REPORT OF

GEOTECHNICAL INVESTIGATION AND PERCOLATION TESTING FOR SUSMP

PROPOSED MULTI-UNIT RESIDENTIAL BUILDING PROJECT

LOT 46, ARB. 2 OF TRACT NO. 482

8554 GLENOAKS BOULEVARD

LOS ANGELES, CALIFORNIA

FOR

GATA BURLINGTON LLC

PROJECT NO. 16-450-02 & 24

JULY 26, 2016

Applied
Earth
Sciences
a division of Applied Soil Technology, Inc.
GEOTECHNICAL & ENVIRONMENTAL ENGINEERING CONSULTANTS

July 26, 2016 16-450-02 & 24

GATA Burlington LLC
530 North kenwood Street,
Suite 1
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Subject: Geotechnical Investigation and Percolation Testing for SUSMP
Proposed Multi-Unit Residential Building Project
Lot 46, Arb.2 of Tract No. 482
8554 Glenoaks Boulevard
Los Angeles, California 91352

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 foundations, grade slabs, and grading. The investigation included subsurface exploration, soil sampling, laboratory testing, percolation testing, engineering evaluation and analysis, consultation and preparation of this report.

During the course of this investigation, the project plans prepared by the offices of Domus Design, were used as reference. The plans were e-mailed to this office on July 22, 2016.

The enclosed Site Plan; Drawing No. 1, shows the approximate locations of the exploratory 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 location of the site with respect to the Regional Topographic Map. Figure No. 3 shows the Regional Geologic Map. The Historically Highest Groundwater Contour Map is shown in Figure No. 4.

The attached Appendix I, describes the method of field exploration. Figure Nos. I-1 through I-4 present summaries of the materials encountered at the location of our borings. Figure No. I-5 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 ad II-2 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.

Appendix IV describes the On Site Percolation Testing, which highlights the on site storm water filtration considerations and discusses recommendations and results.

It should be noted that the presented design recommendations for temporary excavation and foundation are based on our understanding of the depth of excavation, structural setback conditions, 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 multi-unit residential building. The proposed building is expected to be a three-story wood frame structure constructed over 1 to 2 levels of parking garage to depths of 13 to 20 feet. It is anticipated that total excavation depth to the footing levels of the basement will range from about 15 to 22 feet.

Due to the magnitude of the excavation heights, temporary shoring will be required during the course of basement garage construction. The shoring system should be in a form of cantilevered soldier piles where total height of excavation is less than 15 feet. In the areas where the total height of excavation exceed 15 feet, the soldier piles should be laterally supported by internal bracing or anchor tie-back.

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

Structural loading data was not available during this study. For the purpose of this report, it is assumed that maximum loads of the interior columns will be about 450 kips, combined dead plus frequently applied live loads. Perimeter wall footings of the basement are expected to have loads of as much as 18 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;
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. The on site soils can be used as wall backfill material. It is anticipated that, after completion of the site grading work, materials will be exported from the site.

REGIONAL GEOLOGY

The subject site is situated in the northeast San Fernando Valley, at the southern foothills of the Verdugo Mountains, part of the Transverse Ranges Geomorphic Province. The local surface geology consists of coarse-grained quaternary alluvial deposits consisting of sand and gravel, derived from the Verdugo Mountains immediately to the north. Granitic bedrock is thought to underlie the site at depths of between 100 to 200 feet below grade.

As can be seen in Figure No. 3, the site is located within the Verdugo Fault zone. This fault zone is not currently considered active by the state of California. As such the property is not in an Alquist Priolo fault study zone and a fault study was not a part of this scope of work. Nevertheless, owners of properties in fault zones are encouraged to protect their investments from damage related to earthquakes, for example, by purchasing earthquake insurance.

SITE CONDITIONS

SURFACE CONDITIONS

The subject site is located at 8554 Glenoaks Boulevard within the City Of Los Angeles. The Lot consist of a trapezoidal shaped lot with a surface area of about 30,544 square feet. See the enclosed Site Plan; Drawing No. 1, for site location.

At the time of our field investigation, the site was vacant. Hansen Heights Channel is located to the northeast of the project site. Existing off-site building and empty lot occur around the site. See the enclosed Site Plan.

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 Quaternary Alluvium consisting of natural deposits of silty sand and relatively clean sand soils with variable amounts of gravel. Thickness of the existing fill was found to be on the order of two feet at the location of our borings. Deeper fill, however, may be present beneath the existing structures, over the underground tanks, in old utility lines and abandoned private sewage disposal system. 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 silty sand and clean sand. Such soils were found to be generally dense in-place. The results of our laboratory investigations indicated that these materials were of moderate to high strengths.

The soils near the planned foundation levels were found to be consist of generally dense sand. With gravel. The results of our laboratory testing indicated that these materials were of high strengths and low compression.

The soils near the lowest basement garage level were found to be granular in nature. These soils are considered to be virtually non-expansive.

During the course of our investigation, no groundwater was encountered in our borings drilled to a maximum depth of 41 feet. Due to method of drilling (use of continuous casing) caving was not detected. The sand soils, however, are considered to be susceptible to caving within large scale excavation and in drilled holes. 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.

EVALUATION OF LIQUEFACTION POTENTIAL

During the course of our investigation, no groundwater was found in our borings drilled to a maximum depth of 41 feet. The available maps indicate that the historically highest groundwater level at the site occurs more than 80 feet below the existing ground surface. On the basis of the above, it is our opinion that soil liquefaction will not occur at the subject site.

SEISMIC DESIGN CONSIDERATIONS

In accordance with the 2013 California Building code(CBC 2013), the project site can be classified as site D. The mapped spectral accelerations of S_S=2.423 (short period) and S_1 =0.860 (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 (2.423) = 2.423
Sm1=Fv (S1) = 1.5 (0.860) = 1.291
Sds=2/3 (Sms) = 2/3 (2.423) = 1.615 and
Sd1=2/3 (Sm1) = 2/3 (1.291) = 0.860

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 dense, silty sand or clean sand native soils.

It is anticipated that the basement garage excavations will be made through surficial fill and native soils consisting of silty sand and clean sand soils (with variable amounts of gravel). The maximum height of excavation to the perimeter wall footing levels of the basement garage are expected to be on the order of 15 to 22 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. During the course of basement garage construction, temporary shoring should be used. This will consist of soldier piles with and without lateral support. Where total height of excavation is up to 15 feet, cantilevered support system can be used. 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.

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

Caving may occur in typical clean granular soils, during pile drilling. Therefore, during the course of drilling of the piles casing should be available at the site. Also, alternate piles should be drilled so that possible caving in a given hole will not effect the adjacent hole. It is noted that water may also be encountered in the drilled holes close to the existing channel, if pile drilling is made following a storm. Therefore, when placing concrete in water, “treme” should be used. Also, the strength of the concrete should be maintained at least 1,000 psi above those indicated in the project plans and specifications.

The garage floor could be supported on the exposed subgrade, provided that any disturbed soils would be compacted 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 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	vertical
>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 shoring piles can be used as a means of temporary shoring. The soldier piles can then be incorporated into the exterior walls of the garage and be part of the permanent structure.

For temporary construction excavations, the active pressure on cantilever soldier piles may be based on an equivalent fluid density of 30 pounds per cubic foot. See the enclosed supporting engineering calculations. Uniform surcharge may be computed using an active pressure coefficient of 0.35 times the uniform load.

The lateral resistance for piles may be assumed to be offered by available passive pressure developed below the base of the excavation. An allowable maximum passive pressure of 600 pounds per square foot per foot of depth may be used for piles having center-to-center spacing of at least 2-1/2 times the pile diameter. Maximum allowable passive pressure should be limited to 6,600 pounds per square foot. The maximum center-to-center spacing of the vertical shafts should be maintained no greater than 12 feet.

In order to limit local sloughing, it is recommended that lagging be used where soils are exposed between the soldier piles. All wood members left in ground should be pressure treated. Caving may be experienced in the shoring piles below the water level. Therefore, driller's mud should be used to maintain stability of the sides walls of the shoring piles. The driller's mud will then be displaced during placement of concrete. When placing concrete below water, the strength of concrete should be taken at least 1,000 psi above the project specifications.

