REPORT OF
GEOTECHNICAL INVESTIGATION
PROPOSED MIX-USE BUILDING PROJECT
727 SONORA AVENUE
GLENDALE, CALIFORNIA 91201

FOR

SONORA REAL ESTATE GROUP, LLC

PROJECT NO. 18-320-02

OCTOBER 10, 2018

October 10, 2018 18-320-02

Sonora Real Estate Group, LLC
1241 South Glendale Avenue
Suite 302
Glendale, California 91205

Attention: Mr. Karen Sarkisyan

Subject: Report Of Geotechnical Investigation
Proposed Mix-Use Building Project
727 Sonora Avenue
Glendale, California 91201

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/shoring, foundations, basement walls, grade slabs, and grading. The 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 were used as reference. The plans were prepared by the offices of Landmark Design & Construction, LLC.

The enclosed Drawing No. 1, shows the approximate locations of the drilled borings in relation to the site boundaries and the proposed building. 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, and 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 borings.

The attached Appendix II describes the laboratory testing procedures. Figure Nos. II-1 through II-3 present the results of direct shear and consolidation tests performed on selected undisturbed samples.

Appendix III present the construction procedure for anchor shafts and observation and testing requirements during the installation of the tieback anchors.

The presented design recommendations for excavation and foundation are based on the provided plans and assumed structural loading data. This office should be consulted, if the actual structural loading and excavation depths are different from those used during this investigation. Modifications to the presented design recommendations may then be made to reflect the actual conditions.

PROJECT CONSIDERATION

It is our understanding that the proposed project will consist of construction of a mix-use building at the subject site. The proposed building is expected to be a 4-story structure constructed over 3 levels of subterranean parking garage. The lowest basement garage grade is expected to be established at some 35 to 40 feet below grade. The ground floor will have commercial/retail use facing the street and additional parking behind the retail stores having access from the rear alley. The upper three floors will be used for residential units. See the enclosed Cross Sections A-A' and B-B' for building profiles.

It is anticipated that the perimeter walls of the basement garage will be extended to close proximity of the respective property lines. Therefore, during the course of basement garage construction, temporary shoring will be required. The temporary shoring system should be in a form of cantilevered soldier piles (in the areas of ramp) where total height of excavation is no greater than 15 feet. In the areas where total height of excavation exceed 15 feet, the soldier piles should be laterally supported by internal bracing or anchor tie-back.

Unsupported, open excavation slopes with inclinations as recommended in this report may be used for the internal excavations (footings, elevator shafts, etc.).

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 850 kips, combined dead plus frequently applied live loads. Perimeter wall footings are expected to exert loads of on the order of 36 kips per lineal foot.

SITE GRADING

Site grading for the proposed project is expected to involve the following:

1. Excavation in order to establish the lowest level of the basement garage;
2. Backfilling behind retaining walls within the over-excavated areas;
3. Backfilling in the ramp areas; and
4. Subgrade preparation for basement garage slabs.

The wall backfill materials should consist of non-expansive/granular soils. Therefore, the excavated materials from the site can be used for wall backfilling.

SITE CONDITIONS

SURFACE CONDITIONS

The subject site is located at 727 Sonora Avenue, Glendale, California. The site is rectangular in shape covering a plan area of about 12,500 square feet.

At the time of our field investigation, the site was occupied by a commercial (automotive service) business. The site was noted to be generally level.

Existing off-site improvements occur around the site. See the enclosed Site Plan; Drawing No. 1, for detail.

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, sandy silt and relatively clean sand soils with variable amounts of gravel and few cobbles. Thickness of the existing fill was found to be as much as 2 feet at the location of our borings. Deeper fill, however, may be present between and beyond our borings. Such fill soils, however, are expected to be automatically removed by the planned basement garage excavations.

The upper native soils through which the basement garage excavations will be made were found to be medium dense to dense to very dense silty sand and sand and stiff sandy silt. The results of our laboratory investigation indicated that these materials were of moderate to high strengths.

The soils near the planned foundation levels were found to be consist of generally very dense, silty and/or gravely sand soils with little to no fines. The results of our laboratory testing indicated that these materials were of high strengths and low compression.

The site soils (including those at the basement garage level) were found to be granular in nature. These soils are considered to be virtually non-expansive.

GROUNDWATER

During the course of our investigation, no groundwater was encountered in our borings drilled to a maximum depth of 51 feet. The State Maps, however, show the historically highest groundwater in the vicinity of the subject site to be about 35 feet deep. See the enclosed Figure No. 4.

CAVING CONSIDERATIONS

Due to method of drilling (use of continuous auger) caving was not detected during the course of our field exploration program. Typical soils, however, are considered to be susceptible to caving within large scale excavation and in drilled holes.

On the basis of the above, therefore, forming may be required during foundation construction. Also, full lagging (placed from the top as the excavation advances) will be required between the soldier piles. For typical sites, it is common to drill alternate piles during the course of shoring construction. If caving persists in a drilled hole, the boring should be backfilled with 1.5-sack slurry mix and re-drilled the following day.

DE-WATERING

TEMPORARY

Considering that no water was found in our borings drilled to a maximum depth of 51 feet, temporary de-watering (during construction) will not be necessary for this project

PERMANENT

Considering that the base of the proposed building will be established close to, or deeper than the historically highest groundwater level, for a typical project, it is common to use permanent de-watering (post construction). The system is placed beneath the basement slab. This normally consists of trenches (no greater than 25 feet apart) that are filled with pipe and gravel. The collected water would then be diverted to a sump and be pumped out. Due to the current strict environmental rules of discharging water to the curb line and then storm drain, in-lieu of placing permanent de-watering system, it is common to design and basement slab for hydrostatic uplift loads. For the purpose of this project, and assuming a 5 feet fluctuation from the historically highest level, the basement slab would need to be designed for hydrostatic pressure assuming water level near a depth of about 30 feet.

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=2.667 (short period) and S_1 =0.843 (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:

Sms= Fa (Ss) = 1.0 (2.667) = 2.667

Sm1=Fv (S1) = 1.5 (0.843) = 1.265

Sds=2/3 (Sms) = 2/3 (2.667) = 1.778 and

Sd1=2/3 (Sm1) = 2/3 (1.265) = 0.843

EVALUATION OF LIQUEFACTION POTENTIAL

As part of our field exploration, borings were drilled at the subject site to a maximum depth of 51 feet. No groundwater was found in our deep boring.

The State Maps, however, show the historically highest groundwater in the vicinity of the subject site to be shallower about 35 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 10 feet.

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 0.664g) and the predominant earthquake magnitude of 6.90 with 10% probability of exceedance in 50 years (475-year return period) indicated a factor of safety of greater than 1.1. The corresponding seismic related settlements was found to be very small (0.17 inches).

When using higher level peak ground acceleration value of 0.996g 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 7.05, 2% probability of exceedance in 50 years (2475-year return period) again, a factor of safety of greater than 1.1 was obtained. The corresponding seismic settlements was found to be 0.75 inches. See the enclosed Engineering calculations.

Based on the above, therefore, it is our opinion that soil liquefaction will not occur at this 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. Conventional spread footing foundation system could be used for support of the proposed building. The foundation bearing materials are expected to be very dense, relatively clean, gravely sand native soils.

It is anticipated that the basement garage excavations will be made through surficial fill and native soils consisting of mainly sand soils (with variable amounts of fines and gravel) and localized sandy silt lenses. The height of excavation to the perimeter wall footing levels of the basement garage is expected to range from about 35 to 40 feet.

It is anticipated that the perimeter walls of the basement garage of the proposed building will be extended to close proximity of the sides and rear property lines. Therefore, during the course of basement garage construction, temporary shoring will be required. The shoring system should consist of soldier piles with lateral support (interior bracing or tie-back anchor shafts). It is anticipated that one to two rows of anchor shafts (depending upon the magnitude of the vertical cut) will be required for the proposed project. Unsupported, open excavation slopes with inclinations as recommended in this report may be used for the internal excavations (footings, elevator shafts, etc.).

The basement slabs can be supported on the exposed subgrade, provided that any disturbed soils would be compacted in-place to a relative compaction of at least 90 percent at optimum moisture content. All fill soils placed over the interior footings should also be compacted to a relative compaction of at least 90 percent at optimum moisture content. Due to granular nature, soil expansion will not be an issue at this site.

Permanent de-watering (post construction) will be required for this project. This normally consists of excavating trenches below the basement slabs (no greater than 25 feet apart) that are filled with pipe and gravel. The trenches should have a minimum depth of 18 inches (measured from the bas of the slab). The collected water would then be diverted to a sump and be pumped out.

Due to the current strict environmental rules of discharging water to the curb line and then storm drain, in-lieu of placing permanent de-watering system, it is common to design and basement slab for hydrostatic uplift loads. For the purpose of this project, and assuming a 5 feet fluctuation from the historically highest level, the basement slab would need to be designed for hydrostatic pressure assuming water level near a depth of about 30 feet.

The following sections present our specific recommendations for temporary excavations, foundations, lateral design, basement grade slabs, subsurface walls, 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)
0-4	3/4:1
>4	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: Where total height of excavation is no greater than 15 feet, cantilevered soldier piles can be used as a means of temporary shoring. Soldier piles consist of structural steel beams encased in concrete below the basement level and slurry mix within the upper (exposed) portions.

The lateral resistance for cantilevered soldier piles may be assumed to be offered by available passive pressure below the basement level. An allowable passive pressure of 500 pounds per square foot per foot of depth may be used below the basement level for soldier piles having center-to-center spacing of at least 2-1/2 times the pile diameter. Maximum allowable passive pressure should be limited to 6,000 pounds per square foot. The maximum center-to-center spacing of the vertical shafts should be maintained 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.

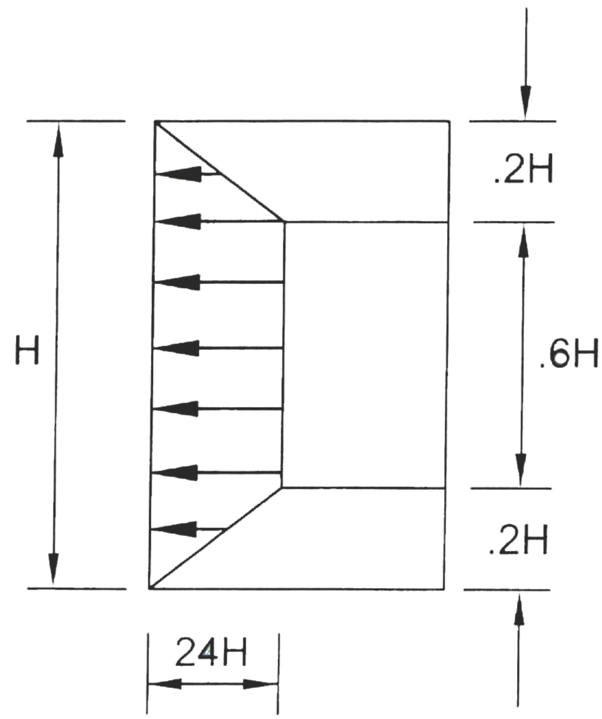
When using cantilevered soldier piles for temporary shoring, the point of fixity (for the purpose of moment calculations), may be assumed to occur at some 2 feet below the base of the excavation. In order to limit local sloughing, it is recommended that lagging be used where fill is exposed between the soldier piles. All wood members left in ground should be pressure treated. The lagging should be designed based on an equivalent fluid pressure of 8 pounds per square foot per foot of depth. Maximum pressure on lagging should be limited to 400 pounds per square foot.

Caving may be experienced during drilling of the shoring piles. The caved holes should be filled with slurry mix and re-drilled the following day. For typical sites, it is common to drill alternate piles during the course of shoring construction.

Braced Shoring: Where total height of excavation exceed 15 feet, the vertical shafts should be laterally supported by internal bracing or anchor tie-back. It is anticipated that one to two rows of anchor shafts will be required for this project. It should be noted that, if tie backs are used, permissions should be obtained to extend the anchor shafts beneath the adjacent properties. Also, the foundations of the off-site structures and utility lines within the anticipated lengths of the tie back anchors should be studied to assure that the existing substructures would not be interfered by the installation of the anchor shafts. The anchor shafts should be tested for the pullout capacities.

