REPORT OF
GEOTECHNICAL INVESTIGATION
PROPOSED NEW MIXED-USE BUILDING PROJECT
3506-3514 VERDUGO ROAD
GLENDALE, CALIFORNIA

FOR

ZOHRABIANS ARCHITECTS

PROJECT NO. 16-428-02

DECEMBER 13, 2017

AES
C. M. APPLIED EARTH SCIENCES
a division of Applied Soil Technology, Inc.
GEOTECHNICAL & ENVIRONMENTAL ENGINEERING CONSULTANTS

December 13, 2017 16-428-02

Zohrabians Architects
3467 Ocean View Blvd. Suite B
Glendale, California 91208

Subject: Geotechnical Investigation
For Proposed New Mixed-Use Building Project
3506-3514 Verdugo Road
Glendale, California 91208

Gentlemen:

INTRODUCTION

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

During the course of this investigation, the provided project plans by the client were used as reference.

The enclosed Site Plan; Drawing No. 1, shows the approximate location of the drilled borings in relation to the site boundaries. This drawing also shows the approximate locations of the Cross Sections A-A’ and B-B’. Drawing Nos. 2 and 3 show the profiles of the Cross Sections A-A’ and B-B’.

Figure No. 1 shows the Site Vicinity Map. Figure No. 2 shows the Regional Topographic Map. Figure No. 3 shows the Regional Geologic Map. Figure No. 4 shows the Historically Highest Groundwater Contour Map.

The attached Appendix I, describes the method of field exploration. Figure Nos. I-1 through I-3 present summaries of the materials encountered at the location of our borings. Figure No. I-4 presents the Uniform Soil Classification System Chart; a guide to the Log of Exploratory Borings.

The attached Appendix II describes the laboratory testing procedures. Figure Nos. II-1 and II-2 present the results of direct shear and consolidation tests performed on selected undisturbed soil samples.

It should be noted that the presented recommendations for excavation and foundation are based on our understanding of the depth of cuts setback conditions, and assumed structural loading. This office should be consulted, if the actual structural loading and excavation depths are different from those used during this investigation.

PROJECT CONSIDERATIONS

It is our understanding that the proposed project will consist of construction of a apartment building at the subject site. The proposed building is expected to be a 4-story wood frame structure constructed over basement.

The basement grade is expected to be established at some 10 feet below grade. Therefore, total height of excavation to the perimeter wall footing levels are expected to be less than 12 feet.

It is anticipated that the perimeter walls of the basement will have variable amounts of horizontal setback from the respective property lines. Where adequate horizontal space beyond the planned line of excavation is available, unsupported, open excavation slopes with gradients as recommended in this report may be used. Where adequate horizontal space is not available, temporary shoring will be required. Such shoring system shall be in a form of cantilevered soldier piles.

Structural loading data was not available at the time of this investigation. For the purpose of this report, it is assumed that maximum concentrated loads of the interior columns will be on the order of 300 kips, combined dead plus frequently applied live loads. Perimeter and interior wall footings of the structure are expected to exert loads of on the order of 6 kips per lineal foot.

ANTICIPATED SITE GRADING WORK

Site grading will involve conducting the following tasks:

1. Excavation of the basement;
2. Subgrade preparation for support of basement grade slabs; and
3. Wall backfilling within the over-excavated areas.

The wall backfill material should be non-expansive and granular in nature. Therefore, the excavated materials from the site can be reused for wall backfilling. After completion of the site grading work, materials will be exported from the site.

SITE CONDITIONS

SURFACE CONDITIONS

The site of the proposed project is located at 3506-3514 Verdugo Road, Glendale, California. The site consist of three rectangular shaped lots and covers a plan area of about 18,000 square feet. See the enclosed Site Plan; Drawing No. 1 for site location.

At the time of our field investigation, the site was occupied by commercial structures which will be removed from the site. The ground surface was noted to be generally level.

SUBSURFACE CONDITIONS

Correlation of the subsoil between the borings was considered to be good. Generally, the site, to the depths explored, was found to be covered by surficial fill underlain by natural deposits of silty sand soil and relatively clean soil with variable amounts of gravel. Thickness of surficial fill was found to be on the order of 2 feet in our borings. Deeper fill, however, may be present beneath the existing structures and in old utility lines. The existing fill is expected to be automatically removed by the planned basement excavation.

The upper soils through which the basement garage excavations will be made is expected to be fill and native soils consisting of mainly silty sand soils with variable amounts of gravel. These soils were found to be generally dense to very dense in-place. The results of our laboratory investigations indicated that these materials were of relatively high strengths.

The subsurface materials near the planned foundation levels were also found to consist of dense, silty sand soils. The results of our laboratory testing indicated that these materials were of higher strengths and low compression.

The soils at the basement garage level were found to be granular in nature. These soils were found to be virtually non-expansive.

During the course of our investigation, no groundwater was encountered in our borings drilled to a maximum depth of 28 feet. Due to the method of drilling (use of continuous auger) caving was not detected in our deep borings. Because of the relatively clean nature of the soils at the basement garage level, forming may be required during foundation construction.

SEISMIC DESIGN CONSIDERATIONS

In accordance with the 2016 California Building Code (CBC 2016), the project site can be classified as site “D”. The mapped spectral accelerations of S_S=2.701 (short period) and S_1 =0.950 (1-second period) can be used for this project. These parameters corresponds to site Coefficients values of F_a =1.0 and F_V =1.5, respectively.

The seismic design parameters would be as follows:

$\mathbf{S_{MS}}$= F_a (S_S) = 1.0 (2.701) = 2.701	$\mathbf{S_{M1}}$=F_v (S_1) = 1.5 (0.950) = 1.425
$\mathbf{S_{DS}}$=2/3 (S_{MS}) = 2/3 (2.701) = 1.801	$\mathbf{S_{D1}}$=2/3 (S_{M1}) = 2/3 (1.425) = 0.950

EVALUATION OF LIQUEFACTION POTENTIAL

As part of our field exploration, one boring was attempted to be drilled to a depth of 51 feet. However, due to the presence of gravel and large bolders, our boring hit refusal at a depth of 28 feet. Water was not encountered in our borings. The State Maps show the historically highest groundwater level at the subject site to be close to a depth of 25 feet. See the enclosed Figure No. 4. For evaluating liquefaction potential at the site, therefore, SPT (Standard Penetration Test) were conducted from a depth of 20 feet.

The blow counts below a depth of 10 feet were found to be consistently above 48 blows. Due to the presence of significant course grained gravel, the standard 6 inch increment penetration was not possible. Therefore, for the purpose of this project we have conducted incremental measurement of the blow count and extrapolated to the standard 12” penetration.

Considering the relatively high SPT values and presence of abundant gravel and occasional cobbles, it is reasonable to assume the SPT values will increase with the increasing of the confining pressure. Therefore, for the purpose of liquefaction evaluation we have conservatively assumed that the SPT values of over 50 for the subsurface material. It is noted that, when the quantity of the large gravel/cobbles in the subsurface materials is relatively large, such as the case at this site, the chances of liquefaction potential reduces significantly.

The results of our liquefaction analysis (using CivilTech program) with lower level peak ground acceleration (PGA) corresponding to 2/3 of PGAm (a value of 0.659g) and the predominant earthquake magnitude of 6.90 with 10% probability of exceedance in 50 years (475-year return period) a factor of safety of greater than 1.1 was obtained for all layers. The corresponding seismic related settlements was found to be insignificant (0.10 inches). See the enclosed engineering calculation sheets.

When using higher level peak ground acceleration value of 0.989g corresponding to PGA based on PGAm (Maximum Considered Earthquake-Geometric Mean, MCEg, adjusted to site effects, ASCE 7-10 Eq. 11.8-1) and the predominant earthquake magnitude of 7.10 with 2% probability of exceedance in 50 years (2475-year return period) a factor of safety of greater than 1.0 was also obtained for all layers. The corresponding seismic related total settlements, however, was found to be on the order of 0.63 inches. On this basis, it is our opinion that soil liquefaction will not be significant at the subject site.

EVALUATION AND RECOMMENDATIONS

GENERAL

Based on the geotechnical engineering data derived from this investigation, the site is considered to be suitable for the proposed development. The existing fill is expected to be automatically removed by the planned basement garage construction.

Conventional spread footing foundation system can be used for support of the proposed building. The foundation bearing materials are expected to be dense, silty sand native soils containing gravel.

As part of the site grading work to establish the proposed finished grades of the basement, temporary excavation will be made. The planned lines of excavation will have variable horizontal setbacks from the respective property lines.

Where adequate horizontal space beyond the line of excavation is available, unsupported, open excavation slopes with gradients as recommended in this report may be used. Where adequate horizontal space is not available, temporary shoring will be required. Such shoring system shall be in a form of cantilevered soldier piles.

The grade slabs can be supported on the finished grades, provided that any fill and disturbed soils would be compacted to a relative compaction of at least 90 percent at optimum moisture content. Although the soils at the basement level are considered to be of low expansion potential, the grade slabs for this project should be at least 5 inches thick and be reinforced with # 4 bars placed at every 18 inches on center.

The following sections present our specific recommendations for temporary excavations, foundations, lateral design, basement grade slabs, subsurface walls, grading, surface drainage, swimming pool and observations during construction.

TEMPORARY EXCAVATION

Unshored Excavations: Where space limitations permit, unshored temporary excavation slopes could be used. Based upon the engineering characteristics of the site upper soils, it is our opinion that temporary excavation slopes in accordance with the following table should be used:

Maximum Depth of Cut (Ft)	**Maximum Slope Ratio (Horizontal:Vertical)**
0-3	Vertical
>3	1:1

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

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

Cantilevered Soldier Piles: In the areas where adequate horizontal distance beyond the planned line of excavation is not available, cantilevered soldier piles should be used as a means of temporary shoring. Soldier piles consist of structural steel beams encased in slurry mix.

The lateral resistance for soldier piles may be assumed to be offered by passive pressure below the basement. An allowable passive pressure of 500 pounds per square foot per foot of depth may be used below the cut for piles having center-to-center spacing of 2-1/2 times the pile diameter. Maximum allowable passive pressure should be limited to 4,800 pounds per square foot. The maximum center-to-center spacing of the vertical shafts should be no greater than 10 feet.

For design of temporary support, active pressure on piles may be computed using an equivalent fluid density of 30 pounds per cubic foot. Uniform surcharge may be computed using an active pressure coefficient of 0.30 times the uniform load.

When using cantilevered soldier piles, the point of fixity may be assumed to occur at some 2 feet below the base of the excavation. In order to limit local sloughing, it is recommended that lagging be used between the soldier piles. All wood members left in ground should be pressure treated. For the purpose of design, lagging pressure should not exceed 400 pounds per square foot.

It should be noted that the recommendations presented in this section are for use in design and for cost estimating purposes prior to construction. The contractor is solely responsible for safety during construction.

TOLERABLE LEVEL OF PILE DEFLECTION AND MONITORING

Where off-site buildings occur within a horizontal distance equal to the depth of cut, the allowable lateral deflection at the tops of the piles should be limited to ½ of one inch. In the areas where the shoring system supports public right-of-way, and where off-site buildings occur outside a horizontal distance equal to the depth of the first row of the lateral support, the allowable lateral deflection piles can be increased to one inch.

The lateral support of the existing off-site buildings should be maintained by the planned temporary shoring for the subject project. The project Structural Engineer should use appropriate surcharge from the off-site building and add to the lateral earth pressure. Proper monitoring program should be maintained during basement garage excavation to assure the shoring pile deflections would not exceed the tolerable limits.

FOUNDATIONS

Conventional spread footing foundation system can be used for support of the proposed building. The foundation bearing materials are expected to be dense, silty sand native soils containing little gravel.

