Geotechnical Engineering Evaluation Report

Proposed DKN Hotel – Ventura Blvd

15485 & 15489 Ventura Blvd
Los Angeles, California

Prepared for:

Mr. Larry Pickell
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September 18, 2018
Project No.: 180543.1

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September 18, 2018
Project No. 180534.1

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Subject: Geotechnical Engineering Evaluation Report
Ventura Boulevard AC Hotel
15485 & 15489 Ventura Boulevard
Los Angeles, California

Dear Mr. Pickell:

In accordance with your request and authorization, Twining Consulting, Inc. is pleased to present the results of our geotechnical engineering evaluation for the proposed Ventura Boulevard AC Hotel in Los Angeles, CA. The project includes constructions of six-story structure with one level of underground parking with an approximate footprint area of 30,000 square feet. The project site is currently occupied by existing buildings, a pool and a paved parking lot which would be demolished prior to construction. Additionally, the project site is surrounded by multiple 2 to 3-story building.

The purpose of this investigation has been to evaluate the potential geologic hazards and subsurface conditions at the site and to provide preliminary recommendations for the proposed development. Our geotechnical evaluation was performed in conformance with Chapter 18A of Title 24, Part 2, Volume 2 of the 2016 California Building Code.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
TWINING CONSULTING, INC.



Rouzbeh Afshar, Ph.D., P.E. 87032
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1. INTRODUCTION

This report presents the results of the preliminary geotechnical evaluation performed by Twining Consulting, Inc. ("Twining") for the proposed hotel to be built at 15485 & 15489 Ventura Blvd Los Angeles, California. The objectives of this study were to evaluate the subsurface conditions of the site and to provide design recommendations for the proposed development.

Our geotechnical evaluation was performed in conformance with Chapter 18 of Title 24, Part 2, Volume 2 of the 2016 California Building Code (CBC).

2. SITE AND PROJECT DESCRIPTION

2.1. Site Location and Existing Conditions

The proposed Ventura Blvd AC Hotel is located at 15485 & 15489 Ventura Blvd in Los Angeles, California, as shown on Figure 1, Site Location Map. The property is bounded by Ventura Blvd on the south, and by multiple two-story and three story residential buildings on the east, west and north. The site is currently occupied by existing buildings, a pool and a paved parking lot. The property and surrounding vicinity are shown on Figure 2, Site Plan. The approximate site coordinates are latitude 34.1547°N and longitude 118.4714°W. The site is located on the Van Nuys, California 7½-Minute Quadrangle (United States Geological Survey, 1997).

2.2. Proposed Project

Based on the information provided to us. The proposed project consists of the construction of a six-story hotel building over a one-level subterranean level parking. The architectural plans by Togawa Smith Martin dated May 3, 2018 were provided to us and were used during the development of this report. The structural loads were not available at the time of preparing this report.

We estimated the maximum axial load on the columns to be approximately 400 kips (dead + live loads) and the wall loading is approximately 16 kips per foot. Should the design loads vary significantly from these values, this office should be contacted to provide revised recommendations.

3. SCOPE OF WORK

To prepare this report, we have performed the following tasks:

3.1. Literature Review

We reviewed readily available background data including previous geotechnical investigation reports, in-house geophysical data, geologic maps, and topographic maps relevant to the subject site in preparation of this report. A partial list of literature reviewed is presented in the "Selected References" section of this report.

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3.2. Field Exploration

Field exploration consisting of the drilling of three 8-inch-diameter exploratory borings was conducted at the site on July 9, 2018. The borings were advanced to the depth approximately 51½ feet, 61½ feet and 31½ below ground surface (bgs). The drilling operation was performed using a CME 75- Rubber Track Mounted Drill, hollow-stem auger drill rig. Additionally, the site has been explored by advancing two exploratory Cone Penetration Testing (CPT) soundings at the site on July 9, 2018. The CPTs were advanced to a maximum depth of 80 feet below the existing ground surface using a 30-ton truck equipped with CPT probe.

The approximate locations of the borings and CPT soundings are shown on Figure 2, Site Plan. Detailed exploration information regarding the borings and CPT soundings is presented in Appendix A.

3.3. Geotechnical Laboratory Testing

Laboratory tests were performed on selected samples obtained from the borings in order to aid in the soil classification and to evaluate the engineering properties of the foundation soils. The following tests were performed in general accordance with ASTM standards:

- In-situ moisture and density;
- Maximum dry density-optimum moisture content;
- 200 Wash Sieve;
- Atterberg Limits;
- Corrosivity;
- Expansion Index;
- R-Value;
- Consolidation;
- Remolded Direct shear; And,
- Direct Shear.

The detailed laboratory test results are presented in Appendix B.

3.4. Engineering Analyses and Report Preparation

We compiled and analyzed the data collected from our site reconnaissance, subsurface evaluation, and laboratory testing, and prepared this report to present our conclusions and recommendations. The analyses included:

- Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials;
- Evaluation of geologic hazards, including site seismicity, liquefaction and seismic settlement potential, and preliminary recommendations for appropriate mitigation measures;
- Evaluation of site-specific seismic design parameters in accordance with provisions of the 2016 California Building Code;
- Evaluation of current and historical groundwater conditions at the site and potential impact on the existing structures;

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- Evaluation of project feasibility and suitability of on-site soils for foundation support; and
- Evaluation of foundation design parameters including soil bearing capacity, lateral resistance, friction coefficient, and seismic considerations.
- Evaluation of shoring design parameters including lateral earth pressure, soil bearing capacity, lateral resistance, friction coefficient, and seismic considerations; and
- Recommendations for pavement structural design and subgrade preparation.

4. SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1. Subsurface Earth Materials

Earth materials encountered during our subsurface investigation consist of a young alluvial fan deposits (geologic map symbol Qa) extending to the total depth of exploration. The alluvial deposits comprise predominantly lean clay and sandy lean clay with thin interlayers of silty sand. Based on our field exploration, the fill depth extends approximately 5.0 feet below the existing grade; however, fill depths greater than this may be present at the site.

Detailed information regarding the exploratory excavations is presented in Appendix A, Field Exploration.

4.2. Groundwater

Groundwater was encountered at a depth of approximately 45 feet below the existing grade in Boring B-1 and at approximately 40 feet in Boring B-2. The historical high depth to groundwater is reportedly at elevation 668 feet MSL (40 to 45 feet below the ground surface) at the project site (County of LA Groundwater Well Data). Groundwater conditions may vary across the site due to stratigraphic and hydrologic conditions, and may change over time as a consequence of seasonal and meteorological fluctuations, or of activities by humans at this and nearby sites.

5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered high during the design life of the proposed improvements. The hazards associated with seismic activity in the vicinity of the site are discussed in the following sections.

5.1. Liquefaction and Seismic Settlement Potential

Liquefaction occurs when the pore pressures generated within a soil mass approach the effective overburden pressure. Liquefaction of soils may be caused by cyclic loading such as that imposed by ground shaking during earthquakes. The increase in pore pressure results in a loss of strength, and the soil then can undergo both horizontal and vertical movements, depending on the site conditions. Other phenomena associated with soil liquefaction include sand boils, ground oscillation, and loss of foundation bearing capacity. Liquefaction is generally known to occur in loose, saturated, relatively clean, fine-grained cohesionless soils at depths shallower

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than approximately 50 feet. Factors to consider in the evaluation of soil liquefaction potential include groundwater conditions, soil type, grain size distribution, relative density, degree of saturation, and both the intensity and duration of ground motion.

Based on our review of the State of California Official Map of Earthquake Zones of Required Investigation for the Van Nuys Quadrangle (California Geological Survey, 1997), the site is located within a Zone of Required Investigation for Liquefaction.

We performed a liquefaction analyses using assumed groundwater level situated at 10 feet below ground surface to evaluate the liquefaction potential. Based on the results of liquefaction potential analysis, the silty sand layers at 25 and 40 feet bgs could potentially susceptible to liquefaction during major earthquake events. The liquefaction-induced total settlement is estimated to be on the order of up to 1-inch with differential settlement of ½ -inch over a 50 feet span.

The detailed liquefaction analyses, including methodology, assumed parameters, and results are included in Appendix C, Liquefaction and Seismic Settlement Analyses.

5.2. Landslides

Based on our review of the referenced geologic maps, literature, and topographic maps, as well as our site investigation, no landslides or related features underlie or are adjacent to the subject site. Due to the relatively level nature of the site and surrounding areas, the potential for landslides at the project site is considered negligible.

5.3. Flooding

The Federal Emergency Management Agency (FEMA) has prepared flood insurance rate maps (FIRMs) for use in administering the National Flood Insurance Program. Based on our review of online FEMA flood mapping, the site is located within Zone X, which is described as “Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.” No additional flooding hazards (e.g. dam inundation hazard) are identified in the City of Los Angeles General Plan Safety Element.

5.4. Tsunamis and Seiches

Tsunamis are waves generated by massive landslides near or under sea water. The site is not located on any State of California – County of Orange Tsunami Inundation Map for Emergency Planning. The potential for the site to be adversely impacted by earthquake-induced tsunamis is considered to be negligible because the site is located miles inland from the Pacific Ocean coast, at an elevation exceeding the maximum height of potential tsunami inundation (California Emergency Management Agency and others, 2009).

Seiches are standing wave oscillations of an enclosed water body after the original driving force has dissipated. The potential for the site to be adversely impacted by earthquake-induced seiches is considered to be negligible due to the lack of any significant enclosed bodies of water located in the vicinity of the site.

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5.5. CBC Seismic Design Parameters

Our recommendations for seismic design parameters have been developed in accordance with 2016 CBC and ASCE 7-10 (ASCE, 2010) standards. The applicable site class is E based on the results of our field investigation. Table 3 presents the seismic design parameters for the site in accordance with 2016 CBC.

Table 3 – 2016 California Building Code Design Parameters

Design Parameters	Value
Site Class	D
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s	2.292 g
Mapped Spectral Acceleration Parameter at at Period 1-Second, S_1	0.846 g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Adjusted MCE_R[1] Spectral Response Acceleration Parameter at Short Period, S_{MS}	2.292 g
1-Second Period Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1}	1.269 g
Short Period Design Spectral Response Acceleration Parameter, S_{DS}	1.528 g
1-Second Period Design Spectral Response Acceleration Parameter, S_{D1}	0.846 g
Peak Ground Acceleration, PGA_M[2]	0.798 g
Seismic Design Category[3]	E

Notes: [1] Risk-targeted maximum considered earthquake
[2] Peak ground acceleration adjusted for site effects
[3] For S_1 greater than or equal to 0.75g, the seismic design category is E

6. GEOTECHNICAL ENGINEERING RECOMMENDATIONS

6.1. General Considerations

Based on the results of our field exploration and engineering analyses, it is our opinion that the proposed development is feasible from a geotechnical standpoint provided that the recommendations in this report are incorporated into the design plans and are implemented during construction.

The following is a list of geotechnical considerations for this project:

- Long term settlement due to consolidation of clay and seismic-induced settlement shall be considered in the structural foundation design. Therefore, mat foundation is recommended as the foundation system. (Section 6.4).

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- Temporary shoring system should consist of solider piles with lagging system (Section 6.7).

- Surcharge weight of the sorrounding building should be incoprporated into the design for the permanent basement wall and temporary shoring system (Section 6.5)

Our geotechnical engineering analyses performed for this report were based on the earth materials encountered during the subsurface exploration for the site. If the design substantially changes, then our geotechnical engineering recommendations would be subject to revision based on our evaluation of the changes. The following sections present our conclusions and recommendations pertaining to the engineering design for this project.

6.2. Corrosive Soils

The potential for the near-surface on-site materials to corrode buried steel and concrete improvements was evaluated. Laboratory testing was performed on one representative sample of on-site soils to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test 643, and the sulfate and chloride tests were performed in accordance with California Tests 417 and 422, respectively. These laboratory test results are presented in Appendix B.

In accordance with the Caltrans (2014) criteria, corrosive soil is defined as the soil has minimum resistivity less than 1,000 ohm-centimeters, or chloride concentration greater than 500 ppm, or sulfate concentration in soils greater than 2,000 ppm, or a pH less than 5.5. Based on the results of resistivity testing we would rate the soil on-site as non-corrosive.

6.2.1.Reinforced Concrete

Laboratory tests indicate that the potential for sulfate attack on concrete in contact with the on-site soils is negligible in accordance with ACI 318, Table 4.3.1. As a minimum, we recommend that Type I or II cement and a water-cement ratio of no greater than 0.5 be used on the project.

Test results also indicate that the potential for chloride attack of reinforcing steel in concrete structures and pipes in contact with soil is negligible.

6.2.2.Metallic

Laboratory resistivity testing indicates that the on-site near-surface soils are considerred severely corrosive to buried ferrous metals. We recommenda that a corrosion specialist may be consulted regarding suitable types of piping and appropriate protection for underground metal conduits as needed.

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6.3. Foundation for the New Building

In order to mitigate the potential building settlement and seismic liquefaction hazard due to presence of shallow groundwater, we recommend the proposed building be supported on a mat foundation.

6.3.1. Mat Foundation

The mat foundation should be designed according to the parameters listed below in Table 2 – Mat Foundation Design Parameters.

Table 2 – Mat Foundation Design Parameters

Mat Foundations	• Allowable bearing capacity of minimum 2,000 psf average; 2,500 psf localized (minimum factor of safety = 3 for static)[1].
Static Settlement	• The settlement was estimated to be approximately 0.5 inches • Please refer to Figure C-2 in Appendix C for detailed settlement contours based on the structural load provided by the structural engineer.
Seismic Settlement	Total settlement is on the order of 1-inch with differential settlement less than ½-inch over 50 feet span.
Allowable Coefficient of Friction	0.25
Unfactored Lateral Passive Resistance	300 pcf (equivalent fluid pressure)
Modulus of Soil Subgrade Reaction	• Average soil subgrade reaction of 8 pci can be used for foundation design.

[1] Pressures may be increased by 1/3 to account for transient loads (i.e. wind and seismic loading.)
[2] Since the mat foundation will be constructed below the current groundwater level, the soil at the subgrade of mat foundation is anticipated to be at near saturated conditions, and most likely requires subgrade stabilization.

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6.3.2.Spread Footings for Surface Improvements

A shallow foundation system may be used for support of the at-grade light structures. The recommended geotechnical foundation design parameters are presented in Table 3 below.

Table 3 – Geotechnical Foundation Design Parameters

Minimum Footing Dimensions	• Continuous footing: At least 12 inches in width, and at least 18 inches in depth. • Square footing: At least 24 inches in width and at least 18 inches in depth.
Allowable Bearing Pressure	• Footing should be supported on compacted fill. • For foundations with the minimum dimensions shown above, a soil bearing pressure of 2,000 pounds per square foot (psf) can be used. • Bearing capacity can increase 250 psf for each additional foot of width, and 350 psf for each additional foot of depth to a maximum allowable capacity of 3,500 psf. • The allowable bearing values may be increased by one-third for transient live loads from wind or earthquake.
Estimated Static Settlement	• Less than 1.0 inches total settlement with differential settlement estimated to be less than 0.5 inch over 50 feet. • The static settlement of the foundation system is expected to occur on initial application of loading.
Allowable Coefficient of Friction Below Footings	0.35 (against granular fill soil)
Unfactored Lateral Passive Resistance	350 pcf (equivalent fluid pressure)

6.4. Retaining Walls

6.4.1.Lateral Earth Pressure

The values presented below assume that the supported grade is level. The values do not include surcharge loads. The recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind the walls and that external hydrostatic pressure will not develop behind the wall.

For walls that are free to rotate at the top (such as cantilevered walls) and have adequate drainage, the lateral earth pressure may be designed for the “active” equivalent fluid pressure (EFP) of 45 pcf.

Walls that are supporting earth that has adequate drainage, and that are restrained against rotation at the top (such as by a floor deck), may be designed for the “at-rest” EFP of 66 pcf.

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Vertical surcharge loads within a 1:1 projection from the bottom of the wall distributed over retained soils should be considered as additional uniform horizontal pressure acting on the wall. The additional horizontal pressure acting on the wall can be estimated as approximately 30% of the magnitude of the vertical surcharge pressure for the "active" conditions. All permanent surcharge loading conditions should be evaluated on a case-by-case basis by the geotechnical engineer.