The anchors normally consist of drilled, cast-in-place concrete shafts stressed against and tied to the vertical soldier piles. These elements are drilled in an inclined manner beneath the adjacent grounds after the basement excavation is reached to the levels of the anchor rows.

When internal bracing or tieback anchors are used against the vertical piles, trapezoidal pressure distribution should be used for design of the temporary shoring. The following sketch shows the recommended lateral earth pressure distribution behind restrained shoring system.



Lateral pressure due to uniform surcharge loads, such as those from existing off-site improvements, should be added to the above pressure diagram. Such loads should be computed using an at-rest pressure coefficient of 0.40 times the assumed uniform loads.

It is noted that, where off-site buildings occur within a horizontal distance equal to the height of excavation, the tolerable limit of lateral movement at the top of the shoring piles could be limited to ½ of one inch. Where the shoring system supports public right-of-way, and where off-site buildings occur at least 20 feet from the planned line of excavation, the tolerable lateral movement at the tops of the shoring piles can be increased to one inch. The temporary shoring should be monitored after the excavation reaches the final depth. The frequency of monitoring would depend on the rate of movement of the piles. The results of monitoring should be provided to the Project Soil and Structural Engineers for review and comment. If excessive lateral movements are noted, additional lateral support system in a form of added tie back anchors or internal bracing may be required.

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

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

A unit friction value of 650 pounds per square foot may be used to calculate the load supporting capacities of the anchor tie backs. This assumes that the concrete will be placed using gravity. For post grouted anchors where the concrete is placed using high pressure (between 700 to 1,000 psi) a skin friction value of 2,500 pounds per square foot can be used.

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

It is possible that the calculated capacities of the anchors based on the given unit friction value would be significantly different from the actual capacities based on the developed friction values. It is, therefore, suggested that the first series of the installed anchors be tested to verify the calculated capacities. The friction value may then be modified based on the actual capacities of the anchor shafts.

The construction procedure of the anchor shafts and observation and testing requirements during the installation of the tieback anchors are presented in the Appendix III attached to this report.

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.

FOUNDATIONS

Conventional spread footing foundation systems could be used to support the proposed building. The foundation bearing materials are expected to be dense to very dense, gravely sand soils with little to no fines.

Exterior and interior footings should have a minimum width of 24 inches. Footings 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 medium dense native soils could be taken as 4,500 pounds per square foot. This value may be increased at a rate of 200 and 400 pounds per square foot for each additional foot of footing width and depth, to a maximum value of 7,500 pounds per square foot.

The above given 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 maximum soil pressure, footings carrying the assumed maximum concentrated loads of 850 kips are expected to settle on the order of one inch. Continuous footings, with loads of about 36 kips per linear foot are expected to settle on the order of 3/4 of one inch. Maximum differential settlements are expected to be on the order of 1/4 of an inch. The major portion of the settlements are expected to occur during construction.

LATERAL DESIGN

Lateral resistance at the base of footings in contact with native soils may be assumed to be the product of the dead load forces and a coefficient of friction of 0.35. 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 4,000 pounds per square foot may be used for footings poured against native soils.

GRADE SLABS

The basement garage slabs can be supported on the exposed subgrade, provided that any disturbed soils would be compacted in-place to a relative compaction of at least 90 percent at optimum moisture content. All fill soils placed over the interior footings should also be compacted to a relative compaction of at least 90 percent at optimum moisture content. Due to granular nature, soil expansion will not be an issue at this site.

The basement garage slab of the proposed building should be equipped with permanent de-watering. Alternatively, the basement garage slabs can be designed for hydrostatic uplift pressure assuming water level near a depth of about 30 feet.

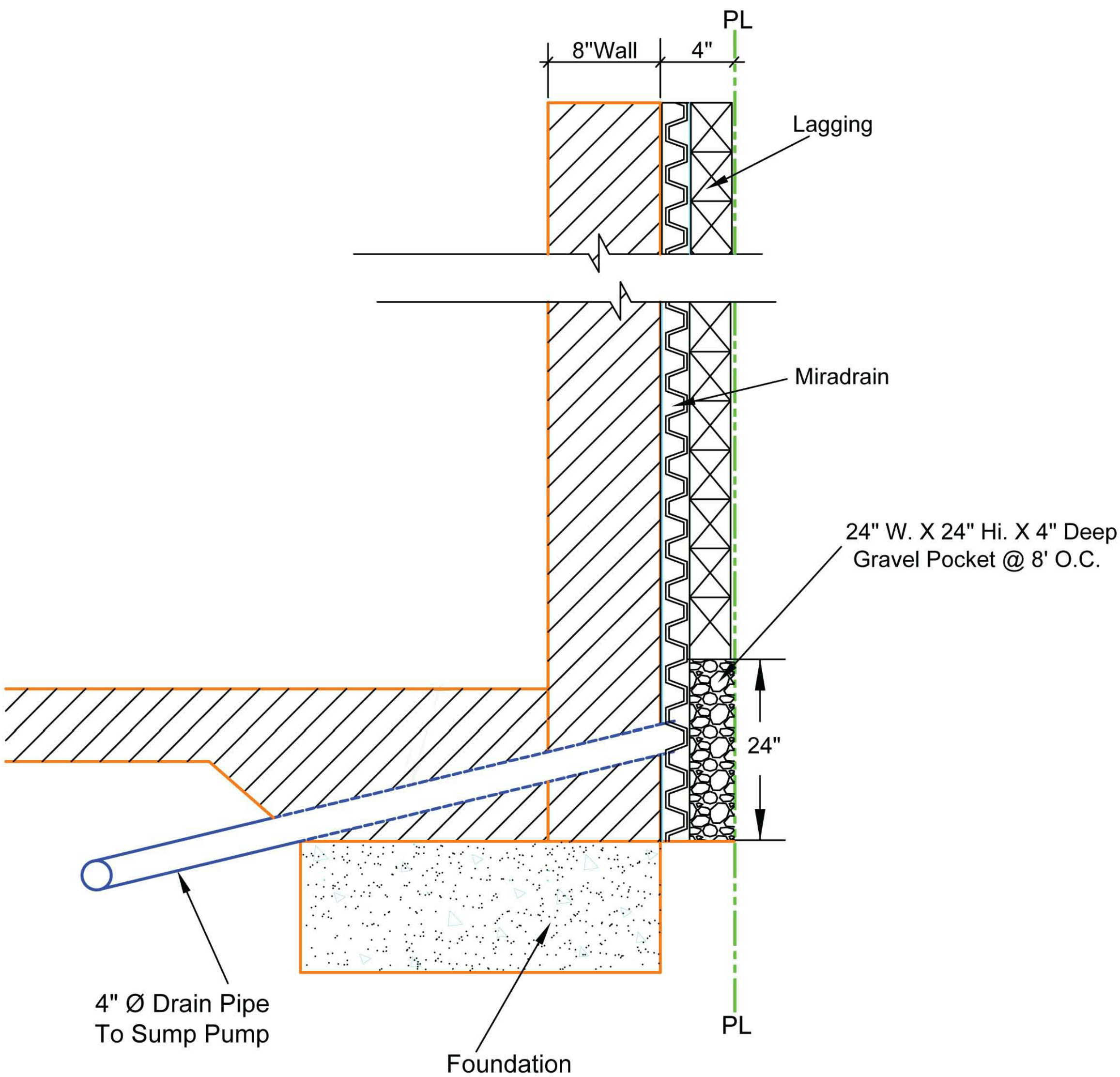
It is recommended that considerations be given to the use of proper waterproofing for all structures that are established below the historically highest groundwater level. The waterproofing should be made by an experienced contractor familiar with similar projects.

BASEMENT WALLS

The cantilevered walls (in the driveway ramp areas) can be designed for an equivalent fluid pressure of 35 pounds per square foot per foot of depth. The perimeter walls of the basement garage that are restrained against rotation should be designed based on “at rest” lateral earth pressure with a magnitude of 61 pounds per square foot per foot of depth (see the enclosed supporting engineering calculation sheets).

The above given pressure assumes that hydrostatic pressure will be relieved from the back of the walls through a properly designed and constructed subdrain system. This normally consists of 4-inch diameter perforated pipes encased in free-draining gravel (at least one cubic foot per lineal foot of the pipes). In order to reduce the chances of siltation which would cause clogging of the drain pipes, 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 basement walls is not available to install standard subdrain (pipe and gravel) an alternative wall backdrain can be used. See the following Sketch No. 1.



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

It is noted that, based on the new Code requirement, the retaining walls higher than 6 feet 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 zero at the base of the excavation and increased upward at a rate of 32 pounds per reducing depth to the maximum value at the ground surface. The seismic lateral earth should be an additive to the active pressure. The point of application of the lateral thrust of the seismic pressure should be assumed 0.6 time the wall height, measured from the top of the wall.

The backs of all subsurface walls should be properly waterproofed. This will help reduce the chances of moisture intrusion into the basement.

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

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. The wall backfill materials should consist of non-expansive granular soils.

Prior to placing any fill, the Soil Engineer should observe the excavation bottoms. In the areas of fill, all soils should be removed until bedrock is exposed. The areas to receive compacted fill should be scarified to a depth of about 8 inches, moistened as required to bring to approximately optimum moisture content, and compacted to at least

90 percent of the maximum dry density as determined by the ASTM Designation D 1557 Compaction Method.

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

1. All fill should be granular in nature. Therefore, the excavated site materials 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 1557-02 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 be expanded to 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 PERCOLATION TESTING

During the course of our original investigation, although no water was found to the maximum depth of 51 feet in our borings, the State maps show the historically highest groundwater level to be near a depth of about 35 feet. As part of our investigation, percolations testing was conducted for dry-well, however the results are not included in the report because the subject site is considered not to be a good candidate for on-site storm water infiltration. The reason is that the base of the proposed building occurs below the historically highest groundwater level. This will not leave the required 10 foot natural filtration zone (as required by the Sanitation District), below the base of the building. 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 very dense native soils. All footing excavations should be observed by a representative of this office before reinforcing is placed.

The depths of soldier piles should be confirmed by a representative of this office before concrete is placed. It is essential to assure that soldier piles are drilled to proper depths and diameters, and in accordance with the project plans and specifications. Also, all anchor shafts should be tested for pull out capacity before locking the design loads. The anchor testing should be made under continuous observation and testing by a representative of this office.

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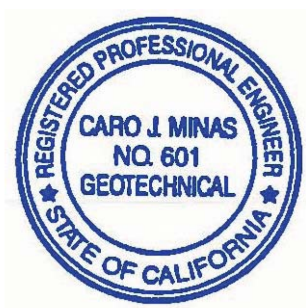
The following Figures and Appendices are attached and complete this report:

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-1through II-3
Appendix III- Construction Procedure For Anchor Tieback

Respectfully Submitted,

APPLIED EARTH SCIENCES

Caro J. Minas, President
Geotechnical Engineer
GE 601



CJM/se

Distribution: (3) Addressee

<u>Average Soil Strength Parameters</u>

Saturated Unit Weight = γs = 125 pcf

Value of Fiction Angle = ϕ = 31 ^{o}

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

$$K_o = 1 - \sin\ 31^o$$

$$K_o = 1 - 0.52$$

$$K_o = 0.48$$

$$\gamma_o = K_o * \gamma$$

$$\gamma_o = 0.48 * 125$$

$$\gamma_o = 60.6$$

At-Rest Equivalent Fluid Density, $\boxed{\gamma_o = \mathbf{61}\ \text{PCF}}$

AT-REST LATERAL EARTH PRESSURE

Basement Walls

FOR: 727 Sonora Ave. Glendale	**DATE:** 10/10/18	PROJECT.NO.:18-320-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		**CALC SHEET No. 1**

<u>Average Soil Strength Parameters</u>

* FIGURE 2 of Naval Facilities Engineering Command

Saturated Unit Weight	ϒ=	125 PCF
Height of Wall	H=	30 Ft.
	PGA_M=	0.996

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

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

K_h= 2/3 * 0.996 / 2

K_h= 0.33

P_{AE} = 3/8 * 125 * 900 * 0.33

P_{AE} = 14006 lb.