Exterior and interior footings should be a minimum of 18 inches wide and should be placed at a minimum depth of 24 inches below the lowest adjacent final grades (in this case, basement level).

The recommended allowable maximum bearing pressure for minimum size footings placed in native soils could be taken as 2,400 pounds per square foot. This value may be increased at a rate of 150 and 300 pounds per square foot for each additional foot of footing width and depth, to a maximum value of 4,000 pounds per square foot.

The above given allowable maximum bearing values are for the total of dead and frequently applied live loads. For short duration transient loading, such as wind or seismic forces, the given values may be increased by one-third.

Under the allowable soil pressure, footings with the assumed collected loads of 300 kips are expected to settle about 3/4 of one inch. Continuous footings, with loads of about 6 kips per linear foot are expected to settle on the order of 5/8 of one inch. Maximum differential settlements are expected to be on the order of 1/4 of an inch.

LATERAL DESIGN

Lateral resistance at the base of footings in contact with native soils may be assumed to be the product of the dead load forces and a coefficient of friction of 0.35. Passive pressure on the face of footings may also be used to resist lateral forces. A passive pressure of zero at the finished grades and increasing at a rate of 250 pounds per square foot per foot of depth to a maximum value of 3,000 pounds per square foot may be used for footings poured against native soils.

GRADE SLABS

On the basis that slab subgrade would be prepared in accordance with the recommendations presented in the preceding sections of this report, grade slabs may be supported on the finished grades that consists of prepared native subgrade or properly compacted fill that has been placed to at least 90 percent relative compaction at optimum moisture content. Although the soils at the basement level are considered to be of low expansion potential, the grade slabs for this project should be at least 5 inches thick and be reinforced with # 4 bars placed at every 18 inches on center.

In the areas where moisture sensitive floor covering is used and slab dampness cannot be tolerated, a vapor-barrier should be used beneath the slabs. This normally consists of a 10-mil polyethylene film covered with 2 inches of clean sand.

BASEMENT WALLS

The perimeter walls of the basement are expected to be buried to a maximum depth of about 10 feet. Static design of these walls (being restrained against rotation) should be based on an equivalent fluid pressure of 55 pounds per square foot per foot of depth. Cantilevered retaining walls (ramp area) can be designed based on an equivalent fluid pressure of 30 pounds per square foot per foot of depth. See the enclosed supporting engineering calculations.

The above given pressures assume that no hydrostatic pressure will occur behind the retaining walls. This will require installation of proper subdrain behind the basement garage walls.

Subdrain normally consists of 4-inch diameter perforated pipes encased in gravel (at least one cubic foot per lineal foot of the pipes). In order to reduce the chances of siltation and drain clogging, the free-draining gravel should be wrapped in filter fabric proper for the site soils.

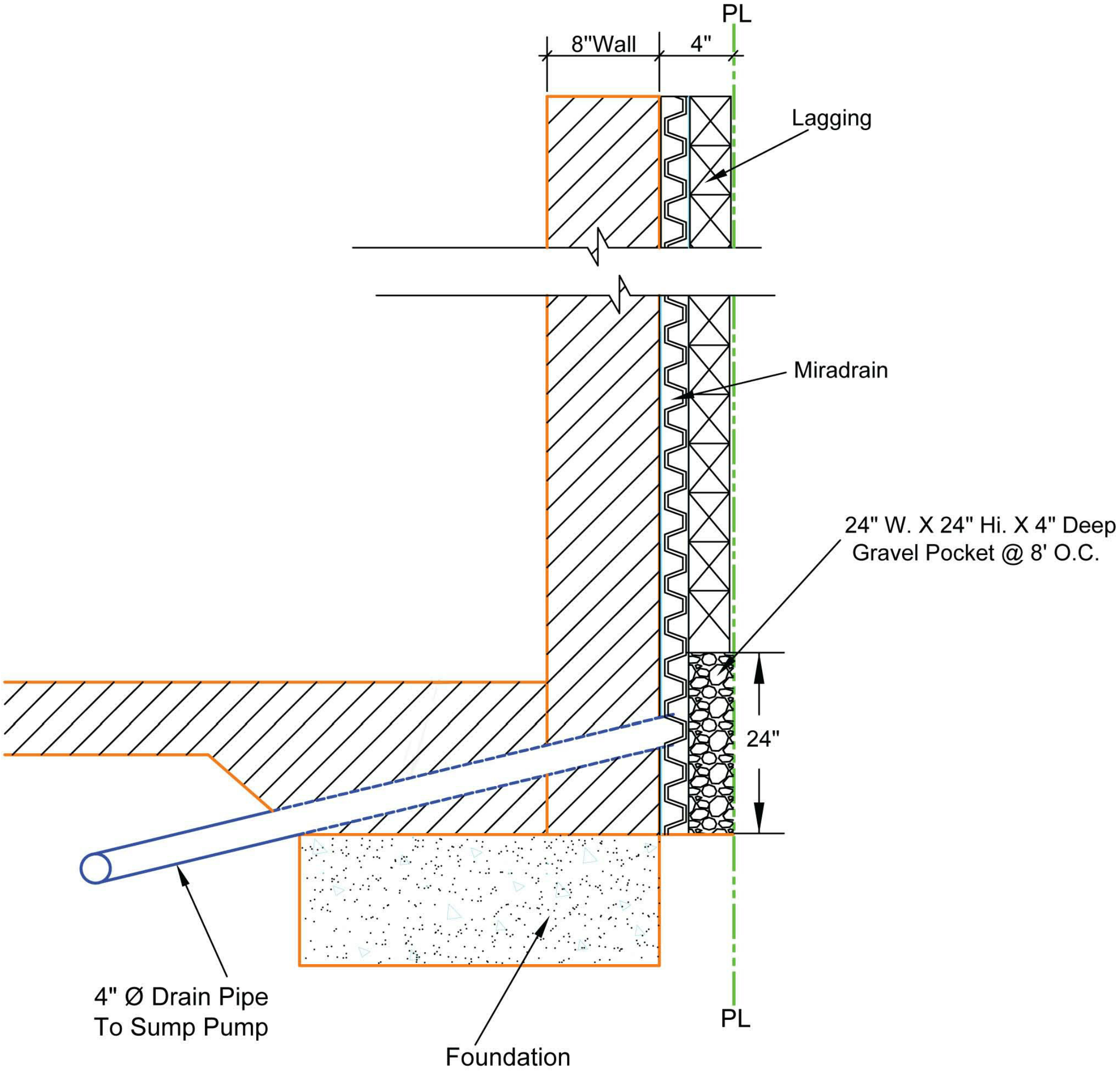
It should be noted that, if adequate space behind the exterior walls of the basement garage is not available to use standard pipe and gravel subdrain, the exterior walls of the basement garage should be equipped with a subdrain similar to those presented on Sketch No. 1 (See next page).

Use of alternative subdrain will require that a “request-for-modification” form with proper fees be submitted to the City Grading Department.

In addition to the lateral earth pressure, the basement garage walls should also be designed for any applicable uniform surcharge loads imposed on the adjacent grounds. For cantilevered retaining walls, the uniform surcharge effects may be computed using a coefficient of 0.30 times the assumed uniform loads. For restrained walls, a coefficient of 0.45 times the assumed uniform loads should be used.

Use of alternative subdrain will require that a “request-for-modification” form with proper fees be submitted to the City Grading Department.

In addition to the lateral earth pressure, the basement garage walls should also be designed for any applicable uniform surcharge loads imposed on the adjacent grounds. For cantilevered retaining walls, the uniform surcharge effects may be computed using a coefficient of 0.30 times the assumed uniform loads. For restrained walls, a coefficient of 0.45 times the assumed uniform loads should be used.



It is noted that, based on the new Code requirement, the basement walls should be designed not only for static, but also for seismic lateral earth pressures. For the purpose of this project, the magnitude of seismic lateral earth pressure should be assumed maximum at the ground surface and reduce at a rate of 30 pounds per square foot per reducing depth to zero at the bottom of excavation. The point of application of the lateral thrust of the seismic pressure should be assumed 0.6 time the wall height, measured from the bottom of the wall.

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

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

GRADING RECOMMENDATIONS

Site grading for the proposed project is expected to include excavation in order to create the basement garage grades and backfilling behind the basement walls. Only the excavated sandy soils should be used for wall backfilling.

Prior to placing any fill, the Soil Engineer should observe the bottoms. The areas to receive fill should be scarified to a depth of 8 inches, moistened to near optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined by the ASTM Designation D 1557 Compaction Method.

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

1. All fill should be granular in nature. Therefore, only the excavated sandy soil from the site may be reused in the areas of compacted fill.

2. Before wall backfilling, subdrain should be installed. The subdrain system should consist of 4-inch diameter perforated pipes embedded in about 1 cubic feet of free draining gravel per foot of pipe. An approved filter fabric should then be wrapped around the free draining gravel in order to reduce the chances of siltation. Non-perforated outlet pipes should then be used to pass through the wall into an interior sump. The subdrain pipes should be laid at a minimum grade of two percent for self-cleaning.

3. The excavated sandy soils from the site are considered to be satisfactory to be reused in the areas of compacted fill and wall backfill provided that rocks larger than 6 inches in diameter are removed.

4. Fill material, approved by the Soil Engineer, should be placed in controlled layers. Each layer should be compacted to at least 90 percent of the maximum unit weight as determined by ASTM designation D 1557 for the material used.

5. The fill soils shall be placed in 8-inch loose layer. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to insure uniformity of material in each layer.

6. When moisture content of the fill is too low, water shall be added and thoroughly dispersed until the moisture content is near optimum. When the moisture content of the fill material is too high to obtain adequate compaction, the fill material shall be aerated by blading or other satisfactory methods until near optimum moisture condition is achieved.

7. Inspection and field density tests should be conducted by the Soil Engineer during grading work to assure that adequate compaction is attained. Where compaction of less than 90 percent is indicated, additional compactive effort should be made with adjustment of the moisture content or layer thickness, as necessary, until at least 90 percent compaction is obtained.

SITE DRAINAGE

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

The site drainage recommendations should also include the following:

1. Having positive slope away from the buildings, as recommended above;
2. Installation of roof drains, area drains and catch basins with appropriate connecting lines;
3. Managing landscape watering;
4. Regular maintenance of the drainage devices;

5. Installing waterproofing or damp proofing, whichever appropriate, beneath concrete grade slabs and behind the basement walls.
6. The owners should be familiar with the general maintenance guidelines of the City requirements.

ON-SITE INFILTRATION CONSIDERATIONS

During the course of our original investigation, no groundwater was found in our borings drilled to a maximum depth of 28 feet. The State maps, however, show the historically highest groundwater level in the vicinity of the subject site to be near a depth of about 25 feet.

Considering that the base of the proposed building will occur near a depth of 10 feet, this will not leave the required 10 foot natural filtration zone (as required by the Sanitation District) below the base of the building. As such, for the proposed project, the site is considered to be a poor candidate for on-site storm water infiltration. Therefore, the storm water should be diverted to areas of planter and any excess water should be carried to the curb line, after going through the required filtration process.

OBSERVATION DURING CONSTRUCTION

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

Drilling of the soldier piles should be made under continuous observation of Deputy Grading Inspector representing this office. It is essential to assure that all shoring piles are drilled to proper depths and diameters.

