

Appendix E

Geotechnical Report

JAVIER NUNEZ
VICE PRESIDENT

JOSELYN GEAGA-ROSENTHAL
LAUREL GILLETTE
GEORGE HOVAGUIMIAN
ELVIN W. MOON



ERIC GARCETTI
MAYOR

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

JOHN WEIGHT
EXECUTIVE OFFICER

SOILS REPORT APPROVAL LETTER

July 12, 2022

LOG # 122052
SOILS/GEOLOGY FILE - 2

Bardas Investment Group
1015 N. Fairfax Avenue
Los Angeles, CA 90046

TRACT: STRONG AND DICKINSON'S HOLLYWOOD HIGH SCHOOL TRACT
(M P 7-79)
BLOCK: B
LOT(S): 3-5, 20-23
LOCATION: 1139-1155 N LAS PALMAS AVE, 1138-1150 N MCCADDEN PL

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Soils Report	2618-14	12/21/2021	Feffer Geological
Oversized Doc (s)	"	"	"

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed three-story building over three subterranean levels. The earth materials at the subsurface exploration locations consist of up to 2 feet of uncertified fill underlain by silts, sands, clay and gravels.

The consultants recommend to support the proposed structures on conventional, mat-type and/or drilled-pile foundations bearing on competent native soils. At-grade structures shall be supported on minimum three feet of compacted fill.

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. Provide a notarized letter from all adjoining property owners allowing tie-back anchors on their property (7006.6).
2. 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).

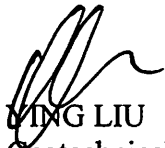
3. All recommendations of the report that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.
4. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans (7006.1). Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.
5. A grading permit shall be obtained for all structural fill and retaining wall backfill (106.1.2).
6. 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.
7. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill (1809.2, 7011.3).
8. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction (7013.12).
9. 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).
10. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring. 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)
11. 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).
12. The soils engineer shall review and approve the shoring and/or underpinning plans prior to issuance of the permit (3307.3.2).
13. 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.
14. Unsurcharged temporary excavations over 5 feet exposing soil shall be trimmed back at a gradient not exceeding 1:1, as recommended.

15. Shoring shall be designed for a minimum EFP of 43 PCF; all surcharge loads shall be included into the design, as recommended.
16. 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.
17. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.
18. All foundations shall derive entire support from native undisturbed soils or minimum three feet of compacted fill, as recommended and shall be approved by the geologist and soils engineer by inspection.
19. The proposed structure and subterranean walls shall be designed to resist uplift and hydrostatic pressures that would develop due to the historic high groundwater level conditions or the current groundwater level, whichever is higher.
20. Footings supported on approved compacted fill or expansive soil 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.
21. The foundation/slab design shall satisfy all requirements of the Information Bulletin P/BC 2017-116 “Foundation Design for Expansive Soils” (1803.5.3).
22. Pile caisson and/or isolated foundation ties are required by LAMC Sections 91.1809.13 and/or 91.1810.3.13. Exceptions and modification to this requirement are provided in Information Bulletin P/BC 2020-030.
23. When water is present in drilled pile holes, the concrete shall be tremied from the bottom up to ensure minimum segregation of the mix and negligible turbulence of the water (1808.8.3).
24. Existing uncertified fill shall not be used for lateral support of deep foundations (1810.2.1).
25. Slabs placed on approved compacted fill shall be at least 5 inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced a maximum of 16 inches on center each way.
26. Concrete floor slabs placed on expansive soil shall be placed on a 4-inch fill of coarse aggregate or on a moisture barrier membrane.
27. 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.
28. Retaining walls shall be designed for the lateral earth pressures specified in the section titled “Retaining Walls” starting on page 20 of the 12/21/2021 report. All surcharge loads shall be included into the design.

29. 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).
30. 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).
31. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector (108.9).
32. 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).
33. Prefabricated drainage composites (Miradrain, Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.
34. All roof, pad and deck drainage shall be conducted to the street in an acceptable manner in non-erosive devices or other approved location in a manner that is acceptable to the LADBS and the Department of Public Works (7013.10).
35. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS (7013.10).
36. Groundwater conditions shall be assessed prior to construction, as recommended on page 17 of the 12/21/2021 report. In the event that temporary dewatering is planned, a supplemental report addressing dewatering and its impact on adjacent structures / properties shall be submitted to the Department for review and approval.
37. 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).
38. All friction pile or caisson drilling and excavations shall be performed under the inspection and approval of the geologist and soils engineer. The geologist shall indicate the distance that friction piles or caissons penetrate into competent [material] bedrock in a written field memorandum. (1803.5.5, 1705.1.2)
39. 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)
40. 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; underpinning;

pile installation;] protection fences; and, dust and traffic control will be scheduled (108.9.1).

41. Installation of shoring, underpinning, slot cutting and/or pile excavations shall be performed under the inspection and approval of the soils engineer and deputy grading inspector (1705.6, 1705.8).
42. The installation and testing of tie-back anchors shall comply with the recommendations included in the report or the standard sheets titled "Requirement for Tie-back Earth Anchors", whichever is more restrictive. [Research Report #23835]
43. 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).
44. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.



MING LIU
Geotechnical Engineer II

Log No. 122052
213-482-0480

cc: Feffer Geological, Project Consultant
LA District Office

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

District	LA	Log No.	122052
----------	----	---------	--------

APPLICATION FOR REVIEW OF TECHNICAL REPORTS

INSTRUCTIONS

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

1. LEGAL DESCRIPTION

Tract: STRONG AND DICKINSON'S HOLLYWOOD HIGH SCHOOL TRACT

Block: B Lots: 3/4/5/20/21/22/23

3. OWNER: BARDAS Investment Group

Address: 1015 N. Fairfax Avenue

City: West Hollywood Zip: 90046

Phone (Daytime):

2. PROJECT ADDRESS:

1139-1155 N. Las Palmas Avenue and 1138-1150 N. Mc Cadden Place Los Angeles, CA 90038

4. APPLICANT Feffer Geological Consulting

Address: 1990 S Bundy Drive Suite 400

City: Los Angeles Zip: 90025

Phone (Daytime): 310 207 5048

E-mail address: eleni@feffergeo.com; josh@feffergeo.com; yvette@feffergeo.com

5. Report(s) Prepared by: Feffer Geological Consulting 6. Report Date(s): 12/21/2021

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

9. Previous Department actions? ☐ YES if yes, provide dates and attach a copy to expedite processing.

Dates:

10. Applicant Signature: J. Hays Position: Geologist

(DEPARTMENT USE ONLY)

REVIEW REQUESTED	FEES	REVIEW REQUESTED	FEES
☒ Soils Engineering	363.00	No. of Lots	
☐ Geology		No. of Acres	
☐ Combined Soils Engr. & Geol.		☐ Division of Land	
☐ Supplemental		Other	
☐ Combined Supplemental		☒ Expedite	181.50
☐ Import-Export Route		☐ Response to Correction	
Cubic Yards:		☐ Expedite ONLY	
		Sub-total	544.50
		Surcharges	129.80
		TOTAL FEE	674.30

Fee Due: 674.30
Fee Verified By: ML Date: 6/13/22
(Cashier Use Only)

1352076 Pd 6/14/22

ACTION BY:

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

For Geology

Date

For Soils

Date

April 27, 2022

File No: 2618-14

BARDAS Investment Group
1015 N. Fairfax Avenue
West Hollywood, CA 90046

Subject: **RECOMMENDATIONS REGARDING DEWATERING**
Proposed Three Story Building Over Three Subterranean Levels
1139-1155 N. Las Palmas Avenue and 1138-1150 N. Mc Cadden Place
Los Angeles, CA 90038

Reference: **GEOTECHNICAL INVESTIGATION**
Proposed Three Story Building Over Three Subterranean Levels
1139-1155 N. Las Palmas Avenue and 1138-1150 N. Mc Cadden Place
Los Angeles, CA 90038
By Feffer Geological Consulting Dated December 21, 2021

TRACT: STRONG AND DICKINSON'S HOLLYWOOD HIGH SCHOOL TRACT
BLOCK: B LOT (S): 3/4/5/20/21/22/23

Dear BARDAS Investment Group:

As stated within the referenced geotechnical report the high historic groundwater level is 20 feet below the ground surface and actual groundwater conditions on nearby sites ranges from 23 to 33 feet below the ground surface.

Excavations that extend below groundwater will require installation of a dewatering system. An experienced, qualified contractor should be hired for design of the dewatering system. We recommend that test wells be installed as needed.

We appreciate the opportunity to be of service. Should you have any questions regarding the information contained in this report, please do not hesitate to contact us.

Sincerely,

FEFFER GEOLOGICAL CONSULTING, INC.

Joshua R. Feffer
Principal Engineering Geologist
C.E.G. 2138



FEFFER

GEOLOGICAL CONSULTING

December 21, 2021

File No: 2618-14

BARDAS Investment Group
1015 N. Fairfax Avenue
West Hollywood, CA 90046

Subject: **GEOTECHNICAL INVESTIGATION**
Proposed Three Story Building Over Three Subterranean Levels
1139-1155 N. Las Palmas Avenue and 1138-1150 N. Mc Cadden Place
Los Angeles, CA 90038

TRACT: STRONG AND DICKINSON'S HOLLYWOOD HIGH SCHOOL TRACT
BLOCK: B LOT (S): 3/4/5/20/21/22/23

Dear BARDAS Investment Group:

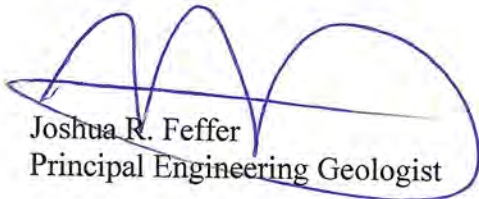
As requested, Feffer Geological Consultants performed a geotechnical investigation at the subject site. The purpose of this investigation was to evaluate the geotechnical conditions at the site in the areas of the proposed construction and to provide geotechnical parameters for design and construction.

Based on our investigation, it is our opinion that the proposed construction is feasible from a geotechnical standpoint provided the recommendations contained herein are incorporated into the project plans and specifications. This report should be reviewed in detail prior to proceeding further with the planned development. When final plans for the proposed construction become available, they should be forwarded to this office for review and comment.

We appreciate the opportunity to be of service. Should you have any questions regarding the information contained in this report, please do not hesitate to contact us.

Sincerely,

FEFFER GEOLOGICAL CONSULTING, INC.


Joshua R. Feffer
Principal Engineering Geologist


Dan Daneshfar
Principal Engineer
P.E. 68377



Distribution: Addressee- (1)

1.0 INTRODUCTION

1.1 PURPOSE

The purpose of this investigation was to evaluate the existing geotechnical conditions at the subject site and to provide design and construction criteria for the proposed mixed-use development.

1.2 SCOPE OF SERVICES

The scope of work performed during this investigation involved the following;

- Research and review of available pertinent geotechnical literature;
- Subsurface exploration consisting of the drilling of four borings (B1, B2, B3, B4,);
- Sampling and logging of the subsurface soils;
- Laboratory testing of selected soil samples collected from the subsurface exploration to determine the engineering properties of the soil;
- Engineering and geologic analysis of the field and laboratory data; and
- Preparation of this report presenting our findings, conclusions, and recommendations for the proposed construction.

1.3 SITE DESCRIPTION

The project site is located on the west side of Las Palmas Drive and is bounded by McCadden Place to the west (Figure 1). The subject site encompasses seven relatively level Parcels (Lots 3-5, 20-23) which have less than five feet of overall elevation change (Figure 2). The subject sites are occupied by an asphalt covered parking lots and one-story building and are surrounded by existing residential and commercial developments. A recent aerial photograph of the site is shown as Figure 3. Surface drainage is by sheet flow to the west and east towards the adjacent streets.

1.4 PROPOSED CONSTRUCTION

Based on the information provided to us, the project will consist of demolishing the existing parking lot and building and the construction of a new three-story building over three subterranean levels. A Site Plan and Cross Sections showing the proposed development are included in Appendix C.

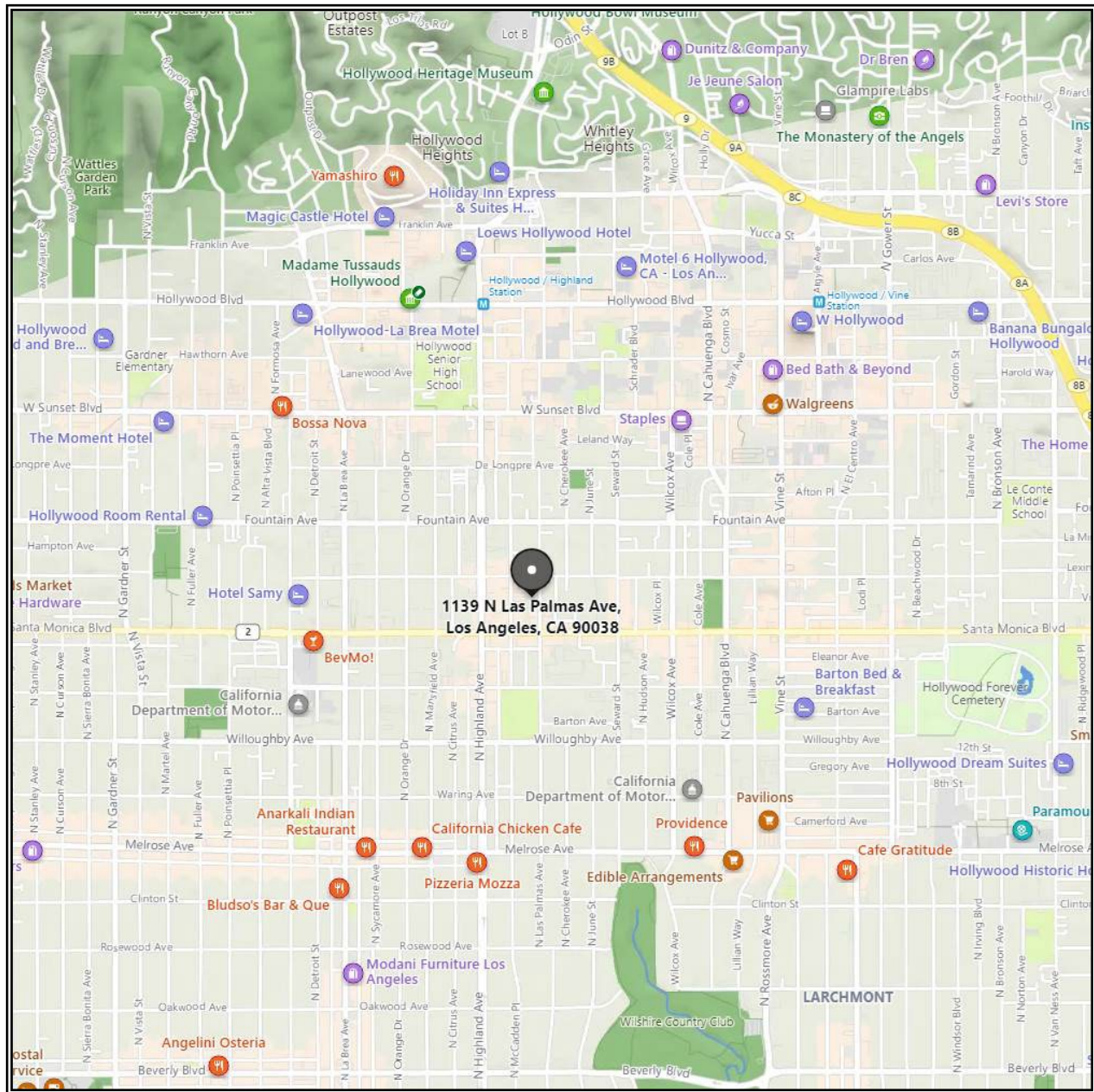


Figure 1. Location map of the subject sites.

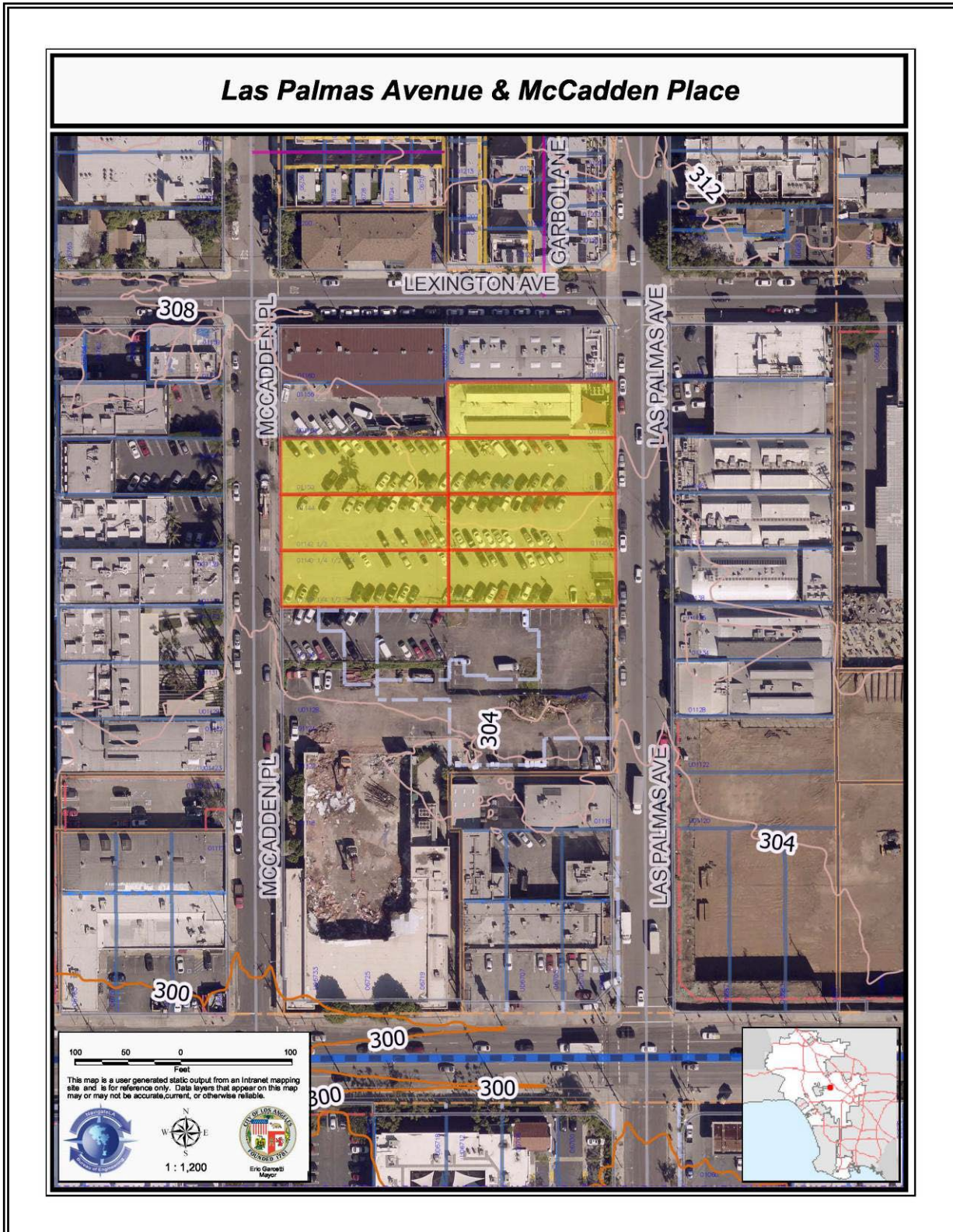


Figure 2. Aerial Photographic with topographic overlay from Navigate LA. Subject sites are highlighted.

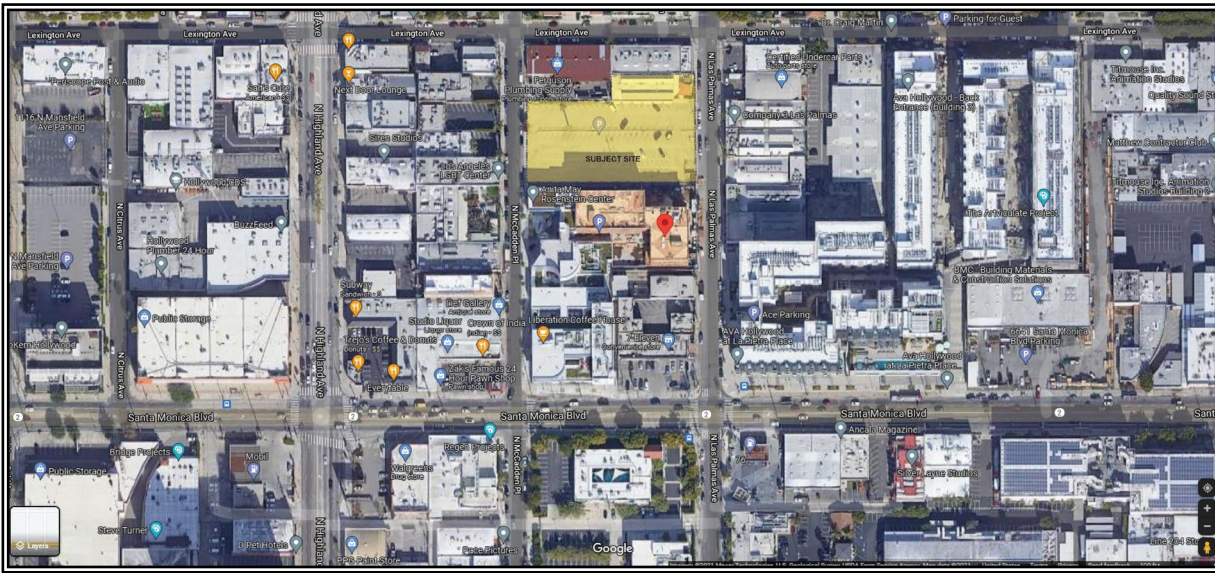


Figure 3. Oblique Aerial Photograph of subject lots and surrounding area, sites are highlighted.

1.5 DOCUMENT REVIEW

The City of Los Angeles Building Department records were researched. The records contained the following Geologic and Soils Engineering Reports for the surrounding properties.

1118 N McCadden Place

Leroy Crandall & Associates performed a Foundation Investigation, Dated August 23, 1960, for a Proposed Workshop Addition. The subsurface investigation consisted of excavating two borings to a maximum depth of forty-one feet below the ground surface and found a few feet of fill over alluvium. The investigation determined that stable competent alluvium was located near the ground surface and recommended that new footings be founded in the natural alluvium or in a new compacted fill cap. The report was approved by the City of Los Angeles on June 22, 1960 File #601-621 and has subsequently been constructed.

6649-6687 Santa Monica Blvd., 1120-1122 Las Palmas Ave., 6624-6648 Lexington Ave.

JB 20113-Z -Addendum Geotechnical Engineering Exploration Update, Proposed Nine-Story, Mixed-Use Development Over Subterranean Garage Parcel A, P.M. 2708, Lots 4-6, Tract 1362, Lot 28, Tonner And Garbutt's Subdivision Of The S. W. Little Tract, And Portions Of Lots 8 And 9, And Lots 10-12 Block A, Strong And Dickinson's Hollywood High School Tract Aka Tentative Tract 067577 6649 -6687 West Santa Monica Boulevard, 1120 -1122 Las Palmas Avenue, and 6624 -6648 Lexington Avenue, Los Angeles, California, Fords Ventures, LLC, dated October 20, 2006

JB 20113-Z -Geotechnical Engineering Exploration, Proposed Multi-Story Mixed-Use Development Over Two-Level Subterranean Garage, Lots 4 and 5, Tract 1362, Lot 28, Tonner & Garbutt's Subdivision of the S. W Little Tract, and Parcel A, P.M. 2708, 6649-6665 Santa Monica Boulevard and 6650 Lexington Avenue, Los Angeles, California, dated July 7, 2005;

JB 20160-B-Geotechnical Engineering Exploration, Proposed Multi-Story Mixed-Use Development Over a Two-Level Subterranean Garage, Lot 6, Tract 1362, 6640 Lexington Avenue, Los Angeles, California, dated August 24, 2005; and

JB 20221-H -Geotechnical Engineering Exploration, Proposed Multi-Story, Mixed-Use Building Over Subterranean Garage, Portions of Lots 8 and 9, Lots 10-12, Block A, Strong and Dickinson's Hollywood High School Tract, 6677-6687 West Santa Monica Boulevard and 1120-1122 North Las Palmas Avenue, Los Angeles, California, dated July 31, 2006.

The City of Los Angeles, Department of Building and Safety, reviewed the July 7, 2005 report by The J. Byer Group, Inc., and issued a Soils Report Correction Letter, Log# 49167, dated August 22, 2005

The City of Los Angeles, Department of Building and Safety, reviewed the October 20, 2006 report by The J. Byer Group, Inc., and issued a Soils Report Approval Letter, Log# 56506, dated February 6, 2006

The J Byer Groups investigation is located nearby on the northeast corner of Santa Monica Boulevard and Las Palmas Avenue and is bounded by Lexington Avenue to the north. Soil reports by the J. Byer Group were based on an extensive subsurface investigation for the construction of a nine-story building over two-levels of subterranean parking. The J. Byer Group determined that a few feet of fill were located over competent alluvium and recommended that conventional foundations be used for support of the new building. Groundwater was observed at a depth of 23 feet below the ground surface. The investigation was approved by the City of Los Angeles on February 5, 2006 under Log #56506.

1118-1136 N McCadden Place & 6719-6733 Santa Monica Boulevard:

Geotechnical Investigation McCadden-Los Angeles LGBT Center Proposed Construction of A Multi-Story Building Over Two Subterranean Levels NE Corner of N McCadden Place and Santa Monica Boulevard 1118-1136 N McCadden Place & 6719-6733 Santa Monica Boulevard By Feffer Geological Consulting Inc; Dated March 19, 2015

City Of Los Angeles Approval Letter Dated May 15, 2015 Log #88073 Plan Modification 1118-1136 N McCadden Place & 6719-6733 Santa Monica Boulevard By Feffer Geological Consulting Inc; Dated October 12, 2015

City Of Los Angeles Approval Letter Dated October 30, 2015 Log #90513

Shoring Pile Addendum 1118-1136 N McCadden Place & 6719-6733 Santa Monica Boulevard By Feffer Geological Consulting Inc; Dated September 21, 2016

Response To City Of Los Angeles Correction Letter Log #95541, Dated November 29, 2016 By Feffer Geological Consulting Inc; Dated February 17, 2017

City Of Los Angeles Approval Letter Dated March 15, 2017 Log #95541-01

Compaction Report I McCadden-Los Angeles LGBTQ Center NE Corner of N McCadden Place and Santa Monica Boulevard 1118-1136 N McCadden Place & 6719-6733 Santa Monica Boulevard By Feffer Geological Consulting Inc; Dated February 15, 2018

City Of Los Angeles Compaction report correction List Dated February 23, 2018, 2018 Log #102100

Response To Compaction Report Correction List McCadden-Los Angeles LGBTQ Center 1118 N McCadden Place Los Angeles, CA 90038 By Feffer Geological Consulting Inc; dated February 15, 2018

City Of Los Angeles Compaction report Approval List Dated May 8, 2018 Log #102100-01

1119 N. McCadden Place:

Geotechnical Investigation McCadden Campus Project – West Site Proposed Construction of Multi-Story Building 1119 N. McCadden Place, Los Angeles, CA 90038 By Feffer Geological Consulting Dated March 19, 2015

Response To City Review Letter, Letter Dated May 8, 2015 Log #88072 By Feffer Geological Consulting Dated May 29, 2015

City Of Los Angeles Approval Letter Dated June 30, 2015 Log #88072-01

Plan Modification 1119 N. McCadden Place, Los Angeles, CA 90038 By Feffer Geological Consulting Dated October 12, 2015

City Of Los Angeles Approval Letter Dated October 30, 2015 Log #90512.