<u>Equivelent Fliud Pressure (EFP)</u>

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

EFP= 2 * 14006 / 900

$\boxed{\text{EFP= } \mathbf{31.13\ \ PCF}}$

SEISMIC LATERAL EARTH PRESSURE

Basement Walls

FOR: 727 Sonora Ave. Glendale	**DATE:** 10/10/2018	**PROJECT NO.:** 18-320-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		**CALC SHEET NO. 2**

Average Soil Strength Parameters

Saturated Unit Weight γ = 125 pcf

C = 195 psf

ϕ = 31 °

Height of Wall

H= 30 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	254.6	31.8	34.47	60.5	13.8	4.7	3.3
					13.8	8.0	

F.S. = ∑ RF / ∑ DF = 7.99 / 13.77 = 0.58

FOR FACTOR OF SAFETY = 1.25 (TEMPORARY)

1.25 (DF) = (RF) + UBF

1.25 * 13.77 = 7.99 + UBF

UBF = 17.21 - 7.99 = 9.22 k/lft.

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

G_h = 20.5 pcf

Therefore use Recommended value of 30 pcf

FOR FACTOR OF SAFETY = 1.5 (PERMANENT)

1.5 (DF) = (RF) + UBF

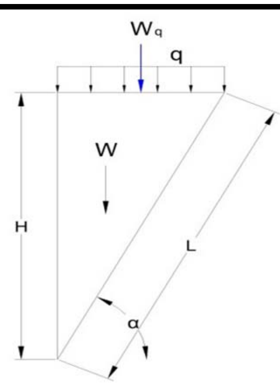
1.5 * 13.77 = 7.99 + UBF

UBF = 20.65 - 7.99 = 12.66 k/lft.

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

G_h = 28.1 pcf

Therefore use Recommended value of 35 pcf



LATERAL EARTH PRESSURE CALCULATIONS

CANTILEVERED SYSTEM

SECTION A-A' - North Facing Basement Walls

FOR: 727 Sonora Ave. Glendale | **DATE:** 10/10/18 | **PROJECT NO.:** 18-320-02

APPLIED EARTH SCIENCES

GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS

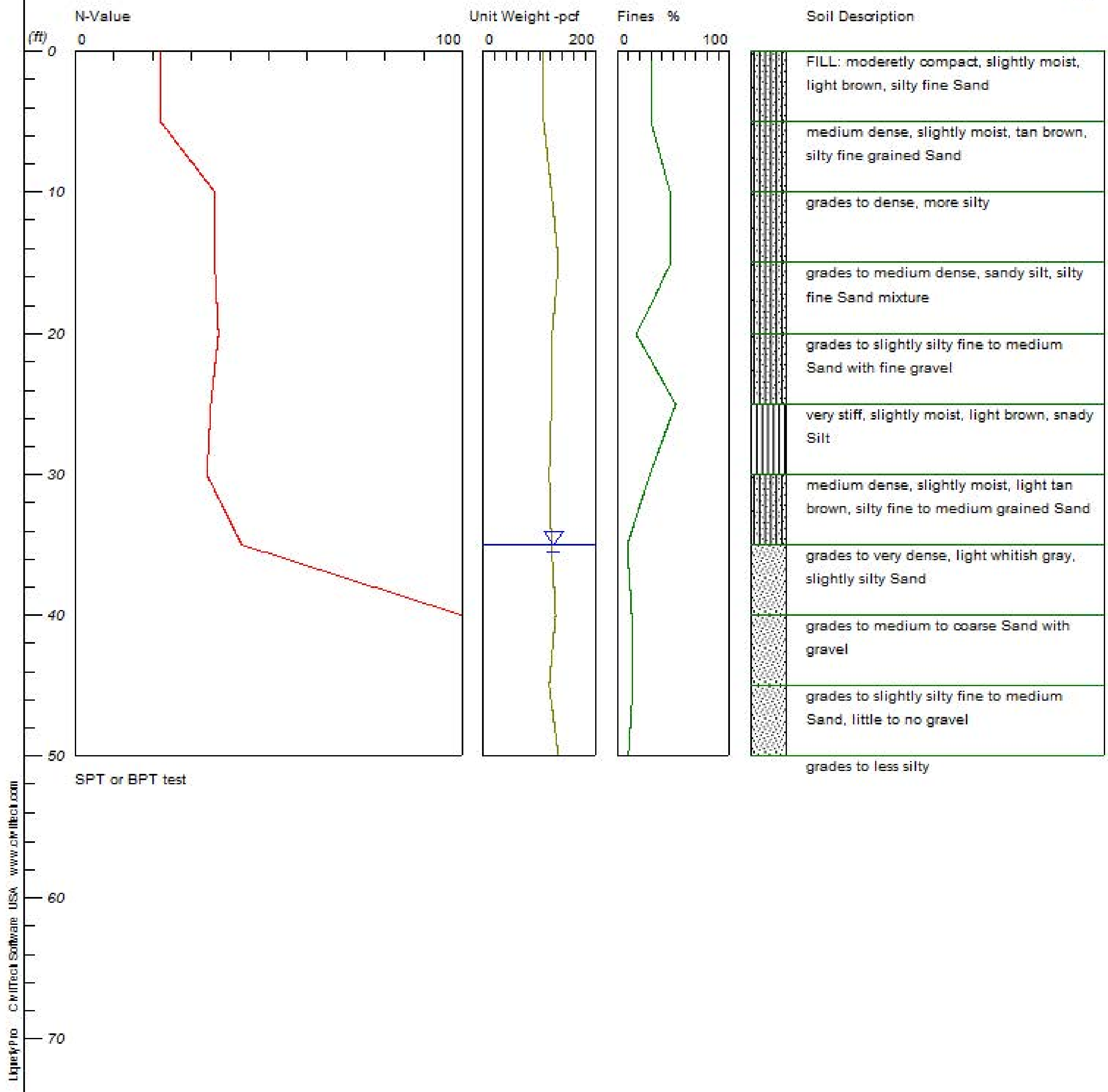
TABLE No. 1

LIQUEFACTION ANALYSIS

727 Sonora Ave. Glendale

Hole No.=B-2   Water Depth=35 ft

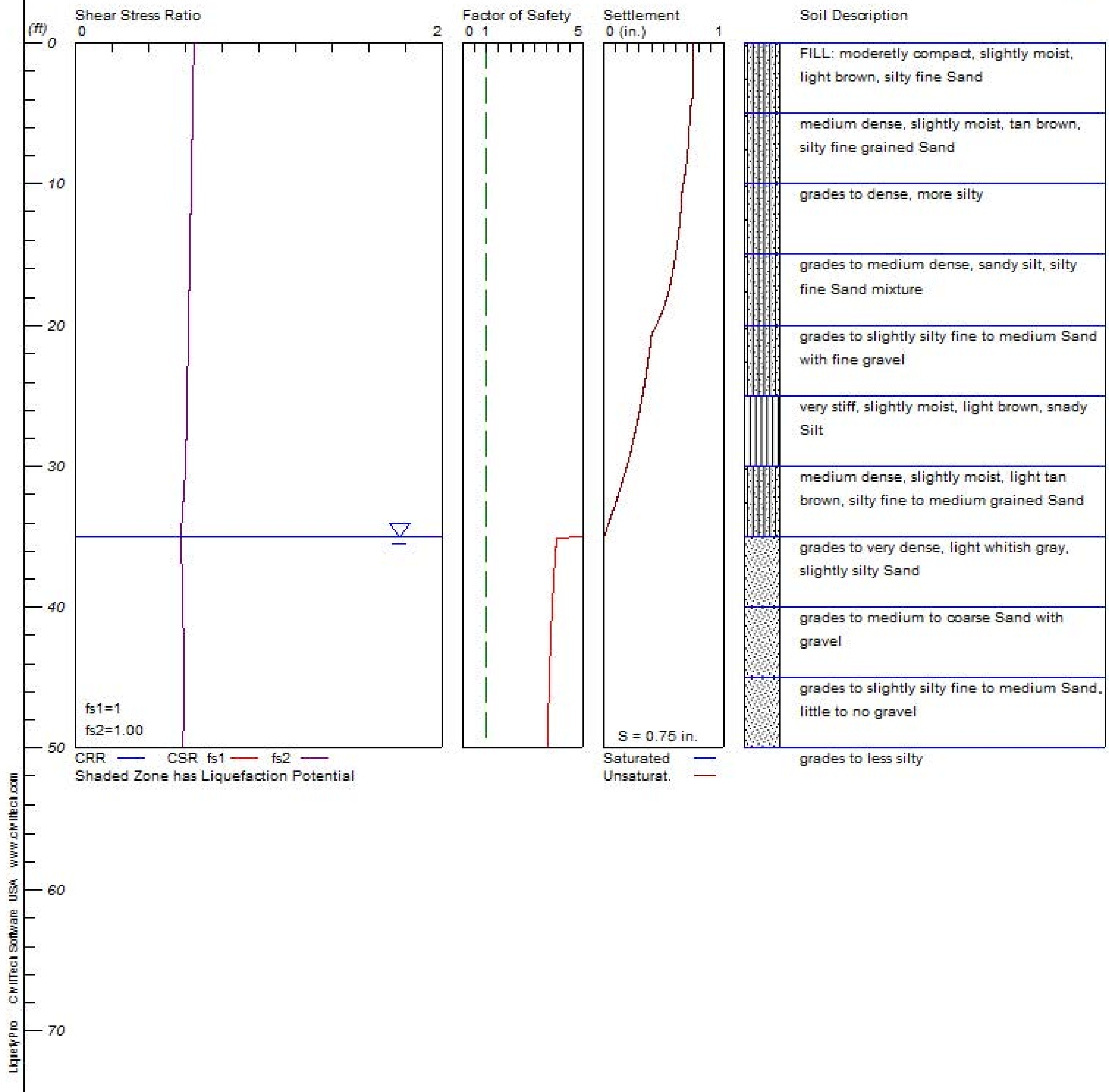
Magnitude=7.05
Acceleration=0.996g



LIQUEFACTION ANALYSIS
727 Sonora Ave. Glendale

Hole No.=B-2 Water Depth=35 ft

Magnitude=7.05
Acceleration=0.996g



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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: P:\Projects-2018\18-320-11, 02 & 24\Engineering-Calculation\Liquefaction\18-320-02_2%.liq
Title: 727 Sonora Ave. Glendale
Subtitle: 18-320-02_2%

Surface Elev.=
Hole No.=B-2
Depth of Hole= 50.00 ft
Water Table during Earthquake= 35.00 ft
Water Table during In-Situ Testing= 55.00 ft
Max. Acceleration= 1 g
Earthquake Magnitude= 7.05

Input Data:
Surface Elev.=
Hole No.=B-2
Depth of Hole=50.00 ft
Water Table during Earthquake= 35.00 ft
Water Table during In-Situ Testing= 55.00 ft
Max. Acceleration=1 g
Earthquake Magnitude=7.05
No-Liquefiable Soils: Based on Analysis

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
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 ft	SPT	gamma pcf	Fines %
0.00	22.00	107.00	30.00
5.00	22.00	107.00	30.00
10.00	36.00	121.00	47.00
15.00	36.00	133.00	48.00
20.00	37.00	122.00	17.00
25.00	35.00	121.00	52.00
30.00	34.00	117.00	29.00
35.00	43.00	121.00	9.00
40.00	100.00	128.00	13.00
45.00	100.00	117.00	14.00

50.00	100.00	133.00	10.00

Output Results:

Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=0.75 in.
Total Settlement of Saturated and Unsaturated Sands=0.75 in.
Differential Settlement=0.377 to 0.497 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.34	0.65	5.00	0.00	0.75	0.75
2.00	2.34	0.64	5.00	0.00	0.75	0.75
4.00	2.34	0.64	5.00	0.00	0.74	0.74
6.00	2.34	0.64	5.00	0.00	0.72	0.72
8.00	2.34	0.64	5.00	0.00	0.70	0.70
10.00	2.34	0.63	5.00	0.00	0.67	0.67
12.00	2.34	0.63	5.00	0.00	0.64	0.64
14.00	2.34	0.63	5.00	0.00	0.62	0.62
16.00	2.34	0.62	5.00	0.00	0.58	0.58
18.00	2.34	0.62	5.00	0.00	0.53	0.53
20.00	2.34	0.62	5.00	0.00	0.44	0.44
22.00	2.34	0.61	5.00	0.00	0.38	0.38
24.00	2.34	0.61	5.00	0.00	0.34	0.34
26.00	2.34	0.61	5.00	0.00	0.30	0.30
28.00	2.35	0.61	5.00	0.00	0.25	0.25
30.00	2.32	0.60	5.00	0.00	0.19	0.19
32.00	2.29	0.59	5.00	0.00	0.12	0.12
34.00	2.27	0.58	5.00	0.00	0.04	0.04
36.00	2.24	0.58	3.88	0.00	0.00	0.00
38.00	2.22	0.58	3.80	0.00	0.00	0.00
40.00	2.19	0.59	3.74	0.00	0.00	0.00
42.00	2.17	0.59	3.68	0.00	0.00	0.00
44.00	2.15	0.59	3.63	0.00	0.00	0.00
46.00	2.13	0.59	3.60	0.00	0.00	0.00
48.00	2.10	0.59	3.57	0.00	0.00	0.00
50.00	2.08	0.59	3.54	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.

