Appendix F
Geotechnical Investigation

CITY OF LOS ANGELES
CALIFORNIA

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

FRANK M. BUSH
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

OSAMA YOUNAN, P.E.
EXECUTIVE OFFICER

GEOLOGY AND SOILS REPORT APPROVAL LETTER

February 6, 2020

LOG # 111755
SOILS/GEOLOGY FILE – 2
LIQ

Stockdale Capitol Partners, LLC
10850 Wilshire Boulevard, Suite 1050
Los Angeles, CA 90024

TRACT: 7555
BLOCK: 4
LOTS: 3 - 9
LOCATION: 650 - 674 S. San Vicente Boulevard

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Soil Report	BG 22581	07/25/2019	Byer Geotechnical, Inc.
Oversized Documents	``	``	``

PREVIOUS REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Inter-Dept. Review Ltr.	Log # 96658	02/15/2017	LADBS
Soil Report	BG 22581	01/19/2017	Byer Geotechnical, Inc.

The Grading Division of the Department of Building and Safety (LADBS) has reviewed the referenced reports dated 01/19/2017 & 07/25/2019. Byer Geotechnical (see pg. 3 in the 07/25/2019 report), have clarified that a 09/20/2017 report was not submitted and that the 07/25/2019 report replaces it. The current 07/25/2019 report included a Site Plan and Sections A, B, C, D and E (presented at a scale of 1 inch = 20 feet). As shown and described on pg. 2 in the 07/25/2019 report, *a twelve (12) level, at-grade, mixed-use building is proposed*.

Additional information in the 07/25/2019 report was provided in the section titled "Metro Purple Line Extension" (pgs. 3 and 4) and the section titled "Site Description" (pg. 5). Appendix I in the 07/25/2019 report included researched information including 2011 exploration performed in close vicinity to the site as part of the Westside Subway Extension of the Metro Purple Line. Appendix II included the Boring Logs and Laboratory Testing Data from 2017. Appendix III included Calculations and Figures.

Based on the information provided, alluvium (unit Qa) overlies earth material assigned to the San Pedro Formation (unit Qsp). The San Pedro Formation described as "bedrock" (see section titled "Earth Materials" on pgs. 7 and 8 in the 07/25/2019 report), was encountered at approximate depths below the explored on-site locations B-1 through B-4 of about 30 feet and at the Subway Extension locations G-127 and 128 at depths between 25 and 29 feet. Be informed that the San Pedro Formation of the lower Pleistocene (as described on pg. 7 and in the regional geology map cited in the report), is generally sand.

As of January 1, 2020, the City of Los Angeles has adopted the new 2020 Los Angeles Building Code (LABC). The 2020 LABC requirements will apply to all projects where the permit application submittal date is after January 1, 2020.

Groundwater was observed in 2011 at depths in the Subway Extension locations shown and on the subject property in 2017 at depths varying from 26 to 41 feet below the existing ground surfaces at these locations. The referenced reports by Byer Geotechnical are acceptable, provided the following conditions are complied with:

1. A supplemental report with recommendations updated to the 2020 LABC shall be submitted, ***if*** the permit application date is after January 1, 2020.

2. Obtain approval from the City Planning Department for the development as currently proposed. Note: (***i***) Based on the LA City Planning Case Tracking System, Vesting Tentative Tract Map 74865 filed 02/26/2017 has a 11/21/2019 assignment date; (***ii***) no subterranean levels are currently proposed or approved, except for retaining walls associated with elevator pit shaft retaining walls.

3. Retaining walls for elevator pit/s shall be designed and constructed as recommended and specified (pgs. 25 – 27 and the recommendations for waterproofing on pg. 30 in the 07/25/2019 report.

4. The recommended equivalent fluid pressure (EFP) for the proposed elevator pit retaining wall shall apply from the top of the freeboard to the bottom of the wall foundation.

5. The retaining wall/s for the elevator pit retaining wall shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soil report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record.

6. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector.

7. Obtain approval from the Department of Public Works – Bureau of Engineering for the development proposed adjacent to the public way.

8. Obtain approval from the appropriate utilities concerning the proposed development. Note: The Site Plan and the City Navigate LA Maps show a storm drain easement at the Lot 4/Lot 5 boundary and, the approximate location of the Metro Line Tunnel is shown on Section B.

9. Infiltration is not considered feasible (pg. 29 in the 07/25/2019 report) and is therefore not approved.

10. Site water shall be conveyed in non-erosive devices to the street or an approved location in a manner acceptable to the LADBS and the Department of Public Works.

11. The building shall be connected to the public sewer system.

12. The recommendations in the reports which are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

13. Final plans shall be based on a recent, updated licensed survey, including but not limited to details

of construction based on existing site conditions, property boundaries, utilities, adjacent streets/alleys, easements (including the tunnel easement if applicable) and the elevator pit retaining walls.

14. Existing fill or loose disturbed alluvium shall not be used for support of foundations, concrete slabs or new fill.

15. As recommended (see pgs. 21 – 25 in the 07/25/2019 report) and specified (see details in paragraphs 2 and 3 on pg. 19 in the 07/25/2019 report), the proposed building shall be supported on a deepened foundation system of friction piles that are extended and embedded a minimum of ten (10) feet into "bedrock" of the San Pedro Formation anticipated at an approximate depth of thirty (30) feet, as approved by inspection by the soil engineer or geologist.

16. The seismic design shall be based on a Site Class D as recommended. All other seismic design parameters shall be reviewed during LADBS building plan check.

17. As recommended (pgs. 20 and 28 in the 07/25/2019 report), the concrete floor slab for the proposed building shall be cast over future certified compacted fill or the slab may be structurally designed to bridge between the friction pile foundation system. As recommended (pg. 28), floor slabs placed on future compacted fill shall be at least five (5) inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced a maximum of 16 inches on center each way.

18. If compacted fill is planned to support the floor slab, the future fill shall extend beyond the footings a minimum distance equal to the depth of the fill below the bottom of footings or a minimum of three feet whichever is greater, as recommended.

19. As recommended, temporary excavations to prepare a compacted fill pad to support the slab on grade shall be restricted to a vertical height of five (5) feet with portions exceeding this specified height sloped to a horizontal to vertical slope gradient not exceeding a horizontal to vertical slope gradient of 1:1.

20. The soils engineer and/or geologist shall inspect all excavations to determine that conditions anticipated in the report have been encountered and to provide recommendations for the correction of hazards found during grading.

21. A grading permit shall be obtained.

22. A copy of the subject and appropriate referenced report/s and this approval letter shall be attached to the District Office and field set of plans. Submit one copy of the above report to the Building Department Plan Checker prior to issuance of the permit.

23. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the State Construction Safety Orders enforced by the State Division of Industrial Safety.

24. Prior to excavation, an initial inspection shall be called at which time the sequence of construction, the recommended friction pile locations, all grading work, protection fences and dust and traffic control will be scheduled. This work shall be performed under the inspection and approval of the soils engineer and deputy grading inspector.

25. The geologist and soils engineer shall review and approve the detailed plans prior to issuance of any permits. This approval shall be by signature on the plans which clearly indicates that they

have reviewed the plans prepared by the design engineer and that the plans include the recommendations in their report.

26. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density. Placement of gravel in lieu of compacted fill is only allowed if complying with LAMC Section 91.7011.3.

27. Prior to the placing of compacted fill, a representative of the consulting soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the LADBS Grading Inspector and the Contractor stating that the soil inspected meets the conditions of the report, but that no fill shall be placed until the LADBS Grading Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be filed in the final compaction report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. The engineer's certificate of compliance shall include the grading permit number and the legal description as described in the permit.

28. Prior to the pouring of concrete, a representative of the consulting soils engineer shall inspect and approve the foundation excavations. The representative shall post a notice on the job site for the LADBS Building Inspector and the Contractor stating that the work so inspected meets the conditions of the report, but that no concrete shall be poured until the LADBS Building Inspector has also inspected and approved the foundation excavations. A written certification to this effect shall be filed with the Department upon completion of the work.

STEPHEN DAWSON
Engineering Geologist II

ALAN DANG
Structural Engineering Associate II

SD/AD:sd/ad
Log No. 111755
213-482-0480

cc: Applicant
Byer Geotechnical, Inc.
LA District Office

BYER GEOTECHNICAL, INC.

January 14, 2020
BG 22581

Stockdale Capital Partners, LLC
10850 Wilshire Boulevard, Suite 1050
Los Angeles, California 90024

Attention: Mr. Matt Porteous

Subject

Transmittal of Geologic and Soils Engineering Exploration
Proposed Twelve-Story, At-Grade Mixed-Use Building
Lots 3 - 9, Block 4, Tract 7555
650 - 674 South San Vicente Boulevard
Los Angeles, California

Dear Mr. Porteous:

Byer Geotechnical has revised our report dated July 25, 2019, which replaces the previously-transmitted report dated July 25, 2019. The revised report describes the geotechnical engineering conditions with respect to the currently-proposed twelve-story, at-grade mixed-use building. All copies of the previously transmitted report dated July 25, 2019, for a 10-story project should be discarded. The reviewing agency for this document is the City of Los Angeles, Department of Building and Safety (LADBS). The reviewing agency requires two unbound copies, one with a wet signature, a CD (PDF format), an application form, and a filing fee. The report has been distributed as follows:

(2) Addressee (Email and Mail)
(2) City of Los Angeles, Department of Building and Safety

Byer Geotechnical will file the report and CD with the LADBS. Please review the report carefully prior to submittal to the governmental agency. Questions concerning the report should be directed to the undersigned. Byer Geotechnical appreciates the opportunity to offer our consultation and advice on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

Raffi S. Babayan
Senior Project Engineer

BYER GEOTECHNICAL, INC.

GEOLOGIC AND SOILS ENGINEERING EXPLORATION
PROPOSED TWELVE-STORY AT-GRADE MIXED-USE BUILDING
LOTS 3 - 9, BLOCK 4, TRACT 7555
650 - 674 SOUTH SAN VICENTE BOULEVARD
LOS ANGELES, CALIFORNIA
FOR STOCKDALE CAPITOL PARTNERS, LLC
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 22581
JULY 25, 2019

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LOS ANGELES, CALIFORNIA
FOR STOCKDALE CAPITOL PARTNERS, LLC
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 22581
JULY 25, 2019

INTRODUCTION

This report has been prepared per our signed Agreement and summarizes findings of Byer Geotechnical, Inc., geologic and soils engineering exploration performed on the subject site. The purpose of this study is to evaluate the nature, distribution, engineering properties, and geologic hazards of the earth materials underlying the site with respect to construction of a twelve-story at-grade mixed-use building. This report is also to respond to the City of Los Angeles, Department of Building and Safety (LADBS), Soils Report Review Letter, Log # 96658, dated February 15, 2017. This report is intended to assist in the design and completion of the proposed project and to reduce geotechnical risks that may affect the project. The professional opinions and advice presented in this report are based upon commonly accepted exploration standards and are subject to the AGREEMENT with TERMS AND CONDITIONS, and the GENERAL CONDITIONS AND NOTICE section of this report. No warranty is expressed or implied by the issuing of this report.

PROPOSED PROJECT

The scope of the proposed project was determined from consultation with Mr. Matt Porteous of Stockdale Capitol Partners, LLC, and the conceptual plans prepared by ZGF Architects, LLC, dated July 25, 2019. Final plans have not been prepared and await the conclusions and recommendations of this report. The project consists of construction of a twelve-story, at-grade mixed-use building. The ground floor will consist of retail space and building amenities. The next four levels above ground floor will consist of parking spaces. The remaining upper seven levels will consist of medical offices. Column loads (dead and live) on foundations are expected to be high. An access ramp to the parking levels is planned in the rear portion of the ground floor, adjacent to the northeast-bounding alley, via San Vicente Boulevard and Orange Street. The existing one- and two-story commercial buildings, as well as the associated parking lots and other improvements, are to be removed from the site.

PRIOR WORK - PRIOR WORK

The geotechnical files and records at the City of Los Angeles, Department of Building and Safety, for the subject property were researched and reviewed as part of the preparation of this report. The following report was located and reviewed:

> *Report of Compaction Tests, Big Five Sporting Goods, Sweetzer Street and San Vicente Boulevard, Los Angeles, California,* by **R.T. Frankian & Associates (RTFA)**, dated October 11, 1976.

This report was prepared to address the grading of the pad of the Big 5 Sporting Goods building that is located in the south portion of the subject property. The upper two feet of clay soil was removed and replaced with compacted decomposed granite. The grading extended to the limits of the existing building, as shown on the enclosed Site Plan. The RTFA compaction report was reviewed and approved by the City of Los Angeles, Department of Building and Safety (LADBS), in their Compaction Report Approval Letter, dated November 5, 1976 (enclosed in Appendix I).

Byer Geotechnical (BG) prepared the following geotechnical report in 2017:

> *Geologic and Soils Engineering Consultation and Preliminary Findings Report, Proposed Vesting Tentative Tract Map and Master Land Use Application, Proposed Medical Office Building, Lots 3 to 9, Block 4, Tract 7555, 650 - 674 South San Vicente Boulevard, Los Angeles, California,* dated January 19, 2017.

The City of Los Angeles, Department of Building and Safety (LADBS), reviewed the report and issued the Soils Report Review Letter, Log # 96658, dated February 15, 2017. In addition, Byer Geotechnical prepared the following report for the subject property:

> *Geologic and Soils Engineering Exploration, Proposed Six- to Thirteen-Story Mixed-Use Building Partially over Four Subterranean Parking Levels, Lots 3 - 9, Block 4, Tract 7555, 650 - 674 South San Vicente Boulevard, Los Angeles, California,* dated September 20, 2017.

That study included four borings that were drilled at the locations shown on the enclosed Site Plan. Recommendations were provided to aid in the design and construction of a six- to thirteen-story mixed-used building partially over four subterranean parking levels. Based on our conversation with Mr. Porteous, the September 20, 2017, report by Byer Geotechnical was not submitted to the LADBS. This report is intended to replace the September 20, 2017, report.

METRO PURPLE LINE EXTENSION

The Metro Purple Line Subway Extension is planned to be constructed under the portion of Wilshire Boulevard immediately south of the subject property. The first phase of the construction work has already commenced between the current western terminus at Wilshire/Western and continuing westward towards the intersection of Wilshire Boulevard and La Cienega Boulevard (see enclosed Metro Rail Map). Extensive subsurface exploration has been performed by several geotechnical consultants along the Purple Line Extension to assess the geotechnical characteristics of the earth materials expected to be encountered along the route. The results of the investigations are presented in a "Preliminary Geotechnical and Environmental Report" prepared by AMEC on behalf of Metro, dated December 2011.

According to the enclosed Metro Plan and Profile, the proposed subway will be between 50 and 70 feet below the existing ground surface of the portion of Wilshire Boulevard south of the site. The northern edge of the tunnel alignment is approximately 70 feet from the southern-most corner of the subject property. The earth materials that will be encountered in the nearby portion of the tunnel are expected to be similar to the earth materials underlying the subject property. These earth materials are shown on the enclosed Metro Plan and Profile and in the Log of Borings G-127 and G-128 prepared by AMEC (Appendix I).

EXPLORATION

The scope of the field exploration was originally determined from our initial site visit and consultation with representatives from Stockdale Capital Partners, LLC. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project, as shown on the enclosed Site Plan and cross sections. The scope of this exploration did not include an assessment of general site environmental conditions for the presence of contaminants in the earth materials and groundwater. Conditions affecting portions of the property outside the area explored are beyond the scope of this report.

Subsurface exploration was conducted on June 14 and July 8, 2017, with the aid of a hollow-stem-auger drill rig. It included drilling four borings (B1 - B4) to approximate depths of 61½ to 96½ feet below existing grade. Samples of the earth materials were obtained and delivered to our soils engineering laboratory for testing and analysis. The borings tailings were visually logged by the project soils engineer and project geologist. Following drilling, logging, and sampling, the borings were backfilled, mechanically tamped, and patched with asphalt.

Office tasks included laboratory testing of selected soil samples, review of published maps and photos for the area, review of our files, review of agency files, preparation of cross sections, preparation of the Site Plan, engineering analysis, and preparation of this report. Earth materials exposed in the borings are described on the enclosed Log of Borings. Appendix II contains a

discussion of the laboratory testing procedures and results. Appendix III contains the results of liquefaction analysis. The proposed project and the locations of the borings are shown on the enclosed Site Plan. Subsurface distribution of the earth materials and the proposed project are shown on Sections A through E.

SITE DESCRIPTION

The subject property consists of seven contiguous lots forming a trapezoidal-shaped, relatively-level parcel in the northern portion of the Los Angeles Basin in the Mid City West section of the city of Los Angeles, California (34.0648° N Latitude, 118.3718° W Longitude). As depicted on the enclosed Aerial Vicinity Map, the property is bounded by South San Vicente Boulevard on the southwest, an alley on the northeast, Sweetzer Avenue on the east, and Orange Street on the northwest. The property is also situated just north of Wilshire Boulevard and is located at least 70 feet from the Metro Purple Line Extension Tunnel. A two-story commercial building (Big 5 Sporting Goods), a one-story building (Montessori School), and associated parking lots and other improvements currently occupy the subject property. A storm-drain easement, containing a 12.5-foot-wide by 10-foot-high box culvert crosses the site along the boundary between Lots 4 and 5 (plan # 29305), immediately underneath the Big 5 Sporting Goods building, as shown on Section A. The surrounding area has been developed with single- and multi-story commercial buildings along San Vicente Boulevard and Wilshire Boulevard, as well as single- and multi-family residential building behind.

Past grading on the site has consisted of preparing a thin compacted-fill blanket for the existing Big 5 Sporting Goods building. Vegetation on the site is sparse and consists of planter areas and a few trees on both sides of the parking lot adjacent to Sweetzer Avenue. Surface drainage is by sheetflow runoff down the contours of the land to the south.

GROUNDWATER

Groundwater was encountered in the borings and was initially measured at approximate depths of 40 to 53 feet. The groundwater level rose to the approximate depths of 34 to 35 feet below existing grade after 30 to 45 minutes (average elevation 109). In *Seismic Hazard Zone Report 026,* the California Geological Survey (CGS) has estimated the historically-highest groundwater level at the site was on the order of 15 feet below ground surface (approximate elevation 128), as shown on the enclosed Historic-High Groundwater Map (CGS, 1998). Seasonal fluctuations in groundwater levels occur due to variations in climate, irrigation, development, and other factors not evident at the time of the exploration. Groundwater levels may also differ across the site. Groundwater can saturate earth materials causing subsidence or instability of slopes.

METHANE ZONES

The City of Los Angeles Ordinance No. 175790 established methane mitigation requirements and includes construction standards to control methane intrusion into buildings. New buildings within a Methane Zone or Methane Buffer Zone must comply with Methane Mitigation Standards established by the Superintendent of Building. The subject property is mapped within a Methane Zone.

A civil engineer experienced with methane detection and remediation should be consulted to conduct the necessary site testing and to provide a methane mitigation system in compliance with the City ordinance. The City of Los Angeles, Department of Building and Safety, has a Methane Mitigation Designer Interest List of consultants that are capable of performing the required testing and/or remediation.

EARTH MATERIALS

Compacted Fill (Cf)

Compacted fill, associated with previous site grading, underlies the existing Big 5 Sporting Goods building and is on the order of two feet thick. The compacted fill was placed under the observation and testing of RTFA, and a compaction report was prepared (see "Research" section of this report). The compacted fill reportedly consists of decomposed granite. Fill was not encountered in the borings that were located within the onsite parking lots.

Alluvium (Qa)

Natural alluvium underlies the subject site and is on the order of 30 feet thick. The alluvium consists generally of clay that is brown, dark brown, and olive-brown, moist to very moist, and medium stiff to very stiff.

Bedrock (Qsp)

The Pleistocene/Pliocene San Pedro Formation underlies the subject property and was encountered in the borings at an approximate depth of 30 feet below existing grade. The upper 25 feet consists of clay and silt that is bluish-gray to gray, moist to very moist, and stiff to very stiff with varying amounts of sand. Below, the formation consists generally of layers of sand and silty sand that is gray, moist to very moist, and medium dense to dense.

AMEC conducted a subsurface exploration in 2011 for the Metro Purple Line Extension Tunnel that is currently under construction below Wilshire Boulevard, as shown on the enclosed Site Plan. Numerous borings were explored and logged along the alignment of the tunnel. Borings G-127 and G-128 are located near the subject site, as shown on the enclosed Metro Plan and Profile. A copy of the boring logs G-127 and G-128 by AMEC is enclosed in Appendix I. Based on the field data

from the borings by AMEC, the San Pedro Formation was encountered at approximate depths of 25 to 29 feet below ground surface in the vicinity of the subject site.

GENERAL SEISMIC CONSIDERATIONS

Regional Faulting

The subject property is located in an active seismic region. Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Geological Survey (CGS), private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies now require earthquake-resistant structures. The purpose of the code seismic-design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

Southern California faults are classified as "active" or "potentially active." Faults from past geologic periods of mountain building that do not display evidence of recent offset are considered "potentially active." Faults that have historically produced earthquakes or show evidence of movement within the past 11,000 years are known as "active faults." No known active faults cross the subject property, and the property is not located within a currently-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2000). Therefore, the potential for future surface rupture onsite is considered nil.

The known regional local active and potentially-active faults that could produce the most significant ground shaking on the site include the Santa Monica, Newport-Inglewood, and Hollywood Faults. Other faults that are located near the site are the Puente Hills blind thrust and the Upper Elysian Park blind thrust. However, these faults are considered inactive (ICBO, 1998). Forty-three faults were found within a 100-kilometer-radius search area from the site using EZ-FRISK V7.65 computer program. The results of seismic-source analysis are listed in Appendix III. The closest mapped

"active" fault is the Santa Monica Fault, a Type B fault that is located 1.4 kilometers (0.9 mile) west of the site. The Santa Monica Fault is capable of producing a maximum moment magnitude of 7.4 and an average slip rate of 1.0 ± 0.5 millimeters per year (Cao et al., 2003). The Elsinore Fault, a Type A fault, is located 30.8 kilometers (19.2 miles) southeast of the site. In addition, the San Andreas Fault, another Type A fault, is located 59.1 kilometers (36.7 miles) northeast of the site. General locations of regional active faults with respect to the subject site are shown on the enclosed Regional Fault Map (Appendix III).

Seismic Design Coefficients

The following tables list the applicable City of Los Angeles Building Code seismic coefficients for the project:

SEISMIC COEFFICIENTS (Based on ASCE 7-10 Standard)		
Latitude = 34.0648° N Longitude = 118.3718° W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 20.3-1, ASCE Standard 7-10	Alluvium/San Pedro Formation - D	
Mapped Spectral Accelerations from Figures 1613.3.1 (1) and 1613.3.1 (2) and USGS	S_s = 2.121 (g)	S_1 = 0.823 (g)
Site Coefficients from Tables 1613.3.3 (1) and 1613.3.3 (2) and USGS	F_A = 1.0	F_V = 1.5
Maximum Considered Spectral Response Accelerations from Equations 16-37 and 16-38, 2013 CBC	S_{MS} = 2.121 (g)	S_{M1} = 1.235 (g)
Design Spectral Response Accelerations from Equations 16-39 and 16-40, 2013 CBC	S_{DS} = 1.414 (g)	S_{D1} = 0.823 (g)
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, adjusted for Site Class effects	PGA_M = 0.815 (g)	

SEISMIC COEFFICIENTS (Based on ASCE 7-16 Standard)		
Latitude = 34.0648° N Longitude = 118.3718° W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 20.3.3, ASCE Standard 7-16	Alluvium/San Pedro Formation - D	
Mapped Spectral Accelerations from Figures 22-1 and 22-2 and USGS	S_s = 2.072 (g)	S_1 = 0.739 (g)
Site Coefficients from Tables 11.4-1 and 11.4-2 and USGS	F_A = 1.0	F_V = Null (g)
Maximum Considered Spectral Response Accelerations from Equations 11.4-1 and 11.4-2	S_{MS} = 2.072 (g)	S_{M1} = Null (g)
Design Spectral Response Accelerations from Equations 11.4-3 and 11.4-4	S_{DS} = 1.381 (g)	S_{D1} = Null (g)
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, adjusted for Site Class effects	PGA_M = 0.977 (g)	

Reference: U.S. Geological Survey, **Geologic Hazards Science Center, U. S. Seismic Design Maps,** http://earthquake.usgs.gov/designmaps/us/application.php

Based on ASCE 7-10 Standard, the mapped spectral response acceleration parameter for the site for a 1-second period (S_1) is greater than 0.75g. Therefore, under the ASCE 7-10 Standard, the project is considered to be in Seismic Design Category E.

Based on ASCE 7-16 Standard, the mapped spectral response acceleration parameter for the site for a 1-second period (S_1) is less than 0.75g. The design spectral response acceleration parameter for the site for a 0.2-second period (S_{DS}) is greater than 0.50g. Therefore, under the ASCE 7-16 Standard, the project is considered to be in Seismic Design Category D.

The principal seismic hazard to the proposed project is strong ground shaking from earthquakes produced by local faults. Modern buildings are designed to resist ground shaking through the use of shear panels, moment frames, and reinforcement. Additional precautions may be taken, including

strapping water heaters and securing furniture to walls and floors. It is likely that the subject property will be shaken by future earthquakes produced in southern California.

Seismic Hazard Deaggregation Analysis

A probabilistic seismic hazard deaggregation analysis was performed for the subject site. Seismic parameters were determined using currently-available earthquake and fault information, utilizing data from the United States Geological Survey (USGS) Earthquake Hazards Program (USGS, 2017). An averaging of three Next Generation Attenuation relations (Chiou-Youngs, 2008; Boore-Atkinson, 2008; and Campbell-Bozorgnia, 2008) were incorporated in the analysis. An average shear-wave velocity (Vs30) of 259 meters-per-second (Site Class D) was used in the analysis. Results of the probabilistic seismic hazard deaggregation analysis are shown in the following table:

Probabilistic Seismic Hazard Deaggregation Analysis		
Latitude = 34.0648° N Longitude = 118.3718° W	Percent Probability of Exceedance in 50 Years	
Shear-Wave Velocity = 259 Meters-per-Second	10%	2%
Return Period	475 Years	2,475 Years
Magnitude of the Predominant Earthquake (Mw)*	6.51	6.51
Distance to the Seismic Source (Km)*	6.46	5.26

* Modal Values (Largest r-m Bin)

Reference: U.S. Geological Survey, 2017, Beta-Unified Hazard Tool, http://earthquake.usgs.gov/hazards/interactive/index.php

Results of the analysis are graphically presented in the enclosed Seismic Hazard Deaggregation Charts 1 and 2 (Appendix III).

Based on a Site Class D, the MCE_G peak ground acceleration adjusted for Site Class effects, PGA_M, is 0.977g using ASCE 7-16 Standard. The pseudo-static seismic coefficient (k_h) was derived according to LADBS memorandum dated July 16, 2014. The horizontal pseudo-static seismic

coefficient (k_h) was selected as one-third of the PGA_M (0.33g) and was used in the seismic calculations for the cantilever and restrained retaining walls. These ground motions could occur at the site during the life of the project.

Site-Specific Ground Motion Analysis

Site-specific ground motion analysis was performed in accordance with Chapter 21 of the American Society of Civil Engineers (ASCE) Standard 7-16. The probabilistic and deterministic seismic response spectra, based on maximum rotated component of spectral response at five-percent damping, are enclosed. The analysis is also based on a probability of exceedance of two percent in 50 years (2,475-return period). A computerized program, EZ-FRISK V7.65, was used to generate the seismic response spectra. An averaging of three Next Generation Attenuation relations (Chiou-Youngs 2007 NGA USGS 2008 MRC; Boore-Atkinson 2008 NGA USGS 2008 MRC; and Campbell-Bozorgnia 2008 NGA USGS 2008 MRC) was incorporated in both the probabilistic and deterministic analyses to estimate ground motions at the subject site. The deterministic response spectrum was generated using the 84th percentile of the maximum rotated component of spectral response at five-percent damping. A shear-wave velocity (Vs30) of 259 meters-per-second (Site Class D) was used in the analysis.

The design response spectrum was generated by multiplying the lesser of the deterministic and probabilistic response spectra by two-thirds, according to Sections 21.2.3 and 21.3 of ASCE Standard 7-16. The deterministic lower-limit response spectrum was determined according to Section 21.2.2 of the ASCE Standard 7-16. Spectral response accelerations for selected periods are shown in the following table:

Spectral Response Accelerations (g)*									
	Fundamental Period (seconds)								
	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Probabilistic MCE_R	1.9878	1.9797	1.9160	1.8335	1.6801	1.5590	1.4352	1.3132	1.2154
Probabilistic (ASCE 7-16)	1.3813	1.3813	1.3813	1.3813	1.3813	1.3813	1.3813	1.3685	1.2317
Deterministic MCE_R (84^{th} Percentile)	1.5330	1.6560	1.6980	1.7030	1.6220	1.5560	1.4610	1.3450	1.2470
Deterministic Lower Limit on MCE_R Response Spectrum	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000
80% Design Response Spectrum	1.1050	1.1050	1.1050	1.1050	1.1050	1.1050	1.1050	1.0950	0.9850
Site-Specific Design Response Spectrum	**1.1050**	**1.1050**	**1.1320**	**1.1350**	**1.1050**	**1.1050**	**1.1050**	**1.0950**	**0.9850**

* Reference: *American Society of Civil Engineers (ASCE), Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Standard 7-16, 2016.*

The data included in the table above are plotted and presented in the enclosed Site-Specific Seismic Response Spectra figure (see Appendix III). Detailed calculations for fundamental periods up to eight seconds are also included in the "Site-Specific Ground Motion Analysis" table (see Appendix III).

As shown on the enclosed Site-Specific Seismic Response Spectra figure, the site-specific design response spectrum is equal or greater than or equal to 80 percent of the probabilistic response spectrum. According to Section 21.3 of ASCE Standard 7-16, the design response spectrum shall not be less than 80 percent of the probabilistic response spectrum.

Based on Section 21.4 of the ASCE Standard 7-16, the design earthquake spectral response acceleration parameters at short period, $S_{DS,}$ and at one-second period, S_{D1}, derived from the site-specific ground motion analysis, are 1.105g and 0.985g, respectively.

Liquefaction

The CGS has mapped the site within an area where historic occurrence of liquefaction or geological, geotechnical, and groundwater conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693 (c) would be required (CGS, 1999), as shown on the enclosed Seismic Hazard Zones Map (Appendix III).

Liquefaction is a process that occurs when saturated alluvial sediments are subjected to repeated strain reversals during an earthquake. The strain reversals cause increased pore water pressure such that the internal pore pressure approaches the overburden stress and the shear strength approaches zero. Liquefied soils may be subject to flow or excessive strain, which may induce settlement. Liquefaction occurs in soils below the groundwater table. Soils commonly subject to liquefaction include loose to medium-dense sand and silty sand. Predominantly fine-grained soils, such as silts and clay, are less susceptible to liquefaction. Generally, medium dense to dense sand-like soils with fines content (percent passing the No. 200 sieve) greater than 35 percent are not considered susceptible to liquefaction. In addition, soils older than 11,500 years, such as the San Pedro Formation, are not considered to be susceptible to liquefaction. In addition, cohesive soils with Plasticity Index (PI) values between 12 and 18 and a saturated moisture content less than 80 percent of the Liquid Limit (LL) are not considered susceptible to liquefaction (CGS, 2008, and Bray and Sancio, 2006). Cohesive soils with PI greater than 18 may be susceptible to liquefaction, if considered sensitive (CGS, 2008). Soil sensitivity is the ratio of the undisturbed shear strength of a cohesive soil to the remolded shear strength at the same water content (Bowles, 1997). Based on the study conducted by Bray and Sancio on soils affected by the 1999 earthquakes in Taiwan and Turkey, soils with a PI greater than 18 tested at low confining effective stresses are not considered susceptible to liquefaction (Bray and Sancio, 2006).

Soils data collected in Boring B1 was utilized to quantify the liquefaction potential of the site. In order to satisfy the requirements of the LADBS, liquefaction analyses were performed based on the following two criteria:

Liquefaction Analysis Input Parameters		
	Criteria 1	Criteria 2
Peak Ground Acceleration (g) (Based on ASCE 7-16 Standard)	0.651 (⅔ PGA_M)	0.977 (PGA_M)
Probability of Exceedance in 50 Years	10%	2%
Return Period	475 Years	2,475 Years
Earthquake Magnitude (Mw)	6.51	6.51
Factor of Safety	1.1	1.0

Reference: LADBS, letter to *Geology and Soils Engineering Firms Practicing in the City of Los Angeles,* dated July 16, 2014.

For a conservative analysis, it was assumed that groundwater rose to the historic-high groundwater level of 15 feet below the ground surface (elevation 109, see "Groundwater" section of this report).

Laboratory testing consisting of Atterberg Limits (ASTM D 4318-10) and sieve analysis by wash method (ASTM D 1140-14) was performed on representative samples of the alluvium collected in Boring B1. The purpose of these tests was to determine the liquid limit, plasticity index (PI), and fines content (percent passing the No. 200 sieve) and incorporate the results in the liquefaction analysis. The results are shown on the Laboratory Testing program in Appendix II, as well as on the enclosed liquefaction calculations (Appendix III).

A liquefaction potential analysis based upon SPT data from Boring B1 is presented in Appendix III on the plates entitled "Liquefaction Susceptibility Analysis: SPT Method." The column labeled "Factor of Safety" lists the calculated safety factor of each 2½-foot-thick layer of soil encountered in the upper 70 feet in Boring B1, and each 5-foot-thick layer of soil encountered between the depths of 70 and 95 feet. In addition, a borehole diameter correction factor (C_B) of 1.15 was incorporated in the analysis to account for the stress relief since the tip of the auger was raised a few inches from the bottom of the hole prior to driving the sampler. The stresses and safety factors for liquefaction were

calculated using the methodology of Youd et al. (2001) and Special Publication 117A (CGS, 2008). Soils with a factor of safety less than 1.1 are considered susceptible to liquefaction.

Based on the results of Atterberg Limits testing conducted on several fine-grained alluvium samples obtained from Boring B1, the plasticity index (PI) of the upper 17½ feet of the clay layers ranges from 18.4 to 27.1 percent. Quantitative evaluation and screening analysis was performed based on Criteria 1 to determine the depths and limits of potentially-liquefiable alluvial layers encountered in Boring B1 below the historic-high groundwater level. The results are summarized in the following table:

Results of Quantitative Evaluation and Screening Analysis (Based on LADBS Criteria 1)										
Boring No.	Layer Depth (feet)	Liquid Limit LL (%)	Plastic Limit PL (%)	Plasticity Index PI (%)	Fines Content (%)	Soil Type & Unit	Moisture Content w_c (%)	$(N_1)_{60cs}$	Screening Criteria (SP 117A, 2008, & Bray 2006)	Result
B1	15.0	37.4	19.0	18.4	60.8	Clay (CL)	28.5	19.2	PI > 18, Not Sensitive	Non-Liquefiable
B1	17.5	49.3	30.7	18.6	89.2	Silt (ML)	21.3	23.1	PI > 18, Not Sensitive	Non-Liquefiable
B1	20.0	-	-	-	84.6	Silt (ML)	26.7	29.6	CRR > CSR	Non-Liquefiable
B1	22.5	31.0	17.0	14.0	68.8	Clay (CL)	23.8 Saturated	25.3	w_c/ LL ≤ 0.8	Non-Liquefiable
B1	25.0	-	-	-	72.4	Silt (ML)	16.0	38.0	CRR > CSR	Non-Liquefiable
B1	27.5	28.6	18.1	10.5	65.4	Clay (CL)	18.8	18.1	PI< 12	Liquefiable

Laboratory testing was conducted on a representative undisturbed sample of the clay soil obtained from Boring B4 at the depth of 22½ feet. This sample exhibits similar behavior to the SPT sample obtained from Boring B1 at the same depth, as evidenced by the results of Atterberg Limits and sieve wash analysis (see Plasticity Charts #5 and #6). In addition, a moisture and density test was conducted on the sample from Boring B1 at the same depth, and the results indicate a moisture of 20.9 percent, with a 100-percent degree of saturation. Therefore, compared to Boring B4, it is reasonable to conclude that the moisture content of the clay soil encountered in Boring B1 at the depth of 22½ feet represents the saturated condition of that layer.

Based on the results of sieve analysis, Atterberg Limits, and saturated moisture content, the alluvium layers encountered between the depths of 15 and 25 are not susceptible to liquefaction.

Therefore, based on the results of liquefaction analysis using Criteria 1, the 2½-foot-thick soil layer encountered at the depth of 27 ½ feet is considered potentially susceptible to liquefaction. Based on Criteria 2, two 2½-foot-thick soil layers, encountered at the depths of 20 and 27½ feet, are considered potentially susceptible to liquefaction.

Dynamic Settlement

Earthquake-induced volumetric strain and dissipation of pore pressure in saturated silts and sands after liquefaction can result in settlement. The potential for liquefaction-induced settlement was calculated using the methodology of Tokimatsu and Seed (1987). The seismic settlement potentials were calculated for the soil layers below the historic-high groundwater level (15 feet below existing grade) and with a factor of safety for liquefaction of less than 1.1 for Criteria 1 and less than 1.0 for Criteria 2, as described in the "Liquefaction" section above. The results are shown in the following table.

Results of Dynamic Settlement Calculations		
	Criteria 1 (inch)	Criteria 2 (inch)
Total Dynamic Settlement	0.71	1.2
Differential Dynamic Settlement (one-half to two-thirds of total dynamic settlement)	0.36 - 0.47	0.6 - 0.8

According to the LADBS, the dynamic settlement calculated based on Criteria 2 is intended to evaluate the deformation of the proposed foundation system so that the proposed building will not lose its ability to carry gravity loads, and collapse of the structure will be prevented.

Lateral Spreading Hazard

Liquefied soils may be subject to lateral spreading flow failure where adjacent to slopes or "free-faces" such as steep slopes or embankments. The subject property is remote to free-faces, slopes, and canals, and a lateral spreading flow failure is not indicated for the potentially-liquefiable alluvial soils. Therefore, it is the opinion of Byer Geotechnical, Inc., that the lateral spreading hazard at the site is considered nil and no mitigation as defined in Public Resources Code Section 2693(c) is required for lateral spreading.

Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water, such as lakes and reservoirs, in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. The site is not located near any lake or reservoir. In addition, the site is at an average elevation of 144 feet above mean sea level and is located approximately 8 miles from the shoreline. Therefore, the risk to the project from seiches or tsunamis is considered nil.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon review of the preliminary plans, review of published maps, four borings, research of available records, laboratory testing, engineering analysis, and years of experience performing similar studies on similar sites. It is the finding of Byer Geotechnical, Inc., that development of the proposed project is feasible from a geologic and soils engineering standpoint, provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The recommended bearing material is the natural alluvium and San Pedro Formation bedrock (bedrock). Bedrock was encountered during the field exploration at an approximate depth of 30 feet below ground surface. A deepened foundation system consisting of friction piles is recommended to support the proposed building. The friction piles should be embedded a minimum of 10 feet into the bedrock. Both the alluvium and San Pedro Formation bedrock may be used to provide skin friction support. Soils to be exposed at finished grade are expected to exhibit a low to moderate expansion potential.

Piles should be located a minimum distance of eight-pile diameters from the existing storm drain. Piles located closer than eight-pile-diameters on-center from the existing storm drain should have the upper portion of the piles, measured from the bottom of the storm-drain structure, placed within a larger diameter steel casing to prevent the imposition of a lateral surcharge on the storm drain structure from piles resisting lateral loads.

Groundwater should be anticipated in friction-pile excavations. Casing should be anticipated for friction-pile excavations.

The concrete slab-on-grade should be cast over three feet of approved compacted fill or the slab may be structurally designed to bridge between the friction piles. Grading should be accomplished prior to the commencement of friction pile excavations.

SITE PREPARATION - REMOVALS

Surficial materials consisting of disturbed alluvium and decomposed granite are present on the site. Remedial grading is recommended to improve site conditions and prepare a firm compacted-fill pad for the slab-on-grade of the proposed building. The decomposed granite and upper three feet of alluvium should be removed and replaced as certified compacted fill. The following general grading specifications may be used in preparation of the grading plan and job specifications. Byer Geotechnical would appreciate the opportunity of reviewing the plans to ensure that these recommendations are included. The grading contractor should be provided with a copy of this report.

A. The area to receive compacted fill should be prepared by removing all vegetation, demolition debris, existing decomposed granite, and upper three feet of alluvium. The exposed excavated area should be observed by the soils engineer/geologist prior to placing compacted fill. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 90 percent of the maximum dry density.

B. The area of the proposed building shall be excavated to a minimum depth of three feet below finished subgrade. The excavated areas shall be observed by the soils engineer/geologist prior to placing compacted fill.

C. Fill, consisting of soil approved by the soils engineer, shall be placed in horizontal lifts, moistened as required, and compacted in six-inch layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks larger than six inches in diameter shall not be used in the fill.

D. The moisture content of the fill should be near the optimum moisture content. When the moisture content of the fill is too wet or dry of optimum, the fill shall be moisture conditioned and mixed until the proper moisture is attained.

E. The fill shall be compacted to at least 90 percent of the maximum laboratory dry density for the material used. The maximum dry density shall be determined by ASTM D 1557-12 or equivalent.

F. Field observation and testing shall be performed by the soils engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until 90 percent relative compaction is obtained. A minimum of one compaction test is required for each 500 cubic yards or two vertical feet of fill placed.

