Ventura County Community College District

PURCHASING DEPARTMENT

DATE: November 8, 2019
TO: All Bidders
FROM: Jo Nell Miller, Purchasing Specialist – jonellmiller@vcccd.edu
SUBJECT: Addendum 1 – Bid 594 Fire Technology Apparatus Building Rough Grading and Storm Drain Improvements

This addendum is hereby made part of the Contract Documents to the same extent as though it was originally included therein and takes precedence over the original documents. The outdated pages must be replaced with any updated and/or changed pages when submitting your bid. ***Acknowledge receipt of all addenda on the Bid Form****.*

The bid opening remains on **Friday, November 22, 2019**. Bids must be received no later than **3:00 p.m**. at 761 E Daily Drive, Suite 200, Camarillo, CA 93010. Properly mark the outside of the exterior envelope on your submitted bid with the Bid Number and Name according to the requirements stated in the bid packet directions.

If you choose not to participate in this particular bid, please notify the listed Purchasing Specialist by email.

It is the responsibility of the Bidder to verify that their proposal has been received by the VCCCD Purchasing Department prior to the opening date. Verification of receipt can be made through the listed Purchasing Specialist.

The attached Engineering Geology and Geotechnical Engineering Report has been added to this project and posted on our website.

The following information is in answer to questions asked during the job walk and via email request. The deadline for questions is Friday November 15, 2019. No further questions will be accepted after that date at 5:00 p.m.

1. **Must an apprentice be active on this project?**
   The contractor must send a request for an apprentice, but the Union may or may not respond.

2. **Is this project only calling out an "A" contractor?**
   An "A" contractor's license is listed on the Cover page, Section 00010 Notice to Contractor Calling for Bid in the bid package and in the Advertisement with the Ventura County Star.

3. **Are any permits required to be pulled by the contractor?**
   No.

4. **Will the contractor have to pay to install meters?**
   Yes, this is the contractor's responsibility.

5. **Will fencing be required around the whole construction area?**
   Because it is not a public accessible site, a construction fence should not be necessary.

6. **When will the construction begin?**
   The first week of January 2020.

7. **Is weekend work allowed?** See 00800 Special Conditions 1.07 and 00700 General Conditions 7.2.1.
   If it doesn't impact anything at FSTA, weekend work is acceptable. The project must be completed on schedule.

8. **What is the Engineer's estimate for this project?**
   As posted on the VCCCD website, the Engineer's estimate is $500,000 - $530,000.

9. **What is the substantial completion expectancy of this project?**
   As posted on the VCCCD website and stated in the bid packet 00310 Sample Agreement and 00800 Special Conditions substantial completions is 90 days from the start date as listed on the Notice to Proceed.

10. **Does the southern portion of the site need to be hydroseeded after the grading is done?**
    Yes, it should include Hydroseeding to prevent erosion from untimely rains.

11. **Can any of the existing storm drain piping be reused?**
    No, all building materials shall be new.

12. **30 Mil Geomembrane Fabric. Please provide a specification for this product. Is it a Liner or a Fabric?**
    It is a liner meeting the Technical guidance manual spec of "A geomembrance line, or other equivalent water proofing. This liner should have a minimum thickness of 30 mils." (TGM BIO-1)

13. **Please provide elevations for the 30 Mil Geomembrane.**
    The answer to this question will be addressed in Addendum 2, as the elevations need further review.

End of Section

ENGINEERING GEOLOGY AND
GEOTECHNICAL ENGINEERING REPORT
FOR
PROPOSED OXNARD COLLEGE FIRE ACADEMY
OXNARD, CALIFORNIA

PROJECT NO.: 302245-001
JUNE 17, 2019

PREPARED FOR
RASMUSSEN & ASSOCIATES

BY
EARTH SYSTEMS PACIFIC
1731-A WALTER STREET
VENTURA, CALIFORNIA 93003

Earth Systems

1731 Walter Street, Suite A | Ventura, CA 93003 | Ph: 805.642.6727 | www.earthsystems.com

June 17, 2019

Project No.: 302245-001
Report No.: 19-6-39

Jay Lomagno
Rasmussen & Associates
21 South California Street, Fourth Floor
Ventura, California 93001

Project: Proposed Oxnard College Fire Academy
Camarillo Area of Ventura County, California

Subject: Engineering Geology and Geotechnical Engineering Report

As authorized, Earth Systems Pacific (Earth Systems) has performed an engineering geology and geotechnical study for a proposed Oxnard College Fire Academy that will be located off the northwest corner of the intersection of Pleasant Valley Road and South Las Posas Road in the Camarillo Airport complex in the Camarillo area of Ventura County, California. The accompanying Engineering Geology and Geotechnical Engineering Report presents the results of our subsurface exploration and laboratory testing programs, and our conclusions and recommendations pertaining to geotechnical aspects of project design. This report completes the scope of services described within our proposal No. VEN-18-05-002, dated May 4, 2018, and authorized by you on June 18, 2018.

We have appreciated the opportunity to be of service to you on this project. Please call if you have any questions, or if we can be of further service.

Respectfully submitted,

EARTH SYSTEMS PACIFIC

PROFESSIONAL GEOLOGIST
PATRICK V. BOALES
No. 1346
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Patrick V Boales

Patrick V. Boales 6-17-19
Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
ANTHONY P. MAZZEI
GE 2823
Exp. 6-30-19
GEOTECHNICAL
STATE OF CALIFORNIA

Anthony P. Mazzei
Geotechnical Engineer

6/17/19

Copies: 4 - Rasmussen and Associates (3 via US mail, 1 via email)
1 - Project File

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PROJECT DESCRIPTION

This report presents results of an Engineering Geology and Geotechnical Engineering study performed for a proposed Oxnard College Fire Academy that will be located off the northwest corner of the intersection of Pleasant Valley Road and South Las Posas Road in the Camarillo area of Ventura County, California. The Fire Academy will be located on a vacant square-shaped site of about 2-acres in the southeast corner of the Camarillo Airport complex, and which presently serves as a detention basin. The detention basin will be relocated to an open field south of the proposed Fire Academy building. The proposed approximate 12,200 square-foot Fire Academy building will be a pre-engineered structure that will be centered in the 2-acre site and surrounded by parking/pavement areas.

The project site is located within one of the liquefaction hazard zones delineated by the California Geological Survey. It is understood that the project will be under the jurisdiction of the Division of the State Architect (DSA).

Topographically, the flat site slopes gently down toward the south. We understand that the site will be raised to match the elevation of the adjacent grade along the north and east sides. Based on a preliminary grading plan, fill thicknesses of approximately 3 to 6.5 feet are expected to be placed beneath the proposed building during site grading. Fill thicknesses within the proposed parking lot will range from approximately 0.5 to 4.5 feet. In other areas of the site to bring it up to finished subgrade elevation, fill thicknesses of approximately 0.5 to 6.5 feet are expected to be placed. Minor cuts will be made around the perimeter of the site to remove high spots, and cuts on the order of about 2.5 to 4 feet will be made for construction of the new detention basin.

We anticipate the proposed building will be a tall one-story pre-engineered structure with a slab-on-grade floor system. As provided by the Project Structural Engineer, we understand the maximum column load will be 30 kips with a maximum wall load of 2 kips per lineal foot. These structural considerations were used as a basis for the recommendations of this report. Because static settlements under the building loads governs the foundation recommendations presented in this report, if actual loads vary significantly from these assumed loads, Earth Systems Pacific (Earth Systems) should be notified since re-evaluation of the recommendations contained in this report may be required.

PURPOSE AND SCOPE OF WORK

The purpose of the geotechnical study that led to this report was to evaluate and analyze the soil conditions of the site with respect to the proposed resort hotel as planned. These conditions include surface and subsurface soil types, expansion potential, settlement potential, bearing capacity, and the presence or absence of subsurface water.

The scope of work performed as part of the overall study included:

1. Performing a reconnaissance of the site.
2. Reviewing available maps and documents relevant to the site geology, seismic setting, and geotechnical conditions.
3. Advancing a total of one (1) cone penetrometer test (CPT-1) sounding to study soil properties and conditions.
4. Drilling, sampling, and logging two (2) exploratory borings (B-1 and B-2) to study soil and groundwater conditions.
5. Two borings (I-1 and I-2) were advanced within the proposed detention basin for use in infiltration testing.
6. Laboratory testing soil samples obtained from the subsurface exploration to determine their physical and engineering properties.
7. Consulting with Owner representatives and design professionals.
8. Analyzing the geotechnical data obtained.
9. Preparing this report.

Contained in this report are:

1. Descriptions and results of field and laboratory tests that were performed.
2. Discussions pertaining to the local geologic, soil, and groundwater conditions.
3. Conclusions pertaining to geohazards that could affect the site.
4. Conclusions and recommendations pertaining to site grading and structural design.

SITE SETTING

The site of the proposed building is a vacant 2-acre square-shaped parcel of land situated west of the existing Oxnard College Fire Academy. The site presently serves as a detention basin for the existing facility. Small earth berms are present along the north, south and west sides of the existing detention basin. An existing paved access road serves as the containment berm along the east side of the existing detention basin. The bottom of the existing detention basin is approximately 6 feet lower than the adjacent paved interior road to the east. We understand that the existing detention basin will be relocated to an open field south of the proposed new Fire Academy. The ground surface outside of the detention basin slopes to the southwest to a small drainage feature running along the west side of the site. Stockpiles of end-dumped soil are present on the site within the proposed parking lot area. The site coordinates are Latitude 34.2077° North and Longitude 119.0733° West.

GEOLOGY

The Camarillo Airport site is located in the Oxnard Plain, which is in the western portion of the Transverse Ranges geologic province. The vicinity of the project is underlain by about 1,500-2,000 feet of relatively horizontal Holocene and Pleistocene alluvial sediments over Tertiary age bedrock units (Jakes, 1979). The Camarillo Fault, a relatively short and steeply-dipping east-west trending fault showing north side up displacement projects to about 2,100 feet north of the project site (C.D.M.G., 1998).

The project site is not within any of the State of California designated seismic hazard zones for earthquake induced landslides or fault rupture but is within a seismic hazard zone for liquefaction potential (C.D.M.G., 2002b).

Although the Camarillo Fault is the nearest fault to the site, the nearest fault of interpreted seismogenic significance is the Simi-Santa Rosa-Springville fault. It is a north dipping reverse fault that strikes along a northeasterly trend. At the closest position relative to the site, the surface trace is approximately 1.3 miles to the northwest. Portions of this fault system are considered "active" by the State.

No faults or landslides were observed to be located on or trending into the subject property during the field study, or during reviews of the referenced geologic literature, or during review of the aerial photographs taken of the site.

GEOLOGIC HAZARDS

Geologic hazards that may impact a site include seismic shaking, fault rupture, landsliding, liquefaction, seismic-induced settlement of dry sands, and flooding.

A. Seismic Shaking

1. Southern California is a seismically active region where the potential for significant ground shaking is universal. Earthquakes of a size large enough to cause structural damage are relatively common in the region. Per the State of California guidelines for these types of reports, when evaluating the seismicity potential of a specific site, it is general practice to look at the historical seismic record of the area and also review the site location with respect to mapped potentially active and active faults. By using this procedure, estimates of maximum ground accelerations are determined for consideration in structural design for buildings. The geotechnical community uses the method even though most are well aware of its shortcomings. The most significant shortcomings relate to the presence of unknown seismogenic intervals between earthquake events on many of the recognized faults. The 1983 Coalinga and 1994 Northridge Earthquakes are examples of relatively large events that occurred on previously unrecognized faults. Man has only been using instruments to monitor earthquakes since the 1930's, which is a relatively short time span considering that the intervals between large earthquakes on some of the regional faults are on the order of thousands of years. Considering the above, an evaluation of site acceleration potential will lead to a value that must be considered an approximation. The structural designers must be aware that there are inherent uncertainties in the determined value or range.

2. The Camarillo area has not experienced any local large earthquakes since records have been kept; however, regional earthquakes have led to significant ground shaking and structural damage. Notable regional earthquakes include the 1812 Santa Barbara Channel and 1857 Fort Tejon events. The epicenter of the 1812 earthquake is thought to have been in the western part of the Santa Barbara Channel. Associated with this earthquake, a tsunami with a disputed run up height of up to 15 feet impacted the Ventura coastal area. On January 9, 1857, the Fort Tejon earthquake with an estimated Richter magnitude of 8.25 impacted the region. According to C.D.M.G., (1975), the earthquake caused the roof of the Mission San Buenaventura to fall in.

3. One measure of ground shaking is intensity. The Modified Mercalli Intensity Scale of ground shaking ranges from I to XII with XII indicating the maximum possible intensity of ground movement. Structural damage begins to occur when the intensity exceeds a value of VI. Southern Ventura County has been mapped by the California Division of Mines and Geology to delineate areas of varying predicted seismic response. The deposits that underlie the subject area are mapped as having a probable maximum intensity of earthquake response of approximately IX on the Modified Mercalli Scale. Historically, the highest estimated intensity in the Camarillo area has been VI (C.D.M.G., 1975, 1994).

4. The school site, like any other site in the region, is subject to relatively severe ground shaking in the event of a maximum earthquake on a nearby fault. In Appendix A is a Regional Fault Location Map that shows the site's relationship to the identified faults in the region. In Appendix C is a summary table listing well-identified faults within about a 35-mile radius of the school, the distance between each fault and the school, and mean earthquake magnitudes that could occur on each of the listed faults. A proprietary program utilizing the State of California's fault model (C.G.S. and USGS, 2008) was used to prepare the list.

5. It is assumed that the 2016 CBC and ASCE 7-10 guidelines will apply for the seismic design parameters. The 2016 CBC includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The seismic design parameters presented herein were determined by the U.S. Seismic Design

Maps "risk-targeted" calculator on the USGS website for the jobsite coordinates (Latitude 34.2077° North and Longitude 119.0733° West). The calculator adjusts for Soil Site Class E, and for Occupancy (Risk) Category III (for schools). The velocity (Vs30) when adjusting for site class was 180 meters per second, as per the default within the U.S. Geological Survey website, but 150 meters per second when calculating site-specific parameters.

For school projects, the seismic design values are referenced to the Maximum Considered Earthquake (MCE) and, by definition, the MCE has a 2% probability of occurrence in a 50-year period. This equates to a return rate of 2,475 years. Spectral acceleration parameters that are applicable to seismic design are presented below and again in Appendix C. It should be noted that the school project carries a seismic importance factor I of 1.25 and that factor has been incorporated into the 2016 California Building Code response spectrum.

The design peak ground acceleration for the site, as calculated by the USGS website, is 0.879 g, although the modified PGA was calculated to be 0.791 g.

The calculated 2016 California Building Code (CBC) and ASCE 7-10 seismic parameters typically used for structural design are included in Appendix C and summarized in the table below.

Site Class (Table 20.3-1 of ASCE 7-10 with 2016 update)	E
Occupancy (Risk) Category	III
Seismic Design Category	E
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	2.374 g
Spectral Response Acceleration at 1 sec. – S_1	0.833 g
Site Coefficient – F_a	0.90
Site Coefficient – F_v	2.40
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.137 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.999 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.425 g
One Second Spectral Response – S_{D1}	1.333 g

Site Modified Peak Ground Acceleration - PGA_M	0.791 g
Note: Values Appropriate for a 2% Probability of Exceedance in 50 Years	

Because the S_1 value is greater than 0.75 g, and the Seismic Design Category is E, a site-specific design analysis is also required. The calculated "site-specific" Short Period Spectral Response (S_{DS}) was found to be 1.140 g, and the 1 Second Spectral Response (S_{D1}) was found to be 1.249 g. The more conservative of the values should be used for design. The adjusted peak ground acceleration (PGA_M) was found to be 0.791 g.

6. California has had several large earthquakes in this century, and studies on the structural effects of the ground shaking have led to changes in the building codes. After the 1933 Long Beach Earthquake, the State of California Field Act was written with the intention of making public schools more earthquake resistant. The intent of the act, as is the intent of the most modern codes, is as follows: "School buildings constructed pursuant to these regulations are expected to resist earthquake forces generated by major earthquakes in California without catastrophic collapse, but may experience some repairable architectural or structural damage". Following the 1971 San Fernando Earthquake, many changes were made to the public-school building codes. After the 1994 Northridge Earthquake, a study of 127 public schools in the Los Angeles area by the State of California Division of the State Architect (1994a) revealed that the intent of the Field Act was being met even when buildings were subjected to horizontal accelerations approaching 0.9 g (much higher than expected) over a large area. None of the schools collapsed and most of the damage that would have caused injury to students, had school been in session, was from failures of non-structural items such as light fixtures, florescent bulbs, suspended ceilings, etc. Most of the schools that experienced these non-structural failures were built before the changes to the building code that applied to these non-structural items. The study also resulted in recommended changes to building codes regarding steel framed school buildings, (State of Calif. Div. of State Architect, 1994b).

B. Fault Rupture

Surficial displacement along a fault trace is known as fault rupture. Fault rupture typically occurs along previously existing fault traces. As mentioned in the "Structure" section above, no existing fault traces were observed to be crossing the site. As a result, it is the opinion of this firm that the potential for fault rupture on this site is low.

C. Landsliding and Rock Fall

As mentioned previously, the subject site is relatively flat. As a result, it appears that the hazards posed by landsliding and rock fall are considered nil.

D. Earthquake-Induced Settlement, Cyclic Softening, and Lateral Spread

Earthquake-induced cyclic loading can be the cause of several significant phenomena, including liquefaction in fine sands and silty sands. Liquefaction results in a loss of strength and can cause structures to settle or even overturn if it occurs in the bearing zone. Cyclic softening in clays during earthquakes has resulted in buildings experiencing foundation failure and ground surface deformation similar to that resultant from liquefaction. If liquefaction or cyclic softening occurs beneath sloping ground, a phenomenon known as lateral spreading can occur. Liquefaction and cyclic softening are typically limited to the upper 50 feet of the subsurface soils. There are a number of conditions that need to be satisfied for liquefaction or cyclic softening to occur. Of primary importance is that groundwater, perched or otherwise, usually must be within the upper 50 feet of soils.

The subject site is located within one of the Liquefaction Hazard Zones delineated by the State of California (C.G.S., 2002b).

Fine sands and silty sands that are poorly graded and lie below the groundwater table are the soils most susceptible to liquefaction. Soils that have I_c values greater than 2.6, soils with plasticity indices (PI) greater than 7, sufficiently dense soils, and/or soils located above the groundwater table are not generally susceptible to liquefaction.

An examination of the conditions existing at the site, in relation to the criteria listed above, indicates the following:

1. Groundwater was encountered in the exploratory borings at a depth of 8 feet below the existing ground surface. However, mapping of historically shallowest groundwater elevations by C.D.M.G. (2002a) indicates groundwater may have risen to within about 13 to 14 feet of the ground surface in the past.
2. Interpretation of the CPT data indicates that the upper 50 feet of the soil profile in CPT-1 includes numerous layers with I_c values greater than 2.6, which is considered the boundary between soils prone and not prone to liquefaction (see CPT Interpretations in Appendix A).
3. Standard penetration tests conducted in the borings, and interpretations of blow counts from CPT data indicate that the near-surface fine-grained soils within the tested depths are generally very soft to stiff, whereas the deeper sands are in a medium dense to dense state.

Based on the above, cyclic mobility analyses were undertaken to analyze liquefaction potentials of soil layers underlying the project site. The analysis was performed in general accordance with the methods proposed by NCEER (1997). In the analysis, the design earthquake was considered to be a 7.2 moment magnitude event, and a peak ground acceleration of 0.791 g, as per the discussion in the "Seismicity and Seismic Design" section of this report.

The analysis for CPT-1 indicated that the majority of the soil layers analyzed in the model had factors of safety that exceeded 1.3 (see Appendix D for calculations), except for the zones between the depths of approximately 24.5 to 27.5 feet, 31.5 to 32 feet, and 36 to 39 feet below the existing ground surface. Zones with factors of safety less than 1.3 are considered potentially liquefiable (C.G.S., 2008, and SCEC, 1999).

The volumetric strain for the potentially liquefiable zones was estimated using a chart derived by Tokimatsu and Seed (1987) after reducing the $N_{1(60)}$ values derived by the analytical program by the calculated "FC Delta" value, then making adjustments for fines content as per Seed (1987) and SCEC (1999). Using this methodology, the volumetric strain was found to be 1.0 inch.

There is a potential for differential areal settlement suggested by our findings. As mentioned previously, the total seismic-induced-related settlement could potentially range up to about 1 inch near sounding CPT-1. (Calculations are included within Appendix E of this report.) According to SCEC (1999), up to about half of the total settlement could be realized as differential settlement. As a result, differential settlement could range up to about 0.5 inch at the ground surface.

According to data generated by Ishihara (National Academy Press, 1985), no "ground" damage would be expected due to the thickness of the non-liquefiable soils above the shallowest liquefiable zone. (Examples of ground damage are sand boils and ground cracks.)

Ground oscillation, which is the other type of lateral spreading, occurs where sites are not adjacent to sloped areas or canyons. It can pose a hazard when corrected standard blow counts ($N1_{(60)}$) in the zones of potential liquefaction are less than 15. The potential ground oscillation was analyzed in accordance with procedures developed by Youd, Hansen and Bartlett (2002). In the analyses, it was assumed that the surface slope was 0.5%, which is equivalent to the 10 feet of fall in 2,000 feet shown near the subject site on the Camarillo Quadrangle. A fines content of 30% was assumed based on averaging the soil types of the potentially liquefiable soils. The cumulative displacement was calculated to be about 0.5 feet (i.e., 6 inches), if all potentially liquefiable zones with $N1_{(60)}$ values of less than 15 were to simultaneously liquefy. (Calculations are included in Appendix D.)