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

CLOSURE

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

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

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

Engineering Calculations
Drawing No. 1 - Site Plan
Drawing No. 2 - Cross Section A-A’
Drawing No. 3 - Cross Section B-B’
Figure No. 1 - Site Vicinity Map
Figure No. 2 - Regional Topographic Map
Figure No. 3 - Regional Geologic Map
Figure No. 4 - Historically Highest Groundwater (contour Map)
Appendix I- Method of Field Exploration
Figure Nos. I-1 through I-4
Appendix II- Methods of Laboratory Testing
Figure Nos. II-1 and II-2

Respectfully submitted,

Applied Earth Sciences

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Distribution: (4)

Average Soil Parameters

Saturated Unit Weight = γs = 121 pcf

Value of Fiction Angle = ϕ = 33 ^{o}

$$K_o = 1 - \sin(\phi)$$

$$K_o = 1 - \sin 33^{o}$$

$$K_o = 1 - 0.54$$

$$K_o = 0.46$$

$$\gamma_o = K_o * \gamma$$

$$\gamma_o = 0.46 * 121$$

$$\gamma_o = 55.1$$

At-Rest Equivalent Fluid Density, $\boxed{\gamma_o = 55 \text{ PCF}}$

AT-REST LATERAL EARTH PRESSURE

Basement Walls

FOR: 3506-3514 Verdugo Blvd.

DATE: 12/13/17

PROJECT NO.: 16-428-02

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CALC SHEET No. 1

Average Soil Parameters

* FIGURE 2 of Naval Facilities Engineering Command

Saturated Unit Weight ϒ= 121 PCF

Height of Wall H= 10 Ft.

PGA_M= 0.989

$$P_{AE} = \frac{3}{8}\gamma H^2(K_h) \qquad *7.2\text{-}78$$

$$K_h = \frac{\frac{2}{3} * PGA_M}{2}$$

K_h= 2/3 * 0.989 / 2

K_h= 0.33

P_{AE} = 3/8 * 121 * 100 * 0.33

P_{AE} = 1496 lb.

Equivelent Fliud Pressure (EFP)

$$EFP = (\frac{2xP_{AE}}{H^2})$$

EFP= 2 * 1496 / 100

EFP= 29.92 PCF

SEISMIC LATERAL EARTH PRESSURE

Basement Walls

FOR: 3506-3514 Verdugo Road

DATE: 12/13/17

PROJECT NO.: 16-428-02

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CALC SHEET NO. 2

Average Soil Parameters

Saturated Unit Weight γ = 121 pcf

C = 102.5 psf

ϕ = 33 ^{o}

Height of Wall

H= 10 ft

Weight of Surcharge Load on Wedge

W_q= 0.3 K

					Driving Force	*Resisting Force*	
SECTION	**A (sf)**	**W (K)**	**L (feet)**	**α (degrees)**	**Wsinαcosα (k)**	**$W\cos^2\alpha\tan\phi$ (k)**	**CLcosα (k)**
I	27.1	3.3	11.38	61.5	1.5	0.5	0.6
					1.5	1.1	

F.S. = ∑ RF / ∑ DF = 1.09 / 1.50 = 0.72

FOR FACTOR OF SAFETY = 1.25 (TEMPORARY)

1.25 (DF) = (RF) + UBF

1.25 * 1.50 = 1.09 + UBF

UBF = 1.88 - 1.09 = 0.79 k/lft.

Equivalent Fluid Density $G_h = 2P/H^2$

G_h = 15.9 pcf

Therefore use Recommended value of 30 pcf

FOR FACTOR OF SAFETY = 1.5 (PERMANENT)

1.5 (DF) = (RF) + UBF

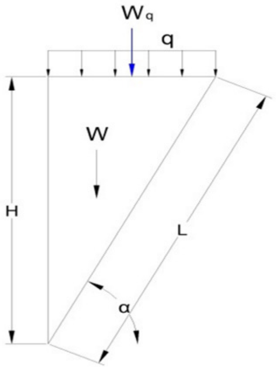
1.5 * 1.50 = 1.09 + UBF

UBF = 2.25 - 1.09 = 1.17 k/lft.

Equivalent Fluid Density $G_h = 2P/H^2$

G_h = 23.4 pcf

Therefore use Recommended value of 35 pcf



LATERAL EARTH PRESSURE CALCULATIONS

CANTILEVERED SYSTEM

SECTION A-A' - North Facing Basement Walls

FOR: 30506-3514 Verdugo Blvd. **DATE:** 12/13/17 **PROJECT NO.:** 16-428-02

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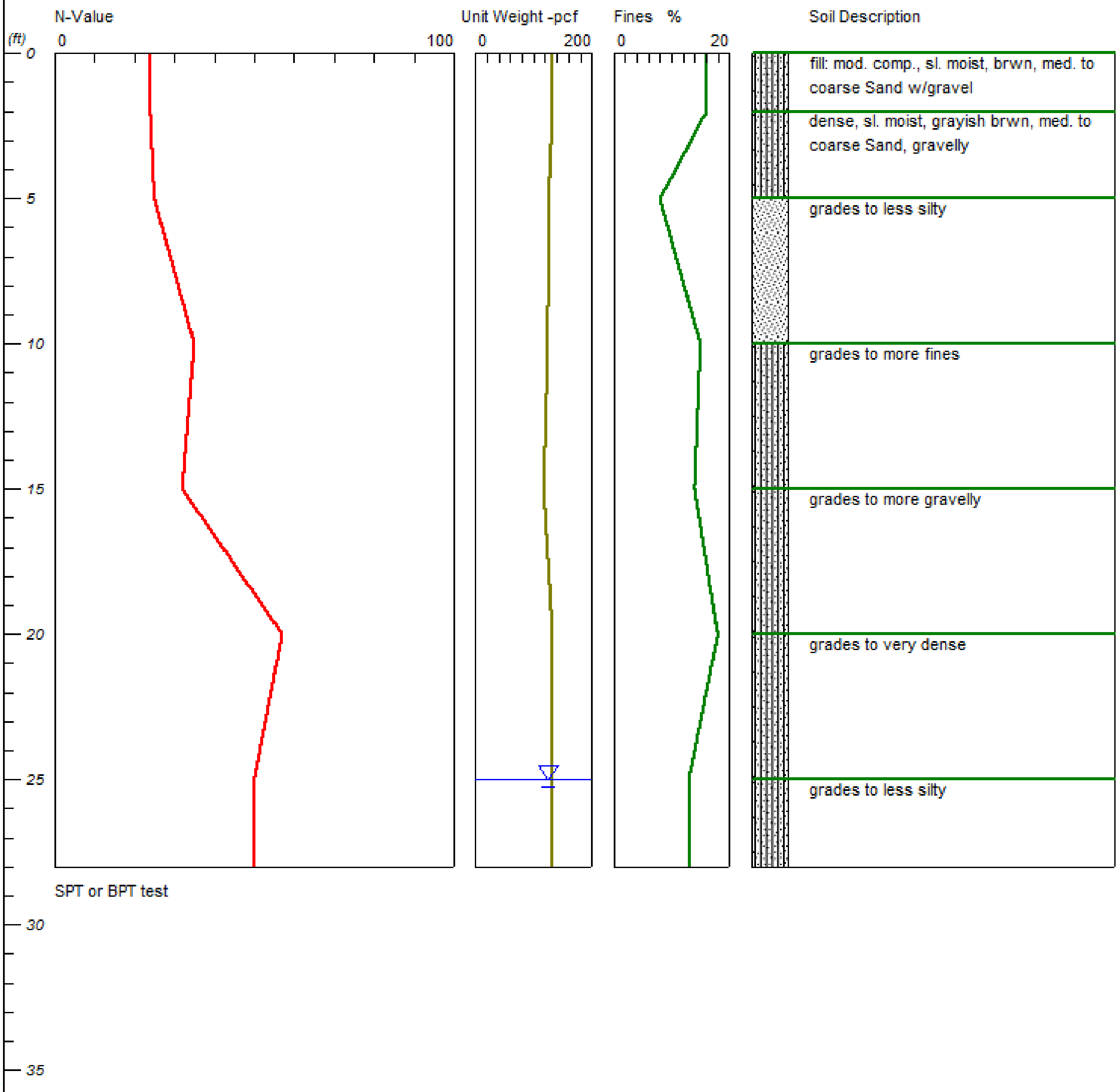
TABLE No. 1

LIQUEFACTION ANALYSIS

3506-3514 Verdugo Rd.

Hole No.=1   Water Depth=25 ft

Magnitude=7.15
Acceleration=0.989g

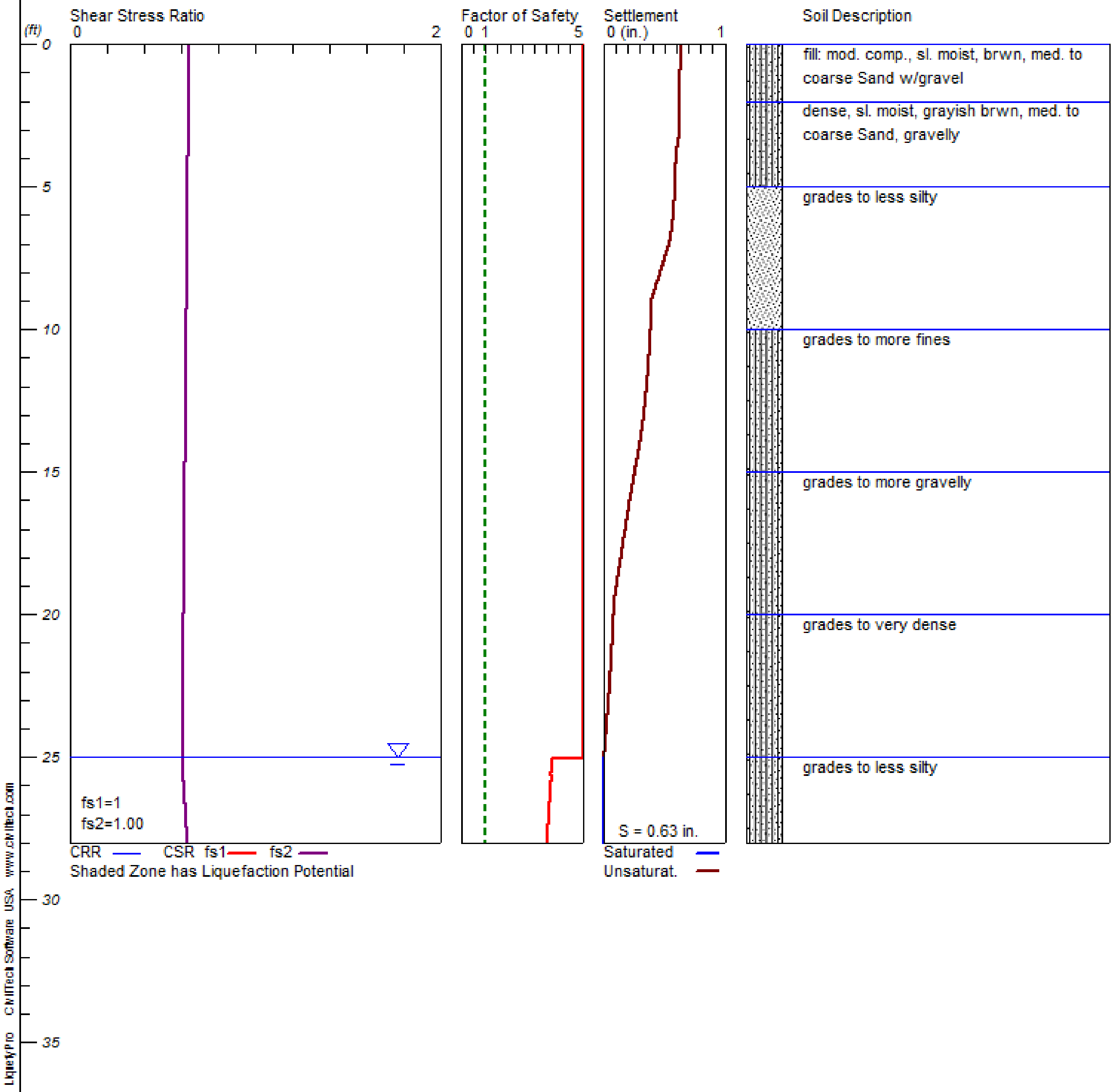


LIQUEFACTION ANALYSIS

3506-3514 Verdugo Rd.

