

ATTACHMENT D

GeoTech Services
Geotechnical Investigation
627-665 S. La Brea Avenue
December 27, 2016.

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February 13, 2019

15-473

CGI
6300 Canoga Avenue, Suite 1100
Woodland Hills, CA 91367

Subject: Change of Soil Engineer of Record
Response To Correction Letter And Report Update
Proposed Mixed-Use Eight Story Building With Seven Residential Floors
On Top Of One Level Of Office and/or Retail Space
With Two Levels of Subterranean Parking
Tract: 5273, M B 55-52
Lot: 38,39,40,41,42,43,44,45,46,47,48
625-665 S. La Brea Avenue
Los Angeles, California

INTRODUCTION

<u>Reference Report/Letters</u>	<u>Report No.</u>	<u>Date(s) of Document</u>	<u>Prepared By</u>
Dept. Correction letter	97427	04/06/2017	LADBS
Soils Report	15-473	12/27/2016	GeoTech Services

This letter confirms that we are in receipt of copies of the above listed geotechnical investigation report and department letter for the subject project.

CHANGE OF RESPONSIBILITY

We have reviewed the reports and concur with all of the recommendations including foundation design recommendations presented in the report except for what is modified in this update report. This office will respond to the correction items issued as listed above, provide update for proposed project and provide observation and testing services during construction and will assume responsibility for the foundation design data contained in previous report in accordance with Code Section 91-7008.5. We accept responsibility for reports and all department conditions of approval.

We will make certain that all of the recommendations presented in the soil investigation report and Grading Division of the City of Los Angeles provisions will be included in the project plans and will be carried out during construction.

All of the recommendations provided in the referenced original report remain applicable for the purpose of design and the proposed development, except as modified herein.

REPORT UPDATE

Since the submittal of the original report, and review letter from the Department there has been following changes in the project, as listed below:

1. the number of subterranean levels in the original soils report was proposed to be three levels. Only two level of subterranean are proposed now. Therefore, their maximum depth of excavation for the construction of the proposed subterranean parking will be reduced by some ten feet.
2. The hight of the building above the subterranean levels has been reduced from eleven stories to eight stories.
3. Addition of a lot number 38 to the north end of the project.

For our revised temporary shoring recommendation see Table 2 shoring design and the diagram in our attachments.

For our revised basement wall recommendations see Table 1: basement wall design within our attachments.

All of the other recommendations from the original soils report remained unchanged.

RESPONSE TO CORRECTION LETTER

Originaly submitted soils report was reviewed by LADBS and the review letter with two correction items were issued. The following two items are response to the correction items . A copy of the review letter is attached to this report.

1. We have revised our recommendations to provied hydrostatic and uplift pressure for the proposed building's basement walls, and mat foundation. Permanent dewatering has been abandoned for this project.
2. In the original soils report extensive dewatering was recommended due to three levels of subtranian, now with reduction to proposed two levels of subtranian

temporary dewatering will be reduced tremendously. Temporary dewatering is conducted for the level of water at present day. Water was encountered in our borings at 19 feet. Therefore temporary dewatering may not be needed or minor. No settlement of neighboring properties is expected due to temporary dewatering.

GeoTech Consultants, Inc. appreciates the opportunity to provide our services on this project. If you have any questions regarding this submittal or you wish to discuss the project further, please do not hesitate to call us.

Respectfully submitted,
GeoTech Consultants, Inc.

Reviewed By:

Vahik Hacopian
President

Behnam Mahmoudkhani
Civil Engineer
CE 88488

VH/BM

CITY OF LOS ANGELES
INTER-DEPARTMENTAL CORRESPONDENCE

SOILS REPORT REVIEW LETTER

April 6, 2017

LOG # 97427
SOILS/GEOLOGY FILE - 2
LIQ

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

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

Tentative Tract: 74779
LOT(S): 39-48
LOCATION: 627-665 S. La Brea Ave.

<u>CURRENT REFERENCE</u> <u>REPORT/LETTER(S)</u>	<u>REPORT</u> <u>No.</u>	<u>DATE(S) OF</u> <u>DOCUMENT</u>	<u>PREPARED BY</u>
Soils Report	15-473	12/27/2016	Geotech Services

The Grading Division of the Department of Building and Safety has reviewed the Tentative Tract 74779 with Los Angeles Department of City Planning receipt stamp dated and the referenced report that provide recommendations for the proposed 13 level mixed used building over 3 levels of basement parking. The earth materials at the subsurface exploration locations consist of up to 5 feet of uncertified fill underlain by native. The consultants recommend to support the proposed structure(s) on mat-type foundations bearing on native undisturbed soils.

The site is located in a designated liquefaction hazard zone as shown on the Seismic Hazard Zones map issued by the State of California. The Liquefaction study included as a part of the report/s demonstrates that the site does not possess a liquefaction potential. This satisfies the requirement of the 2014 Los Angeles City Building Code Section 1802.2.7.

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

(Note: Numbers in parenthesis () refer to applicable sections of the 2017 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. Due to the level of ground water encountered, permanent dewatering is not approved for the proposed project. Revise recommendations for the proposed foundation to be designed for hydrostatic uplift.
2. Provide an evaluation of the potential settlements of adjacent foundations due to the effects of temporary dewatering.

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

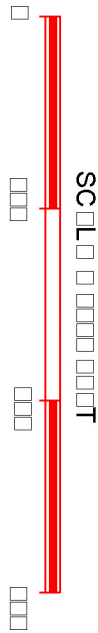
AD

AD/ad

Log No. 97427

213-482-0480

cc: La Brea Bliss LLC, Owner
Geotech Services, Project Consultant
LA District Office



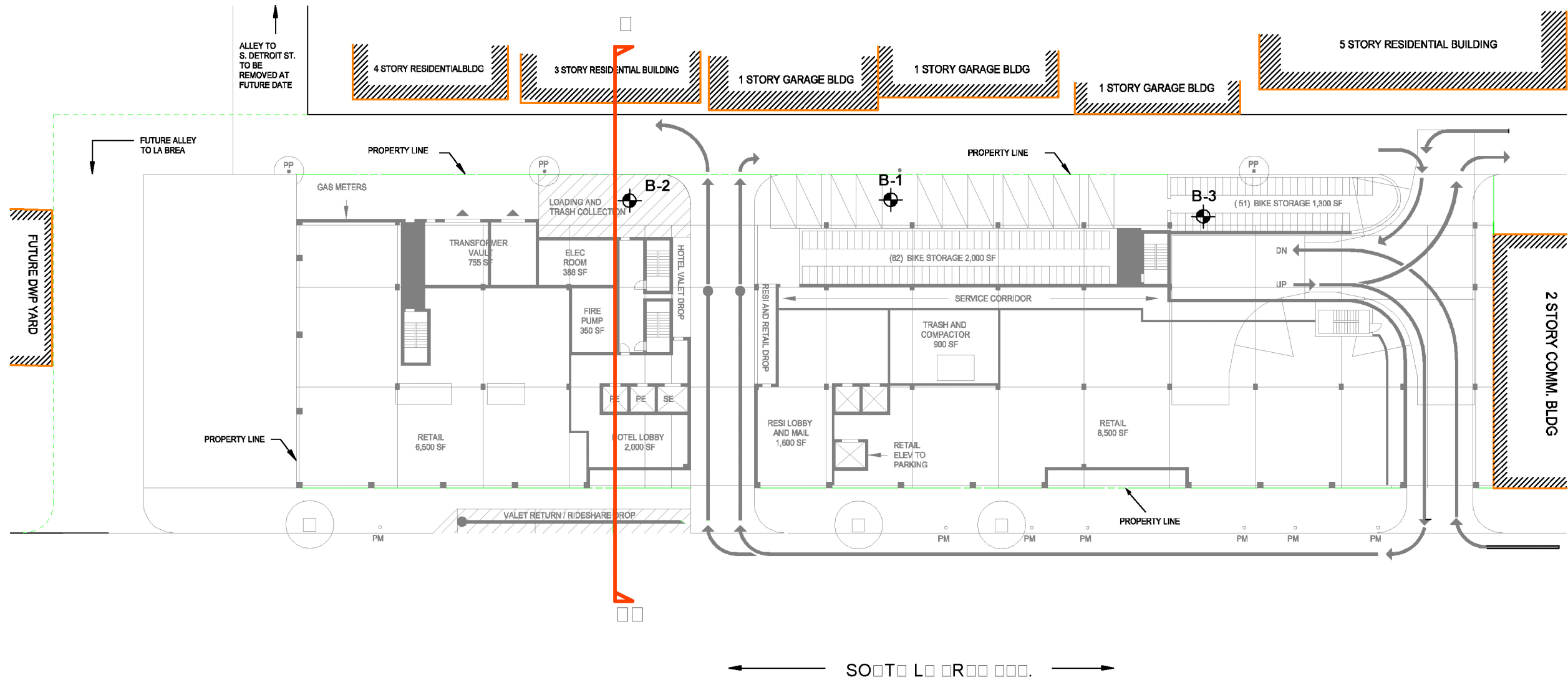
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Feet
Map output from an Intranet mapping
Data layers that appear on this map
may not be current, or otherwise reliable.

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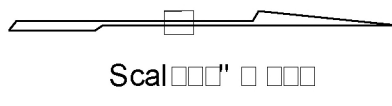
City of Los Angeles, California

GeoTech Consultants, Inc.

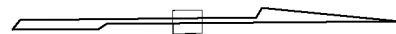
LOT No.



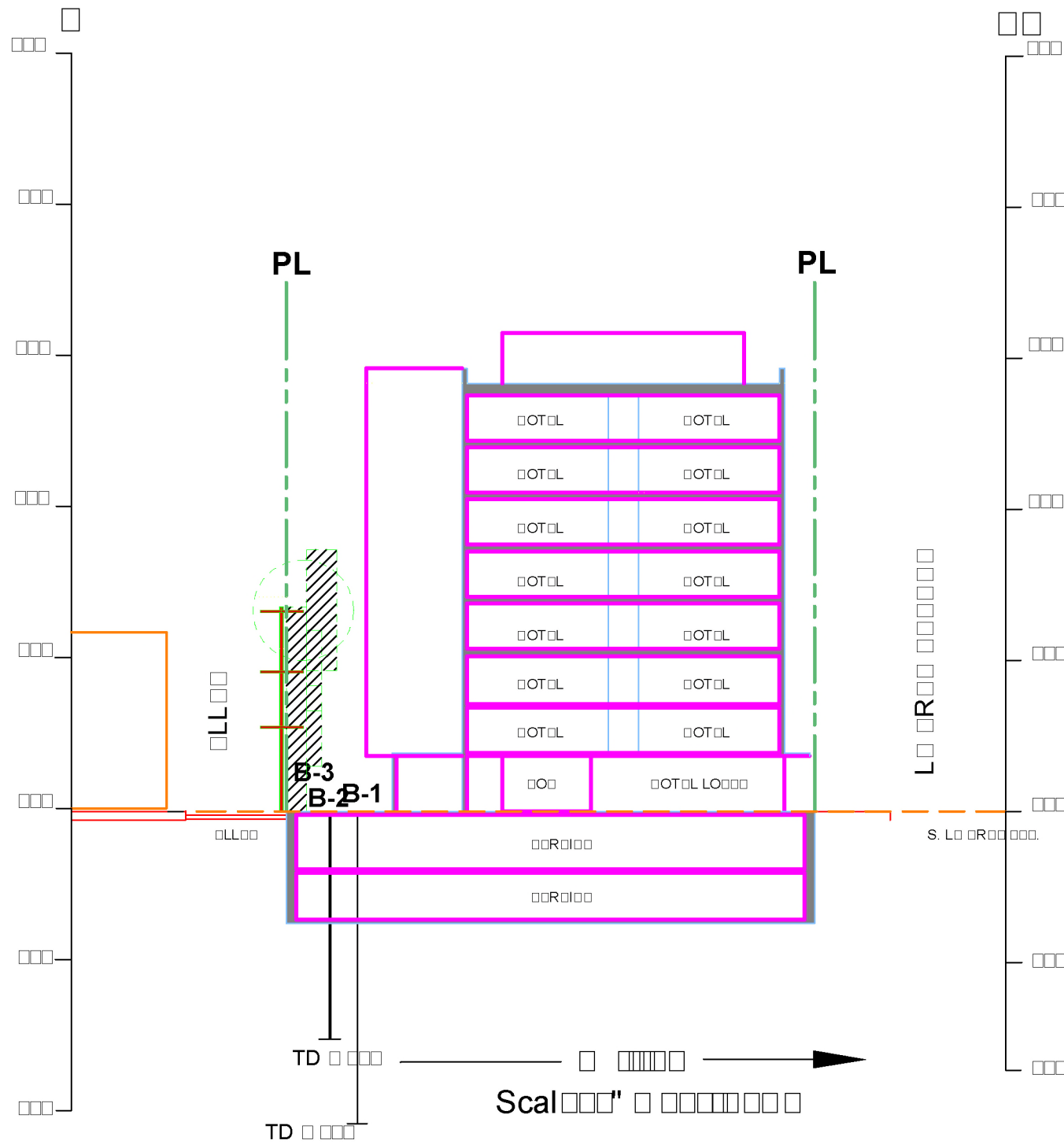
SOIT LOR



SIT PLAN		
Proposed Mixed-Use Building Project		
1000 S. La Brea Avenue, Los Angeles, California		
DR. C-1	DATE January 2024	PROJECT No. 24-001
Project Consultants, Inc.		DRIVER No. 1



Scale 1" = 100'



SITING PLAN

Proposed Multi-story Building Project

1000 S. La Brea Avenue, Los Angeles, California

DR.

City

Date

January 2000

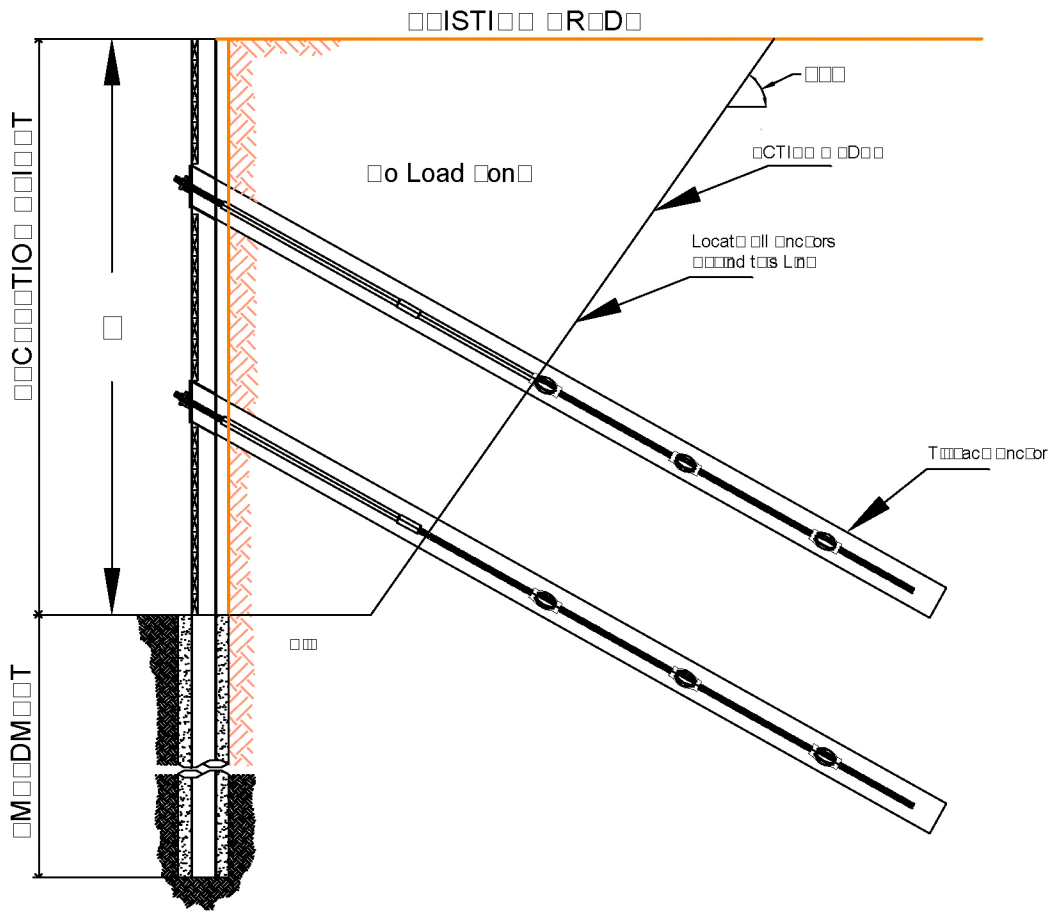
PROJECT No.

1000

Bobcat Consultants, Inc.

DR. No.