Calculations based on the measured liquidity indices indicate that the clay layers tested have sensitivities of 5 or less. As a result, these clay layers do not appear to be sensitive. Hence, cyclic softening of clays and post-liquefaction settlement from consolidation of clays disturbed by a design level earthquake do not appear to be significant at the subject site.

Based on the above, it is the opinion of this firm that a potential for lateral spreading and liquefaction exists at this site. Results of the lateral spreading and liquefaction analyses are included in Appendix D of this report. Due to the fine-grained nature of the near-surface soils at the subject site, seismic induced settlement of dry sand not expected. Mitigation should include designing for the estimated seismically-induced settlements and horizontal displacements related to liquefaction that may be experienced during seismic events. The project Structural Engineer should account for the displacements discussed above when designing the foundation system for the proposed structure.

E. Seismic-Induced Settlement of Dry Sands

Dry sands tend to settle and densify when subjected to earthquake shaking. The amount of settlement is a function of relative density, cyclic shear strain magnitude, and the number of strain cycles. Because the upper 24 feet are predominantly fine-grained soils that are not susceptible to dry sand settlements, it is opinion that the potential for seismically-induced settlement of dry sands at the site is nil.

F. Hydroconsolidation Potential

Hydroconsolidation is a phenomenon whereby dry alluvial soils collapse as they become wetted. Data presented by El-Ehwany and Houston (1990) show that most hydrocollapse occurs as dry soils become wetted to 60% saturation, and that wetting above that level produces little additional collapse.

Because groundwater was encountered in the exploratory borings at a depth of 8 feet below the existing ground surface and the upper 24 feet consists of clayey soils not prone to hydrocollapse, it is opinion that the potential for hydroconsolidation of the soils underlying the site is nil.

G. Flooding

Earthquake-induced flooding types include tsunamis, seiches, and reservoir failure. Due to the inland location of the site, hazards from tsunamis and seiches are considered extremely unlikely.

According to the Ventura County General Plan Hazards Appendix (2013), this site, like most of the Oxnard Plain, is within a dam failure inundation zone. Proper maintenance of these dams is anticipated, and assuming the maintenance continues as planned, the hazard posed by reservoir failure appears to be low.

The site is within an area mapped within Zone X (F.E.M.A., 2019). Zone X is defined as "Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile." From this, it appears that the hazard posed by storm-induced flooding is low.

ENGINEERING GEOLOGY CONCLUSIONS AND RECOMMENDATIONS

Based on the data provided in this report, it appears that the site is suitable for the proposed development from an Engineering Geology standpoint provided that the recommendations provided herein are properly implemented into the project.

SOIL CONDITIONS

Alluvial soils were encountered in Borings B-1 and B-2 and sounding CPT-1 to the maximum depths explored. The near-surface soils within the upper 24 feet consisted predominantly of soft, compressible clays and silts. Below a depth of 24 feet below the ground surface, the alluvial deposits are interbedded, discontinuous strata of medium dense to dense silty sands and poorly-graded sands, and stiff to very stiff, silty clays and clayey silts.

Testing indicates that anticipated bearing soils lie in the "high" expansion range based on an expansion index value of 97. [A locally adopted version of this classification of soil expansion, Table 1809.7, is included in Appendix C of this report.] It appears that soils can be cut by normal grading equipment, but soils are several percent above optimum moisture content.

Groundwater was encountered at a depth of approximately 8 feet below the existing ground surface in both of the exploratory borings drilled for this study. According to mapping by the California Division of Mines and Geology (2002a), historically shallowest groundwater has been as shallow as 13 to 14 feet below the existing ground surface at the site.

A sample of near-surface soils was tested for pH, resistivity, soluble sulfates, and soluble chlorides. The test results provided in Appendix B should be distributed to the design team for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils. It should be noted that the sulfate content (1,955 mg/Kg) is at the upper limits of the "S1" exposure class of Table 4.2.1 of ACI 318; therefore, special concrete designs will be necessary for the measured sulfate contents. Earth Systems recommends that the concrete should have Type V Portland cement, a maximum water-cement ratio of 0.45, and a 28-day compressive strength of 4,500 psi.

Based on criteria established by the County of Los Angeles, measurements of resistivity of near-surface soils (628 ohms-cm) indicate that they are "severely corrosive" to ferrous metal (i.e. cast iron, etc.) pipes.

INFILTRATION FEASIBILITY TESTING

Infiltration testing was performed at the location of the proposed retention basin. Two infiltration tests were performed in accordance with the guidelines referenced in the Ventura County Technical Guidance Manual for Stormwater Quality Control Measures (TGM). A version of the falling-head borehole infiltration test method was used. The test results include both vertical and lateral infiltration from the borehole. Both tests were performed at a depth of 3 feet below the existing ground surface. Deeper testing was not feasible because of the relatively shallow depth to groundwater when the tests were performed (approximately 8 feet). After the borehole walls were drilled, a 2-inch nominal diameter slotted pipe was inserted in each test hole and the annulus between the borehole walls and the slotted pipes backfilled with pea-gravel. About 2 feet of water was then added to the bottom of the test holes and the water depth was monitored until almost all the water had percolated away. Subsequently, the holes were re-filled with about 2 feet of water and the drop in the water depth was measured after a period of time. For these tests, readings were taken at 30-minute intervals in the shallow test hole. The water level was adjusted after every reading. The tests were run until the rate that the water surface dropped had stabilized.

It should be noted that the rate the water surface drops in a borehole is a percolation rate, which is related to, but is not an infiltration rate. Percolation rate ignores the wetted soil surface area into which the water is infiltrating and does not account for the volume of water infiltrated. An infiltration rate considers both factors. Hence, percolation rates (in unit length per unit time) are an overestimation of infiltration rates (also in unit length per unit time). Earth Systems uses the Porchet equation to account for the wetted surface area and volume of water infiltrated to estimate an infiltration rate. Forms of the equation can be found in the Riverside County - Low Impact Development BMP Design Handbook (2001), the South Orange County Version, Technical Guidance Documents Appendices (2017), or in a paper by J.W. Van Hoorn, "Determining Hydraulic Conductivity with the Inversed Auger Hole and Infiltrometer Methods." The Porchet equation in its most simple form is the volume of water infiltrated divided by the product of the change in time and the wetted surface area. By substitution, the equation can be shown to be equal to:

$$\text{Infiltration Rate (inches /hr.)} = \frac{\Delta H * r * 60}{\Delta t * (r + 2H_{avg})}$$

where: ΔH = change is water level (inches)
Δt = change in time (minutes)
r = radius of test hole (inches)
H_{avg} = average height of water in test hole (inches)

The above equation does not account for the gravel pack in the annulus between the borehole wall and the slotted pipe fitted in the test hole. Ignoring the gravel pack inflates the amount of water infiltrated and, hence, yields an unconservative infiltration rate. A method to account for the volume occupied by the gravel (and the slotted pile) and adjust he infiltration rate accordingly is presented in Caltrans Test 750. Earth Systems makes this additional adjustment to our test data. The equation is:

$$\text{Correction Factor} = n * [\ 1 - (O/D)^2] + (I/D)^2$$

Where: n = pea gravel porosity
O = Outside diameter of slotted pipe (inches)
D = Test hole diameter (inches)
I = inside diameter of slotted pipe (inches)

Earth Systems has determined an average porosity for the pea gravel used in our testing. The other values are simple measurements.

The stabilized test infiltration rates for the depths tested and boring locations were determined using the above formulas and the measured percolation rates, and other test data. The data are presented on attached test sheets and summarized as follows:

Boring No.	Boring Depth (feet)	Average Infiltration Rate (in./hr.)
IT-1	3	0.2
IT-2	3	0.1

Both test results failed to satisfy the recommended minimum value infiltration systems (0.5 inches per hour) per the TGM. Hence, the project site does not appear to be suitable for on-site stormwater infiltration.

Please note that there are many factors that influence the infiltration rate. Clear water was used in all our tests, whereas oil residue, silt, organic matter, and other deleterious material will likely be contained in the stormwater. Variations in soil composition and density within the limits of a project site, and within the limits of the proposed stormwater disposal system are likely to affect infiltration characteristics. At a given location in a soil profile, horizontal and vertical infiltration rates can be quite different. The test measures neither but is a composite of the two.

GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

Based on the data provided in this report, it appears that the site is suitable for the proposed development from a Geotechnical Engineering standpoint provided that the recommendations provided herein are properly implemented into the project. Given the site conditions encountered, we conclude that either a rigid foundation system (i.e., mat or "waffle" foundations) or drilled piers should be used for support of the proposed structure. The primary geotechnical considerations from a development standpoint are as follows:

- The potential for about 1 inch of seismic-induced settlement due to liquefaction.
- The potential for about 0.5 feet of horizontal ground displacement due to lateral spreading.

- The upper 24 feet of native soil underlying the site are soft, compressible fine-grained soils that may consolidate or settle significantly under the anticipated structural loads.
- Shallow groundwater at a depth of approximately 8 feet below the existing ground surface.

Under the anticipated structural loads, conventional spread footings supported on at least 2.5 feet of compacted engineered fill could experience settlements on the order of 1.4 inches. Combined with the estimated seismic-induced settlement due to liquefaction of 1 inch, a conventional spread foundation would need to be designed to accommodate about 2.4 inches of total settlement (static and seismic), with a differential settlement of about 1.2 inches over a horizontal distance of 30 feet. Because of the estimated total and differential settlements (static and seismic), Earth Systems believes that a rigid foundation system (i.e., mat or “waffle” foundations) or drilled piers should be used for support of the proposed structure. Therefore, recommendations for a conventional spread foundation system have not been included in this report.

In addition to seismic-induced settlement due to liquefaction and static settlement due to the anticipated structural loads, the soft, compressible fine-grained soils underlying the site may consolidate or settle under the weight of the new fill anticipated to bring the site up to finished subgrade elevation. With as much as 6.5 feet of new fill being placed within the footprint of the proposed building, static settlement of the underlying native soils due to the weight of the new fill could be on the order of 2.7 inches. Settlement of the underlying native soils due to the weight of the new fill will impose downdrag forces on drilled piers, if used for support of the proposed building. Settlement of the underlying native soils due to the the new fill will also affect the proposed parking lot area. Surcharging the site prior to the commencement of construction activities will reduce the amount of settlement due to the weight of the new fill. The height of fill used to surcharge the site and the duration that the surcharge load should remain in order to mitigate the static settlement from the new fill will need to be evaluated if surcharging the site is considered.

Because of the shallow groundwater beneath the site, remedial grading beneath the proposed structure will be limited. In addition, the near-surface soils are expected to be at high moisture contents (i.e., 12 percent or higher above the optimum moisture content), and as a result significant drying will be necessary if the excavated soils are to be used as structural fill. Also, because of the anticipated wet soil conditions at the bottom of any remedial excavations or utility trench excavations, stabilization of the excavation bottoms will be required prior to placing fill.

If a drilled pier foundation system is used to support the proposed building, Earth Systems recommends that the drilled piers do not extend below a depth of 24 feet below the existing ground surface. Piers extending below a depth of 24 feet below the existing ground surface would be subjected to downdrag forces as the piers would penetrate potentially liquefiable zones. The diameter of the piers used to support the proposed structure should be such that the pier can accommodate the anticipated axial and lateral loads from the soils within the upper 24 feet below the existing ground surface. Pile capacity graphs for drilled piers embedded through 3 feet of compacted fill and 24 feet of the native soils underlying the site are presented in Appendix E of this report.

We understand that the existing detention basin that currently occupies the location of the proposed building will be relocated to an open field south of the proposed building. It is recommended that stormwater-related sediments accumulated in the bottom of the basin will be removed until native soils are encountered. The berms along the north, south and west sides should be removed. Assuming that the site will be raised to match the elevation of the adjacent paved interior roads along the north and east sides, fill thicknesses of approximately 3 to 6.5 feet are expected to be placed beneath the proposed building during site grading. Fill thicknesses within the proposed parking lot will range from approximately 0.5 to 4.5 feet. In other areas of the site to bring it up to finished subgrade elevation, fill thicknesses of approximately 0.5 to 6.5 feet are expected to be placed. Assuming these thicknesses of fill are placed to achieve finished subgrade elevations, there should only be limited overexcavation of the existing ground surface. Some overexcavation will be required in isolated areas to achieve the recommended thickness of compacted fill beneath the proposed improvements. The exposed surface in all areas to receive fill would need to be scarified and recompacted prior to fill placement to bring the site to finished grade.

The recommendations presented within do not address post-earthquake performance in regard to flatwork, pavements, etc. It is anticipated that it will not be economically feasible or cost effective to implement engineering measures to mitigate or reduce the potential for the occurrence of seismically-induced settlement across the whole site. The manifestation and effect of seismically-induced differential settlement may generally affect the flatwork, pavement, etc. It is likely that the effects of seismically-induced settlement, should they occur, will most likely require repair in the form of re-leveling portions of the site flatwork and pavement after a major seismic event.

Specific conclusions and recommendations addressing these geotechnical considerations, as well as general recommendations regarding the geotechnical aspects of design and construction, are presented in the following sections

A. Grading

1. Pre-Grading Considerations

a. Plans and specifications should be provided to Earth Systems prior to grading. Plans should include the grading plans, foundation plans, and foundation details.

b. Roof draining systems, if required by the appropriate jurisdictional agency, should be designed so that water is not discharged into bearing soils or near structures.

c. Final site grade should be designed so that all water is diverted away from the structures over paved surfaces, or over landscaped surfaces in accordance with current codes. Water should not be allowed to pond anywhere on the pad.

d. Shrinkage of on-site soils affected by compaction is estimated to be about 20 percent based on an anticipated average compaction of 92 percent.

e. It is recommended that Earth Systems be retained to provide Geotechnical Engineering services during site development and grading, and foundation construction phases of the work to observe compliance with the design concepts, specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

f. Compaction tests shall be made to determine the relative compaction of the fills in accordance with the following minimum guidelines: one test for each 2-foot vertical lift; one test for each 1,000 cubic yards of material placed; and two tests at finished subgrade elevation in the building pad.

2. Rough Grading/Areas of Development

   a. Grading at a minimum should conform to Appendix J in the 2016 California Building Code (CBC), and with the recommendations of the Geotechnical Engineer during construction. Where the recommendations of this report and the cited section of the 2016 CBC are in conflict, the Owner should request clarification from the Geotechnical Engineer.
   b. The existing ground surface should be initially prepared for grading by removing all vegetation, trees, large roots, debris, other organic material, and non-complying fill. Organics and debris should be stockpiled away from areas to be graded, and ultimately removed from the site to prevent their inclusion in fills. Voids created by removal of such material should be properly backfilled and compacted. No compacted fill should be placed unless the underlying soil has been observed by the Geotechnical Engineer.
   c. During abandonment of the existing detention basin, all loose sediments in the bottom of the basin should be removed to expose firm, native soils. The earth berms present along the north, south, and west sides should also be removed to expose native soils. The exposed surfaces should then be scarified to a depth of 6 inches; uniformly moisture-conditioned to above optimum moisture content, and compacted to achieve a relative compaction of between 90 percent of the ASTM D 1557 maximum dry density prior to the placement of engineered fill to achieve final grade.
   d. If a drilled pier foundation system will be used to support the proposed structure, a minimum of 2 feet of compacted fill should be provided below finished subgrade. The limits of the compacted fill should extend at least 5 feet beyond the outside edge of the proposed building footprint. If the thickness of compacted fill is provided by fill placed for raising the site, overexcavation other than the removals discussed above will not be required.
   e. If a mat foundation will be used to support the proposed structure, a minimum of 2 feet of compacted fill should be below the thickened edge. The limits of the compacted fill should extend at least 5 feet beyond the outside edge of the proposed building footprint. If the thickness of compacted fill is provided by fill placed for raising the site, overexcavation other than the removals discussed above will not be required.

f. Areas outside of the building area to receive exterior slabs-on-grade, sidewalks, and pavements should underlain by a minimum of 2 feet of compacted fill below finished subgrade. Some overexcavation will be required in the parking lot area to achieve the 2 feet of compacted fill below finished subgrade. The limits of the compacted fill should extend should extend at least 2 feet beyond the outside edge of the proposed improvement.

g. If overexcavation is not required to achieve the thicknesses of compacted fill beneath the proposed improvements as discussed above, the exposed surface following clearing operations should be scarified to a depth of 6 inches; uniformly moisture-conditioned to above optimum moisture content, and compacted to achieve a relative compaction of between 90 percent of the ASTM D 1557 maximum dry density. Compaction of the prepared subgrade should be verified by testing prior to the placement of engineered fill.

h. If additional overexcavation is required to achieve the thicknesses of compacted fill discussed above, the bottoms of all excavations should be observed by a representative of this firm prior to processing. The exposed surface at the bottoms of the excavations should be scarified to a depth of 6 inches; uniformly moisture-conditioned to above optimum moisture content, and compacted to achieve a relative compaction of between 90 percent of the ASTM D 1557 maximum dry density. Compaction of the prepared subgrade should be verified by testing prior to the placement of engineered fill.

i. Fill material placed against the slopes along the north and east sides of the subject site during site grading should be benched into the existing slopes as the fill placement progresses upward to finished subgrade elevation.

j. Engineered fill should be placed in a series of horizontal layers not exceeding 8 inches in loose thickness, uniformly moisture-conditioned to above optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D 1557 maximum dry density. Compaction of the engineered fill should be verified by testing. Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or if soil conditions are not stable. Discing, tilling, and/or blending may be required to uniformly moisture-condition soils used for engineered fill.

k. On-site soils may be used for fill once they are cleaned of all organic material, rock, debris and irreducible material larger than 6 inches. Excavated soils are expected to be at a high moisture content and drying will be necessary before replacing as compacted backfill.

l. Import soils used to raise site grade should be equal to, or better than, on-site soils in strength, expansion, and compressibility characteristics. Import soil can be evaluated, but will not be prequalified by the Geotechnical Engineer. Final comments on the characteristics of the import will be given after the material is at the project site.

m. Backfill around or adjacent to confined areas (i.e. interior utility trench excavations, etc.) may be performed with a lean sand/cement slurry (maximum 28-day compressive strength of 200 psi) or "flowable fill" material (a mixture of sand/cement/fly ash). The fluidity and lift placement thickness of any such material should be controlled in order to prevent "floating" of any "submerged" structure. Alternatively, a gravel backfill could be used, subject to approval by the Geotechnical Engineer and the City official.

n. If pumping soils or otherwise unstable soils are encountered during the overexcavation, stabilization of the excavation bottom will be required prior to placing fill. This can be accomplished by various means. The first method would include drying the soils as much as possible through scarification, and working thin lifts of "6-inch minus" crushed angular rock into the excavation bottom with small equipment (such as a D-4) until stabilization is achieved. Use of a geotextile fabric such as Mirafi 500X, or Tensar BX-1200, or the equivalent, is another possible means of stabilizing the bottom. If this material is used, it should be laid on the excavation bottom and covered with approximately 12 inches of "6-inch minus" crushed angular rock prior to placement of filter fabric (until the bottom is stabilized). The rock should then be covered with a geotextile filter fabric before placing fill above. It is anticipated that stabilization will probably be necessary due to the existing high moistures of the soils, and due to the shallow groundwater depth. Unit prices should be obtained from the Contractor in advance for this work.

3. <u>Excavations</u>

a. Excavations at the site will typically encounter clays and silts. These materials should be easily excavated with conventional earthmoving equipment.

b. Temporary unshored, unsurcharged, open excavations that are free of seeps and less than 10 feet deep in the **drained soils** may be cut at least 1H:1V (horizontal to vertical) or flatter provided the adjacent ground is not subject to surcharge loading. If excavations dry out, sloughing will occur. No excavation should be made within a 1:1 line projected downward from the outside edge at the base of any existing footing or slab.

c. During the time excavations are open, no heavy grading equipment or other surcharge loads (i.e. excavation spoils) should be allowed within a horizontal distance from the top of any slope equal to the depth of the excavation (both distances measured from the top of the excavation slope).

d. Adequate measures should be taken to protect any structural foundations, pavements, or utilities adjacent to any excavations.

e. All open cuts should be in compliance with applicable Occupational Safety Health Administration (OSHA) regulations (California Construction Safety Orders, Title 8) and should be monitored for evidence of incipient instability. Standard construction techniques should be sufficient for temporary site excavations. Project safety is the responsibility of the Contractor and the Owner. Earth Systems will not be responsible for project safety.

4. Utility Trenches

a. Utility trench backfill should be governed by the provisions of this report relating to minimum compaction standards. In general, on-site service lines may be backfilled with native soils compacted to 90 percent of the maximum dry density. Backfill of offsite service lines will be subject to the specifications of the jurisdictional agency or this report, whichever are greater.

b. Utility trenches running parallel to footings should be located at least 5 feet outside the footing line, or above a 1:1 (horizontal to vertical) projection downward from the outside edge of the bottom of the footing.

c. Compacted native soils should be utilized for backfill below structures. Sand should not be used under structures because it provides a conduit for water to migrate under foundations.

d. Backfill operations should be observed and tested by the Geotechnical Engineer to monitor compliance with these recommendations.

e. Jetting should not be utilized for compaction in utility trenches.

f. If the utility trench depths extend below the depth of the fill placed, the excavated soils are anticipated to be at a high moisture content and drying may be necessary before replacing as compacted backfill. If water is present in trenches, the lower sections of the trenches should be backfilled with gravel to at least 6 inches above the water.

