Attachment I:

Preliminary Geotechnical Investigation Report and LADBS Soils Report Approval Letter

GEOTECHNICAL INVESTIGATION
Proposed 139-Unit 4-Story Multi-Family Building
Over 1½ Levels of Subterranean Parking
Tract: Owensmouth; Block: 50; Lots: 3, 4 & 5
7334 N. Topanga Canyon Boulevard
Canoga Park, California

July 10, 2020
Project No. 30-5538-00

Prepared for:

Alliant Strategic Development, LLC
Attn: Mr. John Shaw
23901 Calabasas Rd., Suite 2092
Calabasas, CA 91302

ΞAGIΞ   **A.G.I. GEOTECHNICAL, INC.**

July 10, 2020   Project No. 30-5538-00

Alliant Strategic Development, LLC
23901 Calabasas Rd., Suite 2092
Calabasas, CA 91302

Attention: Mr. John Shaw

Subject: **GEOTECHNICAL INVESTIGATION**
Proposed 139-Unit 4-Story Multi-Family Building
Over 1½ Levels of Subterranean Parking
APN: 2111-011-030
Tract: Owensmouth; Block: 50; Lots: 3, 4 and 5
7334 N. Topanga Canyon Boulevard
Canoga Park, California

Dear Mr. Shaw:

This report presents the results of the subject investigation and our opinions regarding the soils engineering factors affecting the development of the subject site. This investigation was performed in June and July, 2020 and consisted of field exploration, laboratory testing, engineering analyses of the field and laboratory data and the preparation of this report. *Determination of the presence or not of hazardous or toxic materials in the on-site soils is beyond the scope of this investigation.*

If you have any questions regarding this report, please contact this office.

Respectfully submitted,
A.G.I. GEOTECHNICAL, INC.

Bruce Smith R.G.E. 2673
Senior Engineer

MBS:mbs

Distribution: (4) Alliant Strategic Development, LLC

Enclosures: Location Map (Figure 1)
Plot Plan (Figure 2)
Site Plan (Figure 3)
Boring Logs
Laboratory Test Results
U.S. Seismic Design Maps
USGS Deaggregations
Liquefaction Analyses
Active Earth Pressure Analyses
At-Rest Earth Pressure Analysis
Slot Cut Stability Analysis
Information Bulletin P/BC 2020-083
Information Bulletin P/BC 2017-141
Quadrangle Location Map
Property Line Perimeter Drain Typical
Groundwater Map

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INTRODUCTION

DESCRIPTION OF SITE

The subject site is located on the east side of Topanga Canyon Boulevard between Valerio Street and Wyandotte Street in the Canoga Park area of the City of Los Angeles, California. The property is occupied by a 2-story office building over subterranean parking, paved areas and limited landscaping. Trees are present. The site is bound on the north and south by developed properties, by Topanga Canyon Boulevard on the west and by an alley on the east. The location of the site is shown on the enclosed Location Map, Figure 1.

PROPOSED SITE DEVELOPMENT

Based on information provided to us, we understand development will consist of a 139-unit 4-story multi-family building over 1½ levels of subterranean parking. The lowermost level of subterranean parking is expected to be about 15 feet below existing grade. Structural loads are anticipated to be less than about 10 kips per linear foot for continuous footings and less than about 150 kips for column loads.

FIELD EXPLORATION

Subsurface conditions were explored by drilling two exploratory borings at the approximate locations shown on the Site Plan, Figure 2. The borings were drilled to a maximum depth of 66.5 feet with Standard Penetration Tests (SPT) performed at selected depths. The borings were drilled 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 samples were sealed immediately in watertight containers for shipment to our laboratory. The soil samplers used in our investigation included a 2.50-inch I.D. split barrel sampler lined with 1-inch brass rings (Modified California Sampler, MC) and a 1.5-inch I.D. Standard Penetration Test (SPT) split barrel sampler. The samplers used in the exploratory borings were driven to a depth of 18 inches with a 140-pound hammer falling from a height of 30 inches. The number of blows to drive the samplers 18 inches in three six-inch increments is reported on the enclosed Boring Logs. The blows for the final 12 inches of the 1.5-inch split spoon sampler are the "N" Value from the SPTs.

SUBSURFACE CONDITIONS

Soil Profile

The existing soil profile, as depicted in the borings to the depth explored, consists of alluvium comprised of stiff to very stiff sandy lean clays and medium dense silty and clayey sands in a moist condition. For a more detailed description of the soils encountered in the exploratory borings, please refer to the Boring Logs enclosed with this report.

Groundwater

Groundwater was encountered in both exploratory borings at a depth of 20 feet below the existing ground surface. According to the "Seismic Hazard Evaluation of the Canoga Park 7.5-Minute Quadrangle, Los Angeles County, California" dated 1997 (Revised 2001) by the Department of Conservation - Division of Mines and Geology, historically highest groundwater level has been about 10 feet below the ground surface. The groundwater level may fluctuate because of seasonal changes, injection or extraction of water, variations in temperature and other causes.

LIQUEFACTION POTENTIAL (CYCLIC MOBILITY)

Liquefaction and dry sand settlement analyses were performed using the analytical procedures described in *Tokimatsu, K., and Seed, H. (1987), Evaluation of Settlements in Sands Due to Earthquake Shaking, Youd, T.L., and Idriss, I.M. (1997) "Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", Technical Report NCEER-97-0022, FHWA* and the requirements contained in the City of Los Angeles' memorandum dated July 16, 2014. Seismic settlements discussed herein include both liquefaction and dry sand settlements.

Liquefaction calculations were performed for the historically highest groundwater level located at 10 feet below the ground surface. Calculations were performed for a 475-year return period and a 2475-year return period. The peak ground acceleration for 475 years was evaluated using two-thirds of the PGA_M and a required factor of safety of 1.1. The peak ground acceleration for 2475 years was evaluated using the full PGA_M and a required factor of safety of 1.0. Seismic settlement calculations are enclosed. The results of the liquefaction evaluation are summarized below:

Return Period	Peak Ground Acceleration[1]		Moment Magnitude Mw[2]	Factor of Safety	Calculated Total Settlement	Calculated Differential Settlement
475 years	2/3 PGA$_M$	0.463g	6.47	1.10	2.65	1.77
2475 years	100% PGA$_M$	0.695g	6.56	1.00	4.40	2.93

NOTES: 1) From U.S. Seismic Design Maps website: https://seismicmaps.org/
2) From USGS Deaggregation website: https://earthquake.usgs.gov/hazards/interactive/

The calculated 2.65 inch seismic settlement from the 475-year calculation exceeds the City of Los Angeles' maximum allowable values for a conventional footing foundation system (1.5 inch total, 0.75 inch differential); therefore, a mat foundation is recommended to mitigate the potential for seismic settlement damages. These seismic settlements must be combined with the predicted static settlements for final determination of foundation design requirements. Mat foundation recommendations are discussed subsequently in this report. The 4.40 total and 2.93 inch differential settlement from the 2475-year analysis present no risk of collapse of the structure.

ON-SITE INFILTRATION FACILITIES

The profile soils below a depth of 7 to 10 feet consist primarily of low permeability sandy clays. These soils are unsuitable for on-site infiltration of stormwater. Underlying soils suitable for infiltration are present but are located below the groundwater level in the borings and therefore are unsuitable as well.

SEISMIC DESIGN CRITERIA

Future structures should be designed by the structural engineer in accordance with the applicable Seismic Building Code. Based on our investigation, the subject site is classified as Site Class D in accordance with the 2019 California Building Code that refers to ASCE 7-16.

Per Section 11.4.8 of ASCE 7-16, structures shall be designed for the seismic response coefficient C_S determined by Eq. (12.8-2) for values of $T \leq 1.5\ T_S$, as 1.5 times the value computed in accordance with Eq. (12.8-3) for $T_L \geq T > 1.5\ T_S$, or as 1.5 times the value computed in accordance with Eq. 37.5 (12.8-4) for $T > T_L$ where:

T = the fundamental period of the building
$T_S = S_{D1}/S_{DS}$
T_L = long-period transition period

The Design Spectral Response Acceleration Parameters presented on the following table generated by the U.S. Seismic Design Maps Website (https://seismicmaps.org), may be utilized for seismic design:

2019 CBC Seismic Design Parameters (Site Class D)

Site Location (Latitude, Longitude): (34.1979, -118.5981)				
Spectral Period, T (Seconds)	MCE$_R$ Ground Motion (g)	Site-Modified Spectral Acceleration (g)		Seismic Design Acceleration (g)
0.2	$S_S = 1.500$	$F_a = 1.2$	$S_{MS} = 1.800$	$S_{DS} = 1.200$
1.0	$S_1 = 0.600$	$F_v = 1.7$	$S_{M1} = 1.020$	$S_{D1} = 0.680$
Site Modified Peak Ground Acceleration PGA$_M$ = 0.695 g				
Long-Period Transition Period T$_L$ = 8 Seconds				
Seismic Design Category = D				

If the Seismic Response Coefficient C_s recommended above is not applicable for structural design, our office can perform a Site-Specific Ground Motion Hazard Analysis upon the project structural engineer's request.

Present building codes and construction practices, and the recommendations presented in this report, are intended to minimize structural damage to buildings and prevent loss of life as a result of a moderate or a major earthquake; they are not intended to totally prevent damage to structures, graded slopes and natural hillsides. While it may be possible to design structures and graded slopes to withstand strong ground motion, the construction costs associated with such designs are usually prohibitive, and the design restrictions may be severely limiting. Earthquake insurance is often the only economically feasible form of protection for your property against major earthquake damage. Damage to sidewalks, steps, decks, patios and similar exterior improvements can be expected as these are not normally controlled by the building code.

LABORATORY TESTING

CLASSIFICATION

Soils were classified visually according to the Unified Soil Classification System. Unit weight and moisture determinations were performed for each undisturbed sample. Results of density and moisture determinations, together with classifications, are shown on the enclosed Boring Logs.

LOAD CONSOLIDATION TESTS (ASTM:D-2435)

To investigate the settlement of the soils under the pressure of the proposed foundations, consolidation tests were performed on undisturbed samples of the on-site soils. Axial loads were carried to a maximum of $9,400 lb/ft^2$. To hasten consolidation, investigate the collapse potential and simulate possible adverse field conditions, water was added to an axial load of $2,350 lb/ft^2$. Compressibility of all the soils within the zone of significant stress was investigated and the results considered in our engineering analyses. Graphic plots of the load consolidation curves are included in this report.

DIRECT SHEAR TESTS (ASTM:D-3080)

In order to determine the shear strength of the soils, direct shear tests were performed on undisturbed and remolded samples of the on-site soils. The remolded sample was tested at 90% of the maximum dry density. To simulate possible adverse field conditions, the samples were saturated prior to shearing. Graphic summaries of the test results, including moisture content at the time of shearing, are included in this report.

GRAIN SIZE DISTRIBUTION (ASTM:D-422-63 (2002))

To aid in classification, sieve analyses and hydrometer tests were performed on typical samples of the upper soils. The results of the tests are shown on the enclosed Grain Size Distribution Charts. Fines contents are also noted on the Boring Logs.

MAXIMUM DENSITY/OPTIMUM MOISTURE (ASTM:D-1557)

The maximum density/optimum moisture content relationship was determined for an upper sample of the on-site soils. The test was conducted in accordance with the ASTM:D-1557 standard. A graphic summary of the test result is included in this report.

EXPANSION TEST (ASTM:D-4829)

An expansion test was performed on a remolded sample representative of the upper on-site soils in accordance with ASTM:D-4829 to evaluate its volume change with increasing moisture conditions. The result is as follows:

Location	Depth (ft.)	Expansion Index	Potential Expansion
B-1	0-5	6	Very Low

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

The property is suitable for the proposed construction from a geotechnical engineering standpoint. The construction plans should consider the appropriate soils engineering features of the site. The on-site soils are medium dense to very dense silty sand to sand and stiff to very stiff sandy clays. Groundwater was encountered in the exploratory borings at a depth of 20 feet below the existing ground surface. The on-site soils have a very low potential expansion.

SITE PREPARATION

Debris from demolition, vegetation and underground utility lines to be abandoned should be removed from the site. It is anticipated that excavation to basement level will remove existing structures and soils disturbed by demolition. For any on-grade structures, after clearing the site, the upper three feet of the existing soils should be removed and placed back as compacted fill. The fill pad should extend to at least three feet beyond the building lines in each direction. Fill soils should be cleared of deleterious debris, placed in 6- to 8-inch lifts, brought to about optimum moisture content, and compacted to at least 90% of the maximum density for fine-grained soils and 95% of the maximum density for granular soils as determined by ASTM:D-1557. Clearance and compaction of hardscape areas should be prepared in the same manner in the upper 12 inches of the subgrade. **The placement of the fill should be performed under our observation and testing**

All excavations resulting from removal of existing obstructions (e.g., foundations, tree roots) should be backfilled with soil compacted to at least 90% of the maximum density for fine-grained soils and 95% of the maximum density for granular soils as determined by ASTM:D-1557. If any cesspools or seepage pits are encountered during grading, they should be backfilled with vibrated gravel or slurry mix to five feet below finish grade. The upper five feet should be backfilled with soil compacted by mechanical means.

FOUNDATION DESIGN

Type of Foundation

The proposed building should be supported on a mat foundation bearing on the undisturbed native soils. The mat should be at least 12 inches thick and may be designed for a maximum allowable bearing pressure of 3,000lb/ft^2 and subgrade modulus of 150lb/in^3. The recommended soil bearing pressure may be increased by one-third when designing for wind and seismic forces.

Expected Settlements

If foundations are supported on compacted fill or undisturbed natural soils and are sized for the recommended bearing pressures, static differential settlements are not expected to exceed 0.25 inch in a 30-foot span. Total static settlements are anticipated to be less than 0.5 inch. When combined with the 2.65 inch total seismic settlement and 1.77 inch differential seismic settlement, the overall total and differential settlements should not exceed about 3.2 and 2.0 inches, respectively. The anticipated settlements are acceptable for the recommended mat foundation.

FLOOR SLABS-ON-GRADE

Other than the mat slab, concrete floor slabs-on-grade thickness and reinforcement should reflect the anticipated use of the slabs and should be designed by the structural engineer. Concrete floor slabs-on-grade should be a minimum of four inches (full) thick with minimum reinforcement consisting of No.4 deformed bars spaced a maximum of 16 inches each way. Concrete slabs-on-grade should be underlain by four inches of ½ inch or larger clean aggregate base. In areas where floor coverings or equipment that are sensitive to moisture are contemplated, a 10-mil visqueen moisture barrier should be placed on the granular base beneath the slab.

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.

The recommendations presented above are intended to reduce the potential for random cracking to which concrete flatwork is often prone. Judicious spacing of crack control joints has proven effective in further reducing random cracking. A structural engineer may recommend the desirable spacing. Usually the crack control joints are placed 12 to 15 feet apart in each direction. Factors influencing cracking of concrete flatwork, (other than expansion, settlement and creep of soils), and which should be avoided, include: poor-quality concrete, excessive time passing between the mixing and placement of the concrete (the concrete should be rejected if this time interval exceeds two hours), temperature and wind conditions at the time of placement of the concrete, curing of the concrete and workmanship. The concrete should be maintained in a moist condition (curing) for at least the first seven days after concrete placement. During hot weather, proper attention should be given to the ingredients, production methods, handling, placement, protection and curing to prevent excessive concrete temperature or water evaporation. In hot weather and windy conditions, water evaporates more rapidly from the surface of the concrete flatwork. This requires more frequent moistening of the concrete during the curing period or the use of a protective chemical film to prevent evaporation.

LATERAL RESISTANCE

An allowable lateral bearing of $250lb/ft^2$ per foot of depth may be assumed up to a maximum of $3,500lb/ft^2$. A coefficient of friction between soil and concrete of 0.3 may be used.

LATERAL LOADS

Walls should have adequate drainage to prevent the build-up of hydrostatic pressure. An active equivalent fluid pressure (EFP) of $26lb/ft^3$ was determined using a sliding wedge stability analysis. This is less than typical design values. We recommend that cantilevered walls be designed to resist an active EFP of $30lb/ft^3$. An at-rest EFP of $52lb/ft^3$, or a trapezoidal pressure of $32Hlb/ft^2$ on a 0.2H, 0.6H 0.2H trapezoidal distribution, were calculated using the Jaky formula. These values are recommended for restrained wall design. Calculations are included in this report.

The seismic backfill pressure coefficient for retaining wall design is determined as one-third of PGA_M. A PGA_M of 0.695g was obtained from the U.S. Seismic Design Maps web site. One-third of this value yields an acceleration of 0.232g. For a typical wet unit weight of $110lb/ft^3$, the recommended design seismic pressure is 0.232 x 110 = $26lb/ft^3$ EFP. This pressure is in addition to the static lateral pressures. It is unnecessary to include seismic backfill pressure for restrained walls.

Retaining walls and basement walls subject to surcharge loads should be designed to include the additional lateral pressure determined in accordance with the enclosed LADBS Information Bulletin P/BC 2020-83. A chart solution for this Bulletin method is also included. Lateral loads can also be determined using appropriate Boussinesq equations if details regarding the surcharge loads and locations are available.

HYDROSTATIC DESIGN

The 10-foot historically highest groundwater level is about five feet above the basement floor elevation. Above a depth of 10 feet, basement walls may be designed using lateral loads discussed above. Below 10 feet, the building walls and mat slab should be waterproofed and designed to resist hydrostatic pressures. The lateral wall pressure to be used for submerged backfill may be taken as $90lb/ft^3$ EFP. Hydrostatic uplift on the mat slab should be determined based on a fluid pressure of 62.4lb/ft3 times the height of water above the bottom of the mat.

BACKFILL

All backfill of walls, footings or trenches should be compacted to 90% of the maximum density for fine-grained soils and 95% of the maximum density for granular soils as determined by ASTM:D-1557 **and should be tested by the soils engineer**.

DRAINAGE

Adequate drainage at the site is essential and it should be provided. Rain gutters should be connected to an appropriate drainage system and carried away from the building and into 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 other structures will be significantly influenced by the site drainage conditions.

PLANTERS

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.

CONSTRUCTION CUTS

Construction cuts up to five feet in height may be excavated vertically for their entire length and height. For deeper cuts, we recommend that the backslope above the vertical be laid back to a 1H:1V gradient provided the cuts do not remove lateral support from adjacent buildings or property lines. Removal of lateral support occurs if the cut extends below a 1H:1V line projected downward from the nearest edge of the adjacent property line or building. If lateral support is removed, the construction cuts will need to be completed using the 'A, B, C' slot-cutting method or they should be shored. If the slot-cutting method is used, the cut should be opened at a gradient of 1:1 first, then each slot opened, and the removed soils replaced as engineered compacted fill before the subsequent slot is opened. The slots should not exceed 8 feet in width or 12 feet in height.

An active EFP of 17lb/ft^3 was determined using a sliding wedge stability analysis and factor of safety of 1.25 for temporary shoring. This is less than minimum values. Temporary shoring should be designed to resist an active EFP of 30lb/ft^3. Lateral earth pressure on the lagging may be taken as a uniform pressure of 400lb/ft^2 for either cantilevered or restrained shoring.

Footing foundations for the shoring bracing may be designed for a maximum soil bearing pressure of 2,500lb/ft^2. Tie-back anchors can be designed for an allowable bond stress of 2,500lb/ft^2 for pressure-grouted anchors. The inclination of tiebacks should be between 15 and 45 degrees below horizontal, and the minimum length of the grouted anchors should extend at

least 20 feet beyond the active failure plane. The active failure plane may be taken as 35 degrees from vertical, and the point of fixity may be taken as three feet below the bottom of the excavation.

If piles are used for shoring, a passive resistance of 500lb/ft^2 per foot of depth, up to a maximum of 7,500lb/ft^2, may be used in design. Axial loads on the piles can be resisted using an allowable skin friction of 500lb/ft^2. The piles may be assumed to be fixed at a point located three feet below the bottom of the excavation. Where lateral support of adjacent structures is removed, we recommend that the allowable shoring deflection be no more than 0.5 inch. A maximum deflection of 1.0 inch should be acceptable elsewhere.

If unshored construction cuts are to remain open for more than two weeks or if rain is expected while the construction cuts are open, they should be covered by a plastic membrane kept in placed by holding blocks or driven re-bar at the top and bottom of the membrane. No equipment or personnel should stand closer than ten feet from the top of the temporary cut. All construction cuts should comply with the State of California Construction Safety Orders (CAL/OSHA).

RECOMMENDED INSPECTIONS

It is strongly recommended (and is a condition of use of this report), that the developer ensures that each phase of construction be properly inspected and approved by the local Building Department official.

WORKMAN SAFETY-EXCAVATIONS

It is essential 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 regulations 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 ensure that berms or other devices will prevent surface water from flowing over the top of the excavation or ponding at the top of the excavations.

OBSERVATION

Removal bottoms should be examined and approved by the City inspector and us before any fill is placed. We should examine footing excavations prior to forming or placement of reinforcement steel to confirm that the soil conditions meet the requirements set by this report. Footing excavations should be kept moist and concrete should be placed as soon as possible after excavations are completed, examined and approved by us and the City inspector.

REVIEW

The geotechnical consultants shall review and sign the plans and specifications.

REGULATORY AGENCY REVIEW AND ADDITIONAL CONSULTING

All geotechnical and/or engineering geologic aspects of the proposed development are subject to review and approval by the government reviewing agency. The government reviewing agency may approve or deny any portion of the proposed development which may require additional geotechnical services by this office. Additional geotechnical services may include review responses, supplemental letters, plan reviews, construction/site observations, meetings, etc. The fees for generating additional reports, letters, exploration, analyses, etc. will be billed on a time and material basis.