Compaction Report McCadden Campus Project – West Site Proposed Construction of Multi-Story Building 1119 N. McCadden Place, Los Angeles, CA 90038 By Feffer Geological Consulting dated April 23, 2019

City Of Los Angeles Compaction report Approval List Dated May 10, 2018 Log #108195

Reports were produced by Feffer Geological Consulting. The project consisted of constructing a new multi-story building ranging from three to six stories over two subterranean levels (1118-1136 N McCadden Place & 6719-6733 Santa Monica Boulevard) and new four-story on grade building (1119 N. McCadden Place). The subsurface investigation consisted of drilling 10 borings to a maximum depth of 51.5 feet and encountered up to 5 feet fill over alluvium. It was determined that the proposed construction was feasible from a geotechnical standpoint and the recommended bearing material was the alluvium. The reports were approved by LADBS and the projects have been completed with issued certificates of occupancy.

2.0 INVESTIGATION

2.1 GENERAL

Our field investigation was performed on October 13, 2021 and consisted of a review of site conditions and exploration involving the drilling of four borings (B1-B4) and soil sampling. Our investigation also included laboratory testing of selected soil samples. A brief summary of these various tasks is provided below.

2.2 FIELD EXPLORATION

The subsurface investigation performed at the site consisted of drilling four borings by use of a hollow-stem auger drill rig. The purpose of the exploratory borings was to determine the existing subsurface conditions and to collect subsurface soil in the areas of the proposed construction and throughout the site.

The borings were drilled to a maximum depth of 51.5' below the existing ground surface.

The earth materials encountered in the borings consisted of two feet of fill over alluvium at the sites.

A review of geological maps indicates that the material underlying the subject site is comprised of Alluvium (Qae) of Quaternary age (Figure 4).

The borings were logged by our field geologist using both visual and tactile means. Both bulk and relatively undisturbed soil samples were obtained.

The approximate locations of the Borings are shown on the attached Site Plan included in Appendix C. Detailed boring logs are presented in Appendix A.

2.3 LABORATORY TESTING

Laboratory testing was performed on representative samples obtained during our field exploration. Samples were tested for the purpose of estimating material properties for use in subsequent engineering evaluations. Testing included in-place moisture and density, hydro-response-swell/collapse, maximum density and shear strength testing. A summary of the laboratory test results is included in Appendix B.

The physical properties of the soils were tested at Soil Labworks, LLC. Chemical testing was performed at HDR Schiff. The undersigned geologist and engineer have reviewed the data and concur and accept responsibility for the data therein.

3.0 SITE GEOLOGY, SEISMICITY, POTENTIAL HAZARDS

3.1 SITE GEOLOGY

Regional Geologic Maps¹ and the subsurface exploration indicated that the property is underlain by Quaternary Age Alluvium (Qae) (Figure 4) overlain by a veneer of fill. Descriptions of the materials encountered in our exploratory borings are described below.

3.1.1 Artificial Fill (Af)

The fill consists of sandy clay to clayey sand and gravelly silty sand. The color is brown to dark brown and gray brown. The fill is moist and medium dense to dense. The fill encountered is as deep as two feet below the ground surface.

3.1.2 Quaternary Alluvium (Qae)

The alluvium consists of fine to medium-grained admixtures of silts, sands, clay and gravels, which vary from brown, yellow brown, to orange brown. The Alluvium is slightly moist to moist to wet and varies from dense to very dense. The alluvium is generally weakly horizontally layered with no significant structural planes. Generally, the alluvium becomes more granular with depth.

3.1.3 Groundwater

Groundwater was not encountered during exploration at a depth of fifty-one and a half feet below the ground surface. Historically highest groundwater in this area of Los Angeles is estimated to be twenty feet below the ground surface (Plate 1.2, *Historically Highest Groundwater Contours and Borehole Log Data Locations, Hollywood 7½ Minute Quadrangle in Seismic Hazard Zone Report for the Hollywood Quadrangle*, SHZR-026). Groundwater observed by J. Byer Group for the nearby site was at a depth of twenty-three feet.

¹ Dibblee, T.W., 1991, Geologic Map of the Beverly Hills and Van Nuys (south ½) Quadrangle, Los Angeles County, California, Dibblee Foundation Map, DF #31.

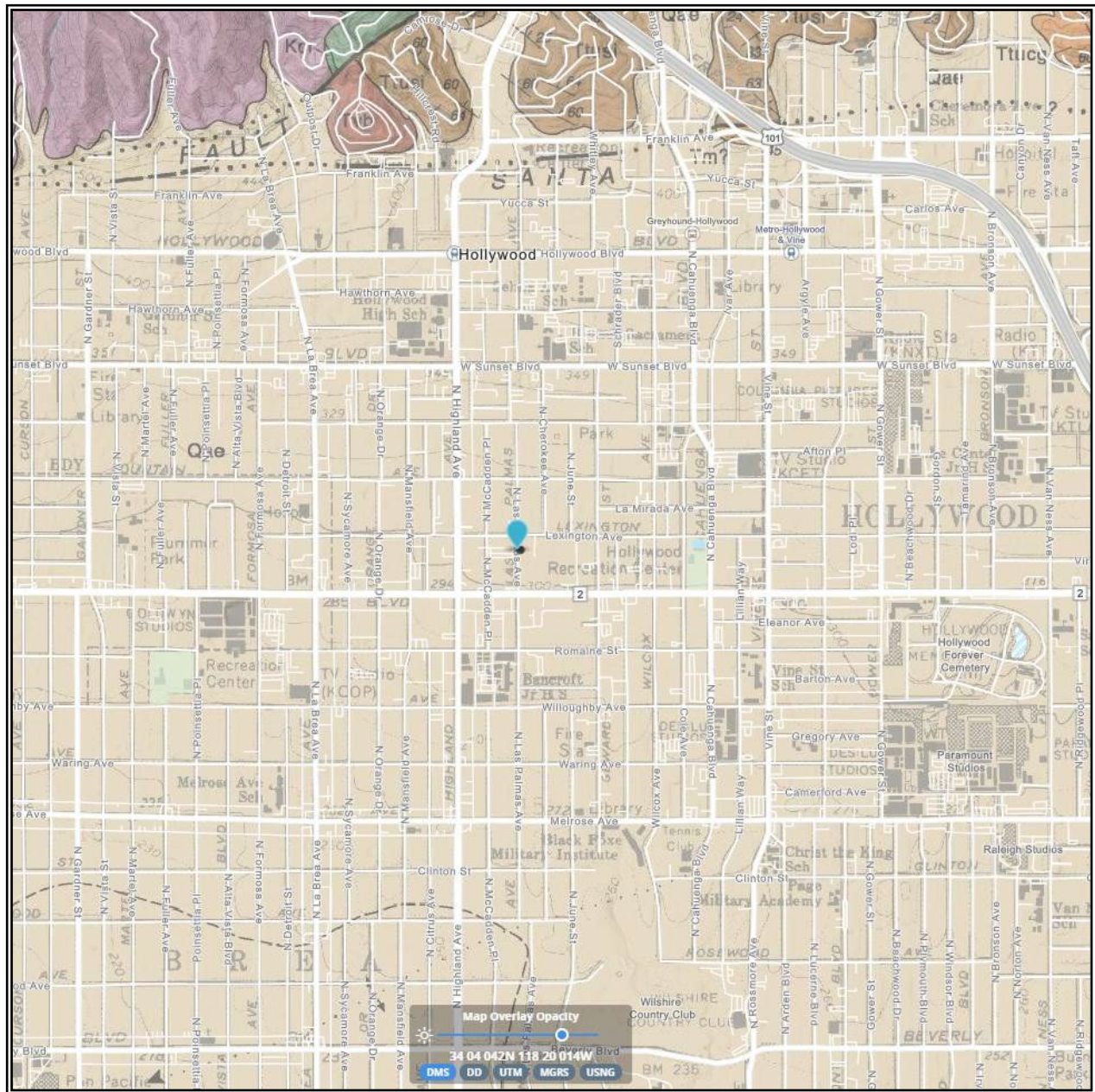


Figure 4. Portion of Dibblee Geologic Map. Site is designated by a dot.

3.2 SEISMICITY

A risk common to all areas of Southern California that should not be overlooked is the potential for damage resulting from seismic events (earthquakes). The site is located within a seismically active area, as is all of Southern California. Although we are not aware of any active faults on or within the immediate vicinity of the site, earthquakes generated on large regional faults such as the San Andreas Fault could affect the site.

The closest known potentially active faults to the site are the Santa Monica-Hollywood and Newport-Inglewood Faults, located within two kilometers. A copy of the recently published California Geological Survey Alquist Priolo Map for the Hollywood Quadrangle is included as Figure 5. As can be seen on the map the Hollywood Fault Zone is located 4,000 feet from the subject site. Since no active faults cross the property, the surface rupture hazard at the site is very low to non-existent.

As can also be seen on Figure 5, the subject site is not located within an area mapped as potentially affected by earthquake induced liquefaction or landslides. Due to the density of the underlying alluvium and the fine content of the soil and the fact that it is not located within an area identified as potentially affected by liquefaction it is our opinion that the liquefaction potential is low.

Due to the distance from the coastline the site is not susceptible to the effects of tsunamis and seiches.

3.3 **2019 CALIFORNIA BUILDING CODE CONSIDERATIONS**

Based on the subsurface investigation and the estimated Vs30 value (317.1 m/s; station name: Hollywood Storage) provided by the USGS², the proposed development may be designed in accordance with seismic considerations contained in the 2019 California Building Code and the following parameters may be considered for design:

A ground motion hazard analysis is required (see Section 11.4.8 of ASCE /SEI 7-16) to be performed in accordance with Section 21.2 for structures on Site Class D with S₁ greater than or equal to 0.2. However, as an alternative of performing the ground motion hazard analysis, a long period coefficient (F_v) of 1.7 may be utilized for calculation of T_s, provided that the value of the Seismic Response Coefficient (C_s) is determined by Equation 12.8-2 for values of the fundamental period of the building (T) less than or equal to 1.5 T_s, and taken as 1.5 times the value computed in accordance with either Equation 12.8-3 for T greater than 1.5 T_s and less than or equal to T_L or Equation 12.8-4 for T greater than T_L.

Mapped Spectral Response Acceleration Parameters:

	S _S	:	2.096g
	S ₁	:	0.752g
Site Class:	D	:	Stiff Soil
Site Coefficients:	F _a	:	1.0
	F _v	:	1.7

Maximum Considered Earthquake Spectral Response Acceleration Parameters:

S _{MS}	:	2.096g
S _{M1}	:	1.278g

Design Spectral Response Acceleration Parameters:

S _{DS}	:	1.398g
S _{D1}	:	0.852g
PGA	:	0.898g
PGA _M	:	0.988g

² United States Geological Survey (USGS), 2022, Earthquake Hazards Program, A Compilation of Vs30 Values in the United States, <https://earthquake.usgs.gov/data/vs30/us/>, website accessed May, 2022

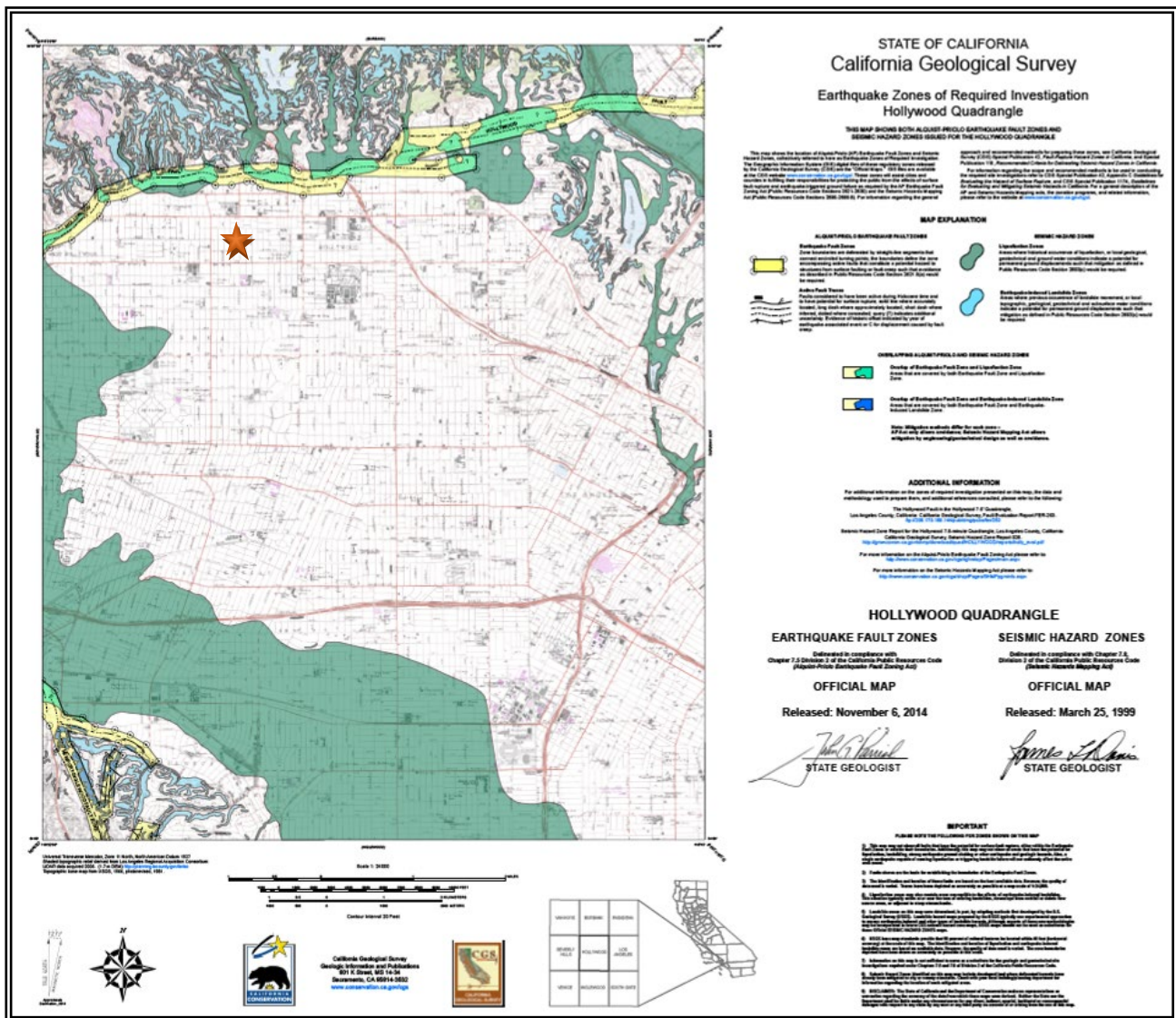


Figure 5. State of California Map showing locations of Alquist Priolo Fault Zones and areas potentially subject to Liquefaction (green) and Landslides (blue). The location of the subject site is shown with a red star.

4.0 GEOTECHNICAL CONSIDERATIONS

4.1 SUBSURFACE SOIL CONDITIONS

Subsurface materials at the site consist of alluvium below a layer of fill observed to be up to two feet at the sites. Laboratory testing indicates that the alluvium has a low potential for consolidation and hydrocollapse. The alluvium at the subject site is competent and capable of supporting engineered structures and appurtenances. The following paragraph provides general discussions about settlement and expansive soil activity.

4.2 SETTLEMENT

Our investigation indicated that the consolidation and hydrocollapse potential of the alluvium at the depth of the proposed construction is low. The in-situ dry densities are high for the samples taken at the foundation level and it is our experience that these soils have a very low potential for consolidation. Recommendations are presented below to mitigate the settlement hazard associated with consolidation of the near surface soils.

4.3 EXPANSIVE SOIL

The on-site, near surface soil was found to possess low to medium expansive characteristics based upon field soil classifications.

4.4 SLOPE STABILITY

The property has less than five feet of overall elevation change and is essentially flat. A slope stability analysis is not required for the property per City of Los Angeles Department of Building and Safety Information Bulletin P/BC 2011-49.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 BASIS

Conclusions and recommendations contained in this report are based upon information provided, information gathered, laboratory testing, engineering and geologic evaluations, experience, and judgment. Recommendations contained herein should be considered minimums consistent with industry practice. More rigorous criteria could be adopted if lower risk of future problems is desired. Where alternatives are presented, regardless of what approach is taken, some risk will remain, as is always the case. Usually, the lowest risk is associated with the greatest cost.

5.2 SITE SUITABILITY

The site is within an area including completed housing and building developments. Geotechnical exploration, analyses, experience, and judgment result in the conclusion that the proposed development is suitable from a geotechnical standpoint.

It is our opinion that the site can be improved without hazard of landslide, slippage, or settlement, and improvement can occur without similar adverse impact on adjoining properties. Realizing this expectation will require adherence to good construction practice, agency and code requirements, the recommendations in this report, and possible addendum recommendations made after plan review and at the time of construction.

It should be realized that the purpose of the seismic design utilizing the above parameters is to safeguard against major structural failures and loss of life, but not to prevent damage altogether. Even if the structural engineer provides designs in accordance with the applicable codes for seismic design, the possibility of damage cannot be ruled out if moderate to strong shaking occurs as a result of a large earthquake. This is the case for essentially all structures in Southern California.

5.4 EARTHWORK

5.4.1 General

Where fill is intended for structural support, a compacted fill cap should extend at least three feet below the bottom of footings or five feet in total depth whichever is greater. If the proposed construction will require grading of the site; it should be done in accordance with good construction practice, minimum code requirements and recommendations to follow. Grading criteria are included within Appendix D.

5.4.2 Site Preparation and Grading

Based on our understanding of the proposed development, we recommend that footings for buildings with subterranean levels be founded in firm alluvium.

If utilized for support of on grade structures, a fill cap should extend a minimum of five feet below the existing ground surface and three feet below the bottom of foundations, whichever is deeper; the bottom of the fill cap should extend into competent natural soil. The fill cap should extend a minimum of five feet outside of the building footprint horizontally. The existing soil can be used for the new fill.

Prior to the start of grading operations, utility lines within the project area, if any, should be located and marked in the field so they can be rerouted or protected during site development. All debris and perishable material should be removed from the site. Although currently not anticipated, all permanent cut and fill slopes should not be constructed steeper than 2:1.

If fill is to be placed the upper six to eight inches of surface exposed by the excavation should be scarified; moisture conditioned to two to four percent over optimum moisture content and compacted to 90 percent relative compaction³. If localized areas of relatively loose soils prevent proper compaction, over-excavation and re-compaction will be necessary.

The existing fill can be used for the compacted fill cap. Import fill should conform to the recommendations included in Appendix D and be "similar to or better" than the existing soil.

5.4.3 Excavation Characteristics

The borings did not encounter hard earth materials. Difficult excavation conditions are not anticipated. However, loose layers of sand may be encountered that will potentially cave during excavation.

5.4.4 Groundwater and Associated Design

The groundwater below our adjacent project to the south site was located at a depth of thirty-three feet and at has been as high as twenty-three feet below the ground surface. Historical

³ Relative compaction refers to the ratio of the in-place dry density of soil to the maximum dry density of the same material as obtained by the "modified proctor" (ASTM D1557-00) test procedure.

groundwater high is at a depth of twenty feet below the ground surface. Wet conditions and actual groundwater may be encountered. If groundwater is encountered, dewatering may be required during construction. Dewatering should be designed by a dewatering contractor and engineer.

The high historic groundwater level is close to the proposed base of foundations. The foundation and any portion of the building below a depth of 20 feet are required to be designed to resist hydrostatic pressures including uplift. Retaining walls below a depth of 20 feet should also be designed for hydrostatic pressures.

Excavations that extend below groundwater will require installation of a dewatering system. An experienced, qualified contractor should be hired for design of the dewatering system. We recommend that test wells be installed as needed.

A mat foundation may be required to provide appropriate waterproofing and resistance to uplift. Recommendations for a mat foundation are provided below.

5.5 FOUNDATION SUPPORT

5.5.1 New Structures

All proposed footings shall be embedded within the competent alluvium, in accordance with the recommendations below. Conventional foundations could be utilized for the proposed development.

Foundation support for the new structures could be derived by utilizing a conventional, shallow foundation system embedded within the competent alluvium.

Allowable design parameters for foundations are provided below.

Minimum depth for interior and exterior footing
(Measured from lowest adjacent grade).....2 feet
Minimum embedment into approved Alluvium 12 inches

Minimum width 1.5 feet

Bearing pressure
a. Sustained loads (lbs. per square foot) 3,000 psf

Resistance to lateral loads
a. Passive soil resistance (lbs. per cubic ft.)

Maximum allowable for Alluvium 5,000 psf
b. Coefficient of sliding friction.....0.35

The allowable bearing pressures are for dead plus long-term live loads and include a factor-of-safety of at least 3.0.

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

Increases in the bearing value of the alluvium are allowable at a rate of 300 pounds per square foot for each additional foot of footing width to a maximum of 5,000 pounds per square foot. For bearing calculations, the weight of the concrete in the footing may be neglected.

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. Footing excavations should be cleaned of all loose soil, moistened, free of shrinkage cracks and approved by the geologist and geotechnical engineer prior to placing forms, steel, or concrete.

Based on the anticipated building loads footings designed and constructed in accordance with the soil criteria included within the referenced report are expected to settle less than $\frac{1}{4}$ to $\frac{1}{2}$ inch in a distance of 20 feet. Differential settlement is expected to be less than $\frac{1}{4}$ inch. The total and differential settlements are within acceptable and allowable tolerances for conventional foundations.

5.5.2 Mat Foundation

For purposes for waterproofing and for resisting design hydrostatic uplift due to the historical high groundwater level being at the level of the basement, a mat foundation may be appropriate. The mat will extend below the highest historical groundwater level and into over-consolidated soils. Existing water is presently at thirty-three feet below the base of the mat. For vertical capacity, the mat may be assumed to have an allowable uniform bearing capacity of 3,000 psf. The bearing value shown above 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.

For computing deflection, a subgrade modulus of 150 kips/ft³ may be assumed. For aesthetic reasons, the deflection should not exceed $\frac{1}{2}$ inch in 30 feet. The mat is not expected to experience any differential settlement.

A rise in the groundwater table will not reduce the bearing capacity of the soils supporting the mat.

5.5.3 Deepened Foundations - Friction Piles

If required drilled, cast-in-place concrete friction piles may be used for support of structures.

Piles should be a minimum of 24 inches in diameter and a minimum of 8 feet into alluvium. Piles may be assumed fixed at three feet into alluvium. The piles may be designed for a skin friction of 400 pounds per square foot for that portion of pile in contact with the alluvium.

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 and alluvium. Passive earth pressure in alluvium may be computed as an equivalent fluid having a density of 400 pounds per cubic foot.

The maximum allowable earth pressure is 4,000 pounds per square foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent.

The structural engineer should follow the Building Code for reduction factors, which are provided below for guidance.

In conformance with Section 1810.2.5 of the Building Code, piles in groups may be subject to reduced lateral and axial capacities to account for group effects.

For piles in groups, the following reduction factors may be assumed for group action. The lateral resistance should be reduced by the factor.

LATERAL RESISTANCE REDUCTION FACTORS FOR PILE GROUPS *AASHTO BRIDGE DESIGN MANUAL 2012			
Pile Spacing in Direction of Loading D = Pile Diameter	Row 1 Leading Row	Row 2	Row 3 & above (Trailing Row)
7D	1.00	1.00	0.90
6.5D	1.00	0.97	0.87
5D	0.86	0.83	0.77
3D	0.75	0.55	0.40

* As amended by Caltrans GeoResearch Group

For piles in groups, the following reduction factors may be assumed for group action. The skin friction axial capacity should be reduced by the factor.

AXIAL CAPACITY REDUCTION FACTORS FOR PILE GROUPS	
Pile Spacing D = Pile Diameter	Group Efficiency
4.5D	0.80
3D	0.70

2D	0.55
1.5D	0.30

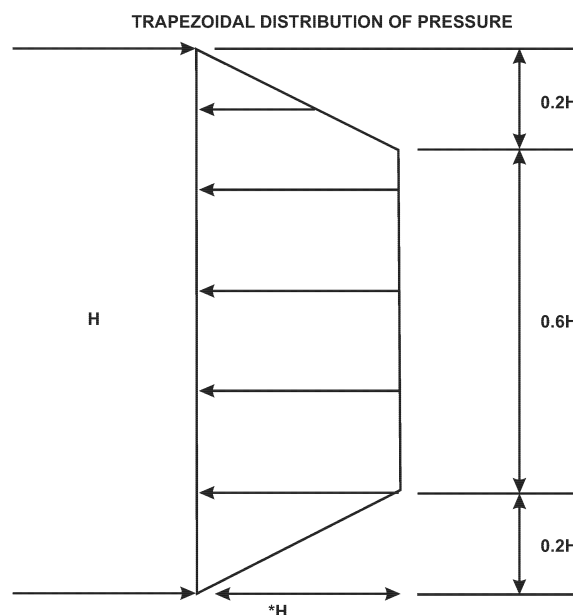
*Figure 3. NAVFAC DM 702-206

5.6 RETAINING WALLS

5.6.1 Retaining Wall

Cantilevered retaining walls up to 35 feet high that support fill, alluvium, and approved retaining wall backfill, may be designed for an equivalent fluid pressure of 55 pounds per cubic foot for level backslopes.

The design at-rest earth pressure on walls is 70 pcf. Restrained/braced retaining walls that are pinned at the top by a non-yielding floor should be for the trapezoidal pressure distribution shown on the adjacent figure of 45 H. The uniform trapezoidal pressure may be assumed over the central six tenths of the wall height. The pressure may be decreased to zero at the top and bottom of the wall.



Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of $\frac{3}{4}$ inch crushed gravel. If a subdrain is not provided the walls should be designed to include hydrostatic pressure and should be designed for an $efp=90$ pcf.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to below grade walls.

Cantilevered retaining walls higher than six feet need to consider a seismic surcharge from the Design Earthquake. The seismic surcharge should be calculated using a factor of safety of 1.0 with the PGA corresponding to $\frac{1}{2}$ of $\frac{2}{3}$ of the PGA_M . The PGA_M is 0.988 and therefore the corresponding seismic design value is 0.33g.

A seismic surcharge for retaining walls designed for active conditions is considered. For a 35-foot-high retaining wall, the static design force is equal to 33.69 kips ($35\text{ft}^2 * 55 \text{ pcf} / 2$).

For a ground motion of 0.36 and a FS of 1.0, the enclosed calculations indicate an unbalanced force under seismic conditions from the maximum considered earthquake is 41049.8 pounds or 41.05 kips.

Since the static design force is lower than the seismic force an additional seismic surcharge of 10 pcf should be added.

5.6.2 Retaining Wall Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557-00. It should be pointed out that the use of heavy compaction equipment in close proximity to retaining walls can result in excess wall movement and/or soil loadings exceeding design values. In this regard, care should be taken during backfilling operations.

5.6.3 Waterproofing

Moisture affecting retaining walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, and/or halite (common salt). Efflorescence is common to retaining walls and generally does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to below grade walls. As aforementioned, the architect, structural engineer, or other qualified waterproofing consultant should develop the actual waterproofing details.

5.7 TEMPORARY EXCAVATIONS

All vertical cuts shall be inspected by our office to verify geologic continuity.