B. Structural Design

1. Mat Foundations

A structural mat slab may be used to minimize the differential settlements resulting from seismic-induced settlement due to liquefaction and static settlement from the anticipated structural loads.

a. The mat foundation may be a conventionally reinforced slab system designed for the anticipated differential settlements.
b. The mat foundation for the proposed building should be supported by compacted fill prepared as recommended in Section A of this report.
c. Due to the expansion potential of the near-surface soils, the thickened edge along the perimeter of the mat foundation should extend at least 27 inches below the lowest adjacent grade.
d. To limit the maximum total settlement under static conditions to about 1 inch, an allowable "net" bearing capacity of 350 pounds per square foot (psf), for loads distributed over the full footprint of the mat foundations, may be utilized for dead and sustained live loads for design of the mat foundation. An allowable "net" bearing capacity of 2,000 psf may be used for thickened edges or other concentrated load areas bearing in compacted native soil. These values include a safety factor of at least 3.0 may be increased by one-third when considering transient loads such as earthquake or wind forces.
e. For subgrade soils beneath the structures consisting of compacted native soil, a modulus of subgrade reaction of k_1 = 100 kips per cubic foot is recommended. The value k_1 reflects a 1-square foot area and must be appropriately corrected for a loaded rectangular area of width B and length, m x B, using the formula: $k_s = (k_1)[(m+0.5)/1.5m]$.

Where: k_1= coefficient of subgrade reaction for 1-foot square plate (100 kcf)
B = width beneath column or bearing wall, in feet, where stresses are imposed on the ground

A value of B should be assumed to estimate the k_s value in the initial structural analysis. Then, the calculated B value (from the initial structural analysis) should be used to re-calculate the k_s value. Additional structural analyses (iterations) should be made using re-calculated k_s values in the same manner, as appropriate, until the B value calculated from the structural analysis is consistent with the B value used to calculate k_s.

f. The actual depth, width, and reinforcement requirements for the mat foundation should be specified by the project Structural Engineer.

g. The Structural Engineer should account for the estimated static and seismically-induced settlements (total and differential) in the mat foundation design.

2. Drilled Pier Foundations

Drilled piers may be used for support of the proposed structure. Piers may consist of drilled, reinforced cast-in-place concrete caissons (cast-in-drilled-hole "CIDH" piles). Steel reinforcing may consist of "rebar cages" or structural steel sections.

a. At a minimum, the new piers should be at least twelve inches (12") in diameter and embedded into firm, native soils. The Geotechnical Engineer should be consulted during pier installation to determine compliance with the geotechnical recommendations.

b. For vertical (axial compression) and uplift capacity, the attached pile capacity graphs may be used. Drilled pier diameters of 1, 1.5, and 2 feet were analyzed, and the results are presented on the attached charts. Side resistance is not allowed to increase beyond a depth equal to 20 pile diameters. Upward resistance is taken as two-thirds of the downward resistance. The downward and upward capacity graphs for drilled piers are presented in Appendix E.

c. The load capacities shown on the attached charts are based upon skin friction with no end bearing. These allowable capacities include a safety factor of 2.0, and may be increased by one-third when considering transient loads such as wind or seismic forces.

d. Reduction in axial capacity due to group effects should be considered for piers spaced at 3 diameters on-center or closer.

e. All piers should be tied together laterally (in both directions) at the top with grade beams. The size, spacing, and reinforcing of grade beams should be determined by the Structural Engineer.

f. The compressive and tensile strength of new pier designs should be checked to verify the structural capacity of the piers. Reinforcement of piers should be specified by the Structural Engineer. The specific method of pier installation will affect the performance of the piers. Earth Systems recommends a meeting with the design team and Contractor to verify that the specific method of pier installation can provide the anticipated load supporting capacity.

g. Lateral (horizontal) loads may be resisted by passive resistance of soil against the piers. An equivalent fluid weight (EFW) of 360 psf per foot of penetration in firm, native soil above the groundwater table may be used for lateral load design. Below the groundwater table, an EFW of 150 pcf may be used. These resisting pressures are ultimate values. The maximum passive pressure used for design should not exceed 3,100 psf.

h. For piers spaced at least 3 diameters apart, an effective width of 2 times the actual pier diameter may be used for passive pressure calculations.

i. Assuming 12-inch diameter piers of reinforced concrete that are fixed against rotation at the head, the “point of fixity” was estimated to be located at least 6 feet below the final ground elevation based on commonly accepted engineering procedures (Lee, 1968). For 18-inch diameter drilled piers, this depth will increase to about 9 feet.

m. It is the Structural Engineer’s responsibility to design the reinforcement for the piers to sustain the imposed axial and lateral loading.

n. Due to the presence of relatively shallow groundwater, temporary casing may be necessary to minimize bore-hole caving during pier construction. Use of special drilling mud or other methods to keep boreholes open during construction may be acceptable upon review by the Geotechnical Engineer.

3. <u>Downdrag Forces on Drilled Piers</u>

   a. Settlement of the underlying native soils due to the weight of the new fill will impose downdrag forces on drilled piers used for support of the proposed building. Downdrag loads will need to be considered by the project Structural Engineer in the design of the drilled piers, if used.

b. A negative skin friction value of 2.1 kips/foot should be used for drilled piers that extend to a depth of at least 20 feet. As previously discussed, piers extending below a depth of 24 feet below the existing ground surface would be subjected to additional downdrag forces as the piers will penetrate potentially liquefiable zones.

c. The downdrag force to be carried by the drilled piers, in addition to the structural loads, can be determined by multiplying the circumference of the drilled piers (in feet) by a negative skin friction value of 2.1 kips/foot.

d. As downdrag occurs, the soils undergoing downdrag will not provide downward capacity for support of the structure. The project Structural Engineer should neglect the downward axial capacity provided in the upper 14 feet shown on the downward capacity graphs for drilled piers presented in Appendix F.

4. Slabs-on-Grade

a. Because of the potential for seismically-induced settlement of the underlying soils, Earth Systems recommends that the interior concrete slabs of the proposed structure should be designed as structural slabs that do not rely on the subgrade soils for support. There is the alternative of allowing them to get damaged, and repairing and/or replacing any damaged portions after a major seismic event. The owner will need to decide whether it is economically feasible or cost effective to design the interior concrete slabs of the proposed structure as structural slabs to mitigate the potential effects of seismically-induced settlement. If the owner decides that allowing them to get damaged and repairing and/or replacing any damaged portions is more economically feasible or cost effective, the interior concrete slabs should be supported on at least 2.5 feet of compacted engineered fill prepared as recommended in Section A of this report for exterior concrete slabs and pavement.

b. Exterior concrete slabs (i.e., flatwork, sidewalks, etc.) will be supported on compacted engineered fill prepared as recommended in Section A of this report.

c. It is recommended that perimeter slabs (walks, patios, etc.) be designed relatively independent of footing stems (i.e. free floating) so foundation adjustment will be less likely to cause cracking.

d. Soils underlying exterior concrete slabs that are in the "high" expansion range should be pre-moistened prior to placing of sand, reinforcing steel, or concrete.

e. Exterior concrete slabs bottomed on soils in the "high" expansion range should be underlain with a minimum of 4 inches of “clean” sand (i.e., 5% fines or less).

f. Where dampness of interior floor slabs of the proposed resort is to be minimized, the slabs should be constructed on a minimum 4-inch-thick layer of capillary break material covered with a high-quality vapor retarder. The vapor retarder should have a minimum thickness of 15 mils, a permeance as tested before and after mandatory conditioning (ASTM E 1745, Section 7.1.2 – 7.1.5) of less than 0.01 perms [grains/(ft^2 hr. in. Hg)], and comply with the ASTM E 1745 Class A requirements.

g. Interior slab surfaces to receive moisture sensitive floor coverings should have considerations for maximum vapor emission levels.

5. <u>Frictional and Lateral Coefficients</u>

a. Resistance to lateral loading may be provided by friction acting on the base of foundations. Assuming the mat foundations will be found into compacted native soils a coefficient of friction of 0.53 may be applied to dead load forces. This value does not include a factor of safety.

b. Passive resistance acting on the sides of the thickened edge of the mat foundation in compacted native soils equal to 310 pcf of equivalent fluid weight may be included for resistance to lateral load. This value does not include a factor of safety.

c. A minimum factor of safety of 1.5 should be used when designing for sliding or overturning.

d. For the building foundations, passive resistance may be combined with frictional resistance provided that a one-third reduction in the coefficient of friction is used.

6. Settlement Considerations

a. With as much as 6.5 feet of new fill being placed within the footprint of the proposed building, static settlement of the underlying native soils due to the weight of the new fill could be on the order of 2.7 inches. Surcharging the site prior to the commencement of construction activities will reduce the amount of settlement due to the weight of the new fill.

b. In the event of a strong seismic event, the native soils underlying the site will undergo seismically-induced settlement due to liquefaction. The estimated seismic-induced settlement is about 1 inch.

c. A maximum static settlement of about 1 inch is anticipated for mat foundations designed for loads to be distributed at a bearing pressure of 350 pounds per square foot (psf) over the full footprint of the mat foundation.

d. A maximum static settlement of about 0.5 inch is anticipated for drilled piers embedded within the upper 24 feet below existing site grade.

e. Differential settlement between adjacent load bearing members should be about one-half the total settlement (static and seismically-induced) over a horizontal distance of 30 feet.

7. Preliminary Asphalt Pavement Sections

a. Based on the exploratory borings drilled by Earth Systems, the near-surface native soils within the proposed paved areas are generally silts and clays that have a low traffic support capacity when recompacted and used as pavement subgrade. A resistance value (R-value) test performed on an untreated sample of the native subgrade soils yielded an R-value of 8.

b. Asphalt pavement sections for untreated subgrade soils are presented below based on an R-value of 8; current Caltrans design procedures, and traffic indices ranging from 4.0 to 7.0. The traffic index (TI) is a measure of traffic wheel loading frequency and intensity of anticipated traffic. For comparison, TI’s between 4 and 6 are often suitable for design of automobile parking areas, TI’s between 5 and 6 are commonly used for design of fire truck access lanes and areas subject to channelized flow with light delivery trucks, and TI’s greater than 6 are common for design of pavements supporting light to moderate bus and truck traffic. Traffic indices assumed above should be reviewed by the project Owner, Architect, and/or Civil Engineer to evaluate their suitability for this project.

TRAFFIC INDEX	ASPHALT-CONCRETE (INCH)	AGGREGATE BASE (INCH)
4.0	3.0	5.5
4.5	3.0	7.5
5.0	4.0	7.0
5.5	4.0	9.0
6.0	4.0	11.0
6.5	5.0	11.0
7.0	5.0	13.0

c. The preliminary paving sections provided above have been designed for the type of traffic indicated. If the pavement is placed before construction on the project is complete, construction loads, which could increase the traffic index values assumed above, should be taken into account.
d. The subgrade soils in the upper 12 inches below the finished subgrade elevation should be properly moisture conditioned to over optimum moisture content and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density. The subgrade soils should be in a stable, non-pumping condition at the time the aggregate base material is placed and compacted.
e. Aggregate base materials should conform to the specifications stated in the 2015 "Greenbook" and be compacted as engineered fill to at least 95 percent compaction.
f. Asphalt paving materials and placement methods should meet specifications stated in the 2015 "Greenbook" for asphalt concrete.
g. Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become continuously wet.
h. All concrete curbs separating pavement and landscaped areas should extend at least 6 inches into the subgrade and below the bottom of the adjacent aggregate base to provide a barrier against lateral migration of landscape water or runoff into the pavement section.
i. Periodic maintenance should be performed to repair degraded areas and seal cracks with appropriate filler.

If imported fill material will be used to raise the site, and differs from the native subgrade soils encountered in our borings and tested in the laboratory, we recommend that a representative sample of the imported fill material be obtained and R-value testing be performed. If the results of the R-value testing vary significantly from those assumed, the pavement sections presented above will need to be revised.

8. Preliminary Concrete Paving Sections

a. For rigid pavements in heavy traffic driveways and access lanes, the following minimum criteria may be used for design:

Concrete thickness (parking area and interior lanes) =	5.75 inches
Concrete thickness (entrance and exterior lanes) =	6.75 inches
PMB or Class II base thickness under concrete =	4.0 inches
Compressive strength of concrete, fc =	4,000 psi at 28 days
Modulus of flexural strength of 4,000 psi concrete =	595 psi
Maximum spacing of contraction joints, each way =	15 feet

b If additional resistance to cracking is desired beyond that provided by the contraction joints, steel reinforcement can be added to the pavement section at approximately 2 inches below the top of concrete; however, reinforcement is not required.

c The preliminary paving sections discussed above have been designed for the type of traffic indicated. If the pavement is placed before construction on the project is complete, construction loads should be taken into account. If bus traffic is expected to exceed 10 per day, these sections should be reevaluated. Traffic should not be allowed on the pavement until 28 days after concrete placement, or until the 28-day design strength is achieved.

ADDITIONAL SERVICES

This report is based on the assumption that an adequate program of monitoring and testing will be performed by Earth Systems during construction to check compliance with the recommendations given in this report. The recommended tests and observations include, but are not necessarily limited to the following:

1. Review of the building and grading plans during the design phase of the project.
2. Observation and testing during site preparation, grading, placing of engineered fill, and foundation construction.
3. Consultation as required during construction.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analysis and recommendations submitted in this report are based in part upon the data obtained from the CPT sounding and the borings advanced on the site. The nature and extent of variations between and beyond the sounding and borings may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

The scope of services did not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statements in this report or on the soil boring logs regarding odors noted, unusual or suspicious items or conditions observed, are strictly for the information of the client.

Findings of this report are valid as of this date; however, changes in conditions of a property can occur with passage of time whether they are due to natural processes or works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur whether they result from legislation or broadening of knowledge. Accordingly, 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 1 year.

In the event that any changes in the nature, design, or location of the structures and other improvements are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed, and conclusions of this report modified or verified in writing.

This report is issued with the understanding that it is the responsibility of the Owner, or of his representative to insure the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

As the Geotechnical Engineers for this project, Earth Systems has striven to provide services in accordance with generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the exclusive use of the Client for the purposes stated in this document for the referenced project only. No third party may use or rely on this report without express written authorization from Earth Systems for such use or reliance.

It is recommended that Earth Systems be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems is not accorded the privilege of making this recommended review, it can assume no responsibility for misinterpretation of the recommendations.

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APPENDIX A

Field Study
Vicinity Map
Regional Fault Map
Regional Geologic Map 1
Regional Geologic Map 2
Seismic Hazard Zones Map
Historical High Groundwater Map
Site Plan
Boring Logs
Symbols Commonly Used on Boring Logs
Unified Soil Classification
Log of Cone Penetrometer Test Sounding
Interpretation of Cone Penetrometer Test Sounding

FIELD STUDY

A. On March 28, 2019, one Cone Penetrometer Test (CPT) sounding was performed to obtain information pertaining to the soil profile. The sounding was advanced to a depth of approximately 50 feet. The CPT sounding was advanced using equipment owned and operated by Middle Earth. During advancement of the cone penetrometer, readings of sleeve friction (in tons per square foot), tip resistance (also in tons per square foot), and friction ratio (in percent) were recorded at 0.05-meter intervals as per ASTM D 5778 and ASTM D 3441. The approximate locations of the test sounding was determined in the field by pacing and sighting, and are shown on the Site Plan in this Appendix.

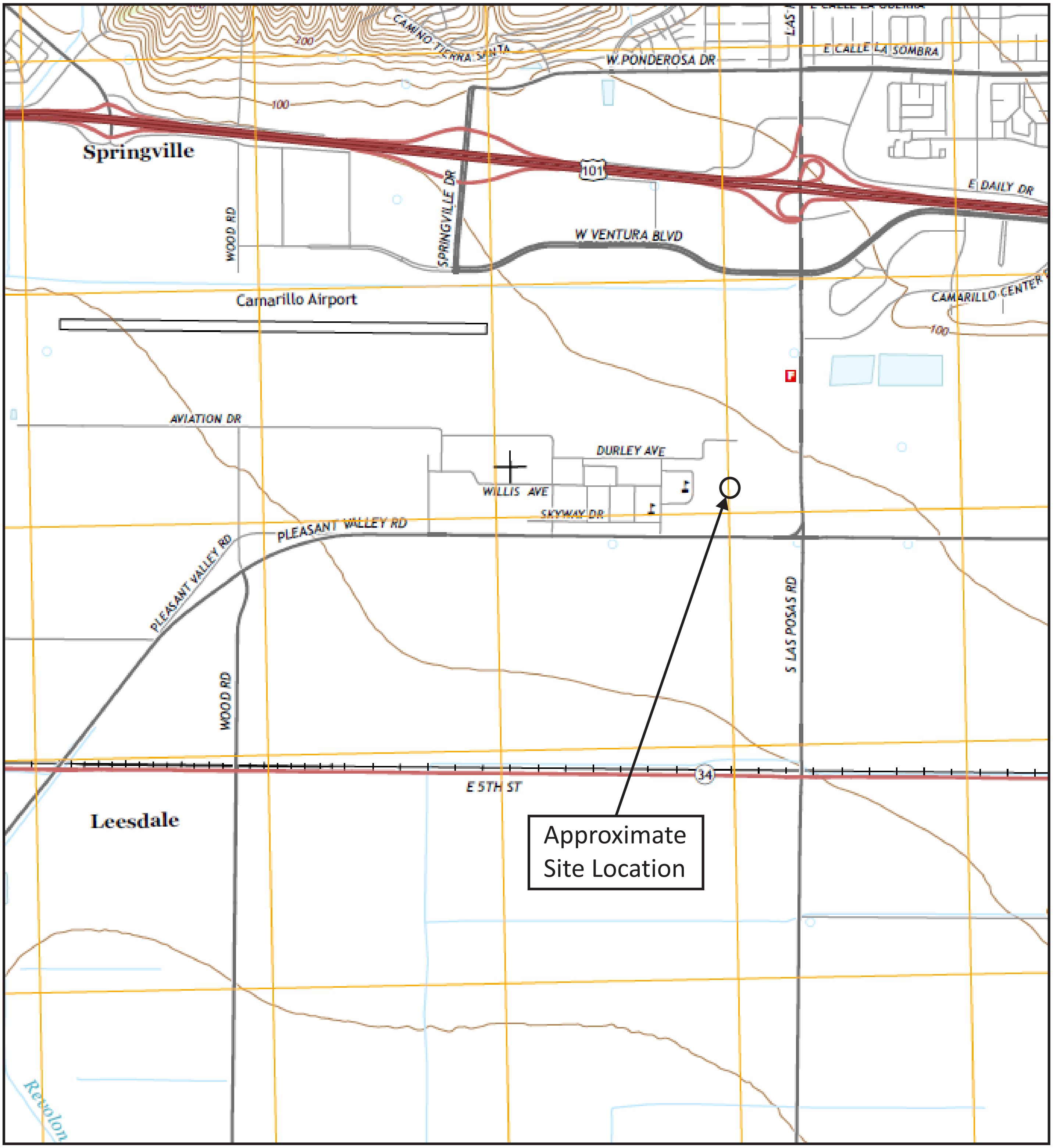
B. On March 19, 2019, two (2) exploratory borings (B-1 and B-2) were drilled to observe the soil profile and to obtain samples for laboratory analysis. Boring depths ranged from approximately 16.5 feet to 31.5 feet below the existing ground surface. The borings were drilled using a hollow stem 8-inch diameter continuous flight auger powered by a CME-75 truck mounted drilling rig. The approximate locations of the test borings were determined in the field by pacing and sighting, and are shown on the Site Plan in this Appendix

C. While onsite for drilling the exploratory borings, two other borings (I-1 and I-2) were drilled for infiltration testing. The borings were drilled using a hollow stem 8-inch diameter continuous flight auger powered by a CME-75 truck mounted drilling rig. The approximate locations of the infiltration test borings were determined in the field by pacing and sighting, and are shown on the Site Plan in this Appendix.

D. Samples were obtained within the test borings with a Modified California (M.C.) ring sampler (ASTM D 3550 with shoe similar to ASTM D 1586), and with a Standard Penetration Test (SPT) sampler (ASTM D 1586). The M.C. sampler has a 3-inch outside diameter, and a 2.42-inch inside diameter when used with brass ring liners (as it was during this study). The SPT sampler has a 2-inch outside diameter and a 1.37-inch inside diameter, but when used without liners, as was done for this project, the inside diameter is 1.63 inches. The samples were obtained by driving the sampler with a 140-pound hammer dropping 30 inches in accordance with ASTM D 1586. The hammer was operated with an automatic trip hammer.

E. A bulk (disturbed) sample of the near-surface materials was obtained from upper 5 feet during the drilling of boring B-1. The sample was secured for classification and testing purposes and represent a mixture of soils and bedrock within the noted depths.

F. The final logs of the test borings represent interpretations of the contents of the field logs and the results of laboratory testing performed on the samples obtained during the subsurface study. The final boring logs, as well as log and interpretation of the CPT sounding are included in this Appendix.



*Taken from USGS Topo Map, Camarillo Quadrangle, California, 2015.