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1 atm (atmosphere) = 1 tsf (ton/ft2)

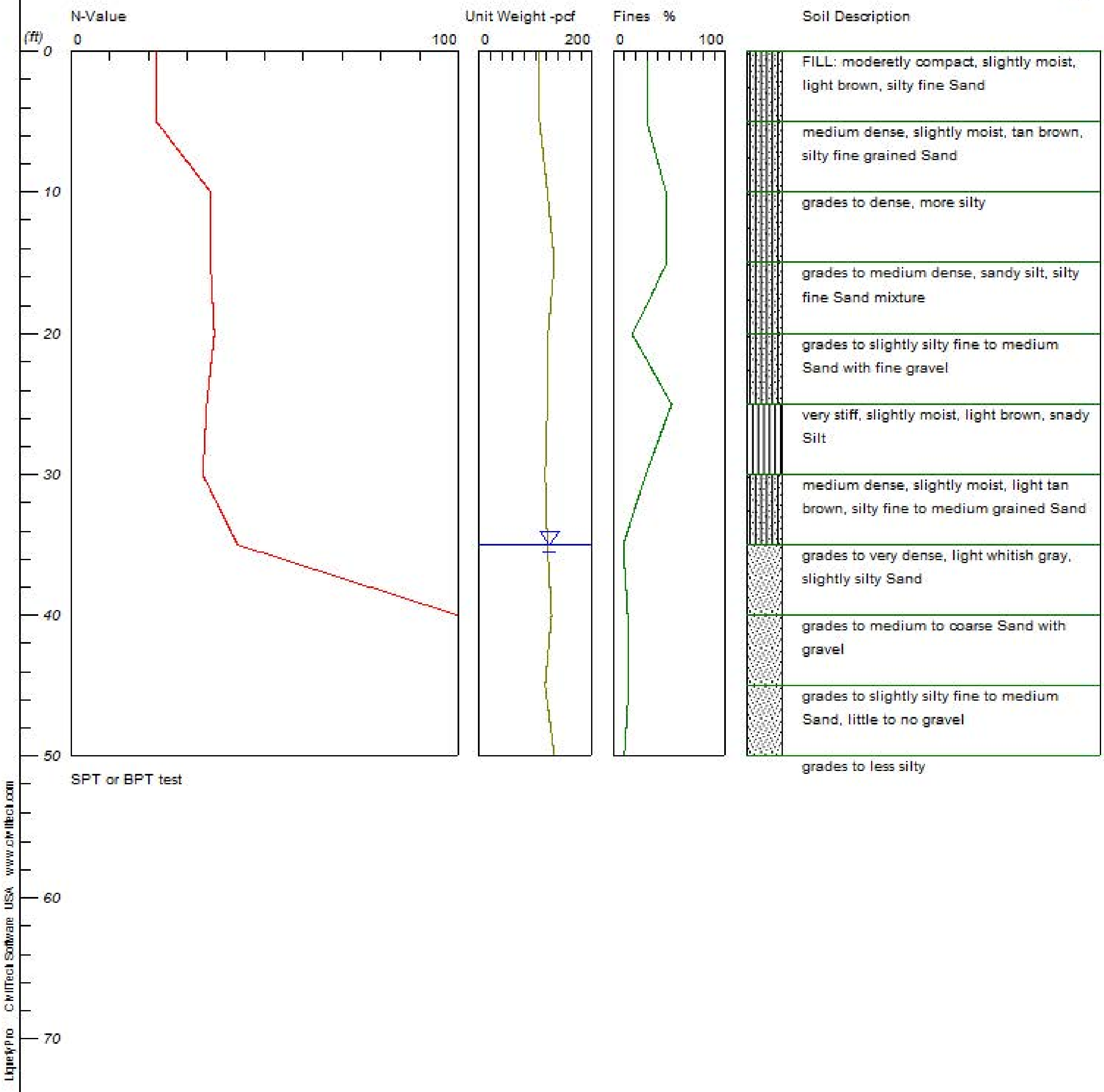
CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated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

LIQUEFACTION ANALYSIS

727 Sonora Ave. Glendale

Hole No.=B-2 Water Depth=35 ft

Magnitude=6.90
Acceleration=0.664g

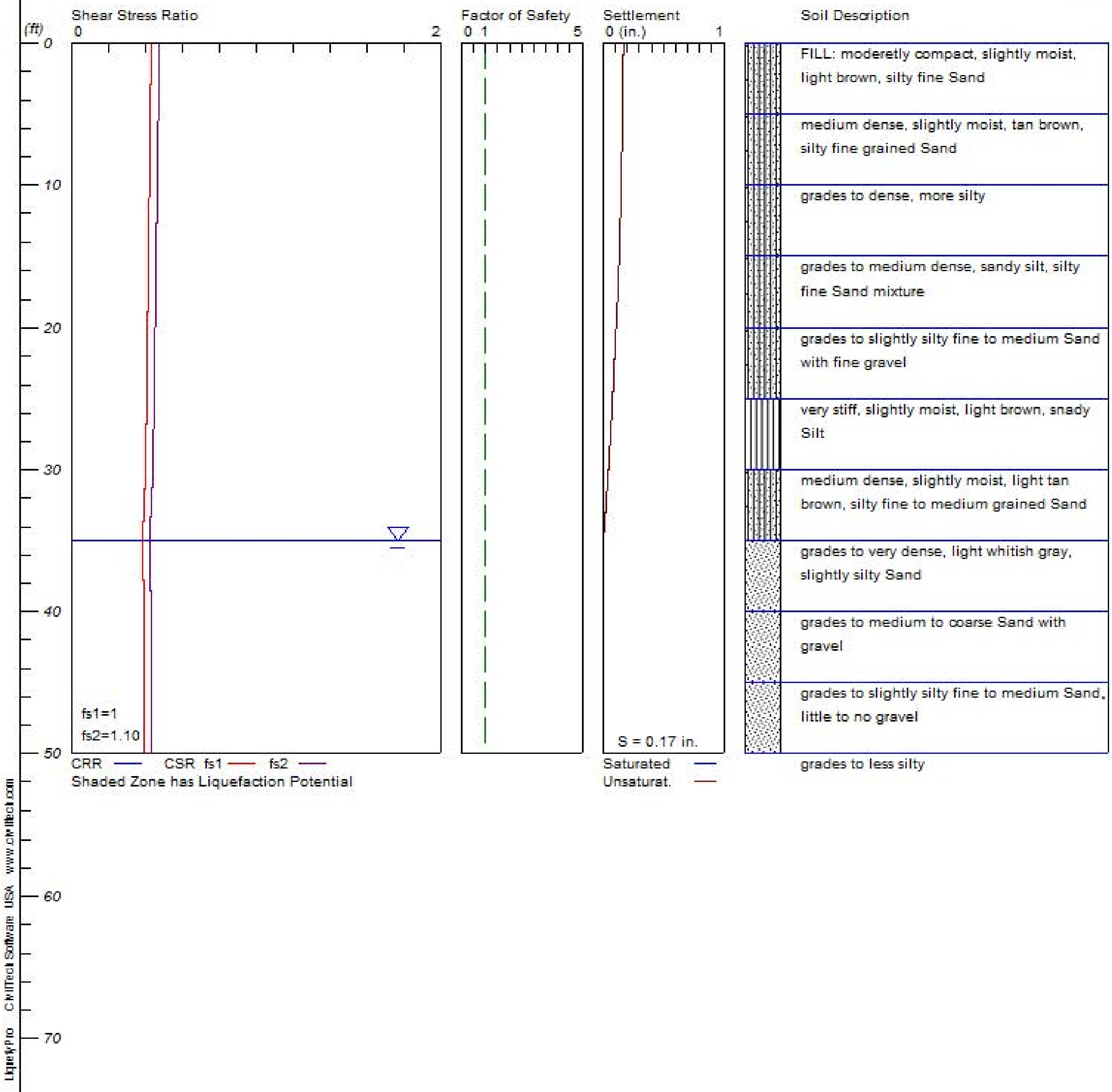


LIQUEFACTION ANALYSIS

727 Sonora Ave. Glendale

Hole No.=B-2 Water Depth=35 ft

Magnitude=6.90
Acceleration=0.664g



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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: P:\Projects-2018\18-320-11, 02 & 24\Engineering-Calculation\Liquefaction\18-320-02_10%.liq
Title: 727 Sonora Ave. Glendale
Subtitle: 18-320-02_10%

Surface Elev.=
Hole No.=B-2
Depth of Hole= 50.00 ft
Water Table during Earthquake= 35.00 ft
Water Table during In-Situ Testing= 55.00 ft
Max. Acceleration= 0.66 g
Earthquake Magnitude= 6.90

Input Data:
Surface Elev.=
Hole No.=B-2
Depth of Hole=50.00 ft
Water Table during Earthquake= 35.00 ft
Water Table during In-Situ Testing= 55.00 ft
Max. Acceleration=0.66 g
Earthquake Magnitude=6.90
No-Liquefiable Soils: Based on Analysis

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
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 ft	SPT	gamma pcf	Fines %
0.00	22.00	107.00	30.00
5.00	22.00	107.00	30.00
10.00	36.00	121.00	47.00
15.00	36.00	133.00	48.00
20.00	37.00	122.00	17.00
25.00	35.00	121.00	52.00
30.00	34.00	117.00	29.00
35.00	43.00	121.00	9.00
40.00	100.00	128.00	13.00
45.00	100.00	117.00	14.00

50.00 100.00 133.00 10.00

Output Results:

Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=0.17 in.
Total Settlement of Saturated and Unsaturated Sands=0.17 in.
Differential Settlement=0.083 to 0.109 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.48	0.43	5.00	0.00	0.17	0.17
2.00	2.48	0.43	5.00	0.00	0.16	0.16
4.00	2.48	0.43	5.00	0.00	0.16	0.16
6.00	2.48	0.43	5.00	0.00	0.16	0.16
8.00	2.48	0.42	5.00	0.00	0.15	0.15
10.00	2.48	0.42	5.00	0.00	0.15	0.15
12.00	2.48	0.42	5.00	0.00	0.14	0.14
14.00	2.48	0.42	5.00	0.00	0.14	0.14
16.00	2.48	0.42	5.00	0.00	0.13	0.13
18.00	2.48	0.41	5.00	0.00	0.12	0.12
20.00	2.48	0.41	5.00	0.00	0.10	0.10
22.00	2.48	0.41	5.00	0.00	0.09	0.09
24.00	2.48	0.41	5.00	0.00	0.08	0.08
26.00	2.48	0.41	5.00	0.00	0.07	0.07
28.00	2.48	0.40	5.00	0.00	0.06	0.06
30.00	2.45	0.40	5.00	0.00	0.04	0.04
32.00	2.42	0.39	5.00	0.00	0.03	0.03
34.00	2.40	0.39	5.00	0.00	0.01	0.01
36.00	2.37	0.39	5.00	0.00	0.00	0.00
38.00	2.34	0.39	5.00	0.00	0.00	0.00
40.00	2.32	0.39	5.00	0.00	0.00	0.00
42.00	2.29	0.39	5.00	0.00	0.00	0.00
44.00	2.27	0.39	5.00	0.00	0.00	0.00
46.00	2.25	0.39	5.00	0.00	0.00	0.00
48.00	2.22	0.39	5.00	0.00	0.00	0.00
50.00	2.20	0.39	5.00	0.00	0.00	0.00