FOUNDATION DESIGN

Friction Piles - Axial Capacity

Cast-in-place, concrete friction piles are recommended to support the proposed building to achieve embedment in the San Pedro Formation bedrock and to avoid surcharging the existing storm-drain. Piles should be a minimum of 24 inches in diameter and a minimum of 10 feet into the San Pedro Formation, which is anticipated at a depth of 30 feet below existing grade (elevation 113). Piles may be assumed fixed into the alluvium below the bottom of the existing storm-drain easement at the point of fixity depths indicated in Table 1, titled "Results of Lateral Pile Analysis" below. The piles may be designed per the enclosed allowable skin friction calculations (see Calculation Sheets #1 - #4, Appendix III) for that portion of pile in contact with the alluvium and San Pedro Formation. Piles spaced more than three-pile diameters center-to-center may be considered isolated for axial capacity. The axial capacity of piles placed in a group closer than three-pile diameters should be reduced. The pile-group-efficiency factor shown in the following table should be applied for the total axial capacity of the individual piles:

Pile Group Efficiency Table - Axial Capacity			
Pile Spacing (in pile diameter "D")	Group Efficiency*		
	2 Piles	3 Piles	4 Piles
2.50 D	88%	80%	76%
2.25 D	87%	78%	73%
2.00 D	85%	75%	70%
1.75 D	83%	72%	67%
1.50 D	81%	69%	63%
1.25 D	79%	64%	57%

** Reference: Converse-Labarre Equation, Bowles, Foundation Analysis and Design,1997.*

Groundwater should be expected in the pile excavations and should be pumped out, or the water may be displaced by pumping concrete from the bottom with a hose. The tip of the hose shall be kept at least five feet below the concrete surface during pumping. When concrete is placed below water, the mix should be adjusted to achieve at least 1,000 pounds-per-square-inch more than the required strength.

Friction Piles - Lateral Design

The skin friction values are for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. Resistance to lateral loading may be provided by passive earth pressure within the alluvium and the San Pedro Formation bedrock.

Passive earth pressure may be computed as an equivalent fluid having a density of 250 pounds-per-cubic-foot for the alluvium and 400 pounds-per-cubic-foot for the San Pedro Formation bedrock. The maximum allowable earth pressure is 6,000 pounds-per-square-foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles

spaced more than eight-pile diameters center-to-center may be considered isolated for lateral capacity in the direction of the lateral movement.

Shear force and bending moment analyses are provided for lateral displacements of one-fourth and one-half of an inch. Twenty-four, 30-, 36-, and 42-inch-diameter piles were analyzed using fixed-head and free-head conditions. The results of lateral pile-load analyses are included in Appendix III. The results are for a single, isolated pile under short-term lateral loading, with no load reduction applied for pile group effect. The analyses were performed using *LPILE 2013.7.07* software by Ensoft, Inc. (Reese and Wang, 1997), which solves the beam of an elastic foundation problem using independent nonlinear lateral springs, commonly referred to as p-y curves, to model the relationship between soil resistance and pile deflection. Bending stiffness of these cast-in-place concrete piles was modeled using the gross section stiffness for 3,000 pounds-per-square-inch concrete. Pile length and shear force estimates for the specified lateral displacements and loading conditions are presented in the following tables:

TABLE 1: RESULTS OF LATERAL PILE ANALYSIS (SINGLE PILE)

Pile Diameter (inches)	Lateral Deflection (inches)	Fixed-Head Loading Condition (Figures 1, 2, 3, & 4)			Free-Head Loading Condition (Figures 5, 6, 7, & 8)		
		Maximum Shear Force (kips)	Maximum Bending Moment (in-kips)	Point of Fixity (feet)	Maximum Shear Force (kips)	Maximum Bending Moment (in-kips)	Point of Fixity (feet)
24	0.25	90.3	4,689	9.0	37.3	1,722	6.0
	0.50	138.3	7,988	10.0	56.2	2,966	6.5
30	0.25	138.7	8,384	10.5	58.1	3,065	7.0
	0.50	212.1	14,208	11.5	87.5	5,267	8.0
36	0.25	198.3	13,528	12.0	83.6	4,914	7.5
	0.50	300.0	22,792	13.0	125.2	8,463	8.5
42	0.25	269.4	20,327	13.5	113.6	7,286	8.5
	0.50	404.1	34,088	14.5	170.2	12,685	9.5

* Point of fixity is measured from the base of pile cap. All piles should be embedded a minimum of 10 feet into San Pedro Formation bedrock (see "Friction Piles - Axial Capacity" section of this report).

The values shown on Table 1 above are based on embedment into the alluvium and San Pedro Formation bedrock below the pile cap.

The lateral capacity of piles placed in a group closer than eight-pile diameters should be reduced. The pile-group-efficiency factor shown in the following table should be applied for the total lateral capacity of the individual piles:

Pile Group Efficiency Table - Lateral Capacity	
Pile Spacing (in pile dimension "D")	Group Efficiency
7 D	85%
6 D	80%
5 D	70%
4 D	60%
3 D	40%

Piles should also be located a minimum distance of eight-pile diameters from the existing storm drain. For piles located closer than eight-pile-diameters on-center from the existing storm drain, the upper portion of the piles, measured from the bottom of the storm-drain structure, should be placed within a larger diameter steel casing to prevent lateral surcharge on the storm drain structure. The inside diameter of the steel casing should be larger than the diameter of the friction pile to allow for lateral movement.

Downdrag Force

The friction piles will extend through potentially-liquefiable soil layers and may be subject to downdrag forces (negative skin friction) along the shaft, caused by the downward movement of the soil above the potentially-liquefiable soil layers as a result of strong ground shaking. The downdrag force may be estimated by assuming a negative skin friction over the surface area of the pile for that portion above the potentially-liquefiable soil layer. As a result, the downdrag force should be applied to that portion of the pile from the pile cap at the top, down to the bottom of the liquefiable soil layer located at a depth of 27½ feet (approximate elevation of 115.5). A negative skin friction

of one-half the downward skin friction, shown on the enclosed "Allowable Pile Axial Capacity" charts (Appendix III), may be used to determine the downdrag force on the piles.

Uplift Forces

Uplift forces on piles may result during a seismic event. To avoid the uplift effect, the piles may be designed to resist uplift. The allowable uplift skin resistance should be taken as one-half the allowable downward skin friction shown on the enclosed Allowable Pile Axial Capacity Charts (Appendix III).

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A maximum settlement of 0.5 inch is anticipated for the friction piles. Differential static settlement should not exceed 0.25 inch across the footprint of the proposed building.

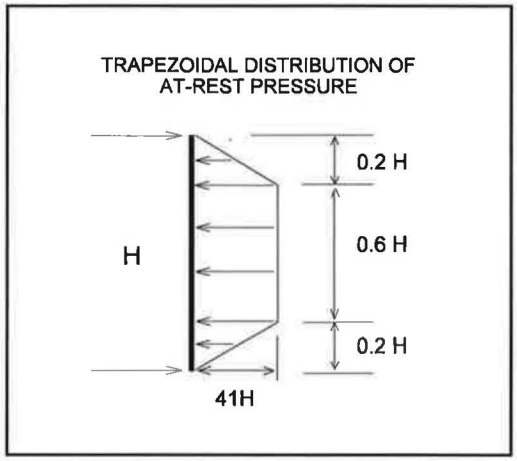
The proposed building will be supported on friction piles to be embedded into the San Pedro Formation bedrock. Therefore, dynamic settlement due to potentially liquefiable alluvial soil layers is not expected for the foundation system.

RETAINING WALLS

General Design

Cantilever retaining walls for the elevator pits up to 12 feet high, with a level backslope and uniform vehicular surcharge of 300 pounds, may be designed for an active equivalent fluid pressure of 43 pounds-per-cubic-foot (see Calculation Sheet #5). Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾-inch crushed gravel.

Elevator pit retaining walls, if restrained, should be designed for an at-rest lateral earth pressure of 41H, where H is the height of the wall. The diagram illustrates the trapezoidal distribution of earth pressure. The design earth pressures assume that the walls are free draining.



Seismic analysis of the proposed cantilever and restrained retaining walls indicates that no additional loading due to seismic forces is required, since the calculated seismic thrust is less than the static active and at-rest design thrusts for a retained height of 12 feet (see Calculation Sheet #6).

Normally, a subdrain system is not placed behind the elevator pit walls. If a subdrain system is omitted behind the elevator-pit walls, these walls should be designed to resist hydrostatic pressure, in addition to the active or at-rest lateral pressures. For a hydrostatic design, restrained retaining walls may be designed for the at-rest lateral earth pressure of 21H (trapezoidal distribution), where H is the height of the wall. This at-rest pressure is calculated based on the buoyant unit weight of the earth materials behind the walls. An additional pressure of 62.4 pounds-per-cubic-foot (triangular distribution) should be applied for design of the hydrostatic case.

Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557-12, or equivalent. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with ¾-inch crushed gravel to within two feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper two feet of backfill above the gravel should consist of a

compacted-fill blanket to the surface. Restrained walls should not be backfilled until the restraining system is in place.

Foundation Design

Elevator pit retaining walls should be supported on friction piles embedded into the bedrock.

Retaining Wall Deflection

It should be noted that non-restrained retaining walls can deflect up to one percent of their height in response to loading. This deflection is normal and results in lateral movement and settlement of the backfill toward the wall. The zone of influence is within a 1:1 plane from the bottom of the wall. Hard surfaces or footings placed on the retaining wall backfill should be designed to avoid the effects of differential settlement from this movement. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill.

TEMPORARY EXCAVATIONS

Temporary excavations will be required during grading to prepare a compacted-fill pad to support the slab-on-grade of the proposed building. The excavations are expected to be up to three feet in height and will expose alluvium. The alluvium is capable of maintaining vertical excavations up to five feet. Where vertical excavations exceed five feet in height, the upper portion should be trimmed to 1:1 (45 degrees).

The geologist should be present during grading to see temporary slopes. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the

excavations nor to flow toward them. No vehicular surcharge should be allowed within three feet of the top of the cut.

FLOOR SLABS

The concrete slab-on-grade should be cast over future compacted fill or the slab may be structurally designed to bridge between the friction piles. The slab should be at least five inches thick and reinforced with a minimum of #4 bars on 16-inch centers, each way. Slabs that will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. The barrier should be sandwiched between the layers of sand, about two inches each, to prevent punctures and aid in the concrete cure. A low-slump concrete may be used to minimize possible curling of the slab. The concrete should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

It should be noted that cracking of concrete slabs is common. The cracking occurs because concrete shrinks as it cures. Control joints, which are commonly used in exterior decking to control such cracking, are normally not used in interior slabs. The reinforcement recommended above is intended to reduce cracking and its proper placement is critical to the performance of the slab. The minor shrinkage cracks, which often form in interior slabs, generally do not present a problem when carpeting, linoleum, or wood floor coverings are used. The slab cracks can, however, lead to surface cracks in brittle floor coverings such as ceramic tile.

EXTERIOR CONCRETE DECKS

Decking should be cast over undisturbed alluvium or approved compacted fill and reinforced with a minimum of #3 bars placed 18 inches on center, each way. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space

between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill. The subgrade should be moistened prior to placing concrete.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street or approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

Low-Impact Development (LID) Requirements

Typically, infiltration systems are utilized in areas underlain by pervious granular earth materials that have high percolation characteristics. In addition, infiltration systems are normally planned at least 10 feet from adjacent property lines or public right-of-way, and 10 feet from a 1:1 plane projected from the bottom of adjacent structural foundations. However, since the site is located within a liquefaction zone and due to the presence of potentially liquefiable soil layers, onsite infiltration is not recommended.

As an alternative, a biofiltration system, a capture-and-reuse system, or equivalent, may be installed on the site in accordance with the City of Los Angeles Best Management Practices (City of Los Angeles, 2011). A planter box may be used to capture and treat storm-water runoff through different soil layers before discharging water to the street storm drain. The planter box should be an impermeable rigid structure that is equipped with an underdrain to prevent water infiltration to the underlying subsurface earth materials. Planter boxes may be situated aboveground and placed adjacent to buildings. Planter boxes should be designed as freestanding and for an inward equivalent fluid pressure of 43 pounds-per-cubic-foot. This fluid pressure includes possible vehicular

surcharge. Byer Geotechnical, Inc., should be provided with the final plans to verify the location of the planter boxes.

Irrigation

Control of irrigation water is a necessary part of site maintenance. Soggy ground and perched water may result if irrigation water is excessively applied. Irrigation systems should be adjusted to provide the minimum water needed. Adjustments should be made for changes in climate and rainfall.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage, and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Equally important is the use of a subdrain that daylights to the atmosphere. The subdrain should be covered with ¾-inch crushed gravel to help the collection of water. Landscape areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

PLAN REVIEW

Formal plans ready for submittal to the building department should be reviewed by Byer Geotechnical. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The building department requires that the geotechnical engineer provide site observations during grading and construction. Foundation excavations should be observed and approved by the geotechnical engineer or geologist prior to placing steel, forms, or concrete. The engineer/geologist should observe bottoms for fill, compaction of fill, temporary excavations, and subdrains. All fill

that is placed should be approved by the geotechnical engineer and the building department prior to use for support of structural footings and floor slabs.

Please advise Byer Geotechnical, Inc., at least 24 hours prior to any required site visit. The building department stamped plans, the permits, and the geotechnical reports should be at the job site and available to our representative. The project consultant will perform the observation and post a notice at the job site with the findings. This notice should be given to the agency inspector.

FINAL REPORTS

The geotechnical engineer will prepare interim and final compaction reports upon request. The geologist will prepare reports summarizing pile excavations.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. The area should be fenced and warning signs posted. All excavations must be covered and secured. Soil generated by foundation excavations should be either removed from the site or placed as compacted fill. Soil should not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep. Water shall not be allowed to saturate open footing trenches.

GENERAL CONDITIONS AND NOTICE

This report and the exploration are subject to the following conditions. Please read this section carefully; it limits our liability.

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by Byer Geotechnical, Inc., and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein have been projected from test excavations on the site and may not reflect any variations that occur between these test excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications, and recommendations requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report, issued and made for the sole use and benefit of the client, is not transferable. Any liability in connection herewith shall not exceed the Phase I fee for the exploration and report or a negotiated fee per the Agreement. No warranty is expressed, implied, or intended in connection with the exploration performed or by the furnishing of this report.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

Byer Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

Raffi S. Babayan
P. E. 72168

Robert I. Zweigler
E. G. 1210/G. E. 2120

RSB:RIZ:mh
F:\22581_Stockdale_Capitol_Geo_and_Soils_7.25.19.wpd

ENCLOSURES AND DISTRIBUTION

Enc: List of References (2 Pages)
LADBS, Soils Report Review Letter, dated February 15, 2017 (2 Pages)
Appendix I - Research
- LADBS, conditional approval letter, dated November 5, 1976
- Metro Plan and Profile
- AMEC, Log of Borings G-127 and G-128 (6 Pages)

Appendix II - Laboratory Testing and Log of Borings (Current Study)
- Laboratory Testing (3 Pages)
- Shear Test Diagrams (2 Pages)
- Consolidation Curves (9 Pages)
- Plasticity Charts (7 Pages)
- Log of Borings B1 - B4 (14 Pages)

Appendix III - Calculations and Figures
- Seismic Sources (2 Pages)
- Seismic Hazard Deaggregation Charts 1 and 2 (2 Pages)
- Site-Specific Ground Motion Analysis (2 Pages)
- Liquefaction Susceptibility Analysis: SPT Method (4 Pages/Sheets)
- Allowable Pile Axial Capacity Charts #1 - #4 (4 Pages)
- Lateral Pile Capacity Charts (16 Pages)
- Retaining Wall Calculation Sheets #5 and #6 (2 Pages)
- Aerial Vicinity Map
- Regional Topographic Map
- Historic Topographic Map
- Regional Geologic Map
- Regional Fault Map
- Seismic Hazard Zones Map
- Historic-High Groundwater Map

In Pocket: Site Plan
Sections A through E (3 Sheets)

xc: (4) Addressee (Email and BG to Submit)

REFERENCES

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REFERENCES (Continued)

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Software

EZ-FRISK 7.65, Risk Engineering, Inc.
LPILE PLUS version 2016-09.001, Ensoft, Inc., Reese and Wang, 1997.

CITY OF LOS ANGELES
CALIFORNIA

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

FRANK M. BUSH
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

OSAMA YOUNAN, P.E.
EXECUTIVE OFFICER

CITY OF LOS ANGELES
INTER-DEPARTMENTAL CORRESPONDENCE

SOILS REPORT REVIEW LETTER

February 15, 2017

LOG # 96658
SOILS/GEOLOGY FILE - 2
LIQ

To: Jim Tokunaga, Deputy Advisory Agency
Department of City Planning
200 N. Spring Street, 7th Floor, Room 750

From: Jesus Adolfo Acosta, Grading Division Chief
Department of Building and Safety

Tentative Tract: 74865
LOT(S): 3-9
LOCATION: 650-674 S. San Vicente Blvd

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Geology Report	22581	01/19/2017	Byer Geotechnical, Inc.

The Grading Division of the Department of Building and Safety has reviewed the Tentative Tract 74865 with Los Angeles Department of City Planning receipt stamp dated 01/27/2017 and the referenced report that provide recommendations for the proposed 11 story medical office over a basement area(up to 4 levels below grade).

The earth materials at the subsurface exploration locations consist of up to 15 feet of uncertified fill underlain by native.

The site is located in a designated liquefaction hazard zone as shown on the Seismic Hazard Zones map issued by the State of California.

The review of the subject report 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 2017 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 a complete soils report that is signed and stamped by a soils engineer.

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 a CD or flash drive, and the appropriate fees will be required for submittal.

AD

AD/ad
Log No. 96658
213-482-0480

cc: SSV Property Owner LLC, Owner
Byer Geotechnical, Inc., Project Consultant
LA District Office

July 25, 2019
BG 22581

APPENDIX I

Research

CITY OF LOS ANGELES
CALIFORNIA

R. J. WILLIAMS
GENERAL MANAGER

DEPARTMENT OF
BUILDING AND SAFETY
402, CITY HALL
LOS ANGELES, CALIF. 90012

TOM BRADLEY
MAYOR

Mr. & Mrs. M. Gaskin
c/o Mr. Rick Faber, A.I.A.
9601 Wilshire Blvd.
Mezzanine, Suite 101
Beverly Hills, CA 90210

TRACT: 7935
LOT: 3 - 6
LOCATION: 676 S. San Vicente Blvd.
(aka 9601 Wilshire Blvd.)

DATE: November 1, 1976
PERMIT: LA 35394-76
DM# 3456

Fill soil classification, per Table 28-A: Lean Clay

Lots having compacted fill: 3 - 6

Approval is granted for compacted fill constructed on the above lots as described in the compaction report dated October 11, 1976, prepared by R. T. Frankian and Associates Report No. 25171-F.

Approval is limited to the area shown in the report and by the following requirements:

A. Footings for one-story wood-frame structures may be dimensioned from Table 17-B without use of the soil bearing value.
B. Footing bearing pressure for all other structures shall not exceed a value of 2,000 lbs. per sq. ft. at 12 inches minimum, below approved compacted surface. into approved bearing soil.
C. Continuous footings per Code Section 91.3012 are required.
D. Dwelling foundations located partially or wholly upon compacted fill ground shall meet the requirements of Section 91.3012.
E. Slope erosion control, planting, and irrigating of fill slopes, and runoff control are required as per Code Section 91.3007.
F. Building or structure footings shall be set back 5 feet from the face of slopes 20 feet or less in vertical height where the slope angle is between 1½ horizontal to 1 vertical and 2 horizontal to 1 vertical. Where the vertical height of slope exceeds 20 feet and the slope angle is as described above, the set back shall be increased 1 foot for each additional 5 feet in vertical height over 20 feet to a maximum set back of 10 feet. For slopes exceeding 100 feet in vertical height, the set back shall be 40 feet except as permitted in Code Section 91.3009(c).
• See Foundation Investigation Report of Jan. 14, 1974, and Board File # 7099-6

R.J. WILLIAMS
General Manager

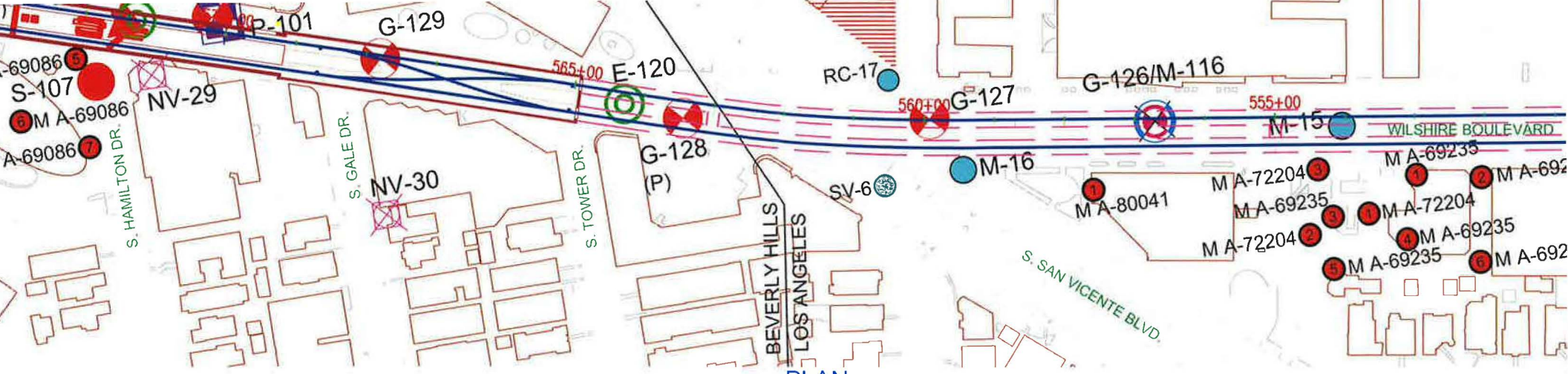
By R

Jack Raymond
Grading Engineer
485-3435 lr

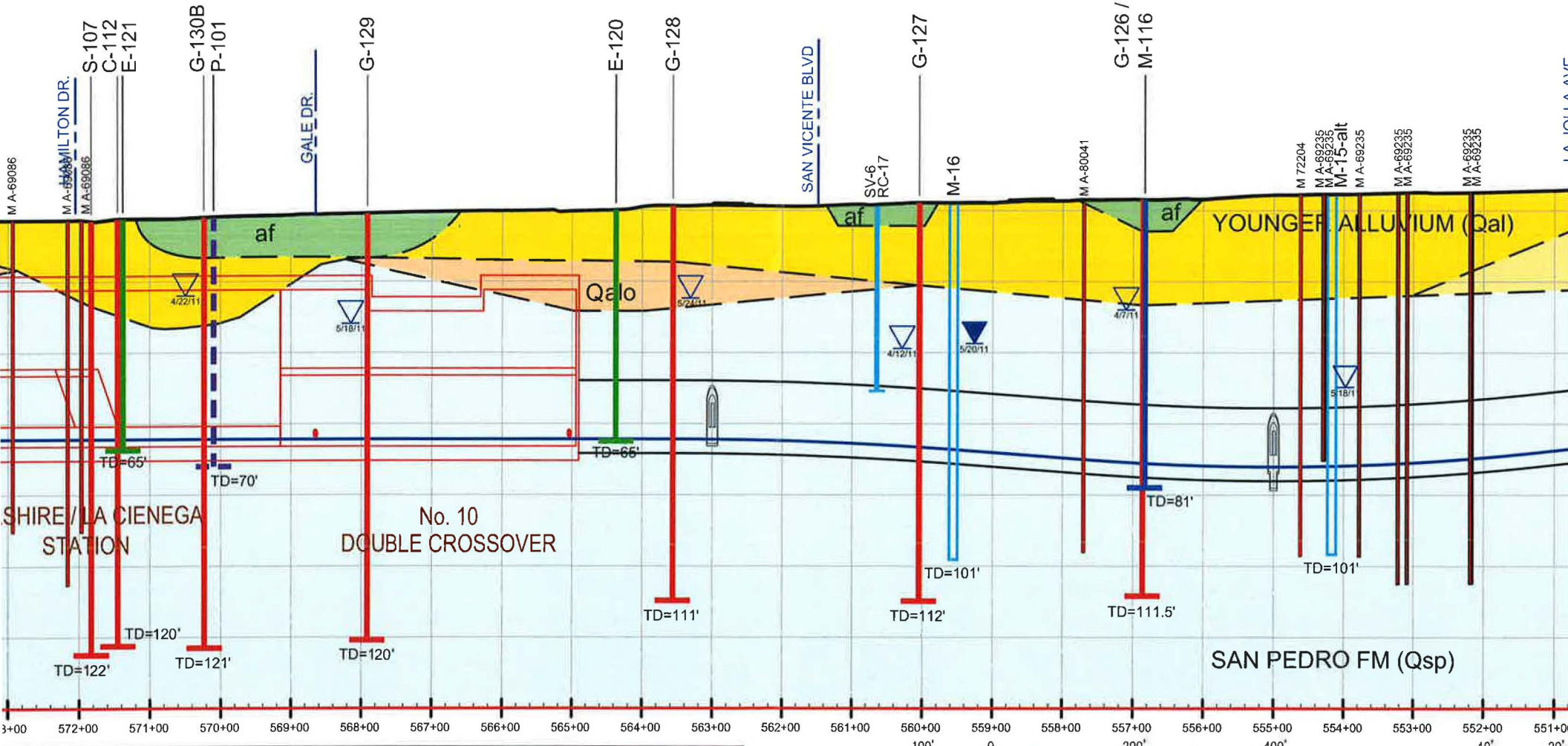
cc: R. T. Frankian and Assoc.
LA Insp-Cruz

B&S B-142 R11.74

0001700001



PLAN



METRO PLAN AND PROFILE

DRILLING COMPANY/DRILLING EQUIPMENT		BORING NO.
C & L Drilling / Mayhew 1000		G-127
DRILLING METHOD	**BOREHOLE LOCATION**	
Rotary Wash	Sta 560195, Rt 5 fcct	
DATES DRILLED	**HOLE DIAMETER**	**GROUND EL.**
4/11/2011 and 4/12/2011	4-7/8 inches	142 feet

GROUND-WATER READINGS
Drilling mud bailed on 4/11/2011. Ground-water level measured at 41 feet below the ground surface on 4/12/2011.

THIS RECORD IS AN INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	OVA (ppm)**	MOISTURE CONTENT (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	PERCENT PASSING No. 200 SIEVE	SAMPLE LOC.	DOWNHOLE TESTS		
140	0										4-inch thick Asphalt Concrete over 18-inch thick Portland Cement Concrete, No Base Course
										SC	**FILL [Af]** CLAYEY SAND - moist, brown to orangish brown, fine to medium-grained
135	5										
										CL	**QUATERNARY YOUNGER ALLUVIUM [Qal]** SANDY LEAN CLAY - very soft, moist, gray, sandy silt interbedded
130	10		0.0	22.5	97	Push					Becomes brown, layers of Sandy Silt
125	15	8	0.1	24.6	-		83				Becomes medium stiff, fine sand
120	20		0.0	27.0	98	7					Becomes brown to light olive brown, trace fine sand
											SAN PEDRO FORMATION [Qsp]
115	25	13	0.0	21.6	-		72			ML	SANDY SILT - stiff, moist, olive brown to olive gray, fine sand, layers of Silty Sand
110	30		0.0	23.3	95	11	46			SM	SILTY SAND - medium dense, moist, greenish gray, fine-grained
105	35	24	0.1	19.3	-					ML	SILT - very stiff, moist, greenish gray, interbeds silt and sand, some clay
	40										

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: DW
Prepared/Date: LH 5/20/2011
Checked/Date: LT 9/22/2011

MTA Westside Subway Extension
Los Angeles, California

amec

LOG OF BORING

Project No.: 4953-10-1561 Figure: A-2.23a

DRILLING COMPANY/DRILLING EQUIPMENT		BORING NO.
C & L Drilling / Mayhew 1000		G-127 (Continued)
DRILLING METHOD: Rotary Wash	BOREHOLE LOCATION: Sta 560+95, Rt 5 feet	
DATES DRILLED: 4/11/2011 and 4/12/2011	HOLE DIAMETER: 4-7/8 inches	GROUND EL.: 142 feet

GROUND-WATER READINGS
Drilling mud bailed on 4/11/2011. Ground-water level measured at 41 feet below the ground surface on 4/12/2011.

THIS RECORD IS AN INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	OVA (ppm)**	MOISTURE CONTENT (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	PERCENT PASSING No. 200 SIEVE	SAMPLE LOC.	DOWNHOLE TESTS		
			0.0	26.3	97	11	61			ML	SANDY SILT - stiff, moist, olive gray, fine to medium sand, occasional coarse
100	45	27	0.6	22.2	-						Becomes very stiff, olive gray, fine to medium sand, some calcium carbonate nodules
95	50		0.1	21.9	100	17				ML	SILT with SAND - very stiff, moist, greenish gray, fine sand
	55	21	0.4	25.8	-		53			CL	SANDY LEAN CLAY - very stiff, moist, gray, fine sand, occasional medium
Tunnel	60		0.0	28.3	90	11				CL	LEAN CLAY - stiff, moist, olive gray, trace sand
	65	10		-	-						(Sample not recovered)
	70		1.4	11.1	-	24	12			SM	SILTY SAND with GRAVEL - medium dense, moist to wet, gray, fine to coarse-grained, fine to coarse gravel (up to 1 inch in size)
70	75	52	1.2	-	-		80			ML	SILT with SAND - hard, moist, gray, fine sand, occasional medium to coarse, some clay
65	80									SP	

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: DW
Prepared/Date: LH 5/20/2011
Checked/Date: LT 9/22/2011

MTA Westside Subway Extension
Los Angeles, California

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LOG OF BORING

Project No.: 4953-10-1561 Figure: A-2.23b

DRILLING COMPANY/DRILLING EQUIPMENT		BORING NO.
C & L Drilling / Mayhew 1000		G-127 (Continued)
DRILLING METHOD	BOREHOLE LOCATION	
Rotary Wash	Sta 560+95, Rt 5 feet	
DATES DRILLED	HOLE DIAMETER	GROUND EL.
4/11/2011 and 4/12/2011	4-7/8 inches	142 feet

GROUND-WATER READINGS
Drilling mud bailed on 4/11/2011. Ground-water level measured at 41 feet below the ground surface on 4/12/2011.

THIS RECORD IS AN INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	OVA (ppm)**	MOISTURE CONTENT (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	PERCENT PASSING No. 200 SIEVE	SAMPLE LOC.	DOWNHOLE TESTS			
60	85		1.6	31.5	91	21		▩				POORLY GRADED SAND - medium dense, wet, gray, fine grained, trace silt
55	90	32	0.2	29.1	-			☒			CL	SANDY LEAN CLAY - hard, moist, gray to light gray
50	95		0.0	16.9	111	25	40	▩			SM	SILTY SAND - medium dense, moist, greenish gray, fine to coarse-grained, occasional gravel (3/8 inch in size)
45	100	37	0.0	25.4	-			☒			CL	SANDY LEAN CLAY - very stiff to hard, moist, greenish gray, fine sand
40	105		0.0	23.7	97	15		▩				Some calcium carbonate nodules
35	110	49	0.0	24.6	-			☒			SM	SILTY SAND - medium dense, moist, gray, fine-grained, layers of Sandy Silt
30	115		0.0	17.7	111	17		▩				END OF BORING AT 112 FEET
25	120											

NOTES:
Hand augered upper 10 feet to avoid damage to utilities.
Borehole grouted with cement bentonite slurry and patched with asphalt concrete.

"N" Value Standard Penetration Test: Number of blows required to drive the SPT sampler 18 inches using a 140 pound automatic hammer falling 30 inches

*Number of blows required to drive the Crandall Sampler 12 inches using a 380 pound hammer falling 18 inches
**Photo Ionization Detector used for OVA readings

Field Tech: DW
Prepared/Date: LH 5/20/2011
Checked/Date: LT 9/22/2011

MTA Westside Subway Extension
Los Angeles, California

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LOG OF BORING

Project No.: 4953-10-1561 Figure: A-2.23c

DRILLING COMPANY/DRILLING EQUIPMENT		BORING NO.
C & L Drilling / Mayhew 1000		G-128
DRILLING METHOD	**BOREHOLE LOCATION**	
Rotary Wash	Sta 563+55, Lt 28 feet	
DATES DRILLED	**HOLE DIAMETER**	**GROUND EL.**
4/14/2011, 5/23/2011 and 5/24/2011	4-7/8 inches	141 feet

GROUND-WATER READINGS

Ground-water level measured at 26 feet below the ground surface on 5/24/2011.

THIS RECORD IS AN INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	OVA (ppm)**	MOISTURE CONTENT (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	PERCENT PASSING No. 200 SIEVE	SAMPLE LOC.	DOWNHOLE TESTS		
140	0										10-inch thick Asphalt Concrete over 4-inch thick Portland Cement Concrete
										GW	**FILL [Af]** - WELL GRADED GRAVEL with SAND - wet, dark gray to black, fine to medium gravel, fine to coarse sand
											QUATERNARY YOUNGER ALUVIUM [Qal]
										CL	SANDY LEAN CLAY - stiff, moist, dark gray, fine to medium sand
135	5		0.0	21.2	101	15	64				
130	10	12	0.0	30.4	-						Becomes dark olive gray
125	15		0.0	34.5	-	3	95		PMT		**QUATERNARY OLDER ALLUVIUM [Qalo]**
										CH	FAT CLAY - soft, moist, olive gray, some silt, trace fine sand
										CL	LEAN CLAY with SAND - very stiff, moist, light brown, fine to medium sand, some root fragments
120	20	18	0.0	25.6	-						
115	25		0.0	19.8	107	7	75				▼ Becomes medium stiff, light yellowish brown, fine sand, trace medium
											SAN PEDRO FORMATION [Qsp]
										CL	LEAN CLAY with SAND - very stiff, moist, light yellowish brown to olive, fine sand, occasional medium
110	30	16	0.0	34.0	-		84				
105	35		0.0	45.7	78	4			PMT		FAT CLAY - soft, moist, dark greenish gray, trace fine sand
										CH	
										SC	CLAYEY SAND with GRAVEL- very dense, wet, bluish gray, fine to medium-grained, fine gravel (up to ¾ inch in size)
	40										

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: HTY
Prepared/Date: YN 6/16/2011
Checked/Date: HP/PE 9/19/2011

MTA Westside Subway Extension
Los Angeles, California

amec

LOG OF BORING

Project No.: 4953-10-1561 Figure: A-2.24a

DRILLING COMPANY/DRILLING EQUIPMENT		BORING NO.
C & L Drilling / Mayhew 1000		G-128 (Continued)
DRILLING METHOD	**BOREHOLE LOCATION**	
Rotary Wash	Sta 563+55, Lt 28 feet	
DATES DRILLED	**HOLE DIAMETER**	**GROUND EL.**
4/14/2011, 5/23/2011 and 5/24/2011	4-7/8 inches	141 feet

GROUND-WATER READINGS

Ground-water level measured at 26 feet below the ground surface on 5/24/2011.

THIS RECORD IS AN INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	OVA (ppm)**	MOISTURE CONTENT (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	PERCENT PASSING No. 200 SIEVE	SAMPLE LOC.	DOWNHOLE TESTS		
100	45	69		20.2	-			☒			Becomes greenish gray
95	50		0.0	19.0	-	18	43	▩			Becomes medium dense
	55	29	0.0	-	-			☒			
Wilshire / La Cienega Station	60		0.0	18.3	111	13	52	▩	PMT	CL	SANDY LEAN CLAY - stiff, moist, greenish gray, fine sand, some medium
	65	68	0.0	19.2	-			☒			Becomes hard, trace fine to coarse gravel
70	70		0.1	22.5	102	12	53	▩			Layer of Sandy Silt, stiff, fine to medium sand
	75									SP-SM	POORLY GRADED SAND with SILT - very dense, wet, dark greenish gray
65	80	58	0.0	23.9	-			☒			

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: HTY
Prepared/Date: YN 6/16/2011
Checked/Date: HP/PE 9/19/2011

MTA Westside Subway Extension
Los Angeles, California

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LOG OF BORING

Project No.: 4953-10-1561 Figure: A-2.24b

DRILLING COMPANY/DRILLING EQUIPMENT		BORING NO.
C & L Drilling / Mayhew 1000		**G-128 (Continued)**
DRILLING METHOD	**BOREHOLE LOCATION**	
Rotary Wash	Sta 563+55, Lt 28 feet	
DATES DRILLED	**HOLE DIAMETER**	**GROUND EL.**
4/14/2011, 5/23/2011 and 5/24/2011	4-7/8 inches	141 feet

GROUND-WATER READINGS

Ground-water level measured at 26 feet below the ground surface on 5/24/2011.

THIS RECORD IS AN INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	OVA (ppm)**	MOISTURE CONTENT (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	PERCENT PASSING No. 200 SIEVE	SAMPLE LOC.	DOWNHOLE TESTS			
60			0.0	25.4	97	16		▨		CH	FAT CLAY with SAND - very stiff, moist, dark greenish gray	
55	85	39	0.2	25.5	-		49	☒		SC	CLAYEY SAND - dense, moist, light greenish gray, fine to medium-grained, trace coarse, trace gravel	
50	90		0.0	13.3	108	15		▨		SW	WELL GRADED SAND - medium dense, moist, dark greenish gray, trace clay	
										CL	SANDY LEAN CLAY - moist, dark greenish gray, fine sand	
45	95	32	0.1	26.4	-			☒		CH	FAT CLAY with SAND - hard, moist, medium sand	
40	100		0.1	31.7	92	9		▨			Becomes stiff, light greenish gray, fine to coarse sand, thin layers of Clayey Sand, fine gravel	
										CL	LEAN CLAY with SAND - moist, dark greenish gray	
35	105	75	0.1	21.4	-			☒		SC	CLAYEY SAND - very dense, wet, fine-grained	
30	110		0.1	28.5	93	12		▨		CL	SANDY LEAN CLAY - stiff, moist, dark greenish gray, fine sand	
25	115										END OF BORING AT 111 FEET	
	120											

NOTES:
Hand augered uppper 5 feet to avoid damage to utilities.
Borehole grouted with cement bentonite and patched with asphalt concrete.

"N" Value Standard Penetration Test: Number of blows required to drive the SPT sampler 18 inches using a 140 pound automatic hammer falling 30 inches

*Number of blows required to drive Crandall Sampler 12 inches using 380 pound hammer falling 18 inches

**Photo Ionization Detector used for OVA readings
Downhole Test: PMT = Pressuremeter

Field Tech: HTY
Prepared/Date: YN 6/16/2011
Checked/Date: HP/PE 9/19/2011

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LOG OF BORING

Project No.: 4953-10-1561 Figure: A-2.24c

July 25, 2019
BG 22581

APPENDIX II

Laboratory Testing and Log of Borings (Current Study)

July 25, 2019
BG 22581

LABORATORY TESTING

Undisturbed and bulk samples of the alluvium and older alluvium were obtained from the borings and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring-lined, barrel sampler conforming to ASTM D 3550-01 with successive drops of the sampler. Experience has shown that sampling causes some disturbance of the sample. However, the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The samples were stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D 2937-10. The moisture content of the samples was determined using the procedures outlined in ASTM D 2216-10. The results are shown on the enclosed Log of Borings.

Maximum Density

The maximum dry density and optimum moisture content of the future compacted fill were determined using the procedures outlined in ASTM D 1557-12, a five-layer standard. The results are shown in the following table.

Boring	Depth (Feet)	Earth Material	Soil Type and Color	Maximum Density (pcf)	Optimum Moisture %	Expansion Index
B4	0 - 10	Alluvium	Clay Dark Brown	117.0	15.0	87 - Moderate

Expansion Test

To find the expansiveness of the soil, a swell test was performed using the procedures outlined in ASTM D 4829-11. Based upon the testing, the near-surface soil is expected to exhibit a moderate expansion potential.

LABORATORY TESTING (Continued)

Shear Tests

Shear tests were performed on samples of the alluvium and older alluvium using the procedures outlined in ASTM D 3080-11 and a strain controlled, direct-shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.025 inch per minute. The samples were tested in an artificially saturated condition. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the enclosed Shear Test Diagrams.

Consolidation

Consolidation tests were performed on *in situ* samples of the alluvium and older alluvium using the procedures outlined in ASTM D 2435-11. Results are graphed on the enclosed Consolidation Curves.

Atterberg Limits

Atterberg limits were determined on representative samples of the alluvium obtained from Borings B1 and B4 using the procedures outlined in ASTM D 4318-10. The tests were performed to assist in the engineering classification of the fine-grained materials and to determine the Liquid Limit (LL) and Plasticity Index (PI). Results of Atterberg Limits are graphed on the enclosed Plasticity Charts and shown in the following table:

Results of Atterberg Limits Laboratory Tests						
Boring No.	Depth (feet)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Soil Type	Reference
B1	2.5	44.7	22.0	22.7	Clay (CL)	Plasticity Chart #1
B1	10.0	59.0	31.9	27.1	Silt (MH)	Plasticity Chart #2
B1	15.0	37.4	19.0	18.4	Clay (CL)	Plasticity Chart #3
B1	17.5	49.3	30.7	18.6	Silt (CL)	Plasticity Chart #4
B1	22.5	31.0	17.0	14.0	Clay (CL)	Plasticity Chart #5
B4	22.5	35.0	19.9	15.1	Clay (CL)	Plasticity Chart #6
B4	27.5	28.6	18.1	10.5	Clay (CL)	Plasticity Chart #7

LABORATORY TESTING (Continued)

Fines Content

Sieve analysis (wash method) was performed on representative samples of the alluvium and older alluvium obtained from Borings B1 and B4 using the procedures outlined in ASTM D 1140-14. The tests were performed to assist in the classification of the soil and to determine the fines content (percent passing #200 sieve). The results are shown on the enclosed Log of Borings B1 and B4, and are summarized in the following table.

