GEOTECHNICAL INVESTIGATION

PROPOSED CHAMINADE HIGH SCHOOL IMPROVEMENTS
7500 CHAMINADE AVENUE, 23241 COHASSET STREET,
23217-23260 SATICOY STREET
7619-7629 WOODLAKE AVENUE
WEST HILLS NEIGHBORHOOD OF LOS ANGELES, CALIFORNIA

TRACT: 2607; LOT: LT1;
TRACT: 2500; LOT: 19;
TRACT: 25733; LOT: LT1; ARB: 1-4

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

CAHMINADE COLLEGE PREPARATORY
WEST HILLS, CALIFORNIA

PROJECT NO. W1547-06-01

REVISED JANUARY 12, 2023

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1547-06-01
Revised January 12, 2023

Chaminade College Preparatory
7500 Chaminade Avenue
West Hills, CA 91304

Attention: Mr. Chris Landon

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED CHAMINADE HIGH SCHOOL IMPROVEMENTS
7500 CHAMINADE AVENUE, 23241 COHASSET STREET,
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TRACT: 2607; LOT: LT1;
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Dear Mr. Landon:

In accordance with your authorization of our proposal dated March 16, 2022, we have performed a geotechnical investigation for the proposed Chaminade High School improvements located at 7500 Chaminade Avenue, 23241 Cohasset Street, 23217-23260 Saticoy Street, 7619-7629 Woodlake Avenue in the City of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

Joe Hicks, M.S.
PE 93183

Susan F. Kirkgard
CEG 1754

Neal Berliner
GE 2576

(EMAIL) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for proposed Chaminade High School improvements located at 7500 Chaminade Avenue, 23241 Cohasset Street, 23217-23260 Saticoy Street, 7619 and 7629 Woodlake Avenue in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on April 18, 2022, and April 19, 2022, by excavating twelve 8-inch diameter borings to depths ranging between approximately 6½ to 51 feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figures 2A through 2D). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 7500 Chaminade Avenue, 23241 Cohasset Street, 23217-23260 Saticoy Street, 7619 and 7629 Woodlake Avenue in the City of Los Angeles, California. The subject property consists of the Main Campus at 7500 Chaminade Avenue, 23241 Cohasset Street, and 2360 Saticoy Street (see Figure 2B), and the North Campus located at 23217-23255 Saticoy Street and 7619-7629 Woodlake Avenue (see Figure 2C).

The Main Campus is currently occupied by the existing Chaminade High School campus located at the northeast corner of the intersection of Chaminade Avenue and Cohasset Street. The Main Campus includes several one- and multi-level classroom, administrative, and ancillary buildings, as well as a sports stadium and athletic fields. The Main Campus is bounded by Keswick Street to the north, by Chaminade Avenue to the west, by Cohasset Street to the south, and by multiple single-story residential structures to the east. The main campus slopes to the south with approximately 42 feet of vertical relief across the site.

The North Campus, north and east of Saticoy Street, is relatively level with approximately 4 feet of vertical relief (see Figure 2C). The North Campus is currently occupied by several single-story commercial structures surrounded by on grade asphalt parking. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets.

It is our understanding that the proposed improvements will include :

- Constructing a pedestrian bridge over Saticoy Street connecting the Main and North Campus
- North Campus:
    - Demolish the existing commercial structures and remove the existing paving
    - Construct a new pool and pool house
    - Construct new sports fields with associated ancillary structures
    - Construct an on-grade parking area
- Main Campus:
    - Demolish 6 of the existing administrative and classroom structures (including 2 portable buildings)
    - Construct a new Main three-story administrative/classroom structure
    - Renovate existing surface parking areas and sports fields.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 300 kips, and wall loads will be up to 4 kips per linear foot.

At this time the proposed building elevations have not yet been finalized. The existing and anticipated proposed site conditions are depicted on the Site Plans and Cross Sections (see Figures 2A through 2E). Once the design phase and foundation configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. PRIOR INVESTIGATION REPORTS

We were provided with a copy of prior geotechnical investigation reports for the site prepared by Geotechnologies, Inc. that include the following:

> *Geotechnical Engineering Investigation, Proposed Performing Arts Center, 7500 Chaminade Avenue, West Hills, California, File No. 19793, dated June 1, 2009, by Geotechnologies, Inc.*
>
> *Stormwater Disposal, Performing Arts Center, Chaminade High School, 7500 Chaminade Avenue, West Hills, California, File No. 19793, dated February 24, 2010, by Geotechnologies, Inc.*
>
> *Response to Geology and Soils Report Correction Letter, Proposed Performing Arts Center, Chaminade High School, 7500 Chaminade Avenue, West Hills, California, File No. 19793, dated May 5, 2010, by Geotechnologies, Inc.*

In 2009, Geotechnologies, Inc. excavated five hollow stem auger borings and three test pits on the west side of the Main Campus for the design of a proposed auditorium/performing arts center. Also, as part of the same investigation, three additional large-diameter borings were excavated in 2010, two in the area west of the existing football stadium, in the parking lot for a proposed infiltration area (borings B6 and B7) and one in the area of the deepest portion of the excavation for the performing arts center (boring B8). The borings ranged from 20 to 40 feet beneath the existing ground surface and the test pits ranged from 5½ to 7½ feet beneath the existing ground surface. The geologic materials encountered in the borings consist of artificial fill, alluvium, and sedimentary bedrock of the Monterey Formation. The depth to bedrock in the explorations ranged from 1 to 16 feet and the bedrock consists of interbedded siltstone, diatomaceous siltstone, siliceous siltstone and sandstone. Boring B8 was downhole logged by a geologist. Bedding in the Monterey Formation bedrock was observed in the borings and test pits to strike N20W to N70E and dip 10 degrees northeast and 12 to 30 degrees southeast. Static groundwater was not encountered in the prior borings; however, water seepage was encountered in boring B7 within the bedrock at a depth of approximately 18 feet below the ground surface. Residual shear tests were performed on bedrock samples collected in boring B8.

Geocon West, Inc. has reviewed the referenced report by Geotechnologies Inc. (2009, 2010). Where applicable, the recommendations presented herein consider the subsurface and laboratory data included in the prior report, as well as our own subsurface and laboratory data. Furthermore, we assume responsibility for the utilization of the exploration and laboratory data presented within the prior geotechnical report. Geocon West, Inc. is the Geotechnical Consultant of Record and will be providing all necessary geotechnical consultation, plan review, design recommendations, inspection and testing services for this project. Where differing, the recommendations presented herein supersede all previous recommendations. The previous investigation reports are presented in Appendix C.

4. GEOLOGIC SETTING

The site is located in the western portion of the San Fernando Valley, an alluvial-filled basin approximately 23 miles wide and 12 miles long (Hitchcock and Wills, 2000). The alluvium within the San Fernando Valley is derived primarily from the Santa Monica Mountains to the south, the Santa Susana Mountains to the north, the Simi Hills to the west, the San Gabriel Mountains to the northeast, and the Verdugo Mountains to the east. The site is located on the valley floor and the surficial alluvial sediments underlying the site were derived primarily from local drainages originating in the Santa Susana Mountains to the north (Hitchcock and Wills, 2000). Locally, isolated bedrock outcrops are present in the site vicinity and shallow bedrock underlies the northern portion of the main campus.

5. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill, Holocene age alluvium and colluvium, and the northern portion of the main campus is underlain by shallow sedimentary bedrock of the Monterey Formation, also called the Modelo Formation (Dibblee, 1992). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

5.1 Artificial Fill

As encountered in the borings, the artificial fill at the site is typically less than 3 feet thick except in the northeastern portion of the Main Campus (B6) and the eastern portion of the North Campus (B2) where fill was encountered to depths of 6 feet in boring B6 and 5 feet in boring B2. The artificial fill generally consists of brown to dark brown or grayish brown sandy clay, sandy silt, and silty sand with varying amounts of gravel. The fill is characterized as dry to moist and soft to hard or loose to medium dense. Artificial fill was encountered in the prior Geotechnologies borings at the site to a maximum depth of 7 feet beneath the ground surface in boring B3. The fill consists of dark brown to yellowish brown sandy and clayey silt, silty clay, and silty sand and is characterized moist and stiff or medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

5.2 Alluvium

The artificial fill is locally underlain by Holocene age alluvium in borings B1 through B4 on the North Campus and in borings B5 through B7 along the east side of the Main Campus and in boring B12 in the southern portion of the Main Campus. The alluvium consists primarily of brown to dark brown, olive brown or dark gray interbedded clay, sandy clay, sandy silt, silty sand, clayey sand, and poorly graded sand with various amounts of gravel. The alluvium is characterized as fine to medium-grained, dry to saturated, and very loose to dense or soft to hard. Alluvium was encountered beneath the fill in the prior Geotechnologies borings B6 and B7, in southern portion of the Main Campus. As logged in the prior borings, the alluvium consists of yellowish brown to dark brown silty sand to sandy silt or clayey silt to silty clay and is characterized moist and stiff or medium dense.

5.3 Colluvium

Colluvium was encountered beneath the artificial fill in borings B10 and B11, south of the existing performing arts center. The colluvium ranges from 2 to 3 feet thick and consists of olive brown to dark brown sandy clay and sandy silt with various amounts of bedrock fragments. The colluvium is characterized as slightly moist to moist, and stiff to hard.

5.4 Monterey Formation

Sedimentary bedrock of the Miocene age Monterey Formation (also known as the Modelo Formation) directly underlies the artificial fill in borings B8 and B9 (adjacent to the proposed administration building in the central portion of the Main Campus) and was encountered below the surficial soils (alluvium and colluvium) in the remainder of the borings. The bedrock at the site consists of interbedded sandstone and siltstone with some localized diatomaceous siltstone and siliceous siltstone beds. The bedrock is characterized as poorly bedded to well-bedded, soft to hard, and slightly to highly weathered with various amounts of oxidation staining. Bedrock was encountered in all borings, including the prior borings by Geotechnologies Inc., except in the current borings B2 and B12. Boring B2 is located on the east side of the North Campus in the vicinity of the proposed pool house and boring B12 is located on the southern portion of the Main Campus. Based on the prior explorations at the site, bedding in the Monterey Formation bedrock was observed in the prior borings and test pits to strike N20W to N70E and dip 10 degrees northeast and 12 to 30 degrees southeast (Geotechnologies, 2009; 2010).

6. GROUNDWATER

Based on a review of the California Division of Mines and Geology (CDMG) Seismic Hazard Evaluation of the Calabasas 7.5-Minute Quadrangle (CDMG, 1997; revised 2001), the historic high groundwater level beneath the site is greater than 10 feet below the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered during site exploration in borings B1 and B12 at depths of approximately 32 feet and 14½ feet beneath the existing ground surface, respectively. However, static groundwater was not encountered in the prior borings by Geotechnologies Inc. (2009, 2010), drilled to a maximum depth of 40 feet below ground surface. Groundwater seepage was encountered within the bedrock at a depth of approximately 18 feet in boring B7 (Geotechnologies, 2010). Based on the reported historic high groundwater levels in the site vicinity (CDMG, 1997; revised 2001), the depth to groundwater encountered in our borings, and the depth of proposed construction, static groundwater is not expected to be encountered during construction or to have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially along the top of the bedrock contact and within impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 8.25).

7. GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

The numerous faults in Southern California include Holocene-active, pre-Holocene, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018a). By definition, a Holocene-active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A pre-Holocene fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2022b; 2018b) for surface fault rupture hazards. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest active fault to the site is the Simi-Santa Rosa Fault Zone, located approximately 7.4 miles to the northwest (USGS, 2006). Other nearby active faults are the Santa Susana Fault, the San Fernando segment of the Sierra Madre Fault Zone, an unnamed fault in North Hollywood, the San Gabriel Fault Zone, and the Verdugo Fault located approximately 8.2 miles north-northeast, 9.7 miles northeast, 13 miles east-southeast, 15 miles north, and 18 miles east of the site, respectively. (Ziony and Jones, 1989; USGS, 2006). The active San Andreas Fault Zone is located approximately 34 miles northeast of the site (USGS, 2006).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Southern California area at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987, M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Southern California area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

7.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	80	E
Long Beach	March 10, 1933	6.4	56	SE
Tehachapi	July 21, 1952	7.5	59	NNW
San Fernando	February 9, 1971	6.6	19	NE
Whittier Narrows	October 1, 1987	5.9	33	ESE
Sierra Madre	June 28, 1991	5.8	36	E
Landers	June 28, 1992	7.3	125	E
Big Bear	June 28, 1992	6.4	103	E
Northridge	January 17, 1994	6.7	6	E
Hector Mine	October 16, 1999	7.1	137	E
Ridgecrest	July 5, 2019	7.1	122	NE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

7.3 Seismic Design Criteria

The following table summarizes the site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *U.S. Seismic Design Maps*, provided by the Structural Engineers Association of California (SEAOC). The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.5g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.6g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.2	Table 1613.2.3(1)
Site Coefficient, F_V	1.7	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.8g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.02g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.2g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.68g	Section 1613.2.4 (Eqn 16-39)

The table on the following page presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.538g	Figure 22-9
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.646g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2022 California Building Code and ASCE 7-16, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.66 magnitude event occurring at a hypocentral distance of 11.17 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.58 magnitude occurring at a hypocentral distance of 13.67 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.4 Liquefaction Potential - General

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

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

The State of California Seismic Hazard Zone Map for the Calabasas Quadrangle (CDMG, 1998; CGS, 2022b) indicates that the southern portion of the Main Campus and the entire North Campus are located in an area designated as having a potential for liquefaction. Consequently, the proposed pool, pool house and pedestrian bridge are located within areas designated as having a potential for liquefaction. Groundwater was encountered during site exploration in borings B1 (North Campus) and B12 (southern portion of the Main Campus) at depths of approximately 32 feet and 14½ feet beneath the existing ground surface, respectively.

7.4.1 North Campus Liquefaction Potential

The North Campus is underlain by artificial fill, potentially liquefiable alluvium, further underlain by Monterey formation bedrock. The depth of the bedrock ranges from 12 feet on the west portion of the property, to 42 feet on the east side of the property.

It is anticipated that the proposed pedestrian bridge structure can be founded on relatively shallow bedrock using deepened foundations (piles) extending through the potentially liquefiable materials. Since structures will be supported on bedrock, it is our opinion that the potential for liquefaction and associated ground deformations to impact structures is considered very low.

Due to the depth of bedrock on the east side of the North Campus, deepened foundations are not considered economically feasible for the construction of the North Campus pool house improvements. Therefore, a liquefaction analysis was performed to determine the feasibility of a shallow foundation system.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

Screening criteria developed by Bray and Sancio (2006) characterize fine-grained soils which are not susceptible to liquefaction as soils with a plasticity index (PI) that is greater than 18 or with a saturated moisture content that is less than 80 percent of the liquid limit. In order to apply the screening criteria, laboratory testing was performed to evaluate the Atterberg Limits and saturated moisture content of select soil samples. Laboratory test results used for the screening criteria are presented as Figures B16 and B17.

The liquefaction analysis was performed for a Design Earthquake level by using a high groundwater table of 10 feet below the ground surface, a magnitude 6.58 earthquake, and a peak horizontal acceleration of 0.431g (⅔PGA_M). The enclosed liquefaction analyses included herein for borings B1 indicate that the alluvial soils below the proposed foundation level would be prone to approximately 2.2 inches of liquefaction settlement during a Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 and 6). The resulting differential settlement at the foundation level is anticipated to be approximately 1½ inches over a distance of 20 feet.

It is our understanding that the intent of the Building Code is to maintain “Life Safety” during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for Maximum Considered Earthquake levels by using a historic high groundwater table of 10 feet below the ground surface, a magnitude 6.66 earthquake, and a peak horizontal acceleration of 0.646g (PGA_M). The enclosed liquefaction analysis, included herein for boring B1 indicates that the alluvial soils below the proposed foundation would be prone to approximately 2½ inches of liquefaction settlement during a Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 7 and 8). The resulting differential settlement at the foundation level is anticipated to be approximately 1¼ inches over a distance of 20 feet.

7.4.2 Main Campus Liquefaction Potential

Based on the anticipated configurations of the proposed improvements, all proposed foundations will be underlain by Monterey formation bedrock. It is recommended that building foundations be founded on bedrock. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations to impact the proposed structure is considered very low.

Once the configuration of the proposed improvements are finalized, additional analysis should be performed as necessary to address liquefaction potential.

7.5 Slope Stability

The topography at the site slopes to the south with 42 feet of elevation difference, and the topography in the immediate site vicinity slopes gently to the southeast. The site is located within a City of Los Angeles Hillside Grading Area but is not located within a city-designated Hillside Ordinance Area (City of Los Angeles, 2022). The County of Los Angeles Safety Element (Leighton, 1990) indicates that the site is within a hillside area. According to the California Geological Survey (CDMG, 1998), the site is not located within an area identified as having a potential for seismic slope instability. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Based on these considerations, the potential for slope stability hazards to impact the proposed development is considered low.

The planned excavation for the proposed administration building will expose bedrock in the majority of the excavation (see Figure 2E). As previously indicated, bedding in the Monterey Formation bedrock was observed in the prior borings and test pits to strike N20W to N70E and dip 10 degrees northeast and 12 to 30 degrees southeast (Geotechnologies, 2009; 2010). If this bedding orientation is consistent across the site, day-lighted (adverse) bedding will be exposed along the west (east- and southeast-facing) and north (south-facing) excavation walls for the proposed administration building. However, due to the limited bedding information collected at the site and the inconsistency of the observed bedding orientations, the bedding orientation at the site should be verified during excavation.

7.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the County of Los Angeles Safety Element (Leighton, 1990), the site is not located within a potential inundation area for an earthquake-induced dam failure. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

7.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis, are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding from a seismically induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2022; LACDPW, 2022).

7.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and there are no oil or gas wells documented at the site or within ½-mile of the site (CalGEM, 2022). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the CalGEM.

The site is not located within a City of Los Angeles Methane Zone or Methane Buffer Zone (City of Los Angeles, 2022). Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

7.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The majority of the site is underlain by shallow bedrock that is not susceptible to subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

8.1.2 Up to 6 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Demolition of the existing structures that occupy the site is anticipated to disturb the upper few feet of existing site soils. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 8.4).

8.1.3 The proposed North Campus pool house (see Figure 2C) is located within a liquefaction hazard zone and underlain by potentially liquefiable soils further underlain by Monterey Formation Bedrock at a depth of 42 feet (see B1). The liquefaction analysis included herein indicates that the alluvial soils below the historic high groundwater level could be prone to approximately 2.2 inches of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). The resulting differential settlement at the ground surface is anticipated to be approximately 1½ inches over a distance of 20 feet. Based on these considerations it is recommended that a reinforced concrete mat foundation system deriving support in newly placed engineered fill may be utilized for support of the proposed North Campus pool house.

8.1.4 The proposed pedestrian bridge is underlain by artificial fill and moderately compressible alluvial soils further underlain by bedrock at a depth between approximately 12 and 15 feet below the existing ground surface based on current boring data (see Borings B3 and B4). It is recommended that a deepened foundation system (piles) deriving support in bedrock be utilized to support the proposed pedestrian bridge.

8.1.5 The proposed Main Campus administration building will be directly underlain by exposed Monterey Formation Bedrock. The bedrock at this location is considered suitable for direct support of the proposed structure. Based on these considerations a conventional spread foundation system and slab-on-grade is considered suitable for the proposed structure.

8.1.6 The grading and foundation recommendations presented herein are intended to minimize the effects of settlement on proposed improvements.

8.1.7 A Summary of the recommended foundation systems for the proposed structures is provided in the table below.

Recommended Foundation Systems

Campus Location	Structure Name	Recommended Foundation Type	Recommended Slab	Recommended Bearing Material
Main Campus	New Administration Building	Conventional Spread Foundations[1]	Conventional Slab-on-Grade	Bedrock
North/Main Campus	Bridge	Deepened Foundations (Piles)[2]	NA	Bedrock
North Campus	Pool House	Mat Foundation[3]	Mat Foundation[3]	Engineered Fill

1. See Section 8.7 and 8.9 for Conventional Foundation Recommendations
2. See Sections 8.10 through 8.12 for Deepened Foundations Recommendations
3. See Section 8.8 and 8.9 for Mat Foundation Recommendations

8.1.8 Groundwater was encountered during site exploration in borings B1 and B12 at depths of approximately 32 feet and 14½ feet beneath the existing ground surface, respectively. Previously, groundwater seepage was encountered within the bedrock at a depth of approximately 18 feet in boring B7 (Geotechnologies, 2010). It is anticipated that the current static groundwater table is sufficiently deep that it will not be encountered during shallow construction excavations, with the exception of a deep drilled excavation such as piles or elevator pistons. However, local seepage could develop and should be expected on top of the bedrock or within the joints and fractures in the bedrock, especially if conducted during the rainy season.

8.1.9 The bedrock encountered in our investigation is slightly to highly weathered and should be rippable with conventional equipment; however, concretions or well cemented layers may be encountered in the bedrock which could make excavation or drilling conditions difficult. Coring or jack-hammering may be required if concretions are encountered and the contractor should be prepared for these conditions.

8.1.10 Excavations on the order of 14 feet in vertical height are anticipated for construction of the proposed administration structure, including foundation depths. Based on the prior explorations at the site, bedding in the Monterey Formation bedrock was observed in the prior borings and test pits to strike N20W to N70E and dip 10 degrees northeast and 12 to 30 degrees southeast (Geotechnologies, 2009; 2010). Unfavorably oriented (adverse) bedding is anticipated to be exposed along the west (east- and southeast-facing) and north (south-facing) excavation walls for the proposed administration building. As shown on the cross sections, excavations for proposed retaining walls along the north and east sides of the property are anticipated to expose bedrock orientations that are favorable with respect to stability of the excavation.

8.1.11 Due to the depth of the excavations and the proximity to the property lines, city streets, and adjacent on-site structures, excavation for the proposed retaining walls will require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required, it is recommended that a soldier pile shoring system be utilized. Where excavation depths exceed 12 feet or surcharges are imposed on the shoring system, raker braces or tie-back anchors may be required in conjunction with the soldier piles. The need for lateral bracing and the acceptable shoring deflection should be determined by the project shoring engineer.

8.1.12 The recommendations for shoring and retaining wall design presented herein incorporate the anticipated surcharge loads generated by out-of-slope bedding.

8.1.13 Excavations must be conducted in a manner that maintains stability and must be observed and approved by the Project Geologist (a representative of Geocon West, Inc.). A geologist should periodically observe the excavation to confirm that the orientation of the exposed bedrock is consistent with the conditions observed during our investigation.

8.1.14 Due to the nature of the proposed design and intent for a partial subterranean level, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

8.1.15 Where new paving is to be placed, it is recommended that all existing fill soils and soft soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required, however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 8.15).

8.1.16 Based on the results of the percolation testing performed at the site, a stormwater infiltration system is not considered feasible for this project. A discussion of the test results is provided in the *Stormwater Infiltration* section of this report (see Section 8.23).

8.1.17 Once the design and foundation loading configurations for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be reevaluated by this office.

8.1.18 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially if granular fill soils are encountered. The surficial bedrock is moderately to highly weathered and should be rippable with conventional equipment; however, concretions or well cemented layers may be encountered in the bedrock which could make excavation or drilling conditions difficult. Coring or jack-hammering may be required if concretions are encountered and the contractor should be prepared for these conditions.

8.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

8.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.21).

8.2.4 The existing site soils encountered during the field investigation near the ground surface are considered to have a “low” to “medium” (EI = 21, 57 and 63) expansive potential and are classified as “expansive” in accordance with the 2022 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the proposed foundations and slabs will derive support in the “expansive” soils.

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils encountered at foundation depths are considered “moderately corrosive” to “corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figures B26 and B27) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approve plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

8.3.2 Laboratory tests were performed on representative samples of the site soils to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figures B26 and B27) and indicate that the on-site materials possess “S0” sulfate exposure to concrete structures as defined by 2022 CBC Section 1904 and ACI 318 Table 19.3.1.1.

8.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

8.4 Grading

8.4.1 Grading is anticipated to include excavation of site soils for foundations, foundations of small outlying structures, and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

8.4.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.

8.4.3 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvium encountered during exploration are suitable for re-use as engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.

8.4.4 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer; in accordance with City policy, concrete and asphalt is not permitted to be mixed into structural fill. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.) and the City of Los Angeles Inspector.

8.4.5 At a minimum, it is recommended that the upper 5 feet of existing earth materials within the proposed North Campus pool house footprint area be excavated and properly compacted for support of the reinforced concrete mat foundation system. Deeper excavations should be conducted as necessary to remove any existing deeper artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

8.4.6 It is recommended that foundations for the proposed Main Campus administrative building and pedestrian bridge penetrate through the existing artificial fill and alluvial soils in order to derive support exclusively in the underlying competent bedrock. All excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.) prior to the placement of engineered fill. If the existing artificial fills and site soils are not properly compacted for slab support, then a structural slab, that derives all support from the pile foundation system, will be required.

8.4.7 It is anticipated that bedrock will be exposed throughout the Main Campus administration building footprint area at the excavation bottom, and the bedrock is considered suitable for support of the foundations and concrete slab-on-grade. Any bedrock that is unintentionally disturbed should be compacted for slab support. At the ground floor level, it is recommended that all existing artificial fill and/or soft colluvium be excavated and properly compacted for slab support. The limits of existing fill and/or soft colluvium removal will be confirmed by the Geocon representative during site grading activities.

8.4.8 All imported fill shall be observed, tested and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. Imported soils should have an expansion index less than 50 and soils corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B26 and B27).

8.4.9 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition). Soils should be moisture conditioned to two percent above optimum moisture content.

8.4.10 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to at least 2 percent above optimum moisture content, and compacted to at least 92 percent relative compaction for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 8.15).

8.4.11 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed to the main structures, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, foundations may derive support directly in the competent undisturbed alluvium, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

8.4.12 Utility trenches should be properly backfilled in accordance with the following requirements. The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. If gravel is used for trench bedding and shading (typical when seepage is present) it must be 3/16-inch rounded birds-eye rock in accordance with the City of LA plumbing department requirements. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill (see Section 8.5). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

8.4.13 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

8.5 Controlled Low Strength Material (CLSM)

8.5.1 Controlled Low Strength Material (CLSM) may be utilized in lieu of compacted soil as engineered fill where approved in writing by the Geotechnical Engineer. Where utilized within the City of Los Angeles use of CLSM is subject to the following requirements:

Standard Requirements

1. CLSM shall be ready-mixed by a City of Los Angeles approved batch plant;
2. CLSM shall not be placed on uncertified fill, on incompetent natural soil, nor below water;
3. CLSM shall not be placed on a sloping surface with a gradient steeper than 5:1 (horizontal to vertical);
4. Placement of the CLSM shall be under the continuous inspection of a concrete deputy inspector;
5. The excavation bottom shall be accepted by the soil engineer and the City Inspector prior to placing CLSM.

Requirements for CLSM that will be used for support of footings

1. The cement content of the CLSM shall not be less than 188 pounds per cubic yard (min. 2 sacks);
2. The excavation bottom must be level, cleaned of loose soils and approved in writing by Geocon prior to placement of the CLSM;
3. The ultimate compressive strength of the CLSM shall be no less than 100 psi when tested on the 28th day per ASTM D4832 (latest edition), Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders. Compression testing will be performed in accordance with ASTM C39 and City of Los Angeles requirements;
4. Samples of the CLSM will be collected during placement, a minimum of one test (two cylinders) for each 50 cubic yards or fraction thereof;
5. Overexcavation for CLSM placement shall extend laterally beyond the footprint of any proposed footings as required for placement of compacted fill, unless justified otherwise by the soil engineer that footings will have adequate vertical and horizontal bearing capacity.

8.6 Foundation Design – General

8.6.1 A Summary of the recommended foundation systems for the proposed structures is provided in the table on the following page.

Recommended Foundation Systems

Campus Location	Structure Name	Recommended Foundation Type	Recommended Slab	Recommended Bearing Material
Main Campus	New Administration Building	Conventional Spread Foundations[1]	Conventional Slab-on-Grade	Bedrock
North/Main Campus	Bridge	Deepened Foundations (Piles)[2]	NA	Bedrock
North Campus	Pool House	Mat Foundation[3]	Mat Foundation[3]	Engineered Fill

1. See Section 8.7 and 8.9 for Conventional Foundation Recommendations
2. See Sections 8.10 through 8.12 for Deepened Foundations Recommendations
3. See Section 8.8 and 8.9 for Mat Foundation Recommendations

8.6.2 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.6.3 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.6.4 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

8.7 Conventional Foundation Design

8.7.1 Where a conventional foundation system is utilized, foundations should derive support in the competent bedrock. Foundations should penetrate through the existing artificial fill, alluvium or colluvium and any soft or highly weathered bedrock, in order to derive support exclusively in competent bedrock. Foundation's excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.7.2 Continuous footings may be designed for an allowable bearing capacity of 4,500 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

8.7.3 Isolated spread foundations may be designed for an allowable bearing capacity of 5,000 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

8.7.4 The allowable soil bearing pressure above may be increased by 300 psf and 700 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 7,000 psf.

8.7.5 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

8.7.6 If depth increases are utilized for the perimeter foundations, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.7.7 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

8.7.8 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

8.7.9 Provided the foundation excavations and the concrete slab-on-grade subgrade exposes undisturbed bedrock, no special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement. Where engineered fill is placed as subgrade soils, due to the expansive nature of the soils, the moisture content in the slab and foundation subgrade should be maintained at 2 percent above optimum moisture content prior to and at the time of concrete placement.