Hole No.=1 Water Depth=25 ft

Magnitude=7.15
Acceleration=0.989g



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                                          LIQUEFACTION ANALYSIS SUMMARY

                                         Copyright by CivilTech Software
                                               www.civiltech.com

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        Licensed to ,   12/13/2017      9:24:57 AM

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02\Engineering-Calculation\Liquefaction\16-428-02_3506-3514 Verdugo Rd_2%.liq
        Title:  3506-3514 Verdugo Rd.
        Subtitle:  16-428-02_2%

        Surface Elev.=
        Hole No.=1
        Depth of Hole= 28.00 ft
        Water Table during Earthquake= 25.00 ft
        Water Table during In-Situ Testing= 30.00 ft
        Max. Acceleration= 0.99 g
        Earthquake Magnitude= 7.15

 Input Data:
        Surface Elev.=
        Hole No.=1
        Depth of Hole=28.00 ft
        Water Table during Earthquake= 25.00 ft
        Water Table during In-Situ Testing= 30.00 ft
        Max. Acceleration=0.99 g
        Earthquake Magnitude=7.15
        No-Liquefiable Soils:   Based on Analysis

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Ishihara / Yoshimine
        3. Fines Correction for Liquefaction: Stark/Olson et al.*
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                   Ce = 1.2
        7. Borehole Diameter,                                         Cb= 1.15
        8. Sampling Method,                                          Cs= 1
        9. User request factor of safety (apply to CSR) ,   User= 1
           Plot two CSR (fs1=1, fs2=User)
        10. Use Curve Smoothing: Yes*
        * Recommended Options

        In-Situ Test Data:
        Depth   SPT     gamma   Fines
        ft              pcf     %
        ____________________________________
        0.00    24.00   133.00  16.00
        2.00    24.00   133.00  16.00
        5.00    25.00   126.00  8.00
        10.00   35.00   125.00  15.00
        15.00   32.00   118.00  14.00
        20.00   57.00   133.00  18.00
        25.00   50.00   131.00  13.00
        ____________________________________

Output Results:
```

```
        Settlement of Saturated Sands=0.00 in.
        Settlement of Unsaturated Sands=0.63 in.
        Total Settlement of Saturated and Unsaturated Sands=0.63 in.
        Differential Settlement=0.316 to 0.417 in.

        Depth   CRRm    CSRfs   F.S.    S_sat.  S_dry   S_all
        ft                              in.     in.     in.
        _______________________________________________________
        0.00    2.26    0.64    5.00    0.00    0.63    0.63
        2.00    2.26    0.64    5.00    0.00    0.63    0.63
        4.00    2.26    0.64    5.00    0.00    0.59    0.59
        6.00    2.26    0.63    5.00    0.00    0.57    0.57
        8.00    2.26    0.63    5.00    0.00    0.46    0.46
        10.00   2.26    0.63    5.00    0.00    0.39    0.39
        12.00   2.26    0.62    5.00    0.00    0.35    0.35
        14.00   2.26    0.62    5.00    0.00    0.30    0.30
        16.00   2.26    0.62    5.00    0.00    0.21    0.21
        18.00   2.26    0.62    5.00    0.00    0.14    0.14
        20.00   2.26    0.61    5.00    0.00    0.08    0.08
        22.00   2.26    0.61    5.00    0.00    0.05    0.05
        24.00   2.26    0.61    5.00    0.00    0.02    0.02
        26.00   2.27    0.61    3.69    0.00    0.00    0.00
        28.00   2.24    0.63    3.53    0.00    0.00    0.00
        _______________________________________________________
        * F.S.<1, Liquefaction Potential Zone
        (F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)

        Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.

