Appendix E
Geotechnical Report

CITY OF LOS ANGELES

CALIFORNIA

ERIC GARCETTI
MAYOR

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

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

SOILS REPORT APPROVAL LETTER

April 28, 2020

LOG # 112843
SOILS/GEOLOGY FILE - 2

Lankershim Crossing, LLC
23622 Calabasas Road, Suite 121
Calabasas, CA 91302

TRACT:	Lankershim Ranch Land and Water Co. (M R 31-39/44)
LOT(S):	PT 24 (Arb.'s 5, 6, 7, 8, 9, 10, 11 & 25) // PT 24 (Arb.'s 5, 13, 37 & 42)
LOCATION:	7918 - 7946 N. Lankershim Blvd. // 11650 - 11664 W. Strathern St.

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Soils Report	BG 23185	03/30/2020	Byer Geotechnical, Inc.
Oversized Doc(s).	``	``	``

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed 7-story mixed-use building over 1-subterranean parking level, and a 2-story at-grade recreation building, as shown on the Site Plan and Cross Sections A through E of the 03/30/2020 report. Retaining walls up to 14 feet high are proposed per the consultants.

Four borings were performed to depths ranging from 21.5 to 41.5 feet. In addition, five borings and three CPT's to depths ranging from 31.5 to 51.5 feet were previously performed by another consultant, along with two shallow percolation tests. The earth materials at the subsurface exploration locations consist of up to 3 feet of uncertified fill underlain by alluvium. According to the consultants, groundwater was not encountered to the maximum depths explored of 51.5 feet, and historically highest groundwater level is on the order of 130 feet below the ground surface. The site is relatively level.

The consultants recommend to support the proposed structure(s) on conventional foundations bearing on native undisturbed soils (for the 7-story building over 1-subterranean parking level), and the 2-story at-grade recreation building on a blanket of properly placed fill a minimum of 3 feet thick below the bottom of the footings.

The referenced report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2020 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. The entire site shall be brought up to the current Code standard (7005.9).

2. Approval shall be obtained from the Department of Public Works, Bureau of Engineering, Development Services and Permits Program for the proposed removal of support and/or retaining of slopes adjoining to public way (3307.3.2).

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3. The soils engineer shall review and approve the detailed plans prior to issuance of any permit. This approval shall be by signature on the plans that clearly indicates the soils engineer has reviewed the plans prepared by the design engineer; and, that the plans included the recommendations contained in their reports (7006.1).

4. All recommendations of the report that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

5. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans (7006.1). Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.

6. A grading permit shall be obtained for all structural fill and retaining wall backfill (106.1.2).

7. 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.

8. If import soils are used, no footings shall be poured until the soils engineer has submitted a compaction report containing in-place shear test data and settlement data to the Grading Division of the Department; and, obtained approval (7008.2).

9. Compacted 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 (7011.3).

10. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill (1809.2, 7011.3).

11. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction (7013.12).

12. Grading shall be scheduled for completion prior to the start of the rainy season, or detailed temporary erosion control plans shall be filed in a manner satisfactory to the Grading Division of the Department and the Department of Public Works, Bureau of Engineering, B-Permit Section, for any grading work in excess of 200 cubic yards (7007.1).

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13. All loose foundation excavation material shall be removed prior to commencement of framing (7005.3).

14. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the General Safety Orders of the California Department of Industrial Relations (3301.1).

15. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring or constructed using ABC slot cuts, as recommended. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

16. Prior to the issuance of any permit that authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation (3307.1).

17. The soils engineer shall review and approve the shoring plans prior to issuance of the permit (3307.3.2).

18. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

19. Unsurcharged temporary excavations exposing fill shall be trimmed back at a gradient not exceeding 1:1, as recommended.

20. Unsurcharged temporary excavation exposing alluvium may be cut vertical up to 5 feet with level backslope, as recommended.

21. Unsurcharged temporary excavation exposing alluvium may be cut vertical up to 4 feet. For excavations over 4 feet, the lower 4 feet may be cut vertically and the portion of the excavation above 4 feet shall be trimmed back at a gradient not exceeding 1:1, as recommended.

22. Shoring shall be designed for the lateral earth pressures as specified in the section titled "Soldier Piles" on page 21 of the 03/30/2020 report; all surcharge loads shall be included into the design. Total lateral load on shoring piles shall be determined by multiplying the recommended EFP by the pile spacing.

23. Shoring shall be designed for a maximum lateral deflection of 1 inch, provided there are no structures within a 1:1 plane projected up from the base of the excavation. Where a structure is within a 1:1 plane projected up from the base of the excavation, shoring shall be designed for a maximum lateral deflection of ½ inch, or to a lower deflection determined by the consultant that does not present any potential hazard to the adjacent structure.

24. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.

25. Surcharged ABC slot-cut method may be used for temporary excavations with each slot-cut not exceeding 5 feet in height and not exceeding 8 feet in width, as recommended. The surcharge load shall not exceed the value given in the report. The soils engineer shall determine the clearance between the excavation and the existing foundation. The soils engineer shall verify in the field if the existing earth materials are stable in the slot-cut excavation. Each slot shall be inspected by the soils engineer and approved in writing prior to any worker access.

26. All foundations shall derive entire support from native undisturbed soils, or a blanket of properly placed fill (a minimum of 3 feet thick below the bottom of the footings), as recommended and approved by the soils engineer by inspection.

27. Footings supported on approved compacted fill shall be reinforced with a minimum of four (4), ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top of the footing.

28. Slabs placed on approved compacted fill shall be at least 3½ inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced a maximum of 16 inches on center each way.

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

30. Cantilevered retaining walls up to 14 feet in height with a level backfill shall be designed for a minimum equivalent fluid pressure (EFP) of 43 PCF, as specified on page 17 of the 03/30/2020 report. All surcharge loads shall be incorporated into the design.

31. Retaining walls higher than 6 feet shall be designed for lateral earth pressure due to earthquake motions as specified on page 18 of the 03/30/2020 report (1803.5.12).

32. Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure as specified on page 18 of the 03/30/2020 report (1610.1). All surcharge loads shall be included into the design.

33. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted in a non-erosive device to the street in an acceptable manner (7013.11).

34. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls 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 soils report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record (1805.4).

35. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector (108.9).

36. Basement walls and floors shall be waterproofed/damp-proofed with an LA City approved "Below-grade" waterproofing/damp-proofing material with a research report number (104.2.6).

37. Prefabricated drainage composites (Miradrain, Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.

38. The structures shall be connected to the public sewer system per P/BC 2020-027.

39. The infiltration facility design and construction shall comply with the minimum requirements specified in the Information Bulletin P/BC 2020-118.

40. The infiltration system (dry well) shall be constructed within the landscaping area at the southeast portion of the site, as recommended on pages 26 and 27 of the 03/30/2020 report.

41. Infiltration shall occur below a depth of 10 feet, as recommended.

42. The construction of the infiltration system shall be provided under the inspection and approval of the soils engineer.

43. An overflow outlet shall be provided to conduct water to the street in the event that the infiltration system capacity is exceeded. (P/BC 2020-118)

44. Approval for the proposed infiltration system from the Bureau of Sanitation, Department of Public Works shall be secured.

45. A minimum distance of 10 feet (in any direction) shall be provided from adjacent proposed/existing footings to the discharge of the proposed infiltration system. A minimum distance of 10 feet horizontally shall be provided from private property lines to the proposed infiltration system.

46. The dry well area between the blank casing and the surround soils shall be sealed to a minimum depth of 10 feet below the bottom of any adjacent foundation with bentonite slurry (or equivalent) to prevent unintended leakage or horizontal infiltration.

47. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS (7013.10).

48. The soils engineer 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 (7008, 1705.6 & 1705.8).

49. Prior to pouring concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the work inspected meets the conditions of the report. No concrete shall be poured until the LADBS Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)

50. Prior to excavation an initial inspection shall be called with the LADBS Inspector. During the initial inspection, the sequence of construction; shoring; ABC slot cuts; protection fences; and, dust and traffic control will be scheduled (108.9.1).

51. Installation of shoring and/or slot cutting shall be performed under the inspection and approval of the soils engineer and deputy grading inspector (1705.6, 1705.8).

52. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the soil inspected meets the conditions of the report. No fill shall be placed until the LADBS Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be included 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. In addition, an Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included (7011.3).

53. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.

GLEN RAAD
Geotechnical Engineer I

Log No. 112843
213-482-0480

cc: GP Design Group, LLC, Applicant
Byer Geotechnical, Inc., Project Consultant
VN District Office

BYER GEOTECHNICAL, INC.

March 30, 2020
BG 23185

Lankershim Crossing, LLC
23622 Calabasas Road #121
Calabasas, California 91302

Attention: Mr. Manny Kozar

Subject

Transmittal of Geotechnical Engineering Exploration
Proposed Seven-Story Mixed-Use Building over One Subterranean Parking Level
and Two-Story At-Grade Recreation Building
Arbs. 5, 6, 7, 8, 9, 10, 11, 13, 25, 37, and 42, Portion of Lot 24,
Lankershim Ranch Land and Water Co. Tract
7918 - 7946 North Lankershim Boulevard and 11650 - 11664 West Strathern Street
North Hollywood, California

Dear Mr. Kozar:

Byer Geotechnical has completed our report dated March 30, 2020, which describes the geotechnical engineering conditions with respect to the proposed project. The reviewing agency for this document is 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. Copies of the report have been distributed as follows:

(4) Addressee (E-mail and Mail)
(1) Lankershim Crossing, LLC, Attention: Liat Kozar (E-mail)

It is our understanding that you or your representative 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.

GEOTECHNICAL ENGINEERING EXPLORATION
PROPOSED SEVEN-STORY MIXED-USE BUILDING OVER ONE SUBTERRANEAN PARKING LEVEL AND TWO-STORY AT-GRADE RECREATION BUILDING

ARBS. 5, 6, 7, 8, 9, 10, 11, 13, 25, 37, AND 42, PORTION OF LOT 24,
LANKERSHIM RANCH LAND AND WATER CO. TRACT
7918 - 7946 NORTH LANKERSHIM BOULEVARD AND
11650 - 11664 WEST STRATHERN STREET
NORTH HOLLYWOOD, CALIFORNIA
FOR LANKERSHIM CROSSING, LLC
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 23185
MARCH 30, 2020

1461 East Chevy Chase Drive, Suite 200 • Glendale, California 91206 • tel 818.549.9959 • fax 818.543.3747 • www.byergeo.com

GEOTECHNICAL ENGINEERING EXPLORATION
PROPOSED SEVEN-STORY MIXED-USE BUILDING OVER
ONE SUBTERRANEAN PARKING LEVEL AND
TWO-STORY AT-GRADE RECREATION BUILDING
ARBS. 5, 6, 7, 8, 9, 10, 11, 13, 25, 37, AND 42, PORTION OF LOT 24,
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FOR LANKERSHIM CROSSING, LLC
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 23185
MARCH 30, 2020

INTRODUCTION

This report has been prepared per our signed Agreement and summarizes findings of Byer Geotechnical, Inc., geotechnical engineering exploration performed on the subject site. The purpose of this study is to evaluate the nature, distribution, and the engineering properties of the earth materials underlying the site with respect to construction of the proposed project. 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. Manny Kozar and the undated preliminary plans prepared by Kamran Tabrizi Architect & Associates. Final plans have not been prepared and await the conclusions and recommendations of this report. The project consists of construction of a seven-story mixed-use building over one subterranean parking level, planned to occupy the majority of the site. In addition, a two-story at-grade Recreation Building is planned in the southeast corner. A cul-de-sac street improvement is planned at the west terminus of Blythe Street, which is located just north of the proposed Recreation Building, as shown on the enclosed Site Plan. The ground floor of the proposed mixed-use building will consist of retail/restaurant spaces fronting on Lankershim Boulevard, and commercial parking spaces in rear. Retaining walls up to 14 feet high are planned to support the excavation for the subterranean parking level. Column loads (dead and live) are expected to be moderate. Grading is anticipated to consist of removal and recompaction of earth materials to create a compacted fill pad for support of the two-story, at-grade Recreation Building. Access to the ground floor and subterranean level parking will be provided by two ramps that are planned in the southwest and northeast corners of the building via Lankershim Boulevard and Strathern Street, respectively.

RESEARCH - PRIOR WORK

The following geotechnical report was prepared for the property by Twining Consulting (Project No. 160511.1):

> *Updated Geotechnical Evaluation Report, Lankershim Square, 7934 Lankershim Boulevard, North Hollywood, California,* dated June 15, 2017.

This report was prepared for a mixed-use residential-shopping center development consisting of three buildings (A, D, and E) up to three stories in height. A subterranean parking garage was planned under Building E. The field exploration included five, 8-inch-diameter hollow-stem-auger borings (TB1 through TB5) and three Cone Penetration Test soundings (CPT1 through CPT3) to approximate depths of 31½ to 51½ feet below ground surface. In addition, two shallow percolation tests (P1 and P2) were conducted onsite to determine the feasibility of infiltrating water into the site soils. Excerpts from the referenced report are included in Appendix I, including a copy of the boring logs and an Exploration Location Map showing the locations of the borings and CPTs by Twining Consulting.

Based on our conversation with the client, that report was not submitted for review by the LADBS. The Twining Consulting report is provided for information purposes only. The field exploration data contained in the above-referenced report was incorporated as part of our work on this project. The conclusions and recommendations included in this report are based solely on the field data, results of laboratory testing, and engineering analysis performed by Byer Geotechnical as part of this study.

EXPLORATION

The scope of the field exploration was determined from our initial site visit and consultation with Mr. Manny Kozar. The undated preliminary plans prepared by Kamran Tabrizi Architect & Associates were a guide to our work on this project. 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.

Exploration was conducted on January 30, 2020, with the aid of a hollow-stem-auger drill rig. It included drilling four borings (BG1 - BG4) to approximate depths of 21½ to 41½ feet below ground surface. 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. Following drilling, sampling, and logging, the borings were backfilled and mechanically tamped.

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.

The proposed project and the locations of the recent borings and previous Twining borings and CPT soundings 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 eleven contiguous relatively-level parcels located in the easternmost portion of the San Fernando Valley, in the North Hollywood section of the city of Los Angeles, California (34.2149° N Latitude, 118.3868° W Longitude). As depicted on the enclosed Aerial Vicinity Map, the property is bounded by Strathern Street and a restaurant on the north, single-family residences and a commercial building on the east and south, and Lankershim Boulevard on the west. It is located approximately 4 miles north of the Ventura (101) Freeway and 1.25 miles east of the Hollywood (170) Freeway. The site is developed with a one-story commercial building on the west portion. The remainder of the property is used as a landscaping and building materials stock yard. The surrounding area has been developed with single-family residences and commercial establishments along Lankershim Boulevard.

Past grading on the site has consisted of placing minor fill to create a level site. Vegetation on the site is sparse. Surface drainage is by sheetflow runoff down the contours of the land to the south.

GROUNDWATER

Groundwater was not encountered in the borings by Byer Geotechnical or by Twining Consulting to a maximum depth of 51½ feet. Based on our review of the hydrological records of Los Angeles County Department of Public Works, groundwater levels in the vicinity of the site ranged from 170.3 to 339.3 feet below existing grade, between 1986 and 2009 (LADPW, 2020). These measurements were obtained from a monitoring well (No. 4918A) that is located approximately one-quarter of a mile east of the subject site.

In *Seismic Hazard Zone Report 08,* the California Geological Survey (CGS) has estimated the historically-highest groundwater level at the site was on the order of 130 feet below ground surface (CGS, 1997), as shown on the enclosed Historic-High Groundwater Map.

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. The subject property is not mapped within either a Methane Zone or Methane Buffer Zone.

EARTH MATERIALS

Fill (Afu)

Fill, associated with previous site grading, underlies the subject site to a maximum observed depth of three feet in the borings by Twining Consulting. Greater depths of fill may occur locally. The fill consists of silty sand that is olive-brown to dark brown, and dry to slightly moist. The existing fill is not suitable for support of any type of structure. Most of the existing fill is expected to be removed during the excavation for the subterranean parking level. Recommendations for ground improvement for the proposed Recreation Building are included in the "Site Preparation-Removals" section of this report.

Alluvium (Qa/Qyf2)

Natural alluvial fan deposits associated with the Pacoima Tujunga Fan typical for this portion of North Hollywood underlie the existing fill and were encountered in the borings. The alluvium predominately consists of layers of poorly-graded to gravelly sand that are tan to olive-brown, dry to slightly moist, and medium dense to very dense.

GEOTECHNICAL CHARACTERISTICS

In-Situ Percolation Testing

To determine the infiltration rate and evaluate the infiltration characteristics of the earth materials underlying the subject site, *in-situ* percolation testing was conducted in Boring BG1, which was drilled to a depth of 41½ feet below ground surface. Following drilling, a PVC pipe was inserted into the boring, covered with a filter sock and surrounded with onsite excavated soil. The test was performed in accordance with the Administrative Manual of the County of Los Angeles, Department of Public Works, Section GS200.2, dated June 30, 2017. The upper 10 feet of the pipe was solid.

The lower 30 feet was screened to allow water infiltration below the upper 10 feet of the earth materials. The boring was presoaked below the depth of 10 feet utilizing water from the drill rig and was allowed to set for at least 30 minutes. Following presoaking, a falling-head percolation test was conducted. The test consisted of ceasing the flow of water into the boring and measuring the drop of the water surface (head) at 10-minute intervals. The test was repeated eight times.

The results of the infiltration rate calculations are shown on the enclosed Calculation Sheet #1. Based on the results of *in-situ* percolation testing, the calculated infiltration rate for the earth materials between the depths of 10 and 40 feet is estimated to be 10.5 inches-per-hour (7.4×10^{-3} centimeters-per-second).

The calculated infiltration rate does not include a reduction factor, which should be determined in accordance with the Administrative Manual, Section GS200.2.

Perched Water

Perched water can occur on finer-grained, relatively impermeable soil layers. Poorly- and well-graded sand layers were encountered at the subject site. Therefore, it is the opinion of Byer Geotechnical that the potential for creating perched groundwater conditions is considered to be very low.

Expansive Soil

The onsite soils have a very low expansion potential.

Hydroconsolidation

The results of dry density and consolidation tests indicate that the subsurface earth materials are medium dense to very dense. Consolidation tests indicate a minor potential for hydroconsolidation, which does not require mitigation.

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 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 Verdugo, Sierra Madre (San Fernando), Santa Monica, and Northridge Faults. Fifty 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 Verdugo Fault, a Type B fault that is located 1.9 kilometers (1.2 miles) east of the site. The Verdugo Fault is capable of producing a maximum moment magnitude of 6.9 and an average slip rate of 0.5 ± 0.5 millimeters per year (Cao et al., 2003). The San Andreas Fault, a Type A fault, is located 44.4 kilometers (27.6 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 table lists the applicable City of Los Angeles Building Code seismic coefficients for the project:

SEISMIC COEFFICIENTS (2020 City of Los Angeles Building Code - Based on ASCE Standard 7-16)		
Latitude = 34.2149° N Longitude = 118.3868° W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 20.3.3, ASCE Standard 7-16	Alluvium - D	
Mapped Spectral Accelerations from Figures 22-1 and 22-2 and USGS	S_s = 2.003 (g)	S_1 = 0.685 (g)
Site Coefficients from Tables 11.4-1 and 11.4-2 and USGS	F_A = 1.0	F_V = 1.7 (g)
Maximum Considered Spectral Response Accelerations from Equations 11.4-1 and 11.4-2	S_{MS} = 2.003 (g)	S_{M1} = 1.165 (g)
Design Spectral Response Accelerations from Equations 11.4-3 and 11.4-4	S_{DS} = 1.335 (g)	S_{D1} = 0.776 (g)
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, adjusted for Site Class effects	PGA_M = 0.903 (g)	

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

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 parameters for the site for a 1-second period (S_{D1}) is greater than 0.20g, and the short period (S_{DS}) is greater than 0.50g. Therefore, 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

Probabilistic seismic hazard deaggregation analysis was performed on the subject site. Seismic parameters were determined using currently-available earthquake and fault information utilizing data from the United States Geological Survey (USGS) National Seismic Hazard Mapping Project (USGS, 2008). An average shear-wave velocity (Vs30) of 259 meters-per-second (Site Class D) was used in the analysis. Hazard deaggregation indicates a predominant modal earthquake magnitude of 6.3 (Mw) at a modal distance of 10 kilometers. The Peak Horizontal Ground Acceleration (PHGA) with a 10-percent probability of exceedance in 50 years is estimated to be 0.55g on the subject site. These ground motions could occur at the site during the life of the project. Results of the analysis are graphically presented in the enclosed "Seismic Hazard Deaggregation Chart" (Appendix III).

Based on a Site Class D, the MCE_G peak ground acceleration adjusted for Site Class effects, PGA_M, is 0.903g. The pseudo-static seismic coefficient (k_h) was derived according to the guidelines of the LADBS memorandum dated July 16, 2014. The horizontal pseudo-static seismic coefficient (k_h) was taken as one-third of the PGA_M (0.30g) and was used in the seismic calculations for the cantilever and restrained subterranean retaining walls.

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)*									
Seismic Response Spectra	Fundamental Period (seconds)								
	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Probabilistic MCE_R	1.9200	1.9363	1.8660	1.7823	1.6389	1.5314	1.4205	1.3099	1.2204
Probabilistic (ASCE 7-16)	1.3353	1.3353	1.3353	1.3353	1.3353	1.3353	1.3353	1.2685	1.1417
Deterministic MCE_R (84th Percentile)	1.2600	1.3430	1.3860	1.3820	1.3000	1.2320	1.1500	1.064	0.9943
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
Site Specific MCE_R	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.4200	1.3100	1.2200
80% Design Response Spectrum	1.0680	1.0680	1.0680	1.0680	1.0680	1.0680	1.0680	1.0150	0.9130
Site-Specific Design Response Spectrum	**1.0680**	**1.0680**	**1.0680**	**1.0680**	**1.0680**	**1.0680**	**1.0680**	**1.0150**	**0.9130**

* 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 shown in the following table:

SITE-SPECIFIC SPECTRAL RESPONSE ACCELERATION PARAMETERS (Based on ASCE Standard 7-16 - Chapter 21)		
Latitude = 34.2149° N Longitude = 118.3868° W	Short Period (0.2s)	One-Second Period
Maximum Considered Spectral Response Accelerations Chapter 21 - Section 21.4	S_{MS} = 1.602 (g)	S_{M1} = 1.370 (g)
Design Spectral Response Accelerations Chapter 21 - Section 21.4	S_{DS} = 1.068 (g)	S_{D1} = 0.913 (g)

Liquefaction

The CGS has not 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, as shown on the enclosed Seismic Hazard Zones Map. The site is underlain by very dense alluvium deposits, and current and historic-high groundwater levels are not present onsite. Therefore, the earth materials underlying the subject site are not considered subject to liquefaction.

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 not located within the flood hazard zone of Hansen Flood Control Basin, which is located approximately three miles north of the subject site. Furthermore, the site is at an average elevation of 805 feet above mean sea level. Therefore, the risk to the project from seiches or tsunamis is considered to be 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 by Byer Geotechnical, five borings and three CPT soundings by Twining Consulting, 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 geotechnical 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 for the proposed seven-story building over one subterranean parking level is undisturbed firm alluvium, which is anticipated at the basement level. The proposed two-story at-grade Recreation Building should be founded on a future compacted fill blanket. The subgrade for the proposed Blythe Street cul-de-sac should be prepared by removing any existing fill and replacing it as compacted fill. Conventional foundations may be used to support the proposed buildings. Soils to be exposed at finished grade are expected to exhibit a very low expansion potential.

Geotechnical issues affecting the project include temporary excavations ranging from 5 to 16 feet in height, including an estimate of the foundation embedment depth, which will be required to construct the subterranean parking level and during grading to create a compacted fill pad for the proposed at-grade building. Temporary shoring consisting of closely-spaced soldier piles and continuous lagging is recommended to facilitate the construction of the subterranean retaining walls abutting property lines as shown on Sections B, C, D, and E. Temporary excavations may be trimmed, where there is sufficient lateral distance from adjacent property lines, as shown on sections A and E. ABC slots are recommended along the east and south perimeters of the proposed at-grade building during grading adjacent to property boundaries. Recommendations for temporary shoring and temporary trimming are included in the "Temporary Excavations" section of this report.

SITE PREPARATION - REMOVALS

Surficial materials consisting of existing fill and disturbed alluvium blanket the site. Remedial grading is recommended to improve site conditions for the recreation building. The existing fill and disturbed alluvium should be removed to a minimum of three feet below the bottom of the footings of the proposed at-grade building 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 of the proposed at-grade building should be prepared by removing all vegetation, demolition debris, existing fill, and disturbed alluvium. The exposed excavated area should be observed by the soils engineer prior to placing compacted fill. Removal depths can be found in the "Site Preparation - Removals" section above. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 95 percent of the maximum dry density.

B. The proposed at-grade building site shall be excavated to a minimum depth of three feet below the bottom of all footings. The excavation shall extend beyond the edge of the exterior footing a minimum of three feet or to the depth of fill below the footing. The excavated areas shall be observed by the soils engineer 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 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 95 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 95 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

Spread Footings

Continuous and/or pad footings may be used to support the proposed buildings, provided they are founded in firm alluvium (seven-story building over a subterranean garage) and future compacted fill (Recreation Building). Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24-inches square. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Future Compacted Fill (At-Grade Building)	24	2,000	0.38	220	4,000
Alluvium (7-Story Building)	24	3,000	0.38	220	6,000

Increases in the bearing value are allowable at a rate of 20 percent for each additional foot of footing width or depth to the maximum bearing values listed in the table above. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing values shown above 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. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

All continuous footings should be reinforced with a minimum of four #4 steel bars: two placed near the top and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks, and approved by the geotechnical engineer prior to placing forms, steel, or concrete.

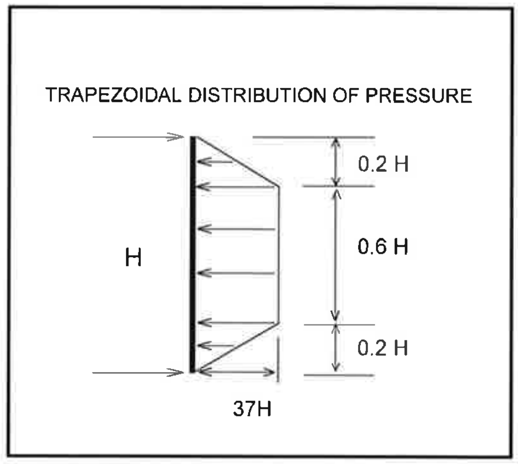
Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A total static settlement of 0.25 to 0.75 inches may be anticipated. Differential settlement should not exceed 0.5 inches within a horizontal distance of 30 feet.

RETAINING WALLS

General Design

Cantilever retaining walls up to 14 feet high, with a level backslope and uniform vehicular surcharge up to 300 pounds, may be designed for an active equivalent fluid pressure of 43 pounds-per-cubic-foot (see Calculation Sheet #2a). Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾-inch crushed gravel.



Subterranean retaining walls, which will be restrained, should be designed for the at-rest lateral earth pressure of 37H, 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. Surcharge loads from vehicular traffic and adjacent buildings should be added to the at-rest pressure for restrained retaining walls. Surcharge loads may be calculated using NAVFAC DM-7.02 Design Manual, LADBS Information Bulletin P/BC 2020-083, or an equivalent method.

Seismic analysis of the proposed cantilever retaining walls indicates an additional 362 pounds of loading due to seismic forces is required, since the calculated seismic thrust is more than the design active thrust for a restrained height up to 14 feet (see Calculation Sheet #2Sa). The seismic load should be applied at a height of 0.3H measured from the base of the wall. Seismic analysis also indicates that no additional loading due to seismic forces is required on restrained retaining walls since the calculated seismic thrusts are less than the design at-rest thrusts for retained heights of up to 14 feet high (see Calculation Sheets #3Sa and #4Sa).

Subterranean retaining walls should be provided with a subdrain covered with a minimum of 12 inches of ¾-inch crushed gravel. An alternative subdrain system consisting of Miradrain and gravel pockets connected to a solid pipe outlet may be used behind the subterranean retaining walls. The gravel pockets should be placed at the bottom of the retaining wall, midway between the shoring bays. A sump pump will be required for basement subdrains. The gravel pockets should be excavated to penetrate the slurry backfill behind the lagging to ensure contact with the earth materials behind the lagging.

Backfill

Retaining wall backfill should be compacted to a minimum of 95 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

Retaining wall footings may be sized per the "Spread Footings" section of this report.

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 to construct the subterranean parking level of the proposed seven-story building and to prepare a compacted fill pad for the proposed at-grade building. The excavations will range from 5 to 16 feet in height, including an estimate of the foundation embedment depth, and will expose minor fill over alluvium. The fill should be trimmed to 1:1 for wall excavations. The alluvium is capable of maintaining vertical excavations up to five feet with level backslope (see Calculation Sheet #8) and up to four feet with a 1:1 backslope (see Calculation Sheet #9).

Vertical excavations removing support from adjacent foundations or property lines will require the use of temporary shoring or slot cutting (ABC method). Closely-spaced soldier piles with wood lagging is recommended to be used as temporary shoring. Design values can be found in the "Soldier Piles" section below. Temporary excavations may consist of a combination of a maximum four-foot vertical and a 1:1 back cut where space permits, as shown on Sections A and E.

The slot cutting method uses the earth as a buttress and allows the excavation to proceed in phases. The initial excavation is made at a slope of 1:1. Alternate slots of eight feet in width may be worked (see Calculation Sheet #10). The remaining earth buttresses should be 16 feet in width. The "A" earth buttresses should be completed and backfilled before the "B" earth buttresses are excavated. The "C" earth buttresses may be excavated upon completion of the backfilling and compaction of the "B" areas.

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.

Soldier Piles

Drilled, cast-in-place concrete soldier piles are recommended as temporary shoring to support excavations to construct the subterranean parking level of the proposed seven-story building and to support offsite improvements. The piles should be a minimum of 18 inches in diameter and a minimum of eight feet into the alluvium below the excavation. Piles may be assumed fixed at three feet into the alluvium below the excavation. The piles may be designed for a skin friction of 500 pounds-per-square-foot for that portion of pile in contact with the alluvium below the excavation. Piles should be spaced a maximum of eight feet on center. Shoring spacing may be increased up to 10 feet on center in local areas such as ramp approaches and corners of shoring. The piles may be designed for the lateral pressures shown in the following table:

Shoring Height (feet)	Type of Surcharge	Maximum Surcharge (pounds)	Active Equivalent Fluid Pressure (pcf)	Trapezoidal Pressure	Reference
16	No Surcharge	-	30	19H	Calculation Sheet #5a
16	Vehicle	300 Uniform Load	32	20H	Calculation Sheet #6a
16	Building	1,200 Line Load	33	21H	Calculation Sheet #7a

If rakers are incorporated in the temporary shoring system, the soldier piles should be designed for the trapezoidal pressures shown on the table above, where H is the shored height.

The equivalent fluid pressure should be multiplied by the pile spacing. The piles may be included in the permanent retaining wall. Where a combination of sloped embankment and shoring is used, the pressure will be greater and must be determined for each combination.

Groundwater is not anticipated in the soldier pile excavations.

Lateral Design

The friction value is 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 below the excavation.

Passive earth pressure may be computed as an equivalent fluid having a density of 220 pounds-per-cubic-foot. 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 3-pile diameters on center may be considered isolated.

Lagging

Continuous lagging is recommended between the soldier piles. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. Lagging should be designed for the recommended earth pressure, but may be limited to a maximum value of 400 pounds-per-square-foot. The space behind lagging should be backfilled with cement slurry.

Lagging should be placed behind the front flange of the shoring steel I-beams. In some cases, the shoring is designed with the lagging behind the rear flange of the shoring steel I-beams. This is to maximize the interior area and position the walls as near the property lines as possible. During the installation of lagging behind the rear flange, the shoring is not supporting the excavation while the lagging is placed and backfilled. This can cause damage to adjacent offsite improvements, such as buildings, site walls, sidewalks, etc. If lagging is to be placed behind the rear flange of the I-beams, the lagging should be installed in slot cuts (ABC method), where lagging is installed and slurry-backfilled in the "A" slots before the "B" and "C" slots are excavated for lagging. Also, the maximum vertical height exposed should be no more than five feet.

Deflection

Some deflection of the shored embankment should be anticipated. Where shoring is planned adjacent to existing structures, it is recommended that lateral deflection not exceed one-half of an inch. For shoring not surcharged by a structure, the allowable deflection is deferred to the structural engineer. If greater deflection occurs during construction, additional bracing or anchors may be necessary to minimize deflection. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

FLOOR SLABS

Floor slabs should be cast over firm alluvium (Subterranean Level) or approved compacted fill (At-Grade Building) 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 approved compacted fill placed in accordance with the "Site Preparation - Removals" section of this report. Decking should be 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.

PAVING

Prior to placing paving in the proposed road cul-de-sac, any existing fill (estimated at 3 feet thick), should be removed and replaced as future compacted fill with a minimum relative compaction of 95 percent. Prior to placing fill, the removal bottom should be observed and scarified, moistened as required to obtain optimum moisture content, and recompacted to 95 percent of the maximum dry density, as determined by ASTM D 1557-12. Trench backfill below paving should be compacted to 95 percent of the maximum dry density. Irrigation water should be prevented from migrating under paving.

