Appendix F

Geotechnical Investigation

Kimley»Horn

GEOTECHNICAL INVESTIGATION
PROPOSED DATA CENTER FACILITY
1977 SATURN STREET
MONTEREY PARK, LOS ANGELES COUNTY
CALIFORNIA

Prepared For **SDCF MONTEREY PARK, LLC**
660 STEAMBOAT ROAD
GREENWICH, CONNECTICUT 06830

Prepared By **LEIGHTON CONSULTING, INC.**
2600 MICHELSON DRIVE SUITE 400
IRVINE, CALIFORNIA 92612

Project Number 19850

March 27, 2024

Revised June 11, 2024

Leighton
a verdantas company

Leighton Consulting, Inc.

March 27, 2024

Revised June 11, 2024

Project No. 19850

SDCF Monterey Park, LLC
660 Steamboat Road
Greenwich, Connecticut 06830

Attention: Mr. Bryan Marsh, Chief Executive Officer

Subject: Geotechnical Investigation
Proposed Data Center Facility
1977 Saturn Street
Monterey Park, Los Angeles County, California

Per our October 11, 2023 proposal, Leighton Consulting, Inc. (Leighton) is pleased to present this geotechnical investigation report for the subject project. This report is intended to meet the requirements of Section 1803A.2 of the 2022 California Building Code (CBC). The purpose of our geotechnical investigation has been to evaluate the geologic and geotechnical conditions, including potential geologic hazards, within the area of and as they relate to the proposed improvements, and to provide geotechnical recommendations for design and construction of the proposed data center facility, equipment/generator yard and electrical substation.

This site is **not** located within a currently designated Alquist-Priolo Special Studies Zone for surface fault rupture. This site is also **not** within a currently designated liquefaction hazard zone. However, as is the case for most of Southern California, strong ground shaking has a strong potential to occur at this site.

This report presents our findings and conclusions regarding this project. Based upon our geotechnical investigation, the proposed improvements are feasible from a geotechnical viewpoint, provided our recommendations are incorporated into the design and construction of the project. The most significant geotechnical issues are shallow potentially compressible soils underlying the site and the potential for strong seismic shaking. These and other geotechnical issues are discussed in this report. Specific recommendations for site grading, foundations and other geotechnical aspects of the project are presented in this report.

We appreciate this opportunity to be of service. If you have any questions regarding this report or if we can be of further service, please call us at your convenience at **(866) *LEIGHTON***, directly at the phone extensions or e-mail addresses listed below.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Site Description

The proposed data center facility, see Figure 1, *Site Location Map (*Latitude N34.0402°, Longitude W-118.1140°) is to encompass the majority of the footprint of the existing industrial building formerly operating as Saturn Electronics, as well as a portion of the adjacent landscaped area to the west of the existing structure. The site is bordered by Saturn Street to the south and southwest, South Orange Avenue to the east, , industrial buildings to the west and southeast-southwest, La Loma Reservoir and La Loma Park to the northwest, and residential developments to the northeast and west-northwest.

Based on review of available in-house geotechnical reports, the site was rough graded in 1978 under the geotechnical observation and testing of Leighton and Associates, Inc. (1978). The project site is formerly denoted as Parcels 1 through 5 of Parcel Map 10094 and Lots 1 and 2 of Tract 33910 (Leighton, 1977). See Figure 2A, *Remedial Grading Map* for lot designation and underlying geologic conditions observed and interpreted during grading and exploration of the site (Leighton, 1977, 1978).

Aerial Image Review: Review of aerial imagery and early topographic maps spanning the timeframe from 1939 to 2020 (NETR, 2023) indicate the site is primarily cut with former site elevations (El.) ranging from approximately El. 350 feet to El. 400 feet. 1939 topographic profiles (5-foot contour interval) indicated a north trending blue line canyon axis bisects the current building footprint. Elevations of this canyon onsite and below the current structure range from approximately El. 340 to El. 360 feet. The former canyon axis is roughly aligned in the approximate center of the existing building. This canyon was filled during prior grading activities at the site (Donald Warren 1971 and Leighton, 1978). Visible in the 1964 aerial image is one water tank on the hill to the northwest and grading activity south of the current site location was underway. 1972 aerial imagery shows a portion of the site and adjacent areas are being graded. By 1980 the canyon had been filled and the current structure constructed.

1.2 Proposed Development

Based on review of the 50-scale conceptual grading plan plotted December 20, 2023 and prepared by Gensler, the proposed development will consist of an approximately 218,000± square foot 1-story rectangular building located on the southern portion of the site along Saturn Street, with an adjacent equipment yard directly north of the building. A western and eastern entryway connecting the development to Saturn Street are proposed, with a fire lane and wrapping around the building and equipment yard. Parking stalls are planned to be located to the north and east of the building and equipment yard. An electrical substation is also planned north of the equipment yard. See Figure 2A, *Boring Location Map* for overlay of the proposed development on current site imagery.

We understand the facility is designed to provide uninterrupted power and cooling services to support the continuous operation and maintenance of servers used for computational processing. The building is organized as Data Hall Galleries flanked by housing cooling units and fans. Each Data Hall is supported by two Electrical Rooms which house electrical gear, panels and batteries. The facility will feature two loading docks, one at each end of the building, for trailer trucks with an area for staging and secure storage of new equipment. Additional spaces will be used for a secure vestibule and lobby, security, two to three fiber entry vaults and fiber cross connect rooms each with diverse optical fiber feed, building restrooms, storage and open office area for facilities and maintenance staff. Electrical gear such as switches and transformers will be in the open yard along with Diesel powered generators.

The conceptual grading plan shows a building Finished Floor Elevation (FFE) of 359.00 feet above mean sea level (msl). Based on existing site grades, the proposed building footprint will require approximately 8 feet of fill on the west end and 11 feet of cut on the east end to reach proposed building grades. Retaining associated with the proposed building pad are currently planned towards the southwest corner of the building with a maximum height of 6 feet and ascending slope towards the building, and towards the southeast corner of the building with a maximum height of 4 feet and descending slope towards the building.

Preliminary structural loads for the proposed building include maximum axial dead loads of 100 kips and live loads of 50 kips for interior columns, and maximum axial dead loads of 50 kips and live loads of 25 kips for the perimeter columns. Exterior perimeter wall loads are estimated as 4 kips per linear foot. Brace frame footings include additional axial loads of 200 kips due to seismic loading.

Retaining walls with maximum heights of 21 feet are currently planned near the northern and northeastern extents of the site. These walls would be primarily a cut condition from existing grades. Retaining walls are not currently designed, however we anticipate that they would be composed of a soil-nail system.

1.3 Previous Site Background

Leighton and Associates Inc., 1977: Leighton performed geotechnical review of grading plans prepared for Parcels 1 through 5 of Parcel Map 10094 and Lots 1 to 13 of Tract 33910 in support of summarizing the geologic and soil engineering properties of then proposed 18-Lot industrial park development. Based on that review, standard cut and fill grading techniques were planned to construct 13 level lots for industrial building construction. Grading entailed construction of 1.5:1 (horizontal:vertical) with cut slopes to 65 feet in height and 1.5:1 fill slopes a maximum of 32 feet in height. Test pit explorations were conducted onsite in support of the exploration. Site materials were characterized as a mixture of clayey silty sands, sandy clays to silts with sand and gravels noting cobbles and boulders were rare. Alluvium in the main drainage obtained a maximum thickness of four feet. Applicable data from this report is included in Appendix B, *Explorations.* Locations of these test pits are shown on Figure 2A.

Proposed cut slopes were anticipated to be stable as designed or buttressed. The keyway planned for the slope on former Lot 5 (Figure 2A) was recommended being at least 15 feet wide and two feet deep inclined to the heel of the drained key.

Leighton and Associates Inc., 1978: Leighton provided geotechnical and geologic observation and testing during grading of the site which commenced on July 10, 1978, and was completed on October 5, 1978. Standard removals of an average depth of three feet removals from the drainage north of Lot 2 (Figure 2A). Fill was placed and compacted to a standard of not less than 90 percent relative compaction. Field density tests were recorded and presented on maps used as the base for Figure 2A in this report. Earth materials observed and interpreted as sedimentary bedrock consisting of sandstone (ss), siltstone (slt) and conglomerate (cg) were reported. Distribution of earth units is shown on Figure 2B. No landslides or major faulting were observed during grading operations. Buttress keyway and subdrains were constructed during grading and the approximate location of these are shown on Figure 2B, *Remedial Grading Map.* The subdrain installed in the north trending canyon north of Lot 2 (Figure 2A) was reported as an 8-inch subdrain tied into the storm drain system offsite. Results of the field density testing and maps showing plotted locations of these tests are included in Appendix A, *Report of Rough Grading.*

Leighton Consulting Inc., 2015: Leighton provided a geotechnical exploration report for the proposed cooling tower addition project at 1980 Saturn Street in Monterey Park, California, immediately southwest of the project site. Field exploration was performed on October 23, 2015, and consisted of drilling, sampling and logging of one hollow-stem auger boring excavated to a depth of 35 feet below existing ground surface (bgs) adjacent to the proposed cooling tower pad. Subsurface soils encountered in the boring drilled for this investigation indicate the site is underlain by approximately 15 feet of compacted artificial fill overlying sedimentary bedrock of the Fernando Formation. The compacted artificial fill soils encountered generally consist of olive brown to reddish brown, slightly moist to moist, clayey sand and sandy clay with variable amounts of fine gravels. Bedrock materials encountered to the depth explored generally consist of reddish brown, slightly moist to moist, gravelly sandstone and conglomerate sandstone with variable amounts of clay within the matrix. Based upon the conditions encountered and results of geotechnical laboratory testing recommendations were provided to support the construction of the cooling tower pad as then proposed. The boring excavated in support of this project is located offsite to the southwest. The approximate location is shown on Figure 2A and the boring log is included in Appendix B.

1.4 Purpose and Scope of Exploration

The purpose of our geotechnical exploration was to evaluate the soil and groundwater conditions at the site through review of available data and subsurface explorations in order to provide geotechnical recommendations to aid in design and construction for the project as currently proposed (see Section 1.2). The scope of this geotechnical exploration included the following tasks:

- *Background Review* – A background review was performed of readily available, relevant geotechnical, civil and geological literature pertinent to the project site. References reviewed in preparation of this report are listed in Section 7.0.

- *Field Exploration* – Field exploration was performed on December 5, 2023, and consisted of drilling, sampling, and logging of five (5) hollow-stem auger borings (designated LB-1 through LB-5). Borings were excavated to depths ranging from approximately 10 feet to 36.5 feet below existing ground surface (bgs) within the footprint of the proposed building as well as the preferred substation location.

Prior to the field exploration, the borings were marked, and Underground Service Alert (USA) was notified for utility clearance. During excavating, bulk and relatively undisturbed samples were obtained from each boring for geotechnical laboratory testing. The borings were logged in the field by a geologist from our staff. Each soil sample collected was reviewed and described in accordance with the Unified Soil Classification System (USCS). The samples were sealed and packaged for transportation to our laboratory. After completion of drilling, the boring was backfilled with soil cuttings generated during the exploration, and surfaces were patched with cold-mix asphalt. The approximate locations of the borings conducted by Leighton are shown on Figure 2A, *Boring Location Map* and Figure 2B, *Remedial Grading Map*. Boring logs are included in Appendix B.

- *Laboratory Testing* – Geotechnical laboratory tests were conducted on selected bulk and undisturbed soil samples obtained from our borings. This laboratory testing program was designed to evaluate geotechnical (physical) characteristics of site soil. A description of geotechnical laboratory test- results are presented in Appendix C, *Laboratory Test Results*. The following laboratory tests were performed:

  — In-situ Moisture Content and Dry Density (ASTM D2216 and ASTM D2937);

  — Expansion Index (ASTM D4829);

  — Modified Proctor Compaction Test (ASTM D1557);

  — Atterberg Limits (ASTM D4318);

  — R Value (DOT CA Test 301);

  — Particle Size Analysis (ASTM D6913);

  — Percentage Passing Sieve No. 200 Sieve (ASTM D1140);

  — Swell or Settlement Potential (ASTM D4546); and

  — Corrosivity (Soluble Sulfate DOT CA Test 417 Part II, Soluble Chloride DOT CA Test 422, pH DOT CA Test 643, and Resistivity DOT CA Test 643).

  The in-situ moisture and density of soil samples at depths are shown on the boring logs included in Appendix B. The results of the remaining laboratory tests are presented in Appendix C.

- *Engineering Analysis* – Data obtained from field explorations and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide recommendations for the project. Geologic cross section interpretations prepared for this site are presented on Plate 2, *Geotechnical Cross Section AA-BB' and CC''*.

- *Report Preparation* - Results of our geologic hazards review and geotechnical exploration have been summarized in this report, presenting our findings, conclusions and geotechnical design recommendations for design and construction of the project as currently proposed.

It should be noted that the recommendations in this report are subject to the limitations presented in Section 6.0 of the report.

2.0 GEOTECHNICAL FINDINGS

2.1 Local Geologic Units and Subsurface Conditions

Presented below are brief descriptions of the geologic units encountered in the exploratory borings completed at the site by Leighton. Detailed descriptions of the geologic units encountered are presented on the boring logs in Appendix B. Geotechnical conditions described on the logs represent the conditions at the actual exploratory excavation locations. Other variations may occur beyond and/or between the excavations. Lines of demarcation between the geologic units and the various earth materials on the logs represent approximated boundaries, and (unless otherwise noted) actual transitions may be gradual. The locations of the subsurface explorations are shown on Plate 1, Figure 2A and 2B. The subsurface profile based on data obtained and interpreted from the borings and available as-built maps (Appendix A) is shown on Plate *2.* The site and surrounding geologic units are presented on Figure 3, *Regional Geology Map*

Artificial Fill Engineered (Map Symbol Afe-1978): As indicated above in Section 1.2, the site was previously rough graded under the geotechnical observation and testing of Leighton and Associates, Inc. (1978). The fill material is locally derived from canyon alluvium and weathered bedrock of the Fernando Formation. These materials are characterized as Silty Sand (SM) with traces of gravel, to Sandy Clayey Silt (CL-ML) to Sandy Clay (CL) with fine to coarse sand to poorly graded Sand with Clay and gravels (SP-CL). Maximum density (Leighton, 1978) and optimum moisture for the fill placed during 1978 indicate optimum moisture content ranging from 8 to 10% and maximum dry density ranging from 125 to 129 pounds per cubic foot (pcf).

Artificial Fill Engineered (Map Symbol Afe-1991): Prior to 1978, we understand portions of the site were used as borrow areas during grading of adjacent areas. It was reported that fill placed on the site during grading of the industrial site to the south occurred in two phases (1961 and 1972) and was controlled by Donald R. Warren and Company for which there is no testing documentation. Based on review of available geotechnical reports for the site (Leighton and Associates, Inc., 1977 and 1978), the site is underlain by to (2) generations of compacted artificial fill with the greatest thickness primarily within a narrow canyon overlying non marine sandstone and conglomerate bedrock of the Fernando Formation. This early generation of fill material interpreted as encountered in current boring LB-2 (Appendix B) is characterized as Clay with Sand (CL-SP) and Silty Clay (ML-CL). Based on recovered data it appears the early generation of fill is primarily confined to the narrow canyon and might be associated with construction of a reported 8-inch diameter vitrified clay pipe (VCP) sewer line, see Figure 2A.

Quaternary Alluvium (Map Symbol Qa): Alluvium, as encountered is interpreted to be limited to the narrow north trending canyon drainage underlying the eastern corner of the existing 2-story building and adjacent parking stalls. This material is overlain by engineered fill and is characterized as dark gray to reddish brown, slightly moist to moist, clay, laminated sandy clay, and poorly-graded sand. The stratigraphy of the subsurface soils encountered in each soil boring is presented in the boring logs (Appendix B). The general subsurface conditions across the site, interpreted from the boring and test pit data (Leighton, 1977) are shown on Plate 2. Three feet of this material was recommended for removal in prior reports prepared by Leighton (1977).

Fernando Formation (Tfsc): The site is geologically mapped as the Fernando Formation (Tfsc), which is described as nonmarine sandstone and conglomerate. The conglomerate within this formation is typically composed of pebbles and cobbles in a friable sandstone matrix. This geologic formation was encountered in all of our geotechnical borings at variable depths, with the shallowest towards the northern portion of the site. Encountered soils within this formation consisted of medium dense to very dense sandstone, which generally classified as Silty Sand.

2.2 Corrosion

Corrosion: In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor for ferrous corrosivity. Based on findings of studies presented in the American Society for Testing and Materials (ASTM) STP 1013 titled “Effects of Soil Characteristics on Corrosion” (February, 1989), an approximate relationship between soil resistivity and soil corrosiveness was developed as shown in Table 1A below.

Table 1A - Soil Corrosivity as a Function of Resistivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very severe corrosion
900 to 2,300	Severely corrosive
2,300 to 5,000	Moderately corrosive
5,000 to 10,000	Mildly corrosive
10,000 to >100,000	Very mildly corrosive

Sulfate Exposure: Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. A potentially high sulfate content could also cause corrosion of reinforcing steel in concrete. Section 1904A of the 2022 California Building Code (CBC) defers to the American Concrete Institute's (ACI's) ACI 318-14 for concrete durability requirements. Table 19.3.1.1 of ACI 318-14 lists "*Exposure categories and classes*," including sulfate exposure as follows:

Table 1B - Sulfate Concentration and Exposure

Soluble Sulfate in Water (parts-per-million)	Water-Soluble Sulfate (SO_4) in soil (percentage by weight)	ACI 318-14 Sulfate Class
0-150	0.00 - 0.10	S0 (negligible)
150-1,500	0.10 - 0.20	S1 (moderate*)
1,500-10,000	0.20 - 2.00	S2 (severe)
>10,000	>2.00	S3 (very severe)

*or seawater

Representative composite, near surface (0 to 5 feet) bulk soil samples obtained for this study were tested to evaluate corrosion potential of material locally derived from the site from previous grading activities (Leighton, 1978). The chemical analysis test results for the onsite soil from our current geotechnical exploration are included in Appendix C of this report and are summarized below.

Table 1C - Corrosivity Test Results

Test Parameter	Test Results	General Classification of Hazard
	LB-2, Sample BB-1	
Water-Soluble Sulfate-SO_4 in Soil (ppm)	136	Negligible sulfate exposure to buried concrete (Exposure Class S0)
Percent by Weight SO_4	0.0136	
Water-Soluble Chloride in Soil (ppm)	50	Non-corrosive to buried concrete (per Caltrans Specifications)
Percent by Weight (Chl^-)	0.005	
pH	7.81	Mildly alkaline
Minimum Resistivity (saturated, ohm-cm)	1,990	Severely corrosive to buried ferrous pipes

Additional corrosion testing should be performed upon completion of grading to confirm the findings and conclusions presented above.

2.3 Expansive Soils

Expansion Index (EI) testing of engineered fill at the completion of grading (Leighton, 1978, Appendix A) below the previously planned lots 1 to 11 and Lots 5 and 10 range from medium expansion (EI=65) to very low (EI=12). Expansion Index testing during investigation of the site (Leighton, 1977) on two samples collected from test pit exploration (TP-3 and TP-11) indicate EI values ranging from 7 to 10 (very low), results are included in Appendix C. Laboratory testing performed during this study on a representative near-surface soil sample within the proposed building footprint indicated an EI of 19 (very low). These combined test results indicate a very low to medium expansion potential for onsite materials. For purposes of this report, the expansion properties of the soil below improvement footprint can be considered as "low". Additional testing of soils upon completion of grading should be performed to confirm the results of the initial testing.

Based on geotechnical laboratory testing performed on selected soil samples collected from the site and review of previous laboratory test results (Appendix A), a synopsis of geotechnical properties of the site soils is provided in Table 2 below. Geotechnical laboratory testing results are presented in Appendix C, *Laboratory Test Results.*

Table 2 – Soil Geotechnical Properties Synopsis

Parameters	Soil Properties
In-situ Moisture:	Dry to very moist
In-situ Density:	Medium dense to very dense
Swell/Expansion Potential:	Mostly granular, swell/expansion potential is **low**.
Collapse Potential:	Not susceptible to collapse when wetted
Strength:	Adequate to provide structural support
Corrosivity:	No sulfate attack of concrete but **severely corrosive to ferrous metals.**

2.4 Groundwater

Groundwater was not encountered during the current exploration drilled to exploration depths of 36.5 feet bgs. Due to shallow bedrock groundwater is not expected to pose a constraint to construction for the project as currently planned.

2.5 Infiltration

In-situ percolation testing was not performed at the site. However, due to shallow bedrock and clayey low permeability engineered fill below the site, infiltration is not considered geotechnically feasible and is therefore not recommended.

3.0 GEOLOGIC/SEISMIC HAZARDS

Geologic and seismic hazards include surface fault rupture, seismic shaking, liquefaction, seismically-induced settlement, lateral spreading, seismically-induced landslides, flooding, seismically-induced flooding, seiches and tsunamis. The following sections discuss these hazards and their potential impact at the project site.

3.1 Faulting

There are no active or potentially active faults known to cross the project site and the site is not located within an Alquist-Priolo Earthquake Fault Zone (CGS, 1986; Bryant and Hart, 2007) and as such, the potential for surface fault rupture at the site is considered low. However, several active and potentially active faults are mapped within 10 km (6.2 miles) of the site. Figure 4, *Regional Fault and Historical Seismicity Map*, shows the proximity of known active and potentially active faults within the region.

3.2 Liquefaction and Lateral Spreading

Liquefaction is the loss of soil strength due to a buildup of excess pore-water pressure during strong and long-duration ground shaking. Liquefaction is associated primarily with loose (low density), saturated, relatively uniform fine- to medium-grained, clean cohesionless soils. As shaking action of an earthquake progresses, soil granules are rearranged and the soil densifies within a short period. This rapid densification of soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, soil shear strength reduces abruptly and temporarily behaves similar to a fluid. For liquefaction to occur there must be:

(1) loose, clean granular soils,

(2) shallow groundwater, **and**

(3) strong, long-duration ground shaking.

Review of both the Earthquake Zones of Required Investigation El Monte Quadrangle, California Geological Survey Zone Map (CGS, 2014) indicate the site is not within an area potentially susceptible to liquefaction (Figure 5, *Seismic Hazard Map*).

With groundwater at a depth greater than 35 feet below grade and relatively medium dense to dense engineered fill overlying hard bedrock below the site, the potential for liquefaction and related lateral spreading effects on the site is considered very low.

3.3 Seismically-Induced Settlement

Seismically-induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event.

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed (1987), and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). Design/historic high groundwater levels of 100 feet below ground surface were used in the analysis.

Our analysis was adjusted to consider the proposed grade changes for the development as currently planned. Based on our preliminary analysis including building pad preparation recommendations presented in this report, a potential for less than 1 inch of seismic settlement is estimated at the project site. Differential seismic settlement estimated as half of the total settlement results in approximately ½ inch over a distance of 30 feet, or angular distortion of 0.0014L. Results of our seismic settlement analysis is presented in Appendix D, *Seismic Analysis*.

3.4 Seismically-Induced Landslides

The proposed project site is not located in an area mapped as potentially susceptible to seismically-induced landslides (Figure 5, *Seismic Hazard Map*). No landslides are mapped or known to exist at the project site or vicinity. The slopes to the northwest below the water tanks are reported as buttressed with engineered fill (Leighton, 1978). Therefore, the potential for seismically induced landslides to affect the site is very low.

3.5 Flooding

As shown on Figure 6, *Flood Hazard and Dam Inundation Map*, the site is located outside of areas recognized by the Federal Emergency Management Agency (FEMA) to within 0.2% annual flood potential (FEMA, 2008). Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of an earthquake. As shown on Figure 6, the site is located inside the dam inundation area of the Garvey Reservoir. Garvey reservoir is classified by the Division of Dam Safety (DSOD) as Dam No. 35-6. Garvey Reservoir is impounded by three embankment dams ranging from 115-foot to 160-foot high with a total length of 5,164 feet. Completed in 1954 the reservoir has a capacity of 1,160 acre feet of treated water. The reservoir regulates flow and pressure for Metropolitan's distribution system. The DSOD classifies it as an extremely high downstream hazard.

According to review of the *General Manager Monthly Report Activities for the Month of June 2021, Metropolitan Water District (MWD) of Southern California*, the Garvey Reservoir is planned for rehabilitation.

Garvey Reservoir Rehabilitation - According to the *Garvey Reservoir rehabilitation Project Proposed Initial Study, Report No. 1642,* prepared by the Metropolitan Water District of Southern California (MWD, 2024) the project rehabilitates Garvey Reservoir by replacing the reservoir cover (initially installed in 1999)liner, junction structure valves, and standby generator; relocates the outdated WQ lab; makes structural improvements to the inlet/outlet tower; assesses the need for subdrain systems below the reservoir liner; and evaluates the feasibility of making modifications inside the reservoir to improve mixing within the reservoir. The existing reservoir cover, liner, and supporting facilities have deteriorated and need rehabilitation to protect water quality and maintain reliable water deliveries. Construction activities would take approximately six years to complete with the first phase of work occurring on the cover liner and intake tower (MWD, 2024).

In February 2021, the dam was given a *Satisfactory Condition Assessment* by the State regulatory agency Department of Water Resources, Safety of Dams. No existing or potential dam safety deficiencies are recognized. Acceptable performance is expected under all loading conditions (static, hydrologic, seismic) in accordance with the minimum applicable state or federal regulatory criteria or tolerable risk guidelines.

Therefore, the potential for earthquake-induced flooding at the site is considered low.

3.6 Seiches-Seismically Induced Inundation

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. There are two above ground water tanks located upslope of the site to the northwest. During seismic events seiches could develop within the water tanks, however design elements such as baffles are required to reduce the potential for seiches in water tanks where overflow or structural failure may result in damage to nearby properties. Criteria for seismic design of water tanks are provided in the American Water Works Association (AWWA) Standards for Design of Steel Water Tanks.

Based on the inland location of the site, tsunami risks are considered negligible.

4.0 FINDINGS AND CONCLUSIONS

Presented below is a summary of findings and conclusions based upon the results of our evaluation of the project site:

- This site is **not** located within a currently designated Alquist-Priolo Special Studies Zone for surface fault rupture. However, as is the case for most of Southern California, strong ground shaking has and will occur at this site. This site is also **not** within a currently designated (March 25, 1998) liquefaction hazard zone. Due to primarily encountered engineered fill, deep groundwater and shallow bedrock, damaging liquefaction and related effects are unlikely to occur at this site. The site is not located in any geologic or seismic hazard zone that could preclude the development of the proposed project.
- Based on review of borings and As-Graded maps (Leighton, 1978) the site is underlain by documented artificial fill (Leighton, 1978) ranging in thickness from 2 to 15± feet overlying bedrock generally consisting of dense to hard sandstone, siltstone and conglomerate. Minor thickness of fill can be expected to vary between explored locations.
- Groundwater was not encountered during the current exploration. Groundwater is not expected to pose a constraint to construction.
- The potential for liquefaction and liquefaction-induced ground failure to occur at the site is considered very low.
- The potential seismically-induced settlement at the site is estimated to be less than 1 inch, with differential seismic settlements of ½ inch over a distance of 30 feet.
- Based on our observations and testing, the onsite soils that will be in contact with the planned structure are expected to have a low expansion potential. Additional testing is recommended at completion of grading. For purposes of design, we recommend using a low expansion index (EI=20 to 50).
- Concrete in contact with the onsite soil is expected to have negligible exposure to water-soluble sulfates (Exposure Class S0) and low exposure to chloride in the soil. The onsite soil, however, is considered severely corrosive to ferrous metal.
- The subsurface materials are anticipated to be readily excavated using conventional earthmoving equipment in good working condition.

- The proposed improvements may be supported on conventional shallow footing foundation systems established on engineered fill.
- Proposed retaining wall systems on the northern and northeastern portions of the site will be cut from existing grade with heights of up to 21 feet. Based on preliminary global stability analyses, a soil nail wall system with adequate nail lengths is recommended to achieve acceptable factors of safety.

Based upon the results of our geotechnical evaluation of the site, the proposed improvements are considered feasible from a geotechnical standpoint.

5.0 RECOMMENDATIONS

The following recommendations have been developed based on the exhibited engineering properties of the onsite soils and their anticipated behavior during and after construction. Recommendations are specifically provided for design of foundations, seismic design considerations, floor slabs, retaining structures, paving, and grading. The proposed structures may be supported on spread-type shallow footing foundation systems established on engineered fill. Leighton should review the grading plan, foundation plans and specifications when they are available to verify that the recommendations presented in this report have been properly interpreted and incorporated. *The local department of building and safety is responsible for implementing the provisions of the applicable building codes. The project would be required to comply with plan review and permitting requirements of the local jurisdiction before the start of construction, as well as comply with the recommendations provided in a final design, and site specific, geotechnical report that is subject to review and approval by the city.*

Loading and bearing pressure diagrams should be provided for our review once prepared to confirm recommendations and settlement estimates remain valid for the project as currently proposed.

5.1 Grading

Project earthwork is expected to include complete demolition/removal of existing surface pavements, building, utilities and complete overexcavation and recompaction of any remaining documented fill soils below new improvement footprints as described in the following subsections.

5.1.1 Site Preparation

After the site is cleared, the soils should be carefully observed for the removal of all unsuitable deposits. We recommend that after removal of pavements and hardscape, and complete demolition of existing structure the subgrade should be evaluated by the engineering geologist.

To reduce the potential for adverse total and differential settlement of the proposed structures, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved. Due to the potential for near-surface compressible soils, we recommend that onsite soils in the proposed building/structure areas and site walls taller than 5 feet be overexcavated to a minimum depth of 5 feet bgs, or to a depth of 3 feet below the bottoms of proposed footings, whichever is deeper. Our geotechnical field representative may evaluate earthwork doing grading to reduce the overexcavation requirements if very dense bedrock is encountered.

Where possible, the removal bottom should extend horizontally a minimum of 5 feet from the outside edges of the building/structure footprint and footings (including columns connected to the buildings), or a distance equal to the depth of overexcavation below the footings, whichever is farther. Where this is not achievable, this should be reviewed on a case-by-case basis.

Any underground obstructions encountered should be removed. Those lines should be removed or rerouted where interfering with proposed construction. *According to prior as graded maps (Appendix A) a canyon subdrain was installed in 1978 and prior grading may have included installation of an 8-inch diameter VCP sewer pipe. It is unknown what elevations these utilities lie below the current building footprint. We recommend confirming the presence or absence of these utilities after demolition and prior to full scale grading of the site.*

Areas outside the building, planned for new asphalt and/or concrete pavement, should be over-excavated to a minimum depth of 24 inches below existing or finish grade, or 18 inches below proposed pavement sections, whichever is deeper.

Ancillary structures, such as lightly loaded electrical equipment pads, should be overexcavated to a minimum depth of 2 feet below existing ground or 18 inches below bottom of proposed footings, whichever is greater. Any overexcavation for small, lightly loaded ancillary structures should extend 3 feet horizontally outside the structure limits, where feasible.

Resulting removal excavation bottom-surfaces should be observed by Leighton Consulting, Inc., prior to placement of any backfill or new construction. After these over-excavations are completed, and prior to fill placement, exposed surfaces should be scarified to a minimum depth of 6 inches, moisture-conditioned to or slightly above optimum moisture content, and recompacted (proof rolled) to a minimum 90 percent relative compaction as determined by ASTM D 1557 standard test method (modified Proctor compaction curve).

5.1.2 Earthwork Observation and Testing

Leighton Consulting, Inc. should observe and test all grading and earthwork, to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential. Project plans and specifications should incorporate recommendations contained in the text of this report.

Variations in site conditions are possible and may be encountered during construction. To confirm correlation between soil data obtained during our field and laboratory testing and actual subsurface conditions encountered during construction, and to observe conformance with approved plans and specifications, it is essential that we be retained to perform continuous or intermittent review during earthwork, excavation and foundation construction phases. Therefore, conclusions and recommendations presented in this report are contingent upon us performing construction observation services.

5.1.3 Fill Placement and Compaction

Onsite soils free of organics, debris and oversized material (greater-than 6 inches in largest dimension) are suitable for use as compacted structural fill. However, any soil to be placed as fill, whether onsite or imported material, should be first viewed by Leighton and then tested if and as necessary, prior to approval for use as compacted fill. All structural fill must be free of hazardous materials.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to within 2 percent of optimum moisture content, and compacted to a minimum 92% relative compaction as determined by ASTM D 1557 standard test method (modified Proctor compaction curve) within building footprints. Aggregate base for pavement sections should be compacted to a minimum of 95% relative compaction. At least the upper 12 inches of the exposed soils in roadways and access drives, parking lots and (concrete – paver) flatwork areas, should be compacted to at least 95 percent relative compaction based on ASTM Test Method D 1557.

Fill Materials: The onsite soils, less any deleterious material or organic matter, can be used in required fills. Cobbles larger than 6 inches in largest diameter should not be used in the fill. Any required import material should consist of relatively non-expansive soils with a very low to low Expansion Index (EI<50). The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be approved by the geotechnical engineer of record prior to being placed at the site.

Surface Drainage: Water should not be allowed to pond or accumulate anywhere except in detention basins. Pad drainage should be designed to collect and direct surface water away from structures to approved drainage facilities. Hardscape drains should be installed and drain to storm water disposal systems. Drainage patterns approved at the time of fine grading should be maintained throughout the life of proposed structures. Irrigation should not be allowed for at least 10 feet horizontally around buildings.

5.1.4 **Reuse of Concrete and Asphalt In Fill** Pulverized demolition concrete free of rebar and other materials and demolished asphalt pavement can be pulverized to particles no-larger-than (≤) 3-inches, and mixed with site soils for use in compacted fill. Blended pulverized concrete and asphalt should be mixed with at least 25% soils by weight. Such materials must be free of and segregated from any hazardous materials and/or organic material of any kind.

5.1.5 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations, and other excavations should be performed in accordance with project plans, specifications and all State of California Occupational Safety and Health Administration (CalOSHA) requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundations should be properly shored to maintain support of these structures.

Temporary excavations should be treated in accordance with CalOSHA excavation regulations. The sides of excavations should be shored or sloped accordingly. CalOSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a ¾:1 (horizontal:vertical) slope for Type A soils, 1:1 for Type B soils, and 1½:1 for Type C soils.

The onsite soils within the proposed overexcavation depths generally conform to CalOSHA Type C soils. Soil type should be evaluated during construction by the designated “competent person” per CalOSHA guidelines. CalOSHA regulations are applicable in areas with no restriction of surrounding ground deformations. Shoring should be designed for areas with deformation restrictions. Heavy construction loads, such as those resulting from stockpiles and heavy machinery, should be kept a minimum distance equivalent to the excavation height or 5 feet, whichever is greater, from the excavation unless the excavation is shored and these surcharges are considered in the design of the shoring system.

5.1.6 Trench Backfill

Pipeline trenches should be backfilled with compacted fill in accordance with this report, and applicable *Standard Specifications For Public Works Construction* (Greenbook), current edition standards. Backfill in and above the pipe zone should be as follows:

- **Pipe Zone**: Any proposed pipe should be placed on properly placed bedding materials. Pipe bedding should extend to a depth in accordance to the pipe manufacturer’s specification. The pipe bedding should extend to least 1 foot over the top of the conduit. The bedding material may consist of compacted free-draining sand, gravel, or crushed rock. If sand is used, the sand should have a sand equivalent greater than 30. As an alternate the pipe bedding zone can be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, conforming to Section 201-6 of the current Edition of the Standard Specifications for Public Works Construction (Greenbook). CLSM bedding should be placed to 1 foot over the top of the conduit, and vibrated. CLSM should not be jetted.

- **Over Pipe Zone**: Above the pipe zone, trenches can be backfilled with excavated on-site soils free of debris, organic and oversized material larger than 3 inches in largest dimension. As an option, the whole trench can be backfilled with one-sack CLSM same as presented above for the pipe bedding zone. Native soil backfill over the pipe-bedding zone should be placed in thin lifts, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 92% relative compaction relative to the ASTM D 1557 laboratory maximum dry density within building footprints. The upper 12-inches under hardscape, parking, paver etc. should be compacted to 95% relative compaction.

5.1.7 Corrosion Protection Measures

Water-soluble sulfates in soil can react adversely with concrete. As referenced in the 2022 California Building Code (CBC), Section 1904A, concrete subject to exposure to sulfates shall comply with requirements set forth in ACI 318. Based on laboratory testing results of the onsite soils from recent and prior investigations, concrete structures in contact with the onsite soil will likely have "negligible" exposure to water-soluble sulfates in the soil (Exposure Class S0). Therefore, common Type II Portland cement may be used for concrete construction in contact with site soils. Subgrade soil should be tested for water-soluble sulfate content prior to final design of the concrete structures once grading is complete. Import fill soil should be geotechnically tested for corrosivity and sulfate attack before import to the site. Further testing of import soils, if needed, should include analytical testing for chemicals of concern prior to import and acceptance.

Based on corrosivity test results of the onsite soils, the onsite soil is considered severely corrosive to ferrous metals. Therefore, based on these results, ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE), polyvinyl chloride (PVC) and/or other non-ferrous pipe when possible. Ferrous pipe can also be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site soils. A corrosion engineer can be consulted for project-specific corrosion protection measures. Corrosion information presented in this report should be provided to your underground utility subcontractors.

5.2 Foundations

Overexcavation and recompaction of the footing subgrade should be performed as detailed in Section 5.1. The following recommendations are based on the onsite soil conditions and soils with a "low" expansion potential. The proposed new structures may be supported on a shallow spread footing foundation system established on engineered fill.