Results of Sieve Analysis (Wash Method) Laboratory Tests							
Boring No.	Depth (feet)	Fines Content (%)	Soil Type	Boring No.	Depth (feet)	Fines Content (%)	Soil Type
B1	2.5	71.9	Clay (CL)	B1	37.5	73.8	Clay w/Sand (CL)
B1	10.0	85.4	Silt (ML)	B1	42.5	61.6	Sandy Clay (CL)
B1	12.5	87.2	Clay (CL)	B1	45.0	52.2	Sandy Clay (CL)
B1	15.0	60.8	Clay (CL)	B1	50.0	62.8	Sandy Silt (ML)
B1	17.5	89.2	Silt (ML)	B1	57.5	32.4	Silty Sand (SM)
B1	20.0	84.6	Silt (ML)	B1	60.0	55.9	Sandy Silt (ML)
B1	22.5	68.8	Clay w/Sand (CL)	B1	85.0	43.5	Silty Sand (SM)
B1	25.0	72.4	Sandy Silt (ML)	B4	22.5	64.5	Sandy Clay (CL)
B1	27.5	65.4	Sandy Clay (CL)	B4	27.5	59.7	Sandy Clay (CL)
B1	32.5	89.9	Clay (CL)	-	-	-	-

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SHEAR TEST DIAGRAM #1

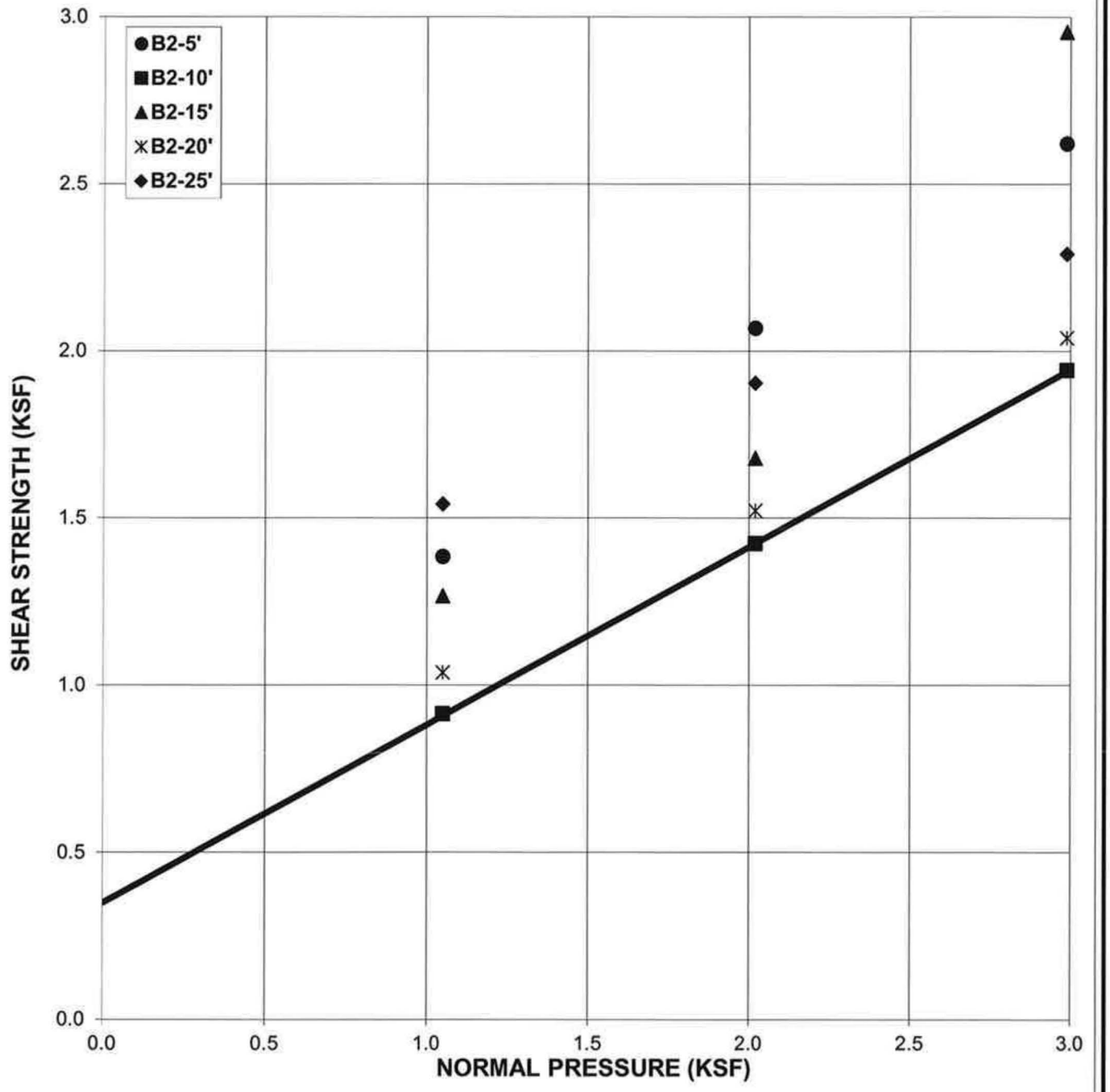
BG: **22581** ENGINEER: **RSB**
CLIENT: **Stockdale Capital Partners, LLC**

EARTH MATERIAL: **Alluvium**

Phi Angle = 28.0 degrees
Cohesion = 350 psf

Average Moisture Content 20.8%
Average Dry Density (pcf) 106.3
Average Saturation 98%

DIRECT SHEAR TEST - ASTM D-3080 (ULTIMATE VALUES)



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SHEAR TEST DIAGRAM #2

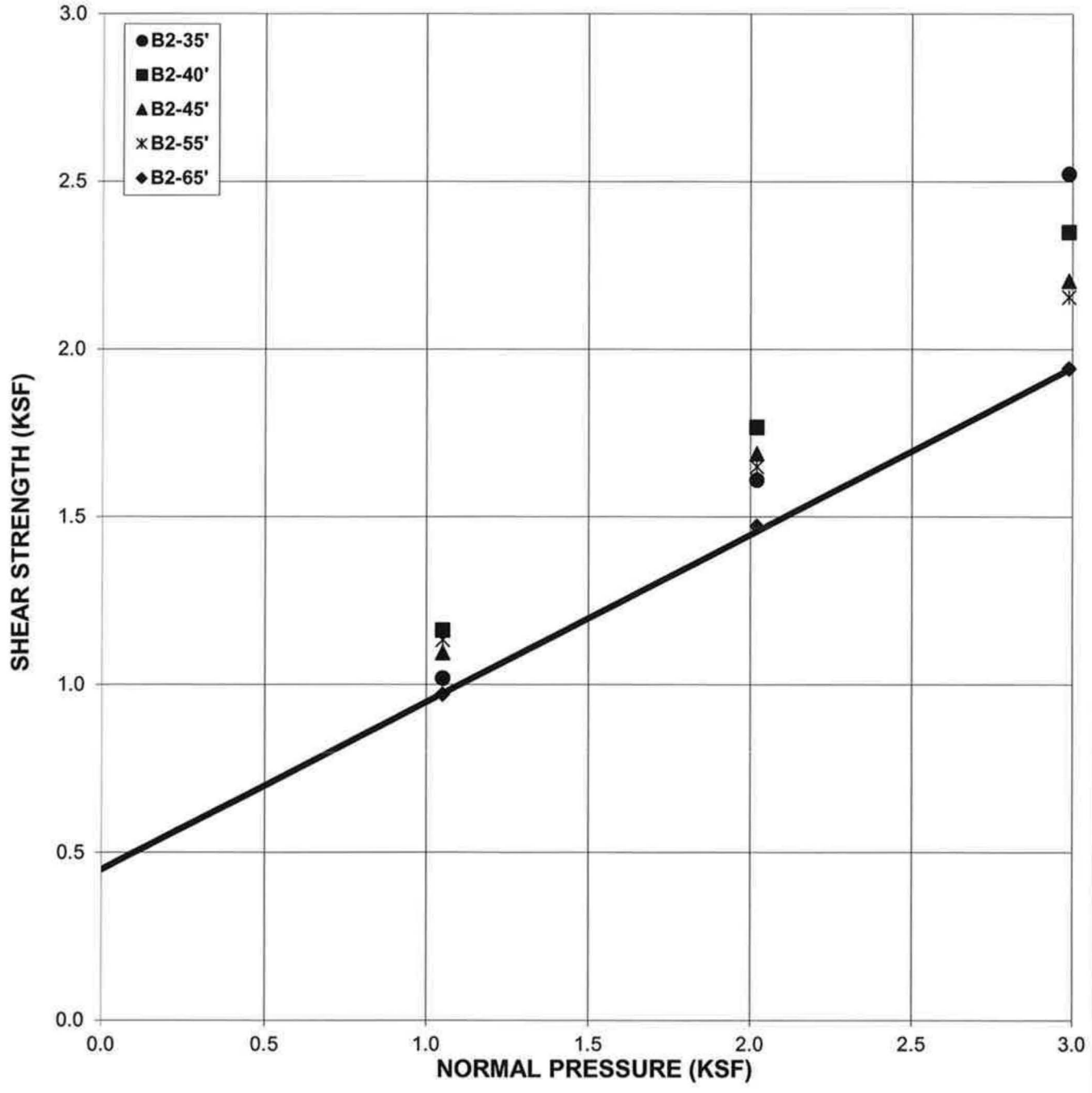
BG: **22581** ENGINEER: **RSB**
CLIENT: **Stockdale Capital Partners, LLC**

EARTH MATERIAL: **San Pedro Formation**

Phi Angle = 26.5 degrees
Cohesion = 450 psf

Average Moisture Content 24.1%
Average Dry Density (pcf) 101.7
Average Saturation 99%

DIRECT SHEAR TEST - ASTM D-3080 (ULTIMATE VALUES)



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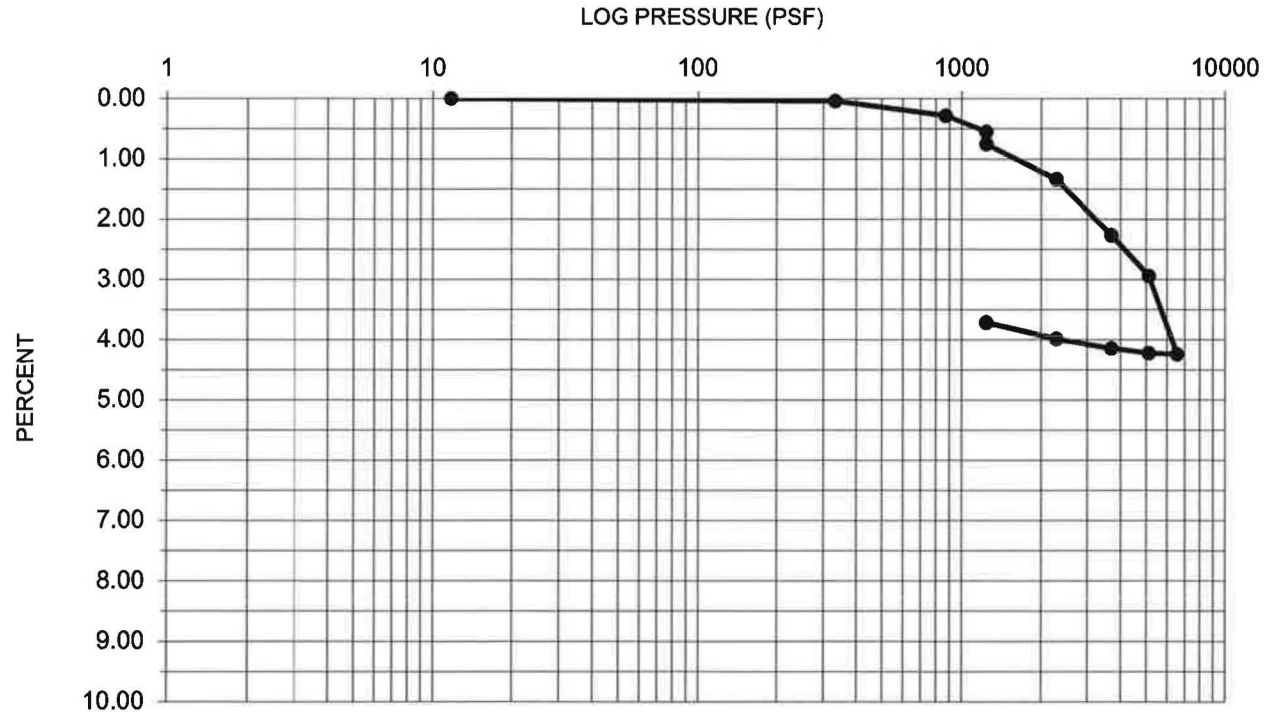
CONSOLIDATION CURVE #1

BG: **22581** ENGINEER: **RSB**

CLIENT: **Stockdale Capital Partners, LC**

Earth Material:	Alluvium		
Sample Location:	B2-15'	Specific Gravity:	2.65
Dry Weight (pcf):	103.4	Initial Void Ratio:	0.60
Initial Moisture:	22.6%	Compression Index (Cc):	0.195
Initial Saturation:	100.0%	Recompression Index (Cr):	0.016
Water Added at (psf)	1237		

CONSOLIDATION DIAGRAM (ASTM D 2435-04)



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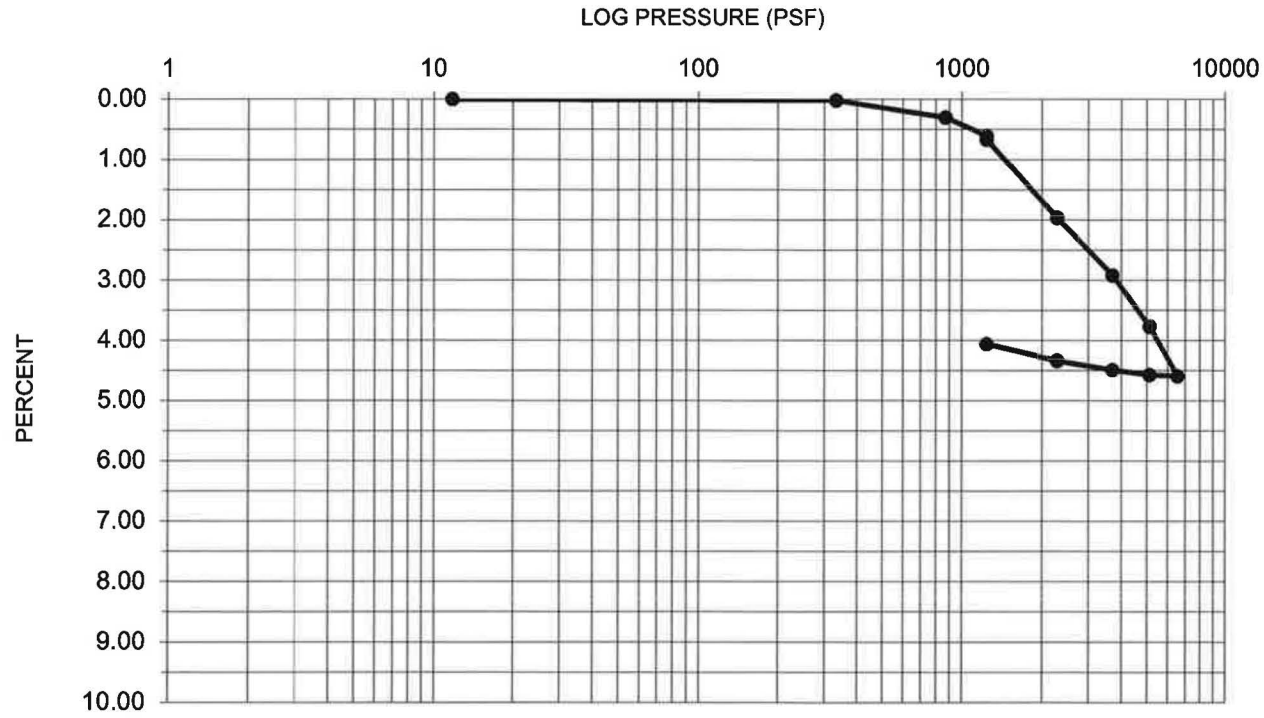
CONSOLIDATION CURVE #2

BG: **22581** ENGINEER: **RSB**

CLIENT: **Stockdale Capital Partners, LC**

Earth Material:	Alluvium		
Sample Location:	B3-25'	Specific Gravity:	2.65
Dry Weight (pcf):	101.0	Initial Void Ratio:	0.64
Initial Moisture:	23.2%	Compression Index (Cc):	0.126
Initial Saturation:	96.4%	Recompression Index (Cr):	0.017
Water Added at (psf)	1237		

CONSOLIDATION DIAGRAM (ASTM D 2435-04)



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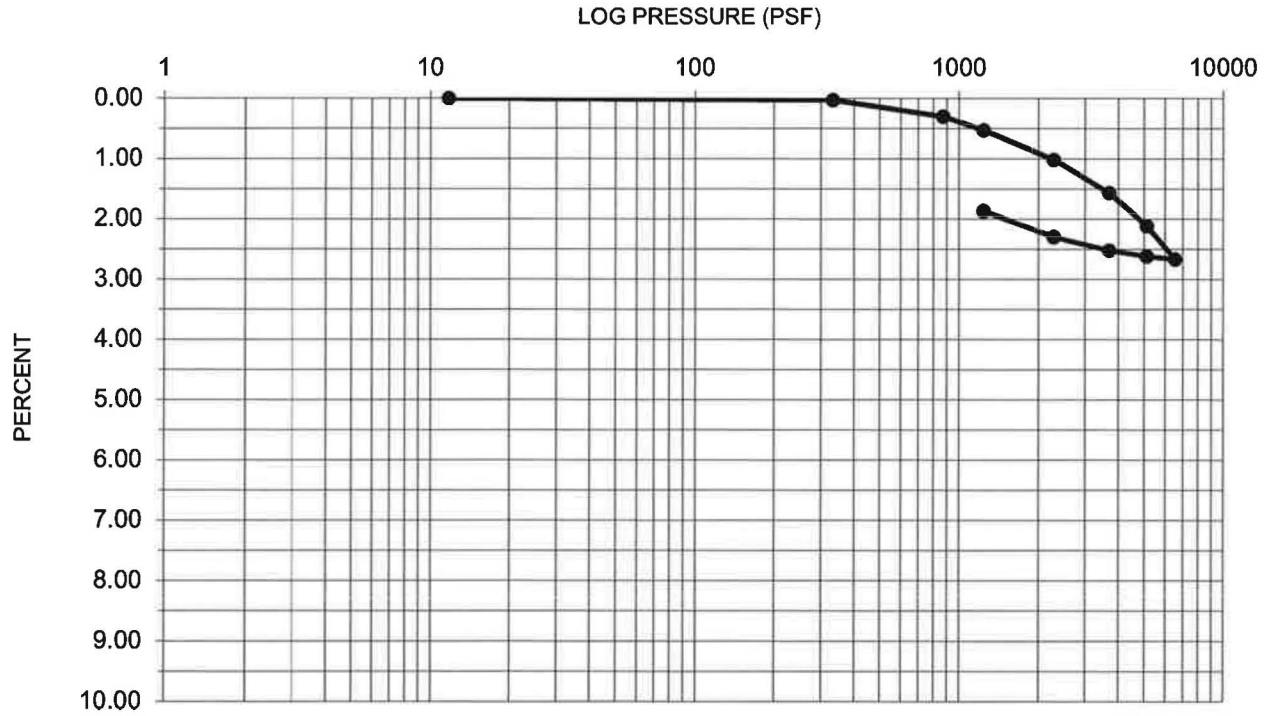
CONSOLIDATION CURVE #3

BG: **22581** ENGINEER: **RSB**

CLIENT: **Stockdale Capital Partners, LC**

Earth Material:	Alluvium		
Sample Location:	B2-27.5'	Specific Gravity:	2.65
Dry Weight (pcf):	112.1	Initial Void Ratio:	0.48
Initial Moisture:	16.6%	Compression Index (Cc):	0.076
Initial Saturation:	92.6%	Recompression Index (Cr):	0.024
Water Added at (psf)	1237		

CONSOLIDATION DIAGRAM (ASTM D 2435-04)



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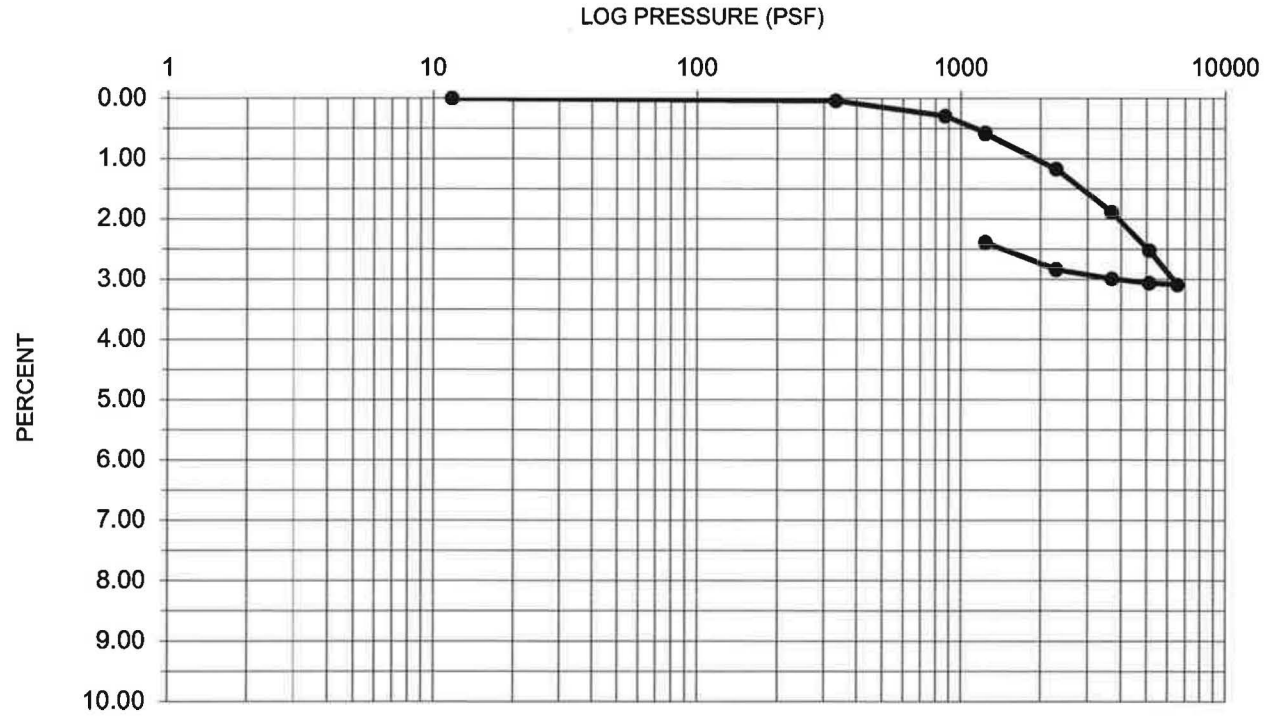
CONSOLIDATION CURVE #4

BG: **22581** ENGINEER: **RSB**

CLIENT: **Stockdale Capital Partners, LC**

Earth Material:	Older Alluvium		
Sample Location:	B3-35'	Specific Gravity:	2.65
Dry Weight (pcf):	102.1	Initial Void Ratio:	0.62
Initial Moisture:	23.4%	Compression Index (Cc):	0.087
Initial Saturation:	100.0%	Recompression Index (Cr):	0.027
Water Added at (psf)	1237		

CONSOLIDATION DIAGRAM (ASTM D 2435-04)



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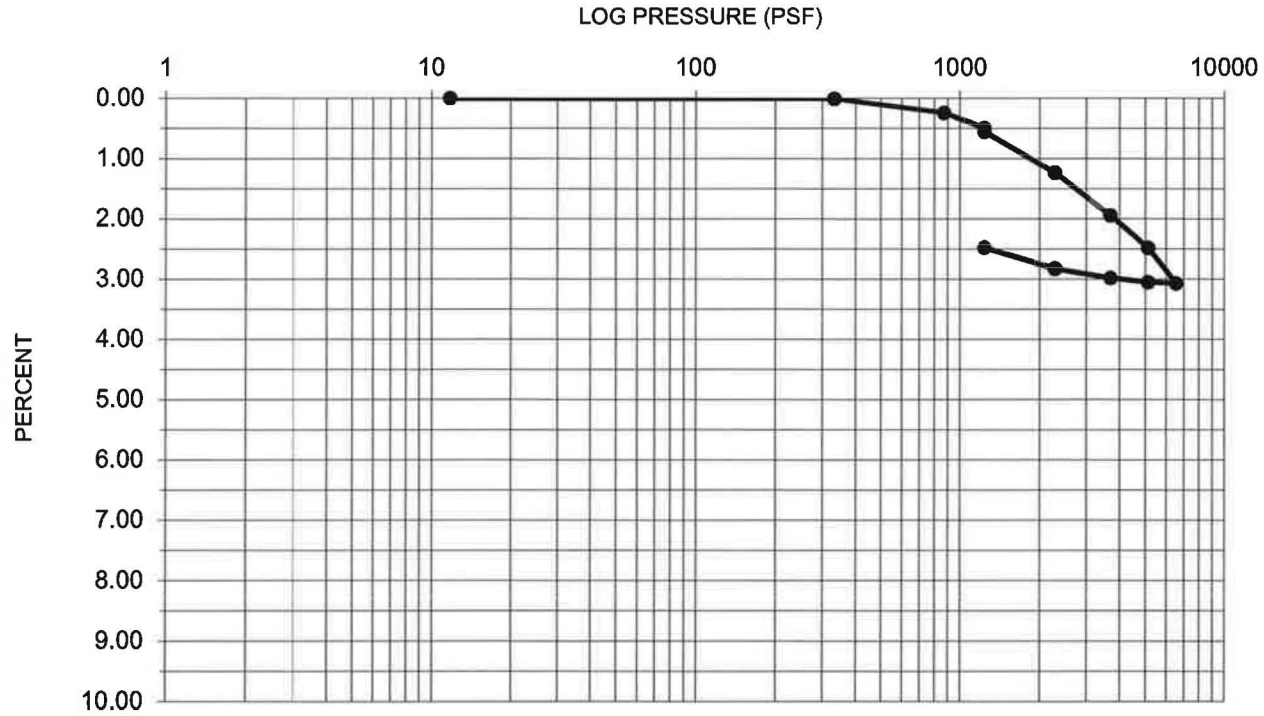
CONSOLIDATION CURVE #5

BG: **22581** ENGINEER: **RSB**

CLIENT: **Stockdale Capital Partners, LC**

Earth Material:	Older Alluvium		
Sample Location:	B2-37.5'	Specific Gravity:	2.65
Dry Weight (pcf):	100.7	Initial Void Ratio:	0.64
Initial Moisture:	21.7%	Compression Index (Cc):	0.091
Initial Saturation:	89.6%	Recompression Index (Cr):	0.022
Water Added at (psf)	1237		

CONSOLIDATION DIAGRAM (ASTM D 2435-04)



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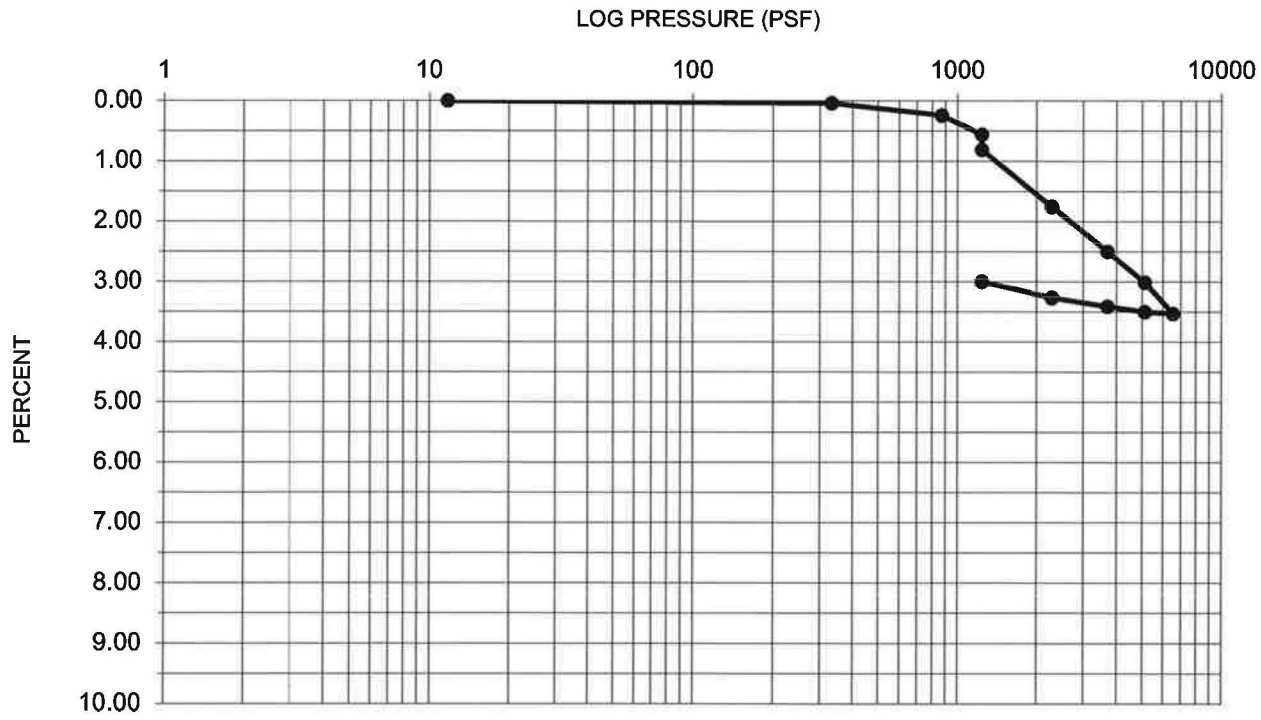
CONSOLIDATION CURVE #6

BG: **22581** ENGINEER: **RSB**

CLIENT: **Stockdale Capital Partners, LC**

Earth Material:	Older Alluvium		
Sample Location:	B2-45'	Specific Gravity:	2.75
Dry Weight (pcf):	111.8	Initial Void Ratio:	0.53
Initial Moisture:	19.2%	Compression Index (Cc):	0.075
Initial Saturation:	98.8%	Recompression Index (Cr):	0.016
Water Added at (psf)	1237		

CONSOLIDATION DIAGRAM (ASTM D 2435-04)



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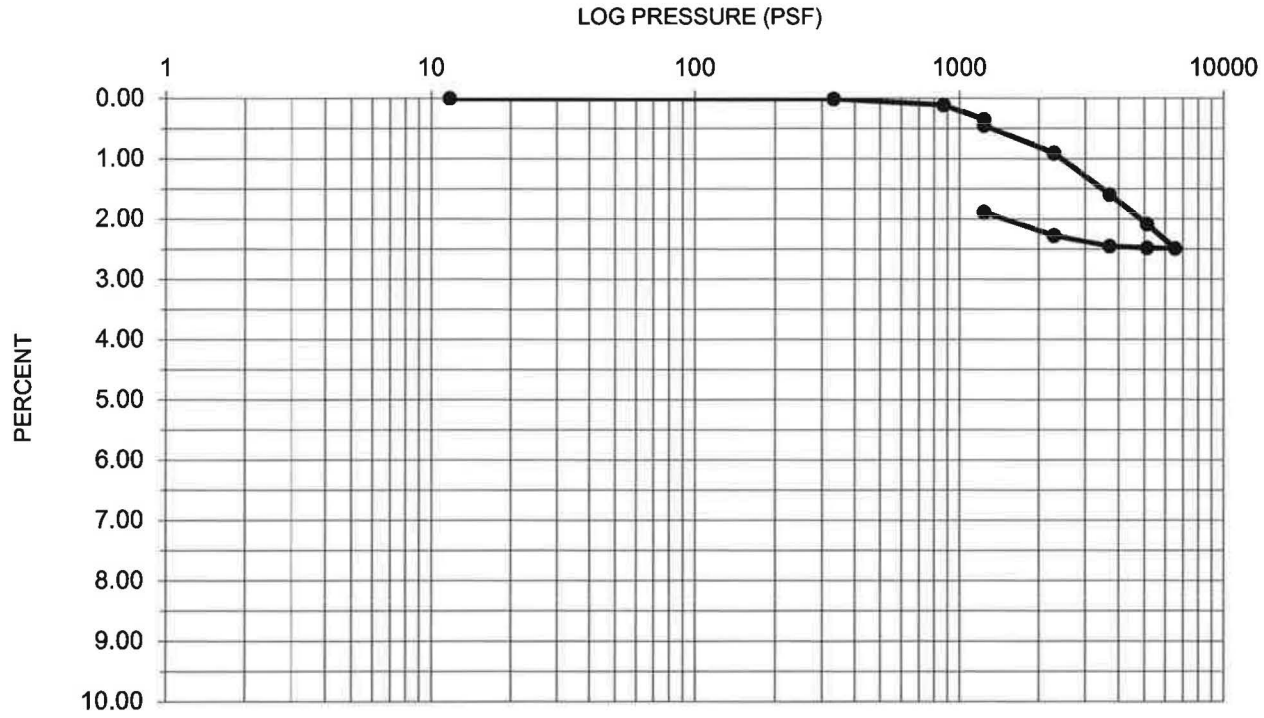
CONSOLIDATION CURVE #7

BG: **22581** ENGINEER: **RSB**

CLIENT: **Stockdale Capital Partners, LC**

Earth Material:	Older Alluvium		
Sample Location:	B2-50'	Specific Gravity:	2.75
Dry Weight (pcf):	114.1	Initial Void Ratio:	0.50
Initial Moisture:	14.6%	Compression Index (Cc):	0.056
Initial Saturation:	79.7%	Recompression Index (Cr):	0.022
Water Added at (psf)	1237		

CONSOLIDATION DIAGRAM (ASTM D 2435-04)



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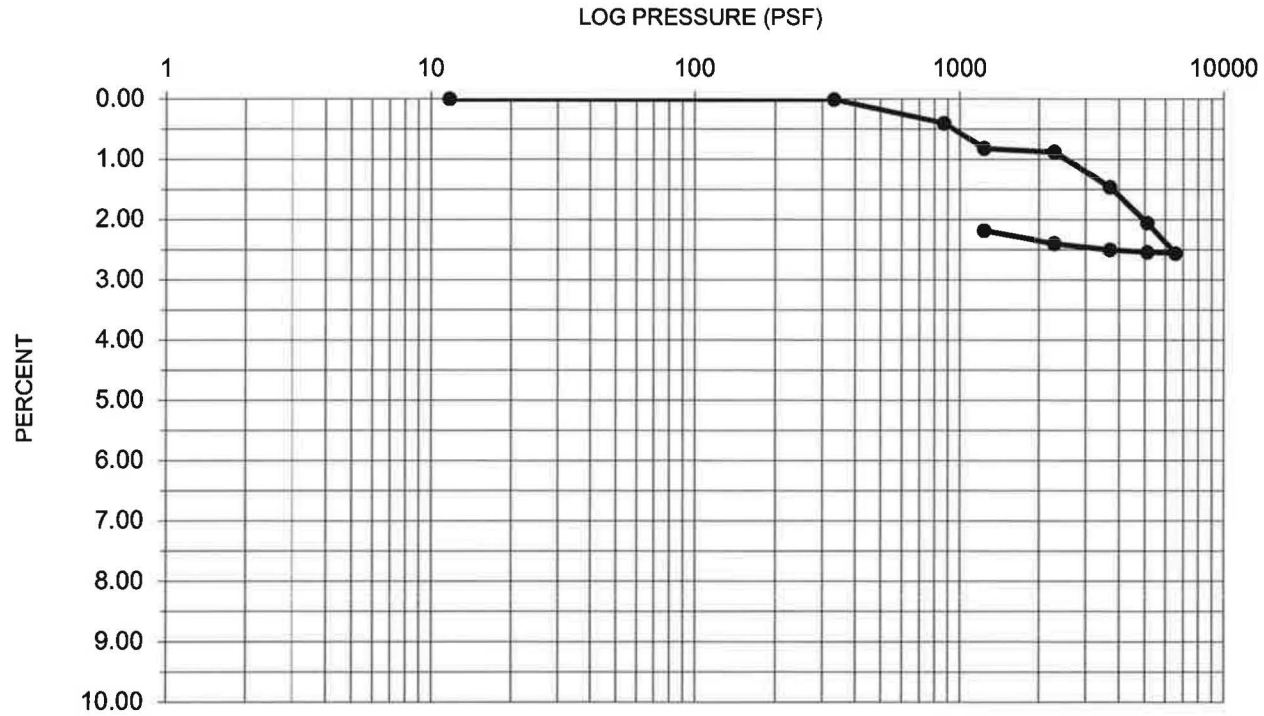
CONSOLIDATION CURVE #8

BG: **22581** ENGINEER: **RSB**

CLIENT: **Stockdale Capital Partners, LC**

Earth Material:	Older Alluvium		
Sample Location:	B2-60'	Specific Gravity:	2.75
Dry Weight (pcf):	108.9	Initial Void Ratio:	0.58
Initial Moisture:	20.6%	Compression Index (Cc):	0.074
Initial Saturation:	98.3%	Recompression Index (Cr):	0.012
Water Added at (psf)	1237		

CONSOLIDATION DIAGRAM (ASTM D 2435-04)



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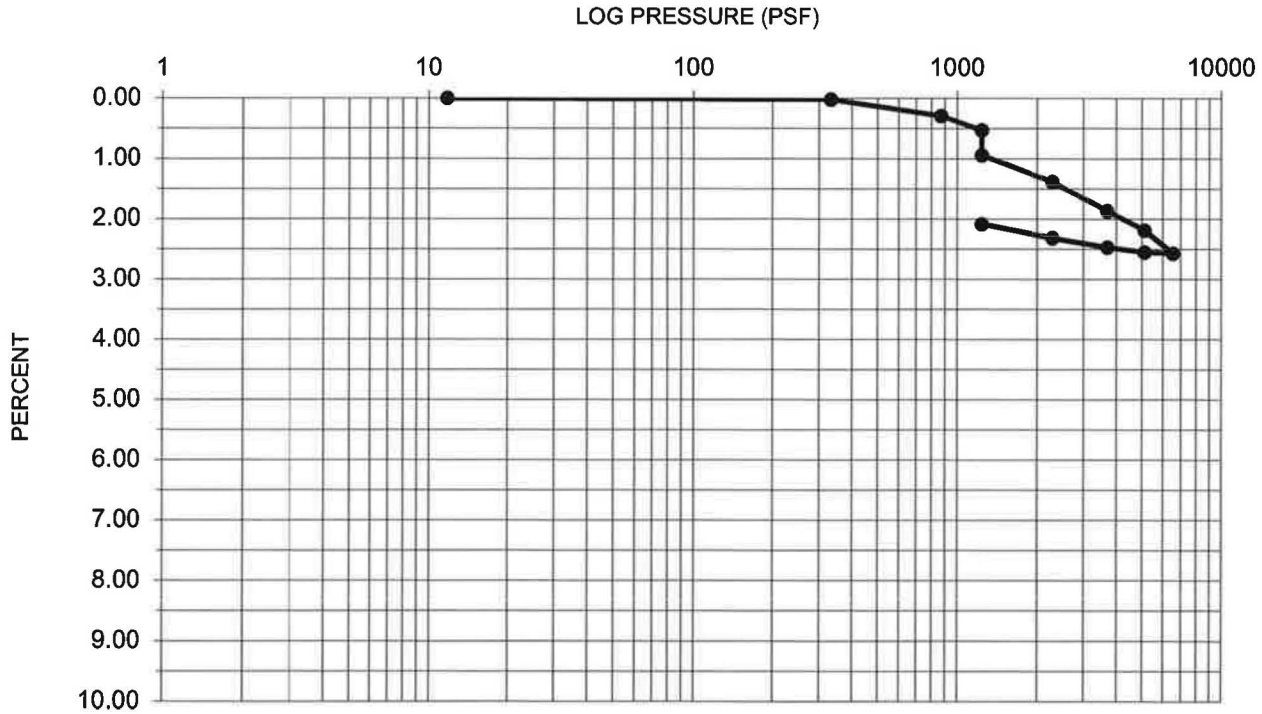
CONSOLIDATION CURVE #9

BG: **22581** ENGINEER: **RSB**

CLIENT: **Stockdale Capital Partners, LC**

Earth Material:	Older Alluvium		
Sample Location:	B2-70'	Specific Gravity:	2.75
Dry Weight (pcf):	102.3	Initial Void Ratio:	0.68
Initial Moisture:	24.7%	Compression Index (Cc):	0.060
Initial Saturation:	100.0%	Recompression Index (Cr):	0.015
Water Added at (psf)	1237		

CONSOLIDATION DIAGRAM (ASTM D 2435-04)



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PLASTICITY CHART #1

BG: **22581** ENGINEER: **RSB**

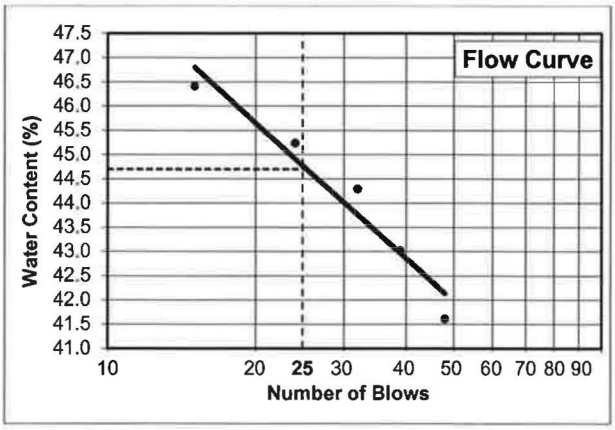
CLIENT: **Stockdale Capital Partners, LLC**

Test Pit No.: B1 **Sample No.:** S1 **Depth of Sample:** 2.5 Feet **Test Date:** 6/26/2017

Soil Description: Clay (CL)

Liquid Limit Determination

Can No.	A	B	C	D	E
Soil Wet Wt. + Can (g)	29.55	30.12	29.95	30.11	30.39
Soil Dry Wt. + Can (g)	26.25	26.70	26.65	26.75	27.34
Wt. of Can (g)	19.14	19.14	19.20	18.94	20.01
Wt. of Dry Soil (g)	7.11	7.56	7.45	7.81	7.33
Wt. of Moisture (g)	3.30	3.42	3.30	3.36	3.05
Water Content (%)	**46.4**	**45.2**	**44.3**	**43.0**	**41.6**
Number of Blows	**15**	**24**	**32**	**39**	**48**