8.7.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavation and exposed soil conditions are consistent with those anticipated. If unanticipated conditions are encountered, foundation modifications may be required.

8.7.11 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.8 Mat Foundation Design

8.8.1 Once the recommended grading has been completed, the proposed pool house structure may be supported by a reinforced mat foundation system deriving support in newly placed engineered fill. The mat foundation system allows for more efficient construction and is better at distributing loads to minimize the effect of potential settlement.

8.8.2 The recommended allowable bearing pressure is 3,000 pounds per square foot (psf). The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.8.3 It is recommended that a modulus of subgrade reaction of 100 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in the undisturbed bedrock. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$\mathrm{K_R} = \mathrm{K}\left[\frac{\mathrm{B}+1}{2\mathrm{B}}\right]^2$$

where:
$\mathrm{K_R}$ = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

8.8.4 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

8.8.5 For seismic design purposes, a coefficient of friction of 0.40 may be utilized between the concrete mat and bedrock without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

8.8.6 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.8.7 Waterproofing of the building slab is suggested for this project. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer.

8.8.8 This office should be provided a copy of the final construction plans so that the recommendations presented herein could be properly reviewed and revised if necessary.

8.9 Foundation Settlement

8.9.1 The enclosed liquefaction settlement analyses indicate that the alluvial soils underlying the North Campus could be susceptible to approximately 2.2 inches of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). The differential settlement at the foundation level is anticipated to be less than 1½ inches over a distance of 20 feet. These settlements are in addition to the static settlements indicated below and must be considered in the structural design.

8.9.2 Where conventional foundations will be utilized, the footings will be underlain by bedrock which is not prone to liquefaction. Therefore, settlement will be limited to static settlement resulting from the building load. The maximum anticipated static settlement for a conventional foundation system designed with the maximum allowable bearing value of 7,000 psf and deriving support in undisturbed bedrock is expected to be less than ½ inch and occur below the heaviest loaded structural element. Differential settlement is not expected to exceed ¼ inch over a distance of 20 feet or between adjacent bedrock-supported foundations.

8.9.3 The North Campus pool house may be supported by a mat foundation system deriving support in newly placed engineered fill. The maximum static settlement for a reinforced mat foundation system deriving support in the newly placed engineered fill with a maximum allowable bearing value of 3,000 psf is expected to be less than 1 inch and occur below the heaviest loaded structural element. Static differential settlement over a distance of 20 feet, is expected to be less than ½ inch. Based on seismic considerations, the proposed North Campus pool house structure should be designed for a combined static and seismically induced differential settlement of 2.0 inches over a distance of 20 feet.

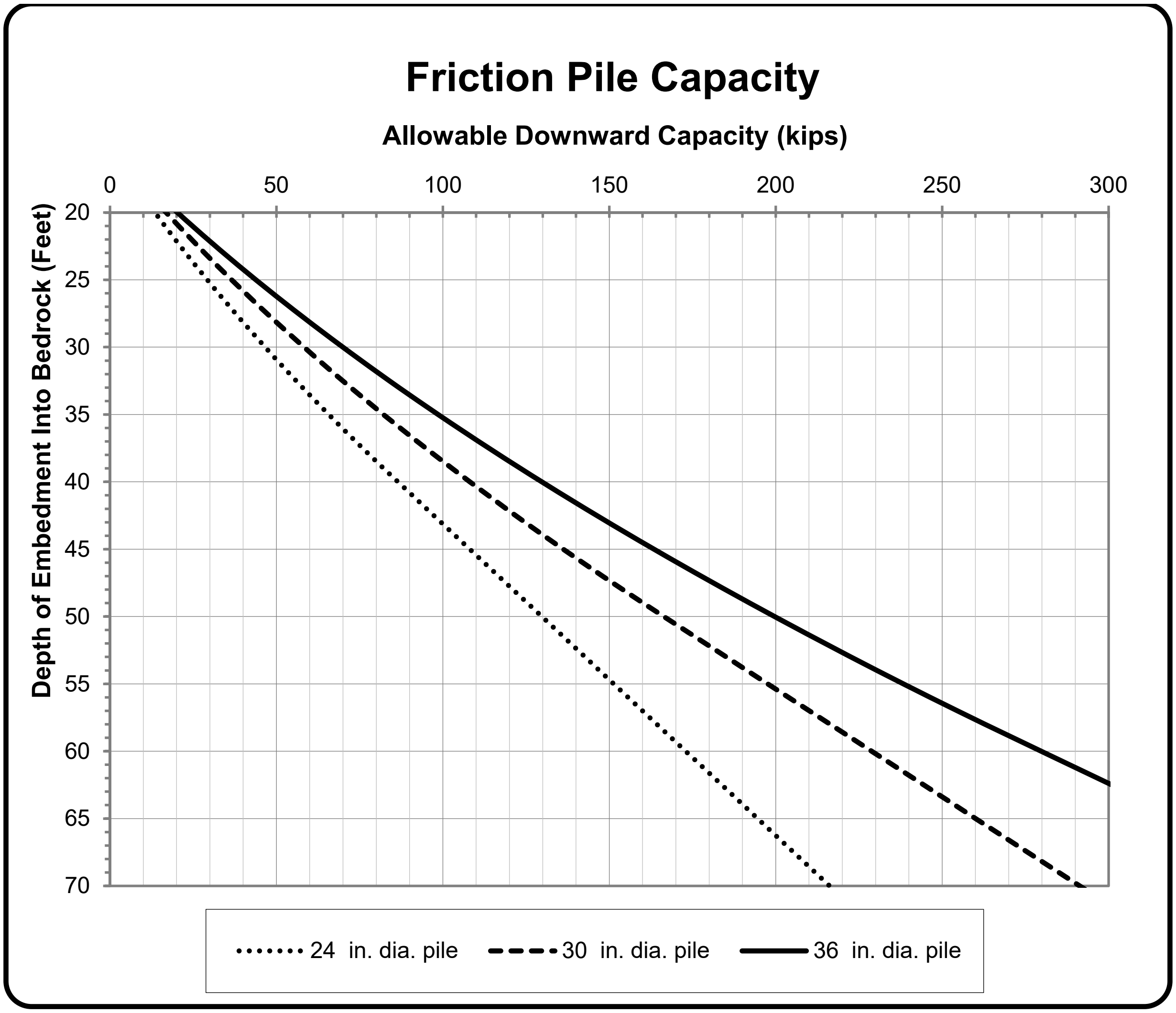
8.9.4 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

8.10 Friction Pile Design

8.10.1 For preliminary design purposes 24-, 30-, and 36-inch diameter drilled cast-in-place friction piles have been evaluated. Friction piles should be embedded a minimum of 20 feet into bedrock. The allowable axial capacities for pile embedment into the competent alluvial soils are provided in the charts below. The axial capacities are based on skin friction; end-bearing capacity is not being considered. The axial capacities also include consideration of down drag forces due to consolidation of the overlying compressible soils as well as down drag from liquefiable soils. An average down drag load of 16 kips, 20 kips, and 24 kips was applied for 24-, 30-, and 36-inch diameter piles respectively.

8.10.2 Once the proposed building configuration proceeds to a more finalized stage, the estimated Downdrag forces should be revised.

8.10.3 Friction piles supporting the proposed on-grade structure at may use the capacities presented in the chart below.



8.10.4 All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the ultimate strength of the bedrock. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

8.10.5 Single pile uplift capacity can be taken as 60 percent of the allowable downward capacity.

8.10.6 The allowable downward capacity and allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

8.10.7 The maximum expected static settlement for the structure supported on friction piles is estimated to be less than ½ inch. Differential settlement between adjacent pile foundations is not expected to exceed ½ inch. The majority of the foundation settlement is expected to occur on initial application of loading and during construction.

8.10.8 For increased resistance to differential foundation movement and lateral drift, the pile tops should be interconnected in two horizontal directions with grade beams or tied with a structural slab. The project structural engineer should provide slab and grade beam design, reinforcement and spacing dependent on anticipated loading. However, for grade beams we recommend a minimum embedment depth below lowest adjacent pad grade of 24 inches and a minimum width of 12 inches. In addition, minimum reinforcement should consist of four No. 4 steel reinforcing bars; two placed near the top of the grade beam and two near the bottom.

8.10.9 If pile spacing is at least three times the maximum dimension of the pile, no reduction in axial capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be performed by Geocon based on pile dimension and spacing. This will be addressed under separate cover, if necessary, as the design progresses.

8.11 End-Bearing Caissons

8.11.1 Drilled, cast-in-placed end-bearing caissons may also be used to support proposed improvements provided the foundations derive support in bedrock. Drilled, cast-in-place end-bearing concrete caissons should be a minimum of 24 inches in diameter. For preliminary design purposes 24, 36, and 48-inch diameter drilled cast-in-place end-bearing caissons have been evaluated. Caissons should be embedded a minimum of 5 feet into the bedrock to be considered fixed. The allowable axial capacities for end-bearing caissons embedded into the competent alluvial soils are provided in the table on the following page. The axial capacities also include consideration of downdrag forces due to liquefaction and or consolidation of the overlying alluvial soils.

8.11.2 End-bearing caissons supporting the proposed structures may use the capacities presented in the following table.

Caisson Diameter (inches)	Depth Embedded into Bedrock (ft)	Axial Capacity* (kips)
24	5	18
	10	27
36	5	58
	10	79
48	5	116
	10	154

*Capacities have been reduced for Buoyancy

8.11.3 All drilled excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the ultimate strength of the bedrock. The compressive and tensile strength of the caisson sections should be checked to verify the structural capacity of the caissons.

8.11.4 The allowable downward capacity and allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

8.11.5 Single caisson uplift capacity may be determined using a frictional resistance of 250 pounds per square foot.

8.11.6 The maximum expected static settlement for the structure supported on end-bearing caissons is estimated to be less than ¾ inch. Differential settlement between adjacent caissons foundations and/or spread foundations in bedrock is not expected to exceed ½ inch. The majority of the foundation settlement is expected to occur on initial application of loading and during construction.

8.11.7 For increased resistance to differential foundation movement and lateral drift, the caisson tops should be interconnected in two horizontal directions with grade beams or tied with a structural slab. The project structural engineer should provide slab and grade beam design, reinforcement and spacing dependent on anticipated loading. However, for grade beams we recommend a minimum embedment depth below lowest adjacent pad grade of 24 inches and a minimum width of 12 inches. In addition, minimum reinforcement should consist of four No. 4 steel reinforcing bars; two placed near the top of the grade beam and two near the bottom.

8.11.8 If caisson spacing is at least three times the maximum dimension of the caisson, no reduction in axial capacity is considered necessary for group effects. If caisson spacing is closer than three diameters, an evaluation for group effects including appropriate reductions should be performed by Geocon based on caisson dimension and spacing.

8.11.9 All loose soils must be completely removed from the bottom of all end-bearing foundation excavations and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.12 Deepened Foundation Installation

8.12.1 Casing may be required if caving occurs in the granular soil layers during deep drilled excavation. The contractor should have casing available and should be prepared to use it. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.12.2 Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design; however, a clean out of the excavation bottom will be required. Where end-bearing caissons are used, all loose soils must be completely removed. Foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete.

8.12.3 Groundwater was encountered at the time of exploration in Borings B1 and B12. Therefore, the contractor should be prepared for groundwater during pile installation if it is encountered. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

8.12.4 A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength of 1,000 psi over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present. Extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by a representative of this firm is required.

8.12.5 Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight.

8.13 Lateral Design

8.13.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.40 may be used with the dead load forces in the undisturbed alluvial soils, bedrock, or properly compacted engineered fill.

8.13.2 Passive earth pressure for the sides of foundations and slabs poured against competent bedrock may be computed as an equivalent fluid having a density of 500 pounds per cubic foot (pcf), with an increase of 200 psf for each additional foot of embedment, up to a maximum earth pressure of 2,700 pcf. Passive earth pressure for the sides of foundations poured against properly compacted engineered fill or undisturbed alluvium may be computed as an equivalent fluid having a density of 230 pounds per cubic foot with a maximum earth pressure of 2,300 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.13.3 Ultimate lateral capacities for ¼ inch deflection of fixed and free-head drilled cast-in place piles are presented in the table below. No factors of safety have been applied to the lateral load values calculated to induce ¼-inch lateral deflection. Lateral capacities provided are for 24-, 30-, and 36-inch diameter drilled cast-in-place concrete piles, penetrating the earth materials encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 psi.

LATERAL LOAD CAPACITIES OF DRILLED CAST-IN-PLACE PILES

FIXED HEAD (NO HEAD ROTATION)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Positive Moment "Mp" (LAT FORCE =P)	Maximum Negative Moment "Mp" (LAT FORCE =P)	Depth to Max Pos. Moment (Feet)	Depth to Zero Moment (Feet)	Depth to Inflection Point (Feet)	MINIMUM PILE LENGTH FOR APPLICABILITY OF LATERAL DESIGN DATA (FEET)
1	24	**43**	**1.4** P	**-5.1** P	**12**	**25**	**6.4**	**25**
2	30	**61**	**1.7** P	**-6.1** P	**15**	**30**	**7.6**	**30**
3	36	**81**	**1.9** P	**-7.1** P	**17**	**35**	**8.8**	**35**

FREE HEAD (HINGED)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Moment "Mp" (LAT FORCE =P)	Depth to Zero Moment (Feet)	Depth to Maximum Moment (Feet)
1	24	**17**	**4.3** P	**23**	**7**
2	30	**25**	**5.2** P	**27**	**9**
3	36	**33**	**6.0** P	**31**	**10**

Lateral capacities are based on 1/4-inch deflection.
Moment magnitudes are presented as a function of the applied lateral load “P”.
"P" is entered in units of kips and the moment magnitude will be in units of kip-feet.
The maximum negative moment is at the rigid, pile to pile cap or grade beam connection at the top of the pile.

8.13.4 Once the project design proceeds to a more finalized state and the foundation system has been selected, an LPile analysis of lateral pile capacity can be performed, if necessary. If piles are spaced at least at least 8 diameters on-center when loaded in-line and at least 3 diameters on-center when loaded in parallel, no reduction in lateral capacity is considered necessary for group effects. If pile spacing is closer, an evaluation for group effects including appropriate reductions should be incorporated into the pile design based on pile dimension, spacing, and the direction of loading. This will be addressed under separate cover if necessary, as the design progresses.

8.14 Concrete Slabs-on-Grade

8.14.1 Concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches of concrete reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade should derive support on either undisturbed bedrock or newly placed engineered fill subsequent to the recommended grading.

8.14.2 Slabs-on-grade that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder selection and design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) as well as ASTM E1745 and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

8.14.3 For seismic design purposes, an allowable coefficient of friction of 0.40 may be utilized between concrete slabs and subgrade soils; and 0.15 for slabs underlain by a vapor retarder.

8.14.4 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moisture conditioned to at least 2 percent above optimum moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

8.14.5 The recommendations of this report are intended to reduce the potential for cracking of slabs due to minor soil movements. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

8.15 Preliminary Pavement Recommendations

8.15.1 Where new paving is to be placed, it is recommended that all existing fill and soft alluvium materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to at least two percent above optimum moisture content, and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

8.15.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

8.15.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking and Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

8.15.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans). The use of Crushed Miscellaneous Base (CMB) in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

8.15.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 92 and 95 percent relative compaction, respectively. as determined by ASTM Test Method D 1557 (latest edition).

8.15.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

8.16 Retaining Wall Design

8.16.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 14 feet. In the event that walls significantly higher than 14 feet are planned, Geocon should be contacted for additional recommendations.

8.16.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Sections 8.6).

8.16.3 Based on the orientation of the proposed excavations with respect to the strike and dip of the bedrock, Adverse bedding conditions are expected for south and east facing excavations. The retaining wall recommendations presented below include consideration of the surcharge due to out-of-slope bedding on the south and east retaining walls. Excavations into bedrock should be observed and approved in writing by the Project Geologist (a representative of Geocon West, Inc.) during excavation to check for the presence of jointing or bedding which may require revised retaining wall recommendations.

8.16.4 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. Calculation of the recommended wall pressures are provided on Figures 9 through 11.

RETAINING WALL WITH LEVEL BACKFILL SURFACE
(North and West Facing Retaining Walls – Favorable Bedding)

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 12	30	54

RETAINING WALL WITH SURCHARGE FROM JOINTING OR BEDDING
(East and South Facing Retaining Walls – Adverse Bedding)

HEIGHT OF RETAINING WALL (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 12	69	95

RETAINING WALL WITH LEVEL BACKFILL SURFACE
(Walls Supporting Alluvial Soils or Engineered Fill)

HEIGHT OF RETAINING WALL (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 12	46	63

8.16.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 110 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

8.16.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Recommendations for the incorporation of surcharges are provided in section 8.24 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

8.16.7 In addition to the recommended earth pressure, the upper 10 feet of the retaining wall adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the wall due to normal street traffic. If the traffic is kept back at least 10 feet from the wall, the traffic surcharge may be neglected.

8.16.8 Seismic lateral forces should be incorporated into the design as necessary, and recommendations for seismic lateral forces are presented below.

8.17 Dynamic (Seismic) Lateral Forces

8.17.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2022 CBC).

8.17.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2022 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-16 Section 11.8.3.

8.18 Retaining Wall Drainage

8.18.1 Retaining walls not designed for hydrostatic pressures should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 12). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

8.18.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 13). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

8.18.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures.

8.18.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

8.19 Elevator Pit Design

8.19.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pits may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 8.6 and 8.16).

8.19.2 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent foundations and should be designed for each condition as the project progresses.

8.19.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 8.18).

8.19.4 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

8.20 Elevator Piston

8.20.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation or shoring pile, or the drilled excavation could compromise the existing foundation or pile support, especially if the drilling is performed subsequent to the foundation or pile construction.

8.20.2 Groundwater should be expected and casing may be required if caving is experienced in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

8.20.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

8.21 Temporary Excavations

8.21.1 Excavations up to 14 feet in height are anticipated for excavation and construction of the proposed administration structure, including foundation system. The excavations are expected to expose artificial fill, alluvium, colluvium and bedrock which are considered stable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, where adverse bedding conditions are not present and where not surcharged by adjacent traffic or structures.

8.21.2 Excavations into bedrock should be observed by the Project Geologist (a representative of Geocon West, Inc.) during excavation to check for the presence of jointing or bedding which may require special recommendations for sloping and/or shoring. Any recommendations deemed necessary will be provided at that time.

8.21.3 Vertical excavations greater than the permissible heights outlined above will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1.5:1 (H:V) slope gradient or flatter up to a maximum height of 10 feet. Temporary unsurcharged embankments may also be sloped back at a uniform 2:1 (H:V) slope gradient or flatter up to a maximum height of 15 feet. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. Shoring recommendations are provided in Section 8.22 of this report.

8.21.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

8.22 Shoring – Soldier Pile Design and Installation

8.22.1 The following information on the design and installation of shoring is preliminary. Review of the final shoring plans and specifications should be made by this office prior to bidding or negotiating with a shoring contractor.

8.22.2 Installation of shoring on a sloping ground surface requires careful consideration of excavation sequencing. Prior to installation of shoring, grading to create a relatively flat pad for equipment access may be required. Excavation of unsupported vertical cuts into a sloping ground surface would remove support from the ascending portion of the slope and create a potentially unstable condition. Unsupported vertical excavation into a slope is not permitted. Equipment access can be created by placement of additional fill to build up a temporary equipment pad against the existing slope.

8.22.3 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Installation of shoring piles by vibration is not recommended at this site due to the dense nature of the underlying bedrock. Where maximum excavation heights are less than 12 feet the soldier piles are typically designed as cantilevers. Where excavations exceed 12 feet or are surcharged, soldier piles may require lateral bracing utilizing drilled tie-back anchors or raker braces to maintain an economical steel beam size and prevent excessive deflection. The size of the steel beam, the need for lateral bracing, and the acceptable shoring deflection should be determined by the project shoring engineer. Should it be determined that temporary tiebacks or rakers are necessary, additional recommendations can be provided under separate cover.

8.22.4 The design embedment of the shoring pile toes must be maintained during excavation activities. The toes of the perimeter shoring piles should be deepened to take into account any required excavations necessary for grading activities, foundations, and/or adjacent drainage systems.

8.22.5 The proposed soldier piles may also be designed as permanent piles. The required pile depth, dimension, spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Walls* section of this report (see Section 8.16).

8.22.6 Drilled cast-in-place soldier piles should be placed no closer than three diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the bedrock below the bottom plane of excavation may be assumed to be 500 pcf, with an increase of 200 psf for each additional foot of embedment. The allowable passive value may be doubled for isolated piles, spaced a minimum of 3 times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed bedrock.

8.22.7 Casing may be required if caving is experienced in the drilled excavation and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.22.8 The contractor should be aware that some difficult drilling conditions could be encountered in the bedrock which could require coring and jack-hammering. The contractor should be prepared for these conditions prior to commencement of drilling activities.

8.22.9 Groundwater seepage was encountered during this site exploration and in past explorations. Therefore, the contractor should be prepared for groundwater during pile installation should the need arise. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

8.22.10 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 pounds per square inch (psi) over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste should be included. The slump should be commensurate to any research report for the admixture, provided that it should also be the minimum for a reasonable consistency for placing when water is present.

8.22.11 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.40 based on uniform contact between the steel beam and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 340 pounds per square foot.

8.22.12 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils and the areas where lagging may be omitted.

8.22.13 The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot.

8.22.14 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following tables, be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained at the top by bracing or tie backs. The recommended active and trapezoidal pressure are provided in the following tables. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculation of the recommended shoring pressures are provided on Figures 14 and 15.

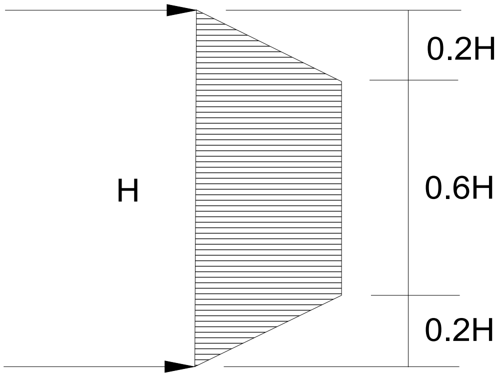
SHORING WALL WITH LEVEL BACKFILL SURFACE
(North and West Facing Shoring Walls – Favorable Bedding)

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE) Triangular	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal (Where H is the height of the shoring in feet)
Up to 14	25	16H

SHORING WALL WITH LEVEL BACKFILL SURFACE
(South and East facing Shoring Walls – Adverse Bedding)

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE) Triangular	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal (Where H is the height of the shoring in feet)
Up to 14	63	40H

Trapezoidal Distribution of Pressure



8.22.15 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures and must be determined for each combination.

8.23 Stormwater Infiltration

8.23.1 During the April 18 and April 19, 2022 site exploration, borings B4 and B7 were utilized to perform percolation testing. The borings were backfilled to the proposed invert elevation with a bentonite seal placed at the bottom of the excavation. Slotted casing was placed in the boring, and the annular space between the casing and excavation was filled with filter pack. The boring was then filled with water to pre-saturate the soils. On April 19, 2022, the casings were refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the measured percolation rate and design infiltration rate, for the earth materials encountered, are provided in the following table. These values have been calculated in accordance with the Boring Percolation Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (June 2021). Percolation test field data and calculation of the measured percolation rate and design infiltration rate are provided on Figures 16 and 17.

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in / hour)	Design Infiltration Rate (in / hour)
B4	SM	4-8	0.12	0.04
B7	CL	2-5	0.04	0.01

8.23.2 The results of the percolation testing indicate that the soils are not conducive to infiltration of stormwater. The design infiltration rate is below the minimum infiltration rate of 0.3 inch per hour recommended in the County guidelines. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.

8.24 Surcharge from Adjacent Structures and Improvements

8.24.1 Additional pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

8.24.2 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

where x is the distance from the face of the excavation or wall to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation or wall, z is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load and $\sigma_H(z)$ is the horizontal pressure at depth z.

8.24.3 It is recommended that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H\,(z) = \sigma_H(z)cos^2(1.1\theta)$$

where x is the distance from the face of the excavation/wall to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth z, θ is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth z.

8.25 Surface Drainage

8.25.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

8.25.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2022 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

8.25.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

8.25.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

8.26 Plan Review

8.26.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project Geotechnical Engineer of Record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

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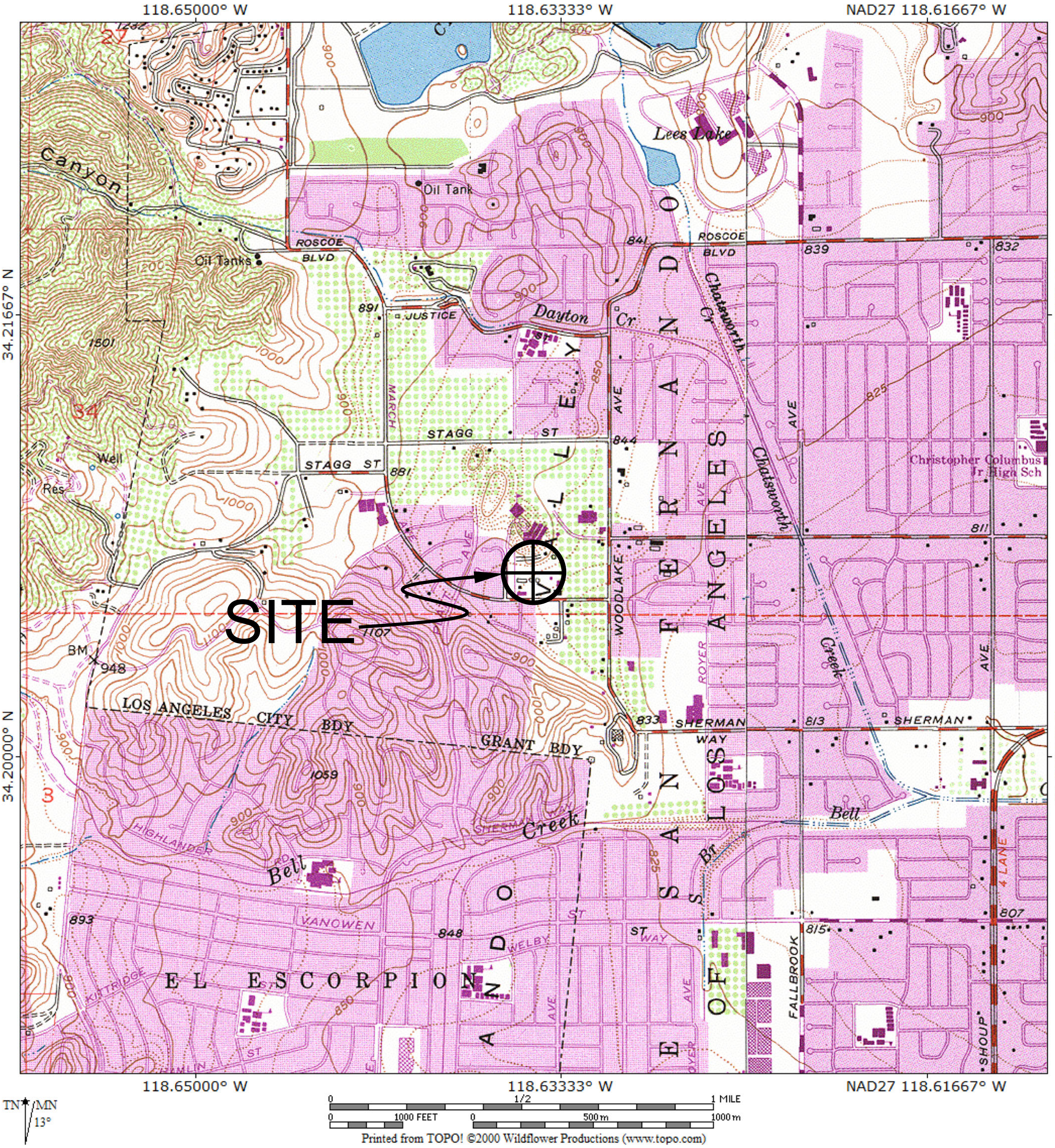

