 10 feet

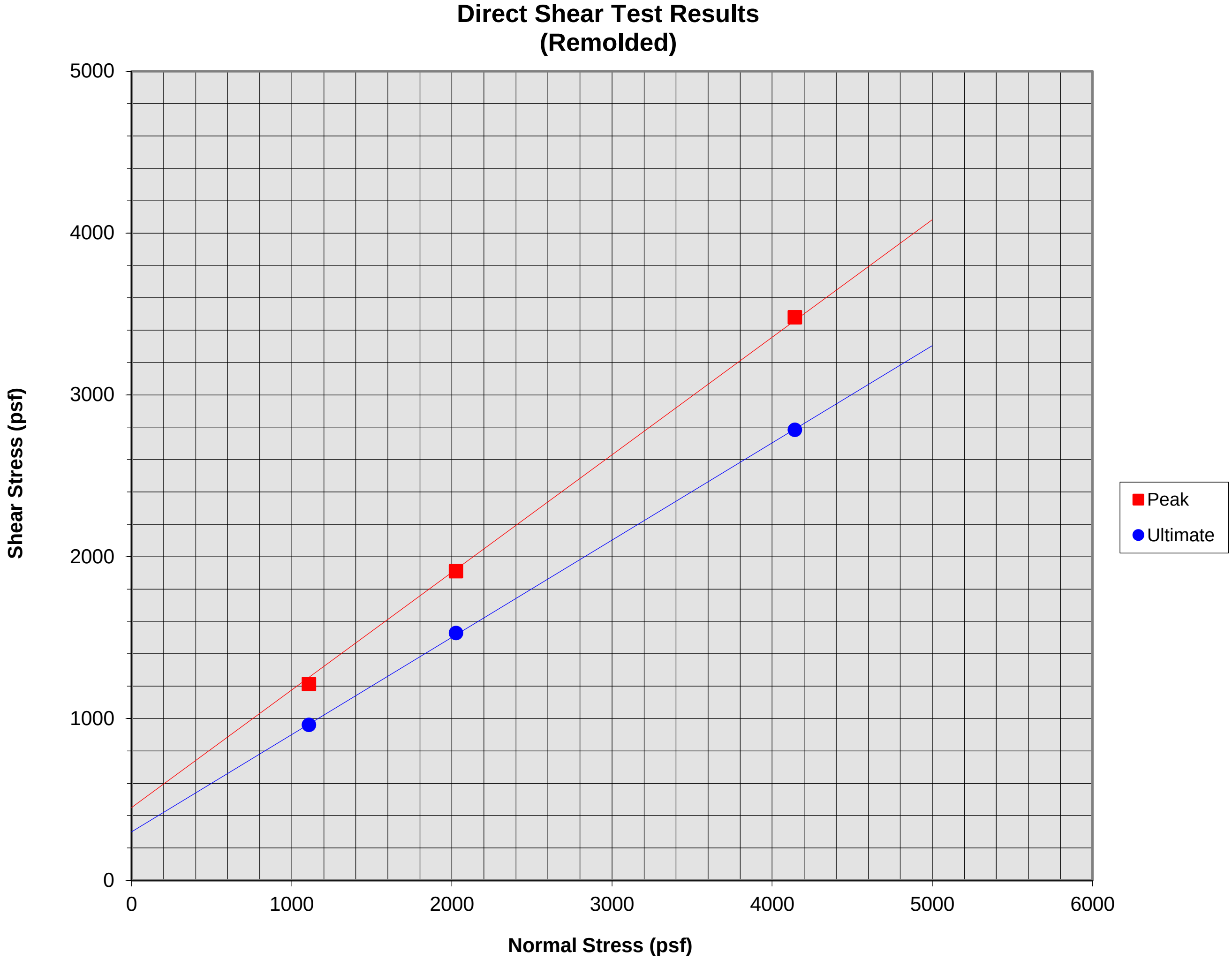
Classification: Gray Brown fine to coarse Sand, trace to little Silt

Sample Data	
Initial Moisture Content	2.0
Final Moisture Content	21.0
Initial Dry Density	107.0
Final Dry Density	N/A
Specimen Diameter (in)	2.4
Specimen Thickness (in)	1.0

Test Results	Peak	Ultimate
ϕ (°)	34.0	32.0
C (psf)	350	200

Proposed Commercial/Industrial Development
Commerce, California
Project No. 20G243-4
PLATE C- 20

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Sample Description: B-3 @ 0 to 5 feet

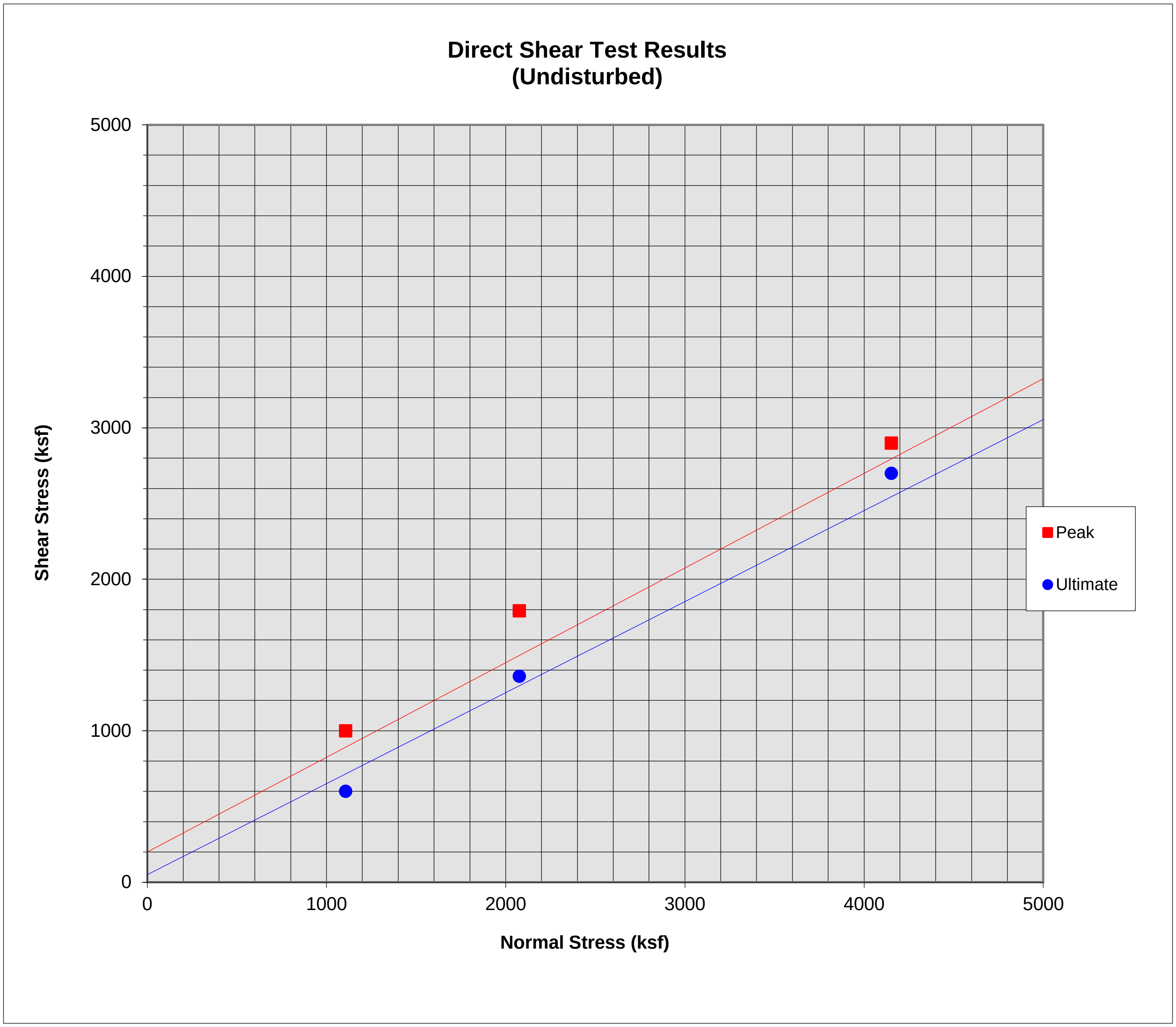
Classification: Gray Brown Silty fine to medium Sand, trace coarse Sand

Sample Data	
Remolded Moisture Content	7.0
Final Moisture Content	16.0
Remolded Dry Density	120.2
Percent Compaction	90.0
Final Dry Density	N/A
Specimen Diameter (in)	2.4
Specimen Thickness (in)	1.0

Test Results	Peak	Ultimate
ϕ (°)	36.0	31.0
C (psf)	450	300

Proposed Commercial/Industrial Development
Commerce, California
Project No. 20G243-4
PLATE C- 21

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



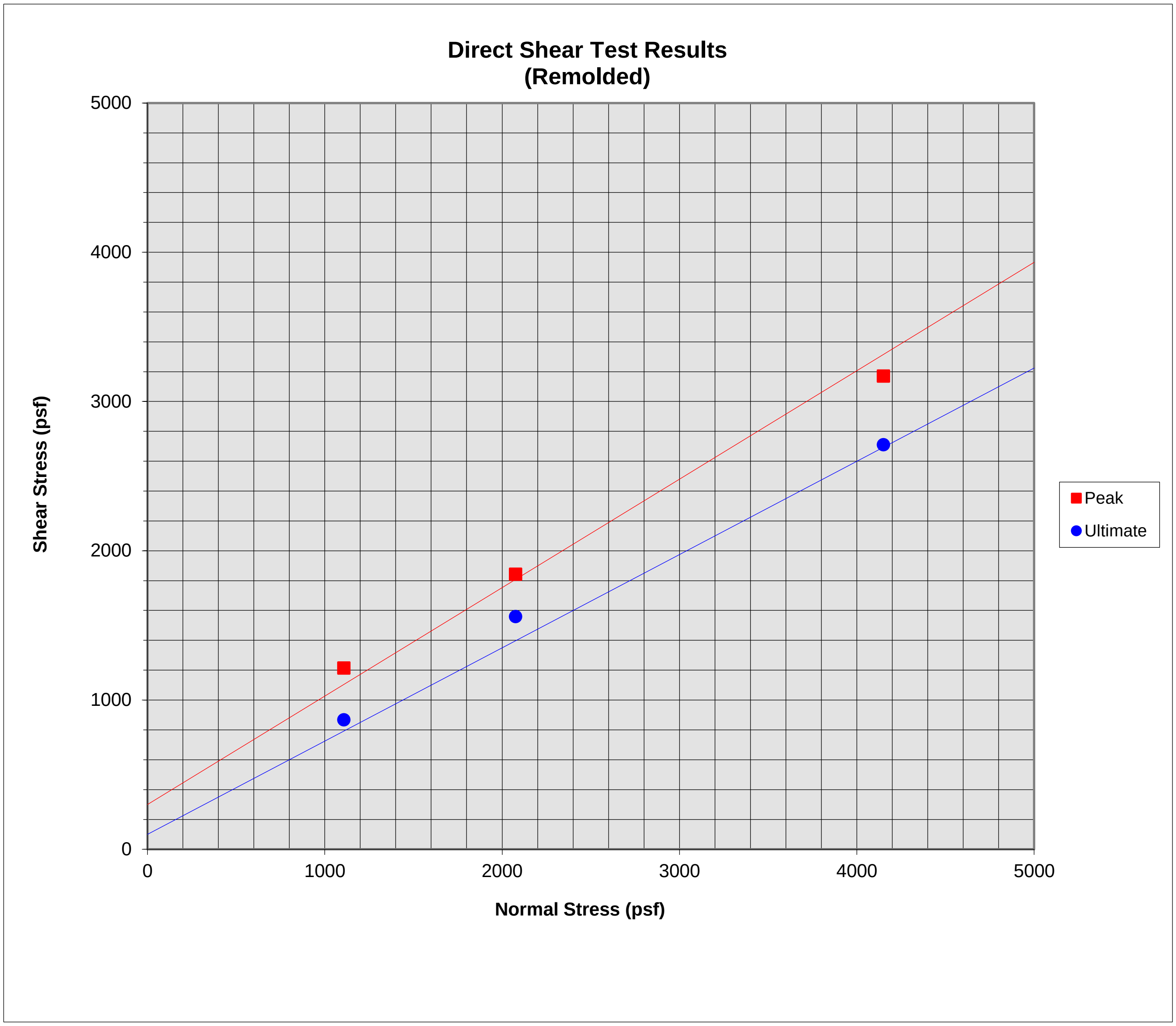
Sample Description: B-8 @ 9 to 10 feet

Classification: Light Brown fine to medium Sand, trace Silt, little coarse Sand

Sample Data			Test Results	
Initial Moisture Content	6.0			
Final Moisture Content	16.0		Peak	Ultimate
Initial Dry Density	107.0	ϕ (°)	32.0	31.0
Final Dry Density	--	C (psf)	200	50
Specimen Diameter (in)	2.4			
Specimen Thickness (in)	1.0			

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C- 22

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Sample Description: B-11 @ 1 to 5 feet

Classification: Gray Brown Silty fine to medium Sand, little fine Gravel

Sample Data	
Remolded Moisture Content	8.0
Final Moisture Content	16.0
Remolded Dry Density	116.6
Percent Compaction	90.0
Final Dry Density	----
Specimen Diameter (in)	2.4
Specimen Thickness (in)	1.0

Test Results	Peak	Ultimate
ϕ (°)	36.0	32.0
C (psf)	300	100

Proposed Commercial/Industrial Development
Los Angeles, California
Project No. 20G243-4
PLATE C- 23

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.

<u>Compacted Fills</u>

- 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.

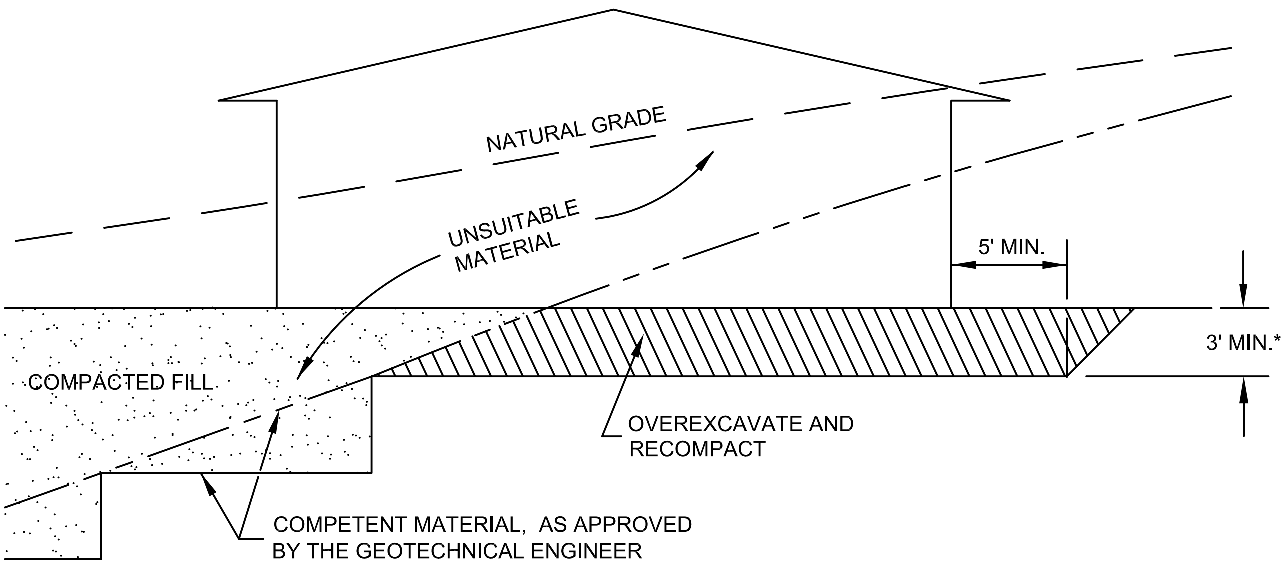
<u>Subdrains</u>

- 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.

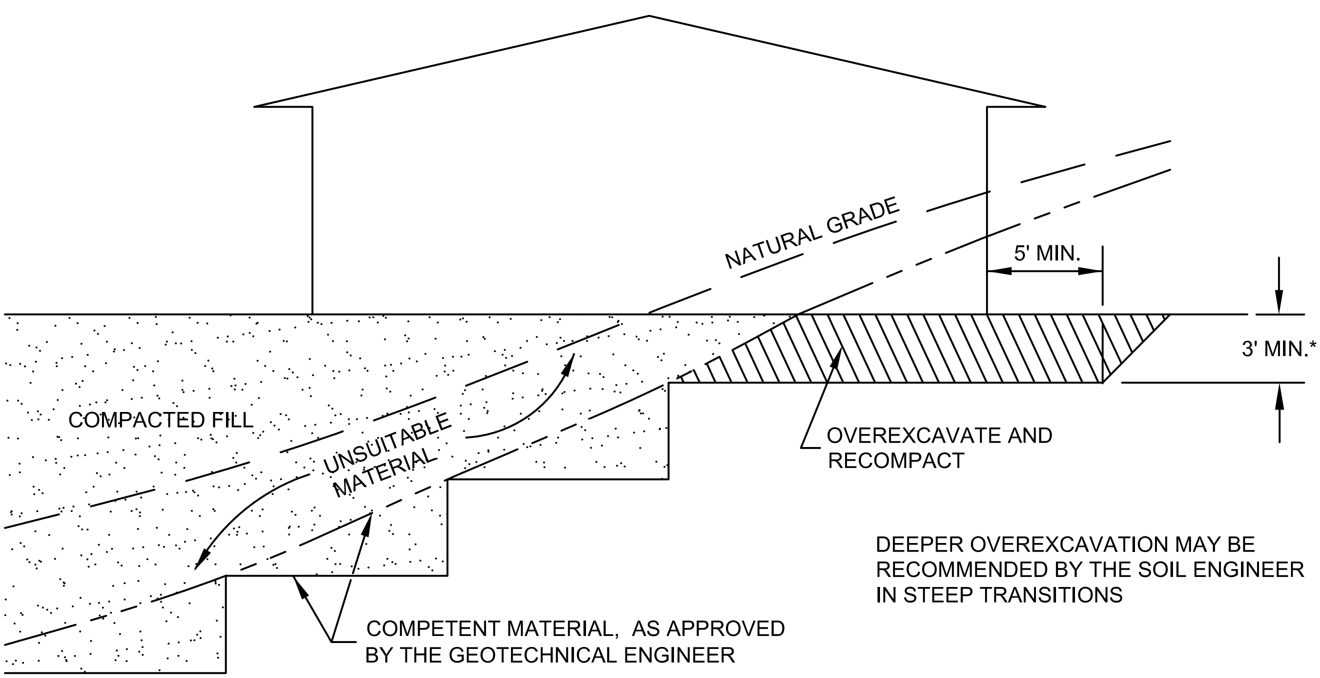
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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

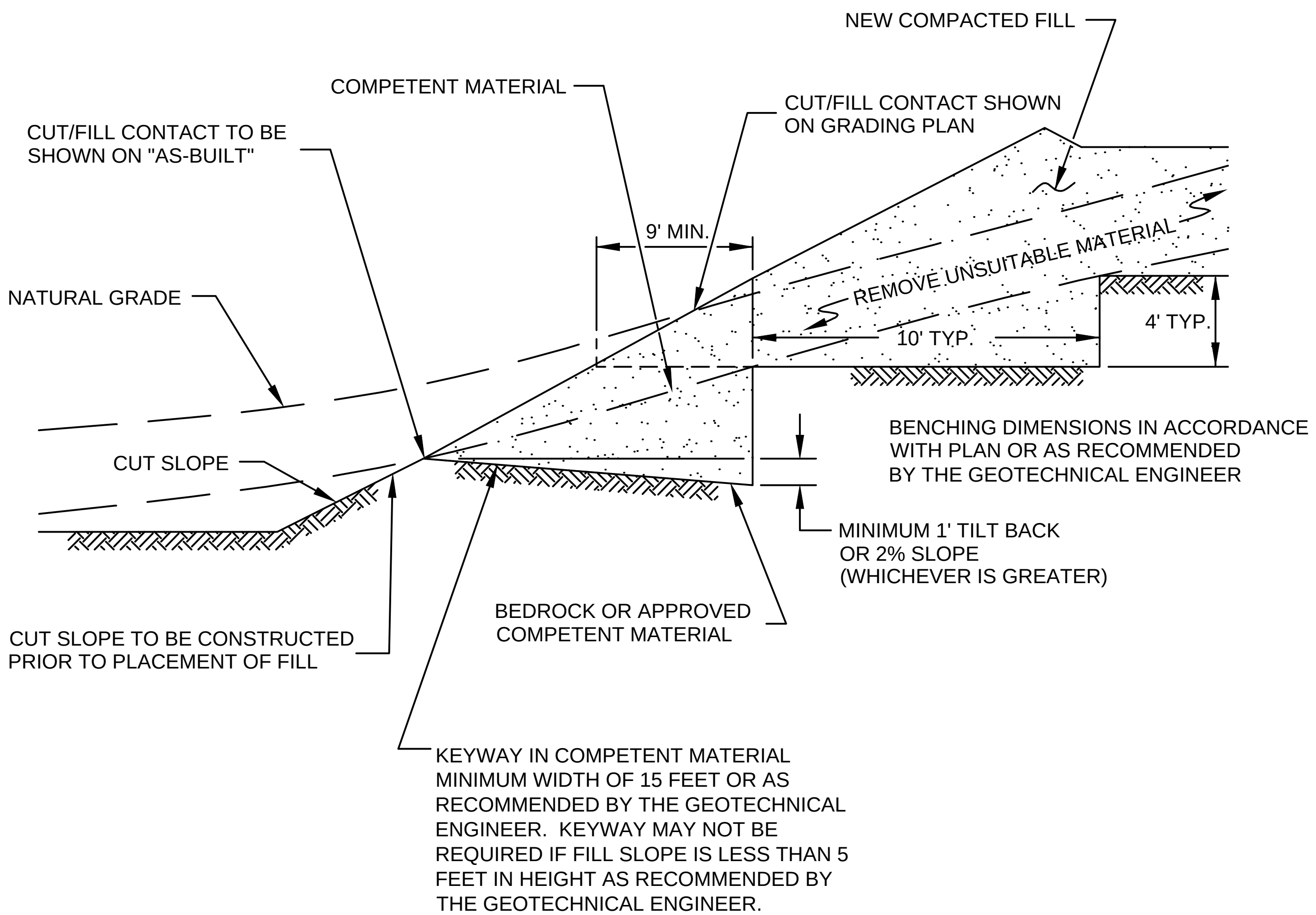
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

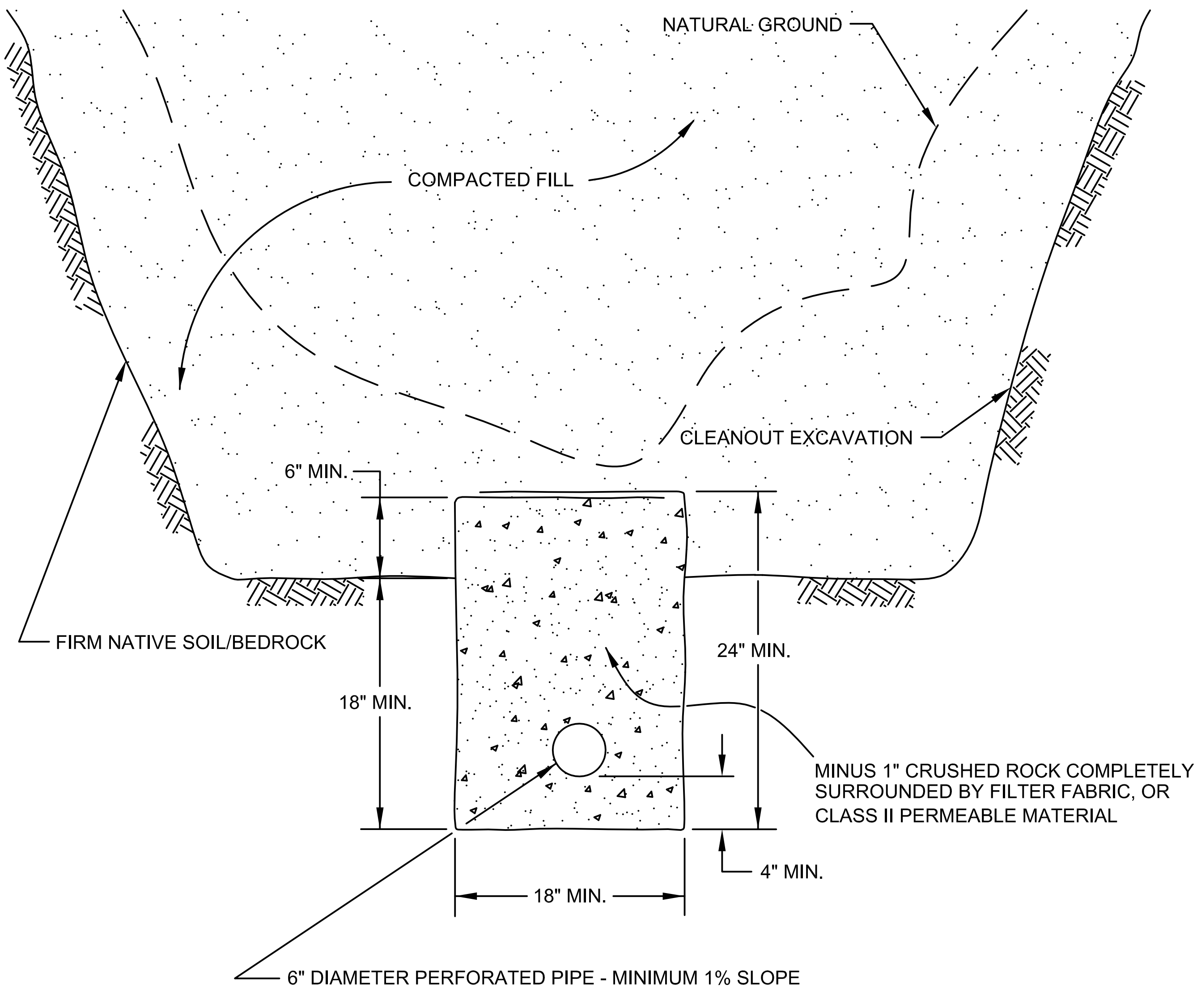
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

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

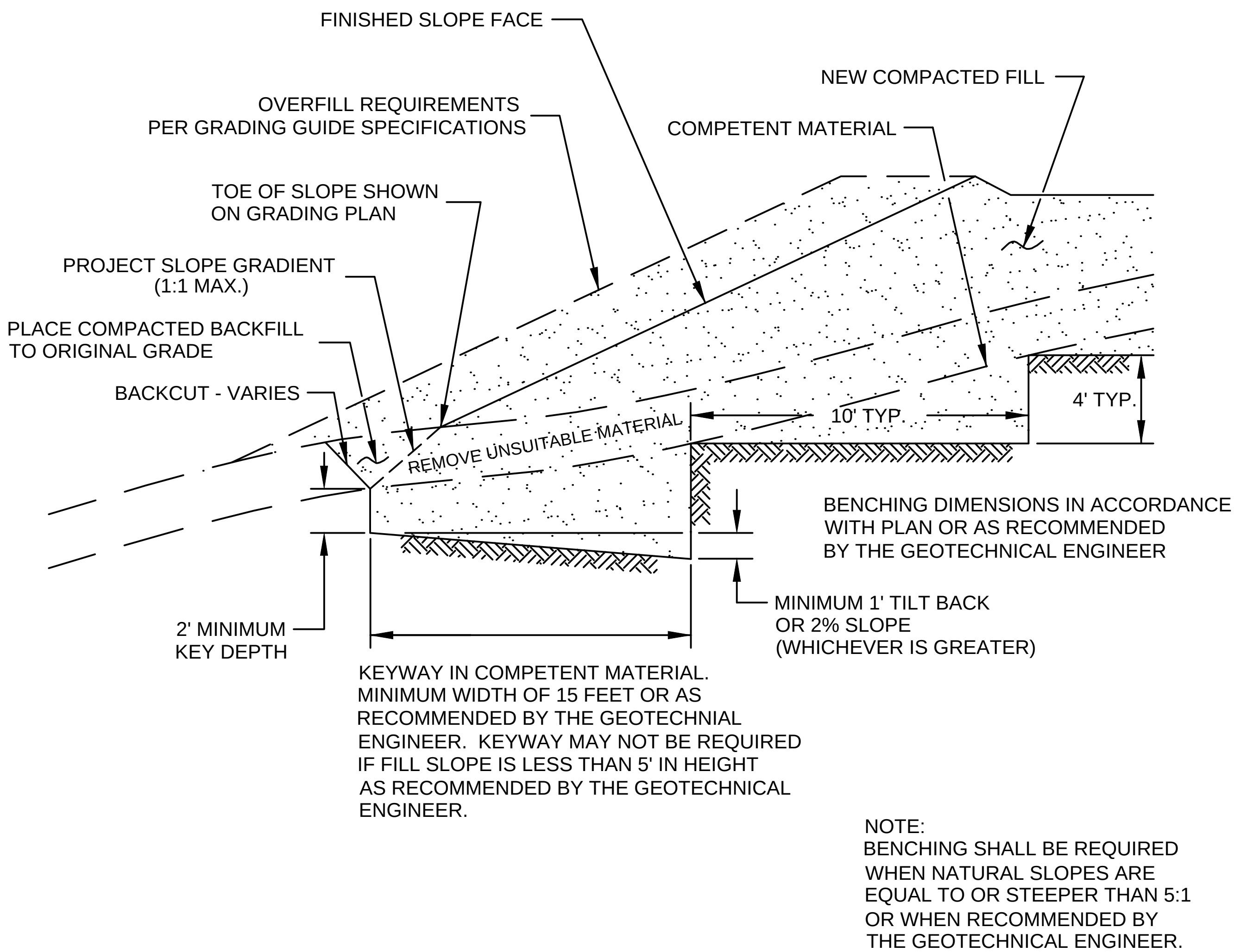
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

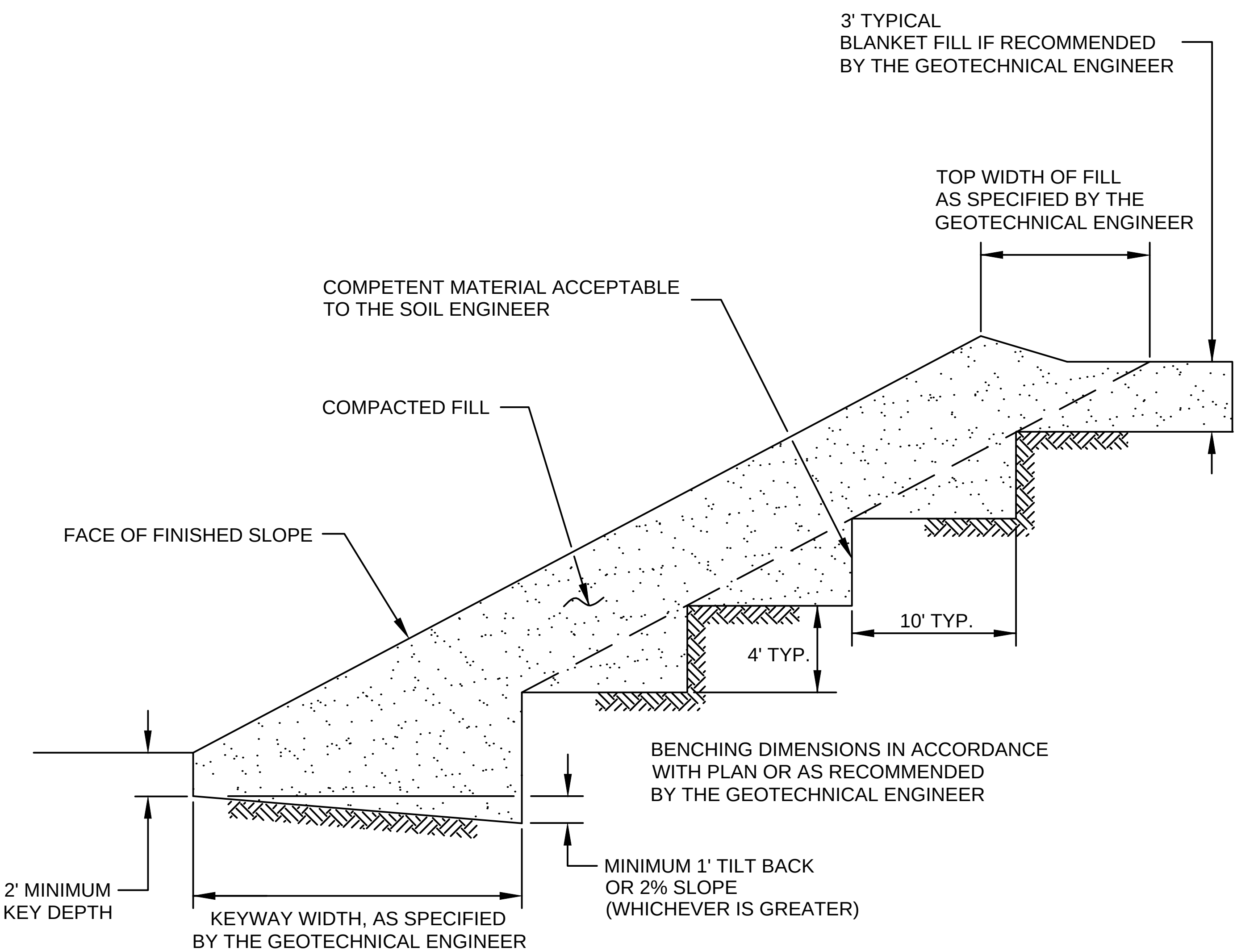
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

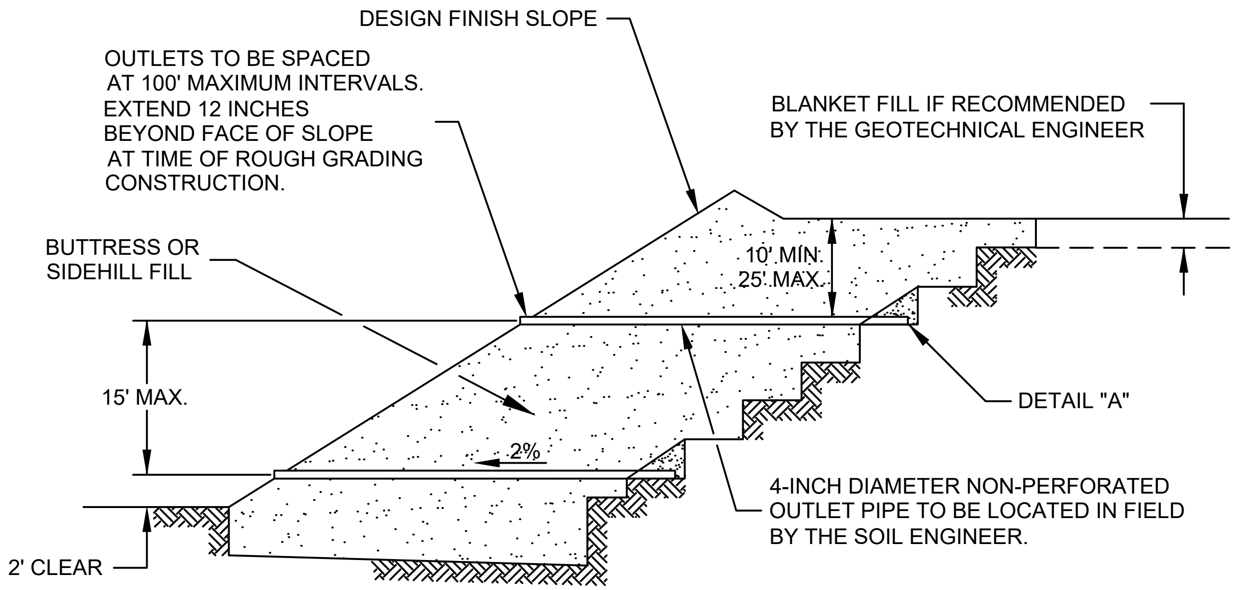
NOT TO SCALE

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)

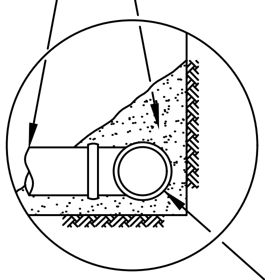
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"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

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW



FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL FIVE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE ABOVE FOR GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

DETAIL "A"

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

SLOPE FILL SUBDRAINS

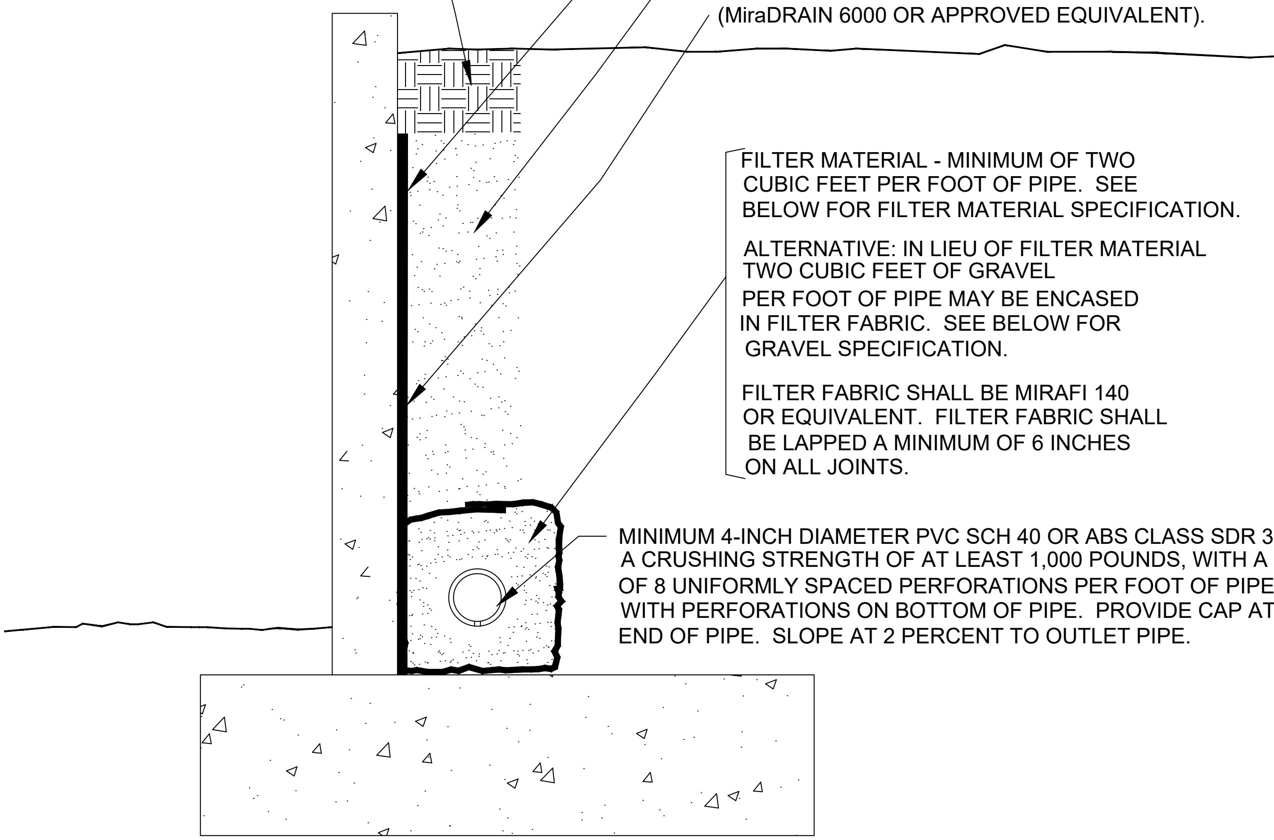
GRADING GUIDE SPECIFICATIONS

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

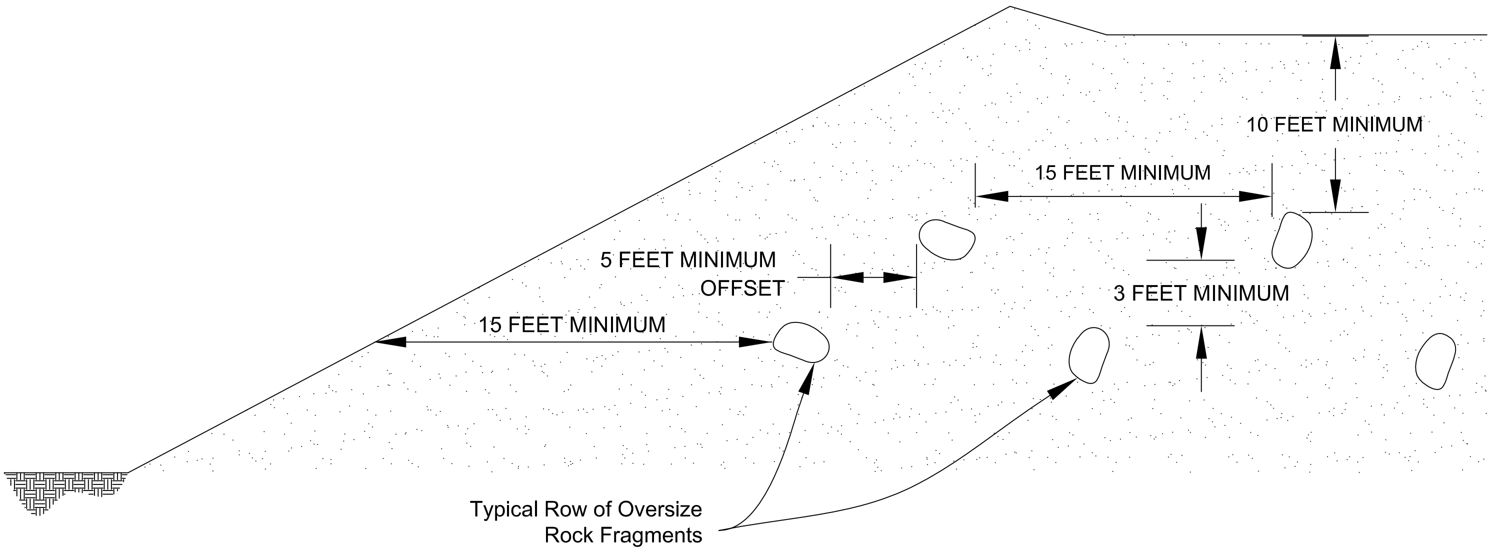
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

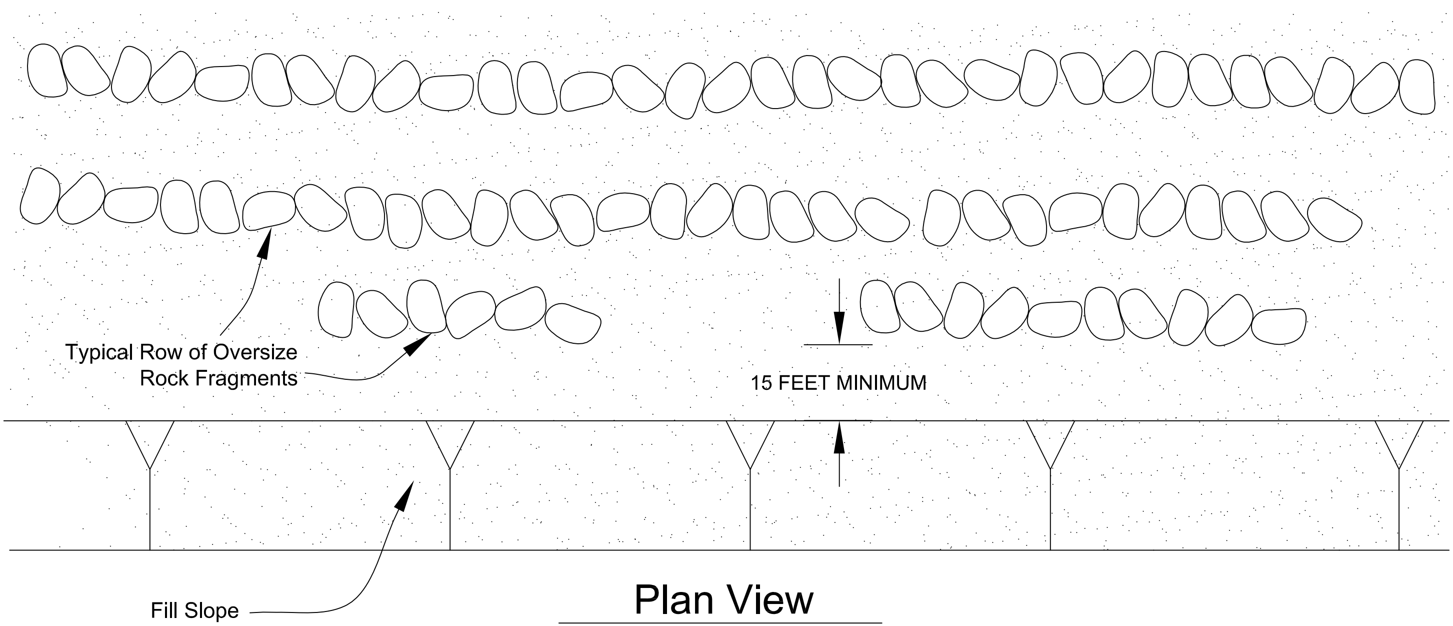
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: GKM	
PLATE D-8	

APPENDIX E

TERRA GEOSCIENCES

GROUND-MOTION SEISMIC ANALYSIS

PROPOSED COMMERCIAL / INDUSTRIAL DEVELOPMENT

1716 EAST 7TH STREET

LOS ANGELES, CALIFORNIA

Project No. 233920-1

February 13, 2023

Prepared for:

Southern California Geotechnical, Inc.
22885 E. Savi Ranch Parkway
Suite E
Yorba Linda, CA 92887

Consulting Engineering Geology & Geophysics

P.O. Box 1090, Loma Linda, CA 92354 • 909 796-4667

Southern California Geotechnical, Inc.
22885 E. Savi Ranch Parkway
Suite E
Yorba Linda, CA 92887

Attention: Mr. Greg Mitchell, Principal Engineer

Regarding: Ground-Motion Seismic Analysis
Proposed Commercial / Industrial Development
1716 East 7th Street
Los Angeles, California
SCG Project No. 20G243-4

INTRODUCTION

At your request, this firm has prepared a ground-motion seismic analysis report for the proposed freezer facility project, as referenced above. The purpose of this study was to evaluate the site-specific ground motion parameters to aid in the seismic design for this project, based on the current 2022 California Building Code (CBC). Our work included performing a seismic shear-wave study for determining the Site Classification and V_{S100} input values for this analysis. The scope of services provided for this evaluation included the following:

- ***Review of available published and unpublished geologic/seismic data in our files pertinent to the site, including a field reconnaissance.***
- ***Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included one traverse for shear-wave velocity analysis purposes.***
- ***Evaluation of the local and regional tectonic setting including performing a site-specific CBC ground motion analysis.***
- ***Preparation of this report presenting our findings, with respect to the seismic design parameters.***

Accompanying Maps and Appendices

Plate 1- Google™ Earth Imagery Map
Plate 2- Seismic Line Location Map
Appendix A - Shear-Wave Survey
Appendix B - Site Specific Ground Motion Analysis
Appendix C - References

PROJECT SUMMARY

Based on the information that has been provided, we understand that a commercial/industrial development is proposed to be constructed at the subject property, along with other associated appurtenances and hardscaping. For this project, we have performed a field reconnaissance, reviewed pertinent available geologic and geotechnical data in our files, along with performing a seismic shear-wave survey.