Un-shored temporary vertical cuts to a height of five feet (5') may be made in soil at the site. Un-shored cuts in soil in excess of five feet (5') soil shall be sloped at a gradient of no steeper than 1:1 (horizontal to vertical) for the portion of the excavation above the vertical cut.

5.7.1 Shoring

Shoring, if required for the cuts should be designed for an equivalent fluid pressure of 43 pcf.

Shoring may consist of cast-in-place concrete piles with wood lagging. Shoring piles should be a minimum of 18 inches in diameter and a minimum of 8 feet into alluvium below the base of the excavation. Piles may be assumed fixed 3 feet below the base of the excavation. For the vertical forces, piles may be designed for a skin friction of 400 pounds per square foot for that portion of pile in contact with the dune deposits. Shoring piles should be spaced a maximum of 10 feet on center.

Shoring that is surcharged by traffic and/or structural loads should be designed to withstand the surcharge. Feffer Geologic is able to assist the shoring engineer in evaluating the surcharge pressure and the point of application from concentrated structural loads.

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 base of the excavation.

Passive earth pressure may be computed as an equivalent fluid having a density of 400 pounds per cubic foot. The maximum allowable earth pressure is 4,000 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 2½ pile diameters on center may be considered isolated.

5.7.2 Earth Anchors

Tie-back anchors may be used to resist lateral loads. Pressure grouted friction anchors are recommended. For design purposes, it is assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 30 degrees with the vertical through the bottom of the excavation. Friction anchors should extend at least 15 feet beyond the potential active wedge or to a greater length if necessary, to develop the desired capacities.

The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following section. For preliminary design purposes, it is estimated that cast-in-place gravity anchors will develop an average value of 300 pounds per square foot. Pressure grouted and post grouted anchors will develop much higher capacities. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least six feet on center, no reduction in the capacity of the anchors need be considered due to group action.

The anchors may be installed at angles of 20 to 40 degrees below the horizontal. Caving and sloughing of the anchor hole should be anticipated and provisions made to minimize such caving and sloughing. To minimize chances of caving and sloughing, that portion of the anchor shaft within the active wedge should be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

At least 10 percent of the initial anchors for a 24-hour 200 percent test and 10 percent additional anchors for quick 200 percent tests. The specific anchors selected for the 200 percent test should be representative and acceptable to the geotechnical engineer. The purpose of the 200 percent

tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Anchor rods of sufficient strength should be installed in these anchors to support the 200 percent test loading. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained. The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour test, the anchor deflection should not exceed 0.75 inch measured after the 200 percent test load is applied. If the anchor movement after the 200 percent load has been applied for 12 hours is less than 0.5 inch, and the movement over the previous four hours has been less than 0.1 inch, the 24-hour test may be terminated.

For the quick 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent test load has been applied should not exceed 0.25 inch during the 30-minute period.

All of the anchors should be pretested to at least 150 percent of the design load; the total deflection during the test should not exceed 12 inches. The rate of creep under the 150 percent test should not exceed 0.1 inch over a 15-minute period for the anchor to be approved for the design loading.

After a satisfactory test, each anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 2 percent from the design load, the load should be reset until the anchor is locked-off within 2 percent of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by a deputy grading inspector under the direction of the geotechnical engineer.

5.7.3 Lagging

Lagging will be required between piles. Due to arching in the soils, the pressure on the lagging will be less than on the shoring piles. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot. The void between the lagging and the back-cut should be slurry-filled and observed by a representative of the geotechnical engineer.

A representative of the geotechnical engineer or geologist should be present during grading to see temporary slopes. All excavations, including caissons, footings, and utility trenches, shall be properly and adequately fenced and/or covered to ensure the safety of all those working on the project.

All temporary excavations shall be stabilized as soon as possible after the initial excavation.

5.7.4 Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of ½ inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in

adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

5.7.6 Monitoring

Because of the depth of the excavation, some mean of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Monitoring of the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. Also, some means of periodically checking the load on selected anchors may be necessary.

5.7.8 SURCHARGE

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Seismic lateral forces should be incorporated into the design as necessary.

In addition to the recommended earth pressure, the upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

5.7.9 Line-Load Surcharges

It is recommended that the horizontal pressure for a line-load surcharge from adjacent footings be calculated using the following equations from NAV-FAC DM 7.2.

For $x/H \leq 0.4$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

For $x/H > 0.4$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

Where x is the distance from the face of the excavation or wall to the vertical line-load, H is the distance from the bottom of the footing to the bottom of the excavation or wall, z is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load, and $\sigma_H(z)$ is the horizontal pressure at depth z.

5.7.10 Vertical Point-Load Surcharges

It is recommended that the horizontal pressure from construction equipment outriggers or adjacent building columns for point-loads be calculated using the following equations from NAV-FAC DM 7.2.

For $x/H \leq 0.4$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

For $x/H > 0.4$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H(z) = \sigma_H(z) \cos^2(1.1\theta)$$

Where x is the distance from the face of the excavation or wall to the vertical point-load, H is the distance from the outriggers/bottom of the column footing to the bottom of the excavation or

wall, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth z , Θ is the angle between a line perpendicular to the excavation/wall where the surcharge is being evaluated.

As an alternative to the equations provided above, the structural engineer may decide to use LADBS Information Bulletin Document No. P/Bc 2020-83 to determine the surcharge loads on basement walls and shoring systems for existing structures located within the 1:1 h:v surcharge influence zone of the excavation and basement.

5.8 SLAB-ON-GRADE

If a slab-on-grade is used for the interior of the building it should be a minimum of five inches thick and reinforced with No. 4 bars at 16 inches on center, both ways. The slab should be underlain by a 10-mil Visqueen plastic membrane sandwiched between two, two-inch thick layers of sand. Green Building Code requirements may supersede the recommendations above. The plastic Visqueen barrier should be sealed at all splices, around plumbing, and at the perimeter of slab areas. Every effort should be made to provide a continuous barrier and care should be taken to not puncture the membrane. The splices between layers should be generously staggered. The slab can be placed directly onto alluvium or two feet of newly compacted fill.

As described above slabs below a depth of twenty feet should be designed to resist hydrostatic uplift forces. A mat foundation may be required.

5.9 EXTERIOR FLATWORK AND AUXILIARY STRUCTURES

Whenever planned, exterior flatwork should be placed directly on alluvium or over a two-foot blanket of approved compacted fill. Five-inch net sections with #4 bars at 18 inches o.c.e.w. are also advised. Control joints should be planned at not more than twelve foot spacing for larger concrete areas. Narrower areas of flatwork such as walkways should have control joints planned at not greater than 1.5 times the width of the walkway. Recommendations provided above for interior slabs can also be used for exterior flatwork, but without a sand layer or Visqueen moisture barrier. Additionally, it is also recommended that at least 12-inch deepened footings be constructed along the edges of larger concrete areas.

Movement of slabs adjacent to structures can be mitigated by doweling slabs to perimeter footings. Doweling should consist of No. 4 bars bent around exterior footing reinforcement. Dowels should be extended at least two feet into planned exterior slabs. Doweling should be spaced consistent with the reinforcement schedule for the slab. With doweling, 3/8-inch minimum thickness expansion joint material should be provided. Where expansion joint material is provided, it should be held down about 3/8 inch below the surface. The expansion joints should be finished with a color matched, flowing, flexible sealer (e.g., pool deck compound) sanded to add mortar-like texture. As an option to doweling, an architectural separation could be provided between the main structures and abutting appurtenant improvements.

Auxiliary structures such as trash enclosures and garden walls can be placed directly on alluvium or on a two-foot blanket of compacted fill.

5.10 CONCRETE

We recommend that the low permeable concrete be utilized at the site to limit moisture transmission through slab and foundation. If groundwater is encountered during construction pumping will be required to lower its level. Any concrete placed below the water table should have an appropriate increase of psi in accordance with the Building Code.

For this purpose, the water/cement ratio to be used at the site should be limited to 0.5 (0.45 preferred). Limited use (subject to approval of mix designs) of a water reducing agent may be included to increase workability. The concrete should be properly cured to minimize risk of shrinkage cracking. One-inch hard rock mixes should be provided. Pea gravel mixes are specifically not recommended but could be utilized for relatively non-critical improvements (e.g., flatwork) and other improvements provided the mix designs consider limiting shrinkage.

Contractors/other designers should take care in all aspects of designing mixes, detailing, placing, finishing, and curing concrete. The mix designers and contractor are advised to consider all available steps to reduce cracking. The use of shrinkage compensating cement or fiber reinforcing should be considered. Mix designs proposed by the contractor should be considered subject to review by the project engineer.

5.11 CONCRETE/SULFATE/CORROSIVITY

Testing of the sulfate content of the soil indicates that only low to moderate levels of sulfate concentrations were encountered in the soil and therefore specialized concrete is not required for the project. We recommend that the low permeable concrete be utilized at the site to limit moisture transmission through slab and foundation. The structural engineer should specify appropriate compressive strength and water-cement ratio. Limited use (subject to approval of mix designs) of a water reducing agent may be included to increase workability. The concrete should be properly cured to minimize risk of shrinkage cracking. One-inch hard rock mixes should be provided. Pea gravel mixes are specifically not recommended but could be utilized for relatively non-critical improvements (e.g., flatwork) and other improvements provided the mix designs consider limiting shrinkage.

Contractors/other designers should take care in all aspects of designing mixes, detailing, placing, finishing, and curing concrete. The mix designers and contractor are advised to consider all available steps to reduce cracking. The use of shrinkage compensating cement or fiber reinforcing should be considered. Mix designs proposed by the contractor should be considered subject to review by the project engineer.

5.12 SOIL CORROSIVITY

According to testing of the site soils, the soils should be expected to be only slightly corrosive to ferrous metals. It is recommended that a consulting corrosion engineer be retained in order to determine the most appropriate protection measures for the project site.

Recommendations that the corrosion engineer may require include the following:

- All steel and wire concrete reinforcement should have at least 3 inches of concrete cover where cast against soil.
- Below-grade ferrous metals should be given a high-quality protective coating, such as plastic tape, extruded polyethylene, hot-applied coal tar enamel, or fusion-bonded epoxy.
- On any type of pipe, coat all bare metal appurtenances such as bolts, valves, joint harnesses, or flexible couplings with a coal tar or rubber-based mastic, coal tar epoxy, moldable sealant, wax tape, or equivalent, after assembly.
- Bond below-grade ferrous metals with non-conductive type joints for electrical continuity.
- Below-grade metals should be electrically insulated (isolated) from dissimilar metals, cement-mortar coated and concrete-encased metals, and above-grade metals, by means of insulated joints.
- Metal pipes penetrating concrete structures such as floors and walls should be provided with plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.
- Bare copper tubing should be bedded and backfilled in clean sand at least 3 inches thick surrounding the tubing. The best corrosion control for hot water copper tubing is placement above-grade. Below-grade hot water copper tubing should be encased in impermeable, unstretched, non-shrink insulation with the joints and seams sealed.

5.13 PAVEMENT DESIGN

The following pavement sections are recommended as minimums:

Traffic Index	Asphalt Thickness	Base Thickness
Light Traffic (T.I.=5) for parking stalls and driveways	3 inches	4 inches
Heavy Traffic (T.I.= 6.5) for loading docks and large truck traffic	4 inches	6 inches

Concrete pavement sections should be a minimum of 6 inches thick and reinforced with #4 bars at 18" on center. A base of 6 inches is required below concrete pavement areas. Control joints should be planned at not more than twelve foot spacing.

All pavement should be placed on a minimum two-foot-thick fill cap that is compacted to a minimum of 95% relative compaction.

5.14 DRAINAGE

Drainage should be directed away from structures via non-erodible conduits to suitable disposal areas. The Civil Engineer should design the drainage system. All enclosed planters should be provided with a suitably located drain or drains and/or flooding protection in the form of weep holes or similar. Preferably, structures should have roof gutters and downspouts tied directly to the area drainage system.

5.15 PLAN REVIEW

When detailed grading and structural plans are developed, they should be forwarded to this office for review and comment.

5.16 AGENCY REVIEW

All soil, geologic, and structural aspects of the proposed development are subject to the review and approval of the governing agency(s). It should be recognized that the governing agency(s) can dictate the manner in which the project proceeds. They could approve or deny any aspect of the proposed improvements and/or could dictate which foundation and grading options are acceptable.

5.17 SUPPLEMENTAL CONSULTING

During construction, a number of reviews by this office are recommended to verify site geotechnical conditions and conformance with the intentions of the recommendations for construction. Although not all possible geotechnical observation and testing services are required by the governing agencies, the more site reviews requested, the lower the risk of future site problems. The following site reviews are advised, some of which will probably be required by the agencies.

Preconstruction/pregrading meeting	Advised
Cut and/or shoring observation.....	Required
Periodic geotechnical observations and testing during grading.....	Required
Reinforcement for all foundations	Advised
Slab subgrade moisture barrier membrane	Advised
Slab subgrade rock placement	Advised
Presaturation checks for all slabs in primary structure areas.....	Required
Presaturation checks for all slabs for appurtenant structures.....	Advised
Slab steel placement, primary and appurtenant structures.....	Advised
Compaction of utility trench backfill.....	Advised

Unless otherwise agreed to in writing, all supplemental consulting services will be provided on an as-needed, time-and-expense, fee schedule basis.

5.18 PROJECT SAFETY

The contractor is the party responsible for providing a safe site. This consultant will not direct the contractor's operations and cannot be responsible for the safety of personnel other than his own representatives on site. The contractor should notify the owner if he is aware of and/or anticipates unsafe conditions. If the geotechnical consultant at the time of construction considers conditions unsafe, the contractor, as well as the owner's representative, will be notified. Within this report the terminology safe or safely may have been utilized. The intent of such use is to imply low risk. Some risk will remain, however, as is always the case.

6.0 REMARKS

Only a portion of subsurface conditions have been reviewed and evaluated. Conclusions, recommendations and other information contained in this report are based upon the assumptions that subsurface conditions do not vary appreciably between and adjacent to observation points. Although no significant variation is anticipated, it must be recognized that variations can occur.

This report has been prepared for the sole use and benefit of our client. The intent of the report is to advise our client on geotechnical matters involving the proposed improvements. It should be understood that the geotechnical consulting provided, and the contents of this report are not perfect. Any errors or omissions noted by any party reviewing this report, and/or any other geotechnical aspect of the project, should be reported to this office in a timely fashion. The client is the only party intended by this office to directly receive the advice. Subsequent use of this report can only be authorized by the client. Any transferring of information or other-directed use by the client should be considered "advice by the client."

Geotechnical engineering is characterized by uncertainty. Geotechnical engineering is often described as an inexact science or art. Conclusions and recommendations presented herein are partly based upon the evaluations of technical information gathered, partly on experience, and partly on professional judgment. The conclusions and recommendations presented should be considered "advice." Other consultants could arrive at different conclusions and recommendations. Typically, "minimum" recommendations have been presented. Although some risk will always remain, lower risk of future problems would usually result if more restrictive criteria were adopted. Final decisions on matters presented are the responsibility of the client and/or the governing agencies. No warranties in any respect are made as to the performance of the project.

APPENDIX ‘A’

Subsurface Investigation Logs

LOG OF EXPLORATORY BORING

Sheet 1 of 1

Job Number:2618-14

Project: BARDAS Investment Group

Date Performed: 11/13/21

Boring No: 1

Boring Location: Asphalt Covered Parking Lot
Las Palmas Avenue & Mc Cadden Place

Drill Type: 8" Hollow Stem CME Drill Rig

Depth in Feet	Blows per 6"	Sample Type		Bedrock/ Soil Description	Color	Density	Moisture
		Undisturbed	Bulk				
0				Artificial Fill (Af): 0-3" Asphalt Sandy clay, contains scattered rock fragments	Mottled brown, medium brown, orange brown	Firm	Moist
2.5				Quaternary Alluvium (Qa):			
5	7/8	R		Sandy clay, contain rock fragments	Mottled brown, gray brown	Stiff	Moist
7.5							
10	4/4	R		Sandy clay	Mottled brown, gray brown medium brown	Firm	Moist
12.5							
15	13/20	R		Sandy clay	Mottled brown, medium brown	Stiff	Moist
17.5							
20	16/16	R		Clayey silty sand	Mottled brown, medium brown, orange brown	Dense	Moist
22.5							
25	10/13	R		Clayey silty sand	Mottled brown, medium brown, orange brown	Dense	Moist
27.5				End At 26.5', Af To 2', No Ground Water, No Caving			
30							
32.5							
35							
37.5							
40							

Sheet 1 of 2

Boring No: 2

Boring Location: Asphalt Covered Parking Lot
Las Palmas Avenue & Mc Cadden Place

Drill Type: 8" Hollow Stem CME Drill Rig

Depth in Feet	Blows per 6"	Sample Type		Bedrock/ Soil Description	Color	Density	Moisture
		Undisturbed	Bulk				
0				Artificial Fill (Af): 0-5" asphalt 5-6" Base Sandy clay, contains scattered gravel	Mottled brown, medium brown, orange brown	Firm	Moist
2.5				Quaternary Alluvium (Qa):			
5	9/13	R		Sandy clay, contain rock fragments	Mottled brown, dark brown, medium brown	Firm	Moist
7.5							
10	7/8	R		Clayey silty sand	Mottled brown, orange brown medium brown	Medium dense	Moist
12.5							
15	10/20	R		Clayey silty sand, gravelly silty sand, contains sub rounded rock fragments	Mottled brown, medium brown	Dense	Moist
17.5							
20	10/17	R		Sandy clay	Mottled brown, medium brown, orange brown	Firm	Moist
22.5							
25	10/20	R		Clayey silty sand	Red brown, medium brown, orange brown	Dense	Moist
27.5							
30	15/17	R		Clayey silty sand	Mottled brown, medium brown, orange brown	Dense	Very wet
32.5							
35	7/7	R		Clayey silty sand	Mottled brown, medium brown, orange brown	Dense	Wet
37.5							
40							

Feffer Geological Consulting

Figure

Sheet 2 of 2

Boring No: 2

Boring Location: Asphalt Covered Parking Lot
Las Palmas Avenue & Mc Cadden Place

Drill Type: 8" Hollow Stem CME Drill Rig

Depth in Feet	Blows per 6"	Sample Type		Bedrock/ Soil Description	Color	Density	Moisture
		Undisturbed	Bulk				
40	13/22	R		Silty sand, sandy clay	Mottled brown, medium brown, orange brown	Dense, stiff	Moist
42.5							
45	15/23	R		Sandy clay	Mottled brown, medium brown, red brown	Stiff	Moist
47.5							
50	9/31	R		Silty sand, gravelly silty sand	Mottled brown, medium brown, orange brown	Dense	Moist
52.5				End At 51.5', Af To 2', No Ground Water, No Caving			
55							
60							
65							
70							
75							
80							

Feffer Geological Consulting

Figure

Sheet 1 of 1

Boring No: 4

Boring Location: Asphalt Covered Parking Lot
Las Palmas Avenue & Mc Cadden Place

Drill Type: 8" Hollow Stem CME Drill Rig

Depth in Feet	Blows per 6"	Sample Type		Bedrock/ Soil Description	Color	Density	Moisture
		Undisturbed	Bulk				
0				Artificial Fill (Af): 0-3" Asphalt, 3-4" Base Silty sand, contains scattered debris	Medium brown, orange brown	Medium dense	Moist
2.5				Quaternary Alluvium (Qa):			
5	9/13	R		Sandy clay	Mottled dark gray, gray brown, medium brown	Stiff	Moist
7.5							
10	9/10	R		Clayey silty sand	Mottled brown, orange brown medium brown	Dense	Moist
12.5							
15	7/11	R		Sandy clay	Mottled brown, medium brown	Stiff	Moist
17.5							
20	16/16	R		Clayey silty sand, contains rock fragments	Mottled brown, medium brown, dark brown	Dense	Moist
22.5							
25	12/18	R		Silty sand	Orange brown	Dense	Moist
27.5							
30				End At 26.5', Af To 2', No Ground Water, No Caving			
32.5							
35							
37.5							
40							

APPENDIX 'B'

Laboratory Testing



TRANSMITTAL LETTER

DATE: December 8, 2021

ATTENTION: Josh Feffer

TO: Feffer Geological Consulting
1990 S. Bundy Drive, 4th Floor
Los Angeles, CA 90025

SUBJECT: Laboratory Test Data
BARDAS Investment Group
Your #3823, HDR Lab #21-1209LAB

COMMENTS: Enclosed are the results for the subject project.

A handwritten signature in black ink, appearing to be 'J. Keegan', written over a horizontal line.

James T. Keegan, MD
Corrosion and Lab Services Section Manager



Table 1 - Laboratory Tests on Soil Samples

*Feffer Geological Consulting
BARDAS Investment Group
Your #3823, HDR Lab #21-1209LAB
8-Dec-21*

Sample ID

B3 @ 4'

Resistivity	Units	
as-received	ohm-cm	20,800
minimum	ohm-cm	920

pH 7.8

Electrical

Conductivity mS/cm 0.36

Chemical Analyses

Cations

calcium	Ca ²⁺	mg/kg	70
magnesium	Mg ²⁺	mg/kg	42
sodium	Na ¹⁺	mg/kg	252
potassium	K ¹⁺	mg/kg	6.2
ammonium	NH ₄ ¹⁺	mg/kg	ND

Anions

carbonate	CO ₃ ²⁻	mg/kg	ND
bicarbonate	HCO ₃ ¹⁻	mg/kg	323
fluoride	F ¹⁻	mg/kg	3.6
chloride	Cl ¹⁻	mg/kg	3.7
sulfate	SO ₄ ²⁻	mg/kg	637
nitrate	NO ₃ ¹⁻	mg/kg	36
phosphate	PO ₄ ³⁻	mg/kg	ND

Other Tests

sulfide	S ²⁻	qual	na
Redox		mV	na

Minimum resistivity and pH per CTM 643, Chloride per CTM 422, Sulfate per CTM 417

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed



SL21.3823
December 15, 2021

Feffer Geological Consulting
1990 S. Bundy Drive
4th Floor
Los Angeles, California 90025

Attn: Joshua R. Feffer

Subject: Laboratory Testing

Site: 1128-1144 & 1145-1155 N Las Palmas
1138-1142 N. McCadden Place
Los Angeles, California

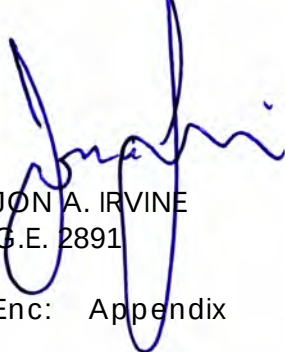
Job: FEFFER/BARDAS INVESTMENT GROUP – 2618-14

Laboratory testing for the subject property was performed by Soil Labworks, LLC., under the supervision of the undersigned Engineer. Samples of the earth materials were obtained from the subject property by personnel of Feffer Geological and transported to the laboratory of Soil Labworks for testing and analysis. The laboratory tests performed are described and results are attached.

Services performed by this facility for the subject property were conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions.

Respectfully Submitted:

SOIL LABWORKS, LLC


JON A. IRVINE
G.E. 2891
Enc: Appendix



2500 Townsgate Road, Suite E, Westlake Village, California 91361
(805) 370-1338 FAX (805) 371-4693

APPENDIX

Laboratory Testing

Sample Retrieval - Drill Rig

Samples of earth materials were obtained at frequent intervals by driving a thick-walled steel sampler conforming to the most recent version of ASTM D 3550/D 3550M-17 with successive drops of a 140 pound hammer falling 30". The earth material was retained in brass rings of 2.416 inches inside diameter and 1.00 inch height. The central portion of the sample was stored in close-fitting, water-tight containers for transportation to the laboratory.

Moisture Density

The field moisture content and dry density were determined for each of the soil samples. The dry density was determined in pounds per cubic foot following ASTM 2937-17e2. The moisture content was determined as a percentage of the dry soil weight conforming to ASTM 2216-19. The results are presented below in the following table. The percent saturation was calculated on the basis of an estimated specific gravity. Description of earth materials used in this report and shown on the attached Plates were provided by the client.

Test Pit/Boring No.	Sample Depth (Feet)	Soil Type	Dry Density (pcf)	Moisture Content (percent)	Percent Saturation ($G_s=2.65$)
B1	5	Alluvium	96.8	22.4	84
B1	10	Alluvium	80.6	32.8	83
B1	15	Alluvium	108.7	20.3	100
B1	20	Alluvium	104.6	20.2	92
B1	25	Alluvium	105.9	19.0	90
B2	5	Alluvium	98.6	22.2	87
B2	10	Alluvium	103.5	17.4	77
B2	15	Alluvium	110.4	11.0	58
B2	20	Alluvium	103.5	22.5	100
B2	25	Alluvium	113.2	17.1	99
B2	30	Alluvium	99.8	23.3	94
B2	35	Alluvium	103.0	23.2	100
B2	40	Alluvium	111.9	18.2	100
B2	45	Alluvium	100.8	24.4	100
B2	50	Alluvium	115.8	15.9	99

Moisture Density (continued)

Test Pit/Boring No.	Sample Depth (Feet)	Soil Type	Dry Density (pcf)	Moisture Content (percent)	Percent Saturation ($G_s=2.65$)
B3	5	Alluvium	100.1	21.2	86
B3	10	Alluvium	99.3	19.2	76
B3	15	Alluvium	104.0	21.9	99
B3	20	Alluvium	107.8	18.4	92
B4	5	Alluvium	103.0	22.4	98
B4	10	Alluvium	98.3	18.9	73
B4	15	Alluvium	99.9	21.5	87
B4	20	Alluvium	103.3	21.1	93
B4	25	Alluvium	114.6	13.4	80

Compaction Character

Compaction tests were performed on bulk samples of the earth materials in accordance with ASTM D1557-12(2021). The results of the tests are provided on the table below and on the "Moisture-Density Relationship", A-Plates. The specific gravity of the fill/alluvium was estimated from the compaction curves.

Test Pit/Boring No.	Sample Depth (Feet)	Soil Type	Maximum Dry Density (pcf)	Optimum Moisture Content (Percent)
B3	4	Fill/Alluvium	119.1	12.1

Shear Strength

The peak and ultimate shear strengths of the alluvium were determined by performing consolidated and drained direct shear tests in conformance with ASTM D3080/D3080M-11. The tests were performed in a strain-controlled machine manufactured by GeoMatic. The rate of deformation was 0.01 inches per minute. Samples were sheared under varying confining pressures, as shown on the "Shear Test Diagrams," B-Plates. The moisture conditions during testing are shown on the following table and on the B-Plates. The samples indicated as saturated were artificially saturated in the laboratory. All saturated samples were sheared under submerged conditions.

Test Pit/Boring No.	Sample Depth (Feet)	Dry Density (pcf)	As-Tested Moisture Content (percent)
B4	5	103.0	29.6
B3	10	99.3	25.1
B2	15	110.4	27.5

Consolidation

One-dimensional consolidation tests were performed on samples of the alluvium in a consolidometer manufactured by GeoMatic in conformance with ASTM D2435/D2435M-11(2020). The tests were performed on 1-inch high samples retained in brass rings. The samples were initially loaded to approximately $\frac{1}{2}$ of the field over-burden pressure and then unloaded to compensate for the effects of possible disturbance during sampling. Loads were then applied in a geometric progression and resulting deformation recorded. Water was added at a specific load to determine the effect of saturation. The results are plotted on the "Consolidation Test," C-Plates.