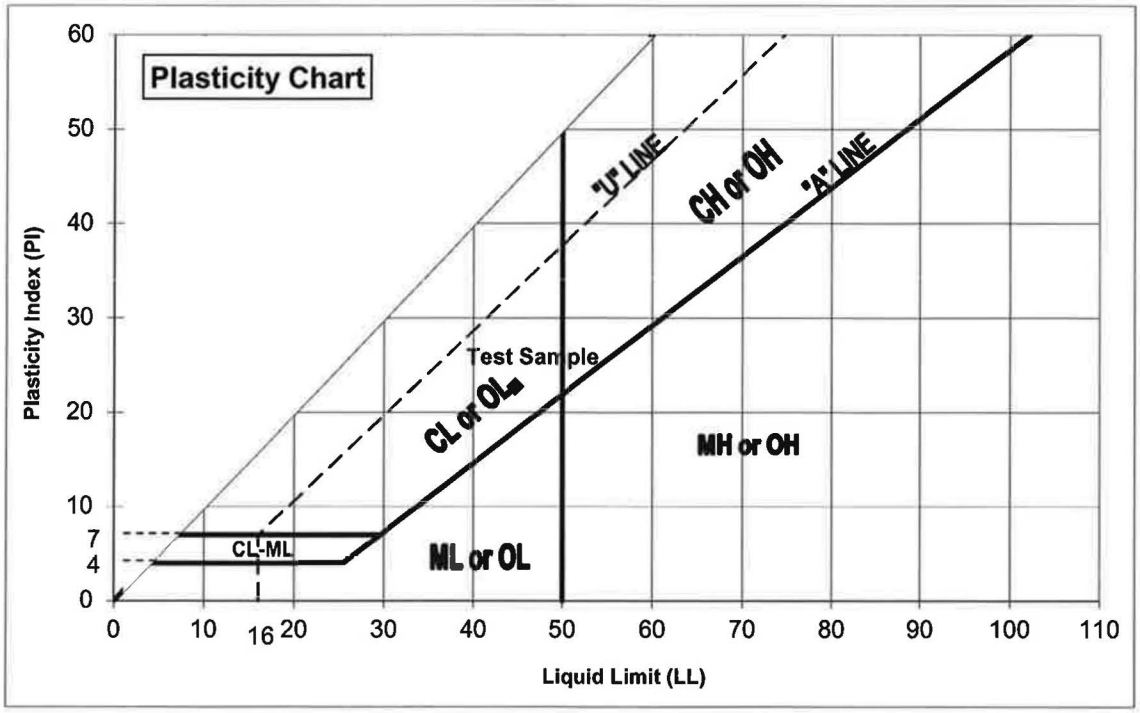


Plastic Limit Determination

Can No.	F				
Soil Wet Wt. + Can (g)	25.78				
Soil Dry Wt. + Can (g)	24.52				
Wt. of Can (g)	18.80				
Wt. of Dry Soil (g)	5.72				
Wt. of Moisture (g)	1.26				
Water Content (%)	**22.0**				

Average Water Content (%) = **22.0**

Liquid Limit, LL = 44.7
Plastic Limit, PL = 22.0
Plasticity Index, PI = 22.7



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PLASTICITY CHART #2

BG: **22581** ENGINEER: **RSB**

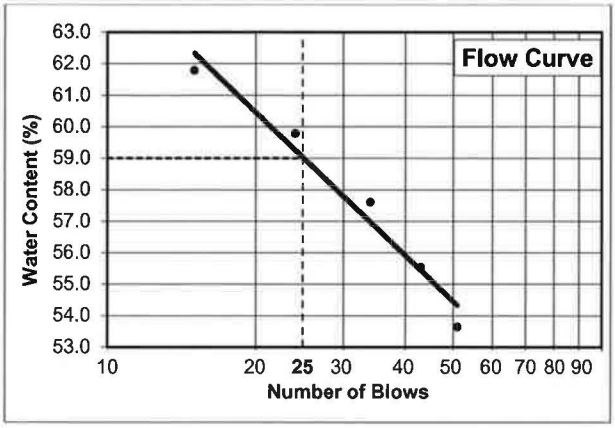
CLIENT: **Stockdale Capital Partners, LLC**

Test Pit No.: B1 **Sample No.:** S4 **Depth of Sample:** 10 Feet **Test Date:** 6/26/2017

Soil Description: Silt (MH)

Liquid Limit Determination

Can No.	A	B	C	D	E
Soil Wet Wt. + Can (g)	29.89	29.99	29.29	30.39	29.43
Soil Dry Wt. + Can (g)	25.75	26.02	25.69	26.22	25.74
Wt. of Can (g)	19.05	19.38	19.44	18.71	18.86
Wt. of Dry Soil (g)	6.70	6.64	6.25	7.51	6.88
Wt. of Moisture (g)	4.14	3.97	3.60	4.17	3.69
Water Content (%)	**61.8**	**59.8**	**57.6**	**55.5**	**53.6**
Number of Blows	**15**	**24**	**34**	**43**	**51**

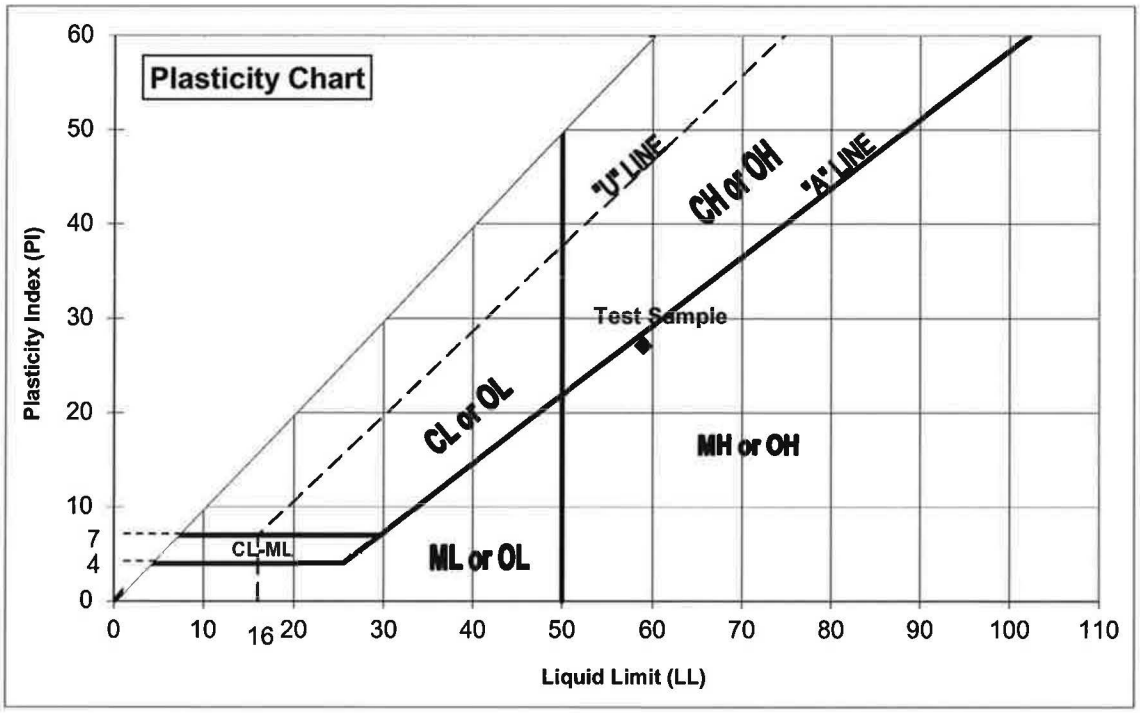


Plastic Limit Determination

Can No.	F				
Soil Wet Wt. + Can (g)	25.97				
Soil Dry Wt. + Can (g)	24.21				
Wt. of Can (g)	18.70				
Wt. of Dry Soil (g)	5.51				
Wt. of Moisture (g)	1.76				
Water Content (%)	**31.9**				

Average Water Content (%) = **31.9**

Liquid Limit, LL = 59.0
Plastic Limit, PL = 31.9
Plasticity Index, PI = 27.1



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PLASTICITY CHART #3

BG: **22581** ENGINEER: **RSB**

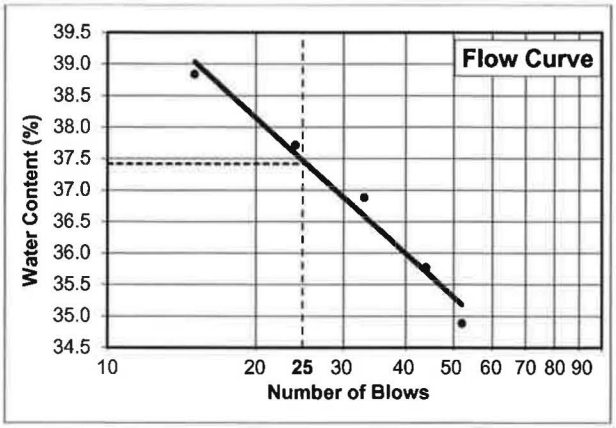
CLIENT: **Stockdale Capital Partners, LLC**

Test Pit No.: B1 **Sample No.:** S6 **Depth of Sample:** 15 feet **Test Date:** 6/27/2017

Soil Description: Clay (CL)

Liquid Limit Determination

Can No.	A	B	C	D	E
Soil Wet Wt. + Can (g)	29.91	29.59	30.55	29.58	30.69
Soil Dry Wt. + Can (g)	26.90	26.72	27.50	26.79	27.92
Wt. of Can (g)	19.15	19.11	19.23	18.99	19.98
Wt. of Dry Soil (g)	7.75	7.61	8.27	7.80	7.94
Wt. of Moisture (g)	3.01	2.87	3.05	2.79	2.77
Water Content (%)	**38.8**	**37.7**	**36.9**	**35.8**	**34.9**
Number of Blows	**15**	**24**	**33**	**44**	**52**

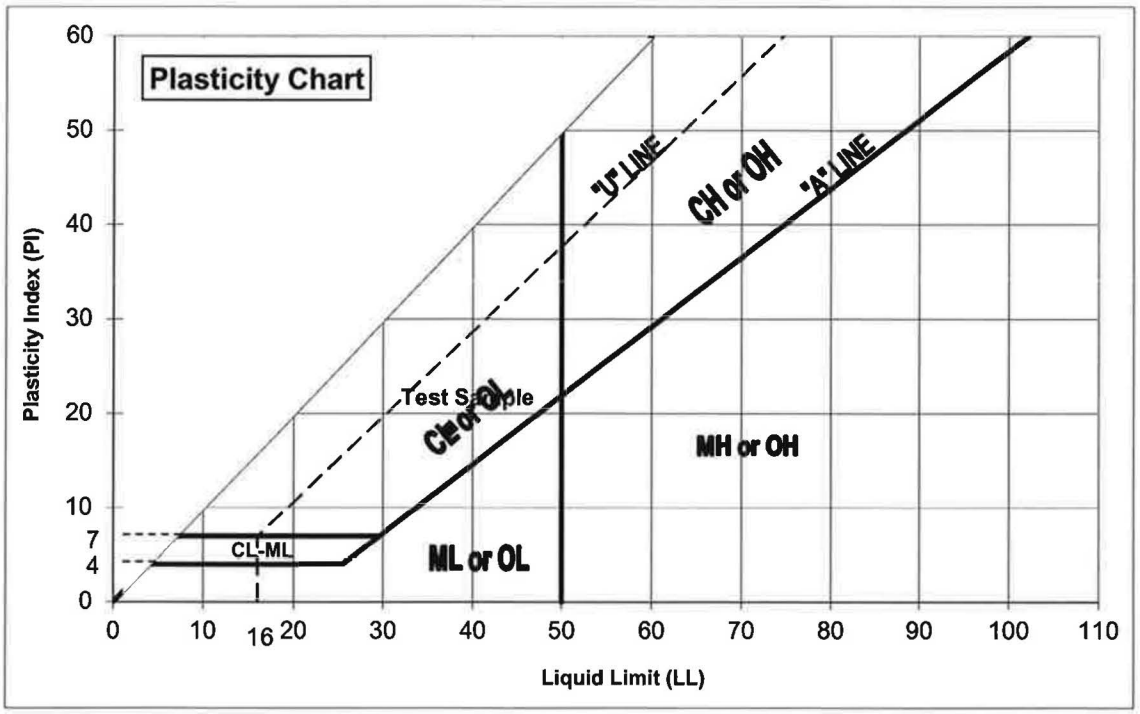


Plastic Limit Determination

Can No.	F				
Soil Wet Wt. + Can (g)	25.95				
Soil Dry Wt. + Can (g)	24.81				
Wt. of Can (g)	18.82				
Wt. of Dry Soil (g)	5.99				
Wt. of Moisture (g)	1.14				
Water Content (%)	**19.0**				

Average Water Content (%) = **19.0**

Liquid Limit, LL = 37.4
Plastic Limit, PL = 19.0
Plasticity Index, PI = 18.4



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PLASTICITY CHART #4

BG: **22581** ENGINEER: **RSB**

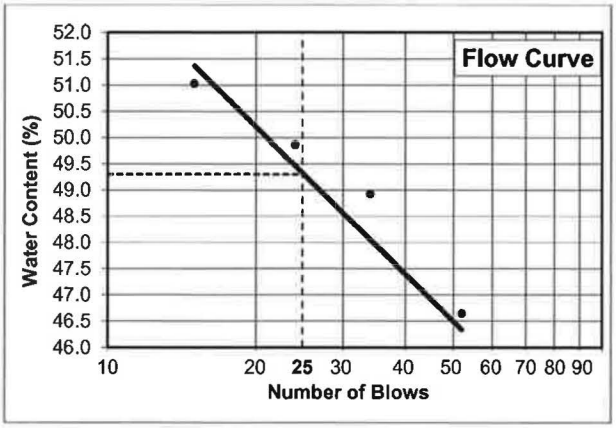
CLIENT: **Stockdale Capital Partners, LLC**

Test Pit No.: B1 **Sample No.:** S7 **Depth of Sample:** 17.5 feet **Test Date:** 6/27/2017

Soil Description: Silt (ML)

Liquid Limit Determination

Can No.	A	B	C	D	E
Soil Wet Wt. + Can (g)	31.15	30.51	30.54	30.32	30.28
Soil Dry Wt. + Can (g)	27.18	26.70	26.90	26.90	26.60
Wt. of Can (g)	19.40	19.06	19.46	19.46	18.71
Wt. of Dry Soil (g)	7.78	7.64	7.44	7.44	7.89
Wt. of Moisture (g)	3.97	3.81	3.64	3.42	3.68
Water Content (%)	**51.0**	**49.9**	**48.9**	**46.0**	**46.6**
Number of Blows	**15**	**24**	**34**	**42**	**52**

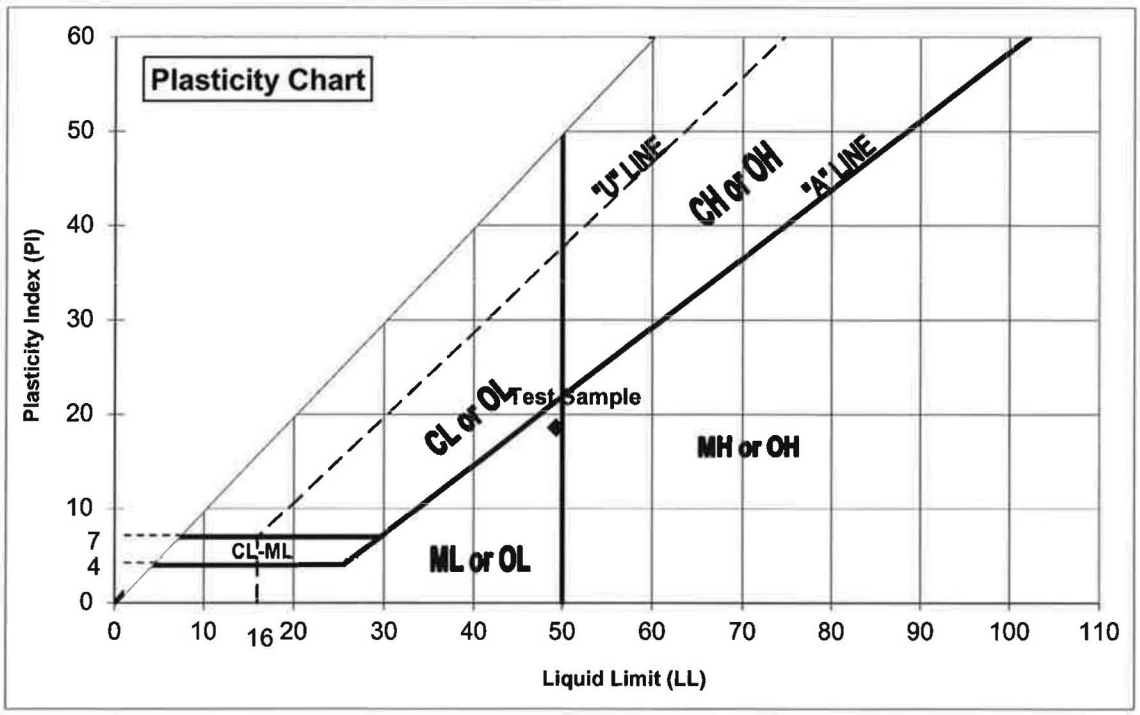


Plastic Limit Determination

Can No.	F				
Soil Wet Wt. + Can (g)	25.68				
Soil Dry Wt. + Can (g)	24.08				
Wt. of Can (g)	18.86				
Wt. of Dry Soil (g)	5.22				
Wt. of Moisture (g)	1.60				
Water Content (%)	**30.7**				

Average Water Content (%) = **30.7**

Liquid Limit, LL = 49.3
Plastic Limit, PL = 30.7
Plasticity Index, PI = 18.6



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PLASTICITY CHART #5

BG: **22581** ENGINEER: **RSB**

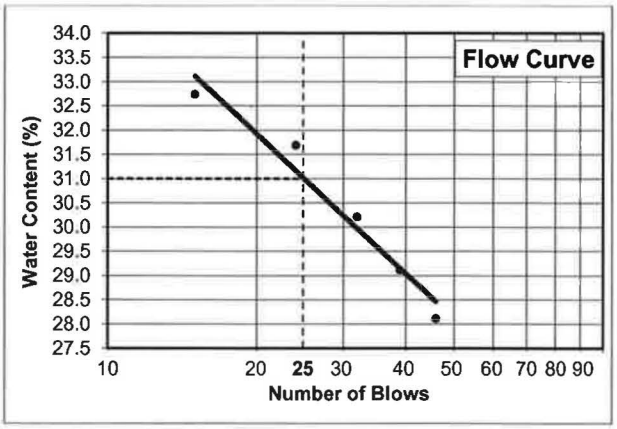
CLIENT: **Stockdale Capital Partners, LLC**

Test Pit No.: B1 **Sample No.:** S9 **Depth of Sample:** 22.5 feet **Test Date:** 6/28/2017

Soil Description: Clay (CL)

Liquid Limit Determination

Can No.	A	B	C	D	E
Soil Wet Wt. + Can (g)	30.26	30.62	29.86	30.78	30.36
Soil Dry Wt. + Can (g)	27.52	27.85	27.38	28.11	28.08
Wt. of Can (g)	19.15	19.11	19.17	18.94	19.97
Wt. of Dry Soil (g)	8.37	8.74	8.21	9.17	8.11
Wt. of Moisture (g)	2.74	2.77	2.48	2.67	2.28
Water Content (%)	**32.7**	**31.7**	**30.2**	**29.1**	**28.1**
Number of Blows	**15**	**24**	**32**	**39**	**46**

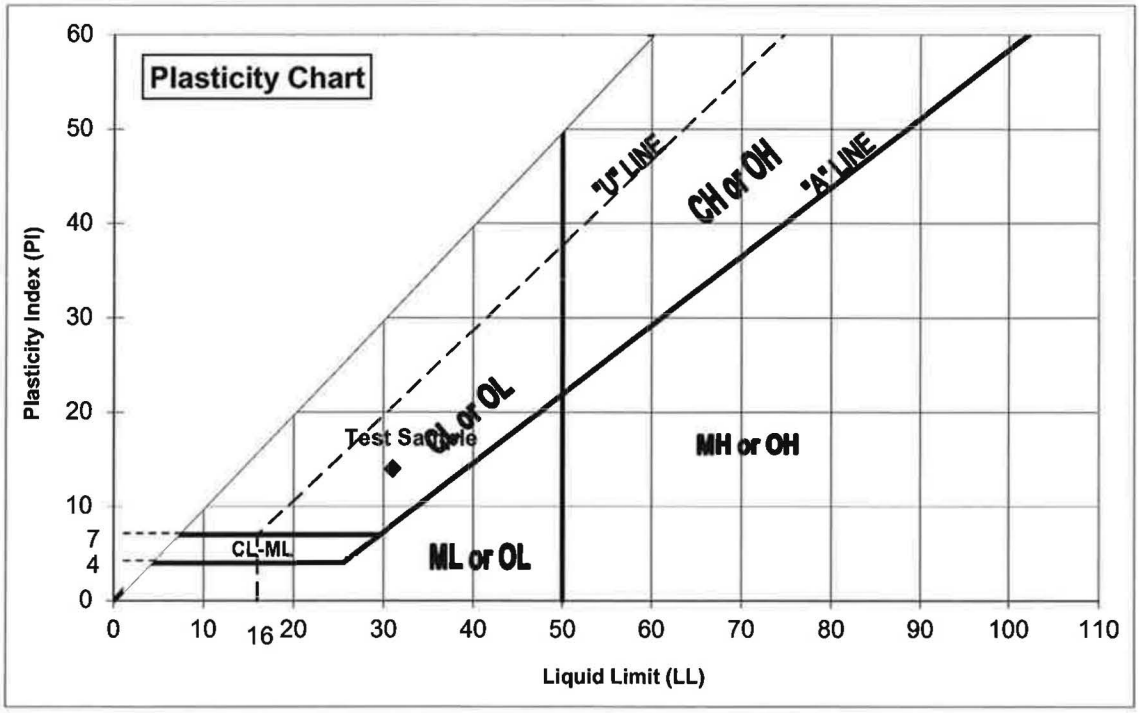


Plastic Limit Determination

Can No.	F				
Soil Wet Wt. + Can (g)	25.88				
Soil Dry Wt. + Can (g)	24.85				
Wt. of Can (g)	18.79				
Wt. of Dry Soil (g)	6.06				
Wt. of Moisture (g)	1.03				
Water Content (%)	**17.0**				

Average Water Content (%) = **17.0**

Liquid Limit, LL = 31.0
Plastic Limit, PL = 17.0
Plasticity Index, PI = 14.0



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PLASTICITY CHART #6

BG: **22581** ENGINEER: **RSB**

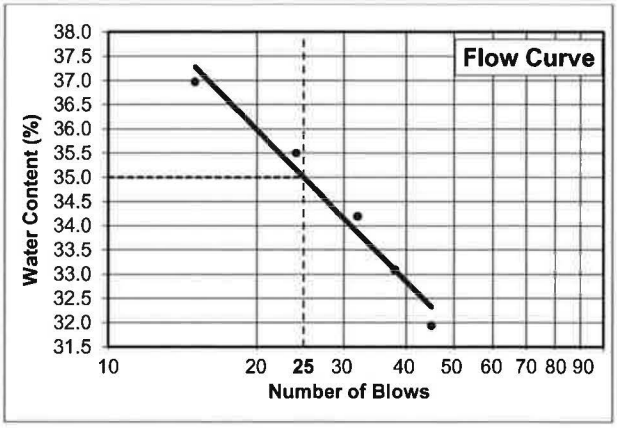
CLIENT: **Stockdale Capital Partners, LLC**

Test Pit No.: B4 **Sample No.:** R4 **Depth of Sample:** 22.5 feet **Test Date:** 7/12/2017

Soil Description: Clay (CL)

Liquid Limit Determination

Can No.	A	B	C	D	E
Soil Wet Wt. + Can (g)	29.81	30.59	30.07	29.87	29.85
Soil Dry Wt. + Can (g)	26.93	27.65	27.30	27.15	27.18
Wt. of Can (g)	19.14	19.37	19.20	18.93	18.82
Wt. of Dry Soil (g)	7.79	8.28	8.10	8.22	8.36
Wt. of Moisture (g)	2.88	2.94	2.77	2.72	2.67
Water Content (%)	**37.0**	**35.5**	**34.2**	**33.1**	**31.9**
Number of Blows	**15**	**24**	**32**	**38**	**45**

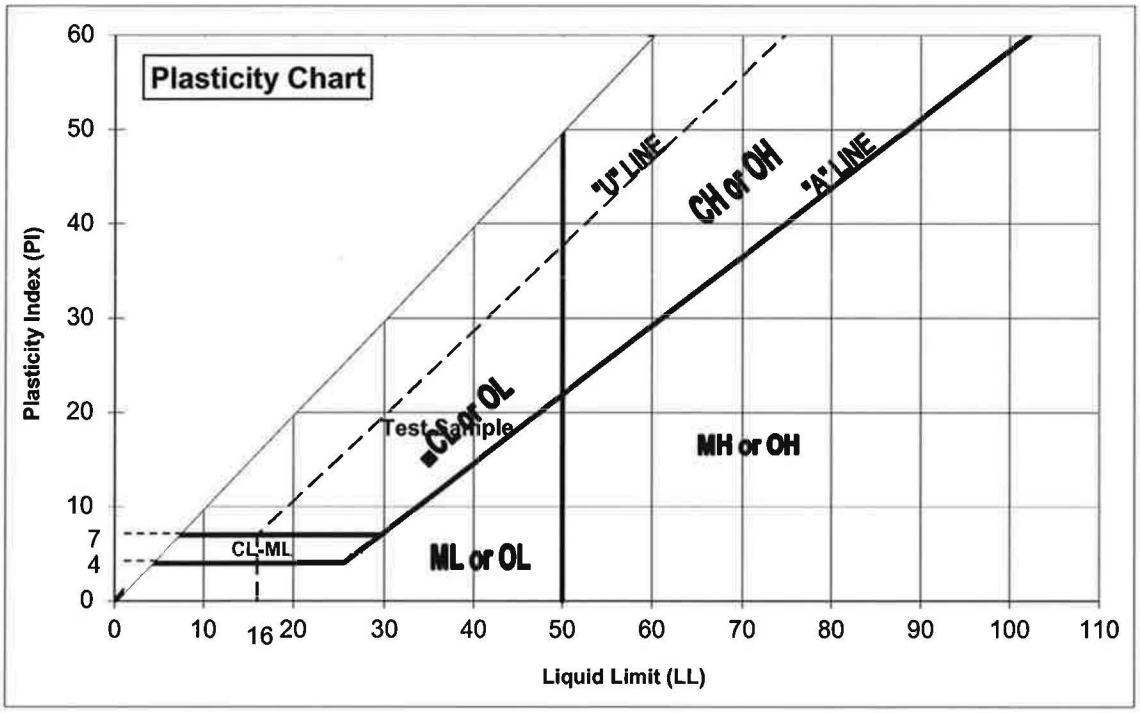


Plastic Limit Determination

Can No.	F				
Soil Wet Wt. + Can (g)	25.78				
Soil Dry Wt. + Can (g)	24.61				
Wt. of Can (g)	18.74				
Wt. of Dry Soil (g)	5.87				
Wt. of Moisture (g)	1.17				
Water Content (%)	**19.9**				

Average Water Content (%) = **19.9**

Liquid Limit, LL = 35.0
Plastic Limit, PL = 19.9
Plasticity Index, PI = 15.1



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PLASTICITY CHART #7

BG: **22581** ENGINEER: **RSB**

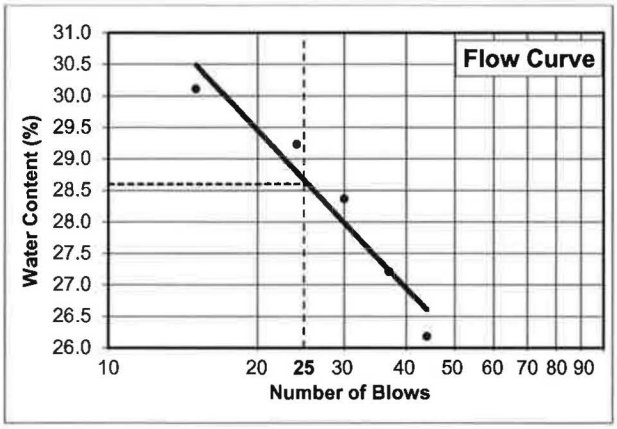
CLIENT: **Stockdale Capital Partners, LLC**

Test Pit No.: B4 **Sample No.:** R5 **Depth of Sample:** 27.5 feet **Test Date:** 7/12/2017

Soil Description: Clay (CL)

Liquid Limit Determination

Can No.	A	B	C	D	E
Soil Wet Wt. + Can (g)	30.48	29.71	29.33	29.66	30.08
Soil Dry Wt. + Can (g)	27.83	27.31	27.14	27.32	27.92
Wt. of Can (g)	19.03	19.10	19.42	18.72	19.67
Wt. of Dry Soil (g)	8.80	8.21	7.72	8.60	8.25
Wt. of Moisture (g)	2.65	2.40	2.19	2.34	2.16
Water Content (%)	**30.1**	**29.2**	**28.4**	**27.2**	**26.2**
Number of Blows	**15**	**24**	**30**	**37**	**44**

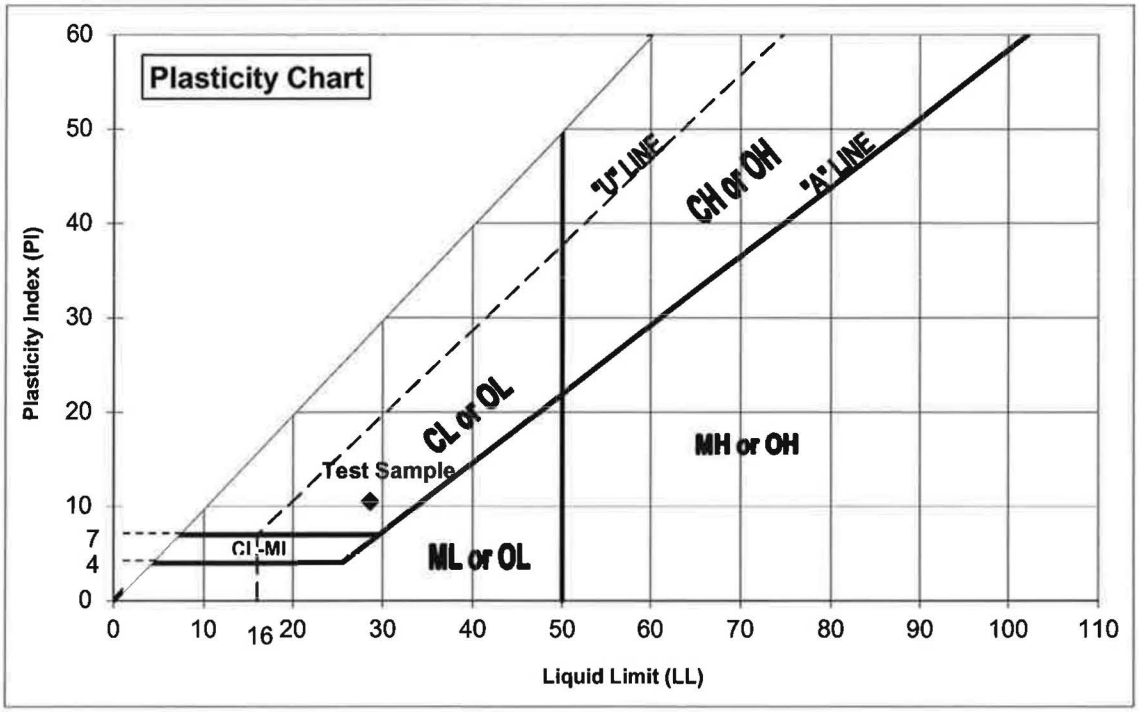


Plastic Limit Determination

Can No.	F				
Soil Wet Wt. + Can (g)	25.63				
Soil Dry Wt. + Can (g)	24.57				
Wt. of Can (g)	18.70				
Wt. of Dry Soil (g)	5.87				
Wt. of Moisture (g)	1.06				
Water Content (%)	**18.1**				

Average Water Content (%) = **18.1**

Liquid Limit, LL = 28.6
Plastic Limit, PL = 18.1
Plasticity Index, PI = 10.5



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LOG OF BORING B1

BG No. 22581
PAGE 1 OF 4

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 6/14/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 5 inches asphalt over 3 inches base (parking lot).								
		(CL) **ALLUVIUM (Qa):** 0.75' - 2.5': CLAY with sand, dark brown, moist, fine sand.		CL						
140		(CL) 2.5': CLAY with sand, dark brown, moist, stiff, fine sand, 71.9% fines, LL= 44.7, PI= 22.7.		CL	S1	4 4 4	13.5			Atterberg Limits, Sieve Wash (-#200)
	5	(CL) 5': CLAY with sand, brown, moist, very stiff, fine sand.		CL	S2	3 8 9	13.3			
135		(CL) 7.5': CLAY, yellowish-brown, moist, very stiff.		CL	S3	5 7 9	25.4			
	10	(MH) 10': SILT, yellowish-brown, moist, stiff, 85.4% fines, LL= 59, PI= 27.1.		MH	S4	3 4 7	24			Atterberg Limits, Sieve Wash (-#200)
130		(CL) 12.5': CLAY, olive-brown to dark brown, moist, stiff, some fine sand, 87.2% fines.		CL	S5	3 5 8	23.7			Sieve Wash (-#200)
	15	(CL) 15': CLAY, brown, moist, stiff, with fine sand, 60.8% fines, LL= 37.4, PI= 18.4.		CL	S6	1 3 5	28.5			Atterberg Limits, Sieve Wash (-#200)
125		(ML) 17.5': SILT, brown, moist, stiff, 89.2% fines, LL= 49.3, PI= 18.6.		ML	S7	2 4 7	21.3			Atterberg Limits, Sieve Wash (-#200)
	20	(ML) 20': SILT, brown, moist, stiff to very stiff, some fine sand, 84.6% fines.		ML	S8	3 7 8	26.7			Sieve Wash (-#200)
120		(CL) 22.5': CLAY, olive-brown, moist, stiff, with fine sand, 68.8% fines, LL= 31, PI= 14.		CL	S9	2 6 9	23.8			Atterberg Limits, Sieve Wash (-#200)
	25									

Standard Penetration Test

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LOG OF BORING B1

BG No. 22581

CLIENT Stockdale Capital Partners, LLC REPORT DATE 7/25/19 DRILL DATE 6/14/17

PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA LOGGED BY RSB

CONTRACTOR Martini Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	(ML) 25': Sandy SILT, light olive-brown, moist, very stiff, fine sand, 72.4% fines.		ML	S10	6 12 12	16			Sieve Wash (-#200)
115		(CL) 27.5': Sandy CLAY, light olive-brown, moist, medium dense, fine sand, 65.4% fines.		CL	S11	2 4 6	18.8			Sieve Wash (-#200)
	30	(CL) **SAN PEDRO FORMATION (Qsp):** 30': Sandy CLAY, light olive-brown, very moist, stiff, fine sand.		CL	S12	2 3 5	34.2			
110		(CL) 32.5': CLAY, light bluish-gray, moist to very moist, stiff, trace fine sand, 89.9% fines.		CL	S13	2 7 7	28.1			Sieve Wash (-#200)
		▽ 34': Groundwater.								
	35	(CL) 35': CLAY, light bluish-gray, moist to very moist, stiff to very stiff, trace fine sand.		CL	S14	3 7 8	25.7			
105		(CL) 37.5': CLAY, light bluish-gray, very moist, stiff, some fine sand, 73.8% fines.		CL	S15	8 10 10	38.1			Sieve Wash (-#200)
	40	(CL) 40': CLAY, light bluish-gray, moist to very moist, stiff to very stiff.		CL	S16	4 5 10	20.7			
100		(CL) 42.5': Sandy CLAY, bluish-gray, moist to very moist, very stiff, fine sand, 61.6% fines.		CL	S17	4 7 9	24.3			Sieve Wash (-#200)
	45	(CL) 45': Sandy CLAY, bluish-gray, moist to very moist, stiff, fine sand, 52.2% fines.		CL	S18	5 6 7	26.6			Sieve Wash (-#200)
95		(ML) 47.5': Sandy SILT, bluish-gray, moist to very moist, stiff, fine sand.		ML	S19	6 6 8	27.4			
	50									

Standard Penetration Test

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LOG OF BORING B1

BG No. 22581
PAGE 3 OF 4

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 6/14/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	50	(ML) 50': Sandy SILT, gray, moist to very moist, stiff, fine sand, 62.8% fines.		ML	S20	2 6 7	27			Sieve Wash (-#200)
90		(ML) 52.5': Sandy SILT, gray, very moist, very stiff, fine sand.		ML	S21	3 8 11	30			
	55	(SP) 55': SAND, gray, very moist, medium dense, fine to medium sand, some fines.		SP	S22	6 9 10	21.7			
85		(SM) 57.5': Silty SAND, gray, very moist, medium dense, fine sand, 32.4% fines.		SM	S23	3 7 14	24.8			Sieve Wash (-#200)
	60	(ML) 60': Sandy SILT, gray, moist to very moist, very stiff, fine to medium sand, 55.9% fines.		ML	S24	2 6 11	23.8			Sieve Wash (-#200)
80		(SP) 62.5': SAND, gray, moist to very moist, dense, fine to medium sand.		SP	S25	4 9 27	16.1			
	65	(SP) 65': SAND, gray, very moist, dense, fine to medium sand.		SP	S26	5 19 24	18.6			
75		(SP) 67.5': SAND, gray, very moist, dense, fine to medium sand, trace coarse sand.		SP	S27	6 17 23	17			
	70	(SP) 70': SAND, gray, very moist, dense, fine to medium sand.		SP	S28	5 8 22	25.4			
70										
	75									

Standard Penetration Test

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LOG OF BORING B1

BG No. 22581
PAGE 4 OF 4

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 6/14/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	75	(SP) 75': SAND, gray, very moist, dense, fine to medium sand.		SP	S29	5 18 29	19.7			
65	80	(SP) 80': SAND, gray, very moist, medium dense, fine to medium sand.		SP	S30	5 9 15	18.3			
60	85	(SM) 85': Silty SAND, gray, moist to very moist, dense, fine to medium sand, 43.5% fines.		SM	S31	7 13 17	16.4			Sieve Wash (-#200)
55	90	(SM) 90': From cuttings: Silty SAND, gray, very moist, medium dense, fine sand.		SM	S32	6 8 15				No Recovery
50	95	(SM) 95': Top: Silty SAND, gray, very moist, medium dense to dense, fine sand.		SM	S33	8 12 16	18.7			
		(CL) Bottom: CLAY, dark gray, moist, very stiff to hard, tough.		CL						

End at 96.5 Feet; Groundwater at 34 Feet.

Standard Penetration Test

BYER GEOTECHNICAL, INC.

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LOG OF BORING B2

BG No. 22581
PAGE 1 OF 3

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 6/14/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 4 inches asphalt over 6 inches base (parking lot).								
		(CL) **ALLUVIUM (Qa):** 0.85' - 2.5': CLAY with sand, dark brown, moist, fine sand.		CL						
140		(CL) 2.5': CLAY with sand, dark brown, moist, stiff, fine sand.		CL	R1	3 7 12	18.8	104.6	85.6	
	5	(CL) 5': CLAY with sand, dark brown, moist, very stiff, fine sand, tough.		CL	R2	5 13 25	16.3	111.5	89.3	Direct Shear
135										
	10	(CL) 10': CLAY, brown, moist, stiff, some fine sand.		CL	R3	4 8 12	25.8	101	100	Direct Shear
130										
	15	(CL) 15': CLAY, brown, moist, stiff, some fine sand.		CL	R4	4 9 21	22.6	106.1	100	Direct Shear, Consolidation
125		(CL) 17.5': CLAY, brown, moist, medium stiff, trace fine sand.		CL	R5	2 4 4	25.7	98.5	100	
	20	(ML) 20': SILT, brown, moist, stiff, some fine sand.		ML	R6	2 6 11	25.2	100.9	100	Direct Shear
120		(CL) 22.5': CLAY, brown, moist, stiff, with fine sand, 62.7% fines.		CL	R7	5 7 9	17.9	103.7	79.8	
	25									

■ Ring Sample

BYER GEOTECHNICAL, INC.

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LOG OF BORING B2

BG No. 22581

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 6/14/17

PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB

CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	(CL) 25': CLAY with sand, brown, moist, very stiff, fine sand.		CL	R8	6 12 14	17.6	111.8	97.2	Direct Shear
115		(CL) 27.5': Sandy CLAY, olive-brown, moist, very stiff, fine sand, some medium sand.		CL	R9	6 12 15	16.6	112.1	92.6	Consolidation
	30	(CL) **SAN PEDRO FORMATION (Qsp):** 30': Sandy CLAY, olive-brown, very moist, stiff, fine sand.		CL	R10	5 9 11	35.7	86.3	100	
110		(CL) 32.5': CLAY, light bluish-gray, moist to very moist, stiff, trace fine sand.		CL	R11	3 7 12	31	93.4	100	
	35	▽ 34': Groundwater.								
		(CL) 35': CLAY, bluish-gray, very moist, stiff to very stiff, trace fine sand, moderately tough.		CL	R12	5 9 16	30.7	93.1	100	Direct Shear
105		(CL) 37.5': CLAY, light bluish-gray, very moist, very stiff, some fine sand.		CL	R13	8 9 26	21.7	100.8	100	Consolidation
	40	(CL) 40': CLAY, light bluish-gray, moist, hard.		CL	R14	4 50	15.8	118.6	100	Direct Shear
100										
	45	(CL) 45': Sandy CLAY, bluish-gray, moist to very moist, very stiff, with fine sand.		CL	R15	8 13 20	19.2	111.8	100	Direct Shear, Consolidation
95										
	50									

■ Ring Sample

BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

LOG OF BORING B2

BG No. 22581
PAGE 3 OF 3

CLIENT Stockdale Capital Partners, LLC REPORT DATE 7/25/19 DRILL DATE 6/14/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA LOGGED BY RSB
CONTRACTOR Martini Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	50	(ML) 50': Sandy SILT, dark bluish-gray, moist, very stiff, fine to medium sand, some coarse sand.		ML	R16	5 16 18	14.6	114.2	100	Consolidation
90	55	(CL) 55': CLAY, dark bluish-gray, very moist, stiff, some fine sand.		CL	R17	4 8 12	28.3	97.6	100	Direct Shear
85	60	(ML) 60': Sandy SILT, dark bluish-gray, moist to very moist, very stiff, fine sand.		ML	R18	8 17 20	20.6	108.9	100	Consolidation
80	65	(ML) 65': Sandy SILT, dark bluish-gray, very moist, very stiff, fine sand.		ML	R19	10 15 23	36.3	87.5	100	Direct Shear
75	70	(SP) 70': SAND, dark gray, very moist, medium dense, fine sand.		SP	R20	6 11 29	24.7	102.3	100	Consolidation

End at 71.5 Feet; Groundwater at 34 Feet.