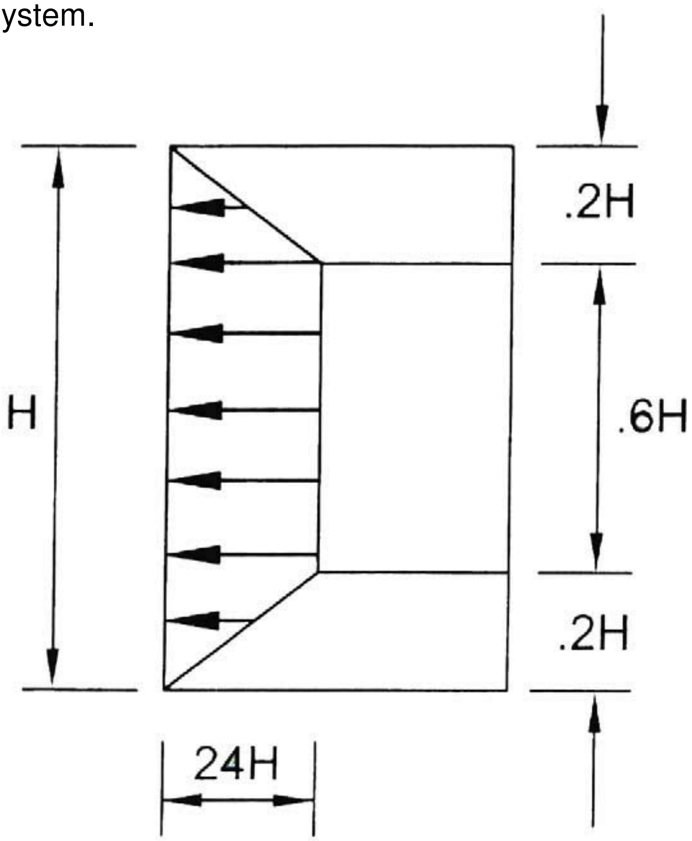
Braced Shoring: Where total height of excavation exceed 15 feet, the vertical shafts should be laterally supported by internal bracing or anchor tie-back. Presence of water and caving should be considered during installation of the anchor tie-backs. See the recommendations in the preceding sections of this report.

It is anticipated that one row of anchor shafts will be required for the proposed project. It should be noted that, if tie backs are used, permissions should be obtained to extend the anchor shafts beneath the adjacent properties.

In addition to the above, 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 0.35 times the assumed uniform loads.

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 600 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.

SHORING PILE DEFLECTION LIMITS

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 excavation, the allowable lateral deflection at the tops of the piles can be increased to one inch.

The temporary shoring should be monitored during the course of basement garage excavation. The report of monitoring should be provided to the Project and Soil

Engineers for review and comment. If excessive lateral movements are noted, additional lateral support system in a form of internal bracing may be required.

FOUNDATIONS

Conventional spread footing foundation systems could be used to support the proposed building. The foundation bearing materials are expected to be dense, silty sand and clean sand native soils with gravel.

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 4,200 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 5,500 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 400 to 600 kips are expected to settle on the order of 7/8 of one inch. Continuous footings, with loads of about 12 to 18 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. 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 4,500 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.

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 6-mil polyethylene film covered with 2 inches of clean sand.

BASEMENT WALLS

Cantilevered walls should be designed for an equivalent fluid pressure of 35 pounds per square foot per foot of depth. The perimeter walls of the basement garage of the proposed building are expected to be buried to a maximum depths of about 13 to 20 feet. Static design of these walls (being restrained against rotation) could be based on an equivalent fluid pressure of 53 pounds per square foot per foot of depth. This assumes that no hydrostatic pressure will occur behind the retaining walls. This will require that proper subdrain be installed behind the basement garage walls. For typical project, subdrain would normally consists of mira-drains nailed to the lagging and extending down to the base of the wall, into gravel pockets which will have solid pipe connections to the base of the slab carrying the collected water into a sump.

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 basement walls greater 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 ½ of the above give static pressure (27 pounds per square foot per decreasing depth) with maximum pressure occurring at top and decreasing to zero at the bottom of the wall in a form of a reverse triangle. 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.

Where adequate space is available, granular fill should be placed and compacted behind the retaining 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.

SITE GRADING

Site grading for the proposed project is expected to include excavation in order to create the basement garage grades and backfilling behind the basement walls. 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 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

Adequate site drainage should be provided to divert roof and surface waters away from the building and from the property through non-erodible drainage devices. In no case should the surface waters be allowed to pond adjacent to the building or in parking lots. A minimum surface 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. Installing roof and area drains and catch basins with proper 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

OBSERVATION DURING CONSTRUCTION

The presented recommendations in this report assume that all foundations will be established in dense to 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-5
  Appendix II-Methods of Laboratory Testing
    Figure Nos. II-1 and II-2
  Appendix III-Construction Procedure For Anchor Tieback
  Appendix IV- On site Percolation testing

Respectfully Submitted,

APPLIED EARTH SCIENCES

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DD/SM/la

Distribution: (5) Addressee

Average Unit Weight = γs = 128 pcf

Average Value of Fiction Angle = ϕ = 36^{o}

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

$$K_o = 1 - \sin 36^{o}$$

$$K_o = 1 - 0.588$$

$$K_o = 0.41$$

$$\gamma_o = K_o * \gamma$$

$$\gamma_o = 0.41 * 128$$

$$\gamma_o = 52.8$$

At-Rest Equivalent Fluid Density, $\boxed{\gamma_o = 53 \text{ PCF}}$

AT-REST LATERAL EARTH PRESSURE

FOR: GATA BURLINGTON LLC

DATE: 7/29/16

PROJECT NO.: 16-450-02

APPLIED EARTH SCIENCES

GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS

CALC SHEET No. 1

Cross Bedding Parameters

* FIGURE 2 of Naval Facilities Engineering Command

ϒ= 128 PCF
C= 255 PSF
ϕ= 36 º
H= 20 Ft.
PGA_M= 0.863

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

*7.2-78

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

K_h= 2/3 * 0.863 / 2
K_h= 0.29

P_{AE} = 3/8 * 128 * 400 * 0.29
P_{AE} = 5523 lb.

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

EFP= 2 * 5523 / 400

EFP= **27.62 PCF**

SEISMIC LATERAL EARTH PRESSURE

FOR: GATA BURLINGTON LLC
DATE: 7/29/16
PROJECT NO.: 16-450-02

APPLIED EARTH SCIENCES
GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS

CALC SHEET NO. 2

					Driving Force	*Resisting Force*	
SECTION	**W (kips)**	**q (ksf)**	**L (feet)**	**α (degrees)**	**Wsinαcosα (k)**	**Wcos^{2}αtanϕ (k)**	**CLcosα (k)**
I	13.06	0	22.4	63	5.3	2.0	2.6

Soil Parameters

Total Unit Weight γ = 128 pcf
C = 255 psf
ϕ = 36 º

Height of Wall

H= 20 ft

Length of Surcharge Load on Wedge

L_{SL}= 0 ft

∑ Resisting Forces= 4.5 K
∑ Driving Force= 5.3 K

F.S. = ∑ RF / ∑ DF = 4.55 / 5.28 = 0.86

FACTOR OF SAFETY = 1.25

1.25 (DF) = (RF) + UBF

1.25 * 5.28 = 4.55 + UBF

UBF = 6.60 - 4.55 = 2.1 k/lft.

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

G_h = 10.3 pcf

Therefore use Recommended value of 30 pcf

LATERAL EARTH PRESSURE CALCULATIONS

CANTILEVERED SYSTEM (TEMPORARY CONDITION)

South Facing Wall - Section A-A'

FOR: GATA BURLINGTON LLC
DATE: 7/29/16
PROJECT NO.: 16-450-02

APPLIED EARTH SCIENCES
GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS

TABLE No. 1

					Driving Force	*Resisting Force*	
SECTION	**W (kips)**	**q (ksf)**	**L (feet)**	**α (degrees)**	**Wsinαcosα (k)**	**W$\cos^{2}$αtanϕ (k)**	**CLcosα (k)**
I	13.06	0	22.4	63	5.3	2.0	2.6

Soil Parameters

Total Unit Weight γ = 128 pcf

C = 255 psf

ϕ = 36 º

Height of Wall

H= 20 ft

Length of Surcharge Load on Wedge

L_{SL}= 0 ft

∑ Resisting Forces= 4.5 K

∑ Driving Force= 5.3 K

F.S. = ∑ RF / ∑ DF = 4.55 / 5.28 = 0.86

FACTOR OF SAFETY = 1.5

1.5 (DF) = (RF) + UBF

1.5 * 5.28 = 4.55 + UBF

UBF = 7.92 - 4.55 = 3.4 k/lft.