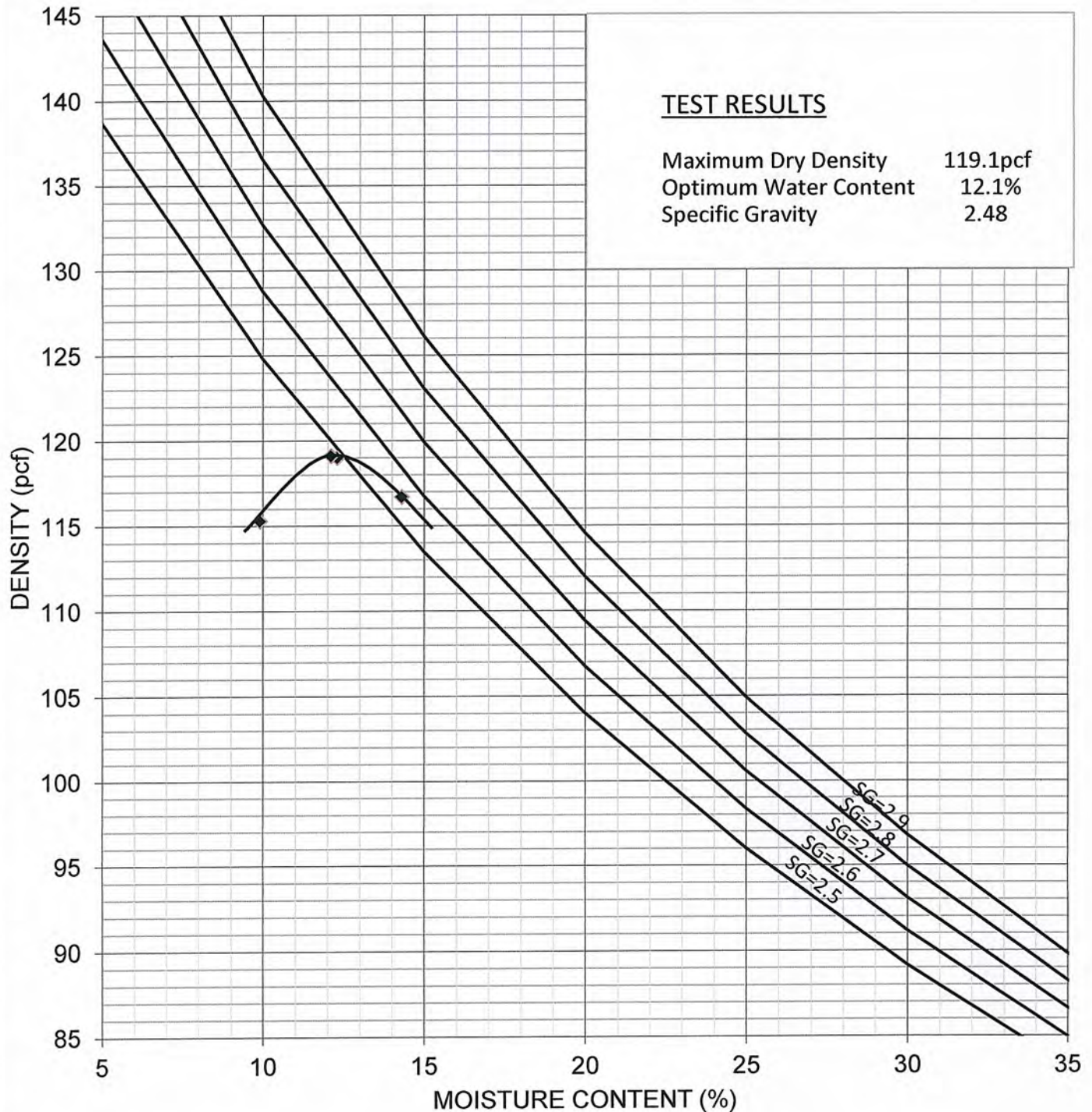


**SOIL
LABWORKS LLC**

MOISTURE-DENSITY RELATIONSHIP A-1

JN: SL21.3823 CONSULTANT: JAI
CLIENT: FEFFER/Bardas-Las Palmas & McCadden PI

B3 @ 4'
EARTH MATERIAL: FILL/ALLUVIUM





SHEAR DIAGRAM B-1

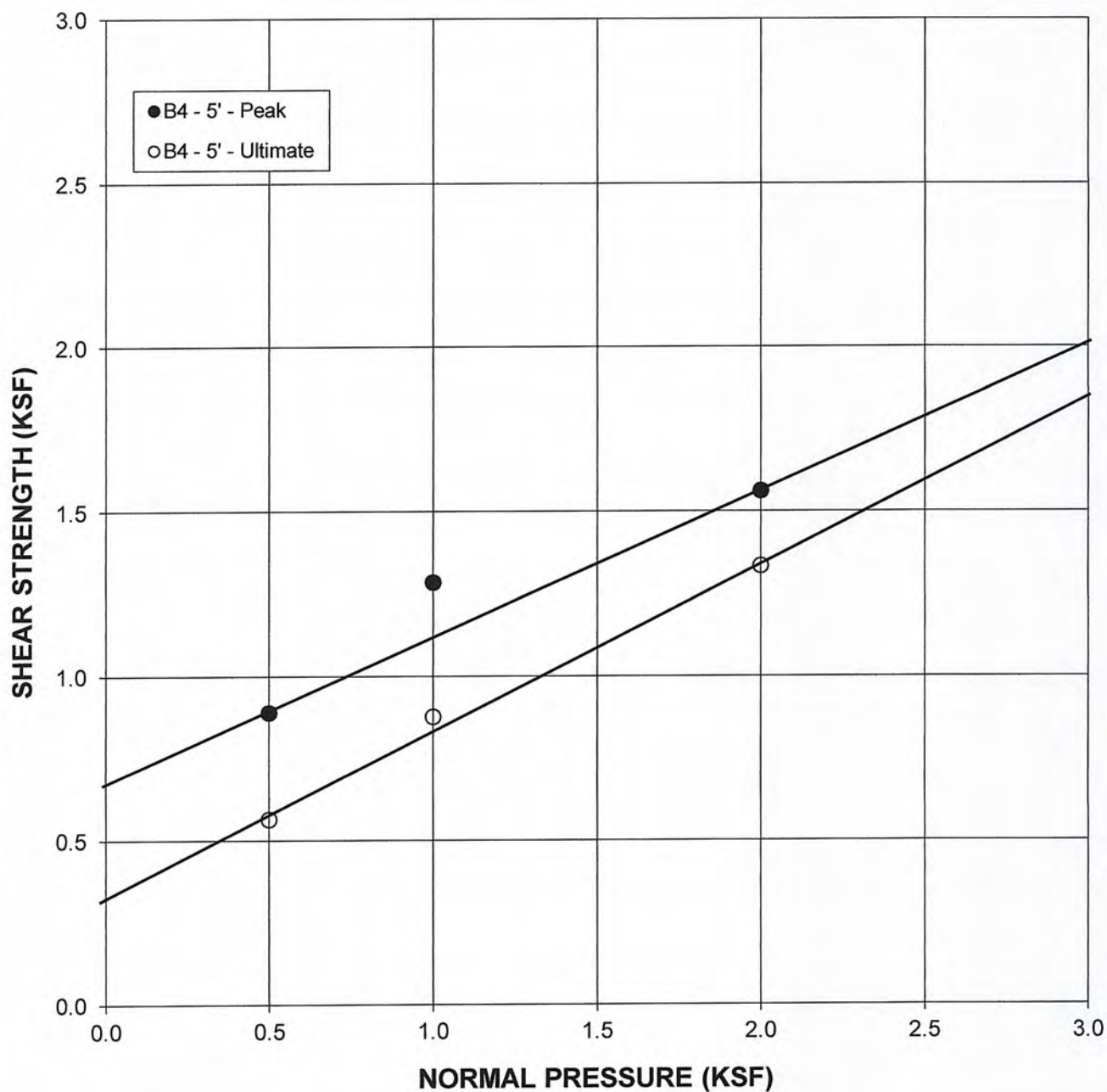
JN: SL21.3823 CONSULTANT JAI
CLIENT: Feffer/Bardas Investment Group

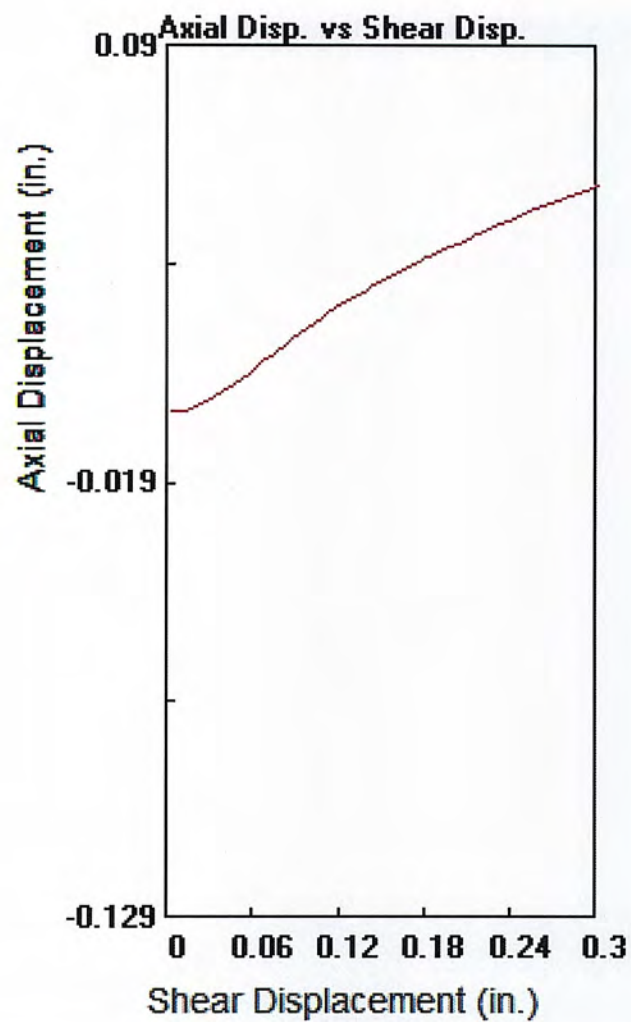
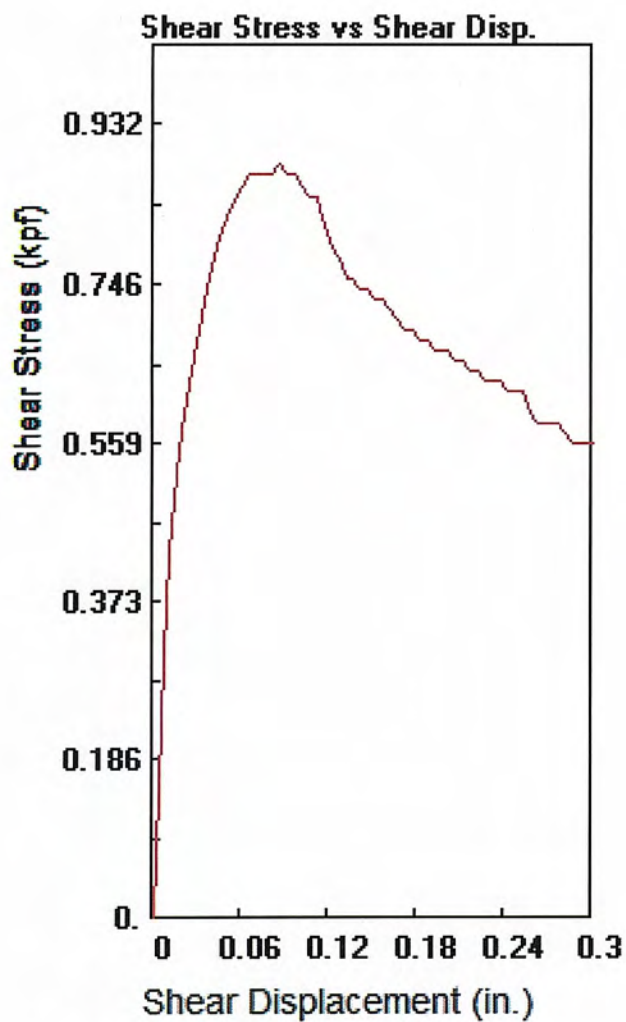
EARTH MATERIAL: ALLUVIUM

	PEAK	ULTIMATE	
Phi Angle	24	27	degrees
Cohesion	680	325	psf

Average Moisture Content	29.6%
Average Dry Density (pcf)	103.0
Percent Saturation	100.0%

DIRECT SHEAR TEST - ASTM D-3080





Parameters

Client: FEFFER/BARDAS

Location: LAS PALMAS MCCADDEN PL

Job # 3823

Soil Type: ALLUVIUM

Sample: 1

Technician: BF

Boring: B4

Axial Load: 500 psf

Depth: 5 ft.

Shear Rate: 0.010 in./sec.

File: 3823B455.dat

Distance: 0.30 in.

Stress at Max Def
888 0.086

Stress at Max Disp
0.296 564

Maximum Load

888 psf

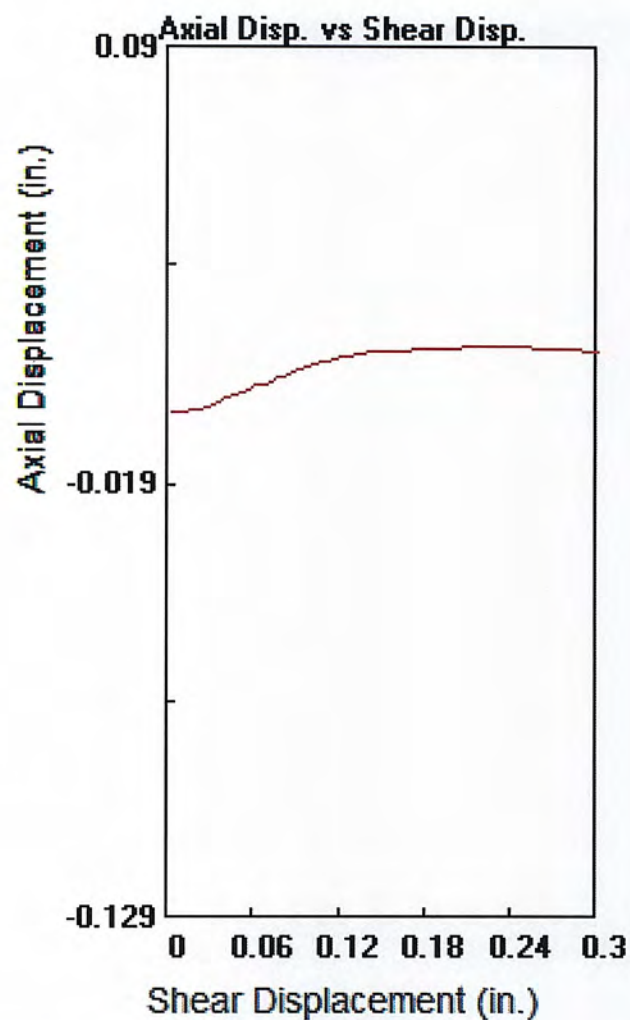
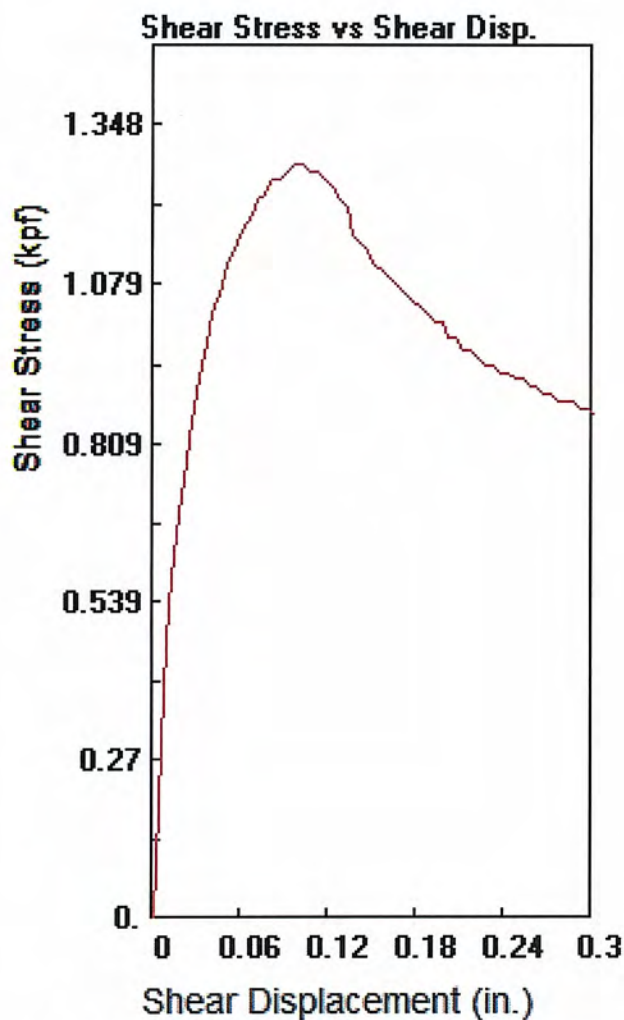
**Shear
Displacement
at maximum
Load**

0.0856 in.

Date

12/10/2021

Soil Labworks



Parameters

Client: FEFFER/BARDAS

Location: LAS PALMAS MCCADDEN PL

Job # 3823

Soil Type: ALLUVIUM

Sample: 2

Technician: BF

Boring: B4

Axial Load: 1000 psf

Depth: 5 ft.

Shear Rate: 0.010 in./sec.

File: 3823B451.dat

Distance: 0.30 in.

Stress at Max Def
1284 0.096

Stress at Max Disp
0.296 876

Maximum Load

1284 psf

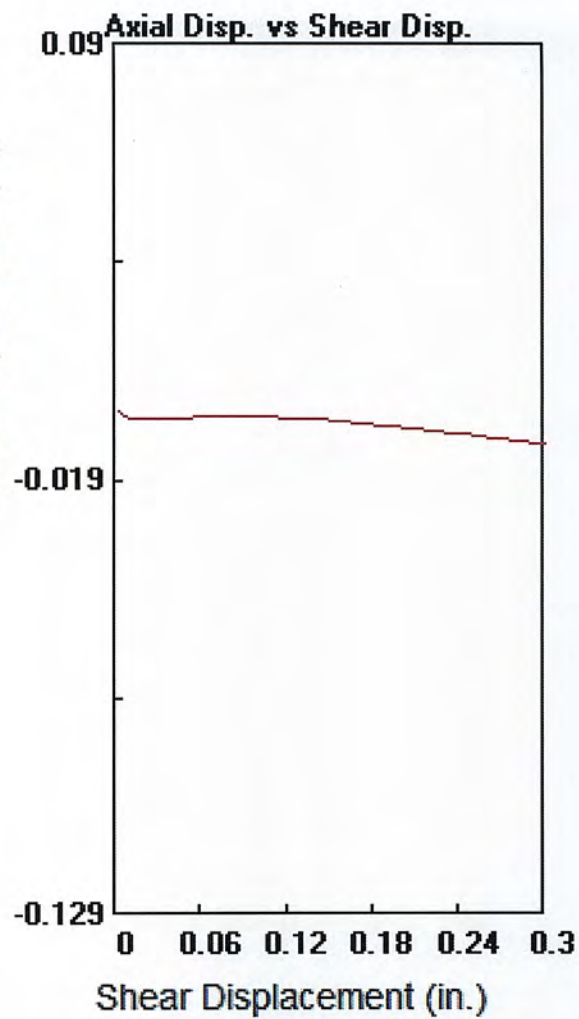
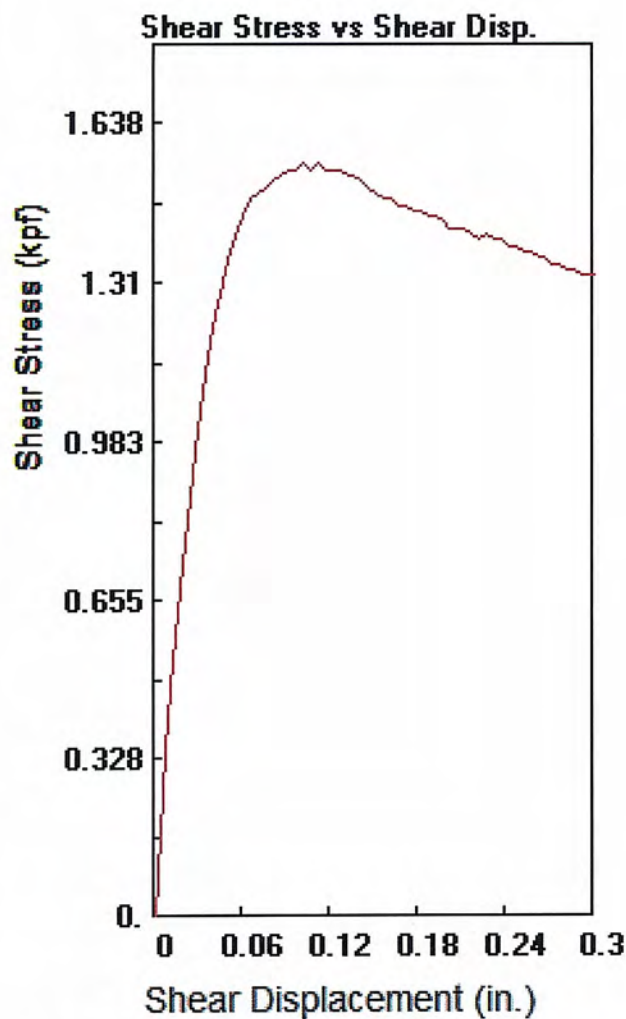
**Shear
Displacement
at maximum
Load**

0.0955 in.

Date

12/10/2021

Soil Labworks



Parameters

Client: FEFFER/BARDAS

Location: LAS PALMAS MCCADDEN PL

Job # 3823

Soil Type: ALLUVIUM

Sample: 3

Technician: BF

Boring: B4

Axial Load: 2000 psf

Depth: 5 ft.

Shear Rate: 0.010 in./sec.

File: 3823B452.dat

Distance: 0.30 in.

Stress at Max Def
1560 0.101

Stress at Max Disp
0.296 1332

Maximum Load

1560 psf

**Shear
Displacement
at maximum
Load**

0.1006 in.

Date

12/10/2021

Soil Labworks



SHEAR DIAGRAM B-2

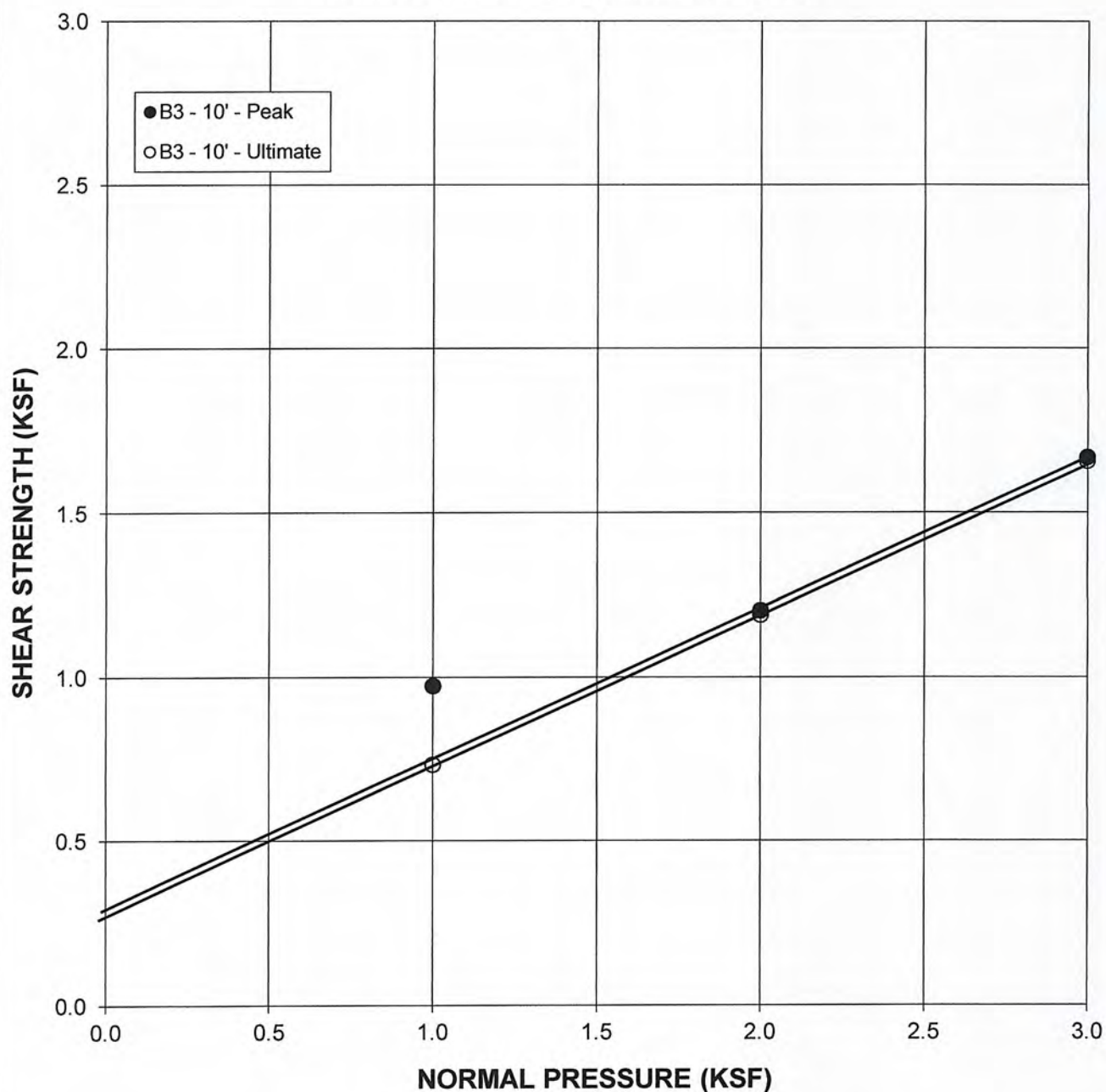
JN: SL21.3823 CONSULTANT JAI
CLIENT: Feffer/Bardas Investment Group

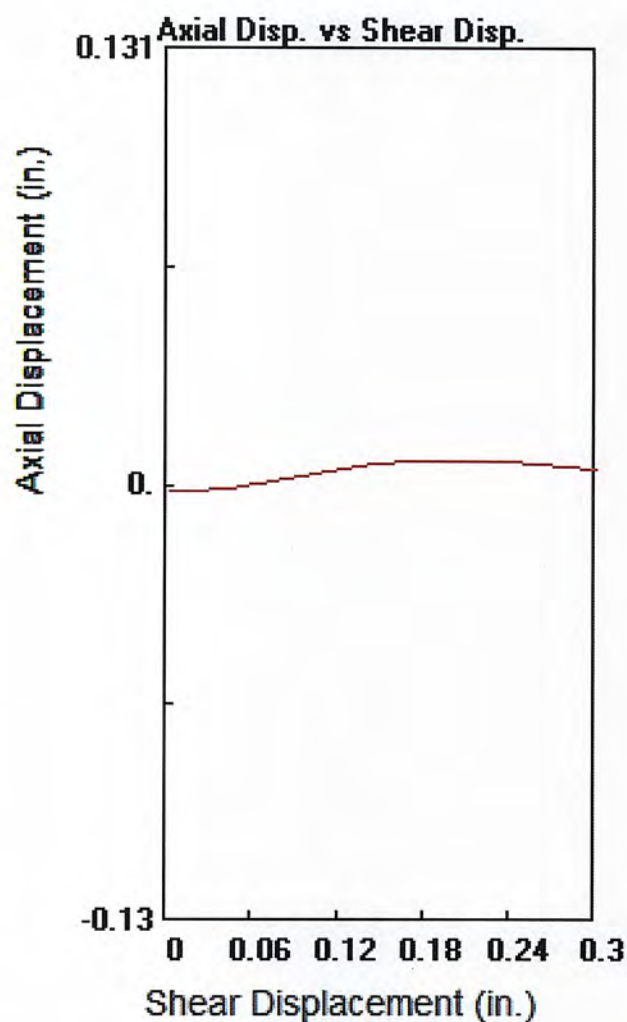
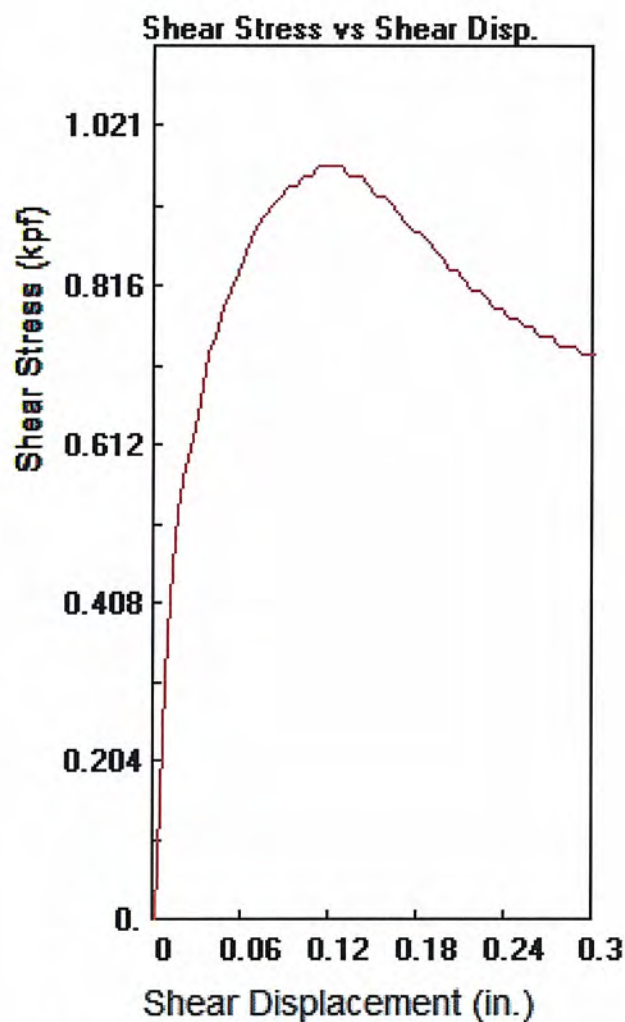
EARTH MATERIAL: ALLUVIUM

	PEAK	ULTIMATE	
Phi Angle	24.5	24.5	degrees
Cohesion	290	275	psf

Average Moisture Content	25.1%
Average Dry Density (pcf)	99.3
Percent Saturation	100.0%

DIRECT SHEAR TEST - ASTM D-3080





Parameters

Client: FEFFER/BARDAS

Location: LAS PALMAS MCCADDEN PL

Job # 3823

Soil Type: ALLUVIUM

Sample: 1

Technician: BF

Boring: B3

Axial Load: 1000 psf

Depth: 10 ft.

