APPENDIX E

Geotechnical Report
(Including LABDS Approval Letter)

BOARD OF
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201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

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

JOHN WEIGHT
EXECUTIVE OFFICER

SOILS REPORT APPROVAL LETTER

May 29, 2025

LOG # 135408
SOILS/GEOLOGY FILE - 2

Stewart Manhattan, Inc.
2233 Honolulu Ave., Suite 313
Montrose, CA 91020

TRACT:	RANCHO CIENEGA O'PASO DE LA TIJERA (D C C 2253 C F 900 PAT 1-259)
LOT(S):	LT 41 (Arb 78)
LOCATION:	3701-3761 W STOCKER ST

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Soils Report	22516	05/23/2025	Geotechnologies, Inc.
Oversized Doc.	"	"	

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed creative campus consisting one- to six-story at-grade buildings.

The earth materials at the subsurface exploration locations consist of up to 7.5 feet of uncertified fill underlain by sandy to clayey silts, silty sands, pebbly sands, cobbly sands, gravelly sands and sands.

The consultants recommend to support the proposed structure(s) on conventional foundations bearing on native undisturbed soils or 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 2023 City of LA Building Code. P/BC numbers refer to the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. 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).

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

3. 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.

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

5. 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.

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

7. Compacted fill shall extend beyond the footings a minimum distance equal to the depth of the fill below the bottom of footings or a minimum of three feet whichever is greater (7011.3).

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

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

10. 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).

11. 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)

12. 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).

13. The soils engineer shall review and approve the shoring and/or underpinning plans prior to issuance of the permit (3307.3.2).

14. Prior to the issuance of the permits, the soils engineer and the structural designer shall evaluate all applicable surcharge loads for the design of the retaining walls and shoring.

15. Unsurcharged temporary excavation may be cut vertical up to 5 feet. Excavations over 5 feet to a maximum height of 20 feet shall be trimmed back at a uniform gradient not exceeding 1:1, from top to bottom of excavation, as recommended.

16. Shoring shall be designed for the lateral earth pressures specified in the section titled "Shoring Design" starting on page 31 of the referenced report; all surcharge loads shall be included into the design.

17. 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.

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

19. All foundations shall derive entire support from native undisturbed soils or compacted fill, as recommended and shall be approved by the geologist and soils engineer by inspection.

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. 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.

23. Concrete floor slabs placed on expansive soil shall be placed on a 4-inch fill of coarse aggregate or on a moisture barrier membrane.

24. 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.

25. Retaining walls shall be designed for the lateral earth pressures specified in the section titled "Retaining Wall Design" starting on page 24 of the referenced report. All surcharge loads shall be included into the design.

26. 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).

27. 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).

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

29. The use of acceptable prefabricated drainage composites (also known as geosynthetic subdrain systems), as an alternative to traditionally accepted methods of draining retained earth, shall be determined during structural plan check.

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

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

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

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

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

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

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

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

38. 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).

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.

YING LIU
Geotechnical Engineer II

Log No. 135408
213-482-0480

cc: Applicant
Geotechnologies, Inc., Project Consultant
LA District Office

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

District	Log No. 135408

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: RANCHO CIENEGA O'PASO DE LA TIJERA
Block: Lots: LT 41

2. PROJECT ADDRESS: 3701-3761 STOCKER STREET, LOS ANGELES

3. OWNER: 3751-3761 STOCKER TRADITIONAL LLC
Address: 1849 GREEN BAY RD. SUITE 280
City: HIGHLAND PARK, IL Zip: 60035
Phone (Daytime):

4. APPLICANT: EFRAIN SANCHEZ, JGM INC.
Address: 6033 WEST CENTURY BLVD. SUITE 601
City: LOS ANGELES, CA Zip: 90045
Phone (Daytime): 310-946-9856
E-mail address: esanchez@jgminc.com

5. Report(s) Prepared by: Geotechnologies, Inc.

6. Report Date(s): May 23, 2024

7. Status of project: ☑ Proposed ☐ Under Construction ☐ Storm Damage

8. Previous site reports? ☐ YES if yes, give date(s) of report(s) and name of company who prepared report(s)

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

10. Applicant Signature: [signature] Position: PROJ. ARCHITECT

(DEPARTMENT USE ONLY)

REVIEW REQUESTED	FEES	REVIEW REQUESTED		FEES
☑ Soils Engineering	363 —	No. of Lots	1	21.40
☐ Geology		No. of Acres	1	
☐ Combined Soils Engr. & Geol.		☐ Division of Land		
☐ Supplemental		Other		
☐ Combined Supplemental		☑ Expedite		192.80
☐ Import-Export Route		☐ Response to Correction		
Cubic Yards:		☐ Expedite ONLY		
		Sub-total		577.20
		Surcharge		136.26
		TOTAL FEE		713.46

Fee Due: 713.46
Fee Verified By: NL Date: 5/8/24
(Cashier Use Only)

Receipt # 2090389

ACTION BY:

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

For Geology — Date

For Soils — Date

Geotechnologies, Inc.
Consulting Geotechnical Engineers
439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

May 23, 2024
File Number 22516

Stewart Manhattan, Inc.
2233 Honolulu Avenue, Suite 313
Montrose, California 91020

Attention: Simeon Stewart II

Subject: Geotechnical Engineering Investigation
Proposed Creative Studio Development
3701 – 3761 Stocker Street, Los Angeles, California

Ladies and Gentlemen:

This letter transmits the Geotechnical Engineering Investigation for the project site prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations, or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
ANDRES LOZANO
No. 92585
Exp. 9-30-25
CIVIL
STATE OF CALIFORNIA

ANDRES E. LOZANO
R.C.E. 92585

AEL:kk

Distribution: (3) Addressee

Email to: [simeon@stewartmanhattan.com]

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Calculation Sheets (6 pages)

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED CREATIVE STUDIO DEVELOPMENT
3701 – 3761 STOCKER STREET
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject property. The purpose of this investigation was to identify the distribution and engineering properties of the earth materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included excavation of six borings, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Survey Plan and Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The proposed development consists of a creative campus development consisting of six new structures. As shown in the enclosed Plot Plan, the proposed structures are described as Building A through F. Building A is proposed to be a 35,000 square foot (sf) three-story at-grade office building, with a finish floor elevation of 160.0 feet above Mean Sea Level (MSL). Building B is proposed to be a 142,200 sf six-story at-grade studio production support and offices building, with a finish floor elevation of 160.0 feet above MSL. Building C is proposed to be a 300 sf one-story at-grade security office, with a finish floor elevation of 179.5 feet above MSL. Building D is proposed to be a 32,000 sf one-story at-grade sound stage and back-of-house facility, with a finish floor

elevation of 185.0 feet above MSL. Building E is proposed to be a 40,000 sf one-story sound stages, mill shop, and office building over one subterranean level daylighting at the lower site elevation. The southwest half of Building E will have a finish floor elevation of 185.0 feet above MSL and the northeast half will have a finish floor elevation of 160.0 feet above MSL. Building F is proposed to be a 122,200 sf five-story at-grade parking structure with community room and back-of-house facility, with a finish floor elevation of 160.0 feet above MSL and a section at the northeast end of the building having a finish floor elevation of 150.5 feet above MSL. The proposed structures and finish floor elevations are illustrated on the attached Cross Sections presented in the Appendix of this report.

As part of the proposed development, the project site will consist of a lower level at the northeast half of the site, with an elevation of 160.0 feet above MSL, and an upper level at the southwest half of the site, with an elevation of 185.0 feet above MSL. Retaining walls are anticipated to achieve the proposed site finish grades. Grading will consist of removal and recompaction of existing unsuitable soils in the area of the at-grade structures, and excavations on the order of 15 feet in depth for the proposed retaining walls and foundation elements.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The property is located at 3701 – 3761 Stocker Street in the City of Los Angeles, California. The site consists of a 5-acre lot bounded by a five-story medical building, two-story commercial building, and asphalt parking lot to the northwest, by Don Felipe Drive to the southwest, by Santa Rosalia Drive to the northeast, and by Stocker Street to the southeast. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

Based upon the topographic survey prepared by Joseph C. Truxaw and Associates, Inc. dated December 9, 2023, the site grade descends gradually from the north to the west, with approximate lowest to highest elevations, ranging from 150.0 to 187.0 feet Above Mean Sea Level (AMSL). This corresponds to an approximate elevation difference of 37 feet across the site for an overall gradient of 16.4 to 1 (horizontal to vertical). The enclosed Survey Plan provides site elevations.

Drainage across the site is by sheetflow along the existing topographic contours towards the adjacent city street. Vegetation on the site consists of trees, shrubbery, and grasses within isolated planters and green spaces. The neighboring developments consist predominantly of commercial, medical and residential structures. The site is currently developed with five multi-story office buildings, four of these structures are constructed over basement levels extending 10 feet below existing grade and asphalt paved parking lots shown on the enclosed Survey Plan. The existing buildings will be demolished as part of the proposed project.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on April 1 and 2, 2024, by excavating six exploratory borings. The exploratory borings varied in depth from 20 to 50 feet. The exploration was prosecuted with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The boring locations are shown on the Survey Plan and Plot Plan, and the geologic materials encountered are logged on Plates A-1 through A-6.

The location of exploratory excavations was determined by information furnished by the client. Elevations of the exploratory excavations were determined by hand level or interpolation from data provided. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

On site geologic materials include fill and alluvium. The distribution of the geologic materials is shown on the Local Geologic Map.

Fill

Fill materials underlying the subject site predominantly consist of sandy silts and silty sands, which are dark brown, yellowish brown, and gray in color, moist, stiff, medium dense to dense, and fine grained. Fill materials were encountered in every boring at depths of 3 to 7.5 feet below the existing site grade.

Alluvium

The fill is in turn underlain by alluvial deposits. The Alluvium consists of sandy to clayey silts, silty sands, pebbly sands, cobbly sands, gravelly sands and sands, which are yellowish brown to dark brown and grayish brown in color, moist, stiff, medium dense to very dense, and fine to coarse grained with variable amounts of gravel. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was not encountered during exploration to a maximum depth of 50 feet below the ground surface. Review of the Seismic Hazard Zone Report (SHZR) for the Hollywood 7 ½-Minute Quadrangle, (CDMG, 1998, Revised 2006), indicates that the historically highest groundwater level at the site is on the order of 15 feet below ground surface.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving could not be directly observed during drilling of the deep borings because the boreholes were cased during drilling, and caving is not possible. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

Research

Available geotechnical reports for the site were reviewed during the preparation of this investigation. Specifically reviewed is the following report prepared by Geodyne Engineering, Inc. (Geodyne).

- Geotechnical Investigation, Proposed Development of Stocker Street Creative, 3701, 3731, 3741, 3751 & 3761 Stocker Street, City of Los Angeles, CA 90008, Project #GD21G185, dated February 17, 2022.

The report was prepared to provide recommendations for a similar proposed development to this current investigation. As part of the geotechnical investigation by Geodyne, seven exploratory borings were excavated to depths ranging from 31.5 to 51.5 feet below the site grade. Fill material was encountered to depths of 1.5 to 4.5 feet below the site grade. The fill is underlain by natural alluvial deposits. Groundwater was not encountered to a maximum explored depth of 51.5 feet below site grade.

The referenced report was never submitted to the City of Los Angeles Division of Building and Safety. Therefore, no recommendations were used and only the exploratory borings were utilized to determine depth of fill. The exploratory boring locations are shown on the enclosed Survey Plan and Plot Plan and the boring logs are included in the Appendix of this report for reference.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the Los Angeles Basin, which in turn, is located in the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-trending reverse faults that form the southern margin of the Transverse Ranges.

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. The basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present day landscape. Erosion of the surrounding mountains has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), Faults may be categorized as Holocene-active, Pre-

Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. As revised in 2018, The Act defines "Holocene-active" Faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 years. It is this recency of fault movement that

the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the Holocene-Active fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known Holocene-active or Pre-Holocene faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The Seismic Hazards Maps of the State of California (CDMG, 1999), does not classify the site as part of the potentially “Liquefiable” area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

Due to the geologic history of the alluvium, the dense consistency, and the depth to the current groundwater level, it is the opinion of this firm that the potential for liquefaction occurring at the site is considered to be remote.

Dynamic Dry Settlement

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures.

Some seismically-induced settlement of the proposed structures should be expected as a result of strong ground-shaking, however, due to the uniform nature of the underlying geologic materials, excessive differential settlements are not expected to occur.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. No major water-retaining structures are located immediately up gradient from the project site. Therefore, the risk of flooding from a seismically-induced seiche is considered to be remote.

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within mapped inundation boundaries due to a seiche or a breached upgradient reservoir.

Landsliding

The topography onsite as well as in the immediate vicinity of the site descends to the northeast with minor elevation changes. Review of the County of Los Angeles Landslide Inventory Map (Leighton, 1990) indicates that the site is not located within an area highly susceptible to landslides. Additionally, the Earthquake Zones of Required Investigation Map of the Hollywood Quadrangle shows that the site is not located within an Earthquake-Induced Landslide Zone (CGS, 2014). Based on these considerations, the potential of landslides negatively affecting the proposed development is considered to be low.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed development is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

Fill materials were encountered during exploration to a depth of 3 to 7.5 feet below the existing grade. The fill is underlain by dense alluvial soils comprising predominantly of a sandy strata with varying amounts of silts, clays, gavels and cobbles intermixed. Groundwater was not encountered during exploration to a maximum excavated depth of 50 feet below the existing grade.

At the time of exploration, the four existing structures to the southwest of the site are serviced by basements extending to 10 feet below grade. Once these structures have been demolished, it is recommended that a backhoe should be utilized to verify depths of fill.

The proposed construction consists of six new structures, comprised of Buildings A through F. Building A is proposed to be a 35,000 square foot (sf) three-story at-grade office building, with a finish floor elevation of 160.0 feet above Mean Sea Level (MSL). Building B is proposed to be a

142,200 sf six-story at-grade studio production support and offices building, with a finish floor elevation of 160.0 feet above MSL. Building C is proposed to be a 300 sf one-story at-grade security office, with a finish floor elevation of 179.5 feet above MSL. Building D is proposed to be a 32,000 sf one-story at-grade sound stage and back-of-house facility, with a finish floor elevation of 185.0 feet above MSL. Building E is proposed to be a 40,000 sf one-story sound stages, mill shop, and office building over one subterranean level daylighting at the lower site elevation. The southwest half of Building E will have a finish floor elevation of 185.0 feet above MSL and the northeast half will have a finish floor elevation of 160.0 feet above MSL. Building F is proposed to be a 122,200 sf five-story at-grade parking structure with community room and back-of-house facility, with a finish floor elevation of 160.0 feet above MSL and a section at the northeast end of the building having a finish floor elevation of 150.5 feet above MSL. Based on the different finish floor elevations and the locations of the proposed structures, recommendations will be provided for both structures that will require a uniform fill pad for support, and for structures that will bear in exposed native alluvial soils.

As part of the proposed development, the project site will consist of a lower level at the northeast half of the site, with an elevation of 160.0 feet above MSL, and an upper level at the southwest half of the site, with an elevation of 185.0 feet above MSL. Retaining walls are anticipated to achieve the proposed site finish grades.

The existing fill materials are not suitable for support of the proposed foundations, floor slabs or additional fill. For the structures built at or near existing grades, it is recommended that all existing fill materials should be completely removed within the building area and recompacted to create a uniform compacted fill pad within the footprint of the proposed structures. The proposed fill pad shall extend a minimum of 3 feet below the bottom of the proposed foundations. Additionally, the fill pad shall be over-excavated horizontally beyond the edge of the new foundations a minimum of 3 feet or for the distance equivalent to the thickness of the newly compacted fill placed below the foundations, whichever is greater. Conventional foundations bearing in the newly placed controlled fill are recommended for foundation support of the proposed at-grade structures.

Based on Cross Sections A-A' and B-B', Building B and the lower level of Building E are anticipated to expose native alluvial soils at the subgrade level. These structures may be supported by conventional foundations bearing in the native alluvial soils expected at the subgrade.

Excavation for the proposed site grading for creation of the upper and lower levels of the site, may require the installation of temporary shoring to provide a stable working area due to the proposed depth, nature of the onsite soils, and the proximity of adjacent structures and property lines. Shoring recommendations are provided in the "Temporary Excavations" section of this report.

Foundations for small outlying structures, such as property line walls, planters, trash enclosures, and canopies, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing in native geologic materials or newly compacted fill.

Percolations testing was performed as part of this geotechnical investigation. The field test results indicate that the site would be suitable for stormwater infiltration. Recommendations for potential stormwater infiltration systems for the proposed development are discussed later in the report.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from borings on the site as indicated and should in no way be construed to reflect any variations which may occur between these borings or which may result from changes in subsurface conditions. Any changes in the design or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

SEISMIC DESIGN CONSIDERATIONS

California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a "Stiff Soil" Profile, according to Table 20.3-1 of ASCE 7-16. This information and the site coordinates were input into the OSHPD seismic utility program (https://www.seismicmaps.org/) in order to calculate ground motion parameters for the site.

CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
California Building Code	2022
ASCE Design Standard	7-16
Risk Category	II
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.946g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.946g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.297g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.686g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.166g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.777g*

** These values are determined based on the tabulated values shown on ASCE 7-16, Table 11.4-2 for the Long-Period Site Coefficient, F_v. It should be noted that the exception to performing a site-specific hazard analysis for structures on Site Class D for values of S_1 greater than or equal to 0.2 presented by ASCE 7-16 may be followed, provided the conditions outlined in the ASCE 7-16 Section 11.4.8 (including Supplement 3 of the ASCE 7 Standards) are implemented in the structural design and analyses. This exception recommends that, where a site-specific ground motion hazard analysis is not required, the S_{M1} determined by eq. (11.4-2) is increased by 50% for all applications of S_{M1} in this standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this standard. This would require that the S_{M1} and S_{D1} values provided in the above table are increased by 50%.*

EXPANSIVE SOILS

The onsite geologic materials are in the very low to moderate expansion range. The Expansion Index was found to be 7, 10, 43, and 50 for bulk samples remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the "Foundation Design" and "Slabs On Grade" sections of this report.

WATER-SOLUBLE SULFATES

The Portland cement portion of concrete is subject to attack when exposed to water-soluble sulfates. Usually the two most common sources of exposure are from soil and marine environments.