COMMENTS

The conclusions and recommendations presented in this report are based on research, site observations and limited subsurface information. The conclusions and recommendations presented are based on the supposition that subsurface conditions do not vary significantly from those indicated. Although no significant variations in subsurface conditions are anticipated, the possibility of significant variations cannot be ruled out. If such conditions are encountered, this consultant should be contacted immediately to consider the need for modification of this project.

This report was prepared for the exclusive use of Alliant Strategic Development, LLC and their design consultants for the specific project outlined herein. This report may not be suitable for use by other parties or other uses. This report is subject to review by regulatory agencies and these agencies may require their approval before the project can proceed. No guarantee that the regulatory public agency or agencies will approve the project is intended, expressed or implied.

One of the purposes of this report is to provide the client with advice regarding geotechnical conditions on the site. It is important to recognize that other consultants could arrive at different conclusions and recommendations. No warranties of future site performance are intended, expressed or implied.

LOCATION MAP

7334 N. Topanga Canyon Blvd., Canoga Park

FIGURE 1

PROJECT NO.	30-5538-00
DATE	4-2020
PREPARED BY	AM
APPROVED BY	JAV

7334 Topanga Canyon Blvd, Canoga Park, CA 91303

Los Angeles

Canoga Park

Canoga Park Chamber of Commerce

B-2

B-1

N. Topanga Canyon Blvd.

2-Story Office
Building Over
Subterranean
Parking

Alley
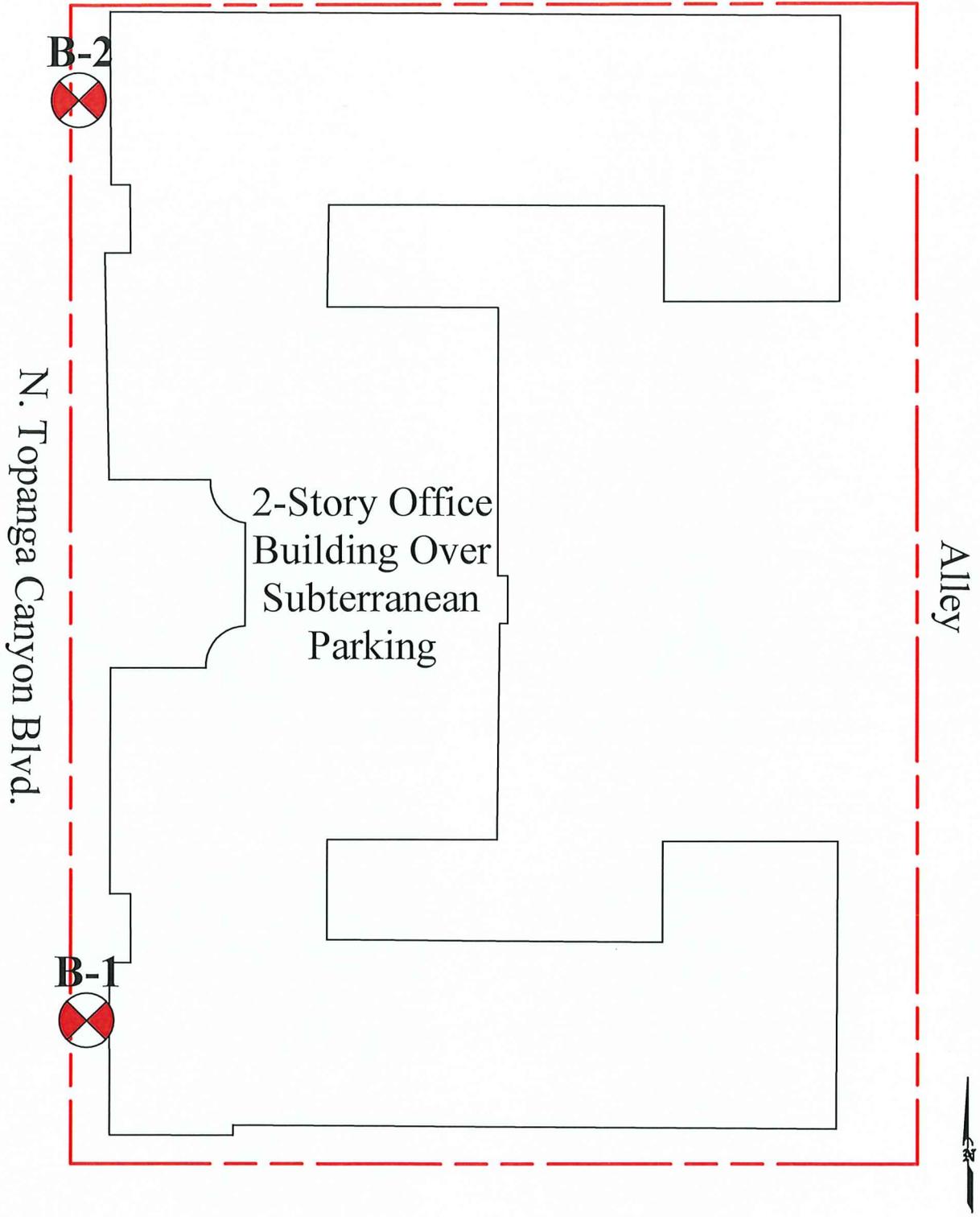
EXPLANATION

B-1 Approximate Location
of Exploratory Boring

Scale 1" = 30'

FIGURE 2

A.G.I. GEOTECHNICAL, INC.
Engineering Geology • Geotechnical Engineering
16555 Sherman Way, Ste. A • Van Nuys, CA 91406
(818) 785-5244 • Fax (818) 785-6251

PLOT PLAN
7334 N. Topanga Canyon Blvd., Canoga Park

PROJECT NO.	30-5538-00
DATE	06-2020
PREPARED BY	WFB
APPROVED BY	JAV

N. Topanga Canyon Blvd.
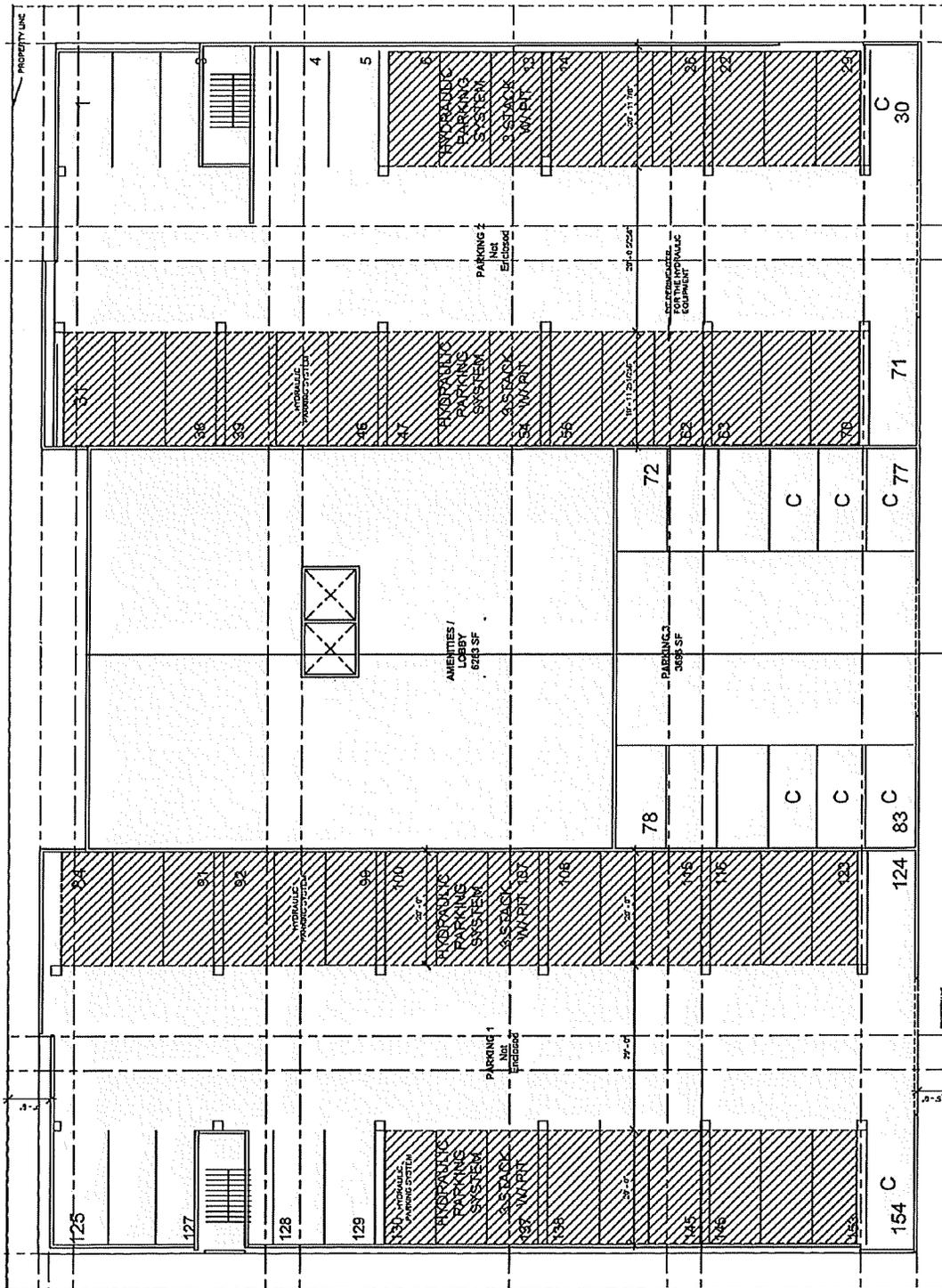
Alley

Scale 1" = 30'

FIGURE 3

A.G.I. GEOTECHNICAL, INC.	**SITE PLAN**	PROJECT NO.	30-5538-00
Engineering Geology • Geotechnical Engineering	7334 N. Topanga Canyon Blvd., Canoga Park	DATE	06-2020
16555 Sherman Way, Ste. A • Van Nuys, CA 91406		PREPARED BY	WFB
(818) 785-5244 • Fax (818) 785-6251		APPROVED BY	JAV

BORING LOGS

LEGEND

⊠ Ring Sample, or Bulk Sample

▤ Standard Penetration Test (SPT)

▽ Ground Water Level

SOIL SIZE	
COMPONENT	SIZE RANGE
Boulders	Above 12"
Cobbles	3"-12"
Gravel	#4 - 3"
coarse	$\frac{3}{4}$" - 3"
fine	#4 - $\frac{3}{4}$"
Sand	#200-#4
coarse	#10-#4
medium	#40-#10
fine	#200-#40
Fines (Silt or Clays)	Below #200

PLASTICITY OF FINE GRAINED SOILS	
PLASTICITY INDEX	VOLUME CHANGE POTENTIAL
0-15	Probably Low
15-30	Probably Moderate
30 or more	Probably High

WATER CONTENT
Dry: No feel of moisture
Damp: Much less than normal moisture
Moist: Normal moisture
Wet: Much greater than normal moisture
Saturated: At or near saturation

RELATIVE DENSITY	
SANDS & GRAVELS	BLOWS PER FOOT
Very loose	0-4
Loose	4-10
Medium dense	10-30
Dense	30-50
Very dense	Over 50

	GROUP SYMBOLS	DESCRIPTIONS	DIVISIONS	
COARSE-GRAINED SOILS (Less than 50% Fines)	GW	Well-graded gravels or gravel-sand mixtures, less than 5% fines	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size	
	GP	Poorly-graded gravels or gravel-sand mixtures, less than 5% fines		
	GM	Silty gravels, gravel-sand silt mixtures, more than 12% fines		
	GC	Clayey gravels, gravel-sand-clay mixtures, more than 12% fines		
	SW	Well-graded sands or gravelly sands, less than 5% fines	SANDS More than half of coarse fraction is smaller than No. 4 sieve size	
	SP	Poorly-graded sands or gravelly sands, less than 5% fines		
	SM	Silty sands, sand-silt mixtures, more than 12% fines		
	SC	Clayey sands, sand-clay mixtures, more than 12% fines		
FINE-GRAINED SOILS (More than 50% Fines)	ML	Inorganic silt, very fine sands, rock flour, silty or clayey fine sands	SILTS AND CLAYS Liquid limit less than 50	
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
	OL	Organic silts or organic silt-clays of low plasticity		
	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	SILTS AND CLAYS Liquid limit less than 50	
	CH	Inorganic clays of high plasticity, fat clays		
	OH	Organic clays of medium to high plasticity		
	PT	Peat, mulch, and other highly organic soils	HIGHLY ORGANIC SOILS	

CONSISTENCY	
CLAYS & SILTS	BLOWS PER FOOT
Very soft	0-2
Soft	2-4
Firm	4-8
Stiff	8-15
Very stiff	15-30
Hard	Over 30

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A.G.I. GEOTECHNICAL, INC.

A.G.I. Geotechnical, Inc.   16555 Sherman Way, Unit A   Van Nuys, California 91406   Telephone: (818) 785-5244   Fax: (818) 785-6251

CLIENT: Alliant Strategic Development, LLC   PROJECT NAME: Proposed 4-Story Building Over 1 1/2 Levels of Subterranean Parking

PROJECT NUMBER: 30-5538-00   PROJECT LOCATION: 7334 N. Topanga Canyon Blvd., Canoga Park

DATE STARTED: 06/04/2020   COMPLETED: 06/04/2020   GROUND ELEVATION: N/A   BORING DIAMETER: 8"

EXCAVATION METHOD: 8" Hollow Stem Auger   GROUND WATER LEVELS: 20'

DRILLING CONTRACTOR: Choice Drilling   SAMPLING METHOD: Autohammer, 140 lb., 30" Drop

LOGGED BY: CWL   CHECKED BY: JAV

DEPTH (ft)	DRIVE SAMPLE	BLOW COUNT (N VALUE)	BULK SAMPLE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	WET UNIT WT. (pcf)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	MATERIAL DESCRIPTION	<200	D 50	Classification
		5/5/10		6.5	103	110				**Alluvium** Light brown Silty Fine SAND (Slightly moist, medium dense)	43		SM
5		2/3/4		12.0							56		
		5/7/9		17.4	96	112				Light brown Sandy CLAY (Moist, stiff)			CL
10		4/4/6		22.0							73		
		5/7/9		22.1	98	119							
15		5/5/6		17.5							53		
		4/6/7		21.5	102	124				Light brown Silty SAND (very moist to wet, medium dense)			SM
20		4/5/8		26.2							49		
		4/7/11		30.3	95	123				Light brown Sandy CLAY (Wet, stiff)			CL
25		6/7/7		38.0							81		
		11/11/13		34.1	92	123							
30		4/5/8		37.4							87		
		5/12/13		17.9	117	137				Dark brown Clayey SAND (Wet, medium dense)			SC

AGI

A.G.I. GEOTECHNICAL, INC.

A.G.I. Geotechnical, Inc.   16555 Sherman Way, Unit A Van Nuys, California 91406   Telephone: (818) 785-5244   Fax: (818) 785-6251

CLIENT: Alliant Strategic Development, LLC PROJECT NAME: Proposed 4-Story Building Over 1 1/2 Levels of Subterranean Parking

PROJECT NUMBER: 30-5538-00   PROJECT LOCATION: 7334 N. Topanga Canyon Blvd., Canoga Park

DATE STARTED: 06/04/2020   COMPLETED: 06/04/2020   GROUND ELEVATION: N/A   BORING DIAMETER: 8"

EXCAVATION METHOD: 8" Hollow Stem Auger   GROUND WATER LEVELS: 20'

DRILLING CONTRACTOR: Choice Drilling   SAMPLING METHOD: Autohammer, 140 lb., 30" Drop

LOGGED BY: CWL   CHECKED BY: JAV

DEPTH (ft)	DRIVE SAMPLE	BLOW COUNT (N VALUE)	BULK SAMPLE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	WET UNIT WT. (pcf)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	MATERIAL DESCRIPTION	<200	D 50	Classification
35													
		5/6/8		24.0						Light brown Sandy CLAY (Wet, stiff)	56		CL
	X	7/7/18		25.6	103	129							
40		5/6/7		32.6							72		
	X	8/9/11		24.8	100	125				Dark brown Clayey Fine SAND (Wet, medium dense)			SC
45		5/9/9		16.6							34		
50		6/8/10		20.5						Light brown Sandy CLAY (Wet, very stiff)	55		CL
55		6/8/11		26.6							69		
60		8/9/10		23.5							53		
65		10/14/16								No Recovery @ 65'			

Total Depth: 66.5'
Water @ 20'

A.G.I. GEOTECHNICAL, INC.

A.G.I. Geotechnical, Inc.   16555 Sherman Way, Unit A   Van Nuys, California 91406   Telephone: (818) 785-5244   Fax: (818) 785-6251

CLIENT: Alliant Strategic Development, LLC   PROJECT NAME: Proposed 4-Story Building Over 1 1/2 Levels of Subterranean Parking

PROJECT NUMBER: 30-5538-00   PROJECT LOCATION: 7334 N. Topanga Canyon Blvd., Canoga Park

DATE STARTED: 06/04/2020   COMPLETED: 06/04/2020   GROUND ELEVATION: N/A   BORING DIAMETER: 8"

EXCAVATION METHOD: 8" Hollow Stem Auger   GROUND WATER LEVELS: 20'

DRILLING CONTRACTOR: Choice Drilling   SAMPLING METHOD: Autohammer, 140 lb., 30" Drop

LOGGED BY: CWL   CHECKED BY: JAV

DEPTH (ft)	DRIVE SAMPLE	BLOW COUNT (N VALUE)	BULK SAMPLE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	WET UNIT WT. (pcf)	ATTERBERG LIMITS			MATERIAL DESCRIPTION	<200	D 50	Classification
							LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX				
	⊠	4/6/7		11.4	99	110				**Alluvium** Light brown Silty Fine SAND (Slightly moist, medium dense)			SM
5	⊠	5/5/7		13.4	96	109							
10	⊠	5/8/9		22.2	97	119				Light brown Sandy CLAY (Moist, stiff)			CL
15	⊠	5/7/11		15.4	114	131							
20	⊠	10/11/12		14.1	114	130				Light brown Silty SAND (Wet, medium dense)			SM
25	⊠	4/7/11		28.5	95	122							
30	⊠	8/11/15		26.3	95	120							

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A.G.I. GEOTECHNICAL, INC.

A.G.I. Geotechnical, Inc.   16555 Sherman Way, Unit A Van Nuys, California 91406   Telephone: (818) 785-5244   Fax: (818) 785-6251

CLIENT: __Alliant Strategic Development, LLC__ PROJECT NAME: __Proposed 4-Story Building Over 1 1/2 Levels of Subterranean Parking__

PROJECT NUMBER: __30-5538-00__   PROJECT LOCATION: __7334 N. Topanga Canyon Blvd., Canoga Park__

DATE STARTED: __06/04/2020__ COMPLETED: __06/04/2020__ GROUND ELEVATION: __N/A__   BORING DIAMETER: __8"__

EXCAVATION METHOD: __8" Hollow Stem Auger__   GROUND WATER LEVELS: __20'__

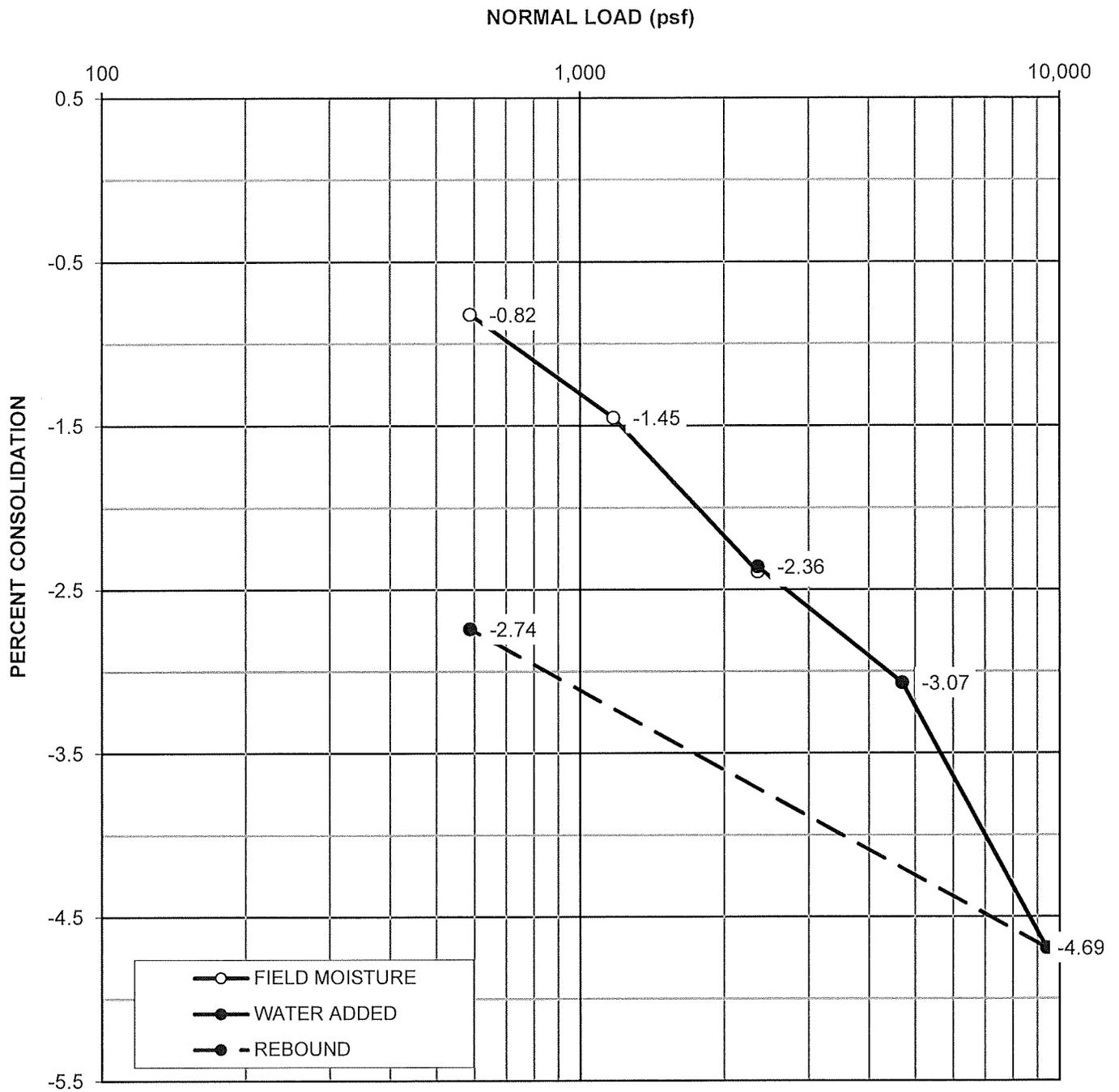DRILLING CONTRACTOR: __Choice Drilling__   SAMPLING METHOD: __Autohammer, 140 lb., 30" Drop__

LOGGED BY: __CWL__   CHECKED BY: __JAV__

DEPTH (ft)	DRIVE SAMPLE	BLOW COUNT (N VALUE)	BULK SAMPLE	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	WET UNIT WT. (pcf)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	MATERIAL DESCRIPTION	<200	D 50	Classification
35													
	✕	7/9/11		20.0	111	134				Brown Clayey SAND (Wet, medium dense)			SC
40										Total Depth: 36.5' Water @ 20'			
45													
50													
55													
60													
65													

LABORATORY TEST RESULTS

NORMAL LOAD (psf)

PERCENT CONSOLIDATION

- ○ FIELD MOISTURE
- ● WATER ADDED
- ● REBOUND

-0.82
-1.45
-2.36
-2.74
-3.07
-4.69

PROJECT NO. 30-5538-00   BORING NO. B-1   DEPTH (FT) 7.5

REPRESENTATIVE FOR   Alluvium
SOIL TYPE AND DESCRIPTION   Sandy CLAY (CL)

HYDROCONSOLIDATION (%) -0.03

A.G.I. GEOTECHNICAL, INC.