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

G_h = 16.9 pcf

Therefore use Recommended value of 35 pcf

LATERAL EARTH PRESSURE CALCULATIONS

CANTILEVERED SYSTEM (PERMANENT CONDITION)

South Facing Wall - Section A-A'

FOR: GATA BURLINGTON LLC

DATE: 7/29/16

PROJECT NO.: 16-450-02

APPLIED EARTH SCIENCES
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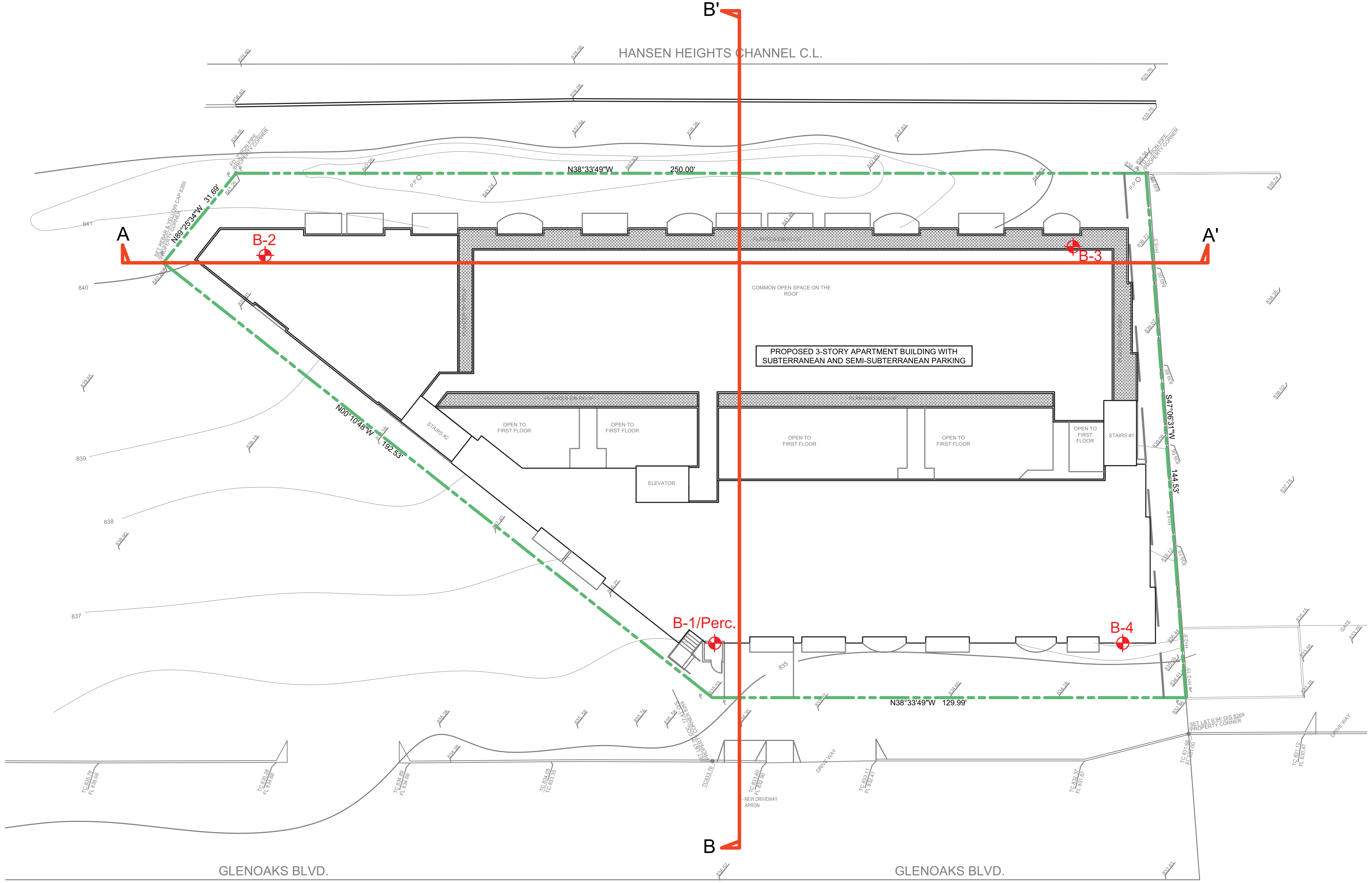
TABLE No. 2



B-4 ⊕ = Location & Number of Boring

N

Scale: 1" = 30'

SITE PLAN

DESCRIPTION:	Proposed Multi-Unit Residential Building Project
FOR:	GATA Burlington LLC
ADDRESS:	8554 Glenoaks Boulevard, Los Angeles, CA 91352

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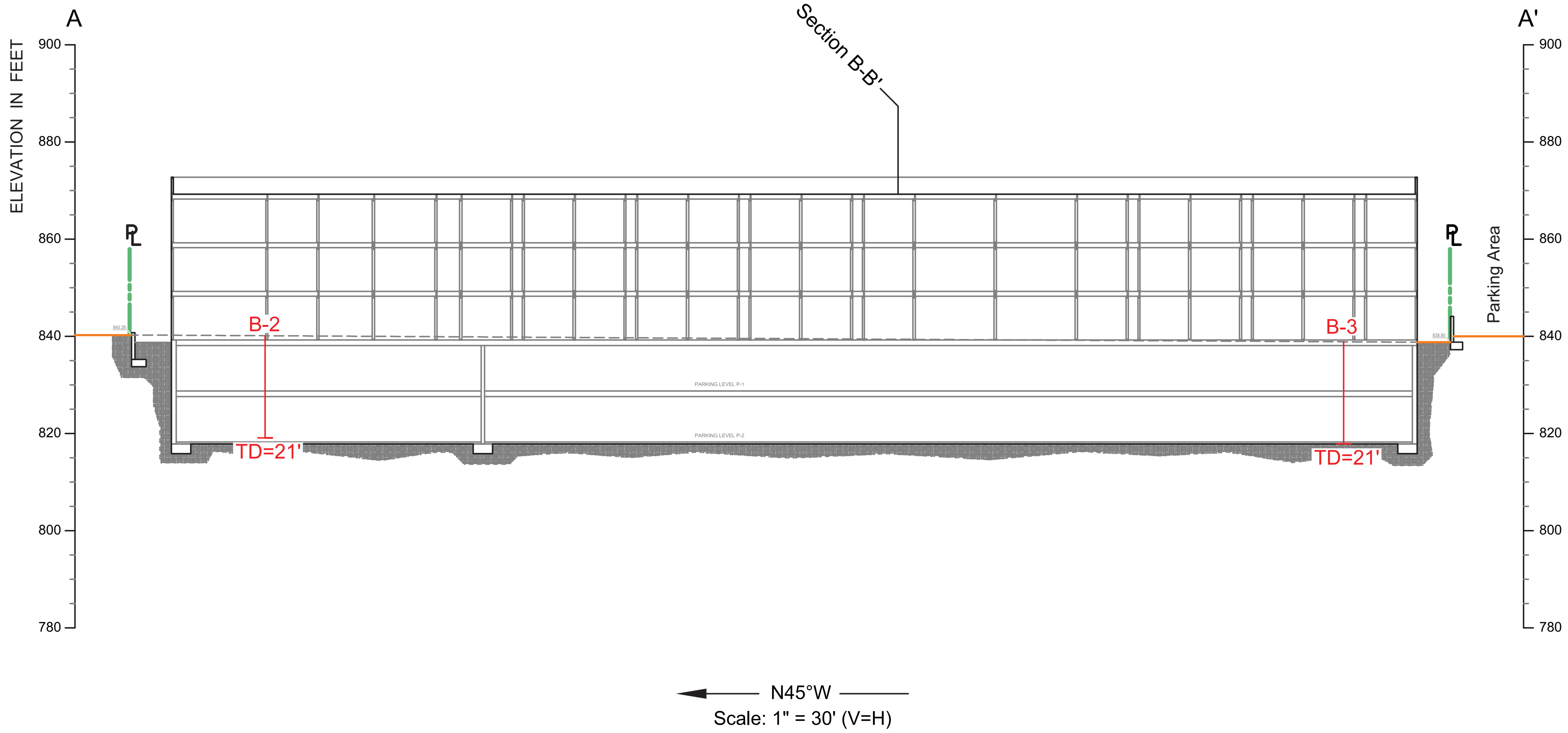
PROJECT No:	16-450-02
DATE:	07 / 26 / 2016
DRAWN BY:	VM
CHECKED BY:	CM
DRAWING No:	1