1



Tension Source

Proposed Masonry Foundation Project

12345 S. La Brea Ave., Los Angeles, California

FOR: City

PROJECT No. 123456

BobTec Consultants, Inc.

Sheet No. 1

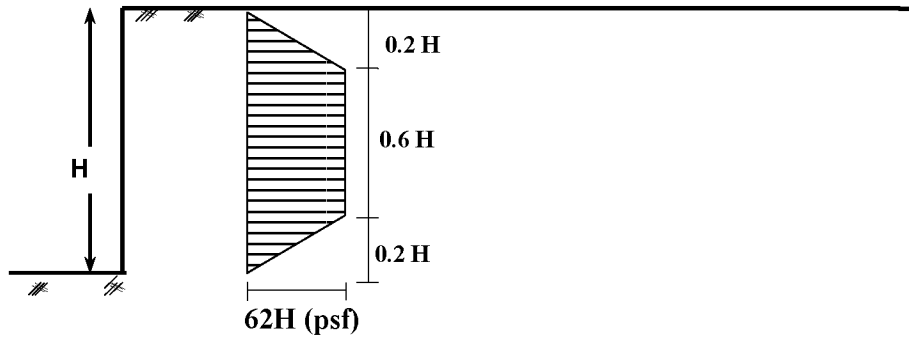
Table 1: Basement Wall Design with Hydrostatic Pressure

Wall Design Recommendations				
Retained Height & Back-slope Gradient (maximum)	Active Pressure Fluid Weight (pcf)	At-Rest Pressure Fluid Weight (pcf)	Restrained Design Earth Pressure (psf)* ¹	Seismically Induced Earth Pressure - Fluid Weight (pcf) * ²
20 (ft) & LEVEL	42	-	$62 \times H$	27

*¹ -Where H is the height of retained soil

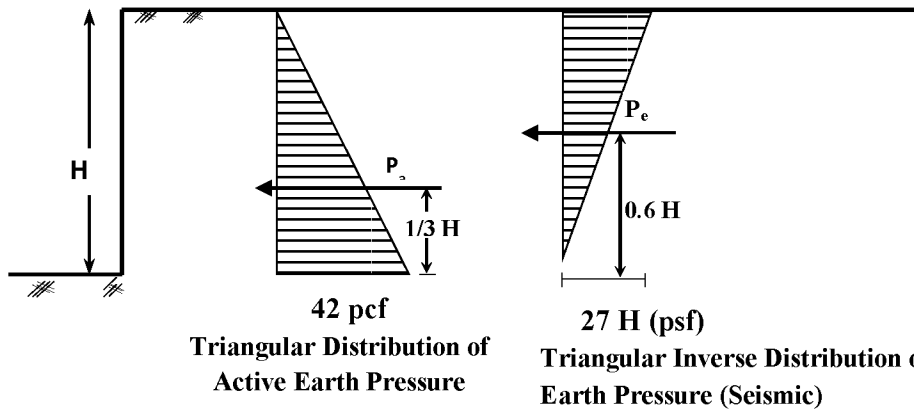
*² - The seismically induced earth pressure should be applied as an inverted triangular pressure

1. Restrained Wall Design Based on At Rest Earth Pressure plus Hydrostatic Pressure



Trapezoidal Distribution of Earth pressure

2. Cantilever Wall Design Based on Active Earth Pressure



**42 pcf
Triangular Distribution of Active Earth Pressure**

**27 H (psf)
Triangular Inverse Distribution of Earth Pressure (Seismic)**

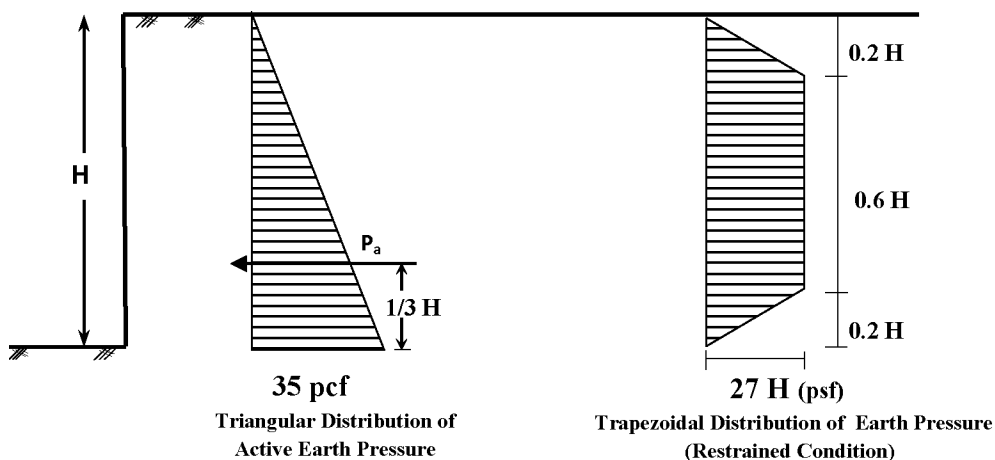
- Restrained Subterranean walls**, “walls for which horizontal movement is restricted at the top”, shall be designed for an At-Rest lateral earth pressure (equivalent fluid weight) as illustrated in the above diagram of **Trapezoidal Distribution of Earth Pressure, 62H(psf)**. Our analysis of restrained and cantilevered retaining walls indicate that load combination of seismic plus static active is lower than the at-reat forces. Therefore, no additional loading due to seismic is required for restrained walls.
- Cantilevered retaining walls** higher than 6 feet shall be designed with the addition of seismic surcharge as illustrated on the above diagrams of Triangular Distribution of Active Earth Pressure and Triangular Inverse distribution of seismic pressure.

Table 2: Shoring Design

Shoring Lateral Pressures Recommendations		
Surface Slope of Retained Material Horizontal to Vertical	Static Equivalent Fluid Weight (pcf)	Restrained Condition Design Earth Pressure (psf)*
Level up to 15 ft.	35	-
UP to 22 ft.	-	27

* -Where H is the retained height of the excavation soil

Shoring Design Based on Active Earth Pressure



Cantilevered soldier pile should be designed to resist an active earth pressure. The active earth pressure condition assumes that a triangular pressure distribution is utilized in the shoring design. If the soldier piles are not allowed to deflect, they shall be designed for the Restrained Condition. Soldier piles designed for the restrained condition should utilize a trapezoidal pressure distribution.

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure (At-Rest Condition) plus Hydrostatic Pressure
 Descript. : 639 S. La Brea Ave. Los Angeles
 Author : Behnam M. Khani
 Date : 1/4/2017

Settings


USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
 Passive earth pressure calculation : Mazindrani (Rankin)
 Earthquake analysis : Mononobe-Okabe
 Shape of earth wedge : Calculate as skew
 Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters			
Permanent design situation			
Reduction coeff. of internal friction :	$\gamma_{m\phi} =$	1.50	[-]
Reduction coeff. of cohesion :	$\gamma_{mc} =$	1.50	[-]
Reduction coeff. of Poisson's ratio :	$\gamma_{mv} =$	1.00	[-]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma} =$	1.00	[-]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma} =$	1.00	[-]

Basic soil parameters

No.	Name	Pattern	ϕ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		24.00	210.0	127.00	67.50	0.00


All soils are considered as cohesionless for at rest pressure analysis.


Soil parameters

Silty Sand

Unit weight : $\gamma = 127.0$ pcf
 Stress-state : effective
 Angle of internal friction : $\phi_{ef} = 24.00^\circ$
 Cohesion of soil : $c_{ef} = 210.0$ psf
 Angle of friction struc.-soil : $\delta = 0.00^\circ$
 Soil : cohesionless
 Saturated unit weight : $\gamma_{sat} = 130.0$ pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	30.00	Silty Sand	

No.	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Terrain profile

Terrain behind the structure is flat.

Water influence

GWT behind the structure lies at a depth of 0.00 ft

Settings of the stage of construction

Design situation : permanent

Analysis No. 1

Pressure at rest behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	ϕ_d [°]	c_d [psf]	γ [pcf]	K_r	Comment
1	22.00	0.00	16.00	140.0	67.50	0.724	

Pressure at rest distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	22.00	1485.0	1375.0	1075.7	1075.7	0.0

Water pressure distribution

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0
2	22.00	1375.0	0.0

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0
2	22.00	2450.7	0.0

Resultant forces

Total horizontal pressure acting on construction = 26957.46 lbf/ft
 Application point of horiz. comp. lies in depth = 14.67 ft
 Total vertical pressure acting on construction = 0.00 lbf/ft
 Dist. of vertical comp. from top of constr. = 0.00 ft

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure (Active Condition)
 Descript. : 639 S. La Brea Ave. Los Angeles
 Author : Behnam M. Khani
 Date : 1/4/2017

Settings


USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
 Passive earth pressure calculation : Mazindrani (Rankin)
 Earthquake analysis : Mononobe-Okabe
 Shape of earth wedge : Calculate as skew
 Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters			
Permanent design situation			
Reduction coeff. of internal friction :	$\gamma_{m\phi} =$	1.50	[-]
Reduction coeff. of cohesion :	$\gamma_{mc} =$	1.50	[-]
Reduction coeff. of Poisson's ratio :	$\gamma_{mv} =$	1.00	[-]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma} =$	1.00	[-]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma} =$	1.00	[-]

Basic soil parameters

No.	Name	Pattern	ϕ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		24.00	210.0	127.00	67.50	0.00


All soils are considered as cohesionless for at rest pressure analysis.


Soil parameters

Silty Sand

Unit weight : $\gamma = 127.0$ pcf
 Stress-state : effective
 Angle of internal friction : $\phi_{ef} = 24.00^\circ$
 Cohesion of soil : $c_{ef} = 210.0$ psf
 Angle of friction struc.-soil : $\delta = 0.00^\circ$
 Soil : cohesionless
 Saturated unit weight : $\gamma_{sat} = 130.0$ pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	30.00	Silty Sand	

No.	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Water influence

Ground water table is located below the structure.

Settings of the stage of construction

Design situation : permanent

Analysis No. 1

Active pressure behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	φ_d [°]	c_d [psf]	γ [pcf]	δ_d [°]	K_a	Comment
1	2.93	0.00	16.00	140.0	127.00	0.00	0.000	
2	19.07	0.00	16.00	140.0	127.00	0.00	0.492	

Active pressure distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	2.93	371.6	0.0	0.0	0.0	0.0
2	2.93	371.6	0.0	0.0	0.0	0.0
	22.00	2794.0	0.0	1375.6	1375.6	0.0

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0
2	2.93	0.0	0.0
3	22.00	1375.6	0.0

Resultant forces

Total horizontal pressure acting on construction = 13118.88 lbf/ft
 Application point of horiz. comp. lies in depth = 15.64 ft
 Total vertical pressure acting on construction = 0.00 lbf/ft
 Dist. of vertical comp. from top of constr. = 0.00 ft

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure (Seismic Condition)
 Descript. : 639 S. La Brea Ave. Los Angeles
 Author : Behnam M. Khani
 Date : 1/4/2017

Settings


USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
 Passive earth pressure calculation : Mazindrani (Rankin)
 Earthquake analysis : Mononobe-Okabe
 Shape of earth wedge : Calculate as skew
 Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters			
Seismic design situation			
Reduction coeff. of internal friction :	$\gamma_{m\phi} =$	1.00	[-]
Reduction coeff. of cohesion :	$\gamma_{mc} =$	1.00	[-]
Reduction coeff. of Poisson's ratio :	$\gamma_{mv} =$	1.00	[-]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma} =$	1.00	[-]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma} =$	1.00	[-]

Basic soil parameters

No.	Name	Pattern	ϕ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		24.00	210.0	127.00	67.50	0.00


All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Silty Sand

Unit weight : $\gamma = 127.0$ pcf
 Stress-state : effective
 Angle of internal friction : $\phi_{ef} = 24.00^\circ$
 Cohesion of soil : $c_{ef} = 210.0$ psf
 Angle of friction struc.-soil : $\delta = 0.00^\circ$
 Soil : cohesionless
 Saturated unit weight : $\gamma_{sat} = 130.0$ pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	30.00	Silty Sand	

No.	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Water influence

Ground water table is located below the structure.

Earthquake

Horizontal seismic coefficient $k_h = 0.2600$

Vertical seismic coefficient $k_v = 0.0000$

Coeff. to compute point of application $k.H = 0.60$

Water below the GWT is restricted.

Settings of the stage of construction

Design situation : seismic

Analysis No. 1

Active pressure behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	φ_d [°]	c_d [psf]	γ [pcf]	δ_d [°]	K_a	Comment
1	5.09	0.00	24.00	210.0	127.00	0.00	0.000	
2	16.91	0.00	24.00	210.0	127.00	0.00	0.324	

Active pressure distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	5.09	646.7	0.0	0.0	0.0	0.0
2	5.09	646.7	0.0	0.0	0.0	0.0
	22.00	2794.0	0.0	905.6	905.6	0.0

Earthquake effects (active earth pressure) - partial results

Layer No.	Thickness [ft]	φ_d [°]	β [°]	ψ [°]	K_a	K_{ae}	$K_{ae}-K_a$	Comment
1	5.09	24.00	0.00	14.57	0.422	0.652	0.230	
2	16.91	24.00	0.00	14.57	0.422	0.652	0.230	

Earthquake effects (active earth pressure)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_D [psf]	Pressure [psf]	Hor. comp. [psf]	Vertical comp. [psf]
1	0.00	0.0	2794.0	643.4	643.4	0.0
	5.09	646.7	2147.3	494.4	494.4	0.0
2	5.09	646.7	2147.3	494.4	494.4	0.0
	22.00	2794.0	0.0	0.0	0.0	0.0

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	514.7	0.0
2	5.09	425.3	0.0
3	22.00	1034.2	0.0

Resultant forces

Total horizontal pressure acting on construction = 14732.37 lbf/ft
 Application point of horiz. comp. lies in depth = 12.73 ft
 Total vertical pressure acting on construction = 0.00 lbf/ft
 Dist. of vertical comp. from top of constr. = 0.00 ft

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure Temporary Condition (Active)
 Descript. : 639 S. La Brea Ave. Los Angeles
 Author : Behnam M. Khani
 Date : 1/4/2017

Settings


USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
 Passive earth pressure calculation : Mazindrani (Rankin)
 Earthquake analysis : Mononobe-Okabe
 Shape of earth wedge : Calculate as skew
 Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters			
Transient design situation			
Reduction coeff. of internal friction :	$\gamma_{m\phi} =$	1.25	[-]
Reduction coeff. of cohesion :	$\gamma_{mc} =$	1.25	[-]
Reduction coeff. of Poisson's ratio :	$\gamma_{mv} =$	1.00	[-]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma} =$	1.00	[-]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma} =$	1.00	[-]

Basic soil parameters

No.	Name	Pattern	ϕ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		24.00	210.0	127.00	67.50	0.00


All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Silty Sand

Unit weight : $\gamma = 127.0$ pcf
 Stress-state : effective
 Angle of internal friction : $\phi_{ef} = 24.00^\circ$
 Cohesion of soil : $c_{ef} = 210.0$ psf
 Angle of friction struc.-soil : $\delta = 0.00^\circ$
 Soil : cohesionless
 Saturated unit weight : $\gamma_{sat} = 130.0$ pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	30.00	Silty Sand	

No.	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Water influence

Ground water table is located below the structure.

Settings of the stage of construction

Design situation : transient

Analysis No. 1

Active pressure behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	φ_d [°]	c_d [psf]	γ [pcf]	δ_d [°]	K_a	Comment
1	3.72	0.00	19.20	168.0	127.00	0.00	0.000	
2	18.28	0.00	19.20	168.0	127.00	0.00	0.420	

Active pressure distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	3.72	472.8	0.0	0.0	0.0	0.0
2	3.72	472.8	0.0	0.0	0.0	0.0
	22.00	2794.0	0.0	1172.3	1172.3	0.0

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0
2	3.72	0.0	0.0
3	22.00	1172.3	0.0

Resultant forces

Total horizontal pressure acting on construction = 10713.21 lbf/ft
 Application point of horiz. comp. lies in depth = 15.91 ft
 Total vertical pressure acting on construction = 0.00 lbf/ft
 Dist. of vertical comp. from top of constr. = 0.00 ft

GeoTech Services

GEOTECHNICAL INVESTIGATION
PROPOSED
MIXED-USE ELEVEN STORY BUILDING WITH NINE RESIDENTIAL FLOORS
ON TOP OF TWO LEVELS OF CREATIVE OFFICE AND RETAIL SPACE
WITH THREE LEVELS OF SUBTERRANEAN PARKING

TRACT: 5273, M.B. 55-52
LOTS: 39,40,41,42,43,44,45,46,47,48

627 – 665 S. LA BREA AVENUE
LOS ANGELES, CALIFORNIA

FOR
CGI STRATEGIES

PROJECT NO. 15-473
DECEMBER 27, 2016

GeoTech Services

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December 27, 2016

15-473

CGI

6300 Canoga Avenue, Suite 1100
Woodland Hills, CA 91367

Subject: Geotechnical Investigation Report

Proposed Mixed-Use Eleven Story Building With Nine Residential Floors
On Top of Two Levels of Creative Office and Retail Space
With Three Levels of Subterranean Parking
Tract: 5273, M B 55-52
Lot: 39,40,41,42,43,44,45,46,47,48
627-665 S. La Brea Avenue
Los Angeles, California

INTRODUCTION

Gentlemen:

This report presents the results of a geotechnical investigation for the subject project. During the course of this investigation, the engineering properties of the subsurface materials were evaluated in order to provide recommendations for design and construction of temporary excavations, foundations, grade slabs, and subsurface walls. The investigation included subsurface exploration, soil sampling, laboratory testing, engineering evaluation and analysis, consultation and preparation of this report.