A representative bulk sample of the near-surface soils was obtained from Boring B1 for laboratory testing to determine the Expansion Index (see Appendix II). The results indicate an Expansion Index (EI) value of 7. Based on a correlation with the EI values, a preliminary R-value of 60 is considered appropriate for design of flexible pavement at the subject site. Based on the Caltrans Design Procedures (Cal Test 301), flexible pavement sections may consist of the following for the Traffic Indices (TI) indicated.

Traffic Index (TI)	Full-Depth Asphalt Thickness (No Base) (Inches)	Asphalt Concrete (AC) Pavement Thickness (Inches)	Class II Aggregate Base Thickness (Inches)
5	3.5	3.0	4.0
6	5.0	3.5	4.0
7	5.5	4.0	4.0
8	6.5	5.0	4.0
9	7.0	5.5	4.0

For rigid concrete pavement, four inches of concrete over six inches of aggregate base can be used. Concrete should be reinforced for heavy load application.

The Class II aggregate base and top one foot of subgrade should be compacted to a minimum of 95 percent of maximum dry density. Crushed aggregate base should meet the requirements of "Greenbook" (Standard Specification for Public Works Construction) Section 200-2.2.

UTILITY-TRENCH BACKFILL

Utility trenches on the subject site may be backfilled with the onsite soil, provided it is free of debris and oversize material. Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent (SE) of 30 or greater. The sand should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified in-place by water jetting. Soil backfill above the bedding sand should be placed in thin, loose layers, moistened as required, and compacted to at least 95 percent of the maximum dry density. The thickness of layers should be based on the type of equipment used for compaction in accordance with the recent edition of Standard Specifications for Public Works Construction (Greenbook).

CEMENT TYPE AND CORROSION PROTECTION

A bulk representative sample of the near-surface soil was obtained from Boring B1 for laboratory testing. Corrosion test results are included in Appendix II. The results indicate that concrete structures in contact with the soils onsite will have negligible exposure to water-soluble sulfates in the soil. According to Table 4.3.1 of Section 4.2 of the ACI 318 Code, Type II cement may be used for concrete construction.

The results of the laboratory testing also indicate that the near-surface soil is considered mildly corrosive to ferrous metals. The corrosion information presented in Appendix II of this report should be provided to the underground utility subcontractor.

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. Planters located next to raised-floor-type construction also should be sealed to the depth of the footings. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

Low-Impact Development (LID) Requirements

It is our understanding that the project is planned to utilize a drywell infiltration system. The coarse granular nature of the earth materials underlying the subject site is considered suitable for an infiltration rate of 10.5 inches-per-hour (7.4 x 10^{-3} centimeters-per-second) for design of the proposed drywell infiltration system. 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.

The following recommendations shall be incorporated into the design and construction of a drywell infiltration system:

- The infiltration system should be located within the landscaping (SE portion) area of the site. The system should not be installed below the footprint of the proposed buildings.

- The upper 10 feet (minimum) of the shaft should be sealed with a pre-cast concrete liner, or equivalent, to avoid any lateral water infiltration. The annular space behind the liner should be backfilled with cement slurry.

- The horizontal distance between the edge of the infiltration system and any adjacent property line should be at least 10 feet.

- The horizontal distance between the edge of the infiltration system and any adjacent structural foundations should be a minimum of 10 feet. In addition, the bottom of the footing should be a minimum of 10 feet from the design zone of saturation.

- The infiltration system shall be designed to overflow to the street in case the drainage capacity is exceeded.

- The excavated shaft should be observed by the soils engineer to verify natural alluvium is exposed prior to construction of the infiltration system.

Byer Geotechnical should be provided with the design plans to verify the location of the infiltration system and to provide additional recommendations, if necessary, depending on the type of the infiltration system to be installed.

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 should observe bottoms for fill, compaction of fill, temporary excavations, soldier pile excavations, lagging and slurry backfill, 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 engineer can also prepare a report summarizing soldier pile excavations, if requested.

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. 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.

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Staff Engineer

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S:\FINAL\BG\23185_Lankershim\23185_Lankershim Crossing_Geotechnical_Engineering Exploration_3.30.20.wpd

ENCLOSURES AND DISTRIBUTION

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Appendix I - Twining Consulting, excerpts from report dated June 15, 2017
- Log of Borings TB1 - TB5 (9 Pages)
- CPT Sounding Charts 1 - 3 (6 Pages)
- Direct Shear Test Diagrams (3 Pages)
- Consolidation Test Curves (2 Pages)
- Percolation Testing Summary
- Exploration Location Map

Appendix II - Laboratory Testing and Log of Borings (Current Study)
- Laboratory Testing (2 Pages)
- Shear Test Diagrams (2 Pages)
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Appendix III - Calculations and Figures
- Seismic Sources (2 Pages)
- Seismic Hazard Deaggregation Chart
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- Aerial Vicinity Map
- Regional Topographic Map
- Historic Topographic Map
- Regional Geologic Map #1 and #2 (2 Pages)
- Regional Fault Map
- Seismic Hazard Zones Map
- Historic-High Groundwater Map

In Pocket: Site Plan
Sections A through E (2 Sheets)

xc: (4) Addressee (E-mail and Mail)
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Software

EZ-FRISK 7.65, Risk Engineering, Inc.

APPENDIX I

Twining Consulting, excerpts from report dated June 15, 2017

DATE DRILLED	7/5/2016	LOGGED BY	DH	BORING NO.	B-1
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	807 ±(MSL)

(TB1)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									Asphalt Concrete: 4 inches
		Bulk		9.6	106.5			SM	FILL: Silty SAND, brown to gray-brown, dry to most, some gravel
						MAX, RDS		SP-SM	ALLUVIUM: Poorly graded SAND with silt, gray-brown, moist, medium dense, medium- to- coarse-grained sand, some gravel
802	5	Driven	16	2.0	104.8	CONSOL			-- dry, some gravel and cobble
797	10	Driven	20						
792	15	Driven	53	2.0	102.1				-- dense
									-- gravel layer encountered (18'-20')
787	20	Driven	25						-- medium dense
									-- gravel layer encountered (22'-23')
782	25	Driven	27	1.6	112.8				-- light brown, medium- to- coarse-grained sand, some gravel
777	30	Driven	31						-- dense
772	35								

TWINING

LOG OF BORING

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	
160511.1	August 2016	FIGURE A - 2

DATE DRILLED	7/5/2016	LOGGED BY	DH	BORING NO.	B-1
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	807 ±(MSL)

(TB1)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
				41					SP-SM	ALLUVIUM: Poorly graded SAND with silt, gray-brown, moist, medium dense, medium- to- coarse-grained sand, some gravel *(continued)*
767	40			38						
762	45			44						-- same
757	50			47						
										Total Depth = 51.5 feet Backfilled on 7/5/2016 Borehole backfilled with soil from cuttings.
752	55									
747	60									
742	65									
737	70									

TWINING

LOG OF BORING

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	
160511.1	August 2016	FIGURE A - 2

DATE DRILLED	7/5/2016	LOGGED BY	DH	BORING NO.	B-2
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	806 ±(MSL)

(TB2)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	U.S.C.S. CLASSIFICATION	DESCRIPTION
							SM	Asphalt Concrete: 2 inches FILL: Silty SAND, brown to gray-brown, moist, medium dense, fine- to-medium-grained sand
							SP-SM	ALLUVIUM: Poorly graded SAND with silt, gray-brown, moist, medium dense, medium-to- coarse-grained sand, some gravel
801	5		13	1.7	107.6	#200		
796	10		27	1.5	90.2	DS		-- dry
791	15		32	1.2	109.1	#200		-- dense, gravel and cobble encountered
786	20		58	3.1	111.9			-- gravelly
781	25		29	2.9	112.0	#200		-- light gray-brown, medium dense
776	30		36					-- dense
771	35							

TWINING

LOG OF BORING

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	FIGURE A - 3
160511.1	August 2016	

BORING LOG 160511.1 - LANKERSHIM.GPJ TWINING LABS.GDT 8/12/16

DATE DRILLED	7/5/2016	LOGGED BY	DH	BORING NO.	B-2
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	806 ±(MSL)

(TB2)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
				34			#200		SP-SM	ALLUVIUM: Poorly graded SAND with silt, gray-brown, moist, medium dense, medium-to- coarse-grained sand, some gravel *(continued)*
766	40			58						-- damp
761	45			37						
756	50			50/3"						-- very dense
										Total Depth = 50.8 feet Backfilled on 7/5/2016 Borehole backfilled with soil from cuttings.
751	55									
746	60									
741	65									
736	70									

TWINING

LOG OF BORING

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	
160511.1	August 2016	FIGURE A - 3

BORING LOG 160511.1 - LANKERSHIM.GPJ TWINING LABS.GDT 8/12/16

DATE DRILLED	7/5/2016	LOGGED BY	DH	BORING NO.	B-3
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	808 ±(MSL)

(TB3)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SM	Asphalt Concrete: 2 inches FILL: Silty SAND, brown to light brown, dry to slightly moist, some gravel
								SP-SM	ALLUVIUM: Poorly graded SAND with silt, gray-brown, moist, medium dense
803	5		24	1.7	115.9	CORR			-- damp, gravel and cobble encountered
798	10		27						
793	15		30	2.0	110.0	DS			-- increase in gravel
788	20		39						-- dense -- less gravel
783	25		57	2.7	116.9				-- light gray-brown
778	30		33						-- medium dense
									Total Depth = 31.5 feet Backfilled on 7/5/2016 Borehole backfilled with soil from cuttings.
773	35								

TWINING

LOG OF BORING

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	FIGURE A - 4
160511.1	August 2016	

DATE DRILLED	7/6/2016	LOGGED BY	DH	BORING NO.	B-4
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	809 ±(MSL)

(TB4)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									SM	Asphalt Concrete: 2 inches FILL: Silty SAND with gravel, light brown/tan, dry, medium dense, fine-to- medium-grained sand
									SP-SM	ALLUVIUM: Poorly graded SAND, light brown/tan, dry, medium dense, medium-grained sand, some gravel
804	5			27	1.2	100.5				
799	10			21	2.5	116.7	CONSOL			-- dry, medium- to- coarse-grained sand
794	15			26	2.4	116.2				
789	20			43	3.3	110.2				-- damp, dense, gravel and cobble layer encountered (20-20.5')
784	25			29	2.7	114.6				-- increase in gravel, moist
779	30			67						-- very dense, gravelly
774	35									

TWINING

LOG OF BORING

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	
160511.1	August 2016	FIGURE A - 5

DATE DRILLED	7/6/2016	LOGGED BY	DH	**BORING NO.**	**B-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	809 ±(MSL)

(TB4)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
				44					SP-SM	ALLUVIUM: Poorly graded SAND, light brown/tan, dry, medium dense, medium-grained sand, some gravel *(continued)* -- dense
769	40			67						-- very dense
764	45			29						-- medium dense
759	50			87/9"						-- very dense
										Total Depth = 51.3 feet Backfilled on 7/6/2016 Borehole backfilled with soil from cuttings.
754	55									
749	60									
744	65									
739	70									

TWINING

LOG OF BORING

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	
160511.1	August 2016	FIGURE A - 5

DATE DRILLED	7/6/2016	LOGGED BY	DH	BORING NO.	B-5
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	808 ±(MSL)

(TB5)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
							RV		SM	Asphalt Concrete: 2 inches FILL: Silty SAND, dark brown to light brown, dry to moist, loose, fine-to- medium-grained, some gravel
									SP-SM	ALLUVIUM: Poorly graded SAND, light brown/tan, moist, medium dense, medium-to- coarse -grained sand, some gravel
803	5			17						
798	10			18	2.7	110.7				
										-- gravel and cobble encountered (12'-13')
793	15			41	2.3	110.2				-- damp, dense
788	20			18	2.5	111.3				-- medium dense, cobble encountered (20'-20.5')
783	25			25						-- increase in gravel, moist
778	30			34						
773	35									

BORING LOG 160511.1 - LANKERSHIM.GPJ TWINING LABS.GDT 8/12/16

TWINING

LOG OF BORING

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	
160511.1	August 2016	FIGURE A - 6

DATE DRILLED	7/6/2016	LOGGED BY	DH	BORING NO.	B-5
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	808 ±(MSL)

(TB5)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
			X	35					SP-SM	ALLUVIUM: Poorly graded SAND, light brown/tan, moist, medium dense, medium-to- coarse -grained sand, some gravel *(continued)* -- dense
768	40		X	32						-- medium-grained
763	45		X	32						
758	50		X	44						
										Total Depth = 51.5 feet Backfilled on 7/6/2016 Borehole backfilled with soil from cuttings.
753	55									
748	60									
743	65									
738	70									

TWINING

LOG OF BORING

Lankershim Square
North Hollywood, California

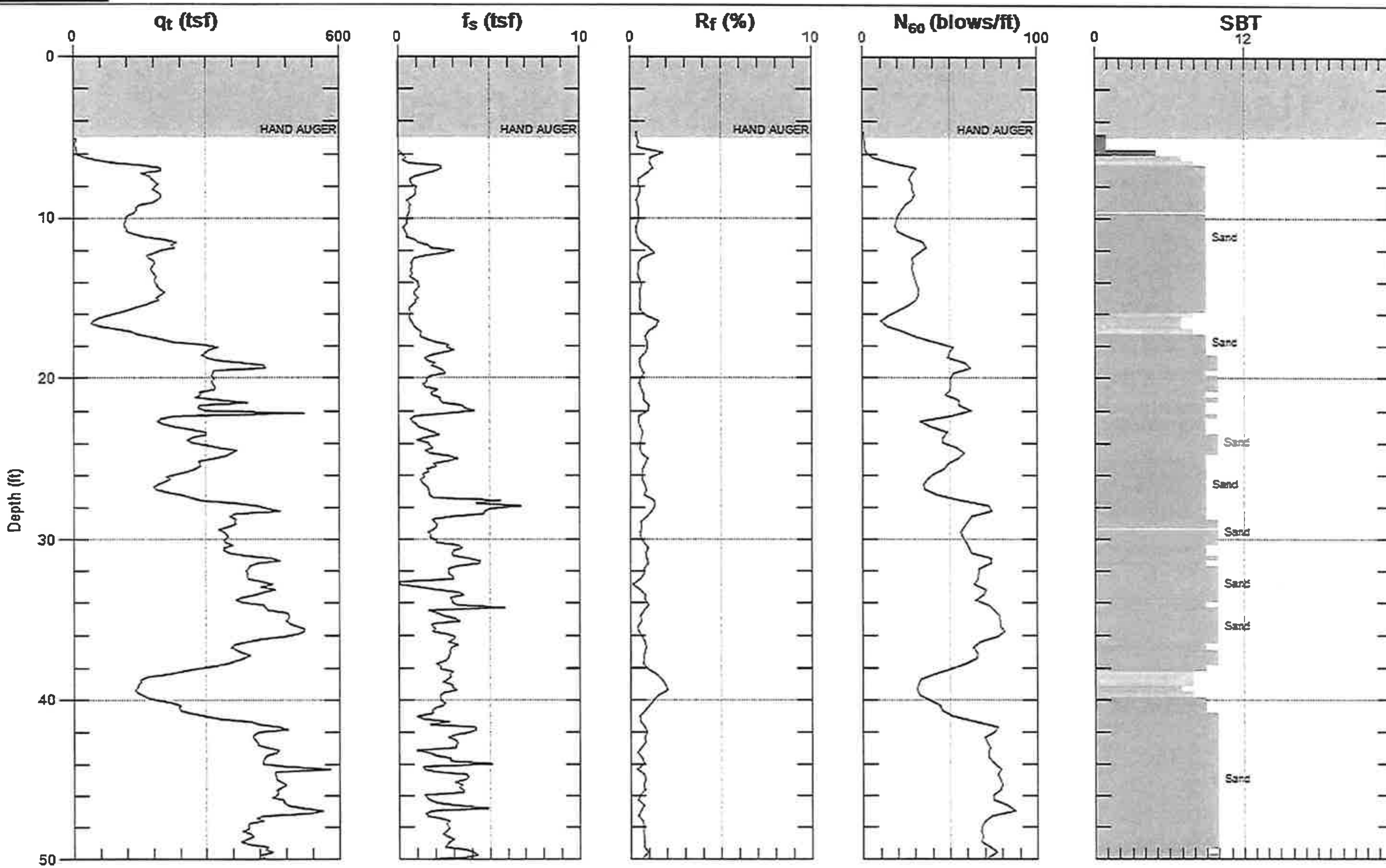
PROJECT NO.	REPORT DATE	FIGURE A - 6
160511.1	August 2016	

GREGG

TWINING CONSULTANTS

Site: NURSERY @7934 LANKERSHIM | Engineer: S.LIN

Sounding: CPT-1 | Date: 7/5/2016 08:28



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

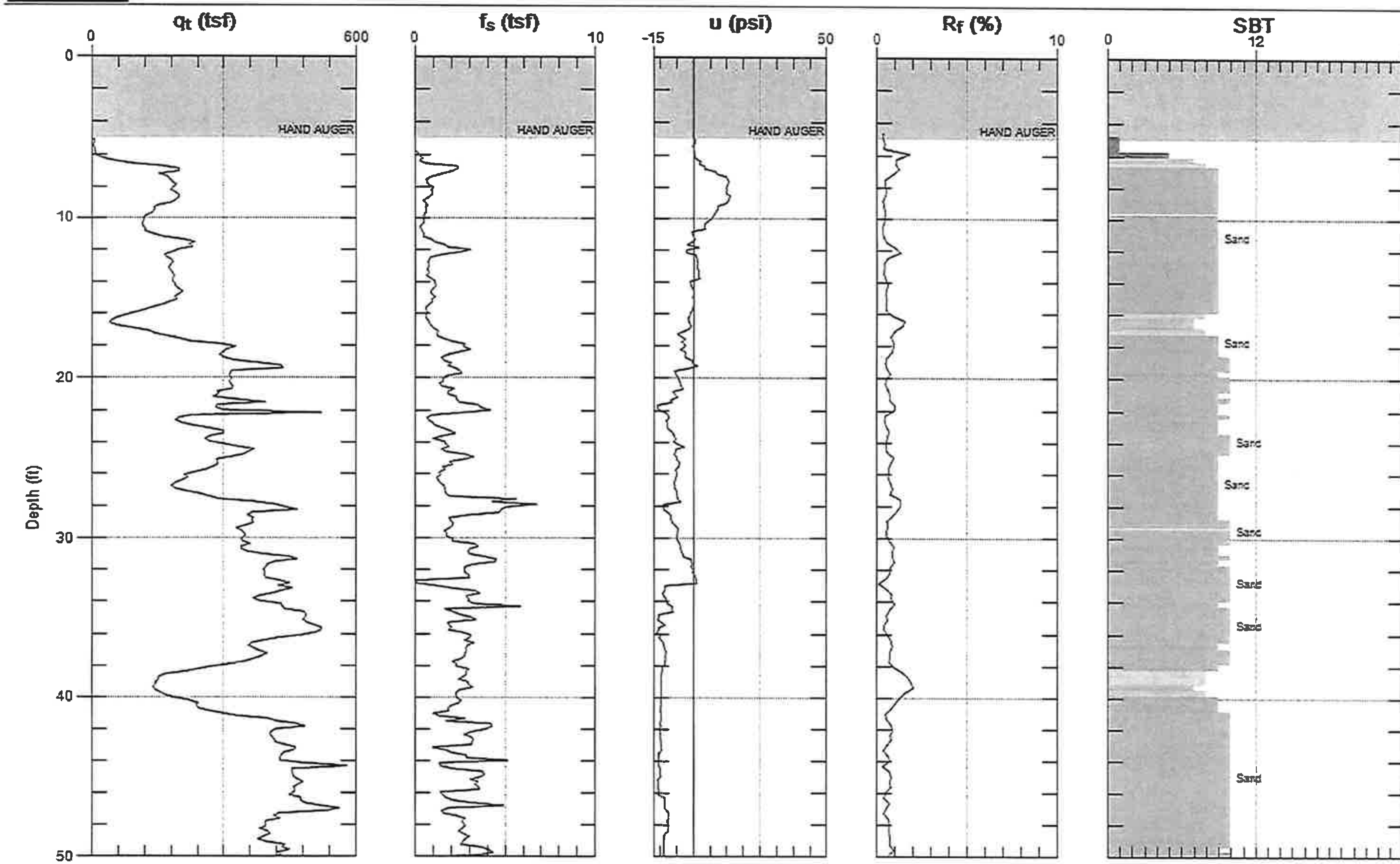
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TWINING CONSULTANTS

Site: NURSERY @7934 LANKE[illegible] Engineer: S.LIN

Sounding: CPT-1 Date: 7/5/2016 08:28

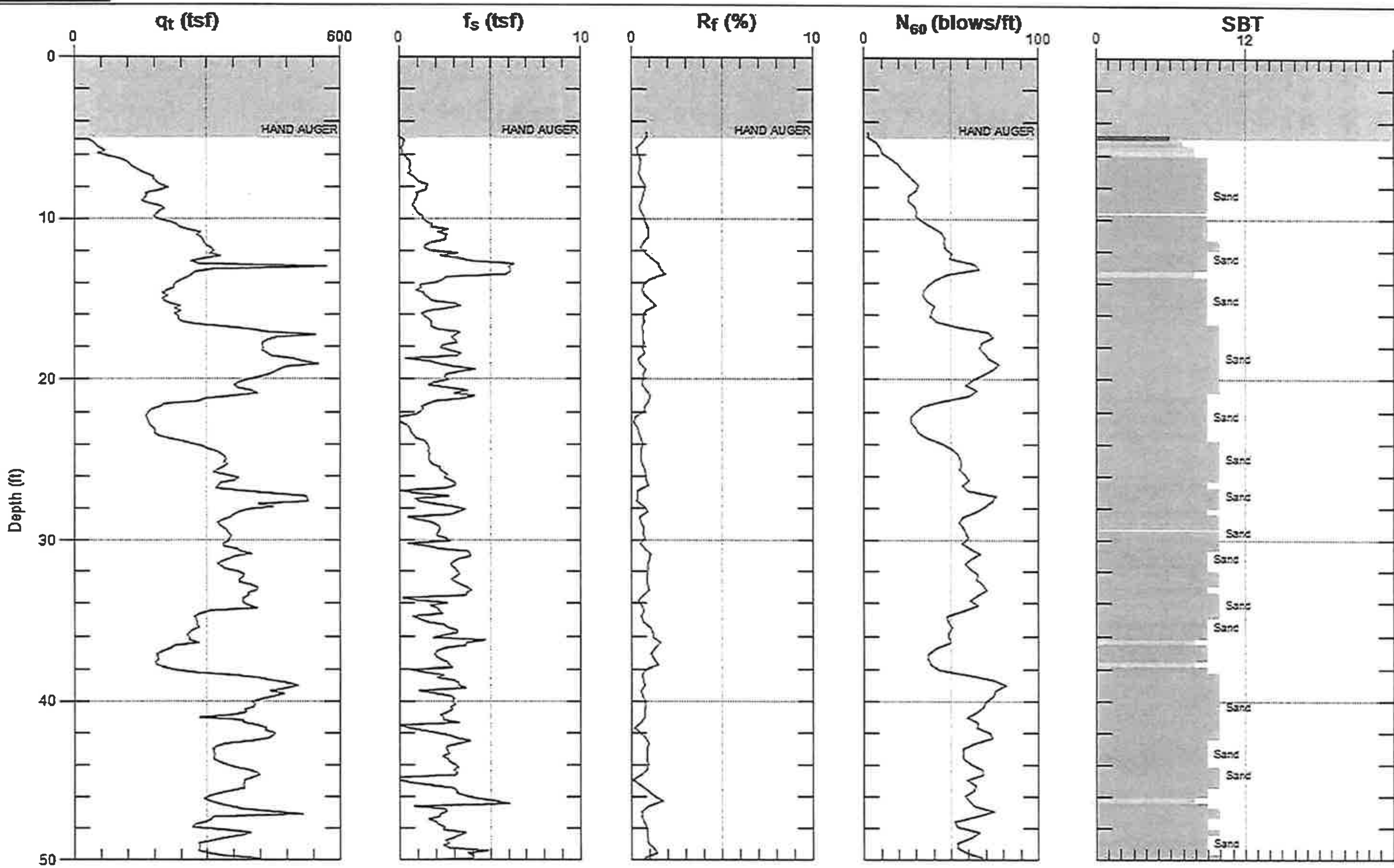


Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TWINING CONSULTANTS**

Site: NURSERY @7934 LANKERSHIM Engineer: S.LIN
Sounding: CPT-2 Date: 7/5/2016 10:44



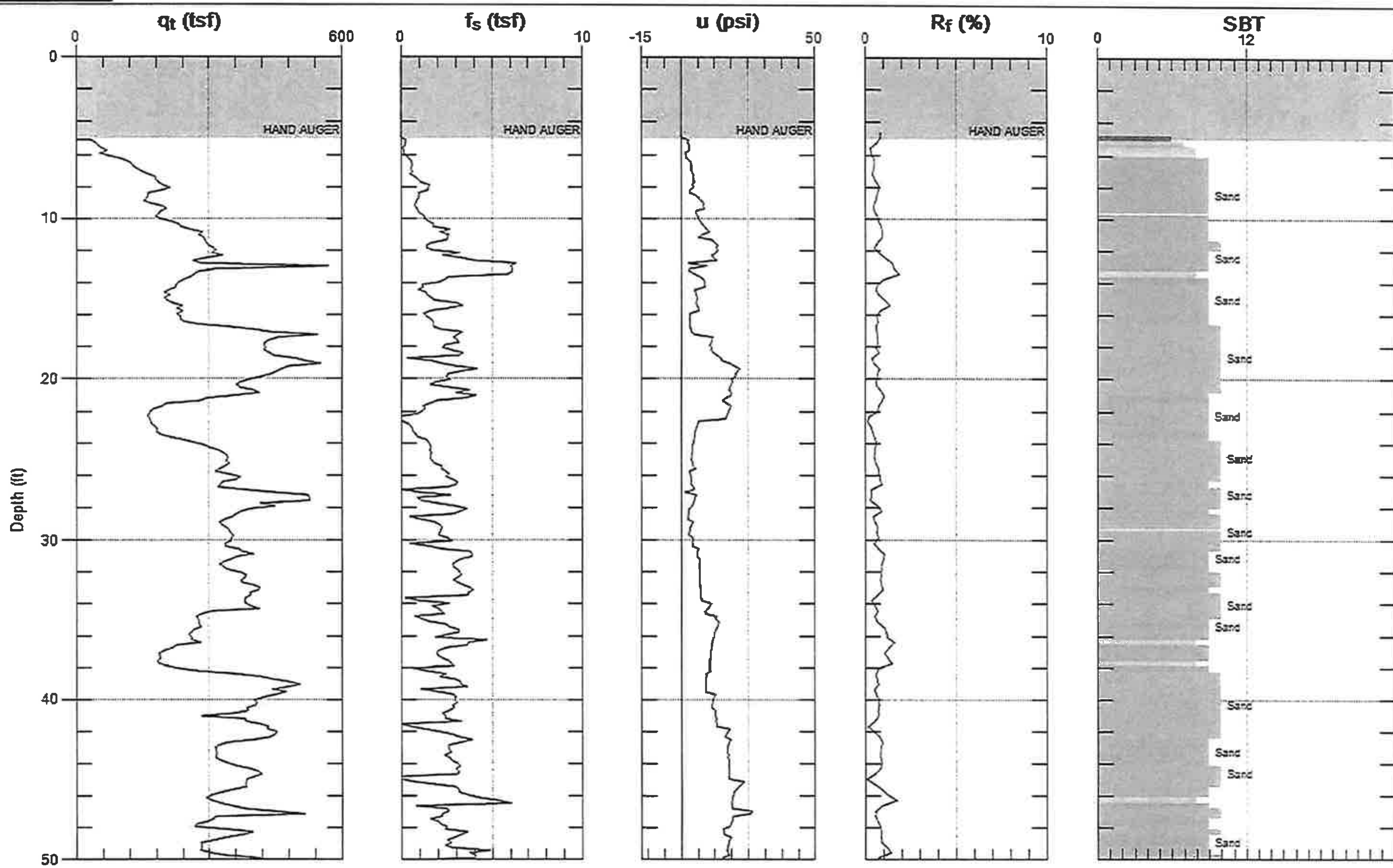
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TWINING CONSULTANTS**

Site: NURSERY @7934 LANKER **Engineer: S.LIN**

Sounding: CPT-2 **Date: 7/5/2016 10:44**



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

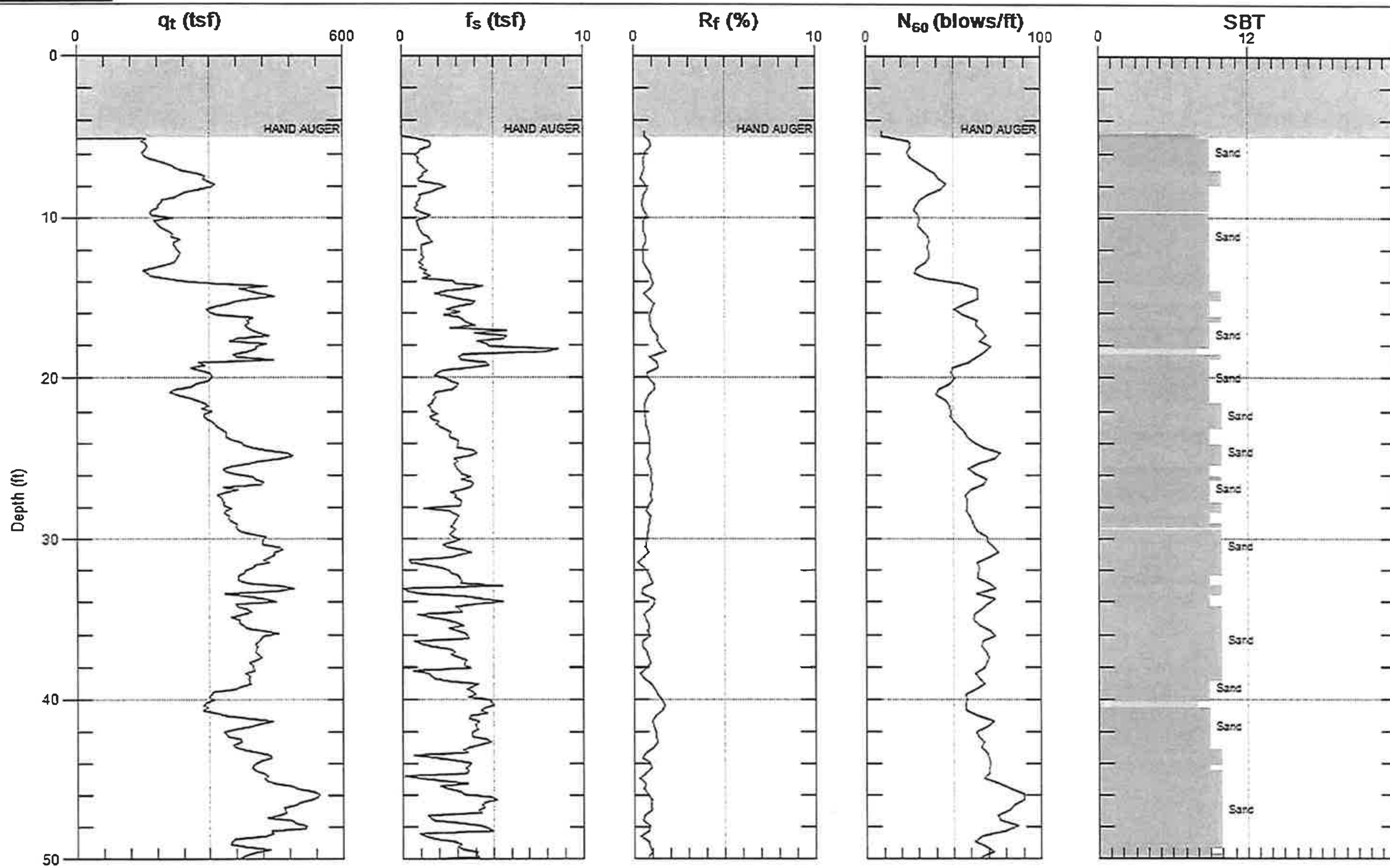
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TWINING CONSULTANTS