The sources of natural sulfate minerals in soils include the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.2% percentage by weight for the soils tested. Based on the most recent revision to American Concrete Institute (ACI) Standard 318, the sulfate exposure is considered to be moderate for geologic materials between 0.1% and 0.2% and Type II cement, a maximum water/cement ratio of 0.50, and minimum strength of 4,000 psi concrete may be utilized for concrete foundations in contact with the site soils.

The design of the concrete mix is not within the area of expertise of the geotechnical engineer. It is recommended that a competent engineer familiar with concrete mix design should develop the recommendations for this project based on the tested moderate sulfate exposure indicated above.

METHANE ZONES

Based on review of the NavigateLA Website, developed by the City of Los Angeles, Bureau of Engineering, Department of Public Works, the site is not located within a City of Los Angeles Methane Zone or Methane Buffer Zone.

GRADING GUIDELINES

The following guidelines may be utilized for the site grading which will be required as part of the proposed development.

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.

- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.

- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Recommended Overexcavation

The proposed at grade building areas shall be excavated such that all fill materials are completely removed and recompacted. The excavations shall extend to a minimum depth of 3 feet below the bottom of all proposed foundations. Additionally, the excavation shall extend at least 3 feet beyond the edge of foundations or for a distance equal to the depth of newly compacted fill placed below the foundations, whichever is greater. It is very important that the positions of the proposed at grade structures are accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Fill materials having more than 15 percent finer than 0.005 millimeters may be compacted to a minimum 90 percent of the maximum density. The majority of the onsite soils will require 95 percent comparative compaction. Comparative compaction is defined, for purposes of these guidelines, as the ratio of the in-place density to the maximum density as determined by applicable ASTM testing.

All fill should be mechanically compacted in layers not more than 8 inches thick. Fill materials shall be moisture conditioned to within 3 percent of the optimum moisture content and sufficiently blended prior to placement as controlled fill. All fill shall be compacted to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. in general accordance with the most recent revision of ASTM D 1557.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Materials larger than 6 inches in maximum dimension shall not be used in the fill. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005

millimeters) of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in general accordance with the most recent revision of ASTM D 1557.

Wet Soils

At the time of exploration, the soils which will be exposed during grading were locally above optimum moisture content. It is anticipated that the excavated materials may require drying and aeration prior to recompaction.

Pumping (yielding or vertical deflection) of the high-moisture content soils at the bottom of the excavation may occur during operation of heavy equipment. Where pumping is encountered, angular minimum ¾-inch gravel and/or crushed concrete should be placed and worked into the subgrade. The exact thickness of the gravel would be a trial and error procedure, and would be determined in the field. It would likely be on the order of 1 to 2 feet thick.

The gravel will help to densify the subgrade as well as function as a stabilization material upon which heavy equipment may operate. It is not recommended that rubber tire construction equipment attempt to operate directly on the pumping subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on the soft subgrade soils will likely result in excessive disturbance to the soils, which in turn will result in a delay to the construction schedule since those disturbed soils would then have to be removed and properly recompacted. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

The simplest method to reduce the moisture content of the on-site soils would involve spreading out the soils in order to dry them naturally while the weather is warm and sunny. As an alternative, dry soils could be imported and used for one of two purposes. The existing saturated soils could be replaced by the dry soils, or the dry soils could be blended with the onsite soils in order to reduce the overall moisture content.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Abandoned Seepage Pits

No abandoned seepage pits were encountered during exploration and none are known to exist on the site. However, should such a structure be encountered during grading, options to permanently abandon seepage pits include complete removal and backfill of the excavation with compacted fill, or drilling out the loose materials and backfilling to within a few feet of grade with slurry, followed by a compacted fill cap.

If the subsurface structures are to be removed by grading, the entire structure should be demolished. The resulting void may be refilled with compacted soil. Concrete and brick generated during the seepage pit removal may be reused in the fill as long as all fragments are less than 6 inches in longest dimension and the debris comprises less than 15 percent of the fill by volume. All grading should comply with the recommendations of this report.

Where the seepage pit structure is to be left in place, the seepage pits should cleaned of all soil and debris. This may be accomplished by drilling. The pits should be filled with minimum 1-½ sack concrete slurry to within 5 feet of the bottom of the proposed foundations. In order to provide a more uniform foundation condition, the remainder of the void should be filled with controlled fill.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

FOUNDATION DESIGN

Based on the site grading to create an upper and lower levels of the project site, it is anticipated that for the structures built at or near existing grades, it is recommended that all existing fill materials should be completely removed within the building area and recompacted to create a uniform compacted fill pad within the footprint of the proposed structures.

Alternatively, for Building B and the lower level of Building E, it is anticipated that native alluvial soils will be exposed at the subgrade. These structures may be supported by conventional foundations bearing in the native alluvial soils expected at the subgrade.

Recommendations for both types of proposed structures are provided herein.

Conventional Foundation

New continuous foundations bearing in a uniform compacted fill pad or native alluvial soils may be designed for a bearing capacity of 2,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

Column foundations bearing in a uniform compacted fill pad or native alluvial soils may be designed for a bearing capacity of 3,000 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 100 pounds per square foot. The bearing capacity increase for each additional foot of depth is 450 pounds per square foot. The maximum recommended bearing capacity is 6,000 pounds per square foot.

The bearing capacities indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

Deepened Footings

Where the recommended compacted fill pad overexcavation cannot be prosecuted such as adjacent to existing buildings or property lines, and for the upper level of Building E, foundations will require deepening to bear in competent native alluvial soils. The deepened portion of the footings may be filled with concrete of the same mix as that specified for the footing. The initial pour would not require reinforcing as it is simply passing the load through to the recommended bearing material. Once the initial pour has hardened, the footing may be reinforced and poured on top of the first pour. Some method of creating a positive bond between the two pours should be employed. Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

Conventional Foundations Adjacent to Buildings or Property Lines

Conventional foundations bearing in controlled fill which are to be constructed adjacent to property lines and/or existing structures should be deepened to bear below a 1:1 plane of foundation action projected up from the toe of the newly placed controlled fill. Foundations which are to be constructed immediately adjacent to property lines and/or existing structures should be deepened to bear solely in native soils.

Where new foundations are proposed adjacent to a deep adjacent foundation, the new foundations should be deepened to match the depth of the adjacent foundation. Where foundation excavations will leave an adjacent foundation unsupported the foundation excavation should be shored as described in the "Temporary Excavations" sections of this report.

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing in properly compacted fill and/or the native alluvial soils. Wall footings may be designed for a bearing capacity of 2,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

Since the recommended bearing capacity is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Foundation Reinforcement

All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.3 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 200 pounds per cubic foot with a maximum earth pressure of 2,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 1 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed ½ inch.

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory geologic materials, if necessary. Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

RETAINING WALL DESIGN

Retaining walls up to 15 feet in height are anticipated for construction of the proposed structures and grading. As a precautionary measure, retaining wall calculations are provided for walls up to 18 feet in height. Retaining walls may be designed as indicated below, depending on whether the walls will be restrained or cantilevered. Retaining wall foundations may be designed in accordance with the provisions of the "Foundation Design" section of this report.

Additional pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures. For traffic surcharge, 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.

Restrained Retaining Walls

Restrained retaining walls up to 18 feet in height and supporting a level back slope may be designed to resist a triangular pressure distribution of at-rest earth pressure. It is recommended the walls be designed to resist the greater of the at-rest pressure, or the active pressure plus the seismic pressure, as discussed in the "Dynamic (Seismic) Earth Pressure" section below.

RESTRAINED BASEMENT WALLS		
	AT-REST EARTH PRESSURE	**ACTIVE EARTH PRESSURE *(To be Combined with Dynamic Seismic Earth Pressure)**
Height of Wall (Feet)	Triangular Distribution of Pressure (Pounds per Cubic Foot)	Triangular Distribution of Pressure (Pounds per Cubic Foot)
Up to 10	71	30*
11 to 18	71	42*

The lateral earth pressures recommended above for retaining walls assumes that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Also, where necessary, the retaining walls should be designed to accommodate any surcharge pressures that may be imposed by adjacent traffic and existing structures.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 25.9 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition. The dynamic earth pressure may be omitted where the retaining wall is 6 feet in height or less.

Cantilever Retaining Walls

Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed utilizing the following table:

HEIGHT OF WALL (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 10	30
11 to 18	42

These lateral earth pressures assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Surcharge from Adjacent Structures

As indicated herein, additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures for retaining walls and shoring design.

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2020-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

Resultant lateral force: $R = (0.3*P*h^2)/(x^2+h^2)$

Location of lateral resultant: $d = x*[(x^2/h^2+1)*\tan^{-1}(h/x)-(x/h)]$

where:

R	=	resultant lateral force measured in pounds per foot of wall width.
P	=	resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length parallel to the wall.
x	=	distance of resultant load from back face of wall measured in feet.
h	=	depth below point of application of surcharge loading to top of wall footing measured in feet.
d	=	depth of lateral resultant below point of application of surcharge loading measure in feet.
$\tan^{-1}(h/x)$	=	the angle in radians whose tangent is equal to h/x.

The structural engineer and shoring engineer may use this equation to determine the surcharge loads based on the loading of the adjacent structures located within the surcharge influence zone.

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, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

Waterproofing is recommended for retaining walls. 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.

Retaining Wall Drainage

All retaining walls shall be provided with a subdrain system in order to minimize the potential for future hydrostatic pressure buildup behind the proposed retaining walls. Subdrains may consist of 4-inch diameter perforated pipes, places with perforated facing down. The pipe shall be encased in at least one foot of gravel around the pipe. The gravel shall be wrapped in filter fabric. The gravel may consist of three-quarter inch to one-inch crushed rock.

As an alternative to the standard perforated subdrain pipe and gravel drainage system, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to one inch crushed rock, wrapped in filter fabric. A collector pipe is placed within the grave which directs collected waters to a sump or standard pipe and grave system constructed under the slab. This method should be approved by the retaining wall designer prior to implementation.

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. If a drainage system is not provided, the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure. In any event, it is recommended that retaining walls be waterproofed.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered during exploration to a depth of 50 feet below existing grade. Therefore, the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density in general accordance with the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

TEMPORARY EXCAVATIONS

Excavations on the order of 5 to 20 feet in vertical height will be required for recommended removal and recompaction of the existing fill for placement of the uniform compacted fil pad, and for the retaining walls, including foundation elements for the site grading. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Vertical excavations exceeding 5 feet, or excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 (h:v) slope gradient to a maximum depth of 20 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if

variations in the geologic material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SHORING DESIGN

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that Geotechnologies, Inc. review the final shoring plans and specifications prior to bidding or negotiating with a shoring contractor.

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tied-back anchors or raker braces.

Soldier Piles – Drilled and Poured

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation, may be assumed to be 500 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed geologic materials.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldiers piled below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Casing may be required should caving be experienced in the granular (saturated) geologic materials. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Additionally, it should be noted that gravels and cobbles were encountered in the underlying soils during exploration. Drilling will be difficult due to the obstructions caused by the gravel and large diameter cobbles, the contractor should be prepared to drill alternate or even every other pile.

Lagging

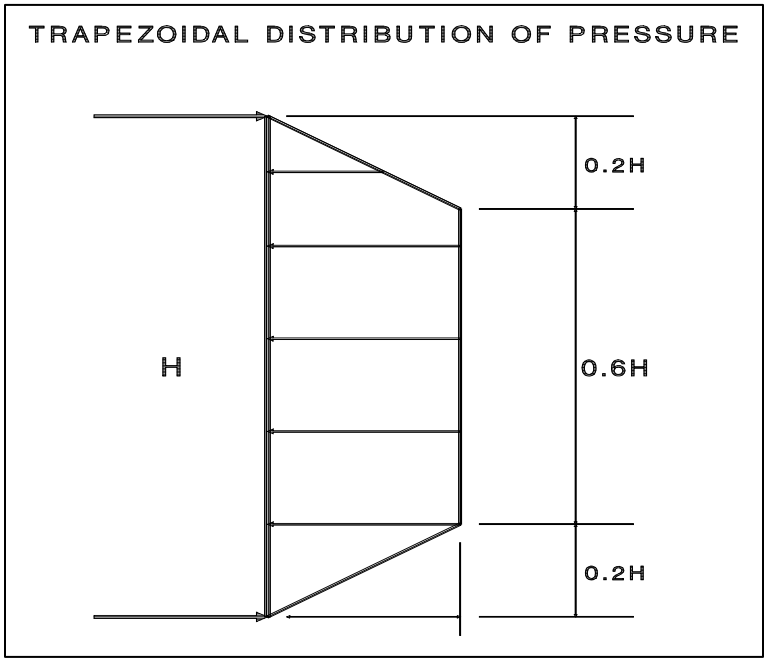
Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but may be limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to ensure uniform support of the excavated embankment.

Lateral Pressures

Cantilevered shoring supporting a level backslope may be designed utilizing a triangular distribution of pressure as indicated in the following table:

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 10	28
11 to 20	35

A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs, with the trapezoidal distribution as shown in the diagram below.



Restrained shoring supporting a level backslope may be designed utilizing a trapezoidal distribution of pressure as indicated in the following table:

HEIGHT OF SHORING "H" (feet)	DESIGN SHORING FOR (Where H is the height of the wall)
Up to 10	18H
11 to 20	22H

Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be applied where the shoring will be surcharged by adjacent traffic or structures.

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge. Anchors should be placed at least 6 feet on center to be considered isolated.

Drilled friction anchors may be designed for a skin friction of 400 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,000 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchor Installation

Tied-back anchors may be installed between 20 and 45 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be

filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge 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.

Tieback Anchor Testing

At least 10 percent of the anchors should be selected for "Quick", 200 percent tests. It is recommended that at least 3 of these anchors be selected for 24-hour, 200 percent tests. It is recommended that the 24-hour tests be performed prior to installation of additional tiebacks. 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. Where satisfactory tests are not achieved on these 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 tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent load has been applied.

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 load has been applied should not exceed 0.25 inches during the 30-minute period.

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

After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. Where post-grouted anchors are utilized, additional post-grouting may be required. The installation and testing of the anchors should be observed by the geotechnical engineer.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. The City of Los Angeles Department of Building and Safety requires limiting shoring deflection to ½-inch at the top of the shored embankment where a structure is within a 1:1 plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed provided there are no structures within a 1:1 plane drawn upward from the base of the excavation. 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.

Raker Brace Foundations

An allowable bearing pressure of 4,000 pounds per square foot may be used for the design a raker foundations bearing in native alluvial soils. This bearing pressure is based on a raker foundation a minimum of 4 feet in width and length as well as 4 feet in depth. The base of the raker foundations

should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

Monitoring

Because of the depth of the excavation, some means 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.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of Geotechnologies, Inc. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations ensure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the geologic material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness, and should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Slabs-on-grade should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent

(or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness, and should be reinforced with a minimum of #3 steel bar on 24-inch centers each way. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, where necessary, it is recommended that a qualified consultant should be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor on various components of the structure.

Where any dampness would be objectionable, it is recommended that floor slabs should be waterproofed. A qualified waterproofing consultant should be engaged in order to recommend a product and/or method which would provide protection from unwanted moisture.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimmable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness (inches)	Base Course (inches)
Passenger Cars	3	4
Moderate Truck	4	6
Heavy Truck	5	9

Concrete paving may be utilized for the project. For concrete paving, the following sections are recommended:

Service	Concrete Pavement Thickness (inches)	Base Course (inches)
Passenger Cars	6	4
Moderate Truck	6	4
Heavy Truck	7½	4

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform to Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), latest edition.

For standard crack control maximum expansion joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete paving should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage, with the exception of any required to disposed of onsite by stormwater regulations, should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

Introduction

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

Percolation Testing

In order to establish a percolation rate for the site soils, Borings B1 and B4 were utilized for percolation test. The Borings B1 and B4 were drilled to depths of 50 and 20 feet below the existing site grade, respectively. The borings were drilled with the aid of a truck-mounted drilling machine, equipped with 8-inch diameter hollowstem augers. Fill materials were encountered in all six borings, to depths ranging from 3 to 7.5 feet below the existing site grade. The fill is in turn underlain by native alluvial soils, consisting predominantly of silty to gravely sands and sands below a depth of 10 feet from ground surface. The geologic materials encountered during excavation are presented on the enclosed Plates A-1 and A-4.

At the completion of drilling, a 2-inch diameter casing was placed within the center of the boreholes for the purpose of conducting percolation testing. The casing consisted of solid PVC pipes from the ground surface to a depth of 30 feet within Boring B1 and 10 feet within Boring B4, and perforated PVC pipes between depths of 30 and 50 feet within Boring B1 and 10 and 20 feet within Boring B4. A sand pack consisting of #3 Monterey Sand was poured into the annular

space around the perforated portion of the casing. A 1-foot thick hydrated bentonite seal was placed over the sand.

After the casing was installed, the boreholes were filled with water for the purpose of pre-soaking for a minimum of 2 hours. After presoaking, the borehole was refilled with water, and the rate of drop in the water level was measured. The percolation test readings were recorded a minimum of 8 times or until a stabilized rate of drop was obtained, whichever occurred first. The percolation testing was performed within the native alluvial soils.

Based on results of the percolation testing, a percolation rate of 300 inches per hour for Boring B1 and 139 inches per hour for Boring B4 may be utilized for design of the proposed deep infiltration dry well systems. Using a safety factor of 3 (as required by the City of Los Angeles), infiltration rates of 100 and 46 inches per hour may be utilized for design of the infiltration system. An applicable infiltration rate of 46 inches per hour may be utilized up to depth of 30 feet and 100 inches per hour may be utilized below a depth of 30 feet.

The Proposed System

The locations for potential stormwater disposal have not been specifically addressed on this site. It is the opinion of this firm that a “dry well” system is possible on this site, however this office should review the plan once it achieves more definition.

The proposed stormwater infiltration system shall only percolate in the underlying native soils, and will be designed to infiltrate at a minimum depth of 15 feet below the lowest level of the adjacent structure. In addition, the proposed infiltration system shall be located a minimum horizontal setback distance of 15 feet from property lines and/or public right of ways. Stormwater infiltration is not allowed within 10 feet (vertically) from the groundwater level. Groundwater was not encountered during exploration conducted to a depth of 50 feet below existing site grade.