B-4

= Location & Number of Boring

TD=10'
(Projected)

CROSS SECTION A-A'

DESCRIPTION:	Proposed Multi-Unit Residential Building Project
FOR:	GATA Burlington LLC
ADDRESS:	8554 Glenoaks Boulevard, Los Angeles, CA 91352

PROJECT No:	16-450-02
DATE:	07 / 26 / 2016
DRAWN BY:	VM
CHECKED BY:	CM
DRAWING No:	2

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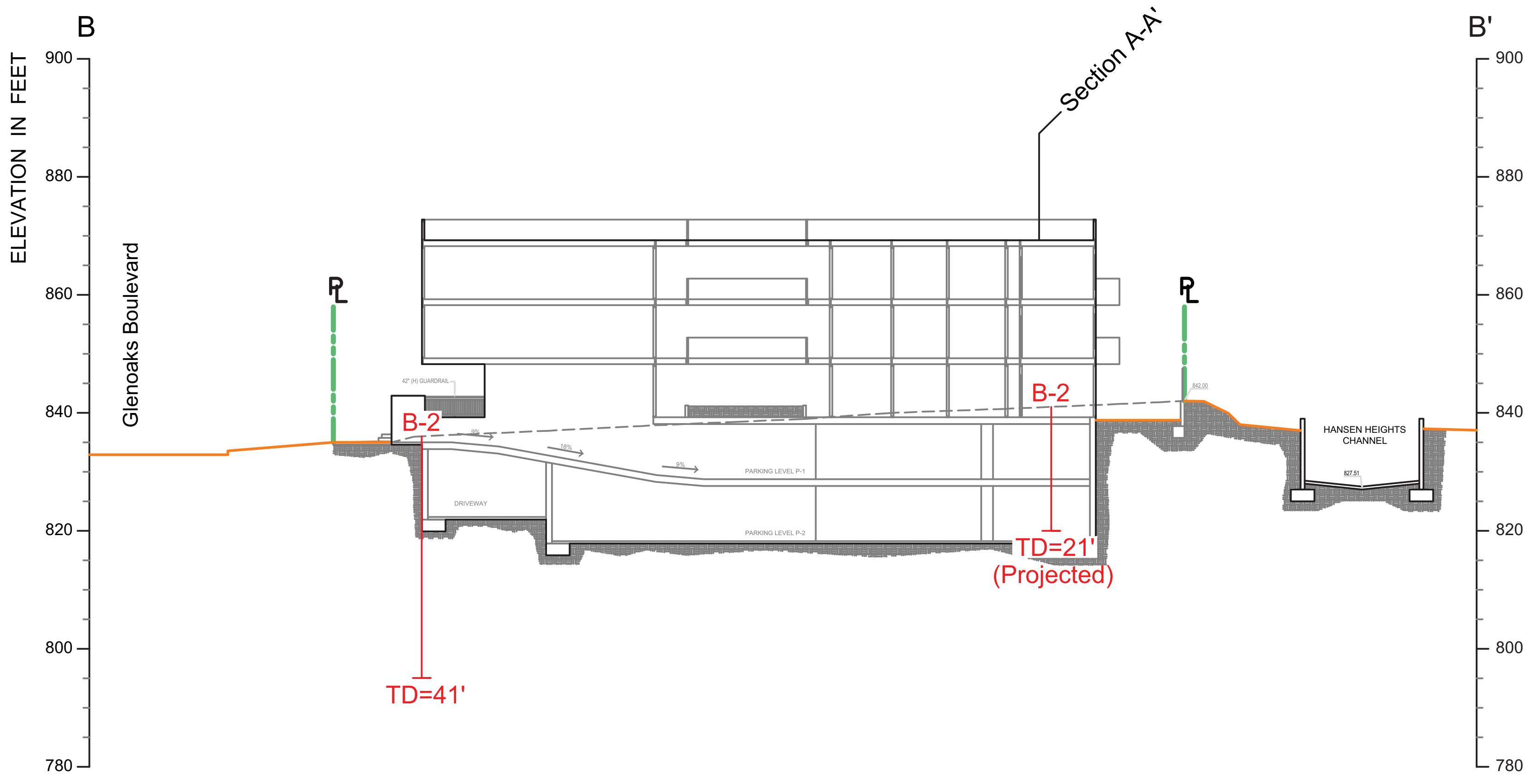

N45°E →

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

B-4

TD=10'
(Projected)

= Location & Number of Boring

CROSS SECTION B-B'

DESCRIPTION: Proposed Multi-Unit Residential Building Project

FOR: GATA Burlington LLC

ADDRESS: 8554 Glenoaks Boulevard, Los Angeles, CA 91352

PROJECT No: 16-450-02

DATE: 07 / 26 / 2016

DRAWN BY: VM

CHECKED BY: CM

DRAWING No: 3

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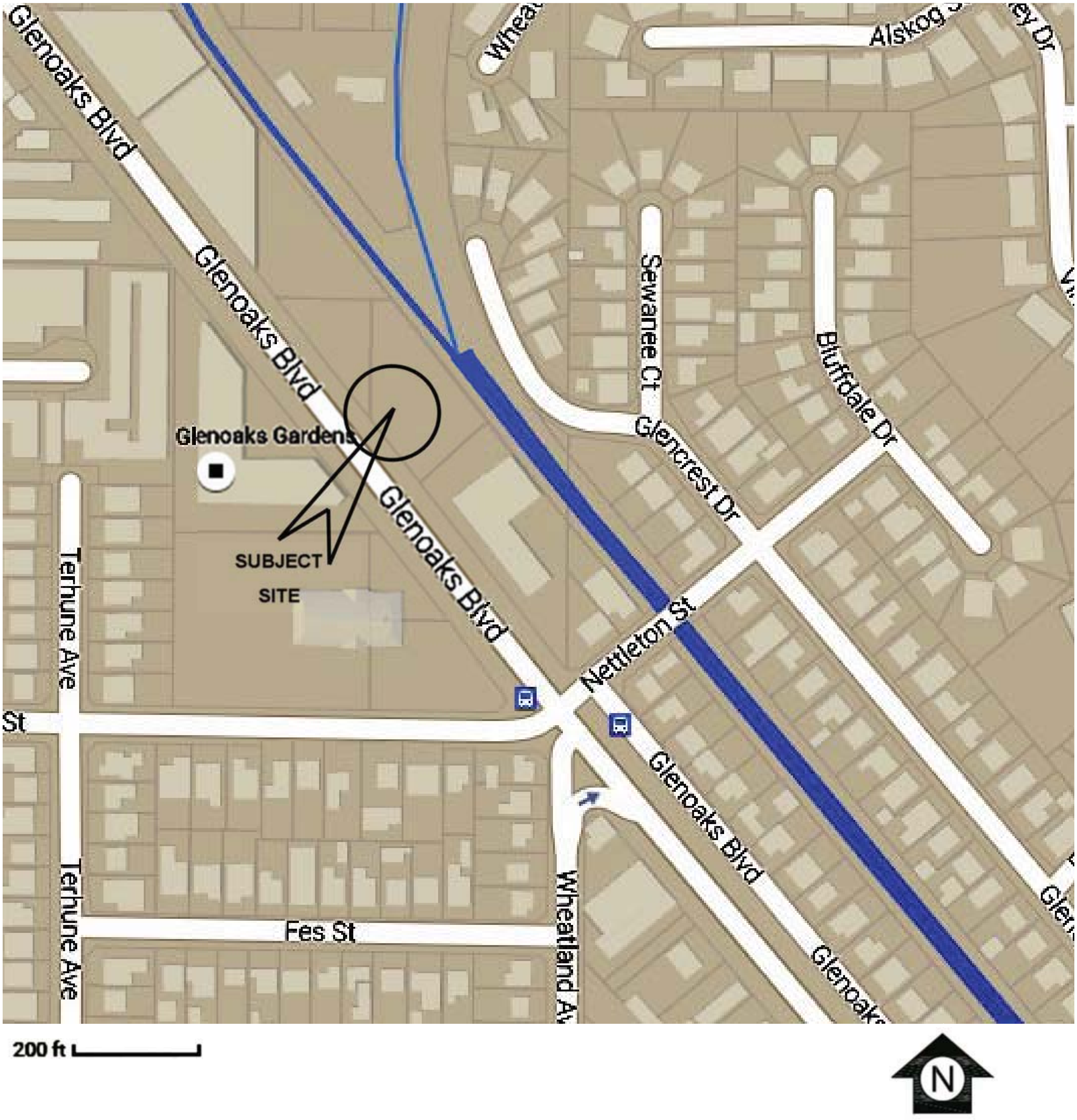


