APPENDIX F: GEOTECHNICAL REPORT

Geotechnical Engineering Report

NEW RESIDENTIAL DEVELOPMENT

7035 Laurel Canyon Blvd
North Hollywood, California 91605

Prepared for:
The Jacmar Companies
2200 W. Valley Boulevard
Alhambra, CA 91803

January 25, 2023
Project No.: 4230.2200062.0000

January 25, 2023
Project No. 4230.2200062.0000

Mr. Phil Silver
The Jacmar Companies
2200 W. Valley Boulevard
Alhambra, CA 91803

Subject: **GEOTECHNICAL ENGINEERING REPORT**
NEW RESIDENTIAL DEVELOPMENT
7035 Laurel Canyon Blvd,
North Hollywood, California 91605

Dear Mr. Silver,

In accordance with your request and authorization, Universal Engineering Sciences, (UES) is pleased to submit the results of our geotechnical investigation for the proposed new residential development located at 7035 Laurel Canyon Blvd in North Hollywood, California 91605. The purpose of our investigation was to evaluate the subsurface conditions and to provide recommendations for design and construction of the proposed development.

Based on our findings, the proposed project is geotechnically feasible, provided that the recommendations in this report are incorporated into the design and are implemented during construction of the project. This report was prepared in accordance with the requirements of the 2023 Los Angeles Amendment to the 2022 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,
Universal Engineering Sciences (UES)

Taha Ashoori, Ph.D., P.E.
Project Engineer

Alexia Mackey, EIT
Staff Scientist

Distribution: one pdf document via email to Addressee

TABLE OF CONTENTS .... Page

Figures

Tables

Appendices

1. INTRODUCTION

This report presents the results of our geotechnical engineering evaluation performed for the proposed new residential development to be located at 7035 Laurel Canyon Blvd, North Hollywood, California (Figure 1, Site Location Map). The purpose of this study has been to evaluate the subsurface conditions at the site and to provide geotechnical recommendations related to the design and construction of the proposed structure.

This report includes a brief description of the proposed development, a discussion regarding the current field exploration, laboratory testing results, description of subsurface conditions, engineering seismology and geological hazards and provides geotechnical conclusions and recommendations.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The approximate site coordinates are latitude 34.19850°N and longitude 118.39724°W and is located at approximately 739 feet above Mean Sea Level (MSL). The site is bounded to the right by the Laurel Canyon Boulevard and neighbors from the north, south and south to the private properties. The northerly portion of the approximately 2.2-acre site has been developed with a 39,000 ft^2 building that is currently being used by the United States Postal Service office and DMV Vehicle registration office. The remaining site is asphalt-paved and used as surface parking lot.

The proposed project consists of a multi-story, mixed-use building totally approximately 34,900 square foot footprint. The building will have 5 sections, consisting of a 2 story, 3 story, 5 story, and 6 story sections, with a 2-level deep subterranean parking garage beneath the latter three sections up to about 24 feet below existing street grade. To implement this, the entire existing building and the parking lot will be demolished. The two-story section directly adjacent to the pool courtyard will be constructed at grade. The proposed structure will be set-back between 8 to 25 feet from the property line and will be used as pedestrian area. The entrance to the building will be accessed from the North Laurel Canyon Blvd. There is no information regarding the type and size of foundation at the time of this writing. However, it is anticipated that the new building will be supported on any types of shallow foundations consisting of continuous footings, spread footings or mat footings.

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 in-house geologic maps, topographic maps, and aerial photographs relevant to the subject site in preparation of this report. The list of documents reviewed is presented in the "References" section of this report.

3.2. Field Exploration

The subsurface soil and groundwater conditions beneath the site were explored by UES on December 15 and 16, 2022 and included drilling, logging, and sampling of 5 exploratory hollow stem auger borings (B-1 through B-5). Boring B-1 and B-3 were advanced to a depth of 51.5 feet below the existing grade. Borings B-2 and B-4 were advanced to depths of 76.5 feet and boring B-5 was advanced to a total depth of 36.5 feet below the existing grade. The drilling operation was performed using a truck-mounted CME 75 drill rig equipped with 8-inch hollow-stem auger.

Prior to initiation of the field exploration program, a field reconnaissance was conducted to observe surface conditions and to mark the locations of the planned subsurface exploration. Underground Service Alert was notified of the exploratory boring locations at least 48 hours prior to drilling.

The borings were surface logged by a California Professional Engineer in general accordance with the visual-manual procedure for description and identification of soils per ASTM D2488. The staff Engineer prepared the recovered samples for subsequent reference and laboratory testing. At the completion of drilling, the borings were backfilled with tamped soil cuttings, and asphalt cold patches were placed to restore the drilled asphalt surface. The approximate locations of the borings are shown on Figure 2 – Site Plan, Boring and Cross Section Locations. Detailed exploration information of soil borings is presented in Appendix A.

3.3. Geotechnical Laboratory Testing

Laboratory tests were performed on selected samples recovered from the borings to aid in the classification of soils and to evaluate pertinent engineering properties of the foundation soils. The following tests were performed:

- In-situ Moisture Content and Dry Density, ASTM D2937;
- Grain Size Distribution, ASTM D6913;
- #200 sieve wash, ASTM D1140;
- Atterberg Limits, ASTM D4318;
- Maximum Density, ASTM D4253;
- Expansion Index, ASTM D4829;
- Consolidation, ASTM D2435;
- Direct Shear, ASTM D3080;
- Compaction test, ASTM 1557;

- Corrosion Testing in Soils:
    - pH and resistivity, CTM 643;
    - Sulphates, CTM 417; and
    - Chlorides, CTM 422.

Laboratory testing was performed in general accordance with applicable ASTM Standards and California Test Methods. 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 geotechnical conclusions and recommendations, including:

- Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials,
- Evaluation of site-specific seismic design parameters in accordance with 2019 California Building Code,
- Corrosion potential of the on-site soils to buried concrete and steel,
- Evaluation of current and historic high groundwater conditions at the site and potential impact on the existing structures and site development,
- Evaluation of project feasibility and suitability of on-site soils for foundation support,
- Evaluation of foundation design parameters including soil bearing capacity, lateral resistance, retaining walls, friction coefficient, settlement estimate and seismic considerations.

4. SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1. Regional Geologic Setting

The subject site is regionally located in central southern part of the Transverse Ranges. The Transverse Range province of southern California is an elongate geomorphic and structural unit that trends essentially east-west across parts of Santa Barbara, Ventura, Los Angeles, San Bernardino, and Riverside Counties. Its name reflects its transverse orientation with respect to the adjacent provinces, especially the Coast Ranges and Sierra Nevada to the north and the Peninsular Ranges to the south (Bailey, 1954). It is located between the San Andreas Fault to the north and the Hollywood Fault to the south.

Locally, the subject site is in the eastern portion of the San Fernando Valley, in the Tujunga Wash, south of San Gabriel Mountains, north of Santa Monica Mountains (Jennings, 1969). According to the Geologic map of the San Fernando and Van Nuys (north 1/2) quadrangles, Los Angeles County, California (Dibblee, 1991), the project site is underlain by Qa- Holocene. The deposit is described as surficial sediments- alluvial gravel, sand, and some clay of the valley areas.

4.2. Subsurface Earth Materials

Earth materials encountered during our subsurface investigation shows that one geologic unit was encountered in our exploration, young Quaternary alluvium valley deposits (Qa). In general, the alluvium consisted of sandy lean clays, clayey sands, and silty clayey sands.

4.3. Groundwater

Groundwater was not encountered during the subsurface investigation. According to the Seismic Hazard Zone Report for Van Nuys 7.5-Minute Quadrangle (1997), the highest groundwater historically is approximately at 65 feet below the ground surface. 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. Based on our findings, we note that the potential for groundwater to impact the proposed improvements is considered low. However, for the design of the building, a groundwater depth of 65 feet should be considered.

It should be noted that groundwater levels may fluctuate due to seasonal variations, rainfall, irrigation, or other factors. Evaluation of such factors is beyond the scope of this study.

4.4. Rippability

Based on our subsurface exploration of the site, the near-surface materials should be generally excavatable with heavy-duty earthwork equipment in good working condition.

4.5. Caving Potential

In general, the near surface soils contain significant amounts of sand and has a medium potential for caving. We recommend that the geotechnical engineer should be notified immediately if caving conditions are encountered during excavations to provide further mitigation recommendations.

4.6. Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content the onsite fill consists of sandy silt within the soils encountered near the ground surface. Generally, this material exhibits “very low” expansion potential.

4.7. Corrosive Soils

The potential for the on-site materials to corrode buried steel and concrete improvements was evaluated. Laboratory testing was performed on representative soil samples to evaluate pH, minimum resistivity, and soluble chloride and sulfate contents. General recommendations to address the corrosion potential of the on-site soils are provided below. Imported fill materials, if used, should be tested to evaluate whether their corrosion potential is more severe than those assumed.

4.7.1. Sulfate Exposure

Laboratory tests indicate that the potential of sulfate attack on concrete in contact with the on-site soils is “negligible” or “S0” exposure in accordance with ACI 318, Table 19.3.1.1. Therefore, restriction on the type of cement, water to cement ratio, and compressive strength is not required.

4.7.2. Ferrous Metals

The results of the laboratory chemical tests performed on a sample of soil collected within the site indicate that the on-site soils are anticipated to likely have a “very high” corrosion potential to buried ferrous metals. A corrosion specialist should be consulted regarding suitable types of piping and necessary protection for underground metal conduits. The corrosion potential of the on-site soils should be verified during construction for each encountered soil type. Imported fill materials should be tested prior to placement to confirm that their corrosion potential is not more severe than the one assumed for the project.

5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS

5.1. Surface Fault Rupture

The subject site is not located within a State of California Alquist-Priolo Earthquake Fault Zone (formerly known as a Special Studies Zone) (CGS, 2018), nor does the County of Los Angeles note faults on the site (County of Los Angeles, 2020). It is our opinion that the likelihood of fault rupture occurring at the site during the design life of the proposed improvements is low.

The nearest active fault and AP zone, the Sierra Madre fault zone, is approximately 5.95 miles north-northeast of the site. The closest fault (inactive) is the Verdugo fault, approximately 2.25 miles northeast of the site. The site is subject to intense ground shaking during a seismic event.

5.2. Liquefaction 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 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.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

A review of the Seismic Hazard Zone Report for the Van Nuys 7.5-Minute Quadrangle (1997), the Earthquake Zones of Required Investigation Van Nuys Quadrangle (1998) and the County of Los Angeles General Plan (2020) indicate the site is located not within a liquefaction or the landslide zone. Based on the anticipated depth of groundwater greater than 50 feet along with dense soil conditions in soil borings, we consider the potential for seismically induced liquefaction to be low.

5.3. Landslides

Based on our review of the referenced geologic maps, literature, topographic maps, aerial photographs, and our subsurface evaluation, no landslides or related features underlie or are adjacent to the subject site. Due to the relatively level and limited gradient changes of the site and surrounding areas, the potential for landslides at the project site is considered negligible.

5.4. 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 the FEMA (2021) flood map, the site is outside the 0.2% annual chance (500-year) floodplain.

5.5. Dam Inundation

Dam Inundation occurs when structural damage to a dam results in a flood. Structural damage to a dam can occur due to earthquakes, liquefaction, landslides, lateral spreading, water overflow and erosion, and design failure. A dam inundation map shows areas that would be susceptible to flooding in the event of dam failure. In 2017, the California Legislature passed a law requiring all state jurisdictional dams, except low hazard dams, to develop inundation maps and emergency action plans. DSOD approves inundation maps, and Cal OES approves emergency action plans. Based on review of the California Department of Water Resources Dam Breach Inundation Maps and County of Los Angeles General Plan, the site is not located within an inundation boundary for nearby dams.

5.6. 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 Los Angeles Tsunami Inundation Map for Emergency Planning. The potential for the site to be adversely impacted by earthquake-induced tsunamis is considered negligible because the site is located approximately 13.8 miles inland from the Pacific Ocean shore, at an elevation exceeding the maximum height of potential tsunami inundation.

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 negligible due to the lack of any significant enclosed bodies of water located in the vicinity of the site.

5.7. Seismic Design Parameters

Our recommendations for seismic design parameters have been developed in accordance with 2022 CBC and ASCE 7-16 (ASCE, 2016) standards. The applicable site class is D based on the results of our field investigation.

Table 1 – 2022 California Building Code Design Parameters presents the seismic design parameters for the site in accordance with 2022 CBC.

Table 1 – 2022 California Building Code Design Parameters
Site coordinates N34.19850° and W-118.39724°

Design Parameters	Value
Site Class	D
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s	1.912 g
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1	0.65 g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.7
Adjusted MCE_R Spectral Response Acceleration Parameter at Short Period, S_{MS}	1.912 g
1-Second Period Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1}	1.105 g
Short Period Design Spectral Response Acceleration Parameter, S_{DS}	1.275 g
1-Second Period Design Spectral Response Acceleration Parameter, S_{D1}	0.737 g
Peak Ground Acceleration, PGA_M	0.858 g
Seismic Design Category	D

Notes: [1] long period coefficient (Fv) of 1.7 may be utilized for calculation of Ts, provided that the value of the Seismic Response Coefficient (Cs) is determined by Equation 12.8-2 for values of the fundamental period of the building (T) less than or equal to 1.5Ts, and taken as 1.5 times the value computed in accordance with either Equation 12.8-3 for T greater than 1.5Ts and less than or equal to TL or Equation 12.8-4 forT greater than TL.

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 summary of the geotechnical considerations for this project:

- Groundwater was not encountered during subsurface investigation. Historic high groundwater was mapped at a depth of approximately 65 feet. Therefore, groundwater is not expected to impact the proposed development.
- Due to anticipated groundwater depth below 50 feet, our potential liquefaction settlement is considered negligible.
- The potential for landslide, flooding, tsunami and seiches to impact the proposed improvement is considered low.
- The site is not located within an AP Zone, however, it is subject to intense ground shaking during a seismic event.
- Based on the laboratory testing, the on-site soils are not expected to cause injurious sulfate attack on concrete with a minimum 28-day compressive strength of 2,500 psi. Based on results of laboratory testing and our local experience with similar soils, the on-site soils are expected to exhibit corrosion potential to ferrous metals. Expansion potential of the on-site soils is very low.
- Based on these considerations, it is recommended a conventional reinforced spread foundation system may be utilized for support of the proposed structure provided foundations derive support in the competent alluvial soils found at the excavation bottom. Foundations should be deepened as necessary to penetrate through any unsuitable soils and derive support in the competent alluvial soils. Any soils unintentionally disturbed should be properly compacted. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the design of a foundation system are provided in this report.
- Excavations on the order of 24 feet in vertical height may be required for construction of the subterranean parking area, including the excavations for the foundation system. Due to the depth of the excavation and the proximity to adjacent offsite structures, excavation of the proposed subterranean levels will require shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed retaining wall and shoring systems should be designed to resist the surcharge imposed by the adjacent offsite structure.
- Due to the nature of the proposed design and intent for subterranean levels, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is

not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

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. Site Preparation and Earthwork

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

6.2.1. General Grading Recommendations

Site preparation should begin with the removal of foundations, pavements, 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, pavement, and fill areas. We recommend that unsuitable materials such as organic matter or oversized material be selectively removed and disposed 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.

6.2.2. Foundation Preparation

The proposed foundations in the subterranean portion of the building should derive support in the competent alluvial soils found at the proposed excavation bottom. Foundations should be deepened as necessary to penetrate through any unsuitable soils and derive support in the competent alluvial soils.

Additionally, in areas where the building is constructed at-grade, overexcavation should be at least 4 feet below the surface, or 2 feet below the proposed bottom of foundation, whichever is deeper to create a relatively uniform support for foundations and slab-on-grade. The lateral extent of the overexcavation should be at least 5 feet beyond the edge of the future footings, where space is available. Deeper excavations may be required in areas where soft, saturated, or unsuitable materials, for example, tree root balls or undocumented fill are encountered. Any soils unintentionally disturbed should be properly compacted.

6.2.3. Flatworks Preparation

Sidewalk, transformer pads and trash enclosures should be scarified to a depth of at least 12 inches below the existing ground surface, or 12 inches below the bottom of the design section (i.e., aggregate base) whichever is lower. Deeper removals may be required in areas where soft, saturated, or unsuitable materials are encountered.

The extent and depths of removal should be evaluated by soil engineer in the field based on the materials exposed. Additional removals may be recommended if loose or soft soils are exposed during grading.

6.2.4. Materials for Fill

On-site soils are suitable to be reused for compaction effort. 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 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 UES prior to importing or filling.

6.2.5. Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed excavation bottom by UES. 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 near optimum moisture content. The scarified materials should then be compacted to 95 percent relative compaction in accordance with the latest version of ASTM Test Method D1557.

Compacted fill should be placed in horizontal lifts of approximately 6 to 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve least 125 percent of the optimum moisture content, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other appropriate compacting rollers, to a relative compaction of 95 percent as evaluated by ASTM D1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

The upper one foot of soils below sidewalks should be processed and compacted to at least 95 percent of relative compaction per ASTM D1557.

6.2.6. Temporary Excavations

The on-site soils are not expected to pose unusual excavation difficulties; therefore, conventional earth-moving equipment may be used. Localized sloughing/raveling of exposed soil intervals should be anticipated. All trench excavations should be performed in accordance with CalOSHA regulations. The on-site soils may be considered a Type C soil, as defined by the current CalOSHA soil classification.