6.4.2.Seismic Lateral Earth Pressure

Retaining walls greater than 6 feet in height should be designed for seismic lateral earth pressures. The "total seismic earth pressure", which is a combination of "static earth pressures" and "incremental seismic pressure" can be designed for an EFP of 70 PCF for both cantilever and restrained walls. The pressure distribution may be considered a triangle with the maximum pressure at the bottom of the wall. The "incremental seismic pressure" can be calculated as the difference of "total seismic earth pressure" and "static active lateral pressure".

6.4.3.Backfill and Drainage of Walls

The backfill material should consist of granular, non-expansive material with an expansion index no greater than 30 and should be approved by the project geotechnical engineer. Based on the soil materials encountered during our exploration, the majority of on-site soils should meet this requirement. Retaining walls should be adequately drained. Adequate backfill drainage is essential in order to provide a free-drained backfill condition and to limit hydrostatic buildup behind walls. The walls should be appropriately waterproofed in accordance with the recommendations of the project architect.

Drainage behind the walls may be provided by a geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, attached to the outside perimeter of the wall. The drain should be placed continuously along the back of the wall and connected to a 4-inch-diameter perforated pipe. The pipe should be sloped at least 1 percent and should be surrounded by 1 cubic foot per foot of ¾-inch crushed rock wrapped in suitable non-woven filter fabric (Mirafi 140NL or equivalent). The crushed rock should meet the requirements defined in Section 200-1.2 of the latest edition of the "Greenbook" Standard Specifications for Public Works Construction. The drain should discharge through a solid pipe to an appropriate outlet.

6.5. Surcharge from Adjacent Buildings

As indicated herein, additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures for retaining walls and shoring design.

The following surcharge equation may be utilized to determine the surcharge loads on the basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

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Resultant lateral force: $R = \frac{0.3\mathrm{Ph}^2}{\mathrm{x}^2+\mathrm{h}^2}$

Location of lateral resultant: $d = \mathrm{x}\left[\left(\frac{x^2}{h^2}+1\right) \times tan^{-1}\left(\frac{h}{x}\right) - \frac{x}{h}\right]$

Where:

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

the structural engineer and shoring engineer may use this equation to determine the surcharge loads based on the loading of the adjacent structures located within the surcharge influence zone.

6.6. Temporary Excavations

Temporary excavations for the demolishing, earthwork, footing and utility trench are expected. We anticipate that unsurcharged excavations with vertical side slopes less than 4 feet high will generally be stable; however, some sloughing of cohesionless sandy materials encountered at the site should be expected.

Where the space is available, temporary, unsurcharged excavation sides over 4 feet in height should be sloped no steeper than an inclination of 1H:1V (horizontal:vertical). Where sloped excavations are created, the tops of the slopes should be barricaded so that vehicles and storage loads do not encroach within 10 feet of the top of the excavated slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Twining should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces.

Excavations shall not undermine the existing adjacent building footings. Where space for sloped excavations is not available, slot-cut or temporary shoring (trench box) may be utilized. For temporary excavations that are less than 6 feet in height adjacent to existing buildings where the excavation extends deeper than the bottom of the existing footing, slot cuts may be utilized. The slots should be no wider than 8 feet and should be excavated in an A-B-C sequence so that there are at least 16 feet spacing between any two excavated slots. The excavated slots should not be left open overnight and should be backfilled on the same day it was excavated before the next set of slots are excavated.

Personnel from Twining should observe the excavations so that any necessary modifications based on variations in the encountered soil conditions can be made. All

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applicable safety requirements and regulations, including CalOSHA requirements, should be met.

6.7. Temporary Shoring

Temporary shoring is anticipated to be placed along the perimeter of the proposed basement parking garage. Based on the assumed finished floor elevation and anticipated foundation excavations, shored walls may be on the order of 10 to 15 feet high.

For vertical excavations less than approximately 15 feet in height, cantilevered shoring may be used. Where cantilevered shoring is used for deeper excavations, the total deflection at the top of the wall tends to exceed acceptable magnitudes. Shoring of excavations deeper than approximately 15 feet may need to be accomplished with the aid of tied-back earth anchors.

The shoring design should be provided by a California Registered Civil Engineer experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and the design should be reviewed by Twining for conformance with the design intent and recommendations.

6.7.1. Lateral Pressures

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the drained soils, with a level surface behind the cantilevered shoring, will exert an equivalent fluid pressure of 45 pcf.

Tied-back or braced shoring should be designed to resist a trapezoidal distribution of lateral earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, the maximum pressure equal to 28H in psf, where H is the height of the shored wall in feet.

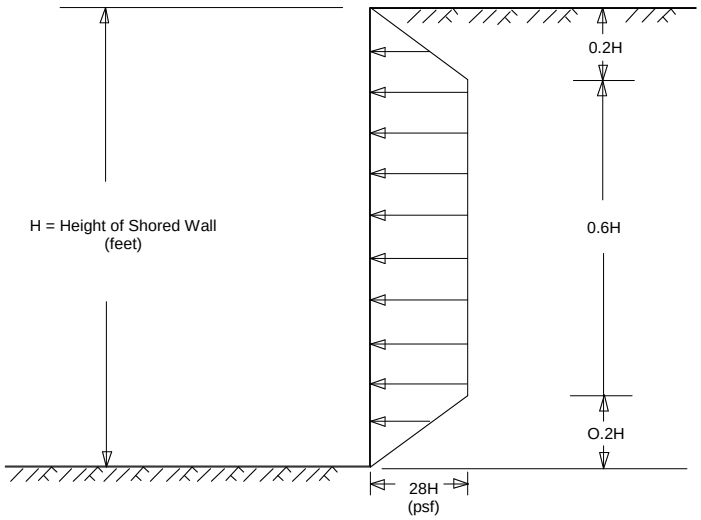


Diagram 1 – Earth Pressure Distribution for Tie-back or Braced Shoring Wall

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Any surcharge (live, including traffic, or dead load) located within a 1:1 plane projected upward from the base of the shored excavation, including adjacent structures, should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind the temporary shoring may be calculated by multiplying the vertical surcharge pressure by 0.30. Lateral load contributions of surcharges located at a distance behind the shored wall may be provided once the load configurations and layouts are known. As a minimum, a 250 psf vertical uniform surcharge is recommended to account for nominal construction and/or traffic loads. More detailed lateral pressure and loading information can be provided, if needed, for specific loading scenarios as recognized through the design process.

6.7.2. Soldier Pile Design

The soldier piles should be designed in accordance with the geotechnical parameters presented in Table 4. Soldier piles should be spaced no closer than 2.5D on center, where D is the diameter of the drilled shaft for the soldier piles.

Table 4: Geotechnical Design Parameters for Soldier Piles

The lateral resistance of an isolated soldier pile drilled or driven into the on-site soils can be calculated using **unfactored lateral passive resistance** equivalent fluid pressure (EFP)	300 pcf
Increase (multiplier) of the ultimate lateral passive resistance due to arching (this value is applicable for soldier piles that are spaced no closer than 2.5 diameters on center)	2.0

The downward component of a tie-back anchor load transferred to the soldier pile may be supported by frictional resistance between the soldier piles and the retained earth, and the skin fiction of the pile shaft below finished excavation grade. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.35 times the horizontal component of anchor load. The allowable downward capacity of a soldier pile below the excavated level may be estimated using an average allowable unit skin friction of 450 psf per foot of embedment below the excavation bottom. This allowable unit skin friction incorporates a factor of safety of 2.0. The upper 1.5D should be neglected when calculating the axial capacity below the excavated level.

Continuous treated timber lagging should be used between the soldier piles. If treated timber is used, the lagging may remain in place. To develop the full lateral resistance, provisions should be taken to assure firm contact between the soldier piles and the soils; for this, we recommend that sand-cement slurry fill behind the lagging be used. For drilled piles, we recommend that piles adjacent to one another be drilled alternately on different days to minimize disturbance to the open excavations.

Drilling of the soldier pile shafts can be accomplished using conventional drilling equipment. Additionally, caving should be anticipated within the upper approximately 40 feet where layers of loose to medium dense clean sand with

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gravel and cobble were encountered during our drilling program. In the event of soil caving, it may be necessary to use casing and/or drilling mud to permit the installation of the soldier piles. Drilled holes for soldier piles should not be left open overnight. Concrete for piles should be placed immediately after the drilling of the hole is complete. The concrete should be pumped to the bottom of the drilled shaft using a tremie. Once concrete pumping is initiated, the bottom of the tremie should remain below the surface of the concrete to prevent contamination of the concrete by soil inclusions. If steel casing is used, the casing should be removed as the concrete is placed. The contractor should consider the use of driven piles or piles that are vibrated into place in lieu of drilled piles to address potential issues related to caving of drilled shafts.

6.7.3. Temporary Shoring Monitoring

Due to the proximity of the excavation to existing improvements, some means of monitoring the performance of the shoring system is recommended. Monitoring should consist of periodic surveying of lateral and vertical locations at the tops of all soldier piles. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system has been finalized. Also, we should review the shoring plans and calculations to evaluate whether our recommendations have been incorporated into the design.

6.8. Excavation Bottom Stabilization

In general, we anticipate that excavation bottoms of the excavations will be stable and should provide suitable support for the proposed improvements. Unstable bottom conditions may be mitigated by overexcavation of the bottom to suitable depths, and/or replacement with a minimum 6- to 12-inch-thick aggregate base based on the field evaluation. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

6.9. At-Grade (Street Grade) Earthwork

In general, earthwork should be performed in accordance with the recommendations presented in this report. Twining should be contacted for questions regarding the recommendations or guidelines presented herein.

6.9.1. Site Preparation

Site preparation should begin with the removal of utility lines, asphalt, concrete, vegetation, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside edges of the proposed excavation and fill areas. We recommend that unsuitable materials such as organic matter or oversized material be selectively removed and disposed of offsite. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed at a legal dump site away from the project area.

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6.9.2. Subgrade Preparation for At-Grade Foundations

Undocumented fill soil within the proposed new building areas should be completely removed by over-excavation. We recommend that the at-grade foundations for the building be supported by at least 3 feet of properly compacted fill to minimize the potential for compression of the upper compressible soils encountered at the site.

The excavation for the building areas should be reviewed by the Geotechnical Engineer of Record prior to placement of any fill. The lateral limits of the overexcavation should extend at least 5 feet beyond the outside edges of new foundations. We anticipate that the majority of on-site soils can be excavated and replaced as compacted fill. Fill should be compacted to at least 90 percent of the maximum dry density as determined by ASTM D 1557.

6.9.1.Flatwork and Pavement Preparation

Exterior Flatwork/Sidewalks: Exterior flatwork and sidewalks subgrade that are not subject to vehicle traffic, and that are outside the limits of grading for the building pad area should bear on relatively undisturbed and competent native soils. Any undocumented fills encountered during site grading should be completely removed or properly compacted to at least 90 percent of the maximum dry density as determined by ASTM D 1557.

Pavements: Pavements subgrade that are subject to vehicle traffic and that are outside the limits of grading for the building pad area should bear on at least 12 inches of engineering fill compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557.

A representative of Twining should observe the bottoms of the excavations prior to placement of any geogrid or aggregate base materials and be present during the placement and compaction of aggregate base materials.

6.9.2.Materials for Fill

On-site soils with an organic content of less than 3 percent by volume (or 1 percent by weight) are suitable for use as fill. Soil material to be used as fill should not contain contaminated materials, rocks, or lumps over 4 inches in largest dimension, and not more than 40 percent larger than ¾ inch. Utility trench backfill material should not contain rocks or lumps over 3 inches in largest dimension. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed of offsite.

Any imported fill material should consist of granular soil having a “very low” expansion potential (that is, expansion index of 20 or less). Import material should also have low corrosion potential (that is, chloride content less than 500 parts per million [ppm], soluble sulfate content of less than 0.1 percent, and pH of 5.5 or higher). Materials to be used as fill should be evaluated by a Twining representative prior to importing or filling.

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6.9.3. Compacted Fill

Prior to placement of compacted fill, the exposed excavation bottoms should be observed by Twining. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 6 inches and watered or dried, as needed, to achieve generally consistent moisture contents at or near the optimum moisture content. The scarified materials should then be compacted to 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557.

Fill materials should be moisture conditioned to approximately 2% above optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass. Continue to place the compacted fill in horizontal lifts of approximately 6 to 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed, mixed, and then compacted by mechanical methods, using multiple wheel pneumatic tired rollers, sheepsfoot rollers, or other appropriate compacting rollers, to a relative compaction of 90 percent as evaluated by the latest version of ASTM D1557. Successive lifts should be treated in a like manner until the desired finish grades are achieved.

The evaluation of compaction by Twining should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify Twining and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

6.9.4. Utility Trench Backfill

Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement. The utility should be bedded with clean sand to at least one foot over the crown. The bedding sand should have a sand equivalent (SE) of 30 or greater. The remainder of trench backfill may be onsite soils compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D1557.

6.9.5.Rippability

The alluvial materials should be generally excavatable with heavy-duty earthwork equipment in good working condition.

6.9.6. Shrinkage/Bulking Due to Compaction

Based on our review of the in-situ soil density data, preliminary volumetric shrinkage on the order of 10 to 15 percent as a result of compaction of onsite soil may be assumed.

6.9.7.Construction Dewatering

Due to the absence of shallow groundwater, dewatering measures are not anticipated to be necessary during excavation operations. If needed, considerations for construction dewatering should include anticipated drawdown,

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volume of pumping, potential for settlement of nearby structures, and groundwater discharge. Disposal of groundwater should be performed in accordance with guidelines of the Regional Water Quality Control Board.

6.10. Flexible Pavement Design

Our pavement structural design is in accordance with Chapter 600 of the Caltrans Highway Design Manual, which is based on a relationship between the gravel equivalent (GE) of the pavement structural materials, the traffic index (TI), and the R-value of the underlying subgrade soil.

We used an R-value of 14 for the subgrade and assumed TI values for our asphalt pavement structural calculations. On this basis, Table 4 provides recommended minimum thicknesses for hot mix asphalt (HMA) and aggregate base sections for different traffic indices.

Table 5 – Recommended Minimum HMA and Base Section Thicknesses

Location	Light Vehicle Parking	Firelane / Truck Drive Way
Traffic Index	**5.0**	**7.0**
HMA Thickness (in)	3.0	6.0
Aggregate Base Thickness (in)	8.0	10.0

Aggregate base should be compacted to 95 percent relative compaction in accordance with the ASTM Test Method D1557.

6.11. Rigid Pavement Design

Table 5 provides minimum thicknesses for Portland Cement Concrete (PCC) pavement sections for anticipated Average Daily Truck Traffic (ADTT) constructed on properly prepared subgrade and aggregate base section compacted to 95 percent of the maximum dry density in accordance with ASTM D1557.

Table 6 – Recommended Minimum PCC Section Thicknesses

ADTT	0.8	130
PCC Thickness (in)	5.0	6.0
Crushed Aggregate Base (in)	6.0	6.0

The above pavement section is based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,500 psi. Transverse contraction joints should not be spaced more than 15 feet and should be cut to a depth of ¼ the thickness of the slab. Longitudinal joints should not be spaced more than 15 feet apart, however, and are not necessary in the pavement adjacent to the curb and gutter section. Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

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6.12. Drainage Control

The control of surface water is essential to the satisfactory performance of the building and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the improvements, even during periods of heavy rainfall. The following recommendations are considered minimal:

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 1 percent.
- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.
- Planters should not be located adjacent to the structures wherever possible. If planters are to be located adjacent to the structures, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.
- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked periodically to detect leakage and they should be turned off during the rainy season.

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7. DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation of excavations will be important to the performance of the proposed development. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

7.1. Plans and Specifications

The design plans and specifications should be reviewed by Twining prior to bidding and construction, as the geotechnical recommendations may need to be reevaluated in the light of the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications. Based on the work already performed, this office is best qualified to provide such review.

7.2. Construction Monitoring

Site preparation, removal of unsuitable soils, assessment of imported fill materials, fill placement, foundation installation, and other site grading operations should be observed and tested. The substrata exposed during the construction may differ from that encountered in the test excavations. Continuous observation by a representative of Twining during construction allows for evaluation of the soil conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where necessary.