Shear Rate: 0.010 in./sec.

File: 3823B3101.dat

Distance: 0.30 in.

Stress at Max Def
972 0.111

Stress at Max Disp
0.296 732

Maximum Load

972 psf

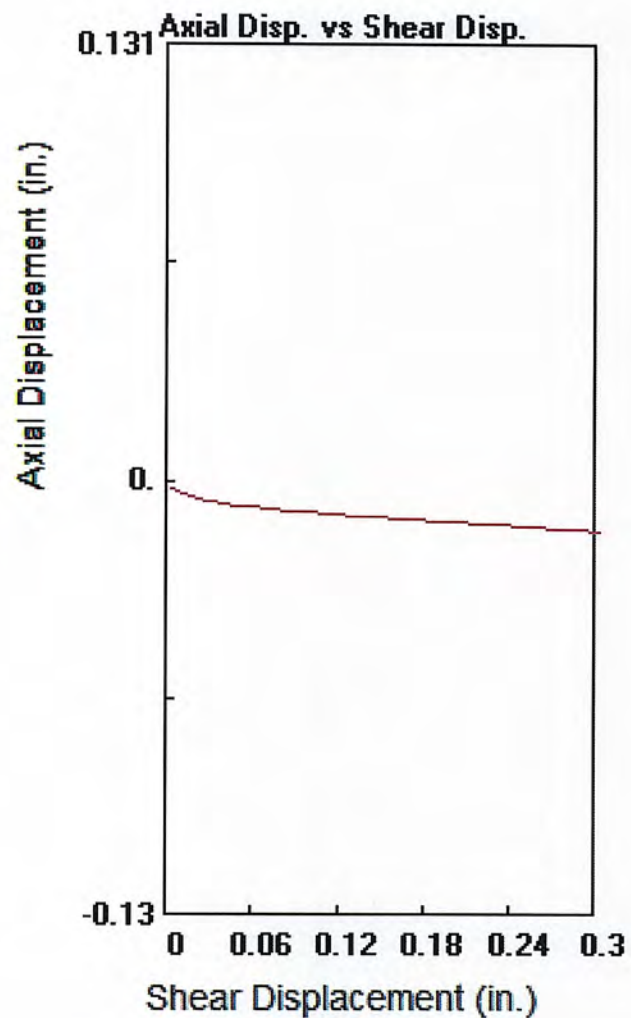
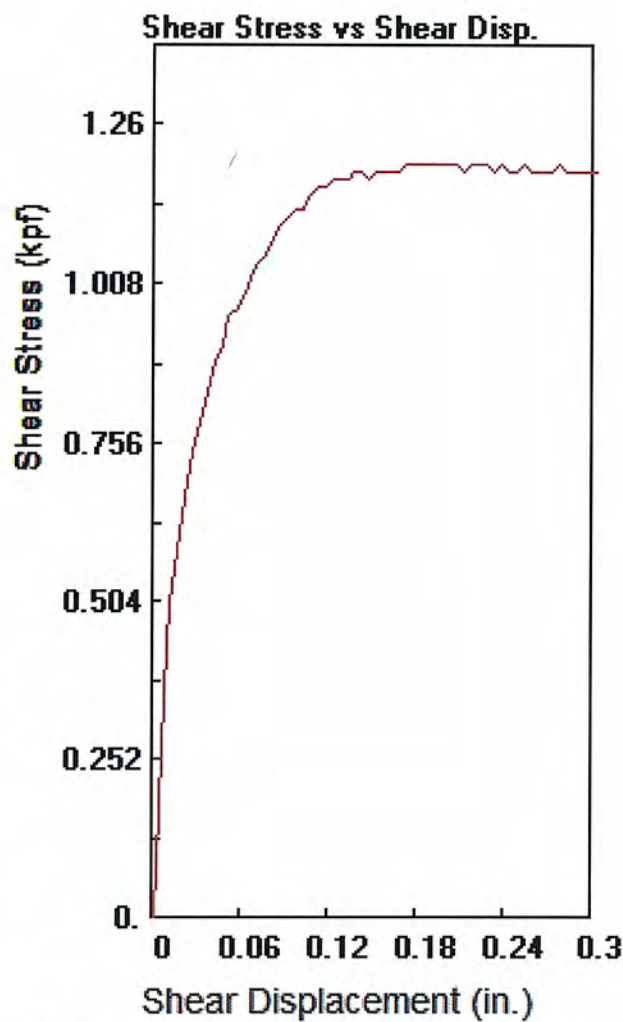
Shear
Displacement
at maximum
Load

0.1106 in.

Date

12/10/2021

Soil Labworks



Parameters

Client: FEFFER/BARDAS

Location: LAS PALMAS MCCADDEN PL

Job # 3823

Soil Type: ALLUVIUM

Sample: 2

Technician: BF

Boring: B3

Axial Load: 2000 psf

Depth: 10 ft.

Shear Rate: 0.010 in./sec.

File: 3823B3102.dat

Distance: 0.30 in.

Stress at Max Def
1200 0.171

Stress at Max Disp
0.296 1188

Maximum Load

1200 psf

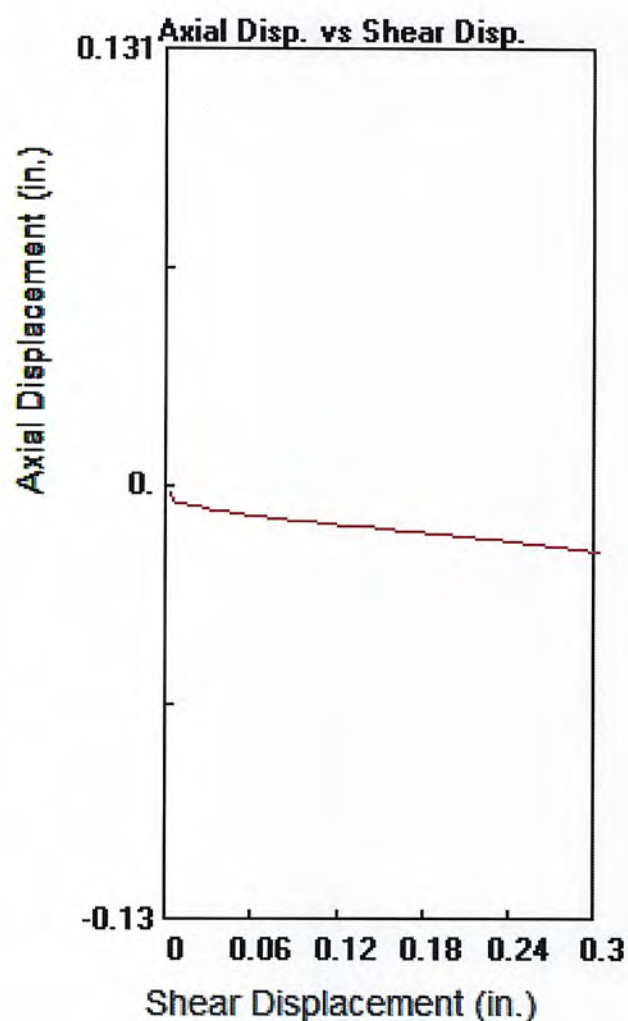
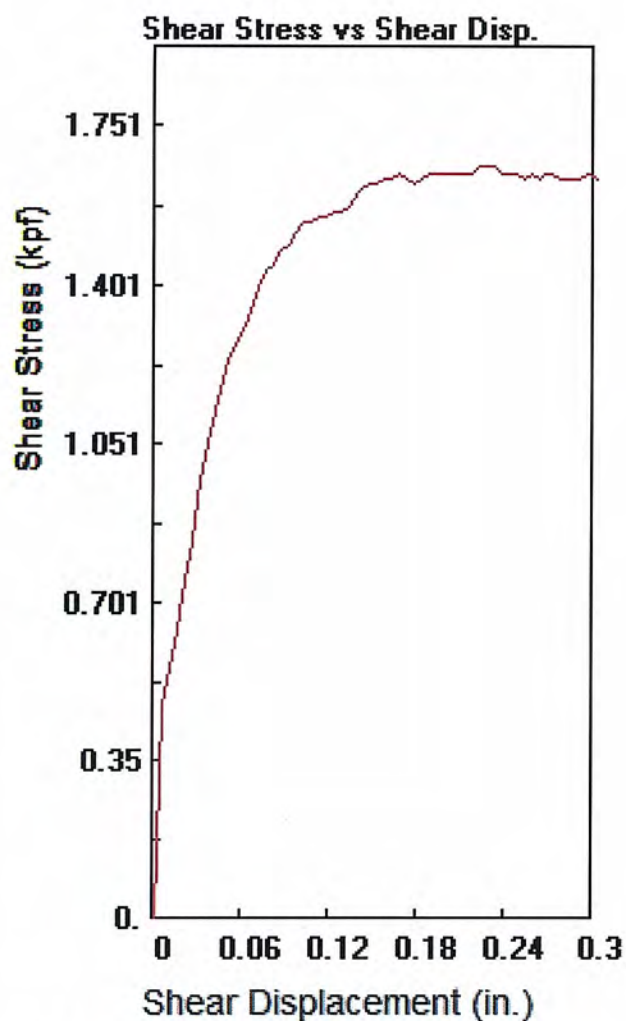
**Shear
Displacement
at maximum
Load**

0.1707 in.

Date

12/10/2021

Soil Labworks



Parameters

Client: FEFFER/BARDAS

Location: LAS PALMAS MCCADDEN PL

Job # 3823

Soil Type: ALLUVIUM

Sample: 3

Technician: BF

Boring: B3

Axial Load: 3000 psf

Depth: 10 ft.

Shear Rate: 0.010 in./sec.

File: 3823B3103.dat

Distance: 0.30 in.

Stress at Max Def
1668 0.221

Stress at Max Disp
0.296 1656

Maximum Load

1668 psf

**Shear
Displacement
at maximum
Load**

0.2207 in.

Date

12/10/2021

Soil Labworks



SHEAR DIAGRAM B-3

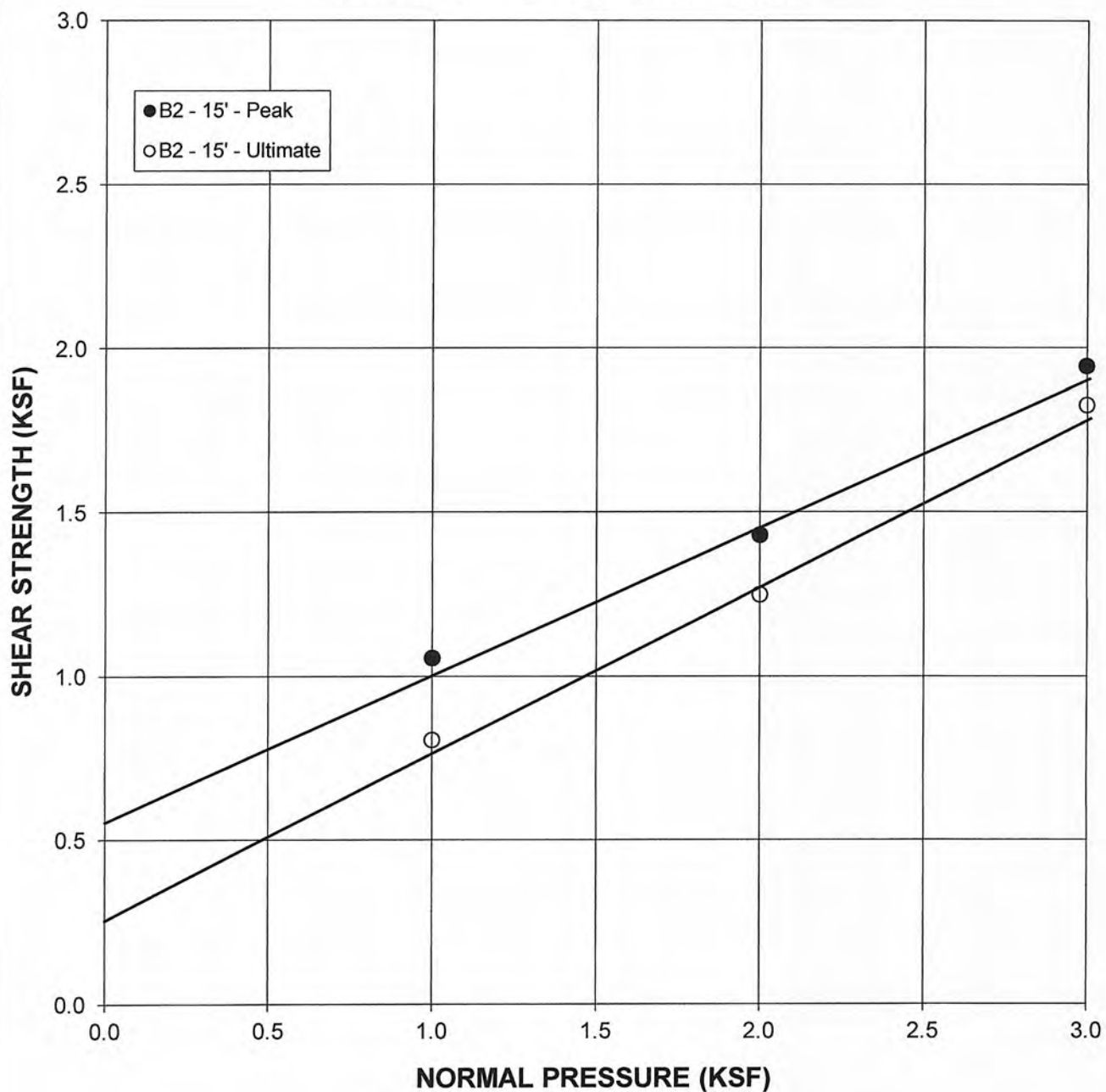
JN: SL21.3823 CONSULTANT JAI
CLIENT: Feffer/Bardas Investment Group

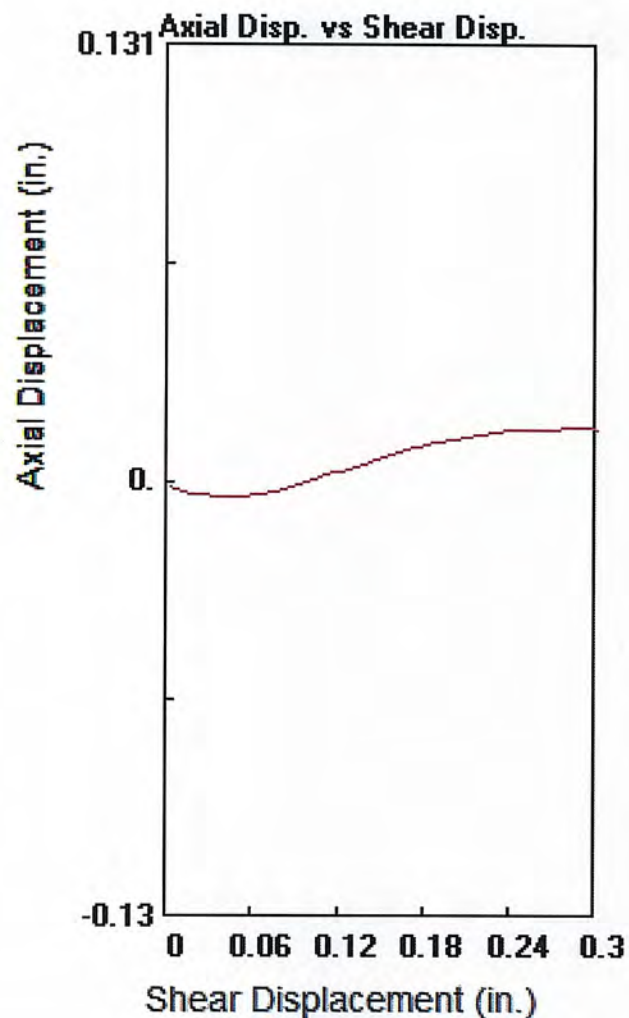
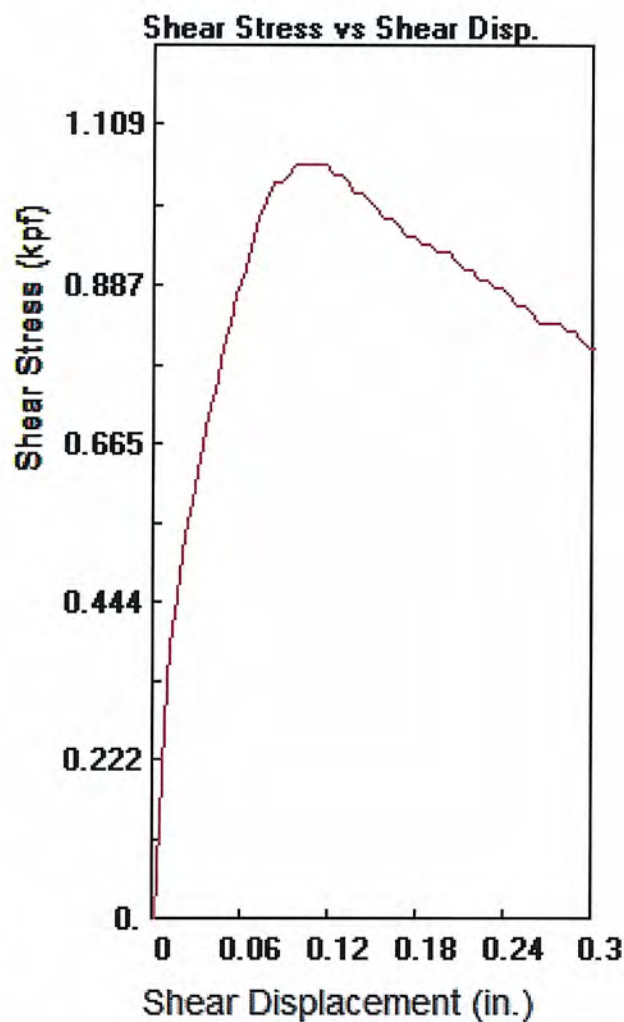
EARTH MATERIAL: ALLUVIUM

	PEAK	ULTIMATE	
Phi Angle	24	27	degrees
Cohesion	560	260	psf

Average Moisture Content	27.5%
Average Dry Density (pcf)	110.4
Percent Saturation	100.0%

DIRECT SHEAR TEST - ASTM D-3080





Parameters

Client: FEFFER/BARDAS

Location: LAS PALMAS MCCADDEN PL

Job # 3823

Soil Type: ALLUVIUM

Sample: 1

Technician: BF

Boring: B2

Axial Load: 1000 psf

Depth: 15 ft.

Shear Rate: 0.010 in./sec.

File: 3823B2151.dat

Distance: 0.30 in.

Stress at Max Def
1056 0.096

Stress at Max Disp
0.296 804

Maximum Load

1056 psf

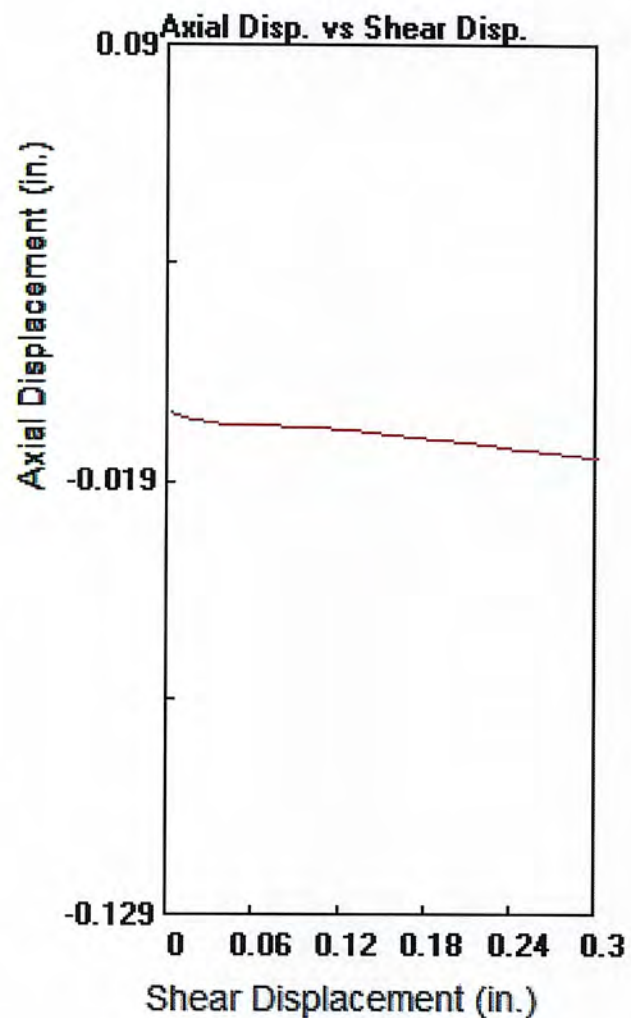
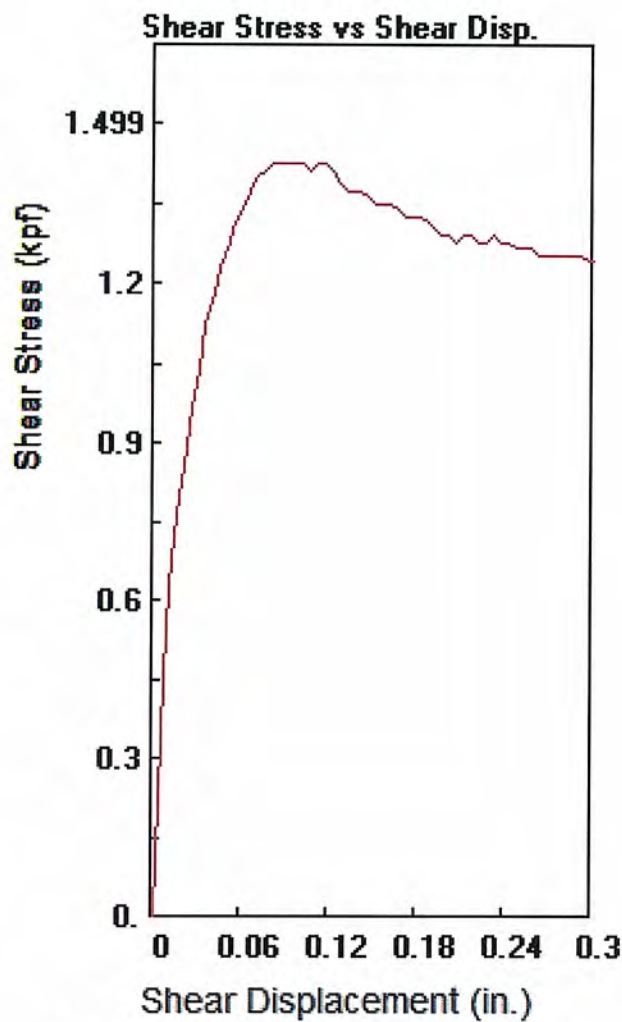
**Shear
Displacement
at maximum
Load**

0.0955 in.

Date

12/10/2021

Soil Labworks



Parameters

Client: FEFFER/BARDAS

Location: LAS PALMAS MCCADDEN PL

Job # 3823

Soil Type: ALLUVIUM

Sample: 2

Technician: BF

Boring: B2

Axial Load: 2000 psf

Depth: 15 ft.

Shear Rate: 0.010 in./sec.

File: 3823B2152.dat

Distance: 0.30 in.