Therefore, it is recommended that the bottom of the proposed infiltration system should not exceed 40 feet below existing site grade.

The proposed infiltration systems should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area, or disposed offsite in an acceptable manner.

The natural alluvial soil encountered below a depth of 15 feet beneath the existing site grades consists primarily of granular soils, and should allow stormwater to percolate in a generally vertical manner. Therefore, there is no potential for creating a perched water condition. The onsite granular soils are very low to low in expansion potential, and are not susceptible to significant hydroconsolidation. In addition, due to the dense nature of the underlying soils, stormwater infiltration should not cause any damage or settlement to any building.

Due to the dense consistency of the underlying natural alluvial soils, liquefaction potential for the site was remote. It is the opinion of this firm that any proposed infiltration of stormwater will not materially impact the liquefaction potential of the site.

Excavation of the proposed stormwater infiltration system may require casing should caving be experience in the granular geologic materials. If casings is used, extreme care should be employed so that the hole is not pulled apart as the casing is withdrawn. Additionally, it should be noted that gravels and cobbles were encountered in the underlying soils during exploration. Drilling will be difficult due to the obstructions caused by the gravel and large diameter cobbles.

It is recommended that the design team, including the structural engineer, waterproofing consultant, plumbing engineer, environmental engineer and landscape architect be consulted in regard to the design and construction of filtration systems.

Recommendations

The design and construction of stormwater infiltration facilities is not the responsibility of the geotechnical engineer. However, based on the experience of this firm, it is recommended that several aspects of the use of such facilities should be considered by the design and construction team:

- All infiltration devices should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area, or disposed offsite in an acceptable manner.

- All connections associated with stormwater infiltration devices should be sealed and water-tight. Water leaking into the subgrade soils can lead to loss of strength, piping, erosion, settlement and/or expansion of the affected earth materials.

- Excavations proposed for the installation of stormwater facilities should comply with the "Temporary Excavations" sections of this (the referenced) reports well as CalOSHA Regulations where applicable.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in

depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly bedrock can contain concretions. Concretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the geologic conditions do not deviate from those disclosed in the investigation. If any variations are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geotechnologies, Inc. should be notified so that supplemental recommendations can be prepared.

This report is issued with the understanding that it is the responsibility of the owner, or the owner's representatives, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineer and are incorporated into the plans. The owner is also responsible to see that the contractor and subcontractors carry out the geotechnical recommendations during construction.

The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Geotechnical observations and testing during construction is considered to be a continuation of the geotechnical investigation. It is, therefore, most prudent to employ the consultant performing the initial investigative work to provide observation and testing services during construction. This practice enables the project to flow smoothly from the planning stages through to completion.

Should another geotechnical firm be selected to provide the testing and observation services during construction, that firm should prepare a letter indicating their assumption of the responsibilities of geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for review. The letter should acknowledge the concurrence of the new geotechnical engineer with the recommendations presented in this report.

EXCLUSIONS

Geotechnologies, Inc. does not practice in the fields of methane gas, radon gas, environmental engineering, waterproofing, dewatering organic substances or the presence of corrosive soils or wetlands which could affect the proposed development including mold and toxic mold. Nothing in this report is intended to address these issues and/or their potential effect on the proposed development. A competent professional consultant should be retained in order to address environmental issues, waterproofing, organic substances and wetlands which might affect the proposed development.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the excavation logs.

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound hammer. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in general accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples in general accordance with the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the “Excavation Logs”, A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed in general accordance with the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

The most recent revision of ASTM 3080 limits the particle size to 10 percent of the diameter of the direct shear test specimen. The sheared sample is inspected by the laboratory technician running the test. The inspection is performed by splitting the sample along the sheared plane and observing the soils exposed on both sides. Where oversize particles are observed in the shear plane, the results are discarded and the test run again with a fresh sample.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests in general accordance with the most recent revision of ASTM D 2435. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D 4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined in general accordance with the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve.

REFERENCES

California Department of Conservation, Division of Mines and Geology, 1998 (Revised 2006), Seismic Hazard Zone Report for the Hollywood 7 ½-Minute Quadrangle, Los Angeles County, California, C.D.M.G. Seismic Hazard Zone Report 026, map scale 1:24,000.

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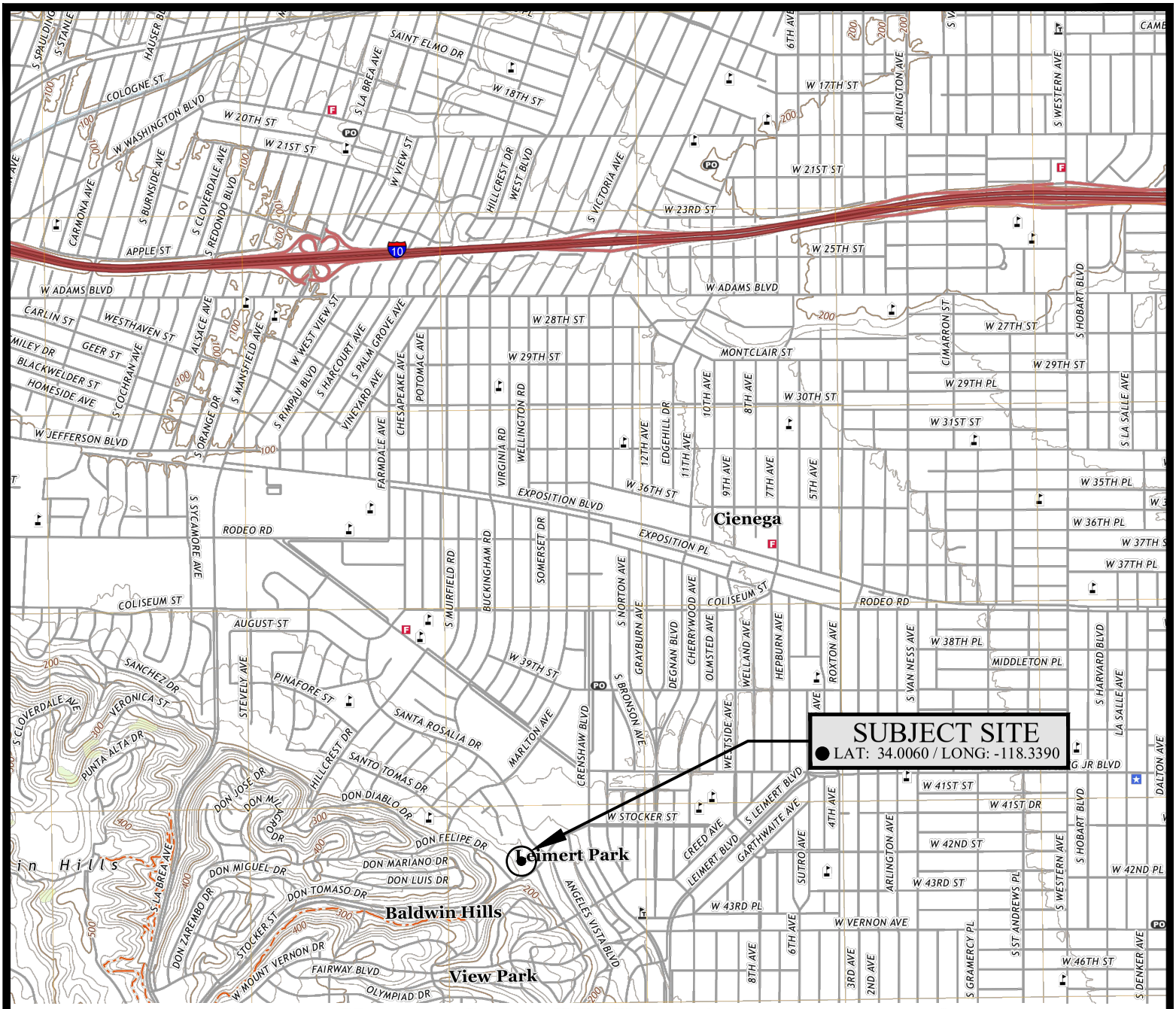
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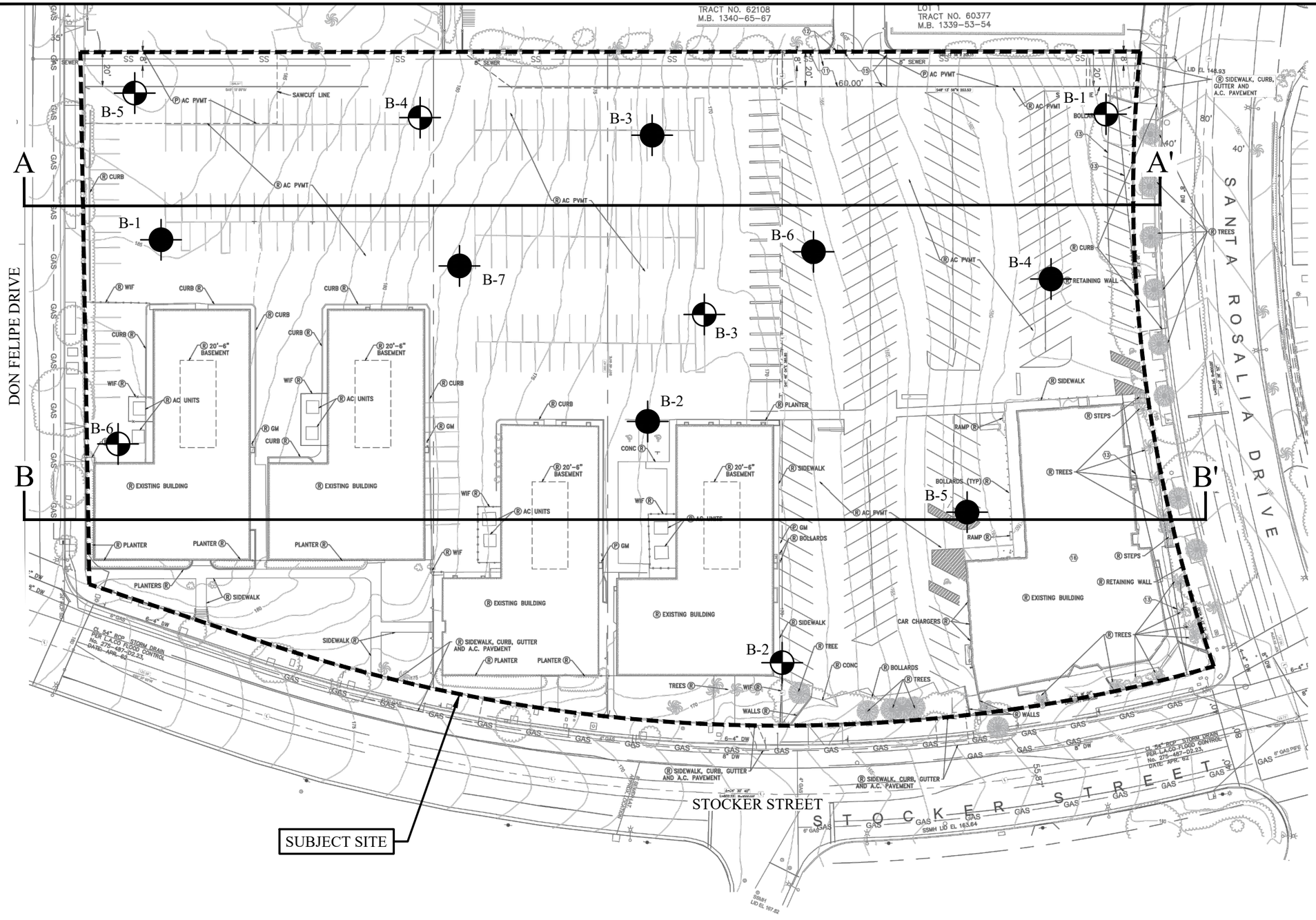
VICINITY MAP

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STEWART MANHATTAN, INC.

FILE NO: 22516

Plot Date: May. 16, 2024
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LEGEND

B-6 — LOCATION AND NUMBER OF BORING (THIS INVESTIGATION)

B-7 — LOCATION AND NUMBER OF BORING (PREVIOUS INVESTIGATION BY GEODYNE ENGINEERING INC. DATED FEBRUARY 17, 2022)

REFERENCE: CONCEPTUAL GRADING PLAN BY JOSEPH C. TRUXAW AND ASSOCIATES INC. DATED 12/08/2023

SURVEY PLAN

Geotechnologies, Inc.
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STEWART MANHATTAN, INC.

Drawn by: JD | File No.: 22516

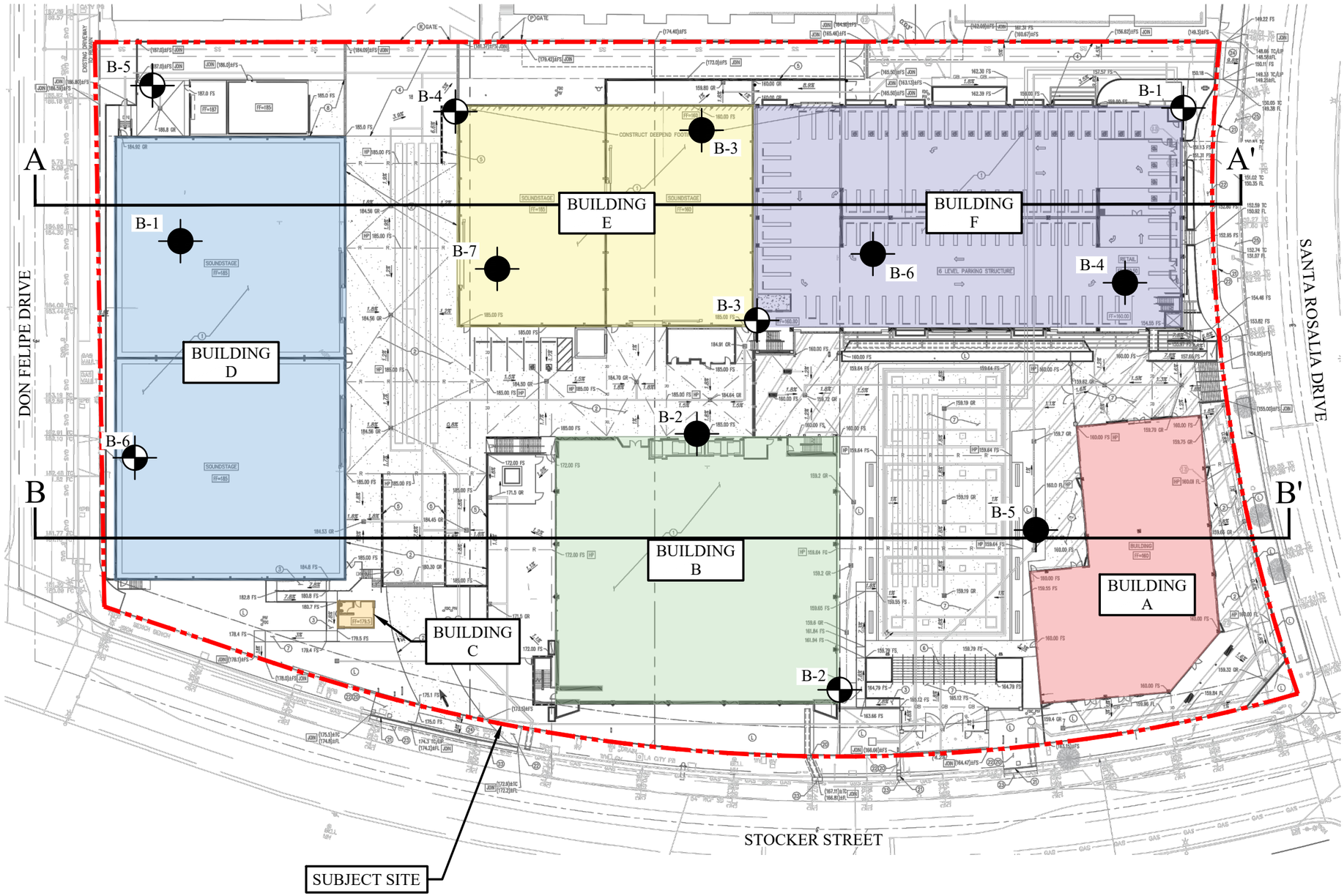
Date: May 2024

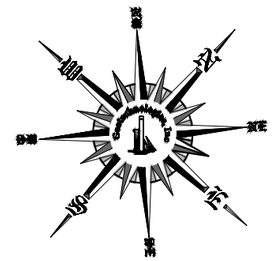
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SCALE IN FEET

0 20 40 60 120

LEGEND

B-6 — LOCATION AND NUMBER OF BORING (THIS INVESTIGATION)

B-7 — LOCATION AND NUMBER OF BORING (PREVIOUS INVESTIGATION BY GEODYNE ENGINEERING INC. DATED FEBRUARY 17, 2022)

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PLOT PLAN

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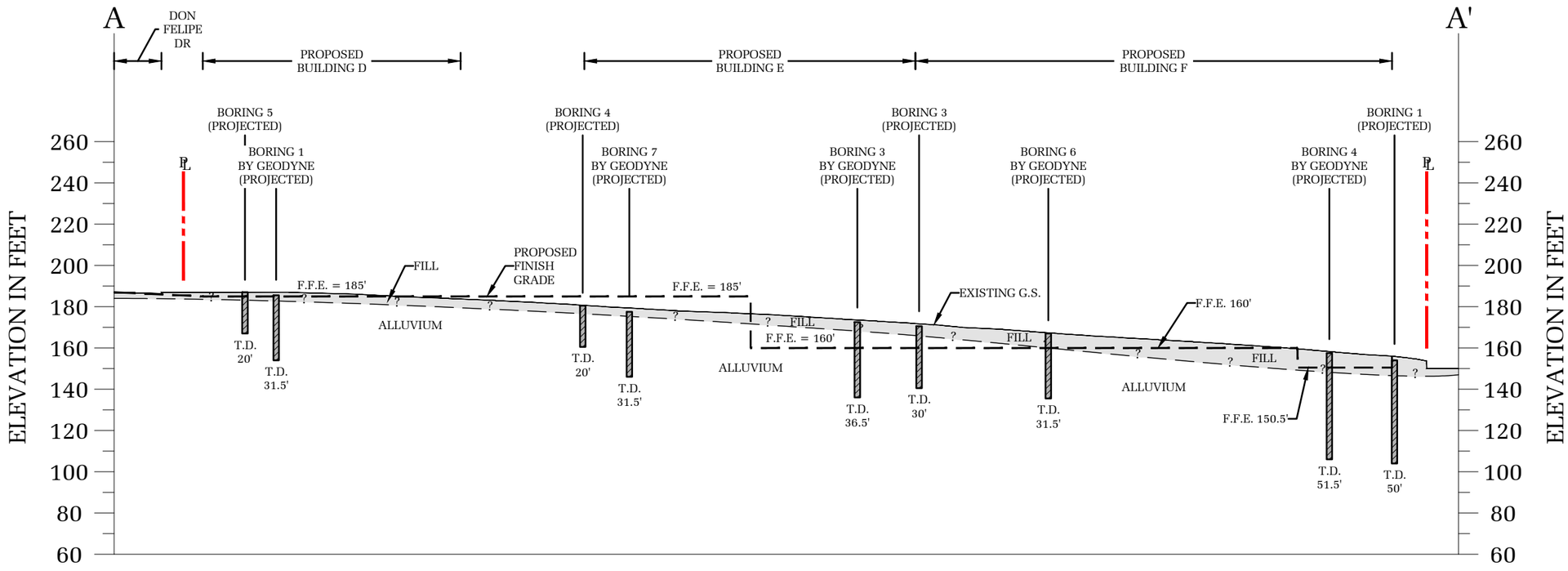
STEWART MANHATTAN, INC.