The project engineer should be notified prior to exposure of subgrades. It is critically important that the engineer be provided with an opportunity to observe all exposed subgrades prior to burial or covering.

8. LIMITATIONS

The recommendations and opinions expressed in this report are based on Twining, Inc.'s review of available background documents, on information obtained from field explorations, and on laboratory testing. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site. In the event that any of our recommendations conflict with recommendations provided by other design professionals, we should be contacted to aid in resolving the discrepancy.

Due to the limited nature of our field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of removal of unsuitable soil, and that additional effort may be required to mitigate them.

Site conditions, including groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result

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of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Twining, Inc. has no control.

Twining's recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Twining to observe grading operations and foundation excavations for the proposed construction. If parties other than Twining are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Twining should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report has been prepared for the exclusive use by the client and its agents for specific application to the proposed project. Land use, site conditions, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of this report and the nature of the new project, Twining may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Twining from any liability resulting from the use of this report by any unauthorized party.

Twining performed its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either express or implied, is made as to the conclusions and recommendations contained in this report.

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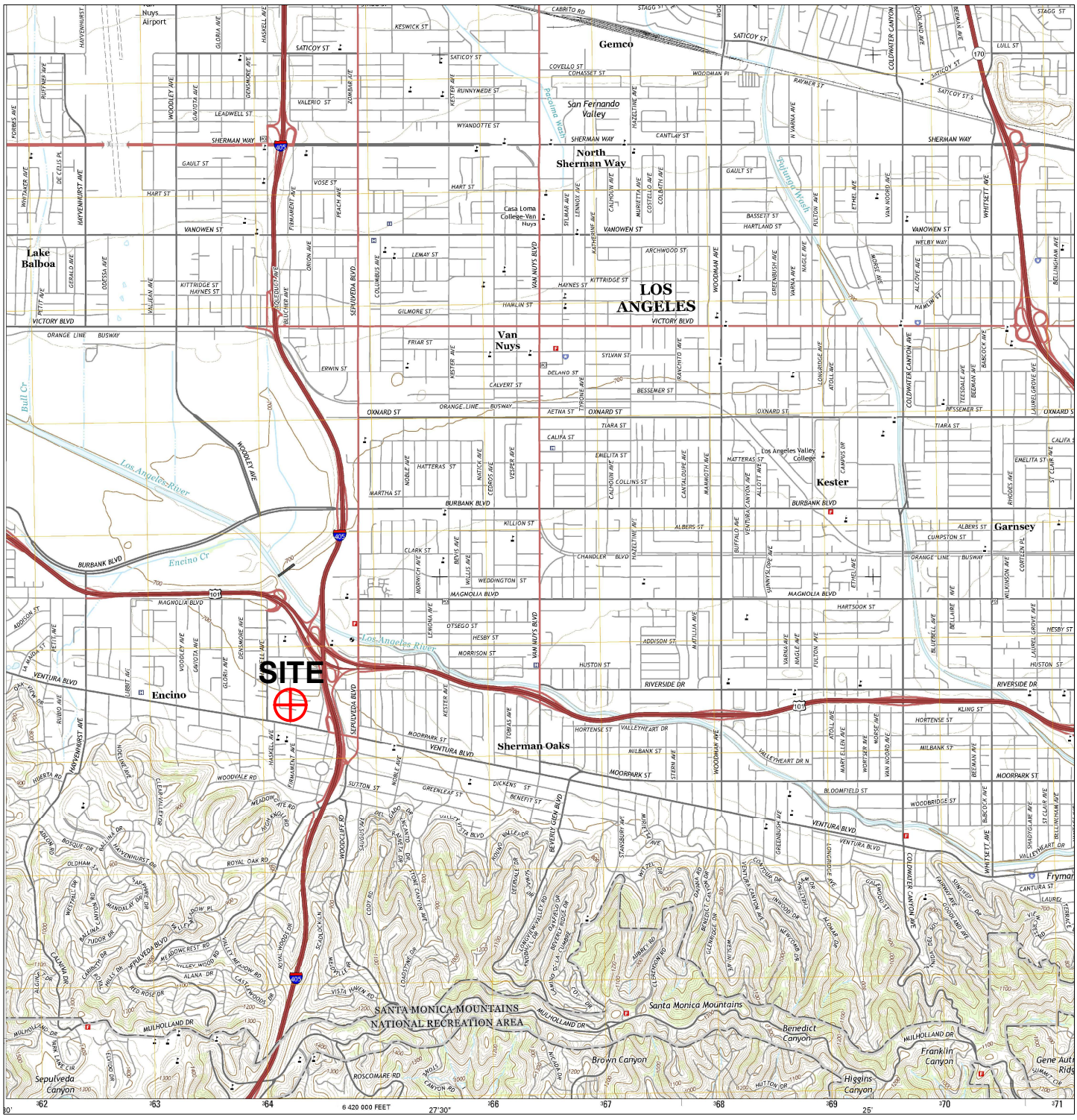
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FIGURES



SCALE IN MILE

0 0.8 1.6

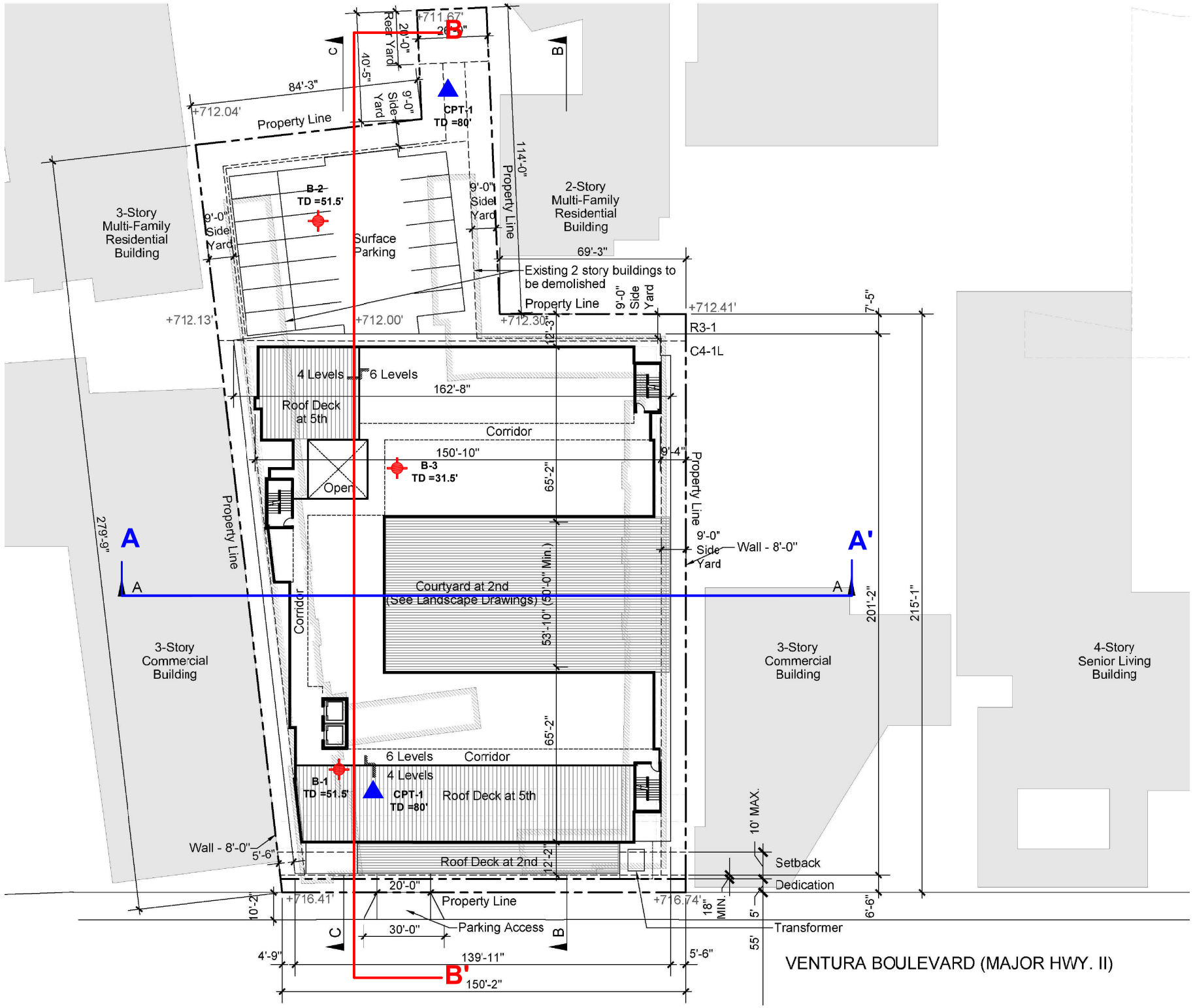
REFERENCE: USGS (2015)

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SITE LOCATION MAP

DKN, Ventura Blvd AC Hotel
15485 & 15489 Ventura Blvd
Sherman Oak, California

PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	FIGURE 1



SCALE IN FEET

0 47 94

NOTE: All dimensions, locations, and directions are approximate.

LEGEND

- HOLLOW-STEM BORING LOCATION AND DEPTH — B-1 TD = 50'
- CONE PENETRATION TEST LOCATION AND DEPTH — CPT-1 TD = 80'
- B / B' — CROSS SECTION
- A / A' — CROSS SECTION

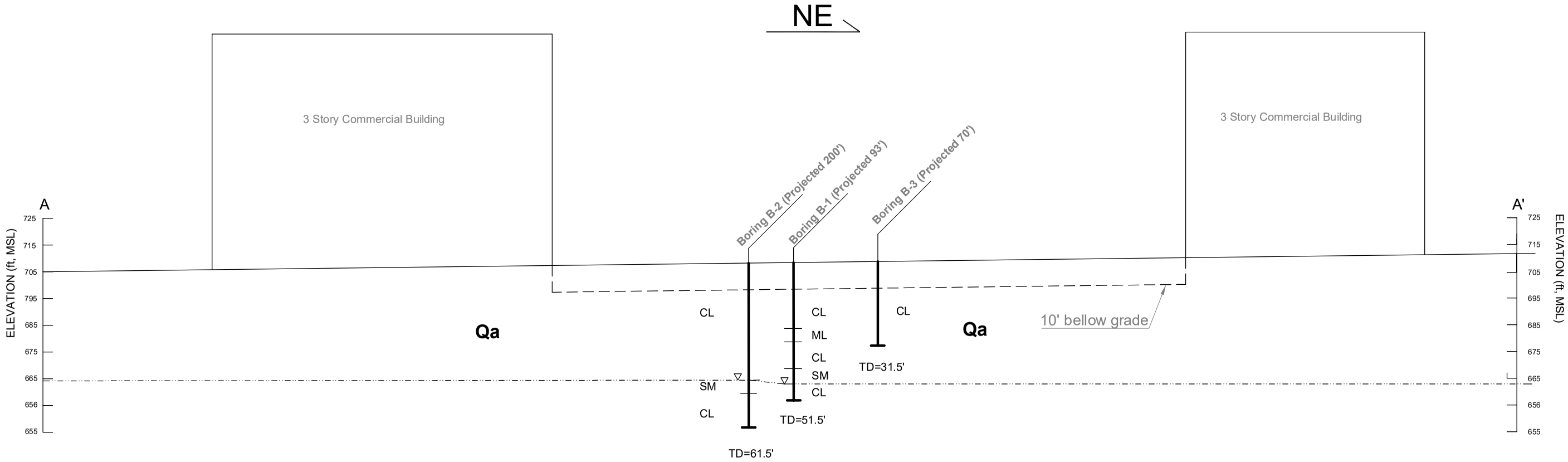
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REFERENCE: GRAPEVINE DEVELOPMENT (2018)

SITE PLAN AND BORING LOCATION MAP

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PROJECT NO.	REPORT DATE	FIGURE 2
180534.1	September 2018	



LEGEND

———	EXISTING GRADE
B-1 / TD=51.5'	APPROXIMATE LOCATION OF EXPLORATORY BORING OR CPT TD = TERMINATION DEPTH IN FEET
– – – – – – – – – –	LIMITS OF PROPOSED EXCAVATION
– · – · – ▽ – · – · –	APPROXIMATE GROUNDWATER ELEVATION

Qa	ALLUVIAL FAN DEPOSITS

SCALE IN FEET

0 40 80

NOTE: All dimensions, locations, and directions are approximate.

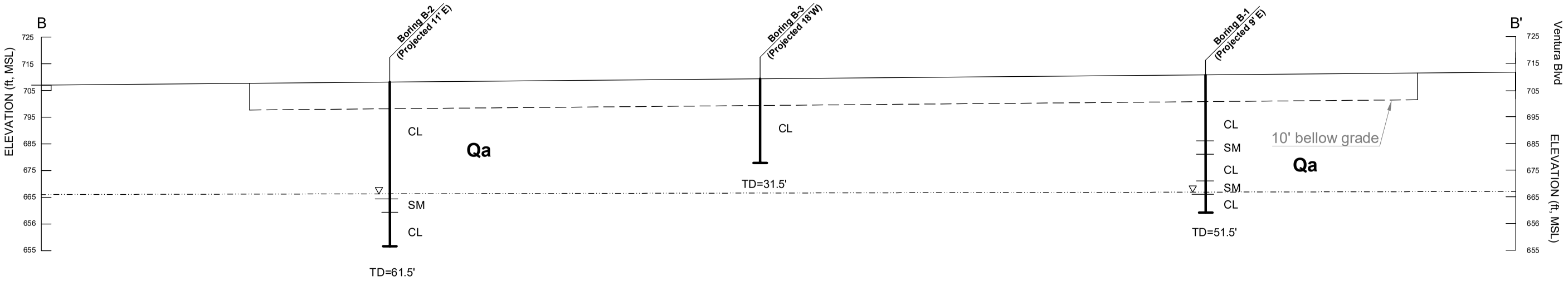
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GEOLOGIC CROSS SECTION A-A'

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PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	FIGURE 2A

SW



LEGEND

EXISTING GRADE

B-1
TD=51.5'
APPROXIMATE LOCATION OF EXPLORATORY BORING OR CPT
TD = TERMINATION DEPTH IN FEET

LIMITS OF PROPOSED EXCAVATION

APPROXIMATE GROUNDWATER ELEVATION

Qa — ALLUVIAL FAN DEPOSITS

SCALE IN FEET
0 40 80

NOTE: All dimensions, locations, and directions are approximate.