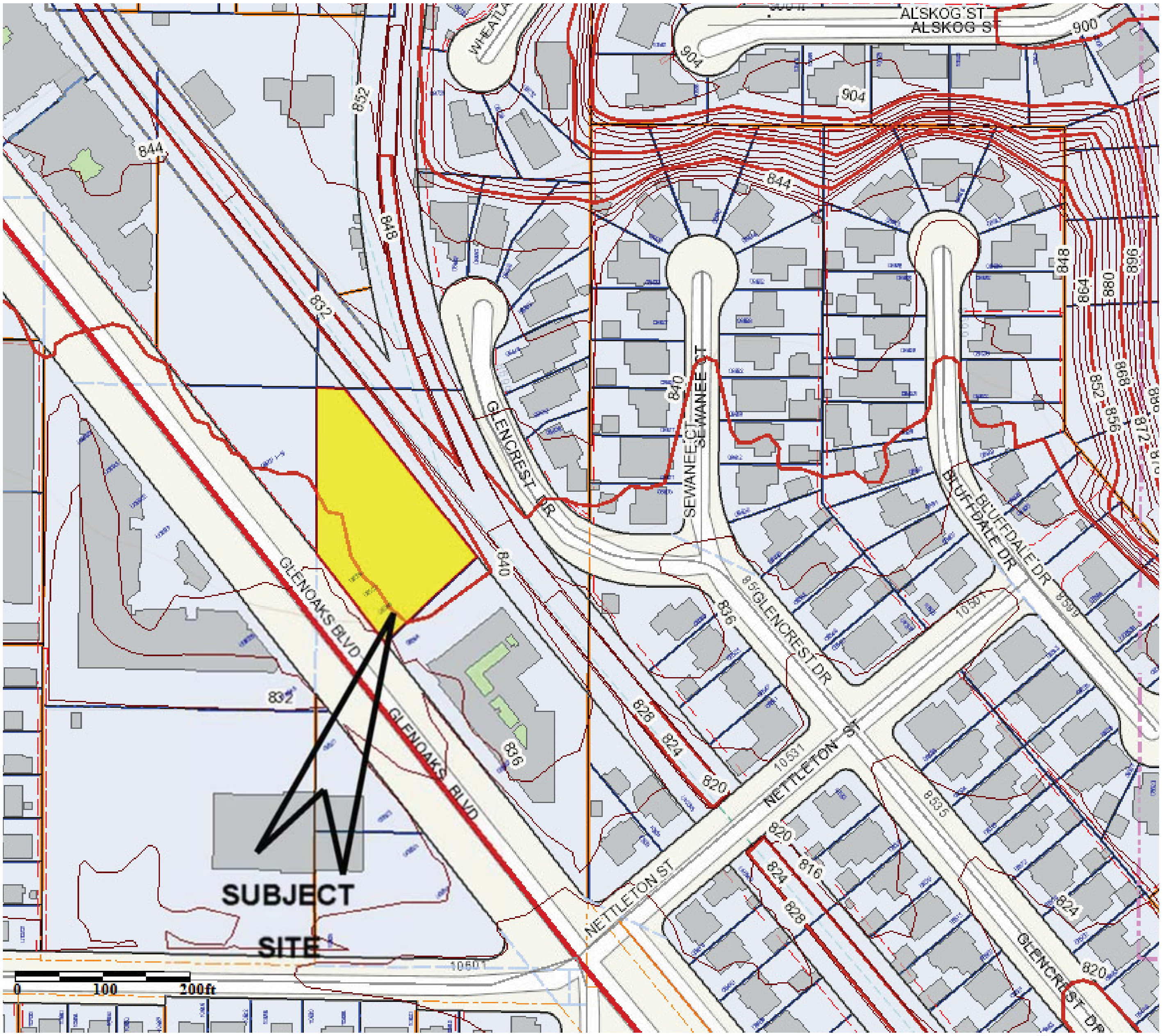
Reference: Portion of Google Maps

SITE VICINITY MAP

Proposed Multi-Unit Residential Building Project

8554 Glenoaks Blvd., Los Angeles, CA 91352

FOR GATA Burlington LLC	DATE 07 / 26 / 2016	PROJECT No. 16-450-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 1