Ring Sample

BYER GEOTECHNICAL, INC.

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818.543.3747 FAX

LOG OF BORING B3

BG No. 22581
PAGE 1 OF 4

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 7/8/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 144 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 4 inches asphalt over 2 inches base (parking lot).								
		(CL) **ALLUVIUM (Qa):** 0.5' - 2.5': CLAY, dark brown, moist, trace fine sand.		CL						
		(CL) 2.5': CLAY, dark brown, moist, medium stiff, trace fine sand.		CL	S1	2 3 4				
140	5	(CL) 5': CLAY, dark brown, moist, stiff, trace fine sand.		CL	R1	5 9 13	18.6	112.3	100	
		(CL) 7.5': CLAY, brown, moist, stiff, some fine sand.		CL	S2	2 5 8				
135	10	(CL) 10': CLAY, tan to brown, moist, stiff to very stiff, trace fine sand.		CL	R2	5 10 13	20.8	104.8	95.5	
130	15	(CL) 15': CLAY, brown, moist, very stiff, some fine sand.		CL	S3	3 10 12				
125	20	(CL) 20': CLAY, brown to olive-brown, very moist, stiff, trace fine sand.		CL	R3	3 7 16	31	96.3	100	
		(CL) 22.5': Sandy CLAY, olive-brown, moist, very stiff, fine to medium sand, some coarse sand.		CL	R4	14 19 16	16.7	102.8	72.8	
120	25									

Standard Penetration Test Ring Sample

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LOG OF BORING B3

BG No. 22581

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 7/8/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 144 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	(ML) 25': Sandy SILT, olive-brown, moist, very stiff, fine sand.		ML	R5	8 13 16	23.2	101	96.4	Consolidation
115	30	(CL) **SAN PEDRO FORMATION (Qsp):** 30': CLAY, olive-brown, moist to very moist, stiff, trace fine sand.		CL	S4	2 4 4				
110		▽ 34': Groundwater.								
	35	(CL) 35': CLAY, light bluish-gray, moist to very moist, very stiff, trace fine sand.		CL	R6	5 14 20	23.4	102.1	100	Consolidation
105	40	(CL) 40': Sandy CLAY, light bluish-gray, moist to very moist, stiff, fine to medium sand.		CL	S5	3 4 10				
100	45	(CL) 45': CLAY, dark bluish-gray, moist to very moist, stiff, some fine sand.		CL	R7	4 8 11	25.7	105.6	100	
95	50									

Standard Penetration Test ▼ Ring Sample ■

BYER GEOTECHNICAL, INC.

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LOG OF BORING B3

BG No. 22581
PAGE 3 OF 4

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 7/8/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 144 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	50	(CL) 50': CLAY, dark bluish-gray, very moist, medium stiff to stiff.		CL	S6	2 3 5				
90	55	(CL) 55': CLAY, dark bluish-gray, very moist, very stiff.		CL	R8	5 16 23	32	95.2	100	
85	60	(ML) 60': Sandy SILT, gray, moist to very moist, stiff, fine to medium sand.		ML	S7	3 5 7				
80	65	(SW) 65': SAND, gray, moist to very moist, dense, fine to coarse sand.		SW	R9	15 25 40	15.2	117.1	100	
75	70	(SM) 70': Silty SAND, gray, very moist, medium dense, fine to medium sand.		SM	S8	5 9 15				
70	75									

Standard Penetration Test

Ring Sample

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LOG OF BORING B3

BG No. 22581

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 7/8/17

PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB

CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 144 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	75	(SW) 75': SAND, dark gray, very moist, dense, fine to coarse sand, some fines.		SW	R10	14 25 37	16.4	112.7	100	
65	80	(CL) 80': Sandy CLAY, gray, very moist, very stiff, fine to medium sand.		CL	S9	4 8 13				

End at 81.5 Feet; Groundwater at 34 Feet.

Standard Penetration Test Ring Sample

BYER GEOTECHNICAL, INC.

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LOG OF BORING B4

BG No. 22581
PAGE 1 OF 3

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 7/8/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 4 inches asphalt over 2 inches base (parking lot).								
		(CL) **ALLUVIUM (Qa):** 0.5' - 2.5': CLAY, dark brown, moist, trace fine sand.		CL						
140		(CL) 2.5': CLAY, dark brown, moist, stiff, trace fine sand.		CL	R1	4 6 8	24.3	99.7	97.7	
	5	(CL) 5': CLAY, brown, moist, medium stiff to stiff, trace fine sand.		CL	Bag1 S1	2 3 5				
135		(CL) 7.5': CLAY, brown, moist, very stiff, some fine to medium sand.		CL	R2	6 11 20	18.8	111.7	100	
	10	(CL) 10': CLAY, brown, moist, stiff, some fine to medium sand.		CL	S2	2 5 9				
130										
	15	(SP) 15': SAND, light yellowish-brown, moist, medium dense, fine to medium sand, some coarse sand, some fines.		SP	R3	6 18 25	7.8	112.5	43.8	
125										
	20	(CL) 20': CLAY, brown, moist, stiff, trace fine sand.		CL	S3	2 4 6				
120		(CL) 22.5': CLAY, olive-brown, moist, very stiff, some fine sand, 64.5% fines, LL= 35, PI= 15.1.		CL	R4	4 12 18	20.9	106.8	100	Atterberg Limits, Sieve Wash (-#200)
	25									

Bulk Sample Ring Sample Standard Penetration Test

BYER GEOTECHNICAL, INC.

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LOG OF BORING B4

BG No. 22581

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 7/8/17
PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	(ML) 25': Sandy SILT, olive-brown, moist, stiff, fine sand.		ML	S4	3 4 7				
115		(CL) 27.5': Sandy CLAY, olive-brown, moist, very stiff, fine sand, 59.7% fines, LL= 28.6, PI= 10.5.		CL	R5	6 18 20	32.3	95.8	100	Atterberg Limits, Sieve Wash (-#200)
	30	(CL) **SAN PEDRO FORMATION (Qsp):** 30': CLAY, olive-brown, moist, very stiff.		CL	R6	8 14 15	25.8	99.4	100	
110										
	35	▽ 35': Groundwater. (CL) 35': CLAY, bluish-gray, moist to very moist, very stiff.		CL	S5	4 7 9				
105										
	40	(CL) 40': Sandy CLAY, light bluish-gray, very moist, medium stiff.		CL	R7	3 4 6	35.6	86.9	100	
100										
	45	(CL) 45': CLAY, bluish-gray, moist to very moist, very stiff, moderately tough.		CL	S6	4 9 12				
95										
	50									

Bulk Sample Ring Sample Standard Penetration Test

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LOG OF BORING B4

BG No. 22581

CLIENT Stockdale Capital Partners, LLC **REPORT DATE** 7/25/19 **DRILL DATE** 7/8/17

PROJECT LOCATION 650 - 674 South San Vicente Blvd., Los Angeles, CA **LOGGED BY** RSB

CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 143 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	50	(ML) 50': Sandy SILT, dark bluish-gray, very moist, very stiff, fine sand.		ML	R8	6 9 14	23.6	101.5	100	
90										
	55	(ML) 55': Sandy SILT, dark bluish-gray, very moist, very stiff, fine sand.		ML	S7	5 9 10				
85										
	60	(ML) 60': Sandy SILT, dark gray, moist, hard, fine to medium sand.		ML	R9	12 42 50	13.3	123.9	100	

End at 61.5 Feet; Groundwater at 35 Feet.

Bulk Sample | Ring Sample | Standard Penetration Test

July 25, 2019
BG 22581

APPENDIX III

Calculations and Figures

SEISMIC SOURCES
EZ-FRISK V7.65

DETERMINISTIC CALCULATION
OF PEAK GROUND ACCELERATION BASED ON DIGITIZED FAULT DATA

BG: 22581
CLIENT: Stockdale Capital Partners, LLC ENGINEER: RSB
PROJECT DESCRIPTION: Proposed 12-Story at-Grade Mixed-Use Building

SITE COORDINATES: LATITUDE: 34.0648
LONGITUDE: -118.3718

SEARCH RADIUS: 100 km

ATTENUATION RELATIONS: CHIOU-YOUNGS (2007) NGA USGS 2008 MRC
BOORE-ATKINSON (2008) NGA USGS 2008 MRC
CAMPBELL-BOZORGNIA (2008) NGA USGS 2008 MRC

SEISMIC SOURCE SUMMARY
DETERMINISTIC SITE PARAMETERS

FAULT NAME	APPROXIMATE DISTANCE		MAXIMUM EATHQUAKE MAGNITUDE	PEAK GROUND ACCELERATION
	(km)	(mi)	(Mw)	(g)
Santa Monica	1.4	0.9	7.4	0.757
Newport-Inglewood	2.9	1.8	7.5	0.624
Hollywood	3.4	2.1	6.7	0.550
Puente Hills (LA)	5.4	3.3	7.0	0.582
Puente Hills	8.5	5.3	7.1	0.523
Elysian Park (Upper)	9.2	5.7	6.7	0.431
Malibu Coast	14.6	9.1	7.0	0.311
Raymond	15.1	9.4	6.8	0.287
Verdugo	16.1	10.0	6.9	0.284
Anacapa-Dume	17.4	10.8	7.2	0.325
Palos Verdes	20.0	12.4	7.3	0.272
Palos Verdes Connected	20.0	12.4	7.7	0.309
Puente Hills (Santa Fe Springs)	22.1	13.8	6.7	0.266
Sierra Madre	22.9	14.2	7.2	0.247
Sierra Madre Connected	22.9	14.2	7.3	0.256
Northridge	23.5	14.6	6.9	0.304

FAULT NAME	APPROXIMATE DISTANCE		MAXIMUM EATHQUAKE MAGNITUDE	PEAK GROUND ACCELERATION
	(km)	(mi)	(Mw)	(g)
Sierra Madre (San Fernando)	23.8	14.8	6.7	0.198
San Gabriel	29.4	18.3	7.3	0.208
Santa Susana, alt 1	30.2	18.7	6.9	0.179
Elsinore	30.8	19.2	7.9	0.250
Puente Hills (Coyote Hills)	35.1	21.8	6.9	0.180
Clamshell-Sawpit	36.1	22.4	6.7	0.142
Holser, alt 1	39.3	24.4	6.8	0.147
Simi-Santa Rosa	40.0	24.8	6.9	0.140
Oak Ridge Connected	44.9	27.9	7.4	0.177
San Jose	44.9	27.9	6.7	0.116
Oak Ridge (Onshore)	47.2	29.4	7.2	0.163
Chino	52.1	32.4	6.8	0.102
San Cayetano	54.7	34.0	7.2	0.124
San Joaquin Hills	57.8	35.9	7.1	0.125
Cucamonga	58.4	36.3	6.7	0.090
Southern San Andreas	59.1	36.7	8.2	0.186
Santa Ynez (East)	72.4	45.0	7.2	0.094
Santa Ynez Connected	72.7	45.2	7.4	0.104
Ventura-Pitas Point	77.2	48.0	7.0	0.088
Pitas Point Connected	77.2	48.0	7.3	0.162
San Jacinto	78.5	48.8	7.9	0.125
Oak Ridge (Offshore)	80.1	49.8	7.0	0.077
Santa Cruz Island	82.4	51.2	7.2	0.081
Channel Islands Thrust	82.4	51.2	7.3	0.104
Mission Ridge-Arroyo Parida-Santa Ana	83.6	52.0	6.9	0.069
Cleghorn	87.9	54.6	6.8	0.059
Red Mountain	91.0	56.5	7.4	0.084

43 Faults found within a 100 km Search Radius.
Closest Fault to the Site: Santa Monica Distance = 1.4 km (0.87mi)
Largest Peak Ground Acceleration: 0.757 g
The San Andreas Fault is Located Aproximately 59.1 km (36.7 mi) from the Site.

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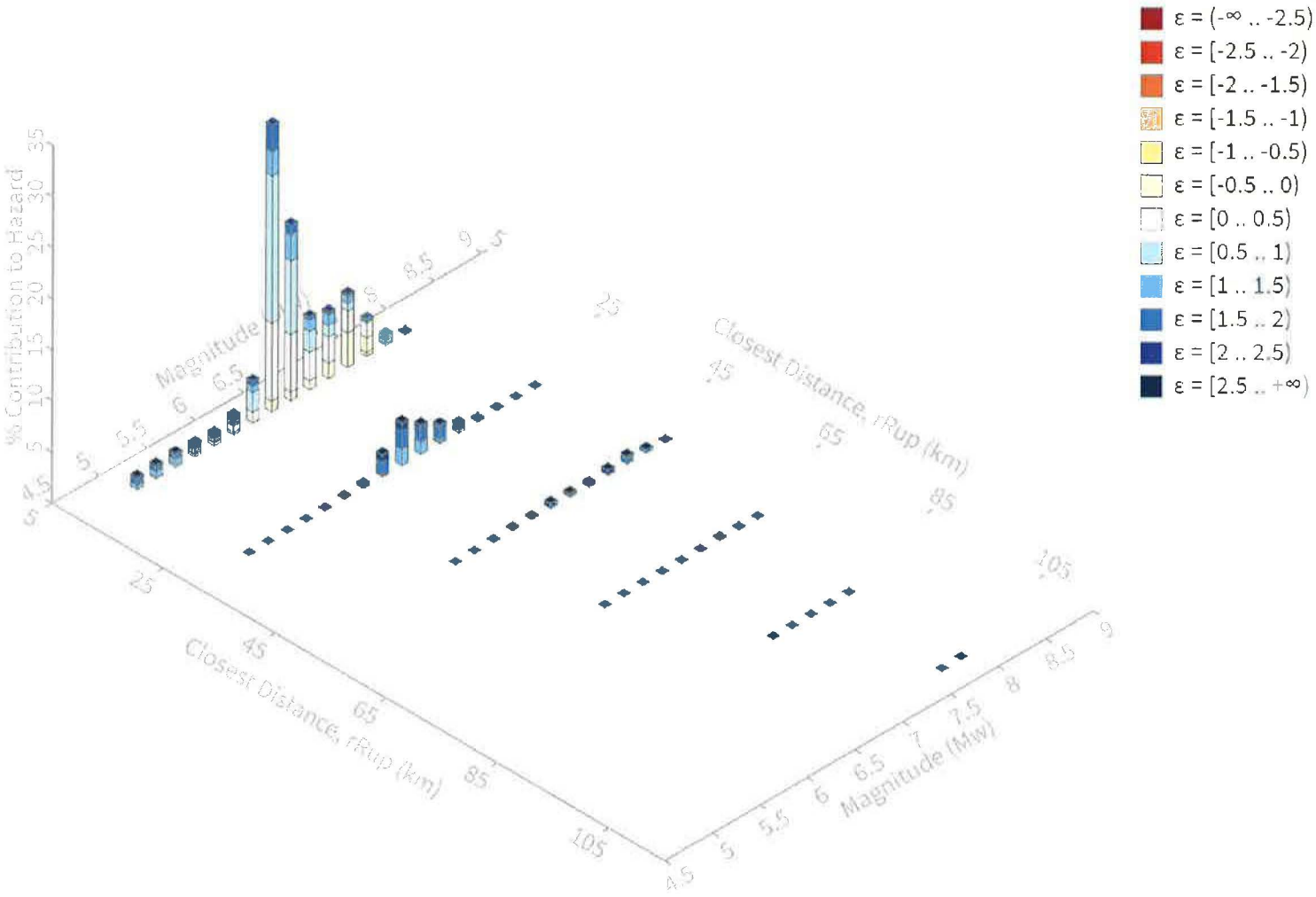
SEISMIC HAZARD DEAGGREGATION CHART 1
(Probability of Exceedance: 10% in 50 years)

BG: 22581

CLIENT: STOCKDALE CAPITAL PARTNERS, LLC

ENGINEER: RSB

REFERENCE: USGS, 2017, Earthquake Hazards Program, Beta - Unified Hazard Tool, Seismic Hazard Deaggregation, Conterminous U.S. 2008 (v3.3.0) Edition, https://earthquake.usgs.gov/hazards/interactive/index.php.



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.47939334 g

Recovered targets

Return period: 504.9298 yrs
Exceedance rate: 0.0019804733 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.1 %

Mean (for all sources)

r: 11.26 km
m: 6.71
ε₀: 0.87 σ

Mode (largest r-m bin)

r: 6.46 km
m: 6.51
ε₀: 0.69 σ
Contribution: 28.08 %

Mode (largest ε₀ bin)

r: 6.14 km
m: 6.51
ε₀: 0.69 σ
Contribution: 14.41 %

Discretization

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

Epsilon keys

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

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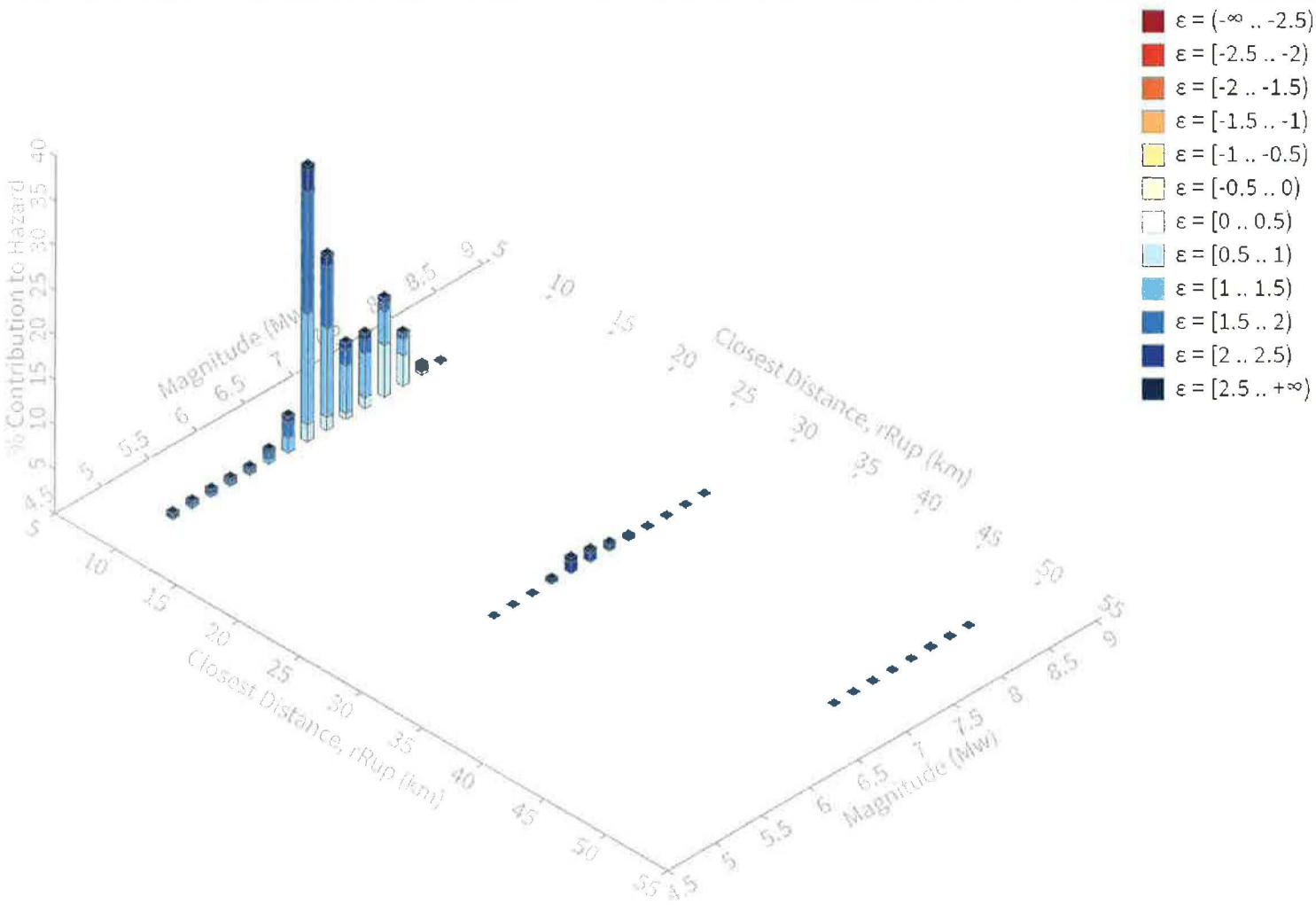
SEISMIC HAZARD DEAGGREGATION CHART 2
(Probability of Exceedance: 2% in 50 years)

BG: 22581

CLIENT: STOCKDALE CAPITAL PARTNERS, LLC

ENGINEER: RSB

REFERENCE: USGS, 2017, Earthquake Hazards Program, Beta - Unified Hazard Tool, Seismic Hazard Deaggregation, Conterminous U.S. 2008 (v3.3.0) Edition, https://earthquake.usgs.gov/hazards/interactive/index.php.



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.78283318 g

Recovered targets

Return period: 2854.168 yrs
Exceedance rate: 0.0003503648 yr^{-1}

Totals

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

Mean (for all sources)

r: 6.49 km
m: 6.76
ε₀: 1.49 σ

Mode (largest r-m bin)

r: 5.26 km
m: 6.51
ε₀: 1.54 σ
Contribution: 30.8 %

Mode (largest ε₀ bin)

r: 6.02 km
m: 6.5
ε₀: 1.69 σ
Contribution: 13.58 %

Discretization

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

Epsilon keys

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

Site-Specific Ground Motion Analysis (Based on ASCE 7-16 Standard)

BG: 22581 Client: Stockdale Capital Partners, LLC

Project Description: Proposed 12-Story Mixed-Use Building Engineer: RSB

Ss (0.2s) =	2.072	Latitude:	34.0648			Periods (seconds):		80% of Sections. 11.4.3 & 11.4.4 of ASCE 7-16	RESULTS Design Values ASCE 7-16 (Section 21.4)
S1 (1s) =	0.739	Longitude:	-118.3718			T_o =	0.178		
Fa =	1.00	Site Class:	D			T_s =	0.892		
Fv =	2.50					T_L =	8		
SMs =	2.072		Fig. 22-18A	S_{MS}=	1.533	<		1.658	1.658
SM1 =	1.848	C_{RS}:	0.897	S_{M1}=	1.478	=		1.478	1.478
SDs =	1.381		Fig. 22-19A	S_{DS}=	1.022	<		1.105	1.105
SD1 =	1.232	C_{R1}:	0.897	S_{D1}=	0.985	=		0.985	0.985

Fundamental Period	Risk Coefficient C_R (Method 1, Section 21.2.1.1, ASCE 7-16)	Probabilistic MCE_R Seismic Response Spectrum (EZ-Frisk & Section 21.2.1.1)	Probabilistic Seismic Response Spectrum (ASCE 7-16, Section 11.4.6)	84th Percentile of Deterministic MCE_R Seismic Response Spectrum (ASCE 7-16)	Deterministic Lower Limit on MCE_R Response Spectrum (ASCE 7-16, Section 21.2.2)	Site Specific MCE_R Spectral Response Acceleration (ASCE 7-16, Section 21.2.3)	80% of Probabilistic Response Spectrum (ASCE 7-16, Section 21.3)	Design Response Spectrum (ASCE 7-16, Section 21.3)
T (sec)		Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)
0.0	0.897	0.9329	0.5525	0.7565	0.600	0.757	0.442	0.504
0.1	0.897	1.5644	1.0182	1.1910	1.050	1.191	0.815	0.815
0.2	0.897	1.9878	1.3813	1.5330	1.500	1.533	1.105	1.105
0.3	0.897	1.9797	1.3813	1.6560	1.500	1.656	1.105	1.105
0.4	0.897	1.9160	1.3813	1.6980	1.500	1.698	1.105	1.132
0.5	0.897	1.8335	1.3813	1.7030	1.500	1.703	1.105	1.135
0.6	0.897	1.6801	1.3813	1.6220	1.500	1.622	1.105	1.105
0.7	0.897	1.5590	1.3813	1.5560	1.500	1.556	1.105	1.105
0.8	0.897	1.4352	1.3813	1.4610	1.500	1.435	1.105	1.105
0.9	0.897	1.3132	1.3685	1.3450	1.500	1.313	1.095	1.095
1.0	0.897	1.2154	1.2317	1.2470	1.500	1.215	0.985	0.985
1.1	0.897	1.1123	1.1197	1.1370	1.364	1.112	0.896	0.896
1.2	0.897	1.0280	1.0264	1.0430	1.250	1.028	0.821	0.821
1.3	0.897	0.9571	0.9474	0.9598	1.154	0.957	0.758	0.758
1.4	0.897	0.8962	0.8798	0.8867	1.071	0.896	0.704	0.704
1.5	0.897	0.8304	0.8211	0.8217	1.000	0.830	0.657	0.657
1.6	0.897	0.7690	0.7698	0.7604	0.938	0.769	0.616	0.616
1.7	0.897	0.7166	0.7245	0.7065	0.882	0.717	0.580	0.580
1.8	0.897	0.6717	0.6843	0.6590	0.833	0.672	0.547	0.547
1.9	0.897	0.6329	0.6482	0.6172	0.789	0.633	0.519	0.519
2.0	0.897	0.5965	0.6158	0.5803	0.750	0.597	0.493	0.493
3.0	0.897	0.3772	0.4106	0.3789	0.500	0.377	0.328	0.328
4.0	0.897	0.2718	0.3079	0.2828	0.375	0.272	0.246	0.246
5.0	0.897	0.2144	0.2463	0.2282	0.300	0.214	0.197	0.197
6.0	0.897	0.1681	0.2053	0.1801	0.250	0.168	0.164	0.164
7.0	0.897	0.1360	0.1760	0.1466	0.214	0.136	0.141	0.141
8.0	0.897	0.1119	0.1540	0.1177	0.188	0.112	0.123	0.123

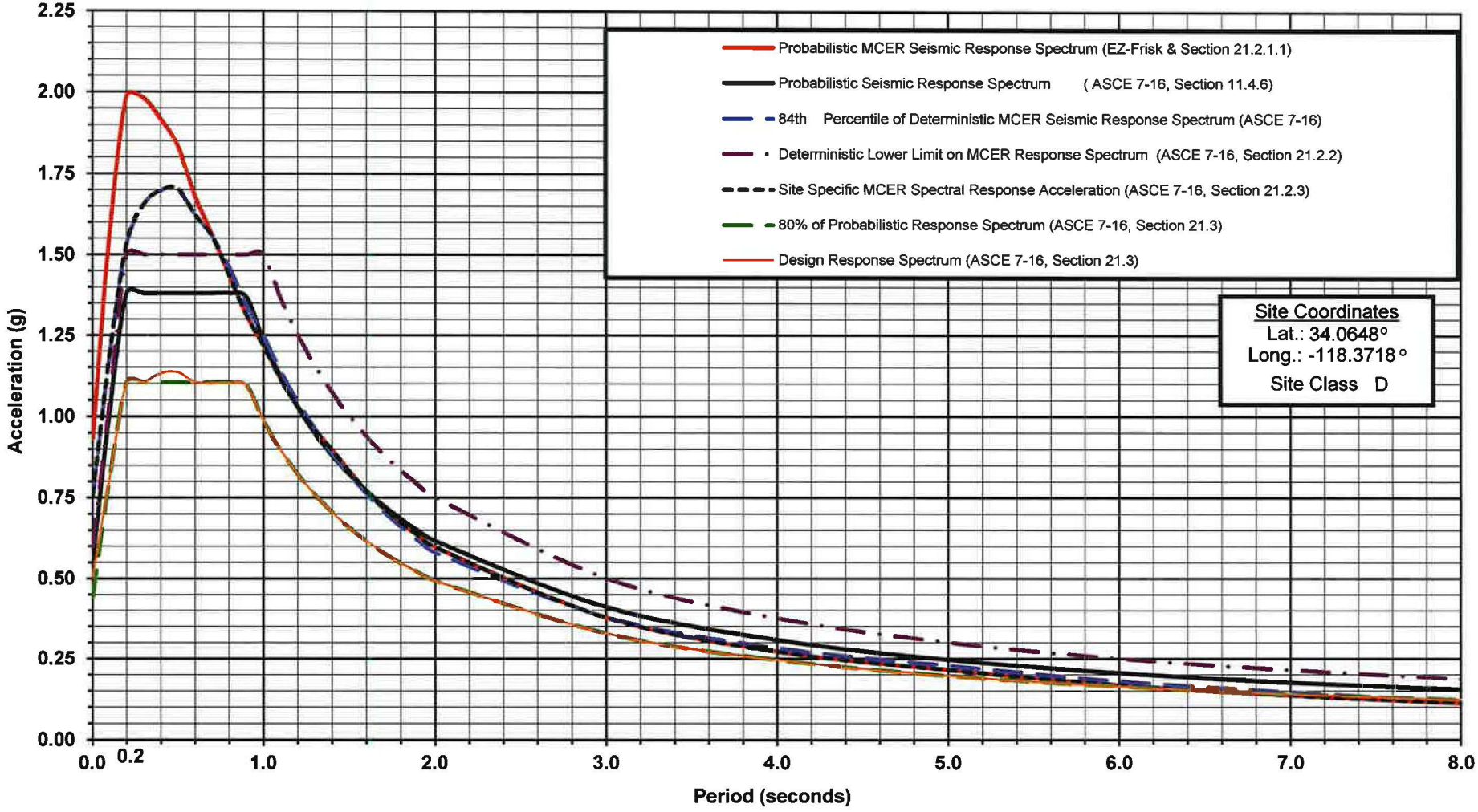
* The Probabilistic and Deterministic Seismic Response Spectra are Based on the Maximum Rotated Component (MRC) of Ground Motion.

References:

- American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Standard ASCE/SEI 7-16, Chapter 21.

- Division of the State Architect (DSA), 2009, Use of the Next Generation Attenuation (NGA) Relations, State of California, Department of General Services, DSA Bulletin 09-01, Effective March 1, 2009.

SEISMIC RESPONSE SPECTRA



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SITE-SPECIFIC SEISMIC RESPONSE SPECTRA

Proposed 12-Story Mixed-Use Building

BG: 22581 Client: Stockdale Capital Partners, LLC

Engineer: RSB

P:\22000 - 22999\22581 Stockdale LLC\Engineering\22581 Ground Motion Analysis 0.84 Percentile_ASCE 7-16

Liquefaction Susceptibility Analysis: SPT Method (475-Yr Return)
(Input Data)

Project No.: 22581 Client: Stockdale Capital Partners, LLC

Project Description.: Proposed 12-Story t-Grade Mixed-Use Building

Engineer: RSB

Boring No.	Top Elevation (ft)	Total Depth (ft)	Existing GW Depth (ft)	Design GW Depth (ft)	Recommended Fill Depth (ft)
B1	143	95	34	15	0

Peak Ground Acceleration:	0.65
Earthquake Magnitude:	6.51
Probability of Exceedance in 50 Years:	10%
Borehole Diamter (inches):	8
Delivered Energy Ratio, ERm (%):	75
Energy Ratio Correction Factor, C_E:	1.25
Borehole Diameter Correction Factor, C_B:	1.15
Rod Length Correction Factor, C_R:	1
Sampler Correction with or without Liners, C_S:	1
Minimum Factor of Safety, FS_{liq}:	1.1

References: *- Youd, T. L., et. al. (2001), Liquefaction Resistane of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, October 2001.*

- Tokimatsu and Seed (1987), Evaluation of Settlements in Sands due to Earthquake Shaking, American Society for Civil Engineers, Journal of Geotechnical Engineering, Vol. 113, No. 8, August, 1987.

- California Geological Survey (2008), Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

- County of Los Angeles, Department of Public Works (2009), Liquefaction/Lateral Spreading, Administrative Manual, Publication No. GS 045-0, May 28, 2009.

(USCS)	(Based on Laboratory Testing, Existing Conditions, and Project Configuration)		FC (%)	PI (%)	LL (%)	Content w_c (%)		γ_t (pcf)	Nm (blows/ft)				(Current) (GW)	(Hist.) (GW)	(2001)				Clean Sand	Coef. r_d				w M
CL	PI>18 & Insensitive		71.9	22.7	44.7			120	8	0.75	8.6	300.0	300.0	300.0	1.70	14.7	5.00	1.20	22.6	0.994	0.420	1.44	0.251	Nor
CL	Above Hist. GW							120	17	0.80	19.6	600.0	600.0	600.0	1.70	33.2	0.00	1.00	33.2	0.988	0.417	1.44	(N1)60cs >= 30	1.1
CL	Above Hist. GW							120	16	0.80	18.4	900.0	900.0	900.0	1.53	28.2	0.00	1.00	28.2	0.983	0.415	1.44	0.377	Nor
MH	PI>18 & Insensitive		85.4	27.1	59.0			120	11	0.85	13.4	1200.0	1200.0	1200.0	1.33	17.8	5.00	1.20	26.4	0.977	0.413	1.44	0.323	Nor
CL	Above Hist. GW		87.2					120	13	0.85	15.9	1500.0	1500.0	1500.0	1.19	18.9	5.00	1.20	27.6	0.971	0.410	1.44	0.357	Nor
CL	PI>18 & Insensitive		60.8	18.4	37.4			120	8	0.95	10.9	1800.0	1800.0	1800.0	1.08	11.8	5.00	1.20	19.2	0.965	0.408	1.44	0.206	Nor
CL	PI>18 & Insensitive		89.2	18.6	49.3			120	11	0.95	15.0	2100.0	2100.0	1944.0	1.00	15.1	5.00	1.20	23.1	0.959	0.438	1.44	0.258	Nor
ML			84.6					120	16	0.95	21.9	2400.0	2400.0	2088.0	0.94	20.5	5.00	1.20	29.6	0.953	0.463	1.44	0.443	0.6
CL	Wc/LL <= 0.8		68.8	14.0	31.0	23.8	0.77	120	14	0.95	19.1	2700.0	2700.0	2232.0	0.89	16.9	5.00	1.20	25.3	0.948	0.485	1.44	0.298	Nor
ML			72.4					120	24	0.95	32.8	3000.0	3000.0	2376.0	0.84	27.5	5.00	1.20	38.0	0.942	0.503	1.44	(N1)60cs >= 30	1.1
CL	PI< 12		65.4	10.5				120	10	0.95	13.7	3300.0	3300.0	2520.0	0.80	10.9	5.00	1.20	18.1	0.936	0.518	1.44	0.193	0.2
CL	Bedrock							120	8	1.00	11.5	3600.0	3600.0	2664.0	0.77	8.8	0.00	1.00	8.8	0.93	0.531	1.44	0.103	Nor
CL	Bedrock		89.9					120	14	1.00	20.1	3900.0	3900.0	2808.0	0.74	14.8	5.00	1.20	22.8	0.91	0.534	1.44	0.254	Nor
CL	Bedrock							120	15	1.00	21.6	4200.0	4137.6	2952.0	0.72	15.4	0.00	1.00	15.4	0.889	0.534	1.44	0.164	Nor
CL	Bedrock		73.8					120	14	1.00	20.1	4500.0	4281.6	3096.0	0.70	14.1	5.00	1.20	22.0	0.869	0.534	1.44	0.242	Nor
CL	Bedrock							120	15	1.00	21.6	4800.0	4425.6	3240.0	0.69	14.9	0.00	1.00	14.9	0.848	0.531	1.44	0.159	Nor
CL	Bedrock		61.6					120	16	1.00	23.0	5100.0	4569.6	3384.0	0.68	15.7	5.00	1.20	23.8	0.828	0.527	1.44	0.270	Nor
CL	Bedrock		52.2					120	13	1.00	18.7	5400.0	4713.6	3528.0	0.67	12.5	5.00	1.20	20.0	0.808	0.523	1.44	0.216	Nor
ML	Bedrock							120	14	1.00	20.1	5700.0	4857.6	3672.0	0.66	13.3	0.00	1.00	13.3	0.787	0.516	1.44	0.143	Nor
ML	Bedrock		62.8					120	13	1.00	18.7	6000.0	5001.6	3816.0	0.65	12.2	5.00	1.20	19.6	0.767	0.510	1.44	0.210	Nor
ML	Bedrock							120	19	1.00	27.3	6300.0	5145.6	3960.0	0.64	17.5	0.00	1.00	17.5	0.747	0.502	1.44	0.186	Nor
SP	Bedrock							120	19	1.00	27.3	6600.0	5289.6	4104.0	0.63	17.3	0.00	1.00	17.3	0.726	0.493	1.44	0.184	Nor
SM	Bedrock		32.4					120	21	1.00	30.2	6900.0	5433.6	4248.0	0.62	18.8	4.85	1.17	27.0	0.706	0.485	1.44	0.338	Nor
ML	Bedrock		55.9					120	17	1.00	24.4	7200.0	5577.6	4392.0	0.62	15.1	5.00	1.20	23.1	0.686	0.475	1.44	0.258	Nor
SP	Bedrock							120	36	1.00	51.8	7500.0	5721.6	4536.0	0.61	31.5	0.00	1.00	31.5	0.665	0.465	1.44	(N1)60cs >= 30	1.1
SP	Bedrock							120	43	1.00	61.8	7800.0	5865.6	4680.0	0.60	37.1	0.00	1.00	37.1	0.645	0.454	1.44	(N1)60cs >= 30	1.1
SP	Bedrock							120	40	1.00	57.5	8100.0	6009.6	4824.0	0.59	34.1	0.00	1.00	34.1	0.625	0.443	1.44	(N1)60cs >= 30	1.1
SP	Bedrock							120	30	1.00	43.1	8400.0	6153.6	4968.0	0.59	25.3	0.00	1.00	25.3	0.604	0.431	1.44	0.298	Nor
SP	Bedrock							120	47	1.00	67.6	9000.0	6441.6	5256.0	0.57	38.7	0.00	1.00	38.7	0.564	0.408	1.44	(N1)60cs >= 30	1.1
SP	Bedrock							120	24	1.00	34.5	9600.0	6729.6	5544.0	0.56	19.3	0.00	1.00	19.3	0.523	0.383	1.44	0.207	Nor

Liquefaction Susceptibility Analysis: SPT Method (2475-Yr Return)
(Input Data)

Project No.: 22581 Client: Stockdale Capital Partners, LLC

Project Description.: Proposed 12-Story t-Grade Mixed-Use Building

Engineer: RSB

Boring No.	Top Elevation (ft)	Total Depth (ft)	Existing GW Depth (ft)	Design GW Depth (ft)	Recommended Fill Depth (ft)
B1	143	95	34	15	0

Peak Ground Acceleration:	0.977
Earthquake Magnitude:	6.51
Probability of Exceedance in 50 Years:	2%
Borehole Diamter (inches):	8
Delivered Energy Ratio, ERm (%):	75
Energy Ratio Correction Factor, C_E:	1.25
Borehole Diameter Correction Factor, C_B:	1.15
Rod Length Correction Factor, C_R:	1
Sampler Correction with or without Liners, C_S:	1
Minimum Factor of Safety, FS_{liq}:	1

References: *- Youd, T. L., et. al. (2001), Liquefaction Resistane of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, October 2001.*

- Tokimatsu and Seed (1987), Evaluation of Settlements in Sands due to Earthquake Shaking, American Society for Civil Engineers, Journal of Geotechnical Engineering, Vol. 113, No. 8, August, 1987.

- California Geological Survey (2008), Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

- County of Los Angeles, Department of Public Works (2009), Liquefaction/Lateral Spreading, Administrative Manual, Publication No. GS 045-0, May 28, 2009.