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GEOLOGIC CROSS SECTION B-B'

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PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	FIGURE 2B



LEGEND

af
Qg Qa

SURFICIAL SEDIMENTS
Undissected alluvial deposits

af *Artificial fill, and cut and fill areas*
Qg *Gravel and sand of major streams, and alluvial ian detritus from San Gabriel Mountains, grades southward into alluvium (**Qa**) as sizes of clasts decrease*
Qa *Alluvial gravel, sand and silt of valleys and floodplains*

N

SCALE IN MILE
0 0.3 0.6

REFERENCE: DIBBLEE AND EHRENSPECK (1999)

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REGIONAL GEOLOGIC MAP

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PROJECT NO.	REPORT DATE	FIGURE 3
180534.1	September 2018	



FAULT ALONG WHICH HISTORIC DISPLACEMENT HAS OCCURRED

HOLOCENE FAULT DISPLACEMENT

LATE QUATERNARY FAULT DISPLACEMENT

QUATERNARY FAULT DISPLACEMENT

PRE-QUATERNARY FAULT DISPLACEMENT

N

SCALE IN MILES

0 11 22

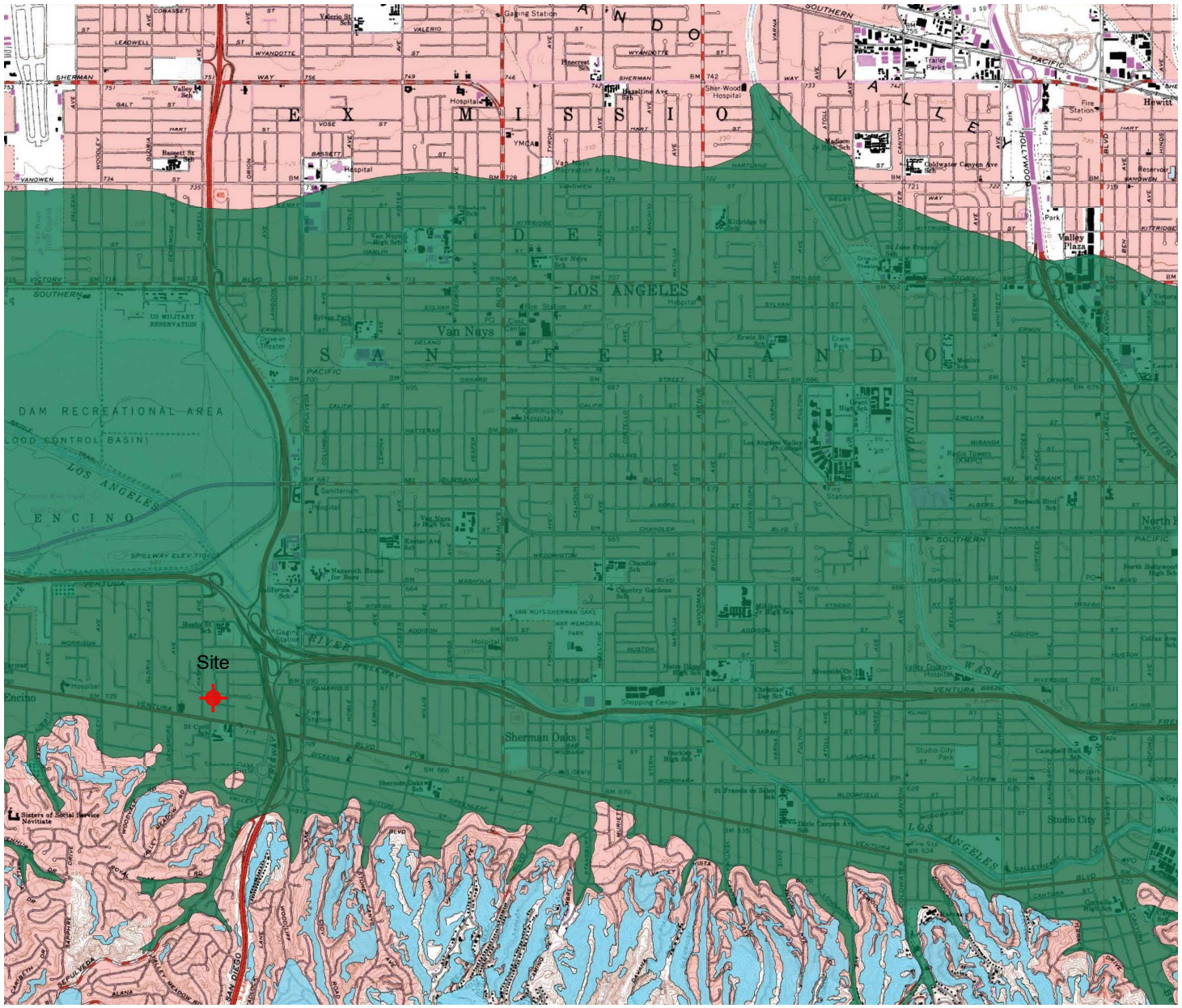
REFERENCE: JENNINGS AND BRYANT (2010)

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REGIONAL FAULT LOCATION MAP

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PROJECT NO.	REPORT DATE	FIGURE 4
180534.1	September 2018	



ALQUIST-PRIOLO EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments that connect encircled turning points; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

1906 C

N

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Overlapping Liquefaction and Earthquake-Induced Landslide Zones
Areas that lie within zones of required investigation for both liquefaction and earthquake-induced landslides.

0 0.5 1

SCALE IN MILES

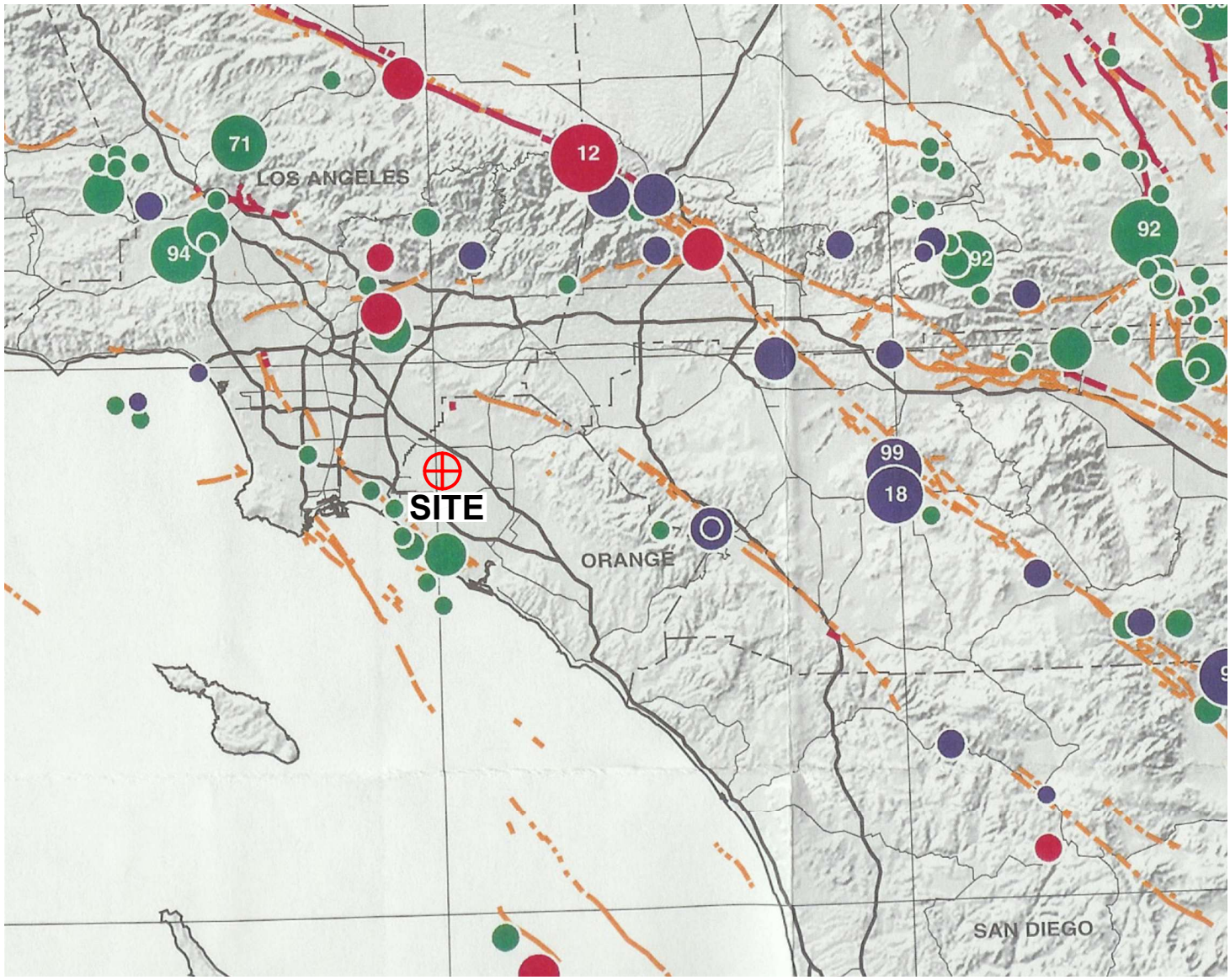
REFERENCE: CALIFORNIA GEOLOGICAL SURVEY (1999)

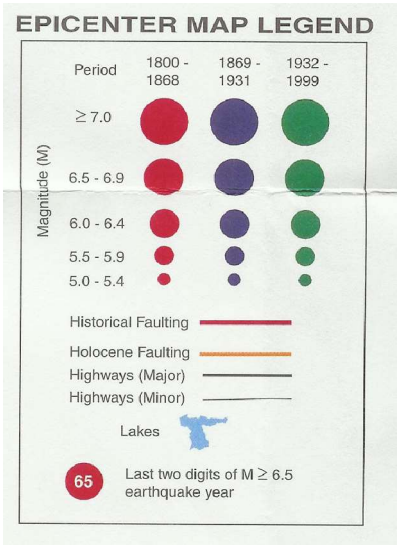
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EARTHQUAKE FAULT ZONES AND SEISMIC HAZARD ZONES MAP

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PROJECT NO.	REPORT DATE	FIGURE
180543.1	September 2018	5





N

REFERENCE: TOPPOZADA ET AL. (2000)

SCALE IN MILES
0 20 40

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HISTORICAL SEISMICITY, 1800-1999

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PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	FIGURE 6

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Appendix A
Field Exploration

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Appendix A
Field Exploration

General

The subsurface exploration program for the proposed project consisted of advancing and logging of three eight-inch diameter exploratory borings to depths of approximately 51 ½ , 61 ½ and 31 ½ feet below existing grade and advancing two Cone Penetration Tests (CPTs) to depths of approximately 80 feet.

The exploratory borings were advanced with a rubber truck mounted CME 75 drill. Drilling was performed by 2R Drilling. CPTs were performed by Middle Earth of Orange, California.

Drilling and Sampling

The Logs of Borings are presented as Figures A-2 through A-4. An explanation of these logs is presented as Figure A-1. The Logs of Borings describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The logs also show the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by a Twining engineer/geologist using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

A California modified sampler was used to obtain drive samples of the soil encountered. This sampler consists of an approximately 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft that is driven a total of 18-inches into the soil at the bottom of the boring. The soil was retained in brass rings for laboratory testing. Additional soil from each drive remaining in the cutting shoe was usually discarded after visually classifying the soil. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs.

Disturbed samples were obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft that is advanced into the soil at the bottom of the drilled hole a total of 18 inches. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs. Soil samples obtained by the SPT were retained in air-tight plastic bags.

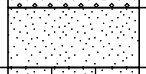
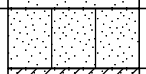
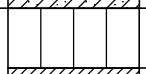
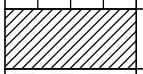
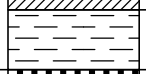
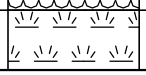
Both the California modified and the SPT sampler were driven by an automatic-trip hammer weighing 140 pounds at a drop height of approximately 30 inches.

Upon completion of drilling, the borings were backfilled by the drilling subcontractor using soil derived from the cuttings. The surfaces were patched using cold-patch asphalt where pavement was present.

CPT Testing

The detailed report of Cone Penetration Tests (CPTs) is attached after the boring logs. Upon completion of CPT probing, the probed holes were backfilled using hydrated bentonite chips.

UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

COARSE-GRAINED SOILS

Relative Density	SPT (blows/ft)	Relative Density (%)
Very Loose	<4	0 - 15
Loose	4 - 10	15 - 35
Medium Dense	10 - 30	35 - 65
Dense	30 - 50	65 - 85
Very Dense	>50	85 - 100

FINE-GRAINED SOILS

Consistency	SPT (blows/ft)
Very Soft	<2
Soft	2 - 4
Medium Stiff	4 - 8
Stiff	8 - 15
Very Stiff	15 - 30
Hard	>30

NOTE: SPT blow counts based on 140 lb. hammer falling 30 inches

Sample Symbol	Sample Type	Description
	SPT	1.4 in I.D., 2.0 in. O.D. driven sampler
	California Modified	2.4 in. I.D., 3.0 in. O.D. driven sampler
	Bulk	Retrieved from soil cuttings
	Thin-Walled Tube	Pitcher or Shelby Tube

LABORATORY TESTING ABBREVIATIONS

ATT	Atterberg Limits
C	Consolidation
CORR	Corrosivity Series
DS	Direct Shear
EI	Expansion Index
GS	Grain Size Distribution
K	Permeability
MAX	Moisture/Density (Modified Proctor)
O	Organic Content
RV	Resistance Value
SE	Sand Equivalent
SG	Specific Gravity
TX	Triaxial Compression
UC	Unconfined Compression

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EXPLANATION FOR LOG OF BORINGS

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PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	A-1

STANDARD LOG EXPLANATION 180138.1 - SUNFLOWER APARTMENTS.GPJ TWINING LABS.GDT 4/6/18

DATE DRILLED	7/9/18	LOGGED BY	SZ	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	45
DRILLING METHOD	HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	712 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
						Corrosivity Max Density		CL	5 " Asphalt Concrete SANDY lean CLAY, softt, dark brown
707	5		3						
702	10		17	8.1	117.6				stiff
697	15		6			Wash #200, ATT			medium stiff
692	20		48	14.9	115.4				very stiff
687	25		20			Wash #200, ATT		SM	SILTY SAND, medium dense, light brown, moist
682	30		34	17.7	107.5			CL	SANDY lean CLAY, very stiff, dark brown
677	35								

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LOG OF BORING

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Sherman Oak, California

PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	A - 2

BORING LOG 180534 DKN VENTURA.GPJ TWINING LABS.GDT 9/5/18

DATE DRILLED	7/9/18	LOGGED BY	SZ	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	45
DRILLING METHOD	HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	712 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
			13			ATT		CL	SANDY lean CLAY, very stiff, dark brown *(continued)* stiff
672	40		23	26.9	97.8	Wash #200		SM	SILTY SAND,medium dense, dark brown, moist
667	45		26			Wash #200		CL	SANDY lean CLAY, very stiff, dark brown
662	50		35	22.2	107.0	Wash #200			
657	55								Total Depth = 51.5 feet Backfilled on 7/9/2018 Groundwater was encountered @45' Borehole backfilled at the completion of testing with cuttings. Surface patched with cold-patch asphalt
652	60								
647	65								
642	70								

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LOG OF BORING

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PROJECT NO.	REPORT DATE	
180534.1	September 2018	FIGURE A - 2

BORING LOG 180534 DKN VENTURA.GPJ TWINING LABS.GDT 9/5/18

DATE DRILLED	7/9/18	LOGGED BY	SZ	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	40
DRILLING METHOD	HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	708 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									6 " Asphalt Concrete
								CL	SANDY lean CLAY, medium stiff, dark brown
703	5		8	15.6	105.9	Consol, DS			
698	10		6						light brown
693	15		17	21.7	98.8	Consol, DS			stiff
688	20		24						very stiff
683	25		48	14.8	115.2				same
678	30		16						same
673	35								

BORING LOG 180534 DKN VENTURA.GPJ TWINING LABS.GDT 9/5/18

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LOG OF BORING

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Sherman Oak, California

PROJECT NO.	REPORT DATE	
180534.1	September 2018	FIGURE A - 3

DATE DRILLED	7/9/18	LOGGED BY	SZ	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	40
DRILLING METHOD	HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	708 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
			24	21.8	103.7			CL	SANDY lean CLAY, medium stiff, dark brown *(continued)*
									stiff
668	40		19	21.2	105.2			SM	SILTY SAND, medium dense, light brown, moist
663	45		35	15.3	114.3			CL	SANDY lean CLAY, very stiff, dark brown, moist
658	50		29						
653	55		70			Wash #200			hard
648	60		30			Wash #200			very stiff
643	65								Total Depth = 61.5 feet Backfilled on 7/9/2018 Groundwater was encountered @40' Borehole backfilled at the completion of testing with cuttings. Surface patched with cold-patch asphalt
638	70								

BORING LOG 180534 DKN VENTURA.GPJ TWINING LABS.GDT 9/5/18

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LOG OF BORING

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Sherman Oak, California

PROJECT NO.	REPORT DATE	
180534.1	September 2018	FIGURE A - 3

DATE DRILLED	7/9/18	LOGGED BY	SZ	**BORING NO.**	**B-3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	707 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
				20.8	117.4	EI, R-Value, Remold DS (90%)		CL	4 " Asphalt Concrete SANDY lean CLAY, soft to medium stiff, dark brown
702	5		4	15.3	109.3				
697	10		14	14.6	114.8	Consol			stiff
692	15		7	15.0	110.2				medium stiff
687	20		15	14.6	114.8	Consol			stiff
682	25		22						very stiff
677	30		32						same
672	35								Total Depth = 31.5 feet Backfilled on 7/9/2018 No groundwater was encountered. Borehole backfilled at the completion of testing with cuttings. Surface patched with cold-patch asphalt

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LOG OF BORING

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PROJECT NO.	REPORT DATE	
180534.1	September 2018	FIGURE A - 4

BORING LOG 180534 DKN VENTURA.GPJ TWINING LABS.GDT 9/5/18

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Appendix B
Laboratory Testing

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Appendix B
Laboratory Testing

Laboratory Moisture Content and Density Tests

The moisture content and dry densities of selected driven samples obtained from the exploratory borings were evaluated in general accordance with the latest version of ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Wash Sieve

The amount of fines passing the No. 200 sieve was evaluated by the wash sieve. The test procedure was in general accordance with ASTM D 1140. The results are presented in Table B-2.