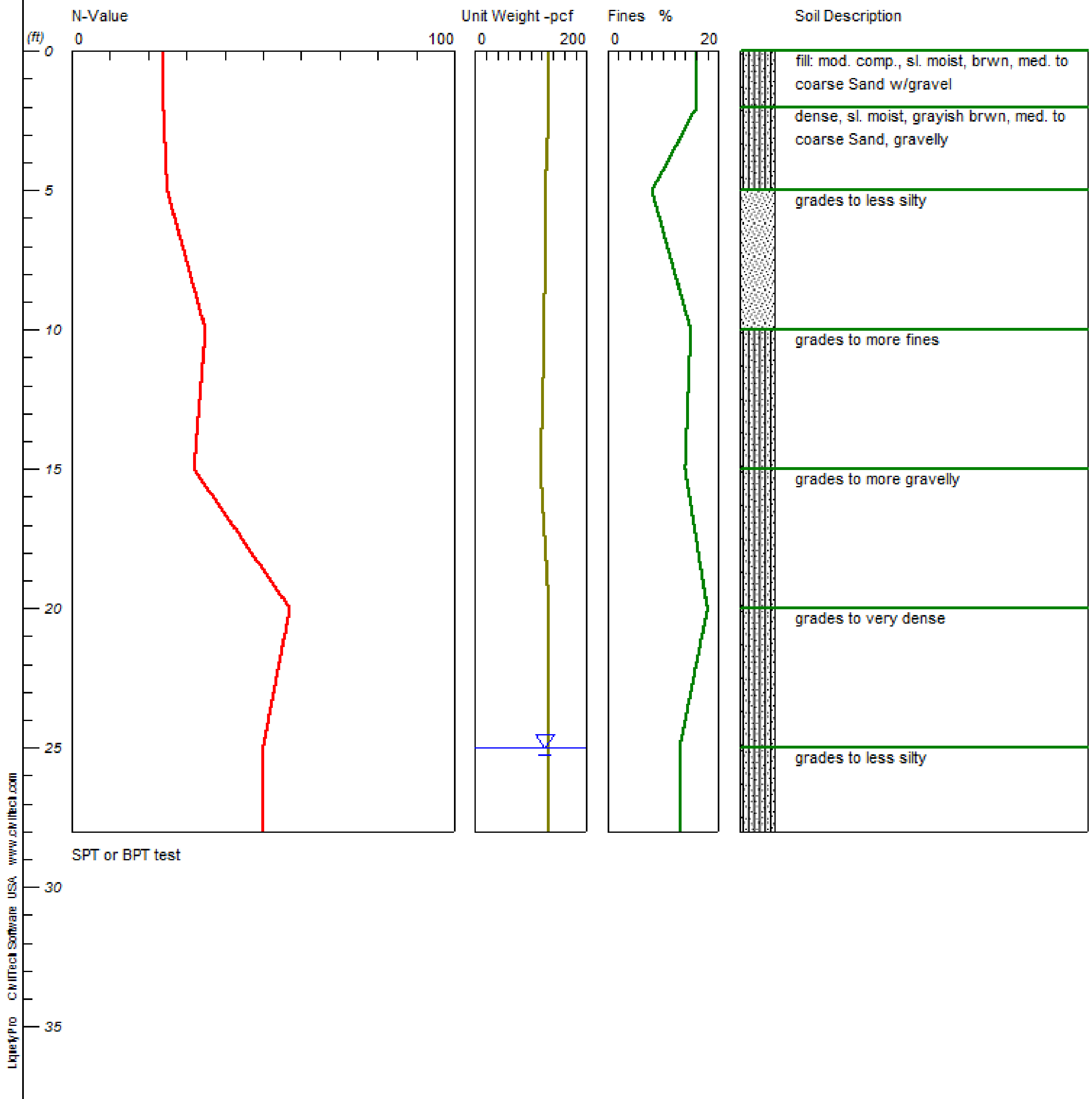
____________________________________________________________________________________
        1 atm (atmosphere) = 1 tsf (ton/ft2)
        CRRm            Cyclic resistance ratio from soils
        CSRsf           Cyclic stress ratio induced by a given earthquake (with user
request factor of safety)
        F.S.            Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        S_sat           Settlement from saturated sands
        S_dry           Settlement from Unsaturated Sands
        S_all           Total Settlement from Saturated and Unsaturated Sands
        NoLiq           No-Liquefy Soils
```

LIQUEFACTION ANALYSIS

3506-3514 Verdugo Rd.

Hole No.=1   Water Depth=25 ft

Magnitude=6.98
Acceleration=0.659g

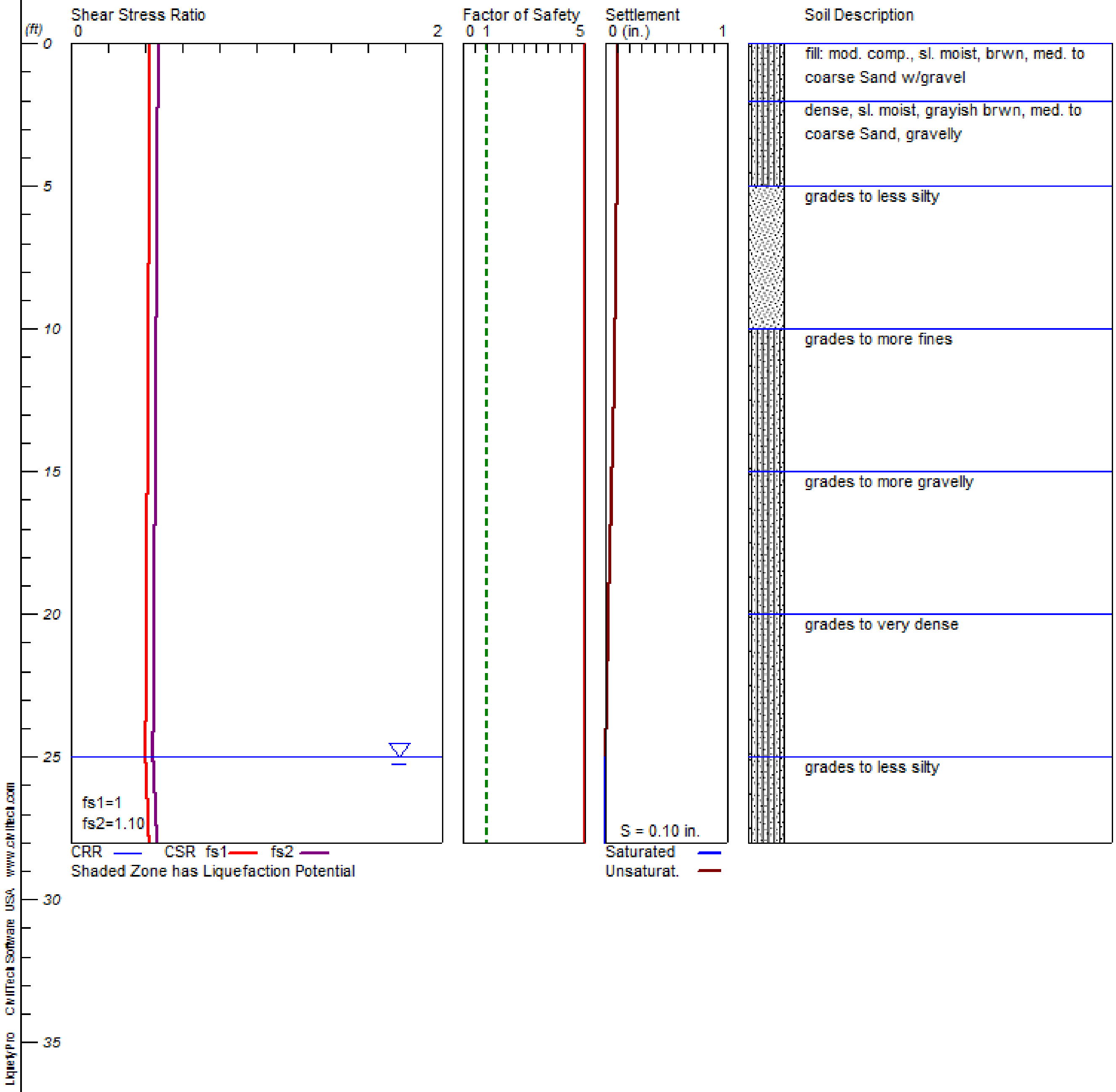


LIQUEFACTION ANALYSIS

3506-3514 Verdugo Rd.

Hole No.=1 Water Depth=25 ft

Magnitude=6.98
Acceleration=0.659g



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*******************
                                          LIQUEFACTION ANALYSIS SUMMARY

                                         Copyright by CivilTech Software
                                               www.civiltech.com

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        Licensed to ,   12/13/2017      9:27:50 AM

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02\Engineering-Calculation\Liquefaction\16-428-02_3506-3514 Verdugo Rd_10%.liq
        Title:  3506-3514 Verdugo Rd.
        Subtitle:  16-428-02_10%

        Surface Elev.=
        Hole No.=1
        Depth of Hole= 28.00 ft
        Water Table during Earthquake= 25.00 ft
        Water Table during In-Situ Testing= 30.00 ft
        Max. Acceleration= 0.66 g
        Earthquake Magnitude= 6.98

 Input Data:
        Surface Elev.=
        Hole No.=1
        Depth of Hole=28.00 ft
        Water Table during Earthquake= 25.00 ft
        Water Table during In-Situ Testing= 30.00 ft
        Max. Acceleration=0.66 g
        Earthquake Magnitude=6.98
        No-Liquefiable Soils:   Based on Analysis

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Ishihara / Yoshimine
        3. Fines Correction for Liquefaction: Stark/Olson et al.*
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                   Ce = 1.2
        7. Borehole Diameter,                                         Cb= 1.15
        8. Sampling Method,                                          Cs= 1
        9. User request factor of safety (apply to CSR) ,   User= 1.1
           Plot two CSR (fs1=1, fs2=User)
        10. Use Curve Smoothing: Yes*
        * Recommended Options

        In-Situ Test Data:
        Depth   SPT     gamma   Fines
        ft              pcf     %
        ____________________________________
        0.00    24.00   133.00  16.00
        2.00    24.00   133.00  16.00
        5.00    25.00   126.00  8.00
        10.00   35.00   125.00  15.00
        15.00   32.00   118.00  14.00
        20.00   57.00   133.00  18.00
        25.00   50.00   131.00  13.00
        ____________________________________

Output Results:
```
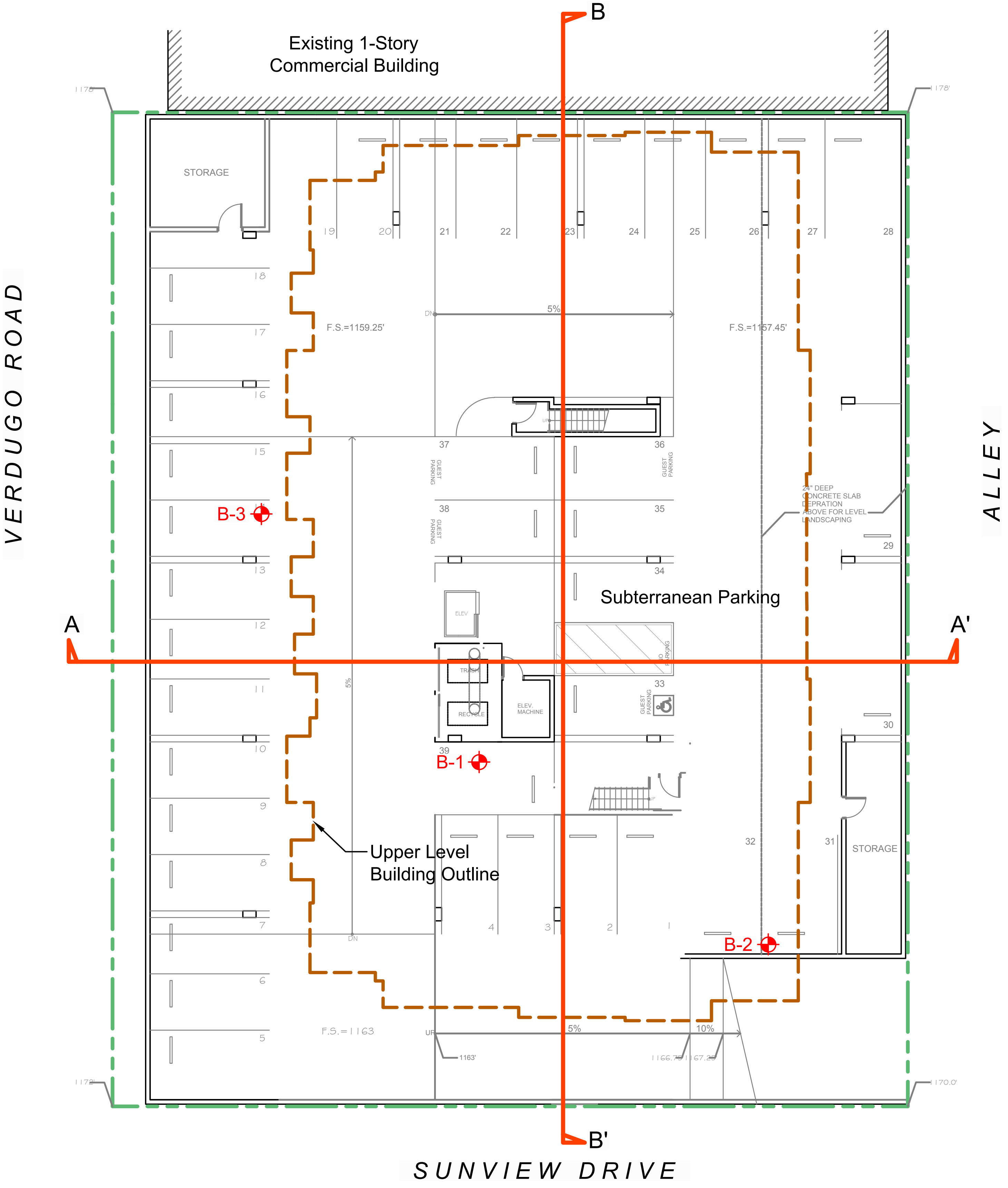

```
        Settlement of Saturated Sands=0.00 in.
        Settlement of Unsaturated Sands=0.10 in.
        Total Settlement of Saturated and Unsaturated Sands=0.10 in.
        Differential Settlement=0.051 to 0.067 in.

        Depth   CRRm    CSRfs   F.S.    S_sat.  S_dry   S_all
        ft                              in.     in.     in.
        _______________________________________________________
        0.00    2.40    0.43    5.00    0.00    0.10    0.10
        2.00    2.40    0.43    5.00    0.00    0.10    0.10
        4.00    2.40    0.42    5.00    0.00    0.10    0.10
        6.00    2.40    0.42    5.00    0.00    0.09    0.09
        8.00    2.40    0.42    5.00    0.00    0.08    0.08
        10.00   2.40    0.42    5.00    0.00    0.08    0.08
        12.00   2.40    0.42    5.00    0.00    0.07    0.07
        14.00   2.40    0.41    5.00    0.00    0.06    0.06
        16.00   2.40    0.41    5.00    0.00    0.05    0.05
        18.00   2.40    0.41    5.00    0.00    0.04    0.04
        20.00   2.40    0.41    5.00    0.00    0.03    0.03
        22.00   2.40    0.41    5.00    0.00    0.02    0.02
        24.00   2.40    0.40    5.00    0.00    0.01    0.01
        26.00   2.41    0.41    5.00    0.00    0.00    0.00
        28.00   2.38    0.42    5.00    0.00    0.00    0.00
        _______________________________________________________
        * F.S.<1, Liquefaction Potential Zone
        (F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)

        Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.

____________________________________________________________________________________
        1 atm (atmosphere) = 1 tsf (ton/ft2)
        CRRm            Cyclic resistance ratio from soils
        CSRsf           Cyclic stress ratio induced by a given earthquake (with user
request factor of safety)
        F.S.            Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        S_sat           Settlement from saturated sands
        S_dry           Settlement from Unsaturated Sands
        S_all           Total Settlement from Saturated and Unsaturated Sands
        NoLiq           No-Liquefy Soils
```



B-3 ⌖ = Location & Number of Boring

Scale: 1" = 20'

Note:
Site plan prepared by using plan drawn by:
-Zohrabians Architects & Builders, Inc.

SITE PLAN

PROJECT No: 16-428-02
DATE: 12 / 13 / 2017
DRAWN BY: VM
CHECKED BY: SM
DRAWING No: 1

DESCRIPTION: Proposed New Mixed-Use Building Project
FOR: Zohrabians Architects
ADDRESS: 3506, 3510, 3512 & 3514 N. Verdugo Blvd. Glendale, CA 91208

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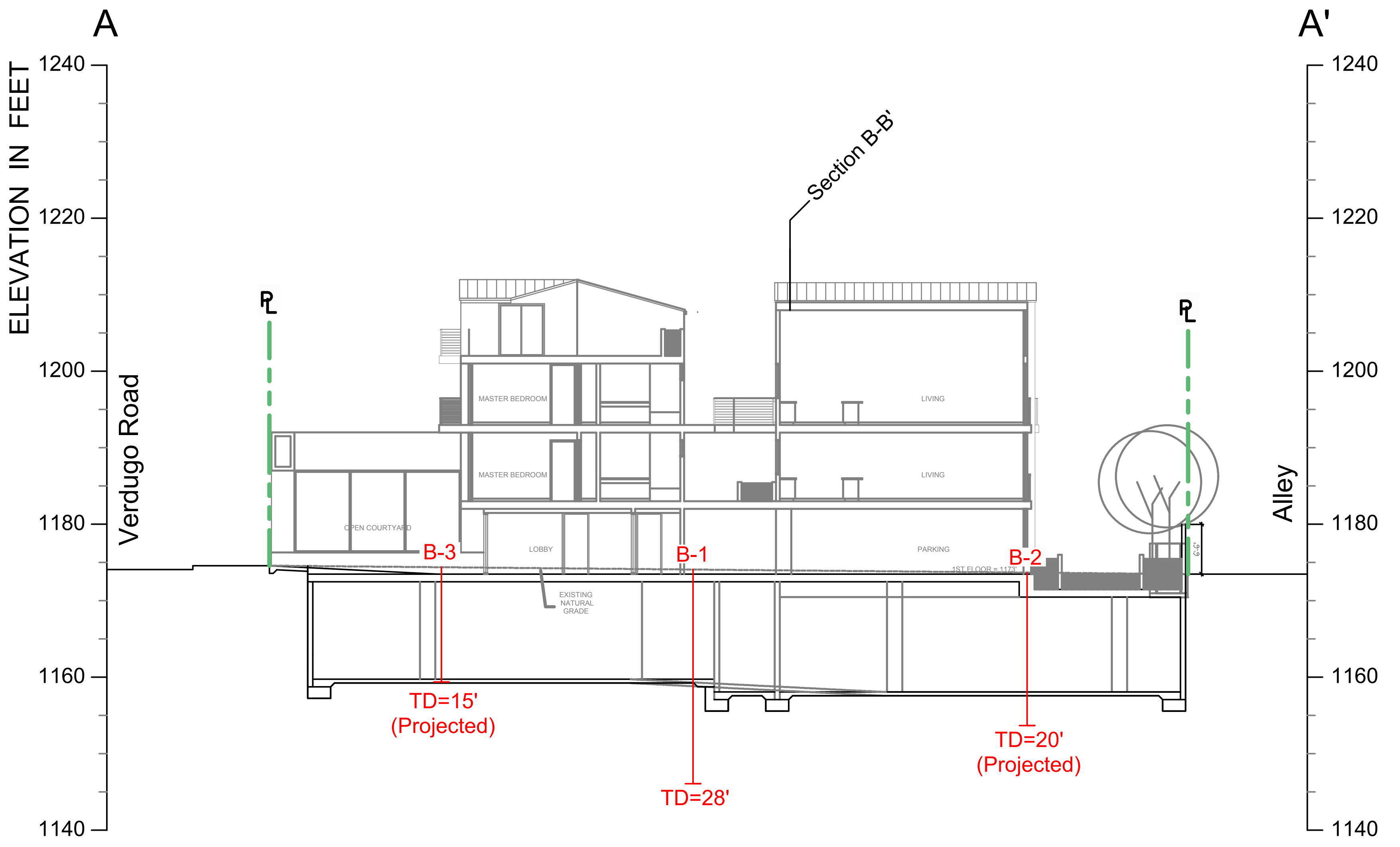
N61°W