NORMAL LOAD (psf)

Consolidation test plot. Y-axis: PERCENT CONSOLIDATION (0.5 to -8.5). X-axis: NORMAL LOAD (psf) logarithmic scale (100, 1,000, 10,000).

Data points (FIELD MOISTURE, open circles): -1.89, -2.32, -2.86
Data points (WATER ADDED, filled circles): -4.09, -5.62, -7.60
Rebound (dashed): -6.70 to -7.60

Legend:
- —○— FIELD MOISTURE
- —●— WATER ADDED
- ●- - REBOUND

PROJECT NO. 30-5538-00   BORING NO.   B-2   DEPTH (FT)   5

REPRESENTATIVE FOR _____ Alluvium _____
SOIL TYPE AND DESCRIPTION _____ Silty SAND (SM) _____

HYDROCONSOLIDATION (%) ____ 1.23 ____

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A.G.I. GEOTECHNICAL, INC.

	Sample ID :	1 ksf	2 ksf	4 ksf			
Initial	Water Content (%)	13.4	13.4	13.4			
	Dry Density (%)	94.9	94.2	93.7			
	Saturation (%)	46.7	45.9	45.3			
Final	Water Content (%)	30.9	31.3	31.6			
	Dry Density (pcf)	93.9	92.8	92.0			
	Saturation (%)	105.0	103.6	102.6			
	Normal Stress (psf)	1067	2134	4269			
	Peak Shear Stress (psf)	1063	1623	2804			
	Residual Shear Stress (psf)	962	1560	2755			
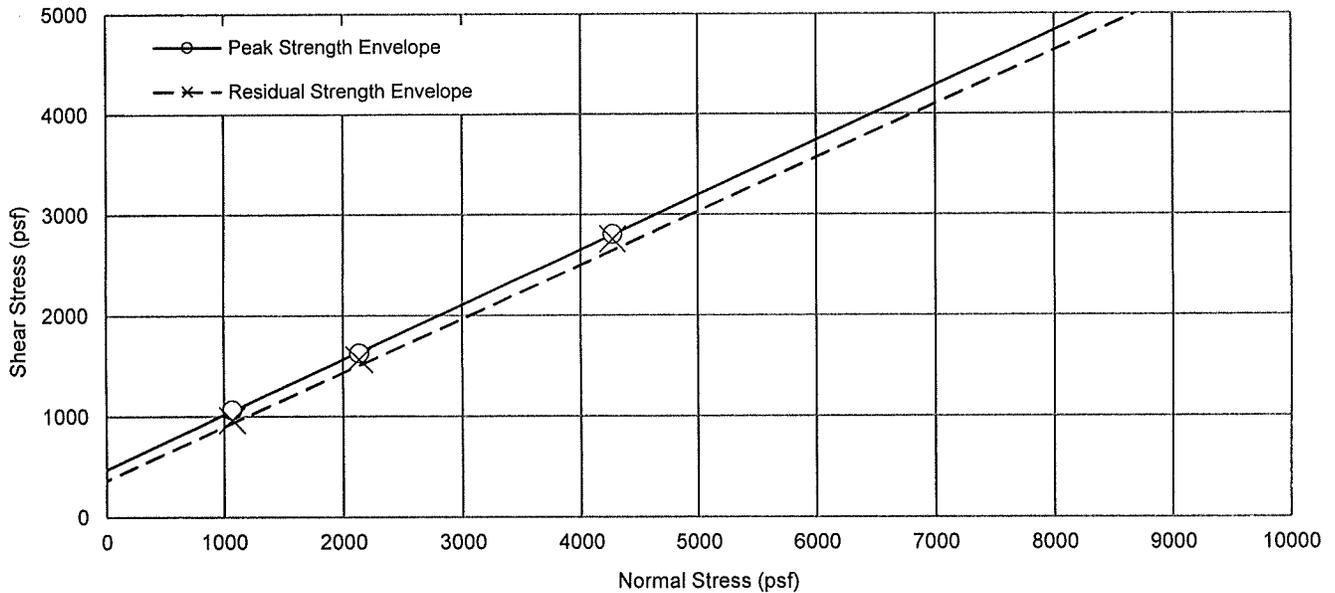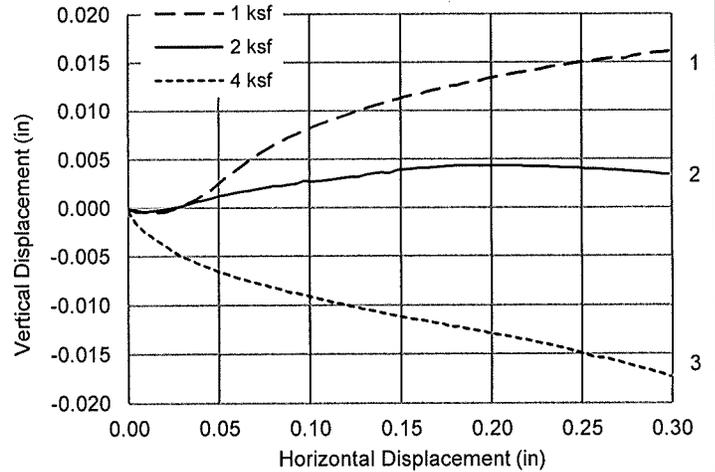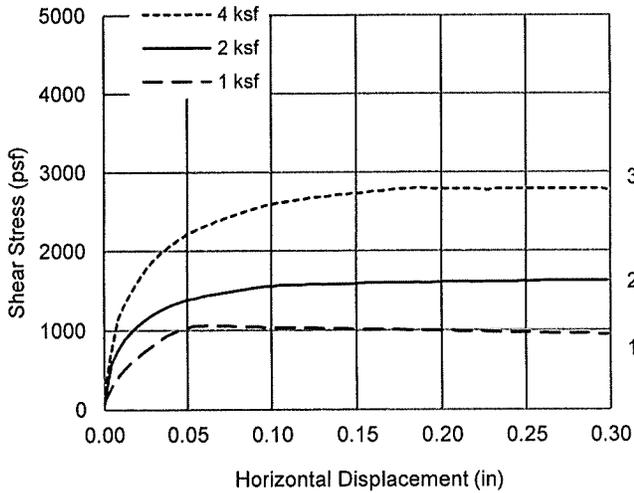
Peak Cohesion, c' (psf):	472	Ultimate Cohesion, c (psf):	364	**DIRECT SHEAR TEST (ASTM:D-3080)**
Peak Friction, φ' (deg):	28.6	Ultimate Friction, Φ' (deg):	28.1	

SAMPLE TYPE: Undisturbed
DESCRIPTION: Silty SAND

LL:
PL:
PI:
% <0.75μ
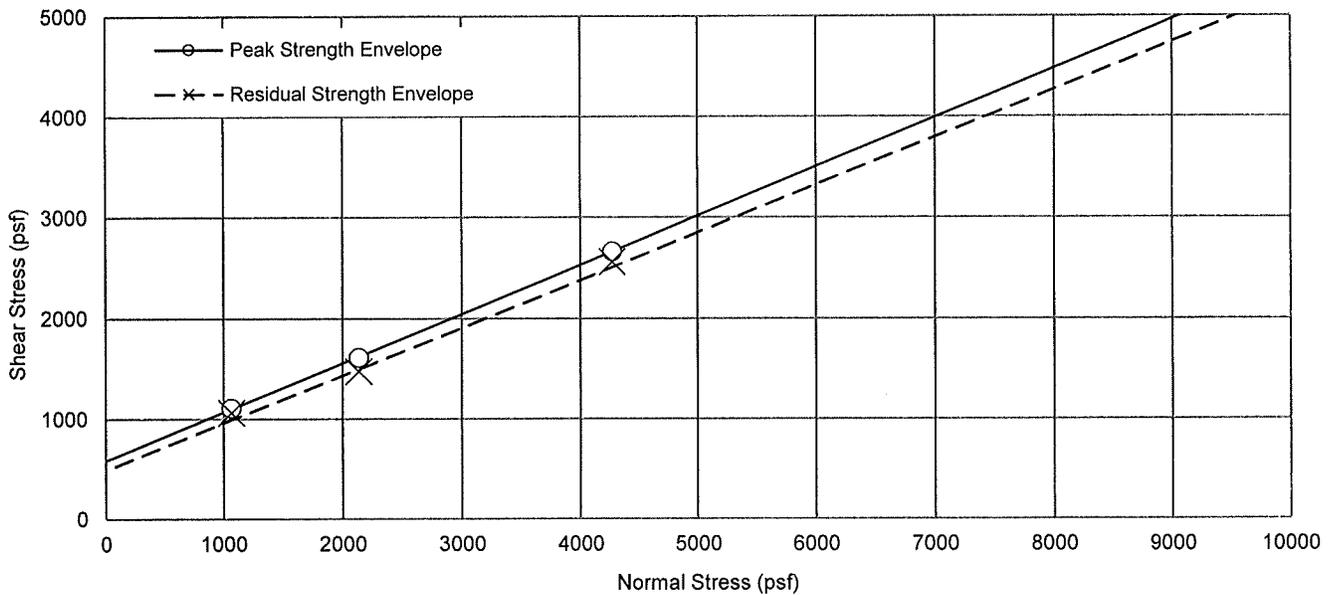% <0.02μ
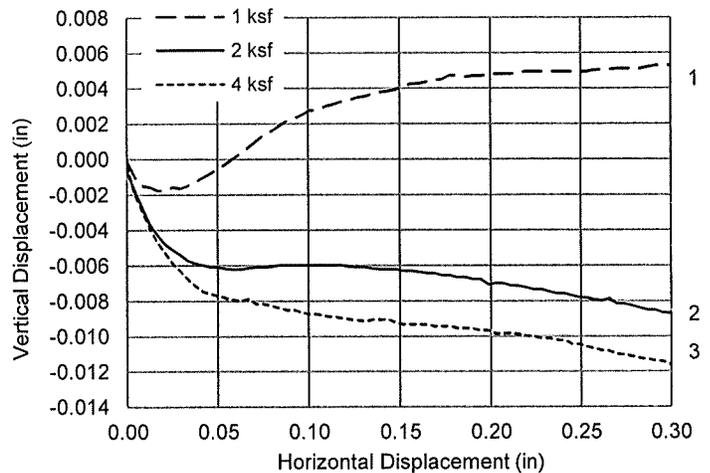EI

USCS:
GEOLOGY:
SYMBOL:
REMARKS:

CLIENT: Alliant Strategic Development, LLC
PROJECT NAME:
LOCATION: 7334 N. Topanga Canyon Blvd.
Canoga Park
SAMPLE LOCATION: B-2 @ 5'

PROJECT NO.: 30-5538-00   TESTED: 06/24/20

AGI

A.G.I. GEOTECHNICAL, INC.

Sample ID :	1 ksf	2 ksf	4 ksf			
Initial Water Content (%)	22.2	22.2	22.2			
Dry Density (%)	96.2	95.6	94.8			
Saturation (%)	79.8	78.6	77.1			
Final Water Content (%)	30.3	30.7	31.0			
Dry Density (pcf)	95.0	94.1	93.0			
Saturation (%)	105.8	104.9	103.1			
Normal Stress (psf)	1067	2134	4269			
Peak Shear Stress (psf)	1107	1610	2663			
Residual Shear Stress (psf)	1061	1472	2556			

Peak Cohesion, c' (psf):	580	Ultimate Cohesion, c (psf):	489	**DIRECT SHEAR TEST (ASTM:D-3080)**
Peak Friction, φ' (deg):	26.0	Ultimate Friction, Φ' (deg):	25.3	

SAMPLE TYPE: Undisturbed
DESCRIPTION: Sandy CLAY

LL:
PL: USCS:
PI: GEOLOGY:
% <0.75µ   SYMBOL:
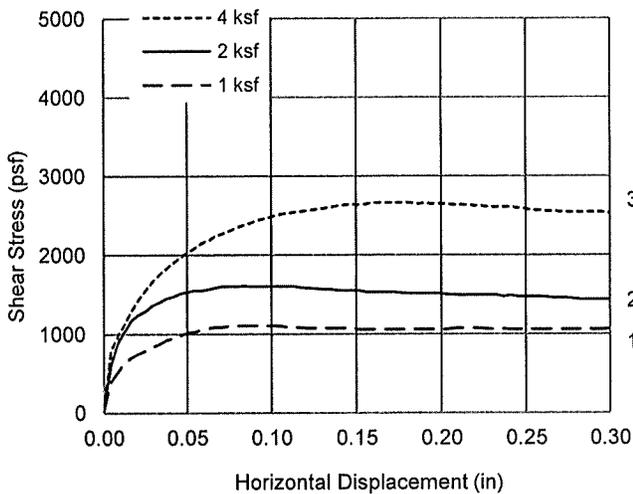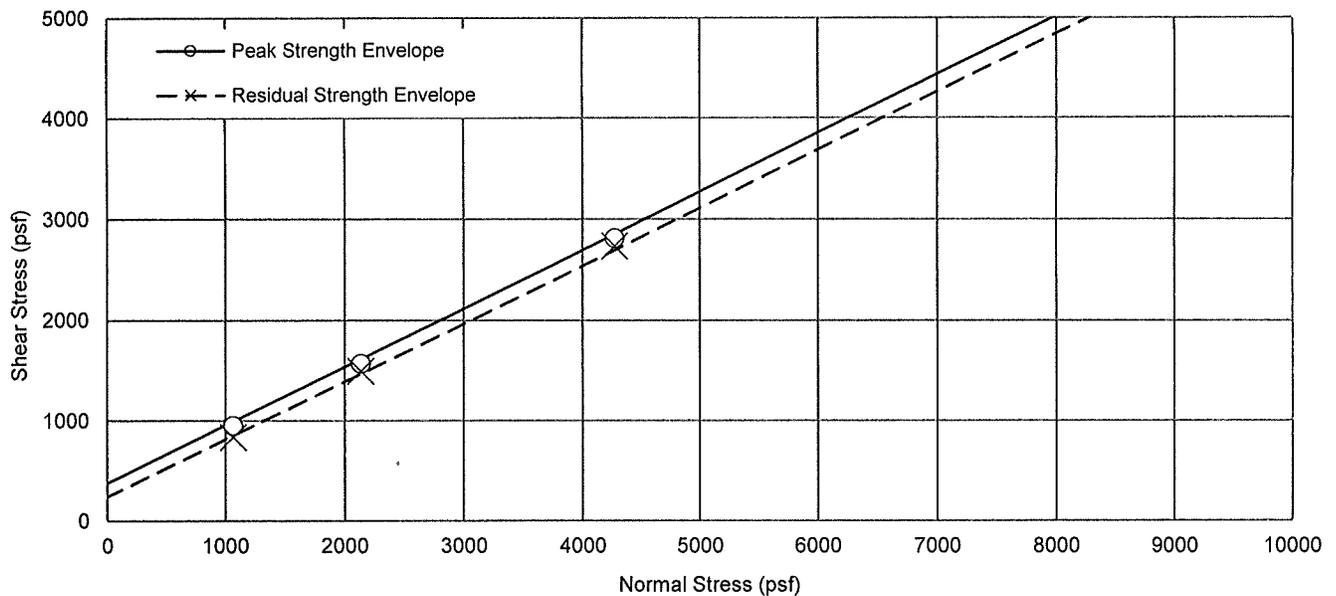% <0.02µ   REMARKS:
EI

CLIENT: Alliant Strategic Development, LLC
PROJECT NAME:
LOCATION: 7334 N. Topanga Canyon Blvd.
Canoga Park
SAMPLE LOCATION: B-2 @ 10'

PROJECT NO.: 30-5538-00   TESTED: 06/25/20

AGI

A.G.I. GEOTECHNICAL, INC.

	Sample ID :	1 ksf	2 ksf	4 ksf			
Initial	Water Content (%)	10.5	10.5	10.5			
Initial	Dry Density (%)	124.3	123.7	123.0			
Initial	Saturation (%)	79.8	78.3	76.7			
Final	Water Content (%)	14.2	14.4	14.6			
Final	Dry Density (pcf)	123.5	122.5	121.5			
Final	Saturation (%)	105.3	103.6	101.9			
	Normal Stress (psf)	1067	2134	4269			
	Peak Shear Stress (psf)	950	1568	2811			
	Residual Shear Stress (psf)	830	1490	2733			

Peak Cohesion, c' (psf):	375	Ultimate Cohesion, c (psf):	240	DIRECT SHEAR TEST (ASTM:D-3080)
Peak Friction, φ' (deg):	30.1	Ultimate Friction, Φ' (deg):	29.9	

SAMPLE TYPE: Remolded
DESCRIPTION: Silty SAND

LL:
PL:
PI:
% <0.75μ
% <0.02μ
EI
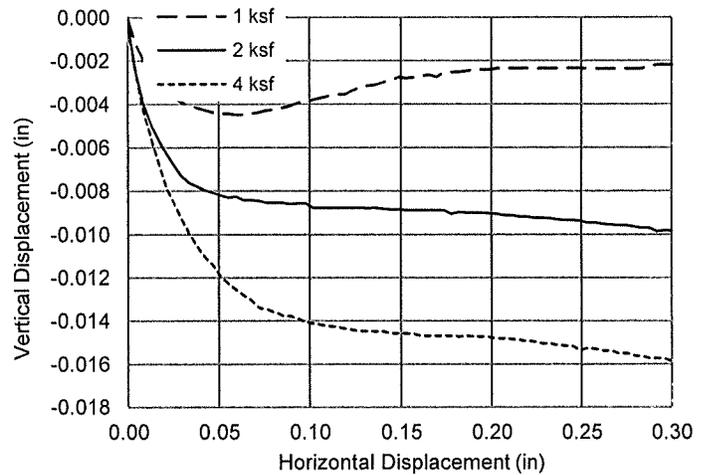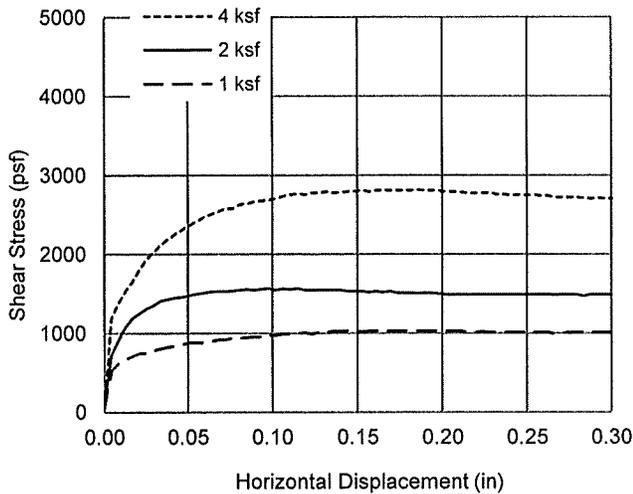
USCS:
GEOLOGY:
SYMBOL:
REMARKS:

CLIENT: Alliant Strategic Development, LLC
PROJECT NAME:
LOCATION: 7334 N. Topanga Canyon Blvd.
Canoga Park
SAMPLE LOCATION: B-1 @ 0-5'

PROJECT NO.: 30-5538-00   TESTED: 06/25/20

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A.G.I. GEOTECHNICAL, INC.