Atterberg Limits

Liquid limit, plastic limit and plasticity index of the soil are evaluated. The test procedure was in general accordance with ASTM D 4318. The results are presented in Figures B-1.

Maximum Dry Density-Optimum Moisture Content

A Modified Proctor test was performed on near-surface soils to evaluate the maximum dry density and optimum water content for compaction. The test was performed in accordance with ASTM D 1557 Method A. The result is summarized below in Table B-1 and a copy of the curve is presented as Figure B-2.

Expansion Index Test

The expansion index was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The result of the Expansion Index test is presented on Table B-4.

Consolidation Test

Consolidation tests were performed on a selected driven soil sample by in general accordance with the latest version of ASTM D2435. The sample was inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test are presented in Figures B-3 through B-5.

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Direct Shear Tests

Direct shear tests were performed on selected remolded and relatively undisturbed soil samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the materials. The samples were inundated during shearing to represent adverse field conditions. The results are summarized in Table B-5 and are illustrated graphically in Figures B-3 and B-4.

Corrosivity

Soil pH and resistivity tests were performed by Anaheim Test Lab on a representative soil sample in general accordance with the latest version of California Test Method 643. The chloride content of the selected sample was evaluated in general accordance with the latest version of California Test Method 422. The sulfate content of the selected samples was evaluated in general accordance with the latest version of California Test Method 417. The test results are presented on Table B-6.

Resistance Value (R-Value)

R-value testing was performed on a select bulk sample of the near-surface soils encountered at the site. The test was performed in general accordance with ASTM D 2844. The results are summarized in Table B-7.

Table B-1
Moisture-Density Relationship Testing
ASTM D 1557 Method A

Boring No.	Depth (feet)	Maximum Dry Density (pcf)	Optimum Water Content (%)
B-1	1 – 5	114.0	12

Table B-2
No. 200 Wash Sieve Results

Boring No.	Depth (feet)	Percent Passing #200
B-1	40	41
B-1	50	58
B-1	15	53
B-1	25	24
B-2	55	52
B-1	45	51
B-2	60	65

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Table B-3
Atterberg Limits Results

Boring No.	Depth (feet)	Liquid Limit	Plastic Limit	Plastic Index	U.S.C.S. Classification
B-1	15	29	19	10	SANDY lean CLAY
B-1	25	NP	NP	NP	SILTY SAND
B-1	35	34	19	15	SANDY lean CLAY

Table B-4
Expansion Index Test Result

Boring No.	Depth (feet)	Expansion Index
B-2	1 – 5	3

Table B-5
Direct Shear Tests

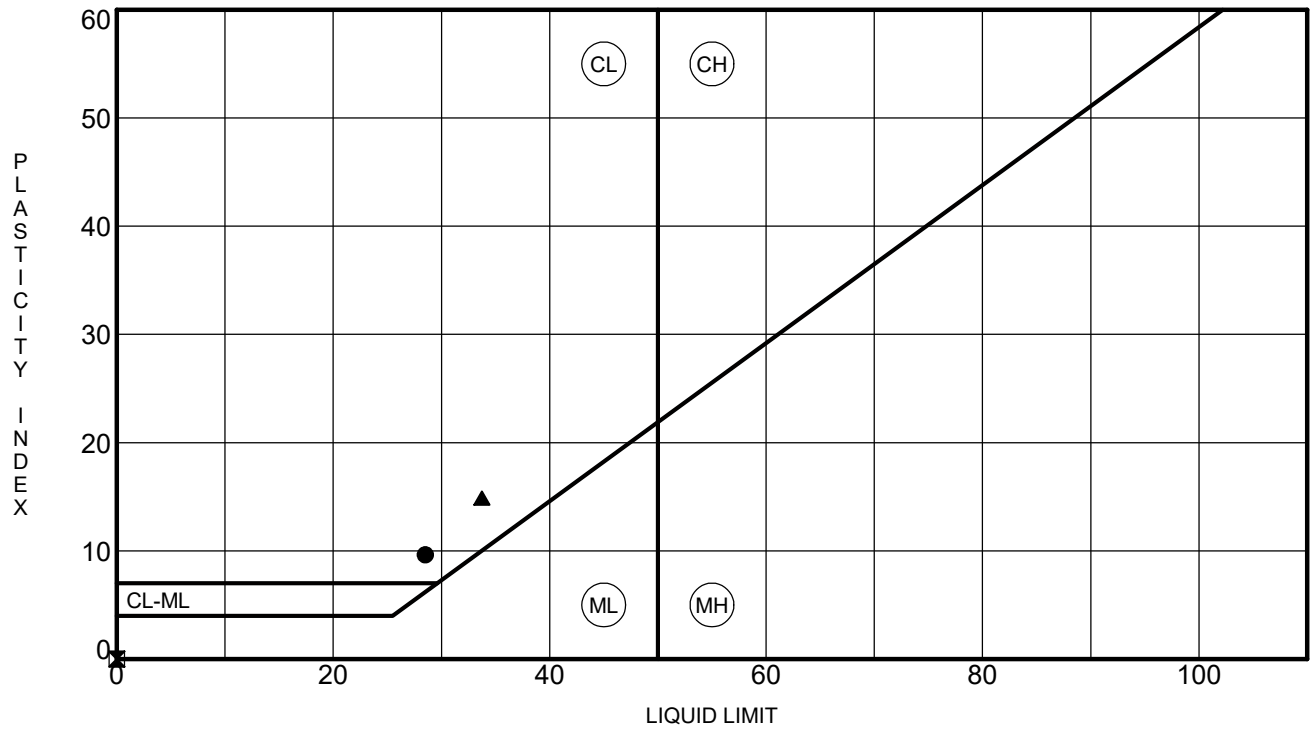
Boring No.	Depth (feet)	Remolded	Peak		Ultimate	
			C (psf)	ϕ (deg)	C (psf)	ϕ (deg)
B-2	5	No	360	25	240	27
B-3	0-5’	Yes	300	26	132	28
B-2	15	No	264	27	84	29

Table B-6
Soil Corrosivity Test Results

Boring No.	Depth (feet)	pH	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-1	1-5	7.1	135	34	2,000

Table B-7
Resistance Value (R-Value)

Boring No.	Depth (feet)	R-Value
B-2	0 – 5	14



	Sample Location	LL	PL	PI	U.S.C.S. Classification
●	**B-1 at 15 ft**	**29**	**19**	**10**	**SANDY lean CLAY**
⧓	**B-1 at 25 ft**	**NP**	**NP**	**NP**	**SILTY SAND**
▲	**B-1 at 35 ft**	**34**	**19**	**15**	**SANDY lean CLAY**

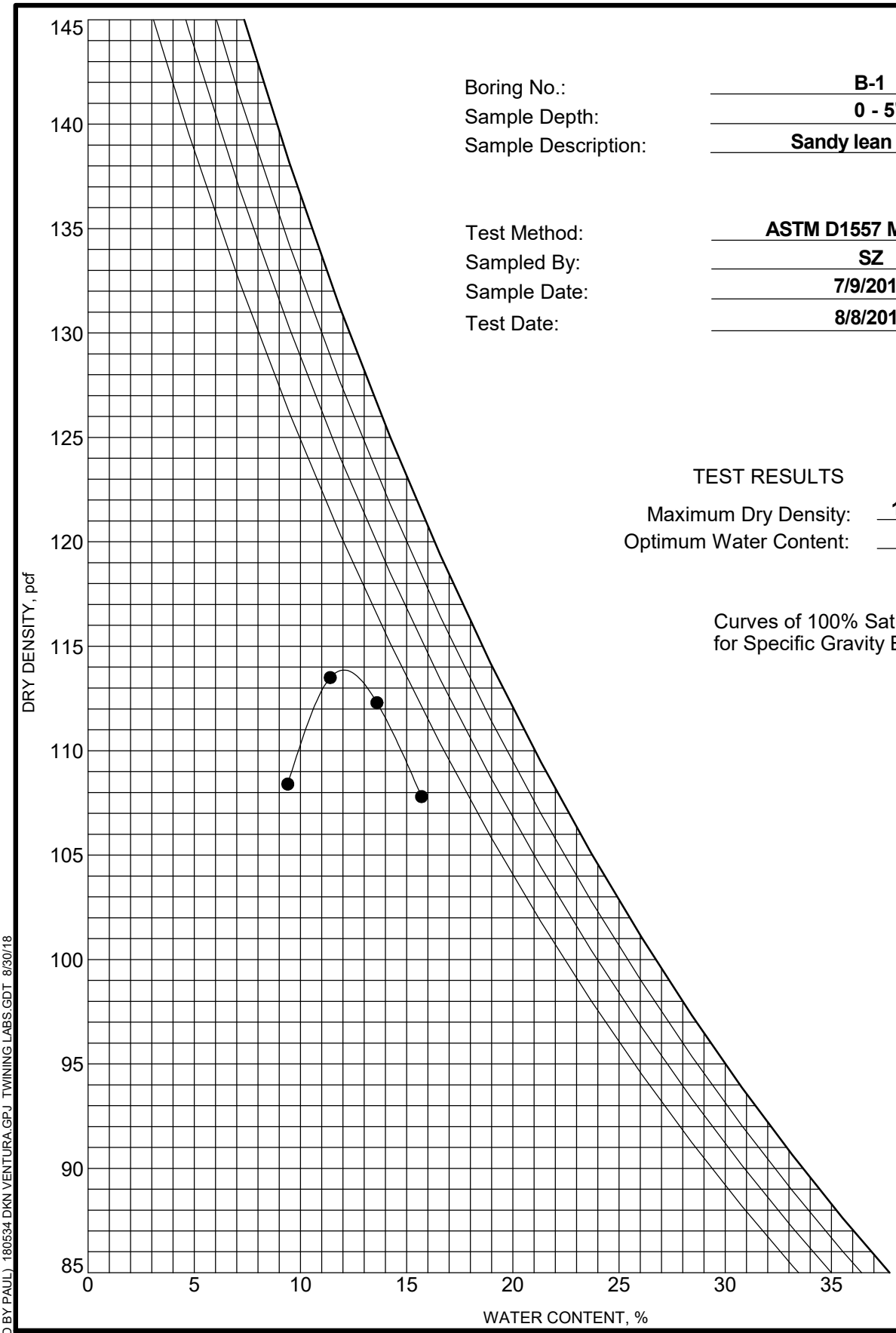
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ATTERBERG LIMITS

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PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	B-1

ATTERBERG LIMITS 180534 DKN VENTURA.GPJ TWINING LABS.GDT 8/30/18



Boring No.:	**B-1**
Sample Depth:	**0 - 5'**
Sample Description:	**Sandy lean CLAY**
Test Method:	**ASTM D1557 Method A**
Sampled By:	**SZ**
Sample Date:	**7/9/2018**
Test Date:	**8/8/2018**

TEST RESULTS

Maximum Dry Density:	**114.0** pcf
Optimum Water Content:	**12.0** %

Curves of 100% Saturation for Specific Gravity Equal to:

2.80

2.70

2.60

2.50

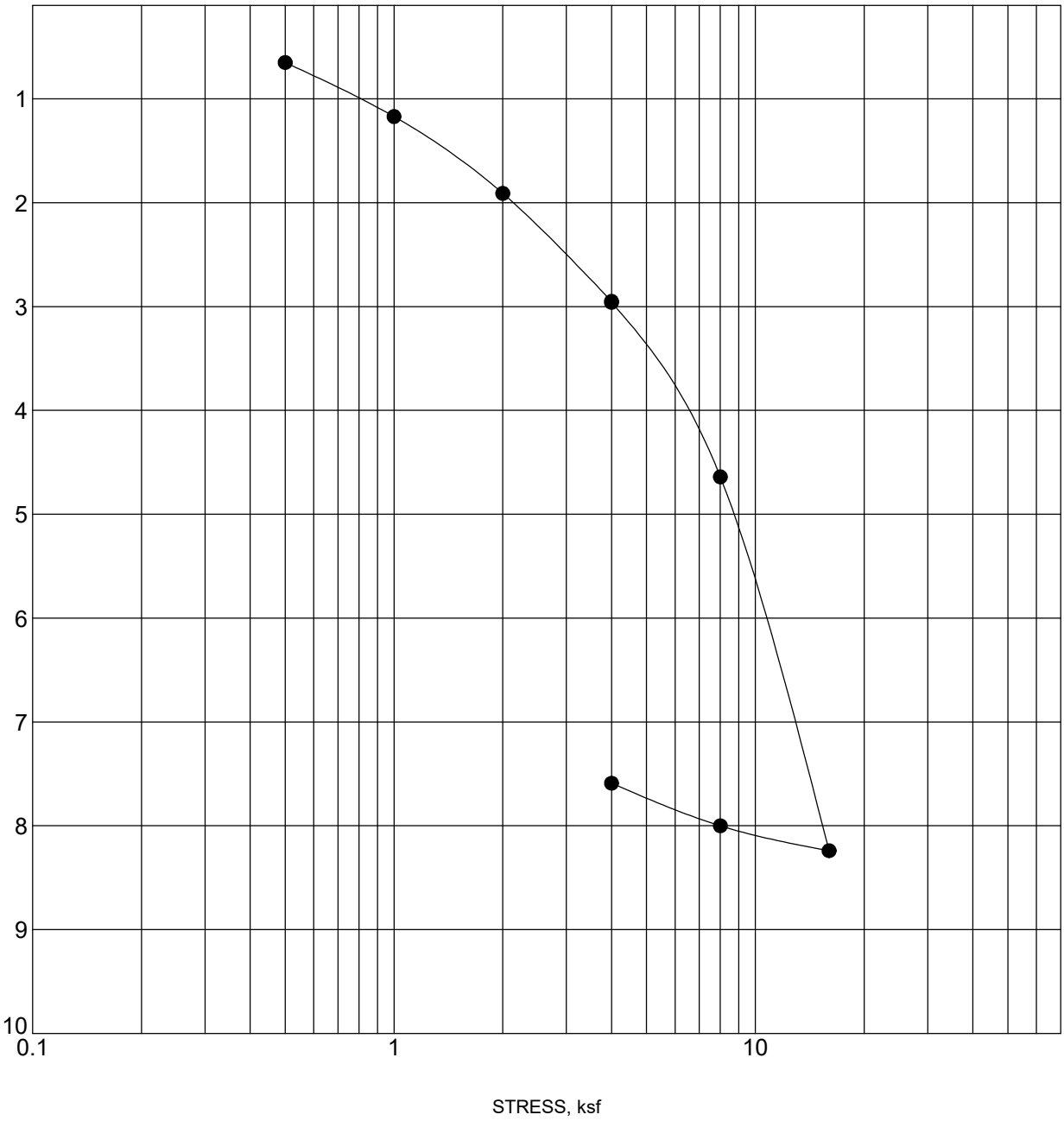
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MOISTURE-DENSITY RELATIONSHIP

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PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	FIGURE B-2

COMPACTION (MODIFIED BY PAUL) 180534 DKN VENTURA.GPJ TWINING LABS.GDT 8/30/18



Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
● **B-2 at 15 ft**	**SANDY lean CLAY**	**98.8**	**21.7**