5.2.1 Shallow Spread Footings

Footings should have a minimum embedment depth of 18 inches, with minimum width of 24 and 12 inches for isolated and continuous footings, respectively.

Bearing Value: An allowable bearing pressure of 2,000 pounds per square foot (psf) may be used, based on an assumed embedment depth of 18 inches and minimum width described above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,500 psf. If higher bearing pressures are required, this should be reviewed on a case-by-case basis and may include additional overexcavation and/or soil reinforcement. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

Settlement: The recommended allowable bearing pressure is generally based on a total allowable, post-construction static settlement of 1 inch. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 30 feet. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

Seismic differential settlement is assumed to be ½ inch over a horizontal distance of 30 feet for the design-level earthquake, or angular distortion of 0.0014L.

Lateral Resistance: Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.35. The passive resistance may be computed using an equivalent fluid pressure of 240 pounds-per-cubic-foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The passive resistance can be increased by one-third when considering short-duration wind or seismic loads. The friction resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

Uplift Resistance: To evaluate uplift resistance provided by the dead weight of soils above the footing, the frustum of soil above the footing may be estimated by a 30 degree outward projection from vertical. A unit weight of 120 pcf may be used for the soil volume within the frustum.

5.2.2 Modulus of Subgrade Reaction

For foundations established in engineered fill, an initial unit modulus of subgrade reaction (k_1) value of 150 pounds per cubic inch (pci) may be used.

The k_1 value presented herein, which corresponds to a 1-foot-square footing, should be reduced as shown below to incorporate foundation size effects:

$$k = k_1 \left(\frac{B+1}{2B}\right)^2$$

where B is the square footing width.

5.2.3 Flagpole-Type Foundations

The following recommendations are applicable for light poles and shade structures/canopies. If large light structure, such as Musco light poles, are planned, then Leighton should review the proposed lighting system and provide supplemental recommendations if required. Similarly, if large shade structures are planned, then Leighton should review the proposed structure systems.

For enhanced sliding and overturning resistance, light poles are often founded on drilled cast-in-place reinforced concrete piers. Therefore, we present geotechnical design parameters for drilled cast-in-place concrete piers to support new light poles.

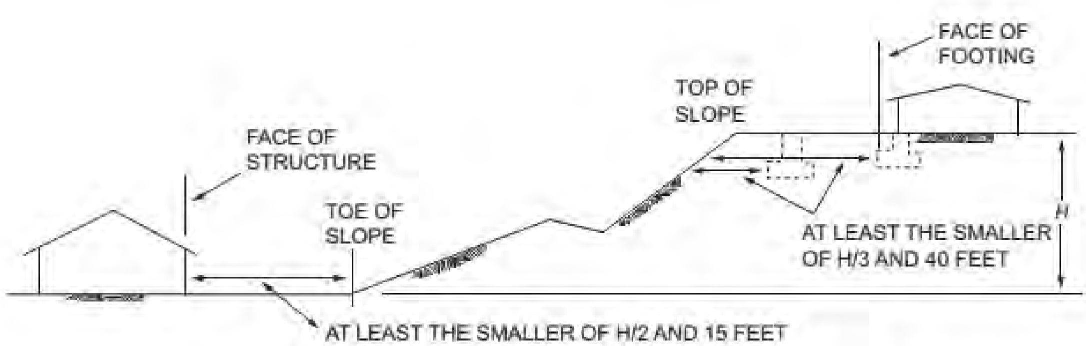
Lateral bearing resistance for proposed light pole pile foundations may be based on allowable lateral earth pressure of Class of Material 5 on Table 1806A.2 of the 2022 CBC, which can be doubled in accordance with 1806A.3.4, ignoring the upper 18 inches of soil in non-paved areas. This lateral bearing value assumes that the pole can tolerate at least a 0.5-inch deflection at the ground surface due to short term loading. Lateral bearing resistance should be computed in accordance with Section 1807A.3.2.1 (unconstrained laterally) of the 2022 CBC.

These recommendations assume that the foundations will be embedded against firm intact soil.

For axial design, we recommend an allowable resistance in compression for these foundations consisting of 200 psf for allowable skin friction, ignoring the bottom one diameter, and an allowable end bearing of 2,500 psf (assuming a cleaned-out bottom). We recommend that the piles be at least 4 pile diameters long. These values are for isolated single piles.

5.2.4 Building Clearance and Foundation Setbacks

All building foundations located near slopes should have a minimum setback per Figure 1808.7.1 of the 2022 California Building Code (CBC). Setback distances should be measured from competent material extending perpendicular to the slope face.



As a minimum, building clearances from the toe of an ascending slope should be equal to one-half of the total slope height (H/2) to a maximum setback of 15 feet per the 2022 CBC section 1808.7.1. and Figure 1808.7.1.

5.3 Seismic Design Parameters

We performed a Site Class analysis in general accordance with ASCE 7-16 analysis with field Standard Penetration Blowcounts (SPT) from the geotechnical borings that extended to a maximum depth of 36.5 feet below existing grade, and considered the proposed site grade adjustments. The western and eastern portion of the proposed building will require approximately 8 feet of fill and 11 feet of cut to reach proposed grades, respectively.

Considering the geotechnical boring with the deepest depth of competent bedrock (Boring LB-2), the site classifies as Site Class D. However, other borings at the site encountered relatively shallow bedrock which resulted in classification as Site Class C. Therefore, we are including seismic parameters for both Site Class D and C for structural evaluation.

The tables below list seismic design parameters for both Site Class C and D based on the 2022 CBC, Section 1613A.3 (ASCE 7-16) methodology:

Table 3A - 2022 CBC Mapped Seismic Parameters – Site Class "C"

Categorization/Coefficients	Code-Based (1) (2)
Site Longitude (decimal degrees) West	-118.1141
Site Latitude (decimal degrees) North	34.0402
Site Class	C
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.918
Mapped Spectral Response Acceleration at 1s Period, S_1	0.689
Short Period Site Coefficient at 0.2s Period, F_a	1.200
Long Period Site Coefficient at 1s Period, F_v	1.400
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	2.302
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	0.965
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.534
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.643
Design Peak Ground Acceleration, PGA_M	0.993

1. All were derived from the SEA web page: https://seismicmaps.org/
2. All coefficients in units of g (spectral acceleration)
3. See Appendix D for details of the seismic evaluation.

Table 3B - 2022 CBC Mapped Seismic Parameters – Site Class "D"

Categorization/Coefficients	Code-Based (1) (2)
Site Longitude (decimal degrees) West	-118.1141
Site Latitude (decimal degrees) North	34.0402
Site Class	D (5)
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.918
Mapped Spectral Response Acceleration at 1s Period, S_1	0.689
Short Period Site Coefficient at 0.2s Period, F_a	1.000
Long Period Site Coefficient at 1s Period, F_v	1.700
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.918
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.171 (4)
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.279
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.781 (4)
Design Peak Ground Acceleration, PGA_M	0.910

1. All were derived from the SEA web page: https://seismicmaps.org/
2. All coefficients in units of g (spectral acceleration)
3. See Appendix D for details of the seismic evaluation.
4. See Section 11.4.8 of ASCE 7-16. A site-specific ground motion hazard analysis in accordance with Section 21.2. of ASCE 7-16 is required for this site. **Per Supplement 3 to ASCE 7-16, a site-specific ground motion hazard analysis is not required where the value of the parameters SM_1 and SD_1 in the table are increased by 50%.**
5. Site Class D, and all of the resulting parameters in this table, may only be used for structures without seismic isolation or seismic damping systems.

The project structural engineer should review the seismic parameters.

5.4 Slabs-on-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with 2022 CBC requirements for soils with a low expansion potential. More stringent requirements may be required by the structural engineer and/or architect; however, slabs-on-grade should have the following minimum recommended components:

- **Subgrade:** The near-surface clayey soils can be expansive and will shrink and swell with changes in the moisture content. Therefore, floor slabs-on-grade and adjacent concrete flatwork should be underlain by at least 18 inches of relatively non-expansive fill (EI<20). Expansion Index tests performed during this study in the area of the proposed building indicated an EI of 19 ("very low").

Existing soils are anticipated to be relatively non-expansive, however this should be further evaluated during earthwork construction. Accordingly, removal and replacement with non-expansive fill will likely not be required.

Slab-on-grade subgrade soil should be moisture conditioned to or within 3% over optimum moisture content, to a minimum depth of 18 inches within building footprints, and compacted to 92% of the modified proctor (ASTM D 1557) laboratory maximum density prior to placing either a moisture barrier, steel and/or concrete. Onsite soil may be suitable for this use; however additional expansion testing should be performed upon completion of grading to verify expansive properties of onsite soil.

- **Moisture Barrier:** A moisture barrier consisting of at least 15-mil-thick Stego-wrap vapor barriers (see: http://www.stegoindustries.com/products/stego_wrap_vapor_barrier.php), or equivalent, should then be placed below slabs where moisture-sensitive floor coverings or equipment will be placed.

- **Reinforced Concrete**: A conventionally reinforced concrete slab-on-grade with a thickness of at least 5 inches should within the building footprint and 5-inches for exterior slab on grade (SOG) be placed in pedestrian areas without heavy loads. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 3 rebar placed at 18 inches on-center, each direction (perpendicularly), mid-depth in the slab.

  Minor cracking of concrete after curing due to expansion, drying and shrinkage is normal and will occur. However, cracking is often aggravated by a high water-to-cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking.

5.4.1 Utilities and Trenches

Where existing pipes cross under footings the footings shall be specifically designed by the engineer in charge. Pipe sleeves shall be provided where pipes cross through footings or footing walls and sleeve clearances shall be designed to account for potential settlement of not less than 1 inch around the pipe. Alternate and approved clearances can be provided by the design professional in charge of the utility.

5.5 Lateral Earth Pressures

Recommended lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft. or pcf. These values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

On-site soils are likely suitable to be used as retaining wall backfill due to its relatively low expansion potential, field and laboratory verification recommended before use. Should site soil be desired for reuse behind retaining walls, if any, the material should be tested to ensure Expansion potential is less than 30 (EI<30). Recommended lateral earth pressures for retaining walls backfilled with sandy soils with drained conditions are as follows:

Table 4 - Retaining Wall Design Earth Pressures

Retaining Wall Condition (Level Backfill)	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active (cantilever)	40
At-Rest (braced)	60
Passive Resistance (compacted fill)	240
Seismic Increment (add to active pressure)	31

*Only for level and drained properly compacted backfill

Walls that are free to rotate or deflect may be designed using active earth pressure. For the basement walls or walls that are fixed against rotation, the at-rest pressure should be used. For seismic condition, the pressure should be distributed as an inverted triangular distribution and the dynamic thrust should be applied at a height of 0.6H above the base of the wall.

Retaining Wall Surcharges: Vertical surcharge loads behind a retaining wall on or in backfill within a 1:1 (horizontal:vertical) plane projection up and out from the retaining wall toe, should be considered as lateral and vertical surcharge. Unrestrained (cantilever) retaining walls should be designed to resist one-third of these surcharge loads applied as a uniform horizontal pressure on the wall. Braced walls should also be designed to resist an additional uniform horizontal-pressure equivalent to one-half of uniform vertical surcharge loads.

In areas where autos and pickup trucks will drive we suggest assuming a uniform vertical surcharge of 300 psf, which would result in active and at-rest horizontal surcharges of 100 psf and 150 psf, respectively. This should be doubled in areas of heavy construction traffic (such as concrete trucks, heavy equipment delivery-trucks, etc.). If crane outrigger loads or other point load sources are applied as wall surcharge, this will require additional analyses based on load source and location relative to the wall.

5.5.1 **Sliding and Overturning** Total depth of retained earth for design of walls and for uplift resistance, should be measured as the vertical height of the stem below the ground surface at the wall face for stem design, or measured at the heel of the footing for overturning and sliding. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing, if drained, or 60 pcf if submerged, for properly compacted backfill.

5.5.2 **Drainage**

Adequate drainage may be provided by a subdrain system positioned behind the walls. Typically, this system consists of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with pervious backfill material described in Section 300-3.5.2 of the Standard Specifications for Public Works Construction (Green Book), Current Edition. This pervious backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in Section 300-8.1 of the Standard Specifications for Public Works Construction (Green Book). The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage geocomposites, or similar, may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill, particularly where horizontal space is limited adjacent to shoring (where walls are cast against shoring). These drainage panels should be connected to the perforated drainpipe at the base of the wall.

5.6 Soil-Nail Retaining Walls

Retaining walls with heights of up to 21 feet are currently proposed on the northern and northeastern portions of the site. Based on the design cut to reach proposed grades, we assume that the retaining wall system would be constructed with soil-nails. Proposed walls include an approximate 1.5H:1V to 2H:1V slope above the wall, extending to the property line.

Soil nail wall systems are advantageous in that they can be constructed in a top-down manner, considering the need for cuts into the existing slope. Based on Caltrans geotechnical manual guidelines, a minimum soil nail length of 15 feet was considered.

Preliminary analysis of global stability using limit equilibrium (i.e. slope stability) techniques was performed on three sections (Section B, D, and E) of the proposed retaining wall. The resultant soil nail lengths to achieve acceptable factors of safety range from approximately 29 to 39 feet (approximately 1.6 to 2.1 times the height of the wall). Our global stability analyses are summarized below.

Table 5 – Soil Nail Wall Global Stability Summary

Section	Wall Height (ft)	Soil Nail Rows	Soil Nail Length	Factor of Safety	
				Static	Pseudo Static
B-B	21	5	39 (top two rows) 35 (remaining)	1.54	1.19
D-D	18	4	34 (top two rows) 29 (remaining)	1.58	1.19
E-E	15	3	32	1.53	1.10

Soil nails were located a minimum of 2.5 feet below the top of wall. Global stability analysis for the proposed retaining wall are included in Appendix D. The following sections include design recommendations for the proposed retaining wall.

5.6.1 Soil Nail Wall Configuration

Based upon the materials that are anticipated to be exposed during cutting and wall construction, the maximum vertical cut performed during nail installation is recommended to be 5 feet. The minimum horizontal spacing of the soils nails is recommended to be 4 feet but should not exceed 5 feet. The rows of soil nails are recommended to be installed 2½ feet from the top of wall, and 2 to 3 feet from the bottom. In addition, nails are recommended to be located no closer than 2½ feet from the ends of the wall. For wall sections under 4.5 feet in height from top of bottom of wall, soil-nails should be centered in the soil cut face. The minimum soil nail length is recommended to be 15 feet. Table 5 provides the minimum soil nail lengths for the wall heights and configurations modeled in order to achieve acceptable global stability factors of safety.

Per Caltrans geotechnical manual guidelines, we recommend that the soil nail reinforcement be composed of No. 8 Grade 75 steel bars.

5.6.2 Pullout Resistance

Pullout resistance used in design depends upon the material in which the nails will be embedded and the manner in which nail construction will be performed. Inclined soil nails constructed by typical gravity flow of grout placement may be analyzed using an assumed design ultimate bond stress of 900 lbs/ft in the encountered fill and 1,100 lbs/ft within granular Fernado Formation bedrock. Bond stress values were derived based on general parameters and soil types shown on Table 4.4 of FHWA guidance documents (FHWA, 2015).

5.6.3 Bearing Capacity and Sliding Resistance

Bearing capacity is not anticipated to be a significant design consideration for the proposed soil nail walls. Evaluation of bearing capacity as part of wall design may be based upon an allowable bearing resistance of 1,000 psf.

Resistance to sliding should be evaluated solely on the sliding resistance of the bearing soils. The coefficient of sliding friction for use in design is recommended to be 0.35.

5.6.4 Wall Drainage

Design of the wall is based upon the presumption of drained soil conditions behind the wall and no additional load demand from the accumulation of water behind the wall. Therefore, an appropriate permanent drainage systems will be required that includes proper connection to a conveyance system to properly discharge water to the storm drain or other device.

The top-down manner in which the wall will be constructed requires the installation of the drainage system during construction of each section of wall as construction proceeds. A drainage system consisting of a geo-composite (e.g., drainboard) is expected to be a feasible alternative.

Vertical drainage panels can be installed as strips between rows of soil nails. The drainage panels should be connected at the base of the wall and outlet to a collective drainage system or to weepholes near the base of the wall. Panel drains should be terminated near the top of the wall and should not be exposed above the top of wall.

5.6.5 Excavation Stability

Field testing may be required to demonstrate the stability of excavation lift height and exposure duration in areas where the maximum lift height exceeds 5 feet or an exposure duration longer than one (1) work shift. When stability testing is not performed, shotcrete should be applied during the same work shift in which excavation has occurred. Completion of the shotcrete facing may be delayed up to 24 hours if the contractor demonstrates that the integrity of the excavated face is maintained.

Stability tests should be conducted by performing staged excavation to produce a neat excavated face no more than 5 feet in front of the location of the final wall face. The height of the excavated face should be as specified in the approved working drawings. The excavated face should be 20 feet long and parallel to the wall alignment. The excavated face should have a constant height within the 20-foot section. Ramps may be excavated outside the 20-foot section to provide construction access. The excavated face should be left open for the duration specified in the approved working drawings.

The excavated face must maintain its integrity without raveling, sloughing, or measurable lateral movement at the completion of the stability test. After written approval by the Engineer, the proposed excavation height may be used in that wall zone as the stand-up height of the excavated face for the duration observed in the stability test.

If at any time the exposed excavated face fails to maintain its integrity without raveling, sloughing, or measurable lateral movement for the duration of time observed in the approved stabilization test, the contractor should immediately stabilize the excavated face and perform additional stability testing as described herein.

5.6.6 Load Testing

Verification and proof testing should be performed on soil nails to verify pullout capacity and bond strength of the soil-nail interface. Tests should be performed during installation in accordance with FHWA guidelines. FHWA recommends proof testing on at least 5 percent of production soil nails in each nail row, and a minimum of one per row.

5.6.7 Corrosion Protection

The results of corrosivity testing performed on representative samples of the material encountered at the boring locations indicates a moderate potential exists for corrosion to occur. The cement grout used to secure the nails in the boreholes will provide both physical and chemical corrosion protection. In addition, the bars are recommended to be epoxy coated and encapsulated with a corrugated plastic sheathing. The grout that fills the annular area around the steel bar should be low in permeability and provide a minimum 1-inch cover.

5.7 Pavement Design

To provide support for paving, the subgrade soils should be prepared as recommended in Section 5.1, Grading. Compaction of the subgrade, including trench backfills, to at least 92 and 95 percent as recommended relative compaction based on ASTM Test Method D 1557 and achieving a firm, hard and unyielding surface will be important for paving support. The upper 12-inches of pavement subgrade should be compacted to 95% relative compaction. The preparation of the paving area subgrade should be performed immediately prior to placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction or Caltrans Specifications. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled.

5.7.1 Base Course

The base course for both asphalt concrete and Portland Cement Concrete paving should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the latest edition of the State of California, Department of Transportation, and Standard Specifications. Alternatively, the base course could meet the specifications for untreated base as defined in Section 200-2 of the latest edition of *Standard Specifications for Public Works Construction* (Greenbook). Crushed Miscellaneous Base (CMB) may be used for the base course provided the geotechnical consultant evaluates and tests it before delivery to the site as CMB tends to result in a thicker support section than crushed aggregate base.

5.7.2 Asphalt Concrete

The required asphalt paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Onsite near-surface soils consisted of clayey/silty sands and sandy clay. Laboratory testing performed on a near-surface clayey bulk sample yielded an R-Value of 18. Assuming that the paving subgrade will consist of similar soils compacted to at least 92 percent relative compaction based on ASTM Test Method D 1557 below 12-inches and 95% relative compaction in the upper 12-inches, the minimum recommended paving thicknesses are presented in the following table:

Area	Traffic Index	Asphalt Concrete (inches)	Base Course (inches)
Light Truck	5	3.0	7.5
Heavy Truck	6	3.5	10.0
Main Drives	7	4.0	12.5

The asphalt paving sections were determined using the 2017 Caltrans design method using assumed Traffic Index values. We can determine the recommended paving and base course thicknesses for other Traffic Indices if required. Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved, and that proper construction procedures are followed.

5.7.3 Portland Cement Concrete Paving

For onsite Portland Cement Concrete (PCC) pavement in truck drive aisles and parking areas, we recommend a minimum of 6.5-inch-thick concrete with dowels at joints, placed on compacted fill subgrade, with the upper 8 inches compacted to a minimum of 95 percent relative compaction. In areas with car traffic only, we recommend a minimum of 5-inch-thick concrete, placed on compacted fill subgrade with the upper 8 inches compacted to a minimum of 95 percent relative compaction.

The PCC pavement sections should be provided with crack-control joints spaced no more than 14 feet on center each way for 6.5-inch-thick concrete, and 12 feet for 5-inch-thick concrete. If sawcuts are used, they should have a minimum depth of ¼ of the slab thickness and made within 24 hours of concrete placement.

5.8 Geotechnical Services During Construction

Our geotechnical recommendations are contingent upon Leighton Consulting, Inc., providing geotechnical observation and testing services during earthwork and foundation construction. There is a potential for encountering deeper fill, underground obstructions or otherwise unacceptable existing soils between or beyond our boring locations. We are unaware of any existing fill placement documentation for the engineered fill placed in 1971 for this site. Therefore, inconsistent existing fill materials may be encountered in the canyon backfill during construction, possibly requiring revised geotechnical recommendations.

Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical exploration, testing and/or analysis may be required based on final plans. Leighton Consulting, Inc. should review site grading and foundation plans when available, to comment further on geotechnical aspects of this project and check to see general conformance of final project plans to recommendations presented in this report.

Leighton Consulting, Inc. should be retained to provide geotechnical observation and testing during excavation and all phases of earthwork. Our conclusions and recommendations should be reviewed and verified by us during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- During all excavation,
- During compaction of all fill materials,
- After excavation of all footings and prior to placement of concrete,
- During utility trench backfilling and compaction,
- During pavement subgrade and base preparation, and/or
- If and when any unusual geotechnical conditions are encountered.

6.0 LIMITATIONS

Leighton's work was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the conclusions and professional opinions included in this report. As in many projects, conditions revealed in excavations may be at variance with our current findings. If this occurs, the changed conditions must be evaluated by the geotechnical consultant and additional recommendations be obtained, as warranted.

The identification and testing of hazardous, toxic or contaminated materials were outside the scope of Leighton's work. Should such materials be encountered at any time, or their existence is suspected, all measures stipulated in local, county, state and federal regulations, as applicable, should be implemented.

This report is issued with the understanding that it is the responsibility of the owner or a duly authorized agent acting on behalf of the owner, to ensure that the information and recommendations contained herein are brought to the attention of the necessary design consultants for the project and incorporated into the plans; and that the necessary steps are taken to see that the contracts carry out such recommendations in the field.

The findings of this report are considered valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on the subject or adjacent properties. In addition, changes in standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may at some future time be invalidated wholly or partially by changes outside Leighton's control.

The conclusions and recommendations in this report are based in part upon data that were obtained from a necessarily limited number of observations, site visits, excavations, samples and testes. Such information can be obtained only with respect to the specific locations explored, and therefore may not completely define all subsurface conditions throughout the site. The nature of many sites is that differing geotechnical and/or geological conditions can occur within small distances and under varying climatic conditions. Furthermore, changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report should be considered preliminary if unanticipated conditions are encountered and additional explorations, testing and analyses may be necessary to develop alternative recommendations.

This report has been prepared for the express use of SDCF Monterey Park, LLC and its design consultants, and only as related expressly to the assessment of the geotechnical constraints of developing the subject site and for construction purposes. This report may not be used by others or for other projects without the express written consent of SDCF Monterey Park, LLC and our firm.

If parties other than Leighton are engaged to provide construction geotechnical services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project by concurring with the findings and recommendations in this report or by providing alternative recommendations. Any persons using this report for bidding or construction purposes should perform such independent investigations as they deem necessary to satisfy themselves as to the surface and/or subsurface conditions to be encountered and the procedures to be used in the performance of work on the subject site.

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FIGURES AND PLATES

Project: 19850	Eng/Geol: JAR
Scale: 1 " = 2,000 '	Date: December 2023

Reference: © 2023 Microsoft Corporation © 2023 Maxar ©CNES (2023) Distribution Airbus DS ©

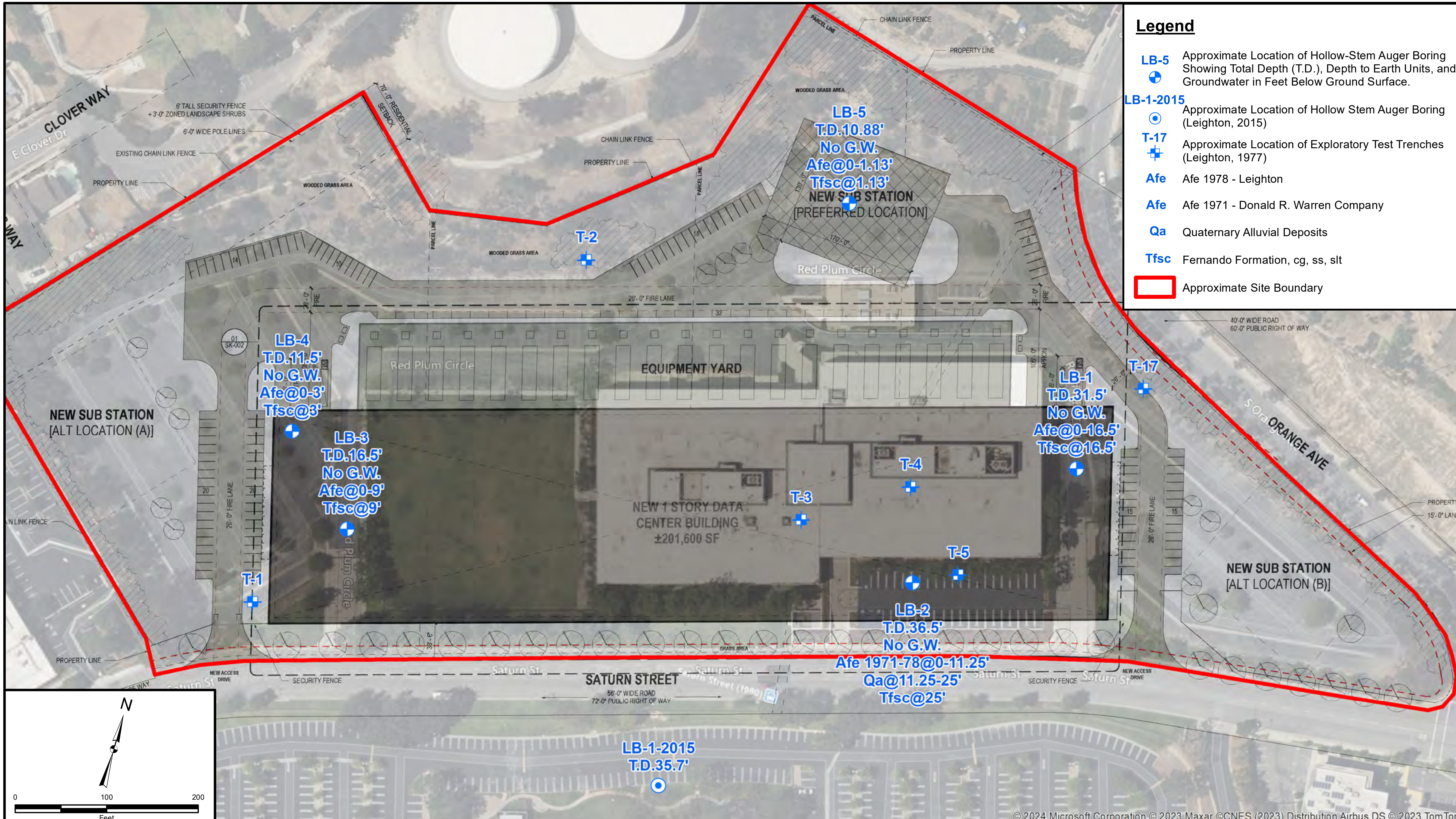
SITE LOCATION MAP

Proposed Data Center Facility
1977 Saturn Street
Monterey Park, California

FIGURE 1

Leighton

Map Saved as Z:\Project Files\SA-TZ\SDCFMONT\19850 - 1977 Saturn Monterey Park\Verdantas Folder Structure\GIS\Maps\19850_F01_SLM_2023-12-07.mxd on 12/7/2023 11:14:46 AM Author: KVM (btran)



Project: 19850	Eng/Geol: JAR
Scale: 1 " = 100 '	Date: February 2024

Base Map: As Shown

Author: (btran)

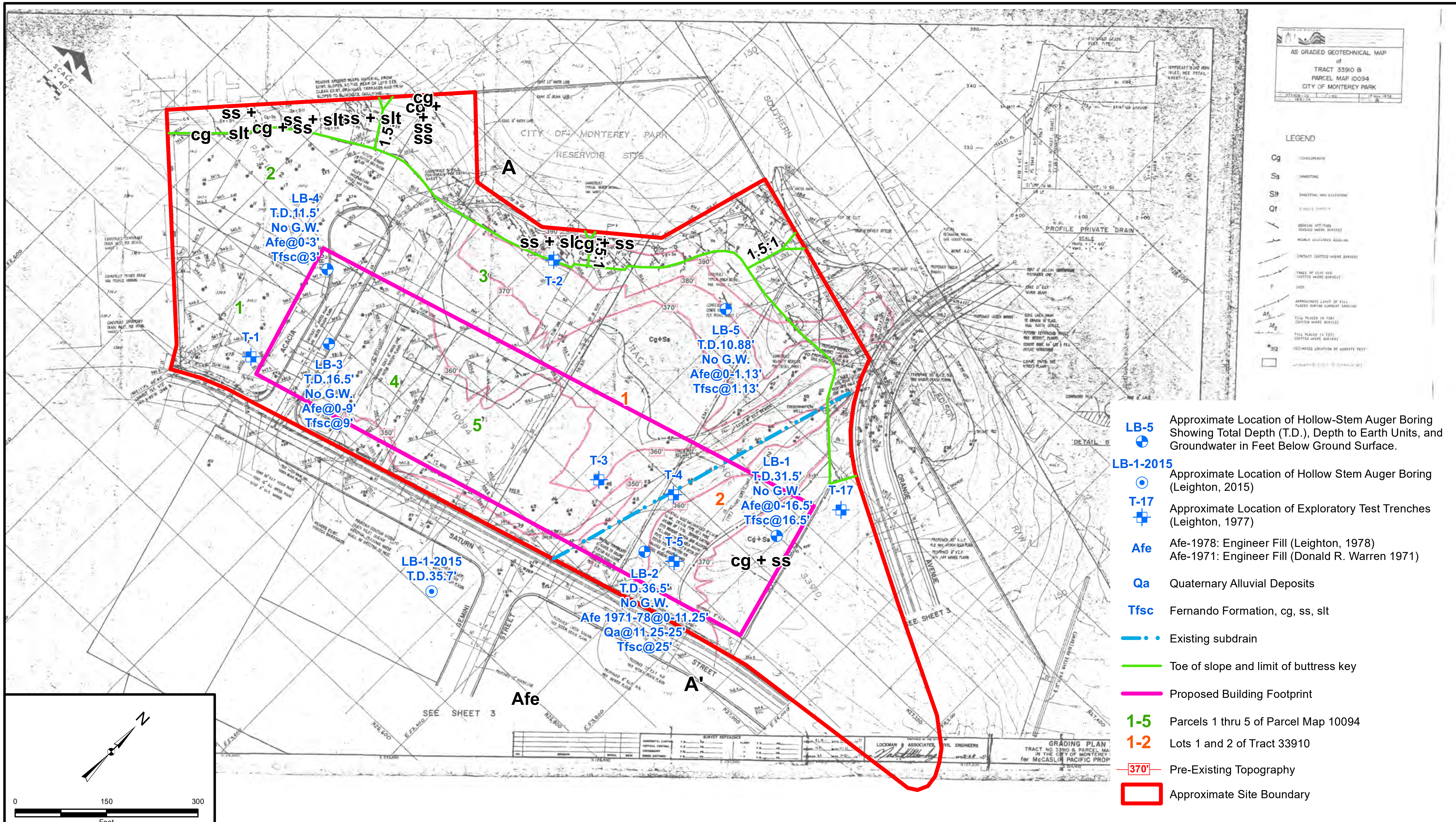
BORING LOCATION MAP

Proposed Data Center Facility
1977 Saturn Street
Monterey Park, California

FIGURE 2A

Leighton

Map Saved as Z:\Project Files\SA-TZ\SDCFMONT\19850 - 1977 Saturn Monterey Park\Verdantas Folder Structure\GIS\Maps\19850_F02A_BLM_2024-02-02.mxd on 1/22/2024 10:22:04 AM



Project: 19850	Eng/Geol: JAR
Scale: 1 " = 150 '	Date: February 2024

Base Map: Grading Plan, Sheet 2 of 3
Dated: 1978 by Lockman & Associates, Civil Engineers

Author: (btran)

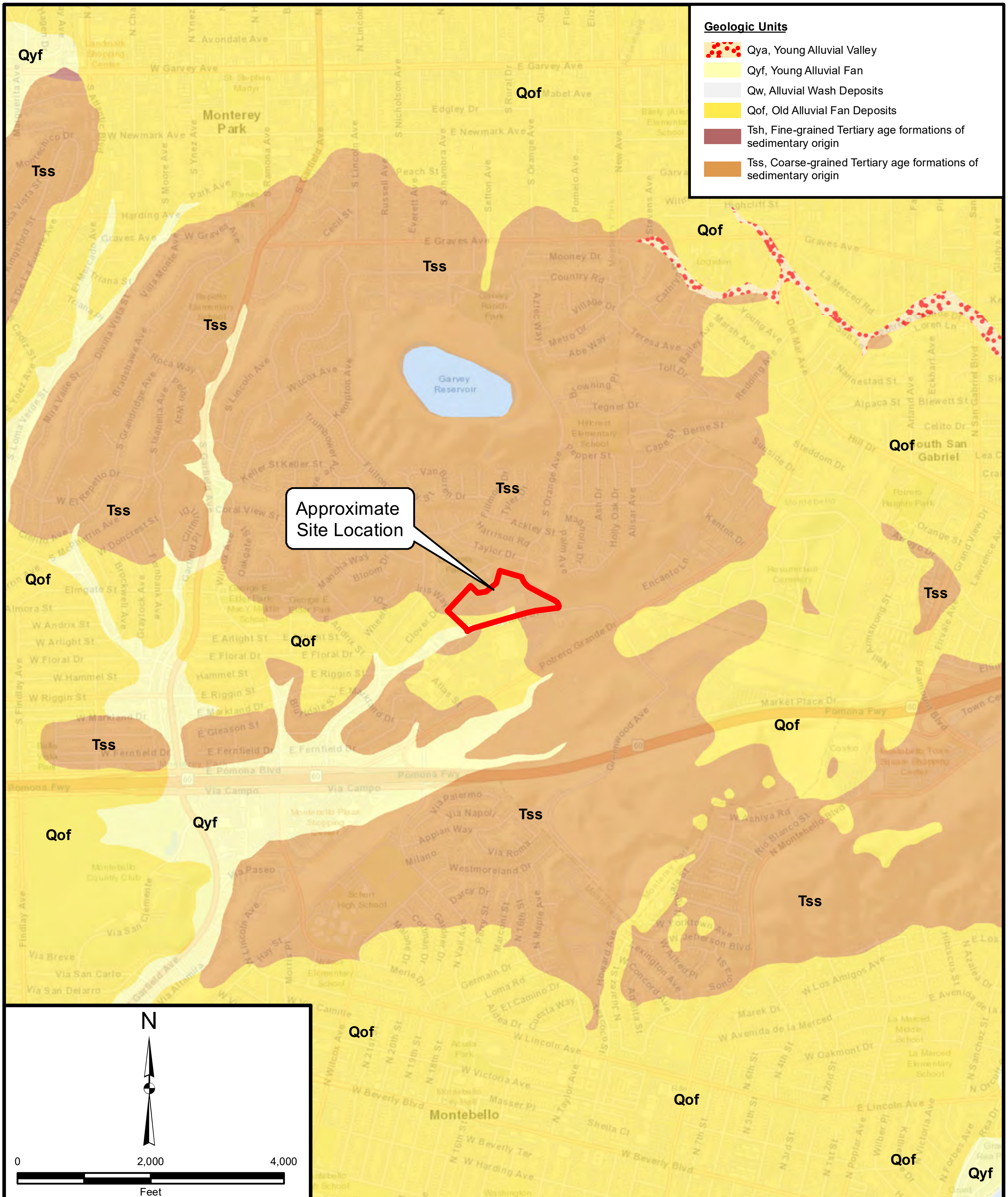
REMEDIAL GRADING MAP (1978)

Proposed Data Center Facility
1977 Saturn Street
Monterey Park, California

FIGURE 2B

Leighton

Map Saved as Z:\Project Files\SA-TZ\SDCFMONT\19850 - 1977 Saturn Monterey Park\Verdantas Folder Structure\GIS\Maps\19850_F02B_RM_2024-02-02.mxd on 1/24/2024 2:31:39 PM



Project: 19850	Eng/Geol: JAR
Scale: 1 " = 2,000 '	Date: December 2023

Reference: Southern California USGS Geology in GIS Format served by California Geological Survey, 2018.