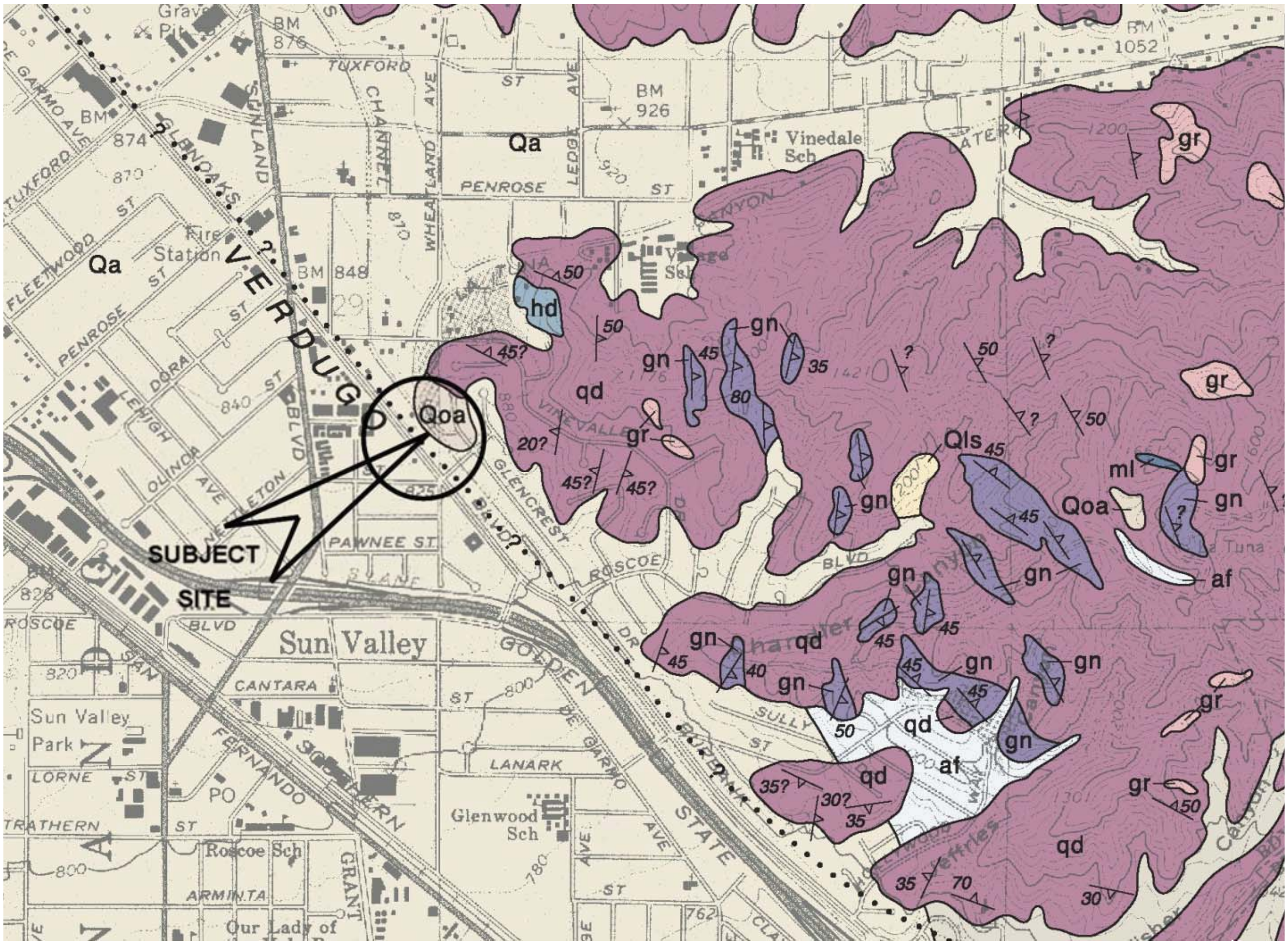
Reference: Navigate LA Los Angeles City

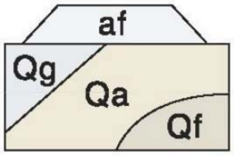
REGIONAL TOPOGRAPHIC MAP

Proposed Multi-Unit Residential Building Project

8554 Glenoaks Blvd., Los Angeles, CA 91352

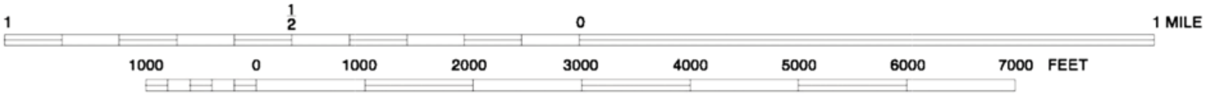
FOR GATA Burlington LLC	DATE 07 / 26 / 2016	PROJECT No. 16-450-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 2





SURFICIAL SEDIMENTS

af *Artificial cut and fill*
Qg *Gravel and sand of major stream channels*
Qa *Alluvium: gravel, sand and clay of valley areas*
Qf *Alluvial fan gravel derived from Verdugo Mountains; may be in part equivalent to* **Qof**



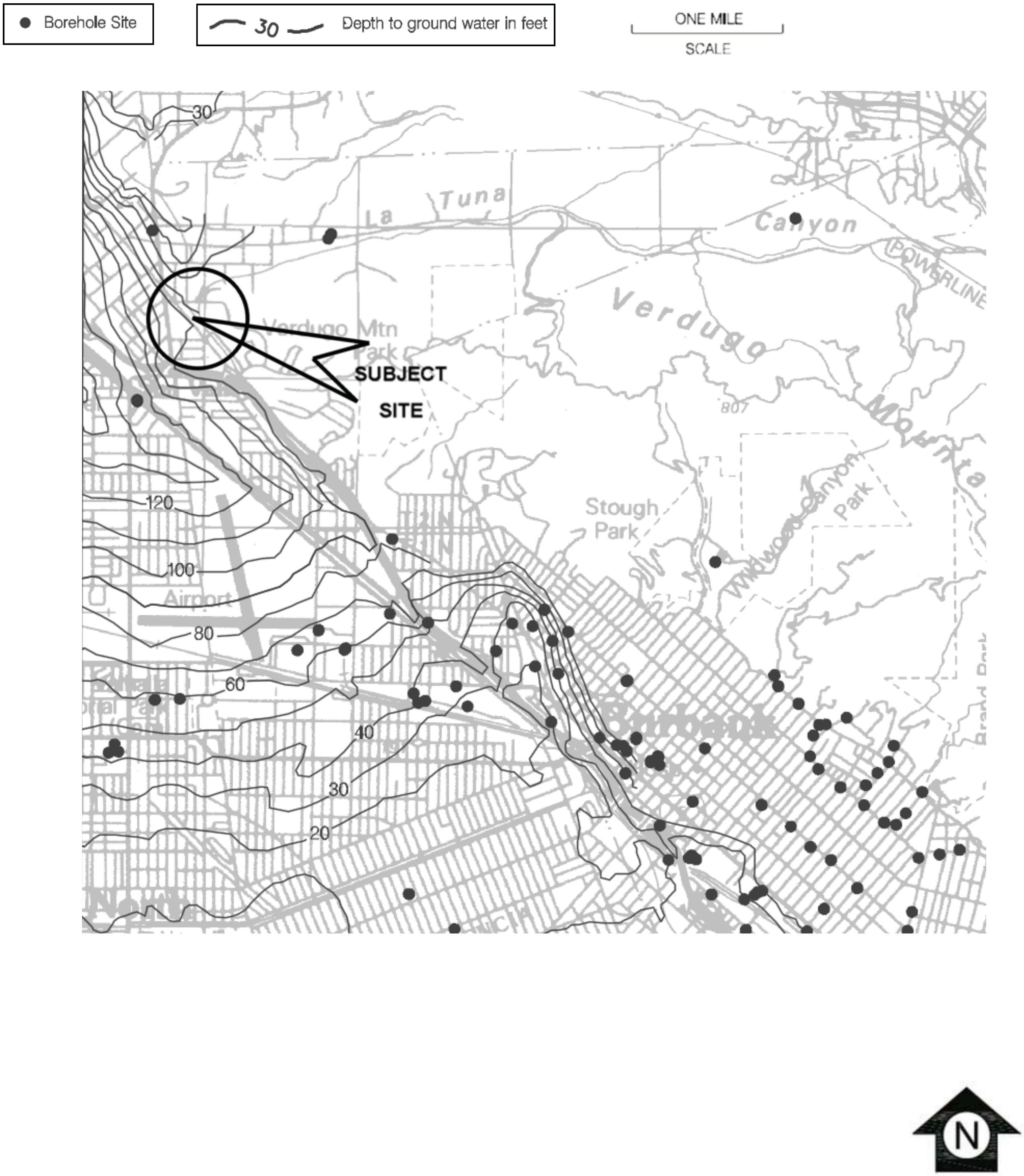
Reference: Dibblee Geologic Map of the Sunland 7.5 Minute Quadrangle

REGIONAL GEOLOGIC MAP

Proposed Multi-Unit Residential Building Project

8554 Glenoaks Blvd., Los Angeles, CA 91352

FOR GATA Burlington LLC	DATE 07 / 26 / 2016	PROJECT No. 16-450-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 3



Reference: Sunland 7.5 Minute Quadrangle

HISTORICALLY HIGHEST GROUNDWATER (Contour Map)

Proposed Multi-Unit Residential Building Project

8554 Glenoaks Blvd., Los Angeles, CA 91352

FOR GATA Burlington LLC

DATE 07 / 26 / 2016

PROJECT No. 16-450-02

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FIGURE No. 4

APPENDIX I

METHOD OF FIELD EXPLORATION

In order to define subsurface conditions at the subject site, four 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 41 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-4 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 July 12, 2016. 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.

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LOG OF BORING NO.1

16-450-02
8554 N. Glenoaks Blvd, Los Angeles, CA 91352

Type: Hollow Stem Auger with 140 lb. Hammer Logged by: Marshall
Location: Front Left of Property

% -200 - △
% Moisture - ●
20 40 60 80

DEPTH, FT	DESCRIPTION OF MATERIAL	SPT BLOWS/FT	BLOWS PER FT	% Moisture	UNIT DRY WT LB/CU FT	% -200
0	(SM) FILL: Sand, moderately compact, slightly moist, light brown/gray, gravelly silty sand.					
	(SM) SAND: Medium dense, slightly moist, light brown, silty fine to coarse grained sand, with gravel.		15	2	111	
5	(SP-SM) Grades to dense, less silty.		24	2	110	
10	(SM) Grades to silty fine to coarse grained sand.		32	8	107	
15	(SP) Grades to gravelly fine to coarse grained sand.		35	2		
20	(SP-GP) Grades to very dense, gravelly, rocky.		92	2	131	
25	(SP) Grades to similar as above. *Big chunks of rock encountered*		88	3	122	
30	(SP) Grades to gravelly fine to coarse grained sand.		100-8"	3	126	
35	(SP) Grades to very rocky.		100-7"	2	127	

COMPLETION DEPTH: 41
DATE: July 12, 2016

DEPTH TO WATER> INITIAL:
FINAL:

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LOG OF BORING NO.1

16-450-02
8554 N. Glenoaks Blvd, Los Angeles, CA 91352

Type: Hollow Stem Auger with 140 lb. Hammer Logged by: Marshall
Location: Front Left of Property