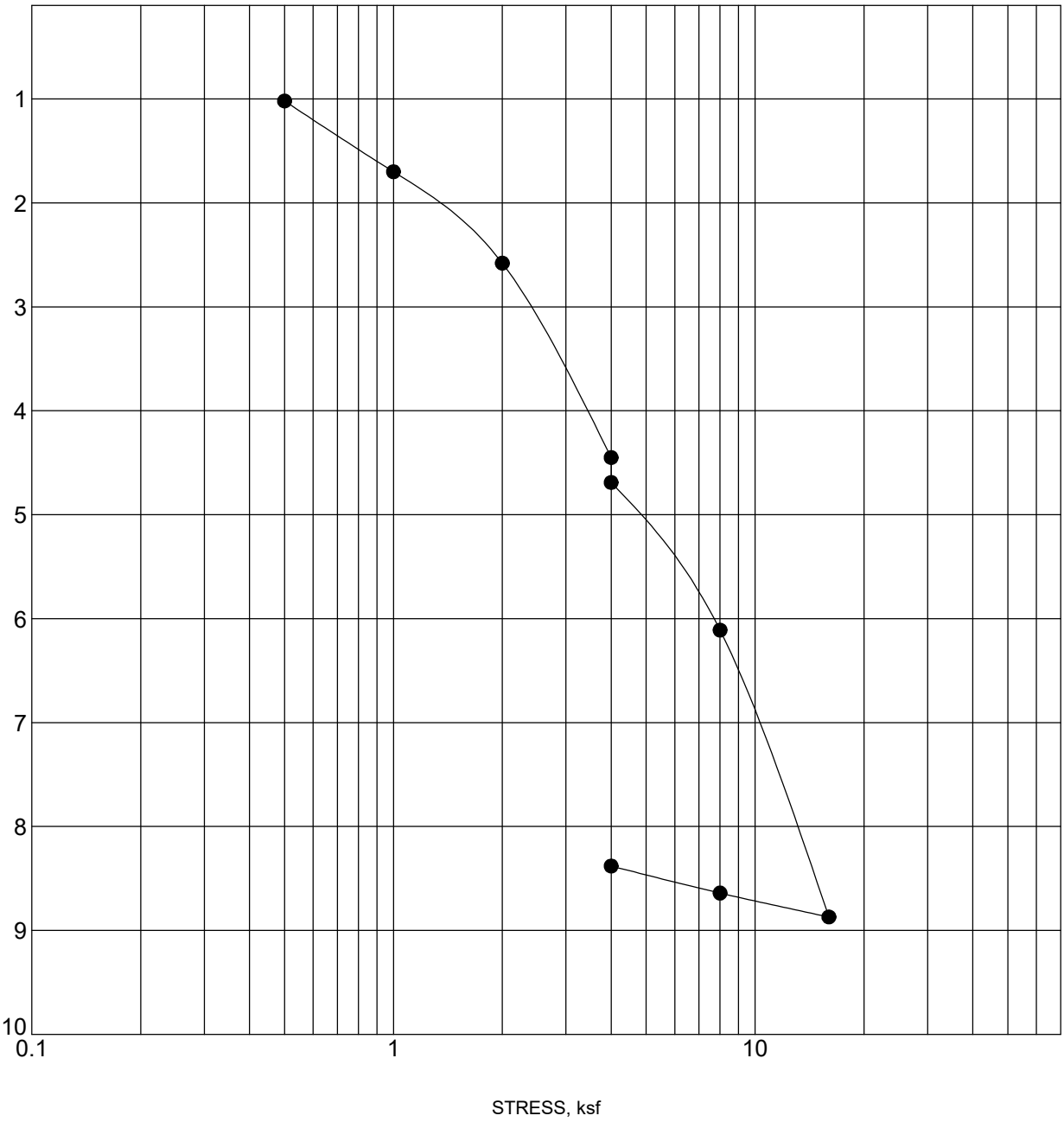
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CONSOLIDATION TEST

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Sherman Oak, California

PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	FIGURE B-3

CONSOL STRAIN 180534 DKN VENTURA.GPJ TWINING LABS.GDT 8/30/18



Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
● **B-2 at 5 ft**	**SANDY lean CLAY**	**105.9**	**15.6**

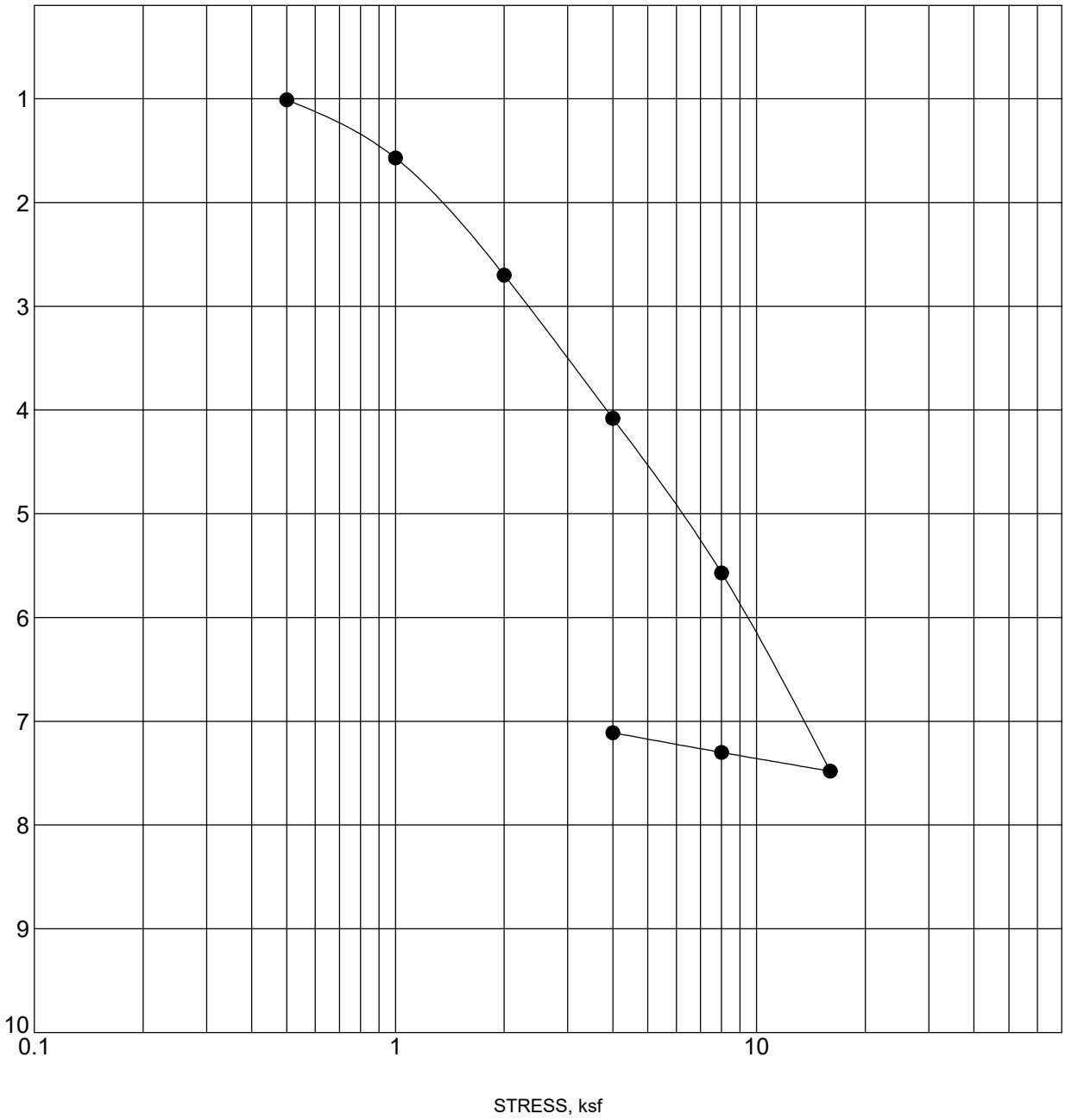
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CONSOLIDATION TEST

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Sherman Oak, California

PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	FIGURE B-4

CONSOL STRAIN 180534 DKN VENTURA.GPJ TWINING LABS.GDT 8/30/18



Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
● **B-3 at 20 ft**	**SANDY lean CLAY**	**114.8**	**14.6**

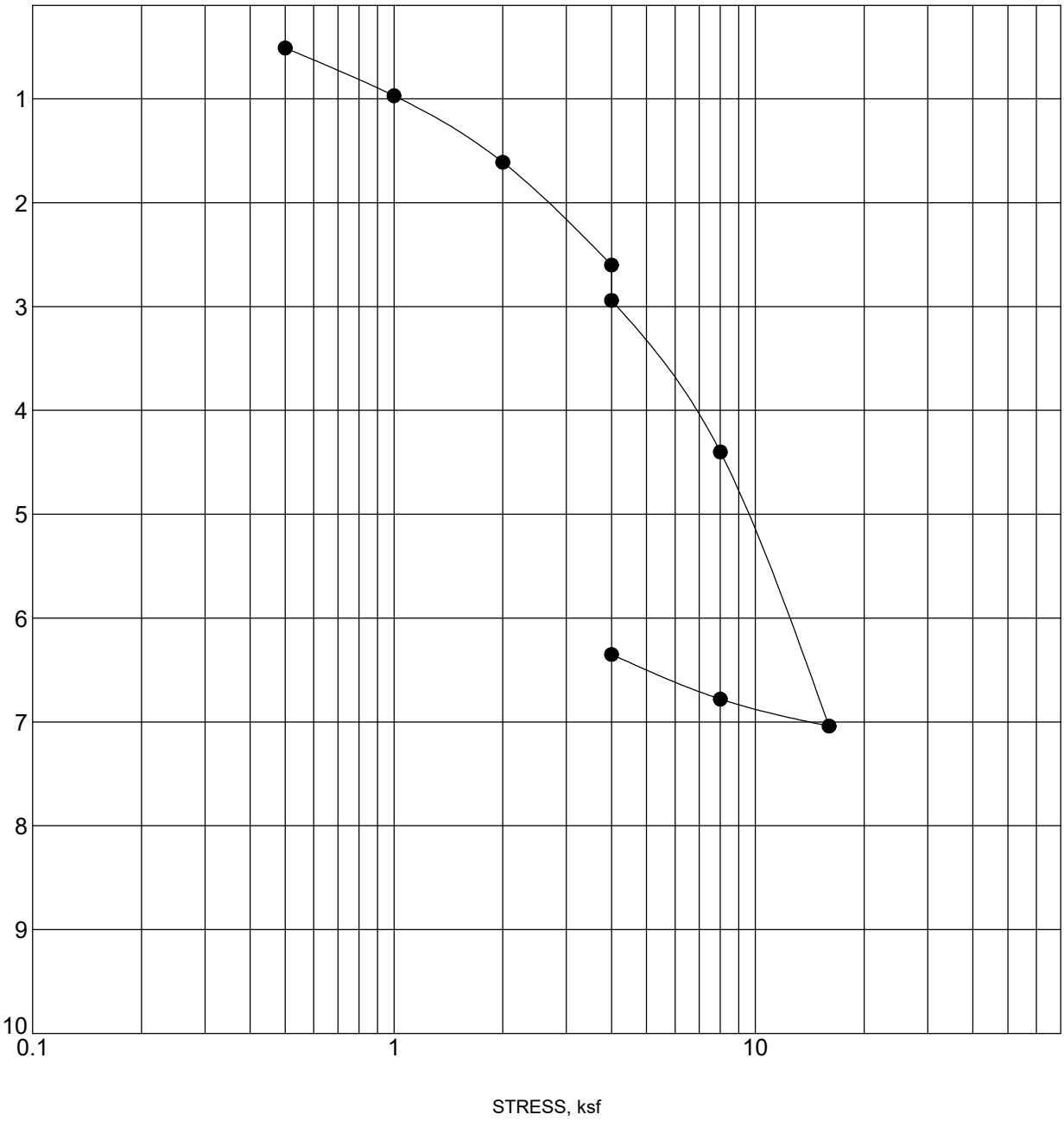
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CONSOLIDATION TEST

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15485 &15489 Ventura Blvd
Sherman Oak, California

PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	FIGURE B-5

CONSOL STRAIN 180534 DKN VENTURA.GPJ TWINING LABS.GDT 8/30/18



Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
● **B-3 at 10 ft**	**SANDY lean CLAY**	**109.3**	**15.3**

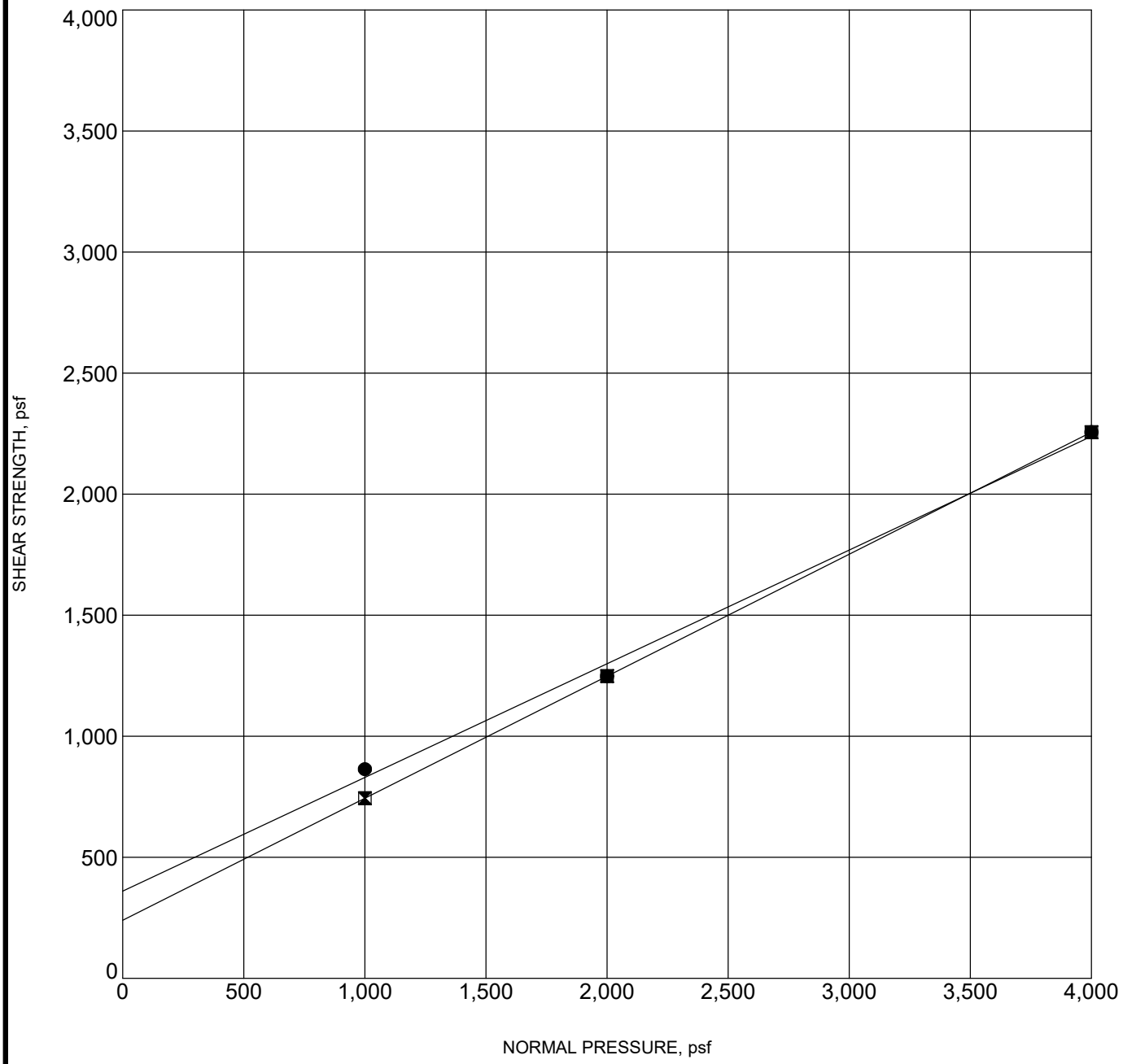
TWINING

CONSOLIDATION TEST

DKN, Ventura Blvd AC Hotel
15485 &15489 Ventura Blvd
Sherman Oak, California

PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	FIGURE B-6

CONSOL STRAIN 180534 DKN VENTURA.GPJ TWINING LABS.GDT 8/30/18



Boring No.:	B-2
Sample Depth (ft):	5
Sample Description:	SANDY lean CLAY
Strain Rate (in./min):	
Dry Density (pcf):	105.9