Scale: 1" = 20' (V=H)



B-3

TD=10'
(Projected)

= Location & Number of Boring

CROSS SECTION A-A'

DESCRIPTION: Proposed New Mixed-Use Building Project

FOR: Zohrabians Architects

ADDRESS: 3506, 3510, 3512 & 3514 N. Verdugo Blvd. Glendale, CA 91208

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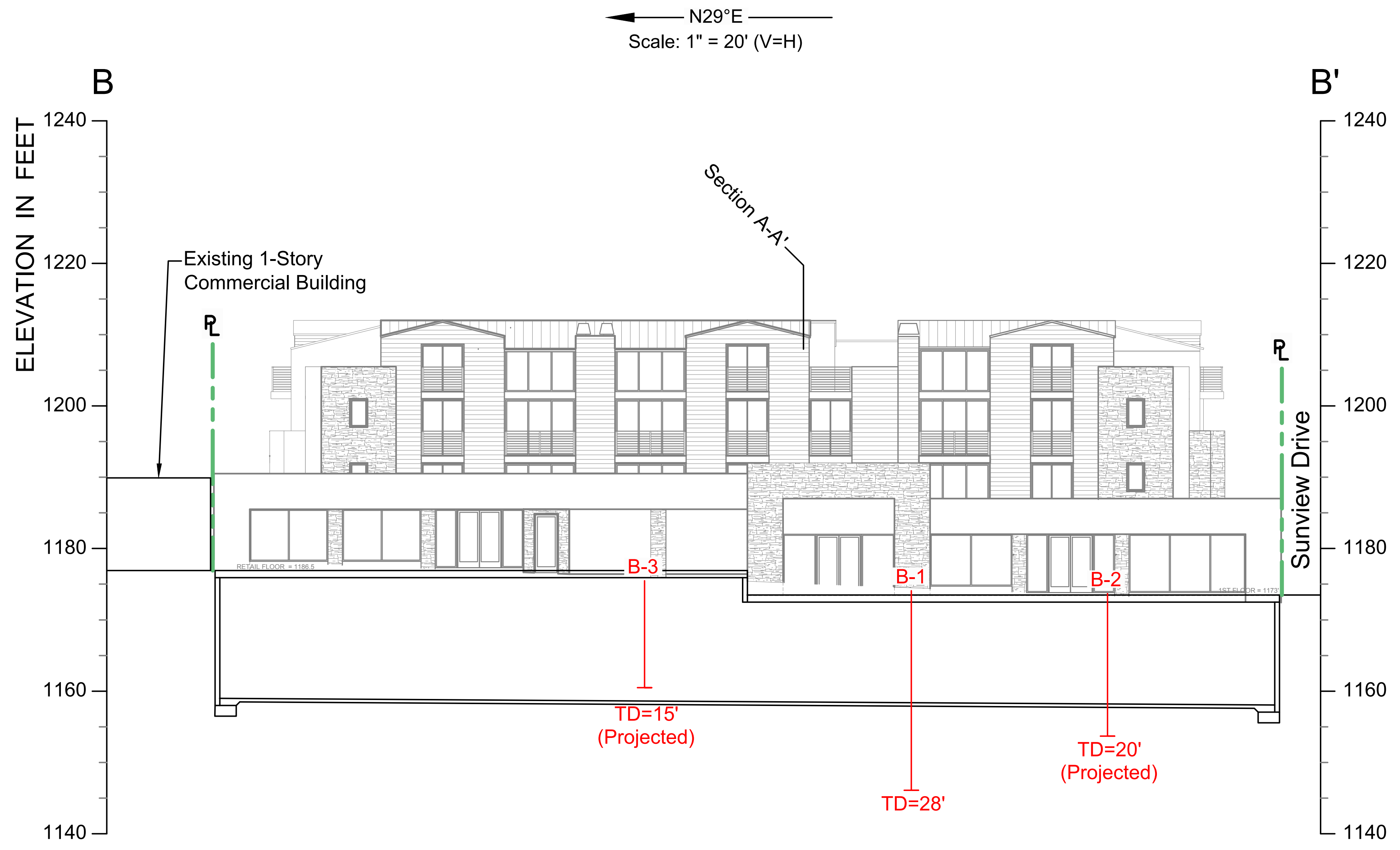
PROJECT No: 16-428-02

DATE: 12 / 13 / 2017

DRAWN BY: VM

CHECKED BY: SM

DRAWING No: 2



B-3
TD=10'
(Projected)
= Location & Number of Boring

CROSS SECTION B-B'

DESCRIPTION: Proposed New Mixed-Use Building Project
FOR: Zohrabians Architects
ADDRESS: 3506, 3510, 3512 & 3514 N. Verdugo Blvd. Glendale, CA 91208

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PROJECT No: 16-428-02
DATE: 12 / 13 / 2017
DRAWN BY: VM
CHECKED BY: SM
DRAWING No: 3

Reference: Portion of Google Maps

SITE VICINITY MAP

Proposed New Mixed-Use Building Project

3506, 3510,3512 & 3514 Verdugo Rd, Glendale CA 91208

FOR	DATE	PROJECT No.
Zohrabians Architects	12 / 13 / 2017	16-428-02

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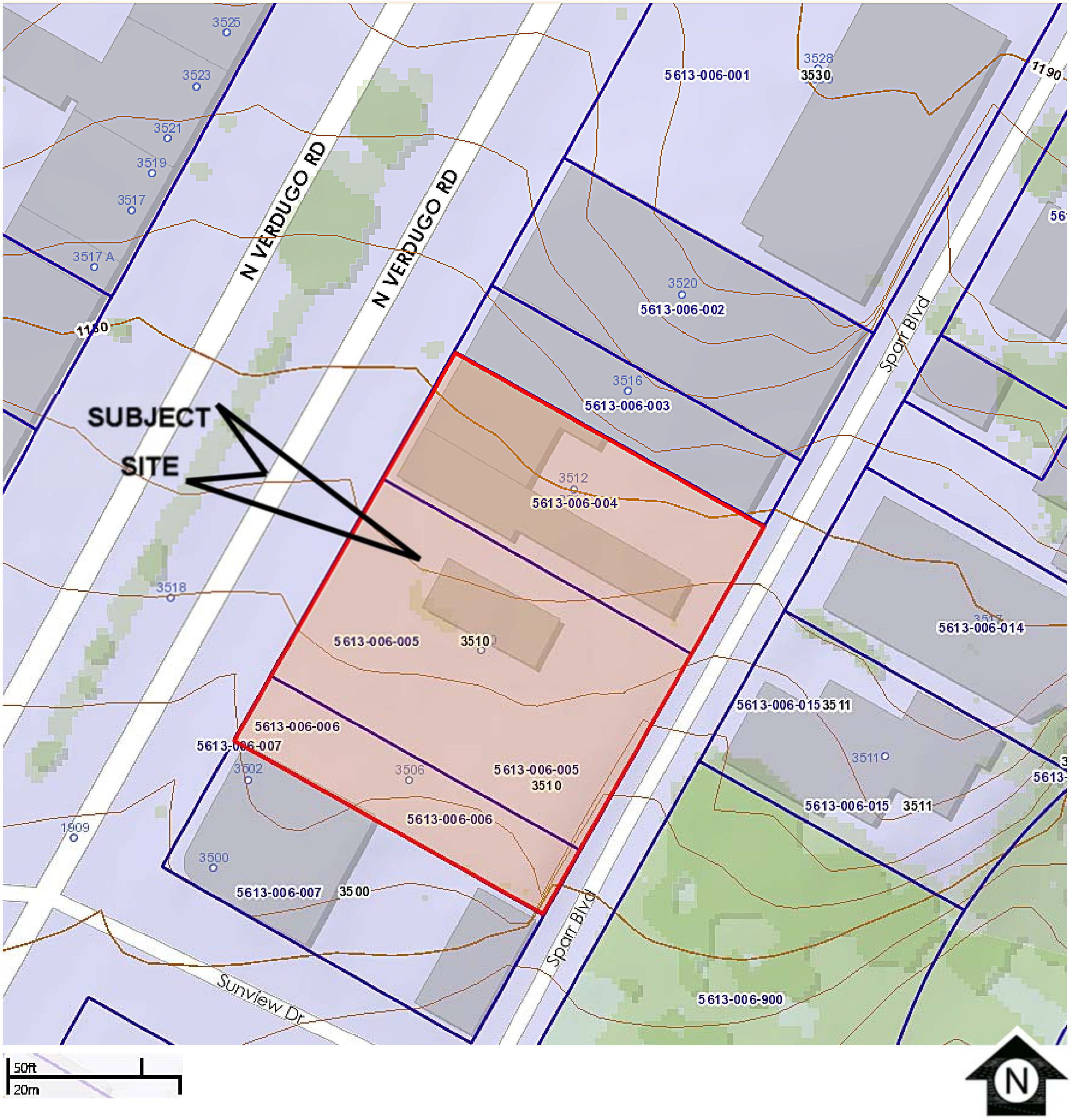
FIGURE No. 1