(USCS)	(Based on Laboratory Testing, Existing Conditions, and Project Configuration)		FC (%)	PI (%)	LL (%)	Content w_c (%)		γ_t (pcf)	Nm (blows/ft)				(Current) (GW)	(Hist.) (GW)	(2001)				Clean Sand	Coef. r_d				w M
CL	PI>18 & Insensitive		71.9	22.7	44.7			120	8	0.75	8.6	300.0	300.0	300.0	1.70	14.7	5.00	1.20	22.6	0.994	0.631	1.44	0.251	No
CL	Above Hist. GW							120	17	0.80	19.6	600.0	600.0	600.0	1.70	33.2	0.00	1.00	33.2	0.988	0.627	1.44	(N1)60cs >= 30	1.0
CL	Above Hist. GW							120	16	0.80	18.4	900.0	900.0	900.0	1.53	28.2	0.00	1.00	28.2	0.983	0.624	1.44	0.377	No
MH	PI>18 & Insensitive		85.4	27.1	59.0			120	11	0.85	13.4	1200.0	1200.0	1200.0	1.33	17.8	5.00	1.20	26.4	0.977	0.620	1.44	0.323	No
CL	Above Hist. GW		87.2					120	13	0.85	15.9	1500.0	1500.0	1500.0	1.19	18.9	5.00	1.20	27.6	0.971	0.617	1.44	0.357	No
CL	PI>18 & Insensitive		60.8	18.4	37.4			120	8	0.95	10.9	1800.0	1800.0	1800.0	1.08	11.8	5.00	1.20	19.2	0.965	0.613	1.44	0.206	No
CL	PI>18 & Insensitive		89.2	18.6	49.3			120	11	0.95	15.0	2100.0	2100.0	1944.0	1.00	15.1	5.00	1.20	23.1	0.959	0.658	1.44	0.258	No
ML			84.6					120	16	0.95	21.9	2400.0	2400.0	2088.0	0.94	20.5	5.00	1.20	29.6	0.953	0.696	1.44	0.443	0.0
CL	Wc/LL <= 0.8		68.8	14.0	31.0	23.8	0.77	120	14	0.95	19.1	2700.0	2700.0	2232.0	0.89	16.9	5.00	1.20	25.3	0.948	0.728	1.44	0.298	No
ML			72.4					120	24	0.95	32.8	3000.0	3000.0	2376.0	0.84	27.5	5.00	1.20	38.0	0.942	0.755	1.44	(N1)60cs >= 30	1.0
CL	PI< 12		65.4	10.5				120	10	0.95	13.7	3300.0	3300.0	2520.0	0.80	10.9	5.00	1.20	18.1	0.936	0.778	1.44	0.193	0.2
CL	Bedrock							120	8	1.00	11.5	3600.0	3600.0	2664.0	0.77	8.8	0.00	1.00	8.8	0.93	0.798	1.44	0.103	No
CL	Bedrock		89.9					120	14	1.00	20.1	3900.0	3900.0	2808.0	0.74	14.8	5.00	1.20	22.8	0.91	0.803	1.44	0.254	No
CL	Bedrock							120	15	1.00	21.6	4200.0	4137.6	2952.0	0.72	15.4	0.00	1.00	15.4	0.889	0.803	1.44	0.164	No
CL	Bedrock		73.8					120	14	1.00	20.1	4500.0	4281.6	3096.0	0.70	14.1	5.00	1.20	22.0	0.869	0.802	1.44	0.242	No
CL	Bedrock							120	15	1.00	21.6	4800.0	4425.6	3240.0	0.69	14.9	0.00	1.00	14.9	0.848	0.798	1.44	0.159	No
CL	Bedrock		61.6					120	16	1.00	23.0	5100.0	4569.6	3384.0	0.68	15.7	5.00	1.20	23.8	0.828	0.792	1.44	0.270	No
CL	Bedrock		52.2					120	13	1.00	18.7	5400.0	4713.6	3528.0	0.67	12.5	5.00	1.20	20.0	0.808	0.785	1.44	0.216	No
ML	Bedrock							120	14	1.00	20.1	5700.0	4857.6	3672.0	0.66	13.3	0.00	1.00	13.3	0.787	0.776	1.44	0.143	No
ML	Bedrock		62.8					120	13	1.00	18.7	6000.0	5001.6	3816.0	0.65	12.2	5.00	1.20	19.6	0.767	0.766	1.44	0.210	No
ML	Bedrock							120	19	1.00	27.3	6300.0	5145.6	3960.0	0.64	17.5	0.00	1.00	17.5	0.747	0.755	1.44	0.186	No
SP	Bedrock							120	19	1.00	27.3	6600.0	5289.6	4104.0	0.63	17.3	0.00	1.00	17.3	0.726	0.741	1.44	0.184	No
SM	Bedrock		32.4					120	21	1.00	30.2	6900.0	5433.6	4248.0	0.62	18.8	4.85	1.17	27.0	0.706	0.728	1.44	0.338	No
ML	Bedrock		55.9					120	17	1.00	24.4	7200.0	5577.6	4392.0	0.62	15.1	5.00	1.20	23.1	0.686	0.714	1.44	0.258	No
SP	Bedrock							120	36	1.00	51.8	7500.0	5721.6	4536.0	0.61	31.5	0.00	1.00	31.5	0.665	0.698	1.44	(N1)60cs >= 30	1.0
SP	Bedrock							120	43	1.00	61.8	7800.0	5865.6	4680.0	0.60	37.1	0.00	1.00	37.1	0.645	0.683	1.44	(N1)60cs >= 30	1.0
SP	Bedrock							120	40	1.00	57.5	8100.0	6009.6	4824.0	0.59	34.1	0.00	1.00	34.1	0.625	0.666	1.44	(N1)60cs >= 30	1.0
SP	Bedrock							120	30	1.00	43.1	8400.0	6153.6	4968.0	0.59	25.3	0.00	1.00	25.3	0.604	0.649	1.44	0.298	No
SP	Bedrock							120	47	1.00	67.6	9000.0	6441.6	5256.0	0.57	38.7	0.00	1.00	38.7	0.564	0.613	1.44	(N1)60cs >= 30	1.0
SP	Bedrock							120	24	1.00	34.5	9600.0	6729.6	5544.0	0.56	19.3	0.00	1.00	19.3	0.523	0.575	1.44	0.207	No

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ALLOWABLE PILE AXIAL CAPACITY

BG: 22581 ENGINEER: RSB

CLIENT: Stockdale Capital Partners, LLC

CHART # 1

References: - *Bowels, J. E., 1997, Foundation Analysis and Design, Fifth Edition, International Edition, Ch. 16.*
- *Das, B. M., 1990, Principles of Foundation Engineering, 2nd Edition, Ch. 8, pp. 444-460.*
- *NAVFAC, 1986, Foundations & Earth Structures, Design Manual 7.02, Ch. 5, Section 3.*

Soil Properties

Depth		Cu	Phi	Density	Earth	Shear
From	To	(psf)	(deg)	(pcf)	Material	Test No.
10	30	350	28	125	Alluvium	1
30	50	450	26.5	125	Bedrock	2

Depth of Groundwater: 34 feet

Pile Properties

Type: Drilled Pile
Material: Concrete
Shape: Circular
End Bearing: No (Friction Pile)
Diameter: **24 Inches**
Factor of Safety: 2
Overburden Pressure: 0 psf

Pile Embedment Depth (feet)	Skin Friction (psf)	Tip Capacity (kips)	Total Capacity (kips)
10	484.9	0.00	30.5
11	514.6	0.00	35.6
12	543.9	0.00	41.0
13	572.8	0.00	46.8
14	601.4	0.00	52.9
15	629.7	0.00	59.3
16	657.6	0.00	66.1
17	684.5	0.00	73.1
18	714.1	0.00	80.8
19	743.5	0.00	88.8
20	772.8	0.00	97.1
21	802.0	0.00	105.8
22	831.0	0.00	114.9
23	859.8	0.00	124.3
24	888.6	0.00	134.0
25	917.1	0.00	144.1
26	945.5	0.00	154.5
27	973.8	0.00	165.2
28	1001.9	0.00	176.3
29	1029.9	0.00	187.7
30	1057.8	0.00	199.4
31	1070.1	0.00	208.4
32	1095.9	0.00	220.3
33	1121.5	0.00	232.5
34	1146.9	0.00	245.0
35	1159.4	0.00	255.0
36	1170.5	0.00	264.8
37	1181.5	0.00	274.7
38	1192.5	0.00	284.7
39	1203.4	0.00	294.9
40	1214.2	0.00	305.2
41	1223.6	0.00	315.2
42	1233.2	0.00	325.4
43	1242.7	0.00	335.8
44	1252.2	0.00	346.2
45	1261.6	0.00	356.7
46	1270.9	0.00	367.3
47	1280.3	0.00	378.1
48	1289.6	0.00	388.9
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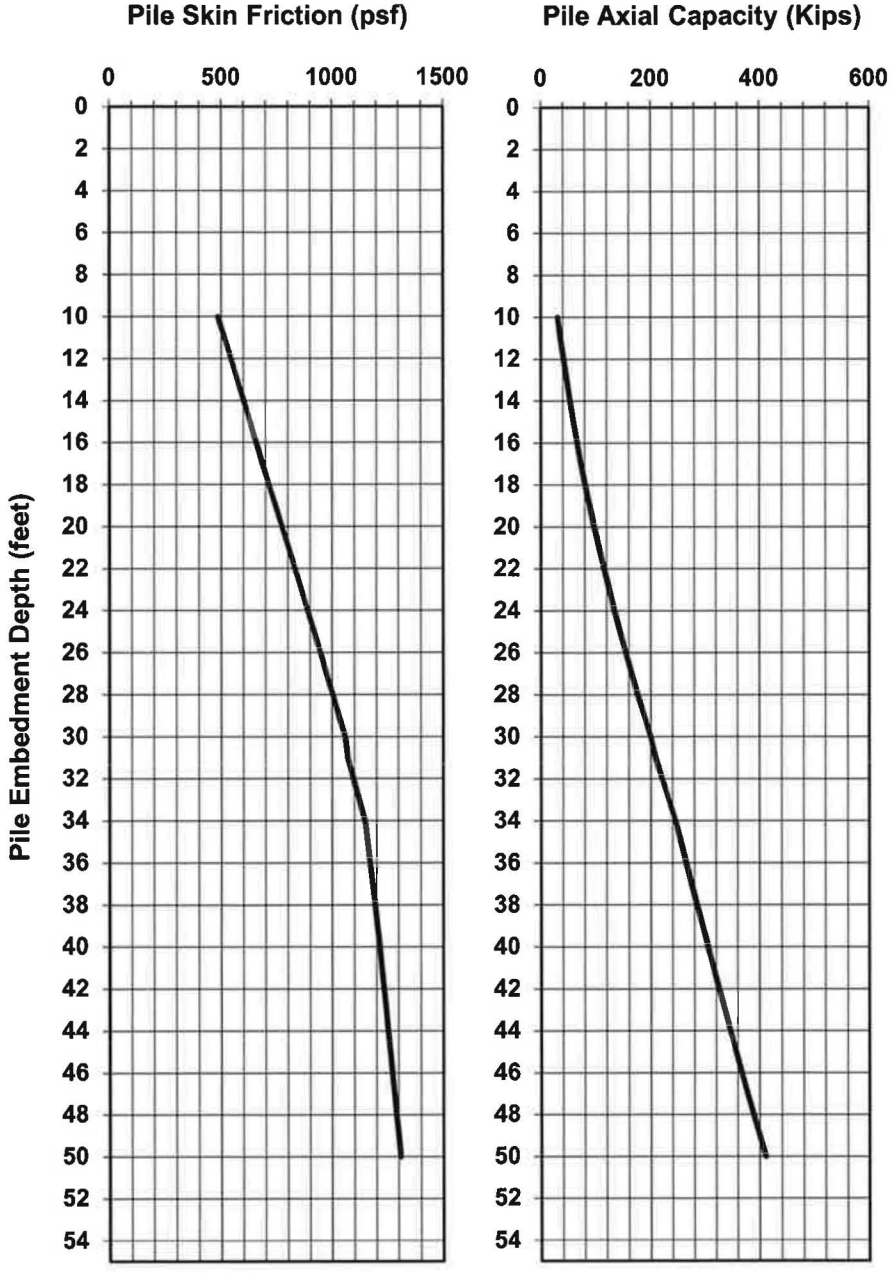

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ALLOWABLE PILE AXIAL CAPACITY

BG: 22581 ENGINEER: RSB

CLIENT: Stockdale Capital Partners, LLC

CHART # 2

References: - *Bowels, J. E., 1997, Foundation Analysis and Design, Fifth Edition, International Edition, Ch. 16.*
- *Das, B. M., 1990, Principles of Foundation Engineering, 2nd Edition, Ch. 8, pp. 444-460.*
- *NAVFAC, 1986, Foundations & Earth Structures, Design Manual 7.02, Ch. 5, Section 3.*

Soil Properties

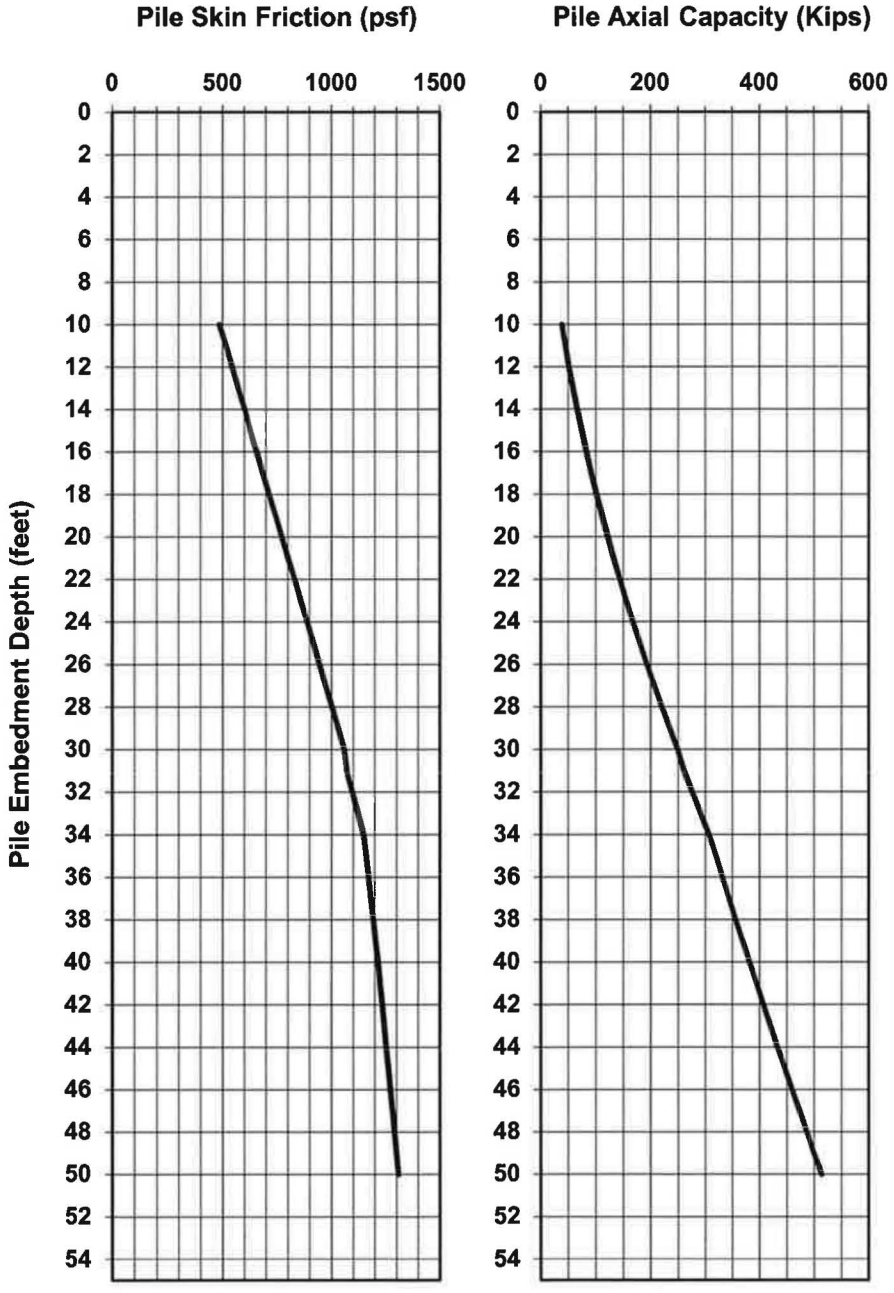
Depth From	Depth To	Cu (psf)	Phi (deg)	Density (pcf)	Earth Material	Shear Test No.
10	30	350	28	125	Alluvium	1
30	50	450	26.5	125	Bedrock	2

Depth of Groundwater: 34 feet

Pile Properties

Type: Drilled Pile
Material: Concrete
Shape: Circular
End Bearing: No (Friction Pile)
Diameter: **30 Inches**
Factor of Safety: 2
Overburden Pressure: 0 psf

Pile Embedment Depth (feet)	Skin Friction (psf)	Tip Capacity (kips)	Total Capacity (kips)
10	484.9	0.00	38.1
11	514.6	0.00	44.5
12	543.9	0.00	51.3
13	572.8	0.00	58.5
14	601.4	0.00	66.1
15	629.7	0.00	74.2
16	657.6	0.00	82.6
17	684.5	0.00	91.4
18	714.1	0.00	101.0
19	743.5	0.00	111.0
20	772.8	0.00	121.4
21	802.0	0.00	132.3
22	831.0	0.00	143.6
23	859.8	0.00	155.3
24	888.6	0.00	167.5
25	917.1	0.00	180.1
26	945.5	0.00	193.1
27	973.8	0.00	206.5
28	1001.9	0.00	220.3
29	1029.9	0.00	234.6
30	1057.8	0.00	249.2
31	1070.1	0.00	260.5
32	1095.9	0.00	275.4
33	1121.5	0.00	290.7
34	1146.9	0.00	306.3
35	1159.4	0.00	318.7
36	1170.5	0.00	331.0
37	1181.5	0.00	343.4
38	1192.5	0.00	355.9
39	1203.4	0.00	368.6
40	1214.2	0.00	381.4
41	1223.6	0.00	394.0
42	1233.2	0.00	406.8
43	1242.7	0.00	419.7
44	1252.2	0.00	432.7
45	1261.6	0.00	445.9
46	1270.9	0.00	459.2
47	1280.3	0.00	472.6
48	1289.6	0.00	486.2
49	1298.8	0.00	499.8
50	1308.0	0.00	513.7



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ALLOWABLE PILE AXIAL CAPACITY

BG: 22581 ENGINEER: RSB

CLIENT: Stockdale Capital Partners, LLC

CHART # 3

References: - *Bowels, J. E., 1997, Foundation Analysis and Design, Fifth Edition, International Edition, Ch. 16.*
- *Das, B. M., 1990, Principles of Foundation Engineering, 2nd Edition, Ch. 8, pp. 444-460.*
- *NAVFAC, 1986, Foundations & Earth Structures, Design Manual 7.02, Ch. 5, Section 3.*

Soil Properties

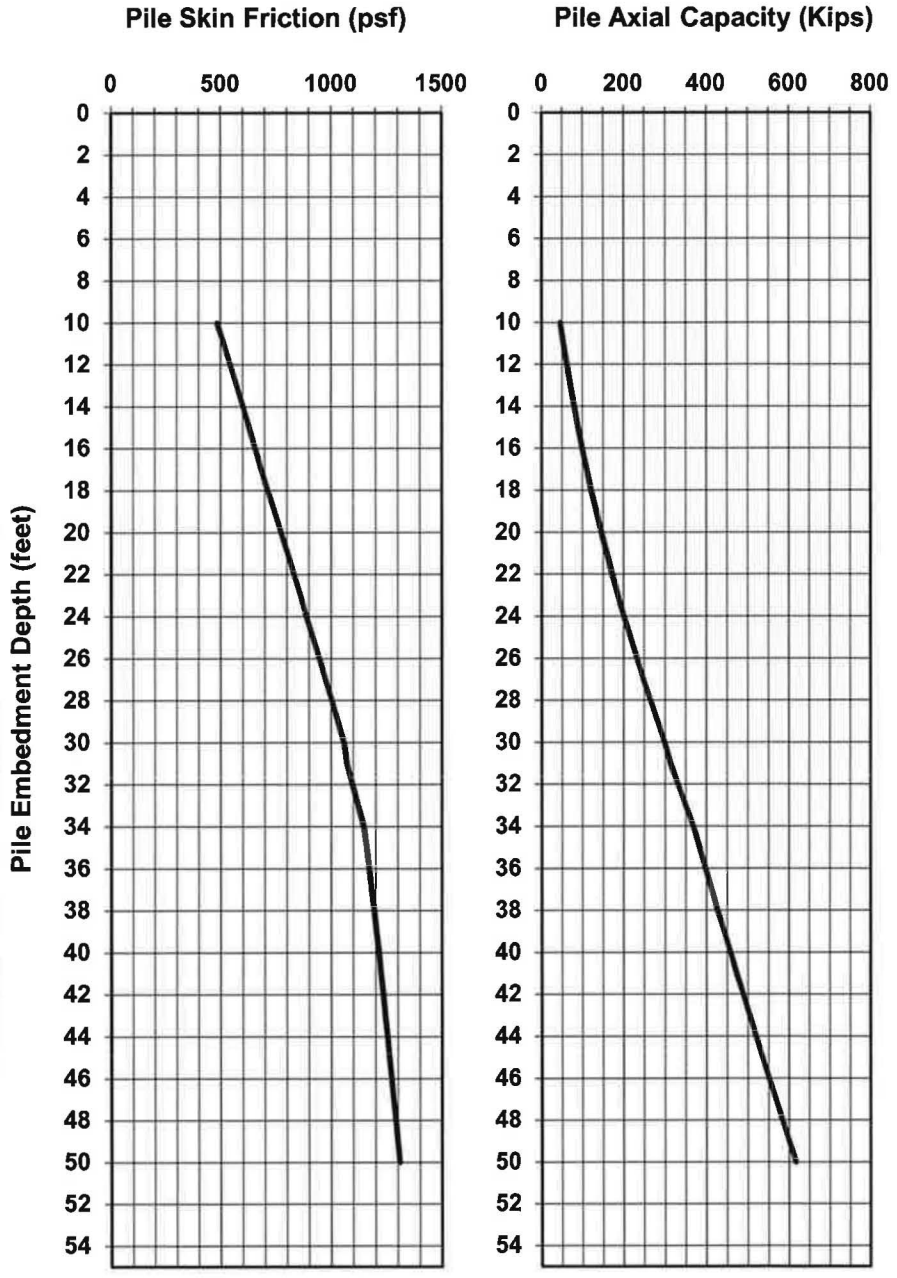
Depth From	Depth To	Cu (psf)	Phi (deg)	Density (pcf)	Earth Material	Shear Test No.
10	30	350	28	125	Alluvium	1
30	50	450	26.5	125	Bedrock	2

Depth of Groundwater: 34 feet

Pile Properties

Type: Drilled Pile
Material: Concrete
Shape: Circular
End Bearing: No (Friction Pile)
Diameter: **36 Inches**
Factor of Safety: 2
Overburden Pressure: 0 psf

Pile Embedment Depth (feet)	Skin Friction (psf)	Tip Capacity (kips)	Total Capacity (kips)
10	484.9	0.00	45.7
11	514.6	0.00	53.3
12	543.9	0.00	61.5
13	572.8	0.00	70.2
14	601.4	0.00	79.4
15	629.7	0.00	89.0
16	657.6	0.00	99.2
17	684.5	0.00	109.7
18	714.1	0.00	121.1
19	743.5	0.00	133.1
20	772.8	0.00	145.7
21	802.0	0.00	158.7
22	831.0	0.00	172.3
23	859.8	0.00	186.4
24	888.6	0.00	201.0
25	917.1	0.00	216.1
26	945.5	0.00	231.7
27	973.8	0.00	247.8
28	1001.9	0.00	264.4
29	1029.9	0.00	281.5
30	1057.8	0.00	299.1
31	1070.1	0.00	312.7
32	1095.9	0.00	330.5
33	1121.5	0.00	348.8
34	1146.9	0.00	367.5
35	1159.4	0.00	382.5
36	1170.5	0.00	397.1
37	1181.5	0.00	412.0
38	1192.5	0.00	427.1
39	1203.4	0.00	442.3
40	1214.2	0.00	457.7
41	1223.6	0.00	472.8
42	1233.2	0.00	488.2
43	1242.7	0.00	503.6
44	1252.2	0.00	519.3
45	1261.6	0.00	535.0
46	1270.9	0.00	551.0
47	1280.3	0.00	567.1
48	1289.6	0.00	583.4
49	1298.8	0.00	599.8
50	1308.0	0.00	616.4



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ALLOWABLE PILE AXIAL CAPACITY

BG: 22581 ENGINEER: RSB
CLIENT: Stockdale Capital Partners, LLC

CHART # 4

References: - *Bowels, J. E., 1997, Foundation Analysis and Design, Fifth Edition, International Edition, Ch. 16.*
- *Das, B. M., 1990, Principles of Foundation Engineering, 2nd Edition, Ch. 8, pp. 444-460.*
- *NAVFAC, 1986, Foundations & Earth Structures, Design Manual 7.02, Ch. 5, Section 3.*

Soil Properties

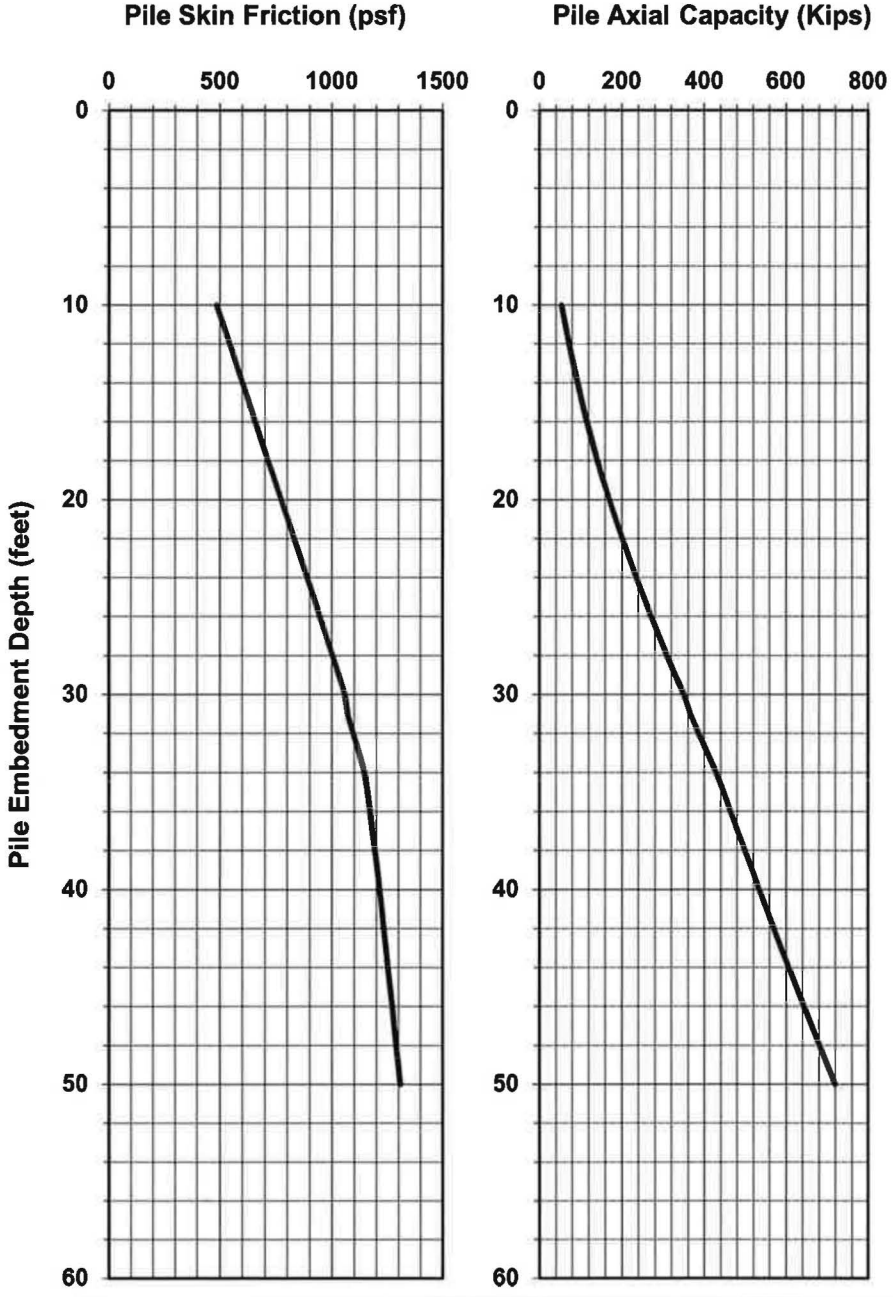
Depth From	Depth To	Cu (psf)	Phi (deg)	Density (pcf)	Earth Material	Shear Test No.
10	30	350	28	125	Alluvium	1
30	50	450	26.5	125	Bedrock	2

Depth of Groundwater: 34 feet

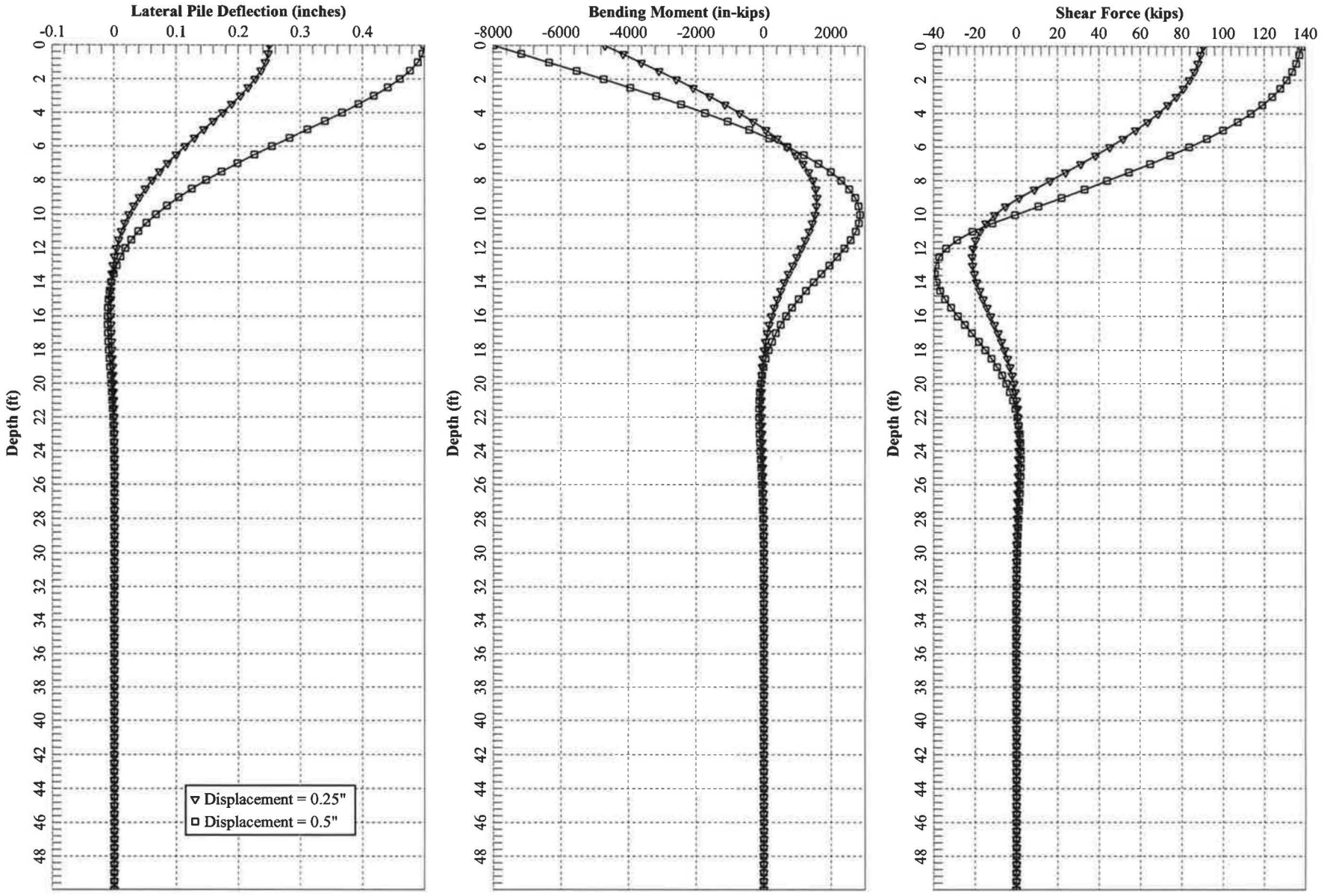
Pile Properties

Type: Drilled Pile
Material: Concrete
Shape: Circular
End Bearing: No (Friction Pile)
Diameter: **42 Inches**
Factor of Safety: 2
Overburden Pressure: 0 psf

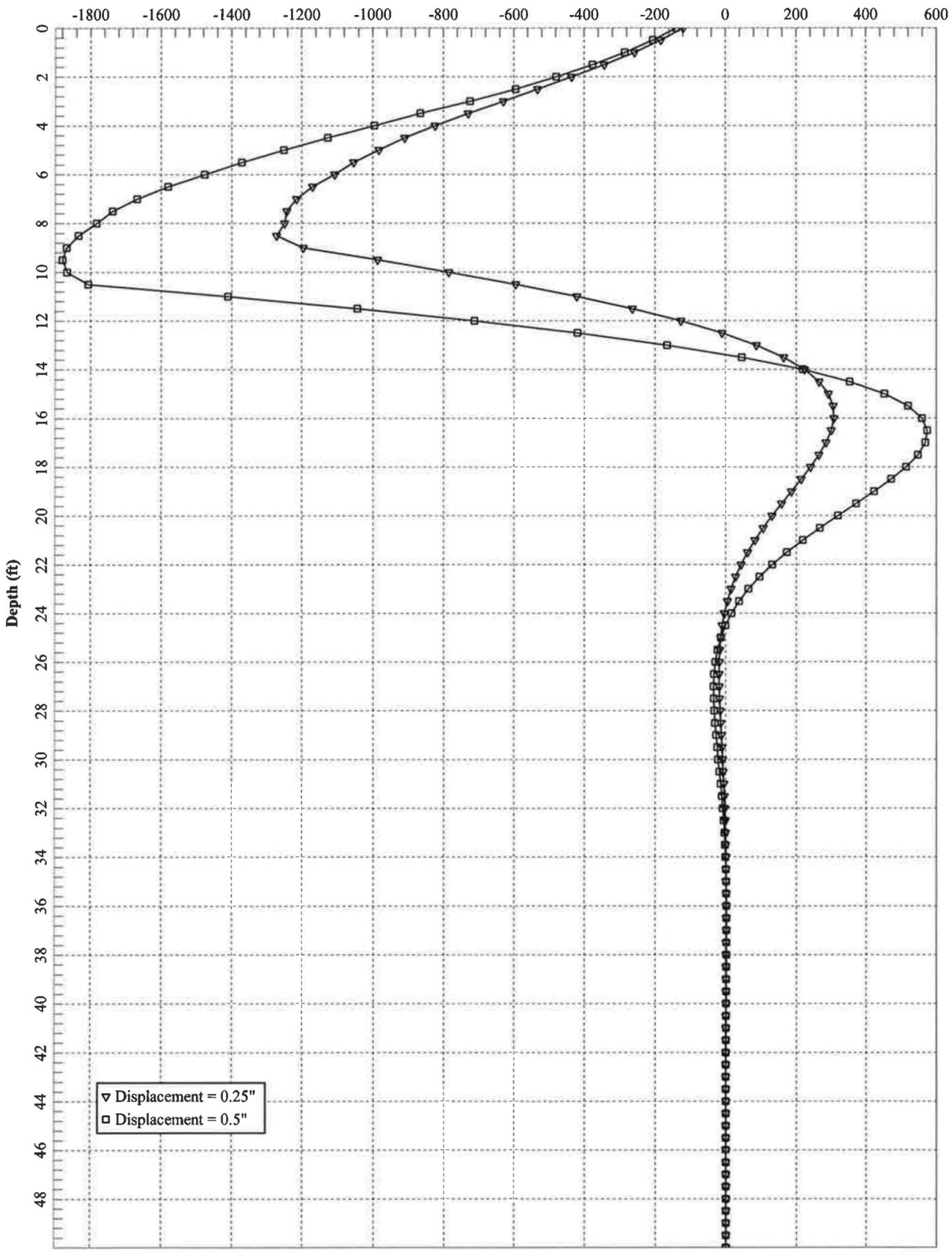
Pile Embedment Depth (feet)	Skin Friction (psf)	Tip Capacity (kips)	Total Capacity (kips)
10	484.9	0.00	53.3
11	514.6	0.00	62.2
12	543.9	0.00	71.8
13	572.8	0.00	81.9
14	601.4	0.00	92.6
15	629.7	0.00	103.9
16	657.6	0.00	115.7
17	684.5	0.00	128.0
18	714.1	0.00	141.3
19	743.5	0.00	155.3
20	772.8	0.00	170.0
21	802.0	0.00	185.2
22	831.0	0.00	201.0
23	859.8	0.00	217.5
24	888.6	0.00	234.5
25	917.1	0.00	252.1
26	945.5	0.00	270.3
27	973.8	0.00	289.1
28	1001.9	0.00	308.5
29	1029.9	0.00	328.4
30	1057.8	0.00	348.9
31	1070.1	0.00	364.8
32	1095.9	0.00	385.6
33	1121.5	0.00	406.9
34	1146.9	0.00	428.8
35	1159.4	0.00	446.2
36	1170.5	0.00	463.3
37	1181.5	0.00	480.7
38	1192.5	0.00	498.3
39	1203.4	0.00	516.0
40	1214.2	0.00	534.0
41	1223.6	0.00	551.6
42	1233.2	0.00	569.5
43	1242.7	0.00	587.6
44	1252.2	0.00	605.8
45	1261.6	0.00	624.2
46	1270.9	0.00	642.8
47	1280.3	0.00	661.6
48	1289.6	0.00	680.6
49	1298.8	0.00	699.8
50	1308.0	0.00	719.1



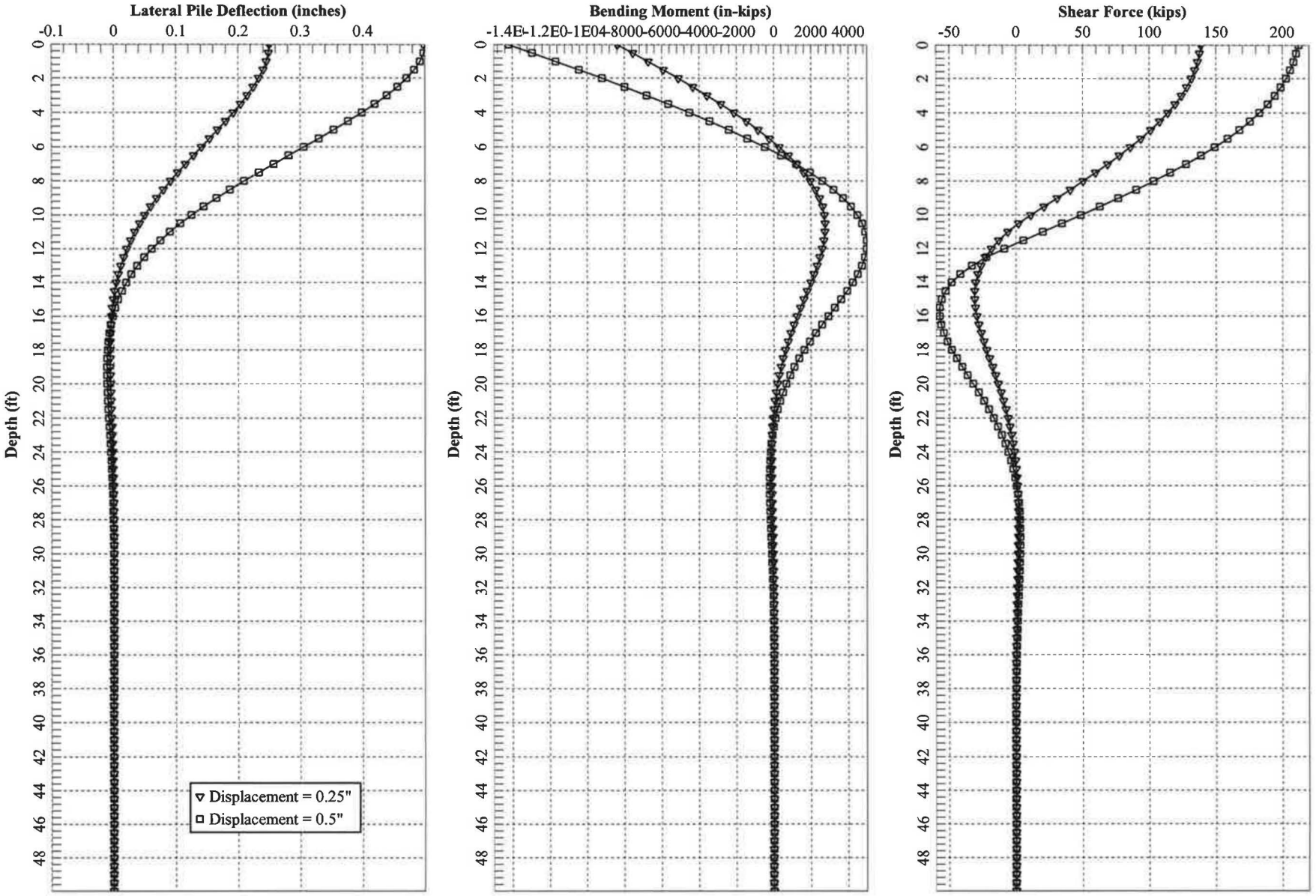
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 24" Dia. - Fixed Head - Single Pile - Fig. 1a



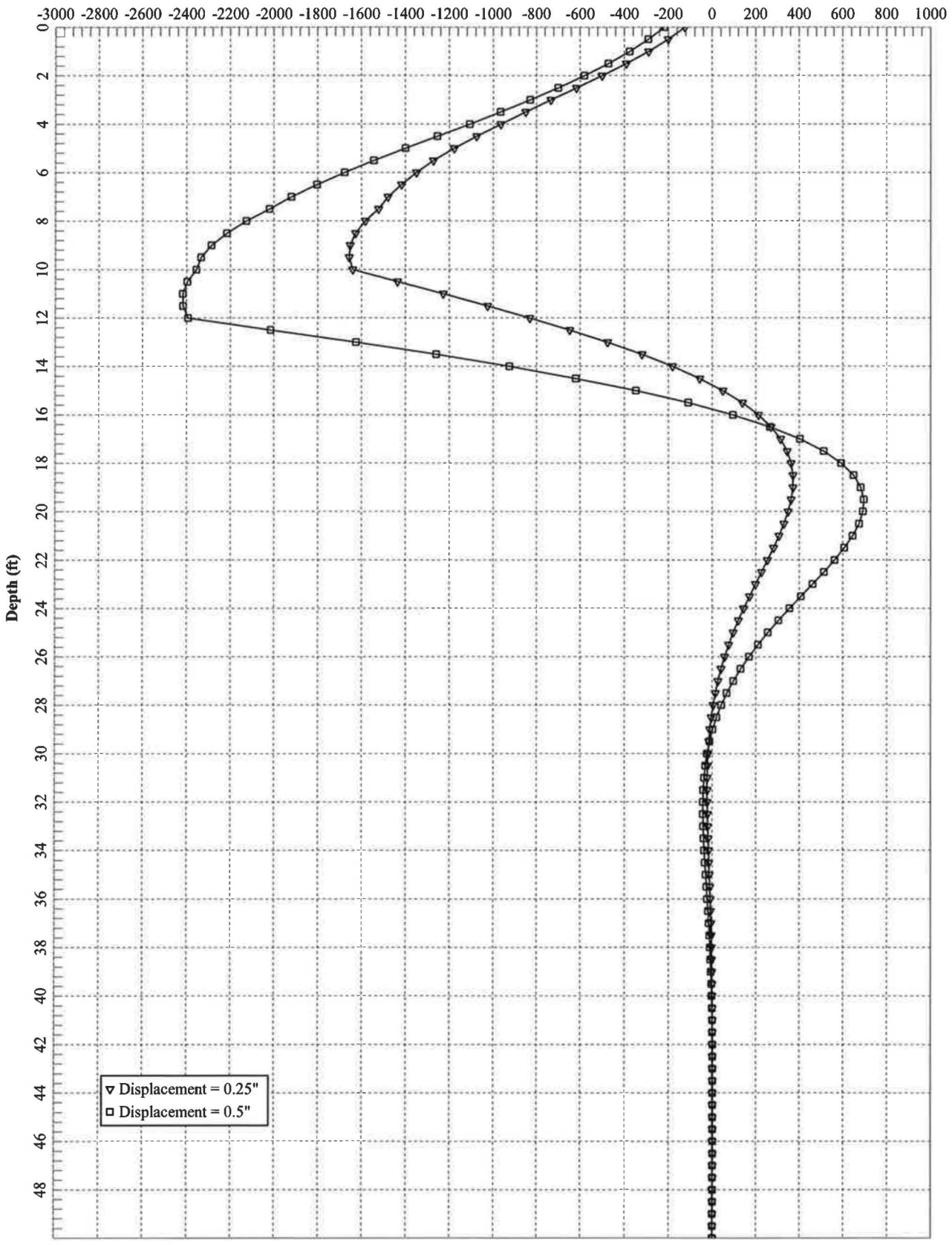
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 24" Dia. - Fixed Head - Single Pile - Fig. 1b
Mobilized Soil Reaction (lb/in)