Stress at Max Def
1428 0.081

Stress at Max Disp
0.296 1248

Maximum Load

1428 psf

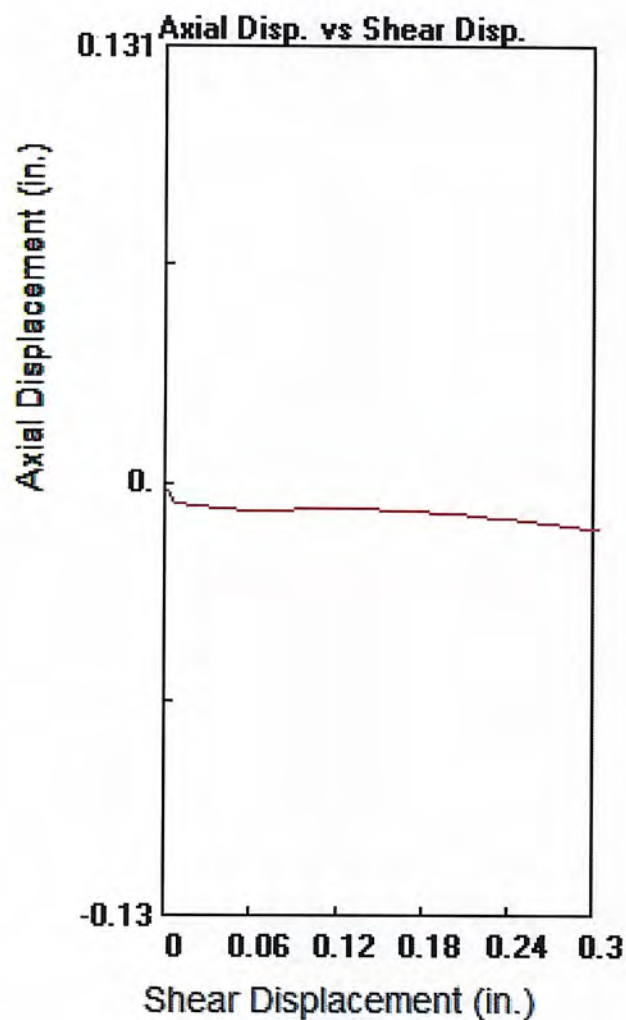
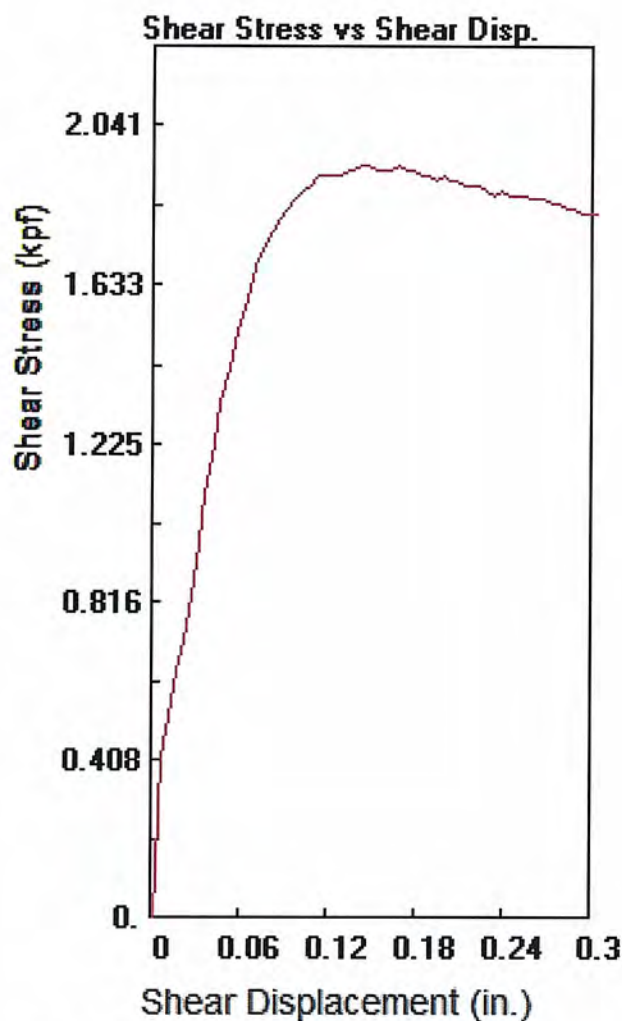
**Shear
Displacement
at maximum
Load**

0.0806 in.

Date

12/10/2021

Soil Labworks



Parameters

Client: FEFFER/BARDAS

Location: LAS PALMAS MCCADDEN PL

Job # 3823

Soil Type: ALLUVIUM

Sample: 3

Technician: BF

Boring: B2

Axial Load: 3000 psf

Depth: 15 ft.

Shear Rate: 0.010 in./sec.

File: 3823B2153.dat

Distance: 0.30 in.

Stress at Max Def
1944 0.141

Stress at Max Disp
0.296 1824

Maximum Load

1944 psf

**Shear
Displacement
at maximum
Load**

0.1407 in.

Date

12/10/2021

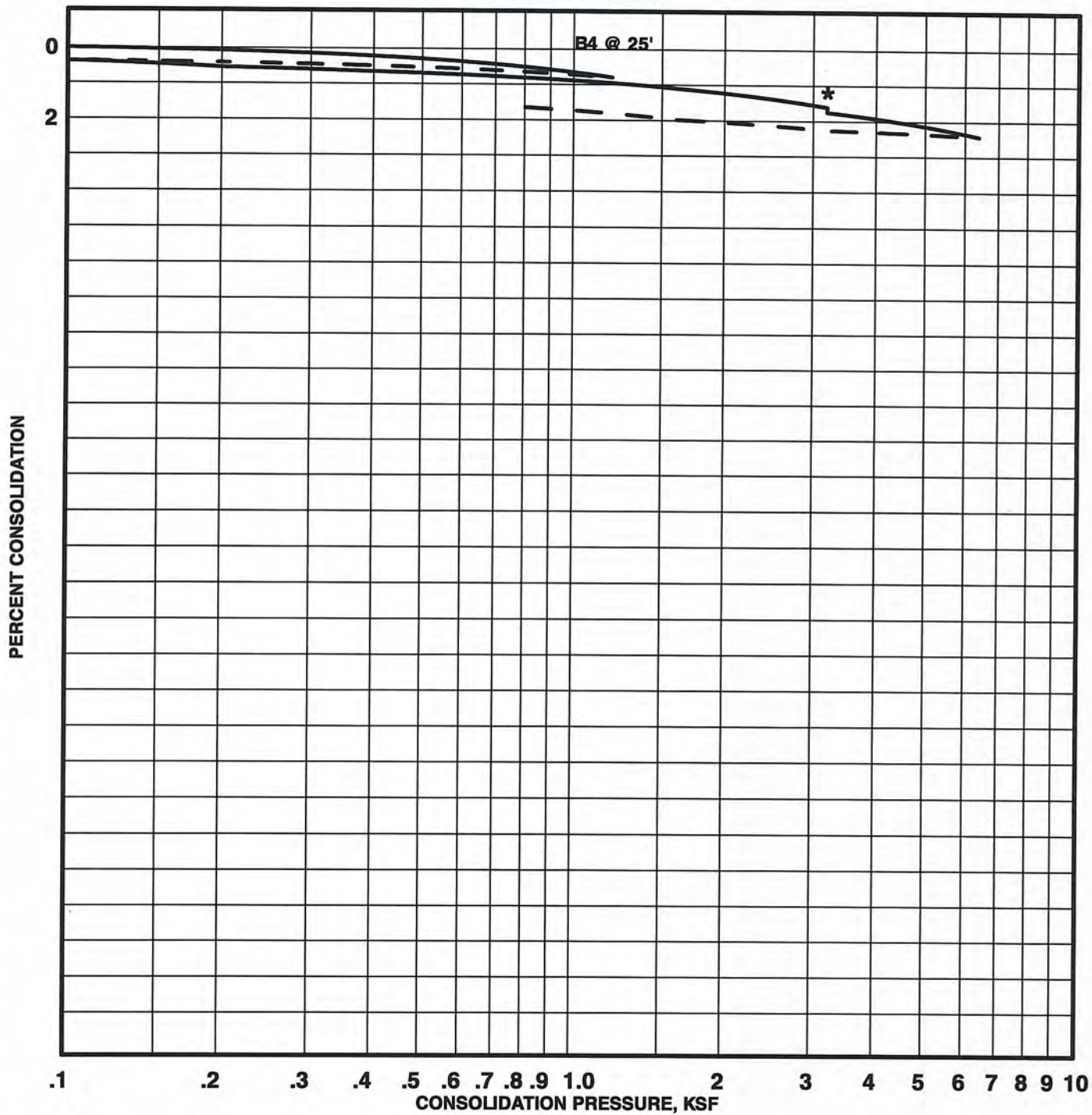
Soil Labworks

CONSOLIDATION TEST

PROJECT: 3823 FEFFER/BARBAS-LAS PALMAS AND MCCADDEN PLACE

SAMPLE: B4 @ 25'

ALLUVIUM



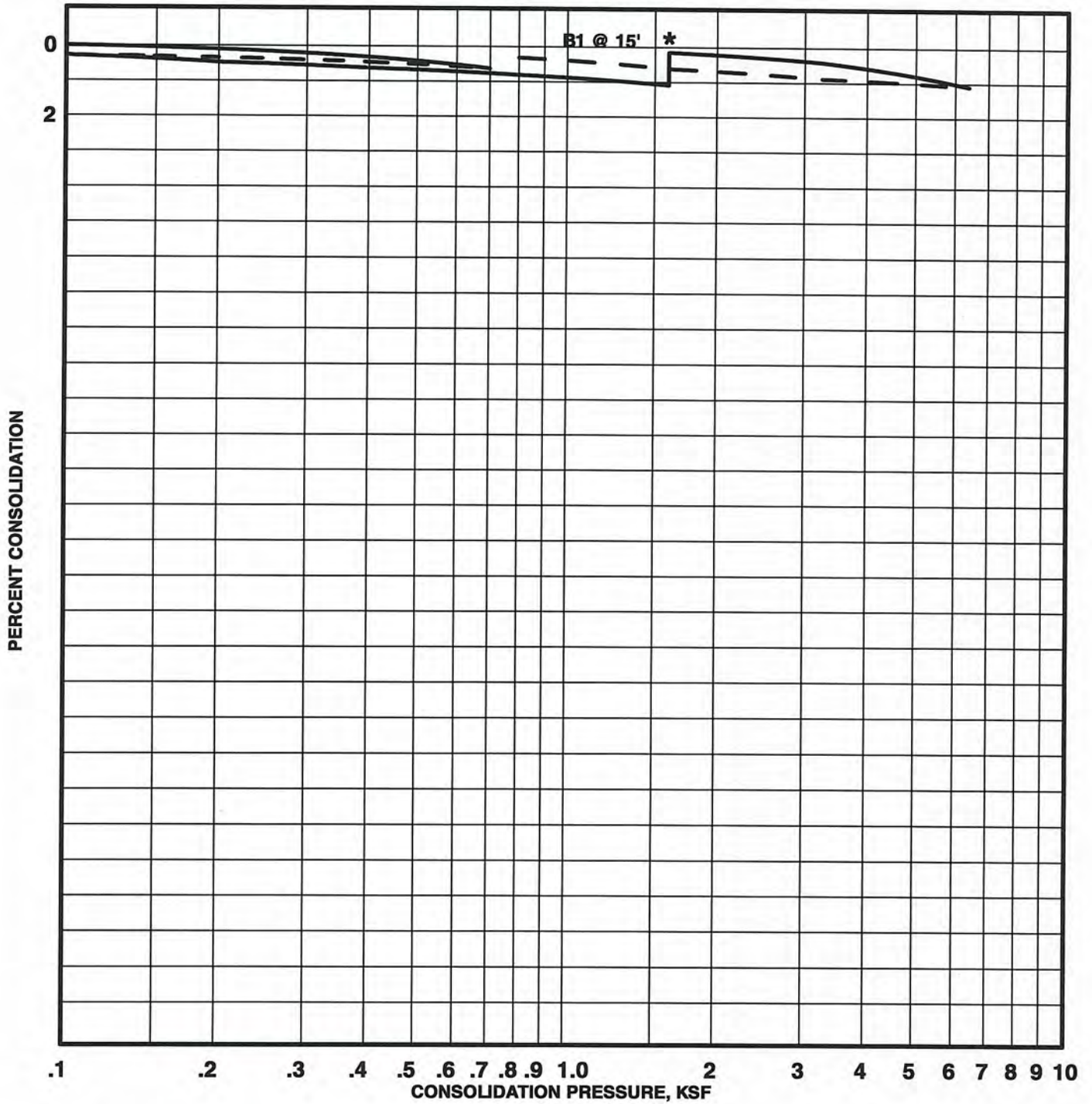
* Water Added

PLATE:

CONSOLIDATION TEST

PROJECT: 3823 FEFFER/BARBAS-LAS PALMAS AND MCCADDEN PLACE
SAMPLE: B1 @ 15'

ALLUVIUM



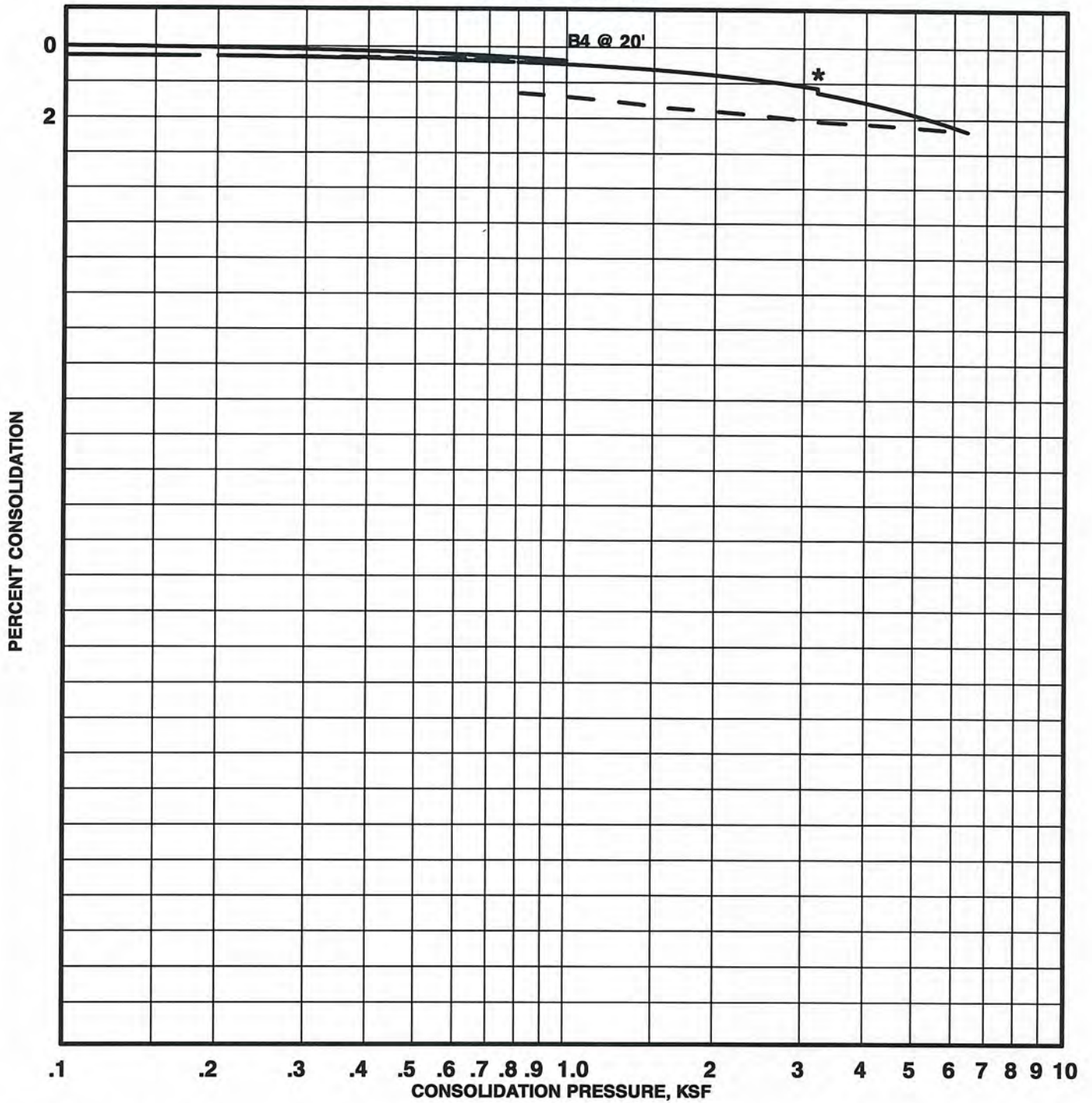
* Water Added

PLATE:

CONSOLIDATION TEST

PROJECT: 3823 FEFFER/BARBAS-LAS PALMAS AND MCCADDEN PLACE
SAMPLE: B4 @ 20'

ALLUVIUM



* Water Added

PLATE:

APPENDIX 'C'

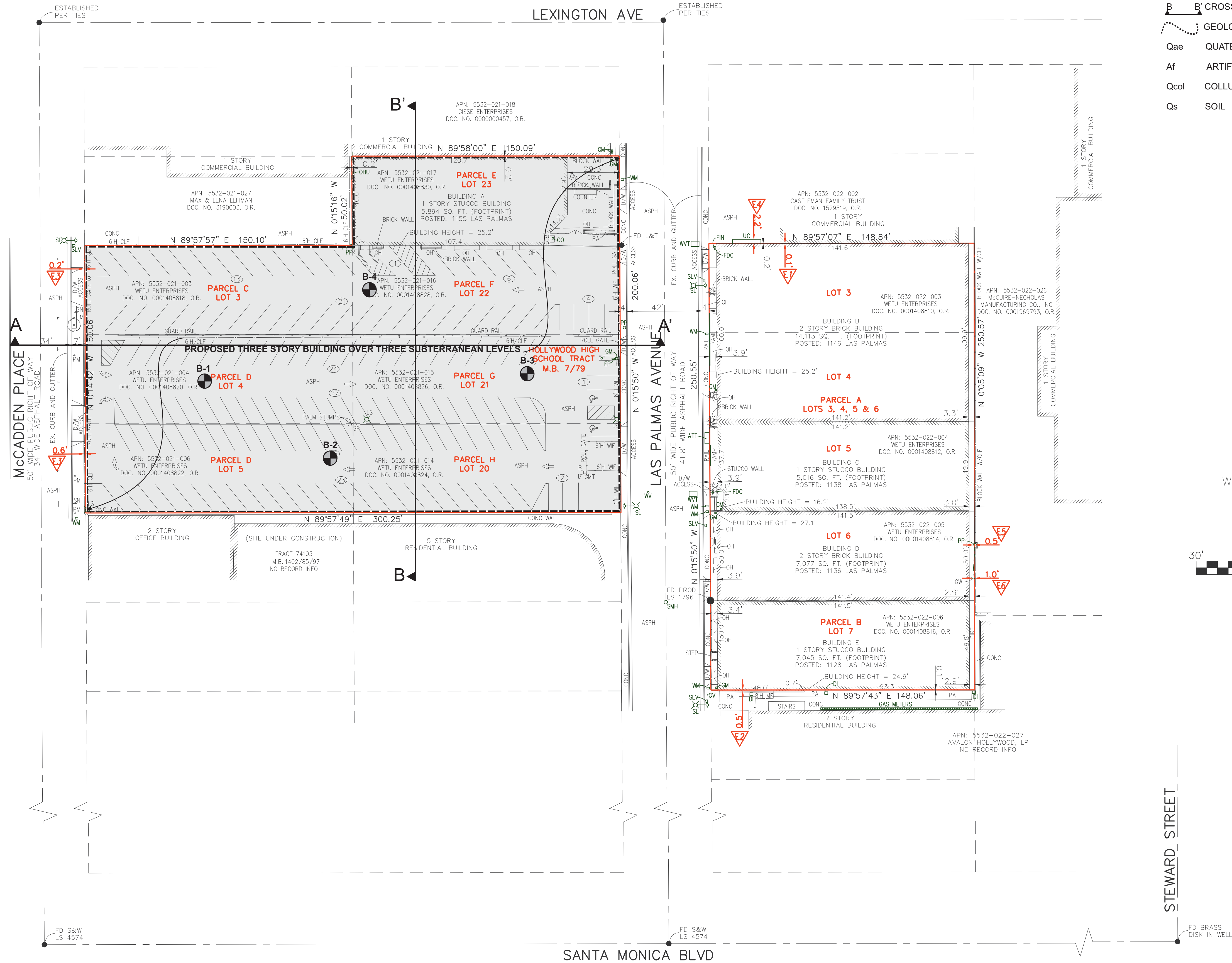
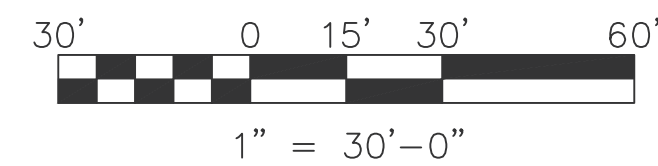
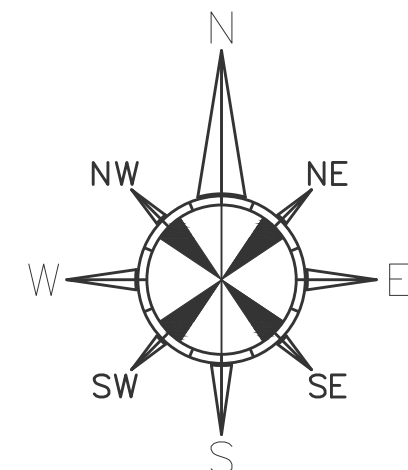
Site Plan & Cross Sections

FEFFER GEOLOGICAL CONSULTING			SITE PLAN		
JB: 2618-14		NAME: BARDAS INVESTMENT GROUP		BY: YMH	
DATE: 12/20/21		SCALE: 1"=30'		LAS PALMAS AVE	
REF:		BASE MAP FROM SURVEY		SITE: & MC CADDEN PL	

FEFFER GEOLOGICAL LEGEND	
B-4	LOCATION OF BORING FEFFER GEOLOGICAL
B	B' CROSS SECTION LINE
	GEOLOGIC CONTACT
Qae	QUATERNARY ALLUVIUM
Af	ARTIFICIAL FILL
Qcol	COLLUVIUM
Qs	SOIL

LEGEND

APN	ASSESSORS' PARCEL NUMBER
ASPH	ASPHALT
B	BOLLARD
CR	CABLE TV RISER
CONC	CONCRETE
DOC.	DOCUMENT
DI	DRAIN INLET
D/W	DRIVEWAY
DCV	DETECTOR CHECK VALVE
EV	ELECTRIC VAULT
FS	FIRE SERVICE
FIN	FIRE LINE INLET
FDC	FIRE DEPARTMENT CONNECTION
FP	FLAG POLE
GM	GAS METER
GTM	GATE MOTOR
GW	GUY WIRE
	HANDICAP SPACE
H	HEIGHT
LS	LIGHT STANDARD
NO.	NUMBER
O.R.	OFFICIAL RECORDS
PA	PLANTER AREA
PL	PROPERTY LINE
PP	POWER POLE
SN	SIGN
SP	SERVICE POLE
SL	STREET LIGHT
SMH	SEWER MANHOLE
SLV	STREET LIGHT VAULT
UC	UTILITY CABINET
W/	WITH
WM	WATER METER
WVT	WATER VAULT
WV	WATER VALVE
	STUCCO OR BLOCK WALL - TYPICAL
MF	METAL FENCE - TYPICAL
WIF	WROUGHT IRON FENCE - TYPICAL
CLF	CHAIN LINK FENCE - TYPICAL
OH	OVERHANG
OHU	OVERHEAD UTILITY LINE
	PAINTED HATCH AREA
	PARKING SPACE COUNTER
	DECIDUOUS TREE
	PALM TREE



ALTA/NSPS LAND TITLE SURVEY

FOR

PROPERTY LOCATED AT

1128, 1136, 1138, 1139, 1145, 1149
and 1155 N. Las Palmas Avenue
and 1138, 1140 and 1142 N. McCadden Place

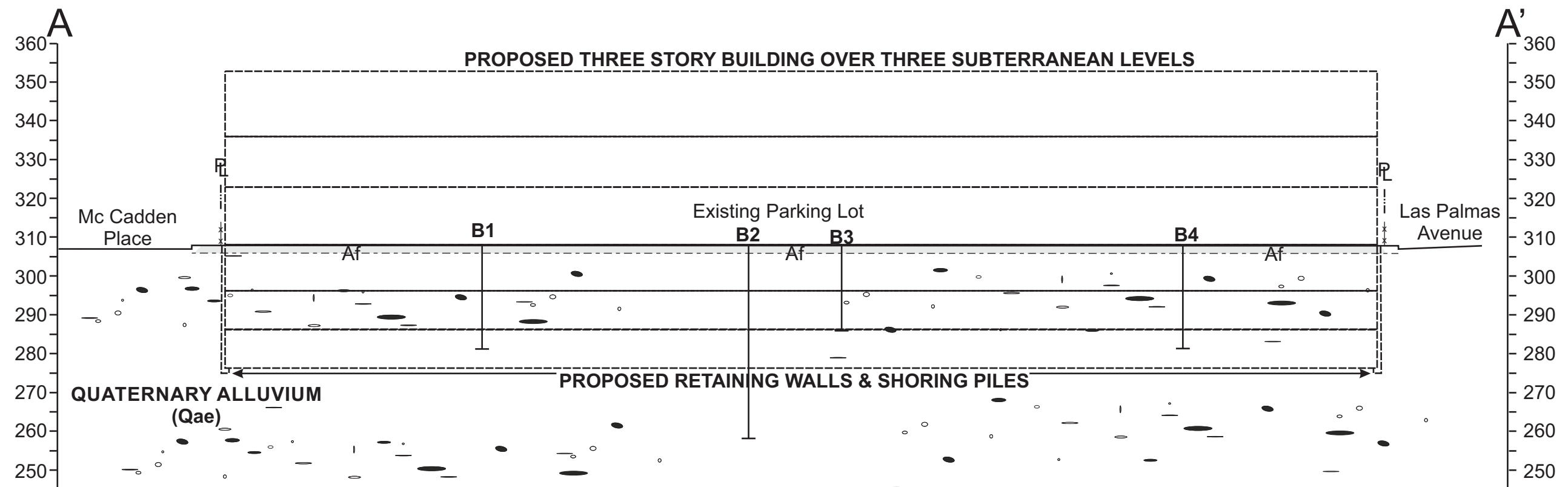
PARTNER
Engineering and Science, Inc.®

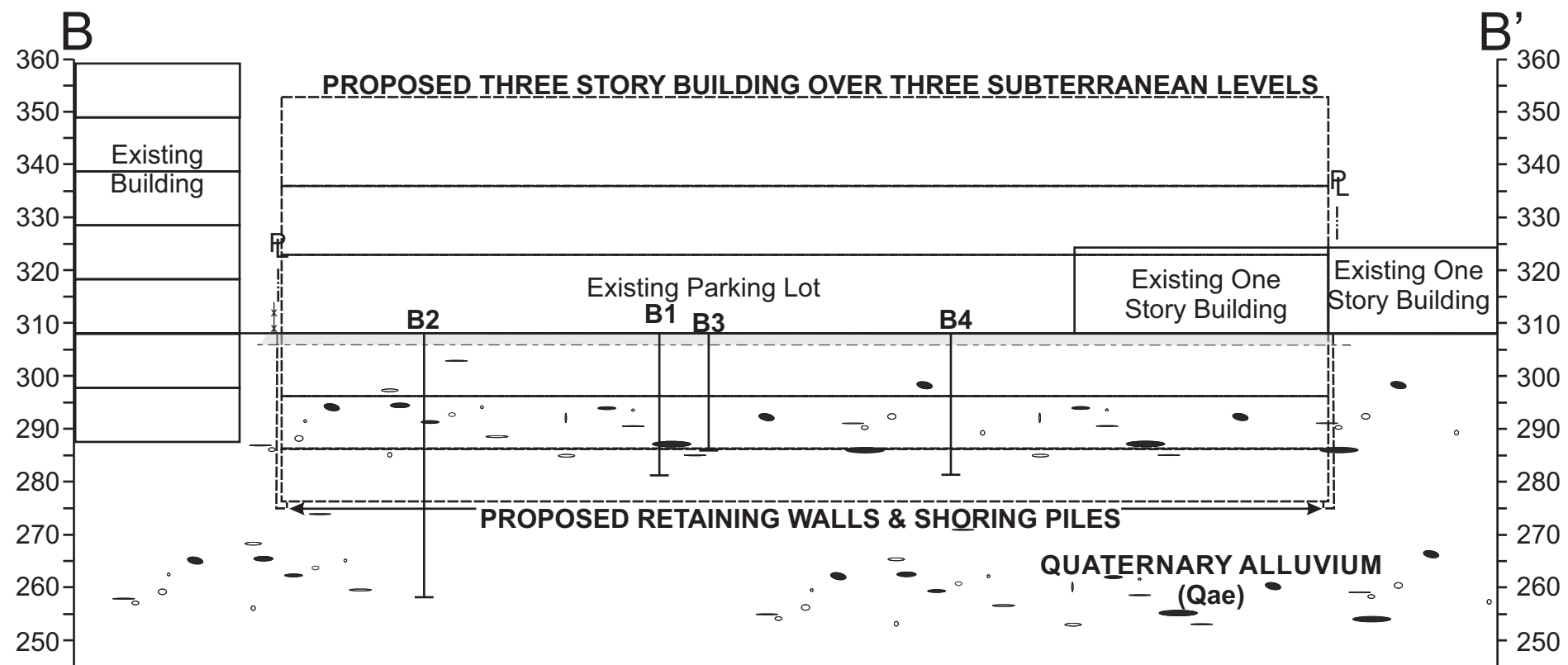
1761 GARRY AVENUE
SANTA ANA, CA 92705
949-930-9095
cdaniels@partneresi.com

LATEST REVISION DATE: 11/29/21
DATE OF MAP OR PLAT: 03/26/21

JN: 21-339659.2

PAGE 2 OF 2





FEFFER GEOLOGICAL CONSULTING		CROSS SECTION B-B'	
NAME: BARDAS		BY: YMH	
JB: 2618-14		INVESTMENT GROUP	
DATE: 12/20/21		LAS PALMAS AVE	
SCALE: 1"=30'		SITE: & MC CADDEN PL	
REF:		TOPOGRAPHY FROM SURVEY	

APPENDIX 'D'

Grading Specifications

STANDARD GRADING SPECIFICATIONS

These specifications present the usual and minimum requirements for grading operations performed under our supervision.