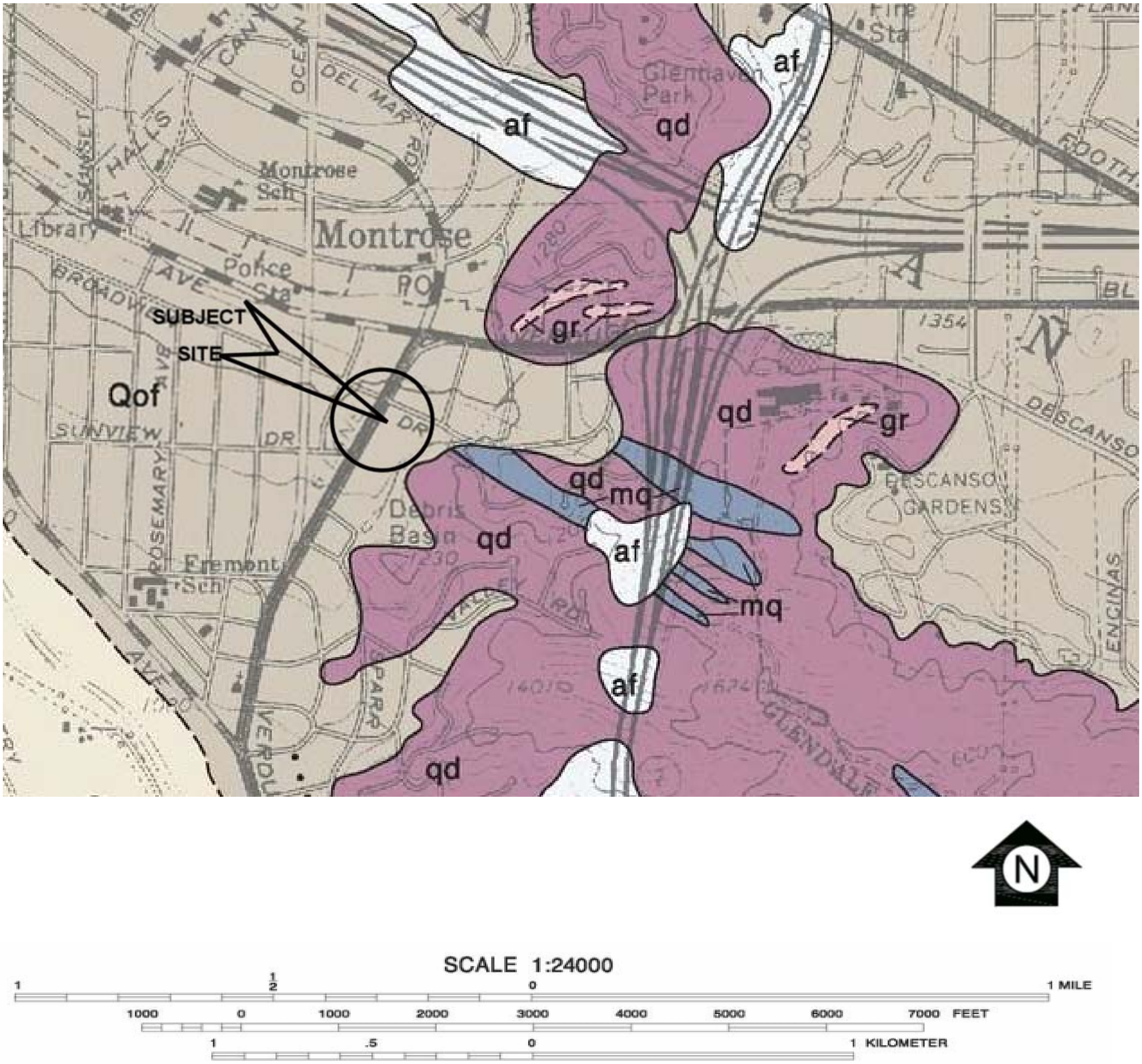
Reference: Navigate LA Los Angeles City

REGIONAL TOPOGRAPHIC

Proposed New Mixed-Use Building Project

3506, 3510,3512 & 3514 Verdugo Rd, Glendale CA 91208

FOR Zohrabians Architects	DATE 12 / 13 / 2017	PROJECT No. 16-428-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 2



OLDER DISSECTED SURFICIAL SEDIMENTS

Qof *Alluvial fan gravel and sand derived from San Gabriel Mountains; some older fans named by Crook, et al., 1987*

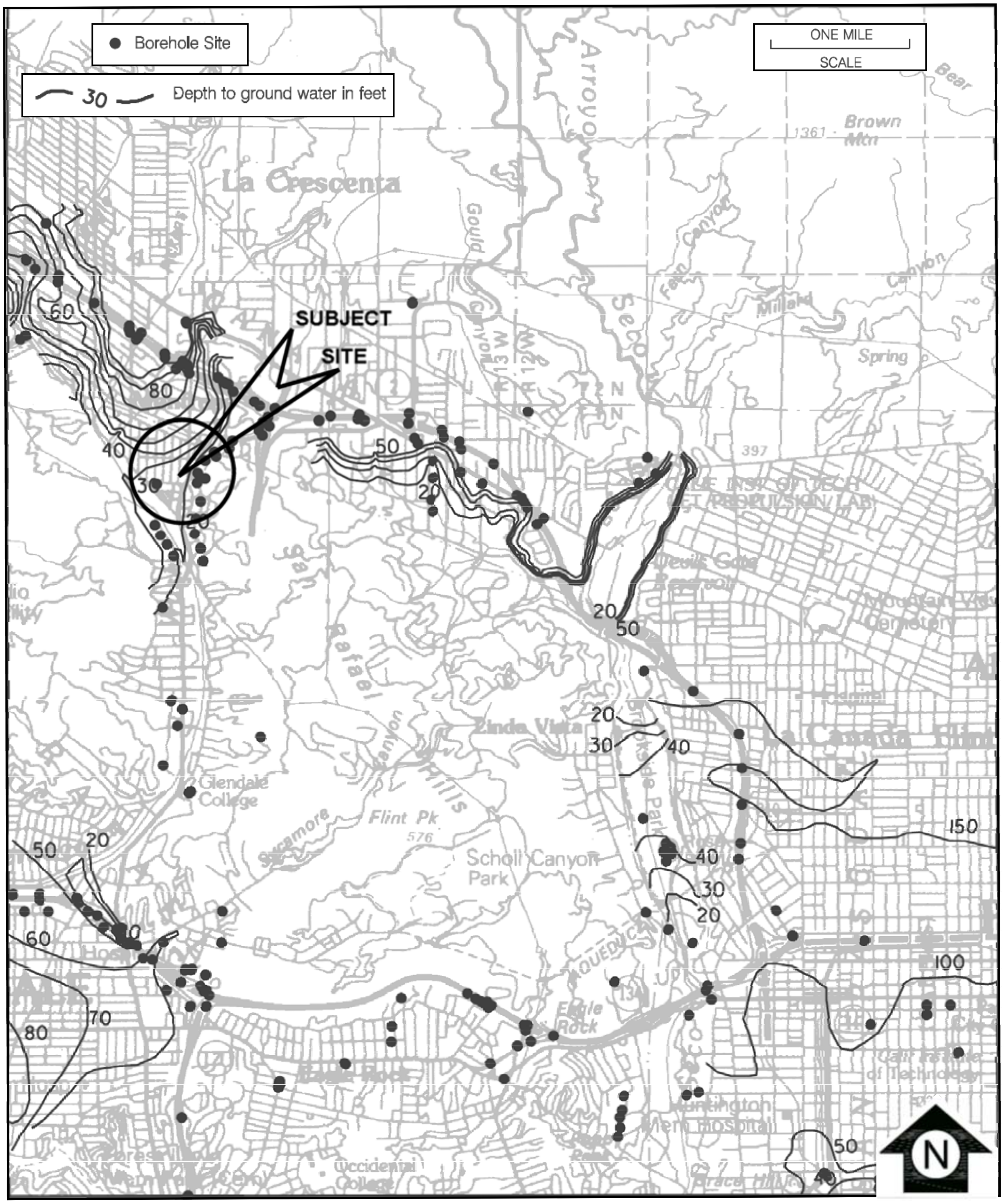
Reference: Dibblee Geologic Map of the Mt. Wilson & Azusa Quadrangle

REGIONAL GEOLOGIC MAP

Proposed New Mixed-Use Building Project

3506, 3510,3512 & 3514 Verdugo Rd, Glendale CA 91208

FOR Zohrabians Architects	DATE 12 / 13 / 2017	PROJECT No. 16-428-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 3



Reference: Pasadena 7.5 Minute Quadrangle

HISTORICALLY HIGHEST GROUNDWATER (Contour Map)

Proposed New Mixed-Use Building Project

3506, 3510,3512 & 3514 Verdugo Rd, Glendale CA 91208

FOR Zohrabians Architects

DATE 12 / 13 / 2017

PROJECT No. 16-428-02

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FIGURE No. 4

APPENDIX I

METHOD OF FIELD EXPLORATION

In order to define subsurface conditions three borings were drilled at the site. The approximate locations of the borings with respect to the existing building are shown on the enclosed Site Plan. The borings were drilled with a hollow stem drilling machine.

Logs of the subsurface materials, as encountered in the borings, were recorded in the field and are presented Figure Nos. I-1 through I-3 within Appendix I. These figures also show the number and approximate depths of each of the recovered soil samples.

With hollow stem drilling, relatively undisturbed samples of the subsoil were obtained by driving a steel sampler with successive drops of a 140-pound sampling hammer free-falling a vertical distance of about 30 inches. The number of blows required for one foot of sampler penetration was recorded at the time of drilling and are shown on the log of exploratory borings. The relatively undisturbed soil samples were retained in brass liner rings 2.5 inches in diameter and 1.0 inch in height.

Field investigation for this project was performed on November 14, 2017. The materials excavated from the test borings were placed back and compacted upon completion of the field work. Such materials may settle. The owner should periodically inspect these areas and notify this office if the settlements create a hazard to person or property.

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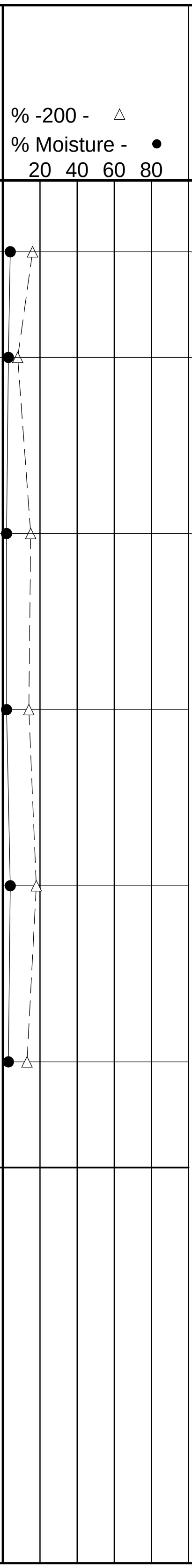
LOG OF BORING NO. 1

16-428-02
3506,3510,3512 & 3514 Verdugo Rd, Glendale, CA 91208

Type: Hollow Stem Auger with 140lb Hammer   Logged by: Ted
Location: *See Site Plan*

DEPTH, FT	DESCRIPTION OF MATERIAL	SPT BLOWS/FT	BLOWS PER FT	% Moisture	UNIT DRY WT LB/CU FT	% -200
0	(SM) FILL: Moderately compact, slightly moist, brown, medium to coarse grained sand with gravel fragments.					
	(SM) SAND: Dense, slightly moist, grayish brown, medium to coarse grained sand, gravelly.		36	4	129	16
5	(SP-SM) Grades to less silty.		37	3	123	8
10	(SM) Grades to more fines.		52	2	122	15
15	(SM) Grades to more gravelly.		48	2	116	14
20	(SM) Grades to very dense.	57		4	128	18
25	(SM) Grades to less silty.	50/3"		3	127	13
30	REFUSAL AT 28 FEET NO WATER ENCOUNTERED HOLE BACKFILLED.					
35						



COMPLETION DEPTH: 28'
DATE: November 14, 2017

DEPTH TO WATER> INITIAL:
FINAL:

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LOG OF BORING NO. 2

16-428-02
3506,3510,3512 & 3514 Verdugo Rd, Glendale, CA 91208

Type: Hollow Stem Auger with 140lb Hammer Logged by: Ted
Location: *See Site Plan*

DEPTH, FT	DESCRIPTION OF MATERIAL	SPT BLOWS/FT	BLOWS PER FT	% Moisture	UNIT DRY WT LB/CU FT	% -200 - △ % Moisture - ● 20 40 60 80	% -200
0	(SM) FILL: Moderately compact, slightly moist, brown, medium to coarse grained sand with gravel fragments.						
	(SP-SM) SAND: Very dense, slightly moist, brown, gravelly, slightly silty sand.		67	1	111		
5	(SP-SM) Grades to dense.		30	3	107		
10	(SM) Grades to more silty.		17	2	96		
15	(SP-SM) Grades to more gravelly, less silty.		32	3	108		
20	(SP-SM) Grades to very dense.		57	3	116		
25							
30							
35							

END OF BORING AT 21 FEET
NO WATER ENCOUNTERED
HOLE BACKFILLED.