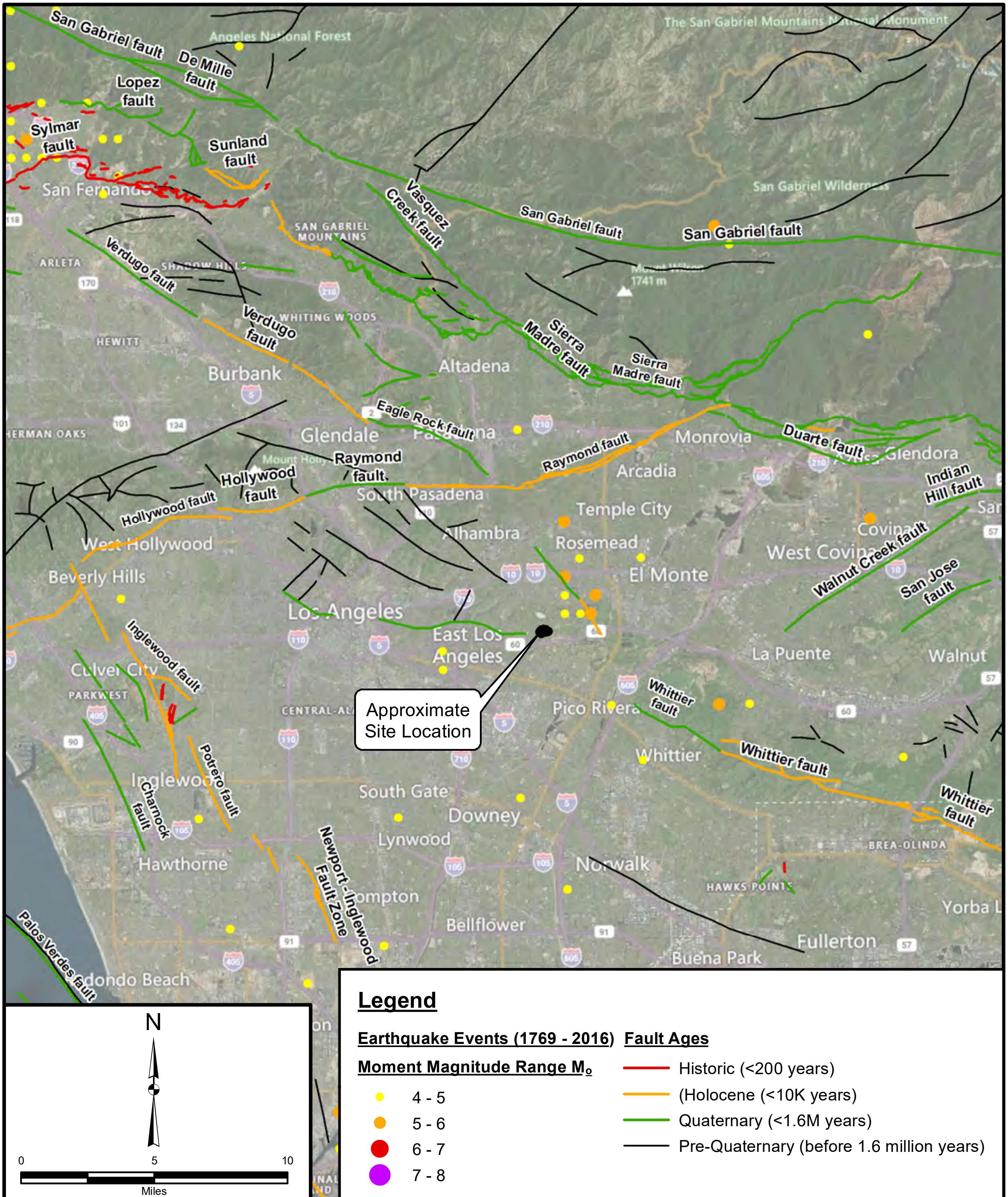
REGIONAL GEOLOGY MAP

Proposed Data Center Facility
1977 Saturn Street
Monterey Park, California

FIGURE 3

Leighton

Map Saved as Z:\Project Files\SA-TZ\SDCFMONT\19850 - 1977 Saturn Monterey Park\Verdantas Folder Structure\GIS\Maps\19850_F03_RGM_2023-12-07.mxd on 12/7/2023 12:00:17 PM (btran)



Project: 19850	Eng/Geol: JAR
Scale: 1 " = 5 miles	Date: December 2023

Reference: ESRI ArcGIS Online 2023
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Author: Leighton Geomatics (btran)

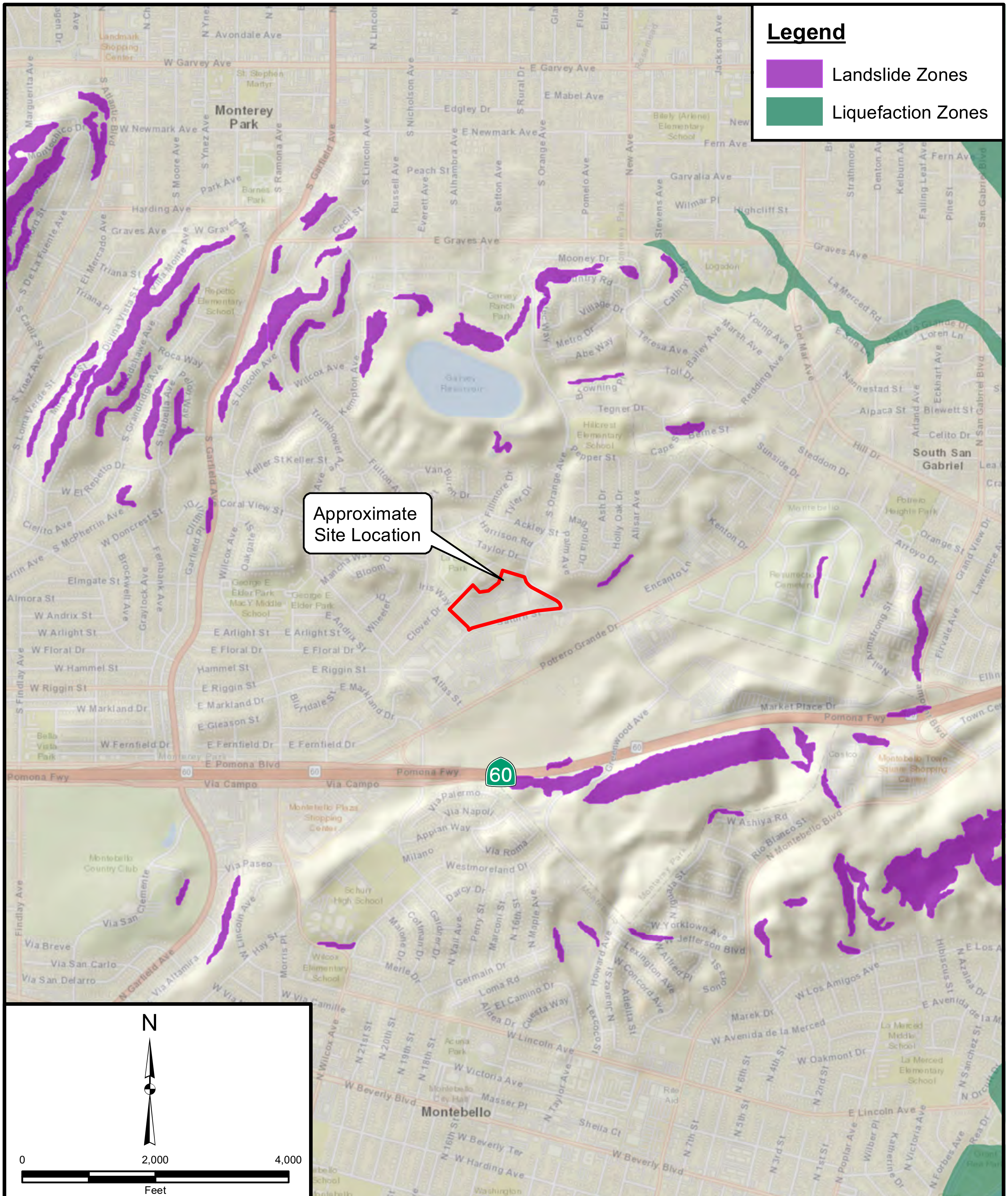
REGIONAL FAULT AND HISTORICAL SEISMICITY MAP

Proposed Data Center Facility
1977 Saturn Street
Monterey Park, California

FIGURE 4

Leighton

Map Saved as Z:\Project Files\SA-TZ\SDCFMONT\19850 - 1977 Saturn Monterey Park\Verdantas Folder Structure\GIS\Maps\19850_F04_RF&HSM_2023-12-07.mxd on 12/7/2023 11:18:02 AM



Project: 19850	Eng/Geol: JAR
Scale: 1 " = 2,000 '	Date: December 2023

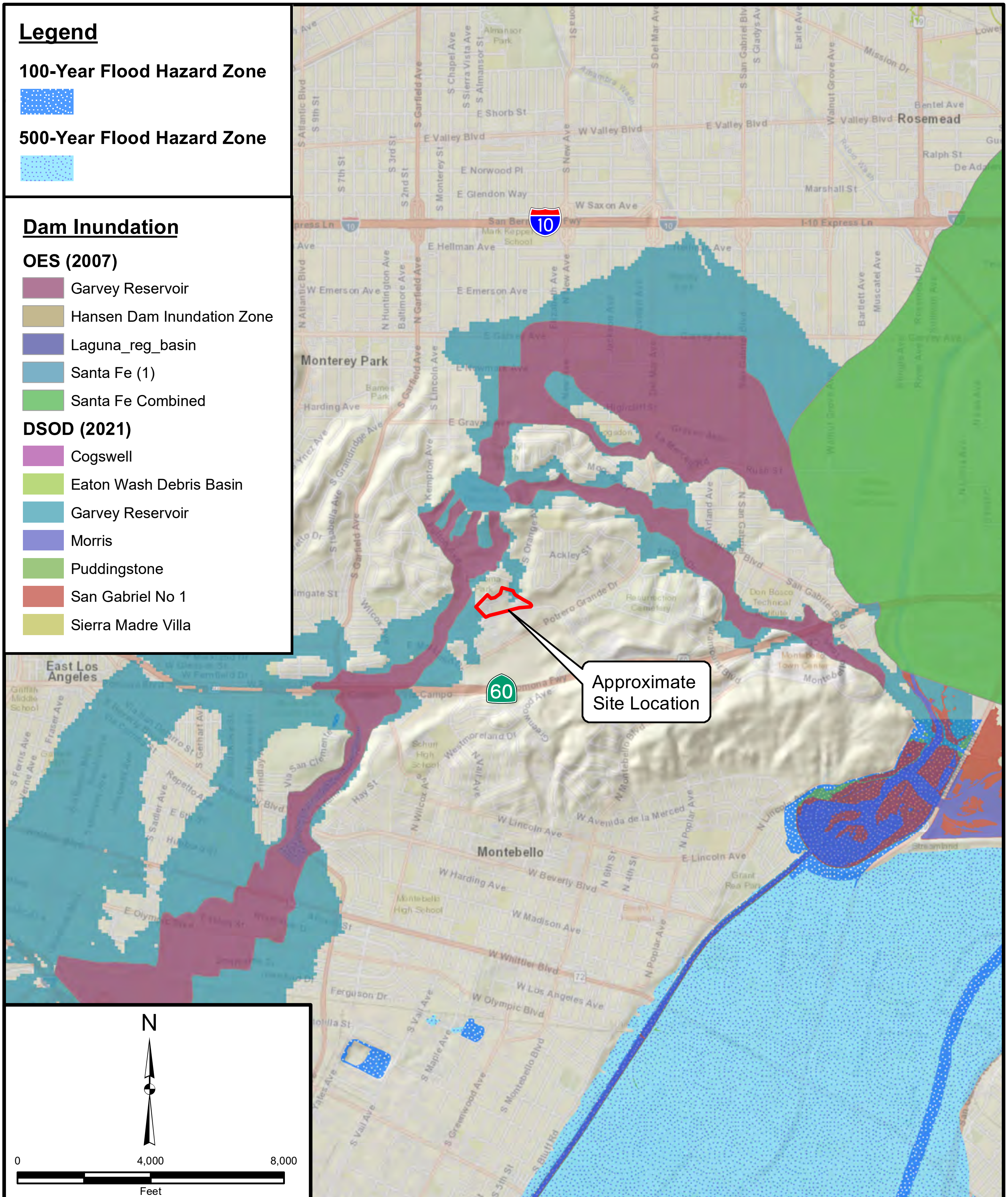
Service Layer Credits: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

SEISMIC HAZARD MAP

Proposed Data Center Facility
1977 Saturn Street
Monterey Park, California

FIGURE 5

Leighton



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Scale: 1 " = 4,000 '	Date: December 2023

Reference: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community
FEMA (http://www.fema.gov/index.shtm), DWR (http://www.dwr.ca.gov)

FLOOD HAZARD AND DAM INUNDATION MAP

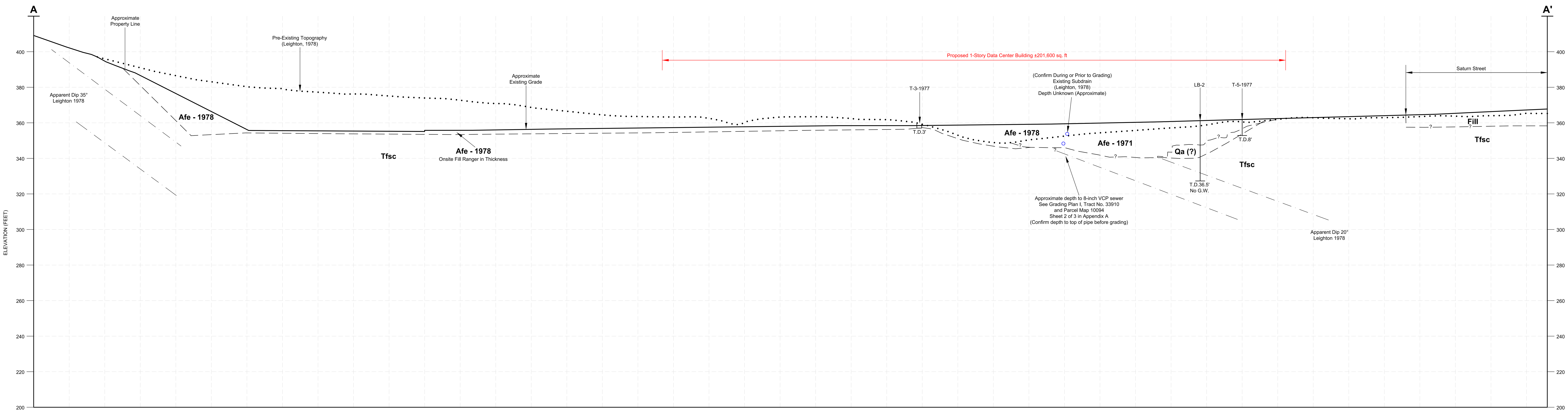
Proposed Data Center Facility
1977 Saturn Street
Monterey Park, California

FIGURE 6

Leighton

Map Saved as Z:\Project Files\SA-TZ\SDCFMONT\19850 - 1977 Saturn Monterey Park\Verdantas Folder Structure\GIS\Maps\19850_F06_FH&DIM_2023-12-07.mxd on 12/7/2023 11:37:42 AM Author: KVM (btran)

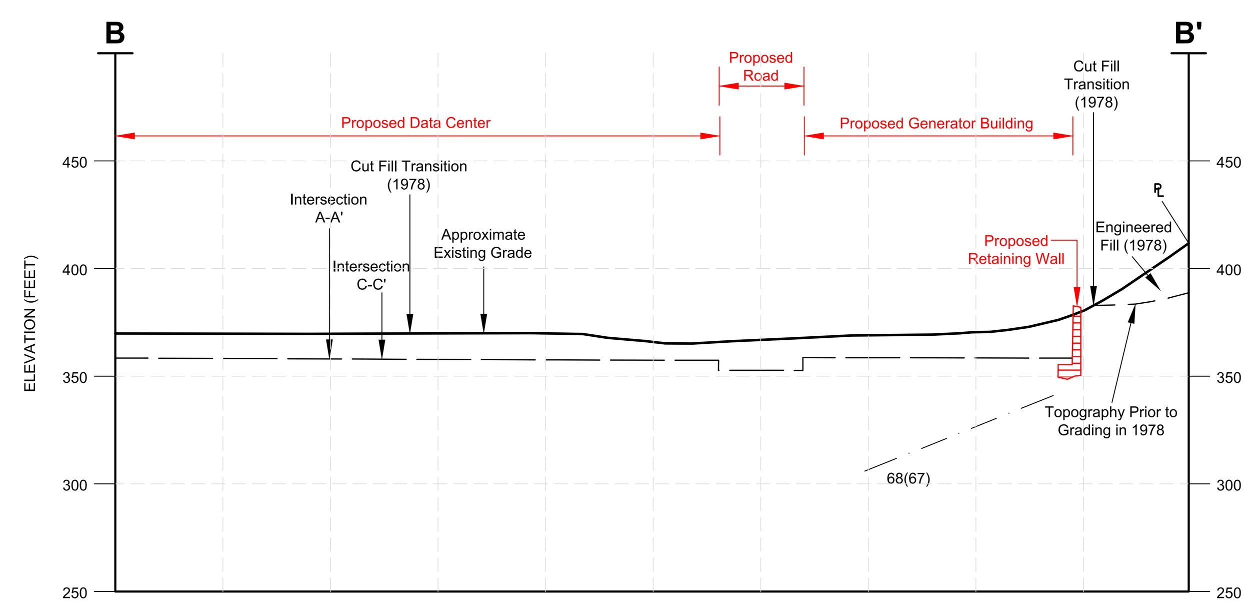

LEGEND
LB-5
Approximate Location of Hollow-Stem Auger Boring Showing Total Depth (T.D.), Depth to Earth Units, and Groundwater in Feet Below Ground Surface.
LB-1-2015
Approximate Location of Hollow Stem Auger Boring (Leighton, 2015)
T-17
Approximate Location of Exploratory Test Trenches (Leighton, 1977)
Tfsc
cg - conglomerate
ss - sandstone
slt - siltstone
Toe EL.347±
Approximate toe of 1.5:1 (horizontal:vertical). Slope with toe of slope elevations in 1978. Elevations shown as approximate See sheet 2 of 3 in Appendix A. (Leighton, 1978)
C F
Approximate location of cut/fill line from As Graded Geotechnical Map in Leighton and Associates (1978). Queried where uncertain.
C C'
Geologic cross sections (see Plate 2) for interpretations.
EXISTING WATER TANKS
APPROXIMATE LIMIT OF TREE CANOPY
EXISTING RESIDENTIAL
EXISTING COMMERCIAL/INDUSTRIAL
EX FENCE TO REMAIN
EX WALL TO REMAIN
EXISTING OPEN SPACE
S ORANGE AVE
SATURN ST
FINISHED FLOOR 359.00
LB-5 T.D.10.88' No G.W. Afe@0-1.13' Tfsc@1.13'
LB-4 T.D.11.5' No G.W. Afe@0-3' Tfsc@3'
LB-3 T.D.16.5' No G.W. Afe@0-9' Tfsc@9'
LB-1 T.D.31.5' No G.W. Afe@0-16.5' Tfsc@16.5'
LB-2 T.D.36.5' No G.W. Afe1971-78@0-11.25' Qa@11.25-25' Tfsc@25'
LB-1-2015 T.D.35.7'
T-1
T-2
T-3
T-4
T-5
T-17
A
A'
B
B'
C
C'
N
0 50 100
Feet
GEOTECHNICAL MAP
Proposed Data Center Facility
1977 Saturn Street
Monterey Park, California
PLATE 1
Scale: 1 " = 50 feet
Date: February 2024
Proj: 19850
Eng/Geol: JAR
Leighton
Base Map: Conceptual Grading Plan, Sheet C1.0 by Gensler (Landscape Architecture)



Afe-1978 - Leighton and Associates Inc., November 16, 1978,
Geotechnical Report of Rough Grading Tract 33910 and Parcel 10094, City of Monterey Park, California

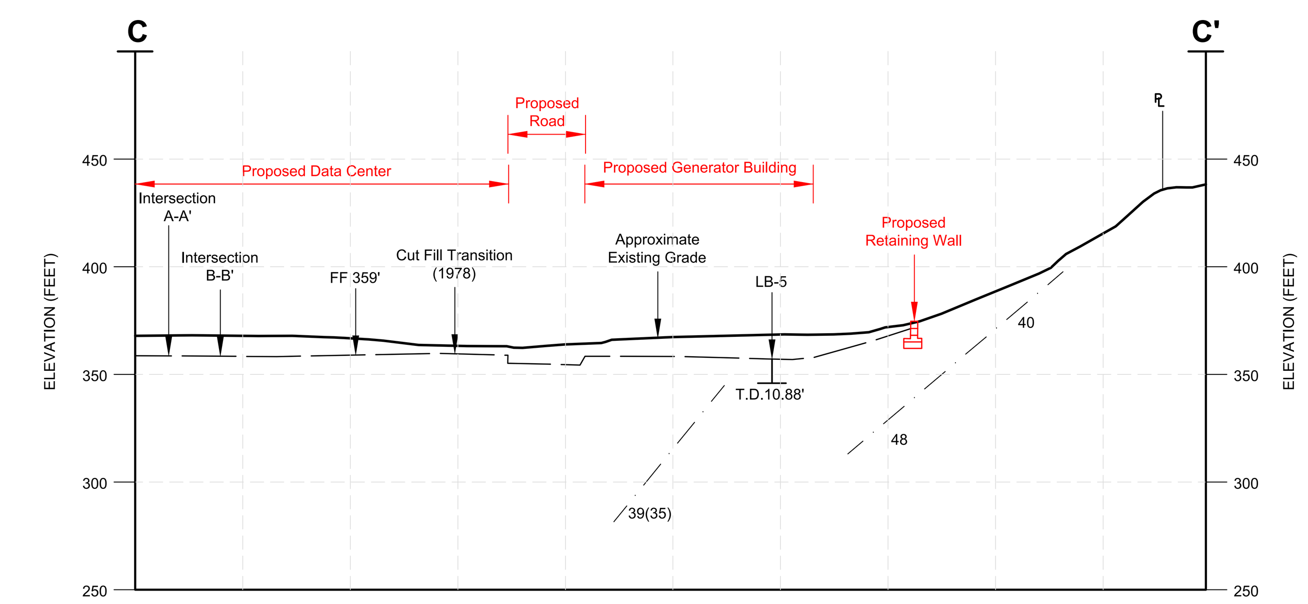
Afe - 1971 - Donald R. Warren Company, September 2, 1971, Final Report on Compaction Tract 31125, Lots 1 Thru 7
and Offsites, Portrero Grande and Saturn Streets, Monterey Park, California

Scale: 1:=20'



N20°E

Scale: 1:=50'



N12°W

Scale: 1:=50'

SEE PLATE 1 FOR CROSS SECTION LOCATIONS

GEOTECHNICAL CROSS SECTIONS A-A', B-B' AND C-C'	PLATE 2
Proposed Data Center Facility 1977 Saturn Street Monterey Park, California	Scale: As Shown
	Date: January 2024
Leighton	Proj: 19850
	Eng/Geol: JAR

APPENDIX A

REPORT OF ROUGH GRADING

Leighton
a verdantas company

1978-11-16 770408.002 GEO REPT OF ROUGH GRDING, TR 33910

LEIGHTON and ASSOCIATES
INCORPORATED

IRVINE OFFICE COPY

SOIL ENGINEERING TESTING GEOLOGY ENVIRONMENTAL SCIENCES

November 16, 1978

Project No. 277408-02

TO: McCaslin Pacific Properties
701 West Foothill Boulevard
Arcadia, California 91006

ATTENTION: Mr. Mark McCaslin

SUBJECT: Geotechnical Report of Rough Grading,
Tract 33910 and Parcel Map 10094,
City of Monterey Park, California

Introduction

In accordance with your authorization of July 14, 1978, we have provided the required geotechnical inspection, testing and mapping during rough grading of the subject areas.

Our services were performed in conformance with the City of Monterey Park requirements.

The purpose of this report is to summarize the geotechnical conditions of the area and to present our recommendations for future construction.

The 40-scale Grading Plan (Sheets 2 and 3 of 3) prepared by Lockman and Associates served as the base for the accompanying As-Graded Geotechnical Map.

Accompanying Map and Appendices

As-Graded Geotechnical Map (40-scale) - In Pocket
Appendix A - Preliminary Foundation Recommendations
Appendix B - Laboratory Test Data
Appendix C - Summary of Field Density Tests
Appendix D - Geotechnical Reports Referenced

21308 PATHFINDER ROAD, SUITE 204, DIAMOND BAR, CALIFORNIA 91765 (714) 598-2856 • (213) 965-4874
IRVINE • WESTLAKE/VENTURA • REDWOOD CITY • DIAMOND BAR/WALNUT • SAN BERNARDINO/RIVERSIDE • SAN DIEGO

Summary of Grading

Rough grading commenced on July 10, 1978 and was completed on October 5, 1978.

Site Preparation

Trash and heavy brush were removed from the site prior to grading. An average depth of 3 feet of unsuitable material was removed prior to fill placement from the drainage area located between Lots 2 and 11.

Fill Placement

Fill materials were spread in level lifts approximately 8 inches in thickness. Moisture was added as necessary by a water truck and the fill materials were mixed to a uniform moisture. Compaction of each lift was then completed through the use of double 5x5 sheepsfoot compactors and self-powered compaction equipment in advance of further fill placement. Fill slopes were either backrolled with a sheepsfoot roller at 4-foot vertical intervals or overfilled and cut back to expose the compacted inner core.

Repair of Previously Existing Slopes

The slopes to the rear of Parcels 2 and 3 of Parcel Map 10094 were repaired as recommended in our report of December 21, 1977 (Referenced Report 2 of Appendix D).

Inspection and Testing

Soil engineering inspection and testing of grading operations was performed full time. Geologic inspection and mapping was conducted periodically as necessary. The standard used for the minimum compaction requirement was 90 percent of ASTM Test Method D1557-70. Field density tests were taken by the Sand Cone Method (ASTM Test Method D1556-64) and the Drive Cylinder Method (ASTM Test Method D2937-71). The approximate locations of tests taken are shown on the accompanying As-Graded Geotechnical Map. Summary sheets of the density tests taken on the project are submitted as Appendix C. Soil types with maximum density and optimum moisture percentage are presented as Appendix C.

Expansion Index Tests (UBC Standard 29-2) were performed on typical materials from the pad areas and low expansion was exhibited in these areas. All lots were sampled and soils were compared texturally to determine in which expansion category they were to be placed.

Summary of Geologic Conditions

The earth materials exposed during grading operations consisted of sedimentary bedrock consisting of sandstone, siltstone and conglomerate.

No landslides or major faulting were exposed by the grading operations.

Adversely-oriented bedding required buttresses to be constructed for support of the cut slopes located southeast of Lots 6 and 10, Tract 33910. The approximate location of the buttress keys and subdrains are shown on the accompanying map.

Four 2 to 3-inch thick clay beds are exposed in portions of Lots 2, 3, 10 and 11 of Tract 33910 as shown on the accompanying map.

Subdrains

The subdrain which was reported to have been installed under the existing fill of the tract north of Lot 2, Tract 33910 could not be located with extensive backhoe probes. An observation well was provided. The approximate location of this well is shown on the accompanying map.

Subdrains were constructed in the buttress fills and are shown on the accompanying map.

Groundwater

Seepages were encountered during excavation of the cut slope to the rear of Lot 1, Tract 33910 and Parcel 3 of Parcel Map 10094, along the property line of Lots 2 and 3, Tract 33910 and in the northeast end of the buttress excavation, Lot 6, Tract 33910.

The seepages in the cut slope have essentially dried up with two minor wet areas remaining as shown on the accompanying map.

It is anticipated that the seepages on the property line of Lots 2 and 3 of Tract 33910 will dry and present no further problems. However, these areas should receive additional evaluation if building foundations or pavement are planned over the moist areas.

The buttress subdrain was extended to drain the seep exposed by the buttress excavation to the rear of Lot 6.

CONCLUSION AND RECOMMENDATIONS

Conclusion

Rough grading was accomplished in a workmanlike manner, in accordance with the requirements of the City of Monterey Park and our recommendations.

Recommendations

1. Foundation recommendations presented in Appendix A are necessarily tentative and preliminary as building layouts, types of construction and building locations are unknown. When these parameters are determined, they should be reviewed by the soil engineer for final foundation recommendations.

   Special foundation requirements may be necessary where foundations cross clay beds or are founded upon both cut and fill.

2. Slopes, both cut and fill, should perform satisfactorily when properly planted and maintained. Positive surface drainage away from the tops of slopes should be maintained.

3. Utility trench backfill should be placed as presented in Referenced Report 1 of Appendix D.

4. Pavement Sections

   Two representative samples were tested for their "R"-Value (Appendix B). In general, the clayey sand with traces of gravel material on the site yield moderate "R"-Values (34 to 37).

   Recommended street, driveway and parking lot sections are 3 inches AC on 4 inches of aggregate base.

   Sections at driveway entrances and service areas with frequent truck traffic should be increased by 2 to 3 inches of additional aggregate base.

   Prior to placement of base, the upper 8 inches of subgrade in bedrock areas must be thoroughly scarified or excavated and recompacted to minimize differential effects of the bedded materials. This subgrade should be moisture conditioned and then compacted to a minimum of 90 percent relative compaction.

5. Observation well should be monitored through one storm season to detect potential presence of free water. Results of the monitoring will determine future disposition of the well. A more permanent cover should be installed on the well to enable the monitoring and maintain security.

6. Construction should be inspected at the following stages by the soil engineer:

   A. After preparation of building plans.

   B. After excavation of foundation trenches.

   C. During backfilling of deep construction. Prior to paving or other construction over shallow backfill.

   D. After compaction of street subgrade and base.

   E. When any unusual conditions are encountered during construction.

Respectfully submitted,

David A. Adams
Project Geologist

Donald E. Brockway
RCE 13552

Steven R. McCollum
Engineering Geologist EG 984

/mp

Distribution: (5) Addressee

(2) Lockman and Associates
Attention: Mr. Norbert W. Weinberg

APPENDIX A

APPENDIX A

PRELIMINARY FOUNDATION RECOMMENDATIONS

1. Footing Type

   Shallow square or continuous-type footings are suitable for support of light to medium construction.

2. Footing Embedment Minimum

One-story or equivalent construction	- 12 inches
Two-story or equivalent construction	- 18 inches
Interior Footing (with minimum 4-inch slab around footing)	- 12 inches
Exterior Column Footing	- 18 inches

3. Minimum Reinforcing of Continuous-Type Footings*

   One No. 4 bar in both top and bottom.

   *Structural consideration may require additional reinforcing.

4. Maximum Allowable Soil Bearing Pressure*

12-inch minimum embedment	- 1500 psf
18-inch minimum embedment	- 2000 psf

   *Increases in bearing pressure may be permissible for extra embedment or footing on bedrock.

5. Slab Thickness Minimum

   4 inches. Additional thickness may be necessary due to special structural considerations.

6. Minimum Slab Reinforcing

   6x6-10/10 WWF carefully placed at the center of the slab thickness. Additional reinforcing may be necessary due to special structural considerations.

APPENDIX B

LEIGHTON and ASSOCIATES
INCORPORATED

MAXIMUM DENSITY TEST RESULTS

SOIL TYPE OR LOCATION	SOIL DESCRIPTION	OPTIMUM MOISTURE(%)	MAXIMUM DRY DENSITY(PCF)
1	Silty Sand with Trace of Gravel	8	129
2	Clayey Sandy Silt with Trace of Gravel	10	125

EXPANSION INDEX TEST RESULTS

SAMPLE LOCATION	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE(%)	VOLUMETRIC SWELL(%)	EXPANSION INDEX	EXPANSIVE CLASSIFICATION
Lots 1, 4 & 5 (Acacia Circle)	9.3	112.3	16.3	1.20	12.0	Very Low
Lots 2 & 3 (Acacia Circle)	11.4	104.0	22.8	3.10	31.0	Low
Lots 1, 6-8 & 11	11.2	104.4	20.2	.07	7.0	Very Low
Lots 2, 3, 9, 12 & 14	11.0	107.0	23.4	4.30	43.0	Low
Lot 4		Not Applicable (Part of Nursery)				Medium
Lots 5 & 10	18.4	85.5	38.2	6.50	65.0	Medium

Project No: 277408-02

Plate No: B-i

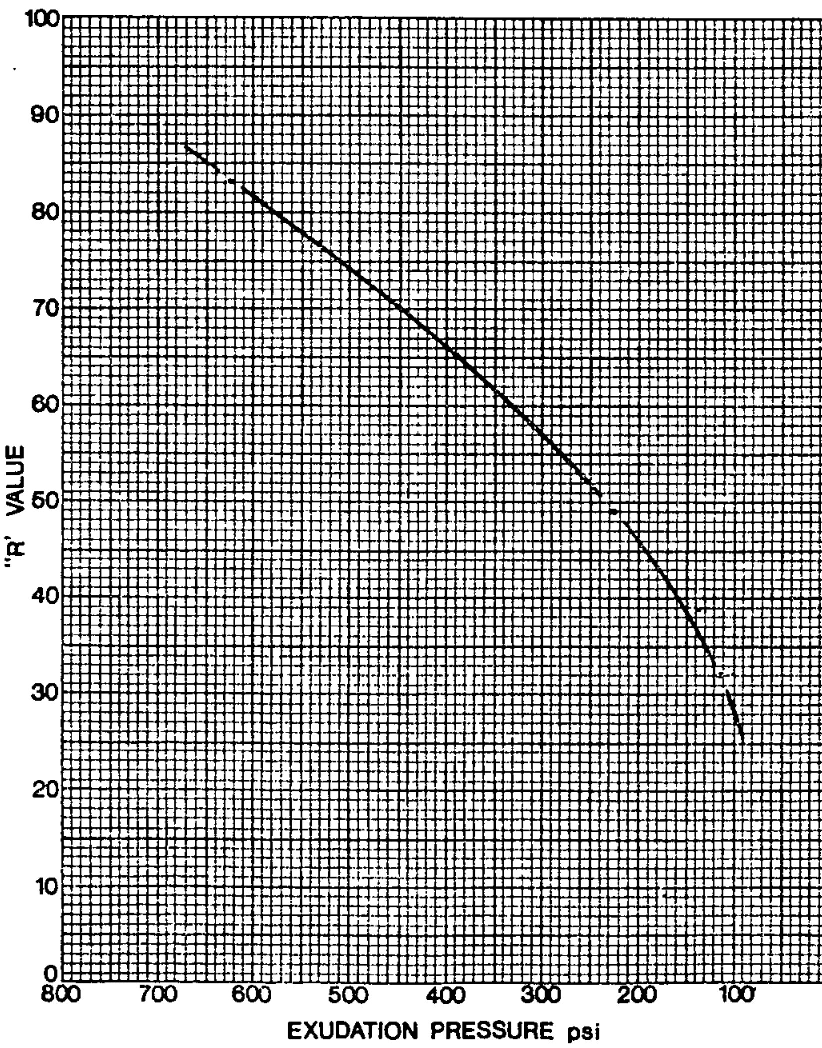
332 A (8/77)

SAMPLE: Gravelly Clayey Coarse Sand
#2, Leighton and Assoc., Proj. 277408-02, Oakwood McCaslin, Orange Avenue

	TEST SPECIMAN		A	B	C	D	E
	DATE TESTED		10-2-78	10-2-78	10-2-78		
SPECIMEN FABRICATION	Compactor Air Pressure	psi	350	350	350		
	Initial Moisture	%	4.0	4.0	4.0		
	Moisture at Compaction	%	10.0	11.8	10.9		
	Briquette Height	in.	2.426	2.503	2.481		
	Density	pcf	127.1	125.6	127.7		
	EXUDATION PRESSURE	psi	625	115	227		
	EXPANSION PRESSURE DIAL		.0004	.0001	.0002		
STABIL-OMETER	P_h at 1000 pounds	psi	12	42	35		
	P_h at 2000 pounds	psi	19	97	69		
	Displacement	turns	3.70	3.45	3.45		
	"R" Value		84	32	49		
	CORRECTED "R" VALUE		83				

"R" VALUE AT 300 PSI EXUDATION PRESSURE = 57

GRAIN SIZE DISTRIBUTION		
SIEVE	AS RECEIVED	AS TESTED
3		
2½		
2		
1½		
1		
¾		
½		
⅜		
#4		
#8		
#16		
#30		
#50		
#100		
#200		
.05mm		
.005mm		
.001mm		
LIQUID LIMIT		
PLASTIC LIMIT		
PLASTICITY INDEX		
SAND EQUIVALENT		



SOUTHERN CALIFORNIA SOIL & TESTING, INC.
17811 SKY PARK CIRCLE DRIVE
IRVINE, CALIFORNIA 92714

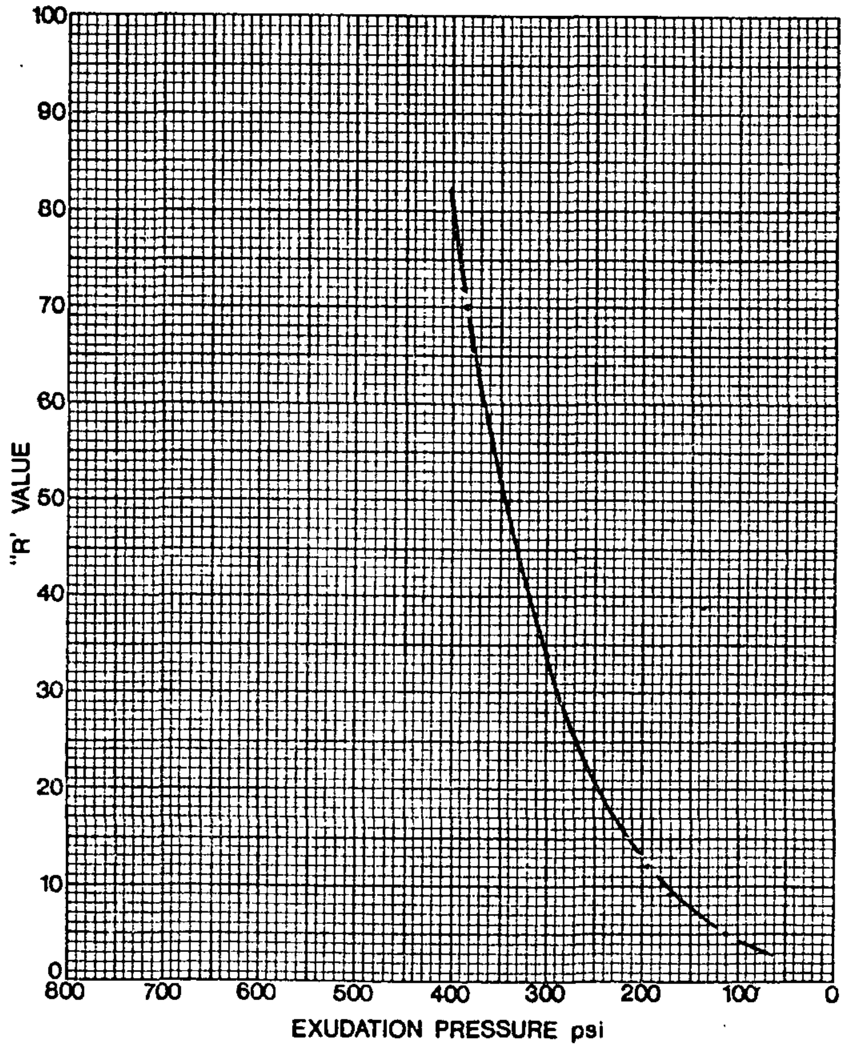
Laboratory Test Results	
BY RMR	DATE 10-2-78
JOB NO. 52023	Plate 1

SAMPLE: Bag #5, Leighton and Asso., Proj. 277408-02, Oakwood McCaslin, Gemini St.