The attached Appendix I, describes the method of field exploration. Appendix II describes the laboratory testing procedures.

Plate No. 1 shows the Site Location. Plate Nos. 2, 3, and 4 show the Seismic Hazard Zone Map, Historically Highest Groundwater Contour and Alluvium Condition.

The enclosed Site Plan & Cross Sections A-A', B-B', C-C' Drawing Nos. 1, 2, 3, 4, show the approximate location of the proposed building and drilled borings in relation to the site boundaries.

Figure Nos. I-1.1, I-1.2 and I-2.1, 1-2.2, and 1-3.1, 1-3.2, present summaries of the materials encountered at the location of our borings (Logs of Borings). Figure No. I-3 presents the Uniform Soil Classification System Chart; a guide to the Log of Exploratory Borings.

Figure Nos. II-1 and II-2 present the results of direct shear and consolidation tests performed on selected undisturbed soil samples.

USGS Design Maps Summary Report & Seismic Parameters Follow.

The table following the USGS Design Maps Summary Report shows all Selected Parameters, Seismic Design Values.

Table 1: Wall Design, table and diagrams are conclusions of the lateral earth pressures on restrained walls calculations and computer printout that follow.

Table 2: Shoring Design, is the result of the lateral earth pressure calculations on the shoring system.

Following is the lateral earth pressure calculations computer printout.

It should be noted that the presented recommendations in this report are based on our understanding of the depth of excavation, structural setback and assumed loading data. This office should be notified if the actual loading and excavation depths are different from those used during this investigation.

PROJECT CONSIDERATIONS

The proposed development consists of a Mixed-use eleven story Building comprised of nine residential floors on top of two levels of creative office and retail space

(the latter of which will be located entirely on the ground floor) on top of three levels of subterranean parking to depth of 32 feet below street level. Approximately 160 units and 24,000 square feet of retail.

The garage structure will be constructed of concrete block exterior walls with a rigid diaphragm (structural concrete deck) at the top.

It is anticipated that the perimeter walls of the basement garage of the proposed building would be extended to close proximity of the respective property lines. Assuming that the parking garage level will be established at some 32 feet below grade, it is anticipated that maximum height of excavation to the perimeter wall footing levels of the basement garage would on the order of 34 feet.

Due to the anticipated height of excavation and the planned extension of the line of excavation to close proximity of the respective property lines, temporary shoring will be required during the course of basement garage construction. Cantilevered soldier piles should be used as a means temporary shoring where total height of excavation does not exceed 15 feet. In areas where total height of excavation exceeds 15 feet, the soldier piles should be held back by tieback or interior bracing.

Where adequate horizontal distance beyond the planned line of basement garage excavation is available, unsupported, open excavation slopes in accordance with the recommendations of this report may be used.

Structural loading data was not available at the time of this investigation. For the purpose of this report, it is assumed that maximum concentrated loads of the interior columns will be between 500 and 800 kips, combined dead plus frequently applied live loads. Perimeter and interior wall footings of the structure are expected to exert loads of on the order of 10 kips per lineal foot.

SITE CONDITIONS

SURFACE CONDITIONS

The site of the proposed project consists of 10 lots located at 637-665 S. La Brea Avenue, Los Angeles, California. The site is rectangular in shape and, practically level and a plan area of about 36,000 square feet. (See the enclosed Site Plan)

At the time of our field investigation, the subject site was occupied by one story commercial buildings (stores) that will be demolished for the new development.

The site is bounded on the north and south by one story stucco retail store buildings and by east and west La Brea Avenue and an Alley respectively.

SUBSURFACE CONDITIONS

SOIL PROFILE

Correlation of the subsoil between the borings was considered to be good. Generally, the site, to the depths explored, was found to be underlain by surficial fill and native soils. Thickness of the surficial fill was found to be less than five feet at the location of our borings. Deeper fill, however, may be present beneath the existing building and in utility lines. The surficial fill, however, is expected to be automatically removed by the planned basement garage excavations.

The existing soil profile as depicted in the boring to the depths explored consists of sand, silty sand, clayey sand, clay, silt to clayey silt soils in a slightly moist to moist and medium dense to dense condition. For a description of the soils encountered in the exploratory boring, please refer to the Logs of Borings enclosed with this report.

GROUNDWATER

Groundwater was encountered at 19 feet in our deepest exploratory borings drilled to maximum of 62 feet below existing ground surface.

According to the map included in the "Seismic Hazard Evaluation of the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California" dated 1998 by the Department of Conservation - Division of Mines and Geology, historical highest groundwater level has been on the order of 10 feet from the ground surface. (See the enclosed Plate No.2) Groundwater level may fluctuate because of seasonal changes, injection or extraction of water, variations in temperature and other causes.

EVALUATION OF LIQUEFACTION POTENTIAL

The site is NOT located within a State of California Liquefaction Seismic Hazard Zone. During the course of our investigation, groundwater was found at depth of 19 feet in our deepest boring No. 1 drilled to depth of 62 feet. The available maps indicate that the historically highest groundwater level at the site was near 10 feet. Liquefaction studies were not performed.

TEMPORARY EXCAVATION

Temporary excavation will be required to construct the subterranean parking level of the proposed building. Where space limitation permit, unshored temporary excavation up to 8 feet could be used, where vertical excavation exceed five feet in height, the upper portion should be trimmed to 1½:1, and excavation greater than 8 feet in vertical height require shoring.

Water should not be allowed to flow over the top of the excavation in an uncontrolled manner. No surcharge should be allowed within a 45-degree line drawn from the bottom of the excavation. Excavation surfaces should be kept moist but not saturated to retard raveling and sloughing during construction.

It would be advantageous, particularly during wet season construction, to place polyethylene plastic sheeting over the slopes. This will reduce the chances of moisture changes within the soil banks and material wash into the excavation.

SEISMIC CONSIDERATIONS

All structures shall be designed per the 2014 LABC. Geologic seismic parameters per current LABC are as follows:

$$\begin{array}{l} S_S = 2.083 \text{ g} \\ S_1 = 0.784 \text{ g} \end{array}$$

$$\begin{array}{l} S_{MS} = 2.083 \text{ g} \\ S_{M1} = 1.176 \text{ g} \end{array}$$

$$\begin{array}{l} S_{DS} = 1.389 \text{ g} \\ S_{D1} = 0.784 \text{ g} \end{array}$$

A copy of the detailed USGS output is included with this report.

EVALUATION AND RECOMMENDATIONS

GENERAL

The property is suitable for the proposed construction from a geotechnical engineering standpoint. The construction plans should take into account the appropriate soils engineering features of the site. The on-site soils are Dense. Ground water was encountered in the exploratory borings at 19 feet drilled to the maximum depth of 62

feet below existing surface. The historically highest groundwater level from the Seismic Hazard Zone Report for the Hollywood 7.5-Minute Quadrangle is on the order of 10 feet.

SOLDIER PILES

Due to magnitude and the depth of excavation and the planned extension of the line of excavation to close proximity of the respective property lines, soldier piles should be used as a means of temporary shoring.

Soldier piles consist of structural steel beams encased in concrete (below the basement garage level) and slurry mix within the exposed depths of excavation.

The soldier piles should be laterally braced utilizing drilled tie-back anchors or raker braces.

The temporary shoring should be designed not only for lateral earth pressure but also against the applicable surcharges from the off-site structures.

The lateral resistance for soldier piles may be assumed to be offered by available passive pressure below the basement level. An allowable passive pressure of 300 pounds per square foot per foot of depth may be used below the basement level for soldier piles having center-to-center spacing of at least $2 \frac{1}{2}$ times the pile diameter. Maximum allowable passive pressure should be limited to 3,000 pounds per square foot. The maximum center-to-center spacing of the vertical shafts should be maintained no greater than 10 feet.

For temporary construction excavations, the active pressure on soldier piles may be computed using an equivalent fluid density of 46 pounds per cubic foot. Uniform surcharge may be computed using an active pressure coefficient of 0.25 times the uniform load.

When using soldier piles for temporary shoring, the point of fixity (for the purpose of moment calculations), may be assumed to occur at some 2 feet below the base of the excavation.

In order to limit local sloughing, it is recommended that lagging be used between the soldier piles where sand is exposed. All wood members left in ground should be pressure treated. The lagging should be designed based on 400 pounds per square foot uniform pressure.

If the construction cuts are open, they should be covered by a plastic membrane kept in place by holding blocks or driven re-bars at the top and bottom of the membrane. No equipment or personnel should stand closer than 10 feet from the top of the temporary cut. We should examine the construction cuts periodically to verify performance. All construction cuts should comply with the State of California Construction Safety Orders (CAL/OSHA)

BRACED SHORING

Where total height of excavation exceeds 15 feet, and in the areas where minor lateral movement at the top of the piles cannot be tolerated, the vertical shafts should be held back with lateral bracing system

Table 2 of this report's attachments, Shoring Design Table and the diagrams that follow represent the recommended trapezoidal lateral earth pressure distributions that should be used to calculate the lateral thrust behind restrained shoring systems at different heights (H).

It is anticipated that Rakers will be used along the north and south (existing offsite buildings and private properties) and Tiebacks along east and west, the public street and alley.

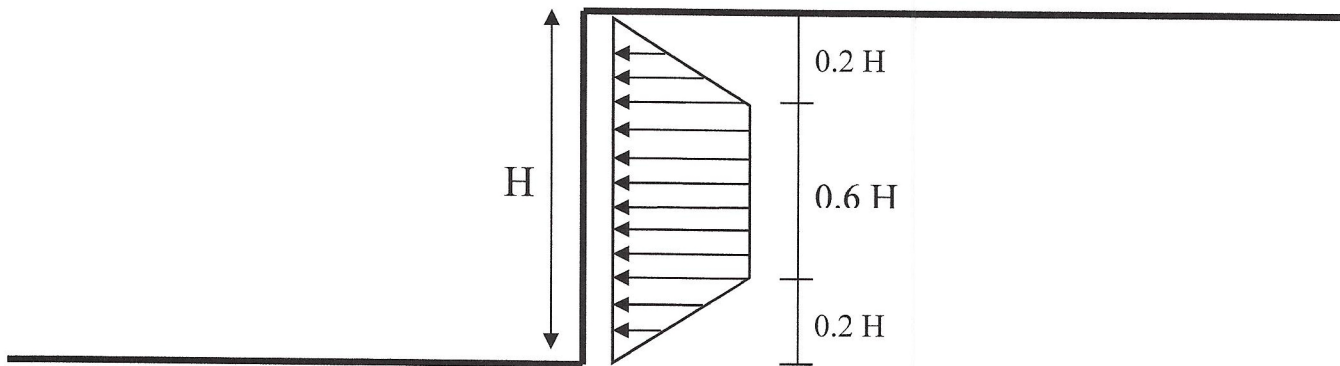
RAKERS:

If internal bracing are used against the vertical piles, the footings of the bracing should be pre-loaded to the anticipated final loads.

Pre-loading of the brace footings would take out the initial settlements, and would reduce the chances of excessive rotations occurring at the top of the vertical shafts.

For the purpose of design, the footings of the bracing that are normally inclined at 45-degree angles should be designed based on a lower allowable maximum soil pressure (2/3 of the allowable maximum bearing value recommended in this report for normal spread footings).

When internal bracing is used against the soldier piles, trapezoidal pressure distribution should be used to calculate the lateral thrust. The following sketch shows the recommended lateral earth pressure distribution behind restrained shoring system.



30H (psf) H = HEIGHT OF SHORING

In order to limit local sloughing, it is recommended that lagging be used between the soldier piles. Lagging may be designed based on a lateral pressure of 400 pounds per square foot. All wood members left in ground should be pressure treated.

The recommendations in this section are for use in design and cost estimating purposes prior to construction. The contractor is solely responsible for safety during construction. All temporary shoring should be permanently supported within 4 weeks.

TIEBACKS:

For the purpose of design, it may be assumed that the potential wedge of failure would be a plane drawn at a 55 degree angle with the horizontal through the bottom of the excavation. Only the portion of the tieback anchor shafts beyond the potential failure wedge should be considered to be effective in resisting lateral loads.

The range of friction values to be used in the lateral capacity design of the anchor shafts is based on several factors, with the upper limit being the strength of the soils. Any disturbance in the soils, such as spauling would reduce the effective friction values around the anchor shafts.

A unit friction value of 250 pounds per square foot may be used to calculate the load supporting capacities of the anchor tie backs. This assumes that the concrete will be placed using gravity. For post grouted anchors where the concrete is placed using

high pressure (between 700 to 1000 psi) a skin friction value of 1,200 pounds per square foot can be used.

Only the frictional resistance developed beyond the assumed failure plane should be used in resisting lateral loads. Structural concrete should be placed on the lower portion of the drilled shafts to the assumed failure plane. Concreting of the anchors should be done by pumping the concrete into the bottom of the shaft. The anchor shaft between the failure plane and the face of the shoring may be backfilled with sand after concrete placement. All temporary shoring should be permanently supported within 4 weeks.

FOUNDATIONS

A mat foundation is being considered for the proposed building. The mat foundation may be designed using a net allowable bearing capacity of 1,500 psf. The allowable bearing value is for total dead loads and frequently applied live loads and may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces.

1. It is recommended that a modulus of subgrade reaction of 150 pound per cubic inch (pci) be utilized for the design of mat foundation bearing on the proposed compacted fill.
2. Mat foundation should be entirely supported by structural compacted fill.
3. The minimum thickness of the mat foundation should be 24 inches. And greater thickness and reinforcement for the mat foundation should be designed by the project structural engineer.
4. Foundation excavation should be observed and approved in writing by the Geotechnical Engineer (a representative of GeoTech Services).

EXPECTED SETTLEMENTS

Under the allowable maximum soil pressure, footings carrying the assumed maximum consent rated loads of 500 kips are expected to settle on the order of one inch. Continuous footings, with loads of about 10 kips per linear foot are expected to

settle on the order of ½ of an inch. Maximum differential settlements are expected to be on the order of ¼ of an inch. Major portion of settlement are expected to occur during construction.

LATERAL DESIGN

Lateral resistance at the base of footings in contact with native soils may be assumed to be the product of the dead load forces and a coefficient of friction of 0.30. Passive pressure on the face of footings may also be used to resist lateral forces.

A passive pressure of zero at the finished grades and increasing at a rate of 300 pounds per square foot per foot of depth to a maximum value of 3,000 pounds per square foot may be used for footings poured against native soils.

BASEMENT WALLS

The perimeter walls of the basement garage of the proposed building are expected to be buried to maximum depth of about 32 feet. Static design of these walls being restrained against rotation could be based on an equivalent fluid pressure of 90 pounds per square foot per foot of depth. In addition to lateral earth pressure, the basement garage walls should also be designed for any applicable uniform surcharge loads imposed on the adjacent grounds such as streets, alley and building.

Uniform surcharge effects may be computed using a coefficient of 0.30 times the assumed uniform loads.

The above given pressure assumes no hydrostatic pressure will occur behind the basement garage walls. This will require that proper subdrain be installed behind the basement walls. Proper subdrain should be installed behind the basement garage walls. Subdrain for basement perimeter walls normally consists of weep holes with a cubic feet of gravel per each weep hole.

Where adequate space is available, granular fill should be placed and compacted behind the retaining walls (after the subdrain is installed) to a relative compaction of at least 90 percent. At least one field density tests should be taken for each 2 feet of the backfill. The degree of compaction of the wall backfill should be verified by the Soil Engineer.

Where space is limited, free-draining gravel should be placed behind the retaining walls. The gravel should then be capped with at least 18 inch thick site soils also compacted to a relative compaction of at least 90 percent. It should be noted that the backfill placed behind the basement garage walls should be made after the concrete decking is cast. All grading surrounding the building should be such to ensure that water drains freely from the site and does not pond.