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

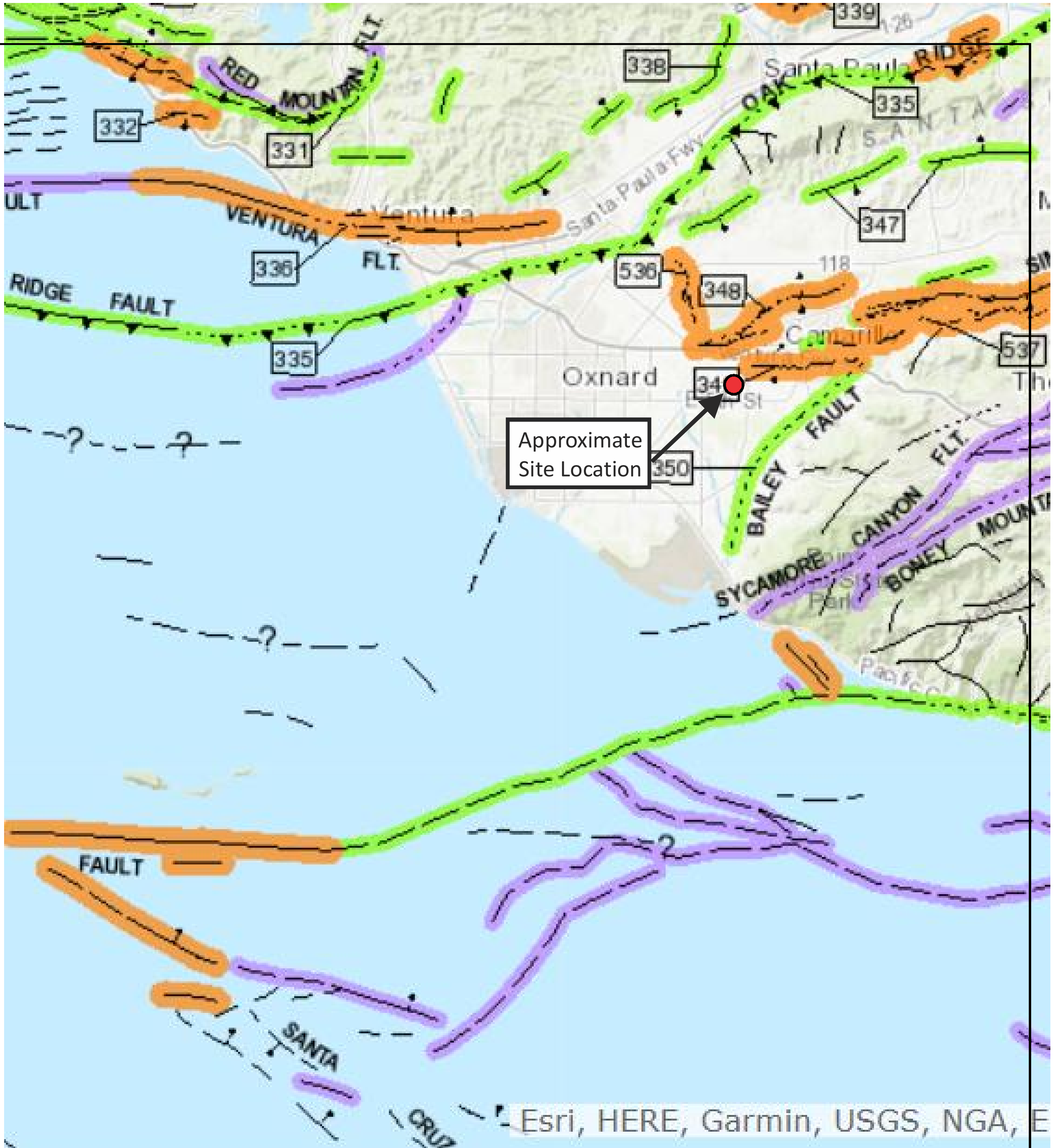
N

VICINITY MAP

Oxnard College Fire Academy
Camarillo, California

Earth Systems

July 2018	302245-001



*Taken from DMG OFR, Fault Activity Map of California , 2010.

Approximate Scale: 1" = 5 Miles

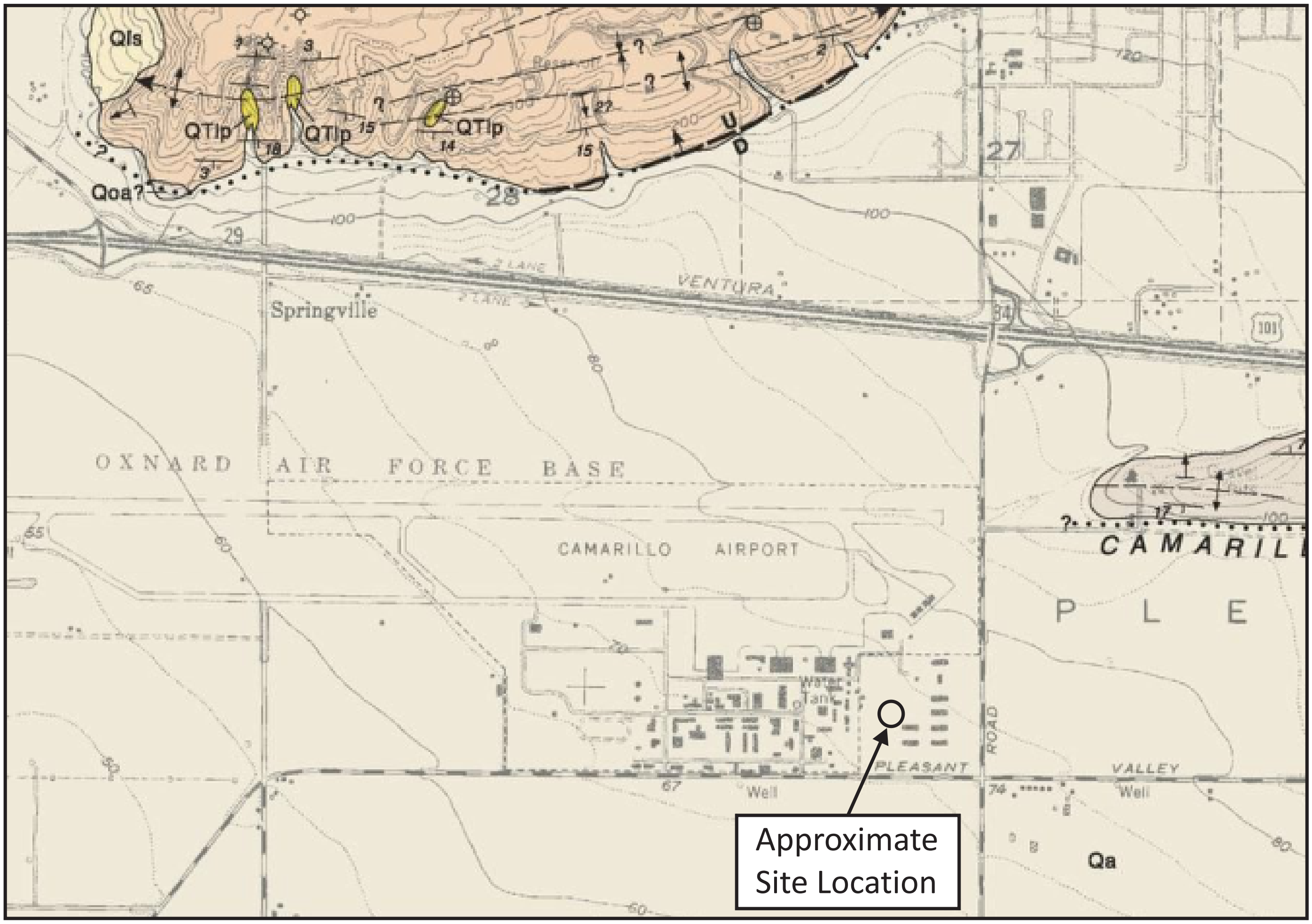
N

REGIONAL FAULT MAP

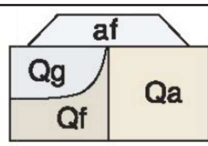
Oxnard College Fire Academy
Camarillo, California

Earth Systems

June 2019	302245-001

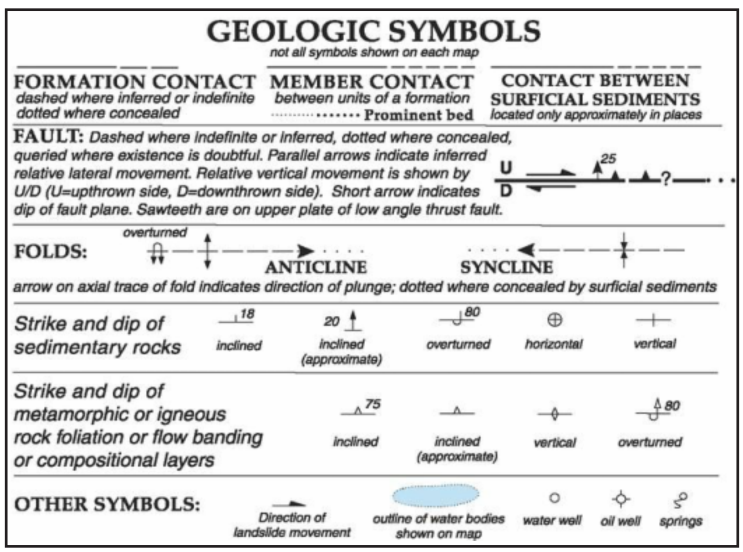


*Taken from Dibblee, Jr., Geologic Map of The Camarillo and Newbury Park Quadrangles, Ventura County, California, 1990, DF-28.



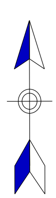
SURFICIAL SEDIMENTS

af *Artificial fill*
Qg *Stream channel sand and gravel*
Qf *Alluvial fan gravel and sand, locally slightly indurated*
Qa *Alluvium: gravel, sand and clay of flatlands*



Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

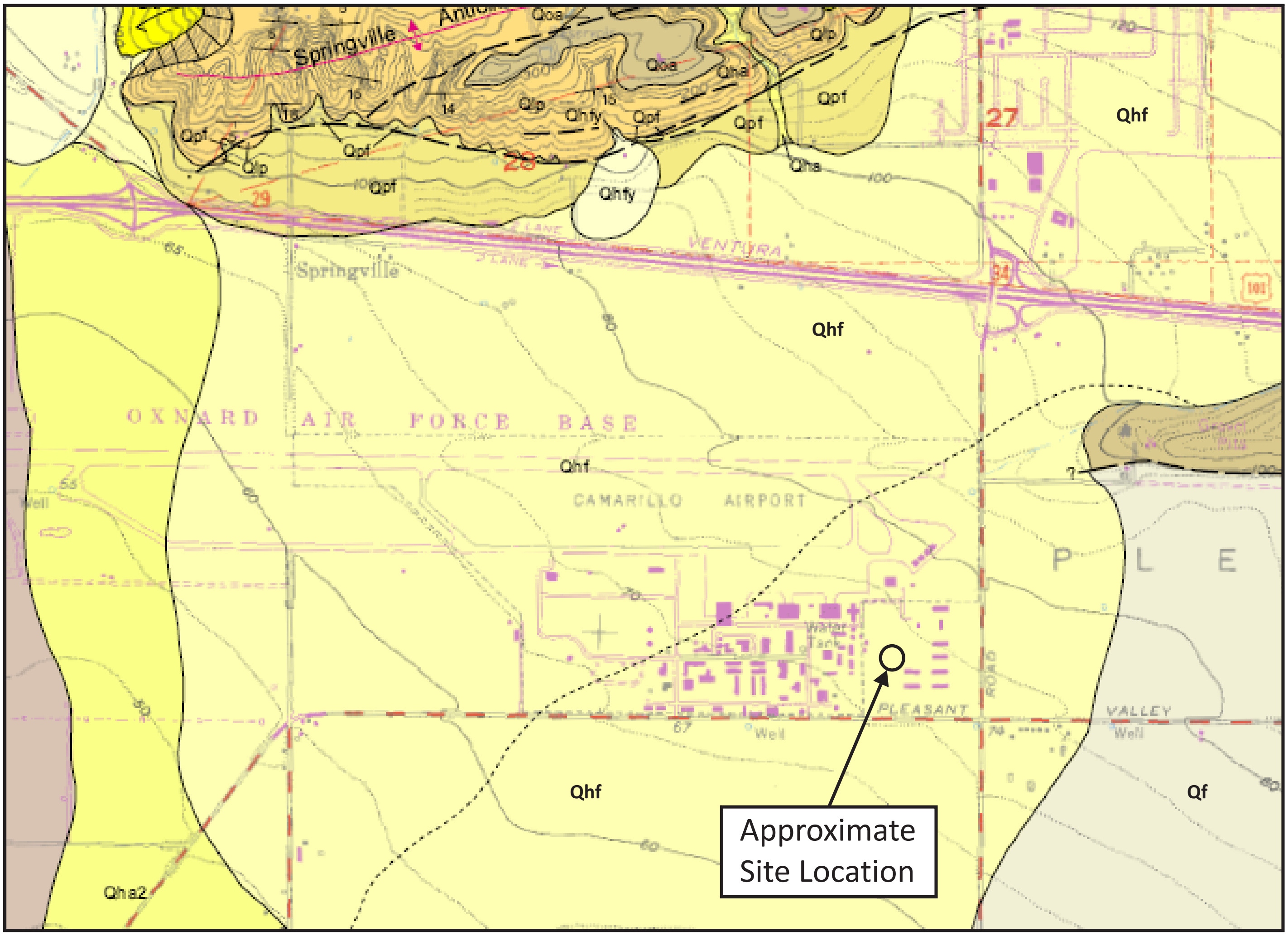


REGIONAL GEOLOGIC MAP 1

Oxnard College Fire Academy
Camarillo, California

Earth Systems

July 2018	302245-001



*Taken from USGS, SCAMP Geologic Map of the Camarillo 7.5' Quadrangle, Ventura County, California, 2004.

Qhf	Alluvial fan deposits (Holocene) - Includes active fan deposits, deposited by streams emanating from mountain canyons to the north onto the alluvial valley floor; deposits originate as debris flows, hyperconcentrated mudflows or braided stream flows; composed of moderately to poorly sorted and moderately to poorly bedded sandy clay with some silt and gravel.
Qf	Alluvial fan deposits (late Pleistocene to Holocene) - Deposited on gently sloping, relatively undissected alluvial surfaces where deposits might be of either late Pleistocene or Holocene age, composed of moderately to poorly sorted sand, gravel, silt , and clay.
————--------	Contact between map units - Generally approximately located or inferred, dotted where concealed.
- - - - - - - - -	Contact between similar map units of different relative age - Recognized by scour and incised channelling features. Generally approximately located.
——— — — —-----?--	Fault - Generally approximately located or inferred, dotted where concealed, queried where location is uncertain.

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

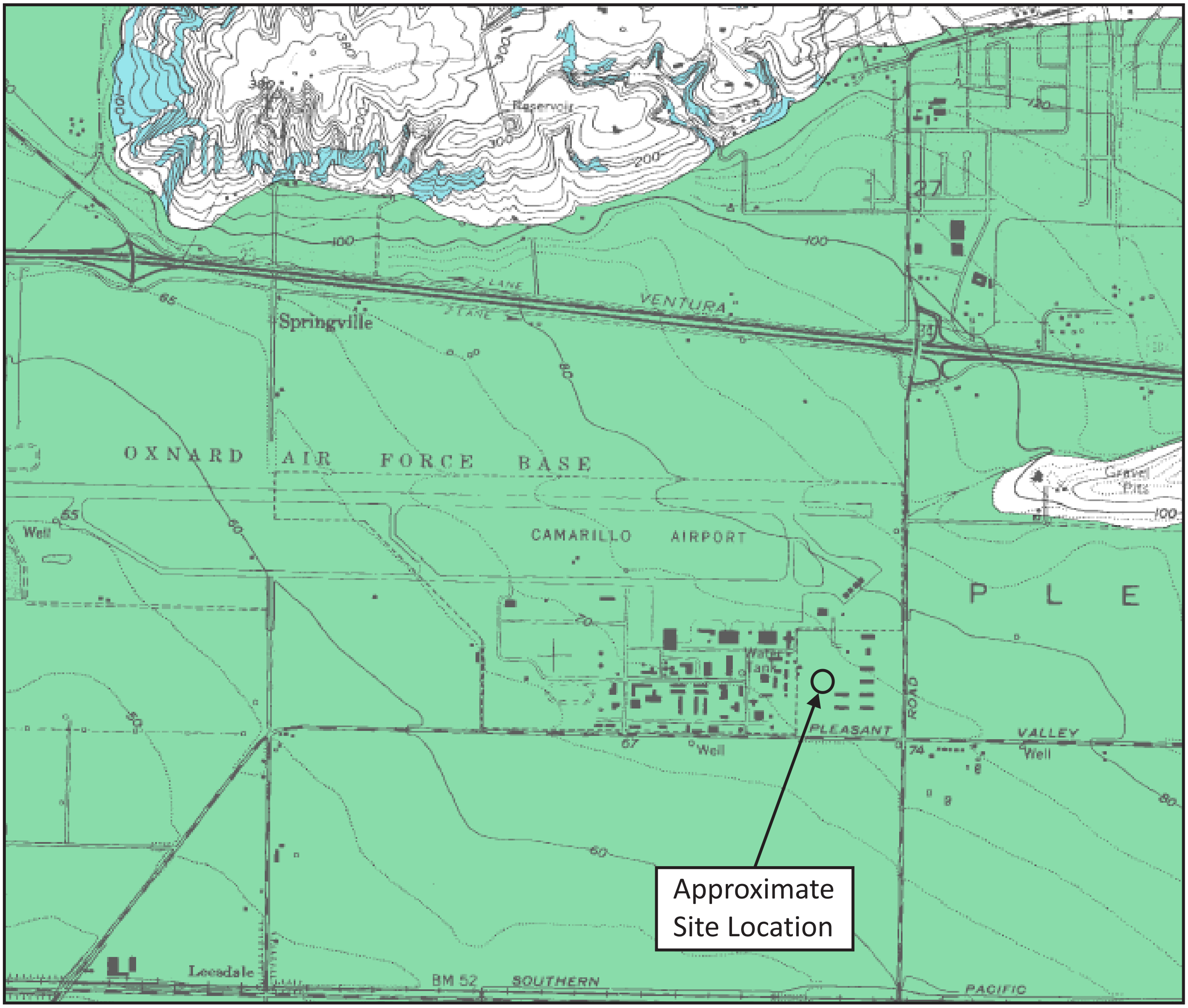
N

REGIONAL GEOLOGIC MAP 2

Oxnard College Fire Academy
Camarillo, California

Earth Systems

July 2018	302245-001



MAP EXPLANATION

Zones of Required Investigation:

Liquefaction

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

Earthquake-Induced Landslides

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

NOTE: Seismic Hazard Zones identified on this map may include developed land where delineated hazards have already been mitigated to city or county standards. Check with your local building/planning department for information regarding the location of such mitigated areas.

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

STATE OF CALIFORNIA

SEISMIC HAZARD ZONES

Delineated in compliance with
Chapter 7.8, Division 2 of the California Public Resources Code
(Seismic Hazards Mapping Act)

CAMARILLO QUADRANGLE

OFFICIAL MAP

Released: February 7, 2002

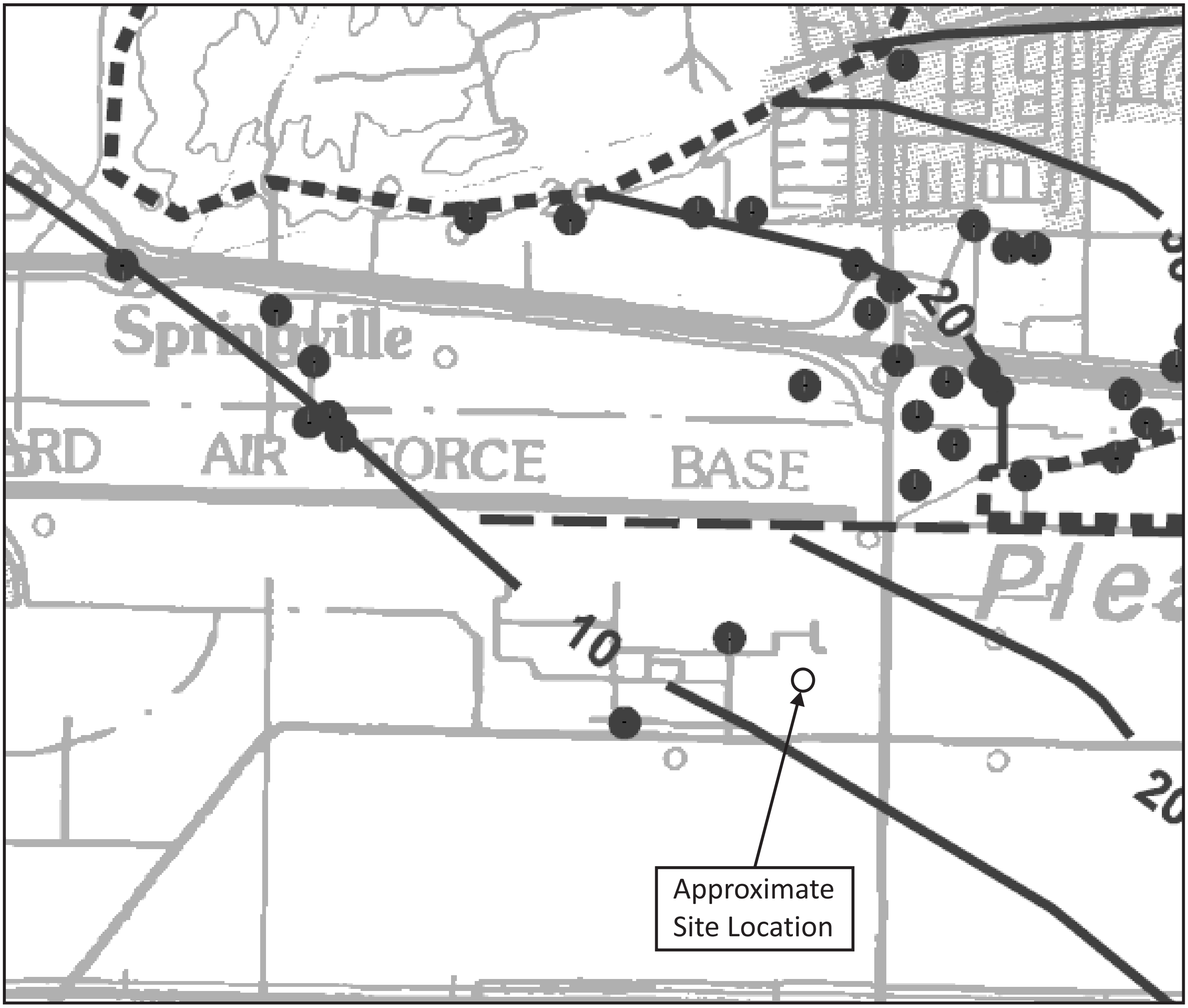
N

SEISMIC HAZARD ZONES MAP

Oxnard College Fire Academy
Camarillo, California

Earth Systems

July 2018	302245-001



*Taken from Seismic Hazard Zone Report For The Camarillo 7.5 Minute Quadrangle, Ventura County, California, 2002.