Unsurcharged excavations: Sides of temporary, unsurcharged excavations less than 20 feet deep should be sloped back at an inclination of 1.5(H):1(V) or flatter. Where space for sloped sides is not available, shoring will be necessary. Additional recommendations for the shoring system are provided in the temporary shoring section of this report.

For any configurations where the depth of the excavation plus the height of any nearby retaining wall or slope exceed 20 feet, a slope stability analysis shall be performed by the Geotechnical Engineer.

Stockpiled (excavated) materials should be placed no closer than 4 feet from the top of the trench, except in areas where an existing retaining wall is within 8 feet of the top of the trench where no stockpiling is allowed. A greater setback may be necessary when considering surcharge loads such as heavy vehicles, concrete trucks and cranes. UES should be advised of such heavy vehicle loadings so that specific setback requirements can be established for the used equipment. Alternatively, a shoring system may be designed to allow reduction in the setback distance.

Personnel from UES should observe the excavation progress so that appropriate modifications to the excavation design may be recommended, if necessary, due to conditions differing from the design assumptions.

6.3. Temporary Shoring

Temporary shoring is anticipated to be placed along the perimeter of the proposed two story subterranean parking. Based on the assumed finished floor elevation and anticipated foundation excavations, shored walls may be on the order of 24 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 UES for conformance with the design intent and recommendations.

6.3.1. 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 density of 36 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 24H in psf, where H is the height of the shored wall in feet.

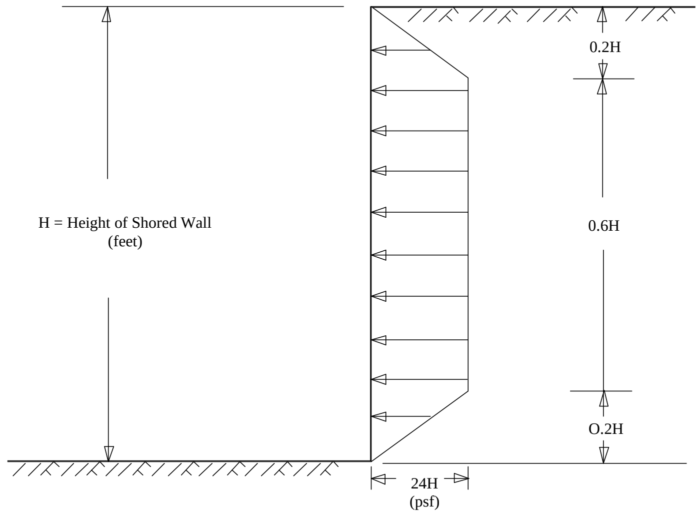


Diagram 1 – Earth Pressure Distribution for Tie-back or Braced Shoring Wall

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.

6.3.2. Surcharge Load

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

Resultant lateral force:

$$R = \frac{0.3 \times P \times h^2}{x^2 + h^2}$$

Location of lateral resultant:

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

Where:

R:	Resultant lateral force measured in pounds per foot of wall width.
P:	Resultant surcharge loads of continuous or isolated footings measured 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. 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.

6.3.3. Soldier Pile Design

The soldier piles should be designed in accordance with the geotechnical parameters presented in Table 2 – Geotechnical Design Parameters for Soldier Beams. 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 2 – Geotechnical Design Parameters for Soldier Beams

Design Parameters	Value
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 density (EFD)	350 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 350 psf per foot of embedment below the excavation bottom up to 3,000 psf. 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 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.

6.3.4. Tie-Back Earth Anchor Design

Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the failure wedge adjacent to the shoring is defined by a plane drawn at nearly Ψ = 28 degrees from the vertical from the toe of the wall. The unbound portion of the anchors should extend at least 15 feet beyond the soldier beam; however, the shoring engineer should evaluate the bonded length required beyond the failure wedge based on the loading on the shoring and the allowable skin friction provided. The bonded length should commence no less than H/5 or 5 feet beyond the failure wedge, whichever is greater.

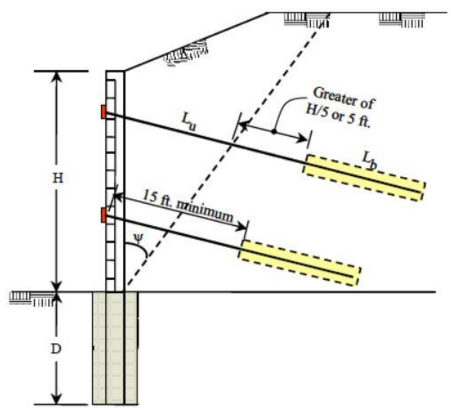


Diagram 2 – Tie-back Bond Length

The capacity of the anchors should be evaluated by testing of initial anchors installed. For preliminary design purposes, conventional drilled anchors (gravity grouted) may be designed for an ultimate bond stress of 15 psi and an ultimate bond stress of 30 psi for grout anchors. Only the resistance developed beyond the failure wedge should be used in resisting lateral loads. If the anchors are spaced at least 6 feet on center, no reduction in the capacity of the anchors need be considered due to group action.

As the proposed tie-back system is intended for temporary use, provisions should be made in the design to de-tension and abandon the tie-backs when the subgrade walls are able to support the lateral loads.

6.3.5. Anchor Testing

We recommend at least 10% of the production anchors be tested to at least 150% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15 minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

To reduce chances of caving during tie-back testing, the portion of the anchor shafts within the failure wedge may need to be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and be flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

6.3.6. Anchor Installation

The anchors may be installed at angles of 15 to 30 degrees below the horizontal. Although we did not encounter caving during drilling for our subsurface investigation, we anticipate that caving may occur during the drilling of tiebacks. The contractor should implement appropriate measures to stabilize the drilled hole such as the installation of steel casing for the loose cohesionless materials or the use of drilling mud. The anchors should be filled with concrete placed by pumping from the tip out. The portion of the anchor tendons within the failure wedge should be sleeved in plastic. If the anchor tendons are sleeved, it is acceptable to grout the entire length of the anchor.

6.3.7. 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.

6.3.8. Construction Dewatering

Due to the absence of shallow groundwater, dewatering measures are not anticipated to be necessary during dry season. However, if excavations are schedule during rainy wet season, such excavations that extend below the recorded historic high groundwater level, such as that for the proposed below grade structures, should be dewatered. The highest recorded historic groundwater levels is approximately 6 feet below the ground surface.

Construction dewatering may include sump pumps, well-points, wells, eductors, or a combination of each strategy to control groundwater. At the below grade structure excavations, the majority of the structure will be constructed within bedrock with a moderate to rapid groundwater flow rate.

In order to reduce the potential for settlement of adjoining structures, groundwater drawdown outside of the excavation should be controlled during pumping in order to limit the drawdown level to be approximately at or above the static groundwater elevation as determined prior to construction.

It is noted that a NPDES permit will be required by the Regional Water Quality Control Board in order to discharge the water. It should be noted that controlling and maintaining of the groundwater during construction is the responsibility of the contractor.

6.4. Foundation Recommendations

At the time of this writing the size and type of foundations are not yet known. However, the proposed residential building may be supported on conventional spread footings established within competent engineered fills or native alluvial soils. Our geotechnical foundation design parameters for the design of shallow foundations for the proposed building are provided in Table 3.

Table 3 – Geotechnical Design Parameters for Foundation

Design Parameters	Values
Bearing Material	• See Foundation Preparation section of this report.
Minimum Footing Dimensions	• At least 12 inches in width and at least 18 inches in depth.
Depth of Embedment	• At least 2 feet below the lowest adjacent grade
Allowable Bearing Pressure	• An allowable bearing capacity of 6,000 psf may be used for the design of foundations at the subterranean level foundations deriving support in competent native soils • 4,000 psf for the design of foundations at-grade level supported on engineering fill • The allowable bearing values may be increased by one-third for transient loads from wind or earthquake.
Estimated Static Settlement	• Less than 1 inch total settlement with differential settlement estimated to be less than 0.5 inch over 30 feet. • The static settlement of the foundation system is expected to occur on initial application of loading.
Allowable Coefficient of Friction Below Footings	• 0.40
Unfactored Lateral Passive Resistance	• 350 pcf (equivalent fluid density, EFD) • Maximum allowable of 4,500 psf
Modulus of Subgrade Reaction (k)*	• 100 pci

As mentioned above, the structural building loads are not provided to us at this time and since the settlement criteria might control the design, the allowable bearing pressure for the spread foundation will be revisited for the final design once loading data becomes available.

6.5. Basement and Retaining Walls

6.5.1. Lateral Earth Pressure

Retaining walls should be designed to resist the earth pressure exerted by the retained soil and water plus any additional lateral force that will be applied to the walls due to surface loads placed at or near the walls. The design criteria for retaining walls are presented in Table 4. Walls that are restrained against lateral deflection should be designed using the at-rest earth pressure. Walls that are free to deflect at their tops may be designed for the active earth pressure.

Table 2 – Design Criteria for Retaining Walls

Backfill Configuration	Earth Pressure	Equivalent Fluid Density (pcf)	
		Drained	Submerged
Level	Active At-Rest Passive	36 55 350	N/A
	Seismic Earth Pressure (Unrestrained Wall)	29 pcf	
	Seismic Earth Pressure (Restrained Wall)	15 pcf	

The resultant force should be applied at a distance of H/3 above the bottom of the wall, where H is the wall height.

For above grade unrestrained walls, an additional seismic pressure, as listed in Table 2, should be added to the active earth pressure, for walls that are retaining 6 or more feet of soil. For restrained walls, the provided seismic pressure in Table 4 should be added to the static at-rest pressure Lateral earth pressures provided above are ultimate values. Therefore, a suitable factor of safety should be applied to these values for design purposes. The appropriate factor of safety will depend on the design condition and should be determined by the project Structural Engineer. Care must be taken during the compaction operations not to overstress the walls. Heavy compaction equipment or other loads should not be allowed within a horizontal distance equal to the height of the wall without additional structural evaluation to review the added stresses to the wall. The above lateral earth pressures do not include the effects of surcharges (e.g., normal plant traffic, adjacent footings, earthwork equipment, etc.) on the wall pressures.

Any surcharge located within a 1 (H):1(V) plane drawn upward from the base of the excavation should be evaluated and the resultant lateral force added to the lateral earth pressure for wall design using the recommended surcharge factor provided in Table 2 above. The surcharge load is a uniform pressure distribution with the resultant acting at a height of H/2 above the bottom of the wall, where H is the wall height. The calculation details are provided in section 6.3.2 of this report. A typical surcharge value for heavy trucks used by is 250 psf.

6.5.1. Backfill and Drainage of Walls

The backfill material behind walls should consist of granular non-expansive material and be approved by the project geotechnical engineer. Based on the soil materials encountered during our exploration, native soils will meet this requirement. Retaining walls should be adequately drained. Adequate backfill drainage is essential to provide a free-drained backfill condition and to limit hydrostatic buildup behind walls. The walls should be appropriately waterproofed. Drainage behind the basement 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% 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 (Public Works Standards, 2015). The drain should discharge through a solid pipe to an appropriate outlet, using a sump/pump system.

Where property line limitations and shoring walls do not allow for the installation of a standard subdrainage system outside the wall, rock pockets may be utilized. The rock pockets should drain through the wall. The pockets should be a minimum of 12 inches in length, width and depth. The pocket should be filled with gravel. The rock pockets should be no more than 8 feet on center.

6.5.2. Waterproofing

Moisture effecting retaining walls is one of the most post construction disputes. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity. It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained on order to recommend a product or method which would provide protection to below grade walls.

6.6. Concrete Slab-On-Grade

At minimum the building slab-on-grade should be at least 5 inches in thickness and should be reinforcement with a minimum of No. 4 bars spaced at 18 inches on-center. Final design of the slab should be provided by the project structural engineer.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements. At minimum, the vapor retarder should consist of 10 mil Stegowrap or equivalent.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. Sand above the vapor retarder is outside of UES purview and should be in accordance with the structural engineer's recommendation.

UES does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure. Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

The recommendations presented above are intended to reduce the potential for cracking of slabs; however, even with the incorporation of the recommendations presented herein, slabs may still exhibit some cracking. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics.

6.7. Drainage Control

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be always maintained. All site drainage, with the exception of any required to disposed of onsite by stormwater regulations, should be collected and transferred to the street in non-erosive drainage devices.

The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

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 UES 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 UES 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 information obtained from our field exploration for 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 excavation operations, for example, the presence 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 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 UES has no control.

UES's recommendations for this site are, to a high degree, dependent upon appropriate quality control of foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for UES to observe foundation excavations for the proposed construction. If parties other than UES 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 and the engineering geologist 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. UES 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 design and construction of the project described herein. Any party other than the client who wishes to use this report for an adjacent or nearby project, shall notify UES of such intended use. 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 project, UES 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 any other party will release UES from any liability resulting from the use of this report by any unauthorized party.

UES has endeavored to perform 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 expressed or implied, is made as to the conclusions and recommendations contained in this report.

9. SELECTED REFERENCES

American Society of Civil Engineers, 2011, Minimum Design Loads for Buildings and Other Structures: ASCE Standard ASCE/SEI 7-16, 608 pp.

ASTM, 2001, "Soil and Rock: American Society for Testing and Materials," vol. 4.08 for ASTM test methods D-420 to D-4914; and vol. 4.09 for ASTM test methods D-4943 to highest number.

Bailey, T. L., & Jahns, R. H. (1954). Geology of the Transverse Range province, southern California.

California Geological Survey, 1997, Seismic Hazard Zone Report for the Van Nuys 7.5-Minute Quadrangle, Los Angeles County, California. SHR Report 08.

California Geological Survey, 1998, Earthquake Zones of Required Investigation Van Nuys Quadrangle.

California Geological Survey, 2008, Guidelines for Evaluation and Mitigation of Seismic Hazards in California: Special Publication 117A, 98 pp.

California Geological Survey, 2018, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California: Special Publication 42, 93 pp

Coduto, Donald P., 1994, Foundation Design: Principles and Practices: Prentice-Hall, Inc, Englewood Cliffs, New Jersey.

County of Los Angeles, 2020, County of Los Angeles General Plan: Chapter 11 Hazards and Safety.
Dibblee, T.W. 1991. Geologic map of the San Fernando and Van Nuys (north 1/2) quadrangles, Los Angeles County, California. Dibblee Geological Foundation. Dibblee Foundation Map DF-33, scale 1:24,000.

FEMA, Federal Emergency Management Agency, National Flood Insurance Program, Flood Insurance Rate Map, Los Angeles County, California and Incorporated areas, Panel No. 06037C1310F, dated September 26, 2008.

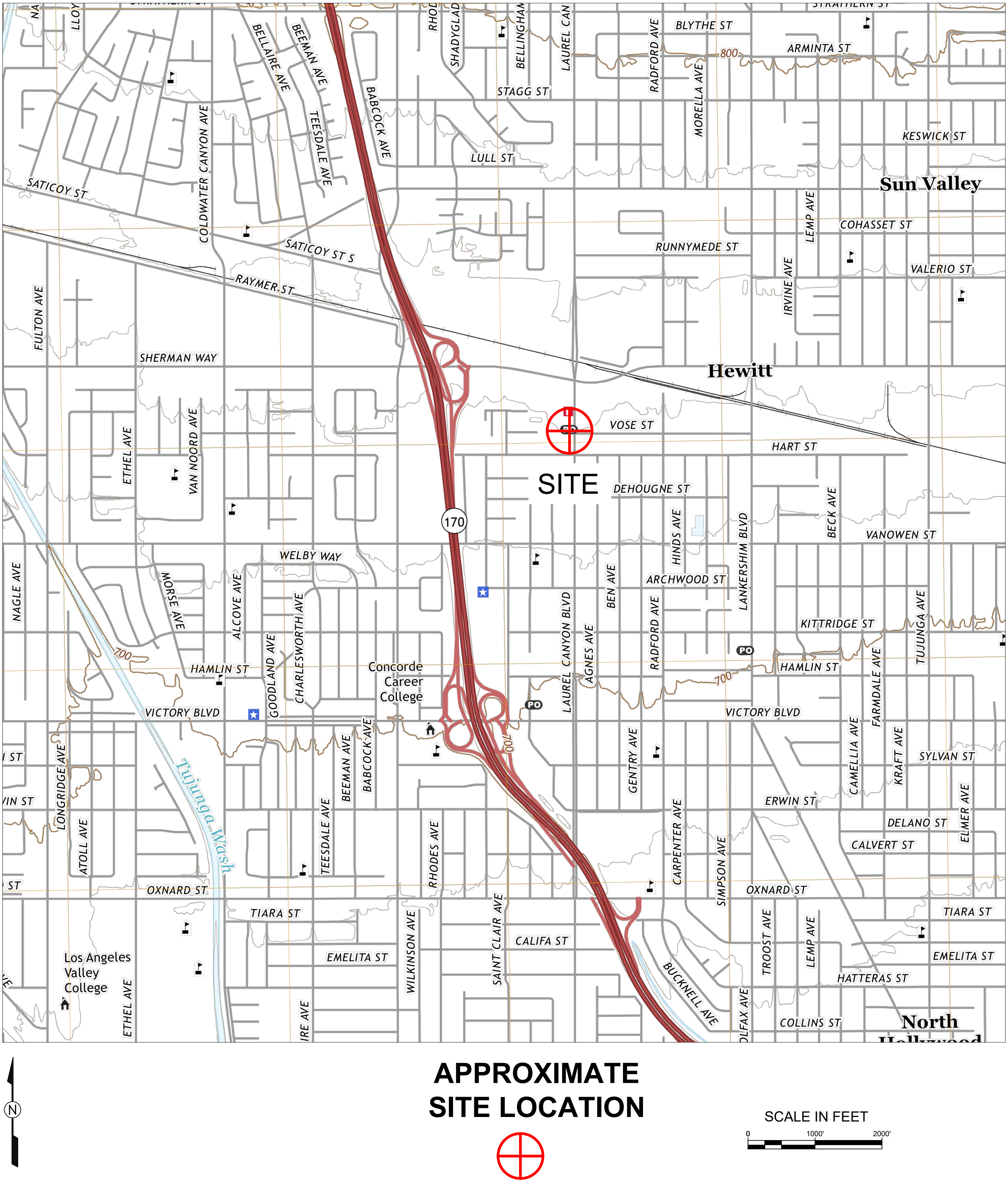
Hitchcock, C.S. and Wills, C.J., 2000, Quaternary geology of the San Fernando Valley, Los Angeles County, California: California Division of Mines and Geology Map Sheet 50, scale 1:48,000.