Shear Strength Parameters	**Peak**	**Ultimate**
Cohesion, C (psf):	360	240
Friction Angle, Ø (deg):	25	27

Initial Moisture (%):	15.6
Final Moisture (%):	19.6

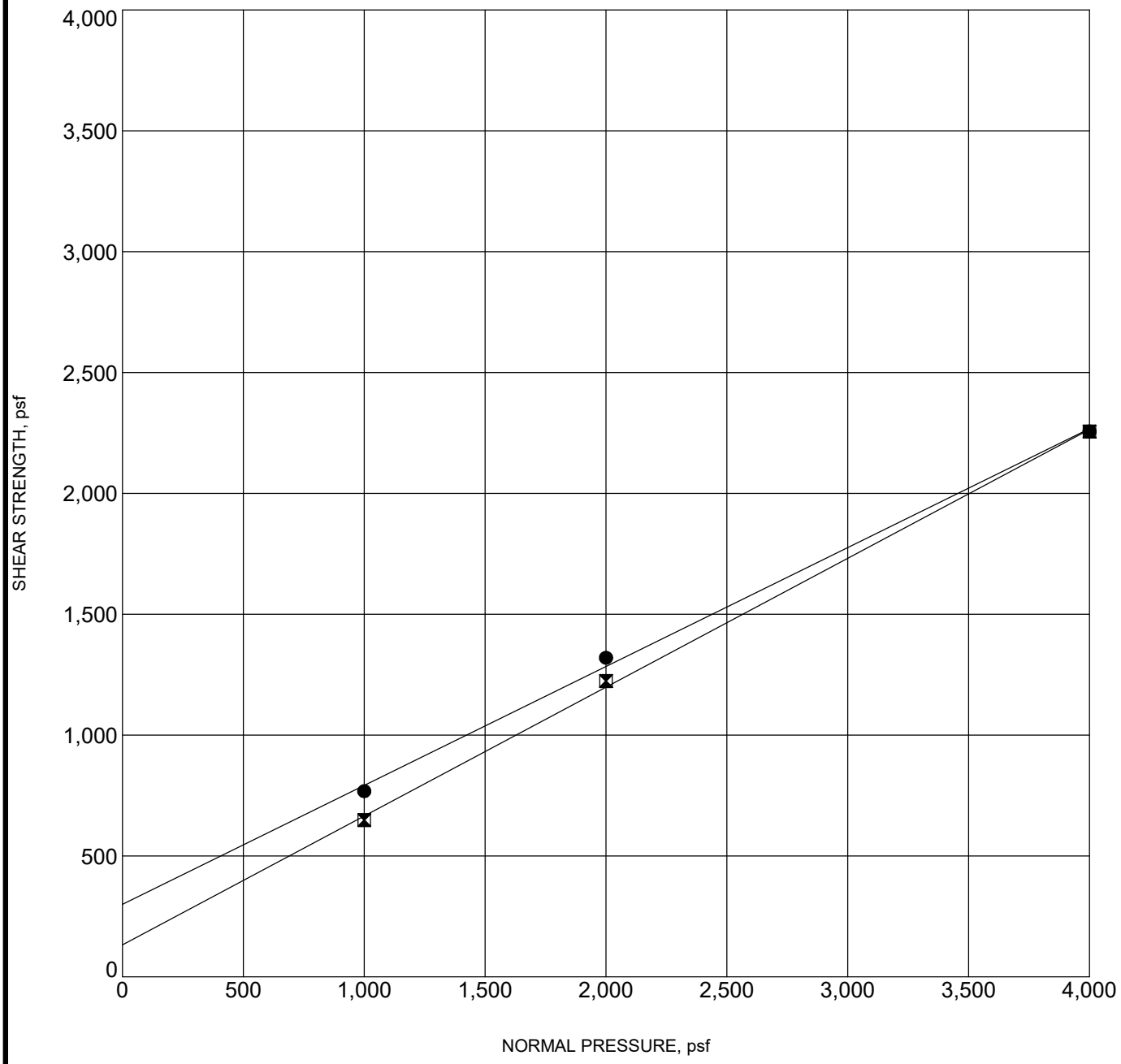
TWINING

DIRECT SHEAR TEST

DKN, Ventura Blvd AC Hotel
15485 &15489 Ventura Blvd
Sherman Oak, California

PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	B-7

DIRECT SHEAR 180534 DKN VENTURA.GPJ TWINING LABS.GDT 8/30/18



Boring No.: B-3
Sample Depth (ft): 0-5'
Sample Description:
Strain Rate (in./min):
Dry Density (pcf): 117.4

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	300	132
Friction Angle, Ø (deg):	26	28

Initial Moisture (%):
Final Moisture (%): 20.8

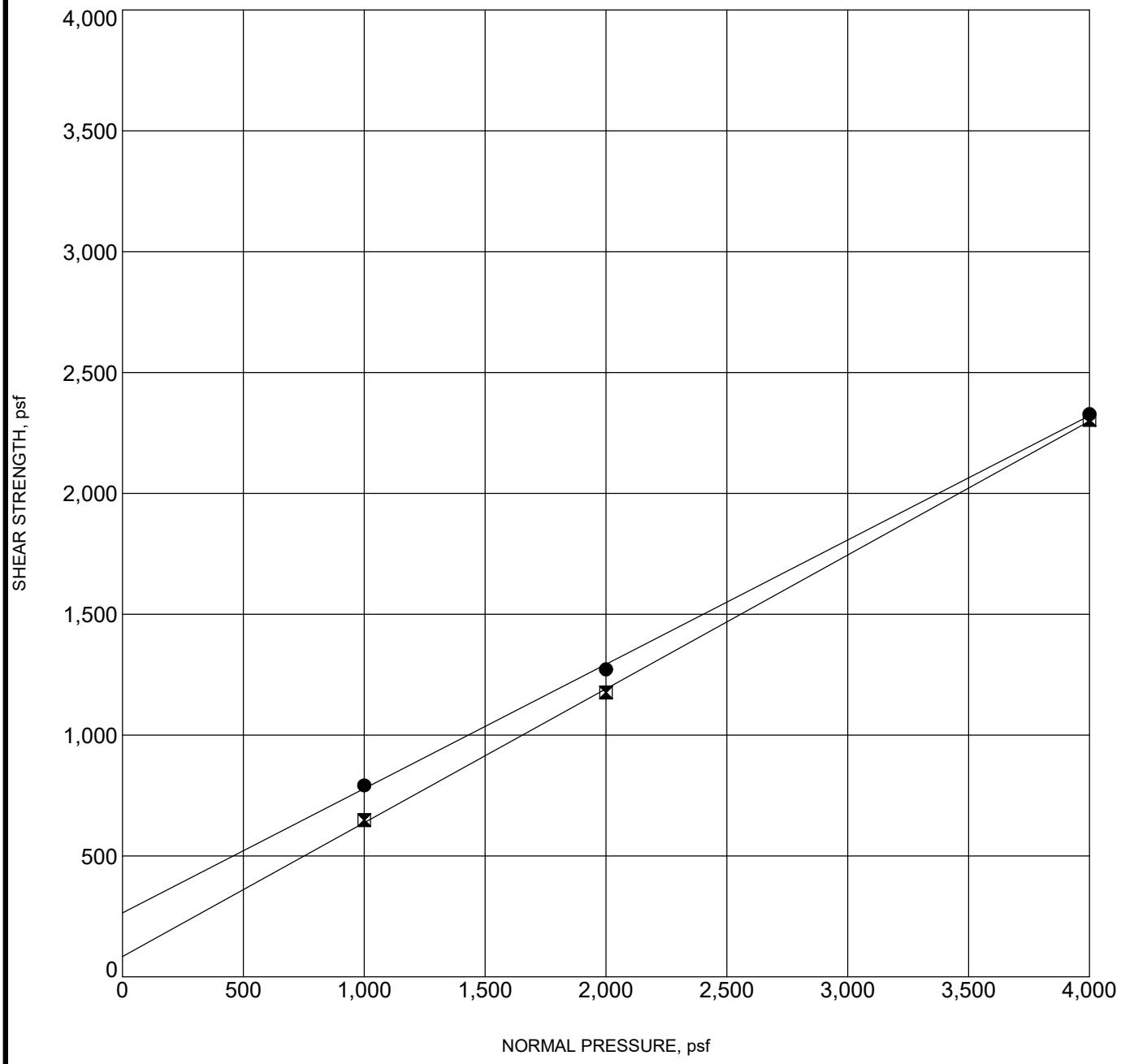
TWINING

DIRECT SHEAR TEST

DKN, Ventura Blvd AC Hotel
15485 &15489 Ventura Blvd
Sherman Oak, California

PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	B-8

DIRECT SHEAR 180534 DKN VENTURA.GPJ TWINING LABS.GDT 8/30/18



Boring No.:	B-2
Sample Depth (ft):	15
Sample Description:	SANDY lean CLAY
Strain Rate (in./min):	
Dry Density (pcf):	98.8

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	264	84
Friction Angle, Ø (deg):	27	29

Initial Moisture (%): 21.7
Final Moisture (%): 19.6

TWINING

DIRECT SHEAR TEST

DKN, Ventura Blvd AC Hotel
15485 &15489 Ventura Blvd
Sherman Oak, California

PROJECT NO.	REPORT DATE	FIGURE
180534.1	September 2018	B-9

DIRECT SHEAR 180534 DKN VENTURA.GPJ TWINING LABS.GDT 8/30/18

18011 Sky Park Circle,
Suite J
Irvine CA 92614

Tel 949.426.3355
Fax 949.553.0371

TWINING CONSULTING

Appendix C
Liquefaction and Seismic Settlement Analysis

18011 Sky Park Circle,
Suite J
Irvine CA 92614

Tel 949.426.3355
Fax 949.553.0371

TWINING CONSULTING

Appendix C
Liquefaction and Seismic Settlement Analysis

Our liquefaction analysis is performed in accordance with the Special Publication 117A; Guidelines for Evaluating and Mitigating Seismic Hazards in California (2008). The field SPT data and laboratory testing results obtained from exploration boring B-1 were used to evaluate the liquefaction and seismic settlement potential. The analysis was performed using a computer software, LiquefyPro, developed by Civil Tech, based on the following seismic parameters:

Table D-1 – Seismic Parameters for Liquefaction Analysis

Earthquake Magnitude M_w	Peak Ground Acceleration (g)	Groundwater Depth (ft)
7	0.762	10

Based on the results of our analyses, the total seismic settlement is estimated to be up to 1-inch with a differential seismic settlement of ½ inch over a 50ft. span.

CORRECTED SPT BLOW COUNTS AND ESTIMATED ES (BORING B-1)

AC HOTEL - VENTURA
LOS ANGELES, CA

			Blow Counts (Blows/Foot)		C_N								NC Sand
Depth (ft)	Rod Length (ft)	Sampler		σ_v (psf)	Seed and Idriss (1982)	C_E	C_B	C_S	C_R	N_{60}	$(N_1)_{60}$	$N_{eqiv.}$	Es
5	8	SPT	3	600	1.48	1.15	1.05	1.2	0.7500	4	3	4	191400
10	13	MC	17	1200	1.25	1.15	1.05	0.6	0.8500	12	11	11	273200
15	18	SPT	6	1800	1.07	1.15	1.05	1.2	0.8500	8	7	7	232800
20	23	MC	48	2400	0.94	1.15	1.05	0.6	0.9500	35	35	30	497800
25	28	SPT	20	3000	0.84	1.15	1.05	1.2	0.9500	28	28	24	426000
30	33	MC	34	3600	0.76	1.15	1.05	0.6	1.0000	25	26	21	396300
35	38	SPT	13	4200	0.69	1.15	1.05	1.2	1.0000	18	19	16	329400
40	43	MC	23	4800	0.63	1.15	1.05	0.6	1.0000	17	17	14	316600
45	48	SPT	26	5400	0.59	1.15	1.05	1.2	1.0000	36	38	31	508800
50	53	MC	35	6000	0.55	1.15	1.05	0.6	1.0000	25	27	25	403600

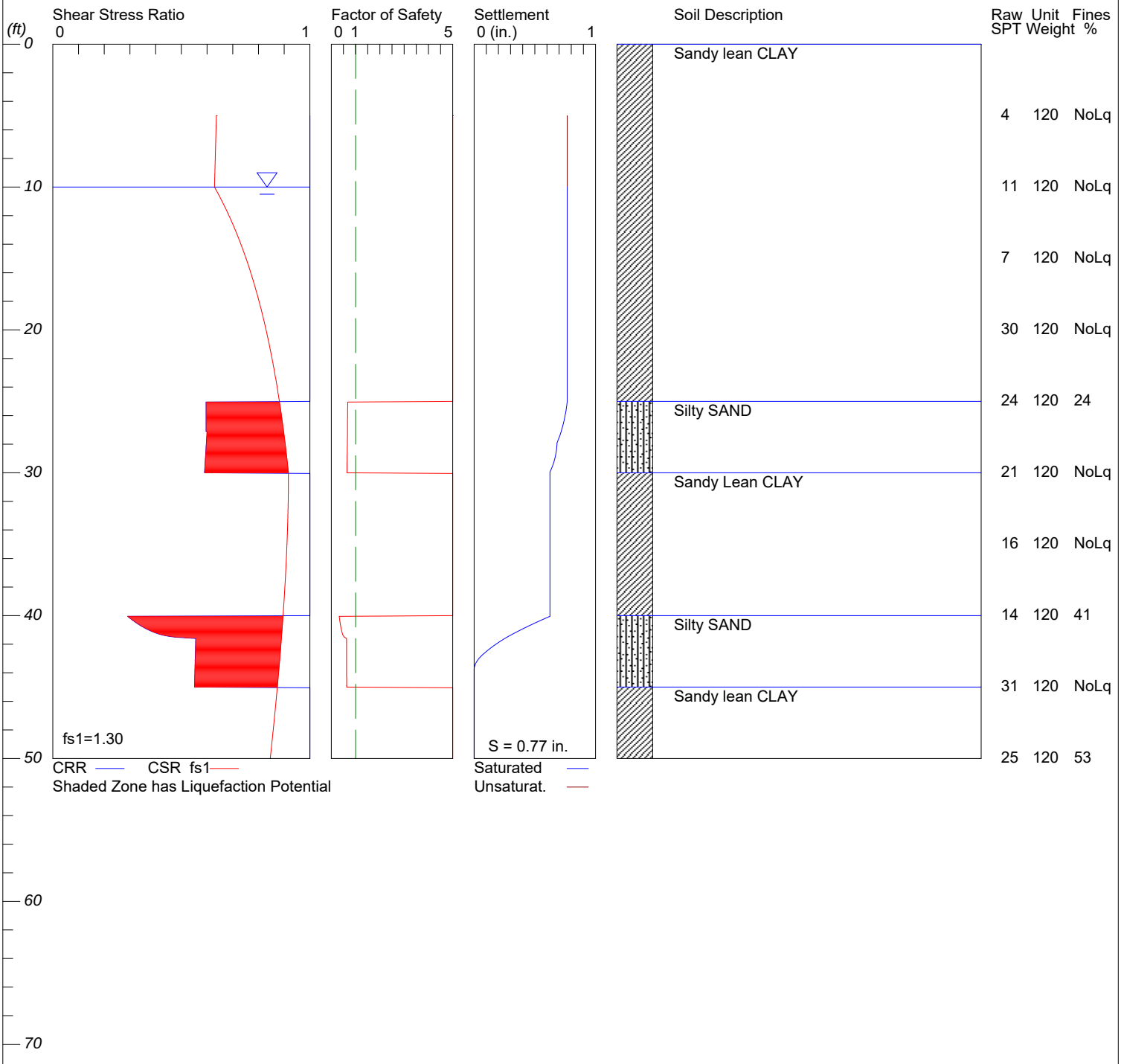
$N_{eqiv.}$ = Equivalent SPT blow counts converted from MC sample using Factor of 0.6
C_N = correction for over-burden
C_B = correction for borehole diameter
C_S = correction for sample type (assume 0.6 for modified California sampler)
C_R = correction for rod length (assuming rod sticks out 2 feet from the ground)
N_{60} = $N_{eqiv.}$ SPT normalized to 60% energy