LEGEND

30 Depth to historically highest ground water level in feet

Borehole Site

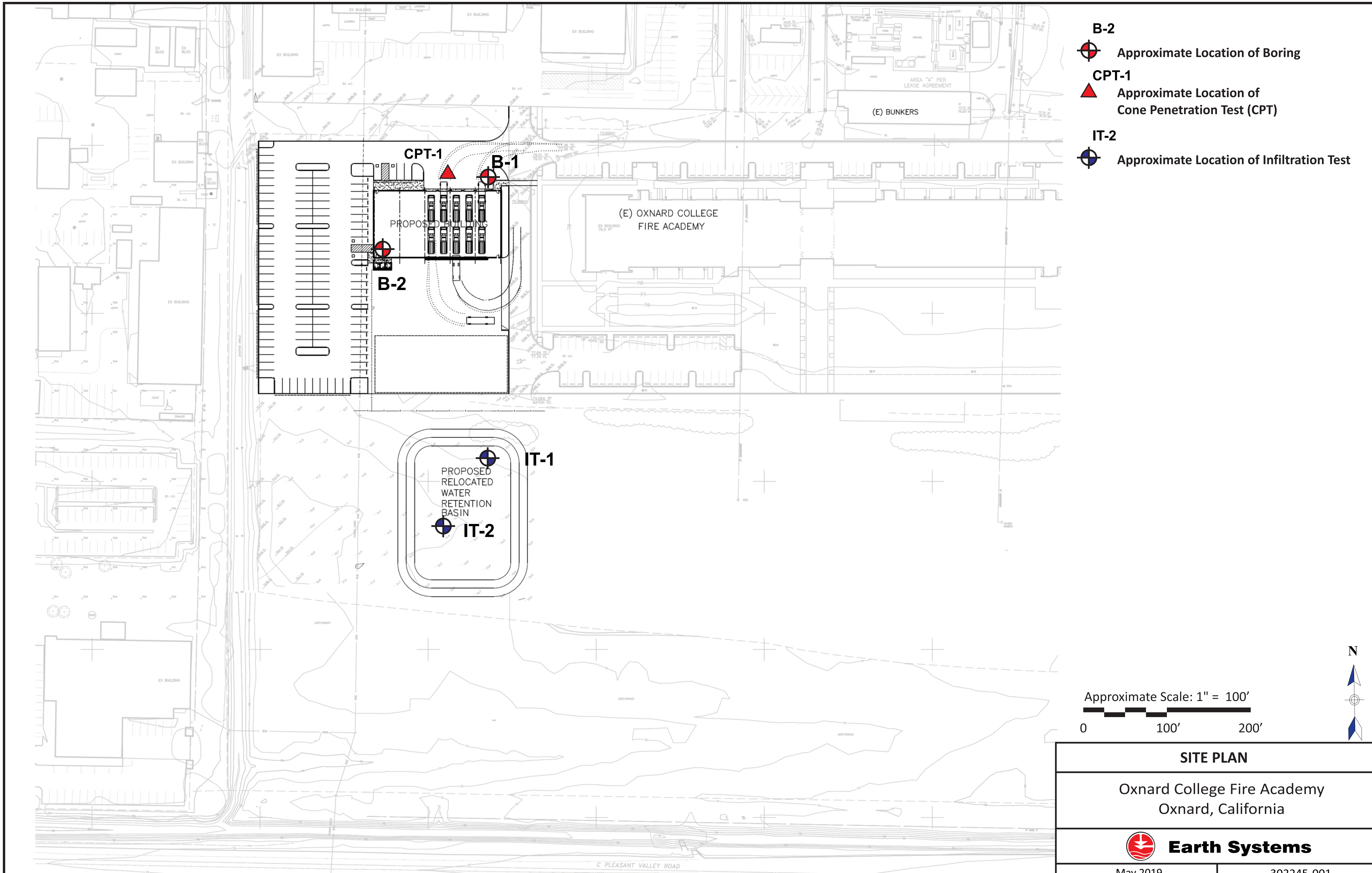
Valley / Mountains Boundary

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

N

HISTORICAL HIGH GROUNDWATER	
Oxnard College Fire Academy Camarillo, California	
Earth Systems	
July 2018	302245-001



SITE PLAN

Oxnard College Fire Academy
Oxnard, California

Earth Systems

May 2019

302245-001

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-1	DRILLING DATE: March 19, 2019
PROJECT NAME: Oxnard College Fire Academy	DRILL RIG: CME-75
PROJECT NUMBER: 302245-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: SC

Vertical Depth	PENETRATION RESISTANCE (BLOWS/6")	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0					
	3/6/8	CL/ML	91.8	23.9	**ALLUVIUM:** Mottled olive brown silty clay to clayey silt; stiff; moist.
5	1/1/2	CL	81.2	36.6	**ALLUVIUM:** Dark yellowish brown silty clay; soft; very moist.
	1/1/2	CL	72.7	45.3	As above; with caliche.
10	1/3/2	CL	78.4	43.1	**ALLUVIUM:** Dark yellowish brown silty clay; minor sand; some caliche; soft; very moist to wet.
15	1/2/3	CH		43.1	**ALLUVIUM:** Interbedded dark yellowish brown fat clay; caliche; medium stiff; wet.
20	2/3/7	SC/ CL			**ALLUVIUM:** Olive brown sandy clay to clayey sand; medium dense to stiff; wet.
25	8/16/19	SM/ SP			**ALLUVIUM:** Interbedded pale yellowish brown fine silty sand and fine sand; dense; wet.
30	8/10/10	SM/ SP			**ALLUVIUM:** Interbedded pale yellowish brown sandy silt; silty sand and fine sand; medium dense; wet.
35					

Sample Type: Bulk, SPT, Mod. Calif. — SYMBOL

Total Depth: 31.5 feet.

Groundwater Depth 8.0 feet.

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-2
PROJECT NAME: Oxnard College Fire Academy
PROJECT NUMBER: 302245-001
BORING LOCATION: Per Plan

DRILLING DATE: March 19, 2019
DRILL RIG: CME-75
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: SC

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
				2/2/2		CL/ML	80.7	39.6	**ALLUVIUM:** Mottled olive brown silty clay to clayey silt; soft; moist.
5				1/1/1		CL	69.8	50.2	Same as above; with caliche and very soft.
10				1/2/3		CL			**ALLUVIUM:** Dark olive brown silty clay; caliche; soft; very moist to wet.
15				1/1/1		CL/ ML			**ALLUVIUM:** Dark olive brown silty clay; caliche; soft; very moist to wet.
20									
25									
30									
35									

Total Depth: 16.5 feet.
Groundwater Depth 8.0 feet.

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

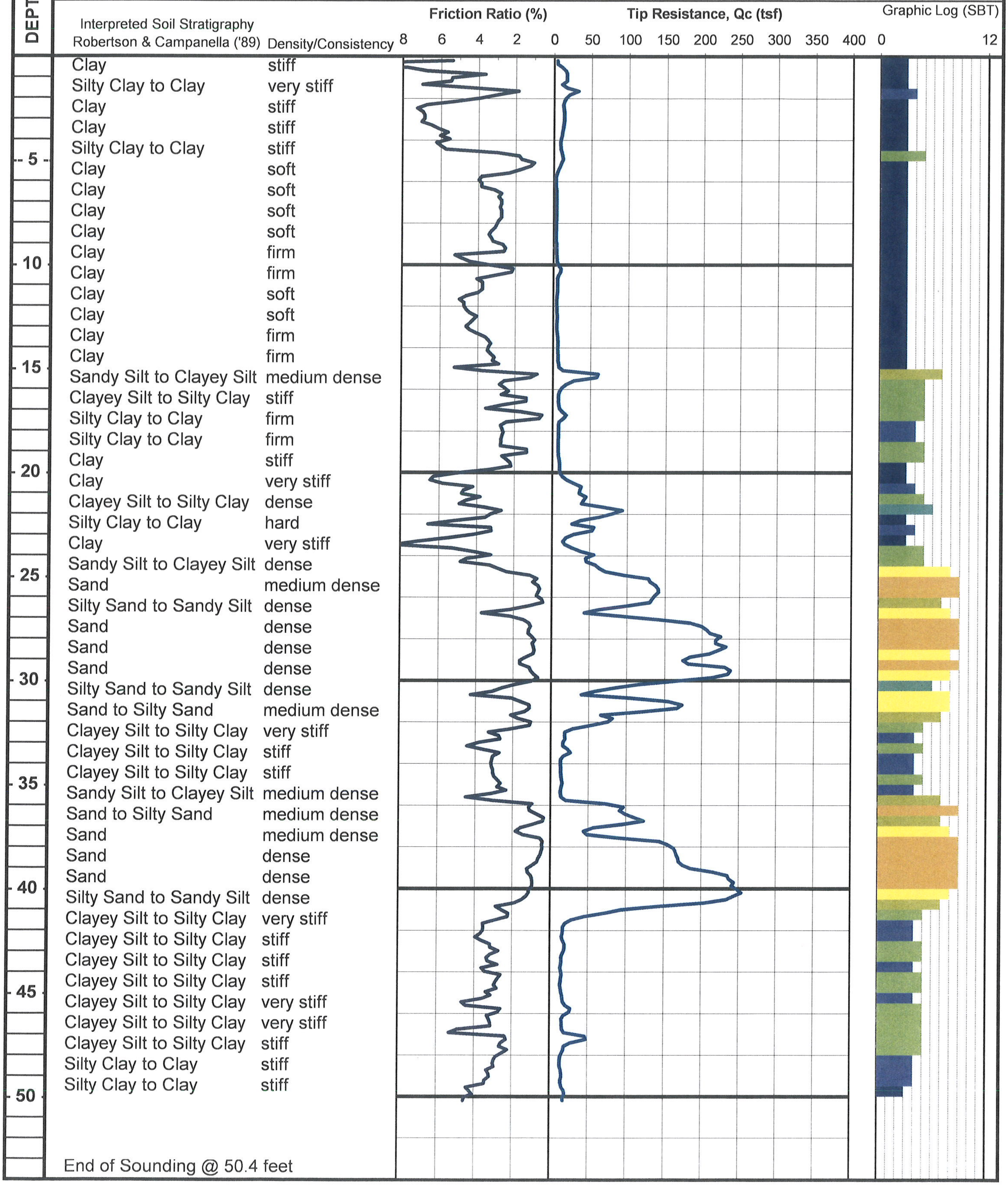
Earth Systems

CPT No: CPT-1
Project Name: Oxnard College Fire Academy
Project No.: 302245-001
Location: See Site Exploration Plan

CPT Vendor: **Middle Earth GeoTesting**
Truck Mounted Electric
Cone with 23-ton reaction
Date: 3/28/2019

DEPTH (FEET)

Interpreted Soil Stratigraphy Robertson & Campanella ('89)	Density/Consistency
Clay	stiff
Silty Clay to Clay	very stiff
Clay	stiff
Clay	stiff
Silty Clay to Clay	stiff
Clay	soft
Clay	soft
Clay	soft
Clay	soft
Clay	firm
Clay	firm
Clay	soft
Clay	soft
Clay	firm
Clay	firm
Sandy Silt to Clayey Silt	medium dense
Clayey Silt to Silty Clay	stiff
Silty Clay to Clay	firm
Silty Clay to Clay	firm
Clay	stiff
Clay	very stiff
Clayey Silt to Silty Clay	dense
Silty Clay to Clay	hard
Clay	very stiff
Sandy Silt to Clayey Silt	dense
Sand	medium dense
Silty Sand to Sandy Silt	dense
Sand	dense
Sand	dense
Sand	dense
Silty Sand to Sandy Silt	dense
Sand to Silty Sand	medium dense
Clayey Silt to Silty Clay	very stiff
Clayey Silt to Silty Clay	stiff
Clayey Silt to Silty Clay	stiff
Sandy Silt to Clayey Silt	medium dense
Sand to Silty Sand	medium dense
Sand	medium dense
Sand	dense
Sand	dense
Silty Sand to Sandy Silt	dense
Clayey Silt to Silty Clay	very stiff
Clayey Silt to Silty Clay	stiff
Clayey Silt to Silty Clay	stiff
Clayey Silt to Silty Clay	stiff
Clayey Silt to Silty Clay	very stiff
Clayey Silt to Silty Clay	very stiff
Clayey Silt to Silty Clay	stiff
Silty Clay to Clay	stiff
Silty Clay to Clay	stiff



End of Sounding @ 50.4 feet

CONE PENETROMETER INTERPRETATION

(based on Robertson & Campanella, 1989)

Project: Oxnard College Fire Academy Project No: 302245-001 Date: 03/28/19

CPT SOUNDING: CPT-1 Plot: 1 Density: 1 SPT N Program developed 2003 by Shelton L. Stringer, GE, Earth Systems Southwest

Est. GWT (feet): **8.0** Dr correlation: 0 Baldi Qc/N: 1 Robertson Phi Correlation: 4 SPT N