To aid in determining the soil Site Classification of the site for ground motion analysis purposes, a seismic shear-wave survey using the multi-channel analysis of surface waves (MASW) and microtremor array measurements (MAM) methods was performed in order to assess the one-dimensional average shear-wave velocity structure beneath the subject site to a depth of at least 100 feet. This survey line was performed within the western portion of the site, which provided the necessary unobstructed survey line length, as well as being representative for the site development. The resultant shear wave velocity (V_S) within the upper 100 feet (30 meters) was then used to determine the Site Classification (ASCE, 2017, Table 20.3-1) of the subject project study area for the seismic analysis. The detailed results of this survey, including the supportive data, are presented within Appendix A for reference.

The site is situated upon a large alluvial plain created as outwash primarily from the San Gabriel and Los Angles Rivers. Locally as mapped by Yerkes (1997), the subject site is shown to be underlain by Holocene age alluvium, generally comprised of unconsolidated and uncemented gravel, sand, silt, and clay silt, sand, and gravel, in turn underlain presumably by progressively older alluvial deposits at depth. A review of the provided Test Boring Log (B-3) drilled at the site by Southern California Geotechnical, Inc. (12/10/20), indicated that the site is locally underlain predominantly by interbedded silty clay, silt, sandy silt, silty fine-grained sand, fine- to medium-grained and fine- to coarse-grained sand, with occasional gravel, to a depth of at least 130 feet, that are in a medium to very dense condition.

The approximate location of the seismic shear-wave traverse (Seismic Line SW-1) is shown on a captured Google™ Earth (2023) image, as presented as the Google™ Earth Imagery Map, Plate 1. Additionally, the seismic survey line is also shown on a partial modified copy of the provided 80-scale “Proposed Boring Location Plan”, as presented on the Seismic Line Location Map, Plate 2. Photographic views of the seismic line traverse have been included within Appendix A for both visual and reference purposes.

SITE-SPECIFIC GROUND MOTION ANALYSIS

As requested, we have performed a site-specific seismic ground motion analysis as discussed above. Geographically, the proposed development project is located at Latitude 34.0342 and Longitude -118.2367 (World Geodetic System of 1984). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the OSHPD Seismic Design Maps Tool web application (OSHPD, 2023) and the California Building Code criteria (CBC, 2022), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (ASCE, 2017).

The results of this site-specific ground motion analysis have been summarized and are tabulated below, with the detailed analysis being presented within Appendix B:

TABLE 1 – SUMMARY OF SEISMIC DESIGN PARAMETERS

Factor or Coefficient	Value
S_S	**1.920g**
S_1	**0.684g**
F_a	**1.0**
F_v	**1.7**
S_{DS}	**1.260g**
S_{D1}	**0.910g**
S_{MS}	**1.885g**
S_{M1}	**1.368g**
T_L	**8 Seconds**
MCE_G PGA	**0.83g**
Shear-Wave Velocity (V_{30})	**1,151.2 ft/sec**
Site Classification	**D**
Risk Category	**II**

CLOSURE

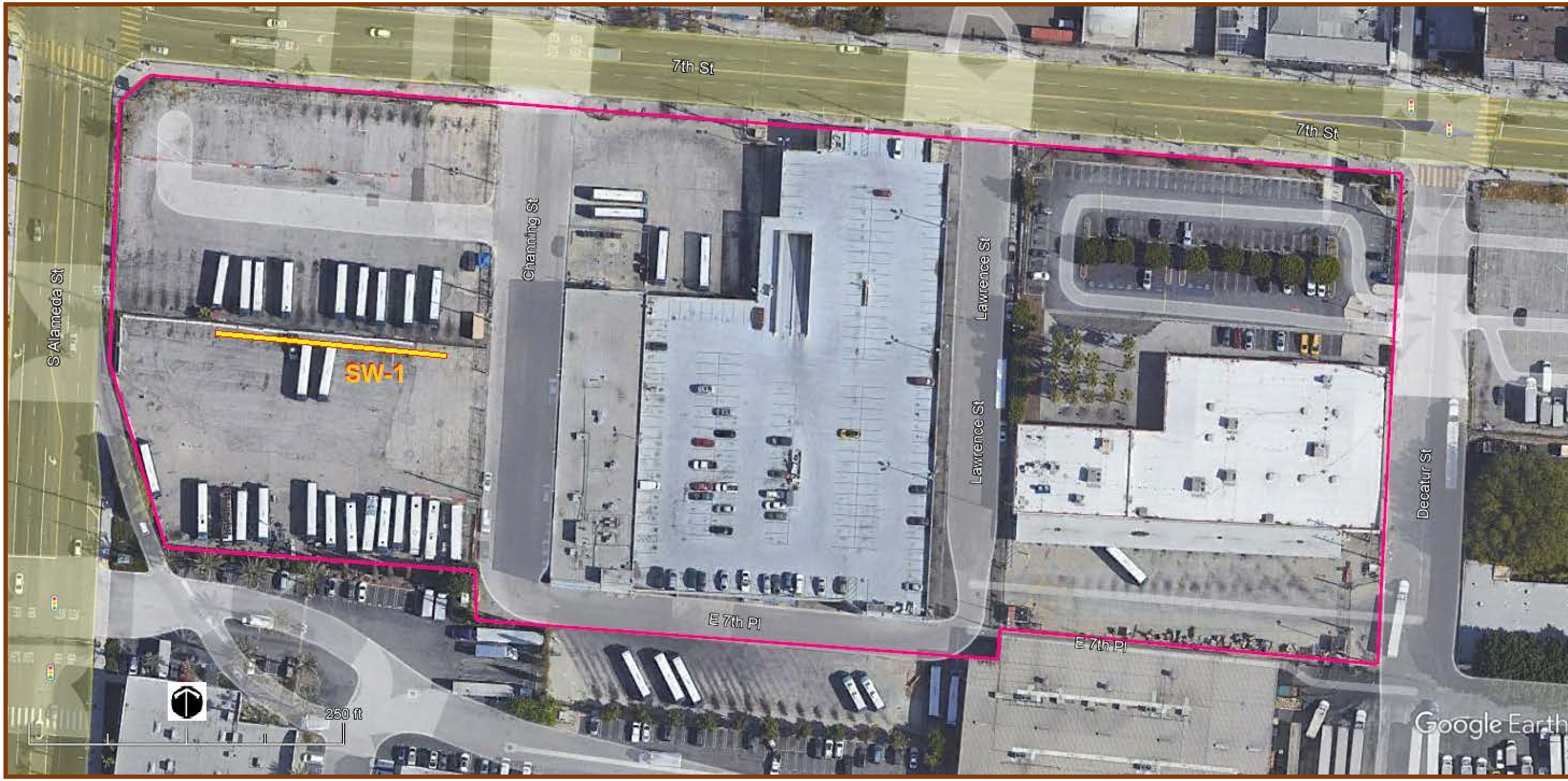
Our conclusions and recommendations are based on an interpretation of available existing geologic, geophysical, geotechnical, and seismic data. No subsurface exploration was performed by this firm for this evaluation. We make no warranty, either express or implied. Should conditions be encountered at a later date or more information becomes available that appear to be different than those indicated in this report, we reserve the right to reevaluate our conclusions and recommendations and provide appropriate mitigation measures, if warranted. If this report is not understood, it is the responsibility of the owner, contractor, engineer, and/or governmental agency, etc., to contact this office for further clarification.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Certified Engineering Geologist
CEG 1459

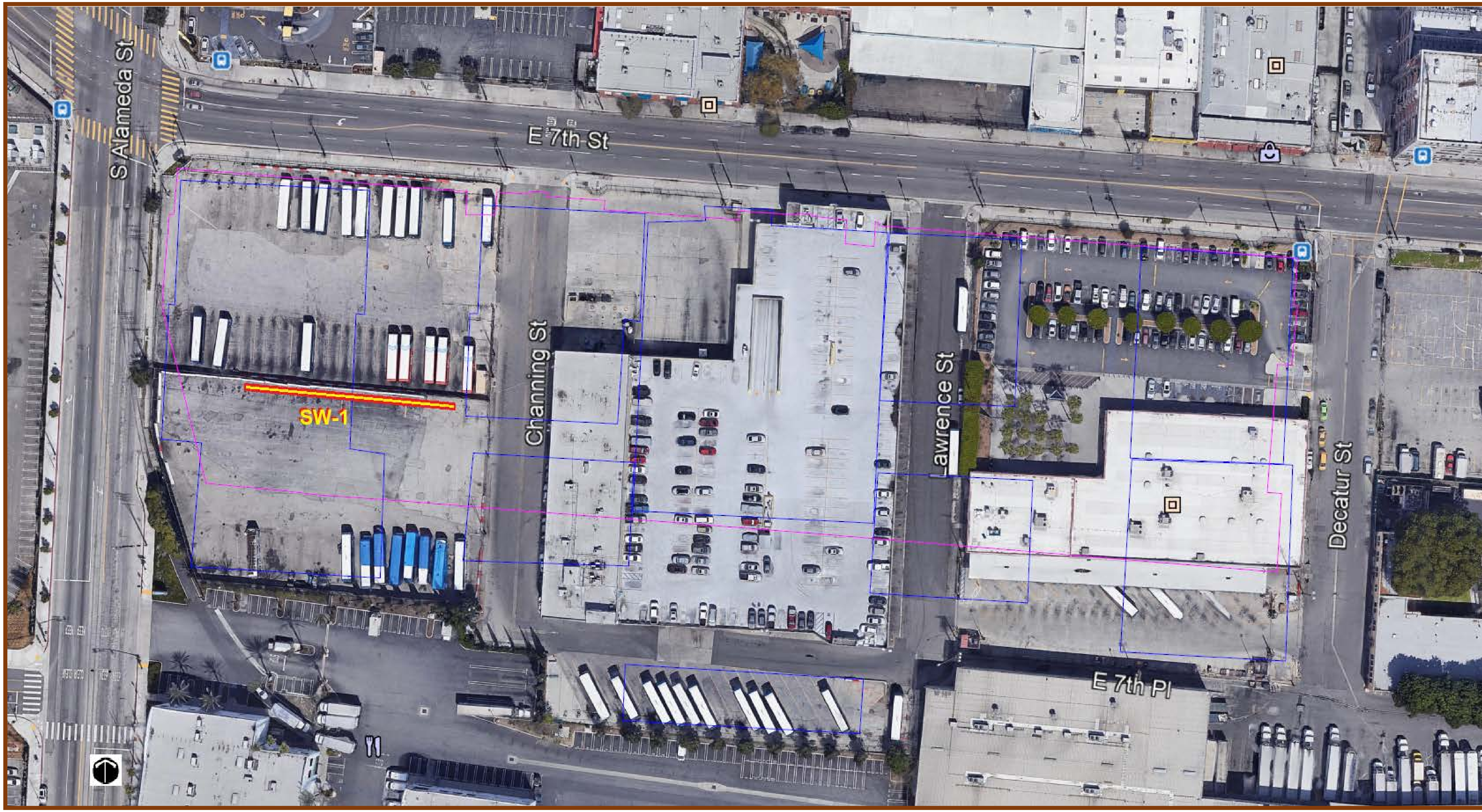
Professional Geophysicist
PGP 1002

GOOGLE™ EARTH IMAGERY MAP



Google™ Earth (2023); Seismic shear-wave survey line (SW-1) shown as yellow line, approximate site boundary outlined in red.

SEISMIC LINE LOCATION MAP



Base Map: Partial modified copy of the provided 80-scale "Proposed Boring Location Plan"; Seismic shear-wave survey line (SW-1) shown as red/yellow line.

APPENDIX A

SHEAR-WAVE SURVEY

SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_S) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

One seismic shear-wave survey traverse was performed at the site as approximated on the Google™ Earth Imagery Map and Seismic Line Location Map, Plates 1 and 2, respectively. For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph. This survey employed both active (MASW) and passive (MAM) source methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005). Both the MASW and MAM survey lines used the same linear geometry array that consisted of a 184-foot-long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals.

For the MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two seismic records were obtained using a 30-foot offset from the beginning and end of the survey line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Each of these shot points used multiple shots (stacking) to improve the signal to noise ratio of the data.

The MAM survey did not require the introduction of artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 20 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Processing

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2004-2021). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies.

Processing of the data proceeded by calculating the dispersion curve from the input data which subsequently created an initial shear-wave model based on the observed data. This initial model was then inverted in order to converge on the best fit of the initial model and the observed data, creating the final shear-wave model (Seismic Line SW-1) as presented within this appendix.

Data Analysis

Data acquisition went very smoothly and the quality was considered to be very good. Analysis revealed that the average shear-wave velocity ("weighted average") in the upper 100 feet of the subject survey area is **1,151.2** feet per second (350.9 meters per second) as shown on the Shear-Wave Model for Seismic Line SW-1, as presented within this appendix. This average velocity classifies the underlying soils to that of Site Class "**D**" ("Stiff Soil"), which has a velocity range from 600 to 1,200 ft/sec (ASCE, 2017; Table 20.3-1).

The "weighted average" velocity is computed from a formula that is used by the ASCE (2017; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V_{100}).

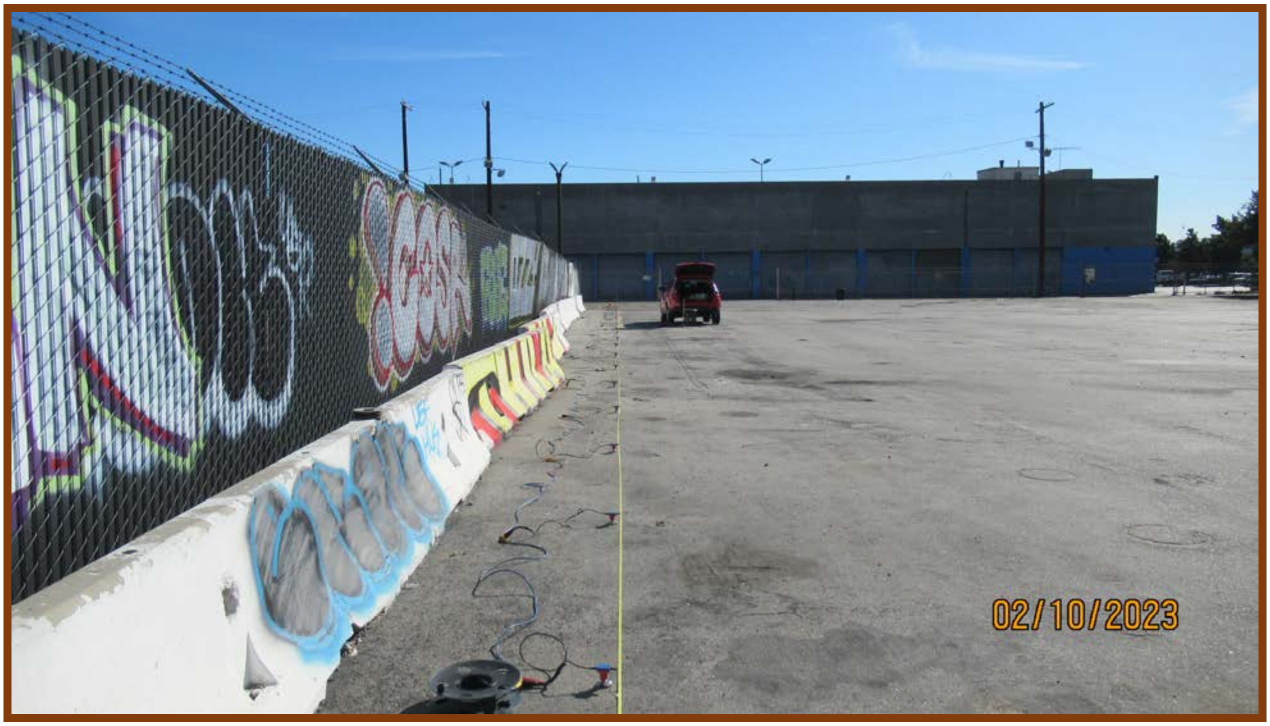
$$Vs = 100/[(d1/v1) + (d2/v2) + ...+ (dn/vn)]$$

Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the 171-foot profile where locally measured. The constrained data is represented by the dark-gray shading on the shear-wave model. The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model, are also included within this appendix, for reference purposes.

Limitations

This survey was performed using "state of the art" geophysical equipment, techniques, and computer software. We make no warranty, either expressed or implied. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation. Compared with traditional borehole shear-wave surveys of which use vertical body waves, the sources of error (if present) using horizontal surface waves for this project are not believed to be greater than 15 percent. It is also important to understand that the fundamental limitation for seismic surveys is known as nonuniqueness, wherein a specific seismic data set does not provide sufficient information to determine a single "true" earth model. Therefore, the interpretation of any seismic data set uses "best-fit" approximations along with the geologic models that appear to be most reasonable for the local area being surveyed.

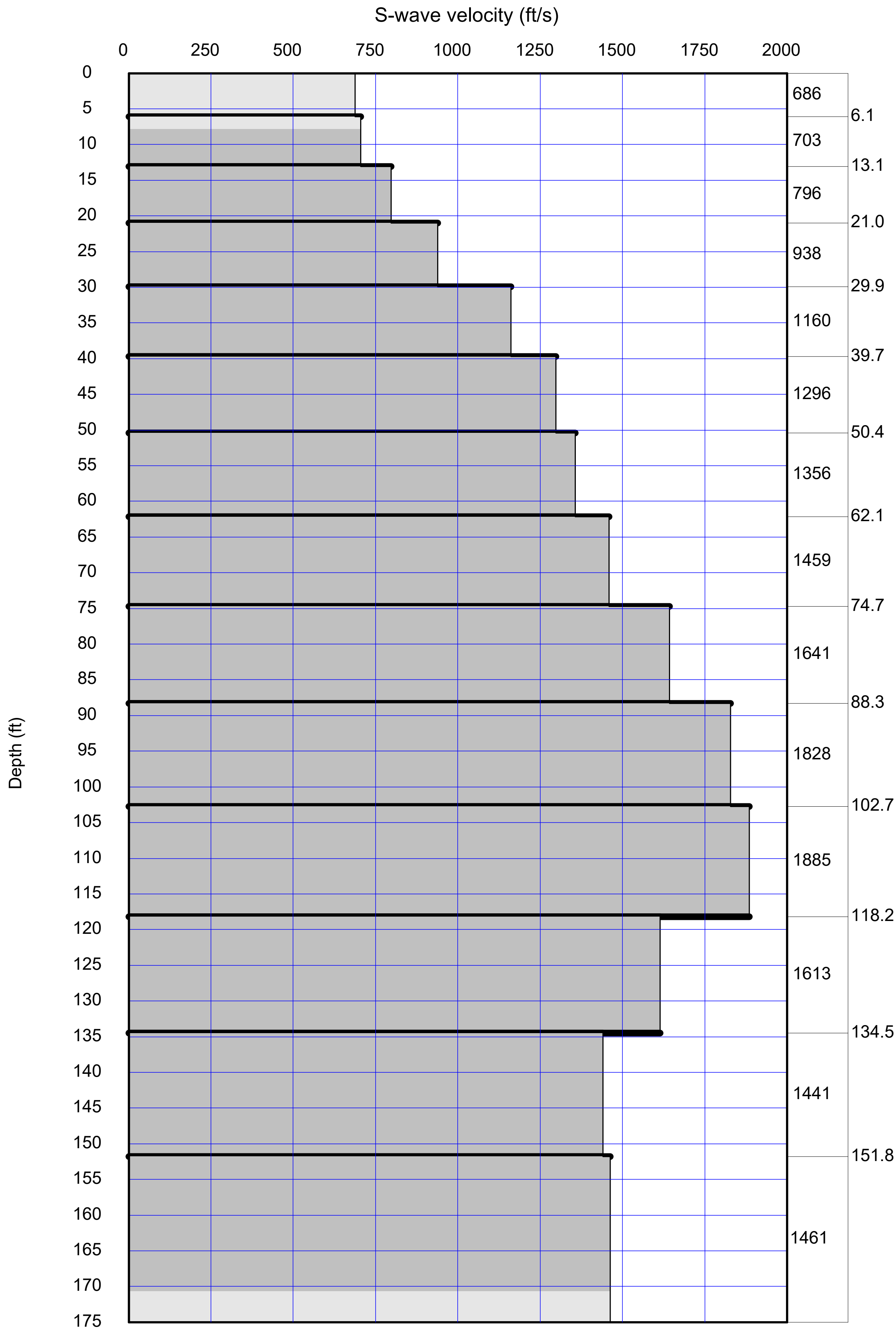
SHEAR-WAVE SURVEY LINE PHOTOGRAPHS



View looking easterly along Seismic Line SW-1.

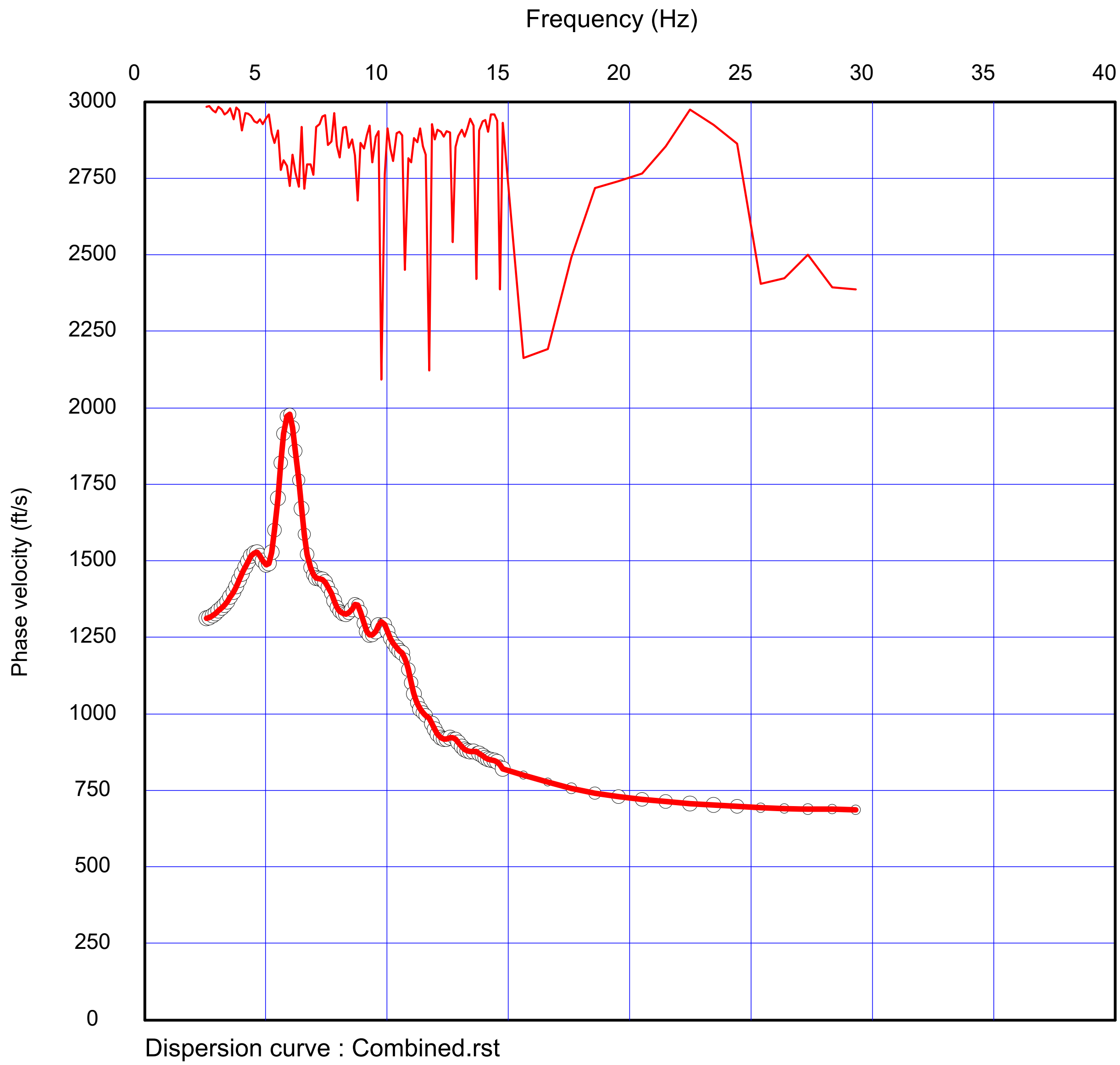
View looking westerly along Seismic Line SW-1.

SEISMIC LINE SW-1



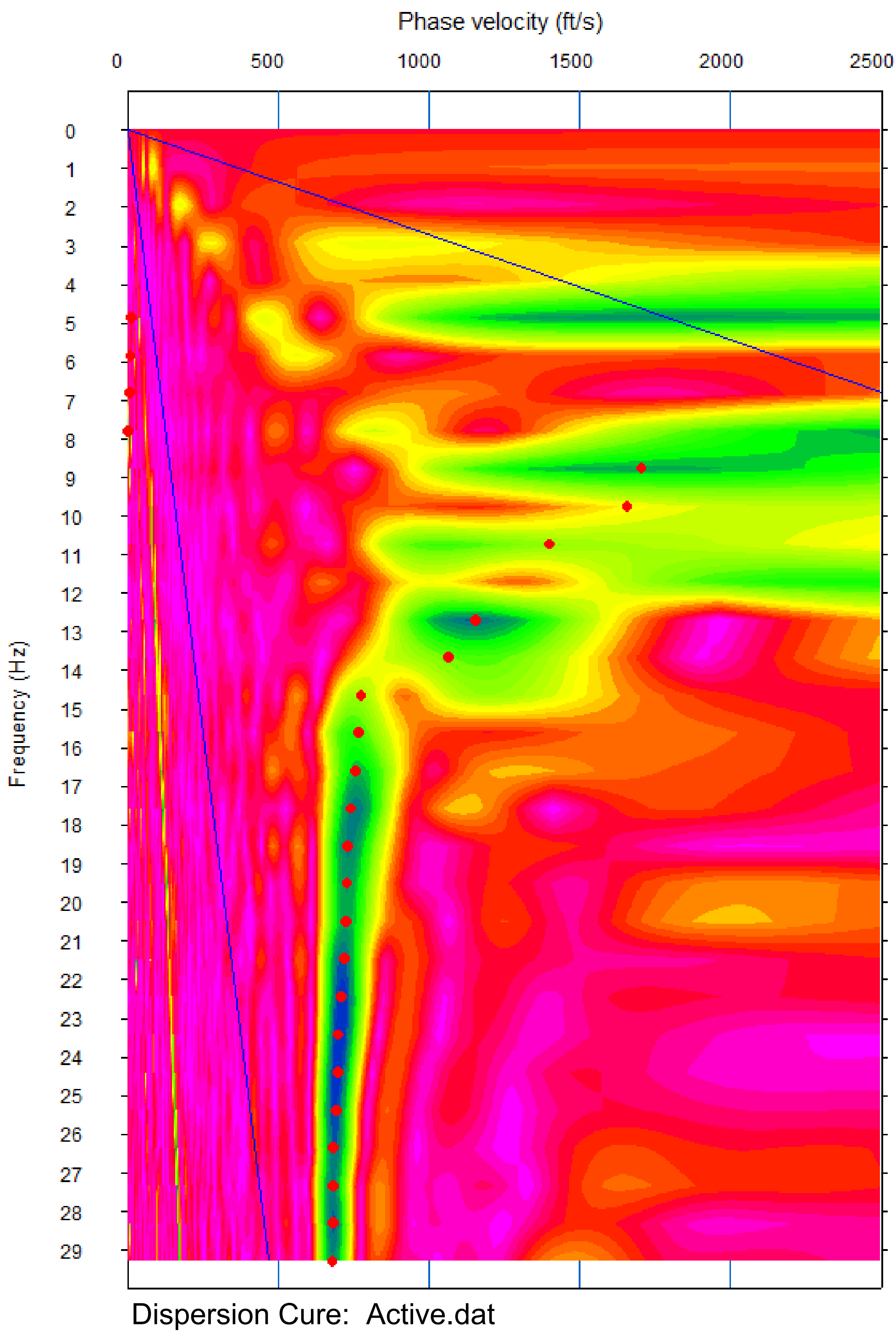
S-wave velocity model (inverted): Final.rst **Average Vs 100ft = 1151.2 ft/sec**

SEISMIC LINE SW-1



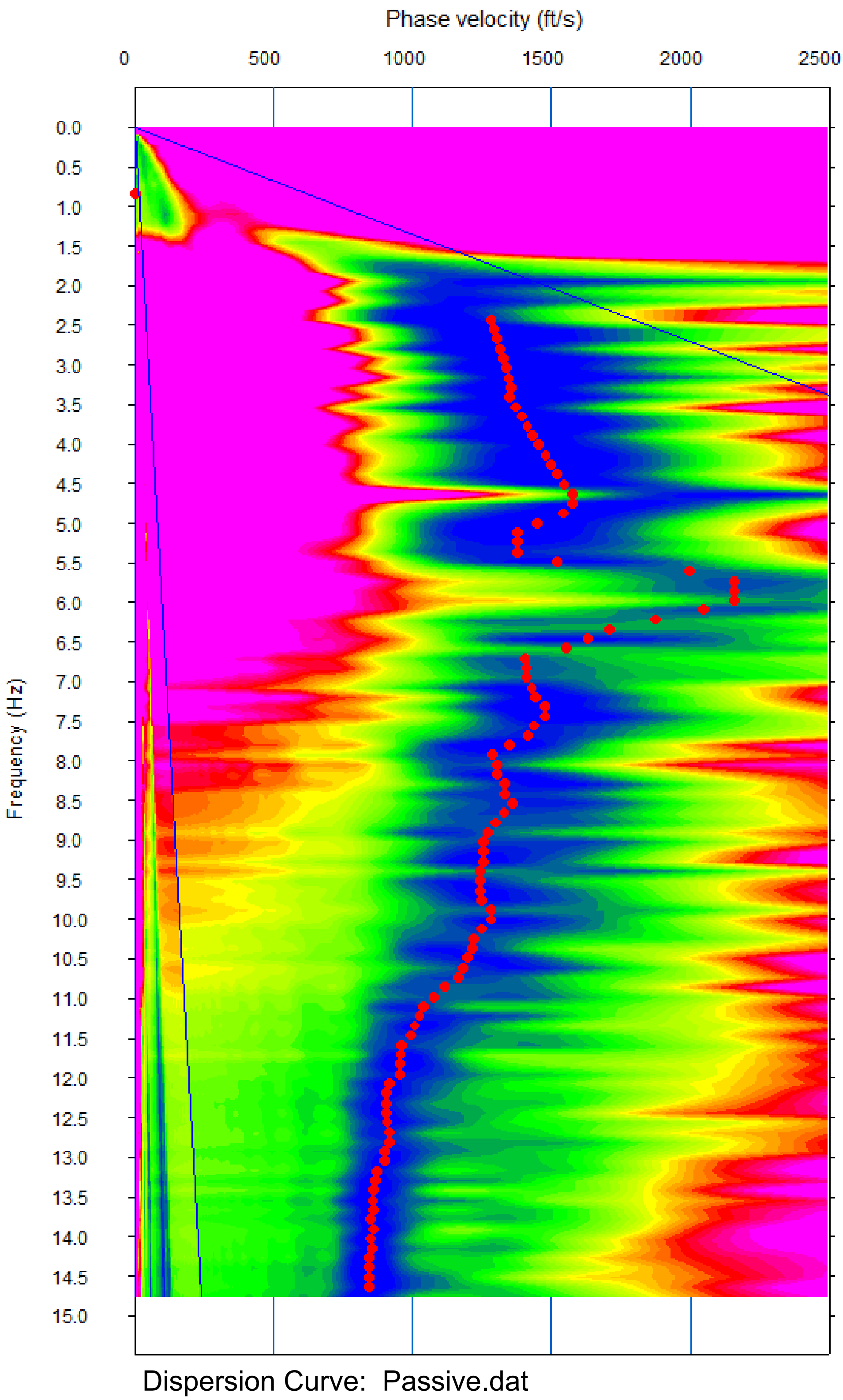
COMBINED DISPERSION CURVE

SEISMIC LINE SW-1



ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



PASSIVE DISPERSION CURVE

APPENDIX B

SITE-SPECIFIC GROUND MOTION ANALYSIS

SITE-SPECIFIC GROUND MOTION ANALYSIS

A detailed summary of the site-specific ground motion analysis, which follows Section 21 of the ASCE Standard 7-16 (2017) and the 2022 California Building Code is presented below, with the Seismic Design Parameters Summary included within this appendix following the summary text.

- **Mapped Spectral Acceleration Parameters (CBC 1613.2.1)**-

  Based on maps prepared by the U.S.G.S (Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for the 0.2 and 1-second Spectral Response Acceleration (5% of Critical Damping; Site Class B/C), a value of **1.920g** for the 0.2 second period (S_S) and **0.684** for the 1.0 second period (S_1) was calculated (ASCE 7-16 Figures 22-1, 22-2 and CBC 1613.2.1).

- **Site Classification (CBC 1613.2.2 & ASCE 7-16 Chapter 20)**-

  Based on the site-specific measured shear-wave value of 1,151.2 feet/second (350.9 m/sec), the soil profile type used should be Site Class **"D."** This Class is defined as having the upper 100 feet (30 meters) of the subsurface being underlain by "Stiff Soil" with average shear-wave velocities of 600 to 1,200 feet/second (180 to 360 meters/second), as detailed within this appendix.

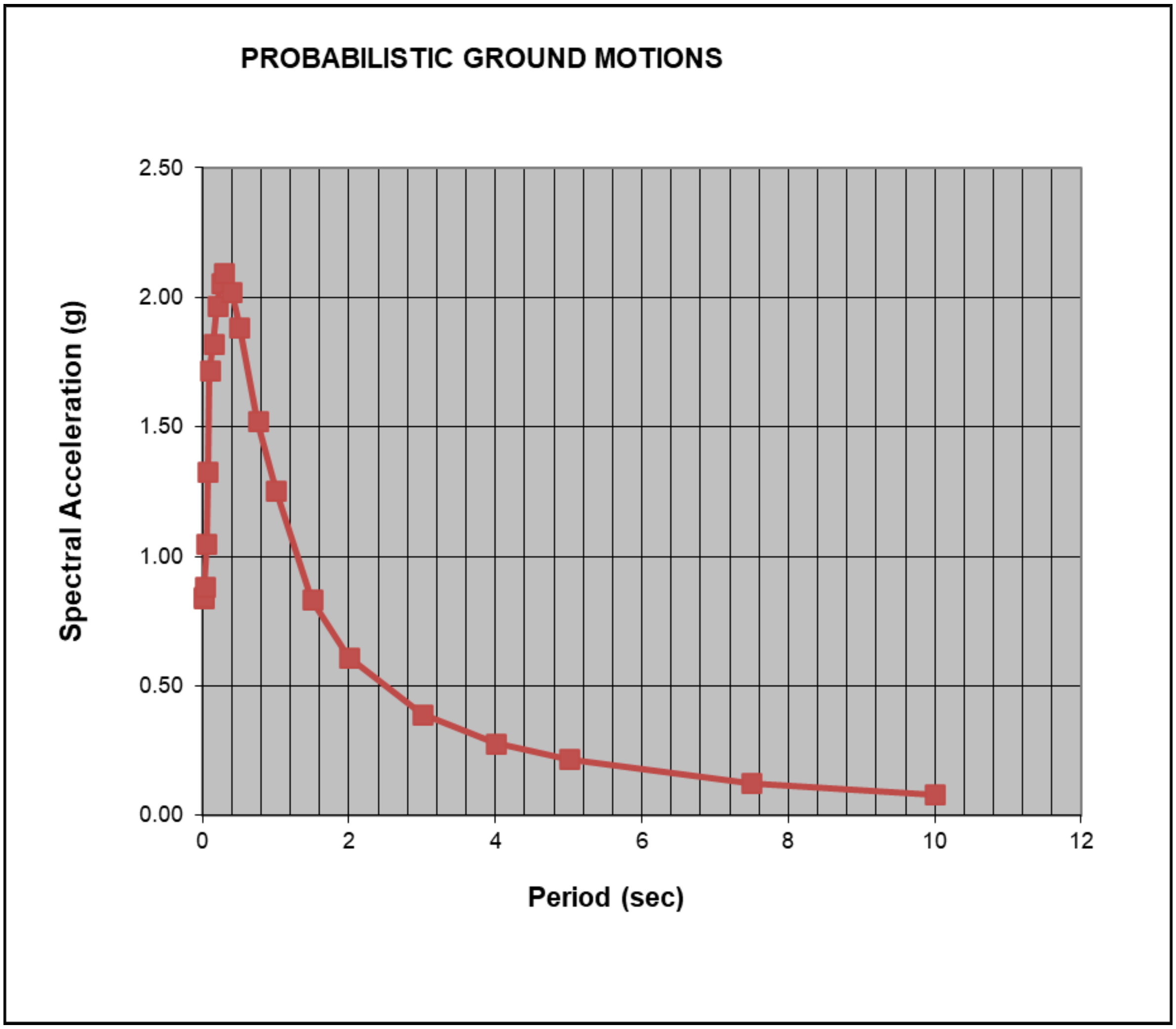
- **Site Coefficients (CBC 1613.2.3)**-

  Based on CBC Tables 1613.2.3(1) and 1613.2.3(2), the site coefficient $\mathbf{F_a = 1.0}$ and $\mathbf{F_v = 1.7}$, respectively.

- **Probabilistic (MCE_R) Ground Motions (ASCE 7 Section 21.2.1.1)**-

  Per Section 21.2.1.1 (**Method 1**), the probabilistic MCE spectral accelerations shall be taken as the spectral response accelerations in the direction of maximum response represented by a five percent damped acceleration response spectrum that is expected to achieve a one percent probability of collapse within a 50-year period.

  The probabilistic analysis included the use of the Open Seismic Hazard Analysis (OpenSHA). The selected Earthquake Rupture Forecast (ERF) was UCERF3 along with a Probability of Exceedance of 2% in 50 Years. The average of four Next Generation Attenuation West-2 Relations (2014 NGA) were utilized to produce a response spectrum. These included Chiou & Youngs (2014), Abrahamsom et al. (2014), Campbell & Bozorgnia (2014), and Boore et al. (2014). The Probabilistic Risk Targeted Response Spectrum was determined as the product of the ordinates of the probabilistic response spectrum and the applicable risk coefficient (C_R). These values were then modified to produce a spectrum based upon the maximum rotated components of ground motion. The resulting MCE_R Response Spectrum is indicated below:



- ***Deterministic Spectral Response Analyses (ASCE 7 Section 21.2.2)***-

The deterministic MCE_R response acceleration at each period shall be calculated as an 84^{th}-percentile 5 percent damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for the characteristic earthquakes on all known active faults within the region shall be used. Analyses were conducted using the average of four Next Generation Attenuation West-2 Relations (2014 NGA), including Chiou & Youngs (2014), Abrahamsom et al. (2014), Campbell & Bozorgnia (2014), and Boore et al. (2014).

Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (UCERF 3; Field et al., 2013) and published geologic data, the nearest six significant faults were evaluated for this analysis, as depicted on the UCERF3 model. The faults used in this analysis were the Elysian Park (upper and lower), Newport-Inglewood, Puente Hills (subsection 4), Compton Thrust, San Andreas, and the Whittier Fault. Of these faults, the controlling faults were found to be the Puente Hills Fault (Subsection 4), Compton Trust, and the Whittier Fault (see Page 3 of 6 in the following "Seismic Design Parameters Summary" further in this appendix).