GRAIN SIZE DISTRIBUTION

PROJECT NO. 30-5538-00	**BORING NO.** B-1	**DEPTH (FT)** 0-5
Liquid Limit (%) -	**Plastic Limit (%)** -	**Plasticity Index** -
Gravel (%) 0.6	**Sand (%)** 56.6	**Silt & Clay (%)** 42.8

D_{10} (mm) - **D_{30} (mm)** - **D_{60} (mm)** - **D_{50} (mm)** -

C_u - **C_c** - **% Fines (< 75υm)** 42.8

REPRESENTATIVE FOR Alluvium

SOIL TYPE AND DESCRIPTION Silty SAND (SM)

U.S. STANDARD SIEVE SIZES

Percent Finer By Weight vs. Grain Size in Millimeters
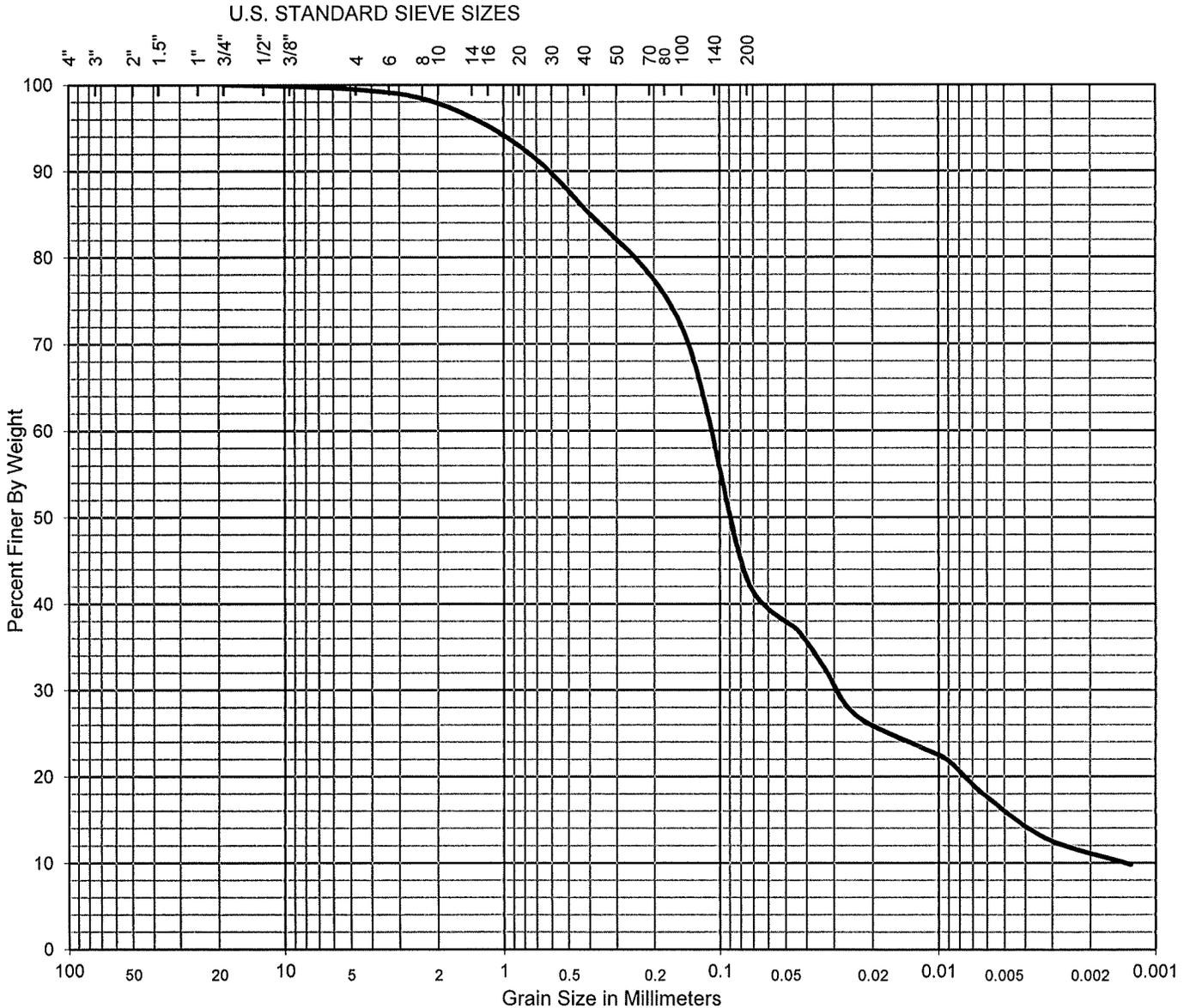
GRAVEL		SAND			SILT & CLAY
Coarse	Fine	Coarse	Medium	Fine	

A.G.I. GEOTECHNICAL, INC.

MAXIMUM DENSITY CURVE

ZERO AIR VOIDS CURVE FOR
SPECIFIC GRAVITY = 2.75

125.0 pcf
@ 10.5%

DRY DENSITY (LBS/CU FT)

MOISTURE CONTENT (% OF DRY WEIGHT)
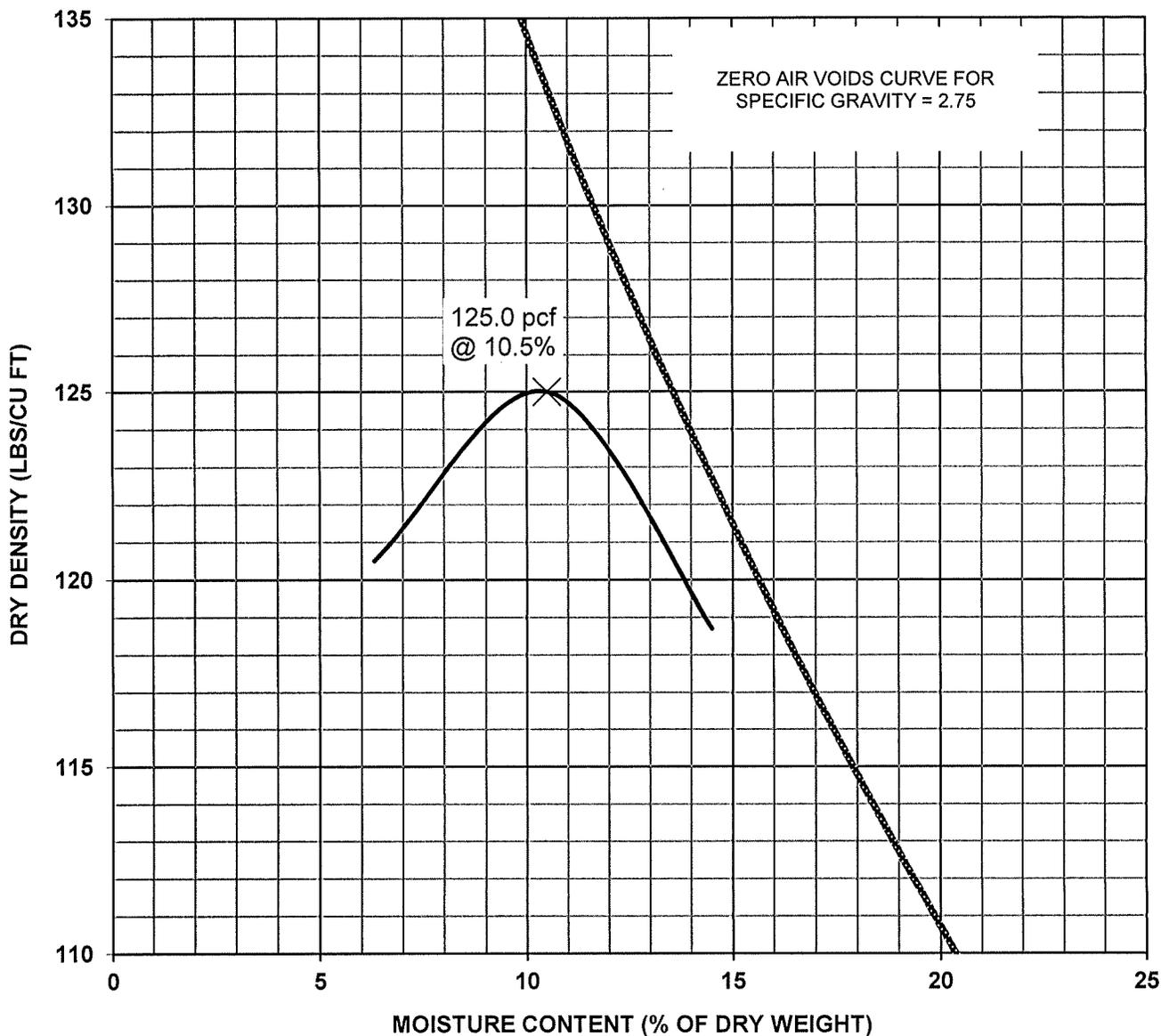
PROJECT NO. 30-5538-00   BORING NO. B-1   DEPTH (FT) 0-5

REPRESENTATIVE FOR _____ Alluvium _____
SOIL TYPE AND DESCRIPTION _____ Silty SAND (SM), (E.I. = 6, Very Low) _____

MAXIMUM DRY DENSITY (LBS/CU FT) _____ 125.0 _____
OPTIMUM MOISTURE CONTENT (% OF DRY WEIGHT) _____ 10.5 _____

METHOD OF COMPACTION
ASTM:D-1557

AGI

A.G.I. GEOTECHNICAL, INC.

U.S. SEISMIC DESIGN MAPS
USGS DEAGGREGATIONS

A.G.I. GEOTECHNICAL, INC.

OSHPD

7334 Topanga Canyon Blvd., Canoga Park

Latitude, Longitude: 34.2038, -118.6055

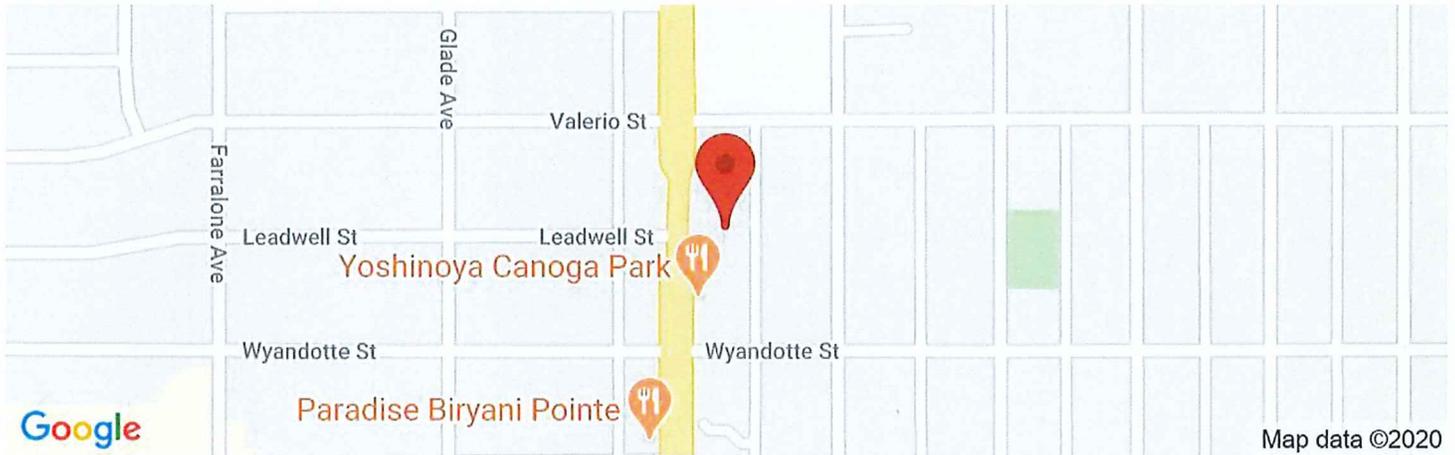| Date	6/25/2020, 11:40:05 AM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.6	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.8	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.2	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.579	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.695	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.839	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.975	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.647	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.709	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.579	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.931	Mapped value of the risk coefficient at short periods
C_{R1}	0.913	Mapped value of the risk coefficient at a period of 1 s

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

∧   Input

Edition

Dynamic: Conterminous U.S. 2014 …

Latitude
Decimal degrees

34.2038

Longitude
Decimal degrees, negative values for western longitudes

-118.6055

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon
Return period in years

475

^ Hazard Curve

Hazard Curves

Annual Frequency of Exceedence vs Ground Motion (g)

Legend:
- Time Horizon 475 years
- Peak Ground Acceleration
- 0.10 Second Spectral Acceleration
- 0.20 Second Spectral Acceleration
- 0.30 Second Spectral Acceleration
- 0.50 Second Spectral Acceleration
- 1.00 Second Spectral Acceleration
- 2.00 Second Spectral Acceleration

Uniform Hazard Response Spectrum

Ground Motion (g) vs Spectral Period (s)

Spectral Period (s): **PGA**
Ground Motion (g): **0.6172**

Component Curves for Peak Ground Acceleration

Annual Frequency of Exceedence vs Ground Motion (g)

Legend:
- Time Horizon 2475 years
- System
- Grid
- Interface

View Raw Data

⌃   Deaggregation

Component

Total

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.61722485 g

Recovered targets

Return period: 519.39729 yrs
Exceedance rate: 0.0019253085 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.19 %

Mean (over all sources)

m: 6.47
r: 14.49 km
ε_0: 1.01 σ

Mode (largest m–r bin)

m: 6.3
r: 11.66 km
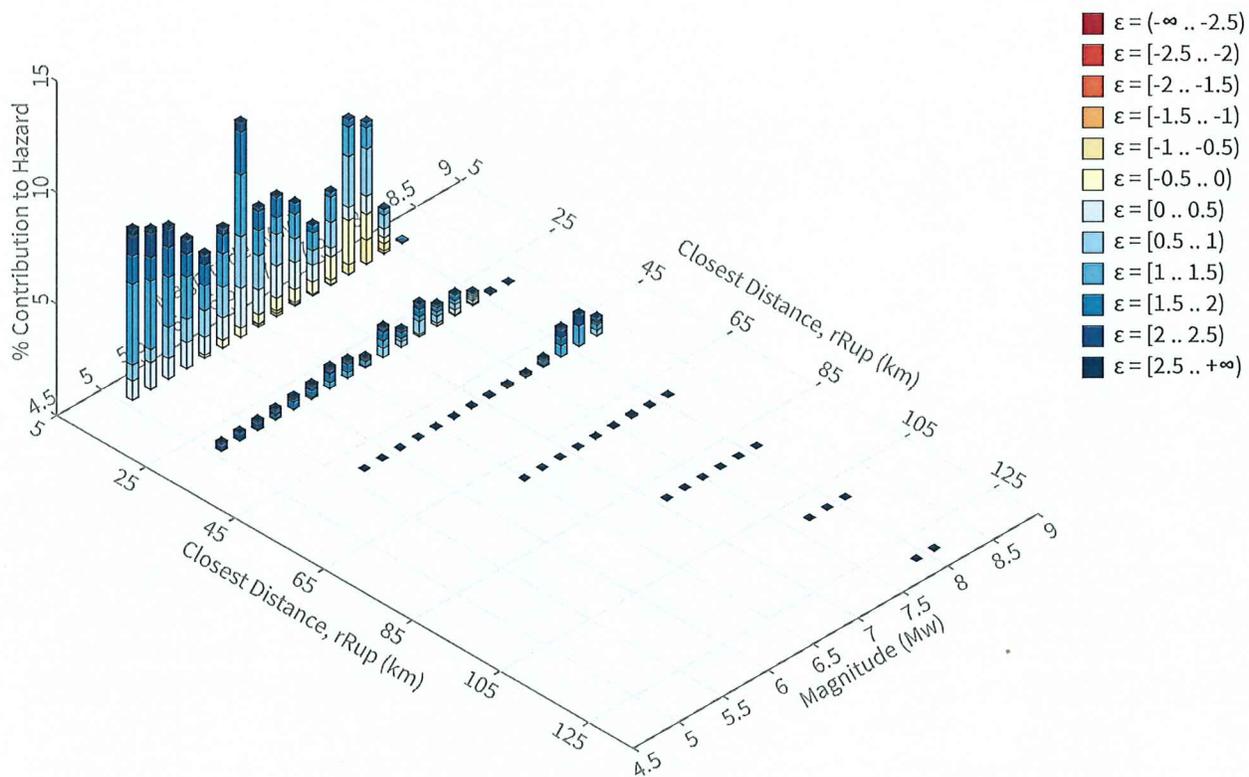**ε_0:** 1 σ
Contribution: 9.61 %

Mode (largest m–r–ε_0 bin)

m: 5.1
r: 7.71 km
ε_0: 1.32 σ
Contribution: 3.61 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							25.74
Santa Susana alt 2 [3]		12.47	7.15	0.58	118.583°W	34.314°N	9.47	6.66
Mission Hills 2011 [1]		10.01	7.14	0.45	118.556°W	34.283°N	27.33	1.94
Santa Susana East (connector) [0]		14.07	6.34	1.13	118.499°W	34.314°N	38.34	1.94
San Andreas (Mojave S) [2]		53.81	8.05	1.65	118.361°W	34.643°N	24.61	1.73
Northridge Hills [0]		9.84	7.66	0.19	118.563°W	34.284°N	23.70	1.62
Compton [4]		16.16	7.57	0.56	118.608°W	34.022°N	180.64	1.25
Northridge [2]		16.18	7.55	0.47	118.550°W	34.343°N	18.08	1.21
Simi-Santa Rosa [0]		14.63	6.87	0.91	118.698°W	34.309°N	324.19	1.14
UC33brAvg_FM31 (opt)	Grid							25.64
PointSourceFinite: -118.606, 34.208		4.85	5.66	0.68	118.606°W	34.208°N	0.00	4.75
PointSourceFinite: -118.606, 34.208		4.85	5.66	0.68	118.606°W	34.208°N	0.00	4.75
PointSourceFinite: -118.606, 34.298		10.98	5.74	1.33	118.606°W	34.298°N	0.00	1.55
PointSourceFinite: -118.606, 34.298		10.98	5.74	1.33	118.606°W	34.298°N	0.00	1.55
PointSourceFinite: -118.606, 34.334		14.37	5.77	1.58	118.606°W	34.334°N	0.00	1.30
PointSourceFinite: -118.606, 34.334		14.37	5.77	1.58	118.606°W	34.334°N	0.00	1.30
PointSourceFinite: -118.606, 34.289		10.05	5.78	1.23	118.606°W	34.289°N	0.00	1.26
PointSourceFinite: -118.606, 34.289		10.05	5.78	1.23	118.606°W	34.289°N	0.00	1.26
PointSourceFinite: -118.606, 34.280		9.08	5.85	1.10	118.606°W	34.280°N	0.00	1.22
PointSourceFinite: -118.606, 34.280		9.08	5.85	1.10	118.606°W	34.280°N	0.00	1.22
UC33brAvg_FM32 (opt)	Grid							25.56
PointSourceFinite: -118.606, 34.208		4.85	5.65	0.68	118.606°W	34.208°N	0.00	4.75
PointSourceFinite: -118.606, 34.208		4.85	5.65	0.68	118.606°W	34.208°N	0.00	4.75
PointSourceFinite: -118.606, 34.298		11.07	5.71	1.36	118.606°W	34.298°N	0.00	1.55
PointSourceFinite: -118.606, 34.298		11.07	5.71	1.36	118.606°W	34.298°N	0.00	1.55
PointSourceFinite: -118.606, 34.334		14.40	5.76	1.58	118.606°W	34.334°N	0.00	1.27
PointSourceFinite: -118.606, 34.334		14.40	5.76	1.58	118.606°W	34.334°N	0.00	1.27
PointSourceFinite: -118.606, 34.289		10.05	5.78	1.23	118.606°W	34.289°N	0.00	1.24
PointSourceFinite: -118.606, 34.289		10.05	5.78	1.23	118.606°W	34.289°N	0.00	1.24
PointSourceFinite: -118.606, 34.280		9.08	5.85	1.10	118.606°W	34.280°N	0.00	1.22
PointSourceFinite: -118.606, 34.280		9.08	5.85	1.10	118.606°W	34.280°N	0.00	1.22
UC33brAvg_FM31	System							23.06
Santa Susana alt 1 [0]		13.25	7.34	0.52	118.544°W	34.310°N	25.54	4.61
Mission Hills 2011 [1]		10.01	6.51	0.78	118.556°W	34.283°N	27.33	2.72
Northridge [2]		16.18	7.34	0.56	118.550°W	34.343°N	18.08	2.18
San Andreas (Mojave S) [2]		53.81	8.05	1.64	118.361°W	34.643°N	24.61	1.73
Northridge Hills [0]		9.84	7.66	0.19	118.563°W	34.284°N	23.70	1.60
Simi-Santa Rosa [0]		14.63	6.96	0.82	118.698°W	34.309°N	324.19	1.46
Compton [4]		16.16	7.42	0.64	118.608°W	34.022°N	180.64	1.13
Santa Susana East (connector) [0]		14.07	6.31	1.15	118.499°W	34.314°N	38.34	1.00

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

∧   Input

Edition

Dynamic: Conterminous U.S. 2014 …

Latitude
Decimal degrees

34.2038

Longitude
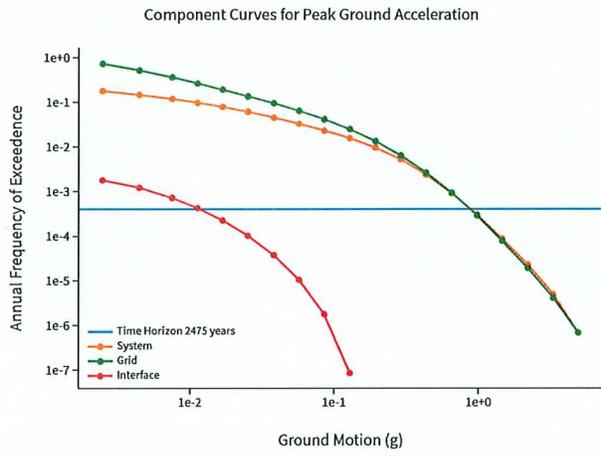Decimal degrees, negative values for western longitudes