Jennings, C.W., and Bryant, W.A., 2010, Fault activity map of California: California Geological Survey Geologic Data Map No. 6, scale 1:750,000.

P.J. Sabatini, D.G. Pass, R.C. Bachus, 1999 "GEOTECHNICAL ENGINEERING CIRCULAR NO. 4 - Ground Anchors and Anchored Systems", Report No. FHWA-IF-99-015, U.S. Department of Transportation, Federal Highway Administration, dated June 1999.

Southern California Earthquake Center Group C, 2001, Active Faults in the North Hollywood Metropolitan Region, SCEC Special Publication Series 1, 47 pp.

FIGURES



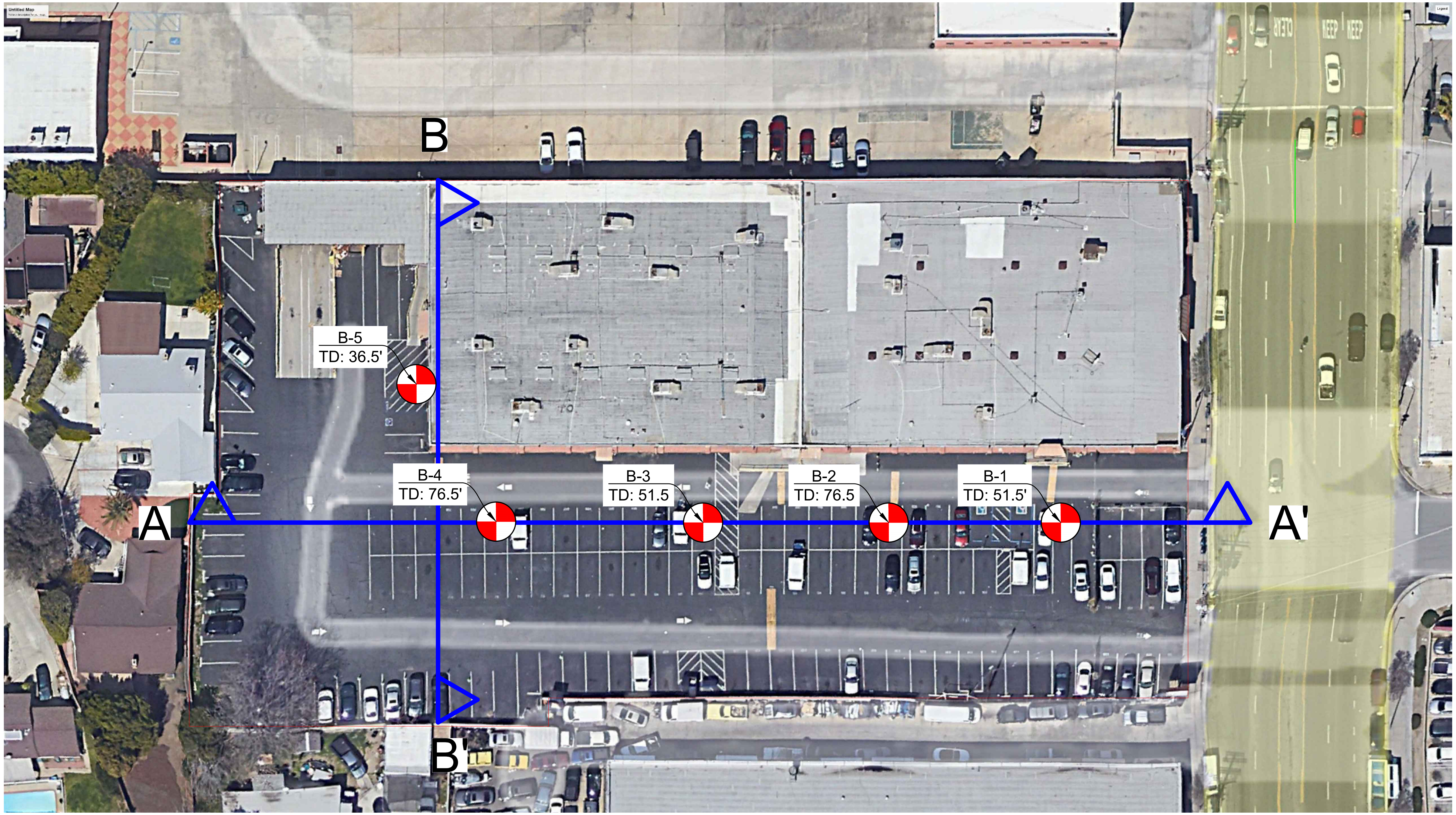
REFERENCE: USGS "Van Nuys 7.5-Minute Quadrangle" (2022)

UNIVERSAL® ENGINEERING SCIENCES

SITE VICINITY LOCATION MAP

Jacmar Companies
7035 Laurel Canyon Blvd
North Hollywood, California

PROJECT NO.	REPORT DATE	FIGURE 1
4230.2200062.0000	January 2023	



N

50 0 25 50 100

(IN FEET)

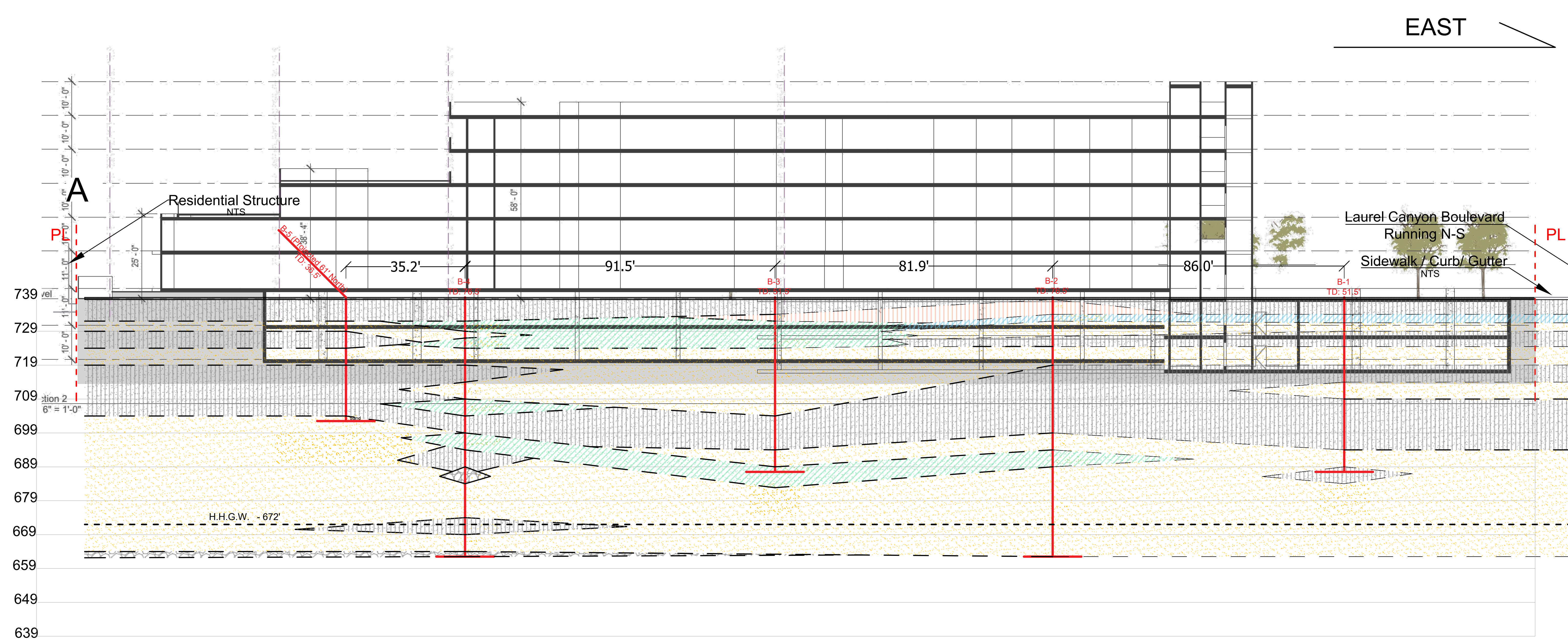
PROJECT BOUNDARY

CROSS SECTION LOCATION

BORING LOCATION

UNIVERSAL ENGINEERING SCIENCES

SITE PLAN, BORING AND CROSS SECTION LOCATIONS		
Jacmar Companies 7035 Laurel Canyon Blvd North Hollywood, California		
PROJECT NO. 4230.2200062.0000	REPORT DATE January 2023	FIGURE 2



LEGEND

SCALE IN FEET

0 10 20

NOTE: All dimensions, locations, and directions are approximate.
REFERENCE: Google Earth (2022)

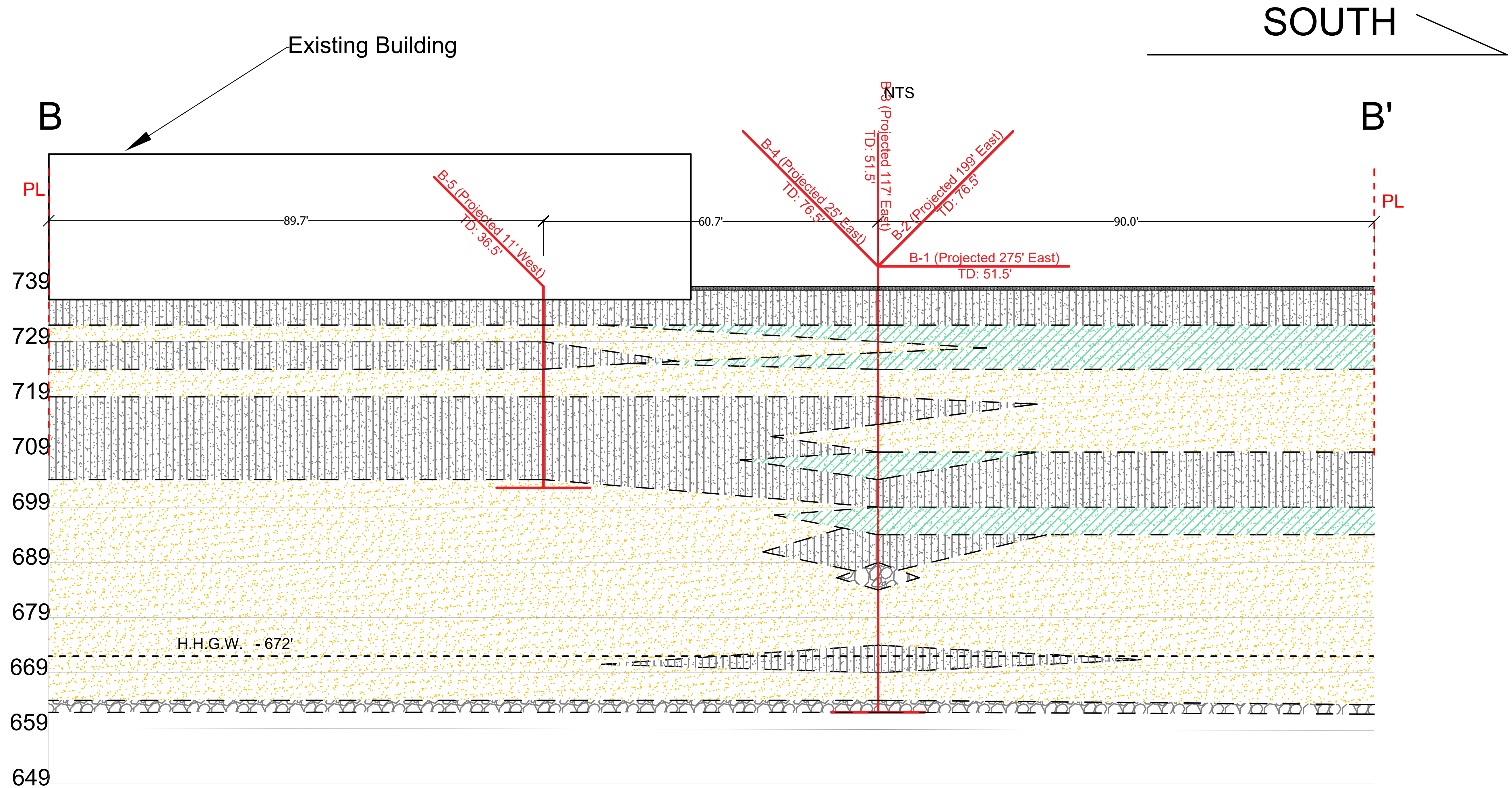
- SAND (SP)
- SAND WITH SILT (SP-SM)
- SAND WITH CLAY (SP-SC)
- PROPERTY LINE
- PAVEMENT
- SILT (ML)
- CLAY WITH SILT (CL-ML)
- GRAVEL (GP-GM)

UNIVERSAL
ENGINEERING SCIENCES

CROSS SECTION A-A'

Jacmar Companies
7035 Laurel Canyon Blvd
North Hollywood, California

PROJECT NO.	REPORT DATE	FIGURE 2A
4230.2200062.0000	January 2023	



LEGEND

SCALE IN FEET
0 15

NOTE: All dimensions, locations, and directions are approximate.
REFERENCE: Google Earth (2022)

SAND (SP)	PAVEMENT
SAND WITH SILT (SP-SM)	GRAVEL (GP-GM)
SAND WITH CLAY (SP-SC)	PROPERTY LINE

UNIVERSAL ENGINEERING SCIENCES

CROSS SECTION B-B'

Jacmar Companies
7035 Laurel Canyon Blvd
North Hollywood, California

PROJECT NO.	REPORT DATE	FIGURE 2B
4230.2200062.0000	January 2023	

APPENDIX A
Field Exploration Boring Logs

UNIVERSAL ENGINEERING SCIENCES
Universal Engineering Sciences
16 Technology Dr., Ste 139
Irvine, CA 92618
Telephone: 949-989-6940

BORING NUMBER B-1

CLIENT The Jacmar Companies
PROJECT NAME 7035 N. Laurel Canyon Blvd.
PROJECT NUMBER 4230.2200062.0000
PROJECT LOCATION 7035 N Laurel Canyon Blvd, North Hollywood, CA
DATE STARTED 12/16/22 **COMPLETED** 12/14/22
GROUND ELEVATION 739 ft MSL **HOLE SIZE** 8 inches
DRILLING CONTRACTOR Choice Drilling
GROUND WATER LEVELS: Not Encountered
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY AM **CHECKED BY** TA
AT END OF DRILLING ---
NOTES 34.198480, -118.396878
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	5" Asphalt, no base	AU								
	(SP-SM) Poorly graded SAND with silt, fine, moist, dark grayish brown, loose, damp	MC	4-5-5 (10)		98.3	7.4				
	(CL-ML) Silty CLAY with SAND, more silt, medium dense, damp	SPT	7-9-10 (19)							
	(SP) Poorly graded SAND with gravel, fine to coarse, light olive brown, loose, damp	MC	8-10-12 (22)		129.5	1.3				
10	(SM) Silty SAND, olive has gravel, loose, damp	SPT	3-4-4 (8)							
	(SP-SM) Poorly graded SAND with silt, fine to medium, olive brown, medium dense, damp	MC	10-12-17 (29)		106.8	4.8				
	(SP) Poorly graded SAND, light olive, medium dense, damp	SPT	6-9-14 (23)							
20	(SP-SM) Poorly graded SAND with silt, olive brown, medium dense, damp	MC	15-15-8 (23)		108.8	3.1				
	(SP) Poorly graded SAND, olive, medium dense, damp	SPT	9-10-12 (22)							
30	(SP-SM) Poorly graded SAND with silt, fine to medium, light olive brown, very dense, moist	MC	12-30-50/5"		117.9	2.8				
	(SP-SM) Poorly graded SAND with silt, light olive brown, medium dense, damp,	SPT	7-10-11 (21)			3.3				7.6
40	(SP-SM) Poorly graded SAND with silt and gravel, very dense, moist	MC	36-50/5"		113.9	3.3				
	(SP) Poorly graded SAND, olive, medium dense	MC	9-20-24 (44)							
50	(SM) Silty SAND with gravel, olive brown, very dense, moist	SPT	100		120.8	3.4	NP	NP	NP	13.2

Bottom of borehole at 51.5 feet.
Backfilled with native clippings.