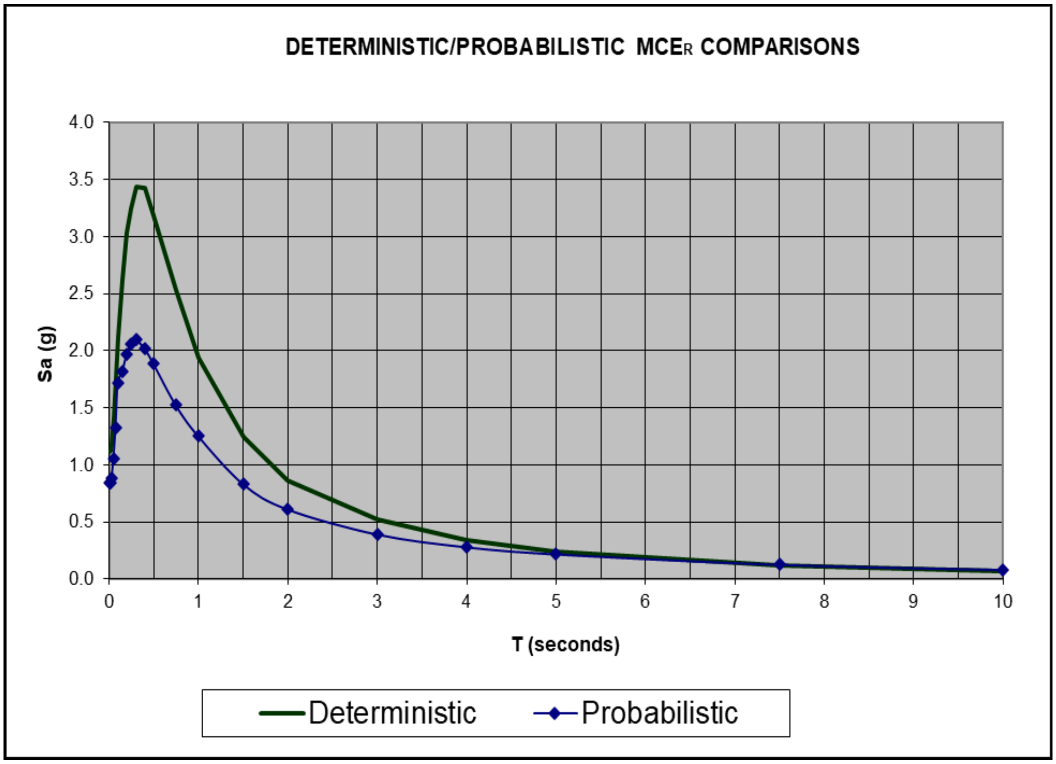
- **Site Specific MCE_R (ASCE 7 Section 21.2.3)**-

The site-specific MCE_R spectral response acceleration at any period, S_{aM}, shall be taken as the lesser of the spectral response accelerations from the probabilistic ground motions of Section 21.2.1 and the deterministic ground motions of Section 21.2.2. The deterministic ground motions were compared with the probabilistic ground motions that were determined in accordance with Section 21.2.1. These are plotted in the following diagram:

Comparison of Deterministic MCE_R Values with Probabilistic MCE_R Values - Section 21.2.3

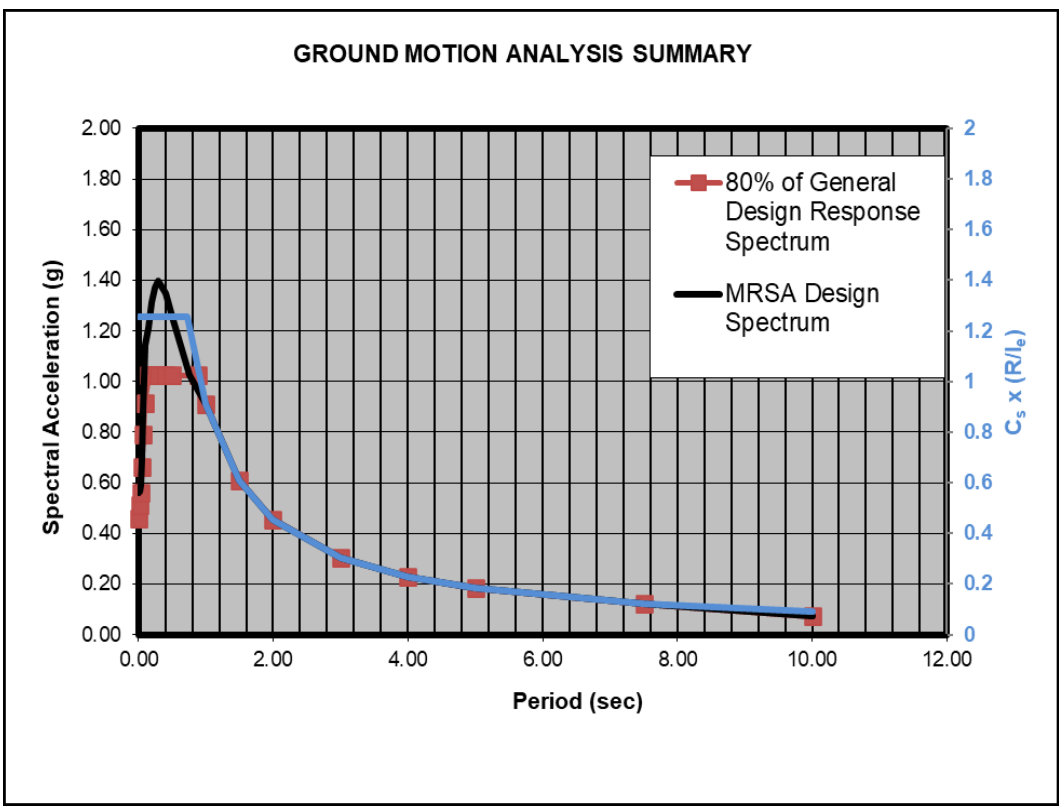
Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	1.11	0.84	0.84	Probabilistic Governs
0.020	1.12	0.84	0.84	Probabilistic Governs
0.030	1.18	0.88	0.88	Probabilistic Governs
0.050	1.41	1.05	1.05	Probabilistic Governs
0.075	1.77	1.33	1.33	Probabilistic Governs
0.100	2.12	1.72	1.72	Probabilistic Governs
0.150	2.62	1.82	1.82	Probabilistic Governs
0.200	3.03	1.97	1.97	Probabilistic Governs
0.250	3.25	2.06	2.06	Probabilistic Governs
0.300	3.44	2.09	2.09	Probabilistic Governs
0.400	3.43	2.02	2.02	Probabilistic Governs
0.500	3.18	1.89	1.89	Probabilistic Governs
0.750	2.53	1.52	1.52	Probabilistic Governs
1.000	1.94	1.25	1.25	Probabilistic Governs
1.500	1.24	0.83	0.83	Probabilistic Governs
2.000	0.86	0.61	0.61	Probabilistic Governs
3.000	0.52	0.39	0.39	Probabilistic Governs
4.000	0.34	0.28	0.28	Probabilistic Governs
5.000	0.24	0.22	0.22	Probabilistic Governs
7.500	0.12	0.12	0.12	Deterministic Governs
10.000	0.07	0.08	0.07	Deterministic Governs

These comparisons are plotted in the following diagram:



- ♦ ***Design Response Spectrum (ASCE 7 Section 21.3)***-

  In accordance with Section 21.3, the Design Response Spectrum was developed by the following equation: $S_a = 2/3S_{aM}$, where S_{aM} is the MCE_R spectral response acceleration obtained from Section 21.1 or 21.2. The design spectral response acceleration shall not be taken less than 80 percent of S_a. These are plotted and compared with 80% of the CBC Spectrum values in the following diagram:



- **Design Acceleration Parameters (ASCE 7 Section 21.4)**-

  Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall obtained from the site-specific spectra at a period of 0.2 s, except that it shall not be taken less than 90 percent of the peak spectral acceleration, S_a, at any period larger than 0.2 s. The parameter S_{D1} shall be taken as the greater of the products of Sa * T for periods between 1 and 5 seconds. The parameters S_{MS}, and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80 percent of the values determined in accordance with Section 11.4.4 for S_{MS}, and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

- **Site Specific Design Parameters** -

  For the 0.2 second period (S_{DS}), a value of 1.26g was computed, based upon the average spectral accelerations. The maximum average acceleration for any period exceeding 0.2 seconds was 1.40g occurring at T=0.30 seconds. This was multiplied by 0.9 to produce a value of 1.26g making this the applicable value. A value of 0.91g was calculated for S_{D1} at a period of 1 second (ASCE 7-16, 21.4). For the MCE_R 0.2 second period, a value of 1.885g (S_{MS}) was computed, along with a value of 1.368g (S_{M1}) for the MCE_R 1.0 second period was also calculated (ASCE 7-16, 21.2.3).

- **Site-Specific MCE_G Peak Ground Accelerations (ASCE 7 Section 21.5)**-

  The probabilistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 0.83g. The deterministic geometric mean peak ground acceleration (largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 1.00g. The site-specific MCE_G peak ground acceleration was calculated to be **0.83g**, which was determined by using the lesser of the probabilistic (0.83g) or the deterministic (1.00g) geometric mean peak ground accelerations, but not taken as less than 80 percent of PGA_M (i.e., 0.90g x 0.80 = 0.72g).

SEISMIC DESIGN PARAMETERS SUMMARY

Project: Proposed Commercial/Industrial Develop Lattitude: 34.0342
Project #: 233920-1 Longitude: -118.2367
Date: 2/13/2023

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

Mapped Acceleration Parameters per ASCE 7-16, Chapter 22

S_s=	1.92	Figure 22-1
S_1=	0.684	Figure 22-2

Site Class per Table 20.3-1

Site Class=	D - Stiff Soil

Site Coefficients per ASCE 7-16 CHAPTER 11

F_a=	1	Table 11.4-1	=	1	For Site Specific Analysis per ASCE7-16 21.3
F_v=	1.7	Table 11.4-2	=	2.50	For Site Specific Analysis per ASCE7-16 21.3

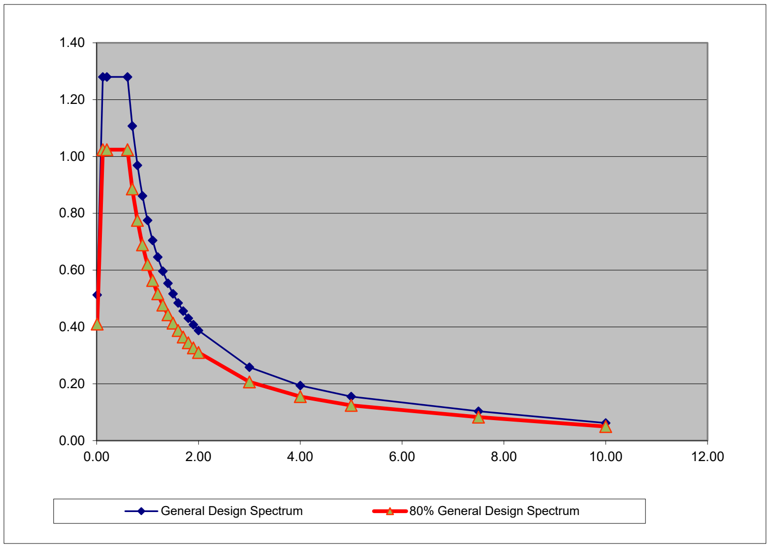
Mapped Design Spectral Response Acceleration Parameters

S_{Ms}=	1.92	Equation 11.4-1	1.92	For Site Specific Analysis per ASCE7-16 21.3
S_{M1}=	1.163	Equation 11.4-2	1.710	For Site Specific Analysis per ASCE7-16 21.3

S_{DS}=	1.280	Equation 11.4-3
S_{D1}=	0.775	Equation 11.4-4

T_0=	0.121	sec	
T_S=	0.606	sec	
T_L=	8	sec	From Fig 22-12
PGA	0.822	g	
F_{PGA}=	1.1		From Table 11.8-1
C_{RS}=	0.901		Figure 22-17
C_{R1}=	0.899		Figure 22-18

Period (T)	Sa (ASCE7-16 - 11.4.6)	80% General Design Spectrum
0.01	0.51	0.41
0.12	1.28	1.02
0.20	1.28	1.02
0.61	1.28	1.02
0.70	1.11	0.89
0.80	0.97	0.78
0.90	0.86	0.69
1.00	0.78	0.62
1.10	0.70	0.56
1.20	0.65	0.52
1.30	0.60	0.48
1.40	0.55	0.44
1.50	0.52	0.41
1.60	0.48	0.39
1.70	0.46	0.36
1.80	0.43	0.34
1.90	0.41	0.33
2.00	0.39	0.31
3.00	0.26	0.21
4.00	0.19	0.16
5.00	0.16	0.12
7.50	0.10	0.08
10.00	0.06	0.05



ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

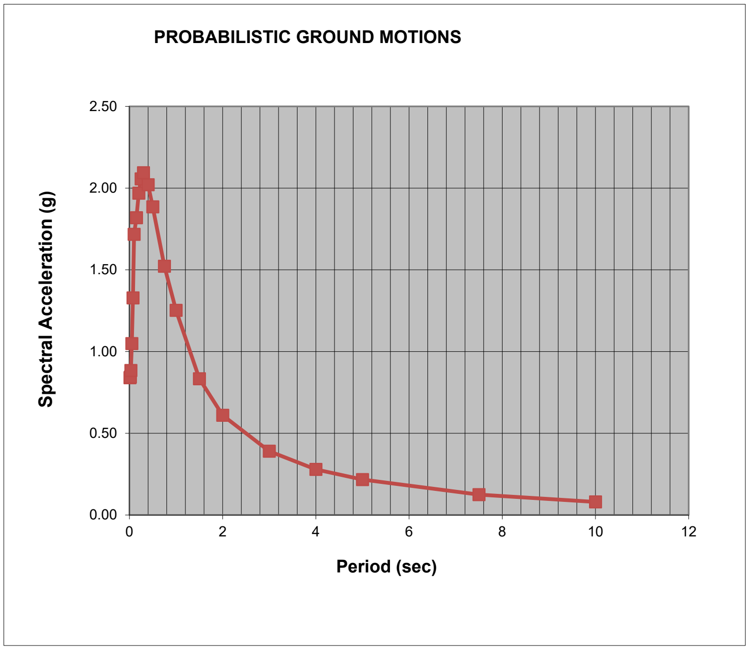
Use Maximum Rotated Horizontal Component?* (Y/N) Y

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships
Earthquake Rupture Forecast - UCERF3 FM 3.2

PROBABILISTIC MCER per 21.2.1.1 Method 1
Risk Coefficients taken from Figures 22-18 and 22-19 of ASCE 7-16
OpenSHA data
2% Probability Of Exceedance in 50 years
Maximum Rotated Horizontal Component determined per ASCE7-16

T	Sa 2% in 50	MCER
0.01	0.93	0.84
0.02	0.94	0.84
0.03	0.98	0.88
0.05	1.16	1.05
0.08	1.47	1.33
0.10	1.73	1.72
0.15	2.02	1.82
0.20	2.19	1.97
0.25	2.28	2.06
0.30	2.32	2.09
0.40	2.24	2.02
0.50	2.09	1.89
0.75	1.69	1.52
1.00	1.39	1.25
1.50	0.93	0.83
2.00	0.68	0.61
3.00	0.43	0.39
4.00	0.31	0.28
5.00	0.24	0.22
7.50	0.14	0.12
10.00	0.09	0.08

S_s=	2.19	1.97
S_1=	1.39	1.25
PGA	0.83 g	



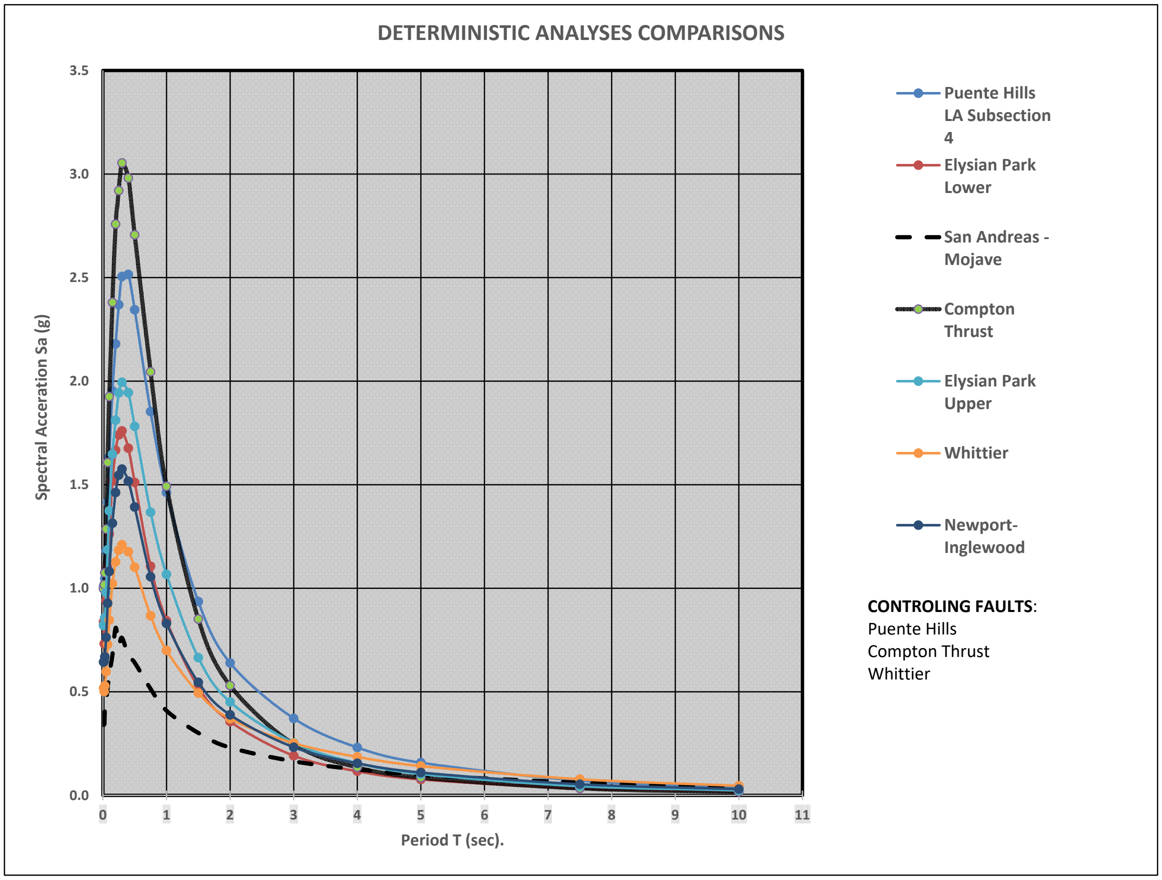
Risk Coefficients:			
C_{RS}	0.901	Figure 22-18	Get from Mapped Values
C_{R1}	0.899	Figure 22-19	
Fa=	1	Table 11.4-1	Per ASCE7-16 - 21.2.3
Is $Sa_{(max)}$<1.2XFa?		NO	If "YES", Probabilistic Spectrum prevails

DETERMINISTIC MCE per 21.2.2

Preliminary Assessment:

Fault	Distance (km)
Puente Hills LA Subsection 4	1.4
Elysian Park Lower	1.6
Whittier	17.5
Compton Thrust	26.5
Newport-Inglewood	11.5
Elysian Park Upper	3.3
San Andreas - Mojave	56.6

Seven faults were considered on the bases of their relative proximities to the site and the contributions to the seismic hazard revealed in disaggregation analyses. The Probabilistic analyses revealed the Puente Hills, Elysian Park (lower) and San Andreas (Mojave S) faults as the major contributors to the hazard. Preliminary screening revealed four faults contributing to the Deterministic hazard.



Input Parameters		Puente Hills LA Subsection 4	Elysian Park Lower	Whittier	Compton Thrust
Fault					
M	= Moment magnitude	7	6.7	7.8	7.5
R_{RUP}	= Closest distance to coseismic rupture (km)	3.2	10.2	17.5	12.9
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	0	0	17.5	0
Rx	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	1.4	1.6	17.5	26.5
U	= Unspecified Faulting Flag (Boore et.al.)	0	0	0	0
F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	1	1	0	1
F_{NM}	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique	0	0	0	1
F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08	1	1	0	1
Z_{TOR}	= Depth to top of coseismic rupture (km)	2.1	10	0	5.2
δ	= Average dip of rupture plane (degrees)	50	22	90	20
V_{S30}	= Average shear-wave velocity in top 30m of site profile	350.9	350.9	350.9	350.9
$F_{Measured}$		1	1	1	1
$Z_{1.0}$	= Depth to Shear Wave Velocity of 1.0 km/sec (km)	0.4	0.4	0.4	0.4
$Z_{2.5}$	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	2.25	2.25	2.25	2.25
Site Class		D	D	D	D
W (km)	= Fault rupture width (km)	28.4	25.3	15	30.4
F_{AS}	= 0 for mainshock; 1 for aftershock	0	0	0	0
σ	=Standard Deviation	1	1	1	1

Deterministic Summary - Section 21.2.2 (Supplement 1)

T	Puente Hills LA Subsection 4	Elysian Park Lower	Whittier	Compton Thrust	Maximum S_a (Average)	Corrected* S_a (per ASCE7-16)	Scaled $S_{a(Average)}$	Controlling Fault
0.010	1.01	0.84	0.52	1.00	1.01	1.11	1.11	Puente Hills LA Subsection
0.020	1.02	0.73	0.50	1.02	1.02	1.12	1.12	Puente Hills LA Subsection
0.030	1.05	0.86	0.53	1.07	1.07	1.18	1.18	Compton Thrust
0.050	1.18	0.94	0.60	1.29	1.29	1.41	1.41	Compton Thrust
0.075	1.41	1.10	0.73	1.61	1.61	1.77	1.77	Compton Thrust
0.100	1.63	1.26	0.85	1.93	1.93	2.12	2.12	Compton Thrust
0.150	1.95	1.52	1.02	2.38	2.38	2.62	2.62	Compton Thrust
0.200	2.18	1.67	1.13	2.76	2.76	3.03	3.03	Compton Thrust
0.250	2.37	1.74	1.18	2.92	2.92	3.25	3.25	Compton Thrust
0.300	2.50	1.76	1.21	3.05	3.05	3.44	3.44	Compton Thrust
0.400	2.52	1.68	1.18	2.98	2.98	3.43	3.43	Compton Thrust
0.500	2.34	1.51	1.10	2.71	2.71	3.18	3.18	Compton Thrust
0.750	1.85	1.11	0.87	2.05	2.05	2.53	2.53	Compton Thrust
1.000	1.46	0.84	0.70	1.49	1.49	1.94	1.94	Compton Thrust
1.500	0.94	0.52	0.49	0.85	0.94	1.24	1.24	Puente Hills LA Subsection
2.000	0.64	0.36	0.37	0.53	0.64	0.86	0.86	Puente Hills LA Subsection
3.000	0.37	0.19	0.25	0.25	0.37	0.52	0.52	Puente Hills LA Subsection
4.000	0.23	0.12	0.19	0.14	0.23	0.34	0.34	Puente Hills LA Subsection
5.000	0.16	0.08	0.14	0.09	0.16	0.24	0.24	Puente Hills LA Subsection
7.500	0.07	0.03	0.08	0.03	0.08	0.12	0.12	Whittier
10.000	0.04	0.02	0.05	0.02	0.05	0.07	0.07	Whittier
PGA	1.00	0.72	0.50	0.99	1.00		1.00	g

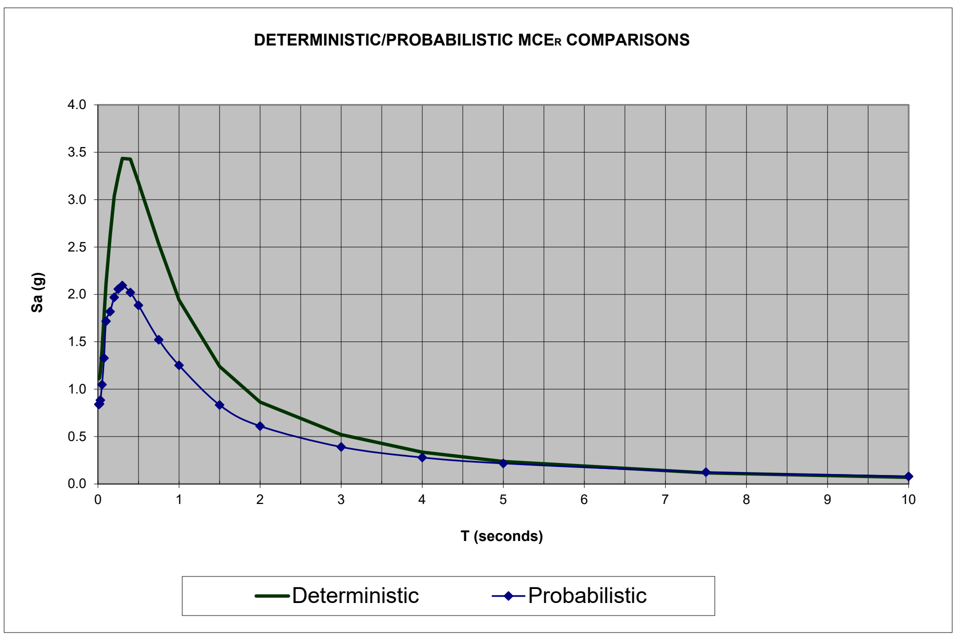
Max Sa=	3.44	
Fa =	1.00	Per ASCE7-16 21.2.2
1.5XFa=	1.5	
Scaling Factor=	1.00	

* Correction is the adjustment for Maximum Rotated Value if Applicable

SITE SPECIFIC MCE_R - Compare Deterministic MCE_R Values (S_a) with Probabilistic MCE_R Values (S_a) per 21.2.3

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships

Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	1.11	0.84	0.84	ProbabilisticGoverns
0.020	1.12	0.84	0.84	ProbabilisticGoverns
0.030	1.18	0.88	0.88	ProbabilisticGoverns
0.050	1.41	1.05	1.05	ProbabilisticGoverns
0.075	1.77	1.33	1.33	ProbabilisticGoverns
0.100	2.12	1.72	1.72	ProbabilisticGoverns
0.150	2.62	1.82	1.82	ProbabilisticGoverns
0.200	3.03	1.97	1.97	ProbabilisticGoverns
0.250	3.25	2.06	2.06	ProbabilisticGoverns
0.300	3.44	2.09	2.09	ProbabilisticGoverns
0.400	3.43	2.02	2.02	ProbabilisticGoverns
0.500	3.18	1.89	1.89	ProbabilisticGoverns
0.750	2.53	1.52	1.52	ProbabilisticGoverns
1.000	1.94	1.25	1.25	ProbabilisticGoverns
1.500	1.24	0.83	0.83	ProbabilisticGoverns
2.000	0.86	0.61	0.61	ProbabilisticGoverns
3.000	0.52	0.39	0.39	ProbabilisticGoverns
4.000	0.34	0.28	0.28	ProbabilisticGoverns
5.000	0.24	0.22	0.22	ProbabilisticGoverns
7.500	0.12	0.12	0.12	Deterministic Governs
10.000	0.07	0.08	0.07	Deterministic Governs



DESIGN RESPONSE SPECTRUM per Section 21.3

DESIGN ACCELERATION PARAMETERS per Section 21.4 (MRSA)

Period	2/3*MCE_R	80% General Design Response Spectrum (per ASCE 7-16 23.3-1)	Design Response Spectrum	TXSa
0.01	0.56	0.44	0.56	
0.02	0.56	0.47	0.56	
0.03	0.59	0.50	0.59	
0.05	0.70	0.56	0.70	
0.08	0.89	0.64	0.89	
0.10	1.14	0.72	1.14	
0.15	1.21	0.87	1.21	
0.20	1.31	1.02	1.31	
0.25	1.37	1.02	1.37	
0.30	1.40	1.02	1.40	
0.40	1.35	1.02	1.35	
0.50	1.26	1.02	1.26	
0.75	1.01	1.02	1.02	
1.00	0.83	0.91	0.91	0.91
1.50	0.56	0.61	0.61	0.91
2.00	0.41	0.46	0.46	0.91
3.00	0.26	0.30	0.30	0.91
4.00	0.19	0.23	0.23	0.91
5.00	0.14	0.18	0.18	0.91
7.50	0.08	0.12	0.12	
10.00	0.05	0.07	0.07	

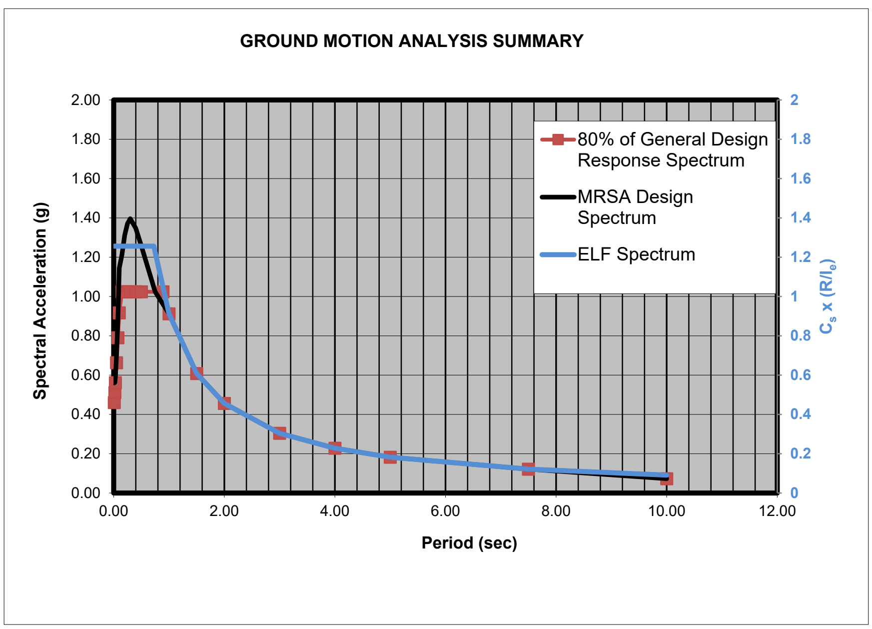
Highest value of S_a for any period exceeding 0.2 sec.=	1.40
90%of Highest Value =	1.26
80% 0f Mapped S_{DS}=	1.02
Maximum TXSa from T=1s-5s =	0.91
80% of Mapped S_{D1}=	0.62

S_{DS}=	1.26
S_{D1}=	0.91
Ts =	0.73

S_{MS}=	1.885
S_{M1}=	1.368

PGA Determination:

Site Coefficient F_{PGA}=	1.1	
Mapped PGA=	0.82	Figure 22-7
PGA_M =	0.90	g
Deterministic PGA =	1.00	g
Probabilistic PGA =	0.83	g
Lesser of Deterministic/Probabilistic =	0.83	g
80% of PGA_{M}=	0.72	g
MCE_G PGA=	0.83	g



APPENDIX C

REFERENCES

REFERENCES

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads and Associated Criteria for Buildings and other Structures, ASCE Standard 7-16, 889pp.

American Society for Testing and Materials, Intl. (ASTM), 2000, Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation, Designation D 5777-00, 13 pp.

California Building Standards Commission (CBSC), 2019, 2022 California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2.

California's Office of Statewide Health Planning and Development (OSHPD), 2023, OSHPD Seismic Design Maps Tool web application, https://seismicmaps.org/.

California State Board for Geologists and Geophysicists, 1998, Department of Consumer Affairs, Guidelines for Geophysical Reports for Environmental and Engineering Geology, 5 pp.

Crice, Douglas B., undated, Shear Waves, Techniques and Systems, Reprinted by Geometrics, Sunnyvale, California.

Dibblee, T.W. Jr., 1989, Geologic Map of the Los Angeles 7.5' Quadrangle, Los Angeles County, California, Dibblee Geology Center Map #DF-22, Scale 1: 24,000.

Field, E.H., et al., 2013, Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3)—The Time-Independent Model, U.S.G.S. Open-File Report 2103-1165, CGS Special Report 228, and Southern California Earthquake Center Publication 1792.

Geometrics, Inc., 2012, StrataVisor™ NZXP Operation and Reference Manual, Revision M, Software Version 9.3, San Jose, California, 234 pp.

Geometrics, Inc., 2004-2021, SeisImager/SW Analysis of Surface Waves, Pickwin Version 6.0.2.1. and WaveEq Version 5.2.6.0.

Google™ Earth, 2023, http://earth.google.com/, Version 7.3.6.9354 (64-bit).

Louie, J.N., 2001, Faster, Better: Shear-Wave Velocity to 100 Meters Depth from Refraction Microtremor Arrays, *in,* Bulletin of the Seismological Society of America, Volume 91, pp. 347-364.

Okada, H., 2003, The Microtremor Survey Method, Society of Exploration Geophysicists, Geophysical Monograph Series Number 12, 135 pp.

Park, C.B, Milner, R.D., Rynden, N., Xia, J., and Ivanov, J., 2005, Combined use of Active and Passive Surface Waves, *in*, Journal of Environmental and Engineering Geophysics, Volume 10, Issue 3, pp. 323-334.

United States Geological Survey (U.S.G.S.), 2021, Hazard Spectrum Application, Version 1.5.2, http://opensha.org.

United States Geological Survey (U.S.G.S.), 2023, Quaternary Faults and Folds in the U.S. database, https://earthquake.usgs.gov/static/lfs/nshm/qfaults/qfaults.kmz.

Wills, C.J. Weldon II, R.J., and Bryant, W.A., 2007, California Fault Parameters for the Nation Seismic Hazard Maps and Working Group on California Earthquake Probabilities, 2007; Appendix A, CGS Special Report 203A.

Yerkes, R.F., 1997, Preliminary Geologic Map of the Los Angeles 7.5' Quadrangle, California, U.S.G.S. Open-File Report 97-254, Scale 1: 24,000.

May 7, 2024

Prologis
17777 Center Court Drive North, Suite 100
Cerritos, California 90703

Attention: Mr. D.J. Arellano
Vice President, Development - Entitlements

Project No.: **20G243-5R2**

Subject: **Results of Infiltration Testing**
Alameda Crossing Development
1716 East 7th Street
Los Angeles, California

References: 1) Geotechnical Feasibility Study, Proposed Commercial/Industrial Development, 1716 East 7th Street, Los Angeles, California, prepared by Southern California Geotechnical, Inc. (SCG) for Prologis, SCG Project No. 20G243-1 dated January 7, 2021.

2) Geotechnical Investigation, Alameda Crossing Development, 1716 East 7th Street, Los Angeles, California, prepared by SCG for Prologis, SCG Project No. 20G243-4R dated May 7, 2024.

Mr. Arellano:

In accordance with your request, we have conducted infiltration testing at the above-referenced subject site (project site) with regards to the proposed Alameda Crossing development (project). We are pleased to present this report summarizing the results of the infiltration testing and our design recommendations.

Scope of Services

The scope of services performed for this project was in accordance with our Proposal No. 22P339R, dated January 19, 2023, and Change Order No. 20G243-CO5R, dated March 6, 2023. The scope of the infiltration testing consisted of site reconnaissance, subsurface exploration, field testing, and engineering analysis to determine the infiltration rates of the onsite soils. The infiltration testing for the shallow infiltration system was performed in general accordance with the guidelines published by the County of Los Angeles – Department of Public Works Geotechnical and Materials Engineering Division. These guidelines are published in Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration (GS200.1), dated June 30, 2021.

Site Description

The project site is located at the southeast corner of South Alameda Street and East 7th Street in Los Angeles, California. The project site is also referenced by the street address 1716 East 7th Street. The project site is bounded to the north by East 7th Street, to the west by South

Alameda Street, to the south by an existing commercial/industrial building, and to the east by Decatur Street. The general location of the project site is illustrated on the Site Location Map, included as Plate 1 of this report.

The project site consists of several rectangular-shaped parcels which total 8.3± acres in size. The three (3) parcels are transected by two (2) north-south trending streets, identified as Channing Street in the west and Laurence Street in the east. The easternmost parcel is developed with a single-story 30,000± ft^2 commercial/industrial building, located in the south-central area of the parcel. The building was previously used as the Los Angeles Greyhound Station. The building is of concrete tilt-up construction, assumed to be supported on conventional shallow foundations with a concrete slab-on-grade floor. The building is surrounded by asphaltic concrete (AC) pavements in the drive and parking areas, concrete flatwork, and landscaped planters that include shrubs and medium to large trees. The existing AC pavements and concrete flatwork are in poor condition with moderate to severe cracking throughout. The central parcel is developed with an 87,000± ft^2 multi-level maintenance service building. The first level of the central portion of the building was previously used as a washing station for buses. Nearly the entire structure is underlain by a large basement. The building is of concrete tilt-up construction, assumed to be supported on conventional shallow foundations with a concrete slab-on-grade floor. The building is generally surrounded by AC pavements and Portland cement concrete (PCC) pavements in the northwestern region. The existing pavements are in poor condition with minor to severe cracking throughout. The eastern and central buildings are vacant but are currently being used by LAPD for training purposes. The remaining parcels are generally developed with AC or PCC pavements with isolated landscaped planters. These pavements are also in poor condition with minor to severe cracking throughout.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth, and visual observations made at the time of the subsurface investigation, the overall site topography slopes downwards to the east at a gradient of less than 1 percent.

Proposed Development

The project site plan provided to our office on February 6, 2024 and enclosed as Plate 2 of this report indicates that the project site will be developed with multiple clusters of buildings. The westernmost region of the project site will be developed with two buildings that will share a common wall, identified as Stage Group A, and will be 60,765± ft^2 in size. The south-central region of the project site will be developed with two buildings which will share a common wall. This structure will be identified as Stage Group B and will be 52,980± ft^2 in size. The easternmost region of the project site will be developed with two buildings that will share a common wall, identified as Stage Group C. This structure will be 60,611± ft^2 in size. The north-central region of the project site will be developed with an eight-level multi-purpose structure identified as "Main Building" which includes six-levels of integrated automobile parking. The Main Building will be 189,671± ft^2 in size. It is assumed that the Main Building structure will be of reinforced concrete and steel-frame construction. The buildings will be surrounded by AC and/or PCC pavements and limited areas of concrete flatwork.

Based on the proposed drainage exhibit, prepared by KPFF, the project civil engineer, the infiltration system will consist of four (4) infiltration pipes that are 8 feet in diameter and 150 feet in length. One (1) of the infiltration pipes will be located in the southeastern area of the project site (identified as Infiltration System "A") and two (2) of the infiltration pipes will be located in the east-central area of the project site (identified as Infiltration System "B"). One (1) of the infiltration pipes will be located in the western area of the project site (identified as Infiltration System "C"). Although the grading plan is not available at this site, the bottoms of the infiltration pipes are expected to range between 10 and 20± feet below the currently existing site grades.

Previous Study

SCG previously conducted a geotechnical feasibility-level study at the project site (Reference No. 1). As a part of this study, three (3) borings (identified as Boring Nos. B-1 through B-3) were advanced to depths of 50 to 130± feet below the existing site grades. AC pavements were encountered at the ground surface at Boring Nos. B-1 and B-3. At these locations, the pavement sections consisted of 2 to 7± inches of AC with no underlying layer of aggregate base. Boring No. B-2 was drilled within the existing PCC pavements. At this location, the pavement section consisted of 8± inches of reinforced PCC, underlain by 6± inches of aggregate base (AB). Artificial fill soils were encountered beneath the existing pavements at all of the boring locations, extending to depths of 2 to 4½± feet below the existing site grades. The fill soils generally consisted of very loose to very dense silty fine sands with varying medium to coarse sand and fine to coarse gravel content. Native alluvium was encountered beneath the fill soils at all of the boring locations, extending to at least the maximum depth explored of 130± feet below the existing site grades. The near-surface alluvium generally consisted of very loose to medium dense sands and silty sands with varying fine gravel content, extending to depths of 12 to 22± feet. At greater depths, the alluvium consisted of dense to very dense silty sands and poorly- to well-graded sands with varying fine to coarse gravel content. Free water was not encountered during the drilling of any of the borings. The static groundwater table is considered to have existed at a depth in excess of 130± feet at the time of the subsurface exploration.

Geotechnical Study

SCG conducted a subsequent geotechnical investigation at the project site (Reference No. 2). As a part of this study, eight (8) borings were advanced to depths of 30 to 50± feet below the currently existing site grades.

AC pavements were encountered at the ground surface at Boring Nos. B-4, B-6, B-7, and B-9. At these locations the pavement sections consisted of 2 to 7± inches AC underlain by 0 to 6± inches of AB. Some of these borings were underlain by an additional 5 to 9± inches of PCC pavements. PCC pavements were encountered at the ground surface at Boring Nos. B-5, B-8, B-10, and B-11. At these locations the pavement sections consisted of 5 to 10± inches of PCC underlain by 0 to 9± inches of AB. Some of these borings were underlain by 7 to 8± inches of additional PCC pavements. Artificial fill soils were encountered beneath the existing pavements at all of the boring locations, extending to depths of 2½ to 8± feet below the existing site grades. The fill soils generally consist of loose to medium dense silty fine to medium sands, fine to medium sands, gravelly fine to coarse sands, and medium stiff clayey silts with variable amounts of gravel content and iron oxide staining. Native alluvium was encountered beneath the fill soils at all of the boring locations, extending to at least the maximum depth explored of 50± feet below the existing site grades. The near-surface alluvium generally consists of loose to very dense fine to coarse sands, silty fine to medium sands, gravelly fine to coarse sands, extending to depths of 12 to 27± feet. At greater depths, the alluvium generally consists of medium dense to very dense silty fine to coarse sands, fine to coarse sands, and gravelly fine to coarse sands. Several of the borings contained interbedded layers of clayey silt. Free water was not encountered during the drilling of any of the borings. The static groundwater table is considered to have existed at a depth in excess of 50± feet at the time of the subsurface exploration.

Subsurface Exploration

A total of four (4) infiltration test borings were drilled within the general area of the eastern proposed infiltration systems. Two (2) of the infiltration test borings were advanced to a depth of 10± feet and two (2) of the infiltration tests were advanced to a depth of 20± feet below existing site grades. Please note that the location of the infiltration systems were altered after the infiltration testing was performed. Therefore, none of the infiltration tests were located in the general area of the western infiltration system. The infiltration test borings were advanced using a truck-mounted drilling rig, equipped with 8-inch-diameter hollow-stem augers. All of the infiltration test borings were logged by a member of our staff. The approximate locations of the infiltration test borings (identified as Infiltration Test Nos. I-1a, I-1b, I-2a, and I-2b) are indicated on the Infiltration Test Location Plan, enclosed as Plate 2 of this report.

At the completion of drilling, a sufficient length of 3-inch-diameter slotted PVC casing was then placed into the test hole, so that the PVC casing extended from the bottom of the test hole to the ground surface. Clean ¾-inch gravel was then installed in the annulus surrounding the PVC casing.

Geotechnical Conditions

PCC pavements were encountered at the ground surface at Infiltration Test Nos. I-1a and I-1b. At these locations, the pavement sections consist of 6± inches of PCC underlain by 2± inches of AB. A second layer of PCC pavements 6± inches in thickness was encountered beneath the AB at these locations. AC pavements were encountered at the ground surface at Infiltration Test Nos. I-2a and I-2b. At these locations, the pavement sections consist of 7± inches of AC underlain by 5± inches of PCC. There pavements were underlain by AB which was 3± inches in thickness. Artificial fill soils were encountered beneath the pavements at all of the infiltration boring locations extending to depths of 3 to 5½± feet below the existing site grades. The fill soils generally consist of loose silty fine to medium sands and fine to medium sands. Native alluvial soils were encountered beneath fill soils at all of the infiltration boring locations, extending to at least the maximum depth explored of 20± feet below the existing site grades. The alluvium generally consists of loose to very dense fine to medium sands, silty fine to coarse sands, and gravelly fine to coarse sands. Free water was not encountered during drilling at the infiltration test locations below ground surface. Based on the previous feasibility study, the static groundwater table is considered to exist at depths in excess of 130± feet below existing site grades. The Boring Logs, which illustrate the conditions encountered at the boring locations, are included with this report.

As part of our research, we reviewed available groundwater data regarding the historic high groundwater level for the site at the time of the subsurface investigation. The primary reference used to determine the historic groundwater depths in this area is the California Geological Survey (CGS) Open File Report 98-20, the Seismic Hazard Zone Report for the Los Angeles 7.5-Minute Quadrangle, which indicates that the historic high groundwater level for the site is greater than 150 feet below the ground surface.

As part of our research, we reviewed readily available groundwater data in order to determine regional groundwater depths. The primary reference used to determine the groundwater depths in the project site area is the California State Water Resources Control Board, GeoTracker, website, https://geotracker.waterboards.ca.gov/. Several monitoring wells on record are located 2,300± north of the project site. Water level readings within these monitoring wells indicate a high groundwater level of 96½± feet below the ground surface, in June 2009. The identified wells provide geotechnically meaningful data regarding groundwater and depth, however, have been abandoned as part of environmental cleanup activities.

A report titled Report of Soil investigation Activities, Greyhound lines, Inc., 1614 East 7th Street, Los Angeles, CA 90021 (Strata Environmental Services Inc, 2016) documents the results of the environmental soil sampling at the project site, and this report was found on the GeoTracker website. The Strata report indicates that the Los Angeles Regional Water Quality Control Board (LARWQCB) has issued a directive letter indicating that the depth to groundwater at the site is 95 feet.

Infiltration Testing

The infiltration testing was performed in general accordance with Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration (GS200.1) published by Los Angeles County Public Works – Geotechnical Engineering and Materials Division, dated June 30, 2021.

Pre-soaking

All of the infiltration test borings were pre-soaked for at least 1 hour to ensure the sand around the annulus of the perforated pipe was fully saturated. The pre-soaking procedure consisted of filling each test boring with clean potable water to an elevation of at least 12± inches above the bottom of each test boring. In accordance with the Los Angeles County guidelines, since the water in all of the infiltration test borings did not completely infiltrate within a 30-minute time period after filling each boring, a falling head test was the appropriate test method.

Infiltration Testing Procedures

After the completion of the pre-soaking process, SCG performed the infiltration testing. A sufficient amount of water was added to the test borings so that the water level was approximately 12± inches higher than the bottom of the borings and less than or equal to the water level used during the pre-soaking process. Readings were taken at 15 to 30-minute intervals at all of the infiltration test locations. A stabilized rate of drop, where the highest and lowest readings from three consecutive readings are within 10 percent of each other, was obtained for each of the test borings. These water level readings are presented on the spreadsheets enclosed with this report. The infiltration rates for each of the timed intervals are also tabulated on the spreadsheets attached to this report.

Infiltration Test No.	Depth (feet)	Soil Description	Measured Infiltration Rate (inches/hour)
I-1a	10	Gravelly fine to coarse Sand, trace Silt	6.2
I-1b	20	Fine to medium Sand, little Silt, trace coarse Sand, trace fine Gravel	9.3
I-2a	20	Silty fine to medium Sand, trace Silt, trace coarse Sand	6.4
I-2b	10	Silty fine to coarse Sand, trace fine Gravel	6.6

Laboratory Testing

Moisture Content

The moisture contents for the recovered soil samples within the borings were 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.

Grain Size Analysis

The grain size distribution of selected soils collected throughout each infiltration test boring have been determined using a range of wire mesh screens. These tests were performed in general accordance with ASTM D-422 and/or ASTM D-1140. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these tests are presented on Plates C-1 through C-4 of this report. The percentage passing the No. 200 sieve is also reported on the Boring Logs.

Design Recommendations

Four (4) infiltration tests were performed at the project site. As noted above, the measured infiltration rate at the shallow infiltration test locations ranged from 6.2 to 9.3 inches per hour. The Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration, GS200.1 prepared by the County of Los Angeles, Department of Public Works, Geotechnical and Materials Division (GMED) on June 30, 2021 dictate that a reduction factor be utilized in the design infiltration rate. The following reduction factors are considered in the design infiltration rate:

Reduction Factor	
Small Diameter Boring	$RF_t = 1$
Site Variability, number of tests, and thoroughness of subsurface investigation	$RF_v = 1$
Long-term siltation plugging and maintenance	$RF_v = 1$
Total Reduction Factor, RF= $RF_t + RF_v + RF_v$	RF = 3
Design Infiltration Rate (DIR) = Measured Infiltration Rate (MIR)/RF	**DIR =** See Below

Based on the results of the infiltration testing, silt content, and reduction factors for the DIR, we recommend the following design infiltration rates:

Infiltration System	Infiltration Test Nos.	Depth (feet)	Design Infiltration Rate (inches per hour)
"A"	I-1a	10	2.1
	I-1b	20	3.1
"B"	I-2a	10	2.1
	I-2b	20	2.2
"C"	See paragraph below		

Please note that none of the infiltration tests were performed within Infiltration System "C", located in the western area of the project site. However, Boring No. B-6 from the current geotechnical investigation was located in the general area of Infiltration System "C". The subsurface conditions at Boring No. B-6 consist of dense to very dense silty fine to coarse sands from 3 to 17± feet and medium dense to dense fine to medium sands with trace fine gravel extending from 17 to at least 30± feet. The soil conditions encountered at Boring No. B-6 are similar to the soil conditions encountered at Infiltration Boring Nos. I-1a, I-1b, I-2a, and I-2b. SCG has been requested to provide a preliminary design infiltration rate for Infiltration System "C". Therefore, based on the similar soil conditions at the infiltration borings and Boring No. B-6, the lowest infiltration rate at the infiltration tests conducted within Infiltration Systems "A" and "B", and an increased reduction of 5 due to the uncertainty, SCG recommends a preliminary design rate of 1.2 inches per hour be used for Infiltration System "C" between 10 and 20± feet. SCG recommends that this infiltration rate be confirmed prior to or at the time of construction of the infiltration system. It should be understood that a redesign of Infiltration System "C" could be necessary if the preliminary design rate is less than the confirmation infiltration rate at the time of construction.