-118.6055

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon
Return period in years

2475

∧ Hazard Curve

Hazard Curves

Annual Frequency of Exceedence vs Ground Motion (g)

Legend:
- Time Horizon 2475 years
- **Peak Ground Acceleration**
- 0.10 Second Spectral Acceleration
- 0.20 Second Spectral Acceleration
- 0.30 Second Spectral Acceleration
- 0.50 Second Spectral Acceleration
- 1.00 Second Spectral Acceleration
- 2.00 Second Spectral Acceleration

Uniform Hazard Response Spectrum

Ground Motion (g) vs Spectral Period (s)

Spectral Period (s): **PGA**
Ground Motion (g): **1.1006**

Component Curves for Peak Ground Acceleration

Annual Frequency of Exceedence vs Ground Motion (g)

Legend:
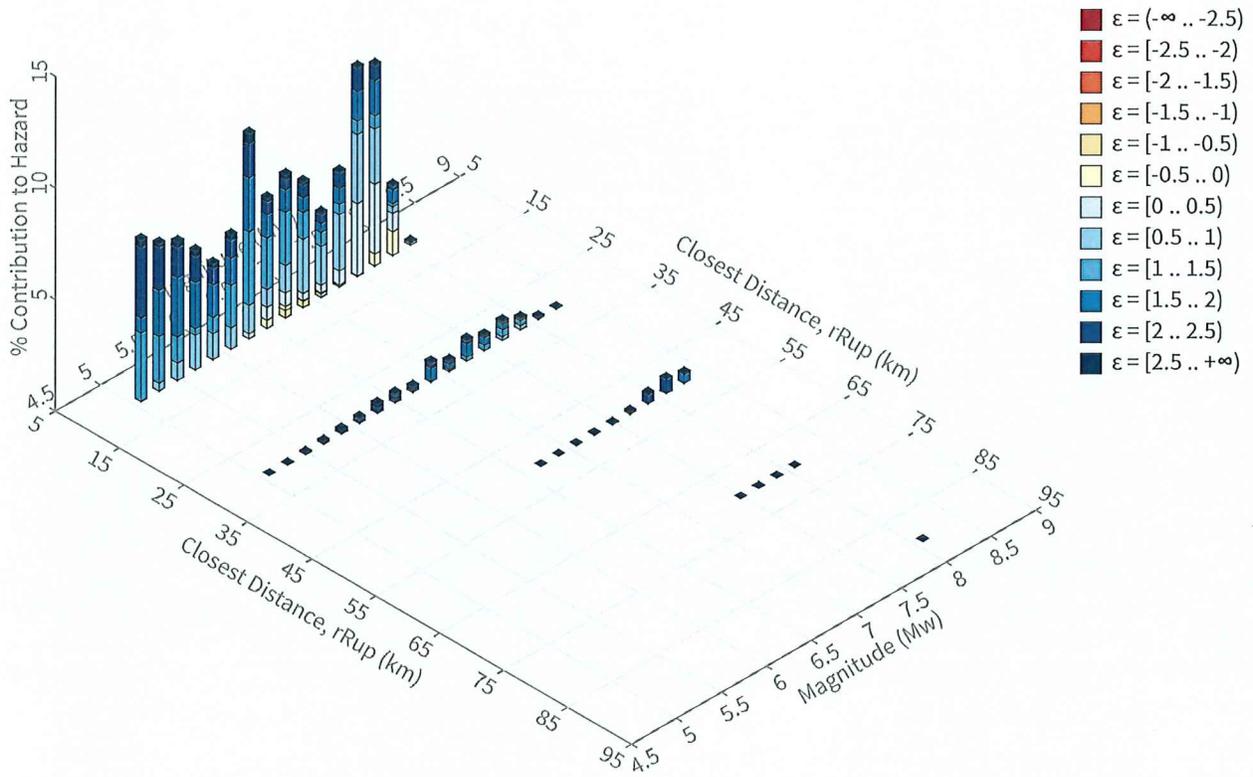- Time Horizon 2475 years
- System
- Grid
- Interface

[View Raw Data](#)

^ Deaggregation

Component

Total

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 1.1005764 g

Recovered targets

Return period: 2884.1341 yrs
Exceedance rate: 0.00034672451 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.08 %

Mean (over all sources)

m: 6.56
r: 12.09 km
ε_0: 1.34 σ

Mode (largest m-r bin)

m: 7.52
r: 13.48 km
ε_0: 0.94 σ
Contribution: 9.37 %

Mode (largest m-r-ε_0 bin)

m: 6.35
r: 9.72 km
ε_0: 1.14 σ
Contribution: 3.27 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ϵ_0	lon	lat	az	%
UC33brAvg_FM32	System							27.40
Santa Susana alt 2 [3]		12.47	7.19	1.01	118.583°W	34.314°N	9.47	8.65
Mission Hills 2011 [1]		10.01	7.18	0.83	118.556°W	34.283°N	27.33	2.74
Northridge Hills [0]		9.84	7.67	0.54	118.563°W	34.284°N	23.70	2.68
Santa Susana East (connector) [0]		14.07	6.37	1.74	118.499°W	34.314°N	38.34	1.58
Northridge [2]		16.18	7.58	1.13	118.550°W	34.343°N	18.08	1.54
Compton [4]		16.16	7.59	1.16	118.608°W	34.022°N	180.64	1.45
Simi-Santa Rosa [0]		14.63	6.93	1.35	118.698°W	34.309°N	324.19	1.17
Anacapa-Dume alt 2 [0]		14.76	7.39	1.17	118.554°W	34.031°N	166.10	1.07
UC33brAvg_FM31 (opt)	Grid							24.30
PointSourceFinite: -118.606, 34.208		4.73	5.70	1.26	118.606°W	34.208°N	0.00	6.16
PointSourceFinite: -118.606, 34.208		4.73	5.70	1.26	118.606°W	34.208°N	0.00	6.16
PointSourceFinite: -118.606, 34.298		10.60	5.83	1.72	118.606°W	34.298°N	0.00	1.24
PointSourceFinite: -118.606, 34.298		10.60	5.83	1.72	118.606°W	34.298°N	0.00	1.24
PointSourceFinite: -118.606, 34.280		8.62	5.99	1.48	118.606°W	34.280°N	0.00	1.18
PointSourceFinite: -118.606, 34.280		8.62	5.99	1.48	118.606°W	34.280°N	0.00	1.18
PointSourceFinite: -118.606, 34.289		9.60	5.90	1.60	118.606°W	34.289°N	0.00	1.11
PointSourceFinite: -118.606, 34.289		9.60	5.90	1.60	118.606°W	34.289°N	0.00	1.11
UC33brAvg_FM32 (opt)	Grid							24.23
PointSourceFinite: -118.606, 34.208		4.73	5.70	1.26	118.606°W	34.208°N	0.00	6.16
PointSourceFinite: -118.606, 34.208		4.73	5.70	1.26	118.606°W	34.208°N	0.00	6.16
PointSourceFinite: -118.606, 34.298		10.70	5.80	1.74	118.606°W	34.298°N	0.00	1.22
PointSourceFinite: -118.606, 34.298		10.70	5.80	1.74	118.606°W	34.298°N	0.00	1.22
PointSourceFinite: -118.606, 34.280		8.62	5.99	1.48	118.606°W	34.280°N	0.00	1.18
PointSourceFinite: -118.606, 34.280		8.62	5.99	1.48	118.606°W	34.280°N	0.00	1.18
PointSourceFinite: -118.606, 34.289		9.59	5.91	1.60	118.606°W	34.289°N	0.00	1.09
PointSourceFinite: -118.606, 34.289		9.59	5.91	1.60	118.606°W	34.289°N	0.00	1.09
UC33brAvg_FM31	System							24.07
Santa Susana alt 1 [0]		13.25	7.37	1.00	118.544°W	34.310°N	25.54	6.09
Mission Hills 2011 [1]		10.01	6.53	1.17	118.556°W	34.283°N	27.33	3.22
Northridge Hills [0]		9.84	7.68	0.54	118.563°W	34.284°N	23.70	2.65
Northridge [2]		16.18	7.44	1.24	118.550°W	34.343°N	18.08	2.54
Simi-Santa Rosa [0]		14.63	7.02	1.31	118.698°W	34.309°N	324.19	1.58
Compton [4]		16.16	7.45	1.28	118.608°W	34.022°N	180.64	1.22

LIQUEFACTION ANALYSES

SPT Liquefaction & Seismic Settlement Evaluation

A.G.I. GEOTECHNICAL, INC.
16555 Sherman Way
Van Nuys, CA 91406
(818) 785-5244 Fax (818) 785-6251

Project:	Alliant Strategic Dev., LLC
Job No:	30-5538-00
Boring:	B-1

Earthquake Magnitude, M :	6.47
Design PGA :	0.463
Magnitude Scaling Factor, r_m :	0.842
Factor, $\varepsilon_{C,N} / \varepsilon_{C,N=15}$:	0.750

Return Period	475	years	Lat : 34.2038
PGA_M	0.695	g	Long : -118.6055
F.O.S	1.1		

SPT N-Value Correction Factors

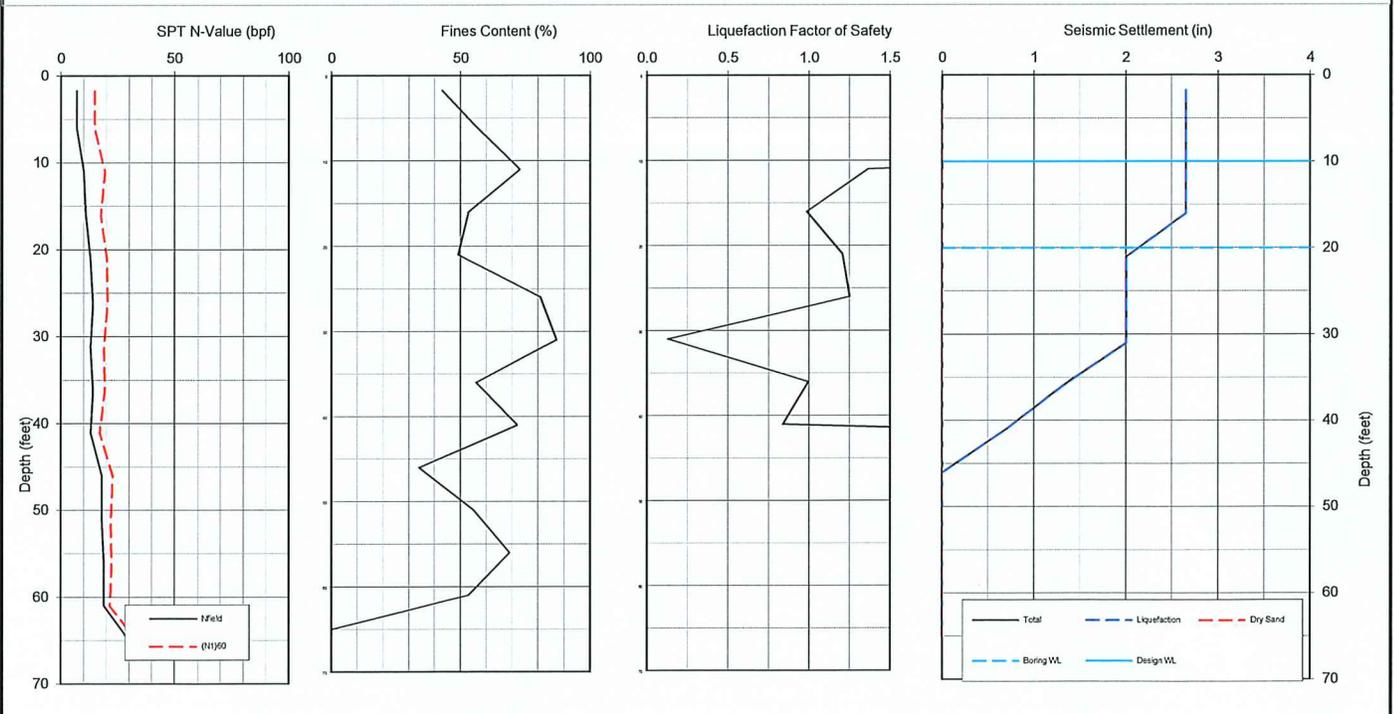|	
Energy Ratio, C_E	1.30
Borehole Diameter, C_B	1.15
Rod Length, C_R	✕
Sampler Type, C_S	1.20
Overall Correction, C_{EBS}	1.79

Boring Water Level (Below Orig), ft :	20.0
Design Water Level (Below Orig), ft :	10.0
Removal Depth (Below Orig), ft :	15.0
Surcharge Fill Height (Above Orig), ft :	0.0
Surcharge Fill Unit Weight γ, pcf :	125.0

LIQUEFACTION SETTLEMENT (in) :	2.65
DRY SAND SETTLEMENT (in) :	0.00
TOTAL SEISMIC SETTLEMENT (in) :	2.65

Layer	Layer Base, z (ft)	Total Unit Weight γ (pcf)	SPT N_{field}	Fines (%)	Incl? (Y/N)	Layer Thickness t (ft)	Layer Midheight z_o (ft)	Design Total Stress σ_o (psf)	Design Effective Stress σ_o' (psf)	Boring Effective Stress σ_b' (psf)	Overburden Correction C_N	Rod Length Corr. C_R	SPT Fines Corr $\delta(N_1)_{60}$	SPT $(N_1)_{60}$	Dry Sett $(N_1)_{60cs}$	r_d	CSR = τ_{ave}/σ_o'
1	3.5	110	7	43	Y	3.50	1.75	193	193	193	1.60	0.750	3.5	15.1	18.5	0.996	0.253
2	8.5	110	7	56	Y	5.00	6.00	660	660	660	1.60	0.750	4.2	15.1	19.3	0.988	0.251
3	13.5	112	10	73	Y	5.00	11.00	1,215	1,153	1,215	1.28	0.850	5.0	19.6	24.6	0.977	0.261
4	18.5	119	11	53	Y	5.00	16.00	1,793	1,418	1,793	1.06	0.850	4.1	17.7	21.8	0.964	0.309
5	23.5	124	13	49	Y	5.00	21.00	2,400	1,714	2,338	0.92	0.950	3.8	20.5	24.3	0.949	0.337
6	28.5	123	14	81	Y	5.00	26.00	3,018	2,019	2,643	0.87	0.950	5.3	20.8	26.1	0.930	0.353
7	33.5	123	13	87	Y	5.00	31.00	3,633	2,322	2,946	0.82	1.000	5.5	19.2	24.7	6.730	2.670
8	38.5	137	14	56	Y	5.00	36.00	4,283	2,660	3,284	0.78	1.000	4.2	19.6	23.8	0.881	0.360
9	43.5	129	13	72	Y	5.00	41.00	4,948	3,013	3,637	0.74	1.000	5.0	17.3	22.3	0.850	0.354
10	48.5	125	18	34	Y	5.00	46.00	5,583	3,336	3,960	0.71	1.000	2.9	22.9	25.8	0.815	0.346
11	53.5	125	18	55	Y	5.00	51.00	6,208	3,649	4,273	0.68	1.000	4.2	22.1	26.3	0.777	0.335
12	58.5	125	19	69	Y	5.00	56.00	6,833	3,962	4,586	0.66	1.000	4.9	22.5	27.4	0.736	0.322
13	63.5	125	19	53	Y	5.00	61.00	7,458	4,275	4,899	0.64	1.000	4.1	21.8	25.9	0.692	0.306
14	66.5	125	30	0	Y	3.00	65.00	7,958	4,526	5,150	0.62	1.000	0.0	33.5	33.5	0.657	0.293

LYR	α	β	Liq FS SPT $(N_1)_{60cs}$	K_σ	CRR_M	Liq FS	Vol Strain (%)	Liq Sett Δs (in)	Sum Liq Sett Δs (in)	Mean Stress σ_m' (psf)	G_{max} (ksf)	$\gamma_{eff}(G_{eff}/G_{max})$		γ_{eff} (%)	$\varepsilon_{C,M=7.5}$ (%)	Dry Sett Δs (in)	Sum Dry Sett Δs (in)	Sum Total Sett (in)
1	5.00	1.20	23.1	1.000	9.999	9.999	0.00	Above WL	2.65	128	600	0.000096		0.0192	0.0237	Removed	0.00	2.65
2	5.00	1.20	23.1	1.000	9.999	9.999	0.00	Above WL	2.65	440	1,125	0.000174		0.0532	0.0615	Removed	0.00	2.65
3	5.00	1.20	28.5	1.000	0.357	1.365	0.00	Removed	2.65	810	1,655	0.000216		0.0569	0.0477	Below WL	0.00	2.65
4	5.00	1.20	26.3	1.000	0.305	0.986	1.08	0.65	2.65	1,195	1,931	0.000270		0.0712	0.0697	Below WL	0.00	2.65
5	5.00	1.20	29.6	1.000	0.407	1.207	0.00	0.00	2.00	1,600	2,318	0.000296		0.0719	0.0608	Below WL	0.00	2.00
6	5.00	1.20	29.9	1.000	0.441	1.251	0.00	0.00	2.00	2,012	2,660	0.000318		0.0724	0.0555	Below WL	0.00	2.00
7	5.00	1.20	28.1	1.000	0.345	0.129	1.16	0.70	2.00	2,422	2,868	0.002567		0.0051	0.0043	Below WL	0.00	2.00
8	5.00	1.20	28.5	1.000	0.358	0.995	1.02	0.61	1.31	2,855	3,076	0.000369		0.0802	0.0695	Below WL	0.00	1.31
9	5.00	1.20	25.8	1.001	0.296	0.836	1.16	0.70	0.70	3,298	3,233	0.000392		0.0831	0.0787	Below WL	0.00	0.70
10	4.93	1.19	32.2	0.985	9.999	9.999	0.00	0.00	0.00	3,722	3,606	0.000380		0.0724	0.0564	Below WL	0.00	0.00
11	5.00	1.20	31.5	0.970	9.999	9.999	0.00	0.00	0.00	4,138	3,825	0.000380		0.0679	0.0516	Below WL	0.00	0.00
12	5.00	1.20	32.0	0.955	9.999	9.999	0.00	0.00	0.00	4,555	4,068	0.000372		0.0617	0.0442	Below WL	0.00	0.00
13	5.00	1.20	31.1	0.941	9.999	9.999	0.00	0.00	0.00	4,972	4,170	0.000373		0.0590	0.0461	Below WL	0.00	0.00
14	0.00	1.00	33.5	0.931	9.999	9.999	0.00	0.00	0.00	5,305	4,696	0.000335		0.0463	0.0235	Below WL	0.00	0.00

References: 1) Tokimatsu, K., and Seed, H. (1987). "Evaluation of Settlements in Sands Due to Earthquake Shaking." Journal of Geotechnical Engineering, ASCE, 113(8), 861-878. 2) Ishii, Y. and Tokimatsu, K. (1988). "Simplified Procedure for the Evaluation of Settlements of Structures During Earthquakes", Proceedings of Ninth World Conference on Earthquake Engineering

SPT N-Value (bpf) — Nfield, (N1)60

Fines Content (%)

Liquefaction Factor of Safety

Seismic Settlement (in) — Total, Liquefaction, Dry Sand, Boring WL, Design WL

SPT Liquefaction & Seismic Settlement Evaluation

A.G.I. GEOTECHNICAL, INC.
16555 Sherman Way
Van Nuys, CA 91406
(818) 785-5244 Fax (818) 785-6251

Project:	Alliant Strategic Dev., LLC
Job No:	30-5538-00
Boring:	B-1

Earthquake Magnitude, M :	6.56
Design PGA :	0.695
Magnitude Scaling Factor, r_m :	0.857
Factor, $\varepsilon_{C,N} / \varepsilon_{C,N=15}$:	0.774

Return Period	2475	years
PGA_M	0.695	g
F.O.S	1.0	
Lat :	34.2038	
Long :	-118.6055	

SPT N-Value Correction Factors

Energy Ratio, C_E	1.30
Borehole Diameter, C_B	1.15
Rod Length, C_R	✕
Sampler Type, C_S	1.20
Overall Correction, C_{EBS}	1.79

Boring Water Level (Below Orig), ft :	20.0
Design Water Level (Below Orig), ft :	10.0
Removal Depth (Below Orig), ft :	15.0
Surcharge Fill Height (Above Orig), ft :	0.0
Surcharge Fill Unit Weight γ, pcf :	125.0