Base Depth meters	Base Depth feet	Avg Tip Qc, tsf	Avg Friction Ratio, %	Soil Classification	USCS	Density or Consistency	Est. Density (pcf)	Qc to N	SPT N(60)	Total po tsf	p'o tsf	F	n	Cq	Norm. Qc1n	2.6 Ic	Clean Sand Qc1n	$N_{1(60)}$	Clean Sand $N_{1(60)}$	Rel. Dens. Dr (%)	Phi (deg.)	Nk: 17 Su (tsf)	OCR
0.15	0.5	9.77	8.81	Clay	CL/CH	stiff	110	1.0	10	0.014	0.014	8.82	0.95	1.70	15.7	3.14		10				0.57	####
0.30	1.0	16.47	4.86	Clay	CL/CH	stiff	110	1.0	16	0.041	0.041	4.87	0.85	1.70	26.5	2.80		16				0.97	####
0.46	1.5	19.37	4.40	Clay	CL/CH	very stiff	110	1.0	19	0.069	0.069	4.42	0.82	1.70	31.1	2.72		19				1.14	84.2
0.61	2.0	16.30	4.06	Silty Clay to Clay	CL	stiff	110	1.5	11	0.096	0.096	4.08	0.83	1.70	26.2	2.75		11				0.95	50.5
0.76	2.5	12.37	7.00	Clay	CL/CH	stiff	110	1.0	12	0.124	0.124	7.08	0.91	1.70	19.9	3.00		12				0.72	29.7
0.91	3.0	13.00	6.92	Clay	CL/CH	stiff	110	1.0	13	0.151	0.151	7.01	0.90	1.70	20.9	2.98		13				0.76	25.5
1.07	3.5	11.43	6.10	Clay	CL/CH	stiff	110	1.0	11	0.179	0.179	6.20	0.91	1.70	18.4	2.99		11				0.66	18.9
1.22	4.0	8.97	5.96	Clay	CL/CH	stiff	110	1.0	9	0.206	0.206	6.10	0.93	1.70	14.4	3.06		9				0.52	12.7
1.37	4.5	9.00	4.92	Clay	CL/CH	stiff	110	1.0	9	0.234	0.234	5.06	0.91	1.70	14.5	3.01		9				0.52	11.3
1.52	5.0	10.83	1.51	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	5	0.263	0.263	1.55	0.80	1.70	17.4	2.64		5				0.62	12.1
1.68	5.5	5.15	2.30	Clay	CL/CH	firm	120	1.0	5	0.293	0.293	2.44	0.92	1.70	8.3	3.02		5				0.29	5.0
1.83	6.0	2.57	3.90	Clay	CL/CH	soft	120	1.0	3	0.323	0.323	4.46	1.00	1.70	4.1	3.41		3				0.13	2.1
1.98	6.5	3.40	2.95	Clay	CL/CH	soft	120	1.0	3	0.353	0.353	3.29	0.99	1.70	5.5	3.24		3				0.18	2.6
2.13	7.0	3.57	2.80	Clay	CL/CH	soft	120	1.0	4	0.383	0.383	3.14	0.98	1.70	5.7	3.21		4				0.19	2.5
2.29	7.5	3.60	2.78	Clay	CL/CH	soft	120	1.0	4	0.413	0.413	3.14	0.98	1.70	5.8	3.21		4				0.19	2.3
2.44	8.0	3.30	3.03	Clay	CL/CH	soft	120	1.0	3	0.443	0.443	3.50	1.00	1.70	5.3	3.26		3				0.17	1.9
2.59	8.5	2.97	3.37	Clay	CL/CH	soft	120	1.0	3	0.473	0.457	4.01	1.00	1.70	4.8	3.33		3				0.15	1.6
2.74	9.0	3.57	2.83	Clay	CL/CH	soft	120	1.0	4	0.503	0.471	3.30	0.99	1.70	5.7	3.22		4				0.18	2.0
2.90	9.5	3.90	4.26	Clay	CL/CH	soft	120	1.0	4	0.533	0.486	4.93	1.00	1.70	6.3	3.29		4				0.20	2.1
3.05	10.0	6.60	3.32	Clay	CL/CH	firm	120	1.0	7	0.563	0.500	3.63	0.92	1.70	10.6	3.02		7				0.36	3.6
3.20	10.5	6.77	3.14	Clay	CL/CH	firm	120	1.0	7	0.593	0.515	3.44	0.92	1.70	10.9	3.00		7				0.37	3.6
3.35	11.0	5.30	3.77	Clay	CL/CH	firm	120	1.0	5	0.623	0.529	4.28	0.96	1.70	8.5	3.14		5				0.28	2.7
3.51	11.5	4.43	4.55	Clay	CL/CH	soft	120	1.0	4	0.653	0.543	5.34	1.00	1.70	7.1	3.26		4				0.23	2.1
3.66	12.0	4.23	4.72	Clay	CL/CH	soft	120	1.0	4	0.683	0.558	5.63	1.00	1.70	6.8	3.29		4				0.22	1.9
3.81	12.5	4.70	4.26	Clay	CL/CH	soft	120	1.0	5	0.713	0.572	5.02	0.99	1.70	7.6	3.23		5				0.24	2.1
3.96	13.0	4.43	4.51	Clay	CL/CH	soft	120	1.0	4	0.743	0.587	5.42	1.00	1.70	7.1	3.27		4				0.23	1.9
4.11	13.5	5.40	3.72	Clay	CL/CH	firm	120	1.0	5	0.773	0.601	4.34	0.96	1.70	8.7	3.14		5				0.28	2.3
4.27	14.0	5.83	3.43	Clay	CL/CH	firm	120	1.0	6	0.803	0.615	3.98	0.95	1.67	9.2	3.10		6				0.31	2.5
4.42	14.5	6.20	3.23	Clay	CL/CH	firm	120	1.0	6	0.833	0.630	3.73	0.94	1.63	9.5	3.07		6				0.33	2.6
4.57	15.0	9.27	3.97	Clay	CL/CH	stiff	120	1.0	9	0.863	0.644	4.38	0.91	1.57	13.8	2.98		9				0.51	3.9
4.72	15.5	48.03	1.55	Silty Sand to Sandy Silt	SM/ML	medium dense	120	3.0	16	0.893	0.659	1.58	0.67	1.37	62.3	2.20	103.2	20	21	57	33		
4.88	16.0	11.28	2.62	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	6	0.923	0.673	2.86	0.86	1.48	15.7	2.82		6				0.62	4.6
5.03	16.5	7.17	1.85	Clayey Silt to Silty Clay	ML/CL	firm	120	2.0	4	0.953	0.687	2.13	0.89	1.47	10.0	2.92		4				0.38	2.7
5.18	17.0	12.57	1.94	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	6	0.983	0.702	2.11	0.83	1.41	16.7	2.73		6				0.70	5.0
5.33	17.5	9.73	1.99	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	5	1.013	0.716	2.22	0.87	1.40	12.9	2.83		5				0.53	3.7
5.49	18.0	7.47	2.68	Silty Clay to Clay	CL	firm	120	1.5	5	1.043	0.731	3.11	0.92	1.41	9.9	3.01		5				0.40	2.6
5.64	18.5	7.27	2.75	Silty Clay to Clay	CL	firm	120	1.5	5	1.073	0.745	3.23	0.93	1.39	9.5	3.03		5				0.38	2.5
5.79	19.0	7.30	1.82	Clayey Silt to Silty Clay	ML/CL	firm	120	2.0	4	1.103	0.759	2.14	0.90	1.35	9.3	2.94		4				0.38	2.4
5.94	19.5	8.73	2.29	Clayey Silt to Silty Clay	ML/CL	firm	120	2.0	4	1.133	0.774	2.64	0.90	1.32	10.9	2.93		4				0.47	2.9
6.10	20.0	10.43	4.94	Clay	CL/CH	stiff	120	1.0	10	1.163	0.788	5.56	0.94	1.32	13.0	3.07		10				0.57	3.5
6.25	20.5	27.93	5.52	Clay	CL/CH	very stiff	120	1.0	28	1.193	0.803	5.76	0.84	1.26	33.3	2.78		28				1.60	10.0
6.40	21.0	38.53	4.46	Silty Clay to Clay	CL	hard	120	1.5	26	1.223	0.817	4.60	0.80	1.23	44.7	2.62		26				2.22	13.7
6.55	21.5	48.67	4.39	Clayey Silt to Silty Clay	ML/CL	medium dense	120	2.0	24	1.253	0.831	4.50	0.77	1.21	55.4	2.55	167.4	27	33	52	35		
6.71	22.0	78.03	3.17	Sandy Silt to Clayey Silt	ML	dense	120	2.5	31	1.283	0.846	3.23	0.70	1.17	86.3	2.31	171.3	34	34	71	37		
6.86	22.5	39.10	5.03	Clay	CL/CH	hard	120	1.0	39	1.313	0.860	5.20	0.81	1.18	43.7	2.66		39				2.25	13.2
7.01	23.0	33.23	4.28	Silty Clay to Clay	CL	very stiff	120	1.5	22	1.343	0.875	4.47	0.81	1.17	36.7	2.67		22				1.90	10.9
7.16	23.5	19.07	6.92	Clay	CL/CH	very stiff	120	1.0	19	1.373	0.889	7.45	0.91	1.17	21.1	3.00		19				1.07	6.0
7.32	24.0	45.90	3.99	Clayey Silt to Silty Clay	ML/CL	medium dense	120	2.0	23	1.403	0.903	4.12	0.78	1.13	49.1	2.56	150.4	24	30	47	34		
7.47	24.5	55.13	3.68	Clayey Silt to Silty Clay	ML/CL	medium dense	120	2.0	28	1.433	0.918	3.78	0.75	1.11	58.0	2.48	154.4	29	31	54	36		
7.62	25.0	98.03	1.44	Sand to Silty Sand	SP/SM	medium dense	120	4.0	25	1.463	0.932	1.46	0.61	1.08	100.2	2.02	132.9	25	27	77	35		
7.77	25.5	135.90	0.79	Sand	SP	medium dense	120	5.0	27	1.493	0.947	0.80	0.53	1.06	136.3	1.74	145.5	28	29	90	35		
7.92	26.0	136.63	0.68	Sand	SP	medium dense	120	5.0	27	1.523	0.961	0.69	0.52	1.05	135.8	1.71	141.4	28	28	90	35		
8.08	26.5	84.13	1.90	Silty Sand to Sandy Silt	SM/ML	medium dense	120	3.0	28	1.553	0.975	1.94	0.66	1.05	83.9	2.16	131.9	28	26	70	36		
8.23	27.0	139.30	1.62	Sand to Silty Sand	SP/SM	dense	120	4.0	35	1.583	0.990	1.64	0.60	1.04	137.0	1.96	171.1	35	34	90	37		
8.38	27.5	204.73	1.19	Sand	SP	dense	120	5.0	41	1.613	1.004	1.20	0.53	1.03	199.0	1.75	213.0	41	43	100	39		
8.53	28.0	220.17	0.97	Sand	SP	dense	120	5.0	44	1.643	1.019	0.98	0.51	1.02	212.2	1.66	215.0	44	43	100	39		
8.69	28.5	221.27	0.93	Sand	SP	dense	120	5.0	44	1.673	1.033	0.94	0.50	1.01	211.7	1.65	212.9	44	43	100	39		
8.84	29.0	179.67	1.56	Sand to Silty Sand	SP/SM	dense	120	4.0	45	1.703	1.047	1.58	0.57	1.01	170.8	1.88	200.2	44	40	99	39		
8.99	29.5	234.37	1.07	Sand	SP	dense	120	5.0	47	1.733	1.062	1.08	0.51	1.00	221.1	1.68	226.9	45	45	100	40		
9.14	30.0	164.17	1.22	Sand to Silty Sand	SP/SM	dense	120	4.0	41	1.763	1.076	1.23	0.56	0.99	153.7	1.83	174.0	40	35	95	38		
9.30	30.5	60.73	3.33	Sandy Silt to Clayey Silt	ML	medium dense	120	2.5	24	1.793	1.091	3.43	0.75	0.98	56.1	2.46	144.0	23	29	53	34		
9.45	31.0	144.33	1.62	Sand to Silty Sand	SP/SM	dense	120	4.0	36	1.823	1.105	1.64	0.60	0.97	132.9	1.97	167.5	34	33	89	37		
9.60	31.5	117.90	1.56	Sand to Silty Sand	SP/SM	medium dense	120	4.0	29	1.853	1.119	1.58	0.61	0.97	107.6	2.02	142.9	28	29	80	35		
9.75	32.0	69.83	1.31	Silty Sand to Sandy Silt	SM/ML	medium dense	120	3.0	23	1.883	1.134	1.34	0.65	0.96	63.1	2.15	97.4	22	19	58	34		
9.91	32.5	21.63	2.73	Clayey Silt to Silty Clay	ML/CL	very stiff	120	2.0	11	1.913	1.148	2.99	0.84	0.93	19.1	2.77		11				1.21	5.2
10.06	33.0	16.67	3.65	Silty Clay to Clay	CL	stiff	120	1.5	11	1.943	1.163	4.13	0.90	0.92	14.5	2.95		11				0.91	3.8
10.21	33.5	21.33	3.14	Clayey Silt to Silty Clay	ML/CL	very stiff	120	2.0	11	1.973	1.177	3.46	0.86	0.91	18.4	2.82		11				1.19	4.9
10.36	34.0	12.70	3.15	Silty Clay to Clay	CL	stiff	120	1.5	8	2.003	1.191	3.74	0.93	0.90	10.8	3.03		8				0.68	2.7
10.52	34.5	13.10	3.05	Silty Clay to Clay	CL	stiff	120	1.5	9	2.033	1.206	3.61	0.92	0.89	11.0	3.01		9				0.70	2.8
10.67	35.0	14.40	2.78	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	7	2.063	1.220	3.25	0.90	0.88	12.0	2.95		7				0.78	3.0
10.82	35.5	12.90	3.34	Silty Clay to Clay	CL	stiff	120	1.5	9	2.093	1.235	3.99	0.93	0.87	10.6	3.05		9				0.69	2.6
10.97	36.0	62.70	1.72	Silty Sand to Sandy Silt	SM/ML	medium dense	120	3.0	21	2.123	1.249	1.78	0.70	0.89	52.8	2.28	100.3	19	20	50	33		
11.13	36.5	106.03	0.64	Sand	SP	medium dense	120	5.0	21	2.153	1.263	0.66	0.56	0.91	90.7	1.83	102.7	19	21	73	33		
11.28	37.0	64.60	1.47	Silty Sand to Sandy Silt	SM/ML	medium dense	120	3.0	22	2.183	1.278	1.52	0.68	0.88	53.7	2.23	94.2	19	19	51	33		
11.43	37.5	93.83	0.81	Sand to Silty Sand	SP/SM	medium dense	120	4.0	23	2.213	1.292	0.83	0.59	0.89	78.8	1.94	97.1	21	19	67	33		
11.58	38.0	160.87	0.46	Sand	SP	medium dense	120	5.0	32	2.243	1.307	0.46	0.50	0.90	136.8	1.60	136.8	28	27	90	36		
11.73	38.5	168.27	0.63	Sand	SP	medium dense	120	5.0	34	2.273	1.321	0.64	0.51	0.89	142.0	1.67	144.6	29	29	91	36		
11.89	39.0	192.23	1.14	Sand	SP	dense	120	5.0	38	2.303	1.335	1.15	0.55	0.88	159.9	1.80	177.1	33	35	96	37		
12.04	39.5	238.43	0.98	Sand	SP	dense	120	5.0	48	2.333	1.350	0.99	0.52	0.88	198.8	1.69	204.7	41	41	100	39		
12.19	40.0	247.43	1.08	Sand	SP	dense	120	5.0	49	2.363	1.364	1.09	0.52	0.88	204.9	1.71	213.8	42	43	100	39		

Earth Systems

CONE PENETROMETER INTERPRETATION

(based on Robertson & Campanella, 1989)

Project: Oxnard College Fire Academy | Project No: 302245-001 | Date: 03/28/19

CPT SOUNDING: CPT-1	Plot: 1	Density:	1	SPT N			Program developed 2003 by Shelton L. Stringer, GE, Earth Systems Southwest
Est. GWT (feet): 8.0		Dr correlation:	0	Baldi	Qc/N: 1	Robertson	Phi Correlation: 4 SPT N

Base Depth meters	Base Depth feet	Avg Tip Qc, tsf	Avg Friction Ratio, %	Soil Classification	USCS	Density or Consistency	Est. Density (pcf)	Qc to N	SPT N(60)	Total po tsf	p'o tsf	F	n	Cq	Norm. Qc1n	2.6 Ic	Clean Sand Qc1n	$N_{1(60)}$	Clean Sand $N_{1(60)}$	Rel. Dens. Dr (%)	Phi (deg.)	Nk: 17 Su (tsf)	OCR
12.34	40.5	227.03	1.56	Sand to Silty Sand	SP/SM	dense	120	4.0	57	2.393	1.379	1.58	0.57	0.86	184.7	1.86	213.0	48	43	100	40		
12.50	41.0	102.37	2.56	Silty Sand to Sandy Silt	SM/ML	medium dense	120	3.0	34	2.423	1.393	2.62	0.69	0.83	80.0	2.27	147.9	29	30	68	36		
12.65	41.5	30.43	2.92	Clayey Silt to Silty Clay	ML/CL	very stiff	120	2.0	15	2.453	1.407	3.17	0.83	0.79	22.7	2.73		15				1.71	6.0
12.80	42.0	16.63	3.61	Silty Clay to Clay	CL	stiff	120	1.5	11	2.483	1.422	4.24	0.92	0.76	12.0	3.02		11				0.89	3.0
12.95	42.5	16.80	3.60	Silty Clay to Clay	CL	stiff	120	1.5	11	2.513	1.436	4.23	0.92	0.75	12.0	3.02		11				0.90	3.0
13.11	43.0	17.67	3.02	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	9	2.543	1.451	3.53	0.90	0.75	12.6	2.96		9				0.95	3.1
13.26	43.5	14.77	3.16	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	7	2.573	1.465	3.82	0.93	0.74	10.3	3.05		7				0.78	2.5
13.41	44.0	14.50	3.24	Silty Clay to Clay	CL	stiff	120	1.5	10	2.603	1.479	3.94	0.94	0.73	10.0	3.07		10				0.77	2.4
13.56	44.5	13.97	2.87	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	7	2.633	1.494	3.53	0.94	0.72	9.6	3.05		7				0.73	2.3
13.72	45.0	14.93	3.12	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	7	2.663	1.508	3.80	0.93	0.72	10.1	3.05		7				0.79	2.4
13.87	45.5	17.97	4.25	Silty Clay to Clay	CL	stiff	120	1.5	12	2.693	1.523	5.00	0.94	0.71	12.1	3.06		12				0.97	3.0
14.02	46.0	23.40	2.90	Clayey Silt to Silty Clay	ML/CL	very stiff	120	2.0	12	2.723	1.537	3.28	0.87	0.72	16.0	2.86		12				1.29	4.0
14.17	46.5	15.73	3.18	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	8	2.753	1.551	3.85	0.93	0.70	10.4	3.05		8				0.83	2.5
14.33	47.0	32.23	3.74	Clayey Silt to Silty Clay	ML/CL	very stiff	120	2.0	16	2.783	1.566	4.09	0.86	0.71	21.8	2.81		16				1.80	5.6
14.48	47.5	21.23	2.50	Clayey Silt to Silty Clay	ML/CL	very stiff	120	2.0	11	2.813	1.580	2.88	0.88	0.70	14.1	2.86		11				1.16	3.5
14.63	48.0	14.43	2.78	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.0	7	2.843	1.595	3.47	0.94	0.68	9.3	3.06		7				0.76	2.2
14.78	48.5	13.90	3.11	Silty Clay to Clay	CL	stiff	120	1.5	9	2.873	1.609	3.92	0.95	0.67	8.8	3.11		9				0.72	2.1
14.94	49.0	15.20	3.29	Silty Clay to Clay	CL	stiff	120	1.5	10	2.903	1.623	4.07	0.95	0.67	9.6	3.09		10				0.80	2.3
15.09	49.5	16.33	4.05	Silty Clay to Clay	CL	stiff	120	1.5	11	2.933	1.638	4.94	0.95	0.66	10.2	3.12		11				0.86	2.5
15.24	50.0	18.23	4.40	Clay	CL/CH	stiff	120	1.0	18	2.963	1.652	5.25	0.95	0.66	11.3	3.10		18				0.98	2.8

APPENDIX B

Laboratory Testing

Tabulated Laboratory Test Results

Individual Laboratory Test Results

Table 1809.7

LABORATORY TESTING

A. Samples were reviewed along with field logs to determine which would be analyzed further. Those chosen for laboratory analysis were considered representative of soils that would be exposed and/or used during grading, and those deemed to be within the influence of proposed structures. Test results are presented in graphic and tabular form in this Appendix.

B. In-situ Moisture Content and Unit Dry Weight for the ring samples were determined in general accordance with ASTM D 2937.

C. The relative strength characteristics were determined from the results of Direct Shear tests on relatively undisturbed samples of formational bedrock and on a remolded sample of the near-surface soils. The compacted sample was remolded to approximately 90% of the maximum dry density (ASTM D 1557). Specimens were placed in contact with water at least 24 hours before testing, and were then sheared under normal loads ranging from 1 to 3 ksf in general accordance with ASTM D 3080. The samples were sheared to sufficient strains so that both peak and ultimate values were evaluated. The relatively undisturbed samples of formational bedrock were sheared to sufficient strains so that peak, ultimate, and residual values were evaluated.

D. An expansion index test was performed on a bulk soil sample in accordance with ASTM D 4829. The sample was surcharged under 144 pounds per square foot at moisture content of near 50 percent saturation. The sample was then submerged in water for 24 hours, and the amount of expansion was recorded with a dial indicator.

E. A maximum density test was performed to estimate the moisture-density relationship of typical near-surface materials. The test was performed in accordance with ASTM D 1557.

F. The gradation characteristics of certain samples were evaluated by hydrometer (in accordance with ASTM D 422) and sieve analysis procedures. The samples were soaked in water until individual soil particles were separated, then washed on the No. 200 mesh sieve, oven dried, weighed to calculate the percent passing the No. 200 sieve, and mechanically sieved. Additionally, hydrometer analyses were performed to assess the grain size distribution of the particles that passed the No. 200 screen. The hydrometer portions of the tests were run using sodium hexametaphosphate as a dispersing agent.

G. The Plasticity Indices of selected samples were evaluated in accordance with ASTM D 4318.

H. One resistance value (R-value) test was conducted on a bulk sample secured during the field study from within the proposed paved parking lot. The test was performed in accordance with California Method 301. Three specimens at different moisture contents were tested for each sample and the R-Values at 300 psi exudation pressure were determined from the plotted results.

I. A portion of the bulk sample collected in boring B-1 was sent to another laboratory for analyses of soil pH, resistivity, chloride contents, and sulfate contents. Soluble chloride and sulfate contents were determined on a dry weight basis. Resistivity testing was performed in accordance with California Test Method 424, wherein the ratio of soil to water was 1:3.

TABULATED LABORATORY TEST RESULTS

	REMOLDED SAMPLE
TEST PIT/BORING AND DEPTH	B-1 @ 0'-5'
USCS	CL
MAXIMUM DRY DENSITY (pcf)	113.0
OPTIMUM MOISTURE (%)	11.5
COHESION (PSF)	250* 220**
ANGLE OF INTERNAL FRICTION	28°* 28°**
EXPANSION INDEX	97
pH	8.1
SOLUBLE CHLORIDES (mg/kg)	110
RESISTIVITY (ohms-cm)	628
SOLUBLE SULFATES (mg/Kg)	1,955

* = Peak Strength Parameters; ** = Ultimate Strength Parameters

	RELATIVELY UNDISTURBED SAMPLES	
BORING AND DEPTH	B-1 @ 5'	B-1 @ 15'
USCS	CL	CH
IN-PLACE DRY DENSITY (PCF)	81.2	--
IN-PLACE MOISTURE (%)	36.6	43.1
LIQUID LIMIT	44	62
PLASTIC LIMIT	23	23
PLASTICITY INDEX	21	39
GRAIN SIZE DISTRIBUTION (%)		
GRAVEL	0.0	0.0
SAND	11.7	6.2
SILT	55.5	36.5
CLAY (2ɥm to 5ɥm)	8.2	14.3
CLAY (≤2ɥm)	24.6	43.0

UNIT DENSITIES AND MOISTURE CONTENT

ASTM D2937 & D2216

Job Name: Oxnard College Fire Academy

Sample Location	Depth (feet)	Unit Dry Density (pcf)	Moisture Content (%)	USCS Group Symbol
B1	2.5	91.8	23.9	CL/ML
B1	5	81.2	36.6	CL
B1	7.5	72.7	45.3	CL
B1	10	78.4	43.1	CL
B1	15	---	43.1	CH
B2	2.5	80.7	39.6	CL/ML
B2	10	69.8	50.2	CL

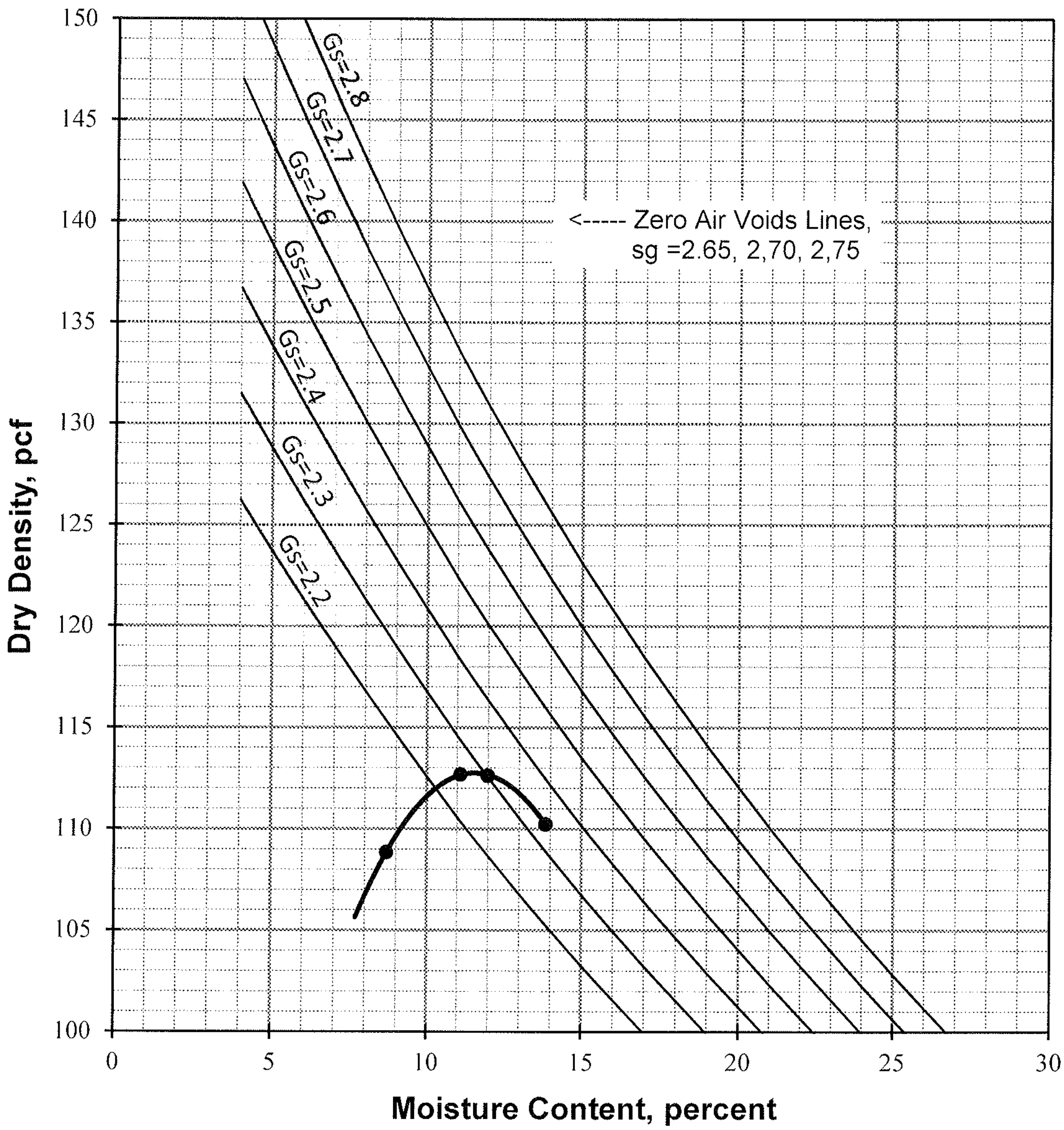
File Number: 302245-001 Lab Number: 098127