Additional Recommendations

We recommend that a representative from the geotechnical engineer be on-site during the construction of the proposed infiltration systems to identify the soil classification at the base of each system. It should be confirmed that the soils at the base of the proposed infiltration systems correspond with those presented in this report to ensure that the performance of the systems will be consistent with the rates reported herein.

The design of the proposed dry well infiltration systems should be performed by the project civil engineer, in accordance with the City of Los Angeles and/or Los Angeles County guidelines.

However, it is recommended that the system be constructed so as to facilitate removal of silt and clay, or other deleterious materials from any water that may enter the systems. The presence of such materials would decrease the effective infiltration rates. **The infiltration rates recommended above are based on the assumption that only clean water will be introduced to the subsurface profile. Any fines, debris, or organic materials could significantly impact the infiltration rates.** It should be noted that the recommended infiltration rates are based on infiltration testing at four (4) discrete locations and the overall infiltration rates of the infiltration systems could vary.

Infiltration Rate Considerations

The infiltration rates presented herein was determined in accordance with the Los Angeles County guidelines and are considered valid only for the time and place of the actual test. Varying subsurface conditions will exist in other areas of the site, which could alter the recommended infiltration rates presented above. The infiltration rates will decline over time between maintenance cycles as silt or clay particles accumulate on the BMP surface. The infiltration rate is highly dependent upon a number of factors, including density, silt and clay content, grainsize distribution throughout the range of particle sizes, and particle shape. Small changes in these factors can cause large changes in the infiltration rates.

Infiltration rates are based on unsaturated flow. As water is introduced into soils by infiltration, the soils become saturated and the wetting front advances from the unsaturated zone to the saturated zone. Once the soils become saturated, infiltration rates become zero, and water can only move through soils by hydraulic conductivity at a rate determined by pressure head and soil permeability. Changes in soil moisture content will affect the infiltration rate. Infiltration rates should be expected to decrease until the soils become saturated. Soil permeability values will then govern groundwater movement. Permeability values may be on the order of 10 to 20 times less than infiltration rates. The infiltration system designer should incorporate adequate factors of safety and allow for overflow design into appropriate traditional storm drain systems, which would transport storm water off-site.

Construction Considerations

The infiltration rates presented in this report are specific to the tested locations and tested depths. Infiltration rates can be significantly reduced if the soils are exposed to excessive disturbance or compaction during construction. Compaction of the soils at the bottom of the infiltration system can significantly reduce the infiltration ability of the basins. Therefore, the subgrade soils within proposed infiltration system areas should not be over-excavated, undercut or compacted in any significant manner. **It is recommended that a note to this effect be added to the project plans and/or specifications.**

We recommend that a representative from the geotechnical engineer be on-site during the construction of the proposed infiltration systems to identify the soil classification at the base of each system. It should be confirmed that the soils at the base of the proposed infiltration systems correspond with those presented in this report to ensure that the performance of the systems will be consistent with the rates reported herein.

We recommend that scrapers and other rubber-tired heavy equipment not be operated on the basin bottom, or at levels lower than 2 feet above the bottom of the system, particularly within basins. As such, the bottom 24 inches of the infiltration systems should be excavated with non-rubber-tired equipment, such as excavators.

Chamber Maintenance

The project may include below-grade chamber systems. Water flowing into these systems will carry some level of sediment. Wind-blown sediments will also contribute to sediment deposition at the bottom of the chamber. This layer has the potential to significantly reduce the infiltration rate of the basin subgrade soils. Therefore, a formal chamber maintenance program should be established to ensure that these silt and clay deposits are removed from the system on a regular basis.

Location of Infiltration Systems

The use of on-site storm water infiltration systems carries a risk of creating adverse geotechnical conditions. Increasing the moisture content of the soil can cause the soil to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Overlying structures and pavements in the infiltration area could potentially be damaged due to saturation of the subgrade soils. **The proposed infiltration systems for this project site should be located at least 25 feet away from any descending slopes or structures, including retaining walls.** Even with this provision of locating the infiltration system at least 25 feet from the building(s), it is possible that infiltrating water into the subsurface soils could have an adverse effect on the proposed or existing structures. It should also be noted that utility trenches which happen to collect storm water can also serve as conduits to transmit storm water toward the structure, depending on the slope of the utility trench. Therefore, consideration should also be given to the proposed locations of underground utilities which may pass near the proposed infiltration system.

The infiltration system designer should also give special consideration to the effect that the proposed infiltration systems may have on nearby subterranean structures, open excavations, or descending slopes. In particular, infiltration systems should not be located near the crest of descending slopes, particularly where the slopes are comprised of granular soils. Such systems will require specialized design and analysis to evaluate the potential for slope instability, piping failures and other phenomena that typically apply to earthen dam design. This type of analysis is beyond the scope of this infiltration test report, but these factors should be considered by the infiltration system designer when locating the infiltration systems.

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, structural engineer, and/or civil engineer. The design of the proposed dry well infiltration system is the responsibility of the civil engineer. The role of the geotechnical engineer is limited to determination of infiltration rate only. SCG assumes no responsibility for the design or performance of the proposed stormwater infiltration system. 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 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 testing 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.

Closure

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.

Michelle Krizek
Staff Geologist

PROFESSIONAL GEOLOGIST
DARYL R. KAS
No. 2467
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

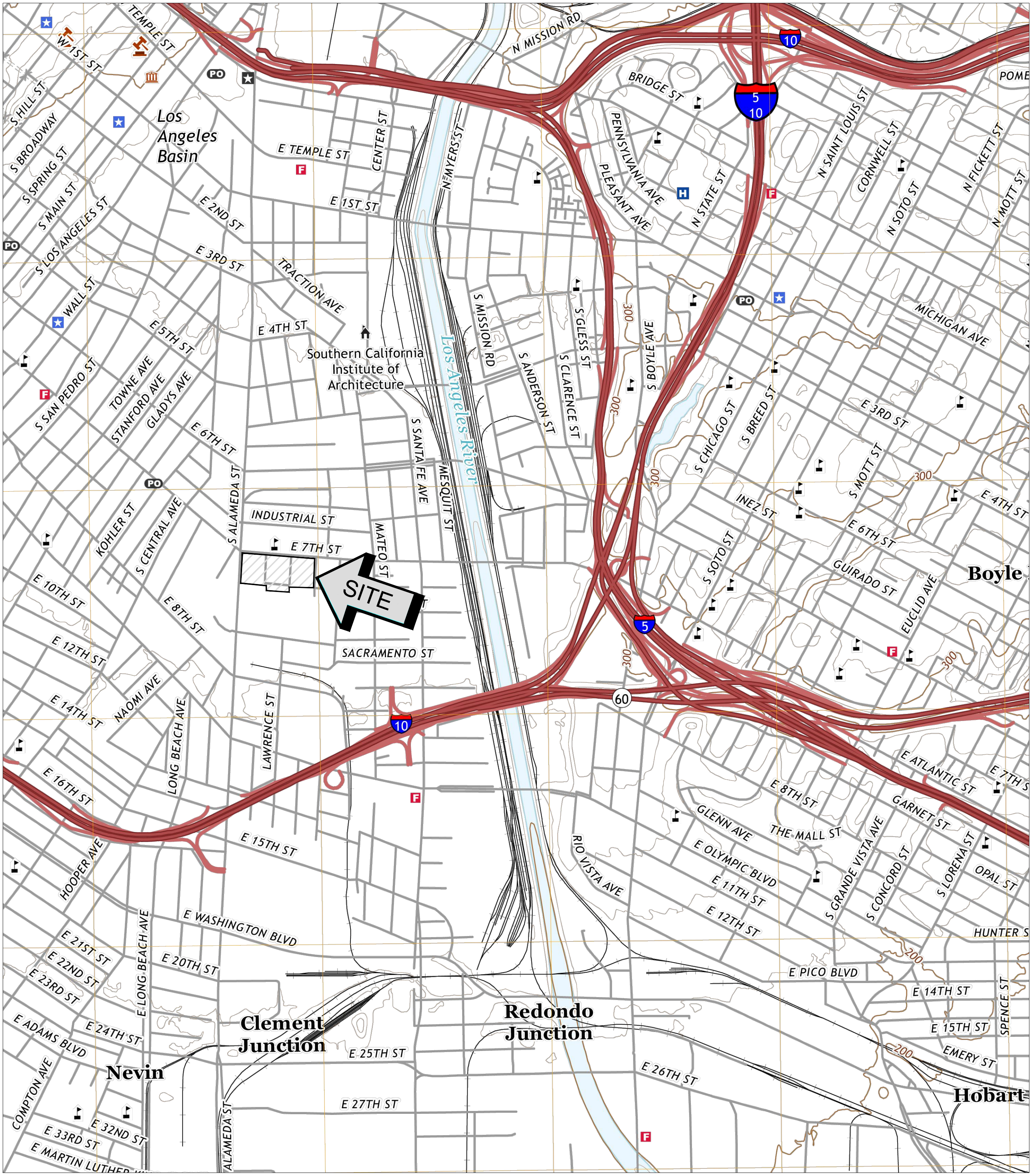
Daryl Kas, CEG 2467
Senior Geologist

Gregory K. Mitchell, GE 2364
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
GREGORY K. MITCHELL
No. 2364
GEOTECHNICAL
STATE OF CALIFORNIA

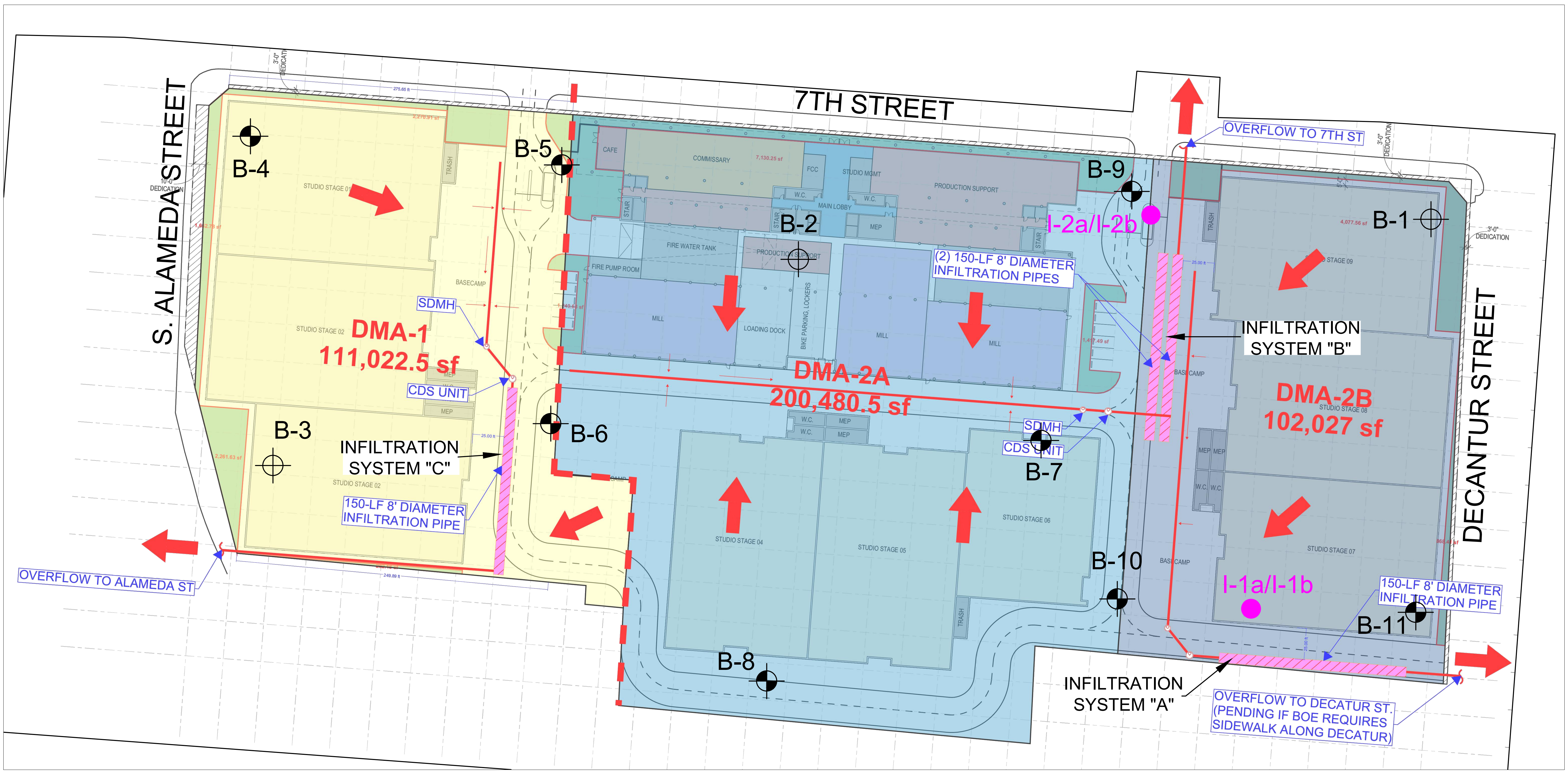
Distribution: (1) Addressee

Enclosures: Plate 1 – Site Location Map
Plate 2 – Infiltration Test Location Plan
Boring Log Legend and Logs (6 Pages)



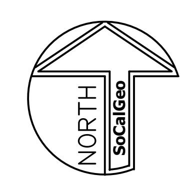
SOURCE: USGS TOPOGRAPHIC MAP OF THE LOS ANGELES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA, 2022.

SITE LOCATION MAP	
ALAMEDA CROSSING DEVELOPMENT	
LOS ANGELES, CALIFORNIA	
SCALE: 1" = 2000'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: MK CHKD: GKM	
SCG PROJECT 20G243-5R2	
PLATE 1	



GEOTECHNICAL LEGEND

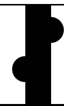
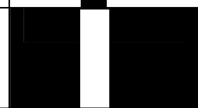
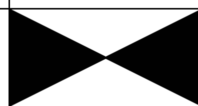
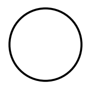
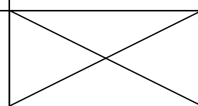
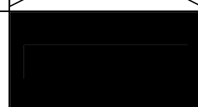
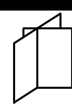
- APPROXIMATE INFILTRATION TEST LOCATION
- PROPOSED INFILTRATION SYSTEM
- APPROXIMATE BORING LOCATION (SCG PROJECT NO. 20G243-4R)
- PREVIOUS BORING LOCATION (SCG PROJECT NO. 20G243-1)



NOTE: PROPOSED DRAINAGE EXHIBIT PROVIDED BY KPFF

INFILTRATION TEST LOCATION PLAN	
ALAMEDA CROSSING DEVELOPMENT	
LOS ANGELES, CALIFORNIA	
SCALE: 1" = 80'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: MK CHKD: GKM	
SCG PROJECT 20G243-5R2	
PLATE 2	

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	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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. I-1a

JOB NO.: 20G243-5R2
PROJECT: Alameda Crossing Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/13/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: Not Applicable
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					<u>PORTLAND CEMENT CONCRETE:</u> 6± inches Portland Cement Concrete underlain by 2± inches Aggregate Base, underlain by 6± inches Portland Cement Concrete							Hand Auger cleared to 5 feet
					<u>FILL:</u>Brown fine to medium Sand, trace Silt, trace to little coarse Sand, trace fine Gravel, medium dense-dry to damp							
					<u>ALLUVIUM:</u>Brown fine to medium Sand, trace Silt, trace to little coarse Sand, trace fine Gravel, medium dense-dry to damp							
5	X	14					3					
	X	29			Brown Gravelly fine to coarse Sand, trace Silt, medium dense-dry to damp		3			3		@ 8.5 feet, Poor Sample Recovery
10												

Boring Terminated at 10'

TBL 20G243-5.GPJ SOCALGEO.GDT 3/20/24

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. I-1b

JOB NO.: 20G243-5R2
PROJECT: Alameda Crossing Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/13/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: Not Applicable
READING TAKEN: At Completion

SURFACE ELEVATION: 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
					PORTLAND CEMENT CONCRETE: 6± inches Portland Cement Concrete underlain by 2± inches Aggregate Base, underlain by 6± inches Portland Cement Concrete							Hand Auger cleared to 5 feet
					FILL: Light Brown fine to medium Sand, little coarse Sand, trace fine to coarse Gravel, trace Silt, medium dense-damp to moist							
					ALLUVIUM: Light Brown fine to medium Sand, little coarse Sand, trace fine to coarse Gravel, trace Silt, medium dense-damp to moist							
5	X	12					4					
	X	7			@ 8.5 feet, trace to little Silt, trace to little fine to coarse Gravel, loose		11					
10												
					Gray Brown Gravelly fine to coarse Sand, trace Silt, occasional Cobble, dense-damp							
	X	36					3					
15												
					Gray Brown fine to medium Sand, little Silt, trace coarse Sand, trace fine Gravel, medium dense-damp							
	X	18					5			9		
20					Boring Terminated at 20 feet							

TBL 20G243-5.GPJ SOCALGEO.GDT 3/20/24

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. I-2a

JOB NO.: 20G243-5R2
PROJECT: Alameda Crossing Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/10/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: Not Applicable
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ASPHALTIC CONCRETE: 7± inches Asphaltic Concrete underlain by 5± inches Portlant Cement Concrete, underlain by 3± inches Aggregate Base							@ 3.5 feet Poor Sample Recovery
					FILL: Gray Brown Silty fine to medium Sand, trace Clay, trace coarse Sand, trace to little fine to coarse Gravel, loose-damp							
	X	8					9					
5					ALLUVIUM: Dark Gray Brown Silty fine Sand to fine Sandy Silt with trace Clay interbedded with Light Brown fine to medium Sandy layers, trace fine Gravel, medium dense-very moist							
	X	13					20					
	X	12			Gray Brown fine to medium Sand, trace Silt, trace coarse Sand, medium dense-dry		1			6		
10												

Boring Terminated at 10 feet

TBL 20G243-5.GPJ SOCALGEO.GDT 3/20/24

BORING NO. I-2b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 20G243-5R2
PROJECT: Alameda Crossing Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/10/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: Not Applicable
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ASPHALTIC CONCRETE: 7± inches Asphaltic Concrete underlain by 5± inches Portland Cement Concrete, underlain by 3± inches Aggregate Base							
	X	24			FILL: Gray Brown fine to medium Sand, trace Silt, trace to little coarse Sand, trace fine Gravel, medium dense-damp		4					
	X	26			ALLUVIUM: Gray Brown fine to medium Sand, trace Silt, trace to little coarse Sand, trace fine Gravel, medium dense to dense-damp		12					
5												
	X	39					7					
	X	39			@ 8.5 feet, trace to little fine to coarse Gravel, dry to damp		8					
10												
	X	50/2"			@ 13.5 trace to little Silt, very dense		3					
15												
					Gray Brown Silty fine to coarse Sand, trace fine Gravel, medium dense-damp							
	X	24					3					@ 18.5 feet Poor Sample Recovery
20												

Boring Terminated at 20 feet

TBL 20G243-5.GPJ SOCALGEO.GDT 3/20/24

INFILTRATION CALCULATIONS

Project Name	Alameda Crossing Development
Project Location	Los Angeles, California
Project Number	20G243-5R2
Engineer	Michelle Krizek

Test Hole Radius	4.00	(in)
Test Depth	10.15	(ft)

Infiltration Test Hole: I-1a

Start Time for Pre-Soak	8:43 AM	Water Remaining in Boring (Y/N)	Y
Start Time for Standard	9:49 AM	Time Interal Between Readings	30min

Interval Number		Time	Time Interval (min)	Water Depth (ft)	Change in Water Level (ft)	Average Head Height (ft)	Measured Infiltration Rate Q (in/hr)	Reduction Factor (RF)	Design Infiltration Rate Q (in/hr)
1	Initial	9:49 AM	30.0	6.50	3.40	2.0	6.4	3.0	2.1
	Final	10:19 AM		9.90					
2	Initial	10:22 AM	30.0	6.50	3.34	2.0	6.2	3.0	2.1
	Final	10:52 AM		9.84					
3	Initial	10:55 AM	30.0	6.50	3.35	2.0	6.3	3.0	2.1
	Final	11:25 AM		9.85					
4	Initial	11:28 AM	30.0	6.50	3.35	2.0	6.3	3.0	2.1
	Final	11:58 AM		9.85					
5	Initial	12:04 PM	30.0	6.50	3.33	2.0	6.2	3.0	2.1
	Final	12:34 PM		9.83					
6	Initial	12:36 PM	30.0	6.50	3.33	2.0	6.2	3.0	2.1
	Final	1:06 PM		9.83					

Design Infiltration Rate = (Measured Infiltration Rate)/(Reduction Factor)
Reduction Factor (RF) = $RF_t+RF_v+RF_s$

Reduction Factors	
Double-ring Infiltrometer	RF_t = 1 to 3
Shallow Test Pit	
Small Diameter Boring	
Large Diameter Boring	
High Fow-rate	RF_t = 3
Grain Size Analysis Method	RF_t = 2 to 3
Site variability, number of tests and thoroughness of subsurface investigation	RF_v = 1 to 3
Long-term siltation, plugging, and maintenance	RF_s = 1 to 3

$$Q = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Where: Q = Measured Infiltration Rate (in inches per hour)
ΔH = Change in Height (Water Level) over the time interval
r = Test Hole (Borehole) Radius
Δt = Time Interval
H_{avg} = Average Head Height over the time interval

INFILTRATION CALCULATIONS

Project Name	Alameda Crossing Development
Project Location	Los Angeles, California
Project Number	20G243-5R2
Engineer	Michelle Krizek

Test Hole Radius	4.00	(in)
Test Depth	20.10	(ft)

Infiltration Test Hole	I-1b

Start Time for Pre-Soak	9:23 AM	Water Remaining in Boring (Y/N)	Y
Start Time for Standard	10:23 AM	Time Interal Between Readings	30min

Interval Number		Time	Time Interval (min)	Water Depth (ft)	Change in Water Level (ft)	Average Head Height (ft)	Measured Infiltration Rate Q (in/hr)	Reduction Factor (RF)	Design Infiltration Rate Q (in/hr)
1	Initial	10:23 AM	15.0	5.10	11.44	9.3	9.7	3.0	3.2
	Final	10:38 AM		16.54					
2	Initial	10:45 AM	15.0	5.10	11.30	9.4	9.5	3.0	3.2
	Final	11:00 AM		16.40					
3	Initial	11:08 AM	15.0	5.10	11.25	9.4	9.4	3.0	3.1
	Final	11:23 AM		16.35					
4	Initial	11:26 AM	15.0	5.10	11.15	9.4	9.3	3.0	3.1
	Final	11:41 AM		16.25					
5	Initial	11:46 AM	15.0	5.10	11.14	9.4	9.3	3.0	3.1
	Final	12:01 PM		16.24					
6	Initial	12:41 PM	15.0	5.10	11.14	9.4	9.3	3.0	3.1
	Final	12:56 PM		16.24					

Design Infiltration Rate = (Measured Infiltration Rate)/(Reduction Factor)
Reduction Factor (RF) = $RF_t + RF_v + RF_s$

Reduction Factors	
Double-ring Infiltrometer	RF_t = 1 to 3
Shallow Test Pit	
Small Diameter Boring	
Large Diameter Boring	
High Fow-rate	RF_t = 3
Grain Size Analysis Method	RF_t = 2 to 3
Site variability, number of tests and thoroughness of subsurface investigation	RF_v = 1 to 3
Long-term siltation, plugging, and maintenance	RF_s = 1 to 3

$$Q = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Where:
Q = Measured Infiltration Rate (in inches per hour)
ΔH = Change in Height (Water Level) over the time interval
r = Test Hole (Borehole) Radius
Δt = Time Interval
H_{avg} = Average Head Height over the time interval

INFILTRATION CALCULATIONS

Project Name	Alameda Crossing Development
Project Location	Los Angeles, California
Project Number	20G243-5R2
Engineer	Michelle Krizek

Test Hole Radius	4.00	(in)
Test Depth	10.08	(ft)

Infiltration Test Hole: I-2a

Start Time for Pre-Soak	1:15 PM	Water Remaining in Boring (Y/N)	Y
Start Time for Standard	2:22 PM	Time Interal Between Readings	30min

Interval Number		Time	Time Interval (min)	Water Depth (ft)	Change in Water Level (ft)	Average Head Height (ft)	Measured Infiltration Rate Q (in/hr)	Reduction Factor (RF)	Design Infiltration Rate Q (in/hr)
1	Initial	2:22 PM	30.0	6.70	3.25	1.8	6.8	3.0	2.3
	Final	2:52 PM		9.95					
2	Initial	2:55 PM	30.0	6.70	3.23	1.8	6.7	3.0	2.2
	Final	3:25 PM		9.93					
3	Initial	3:28 PM	30.0	6.70	3.19	1.8	6.5	3.0	2.2
	Final	3:58 PM		9.89					
4	Initial	4:00 PM	30.0	6.70	3.16	1.8	6.4	3.0	2.1
	Final	4:30 PM		9.86					
5	Initial	4:33 PM	30.0	6.70	3.15	1.8	6.4	3.0	2.1
	Final	5:03 PM		9.85					
6	Initial	5:05 PM	30.0	6.70	3.15	1.8	6.4	3.0	2.1
	Final	5:35 PM		9.85					

Design Infiltration Rate = (Measured Infiltration Rate)/(Reduction Factor)
Reduction Factor (RF) = $RF_t+RF_v+RF_s$

Reduction Factors	
Double-ring Infiltrometer	RF_t = 1 to 3
Shallow Test Pit	
Small Diameter Boring	
Large Diameter Boring	
High Fow-rate	RF_t = 3
Grain Size Analysis Method	RF_t = 2 to 3
Site variability, number of tests and thoroughness of subsurface investigation	RF_v = 1 to 3
Long-term siltation, plugging, and maintenance	RF_s = 1 to 3

$$Q = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Where:
- Q = Measured Infiltration Rate (in inches per hour)
- ΔH = Change in Height (Water Level) over the time interval
- r = Test Hole (Borehole) Radius
- Δt = Time Interval
- H_{avg} = Average Head Height over the time interval

INFILTRATION CALCULATIONS

Project Name	Alameda Crossing Development
Project Location	Los Angeles, California
Project Number	20G243-5R2
Engineer	Michelle Krizek

Test Hole Radius	4.00	(in)
Test Depth	20.10	(ft)

Infiltration Test Hole: I-2b

Start Time for Pre-Soak	7:48 AM	Water Remaining in Boring (Y/N)	Y
Start Time for Standard	9:18 AM	Time Interal Between Readings	30min

Interval Number		Time	Time Interval (min)	Water Depth (ft)	Change in Water Level (ft)	Average Head Height (ft)	Measured Infiltration Rate Q (in/hr)	Reduction Factor (RF)	Design Infiltration Rate Q (in/hr)
1	Initial	9:18 AM	30.0	14.00	5.79	3.2	6.9	3.0	2.3
	Final	9:48 AM		19.79					
2	Initial	10:04 AM	30.0	14.00	5.75	3.2	6.8	3.0	2.3
	Final	10:34 AM		19.75					
3	Initial	10:37 AM	30.0	14.00	5.74	3.2	6.8	3.0	2.3
	Final	11:07 AM		19.74					
4	Initial	11:11 AM	30.0	14.00	5.72	3.2	6.7	3.0	2.2
	Final	11:41 AM		19.72					
5	Initial	11:45 AM	30.0	14.00	5.68	3.3	6.6	3.0	2.2
	Final	12:15 PM		19.68					
6	Initial	12:19 PM	30.0	14.00	5.67	3.3	6.6	3.0	2.2
	Final	12:49 PM		19.67					

Design Infiltration Rate = (Measured Infiltration Rate)/(Reduction Factor)

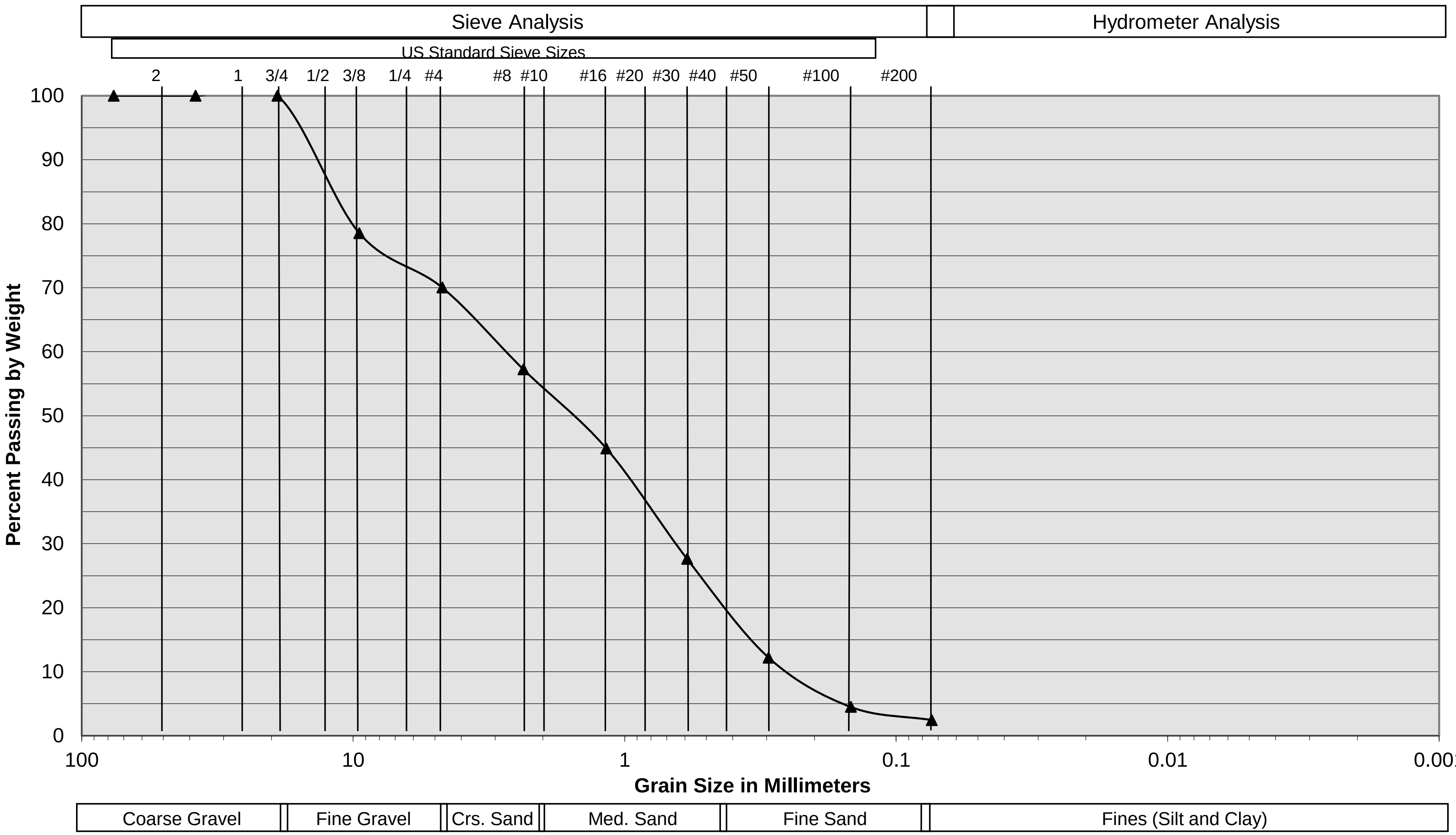
Reduction Factor (RF) = $RF_t + RF_v + RF_s$

Reduction Factors	
Double-ring Infiltrometer	RF_t = 1 to 3
Shallow Test Pit	
Small Diameter Boring	
Large Diameter Boring	
High Fow-rate	RF_t = 3
Grain Size Analysis Method	RF_t = 2 to 3
Site variability, number of tests and thoroughness of subsurface investigation	RF_v = 1 to 3
Long-term siltation, plugging, and maintenance	RF_s = 1 to 3

$$\boxed{Q = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}}$$

Where:
- Q = Measured Infiltration Rate (in inches per hour)
- ΔH = Change in Height (Water Level) over the time interval
- r = Test Hole (Borehole) Radius
- Δt = Time Interval
- H_{avg} = Average Head Height over the time interval

Grain Size Distribution



Sample Description	I-1a @ 8½ feet
Soil Classification	Brown Gravelly fine to coarse Sand, trace Silt

Alameda Crossing Development
Los Angeles, California
Project No. 20G243-5R2
PLATE C- 1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

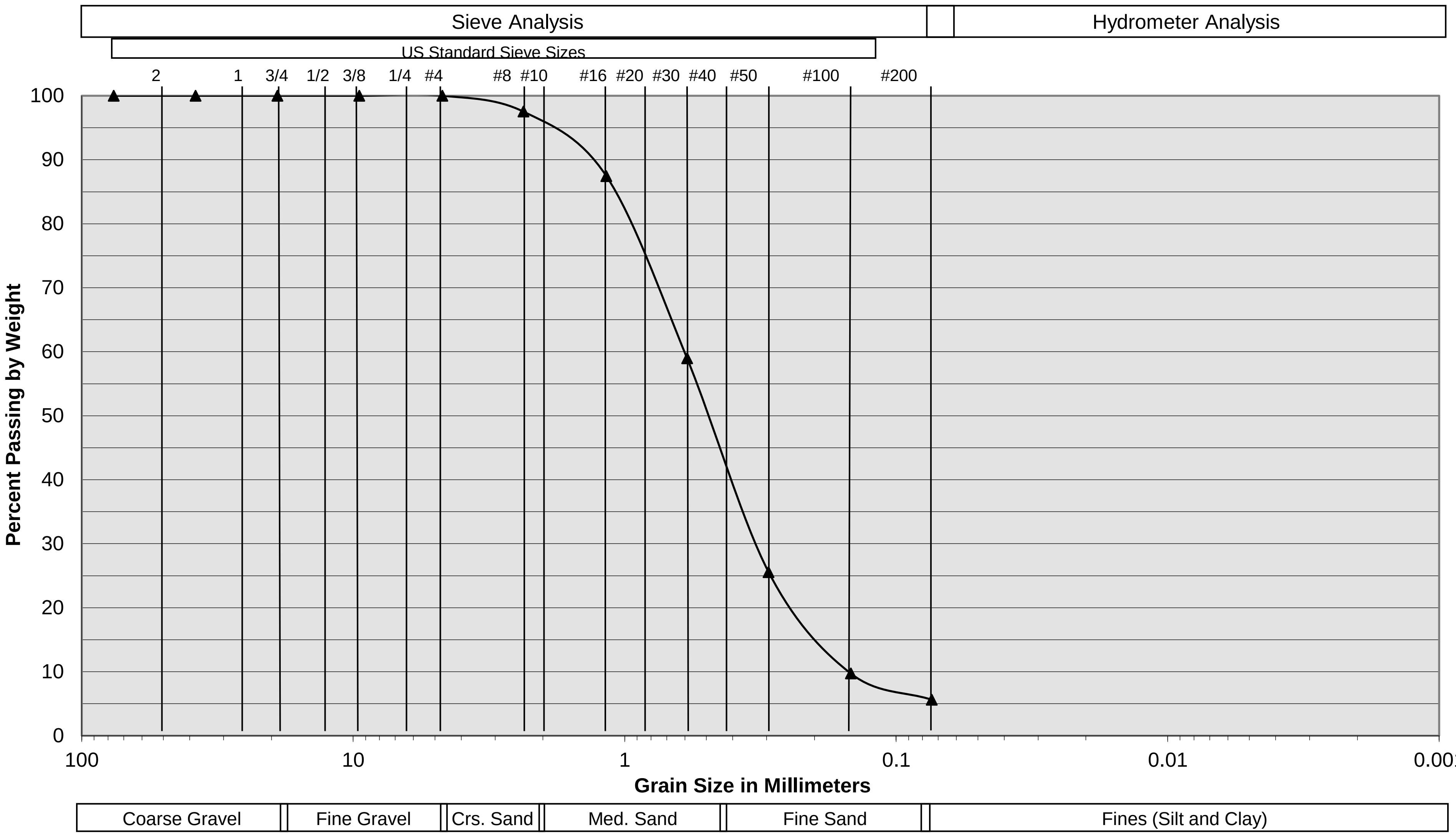
Grain Size Distribution

Sample Description	I-1b @ 18½ feet
Soil Classification	Brown fine to medium Sand, little Silt, trace coarse Sand, trace fine Gravel

Alameda Crossing Development
Los Angeles, California
Project No. 20G243-5R2
PLATE C- 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Grain Size Distribution

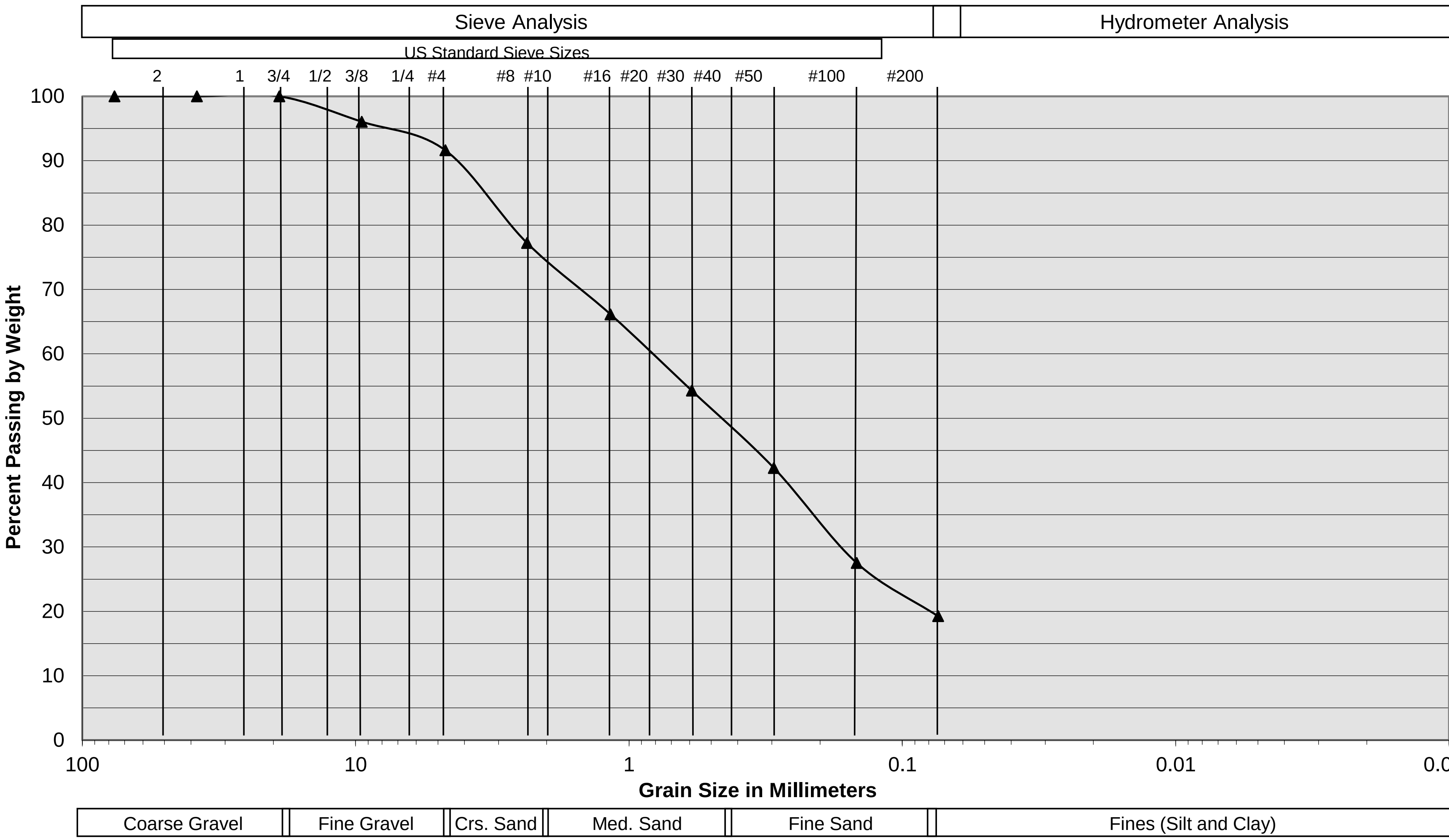


Sample Description	I-2a @ 8½ feet
Soil Classification	Light Brown fine to medium sand, trace Silt, trace coarse Sand

Alameda Crossing Development
Los Angeles, California
Project No. 20G243-5R2
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Grain Size Distribution



Sample Description	I-2b @ 18½ feet
Soil Classification	Gray Brown Silty fine to coarse Sand, trace fine Gravel

Alameda Crossing Development
Los Angeles, California
Project No. 20G243-5R2
PLATE C- 4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

October 17, 2024

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Prologis
2141 Rosecrans Avenue, Suite 1151
El Segundo, California 90245

Attention: Mr. Jonathan Payne
VP, Development Officer

Project No.: **20G243-6**

Subject: **Response to City of Los Angeles Review Letter**
Alameda Crossing Development
1716 East 7th Street
Los Angeles, California

References: 1) Preliminary Geotechnical Investigation, Alameda Crossing Development, 1716 East 7th Street, Los Angeles, California, prepared for Prologis by Southern California Geotechnical, Inc. (SCG), SCG Project No. 20G243-4R, dated May 7, 2024.

2) Results of Infiltration Testing, Alameda Crossing Development, 1716 East 7th Street, Los Angeles, California, prepared for Prologis by SCG, SCG Project No. 20G243-5R2, dated May 7, 2024.

3) Soils Report Review Letter, prepared by the City of Los Angeles, dated June 14, 2024, Log # 130835.

Mr. Payne:

In accordance with your request, we have prepared this letter to respond to comments issued by the City of Los Angeles, Department of Building and Safety following their review of the above referenced reports. The review comments prepared by LADBS are reproduced below, followed by our responses. A copy of the review letter is included with this correspondence for reference purposes.

Response to Review Letter

Each of the City of Los Angeles review comments is presented below followed by our response. A copy of the correction letter is enclosed with this correspondence for reference purposes.

LADBS1: *Provide geologic cross sections illustrating existing and proposed grades and structures.*

SCG1: Geologic cross sections have been provided as an attachment to this report as well as contained within the updated geotechnical report.

LADBS2: *The Geotechnical Engineer of Record shall provide geotechnical design parameters to support the geopier (or Rammed Aggregate Pier, RAM) design. As a minimum,* *the report shall include: the allowable bearing pressure, depth to groundwater, unit weight of soil, soil friction angle, elastic modulus, compression/re-compression indexes, OCR.*

SCG2: The referenced geotechnical report has been updated to only include the Rammed Aggregate Columns (RACs) recommendations for remediation within the main building. The kind of RACs is also known as Geopiers. The referenced geotechnical report includes all the required parameters for the ground improvement design, which consist of RACs. The following sections and plates include the required information:

- Section 4.2 includes the depth to groundwater.
- Section 6.5 includes the allowable bearing pressure for foundations founded on the RAC system.
- Section 6.8 includes the unit weight of the soil and soil friction angle.
- Plates C-1 through C-17 include information regarding the elastic modulus, compression/recompression indexes, and OCR.

The updated report will be submitted along with this response report.

LADBS3: *Clarify if the proposed pile foundation are to be embedded in new certified fill or native soils.*

SCG3: At the time of the referenced report, a foundation design had not been completed. Since we issued the referenced report, the client has hired on a ground improvement consultant who will be installing RAC. This system is also known as Geopiers. The updated report has had the pile/deep foundation option removed. Pile/deep foundations will not be implemented at the site.

LADBS4: *Provide geopier design performed by a designer certified by the Geopier Foundation Company. The Geotechnical Engineer of Record (GEOR) shall state that they have reviewed, and concurs with the design. The geopier design shall include, as a minimum, the following:*

a. *Engineering design of the RAP system, including: bearing capacity and settlement analysis without and with RAP, design calculations, range of pier diameters and depths, replacement ratio and acceptable aggregate types and size specifications.*

b. *Requirements for an indicator RAP "Modulus Test" (MT) program, and other field-testing methods and procedures.*

c. *The location of the proposed MT, test pier dimensions, acceptable methods of installation and approval criteria.*

d. *Geopier setbacks from adjacent property lines.*

SCG4: Geopier plans and calculations have been prepared by Western Ground Improvements, Inc. (WGI) and reviewed by SCG. A ground improvement plan review letter will be submitted along with this response report. The ground improvement plans and calculations were found to include all of the above requirements.

LADBS5: *Provide retaining wall/basement design calculations and recommendations for lateral earth pressure due to earthquake motion for walls higher than 6 feet, as required by section 1803.5.12 of the 2023 Los Angeles Building Code.*

SCG5: It is our understanding that no retaining wall/basements over 6 feet will be utilized at the site based on a review of the project plans. Therefore, seismic lateral earth pressures are not required and have not been provided.

LADBS6: *For the proposed on-site infiltration system, provide an evaluation on the following items (please refer to Information Bulletin P/BC 2020-118, which can be downloaded from our web site www.ladbs.org):*

a. *Potential on creating perched groundwater conditions.*

b. *Presence of potential expansive soils and/or susceptibility for hydroconsolidation.*

c. *Influence of the proposed infiltration system on adjacent proposed/existing foundations and retaining walls.*

SCG6: We have prepared a response for each comment provided by LADBS below:

a. Based on boring logs included in both the geotechnical and infiltration reports, the soil profile of the project site consists mainly of sands. When fine grained materials were encountered, they were either encountered at great depths (greater than 50 feet from existing site grades) or classified as fill. Proposed infiltration systems should not be installed within fill materials. It is for these reasons that potential perching of storm water infiltration is considered low (see page 16 of the geotechnical report).

b. Based on the consolidation/collapse test results presented within the submitted geotechnical report (see page 10 and Plates C-1 through C-17 in Appendix C of the geotechnical report), the potential for hydroconsolidation is considered low.

c. The submitted infiltration report (see "Location of Infiltration Systems" on page 9) states that, "The proposed infiltration systems for this project site should be located at least 25 feet away from any descending slopes or structures, including retaining walls." This statement was included to protect the proposed improvements and explains the influence of the proposed infiltration system on adjacent proposed/existing foundations and retaining walls. Based on the test results, a hydroconsolidation value of approximately 1 percent can be expected.