LIQUEFACTION SETTLEMENT (in) :	4.40
DRY SAND SETTLEMENT (in) :	0.00
TOTAL SEISMIC SETTLEMENT (in) :	4.40

Layer	Layer Base, z (ft)	Total Unit Weight γ (pcf)	SPT N_{field}	Fines (%)	Incl? (Y/N)	Layer Thickness t (ft)	Layer Midheight z_o (ft)	Design Total Stress σ_o (psf)	Design Effective Stress σ_o' (psf)	Boring Effective Stress σ_b' (psf)	Overburden Correction C_N	Rod Length Corr. C_R	SPT Fines Corr $\delta(N_1)_{60}$	SPT $(N_1)_{60}$	Dry Sett $(N_1)_{60cs}$	r_d	CSR = τ_{ave} / σ_o'
1	3.5	110	7	43	Y	3.50	1.75	193	193	193	1.60	0.750	3.5	15.1	18.5	0.996	0.386
2	8.5	110	7	56	Y	5.00	6.00	660	660	660	1.60	0.750	4.2	15.1	19.3	0.988	0.383
3	13.5	112	10	73	Y	5.00	11.00	1,215	1,153	1,215	1.28	0.850	5.0	19.6	24.6	0.977	0.399
4	18.5	119	11	53	Y	5.00	16.00	1,793	1,418	1,793	1.06	0.850	4.1	17.7	21.8	0.964	0.472
5	23.5	124	13	49	Y	5.00	21.00	2,400	1,714	2,338	0.92	0.950	3.8	20.5	24.3	0.949	0.515
6	28.5	123	14	81	Y	5.00	26.00	3,018	2,019	2,643	0.87	0.950	5.3	20.8	26.1	0.930	0.538
7	33.5	123	13	87	Y	5.00	31.00	3,633	2,322	2,946	0.82	1.000	5.5	19.2	24.7	0.908	0.550
8	38.5	137	14	56	Y	5.00	36.00	4,283	2,660	3,284	0.78	1.000	4.2	19.6	23.8	0.881	0.549
9	43.5	129	13	72	Y	5.00	41.00	4,948	3,013	3,637	0.74	1.000	5.0	17.3	22.3	0.850	0.540
10	48.5	125	18	34	Y	5.00	46.00	5,583	3,336	3,960	0.71	1.000	2.9	22.9	25.8	0.815	0.528
11	53.5	125	18	55	Y	5.00	51.00	6,208	3,649	4,273	0.68	1.000	4.2	22.1	26.3	0.777	0.512
12	58.5	125	19	69	Y	5.00	56.00	6,833	3,962	4,586	0.66	1.000	4.9	22.5	27.4	0.736	0.491
13	63.5	125	19	53	Y	5.00	61.00	7,458	4,275	4,899	0.64	1.000	4.1	21.8	25.9	0.692	0.468
14	66.5	125	30	0	Y	3.00	65.00	7,958	4,526	5,150	0.62	1.000	0.0	33.5	33.5	0.657	0.447

LYR	α	β	Liq FS SPT $(N_1)_{60cs}$	K_σ	CRR_M	Liq FS	Vol Strain (%)	Liq Sett Δs (in)	Sum Liq Sett Δs (in)	Mean Stress σ_m' (psf)	G_{max} (ksf)	$\gamma_{eff}(G_{eff}/G_{max})$		γ_{eff} (%)	$\varepsilon_{C,M=7.5}$ (%)	Dry Sett Δs (in)	Sum Dry Sett Δs (in)	Sum Total Sett (in)
1	5.00	1.20	23.1	1.000	9.999	9.999	0.00	Above WL	4.40	128	600	0.000145		0.1334	0.1585	Removed	0.00	4.40
2	5.00	1.20	23.1	1.000	9.999	9.999	0.00	Above WL	4.40	440	1,125	0.000262		0.1765	0.1965	Removed	0.00	4.40
3	5.00	1.20	28.5	1.000	0.357	0.894	1.03	Removed	4.40	810	1,655	0.000324		0.1647	0.1335	Below WL	0.00	4.40
4	5.00	1.20	26.3	1.000	0.305	0.646	1.37	0.82	4.40	1,195	1,931	0.000404		0.2177	0.2048	Below WL	0.00	4.40
5	5.00	1.20	29.6	1.000	0.407	0.791	1.17	0.70	3.58	1,600	2,318	0.000444		0.2078	0.1691	Below WL	0.00	3.58
6	5.00	1.20	29.9	1.000	0.441	0.819	1.04	0.63	2.87	2,012	2,660	0.000477		0.2010	0.1487	Below WL	0.00	2.87
7	5.00	1.20	28.1	1.000	0.345	0.627	1.16	0.69	2.25	2,422	2,868	0.000519		0.2134	0.1696	Below WL	0.00	2.25
8	5.00	1.20	28.5	1.000	0.358	0.652	1.23	0.74	1.55	2,855	3,076	0.000554		0.2173	0.1815	Below WL	0.00	1.55
9	5.00	1.20	25.8	1.001	0.296	0.547	1.36	0.81	0.81	3,298	3,233	0.000588		0.2220	0.2025	Below WL	0.00	0.81
10	4.93	1.19	32.2	0.985	9.999	9.999	0.00	0.00	0.00	3,722	3,606	0.000570		0.1832	0.1382	Below WL	0.00	0.00
11	5.00	1.20	31.5	0.970	9.999	9.999	0.00	0.00	0.00	4,138	3,825	0.000570		0.1672	0.1234	Below WL	0.00	0.00
12	5.00	1.20	32.0	0.955	9.999	9.999	0.00	0.00	0.00	4,555	4,068	0.000558		0.1478	0.1032	Below WL	0.00	0.00
13	5.00	1.20	31.1	0.941	9.999	9.999	0.00	0.00	0.00	4,972	4,170	0.000559		0.1393	0.1060	Below WL	0.00	0.00
14	0.00	1.00	33.5	0.931	9.999	9.999	0.00	0.00	0.00	5,305	4,696	0.000503		0.1052	0.0523	Below WL	0.00	0.00

References: 1) Tokimatsu, K., and Seed, H. (1987). "Evaluation of Settlements in Sands Due to Earthquake Shaking." Journal of Geotechnical Engineering, ASCE, 113(8), 861-878. 2) Ishii, Y. and Tokimatsu, K. (1988). "Simplified Procedure for the Evaluation of Settlements of Structures During Earthquakes", Proceedings of Ninth World Conference on Earthquake Engineering
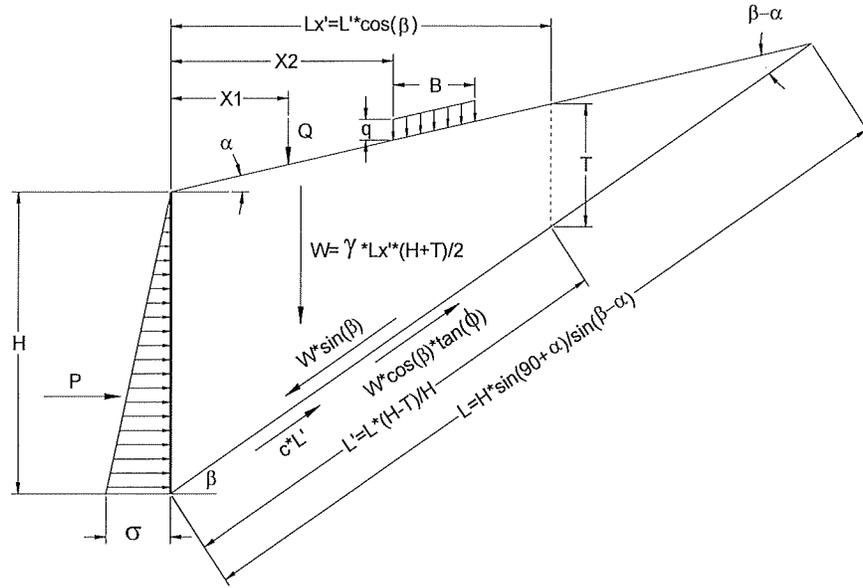

SPT N-Value (bpf) · Fines Content (%) · Liquefaction Factor of Safety · Seismic Settlement (in)
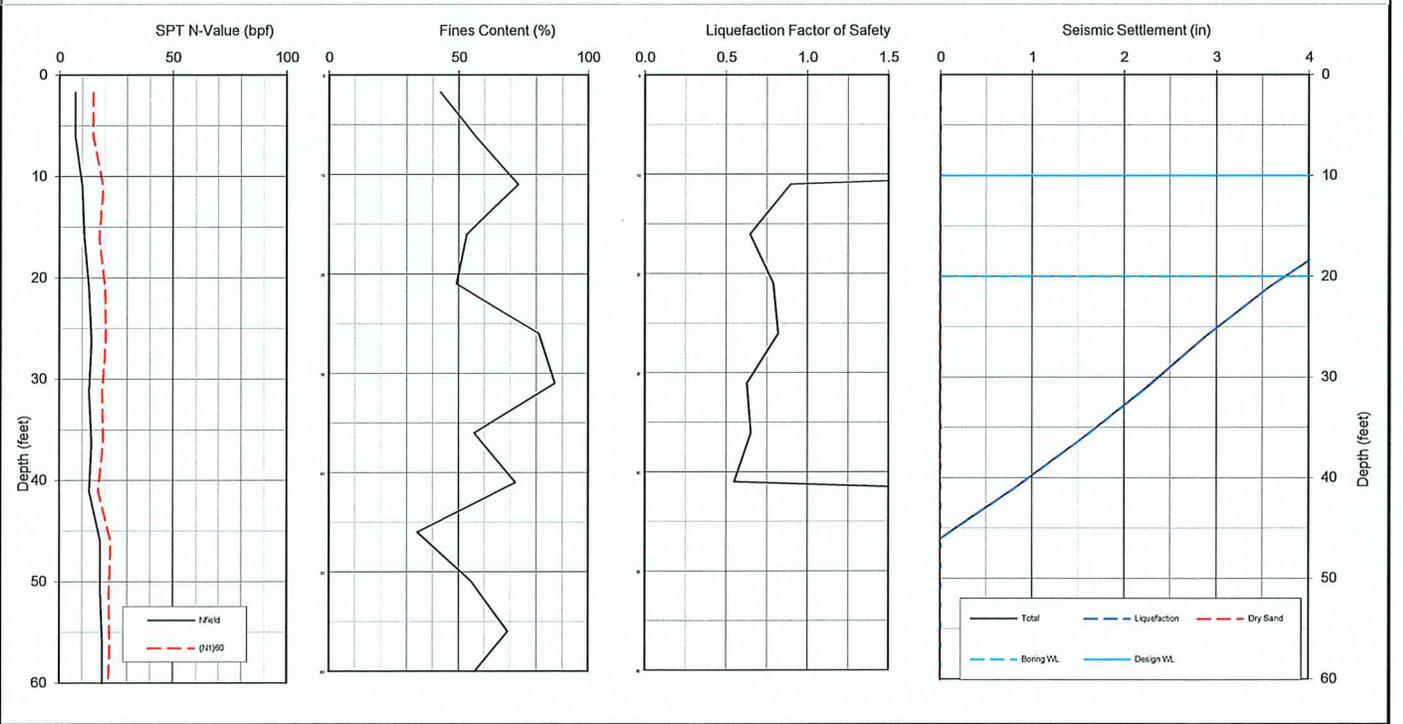
ACTIVE EARTH PRESSURE

ANALYSES

ACTIVE EFP FROM SLIDING WEDGE ANALYSIS
CANTILEVERED WALL

$Lx' = L' \cdot \cos(\beta)$

$\beta - \alpha$

X2

X1

B

α

Q

q

$W = \gamma \cdot Lx' \cdot (H+T)/2$

T

H

P

$W \cdot \sin(\beta)$

$W \cdot \cos(\beta) \cdot \tan(\phi)$

$L = H \cdot \sin(90+\alpha)/\sin(\beta-\alpha)$

$c \cdot L'$

$L' = L \cdot (H-T)/H$

β

σ

Input Description	Value
Wall Height, H (ft)	15.0
Back Slope Angle, α (deg)	0.0
Line Load, Q (plf)	0
Line Load Distance, X1 (ft)	0
Strip Load, q (psf)	300
Strip Load Distance, X2 (ft)	0.0
Strip Load Width, B (ft)	10.0
Unit Weight, γ (pcf)	110
Cohesion, c (psf)	364
Friction Angle, ϕ (deg)	28.1
Horizontal Seismic Load, k_h (g)	0.00
Vertical Seismic Load, k_v (g)	0.00
Required Factor of Safety, FS	1.50

——— Static Only

— — — Static+Seismic

- - - - - - Static+Seismic+Surcharge

Output Description	Value
Static-Critical Failure Angle, β (deg)	52.0
Total Failure Length, L	19.0
Maximum Reaction, P (lb)	1851
Maximum Equivalent Fluid Pressure, σ (pcf)	16.4
Equivalent Fluid Pressure Coefficient, Ka	0.150
Static+Seismic-Critical Failure Angle, β (deg)	52.0
Total Failure Length, L	19.0
Maximum Reaction, P (lb)	1851
Maximum Equivalent Fluid Pressure, σ (pcf)	16.4
Equivalent Fluid Pressure Coefficient, Ka	0.150
Static+Seismic+Surcharge-Critical β (deg)	52.9
Total Failure Length, L	18.8
Maximum Reaction, P (lb)	2945
Maximum Equivalent Fluid Pressure, σ (pcf)	26.2
Equivalent Fluid Pressure Coefficient, Ka	0.238
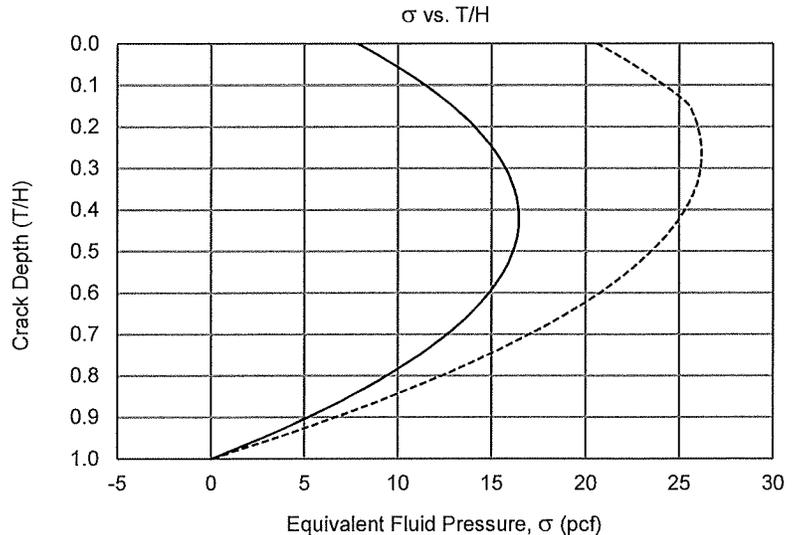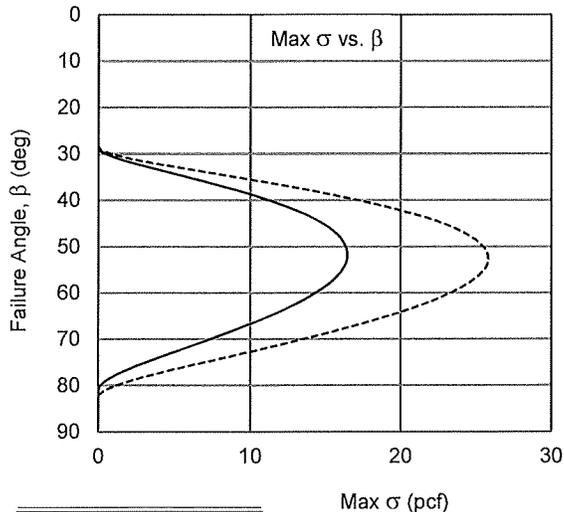
Max σ vs. β

Failure Angle, β (deg)

Max σ (pcf)

σ vs. T/H

Crack Depth (T/H)

Equivalent Fluid Pressure, σ (pcf)

Proj No. : 30-5538-00	Date: July 2020
Proj Name: 7334 N. Topanga Canyon Blvd.	
Calc. By: MBS	

ACTIVE EFP FROM SLIDING WEDGE ANALYSIS
TEMPORARY SHORING

Diagram labels:
$Lx'=L'*cos(\beta)$, $X2$, $X1$, B, $\beta-\alpha$, α, Q, q, T, $W=\gamma*Lx'*(H+T)/2$, $W*sin(\beta)$, $W*cos(\beta)*tan(\phi)$, $L=H*sin(90+\alpha)/sin(\beta-\alpha)$, $c*L'$, $L'=L*(H-T)/H$, H, P, β, σ

Input Description	Value
Wall Height, H (ft)	15.0
Back Slope Angle, α (deg)	0.0
Line Load, Q (plf)	0
Line Load Distance, X1 (ft)	0
Strip Load, q (psf)	300
Strip Load Distance, X2 (ft)	0.0
Strip Load Width, B (ft)	15.0
Unit Weight, γ (pcf)	110
Cohesion, c (psf)	364
Friction Angle, ϕ (deg)	28.1
Horizontal Seismic Load, k_h (g)	0.00
Vertical Seismic Load, k_v (g)	0.00
Required Factor of Safety, FS	1.25

Output Description	Value
Static-Critical Failure Angle, β (deg)	54.7
Total Failure Length, L	18.4
Maximum Reaction, P (lb)	1072
Maximum Equivalent Fluid Pressure, σ (pcf)	9.5
Equivalent Fluid Pressure Coefficient, Ka	0.087
Static+Seismic-Critical Failure Angle, β (deg)	54.7
Total Failure Length, L	18.4
Maximum Reaction, P (lb)	1072
Maximum Equivalent Fluid Pressure, σ (pcf)	9.5
Equivalent Fluid Pressure Coefficient, Ka	0.087
Static+Seismic+Surcharge-Critical β (deg)	54.7
Total Failure Length, L	18.4
Maximum Reaction, P (lb)	1952
Maximum Equivalent Fluid Pressure, σ (pcf)	17.4
Equivalent Fluid Pressure Coefficient, Ka	0.158

Legend:
— Static Only
— — Static+Seismic
----- Static+Seismic+Surcharge

Max σ vs. β — Failure Angle, β (deg) vs Max σ (pcf)

σ vs. T/H — Crack Depth (T/H) vs Equivalent Fluid Pressure, σ (pcf)

AT-REST EARTH PRESSURE ANALYSIS

ANALYSIS

AT-REST EARTH PRESSURE FROM JAKY'S Ko EQUATION

TRIANGULAR

TRAPEZOIDAL
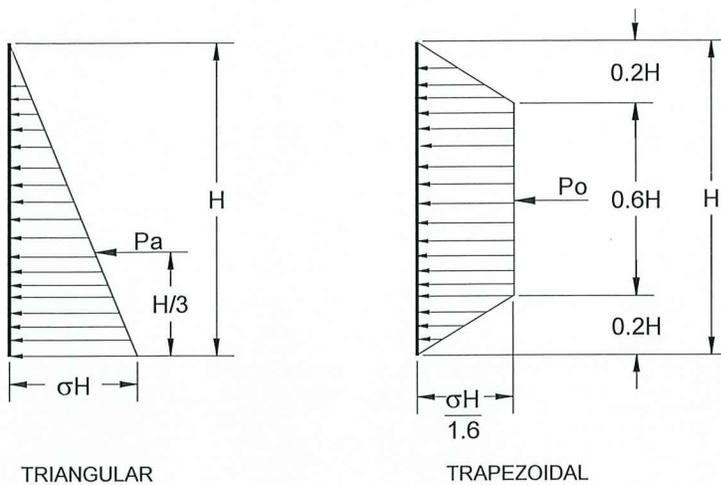
UNIT WEIGHT, γ (lb/ft^2)	110.0	
FRICTION ANGLE, ϕ ' (deg)	28.1	
JAKY'S Ko, σ (lb/ft^2)		
$\sin \phi$	0.471	
Ko	0.473	
σH (lb/ft^2)	52	(triangular)
σH/1.6 (lb/ft^2)	32	xH (trapezoidal)

Normally Consolidated Soils, Jaky's K_o Equation

In 1944 J. Jaky's paper "The Coefficient of Earth Pressure at Rest" presented his theoretical derivation of K_o:

$$K_o = (1 - \sin \varphi')\frac{\left(1 + \frac{2}{3}\sin \varphi'\right)}{(1 + \sin \varphi')}$$

where φ' is the effective angle of internal friction. The above equation can be simplified to the following approximation:

$$K_o = (1 - \sin \varphi').$$

The difference in the calculated values is shown in Fig 3, and ranges from 9 percent at low friction angles to 16 percent at high friction angles. However, "considering the difficulty of making an appropriate choice for φ' for a given soil, this approximation is sufficiently accurate for most engineering purposes" (Wroth, 1972).