Universal Engineering Sciences
16 Technology Dr., Ste 139
Irvine, CA 92618
Telephone: 949-989-6940

BORING NUMBER B-2

CLIENT The Jacmar Companies
PROJECT NAME 7035 N. Laurel Canyon Blvd.
PROJECT NUMBER 4230.2200062.0000
PROJECT LOCATION 7035 N Laurel Canyon Blvd, North Hollywood, CA
DATE STARTED 12/16/22 **COMPLETED** 12/14/22
GROUND ELEVATION 739 ft MSL **HOLE SIZE** 8 inches
DRILLING CONTRACTOR Choice Drilling
GROUND WATER LEVELS: Not Encountered
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY AM **CHECKED BY** TA
AT END OF DRILLING ---
NOTES 34.198476, -118.397115
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	8" Asphalt, no base									
	(ML) Silt, fine, dark olive brown, loose, moist	MC AU	6-7-9 (16)		97.6	19.1				
	(CL-ML) Silty CLAY, brown to olive, medium stiff, moist	SPT	2-2-3 (5)							
10	(SP) Poorly graded SAND with gravel, fine to coarse, light gray, loose, damp	MC	3-6-11 (17)		117.6	2.8				
	(SP) Poorly graded SAND, olive, medium dense, moist	SPT	7-9-12 (21)							
	(SP-SM) Poorly graded SAND with silt trace fine gravel, fine to coarse, light gray, medium dense, moist	MC	11-16-25 (41)		110.4	3.2				
	(SP) Poorly graded SAND, olive, dense, damp	SPT	15-22-38 (60)							
20	(SP-SM) Poorly graded SAND with silt trace with fine gravel, fine to coarse, light olive brown, very dense, damp	MC	46-50/3"		121.6	3.2				
	(SP-SC) Poorly graded sand with silt and gravel, olive, medium dense, moist	SPT	11-13-18 (31)							
30	(SP-SM) Poorly graded SAND with silt with trace fine gravel, fine to medium, olive brown, dense, damp	MC	15-32-44 (76)		110.0	4.4				
	(SC-SM) Silty, clayey SAND with gravel, stiff, moist	SPT	17-20-25 (45)							
40	(SP) Poorly graded SAND with gravel, fine to coarse, light olive brown, very dense, moist	MC	35-42-50/4"		115.6	2.8				
	(SP-SC) Poorly graded SAND with clay trace gravel, olive, medium dense, moist	SPT	21-35-32 (67)							
	Hard layer, drilled through									
50	(SP) Poorly graded SAND with gravel, medium to coarse, more angular gravel then rounded, light brown, very dense, damp	MC	50-50/6"		121.9	2.2				
	Very hard rock									
	(SP) Poorly graded SAND with gravel, olive and grey, very dense, dry	SPT	46-50/4"							
60		MC								
	Poor recovery, cobble, and gravel									
	(SP) Poorly graded SAND with gravel, fine to coarse, olive brown, poor recovery, very dense, moist	SPT	21-50/5"			2.7				
70	Poor recovery, cobble, and gravel	MC								
		SPT								

Bottom of borehole at 76.5 feet.
Backfilled with native clippings.

Universal Engineering Sciences
16 Technology Dr., Ste 139
Irvine, CA 92618
Telephone: 949-989-6940

BORING NUMBER B-3

CLIENT The Jacmar Companies
PROJECT NAME 7035 N. Laurel Canyon Blvd.
PROJECT NUMBER 4230.2200062.0000
PROJECT LOCATION 7035 N Laurel Canyon Blvd, North Hollywood, CA
DATE STARTED 12/15/22 **COMPLETED** 12/14/22
GROUND ELEVATION 739 ft MSL **HOLE SIZE** 8 inches
DRILLING CONTRACTOR Choice Drilling
GROUND WATER LEVELS: Not Encountered
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY AM **CHECKED BY** TA
AT END OF DRILLING ---
NOTES 34.198476, -118.397379
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		8" Asphalt, no base	AU								
		(SC-SM) Silty, clayey SAND trace gravel, dark brown, medium stiff, damp	SPT	2-2-3 (5)			10.6				
		(ML) Silt, fine, dark olive brown, medium dense, moist	MC	5-6-8 (14)		98.8	12.3				
		(SM) Silty SAND, olive brown	SPT	5-7-9 (16)			5.7	NP	NP	NP	13.8
10		(SC-SM) Silty, clayey SAND trace fine gravel, fine to medium, dark brown, medium dense, damp	MC	8-12-16 (28)		107.5	11.5				
		(SC) Clayey SAND trace gravel, olive, stiff, damp	SPT	7-9-13 (22)							
		(SP) Poorly graded SAND with gravel, medium to coarse, light olive brown, dense, damp				113.3	2.6				
20		(SP) Poorly graded SAND, olive, medium dense, damp	SPT	19-30-39 (69)							
		(SP) Poorly graded SAND with fine gravel, fine to coarse, light olive brown, medium dense, damp	MC	38-50/5"		106.3	2.6				
30		(SP) Poorly graded SAND, light to medium olive, dense, damp	SPT	14-19-30 (49)							
		(SP-SM) Poorly graded SAND with silt trace fine gravel, fine to coarse, olive brown, very dense, moist	MC	39-50/6"		114.5	3.0				
40		(SM) Poorly graded SAND with silt trace gravel, olive brown, dense, moist	SPT	20-32-40 (72)			13.2	NP	NP	NP	8.4
		(SP) Poorly graded SAND with fine gravel, fine to coarse, grayish brown, very dense, moist	MC	38-50/5"		113.5	2.6				
50		(SP-SC) Poorly graded sand with clay and gravel, olive, very dense, moist	SPT	39-50/5"							

Bottom of borehole at 51.5 feet.
Backfilled with native clippings.

UNIVERSAL ENGINEERING SCIENCES

Universal Engineering Sciences
16 Technology Dr., Ste 139
Irvine, CA 92618
Telephone: 949-989-6940

BORING NUMBER B-4

CLIENT The Jacmar Companies
PROJECT NAME 7035 N. Laurel Canyon Blvd.
PROJECT NUMBER 4230.2200062.0000
PROJECT LOCATION 7035 N Laurel Canyon Blvd, North Hollywood, CA
DATE STARTED 12/15/22 **COMPLETED** 12/14/22
GROUND ELEVATION 739 ft MSL **HOLE SIZE** 8 inches
DRILLING CONTRACTOR Choice Drilling
GROUND WATER LEVELS: Not Encountered
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY AM **CHECKED BY** TA
AT END OF DRILLING ---
NOTES 34.198470, -118.397680
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		8" Asphalt, nob base	AU								
		(SP-SM) Poorly graded SAND with silt, olive to brown, loose, moist	SPT	5-6-7 (13)							
		(SM) Silty SAND with gravel, fine to medium, olive brown, medium dense, moist	MC	15-18-30 (48)		109.6	4.7				
10		(SP-SC) Poorly graded sand with clay trace gravel, loose, moist	SPT	5-6-10 (16)							
		(SP) Poorly graded SAND trace fine gravel, fine to coarse, olive brown, medium dense, moist	MC	14-19-26 (45)		112.3	3.0				
		(SP-SC) Poorly graded sand with clay and gravel, olive brown, loose, moist	SPT	6-8-10 (18)							
		(SP) Poorly graded SAND with trace gravel, fine to coarse, light olive brown, dense, moist	MC	20-32-45 (77)		113.9	3.6				
20		(SP-SM) Poorly graded SAND with silt, brown, medium dense, moist	SPT	5-6-9 (15)			3.7				5.9
		(SP) Poorly graded SAND with fine gravel, fine to coarse, olive brown, very dense, moist	MC	35-50/6"		116.0	3.2				
30		(SP-SC) Poorly graded sand with clay trace gravel, light olive to gray, medium dense, moist	SPT	10-13-19 (32)							
		(SP-SM) Poorly graded SAND with silt, fine to medium some coarse, olive brown, very dense, moist	MC	39-50/6"		110.5	3.8				
40		(SP-SC) Prooly graded sand with clay trace gravel, olive dense, moist	SPT	18-27-36 (63)							
		(SM) Silty SAND with gravel, fine to coarse, olive brown, very dense, moist	MC	41-50/3"		122.0	6.3				
50		No recovery, rock, very dense	SPT	50/6"							
		(SP) Poorly graded SAND with gravel, fine to coarse, dark brown, very dense, moist	MC	50/6"		136.1	3.0				
60		(SP) Poorly graded SAND, light olive to gray, very dense, moist	SPT	38-50/5"							
		(SP-SM) Poorly graded SAND with silt and gravel, fine to coarse, light olive brown	MC	50-50/2"		128.1	3.5				
70		(SP) Poorly graded SAND, gray, little recovery, very dense, damp	SPT	25-50/1"							
		(GP-GM) Poorly graded GRAVEL with silt with fine to coarse sand, dark olive brown, very dense, damp	MC	50-50/6"		132.0	3.1				

Bottom of borehole at 76.5 feet. Backfilled with native clippings.

UNIVERSAL ENGINEERING SCIENCES

Universal Engineering Sciences
16 Technology Dr., Ste 139
Irvine, CA 92618
Telephone: 949-989-6940

BORING NUMBER B-5

CLIENT The Jacmar Companies
PROJECT NAME 7035 N. Laurel Canyon Blvd.
PROJECT NUMBER 4230.2200062.0000
PROJECT LOCATION 7035 N Laurel Canyon Blvd, North Hollywood, CA
DATE STARTED 12/15/22 **COMPLETED** 12/14/22
GROUND ELEVATION 739 ft MSL **HOLE SIZE** 8 inches
DRILLING CONTRACTOR Choice Drilling
GROUND WATER LEVELS: Not Encountered
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY AM **CHECKED BY** TA
AT END OF DRILLING ---
NOTES 34.198629, -118.397790
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		8" Asphalt, no base	AU								
		(SM) Silty SAND, fine, dark brown, loose, damp	SPT	3-2-3 (5)			11.0				
5		(SP-SM) Poorly graded SAND with silt, olive brown, medium dense, damp	MC	6-8-11 (19)		93.5	9.9				
		(SP) Poorly graded SAND, olive, medium dense, damp	SPT	9-13-14 (27)							
10		(SP-SM) Poorly graded SAND with silt and trace fine gravel, fine to coarse, dark brown, medium dense, moist	MC	9-14-27 (41)		112.5	5.5				
		(SP-SM) Poorly graded SAND with silt, medium dense, damp	SPT	8-11-17 (28)							
15		(SP) Poorly graded SAND with trace fine gravel, fine to coarse, yellowish brown, medium dense, damp	MC	24-25-25 (50)		111.1	3.3				
20		(SP-SM) Poorly graded SAND with silt and gravel, light olive brown, loose, damp	SPT	3-5-6 (11)			2.2				5.5
25		(SP-SM) Poorly graded SAND with silt and fine gravel, fine to coarse, light olive brown, medium dense, damp	MC	20-28-40 (68)		113.5	3.5				
30		(SP-SM) Poorly graded SAND with silt, olive, medium dense, damp	SPT	9-11-15 (26)							
35		(SP) Poorly graded SAND with gravel, fine to coarse, light olive brown, very dense	MC	24-42-50/4"		116.8	2.5				

Bottom of borehole at 36.5 feet.
Backfilled with native clippings.

Appendix A
Field Exploration Boring Logs

General

The subsurface exploration program for the proposed project consisted of logging five 8-inch diameter exploratory borings conducted at the site on December 15 and 16, 2022. The borings were advanced to depths of approximately 36.5, 51.5, and 76.5 feet below the existing grades. The drilling operation was performed using a truck mounted CME-75 hollow-stem-auger drill rig performed by Choice Drilling.

Drilling and Sampling

The Boring Logs are presented in the following pages. The log also shows the boring number and drilling date. The borings were logged by a 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.

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 plastic bags.

A California modified sampler was used to obtain drive samples of the soil encountered. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft that was driven a total of 12-inches into the soil at the bottom of the boring by a safety hammer weighing 140 pounds at a drop height of approximately 30 inches. 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.

Upon completion of the borings, the boreholes were backfilled with soil from the cuttings.

APPENDIX B
Laboratory Testing

Appendix B
Laboratory Testing

ASTM D 2488 - Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

ASTM D 2937- In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

ASTM D 1140 - 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.

ASTM D 1557 - Maximum Dry Density and Optimum Moisture Content
The maximum dry density and optimum moisture content of the material of selected bulk samples obtained from the exploratory borings were evaluated in general accordance with the latest version of ASTM D 1557.

ASTM D 4318 - Atterberg Limit
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS).

ASTM D 4829 - Expansion Index Test
The expansion index of a selected material 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.

ASTM D 3080 - Direct Shear Tests
A direct shear test was performed on relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions.

ASTM D 2435 - Consolidation Test
A Consolidation tests was performed on a selected driven soil sample 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.

Soil Corrosivity

Soil pH and resistivity tests were performed by **Project X Corrosion** on a representative soil sample in general accordance with the latest version of ASTM D4972 and ASTM G187, respectively. The chloride content of the selected sample was evaluated in general accordance with the latest version of ASTM D4327. The sulfate content of the selected samples was evaluated in general accordance with the latest version of ASTM D4327.

Universal Engineering Sciences
16 Technology Dr.,Ste 139
Irvine, CA 92618
Telephone: 949-537-3222

SUMMARY OF LABORATORY RESULTS

CLIENT The Jacmar Companies

PROJECT NAME 7035 N. Laurel Canyon Blvd.

PROJECT NUMBER 4230.2200062.0000

PROJECT LOCATION 7035 N Laurel Canyon Blvd, North Hollywood, CA

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)
B-1	2.5						SP-SM	7.4	98.3
B-1	7.5						SP	1.3	129.5
B-1	12.5						SP-SM	4.8	106.8
B-1	20.0						SP-SM	3.1	108.8
B-1	30.0						SP-SM	2.8	117.9
B-1	35.0				9.5	7.6	SP-SM	3.3	
B-1	40.0						SP-SM	3.3	113.9
B-1	50.0	NP	NP	NP	>4.75	13.2	SM	3.4	120.8
B-2	2.5						ML	19.1	97.6
B-2	7.5						SP	2.8	117.6
B-2	12.5						SP-SM	3.2	110.4
B-2	20.0						SP-SM	3.2	121.6
B-2	30.0						SP-SM	4.4	110.0
B-2	40.0						SP	2.8	115.6
B-2	50.0						SP	2.2	121.9
B-2	65.0						SP	2.7	
B-3	0.0-5.0						SC-SM	10.6	
B-3	5.0						ML	12.3	98.8
B-3	7.5	NP	NP	NP	>4.75	13.8	SM	5.7	
B-3	10.0						SC-SM	11.5	107.5
B-3	15.0						SP	2.6	113.3
B-3	25.0						SP	2.6	106.3
B-3	35.0						SP-SM	3.0	114.5
B-3	40.0	NP	NP	NP	>4.75	8.4	SM	13.2	
B-3	45.0						SP	2.6	113.5
B-4	5.0						SM	4.7	109.6
B-4	10.0						SP	3.0	112.3
B-4	15.0						SP-SC	3.6	113.9
B-4	20.0				9.5	5.9	SP-SM	3.7	
B-4	25.0						SP	3.2	116.0
B-4	35.0						SP-SM	3.8	110.5
B-4	45.0						SM	6.3	122.0
B-4	55.0						SP	3.0	136.1
B-4	65.0						SP-SM	3.5	128.1
B-4	75.0						GP-GM	3.1	132.0
B-5	2.5						SM	11.0	
B-5	5.0						SP-SM	9.9	93.5
B-5	10.0						SP-SM	5.5	112.5
B-5	15.0						SP	3.3	111.1
B-5	20.0				>4.75	5.5	SP-SM	2.2	
B-5	25.0						SP-SM	3.5	113.5
B-5	35.0						SP	2.5	116.8

Boring No.	B-1	B-3	B-3	B-5				
Sample No.								
Depth (ft.)	50	7.5	40	20				
Sample Type	Ring	SPT	SPT	SPT				
Visual Soil Classification	Olive Brown Silty Sand with Gravel (SM)	Olive Brown Silty Sand (SM)	Olive Brown Poorly Graded Sand with Silt (SP-SM) *trace Gravel	Light Olive Brown Poorly Graded Sand with Silt and Gravel (SP-SM)				
Total Sample Weight (Coarse Fraction)								
Total Sample Weight (g):	516.92	375.36	382.58	473.37				
Plus #4 Weight (g):	97.70	11.10	50.31	74.71				
Percent Retained Coarse Fraction (+ #4):	18.9	3.0	13.2	15.8				
Weight of Moist Sample + Container (g):	1028.86	543.36	547.55	877.53				
Weight of Dry Sample + Container (g):	1011.21	521.81	534.29	866.96				
Weight of Container (g):	494.29	146.45	151.71	393.59				
Moisture Content (%):	3.4	5.7	3.5	2.2				
Container No.:	sr	t2	t1	3				
Weight After Wash								
Dry Weight of Sample + Container (g):	943.11	470.01	502.15	841.10				
Weight of Container (g):	494.29	146.45	151.71	393.59				
Dry Weight of Sample (gm):	448.82	323.56	350.44	447.51				
% Retained No. 200 Sieve	86.8	86.2	91.6	94.5				
% Passing No. 200 Sieve	**13.2**	**13.8**	**8.4**	**5.5**				