Closure

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.

Ricardo Frias, RCE 91722
Project Engineer

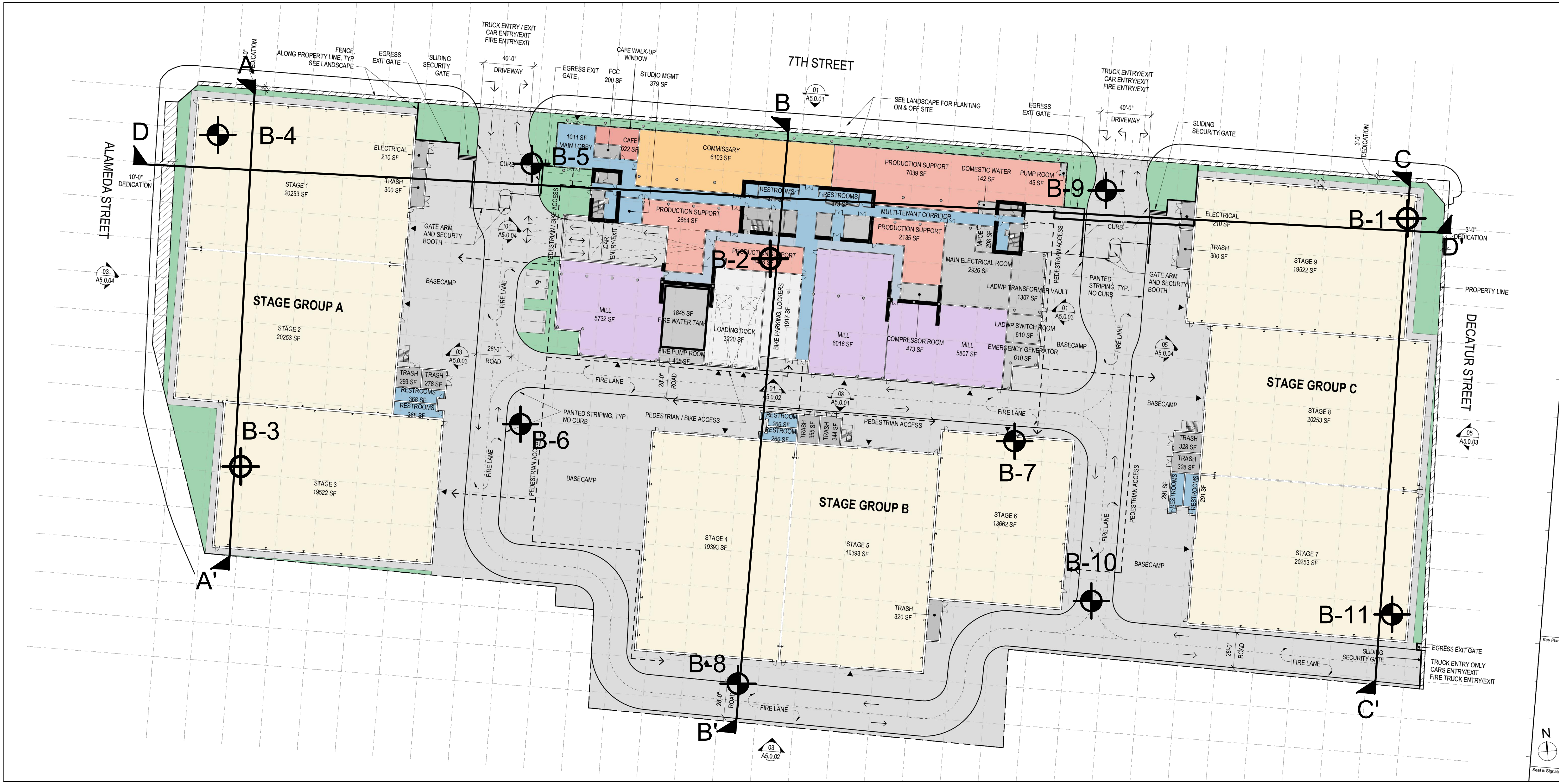
Gregory K. Mitchell, GE 2364
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER GREGORY K. MITCHELL No. 2364 GEOTECHNICAL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER RICARDO FRIAS No. 91772 CIVIL STATE OF CALIFORNIA

Enclosures: Plate 2: Boring and Cross Section Location Plan
Plate 5: Geotechnical Cross Sections
City of Los Angeles Soils Report Review Letter, Log #130835

Distribution: (1) Addressee



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION (SCG PROJECT NO. 20G243-4)

APPROXIMATE BORING LOCATION (SCG PROJECT NO. 20G243-1)

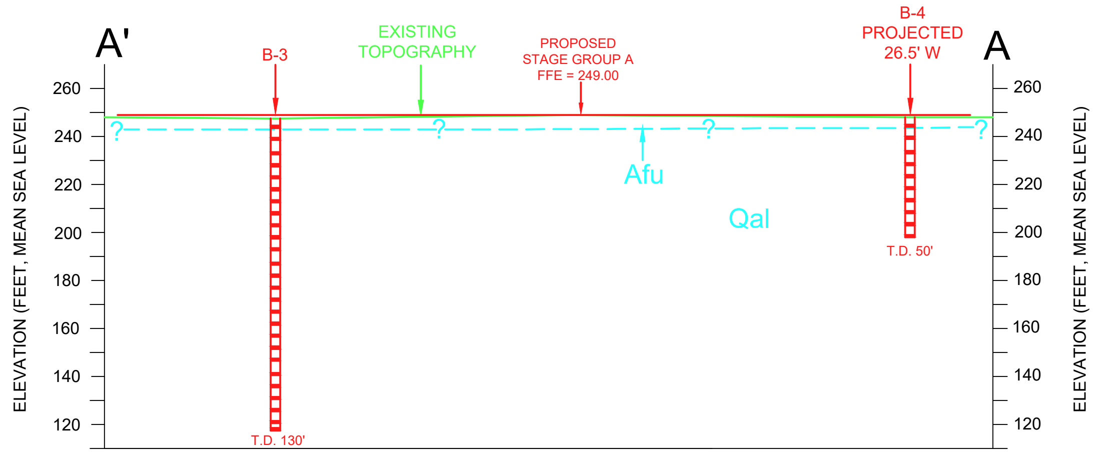
A — A' APPROXIMATE CROSS SECTION LOCATION

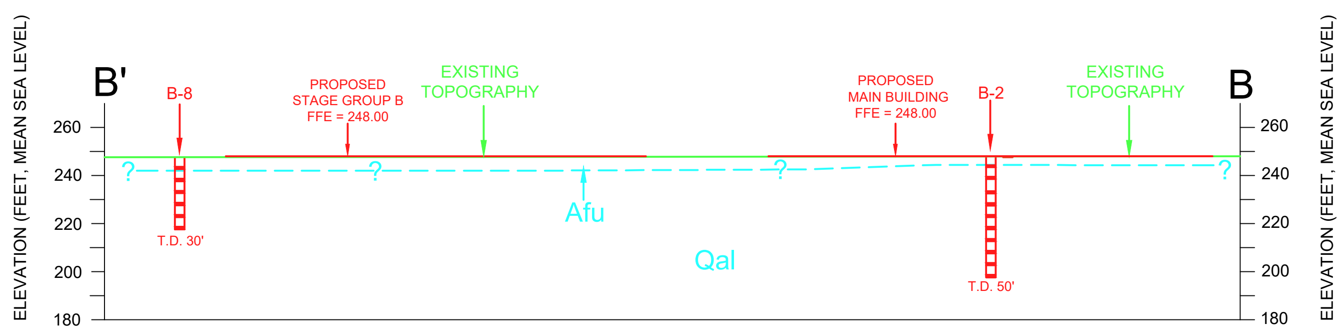
NORTH SoCalGeo

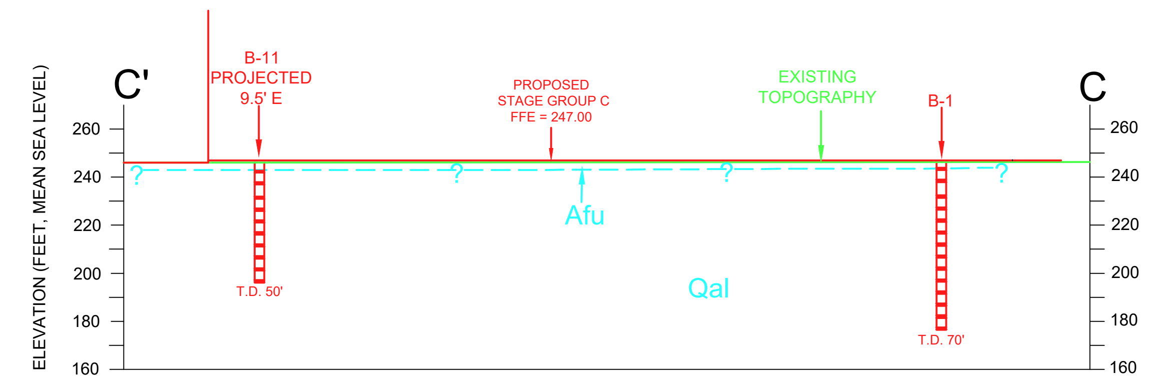
NOTE: CONCEPTUAL SITE PLAN PROVIDED BY RELATIVITY ARCHITECTS.

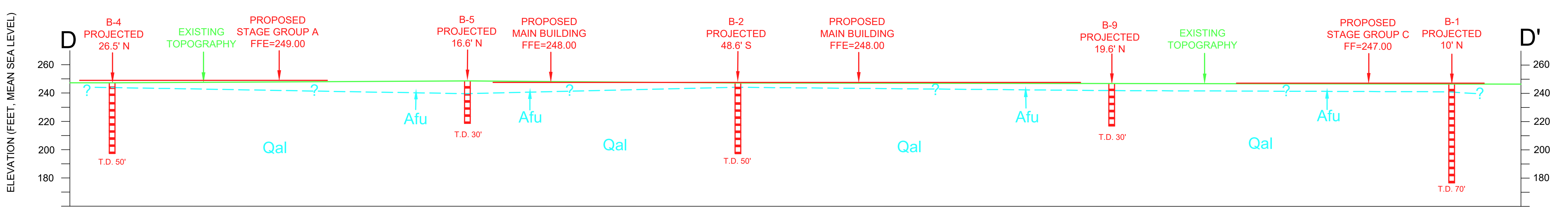
BORING AND CROSS SECTION LOCATION PLAN
ALAMEDA CROSSING DEVELOPMENT
LOS ANGELES, CALIFORNIA

SCALE: 1" = 80'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: DE CHKD: RF	
SCG PROJECT 20G243-4R2	
PLATE 2	









GEOLOGICAL UNITS

Afu - UNDOCUMENTED ARTIFICIAL FILL

Qal - ALLUVIUM

GEOTECHNICAL LEGEND

CONTACT QUERIED WHERE UNCERTAIN

PROPOSED BUILDING ELEVATION

EXISTING ELEVATION

GEOTECHNICAL CROSS SECTIONS

ALAMEDA CROSSING DEVELOPMENT

LOS ANGELES, CALIFORNIA

SCALE: 1" = 80'

DRAWN: DE
CHKD: RF

SCG PROJECT
20G243-4R2

PLATE 5

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

BOARD OF
BUILDING AND SAFETY
COMMISSIONERS

JAVIER NUNEZ
PRESIDENT

JACOB STEVENS
VICE PRESIDENT

CORISSA HERNANDEZ
MOISES ROSALES
NANCY YAP

CITY OF LOS ANGELES

CALIFORNIA

KAREN BASS
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

JOHN WEIGHT
EXECUTIVE OFFICER

SOILS REPORT REVIEW LETTER

June 14, 2024

LOG # 130835
SOILS/GEOLOGY FILE - 2

ProLogis
2141 Rosecrans Ave. #1151
El Segundo, CA 90245

TRACT:	E. B. MILLAR TRACT
BLOCK:	C
LOT(S):	3
LOCATION:	1716 E. 7th St.

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Soils Report	20G243-4R	05/07/2024	SoCalGeo
Addendum Report	20G243-5R2	05/07/02024	SoCalGeo

PREVIOUS REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Dept. Approval Letter	123370-01	01/18/2023	LADBS
Soils Report	20G243-3	12/14/2022	SoCalGeo
Dept. Review Letter	123370	10/20/2022	LADBS
Soils Report	20G243-2	09/22/2022	SoCalGeo

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provide recommendations for the proposed main building: 8 level mixed use structure (6 levels of parking), and three, one-story structures (i.e., Stage Groups A, B, and C). The earth materials at the subsurface exploration locations consist of up to 8 feet of uncertified fill underlain by native soils.The consultants recommend to support the proposed structure(s) on conventional, mat-type, drilled-pile and/or rammed aggregate piers foundations bearing on properly placed fill.

The Department previously conditionally approved the above referenced reports for the proposed industrial building and studio(for EIR and CEQA study purposes only) in a letter dated 01/18/2023, Log #123370-01.

As of January 1, 2023, the City of Los Angeles has adopted the new 2023 Los Angeles Building Code (LABC). The 2023 LABC requirements will apply to all projects where the permit application submittal date is after January 1, 2023.

AN EQUAL EMPLOYMENT OPPORTUNITY - AFFIRMATIVE ACTION EMPLOYER

The review of the subject report(s) cannot be completed at this time and will be continued upon submittal of an addendum to the report which shall include, but not be limited to, the following:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2023 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. Provide geological cross sections illustrating existing and proposed grades and structures.

2. The Geotechnical Engineer of Record shall provide geotechnical design parameters to support the geopier (or Ram Aggregate Pier, RAM) design. As a minimum, the report shall include: the allowable bearing pressure, depth to groundwater, unit weight of soil, soil friction angle, elastic modulus, compression/re-compression indexes, OCR.

3. Clarify if the proposed pile foundation are to be embedded in new certified fill or native soils.

4. Provide geopier design performed by a designer certified by the Geopier Foundation Company. The Geotechnical Engineer of Record (GEOR) shall state that they have reviewed, and concurs with the design. The geopier design shall include, as a minimum, the following:

    a. Engineering design of the RAP system, including: bearing capacity and settlement analysis without and with RAP, design calculations, range of pier diameters and depths, replacement ratio and acceptable aggregate types and size specifications.

    b. Requirements for an indicator RAP "Modulus Tests" (MT) program, and other field-testing methods and procedures.

    c. The location of the proposed MT, test pier dimensions, acceptable methods of installation and approval criteria.

    d. Geopier setbacks from adjacent property lines.

5. Provide retaining wall/basement design calculations and recommendations for lateral earth pressure due to earthquake motions for walls higher than 6 feet, as required by section 1803.5.12 of the 2023 Los Angeles Building Code.

6. For the proposed on-site infiltration system, provide an evaluation on the following items (please refer to Information Bulletin P/BC 2020-118, which can be downloaded from our web site www.ladbs.org):

    a. Potential on creating perched ground water conditions.

    b. Presence of potential expansive soils and/or susceptibility for hydroconsolidation.

    c. Influence of the proposed infiltration system on adjacent proposed/existing foundations and retaining walls.

The soils engineer shall prepare a report containing an itemized response to the review items indicated in this letter. If clarification concerning the review letter is necessary, the report review engineer may be contacted. Two copies of the response report, including one unbound wet-signed original for archiving purposes, a pdf-copy of the complete report in flash drive, and the appropriate fees will be required for submittal.

ALAN DANG
Structural Engineering Associate II

AD/ad
Log No. 130835
213-482-0480

cc: SoCalGeo, Project Consultant
LA District Office

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

District LA	Log No. 130835

APPLICATION FOR REVIEW OF TECHNICAL REPORTS

INSTRUCTIONS

A. Address all communications to the Grading Division, LADBS, 221 N. Figueroa St., 12th Fl., Los Angeles, CA 90012
Telephone No. (213)482-0480.
B. Submit two copies (three for subdivisions) of reports, one "pdf" copy of the report on a CD-Rom or flash drive, and one copy of application with items "1" through "10" completed.
C. Check should be made to the City of Los Angeles.

1. LEGAL DESCRIPTION
Tract: See Attachment "A"
Block: Lots:

2. PROJECT ADDRESS:
1716 East 7th Street; see "Attachment A" for additional addresses.

3. OWNER: 1614 E 7th Street LLC c/o Arteen Mnayan, Mayer Brown LLP
Address: 333 S. Grand Avenue, 47th Floor
City: CA Zip: 90071
Phone (Daytime): 213-229-5158

4. APPLICANT 1614 E 7th Street LLC c/o Arteen Mnayan, Mayer Brown LLP
Address: Same as owner.
City: Zip:
Phone (Daytime): 213-229-5158
E-mail address: amnayan@mayerbrown.com

5. Report(s) Prepared by:
Southern California Geotechnical, Preliminary Geotechnical Investigation Alameda Crossing Development (20G243-4R)

6. Report Date(s):
May 7, 2024

7. Status of project: ☑ Proposed ☐ Under Construction ☐ Storm Damage
8. Previous site reports? ☐ YES if yes, give date(s) of report(s) and name of company who prepared report(s)

9. Previous Department actions? ☐ YES if yes, provide dates and attach a copy to expedite processing.
Dates:
10. Applicant Signature: [signature] Position: Representative

(DEPARTMENT USE ONLY)

REVIEW REQUESTED	FEES	REVIEW REQUESTED		FEES
☒ Soils Engineering	363.00	No. of Lots	19	$90
☐ Geology		No. of Acres		
☐ Combined Soils Engr. & Geol.		☐ Division of Land		
☐ Supplemental		Other		
☐ Combined Supplemental		☒ Expedite		226.50
☐ Import-Export Route		☐ Response to Correction		
Cubic Yards:		☐ Expedite ONLY		
		Sub-total		685.50
		Surcharge		153.50
		TOTAL FEE		839.00

Fee Due: 839.00
Fee Verified By: Am Date: 5/22/24
(Cashier Use Only)

Receipt # 1846164

Paid on 5/28/24

ACTION BY:

THE REPORT IS: ☐ NOT APPROVED
☐ APPROVED WITH CONDITIONS ☐ BELOW ☐ ATTACHED

For Geology Date

For Soils Date

October 17, 2024

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Prologis
2141 Rosecrans Avenue, Suite 1151
El Segundo, California 90245

Attention: Mr. Jonathan Payne
VP, Development Officer

Project No.: **20G243-7**

Subject: **Ground Improvement Plan Review**
Alameda Crossing Development
1716 East 7th Street
Los Angeles, California

Reference: Preliminary Geotechnical Investigation, Alameda Crossing Development, 1716 East 7th Street, Los Angeles, California, prepared for Prologis by Southern California Geotechnical, Inc. (SCG), SCG Project No. 20G243-4R2, dated October 9, 2024.

Mr. Payne:

In accordance with the requirements of the City of Los Angeles, Department of Building and Safety, we have reviewed the ground improvement plans and calculations for the proposed development at the subject site. These plans and calculations were reviewed for conformance with the assumptions, conclusions, and recommendations made in the above-referenced geotechnical report. The ground improvement plans and calculations for this project were prepared by Western Ground Improvement, Inc. (WGI). The plans reviewed by our office are identified as follows:

- Sheet GP0.1, Geopier Notes and Details, dated October 3, 2024.
- Sheet GP0.2, Geopier Details, dated October 3, 2024.
- Sheet GP0.3, Geopier Schedule, dated October 3, 2024.
- Sheet GP1.1, Geopier Location Plan, dated October 3, 2024.

Additionally, we have attached a Request for Modification of Building Ordinances (RFM) to this letter, which will be submitted to the City of Los Angeles, Department of Building and Safety for review and approval. This RFM was prepared by WGI and requests that the area replacement ratio be less than the minimum of 30% that is referenced in the LARR 26139.

Based on our review, the ground improvement plans and calculations have been prepared in accordance with the recommendations of the referenced geotechnical report. It should be noted that our review was limited to the geotechnical aspects of the project and no representations as to the suitability of the ground improvement design are intended.

Closure

We appreciate the opportunity to be of continued service on this project. If there are any questions concerning this matter, please contact our office at your convenience.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Ricardo Frias, RCE 91722
Project Engineer

Gregory K. Mitchell, GE 2364
Principal Engineer

Enclosures: City of Los Angeles: Request for Modification of Building Ordinances

Distribution: (1) Addressee

REQUEST FOR MODIFICATION OF BUILDING ORDINANCES

UNDER AUTHORITY OF L.A.M.C. SECTION 98.0403

CITY LA DBS — DEPARTMENT OF BUILDING AND SAFETY

PERMIT APP. #:	**DATE:** 10/03/2024	**For City Dept. Use Only**
JOB ADDRESS: 1716 E 7th Street, Los Angeles, CA 90021		Building Zoning Grading Shoring Mech. Elec. Plumb. Green D.A. Misc.
Tract: E.B Miller Tract	**Block:** C **Lot:** 3	
Owner: Prologis	Petitioner: Western Ground Improvement	
Address: 2141 Rosecrans Avenue	Address: 209 Avenida Del Mar, Ste. 201B	
City El Segundo · State CA · Zip 90245 · Phone 610-722-4139	City San Clemente · State CA · Zip 92672 · Phone 310-951-2986	

REQUEST (SUBMIT PLANS OR ADDITIONAL SHEETS AS NECESSARY) — CODE SECTIONS: LARR 26139

(1) For the spread footing foundations supported by Geopier Rammed Aggregate Piers (RAPs), we request that the area replacement ratio (Ra) be less than the minimum of 30% that is referenced in the LARR 26139.

JUSTIFICATION (SUBMIT PLANS OR ADDITIONAL SHEETS AS NECESSARY)

As shown in the footings calculations, the factor of safety (FS) is 2.0 or greater, and the bearing capacity calculations show a FS of 5.5 or greater.

The anticipated settlement under the footings is about 1-inch or less.

Ryan Bulatao	Ryan Bulatao	Regional Manager - Los Angeles
Owner/Petitioner Name (Print)	**(Signature)**	**Position**

FOR CITY DEPARTMENT'S USE ONLY BELOW THIS LINE

Concurrences required from the following Department(s)

Department	Print Name	Sign	Approved	Denied
☐ Los Angeles Fire Department	Print Name	Sign	☐	☐
☐ Public Works Bureau of Engineering	Print Name	Sign	☐	☐
☐ Department of City Planning	Print Name	Sign	☐	☐
☐ Department of County Health	Print Name	Sign	☐	☐
☐ Other ________	Print Name	Sign	☐	☐

DEPARTMENT ACTION

☐ **GRANTED** ☐ **DENIED**

Reviewed by: (Staff) (Print) *Sign* *Date*

Action taken by: (Supervisor) (Print) *Sign* *Date*

NOTE: IN CASE OF DENIAL, SEE PAGE #2 OF THIS FORM FOR APPEAL PROCEDURES

CONDITIONS OF APPROVAL (Continued on Page 2):

For Cashiers Use Only

(PROCESS ONLY WHEN FEES ARE VERIFIED)

(DEPARTMENT USE ONLY)

FEES

Appeal Processing Fee.. (No. of Items) =	X $130 + $39/addl	= ______
Inspection Fee .... (No of Insp.) =	X $ 84.00	= ______
Research Fee ... (Total Hours Worked) =	X $104.00	= ______
Subtotal		= ______
Development Services Center Surcharge	X 3%	= ______
Systems Development Surcharge	X 6%	= ______
Total Fees		= ______

Fees verified by:

Print and Sign ________________

CONDITIONS OF APPROVAL (Continued from Page 1)

CITY OF LOS ANGELES
BOARD OF BUILDING AND SAFETY/DISABLED ACCESS COMMISSION APPEAL FORM

(Must be Attached to the Modification Request Form, Page 1)

AFFIDAVIT – LADBS BOARD OF BUILDING AND SAFETY COMMISSIONERS – RESOLUTION NO. 832-93

I, ______________________________ do state and swear as follows:
(Print or Type Name of the Person Signing this Form)

1. The name and mailing address of the owner of the property (as defined in the resolution 832-93) at ______________________ as shown on the appeal application (LADBS Com 31) are correct, and
2. The owner of the property as shown on the appeal application will be made aware of the appeal and will receive a copy of the appeal.

I declare under PENALTY OF PERJURY that the forgoing is true and correct.

Owner's Name(s) ______________________ (Please Type or Print) ______________________ (Please Type or Print)

Owner's Signature(s) ______________________ (Please Sign) (Two Officers' Signatures Required for Corporations)

Name of Corporation ______________________ (Please Print Name of Corporation) ______________________ (Please Type or Print)

Dated this ________ day of ______________________ 20______

CALIFORNIA ALL-PURPOSE ACKNOWLEDGEMENT------------------------SIGNATURE(S) MUST BE NOTARIZED

State of **CALIFORNIA** County of ________________ on ______________________

before me, ______________________, personally appeared ______________________,
Name, Title of Officer (e.g. Jane Doe, Notary Public) Name(s) of Signer(s)

who proved to me on the basis of satisfactory evidence to be the person(s) whose name(s) is/are subscribed to the within instrument and acknowledged to me that he/she/they executed the same in his/her/their authorized capacity(ies), and that by his/her/their signature(s) on the instrument in person(s), or the entity upon behalf of which the person(s) acted, executed the instrument. **I certify under PENALTY OF PERJURY under the laws of the State of California that the foregoing is true and correct.**

WITNESS my hand and official seal. Signature ______________________

As a covered entity under Title II of the Americans with Disabilities Act, the City of Los Angeles does not discriminate on the basis of disability and, upon request, will provide reasonable accommodation to ensure equal access to its programs, services and activities.

APPEAL OF DEPARTMENT ACTION TO THE BOARD OF BUILDING AND SAFETY COMMISSIONERS/DISABLED ACCESS APPEALS COMMISSION

Applicant's Name ______________________ Applicant's Title ______________________

Signature ______________________ Date ______________________

FEES

(DEPARTMENT USE ONLY)

Board Fee .... (No. of Items)		X	$354.00	= ________
Inspection Fee .... (No of Insp.) =		X	$84.00	= ________
Research Fee ... (Total Hours Worked) =		X	$104.00	= ________
Subtotal..				= ________
Development Services Center Surcharge		X	3%	= ________
Systems Development Surcharge		X	6%	= ________
Total Fees..				= ________

Fees verified by:

Print and Sign ______________________

For Cashiers Use Only
(PROCESS ONLY WHEN FEES ARE VERIFIED)

December 10, 2024
(Revised February 3, 2025)

Prologis
2141 Rosecrans Avenue, Suite 1151
El Segundo, California 90245

Attention: Mr. Jonathan Payne
VP, Development Officer

Project No.: **20G243-8R2**

Subject: **Response to Soils Report Review Letter (November 22, 2024)**
Alameda Crossing Development
1716 East 7th Street
Los Angeles, California

References: 1) Preliminary Geotechnical Investigation, Alameda Crossing Development, 1716 East 7th Street, Los Angeles, California, prepared for Prologis by Southern California Geotechnical, Inc. (SCG), SCG Project No. 20G243-4R2, dated October 9, 2024.

2) Soils Report Review Letter, prepared by City of Los Angeles, Department of Building and Safety, dated November 22, 2024, Log# 130835-01.

Mr. Payne:

This letter provides our response to the Soils Report Review letter generated by the City of Los Angeles Department of Building and Safety (LADBS) dated November 22, 2024. A copy of the review sheet is attached with this correspondence for reference purposes. Only one of the review comments required a geotechnical response and is presented below.

LAC 2: *As previously requested, provide recommendations for geopier design which shall include, as a minimum the following:*

a. *Engineering design of the RAP system, including: bearing capacity and settlement analysis without and with R.A.P., design calculations, range of pier diameters and depths, replacement ratio and acceptable aggregate types and size specifications.*
b. *Requirements for an indicator RAP "Modulus Test" (MT) program, and other field-testing methods and procedures.*
c. *The location of the proposed MT, test pier dimensions, acceptable methods of installation and approval criteria.*
d. *Geopier setbacks from adjacent property lines.*

SCG 2: a. Detailed information regarding the engineering design of the rammed aggregate piers (RAP) system is included in the "Design Submittal for a Geopier Foundation System," plan and calculations prepared by Western Ground Improvement (WGI), and attached to this report (WGI Report). Please refer to Sheet GP0.1 of the WGI plan and calculations, included herein, which includes RAP design parameters, estimated settlement, and schedule. The allowable bearing pressure using the RAP system is 8,000 lbs/ft^2, with associated settlements of ¾ to 1± inch. Without the RAP system, and along with the remedial grading recommendations presented in the project soils report (Reference 1), the allowable bearing pressure would be limited to 2,500 lbs/ft^2, with an estimated settlement of 1± inch, as indicated in Section 6.5 of the referenced soils report. Based on the data provided on Sheet GP0.1 of the WGI Report, the Geopier diameter is 20 inches and the minimum design shaft length will range form 10 to 15 feet. Per the WGI Report, the replacement ratio will range between 8 to 18 percent. The aggregate types should be in accordance with WGI's recommendations based on the method of installation and with Condition #12 of LARR 26139, which consists of aggregate in accordance with ASTM D-1241, or other aggregate approved by the designer.

b. Requirements for the Geopier Modulus Test (MT) program are included on Sheet GP0.3 of the WGI Report, and on Pages 44 and 46 attached with this response report. These requirements are in accordance with Conditions #15, #16, and #17 of LARR 26139. Additional field tests will be in accordance with Condition #22 of LARR 26139.

c. The location of the MT program is indicated on Figure 1, attached with this report. Criteria for the MT program are included on GP0.3 and Page 44 attached with this report. These requirements are in accordance with Conditions #15, #16, and #17 of LARR 26139.

d. Geopier setbacks from adjacent property lines should be in accordance with Condition #5 of LARR 26139, which indicates a minimum distance of 8 feet.

LAC S4: *ABC slot-cuts were mentioned on page 21 of the report dated 10/17/2024 (20G243-7), however incomplete recommendations were provided. Provide complete ABC slot-cut recommendations and calculations considering the maximum height and width of the slot, and surcharge load from the existing foundations.*

SCG S4: In accordance with LADBS' request, we have performed slot-cut calculations for the site. Based on our review of the preliminary architectural plans, some of the new buildings will be constructed within close proximity of the property lines and the public right-of-way. In isolated locations, remedial grading for the new building areas will likely extend to the property line. In order to protect the existing public right-of-way, A-B-C slot cuts may be necessary in some localized areas. Based on the direct shear testing performed as part of the Reference 1 report, the existing soils are expected to possess an average internal friction angle of at least 31 degrees and a average cohesion of 150 lbs/ft^2. Based on our review of the preliminary architectural plans, excavations along property line will not expose areas of new foundations. Therefore, surcharge loads from existing structures were not considered necessary for the analysis.

Based on the subsurface profile identified in the Reference 1 report, above, excavations for the new building areas will likely extend to a depth of up to 8 feet below existing site grades. Therefore, the slot cut excavations were analyzed for excavations that are 8 feet deep, and no more than 6 feet wide. The results of the slot cutting calculations are presented on the enclosed spreadsheet.

The results of the slot cutting calculations indicate that the proposed A-B-C slots possess a factor of safety of at least 1.5. The safety factor of 1.5 is the acceptable standard when evaluating the stability of cut, fill, and natural slopes.

The Soils Report Review letter also requested a formal submission of the Request for Modification application that was included in the report dated 10/17/2024 (20G243-7) (see LADBS comment LAC3). Enclosed herein is the formal submission of the Request for Modification application (also prepared by WGI).

Closure

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.

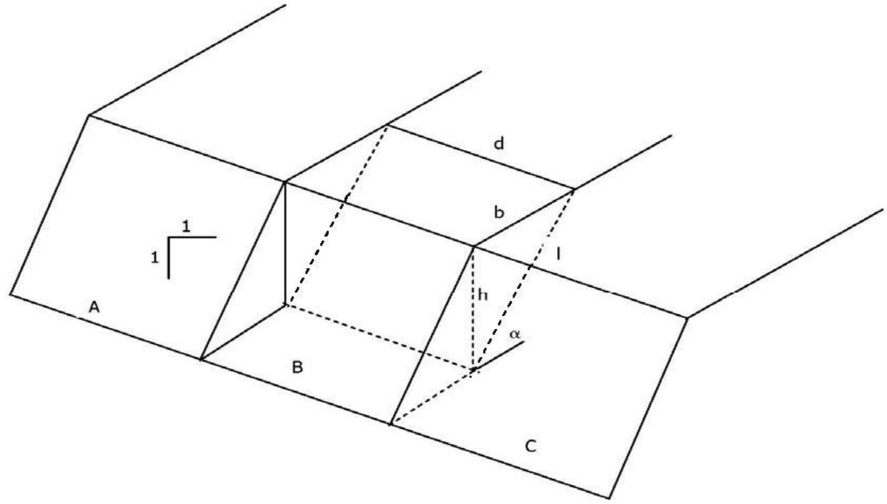
Pablo Montes Jr.
Project Engineer

Gregory K. Mitchell, GE 2364
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
GREGORY K. MITCHELL
No. 2364
GEOTECHNICAL
STATE OF CALIFORNIA

Enclosures: Slot Cutting Calculations
GMED Geologic and Soils Review Sheet
Geopier RFM Submittal – 2024-12-10
Geopier RFM Submittal Confirmation -2024-12-10

Distribution: (1) Addressee
(1) Mayer Brown LLP



Symbol Definition

Proposed Slot Width	d =	8	ft
Height of Exposed Cut	h =	8	ft
Moist Unit Weight of Soil	γ_n =	120	lbs/ft^3
Soil Internal Angle of Friction	ϕ =	31	degrees
Soil Cohesion	c =	150	lbs/ft^2
Surcharge Due to Adjacent Footing	q =	0	lbs/ft^2

Slot Cutting Analysis

Inclination of Active Failure Plane	α =	$45 + \phi/2$	degrees
	α =	60.5	degrees
Coefficient of At-Rest Pressure	K_0 =	$1 - \sin(\phi)$	
	K_0 =	0.485	
Width of Side Shear	b =	$h / \tan(\alpha)$	ft
	b =	4.53	ft
Area of Side Shear (1 side)	A =	1/2 * b * h	ft^2
	A =	18.10	ft^2
Side Shear Force (1 side)	F =	$A * (1/3 * \gamma n * h * K_0 * \tan(\phi) + c)$	
	F =	18.1 * (1/3 * 120 * 8 * 0.485 * tan(31°) + 150)	
	F =	4404	lbs
Weight of Sliding Mass	W =	$A * \gamma_n$	lbs/ft
	W =	2173	lbs/ft
Driving Force	F_D =	$d * [W * \cos(\alpha) * \sin(\alpha) + q * \cos(\alpha)]$	
	F_D =	8 * [2173 * cos(60.5°) * sin(60.5°) + 0 * cos(60.5°)]	
	F_D =	7449	lbs/ft^2
Resisting Force	F_R =	$d *[(W * \cos^2(\alpha) * \tan(\phi) + (c * b)] + 2 * F$	
	F_R =	8 * [2173 * cos^2(60.5°) * tan(31°) + (150 * 4.53)] + 2 * 4404	
	F_R =	16772	lbs/ft^2
Factor of Safety	FS =	F_R / F_D	
	FS =	16772 / 7449	
	FS =	2.25	

Source: City of Los Angeles Slot Cutting Procedure, date unknown

SLOT CUTTING ANALYSIS
Alameda Crossing Development
Los Angeles, California
Project No. 20G243-8

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BOARD OF
BUILDING AND SAFETY
COMMISSIONERS

JACOB STEVENS
PRESIDENT

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CALIFORNIA

KAREN BASS
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

JOHN WEIGHT
EXECUTIVE OFFICER

SOILS REPORT REVIEW LETTER

November 22, 2024

LOG # 130835-01
SOILS/GEOLOGY FILE - 2

ProLogis
2141 Rosecrans Ave. #1151
El Segundo, CA 90245

TRACT:	E. B. MILLAR TRACT
BLOCK:	C
LOT(S):	3
LOCATION:	1716 E. 7th St.

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Soils Report	20G243-6	10/17/2024	SoCalGeo
Addendum Report	20G243-7	10/17/2024	``
Update Report	20G243-4R2	10/17/2024	``
Addendum Report	20G243-5R2	05/07/2024	``

PREVIOUS REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Dept. Review Letter	130835	06/14/2024	LADBS
Soils Report	20G243-4R	05/07/2024	SoCalGeo
Addendum Report	20G243-5R2	05/07/02024	SoCalGeo
Dept. Approval Letter	123370-01	01/18/2023	LADBS
Soils Report	20G243-3	12/14/2022	SoCalGeo
Dept. Review Letter	123370	10/20/2022	LADBS
Soils Report	20G243-2	09/22/2022	SoCalGeo

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provide recommendations for the proposed main building: 8 level mixed use structure (6 levels of parking), and three, one-story structures (i.e., Stage Groups A, B, and C). The earth materials at the subsurface exploration locations consist of up to 8 feet of uncertified fill underlain by native soils.The consultants recommend to support the proposed structure(s) on conventional, mat-type, and/or rammed aggregate piers foundations bearing on properly placed fill or native soils.

The review of the subject report(s) cannot be completed at this time and will be continued upon submittal of an addendum to the report which shall include, but not be limited to, the following:

AN EQUAL EMPLOYMENT OPPORTUNITY - AFFIRMATIVE ACTION EMPLOYER

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2023 City of LA Building Code. P/BC numbers refer to the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. Provide an itemized response to the review items indicated in this letter. Do not revise existing reports or use solely references as a response to review items.

2. As previously requested, provide recommendations for the geopier design which shall include, as a minimum, the following:

    a. Engineering design of the RAP system, including: bearing capacity and settlement analysis without and with RAP, design calculations, range of pier diameters and depths, replacement ratio and acceptable aggregate types and size specifications.

    b. Requirements for an indicator RAP "Modulus Tests" (MT) program, and other field-testing methods and procedures.

    c. The location of the proposed MT, test pier dimensions, acceptable methods of installation and approval criteria.

    d. Geopier setbacks from adjacent property lines.

3. A Request for Modification application was included in the report dated 10/17/2024 (20G243-7), however was not formally submitted. Provide a formal submission of the application using the online submission portal or in-person drop off. Note: https://ladbs.org/forms-publications/forms/requests-for-modifications may be use for online submission

4. ABC slot-cuts were mentioned on page 21 of the report dated 10/17/2024 (20G243-4R2), however incomplete recommendations were provided. Provide complete ABC slot-cut recommendations and calculations considering the maximum height and width of the slot, and surcharge load from the existing foundation.

If clarification concerning the review letter is necessary, the report review engineer may be contacted. Two copies of the response report, including one unbound wet-signed original for archiving purposes, a pdf-copy of the complete report in flash drive, and the appropriate fees will be required for submittal.

ALAN DANG
Structural Engineering Associate II

AD/ad
Log No. 130835-01
213-482-0480

cc: SoCalGeo, Project Consultant
LA District Office

CITY LA DBS DEPARTMENT OF BUILDING AND SAFETY

REQUEST FOR MODIFICATION OF BUILDING ORDINANCES

UNDER AUTHORITY OF L.A.M.C. SECTION 98.0403

PERMIT APP. #:	**DATE:** 12/10/2024	**For City Dept. Use Only**
JOB ADDRESS: 1716 E 7th Street, Los Angeles, CA 90021		**Building** **Zoning** **Grading** **Shoring** **Mech.** **Elec.** **Plumb.** **Green** **D.A.** **Misc.**
Tract: E.B Miller Tract	**Block:** C **Lot:** 3	

Owner: Prologis	Petitioner: Western Ground Improvement
Address: 2141 Rosecrans Avenue	Address: 209 Avenida Del Mar, Ste. 201B
City El Segundo, State CA, Zip 90245, Phone 610-722-4139	City San Clemente, State CA, Zip 92672, Phone 310-951-2986

REQUEST (SUBMIT PLANS OR ADDITIONAL SHEETS AS NECESSARY) — CODE SECTIONS: LARR 26139

(1) For the spread footing foundations supported by Geopier Rammed Aggregate Piers (RAPs), we request that the area replacement ratio (Ra) be less than the minimum of 30% that is referenced in the LARR 26139.

JUSTIFICATION (SUBMIT PLANS OR ADDITIONAL SHEETS AS NECESSARY)

As shown in the footings calculations, the factor of safety (FS) is 2.0 or greater, and the bearing capacity calculations show a FS of 5.5 or greater.

The anticipated settlement under the footings is about 1-inch or less.

Ryan Bulatao	Ryan Bulatao	Regional Manager - Los Angeles
Owner/Petitioner Name (Print)	**(Signature)**	**Position**

FOR CITY DEPARTMENT'S USE ONLY BELOW THIS LINE

Concurrences required from the following Department(s)			Approved	Denied
☐ Los Angeles Fire Department	Print Name____	Sign____	☐	☐
☐ Public Works Bureau of Engineering	Print Name____	Sign____	☐	☐
☐ Department of City Planning	Print Name____	Sign____	☐	☐
☐ Department of County Health	Print Name____	Sign____	☐	☐
☐ Other ____	Print Name____	Sign____	☐	☐

DEPARTMENT ACTION

☐ **GRANTED** ☐ **DENIED**

Reviewed by: (Staff) (Print) *Sign* *Date*

Action taken by: (Supervisor) (Print) *Sign* *Date*

NOTE: IN CASE OF DENIAL, SEE PAGE #2 OF THIS FORM FOR APPEAL PROCEDURES

CONDITIONS OF APPROVAL (Continued on Page 2):

For Cashiers Use Only
(PROCESS ONLY WHEN FEES ARE VERIFIED)

FEES **(DEPARTMENT USE ONLY)**

Appeal Processing Fee.. (No. of Items) =	X $130 + $39/addl	= ____
Inspection Fee .... (No of Insp.) =	X $ 84.00	= ____
Research Fee ... (Total Hours Worked) =	X $104.00	= ____
Subtotal		= ____
Development Services Center Surcharge	X 3%	= ____
Systems Development Surcharge	X 6%	= ____
Total Fees		= ____

Fees verified by:

Print and Sign ____

Permit App #:	Job Address: 1716 E 7th Street, Los Angeles, CA 90021

CONDITIONS OF APPROVAL (Continued from Page 1)

CITY OF LOS ANGELES
BOARD OF BUILDING AND SAFETY/DISABLED ACCESS COMMISSION APPEAL FORM

(Must be Attached to the Modification Request Form, Page 1)

AFFIDAVIT – LADBS BOARD OF BUILDING AND SAFETY COMMISSIONERS – RESOLUTION NO. 832-93

I, ______________________________ do state and swear as follows:
(Print or Type Name of the Person Signing this Form)

1. The name and mailing address of the owner of the property (as defined in the resolution 832-93) at ______________________ as shown on the appeal application (LADBS Com 31) are correct, and
2. The owner of the property as shown on the appeal application will be made aware of the appeal and will receive a copy of the appeal.

I declare under PENALTY OF PERJURY that the forgoing is true and correct.

Owner's Name(s) ______________________________ ______________________________
(Please Type or Print) (Please Type or Print)

Owner's Signature(s) ______________________________ (Two Officers' Signatures Required for Corporations)
(Please Sign)

Name of Corporation ______________________________ ______________________________
(Please Print Name of Corporation) (Please Type or Print)

Dated this __________ day of ______________________________ 20______

CALIFORNIA ALL-PURPOSE ACKNOWLEDGEMENT------------------------SIGNATURE(S) MUST BE NOTARIZED

State of **CALIFORNIA** County of ____________________ on ______________________________

before me, ______________________________, personally appeared ______________________________,
Name, Title of Officer (e.g. Jane Doe, Notary Public) Name(s) of Signer(s)

who proved to me on the basis of satisfactory evidence to be the person(s) whose name(s) is/are subscribed to the within instrument and acknowledged to me that he/she/they executed the same in his/her/their authorized capacity(ies), and that by his/her/their signature(s) on the instrument in person(s), or the entity upon behalf of which the person(s) acted, executed the instrument. **I certify under PENALTY OF PERJURY under the laws of the State of California that the foregoing is true and correct.**

WITNESS my hand and official seal. Signature ______________________________

As a covered entity under Title II of the Americans with Disabilities Act, the City of Los Angeles does not discriminate on the basis of disability and, upon request, will provide reasonable accommodation to ensure equal access to its programs, services and activities.

APPEAL OF DEPARTMENT ACTION TO THE BOARD OF BUILDING AND SAFETY COMMISSIONERS/DISABLED ACCESS APPEALS COMMISSION

Applicant's Name ______________________________ Applicant's Title ______________________________

Signature ______________________________ Date ______________________________

FEES

(DEPARTMENT USE ONLY)

Board Fee .... (No. of Items)		X	$354.00	= ________
Inspection Fee .... (No of Insp.) =		X	$84.00	= ________
Research Fee ... (Total Hours Worked) =		X	$104.00	= ________
Subtotal				= ________
Development Services Center Surcharge		X	3%	= ________
Systems Development Surcharge		X	6%	= ________
Total Fees				= ________

Fees verified by:

Print and Sign ______________________________

For Cashiers Use Only

(PROCESS ONLY WHEN FEES ARE VERIFIED)

WGI | GEOPIER®

Western Ground Improvement, Inc.
209 Avenida Del Mar
Suite 201B
San Clemente, CA 92672

Tel: (310) 717-3428
www.westerngroundimprovement.com

October 3, 2024

Jonathan Payne
Prologis
2141 Rosecrans Avenue, suite 1151
El Segundo, CA 90245

Re: Design Submittal for a *Geopier®* Foundation System
Alameda Crossing
1716 East 7th Street
Los Angeles, CA 90021
GFC Project No.: GLA-229 / NLA-126

Dear Mr. Payne,

Western Ground Improvement, Inc. has completed the Geopier® foundation design for above project. The following documents are included herein:

- Geopier Design Drawing GP0.1: Geopier Notes & Details
- Geopier Design Drawing GP0.2: Geopier Details
- Geopier Design Drawing GP0.3: Geopier Schedules
- Geopier Design Drawing GP1.1: Geopier Location Plan

We are pleased to have provided you with our design services. If you have any questions, please contact this office.