Site: NURSERY @7934 LANKERSHIM Engineer: S.LIN

Sounding: CPT-3 Date: 7/5/2016 09:40



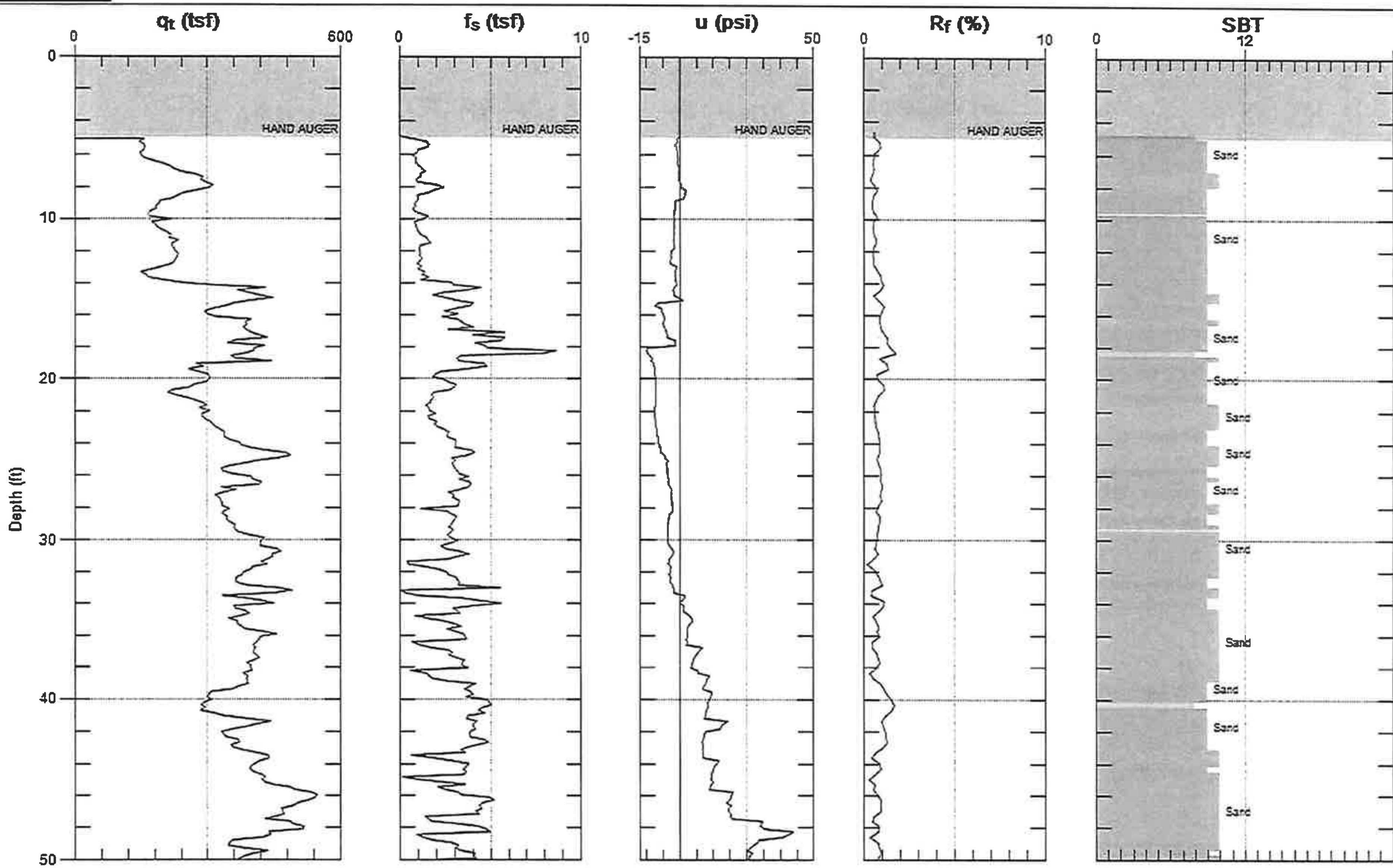
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TWINING CONSULTANTS**

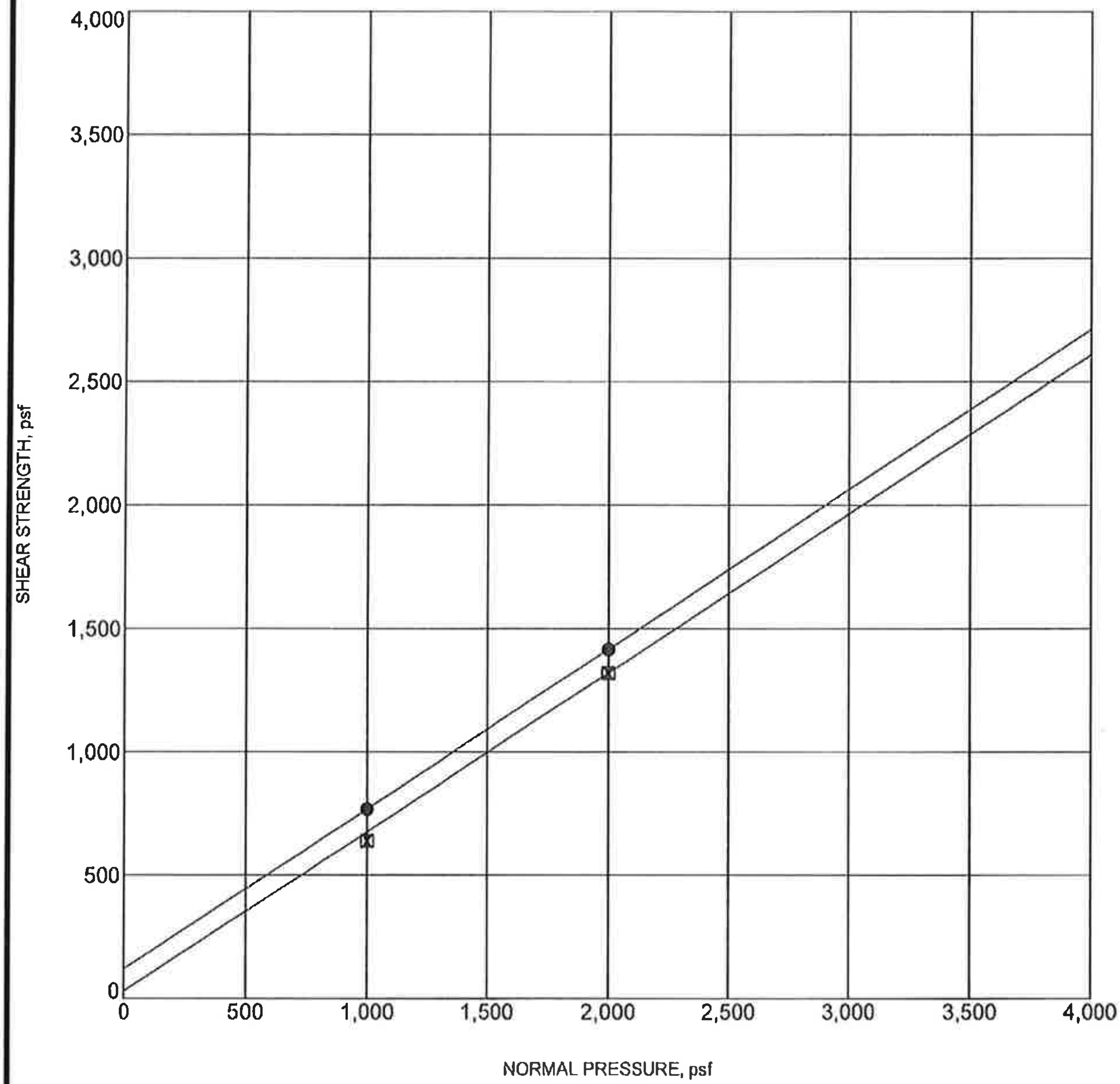
Site: NURSERY @7934 LANKERSHIM Engineer: S.LIN

Sounding: CPT-3 Date: 7/5/2016 09:40



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



Shear Strength Parameters

		Peak	Ultimate
Boring No.:	B-1		
Sample Depth (ft):	1-4		
Sample Description:	Poorly graded SAND with silt		
Strain Rate (in./min):	0.005		
Dry Density (pcf):	106.5		
Cohesion, C (psf):		120	0
Friction Angle, Ø (deg):		33	33
Initial Moisture (%):		9.6	
Final Moisture (%):		15.6	

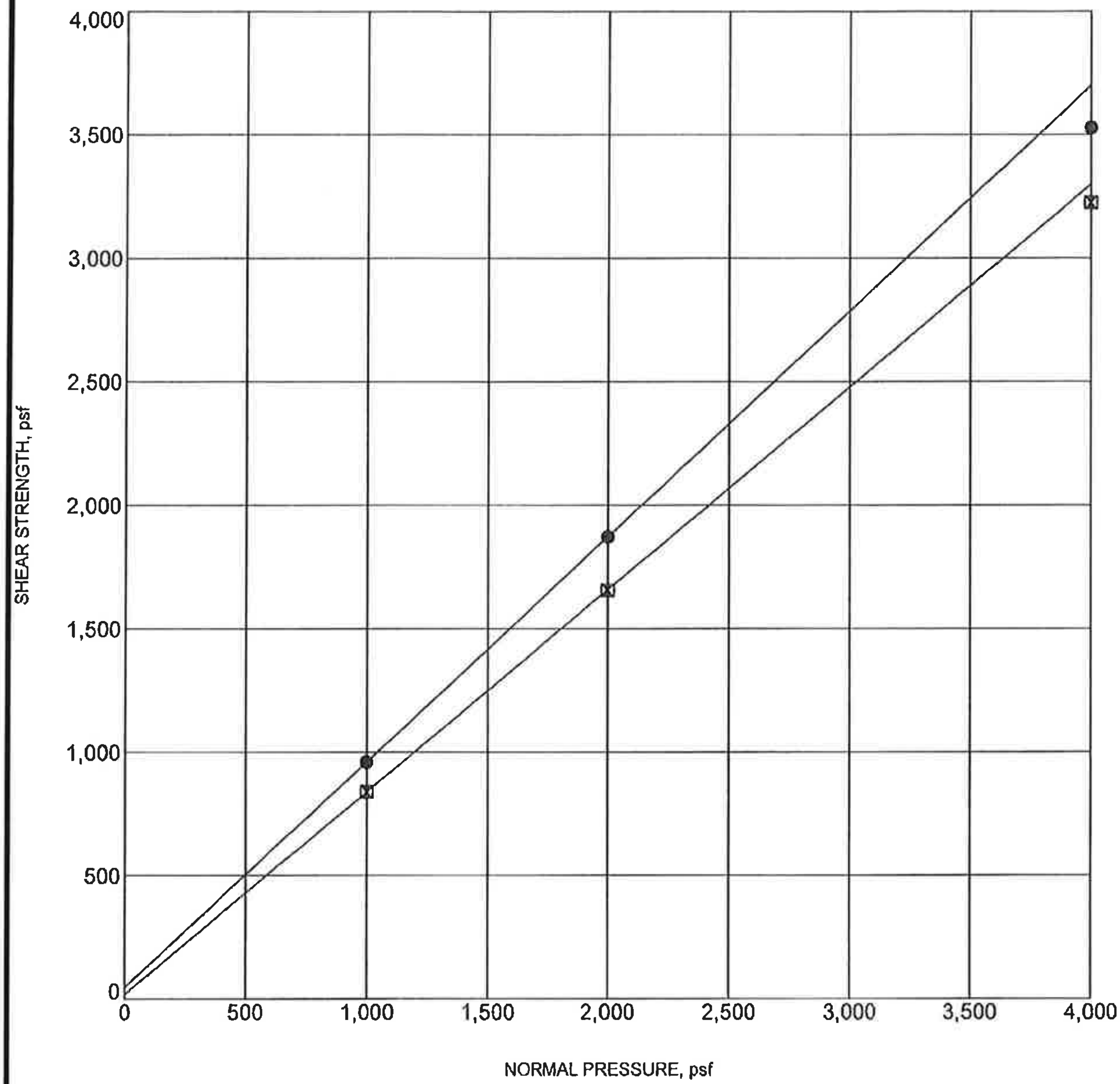
DIRECT SHEAR 160511.1 - LANKERSHIM.GPJ TWINING LABS.GDT 7/27/16

TWINING

REMOLDED DIRECT SHEAR TEST

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	FIGURE B-2
160511.1	August 2016	



Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	40	20
Friction Angle, Ø (deg):	42	40

Boring No.: B-2
Sample Depth (ft): 10
Sample Description: Poorly graded SAND with silt
Strain Rate (in./min): 0.005
Dry Density (pcf): 90.2

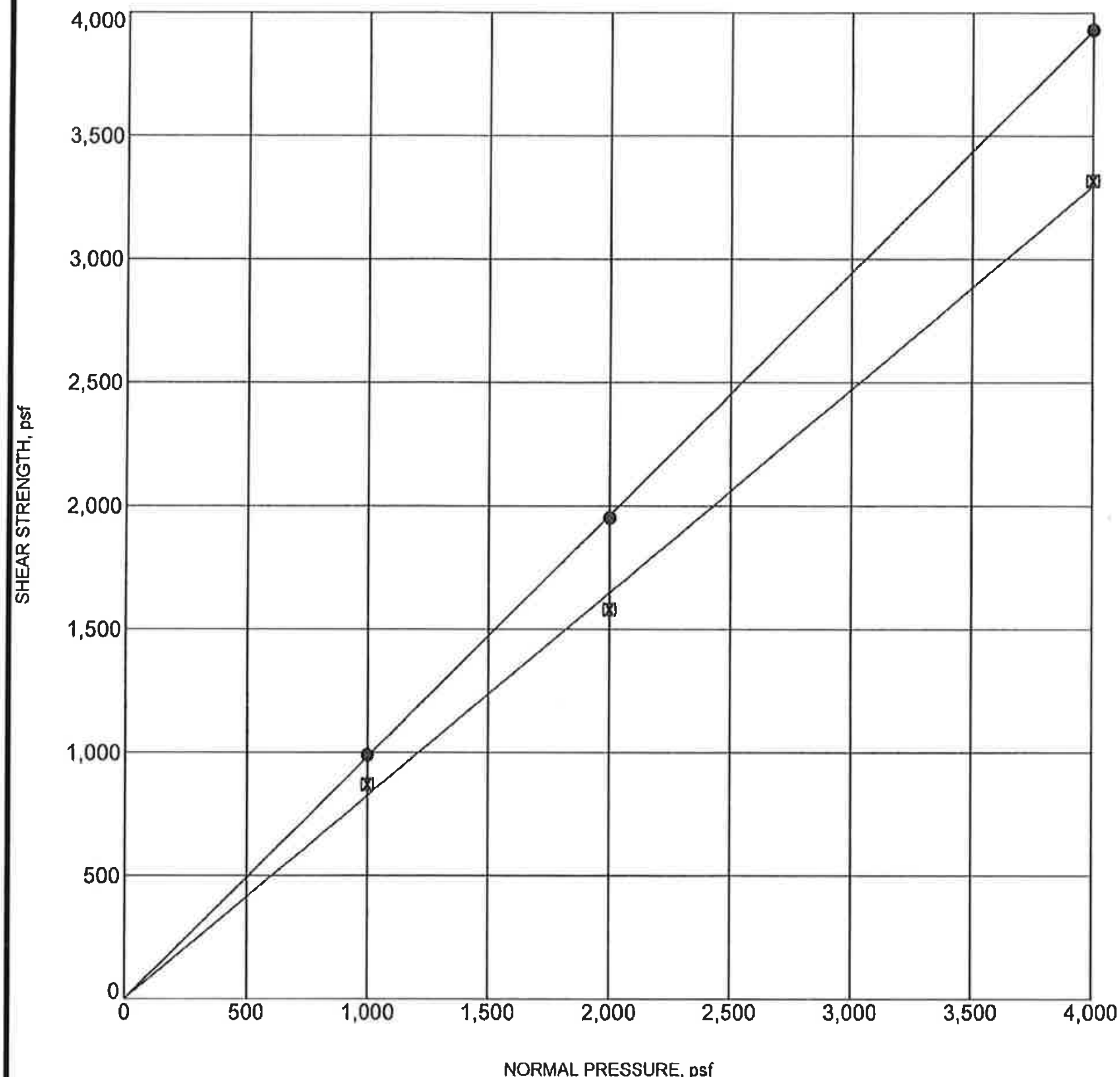
Initial Moisture (%): 1.5
Final Moisture (%): 19.3

TWINING

DIRECT SHEAR TEST

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	FIGURE B-3
160511.1	August 2016	



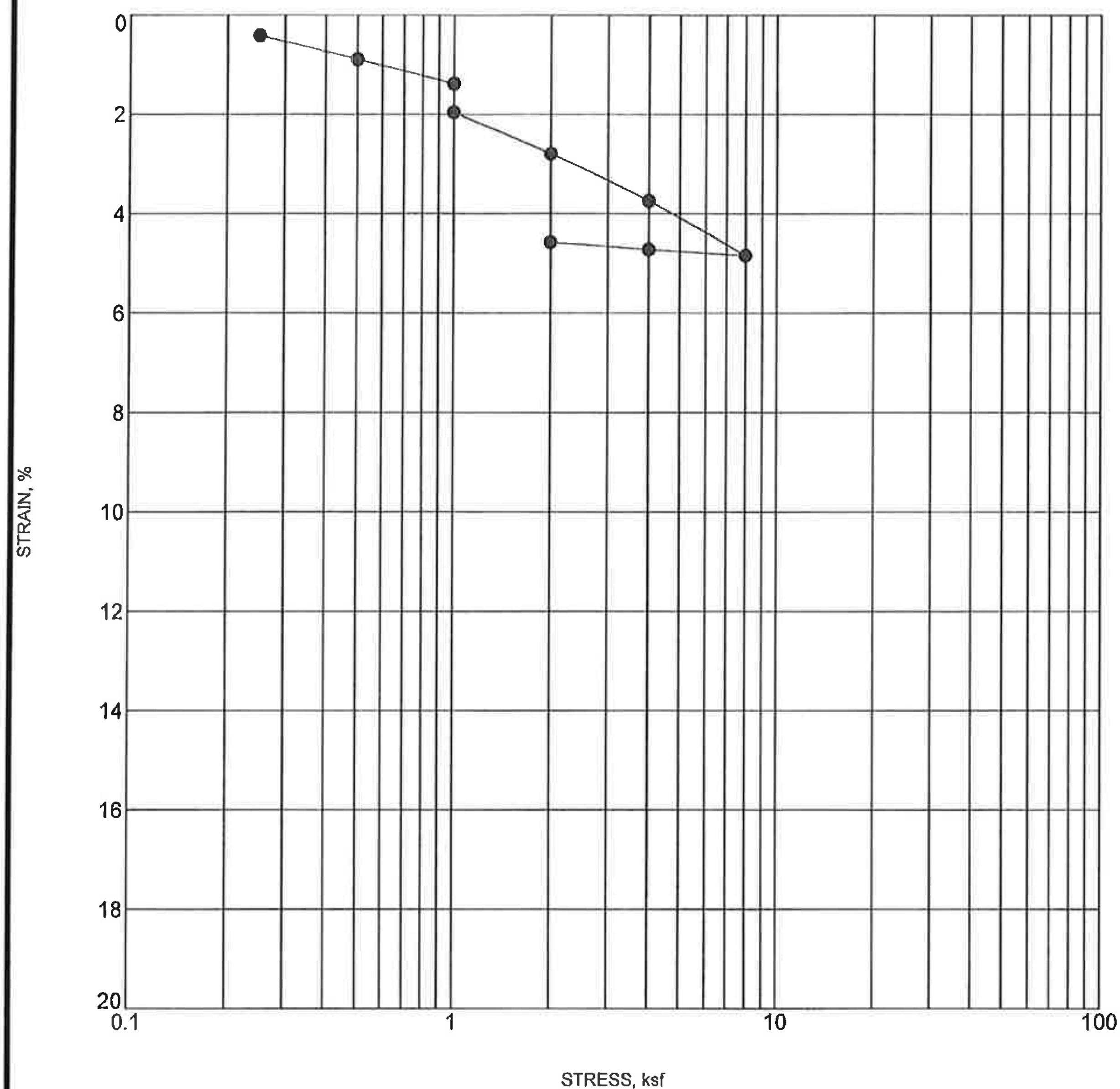
			Shear Strength Parameters	
			Peak	**Ultimate**
Boring No.:	B-3			
Sample Depth (ft):	15	**Cohesion, C (psf):**	0	0
Sample Description:	Poorly graded SAND with silt	**Friction Angle, Ø (deg):**	44	40
Strain Rate (in./min):	0.005			
Dry Density (pcf):	110.0	**Initial Moisture (%):**	2.0	
		Final Moisture (%):	19.1	

TWINING

DIRECT SHEAR TEST

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	FIGURE B-4
160511.1	August 2016	



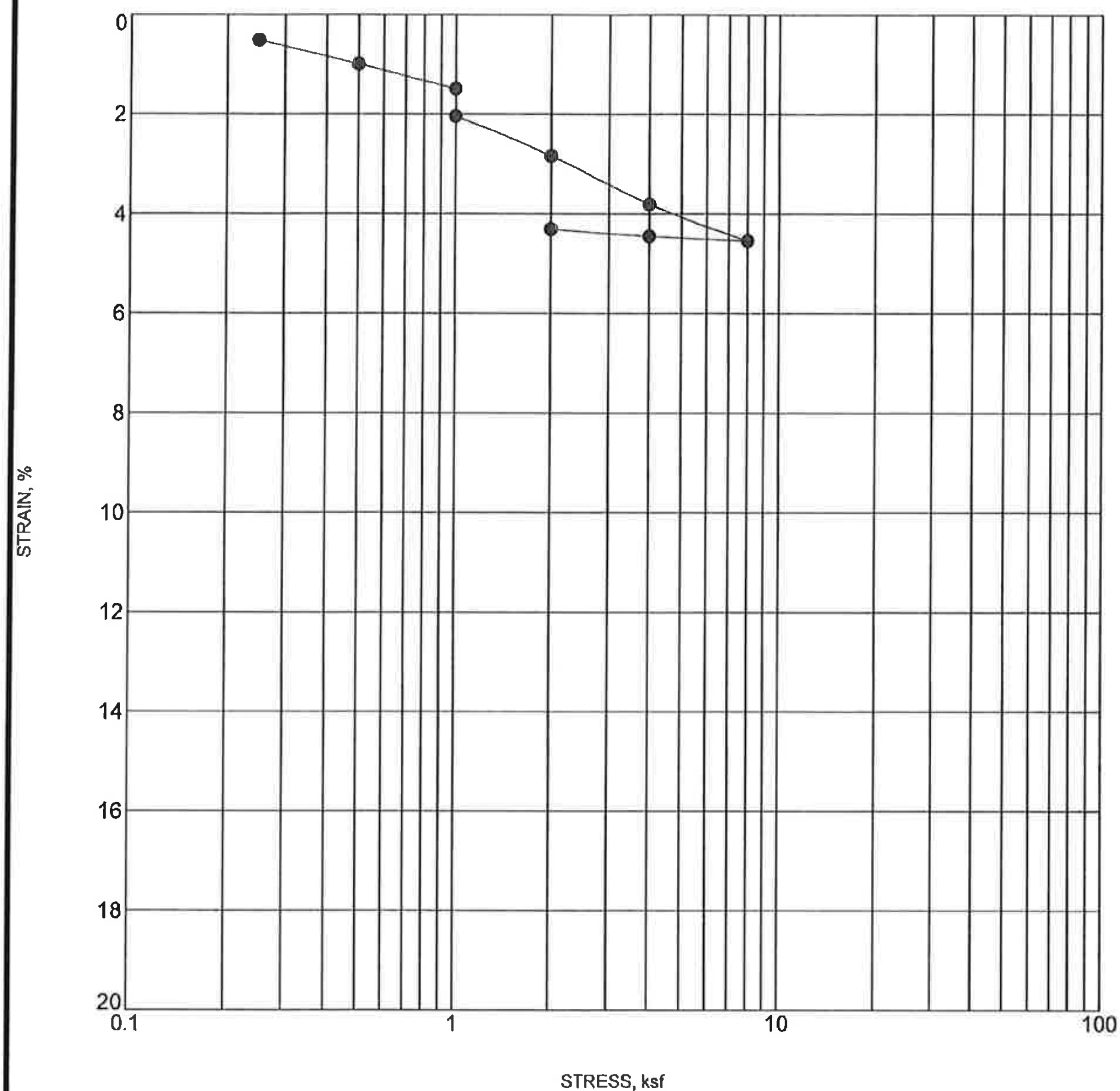
	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-1 at 5 ft**	**Poorly graded SAND with silt and gravel**	**102.5**	**2.0**

TWINING

CONSOLIDATION TEST

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	
160511.1	August 2016	FIGURE B-5



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-4 at 10 ft**	**Poorly graded SAND**	**98.3**	**1.2**

TWINING

CONSOLIDATION TEST

Lankershim Square
North Hollywood, California

PROJECT NO.	REPORT DATE	FIGURE B-6
160511.1	August 2016	

Appendix C
Percolation Testing

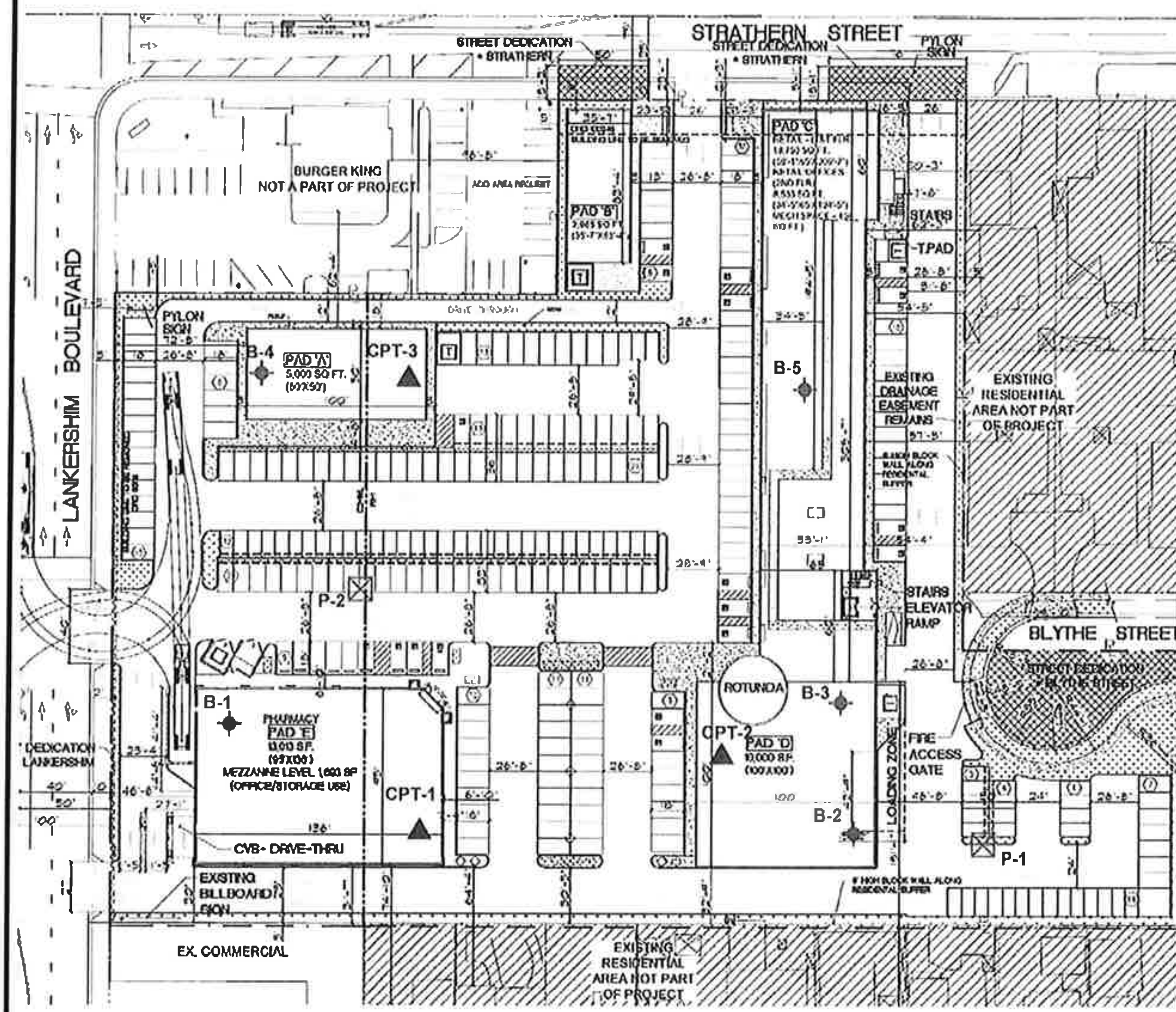
Two percolation test borings were excavated at the project site as shown on Figure 2 – Boring and CPT Location Map. One boring was advanced to approximately 4 feet below the existing ground surface and the second boring was advanced to approximately 9 feet below the existing ground surface. Percolation testing was performed in a 8-inch diameter soil borings on July 5 and 6, 2016 in general conformance with the County of Los Angeles requirements.

The purpose of the tests was to evaluate the infiltration rates of the subgrade soils. Approximately 2 inches of coarse gravel was placed at the bottoms of the boreholes to prevent scouring during testing. A 2-inch diameter slotted screening PVC pipe was inserted in the borehole and coarse gravel was used to fill around the pipes. The boreholes were presoaked prior to testing. After the completion of presoaking, the borings were filled with water at a minimum depth of 12 Inches above the bottom of excavation. Measurements were taken at approximate10-minute intervals and then refill to same water depth for a total of 8 readings. A summary of results is presented in Table C-1 and the detailed data is attached.

Upon completion of the borings and testing, the boreholes were backfilled with soil from the cuttings as noted in the Log of Boring.

Table C-1 - Summary of Percolation Test Results

Test Location	Depth of Test Hole (in.)	Design Infiltration Rate (in/hr)
P-1	108	**6.34**
P-2	48	**6.39**



LEGEND

CPT-1 ▲ Approximate CPT location and number

B-1 ⌖ Approximate boring location and number

P-1 ☒ Approximate percolation location and number

N

Reference: CCA Architect (2008)

TWINING CONSULTING

EXPLORATION LOCATION MAP

Lankershim Square
7934 Lankershim Boulevard
Hollywood, California

PROJECT NO. 160511.1	REPORT DATE August 2016	FIGURE 2

APPENDIX II

Laboratory Testing and Log of Borings (Current Study)

APPENDIX II

LABORATORY TESTING

Undisturbed and bulk samples of 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. Remolded samples were prepared at 95 percent of the maximum dry density. The remolded samples were tested for shear strength.

Boring	Depth (Feet)	Earth Material	Soil Type and Color	Maximum Density (pcf)	Optimum Moisture %	Expansion Index
B1	0 - 10	Alluvium	Sand, Tan	127.0	11.0	7 - Very Low

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 upper ten feet of the earth materials are expected to exhibit a very low expansion potential.

APPENDIX II (Continued)

Shear Tests

Shear tests were performed on samples of the future compacted fill and 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 future compacted fill using the procedures outlined in ASTM D 2435-11. Results are graphed on the enclosed Consolidation Curves.

Corrosion

A bulk representative sample of the near-surface soils was transported to Environmental Geotechnology Laboratory for chemical testing. The testing was performed in accordance with Caltrans Standards 643 (pH), 422 (Chloride Content), 417 (Sulfate Content), and 532 (Resistivity). The results of the testing are reported in the following table:

CHEMICAL TEST RESULTS TABLE

Sample	Depth (Feet)	pH	Chloride (PPM)	Sulfate (%)	Resistivity (Ohm-cm)
BG1	0-10	8.04	125	0.001	20,000

The chloride and sulfate contents of the soil are negligible and not a factor in corrosion. The pH is near neutral and not a factor. The resistivity indicates that the soil is considered mildly corrosive to ferrous metals.