	TEST SPECIMAN		A	B	C	D	E
	DATE TESTED		10-2-78	10-2-78	10-2-78		
SPECIMEN FABRICATION	Compactor Air Pressure	psi	130	290	350		
	Initial Moisture	%	6.9	6.9	6.9		
	Moisture at Compaction	%	12.2	10.4	8.7		
	Briquette Height	in.	2.496	2.485	2.46		
	Density	pcf	126.0	129.7	132.1		
	EXUDATION PRESSURE	psi	111	195	382		
	EXPANSION PRESSURE DIAL		-0-	.0004	.0008		
STABIL-OMETER	P_h at 1000 pounds	psi	72	59	17		
	P_h at 2000 pounds	psi	151	134	37		
	Displacement	turns	3.25	3.50	3.50		
	"R" Value		5	12	70		
	CORRECTED "R" VALUE						

"R" VALUE AT 300 PSI EXUDATION PRESSURE = 34

GRAIN SIZE DISTRIBUTION		
SIEVE	AS RECEIVED	AS TESTED
3		
2½		
2		
1½		
1		
¾		
½		
⅜		
#4		
#8		
#16		
#30		
#50		
#100		
#200		
.05mm		
.005mm		
.001mm		
LIQUID LIMIT		
PLASTIC LIMIT		
PLASTICITY INDEX		
SAND EQUIVALENT		



SOUTHERN CALIFORNIA SOIL & TESTING, INC.
17811 SKY PARK CIRCLE DRIVE
IRVINE, CALIFORNIA 92714

Laboratory TEst Results	
BY DS/RMR	DATE 10-2-78
JOB NO. 52023	Plate 2

APPENDIX C

LEIGHTON and ASSOCIATES
INCORPORATED

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
1	7/10	N26825, E52920	2	341	112.5	125.0	8.0	10.0	90
2	7/10	N26900, E52960	2	343	112.0	125.0	9.3	10.0	90
3	7/11	N27080, E52850	2	344	116.6	125.0	8.3	10.0	93
4	7/11	N26910, E52850	2	344	117.1	125.0	8.3	10.0	94
5	7/12	N26970, E52930	2	345	123.6	125.0	12.3	10.0	99
6	7/12	N27115, E52910	2	344	115.0	125.0	12.3	8.0	92
7	7/13	N27050, E52710	2	343	112.4	125.0	12.3	10.0	90
8	7/13	N27155, E52820	2	341	117.0	125.0	8.7	10.0	94
9	7/13	N26930, E52960	2	342	121.9	125.0	14.0	10.0	98
10	7/13	N26860, E52890	2	342	120.0	125.0	11.0	10.0	96
11	7/14	N27060, E52810	1	346	122.2	129.0	22.0	8.0	95
12	7/14	N27020, E52780	1	346	116.6	129.0	14.0	8.0	90
13	7/14	N27090, E52770	2	346	112.8	125.0	13.0	10.0	90
14	7/14	N27035, E52730	2	344	112.1	125.0	10.0	10.0	90
15	7/14	N27095, E52890	1	344	121.5	129	11.0	8.0	94
16	7/14	N27140, E52760	1	345	118.7	129.0	13.6	8.0	92
17	7/17	N27115, E52825	1	346	115.6	129.0	6.0	10.0	90
18	7/17	N27195, E52815	2	346	123.9	125.0	5.0	10.0	99
19	7/17	N27225, E52880	2	346	117.6	125.0	12.3	10.0	94
20	7/17	N27160, E52900	2	347	116.8	125.0	12.0	10.0	93
21	7/17	N27225, E52920	2	346	119.3	125.0	12.3	10.0	95
22	7/17	N27140, E52950	2	347	117.5	125.0	13.0	10.0	94
23	7/18	N26940, E53100	1	347	115.8	129.0	9.0	8.0	90
24	7/18	N27045, E53810	1	348	118.2	129.0	10.0	8.0	92

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
25	7/18	N26980, E53125	2	350	112.3	125.0	8.0	10.0	90
26	7/18	N27020, E53100	2	351	112.2	125.0	10.0	10.0	90
27	7/19	N26970, E53220	2	346	118.8	125.0	8.7	10.0	95
28	7/19	N26920, E53230	2	347	113.5	125.0	12.3	10.0	91
29	7/19	N26935, E53310	2	348	112.6	125.0	10.0	10.0	90
30	7/19	N26810, E53240	2	342	120.5	125.0	14.0	8.0	93
31	7/19	N26875, E53260	1	343	126.4	129.0	10.0	8.0	98
32	7/19	N26860, E53321	1	345	119.1	129.0	10.0	8.0	92
33	7/19	N26825, E53280	1	346	118.7	129.0	9.6	8.0	92
34	7/20	N26960, E53300	1	350	117.6	129.0	9.0	8.0	91
35	7/20	N26960, E53385	1	348	118.2	129.0	10.0	8.0	92
36	7/20	N26880, E53380	1	348	119.1	129.0	10.0	8.0	92
37	7/20	N26870, E53345	1	350	119.2	129.0	9.0	8.0	92
38	7/20	N26895, E53210	1	350	119.3	129.0	9.0	8.0	92
39	7/20	N26975, E53480	1	364	120.9	129	10.0	8.0	93
40	7/21	N27490, E53620	1	370	121.6	129.0	11.7	10.0	94
41	7/21	N27380, E53705	2	363	118.1	125.0	11.7	10.0	94
42	7/21	N27240, E53560	1	362	118.0	129.0	11.7	8.0	91
43	7/21	N27135, E53590	1	365	116.5	129.0	10.0	8.0	90
44	7/21	N27220, E53630	1	364	117.3	129.0	11.7	8.0	91
45	7/21	N27120, E53540	1	366	117.3	129.0	10.0	8.0	91
46	7/24	N27050, E53500	2	366	114.6	125.0	11.0	10.0	92
47	7/24	N27015, E53565	1	364	117.8	129.0	12.0	8.0	91
48	7/24	N27015, E53580	1	366	119.8	129.0	11.0	8.0	93

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf Field	Dry Dens., pcf Maximum	Moisture, % Field	Moisture, % Opt.	Relative Compaction
	1978								
49	7/24	N27420, E53620	1	373	125.5	129.0	12.0	8.0	97
50	7/25	N27440, E53670	2	372	113.9	125.0	11.0	8.0	91
51	7/25	N27425, E53700	1	365	116.3	129.0	11.0	8.0	93
52	7/25	N27305, E53650	1	374	119.6	125.9	10.0	8.0	93
53	7/25	N21165, E53585	1	374	118.3	129.0	10.0	8.0	92
54	7/25	N27455, E53625	1	374	119.1	129.0	10.0	8.0	92
55	7/25	N27460, E53680	1	376	122.9	129.0	9.0	8.0	95
56	7/25	N26905, E53515	1	348	116.5	129.0	10.0	8.0	90
57	7/26	N27090, E53545	1	368	121.6	129.0	10.0	8.0	94
58	7/26	N26930, E53520	1	350	115.7	129.0	10.0	8.0	90
59	7/26	N27350, E53610	2	376	114.0	125.0	8.0	10.0	91
60	7/26	N27340, E53660	2	378	113.4	125.0	8.0	10.0	90
61	7/27	N27190, E53575	1	375	115.9	129.0	9.8	8.0	90
62	7/27	N27080, E53580	1	370	118.0	129.0	28.0	8.0	91
63	7/27	N27015, E53480	1	368	116.6	129.0	11.6	8.0	90
64	7/27	N28050, E53525	1	368	116.3	129.0	11.0	8.0	90
65	7/27	N27370, E53660	2	374	114.9	125.0	11.6	10.0	92
66	7/27	N26980, E54325	1	360	120.9	129.0	10.0	8.0	94
67	7/27	N26840, E54220	1	359	119.2	129.0	10.0	8.0	92
68	7/28	N26960, E54150	1	359	116.5	129.0	10.0	8.0	90
69	7/28	N26880, E54332	1	372	115.9	129.0	11.3	8.0	90
70	7/31	N26920, E54235	1	356	117.3	129.0	11.0	8.0	91
71	7/31	N26880, E54180	1	357	115.8	129.0	9.0	8.0	90
72	7/31	N26960, E54235	1	361	116.3	129.0	11.0	8.0	90

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
73	7/31	N26905, E53920	1	354	118.6	129.0	14.6	8.0	92
74	8/1	N26740, E53965	1	357	116.3	129.0	11.0	8.0	90
75	8/1	N26860, E53945	1	356	122.6	129.0	14.6	8.0	91
76	8/1	N26880, E54012	1	358	117.0	129.0	12.0	8.0	91
77	8/1	N26580, E54040	1	372	118.2	129.0	10.0	8.0	92
78	8/1	N26705, E54180	2	371	115.9	125.0	10.0	10.0	93
79	8/2	N26620, E54093	2	374	113.5	125.0	9.6	10.0	91
80	8/2	N26670, E54130	1	375	119.0	129.0	9.3	8.0	92
81	8/3	N26600, E54070	1	376	115.6	129.0	10.0	8.0	90
82	8/3	N26935, E54375	2	373	113.2	125.0	6.0	10.0	91
83	8/3	N26815, E54220	1	361	117.2	129.0	11.0	8.0	91
84	8/3	N26585, E54055	2	378	113.5	125.0	12.0	8.0	91
85	8/3	N26680, E54150	1	376	115.9	129.0	23.0	8.0	90
86	8/3	N26745, E54052	1	358	115.9	129.0	10.0	8.0	90
87	8/4	N26570, E54050	1	380	123.0	129.0	23.0	8.0	95
88	8/4	N26890, E53950	1	358	118.8	129.0	18.0	8.0	92
89	8/4	N27020, E54400	1	380	116.6	129.0	9.0	8.0	90
90	8/4	N26940, E54350	2	375	114.1	125.0	9.0	10.0	91
91	8/4	N26840, E53960	1	357	115.6	129.0	10.0	8.0	90
92	8/4	N26895, E54290	1	360	119.3	129.0	12.3	8.0	92
93	8/4	N26950, E54180	1	361	119.6	129.0	12.0	8.0	93
94	8/5	N26930, E54085	1	360	115.8	129.0	9.0	8.0	90
95	8/5	N26775, E53985	1	359	118.4	129.0	9.0	8.0	91
96	8/5	N26980, E54360	1	377	115.6	129.0	10.0	8.0	90

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf Field	Dry Dens., pcf Maximum	Moisture, % Field	Moisture, % Opt.	Relative Compaction
	1978								
97	8/5	N26610, E54060	1	380	118.1	129.0	13.2	8.0	91
98	8/5	N26540, E54010	1	350	117.3	129.0	10.0	8.0	90
99	8/5	N26475, E53985	1	336	118.5	129.0	9.0	8.0	91
100	8/5	N26820, E53940	1	360	123.1	129.0	11.0	8.0	95
101	8/6	N26715, E53055	1	340	117.0	129.0	18.0	8.0	90
102	8/6	N26640, E53140	1	342	120.2	129.0	9.0	8.0	93
103	8/6	N26450, E53150	1	341	118.3	129.0	10.0	8.0	91
104	8/6	N26485, E54015	1	356	117.4	129.0	10.0	8.0	91
105	8/6	N26735, E53130	1	341	119.3	129.0	9.0	8.0	92
106	8/6	N26415, E53185	1	343 R.T. on 107	115.4	129.0	9.0	8.0	89
107	8/6	N26415, E53185	1	343 R.T. of 106	124.7	129.0	9.0	8.0	96
108	8/6	N26520, E54025	1	354	119.6	129.0	8.0	8.0	92
109	8/6	N26870, E53490	1	352	117.6	129.0	30.0	8.0	91
110	8/7	N26840, E53255	1	344	116.5	129.0	10.0	8.0	90
111	8/7	N26990, E53485	1	351	118.1	129.0	11.0	8.0	91
112	8/7	N26915, E53480	1	354	119.8	129.0	11.0	8.0	92
113	8/7	N26810, E53290	1	347	118.4	129.0	9.0	8.0	91
114	8/7	N26660, E53260	1	350	115.8	129.0	9.0	8.0	90
115	8/7	N26615, E53070	1	343	116.9	129.0	8.0	8.0	90
116	8/7	N26445, E53240	2	342	112.5	125.0	8.0	10.0	90
117	8/7	N26640, E53220	1	343	115.8	129.0	9.0	8.0	90
118	8/7	N26545, E53100	1	343	117.2	129.0	11.0	8.0	90
119	8/7	N26820, E53255	1	346	116.3	129.0	11.0	8.0	90

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
120	8/7	N26900, E53420	1	355	116.5	129.0	11.0	8.0	90
121	8/8	N26825, E53372	1	349	121.2	129.0	12.6	8.0	93
122	8/8	N26720, E53262	1	350	118.1	129.0	11.0	8.0	91
123	8/8	N26675, E53035	1	343	120.0	129.0	10.0	8.0	93
124	8/8	N26935, E53460	1	356	119.1	129.0	10.0	8.0	92
125	8/8	N26350, E53250	1	344	115.6	129.0	10.0	8.0	90
126	8/8	N26560, E53165	1	345	120.4	129.0	15.0	8.0	93
127	8/8	N27510, E53728	1	374	122.7	129.0	14.6	8.0	95
128	8/8	N27585, E53680	1	375	117.2	129.0	11.0	8.0	90
129	8/8	N27460, E53807	1	382	118.4	129.0	12.3	8.0	91
130	8/8	N27530, E53695	1	380	115.6	129.0	10.0	8.0	90
131	8/8	N26905, E53420	1	356	116.0	129.0	12.0	8.0	90
132	8/8	N27570, E53750	1	372	115.5	129.0	8.0	8.0	90
133	8/8	N27420, E53785	1	380	116.3	129.0	11.0	8.0	90
134	8/9	N27485, E53790	1	384	119.1	129.0	10.0	8.0	92
135	8/9	N27610, E53660	1	378	120.6	129.0	11.0	8.0	93
136	8/9	N27530, E53660	1	376	115.8	129.0	9.0	8.0	90
137	8/9	N27510, E53755	1	376	120.0	129.0	10.0	8.0	93
138	8/9	N27618, E53740	2	392	113.0	125.0	7.0	10.0	90
139	8/9	N26590, E53400	2	347	111.9	125.0	8.0	10.0	90
140	8/9	N26485, E53320	2	347	115.8	125.0	9.0	10.0	92
141	8/9	N26590, E53245	1	344	117.6	129.0	9.0	8.0	91
142	8/9	N26675, E53285	1	352	117.7	129.0	8.0	8.0	91

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
143	8/9	N26525, E53380	1	349	120.9	129.0	10.0	8.0	93
144	8/9	N26795, E53330	1	351	120.0	129.0	10.0	8.0	93
145	8/10	N26455, E53360	1	350	117.3	129.0	10.0	8.0	90
146	8/10	N26655, E53105	1	345	117.2	129.0	11.0	8.0	90
147	8/10	N26435, E53130	1	343	115.7	129.0	10.0	8.0	90
148	8/10	N26500, E53245	1	345	119.1	129.0	22.0	8.0	92
149	8/10	N26770, E53170	1	343	117.8	129.0	12.0	8.0	91
150	8/10	N26485, E53140	1	344	117.2	129.0	11.0	8.0	90
151	8/10	N26395, E53160	1	345	116.9	129.0	8.0	8.0	90
152	8/10	N26695, E53130	1	346	115.7	129.0	14.0	8.0	90
153	8/10	N26790, E53255	1	347	117.0	129.0	8.0	8.0	91
154	8/10	N26715, E53380	1	355	120.2	129.0	9.0	8.0	93
155	8/10	N26580, E53340	2	349	113.5	125.0	7.0	10.0	90
156	8/11	N26445, E53170	1	347	116.0	129.0	8.0	8.0	90
157	8/11	N26565, E53120	1	347	115.8	129.0	13.6	8.0	90
158	8/11	N26735, E53185	1	345	120.6	129.0	14.0	8.0	93
159	8/11	N26705, E53065	2	345	112.0	125.0	9.3	10.0	90
160	8/11	N26525, E53220	1	347	119.4	129.0	8.0	8.0	92
161	8/11	N26425, E53215	1	345	117.1	129.0	11.0	8.0	90
162	8/11	N26375, E53325	1	346	116.2	129.0	11.0	8.0	90
163	8/11	N26460, E53280	1	349	119.2	129.0	9.0	8.0	92
163	8/11	N26460, E53280	1	349	119.2	129.0	9.0	8.0	92
164	8/11	N26420, E53160	1	347	118.3	129.0	9.0	8.0	91
165	8/11	N26480, E53420	2	349	114.8	125.0	10.0	10.0	91

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
166	8/11	N26490, E53355	1	351	116.9	129.0	8.0	8.0	90
167	8/14	N26415, E53140	1	349	115.5	129.0	8.0	8.0	90
168	8/14	N26640, E53040	1	346	116.0	129.0	14.0	8.0	90
169	8/14	N26770, E54040	1	362	119.9	129.0	12.0	8.0	92
170	8/14	N26900, E53905	1	356	119.3	129.0	13.0	8.0	92
171	8/14	N26520, E53115	1	349	117.8	129.0	12.0	8.0	91
172	8/14	N26480, E53370	1	346	120.0	129.0	14.0	8.0	93
173	8/14	N26355, E53250	1	342	119.3	129.0	13.0	8.0	92
174	8/14	N26500, E53380	2	358	114.5	125.0	13.6	10.0	91
175	8/14	N26915, E54320	1	362	116.6	129.0	9.0	8.0	90
176	8/14	N26960, E54275	1	362	118.1	129.0	11.0	8.0	91
177	8/15	N26890, E54230	2	358	112.2	125.0	9.0	10.0	90
178	8/15	N26960, E54330	2	362	112.2	125.0	8.0	10.0	90
179	8/15	N26845, E54125	1	360	117.0	129.0	12.0	8.0	90
180	8/15	N26880, E54090	1	360	118.0	129.0	7.0	8.0	91
181	8/15	N26915, E54040	1	360	116.6	129.0	9.0	8.0	90
182	8/15	N26820, E53830	1	356	121.7	129.0	10.0	8.0	94
183	8/15	N26620, E53950	1	353	116.9	129.0	8.0	8.0	90
184	8/15	N26390, E53290	1	346	116.6	129.0	9.0	8.0	90
185	8/16	N26515, E54010	1	360	121.0	129.0	10.0	8.0	93
186	8/16	N27535, E53740	1	382	122.7	129.0	14.0	8.0	95
187	8/16	N27500, E53855	1	384	115.5	129.0	10.0	8.0	90
188	8/16	N27470, E53785	1	386	123.1	129.0	15.0	8.0	95
189	8/16	N27570, E53915	1	386	117.6	129.0	9.0	8.0	91

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
190	8/16	N27510, E53785	1	380	116.5	129.0	10.0	8.0	90
191	8/16	N27560, E53790	1	385	118.1	129.0	11.0	8.0	91
192	8/16	N27520, E53770	1	385	116.2	129.0	7.0	8.0	90
193	8/17	N26715, E53280	1	352	119.1	129.0	10.0	8.0	92
194	8/17	N26800, E53300	1	354	119.6	129.0	8.0	8.0	92
195	8/17	N26685, E53180	1	346	118.2	129.0	10.0	8.0	91
196	8/17	N26750, E53085	1	345	119.8	129.0	11.0	8.0	92
197	8/17	N26620, E53325	1	351	116.3	129.0	11.0	8.0	90
198	8/18	N26540, E54023	1	362	118.2	129.0	10.0	8.0	91
199	8/18	N26770, E53830	1	356	117.4	129.0	14.0	8.0	91
200	8/18	N26940, E54285	1	364	117.6	129.0	9.0	8.0	91
201	8/18	N26870, E54250	1	366	119.1	129.0	10.0	8.0	92
202	8/18	N26905, E53970	1	362	116.5	129.0	10.0	8.0	90
203	8/18	N26500, E54000	1	365	120.4	129.0	8.0	8.0	93
204	8/18	N26905, E54104	1	365	115.8	129.0	9.0	8.0	90
205	8/18	N26915, E54275	1	366	117.4	129.0	10.0	8.0	91
206	8/18	N26855, E53485	1	352	116.1	129.0	12.0	8.0	90
207	8/18	N26740, E53420	1	355	117.0	129.0	12.0	8.0	90
208	8/18	N26685, E54170	1	383	116.3	129.0	11.0	8.0	90
209	8/21	N26760, E54010	1	369	116.4	129.0	8.0	8.0	90
210	8/21	N26625, E54090	1	383	117.6	129.0	9.0	8.0	91
211	8/21	N26610, E54070	1	374	118.1	129.0	10.0	8.0	91
212	8/21	N26610, E54070	1	383	118.3	129.0	14.0	8.0	91

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
213	8/21	N26840, E54065	1	366	118.9	129.0	11.0	8.0	92
214	8/21	N26920, E54165	1	368	116.0	129.0	8.0	8.0	90
215	8/21	N26865, E53965	1	367	121.6	129.0	11.0	8.0	94
216	8/21	N26845, E53782	1	357	119.1	129.0	10.0	8.0	92
217	8/21	N26540, E54040	1	370	119.4	129.0	12.6	8.0	93
218	8/21	N26610, E54100	1	383	119.0	129.0	13.0	8.0	92
219	8/22	N26890, E54268	1	369	115.8	129.0	9.0	8.0	90
220	8/22	N26685, E54010	1	362	117.6	129.0	9.0	8.0	91
221	8/22	N26920, E54380	1	378	120.9	129.0	10.0	8.0	93
222	8/22	N26665, E54140	1	384	118.7	129.0	8.0	8.0	92
223	8/22	N26845, E54100	1	363	118.9	129.0	11.0	8.0	92
224	8/22	N27025, E54375	1	378	117.0	129.0	12.0	8.0	90
225	8/23	N26900, E54000	1	364	118.2	129.0	12.0	8.0	91
226	8/23	N26960, E54420	1	380	115.5	129.0	11.0	8.0	90
227	8/23	N26850, E54000	1	360	121.1	129.0	9.0	8.0	93
228	8/23	N26500, E54040	1	371	116.9	129.0	8.0	8.0	90
229	8/23	N26950, E53545	1	349	119.8	129.0	11.0	8.0	92
230	8/24	N26980, E53740	1	359	116.1	129.0	12.0	8.0	90
231	8/24	N26945, E54300	1	370	117.2	129.0	11.0	8.0	90
232	8/24	N27535, E53890	1	384	118.5	129.0	9.0	8.0	91
233	8/25	N27450, E53780	1	387	120.0	129.0	10.0	8.0	93
234	8/25	N27500, E53760	1	386	116.9	129.0	8.0	8.0	90
235	8/25	N27540, E53715	1	388	117.4	129.0	10.0	8.0	91

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
236	8/25	N27465, E53770	1	388	117.2	129.0	11.0	8.0	90
237	8/25	N27500, E53825	1	388	121.6	129.0	12.0	8.0	94
238	8/28	N27490, E53800	1	390	118.1	129.0	11.0	8.0	91
239	8/28	N27495, E53745	1	390	116.3	129.0	11.0	8.0	90
240	8/28	N27620, E53705	1	394	119.3	129.0	9.0	8.0	92
241	8/28	N27545, E53765	1	390	117.3	129.0	10.0	8.0	90
242	8/29	N27480, E53770	1	392	118.2	129.0	10.0	8.0	91
243	8/29	N27590, E53730	1	396	118.9	129.0	11.0	8.0	92
244	8/29	N27530, E53755	1	392	116.6	129.0	9.0	8.0	90
245	8/29	N27555, E53765	1	398	117.3	129.0	10.0	8.0	90
246	8/30	N27635, E53720	1	400	117.8	129.0	12.0	8.0	91
247	8/30	N27230, E52970	1	347	117.8	129.0	13.6	8.0	91
248	8/30	N28065, E53050	1	348	117.6	129.0	9.0	8.0	91
249	8/31	N27020, E52960	1	347	116.9	129.0	8.0	8.0	90
250	8/31	N26820, E53115	1	342	117.6	129.0	9.0	8.0	91
251	8/31	N27050, E52780	1	347	120.0	129.0	10.0	8.0	93
252	8/31	N26890, E52900	1	346	119.1	129.0	10.0	8.0	92
253	9/1	N27180, E52850	1	348	116.5	129.0	10.0	8.0	90
254	9/1	N26880, E53070	1	344	120.0	129.0	10.0	8.0	93
255	9/1	N26850, E53270	1	348	117.2	129.0	11.0	8.0	90
256	9/1	N26940, E53575	1	351	119.5	129.0	8.0	8.0	92
257	9/5	N27290, E53710	1	370	119.5	129.0	8.0	8.0	92
258	9/5	N27230, E52905	1	349	121.1	129.0	9.0	8.0	93

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
259	9/7	N27050, E53060	1	348	117.4	129.0	10.0	8.0	91
260	9/7	N27115, E52770	1	349	118.1	129.0	11.0	8.0	91
261	9/7	N26960, E52875	1	348	118.1	129.0	11.0	8.0	91
262	9/8	N27050, E52940	1	349	117.8	129.0	12.0	8.0	91
263	9/8	N27075, E53725	1	357	116.3	129.0	11.0	8.0	90
264	9/8	N26950, E53640	1	352	117.2	129.0	11.0	8.0	90
265	9/11	N26935, E53145	1	352	115.4	129.0	13.6	8.0	90
266	9/11	N27125, E52990	1	349	116.9	129.0	8.0	8.0	90
267	9/11	N27155, E52790	1	346	116.3	129.0	11.0	8.0	90
268	9/12	N26940, E53040	1	352	118.3	129.0	18.0	8.0	91
269	9/12	N27135, E52870	1	351	117.8	129.0	12.0	8.0	91
270	9/12	N26965, E52970	1	348	122.3	129.0	11.0	8.0	94
271	9/13	N27100, E53050	1	350	117.6	129.0	9.0	8.0	91
272	9/13	N26835, E53175	1	346	119.8	129.0	7.0	8.0	92
273	9/13	N26890, E53030	1	347	117.6	129.0	9.0	8.0	91
274	9/14	N26880, E53545	1	354	118.2	129.0	10.0	8.0	91
275	9/14	N26860, E52955	1	341	119.1	129.0	10.0	8.0	92
276	9/14	N27175, E52910	1	351	119.5	129.0	8.0	8.0	92
277	9/15	N26540, E54040	1	373	119.6	129.0	8.0	8.0	92
278	9/15	N26550, E54020	1	375	120.0	129.0	10.0	8.0	93
279	9/15	N26470, E53965	1	377	122.3	129.0	10.0	8.0	94
280	9/15	N26520, E54000	1	379	118.7	129.0	8.0	8.0	92
281	9/18	N26540, E54050	1	380	120.4	129.0	13.6	8.0	93

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
282	9/18	N26465, E54000	1	382	117.3	129.0	10.0	8.0	90
283	9/18	N26510, E53985	1	384	117.2	129.0	11.0	8.0	90
284	9/19	N26495, E54015	1	386	117.0	129.0	12.0	8.0	90
285	9/19	N26485, E53985	1	386	122.6	129.0	10.0	8.0	95
286	9/19	N26440, E54000	1	388	123.2	129.0	11.0	8.0	95
287	9/20	N26940, E54215	1	370	117.2	129.0	11.0	8.0	90
288	9/20	N26830, E54060	1	365	118.0	129.0	7.0	8.0	91
289	9/20	N26870, E53925	1	366	116.4	129.0	6.0	8.0	90
290	9/21	N26915, E54212	1	372	116.3	129.0	11.0	8.0	90
291	9/21	N26850, E54295	1	372	119.8	129.0	11.0	8.0	92
292	9/21	N26985, E54028	1	367	116.4	129.0	15.0	8.0	90
293	9/22	N26810, E54145	1	369	117.1	129.0	8.0	8.0	91
294	9/22	N26980, E54260	1	375	117.8	129.0	12.0	8.0	91
295	9/22	N26960, E54390	1	382	118.2	129.0	10.0	8.0	91
296	9/22	N26790, E54240	1	378	124.5	129.0	7.0	8.0	96
297	9/25	N26880, E54210	1	374	121.2	129.0	12.0	8.0	93
298	9/25	N26825, E53965	1	368	117.3	129.0	10.0	8.0	90
299	9/25	N26950, E54105	1	370	116.1	129.0	13.6	8.0	90
300	9/26	N26950, E54245	1	380	116.5	129.0	10.0	8.0	90
301	9/26	N26500, E53440	1	350	119.8	129.0	11.0	8.0	92
302	9/26	N26695, E53445	1	359	116.5	129.0	12.5	8.0	90
303	9/26	N26570, E53290	1	351	116.1	129.0	12.0	8.0	90
304	9/26	N26750, E53460	1	358	119.6	129.0	8.0	8.0	92

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

SUMMARY OF FIELD DENSITY TESTS

Project No.: 277408-02 McCASLIN PACIFIC PROPERTIES

Test No.	Test Date	Test Location	Soil Type	Comments and Test Elev.	Dry Dens., pcf		Moisture, %		Relative Compaction
					Field	Maximum	Field	Opt.	
	1978								
305	9/27	N26760, E53285	1	359	118.7	129.0	13.6	8.0	92
306	9/28	N26670, E53370	1	360	118.1	129.0	11.0	8.0	91
307	9/28	N26650, E53175	1	347	118.4	129.0	9.0	8.0	92
308	9/28	N26500, E53255	1	351	121.5	129.0	11.0	8.0	94
309	9/29	N26545, E53370	1	352	117.4	129.0	10.0	8.0	91
310	9/29	N26800, E53440	1	360	121.3	129.0	8.0	8.0	94
311	9/29	N26615, E53385	1	354	118.2	129.0	10.0	8.0	91
312	9/29	N26540, E53350	1	355	122.6	129.0	6.8	8.0	95

Field density tests were performed in accordance with ASTM D1556-64
* Denotes Driven Sampler Method of Testing, ASTM D2937-71
Maximum densities and optimum moistures were determined in accordance with ASTM D1557-70
Form 202A (8/77)

APPENDIX D

LEIGHTON and ASSOCIATES
INCORPORATED

APPENDIX D

GEOTECHNICAL REPORTS REFERENCED

1. Leighton and Associates, Inc., 1977, Geotechnical Review of Grading Plan Tract 33910 and Parcels 1 Through 5, Parcel Map 10094, City of Monterey Park, dated December 12, 1977.

2. ------, 1977, Addendum to Geotechnical Review of Grading Plan, Tract 33910 and Parcels 1 Through 5, Parcel Map 10094, City of Monterey Park, dated December 21, 1977.

3. ------, 1978, Seepage From Cut Slope, Existing Downdrain Outlet, Tract 33910, City of Monterey Park, County of Los Angeles, California, dated August 2, 1978.

4. ------, 1978, Cut Slope North of Proposed Orange Avenue, City of Monterey Park, California, dated August 18, 1978.


SEE SHEET 2
SCALE 1" = 40'
SATURN
STREET
GEMINI
ORANGE
AVENUE
SOUTHERN CALIFORNIA
EDISON
POTRERO GRANDE DRIVE
SECTION H-H
DAYLIGHT FILL
DAYLIGHT LINE
GREEN HOUSE
R/W
PHOTOGRAMMETRY BY DON READ CORP. ANAHEIM, CALIFORNIA
LEIGHTON and ASSOCIATES
AS-GRADED GEOTECHNICAL MAP OF TRACT 33910 AND PARCEL MAP 10094 CITY OF MONTEREY PARK
REV. 7-20-78
DESIGN K.L.W. DATE 11-21 1977
DRAWN C.A. DATE 11-21 1977
CHECKED A.V. DATE 11-21 1977
PREPARED IN THE OFFICE OF LOCKMAN & ASSOCIATES, CIVIL ENGINEERS
GRADING PLAN II
TRACT NO. 33910 & PARCEL MAP 10094
IN THE CITY OF MONTEREY PARK
for McCASLIN PACIFIC PROPERTIES
JOB NO. 77-702
SHEET 3 OF 3
DR 77156

NO.	REVISION	INITIAL	DATE

SURVEY REFERENCE				
HORIZONTAL CONTROL	F.B.	PG.	CLIENT F.B.	PG.
VERTICAL CONTROL	F.B.	PG.	" F.B.	PG.
TOPOGRAPHY	F.B.	PG.	" F.B.	PG.
CROSS SECTIONS	F.B.	PG.	" F.B.	PG.