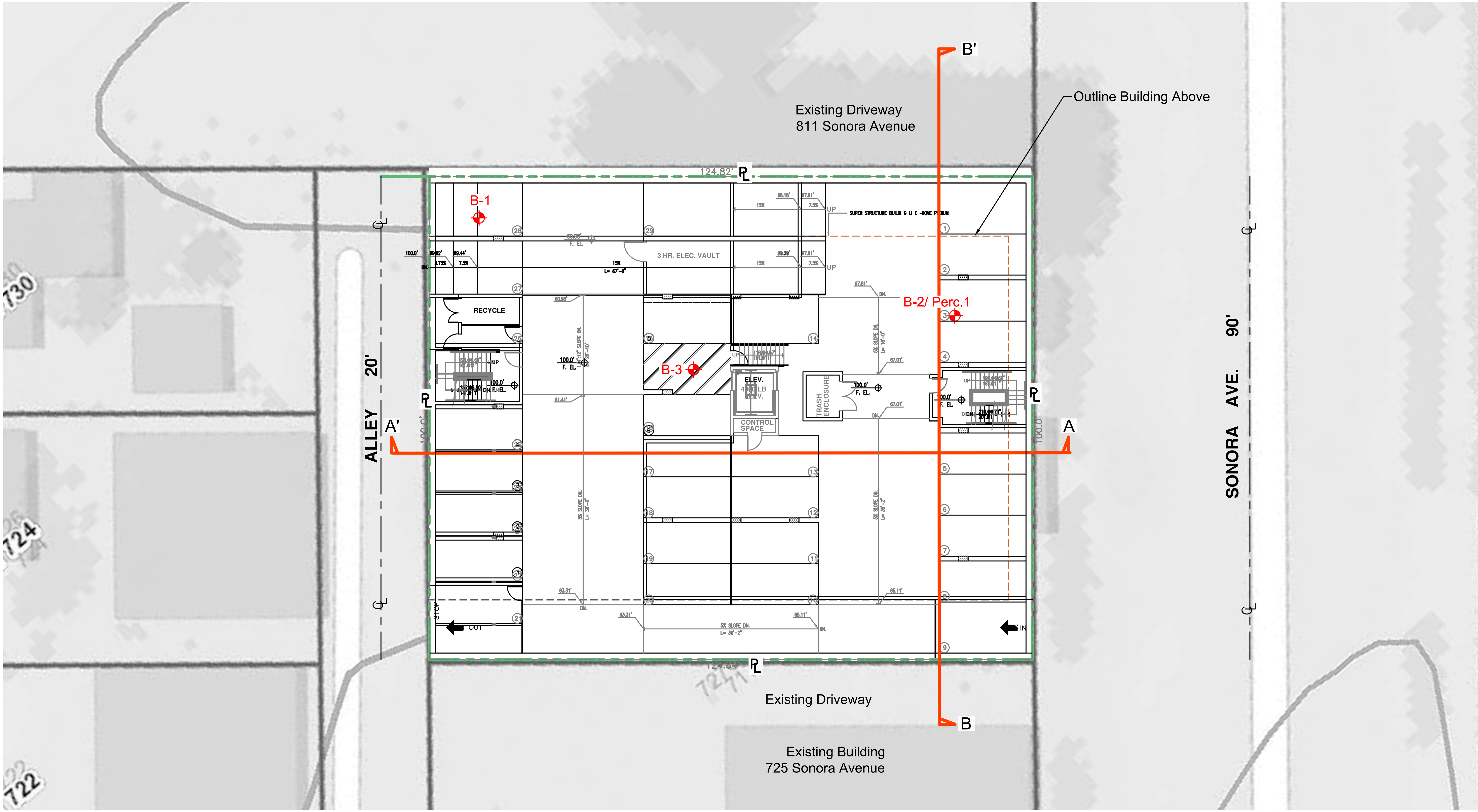
* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

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



LEGEND:

B-2 = Location & Number of Boring

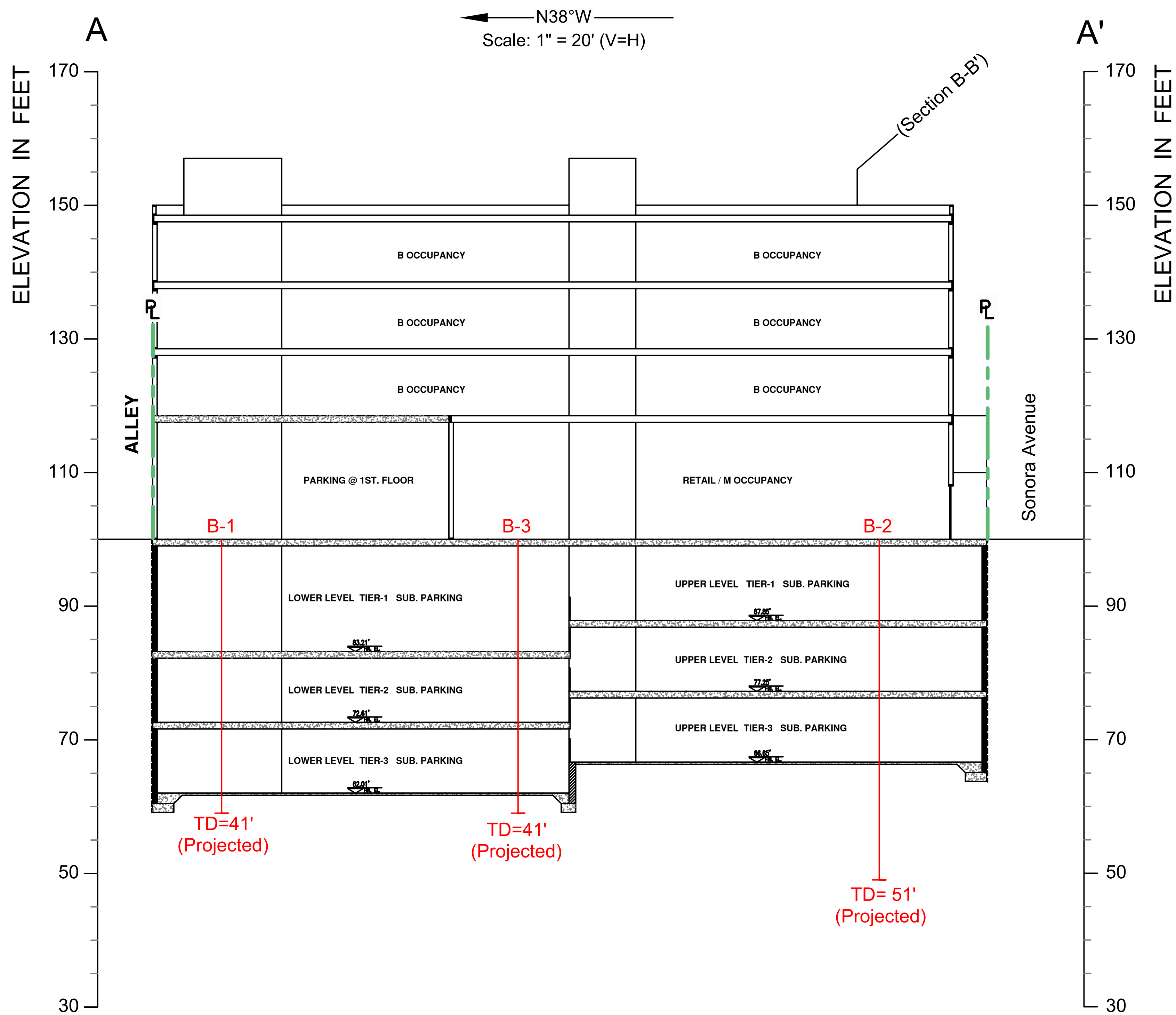
Note:

Site plan prepared by using plan drawn by:
-Landmark Design & Construction, LLC
-Topographic Lines outside subject property are based on LA County GIS Map.

N

Scale: 1" = 20'

SITE MAP		PROJECT No: 18-320-02	
DESCRIPTION:	Proposed Mix-Use Building Project	DATE:	10 / 09 / 2018
FOR:	Sonora Real Estate Group, LLC	DRAWN BY:	TG
ADDRESS:	727 Sonora Avenue, Glendale, CA 91201	CHECKED BY:	CM
Applied Earth Sciences — GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS	www.aessoil.com (818) 552-6000	DRAWING No:	1



LEGEND:

B-2 = Location & Number of Boring

TD=10'
(Projected)

CROSS SECTION A-A'

DESCRIPTION:	Proposed Mix-Use Building Project
FOR:	Sonora Real Estate Group, LLC
ADDRESS:	727 Sonora Avenue, Glendale, CA 91201

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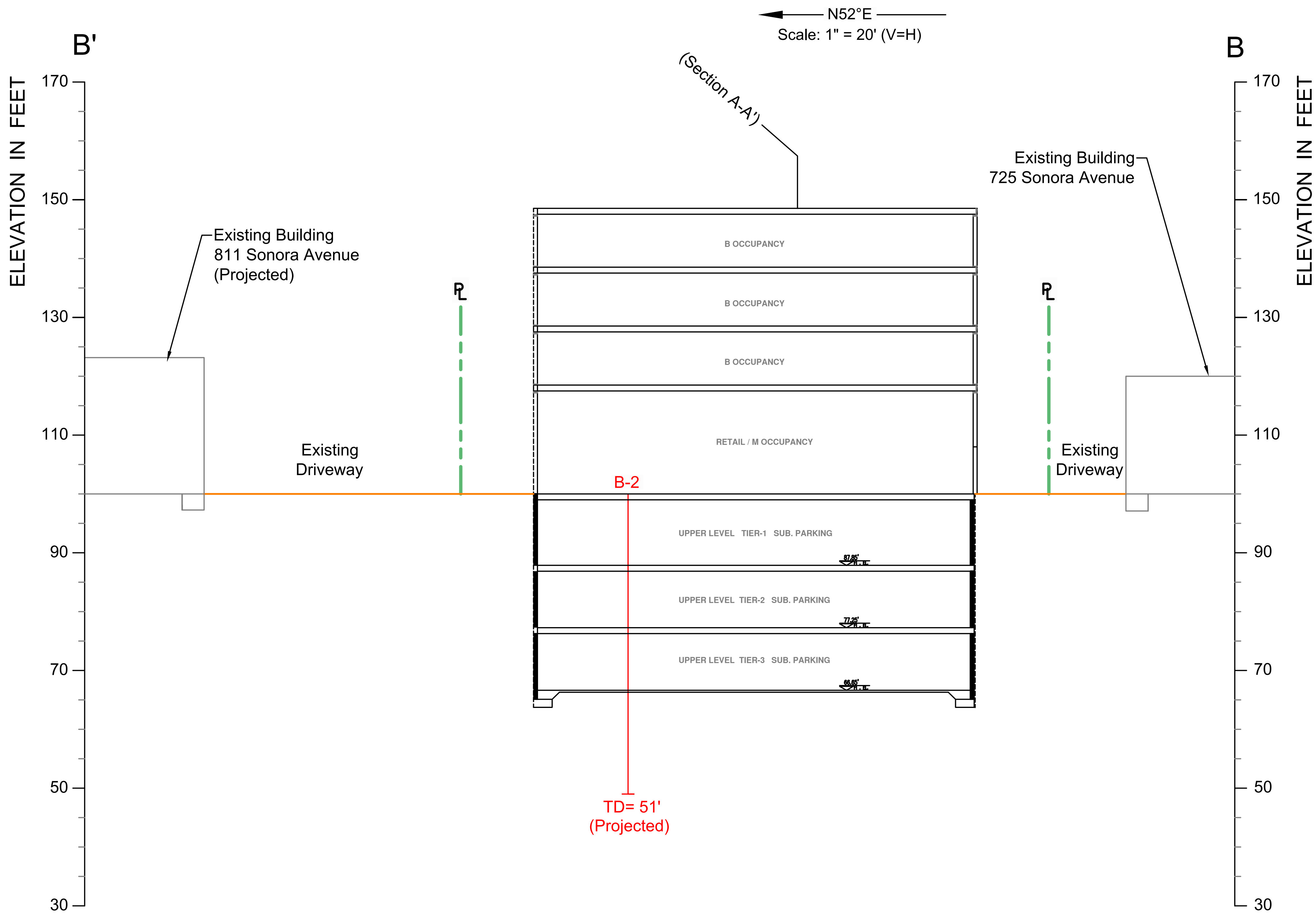
www.aessoil.com
(818) 552-6000