Sincerely,
Western Ground Improvement, Inc.

Ryan Bulatao, G.E., P.E.
Regional Manager

REGISTERED PROFESSIONAL ENGINEER
RYAN PAUL BULATAO
GE 3021
GEOTECHNICAL
STATE OF CALIFORNIA

GEOPIER DESIGN NOTES:

1. GEOPIER FOUNDATION SUPPORT IS AS DESIGNED BY GEOPIER FOUNDATION COMPANY, DAVIDSON, NORTH CAROLINA (DESIGNER).
2. THESE DESIGN DRAWINGS ARE PREPARED BY THE DESIGNER FOR USE IN GEOPIER CONSTRUCTION. THE GEOPIER SYSTEM SHALL BE INSTALLED BY APPROVED INSTALLERS LICENSED BY GEOPIER FOUNDATION COMPANY. UNAUTHORIZED USE OF THESE DRAWINGS IS PROHIBITED.
3. THE GEOPIER FOUNDATION DESIGN IS BASED ON THE GEOTECHNICAL INFORMATION PROVIDED IN THE SUBSURFACE EXPLORATION BY SOUTHERN CALIFORNIA GEOTECHNICAL IN THE REPORT DATED 05/07/24. GEOPIER FOUNDATION COMPANY HAS RELIED ON THIS INFORMATION AND WE HAVE NO REASON TO SUSPECT ANY OF THE INFORMATION IN THE REPORT IS IN ERROR. GEOPIER FOUNDATION COMPANY IS NOT RESPONSIBLE FOR ERRORS OR OMISSIONS IN THE REPORT THAT MAY AFFECT THE PARAMETER VALUES IN OUR DESIGN. IF THE SUBSURFACE OR SITE CONDITIONS DIFFER FROM THOSE UTILIZED IN THE DESIGN THE DESIGNER SHALL BE NOTIFIED IMMEDIATELY.
4. THE ALLOWABLE BEARING PRESSURE FOR FOUNDATIONS SUPPORTED BY GEOPIER ELEMENTS IS AS REFERENCED IN DETAIL 1/GP0.1. THE GEOPIER LAYOUT IS DESIGNED TO PROVIDE SETTLEMENT CONTROL BASED ON SERVICE LOADS PROVIDED BY MAGNUSSON KLEMENCIC ASSOCIATES. IN THE EVENT THE STRUCTURAL LOADS VARY, THE DESIGNER SHALL BE NOTIFIED IMMEDIATELY.
5. FOOTING ELEVATIONS ARE THE RESPONSIBILITY OF THE GENERAL CONTRACTOR AND SHALL BE REPORTED IN WRITING TO THE INSTALLER'S QC REPRESENTATIVE PRIOR TO INSTALLING GEOPIER ELEMENTS.

GEOPIER LAYOUT NOTES:

1. GEOPIER ELEMENT LAYOUT IS THE RESPONSIBILITY OF THE GENERAL CONTRACTOR. GEOPIER ELEMENTS SHALL BE INSTALLED IN THE FIELD WITHIN 6 INCHES OF LOCATIONS SHOWN ON THESE PLANS.
2. GEOPIER ELEMENTS ARE LOCATED RELATIVE TO THE INTERSECTION OF REFERENCE GRID LINES OR AT THE CENTERLINE OF STRIP FOOTINGS, UNLESS DIMENSIONED OTHERWISE. PLEASE REFER TO THE "FOOTING DETAILS" ON SHEET GP0.2 FOR SPECIFIC PIER LOCATIONS AND DIMENSIONS RELATIVE TO THE FOOTING.
3. THE "GEOPIER LOCATION PLAN" AND "FOOTING DETAILS" PROVIDE GEOPIER ELEMENT NUMBER, LOCATION, AND LAYOUT ONLY. FOOTING LOCATIONS, SIZES, AND ORIENTATION SHOWN ON THESE PLANS ARE FOR INFORMATION ONLY. PLEASE REFER TO THE STRUCTURAL PLANS FOR SPECIFIC FOUNDATION DIMENSIONS AND LOCATIONS. THE DESIGNER ACCEPTS NO RESPONSIBILITY FOR THE LOCATION OF FOOTINGS SHOWN ON THESE PLANS. THE DESIGNER SHALL BE NOTIFIED IMMEDIATELY IF INFORMATION ON THESE PLANS CONFLICTS WITH STRUCTURAL OR ARCHITECTURAL DRAWINGS.

UTILITIES/OBSTRUCTION NOTES:

1. UTILITY LOCATIONS ARE THE RESPONSIBILITY OF THE GENERAL CONTRACTOR. THE DESIGNER SHALL BE NOTIFIED OF ANY CONFLICTS WITH GEOPIER LOCATIONS SHOWN ON THE PLANS. NEW TEMPORARY UTILITY EXCAVATIONS SHALL BE LIMITED TO THE ZONE DEPICTED ON DETAIL 2 OF THIS SHEET. IF EXCAVATIONS ARE PLANNED WITHIN THE GEOPIER "NO DIG" ZONE, THE DESIGNER SHALL BE NOTIFIED IMMEDIATELY TO DISCUSS EXCAVATION OPTIONS.
2. IF OBSTRUCTIONS ARE ENCOUNTERED DURING GEOPIER INSTALLATION THAT CANNOT BE REMOVED WITH CONVENTIONAL GEOPIER INSTALLATION EQUIPMENT, THE GENERAL CONTRACTOR SHALL BE RESPONSIBLE FOR REMOVING THE OBSTRUCTIONS. IF THE GENERAL CONTRACTOR DOES NOT DO SO IN A TIMELY MANNER THAT DOES NOT INTERRUPT GEOPIER PRODUCTION, THE INSTALLER MAY REMOVE OBSTRUCTION(S) AND SHALL BE REIMBURSED FOR COSTS INCURRED, INCLUDING LABOR, EQUIPMENT, AND MATERIALS. IN THE EVENT OBSTRUCTIONS ARE ENCOUNTERED BELOW THE DESIGNED BOTTOM OF FOOTING ELEVATION THE OBSTRUCTION SHALL BE REMOVED AS OUTLINED ABOVE. THE RESULTING EXCAVATION SHALL THEN BE BACKFILLED AND COMPACTED IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS FOR STRUCTURAL FILL. THE AREA SHALL BE TESTED BY THE OWNER'S TESTING AGENCY AND THE COMPACTION TEST RESULTS SHALL BE SUBMITTED TO THE INSTALLER AND THE DESIGNER.

CONSTRUCTION NOTES FOR CONCRETE FOUNDATIONS SUPPORTED BY GEOPIER® ELEMENTS:

1. ALL EXCAVATIONS FOR FOUNDATIONS SUPPORTED BY GEOPIER ELEMENTS SHALL BE PREPARED IN THE FOLLOWING MANNER BY THE GENERAL CONTRACTOR: OVEREXCAVATION BELOW THE BOTTOM OF FOUNDATION SHALL BE LIMITED TO THREE INCHES. THIS INCLUDES LIMITING THE TEETH OF EXCAVATORS FROM OVEREXCAVATION BEYOND THREE INCHES BELOW THE FOUNDATION ELEVATION.
2. FOUNDATION CONCRETE SHALL BE PLACED IMMEDIATELY FOLLOWING FOUNDATION EXCAVATION AND APPROVAL, PREFERABLY THE SAME DAY AS THE EXCAVATION. FOUNDATION CONCRETE SHALL BE PLACED ON THE SAME DAY IF THE FOUNDATION IS BEARING ON MOISTURE-SENSITIVE SOILS. IF SAME DAY PLACEMENT OF FOUNDATION CONCRETE IS NOT POSSIBLE, OPEN EXCAVATIONS SHALL BE PROTECTED FROM SURFACE WATER ACCUMULATION. A LEAN CONCRETE MUD-MAT MAY BE USED TO ACCOMPLISH THIS. GENERAL CONTRACTOR IS RESPONSIBLE FOR PROTECTION OF THE FINAL FOOTING SUBGRADE AND GEOPIER ELEMENTS FROM SURFACE WATER ACCUMULATION.
3. PRIOR TO CONCRETE OR MUD MAT PLACEMENT, THE TOP OF THE EXCAVATED SOIL AND GEOPIER ELEMENTS SHALL BE COMPACTED WITH A STANDARD, HAND-OPERATED IMPACT COMPACTOR (I.E. JUMPING JACK COMPACTOR). COMPACTION SHALL BE PERFORMED OVER THE ENTIRE FOUNDATION SUBGRADE TO COMPACT ANY LOOSE SURFACE SOIL AND LOOSE SURFACE GEOPIER AGGREGATE.
4. WATER SHALL NOT BE ALLOWED TO ACCUMULATE IN THE FOUNDATION EXCAVATIONS PRIOR TO CONCRETE PLACEMENT OR ALLOWED TO ACCUMULATE OVER THE POURED FOUNDATION.
5. EXCAVATION AND SURFACE COMPACTION OF ALL FOUNDATION SUBGRADES SHALL BE THE RESPONSIBILITY OF THE GENERAL CONTRACTOR.
6. THE TESTING AGENCY SHALL INSPECT EACH FOUNDATION AND APPROVE IT IN WRITING ON THE SAME DAY THAT THE CONCRETE OR MUD MAT IS PLACED IN THE FOUNDATION EXCAVATION. THE APPROVAL SHALL STATE THAT ALL FOUNDATION SUBGRADE, INCLUDING MATRIX SOILS AND GEOPIER TOPS, HAVE NOT BEEN OVEREXCAVATED MORE THAN THREE-INCHES BELOW THE BOTTOM OF THE FOUNDATION, HAVE BEEN KEPT FREE OF WATER ACCUMULATION, AND HAVE BEEN REASONABLY COMPACTED WITH A HAND-HELD MECHANICAL IMPACT COMPACTOR ON THE SAME DAY THAT THE CONCRETE WAS PLACED.
7. IN THE EVENT THAT FOUNDATION BOTTOM PREPARATIONS, AS DESCRIBED ABOVE, ARE NOT PERFORMED OR DOCUMENTED IN ACCORDANCE WITH THIS SECTION, ANY WRITTEN OR IMPLIED WARRANTY WITH RESPECT TO GEOPIER FOUNDATION PERFORMANCE CAN BY CONSIDERED VOID.

GEOPIER TESTING NOTES:

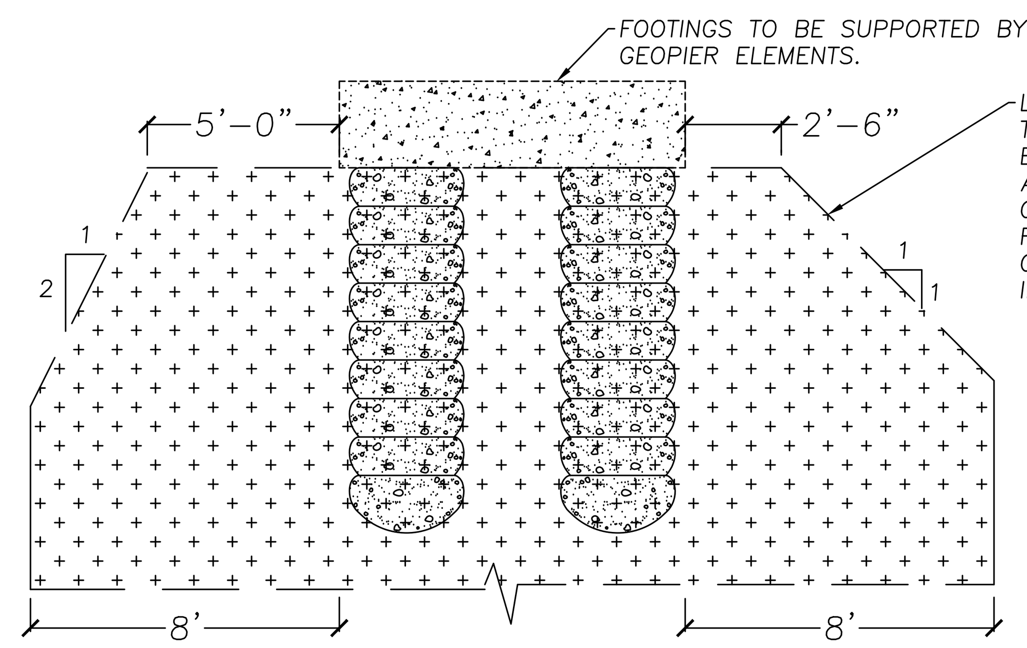
1. A QUALIFIED, FULL-TIME QUALITY CONTROL (QC) REPRESENTATIVE PROVIDED BY THE GEOPIER INSTALLER (INSTALLER) SHALL BE RESPONSIBLE FOR INSTALLATION OF THE GEOPIER ELEMENTS IN ACCORDANCE WITH THE DESIGN AND SHALL REPORT ALL GEOPIER FOUNDATION CONSTRUCTION ACTIVITIES TO THE DESIGNER. IF AUTHORIZED BY THE OWNER, THE QC REPRESENTATIVE SHALL COORDINATE QC ACTIVITIES WITH THE TESTING AGENCY HIRED BY THE OWNER. UNDER NO CIRCUMSTANCES SHALL THE TESTING AGENCY DIRECT GEOPIER INSTALLATION PROCEDURES.
2. GEOPIER ELEMENT DESIGN SHALL BE CONFIRMED BY A MODULUS TEST PERFORMED AT THE SITE. PLEASE REFER TO THE DESIGN SUBMITTAL FOR TEST LOCATION AND SPECIFICATIONS.
3. GEOPIER ELEMENTS SHALL BE BASED ON THE FOLLOWING CRITERIA UNLESS OTHERWISE APPROVED IN WRITING BY THE DESIGNER:
   A. DEPTHS SHALL BE WITHIN 3 INCHES OR DEEPER THAN THE DEPTHS SHOWN ON THE PLANS UNLESS REFUSAL IS ENCOUNTERED ON FORMATIONAL MATERIAL.
   B. CROWD STABILIZATION TEST (CST) SHALL BE PERFORMED ON THE FIRST FIVE (5) INSTALLED PIERS (INCLUDING PRE-PRODUCTION PIERS) TO ESTABLISH ACCEPTANCE CRITERIA FOR THE MAXIMUM ALLOWABLE DEFLECTION OF THE MANDREL UNDER THE FULL-STATIC CROWD PRESSURE OF THE CLOSED-ENDED MANDREL. CST SHALL BE PERFORMED BY SHUTTING THE HAMMER ENERGY OFF AT THE TOP OF A COMPACTED LIFT IN THE BOTTOM ONE-HALF OF THE PIER. ONCE THE HAMMER ENERGY IS OFF AND THE MANDREL IS RESTING ON TOP OF THE LAST COMPACTED LIFT, STATIC CROWD PRESSURE SHALL BE APPLIED TO THE PIER FOR A PERIOD OF TEN SECONDS. THE CORRESPONDING DEFLECTION OF THE MANDREL IS THEN NOTED AND RECORDED. RESULTS OF THE INITIAL CSTS SHALL BE PROVIDED TO THE DESIGNER FOR REVIEW AND ESTABLISHMENT OF ACCEPTANCE CRITERIA AND FREQUENCY OF CSTS. THE FREQUENCY OF CSTS MAY VARY DEPENDING ON THE SOIL CONDITIONS; HOWEVER, CSTS SHALL BE PERFORMED ON NO LESS THAN 20% OF THE PRODUCTION PIERS OR AS APPROVED BY THE DESIGNER.
   C. GEOPIER ELEMENT AGGREGATE SHALL BE APPROVED BY THE DESIGNER AND THE SAME AGGREGATE USED IN A SUCCESSFUL MODULUS TEST UNLESS OTHERWISE APPROVED IN WRITING BY THE DESIGNER.
4. GEOPIER ELEMENTS NOT MEETING DESIGN REQUIREMENTS SHALL BE REINSTALLED UNLESS OTHERWISE APPROVED IN WRITING BY THE DESIGNER.
5. SPECIAL INSPECTION IS REQUIRED AS NOTED IN DETAIL 3 ON SHEET GP0.1

Design Parameter	Value
Allowable bearing pressure (ksf)	8
Depth to groundwater (ft)	95
Total unit weight of soil (pcf)	120
Soil friction angle (degrees)	30

Type / Mark	Maximum Load, (kips, klf)	Width, (ft)	Length, (ft)	Thickness, (in)	Geopier® Diameter, (in)	Maximum Number of Geopier® Elements per Footing	Maximum Geopier® spacing, (ft)	Minimum Design Shaft Length, (ft)	Anticipated Settlement, (in) (1)	Notes
F1	510	10.0	10.0	30	20	6	-	15	3/4	
F2	840	12.0	12.0	36	20	10	-	14	3/4	
F3	1280	14.0	14.0	39	20	16	-	15	1	
F4	1610	16.0	16.0	42	20	20	-	14	1	
MAT	9400	45.0	62.5	72	20	120	-	10	1	
MAT	25700	62.5	125.0	72	20	380	-	10	1	
MAT	10250	52.0	63.0	72	20	120	-	10	1	

NOTES: *(1) Anticipated settlement is estimated to the nearest 1/4 inch.*

1 GEOPIER® DESIGN PARAMETERS AND ESTIMATED SETTLEMENT

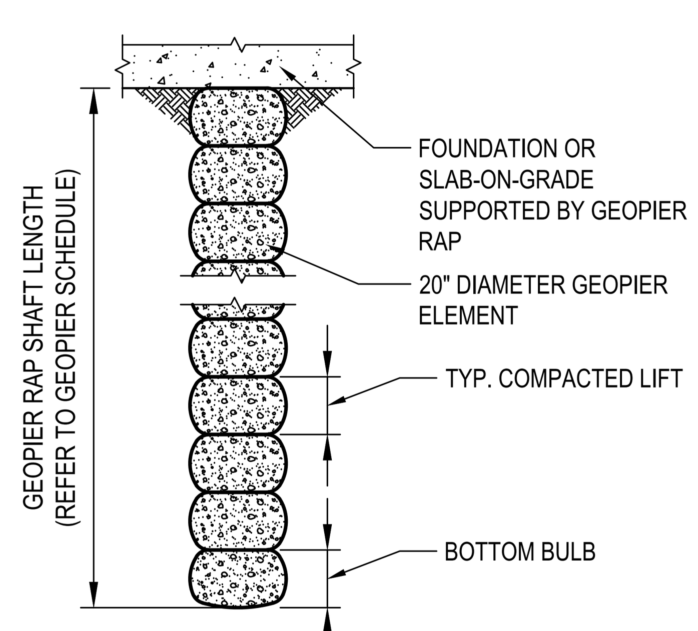


LIMITS OF DISTURBANCE: ADJACENT TEMPORARY UTILITY EXCAVATIONS MUST BE OUTSIDE THIS ZONE AND IN ACCORDANCE WITH PROJECT GEOTECHNICAL ENGINEER RECOMMENDATIONS. IF THIS IS NOT THE CASE THE DESIGNER MUST BE NOTIFIED IMMEDIATELY.

2 ADJACENT TEMPORARY EXCAVATION DETAIL
NOT TO SCALE

ADJACENT TEMPORARY EXCAVATION NOTES:

1. THIS DETAIL APPLIES TO EXCAVATIONS ONLY WITHIN A DISTANCE OF 8 FEET FROM THE EDGE OF THE GEOPIER ELEMENTS.
2. THIS DETAIL DOES NOT APPLY TO MASS EXCAVATION OR SITE GRADING.
3. THE PROJECT GEOTECHNICAL ENGINEER'S RECOMMENDATIONS SHALL BE FOLLOWED FOR TEMPORARY OR PERMANANT SLOPES.



4 TYPICAL GEOPIER® ELEMENT - IMPACT SYSTEM
NOT TO SCALE

SCHEDULE OF SPECIAL INSPECTION SERVICES - RAMMED AGGREGATE PIERS					
#	Item	Extent		Agency	Comments
		Cont.	Periodic		
1	Verify aggregate used in Geopier elements	X		QC	
2	Type and number of lifts	X		QC	
3	Pier dimensions and elevations	X		QC	
4	Rammer Energy	X		QC	
5	Observation of the installation	X		QC	
6	Modulus test		X	QC	
7	Crowd stabilization test (CST)		X	QC	

Notes: *(QC) - Quality Control by an approved testing laboratory, provided by Geopier installer.*

3 SCHEDULE OF SPECIAL INSPECTION SERVICES

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ZONE	REV	DESCRIPTION	DATE	APPROVED

REVISIONS

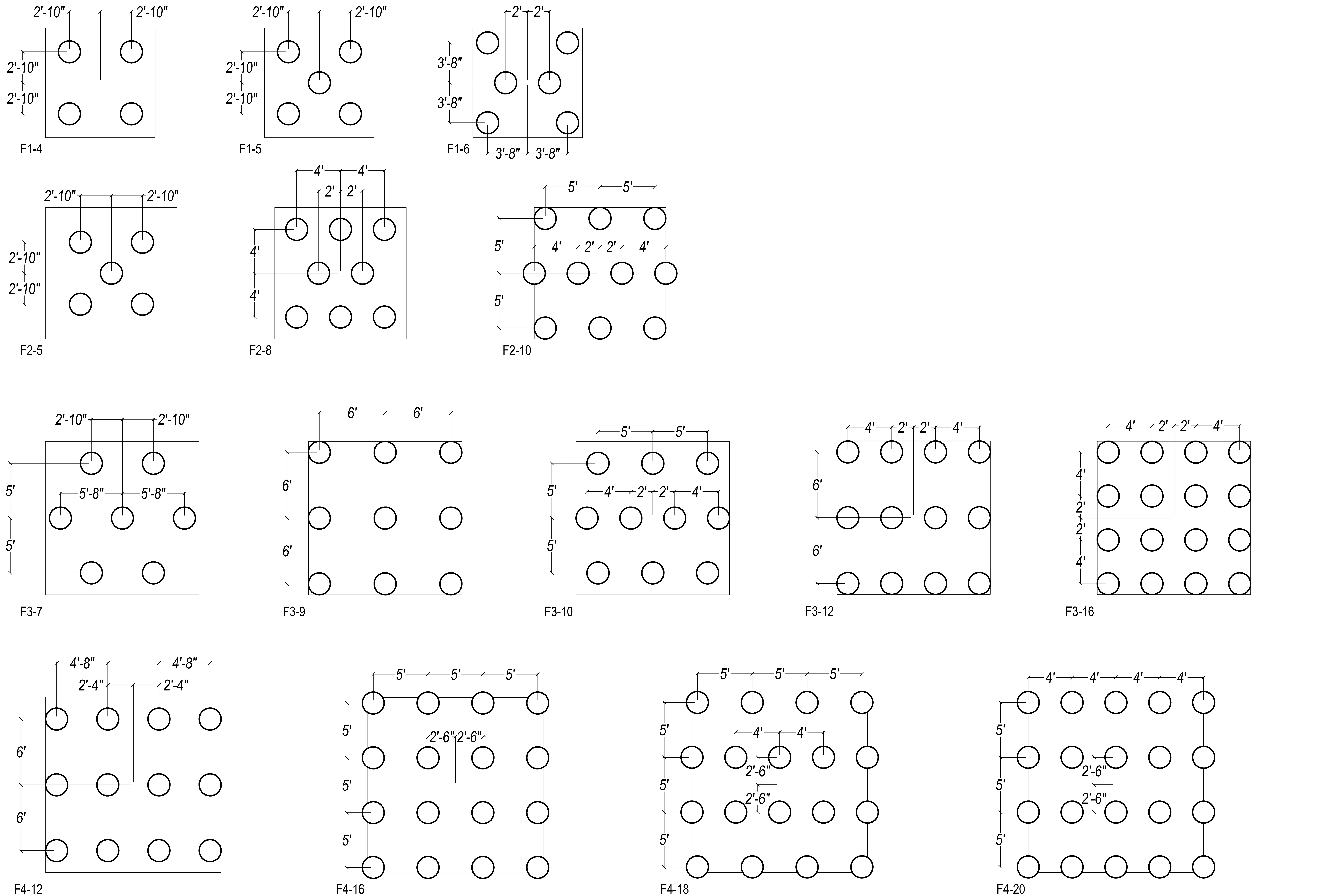
GEOPIER® NOTES AND DETAILS

ALAMEDA CROSSING
1716 EAST 7TH STREET
LOS ANGELES, CA

WGI GEOPIER®

PROJECT NUMBER GLA-229 / NLA-126
DRAWN BY RB
CHECK BY KH
DATE 10/03/24
SHEET NUMBER

GP0.1



1 FOOTING DETAILS
NOT TO SCALE

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REVISIONS				
ZONE	REV	DESCRIPTION	DATE	APPROVED

GEOPIER® DETAILS

REGISTERED PROFESSIONAL ENGINEER RYAN PAUL BULATAO GE 3021 GEOTECHNICAL STATE OF CALIFORNIA

ALAMEDA CROSSING
1716 EAST 7TH STREET
LOS ANGELES, CA

WGI GEOPIER®

PROJECT NUMBER GLA-229 / NLA-126
DRAWN BY RB
CHECK BY KH
DATE 10/03/24
SHEET NUMBER
GP0.2

Geopier Foundation Company®

GEOPIER®

Project Name: Ladbs - Alameda Crossing
Project Location: Los Angeles, Ca
Project Number: GLA-229 / NLA-126

GEOPIER® SCHEDULE

Column Line	Geopier® Number(s)	Type / Mark	Top of Footing Elevation, (ft)	Width, (ft)	Length, (ft)	Thickness, (in)	Geopier® Shaft Length, (ft)	Finish Floor Elevation, (ft) (1)	Top of Geopier® Elevation, (ft)	Notes
1-A	1 - 10	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
2-A	11 - 19	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
3-A	20 - 28	F3	-0.50	14.0	14.0	39	15	0.00	-3.75	
4-A	29 - 38	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
5-A	39 - 48	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
6-A	49 - 57	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
7-A	58 - 66	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
8-A	67 - 76	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
9-A	77 - 86	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
10-A	87 - 98	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
11-A	99 - 110	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
12-A	111 - 122	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
13-A	123 - 132	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
14-A	133 - 144	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
1-B	145 - 160	F4	-0.50	15.0	16.0	42	17	0.00	-4.00	
3.2-B	161 - 172	F3	-0.50	14.0	14.0	39	14	0.00	-3.75	
4-B	173 - 188	F4	-0.50	15.0	16.0	42	17	0.00	-4.00	
5-B	189 - 204	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
10-B	205 - 220	F4	-0.50	15.0	16.0	42	17	0.00	-4.00	
11.8-B	221 - 238	F4	-0.50	15.0	16.0	42	17	0.00	-4.00	
12-B	239 - 250	F4	-0.50	15.0	16.0	42	17	0.00	-4.00	
14-B	251 - 270	F4	-0.50	15.0	16.0	42	16	0.00	-4.00	
3.2-C	271 - 278	F2	-0.50	12.0	12.0	36	17	0.00	-3.50	
3.8-C	279 - 288	F2	-0.50	12.0	12.0	36	17	0.00	-3.50	
4.5-C	289 - 298	F2	-0.50	12.0	12.0	36	17	0.00	-3.50	
5.2-C	299 - 306	F2	-0.50	12.0	12.0	36	17	0.00	-3.50	
9.8-C	307 - 313	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
10.5-C	314 - 322	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
11.2-C	323 - 331	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
11.8-C	332 - 338	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
1.2-D	339 - 342	F1	-0.50	10.0	10.0	30	12	0.00	-3.00	
13.8-D	343 - 346	F1	-0.50	10.0	10.0	30	12	0.00	-3.00	
1.2-E	347 - 351	F2	-0.50	12.0	12.0	36	17	0.00	-3.50	
1.8-E	352 - 363	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
2.5-E	364 - 375	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
3.2-E	376 - 384	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
5.8-E	385 - 391	F3	-0.50	14.0	14.0	39	15	0.00	-3.75	
6.5-E	392 - 400	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
7.2-E	401 - 409	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
7.8-E	410 - 418	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
8.5-E	419 - 427	F3	-0.50	14.0	14.0	39	17	0.00	-3.75	
9.2-E	428 - 434	F3	-0.50	14.0	14.0	39	15	0.00	-3.75	
11.8-E	435 - 443	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
12.5-E	444 - 455	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
13.2-E	456 - 467	F3	-0.50	14.0	14.0	39	16	0.00	-3.75	
13.8-E	468 - 472	F2	-0.50	12.0	12.0	36	17	0.00	-3.50	
1.2-F	473 - 476	F1	-0.50	10.0	10.0	30	12	0.00	-3.00	
1.8-F	477 - 480	F1	-0.50	10.0	10.0	30	12	0.00	-3.00	
13.2-F	481 - 484	F1	-0.50	10.0	10.0	30	12	0.00	-3.00	
13.8-F	485 - 488	F1	-0.50	10.0	10.0	30	12	0.00	-3.00	
1.8-G	489 - 494	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
2.5-G	495 - 500	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
3.2-G	501 - 505	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
3.8-G	506 - 509	F1	-0.50	10.0	10.0	30	14	0.00	-3.00	
4.5-G	510 - 513	F1	-0.50	10.0	10.0	30	14	0.00	-3.00	
5.2-G	514 - 517	F1	-0.50	10.0	10.0	30	14	0.00	-3.00	
5.8-G	518 - 522	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
6.5-G	523 - 528	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
7.2-G	529 - 534	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
7.8-G	535 - 540	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
8.5-G	541 - 546	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
9.2-G	547 - 551	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
9.8-G	552 - 555	F1	-0.50	10.0	10.0	30	14	0.00	-3.00	
10.5-G	556 - 559	F1	-0.50	10.0	10.0	30	14	0.00	-3.00	
11.2-G	560 - 563	F1	-0.50	10.0	10.0	30	14	0.00	-3.00	
11.8-G	564 - 568	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
12.5-G	569 - 574	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
13.2-G	575 - 580	F1	-0.50	10.0	10.0	30	16	0.00	-3.00	
NOT USED	581 - 599	NOT USED								
1.5/3-A.5/D.5	600 - 716	MAT	-0.50	45.0	62.5	72	12	0.00	-6.50	
5.5/9.5-A.5/D.5	717 - 1093	MAT	-0.50	62.5	1215.0	72	11	0.00	-6.50	
12/13.5-A.5/D.5	1094 - 1209	MAT	-0.50	45.0	62.5	72	12	0.00	-6.50	
3.5/5.5-D.5/E.5	1210 - 1329	MAT	-0.50	52.0	63.0	72	12	0.00	-6.50	
9.5/11.5-D.5/E.5	1330 - 1449	MAT	-0.50	52.0	63.0	72	12	0.00	-6.50	

NOTES:

(1) Structure Elevation 0.00 ft is equivalent to Site Civil Elevation/Finished Floor Elevation 248.00 ft.
(2) Top of footing elevation shall be confirmed prior to Geopier element installation.
(3) Structural Engineer shall verify footing dimensions prior to Geopier element installation.

1 GEOPIER SCHEDULE

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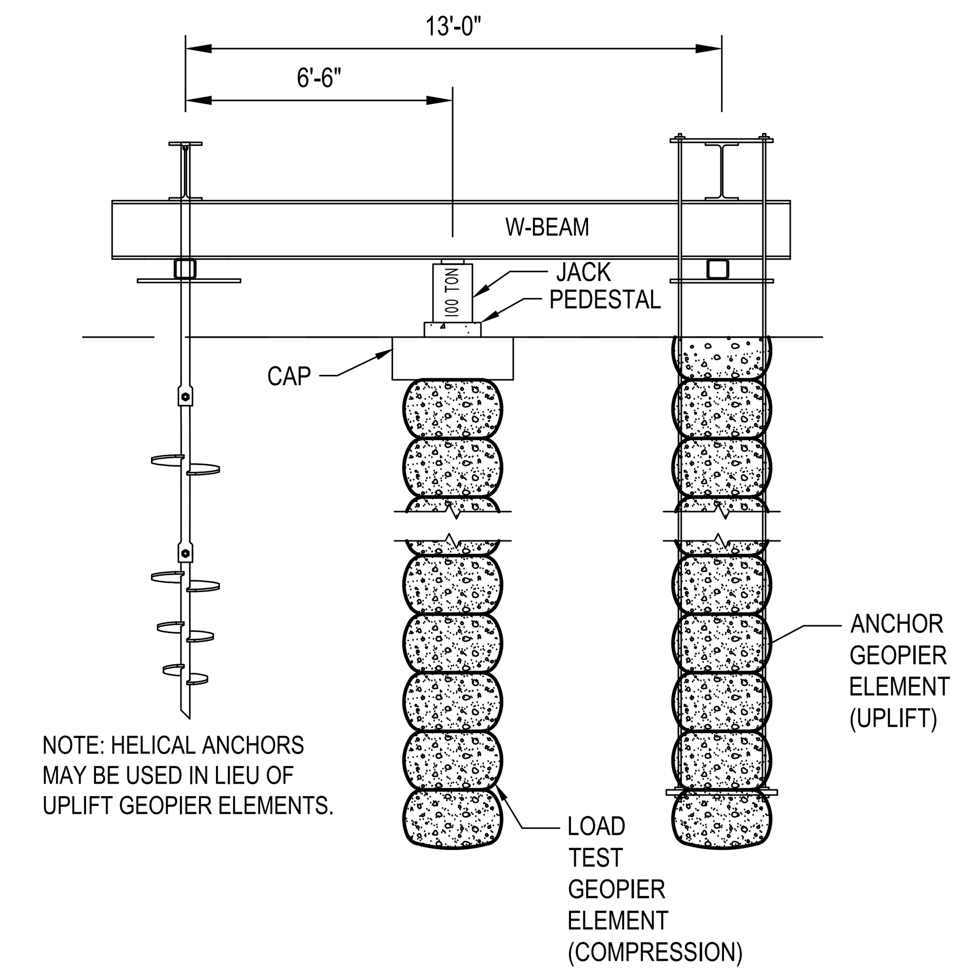


2 GEOPIER MODULUS TEST SETUP
NOT TO SCALE

Geopier Foundation Company®

GEOPIER®

Project Name: Ladbs - Alameda Crossing
Project Location: Los Angeles, Ca
Project Number: GLA-229 / NLA-126

Geopier® Modulus Test Schedule

Geopier Design Stress: 28,035 psf
Geopier Element Design Diameter: 20 in.
Geopier Design Modulus: 285 pci

Modulus Test Location: Near B-2 (see Figure 1)
Test Geopier Element Shaft Length: 17 ft
Concrete Cap Thickness: 3.5 ft
Total Geopier Element Depth: 20.5 ft

Load No.	Ram Load, (kips)	Percent of Design Stress	Minimum Duration	Maximum Duration	Remarks
	3.06	5.0%	N/A	N/A	seating load
1	10.20	16.7%	15 min	60 min	
2	20.39	33.3%	15 min	60 min	
3	30.58	50.0%	15 min	60 min	
4	40.78	66.7%	15 min	60 min	
5	50.97	83.3%	15 min	60 min	
6	61.16	100.0%	15 min	60 min	
7	71.34	116.6%	60 min	240 min	
8	81.55	133.3%	15 min	60 min	
9	91.75	150.0%	15 min	60 min	
10	122.33	200.0%	15 min	60 min	
11	61.16	100.0%	N/A	N/A	rebound, unload
12	40.37	66.0%	N/A	N/A	rebound, unload
13	20.18	33.0%	N/A	N/A	rebound, unload
14	3.06	5.0%	N/A	N/A	rebound, unload

Notes:
1 - The Geopier element to be used in the modulus load testing should be installed in a manner similar to production, at least 4 days prior to testing, so that pore-pressures have adequate time to dissipate.
2 - The modulus load test shall be performed to a stress not less than 200% of the design maximum top-of-pier stress indicated in the Geopier Design Calculations.
3 - The modulus load test Geopier element shall be installed to a depth of 20.5 feet below the ground surface with a 3.5-foot thick unreinforced concrete leveling pad.
4 - A telltale shall be installed in the bottom one-third of the tested Geopier element. Telltale deflections shall be monitored concurrent with top of Geopier deflections during the modulus load test.
5 - The modulus load test setup shall be as shown on Geopier Construction Drawing GP0.2. Helical anchors should be installed in accordance with manufacturers specifications.
6 - A representative of the owner's geotechnical consultant should be present to witness the load test.

3 GEOPIER MODULUS TEST SCHEDULE

ZONE	REV	DESCRIPTION	DATE	APPROVED

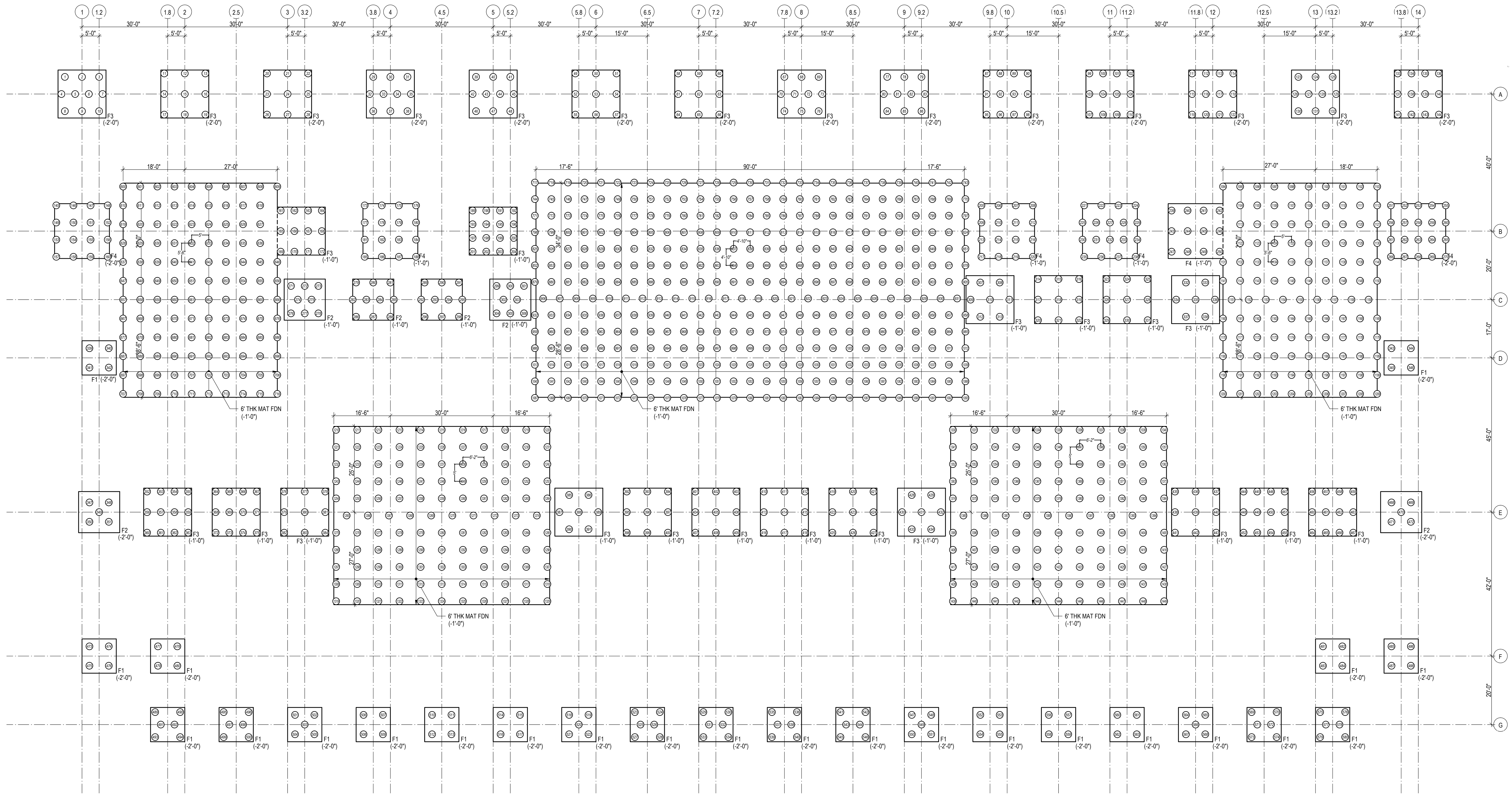
REVISIONS

GEOPIER® SCHEDULES

ALAMEDA CROSSING
1716 EAST 7TH STREET
LOS ANGELES, CA

WGI GEOPIER®

PROJECT NUMBER GLA-229 / NLA-126
DRAWN BY RB
CHECK BY KH
DATE 10/03/24
SHEET NUMBER GP0.3



NORTH

1 GEOPIER® LOCATION PLAN
1/16" = 1'-0"

LEGEND

○	TYP. 20" GEOPIER ELEMENT
①	TYP. GEOPIER ELEMENT NUMBER

GEOPIER LOCATION PLAN NOTES:

1. FOOTING CONCRETE SHALL BE PLACED DIRECTLY ON TOP OF EXPOSED GEOPIER ELEMENTS.
2. ALL EXISTING AND PROPOSED UTILITIES WITHIN AND ADJACENT TO THE PROPOSED BUILDING FOOTPRINT SHALL BE FIELD VERIFIED BY THE GENERAL CONTRACTOR AND COORDINATED WITH THE GEOPIER INSTALLER BEFORE GEOPIER ELEMENT INSTALLATION SHALL PROCEED.
3. THESE DRAWINGS ARE FOR GEOPIER LOCATION ONLY, AND ARE BASED ON THE STRUCTURAL DRAWINGS PROVIDED BY MAGNUSSON KLEMENCIC ASSOCIATES ON SHEET S2.00 DATED 08/20/24. REFER TO MAGNUSSON KLEMENCIC ASSOCIATES DRAWINGS FOR FOOTING LAYOUT AND ORIENTATION.
4. GEOPIER ELEMENTS SHALL BE LOCATED IN THE FIELD AS SHOWN, DIMENSIONED FROM CONTROL POINTS ESTABLISHED FROM STRUCTURAL AND/OR ARCHITECTURAL PLANS.

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ZONE	REV	DESCRIPTION	DATE	APPROVED
REVISIONS				

GEOPIER® LOCATION PLAN

REGISTERED PROFESSIONAL ENGINEER RYAN PAUL BULATAO GE 3021 GEOTECHNICAL STATE OF CALIFORNIA

ALAMEDA CROSSING
1716 EAST 7TH STREET
LOS ANGELES, CA

WGI GEOPIER®

PROJECT NUMBER	GLA-229 / NLA-126
DRAWN BY	RB
CHECK BY	KH
DATE	10/03/24
SHEET NUMBER	GP1.1

WGI GEOPIER®

Western Ground Improvement, Inc.
209 Avenida Del Mar
Suite 201B
San Clemente, CA 92672

Tel: (310) 717-3428
www.westerngroundimprovement.com

October 3, 2024

Jonathan Payne
Prologis
2141 Rosecrans Avenue, suite 1151
El Segundo, CA 90245

Re: Calculations Package for a *Geopier*® Foundation System for a Mat Foundation
Alameda Crossing
1716 East 7th Street
Los Angeles, CA 90021
GFC Project No.: GLA-229 / NLA-126

Dear Mr. Payne,

Western Ground Improvement, Inc. has completed the Geopier® foundation design for the above project. The design is based on geotechnical information provided by Southern California Geotechnical. in their report dated April 7, 2024. Structural design loads are as provided by Magnusson Klemencic Associates. The following documents are included herein:

- Geopier settlement calculations for square footings
- Geopier settlement calculations for rectangular footings
- Geopier bearing capacity calculation
- Design Parameters Calculations

We are pleased to have provided you with our design services. If you have any questions, please contact this office.

Sincerely,
Western Ground Improvement, Inc.