Drawn by: JD | File No.: 22516

Date: May 2024

Plot Date: May. 16, 2024
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N42W



SCALE IN FEET

0 20 40 60 120

REFERENCE: CONCEPTUAL GRADING PLAN (SHEETS C-2 & C-3) BY JOSEPH C. TRUXAW AND ASSOCIATES INC. DATED 12/08/2023

CROSS SECTION A-A'

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Drawn by: JD | File No.: 22516

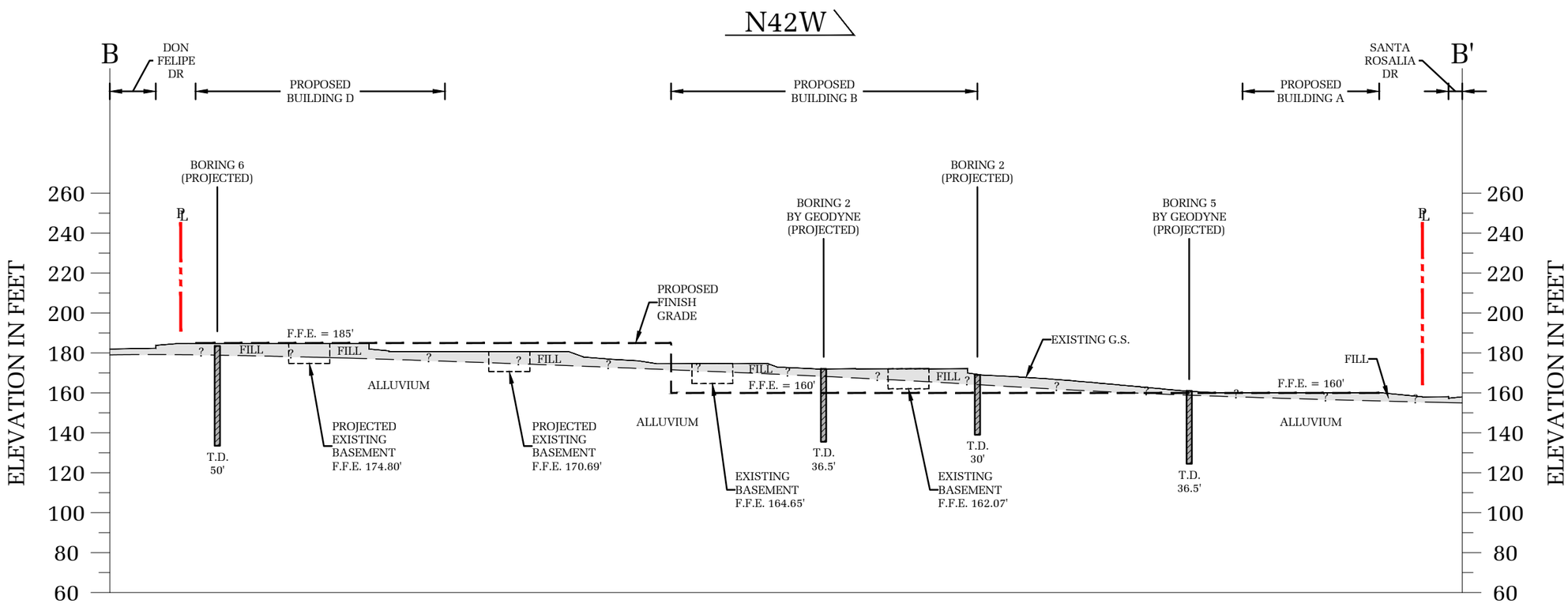
Date: May 2024

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Plot Date: May. 16, 2024

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SCALE IN FEET

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REFERENCE: CONCEPTUAL GRADING PLAN (SHEETS C-2 & C-3) BY JOSEPH C. TRUXAW AND ASSOCIATES INC. DATED 12/08/2023

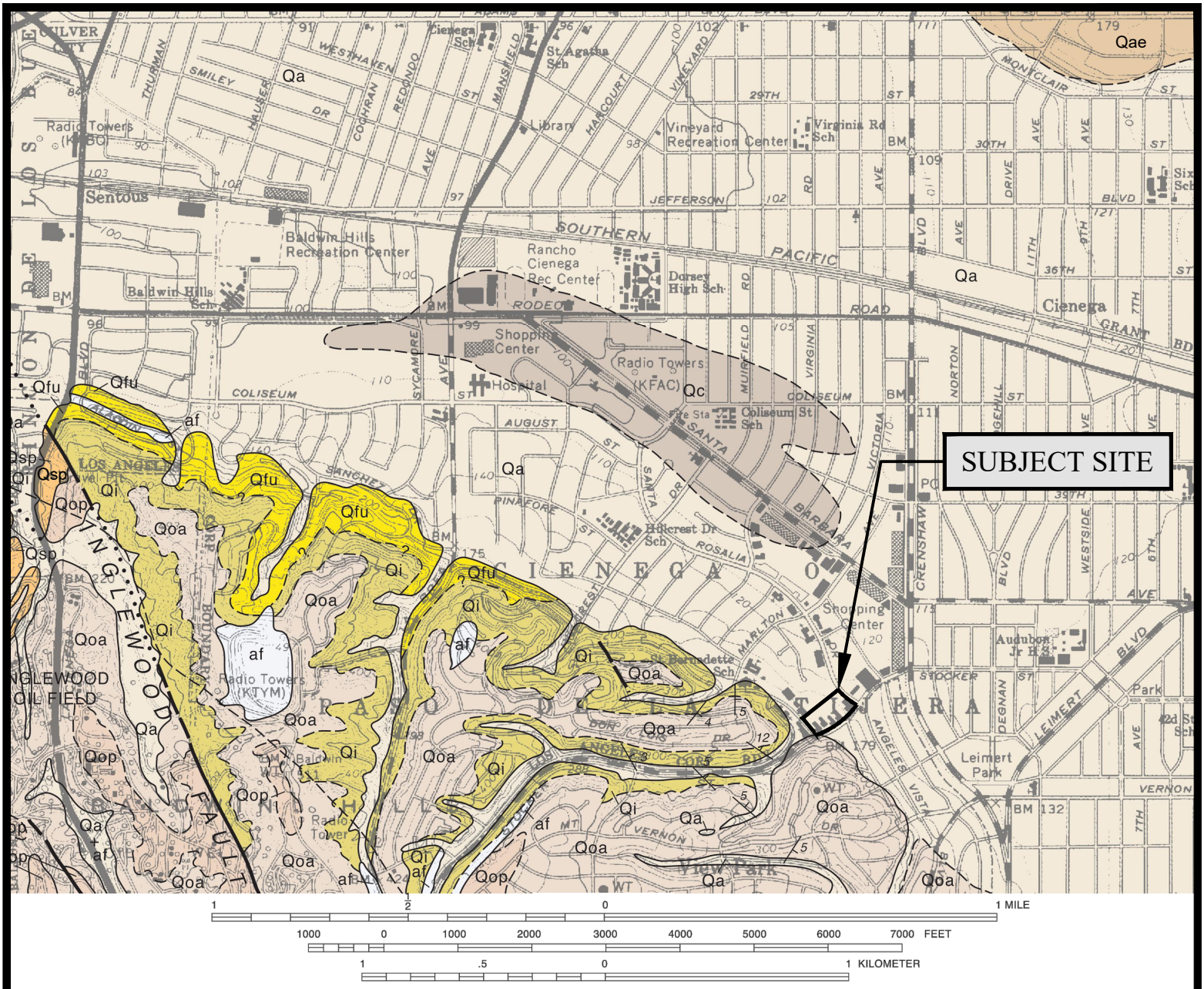
CROSS SECTION B-B'

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STEWART MANHATTAN, INC.

Drawn by: JD | File No.: 22516

Date: May 2024



LEGEND

af: Surficial Sediments - Artificial fill

Qc: Clay and sand of pre-development marshlands

Qa: Alluvium: clay, sand and gravel; includes gravel and sand of minor stream channels

Qae: Older Surficial Sediments - Similar to Qa, but slightly elevated and dissected; includes alluvial fan sediments

Qoa: Older alluvium, fray to light brown pebble-gravel, sand, silt and slay of detritus derived from Santa Monica Mountains; similar to Qae, but slightly consolidated; in Baldwin Hills designated as Baldwin Hills sandy gravel, where it is much dissected and eroded

Qsp: Shallow Marine Sediments - San Pedro Sand: light gray to light brown sand, fine to coarse grained, pebbly; locally contains shell fragments

Qi: Inglewood Formation: light gray, friable; fine grained sandstone and interbedded soft gray siltstone

Qfu: Upper Fernando (?) Formation: soft gray massive silty claystone, base not exposed

Folds - arrow on axial trace of fold indicates direction of plunge

Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

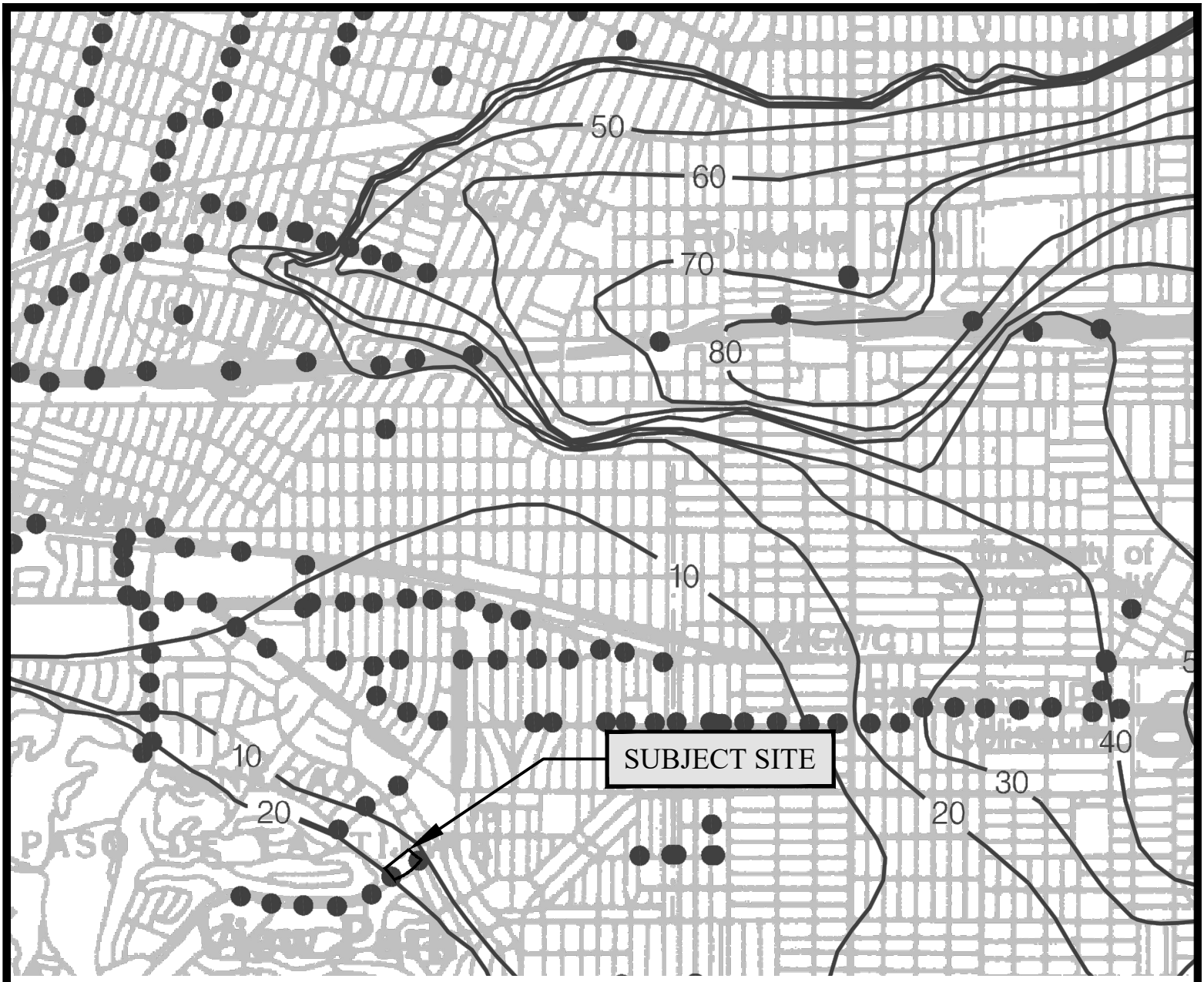
REFERENCE: T.W. DIBBLEE (EDITED 2010) GEOLOGIC MAP OF THE HOLLYWOOD & SOUTH HALF BURBANK QUADRANGLES (#DF-30)

GEOLOGIC MAP (DIBBLEE)

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STEWART MANHATTAN, INC.

FILE NO: 22516



ONE MILE

SCALE

N

LEGEND

- Borehole Site
- 30 — Depth to ground water in feet

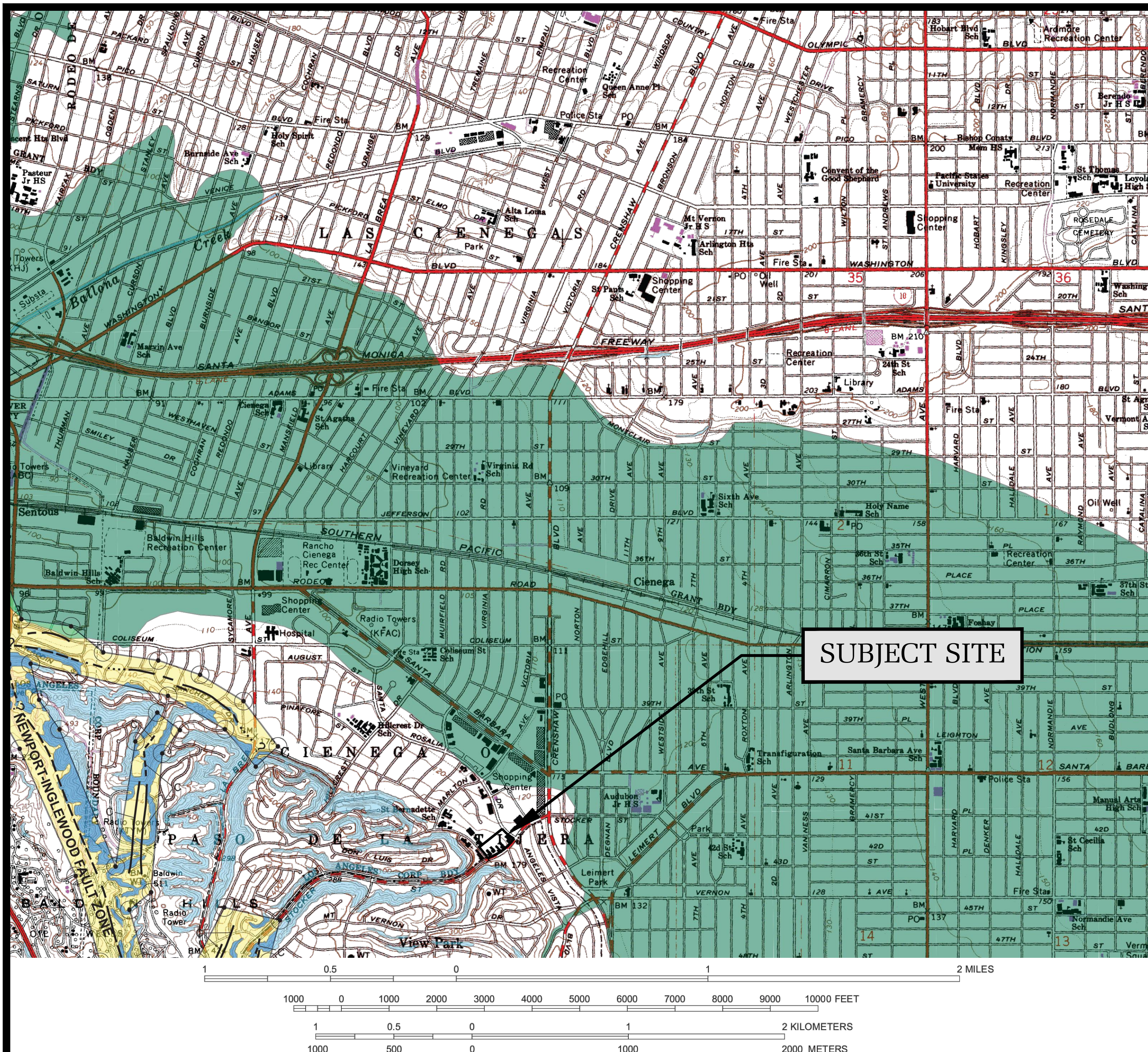
REFERENCE: SEISMIC HAZARD ZONE REPORT, 026 HOLLYWOOD, 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA 1998, (REV 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS MAP

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STEWART MANHATTAN, INC.