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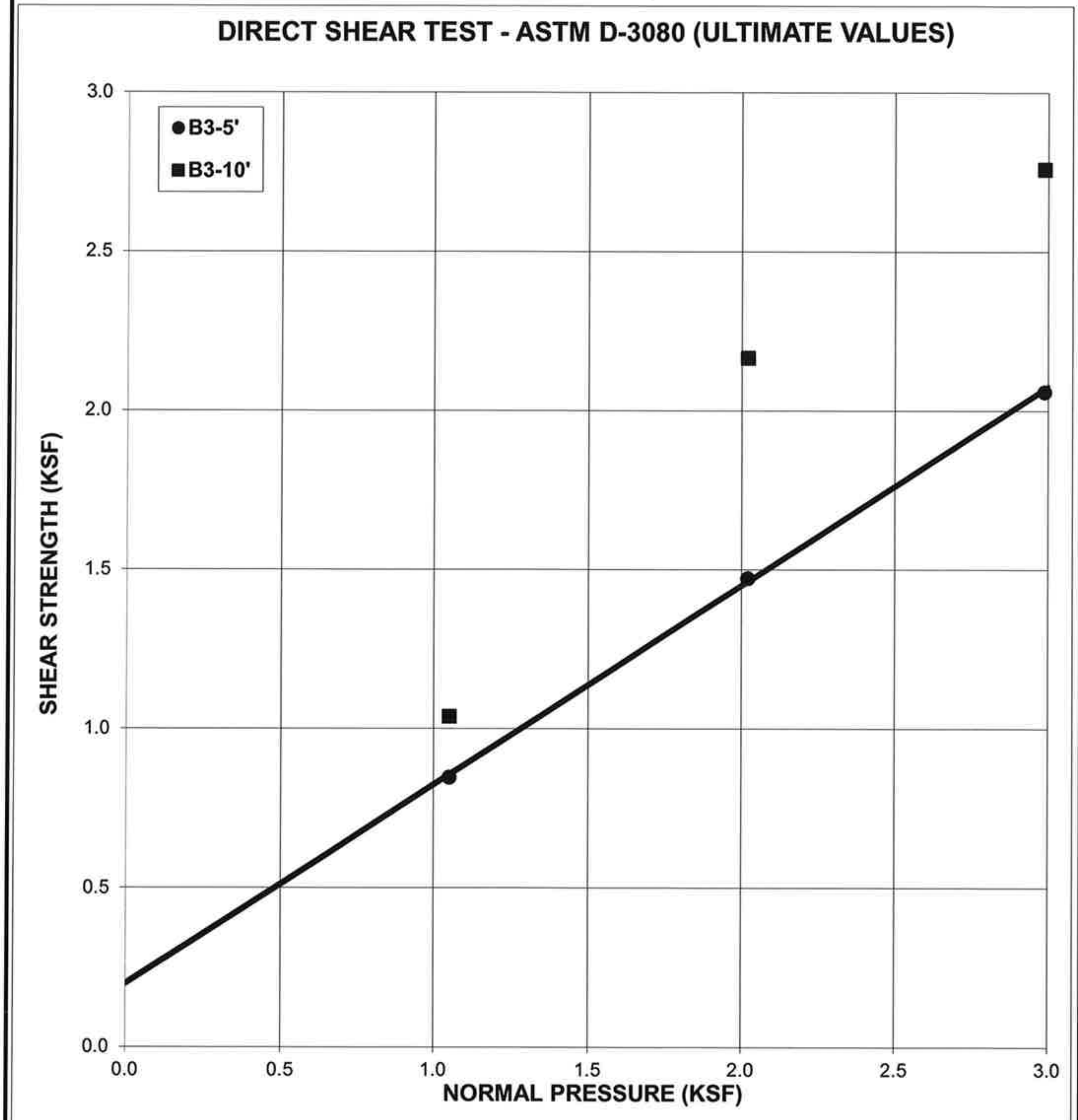
1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

SHEAR TEST DIAGRAM #1

BG: 23185 ENGINEER: JHP
CLIENT: Lankershim Crossing, LLC

EARTH MATERIAL: **Alluvium**

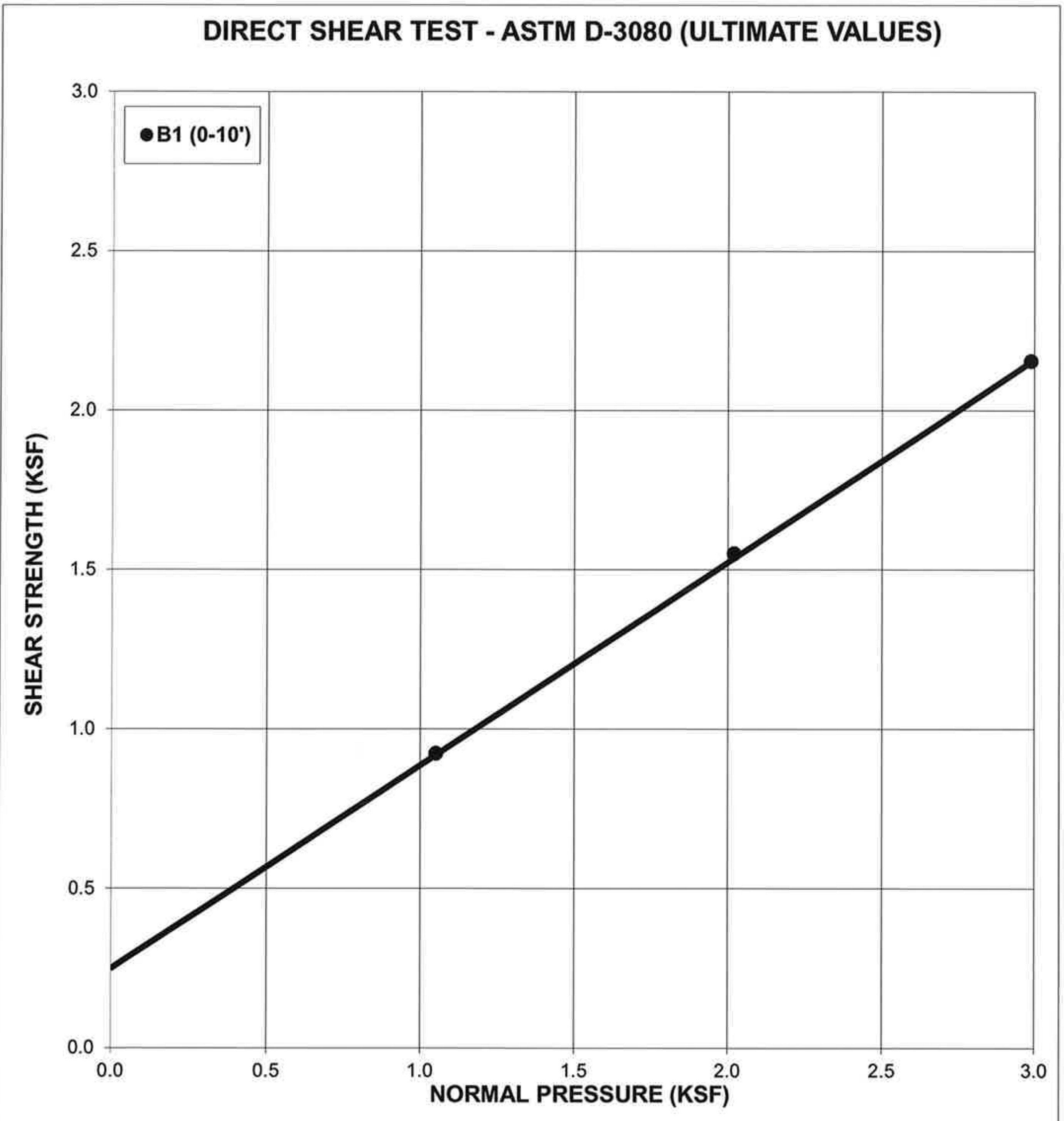
Phi Angle =	**32.0**	**degrees**	**Average Moisture Content**	**18.9%**
Cohesion =	**200**	**psf**	**Average Dry Density (pcf)**	**110.6**
			Average Saturation	**99%**



BYER GEOTECHNICAL INC.	SHEAR TEST DIAGRAM #2
1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206 tel 818.549.9959 fax 818.543.3747	BG: 23185 ENGINEER: JHP CLIENT: Lankershim Crossing, LLC EARTH MATERIAL: **Future Compacted Fill (Remolded to 95%)**

Phi Angle = 32.5 degrees
Cohesion = 250 psf

Moisture Content 14.3%
Dry Density (pcf) 120.7
Saturation 99%



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CONSOLIDATION CURVE #1

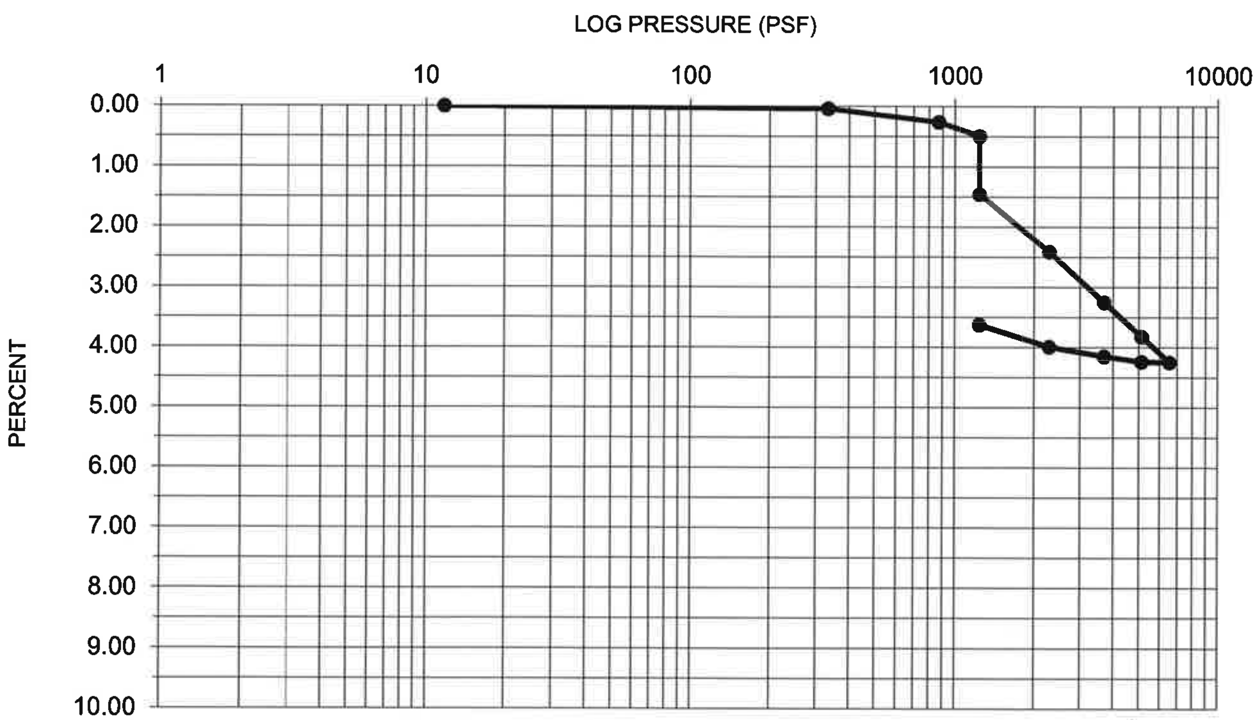
BG: **23185** ENGINEER: **JHP**

CLIENT: **Lankershim Crossing, LLC**

Earth Material:	Alluvium
Sample Location:	B2-10'
Dry Weight (pcf):	108.0
Initial Moisture:	2.2%
Initial Saturation:	11.0%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.53
Compression Index (Cc):	0.062
Recompression Index (Cr):	0.021

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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CONSOLIDATION CURVE #2

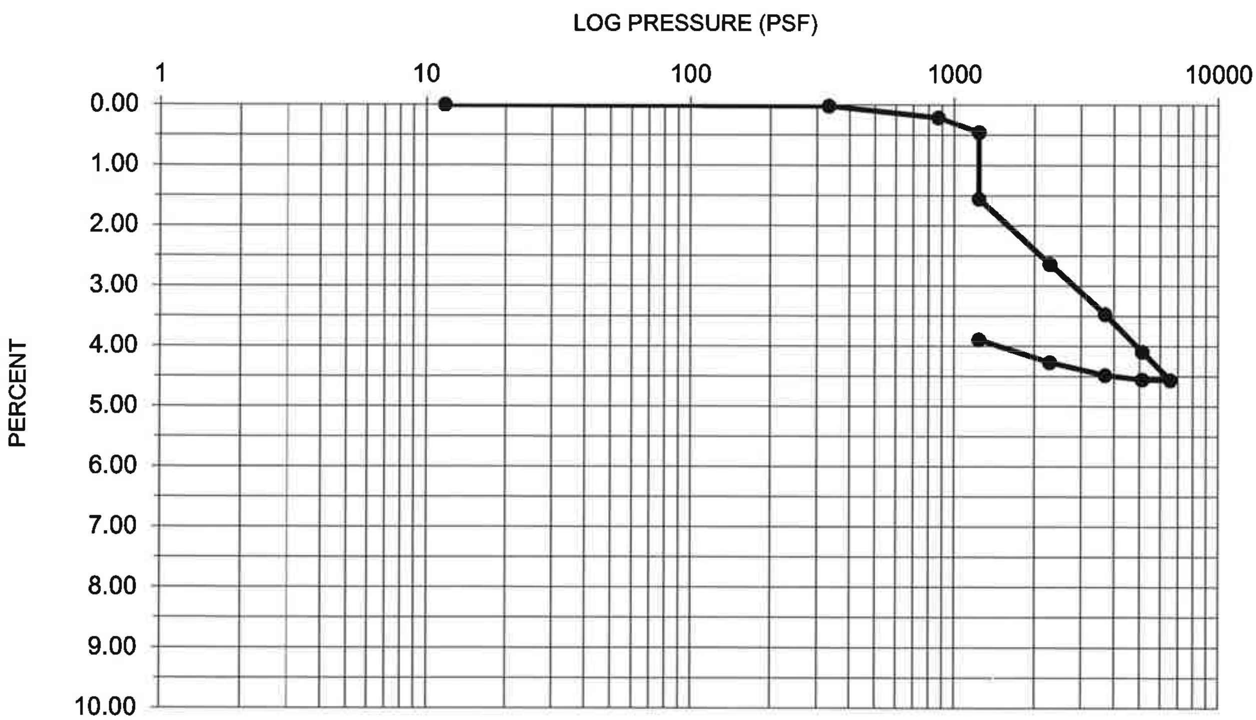
BG: 23185 ENGINEER: JHP

CLIENT: Lankershim Crossing, LLC

Earth Material:	Alluvium
Sample Location:	B1-15'
Dry Weight (pcf):	119.7
Initial Moisture:	3.3%
Initial Saturation:	22.9%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.38
Compression Index (Cc):	0.061
Recompression Index (Cr):	0.020

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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CONSOLIDATION CURVE #3

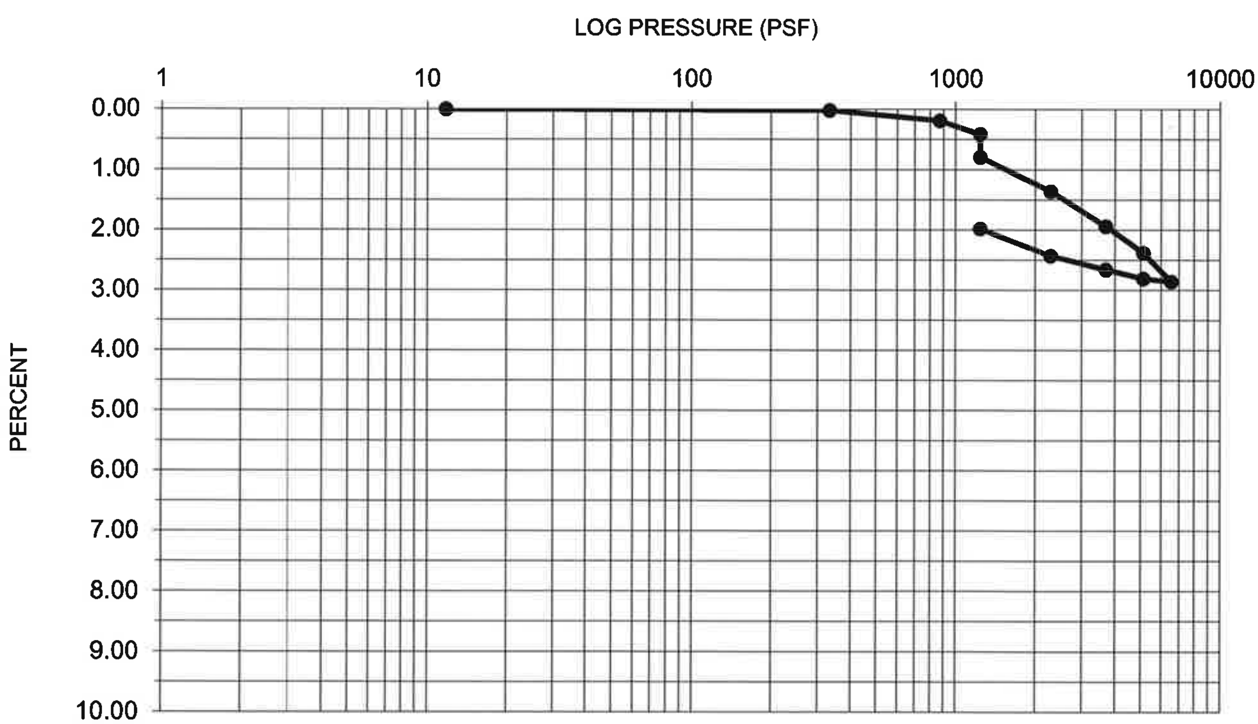
BG: **23185** ENGINEER: **JHP**

CLIENT: **Lankershim Crossing, LLC**

Earth Material:	Alluvium
Sample Location:	B3-20'
Dry Weight (pcf):	107.4
Initial Moisture:	3.8%
Initial Saturation:	18.7%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.54
Compression Index (Cc):	0.069
Recompression Index (Cr):	0.026

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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CONSOLIDATION CURVE #4

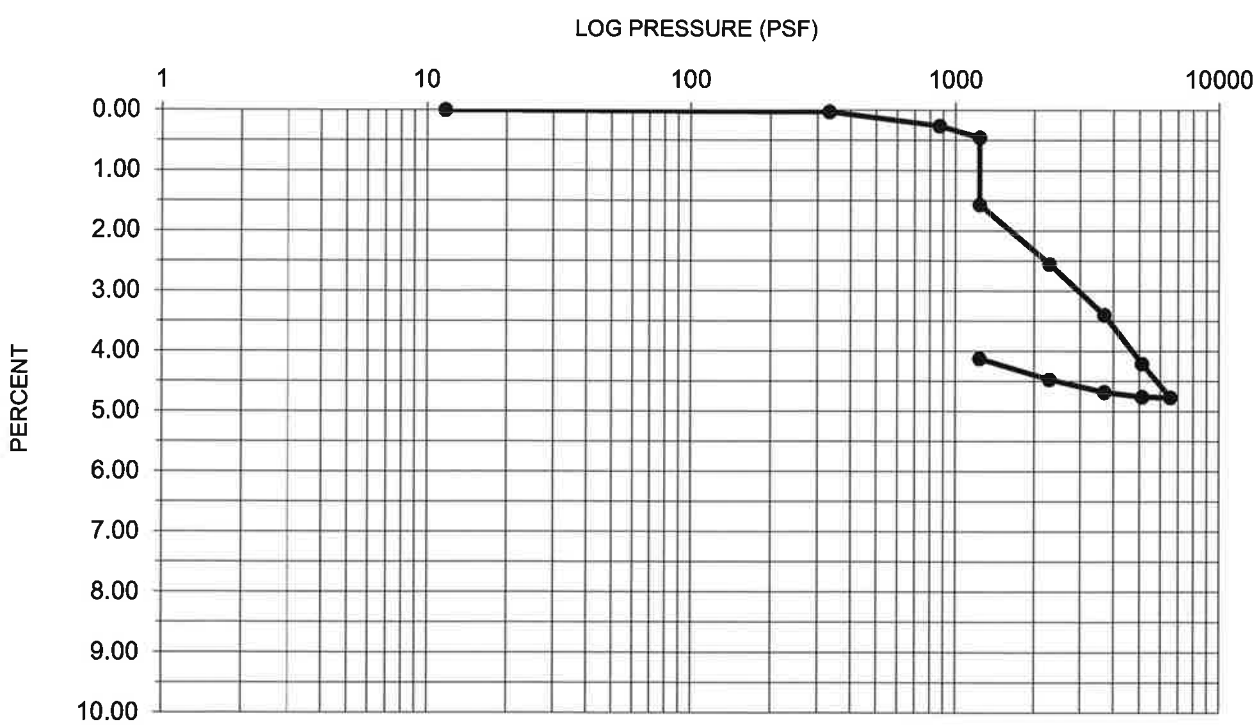
BG: 23185 ENGINEER: JHP

CLIENT: Lankershim Crossing, LLC

Earth Material:	Alluvium
Sample Location:	B1-25'
Dry Weight (pcf):	118.4
Initial Moisture:	2.5%
Initial Saturation:	16.7%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.40
Compression Index (Cc):	0.080
Recompression Index (Cr):	0.018

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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CONSOLIDATION CURVE #5

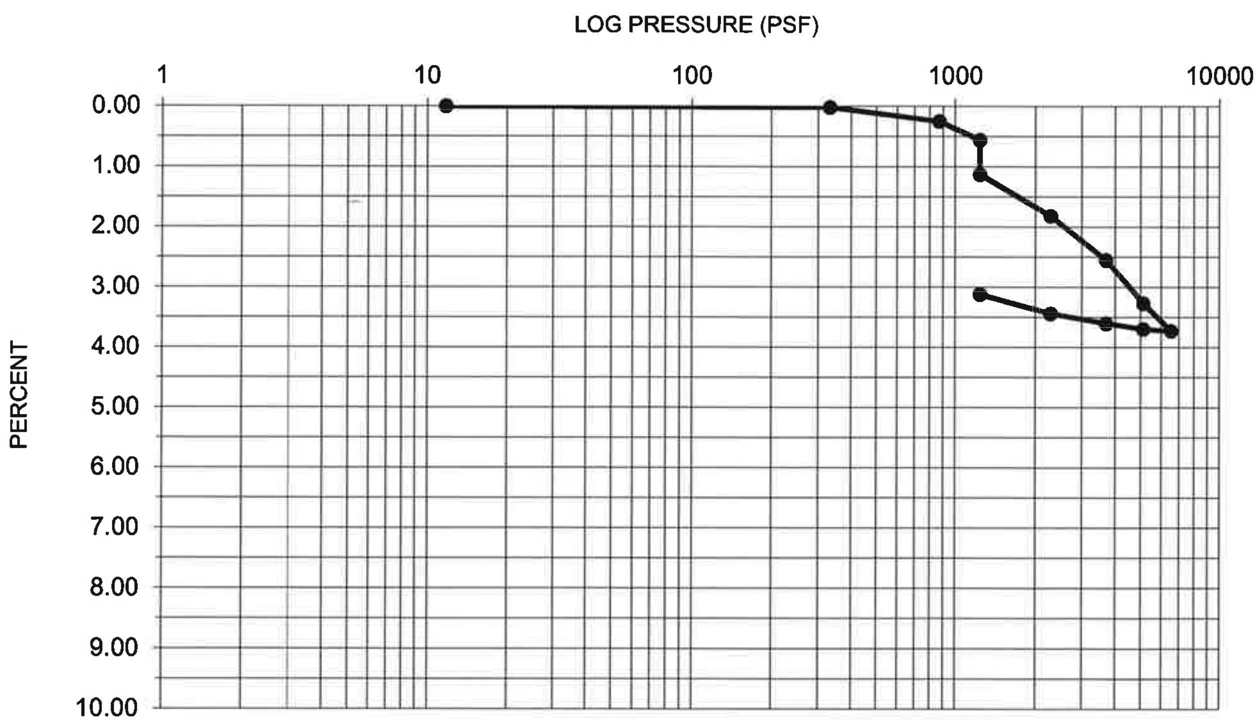
BG: **23185** ENGINEER: **JHP**

CLIENT: **Lankershim Crossing, LLC**

Earth Material:	Alluvium
Sample Location:	B3-30'
Dry Weight (pcf):	117.9
Initial Moisture:	2.8%
Initial Saturation:	18.4%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.40
Compression Index (Cc):	0.071
Recompression Index (Cr):	0.017

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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CONSOLIDATION CURVE #6

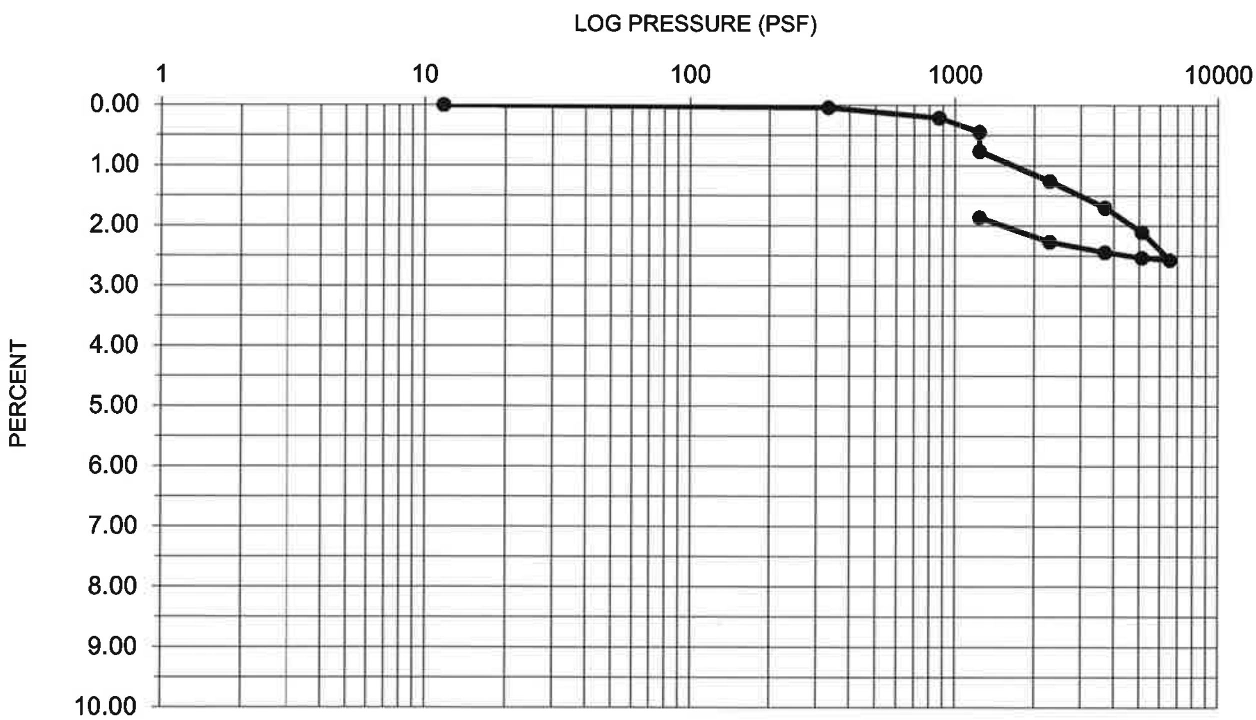
BG: 23185 ENGINEER: JHP

CLIENT: Lankershim Crossing, LLC

Earth Material:	Alluvium
Sample Location:	B1-35'
Dry Weight (pcf):	109.0
Initial Moisture:	5.1%
Initial Saturation:	26.1%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.52
Compression Index (Cc):	0.065
Recompression Index (Cr):	0.023

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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CONSOLIDATION CURVE #7

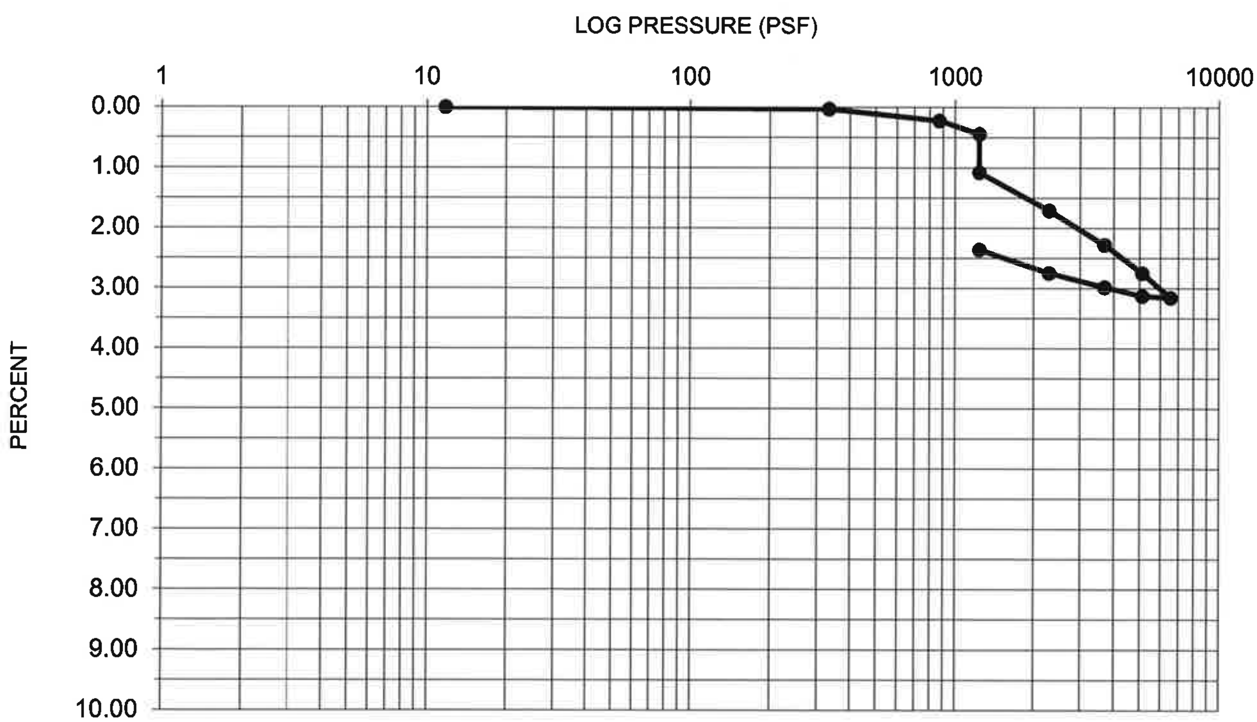
BG: 23185 ENGINEER: JHP

CLIENT: Lankershim Crossing, LLC

Earth Material:	Future Compacted Fill
Sample Location:	B1 (0 -10')
Dry Weight (pcf):	120.7
Initial Moisture:	11.0%
Initial Saturation:	76.4%
Water Added at (psf)	1237

Specific Gravity:	2.68
Initial Void Ratio:	0.39
Compression Index (Cc):	0.053
Recompression Index (Cr):	0.020

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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LOG OF BORING B1

BG No. 23185

CLIENT Lankershim Crossing, LLC — REPORT DATE 3/30/20 — DRILL DATE 1/30/20

PROJECT LOCATION 7918-7946 N. Lankershim Blvd., North Hollywood, CA — LOGGED BY JHP

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 803.5 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	(SM) Surface: Gravel. **FILL (Afu):** 0 - 1.5': Silty SAND, olive-brown, dry, fine to medium sand.		SM						
		(SP) **ALLUVIUM (Qa):** 1.5'-2.5': SAND, tan, dry, medium dense, fine to medium sand, some coarse sand, some fine gravel, subangular.		SP SP						
800		(SP) 2.5': SAND, tan, dry, medium dense, fine to medium sand, some coarse sand, some fine gravel, subangular.			R1	5 8 10	1.9	111.3	10	Direct Shear (*in-situ*) Max, EI, Corrosion Suite, Remoded Shear (95%), Remolded Consolidation (95%)
	5	(SP) 5': SAND, tan, dry, medium dense, fine to medium sand, some coarse sand, some fine gravel, subangular.		SP	Bag1 S1	3 6 8	4.1			
795		(SW) 7.5': Gravelly SAND, light tan, dry, medium dense, fine to coarse sand, fine gravel, subrounded.		SW	R2	12 17 21	1.9	119.4	13	Direct Shear (*in-situ*)
	10	(SW) 10': Gravelly SAND, light tan, dry, medium dense, fine to coarse sand, fine gravel, subrounded.		SW	S2	4 14 7	1.9			
790										
	15	(SM) 15': Silty SAND, olive-brown, dry, dense, fine to medium sand, some fine gravel, 15.2% fines.		SM	R3	5 14 38	3.3	119.6	23	Consolidation, Sieve Wash (-#200)
785										
	20	(SP) 20': SAND, light olive-brown, dry, medium dense, fine to medium sand, some coarse sand, some fine gravel.		SP	S3	6 10 11	2.9			
780										
	25									

Bulk Sample — Ring Sample — Standard Penetration Test

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LOG OF BORING B1

BG No. 23185

CLIENT Lankershim Crossing, LLC REPORT DATE 3/30/20 DRILL DATE 1/30/20

PROJECT LOCATION 7918-7946 N. Lankershim Blvd., North Hollywood, CA LOGGED BY JHP

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 803.5 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	(SW) 25': Gravelly SAND, tan, dry, very dense, fine to coarse sand, fine to coarse gravel.		SW	R4	22 31 50/5"	2.5	118.4	17	Consolidation
775										
	30	(SP) 30': SAND, light olive-brown, dry, dense, fine to medium sand, some coarse sand, some fine gravel.		SP	S4	10 14 18	4			
770										
	35	(SP-SM) 35': SAND with silt, gray- to olive-brown, dry, dense, fine to medium sand, some coarse sand, 11.5% fines.		SP-SM	R5	12 22 42	5.1	109	26	Consolidation, Sieve Wash (-#200)
765										
	40	(SW) 40': Gravelly SAND, tan, dry, very dense, fine to coarse sand, fine to coarse gravel.		SW	S5	30 39 50	2.6			

End at 41.5 Feet; No Groundwater; Fill to 1.5 Feet.

Bulk Sample | Ring Sample | Standard Penetration Test

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LOG OF BORING B2

BG No. 23185

PAGE 1 OF 1

CLIENT Lankershim Crossing, LLC **REPORT DATE** 3/30/20 **DRILL DATE** 1/30/20

PROJECT LOCATION 7918-7946 N. Lankershim Blvd., North Hollywood, CA **LOGGED BY** JHP

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** 805 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
805	0	(SM) Surface: Exposed earth. **FILL (Afu):** 0 - 2.5': Silty SAND, dark brown, slightly moist, fine to medium sand, some fine gravel.		SM	Bag2					
		(SW) **ALLUVIUM (Qa):** 2.5': Gravelly SAND, olive to tan, dry, medium dense, fine to coarse sand, fine to coarse gravel up to 2 inches, subangular.		SW	R1	9 9 10	2.4	120.4	17	
800	5	(SW) 5': Gravelly SAND, tan, dry, medium dense, fine to coarse sand, fine to coarse gravel up to 2 inches, subangular.		SW	R2	7 10 12	2.8	119.4	19	
		(SP) 7.5': SAND, tan, dry, loose, fine to medium sand, some coarse sand.		SP	S1	2 2 3	3.8			
795	10	(SP) 10': SAND, tan, dry, very dense, fine to medium sand, some coarse sand.		SP	R3	8 27 50/5"	2.2	108	11	Consolidation
790	15	(SP) 15': SAND, tan, dry, medium dense, fine to medium sand, some coarse sand, some fine gravel.		SP	S2	3 6 9	3.2			
785	20	(SW) 20': Gravelly SAND, very light gray, dry, very dense, fine to coarse sand, fine to coarse gravel.		SW	R4	21 30 39	2.4	130.5	24	

End at 21.5 Feet; No Groundwater; Fill to 2.5 Feet.