PROJECT No:	18-320-02
DATE:	10 / 10 / 2018
DRAWN BY:	TG
CHECKED BY:	SM
DRAWING No:	2



LEGEND:

B-2 = Location & Number of Boring
TD=10'
(Projected)

CROSS SECTION B-B'

DESCRIPTION:	Proposed Mix-Use Building Project	PROJECT No:	18-320-02
FOR:	Sonora Real Estate Group, LLC	DATE:	10 / 10 / 2018
ADDRESS:	727 Sonora Avenue, Glendale, CA 91201	DRAWN BY:	TG
		CHECKED BY:	SM
Applied Earth Sciences	GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS www.aessoil.com (818) 552-6000	DRAWING No:	3

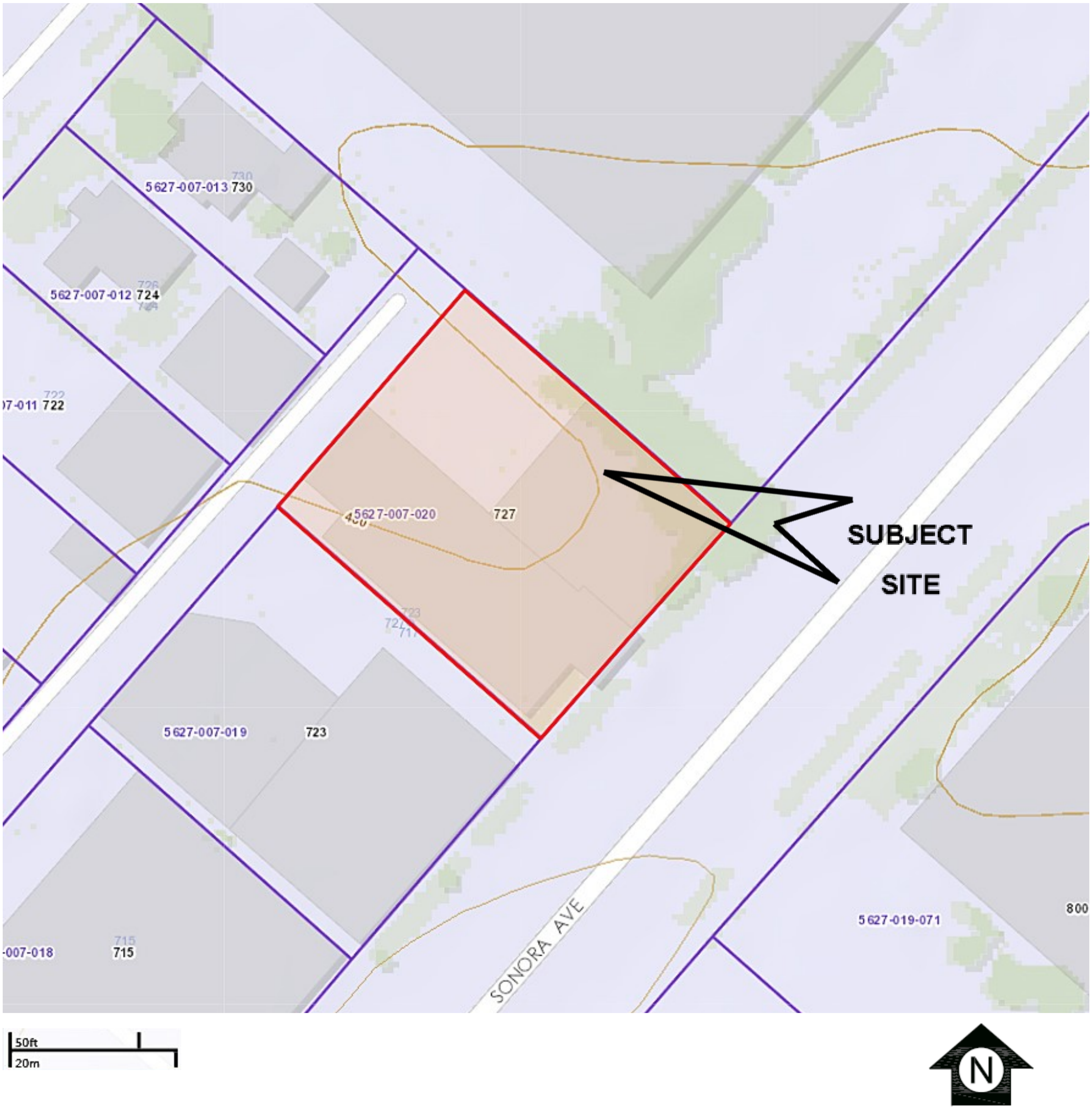
Reference: Portion of Google Maps

SITE VICINITY MAP

Preliminary Mix-Use Building Project

727 Sonora Avenue, Glendale, CA 91201

FOR	DATE	PROJECT No.
Sonora Real Estate Group, LLC	10 / 10 / 2018	18-320-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 1



Reference: Los Angeles County GIS Map

REGIONAL TOPOGRAPHIC MAP

Preliminary Mix-Use Building Project 727 Sonora Avenue, Glendale, CA 91201		
FOR Sonora Real Estate Group, LLC	DATE 10 / 10 / 2018	PROJECT No. 18-320-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 2

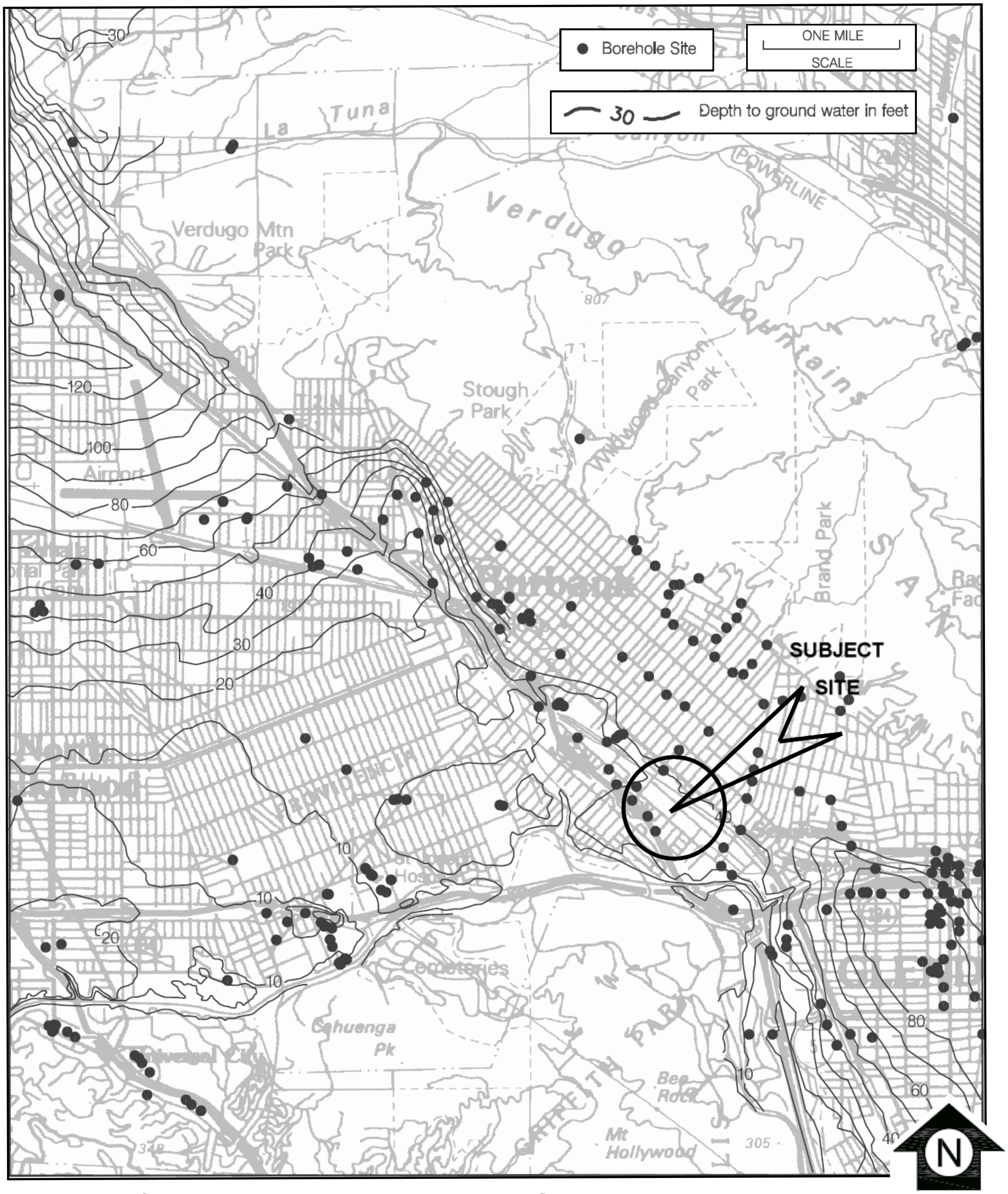
Reference: Dibblee Geologic Map of the Burbank Quadrangle

REGIONAL GEOLOGIC MAP

Preliminary Mix-Use Building Project

727 Sonora Avenue, Glendale, CA 91201

FOR	DATE	PROJECT No.
Sonora Real Estate Group, LLC	10 / 10 / 2018	18-320-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 3



Reference: Burbank 7.5 Minute Quadrangle

HISTORICALLY HIGHEST GROUNDWATER (Contour Map)

Preliminary Mix-Use Building Project

727 Sonora Avenue, Glendale, CA 91201

FOR	DATE	PROJECT No.
Sonora Real Estate Group, LLC	10 / 10 / 2018	18-320-02

APPLIED EARTH SCIENCES
GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS

FIGURE No. 4

APPENDIX I
METHOD OF FIELD EXPLORATION

In order to define subsurface conditions at the subject site, three borings were drilled on the site. The approximate locations of the drilled borings are shown on the enclosed Site Plan. Borings were extended to a maximum depths of 51 feet below grade. 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 August 2, 2018. The material excavated from the borings was placed back and compacted upon completion of the field work. Such material may settle. The owner should periodically inspect these areas and notify this office if the settlement creates a hazard to persons or property.