DESIGN R.L.M. DATE 11-21 1977
DRAWN S.V. DATE 11-21 1977
CHECKED A.V. DATE 11-21 1977

PREPARED IN THE OFFICE OF
LOCKMAN & ASSOCIATES, CIVIL ENGINEERS
BY [signature] DATE 12-28 1977

GRADING PLAN I
TRACT NO. 33910 & PARCEL MAP 10094
IN THE CITY OF MONTEREY PARK
for McCASLIN PACIFIC PROPERTIES

JOB NO. 77-702
SHEET 2 of 3

LEIGHTON and ASSOCIATES

AS GRADED GEOTECHNICAL MAP of TRACT 33910 & PARCEL MAP 10094 CITY OF MONTEREY PARK

Project 277408-02	Scale 1" = 40	Date 15 Nov 1978
Engineer Geologist DEB / DA		Drafted By gjg

LEGEND

Cg	CONGLOMERATE
Ss	SANDSTONE
Slt	SANDSTONE AND SILTSTONE
Qt	TERRACE DEPOSIT
	BEDDING ATTITUDE (DASHED WHERE BURIED)
	POORLY DEVELOPED BEDDING
	CONTACT (DOTTED WHERE BURIED)
	TRACE OF CLAY BED (DOTTED WHERE BURIED)
	SEEP
	APPROXIMATE LIMIT OF FILL PLACED DURING CURRENT GRADING
Af_1	FILL PLACED IN 1961 (DOTTED WHERE BURIED)
Af_2	FILL PLACED IN 1971 (DOTTED WHERE BURIED)
•312	ESTIMATED LOCATION OF DENSITY TEST
	APPROXIMATE LIMIT OF BUTTRESS KEY

APPENDIX B

EXPLORATION LOGS

Appendix B.1 – Exploration Boring Logs (2023)

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GEOTECHNICAL BORING LOG LB-1

Project No.	19850	**Date Drilled**	12-1-23
Project	1977 Saturn Street Data Center Project	**Logged By**	LFO
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	364'
Location	See Figure 2A - Boring Location Map	**Sampled By**	LFO

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0								@Surface: 3.5" asphalt concrete over 9" base	
								SP-SM	**Engineered Fill, Compacted (Afe) - Leighton 1978** @1.04': SAND w/ Silt, reddish brown, predominantly fine sand, trace medium sand, some clay	
360				S-1	3 5 7			SM	@3.0': Silty SAND, reddish brown mottled grayish brown, slightly moist, dense, predominately medium sand, trace coarse sand, some clay, some fine subrounded gravels (% Passing No. 200 Sieve = 23.9%).	-200
	5			R-2	7 20 25	109.5	14.0	SM	@5.0': Silty SAND, reddish brown mottled grayish brown, slightly moist, dense, predominately medium sand, trace coarse sand, some clay, some fine subrounded gravels.	
355				S-3	3 7 10			SC	@7.5': Clayey SAND, light yellowish brown, slightly moist, stiff, low plasticity (Plasticity Index = 14), Fe-stained, predominantly fine sand (% Passing No. 200 Sieve = 35.9%)	-200, AL
	10			R-4	7 15 26	103.9	22.8	CLs	@10': Sandy CLAY, medium brown, hard, slightly moist, fine-grained sand particles (quartz and feldspar), quartz is light gray translucent, feldspar appears as white chips, subangular, quartz grains subangular to subrounded, few small root hairs, no reaction with HCl, poorly developed structure, moderate clay development on parting surfaces (shrink and swell).	
350	15			S-5	4 9 13			CL-ML	@15': Clayey SILT with very fine Sand, light medium brown, hard, poorly developed blocky structure, moderate clay development on parting surfaces (internal shear planes, shrink and swell).	
									Pliocene-Pleistocene Fernando Formation (Tfsc)	
345	20			S-6	7 14 20		19.7		@20': SANDSTONE, yellowish brown, hard, fine sand (% Passing No. 200 Sieve = 44.1%), slightly moist, slightly micaceous, contains Fe-stained veins, degrades to Clayey SAND.	-200
340	25			S-7	15 29 50/6"				@25': Conglomerate SANDSTONE, reddish to orange brown, hard, medium to coarse sand, medium subrounded gravels, slightly moist, micaceous	
335	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-1

Project No.	19850	**Date Drilled**	12-1-23
Project	1977 Saturn Street Data Center Project	**Logged By**	LFO
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	364'
Location	See Figure 2A - Boring Location Map	**Sampled By**	LFO

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			S-8	15 38 46				@30': Conglomerate SANDSTONE, reddish to orange brown, hard, medium to coarse sand, predominantly medium subrounded gravel, some coarse subrounded slate and quartzite-derived gravel, slightly moist, micaceous	
									T.D. 31.5' bgs **No groundwater encountered during drilling.** **Boring backfilled with soil cuttings and patched with cold-mix asphalt.**	
330										
	35									
325										
	40									
320										
	45									
315										
	50									
310										
	55									
305										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-2

Project No.	19850	**Date Drilled**	12-1-23
Project	1977 Saturn Street Data Center Project	**Logged By**	LFO
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	364'
Location	See Figure 2A - Boring Location Map	**Sampled By**	LFO

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								@Surface: 1" asphalt concrete over 5" base	
				BB-1				SCg	**Engineered Fill, Compacted (Afe) - Leighton 1978** @6": Clayey SAND, light yellow brown, predominantly fine sand, trace medium to coarse sand, trace fine gravel, low plasticity, slightly moist @2': Coarse subrounded gravels ranging from 2 to 5" diameter, quartzite-derived	CR, EI, MD, RV
360	5			S-1	3 5 7		11.8	SC	@5': Clayey SAND, brown, very stiff, predominantly medium sand, some coarse sand, some fine to medium subrounded gravel (% Passing No. 200 Sieve = 33.7%), low plasticity (Plasticity Index = 15), slightly moist	-200, AL
				R-2	10 18 21	115.4	10.4	SC	**Engineered Fill, Compacted (Afe) - 1971** @7.5': Clayey SAND, olive, dense, coarse sand, some subrounded fine to medium gravel, moist	CO
355	10			S-3	3 6 6			CL-SP	@10': CLAY w/ Sand, dark gray, very stiff, fine sand, trace fine gravel, low plasticity, moist, Fe-stained	
								CL-ML	**Quaternary Alluvium (Qa)** @11.25': Silty CLAY, dark gray, low plasticity, moist	
350	15			S-4	3 6 9			sCL	@15': Sandy Lean CLAY, reddish brown mottled gray, very stiff, predominantly fine sand, trace medium sand, low plasticity, slightly moist, thinly laminated	
345	20			S-5	2 4 7		22.8	sCL	@20': Sandy Lean CLAY, brown, very stiff, fine sand, some silt (% Passing No. 200 Sieve = 54.1%), low plasticity (Plasticity Index = 11), moist	-200, AL
340	25			S-6	6 8 11				**Pliocene-Pleistocene Fernando Formation (Tfsc)** @25': Silty SANDSTONE, reddish to orange brown, hard, fine sand, low plasticity, moist, thinly laminated, heavily Fe-stained with depth, slightly micaceous	
335	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

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GEOTECHNICAL BORING LOG LB-2

Project No.	19850	**Date Drilled**	12-1-23
Project	1977 Saturn Street Data Center Project	**Logged By**	LFO
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	364'
Location	See Figure 2A - Boring Location Map	**Sampled By**	LFO

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			S-7	16 20 24				@30': Silty SANDSTONE, dark reddish brown, hard, fine sand, slightly moist, Fe-stained, contains fractured white shells	
330										
	35			S-8	7 17 25				@35': Silty SANDSTONE, yellow brown, hard, recovered as Silty SAND, predominantly fine sand, trace coarse sand (% Passing No. 200 Sieve = 16.7%), slightly moist	-200
									T.D. 36.5' bgs **No groundwater encountered during drilling.** **Boring backfilled with soil cuttings and patched with cold-mix asphalt.**	
325										
	40									
320										
	45									
315										
	50									
310										
	55									
305										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-3

Project No.	19850	**Date Drilled**	12-1-23
Project	1977 Saturn Street Data Center Project	**Logged By**	LFO
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	345'
Location	See Figure 2A - Boring Location Map	**Sampled By**	LFO

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
345	0							@Surface: 3" asphalt concrete over 11" base	
							sCL	**Engineered Fill, Compacted (Afe) - Leighton 1978** @14": Sandy CLAY, yellow brown mottled reddish brown, predominantly fine to medium sand, some coarse sand, low plasticity, moist	
							SC	@3': Clayey SAND, yellow brown, predominantly fine to medium sand, some coarse sand, trace fine gravel, low plasticity, moist	
340	5		R-1	9 24 45			sCL	@5': Sandy CLAY, reddish brown, hard, predominantly fine to medium sand, some coarse sand, low plasticity, moist, weakly laminated	
			S-2	4 6 28			SM	@7.4': Silty SAND, yellowish brown, very dense, medium sand (% Passing No. 200 Sieve = 36.4%), low plasticity, slightly moist	-200
								Pliocene-Pleistocene Fernando Formation (Tfsc) @9': Gravelly SANDSTONE, reddish orange to white, medium to coarse sand, fine to medium gravel, slightly moist	
335	10		R-3	36 50/4"	118.6	8.6		@10': Gravelly SANDSTONE, reddish orange to white, hard, medium to coarse sand, fine to medium gravel derived from quartzite, schist, and granite, slightly moist	
330	15		S-4	24 38 40				@15': Silty SANDSTONE with Gravel, yellowish brown, hard, medium to coarse sand (% Passing No. 200 Sieve = 17.8%), some fine to medium gravel derived from quartzite, schist, and granite, slightly moist	-200
								T.D. 16.5' bgs **No groundwater encountered during drilling.** **Boring backfilled with soil cuttings and patched with cold-mix asphalt.**	
325	20								
320	25								
315	30								

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-4

Project No.	19850	**Date Drilled**	12-1-23
Project	1977 Saturn Street Data Center Project	**Logged By**	LFO
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	345'
Location	See Figure 2A - Boring Location Map	**Sampled By**	LFO

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
345	0								@Surface: 6.5" asphalt concrete over 5" base	
				BB-1				SC	**Engineered Fill, Compacted (Afe) - Leighton 1978** @11.5": Clayey SAND, yellowish brown, predominantly medium sand, some coarse sand, trace coarse subrounded gravel (Sieve Analysis: Gravel = 8%, Sand = 61%, Fines = 31%), low plasticity, slightly moist	SA
									Pliocene-Pleistocene Fernando Formation (Tfsc) @3': Conglomerate SANDSTONE, reddish brown, predominantly medium to coarse sand, medium subrounded gravel, some gray clay blebs, slightly moist	
340	5			S-1	37 37 38				@5': hard	
				R-2	29 50/4"	105.9	7.7		@7.5': hard	
335	10			S-3	31 36 33				@10': Conglomerate SANDSTONE, reddish brown, very dense, predominantly medium to coarse sand, medium subrounded gravel derived from quartzite and granite, some gray clay blebs, slightly moist	
									T.D. 11.5' bgs **No groundwater encountered during drilling.** **Boring backfilled with soil cuttings and patched with cold-mix asphalt.**	
330	15									
325	20									
320	25									
315	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

GEOTECHNICAL BORING LOG LB-5

Project No.	19850	**Date Drilled**	12-1-23
Project	1977 Saturn Street Data Center Project	**Logged By**	LFO
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	362'
Location	See Figure 2A - Boring Location Map	**Sampled By**	LFO

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0								@Surface: 2.5" asphalt concrete over 11" base	
360				BB-1					**Pliocene-Pleistocene Fernando Formation (Tfsc)** @13": SANDSTONE, yellow brown, predominantly fine sand, trace medium to coarse sand, trace fine to coarse subrounded gravel, some clay, moist	
	5			R-1	41 50/3"	111.2	9.1		@5': Gravelly SANDSTONE, yellow brown, hard, medium to coarse sand, fine subrounded gravel, moist, micaceous	
355				S-2	18 50/6"				@7.5': Gravelly SANDSTONE, yellow brown, hard, medium to coarse sand, predominantly fine subrounded gravel, some medium subrounded gravel derived from biotite schist and quartzite, moist, micaceous @8.5': Fe-staining in sandstone matrix	
	10			R-3	33 50/4.5"	112.6	11.9		@10': Gravelly SANDSTONE, yellow brown, hard, medium to coarse sand, predominantly fine subrounded gravel, some medium subrounded gravel derived from biotite schist and quartzite, moist, micaceous, interbedded with 1-2" thick orange brown shale	
350									**T.D. 10.88' bgs** **No groundwater encountered during drilling.** **Boring backfilled with soil cuttings and patched with cold-mix asphalt.**	
	15									
345										
	20									
340										
	25									
335										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

Appendix B.2 – Exploration Log (2015 Offsite)

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GEOTECHNICAL BORING LOG LB-1

Project No.	11173.001	**Date Drilled**	10-23-15
Project	Swinerton Builders - Monterey Park	**Logged By**	ARR
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	359'
Location	See Figure 2, Exploration Location Map	**Sampled By**	ARR

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet: 355, 350, 345, 340, 335, 330
Depth Feet: 0, 5, 10, 15, 20, 25, 30

Graphic Log: N S

Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
BB-1				SC	**Artificial Fill, compacted (Afc)** @Surface: Grass sod over Clayey SAND with Gravel (SC), dry to slightly moist, fine sand, very weathered angular granitic gravel, trace clay, roots	MD, AL, SA, CR
					@3': becomes light yellowish to reddish brown, fine sand, fine and coarse gravel	
R-1	9 11 11	119.6	12.3	SC-CL	@5': Clayey SAND (SC) to Sandy CLAY (CL), mottled brown, gray and reddish brown, moist, medium dense, predominantly fine sand with few medium to coarse sand	DS
S-1	5 6 9			SC	@7.5': Clayey SAND with Gravel (SC), mottled reddish to olive brown, moist, medium dense, fine to medium sand, fine very weathered gravel	
R-2	8 9 13	119.4	12.9	SC-CL	@10': Clayey SAND (SC) to Sandy CLAY (CL), mottled light brown to orange brown, moist, medium dense, fine to medium sand, trace coarse sand, trace gravel	DS
S-2	3 2 2			CL	@12.5': Silty sandy CLAY (CL), dark olive brown, very moist, soft, trace crushed very weathered gravel, gray fine sand lenses	
R-3	8 15 16	121.7	11.3		**Fernando Formation (Tfsc)** @15': Clayey Gravelly SANDSTONE, grayish brown, moist, dense, predominantly fine to medium sand, some coarse sand, very weathered, granitic, basalt and quartz derived gravel	DS
S-3	10 20 45				@17.5': Gravelly SANDSTONE, light reddish brown, slightly moist to moist, very dense, fine to coarse sand, fine very weathered gravel, trace clay	
R-4	26 36 50/6"				@20': same as above	
S-4	25 40 50/6"				@22.5': same as above	
R-5	35 50/6"				@25': Conglomerate SANDSTONE, mottled olive brown to reddish brown, moist, very dense, fine to coarse sand, fine and coarse gravel, trace clay in matrix	
S-5	15 35 37				@27.5': Gravelly SANDSTONE, light reddish brown, moist, very dense, fine to coarse sand, fine gravel, trace clay	

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER	UU	UNCONSOLIDATED UNDRAINED TRIAXIAL
CU	UNDRAINED TRIAXIAL	RV	R VALUE	SW	SWELL OR SETTLEMENT

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-1

Project No.	11173.001	**Date Drilled**	10-23-15
Project	Swinerton Builders - Monterey Park	**Logged By**	ARR
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	359'
Location	See Figure 2, Exploration Location Map	**Sampled By**	ARR

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-6	50/6"				@30': Conglomerate SANDSTONE, reddish brown, moist, very dense, fine to coarse sand, fine and coarse gravel, trace clay in matrix	
				S-6	24 31 33				@32.5': Gravelly SANDSTONE, light reddish brown, moist, very dense, fine to coarse sand, fine gravel, trace clay	
325										
	35			R-7	35 50/2"				@35': Clayey SANDSTONE with Gravel, reddish brown, very moist, predominantly medium sand, few fine and coarse sand, trace coarse angular gravel	
320	40								**Notes:** Total Depth: 35.7 feet bgs No groundwater encountered during drilling Boring backfilled with soil cuttings and capped with sod	
315	45									
310	50									
305	55									
300	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH
- UU UNCONSOLIDATED UNDRAINED TRIAXIAL
- SW SWELL OR SETTLEMENT

Appendix B.3 – Test Pits (1977)

APPENDIX A

GEOTECHNICAL TRENCH LOGS

TRENCH 1 - Parcel 1

0 to 4 feet	-	FILL, SILTY SAND WITH GRAVEL: Damp, light brown to tan, pieces of gray sandstone, dense. (SM)
4 to 12.5 feet	-	ALLUVIUM, CLAYEY SILTY SAND WITH GRAVEL: Dry to damp, dark brown, highly porous with root hairs, moist below 7 feet--still porous, loose to medium-dense with depth. (SM)
12.5 to 14 feet	-	BEDROCK, SANDSTONE: Damp to moist, rusty brown, dense. (SM)

TRENCH 2 - Parcel 3

0 to 7 feet	-	BEDROCK, SANDSTONE: Damp, yellow/orange, 5-inch pebble/cobble conglomerate interbedded, very dense, bedding: N85°W, 40°SW. (SW)

TRENCH 3 - Lot 1

0 to 1.5 feet	-	FILL, CLAYEY SILTY SAND WITH GRAVEL: Dry, brown, very loose. SC
1.5 inches to 3.5 feet	-	BEDROCK; CLAYEY SAND AND GRAVEL WITH COBBLE TO 6 INCHES: Moist, rusty brown, very dense. SW-SC

TRENCH 4 - Lot 2

0 to 2 feet	-	ALLUVIUM, CLAYEY SILTY SAND WITH GRAVEL: Dry to damp, dark brown, loose. (SM)
2 to 3.5 feet	-	BEDROCK, CLAYEY SAND AND GRAVEL WITH COBBLE TO 6 INCHES: Damp to moist, rusty brown, dense. (SC)

TRENCH 5 - Lot 2

0 to 4.5 feet	-	ALLUVIUM, CLAYEY SILTY SAND WITH GRAVEL: Dry, dark brown, highly porous with root hairs, loose, damp and very dense from 2 feet to 4.5 feet. (SM)
4.5 to 5.5 feet	-	ALLUVIUM, COARSE SAND AND GRAVEL: Damp, tan, dense. (SP)

APPENDIX A

GEOTECHNICAL TRENCH LOGS (CONTINUED)

TRENCH 5 (Continued)

5.5 to 6 feet 4 inches	- ALLUVIUM, CLAYEY SILTY FINE SAND: Moist, light brown, dense. (SC)
6 feet 4 inches to 7 feet 4 inches	- ALLUVIUM, COARSE SAND AND GRAVEL: Moist, light brown, dense. (SP)
7 feet 4 inches to 7 feet 8 inches	- BEDROCK, SANDSTONE: Medium brown, medium dense. (SW)

TRENCH 6 - Lot 7

0 to 2 feet	- BEDROCK CLAYEY SAND AND GRAVEL WITH COBBLE TO 6 INCHES: Moist, rusty brown, dense to very dense. (SC)

TRENCH 7 - Lot 5

0 to 2 feet	- BEDROCK, CLAYEY SAND AND GRAVEL WITH COBBLE TO 6 INCHES: Moist, rusty brown, dense to very dense. (SC)

TRENCH 8 - Lot 8

0 to 5.5 feet	- FILL, SANDY SILTY CLAY: Dry to damp, medium brown, pieces of wood, brick, asphaltic concrete, soft. (CL)
5.5 to 7 feet	- BEDROCK, SANDSTONE: Damp, rusty brown, weathered, dense. (SP-SC)

TRENCH 9 - Lot 11

0 to .5 feet	- FILL, CLAYEY SILTY SAND WITH GRAVEL: Dry, dark brown, loose. (SM)
.5 to 3 feet	- BEDROCK, CLAYEY SAND AND GRAVEL WITH COBBLE TO 6 INCHES: Moist, rusty brown, dense to very dense. (SC)

APPENDIX A

GEOTECHNICAL TRENCH LOGS (CONTINUED)

TRENCH 10 - Lot 10

0 to .5 feet - FILL, CLAYEY SAND AND GRAVEL WITH COBBLE TO 4 INCHES: Dry to damp, rusty brown, pieces of concrete, medium dense. (SC)

.5 to 6.75 feet - ALLUVIUM, CLAYEY SAND WITH GRAVEL: Damp, medium brown, porous, becomes moist with depth, medium dense. (SC)

6.75 to 8 feet - ALLUVIUM, SANDY CLAY: Moist, medium brown, stiff. (CL)

8 to 10 feet - BEDROCK, SANDSTONE: Moist, rusty brown, dense. (SM)

TRENCH 11 - Lot 10

0 to 6.5 feet - FILL, SILTY FINE SAND WITH TRACES OF GRAVEL: Moist, tan, few 6± inch lifts of dark brown clayey sand, medium dense. SM

6.5 to 9 feet - ALLUVIUM, CLAYEY SAND WITH GRAVEL: Moist, medium brown, porous, medium dense. (SC)

9 to 10 feet 8 inches - BEDROCK, SANDSTONE: Moist, rusty brown, dense. (SM)

TRENCH 12 - Lot 10

0 to 2 feet - BEDROCK, SANDSTONE: Damp, rusty brown, dense. (SM)

2 to 2.5 feet - BEDROCK, SANDSTONE: Damp, gray with rust specks. (SW)

TRENCH 13 - Lot 11

0 to 17.5 feet - FILL, CLAYEY SAND WITH GRAVEL, SANDY SILTY CLAY, SILTY SAND ALTERNATING: Dry to damp, mottled browns, piece of brick at 4 feet, dry 8± inch lift at 3 feet 8 inches, moist below 4 feet 6 inches, stiff and dense. SC

TRENCH 14 - Lot 13

0 to 12 feet - FILL, CLAYEY SAND WITH GRAVEL, SANDY SILTY CLAY, SILTY SAND ALTERNATING: Dry to damp, mottled browns, loose from 4 to 5 feet, pieces of concrete and asphaltic concrete at 7 feet. (SC)

APPENDIX A

GEOTECHNICAL TRENCH LOGS (CONTINUED)

TRENCH 14 (Continued)

12 to 13.5 feet	- BEDROCK, CLAYEY SAND AND GRAVEL WITH COBBLE TO 6 INCHES: Moist, rusty brown, root hairs, dense to very dense. (SC)

TRENCH 15 - Lot 12

0 to 4.5 feet	- FILL, CLAYEY SAND WITH GRAVEL, SANDY SILTY CLAY, SILTY SAND ALTERNATING: Dry to damp, mottled browns, stiff and dense. (SC)
4.5 to 6.5 feet	- ALLUVIUM, CLAYEY SILTY SAND WITH GRAVEL: Dry-damp, dark brown, highly porous with root hairs, medium dense. (SM)
6.5 to 8 feet	- BEDROCK, SANDSTONE: Moist, rusty brown, porous with root hairs, dense. (SM)

TRENCH 16 - Lot 12

0 to 4 feet	- ALLUVIUM, CLAYEY SILTY SAND WITH GRAVEL: Dry, dark brown, highly porous with root hairs, medium dense. (SM)
4 to 9 feet	- ALLUVIUM, SANDY SILTY CLAY WITH GRAVEL: Moist, medium brown, highly porous, stiff. (CL)
9 to 10 feet	- BEDROCK, SANDSTONE: Very moist, rusty brown, dense. (SM)

TRENCH 17 - Lot 3

0 to 16 inches	- ALLUVIUM, CLAYEY SILTY SAND WITH GRAVEL: Dry, dark brown, highly porous with root hairs, dense. (SM)
16 inches to 2.5 feet	- ALLUVIUM, CLAYEY SILTY SAND WITH GRAVEL: Damp, medium brown, very dense. (SC)
2.5 to 3 feet	- BEDROCK, SANDSTONE: Damp, rusty brown, slightly porous with root hairs, dense. (SM)

APPENDIX C

LABORATORY TEST RESULTS

APPENDIX C - GEOTECHNICAL LABORATORY TESTING

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of physical and mechanical properties of soils underlying this campus at proposed improvements, and to aid in verifying soil classification. This geotechnical testing was performed at our Irvine laboratory (DSA LEA 63).

Percentage Passing Sieve No. 200 Sieve (-200): Selected soil samples were wet-wash sieved through a No. 200 U.S. Standard brass sieve in accordance with ASTM Test Methods D 1140 to measure percent fines (silts and clays). This data was used to refine the Unified Soil Classification for tested soil samples. Test results are presented in this appendix and on logs in Appendix B.

Modified Proctor Compaction Curve (MD): Laboratory modified Proctor compaction curves (ASTM D 1557) were established for bulk soil-samples to determine sample-specific modified Proctor laboratory maximum dry density and optimum moisture content. Results of these tests are presented on the following "*Modified Proctor Compaction Test*" sheets in this appendix.

Sieve Analysis (SA): Selected composite soil samples were tested for grain size distribution in accordance with ASTM Test Methods D 6913 to measure grain size. This data was used to refine the Unified Soil Classification for tested soil samples. Test results are presented in this appendix and on logs in Appendix B.

Atterberg Limit Tests: Atterberg Limit tests were performed, in general accordance with ASTM Test Method D 4318, selected soils containing clay material to determine soil compressibility. Following the Dry Preparation Procedure, the soil was dried in an oven at or less than 60° C, pulverized in a mortar and rubber-tipped pestle, and repeatedly sieved using a No. 40 (425-μm) sieve until material retained in the sieve consists of individual grains. The soil that passed through the No. 40 sieve was then placed in a cup and patted with a spatula to a thickness of ~10 mm. A groove was then made in the soil pat from the center to the rim of the of cup and placed in a liquid limit device, where the cup was lifted and dropped until the groove closes (between 25 and 35 drops). The number of drops and moisture content of the soil were then used to determine the soil's liquid limit and plasticity index. Test results are presented in this appendix and on logs in Appendix B.

Swell Potential or Settlement (CO): Selected composite soil samples were tested for wetting-induced swell/collapse strains to develop estimates of heave or settlement of a confined soil profile in accordance with ASTM D 4546. The load-induced strains after wetting are used to estimate stress-induced settlement following wetting-induced heave or settlement. Test results are presented in this appendix and on logs in Appendix B.

Corrosivity Tests: To evaluate corrosion potential of subsurface soils at the site, we tested a bulk sample collected during our subsurface exploration for pH, electrical resistivity (CTM 643), soluble sulfate content (CTM 417 Part II) and soluble chloride content (CTM 422) testing. Results of these tests are enclosed at the end of this appendix.

R-Value Tests (RV): Selected samples were tested in accordance with DOT CA Test 301. The R-Value test measures the response of a compacted sample of soil or aggregate to a vertically applied pressure under specific conditions. This test is used by Caltrans for pavement design, replacing the California bearing ratio test. The R-value of a material is determined when the material is in a state of saturation such that water will be exuded from the compacted test specimen when a 16.8 kN load (2.07 MPa) is applied to test a series of specimens prepared at different moisture contents. R-Value is used in pavement design, with the thickness of each layer dependent on the R-value of the layer below and the expected level of traffic loading, expressed as a Traffic Index. Results of these tests are enclosed at the end of this appendix.

Expansion Tests (EI): In accordance with ASTM D 4829 the specimen is compacted into a metal ring so that the degree of saturation is between 40 and 60 % and the specimen and the ring are placed in a consolidometer. A vertical confining pressure of 1 psi is applied to the specimen and then the specimen is inundated with distilled water. The deformation of the specimen is recorded for 24 hours or until the rate of deformation becomes less than 0.005 mm/hour. The Expansion Index, EI, is used to measure a basic index property of soil and therefore, the EI is comparable to other indices such as the liquid limit, plastic limit, and plasticity index of soils. Results of these tests are enclosed at the end of this appendix.

Appendix C.1 – Lab Results (current)

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: 1977 Saturn Street — Tested By : GEB/JD — Date: 12/13/23

Project No. : 19850 — Checked By: J. Ward — Date: 12/29/23

Boring No.	LB-2			
Sample No.	BB-1			
Sample Depth (ft)	0-5			
Soil Identification:	Light yellowish brown (SC)g			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.17			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	402			
Crucible No.	4			
Furnace Temperature (°C)	860			
Time In / Time Out	9:00/9:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	21.6373			
Wt. of Crucible (g)	21.6340			
Wt. of Residue (g) (A)	0.0033			
PPM of Sulfate (A) x 41150	135.79			
PPM of Sulfate, Dry Weight Basis	**136**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.7			
PPM of Chloride (C -0.2) * 100 * 30 / B	50			
PPM of Chloride, Dry Wt. Basis	**50**			

pH TEST, DOT California Test 643

pH Value	**7.81**			
Temperature °C	22.9			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	1977 Saturn Street	Tested By :	G. Berdy Date: 12/13/23
Project No. :	19850	Checked By:	J. Ward Date: 12/29/23
Boring No.:	LB-2	Depth (ft.) :	0-5
Sample No. :	BB-1		

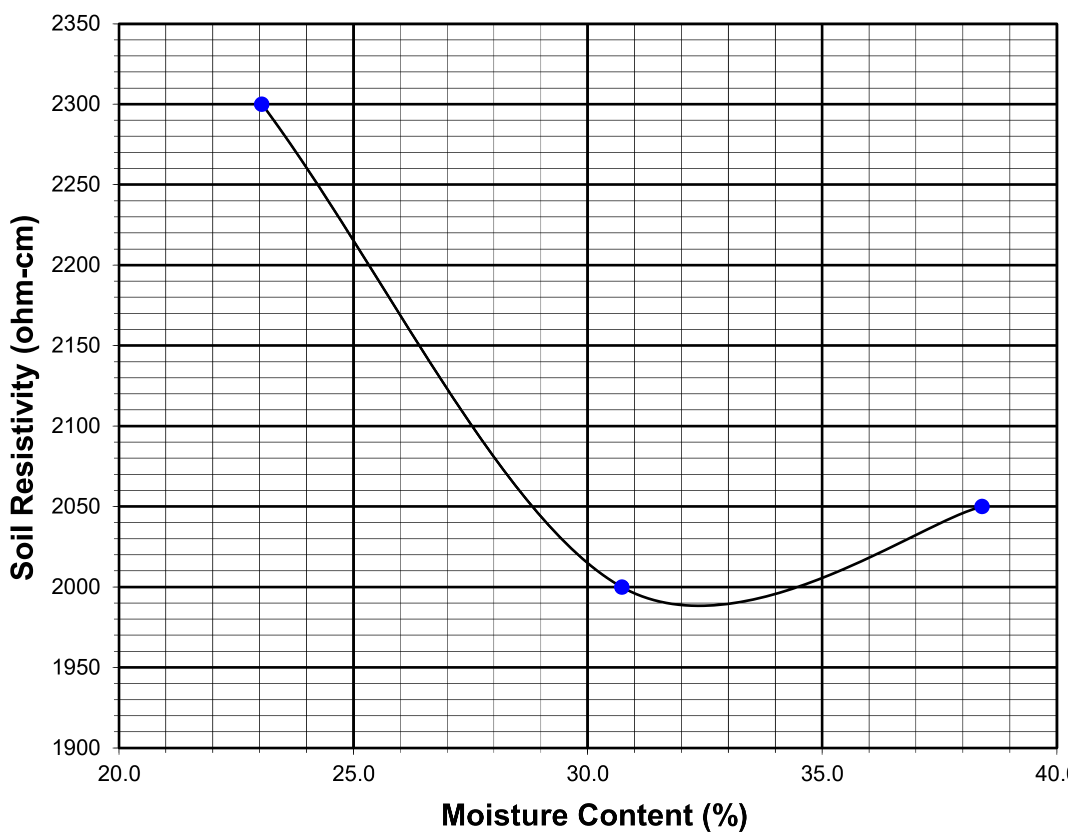
Soil Identification:* Light yellowish brown (SC)g

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	23.05	2300	2300
2	40	30.73	2000	2000
3	50	38.41	2050	2050
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.18
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1990	32.4	136	50	7.81	22.9



EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	1977 Saturn Street	Tested By:	GEB/ACS	Date:	12/13/23
Project No.:	19850	Checked By:	J. Ward	Date:	12/29/23
Boring No.:	LB-2	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Light yellowish brown clayey sand with gravel (SC)g				

Dry Wt. of Soil + Cont. (g)	1000.00
Wt. of Container No. (g)	0.00
Dry Wt. of Soil (g)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0175
Wt. Comp. Soil + Mold (g)	619.50	444.00
Wt. of Mold (g)	204.30	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	824.10	648.30
Dry Wt. of Soil + Cont. (g)	757.40	585.90
Wt. of Container (g)	0.00	204.30
Moisture Content (%)	8.81	16.35
Wet Density (pcf)	125.2	131.6
Dry Density (pcf)	115.1	113.1
Void Ratio	0.465	0.490
Total Porosity	0.317	0.329
Pore Volume (cc)	65.7	69.3
Degree of Saturation (%) [S_{meas}]	**51.2**	90.1

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

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
12/13/23	10:26	1.0	0	0.4475
12/13/23	10:36	1.0	10	0.4465
Add Distilled Water to the Specimen				
12/13/23	11:00	1.0	24	0.4580
12/14/23	8:22	1.0	1306	0.4650
12/14/23	10:00	1.0	1404	0.4650

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**19**

MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Leighton

Project Name:	1977 Saturn Street	Tested By:	R. Cast	Date:	12/13/23
Project No.:	19850	Input By:	J. Ward	Date:	12/29/23
Boring No.:	LB-2	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Light yellowish brown clayey sand with gravel (SC)g				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

Preparation Method:	X	Moist	Scalp Fraction (%)	
		Dry	#3/4	
Compaction Method	X	Mechanical Ram	#3/8	11.4
		Manual Ram	#4	

Rammer Weight (lb.) = 10.0
Height of Drop (in.) = 18.0
Mold Volume (ft³) 0.03320

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3698	3806	3901	3883		
Weight of Mold (g)	1808	1808	1808	1808		
Net Weight of Soil (g)	1890	1998	2093	2075		
Wet Weight of Soil + Cont. (g)	713.4	843.5	803.7	1148.4		
Dry Weight of Soil + Cont. (g)	686.4	793.9	741.1	1036.5		
Weight of Container (g)	74.5	88.0	77.3	88.9		
Moisture Content (%)	4.41	7.03	9.43	11.81		
Wet Density (pcf)	125.5	132.7	139.0	137.8		
Dry Density (pcf)	120.2	124.0	127.0	123.2		

Maximum Dry Density (pcf) 127.0 **Optimum Moisture Content (%)** 9.5

Corrected Dry Density (pcf) 130.7 **Corrected Moisture Content (%)** 8.5

☐ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☒ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

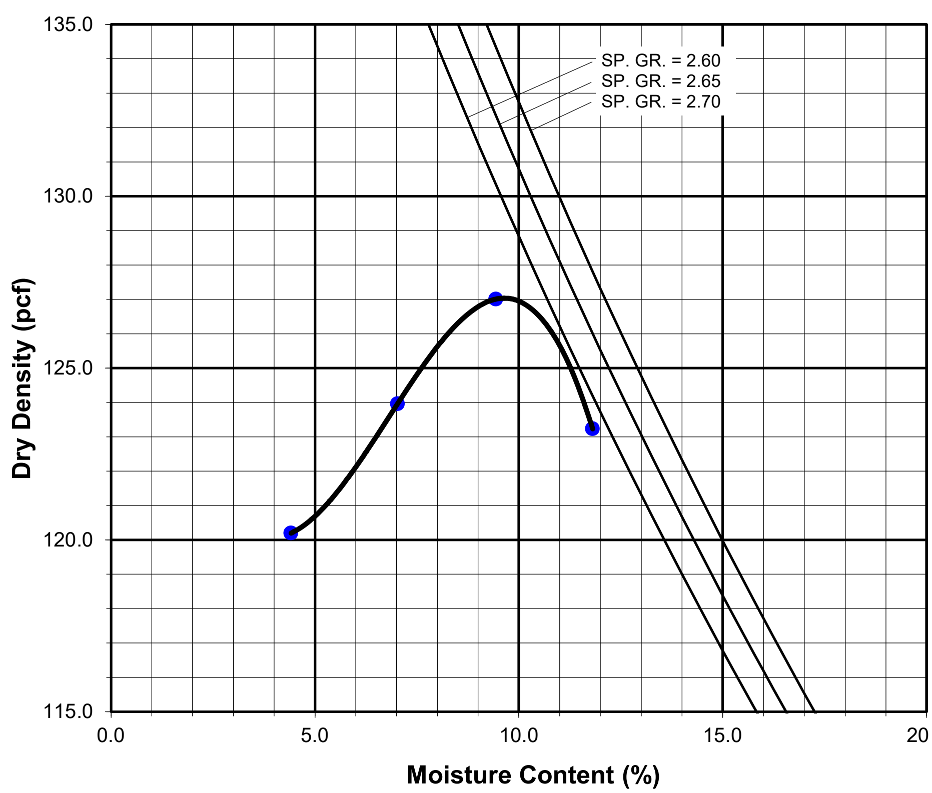
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Boring No.	LB-1	LB-1	LB-1	LB-2	LB-2	LB-3	LB-3	
Sample No.	S-1	S-3	S-6	S-5	S-8	S-2	S-4	
Depth (ft.)	3.0	7.5	20.0	20.0	35.0	7.5	15.0	
Sample Type	SPT	SPT	SPT	SPT	SPT	SPT	SPT	
Soil Identification	Reddish brown silty sand (SM)	Light yellowish brown clayey sand (SC)	Yellowish brown clayey sand (SC)	Brown sandy lean clay s(CL)	Yellowish brown silty sand (SM)	Yellowish brown silty sand (SM)	Yellowish brown silty sand (SM)	
Moisture Correction								
Wet Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Dry Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Weight of Container (g)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
Moisture Content (%)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Sample Dry Weight Determination								
Weight of Sample + Container (g)	809.7	581.9	1040.8	690.8	845.2	748.8	849.8	
Weight of Container (g)	220.2	108.0	300.2	109.9	236.6	82.6	72.7	
Weight of Dry Sample (g)	589.5	473.9	740.6	580.9	608.6	666.2	777.1	
Container No.:								
After Wash								
Method (A or B)	A	A	A	A	A	A	A	
Dry Weight of Sample + Cont. (g)	669.0	411.8	714.0	376.4	743.3	506.1	711.5	
Weight of Container (g)	220.2	108.0	300.2	109.9	236.6	82.6	72.7	
Dry Weight of Sample (g)	448.8	303.8	413.8	266.5	506.7	423.5	638.8	
% Passing No. 200 Sieve	**23.9**	**35.9**	**44.1**	**54.1**	**16.7**	**36.4**	**17.8**	
% Retained No. 200 Sieve	76.1	64.1	55.9	45.9	83.3	63.6	82.2	

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: 1977 Saturn Street
Project No.: 19850
Tested By: K. Jumig
Date: 12/12/23

Passing 200 LB-1, LB-2, LB-3 (not split)

Boring No.	LB-2								
Sample No.	S-1								
Depth (ft.)	5.0								
Sample Type	SPT								
Soil Identification	Brown clayey sand (SC)								
No Moisture Correction; ASTM D 1140 modified to include splitting the sample on the #4 sieve									
Total Sample Dry Weight Determination									
Dry Weight of Soil + Container (g)	745.9								
Weight of Container (g)	109.1								
Dry Weight of Soil (g)	636.8								
Sample Dry Weight Determination, Retained on Sieve #4									
Dry Weight of Sample + Cont. (g)	131.9								
Weight of Container (g)	77.3								
Weight of Dry Sample (g)	54.6								
Sample Dry Weight Determination, Passing Sieve #4									
Dry Weight of Sample + Cont. (g)	497.8								
Weight of Container (g)	109.1								
Weight of Dry Sample (g)	388.7								
After Wash									
Method (A or B)	A								
Dry Weight of Sample + Cont. (g)	354.5								
Weight of Container (g)	109.1								
Weight of Dry Sample (g)	245.4								
% Passing No. 4 Sieve	91.4								
% Retained No. 4 Sieve	8.6								
% Passing No. 200 Sieve	**33.7**								

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: 1977 Saturn Street
Project No.: 19850
Tested By: K. Jumig Date: 12/12/23

ATTERBERG LIMITS
ASTM D 4318

Project Name:	1977 Saturn Street	Tested By:	J. Domingo	Date:	12/14/23
Project No. :	19850	Input By:	J. Domingo	Date:	12/15/23
Boring No.:	LB-1	Checked By:	J. Ward		
Sample No.:	S-3	Depth (ft.)	7.5		
Soil Identification:	Light yellowish brown clayey sand (SC)				