UNIVERSAL® ENGINEERING SCIENCES

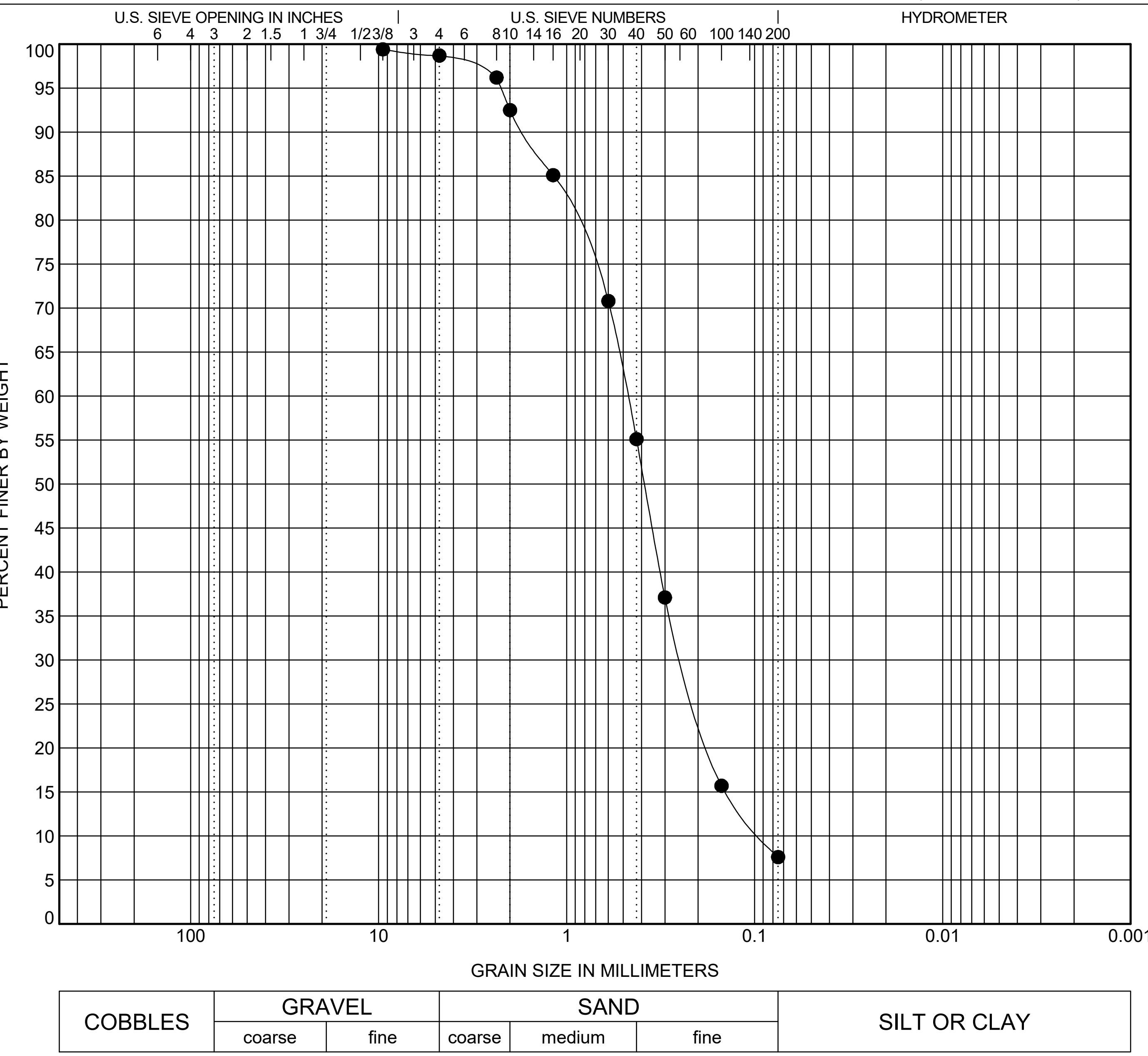
PERCENT PASSING No. 200 SIEVE
ASTM D 1140

Project Name: **7035 N. Laurel Canyon Blvd.**
Project No.: **4230.2200062.0000**
Client: The Jacmar Companies
Tested By: SE Date: 12/19/22

GRAIN SIZE DISTRIBUTION

Universal Engineering Sciences
16 Technology Dr.,Ste 139
Irvine, CA 92618
Telephone: 949-537-3222

CLIENT The Jacmar Companies
PROJECT NAME 7035 N. Laurel Canyon Blvd.
PROJECT NUMBER 4230.2200062.0000
PROJECT LOCATION 7035 N Laurel Canyon Blvd, North Hollywood, CA



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**35.0**	**(SP-SM) Poorly graded SAND with silt, light olive brown**				**1.30**	**5.14**

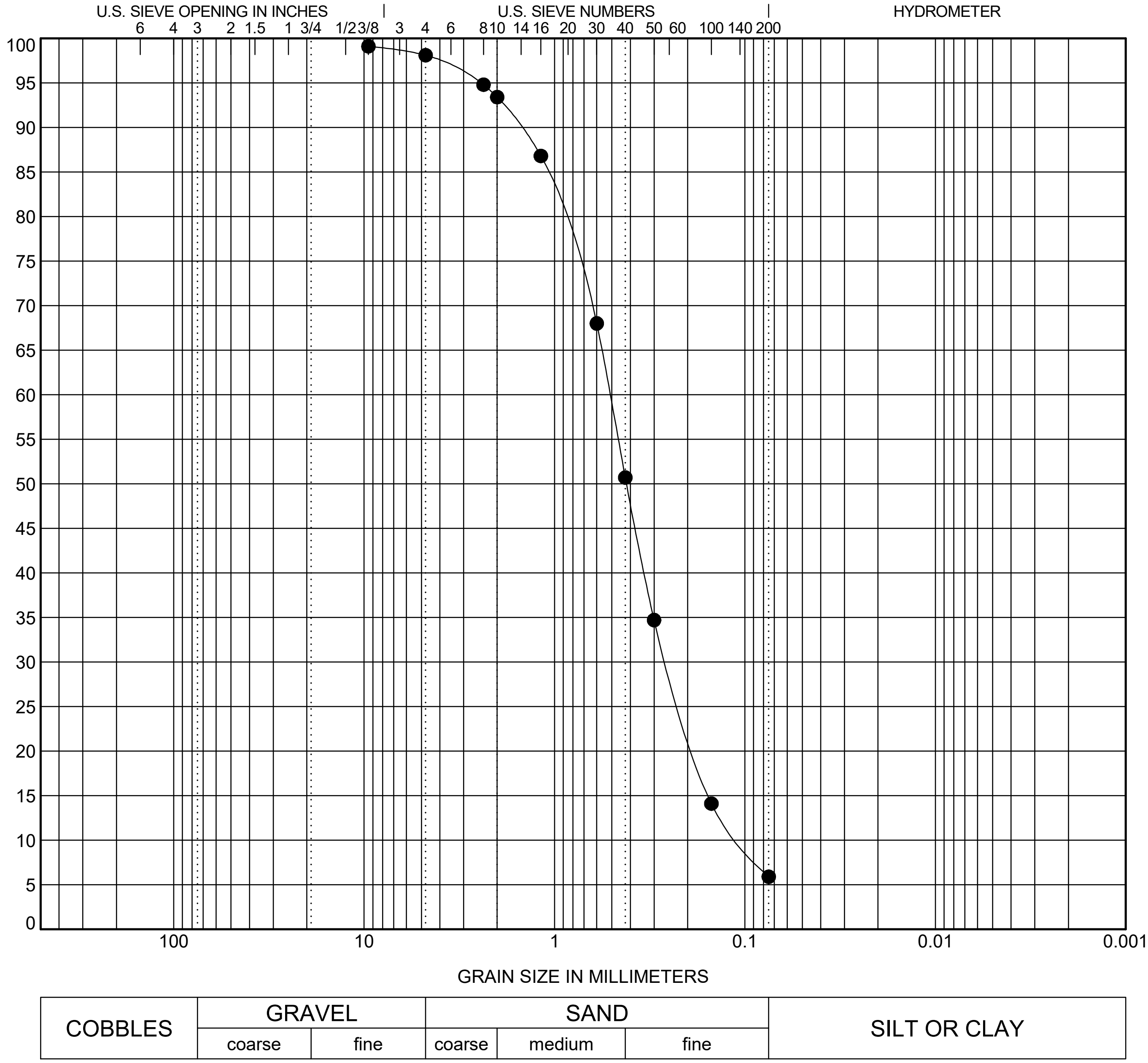
	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**35.0**	**9.5**	**0.473**	**0.238**	**0.092**	**1.3**	**91.1**	**7.6**	

Universal Engineering Sciences
16 Technology Dr.,Ste 139
Irvine, CA 92618
Telephone: 949-537-3222

GRAIN SIZE DISTRIBUTION

CLIENT The Jacmar Companies **PROJECT NAME** 7035 N. Laurel Canyon Blvd.

PROJECT NUMBER 4230.2200062.0000 **PROJECT LOCATION** 7035 N Laurel Canyon Blvd, North Hollywood, CA



BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
● **B-4**	**20.0**	**(SP-SM) Poorly graded SAND with silt, brown**				**1.21**	**4.82**

BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **B-4**	**20.0**	**9.5**	**0.512**	**0.256**	**0.106**	**1.9**	**92.2**	**5.9**	

UNIVERSAL ENGINEERING SCIENCES

Moisture and Density Data

7035 N Laurel Canyon Blvd.

4230.2200062.0000

Boring No.	B-1	B-1	B-1	B-1	B-1	B-1	B-1
Depth	2.5	7.5	12.5	20	30	40	50
Number of Rings	5	4	5	5	5	5	5
Weight of Soil and Ring	860.11	811.99	898.02	900.01	954.14	933.21	977
Weight of Rings	222.75	178.2	222.75	222.75	222.75	222.75	222.75
Wet Weight of Soil (g)	637.36	633.79	675.27	677.26	731.39	710.46	754.25
Wet Weight of Soil (lb)	1.405	1.397	1.489	1.493	1.612	1.566	1.663
Wet Density (PCF)	105.6	131.2	111.9	112.2	121.2	117.7	124.9
Wet Weight of Soil and Tare	156.82	613.91	147.91	132.77	130.95	188.79	1028.86
Weight of Tare	31.53	325.46	31.58	31.64	31.7	31.8	494.29
Dry weight of Soil and Tare	148.19	610.18	142.61	129.76	128.26	183.71	1011.21
Moisture Loss	8.63	3.73	5.3	3.01	2.69	5.08	17.65
Dry Weight of Soil	116.66	284.72	111.03	98.12	96.56	151.91	516.92
Moisture Content	7.4%	1.3%	4.8%	3.1%	2.8%	3.3%	3.4%
Dry Density (PCF)	98.3	129.5	106.8	108.8	117.9	113.9	120.8

UNIVERSAL
ENGINEERING SCIENCES

Moisture and Density Data

7035 N Laurel Canyon Blvd.

4230.2200062.0000

Boring No.	B-2	B-2	B-2	B-2	B-2	B-2	B-2	B-2						
Depth	2.5	7.5	12.5	20	30	40	50	65						
Number of Rings	5	5	5	5	5	5	5							
Weight of Soil and Ring	924.22	952.29	910.61	980.19	915.51	940.12	975.19							
Weight of Rings	222.75	222.75	222.75	222.75	222.75	222.75	222.75							
Wet Weight of Soil (g)	701.47	729.54	687.86	757.44	692.76	717.37	752.44							
Wet Weight of Soil (lb)	1.546	1.608	1.516	1.670	1.527	1.582	1.659							
Wet Density (PCF)	116.2	120.8	113.9	125.5	114.8	118.8	124.6							
Wet Weight of Soil and Tare	157	187.07	186.52	176.61	181.46	202.77	210.81	194.24						
Weight of Tare	31.64	31.57	31.64	31.36	31.81	31.61	31.88	31.87						
Dry weight of Soil and Tare	136.9	182.85	181.77	172.15	175.21	198.06	206.9	189.91						
Moisture Loss	20.1	4.22	4.75	4.46	6.25	4.71	3.91	4.33						
Dry Weight of Soil	105.26	151.28	150.13	140.79	143.4	166.45	175.02	158.04						
Moisture Content	19.1%	2.8%	3.2%	3.2%	4.4%	2.8%	2.2%	2.7%						
Dry Density (PCF)	97.6	117.6	110.4	121.6	110.0	115.6	121.9							

UNIVERSAL ENGINEERING SCIENCES

Moisture and Density Data

7035 N Laurel Canyon Blvd.

4230.2200062.0000

Boring No.	B-3	B-3	B-3	B-3	B-3	B-3	B-3							
Depth	0-5	5	10	15	25	35	45							
Number of Rings		5	5	5	5	5	5							
Weight of Soil and Ring		892.31	946.16	923.91	881.1	935.15	925.61							
Weight of Rings		222.75	222.75	222.75	222.75	222.75	222.75							
Wet Weight of Soil (g)		669.56	723.41	701.16	658.35	712.4	702.86							
Wet Weight of Soil (lb)		1.476	1.595	1.546	1.451	1.571	1.550							
Wet Density (PCF)		110.9	119.8	116.1	109.1	118.0	116.4							
Wet Weight of Soil and Tare	150.64	145.67	181.45	202.81	169.56	205.1	111.35							
Weight of Tare	31.64	31.69	31.65	31.68	31.85	31.71	20.74							
Dry weight of Soil and Tare	139.25	133.2	166.06	198.55	166.12	200.01	109.06							
Moisture Loss	11.39	12.47	15.39	4.26	3.44	5.09	2.29							
Dry Weight of Soil	107.61	101.51	134.41	166.87	134.27	168.3	88.32							
Moisture Content	10.6%	12.3%	11.5%	2.6%	2.6%	3.0%	2.6%							
Dry Density (PCF)		98.8	107.5	113.3	106.3	114.5	113.5							

UNIVERSAL ENGINEERING SCIENCES

Moisture and Density Data

7035 N Laurel Canyon Blvd.

4230.2200062.0000

Boring No.	B-4	B-4	B-4	B-4	B-4	B-4	B-4	B-4	B-4					
Depth	5	10	15	25	35	45	55	65	75					
Number of Rings	5	5	5	5	5	2	4	5	5					
Weight of Soil and Ring	915.65	921.21	935.51	945.21	915.07	402.29	855.61	1023.41	1044.11					
Weight of Rings	222.75	222.75	222.75	222.75	222.75	89.1	178.2	222.75	222.75					
Wet Weight of Soil (g)	692.9	698.46	712.76	722.46	692.32	313.19	677.41	800.66	821.36					
Wet Weight of Soil (lb)	1.528	1.540	1.571	1.593	1.526	0.690	1.493	1.765	1.811					
Wet Density (PCF)	114.8	115.7	118.1	119.7	114.7	129.7	140.3	132.6	136.1					
Wet Weight of Soil and Tare	109.59	534.91	102.95	187.65	161.24	165.1	230.36	178.6	252.49					
Weight of Tare	21.59	348.88	31.67	31.65	31.87	31.62	31.69	31.86	31.58					
Dry weight of Soil and Tare	105.61	529.41	100.46	182.84	156.54	157.18	224.51	173.61	245.91					
Moisture Loss	3.98	5.5	2.49	4.81	4.7	7.92	5.85	4.99	6.58					
Dry Weight of Soil	84.02	180.53	68.79	151.19	124.67	125.56	192.82	141.75	214.33					
Moisture Content	4.7%	3.0%	3.6%	3.2%	3.8%	6.3%	3.0%	3.5%	3.1%					
Dry Density (PCF)	109.6	112.3	113.9	116.0	110.5	122.0	136.1	128.1	132.0					

Moisture and Density Data

7035 N Laurel Canyon Blvd.

4230.2200062.0000

Boring No.	B-5	B-5	B-5	B-5	B-5	B-5								
Depth	2.5	5	10	15	25	35								
Number of Rings		5	5	5	5	5								
Weight of Soil and Ring		843.00	939.19	915.59	932.21	945.59								
Weight of Rings		222.75	222.75	222.75	222.75	222.75								
Wet Weight of Soil (g)		620.25	716.44	692.84	709.46	722.84								
Wet Weight of Soil (lb)		1.367	1.579	1.527	1.564	1.594								
Wet Density (PCF)		102.7	118.7	114.8	117.5	119.7								
Wet Weight of Soil and Tare	91.6	148.51	172.36	143.86	157.3	211.61								
Weight of Tare	20.74	31.57	31.68	31.86	31.87	31.68								
Dry weight of Soil and Tare	84.56	138.01	165.04	140.31	153.01	207.15								
Moisture Loss	7.04	10.5	7.32	3.55	4.29	4.46								
Dry Weight of Soil	63.82	106.44	133.36	108.45	121.14	175.47								
Moisture Content	11.0%	9.9%	5.5%	3.3%	3.5%	2.5%								
Dry Density (PCF)		93.5	112.5	111.1	113.5	116.8								