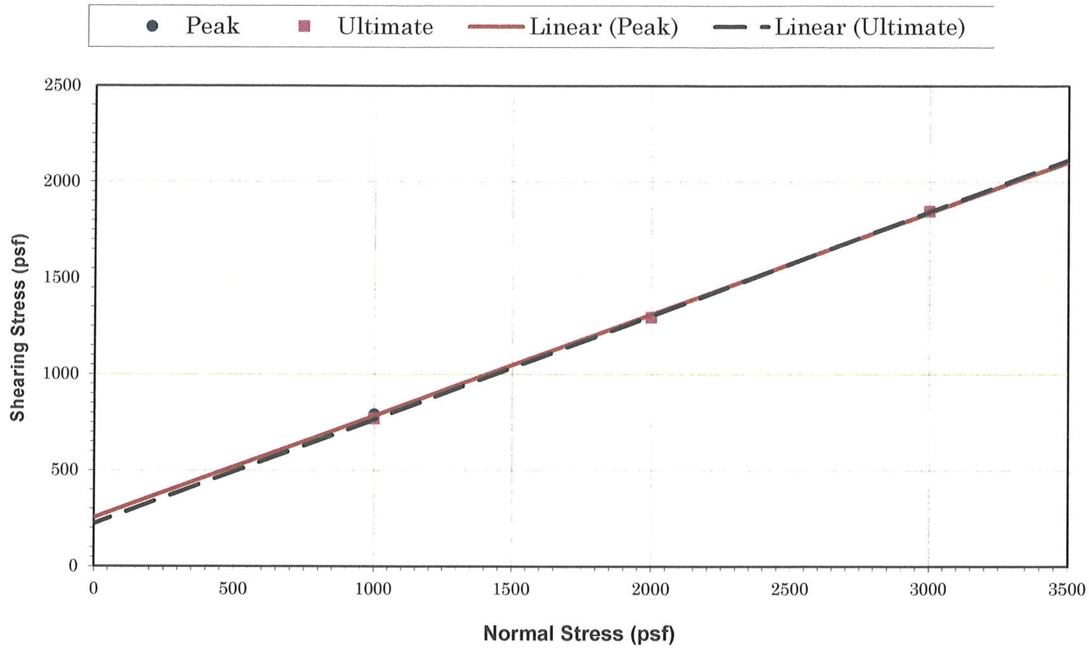
MAXIMUM DENSITY / OPTIMUM MOISTURE ASTM D 1557-12 (Modified)

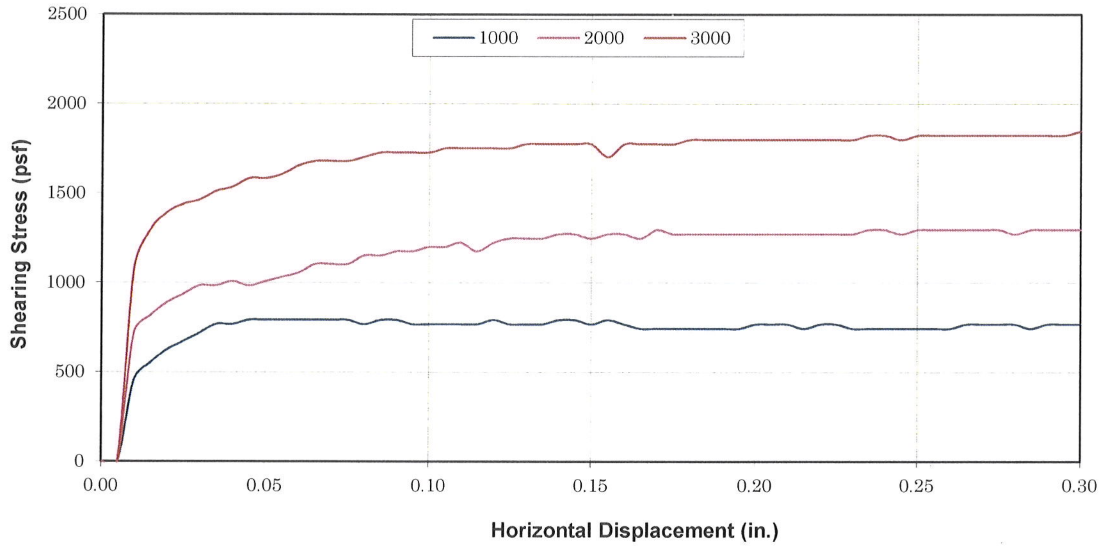
Job Name:	Oxnard College Fire Academy	Procedure Used: B
Sample ID:	B-1 @ 0-5'	Prep. Method: Moist
Date:	4/29/2019	Rammer Type: Automatic
Description:	Greyish Brown Clayey Silt/Silty Clay	
SG:	2.28	

Maximum Density: **113 pcf**
Optimum Moisture: **11.5%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	0.0







DIRECT SHEAR DATA*

Sample Location: B-1 @ 0-5'
Sample Description: Clayey Silt/Silty Clay
Dry Density (pcf): 100.4
Intial % Moisture: 11.6
Average Degree of Saturation: 100.0
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	792	1296	1848
Ultimate stress (psf)	768	1296	1848

	Peak	Ultimate
ϕ Angle of Friction (degrees):	28	28
c Cohesive Strength (psf):	250	220

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Oxnard College Fire Academy

Earth

5/22/2019 | 302245-001

File No.: 302245-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

Job Name: Oxnard College Fire Academy
Sample ID: B-1 @ 0-5'
Soil Description: CL/ML

Initial Moisture, %:	10.3
Initial Compacted Dry Density, pcf:	107.4
Initial Saturation, %:	49
Final Moisture, %:	26.6
Volumetric Swell, %:	9.7

Expansion Index: **97** **High**

EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
130+	Very High

MECHANICAL ANALYSIS

CTM 203-08

Job Name:	Oxnard College Fire Academy
Job No.:	302245-001
Sample ID:	**B-1 @ 15'**
Soil Description:	**CH**
Hydrometer ID:	504229

Hydroscopic Moisture

Air Dry Wt, g:	100.0
Oven Dry Wt, g	100.0
% Moisture:	0.0

Air Dry Sample Wt., g:	346.3
Corrected Wt., g:	346.3

Sieve Analysis for +#10 Material

Sieve Size	Wt Ret	% Ret	% Passing
1/2 inch	0.0	0.00	100.00
3/8 inch	0.0	0.00	100.00
#4	0.0	0.00	100.00
#8	0.0	0.00	100.00
#10	0.0	0.00	100.00

Air Dry Hydro Sample Wt., g:	63
Corrected Wt., g:	63.0
Calculation Factor	0.6300

Hydrometer Analysis for <#10 Material

Start time: 2:04:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	2:04:20 AM	64	21	4.9	59.1
1 hour	3:04:00 AM	41	21	4.9	36.1
6 hour	8:04:00 AM	32	21	4.9	27.1

% Gravel:	0.0
% Sand(2mm - 74µm):	6.2
% Silt(74µm- 5µm):	36.5
% Clay(5µm - 2µm):	14.3
% Clay(≤2µm):	43.0

MECHANICAL ANALYSIS

CTM 203-08

Job Name:	Oxnard College Fire Academy
Job No.:	302245-001
Sample ID:	**B-1 @ 5'**
Soil Description:	**CL**
Hydrometer ID:	504229

Hydroscopic Moisture

Air Dry Wt, g:	100.0
Oven Dry Wt, g	100.0
% Moisture:	0.0
Air Dry Sample Wt., g:	467.8
Corrected Wt., g:	467.8

Sieve Analysis for +#10 Material

Sieve Size	Wt Ret	% Ret	% Passing
1/2 inch	0.0	0.00	100.00
3/8 inch	0.0	0.00	100.00
#4	0.0	0.00	100.00
#8	0.7	0.15	99.85
#10	1.5	0.32	99.68

Air Dry Hydro Sample Wt., g:	61.1
Corrected Wt., g:	61.1
Calculation Factor	0.6130

Hydrometer Analysis for <#10 Material

Start time: 2:11:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	2:11:20 AM	59	21	4.9	54.1
1 hour	3:11:00 AM	25	21	4.9	20.1
6 hour	8:11:00 AM	20	21	4.9	15.1

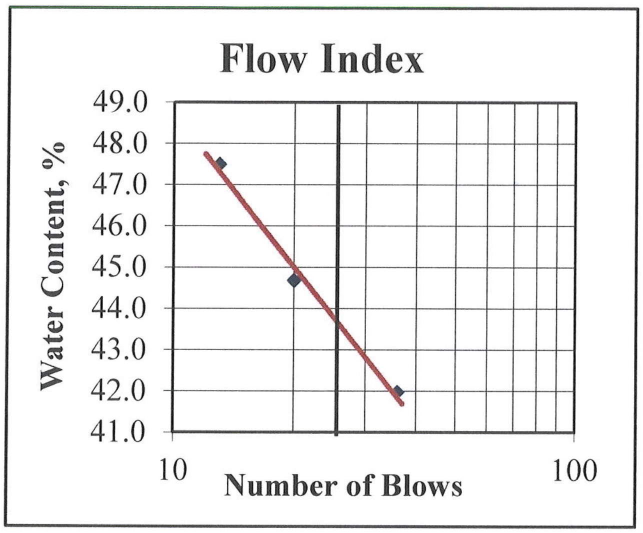
% Gravel:	0.0
% Sand(2mm - 74µm):	11.7
% Silt(74µm- 5µm):	55.5
% Clay(5µm - 2µm):	8.2
% Clay(≤2µm):	24.6

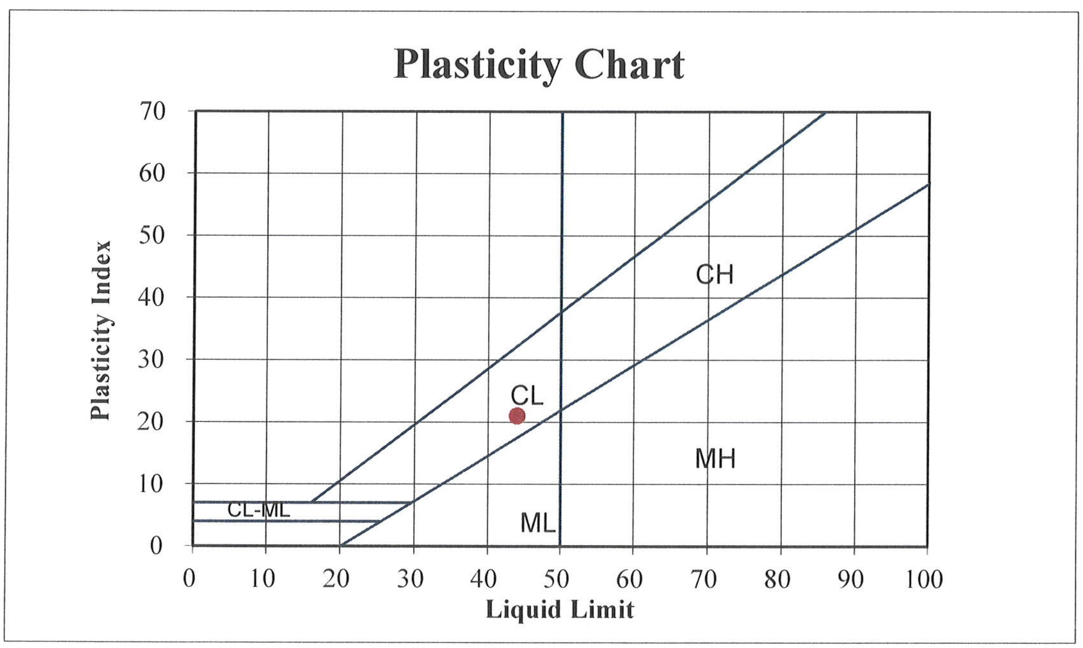
PLASTICITY INDEX

ASTM D-4318

Job Name: Oxnard College Fire Academy
Sample ID: B-1 @ 5'
Soil Description: CL

DATA SUMMARY				TEST RESULTS	
Number of Blows:	13	20	36	**LIQUID LIMIT**	**44**
Water Content, %	47.5	44.7	42.0	**PLASTIC LIMIT**	**23**
Plastic Limit:	23.7	23.2		**PLASTICITY INDEX**	**21**





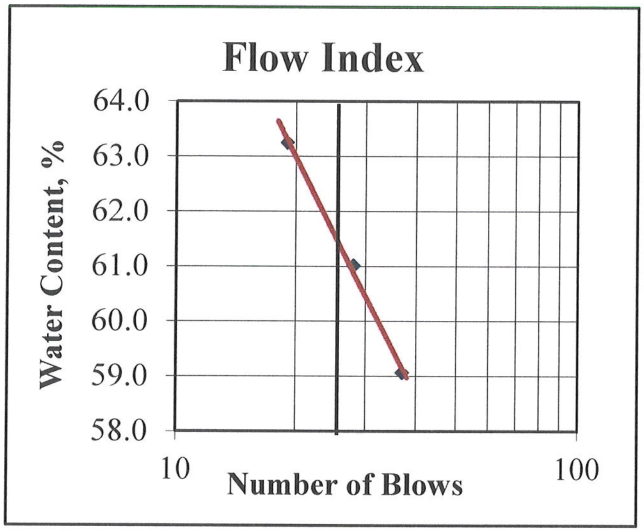
File No.: 302245-001 May 7, 2019

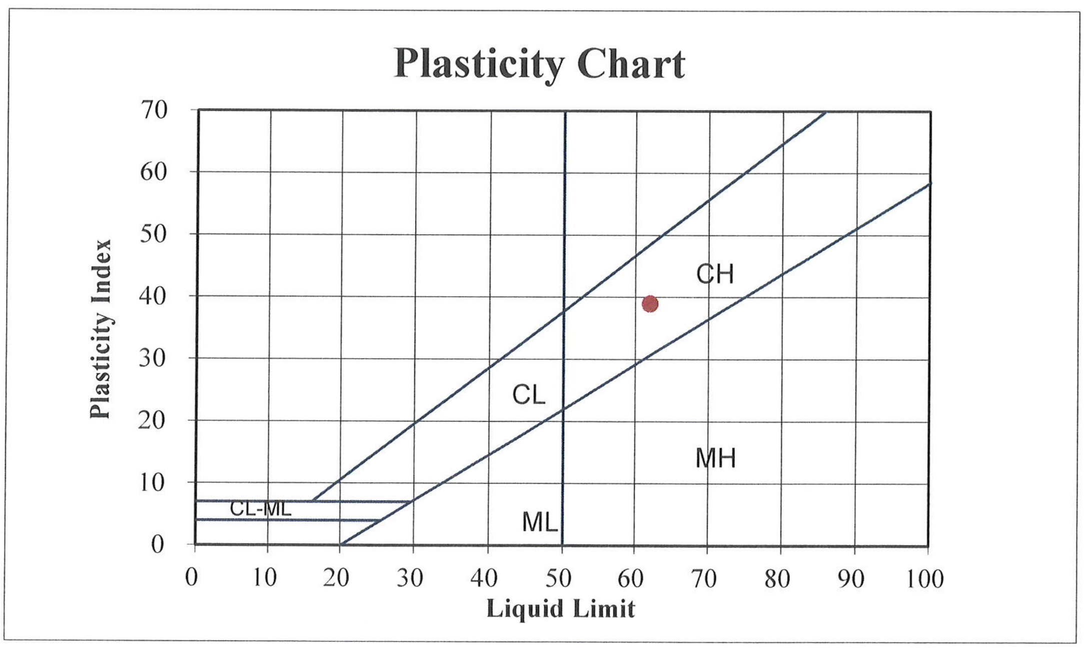
PLASTICITY INDEX

ASTM D-4318

Job Name: Oxnard College Fire Academy
Sample ID: B-1 @ 15'
Soil Description: CH

DATA SUMMARY				TEST RESULTS	
Number of Blows:	19	28	37	**LIQUID LIMIT**	**62**
Water Content, %	63.3	61.0	59.1	**PLASTIC LIMIT**	**23**
Plastic Limit:	23.3	23.5		**PLASTICITY INDEX**	**39**



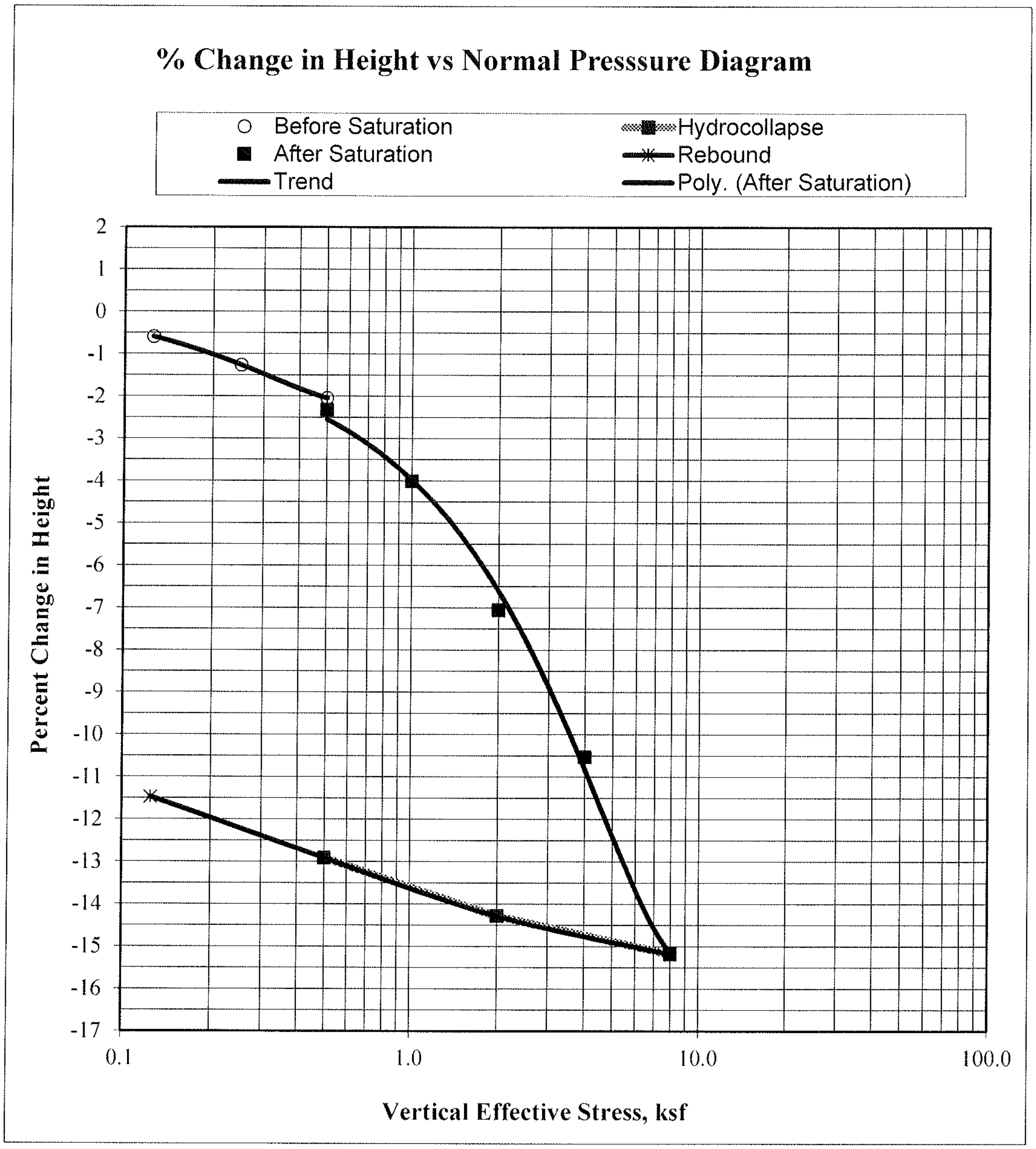


CONSOLIDATION TEST

ASTM D 2435-90 & D5333

Oxnard College Fire Academy
B-1 @ 5'
CL
Ring Sample

Initial Dry Density: 78.9 pcf
Initial Moisture, %: 36.6%
Specific Gravity: 2.67 (assumed
Initial Void Ratio: 1.113