LOG OF BORING NO.1

18-320-02
727 Sonora Avenue Glendale, CA 91201

Type: Hollow Stem Auger, With 140 Lb Hammer Logged by: Sevada
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, light brown, silty fine sand.					
	(SM) SAND: Medium dense, slightly moist, light brown, silty fine to medium grained sand.					
5	(SM) Grades to brown, slightly silty fine grained sand.		14	3	104	
10	(SM) Grades to dense, more silty.		37	8	119	
15	(SM) Grades to slightly gravelly.		28	7	110	
20	(SP/SM) Grades to more sandy.		35	9	119	
25	(SM) Grades to moist, more silty.		26	12	107	
30	(SM/GM) Grades to very dense, light whitish brown, slightly silty, gravelly fine to medium sand.		65	3	119	
35	(SM) Grades to dense, fine sand with little to no gravel.		38	7	113	

% -200 - △
% Moisture - ●
20 40 60 80

COMPLETION DEPTH: 41' DEPTH TO WATER> INITIAL:
DATE: August 2, 2018 FINAL:

LOG OF BORING NO.1

18-320-02
727 Sonora Avenue Glendale, CA 91201

Type: Hollow Stem Auger, With 140 Lb Hammer   Logged by: Sevada
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
40			(SM/GM) Grades to very dense, gravelly fine to medium sand.		52	2	122	●	
			End of Boring @ 41 Feet NO Groundwater Encountered Hole Backfilled						
45									
50									
55									
60									
65									
70									
75									

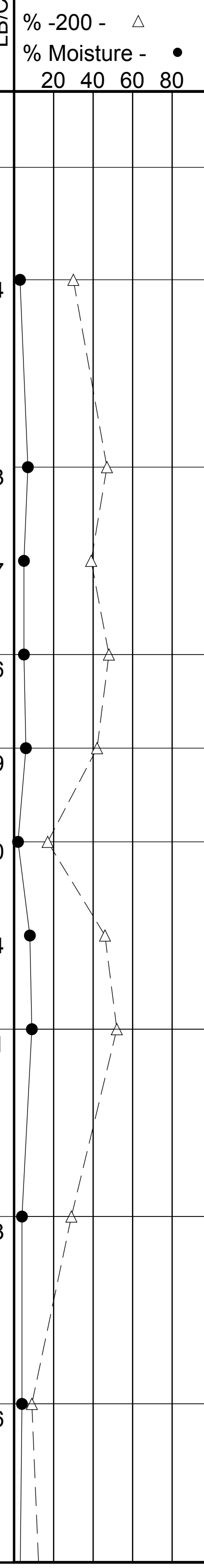
COMPLETION DEPTH: 41   DEPTH TO WATER> INITIAL:
DATE: August 2, 2018   FINAL:

LOG OF BORING NO.2

18-320-02
727 Sonora Avenue Glendale, CA 91201

Type: Hollow Stem Auger, With 140 Lb Hammer   Logged by: Sevada
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, light brown, silty fine sand.					
	(SM) SAND: Medium dense, slightly moist, light brown, silty fine to medium grained sand.					
5	(SM) Grades to tan brown, silty fine grained sand.		33	3	104	30
10	(SM) Grades to dense, more silty.	36		7	113	47
	(SM) Grades to very dense.		64	5	117	39
15	(SM/ML) Grades to dense, sandy silt, silty fine sand mixture.	36		5	126	48
	(SM) Grades to very dense, less silty.		57	6	119	42
20	(SM) Grades to dense, slightly silty fine to medium sand with fine gravel.	37		2	120	17
	(SM) Grades to very dense, more silty.		58	8	114	46
25	(ML) SILT: Very stiff, slightly moist, light brown, sandy silt.	35		9	111	52
30	(SM) SAND: Dense, slightly moist, light tan brown, silty fine to medium sand.	34		4	113	29
35	(SP/SM) Grades to very dense, light whitish gray, little to no silt.	43		4	116	9



COMPLETION DEPTH: 51'   DEPTH TO WATER> INITIAL:
DATE: August 2, 2018   FINAL:

LOG OF BORING NO.2

18-320-02
727 Sonora Avenue Glendale, CA 91201

Type: Hollow Stem Auger, With 140 Lb Hammer   Logged by: Sevada
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
40			(SP/SM) Grades to medium to coarse sand with gravel.	60/6"		3	124		13
45			(SP/SM) Grades to slightly silty fine to medium sand, little to no gravel.	60/6"		3	114		14
50			(SP) Grades to less silty.	61/8"		2	130		10
55			End of Boring @ 51 Feet NO Groundwater Encountered Percolation @ 40'-50'						
60									
65									
70									
75									

COMPLETION DEPTH: 51'
DATE: August 2, 2018

DEPTH TO WATER> INITIAL:
FINAL:

LOG OF BORING NO.3

18-320-02
727 Sonora Avenue Glendale, CA 91201

Type: Hollow Stem Auger, With 140 Lb Hammer Logged by: Sevada
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/GM) FILL: Moderately compact, slightly moist, light gray, silty sand with gravel.					
	(SM/GM) SAND: Dense, slightly moist, light gray, silty fine to medium sand with coarse gravel mixture.					
5	(SM/GM) Similar as above.		39	2	133	
10	(SM) Grades to very dense, silty fine sand with little to no gravel.		43	8	98	
15	(SM) Grades to dense.		24	8	111	
20	(SM) Grades to less silty.		28	7	109	
25	(SM) Grades to slightly more sandy.		35	8	121	
30	(SP/SM) Grades to medium to coarse sand, little to no silt.		37	3	119	
35	(SP/SM) Grades to light grayish brown, fine sand.		40	2	109	

% -200 - △
% Moisture - ●
20 40 60 80

COMPLETION DEPTH: 41'
DATE: August 2, 2018

DEPTH TO WATER> INITIAL:
FINAL:

LOG OF BORING NO.3

18-320-02
727 Sonora Avenue Glendale, CA 91201

Type: Hollow Stem Auger, With 140 Lb Hammer   Logged by: Sevada
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
40			(Sp/SM) Grades to very dense, light whitish gray, fine to medium sand with gravel.		46	3	114	●	
			End of Boring @ 41 Feet NO Groundwater Encountered Hole Backfilled						
45									
50									
55									
60									
65									
70									
75									

COMPLETION DEPTH: 41'   DEPTH TO WATER> INITIAL:
DATE: August 2, 2018   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.

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

PARTICLE SIZE LIMITS

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

NO. 200 NO. 40 NO. 10 NO. 4 $\frac{3}{4}$ in. 3 in. (12 in.)

U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

JOB NAME : Proposed Commercial Mixed Use Building
727 Sonora Avenue
Glendale, CA 91201

JOB No. 18-320-02

FIGURE No. I-4

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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 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

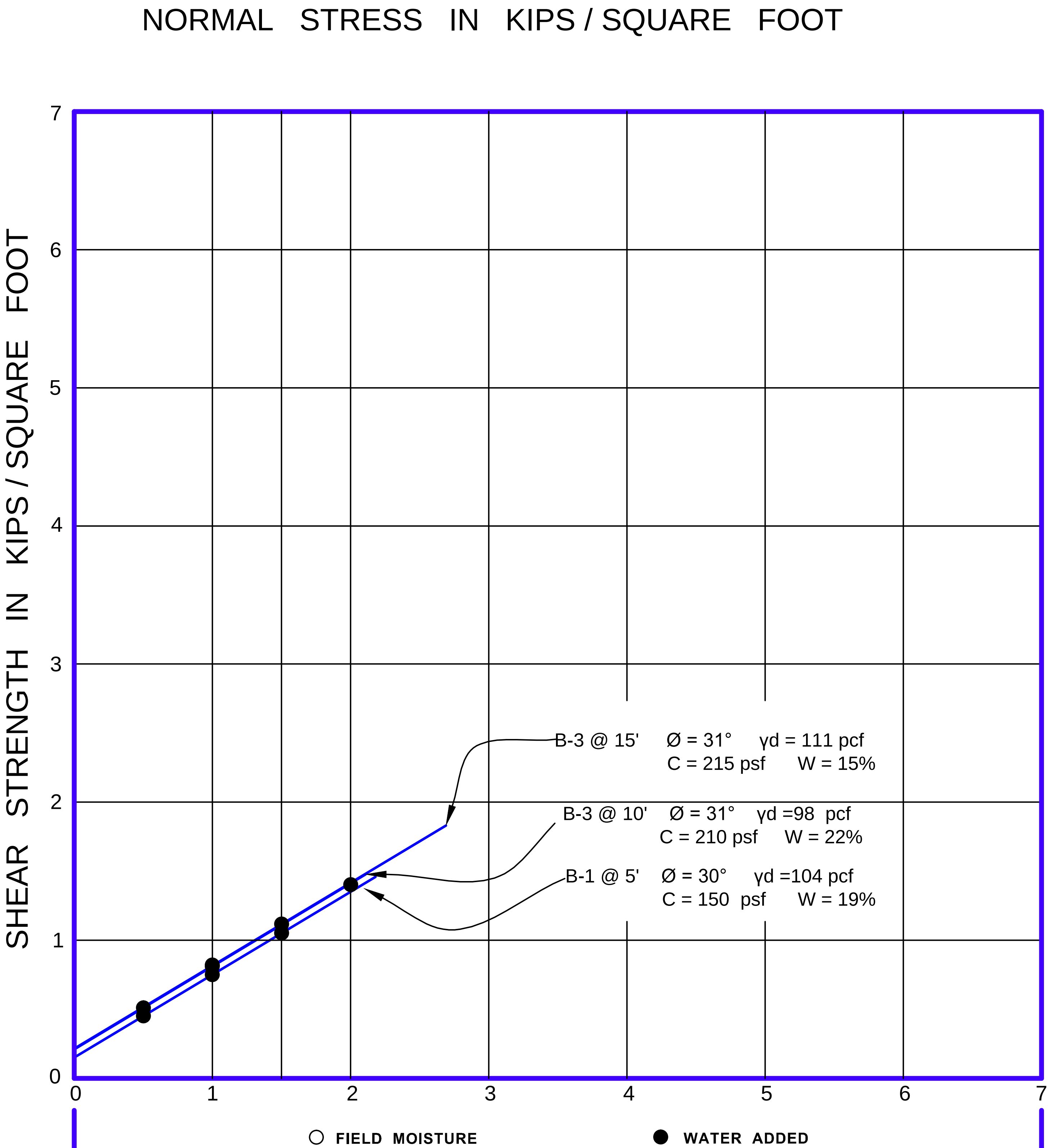
Shear tests were made with a direct shear machine at a constant rate of strain. The machine is designed to test the materials without completely removing the samples from the brass rings. The rate of shear was determined through determination of the rate of consolidation of the foundation bearing materials. 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 Laboratory Direct Shear Test Method. The Ultimate shear strength results of direct shear tests are presented on Figure Nos. II-1 and II-2 within this Appendix.

Consolidation

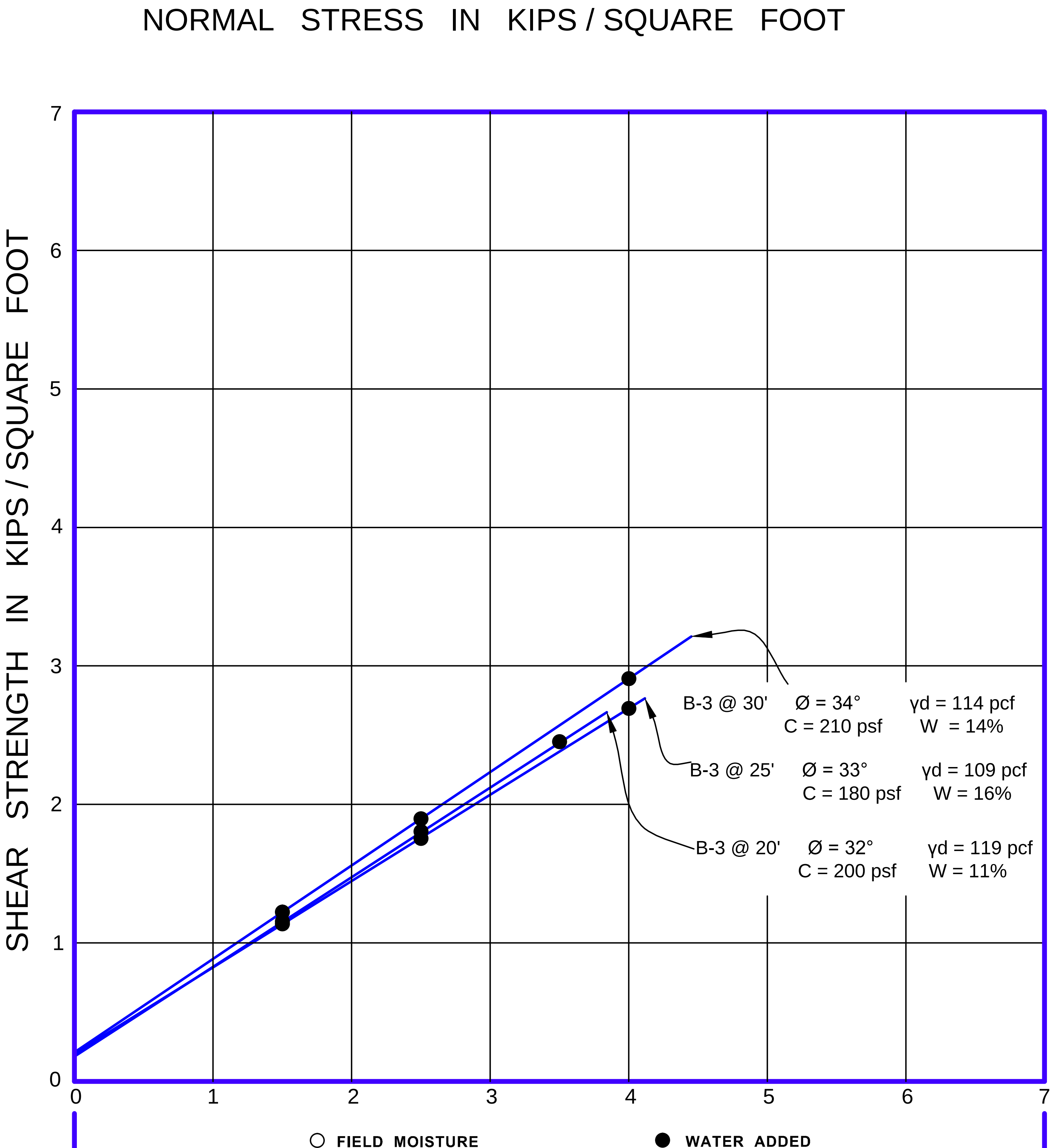
The apparatus used for the consolidation tests is designed to receive the undisturbed brass ring of soil as it comes from the field. Loads were applied to the test specimen in several increments, and the resulting deformations were recorded at time intervals. Porous stones were placed in contact with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