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, CALABASAS, CA QUADRANGLE

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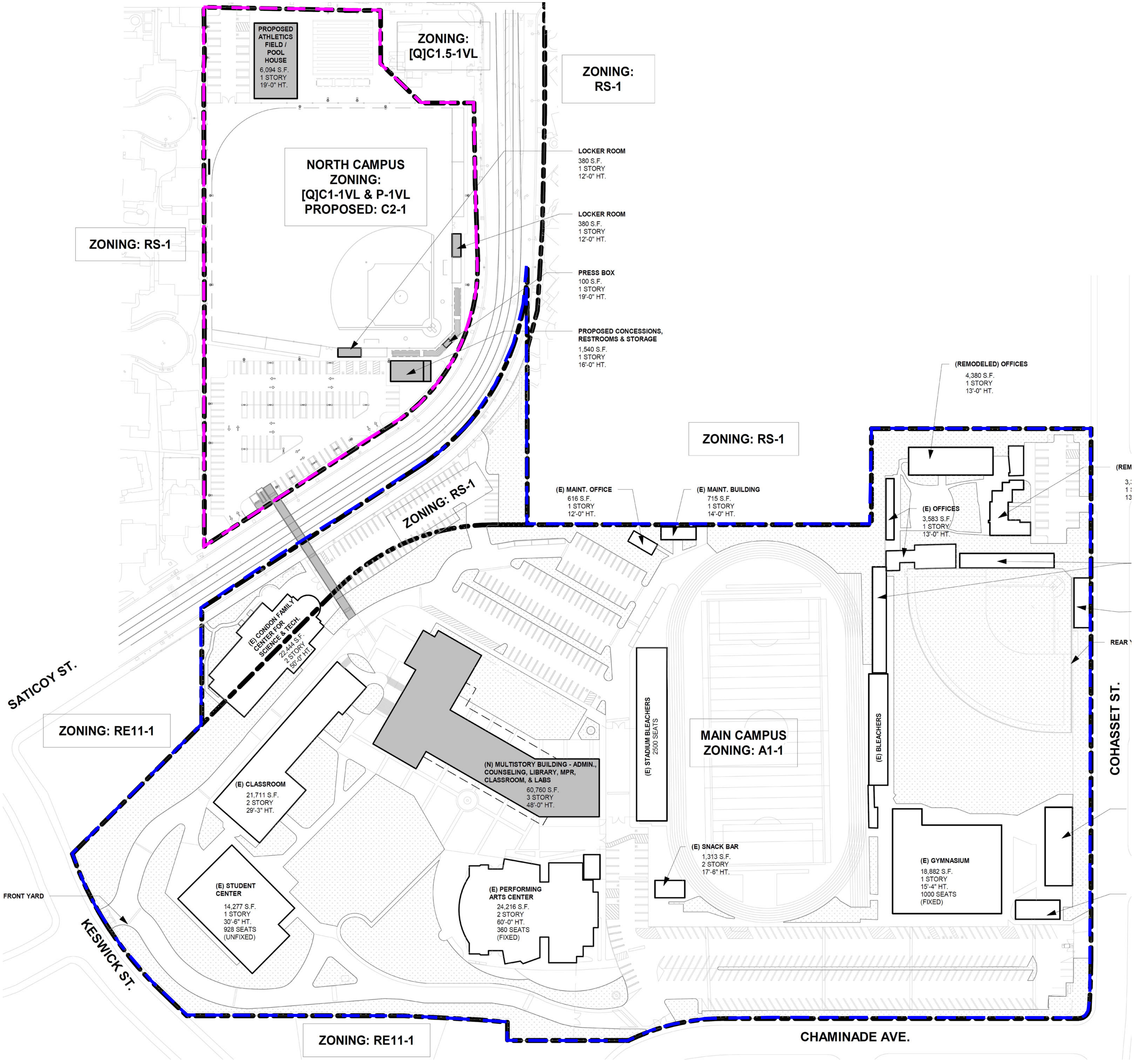
DRAFTED BY: RA	CHECKED BY: SFK

VICINITY MAP

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 2023	PROJECT NO. W1547-06-01	FIG. 1



LEGEND

NORTH CAMPUS LIMITS

MAIN CAMPUS LIMITS

0 150' 300'

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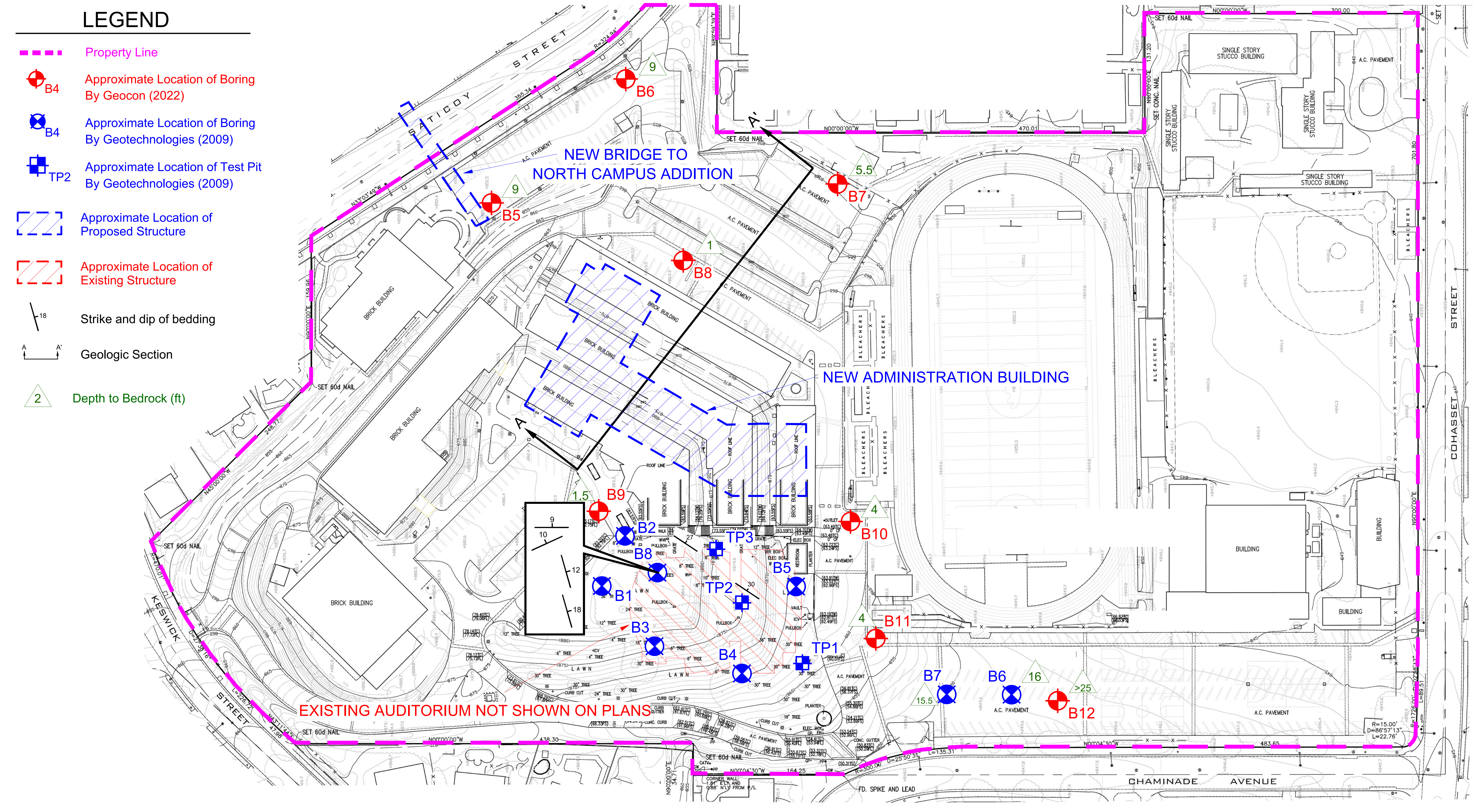
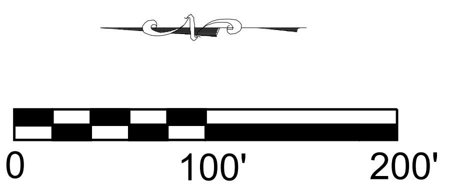
SITE PLAN

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023	PROJECT NO. W1547-06-01	FIG. 2A





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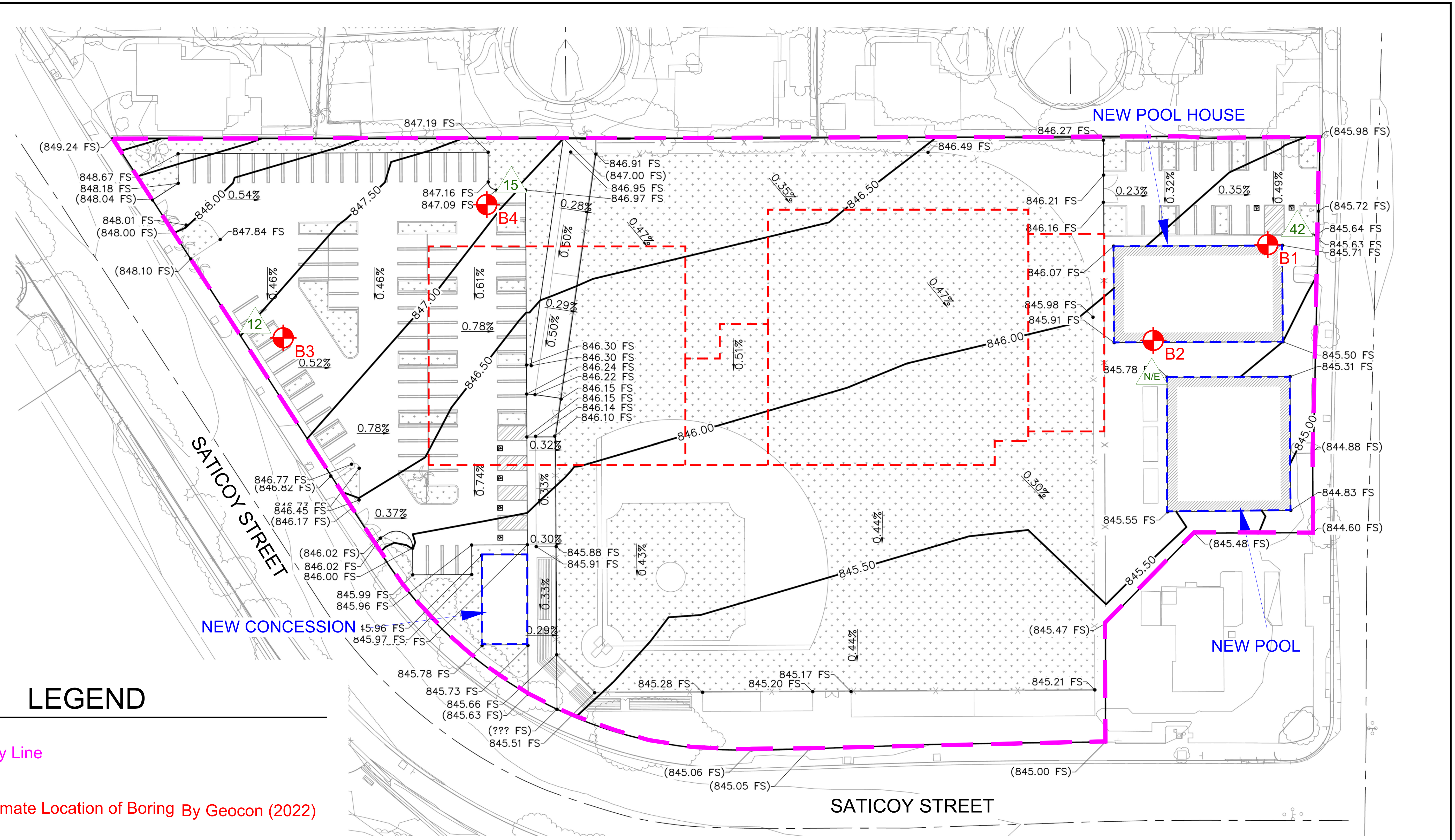
SITE PLAN (MAIN CAMPUS)

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023	PROJECT NO. W1547-06-01	FIG. 2B



LEGEND

▬ ▬ Property Line

B4 Approximate Location of Boring By Geocon (2022)

▨ Approximate Location of Proposed Structure

Limits of Existing Structures To Be Demolished

2 Depth to Bedrock (ft)

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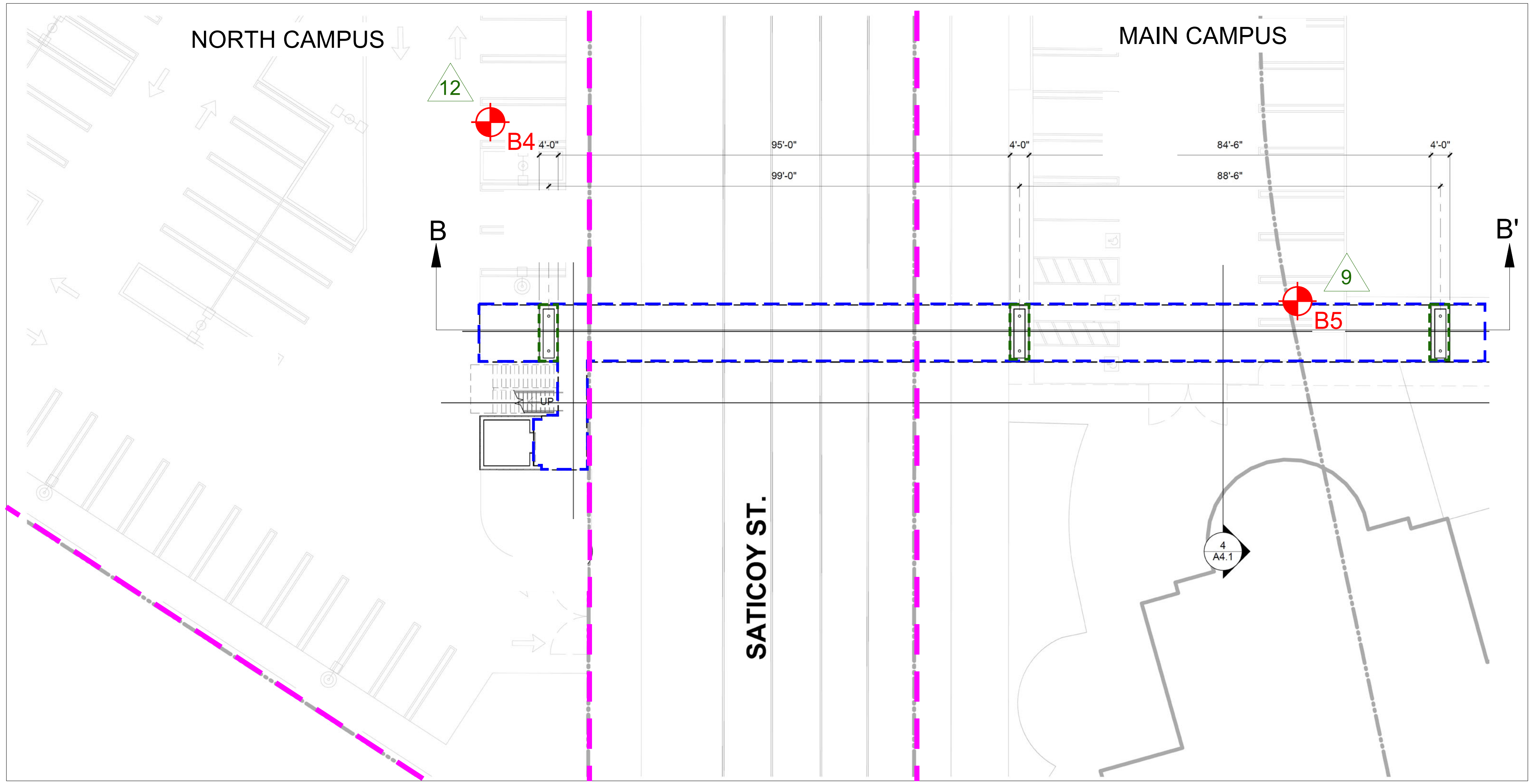
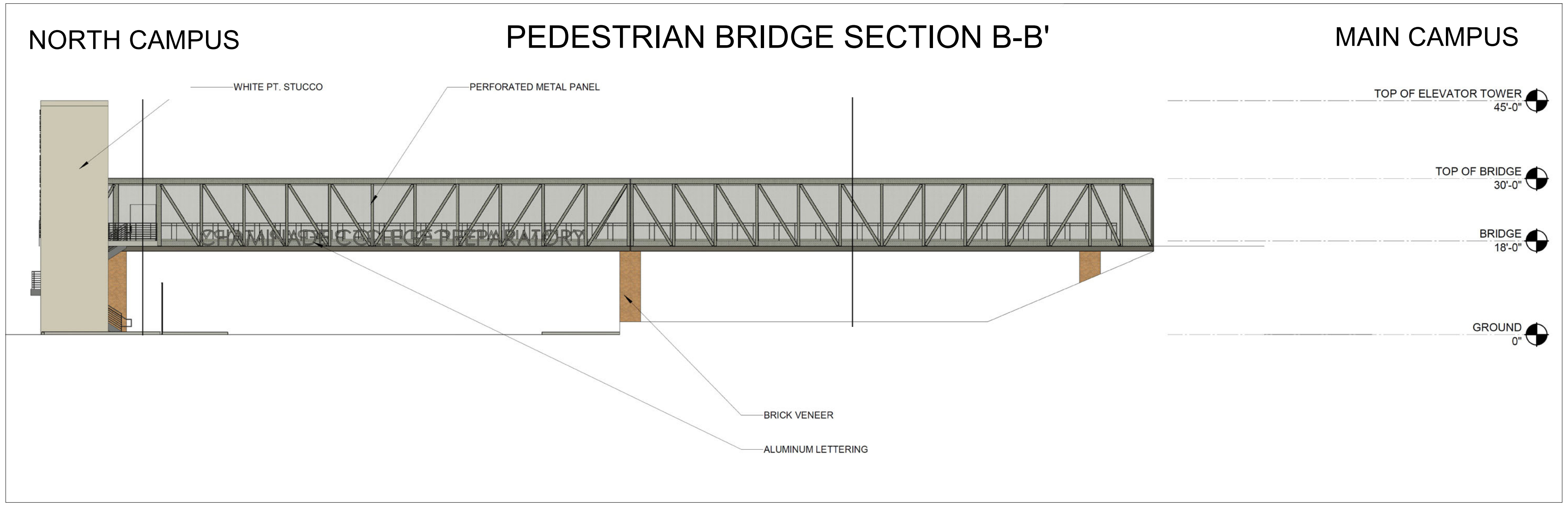
SITE PLAN (NORTH CAMPUS)

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023 | PROJECT NO. W1547-06-01 | FIG. 2C

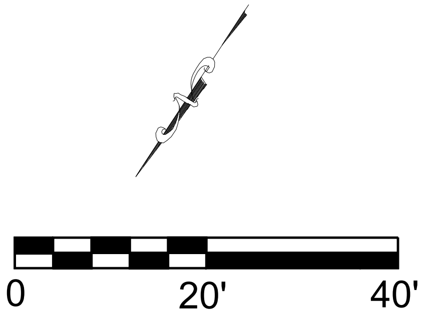


PEDESTRIAN BRIDGE SECTION B-B'



LEGEND

▬ ▬	Property Line
⊕ B4	Approximate Location of Boring By Geocon (2022)
▨	Limits of Proposed Pedestrian Bridge
▨	Limits of Proposed Foundation
△ 12	Depth to Bedrock (ft)



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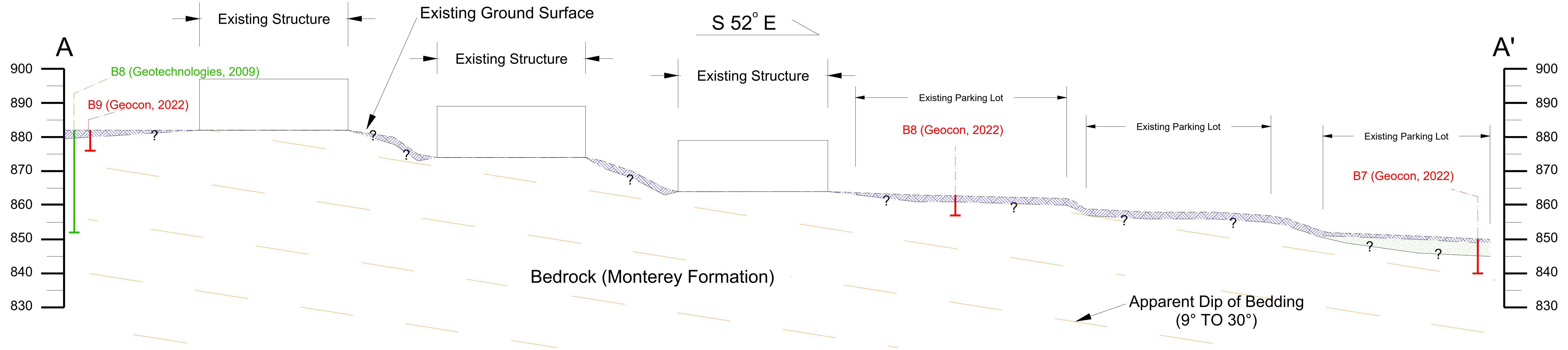
SITE PLAN (PEDESTRIAN BRIDGE)

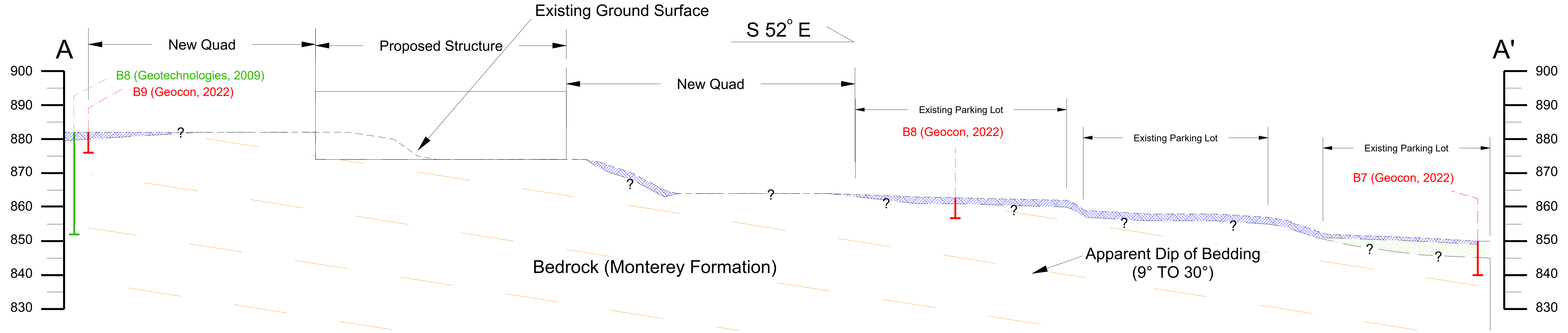
CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023
PROJECT NO. W1547-06-01
FIG. 2D

EXISTING CONDITION



PROPOSED CONDITION



▨ Fill

▨ Colluvium/Alluvium

▨ Bedrock (Monterey Formation)

0 30' 60'

NOT FOR CONSTRUCTION; FOR ILLUSTRATION PURPOSES ONLY

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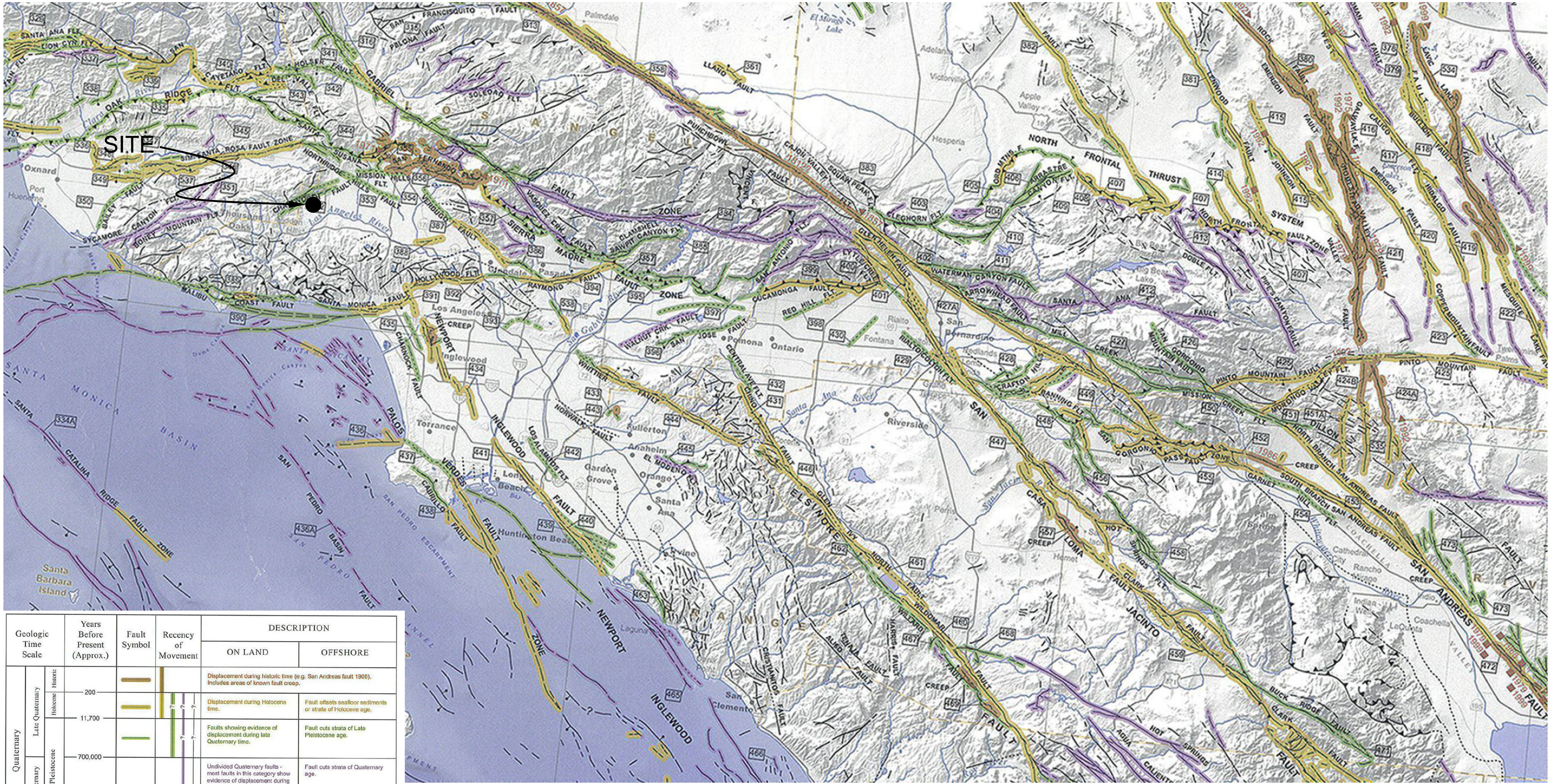
CROSS SECTIONS

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

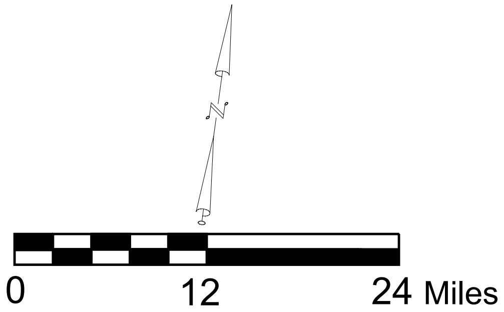
JAN. 2023 PROJECT NO. W1547-06-01 FIG. 2E

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic		🟫		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟨	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.