COMPLETION DEPTH: 21'
DATE: November 14, 2017

DEPTH TO WATER> INITIAL:
FINAL:

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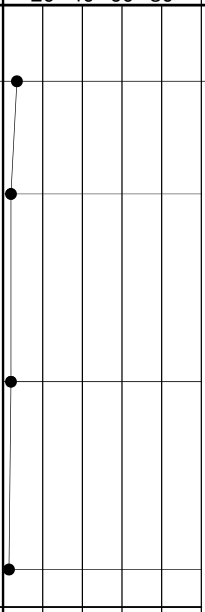
LOG OF BORING NO. 3

16-428-02
3506,3510,3512 & 3514 Verdugo Rd, Glendale, CA 91208

Type: Hollow Stem Auger with 140lb Hammer   Logged by: Ted
Location: *See Site Plan*

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	SPT BLOWS/FT	BLOWS PER FT	% Moisture	UNIT DRY WT LB/CU FT	% -200 - △ % Moisture - ● 20 40 60 80	% -200
0			(SM) FILL: Moderately compact, slightly moist, brown, medium to coarse grained sand with gravel fragments.		20	7	119		
			(SP-SM) SAND: Dense, slightly moist, brown, gravelly silty sand.						
5			(SP-SM) Grades to less silty, more gravelly.		31	4	116		
10			(SP-SM) Similar as above.		38	4	115		
15			(SP-SM) Grades to very dense.		89/3"	3	112		
			END OF BORING AT 16 FEET NO WATER ENCOUNTERED HOLE BACKFILLED.						
20									
25									
30									
35									



COMPLETION DEPTH: 16'
DATE: November 14, 2017

DEPTH TO WATER> INITIAL:
FINAL:

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

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

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO. 200 NO. 40 NO. 10 NO. 4 ¾ in. 3 in. (12 in.)

U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

JOB NAME : Proposed New Mixed-Use Building Project
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FIGURE No. I-4

APPENDIX II

LABORATORY TESTING PROCEDURES

Moisture Density

The moisture-density information provides a summary of soil consistency for each stratum and can also provide a correlation between soils found on this site and other nearby sites. The tests were performed using ASTM D 2216-04 Laboratory Determination of water content Test Method. The dry unit weight and field moisture content were determined for each undisturbed sample, and the results are shown on log of exploratory borings.

Shear Tests

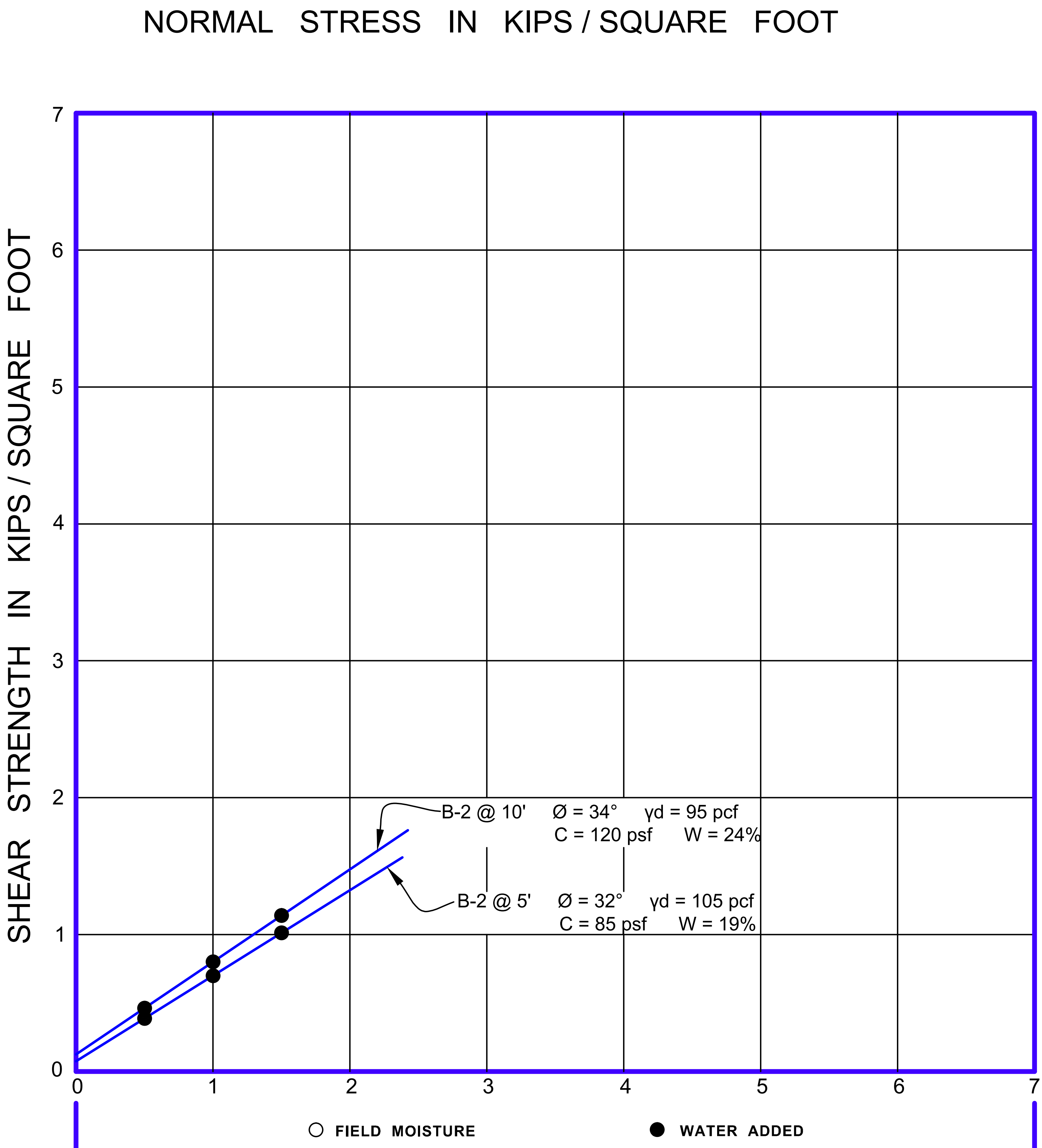
Shear tests were made with a direct shear machine at a constant rate of strain. The machine is designed to test the materials without completely removing the samples from the brass rings. The rate of shear was determined through determination of the rate of consolidation of the foundation bearing materials. For the proposed project, a rate of 0.005 was selected.

A range of normal stresses was applied vertically, and the shear strength was progressively determined at each load in order to determine the internal angle of friction and the cohesion. The tests were performed using ASTM D 3080-04 Laboratory Direct Shear Test Method. The Ultimate shear strength results of direct shear tests are presented on Figure No. II-1 within this Appendix.

Consolidation

The apparatus used for the consolidation tests is designed to receive the undisturbed brass ring of soil as it comes from the field. Loads were applied to the test specimen in several increments, and the resulting deformations were recorded at time intervals. Porous stones were placed in contact with the top and bottom of the specimen to permit the ready addition or release of water. ASTM D 2435-04 Laboratory Consolidation Test Method.

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



DIRECT SHEAR TESTS

JOB NAME : Proposed New Mixed-Use Building Project
3506, 3510, 3512 & 3514 N. Verdugo Blvd.
Glendale, CA 91208

JOB No. 16-428-02

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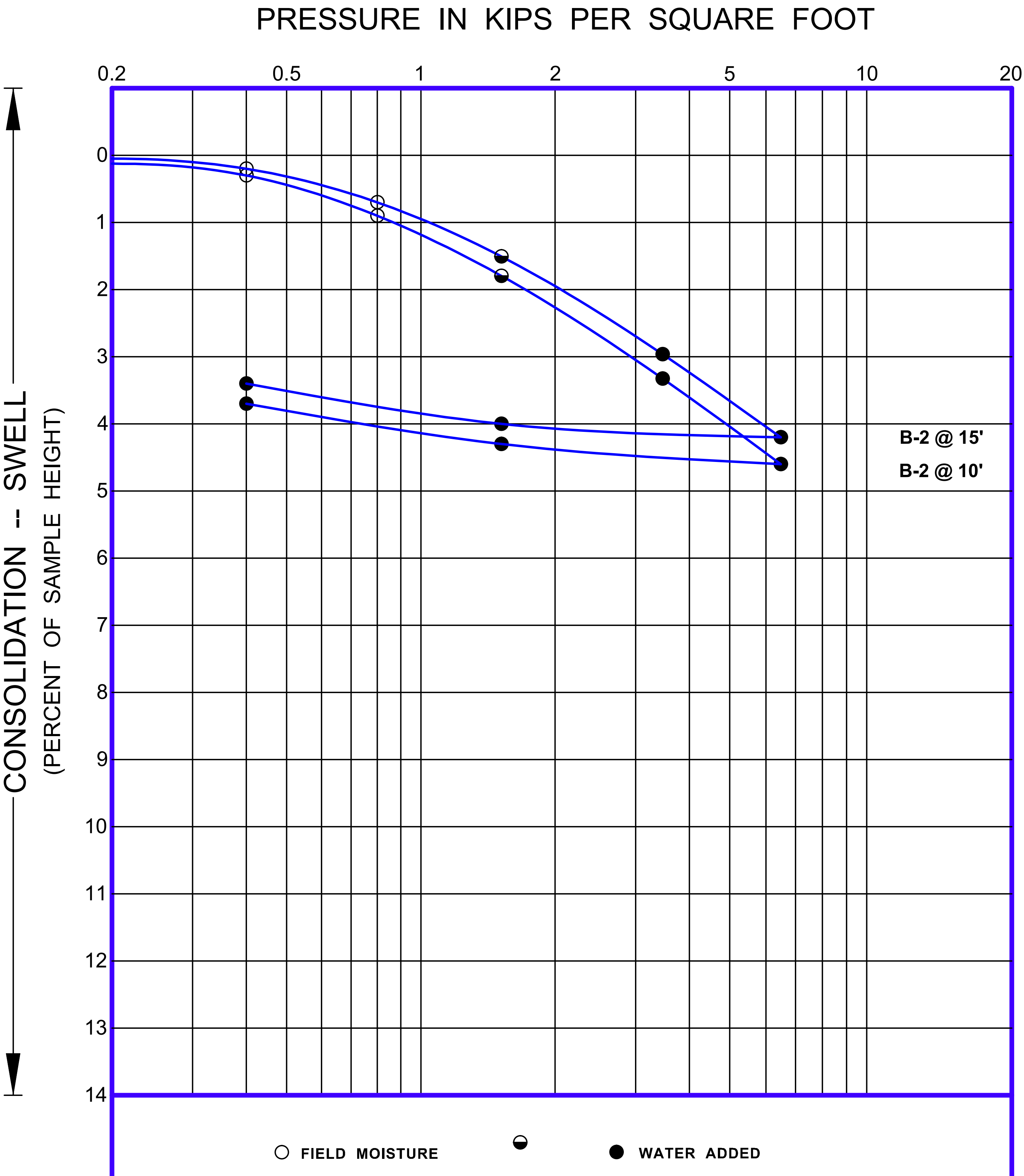
FIGURE No. II - 1



SWELL - CONSOLIDATION TESTS

JOB NAME :	Proposed New Mixed-Use Building Project 3506, 3510, 3512 & 3514 N. Verdugo Blvd. Glendale, CA 91208	JOB No.	16-428-02
Applied Earth Sciences	GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS www.aessoil.com (818) 552-6000	FIGURE No.	II - 2