Bulk Sample   Ring Sample   Standard Penetration Test

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LOG OF BORING B3

BG No. 23185

CLIENT Lankershim Crossing, LLC REPORT DATE 3/30/20 DRILL DATE 1/30/20
PROJECT LOCATION 7918-7946 N. Lankershim Blvd., North Hollywood, CA LOGGED BY JHP
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 807 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	(SM) Surface: Exposed earth. **FILL (Afu):** 0 - 2.5': Silty SAND, brown, dry, fine to medium sand.		SM						
805		(SW) **ALLUVIUM (Qa):** 2.5': Gravelly SAND, light tan, dry, medium dense, fine to coarse sand, fine gravel.		SW	S1	8 6 9	5			
	5	(SW) 5': Gravelly SAND, light tan, dry, medium dense, fine to coarse sand, fine gravel.		SW	R1	9 9 11	2.1	109	9	Direct Shear
800		(SW) 7.5': Gravelly SAND, light tan, dry, medium dense, fine to coarse sand, fine gravel.		SW	S2	7 7 8	2.1			
	10	(SW) 10': Gravelly SAND, light tan, dry, dense, fine to coarse sand, fine gravel.		SW	R2	11 20 25	2	118.1	13	Direct Shear
795										
	15	(SW) 15': Gravelly SAND, light tan, dry, dense, fine to coarse sand, fine gravel.		SW	S3	6 16 17	2.3			
790										
	20	(SP) 20': Gravelly SAND, tan to light olive, dry, very dense, fine to coarse sand, fine gravel.		SP	R3	12 30 50/3"	3.8	107.4	19	Consolidation
785										
	25									

Standard Penetration Test   Ring Sample

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LOG OF BORING B3

BG No. 23185

CLIENT Lankershim Crossing, LLC REPORT DATE 3/30/20 DRILL DATE 1/30/20

PROJECT LOCATION 7918-7946 N. Lankershim Blvd., North Hollywood, CA LOGGED BY JHP

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 807 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	(SP) 25': Gravelly SAND, very light olive-brown, dry, very dense, fine to coarse sand, fine gravel.		SP	S4	30 27 25	2.2			
780										
	30	(SP) 30': Gravelly SAND, very light olive-brown, dry, very dense, fine to coarse sand, fine gravel.		SP	R4	23 37 44	2.8	117.9	18	Consolidation

End at 31.5 Feet; No Groundwater; Fill to 2.5 Feet.

Standard Penetration Test

Ring Sample

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LOG OF BORING B4

BG No. 23185

PAGE 1 OF 2

CLIENT Lankershim Crossing, LLC **REPORT DATE** 3/30/20 **DRILL DATE** 1/30/20

PROJECT LOCATION 7918-7946 N. Lankershim Blvd., North Hollywood, CA **LOGGED BY** JHP

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** 804 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	(SM) Surface: Exposed earth. **FILL (Afu):** 0 - 2.5': Silty SAND, brown, dry, fine to medium sand.		SM						
800		(SP) **ALLUVIUM (Qa):** 2.5': SAND, very light olive, dry, medium dense, fine to medium sand, trace coarse sand.		SP	S1	3 6 6	4.8			
	5	(SW) 5': Gravelly SAND, light tan, dry, medium dense, fine to coarse sand, fine gravel.		SW	R1	6 9 19	1.9	119.8	13	
795		(SW) 7.5': Gravelly SAND, tan, dry, dense, fine to coarse sand, fine gravel.		SW	S2	4 12 18	2.7			
	10	(SW) 10': Gravelly SAND, tan, dry, medium dense, fine to coarse sand, fine gravel.		SW	R2	10 16 21	3.1	116.4	19	
790										
	15	(SW) 15': Gravelly SAND, very light gray, dry, dense, fine to coarse sand, fine gravel.		SW	S3	8 20 37	1.5			
785										
	20	(SP) 20': Gravelly SAND, very light gray, dry, very dense, fine to coarse sand, fine gravel.		SP	R3	20 40 41	2.5	125.6	21	
780										
	25									

Standard Penetration Test   Ring Sample   No Recovery

BYER GEOTECHNICAL, INC.

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LOG OF BORING B4

BG No. 23185

CLIENT Lankershim Crossing, LLC REPORT DATE 3/30/20 DRILL DATE 1/30/20

PROJECT LOCATION 7918-7946 N. Lankershim Blvd., North Hollywood, CA LOGGED BY JHP

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 804 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	(SP) 25': SAND with gravel, light olive, dry, dense, fine to medium sand, some coarse sand, some fine gravel, angular.		SP	S4	9 16 25	3.2			
775										
	30	(SP) 30': SAND with gravel, light olive, dry, dense, fine to medium sand, some coarse sand, some fine gravel, angular. Note: Cobble stuck in sampler tip.		SP		50/2"				No Recovery

End at 31.5 Feet; No Groundwater; Fill to 2.5 Feet.

Standard Penetration Test | Ring Sample | No Recovery

APPENDIX III

Calculations and Figures

SEISMIC SOURCES
EZ-FRISK V7.65

DETERMINISTIC CALCULATION
OF PEAK GROUND ACCELERATION BASED ON DIGITIZED FAULT DATA

BG: 23185 ANALYSIS DATE: 3/23/2020
CLIENT: Lankershim Crossing, LLC ENGINEER: JHP
PROJECT DESCRIPTION: Proposed 7-Story Mixed-Use Building over One Subterranean Level

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SITE COORDINATES: LATITUDE: 34.2149
LONGITUDE: -118.3868

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)
Verdugo	1.9	1.2	6.9	0.597
Sierra Madre (San Fernando)	7.9	4.9	6.7	0.393
Sierra Madre Connected	7.9	4.9	7.3	0.439
Santa Monica	10.6	6.6	7.4	0.574
Sierra Madre	11.0	6.8	7.2	0.375
Hollywood	12.1	7.5	6.7	0.377
Northridge	12.3	7.7	6.9	0.501
Elysian Park (Upper)	14.4	8.9	6.7	0.326
San Gabriel	14.9	9.2	7.3	0.321
Santa Susana, alt 1	15.7	9.8	6.9	0.288
Puente Hills (LA)	17.4	10.8	7.0	0.326
Raymond	18.2	11.3	6.8	0.262
Newport-Inglewood	19.1	11.9	7.5	0.296
Puente Hills	19.1	11.9	7.1	0.324
Malibu Coast	23.5	14.6	7.0	0.239
Anacapa-Dume	24.2	15.0	7.2	0.277

FAULT NAME	APPROXIMATE DISTANCE (km)	(mi)	MAXIMUM EATHQUAKE MAGNITUDE (Mw)	PEAK GROUND ACCELERATION (g)
Holser, alt 1	25.7	16.0	6.8	0.216
Puente Hills (Santa Fe Springs)	27.4	17.0	6.7	0.230
Simi-Santa Rosa	30.1	18.7	6.9	0.185
Palos Verdes	31.4	19.5	7.3	0.205
Palos Verdes Connected	31.4	19.5	7.7	0.237
Clamshell-Sawpit	32.5	20.2	6.7	0.171
Oak Ridge Connected	35.6	22.1	7.4	0.219
Oak Ridge (Onshore)	36.4	22.6	7.2	0.207
Elsinore	39.6	24.6	7.9	0.217
San Cayetano	42.3	26.3	7.2	0.164
Puente Hills (Coyote Hills)	42.6	26.5	6.9	0.166
Southern San Andreas	44.4	27.6	8.2	0.231
San Jose	49.4	30.7	6.7	0.116
Chino	58.0	36.0	6.8	0.102
Cucamonga	58.1	36.1	6.7	0.100
Santa Ynez (East)	60.1	37.4	7.2	0.123
Santa Ynez Connected	60.3	37.5	7.4	0.135
Ventura-Pitas Point	70.8	44.0	7.0	0.106
Pitas Point Connected	70.8	44.0	7.3	0.124
San Joaquin Hills	71.4	44.4	7.1	0.172
Imp Extensional Gridded, Char, Normal	55.0	34.2	7.0	0.106
Imp Extensional Gridded, Char, Strike Slip	55.0	34.2	7.0	0.128
Imp Extensional Gridded, GR, Normal	54.9	34.2	7.0	0.106
Imp Extensional Gridded, GR, Strike Slip	54.9	34.2	7.0	0.128
Mission Ridge-Arroyo Parida-Santa Ana	75.1	46.7	6.9	0.086
San Jacinto	76.0	47.3	7.9	0.139
Oak Ridge (Offshore)	78.6	48.8	7.0	0.087
Garlock	81.1	50.4	7.7	0.122
Pleito	83.1	51.6	7.1	0.087
Channel Islands Thrust	83.5	51.9	7.3	0.112
Santa Cruz Island	84.4	52.5	7.2	0.088
Red Mountain	85.4	53.1	7.4	0.100
Cleghorn	85.4	53.1	6.8	0.069
North Channel	96.1	59.7	6.8	0.067

50 Faults found within a 100 km Search Radius.
Closest Fault to the Site: Verdugo Distance = 1.94 km (1.21mi)
Largest Peak Ground Acceleration: 0.597 g
The San Andreas Fault is Located Aproximately 44.4 km (27.6 mi) from the Site.

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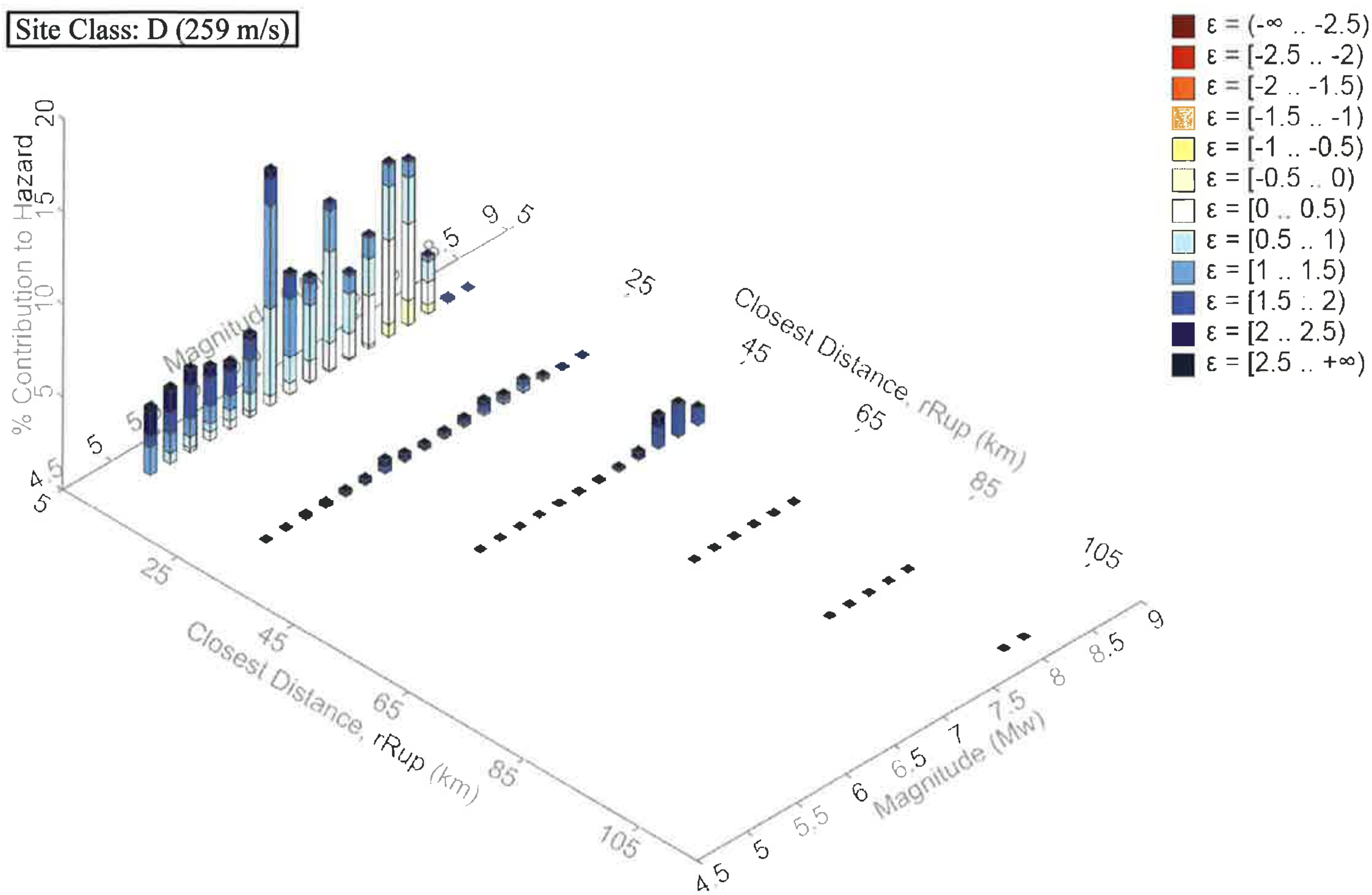
SEISMIC HAZARD DEAGGREGATION CHART
(Probability of Exceedance: 10% in 50 years)

BG: 23185 CLIENT: LANKERSHIM CROSSING LLC

ENGINEER: JHP

REFERENCE: USGS, 2019, Earthquake Hazards Program, Beta - Unified Hazard Tool, Seismic Hazard Deaggregation, Conterminous U.S. 2014 (update) (v4.2.0) 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: 0.55006282 g

Recovered targets

Return period: 512.48056 yrs
Exceedance rate: 0.0019512935 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.13 %

Mode (largest m-r bin)

m: 6.3
r: 9.7 km
εo: 1.1 σ
Contribution: 12.65 %

Mode (largest m-r-εo bin)

m: 6.29
r: 8.97 km
εo: 1.16 σ
Contribution: 5.53 %

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

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

BG: 23185 Client: Lankershim Crossing, LLC Date: 3/23/20
Project Description: Proposed 7-Story Mixed-Use Building Engineer: RSB

Ss (0.2s) =	2.003	Latitude:	34.2149			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) =	0.685	Longitude:	-118.3868			T_o =	0.171		
Fa =	1.00	Site Class:	D			T_s =	0.855		
Fv =	2.50					T_L =	8		
SMs =	2.003		Fig. 22-18A	S_{MS}=	1.442		<	1.602	**1.602**
SM1 =	1.713	C_{RS}:	0.913	S_{M1}=	1.370		=	1.370	**1.370**
SDs =	1.335		Fig. 22-19A	S_{DS}=	0.961		<	1.068	**1.068**
SD1 =	1.142	C_{R1}:	0.902	S_{D1}=	0.913		=	0.913	**0.913**

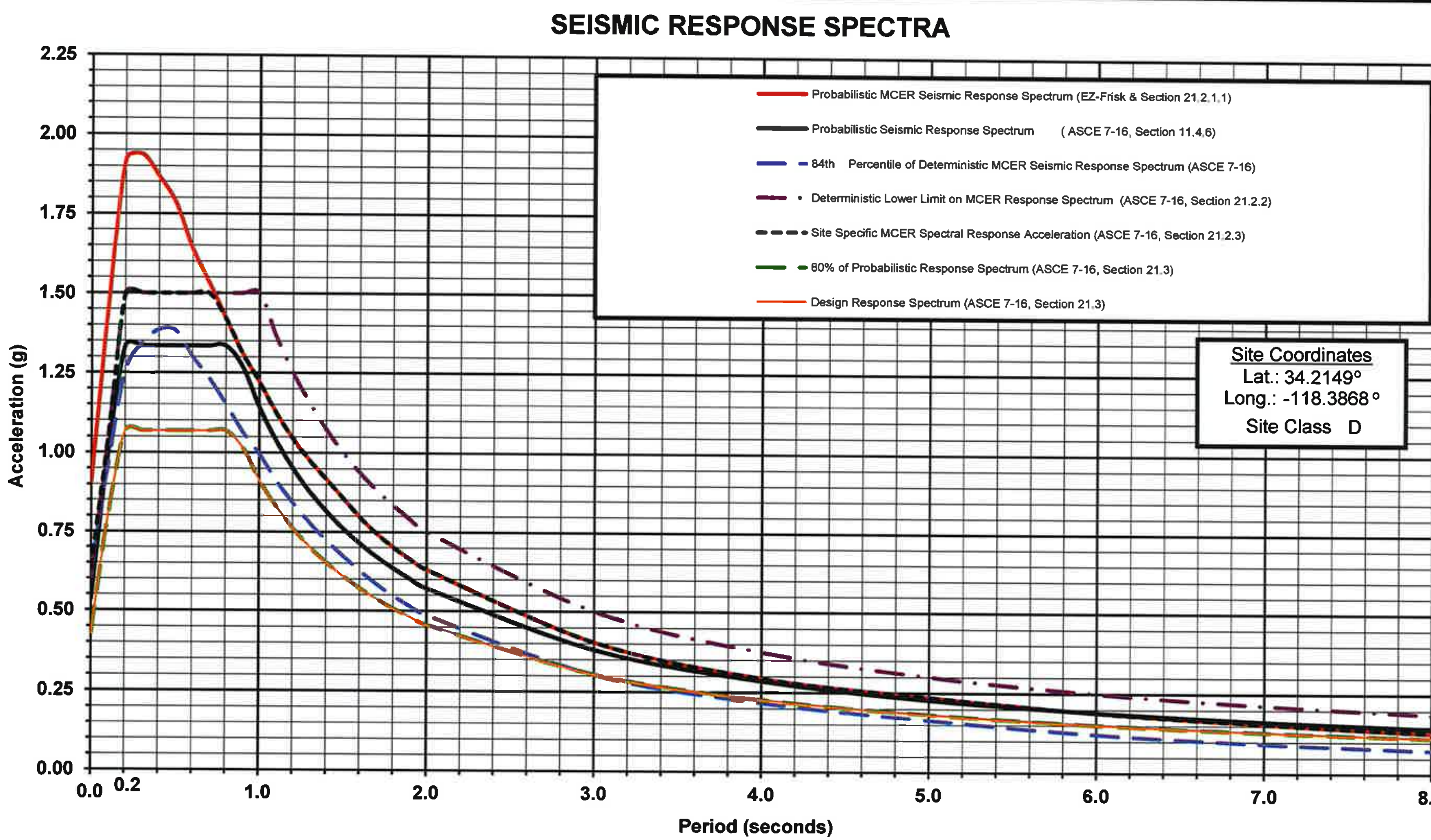
Fundamental Period	Risk Coefficient C_R (Method 1, Section 21.2.1.1, ASCE 7-16)	Probabilistic MCE_R Seismic Response Spectrum (EZ-Frisk & Section 21.2.1.1)	Probabilistic Seismic Response Spectrum (ASCE 7-16, Section 11.4.6)	84th Percentile of Deterministic MCE_R Seismic Response Spectrum (ASCE 7-16)	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	0.913	0.9079	0.5341	0.6303	0.600	0.630	0.427	0.427
0.1	0.913	1.4645	1.0027	0.9651	1.050	1.050	0.802	0.802
0.2	0.913	1.9200	1.3353	1.2600	1.500	1.500	1.068	1.068
0.3	0.912	1.9363	1.3353	1.3430	1.500	1.500	1.068	1.068
0.4	0.910	1.8660	1.3353	1.3860	1.500	1.500	1.068	1.068
0.5	0.909	1.7823	1.3353	1.3820	1.500	1.500	1.068	1.068
0.6	0.908	1.6389	1.3353	1.3000	1.500	1.500	1.068	1.068
0.7	0.906	1.5314	1.3353	1.2320	1.500	1.500	1.068	1.068
0.8	0.905	1.4205	1.3353	1.1500	1.500	1.420	1.068	1.068
0.9	0.903	1.3099	1.2685	1.0640	1.500	1.310	1.015	1.015
1.0	0.902	1.2204	1.1417	0.9943	1.500	1.220	0.913	0.913
1.1	0.902	1.1248	1.0379	0.9126	1.364	1.125	0.830	0.830
1.2	0.902	1.0445	0.9514	0.8414	1.250	1.045	0.761	0.761
1.3	0.902	0.9760	0.8782	0.7788	1.154	0.976	0.703	0.703
1.4	0.902	0.9173	0.8155	0.7232	1.071	0.917	0.652	0.652
1.5	0.902	0.8572	0.7611	0.6733	1.000	0.857	0.609	0.609
1.6	0.902	0.7970	0.7135	0.6260	0.938	0.797	0.571	0.571
1.7	0.902	0.7455	0.6716	0.5840	0.882	0.746	0.537	0.537
1.8	0.902	0.7013	0.6343	0.5470	0.833	0.701	0.507	0.507
1.9	0.902	0.6632	0.6009	0.5144	0.789	0.663	0.481	0.481
2.0	0.902	0.6298	0.5708	0.4855	0.750	0.630	0.457	0.457
3.0	0.902	0.4039	0.3806	0.3026	0.500	0.404	0.304	0.304
4.0	0.902	0.2932	0.2854	0.2176	0.375	0.293	0.228	0.228
5.0	0.902	0.2362	0.2283	0.1644	0.300	0.236	0.183	0.183
6.0	0.902	0.1893	0.1903	0.1222	0.250	0.189	0.152	0.152
7.0	0.902	0.1557	0.1631	0.0941	0.214	0.156	0.130	0.130
8.0	0.902	0.1288	0.1427	0.0742	0.188	0.129	0.114	0.114

* 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.



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SITE-SPECIFIC SEISMIC RESPONSE SPECTRA

Proposed 7-Story Mixed-Use Building

BG: 23185 Client: Lankershim Crossing, LLC

Engineer: RSB Date: March 23, 2020

In-Situ Percolation Test Results - Falling Head Method

Calculation Sheet #: 1

BG No.: 23185
Client: Lankershim Crossing, LLC
Project Name: Proposed 7-Story Mixed-Used Building over One Subterranean Level

Date Excavated: 1/30/2020
Date Tested: 1/30/2020
Tested by: JHP

Input Data:

Boring No.	Date of Presoak	Time of Presoak	Top of Perforation Depth (ft)	Perc Hole Depth (ft)	Approx. Hole Diam. (in)	Pipe ID (in)
B1*	1/30/20	9:26 AM	10	40	8	2

Soil Distribution
0 - 7.5 ft: Poorly Graded Sand (SP)
7.5 - 15 ft: Well Graded Sand (SW)
15 - 20 ft: Silty Sand (SM)
20 - 25 ft: Poorly Graded Sand (SP)
25 - 30 ft: Well Graded Sand (SW)
30 - 40 ft: Sand with Silt (SP-SM)

Falling Head Percolation Test Data and Results:

Test Number	Initial Time of Reading	Final Time of Reading	Elapsed Time (min.)	Initial Water Depth d_1, (ft)	Surface Area (sq-ft)	Final Water Depth (ft)	Water Level Drop Δd, (ft)	Initial Vol. of Water (cu-ft)	Final Vol. of Water (cu-ft)	Vol. of Water Discharge (cu-ft)	Infiltration Rate (in./hr.)
1	9:45:00	9:55:00	10	36.0	75.7	2.5	33.5	12.6	0.9	11.7	11.12
2	9:56:00	10:06:00	10	36.0	75.7	3.0	33.0	12.6	1.0	11.5	10.95
3	10:41:00	10:51:00	10	33.0	69.5	3.5	29.5	11.5	1.2	10.3	10.67
4	10:51:00	11:01:00	10	32.0	67.4	3.5	28.5	11.2	1.2	9.9	10.63
5	12:11:00	12:21:00	10	33.0	69.5	4.0	29.0	11.5	1.4	10.1	10.49
6	12:22:00	12:32:00	10	32.0	67.4	3.9	28.2	11.2	1.3	9.8	10.50
7	12:34:00	12:44:00	10	32.0	67.4	3.9	28.2	11.2	1.3	9.8	10.50
8	12:45:00	12:55:00	10	32.0	67.4	3.9	28.2	11.2	1.3	9.8	10.50
										Calculated Infiltration Rate (in/hr) =	**10.5**

*** See Site Plan for boring location.**

Reference: County of Los Angeles, 2017, Administrative Manual, Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration, Department of Public Works, GS200.2, dated June 30, 2017.

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RETAINING WALL CALCULATION

BG **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#2a**
Cantilevered Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	200.0 psf
Phi Angle, φ	32.0 degrees
Density, γ	125.0 pcf
Anisotropic Strength Function	NO
Restraining Device	RETAINING WALL
Type	CANTILEVERED
Retained Height, H	14 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	STATIC

Calculation Safety Factor, FS 1.5

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid	1371 trials
Trial Wedges Analyzed, Secondary Search Window	324 trials
Critical Failure Angle, α	53.2 degrees
Area of Critical Wedge	73.1 square feet
Length of Critical Failure Plane, L	16.4 feet
Depth of Critical Tension Crack	0.9 feet
Horizontal Upslope Distance to Critical Tension Crack	9.9 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	22.6 degrees
Factored Cohesion on Critical Slip Plane, C'	133.3 psf
Weight of Critical Wedge, W	9,142 pounds
External Surcharge on Critical Wedge, V	1,605 pounds
Static Gravitational Driving Force, W'	10,747 pounds
Mobilized Cohesive Force, C'L	2,191 pounds
Mobilized Frictional Force, R	10,443 pounds
Calculated Unbalanced Force, P	3,995 pounds
Calculated Horizontal Unbalanced Force, P_h	3,995 pounds
Calculated Equivalent Fluid Pressure	40.8 pcf

RECOMMENDED DESIGN PARAMETERS

Design Equivalent Fluid Pressure, EFP	43.0 pcf
Design Horizontal Force	4,214 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	14		
(0,14)	(0,14)			
(5,14)	(5,14)			Uniform Load: 300 psf
(15,14)	(15,14)			
(18,14)	(18,14)			
(20,14)	(20,14)			
(25,14)	(25,14)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED CANTILEVERED RETAINING WALL, WITH A RETAINED HEIGHT OF UP TO 14 FEET, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE (EFP) OF 43 POUNDS PER CUBIC FOOT.

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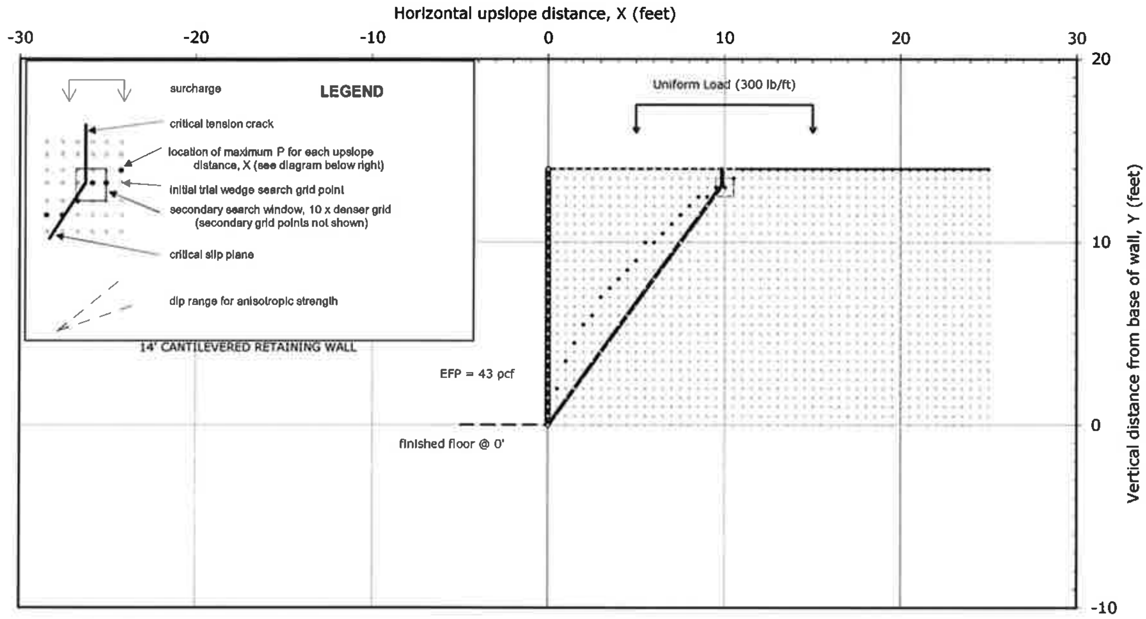
RETAINING WALL CALCULATION

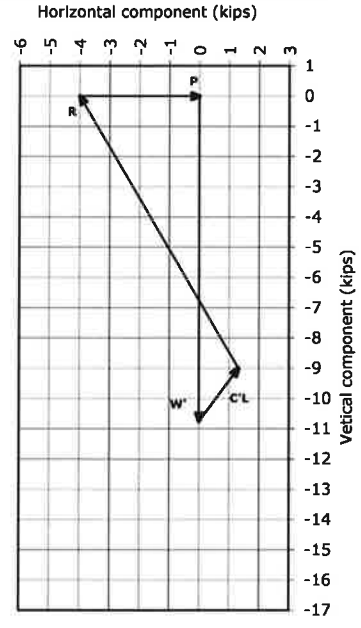
BG: **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#2b**
Cantilevered Retaining Wall, basement

Cross Section and Critical Active Wedge



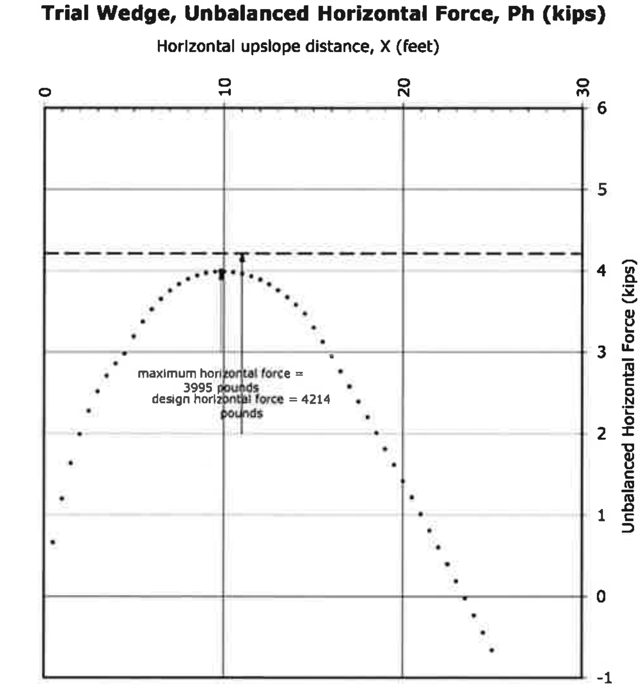
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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RETAINING WALL CALCULATION

BG **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#2Sa**
Cantilevered Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE PSEUDO-STATIC (MONONOBE-OKABE) METHOD FOR SEISMIC LOADING.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	200.0 psf
Phi Angle, φ	32.0 degrees
Density, γ	125.0 pcf
Anisotropic Strength Function	NO
Restraining Device	RETAINING WALL
Type	CANTILEVERED
Retained Height, H	14 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	SEISMIC
PGA_M	0.90 g

Pseudostatic Coefficients:

horizontal , K_h***	0.30 g
vertical, K_v****	0.00 g
Calculation Safety Factor, FS	1

* Critical wedge 'sees' only portion of regional backslope

*** Calculated using methodology of Abrahamson and Silva (1986)

**** Kv > 0 indicates downward acceleration and upward inertial force

CALCULATION OUTPUT

Use Critical Trial Wedge From Static Case

Critical Failure Angle, α	53.2 degrees
Area of Critical Wedge	73.1 square feet
Length of Critical Failure Plane, L	16.4 feet
Depth of Critical Tension Crack	0.9 feet
Horizontal Upslope Distance to Critical Tension Crack	9.9 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	32.0 degrees
Factored Cohesion on Critical Slip Plane, C'	200.0 psf
Weight of Critical Wedge, W	9,142 pounds
External Surcharge on Critical Wedge, V	1,605 pounds
Pseudo-Static (Gravitational + Dynamic) Driving Force, Wd	11,223 pounds
Mobilized Cohesive Force, C'L	3,286 pounds
Mobilized Frictional Force, R	8,704 pounds
Calculated Unbalanced Force, P	4,576 pounds
Calculated Horizontal Unbalanced Force, P_h	4,576 pounds

RECOMMENDED DESIGN PARAMETERS

Calculated Pseudo-Static Horizontal Force	4,576 pounds
Recommended Static Horizontal Force from sheet 2a	4,214 pounds
Calculated Seismic Force ***	362 pounds

*** the seismic force should be applied at 0.6H, where H is the retained height

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	14		
(0,14)	(0,14)			
(5,14)	(5,14)			Uniform Load: 300 psf
(15,14)	(15,14)			
(18,14)	(18,14)			
(20,14)	(20,14)			
(25,14)	(25,14)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATED SEISMIC FORCE ON THE WALL IS THE DIFFERENCE BETWEEN THE PSEUDO-STATIC AND STATIC FORCE, AND IS 362 POUNDS. THE WALL SHOULD BE DESIGNED FOR THIS FORCE IN ADDITION TO THE RECOMMENDED DESIGN PARAMETERS ON SHEET 2A. THE SEISMIC FORCE MAY BE APPLIED AT 0.3H ABOVE THE BASE, WHERE H IS THE RETAINED HEIGHT.