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PHONE (818) 841-8388 - FAX (818) 841-1704

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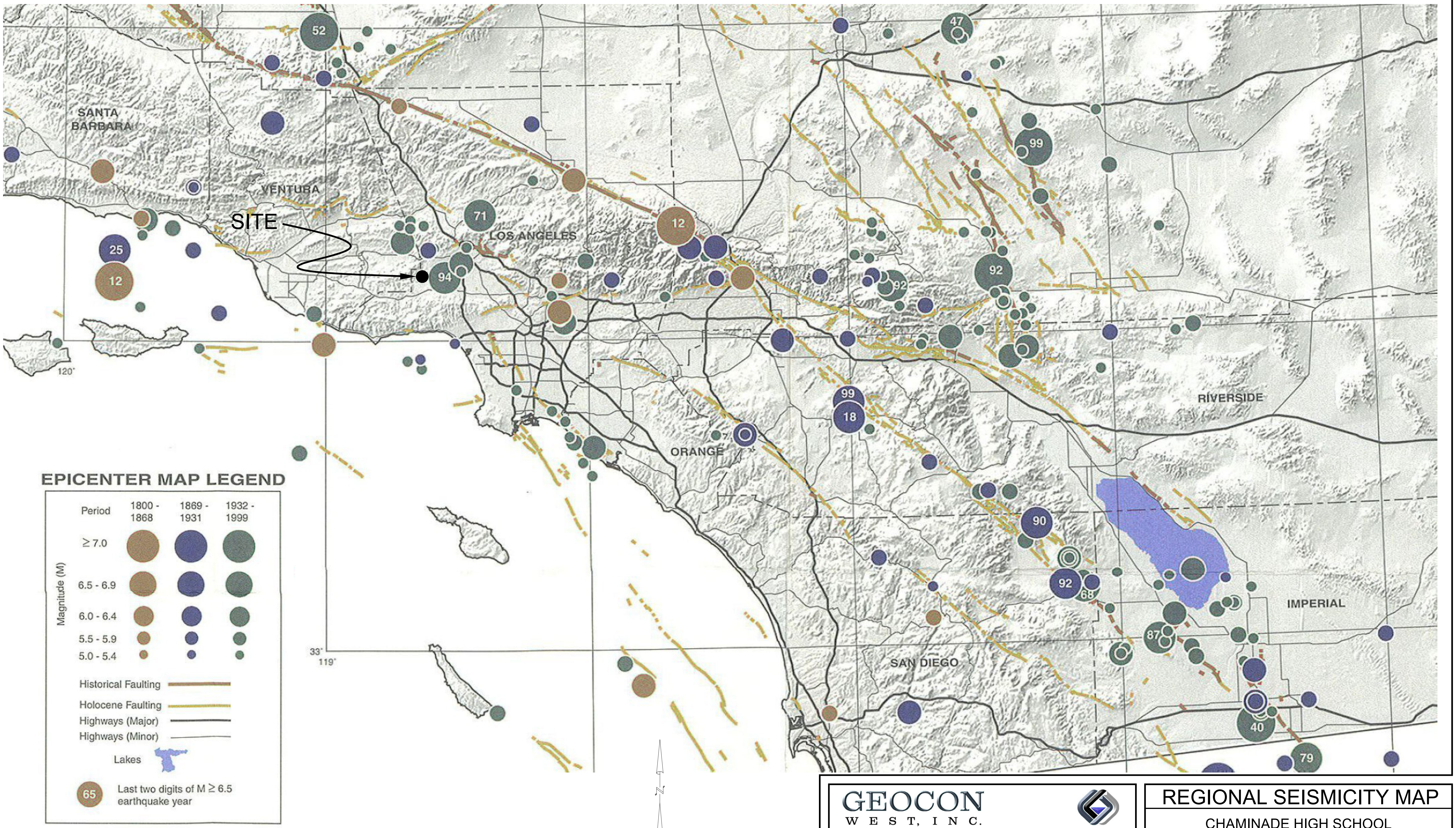
REGIONAL FAULT MAP

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023
PROJECT NO. W1547-06-01
FIG. 3



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REGIONAL SEISMICITY MAP

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023
PROJECT NO. W1547-06-01
FIG.4

GEOCON

Project: Chaminade N Campus Extensior
File No. : W1547-06-01
Boring : B1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.58
Peak Horiz. Acceleration PGA_M (g):	0.646
2/3 PGA_M (g):	0.431
Calculated Mag.Wtg.Factor:	0.719
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	32.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.29
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.998	0.201	--
2.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.993	0.200	--
3.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.989	0.199	--
4.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.984	0.198	--
5.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.979	0.197	--
6.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.975	0.196	--
7.0	120.0	0	2.0	5.0	1	30	31	1.636	9.7	120.0	0.106	0.970	0.195	--
8.0	120.0	0	2.0	5.0	1	30	31	1.523	9.4	120.0	0.104	0.966	0.194	--
9.0	120.0	0	2.0	5.0	1	30	31	1.431	9.2	120.0	0.101	0.961	0.193	--
10.0	120.0	0	2.0	5.0	1	30	31	1.353	9.0	120.0	0.100	0.957	0.193	--
11.0	120.0	1	4.0	10.0	0	79		1.287	13.0	57.6	~	0.952	0.197	~
12.0	120.0	1	4.0	10.0	0	79		1.230	12.7	57.6	~	0.947	0.205	~
13.0	120.0	1	7.0	15.0	0	56		1.180	17.3	57.6	~	0.943	0.212	~
14.0	120.0	1	7.0	15.0	0	56		1.135	16.9	57.6	~	0.938	0.218	~
15.0	120.0	1	7.0	15.0	0	56		1.095	16.6	57.6	~	0.934	0.224	~
16.0	120.0	1	7.0	15.0	0	56		1.060	16.3	57.6	~	0.929	0.229	~
17.0	120.0	1	7.0	15.0	0	56		1.027	16.0	57.6	~	0.925	0.234	~
18.0	120.0	1	7.0	15.0	0	56		0.997	15.7	57.6	~	0.920	0.238	~
19.0	120.0	1	7.0	15.0	0	56		0.970	15.5	57.6	~	0.915	0.242	~
20.0	120.0	1	14.0	20.0	1	38	68	0.945	25.3	57.6	0.291	0.911	0.246	1.18
21.0	120.0	1	14.0	20.0	1	38	68	0.921	24.9	57.6	0.283	0.906	0.249	1.14
22.0	120.0	1	10.0	25.0	1	57	56	0.900	20.3	57.6	0.222	0.902	0.251	0.88
23.0	120.0	1	10.0	25.0	1	57	56	0.879	20.0	57.6	0.218	0.897	0.254	0.86
24.0	120.0	1	10.0	25.0	1	57	56	0.860	19.7	57.6	0.215	0.893	0.256	0.84
25.0	120.0	1	10.0	25.0	1	57	56	0.843	19.5	57.6	0.212	0.888	0.258	0.82
26.0	120.0	1	10.0	25.0	1	57	56	0.826	19.2	57.6	0.209	0.883	0.260	0.81
27.0	120.0	1	10.0	25.0	1	57	56	0.810	19.0	57.6	0.207	0.879	0.262	0.79
28.0	120.0	1	10.0	25.0	1	57	56	0.795	18.8	57.6	0.204	0.874	0.263	0.78
29.0	120.0	1	10.0	25.0	1	57	56	0.781	18.6	57.6	0.202	0.870	0.264	0.76
30.0	120.0	1	11.0	30.0	0	86		0.768	20.1	57.6	~	0.865	0.265	~
31.0	120.0	1	11.0	30.0	0	86		0.755	19.9	57.6	~	0.861	0.266	~
32.0	120.0	1	11.0	30.0	0	86		0.746	19.7	57.6	~	0.856	0.267	~
33.0	120.0	1	22.0	32.5	1	0	78	0.741	25.2	57.6	0.283	0.851	0.268	1.06
34.0	120.0	1	22.0	32.5	1	0	78	0.735	25.0	57.6	0.280	0.847	0.268	1.04
35.0	120.0	1	8.0	35.0	0	93		0.730	16.0	57.6	~	0.842	0.269	~
36.0	120.0	1	8.0	35.0	0	93		0.724	16.0	57.6	~	0.838	0.269	~
37.0	120.0	1	8.0	35.0	0	93		0.719	15.9	57.6	~	0.833	0.269	~
38.0	120.0	1	19.0	37.5	1	0	70	0.714	21.0	57.6	0.221	0.829	0.270	0.82
39.0	120.0	1	19.0	37.5	1	0	70	0.709	20.9	57.6	0.219	0.824	0.270	0.81
40.0	120.0	1	19.0	37.5	1	0	70	0.704	20.7	57.6	0.218	0.819	0.270	0.81
41.0	120.0	1	12.0	40.0	0	62		0.700	20.0	57.6	~	0.815	0.270	~
42.0	120.0	1	12.0	40.0	0	62		0.695	19.9	57.6	~	0.810	0.269	~
43.0	120.0	1	31.0	45.0	0	73		0.690	40.1	57.6	~	0.806	0.269	~
44.0	120.0	1	31.0	45.0	0	73		0.686	39.9	57.6	~	0.801	0.269	~
45.0	120.0	1	31.0	45.0	0	73		0.681	39.7	57.6	~	0.797	0.269	~
46.0	120.0	1	31.0	45.0	0	73		0.677	39.5	57.6	~	0.792	0.268	~
47.0	120.0	1	31.0	50.0	0	73		0.673	39.3	57.6	~	0.787	0.268	~
48.0	120.0	1	31.0	50.0	0	73		0.669	39.1	57.6	~	0.783	0.267	~
49.0	120.0	1	31.0	50.0	0	73		0.665	38.9	57.6	~	0.778	0.267	~
50.0	120.0	1	31.0	50.0	0	73		0.661	38.7	57.6	~	0.774	0.266	~

Figure 5

GEOCON

Project: Chaminade N Campus Extens
File No. : W1547-06-01
Boring : B1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.58
PGAM (g):	0.646
2/3 PGAM (g):	0.43
Calculated Mag.Wtg.Factor:	0.719
Historic High Groundwater:	10.0
Groundwater @ Exploration:	32.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e$_{15}$} (%)	EQ. SETTLE. Pe (in.)
1.0	2	120.0	0.030	0.030	31	10	0.280	--	0.00	0.00
2.0	2	120.0	0.090	0.090	31	10	0.280	--	0.00	0.00
3.0	2	120.0	0.150	0.150	31	10	0.280	--	0.00	0.00
4.0	2	120.0	0.210	0.210	31	10	0.280	--	0.00	0.00
5.0	2	120.0	0.270	0.270	31	10	0.280	--	0.00	0.00
6.0	2	120.0	0.330	0.330	31	10	0.280	--	0.00	0.00
7.0	2	120.0	0.390	0.390	31	10	0.280	--	0.00	0.00
8.0	2	120.0	0.450	0.450	31	9	0.280	--	0.00	0.00
9.0	2	120.0	0.510	0.510	31	9	0.280	--	0.00	0.00
10.0	2	120.0	0.570	0.570	31	9	0.280	--	0.00	0.00
11.0	4	120.0	0.630	0.614		13	0.287	~	0.00	0.00
12.0	4	120.0	0.690	0.643		13	0.300	~	0.00	0.00
13.0	7	120.0	0.750	0.672		17	0.313	~	0.00	0.00
14.0	7	120.0	0.810	0.701		17	0.324	~	0.00	0.00
15.0	7	120.0	0.870	0.730		17	0.334	~	0.00	0.00
16.0	7	120.0	0.930	0.758		16	0.343	~	0.00	0.00
17.0	7	120.0	0.990	0.787		16	0.352	~	0.00	0.00
18.0	7	120.0	1.050	0.816		16	0.360	~	0.00	0.00
19.0	7	120.0	1.110	0.845		15	0.368	~	0.00	0.00
20.0	14	120.0	1.170	0.874	68	25	0.375	1.18	0.00	0.00
21.0	14	120.0	1.230	0.902	68	25	0.382	1.14	0.00	0.00
22.0	10	120.0	1.290	0.931	56	20	0.388	0.88	1.40	0.17
23.0	10	120.0	1.350	0.960	56	20	0.394	0.86	1.40	0.17
24.0	10	120.0	1.410	0.989	56	20	0.399	0.84	1.60	0.19
25.0	10	120.0	1.470	1.018	56	19	0.405	0.82	1.60	0.19
26.0	10	120.0	1.530	1.046	56	19	0.410	0.81	1.60	0.19
27.0	10	120.0	1.590	1.075	56	19	0.414	0.79	1.60	0.19
28.0	10	120.0	1.650	1.104	56	19	0.419	0.78	1.60	0.19
29.0	10	120.0	1.710	1.133	56	19	0.423	0.76	1.60	0.19
30.0	11	120.0	1.770	1.162		20	0.427	~	0.00	0.00
31.0	11	120.0	1.830	1.190		20	0.431	~	0.00	0.00
32.0	11	120.0	1.890	1.219		20	0.434	~	0.00	0.00
33.0	22	120.0	1.950	1.248	78	25	0.438	1.06	1.00	0.12
34.0	22	120.0	2.010	1.277	78	25	0.441	1.04	1.00	0.12
35.0	8	120.0	2.070	1.306		16	0.444	~	0.00	0.00
36.0	8	120.0	2.130	1.334		16	0.447	~	0.00	0.00
37.0	8	120.0	2.190	1.363		16	0.450	~	0.00	0.00
38.0	19	120.0	2.250	1.392	70	21	0.453	0.82	1.40	0.17
39.0	19	120.0	2.310	1.421	70	21	0.455	0.81	1.40	0.17
40.0	19	120.0	2.370	1.450	70	21	0.458	0.81	1.40	0.17
41.0	12	120.0	2.430	1.478		20	0.460	~	0.00	0.00
42.0	12	120.0	2.490	1.507		20	0.463	~	0.00	0.00
43.0	31	120.0	2.550	1.536		40	0.465	~	0.00	0.00
44.0	31	120.0	2.610	1.565		40	0.467	~	0.00	0.00
45.0	31	120.0	2.670	1.594		40	0.469	~	0.00	0.00
46.0	31	120.0	2.730	1.622		39	0.471	~	0.00	0.00
47.0	31	120.0	2.790	1.651		39	0.473	~	0.00	0.00
48.0	31	120.0	2.850	1.680		39	0.475	~	0.00	0.00
49.0	31	120.0	2.910	1.709		39	0.477	~	0.00	0.00
50.0	31	120.0	2.970	1.738		39	0.479	~	0.00	0.00

TOTAL SETTLEMENT = 2.2 INCHES

Figure 6

GEOCON

Project: Chaminade N Campus Extensior
File No. : W1547-06-01
Boring : B1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.66
Peak Horiz. Acceleration PGA_M (g):	0.646
Calculated Mag.Wtg.Factor:	0.741
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	32.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.29
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

P200 Values are preliminary

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.998	0.311	--
2.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.993	0.309	--
3.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.989	0.308	--
4.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.984	0.306	--
5.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.979	0.305	--
6.0	120.0	0	2.0	5.0	1	30	31	1.700	9.8	120.0	0.108	0.975	0.303	--
7.0	120.0	0	2.0	5.0	1	30	31	1.636	9.7	120.0	0.106	0.970	0.302	--
8.0	120.0	0	2.0	5.0	1	30	31	1.523	9.4	120.0	0.104	0.966	0.301	--
9.0	120.0	0	2.0	5.0	1	30	31	1.431	9.2	120.0	0.101	0.961	0.299	--
10.0	120.0	0	2.0	5.0	1	30	31	1.353	9.0	120.0	0.100	0.957	0.298	--
11.0	120.0	1	4.0	10.0	0	79		1.287	13.0	57.6	~	0.952	0.304	~
12.0	120.0	1	4.0	10.0	0	79		1.230	12.7	57.6	~	0.947	0.316	~
13.0	120.0	1	7.0	15.0	0	56		1.180	17.3	57.6	~	0.943	0.328	~
14.0	120.0	1	7.0	15.0	0	56		1.135	16.9	57.6	~	0.938	0.338	~
15.0	120.0	1	7.0	15.0	0	56		1.095	16.6	57.6	~	0.934	0.347	~
16.0	120.0	1	7.0	15.0	0	56		1.060	16.3	57.6	~	0.929	0.355	~
17.0	120.0	1	7.0	15.0	0	56		1.027	16.0	57.6	~	0.925	0.362	~
18.0	120.0	1	7.0	15.0	0	56		0.997	15.7	57.6	~	0.920	0.369	~
19.0	120.0	1	7.0	15.0	0	56		0.970	15.5	57.6	~	0.915	0.374	~
20.0	120.0	1	14.0	20.0	1	38	68	0.945	25.3	57.6	0.291	0.911	0.380	0.77
21.0	120.0	1	14.0	20.0	1	38	68	0.921	24.9	57.6	0.283	0.906	0.385	0.74
22.0	120.0	1	10.0	25.0	1	57	56	0.900	20.3	57.6	0.222	0.902	0.389	0.57
23.0	120.0	1	10.0	25.0	1	57	56	0.879	20.0	57.6	0.218	0.897	0.393	0.56
24.0	120.0	1	10.0	25.0	1	57	56	0.860	19.7	57.6	0.215	0.893	0.396	0.54
25.0	120.0	1	10.0	25.0	1	57	56	0.843	19.5	57.6	0.212	0.888	0.399	0.53
26.0	120.0	1	10.0	25.0	1	57	56	0.826	19.2	57.6	0.209	0.883	0.402	0.52
27.0	120.0	1	10.0	25.0	1	57	56	0.810	19.0	57.6	0.207	0.879	0.405	0.51
28.0	120.0	1	10.0	25.0	1	57	56	0.795	18.8	57.6	0.204	0.874	0.407	0.50
29.0	120.0	1	10.0	25.0	1	57	56	0.781	18.6	57.6	0.202	0.870	0.409	0.49
30.0	120.0	1	11.0	30.0	0	86		0.768	20.1	57.6	~	0.865	0.410	~
31.0	120.0	1	11.0	30.0	0	86		0.755	19.9	57.6	~	0.861	0.412	~
32.0	120.0	1	11.0	30.0	0	86		0.746	19.7	57.6	~	0.856	0.413	~
33.0	120.0	1	22.0	32.5	1	0	78	0.741	25.2	57.6	0.283	0.851	0.414	0.68
34.0	120.0	1	22.0	32.5	1	0	78	0.735	25.0	57.6	0.280	0.847	0.415	0.67
35.0	120.0	1	8.0	35.0	0	93		0.730	16.0	57.6	~	0.842	0.416	~
36.0	120.0	1	8.0	35.0	0	93		0.724	16.0	57.6	~	0.838	0.416	~
37.0	120.0	1	8.0	35.0	0	93		0.719	15.9	57.6	~	0.833	0.417	~
38.0	120.0	1	19.0	37.5	1	0	70	0.714	21.0	57.6	0.221	0.829	0.417	0.53
39.0	120.0	1	19.0	37.5	1	0	70	0.709	20.9	57.6	0.219	0.824	0.417	0.53
40.0	120.0	1	19.0	37.5	1	0	70	0.704	20.7	57.6	0.218	0.819	0.417	0.52
41.0	120.0	1	12.0	40.0	0	62		0.700	20.0	57.6	~	0.815	0.417	~
42.0	120.0	1	12.0	40.0	0	62		0.695	19.9	57.6	~	0.810	0.417	~
43.0	120.0	1	31.0	45.0	0	73		0.690	40.1	57.6	~	0.806	0.416	~
44.0	120.0	1	31.0	45.0	0	73		0.686	39.9	57.6	~	0.801	0.416	~
45.0	120.0	1	31.0	45.0	0	73		0.681	39.7	57.6	~	0.797	0.415	~
46.0	120.0	1	31.0	45.0	0	73		0.677	39.5	57.6	~	0.792	0.415	~
47.0	120.0	1	31.0	50.0	0	73		0.673	39.3	57.6	~	0.787	0.414	~
48.0	120.0	1	31.0	50.0	0	73		0.669	39.1	57.6	~	0.783	0.413	~
49.0	120.0	1	31.0	50.0	0	73		0.665	38.9	57.6	~	0.778	0.413	~
50.0	120.0	1	31.0	50.0	0	73		0.661	38.7	57.6	~	0.774	0.412	~

GEOCON

Project: Chaminade N Campus Extensi
File No. : W1547-06-01
Boring : B1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.66
PGA_M (g):	0.646
Calculated Mag.Wtg.Factor:	0.741
Historic High Groundwater:	10.0
Groundwater @ Exploration:	100.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain $[e_{15}\}$ (%)	EQ. SETTLE. Pe (in.)
1.0	2	120.0	0.030	0.030	31	10	0.420	--	0.00	0.00
2.0	2	120.0	0.090	0.090	31	10	0.420	--	0.00	0.00
3.0	2	120.0	0.150	0.150	31	10	0.420	--	0.00	0.00
4.0	2	120.0	0.210	0.210	31	10	0.420	--	0.00	0.00
5.0	2	120.0	0.270	0.270	31	10	0.420	--	0.00	0.00
6.0	2	120.0	0.330	0.330	31	10	0.420	--	0.00	0.00
7.0	2	120.0	0.390	0.390	31	10	0.420	--	0.00	0.00
8.0	2	120.0	0.450	0.450	31	9	0.420	--	0.00	0.00
9.0	2	120.0	0.510	0.510	31	9	0.420	--	0.00	0.00
10.0	2	120.0	0.570	0.570	31	9	0.420	--	0.00	0.00
11.0	4	120.0	0.630	0.614		13	0.431	~	0.00	0.00
12.0	4	120.0	0.690	0.643		13	0.450	~	0.00	0.00
13.0	7	120.0	0.750	0.672		17	0.469	~	0.00	0.00
14.0	7	120.0	0.810	0.701		17	0.485	~	0.00	0.00
15.0	7	120.0	0.870	0.730		17	0.501	~	0.00	0.00
16.0	7	120.0	0.930	0.758		16	0.515	~	0.00	0.00
17.0	7	120.0	0.990	0.787		16	0.528	~	0.00	0.00
18.0	7	120.0	1.050	0.816		16	0.540	~	0.00	0.00
19.0	7	120.0	1.110	0.845		15	0.552	~	0.00	0.00
20.0	14	120.0	1.170	0.874	68	25	0.562	0.77	1.10	0.13
21.0	14	120.0	1.230	0.902	68	25	0.572	0.74	1.30	0.16
22.0	10	120.0	1.290	0.931	56	20	0.582	0.57	1.40	0.17
23.0	10	120.0	1.350	0.960	56	20	0.590	0.56	1.40	0.17
24.0	10	120.0	1.410	0.989	56	20	0.599	0.54	1.60	0.19
25.0	10	120.0	1.470	1.018	56	19	0.607	0.53	1.60	0.19
26.0	10	120.0	1.530	1.046	56	19	0.614	0.52	1.60	0.19
27.0	10	120.0	1.590	1.075	56	19	0.621	0.51	1.60	0.19
28.0	10	120.0	1.650	1.104	56	19	0.628	0.50	1.60	0.19
29.0	10	120.0	1.710	1.133	56	19	0.634	0.49	1.60	0.19
30.0	11	120.0	1.770	1.162		20	0.640	~	0.00	0.00
31.0	11	120.0	1.830	1.190		20	0.646	~	0.00	0.00
32.0	11	120.0	1.890	1.219		20	0.651	~	0.00	0.00
33.0	22	120.0	1.950	1.248	78	25	0.656	0.68	1.10	0.13
34.0	22	120.0	2.010	1.277	78	25	0.661	0.67	1.10	0.13
35.0	8	120.0	2.070	1.306		16	0.666	~	0.00	0.00
36.0	8	120.0	2.130	1.334		16	0.670	~	0.00	0.00
37.0	8	120.0	2.190	1.363		16	0.675	~	0.00	0.00
38.0	19	120.0	2.250	1.392	70	21	0.679	0.53	1.40	0.17
39.0	19	120.0	2.310	1.421	70	21	0.683	0.53	1.40	0.17
40.0	19	120.0	2.370	1.450	70	21	0.687	0.52	1.40	0.17
41.0	12	120.0	2.430	1.478		20	0.690	~	0.00	0.00
42.0	12	120.0	2.490	1.507		20	0.694	~	0.00	0.00
43.0	31	120.0	2.550	1.536		40	0.697	~	0.00	0.00
44.0	31	120.0	2.610	1.565		40	0.700	~	0.00	0.00
45.0	31	120.0	2.670	1.594		40	0.704	~	0.00	0.00
46.0	31	120.0	2.730	1.622		39	0.707	~	0.00	0.00
47.0	31	120.0	2.790	1.651		39	0.709	~	0.00	0.00
48.0	31	120.0	2.850	1.680		39	0.712	~	0.00	0.00
49.0	31	120.0	2.910	1.709		39	0.715	~	0.00	0.00
50.0	31	120.0	2.970	1.738		39	0.718	~	0.00	0.00

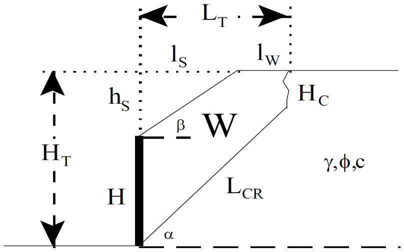
TOTAL SETTLEMENT = 2.5 INCHES

Figure 8

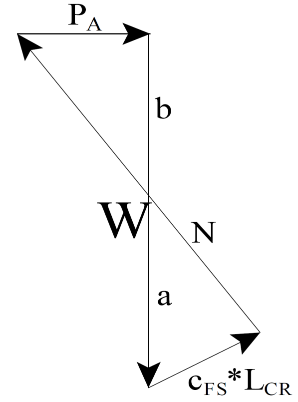
Retaining Wall Design with Transitioned Backfill
(Retaining Favorable Bedding Conditions)
(Vector Analysis)

Input:

Retaining Wall Height	(H)	12.00 feet
Slope Angle of Backfill	(b)	0.0 degrees
Height of Slope above Wall	(h_s)	0.0 feet
Horizontal Length of Slope	(l_s)	0.0 feet
Total Height (Wall + Slope)	(H_T)	12.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	34.8 degrees
Cohesion of Retained Soils	(c)	234.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(f_{FS})	24.9 degrees
	(c_{FS})	156.0 psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet²	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	4.7	61	7648.0	10.4	4272.5	3375.5	1237.9
46	4.5	60	7458.1	10.4	4081.2	3376.9	1305.7
47	4.4	58	7261.4	10.4	3900.0	3361.3	1367.6
48	4.3	56	7060.0	10.4	3729.0	3331.1	1423.5
49	4.2	55	6855.8	10.3	3567.7	3288.2	1473.6
50	4.1	53	6650.1	10.3	3415.7	3234.4	1517.8
51	4.1	52	6443.8	10.2	3272.5	3171.3	1556.3
52	4.0	50	6237.7	10.1	3137.6	3100.2	1589.1
53	4.0	48	6032.4	10.0	3010.3	3022.1	1616.3
54	4.0	47	5828.1	9.9	2890.1	2938.0	1637.9
55	3.9	45	5625.3	9.8	2776.6	2848.8	1654.0
56	3.9	43	5424.2	9.8	2669.1	2755.1	1664.6
57	3.9	42	5224.7	9.6	2567.1	2657.6	1669.7
58	3.9	40	5027.1	9.5	2470.3	2556.8	1669.3
59	3.9	39	4831.4	9.4	2378.2	2453.2	1663.4
60	3.9	37	4637.5	9.3	2290.2	2347.2	1652.1
61	4.0	36	4445.4	9.2	2206.2	2239.2	1635.3
62	4.0	34	4255.0	9.1	2125.5	2129.5	1612.9
63	4.0	33	4066.3	8.9	2047.9	2018.4	1584.9
64	4.1	31	3879.1	8.8	1972.9	1906.2	1551.3
65	4.2	30	3693.3	8.7	1900.3	1793.1	1512.0
66	4.2	28	3508.8	8.5	1829.5	1679.3	1467.0
67	4.3	27	3325.3	8.3	1760.3	1565.0	1416.1
68	4.4	25	3142.8	8.2	1692.2	1450.6	1359.3
69	4.5	24	2960.9	8.0	1624.7	1336.2	1296.6
70	4.7	22	2779.5	7.8	1557.5	1222.0	1227.9



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 1669.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall) At-Rest= γ*(1-sin(ϕ))

$EFP = 2*P_A/H^2$

EFP 23.2 pcf 53.7 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf 54 pcf

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3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: NDB

RETAINING WALL CALCULATION

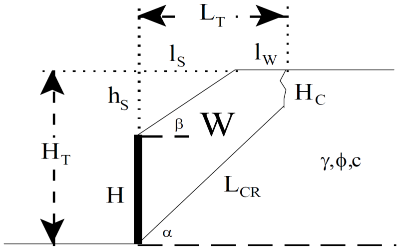
CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023 | PROJECT NO. W1547-06-01 | FIG. 9

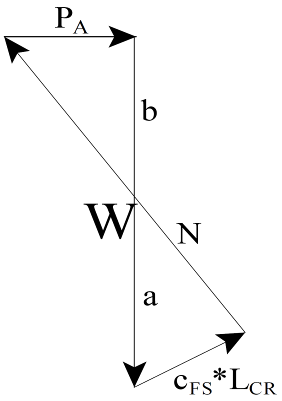
Retaining Wall Design with Transitioned Backfill
(Retaining Adverse Bedding Conditions)
(Vector Analysis)

Input:

Retaining Wall Height	(H)	12.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	12.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	14.0	degrees
Cohesion of Retained Soils	(c)	120.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	9.4	degrees
	(c_{FS})	80.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	1.5	71	8852.7	14.8	2008.2	6844.5	4893.5
46	1.5	68	8550.7	14.6	1929.0	6621.7	4911.0
47	1.5	66	8258.2	14.3	1855.3	6403.0	4924.3
48	1.5	64	7974.7	14.1	1786.4	6188.3	4933.5
49	1.5	62	7699.6	13.9	1722.0	5977.5	4938.5
50	1.5	59	7432.3	13.7	1661.8	5770.5	4939.4
51	1.5	57	7172.3	13.5	1605.3	5567.0	4936.2
52	1.5	55	6919.3	13.3	1552.3	5367.1	4928.8
53	1.5	53	6672.9	13.1	1502.4	5170.5	4917.4
54	1.5	51	6432.5	12.9	1455.4	4977.1	4901.7
55	1.5	50	6197.9	12.8	1411.1	4786.7	4881.7
56	1.6	48	5968.7	12.6	1369.3	4599.4	4857.3
57	1.6	46	5744.5	12.4	1329.8	4414.8	4828.5
58	1.6	44	5525.2	12.3	1292.3	4232.9	4795.0
59	1.6	42	5310.3	12.1	1256.7	4053.6	4756.7
60	1.6	41	5099.7	12.0	1223.0	3876.7	4713.3
61	1.7	39	4893.0	11.8	1190.8	3702.2	4664.8
62	1.7	38	4690.1	11.7	1160.1	3529.9	4610.7
63	1.7	36	4490.6	11.5	1130.8	3359.8	4550.9
64	1.8	34	4294.3	11.4	1102.7	3191.6	4484.9
65	1.8	33	4101.2	11.2	1075.7	3025.4	4412.4
66	1.9	31	3910.8	11.1	1049.7	2861.1	4332.9
67	1.9	30	3723.0	11.0	1024.5	2698.5	4246.1
68	2.0	28	3537.7	10.8	1000.0	2537.7	4151.3
69	2.0	27	3354.6	10.7	976.2	2378.4	4047.9
70	2.1	25	3173.5	10.5	952.8	2220.7	3935.2



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 4939.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall) At-Rest= γ*(1-sin(ϕ))

$EFP = 2*P_A/H^2$

EFP 68.6 pcf 94.8 pcf

Design Wall for an Equivalent Fluid Pressure: **69 pcf** **95** pcf

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3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: NDB

RETAINING WALL CALCULATION

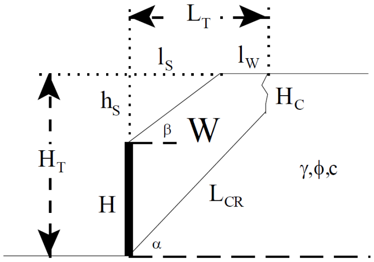
CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023 | PROJECT NO. W1547-06-01 | FIG. 10

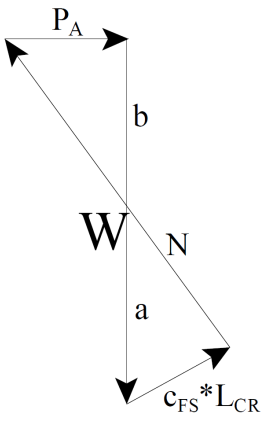
Retaining Wall Design with Transitioned Backfill
(Retaining Alluvium or Engineered Fill)
(Vector Analysis)

Input:

Retaining Wall Height	(H)	12.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	12.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	29.8	degrees
Cohesion of Retained Soils	(c)	92.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	20.9	degrees
	(c_{FS})	61.3	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	1.6	71	8842.5	14.7	2066.1	6776.4	3031.6
46	1.6	68	8545.2	14.5	1961.0	6584.1	3084.7
47	1.5	66	8256.6	14.3	1864.8	6391.9	3131.7
48	1.5	64	7976.4	14.1	1776.4	6200.0	3173.1
49	1.5	62	7704.0	13.9	1695.0	6009.0	3208.9
50	1.5	60	7439.2	13.8	1620.0	5819.2	3239.3
51	1.5	57	7181.3	13.6	1550.5	5630.8	3264.4
52	1.4	55	6930.1	13.4	1486.2	5443.9	3284.3
53	1.4	53	6685.2	13.2	1426.5	5258.7	3299.2
54	1.4	52	6446.3	13.1	1371.0	5075.3	3308.9
55	1.4	50	6213.0	12.9	1319.3	4893.7	3313.6
56	1.4	48	5984.9	12.8	1270.9	4714.0	3313.4
57	1.4	46	5761.9	12.6	1225.8	4536.1	3308.1
58	1.4	44	5543.5	12.5	1183.4	4360.1	3297.9
59	1.4	43	5329.6	12.3	1143.7	4185.9	3282.5
60	1.5	41	5119.9	12.2	1106.3	4013.6	3262.1
61	1.5	39	4914.1	12.0	1071.2	3843.0	3236.4
62	1.5	38	4712.1	11.9	1037.9	3674.1	3205.5
63	1.5	36	4513.5	11.8	1006.5	3507.0	3169.1
64	1.5	35	4318.2	11.6	976.8	3341.4	3127.2
65	1.6	33	4126.0	11.5	948.5	3177.4	3079.5
66	1.6	31	3936.6	11.4	921.6	3015.0	3025.8
67	1.6	30	3750.0	11.3	896.0	2854.0	2966.0
68	1.7	29	3565.8	11.1	871.4	2694.4	2899.8
69	1.7	27	3383.9	11.0	847.8	2536.2	2826.9
70	1.8	26	3204.2	10.9	825.0	2379.2	2746.9



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 3313.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall) At-Rest= γ*(1-sin(ϕ))

$EFP = 2*P_A/H^2$

EFP 46.0 pcf 62.9 pcf

Design Wall for an Equivalent Fluid Pressure: 46 pcf 63 pcf

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3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
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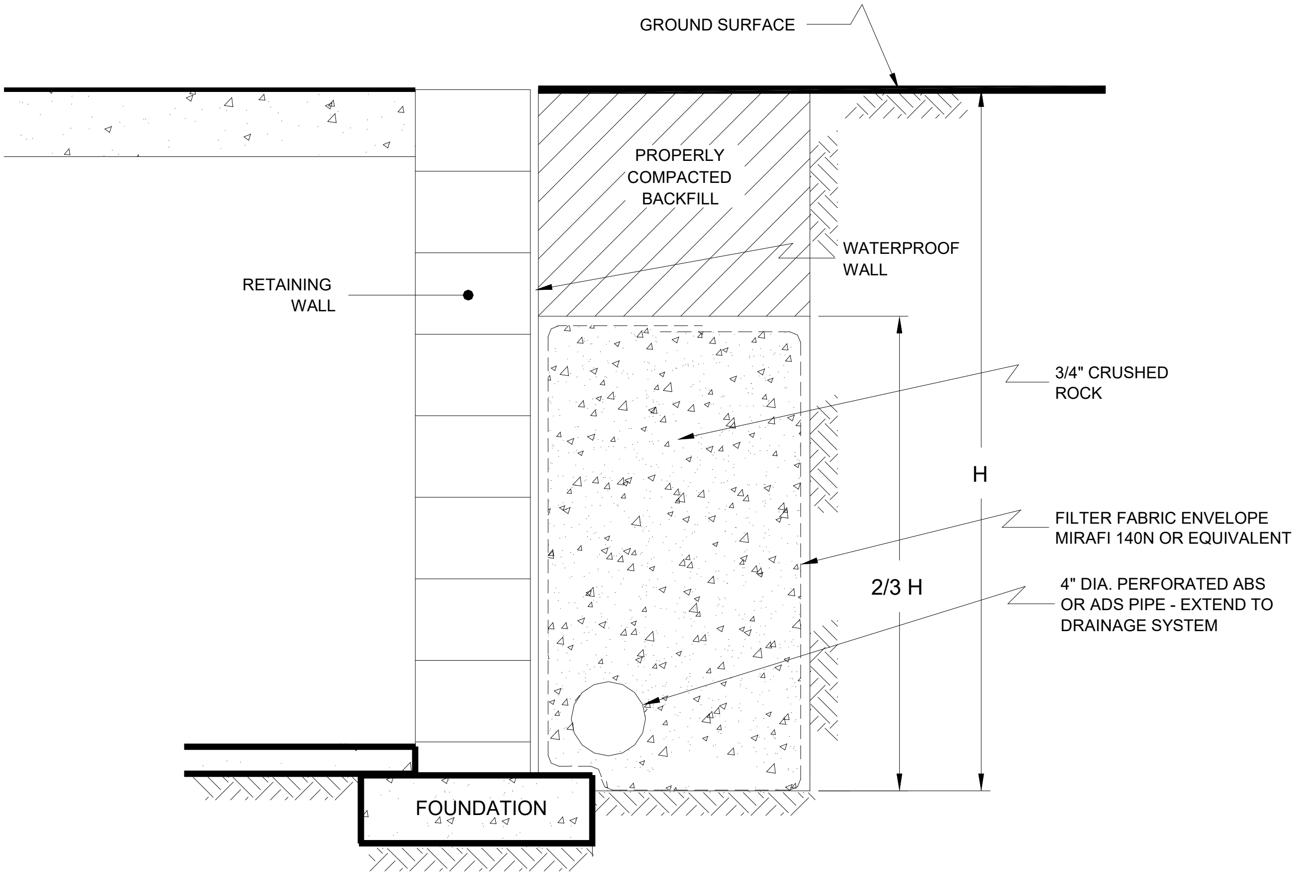
DRAFTED BY: JMH CHECKED BY: NDB

RETAINING WALL CALCULATION

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023 PROJECT NO. W1547-06-01 FIG. 11



NO SCALE

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RETAINING WALL DRAIN DETAIL

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
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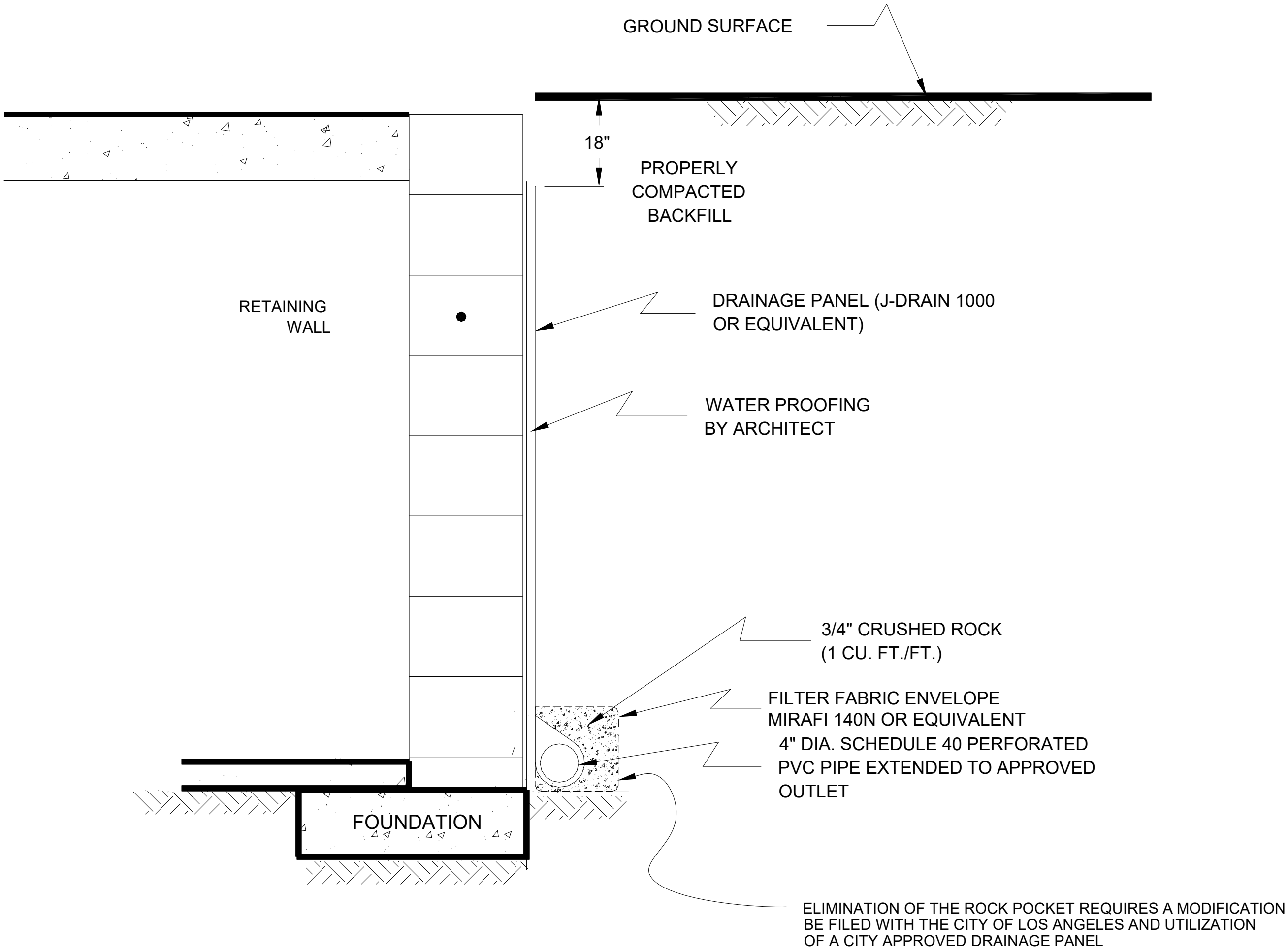
JAN. 2023 | PROJECT NO. W1547-06-01 | FIG. 12



NO SCALE

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RETAINING WALL DRAIN DETAIL

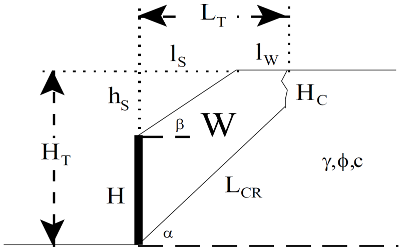
CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023 | PROJECT NO. W1547-06-01 | FIG. 13

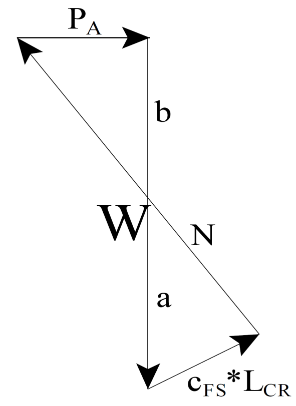
Shoring Design with Transitioned Backfill
(Retaining Favorable Bedding Conditions)
(Vector Analysis)

Input:

Shoring Height	(H)	14.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	14.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	34.8	degrees
Cohesion of Retained Soils	(c)	234.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	29.1	degrees
	(c_{FS})	187.2	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	6.7	75	9405.6	10.3	6116.9	3288.6	938.4
46	6.5	74	9301.4	10.5	5881.2	3420.2	1040.8
47	6.2	73	9157.1	10.6	5643.6	3513.6	1136.6
48	6.0	72	8983.0	10.7	5409.3	3573.7	1225.3
49	5.9	70	8786.8	10.8	5181.7	3605.1	1306.8
50	5.7	69	8574.2	10.8	4962.6	3611.6	1381.0
51	5.6	67	8349.6	10.8	4752.9	3596.7	1447.7
52	5.5	65	8116.2	10.8	4553.1	3563.1	1507.0
53	5.4	63	7876.5	10.8	4363.1	3513.4	1558.8
54	5.3	61	7632.4	10.8	4182.6	3449.8	1603.2
55	5.2	59	7385.3	10.7	4011.3	3374.0	1640.2
56	5.2	57	7136.4	10.7	3848.7	3287.7	1669.8
57	5.1	55	6886.4	10.6	3694.2	3192.2	1692.0
58	5.1	53	6636.2	10.5	3547.3	3088.8	1706.9
59	5.1	51	6386.0	10.4	3407.5	2978.5	1714.5
60	5.1	49	6136.3	10.3	3274.0	2862.3	1714.8
61	5.1	47	5887.3	10.2	3146.4	2740.9	1707.7
62	5.1	45	5639.1	10.0	3024.1	2615.0	1693.4
63	5.2	43	5391.9	9.9	2906.6	2485.3	1671.7
64	5.2	41	5145.6	9.8	2793.2	2352.4	1642.6
65	5.3	39	4900.2	9.6	2683.4	2216.8	1606.2
66	5.4	37	4655.7	9.5	2576.7	2079.0	1562.4
67	5.5	35	4411.8	9.3	2472.5	1939.3	1511.1
68	5.6	33	4168.4	9.1	2370.1	1798.3	1452.4
69	5.7	31	3925.3	8.9	2268.9	1656.4	1386.2
70	5.8	29	3682.3	8.7	2168.3	1514.0	1312.6



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 1714.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 17.5 pcf

Design Shoring for an Equivalent Fluid Pressure: 25 pcf

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ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH CHECKED BY: NDB

SHORING WALL CALCULATION

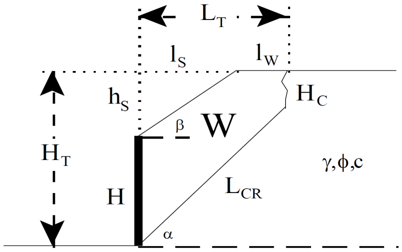
CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS, CALIFORNIA

JAN. 2023 PROJECT NO. W1547-06-01 FIG. 14

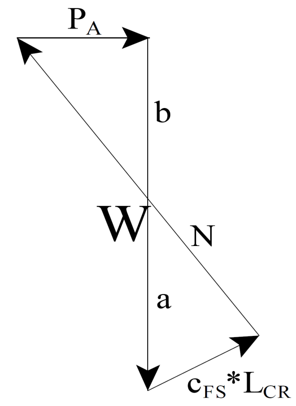
Shoring Design with Transitioned Backfill
(Retaining Adverse Bedding Conditions)
(Vector Analysis)

Input:

Shoring Height	(H)	14.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	14.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	14.0	degrees
Cohesion of Retained Soils	(c)	120.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	11.3	degrees
	(c_{FS})	96.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	1.9	96	12019.9	17.1	2897.6	9122.3	6088.4
46	1.9	93	11611.0	16.8	2779.6	8831.4	6119.6
47	1.9	90	11214.8	16.6	2669.8	8545.0	6144.6
48	1.9	87	10830.5	16.3	2567.5	8263.0	6163.5
49	1.9	84	10457.5	16.1	2472.0	7985.5	6176.3
50	1.9	81	10094.9	15.8	2382.6	7712.3	6183.1
51	1.9	78	9742.3	15.6	2299.0	7443.4	6183.9
52	1.9	75	9399.0	15.4	2220.5	7178.5	6178.8
53	1.9	73	9064.5	15.2	2146.8	6917.7	6167.7
54	1.9	70	8738.2	15.0	2077.5	6660.7	6150.5
55	1.9	67	8419.6	14.8	2012.2	6407.4	6127.3
56	1.9	65	8108.3	14.6	1950.5	6157.7	6097.8
57	1.9	62	7803.8	14.4	1892.3	5911.5	6061.9
58	2.0	60	7505.8	14.2	1837.2	5668.6	6019.6
59	2.0	58	7213.8	14.0	1784.9	5428.9	5970.4
60	2.0	55	6927.6	13.9	1735.3	5192.3	5914.4
61	2.0	53	6646.6	13.7	1688.0	4958.6	5851.0
62	2.1	51	6370.7	13.5	1643.0	4727.7	5780.2
63	2.1	49	6099.5	13.3	1600.0	4499.5	5701.4
64	2.2	47	5832.6	13.2	1558.8	4273.8	5614.2
65	2.2	45	5569.8	13.0	1519.2	4050.7	5518.3
66	2.3	42	5310.9	12.8	1481.0	3829.9	5413.0
67	2.3	40	5055.4	12.7	1444.1	3611.3	5297.9
68	2.4	38	4803.3	12.5	1408.3	3394.9	5172.2
69	2.5	36	4554.1	12.3	1373.4	3180.7	5035.2
70	2.6	34	4307.6	12.2	1339.1	2968.5	4886.1



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 6183.9 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 63.1 pcf

Design Shoring for an Equivalent Fluid Pressure: 63 pcf

GEOCON
WEST, INC.

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WEST HILLS, CALIFORNIA

JAN. 2023 PROJECT NO. W1547-06-01 FIG. 15

BORING PERCOLATION TEST FIELD LOG

Date: Tuesday, April 19, 2022
Project Number: W1547-06-01
Project Location: Chaminade HS
Earth Description: SM
Tested By: Name
Liquid Description: Water
Measurement Method: Sounder

Boring/Test Number: Boring 4 / Test 1
Diameter of Boring: 8 inches
Diameter of Casing: 2 inches
Depth of Boring: 8 feet
Depth to Invert of BMP: 4 feet
Depth to Water Table: 30 feet
Depth to Initial Water Depth (d_1): 48 inches

Start Time for Pre-Soak: 9:30 AM
Start Time for Standard: 10:30 AM

Water Remaining in Boring (Y/N): Yes
Standard Time Interval Between Readings: 30 min

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	10:30 AM	11:00 AM	30	4.2	
2	11:00 AM	11:30 AM	30	3.7	
3	11:30 AM	12:00 PM	30	2.6	
4	12:00 PM	12:30 PM	30	1.8	
5	12:30 PM	1:00 PM	30	1.4	
6	1:00 PM	1:30 PM	30	1.6	Stabilized Readings
7	1:30 PM	2:00 PM	30	1.4	Achieved with Readings
8	2:00 PM	2:30 PM	30	1.4	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches
Test Section Height, h: 48.0 inches

$Test\ Section\ Surface\ Area, A = 2\pi rh + \pi r^2$

A = **1257** in^2

$Discharged\ Water\ Volume, V = \pi r^2 \Delta d$

$Percolation\ Rate = \left(\frac{V/A}{\Delta T}\right)$

Reading 6	V =	**78**	in^3	Percolation Rate =	**0.12**	inches/hour
Reading 7	V =	**72**	in^3	Percolation Rate =	**0.12**	inches/hour
Reading 8	V =	**72**	in^3	Percolation Rate =	**0.12**	inches/hour

Measured Percolation Rate = **0.12** inches/hour

Reduction Factors

Boring Percolation Test, RF_t = 1
Site Variability, RF_v = 1
Long Term Siltation, RF_s = 1

$Total\ Reduction\ Factor, RF = RF_t + RF_v + RF_s$

Total Reduction Factor = 3

Design Infiltration Rate

$Design\ Infiltration\ Rate = Measured\ Percolation\ Rate\ /RF$

Design Infiltration Rate = **0.04** inches/hour

FIGURE 16

BORING PERCOLATION TEST FIELD LOG

Date: Tuesday, April 19, 2022
Project Number: W1547-06-01
Project Location: Chaminade HS
Earth Description: CL
Tested By: Name
Liquid Description: Water
Measurement Method: Sounder

Boring/Test Number: Boring 7 / Test 1
Diameter of Boring: 8 inches
Diameter of Casing: 2 inches
Depth of Boring: 5 feet
Depth to Invert of BMP: 2 feet
Depth to Water Table: 30 feet
Depth to Initial Water Depth (d_1): 24 inches

Start Time for Pre-Soak: 9:30 AM
Start Time for Standard: 10:30 AM

Water Remaining in Boring (Y/N): Yes
Standard Time Interval Between Readings: 30 min

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	10:30 AM	11:00 AM	30	0.6	
2	11:00 AM	11:30 AM	30	0.4	
3	11:30 AM	12:00 PM	30	0.4	
4	12:00 PM	12:30 PM	30	0.4	
5	12:30 PM	1:00 PM	30	0.4	
6	1:00 PM	1:30 PM	30	0.4	Stabilized Readings
7	1:30 PM	2:00 PM	30	0.4	Achieved with Readings
8	2:00 PM	2:30 PM	30	0.4	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches
Test Section Height, h: 36.0 inches

$Test\ Section\ Surface\ Area, A = 2\pi rh + \pi r^2$

A = **955** in^2

$Discharged\ Water\ Volume, V = \pi r^2 \Delta d$

$Percolation\ Rate = \left(\frac{V/A}{\Delta T}\right)$

Reading 6 V = **18** in^3
Reading 7 V = **18** in^3
Reading 8 V = **18** in^3

Percolation Rate = **0.04** inches/hour
Percolation Rate = **0.04** inches/hour
Percolation Rate = **0.04** inches/hour

Measured Percolation Rate = **0.04** inches/hour

Reduction Factors

Boring Percolation Test, RF_t = 1
Site Variability, RF_v = 1
Long Term Siltation, RF_s = 1

$Total\ Reduction\ Factor, RF = RF_t + RF_v + RF_s$

Total Reduction Factor = 3

Design Infiltration Rate

$Design\ Infiltration\ Rate = Measured\ Percolation\ Rate\ /RF$

Design Infiltration Rate = **0.01** inches/hour

FIGURE 17

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on April 18, 2022, and April 19, 2022 by excavating twelve 7-inch diameter borings to depths ranging between approximately 6½ feet to 51 feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were obtained from the borings and Standard Penetration Tests (SPTs) were performed in borings B1, B3 and B5.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 and A12. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The locations of the borings are shown on Figures 2B through 2D.

PROJECT NO. W1547-06-01

BORING 1

ELEV. (MSL.) ________ DATE COMPLETED **04/18/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 3.5" BASE: 3.5"** **ARTIFICIAL FILL** Silty Sand, poorly graded, loose, blackish brown, slightly moist, fine-grained, some medium-grained.			
2	B1@2.5'			ML	**ALLUVIUM** Sandy Silt, soft, moist, brown to dark brown.	9	107.9	11.2
4				SM	Silty Sand, poorly graded, very loose, dry, brown, fine-grained.			
6	B1@5'					2		
8	B1@7.5'				- loose	14	88.5	7.9
10	B1@10'			CL	Sandy Clay, soft, brown, moist. - increase in silt	4		
12	B1@12.5'			CL	Clay with Sand, firm, moist, brown, fine-grained.	20	105.6	13.7
14	B1@15'				- loose	7		
18	B1@17.5'				- medium dense, dark brown	20		
20	B1@20'			SM	Silty Sand, poorly graded, medium dense, slightly moist, brown, fine-grained, some medium-grained.	14		
22	B1@22.5'			ML	Sandy Silt, stiff, slightly moist to moist, olive brown with oxidation.	34	110.9	16.7
24	B1@25'				- firm, slightly moist, dark brown, fine-grained	10		
28	B1@27.5'					21		
				CL	Sandy Clay, firm, moist, olive brown, some oxidation.			

W1547-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 2

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING 1

ELEV. (MSL.) _______ DATE COMPLETED **04/18/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'			CL		11		
32	B1@32.5'		▼	SP	Sand, poorly graded, medium dense, saturated, brown, fine- to medium-grained, some coarse-grained.	40	107.8	19.8
34	B1@35'			CL	Clay, soft, wet, olive brown.	8		
36								
38	B1@37.5'			SP	Sand, poorly graded, medium dense, saturated, brown with dark brown mottles, some sandstone clasts.	35	117.6	15.2
40	B1@40'			CL	Sandy Clay, firm, wet, olive brown.	12		
42	B1@42.5'				**MONTEREY FORMATION** Siltstone, poorly bedded, highly weathered, olive brown with oxidation mottles, soft.	50 (4")		
44								
46	B1@45'					31		
48	B1@47.5'					61	104.3	23.0
50	B1@50'					31		

Total depth of boring: 51 feet
Fill to 2 feet.
Groundwater encountered at 32 feet.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A1,
Log of Boring 1, Page 2 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. W1547-06-01

BORING 2

ELEV. (MSL.) _______ DATE COMPLETED **04/18/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 6" BASE: NONE** **ARTIFICIAL FILL** Sandy Silt, soft, slightly moist, dark brown, gravel throughout.			
3	B2@3'					8	89.8	12.3
5					**ALLUVIUM** Silty Sand, poorly graded, loose, slightly moist, brown, fine-grained.			
6	B2@6'			SM		9	110.8	4.9
8					- decrease in silt content			
9	B2@9'					10	119.8	13.0
9.5					Sandy Silt, soft, moist, brown.			
12	B2@12'			ML		11	100.5	15.0
15	B2@15'					13	109.3	13.3

Total depth of boring: 15.5 feet
Fill to 5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1547-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. W1547-06-01

BORING 3

ELEV. (MSL.) _______ DATE COMPLETED **04/18/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				**AC: 5" BASE: NONE** **ARTIFICIAL FILL** Sandy Clay, soft, moist, dark brown.			
2	B3@2.5'		ML	**ALLUVIUM** Sandy Silt, soft, moist, brown.	8	111.3	14.0
4				Sand with Silt, poorly graded, loose, slightly moist, brown, fine-grained.			
6	B3@5'		SP-SM		4		
8	B3@7.5'		SC	Clayey Sand, poorly graded, loose, slightly moist, brown, fine-grained.	14	115.3	14.4
10	B3@10'		SP-SM	Sand with Silt, poorly graded, loose, slightly moist, brown, fine-grained.	4		
12	B3@12.5'			**MONTEREY FORMATION** Silty Sandstone, poorly bedded, highly weathered, dark grayish brown, dry.	55		
14							
16	B3@15'				30		
18	B3@17.5'			- reddish brown	50 (4")	124.5	15.1
20	B3@20'				30		

Total depth of boring: 21 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A3,
Log of Boring 3, Page 1 of 1

W1547-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. W1547-06-01

BORING 4

ELEV. (MSL.) _______ DATE COMPLETED **04/18/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-4'	☒ ▨			**AC: 4" BASE: NONE** **ARTIFICIAL FILL** Sandy Silt, soft, moist, dark brown, some gravel.			
2		☒ ▨						
	B4@3'	■ ▨				11	101.2	10.6
4				SM	**ALLUVIUM** Silty Sand, poorly graded, loose, slightly moist, brown, fine-grained, some medium-grained.			
6	B4@6'	■				12	98.7	9.2
8				ML	Silt with Sand, firm, moist, dark brown, fine-grained.			
	B4@9'	■				19	113.6	16.5
10								
12	B4@12'	■				15	104.9	13.1
14								
	B4@15'	■			**MONTEREY FORMATION** Siltstone, poorly bedded, highly weathered, dark grayish brown, soft, dry.	46	119.4	13.0

Total depth of boring: 15.5 feet
Fill to 3.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1547-06-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
☒ ... DISTURBED OR BAG SAMPLE
◧ ... STANDARD PENETRATION TEST
◪ ... CHUNK SAMPLE
■ ... DRIVE SAMPLE (UNDISTURBED)
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. W1547-06-01

BORING 5

ELEV. (MSL.) _______ DATE COMPLETED **04/18/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Silty Sand, poorly graded, medium dense, brown, slightly moist, fine- to medium-grained, gravel.			
2	B5@2.5'				**ALLUVIUM** Sandy Clay, stiff, slightly moist, dark gray, bedrock fragments.	39	97.3	25.5
4								
	B5@5'			CL	- tree roots	21		
6								
	B5@7.5'					29	94.5	17.0
8								
10	B5@10'				**MONTEREY FORMATION** Sandstone, poorly bedded, highly weathered, dark grayish brown, hard, dry.	50 (2")		
12								
	B5@12.5'				- reddish brown	50 (6")	118.5	13.4
14								
	B5@15'					46		
16								

Total depth of boring: 16 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1547-06-01 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. W1547-06-01

BORING 6

ELEV. (MSL.) _______ DATE COMPLETED **04/18/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 3" BASE: 10"** **ARTIFICIAL FILL**			
2	B6@2'				Silty Sand, poorly graded, medium dense, dry, grayish brown, fine- to medium-grained, gravel.	36	116.6	15.1
					Sandy Clay, hard, moist, bluish gray and dark gray, some gravel.			
4								
	B6@5'				- dark gray	60	111.6	12.6
6					**ALLUVIUM**			
	B6@7'			SM	Silty Sand, poorly graded, loose, slightly moist, brown, fine-grained.	16	101.8	5.7
8								
					MONTEREY FORMATION			
10	B6@10'				Siltstone, poorly bedded, highly weathered, dark grayish brown, soft, dry.	44	109.2	15.1
12	B6@12'				Sandstone, poorly bedded, highly weathered, reddish brown, hard, dry.	50 (6")	105.6	12.4
14	B6@14'					50 (6")	102.7	15.9

Total depth of boring: 14.5 feet
Fill to 6 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

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Figure A6,
Log of Boring 6, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
☒ ... DISTURBED OR BAG SAMPLE
◧ ... STANDARD PENETRATION TEST
◪ ... CHUNK SAMPLE
■ ... DRIVE SAMPLE (UNDISTURBED)
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. W1547-06-01

BORING 7

ELEV. (MSL.) ________ DATE COMPLETED **04/19/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, poorly graded, medium dense, slightly moist, brown to dark brown, fine-grained, some gravel.			
2	B7@2.5'			CL	**ALLUVIUM** Sandy Clay, stiff, moist, blackish brown, some bedrock fragments throughout.	34	108.8	19.5
4	B7@5'					50 (5")	100.4	23.3
6 8 10	B7@10'				**MONTEREY FORMATION** Sandstone, thinly bedded, slightly weathered, olive brown, some oxidation, hard, dry.	75	73.4	44.7

Total depth of boring: 10.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A7,
Log of Boring 7, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ☒ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. W1547-06-01

BORING 8

ELEV. (MSL.) ______ DATE COMPLETED **04/19/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 5.5" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, poorly graded, medium dense, slightly moist, brown, fine-grained, some medium-grained.			
2								
	B8@3'				**MONTEREY FORMATION** Sandstone, thinly bedded, slightly weathered, light olive brown, dry, hard, some oxidation striping.	63		
4								
6	B8@6'					87	86.0	16.1

Total depth of boring: 6.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Concrete patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A8,
Log of Boring 8, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. W1547-06-01

BORING 9

ELEV. (MSL.) ________ DATE COMPLETED **04/19/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4.5" BASE: 4.5"** **ARTIFICIAL FILL** Sandy Silt, firm, moist, dark brown.			
2	B9@2'				**MONTEREY FORMATION** Sandstone with Siltstone Interbeds, thinly bedded, slighty weathered, light olive brown, dry, hard.	50 (4")	78.9	43.9
4								
6	B9@6'				Siltstone with Sandstone Interbeds, poorly bedded to thinly bedded, slightly weathered, olive brown with oxidation mottles.	40	82.1	40.3

Total depth of boring: 6.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Concrete patched with black dye.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A9,
Log of Boring 9, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING 10

ELEV. (MSL.) _______ DATE COMPLETED **04/19/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 5" BASE: 6"**			
				ML	**ARTIFICIAL FILL** Silty Sand, poorly graded, medium dense, slightly moist, brown, fine- to medium-grained.			
2								
	B10@3' ■				**COLLUVIUM** Sandy Silt, hard, moist, olive brown, sandstone clasts throughout.	48		
4					**MONTEREY FORMATION** Siltstone, thinly bedded, slightly weathered, light olive brown, dry, hard.			
6	B10@6' ■					40	81.5	39.2
8					- Sandstone Interbeds and oxidation striping			
	B10@9' ■					52	78.6	42.7

Total depth of boring: 9.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Concrete patched with black dye.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A10,
Log of Boring 10, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. W1547-06-01

BORING 11

ELEV. (MSL.) _______ DATE COMPLETED **04/19/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-10'				**AC: 6" BASE: 3"** **ARTIFICIAL FILL** Sandy Silt, firm, slightly moist, dark brown, some gravel.			
2	B11@3'			CL	**COLLUVIUM** Sandy Clay, stiff, slightly moist, dark brown with olive brown mottles.	40		
4	B11@6'				**MONTEREY FORMATION** Siltstone, poorly bedded, highly weathered, olive brown with oxidation striping, slightly moist, soft.	59	85.8	34.6
8	B11@9'				Sandstone, thinly bedded, slightly weathered, light gray to olive brown, dry, hard.	66	74.3	29.5
10					Total depth of boring: 10 feet Fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. Concrete patched with black dye. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

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Figure A11,
Log of Boring 11, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. W1547-06-01

BORING 12

ELEV. (MSL.) _______ DATE COMPLETED **04/19/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 5.5" BASE: NONE** **ARTIFICIAL FILL** Sandy Silt, soft, moist, olive brown with dark brown mottles, some gravel and bedrock fragments.			
2								
	B12@3'					14	89.2	32.2
4					**ALLUVIUM** Sandy Silt, firm, moist, brown.			
6	B12@6'			ML	- dark brown to brown	10		
8								
	B12@9'				- increase in sand content	19	92.1	8.3
10					Clay, hard, moist, dark brown, trace sand, calcium stringers throughout.			
12	B12@12'					44	105.3	21.8
14			▼	CL				
	B12@15'					20	98.5	22.9
16								
18								
					Clayey Sand, poorly graded, medium dense, moist, olive brown, fine-grained.			
20	B12@20'			SC	- bluish green	39	106.1	19.1
22								
					Sand, poorly graded, dense, saturated, dark gray, fine- to medium-grained, some silt.			
24				SP				
	B12@25'				- strong sulfur odor	62	104.4	20.7

Total depth of boring: 25.5 feet
Fill to 3.5 feet.
Groundwater encountered at 14.6 feet.
Backfilled with soil cuttings and tamped.
Concrete patched with black dye.