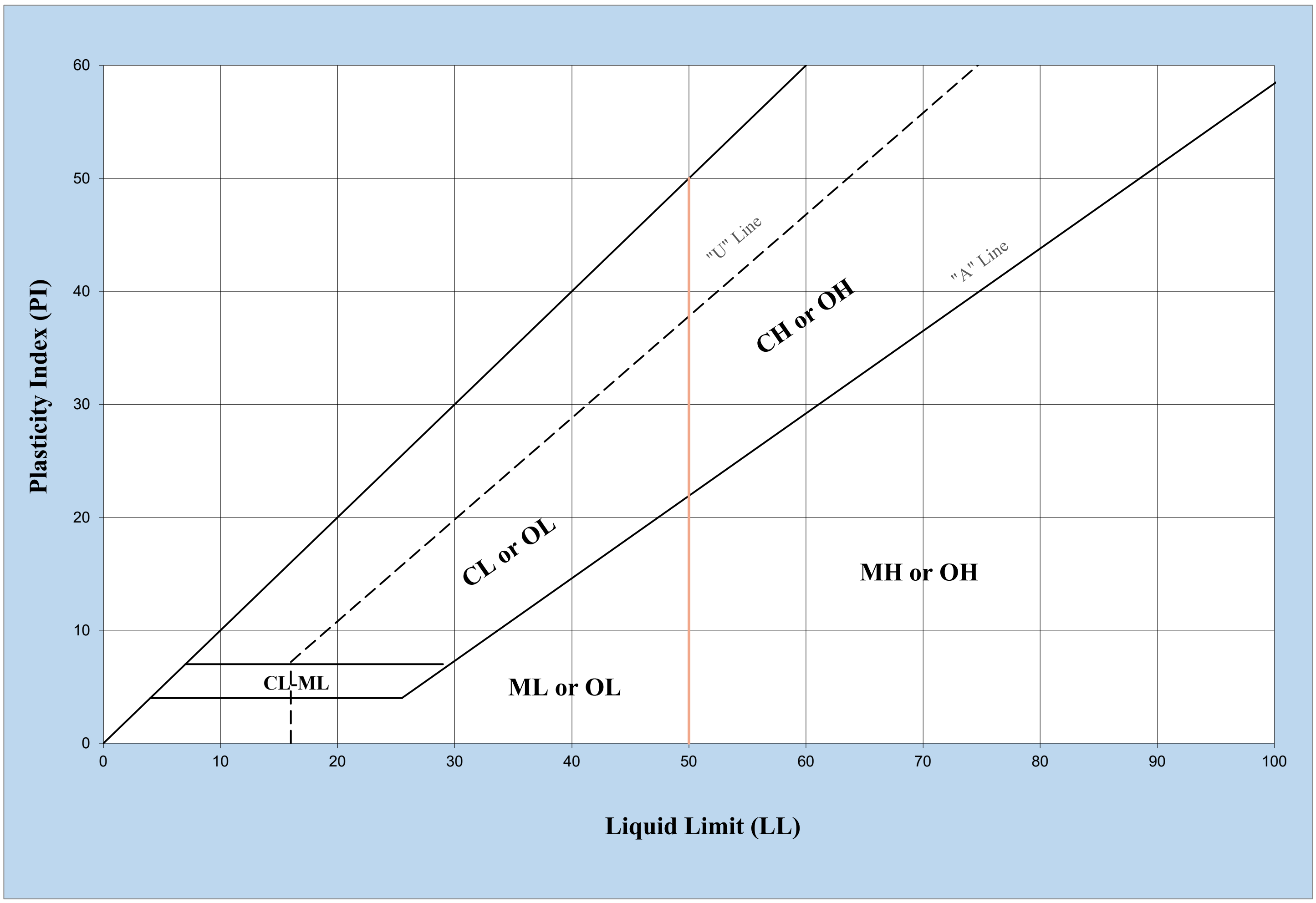
UNIVERSAL® ENGINEERING SCIENCES
16 Technology Dr, Suite 139
Irvine, CA 92618
949 989 6941

Lab Accreditation Number:

Atterberg Limits

Job Number:	4230.2200062.0000
Client:	The Jacmar Companies
Project Name:	7035 N. Laurel Canyon Blvd.
Permit Number:	---
Material Source	Boring 1
Sample Location:	B-1 @ 50 ft.

Sample ID	B-1 @ 50 ft.
Date Received	12/19/2022
Sampled By:	AM
Date Sampled:	12/15/2022
Tested By:	SE
Test Method	D4318



Sample Point

LL: NP PL: NP PI: NP

Soil Classification: Olive Brown Silty Sand with Gravel (SM)

Reviewed by: SE

Date Reviewed: 12/28/2022

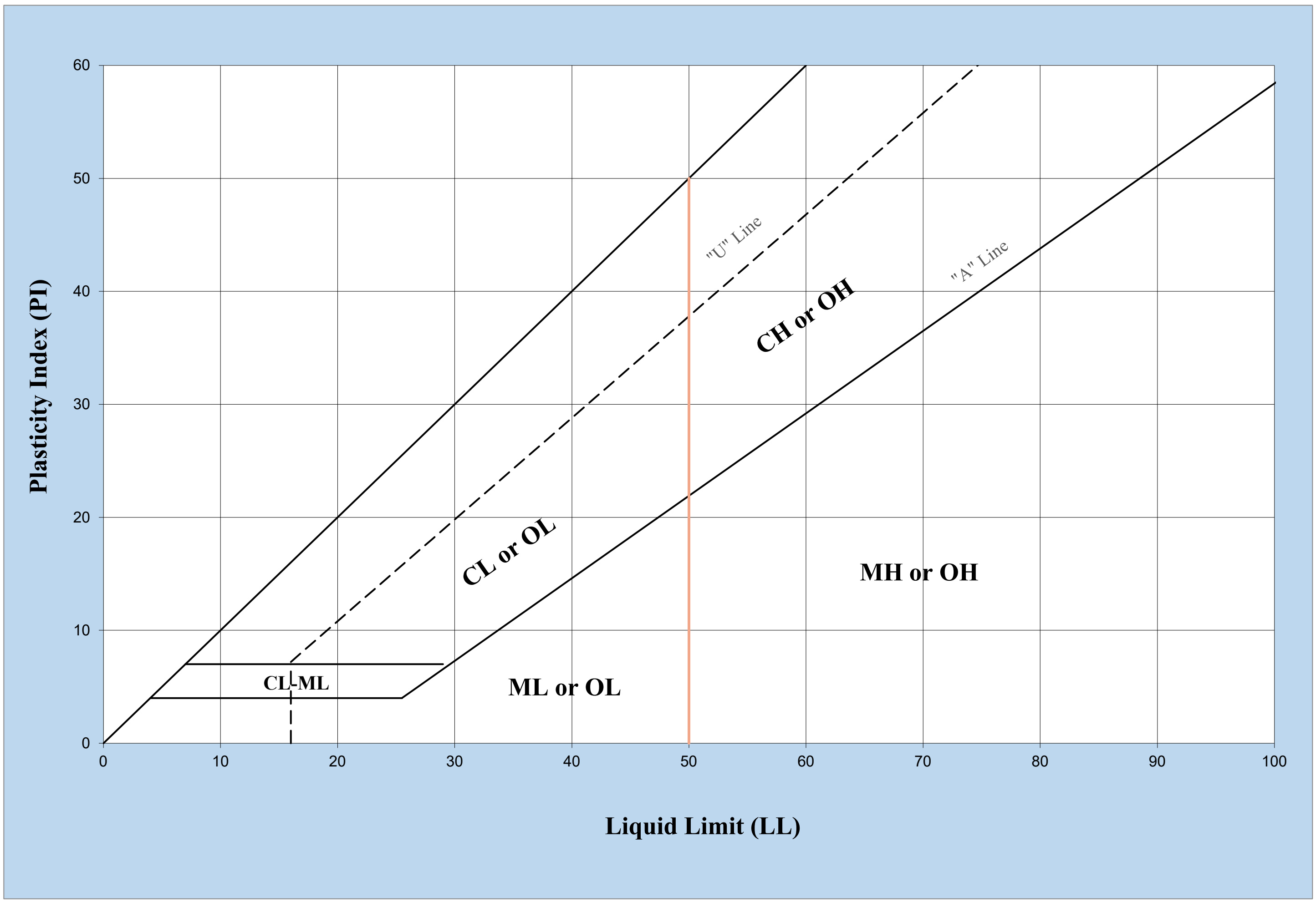
16 Technology Dr, Suite 139
Irvine, CA 92618
949 989 6941

Lab Accreditation Number:

Atterberg Limits

Job Number:	4230.2200062.0000
Client:	The Jacmar Companies
Project Name:	7035 N. Laurel Canyon Blvd.
Permit Number:	---
Material Source	Boring 3
Sample Location:	B-3 @ 7.5 ft.

Sample ID	B-3 @ 7.5 ft.
Date Received	12/19/2022
Sampled By:	AM
Date Sampled:	12/15/2022
Tested By:	SE
Test Method	D4318



Sample Point

LL: NP PL: NP PI: NP

Soil Classification: Olive Brown Silty Sand (SM)

Reviewed by: SE

Date Reviewed: 12/28/2022

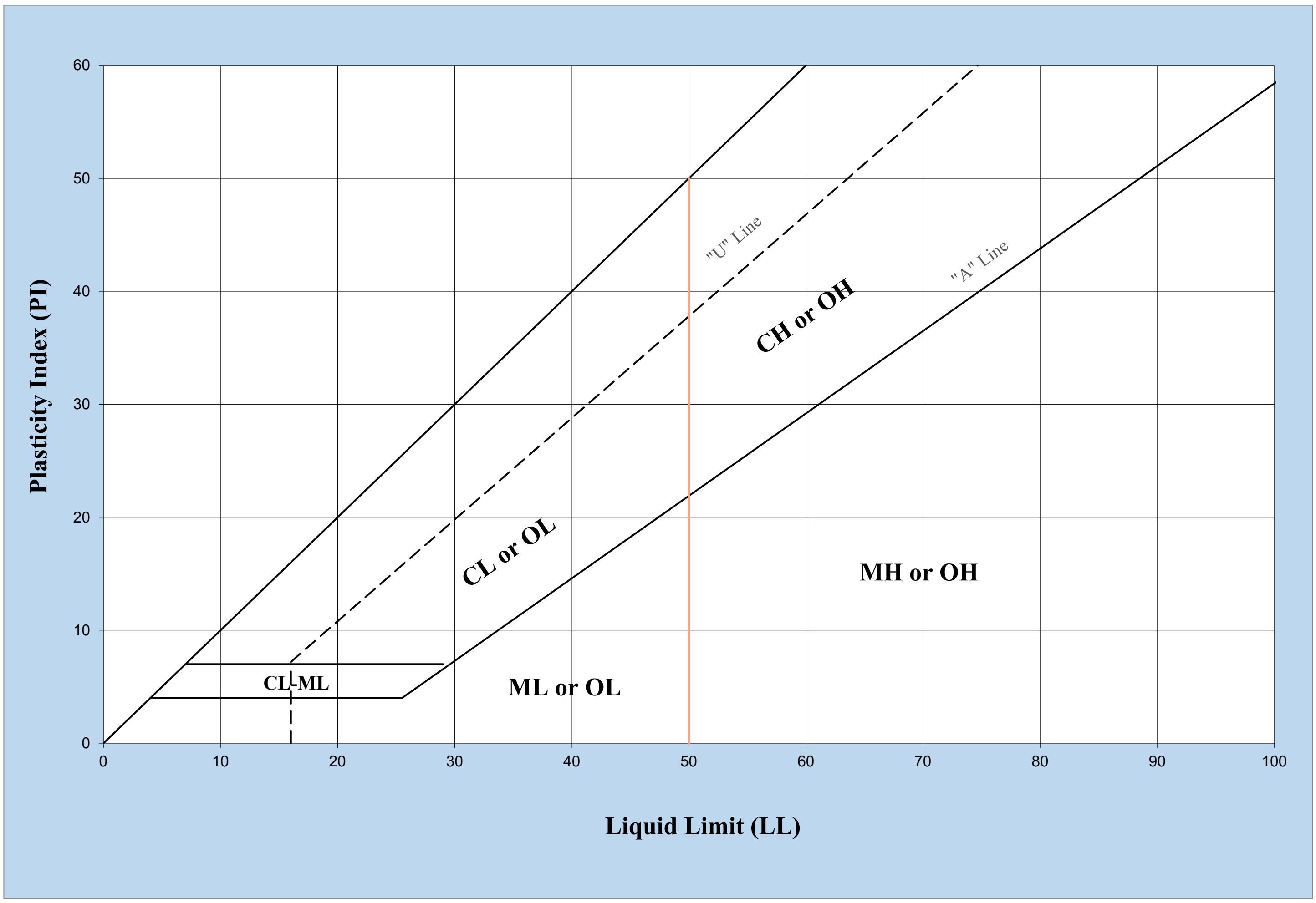
UNIVERSAL® ENGINEERING SCIENCES

16 Technology Dr, Suite 139
Irvine, CA 92618
949 989 6941

Lab Accreditation Number:

Atterberg Limits

Job Number:	4230.2200062.0000	Sample ID	B-3 @ 40 ft.
Client:	The Jacmar Companies	Date Received	12/19/2022
Project Name:	7035 N. Laurel Canyon Blvd.	Sampled By:	AM
Permit Number:	---	Date Sampled:	12/15/2022
Material Source	Boring 3	Tested By:	SE
Sample Location:	B-3 @ 40 ft.	Test Method	D4318



Sample Point

LL:	PL:	PI:
NP	NP	NP

Soil Classification: Olive Brown Poorly Graded Sand with Silt (SP-SM) *trace Gravel

Reviewed by: SE

Date Reviewed: 12/28/2022

UNIVERSAL ENGINEERING SCIENCES

Expansion Index of Soils - D4829

Project Name:	**7035 N. Laurel Canyon Blvd.**		Date Sampled:	12/16/2022
Project No.:	**4230.2200062.0000**		Sampled By:	AM
Boring No.:	B-3 Bulk		Date Tested:	1/6/2023
Sample No.:	Bulk		Tested By:	SE
Depth (ft.):	0-5'			
Sample Prep.:	Dry			
Description:	Dark Brown Silty, Clayey Sand (SC-SM) *trace Gravel			

Dry Weight of Soil + Cont. (g) *(M_t)*	1516.1
Dry Weight of Soil (g)	1516.1
Weight Soil Retained on #4 *(M_{cf})*	86.1
Percent Soil Retained on #4 *(M_{cf})*	5.4
Weight of Soil Passing *(M_p)*	1430.0
Sieve Percent Passing #4	94.6%

MOLDED SPECIMEN	Before Test	AFTER TEST
Specimen Diameter (in.)	4.00	
Specimen Height (in.) *(H_i)*	1.00	
Wt. Comp. Soil + Ring (g) *(M_{sr})*	774.14	
Wt. of Ring (g)	368.60	
Specific Gravity (Assumed)	2.60	
Ring Factor	0.3014	
Wet Wt. of Soil + Cont. (g)	155.15	
Dry Wt. of Soil + Cont. (g)	143.13	
Wt. of Container (g)	0.00	
Water Content (%) *(w)*	8.4	14.9
Wet Density (pcf)	122.2	
Dry Density (pcf)	112.8	
Degree of Saturation (%) [S meas]	49.8	

SPECIMEN INUNDATION in distilled water for the period of 24h or expansion rate < 0.0002 in./h

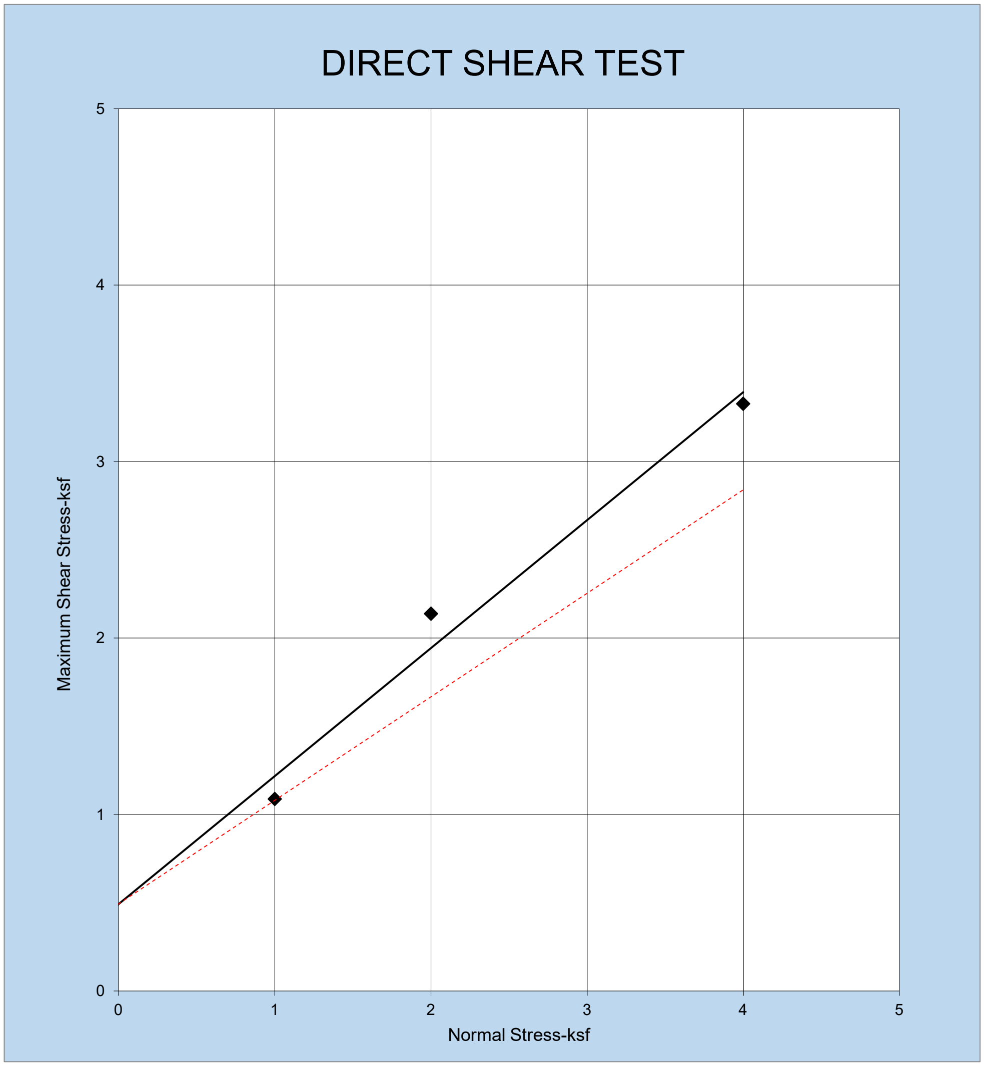
Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Reading
1/6/2023	0800	1	0	0.000
1/6/2023	0810	1	10	0.000
1/7/2023	0800	1	1440	0.0019
			2	**Very Low**