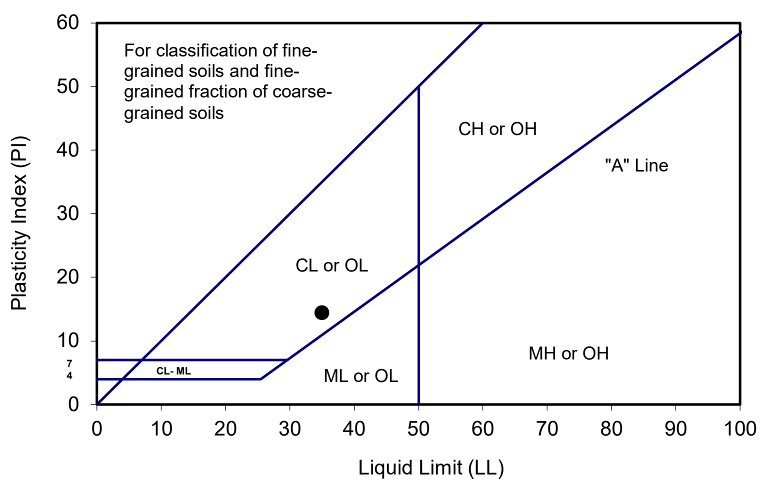
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	24	18	
Wet Wt. of Soil + Cont. (g)	9.80	9.74	23.28	23.60	23.10	
Dry Wt. of Soil + Cont. (g)	8.31	8.27	17.62	17.73	17.14	
Wt. of Container (g)	1.12	1.08	1.02	1.07	1.05	
Moisture Content (%) [Wn]	20.72	20.45	34.10	35.23	37.04	

Liquid Limit	**35**
Plastic Limit	**21**
Plasticity Index	**14**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 10.95

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



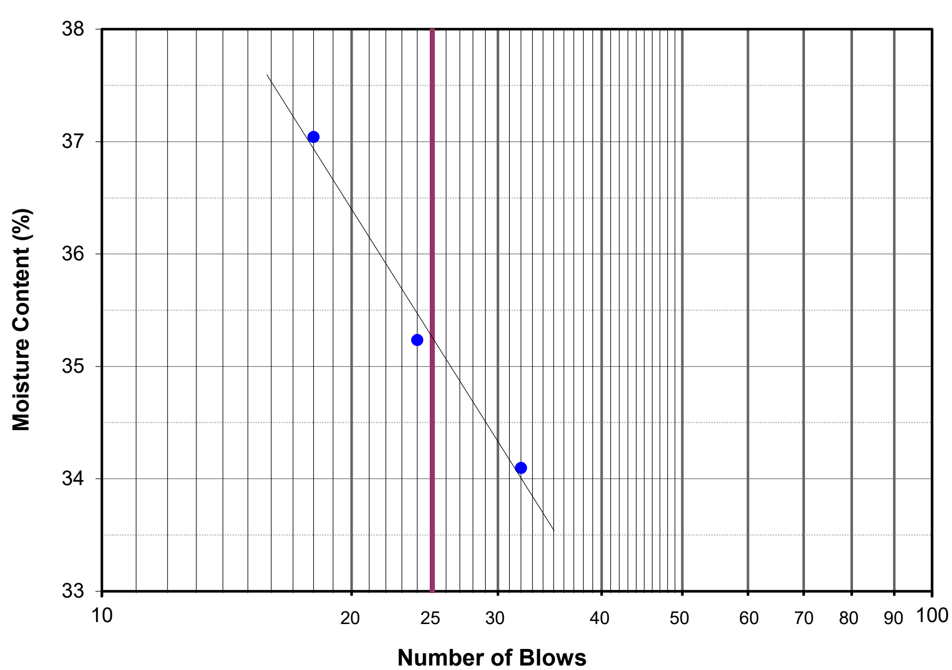
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



ATTERBERG LIMITS
ASTM D 4318

Project Name:	1977 Saturn Street	Tested By:	J. Domingo	Date:	12/14/23
Project No. :	19850	Input By:	J. Domingo	Date:	12/15/23
Boring No.:	LB-2	Checked By:	J. Ward		
Sample No.:	S-1	Depth (ft.)	5.0		
Soil Identification:	Brown clayey sand (SC)				

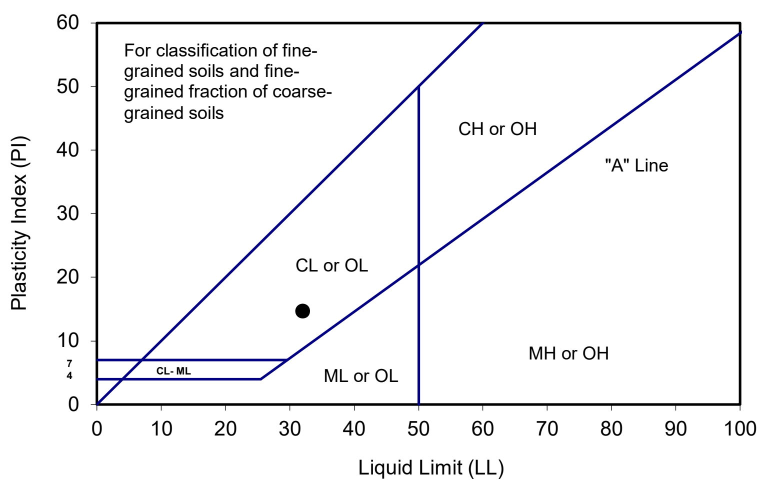
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			35	26	17	
Wet Wt. of Soil + Cont. (g)	9.46	9.39	23.40	23.04	23.14	
Dry Wt. of Soil + Cont. (g)	8.21	8.17	18.12	17.71	17.57	
Wt. of Container (g)	1.07	1.07	1.05	1.02	1.01	
Moisture Content (%) [Wn]	17.51	17.18	30.93	31.94	33.64	

Liquid Limit	**32**
Plastic Limit	**17**
Plasticity Index	**15**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 8.76

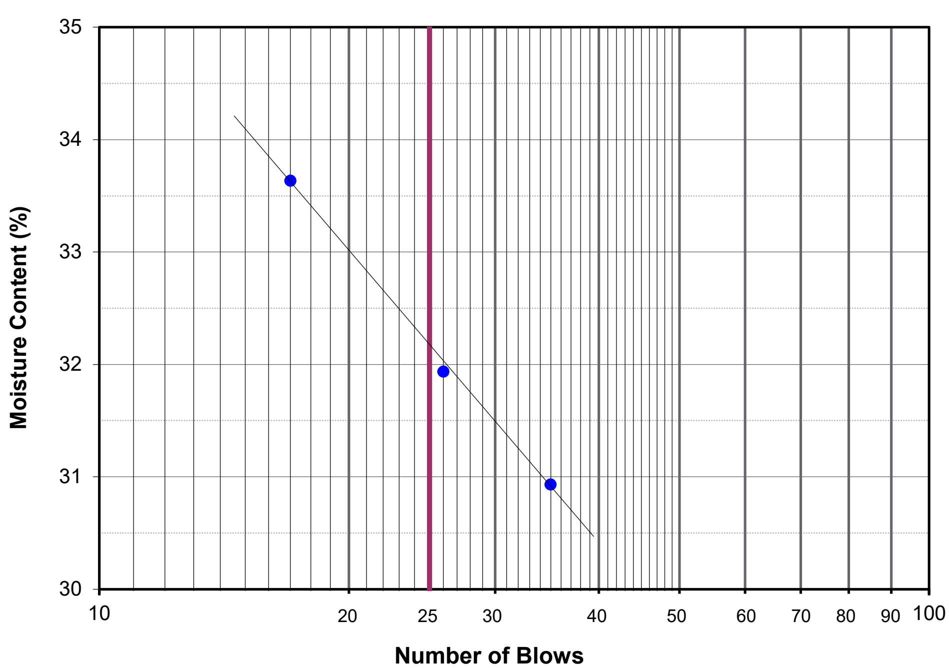
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation Multipoint - Wet
- [X] Dry Preparation Multipoint - Dry
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Leighton

ATTERBERG LIMITS

ASTM D 4318

Project Name:	1977 Saturn Street	Tested By:	J. Domingo	Date:	12/15/23
Project No. :	19850	Input By:	J. Domingo	Date:	12/18/23
Boring No.:	LB-2	Checked By:	J. Ward		
Sample No.:	S-5	Depth (ft.)	20.0		
Soil Identification:	Brown sandy lean clay s(CL)				

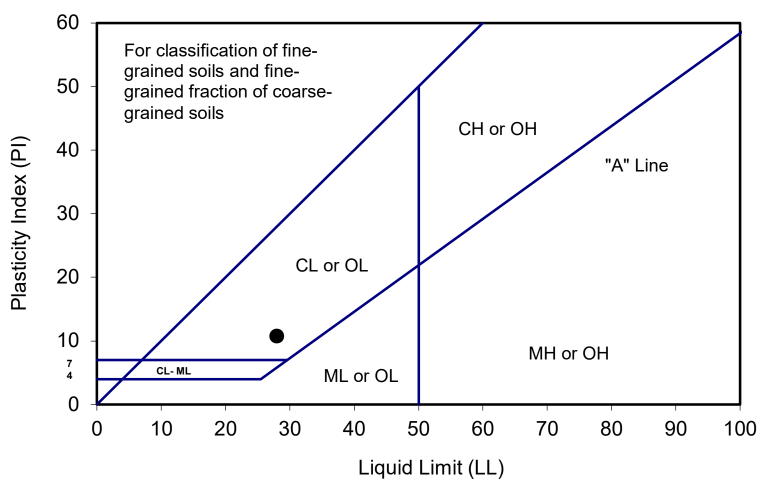
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			33	25	16	
Wet Wt. of Soil + Cont. (g)	9.70	9.72	23.55	23.43	23.12	
Dry Wt. of Soil + Cont. (g)	8.42	8.45	18.85	18.54	18.07	
Wt. of Container (g)	0.99	1.09	1.09	1.02	1.03	
Moisture Content (%) [Wn]	17.23	17.26	26.46	27.91	29.64	

Liquid Limit	**28**
Plastic Limit	**17**
Plasticity Index	**11**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 5.84

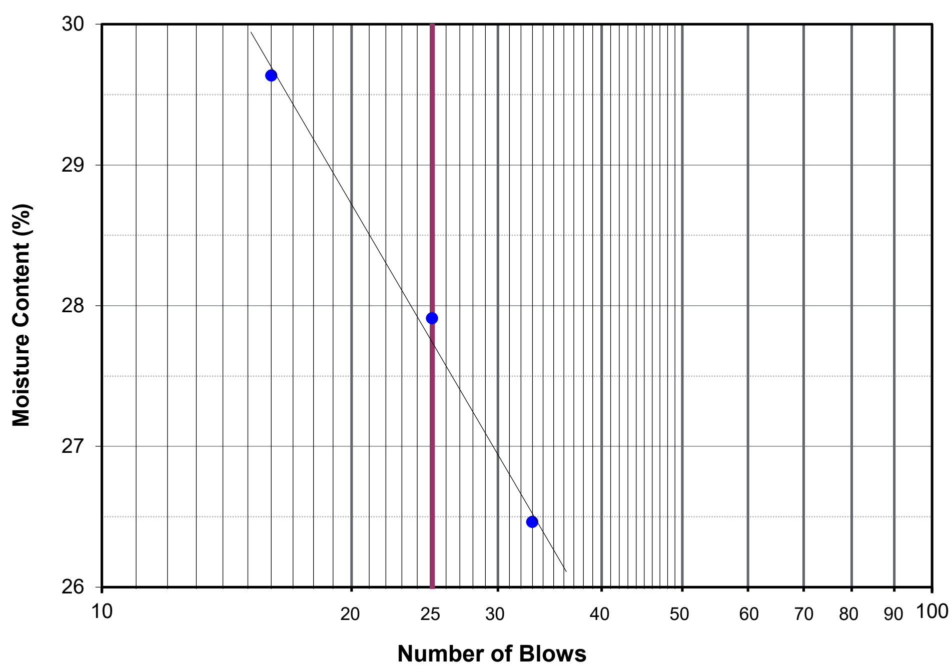
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test

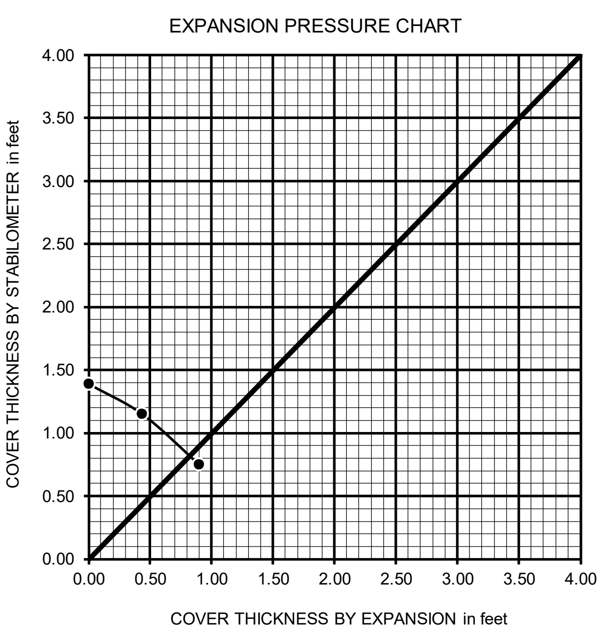


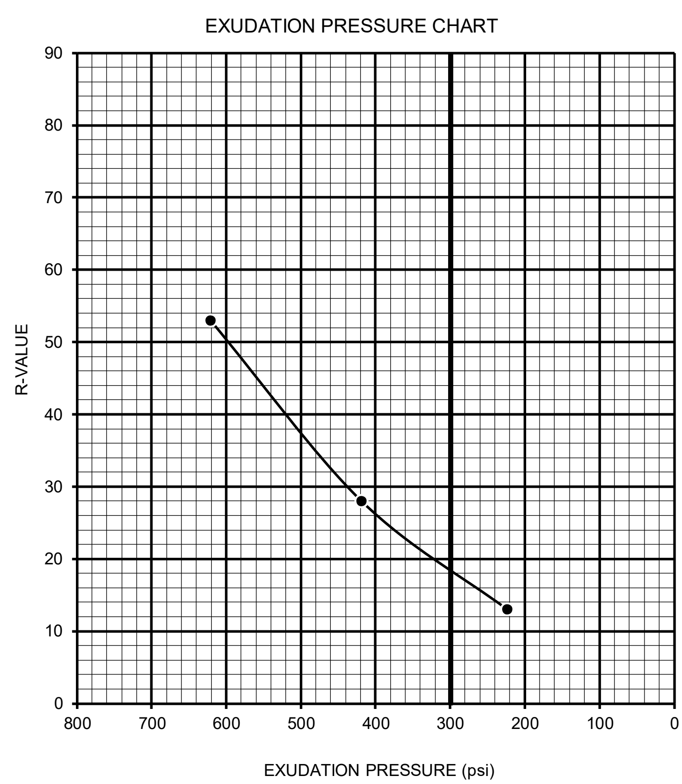
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	1977 Saturn Street	PROJECT NUMBER:	19850
BORING NUMBER:	LB-2	DEPTH (FT.):	0-5
SAMPLE NUMBER:	BB-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Light yellowish brown clayey sand with gravel (SC)g	DATE COMPLETED:	12/15/2023

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	10.4	11.7	13.4
HEIGHT OF SAMPLE, Inches	2.43	2.47	2.51
DRY DENSITY, pcf	128.9	126.4	121.6
COMPACTOR PRESSURE, psi	120	90	50
EXUDATION PRESSURE, psi	621	418	224
EXPANSION, Inches x 10exp-4	27	13	0
STABILITY Ph 2,000 lbs (160 psi)	57	100	128
TURNS DISPLACEMENT	3.75	3.90	4.20
R-VALUE UNCORRECTED	55	28	13
R-VALUE CORRECTED	53	28	13

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.75	1.15	1.39
EXPANSION PRESSURE THICKNESS, ft.	0.90	0.43	0.00





R-VALUE BY EXPANSION:	48
R-VALUE BY EXUDATION:	18
EQUILIBRIUM R-VALUE:	18

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D6913

Leighton

Project Name: 1977 Saturn Street
Project No.: 19850
Boring No.: LB-4
Sample No.: BB-1
Soil Identification: Yellowish brown clayey sand (SC)

Tested By: K. Jumig   Date: 12/12/23
Checked By: J. Ward   Date: 12/29/23
Depth (feet): 0-5

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	A-14	912	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	3456.3	617.8	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	225.0	106.0	Wt. of Container No.______(g)	1.0	1.0
Dry Wt. of Soil (g)	3231.3	511.8	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve		
	Container No.	912
	Wt. of Dry Soil + Container (g)	451.6
	Wt. of Container (g)	106.0
	Dry Wt. of Soil Retained on # 200 Sieve (g)	345.6

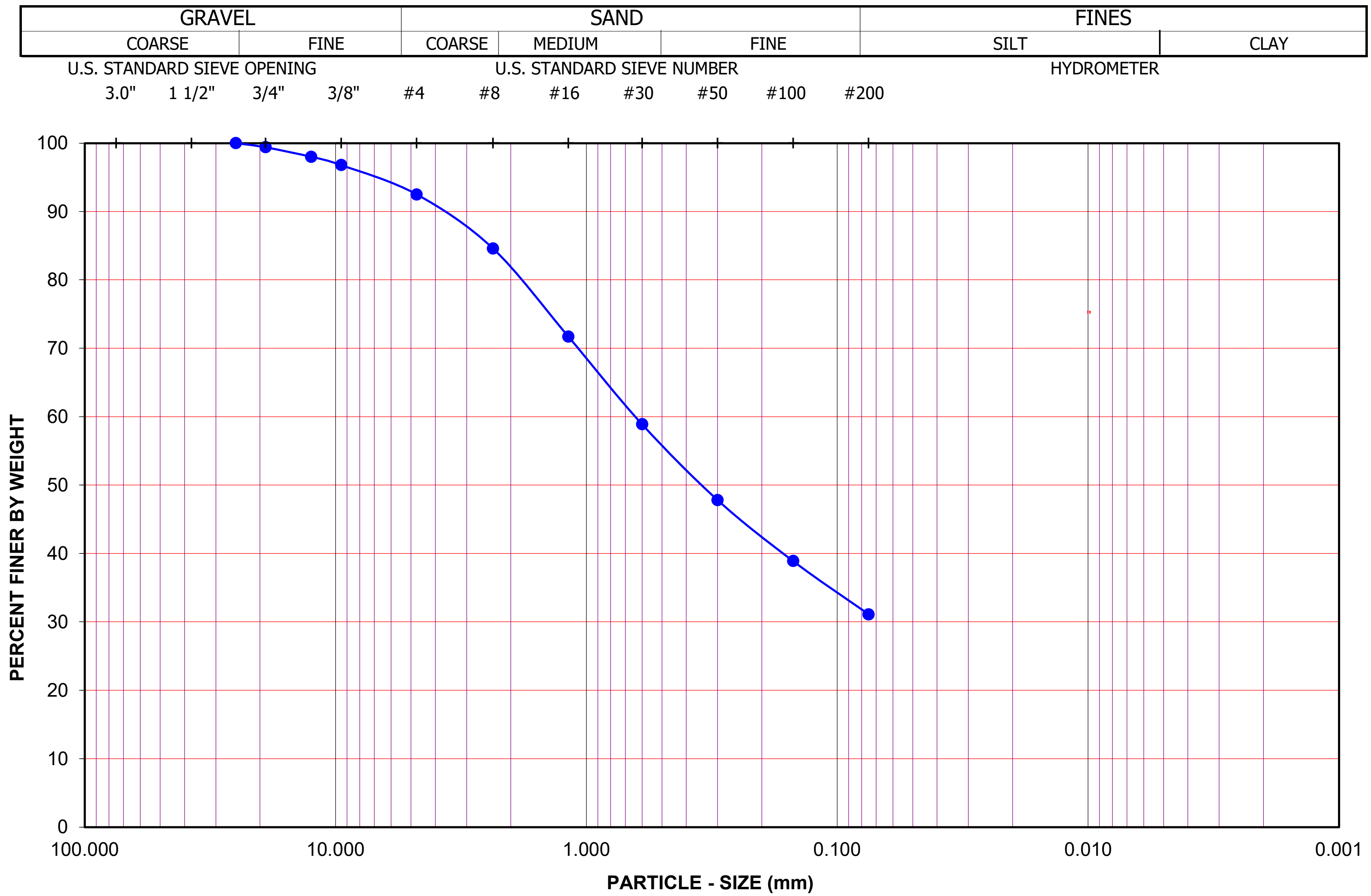
U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
1 1/2"	37.5			
1"	25.0	0.0		100.0
3/4"	19.0	19.8		99.4
1/2"	12.5	63.4		98.0
3/8"	9.5	102.1		96.8
#4	4.75	242.9		92.5
#8	2.36		43.9	84.6
#16	1.18		115.1	71.7
#30	0.600		186.0	58.9
#50	0.300		247.5	47.8
#100	0.150		296.4	38.9
#200	0.075		339.6	31.1
PAN				

GRAVEL: **8 %**
SAND: **61 %**
FINES: **31 %**
GROUP SYMBOL: **SC**

Cu = D60/D10 = ______

Cc = (D30)²/(D60*D10) = ______

Remarks: ______



Project Name: 1977 Saturn Street

Project No.: 19850

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: LB-4 Sample No.: BB-1

Depth (feet): 0-5 Soil Type : SC

Soil Identification: Yellowish brown clayey sand (SC)

GR:SA:FI : (%) 8 : 61 : 31

Dec-23

ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
ASTM D 4546

Project Name:	1977 Saturn Street	Tested By:	G. Bathala	Date:	12/12/23
Project No.:	19850	Checked By:	J. Ward	Date:	12/29/23
Boring No.:	LB-2	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.)	7.5		
Sample Description:	Olive clayey sand (SC)				

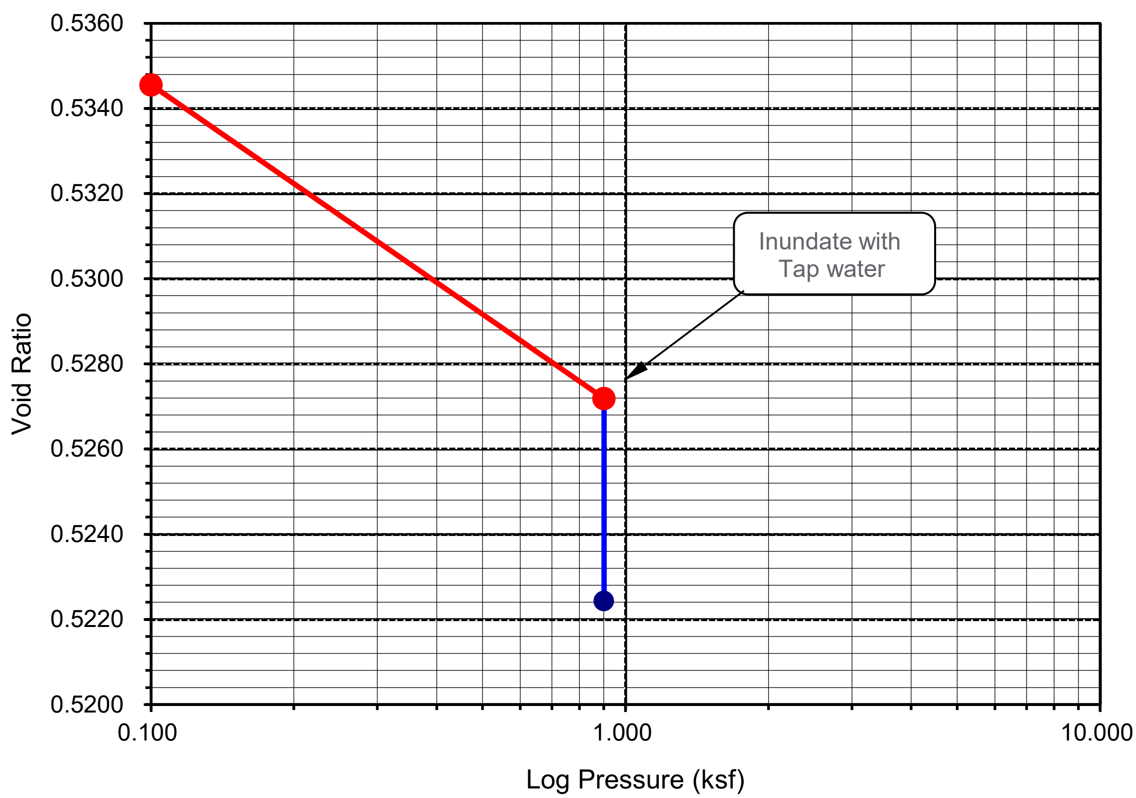
Initial Dry Density (pcf):	109.8
Initial Moisture (%):	11.62
Initial Length (in.):	1.0000
Initial Dial Reading:	0.1912
Diameter(in):	2.415

Final Dry Density (pcf):	111.7
Final Moisture (%) :	13.8
Initial Void ratio:	0.5349
Specific Gravity(assumed):	2.70
Initial Saturation (%)	58.7

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.1914	0.9998	0.00	-0.02	0.5346	-0.02
0.900	0.2007	0.9905	0.45	-0.95	0.5272	-0.50
H2O	0.2038	0.9874	0.45	-1.26	0.5224	-0.81

Percent Swell (+) / Settlement (-) After Inundation = -0.31

Void Ratio - Log Pressure Curve



Appendix C.2 – Lab Results (2015 Offsite)

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	Swinerton/Monterey Park	Tested By :	G. Berdy	Date:	10/23/15
Project No. :	11173.001	Data Input By:	J. Ward	Date:	10/28/15

Boring No.	LB-1			
Sample No.	BB-1			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown SC			
Wet Weight of Soil + Container (g)	180.35			
Dry Weight of Soil + Container (g)	178.00			
Weight of Container (g)	66.56			
Moisture Content (%)	2.11			
Weight of Soaked Soil (g)	100.27			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	62			
Crucible No.	15			
Furnace Temperature (°C)	850			
Time In / Time Out	10:00/10:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	20.3279			
Wt. of Crucible (g)	20.3245			
Wt. of Residue (g) (A)	0.0034			
PPM of Sulfate (A) x 41150	139.91			
PPM of Sulfate, Dry Weight Basis	**143**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	1.4			
PPM of Chloride (C -0.2) * 100 * 30 / B	120			
PPM of Chloride, Dry Wt. Basis	**123**			

pH TEST, DOT California Test 643

pH Value	**6.88**			
Temperature °C	21.3			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

Project Name:	Swinerton/Monterey Park	Tested By :	G. Berdy	Date:	10/26/15
Project No. :	11173.001	Data Input By:	J. Ward	Date:	10/28/15
Boring No.:	LB-1	Depth (ft.) :	0-5		
Sample No. :	BB-1				

Soil Identification:* Olive brown SC

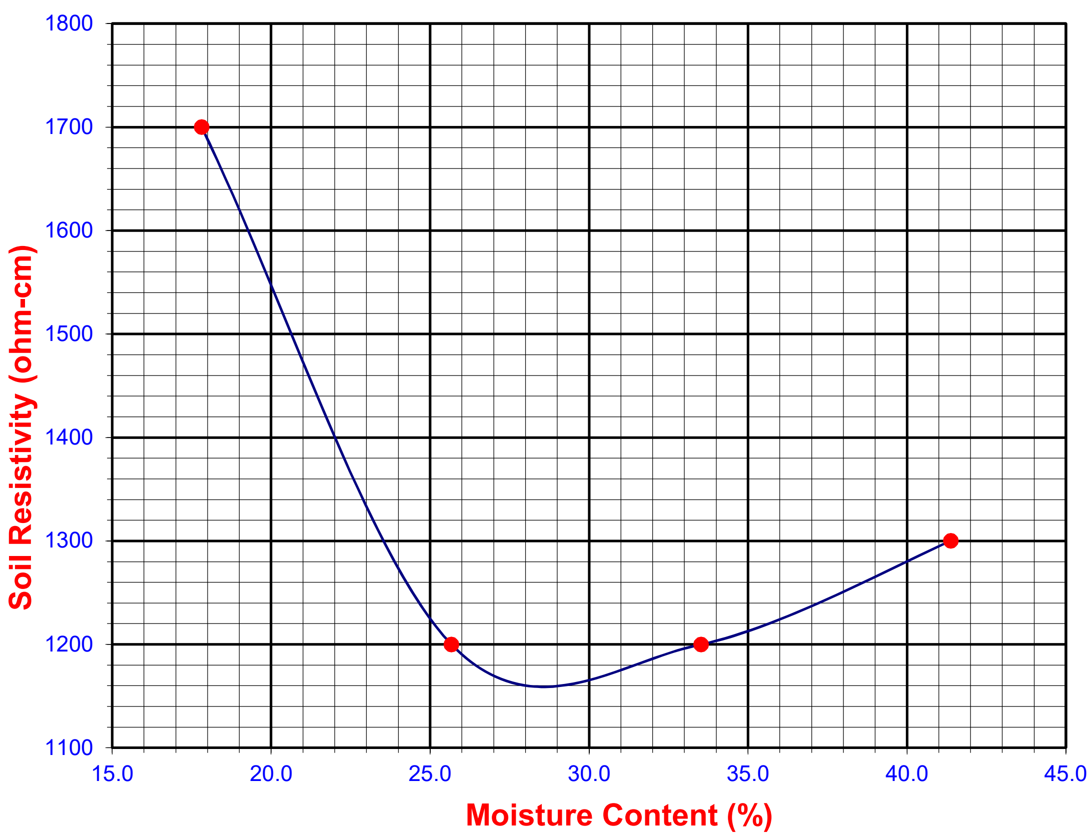
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	17.82	1700	1700
2	30	25.67	1200	1200
3	40	33.53	1200	1200
4	50	41.38	1300	1300
5				

Moisture Content (%) (MCi)	2.11
Wet Wt. of Soil + Cont. (g)	180.35
Dry Wt. of Soil + Cont. (g)	178.00
Wt. of Container (g)	66.56
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000

$MC = (((1+Mci/100)x(Wa/Wt+1))-1)x100$

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1160	**28.5**	**143**	**123**	**6.88**	**21.3**



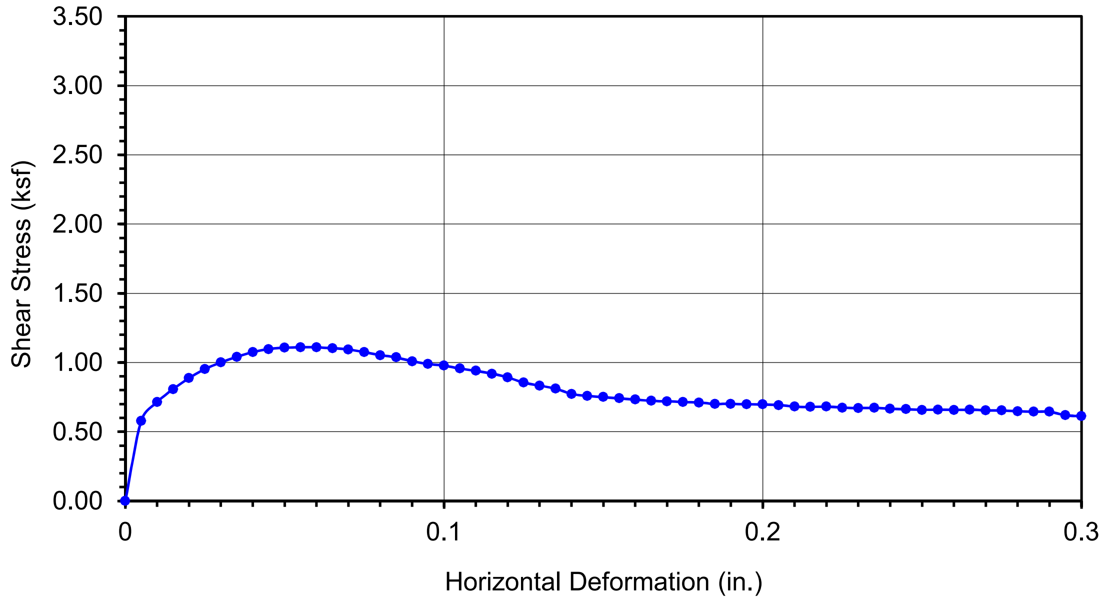
DIRECT SHEAR TEST

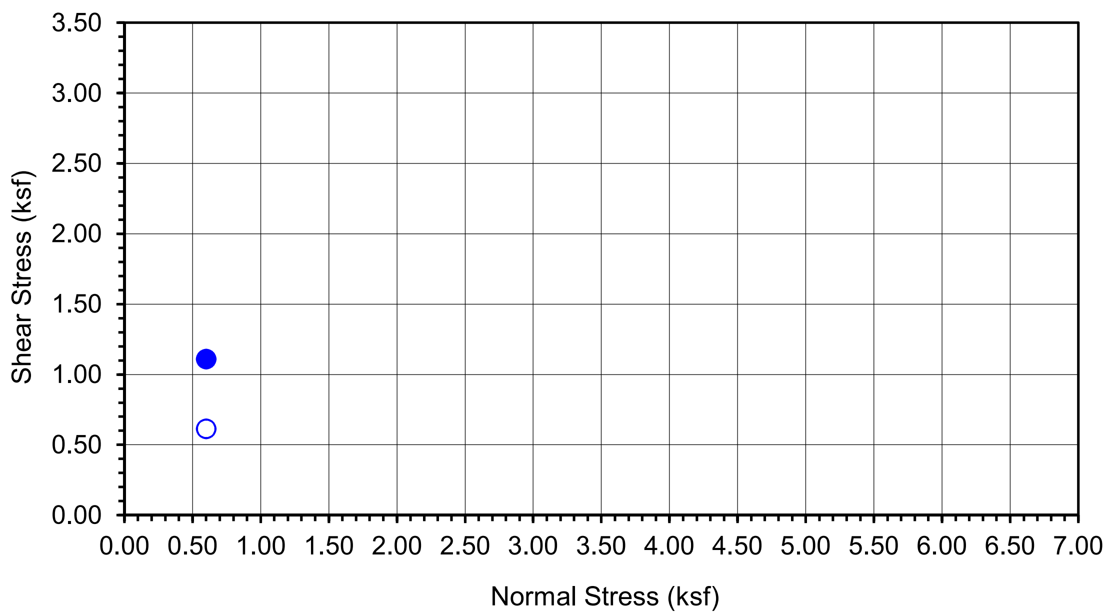
Consolidated Undrained

Project Name:	Swinerton/Monterey Park	Tested By:	G. Bathala	Date:	10/23/15
Project No.:	11173.001	Checked By:	J. Ward		
Boring No.:	LB-1	Sample Type:	Ring		
Sample No.:	R-1	Depth (ft.):	5.0		

Soil Identification: Olive brown sandy lean clay s(CL)

Sample Diameter(in):	2.415		
Sample Thickness(in.):	1.000		
Weight of Sample + ring(gm):	204.67		
Weight of Ring(gm):	43.23		
Before Shearing			
Weight of Wet Sample+Cont.(gm):	158.39		
Weight of Dry Sample+Cont.(gm):	145.16		
Weight of Container(gm):	37.58		
Vertical Rdg.(in): Initial	0.2559		
Vertical Rdg.(in): Final	0.2584		
After Shearing			
Weight of Wet Sample+Cont.(gm):	200.29		
Weight of Dry Sample+Cont.(gm):	181.00		
Weight of Container(gm):	38.24		
Specific Gravity (Assumed):	2.70		
Water Density(pcf):	62.43		





Boring No.	**LB-1**
Sample No.	**R-1**
Depth (ft)	**5**

Sample Type:

Ring

Soil Identification:
Olive brown sandy lean clay s(CL)

Normal Stress (kip/ft²)	0.600		
Peak Shear Stress (kip/ft²)	● 1.110		
Shear Stress @ End of Test (ksf)	○ 0.613		
Deformation Rate (in./min.)	0.0500		
Initial Sample Height (in.)	1.000		
Diameter (in.)	2.415		
Initial Moisture Content (%)	12.30		
Dry Density (pcf)	119.6		
Saturation (%)	81.0		
Soil Height Before Shearing (in.)	0.9975		
Final Moisture Content (%)	13.5		