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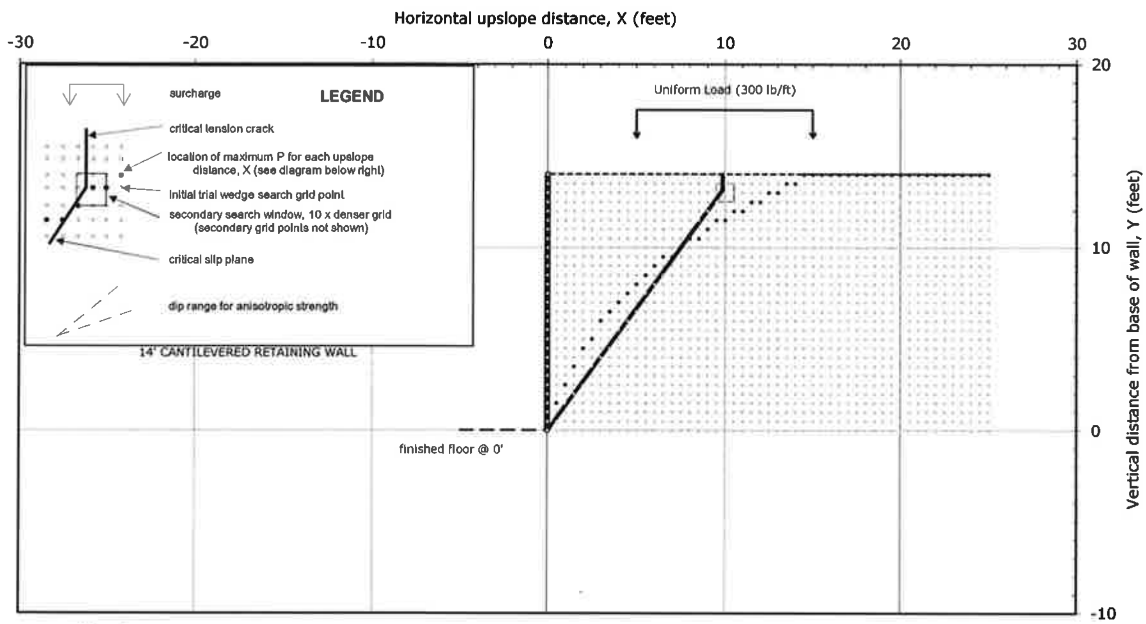
RETAINING WALL CALCULATION

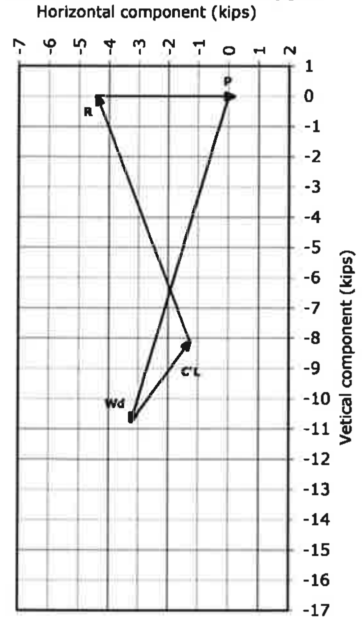
BG: **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#2Sb**
Cantilevered Retaining Wall, basement

Cross Section and Critical Active Wedge



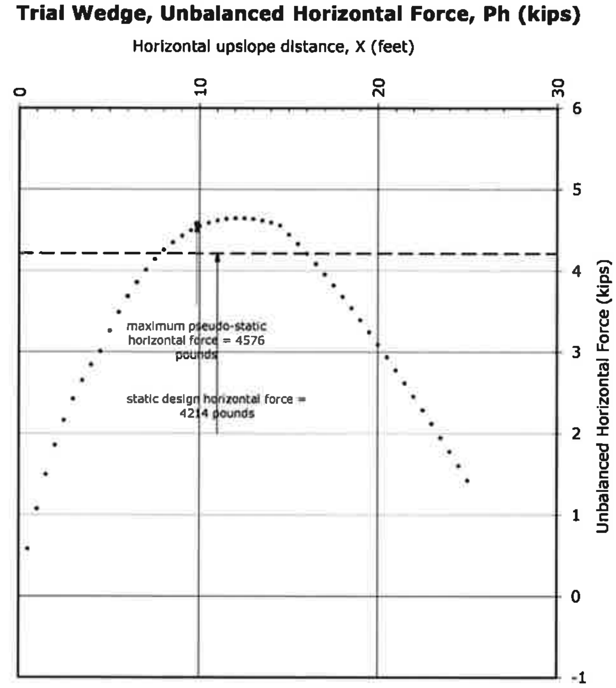
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the pseudo-static (gravitational and dynamic) driving force, Wd; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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RETAINING WALL CALCULATION

BG **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#3a**
Restrained Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED RESTRAINED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	200.0 psf
Phi Angle, φ	32.0 degrees
Density, γ	125.0 pcf
Anisotropic Strength Function	NO
Restraining Device	RETAINING WALL
Type	RESTRAINED
Retained Height, H	14 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	NO
General Backslope Condition*	level
Loading	STATIC
Calculation Safety Factor, FS	1.5

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid	1371 trials
Trial Wedges Analyzed, Secondary Search Window	324 trials
Critical Failure Angle, α	56.3 degrees
Area of Critical Wedge	61.9 square feet
Length of Critical Failure Plane, L	13.0 feet
Depth of Critical Tension Crack	3.2 feet
Horizontal Upslope Distance to Critical Tension Crack	7.2 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	22.6 degrees
Factored Cohesion on Critical Slip Plane, C'	133.3 psf
Weight of Critical Wedge, W	7,740 pounds
External Surcharge on Critical Wedge, V	0 pounds
Static Gravitational Driving Force, W'	7,740 pounds
Mobilized Cohesive Force, C'L	1,731 pounds
Mobilized Frictional Force, R	7,572 pounds
Calculated Unbalanced Force, P	3,241 pounds
Calculated Horizontal Unbalanced Force, P_h	3,241 pounds
Calculated Trapezoidal Design Pressure *	20.7 H psf
Calculated At-Rest Equivalent Fluid Pressure **	58.8 pcf
Calculated At-Rest Trapezoidal Earth Pressure *	36.7 H psf

RECOMMENDED DESIGN PARAMETERS

Trapezoidal Design Pressure, TDP*	37 H psf
Design Horizontal Force	5,802 pounds

* H is restrained height, see report for diagram of trapezoidal pressure distribution
** at-rest equivalent fluid pressure is calculated as: γ (1- sin(φ))

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	14		
(0,14)	(0,14)			
(5,14)	(5,14)			
(15,14)	(15,14)			
(5,14)	(5,14)			
(10,14)	(10,14)			
(25,14)	(25,14)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED RESTRAINED RETAINING WALL, WITH A RETAINED HEIGHT OF UP TO 14 FEET, MAY BE DESIGNED FOR A TRAPEZOIDAL DESIGN PRESSURE (TDP) OF 37 H POUNDS PER SQUARE FOOT, WHERE H IS THE RETAINED HEIGHT. SEE REPORT FOR DIAGRAM OF TRAPEZOIDAL PRESSURE DISTRIBUTION.

THE STATIC DESIGN IS GOVERNED BY THE AT-REST CONDITION.

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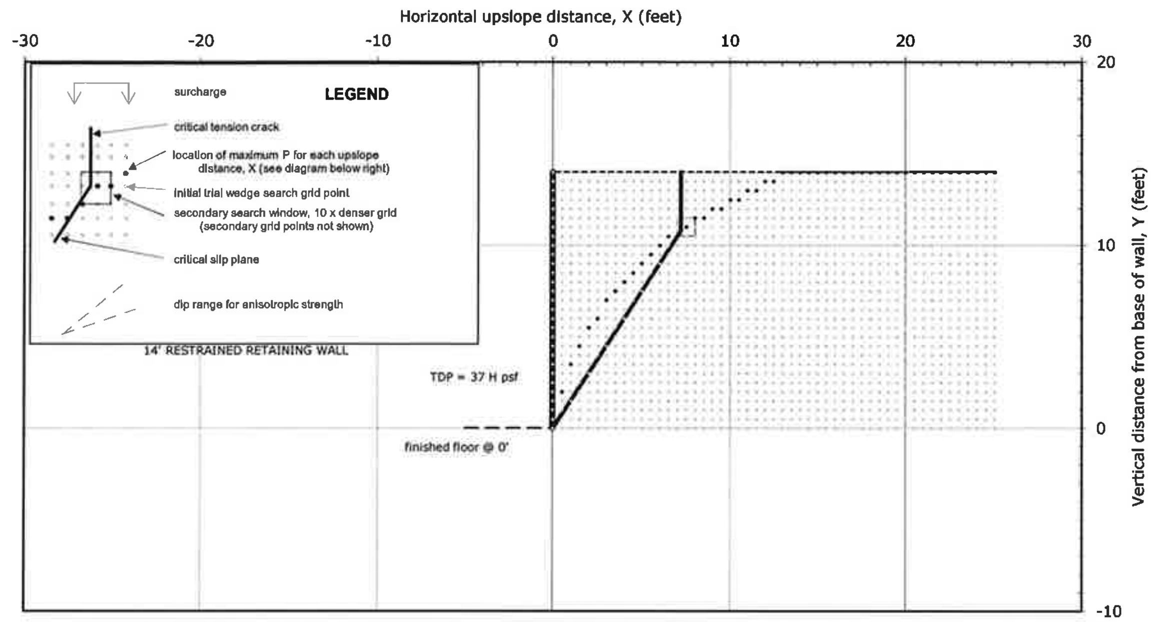
RETAINING WALL CALCULATION

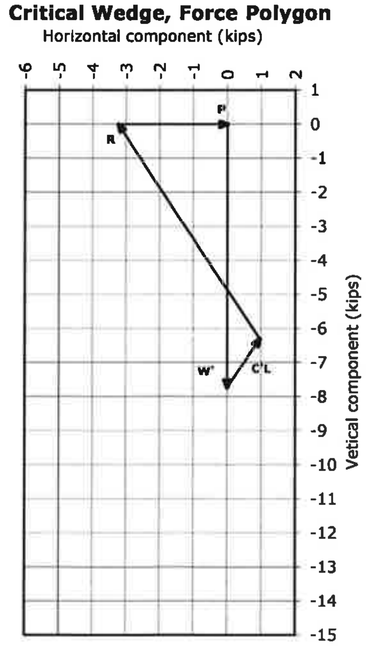
BG: **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#3b**
Restrained Retaining Wall, basement

Cross Section and Critical Active Wedge



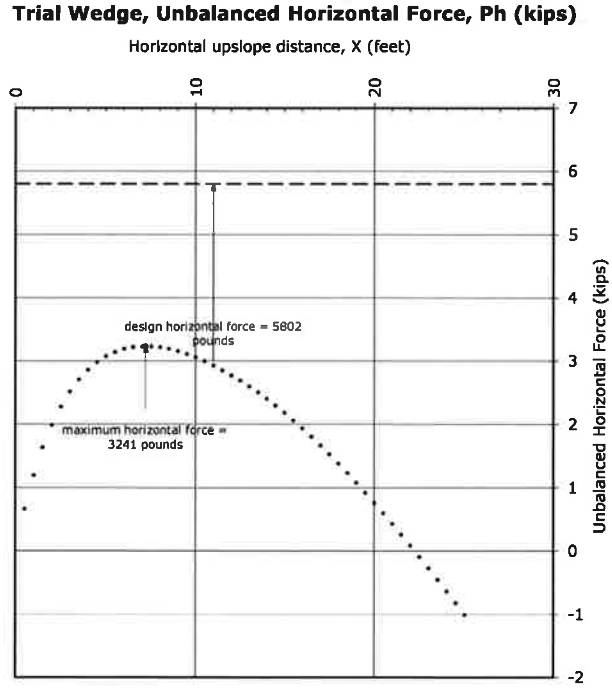
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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RETAINING WALL CALCULATION

BG **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#3Sa**
Restrained Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED RESTRAINED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE PSEUDO-STATIC (MONONOBE-OKABE) METHOD FOR SEISMIC LOADING.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	200.0 psf
Phi Angle, φ	32.0 degrees
Density, γ	125.0 pcf
Anisotropic Strength Function	NO
Restraining Device	RETAINING WALL
Type	RESTRAINED
Retained Height, H	14 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	SEISMIC
PGA_M	0.90 g

Pseudostatic Coefficients:

horizontal , K_h***	0.30 g
vertical, K_v****	0.00 g
Calculation Safety Factor, FS	1

* Critical wedge 'sees' only portion of regional backslope

*** Calculated using methodology of Abrahamson and Silva (1986)

**** Kv > 0 indicates downward acceleration and upward inertial force

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	14		
(0,14)	(0,14)			
(5,14)	(5,14)			Uniform Load: 300 psf
(15,14)	(15,14)			
(5,14)	(5,14)			
(10,14)	(10,14)			
(25,14)	(25,14)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CALCULATION OUTPUT

Use Critical Trial Wedge From Static Case

Critical Failure Angle, α	53.7 degrees
Area of Critical Wedge	71.5 square feet
Length of Critical Failure Plane, L	16.1 feet
Depth of Critical Tension Crack	1.1 feet
Horizontal Upslope Distance to Critical Tension Crack	9.5 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	32.0 degrees
Factored Cohesion on Critical Slip Plane, C'	200.0 psf
Weight of Critical Wedge, W	8,936 pounds
External Surcharge on Critical Wedge, V	1,500 pounds
Pseudo-Static (Gravitational + Dynamic) Driving Force, Wd	10,898 pounds
Mobilized Cohesive Force, C'L	3,212 pounds
Mobilized Frictional Force, R	8,447 pounds
Calculated Unbalanced Force, P	4,432 pounds
Calculated Horizontal Unbalanced Force, P_h	4,432 pounds

RECOMMENDED DESIGN PARAMETERS

Calculated Pseudo-Static Horizontal Force	4,432 pounds
Recommended Static Horizontal Force from sheet 3a	5,802 pounds

CONCLUSIONS

THE CALCULATED STATIC FORCE EXCEEDS THE CALCULATED PSEUDO-STATIC FORCE. THEREFORE, THE RECOMMENDED DESIGN PARAMETERS ON SHEET 3A ARE SUFFICIENT.

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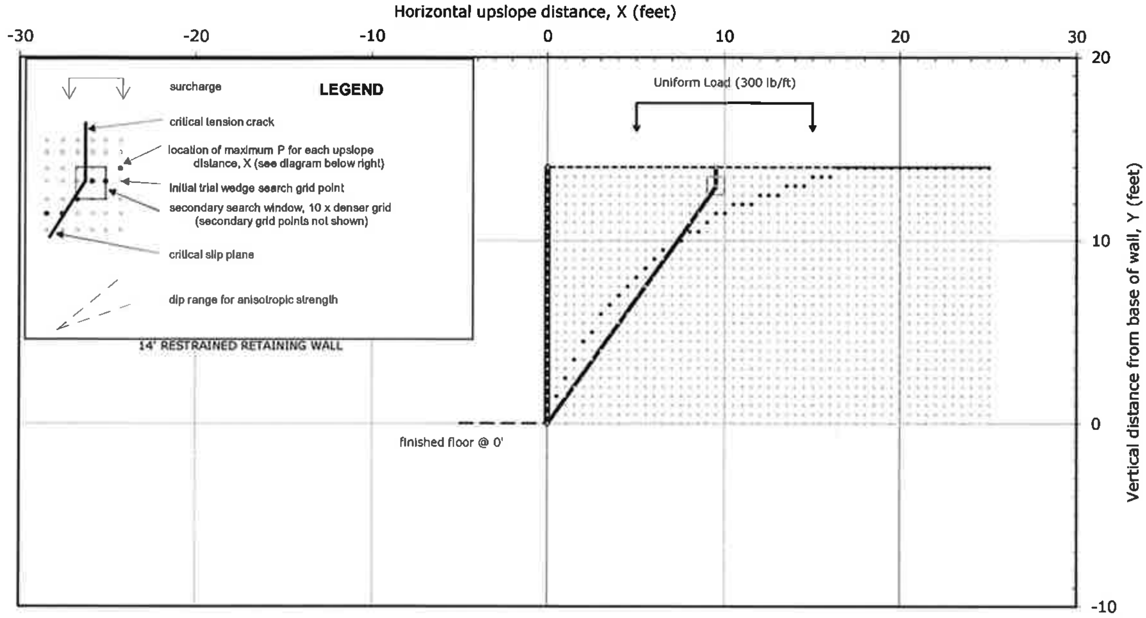
RETAINING WALL CALCULATION

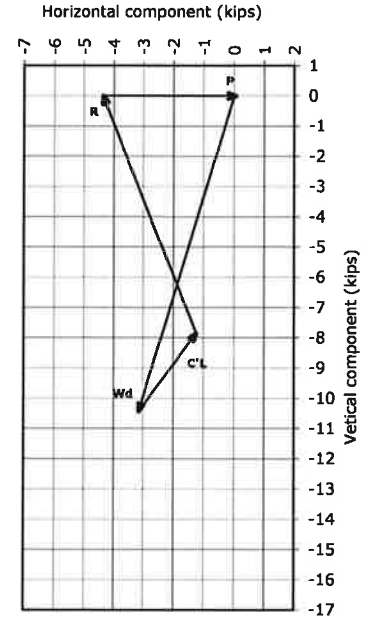
BG: **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#3Sb**
Restrained Retaining Wall, basement

Cross Section and Critical Active Wedge



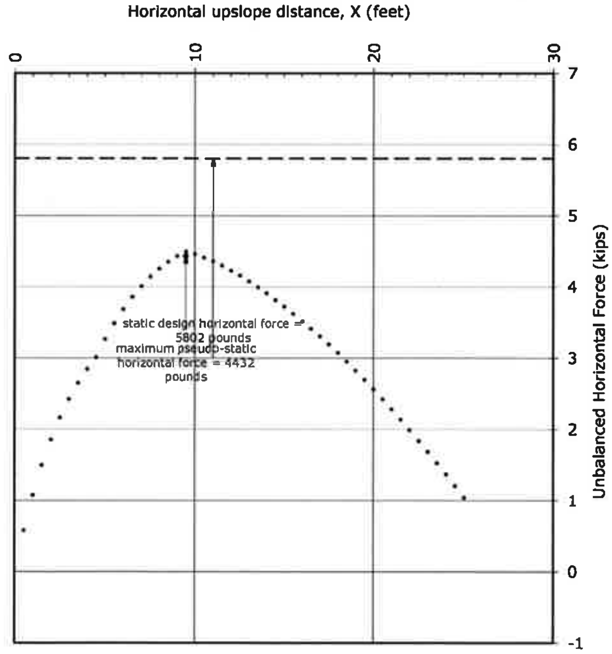
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the pseudo-static (gravitational and dynamic) driving force, Wd; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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RETAINING WALL CALCULATION

BG **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#4Sa**
Restrained Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED RESTRAINED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE PSEUDO-STATIC (MONONOBE-OKABE) METHOD FOR SEISMIC LOADING.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	200.0 psf
Phi Angle, φ	32.0 degrees
Density, γ	125.0 pcf
Anisotropic Strength Function	NO
Restraining Device	RETAINING WALL
Type	RESTRAINED
Retained Height, H	14 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	SEISMIC
PGA_M	0.90 g

Pseudostatic Coefficients:

horizontal , K_h***	0.30 g
vertical, K_v****	0.00 g
Calculation Safety Factor, FS	1

* Critical wedge 'sees' only portion of regional backslope

*** Calculated using methodology of Abrahamson and Silva (1986)

**** Kv > 0 indicates downward acceleration and upward inertial force

CALCULATION OUTPUT

Use Critical Trial Wedge From Static Case

Critical Failure Angle, α	60.8 degrees
Area of Critical Wedge	51.8 square feet
Length of Critical Failure Plane, L	12.3 feet
Depth of Critical Tension Crack	3.3 feet
Horizontal Upslope Distance to Critical Tension Crack	6.0 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	32.0 degrees
Factored Cohesion on Critical Slip Plane, C'	200.0 psf
Weight of Critical Wedge, W	6,469 pounds
External Surcharge on Critical Wedge, V	1,200 pounds
Pseudo-Static (Gravitational + Dynamic) Driving Force, Wd	8,009 pounds
Mobilized Cohesive Force, C'L	2,462 pounds
Mobilized Frictional Force, R	6,300 pounds
Calculated Unbalanced Force, P	4,296 pounds
Calculated Horizontal Unbalanced Force, P_h	4,296 pounds

RECOMMENDED DESIGN PARAMETERS

Calculated Pseudo-Static Horizontal Force	4,296 pounds
Recommended Static Horizontal Force from sheet 4a	5,802 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	14		
(0,14)	(0,14)			
(1,14)	(1,14)			Line Load: 1200 psf
(2,14)	(2,14)			
(5,14)	(5,14)			
(10,14)	(10,14)			
(25,14)	(25,14)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATED STATIC FORCE EXCEEDS THE CALCULATED PSEUDO-STATIC FORCE. THEREFORE, THE RECOMMENDED DESIGN PARAMETERS ON SHEET 4A ARE SUFFICIENT.

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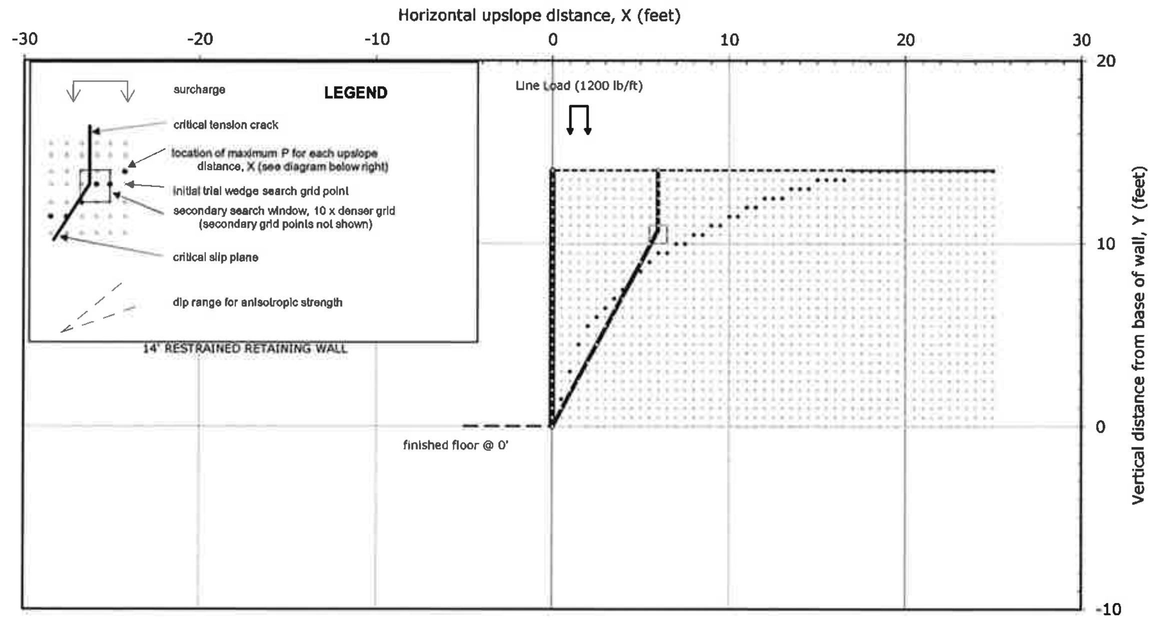
RETAINING WALL CALCULATION

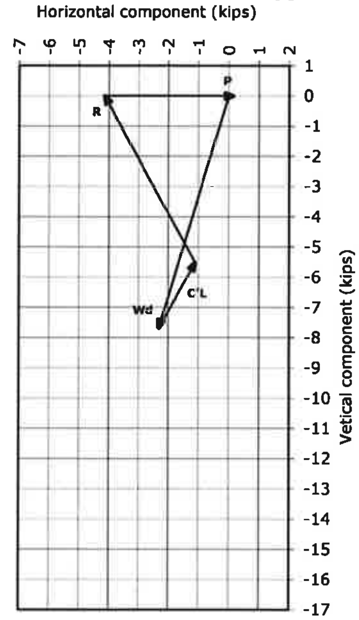
BG: **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#4Sb**
Restrained Retaining Wall, basement

Cross Section and Critical Active Wedge



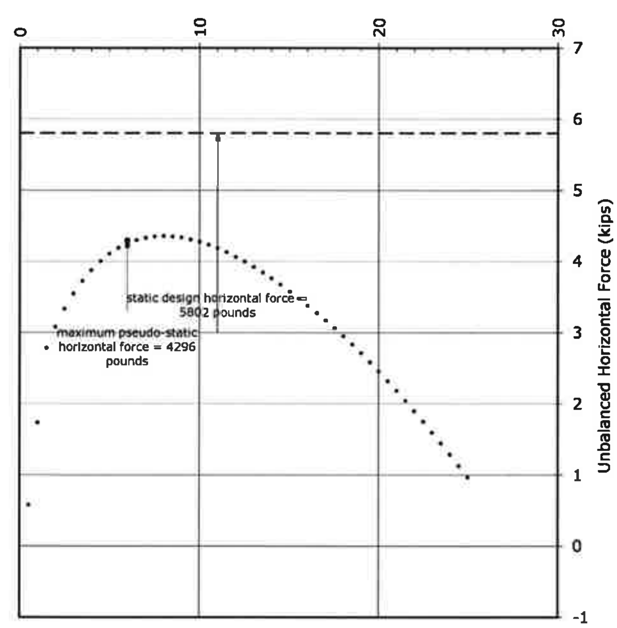
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the pseudo-static (gravitational and dynamic) driving force, Wd; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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SHORING PILE CALCULATION

BG **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#5a**
Cantilevered Shoring Pile, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED SHORING PILE. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	200.0 psf
Phi Angle, φ	32.0 degrees
Density, γ	125.0 pcf
Anisotropic Strength Function	NO
Restraining Device	SHORING PILE
Type	CANTILEVERED
Retained Height, H	16 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	NO
General Backslope Condition*	level
Loading	STATIC
Calculation Safety Factor, FS	1.25

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid	1575 trials
Trial Wedges Analyzed, Secondary Search Window	324 trials
Critical Failure Angle, α	58.4 degrees
Area of Critical Wedge	73.5 square feet
Length of Critical Failure Plane, L	13.9 feet
Depth of Critical Tension Crack	4.2 feet
Horizontal Upslope Distance to Critical Tension Crack	7.3 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	26.6 degrees
Factored Cohesion on Critical Slip Plane, C'	160.0 psf
Weight of Critical Wedge, W	9,193 pounds
External Surcharge on Critical Wedge, V	0 pounds
Static Gravitational Driving Force, W'	9,193 pounds
Mobilized Cohesive Force, C'L	2,227 pounds
Mobilized Frictional Force, R	8,587 pounds
Calculated Unbalanced Force, P	3,358 pounds
Calculated Horizontal Unbalanced Force, P_h	3,358 pounds
Calculated Equivalent Fluid Pressure	26.2 pcf

RECOMMENDED DESIGN PARAMETERS

Design Equivalent Fluid Pressure, EFP	30.0 pcf
Design Horizontal Force	3,840 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	16		
(0,16)	(0,16)			
(1,16)	(1,16)			
(2,16)	(2,16)			
(5,16)	(5,16)			
(10,16)	(10,16)			
(25,16)	(25,16)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED CANTILEVERED SHORING PILE, WITH A RETAINED HEIGHT OF UP TO 16 FEET, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE (EFP) OF 30 POUNDS PER CUBIC FOOT. FOR PILES, THE PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

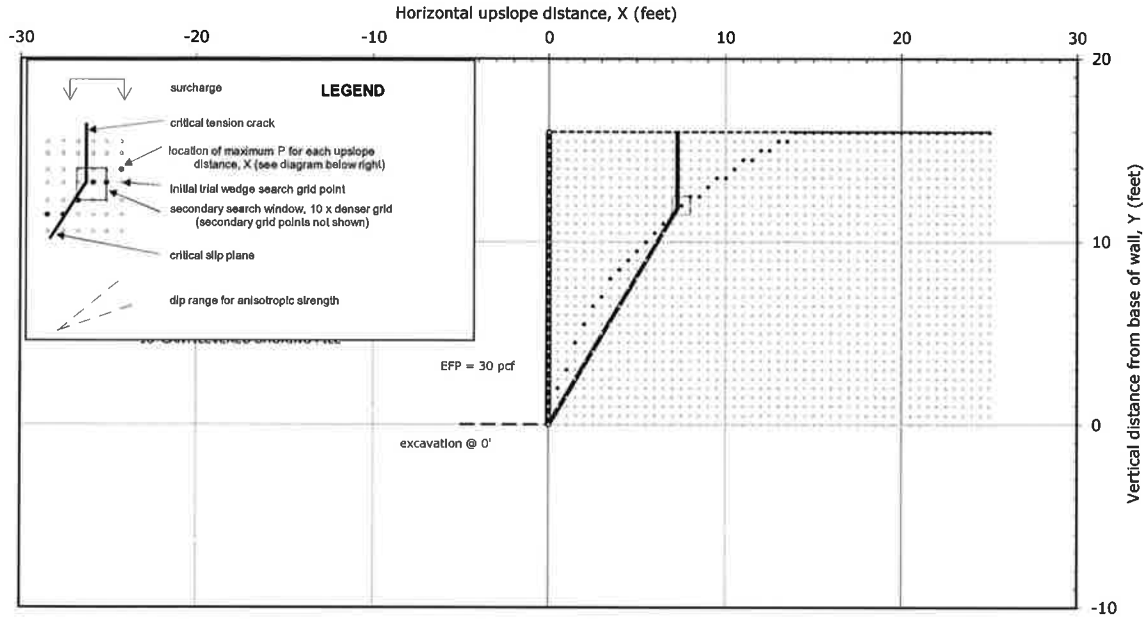
BYER GEOTECHNICAL INC.

1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

SHORING PILE CALCULATION

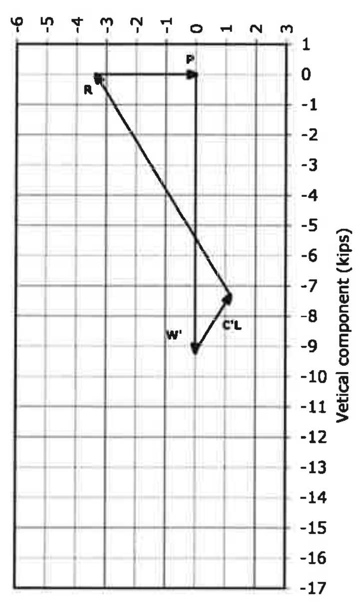
BG: **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#5b**
Cantilevered Shoring Pile, basement

Cross Section and Critical Active Wedge



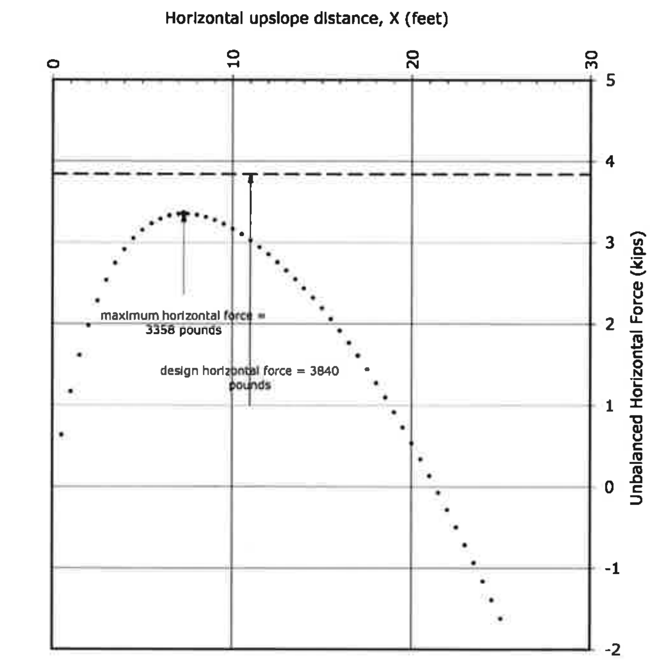
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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SHORING PILE CALCULATION

BG 23185 CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#6a**
Cantilevered Shoring Pile, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED SHORING PILE. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	200.0 psf
Phi Angle, φ	32.0 degrees
Density, γ	125.0 pcf
Anisotropic Strength Function	NO
Restraining Device	SHORING PILE
Type	CANTILEVERED
Retained Height, H	16 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	STATIC
Calculation Safety Factor, FS	1.25

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid	1575 trials
Trial Wedges Analyzed, Secondary Search Window	324 trials
Critical Failure Angle, α	55.9 degrees
Area of Critical Wedge	85.6 square feet
Length of Critical Failure Plane, L	17.2 feet
Depth of Critical Tension Crack	1.8 feet
Horizontal Upslope Distance to Critical Tension Crack	9.7 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	26.6 degrees
Factored Cohesion on Critical Slip Plane, C'	160.0 psf
Weight of Critical Wedge, W	10,705 pounds
External Surcharge on Critical Wedge, V	1,545 pounds
Static Gravitational Driving Force, W'	12,250 pounds
Mobilized Cohesive Force, C'L	2,754 pounds
Mobilized Frictional Force, R	11,437 pounds
Calculated Unbalanced Force, P	4,059 pounds
Calculated Horizontal Unbalanced Force, P_h	4,059 pounds
Calculated Equivalent Fluid Pressure	31.7 pcf

RECOMMENDED DESIGN PARAMETERS

Design Equivalent Fluid Pressure, EFP	32.0 pcf
Design Horizontal Force	4,096 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	16		
(0,16)	(0,16)			
(5,16)	(5,16)			Uniform Load: 300 psf
(15,16)	(15,16)			
(18,16)	(18,16)			
(20,16)	(20,16)			
(25,16)	(25,16)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED CANTILEVERED SHORING PILE, WITH A RETAINED HEIGHT OF UP TO 16 FEET, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE (EFP) OF 32 POUNDS PER CUBIC FOOT. FOR PILES, THE PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

BYER GEOTECHNICAL INC.