UNIVERSAL ENGINEERING SCIENCES

Lab Accreditation number:

Direct Shear Test

Job Number:	4230.2200062.0000	Sample ID:	B-1 @ 20 ft.
Client:	The Jacmar Companies	Date Received:	12/19/2022
Project Name:	7035 N. Laurel Canyon Blvd.	Tested By:	SE
Permit Number:	---	Test Method:	ASTM D3080
Material Source:	Boring 1	Sampled By:	AM
Sample Location:	B-1 @ 20 ft.	Date Sampled:	12/15/2022



Sample No.	Sample Depth (ft)	Normal Stress (ksf)	Maximum Shear Stress (ksf)	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Final Moisture Content (%)
B-1	**20**	1.0	1.09	110.3	3.1	107.0	21.3
B-1	**20**	2.0	2.14	108.0	3.1	104.8	22.1
B-1	**20**	4.0	3.33	107.6	3.1	104.4	21.1

Sample Type: Undisturbed **Test Condition:** Saturated **Sample Description:** Olive Brown Poorly Graded Sand with Silt (SP-SM)

Peak Test Results

Cohesion (psf):	490
Friction Angle (degrees):	36
Shear Rate (in/min)	0.0083

Residual Test Results

Cohesion (psf):	490
Friction Angle (degrees):	30
Shear Rate (in/min):	0.0083

Reviewed by: SE

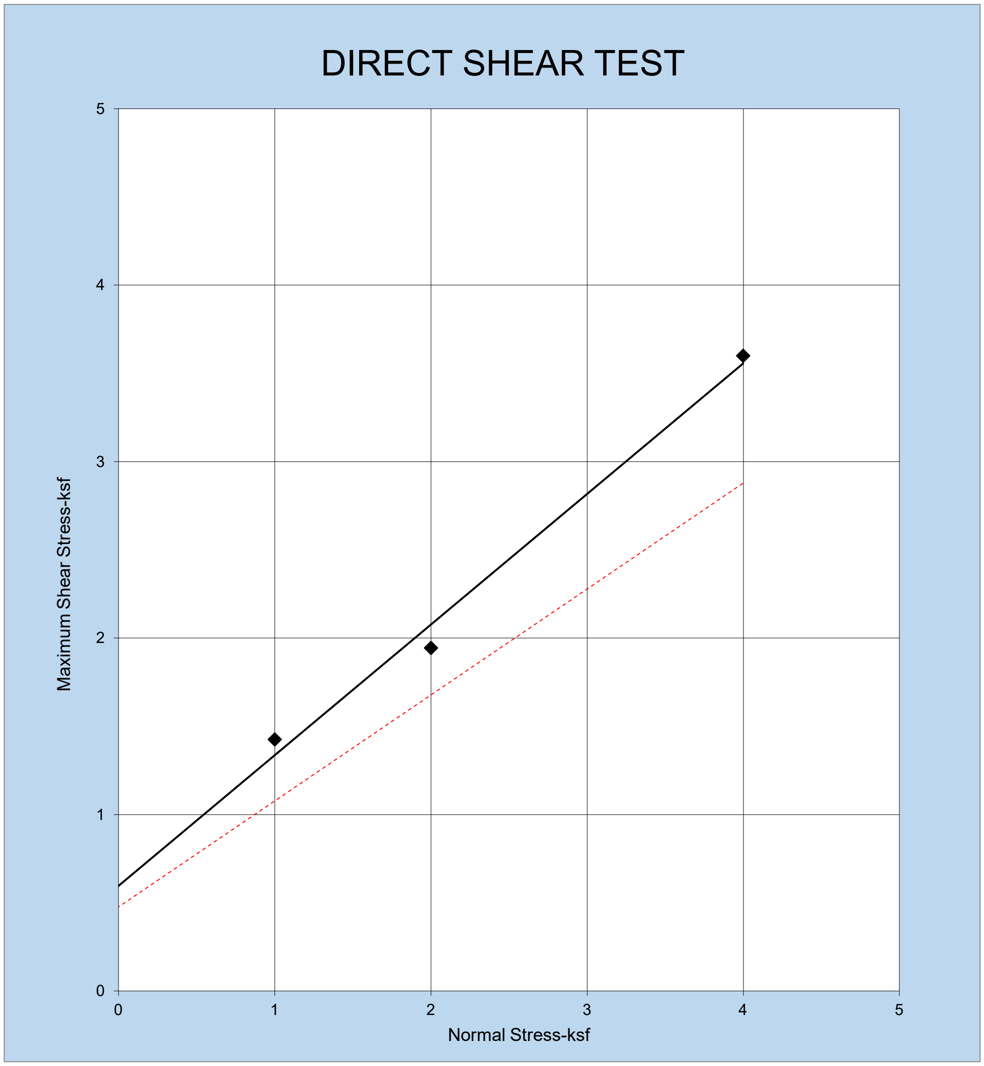
Date Reviewed: 12-26-22

UNIVERSAL ENGINEERING SCIENCES

Lab Accreditation number:

Direct Shear Test

Job Number:	4230.2200062.0000	Sample ID:	B-1 @ 30 ft.
Client:	The Jacmar Companies	Date Received:	12/19/2022
Project Name:	7035 N. Laurel Canyon Blvd.	Tested By:	SE
Permit Number:	---	Test Method:	ASTM D3080
Material Source:	Boring 1	Sampled By:	AM
Sample Location:	B-1 @ 30 ft.	Date Sampled:	12/15/2022



Sample No.	Sample Depth (ft)	Normal Stress (ksf)	Maximum Shear Stress (ksf)	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Final Moisture Content (%)
B-1	**20**	1.0	1.43	122.6	2.8	119.3	16.8
B-1	**30**	2.0	1.94	120.0	2.8	116.7	17.0
B-1	**30**	4.0	3.60	117.6	2.8	114.4	17.7

Sample Type: Undisturbed **Test Condition:** Saturated **Sample Description:** L. Olive Brown Poorly Graded Sand with Silt (SP-SM

Peak Test Results

Cohesion (psf):	600
Friction Angle (degrees):	36
Shear Rate (in/min)	0.0083

Residual Test Results

Cohesion (psf):	480
Friction Angle (degrees):	31
Shear Rate (in/min):	0.0083

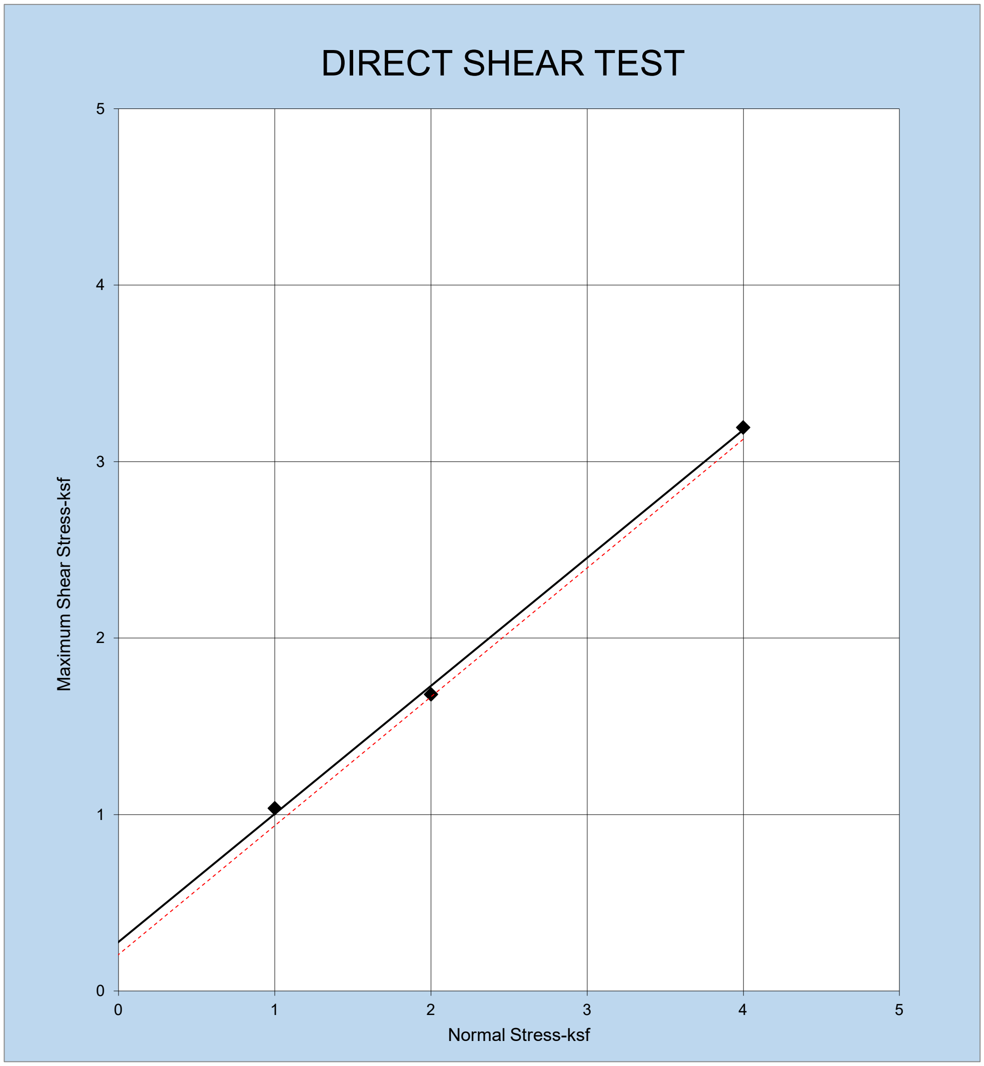
Reviewed by: SE

Date Reviewed: 12-28-22

Lab Accreditation number:

Direct Shear Test

Job Number:	4230.2200062.0000	Sample ID:	B-3 @ 25 ft.
Client:	The Jacmar Companies	Date Received:	12/19/2022
Project Name:	7035 N. Laurel Canyon Blvd.	Tested By:	SE
Permit Number:	---	Test Method:	ASTM D3080
Material Source:	Boring 3	Sampled By:	AM
Sample Location:	B-3 @ 25 ft.	Date Sampled:	12/15/2022



Sample No.	Sample Depth (ft)	Normal Stress (ksf)	Maximum Shear Stress (ksf)	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Final Moisture Content (%)
B-3	**25**	1.0	1.03	112.7	2.8	109.6	19.3
B-3	**25**	2.0	1.68	116.5	2.8	113.3	17.3
B-3	**25**	4.0	3.19	114.8	2.8	111.7	17.4

Sample Type: Undisturbed **Test Condition:** Saturated **Sample Description:** L. Olive Brown Poorly Graded Sand with Gravel (SP

Peak Test Results

Cohesion (psf):	280
Friction Angle (degrees):	36
Shear Rate (in/min)	0.0083

Residual Test Results

Cohesion (psf):	210
Friction Angle (degrees):	36
Shear Rate (in/min):	0.0083

Reviewed by: SE

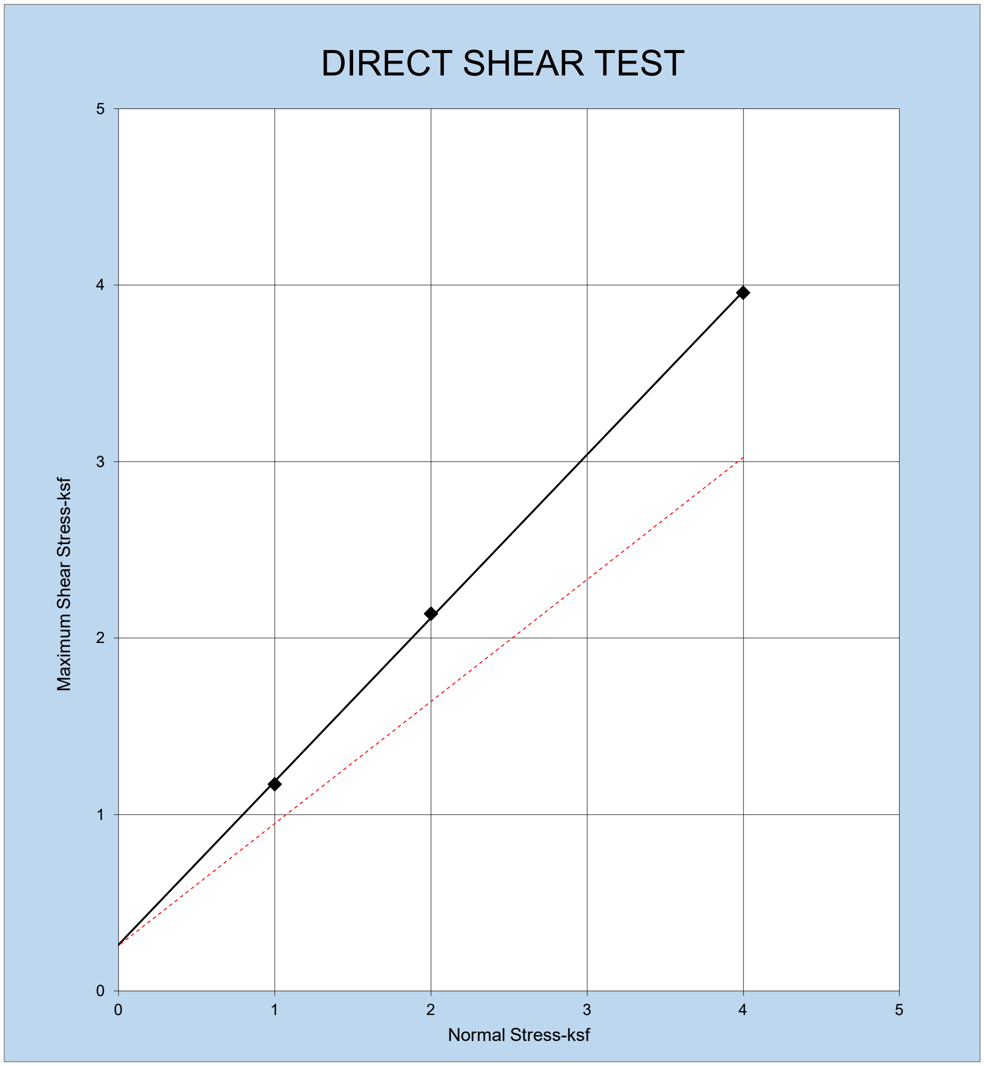
Date Reviewed: 12-28-22

UNIVERSAL ENGINEERING SCIENCES

Lab Accreditation number:

Direct Shear Test

Job Number:	4230.2200062.0000	Sample ID:	B-4 @ 15 ft.
Client:	The Jacmar Companies	Date Received:	12/19/2022
Project Name:	7035 N. Laurel Canyon Blvd.	Tested By:	SE
Permit Number:	---	Test Method:	ASTM D3080
Material Source:	Boring 4	Sampled By:	AM
Sample Location:	B-4 @ 15 ft.	Date Sampled:	12/15/2022



Sample No.	Sample Depth (ft)	Normal Stress (ksf)	Maximum Shear Stress (ksf)	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Final Moisture Content (%)	
B-4	**15**	1.0	1.17	108.6	3.6	104.8	20.2	
B-4	**15**	2.0	2.14	111.7	3.6	107.8	19.1	
B-4	**15**	4.0	3.96	113.5	3.6	109.6	17.3	

Sample Type: Undisturbed **Test Condition:** Saturated **Sample Description:** L. Brown Poorly Graded Sand (SP) *trace Gravel