Fig 3: Comparison of Several K_o Equations for Normally Consolidated Soils

SLOT CUT STABILITY ANALYSIS

SLOT CUT STABILITY ANALYSIS

Soil:
γ, c, ϕ
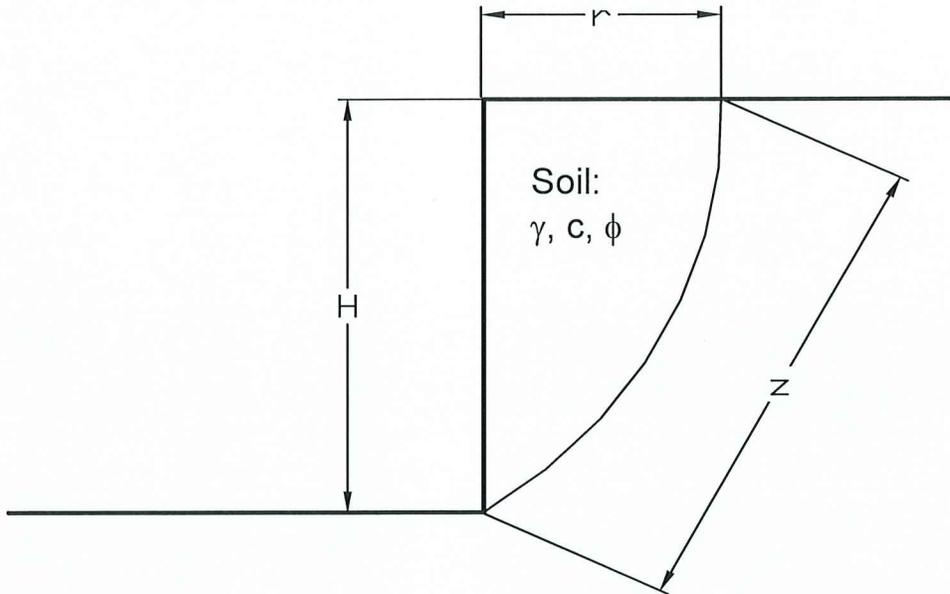
Description	Value
Unit Weight, γ (pcf)	110.0
Friction, ϕ (deg)	28.1
Cohesion, c (psf)	364

Description	Value
Cut Height, H (ft)	12.0
Failure Radius, r (ft)	4.0
Failure Width, B = 2r (ft)	8.0

Description	Value
Volume, $V = \pi r^2 H / 4$ (ft^3)	151
Weight, $W = V\gamma$ (lb)	16,610
Surcharge, Q (lb)	8,000
Weight+Surcharge, W + Q, (lb)	24,610

Description	Value
Surface Area, $A = 0.5236r\,((r^2+4H^2)^{3/2} - r^3)$ (ft^2)	104
Driving Force, $F_D = WH / (r^2+H^2)^{1/2}$ (lb)	23,347
Normal Force, $F_N = Wr / (r^2+H^2)^{1/2}$ (lb)	7,782
Frictional Resistance, $R_F = F_N \tan\phi$ (lb)	4,155
Cohesive Resistance, $R_C = A\,c$ (lb)	37,856
Total Resistance, $R = R_F + R_C$ (lb)	42,011
Factor of Safety, $FS = R / F_D$	1.80

INFORMATION BULLETIN
P/BC 2020-083

RETAINING WALL DESIGN

This information bulletin provides general criteria for design of retaining walls. In particular, guidelines include:

- Minimum static design earth pressures retaining level and sloping ground;
- Vertical surcharge loads on walls;
- Seismic lateral earth pressure on retaining walls; and,
- Acceptable engineering criteria for retaining wall design.

Alternative design procedures justified in a geotechnical report may also be approved.

Design of retaining walls as presented in this Bulletin are in accordance with Sections 1610.1 and 1807.2 of the City of Los Angeles Building Code (LABC).

I.   SOIL LATERAL LOADS

LABC 1610.1 General. Foundation walls and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless determined otherwise by a geotechnical investigation in accordance with Section 1803. Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of un-drained backfill unless a drainage system is installed in accordance with Sections 1805.4.2 and 1805.4.3.

Exception: Foundation walls extending not more than 8 feet (2438 mm) below grade and laterally supported at the top by flexible diaphragms shall be permitted to be designed for active pressure.

Maximum values presented in Table 1610.1 shall be used for design, unless a geotechnical investigation determines the type of material retained or justifies lower values or both.

Table 1610.1 does not provide design lateral soil loads for retaining sloping ground. Therefore, a geotechnical investigation report shall be provided when walls will retain sloping ground.

II. RETAINING WALL DESIGN

LABC 1807.2.2 Design lateral soil loads. Retaining walls shall be designed for the lateral soil loads set forth in Section 1610.

LABC 1807.2.3 Safety factor. Retaining walls shall be designed to resist the lateral action of soil to produce sliding and overturning with minimum safety factor of 1.5 in each case. The load combinations of Section 1605 shall not apply to this requirement. Instead, design shall be based on 0.7 times nominal earthquake loads, 1.0 times other nominal loads, and investigation with one or more of the variable loads set to zero. The safety factor against lateral sliding shall be taken as the available soil resistance at the base of the retaining wall foundation divided by the net lateral force applied to the retaining wall.

Exception: Where earthquake loads are included, the minimum safety factor for retaining wall sliding and overturning shall be 1.1.

III. MINIMUM DESIGN STATIC ACTIVE LATERAL EARTH PRESSURES FOR RETAINING WALLS SUPPORTING LEVEL AND SLOPING GROUND WHEN A GEOTHECNICAL INVESTIGATION REPORT IS PROVIDED

The design static active equivalent fluid pressure (EFP) for walls that retain drained earth[1] when a geotechnical investigation report is provided shall not be less than the values shown in Table 1. The horizontal resultant force is determined as illustrated in Figure 1. A vertical component equal to one third of the horizontal force so obtained may be assumed at the plane of contact between the retained soil and wall surface when considering the total resisting moment taken at the toe of the wall. Such a vertical component is not permitted when filter fabric is used behind retaining walls.

The depth of the retained earth shall be the vertical distance below the ground surface measured at the wall face of stem design or measured at the heel of the footing for overturning and sliding.

TABLE 1 Minimum Static Equivalent Fluid Pressures

Surface Slope of Retained Material* Horizontal (H) to Vertical (V)	Equivalent Fluid Pressure γ_{EFP} (pounds per cubic foot, pcf)
LEVEL (0° angle)	30
5 to 1	32
4 to 1	35
3 to 1	38
2 to 1	43
1.5 to 1	55
1 to 1 (45° angle)	80

[1] Drainage system shall be installed in accordance with LABC Section 1805.4.2 and 1805.4.3.

* Where the surface slope of the retained earth varies, the design slope shall be obtained by connecting a line from the top of the wall to the highest point on the slope whose limits are within the horizontal distance from the stem equal to the stem height of the wall.
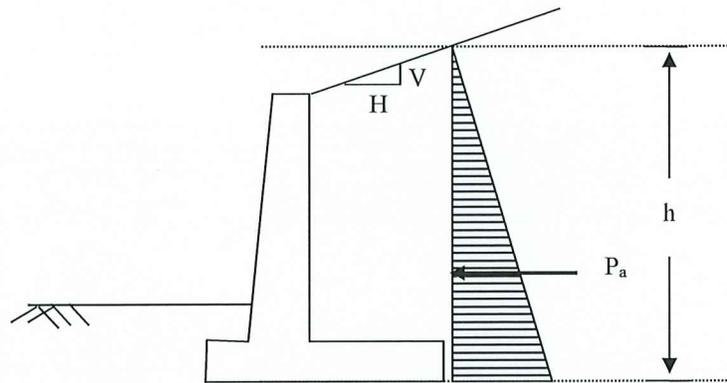

Figure 1 – Horizontal Resultant Force

$$P_a = 0.5 * \gamma_{EFP} * h^2 \ (in\ pounds); \qquad Equation\ 1$$

$$applied\ at \frac{1}{3}h\ measured\ from\ bottom\ of\ wall\ footings$$

IV.   METHODS OF DETERMINING VERTICAL SURCHARGE LOADS ON WALLS

Any superimposed vertical loading, except retained earth, shall be considered as surcharge and provided for in the design. Uniformly distributed loads may be considered as equivalent added depth of retained earth. Surcharge loading due to continuous or isolated footings can be determined by Equations 2 and 3, and as illustrated in Figure 2, or by an equivalent method approved by the Superintendent of Building. Equation 2 is limited to retaining walls that are permitted to be designed for active pressure[2]. This method shall also be limited to the design of retaining walls only under vertical surcharge. Retaining walls under lateral surcharge shall be designed by licensed civil/structural engineer with approval from the Department. The Superintendent of Building may require a site-specific geotechnical investigation prior to approving a permit for such a wall.

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[2] Per LABC section 1610.1: Retaining walls free to move and rotate at the top shall be permitted to be designed for active pressure.
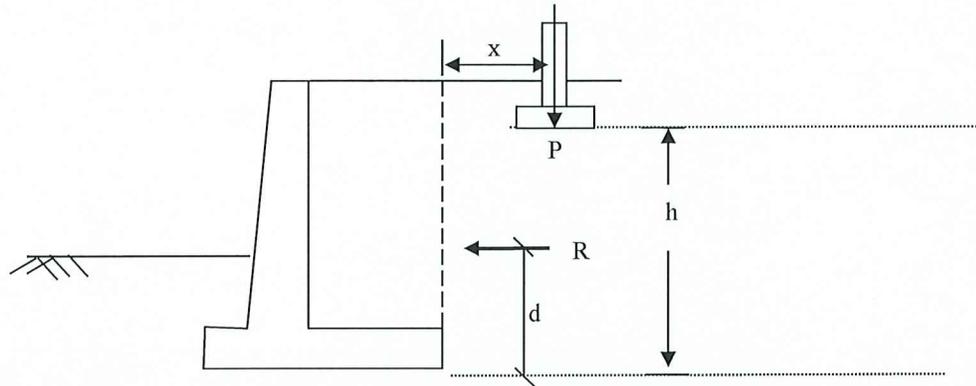

Figure 2 - Vertical Surcharge Loads

Resultant lateral force:

$$R = \frac{0.3\,P\,h^2}{x^2 + h^2};\qquad Equation\ 2$$

Location lateral resultant:

$$d = x\left[\left(\frac{x^2}{h^2} + 1\right)\left(tan^{-1}\frac{h}{x}\right) - \left(\frac{x}{h}\right)\right];\qquad Equation\ 3$$

Where:

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

Loads applied within a horizontal distance equal to the wall height (i.e. x ≤ h), measured from the back face of the wall footings, shall be considered as surcharge.

For isolated footings that have a width parallel to the wall less than 3 feet, "R" may be reduced to one-sixth the calculated value.

The resultant lateral force "R" shall be assumed to be uniform for the length of footing parallel to the wall and to diminish uniformly to zero at the distance "x" beyond the ends of the footing, as shown in Figure 3.

Vertical pressure due to surcharge applied to the top of the wall footing may be considered to spread uniformly within the limits of the stem and planes making an angle of 45 degrees with the vertical, as shown in Figure 3.
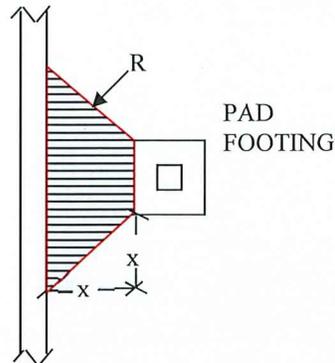

Figure 3 - Vertical Surcharge Loads, Plan View

Guidelines for determining live loads surcharge from sidewalk pedestrian traffic and stret traffic are provided in the Information Bulletin P/BC 2020-141.

V. METHOD FOR DETERMINING SEISMIC LATERAL EARTH PRESSURE ON RETAINING WALLS

Section **1803.5.12** of the LABC specifies that for Seismic Design Categories D through F, retaining walls supporting more than 6 feet of backfill shall be designed for seismic lateral earth pressures due to design earthquake ground motions.

The seismic lateral earth pressure for walls retaining level ground can be calculated using the Equation 4, based on Seed and Whitman (1970)[3]:

$$\gamma_{EFP\ (seismic)} = \frac{3}{4} k_h \gamma_{soil}; \qquad Equation\ 4$$

Where:

$\gamma_{EFP\ (seismic)}$ is the seismic increment expressed as equivalent fluid pressure (pcf);

k_h is the seismic lateral earth pressure coefficient equivalent to one-half of two-thirds of PGA$_M$;

γ_{soil} is the unit weight of the retained soils, may be taken as 120 pcf without a soils report.

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[3] Seed, H.B. and Whitman, R.V., 1970, Design of Earth Retaining Structures for Dynamic Loads, *ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures,* pp 103-147.

The seismic lateral earth pressure shall be applied in addition to the static lateral earth pressure, and can be applied assuming an inverted triangular distribution, with the resultant applied at a height of 2/3 h measured from the bottom of wall footings.

Example: For a site located at 201 N. Figueroa St, for Site Class C, the PGA$_M$ is 0.94g. The seismic lateral earth pressure can be calculated as the following:

$$\gamma_{EFP\ (seismic)} = \frac{3}{4}k_h\gamma_{soil} = \frac{3}{4} \times \frac{1}{2} \times \frac{2}{3} \times 0.94 \times 120pcf = 28.2\ pcf;$$

VI.   ACCEPTABLE ENGINEERING CRITERIA FOR RETAINING WALL DESIGN

LABC 1807.2.1 Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift.

a.   Bearing Pressure and Overturning

Minimum values presented in LABC Table 1806.2[4] shall be used for design, unless a geotechnical investigation determines the type of material for foundation support or justifies higher load-bearing values or both. The resultant of vertical loads and lateral pressures shall pass through the middle one third of the base.

b.   Lateral Pressures

Retaining walls shall be restrained against sliding by lateral sliding resistance of the base against the earth, by lateral bearing pressure against the soil, or by a combination of the two[5]. Minimum values presented in LABC Table 1806.2 shall be used for design, unless a geotechnical investigation determines the type of material for lateral bearing and lateral sliding resistance or justifies higher allowable lateral bearing and lateral sliding resistance values or both.

When used, keys shall be assumed to lower the plane of lateral sliding resistance and the depth of lateral bearing to the level of the bottom of the key. Lateral bearing pressures shall be assumed to act on a vertical plane located at the toe of the footing.

VII.   SPECIAL CONDITION

The Superintendent of Building may require a site-specific soil investigation before approving any permit for a retaining wall whenever, the following exist: the adequacy of the foundation material to support a wall is questionable; an unusual surcharge condition exists such as
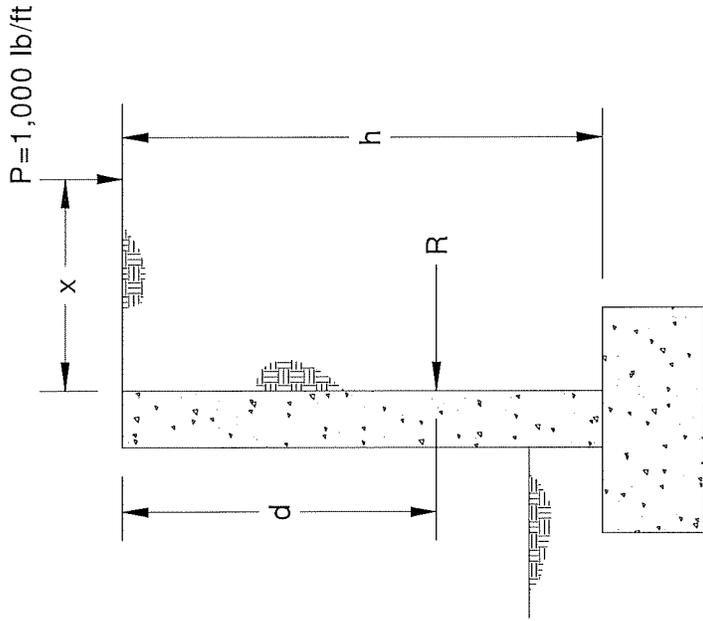
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[4] Per LABC 1806.2: Mud, organic silt, organic clay, peat or **unprepared fill** shall not be assumed to have a presumptive load-bearing capacity.
[5] Reference code section LABC 1806.3.1.

seepage pressure; or when the retained earth is so stratified or of such a character as to invalidate normal design assumptions..

Additionally, the footings for all retaining walls shall extend a minimum of 24 inches below the natural and finish grades in accordance with the requirements contained in Information Bulletin P/BC 2020-116 for expansive soils conditions unless a soil report indicates expansive soils do not exist at the site.

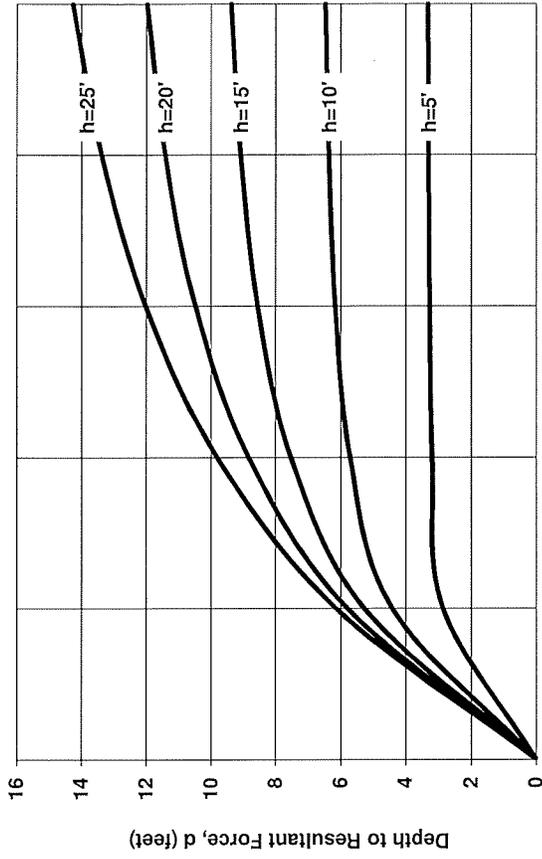# LATERAL SURCHARGE DUE TO ADJACENT 1000 LB/FT LINE LOAD (RESTRAINED WALL)*

Top chart (Depth to Resultant Force):

Depth to Resultant Force, d (feet) — vertical axis: 0, 2, 4, 6, 8, 10, 12, 14, 16

Curves labeled: h=25', h=20', h=15', h=10', h=5'

Bottom chart (Lateral Surcharge Resultant):

Lateral Surcharge Resultant, R (lb/ft) — vertical axis: 0, 50, 100, 150, 200, 250, 300

Distance From Wall, x (feet) — horizontal axis: 0, 5, 10, 15, 20, 25

Curves labeled: h=25', h=20', h=15', h=10', h=5'

P=1,000 lb/ft

Resultant Lateral Force: $R = \dfrac{0.3Ph}{x^2 + h^2}$
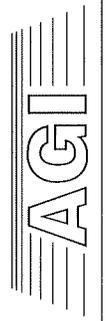
Location Lateral Resultant:

$d = x \left[\left(\dfrac{x^2}{h^2} + 1 \right) \left(\tan^{-1} \dfrac{h}{x} \right) - \left(\dfrac{x}{h} \right) \right]$

*NOTE: Use 1/2 chart values for unrestrained wall or shoring
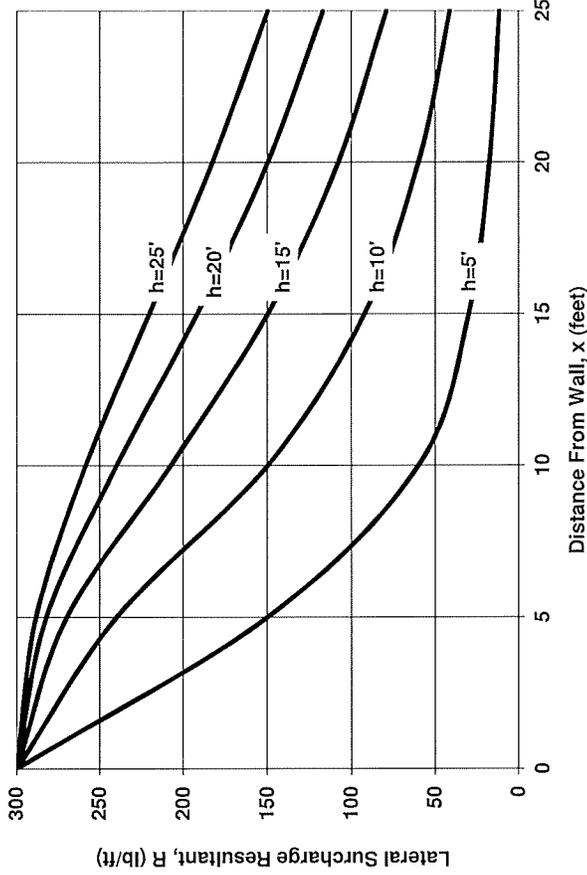
INFORMATION BULLETIN
P/BC 2017-141

A.G.I. GEOTECHNICAL, INC.

CITY LA DBS
DEPARTMENT OF BUILDING AND SAFETY

INFORMATION BULLETIN / PUBLIC - BUILDING CODE
REFERENCE NO.:   LABC 3307.3   Effective: 01-01-2017
DOCUMENT NO.: **P/BC 2017- 141**   Revised:
Previously Issued As: P/BC 2014 - 141

GUIDELINES FOR DETERMINING LIVE LOADS SURCHARGE FROM SIDEWALK PEDESTRIAN TRAFFIC AND STREET TRAFFIC

Introduction

This Information Bulletin provides guidelines for determining live loads due to sidewalk pedestrian traffic and street traffic for temporary shoring design adjacent to the public way. Surcharge loads shall be applied where vehicular load or pedestrian loads are expected to act on the surface behind a shored excavation or retaining wall within a distance equal to the height of the excavation or wall.

Based on the study performed by Kim and Barker (2002), the American Association of State Highway and Transportation Officials (AASHTO) provided a guideline for determining the equivalent height of soil for vehicular loading on retaining wall and shoring parallel to traffic (AASHTO 3.11.6). AASHTO Article 3.11.6.2 also provides surcharge pressures on retaining walls and shoring due to point, line, and strip loads based on elasticity solution (Boussinesq, 1876). Based on AASHTO recommendations, the following three methods for determining surcharge pressure on retaining walls and temporary shoring are generally acceptable to the Department. **Note: Regardless of the method used, in no case shall the traffic surcharge pressure be less than 60 psf for cantilever condition and 90 psf for braced condition. This pressure shall be considered with rectangular distribution applied horizontally on the face of the shoring.**

I.   Simple Method Using Equivalent Soil Heights for Live Loads (Method A)

Method A is applicable where no specific recommendations for traffic surcharge are provided in the Soils Report. Method A uses the following equation to determine the lateral surcharge pressure on retaining wall and shoring.

$$q = \gamma_{EFP} \times H_{eq}$$

Where: q = lateral surcharge pressure (psf) in rectangular distribution
         γ_{EFP} = equivalent fluid pressure (pcf) for shoring design
         H_{eq} = equivalent height of soil from "Table 1" below

Table 1*

Equivalent Height of Soil for Vehicular Loading on Retaining Wall and Shoring Parallel to Traffic

Excavation/Wall Height (ft)	Distance from the edge of excavation (ft)	
	0.0 ft	1.0 ft or further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

* From Table 3.11.6.4-2 of the AASHTO document referenced above.