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 11173.001

Swinerton/Monterey Park

10-15

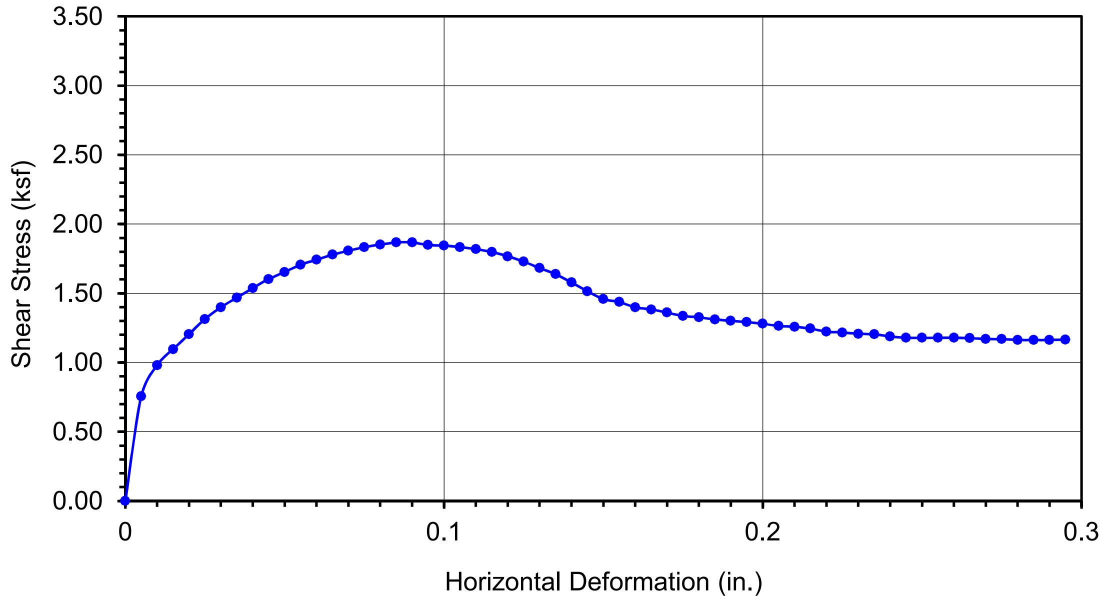
Leighton

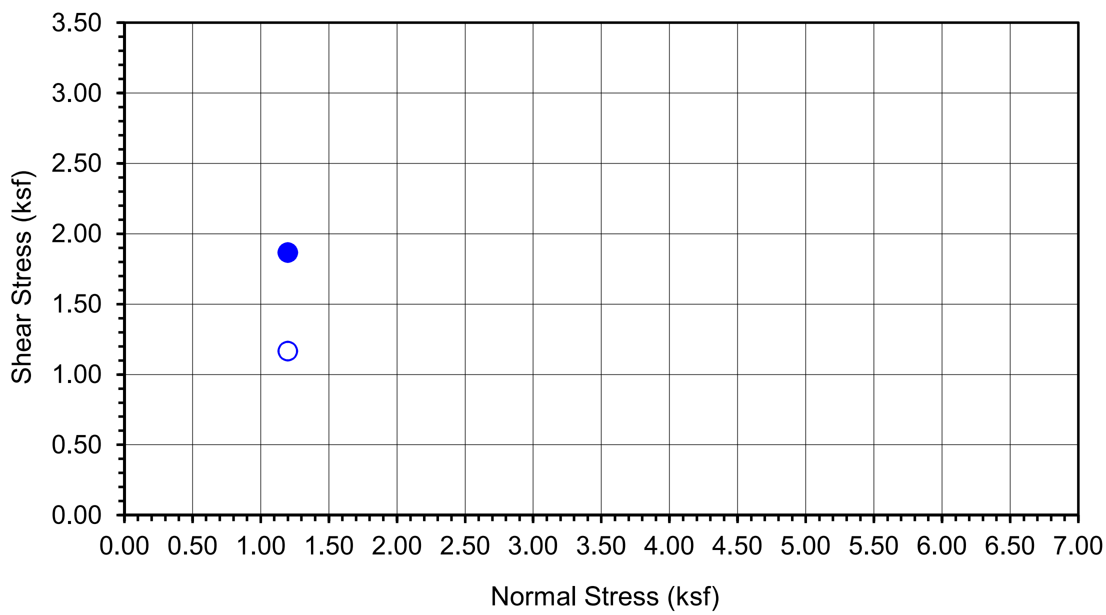
DIRECT SHEAR TEST

Consolidated Undrained

Project Name: Swinerton/Monterey Park
Tested By: G. Bathala
Date: 10/23/15
Project No.: 11173.001
Checked By: J. Ward
Boring No.: LB-1
Sample Type: Ring
Sample No.: R-2
Depth (ft.): 10.0
Soil Identification: Dark olive gray sandy lean clay s(CL)

Sample Diameter(in):	2.415		
Sample Thickness(in.):	1.000		
Weight of Sample + ring(gm):	205.51		
Weight of Ring(gm):	43.48		
Before Shearing			
Weight of Wet Sample+Cont.(gm):	219.15		
Weight of Dry Sample+Cont.(gm):	198.55		
Weight of Container(gm):	38.76		
Vertical Rdg.(in): Initial	0.2651		
Vertical Rdg.(in): Final	0.2784		
After Shearing			
Weight of Wet Sample+Cont.(gm):	213.75		
Weight of Dry Sample+Cont.(gm):	194.79		
Weight of Container(gm):	53.87		
Specific Gravity (Assumed):	2.70		
Water Density(pcf):	62.43		





Boring No.	**LB-1**
Sample No.	**R-2**
Depth (ft)	**10**

Sample Type:

Ring

Soil Identification:

Dark olive gray sandy lean clay s(CL)

Normal Stress (kip/ft²)	1.200		
Peak Shear Stress (kip/ft²)	● 1.867		
Shear Stress @ End of Test (ksf)	○ 1.166		
Deformation Rate (in./min.)	0.0500		
Initial Sample Height (in.)	1.000		
Diameter (in.)	2.415		
Initial Moisture Content (%)	12.89		
Dry Density (pcf)	119.4		
Saturation (%)	84.5		
Soil Height Before Shearing (in.)	0.9867		
Final Moisture Content (%)	13.5		

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Undrained

Project No.: 11173.001

Swinerton/Monterey Park

10-15

Leighton

DIRECT SHEAR TEST

Consolidated Undrained

Project Name: Swinerton/Monterey Park | Tested By: G. Bathala | Date: 10/23/15

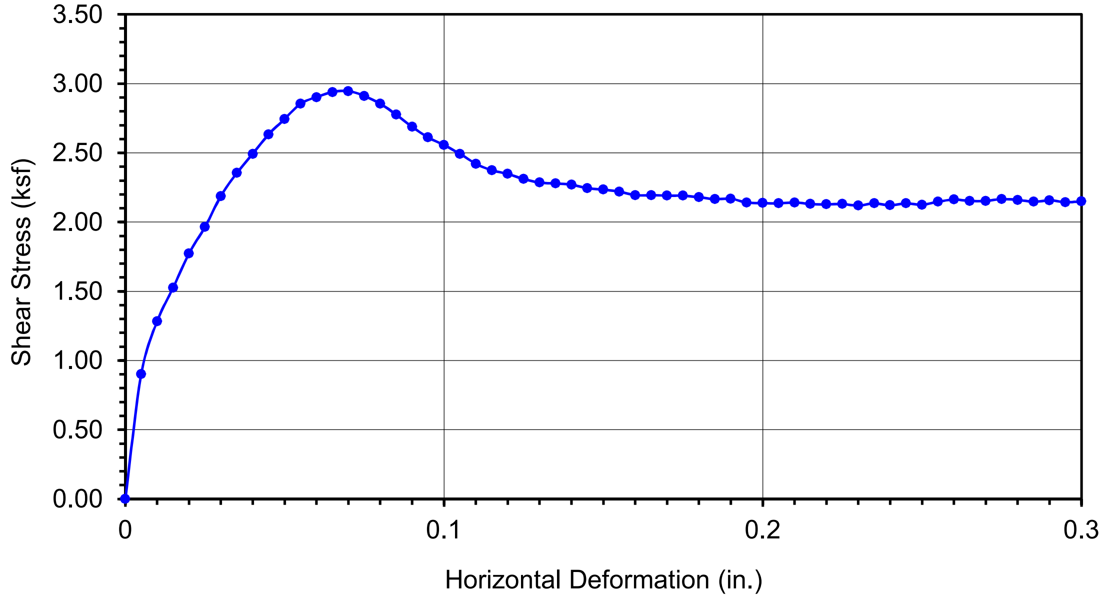
Project No.: 11173.001 | Checked By: J. Ward

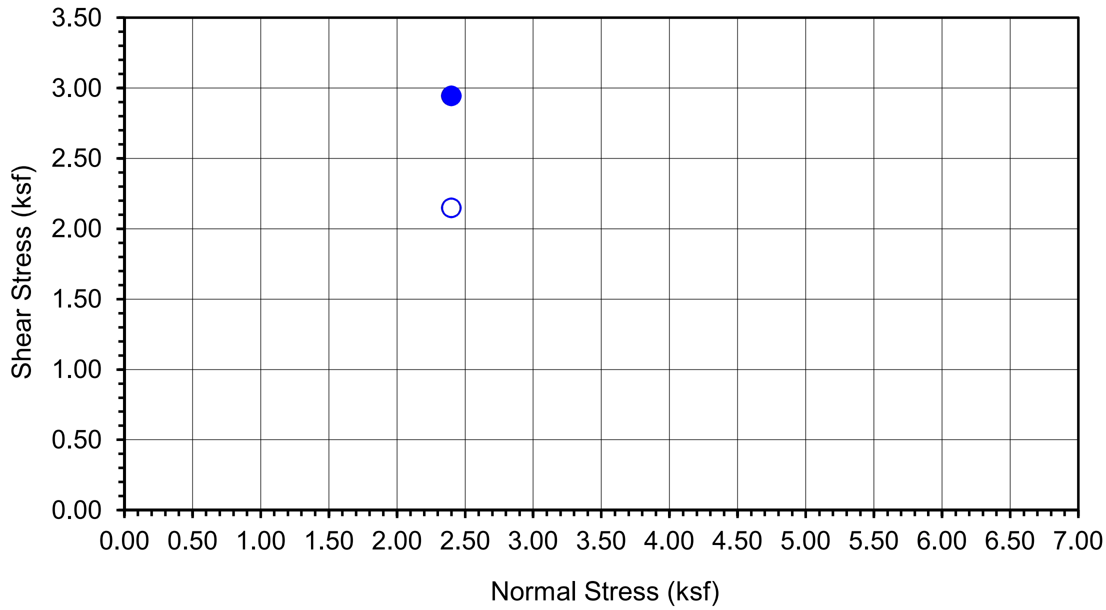
Boring No.: LB-1 | Sample Type: Ring

Sample No.: R-3 | Depth (ft.): 15.0

Soil Identification: Olive gray sandy lean clay s(CL)

Sample Diameter(in):	2.415		
Sample Thickness(in.):	1.000		
Weight of Sample + ring(gm):	205.98		
Weight of Ring(gm):	43.07		
Before Shearing			
Weight of Wet Sample+Cont.(gm):	207.82		
Weight of Dry Sample+Cont.(gm):	190.70		
Weight of Container(gm):	39.19		
Vertical Rdg.(in): Initial	0.0000		
Vertical Rdg.(in): Final	-0.0126		
After Shearing			
Weight of Wet Sample+Cont.(gm):	198.86		
Weight of Dry Sample+Cont.(gm):	181.42		
Weight of Container(gm):	37.58		
Specific Gravity (Assumed):	2.70		
Water Density(pcf):	62.43		





Boring No.	**LB-1**
Sample No.	**R-3**
Depth (ft)	**15**

Sample Type:

Ring

Soil Identification:
Olive gray sandy lean clay s(CL)

Normal Stress (kip/ft²)	2.400		
Peak Shear Stress (kip/ft²)	● 2.946		
Shear Stress @ End of Test (ksf)	○ 2.150		
Deformation Rate (in./min.)	0.0500		
Initial Sample Height (in.)	1.000		
Diameter (in.)	2.415		
Initial Moisture Content (%)	11.30		
Dry Density (pcf)	121.7		
Saturation (%)	79.3		
Soil Height Before Shearing (in.)	0.9874		
Final Moisture Content (%)	12.1		

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 11173.001

Swinerton/Monterey Park

10-15

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Swinerton/Monterey Park	Tested By:	O. Figueroa	Date:	10/26/15
Project No.:	11173.001	Input By:	J. Ward	Date:	10/28/15
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Olive brown clayey sand (SC)				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

			Scalp Fraction (%)	
Preparation Method:	X	Moist	#3/4	
		Dry		
Compaction Method	X	Mechanical Ram	#3/8	6.3
		Manual Ram	#4	

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) 0.03340

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3906	3993	3925			
Weight of Mold (g)	1851	1851	1851			
Net Weight of Soil (g)	2055	2142	2074			
Wet Weight of Soil + Cont. (g)	410.2	443.0	453.4			
Dry Weight of Soil + Cont. (g)	387.0	409.6	409.9			
Weight of Container (g)	39.2	39.0	39.2			
Moisture Content (%)	6.67	9.01	11.73			
Wet Density (pcf)	135.6	141.4	136.9			
Dry Density (pcf)	127.2	129.7	122.5			

Maximum Dry Density (pcf)	**130.0**	**Optimum Moisture Content (%)**	**8.5**
Corrected Dry Density (pcf)	**132.0**	**Corrected Moisture Content (%)**	**8.0**

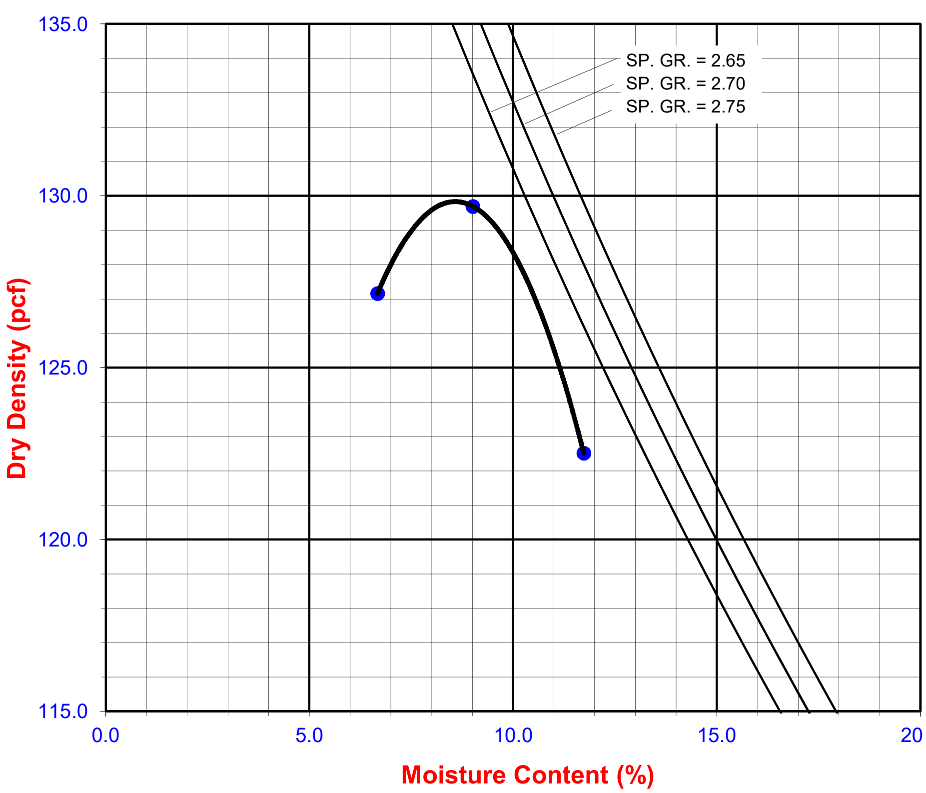
☐ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☒ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:
11:63:26
GR:SA:FI

Atterberg Limits:
30,16,14
LL,PL,PI



ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	Swinerton/Monterey Park	Tested By:	G. Bathala	Date:	10/24/15
Project No. :	11173.001	Input By:	J. Ward	Date:	10/28/15
Boring No.:	LB-1	Checked By:	J. Ward		
Sample No.:	BB-1	Depth (ft.)	0-5		
Soil Identification:	Olive brown clayey sand (SC)				

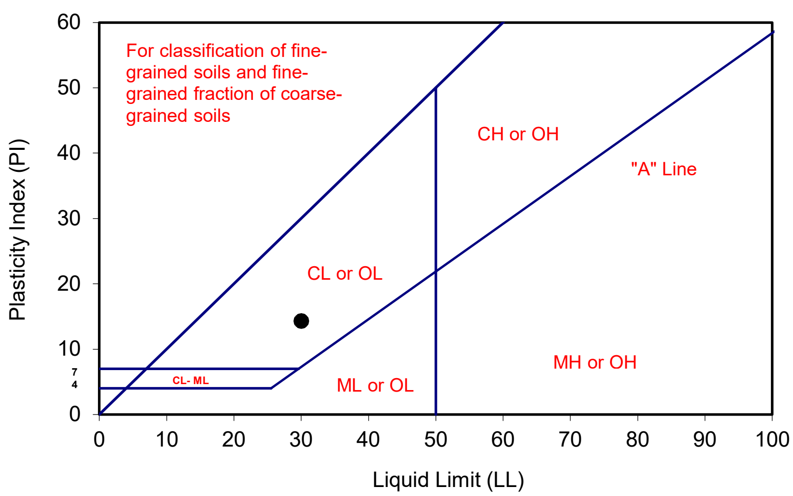
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			34	25	15	
Wet Wt. of Soil + Cont. (g)	20.33	18.43	30.42	29.94	30.02	
Dry Wt. of Soil + Cont. (g)	19.12	17.52	26.59	26.12	25.98	
Wt. of Container (g)	11.51	11.66	13.50	13.55	13.55	
Moisture Content (%) [Wn]	15.90	15.53	29.26	30.39	32.50	

Liquid Limit	**30**
Plastic Limit	**16**
Plasticity Index	**14**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) = 7.3

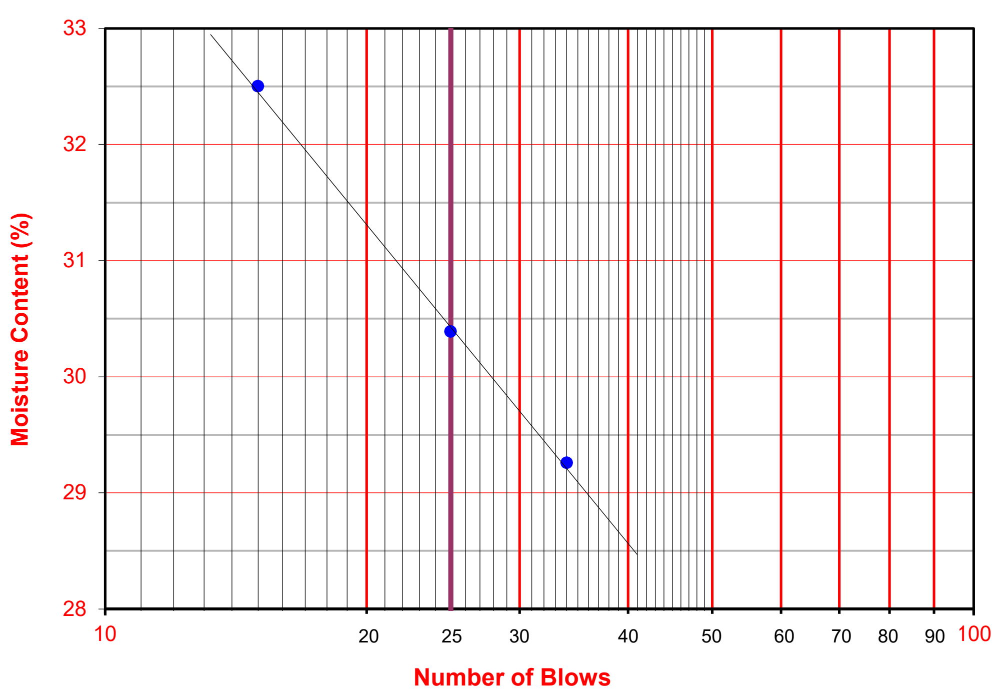
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

[ ]	Wet Preparation Multipoint - Wet
[X]	Dry Preparation Multipoint - Dry
[X]	Procedure A Multipoint Test
[ ]	Procedure B One-point Test



PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Leighton

Project Name: Swinerton/Monterey Park
Tested By: G. Bathala Date: 10/24/15
Project No.: 11173.001
Checked By: J. Ward Date: 10/28/15
Boring No.: LB-1
Depth (feet): 0-5
Sample No.: BB-1
Soil Identification: Olive brown clayey sand (SC)

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	SP-04	YK	Wt. of Air-Dry Soil + Cont.(g)	0.00	0.00
Wt. Air-Dried Soil + Cont.(g)	5932.1	773.2	Wt. of Dry Soil + Cont. (g)	0.00	0.00
Wt. of Container (g)	790.1	252.4	Wt. of Container No.______(g)	1.00	1.00
Dry Wt. of Soil (g)	5142.0	520.8	Moisture Content (%)	0.00	0.00

Passing #4 Material After Wet Sieve		
	Container No.	YK
	Wt. of Dry Soil + Container (g)	626.5
	Wt. of Container (g)	252.4
	Dry Wt. of Soil Retained on # 200 Sieve (g)	374.1

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
3"	75.000			
1 1/2"	37.500	0.0		100.0
3/4"	19.000	92.0		98.2
3/8"	9.500	325.4		93.7
#4	4.750	585.5		88.6
#8	2.360		36.9	82.3
#16	1.180		87.9	73.6
#30	0.600		151.6	62.8
#50	0.300		214.9	52.0
#100	0.150		303.4	37.0
#200	0.075		368.4	25.9
PAN				

GRAVEL: **11 %**

SAND: **63 %**

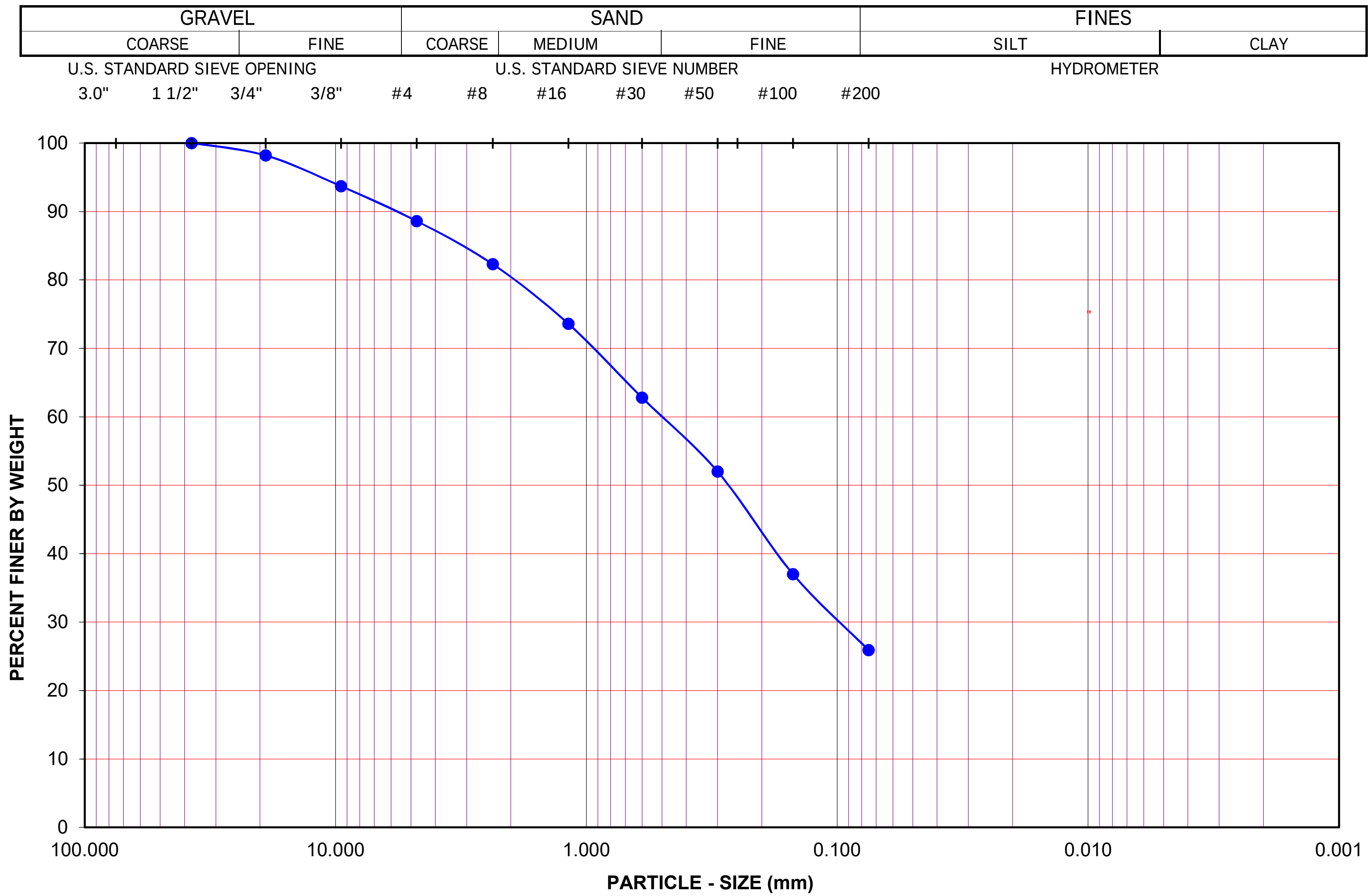
FINES: **26 %**

GROUP SYMBOL: **SC**

Cu = D60/D10 = ______

Cc = (D30)²/(D60*D10) = ______

Remarks: ______



Project Name: Swinerton/Monterey Park

Project No.: 11173.001

Boring No.: LB-1 Sample No.: BB-1

Depth (feet): 0-5 Soil Type : SC

Soil Identification: Olive brown clayey sand (SC)

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 11 : 63 : 26

10/28/15

Appendix C.3 – Lab Results (1977)

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APPENDIX B

LABORATORY TEST DATA

U.S. Standard Sieve Size		T-1 @2½'	T-1 @4½'	T-3 @2'	T-8 @5'	T-10 @5½'	T-11 @4'	T-13 @4'	T-14 @4½'	T-15 @5'
				% Passing Size Shown						
3"	Gravel			100						
2"	Gravel			97						
1½"	Gravel			92			100	100		
1"	Gravel			87			97	99		
3/4"	Gravel			82			94	97		
½"	Gravel			77			91	95		
3/8"	Gravel			72			88	93		
No. 4	Gravel			60			82	88		
No. 8	Sand			50			76	84		
No. 16	Sand			35			71	76		
No. 30	Sand			23			63	68		
No. 50	Sand			16			57	57		
No. 100	Sand			12			49	44		
No. 200	Sand			9			23	30		
	Silt & Clay									
U.S.C. Group Symbol				SW-SC			SM	SC		
Sand Equivalent				19			12	13		
In-Place Moisture		9.8	5.5		7.3	5.6	16.3	5.5	6.2	5.2
In-Place Dry Density		107.7	91.1		105.7	110.2	104.2	121.0	99.8	102.6
Optimum Moisture		10					10	9		
Maximum Dry Density		126.0					124.0	130.0		
Relative Compaction		86.0					84.0	93.0		

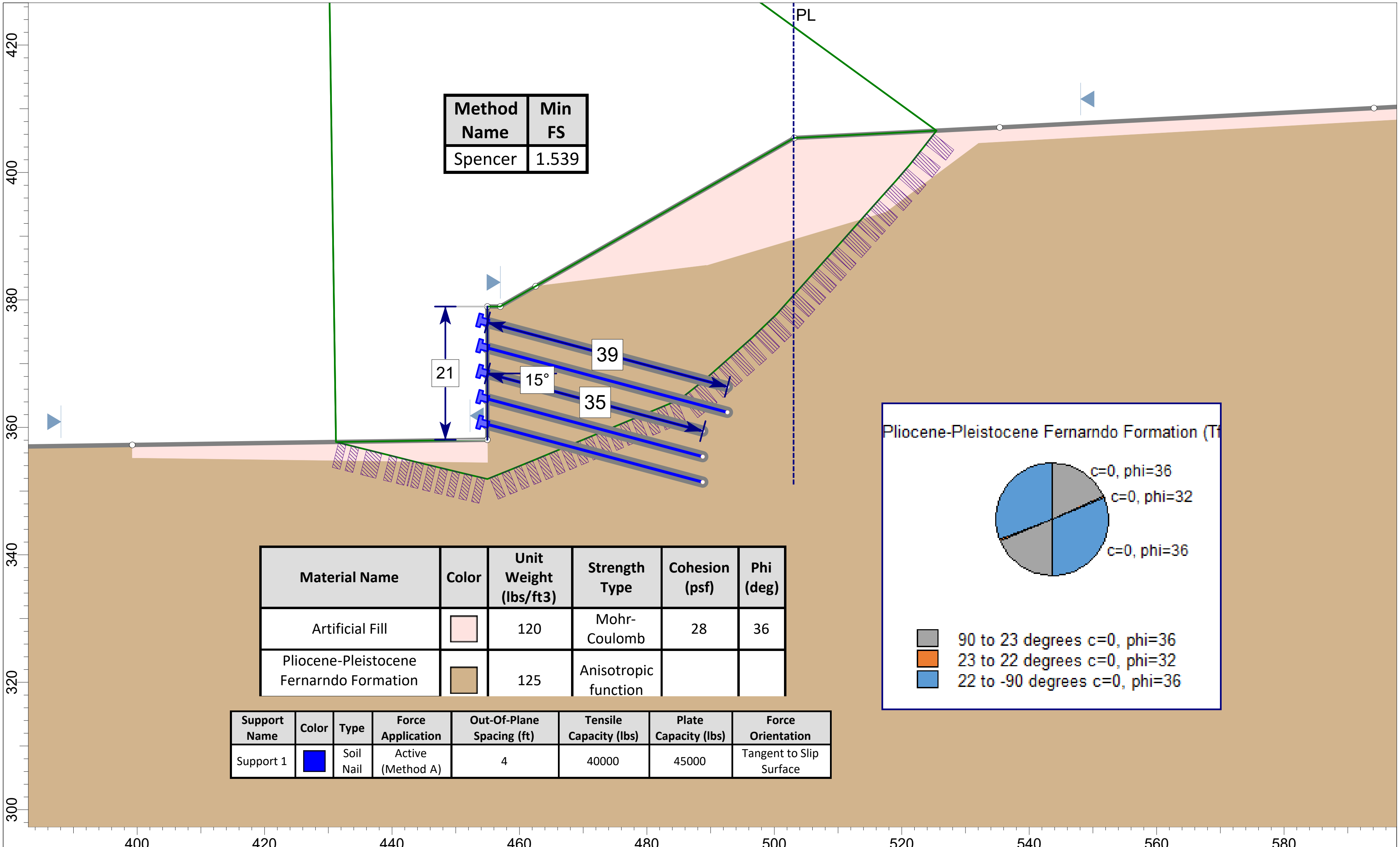
APPENDIX B

LABORATORY TEST DATA (CONTINUED)

	T-1 @2½'	T-1 @4½'	T-3 @2'	T-8 @5'	T-10 @5½'	T-11 @4'	T-13 @4'	T-14 @4½'	T-15 @5'
Expansion Index Data									
Molded Moisture			9.5			11.2			
Molded Dry Density			110.8			104.4			
Moisture After Test			17.2			20.2			
Volumetric Swell			1.0%			0.7%			
Expansion Index			10			7			
Potential Expansion			Very Low			Very Low			

APPENDIX D

SLOPE STABILITY RUNS



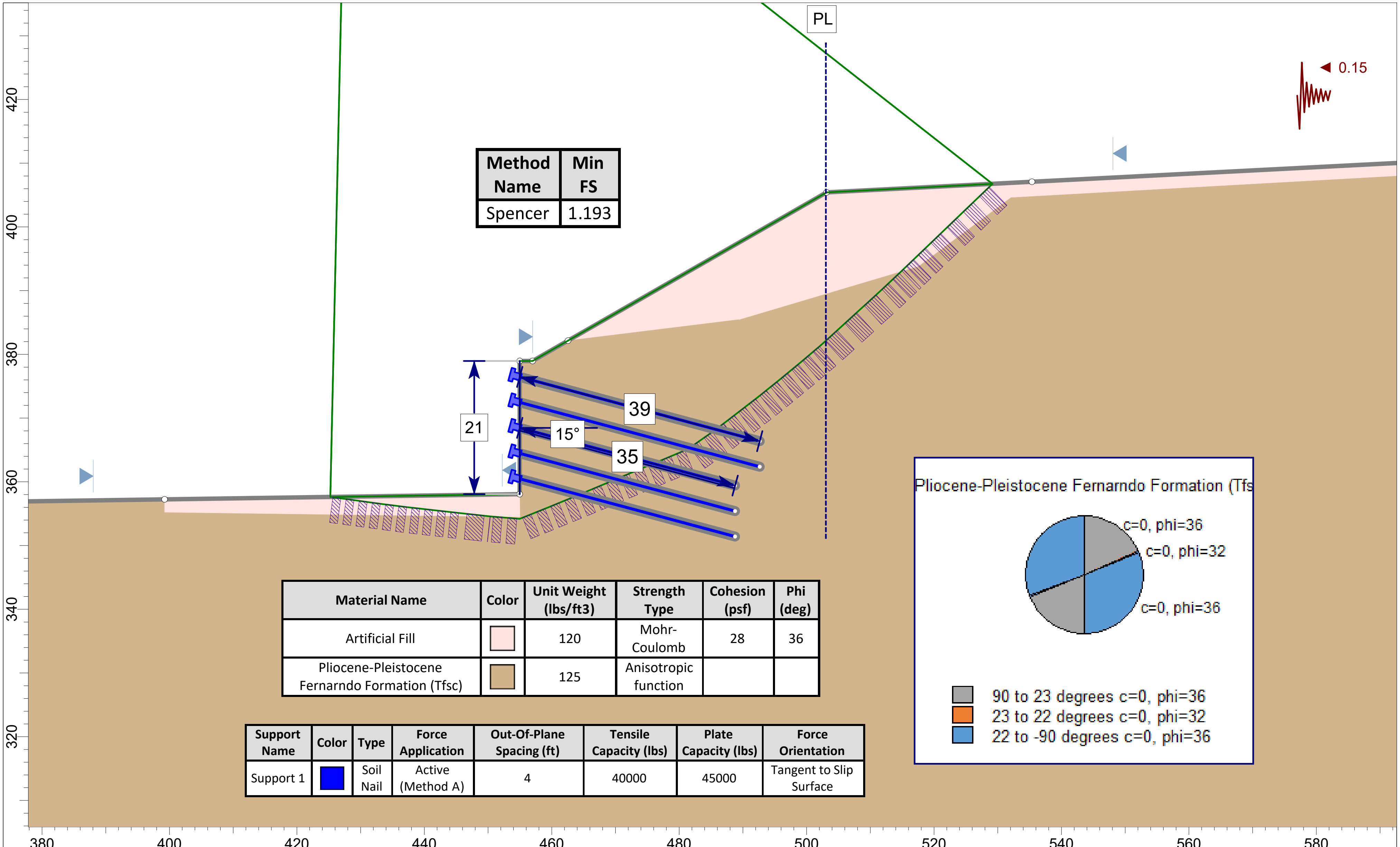
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Artificial Fill		120	Mohr-Coulomb	28	36
Pliocene-Pleistocene Fernarndo Formation		125	Anisotropic function		

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Force Orientation
Support 1		Soil Nail	Active (Method A)	4	40000	45000	Tangent to Slip Surface

Project	Proposed Data Center Facility, 1977 Saturn Street, Monterrey Park		
Group	Section B-B	*Scenario*	Static
Drawn By	LPM	*Company*	Leighton Consulting, Inc.
Date	1/30/2024	*File Name*	Section B-B'.slmd

Leighton

SLIDEINTERPRET 9.018



Method Name	Min FS
Spencer	1.193

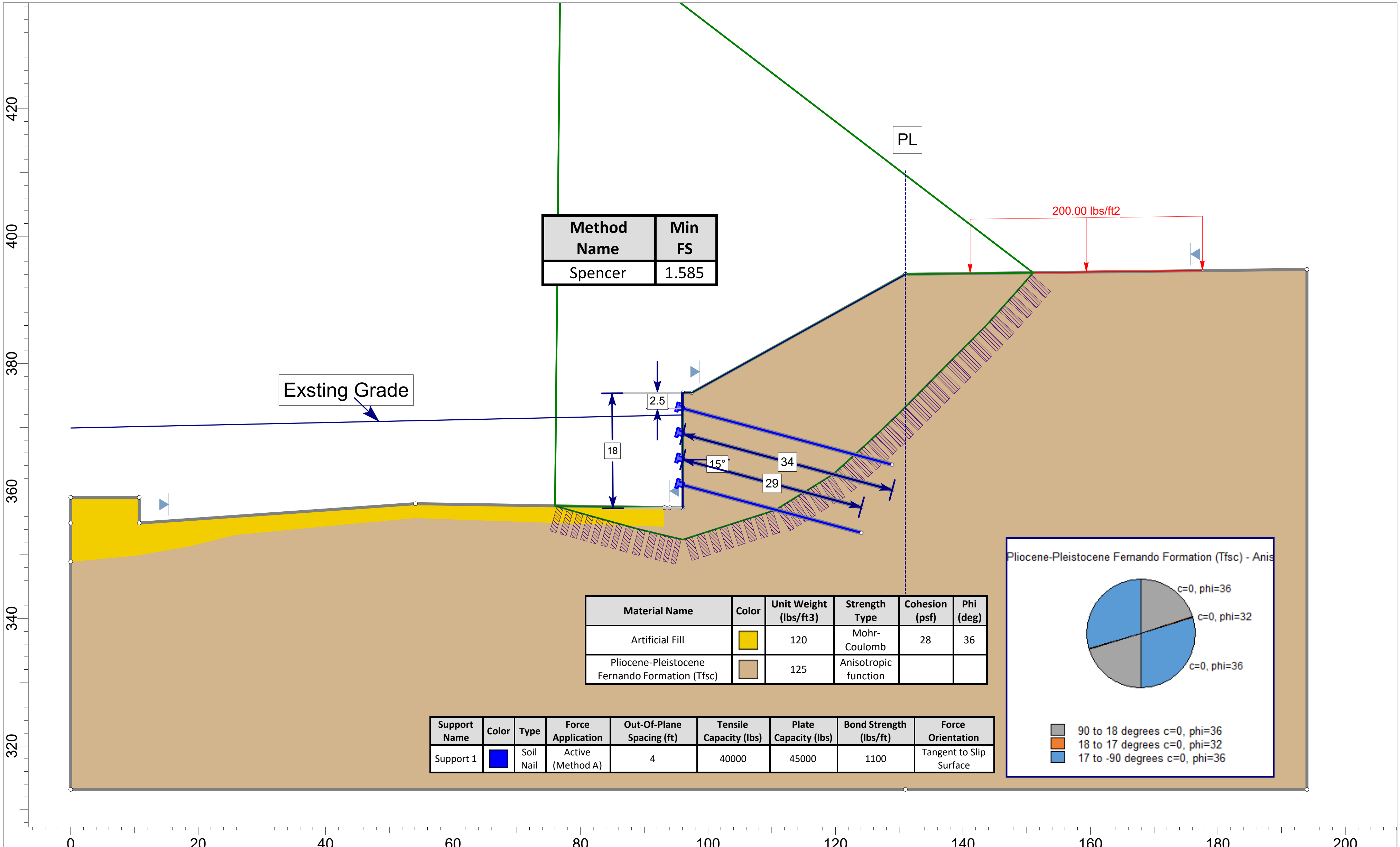
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Artificial Fill		120	Mohr-Coulomb	28	36
Pliocene-Pleistocene Fernarndo Formation (Tfsc)		125	Anisotropic function		