Restrained undrained walls should be designed to resist a triangular pressure distribution of at-rest earth pressure and hydrostatic pressure. The hydrostatic pressure for design purpose would be 62.4 pound per cubic foot. Additional earth pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

WATERPROOFING

Retaining walls are susceptible to moisture penetration and therefore, a waterproofing system on the backside of the basement walls can be installed to provide the walls with a protection against moisture penetration.

It is recommended that retaining walls be waterproof. 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.

DRAINAGE

Adequate site drainage is absolutely essential at the site and it should be provided. Roof drainage should be connected to an appropriate drainage system and carried away from the building and to the street. Yard drainage should be kept adequate to prevent ponding of water and saturation of the soils. Water should be directed to the street in an approved manner. Future performance of the building and appurtenances will be significantly influenced by the site drainage conditions. Planters and lawns adjacent to the building should be avoided. If planters are planned adjacent to the building, they should have the bottom and walls waterproofed and a drain installed to

carry irrigation water away from the footing areas. Site drainage should be provided to divert roof and surface waters from the property through non-erodible drainage devices to the street. In no case should the surface waters be allowed to pond adjacent to building or behind the basement garage walls. A minimum slope of one and two percent are recommended for paved and unpaved areas, respectively.

TEMPORARY AND PERMANENT DEWATERING

During the course of our field investigation water was found at depth of 19 feet in our exploratory borings drilled to maximum of 62 feet.

Therefore, temporary dewatering is recommended for the construction period of this project. Due to present and historically highest groundwater being on the order of 10 feet, permanent dewatering trenches should be installed below the slab of the lowest subterranean level of the proposed building.

Subdrain, consisting of 2-foot wide trenches extended at least 18 inches below grade should be filled with free-draining gravel which are diverted by a 4 inches in diameter perforated pipe to a sump pump. As water level rises above a certain level, the pumps will become activated and pump the collected water to the curb line. Groundwater dewatering system for basement should be design by a certified hydrogeologist or other professional licensed by the state of California.

SURFACE WATER INFILTRATION

As a part of the proposed project, the surface and stormwater should be collected and taken to the curb line. On-site infiltration (in a form of "dry well"), for this project, is not recommended due to the following:

The proposed building will have a subterranean level at 32 feet below the original ground level. Therefore, the total height of excavation to the perimeter wall footing levels of the basement garage are expected to be on the order of 34 feet.

The drywell discharge should extend to minimum of 44 feet below ground to provide 10 feet of distance between the foundations and the discharge as minimum requirement.

Anyhow, water infiltration is not allowed below water table. Therefore the infiltration facilities do not meet the minimum requirements. As such, installation of the drywell can have long adverse effect on long term foundation and building performance.

SOLDIER BEAM SURVEY MONITORING (BY OWNER)

1. Soldier beam survey monitoring shall be conducted on a periodic until the permanent structure is capable of supporting the imposed lateral loads.
2. A photographic/video survey of the adjacent street and structures should be performed to establish the pre-excavation base-line conditions. Prior to any excavation, survey monitoring control points and initial soldier beam offsets shall be established to monitor the horizontal and vertical movement of the soldier beams and adjacent structures.
3. Control points, initial soldier beam offsets and monitoring performance of components of tieback anchor system for vertical and horizontal movement shall be established weekly by a licensed Surveyor under the direction and to the satisfaction of the Soil Engineer. The monitoring shall consist of readings of the vertical and lateral movement of the shoring wall.
4. Initial and periodic soldier beam readings shall be submitted to Department of Public Works, Building & Safety, General Contractor, Shoring Sub-contractor, Shoring Engineer and Soils Engineer.
5. Monitoring readings shall be submitted within 3 working days after they are conducted. Additional reading shall be obtained when requested.
6. Control points shall be established outside the areas of influence of the shoring system to ensure the accuracy of the monitoring readings.
7. If any horizontal or vertical movement of the soldier beams reaches one inch (one-half inch adjacent to existing structures), the Soils Engineer and Shoring Engineer shall evaluate such movements and recommend corrective measures, if necessary, before excavation continues.

7. If any horizontal or vertical movement of the soldier beams reaches one inch (one-half inch adjacent to existing structures), the Soils Engineer and Shoring Engineer shall evaluate such movements and recommend corrective measures, if necessary, before excavation continues.

GRADE SLABS

The slabs-on-grade thickness and reinforcement should reflect the anticipated use of the slab and should be designed by the Structural Engineer.

The mat foundation system for the proposed project should be designed to stand the uplift pressure of the water.

The floor slabs-on- grade for areas other than the basement floor should be a minimum of 5 inches thick with minimum reinforcement consisting of #4 bars spaced maximum at 16 inches each way (#4 @ maximum 16" o.c. each way) placed slightly above the slab mid-height. In areas where floor coverings or equipment that are sensitive to moisture are contemplated, a 10-mil visqueen moisture barrier should be placed beneath the slab with one inch of clean sand between the concrete slabs and the visqueen to aid in curing and to prevent puncture of the visqueen. Cracking of reinforced concrete is a relatively common occurrence. Some cracking of reinforced concrete, including slabs, can be anticipated. Irregularities in new slabs are also common. If cracking of slabs cannot be tolerated, heavily reinforced structural slabs are an option.

SITE PREPARATION

Debris from demolition and underground utility lines to be abandoned should be removed from the building area. All excavations resulting from removal of existing obstructions should be backfilled with soil compacted to at least 90 percent of the maximum density as determined by ASTM:D-1557. If any cesspools or seepage pits are encountered during shoring, they should be backfilled with vibrated gravel or slurry mix to 5 feet below finish grade. The upper 5 feet should be backfilled with soil compacted by mechanical means.

FILL PLACEMENT

Fill soils, if any, should be cleansed of deleterious debris, placed in 6 to 8 inch lifts, brought to about optimum moisture content, and compacted to at least 90 percent of the maximum density for granular soils. The placement of the fill should be performed under our observation and testing.

SITE GRADING

Site grading for the proposed project is expected to include excavation in order to create the basement garage grades and backfilling behind the basement walls and ramp areas. Prior to placing any fill, the Soil Engineer should observe the excavation bottoms. The areas to receive compacted fill should be scarified to a depth of about 8 inches, moistened as required to bring to approximately optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined by the ASTM Designation D 1557-13 Compaction Method.

General guidelines regarding site grading are presented below which may be included in the earthwork specification. It is recommended that all fill be placed under engineering observation and in accordance with the following guidelines.

1. All fill should be granular in nature. Therefore, the excavated silty sand soil from the site may be reused in the areas of compacted fill.
2. Before wall backfilling, subdrain should be installed. The subdrain system should consist of 4-inch diameter perforated pipes embedded in about 1 cubic feet of free draining gravel per foot of pipe. An approved filter fabric should then be wrapped around the free draining gravel in order to reduce the chances of siltation. Non-perforated outlet pipes should then be used to pass through the wall into an interior sump. The subdrain pipes should be laid at a minimum grade of two percent for self cleaning.
3. The excavated sandy soils from the site are considered to be satisfactory to be reused in the areas of compacted fill and wall backfill provided that rocks larger than 6 inches in diameter are removed.
4. Fill material, approved by the Soil Engineer, should be placed in controlled layers. Each layer should be compacted to at least 90 percent of the maximum unit weight as determined by ASTM designation D 1557-12 for the material used.

5. The fill material shall be placed in layers which, when compacted, shall not exceed 8 inches per layer. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to insure uniformity of material in each layer.
6. When moisture content of the fill material is too low to obtain adequate compaction, water shall be added and thoroughly dispersed until the moisture content is near optimum.
7. When the moisture content of the fill material is too high to obtain adequate compaction, the fill material shall be aerated by blading or other satisfactory methods until near optimum moisture condition is achieved.
8. Inspection and field density tests should be conducted by the Soil Engineer during grading work to assure that adequate compaction is attained. Where compaction of less than 90 percent is indicated, additional compactive effort should be made with adjustment of the moisture content or layer thickness, as necessary, until at least 90 percent compaction is obtained.

OBSERVATION DURING CONSTRUCTION

The presented recommendations in this report assume that all structural foundations will be established in dense native soils. All footing excavations should be observed by a representative of this office before reinforcing is placed.

The depths of cantilevered soldier piles should be confirmed by a representative of this office before concrete is placed. It is essential to assure that soldier piles are drilled to proper depths and diameters, and in accordance with the project plans and specifications.

Site grading work, such as wall backfilling, and subgrade preparation for basement slab support, should be conducted under observation and testing by a representative of this firm. All backfill soils should be properly compacted to at least 90 percent relative compaction. For proper scheduling, please notify this office at least 24 hours before any observation work is required.

WORKMAN SAFETY-EXCAVATIONS

It is necessary for the contractor to provide adequate shoring and safety equipment as required by the State or Federal OSHA regulations. All regulations of the State or Federal OSHA should be followed before allowing workmen in a trench or other

excavation. If excavations are to be made during the rainy season, particular care should be given to insure that berms or other devices will prevent surface water from flowing over the top of the excavations or ponding at the top of the excavations.

CLOSURE

The findings and recommendations presented in this report were based on the results of our field and laboratory investigations combined with professional engineering experience and judgment. The report was prepared in accordance with generally accepted engineering principles and practice. We make no other warranty, either express or implied.

It is noted that the conclusions and recommendations presented are based on exploration "window" borings and excavations which is in conformance with accepted engineering practice. Some variations of subsurface conditions are common between "windows" and major variations are possible.

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The following Figures and Appendices are attached and complete this report:

Appendix I-Method of Field Exploration
Appendix II-Methods of Laboratory Testing
Site Location Plate No. 1
Seismic Hazard Zone Map Plate No. 2
Historically Highest Groundwater Contour Map Plate No. 3
Seismic Hazard Map (Alluvium Condition) Plate No. 3
Site Plan – Drawing No. 1
Cross Sections-Drawing Nos. 2, 3, 4
Figure Nos. I-1.1 through I-3
Figure Nos. II-1 and II-2
Summary of Calculation
USGS Design Maps Summary Report – Seismic Parameters
Table of Seismic Design Values
Table 1: Wall Design
Table 2: Shoring Design
Computer printout of the lateral earth pressure calculations (8 pages)

Respectfully Submitted,

GeoTech Services

VahikHacopian


AraJitechian
Civil Engineer
CE 54893



APPENDIX I

METHOD OF FIELD EXPLORATION

Subsurface conditions were explored by drilling three (3) exploratory borings at the locations shown on the Site Plan. The boring were drilled to a maximum depth of 62 feet below existing grade using a truck mounted 8-inch diameter hollow stem flight auger.

The drilling of the borings was supervised by our field engineer who logged the materials brought up from the borings. Undisturbed and bulk samples were collected at depths appropriate to the investigation. The undisturbed sampler utilized in our investigation included our 2.50 inch I.D. drive barrel lined with 1 inch brass rings. The sampler used in the exploratory borings was driven to a depth of 12 inches with a 140-pound hammer falling through a height of 30 inches. The number of blows to drive the sampler 12 inches is shown on the attached Logs of Borings.

APPENDIX II

LABORATORY TESTING PROCEDURES

Moisture Density

The moisture-density information provides a summary of soil consistency for each stratum and can also provide a correlation between soils found on this site and other nearby sites. The dry unit weight and field moisture content were determined for each undisturbed sample, and the results are shown on the log of exploratory borings.

Shear Tests

Shear tests were made with a direct shear machine at a constant rate of strain. The machine is designed to test the soil without completely removing the samples from the brass rings. A range of normal stresses were applied vertically, and the shear strength was progressively determined at each load in order to determine the internal angle of friction and the cohesion. The results of direct shear tests are presented on Figure No. II-1 within this Appendix.

Consolidation

The apparatus used for the consolidation tests is designed to receive the undisturbed brass ring of soil as it comes from the field. Loads were applied to the test specimen in several increments, and the resulting deformations were recorded at selected time intervals. Porous stones were placed in contact with the top and bottom of the specimen to permit the ready addition or release of water.

Undisturbed specimens were tested at the field and added water conditions. The test results are shown on Figure No. II-2.