FILE NO: 22516



LEGEND

LIQUEFACTION ZONES

EARTHQUAKE-INDUCED LANDSLIDE ZONES

ACTIVE FAULT TRACES

EARTHQUAKE FAULT ZONES

REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, HOLLYWOOD QUADRANGLE (CGS, 2014)

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

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Consulting Geotechnical Engineers

STEWART MANHATTAN, INC.

FILE NO: 22516

BORING LOG NUMBER 1

Stewart Manhattan Inc. **Date: 04/01/24** **Elevation: 154.0'***

File No. 22516 **Method: 8-inch diameter Hollow Stem Auger**

kk

*Based on Survey prepared by Joseph C. Truxaw and Associates, Inc. dated 12/8/2023

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		5" Asphalt over 3" Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	18	11.8	121.6	-		
				3 --		
				-		
				4 --		
				-		
5	10	12.2	SPT	5 --		
				-		Silty Sand, dark and yellowish brown, moist, medium dense,
				6 --		fine grained
				-		
				7 --		
7.5	20	17.7	112.7	-		
				8 --	SM/SW	ALLUVIUM: Silty Sand to Pebbly Sand, yellowish brown, moist,
				-		medium dense, fine to coarse grained
				9 --		
				-		
10	11	5.9	SPT	10 --		
				-	SM/SP	Silty Sand to Sand, yellowish brown, moist, medium dense, fine
				11 --		to medium grained
				-		
				12 --		
12.5	34	6.4	115.2	-		
				13 --		
				-		
				14 --		
				-		
15	19	9.6	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	52	4.2	123.7	-		
				18 --		
				-		
				19 --		
				-		
20	24	14.9	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	52	6.3	115.9	-		
				23 --		Silty Sand to Sand, dark and yellowish brown, moist, medium
				-		dense, fine to medium grained
				24 --		
				-		
25	22	5.7	SPT	25 --		
				-		

BORING LOG NUMBER 1

Stewart Manhattan Inc.

File No. 22516

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	47	7.0	107.9	28 --		
				29 --		
30	27	12.4	SPT	30 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test
				31 --		
				32 --		
32.5	78	4.4	115.6	33 --		
				34 --		
35	33	10.2	SPT	35 --	SM	Silty Sand, grayish brown, moist, medium dense, fine grained
				36 --		
				37 --		
37.5	32 50/3"	8.2	109.6	38 --		
				39 --		
40	32 50/5"	2.3	SPT	40 --	SP	Sand, yellowish brown, moist, very dense, fine to medium grained
				41 --		
				42 --		
42.5	100/8"	2.1	120.3	43 --	SP/SW	Sand to Gravelly Sand, yellowish brown, moist, very dense, fine to coarse grained
				44 --		
45	50/6"	1.9	SPT	45 --		
				46 --		
				47 --		
47.5	100/8"	4.8	118.6	48 --	SM/SW	Silty Sand to Cobbly Sand, yellow and grayish brown, moist, very dense, fine to coarse grained
				49 --	SM/SP	Silty Sand to Sand, dark and grayish brown, moist, medium dense to dense
50	45	11.0	SPT	50 --		Total Depth 50 feet No Water Fill to 7.5 feet

BORING LOG NUMBER 2

Stewart Manhattan Inc. **Date: 04/01/24** **Elevation: 169.0'***

File No. 22516 **Method: 8-inch diameter Hollow Stem Auger**

kk

*Based on Survey prepared by Joseph C. Truxaw and Associates, Inc. dated 12/8/2023

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4" Asphalt over 3" Base
				-		
				1 --		FILL: Silty Sand, dark brown and gray, moist, medium dense, fine grained
				-		
				2 --		
2.5	8	8.5	110.8	-		
				3 --		
				-		
				4 --		
				-		
5	26	10.4	121.9	5 --		
				-	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	55	12.1	118.5	10 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, stiff, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	54	6.7	113.8	15 --		
				-	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	68	2.2	109.7	20 --		
				-	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, dense, fine to medium grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	71	3.3	114.8	25 --		
				-		

BORING LOG NUMBER 2

Stewart Manhattan Inc.

File No. 22516

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	100/8"	2.3	Disturbed	30 -- - 31 -- - 32 -- - 33 -- - 34 -- - 35 -- - 36 -- - 37 -- - 38 -- - 39 -- - 40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -	SW	Cobbly Sand, grayish brown, moist, very dense, fine to coarse grained Total Depth 30 feet No Water Fill to 5 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test

BORING LOG NUMBER 3

Stewart Manhattan Inc. **Date: 04/01/24** **Elevation: 170.5'***

File No. 22516 **Method: 8-inch diameter Hollow Stem Auger**

kk

*Based on Survey prepared by Joseph C. Truxaw and Associates, Inc. dated 12/8/2023

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		6" Asphalt over 3" Base
				1 --		
				2 --		FILL: Silty Sand, dark brown and gray, moist, medium dense, fine grained
2.5	29	13.3	120.2	3 --		
				4 --		
				5 --	SM	ALLUVUIM: Silty Sand, dark brown, moist, medium dense, fine grained
5	41	17.4	113.4	6 --	ML	Sandy Silt, dark brown, moist, stiff
				7 --		
				8 --		
				9 --		
10	50	15.0	120.6	10 --	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, dense, fine grained
				11 --		
				12 --		
				13 --		
				14 --		
15	27	9.7	109.4	15 --		
				16 --		
				17 --		
				18 --		
				19 --		
20	76	12.3	113.6	20 --	SM	Silty Sand, dark brown, moist, dense, fine grained
				21 --		
				22 --		
				23 --		
				24 --		
25	88	5.0	110.7	25 --	SP/SW	Sand to Gravelly Sand, dark and yellowish brown, moist, dense, fine to coarse grained

BORING LOG NUMBER 3

Stewart Manhattan Inc.

File No. 22516

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	49	31.1	92.3	30 -- - 31 -- - 32 -- - 33 -- - 34 -- - 35 -- - 36 -- - 37 -- - 38 -- - 39 -- - 40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -	SM	Silty Sand, dark and grayish brown, moist, medium dense, fine grained Total Depth 30 feet No Water Fill to 4 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 4

Stewart Manhattan Inc. **Date: 04/02/24** **Elevation: 180.5'***

File No. 22516 **Method: 8-inch diameter Hollow Stem Auger**

kk

*Based on Survey prepared by Joseph C. Truxaw and Associates, Inc. dated 12/8/2023

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		5" Asphalt, over 2" Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	35	17.5	114.6	-		
				3 --		
				-	ML	ALLUVIUM: Sandy to Clayey Silt, dark brown, moist, stiff
				4 --		
				-		
5	28	17.5	111.9	5 --		
				-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	56	5.1	120.1	10 --		
				-	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, medium
				11 --		dense, fine grained
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	32	8.2	104.8	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained
20	37	13.9	108.5	20 --		
				-		Total Depth 20 feet
				21 --		No Water
				-		Fill to 3 feet
				22 --		
				-		NOTE: The stratification lines represent the approximate
				23 --		boundary between earth types; the transition may be gradual.
				-		
				24 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				25 --		Modified California Sampler used unless otherwise noted
				-		

BORING LOG NUMBER 5

Stewart Manhattan Inc. **Date: 04/02/24** **Elevation: 187.0'***

File No. 22516 **Method: 8-inch diameter Hollow Stem Auger**

kk

*Based on Survey prepared by Joseph C. Truxaw and Associates, Inc. dated 12/8/2023

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4" Asphalt, over 2" Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense, stiff, fine grained
				-		
				2 --		
2.5	45	13.3	120.1	-		
				3 --		
				-	SM/ML	ALLUVIUM: Silty Sand to Sandy Silt, dark brown, moist, medium dense, stiff, fine grained
				4 --		
				-		
5	60	8.9	127.5	5 --	SM/SP	Silty Sand to Sand, dark brown, moist, dense, fine grained
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	35	9.7	113.1	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	42	6.5	113.3	15 --		
				-	SP	Sand, yellowish brown, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	36	22.2	101.4	20 --		
				-		Total Depth 20 feet No Water Fill to 3 feet
				21 --		
				-		
				22 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				23 --		
				-		
				24 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				-		
				25 --		
				-		

BORING LOG NUMBER 6

Stewart Manhattan Inc. **Date: 04/02/24** **Elevation: 183.5'***

File No. 22516 **Method: 8-inch diameter Hollow Stem Auger**

kk *Based on Survey prepared by Joseph C. Truxaw and Associates, Inc. dated 12/8/2023

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0		FILL: Silty Sand, dark brown, moist, dense, fine grained
				1		
				2		
2.5	23	11.5	122.0			
				3	SM	ALLUVIUM: Silty Sand, dark and yellowish brown, moist, medium dense, fine grained
				4		
5	12	11.7	SPT	5		
				6		
				7		
7.5	56	22.1	104.9			
				8		
				9		
10	21	15.8	SPT	10	SM/SP	Silty Sand to Sand, yellowish brown, moist, medium dense, fine grained
				11		
				12		
12.5	38	5.6	113.7			
				13	SP	Sand, yellowish brown, moist, medium dense, fine grained
				14		
15	29	11.1	SPT	15	SM/SP	Silty Sand to Sand, yellowish brown, moist, medium dense, fine grained
				16		
				17		
17.5	70	4.2	118.6			
				18		
				19		
20	22	11.8	SPT	20		
				21		
				22		
22.5	66	9.4	113.6			
				23	SP	Sand, yellowish brown, moist, dense, fine grained
				24		
25	50	5.4	SPT	25		Sand, yellowish brown, moist, dense, fine grained, minor gravel

NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.

Used 8-inch diameter Hollow-Stem Auger
140-lb. Automatic Hammer, 30-inch drop
Modified California Sampler used unless otherwise noted

SPT=Standard Penetration Test

BORING LOG NUMBER 6

Stewart Manhattan Inc.

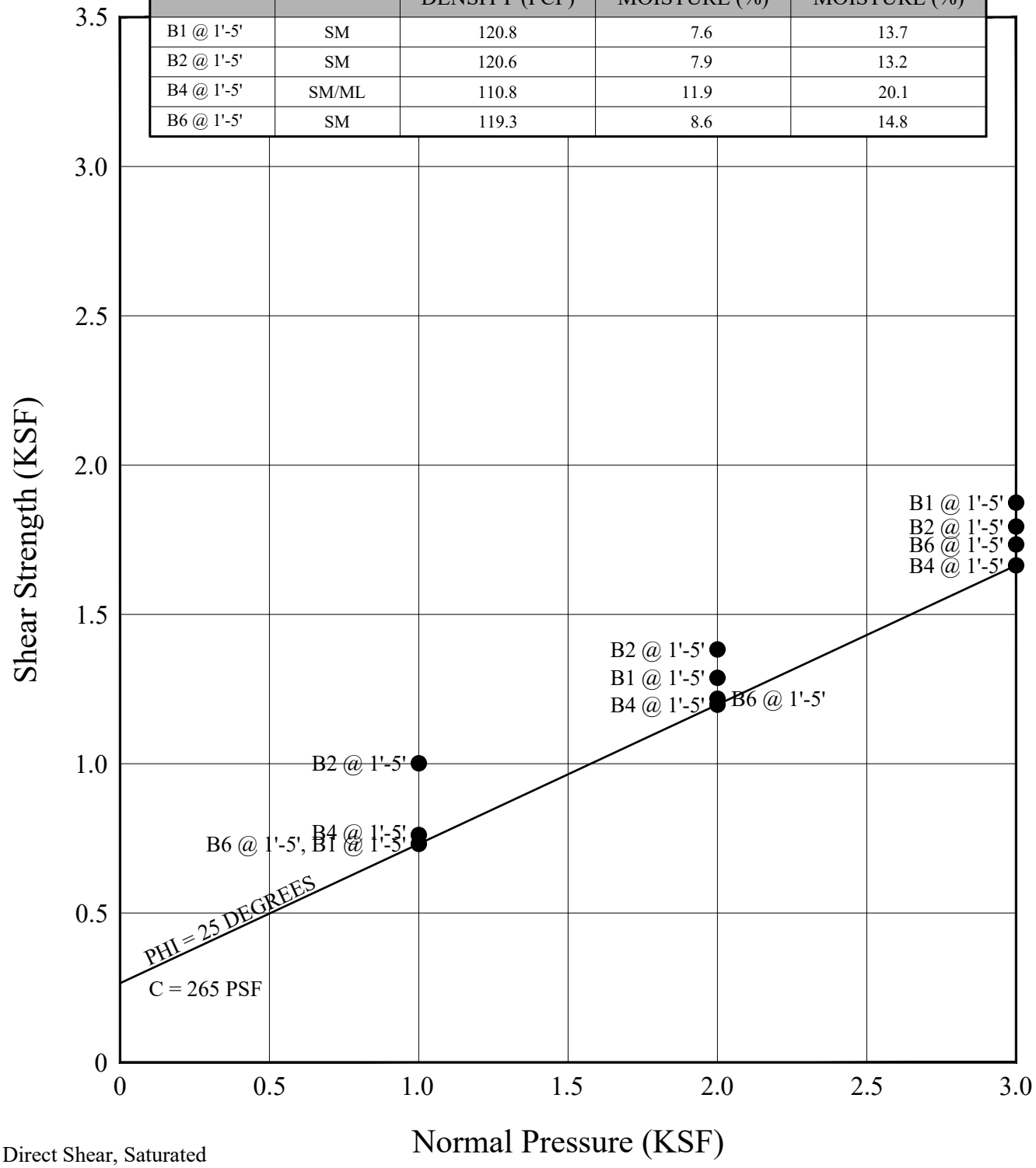
File No. 22516

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	42	7.9	105.3	28 --		Sand, yellowish brown, moist, medium dense, fine grained
				29 --		
30	58	7.8	SPT	30 --		Sand, dark and yellowish brown, moist, dense, fine to coarse grained
				31 --		
				32 --		
32.5	100/7"	1.7	116.2	33 --	SP/SW	Sand to Gravelly Sand, dark and yellowish brown, moist, very dense, fine to coarse grained
				34 --		
35	90	1.8	SPT	35 --	SP	Sand, dark and yellowish brown
				36 --		
				37 --		
37.5	40 50/4"	14.4	94.4	38 --	SM	Silty Sand, grayish brown, moist, very dense, fine grained
				39 --		
40	70	15.3	SPT	40 --		
				41 --		
				42 --		Silty Sand, dark and grayish brown, moist, very dense, fine grained
42.5	50/5"	16.5	113.7	43 --		
				44 --		
45	76	3.8	SPT	45 --	SP	Sand, yellow and grayish brown, moist, very dense, fine to medium grained
				46 --		
				47 --		
47.5	50/4"	4.4	111.9	48 --		
				49 --	SM	Silty Sand, dak and yellowish brown, moist, medium dense, fine grained
50	29	28.5	SPT	50 --		Total Depth 50 feet No Water Fill to 3 feet

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 1'-5'	SM	120.8	7.6	13.7
B2 @ 1'-5'	SM	120.6	7.9	13.2
B4 @ 1'-5'	SM/ML	110.8	11.9	20.1
B6 @ 1'-5'	SM	119.3	8.6	14.8



● Direct Shear, Saturated

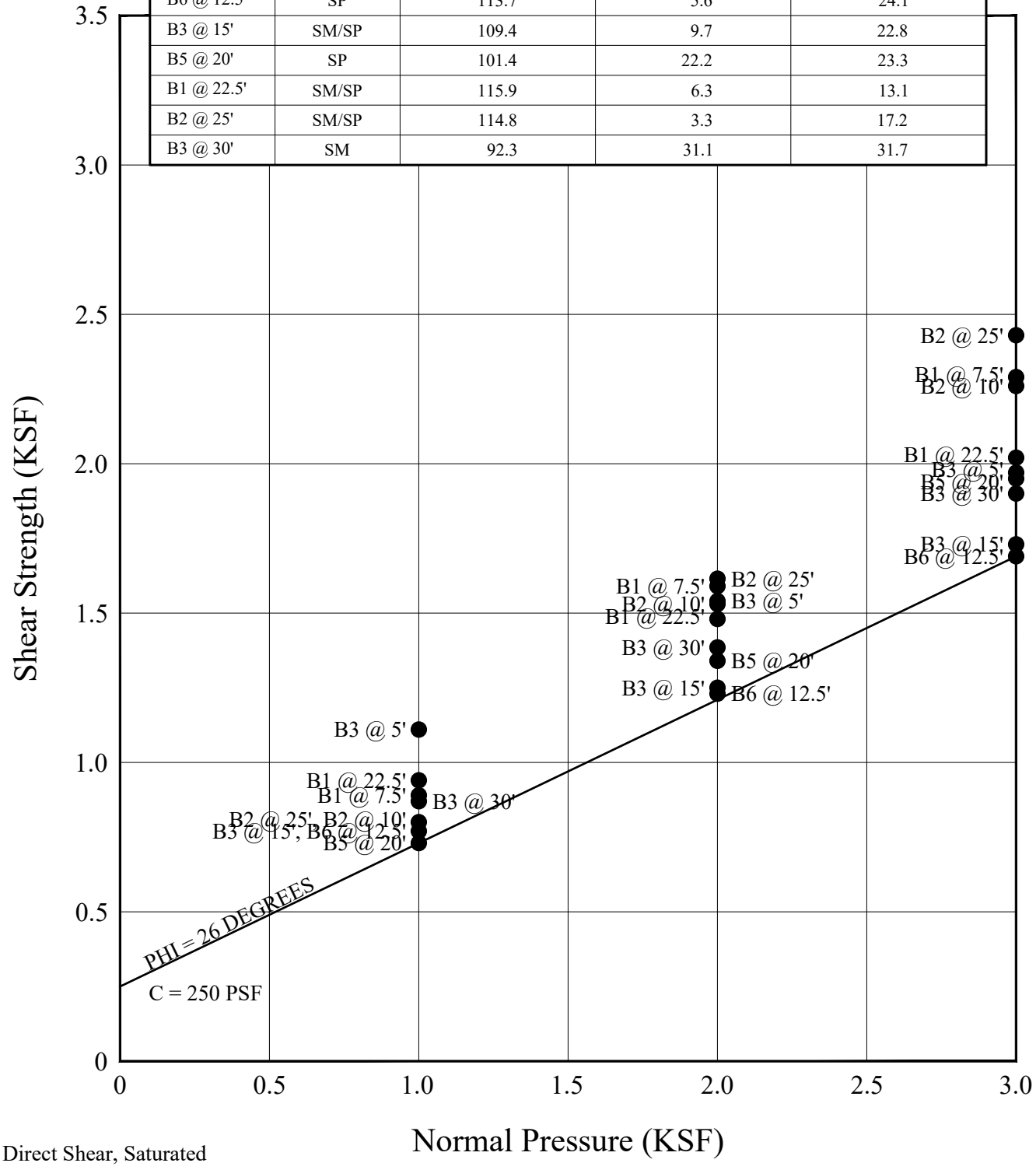
SHEAR TEST DIAGRAM (ASTM D3080)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

STEWART MANHATTAN INC.