*Penetration resistance for 140-pound hammer falling 30 inches by

W1547-06-01 BORING LOGS.GPJ

Figure A12,
Log of Boring 12, Page 1 of 2

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. W1547-06-01

BORING 12

ELEV. (MSL.) _______ DATE COMPLETED **04/19/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					auto-hammer. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

Figure A12,
Log of Boring 12, Page 2 of 2

W1547-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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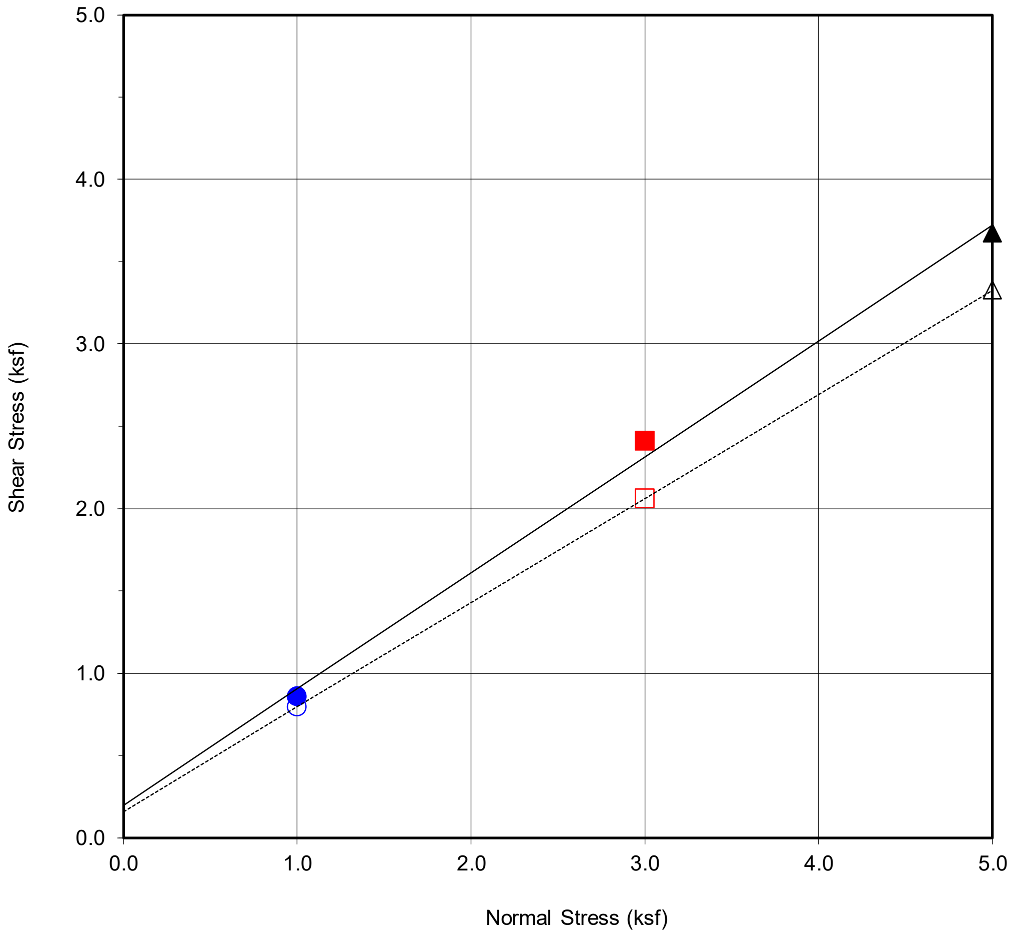
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the “American Society for Testing and Materials (ASTM)”, or other suggested procedures. Selected samples were tested for direct shear strength, consolidation characteristics, Atterburg limits, grain size, corrosivity, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B27. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B1**
Sample No.	**B1@2.5'**
Depth (ft)	**2.5**
Sample Type:	Ring

Soil Identification:

Sandy Silt (ML)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	200	35.2
Ultimate	162	32.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.86	🟥 2.41	▲ 3.68
Shear Stress @ End of Test (ksf)	○ 0.80	□ 2.06	△ 3.33
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.5	12.1	13.6
Initial Dry Density (pcf)	109.0	110.7	111.2
Initial Degree of Saturation (%)	61.6	62.6	71.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.7	14.8	14.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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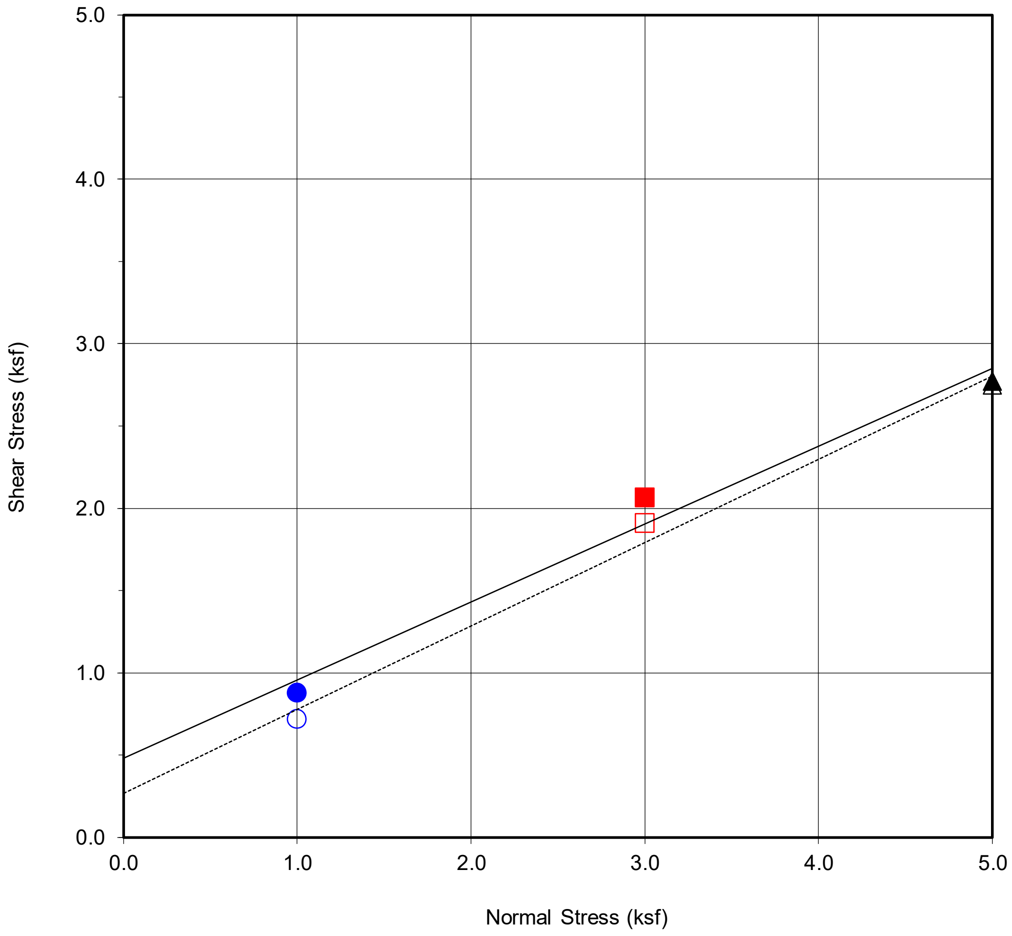
Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B1



Boring No.	**B1**
Sample No.	**B1@17.5'**
Depth (ft)	**17.5'**
Sample Type:	Ring

Soil Identification:

Clay with Sand (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	482	25.4
Ultimate	271	26.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.88	■ 2.06	▲ 2.77
Shear Stress @ End of Test (ksf)	○ 0.72	□ 1.91	△ 2.75
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	16.1	17.0	19.0
Initial Dry Density (pcf)	116.0	110.3	107.3
Initial Degree of Saturation (%)	95.7	87.0	89.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.1	17.8	17.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
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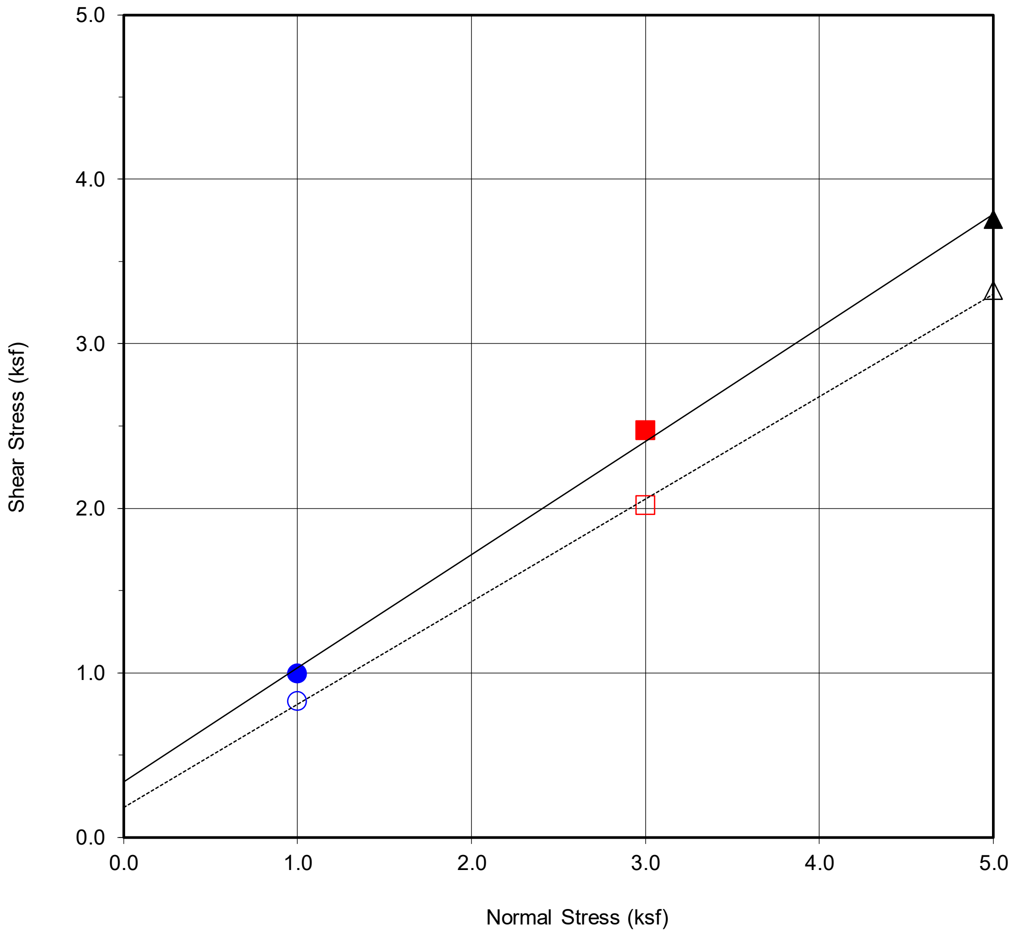
Figure B2



Boring No.	**B1**
Sample No.	**B1@27.5'**
Depth (ft)	**27.5'**
Sample Type:	Ring

Soil Identification:		
Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	338	34.6
Ultimate	184	32.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.00	■ 2.47	▲ 3.76
Shear Stress @ End of Test (ksf)	○ 0.83	□ 2.02	△ 3.32
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	19.6	20.6	20.6
Initial Dry Density (pcf)	106.3	104.0	105.0
Initial Degree of Saturation (%)	90.3	89.6	91.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.0	20.9	20.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JMH

Project No.: W1547-06-01

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7500 CHAMINADE AVENUE
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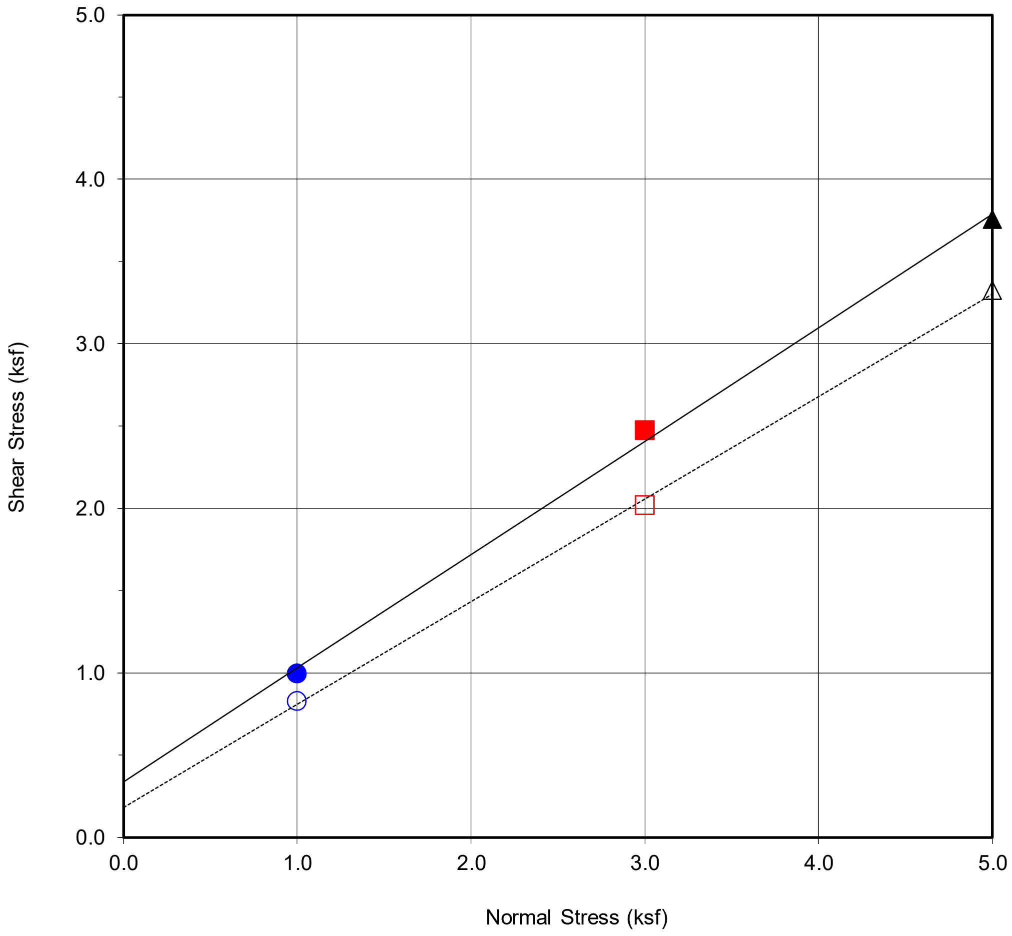
Figure B3



Boring No.	**B1**
Sample No.	**B1@42.5'**
Depth (ft)	**42.5'**
Sample Type:	Ring

Soil Identification:

Siltstone

Strength Parameters

	C (psf)	ϕ (°)
Peak	338	34.6
Ultimate	184	32.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.00	■ 2.47	▲ 3.76
Shear Stress @ End of Test (ksf)	○ 0.83	□ 2.02	△ 3.32
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	16.9	22.6	16.7
Initial Dry Density (pcf)	115.0	113.8	116.7
Initial Degree of Saturation (%)	98.0	126.7	101.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.1	18.5	16.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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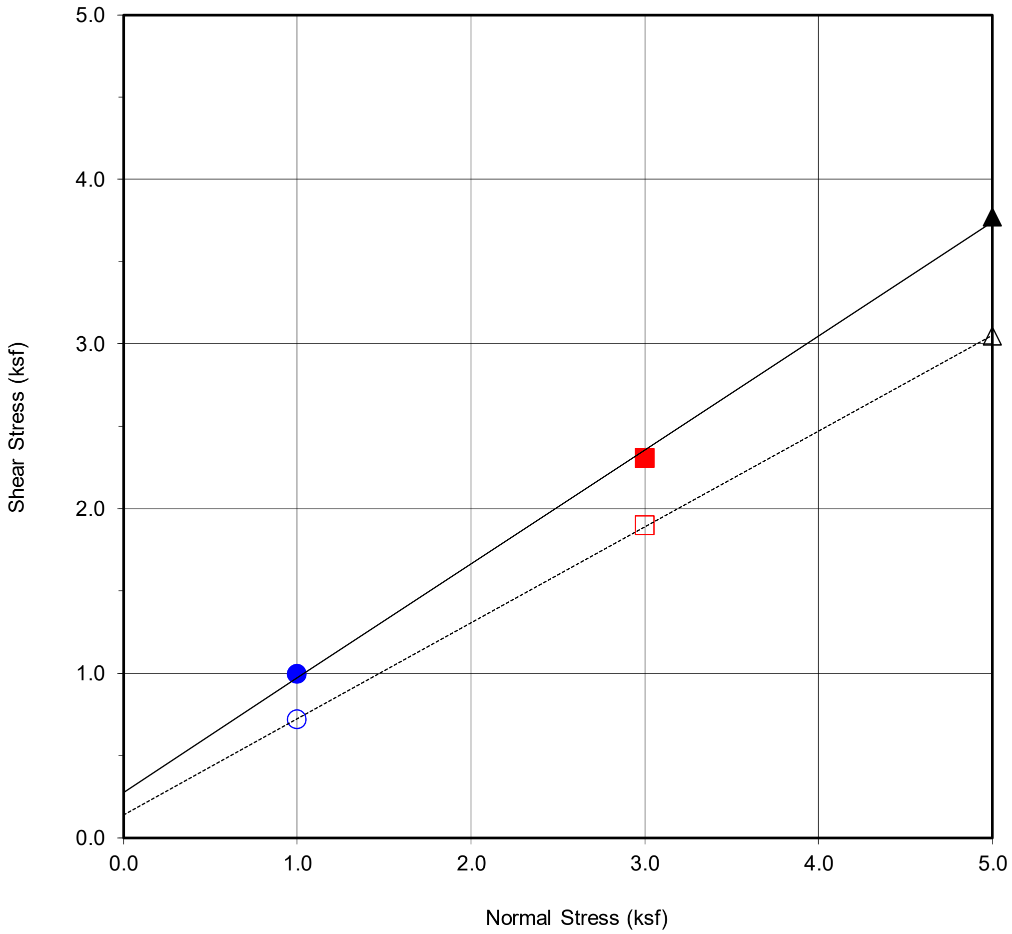
Figure B4



Boring No.	**B3**
Sample No.	**B3@12.5'**
Depth (ft)	**12.5'**
Sample Type:	Ring

Soil Identification:		
Silty Sandstone		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	276	34.7
Ultimate	141	30.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.00	■ 2.30	▲ 3.77
Shear Stress @ End of Test (ksf)	○ 0.72	□ 1.90	△ 3.05
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	14.1	16.9	15.3
Initial Dry Density (pcf)	115.1	115.5	114.1
Initial Degree of Saturation (%)	81.9	99.2	86.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.9	18.4	16.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

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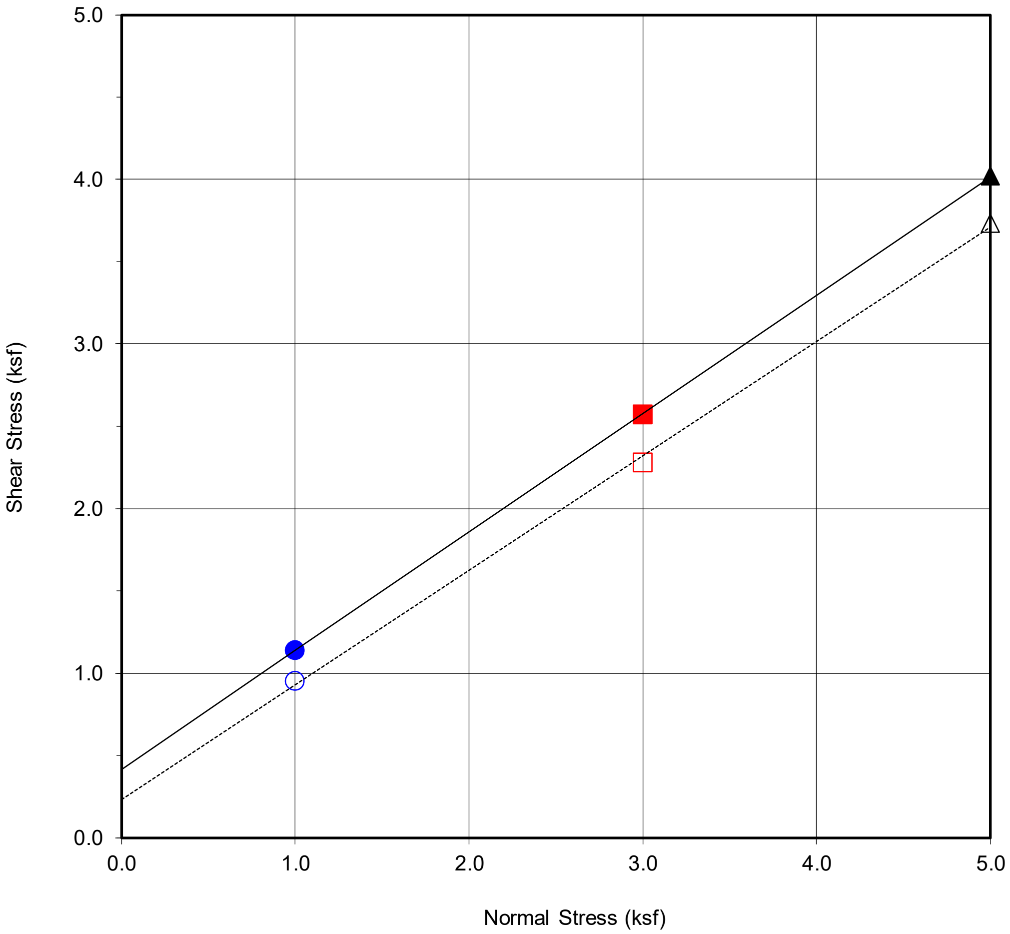
Figure B5



Boring No.	**B8**
Sample No.	**B8@3'**
Depth (ft)	**3'**
Sample Type:	Ring

Soil Identification:		
Sandstone		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	417	35.8
Ultimate	234	34.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.14	■ 2.57	▲ 4.02
Shear Stress @ End of Test (ksf)	○ 0.95	□ 2.28	△ 3.73
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	43.9	31.5	36.1
Initial Dry Density (pcf)	76.2	88.9	82.7
Initial Degree of Saturation (%)	98.0	94.9	93.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	50.5	36.0	39.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

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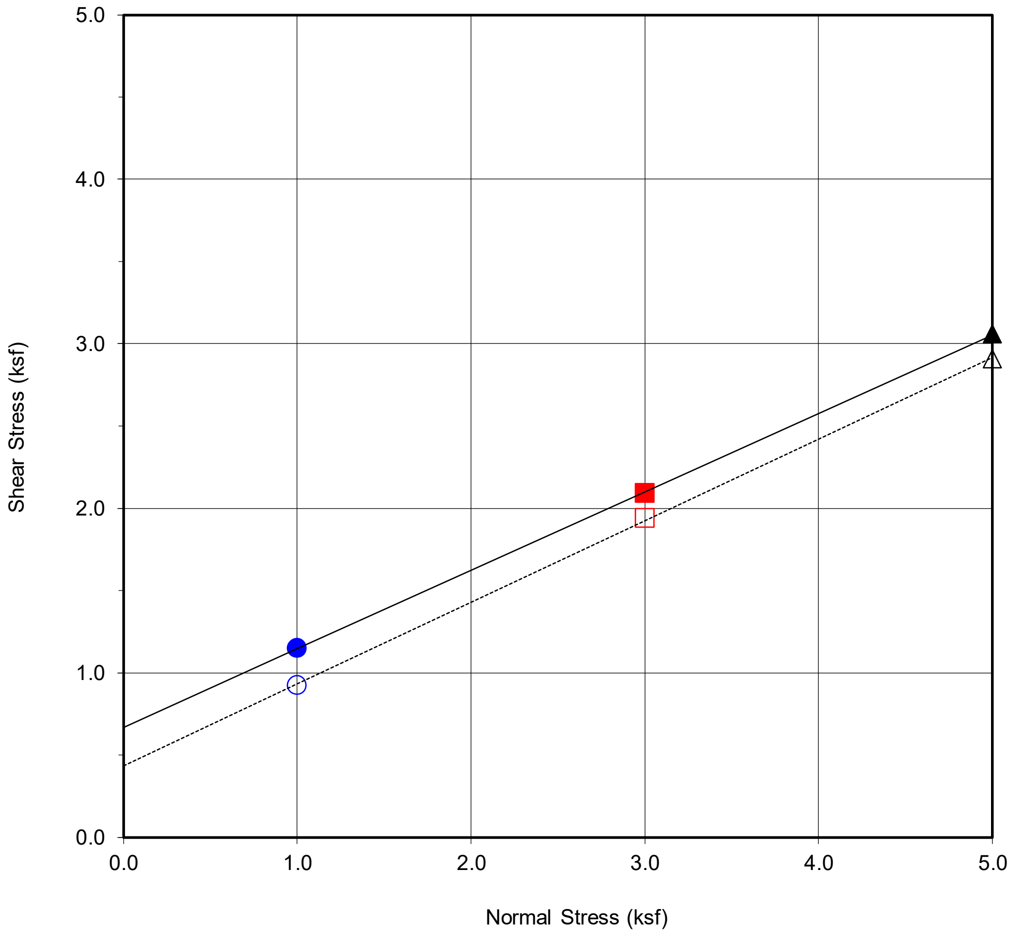
Figure B6