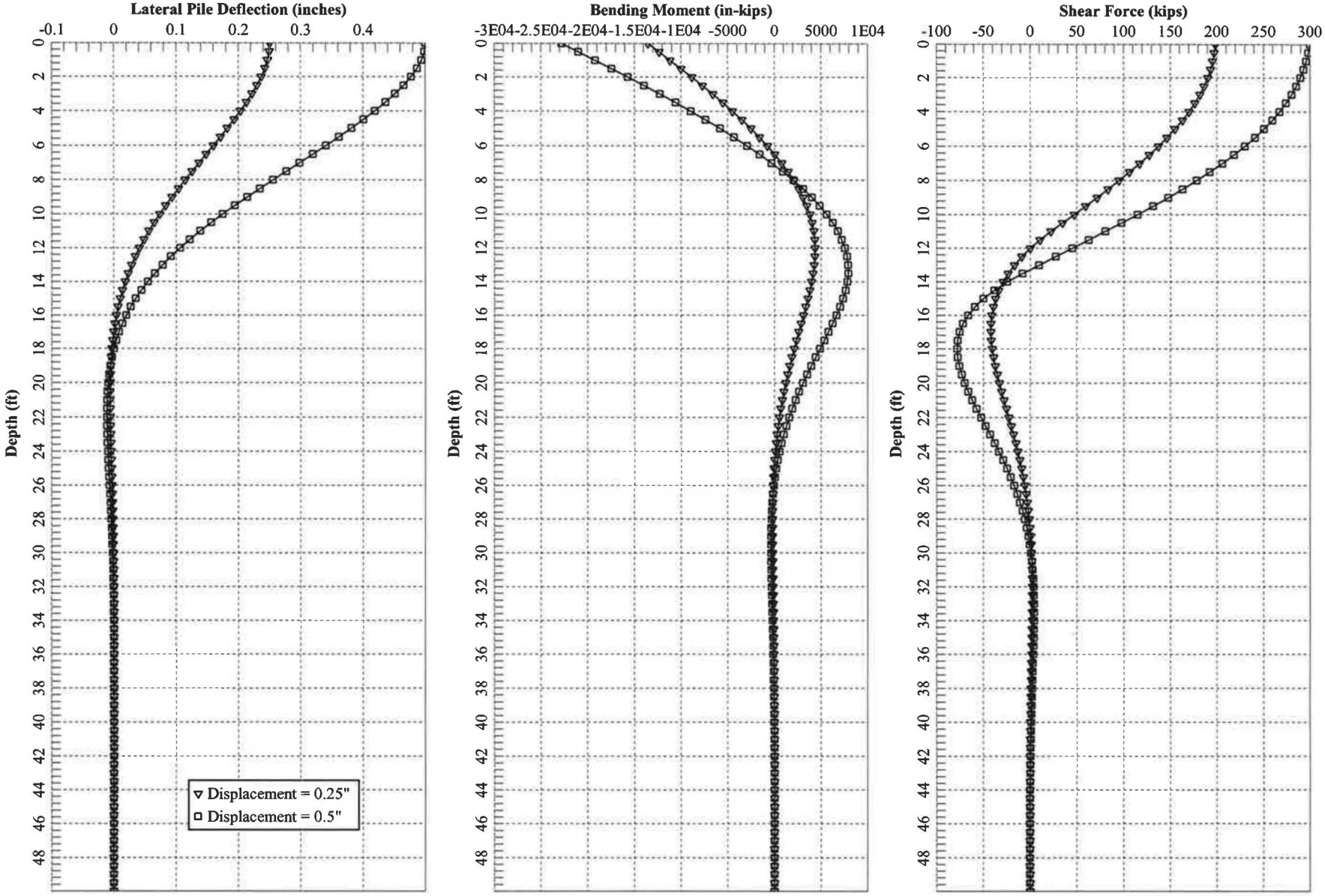
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 30" Dia. - Fixed Head - Single Pile - Fig. 2a



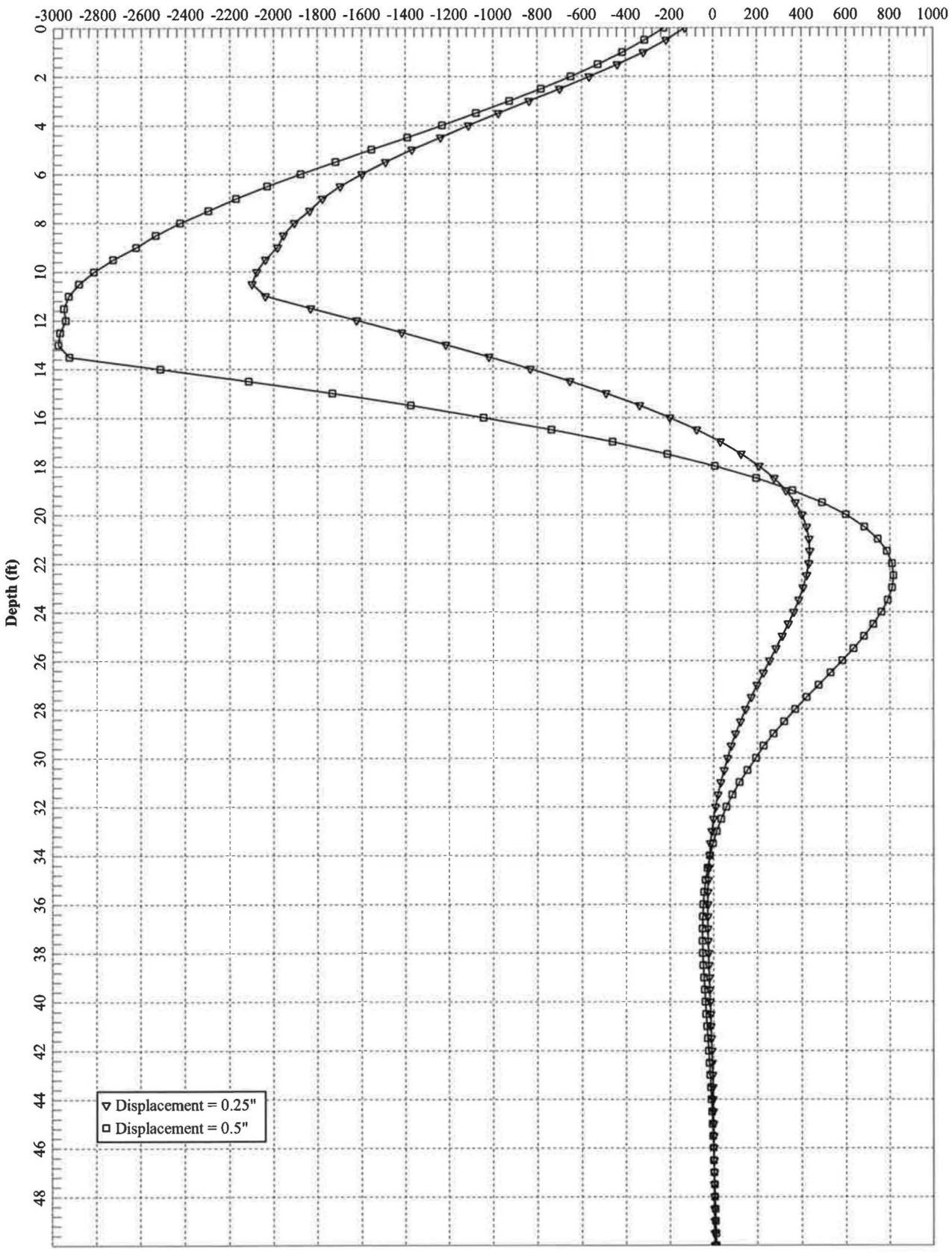
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 30" Dia. - Fixed Head - Single Pile - Fig. 2b
Mobilized Soil Reaction (lb/in)



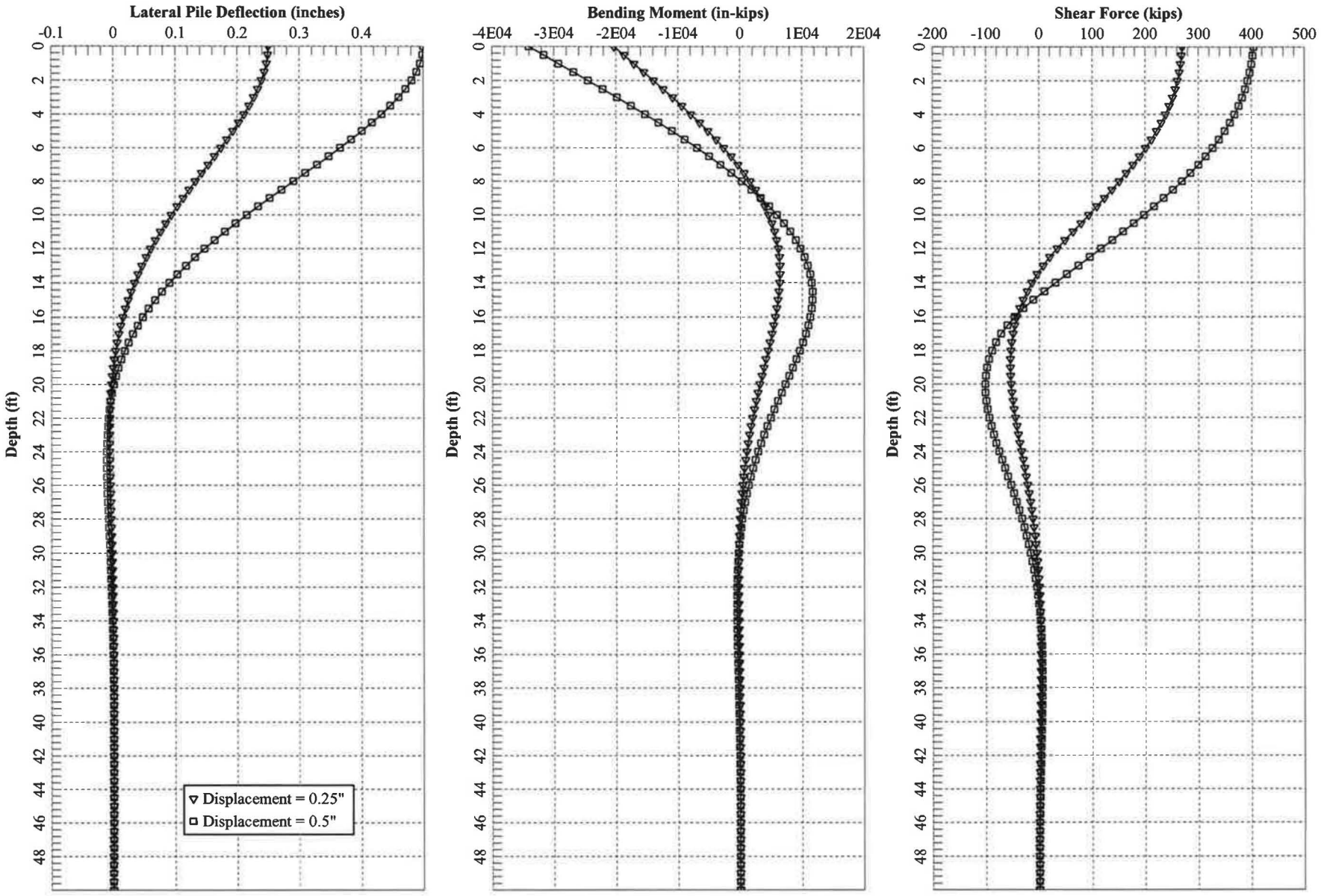
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 36" Dia. - Fixed Head - Single Pile - Fig. 3a



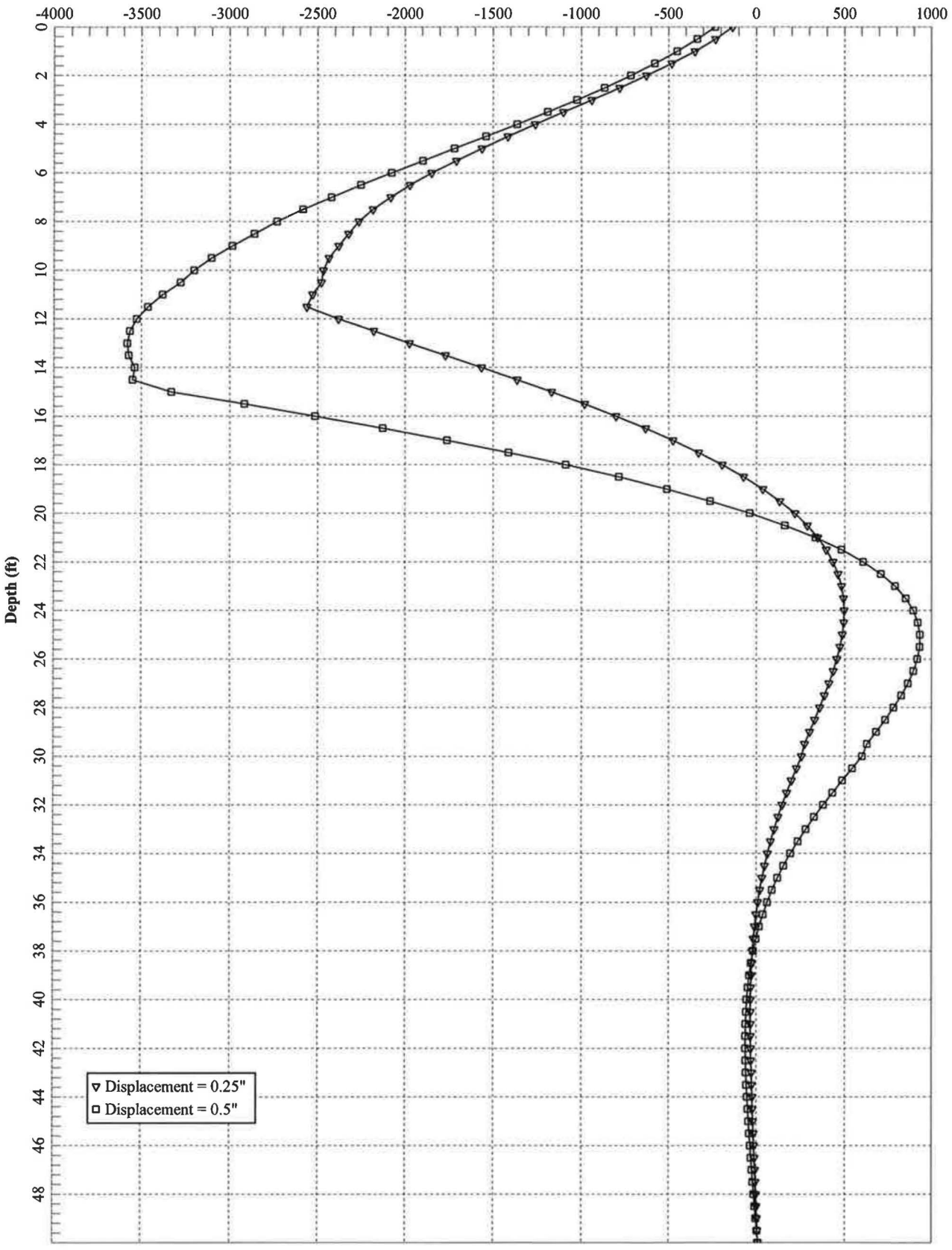
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 36" Dia. - Fixed Head - Single Pile - Fig. 3b
Mobilized Soil Reaction (lb/in)



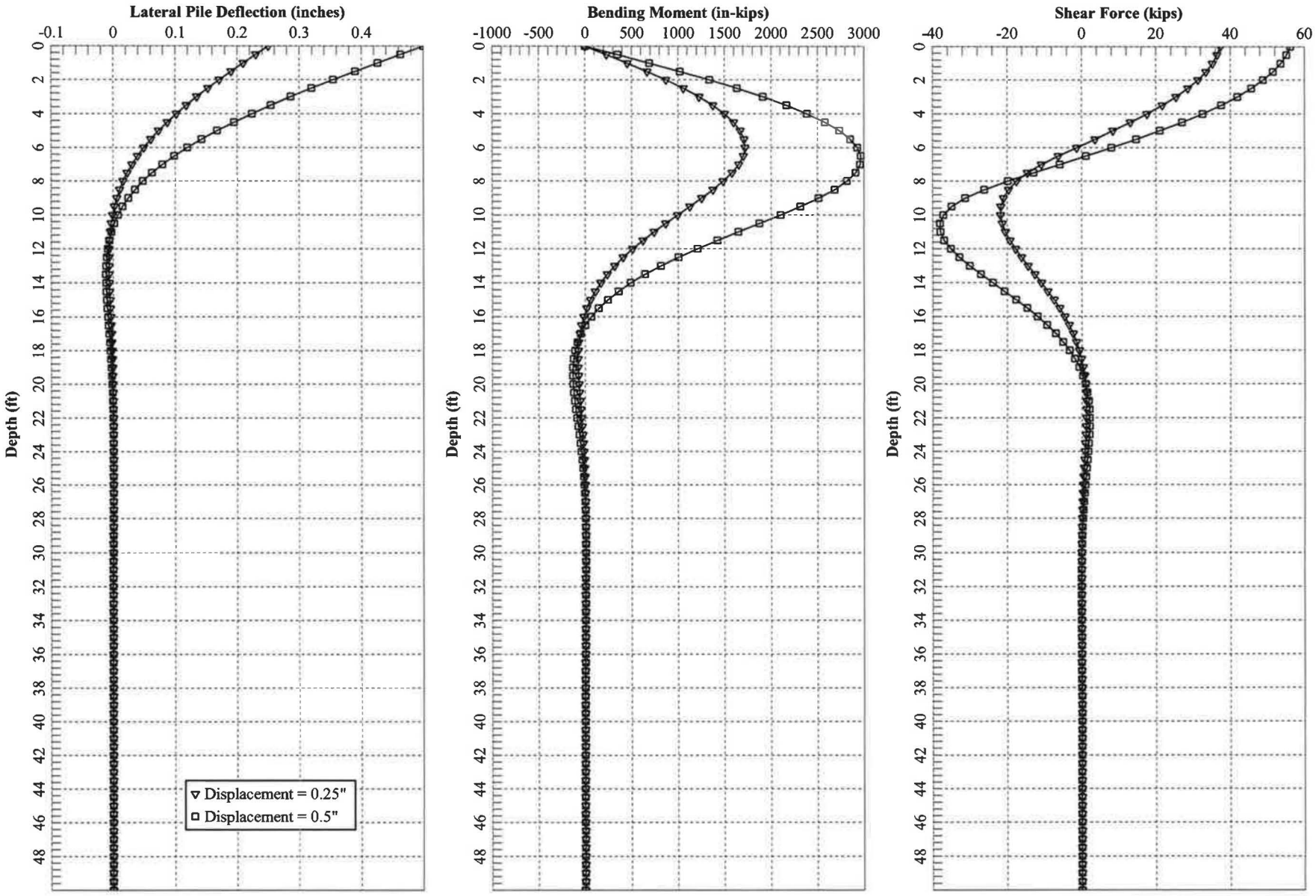
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 42" Dia. - Fixed Head - Single Pile - Fig. 4a



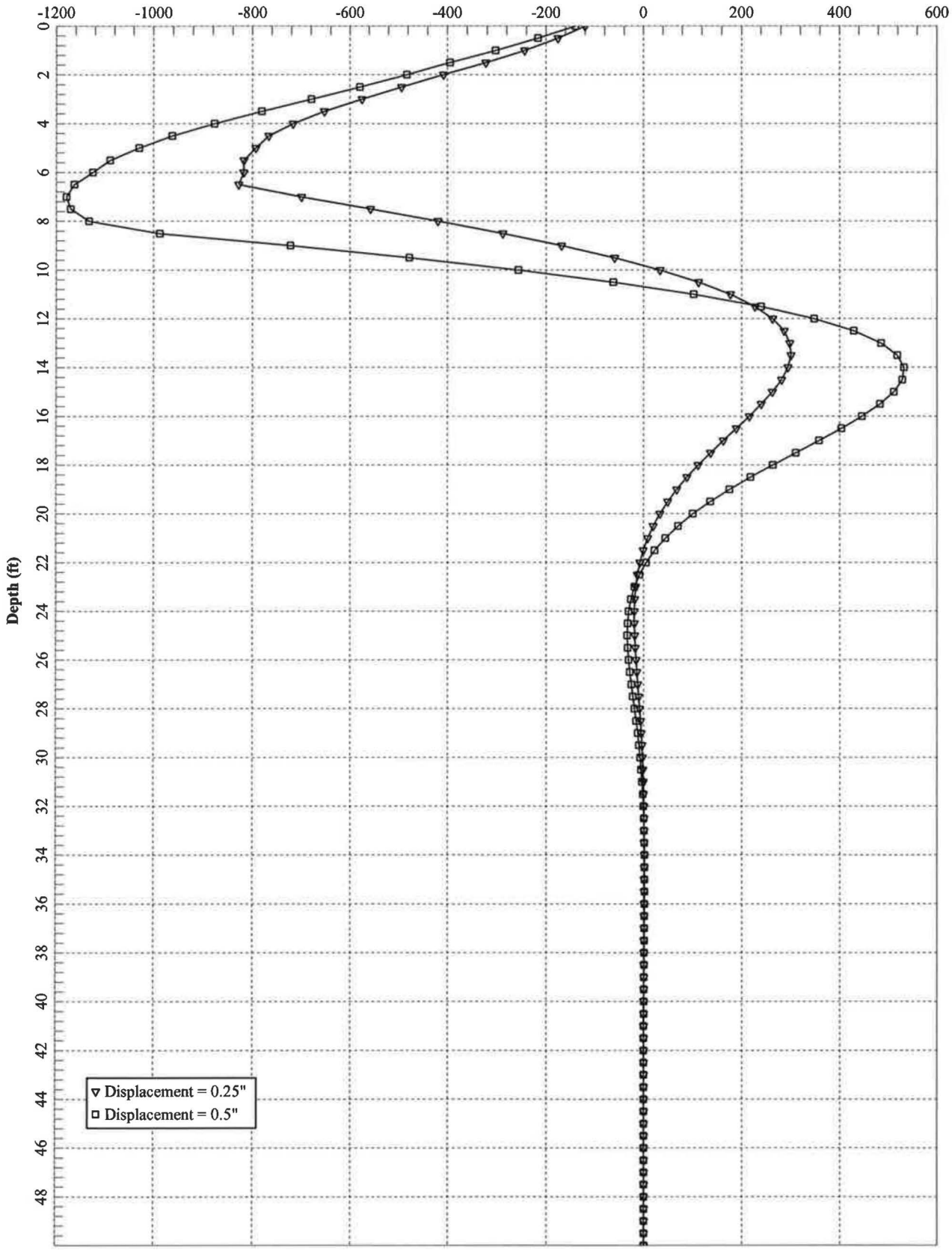
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 42" Dia. - Fixed Head - Single Pile - Fig. 4b
Mobilized Soil Reaction (lb/in)



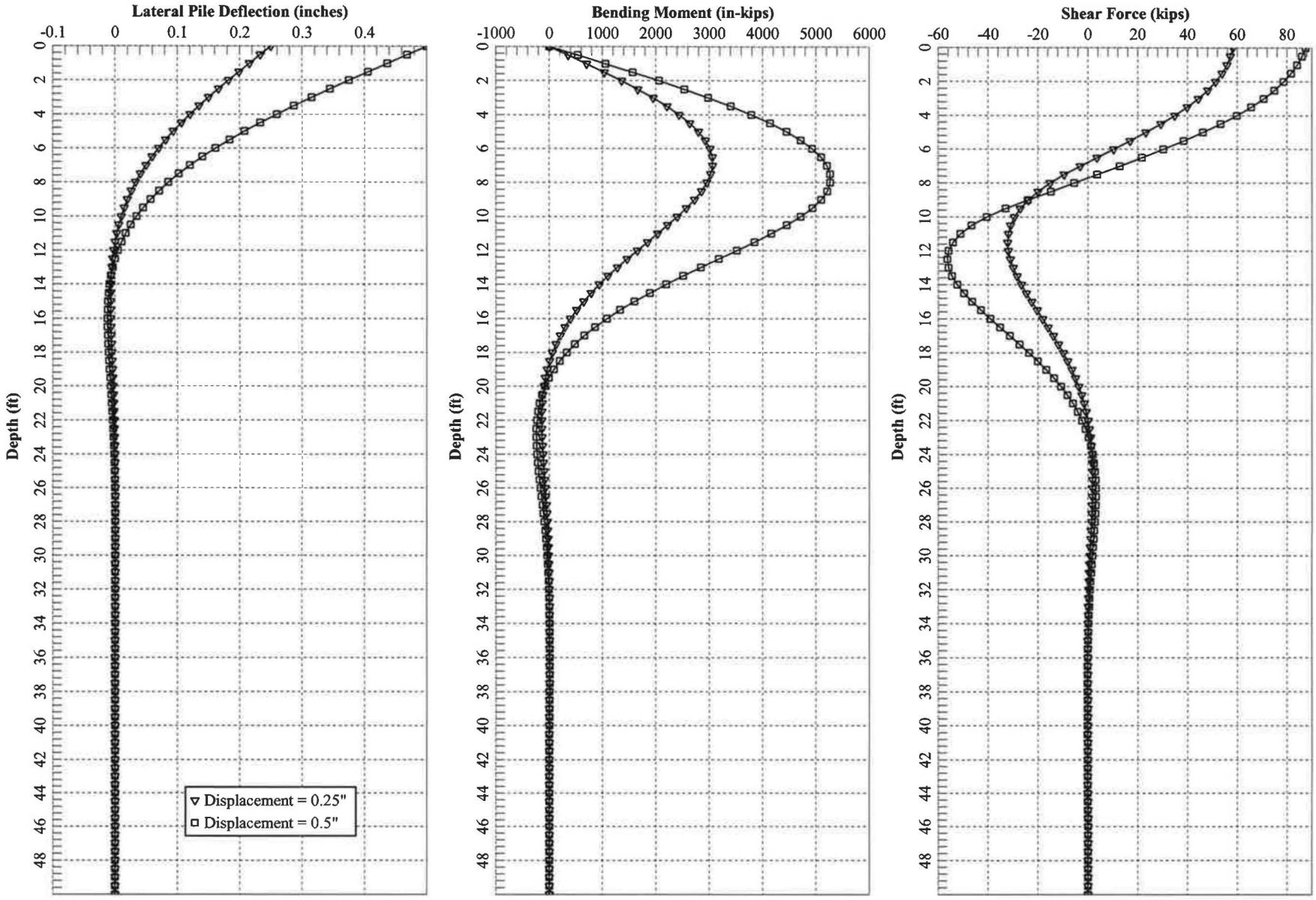
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 24" Dia. - Free Head - Single Pile - Fig. 5a



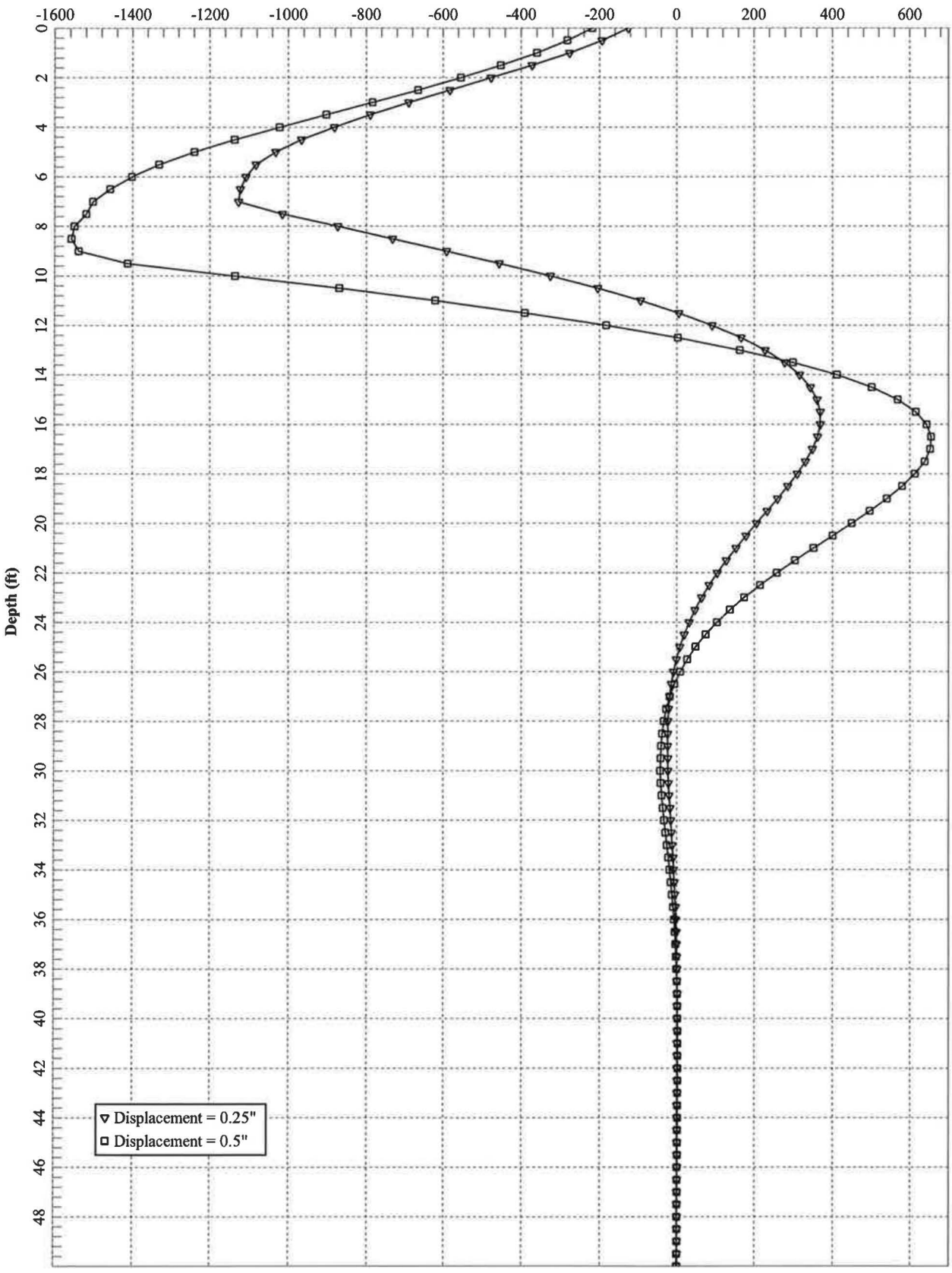
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 24" Dia. - Free Head - Single Pile - Fig. 5b
Mobilized Soil Reaction (lb/in)



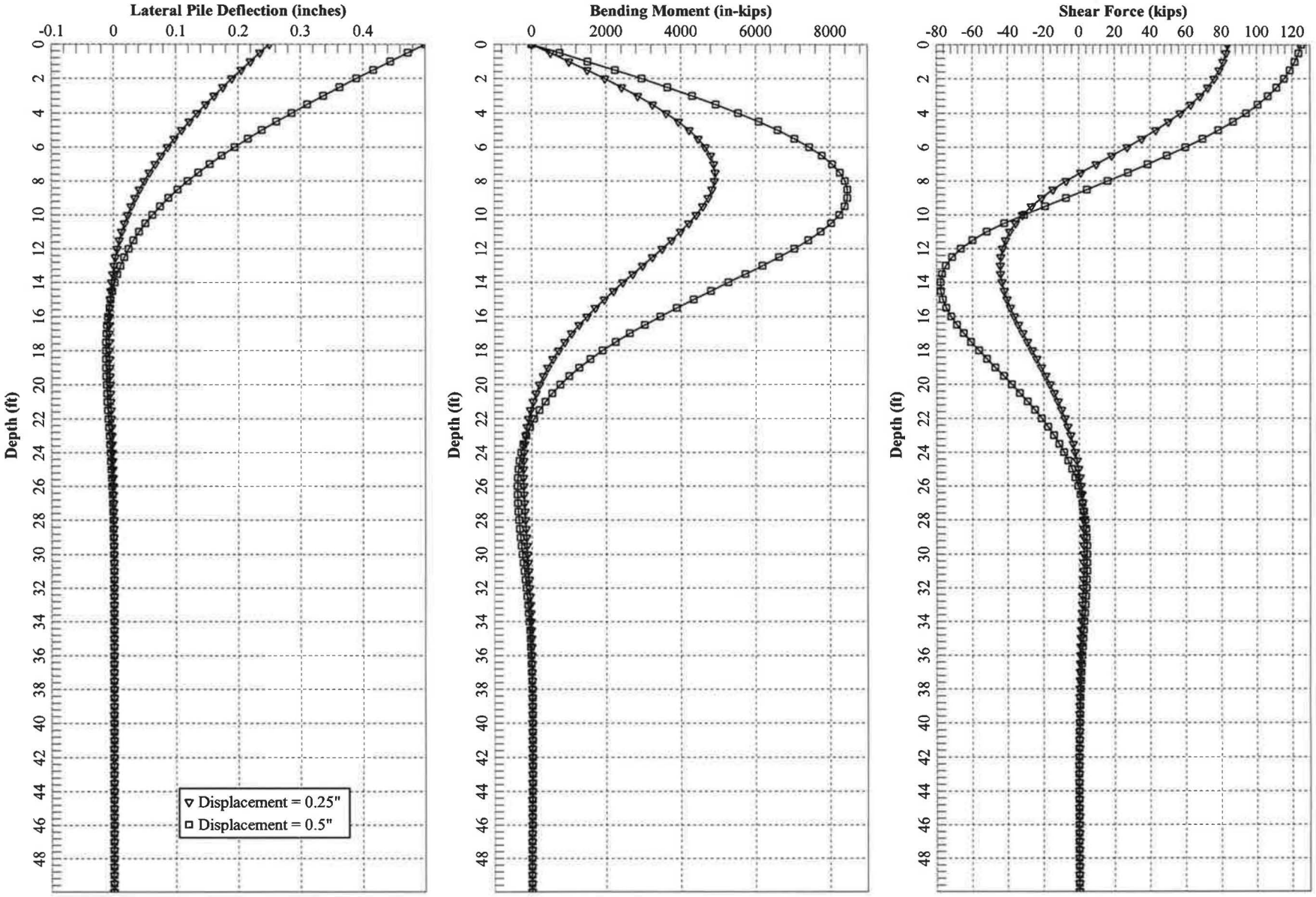
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 30" Dia. - Free Head - Single Pile - Fig. 6a



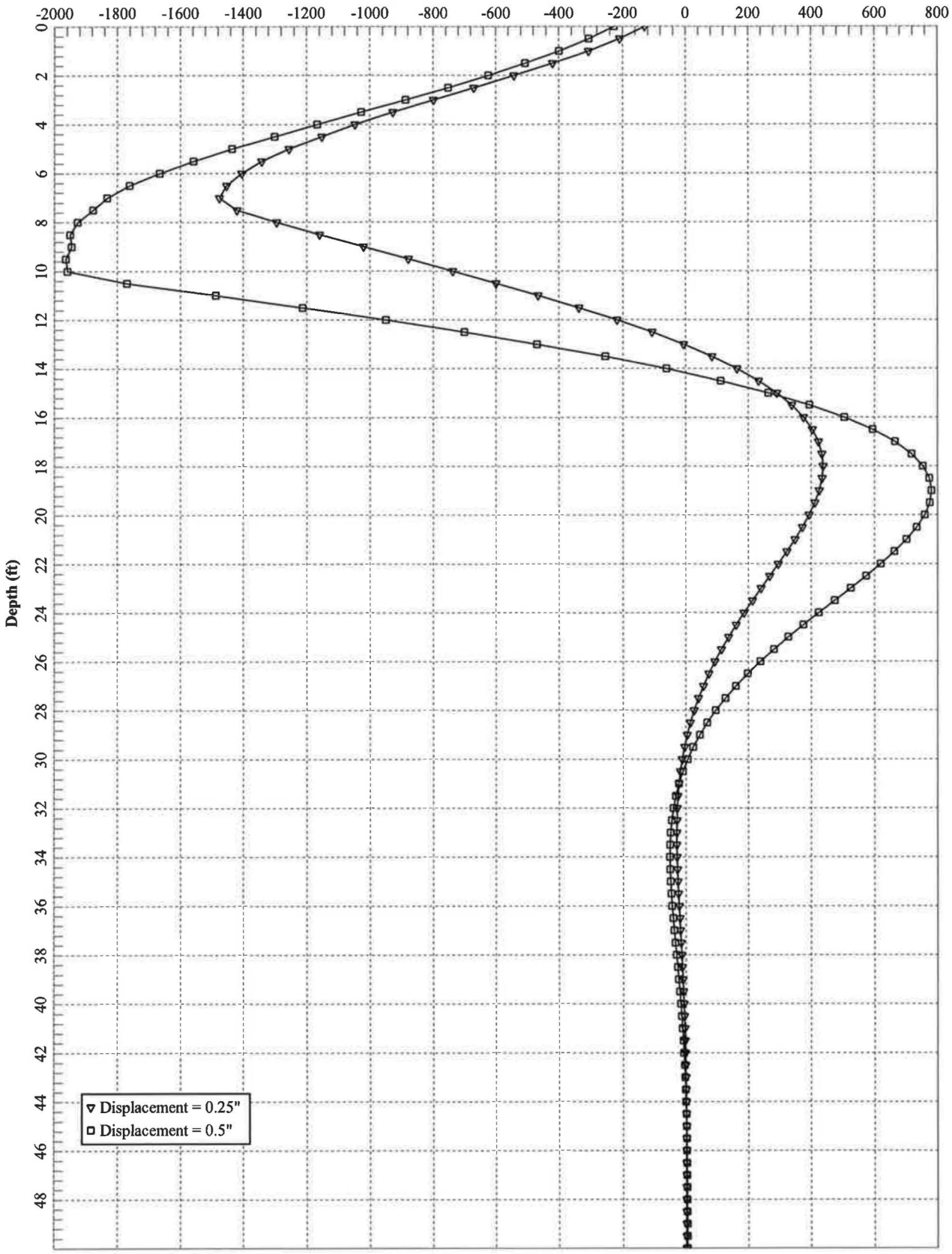
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 30" Dia. - Free Head - Single Pile - Fig. 6b
Mobilized Soil Reaction (lb/in)



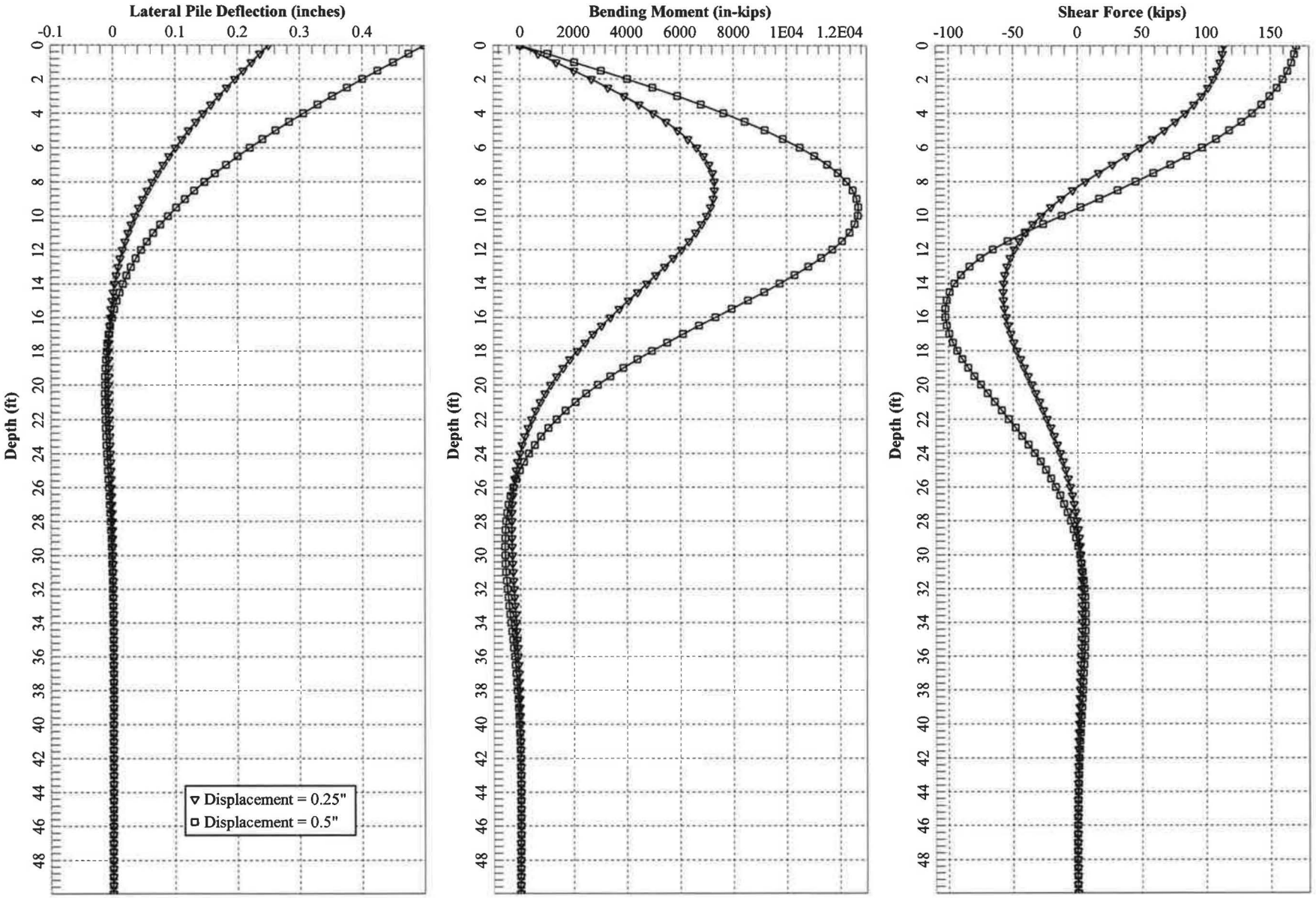
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 36" Dia. - Free Head - Single Pile - Fig. 7a