FILE NO: 22516 | PLATE: B-1

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3 @ 5'	SM	113.4	17.4	20.9
B1 @ 7.5'	SM	112.7	17.7	19.5
B2 @ 10'	SM/ML	118.5	12.1	24.6
B6 @ 12.5'	SP	113.7	5.6	24.1
B3 @ 15'	SM/SP	109.4	9.7	22.8
B5 @ 20'	SP	101.4	22.2	23.3
B1 @ 22.5'	SM/SP	115.9	6.3	13.1
B2 @ 25'	SM/SP	114.8	3.3	17.2
B3 @ 30'	SM	92.3	31.1	31.7



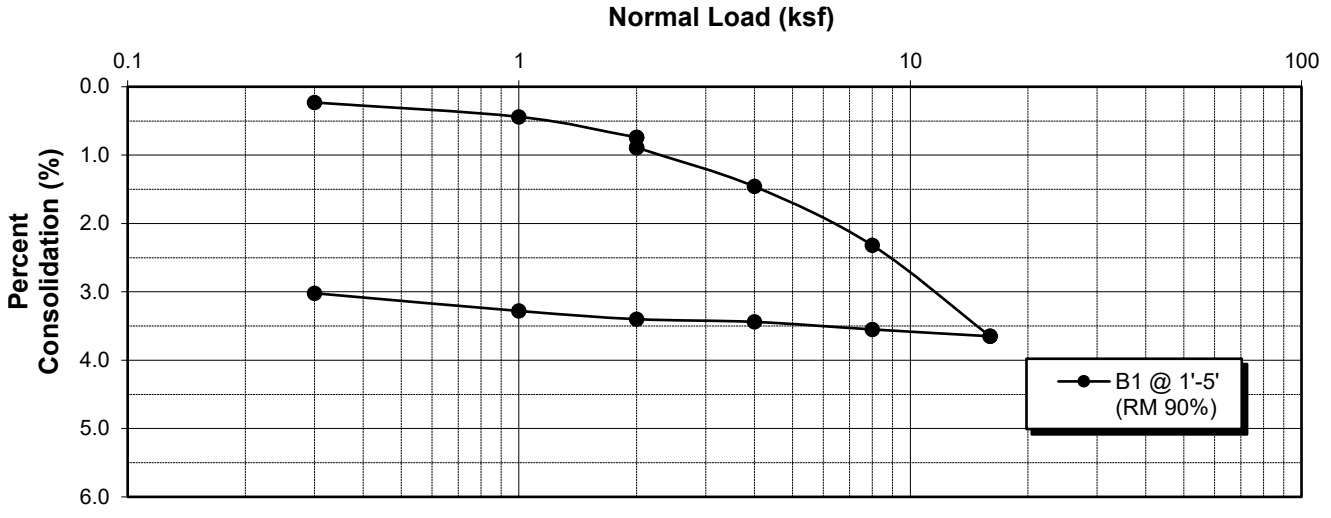
● Direct Shear, Saturated

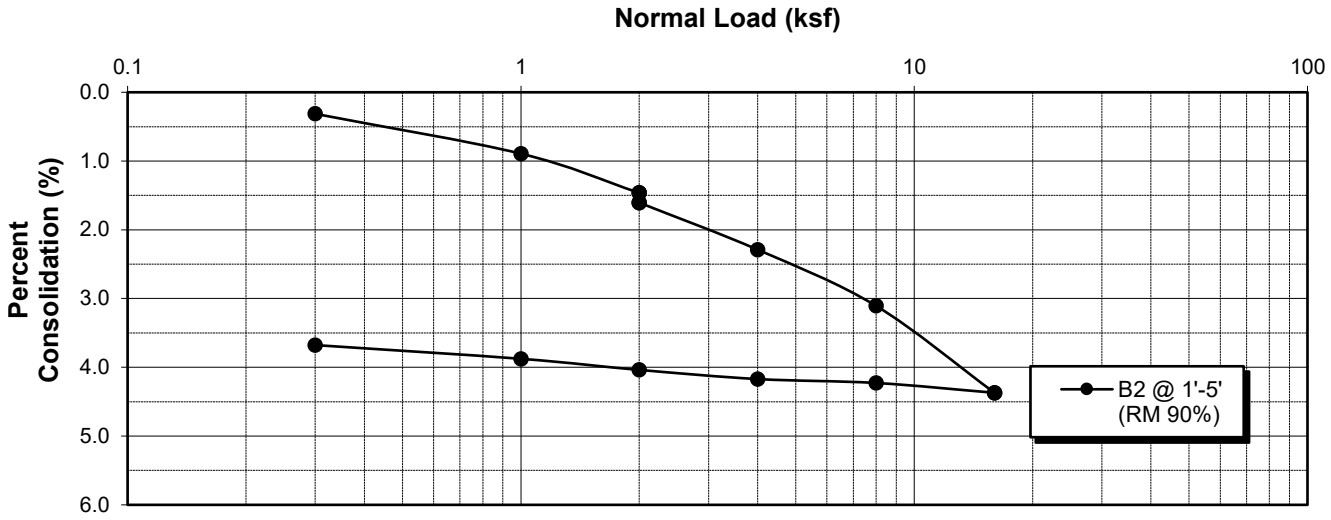
SHEAR TEST DIAGRAM (ASTM D3080)

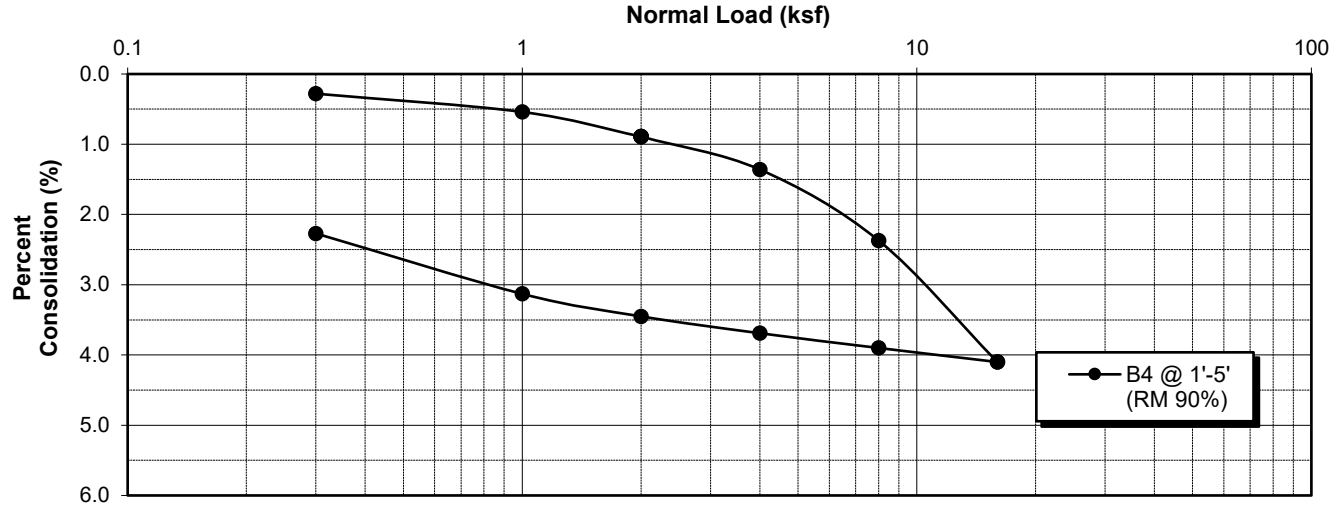
Geotechnologies, Inc.
Consulting Geotechnical Engineers

STEWART MANHATTAN INC.

FILE NO: 22516 | PLATE: B-2







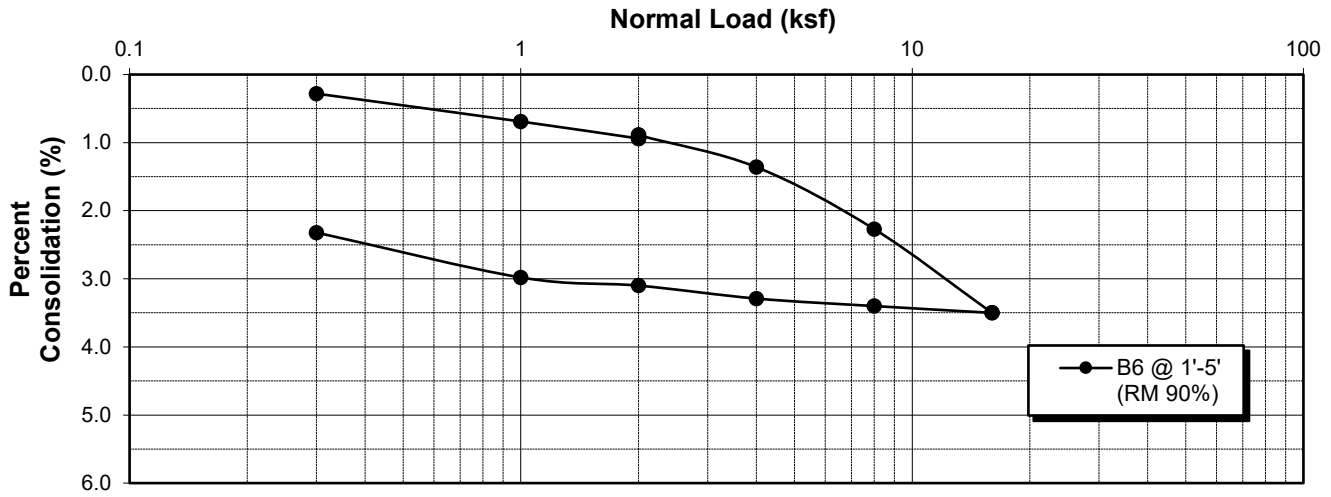
Water added at 2 KSF

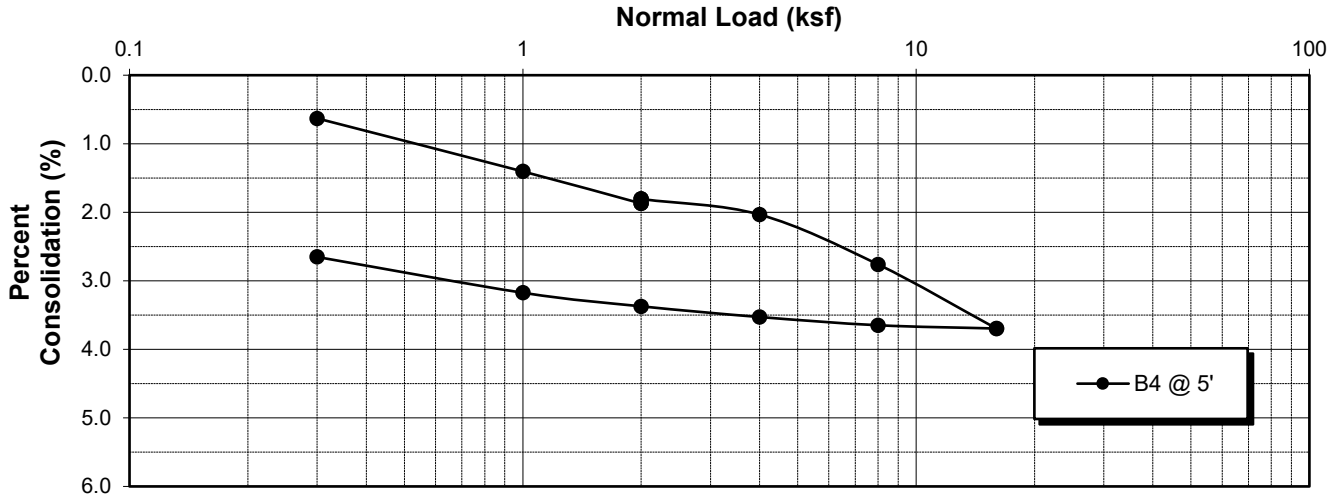
CONSOLIDATION (ASTM D2435)

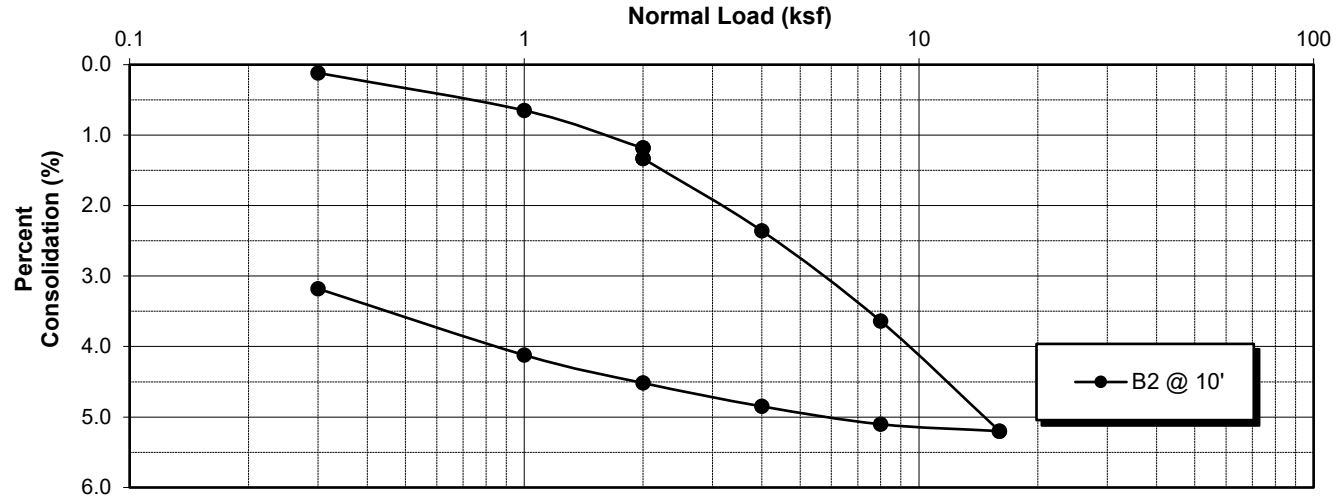
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: STEWART MANHATTAN INC.

FILE NO.: 22516 | PLATE: C-1







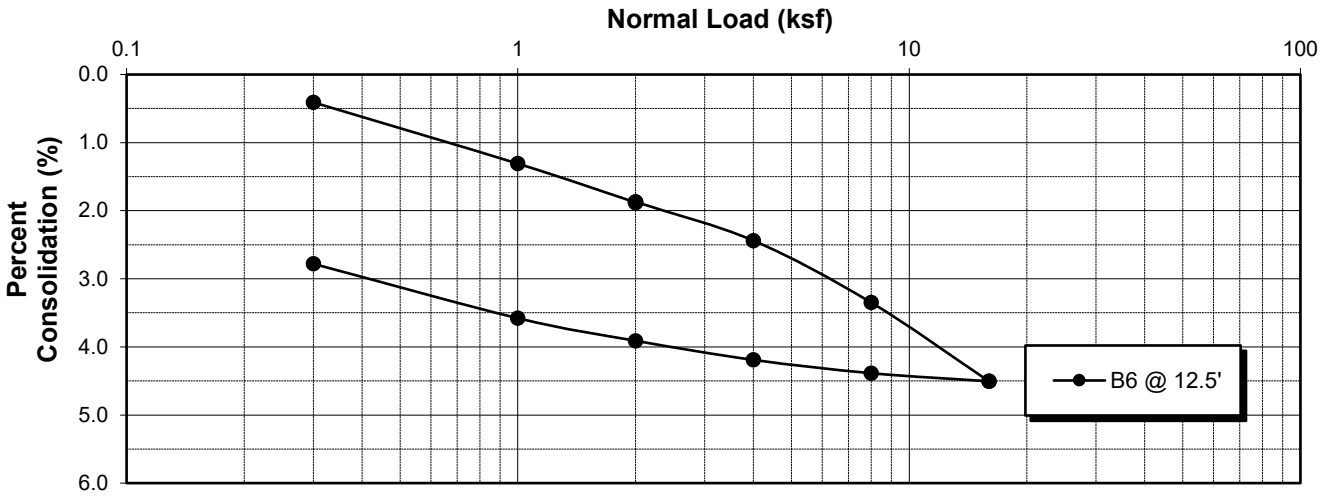
Water added at 2 KSF

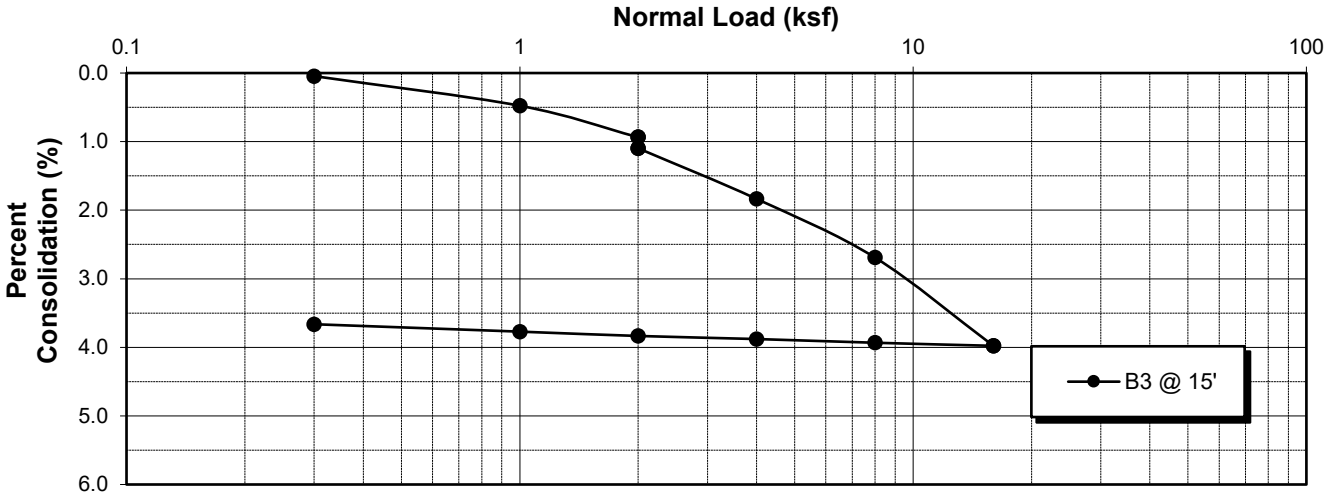
CONSOLIDATION(ASTM D2435)