Peak Test Results

Cohesion (psf):	260
Friction Angle (degrees):	43
Shear Rate (in/min)	0.0083

Residual Test Results

Cohesion (psf):	260
Friction Angle (degrees):	35
Shear Rate (in/min):	0.0083

Reviewed by: SE

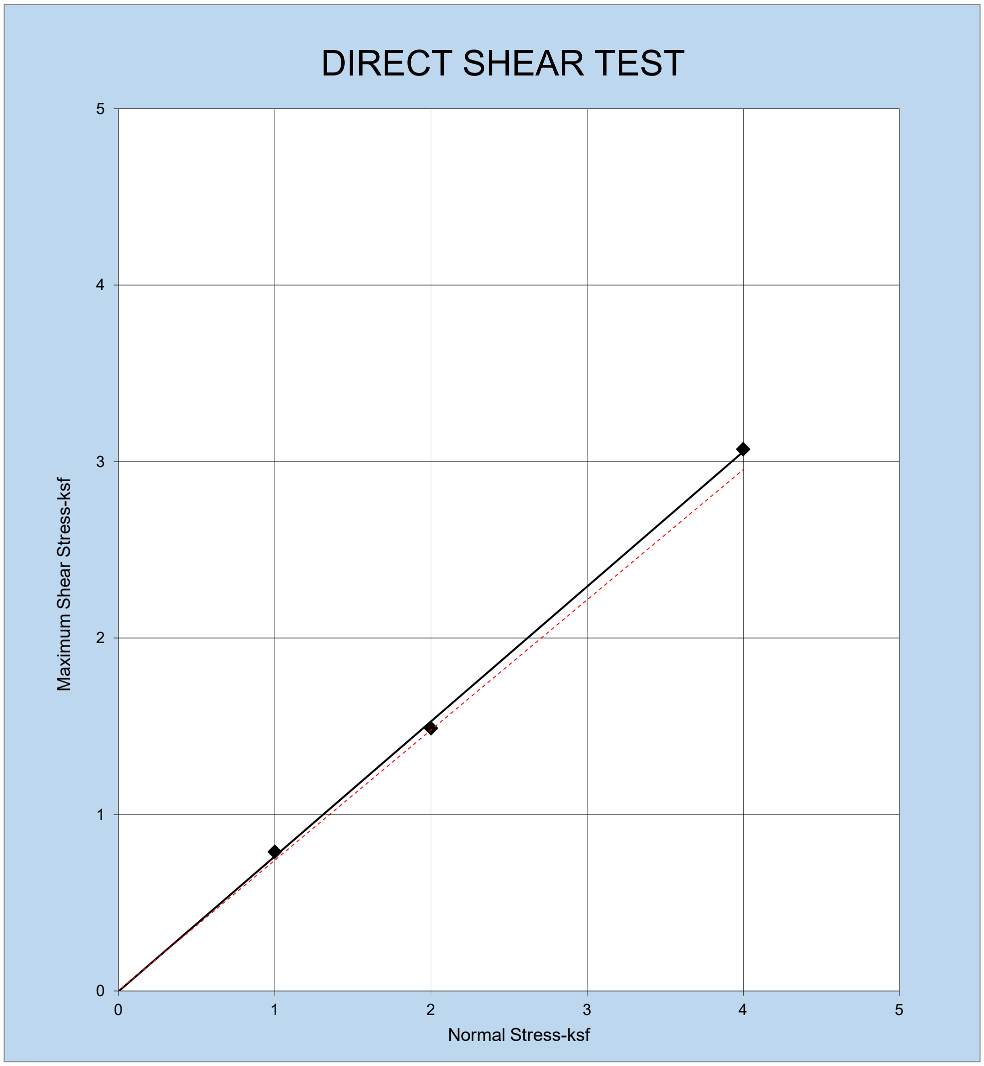
Date Reviewed: 12-28-22

UNIVERSAL® ENGINEERING SCIENCES

Lab Accreditation number:

Direct Shear Test

Job Number:	4230.2200062.0000	Sample ID:	B-5 @ 5 ft.
Client:	The Jacmar Companies	Date Received:	12/19/2022
Project Name:	7035 N. Laurel Canyon Blvd.	Tested By:	SE
Permit Number:	---	Test Method:	ASTM D3080
Material Source:	Boring 5	Sampled By:	AM
Sample Location:	B-5 @ 5 ft.	Date Sampled:	12/15/2022



Sample No.	Sample Depth (ft)	Normal Stress (ksf)	Maximum Shear Stress (ksf)	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Final Moisture Content (%)
B-5	**5**	1.0	0.79	102.6	9.9	93.4	27.4
B-5	**5**	2.0	1.49	106.6	9.9	97.0	25.5
B-5	**5**	4.0	3.07	103.7	9.9	94.4	22.9

Sample Type: Undisturbed **Test Condition:** Saturated **Sample Description:** Olive Brown Poorly Graded Sand with Silt (SP-SM)

Peak Test Results

Cohesion (psf):	0
Friction Angle (degrees):	37
Shear Rate (in/min)	0.0083

Residual Test Results

Cohesion (psf):	0
Friction Angle (degrees):	36
Shear Rate (in/min):	0.0083

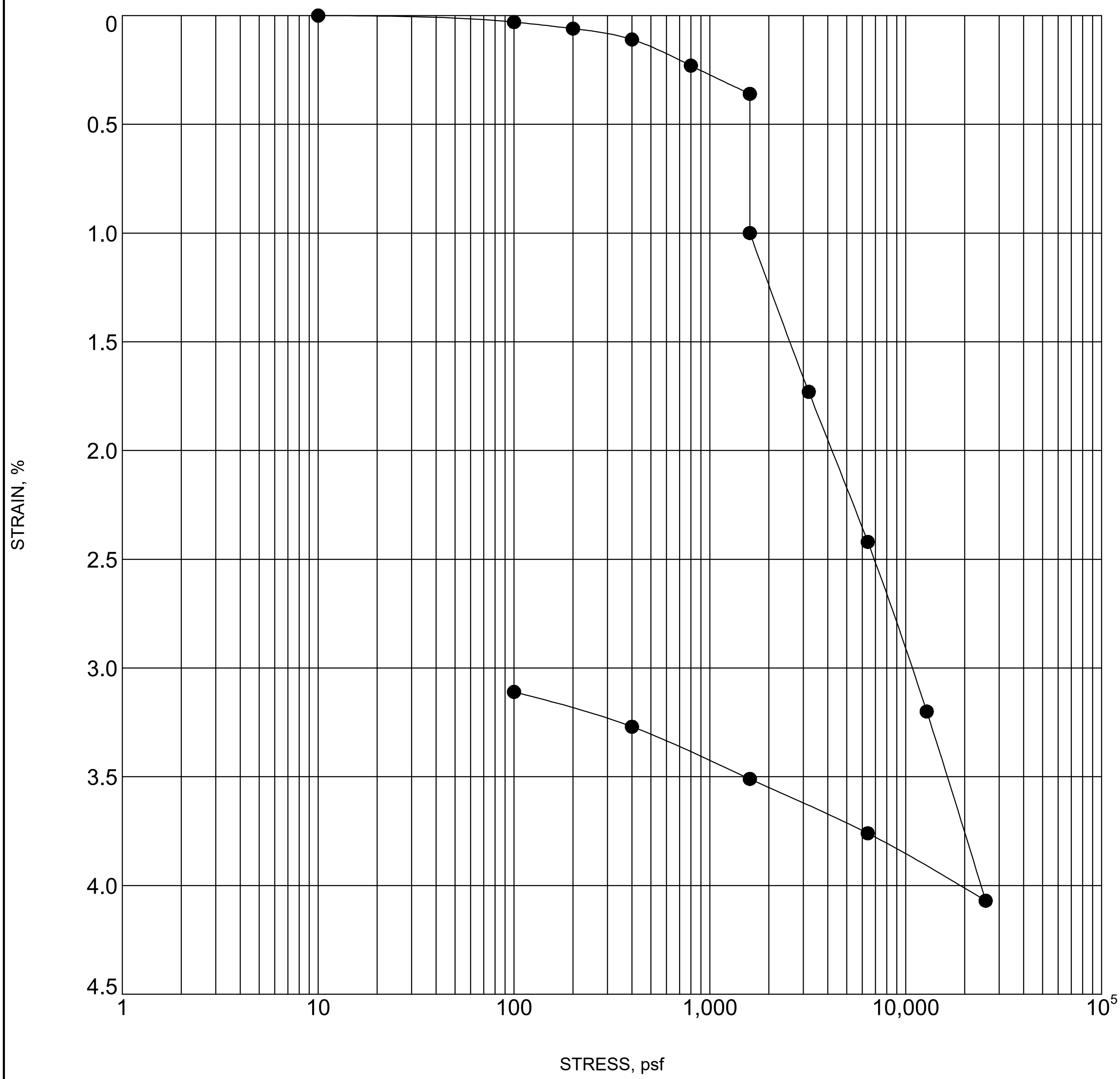
Reviewed by: SE

Date Reviewed: 12-28-22

Universal Engineering Sciences
16 Technology Dr.,Ste 139
Irvine, CA 92618
Telephone: 949-537-3222

CONSOLIDATION TEST

CLIENT The Jacmar Companies
PROJECT NAME 7035 N. Laurel Canyon Blvd.
PROJECT NUMBER 4230.2200062.0000
PROJECT LOCATION 7035 N Laurel Canyon Blvd, North Hollywood, CA



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-1**	**30.0**	**(SP-SM) Poorly graded SAND with silt, light olive brown**	**109.3**	**2.8**

CONSOL STRAIN - GINT STD US LAB.GDT - 1/10/23 11:50 - C:\USERS\SDARYAEI\ONEDRIVE - UNIVERSAL ENGINEERING-TEAM UES\DESKTOP\4230.2200062.0000 7035 N LAUREL CANYON BLVD.GPJ

Universal Engineering Sciences
16 Technology Dr.,Ste 139
Irvine, CA 92618
Telephone: 949-537-3222

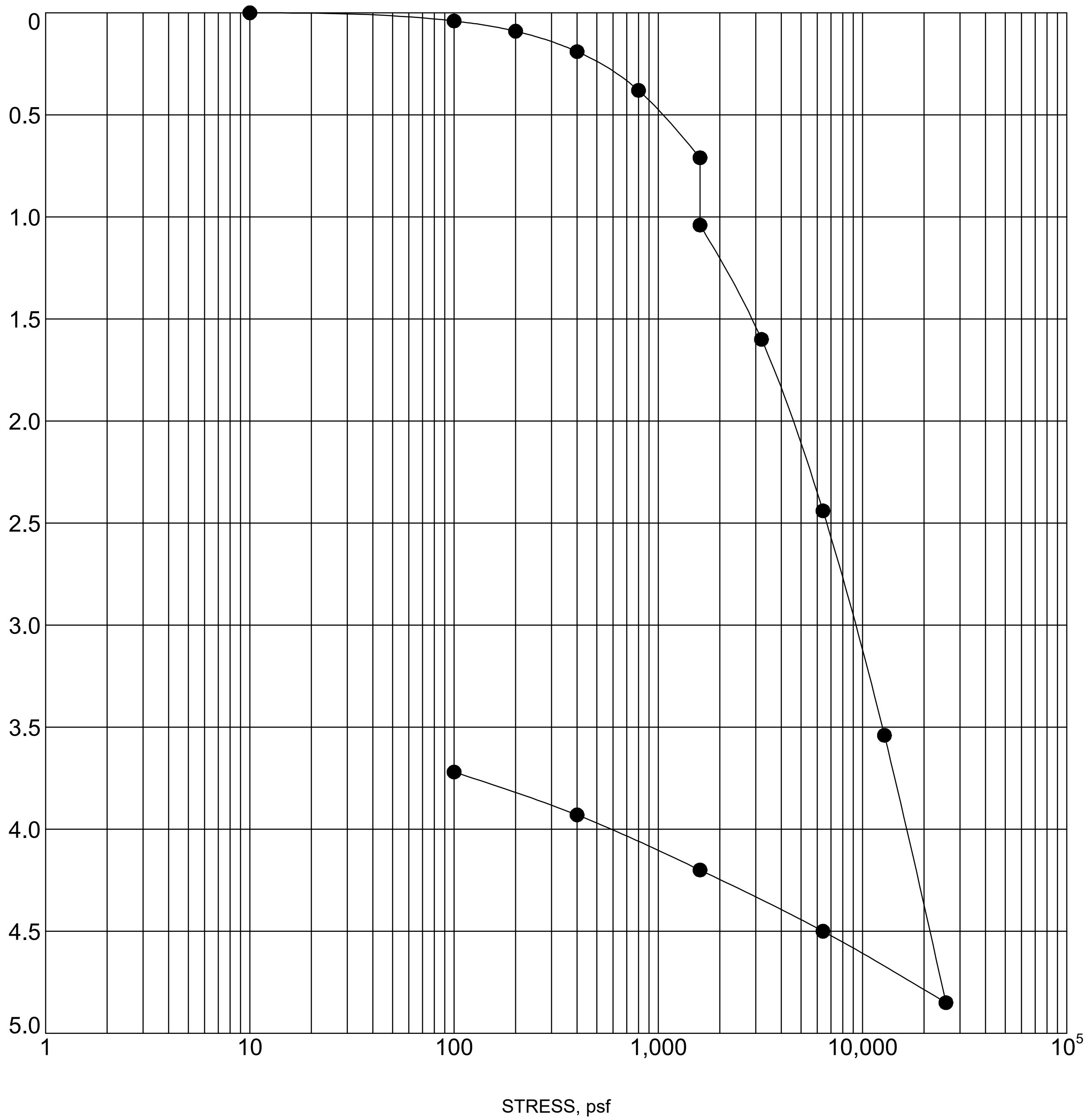
CONSOLIDATION TEST

CLIENT The Jacmar Companies

PROJECT NAME 7035 N. Laurel Canyon Blvd.

PROJECT NUMBER 4230.2200062.0000

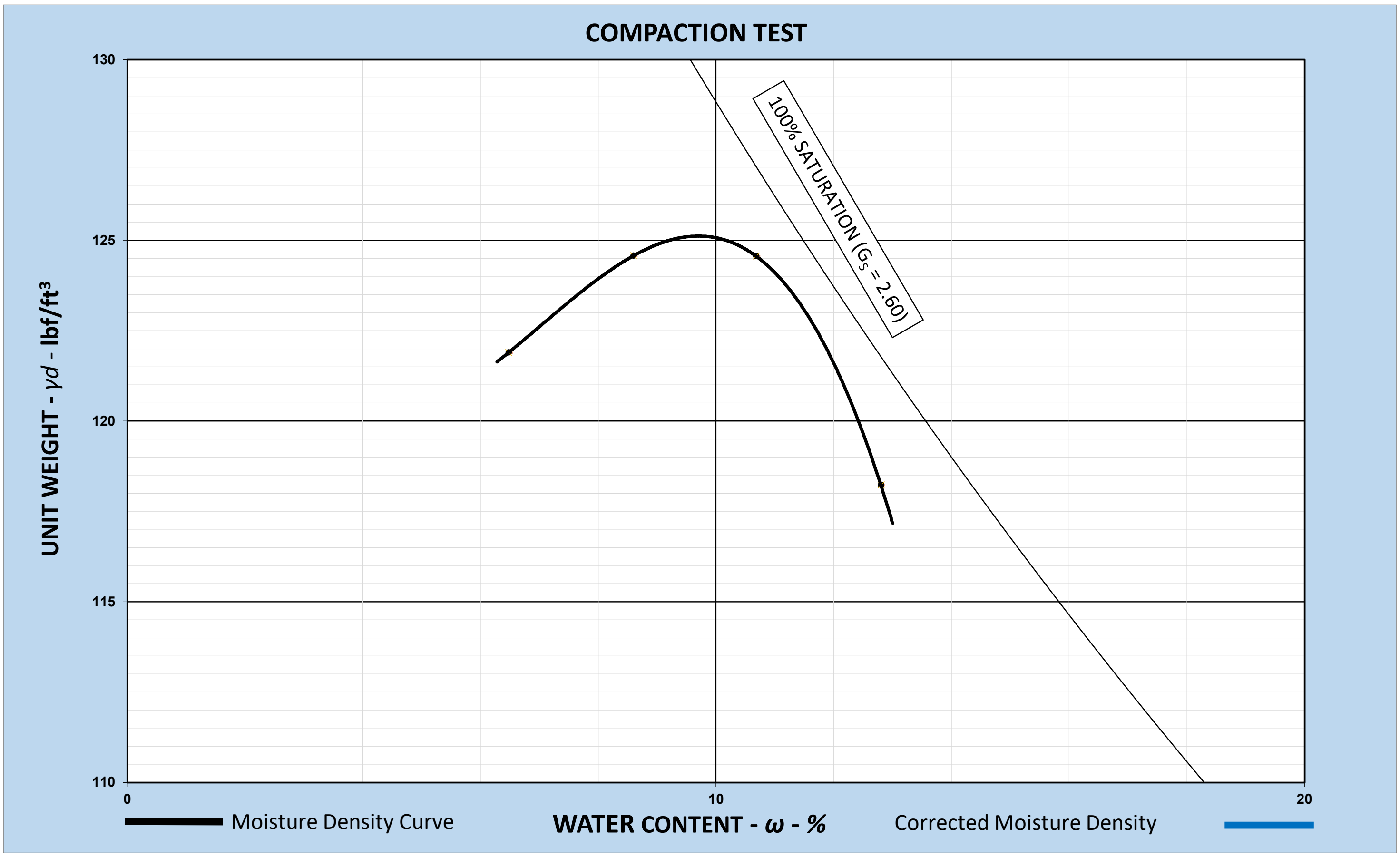
PROJECT LOCATION 7035 N Laurel Canyon Blvd, North Hollywood, CA



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-3**	**25.0**	**(SP) Poorly graded SAND, light olive brown**	**109.5**	**2.6**

Laboratory Compaction Characteristics of Soil Using Modified Effort - D1557

Job Number:	4230.2200062.0000	Sample ID:	B-3 Bulk
Client:	The Jacmar Companies	Date Received:	12/19/2022
Project Name:	7035 N. Laurel Canyon Blvd	Sampled By:	AM
Permit Number:	---	Date Sampled:	12/16/2022
Material Source:	Boring 3	Tested By:	SD
Sample Location:	B-3 Bulk	Test Method and Procedure:	ASTM D1557 B
Depth:	0-5 ft.	Preparation Method:	Moist
		Type of Rammer:	Mechanical



Maximum Dry Unit Weight	**125.1 pcf**
Optimum Water Content	**9.7 %**

Specific Gravity Used For Zero Air Voids Curve	2.60	% Fraction Retained	3.4
Sieve Used	3/8"	% Fraction Passing	96.6

USCS Description: Dark Brown Silty, Clayey Sand (SC-SM) *trace Gravel

Reviewed by: SE

Date Reviewed: 12/29/2022

Soil Analysis Lab Results

Client: Universal Engineering
Job Name: 7035 N. Laurel Canyon Blvd.
Client Job Number: 4230.2200062.0000
Project X Job Number: S221228A
January 3, 2023

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)	
B-3 SPT	20.0	26.0	0.0026	29.1	0.0029	214,400	30,820	7.1

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)