Undisturbed specimens were tested at the field and added water conditions. The test results are shown on Figure No. II-3 within this Appendix.



DIRECT SHEAR TESTS

JOB NAME :	Proposed Commercial Mixed Use Building 727 Sonora Avenue Glendale, CA 91201	JOB No. 18-320-02
Applied Earth Sciences	GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS www.aessoil.com (818) 552-6000	FIGURE No. II - 1



DIRECT SHEAR TESTS

JOB NAME : Proposed Commercial Mixed Use Building
727 Sonora Avenue
Glendale, CA 91201

JOB No. 18-320-02

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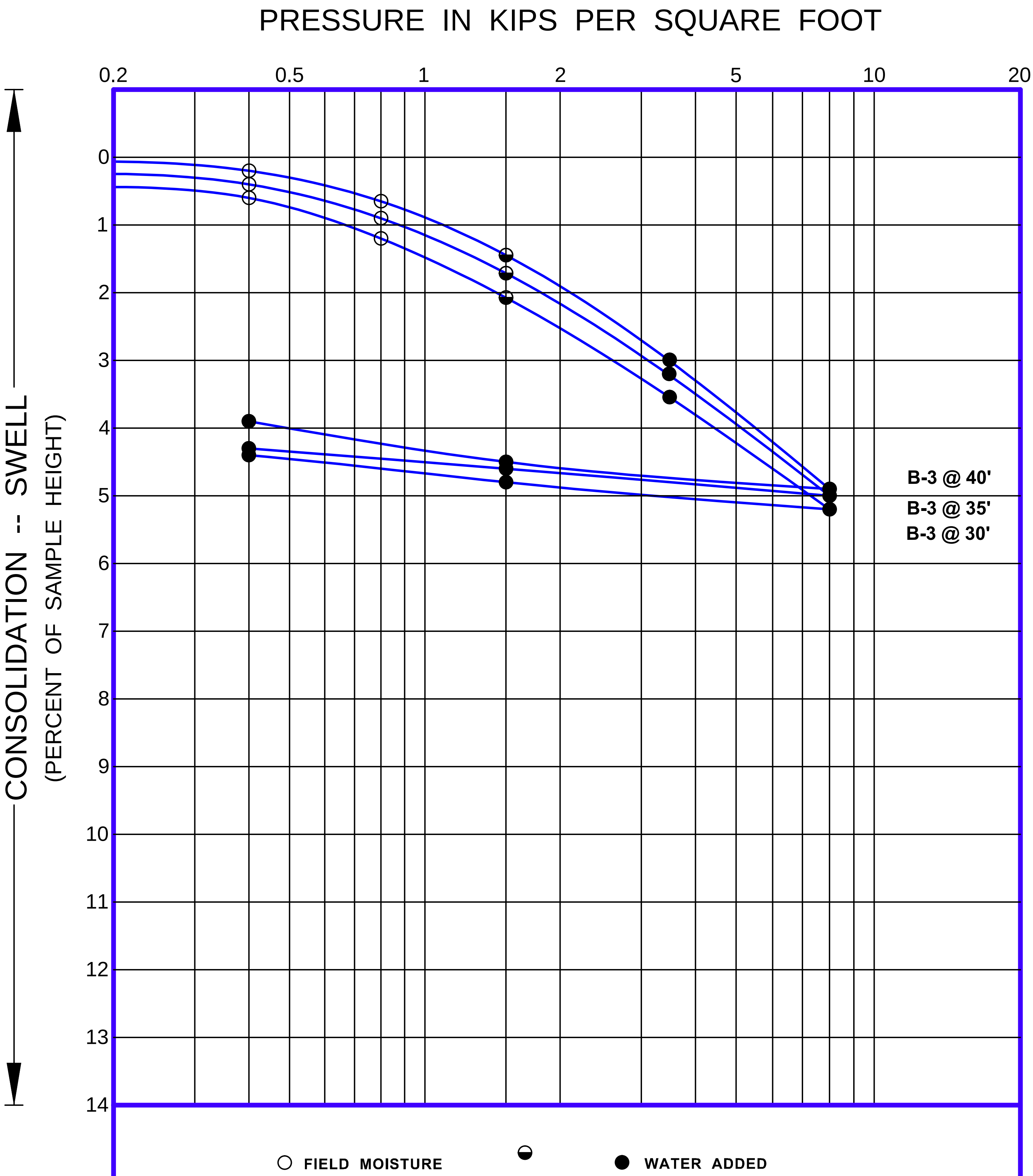
FIGURE No. II - 2



SWELL - CONSOLIDATION TESTS

JOB NAME :	Proposed Commercial Mixed Use Building 727 Sonora Avenue Glendale, CA 91201	JOB No. 18-320-02
Applied Earth Sciences	GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS www.aessoil.com (818) 552-6000	FIGURE No. II - 3

APPENDIX III

CONSTRUCTION PROCEDURE FOR ANCHOR SHAFTS
AND
OBSERVATION AND TESTING REQUIREMENTS DURING THE INSTALLATION OF THE TIEBACK ANCHORS

STANDARD CONSTRUCTION PROCEDURE FOR TEMPORARY SHORING

INTRODUCTION

This section presents a description of the normal construction procedure for installation and testing of concrete anchor shafts against vertical soldier piles. For design of the anchor shafts, refer to the body of the report for the recommended skin friction values.

EXCAVATION PROCEDURE

After the vertical soldier piles are installed, the initial excavation will be extended some 3 feet below the levels of the rows of tiebacks. After the anchor shafts are installed and tested, the excavation will be extended to 3 feet below the next row of tie-back. The procedure will be continued to the lowest basement garage level which is expected to be established at some 35 feet below grade.

TIEBACK CONSTRUCTION

Tieback anchors are normally designed to take loads through skin friction. The portion of the anchor shaft that is considered to be effective in taking pull out loads is the length of the member beyond the potential wedge of the failure. Refer to the body of the report for the recommended inclination of the potential wedge of the failure.

Installation and testing of the tieback anchors should be done under continuous observation and testing of the Soil Engineer. Should significant variations in the soil conditions be encountered during the installation of the anchor shafts, the Soil Engineer will modify the skin friction values to reflect the actual soil conditions.

During the course of our field exploration caving was not detected, due to the method of drilling. However, it should be noted that, if caving is experienced during the excavation of the tieback anchors, it would be necessary to modify the construction procedure (use of casing, etc.).

CONCRETING

After each of the anchors are drilled, foundation grade concrete is placed in the excavated holes using a pump. The concrete is placed only to the level of the potential wedge of failure. After the anchor is tested and approved, the portion of the anchor between the face of the excavation and potential wedge of failure is filled with sand slurry mixture to help maintain the excavation.

SURFACE LOADS

The temporary shoring are designs for lateral earth pressure an any surcharge loads imposed by the existing improvements around the site. In addition, the temporary shoring system should be designed for future loads such as crane and other equipment which operate at close proximity of the top of excavation.

TESTING

The recommended shoring pressures in the report are based on a factor of safety of 1.5. If the anchors are successfully loaded to about 150 percent of the design loads, the overall factor of safety of the shoring system would be on the order of 2. It is customary to test at least one anchor per face of excavation per rows of anchors, for long term loading conditions (24 -hour loading). Load-deflection data for each anchor should be maintained during the testing. Pull out loads are normally applied in increments of 50%, 100% and 150% of the design loads. Once the full 150% design load is applied, the test load is maintained and the deflection of the anchor is recorded. During this stage of testing, the deflection of the anchor during a 15 minute period should not exceed 1/10 of one inch. The total deflection of the anchor should be less than 12 inches, although larger deflections may be accepted provided that both the shoring Engineer and the Soil Engineer approve each such anchors. For long term anchor testing, the 150 percent of the design load is normally applied for a period of 24 hours. If the deflection of the anchor, under 150 percent of the design load, is less than 1/10 of one inch for a period of 4 hours, the test may be considered satisfactory provided that the 150% load has been applied for at least 8 hours.

FAILED ANCHORS

The anchors which do not pass the required pull out test as indicated above are considered to be failed anchors. The modified capacity of the failed anchors would be 2/3 of the available pull out force of the anchors. Additional resistance in a form of supplemental anchors or rakers should then be installed to compensate for the difference between the design and available loads. The failed anchors would then be locked off at 2/3 of the available capacity of the anchor which results a deflection of no more than 1/10 of one inch during a 15 minute period. Since it will be necessary to extend the excavation below the row of anchor in order to install a replacement anchor, it would be advisable to lock off the failed anchor at some value between 2/3 and full available capacity of the anchor. The Soil Engineer and the Shoring Engineer are to provide specific recommendations for the lock off loads for each failed anchor.

LOCK OFF LOADS

After each anchor has been tested and approved by the Soil Engineer, the anchor should be locked off at the design load. The lock off load should be maintained within 90 to 110 percent of the designed load.

CONTINUED EXCAVATION

After each any every anchor in a given face is tested and approved, the excavation can then be extended below the drill bench levels. The Soil Engineer may permit local excavations to be extended below the drill bench elevation where it would be required for construction of replacement anchors.

MONITORING

It is important that an accurate monitoring of the shoring system be maintained during basement construction. Both the horizontal and vertical deflections of the soldier piles should be recorded.

The vertical and horizontal movement of the shoring system should be recorded on a weekly basis and the results be submitted to Soil and Shoring Engineers for review and comment. The accuracy of the reading should be within 0.01 of a foot. The record should be produced in a readily understandable form. The surveyor should submit to the Soil Engineer, prior to the start of excavation, a plan which would indicate the method selected for monitoring of the excavation.

Monitoring of the excavation performance should be initiated from the beginning of the initial excavation. The weekly monitoring may be modified as the job progresses. Once the subterranean garage has been constructed and the tieback have been de-tensioned, monitoring of the performance will no longer be required.

DEFLECTIONS

The maximum depth of excavation is expected to be an the order of 20 feet. Considering the factor of safety of the overall shoring system, it is anticipated that horizontal deflections at the top the soldier piles may reach about one inch. Where off-site buildings are present, the deflection at the top of the piles should be limited to ¼ of one inch.

It is possible that, locally, deflections at the top of the soldier piles may exceed the anticipated values. Should this occur, the Soil and Shoring Engineers should be consulted to provide remedial measures such as installation of additional support system.