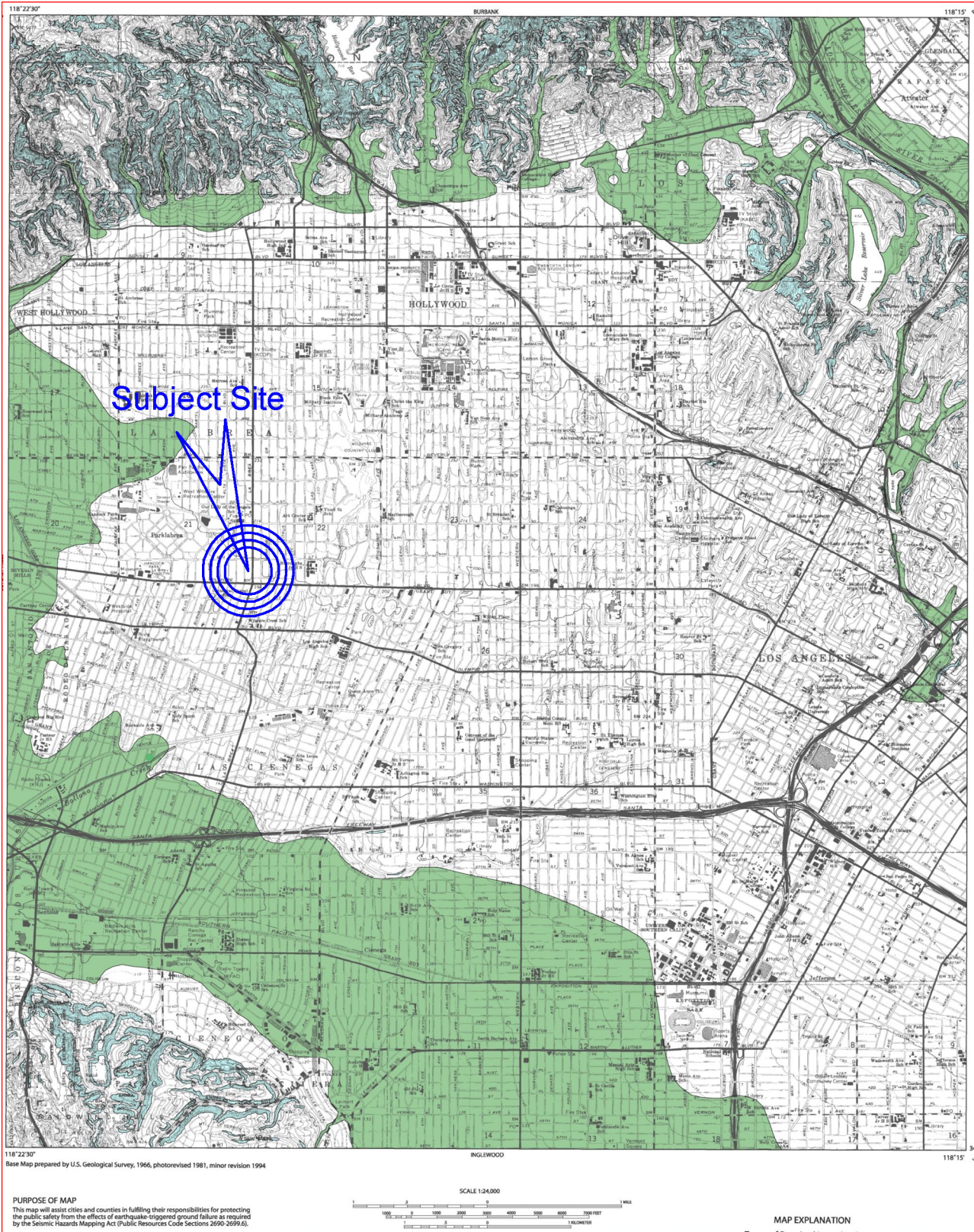
SITE LOCATION

JOB NAME : 639 S La Brea Ave Los Angeles, California

JOB No. 15-473

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PLATE No. 1



SEISMIC HAZARD ZONES MAP

JOB NAME : 639 S La Brea Ave. Los Angeles, California

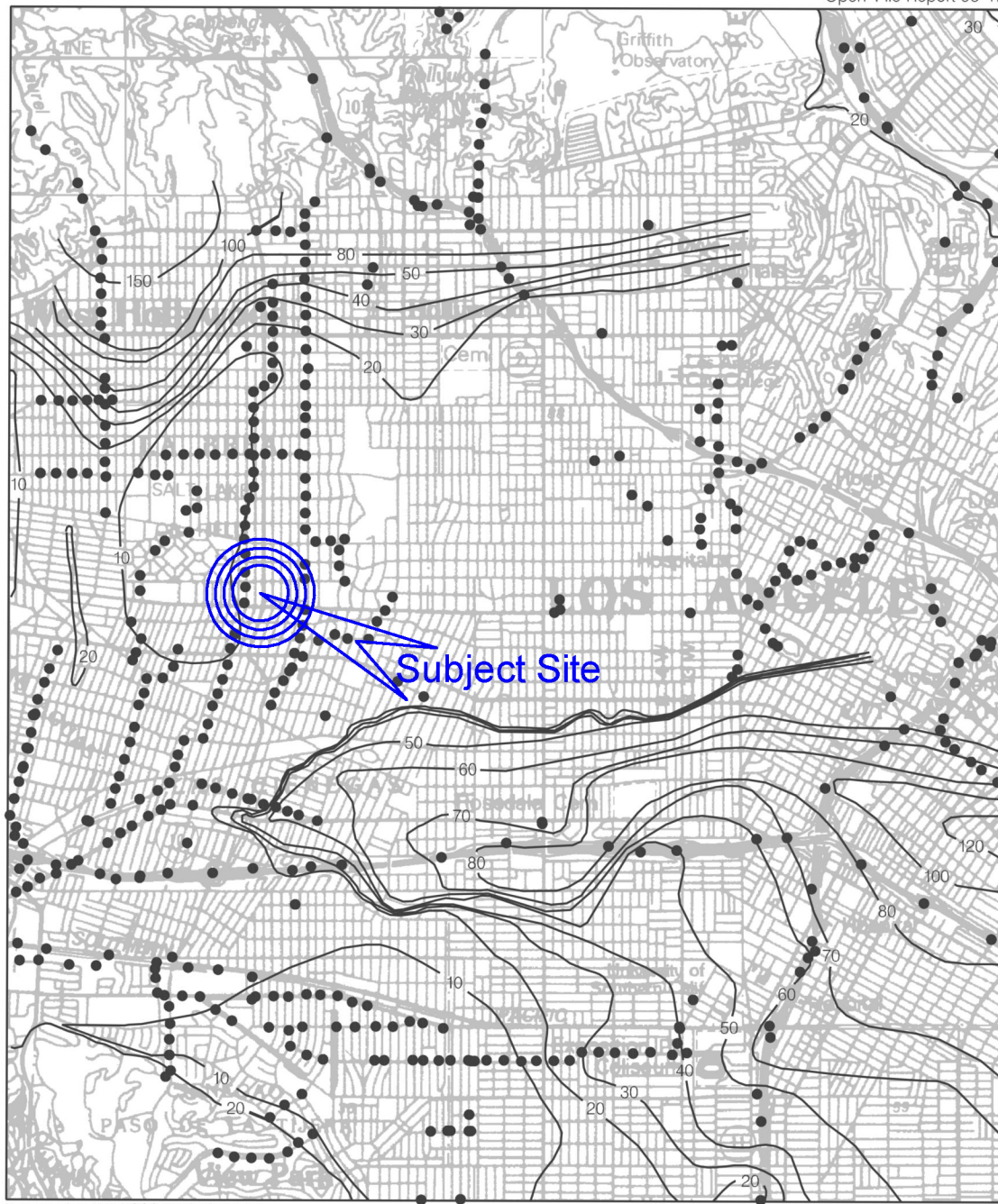
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PLATE No.

2



Base map enlarged from U.S.G.S. 30 x 60-minute series

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

Borehole Site

 — 30 — Depth to ground water in feet

ONE MILE
 SCALE

HISTORICALLY HIGHEST GROUND WATER CONTURS

JOB NAME : 639 S La Brea Ave. Los Angeles, California

JOB No.

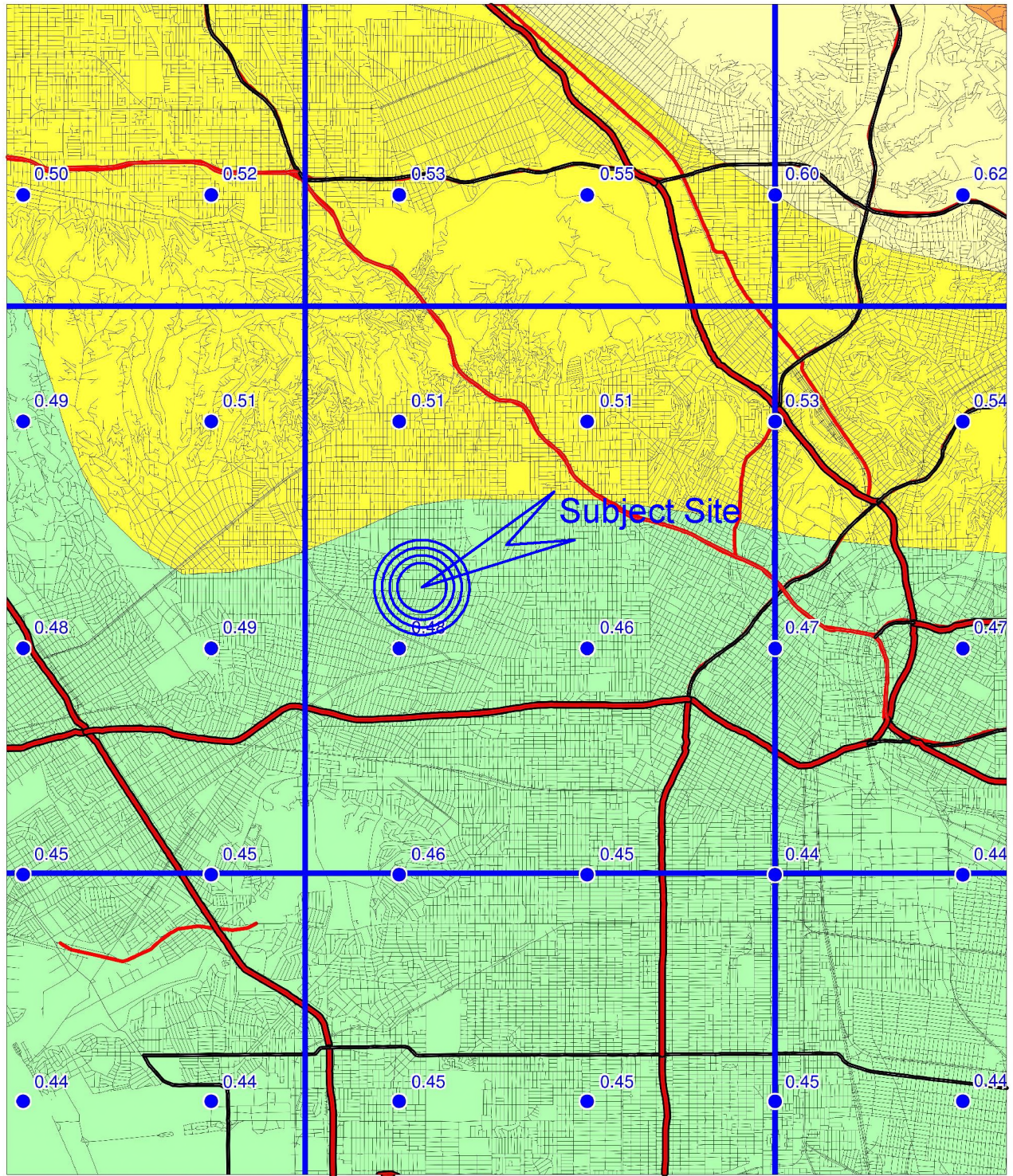
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PLATE No.

3

ALLUVIUM CONDITIONS



Base map modified from MapInfo Street Works ©1998 MapInfo Corporation

Department of Conservation
Division of Mines and Geology



Figure 3.3

ALLUVIUM CONDITION

JOB NAME : 639 S La Brea Ave. Los Angeles, California

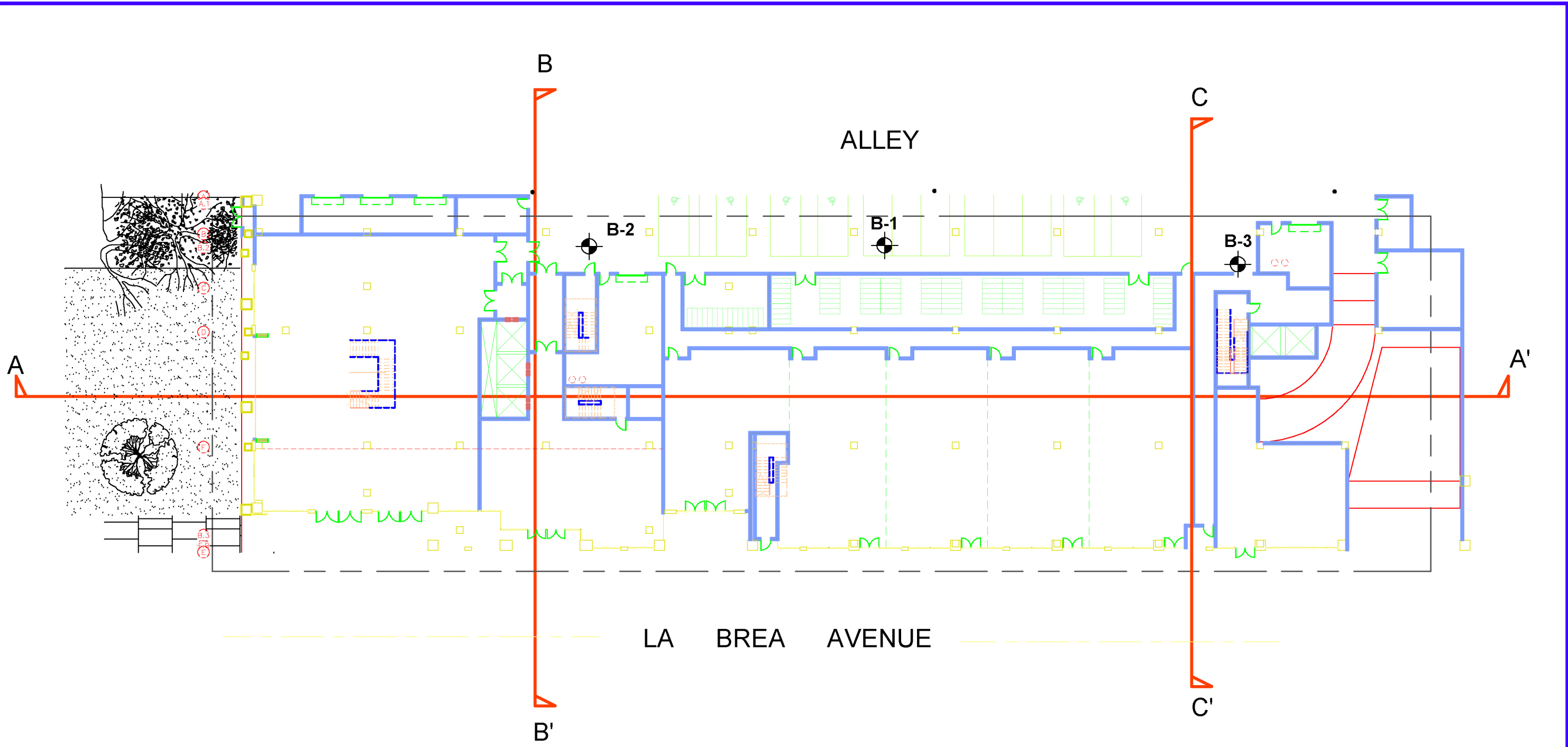
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15-473

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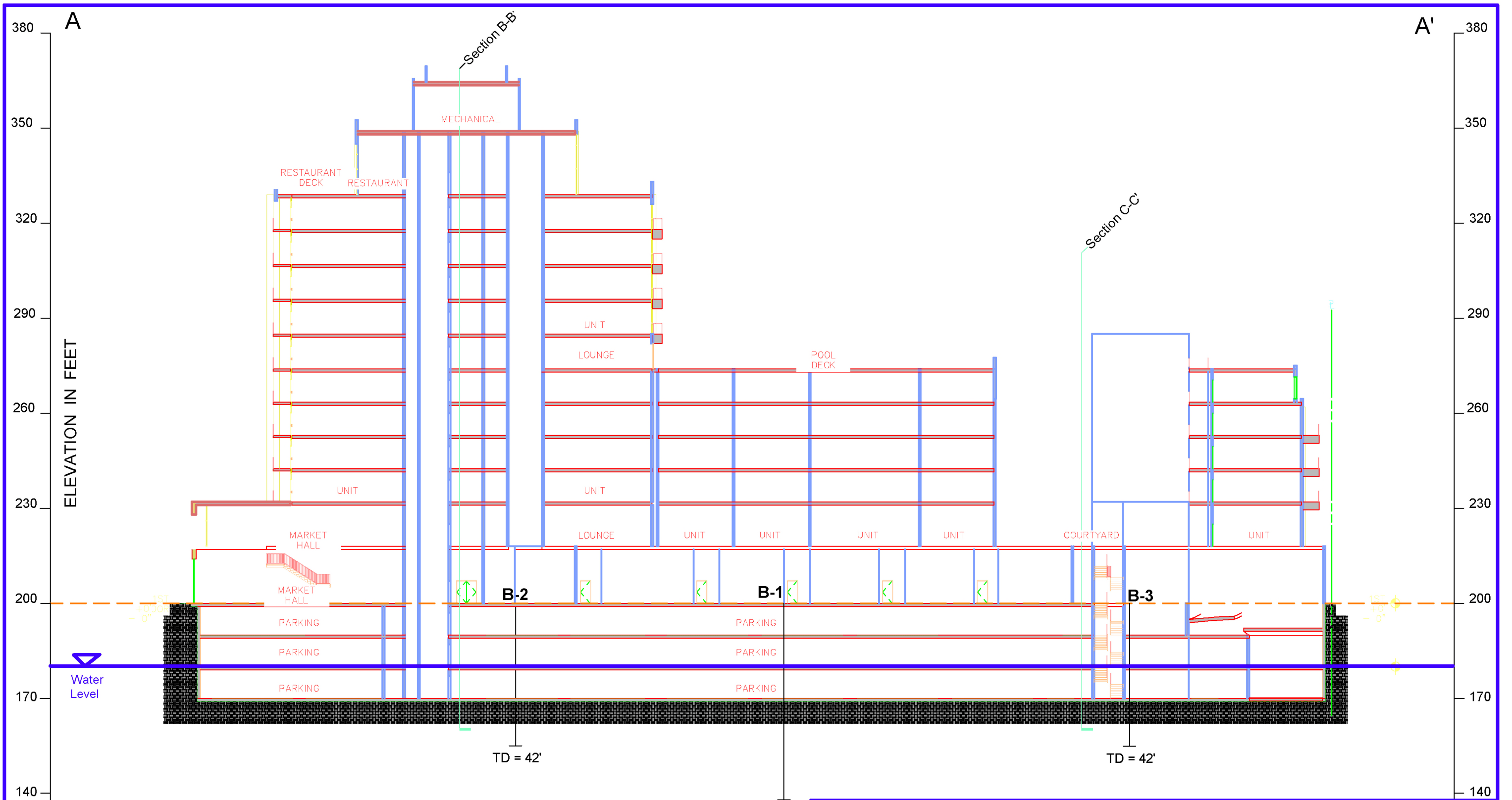
PLATE No.