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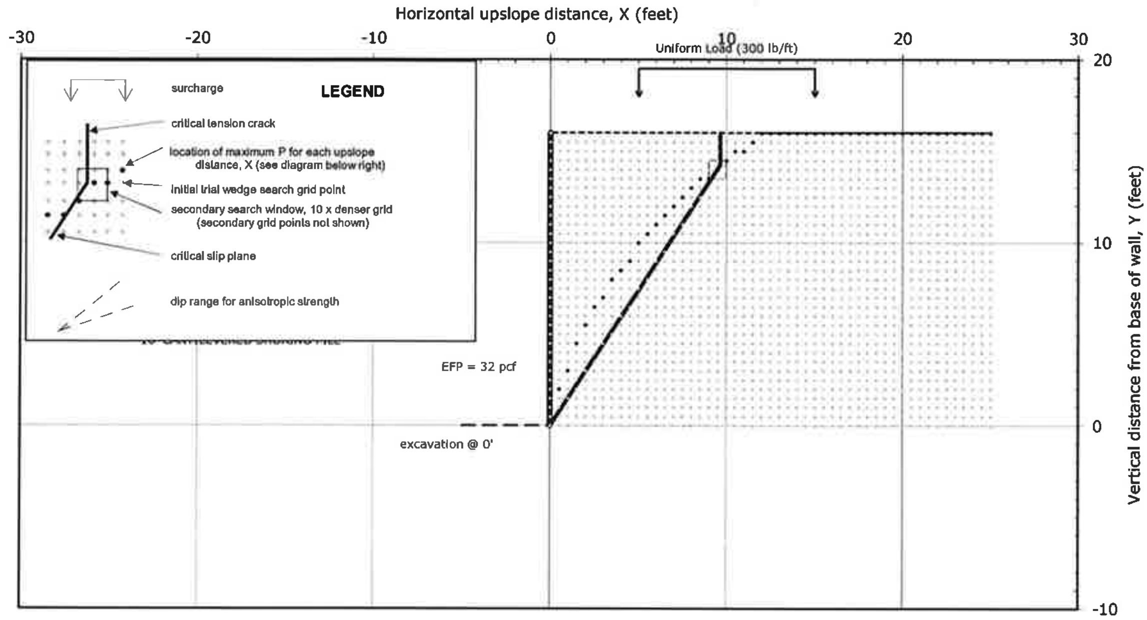
SHORING PILE CALCULATION

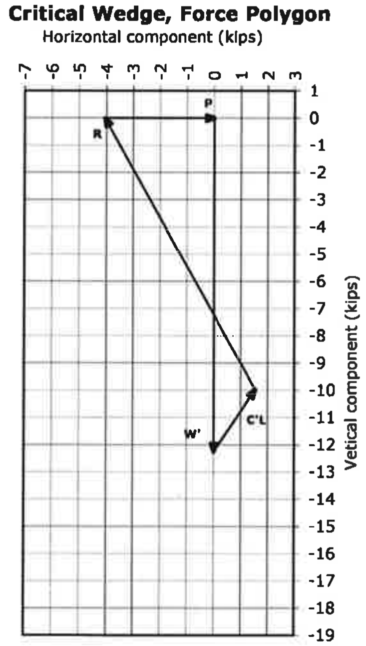
BG: **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#6b**
Cantilevered Shoring Pile, basement

Cross Section and Critical Active Wedge



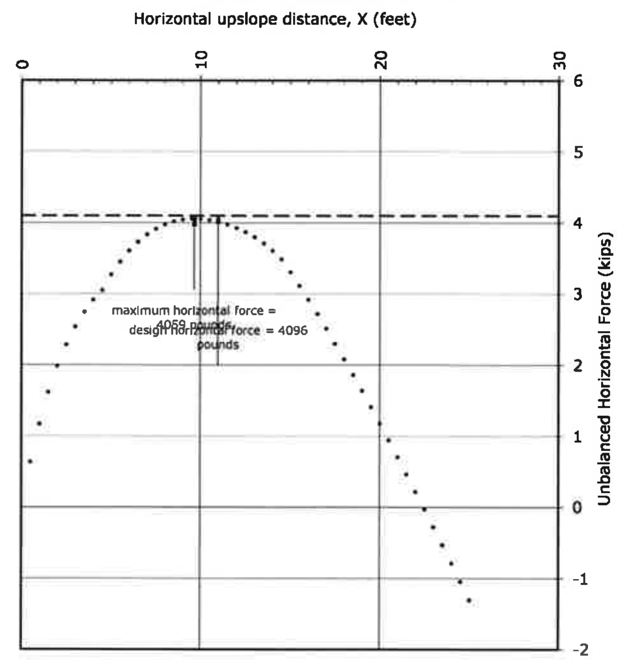
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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SHORING PILE CALCULATION

BG **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#7a**
Cantilevered Shoring Pile, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED SHORING PILE. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	200.0 psf
Phi Angle, φ	32.0 degrees
Density, γ	125.0 pcf
Anisotropic Strength Function	NO
Restraining Device	SHORING PILE
Type	CANTILEVERED
Retained Height, H	16 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	STATIC

Calculation Safety Factor, FS 1.25

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid	1575 trials
Trial Wedges Analyzed, Secondary Search Window	324 trials
Critical Failure Angle, α	61.5 degrees
Area of Critical Wedge	64.6 square feet
Length of Critical Failure Plane, L	13.4 feet
Depth of Critical Tension Crack	4.2 feet
Horizontal Upslope Distance to Critical Tension Crack	6.4 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	26.6 degrees
Factored Cohesion on Critical Slip Plane, C'	160.0 psf
Weight of Critical Wedge, W	8,080 pounds
External Surcharge on Critical Wedge, V	1,200 pounds
Static Gravitational Driving Force, W'	9,280 pounds
Mobilized Cohesive Force, C'L	2,148 pounds
Mobilized Frictional Force, R	9,020 pounds
Calculated Unbalanced Force, P	4,145 pounds
Calculated Horizontal Unbalanced Force, P_h	4,145 pounds
Calculated Equivalent Fluid Pressure	32.4 pcf

RECOMMENDED DESIGN PARAMETERS

Design Equivalent Fluid Pressure, EFP	33.0 pcf
Design Horizontal Force	4,224 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	16		
(0,16)	(0,16)			
(1,16)	(1,16)			Line Load: 1200 psf
(2,16)	(2,16)			
(18,16)	(18,16)			
(20,16)	(20,16)			
(25,16)	(25,16)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED CANTILEVERED SHORING PILE, WITH A RETAINED HEIGHT OF UP TO 16 FEET, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE (EFP) OF 33 POUNDS PER CUBIC FOOT. FOR PILES, THE PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

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INC.

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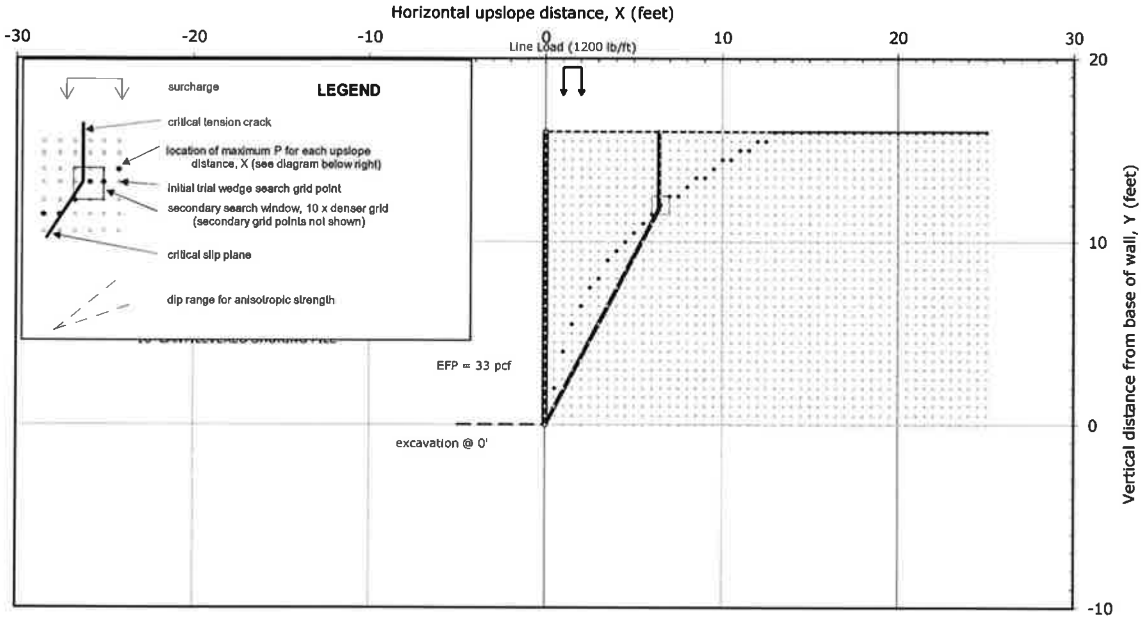
tel 818.549.9959 fax 818.543.3747

SHORING PILE CALCULATION

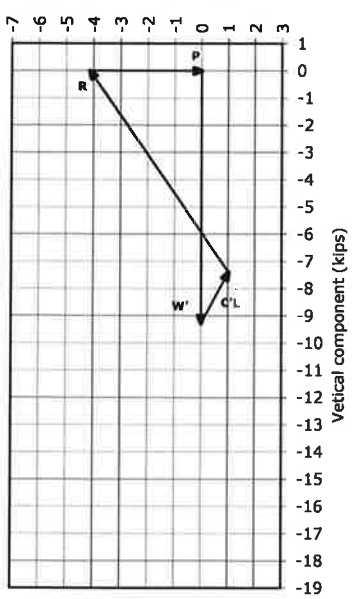
BG: **23185** CLIENT: **Lankershim Crossing, LLC**
CONSULTANT: **JHP**
SHEET: **#7b**
Cantilevered Shoring Pile, basement

Cross Section and Critical Active Wedge



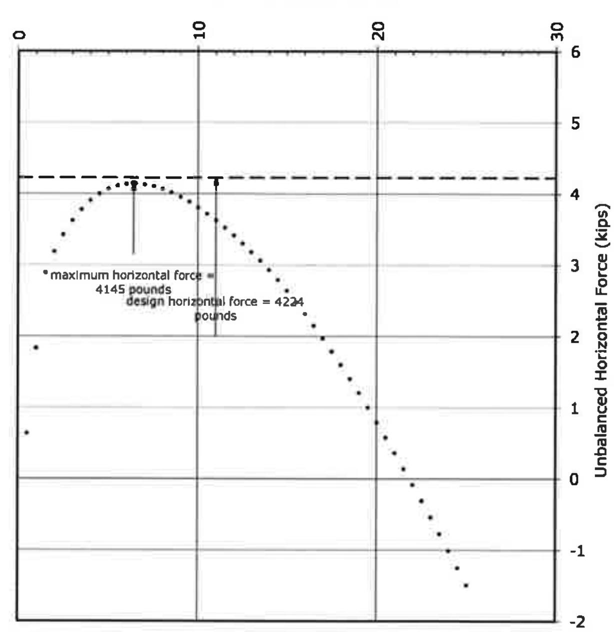
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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TEMPORARY EXCAVATION HEIGHT

BG: 23185 ENGINEER: JHP
CLIENT: Lankershim Crossing, LLC

CALCULATION SHEET # 8

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST). THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium	WALL HEIGHT:	5 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 degrees	SURCHARGE TYPE:	u Uniform
DENSITY:	125 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	160.0 psf	FINAL TENSION CRACK:	20 feet
PHID = ATAN(TAN(PHI)/FS) =		26.6 degrees	

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	51 degrees
AREA OF TRIAL FAILURE WEDGE	7.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	941.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	969 trials
LENGTH OF FAILURE PLANE	3.2 feet
DEPTH OF TENSION CRACK	2.5 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	2.0 feet
CALCULATED HORIZONTAL THRUST	**-71.8 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-5.7 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**5.0 feet**

CONCLUSIONS:

THE CALCULATION INDICATES THAT THE TEMPORARY VERTICAL EXCAVATIONS UP TO FIVE FEET HIGH WITH LEVEL BACKSLOPE HAVE A NEGATIVE THRUST AND ARE TEMPORARILY STABLE.

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TEMPORARY EXCAVATION HEIGHT

BG: 23185 ENGINEER: JHP
CLIENT: Lankershim Crossing, LLC

CALCULATION SHEET # 9

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST). THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium	WALL HEIGHT:	5 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	45 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 degrees	SURCHARGE TYPE:	u Uniform
DENSITY:	125 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	160.0 psf	FINAL TENSION CRACK:	20 feet

PHID = ATAN(TAN(PHI)/FS) = 26.6 degrees

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	51 degrees
AREA OF TRIAL FAILURE WEDGE	7.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	941.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1938 trials
LENGTH OF FAILURE PLANE	3.2 feet
DEPTH OF TENSION CRACK	3.5 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	2.0 feet
CALCULATED HORIZONTAL THRUST	**-71.8 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-9.0 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**4.0 feet**

CONCLUSIONS:

THE CALCULATION INDICATES THAT THE TEMPORARY VERTICAL EXCAVATIONS UP TO FOUR FEET HIGH WITH 1:1 BACKSLOPE HAVE A NEGATIVE THRUST AND ARE TEMPORARILY STABLE.

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SLOT CUT ANALYSIS

BG: **23185** ENGINEER: **JHP**
CLIENT: **Lankershim Crossing, LLC**

CALCULATION SHEET # **10**

CALCULATE THE FACTOR OF SAFETY OF SLOT CUT EXCAVATIONS. ASSUME COHESIVE AND FRICTIONAL RESISTANCE ALONG THE SIDES OF SLOTS AS WELL AS THE FAILURE SURFACE. THE HORIZONTAL PRESSURE ON THE SIDES OF THE SLOTS IS THE AT-REST PRESSURE (1-SIN(phi)).

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium	EXCAVATION HEIGHT:	5 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	1000 pounds
PHI ANGLE:	32 degrees	SURCHARGE TYPE:	P Point
DENSITY:	125 pcf	INITIAL FAILURE ANGLE:	20 degrees
SLOT BOUNDARY CONDITIONS		FINAL FAILURE ANGLE:	70 degrees
SLOT CUT WIDTH:	8 feet	INITIAL TENSION CRACK:	1 feet
COHESION:	200 psf	FINAL TENSION CRACK:	20 feet
PHI ANGLE:	32 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	60 degrees
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	1.0 feet
DEPTH OF TENSION CRACK	3.3 feet
TOTAL EXTERNAL SURCHARGE	1000.0 pounds
VOLUME OF FAILURE WEDGE	33.1 ft^3
WEIGHT OF FAILURE WEDGE	5134.0 pounds
LENGTH OF FAILURE PLANE	2.0 feet
SURFACE AREA OF FAILURE PLANE	16 ft^2
SURFACE AREA OF SIDES OF SLOTS	4.1 ft^2
NUMBER OF TRIAL WEDGES ANALYZED	548 trials
TOTAL RESISTING FORCE ALONG WEDGE BASE (FrB)	2147.7 pounds
TOTAL RESISTING FORCE ALONG WEDGE SIDES (FrS)	1242.7 pounds
RESULTANT HORIZONTAL COMPONENT OF FORCE	**-24.0 pounds**
CALCULATED FACTOR OF SAFETY	**1.49**

CONCLUSIONS:

THE CALCULATION INDICATES THAT SLOTS CUTS UP TO 8 FEET WIDE AND 5 FEET HIGH IN NATURAL ALLUVIUM HAVE A SAFETY FACTOR GREATER THAN 1.25 AND ARE TEMPORARILY STABLE.

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AERIAL VICINITY MAP

BG: 23185 LANKERSHIM CROSSING, LLC

CONSULTANT: JHP/RSB

DRAWN BY: AS

SCALE: 1" = 200'

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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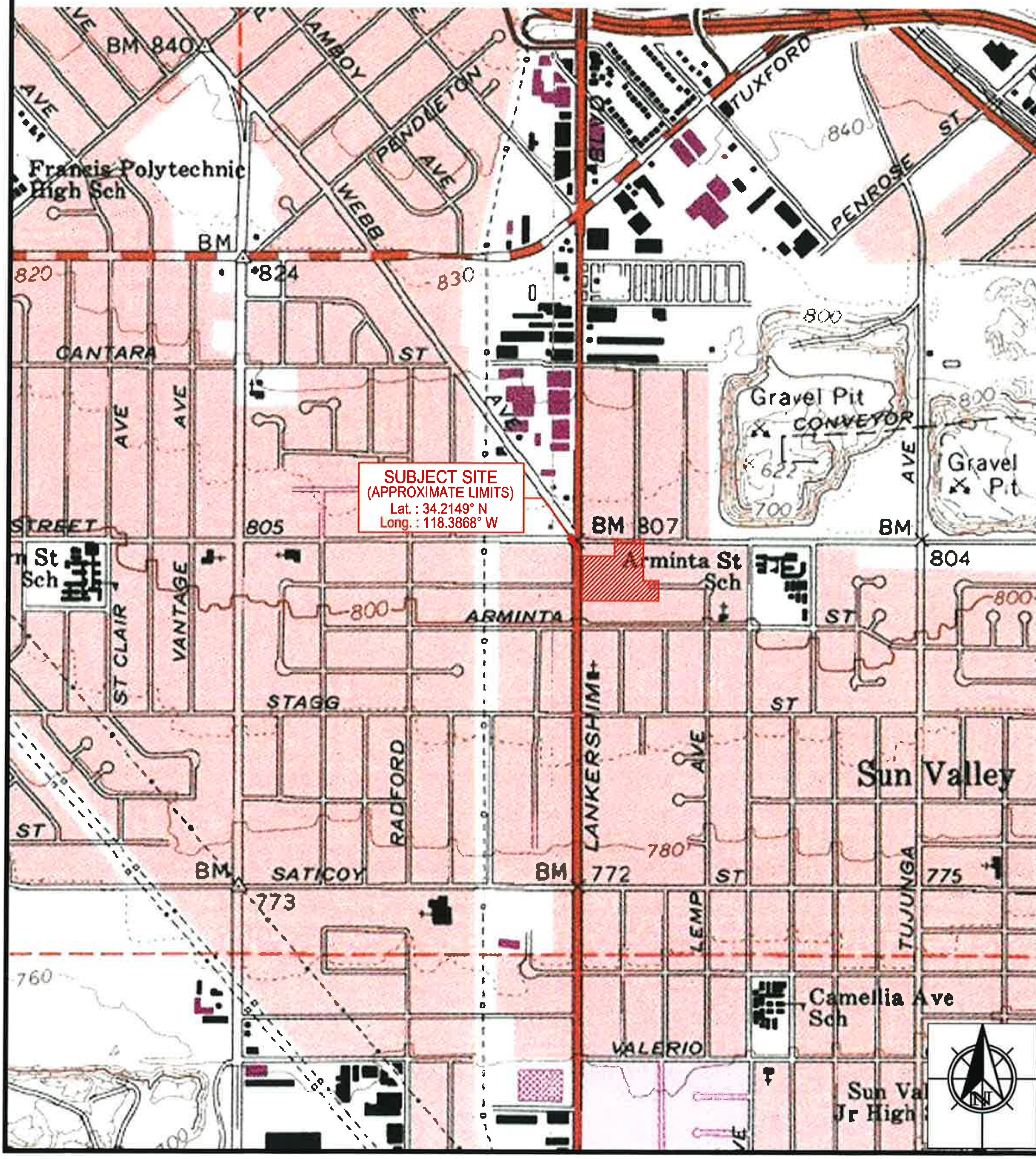
REGIONAL TOPOGRAPHIC MAP

BG: 23185	LANKERSHIM CROSSING, LLC
CONSULTANT : JHP/RSB	SCALE: 1" = 1000'
DRAWN BY : AS	

REFERENCE: USGS TOPOGRAPHIC MAP, VAN NUYS 7.5-MINUTE SERIES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA CREATED 1981.



PLOT DATE/TIME: 2/28/2020 - 1:20pm

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HISTORIC TOPOGRAPHIC MAP

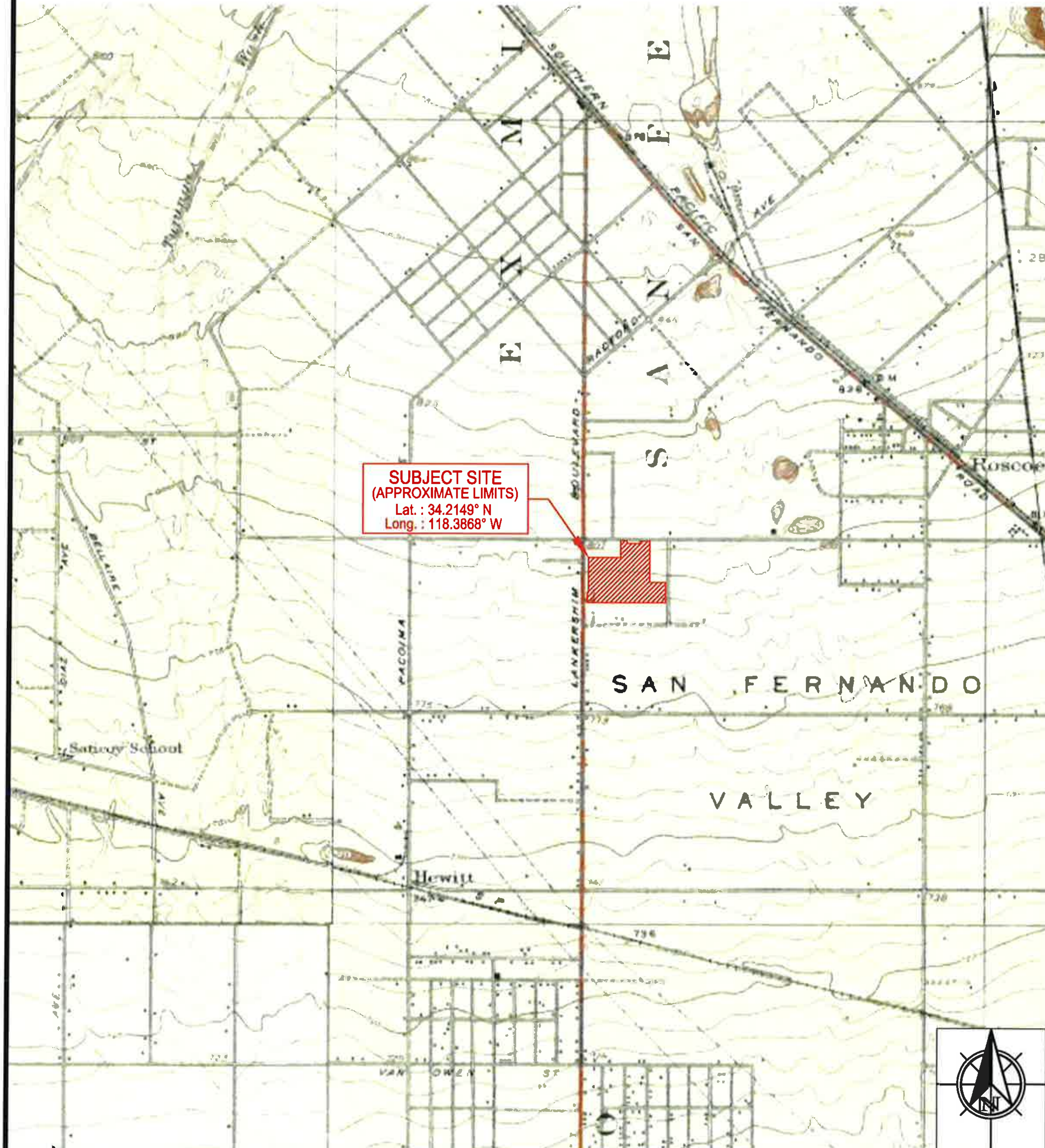
BG: 23185 LANKERSHIM CROSSING, LLC

CONSULTANT : JHP/RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: USGS TOPOGRAPHIC MAP, SUNLAND 6-MINUTE SERIES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA CREATED 1926.



PLOT DATE/TIME: 2/28/2020 - 1:20pm

CAD FILE: S:\LandProjects\23185 Lankershim Crossing_No Ho\DWG\HISTORIC TOPOGRAPHIC MAP.dwg

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REGIONAL GEOLOGIC MAP #1

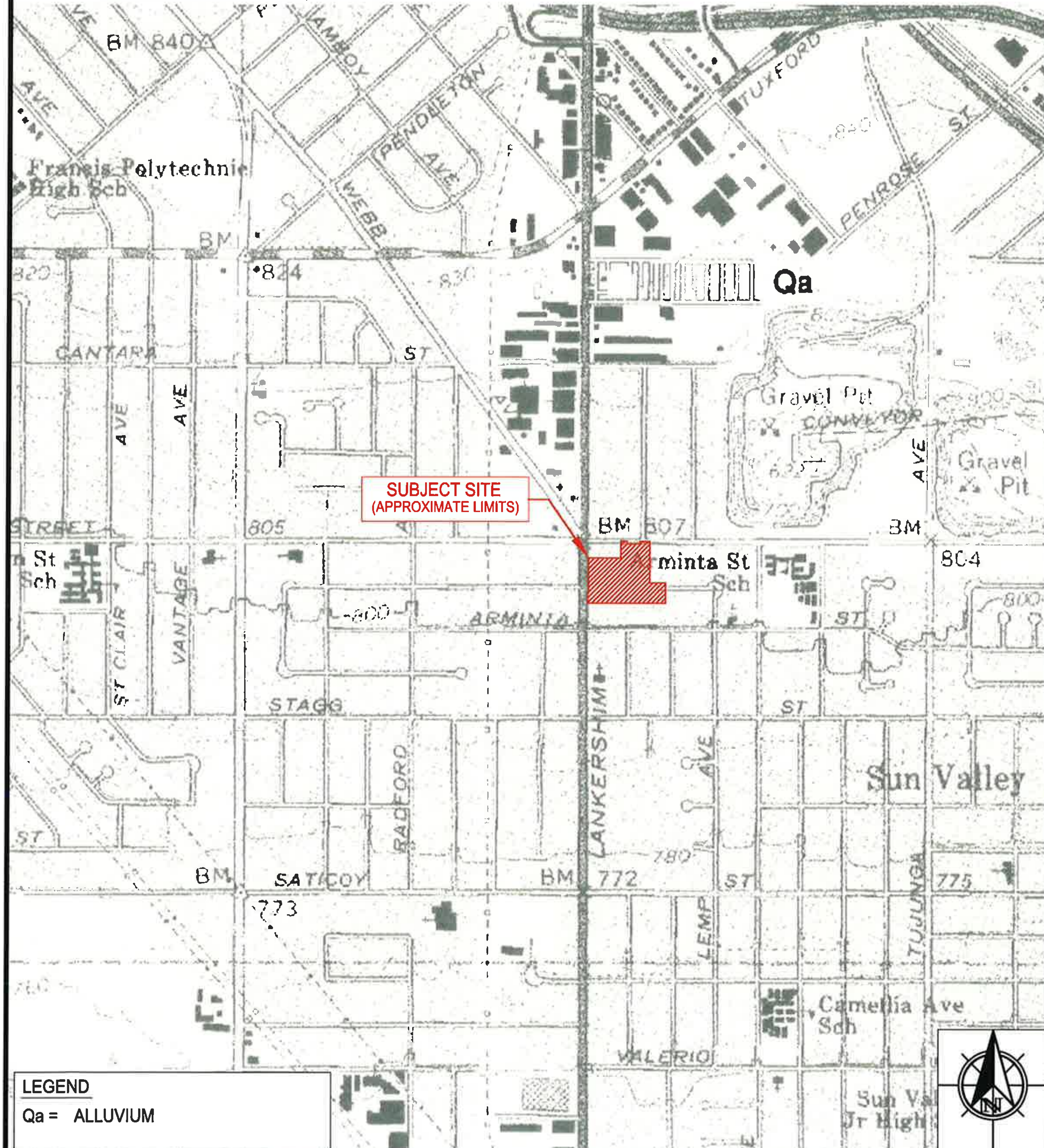
BG: 23185 LANKERSHIM CROSSING, LLC

CONSULTANT : JHP/RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: DIBBLEE, T.W. (1991), GEOLOGIC MAP OF SAN FERNANDO AND VAN NUYS (NORTH 1/2) QUADRANGLES, LOS ANGELES, CALIFORNIA DIBBLEE GEOLOGICAL FOUNDATION, MAP DF-33.



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REGIONAL GEOLOGIC MAP #2

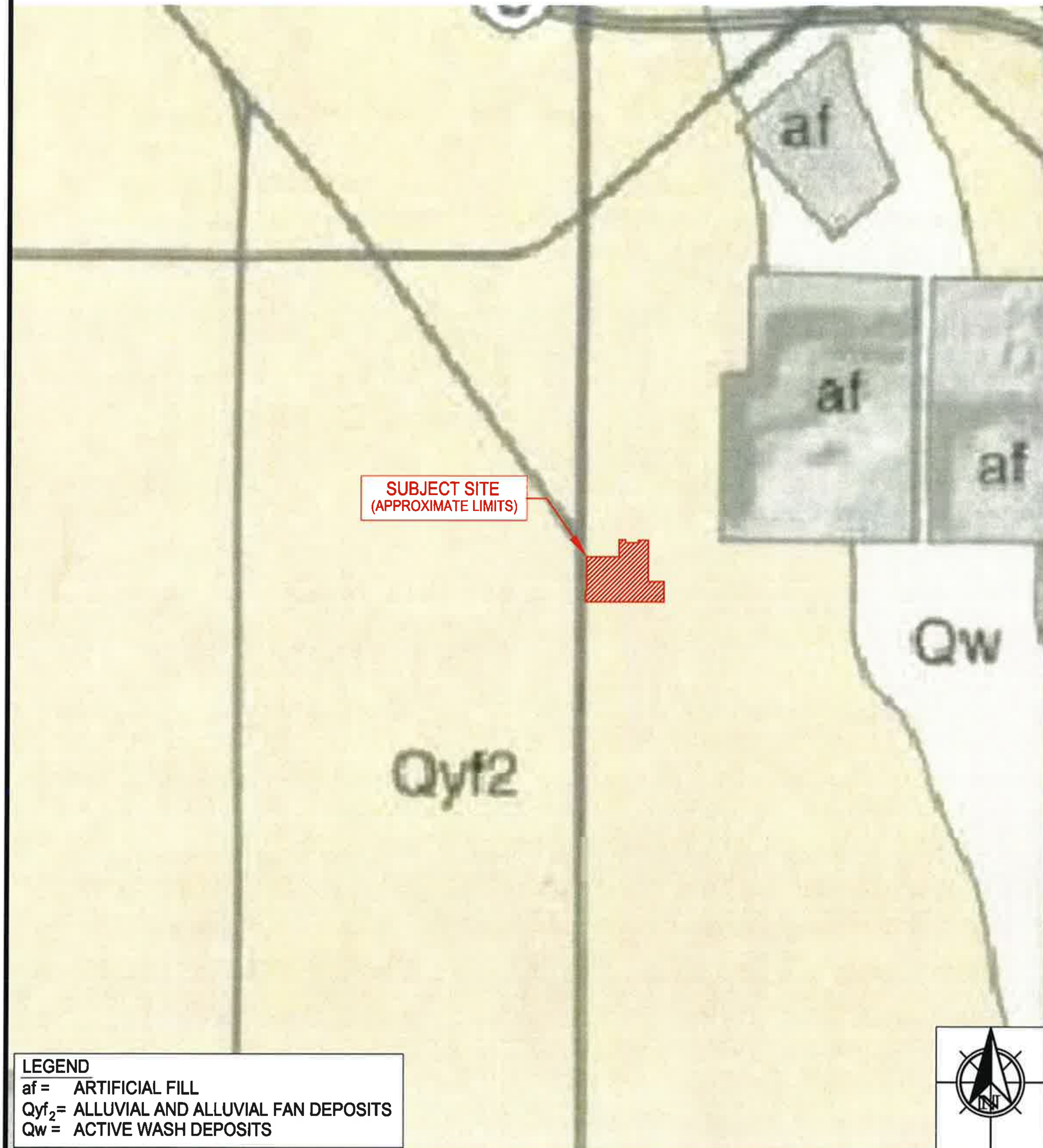
BG: 23185 LANKERSHIM CROSSING, LLC

CONSULTANT : JHP/RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: QUATERNARY GEOLOGY OF THE SAN FERNANDO VALLEY, LOS ANGELES COUNTY, CALIFORNIA 2000 BY C.S. HITCHCOCK AND C.J. WILLS.