Ryan Bulatao, G.E., P.E.
Regional Manager

REGISTERED PROFESSIONAL ENGINEER
RYAN PAUL BULATAO
GE 3021
GEOTECHNICAL
STATE OF CALIFORNIA

GEOPIER® Foundation Company

GEOPIER®

Project Name/Location:	**LADBS - Alameda Crossing, Los Angeles, CA**
Project No.:	**GLA-229 / NLA-126**
Engineer:	**RB**
Date:	**10/2/2024**

SQUARE FOOTINGS

Version 3.0.6 August 2013

INPUT PARAMETER VALUES:

Parameter	Symb	Val.
Constructed RAP diameter (in)	d	20
Depth to groundwater (ft)	dgw	95
Total unit weight of soil (pcf)	g	120
Soil frict. angle (degr)	f	30
Max. hor. pressure (psf)	pmax	2500
From Table 4.2:		
Allowable bearing press. (ksf)	qall	8
RAP stiffn. modulus (pci)	kg	285
Soil stiffness modulus (pci)	km	22

TOP OF PIER STRESS - SQUARE FOOTINGS

Parameter	Symb	Equation	F1	F2	F3	F4			
Sustained column load (kips)	p		510	840	1280	1610			
Required footing width (ft)	Br	sqrt(p/qall)	7.98	10.25	12.65	14.19			
Selected footing width (ft)	B		10	12	14	16			
Sustained bearing pressure (ksf)	q	p/(B*B)	5.10	5.83	6.53	6.29			
Selected No. RAP elems	N		6	10	16	20			
Area replacement ratio	Ra	N*Ag/(B*B)	0.131	0.152	0.178	0.170			
Stiffness ratio	Rs	kg/km	13.0	13.0	13.0	13.0			
Stress at top of GP (ksf)	qg	q*Rs/(Rs*Ra-Ra+1)	25.76	26.88	27.04	26.82			
Load at top of GP (kips)	Qg	qg*Ag	56.2	58.6	59.0	58.5			

SHAFT LENGTH REQUIREMENTS

Parameter	Symb	Equation	F1	F2	F3	F4			
Depth of Embedment	Df		2.5	3.0	3.0	3.5			
Trial shaft length (ft)	Hs		16.0	17.0	17.0	16.0			
Drill depth (ft)	Hdrill	Df+Hs	19	20	20	20			
Frictional resistance force (kips)	Qs	fs*pi*d*Hs	110	120	120	114			
Allowable tensile resistance (kips)	Qsall	Qs/2	55	60	60	57			
Allowable end-bearing rest. (kips)	Qeb	Qeb	0	0	0	0			
Factor of Safety	FS	Qs/Qg	2.0	2.0	2.0	2.0			
Is shaft long enough?		Qs+Qeb>Pcdem?	**ok**	**ok**	**ok**	**ok**			

INPUT PARAMETER VALUES:

Upper Zone Elastic Parameters Parameter	Sym	Val
Pier Modulus Layer 1 (ksf)	Eg1	3900
Pier Modulus Layer 2 (ksf)	Eg2	3900
Pier Modulus Layer 3 (ksf)	Eg3	3900
Pier Modulus Layer 4 (ksf)	Eg4	3900
Pier Modulus Layer 5 (ksf)	Eg5	3900
Soil Modulus Layer 1 (ksf)	Em1	350
Soil Modulus Layer 2 (ksf)	Em2	350
Soil Modulus Layer 3 (ksf)	Em3	350
Soil Modulus Layer 4 (ksf)	Em4	350
Soil Modulus Layer 5 (ksf)	Em5	350

UPPER ZONE SETTLEMENT

Parameter	Symb	Equation							
UZ Settlement Approach		1-Stiffness, 2-Modulus	2	2	2	2	1	1	1
Thickness of UZ sublayer 1(ft)	H_{uz1}		3.5	3.0	3.0	2.5			
Thickness of UZ sublayer 2 (ft)	H_{uz2}		5.0	5.0	5.0	5.0			
Thickness of UZ sublayer 3 (ft)	H_{uz3}		7.0	7.0	7.0	7.0			
Thickness of UZ sublayer 4 (ft)	H_{uz4}		2.0	3.5	3.5	3.0			
Thickness of UZ sublayer 5 (ft)	H_{uz5}								
Total UZ Thickness OK?		Huz = Hs + d	**ok**	**ok**	**ok**	**ok**			
Composite Modulus Layer 1 (ksf)	E_{comp1}	Eg1Ra + Em1(1-Ra)	815	888	982	955			
Composite Modulus Layer 2 (ksf)	E_{comp2}	Eg2Ra + Em2(1-Ra)	815	888	982	955			
Composite Modulus Layer 3 (ksf)	E_{comp3}	Eg3Ra + Em3(1-Ra)	815	888	982	955			
Composite Modulus Layer 4 (ksf)	E_{comp4}	Eg4Ra + Em4(1-Ra)	815	888	982	955			
Composite Modulus Layer 5 (ksf)	E_{comp5}	Eg5Ra + Em5(1-Ra)	815	888	982	955			
Sett. of LZ sublayer 1 (in)	s_{uz1}	qg/kg or q*Iσ-vag*H/Ecomp	0.25	0.23	0.24	0.20			
Sett. of LZ sublayer 2 (in)	s_{uz2}	$q^*I\sigma\text{-}2^*H_{uz2}/E_{comp2}$	0.23	0.29	0.32	0.35			
Sett. of LZ sublayer 3 (in)	s_{uz3}	$q^*I\sigma\text{-}3^*H_{uz3}/E_{comp3}$	0.14	0.20	0.24	0.30			
Sett. of LZ sublayer 4 (in)	s_{uz4}	$q^*I\sigma\text{-}4^*H_{uz4}/E_{comp4}$	0.02	0.06	0.07	0.08			
Sett. of LZ sublayer 5 (in)	s_{uz5}	$q^*I\sigma\text{-}5^*H_{uz5}/E_{comp5}$	0.00	0.00	0.00	0.00			
Total Upper Zone Settlement (in)	s_{uz}	$s_{uz1}+s_{uz2}+s_{uz3}+s_{uz4}+s_{uz5}$	0.64	0.78	0.88	0.92			

INPUT PARAMETER VALUES:

Parameter	Symb	Val.
Allowable end-bearing (kips)	Qeb	0.0
E or c_ε for LZ sublyr 1	E_1 / $c_{\varepsilon1}$	1250
E or c_ε for LZ sublyr 2	E_2 / $c_{\varepsilon2}$	1250
E or c_ε for LZ sublyr 3	E_3 / $c_{\varepsilon3}$	2000
E or c_ε for LZ sublyr 4	E_4 / $c_{\varepsilon4}$	2000
E or c_ε for LZ sublyr 5	E_5 / $c_{\varepsilon5}$	0
Calc. settlement to X*B	X	2

LOWER ZONE SETTLEMENTS

Parameter	Symb	Equation	F1	F2	F3	F4			
Dpth to bottm of LZ from ftg (ft)	X*B	X*B	20	24	28	32			
Upper zone thickness (ft)	H_{uz}	Hs+d	17.7	18.7	18.7	17.7			
Lower zone thickness (ft)	H_{lz}	H2b-Hlz	2.4	5.4	9.4	14.4			
Thickness of LZ sublayer 1(ft)	H_{lz1}								
Thickness of LZ sublayer 2 (ft)	H_{lz2}		2.4	1.3	1.3	1.8			
Thickness of LZ sublayer 3 (ft)	H_{lz3}			4.1	5.0	5.0			
Thickness of LZ sublayer 4 (ft)	H_{lz4}				3.1	7.6			
Thickness of LZ sublayer 5 (ft)	H_{lz5}								
Total LZ thickness ok?			**ok**	**ok**	**ok**	**ok**			
E or c_ε for LZ sublyr 1	E_1 / $c_{\varepsilon1}$	E (ksf) or c_ε	1250	1250	1250	1250			
E or c_ε for LZ sublyr 2	E_2 / $c_{\varepsilon2}$	E (ksf) or c_ε	1250	1250	1250	1250			
E or c_ε for LZ sublyr 3	E_3 / $c_{\varepsilon3}$	E (ksf) or c_ε	2000	2000	2000	2000			
E or c_ε for LZ sublyr 4	E_4 / $c_{\varepsilon4}$	E (ksf) or c_ε	2000	2000	2000	2000			
E or c_ε for LZ sublyr 5	E_5 / $c_{\varepsilon5}$	E (ksf) or c_ε	0	0	0	0			
Initial stress for sublyr 1 (ksf)	P'_{o1}		2.420	2.600	2.600	2.540			
Initial stress for sublyr 2 (ksf)	P'_{o2}		2.564	2.680	2.680	2.650			
Initial stress for sublyr 3 (ksf)	P'_{o3}		2.708	3.004	3.060	3.060			
Initial stress for sublyr 4 (ksf)	P'_{o4}		2.708	3.248	3.544	3.814			
Initial stress for sublyr 5 (ksf)	P'_{o5}		2.708	3.248	3.728	4.268			
Ftg stress on sublyr 1 (ksf)	ΔP1	q*I	0.69	0.98	1.43	1.84			
Ftg stress on sublyr 2 (ksf)	ΔP2	q*I	0.61	0.93	1.34	1.70			
Ftg stress on sublyr 3 (ksf)	ΔP3	q*I	0.55	0.74	1.04	1.30			
Ftg stress on sublyr 4 (ksf)	ΔP4	q*I	0.55	0.63	0.78	0.85			
Ftg stress on sublyr 5 (ksf)	ΔP5	q*I	0.55	0.63	0.71	0.68			
Sett. of LZ sublayer 1 (in)	s_{lz1}	DP1*Hlz1/E1	0.00	0.00	0.00	0.00			
Sett. of LZ sublayer 2 (in)	s_{lz2}	DP2*Hlz2/E2	0.01	0.01	0.02	0.03			
Sett. of LZ sublayer 3 (in)	s_{lz3}	DP3*Hlz3/E3	0.00	0.02	0.03	0.04			
Sett. of LZ sublayer 4 (in)	s_{lz4}	DP4*Hlz4/E4	0.00	0.00	0.01	0.04			
Sett. of LZ sublayer 5 (in)	s_{lz5}	ce5*Hlz5*log((Po5+DP5)/Po5)	0.00	0.00	0.00	0.00			
Total lower zone sett. (in)	s_{lz}	$s_{lz1}+s_{lz2}+s_{lz3}+s_{lz4}+s_{lz5}$	0.0	0.0	0.1	0.1			
Total UZ + LZ settlement (in)	s		0.7	0.8	0.9	1.0			

GEOPIER® **Foundation Company**

Project Name/Location: **LADBS - Alameda Crossing, Los Angeles, CA**
Project No.: **GLA-229 / NLA-126**
Engineer: **RB**
Date: **10/2/2024**

RECTANGULAR FOOTINGS

Version 3.0.6 August 2013

INPUT PARAMETER VALUES:

Parameter	Symb	Val.
Constructed RAP diameter (in)	d	20
Depth to groundwater (ft)	dgw	95
Total unit weight of soil (pcf)	g	120
Soil frict. angle (degr)	f	30
Max. hor. pressure (psf)	pmax	2500
From Table 4.2:		
Allowable bearing press. (ksf)	qall	8
RAP stiffn. modulus (pci)	kg	285
Soil stiffness modulus (pci)	km	22

TOP OF PIER STRESS - RECTANGULAR FOOTINGS

Parameter	Symb	Equation							
Sustained column load (kips)	p		9400	25700	10250				
Selected footing width (ft)	B		45.00	62.50	52.00				
Required footing length (ft)	Lr		26.11	51.40	24.64				
Selected footing length (ft)	L		62.50	125.00	63.00				
Sustained bearing pressure (ksf)	q	p/(B*L)	3.34	3.29	3.13				
Selected No. RAP elems	N		120	380	120				
Area replacement ratio	Ra	N*Ag/(B*L)	0.093	0.106	0.080				
Stiffness ratio	Rs	kg/km	13.0	13.0	13.0				
Stress at top of GP (ksf)	qg	q*Rs/(Rs*Ra-Ra+1)	20.49	18.79	20.73				
Load at top of GP (kips)	Qg	qg*Ag	44.7	41.0	45.2				

SHAFT LENGTH REQUIREMENTS

Depth of Embedment	Df		6.0	6.0	6.0				
Trial shaft length (ft)	Hs		12.0	11.0	12.0				
Drill depth (ft)	Hdrill	Df+Hs	18	17	18				
Frictional resistance force (kips)	Qs	fs*pi*d*Hs	90	83	90				
Allowable tensile resistance (kips)	Qsall	Qs/2	45	41	45				
Allowable end-bearing rest. (kips)	Qeb	Qeb	0	0	0				
Factor of Safety	FS	Qs/Qg	2.0	2.0	2.0				
Is shaft long enough?		Qs+Qeb>Pcdem?	**ok**	**ok**	**ok**				

INPUT PARAMETER VALUES:

Upper Zone Elastic Parameters		
Parameter	Sym	Val
Pier Modulus Layer 1 (ksf)	Eg1	3900
Pier Modulus Layer 2 (ksf)	Eg2	3900
Pier Modulus Layer 3 (ksf)	Eg3	3900
Pier Modulus Layer 4 (ksf)	Eg4	3900
Pier Modulus Layer 5 (ksf)	Eg5	3900
Soil Modulus Layer 1 (ksf)	Em1	350
Soil Modulus Layer 2 (ksf)	Em2	350
Soil Modulus Layer 3 (ksf)	Em3	350
Soil Modulus Layer 4 (ksf)	Em4	350
Soil Modulus Layer 5 (ksf)	Em5	350

UPPER ZONE SETTLEMENT

Parameter	Symb	Equation							
UZ Settlement Approach		1-Stiffness, 2-Modulus	2	2	2	1	1	1	1
Thickness of UZ sublayer 1(ft)	H_{uz1}								
Thickness of UZ sublayer 2 (ft)	H_{uz2}		5	5	5				
Thickness of UZ sublayer 3 (ft)	H_{uz3}		7	7	7				
Thickness of UZ sublayer 4 (ft)	H_{uz4}		2	1	2				
Thickness of UZ sublayer 5 (ft)	H_{uz5}								
Total UZ Thickness OK?		Huz = Hs +d	**ok**	**ok**	**ok**				
Composite Modulus Layer 1 (ksf)	E_{comp1}	Eg1Ra + Em1(1-Ra)	680	727	634				
Composite Modulus Layer 2 (ksf)	E_{comp2}	Eg2Ra + Em2(1-Ra)	680	727	634				
Composite Modulus Layer 3 (ksf)	E_{comp3}	Eg3Ra + Em3(1-Ra)	680	727	634				
Composite Modulus Layer 4 (ksf)	E_{comp4}	Eg4Ra + Em4(1-Ra)	680	727	634				
Composite Modulus Layer 5 (ksf)	E_{comp5}	Eg5Ra + Em5(1-Ra)	680	727	634				
Sett. of UZ sublayer 1 (in)	s_{uz1}	qg/kg or q*Iσ-vag*H/Ecomp	0.00	0.00	0.00				
Sett. of UZ sublayer 2 (in)	s_{uz2}	q*Iσ-2*H_{uz2}/E_{comp2}	0.29	0.27	0.30				
Sett. of UZ sublayer 3 (in)	s_{uz3}	q*Iσ-3*H_{uz3}/E_{comp3}	0.40	0.38	0.41				
Sett. of UZ sublayer 4 (in)	s_{uz4}	q*Iσ-4*H_{uz4}/E_{comp4}	0.08	0.03	0.08				
Sett. of UZ sublayer 5 (in)	s_{uz5}	q*Iσ-5*H_{uz5}/E_{comp5}	0.00	0.00	0.00				
Total Upper Zone Settlement (in)	s_{uz}	$s_{uz1}+s_{uz2}+s_{uz3}+s_{uz4}+s_{uz5}$	0.78	0.68	0.79				

INPUT PARAMETER VALUES:

Parameter	Symb	Val.
Allowable end-bearing (kips)	Qeb	0.0
E or c_ε for LZ sublyr 1	$E_1 / c_{\varepsilon 1}$	1250
E or c_ε for LZ sublyr 2	$E_2 / c_{\varepsilon 2}$	1250
E or c_ε for LZ sublyr 3	$E_3 / c_{\varepsilon 3}$	2000
E or c_ε for LZ sublyr 4	$E_4 / c_{\varepsilon 4}$	2000
E or c_ε for LZ sublyr 5	$E_5 / c_{\varepsilon 5}$	0
Calc. settlement to X*B	X	2

LOWER ZONE SETTLEMENTS

Parameter	Symb	Equation	0	0	0				
Dpth to bottm of LZ from ftg (ft)	X*B	X*Beq	106.1	176.8	114.5				
Upper zone thickness (ft)	H_{uz}	Hs+d	13.7	12.7	13.7				
Lower zone thickness (ft)	H_{lz}	H2b-Hlz	20	20	20				
Thickness of LZ sublayer 1(ft)	H_{lz1}								
Thickness of LZ sublayer 2 (ft)	H_{lz2}		3	4	3				
Thickness of LZ sublayer 3 (ft)	H_{lz3}		5	5	5				
Thickness of LZ sublayer 4 (ft)	H_{lz4}		10	10	10				
Thickness of LZ sublayer 5 (ft)	H_{lz5}		2	1	2				
Total thickness ok?			**ok**	**ok**	**ok**				
E or c_ε for LZ sublyr 1	$E_1 / c_{\varepsilon 1}$	E (ksf) or c_ε	1250	1250	1250				
E or c_ε for LZ sublyr 2	$E_2 / c_{\varepsilon 2}$	E (ksf) or c_ε	1250	1250	1250				
E or c_ε for LZ sublyr 3	$E_3 / c_{\varepsilon 3}$	E (ksf) or c_ε	2000	2000	2000				
E or c_ε for LZ sublyr 4	$E_4 / c_{\varepsilon 4}$	E (ksf) or c_ε	2000	2000	2000				
E or c_ε for LZ sublyr 5	$E_5 / c_{\varepsilon 5}$	E (ksf) or c_ε	0	0	0				
Initial stress for sublyr 1 (ksf)	P'_{o1}		2.360	2.240	2.360				
Initial stress for sublyr 2 (ksf)	P'_{o2}		2.560	2.500	2.560				
Initial stress for sublyr 3 (ksf)	P'_{o3}		3.060	3.060	3.060				
Initial stress for sublyr 4 (ksf)	P'_{o4}		3.960	3.960	3.960				
Initial stress for sublyr 5 (ksf)	P'_{o5}		4.660	4.600	4.660				
Ftg stress on sublyr 1 (ksf)	ΔP1	q*I	3.10	3.24	2.94				
Ftg stress on sublyr 2 (ksf)	ΔP2	q*I	3.02	3.21	2.88				
Ftg stress on sublyr 3 (ksf)	ΔP3	q*I	2.79	3.12	2.68				
Ftg stress on sublyr 4 (ksf)	ΔP4	q*I	2.33	2.92	2.29				
Ftg stress on sublyr 5 (ksf)	ΔP5	q*I	1.98	2.74	1.98				
Sett. of LZ sublayer 1 (in)	s_{lz1}	DP1*Hlz1/E1	0.00	0.00	0.00				
Sett. of LZ sublayer 2 (in)	s_{lz2}	DP2*Hlz2/E2	0.10	0.13	0.09				
Sett. of LZ sublayer 3 (in)	s_{lz3}	DP3*Hlz3/E3	0.08	0.09	0.08				
Sett. of LZ sublayer 4 (in)	s_{lz4}	DP4*Hlz4/E4	0.14	0.18	0.14				
Sett. of LZ sublayer 5 (in)	s_{lz5}	ce5*Hlz5*log((Po5+DP5)/Po5)	0.00	0.00	0.00				
Total lower zone sett. (in)	s_{lz}	$s_{lz1}+s_{lz2}+s_{lz3}+s_{lz4}+s_{lz5}$	0.32	0.40	0.31				
Total UZ + LZ settlement (in)	s		1.1	1.1	1.1				

STATIC BEARING CAPACITY

GEOPIER®

Project: LADBS - Alameda Crossing
No. GLA-229 / NLA-126
Engr: RB
Date: 10/2/2024

Footing F1

Parameter Values:

Foundation Data and RAP Geometry:						
Footing Contact Pressure	q =	5100	(psf)	Total Column Load, P (kips)	=	510
Footing Length	L =	10	(ft)	Footing Area (sq. ft)	=	100
Footing Width	B =	10	(ft)	Total Pier Area (sq. ft)	=	13.08
Footing Depth	D_f =	2.5	(ft)	Area Ratio (granular layers)	=	0.131
Equivalent Width	B_{eq} =	10.0	(ft)	Stress Applied to Piers (granular) (psf)	=	25,767
Pier diameter	d =	20	(in)	Stress Applied to Matrix Soil (psf)	=	1,989
Number of Piers	N =	6		Relative Stiffness Ratio	=	13
Pier Modulus	k_g =	285	(pci)	Individual Pier Load (kips)	=	56.19
Depth to GWL Below Finish Floor	=	95	(ft)			

Matrix Soil Data:				
Upper Zone:				
UZ Soil Modulus	k_m	=	22	(pci)
Undrained Strength	S_u	=	0	(psf)
Cohesion	c	=	0	(psf)
Friction Angle	Φ_s	=	30	(degrees)
Total Unit Weight:				
Above D_f	γ_2	=	120	(pcf)
Below D_f	γ_1	=	120	(pcf)

RAP Design Parameter Values:					
Shaft Drill Depth		=	16	(ft.)	
Effective Shaft	$H_{eff.}$	=	17.7	(ft.)	*** effective pier length for soil bearing capacity and settlement calculations*
Friction Angle	Φ_{gp}	=	45	(degrees)	
Unit weight:					
Total	γ_{gp}	=	120	(pcf)	*Conservative*

STATIC BEARING CAPACITY

GEOPIER®

Global Shear (Upper Zone):

Shearing Within The Geopier-Reinforced Soil Matrix:

Composite Soil Strength Parameters:

R_a Reduction Factor =	0.4		
Soil Stress Concentration Factor =	2.5		*(Reduced R_s to account for vert. stress decrease with depth)*
Effective R_s =	0.05		
$\Phi_{comp.}$ =	32	degrees	
$C_{comp.}$ =	0	psf	*(based on value entered for C)*

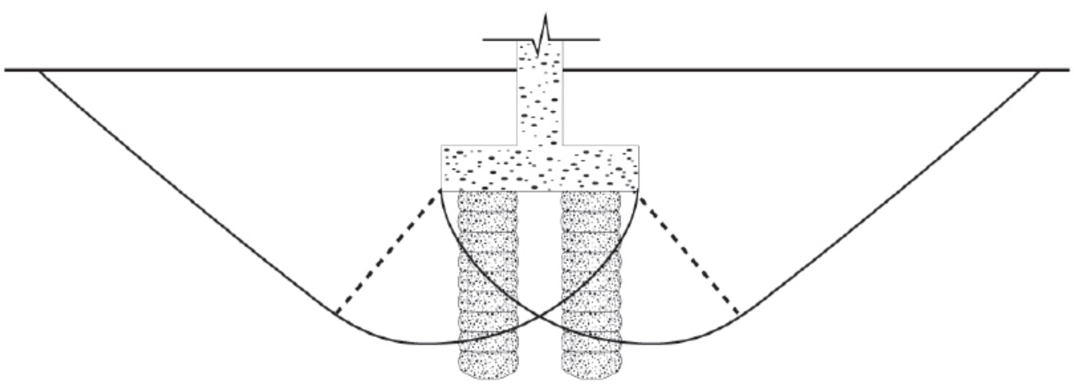
$$q_{ult.} = k_1(C_{comp.}N_c) + k_2(\gamma_1 B N_\gamma) + \gamma_2 D_f N_q$$

where:

k_1 =	1.3	k_1 = 1.3 for square and rectangular footings; and 1.0 for continuous footings
k_2 =	0.5	k_2 = 0.5 for square, rectangular and continuous footings
N_c =	47	(Terzaghi General Shear Factors)
N_γ =	31	
N_q =	32	

$q_{ult.}$ =	$k_1(C_{comp.}N_c)$ +	$k_2(\gamma_1 B N_\gamma)$ +	$\gamma_2 D_f N_q$
$q_{ult.}$ =	-	18,600	9,600
$q_{ult.}$ =	28,200 psf		

For Design Footing Stress = 5100 psf **FS = 5.5**



STATIC BEARING CAPACITY

GEOPIER®

Project:	LADBS - Alameda Crossing
No.	GLA-229 / NLA-126
Engr:	RB
Date:	10/2/2024

Footing F2

Parameter Values:

Foundation Data and RAP Geometry:

Footing Contact Pressure	q =	5830	(psf)	Total Column Load, P (kips)	=	840
Footing Length	L =	12	(ft)	Footing Area (sq. ft)	=	144
Footing Width	B =	12	(ft)	Total Pier Area (sq. ft)	=	21.81
Footing Depth	D_f =	2.5	(ft)	Area Ratio (granular layers)	=	0.151
Equivalent Width	B_{eq} =	12.0	(ft)	Stress Applied to Piers (granular) (psf)	=	26,875
Pier diameter	d =	20	(in)	Stress Applied to Matrix Soil (psf)	=	2,075
Number of Piers	N =	10		Relative Stiffness Ratio	=	13
Pier Modulus	k_g =	285	(pci)	Individual Pier Load (kips)	=	58.60
Depth to GWL Below Finish Floor	=	95	(ft)			

Matrix Soil Data:

Upper Zone:

UZ Soil Modulus	k_m	=	22	(pci)
Undrained Strength	S_u	=	0	(psf)
Cohesion	c	=	0	(psf)
Friction Angle	Φ_s	=	30	(degrees)
Total Unit Weight:				
Above D_f	γ_2	=	120	(pcf)
Below D_f	γ_1	=	120	(pcf)

RAP Design Parameter Values:

Shaft Drill Depth		=	17	(ft.)	
Effective Shaft	$H_{eff.}$	=	18.7	(ft.)	*** effective pier length for soil bearing capacity and settlement calculations*
Friction Angle	Φ_{gp}	=	45	(degrees)	
Unit weight:					
Total	γ_{gp}	=	120	(pcf)	*Conservative*

STATIC BEARING CAPACITY

GEOPIER®

Global Shear (Upper Zone):

Shearing Within The Geopier-Reinforced Soil Matrix:

Composite Soil Strength Parameters:

R_a Reduction Factor =	0.4	
Soil Stress Concentration Factor =	2.5	*(Reduced R_s to account for vert. stress decrease with depth)*
Effective R_s =	0.06	
$\Phi_{comp.}$ =	33 degrees	
$C_{comp.}$ =	0 psf	*(based on value entered for C)*

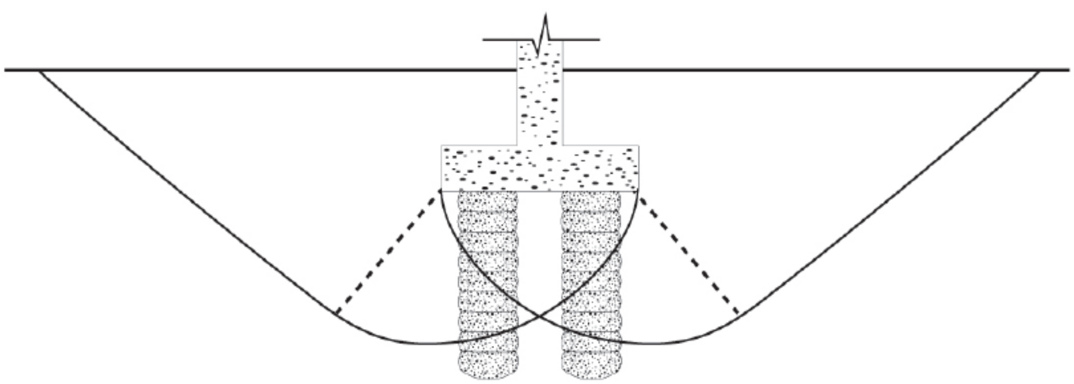
$$q_{ult.} = k_1(C_{comp.}N_c) + k_2(\gamma_1 BN_\gamma) + \gamma_2 D_f N_q$$

where:

k_1 =	1.3	k_1 = 1.3 for square and rectangular footings; and 1.0 for continuous footings
k_2 =	0.5	k_2 = 0.5 for square, rectangular and continuous footings
N_c =	47	(Terzaghi General Shear Factors)
N_γ =	31	
N_q =	32	

$q_{ult.}$ =	$k_1(C_{comp.}N_c)$ +	$k_2(\gamma_1 BN_\gamma)$ +	$\gamma_2 D_f N_q$
$q_{ult.}$ =	-	22,320	9,600
$q_{ult.}$ =	31,920 psf		

For Design Footing Stress = 5830 psf **FS = 5.5**



STATIC BEARING CAPACITY

GEOPIER®

Project: LADBS - Alameda Crossing
No. GLA-229 / NLA-126
Engr: RB
Date: 10/2/2024

Footing F3

Parameter Values:

Foundation Data and RAP Geometry:						
Footing Contact Pressure	q =	6530 (psf)	Total Column Load, P (kips)	=	1,280
Footing Length	L =	14 (ft)	Footing Area (sq. ft)	=	196
Footing Width	B =	14 (ft)	Total Pier Area (sq. ft)	=	34.89
Footing Depth	D_f =	2.5 (ft)	Area Ratio (granular layers)	=	0.178
Equivalent Width	B_{eq} =	14.0 (ft)	Stress Applied to Piers (granular) (psf)	=	27,044
Pier diameter	d =	20 (in)	Stress Applied to Matrix Soil (psf)	=	2,088
Number of Piers	N =	16	Relative Stiffness Ratio	=	13
Pier Modulus	k_g =	285 (pci)	Individual Pier Load (kips)	=	58.97
Depth to GWL Below Finish Floor	=	95 (ft)			

Matrix Soil Data:			
Upper Zone:			
UZ Soil Modulus	k_m	=	22 (pci)
Undrained Strength	S_u	=	0 (psf)
Cohesion	c	=	0 (psf)
Friction Angle	Φ_s	=	30 (degrees)
Total Unit Weight:			
Above D_f	γ_2	=	120 (pcf)
Below D_f	γ_1	=	120 (pcf)

RAP Design Parameter Values:				
Shaft Drill Depth		=	17 (ft.)	
Effective Shaft	$H_{eff.}$	=	18.7 (ft.)	*** effective pier length for soil bearing capacity and settlement calculations*
Friction Angle	Φ_{gp}	=	45 (degrees)	
Unit weight:				
Total	γ_{gp}	=	120 (pcf)	*Conservative*

STATIC BEARING CAPACITY

Global Shear (Upper Zone):

Shearing Within The Geopier-Reinforced Soil Matrix:

Composite Soil Strength Parameters:

R_a Reduction Factor =	0.4		
Soil Stress Concentration Factor =	2.5		*(Reduced R_s to account for vert. stress decrease with depth)*
Effective R_s =	0.07		
$\Phi_{comp.}$ =	33	degrees	
$C_{comp.}$ =	0	psf	*(based on value entered for C)*

$$q_{ult.} = k_1(C_{comp.}N_c) + k_2(\gamma_1 BN_\gamma) + \gamma_2 D_f N_q$$

where:

k_1 = 1.3 — k_1 = 1.3 for square and rectangular footings; and 1.0 for continuous footings

k_2 = 0.5 — k_2 = 0.5 for square, rectangular and continuous footings

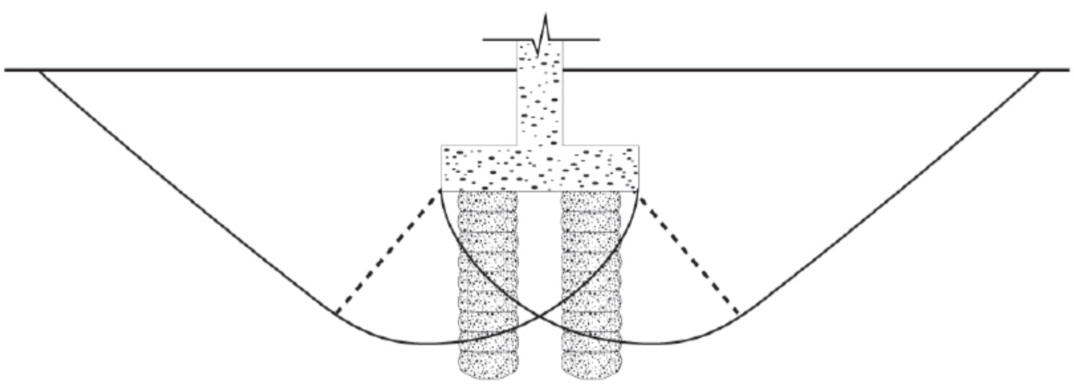
$$\left.\begin{array}{l} N_c = 47 \\ N_\gamma = 31 \\ N_q = 32 \end{array}\right\} \text{(Terzaghi General Shear Factors)}$$

$q_{ult.}$ = $k_1(C_{comp.}N_c)$ + $k_2(\gamma_1 BN_\gamma)$ + $\gamma_2 D_f N_q$

$q_{ult.}$ = - 26,040 9,600

$q_{ult.}$ = 35,640 psf

For Design Footing Stress = 6530 psf **FS = 5.5**



STATIC BEARING CAPACITY

GEOPIER®

Project: LADBS - Alameda Crossing
No. GLA-229 / NLA-126
Engr: RB
Date: 10/2/2024

Footing F4

Parameter Values:

Foundation Data and RAP Geometry:						
Footing Contact Pressure	q =	6290	(psf)	Total Column Load, P (kips)	=	1,610
Footing Length	L =	16	(ft)	Footing Area (sq. ft)	=	256
Footing Width	B =	16	(ft)	Total Pier Area (sq. ft)	=	43.61
Footing Depth	D_f =	2.5	(ft)	Area Ratio (granular layers)	=	0.170
Equivalent Width	B_{eq} =	16.0	(ft)	Stress Applied to Piers (granular) (psf)	=	26,835
Pier diameter	d =	20	(in)	Stress Applied to Matrix Soil (psf)	=	2,071
Number of Piers	N =	20		Relative Stiffness Ratio	=	13
Pier Modulus	k_g =	285	(pci)	Individual Pier Load (kips)	=	58.51
Depth to GWL Below Finish Floor	=	95	(ft)			

Matrix Soil Data:				
Upper Zone:				
UZ Soil Modulus	k_m	=	22	(pci)
Undrained Strength	S_u	=	0	(psf)
Cohesion	c	=	0	(psf)
Friction Angle	Φ_s	=	30	(degrees)
Total Unit Weight:				
Above D_f	γ_2	=	120	(pcf)
Below D_f	γ_1	=	120	(pcf)

RAP Design Parameter Values:					
Shaft Drill Depth		=	16	(ft.)	
Effective Shaft	$H_{eff.}$	=	17.7	(ft.)	*** effective pier length for soil bearing capacity and settlement calculations*
Friction Angle	Φ_{gp}	=	45	(degrees)	
Unit weight:					
Total	γ_{gp}	=	120	(pcf)	*Conservative*

STATIC BEARING CAPACITY

Global Shear (Upper Zone):

Shearing Within The Geopier-Reinforced Soil Matrix:

Composite Soil Strength Parameters:

R_a Reduction Factor =	0.4	
Soil Stress Concentration Factor =	2.5	*(Reduced R_s to account for vert. stress decrease with depth)*
Effective R_s =	0.07	
$\Phi_{comp.}$ =	33 degrees	
$C_{comp.}$ =	0 psf	*(based on value entered for C)*

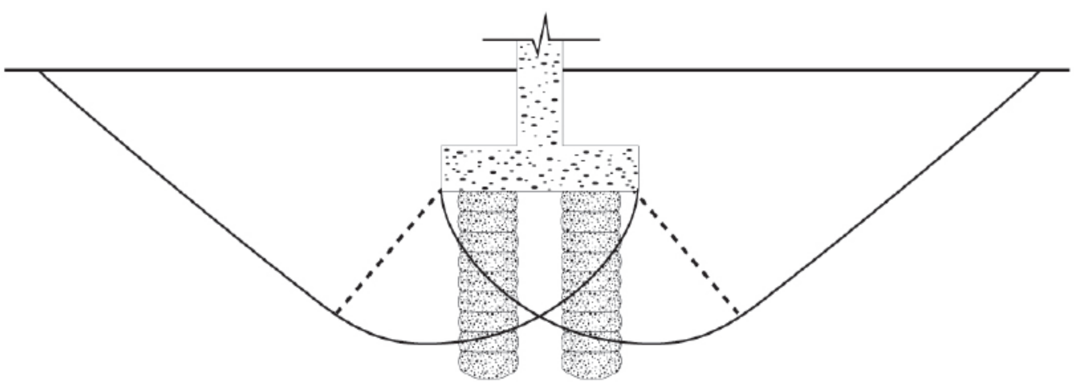
$$q_{ult.} = k_1(C_{comp.}N_c) + k_2(\gamma_1 B N_\gamma) + \gamma_2 D_f N_q$$

where:

k_1 =	1.3	k_1 = 1.3 for square and rectangular footings; and 1.0 for continuous footings
k_2 =	0.5	k_2 = 0.5 for square, rectangular and continuous footings
N_c =	47	(Terzaghi General Shear Factors)
N_γ =	31	
N_q =	32	

$q_{ult.}$ =	$k_1(C_{comp.}N_c)$ +	$k_2(\gamma_1 B N_\gamma)$ +	$\gamma_2 D_f N_q$
$q_{ult.}$ =	-	29,760	9,600
$q_{ult.}$ =	39,360 psf		

For Design Footing Stress = 6290 psf **FS = 6.3**



STATIC BEARING CAPACITY

GEOPIER®

Project: LADBS - Alameda Crossing
No. GLA-229 / NLA-126
Engr: RB
Date: 10/2/2024

Large Footing 1

Parameter Values:

Foundation Data and RAP Geometry:						
Footing Contact Pressure	q =	3340	(psf)	Total Column Load, P (kips)	=	9,394
Footing Length	L =	62.5	(ft)	Footing Area (sq. ft)	=	2,813
Footing Width	B =	45	(ft)	Total Pier Area (sq. ft)	=	261.67
Footing Depth	D_f =	6	(ft)	Area Ratio (granular layers)	=	0.093
Equivalent Width	B_{eq} =	53.0	(ft)	Stress Applied to Piers (granular) (psf)	=	20,485
Pier diameter	d =	20	(in)	Stress Applied to Matrix Soil (psf)	=	1,581
Number of Piers	N =	120		Relative Stiffness Ratio	=	13
Pier Modulus	k_g =	285	(pci)	Individual Pier Load (kips)	=	44.67
Depth to GWL Below Finish Floor	=	95	(ft)			

Matrix Soil Data:				
Upper Zone:				
UZ Soil Modulus	k_m	=	22	(pci)
Undrained Strength	S_u	=	0	(psf)
Cohesion	c	=	0	(psf)
Friction Angle	Φ_s	=	30	(degrees)
Total Unit Weight:				
Above D_f	γ_2	=	120	(pcf)
Below D_f	γ_1	=	120	(pcf)

RAP Design Parameter Values:					
Shaft Drill Depth		=	12	(ft.)	
Effective Shaft	$H_{eff.}$	=	13.7	(ft.)	*** effective pier length for soil bearing capacity and settlement calculations*
Friction Angle	Φ_{gp}	=	45	(degrees)	
Unit weight:					
Total	γ_{gp}	=	120	(pcf)	*Conservative*

STATIC BEARING CAPACITY

GEOPIER®

Global Shear (Upper Zone):

Shearing Within The Geopier-Reinforced Soil Matrix:

Composite Soil Strength Parameters:

R_a Reduction Factor =	0.4		
Soil Stress Concentration Factor =	2.5		*(Reduced R_s to account for vert. stress decrease with depth)*
Effective R_s =	0.04		
$\Phi_{comp.}$ =	32	degrees	
$C_{comp.}$ =	0	psf	*(based on value entered for C)*

$$q_{ult.} = k_1(C_{comp.}N_c) + k_2(\gamma_1 B N_\gamma) + \gamma_2 D_f N_q$$

where:

k_1 = 1.3 — k_1 = 1.3 for square and rectangular footings; and 1.0 for continuous footings

k_2 = 0.5 — k_2 = 0.5 for square, rectangular and continuous footings

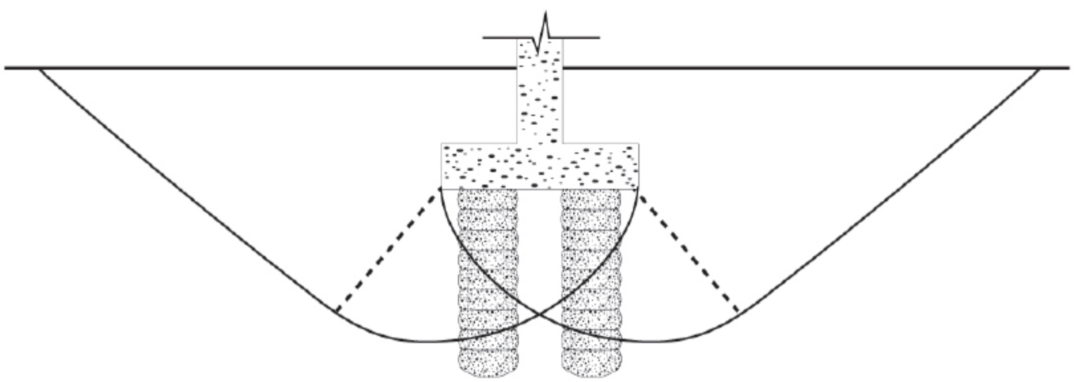
$$\left.\begin{aligned} N_c &= 47 \\ N_\gamma &= 31 \\ N_q &= 32 \end{aligned}\right\} \text{(Terzaghi General Shear Factors)}$$

$q_{ult.}$ = $k_1(C_{comp.}N_c)$ + $k_2(\gamma_1 B N_\gamma)$ + $\gamma_2 D_f N_q$

$q_{ult.}$ = - 83,700 23,040

$q_{ult.}$ = 106,740 psf

For Design Footing Stress = 3340 psf **FS = 32.0**



STATIC BEARING CAPACITY

GEOPIER®

Project: LADBS - Alameda Crossing
No. GLA-229 / NLA-126
Engr: RB
Date: 10/2/2024

Large Footing 2

Parameter Values:

Foundation Data and RAP Geometry:						
Footing Contact Pressure	q =	3290 (psf)	Total Column Load, P (kips)	=	25,703
Footing Length	L =	125 (ft)	Footing Area (sq. ft)	=	7,813
Footing Width	B =	62.5 (ft)	Total Pier Area (sq. ft)	=	828.61
Footing Depth	D_f =	6 (ft)	Area Ratio (granular layers)	=	0.106
Equivalent Width	B_{eq} =	88.4 (ft)	Stress Applied to Piers (granular) (psf)	=	18,793
Pier diameter	d =	20 (in)	Stress Applied to Matrix Soil (psf)	=	1,451
Number of Piers	N =	380	Relative Stiffness Ratio	=	13
Pier Modulus	k_g =	285 (pci)	Individual Pier Load (kips)	=	40.98
Depth to GWL Below Finish Floor	=	95 (ft)			

Matrix Soil Data:			
Upper Zone:			
UZ Soil Modulus	k_m	=	22 (pci)
Undrained Strength	S_u	=	0 (psf)
Cohesion	c	=	0 (psf)
Friction Angle	Φ_s	=	30 (degrees)
Total Unit Weight:			
Above D_f	γ_2	=	120 (pcf)
Below D_f	γ_1	=	120 (pcf)

RAP Design Parameter Values:				
Shaft Drill Depth		=	11 (ft.)	
Effective Shaft	$H_{eff.}$	=	12.7 (ft.)	*** effective pier length for soil bearing capacity and settlement calculations*
Friction Angle	Φ_{gp}	=	45 (degrees)	
Unit weight:				
Total	γ_{gp}	=	120 (pcf)	*Conservative*

STATIC BEARING CAPACITY

Global Shear (Upper Zone):

Shearing Within The Geopier-Reinforced Soil Matrix:

Composite Soil Strength Parameters:

R_a Reduction Factor =	0.4	
Soil Stress Concentration Factor =	2.5	*(Reduced R_s to account for vert. stress decrease with depth)*
Effective R_s =	0.04	
$\Phi_{comp.}$ =	32 degrees	
$C_{comp.}$ =	0 psf	*(based on value entered for C)*

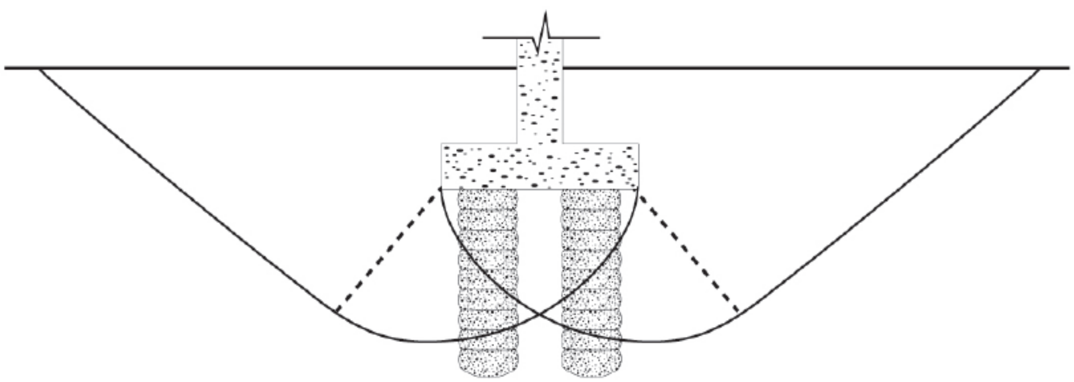
$$q_{ult.} = k_1(C_{comp.}N_c) + k_2(\gamma_1 B N_\gamma) + \gamma_2 D_f N_q$$

where:

k_1 =	1.3	k_1 = 1.3 for square and rectangular footings; and 1.0 for continuous footings
k_2 =	0.5	k_2 = 0.5 for square, rectangular and continuous footings
N_c =	47	(Terzaghi General Shear Factors)
N_γ =	31	
N_q =	32	

$q_{ult.}$ =	$k_1(C_{comp.}N_c)$ +	$k_2(\gamma_1 B N_\gamma)$ +	$\gamma_2 D_f N_q$
$q_{ult.}$ =	-	116,250	23,040
$q_{ult.}$ =	139,290 psf		

For Design Footing Stress = 3290 psf **FS = 42.3**



STATIC BEARING CAPACITY

GEOPIER®

Project:	LADBS - Alameda Crossing
No.	GLA-229 / NLA-126
Engr:	RB
Date:	10/2/2024

Large Footing 3

Parameter Values:

Foundation Data and RAP Geometry:						
Footing Contact Pressure	q =	3130	(psf)	Total Column Load, P (kips)	=	10,254
Footing Length	L =	63	(ft)	Footing Area (sq. ft)	=	3,276
Footing Width	B =	52	(ft)	Total Pier Area (sq. ft)	=	261.67
Footing Depth	D_f =	6	(ft)	Area Ratio (granular layers)	=	0.080
Equivalent Width	B_{eq} =	57.2	(ft)	Stress Applied to Piers (granular) (psf)	=	20,742
Pier diameter	d =	20	(in)	Stress Applied to Matrix Soil (psf)	=	1,601
Number of Piers	N =	120		Relative Stiffness Ratio	=	13
Pier Modulus	k_g =	285	(pci)	Individual Pier Load (kips)	=	45.23
Depth to GWL Below Finish Floor	=	95	(ft)			

Matrix Soil Data:				
Upper Zone:				
UZ Soil Modulus	k_m	=	22	(pci)
Undrained Strength	S_u	=	0	(psf)
Cohesion	c	=	0	(psf)
Friction Angle	Φ_s	=	30	(degrees)
Total Unit Weight:				
Above D_f	γ_2	=	120	(pcf)
Below D_f	γ_1	=	120	(pcf)

RAP Design Parameter Values:					
Shaft Drill Depth		=	12	(ft.)	
Effective Shaft	$H_{eff.}$	=	13.7	(ft.)	*** effective pier length for soil bearing capacity and settlement calculations*
Friction Angle	Φ_{gp}	=	45	(degrees)	
Unit weight:					
Total	γ_{gp}	=	120	(pcf)	*Conservative*

STATIC BEARING CAPACITY

Global Shear (Upper Zone):

Shearing Within The Geopier-Reinforced Soil Matrix:

Composite Soil Strength Parameters:

R_a Reduction Factor =	0.4	
Soil Stress Concentration Factor =	2.5	*(Reduced R_s to account for vert. stress decrease with depth)*
Effective R_ε =	0.03	
$\Phi_{comp.}$ =	31 degrees	
$C_{comp.}$ =	0 psf	*(based on value entered for C)*

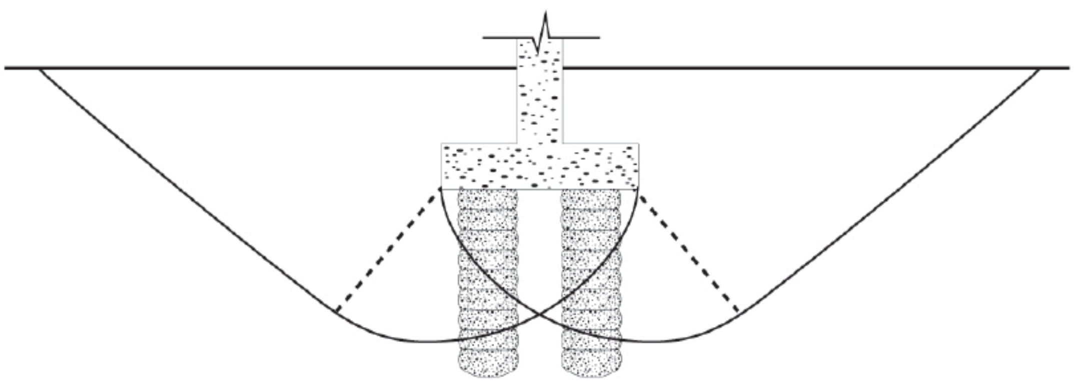
$$q_{ult.} = k_1(C_{comp.}N_c) + k_2(\gamma_1 B N_\gamma) + \gamma_2 D_f N_q$$

where:

k_1 =	1.3	k_1 = 1.3 for square and rectangular footings; and 1.0 for continuous footings
k_2 =	0.5	k_2 = 0.5 for square, rectangular and continuous footings
N_c =	36	(Terzaghi General Shear Factors)
N_γ =	20	
N_q =	22	

$q_{ult.}$ =	$k_1(C_{comp.}N_c)$ +	$k_2(\gamma_1 B N_\gamma)$ +	$\gamma_2 D_f N_q$
$q_{ult.}$ =	-	62,400	15,840
$q_{ult.}$ =	78,240 psf		

For Design Footing Stress = 3130 psf **FS = 25.0**



SUMMARY OF SOIL PARAMETERS

Approximate Sublayer Elevations (depth)	Zone	$E_{m,soil}$ (ksf)
~EL +228’ (0’ to 20’)	Upper	350
EL 228' to EL 225' (20’ to 23’)	Lower	1250
EL 225' to EL 210' (23’ to 38’)	Lower	2000
Below EL 210' (38’)	Lower	Assumed Incompressible

$E_{m,soil}$ – Matrix/Soil Modulus Value

MODULUS OF MATRIX SOILS CORRELATION

Soil Type	q_c/N
Silts, sandy silts, slightly cohesive silt-sand mixtures	2.0

Reference: Schmertmann, J.H. (1970) “Static Cone to Compute Static Settlement Over Sand” Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers. Vol. 96, No. SM3, May 1970.

$$s_u = \frac{q_t - \sigma_v}{N_{kt}}$$

Reference: Robertson, P.K. and Cabal K.L. (2015) “Guide to Cone Penetration Testing for Geotechnical Engineering” Gregg Drilling & Testing, Inc., Signal Hill, CA. 6th Ed.