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) Grades to similar as above.		95	3	126	●	
			END @ 41' NO WATER PERC INSTALLED (30-40')						
45									
50									
55									
60									
65									
70									
75									

COMPLETION DEPTH: 41
DATE: July 12, 2016

DEPTH TO WATER> INITIAL:
FINAL:

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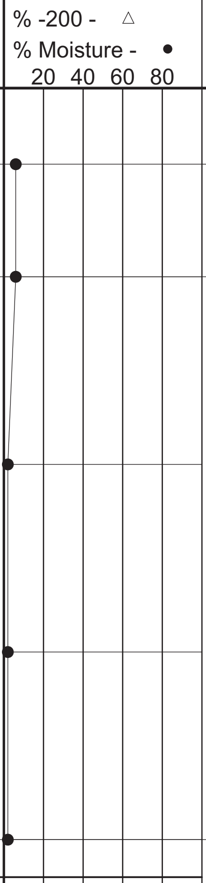
LOG OF BORING NO.2

16-450-02
8554 N. Glenoaks Blvd, Los Angeles, CA 91352

Type: Hollow Stem Auger with 140 lb Hammer
Logged by: Marshall
Location: Rear Left of Property

DEPTH, FT	DESCRIPTION OF MATERIAL	SPT BLOWS/FT	BLOWS PER FT	% Moisture	UNIT DRY WT LB/CU FT	% -200
0	(SM) FILL: Sand, moderately compact, slightly moist, brown, silty sand with gravel.					
			12	6	109	
	(SP-SM) SAND: Medium dense, slightly moist, light brown, fine to coarse sand with some gravel, slightly sandy.					
5			27	6	122	
	(SP) Grades to dense, gravelly fine to coarse grained sand					
10	(SP) Grades to similar as above.		28	2	114	
15	(SP) Grades to very dense, gray.		72	2	136	
20	(SP-GP) Grades to dense, more gravel.		32	2	131	



END @ 21'
NO WATER
HOLE BACKFILLED

COMPLETION DEPTH: 21
DATE: July 12, 2016

DEPTH TO WATER> INITIAL:
FINAL:

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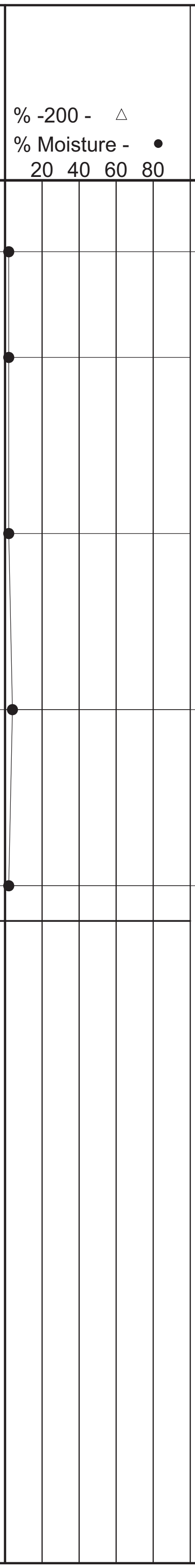
LOG OF BORING NO.3

16-450-02
8554 N. Glenoaks Blvd, Los Angeles, CA 91352

Type: Hollow Stem Auger with 140 lb Hammer Logged by: Marshall
Location: Rear Right of Property

DEPTH, FT	DESCRIPTION OF MATERIAL	SPT BLOWS/FT	BLOWS PER FT	% Moisture	UNIT DRY WT LB/CU FT	% -200
0	(SM) FILL: Sand, moderately compact, slightly moist, light brown, gravelly sand with silt.					
	(SP) SAND: Medium dense, slightly moist, light brown, gravelly fine to coarse grained sand.		14	2	111	
5	(SP) Grades to similar as above.		15	2	116	
10	(SP) Grades to gray, very dense.		37	2	114	
15	(SM) Grades to light brown, sily fine to medium grained sand, some gravel.		25	4	101	
20	(SP) Grades to gravelly fine to coarse grained sand.		34	2	122	
25	END @ 21' NO WATER HOLE BACKFILLED					
30						
35						



COMPLETION DEPTH: 21 DEPTH TO WATER> INITIAL:
DATE: July 12, 2016 FINAL:

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LOG OF BORING NO.4

16-450-02
8554 N. Glenoaks Blvd, Los Angeles, CA 91352

Type: Hollow Stem Auger with 140 lb Hammer Logged by: Marshall
Location: Front Right of Property

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: Sand< moderately compact, slightly moist, brown, silty sand with gravel.						
			(SM) SAND: Medium dense, slightly moist, light brown, gravelly fine to coarse grained sand with silt.						
5		■	(SP) Grades to gravelly fine to coarse grained sand.		10	2	113	●	
10		■	(SP-SM) Grades to gravelly fine to coarse grained sand with some silt.		16	2	111	●	
15		■	(SP) Grades to gravelly fine to coarse sand *Large Rocks*		100-5"	2	109	●	
20									
25									
30									
35									

END @ 16'
NO WATER
HOLE BACKFILLED

COMPLETION DEPTH: 16
DATE: July 12, 2016

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 $^3\!/_4$ in. 3 in. (12 in.)

U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

JOB NAME : Proposed New Multi-Unit Residential Building
8554 Glenoaks Blvd.
Los Angeles, CA 91352

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FIGURE No. I-5

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

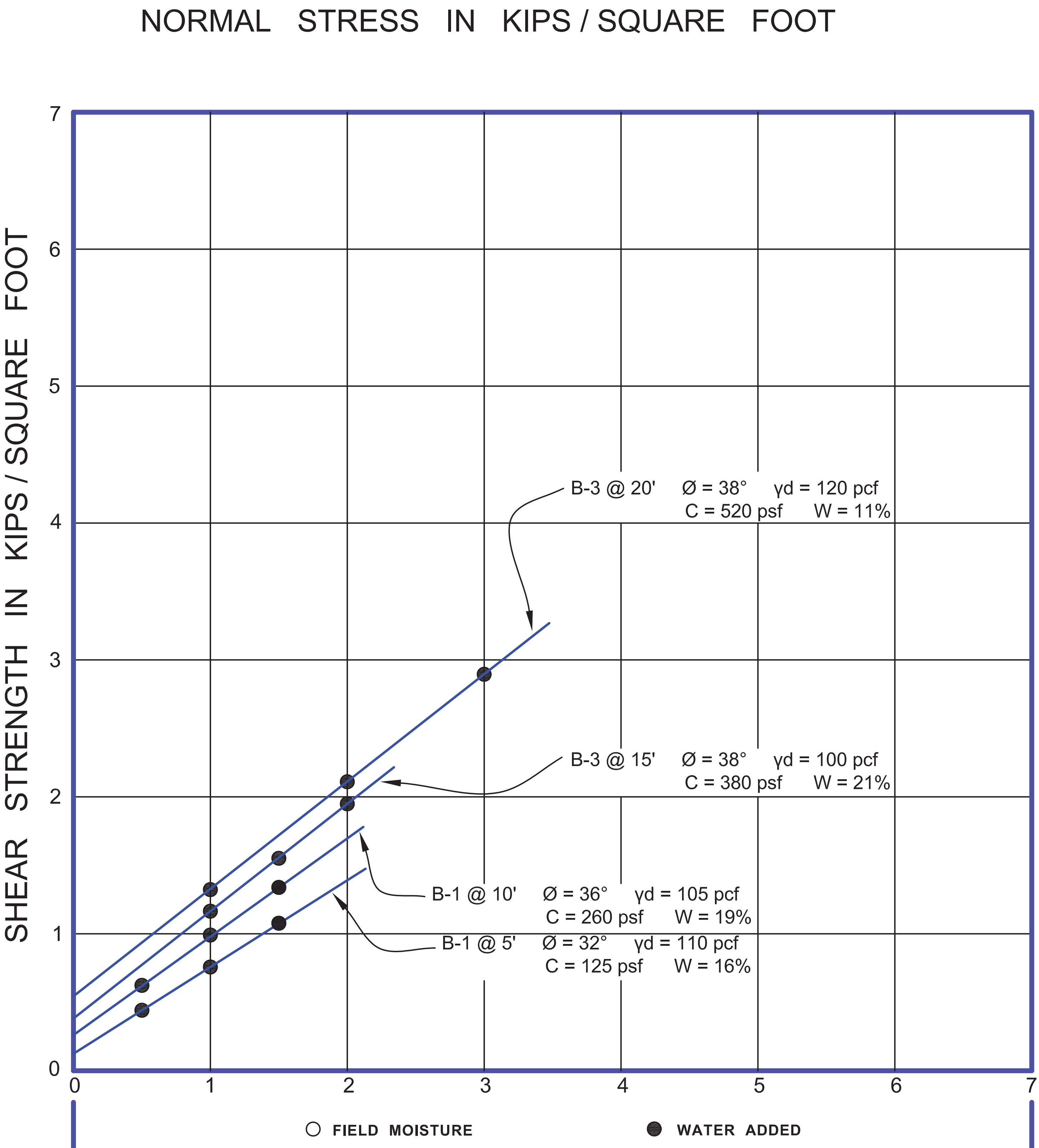
Shear tests were made with a direct shear machine at a constant rate of strain. The machine is designed to test the materials without completely removing the samples from the brass rings. The rate of shear was determined through determination of the rate of consolidation of the foundation bearing materials. 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 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 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 Multi-Unit Residential Building
8554 Glenoaks Blvd.
Los Angeles, CA 91352