PLOT DATE/TIME: 3/25/2020 – 10:28am

CAD FILE: S:\LandProjects\23185 Lankershim Crossing_No Ho\DWG\REGIONAL GEOLOGIC MAP #2.dwg

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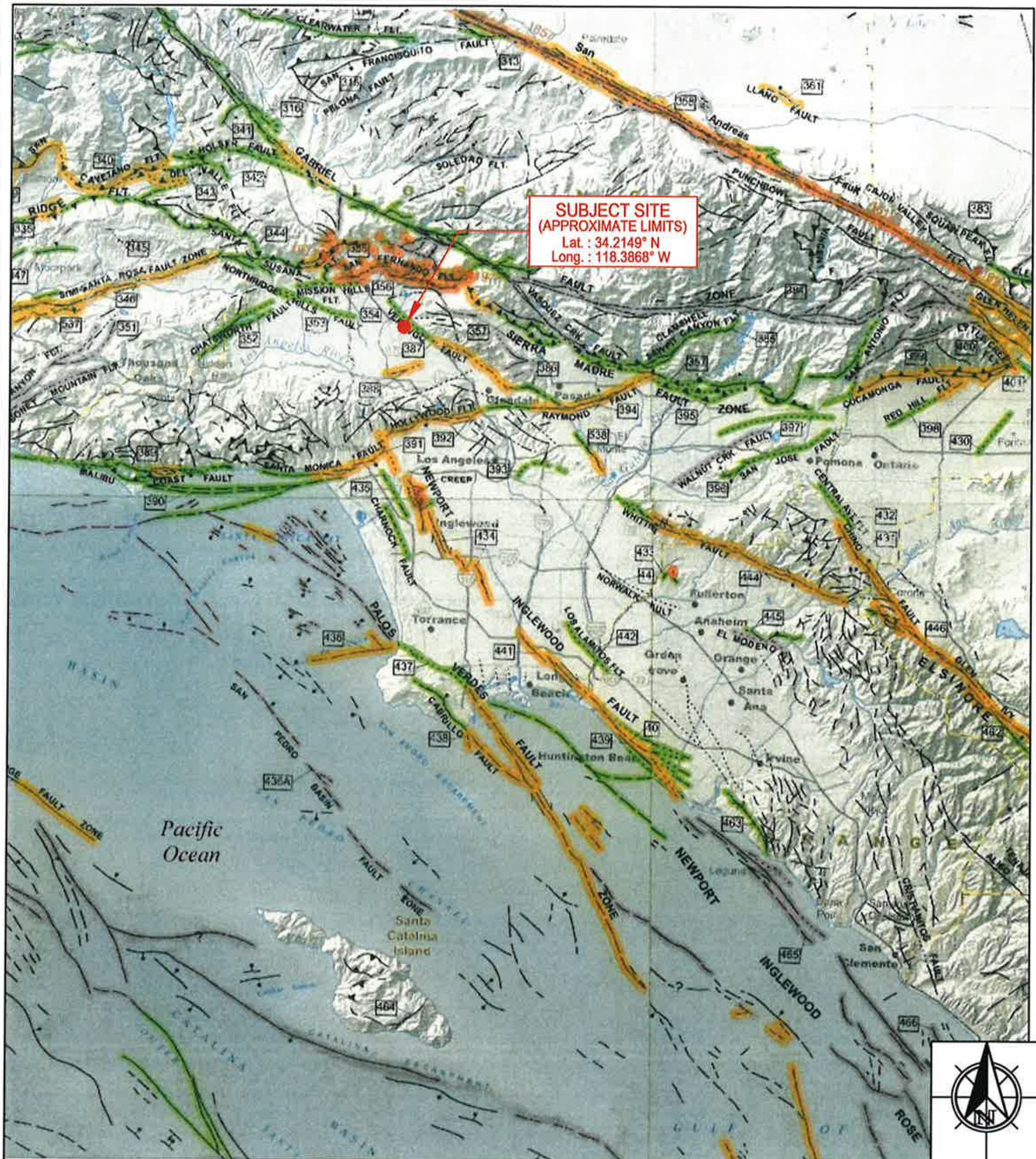
818.543.3747 FAX

REGIONAL FAULT MAP

BG: 23185 LANKERSHIM CROSSING, LLC

CONSULTANT : JHP/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.



CAD FILE: S:\LandProjects\23185 Lankershim Crossing_No 1\DWG\REGIONAL FAULT MAP.dwg PLOT DATE/TIME: 2/28/2020 - 1:20pm

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

SEISMIC HAZARD ZONES MAP

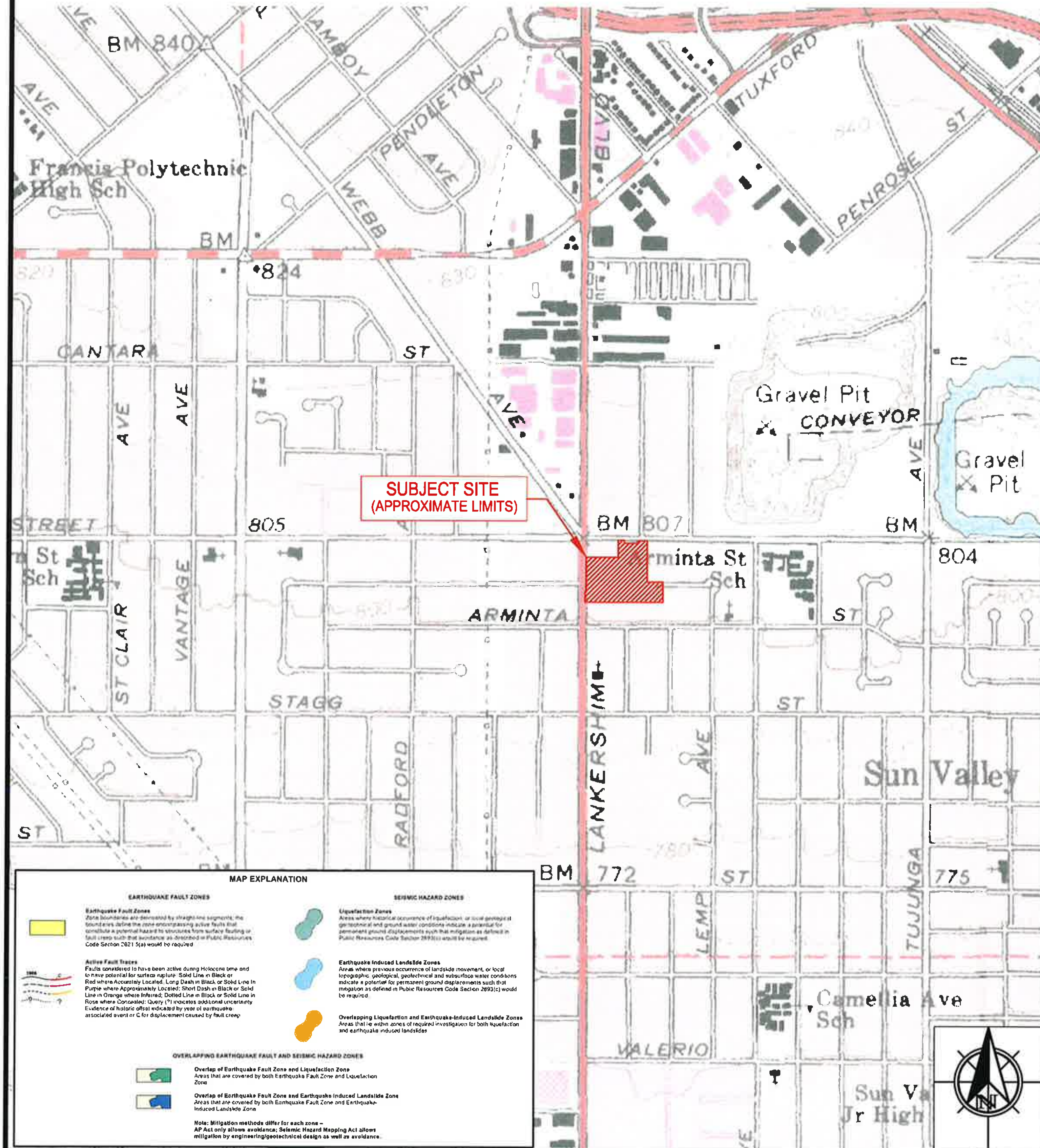
BG: 23185 LANKERSHIM CROSSING, LLC

CONSULTANT : JHP/RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION VAN NUYS QUADRANGLE; SEISMIC HAZARD ZONES, DATED FEBRUARY 01, 1998.



CAD FILE: S:\LandProjects\23185 Lankershim Crossing_No Ho\DWG\SEISMIC HAZARD ZONES MAP.dwg PLOT DATE/TIME: 2/28/2020 - 1:21pm

BYER GEOTECHNICAL INC.

1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
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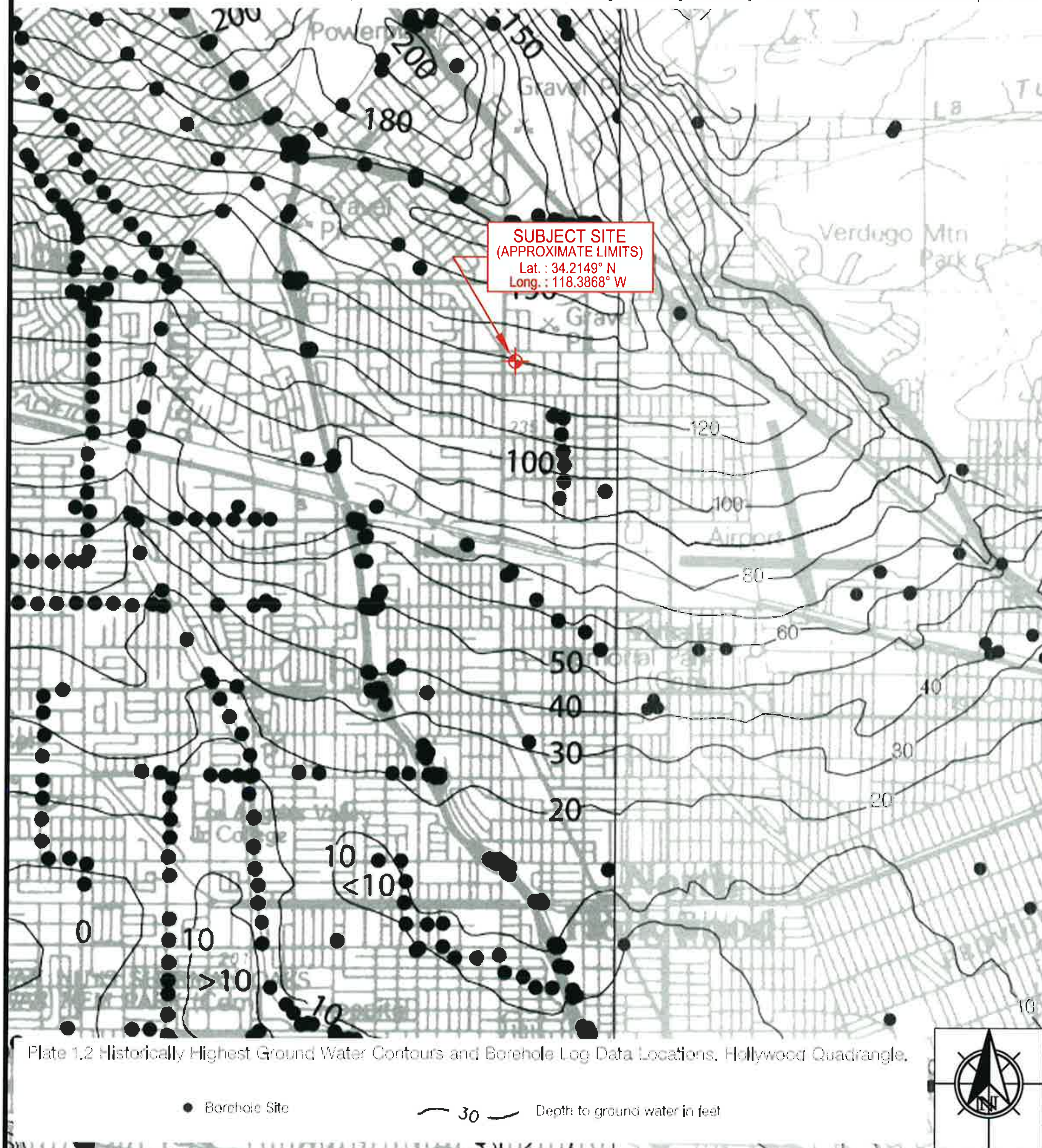
HISTORIC-HIGH GROUNDWATER MAP

BG: 23185	LANKERSHIM CROSSING, LLC
CONSULTANT : JHP/RSB	SCALE: 1" = 4000'
DRAWN BY : AS	

REFERENCE: CGS, 1997, Seismic Hazard Zone Report for the Van Nuys 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 08, and CGS, 1998, Seismic Hazard Zone Report for the Burbank 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 016.



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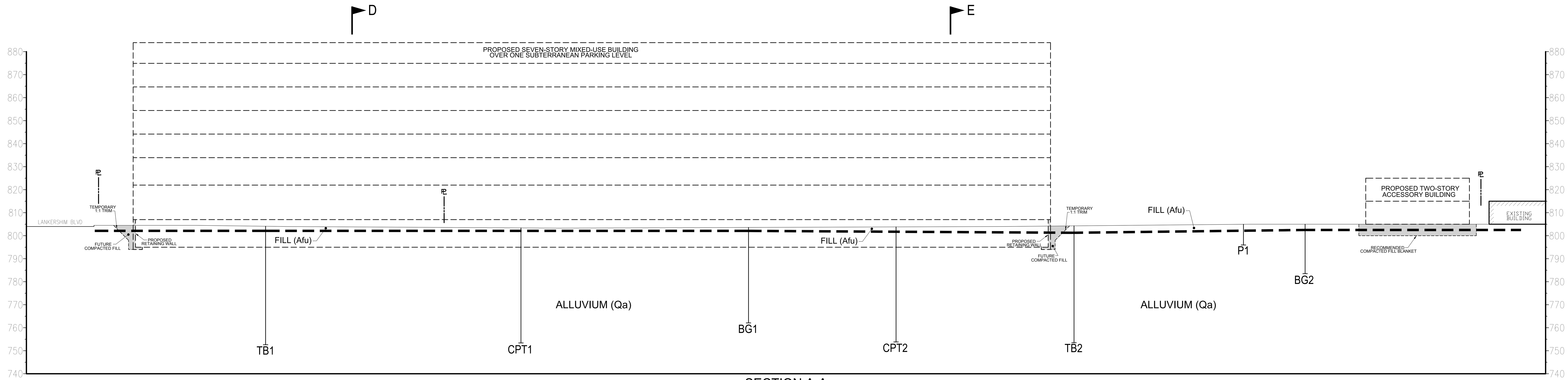
818.543.3747 FAX

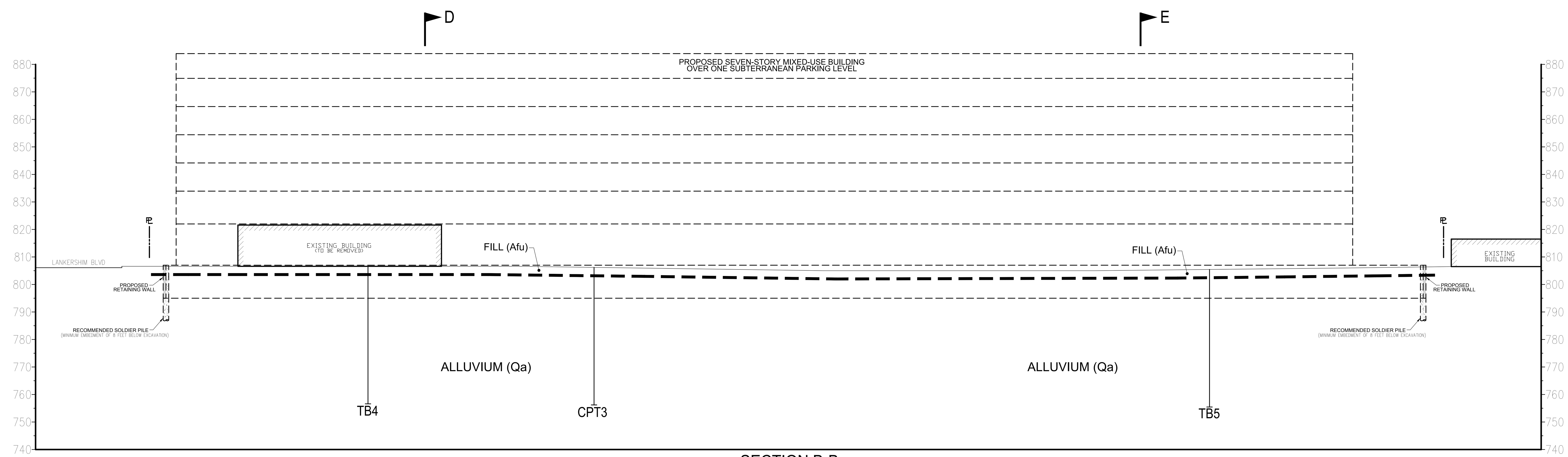
SITE PLAN

BG: 23185 LANKERSHIM CROSSING, LLC

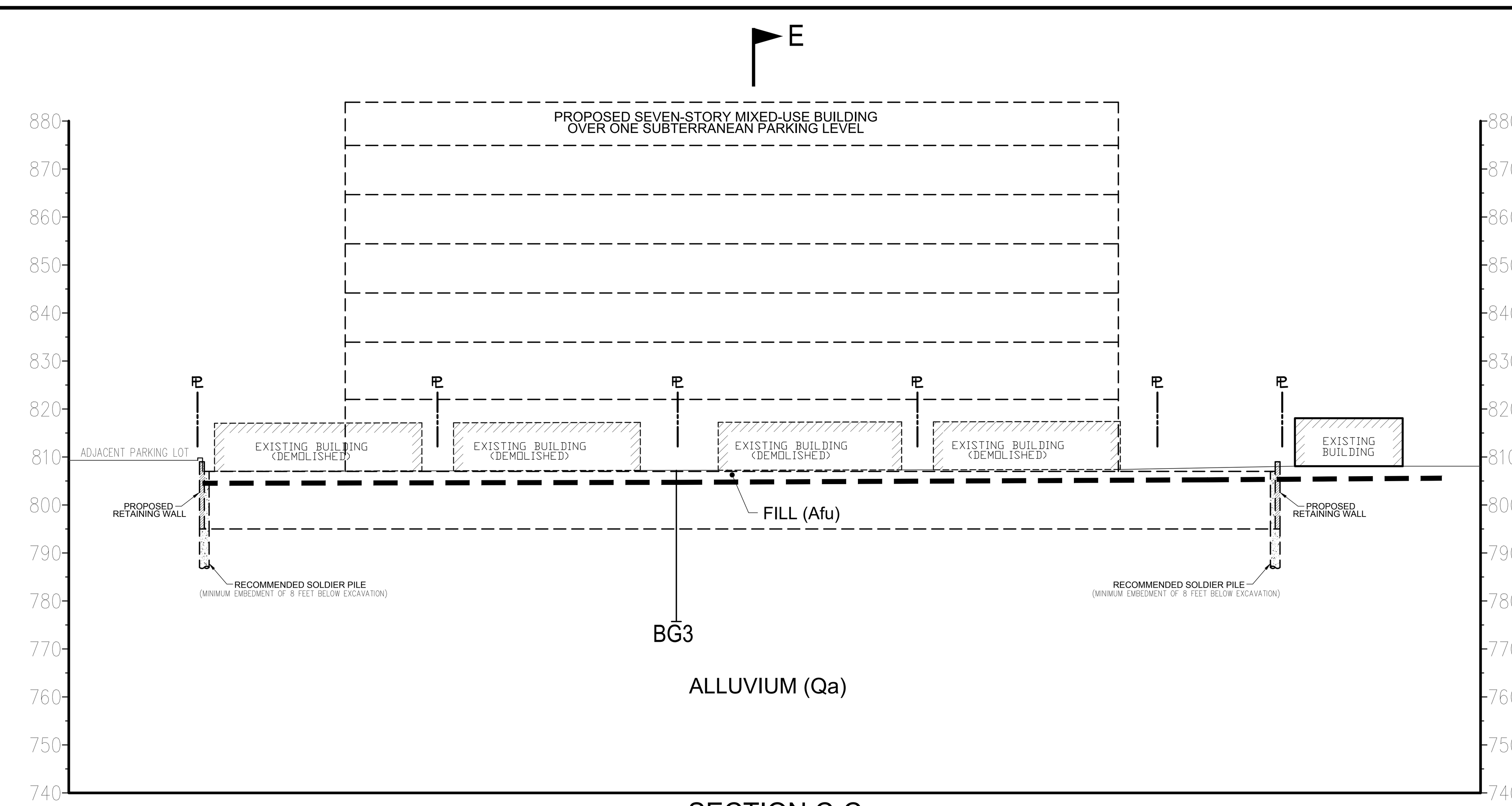
CONSULTANT : JHP/RSB
DRAWN BY : AS
SCALE: 1" = 20'



SECTION A-A



SECTION B-B

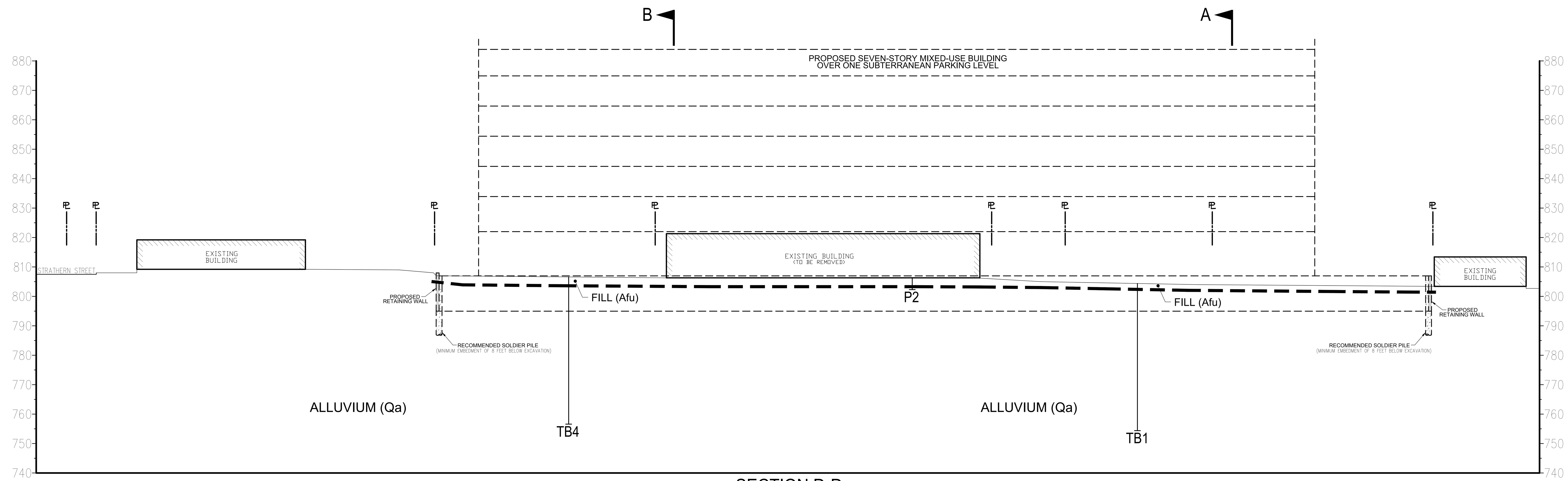


SECTION C-C

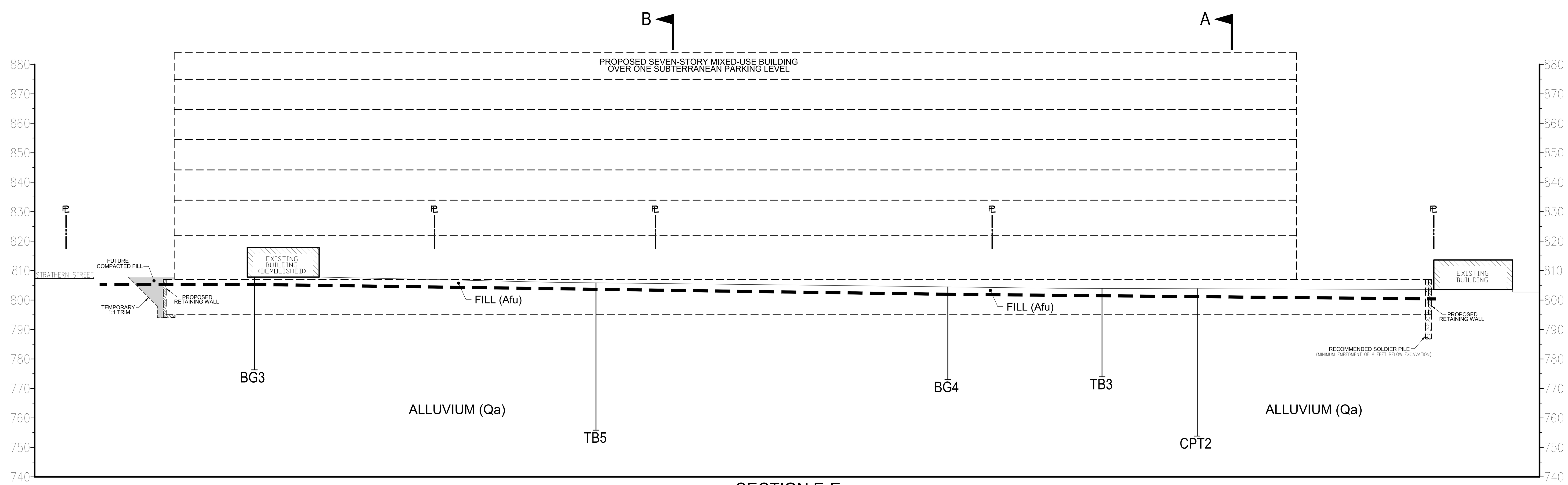
MARCH 30, 2020

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

SECTIONS A, B, & C	
BG: 23185 LANKERSHIM CROSSING, LLC	
CONSULTANT : JHP/RSB	SCALE: 1" = 20'
DRAWN BY : AS	



SECTION D-D



SECTION E-E

MARCH 30, 2020

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

SECTIONS D & E

BG: 23185 LANKERSHIM CROSSING, LLC

CONSULTANT : JHP/RSB

DRAWN BY : AS

SCALE: 1" = 20'

CAD FILE: S:\LandProjects\23185 Lankershim Crossing_No Ho\DWG\SITE PLAN_022...

CITY OF LOS ANGELES
CALIFORNIA

ERIC GARCETTI
MAYOR

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

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

SOILS REPORT APPROVAL LETTER

April 28, 2020

LOG # 112843
SOILS/GEOLOGY FILE - 2

Lankershim Crossing, LLC
23622 Calabasas Road, Suite 121
Calabasas, CA 91302

TRACT:	Lankershim Ranch Land and Water Co. (M R 31-39/44)
LOT(S):	PT 24 (Arb.'s 5, 6, 7, 8, 9, 10, 11 & 25) // PT 24 (Arb.'s 5, 13, 37 & 42)
LOCATION:	7918 - 7946 N. Lankershim Blvd. // 11650 - 11664 W. Strathern St.

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Soils Report	BG 23185	03/30/2020	Byer Geotechnical, Inc.
Oversized Doc(s).	"	"	"

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed 7-story mixed-use building over 1-subterranean parking level, and a 2-story at-grade recreation building, as shown on the Site Plan and Cross Sections A through E of the 03/30/2020 report. Retaining walls up to 14 feet high are proposed per the consultants.

Four borings were performed to depths ranging from 21.5 to 41.5 feet. In addition, five borings and three CPT's to depths ranging from 31.5 to 51.5 feet were previously performed by another consultant, along with two shallow percolation tests. The earth materials at the subsurface exploration locations consist of up to 3 feet of uncertified fill underlain by alluvium. According to the consultants, groundwater was not encountered to the maximum depths explored of 51.5 feet, and historically highest groundwater level is on the order of 130 feet below the ground surface. The site is relatively level.

The consultants recommend to support the proposed structure(s) on conventional foundations bearing on native undisturbed soils (for the 7-story building over 1-subterranean parking level), and the 2-story at-grade recreation building on a blanket of properly placed fill a minimum of 3 feet thick below the bottom of the footings.

The referenced report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2020 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. The entire site shall be brought up to the current Code standard (7005.9).

2. Approval shall be obtained from the Department of Public Works, Bureau of Engineering, Development Services and Permits Program for the proposed removal of support and/or retaining of slopes adjoining to public way (3307.3.2).

   6262 Van Nuys Blvd. Ste 351, Van Nuys (818) 374-4605

3. The soils engineer shall review and approve the detailed plans prior to issuance of any permit. This approval shall be by signature on the plans that clearly indicates the soils engineer has reviewed the plans prepared by the design engineer; and, that the plans included the recommendations contained in their reports (7006.1).

4. All recommendations of the report that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

5. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans (7006.1). Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.

6. A grading permit shall be obtained for all structural fill and retaining wall backfill (106.1.2).

7. 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.

8. If import soils are used, no footings shall be poured until the soils engineer has submitted a compaction report containing in-place shear test data and settlement data to the Grading Division of the Department; and, obtained approval (7008.2).

9. Compacted 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 (7011.3).

10. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill (1809.2, 7011.3).

11. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction (7013.12).

12. Grading shall be scheduled for completion prior to the start of the rainy season, or detailed temporary erosion control plans shall be filed in a manner satisfactory to the Grading Division of the Department and the Department of Public Works, Bureau of Engineering, B-Permit Section, for any grading work in excess of 200 cubic yards (7007.1).

13. All loose foundation excavation material shall be removed prior to commencement of framing (7005.3).

14. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the General Safety Orders of the California Department of Industrial Relations (3301.1).

15. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring or constructed using ABC slot cuts, as recommended. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

16. Prior to the issuance of any permit that authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation (3307.1).

17. The soils engineer shall review and approve the shoring plans prior to issuance of the permit (3307.3.2).

18. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

19. Unsurcharged temporary excavations exposing fill shall be trimmed back at a gradient not exceeding 1:1, as recommended.

20. Unsurcharged temporary excavation exposing alluvium may be cut vertical up to 5 feet with level backslope, as recommended.

21. Unsurcharged temporary excavation exposing alluvium may be cut vertical up to 4 feet. For excavations over 4 feet, the lower 4 feet may be cut vertically and the portion of the excavation above 4 feet shall be trimmed back at a gradient not exceeding 1:1, as recommended.

22. Shoring shall be designed for the lateral earth pressures as specified in the section titled "Soldier Piles" on page 21 of the 03/30/2020 report; all surcharge loads shall be included into the design. Total lateral load on shoring piles shall be determined by multiplying the recommended EFP by the pile spacing.

23. Shoring shall be designed for a maximum lateral deflection of 1 inch, provided there are no structures within a 1:1 plane projected up from the base of the excavation. Where a structure is within a 1:1 plane projected up from the base of the excavation, shoring shall be designed for a maximum lateral deflection of ½ inch, or to a lower deflection determined by the consultant that does not present any potential hazard to the adjacent structure.

24. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.

25. Surcharged ABC slot-cut method may be used for temporary excavations with each slot-cut not exceeding 5 feet in height and not exceeding 8 feet in width, as recommended. The surcharge load shall not exceed the value given in the report. The soils engineer shall determine the clearance between the excavation and the existing foundation. The soils engineer shall verify in the field if the existing earth materials are stable in the slot-cut excavation. Each slot shall be inspected by the soils engineer and approved in writing prior to any worker access.

26. All foundations shall derive entire support from native undisturbed soils, or a blanket of properly placed fill (a minimum of 3 feet thick below the bottom of the footings), as recommended and approved by the soils engineer by inspection.

27. Footings supported on approved compacted fill shall be reinforced with a minimum of four (4), ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top of the footing.

28. Slabs placed on approved compacted fill shall be at least 3½ inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced a maximum of 16 inches on center each way.

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

30. Cantilevered retaining walls up to 14 feet in height with a level backfill shall be designed for a minimum equivalent fluid pressure (EFP) of 43 PCF, as specified on page 17 of the 03/30/2020 report. All surcharge loads shall be incorporated into the design.

31. Retaining walls higher than 6 feet shall be designed for lateral earth pressure due to earthquake motions as specified on page 18 of the 03/30/2020 report (1803.5.12).

32. Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure as specified on page 18 of the 03/30/2020 report (1610.1). All surcharge loads shall be included into the design.

33. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted in a non-erosive device to the street in an acceptable manner (7013.11).

34. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls 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 soils report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record (1805.4).

35. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector (108.9).

36. Basement walls and floors shall be waterproofed/damp-proofed with an LA City approved "Below-grade" waterproofing/damp-proofing material with a research report number (104.2.6).

37. Prefabricated drainage composites (Miradrain, Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.

38. The structures shall be connected to the public sewer system per P/BC 2020-027.

39. The infiltration facility design and construction shall comply with the minimum requirements specified in the Information Bulletin P/BC 2020-118.

40. The infiltration system (dry well) shall be constructed within the landscaping area at the southeast portion of the site, as recommended on pages 26 and 27 of the 03/30/2020 report.

41. Infiltration shall occur below a depth of 10 feet, as recommended.

42. The construction of the infiltration system shall be provided under the inspection and approval of the soils engineer.

43. An overflow outlet shall be provided to conduct water to the street in the event that the infiltration system capacity is exceeded. (P/BC 2020-118)

44. Approval for the proposed infiltration system from the Bureau of Sanitation, Department of Public Works shall be secured.

45. A minimum distance of 10 feet (in any direction) shall be provided from adjacent proposed/existing footings to the discharge of the proposed infiltration system. A minimum distance of 10 feet horizontally shall be provided from private property lines to the proposed infiltration system.

46. The dry well area between the blank casing and the surround soils shall be sealed to a minimum depth of 10 feet below the bottom of any adjacent foundation with bentonite slurry (or equivalent) to prevent unintended leakage or horizontal infiltration.

47. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS (7013.10).

48. The soils engineer 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 (7008, 1705.6 & 1705.8).

49. Prior to pouring concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the work inspected meets the conditions of the report. No concrete shall be poured until the LADBS Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)

50. Prior to excavation an initial inspection shall be called with the LADBS Inspector. During the initial inspection, the sequence of construction; shoring; ABC slot cuts; protection fences; and, dust and traffic control will be scheduled (108.9.1).

51. Installation of shoring and/or slot cutting shall be performed under the inspection and approval of the soils engineer and deputy grading inspector (1705.6, 1705.8).

52. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the soil inspected meets the conditions of the report. No fill shall be placed until the LADBS Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be included 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. In addition, an Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included (7011.3).

53. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.

GLEN RAAD
Geotechnical Engineer I

Log No. 112843
213-482-0480

cc: GP Design Group, LLC, Applicant
Byer Geotechnical, Inc., Project Consultant
VN District Office