LIQUEFACTION ANALYSIS

AC Hotel

Hole No.=B-1 Water Depth=10 ft Surface Elev.=707

Magnitude=7
Acceleration=0.762g



LiquefyPro CivilTech Software USA www.civiltech.com

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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: T:\Projects\2018 TLSC Projects\180534.1 - TC - DKN - Ventura Boulevard AC Hotel\Liquefaction analysis\DKN Ventura Liqufaction Analysis-SPT1.liq
Title: AC Hotel
Subtitle: SPT-1

Surface Elev.=707
Hole No.=B-1
Depth of Hole= 50.00 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration= 0.76 g
Earthquake Magnitude= 7.00

Input Data:
Surface Elev.=707
Hole No.=B-1
Depth of Hole=50.00 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration=0.76 g
Earthquake Magnitude=7.00
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: Post Liquefaction
5. Settlement Calculation in: Liq. zone only
6. Hammer Energy Ratio, Ce = 1.30
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1.1
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
5.00	4.00	120.00	NoLiq
10.00	11.00	120.00	NoLiq
15.00	7.00	120.00	NoLiq
20.00	30.00	120.00	NoLiq
25.00	24.00	120.00	24.00
30.00	21.00	120.00	NoLiq
35.00	16.00	120.00	NoLiq
40.00	14.00	120.00	41.00
45.00	31.00	120.00	NoLiq
50.00	25.00	120.00	53.00

Output Results:

Settlement of Saturated Sands=0.77 in.
Settlement of Unsaturated Sands=0.00 in.
Total Settlement of Saturated and Unsaturated Sands=0.77 in.
Differential Settlement=0.383 to 0.506 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
5.00	2.00	0.64	5.00	0.77	0.00	0.77
6.00	2.00	0.63	5.00	0.77	0.00	0.77
7.00	2.00	0.63	5.00	0.77	0.00	0.77
8.00	2.00	0.63	5.00	0.77	0.00	0.77
9.00	2.00	0.63	5.00	0.77	0.00	0.77
10.00	2.00	0.63	5.00	0.77	0.00	0.77
11.00	2.00	0.66	5.00	0.77	0.00	0.77
12.00	2.00	0.69	5.00	0.77	0.00	0.77
13.00	2.00	0.71	5.00	0.77	0.00	0.77
14.00	2.00	0.73	5.00	0.77	0.00	0.77
15.00	2.00	0.75	5.00	0.77	0.00	0.77
16.00	2.00	0.77	5.00	0.77	0.00	0.77
17.00	2.00	0.79	5.00	0.77	0.00	0.77
18.00	2.00	0.80	5.00	0.77	0.00	0.77
19.00	2.00	0.82	5.00	0.77	0.00	0.77
20.00	2.00	0.83	5.00	0.77	0.00	0.77
21.00	2.00	0.84	5.00	0.77	0.00	0.77
22.00	2.00	0.85	5.00	0.77	0.00	0.77
23.00	2.00	0.86	5.00	0.77	0.00	0.77
24.00	2.00	0.87	5.00	0.77	0.00	0.77
25.00	2.00	0.88	5.00	0.77	0.00	0.77

26.00	0.60	0.89	0.67*	0.75	0.00	0.75
27.00	0.60	0.90	0.66*	0.72	0.00	0.72
28.00	0.60	0.90	0.66*	0.68	0.00	0.68
29.00	0.59	0.91	0.65*	0.66	0.00	0.66
30.00	0.59	0.92	0.64*	0.62	0.00	0.62
31.00	2.00	0.92	5.00	0.62	0.00	0.62
32.00	2.00	0.92	5.00	0.62	0.00	0.62
33.00	2.00	0.91	5.00	0.62	0.00	0.62
34.00	2.00	0.91	5.00	0.62	0.00	0.62
35.00	2.00	0.91	5.00	0.62	0.00	0.62
36.00	2.00	0.91	5.00	0.62	0.00	0.62
37.00	2.00	0.91	5.00	0.62	0.00	0.62
38.00	2.00	0.90	5.00	0.62	0.00	0.62
39.00	2.00	0.90	5.00	0.62	0.00	0.62
40.00	2.00	0.90	5.00	0.62	0.00	0.62
41.00	0.37	0.89	0.42*	0.38	0.00	0.38
42.00	0.56	0.89	0.63*	0.18	0.00	0.18
43.00	0.55	0.88	0.63*	0.04	0.00	0.04
44.00	0.55	0.88	0.63*	0.00	0.00	0.00
45.00	0.55	0.87	0.63*	0.00	0.00	0.00
46.00	2.00	0.87	5.00	0.00	0.00	0.00
47.00	2.00	0.86	5.00	0.00	0.00	0.00
48.00	2.00	0.86	5.00	0.00	0.00	0.00
49.00	2.00	0.85	5.00	0.00	0.00	0.00
50.00	2.00	0.85	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

CORRECTED SPT BLOW COUNTS AND ESTIMATED ES (BORING B-2)

AC HOTEL - VENTURA
LOS ANGELES, CA

			Blow Counts (Blows/Foot)		C_N								NC Sand
Depth (ft)	Rod Length (ft)	Sampler		σ_v (psf)	Seed and Idriss (1982)	C_E	C_B	C_S	C_R	N_{60}	$(N_1)_{60}$	$N_{eqiv.}$	Es
5	8	MC	8	650	1.46	1.15	1.05	0.6	0.7500	6	5	5	208000
10	13	SPT	6	1300	1.21	0.92	1.05	1.2	0.8500	7	6	7	216200
15	18	MC	17	1950	1.04	0.92	1.05	0.6	0.8500	10	9	11	248500
20	23	SPT	24	2600	0.91	0.92	1.05	1.2	0.9500	26	26	29	415000
25	28	MC	48	3250	0.80	0.92	1.05	0.6	0.9500	28	28	30	428200
30	33	SPT	16	3900	0.72	0.92	1.05	1.2	1.0000	18	19	19	326600
35	38	MC	24	4550	0.66	0.92	1.05	0.6	1.0000	14	15	15	289100
40	43	SPT	19	5200	0.60	0.92	1.05	1.2	1.0000	21	22	23	359800
45	48	MC	35	5850	0.55	0.92	1.05	0.6	1.0000	20	21	22	352900
50	53	SPT	29	6500	0.52	0.92	1.05	1.2	1.0000	32	34	35	470200
55	58	MC	70	7150	0.48	0.92	1.05	0.6	1.0000	41	43	44	555700
60	63	SPT	30	7800	0.45	0.92	1.05	1.2	1.0000	33	35	36	481200

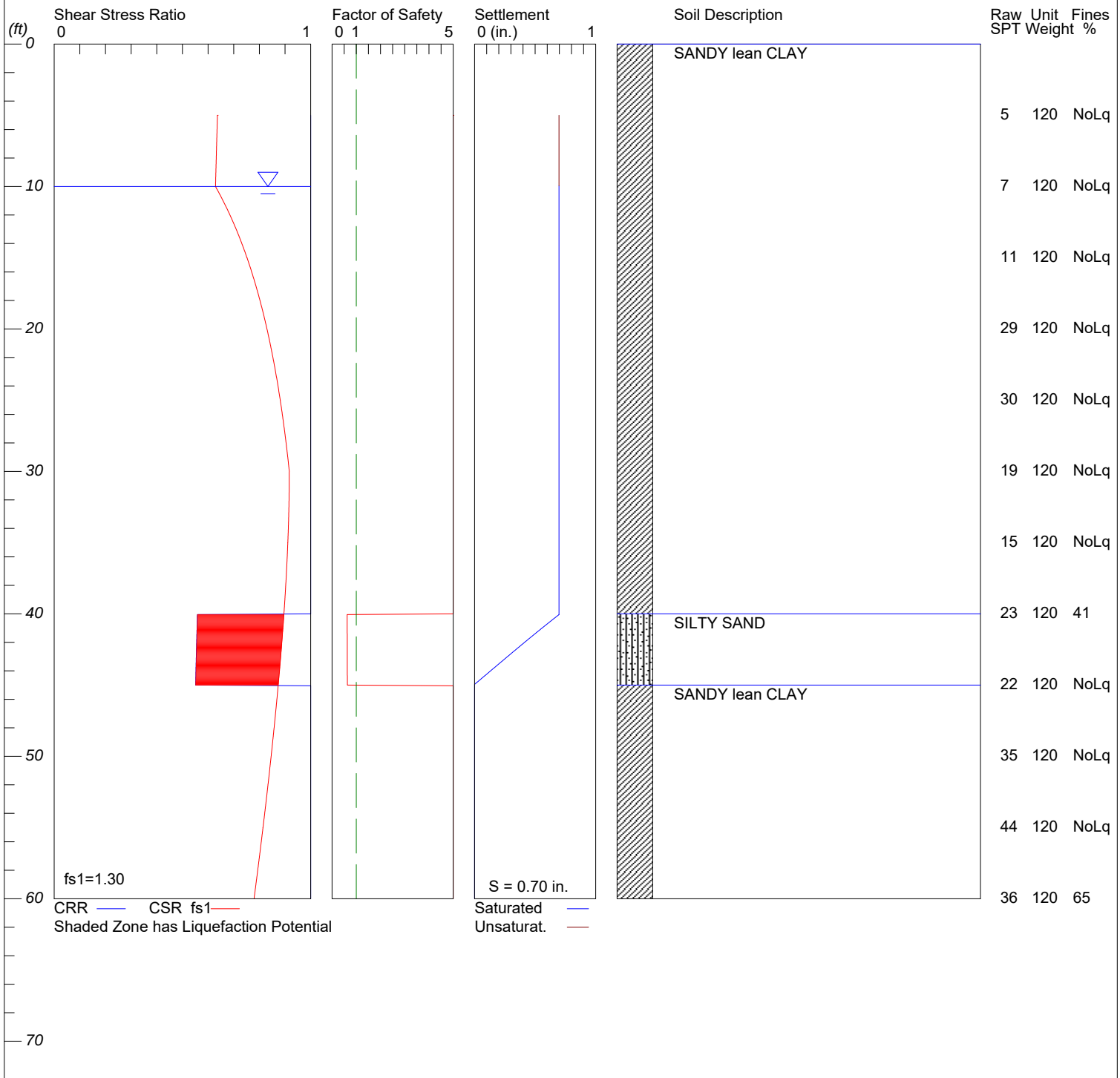
$N_{eqiv.}$ = Equivalent SPT blow counts converted from MC sample using Factor of 0.6
C_N = correction for over-burden
C_B = correction for borehole diameter
C_S = correction for sample type (assume 0.6 for modified California sampler)
C_R = correction for rod length (assuming rod sticks out 2 feet from the ground)
N_{60} = $N_{eqiv.}$ SPT normalized to 60% energy

LIQUEFACTION ANALYSIS

DKN ventura

Hole No.=B-2 Water Depth=10 ft Surface Elev.=707

Magnitude=7
Acceleration=0.762g



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LIQUEFACTION ANALYSIS SUMMARY

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Title: DKN ventura
Subtitle: SPT-2

Surface Elev.=707
Hole No.=B-2
Depth of Hole= 60.00 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration= 0.76 g
Earthquake Magnitude= 7.00

Input Data:
Surface Elev.=707
Hole No.=B-2
Depth of Hole=60.00 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration=0.76 g
Earthquake Magnitude=7.00
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: Post Liquefaction
5. Settlement Calculation in: Liq. zone only
6. Hammer Energy Ratio, Ce = 1.30
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1.1
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
5.00	5.00	120.00	NoLiq
10.00	7.00	120.00	NoLiq
15.00	11.00	120.00	NoLiq
20.00	29.00	120.00	NoLiq
25.00	30.00	120.00	NoLiq
30.00	19.00	120.00	NoLiq
35.00	15.00	120.00	NoLiq
40.00	23.00	120.00	41.00
45.00	22.00	120.00	NoLiq
50.00	35.00	120.00	NoLiq
55.00	44.00	120.00	NoLiq
60.00	36.00	120.00	65.00

Output Results:

Settlement of Saturated Sands=0.70 in.
Settlement of Unsaturated Sands=0.00 in.
Total Settlement of Saturated and Unsaturated Sands=0.70 in.
Differential Settlement=0.349 to 0.460 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
5.00	2.00	0.64	5.00	0.70	0.00	0.70
6.00	2.00	0.63	5.00	0.70	0.00	0.70
7.00	2.00	0.63	5.00	0.70	0.00	0.70
8.00	2.00	0.63	5.00	0.70	0.00	0.70
9.00	2.00	0.63	5.00	0.70	0.00	0.70
10.00	2.00	0.63	5.00	0.70	0.00	0.70
11.00	2.00	0.66	5.00	0.70	0.00	0.70
12.00	2.00	0.69	5.00	0.70	0.00	0.70
13.00	2.00	0.71	5.00	0.70	0.00	0.70
14.00	2.00	0.73	5.00	0.70	0.00	0.70
15.00	2.00	0.75	5.00	0.70	0.00	0.70
16.00	2.00	0.77	5.00	0.70	0.00	0.70
17.00	2.00	0.79	5.00	0.70	0.00	0.70
18.00	2.00	0.80	5.00	0.70	0.00	0.70
19.00	2.00	0.82	5.00	0.70	0.00	0.70
20.00	2.00	0.83	5.00	0.70	0.00	0.70
21.00	2.00	0.84	5.00	0.70	0.00	0.70
22.00	2.00	0.85	5.00	0.70	0.00	0.70
23.00	2.00	0.86	5.00	0.70	0.00	0.70

24.00	2.00	0.87	5.00	0.70	0.00	0.70
25.00	2.00	0.88	5.00	0.70	0.00	0.70
26.00	2.00	0.89	5.00	0.70	0.00	0.70
27.00	2.00	0.90	5.00	0.70	0.00	0.70
28.00	2.00	0.90	5.00	0.70	0.00	0.70
29.00	2.00	0.91	5.00	0.70	0.00	0.70
30.00	2.00	0.92	5.00	0.70	0.00	0.70
31.00	2.00	0.92	5.00	0.70	0.00	0.70
32.00	2.00	0.92	5.00	0.70	0.00	0.70
33.00	2.00	0.91	5.00	0.70	0.00	0.70
34.00	2.00	0.91	5.00	0.70	0.00	0.70
35.00	2.00	0.91	5.00	0.70	0.00	0.70
36.00	2.00	0.91	5.00	0.70	0.00	0.70
37.00	2.00	0.91	5.00	0.70	0.00	0.70
38.00	2.00	0.90	5.00	0.70	0.00	0.70
39.00	2.00	0.90	5.00	0.70	0.00	0.70
40.00	2.00	0.90	5.00	0.70	0.00	0.70
41.00	0.56	0.89	0.62*	0.56	0.00	0.56
42.00	0.56	0.89	0.63*	0.41	0.00	0.41
43.00	0.55	0.88	0.63*	0.27	0.00	0.27
44.00	0.55	0.88	0.63*	0.13	0.00	0.13
45.00	0.55	0.87	0.63*	0.00	0.00	0.00
46.00	2.00	0.87	5.00	0.00	0.00	0.00
47.00	2.00	0.86	5.00	0.00	0.00	0.00
48.00	2.00	0.86	5.00	0.00	0.00	0.00
49.00	2.00	0.85	5.00	0.00	0.00	0.00
50.00	2.00	0.85	5.00	0.00	0.00	0.00
51.00	2.00	0.84	5.00	0.00	0.00	0.00
52.00	2.00	0.83	5.00	0.00	0.00	0.00
53.00	2.00	0.83	5.00	0.00	0.00	0.00
54.00	2.00	0.82	5.00	0.00	0.00	0.00
55.00	2.00	0.81	5.00	0.00	0.00	0.00
56.00	2.00	0.81	5.00	0.00	0.00	0.00
57.00	2.00	0.80	5.00	0.00	0.00	0.00
58.00	2.00	0.79	5.00	0.00	0.00	0.00
59.00	2.00	0.79	5.00	0.00	0.00	0.00
60.00	2.00	0.78	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.

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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