4



Scale: 1" = 30'

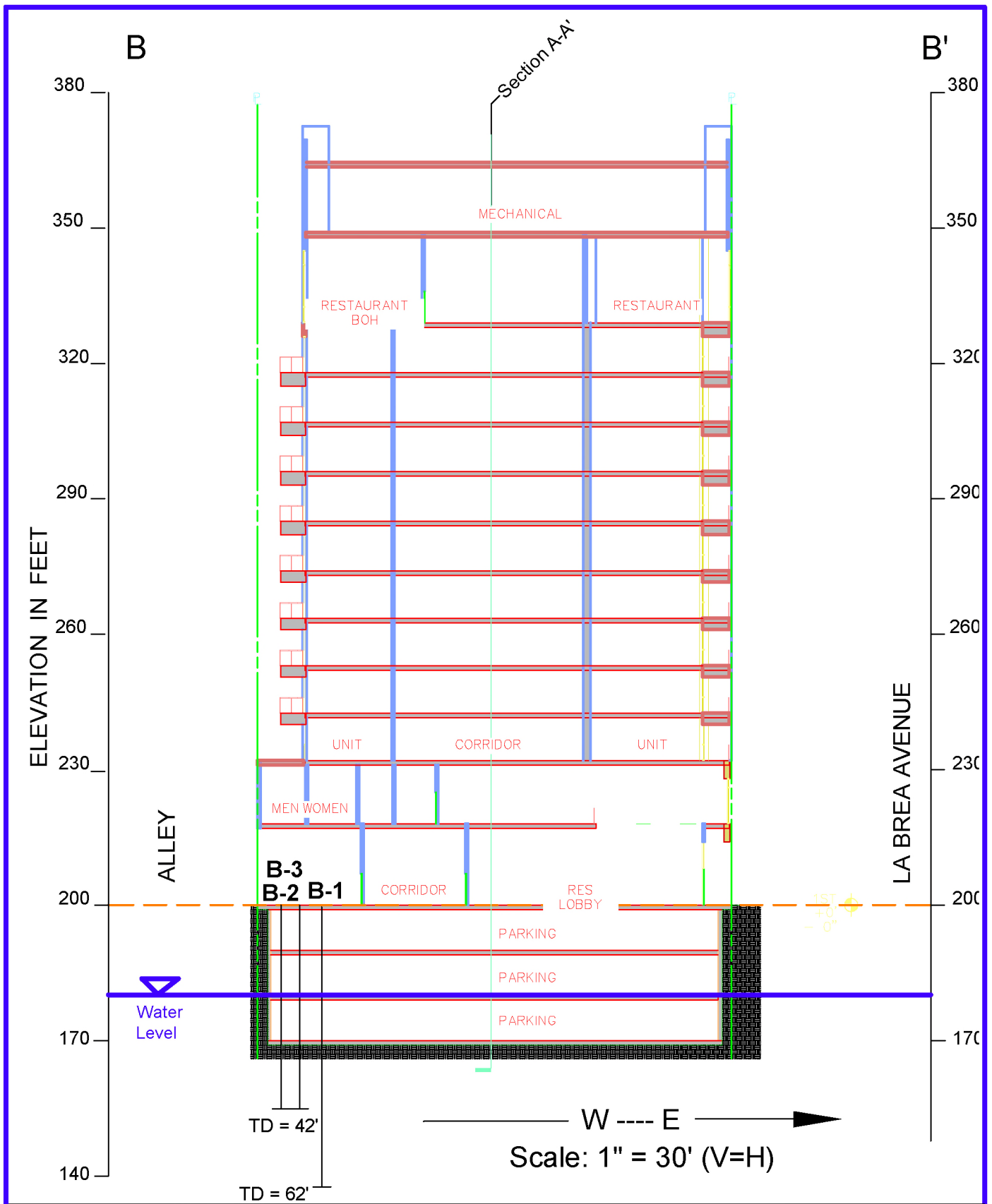
SITE PLAN		
Proposed Mix Use Building Project		637-665 S. La Brea Avenue, Los Angeles, California
FOR. CGI	DATE December 2016	PROJECT No. 15 - 473
GeoTech Services		DRAWING No. 1



S ---- N →

Scale: 1" = 30' (V=H)

CROSS SECTION A-A'		
Proposed Mix Use Building Project		637-665 S. La Brea Avenue, Los Angeles, California
FOR. CGI	DATE December 2016	PROJECT No. 15 - 473
GeoTech Services		DRAWING No. 2



Proposed Mix Use Building Project

637-665 S. La Brea Avenue, Los Angeles, California

FOR. CGI

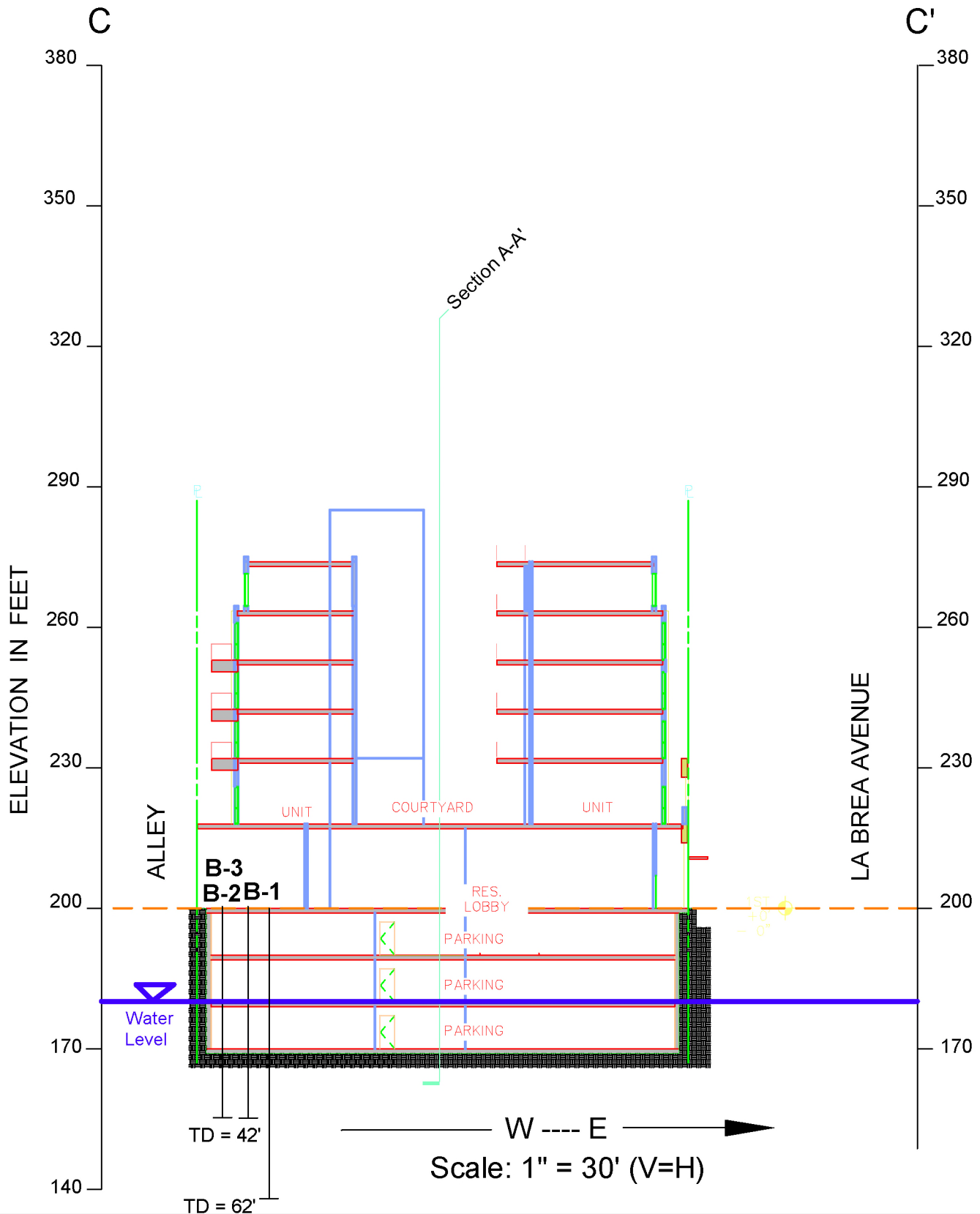
DATE December 2016

PROJECT No. 15 - 473

GeoTech Services

CROSS SEC'N B-B'

DRAWING No. 3



Proposed Mix Use Building Project

637-665 S. La Brea Avenue, Los Angeles, California

FOR.

CGI

DATE

December 2016

PROJECT No.

15 - 473

GeoTech Services

CROSS SEC'N C-C'


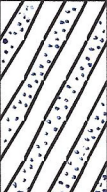
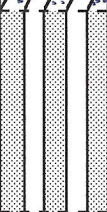
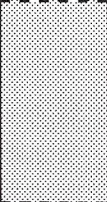
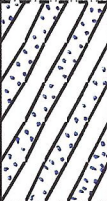
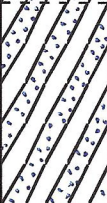
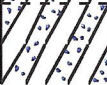
DRAWING No.

4

BORING No. 1

DATE EXCAVATED: 11/30/2015

LOGGED BY: JOSEPH

DEPTH IN FEET	DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% PASSING #200	BLOWS PER FOOT	MATERIAL TYPE	MATERIAL SYMBOL	MATERIAL	DESCRIPTION
3					CLAY (CL)		Fill:	Sandy Clay, Stiff, Moist, Dark Brown
5	104	22		27	SAND (SC)			Silty-Clayey Sand (fine to medium grained), Dense, Moist, Brown
10	108	18		23	SAND (SM)			Silty Sand, (fine grained), medium Dense, Moist, Brown
15	101	19		17	SAND (SP)			Fine to medium Sand, medium Dense, Very Moist, Brown
20	102	21		15	SAND (SC-SM)			Clayey-Silty Sand, medium Dense, Wet, Olive Brown
25	103	23		10	SAND (SC)			Clayey Sand, Medium, Fine to Medium, Wet, Olive Gray
30	104	28		30	SAND (SC)			Grades to Dense, Light Brown

LOG OF BORING

JOB NAME: 639 S La Brea Ave. Los Angeles CA.

JOB No. 15-473

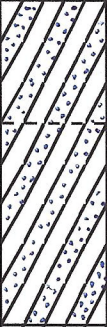





GeoTech Services

FIGURE NO : I-1.1

BORING No. 1 (continued)

DATE EXCAVATED: 11/30/2015

LOGGED BY: JOSEPH

DEPTH IN FEET	DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% PASSING #200	BLOWS PER FOOT	MATERIAL TYPE	MATERIAL SYMBOL	MATERIAL DESCRIPTION
Continued from previous page							
35	110	21		61	SAND (SC)		Grade to Very Dense
40	96	27		52	CLAY (CL)		Sandy Clay, Hard, Wet, Light Brown
45	109	21		42	SAND (SM)		Silty Sand, Dense, Fine to Medium, Wet, Light Brown, Trace of Clay.
50	97	29		30	SILT (ML)		Sandy Silt, Very Stiff, Wet, Greenish Gray, Trace of Clay.
55	104	24		54	SAND (SM)		Silty Sand, Very Dense, Fine to Medium, Wet, Greenish Gray, Trace of Clay
60	104	25		82			No Change
End of Boring @ 62 feet, Water @ 19 feet No Caving							

LOG OF BORING

JOB NAME: 639 S La Brea Ave. Los Angeles CA.

JOB No. 15-473

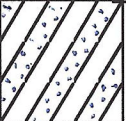
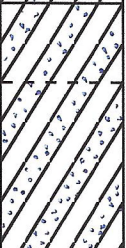
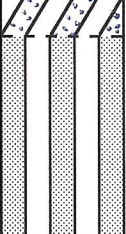
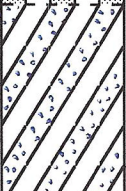

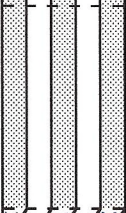
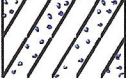
GeoTech Services

FIGURE NO : I-1.2

BORING No. 2

DATE EXCAVATED: 11/30/2015

LOGGED BY: JOSEPH

DEPTH IN FEET	DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% PASSING #200	BLOWS PER FOOT	MATERIAL TYPE	MATERIAL SYMBOL	MATERIAL DESCRIPTION
3					SAND (SC)		<p>↓ Fill: Clayey Sand, Medium, Fine to Medium, Moist, Very Dark Brown</p>
5	115	8		23	SAND (SC)		Clayey Sand, Medium, Fine to Medium, Moist, Dark Brown
10	116	17		50/6"	SAND (SM)		Silty Sand with slightly Clay, Very Dense, Moist, Light Brown
15	106	23		37	SAND (SC)		Clayey Sand, Dense, Moist, Light Brown
20	110	21		29	SAND (SM)		Silty-Clayey Sand, Medium, Wet, Brown
25	101	27		20	SAND (SM)		
30	105	29		17	SAND (SC)		Clayey Sand, Medium, Fine to Medium, Wet, Light Brown

LOG OF BORING

JOB NAME: 639 S La Brea Ave. Los Angeles CA.

JOB No. 15-473

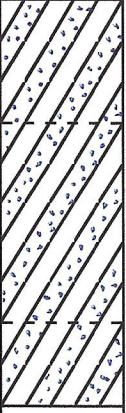

GeoTech Services

FIGURE NO : I-2.1

BORING No. 2 (continued)

DATE EXCAVATED: 11/30/2015

LOGGED BY: JOSEPH

DEPTH IN FEET	DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% PASSING #200	BLOWS PER FOOT	MATERIAL TYPE	MATERIAL SYMBOL	MATERIAL DESCRIPTION
<div style="text-align: center;">35</div> <div style="text-align: center;">40</div>	111	22		36	SAND (SC)		<p>Continued from previous page</p> <p>Clayey Sand, Dense, Fine to Medium, Wet, Brown</p> <p>Grade to Very Dense</p> <p>End of Boring @ 42 feet, Water @ 19 feet No Caving</p>
	108	25		55	SAND (SC)		

LOG OF BORING

JOB NAME: 639 S La Brea Ave. Los Angeles CA.	JOB No. 15-473
GeoTech Services	FIGURE NO : I-2.2

BORING No. 3

DATE EXCAVATED: 12/05/2016

LOGGED BY: JOSEPH

DEPTH IN FEET	DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% PASSING #200	BLOWS PER FOOT	MATERIAL TYPE	MATERIAL SYMBOL	MATERIAL DESCRIPTION
3					SILT (ML)		<p>↓ Fill: Sandy-Clayey Silt, Compacted, Moist, Dark Brown</p>
5	111	17		26	SILT (ML)		Sandy Silt with slightly Clay, Stiff, Moist, Light Brown
10	107	19		18	SAND (SM)		Silty Sand, medium Dense, Moist, Light Brown
15	102	19		11	SILT (ML)		Sandy-Clayey Silt, Soft, Moist, Light Brown
20	108	20		17	SAND (SC-SM)		Silty-Clayey Sand, medium Dense, Wet, Brown
25	105	18		28	SAND (SM)		Silty Sand (fine to medium grained) slightly Clayey, Medium Dense, Wet, Very Light Brown
30	116	17		34	SAND (SM)		Grades to Dense

LOG OF BORING

JOB NAME: 639 S. La Brea Ave., Los Angeles, CA

JOB No. 15-473

GeoTech Services

FIGURE NO : I-3.1

MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAME	
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW Well graded gravels, gravel - sand mixtures, little or no fines.	
		GRAVELS WITH FINES (Appreciable amt. of fines)	GP Poorly graded gravels or gravel-sand mixtures, little or no fines.	
			GM Silty gravels, gravel-sand-silt mixtures.	
			GC Clayey gravels, gravel-sand-clay mixtures.	
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW Well graded sands, gravelly sands, little or no fines.	
		SANDS WITH FINES (Appreciable amt. of fines)	SP Poorly graded sands or gravelly sands, little or no fines.	
			SM Silty sands, sand-silt mixtures.	
		FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)	ML Organic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
				CL Organic clay of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
				OL Organic silts and organic silty clays of low plasticity.
SILTS AND CLAYS (Liquid limit GREATER than 50)	MH Organic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.			
	CH Organic clays of high plasticity, fat clays.			
	OH Organic clays of medium to high plasticity, organic silts.			
HIGHLY ORGANIC SOILS		Pt Peat and other highly organic soils.		

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		
	NO. 200	NO. 40	NO. 10	NO. 4	3/4 in.	3 in.	(12 in.)