Boring No.	**B11**
Sample No.	**B11@3'**
Depth (ft)	**3'**
Sample Type:	Ring

Soil Identification:		
Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	670	25.5
Ultimate	435	26.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.15	■ 2.09	▲ 3.06
Shear Stress @ End of Test (ksf)	○ 0.92	□ 1.94	△ 2.91
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	27.1	26.4	25.8
Initial Dry Density (pcf)	93.3	91.9	94.1
Initial Degree of Saturation (%)	90.5	85.4	87.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	30.6	31.3	28.2

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

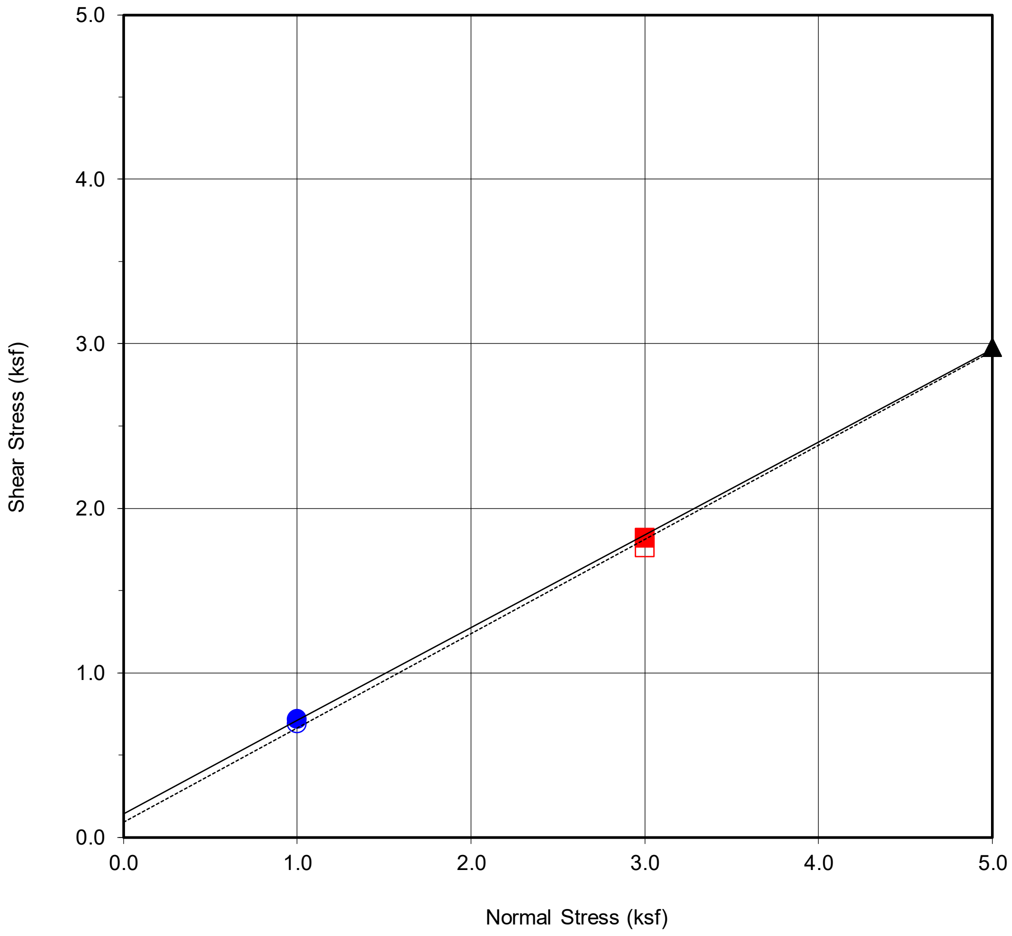
Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B7



Boring No.	B12
Sample No.	**B12@6'**
Depth (ft)	**6'**
Sample Type:	Ring

Soil Identification:		
Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	145	29.5
Ultimate	92	29.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.72	■ 1.82	▲ 2.98
Shear Stress @ End of Test (ksf)	○ 0.69	□ 1.76	△ 2.98
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	67.8	20.2	16.7
Initial Dry Density (pcf)	101.6	95.5	107.6
Initial Degree of Saturation (%)	277.7	71.3	79.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.3	20.4	17.9

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

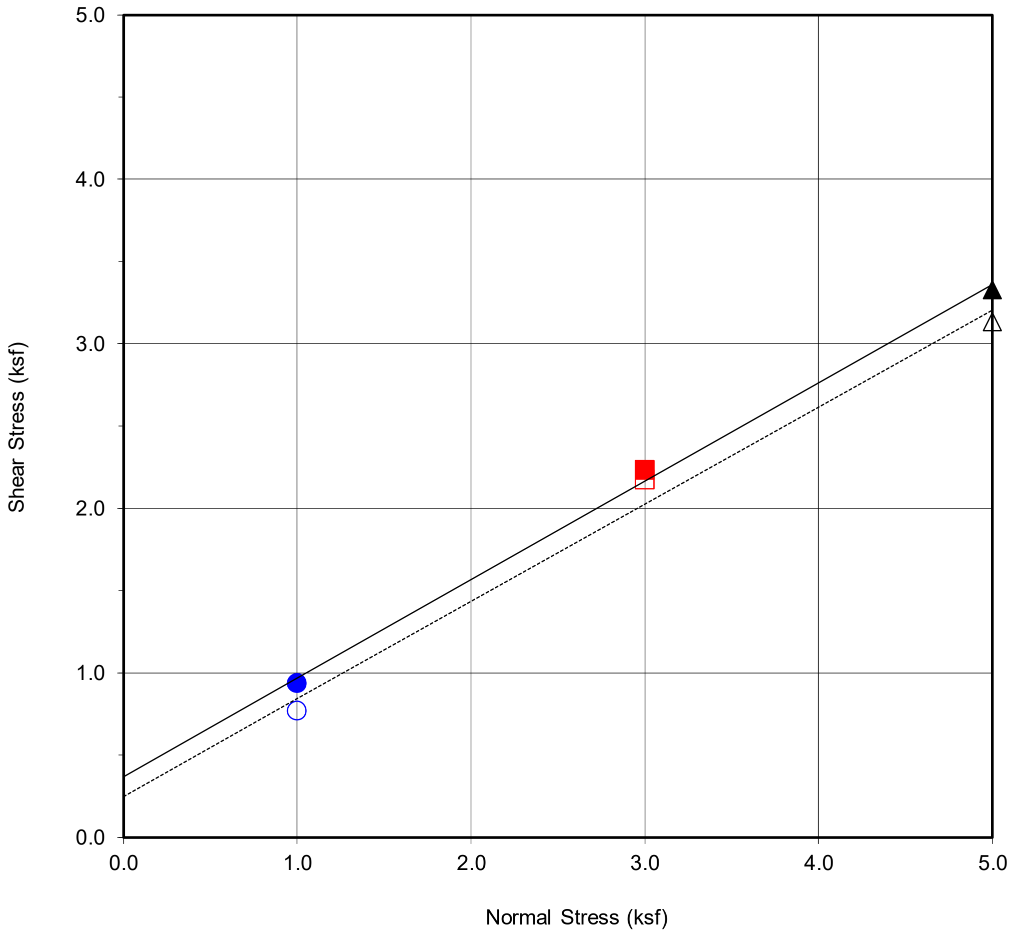
Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B8



Boring No.	**B11**
Sample No.	**B11@0-10'**
Depth (ft)	**0-10'**
Sample Type:	Bulk

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.94	■ 2.23	▲ 3.33
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.17	△ 3.13
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	17.5	17.3	17.4
Initial Dry Density (pcf)	90.0	90.0	90.0
Initial Degree of Saturation (%)	54.1	53.7	53.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	32.5	31.6	30.8

Soil Identification:

Sandy Silt with some Bedrock Fragments (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	371	30.9
Ultimate	251	30.6

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

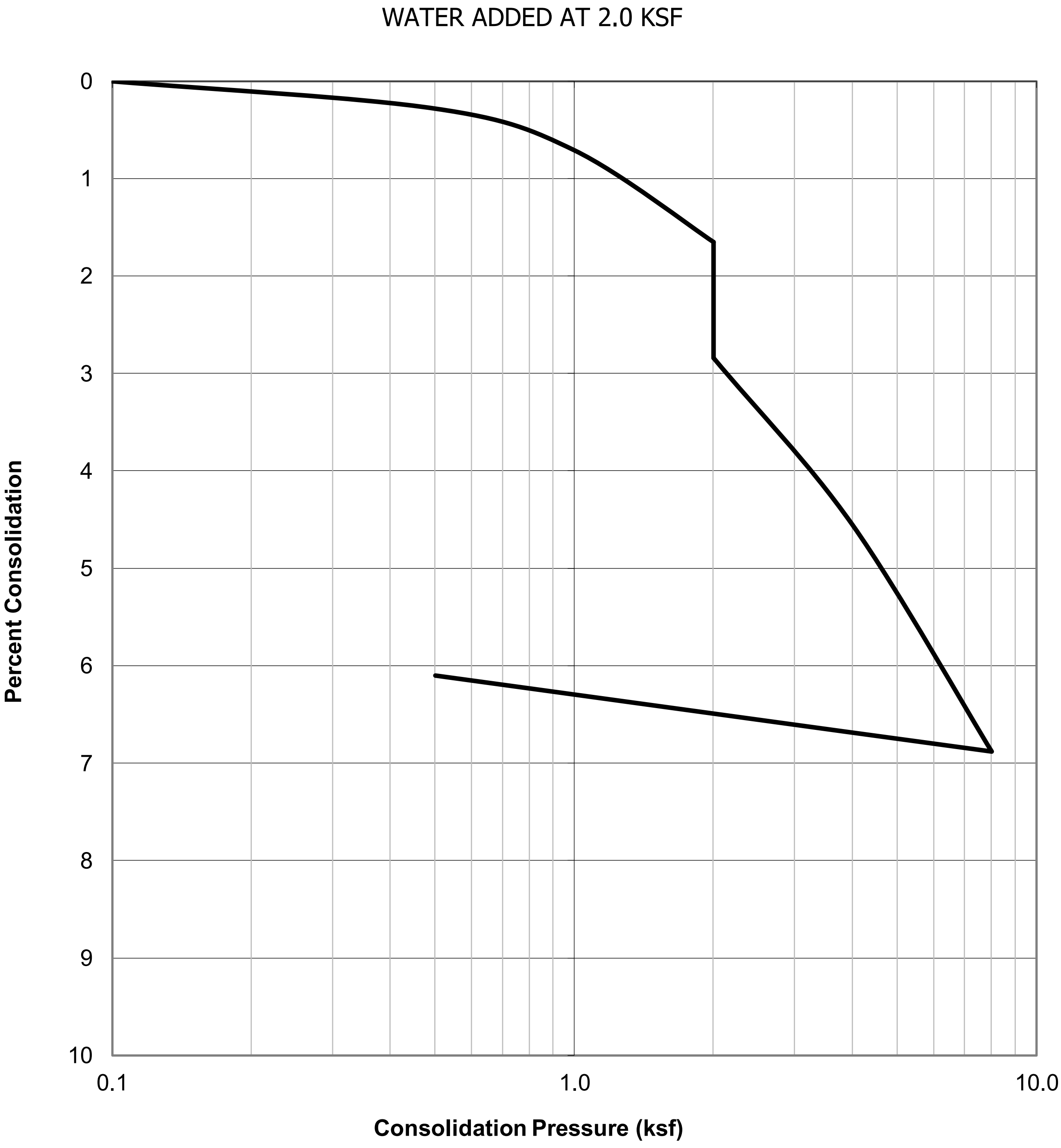
Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B9



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@2.5	Sandy Silt (ML)	105.0	11.2	15.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

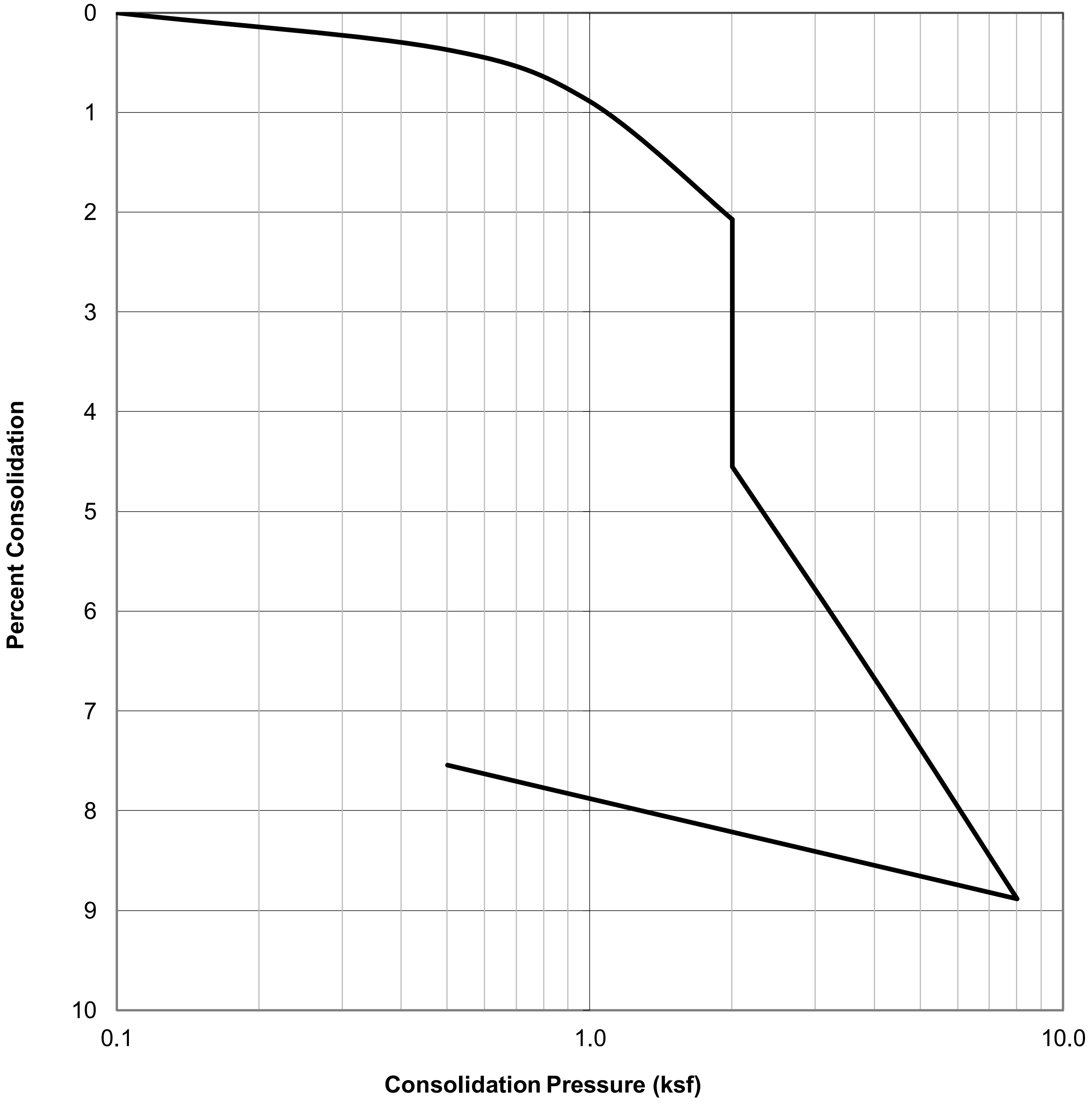
Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@7.5	Silty Sand (SM)	89.9	7.9	16.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

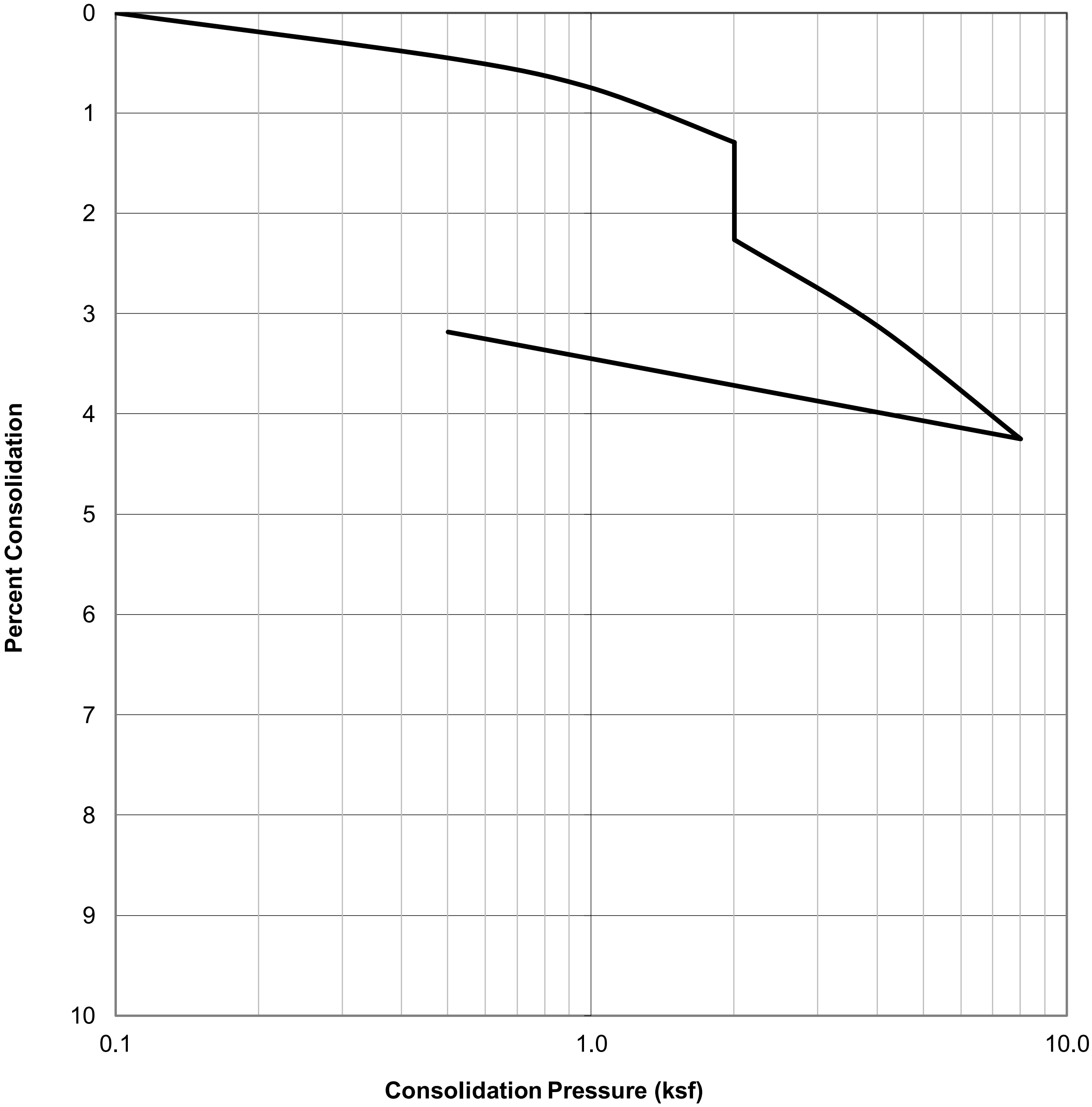
Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@12.5	Clay with Sand (CL)	104.5	13.7	17.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

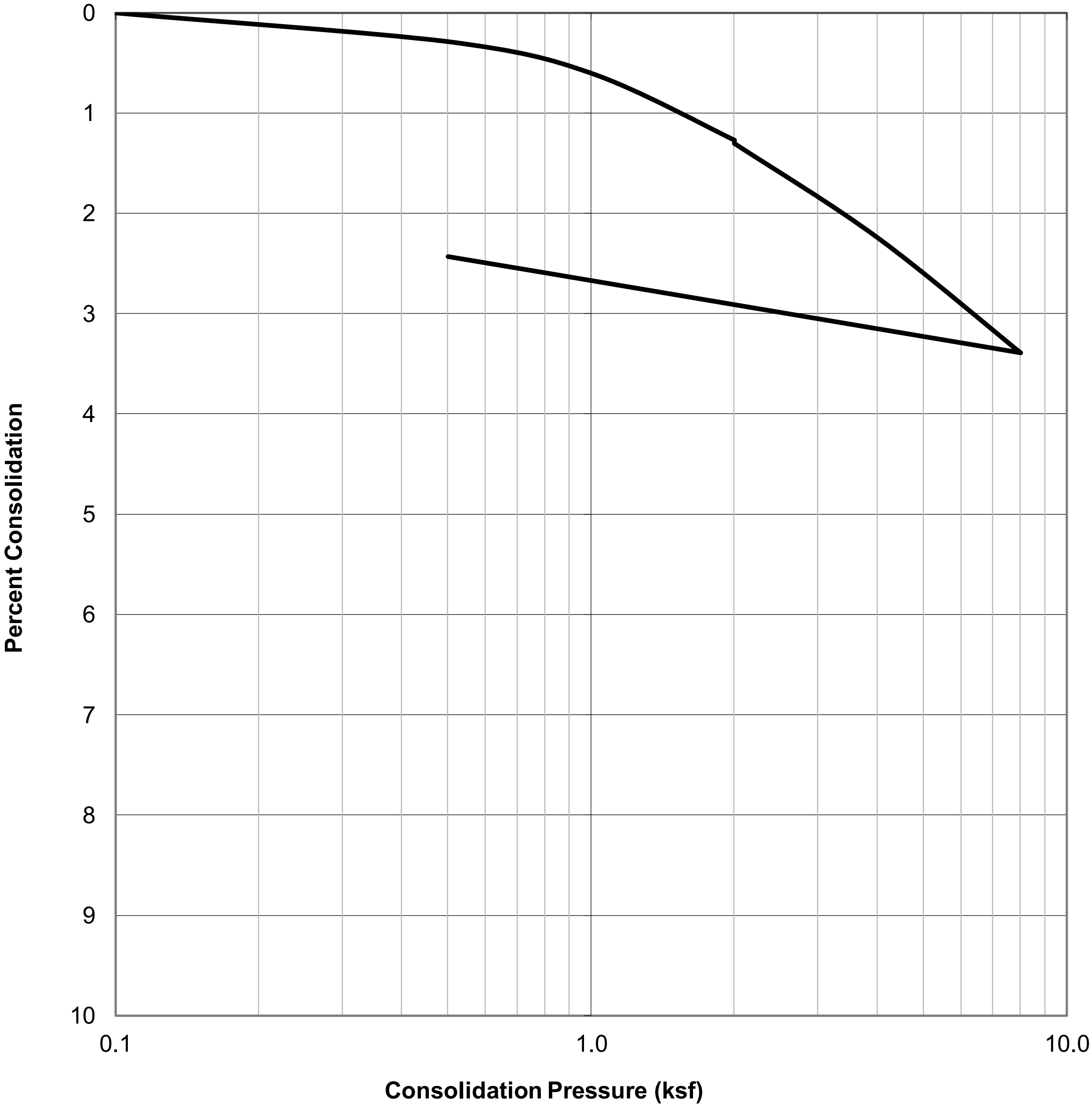
Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B12

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@22.5	Sandy Silt (ML)	111.5	16.7	17.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

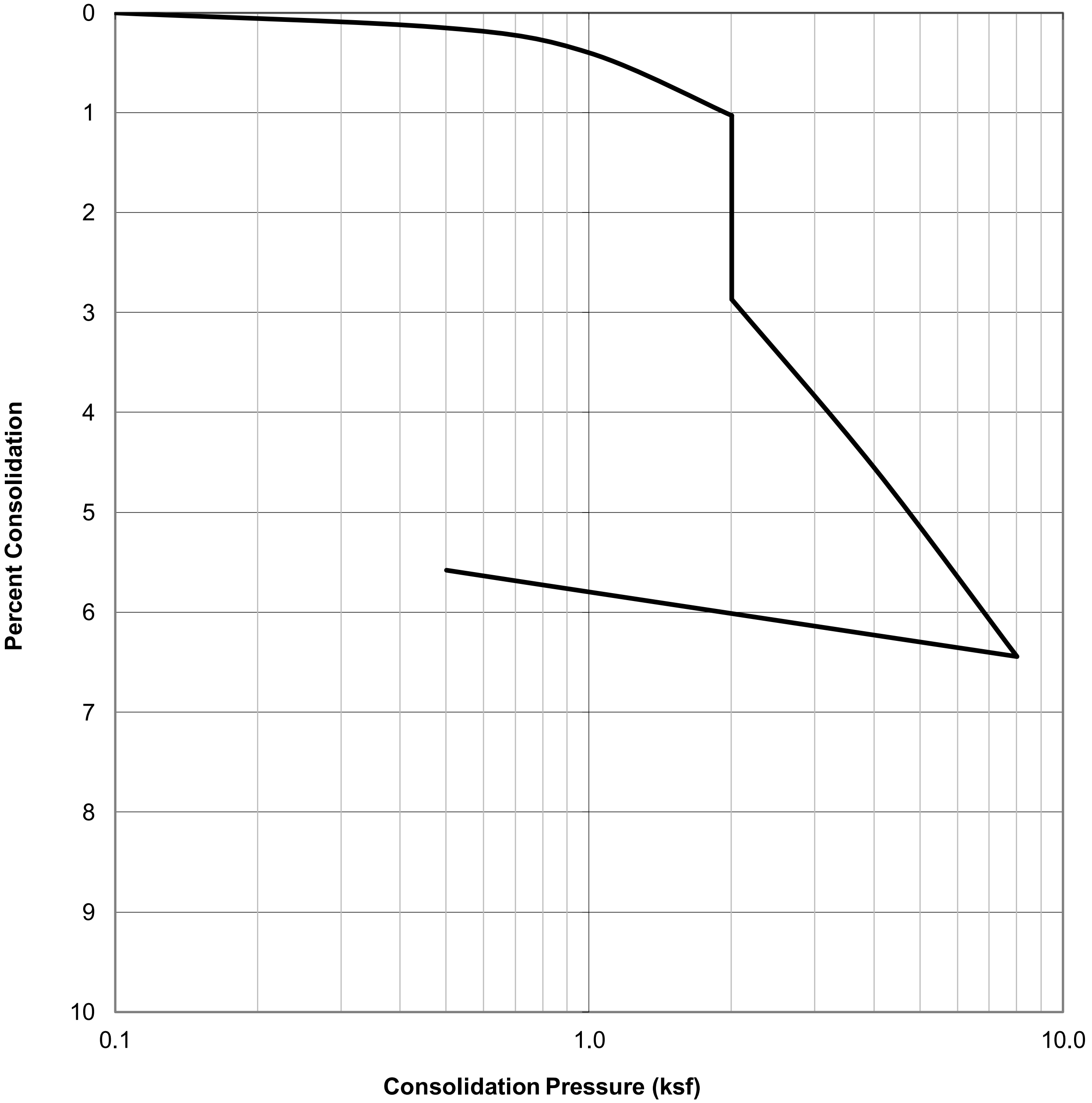
Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B13

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@6	Silty Sand (SM)	101.0	4.9	17.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

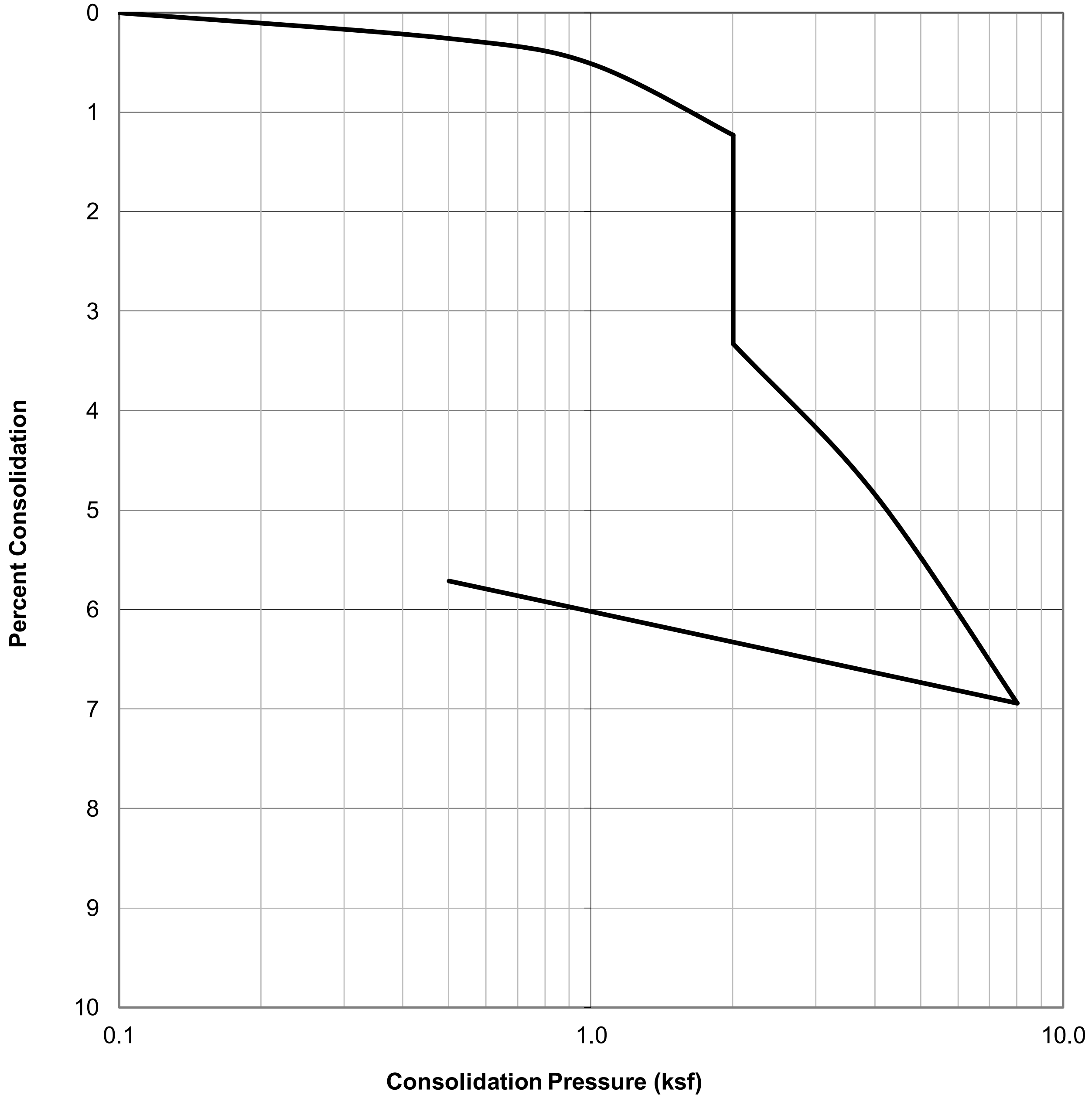
Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B14