Example:

Given: Active equivalent fluid pressure γ_{EFP} is 30 pcf
            Surcharge location is 0 feet from shoring/retaining wall
            Height of retaining wall/shoring is 10 feet

Traffic Surcharge $q = \gamma_{EFP} \times H_{eq}$= 30 pcf (Given in this example) x 3.5 ft (From Table 1) = 105 psf. This surcharge shall apply as a rectangular distribution to the full height of shoring.

II.   Site-Specific Calculation Using Equivalent Soil Heights for Live Loads (Method B)

Method B is applicable where site-specific lateral earth pressure coefficients are provided in the Soils Report approved by the Grading Division. Method B uses the following equation to determine the lateral surcharge pressure on retaining wall and shoring.

$$q = k \times \gamma_s \times H_{eq}$$

Where: q = lateral surcharge pressure (psf) in rectangular distribution
            k = active or at-rest earth pressure coefficient from Soils Report
            γ_s = total unit weight of soil (pcf)
            H_{eq} = equivalent height of soil from "Table 1" above

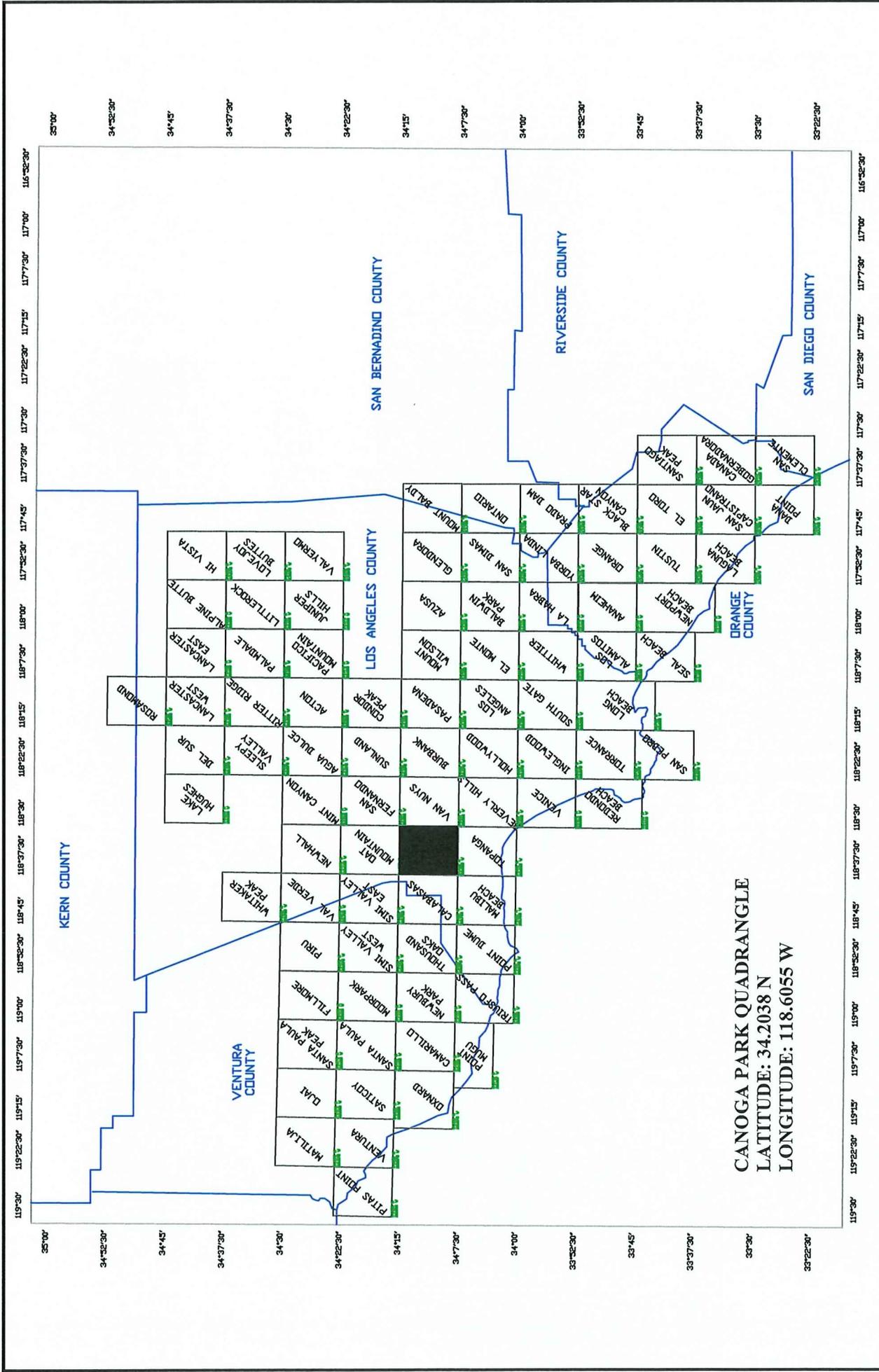## III.   Site-Specific Calculation Using Elasticity Solutions (Method C)

As discussed above, elasticity solutions included in AASHTO LRFD 2012 Bridge Design Specifications, 6th Edition (Article 3.11.6.2) are acceptable to the Department. Method C is used for more complex conditions, such as when heavy construction equipment (crane, etc.) will surcharge a shored excavation. Specific calculations for this method shall be determined by either the soils engineer of record or the project shoring engineer.

If the specific calculations are provided by the soils engineer in the soils report, such report shall be approved by the Grading Division.

As a covered entity under Title II of the Americans with Disabilities Act, the City of Los Angeles does not discriminate on the basis of disability and, upon request, will provide reasonable accommodation to ensure equal access to its programs, services and activities.

Page 2 of 2

QUADRANGLE LOCATION MAP

QUADRANGLE LOCATION MAP

7334 N. Topanga Canyon Blvd., Canoga Park

CANOGA PARK QUADRANGLE
LATITUDE: 34.2038 N
LONGITUDE: 118.6055 W

PROJECT NO.	30-5538-00
DATE	06-2020
PREPARED BY	WFB
APPROVED BY	JAV

A.G.I. GEOTECHNICAL, INC.

Engineering Geology • Geotechnical Engineering

16555 Sherman Way, Unit A • Van Nuys, CA 91406
(818) 785-5244 • Fax (818) 785-6251

KERN COUNTY

VENTURA COUNTY

LOS ANGELES COUNTY

SAN BERNADINO COUNTY

RIVERSIDE COUNTY

SAN DIEGO COUNTY

ORANGE COUNTY

PROPERTY LINE PERIMETER
DRAIN TYPICAL

SHOTCRETE WALL

DRAINAGE COMPOSITE

WOOD LAGGING

FLOOR SLAB (OR MAT)
ON-GRADE

12" WIDE x 12" HIGH x 12" DEEP
GRAVEL POCKET @ 8' O.C.

FOUNDATION

4" Ø DRAIN PIPE
TO SUMP PUMP

(NOT TO SCALE)

**PROPERTY LINE PERIMETER
DRAIN TYPICAL**
7334 N. Topanga Canyon Blvd., Canoga Park

PROJECT NO.	30-5538-00
DATE	06-2020
PREPARED BY	WFB
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GROUNDWATER MAP
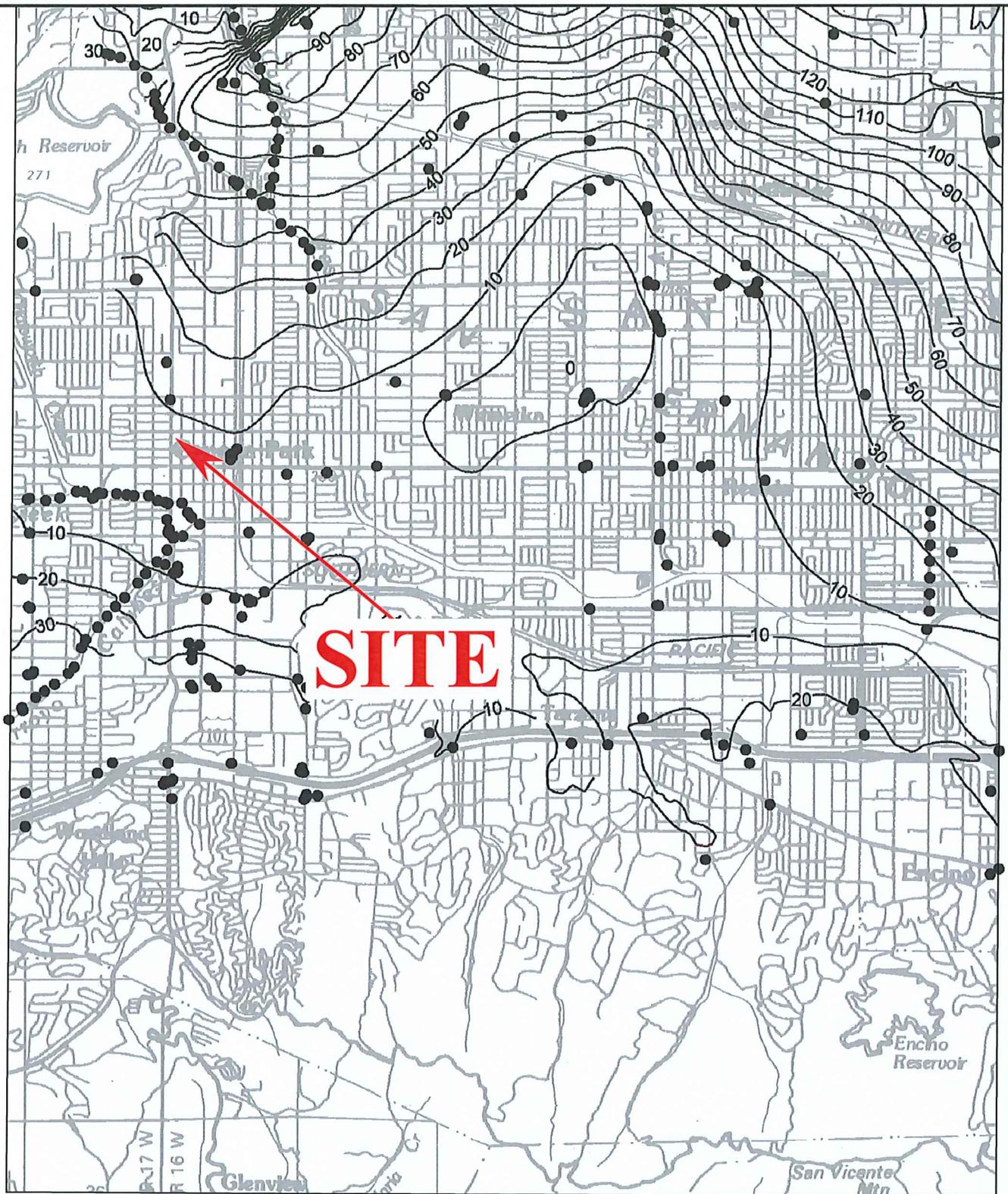
SITE

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

34°07'30"

118°30'

Plate 1.2 Historically highest ground water contours and borehole locations, Canoga Park 7.5–minute Quadrangle, California.

● Borehole Site   〜 30 〜 Depth to ground water in feet

N

ONE MILE
SCALE

GROUNDWATER MAP

7334 N. Topanga Canyon Blvd., Canoga Park

PROJECT NO.	30–5538–00
DATE	06–2020
PREPARED BY	WFB
APPROVED BY	JAV

May 7, 2021   Project No. 30-5538-03

Alliant Strategic Development, LLC
23901 Calabasas Rd., Suite 2092
Calabasas, CA 91302

Attention: Mr. John Shaw

Subject: **ADDENDUM REPORT – PROJECT REVISIONS**
                Proposed 149-Unit 5-Story Multi-Family Building
                Over Parking On-Grade
                APN 2111-011-030
                Tract: Owensmouth; Block: 50; Lots: 3, 4 and 5
                7334 N. Topanga Canyon Boulevard
                Canoga Park, California

References: **ADDENDUM REPORT – FOUNDATION SUPPORT**
                Proposed 139-Unit 4-Story Multi-Family Building
                Over Partial Subterranean Parking
                APN 2111-011-030
                Tract: Owensmouth; Block: 50; Lots: 3, 4 and 5
                7334 N. Topanga Canyon Boulevard
                Canoga Park, California
                Prepared by A.G.I. Geotechnical, Inc., Project No. 30-5538-02
                dated March 16, 2021

                ADDENDUM REPORT - BUILDING MODIFICATIONS
                Proposed 139-Unit 4-Story Multi-Family Building
                Over Partial Subterranean Parking
                APN 2111-011-030
                Tract: Owensmouth; Block: 50; Lots: 3, 4 and 5
                7334 N. Topanga Canyon Boulevard
                Canoga Park, California
                Prepared by A.G.I. Geotechnical, Inc., Project No. 30-5538-01
                dated July 30, 2020
                City Log #116296

                GEOTECHNICAL INVESTIGATION
                Proposed 139-Unit 4-Story Multi-Family Building
                Over 1½ Levels of Subterranean Parking
                APN 2111-011-030
                Tract: Owensmouth; Block: 50; Lots: 3, 4 and 5
                7334 N. Topanga Canyon Boulevard
                Canoga Park, California
                Prepared by A.G.I. Geotechnical, Inc., Project No. 30-5538-00
                dated July 10, 2020
                City Log #116296

Dear Mr. Shaw:

This report has been prepared to formally notify the City of Los Angeles that the subject project has been revised from a proposed 139-unit 4-story multi-family building over partial subterranean parking to a proposed 149-unit 5-story multi-family building over parking on-grade.

Our referenced Addendum Report dated March 16, 2021 presented recommendations for support of the new mat foundation on compacted fill overlying an existing mat foundation. It is our understanding that the existing foundation(s) might be conventional footings rather than a mat. We recommend that the new mat foundation be supported on compacted fill overlying the buried foundation(s) for either case. The attached Foundation Detail illustrates the proposed construction and presents earthwork and foundation design recommendations.

All other recommendations contained in our referenced reports remain in effect and should be followed unless specifically modified herein.

If you have any questions regarding this report, please contact this office.

Respectfully submitted,
A.G.I. GEOTECHNICAL, INC.

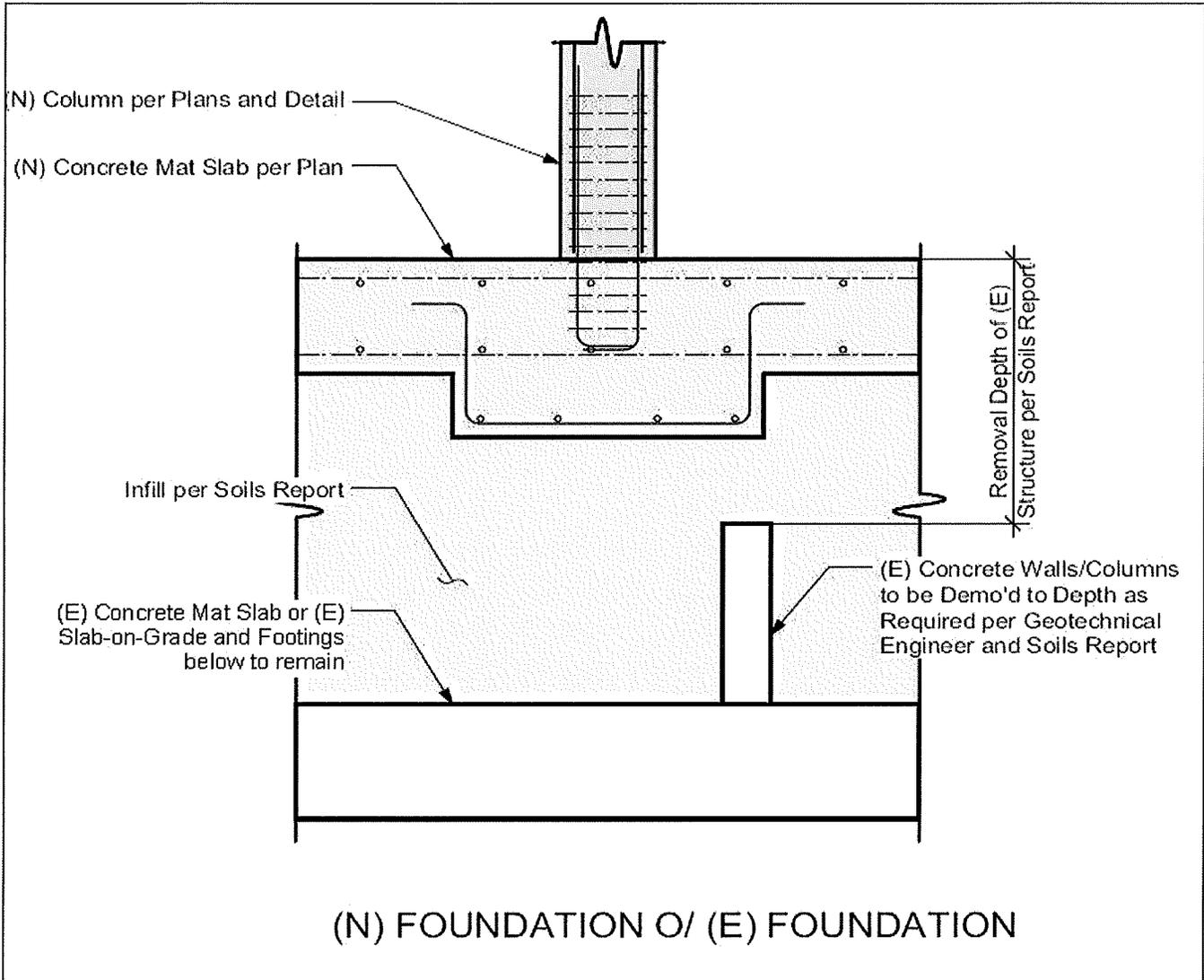Bruce Smith R.G.E. 2673
Senior Engineer

MBS:mbs

Distribution: (4) Alliant Strategic Development, LLC

Enclosure: New/Existing Foundation Detail

NEW OVER EXISTING FOUNDATION
DRAWING SHEET S-2.1

(N) Column per Plans and Detail ———

(N) Concrete Mat Slab per Plan ———

Removal Depth of (E) Structure per Soils Report

Infill per Soils Report ———

(E) Concrete Mat Slab or (E) Slab-on-Grade and Footings below to remain ———

(E) Concrete Walls/Columns to be Demo'd to Depth as Required per Geotechnical Engineer and Soils Report

(N) FOUNDATION O/ (E) FOUNDATION

1. Completely remove existing walls, columns and soil down to top of existing slab.
2. Place on-site soils compacted to 90% ASTM:D-1557 up to bottom of new mat.
3. New mat may be designed for a maximum allowable bearing pressure of $3,000 lb/ft^2$ and subgrade modulus of $150 lb/in^3$.
1. Total and diffential settlements of about 3.2 and 2.0-inches, respecfully, are anticipated.

CITY OF LOS ANGELES
CALIFORNIA

ERIC GARCETTI
MAYOR

SOILS REPORT APPROVAL LETTER

August 6, 2021

LOG # 117664-01
SOILS/GEOLOGY FILE - 2
LIQ

Alliant Strategic Development, LLC
23901 Calabasas Rd.
Calabasas, CA 91302

TRACT: OWENSMOUTH (M R 19-36 (SHT 1)
BLOCK: 50
LOT(S):   3, 4 & 5
LOCATION: 7334 N. Topanga Canyon Blvd.

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Addendum Report	30-5538-04	08/04/2021	AGI Geotechnical, Inc.

PREVIOUS REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Dept. Review Letter	117664	06/23/2021	LADBS
Addendum Report	30-5538-03	05/07/2021	AGI Geotechnical, Inc.
Dept. Approval Letter	116296	03/17/2021	LADBS
Addendum Report	30-5538-01	07/30/2020	AGI Geotechnical, Inc.
Soils Report	30-5538-00	07/10/2020	AGI Geotechnical, Inc.

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed 149 unit 5-story multifamily building over parking on grade. No basement is proposed. The earth materials at the subsurface exploration locations consist of native soils. The consultants recommend to remove all buried foundations and support the proposed structure(s) on mat-type foundations bearing on properly placed compacted fill.

The Department previously conditionally approved the above referenced reports for the proposed 139-unit 4-story multifamily building over partial subterranean parking in a letter dated 03/17/2021, Log #116296. The earth materials at the subsurface exploration locations consist of native soils. The consultants recommended to support the proposed structure(s) on mat-type foundations bearing on native undisturbed soils.

Groundwater was encountered at 20 feet below the ground surface, and depth to the historical high groundwater level is about 10 feet below the surface, according to the consultants.

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 soils are subject to liquefaction. The earthquake induced total and differential settlements are calculated to be 2.65 and 1.77 inches, respectively. To mitigate the earthquake induced settlements it is proposed to use a mat foundation.

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

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2020 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1.   All conditions of the above referenced Department approval letter Log#116296, dated 03/17/2021 shall apply except as specifically modified herein.

2.   Placement of the new building above old foundations and slab is not approved. All uncertified fill, old foundations, slabs, walls and columns and any disturbed soils due to demolition shall be removed.

3.   All foundations shall derive entire support from properly placed compacted fill, as recommended and approved by the soils engineer by inspection.

4.   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).

5.   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).

LEILA ETAAT
Structural Engineering Associate II

LE/le
Log No. 117664-01
213-482-0480

cc: Applicant
        AGI Geotechnical, Inc., Project Consultant
        VN District Office