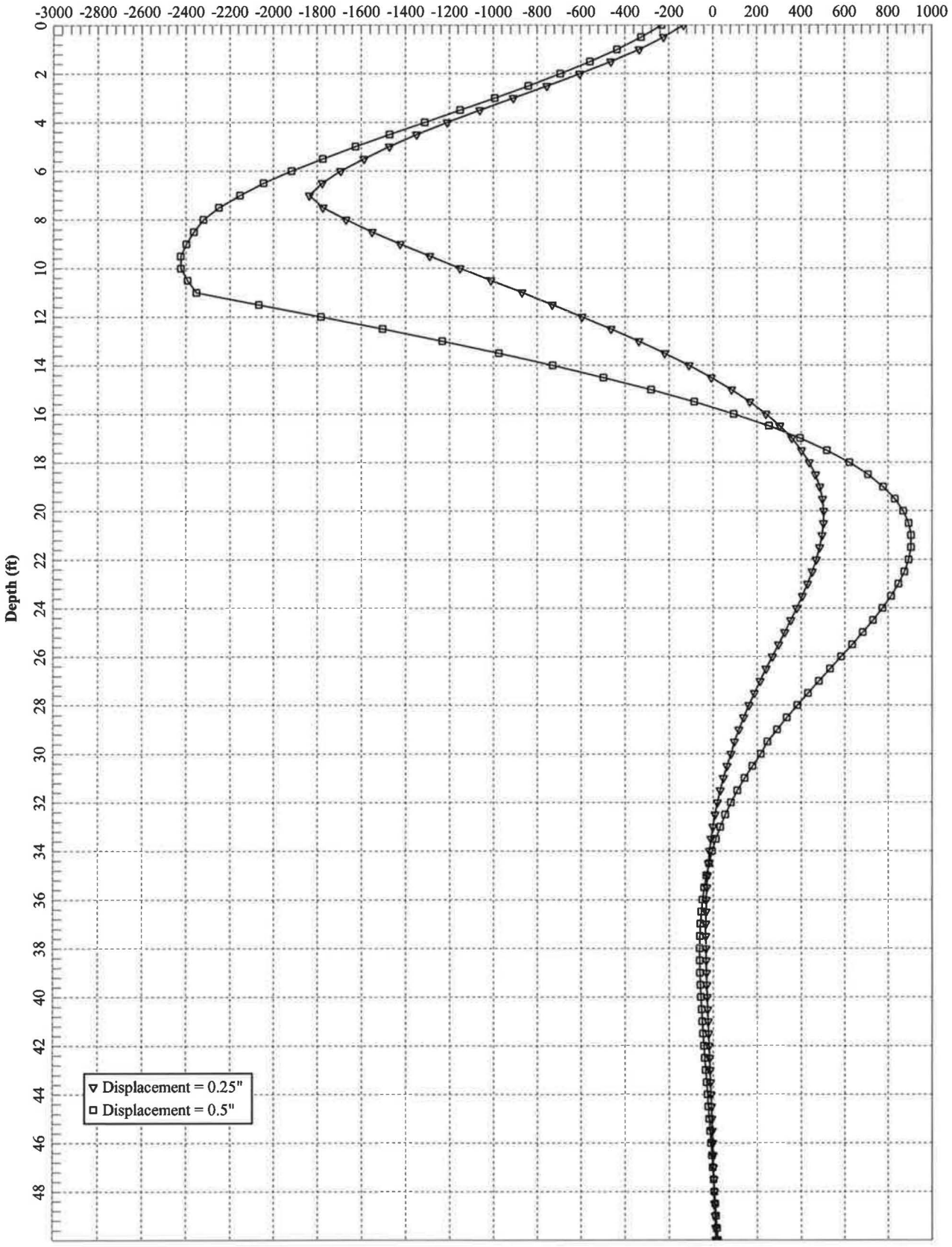
BG 22581 - Stockdale Capital Partners, LLC - LPILE - 36" Dia. - Free Head - Single Pile - Fig. 7b
Mobilized Soil Reaction (lb/in)



BG 22581 - Stockdale Capital Partners, LLC - LPILE - 42" Dia. - Free Head - Single Pile - Fig. 8a



BG 22581 - Stockdale Capital Partners, LLC - LPILE - 42" Dia. - Free Head - Single Pile - Fig. 8b
Mobilized Soil Reaction (lb/in)



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RETAINING WALL

BG: **22581** ENGINEER: **RSB**
CLIENT: **Stockdale Capital Partners, LLC**
CALCULATION SHEET # **5**

CALCULATE THE DESIGN ACTIVE EQUIVALENT FLUID PRESSURE (EFP) FOR THE PROPOSED RETAINING WALL. ASSUME BACKFILL IS SATURATED AND THERE IS NO HYDROSTATIC PRESSURE THE RETAINED HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium	WALL HEIGHT	12 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	350 psf	SURCHARGE:	300 pounds
PHI ANGLE:	28 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	125 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	233.3 psf	FINAL TENSION CRACK:	20 feet

PHID = ATAN(TAN(PHI)/FS) = 19.5 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	53 degrees
AREA OF TRIAL FAILURE WEDGE	51.5 square feet
TOTAL EXTERNAL SURCHARGE	1800.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	8235.9 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1020 trials
LENGTH OF FAILURE PLANE	11.6 feet
DEPTH OF TENSION CRACK	2.7 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	7.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**2380.5 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**33.1 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**43.0 pcf**

CONCLUSION:

THE CALCULATION INDICATES THAT CANTILEVER RETAINING WALLS UP TO 12 FEET HIGH, WITH LEVEL BACKSLOPE AND VEHICULAR SURCHARGE, MAY BE DESIGNED FOR AN ACTIVE EQUIVALENT FLUID PRESSURE OF 43 POUNDS-PER-CUBIC-FOOT.

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RETAINING WALL

BG: **22581** ENGINEER: **RSB**
CLIENT: **Stockdale Capital Partners, LLC**

CALCULATION SHEET # **6**

CALCULATE THE DESIGN SEISMIC FORCE FOR THE PROPOSED RETAINING WALL. ASSUME BACKFILL IS SATURATED AND THERE IS NO HYDROSTATIC PRESSURE THE RETAINED HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium	WALL HEIGHT	12 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	350 psf	SURCHARGE:	300 pounds
PHI ANGLE:	28 degrees	SURCHARGE TYPE:	u Uniform
DENSITY	125 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	350.0 psf	FINAL TENSION CRACK:	20 feet

PHID = ATAN(TAN(PHI)/FS) = 28.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.33 g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	47 degrees
AREA OF TRIAL FAILURE WEDGE	61.7 square feet
TOTAL EXTERNAL SURCHARGE	2100.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	9810.5 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1020 trials
LENGTH OF FAILURE PLANE	11.7 feet
DEPTH OF TENSION CRACK	3.4 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	8.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**2781.6 pounds**

CONCLUSIONS:

THE CALCULATION INDICATES THAT NO ADDITIONAL SEISMIC LOADING IS REQUIRED FOR CANTILEVER AND RESTRAINED RETAINING WALLS UP TO 12 FEET HIGH, WITH LEVEL BACKSLOPE AND VEHICULAR SURCHARGE (CALCULATED SEISMIC THRUST IS LESS THAN THE ACTIVE THRUST OF 3,096 POUNDS, AND AT-REST THRUST OF 4,723.2 POUNDS).

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AERIAL VICINITY MAP

BG: 22581 STOCKDALE CAPITAL PARTNERS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 100'

REFERENCE: LOS ANGELES COUNTY DEPARTMENT OF REGIONAL PLANNING, GIS-NET, 2013, http://gis.planning.lacounty.gov/GIS-NET_Public/Viewer.html

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REGIONAL TOPOGRAPHIC MAP

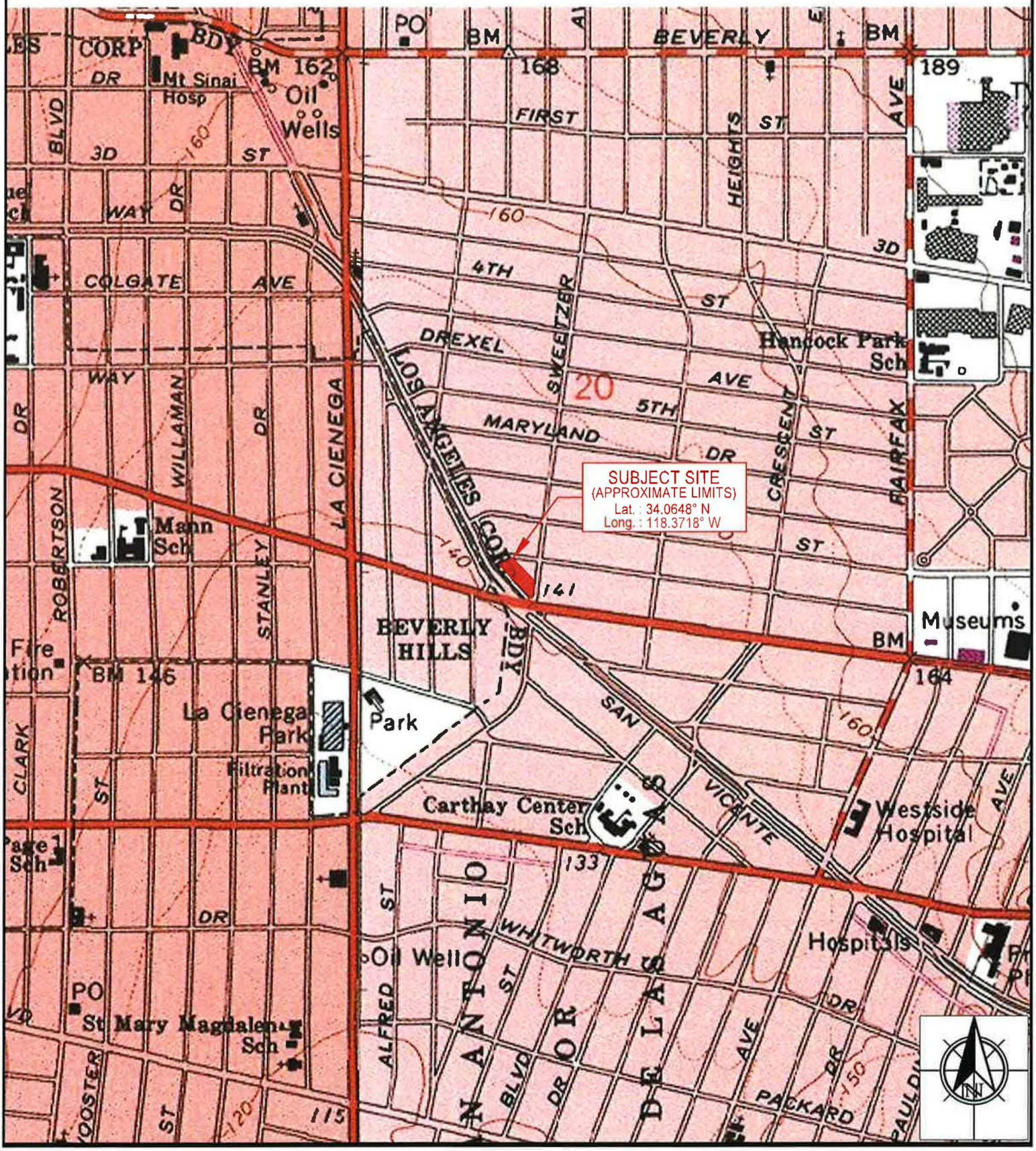
BG: 22581 STOCKDALE CAPITAL PARTNERS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: USGS TOPOGRAPHIC MAP, HOLLYWOOD 7.5-MINUTE SERIES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA CREATED 1964.



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HISTORIC TOPOGRAPHIC MAP

BG: 22581 STOCKDALE CAPITAL PARTNERS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: USGS TOPOGRAPHIC MAP, HOLLYWOOD 6-MINUTE SERIES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA CREATED 1942.

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REGIONAL GEOLOGIC MAP

BG: 22581 STOCKDALE CAPITAL PARTNERS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: DIBBLEE, T.W. (1991), GEOLOGIC MAP OF THE HOLLYWOOD AND BURBANK (SOUTH 1/2) QUADRANGLES, LOS ANGELES, CALIFORNIA DIBBLEE GEOLOGICAL FOUNDATION, MAP DF-30.
– DIBBLEE, T.W. (1991), GEOLOGIC MAP OF THE BEVERLY HILLS AND VAN NUYS (SOUTH 1/2) QUADRANGLES, LOS ANGELES, CALIFORNIA DIBBLEE GEOLOGICAL FOUNDATION, MAP DF-31.

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REGIONAL FAULT MAP

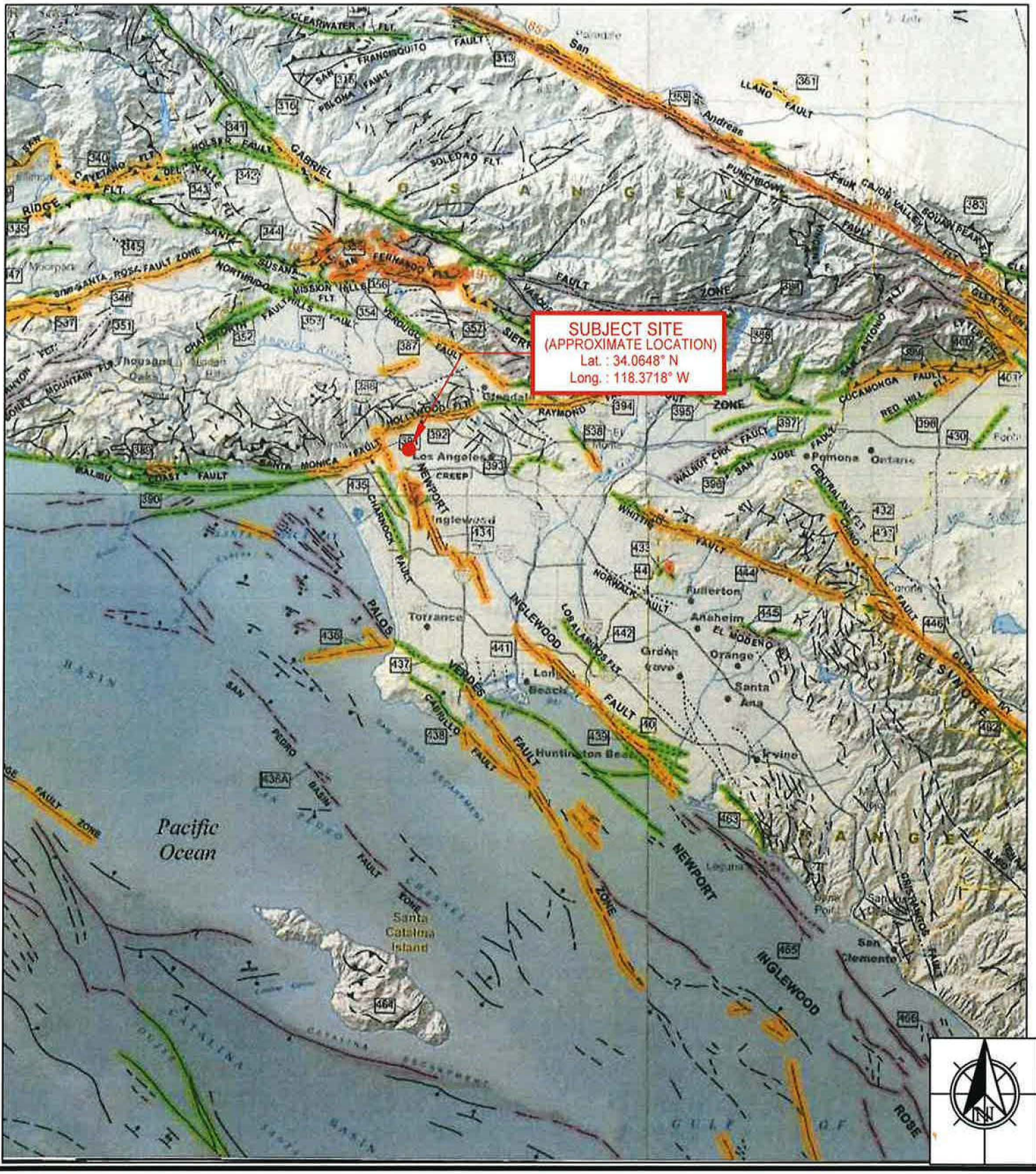
BG: 22581 STOCKDALE CAPITAL PARTNERS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 12 MILES

REFERENCE: JENNINGS, C.W., AND BRYANT, W.A.,2010, FAULT ACTIVITY MAP OF CALIFORNIA GEOLOGICAL SURVEY, 150th ANNIVERSARY, MAP No 6.



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SEISMIC HAZARD ZONES MAP

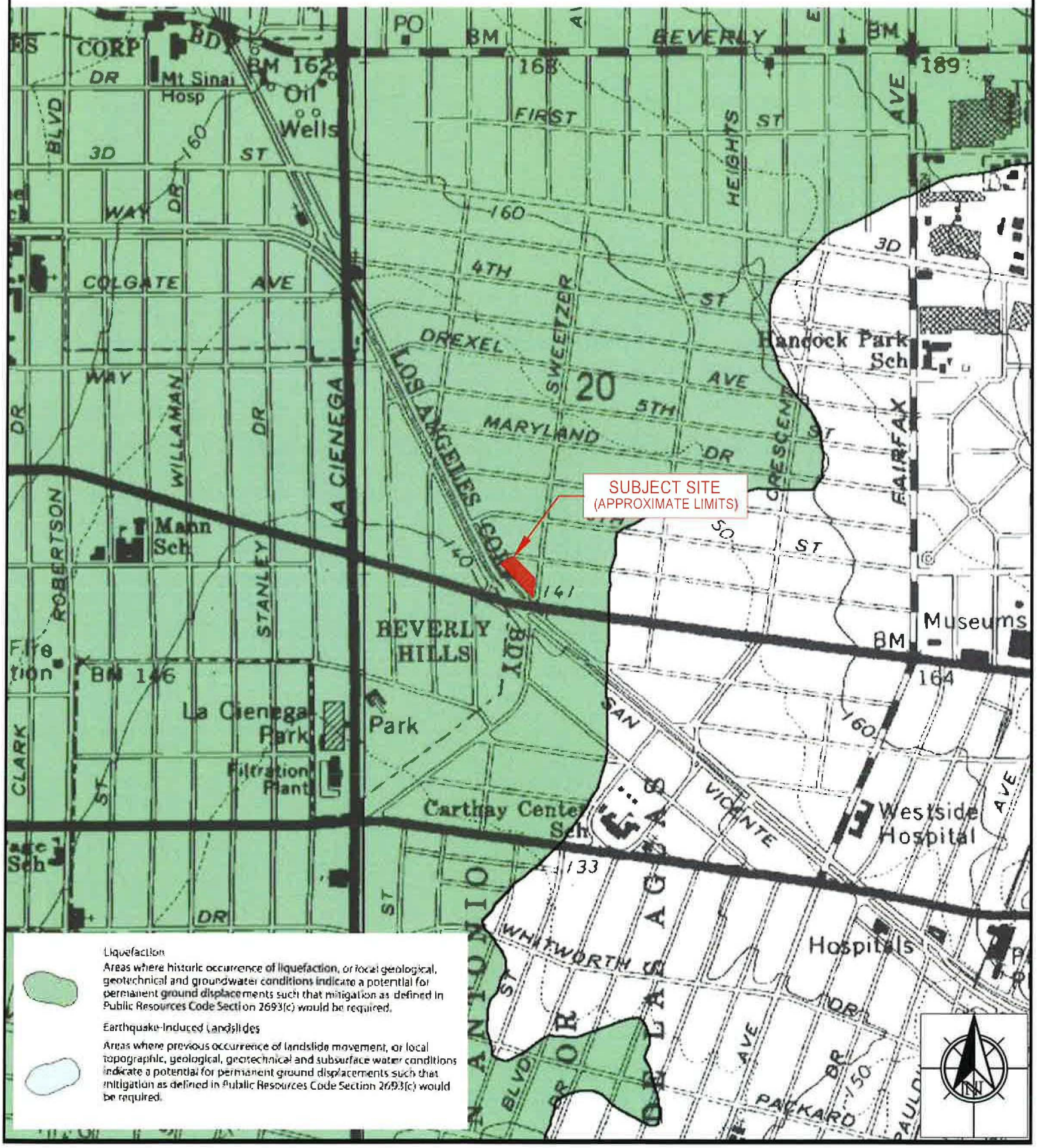
BG: 22581 STOCKDALE CAPITAL PARTNERS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: STATE OF CALIFORNIA SEISMIC HAZARD ZONES, HOLLYWOOD & BEVERLY HILLS QUADRANGLE OFFICIAL MAP, CALIFORNIA GEOLOGICAL SURVEY, DATED MARCH 25, 1999.



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HISTORIC-HIGH GROUNDWATER MAP

BG: 22581 STOCKDALE CAPITAL PARTNERS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 4000'

REFERENCE: CGS, 1998, Seismic Hazard Zone Report for the Beverly Hills 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 023, and CGS, 1998, Seismic Hazard Zone Report for the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 026.

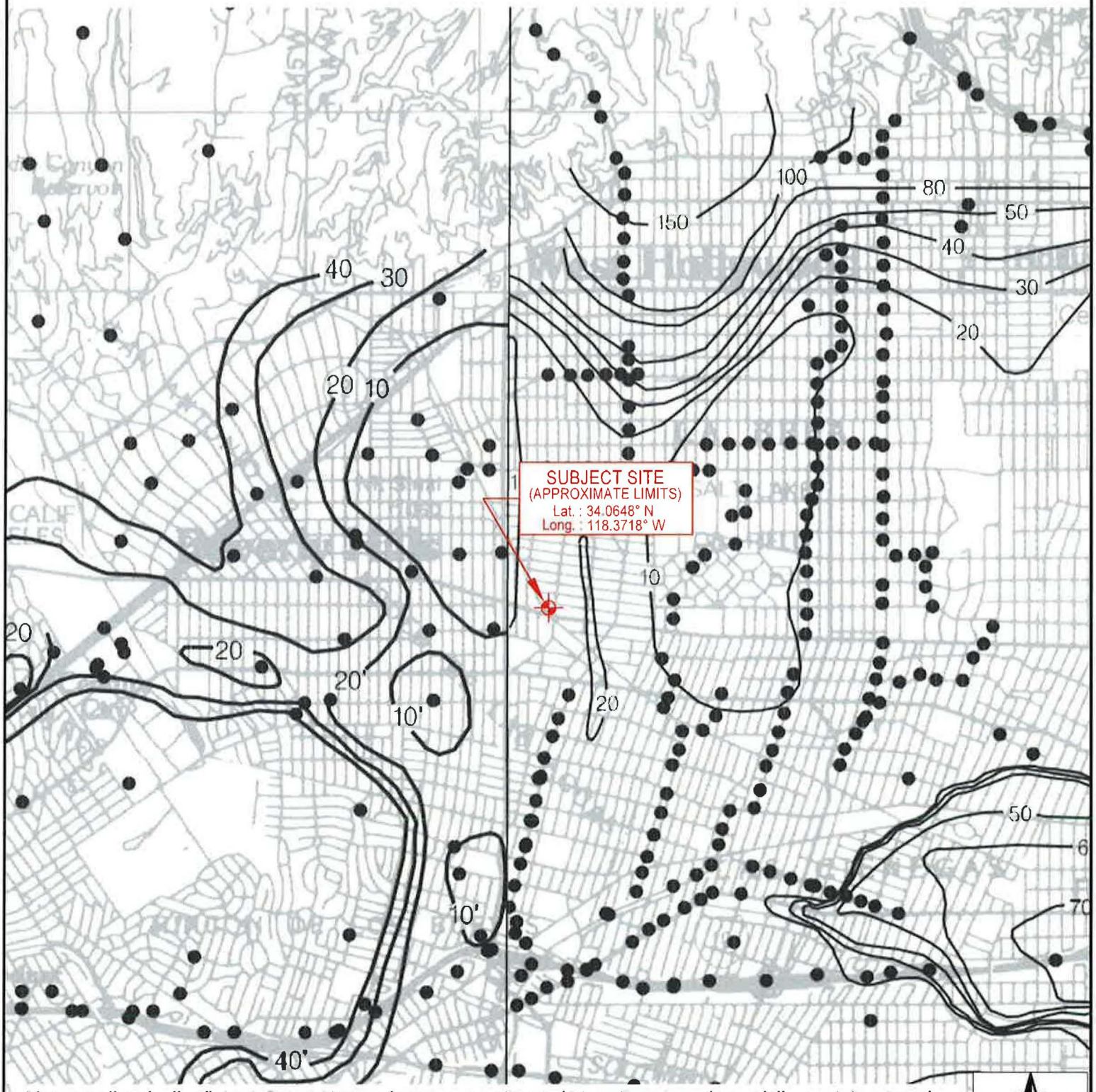
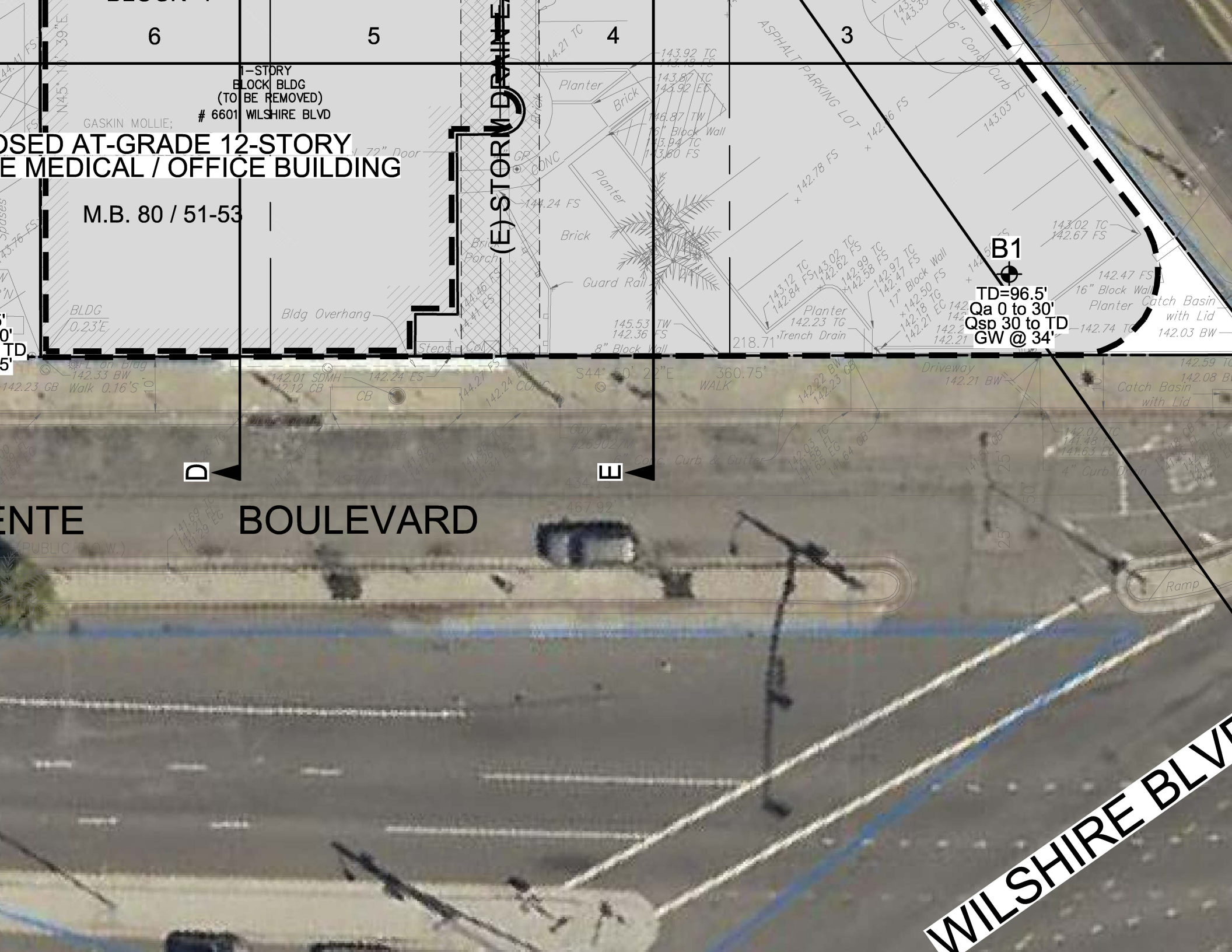
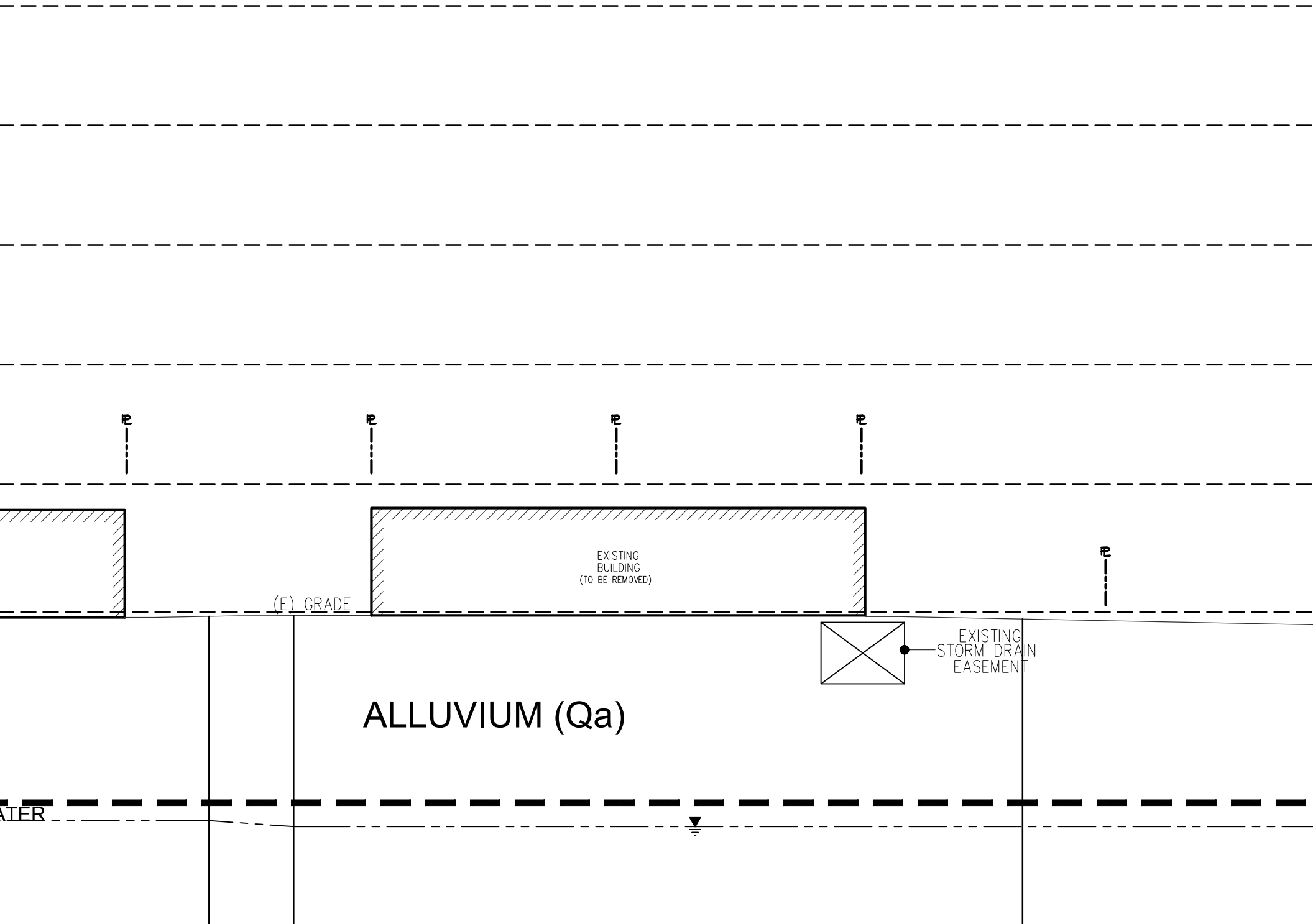


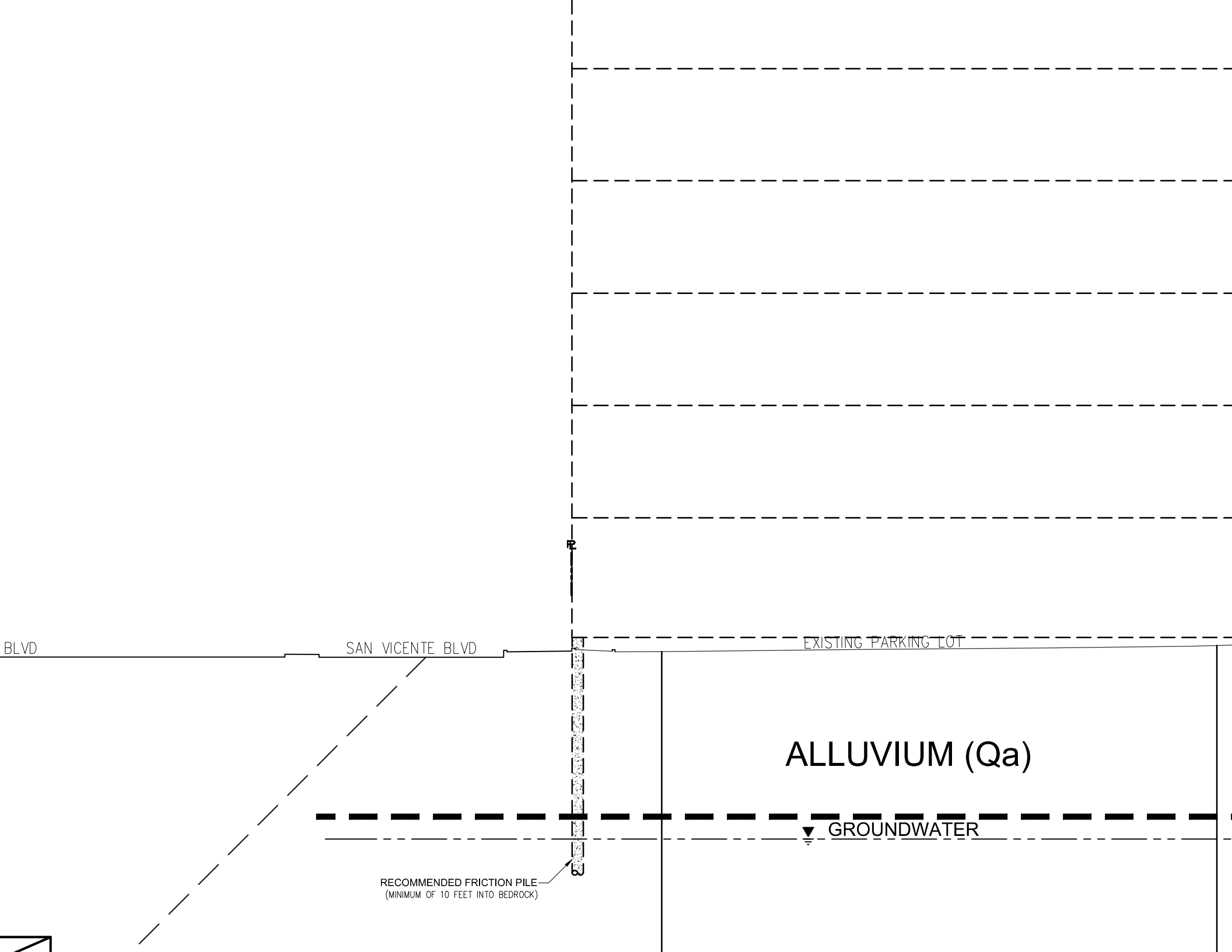
Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, Hollywood Quadrangle.

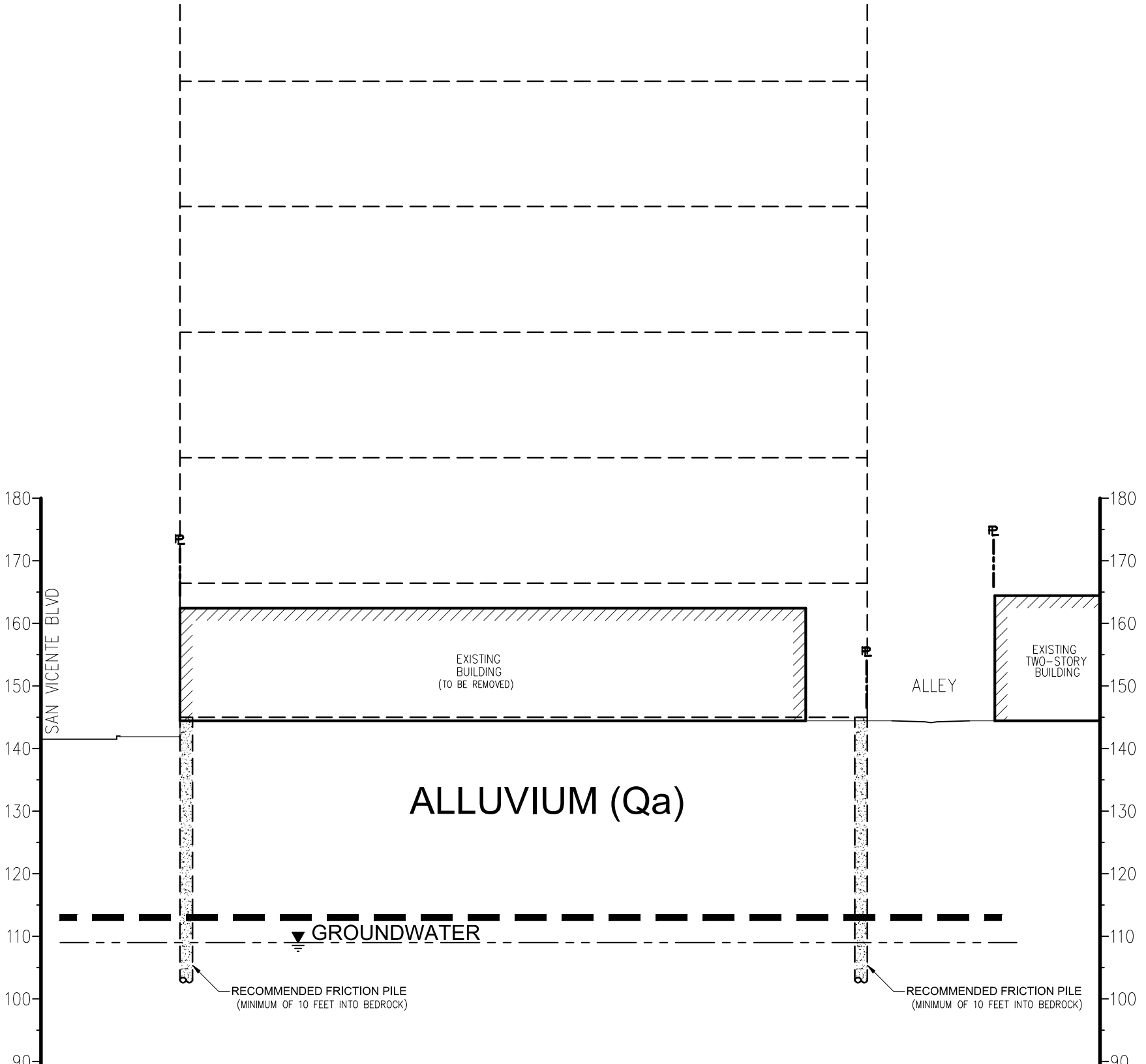
● Borehole Site

30 Depth to ground water in feet


6
5
4
3
1-STORY
BLOCK BLDG
(TO BE REMOVED)
6601 WILSHIRE BLVD
GASKIN MOLLIE;
OSED AT-GRADE 12-STORY
E MEDICAL / OFFICE BUILDING
M.B. 80 / 51-53
(E) STORM DRAIN
B1
TD=96.5'
Qa 0 to 30'
Qsp 30 to TD
GW @ 34'
D
E
ENTE
BOULEVARD
WILSHIRE BLVD
Planter
Brick
Guard Rail
Trench Drain
Driveway
Catch Basin
with Lid
Ramp
Bldg Overhang
ASPHALT PARKING LOT
Curb & Gutter
WALK
218.71'
360.75'
467.92'


EXISTING
BUILDING
(TO BE REMOVED)
(E) GRADE
EXISTING
STORM DRAIN
EASEMENT
ALLUVIUM (Qa)
ATER


BLVD
SAN VICENTE BLVD
EXISTING PARKING LOT
ALLUVIUM (Qa)
GROUNDWATER
RECOMMENDED FRICTION PILE
(MINIMUM OF 10 FEET INTO BEDROCK)


180
170
160
150
140
130
120
110
100
90
SAN VICENTE BLVD
EXISTING
BUILDING
(TO BE REMOVED)
ALLEY
EXISTING
TWO-STORY
BUILDING
ALLUVIUM (Qa)
GROUNDWATER
RECOMMENDED FRICTION PILE
(MINIMUM OF 10 FEET INTO BEDROCK)
RECOMMENDED FRICTION PILE
(MINIMUM OF 10 FEET INTO BEDROCK)