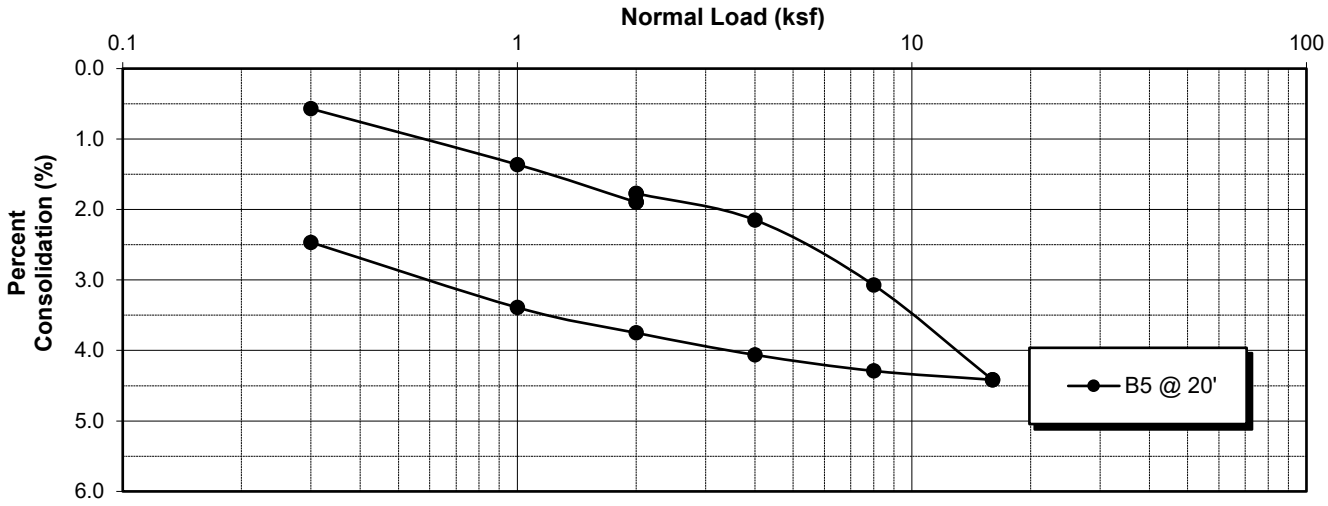
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PROJECT: STEWART MANHATTAN INC.

FILE NO.: 22516 | PLATE: C-2







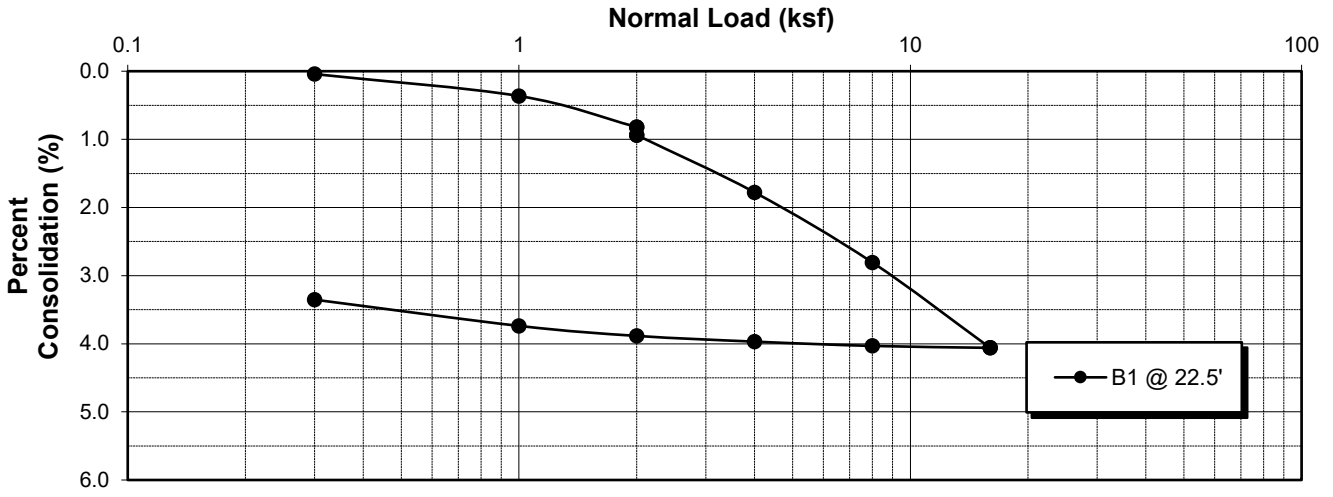
Water added at 2 KSF

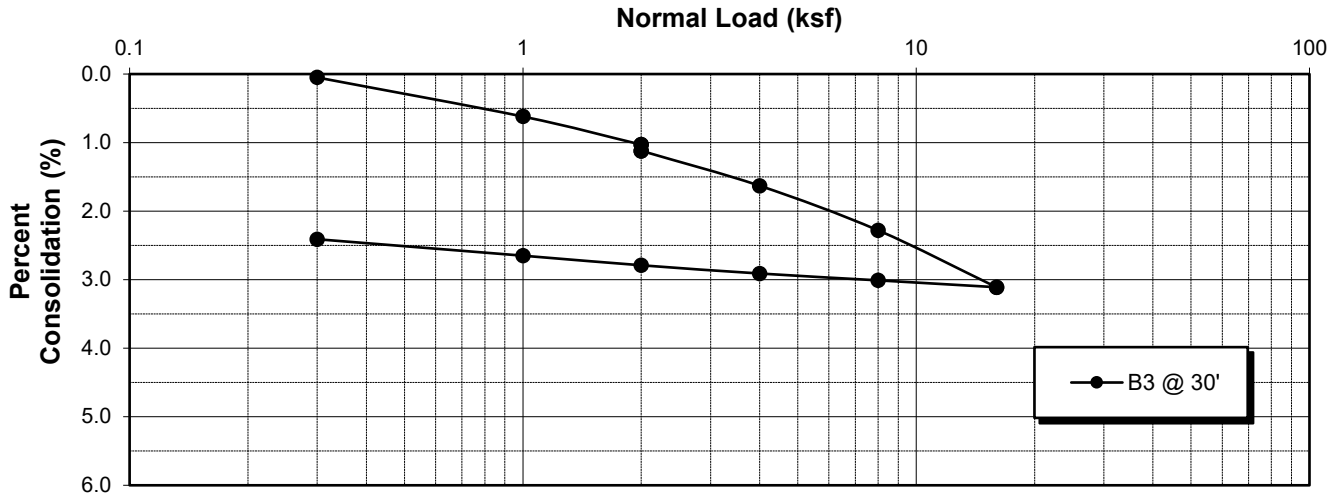
CONSOLIDATION(ASTM D2435)

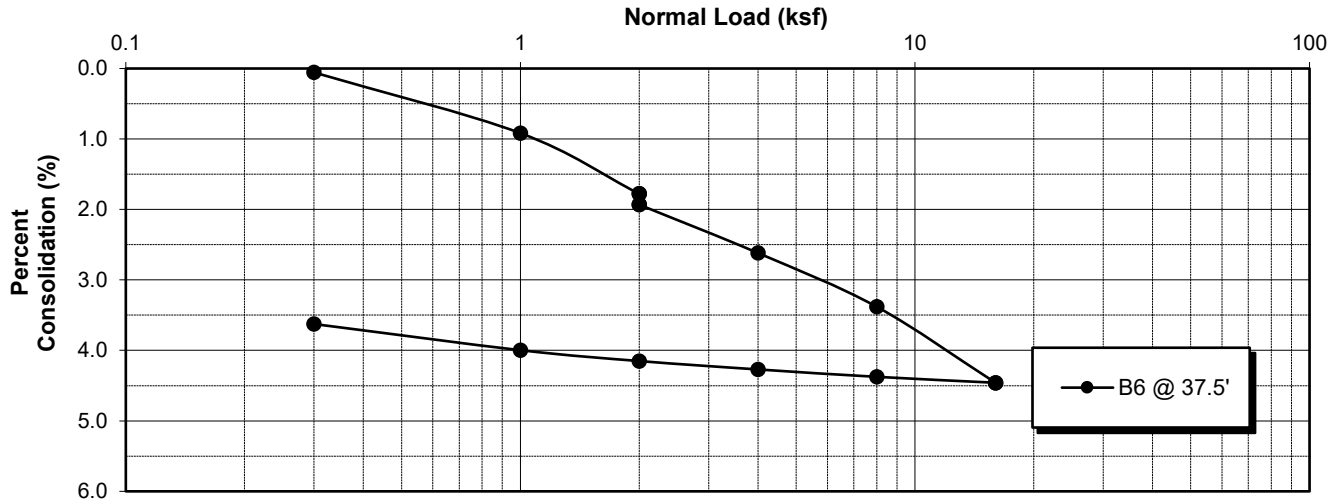
Geotechnologies, Inc.
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PROJECT: STEWART MANHATTAN INC.

FILE NO.: 22516 | PLATE: C-3







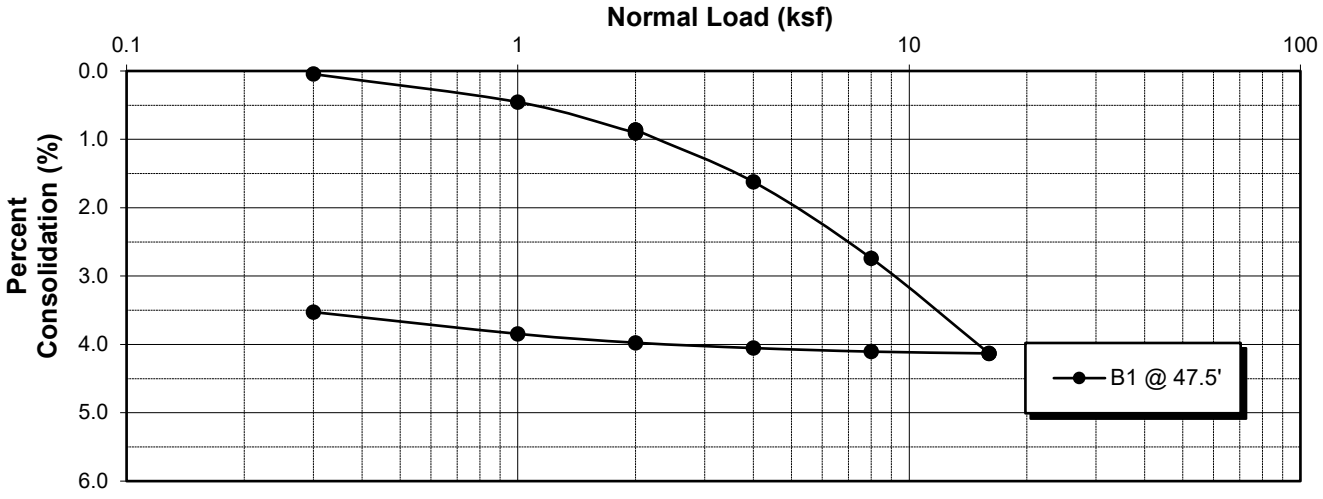
Water added at 2 KSF

CONSOLIDATION(ASTM D2435)

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Consulting Geotechnical Engineers

PROJECT: STEWART MANHATTAN INC.

FILE NO.: 22516 | PLATE: C-4



Water added at 2 KSF

CONSOLIDATION (ASTM D2435)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: STEWART MANHATTAN INC.

FILE NO.: 22516 | PLATE: C-5

LABORATORY COMPACTION CHARACTERISTICS
(ASTM D1557)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'	B4 @ 1'-5'	B6 @ 1'-5'
SOIL TYPE	SM	SM	SM/ML	SM/SC
MAXIMUM DRY DENSITY (PCF)	134.2	134.0	123.1	132.5
OPTIMUM MOISTURE %	7.6	7.9	11.9	8.6

EXPANSION INDEX
(ASTM D4829)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'	B4 @ 1'-5'	B6 @ 1'-5'
SOIL TYPE	SM	SM	SM/ML	SM/SC
EXPANSION INDEX UBC STANDARD 18-2	10	7	43	50
EXPANSION CHARACTER	VERY LOW	VERY LOW	LOW	MODERATE

SULFATE CONTENT
(CALIFORNIA TEST 417)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'	B4 @ 1'-5'	B6 @ 1'-5'
SULFATE CONTENT: (Percentage by Weight)	<0.1%	<0.2%	<0.2%	<0.2%

SULFATE CONTENT
(CALIFORNIA TEST 417)

SAMPLE	B1 @ 10'	B1 @ 20'	B3 @ 15'
SULFATE CONTENT: (Percentage by Weight)	<0.1%	<0.1%	<0.1%



COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

STEWART MANHATTAN INC.

FILE NO: 22516 | PLATE: D

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 1
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
		3.5" Asphalt Concrete,						
	SM	3.5"-18" Artificial FILL (Af) Medium brown silty sand, moist,						
		1.5'-31.5' Alluvium (Qa)						
	SM	Olive gray/brown silty sand, fine, moist, few pebbles & small gravel,	11 14 32	6.7	118.9			
5	SP SM	sand to silty sand, very moist,	6 8 13	12.8	101.3			
10	SP	sand to sand slightly silty, few pebbles, moist,	11 15 18	7.1	113.2			
15	SM	Olive brown/ gray with white and rust iron stains Interbed layer of sand, silty sand and gravelly sand	8 10 13	8.3	108.5			

Cont. next page....

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 1
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
20	SM	Olive brown/ gray with white and rust iron stains Interbed layer of sand, silty sand and gravelly sand	14 12 16	8.1	114.9			
25	SP	becomes sand slightly silty, moist,	12 15 16	5.6	113.7			
30	SM		13 18 23	8.3	115.1			
35		**End of boring @ 31.5-ft** **No cavings or groundwater** **Boring backfilled with cuttings on 12/10/21**						

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 2
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
		6" Asphalt Concrete over 2" Aggregate Base						
	SM	8"-36" Artificial FILL (Af) Dark gray silty sand, trace clay, moist-very moist,	5					
	SM		5 6	13.2	104.8			
		3.0'-36.5' Alluvium (Qa) Dark olive gray/brown silty sand, fine, moist, few pebbles & small gravel,						
5	SM		13 15 16	12.5	121.0			
10	SM SP	mixture of silty sand and fine poorly graded sand,	5 7 13	9.6	113.8			
15	SM	Becones olive gray silty sand to sand slightly silty, pebbles & small size gravels,	3 5 6	10.7	105.5			

Cont. next page....

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 2
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
20	SM	Olive gray silty sand to sand slightly silty, pebbles & small size gravels,	3 6 11	11.1	107.3			
25	SM SP	mixture of silty sand and fine poorly graded sand,	5 9 13	10.5	108.7			
30	SM ML	silty sand/sandy silt, few small gravel,	7 11 13	8.9	111.3			
35	SM	becomes gravelly silty sand,	6 13 18	10.7	114.3			
		End of boring @ 36.5-ft **No cavings or groundwater** **Boring backfilled with cuttings on 12/10/21**						

Geodyne Engineering, Inc.

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 3
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk

Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	Undisturbed	Bulk	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
	SC-SM			6" Asphalt Concrete 0.5'-4.5' Artificial FILL (Af) Dark gray silty, clayey sand, moist-very moist,	7					
	SC-SM	X			9 13	13.2	108.7			Expansion Index Atterberg Limits % Passing #200
5	SM ML	X		**4.5'-36.5' Alluvium (Qa)** Light to med. olive brown/gray silty sand/ sandy silt	10 11 17	14.4	112.2			
10	SM ML	X		mixture of poorly graded fine silty sand and sandy silt,	7 7 9	9.8	98.2			
15	SW	X		becomes fine to coarse sand slightly silty to clean, pebbles & gravels	6 7 19	4.1	113.5			

Cont. next page....

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 3
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
20	SP SM	mixture of poorly graded fine sand and silty sand	7 9 17	8.5	108.3			
25	SP SM	same as above descriptions,	7 12 14	8.7	110.5			
30	SM ML	silty sand/sandy silt,	5 9 14	10.7	107.5			
35	SM ML	same as above descriptions,	8 13 15	9.9	109.1			

End of boring @ 36.5-ft
No cavings or groundwater
Boring backfilled with cuttings on 12/10/21

Geodyne Engineering, Inc.

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 4
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
	SM	5" Asphalt Concrete, **0.5"-3.0' Artificial FILL (Af)** Dark gray silty sand, moist-very moist,						
	SM		5 6 9	13.8	105.7			Corrosion Series
		3.0'-51.5' Alluvium (Qa) Olive gray/brown silty sand, very fine to fine, few pebbles, moist,						
5	SM		5 7 8	9.3	100.2			Consolidation
10	SM	same as above descriptions	6 8 16	7.4	108.6			Direct Shear Consolidation
15	SM	becomes fine-coarse, few small rocks 3/8"-1/2"	10 12 15	5.7	97.7			

Cont. next page....

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 4
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
20	SM	Olive gray/brown silty sand, very fine to fine, few pebbles, moist,	14 19 38	5.8	116.2			Consolidation
25	SP	becomes sand slightly silty	9 12 15	8.5	91.7			
30	SM ML	Becomes very fine silty sand/sandy silt, damp-moist,	13 16 35	8.2	108.9			
35	SM ML	Olive grayish green with rust iron stains,	7 10 16	5.6				

Cont. next page....