GENERAL

- 1) The Geotechnical Engineer and Engineering Geologist are the developer's representative on the project.
- 2) All clearing, site preparation or earth work performed on the project shall be conducted by the contractor under the supervision of the Geotechnical Engineer.
- 3) It is the contractor's responsibility to prepare the ground surface to receive the fills to the satisfaction of the Geotechnical Engineer and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Engineer. The contractor shall also remove all material considered unsatisfactory by the Geotechnical Engineer.
- 4) It is the contractor's responsibility to have suitable and sufficient compaction equipment on the job site to handle the amount of fill being placed. If necessary, excavation equipment will be shut down to permit completion of compaction. Sufficient watering apparatus will also be provided by the contractor, with due consideration for the fill material, rate of placement and time of year.
- 5) A final report shall be issued by our firm outlining the contractor's conformance with these specifications.

SITE PREPARATION

- 1) All vegetation and deleterious materials such as rubbish shall be disposed of off-site. Soil, alluvium or rock materials determined by the Geotechnical Engineer as being unsuitable for placement in compacted fills shall be removed and wasted from the site. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Engineer.
- 2) The Engineer shall locate all houses, sheds, sewage disposal systems, large trees or structures on the site or on the grading plan to the best of his knowledge prior to preparing the ground surface.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines, or others not located prior to grading are to be removed or treated in a manner prescribed by the Geotechnical Engineer.

3) After the ground surface to receive fill has been cleared, it shall be scarified, disced or bladed by the contractor until it is uniform and free from ruts, hollows, hummocks or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted as specified. If the scarified zone is greater than twelve inches (12") in depth, the excess shall be removed and placed in lifts restricted to six inches (6").

Prior to placing fill, the ground surface to receive fill shall be inspected, tested and approved by the Geotechnical Engineer.

PLACING, SPREADING AND COMPACTION OF FILL MATERIALS

1) The selected fill material shall be placed in layers which when compacted shall not exceed six inches (6") in thickness. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to insure uniformity of material and moisture of each layer.

2) Where the moisture content of the fill material is below the limits specified by the Geotechnical Engineer, water shall be added until the moisture content is as required to assure thorough bonding and thorough compaction.

3) Where the moisture content of the fill material is above the limits specified by the Geotechnical Engineer, the fill materials shall be aerated by blading or other satisfactory methods until the moisture content is adequate.

COMPACTED FILLS

1) Any material imported or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Engineer. Roots, tree branches or other matter missed during clearing shall be removed from the fill as directed by the Geotechnical Engineer.

2) Rock fragments less than six inches (6") in diameter may be utilized in the fill, provided:

- a) They are not placed in concentrated pockets.
- b) There is a sufficient percentage of fine-grained material to surround the rocks.
- c) The distribution of the rocks is supervised by the Geotechnical Engineer.

3) Rocks greater than six inches (6") in diameter shall be taken off-site, or placed in accordance with the recommendations of the Geotechnical Engineer in areas designated as suitable for rock disposal. Details for rock disposal such as location, moisture control, percentage of rock placed, will be referred to in the "Conclusions and Recommendations" section of the geotechnical report.

If the rocks greater than six inches (6") in diameter were not anticipated in the preliminary geotechnical and geology report, rock disposal recommendations may not have been made in the "Conclusions and Recommendations" section. In this case, the contractor shall notify the Geotechnical Engineer if rocks greater than six inches (6") in diameter are encountered. The Geotechnical Engineer will then prepare a rock disposal recommendation or request that such rocks be taken off-site.

4) Representative samples of materials to be utilized as compacted fill shall be analyzed in the laboratory by the Geotechnical Engineer to determine their physical properties. If any materials other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Engineer as soon as possible.

Material that is spongy, subject to decay or otherwise considered unsuitable shall not be used in the compacted fill.

5) Each layer shall be compacted to a minimum of ninety percent (90%) of the maximum density in compliance with the testing method specified by the controlling governmental agency (ASTM D-1557).

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive soil conditions, the area to receive fill compacted to less than ninety percent (90%) shall either be delineated on the grading plan or appropriate reference made to the area in the geotechnical report.

6) Compaction shall be by sheeps foot roller, multi-wheeled pneumatic tire roller, or other types of acceptable rollers. Rollers shall be of such design that they will be able to compact the fill to the specified density. Rolling shall be accomplished while the fill material is at the specified moisture content. The final surface of the lot areas to receive slabs-on-grade should be rolled to a smooth, firm surface.

7) Field density tests shall be made by the Geotechnical Engineer of the compaction of each layer of fill. Density tests shall be made at intervals not to exceed two feet (2') of fill height provided all layers are tested. Where the sheeps foot rollers are used, the soil may be disturbed to a depth of several inches and density readings shall be taken in the compacted material below the disturbed surface. When these readings indicate the density of any layer of fill or portion thereof is below the required ninety percent (90%) density, the particular layer or portion shall be reworked until the required density has been obtained.

8) Buildings shall not span from cut to fill. Cut areas shall be over excavated and compacted to provide a fill mat of three feet (3').

FILL SLOPES

1) All fills shall be keyed and benched through all top soil, colluvium, alluvium, or creep material into sound bedrock or firm material where the slope receiving fill exceeds a ratio of five (5) horizontal to one (1) vertical, in accordance with the recommendations of the Geotechnical Engineer.

2) The key for side hill fills shall be a minimum of fifteen feet (15') within bedrock or firm materials, unless otherwise specified in the geotechnical report.

3) Drainage terraces and subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, or with the recommendations of the Geotechnical Engineer.

4) The Contractor will be required to obtain a minimum relative compaction of ninety percent (90%) out to the finish slope face of fill slopes, buttresses, and stabilization fills. This may be achieved by either over-building

the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure which produces the required compaction.

5) All fill slopes should be planted or protected from erosion by methods specified in the geotechnical report and by the governing agency.

6) Fill-over-cut slopes shall be properly keyed through topsoil, colluvium, or creep material into rock or firm materials. The transition zone shall be stripped of all soil prior to placing fill.

CUT SLOPES

1) The Engineering Geologist shall inspect all cut slopes excavated in rock, lithified, or formation material at vertical intervals not exceeding ten feet (10').

2) If any conditions not anticipated in the preliminary report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints, or fault planes, are encountered during grading, these conditions shall be analyzed by the Engineering Geologist and Geotechnical Engineer; and recommendations shall be made to treat these problems.

3) Cut slope that face in the same direction as the prevailing drainage shall be protected from slope wash by a non-erosive interceptor swale placed at the top of the slope.

4) Unless otherwise specified in the geological and geotechnical report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of the controlling governmental agencies.

5) Drainage terraces shall be constructed in compliance with the ordinances of controlling governmental agencies, or with the recommendations of the Geotechnical Engineer or Engineering Geologist.

GRADING CONTROL

1) Inspection of the fill placement shall be provided by the Geotechnical Engineer during the progress of grading.

2) In general, density tests should be made at intervals not exceeding two feet (2') of fill height or every five hundred (500) cubic yards of fill placed. These criteria will vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests shall be made to verify that the required compaction is being achieved.

3) Density tests should also be made on the surface materials to receive fill as required by the Geotechnical Engineer.

4) All clean-out, processed ground to receive fill, key excavations, subdrains, and rock disposal must be inspected and approved by the Geotechnical Engineer prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Engineer when such areas are ready for inspection.

CONSTRUCTION CONSIDERATIONS

1) Erosion control measures, when necessary, shall be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.

2) Upon completion of grading and termination of inspections by the Geotechnical Engineer, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Engineer or Engineering Geologist.

3) Care shall be taken by the contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of a permanent nature on or adjacent to the property.

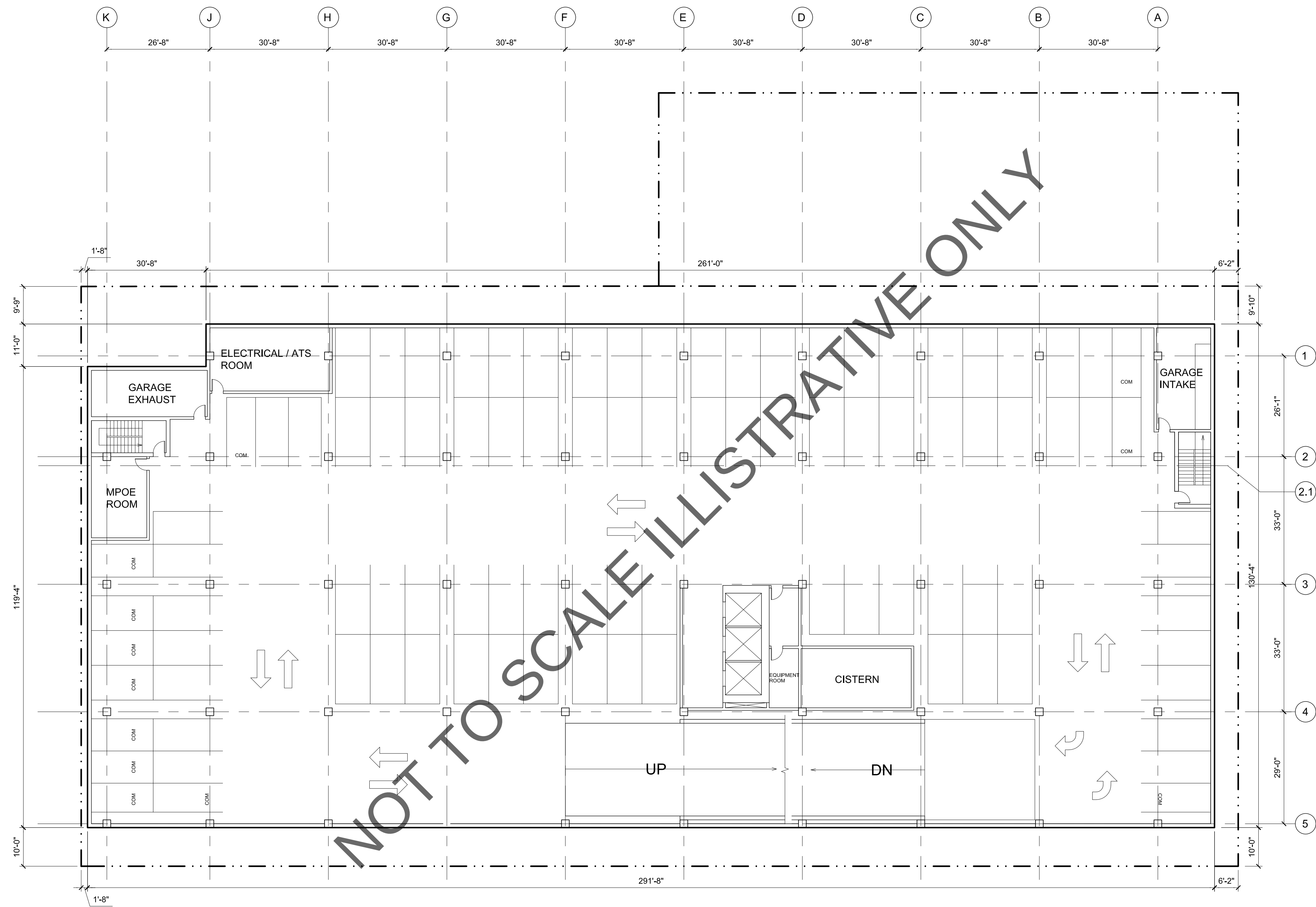
APPENDIX 'E'

Architectural Development Plans

LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

No.	Description	Date

Drawing Title:	
PARKING LEVEL 03 PLAN	
Scale: 1/16" = 1'-0"	
Project No: 2103	Project North: ↑
Drawing No: A - P3	Rev No
NOV. 11, 2021	



LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

No.	Description	Date

Drawing Title:
**PARKING
LEVEL 02 PLAN**

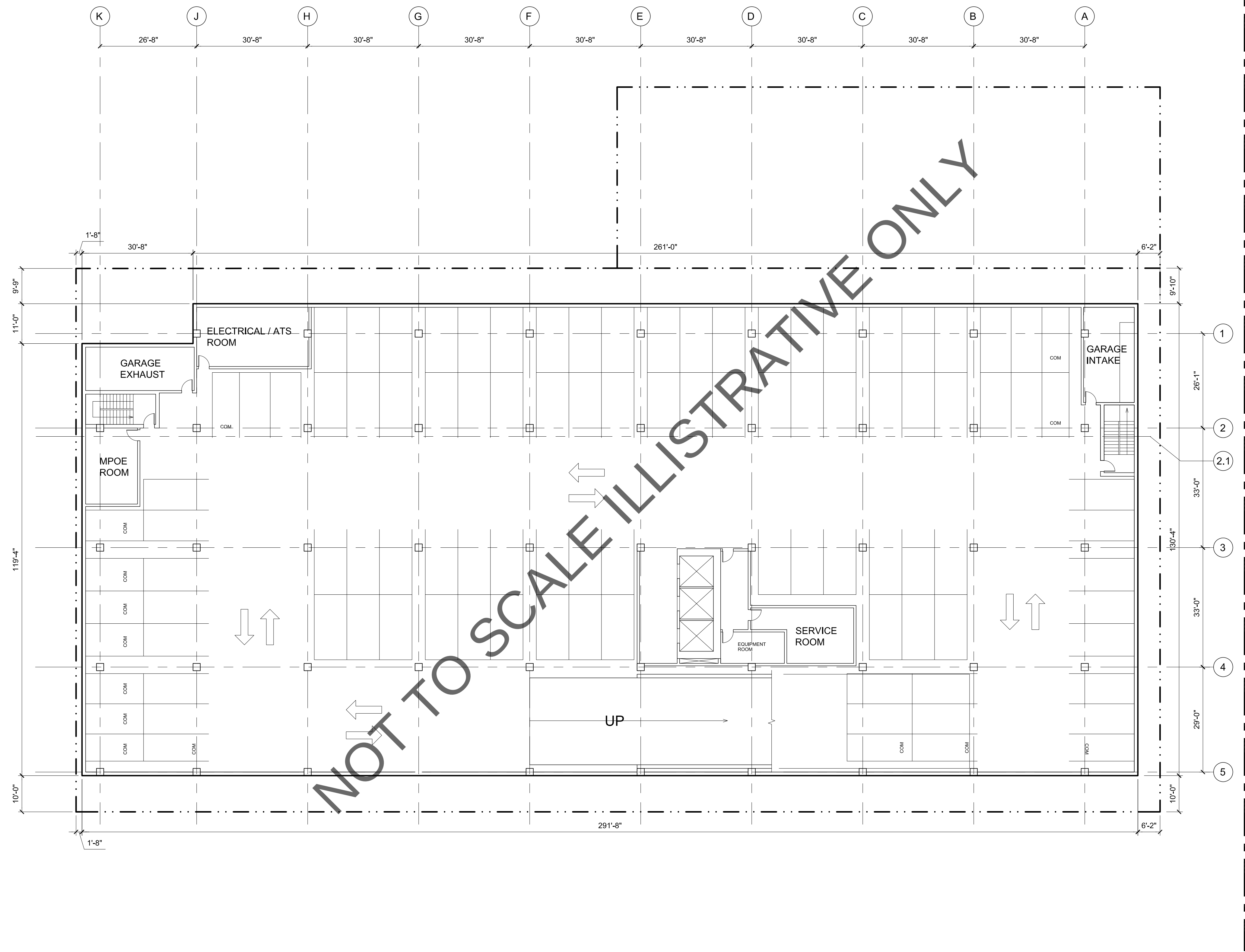
Scale:
1/16" = 1'-0"

Project No:
2103

Project North:
↑

Drawing No:
A - P2
NOV. 11, 2021

Rev No



LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

No.	Description	Date

Drawing Title:
**PARKING
LEVEL 01 PLAN**

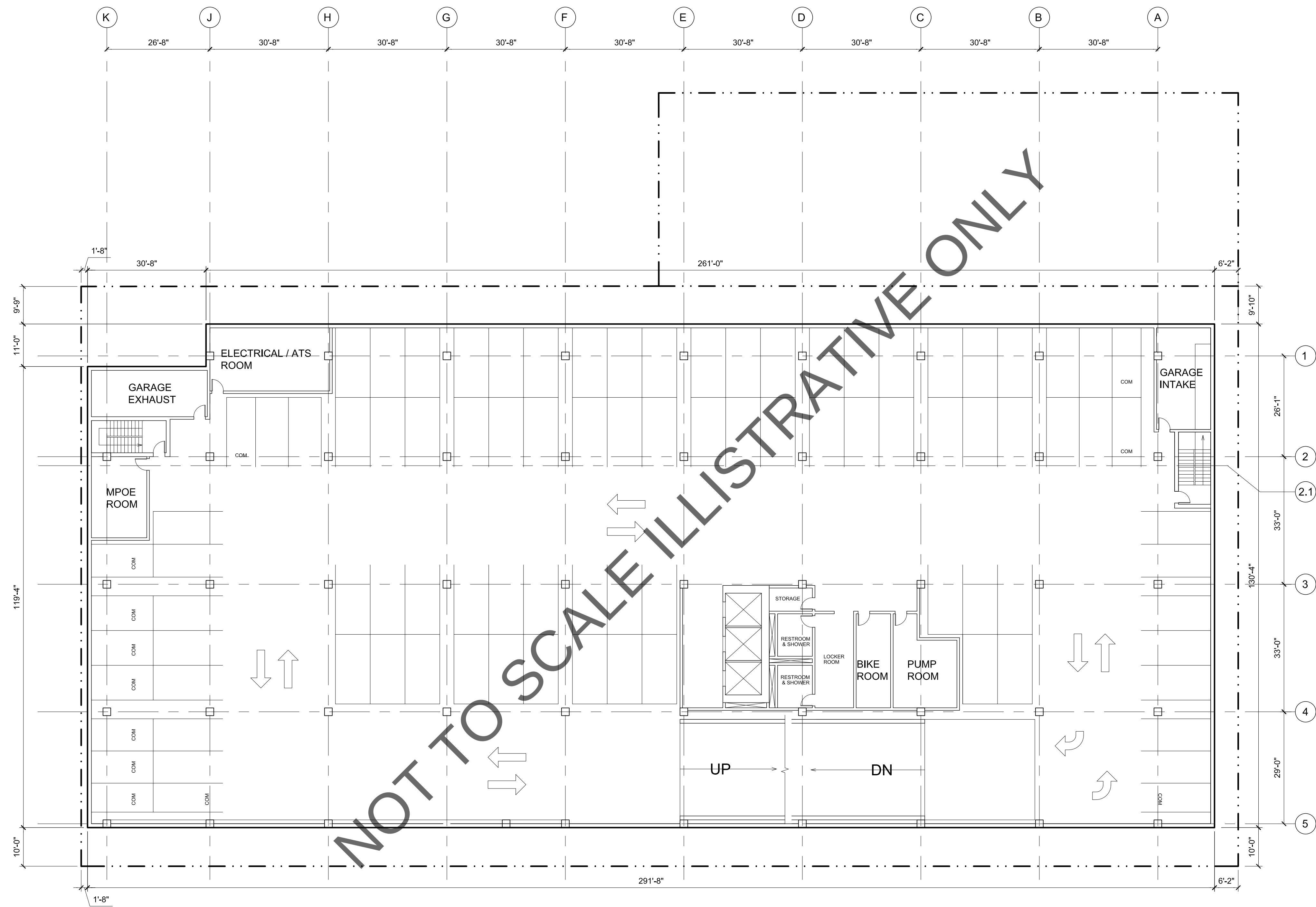
Scale:
1/16" = 1'-0"

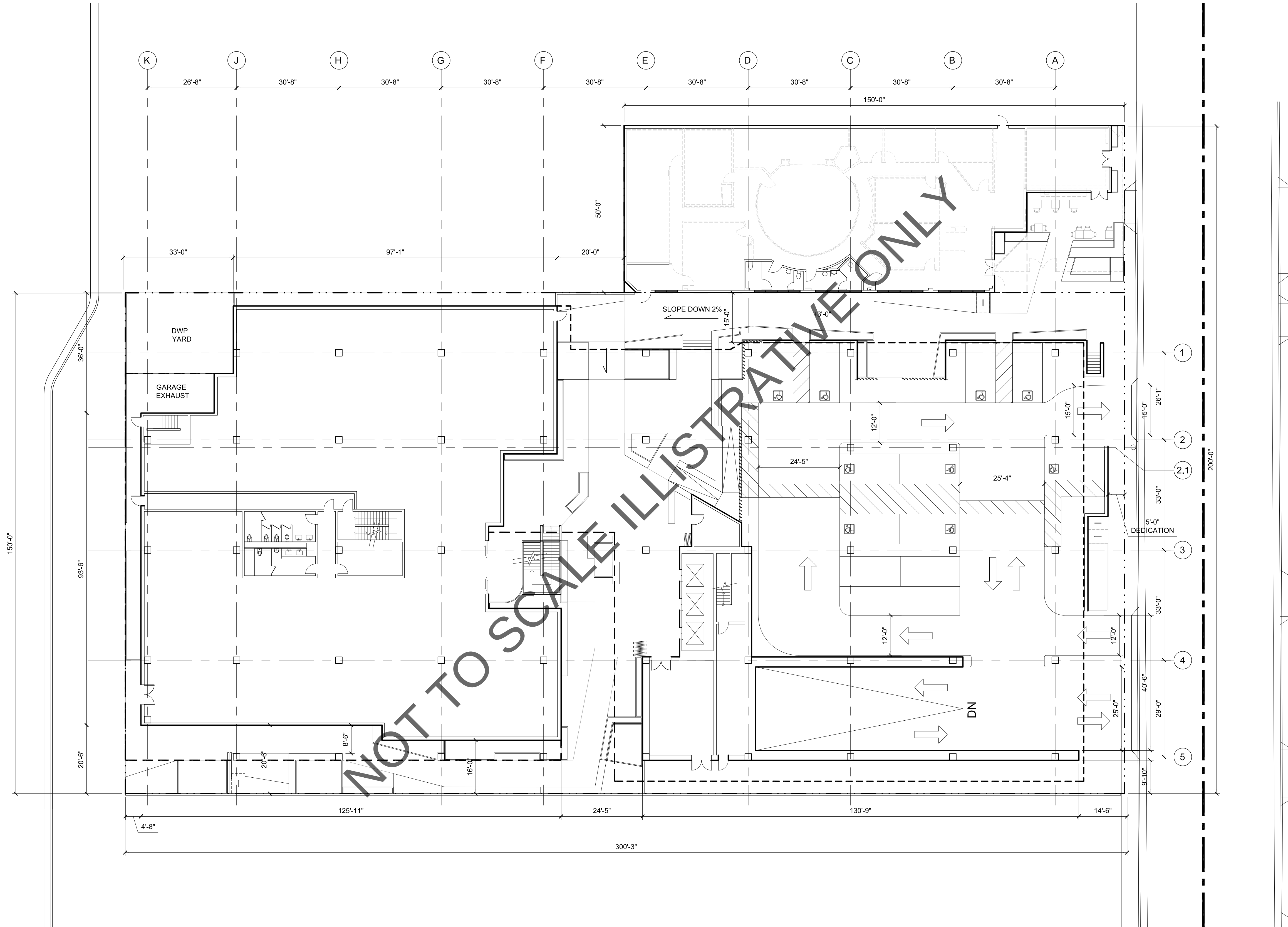
Project No:
2103

Project North:
↑

Drawing No:
A - P1
NOV. 11, 2021

Rev No





LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

No.	Description	Date

Drawing Title:
**GROUND FLOOR
PLAN**

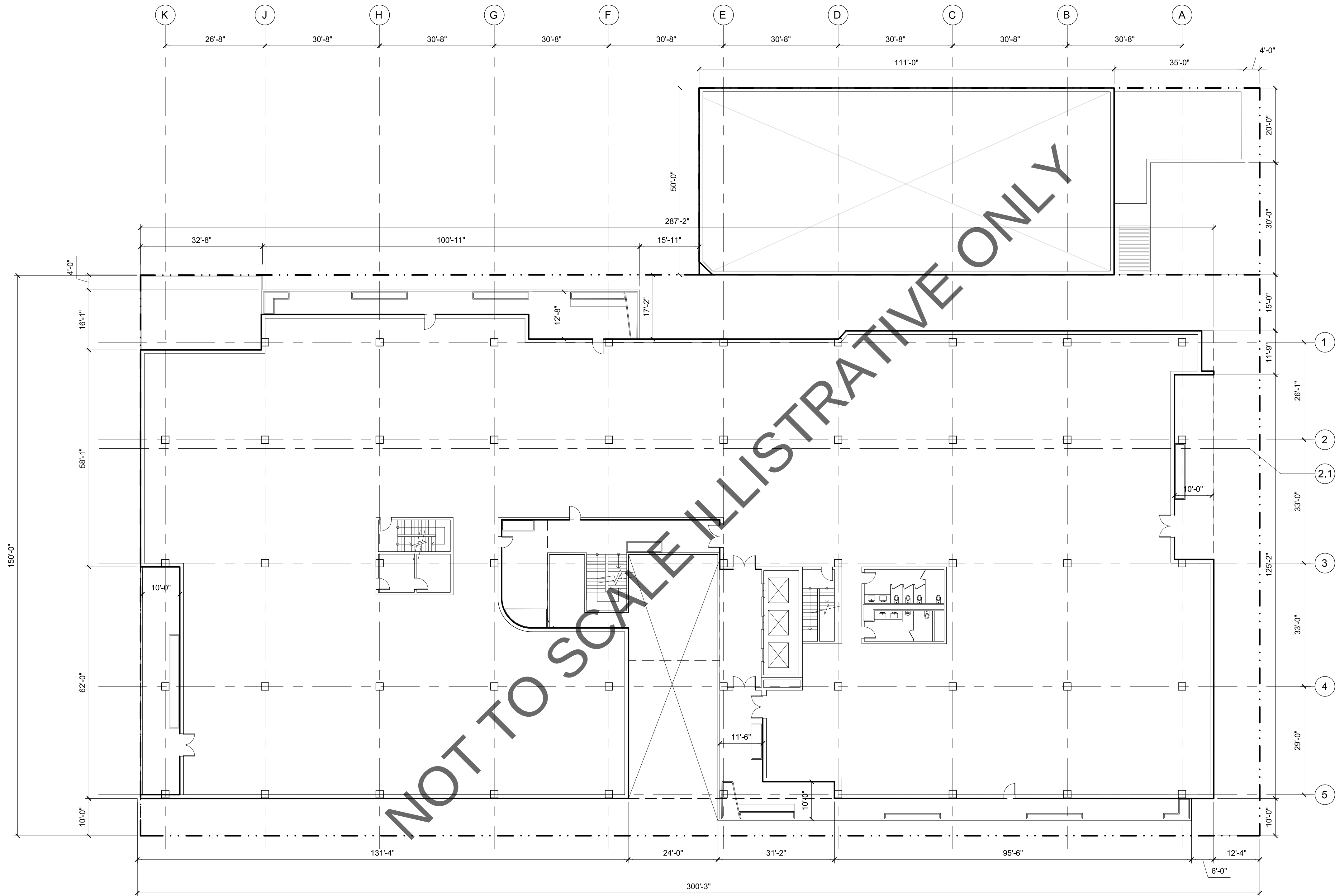
Scale:
1/16" = 1'-0"