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Force Orientation
Support 1		Soil Nail	Active (Method A)	4	40000	45000	Tangent to Slip Surface

Project	Proposed Data Center Facility, 1977 Saturn Street, Monterrey Park		
Group	Section B-B	*Scenario*	Pseudo-Static
Drawn By	LPM	*Company*	Leighton Consulting, Inc.
Date	1/30/2024	*File Name*	Section B-B'.slmd

Leighton

SLIDEINTERPRET 9.018



Method Name	Min FS
Spencer	1.585

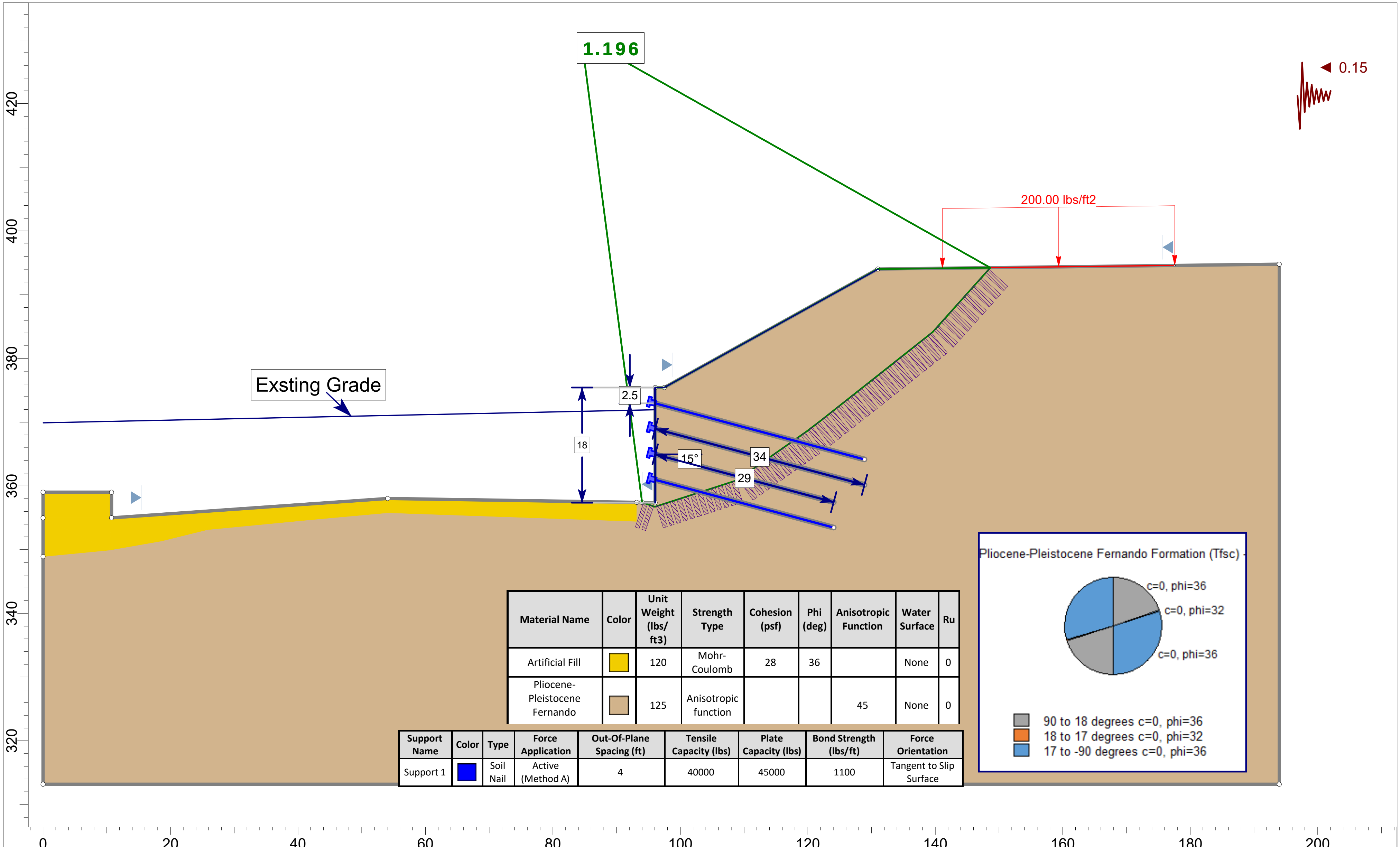
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Artificial Fill		120	Mohr-Coulomb	28	36
Pliocene-Pleistocene Fernando Formation (Tfsc)		125	Anisotropic function		

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Bond Strength (lbs/ft)	Force Orientation
Support 1		Soil Nail	Active (Method A)	4	40000	45000	1100	Tangent to Slip Surface

Leighton

SLIDEINTERPRET 9.018

Project	Proposed Data Center Facility, 1977 Saturn Street, Monterrey Park
Group Section D-D	*Scenario* Static
Drawn By LPM	*Company* Leighton Consulting, Inc.
Date 1/30/2024	*File Name* Section D-D'.slmd



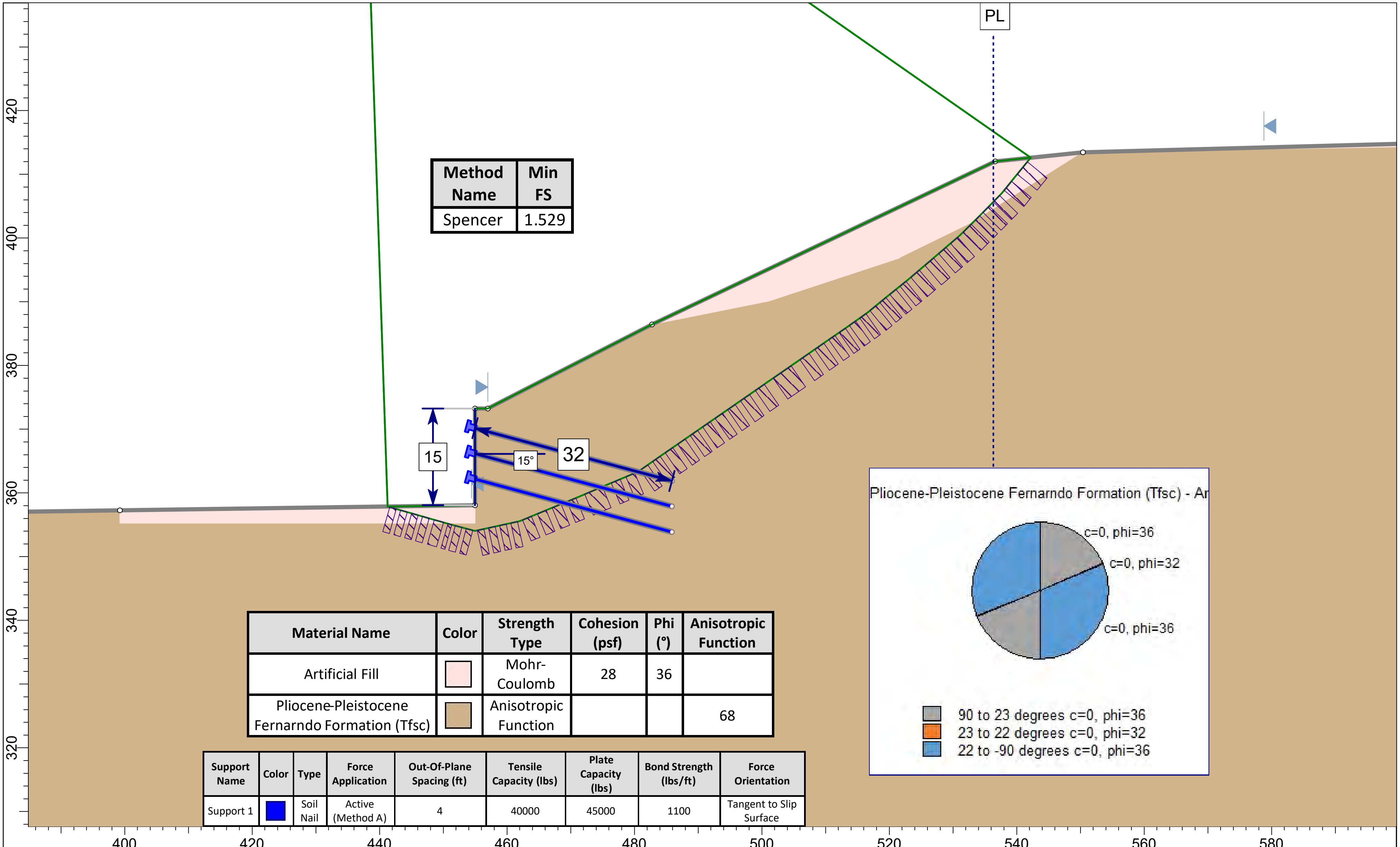
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Anisotropic Function	Water Surface	Ru
Artificial Fill		120	Mohr-Coulomb	28	36		None	0
Pliocene-Pleistocene Fernando		125	Anisotropic function			45	None	0

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Bond Strength (lbs/ft)	Force Orientation
Support 1		Soil Nail	Active (Method A)	4	40000	45000	1100	Tangent to Slip Surface

Leighton

SLIDEINTERPRET 9.018

Project	Proposed Data Center Facility, 1977 Saturn Street, Monterrey Park		
Group	Section D-D	*Scenario*	Pseudo-Static
Drawn By	LPM	*Company*	Leighton Consulting, Inc.
Date	1/30/2024	*File Name*	Section D-D'.slmd



Method Name	Min FS
Spencer	1.529

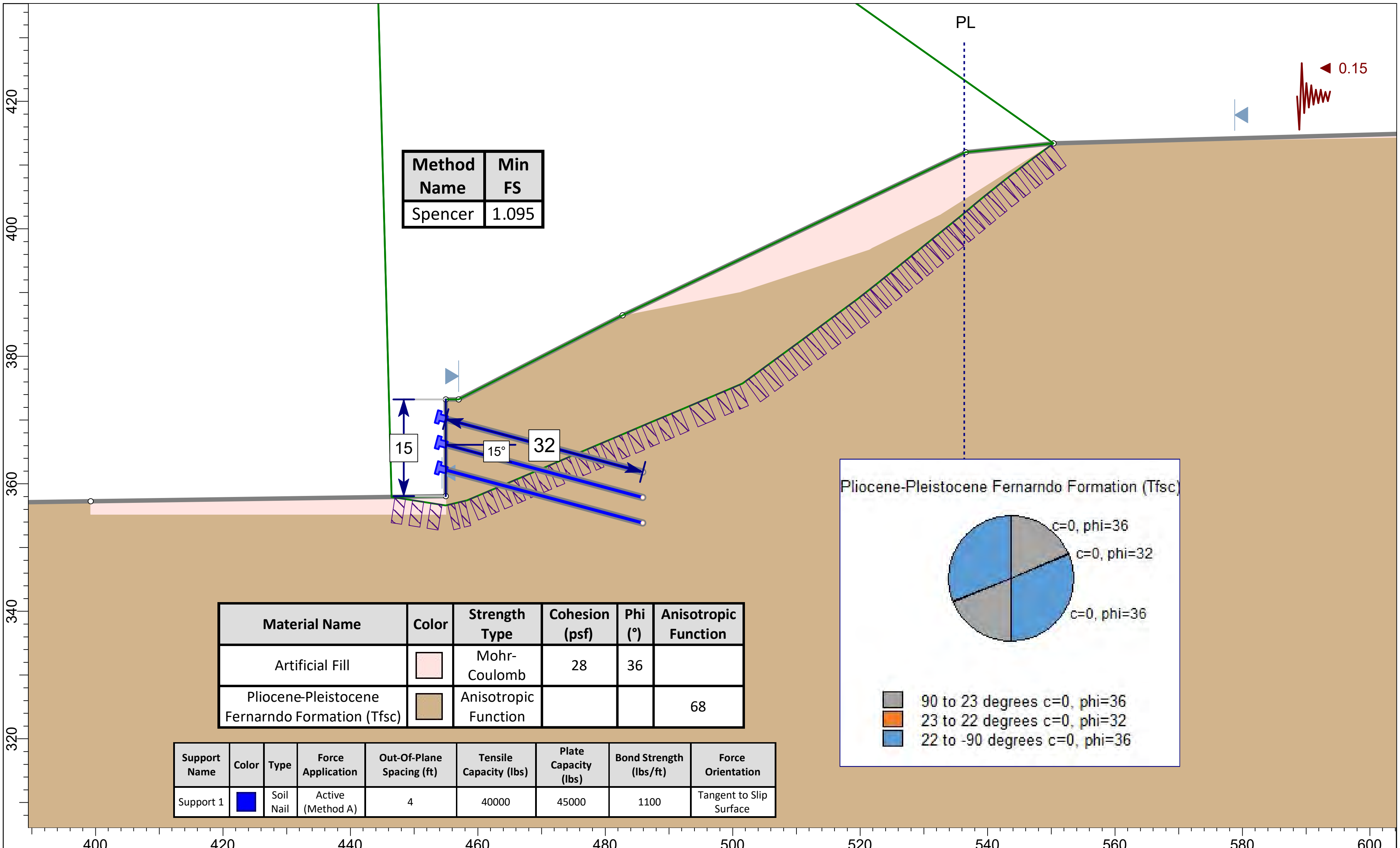
Material Name	Color	Strength Type	Cohesion (psf)	Phi (°)	Anisotropic Function
Artificial Fill		Mohr-Coulomb	28	36	
Pliocene-Pleistocene Fernarndo Formation (Tfsc)		Anisotropic Function			68

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Bond Strength (lbs/ft)	Force Orientation
Support 1		Soil Nail	Active (Method A)	4	40000	45000	1100	Tangent to Slip Surface

Project	Proposed Data Center Facility, 1977 Saturn Street, Monterrey Park		
Group	Section E-E	*Scenario*	Static
Drawn By	LPM	*Company*	Leighton Consulting, Inc.
Date	1/31/2024	*File Name*	Section E-E'.slmd

Leighton

SLIDEINTERPRET 9.031



Method Name	Min FS
Spencer	1.095

Material Name	Color	Strength Type	Cohesion (psf)	Phi (°)	Anisotropic Function
Artificial Fill		Mohr-Coulomb	28	36	
Pliocene-Pleistocene Fernarndo Formation (Tfsc)		Anisotropic Function			68

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Bond Strength (lbs/ft)	Force Orientation
Support 1		Soil Nail	Active (Method A)	4	40000	45000	1100	Tangent to Slip Surface

SLIDEINTERPRET 9.03

Leighton

Project	Proposed Data Center Facility, 1977 Saturn Street, Monterrey Park		
Group	Section E-E	*Scenario*	Pseudo Static
Drawn By	LPM	*Company*	Leighton Consulting, Inc.
Date	1/31/2024	*File Name*	Section E-E'.slmd

APPENDIX E

SEISMIC

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Determination of Site Class and Estimation of Shear Wave Velocity

Project: 19850 - 1977 Saturn

Depth (ft)	di, Layer Thick (ft)	Field Blow Counts, Ni Corrected for Cs and sampler type Blows per foot (bpf) LB-1	LB-2	LB-3	LB-4	Average Ni (bpf)	Ni Hammer Corr: 1.3	di / Ni
5	6		14	15		15	19	0.32
7	2.5	----------------	10	15		13	16	0.15
10	4		10	12		11	14	0.28
15	5		19	34	------------	27	34	0.15
20	5		44	80		62	81	0.06
25	5		42			42	55	0.09
30	5		42	*Assumed based on deepest blowcount		42	55	0.09
35	5		42	and blowcounts within bedrock.		42	55	0.09
40	7.5		42			42	55	0.14
50	10		42			42	55	0.18
60	10		70		**Legend:**	70	91	0.11
70	10		70		(pink) Approx. New Fill for Pad Grade	70	91	0.11
80	10		70		--------- Approx. Bedrock Contact	70	91	0.11
90	10		70			70	91	0.11
100	5		70			70	91	0.05
Summation	100							2.05
							Navg = Sum(di) / Sum(di / Ni) =	**49**

Extract of ASCE 7-16 Table 20.3-1 Site Classification (2022 CBC 1613.2.2):

Site Class	Soil Profile Name	Avg. N upper 100' from	to	Vs30 (ft/sec) from	to	Vs30 (m/s) from	to	Site Avg N	Interpolated vs30 (ft/s)
A	Hard Rock	-		5000	10000	1524	3048		
B	Rock	-		2500	5000	762	1524		
C	VD soil & soft rock	50.001	100	1200	2500	366	762		
D	Stiff Soil	15	50	600	1200	183	366	**49**	**1180**
E	Soft Soil	0	14.999	0	600	0	183		
F		-	-			0	0		

Site class, Table 20.3-1: D

Estimation of Average Shear Wave Velocity in upper 100 ft (Vs30):

	ft/s	m/s
Approx. Vs30 (interpolation of Table 20.3-1) =	1180	360
Approx. Vs30 sands (Imai and Tonouchi, 1982) =	1186	362
Approx. Vs30 sands (Sykora and Stokoe, 1983) =	1000	305
Approx. Vs30 (Maheswari, Boominathan, Dodagoudar, 2009) =	971	296

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

OSHPD

Latitude, Longitude: 34.0402, -118.1141

Date	1/8/2024, 5:31:51 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.918	MCE_R ground motion. (for 0.2 second period)
S_1	0.689	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.918	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.279	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.828	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.91	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.918	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.145	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.469	Factored deterministic acceleration value. (0.2 second)
S1RT	0.689	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.771	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.773	Factored deterministic acceleration value. (1.0 second)
PGAd	0.994	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.828	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.894	Mapped value of the risk coefficient at short periods
C_{R1}	0.894	Mapped value of the risk coefficient at a period of 1 s
C_V	1.484	Vertical coefficient

Determination of Site Class and Estimation of Shear Wave Velocity

Project: 19850 - 1977 Saturn

Depth (ft)	di, Layer Thick (ft)	Field Blow Counts, Ni Corrected for Cs and sampler type Blows per foot (bpf)				Average Ni (bpf)	Ni Hammer Corr:	di / Ni
		LB-1	LB-2	LB-3	LB-4		1.3	
5	6	22	14	15	15	17	21	0.28
7	2.5	22	10	15	15	16	20	0.12
10	4	34	10	12	15	18	23	0.17
15	5	79	19	34	80	53	69	0.07
20	5	84	44	80	69	69	90	0.06
25	5		42	78		60	78	0.06
30	5		42			42	55	0.09
35	5		42			42	55	0.09
40	7.5		42			42	55	0.14
50	10		70			70	91	0.11
60	10		70			70	91	0.11
70	10		70			70	91	0.11
80	10		70			70	91	0.11
90	10		70			70	91	0.11
100	5		70			70	91	0.05
Summation	100							1.69
						Navg = Sum(di) / Sum(di / Ni) =		59

*Assumed based on deepest blowcount and blowcounts within bedrock.

Legend:
- (pink shading) Approx. New Fill for Pad Grade
- --------- Approx. Bedrock Contact

Extract of ASCE 7-16 Table 20.3-1 Site Classification (2022 CBC 1613.2.2):

Site Class	Soil Profile Name	Avg. N upper 100' from	Avg. N upper 100' to	Vs30 (ft/sec) from	Vs30 (ft/sec) to	Vs30 (m/s) from	Vs30 (m/s) to	Site Avg N	Interpolated vs30 (ft/s)
A	Hard Rock	-		5000	10000	1524	3048		
B	Rock	-		2500	5000	762	1524		
C	VD soil & soft rock	50.001	100	1200	2500	366	762	**59**	**1435**
D	Stiff Soil	15	50	600	1200	183	366		
E	Soft Soil	0	14.999	0	600	0	183		
F		-	-			0	0		

Site class, Table 20.3-1: C

Estimation of Average Shear Wave Velocity in upper 100 ft (Vs30):

	ft/s	m/s
Approx. Vs30 (interpolation of Table 20.3-1) =	1435	437
Approx. Vs30 sands (Imai and Tonouchi, 1982) =	1259	384
Approx. Vs30 sands (Sykora and Stokoe, 1983) =	1053	321
Approx. Vs30 (Maheswari, Boominathan, Dodagoudar, 2009) =	1029	313

Latitude, Longitude: 34.0402, -118.1141

Date	1/8/2024, 5:32:44 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.918	MCE_R ground motion. (for 0.2 second period)
S_1	0.689	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.302	Site-modified spectral acceleration value
S_{M1}	0.965	Site-modified spectral acceleration value
S_{DS}	1.534	Numeric seismic design value at 0.2 second SA
S_{D1}	0.643	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.4	Site amplification factor at 1.0 second
PGA	0.828	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.993	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.918	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.145	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.469	Factored deterministic acceleration value. (0.2 second)
S1RT	0.689	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.771	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.773	Factored deterministic acceleration value. (1.0 second)
PGAd	0.994	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.828	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.894	Mapped value of the risk coefficient at short periods
C_{R1}	0.894	Mapped value of the risk coefficient at a period of 1 s
C_V	1.284	Vertical coefficient

Liquefaction Susceptibility Analysis: SPT Method

Leighton

Youd and Idriss (2001), Martin and Lew (1999)

Description: 1977 Saturn Monterrey Park; Case 3; PGAm 0.993; design GW 100; Overex./scarify 4
Project No.: 19850
Jan 2024

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex or mitig depth (ft)	Ground Surface Elev (ft)	design gw elve
LB-1	100	100		4	370	270
LB-2	100	100		4	370	270
LB-3	100	100		4	351	251
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates X (ft)	Y (ft)

General Parameters:

a_{max} = 0.99g
M_W = 6.91
MSF eq: 1
MSF = 1.23
Hammer Efficiency = 84
C_E = 1.40
C_B = 1
C_S for SPT? TRUE
Unlined, but room for liner
Rod Stickup (feet) = 3
Ring sample correction = 0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Leighton

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: 1977 Saturn Monterrey Park; Case 3; PGAm 0.993; design GW 100; Overex./scarify 4

Project No.: 19850

Boring No.	Approx. Layer Depth (ft)	Elevation (ft)	SPT Depth (ft)	Approx Layer Thickness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settlement) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 2.5	369	1	2.5	OX	36	120	50	1	1.3	65.0	120	116.0	144.2	120	0.64	0.52	**NonLiq**	144.2	0.00	0.00	0.1
LB-1	2.5 to 4.0	366	4	1.5	OX	50	120	50	1	1.3	65.0	480	116.0	144.2	480	0.64	0.52	**NonLiq**	144.2	0.00	0.00	0.1
LB-1	4.0 to 6.5	366	4	2.5		50	120	22	1	1.3	28.6	480	51.1	66.3	480	0.64	0.52	**NonLiq**	66.3	0.06	0.02	0.1
LB-1	6.5 to 11.5	361	9	5.0		**44**	120	34	1	1.3	44.2	1080	68.8	87.6	1080	0.63	0.51	**NonLiq**	87.6	0.05	0.03	0.1
LB-1	11.5 to 16.5	356	14	5.0		44	120	89	1	1.3	115.7	1680	153.5	189.2	1680	0.62	0.51	**NonLiq**	189.2	0.02	0.01	0.0
LB-1	16.5 to 21.0	351	19	4.5		44	120	84	1	1.3	109.2	2280	139.0	171.8	2280	0.62	0.50	**NonLiq**	171.8	0.03	0.01	0.0
LB-2	0 to 4.0	366	4	4.0	OX	54	120	50	1	1.3	65.0	480	116.0	144.2	480	0.64	0.52	**NonLiq**	144.2	0.00	0.00	0.6
LB-2	4.0 to 6.5	366	4	2.5		54	120	15	1	1.3	19.5	480	34.8	46.8	480	0.64	0.52	**NonLiq**	46.8	0.09	0.03	0.6
LB-2	6.5 to 11.5	361	9	5.0		**54**	120	11	1	1.21	13.3	1080	20.7	29.8	1080	0.63	0.51	**NonLiq**	29.8	0.76	0.46	0.6
LB-2	11.5 to 16.5	356	14	5.0		30	120	19	1	1.3	24.7	1680	32.8	42.5	1680	0.62	0.51	**NonLiq**	42.5	0.07	0.04	0.1
LB-2	16.5 to 21.5	351	19	5.0		17	120	44	1	1.3	57.2	2280	72.8	80.2	2280	0.62	0.50	**NonLiq**	80.2	0.06	0.03	0.1
LB-2	21.5 to 26.0	346	24	4.5		**17**	120	42	1	1.3	54.6	2880	61.8	68.6	2880	0.61	0.49	**NonLiq**	68.6	0.11	0.06	0.1
LB-3	0 to 4.0	348.5	2.5	4.0	OX	35	120	50	1	1.3	65.0	300	116.0	144.2	300	0.64	0.52	**NonLiq**	144.2	0.00	0.00	0.1
LB-3	4.0 to 4.3	348.5	2.5	0.3		35	120	15	1	1.3	19.5	300	34.8	46.8	300	0.64	0.52	**NonLiq**	46.8	0.03	0.00	0.1
LB-3	4.3 to 9.5	345	6	5.3		35	120	15	1	1.3	19.5	720	34.8	46.8	720	0.64	0.52	**NonLiq**	46.8	0.05	0.03	0.1
LB-3	9.5 to 14.0	338	13	4.5		50	120	69	2	1	44.9	1560	61.7	79.1	1560	0.63	0.51	**NonLiq**	79.1	0.03	0.02	0.1
LB-3	14.0 to 16.5	336	15	2.5		**36**	120	34	1	1.3	44.2	1800	56.6	73.0	1800	0.62	0.51	**NonLiq**	73.0	0.04	0.01	0.1
LB-3	16.5 to 20.5	333	18	4.0		18	120	90	2	1	58.5	2160	76.5	84.8	2160	0.62	0.50	**NonLiq**	84.8	0.05	0.02	0.0
LB-3	20.5 to 25.0	328	23	4.5		**18**	120	78	1	1.3	101.4	2760	117.3	128.3	2760	0.61	0.50	**NonLiq**	128.3	0.04	0.02	0.0

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (4.2.0)

Latitude

Decimal degrees

34.0402

Longitude

Decimal degrees, negative values for western longitudes

-118.1141

Site Class

360 m/s (C/D boundary)

Spectral Period

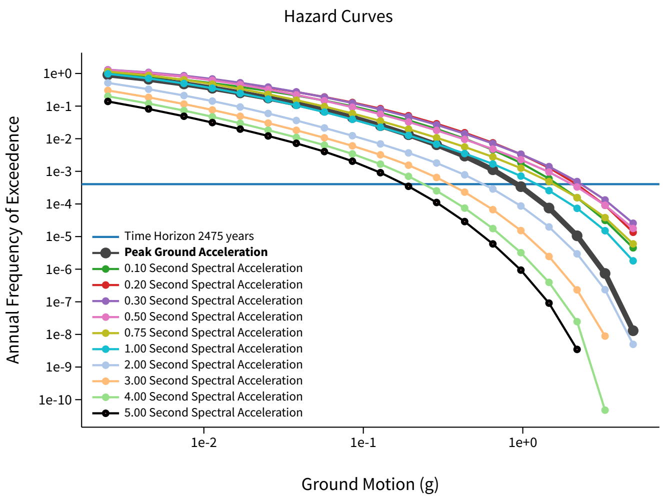
Peak Ground Acceleration

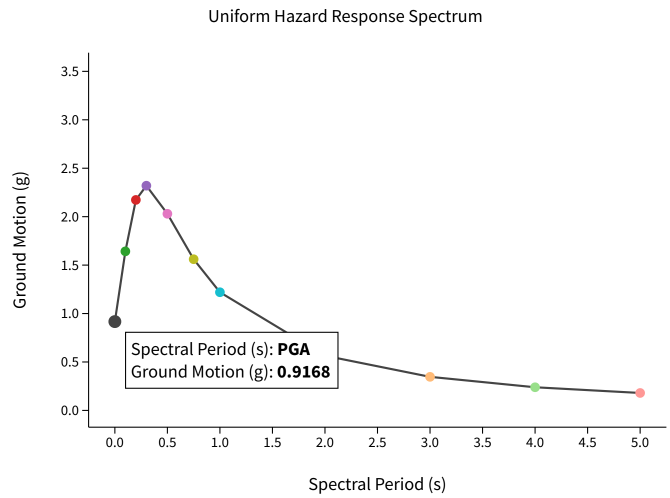
Time Horizon

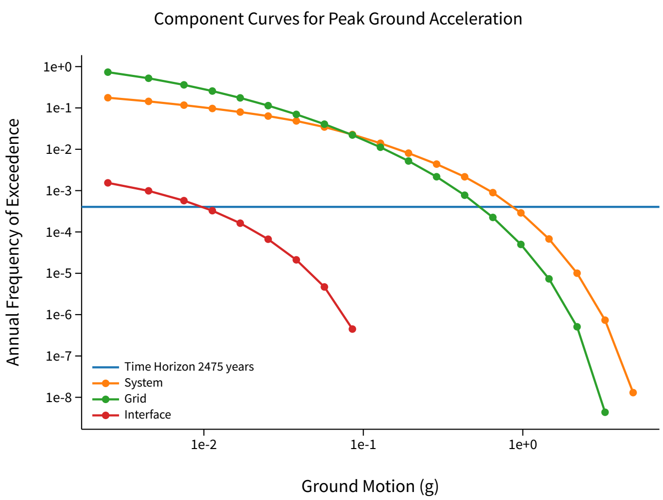
Return period in years

2475

Hazard Curve





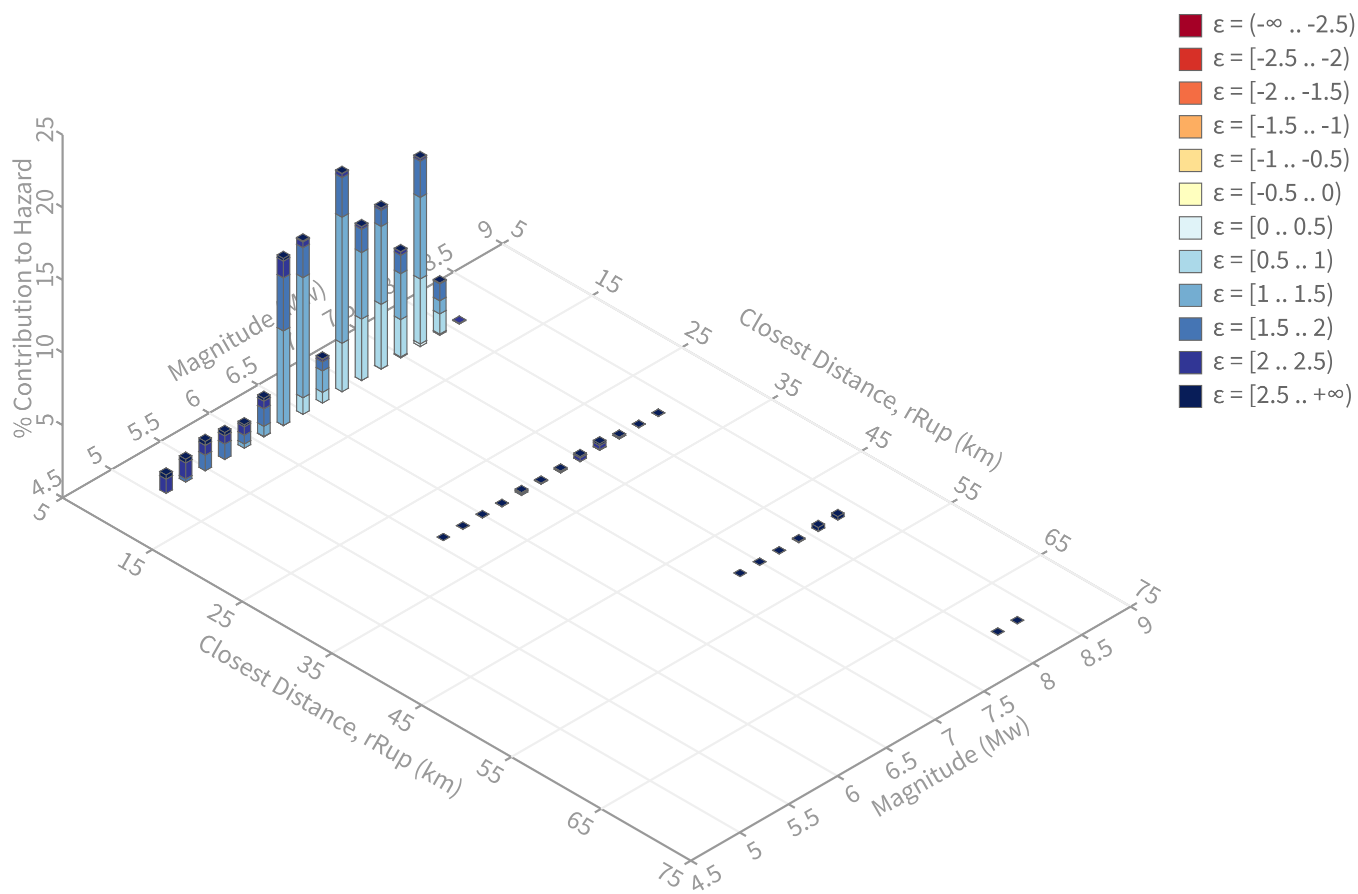


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.91681803 g

Recovered targets

Return period: 2891.2721 yrs
Exceedance rate: 0.00034586852 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 6.91
r: 9.54 km
ε_0: 1.37 σ

Mode (largest m-r bin)

m: 6.91
r: 8.16 km
ε_0: 1.23 σ
Contribution: 15.04 %

Mode (largest m-r-ε_0 bin)

m: 6.91
r: 8.39 km
ε_0: 1.21 σ
Contribution: 8.68 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							43.57
Elysian Park (Upper) [0]		5.49	6.48	1.36	118.127°W	34.070°N	340.29	12.63
Elysian Park (Upper) [1]		7.46	7.15	1.18	118.169°W	34.072°N	305.53	5.80
Puente Hills [3]		8.53	7.12	0.93	118.167°W	33.989°N	220.95	4.88
Compton [1]		15.95	7.22	1.15	118.257°W	33.805°N	206.76	4.22
Whittier alt 1 [7]		7.89	6.88	1.61	118.046°W	33.999°N	126.31	4.03
Raymond [1]		9.28	7.28	1.56	118.132°W	34.121°N	349.54	3.21
Compton [2]		15.95	7.51	1.17	118.286°W	33.817°N	212.59	2.09
Puente Hills [2]		8.63	7.32	0.93	118.143°W	33.972°N	199.31	1.49
UC33brAvg_FM32	System							42.19
Elysian Park (Upper) [0]		5.49	7.06	1.11	118.127°W	34.070°N	340.29	11.33
Puente Hills (Santa Fe Springs) [1]		9.34	6.94	0.98	118.093°W	33.934°N	170.63	6.01
Compton [1]		15.95	7.29	1.12	118.257°W	33.805°N	206.76	4.56
Whittier alt 2 [6]		7.93	7.11	1.31	118.046°W	33.998°N	126.83	3.87
Puente Hills (LA) [0]		5.69	7.17	0.74	118.116°W	33.990°N	182.24	3.19
Raymond [1]		9.28	7.28	1.56	118.132°W	34.121°N	349.54	2.94
Elysian Park (Upper) [1]		7.46	7.14	1.38	118.169°W	34.072°N	305.53	2.89
Puente Hills (Coyote Hills) [1]		11.47	7.28	1.34	118.044°W	33.915°N	155.27	1.93
Compton [2]		15.95	7.48	1.18	118.286°W	33.817°N	212.59	1.81
UC33brAvg_FM32 (opt)	Grid							7.35
PointSourceFinite: -118.114, 34.081		6.73	5.71	1.76	118.114°W	34.081°N	0.00	1.47
PointSourceFinite: -118.114, 34.081		6.73	5.71	1.76	118.114°W	34.081°N	0.00	1.47
UC33brAvg_FM31 (opt)	Grid							6.89
PointSourceFinite: -118.114, 34.081		6.76	5.69	1.77	118.114°W	34.081°N	0.00	1.51
PointSourceFinite: -118.114, 34.081		6.76	5.69	1.77	118.114°W	34.081°N	0.00	1.51