Conversion:

1) Conservatively use N_{kt}= 18

   $s_u = \frac{q_t-\sigma_v}{18}$ generally simplifies to $s_u = \frac{q_t}{20}$ with same result.

2) Determine s_u

   $\frac{q_c}{N} = 2 \rightarrow q_c$=2N; Substitute q_c for $q_t(tsf)$, results in $s_u(tsf) = \frac{2N}{20}$

   For Upper Zone **Sandy/Silty** soils, N values range from 11 to 25. Conservatively use N = 10 blows per foot

$$s_u = \frac{2 \times 10}{20} = 1tsf = 2ksf$$

3) Matrix Soil Modulus (Em)

$$E_m = 100 s_u F$$

Where F is the Geopier improvement factor. Use F = 1.5

$$E_m = 100 \times 2.4 \times 1.5 = 360ksf \ \rightarrow USE\ \mathbf{350ksf}$$

For the **Lower Zone** Sand soil layers, use typical Elastic Soil Modulus (Es) values for dense to very dense sand/gravel.

Dense Sand: Es ranges from 50 to 81 (MPa) → 1044 to 1692 (ksf). USE **1250 ksf**

Dense Sand and Gravel: Es ranges from 100 to 200 (MPa) → 2089 to 4177 (ksf). USE **2000 ksf**

Reference: Bowles, Joseph E. (1997) "Foundation Analysis and Design" 5th Edition.

Eg Calculation

From Technical Note 1.2.6

$$E_g = \frac{k_g H_s' m}{2}$$

where: H'_s = The length of RAP element required to resist applied top-of-pier load entirely in side shearing (typically 9 feet)

TABLE 4.2 - Preliminary Values for Geopier™ Foundation Design*

SPT = N Blows Per Foot All Soils	UCS, psf Fine-Grained Soils	Sands & Sandy Silts			Silts & Clays			Peat		
		Allowable Composite Footing Bearing Pressure, psf(1)	Geopier™ Element & Footing Segment Capacity, kips(2)	Geopier™ Element Stiffness Modulus, pci(3)	Allowable Composite Footing Bearing Pressure, psf(1)	Geopier™ Element & Footing Segment Capacity, kips(2)	Geopier™ Element Stiffness Modulus, pci(3)	Allowable Composite Footing Bearing Pressure, psf(1)	Geopier™ Element & Footing Segment Capacity, kips(2)	Geopier™ Element Stiffness Modulus, pci(3)
1-3	200-1000	5000	65	165	4500	50	125	3500	30	75
4-6	1001-2300	6000	90	225	5000	70	175	4000	45	110
7-9	2301-3500	7000	105	260	6000	85	210	5000	55	125
10-12	3501-4600	8000	115	285	7000	100	250	N/A	N/A	N/A
13-16	4601-6000	8500	125	310	7000	105	260	N/A	N/A	N/A
17-25	6001-8000	9000	130	325	7500	110	275	N/A	N/A	N/A
Over 25	Over 8000	10,000	145	360	8000	120	300	N/A	N/A	N/A

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Depth (ft)	Elevation (ft)	k_g [fromTable 4.2] (pci)	k_g (kips/ft³)	H'ₛ (ft)	m	E_g (ksf)	**USE E_g (ksf)**
0 to 20	+228	285	492.48	9	1.8	3989.1	**3900**

k_g Calculation

The soil stiffness modulus (km) is calculated by performing unimproved settlement calculations for the various square footing sizes using the actual footing loads. The actual bearing pressure is divided by the calculated settlement to determine the km value (see last row of calculations). The average of all the km values is used for the Geopier design. The average km for this project is 22 pci. The calculations for km are attached.

WGI | GEOPIER®

October 3, 2024

Jonathan Payne
Prologis
2141 Rosecrans Avenue, suite 1151
El Segundo, CA 90245

Re: Quality Control Package for a *Geopier*® Foundation System
Alameda Crossing
1716 East 7th Street
Los Angeles, CA 90021
GFC Project No.: GLA-229 / NLA-126

Dear Mr. Payne,

Western Ground Improvement, Inc. has completed the Geopier® foundation design for above project. The following documents are included herein:

- Geopier Quality Control Package

We are pleased to have provided you with our design services. If you have any questions, please contact this office.

Sincerely,
Western Ground Improvement, Inc.

Ryan Bulatao, G.E., P.E.
Regional Manager

REGISTERED PROFESSIONAL ENGINEER
RYAN PAUL BULATAO
GE 3021
GEOTECHNICAL
STATE OF CALIFORNIA

QUALITY CONTROL PACKAGE FOR GEOPIER FOUNDATIONS
(Copy to be provided to Owner's QA Representative)

Project:	**Alameda Crossing** **Los Angeles, CA**
Project Number:	**GLA-229 / NLA-126**
Geopier Designer:	**Ryan Bulatao, G.E., P.E.**
Mobile:	**310.717.3428**
E-Mail:	**ryan@westerngroundimprovement.com**
Geotechnical Engineer:	**Southern California Geotechnical**
Contact:	**Ricardo Frias, PE**
Phone:	**714.685.1115**
Structural Engineer:	**Magnusson Klemencic Associates**
Contact:	**Ian McFarlane, SE, PE**
Phone:	**206.292.1200**
Referenced Drawings:	**S2.00**
Date of Drawings:	**08/20/24**

Anticipated Geotechnical Conditions:

The subsurface conditions generally consist of about 2 to 8 feet of fill primarily consisting of loose to medium dense sand and silty sand; underlain by alluvial soils consisting of loose sands to a depth of about 12 feet; underlain by medium to very dense sands.
Groundwater was not encountered during the full depth of exploration to about 50 feet.

Potential Anomalies:

None.

Materials to be Encountered at Bottom of Shaft:

Bottom of Geopier elements shall be in native alluvial soils.

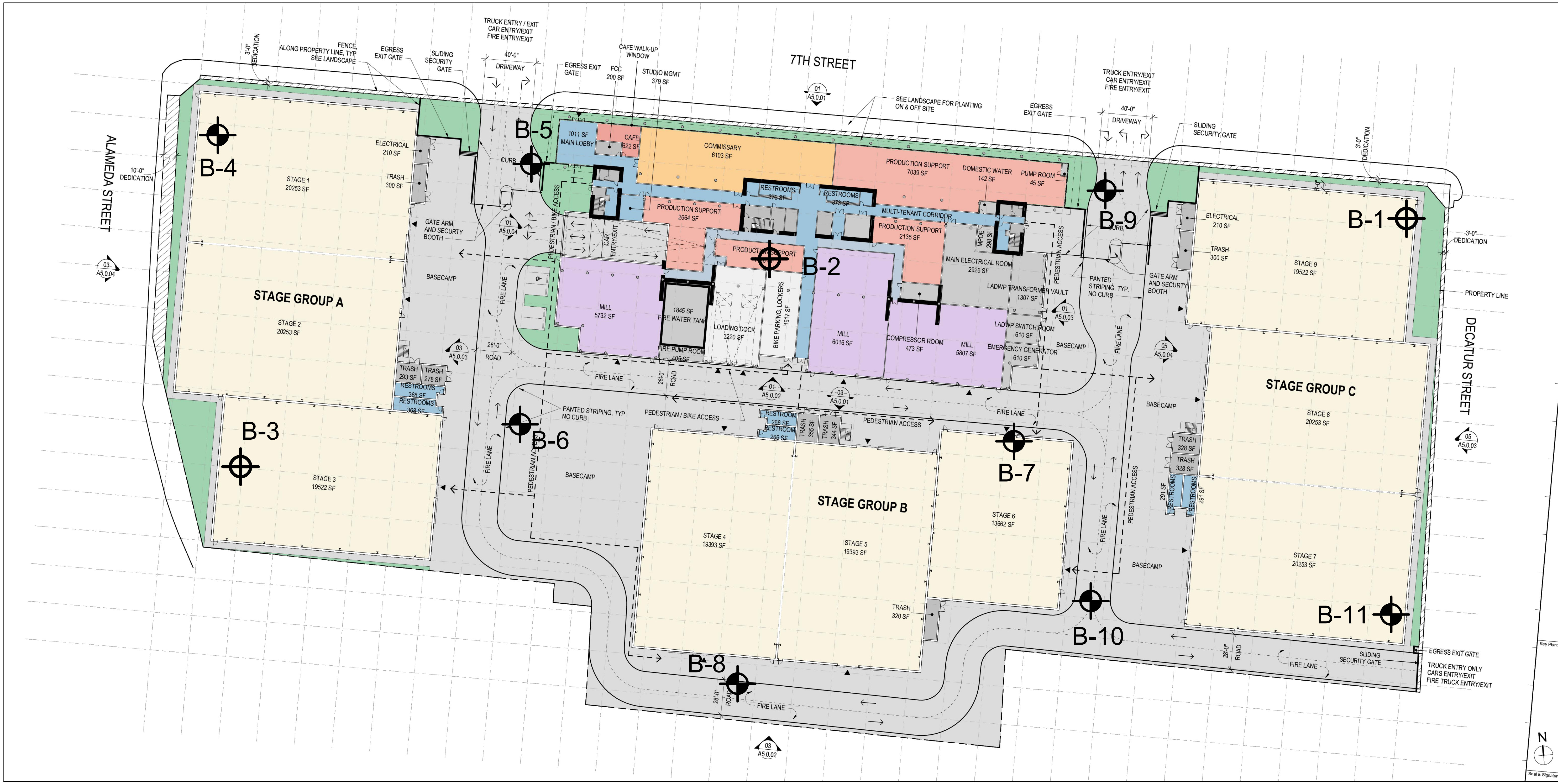
Other Items:

None.

ATTACHMENTS – **GEOTECHNICAL INFORMATION**
GEOPIER TEST SCHEDULES
MODULUS TEST LOCATIONS

GEOTECHNICAL INFORMATION

The attached boring logs have been prepared by others and are included solely for reference purposes. The boring logs should be used for information only and are not intended to represent geotechnical recommendations for this project. The project geotechnical report should be reviewed in its entirety for more information.



GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- PREVIOUS BORING LOCATION (SCG PROJECT NO. 20G243-1)

NORTH

NOTE: SITE PLAN PROVIDED BY CLIENT.

BORING LOCATION PLAN

ALAMEDA CROSSING DEVELOPMENT

LOS ANGELES, CALIFORNIA

SCALE: 1" = 80'

DRAWN: RF
CHKD: GKM

SCG PROJECT
20G243-4R

PLATE 2

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 12/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 28 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
				CONCRETE: 8± inches Portland Cement Concrete with 6± inches of Aggregate Base							
	X	22		FILL: Brown Silty fine Sand, trace medium to coarse Sand, mottled, medium dense-damp	111	5					EI = 3 @ 0 to 5 feet
	X	26		ALLUVIUM: Gray fine Sand, trace to little Silt, medium dense-damp	88	2					
5	X	5		Light Gray Brown fine to coarse Sand, trace Silt, trace to little fine Gravel, medium dense-dry to damp	106	12					
	X	3		Dark Brown Silty fine to coarse Sand, trace fine Gravel, very loose-moist to very moist	90	12					
10	X	6		Light Gray Brown fine to medium Sand, trace coarse Sand, trace Silt, loose-dry to damp	103	2					
				Gray Brown to Dark Gray Brown Silty fine Sand, trace Iron Oxide staining, loose-dry							
15	X	20		Light Gray Brown to Gray Brown fine to coarse Sand, trace fine Gravel, medium dense-dry to damp	106	2					
20	X	42		Gray Brown fine to medium Sand, trace coarse Sand, trace Silt, medium dense-damp to moist	102	6					
25	X	50/5"		Gray Brown fine to coarse Sand, trace to little fine Gravel, trace Silt, very dense-dry to damp	119	2					
30	X	50/5"			103	2					
35	X	58				3					
	X	50/3"				2					

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 12/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 28 feet
READING TAKEN: At 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
					Gray Brown fine to coarse Sand, trace to little fine Gravel, trace Silt, very dense-dry to damp							
	X	50/5"					2					
45												
	X	50/5"					2					
50												
					Boring Terminated at 50'							

TBL 20G243-4 B1-B3.GPJ SOCALGEO.GDT 3/9/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					CONCRETE: 5± inches Portland Cement Concrete with 9 inches of Aggregate Base over 7 inches of Portland Cement Concrete		5					
					FILL: Brown Silty fine to medium Sand, little coarse Sand, trace fine Gravel, loose-damp		6					
5		9	1.0		FILL: Dark Brown Clayey Silt, trace iron oxide staining, medium stiff-very moist		40					
10		15			ALLUVIUM: Light Brown fine to coarse Sand, trace to little fine to coarse Gravel, trace Silt, medium dense-dry to damp		8					
15		20					2					
20		20					3					
25		48			@ 23½ feet, little fine to coarse Gravel, dense		3					
		46					3					
30					Gray Brown Silt, little fine Sand, little iron oxide staining, dense-very moist		32					
		41			Brown Silty fine to medium Sand, little coarse Sand, trace fine to coarse Gravel, dense-damp		3					
					Boring Terminated at 32½' and grouted at completion							

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ASPHALT: 4± inches Ashaltic Concrete with 6 inches of Aggregate Base							
	X	24			FILL: Dark Gray Brown Gravelly fine to coarse Sand, little Silt, medium dense-damp		4					
	X	26			ALLUVIUM: Brown Silty fine to coarse Sand, trace fine to coarse Gravel, medium dense to dense-damp to moist		12			24		
5												
	X	39			@ 6 feet, little fine to coarse Gravel		7					
	X	39					8			17		
10												
	X	50/2"			@ 13½ feet, very dense-damp		3					
15												
					Light Brown fine to medium Sand, trace fine Gravel, trace coarse Sand, trace Silt, medium dense to dense-damp							
	X	24					3			4		
20												
	X	30			@ 23½ feet, trace fine to coarse Gravel, trace Silt		4					
25												
	X	24			@ 28½ feet, medium dense		3					
30												
					Boring Terminated at 30' and grouted at completion							

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ASPHALT: 2± inches Ashaltic Concrete with 8 inches of Portland Cement Concrete							
					FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, loose-damp		3					
					ALLUVIUM: Brown fine to medium Sand, little Silt, trace coarse Sand, trace fine Gravel, medium dense-damp		5					
5		15			Brown Silty fine to medium Sand, little coarse Sand, little fine to coarse Gravel, loose to medium dense-dry to damp		3					Disturbed Sample
		12					2					Disturbed Sample
10		27										No Sample Recovery
					Light Brown fine to coarse Sand, little fine to coarse Gravel, trace Silt, dense to very dense-dry to damp							
15		65/10"				116	2					
20		29			@ 19 feet, little Silt, medium dense	111	2					
25		47			@ 24 feet, dense	114	2					
					Brown Gravelly fine to coarse Sand, trace to little Silt, occasional Cobbles, dense-dry to damp							
30		51				107	2					
					Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, dense-dry to damp							
		39					2					

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

TEST BORING LOG **PLATE B-7a**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/9/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At 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
					Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, dense-dry to damp							
	X	30			@ 38½ feet, little fine to coarse Gravel, occasional Cobbles, medium dense to dense		4					
40												
	X	50/3"			@ 43½ feet, occasional Cobbles, very dense		2					
45												
	X	50/5"			@ 48½ feet, occasional Cobbles, very dense		2					
50												
					Boring Terminated at 50' and grouted at completion							

TBL 20G243-4 B4-B11.GPJ SOCALGEO.GDT 3/10/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 20G243-4
PROJECT: Proposed C/I Development
LOCATION: Los Angeles, California

DRILLING DATE: 2/8/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ASPHALT: 7± inches Ashaltic Concrete with 5 inches of Portland Cement Concrete over 3 inches of Aggregate Base							
	X	18			FILL:Dark Brown Silty fine to medium Sand, trace coarse Sand, mottled, medium dense-moist	105	11					
					FILL:Gray Brown Silty fine Sand, loose-moist							
	X	11			ALLUVIUM:Light Brown fine to medium Sand, trace fine to coarse Gravel, loose-damp to moist	95	11					
5	X	12				92	3			1		
	X	14				95	3					
	X	20			Gray Brown fine to coarse Sand, some Silt, trace fine Gravel, medium dense-damp		4			11		Disturbed Sample
10												
					Light Brown Silty fine Sand, little medium Sand, dense-moist							
	X	39				111	12			25		
15					Light Brown Gravelly fine to coarse Sand, little Silt, medium dense to dense-dry to damp							
	X	18					2			4		
20												
	X	24					3					
25												
	X	26					4					
30					Boring Terminated at 30'							

GEOPIER TEST SCHEDULES

Geopier Foundation Company®

GEOPIER®

Project Name:	Ladbs - Alameda Crossing
Project Location:	Los Angeles, Ca
Project Number:	GLA-229 / NLA-126

Geopier® Modulus Test Schedule

Geopier Design Stress:	28,035	psf
Geopier Element Design Diameter:	20	in.
Geopier Design Modulus:	285	pci

Modulus Test Location:	Near B-2 (see Figure 1)
Test Geopier Element Shaft Length:	17 ft
Concrete Cap Thickness:	3.5 ft
Total Geopier Element Depth:	20.5 ft

Load No.	Ram Load, (kips)	Percent of Design Stress	Minimum Duration	Maximum Duration	Remarks
	3.06	5.0%	N/A	N/A	seating load
1	10.20	16.7%	15 min	60 min	
2	20.39	33.3%	15 min	60 min	
3	30.58	50.0%	15 min	60 min	
4	40.78	66.7%	15 min	60 min	
5	50.97	83.3%	15 min	60 min	
6	61.16	100.0%	15 min	60 min	
7	71.34	116.6%	60 min	240 min	
8	81.55	133.3%	15 min	60 min	
9	91.75	150.0%	15 min	60 min	
10	122.33	200.0%	15 min	60 min	
11	61.16	100.0%	N/A	N/A	rebound, unload
12	40.37	66.0%	N/A	N/A	rebound, unload
13	20.18	33.0%	N/A	N/A	rebound, unload
14	3.06	5.0%	N/A	N/A	rebound, unload

Notes:

1 - The Geopier element to be used in the modulus load testing should be installed in a manner similar to production, at least 4 days prior to testing, so that pore-pressures have adequate time to dissipate.

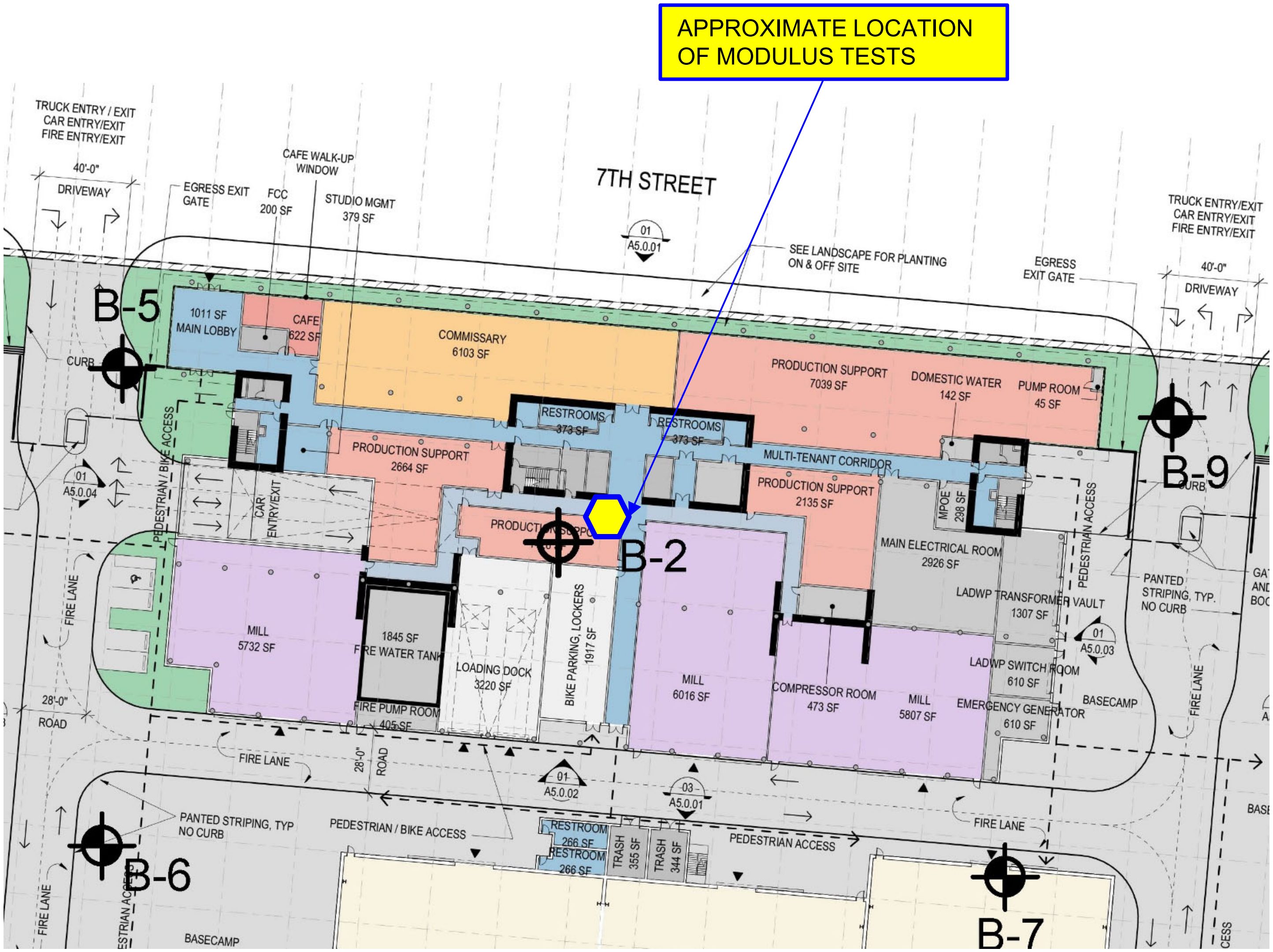
2 - The modulus load test shall be performed to a stress not less than 200% of the design maximum top-of-pier stress indicated in the Geopier Design Calculations.

3 - The modulus load test Geopier element shall be installed to a depth of 20.5 feet below the ground surface with a 3.5-foot thick unreinforced concrete leveling pad.

4 - A telltale shall be installed in the bottom one-third of the tested Geopier element. Telltale deflections shall be monitored concurrent with top of Geopier deflections during the modulus load test.

5 - The modulus load test setup shall be as shown on Geopier Construction Drawing GP0.2. Helical anchors should be installed in accordance with manufacturers specifications.

6 - A representative of the owner's geotechnical consultant should be present to witness the load test.



REFERENCE: Southern California Geotechnical, Preliminary Geotechnical Investigation, 1716 East 7th Street, dated May 7, 2024. Boring Location Plan, Plate 2.

MODULUS TEST LOCATION

Date: OCT 2024	Project No.: GLA-229 / NLA-126	Project: LADBS – Alameda Crossing	Figure 1

WGI | GEOPIER

Modification System

(/PMATS/Customer/Home/Index)

Ask CHIP Dashboard (/PMATS/Customer/)

Arteen Mnayan (https://angeleno.lacity.org/settings)

Sign out (/PMATS/Landing/Logout)

More Information

Modification ID: 2855
Permit Application:
Address: 1716 E 7TH ST
Modification Type: Blank Form

Submitted For Review ▶ Review In Process ▶ Review Result ▶ Fees Pending ▶ Fees Paid ▶ Final Review ▶ Modification Complete ▶

Your Request for a Modification has been received and will be reviewed by the appropriate LADBS staff. Please notify your Plan Checker/Inspector/Engineer or Geologist of your submission. Please reference your Modification ID # when communicating with the staff during your plan check, inspection, response to a Code Enforcement Order, or geology/soils report review process.

You will be notified via email when there is an update to the application status.

Back (/PMATS/Customer) View Modification

REQUEST FOR MODIFICATION OF BUILDING ORDINANCES

UNDER AUTHORITY OF L.A.M.C. SECTION 98.0403

CITY LA DBS — DEPARTMENT OF BUILDING AND SAFETY

PERMIT APP. #: No Permit or CEIS # Provided	**MODIFICATION#:** 2855	**DATE:** 12/10/2024	**For City Dept. Use Only** Grading Grading

JOB ADDRESS: 1716 E 7TH ST

Tract: E. B. MILLAR TRACT

Block: C

Lot: 3

Owner: Prologis	Petitioner: Mnayan, Arteen
Address: 2141 Rosecrans Avenue	Address: 333 S. Grand Ave, 47th Floor
City: El Segundo; State: CA; Zip: 90245; Phone: 213-229-5158	City: Los Angeles; State: CA; Zip: 90071; Phone: 213-229-5158

REQUEST (SUBMIT PLANS OR ADDITIONAL SHEETS AS NECESSARY) | CODE SECTIONS:

For the spread footing foundations supported by Geopier Rammed Aggregate Piers (RAPs), we request that the area replacement ratio (Ra) be less than the minimum of 30% that is referenced in the LARR 26139.

JUSTIFICATION (SUBMIT PLANS OR ADDITIONAL SHEETS AS NECESSARY)

As shown in the footings calculations, the factor of safety (FS) is 2.0 or greater, and the bearing capacity calculations show a FS of 5.5 or greater. The anticipated settlement under the footings is about 1-inch or less.

Mnayan, Arteen	*Mnayan, Arteen*	Agent for Owner
Owner/Petitioner Name (Print)	**(Electronic Signature via Online Modification)**	**Position**

FOR CITY DEPARTMENT'S USE ONLY BELOW THIS LINE

DEPARTMENT ACTION

[X] **GRANTED** [ ] **DENIED**

Alan Dang — 02/05/2025
Reviewed by: (Staff) (Print) ***Date***

Jesus Acosta — 02/05/2025
Action taken by: (Supervisor) (Print) ***Date***

NOTE: IN CASE OF DENIAL, SEE PAGE #3 OF THIS FORM FOR APPEAL PROCEDURES

CONDITIONS OF APPROVAL (Continued on Page 2):

FEES

(DEPARTMENT USE ONLY)

Appeal Processing Fee ..(No. of Items) =	1	X $130 + $39/addl	= $	130.00
Inspection Fee ....(No of Insp.) =	0	X $ 84.00	= $	0.00
Research Fee ... (Total Hours Worked) =	2	X $104.00	= $	208.00
Subtotal..			= $	338.00
Development Services Center Surcharge		X 3%	= $	10.14
Systems Development Surcharge		X 6%	= $	20.28
Total Fees			= **$**	**368.42**

Fees verified by: Alan Dang

For Cashiers Use Only
(PROCESS ONLY WHEN FEES ARE VERIFIED)

Payment Date: 01/17/2025
Receipt Number: 2009174
Total Fees: $368.42

CONDITIONS OF APPROVAL (Continued from Page 1)

RFM#29585

See department approval letter dated 2/5/25, Log# 130835-02

CONCURRENCES REQUIRED

Concurrences required from the following Department(s):		Approved	Denied
☐ Los Angeles Fire Department		☐	☐
☐ Public Works Bureau of Engineering		☐	☐
☐ Department of City Planning		☐	☐
☐ Department of County Health		☐	☐
☐ Coastal Commission		☐	☐
☐ Public Works Bureau of Sanitation		☐	☐
☐ LADBS Permit and Engr		☐	☐
☐ LADBS Inspection, Residential		☐	☐
☐ LADBS Inspection, Commercial		☐	☐
☐		☐	☐
☐		☐	☐
☐		☐	☐

CITY OF LOS ANGELES
BOARD OF BUILDING AND SAFETY/DISABLED ACCESS COMMISSION APPEAL FORM

(Must be Attached to the Modification Request Form, Page 1)

AFFIDAVIT – LADBS BOARD OF BUILDING AND SAFETY COMMISSIONERS – RESOLUTION NO. 832-93

I,____________________do state and swear as follows:
(Print or Type Name of the Person Signing this Form)

1. The name and mailing address of the owner of the property (as defined in the resolution 832-93) at________________as shown on the appeal application (LADBS Com 31) are correct, and
2. The owner of the property as shown on the appeal application will be made aware of the appeal and will receive a copy of the appeal.

I declare under PENALTY OF PERJURY that the forgoing is true and correct.

Owner's Name(s) ________________ (Please Type or Print) ________________ (Please Type or Print)

Owner's Signature(s)________________(Two Officers' Signatures Required for Corporations)
(Please Sign)

Name of Corporation ________________ (Please Print Name of Corporation) ________________ (Please Type or Print)

Dated this________day of________________20_____

CALIFORNIA ALL-PURPOSE ACKNOWLEDGEMENT ------------------ SIGNATURE(S) MUST BE NOTARIZED

State of **CALIFORNIA** County of________________on________________

before me,________________, personally appeared________________,
Name, Title of Officer (e.g. Jane Doe, Notary Public) Name(s) of Signer(s)

who proved to me on the basis of satisfactory evidence to be the person(s) whose name(s) is/are subscribed to the within instrument and acknowledged to me that he/she/they executed the same in his/her/their authorized capacity(ies), and that by his/her/their signature(s) on the instrument in person(s), or the entity upon behalf of which the person(s) acted, executed the instrument. **I certify under PENALTY OF PERJURY under the laws of the State of California that the foregoing is true and correct.**

WITNESS my hand and official seal. Signature ________________

As a covered entity under Title II of the Americans with Disabilities Act, the City of Los Angeles does not discriminate on the basis of disability and, upon request, will provide reasonable accommodation to ensure equal access to its programs, services and activities.

APPEAL OF DEPARTMENT ACTION TO THE BOARD OF BUILDING AND SAFETY COMMISSIONERS/DISABLED ACCESS APPEALS COMMISSION

Applicant's Name ________________ Applicant's Title ________________

Signature ________________ Date ________________

FEES

(DEPARTMENT USE ONLY)

Board Fee ....(No. of Items)	X	$130.00	=	0.00
Inspection Fee................ (No of Insp.) =	X	$84.00	=	0.00
Research Fee.... (Total Hours Worked) =	X	$104.00	=	0.00
Subtotal			=	0.00
Development Services Center Surcharge	X	3%	=	0.00
Systems Development Surcharge	X	6%	=	0.00
Total Fees			=	0.00

Fees verified by:
Print and Sign ________________

For Cashiers Use Only
(PROCESS ONLY WHEN FEES ARE VERIFIED)

BOARD OF
BUILDING AND SAFETY
COMMISSIONERS

JACOB STEVENS
PRESIDENT

NANCY YAP
VICE PRESIDENT

CORISSA HERNANDEZ
JAVIER NUNEZ
MOISES ROSALES

CITY OF LOS ANGELES
CALIFORNIA

KAREN BASS
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

JOHN WEIGHT
EXECUTIVE OFFICER

SOILS REPORT APPROVAL LETTER

February 5, 2025

LOG # 130835-02
SOILS/GEOLOGY FILE - 2

ProLogis
2141 Rosecrans Ave. #1151
El Segundo, CA 90245

TRACT:	E. B. MILLAR TRACT (MR 13-91)
BLOCK:	C
LOT(S):	3
LOCATION:	1716 E. 7th St.

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Addendum Report	20G243-8R2	02/03/2025	SoCalGeo
Oversized Docs.	``	``	``
Request for Modification	29585	12/10/2024	LADBS

PREVIOUS REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Dept. Review Letter	130835-01	11/22/2024	LADBS
Soils Report	20G243-6	10/17/2024	SoCalGeo
Addendum Report	20G243-7	10/17/2024	``
Update Report	20G243-4R2	10/17/2024	``
Addendum Report	20G243-5R2	05/07/2024	``
Dept. Review Letter	130835	06/14/2024	LADBS
Soils Report	20G243-4R	05/07/2024	SoCalGeo
Addendum Report	20G243-5R2	05/07/02024	SoCalGeo
Dept. Approval Letter	123370-01	01/18/2023	LADBS
Soils Report	20G243-3	12/14/2022	SoCalGeo
Dept. Review Letter	123370	10/20/2022	LADBS
Soils Report	20G243-2	09/22/2022	SoCalGeo

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provide recommendations for the proposed main building: 8 level mixed use structure (6 levels of parking), and three, one-story structures (i.e., Stage Groups A, B, and C). The earth materials at the subsurface exploration locations consist of up to 8 feet of uncertified fill underlain by native soils.The consultants recommend to support the proposed structure(s) on conventional or mat-type on properly placed fill, and rammed aggregate piers foundations bearing on native soils.

AN EQUAL EMPLOYMENT OPPORTUNITY - AFFIRMATIVE ACTION EMPLOYER

The Department previously conditionally approved the above referenced reports for the proposed industrial building and studio (for EIR and CEQA study purposes only) in a letter dated 01/18/2023, Log #123370-01.

The referenced Request for Modification(s) with File No(s). 29585 to allow the area replacement ratio (Ra) to be less than the minimum of 30% that is referenced in the LARR 26139 for the spread footing foundations supported by Geopier Rammed Aggregate Piers (RAPs), is acceptable provided the conditions listed in this letter are complied with during site development.

As of January 1, 2023, the City of Los Angeles has adopted the new 2023 Los Angeles Building Code (LABC). The 2023 LABC requirements will apply to all projects where the permit application submittal date is after January 1, 2023.

The referenced report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2023 City of LA Building Code. P/BC numbers refer to the applicable Information Bulletin. Information Bulletins can be accessed on the internet at dbs.lacity.gov.)

1. All conditions of the above referenced Department approval letter(s) shall apply except as specifically modified herein. All references to prior building code sections and information bulletins in the referenced Department approval letter(s) shall be deemed to reference applicable building code sections and information bulletins.

2. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring or constructed using ABC slot cuts. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

3. Shoring shall be designed for the lateral earth pressures specified on page 30 and plate 3 pf the of the 10/17/2024 report; all surcharge loads shall be included into the design.

4. ABC slot-cut method may be used for unsurcharged temporary excavations with each slot not exceeding 8 feet in height and not exceeding 8 feet in width, as recommended. The soils engineer shall verify in the field if the existing earth materials are stable in the slot-cut excavation. Each slot shall be inspected by the soils engineer and approved in writing prior to any worker access.

5. All conventional and mat foundations shall derive entire support from properly placed fill, as recommended.

6. The proposed ground improvement system is Rammed Aggregate Pier.

7. Property lines and excavations adjacent to geopier elements shall be setback a minimum distance of 8 feet, as recommended.

8. The length, diameter and number of geopier elements for each footing is summarized on Sheet GP0.1 of the WGI report, as referenced on page 2 of the current report dated 02/03/2025.

9. The aggregate to be rammed into the drilled holes for the proposed Aggregate Piers construction shall be graded to avoid fine migration, and shall conform to the minimum standards of Class 2 Aggregate Base per the Standard Specifications of State of California, as recommended.

10. At least one (1) RAP, Modulus Test (MT) per every 500 piers constructed up to 1500 piers, and then one (1) test per every 1000 piers thereafter, shall be installed and tested to verify the installation methods, soil conditions, etc. The tested RAP shall be installed such that the testing conditions match the proposed conditions (i.e., top and tip RAP elevations, overburden pressures, etc. are the same for the test and production RAPs).

    a. The maximum load applied during the modulus load test shall equal to 200% of the maximum design stress. Loading procedure B (Maintained Test) of ASTM D 1143 is required.

    b. The load test evaluation method shall satisfy a deflection criterion established by the project specifications. In the absence of an over-riding criteria, use 1-inch deflection or less at 200% the design load. The project specifications shall not specify a deflection criterion of less than 1-inch deflection at 200% the design load.

    c. A supplementary report providing the information, test results, and subsequent recommendations on the load-tests shall be submitted to the Department for approval. The report shall indicate the measured deflection, the geopier stiffness modulus and geopier capacity Qcell.

11. The installation of the Aggregate Piers shall be performed under the inspections and approvals of the soils engineer.

12. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

13. Retaining walls shall be designed for the lateral earth pressures specified in the section titled "Retaining Walls" starting on page 28 of the 10/17/2024 report. All surcharge loads shall be included into the design.

14. The use of acceptable prefabricated drainage composites (also known as geosynthetic subdrain systems), as an alternative to traditionally accepted methods of draining retained earth, shall be determined during structural plan check.

15. The infiltration facility design and construction shall comply with the minimum requirements specified in the Information Bulletin P/BC 2023-118.

16. The infiltration system shall be constructed at the location shown on the drawing attached to the current report.

17. The construction of the infiltration system shall be provided under the inspection and approval of the soils engineer.

18. An overflow outlet shall be provided to conduct water to the street in the event that the infiltration system capacity is exceeded. (P/BC 2023-118)

19. Approval for the proposed infiltration system from the Bureau of Sanitation, Department of Public Works shall be secured.

20. A minimum distance of 10 feet (in any direction) shall be provided from adjacent proposed/existing footings to the proposed infiltration system. A minimum distance of 10 feet horizontally shall be provided from private property lines to the proposed infiltration system.

21. Installation of shoring, underpinning, slot cutting and/or pile excavations shall be performed under the inspection and approval of the soils engineer and deputy grading inspector (1705.6, 1705.8).

22. A supplemental report shall be provided in the event any deviation to the currently proposed project configuration, as presented and as shown in the plans and cross sections included in the approved reports, is made. This shall include but not limited to: relocation, change in any dimension, change in the number of stories above or below grade of any of the proposed structures; addition of any structure(s), such as retaining walls, decks, swimming pools, driveways, access roads, living quarters, etc.; or, additional permanent grading or temporary grading for construction purposes that are not described and not shown in the plans and cross sections included in the approved reports.

ALAN DANG
Structural Engineering Associate II

AD/ad
Log No. 130835-02
213-482-0480

cc: SoCalGeo, Project Consultant
LA District Office

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

District 1A	Log No. 130835-2

APPLICATION FOR REVIEW OF TECHNICAL REPORTS

INSTRUCTIONS

A. Address all communications to the Grading Division, LADBS, 221 N. Figueroa St., 12th Fl., Los Angeles, CA 90012
Telephone No. (213)482-0480.
B. Submit two copies (three for subdivisions) of reports, one "pdf" copy of the report on a CD-Rom or flash drive, and one copy of application with items "1" through "10" completed.
C. Check should be made to the City of Los Angeles.

1. LEGAL DESCRIPTION
Tract: See Attachment "A"
Block: Lots:

2. PROJECT ADDRESS:
1716 East 7th Street; see "Attachment A" for additional addresses

3. OWNER: 1614 E 7th Street LLC c/o Arteen Mnayan, Mayer Brown LLP
Address: 333 S. Grand Avenue, 47th Floor
City: CA Zip: 90071
Phone (Daytime): 213-229-5158

4. APPLICANT 1614 E 7th Street LLC c/o Arteen Mnayan, Mayer Brown LLP
Address: Same as owner.
City: Zip:
Phone (Daytime): 213-229-5158
E-mail address: amnayan@mayerbrown.com

5. Report(s) Prepared by:
6. Report Date(s):
Report prepared by SoCalGeo dated December 10,2024: Response to Soils Report Review Letter, dated November 22, 2024, Log # 130835-01.

7. Status of project: ☑ Proposed ☐ Under Construction ☐ Storm Damage

8. Previous site reports? ☑ YES if yes, give date(s) of report(s) and name of company who prepared report(s)
Response to City of Los Angeles Review Letter, Alameda Crossing Development (20G243-6) dated October 17, 2024; Preliminary Geotechnical Investigation (Revised October 2024)(20G243-4R2) dated October 17, 2024; and Ground Improvement Plan Review (20G243-7) dated October 17, 2024; Preliminary Geotechnical Investigation, Alameda Crossing Development, 1716 East 7th Street, Los Angeles, California, prepared for Prologis by Southern California Geotechnical, Inc. (SCG), SCG Project No. 20G243-4R, dated May 7, 2024; and Results of Infiltration Testing, Alameda Crossing Development, 1716 East 7th Street, Los Angeles, California, prepared for Prologis by SCG, SCG Project No. 20G243-5R2, dated May 7, 2024

9. Previous Department actions? ☑ YES if yes, provide dates and attach a copy to expedite processing.
Dates: Soils Report Review Letters, prepared by the City of Los Angeles, dated June 14, 2024, Log # 130835; and dated November 22, 2024, Log # 130835-01.

10. Applicant Signature: Arteen Mnayan Position: Representative

(DEPARTMENT USE ONLY)

REVIEW REQUESTED	FEES	REVIEW REQUESTED		FEES
☒ Soils Engineering		No. of Lots		
☐ Geology		No. of Acres		
☐ Combined Soils Engr. & Geol.		☐ Division of Land		
☐ Supplemental		Other		
☐ Combined Supplemental		☒ Expedite		90.75
☐ Import-Export Route		☒ Response to Correction		181.50
Cubic Yards:		☐ Expedite ONLY		
		Sub-total		272.25
		Surcharge		69.91
		TOTAL FEE		342.16

Fee Due: 342.16
Fee Verified By: AM Date: 12/16/24
(Cashier Use Only)

Receipt #
1991163
Paid on
12/16/24

ACTION BY:

THE REPORT IS: ☐ NOT APPROVED
☐ APPROVED WITH CONDITIONS ☐ BELOW ☐ ATTACHED

For Geology Date

For Soils Date