Project No:
2103

Project North:
↑

Drawing No:
A - 101

Rev No
NOV. 11, 2021



LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

No.	Description	Date

Drawing Title:
**SECOND FLOOR
PLAN**

Scale:
1/16" = 1'-0"

Project No:
2103

Project North:
▲

Drawing No:
A - 102

Rev No
NOV. 11, 2021



LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

No.	Description	Date

Drawing Title:
**THIRD FLOOR
PLAN**

Scale:
1/16" = 1'-0"

Project No:
2103

Project North:
↑

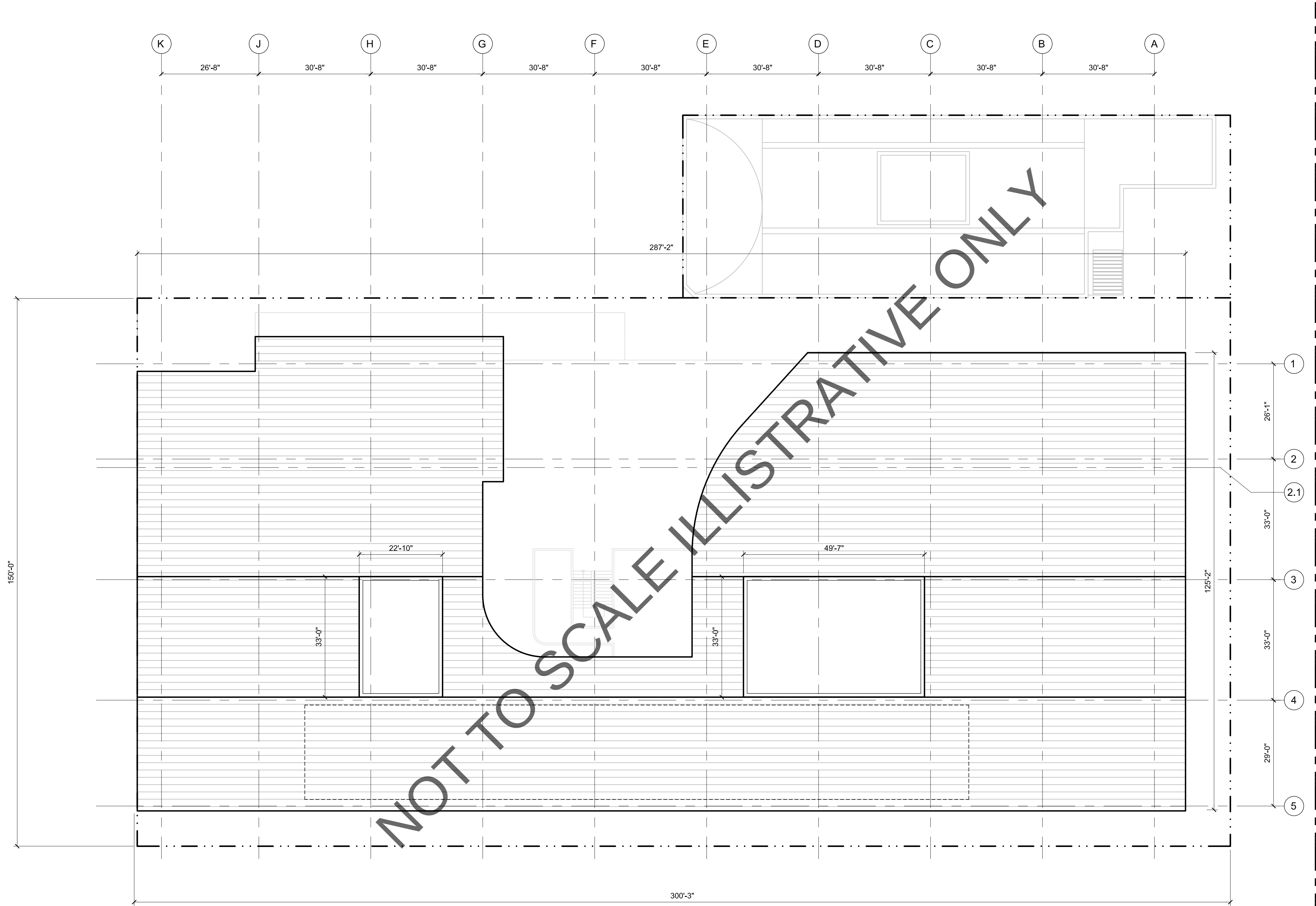
Drawing No:
A - 103

Rev No
NOV. 11, 2021

LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

No.	Description	Date

Drawing Title:	
ROOF FLOOR PLAN	
Scale: 1/16" = 1'-0"	
Project No: 2103	Project North: ▲
Drawing No: A - 104	Rev No
NOV. 11, 2021	



LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

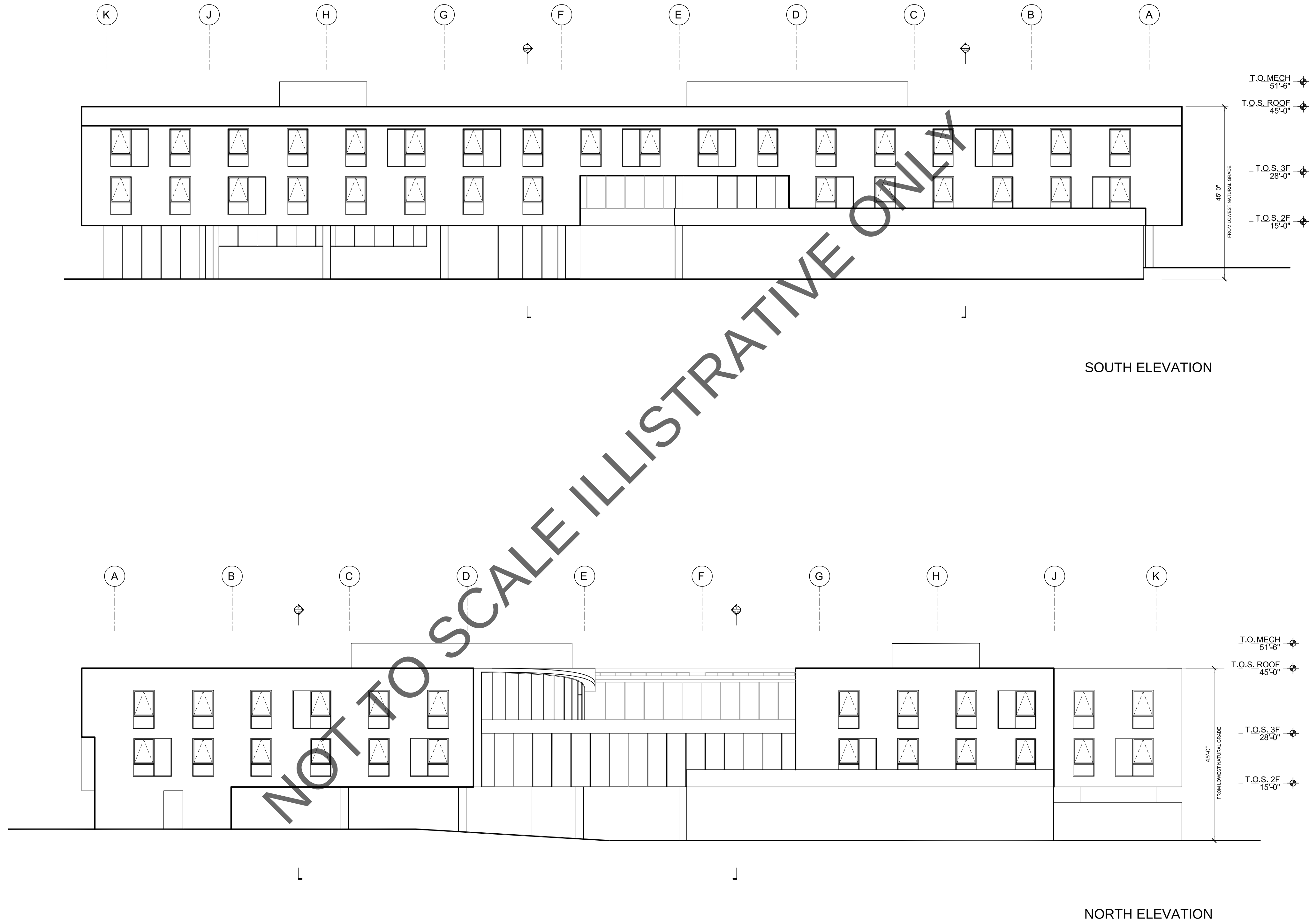
No.	Description	Date

Drawing Title:
**NORTH & SOUTH
ELEVATIONS**

Scale:
1/16" = 1'-0"

Project No: 2103	Project North: ▲
---------------------	---------------------

Drawing No: A - 201 NOV. 11, 2021	Rev No
--	--------



LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

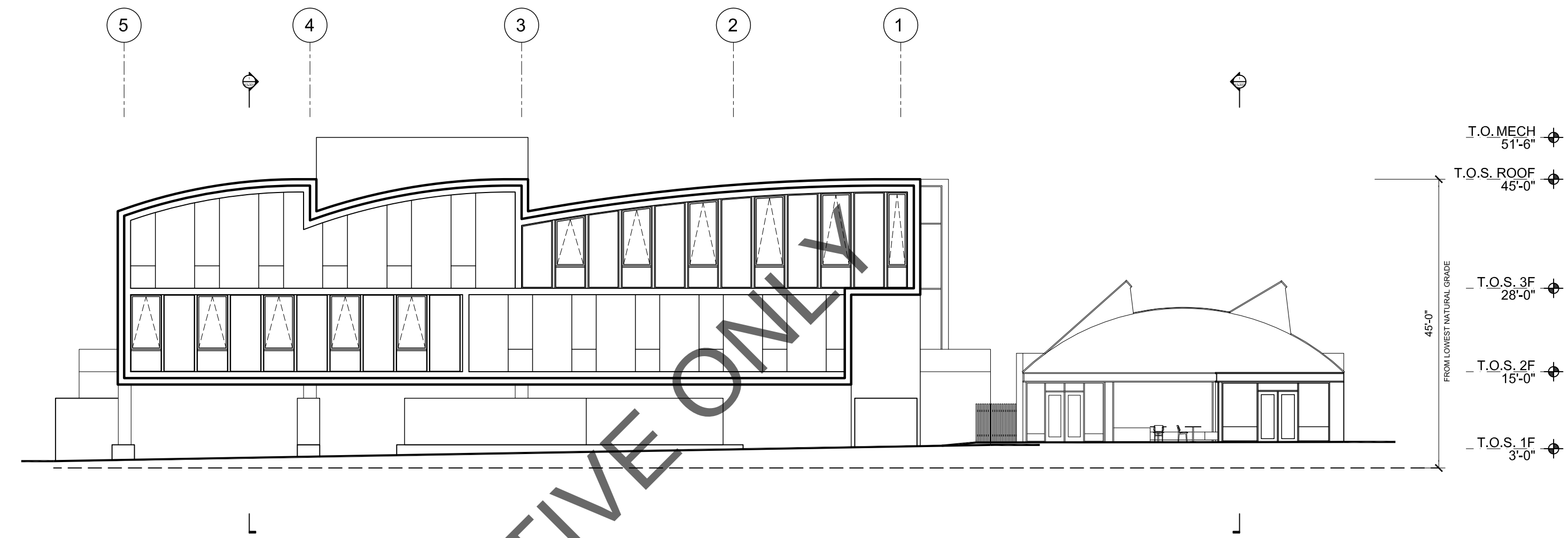
No.	Description	Date

Drawing Title:
**EAST & WEST
ELEVATIONS**

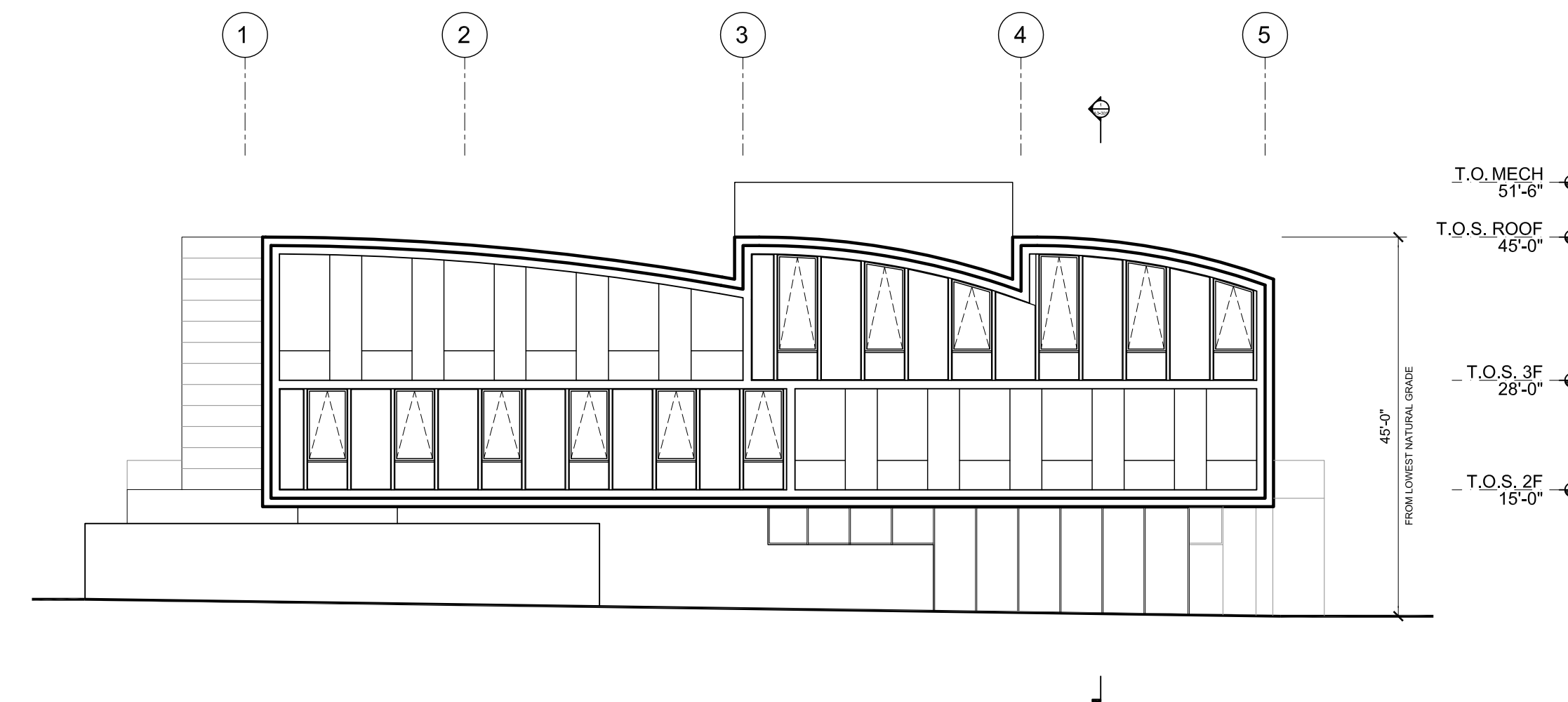
Scale:
1/16" = 1'-0"

Project No: 2103	Project North: ▲
---------------------	---------------------

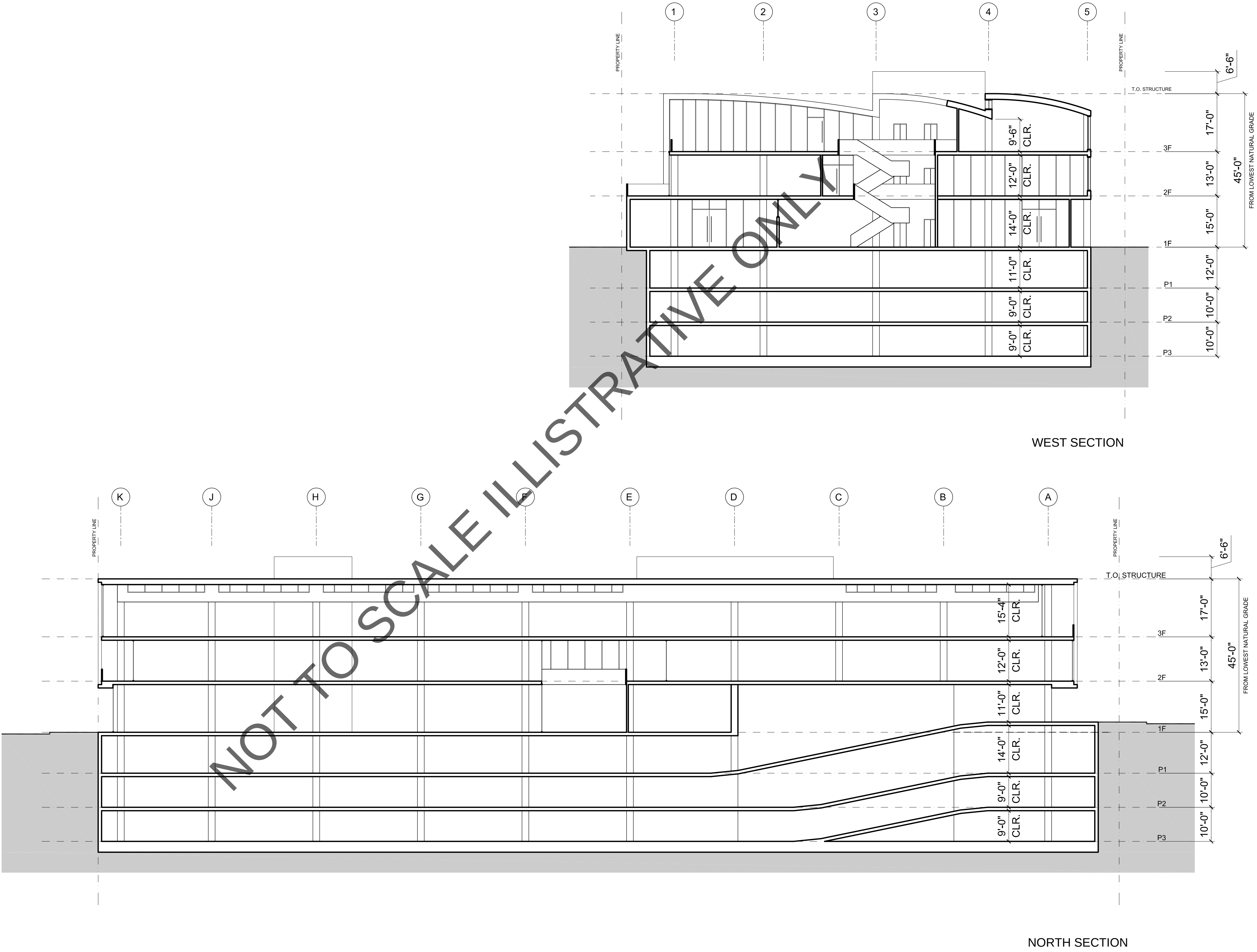
Drawing No: A - 202	Rev No: NOV. 11, 2021
-------------------------------	--------------------------



EAST ELEVATION



WEST ELEVATION





923 E. 3rd Street #107
Los Angeles, CA 90013

LAS PALMAS
1149 LAS PALMAS
LOS ANGELES, CA 90038

No.	Description	Date

Drawing Title:

BUILDING SECTIONS

Scale:

1/16" = 1'-0"

Project No:

2103

Project North:

↑

Drawing No:

A - 203

Rev No:

NOV. 11, 2021

APPENDIX 'F'

Engineering Analysis

RETAINING WALL

IC: **2618-14** CONSULT: **YMH**
CLIENT: **BARDAS Investment Group**

CALCULATION SHEET #

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBÉ-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium	WALL HEIGHT	35 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	325 psf	SURCHARGE:	300 pounds
PHI ANGLE:	27 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	126 pcf	INITIAL FAILURE ANGLE:	10 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	10 degrees	INITIAL TENSION CRACK:	4 feet
CD (C/FS):	325.0 psf	FINAL TENSION CRACK:	35 feet
PHID = $\text{ATAN}(\text{TAN}(\text{PHI})/\text{FS}) =$	27.0 degrees		
HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h)		0.36 %g	
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v)		0 %g	

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	41 degrees
AREA OF TRIAL FAILURE WEDGE	692.6 square feet
TOTAL EXTERNAL SURCHARGE	9300.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	96562.8 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1952 trials
LENGTH OF FAILURE PLANE	46.4 feet
DEPTH OF TENSION CRACK	4.6 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	35.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	43889.9 pounds

FOR A GROUND MOTION OF 0.36 AND A FS OF 1.0, THE ENCLOSED CALCULATIONS INDICATE AN UNBALANCED FORCE UNDER SEISMIC CONDITIONS FROM THE MAXIMUM CONSIDERED EARTHQUAKE IS 43889.9 POUNDS OR 43.89 KIPS.

RETAINING WALL

IC: **2618-14** CONSULT: **YMH**
CLIENT: **BARDAS Investment Group**

CALCULATION SHEET #

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBÉ-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium	WALL HEIGHT	35 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	325 psf	SURCHARGE:	300 pounds
PHI ANGLE:	27 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	126 pcf	INITIAL FAILURE ANGLE:	10 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	10 degrees	INITIAL TENSION CRACK:	4 feet
CD (C/FS):	216.7 psf	FINAL TENSION CRACK:	35 feet
PHID = $\text{ATAN}(\text{TAN}(\text{PHI})/\text{FS})$ =	18.8 degrees		
HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h)			0 %g
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v)			0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	52 degrees
AREA OF TRIAL FAILURE WEDGE	475.0 square feet
TOTAL EXTERNAL SURCHARGE	6300.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	66152.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1952 trials
LENGTH OF FAILURE PLANE	40.6 feet
DEPTH OF TENSION CRACK	3.0 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	25.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	30532.4 pounds
CALCULATED EQUIVALENT FLUID PRESSURE	49.8 pcf
DESIGN EQUIVALENT FLUID PRESSURE	55.0 pcf

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALL MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 55 POUNDS PER CUBIC FOOT.

SHORING PILE

IC: **2618-14** CONSULT: **YMH**
CLIENT: **BARDAS Investment Group**

CALCULATION SHEET #

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBÉ-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium	RETAINED LENGTH	35 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	325 psf	SURCHARGE:	300 pounds
PHI ANGLE:	27 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	126 pcf	INITIAL FAILURE ANGLE:	10 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	10 degrees	INITIAL TENSION CRACK:	4 feet
CD (C/FS):	260.0 psf	FINAL TENSION CRACK:	35 feet
PHID = ATAN(TAN(PHI)/FS) =	22.2 degrees		
HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h)			0 %g
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v)			0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	54 degrees
AREA OF TRIAL FAILURE WEDGE	440.9 square feet
TOTAL EXTERNAL SURCHARGE	5700.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	61259.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1952 trials
LENGTH OF FAILURE PLANE	39.1 feet
DEPTH OF TENSION CRACK	3.3 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	23.0 feet
CALCULATED THRUST ON PILE	24385.0 pounds
CALCULATED EQUIVALENT FLUID PRESSURE	39.8 pcf
DESIGN EQUIVALENT FLUID PRESSURE	43.0 pcf

THE CALCULATION INDICATES THAT THE PROPOSED SHORING PILES MAY MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 43 POUNDS PER CUBIC FOOT. THE FLUID PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

TEMPORARY EXCAVATION HEIGHT

IC: **2618-14** CONSULT: **YMH**
CLIENT: **BARDAS Investment Group**

CALCULATION SHEET #

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:	325 psf	SURCHARGE:	300 pounds
PHI ANGLE:	27 degrees	SURCHARGE TYPE:	U Uniform
DENSITY:	126 pcf	INITIAL FAILURE ANGLE:	17 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	260.0 psf	FINAL TENSION CRACK:	5 feet
PHID = ATAN(TAN(PHI)/FS) =	22.2 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	48 degrees
AREA OF TRIAL FAILURE WEDGE	23.6 square feet
TOTAL EXTERNAL SURCHARGE	900.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	3875.8 pounds
NUMBER OF TRIAL WEDGES ANALYZED	216 trials
LENGTH OF FAILURE PLANE	7.5 feet
DEPTH OF TENSION CRACK	4.4 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	5.0 feet
CALCULATED HORIZONTAL THRUST	-123.1 pounds
CALCULATED EQUIVALENT FLUID PRESSURE	-9.8 pcf
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	5.0 feet

CONCLUSIONS:

THE CALCULATION INDICATES THAT THE TEMPORARY EXCAVATIONS UP TO 5 FEET HIGH IN ALLUVIUM WITH A 1:1 BACKSLOPE HAVE A NEGATIVE THRUST AND ARE TEMPORARILY STABLE.



2618-14

1139 N Las Palmas Ave, Los Angeles, CA 90038, USA

Latitude, Longitude: 34.0918975, -118.3366343



Date	12/21/2021, 1:01:10 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	2.097	MCE_R ground motion. (for 0.2 second period)
S_1	0.752	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.516	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.677	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.898	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.078	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
S_{sRT}	2.097	Probabilistic risk-targeted ground motion. (0.2 second)
S_{sUH}	2.338	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
S_{sD}	2.273	Factored deterministic acceleration value. (0.2 second)
S_{1RT}	0.752	Probabilistic risk-targeted ground motion. (1.0 second)
S_{1UH}	0.838	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S_{1D}	0.754	Factored deterministic acceleration value. (1.0 second)
$PGAd$	0.922	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.897	Mapped value of the risk coefficient at short periods

APPENDIX ‘G’

Research

See CD