U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

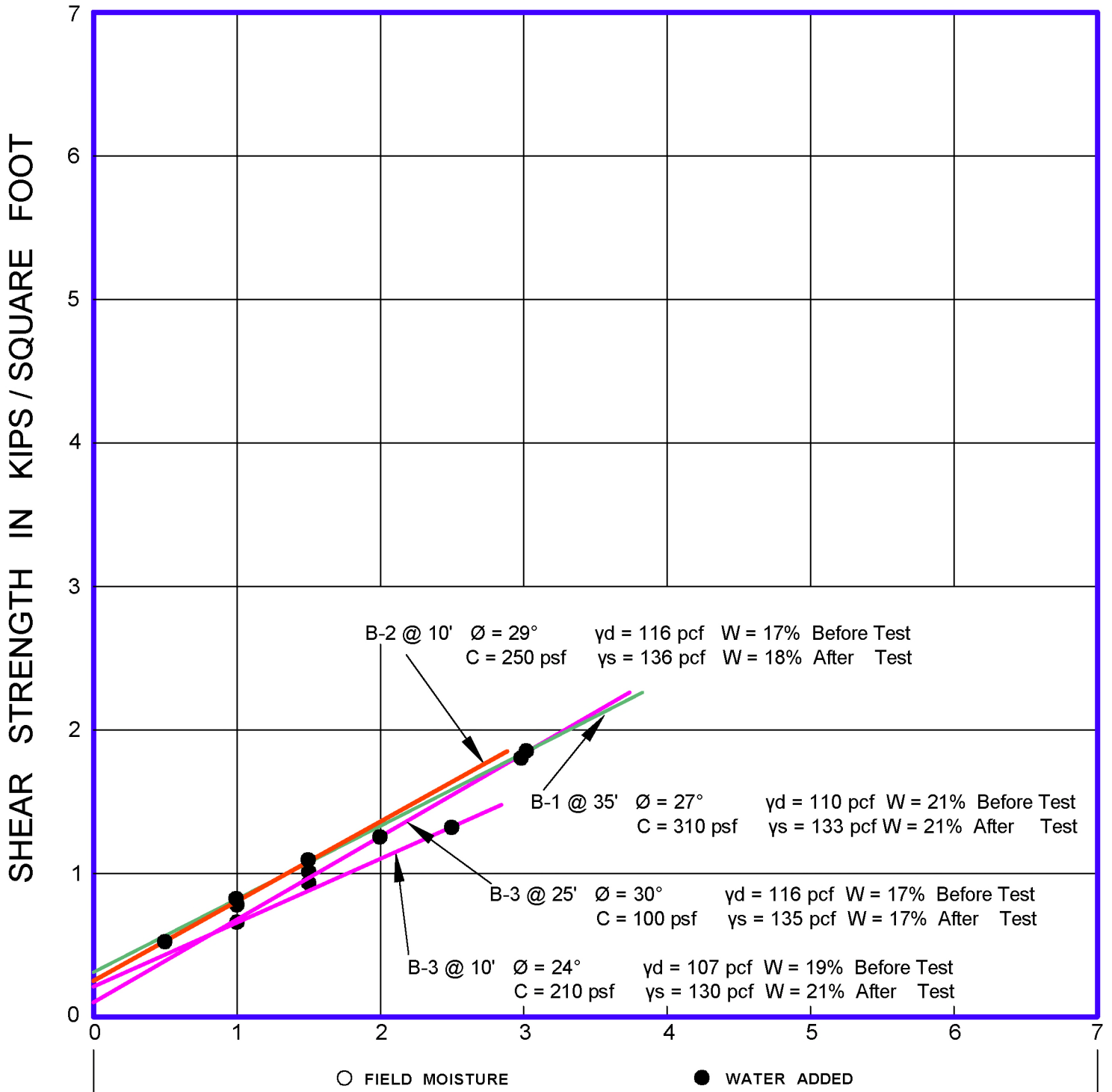
JOB NAME : 639 S La Brea Ave. Los Angeles, California

JOB No. 15-473

Geo Tech Services

FIGURE No. I-3

NORMAL STRESS IN KIPS / SQUARE FOOT



DIRECT SHEAR TESTS

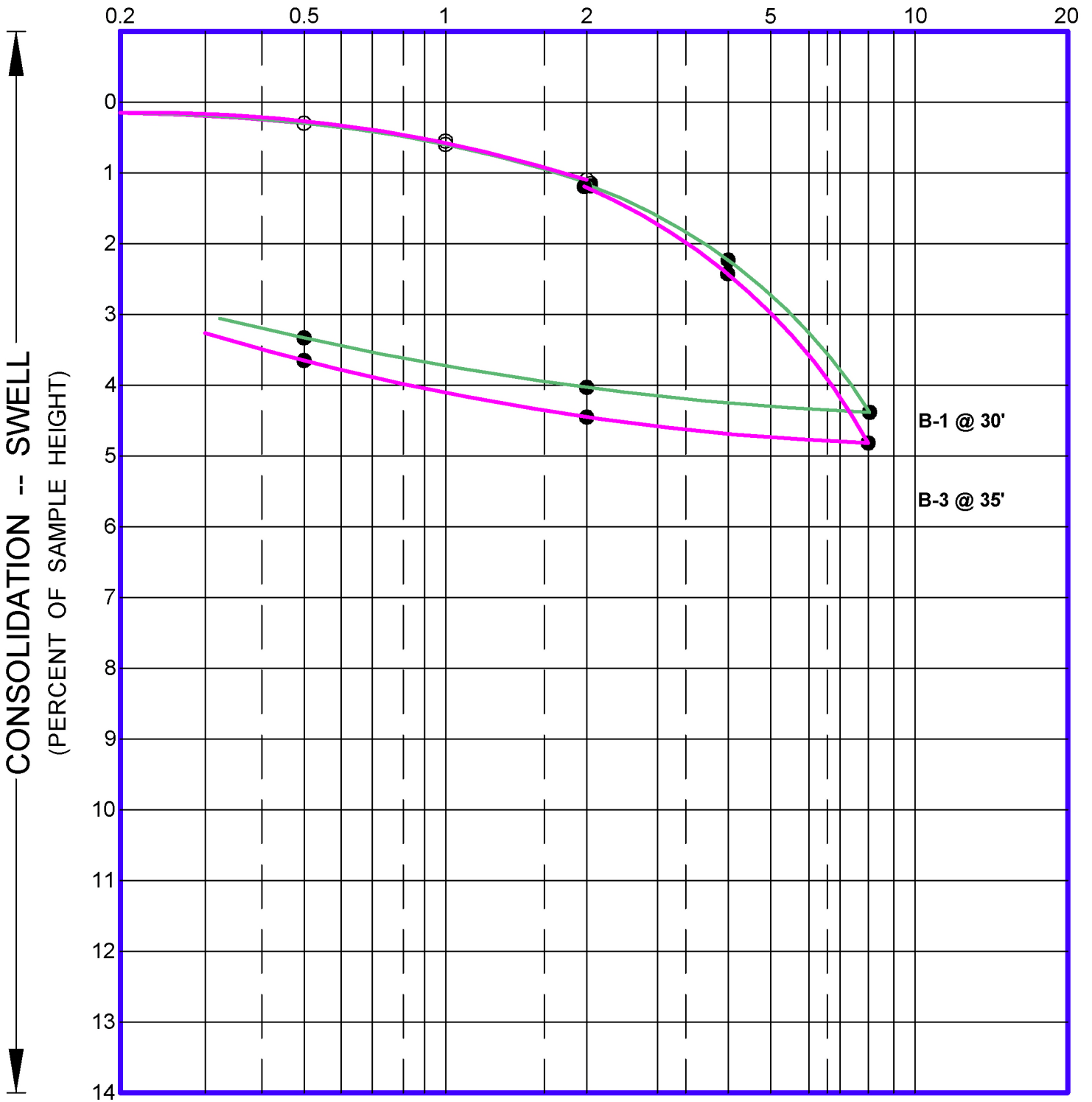
JOB NAME 639 S La Brea Ave. Los Angeles, California

JOB No. 15-473

Geo Tech Services

FIGURE No. II - 1

PRESSURE IN KIPS PER SQUARE FOOT



○ FIELD MOISTURE

● WATER ADDED

SWELL - CONSOLIDATION TESTS

JOB NAME 639 S La Brea Ave. Los Angeles, California

JOB No. 15-473

Geo Tech Services

FIGURE No. II - 2

USGS Design Maps Summary Report

User-Specified Input

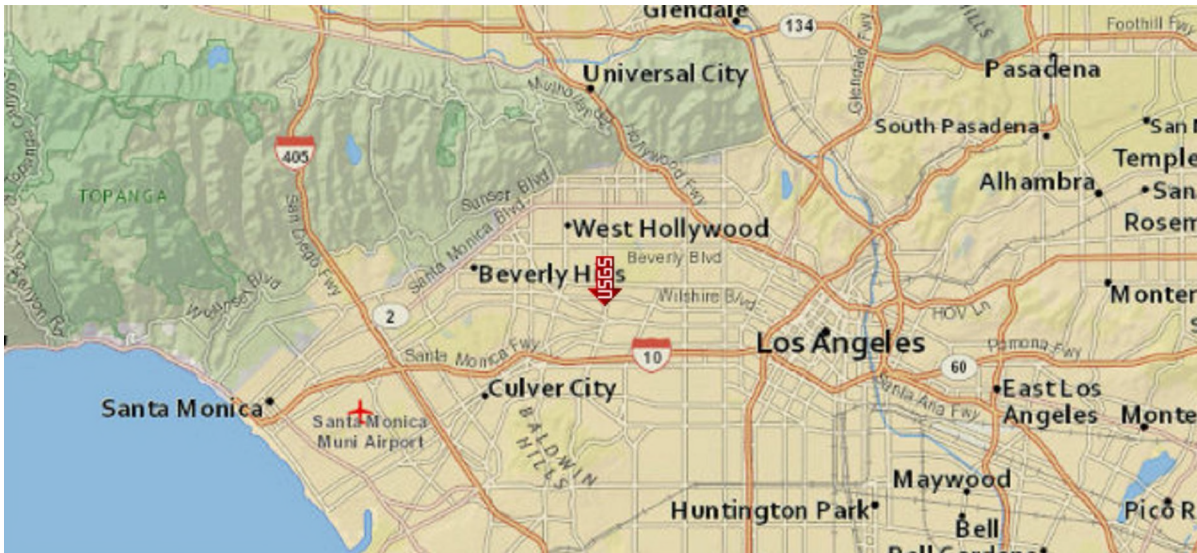
Report Title GeoTech Services Job No. 15-473 (639 S. La Brea, CA)
Wed January 4, 2017 18:55:36 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 34.06369°N, 118.34439°W

Site Soil Classification Site Class D – “Stiff Soil”

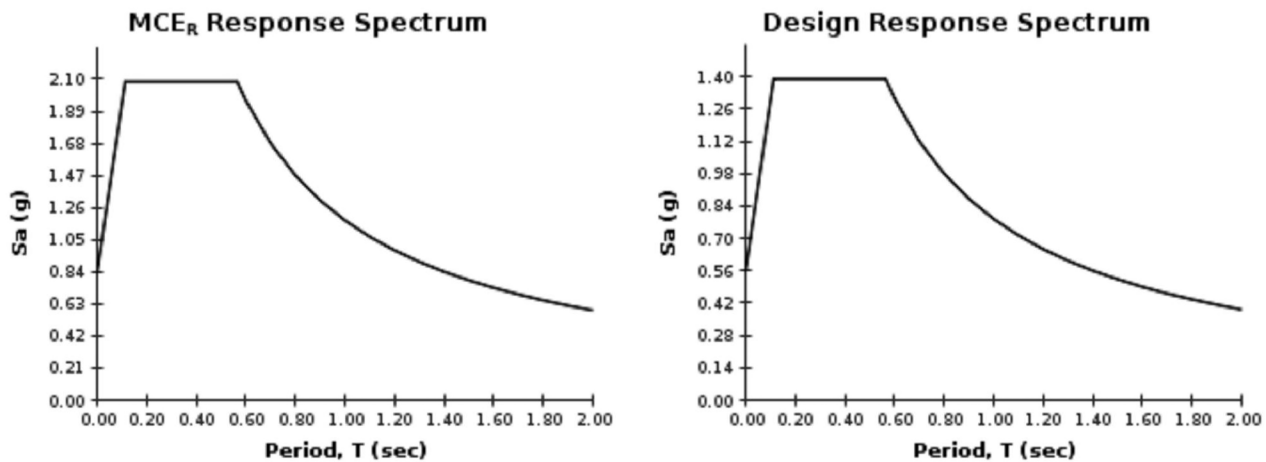
Risk Category I/II/III



USGS-Provided Output

$S_s = 2.083 \text{ g}$	$S_{MS} = 2.083 \text{ g}$	$S_{DS} = 1.389 \text{ g}$
$S_1 = 0.784 \text{ g}$	$S_{M1} = 1.176 \text{ g}$	$S_{D1} = 0.784 \text{ g}$

For information on how the S_s and S_1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the “2009 NEHRP” building code reference document.

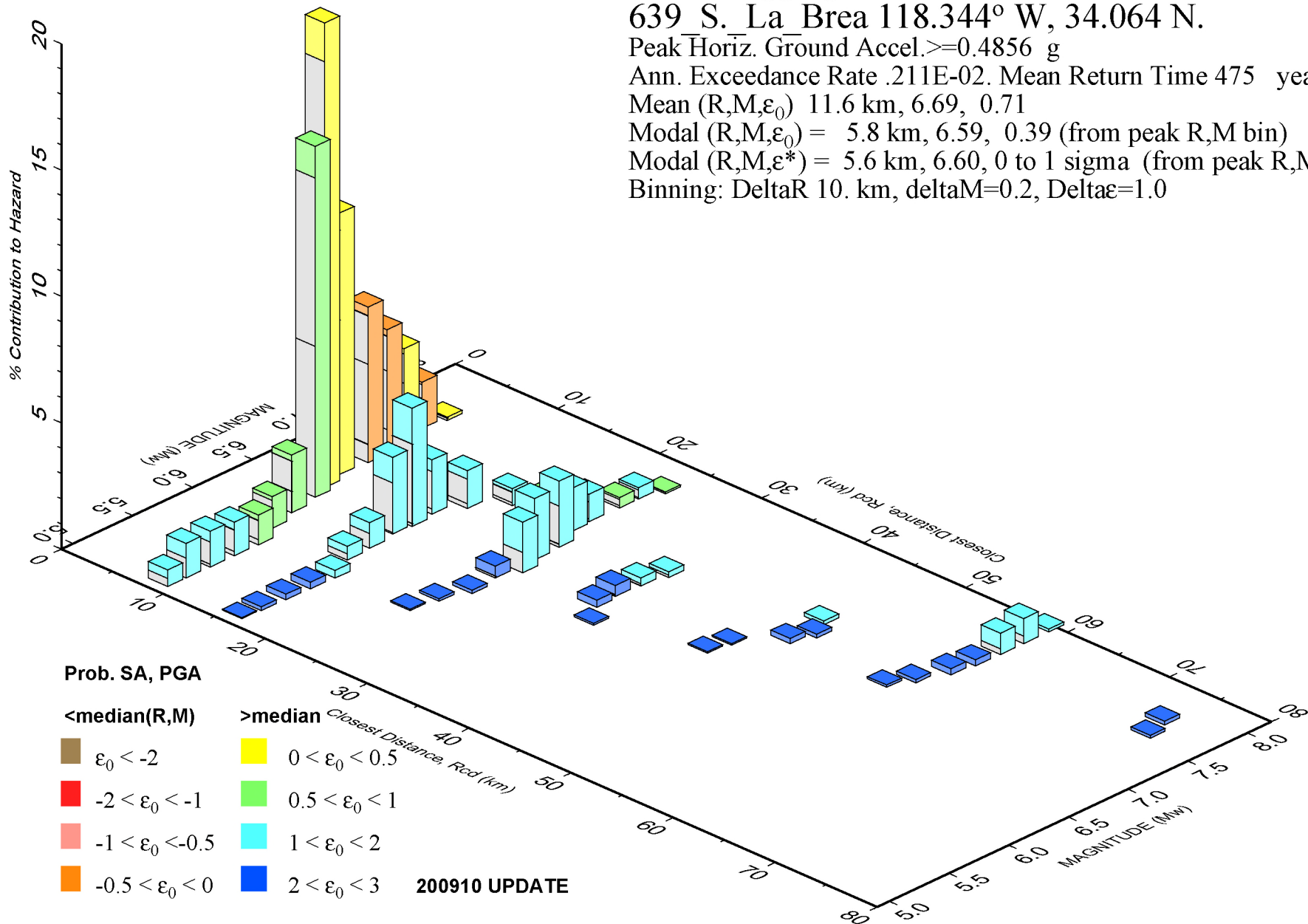


For PGA_M , T_L , C_{RS} , and C_{R1} values, please [view the detailed report](#).

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

PSH Deaggregation on NEHRP D soil
 639 S. La Brea 118.344° W, 34.064 N.

Peak Horiz. Ground Accel. ≥ 0.4856 g
 Ann. Exceedance Rate .211E-02. Mean Return Time 475 years
 Mean (R,M, ϵ_0) 11.6 km, 6.69, 0.71
 Modal (R,M, ϵ_0) = 5.8 km, 6.59, 0.39 (from peak R,M bin)
 Modal (R,M, ϵ^*) = 5.6 km, 6.60, 0 to 1 sigma (from peak R,M, ϵ bin)
 Binning: DeltaR 10. km, deltaM=0.2, Delta ϵ =1.0



Prob. SA, PGA

<median(R,M)

>median

$\epsilon_0 < -2$

$0 < \epsilon_0 < 0.5$

$-2 < \epsilon_0 < -1$

$0.5 < \epsilon_0 < 1$

$-1 < \epsilon_0 < -0.5$

$1 < \epsilon_0 < 2$

$-0.5 < \epsilon_0 < 0$

$2 < \epsilon_0 < 3$

200910 UPDATE

PSH Deaggregation on NEHRP D soil
 639 S. La Brea 118.344° W, 34.064 N.