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@9	Silty Sand (SM)	103.6	15.0	18.0

GEOCON

CONSOLIDATION TEST RESULTS

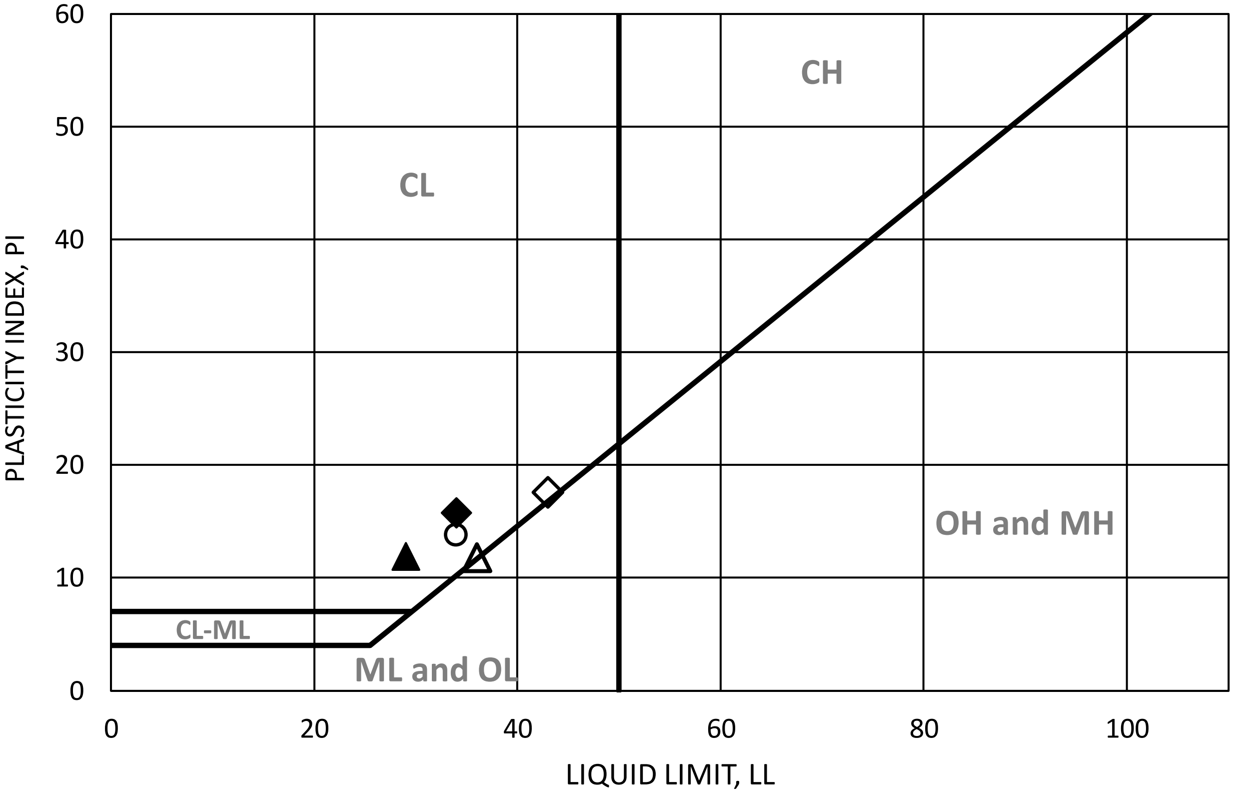
ASTM D-2435

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B1	5'	N/P	N/P	N/P		
◆	B1	10'	34	18	16	26	CL
▲	B1	15'	29	17	12	22	CL
●	B1	20'	N/P	N/P	N/P		
□	B1	25'	N/P	N/P	N/P		
◇	B1	30'	43	25	18		CL
△	B1	35'	36	24	12	23	CL-ML
○	B1	40'	34	20	14	22	CL

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

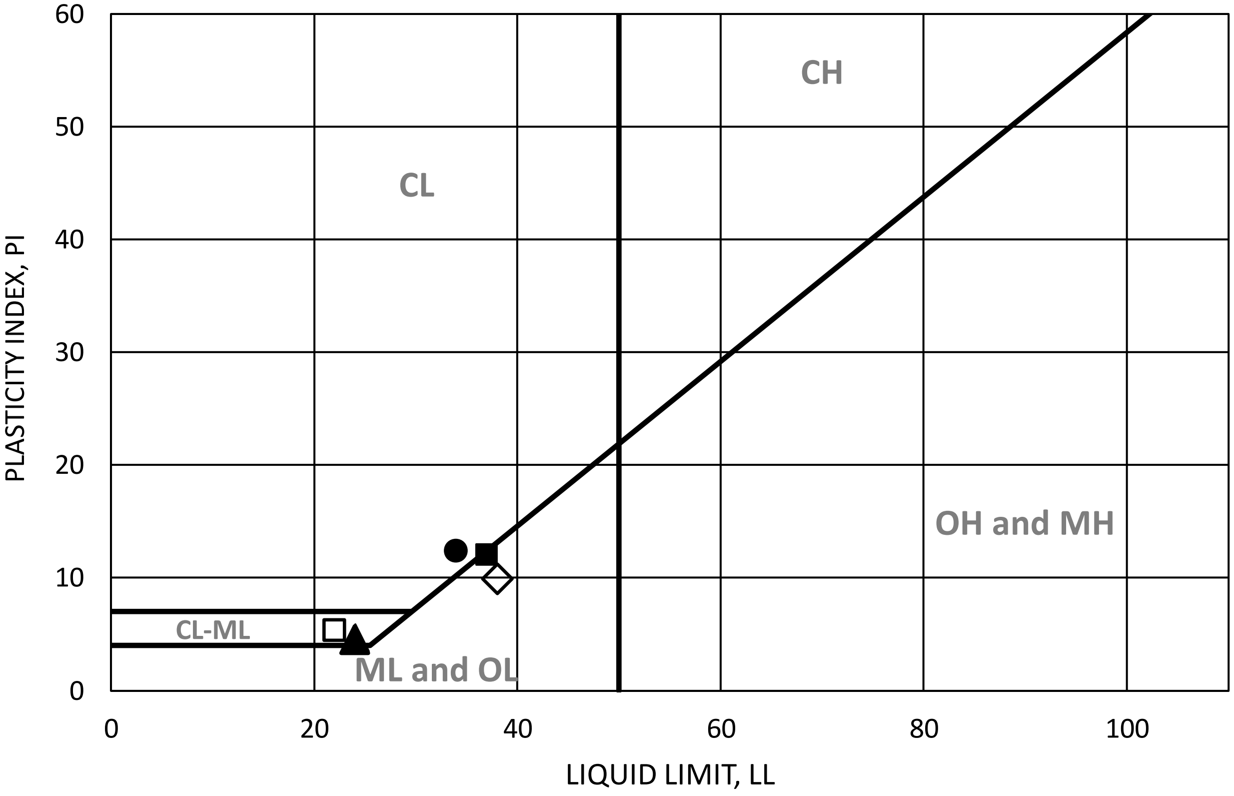
Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B16



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B1	45'	37	25	12		ML
◆	B3	5'	N/P	N/P	N/P		
▲	B3	10'	24	19	5		ML
●							
□							
◇							
△							
○							

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

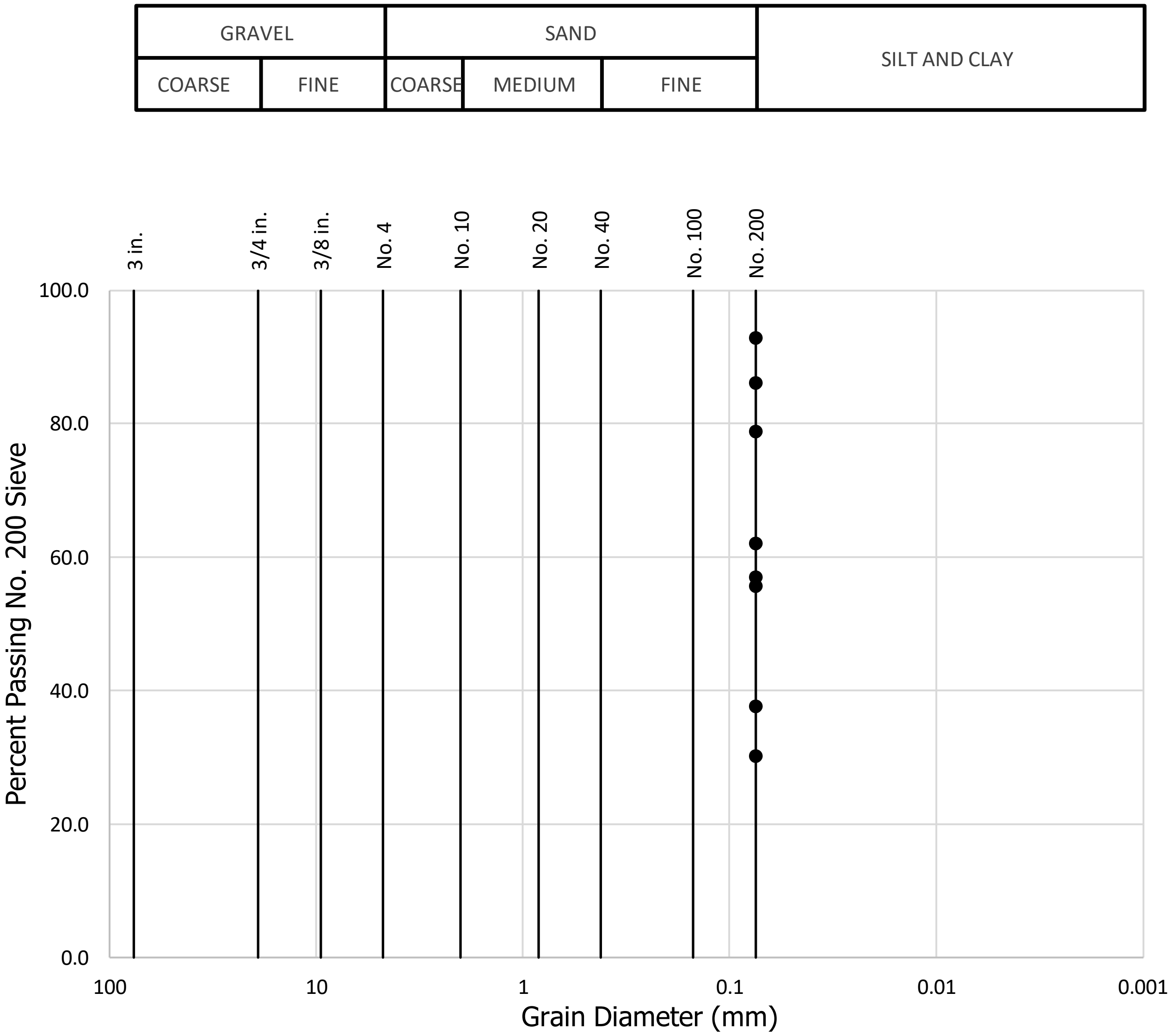
Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B17



Sample No.	Percent Passing No. 200 Sieve
B1 @ 5'	30.2
B1 @ 10'	78.9
B1 @ 15'	55.7
B1 @ 20'	37.6
B1 @ 25'	57.0
B1 @ 30'	86.1
B1 @ 35'	92.9
B1 @ 40'	62.1

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

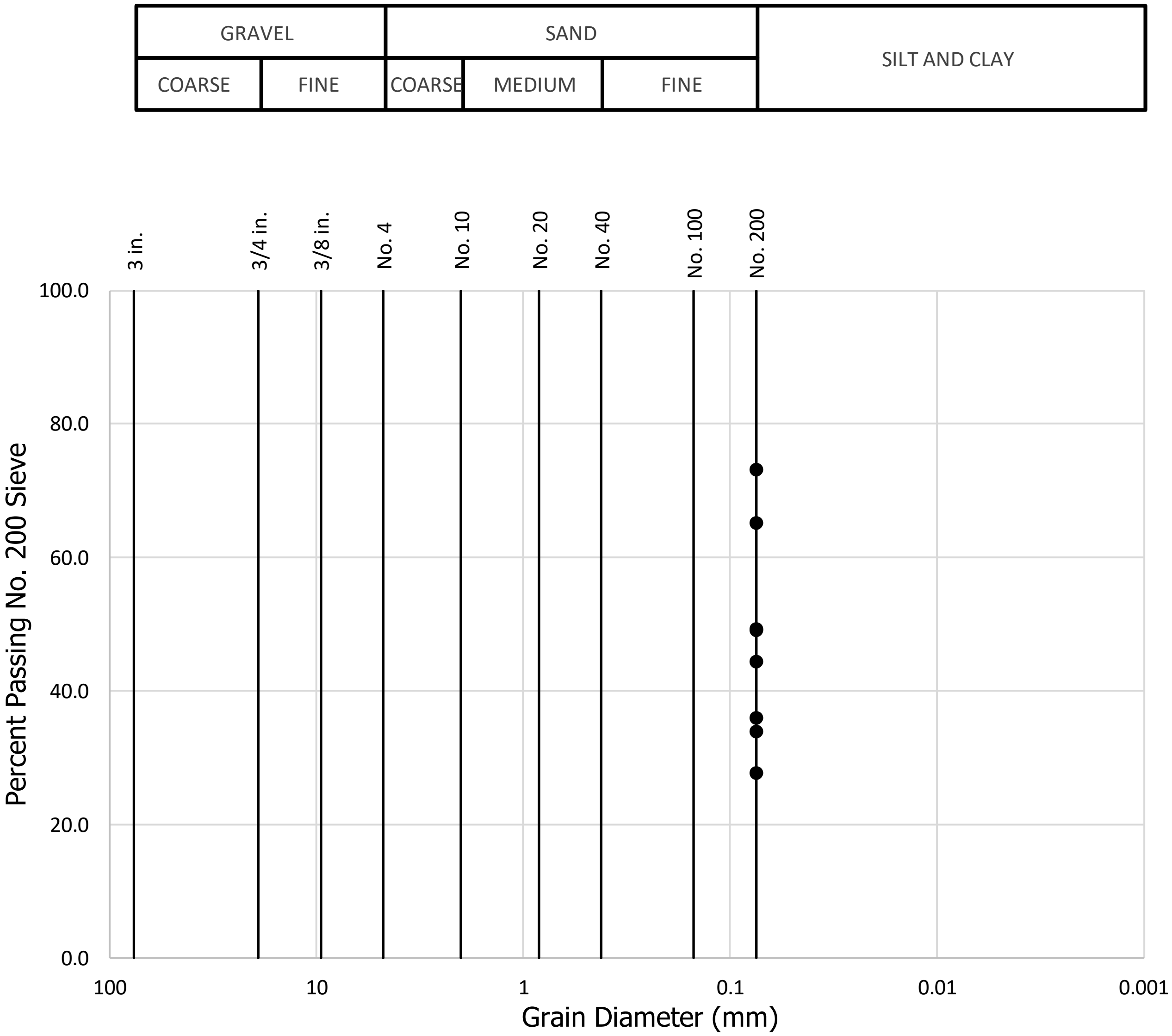
Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B18



Sample No.	Percent Passing No. 200 Sieve
B1 @ 45'	73.2
B3 @ 5'	49.3
B3 @ 10'	49.1
B3 @ 15'	44.4
B3 @ 20'	33.9
B5 @ 5'	65.1
B5 @ 10'	27.7
B6 @ 14'	36.0

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B19

B4@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	782.9	801.2
Wt. of Mold	(gm)	367.6	367.6
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	487.4	801.2
Dry Wt. of Soil + Cont.	(gm)	463.9	382.8
Wt. of Container	(gm)	187.4	367.6
Moisture Content	(%)	8.5	13.3
Wet Density	(pcf)	125.3	130.6
Dry Density	(pcf)	115.5	115.3
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	65.2	69.6
Degree of Saturation	(%) [S_{meas}]	50.3	73.1

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/11/2022	10:00	1.0	0	0.3005
5/11/2022	10:10	1.0	10	0.2965
Add Distilled Water to the Specimen				
5/12/2022	10:00	1.0	1430	0.3175
5/12/2022	11:00	1.0	1490	0.3175

Expansion Index (EI meas) =	21
Expansion Index (Report) =	**21**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2019 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B20

B1@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.1
Wt. Comp. Soil + Mold	(gm)	726.3	759.8
Wt. of Mold	(gm)	367.8	367.8
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	487.4	759.8
Dry Wt. of Soil + Cont.	(gm)	448.3	311.7
Wt. of Container	(gm)	187.4	367.8
Moisture Content	(%)	15.0	25.7
Wet Density	(pcf)	108.1	118.1
Dry Density	(pcf)	94.0	93.9
Void Ratio		0.8	0.9
Total Porosity		0.4	0.5
Pore Volume	(cc)	91.5	103.3
Degree of Saturation	(%) [S_{meas}]	51.5	77.7

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/11/2022	10:00	1.0	0	0.3055
5/11/2022	10:10	1.0	10	0.3045
Add Distilled Water to the Specimen				
5/12/2022	10:00	1.0	1430	0.3615
5/12/2022	11:00	1.0	1490	0.3615

Expansion Index (EI meas) =	57
Expansion Index (Report) =	**57**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2019 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B21

B11@0-10'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.1
Wt. Comp. Soil + Mold	(gm)	708.7	749.5
Wt. of Mold	(gm)	367.6	367.6
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	487.4	749.5
Dry Wt. of Soil + Cont.	(gm)	442.5	290.1
Wt. of Container	(gm)	187.4	367.6
Moisture Content	(%)	17.6	31.7
Wet Density	(pcf)	102.9	115.1
Dry Density	(pcf)	87.5	87.4
Void Ratio		0.9	1.0
Total Porosity		0.5	0.5
Pore Volume	(cc)	99.6	112.5
Degree of Saturation	(%) [S_{meas}]	51.7	81.6

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/11/2022	10:00	1.0	0	0.2935
5/11/2022	10:10	1.0	10	0.293
Add Distilled Water to the Specimen				
5/12/2022	10:00	1.0	1430	0.3555
5/12/2022	11:00	1.0	1490	0.3555

Expansion Index (EI meas) =	62.5
Expansion Index (Report) =	**63**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2019 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

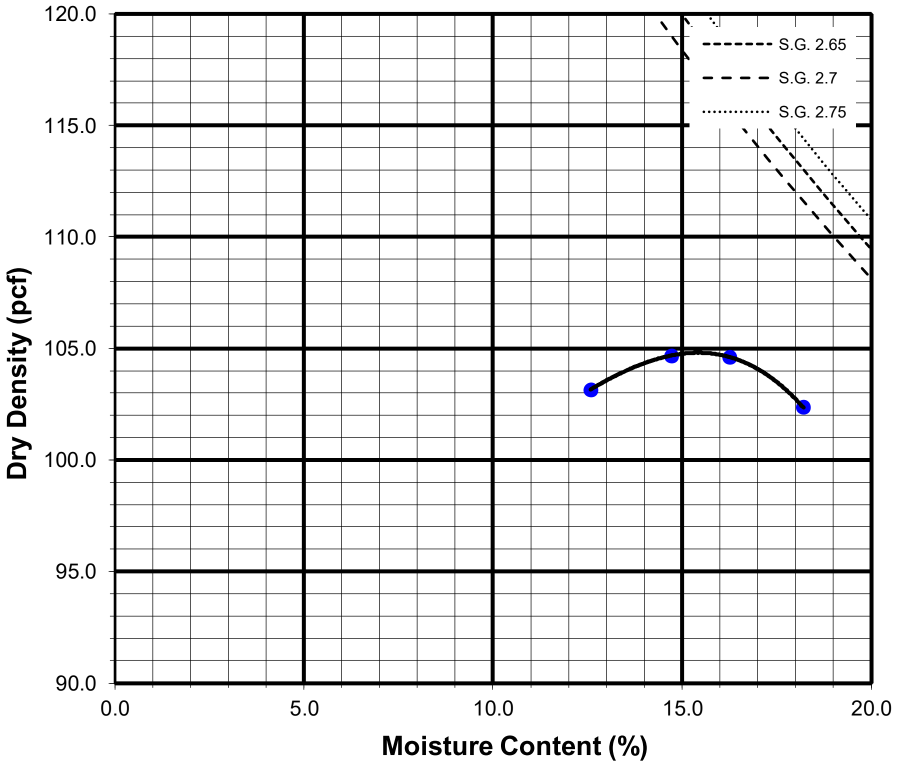
Figure B22

Sample No:

B1@0-5'	Dark Brown Sandy Silt (ML)

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5857	5917	5940	5930		
Weight of Mold (g)	4103	4103	4103	4103		
Net Weight of Soil (g)	1754	1814	1837	1828		
Wet Weight of Soil + Cont. (g)	656.9	624.1	660.7	659.3		
Dry Weight of Soil + Cont. (g)	597.5	561.5	587.0	580.5		
Weight of Container (g)	125.3	136.2	133.5	147.5		
Moisture Content (%)	12.6	14.7	16.3	18.2		
Wet Density (pcf)	116.1	120.1	121.6	121.0		
Dry Density (pcf)	103.2	104.7	104.6	102.4		

Maximum Dry Density (pcf)	**105.0**	**Optimum Moisture Content (%)**	**15.5**



Preparation Method: A

GEOCON

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

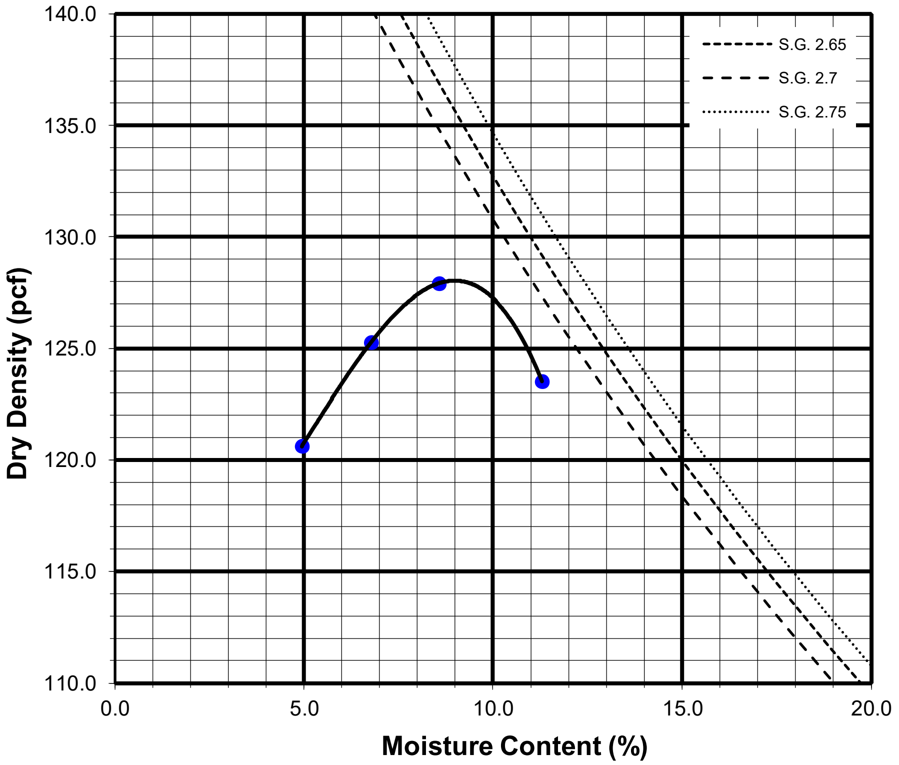
Figure B23

Sample No:

B4@0-3'	Dark Brown Sandy Silt with some Gravel (ML)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6015	6123	6201	6179		
Weight of Mold	(g)	4103	4103	4103	4103		
Net Weight of Soil	(g)	1912	2021	2098	2077		
Wet Weight of Soil + Cont.	(g)	625.8	762.6	748.1	675.1		
Dry Weight of Soil + Cont.	(g)	602.6	723.6	699.7	621.4		
Weight of Container	(g)	133.7	148.5	135.3	145.9		
Moisture Content	(%)	4.9	6.8	8.6	11.3		
Wet Density	(pcf)	126.6	133.8	138.9	137.5		
Dry Density	(pcf)	120.6	125.3	127.9	123.5		

Maximum Dry Density (pcf)	**128.0**	**Optimum Moisture Content (%)**	**8.5**



Preparation Method: A

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

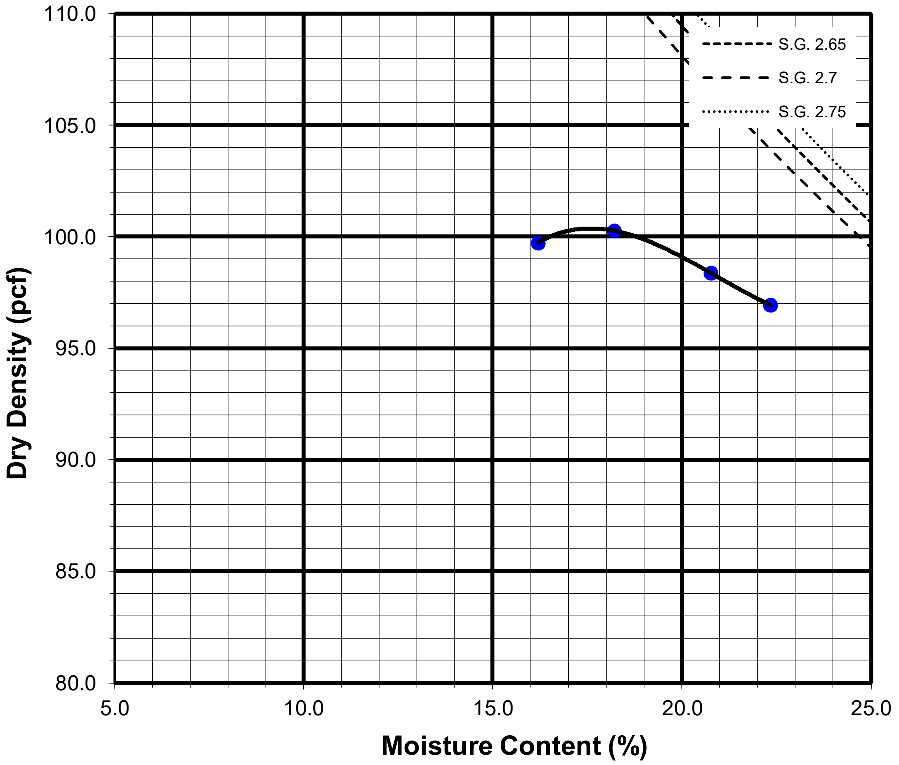
Figure B24

Sample No:

B11@0-10'	Sandy Silt with Siltstone (ML)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	5853	5893	5897	5894		
Weight of Mold	(g)	4103	4103	4103	4103		
Net Weight of Soil	(g)	1750	1790	1795	1791		
Wet Weight of Soil + Cont.	(g)	691.4	676.2	633.2	626.5		
Dry Weight of Soil + Cont.	(g)	615.6	594.8	549.6	535.0		
Weight of Container	(g)	147.3	147.8	146.8	125.4		
Moisture Content	(%)	16.2	18.2	20.8	22.3		
Wet Density	(pcf)	115.9	118.5	118.8	118.6		
Dry Density	(pcf)	99.7	100.3	98.4	96.9		

Maximum Dry Density (pcf)	**100.0**	**Optimum Moisture Content (%)**	**18.0**



Preparation Method: A

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: JMH

GEOCON

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B25

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS

CALIFORNIA TEST NO. 643

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 0-5'	8.1	1500 (Corrosive)
B4 @ 0-3-	8.6	2500 (Moderately Corrosive)
B5 @ 0-5'	8.6	4000 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS

EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1@0-5'	0.027
B4@0-3'	0.009
B5@0-5'	0.012

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS

CALIFORNIA TEST NO. 417

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1@0-5'	0.000	S0
B4@0-3'	0.000	S0
B5@0-5'	0.000	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B8 @ 0-5'	8.3	3000 (Moderately Corrosive)
B11 @ 0-10'	8.5	1600 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B8@0-5'	0.008
B11@0-10'	0.008

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B8@0-5'	0.000	S0
B11@0-10'	0.000	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: JMH

Project No.: W1547-06-01

CHAMINADE HIGH SCHOOL
7500 CHAMINADE AVENUE
WEST HILLS CALIFORNIA

JAN. 23

Figure B27

APPENDIX C

APPENDIX C

PREVIOUS REPORTS

(USB Only)