JOB No.
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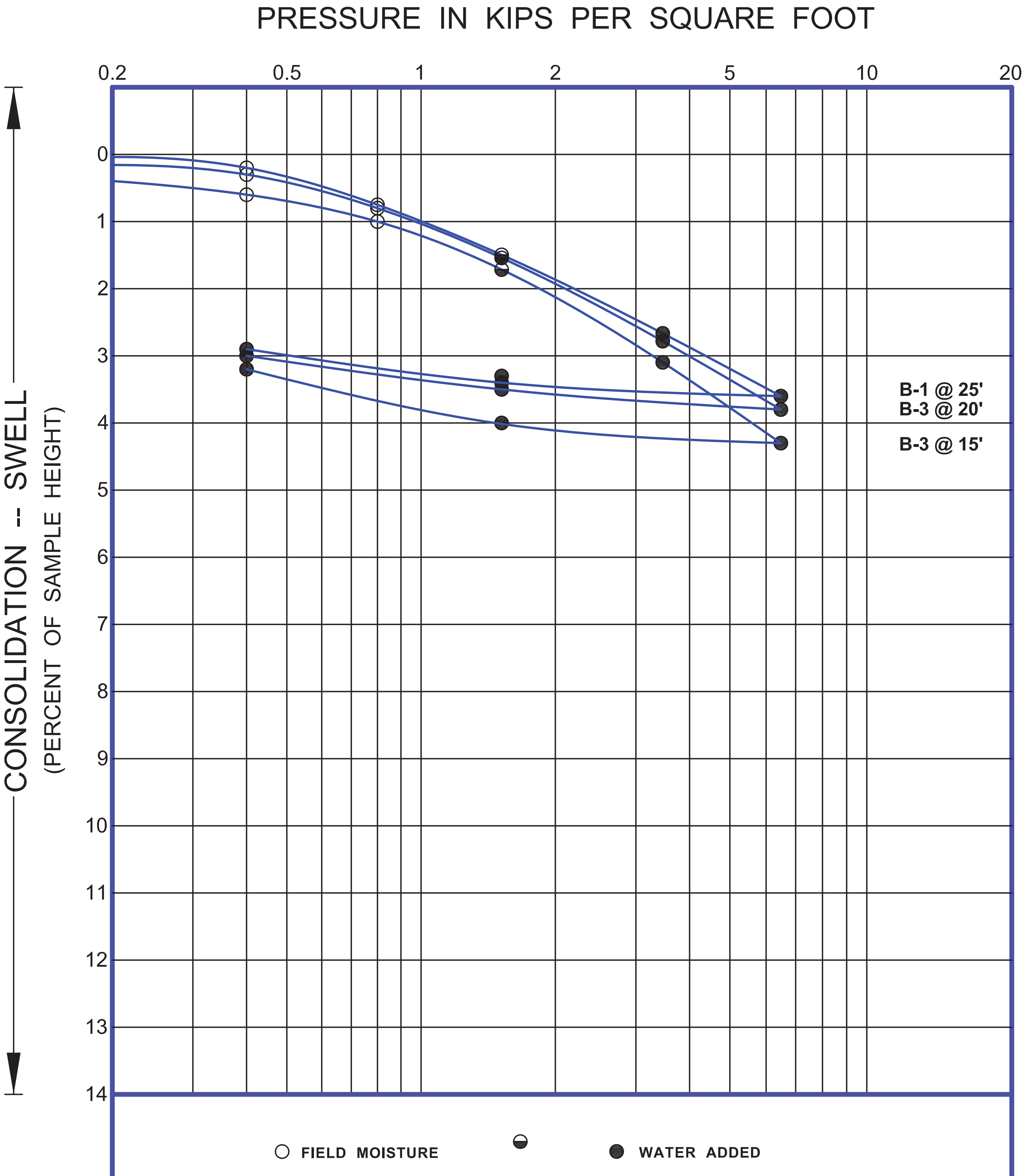
FIGURE No.
II - 1



SWELL - CONSOLIDATION TESTS

JOB NAME : Proposed New Multi-Unit Residential Building
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Los Angeles, CA 91352

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FIGURE No. II - 2

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 40 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.

APPENDIX IV

ON-SITE PERCOLATION TESTING

ON-SITE STORM WATER FILTRATION CONSIDERATIONS

It is our understanding that, as part of the development of the subject site, an on-site storm water infiltration system should be used. This normally consist of diversion of the storm water into a system that will allow infiltration into the ground. For a typical project with basement garage and perimeter walls being extended to close proximity of the respective property lines, it is common to utilize a “dry well” system. The zone of infiltration is normally kept away from existing and proposed building foundations and private property lines a minimum horizontal distance of at least 10 feet.

The subject project has a basement garage with perimeter walls extended to close proximity of the respective property lines. Using a “dry well” method and discharging the storm water some 10 feet below the building foundation, will not compromise the strength of the subgrade and associated foundation performance, because the soils below the base of the proposed building consists mainly of sand (silty to relatively clean.

For the purpose of this project, on-site percolation test was conducted in our Boring No. 1, drilled for soil testing. See log of Boring No. 1 (Figure No. I-1) in Appendix I of this report. The approximate location of the boring within which the percolation test was conducted, with respect to the site boundaries, is shown on the enclosed Site Plan; Drawing No. 1.

The boring within which the percolation test was conducted was drilled with a hollow stem drilling machine having a diameter of 8 inches. Before the percolation testing was initiated, a 3-inch diameter pipe surrounded by gravel was installed in the boring. Since the proposed building will have one to two levels of subterranean parking garage, the percolation was forced to occur below a depth of 30 feet where sand layer was found. Solid pipe was used within the top 30 feet of the percolation boring.

The percolation testing for this project were performed on July 12, 2016.

DISCUSSION OF RESULTS AND RECOMMENDATIONS

As can be seen from the log of borings contained in Appendix I, the native soils below a depth of about 30 feet consists of clean sand soils. The sand layers are considered to be of medium to high permeable character, therefore, have ability to absorb water. As such, the subject site is considered to be a good candidate for use of an on-site storm water infiltration systems. Due to space limitations, “dry well” system should be used for on-site storm water infiltration.

The proposed project will have basement garage with footings established some 20 feet below grade. For the purpose of this project, the percolation zone should occur some 10 feet below the base of the building footings which will be established at an estimated depth of about 30 feet. At the given depth, application of water into the sandy subgrade will not have adverse effects on the subgrade within the zone of foundation pressure.

The lower portion of the pipe (some 10 feet below the base of the footings) should be perforated and encased in gravel. The top portion of the pipe (within 10 feet from the base of the footing) should be solid.

The system should be designed so that any excess water not infiltrated into the subsoil would be diverted into the planter areas first and then to the street (after going through the required filtration process).

Based on the results of our in-situ percolation testing, and for design, it should be assumed that one square foot of exposed area below a depth of 30 feet can absorb some 10 gallons of water per hour. Note that no factor of safety is applied to the calculated value. The actual percolation rate may be significantly different based on the actual soil exposures along the perimeter of the dry wells.