Peak Horiz. Ground Accel. ≥ 0.8188 g
 Ann. Exceedance Rate .401E-03. Mean Return Time 2475 years
 Mean (R,M, ϵ_0) 7.2 km, 6.73, 1.26
 Modal (R,M, ϵ_0) = 5.5 km, 6.59, 1.22 (from peak R,M bin)
 Modal (R,M, ϵ^*) = 5.5 km, 6.59, 1 to 2 sigma (from peak R,M, ϵ bin)
 Binning: DeltaR 10. km, deltaM=0.2, Delta ϵ =1.0

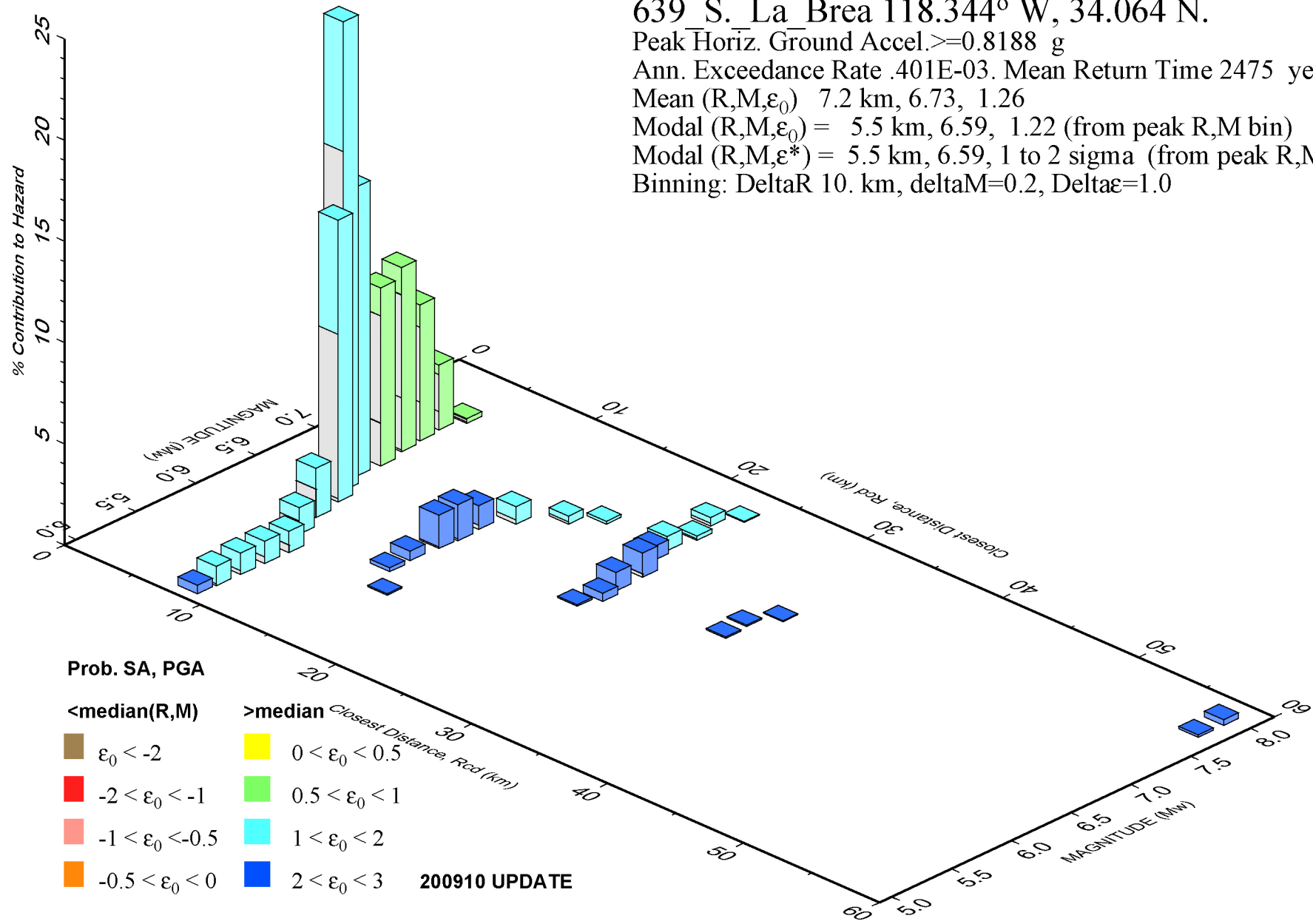
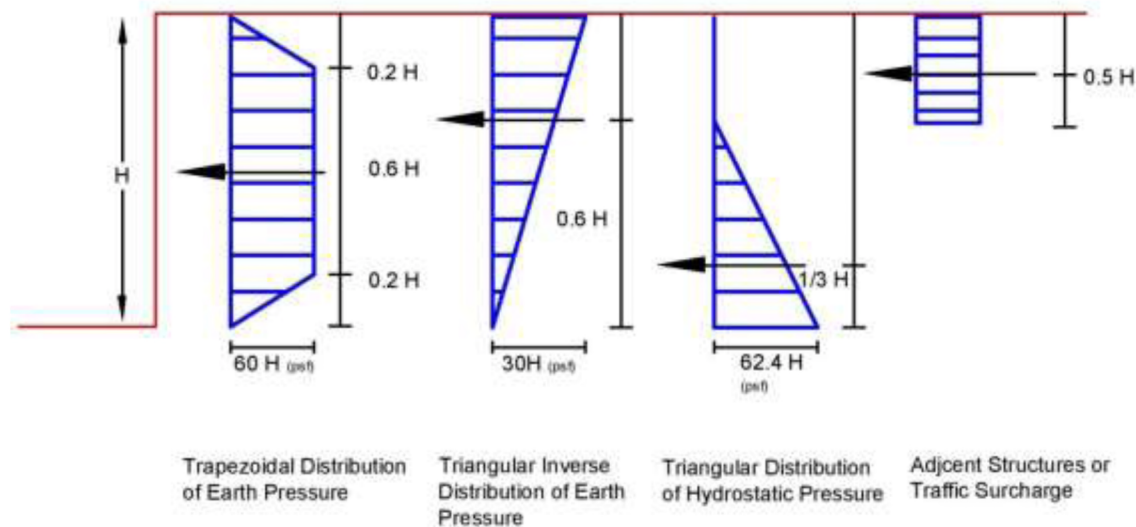


Table 1: Wall Design

Wall Design Recommendations				
Retained Height & Back-slope Gradient (maximum)	Active Pressure Fluid Weight (pcf)	At-Rest Pressure Fluid Weight (pcf)	Restrained Design Earth Pressure (psf)* ¹	Seismically Induced Earth Pressure - Fluid Weight (pcf) * ²
(ft) & LEVEL	-	90	55× H	27

*¹ -Where H is the height of retained soil

*² - The seismically induced earth pressure should be applied as an inverted triangular pressure

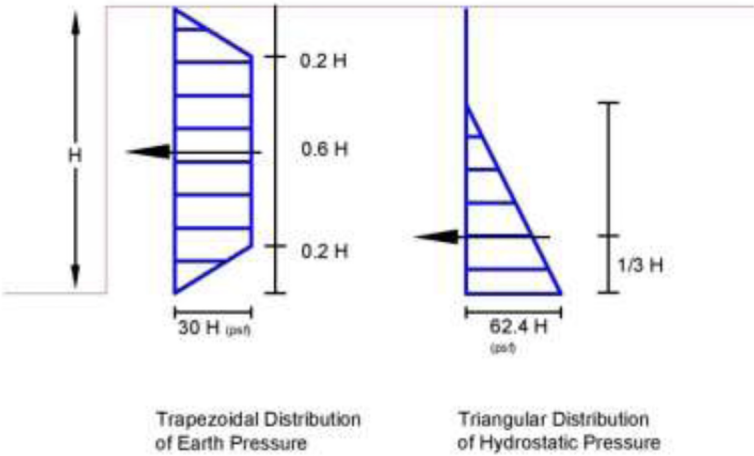


Restrained walls, walls for which horizontal movement is restricted at the top, shall be designed for an At-Rest earth pressure and the restrained condition. Retaining walls designed for the restrained condition should be designed for the At-Rest fluid weight, and should utilize a trapezoidal pressure distribution with the Restrained Condition Designed Earth Pressure.

Table 2: Shoring Design

Shoring Lateral Pressures Recommendations		
Surface Slope of Retained Material Horizontal to Vertical & Height	Static Equivalent Fluid Weight (pcf)	Restrained Condition Design Earth Pressure (psf)*
Level & 16 to 30 ft.	46	$30 \times H$

* -Where H is the retained height of the excavation soil



Cantilevered soldier pile should be designed to resist an active earth pressure. The active earth pressure condition assumes that a triangular pressure distribution is utilized in the shoring design. If the soldier piles are not allowed to deflect, they shall be designed for the Restrained Condition. Soldier piles designed for the restrained condition should utilize a trapezoidal pressure distribution.

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure (At-Rest Condition)
 Descript. : 639 S. La Brea Ave. Los Angeles
 Author : Behnam M. Khani
 Date : 1/4/2017

Settings


USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
 Passive earth pressure calculation : Mazindrani (Rankin)
 Earthquake analysis : Mononobe-Okabe
 Shape of earth wedge : Calculate as skew
 Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters			
Permanent design situation			
Reduction coeff. of internal friction :	$\gamma_{m\phi} =$	1.50	[-]
Reduction coeff. of cohesion :	$\gamma_{mc} =$	1.50	[-]
Reduction coeff. of Poisson's ratio :	$\gamma_{mv} =$	1.00	[-]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma} =$	1.00	[-]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma} =$	1.00	[-]

Basic soil parameters

Number	Name	Pattern	ϕ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		24.00	210.0	127.00	67.50	0.00


All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Silty Sand

Unit weight : $\gamma = 127.0$ pcf
 Stress-state : effective
 Angle of internal friction : $\phi_{ef} = 24.00^\circ$
 Cohesion of soil : $c_{ef} = 210.0$ psf
 Angle of friction struc.-soil : $\delta = 0.00^\circ$
 Soil : cohesionless
 Saturated unit weight : $\gamma_{sat} = 130.0$ pcf

Geological profile and assigned soils

Number	Layer [ft]	Assigned soil	Pattern
1	15.00	Silty Sand	

Number	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Terrain profile

Terrain behind the structure is flat.

Water influence

Ground water table is located below the structure.

Settings of the stage of construction

Design situation : permanent

Analysis No. 1

Pressure at rest behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	φ_d [°]	c_d [psf]	γ [pcf]	K_r	Comment
1	15.00	0.00	16.00	140.0	127.00	0.724	

Pressure at rest distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	15.00	1905.0	0.0	1379.9	1379.9	0.0

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. Z [ft]	F_{vert} [lbf/ft]	App.Pt. X [ft]	Design coefficient
Pressure at rest	10349.3	10.00	0.0	0.00	1.000

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0
2	15.00	1379.9	0.0

Resultant forces

Total horizontal pressure acting on construction = 10349.33 lbf/ft
 Application point of horiz. comp. lies in depth = 10.00 ft
 Total vertical pressure acting on construction = 0.00 lbf/ft
 Dist. of vertical comp. from top of constr. = 0.00 ft

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure (Seismic Condition)
 Descript. : 639 S. La Brea Ave. Los Angeles
 Author : Behnam M. Khani
 Date : 1/4/2017

Settings


USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
 Passive earth pressure calculation : Mazindrani (Rankin)
 Earthquake analysis : Mononobe-Okabe
 Shape of earth wedge : Calculate as skew
 Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters			
Seismic design situation			
Reduction coeff. of internal friction :	$\gamma_{m\phi} =$	1.00	[-]
Reduction coeff. of cohesion :	$\gamma_{mc} =$	1.00	[-]
Reduction coeff. of Poisson's ratio :	$\gamma_{mv} =$	1.00	[-]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma} =$	1.00	[-]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma} =$	1.00	[-]

Basic soil parameters

Number	Name	Pattern	ϕ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		24.00	210.0	127.00	67.50	0.00


All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Silty Sand

Unit weight : $\gamma = 127.0$ pcf
 Stress-state : effective
 Angle of internal friction : $\phi_{ef} = 24.00^\circ$
 Cohesion of soil : $c_{ef} = 210.0$ psf
 Angle of friction struc.-soil : $\delta = 0.00^\circ$
 Soil : cohesionless
 Saturated unit weight : $\gamma_{sat} = 130.0$ pcf

Geological profile and assigned soils

Number	Layer [ft]	Assigned soil	Pattern
1	15.00	Silty Sand	

Number	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Terrain profile

Terrain behind the structure is flat.

Water influence

Ground water table is located below the structure.

Earthquake

Horizontal seismic coefficient $k_h = 0.2600$

Vertical seismic coefficient $k_v = 0.0000$

Coeff. to compute point of application $k.H = 0.60$

Water below the GWT is restricted.

Settings of the stage of construction

Design situation : seismic

Analysis No. 1

Active pressure behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	Φ_d [°]	c_d [psf]	γ [pcf]	δ_d [°]	K_a	Comment
1	5.09	0.00	24.00	210.0	127.00	0.00	0.000	
2	9.91	0.00	24.00	210.0	127.00	0.00	0.279	

Active pressure distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	5.09	646.7	0.0	0.0	0.0	0.0
2	5.09	646.7	0.0	0.0	0.0	0.0
	15.00	1905.0	0.0	530.6	530.6	0.0

Earthquake effects (active earth pressure) - partial results

Layer No.	Thickness [ft]	Φ_d [°]	β [°]	ψ [°]	K_a	K_{ae}	$K_{ae}-K_a$	Comment
1	5.09	24.00	0.00	14.57	0.422	0.652	0.230	
2	9.91	24.00	0.00	14.57	0.422	0.652	0.230	

Earthquake effects (active earth pressure)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_D [psf]	Pressure [psf]	Hor. comp. [psf]	Vertical comp. [psf]
1	0.00	0.0	1905.0	438.7	438.7	0.0
	5.09	646.7	1258.3	289.7	289.7	0.0
2	5.09	646.7	1258.3	289.7	289.7	0.0
	15.00	1905.0	0.0	0.0	0.0	0.0

Forces acting on construction

Name	F _{hor} [lb/ft]	App.Pt. Z [ft]	F _{vert} [lb/ft]	App.Pt. X [ft]	Design coefficient
Active pressure	2628.7	11.70	0.0	0.00	1.000
Earthq.- act.pressure	3289.9	6.00	0.0	0.00	1.000

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	350.9	0.0
2	5.09	261.6	0.0
3	15.00	618.4	0.0

Resultant forces

Total horizontal pressure acting on construction = 5918.60 lb/ft
Application point of horiz. comp. lies in depth = 8.53 ft
Total vertical pressure acting on construction = 0.00 lb/ft
Dist. of vertical comp. from top of constr. = 0.00 ft

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure (Active Condition)
 Descript. : 639 S. La Brea Ave. Los Angeles
 Author : Behnam M. Khani
 Date : 1/4/2017

Settings


USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
 Passive earth pressure calculation : Mazindrani (Rankin)
 Earthquake analysis : Mononobe-Okabe
 Shape of earth wedge : Calculate as skew
 Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters			
Transient design situation			
Reduction coeff. of internal friction :	$\gamma_{m\phi} =$	1.25	[-]
Reduction coeff. of cohesion :	$\gamma_{mc} =$	1.25	[-]
Reduction coeff. of Poisson's ratio :	$\gamma_{mv} =$	1.00	[-]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma} =$	1.00	[-]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma} =$	1.00	[-]

Basic soil parameters

Number	Name	Pattern	ϕ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		24.00	210.0	127.00	67.50	0.00


All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Silty Sand

Unit weight : $\gamma = 127.0$ pcf
 Stress-state : effective
 Angle of internal friction : $\phi_{ef} = 24.00^\circ$
 Cohesion of soil : $c_{ef} = 210.0$ psf
 Angle of friction struc.-soil : $\delta = 0.00^\circ$
 Soil : cohesionless
 Saturated unit weight : $\gamma_{sat} = 130.0$ pcf

Geological profile and assigned soils

Number	Layer [ft]	Assigned soil	Pattern
1	15.00	Silty Sand	

Number	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Terrain profile

Terrain behind the structure is flat.

Water influence

Ground water table is located below the structure.

Settings of the stage of construction

Design situation : transient

Analysis No. 1

Active pressure behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	Φ_d [°]	c_d [psf]	γ [pcf]	δ_d [°]	K_a	Comment
1	3.72	0.00	19.20	168.0	127.00	0.00	0.000	
2	11.28	0.00	19.20	168.0	127.00	0.00	0.380	

Active pressure distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	3.72	472.8	0.0	0.0	0.0	0.0
2	3.72	472.8	0.0	0.0	0.0	0.0
	15.00	1905.0	0.0	723.3	723.3	0.0

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. Z [ft]	F_{vert} [lbf/ft]	App.Pt. X [ft]	Design coefficient
Active pressure	4078.5	11.24	0.0	0.00	1.000

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0
2	3.72	0.0	0.0
3	15.00	723.3	0.0

Resultant forces

Total horizontal pressure acting on construction = 4078.52 lbf/ft
 Application point of horiz. comp. lies in depth = 11.24 ft
 Total vertical pressure acting on construction = 0.00 lbf/ft
 Dist. of vertical comp. from top of constr. = 0.00 ft