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 4
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
40	SM ML	Olive gray very fine silty sand/sandy silt, damp-moist,	50-6"	4.3				
45	SM ML	Oliver grayish green with rust iron stains,	50-6"	5.1				
50	SM ML		50-6"	4.5				
		End of boring @ 51.5-ft **No cavings or groundwater** **Boring backfilled with cuttings on 12/10/21** **Difficult drilling/grinding below 35 ft**						
55								

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 5
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
	SM	5" Asphalt Concrete 5"-36" Artificial FILL (Af) Dark grayish brown silty sand, trace clay, moist,						
	SM		4 6 13	14.1	105.5			
		3.0'-36.5' Alluvium (Qa) Olive gray/brown silty sand, fine, moist,						
5	SM		5 7 19	13.3	118.2			
10	SM SP	same as above descriptions,	7 7 9	12.9	108.3			
15	SM	same as above descriptions,	5 6 8	11.7	106.8			

Cont. next page....

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 5
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
20	SM	Olive gray silty sand to sand slightly silty,	6 10 18	11.1	107.3			
25	SM ML	silty sand/sandy silt, few small gravel,	7 11 17	10.6	111.5			
30	SM ML	same as above descriptions, becomes gravelly,	6 13 15	9.7	110.5			
35	SM ML		13 15 17	12.3	117.1			
		End of boring @ 36.5-ft **No cavings or groundwater** **Boring backfilled with cuttings on 12/10/21**						

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 6
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	Undisturbed	Bulk	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
	SM			5" Asphalt Concrete, 5"-18" Artificial FILL (Af) Dark gray silty sand, trace clay, very moist,						
				1.5'-31.5' Alluvium (Qa)						
	SM	X		Olive gray/brown silty sand, fine, moist,	3 4 6	14.3	105.1			
5	SM	X		same as above descriptions,	3 6 9	12.5	106.7			
10	SM	X		few pebbles & grevel size 3/8"	5 6 8	6.3	107.3			
15	SM ML	X		Interbed layer of fine silty sand/sandy silt	7 9 12	8.7	109.1			

Cont. next page....

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 6
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
20	SM	Olive gray/brown fine silty sand, moist,	11 13 15	8.9				
25	SM ML	fine silty sand/sandy silt,	7 11 18	11.3				
30	SM ML	same as above descriptions,	11 13 21	8.7				
		End of boring @ 31.5-ft **No cavings or groundwater** **Boring backfilled with cuttings on 12/10/21**						
35								

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 7
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk
Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
		5.5" Asphalt Concrete,						
	SM	5.5"-24" Artificial FILL (Af) Dark gray silty sand, trace clay, very moist,						
		2.0'-31.5' Alluvium (Qa)						
		Olive gray/brown fine silty sand, moist,	6					
	SM	few pebbles & small gravel,	8 13	13.5	104.7			
5	SM	same as above descriptions,	9 10 18	10.1	108.3			
10	SM	same as above descriptions,	10 11 11	10.7	107.8			
15	SM ML	silty sand/sandy silt,	8 13 17	7.5	113.3			

Cont. next page....

LOG OF EXPLORATORY BORING

Project No. GD21G185
Project: **STOCKER STREET CREATIVE, LA**

Date: 12/10/2021
Driller: Discovery/Duxbury Drilling

Boring No: 7
Boring Location: See Plot Plan
Groundwater: Not Encountered
Drill Type: CME 75 Automatic Hammer
Boring Hole Diameter (in): 8
Hammer: 140 Lbs, 30" Drop

Sample Type: Undisturbed, Bulk

Thin Wall Tube | 2.5" Ring Sample | Bulk Sample | Standard Split Spoon Sample | Static Water Table

Depth in Feet	Soil Type	SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Other Tests
20	SM	Olive gray/brown fine silty sand, moist,	9 11 13	7.7				
25	SM ML	fine silty sand/sandy silt,	6 13 15	9.9				
30	SM ML	same as above descriptions,	13 15 17	7.8				
		End of boring @ 31.5-ft **No cavings or groundwater** **Boring backfilled with cuttings on 12/10/21**						
35								

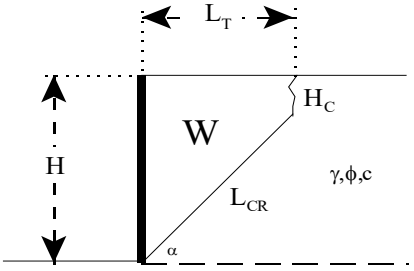
Geotechnologies, Inc.

Project: Stewart Manhattan, Inc.
File No.: 22516
Description: Retaining Walls up to 10 feet

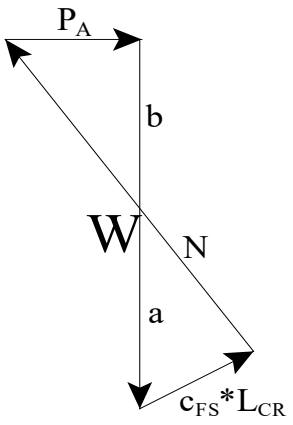
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	10.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	26.0	degrees
Cohesion of Retained Soils	(c)	250.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	18.0	degrees
	(c_{FS})	166.7	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	4.0	42	5274.1	8.6	2987.6	2286.5	1164.4
46	3.9	41	5122.4	8.5	2869.0	2253.5	1197.6
47	3.8	40	4970.4	8.4	2756.3	2214.1	1226.7
48	3.8	39	4818.6	8.4	2649.3	2169.3	1251.8
49	3.8	37	4667.4	8.3	2547.8	2119.6	1273.0
50	3.7	36	4517.2	8.2	2451.4	2065.8	1290.2
51	3.7	35	4368.0	8.1	2359.7	2008.4	1303.6
52	3.7	34	4220.2	8.0	2272.4	1947.8	1313.2
53	3.7	33	4073.8	7.9	2189.4	1884.5	1318.9
54	3.7	31	3928.9	7.8	2110.1	1818.8	1320.9
55	3.7	30	3785.5	7.7	2034.3	1751.1	1319.0
56	3.7	29	3643.5	7.6	1961.8	1681.7	1313.3
57	3.7	28	3503.0	7.5	1892.3	1610.7	1303.8
58	3.7	27	3364.0	7.4	1825.4	1538.5	1290.4
59	3.8	26	3226.3	7.3	1761.0	1465.3	1273.2
60	3.8	25	3089.9	7.2	1698.7	1391.2	1252.1
61	3.8	24	2954.7	7.0	1638.3	1316.4	1227.0
62	3.9	23	2820.6	6.9	1579.5	1241.0	1198.0
63	4.0	21	2687.5	6.8	1522.1	1165.3	1164.8
64	4.0	20	2555.2	6.7	1465.8	1089.4	1127.6
65	4.1	19	2423.7	6.5	1410.3	1013.4	1086.3
66	4.2	18	2292.8	6.4	1355.3	937.5	1040.7
67	4.3	17	2162.3	6.2	1300.5	861.7	990.9
68	4.4	16	2032.0	6.0	1245.6	786.4	936.8
69	4.6	15	1901.7	5.8	1190.0	711.7	878.4
70	4.7	14	1771.1	5.6	1133.5	637.7	815.8



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 1320.85 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 26.4 pcf

Design Wall for an Equivalent Fluid Pressure: **30 pcf**

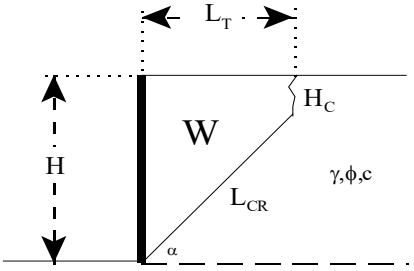
Geotechnologies, Inc.

Project: Stewart Manhattan, Inc.
File No.: 22516
Description: Retaining Walls up to 18 feet

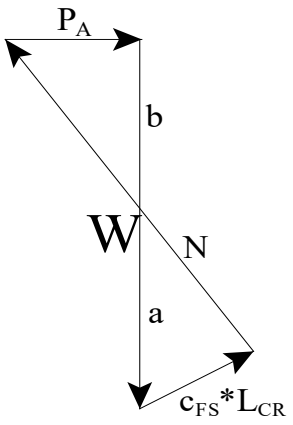
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	18.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	26.0	degrees
Cohesion of Retained Soils	(c)	250.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	18.0	degrees
	(c_{FS})	166.7	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	4.0	154	19274.1	19.9	6939.1	12335.0	6281.7
46	3.9	149	18642.1	19.6	6625.1	12016.9	6386.2
47	3.8	144	18025.6	19.4	6333.8	11691.8	6477.6
48	3.8	139	17424.3	19.1	6063.1	11361.2	6556.1
49	3.8	135	16837.4	18.9	5811.1	11026.4	6622.1
50	3.7	130	16264.5	18.6	5576.0	10688.6	6675.8
51	3.7	126	15705.0	18.4	5356.4	10348.6	6717.3
52	3.7	121	15158.2	18.2	5150.9	10007.3	6746.9
53	3.7	117	14623.6	17.9	4958.2	9665.3	6764.7
54	3.7	113	14100.5	17.7	4777.3	9323.1	6770.6
55	3.7	109	13588.4	17.5	4607.2	8981.2	6764.8
56	3.7	105	13086.6	17.3	4446.8	8639.8	6747.2
57	3.7	101	12594.7	17.1	4295.4	8299.4	6717.8
58	3.7	97	12112.1	16.8	4152.1	7960.0	6676.4
59	3.8	93	11638.3	16.6	4016.3	7622.0	6622.9
60	3.8	89	11172.8	16.4	3887.4	7285.4	6557.0
61	3.8	86	10715.0	16.2	3764.5	6950.5	6478.7
62	3.9	82	10264.5	16.0	3647.3	6617.2	6387.4
63	4.0	79	9820.8	15.8	3535.1	6285.7	6283.0
64	4.0	75	9383.5	15.6	3427.4	5956.1	6165.1
65	4.1	72	8952.0	15.3	3323.7	5628.3	6033.1
66	4.2	68	8526.0	15.1	3223.4	5302.6	5886.6
67	4.3	65	8104.9	14.9	3126.1	4978.9	5725.1
68	4.4	62	7688.3	14.6	3031.1	4657.2	5547.9
69	4.6	58	7275.8	14.4	2938.0	4337.8	5354.4
70	4.7	55	6866.7	14.1	2846.1	4020.6	5143.9



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 6770.63 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 41.8 pcf

Design Wall for an Equivalent Fluid Pressure: **42 pcf**

Geotechnologies, Inc.

Project: Stewart Manhattan, Inc.
File No.: 22516

Soil Weight	γ	125	pcf
Internal Friction Angle	ϕ	26	degrees
Cohesion	c	250	psf
Height of Retaining Wall	H	18	feet

Restrained Retaining Wall Design based on At Rest Earth Pressure

$\sigma'_h = K_o\sigma'_v$

$K_o = 1 - \sin\phi$ 0.562

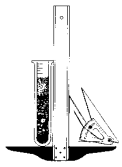
$\sigma'_v = \gamma H$ 2250.0 psf

σ'_h = 1263.7 psf

EFP = 70.2 pcf

P_o = 11373.0 lbs/ft (based on a triangular distribution of pressure)

Design wall for an EFP of 71 pcf



Geotechnologies, Inc.

Project: Stewart Manhattan, Inc.
File No.: 22516

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	18.0	feet
Retained Soil Unit Weight:	(γ)	125.0	pcf
Peak Ground Acceleraction:	(PGA_M)	0.92	g
Horizontal Ground Acceleration:	(k_h)	0.31	g

Seismic Increment (ΔP_{AE}):

$k_h = 0.5*0.67*PGA_M$

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.75*k_h)$

ΔP_{AE} = 4665.5 lbs/ft

$T*(2/3)*H = \Delta P_{AE}*0.6*H$

T = 4199.0 lbs/ft

$EFP = 2*T/H^2$

EFP = 25.9 pcf

triangular distribution of pressure, applied to the proposed retaining wall.

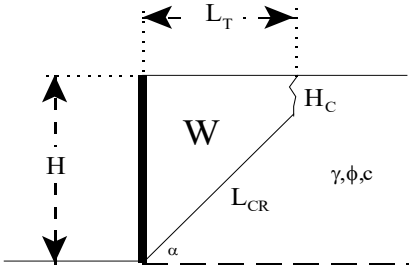
Geotechnologies, Inc.

Project: Stewart Manhattan Inc.
File No.: 22516
Description: Shoring Walls up to 10 feet

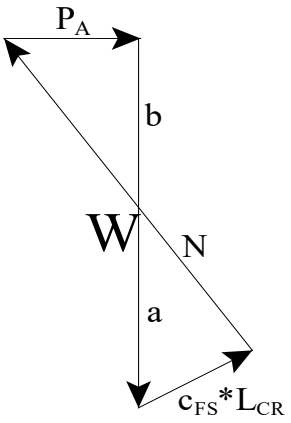
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	10.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	26.0	degrees
Cohesion of Retained Soils	(c)	250.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	21.3	degrees
	(c_{FS})	200.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.2	36	4529.0	6.7	3117.3	1411.6	619.2
46	5.1	36	4442.3	6.8	3015.5	1426.8	655.8
47	5.0	35	4346.2	6.8	2913.9	1432.3	688.9
48	5.0	34	4242.9	6.8	2813.6	1429.3	718.4
49	4.9	33	4133.8	6.8	2715.2	1418.6	744.3
50	4.8	32	4020.4	6.7	2619.2	1401.2	766.6
51	4.8	31	3903.5	6.7	2525.8	1377.7	785.3
52	4.7	30	3784.0	6.7	2435.1	1348.8	800.4
53	4.7	29	3662.5	6.6	2347.2	1315.3	811.9
54	4.7	28	3539.6	6.6	2262.0	1277.5	819.7
55	4.7	27	3415.5	6.5	2179.5	1236.0	823.9
56	4.7	26	3290.7	6.4	2099.4	1191.3	824.4
57	4.7	25	3165.4	6.3	2021.7	1143.7	821.4
58	4.7	24	3039.7	6.2	1946.1	1093.6	814.7
59	4.7	23	2913.7	6.1	1872.4	1041.3	804.4
60	4.8	22	2787.6	6.0	1800.4	987.2	790.5
61	4.8	21	2661.3	5.9	1729.8	931.5	772.9
62	4.9	20	2534.9	5.8	1660.5	874.4	751.7
63	4.9	19	2408.4	5.7	1592.0	816.3	727.0
64	5.0	18	2281.6	5.5	1524.2	757.4	698.6
65	5.1	17	2154.5	5.4	1456.6	697.9	666.6
66	5.2	16	2027.0	5.2	1388.9	638.1	631.1
67	5.3	15	1898.8	5.1	1320.7	578.2	592.2
68	5.5	14	1770.0	4.9	1251.5	518.5	549.9
69	5.6	13	1640.1	4.7	1180.8	459.3	504.4
70	5.8	12	1508.9	4.5	1108.1	400.8	456.0



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 824.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 16.5 pcf

Design Shoring for an Equivalent Fluid Pressure: 28 pcf

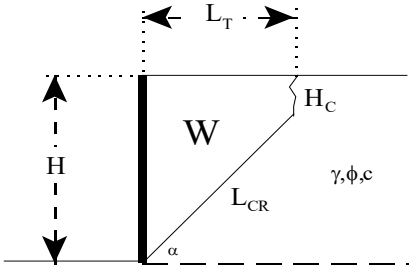
Geotechnologies, Inc.

Project: Stewart Manhattan Inc.
File No.: 22516
Description: Shoring Walls up to 20 feet

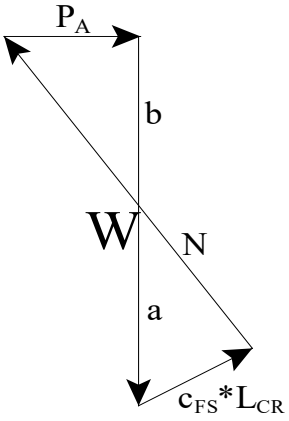
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	26.0	degrees
Cohesion of Retained Soils	(c)	250.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	21.3	degrees
	(c_{FS})	200.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.2	186	23279.0	20.9	9676.7	13602.3	5966.7
46	5.1	180	22549.0	20.7	9217.5	13331.4	6127.5
47	5.0	175	21830.9	20.5	8791.7	13039.2	6271.1
48	5.0	169	21125.5	20.2	8396.4	12729.0	6397.9
49	4.9	163	20433.0	20.0	8028.8	12404.2	6508.2
50	4.8	158	19753.5	19.8	7686.4	12067.1	6602.4
51	4.8	153	19086.9	19.6	7366.9	11720.0	6680.9
52	4.7	147	18433.1	19.4	7068.4	11364.7	6743.8
53	4.7	142	17791.6	19.1	6788.9	11002.8	6791.5
54	4.7	137	17162.2	18.9	6526.8	10635.5	6823.9
55	4.7	132	16544.4	18.7	6280.5	10263.9	6841.3
56	4.7	128	15937.8	18.5	6048.7	9889.0	6843.6
57	4.7	123	15341.8	18.3	5830.2	9511.6	6831.0
58	4.7	118	14756.0	18.0	5623.6	9132.3	6803.3
59	4.7	113	14179.9	17.8	5428.1	8751.8	6760.5
60	4.8	109	13612.9	17.6	5242.5	8370.4	6702.4
61	4.8	104	13054.6	17.4	5065.9	7988.7	6628.8
62	4.9	100	12504.5	17.1	4897.5	7607.0	6539.6
63	4.9	96	11962.0	16.9	4736.4	7225.6	6434.4
64	5.0	91	11426.6	16.7	4581.8	6844.8	6312.8
65	5.1	87	10897.7	16.4	4433.0	6464.8	6174.6
66	5.2	83	10375.0	16.2	4289.2	6085.8	6019.3
67	5.3	79	9857.8	15.9	4149.5	5708.2	5846.3
68	5.5	75	9345.5	15.7	4013.4	5332.1	5655.3
69	5.6	71	8837.5	15.4	3879.8	4957.8	5445.6
70	5.8	67	8333.4	15.1	3748.0	4585.4	5216.7



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 6843.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 34.2 pcf

Design Shoring for an Equivalent Fluid Pressure: 35 pcf