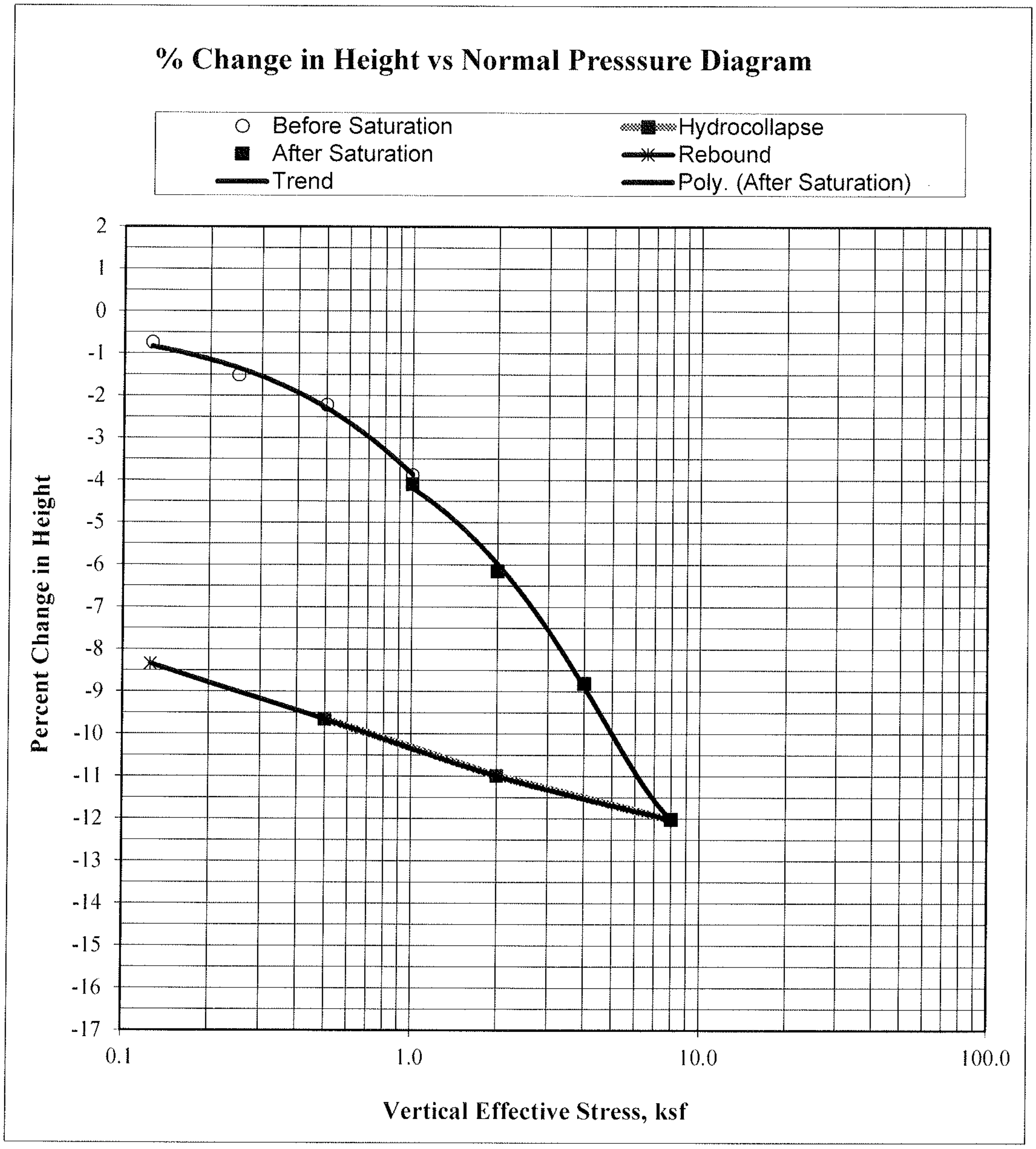


CONSOLIDATION TEST

ASTM D 2435-90 & D5333

Oxnard College Fire Academy
B-1 @ 10'
CL
Ring Sample

Initial Dry Density: 76.2 pcf
Initial Moisture, %: 43.1%
Specific Gravity: 2.67 (assumed
Initial Void Ratio: 1.189



CERTIFICATE OF ANALYSIS

Client: Earth Systems Pacific
CAS LAB NO: 190628-01
Sample ID: B-1@0-5
Analyst: GP

Date Sampled: 04/02/19
Date Received: 04/02/19
Sample Matrix: Soil

WET CHEMISTRY ANALYSIS SUMMARY

COMPOUND	RESULTS	UNITS	DF	PQL	METHOD	ANALYZED
pH (Corrosivity)	8.1	S.U.	1	---	9045	04/03/19
Resistivity*	628	Ohms-cm	1	---	SM 120.1M	04/03/19
Chloride	110	mg/Kg	2	1.2	300.0M	04/03/19
Sulfate	1955	mg/Kg	4	2.4	300.0M	04/03/19

*Sample was extracted using a 1:3 ratio of soil and DI water.

DF: Dilution Factor
PQL: Practical Quantitation Limit
BQL: Below Quantitation Limit
mg/Kg: Milligrams/Kilograms(ppm)

APPENDIX C

Site Class Determination Calculation
2016 CBC & ASCE 7-10 Seismic Parameters
OSHPD Design Maps Report
Spectral Response Values Table
Response Spectra Curves
Fault Parameters

	EARTH SYSTEMS	Job Number:	302245-001
		Job Name:	Oxnard College Fire Academy
		Calc Date:	6/17/2019
		CPT/Boring ID:	

Use "SPT N_{60}" if correlated from CPT.
Use "Raw SPT blow/ft" if from SPT/ModCal.
Input Number Max Limit = 100.

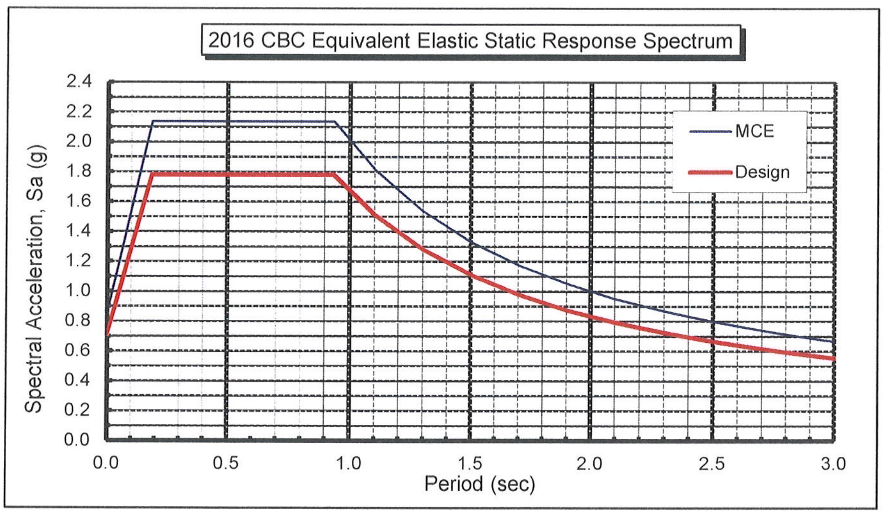
Depth (ft)	SPT N	Sublayer Thick (ft)	Sublayer Thick/N
5.0	3.0	5.0	1.667
10.0	4.0	5.0	1.250
15.0	3.0	5.0	1.667
20.0	4.0	5.0	1.250
25.0	4.0	5.0	1.250
30.0	8.0	5.0	0.625
35.0	8.0	5.0	0.625
40.0	11.0	5.0	0.455
45.0	22.0	5.0	0.227
50.0	30.0	5.0	0.167
52.5	23.0	2.5	0.109
55.0	33.0	2.5	0.076
100.0	30.0	45.0	1.500

Total Thickness of Soil =	100.00	ft
N-bar Value =	9.2	*
Site Classification =	Class E	

*Equation 20.4-2 of ASCE 7-10

2016 California Building Code (CBC) (ASCE 7-10) Seismic Design Parameters

			CBC Reference	ASCE 7-10 Reference
Seismic Design Category		**E**	Table 1613.5.6	Table 11.6-2
Site Class		**E**	Table 1613.5.2	Table 20.3-1
Latitude:		34.208 N		
Longitude:		-119.073 W		
Maximum Considered Earthquake (MCE) Ground Motion				
Short Period Spectral Reponse	S_S	**2.374 g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	S_1	**0.833 g**	Figure 1613.5	Figure 22.4
Site Coefficient	F_a	0.90	Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F_v	2.40	Table 1613.5.3(2)	Table 11-4.2
	S_{MS}	2.137 g	$= F_a * S_S$	
	S_{M1}	1.999 g	$= F_v * S_1$	
Design Earthquake Ground Motion				
Short Period Spectral Reponse	S_{DS}	**1.424 g**	$= 2/3 * S_{MS}$	
1 second Spectral Response	S_{D1}	**1.333 g**	$= 2/3 * S_{M1}$	
	To	0.19 sec	$= 0.2 * S_{D1}/S_{DS}$	
	Ts	0.94 sec	$= S_{D1}/S_{DS}$	
Seismic Importance Factor	I	1.25	Table 1604.5	Table 11.5-1
	F_{PGA}	0.90		



Period T (sec)	Design Sa (g)
0.00	0.712
0.05	0.998
0.19	1.781
0.94	1.781
1.10	1.515
1.30	1.282
1.50	1.111
1.70	0.980
1.90	0.877
2.10	0.793
2.30	0.724
2.50	0.666
2.70	0.617
2.90	0.574
3.10	0.537
3.30	0.505

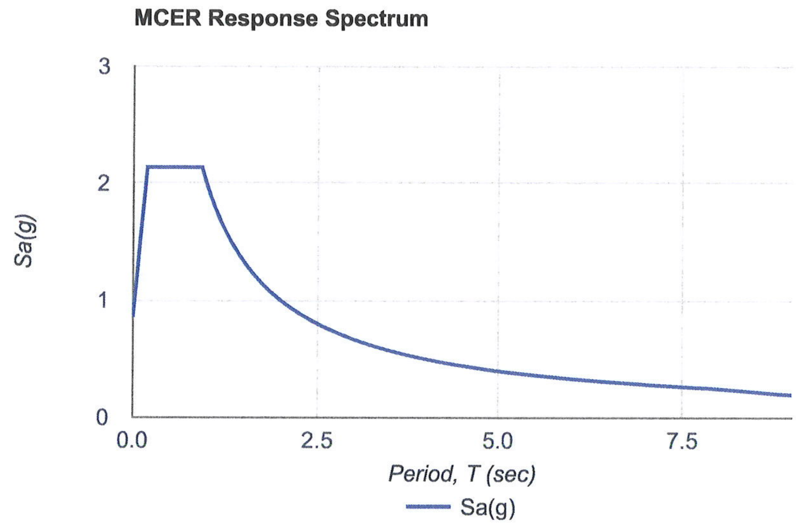
OSHPD

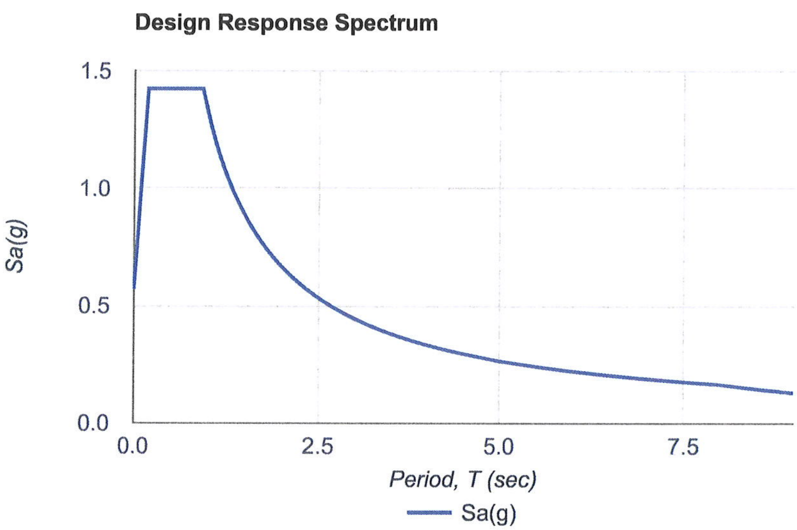
Latitude, Longitude: 34.2077, -119.0733

Date	6/15/2019, 6:52:35 AM
Design Code Reference Document	ASCE7-10
Risk Category	III
Site Class	E - Soft Clay Soil

Type	Value	Description
S_S	2.374	MCE_R ground motion. (for 0.2 second period)
S_1	0.833	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.137	Site-modified spectral acceleration value
S_{M1}	1.999	Site-modified spectral acceleration value
S_{DS}	1.425	Numeric seismic design value at 0.2 second SA
S_{D1}	1.332	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	E	Seismic design category
F_a	0.9	Site amplification factor at 0.2 second
F_v	2.4	Site amplification factor at 1.0 second
PGA	0.879	MCE_G peak ground acceleration
F_{PGA}	0.9	Site amplification factor at PGA
PGA_M	0.791	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.429	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.613	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.374	Factored deterministic acceleration value. (0.2 second)
S1RT	0.864	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.923	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.833	Factored deterministic acceleration value. (1.0 second)
PGAd	0.879	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.93	Mapped value of the risk coefficient at short periods
C_{R1}	0.937	Mapped value of the risk coefficient at a period of 1 s





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Spectral Response Values

Probabilistic and Deterministic Response Spectra for MCE compared to Code Spectra

for 5% Viscous Damping Ratio

Natural Period T (seconds)	GeoMean Probab. 2% in 50 yr MCE Spectrum (1) 2475-yr	Max Rotated Probab. 2% in 50 yr MCEr (2) 2475-yr	Max 84th Percentile Determ. MCE Spectrum (3)	Determ. Lower Limit MCE Spectrum (4)	Determ. MCE Spectrum (5) max(3,4)	**Site Specific MCE Spectrum** (6) min(2.5)	**2013 CBC MCE Spectrum** (7)	Site Specific Design Spectrum (8) 2/3*(6)*	2013 CBC Design Spectrum (9) 2/3*(7)
0.00	0.707	0.723	0.667	0.540	0.667	**0.667**	**0.855**	0.456	0.570
0.05	0.903	0.924	0.745	0.730	0.745	**0.745**	**1.197**	0.638	0.798
0.10	1.100	1.125	0.966	0.920	0.966	**0.966**	**1.540**	0.821	1.026
0.15	1.288	1.318	1.173	1.110	1.173	**1.173**	**1.882**	1.004	1.255
0.20	1.476	1.510	1.251	1.299	1.299	**1.299**	**2.137**	1.140	1.424
0.30	1.581	1.619	1.335	1.350	1.350	**1.350**	**2.137**	1.140	1.424
0.40	1.561	1.673	1.413	1.350	1.413	**1.413**	**2.137**	1.140	1.424
0.50	1.541	1.725	1.525	1.350	1.525	**1.525**	**2.137**	1.140	1.424
0.75	1.380	1.613	1.656	1.350	1.656	**1.613**	**2.137**	1.140	1.424
1.00	1.219	1.485	1.628	1.350	1.628	**1.485**	**1.999**	1.066	1.333
1.50	0.994	1.211	1.508	0.960	1.508	**1.211**	**1.333**	0.807	0.889
2.00	0.769	0.937	1.334	0.720	1.334	**0.937**	**1.000**	0.624	0.666

Crs: 0.930 * > 80% of (9)
Cr1: 0.937

Probabilistic Spectrum from 2008 USGS Ground Motion Mapping Program adjusted for site conditions and maximum rotated component of ground motion using NGA, Column 2 has risk coefficients Cr applied.

Reference: ASCE 7-10, Chapters 21.2, 21.3, 21.4 and 11.4

Mapped MCE Acceleration Values			Site Coefficients		**Site-Specific Design Acceleration Values**		
PGA	0.879	g	F_{PGA}	0.90	**PGA_M**	**0.791**	**g**
Ss	2.374	g	F_a	0.90	**S_{DS}**	**1.140**	**g**
S_1	0.833	g	F_v	2.40	**S_{D1}**	**1.249**	**g**

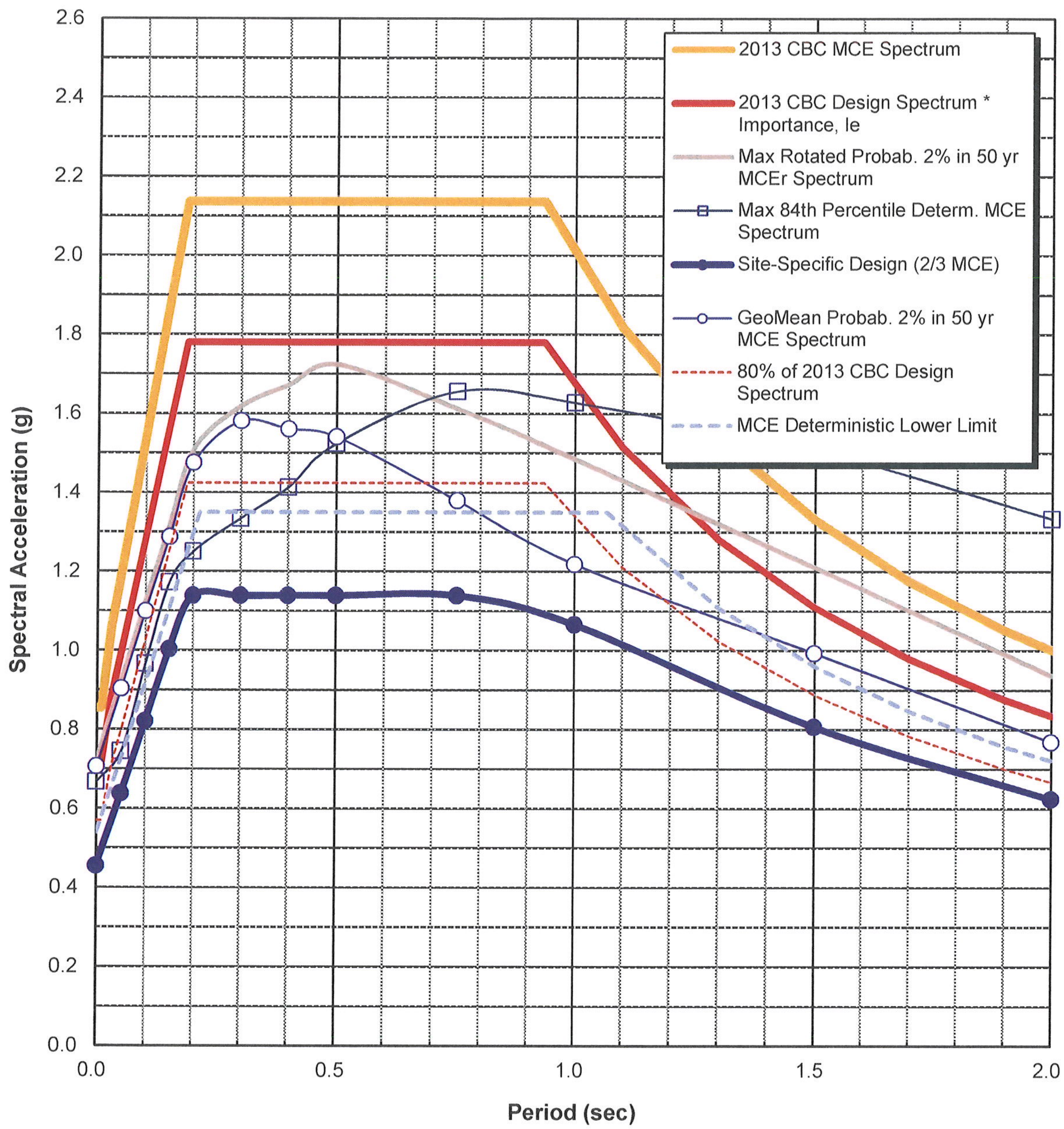
Spectral Amplification Factor for different viscous damping, D (%):

0.5%	2%	10%	20%
1.50	1.23	0.83	0.67

1 g = 980.6 cm/sec^2 =32.2 ft/sec^2
PSV (ft/sec) = 32.2(Sa)T/(2π)

Key: Probab. = Probabilistic, Determ. = Deterministic, MCE = Maximum Considered Earthquake

RESPONSE SPECTRA



Based on USGS National Strong Ground Motion Interactive Deaggregation Website using 2008 Parameters

Site Class: E
Latitude: 34.2077
Longitude: -119.0733

Response Spectra Curves

Oxnard College Fire Academy
File No.: 302245-001

Earth Systems

Table 1
Fault Parameters

Fault Section Name	Distance (miles)	Distance (km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
Simi-Santa Rosa	1.3	2.1	60	346	30	39	B	**7.2**		1
Oak Ridge (Onshore)	6.2	9.9	65	159	90	49	B	**7.4**		4
Ventura-Pitas Point	9.9	16.0	64	353	60	44	B	**6.9**		1
Malibu Coast (Extension), alt 1	10.3	16.5	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	10.3	16.5	74	4	30	35	B'	**6.9**		
Oak Ridge (Offshore)	11.8	19.0	32	180	90	38	B	**6.9**		3
Malibu Coast, alt 1	13.7	22.1	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	13.7	22.1	74	3	30	38	B	**6.9**		0.3
San Cayetano	15.0	24.2	42	3	90	42	B	**7.2**		6
Sisar	15.5	25.0	29	168	na	20	B'	**7.0**		
Red Mountain	16.0	25.7	56	2	90	101	B	**7.4**		2
Anacapa-Dume, alt 1	16.3	26.2	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	16.3	26.2	41	352	60	65	B	**7.2**		3
Channel Islands Thrust	16.6	26.7	20	354	90	59	B	**7.3**		1.5
Mission Ridge-Arroyo Parida-Santa Ana	18.3	29.4	70	176	90	69	B	**6.8**		0.4
Santa Cruz Island	18.8	30.3	90	188	30	69	B	**7.1**		1
Santa Susana, alt 1	20.4	32.8	55	9	90	27	B	**6.8**		5
Santa Susana, alt 2	20.6	33.2	53	10	90	43	B'	**6.8**		
Shelf (Projection)	20.7	33.3	17	21	na	70	B'	**7.8**		
Channel Islands Western Deep Ramp	21.3	34.3	21	204	90	62	B'	**7.3**		
North Channel	21.5	34.6	26	10	90	51	B	**6.7**		1
Northridge Hills	21.9	35.3	31	19	90	25	B'	**7.0**		
Santa Ynez (East)	22.9	36.9	70	172	0	68	B	**7.2**		2
Pitas Point (Lower)-Montalvo	23.2	37.4	16	359	90	30	B	**7.3**		2.5
Del Valle	23.8	38.3	73	195	90	9	B'	**6.3**		
Holser, alt 1	24.2	39.0	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	24.2	39.0	58	182	90	17	B'	**6.7**		
San Pedro Basin	24.3	39.0	88	51	na	69	B'	**7.0**		
Santa Monica Bay	24.8	39.8	20	44	na	17	B'	**7.0**		
Northridge	25.2	40.5	35	201	90	33	B	**6.8**		1.5
Pine Mtn	25.6	41.1	45	5	na	62	B'	**7.3**		
Santa Cruz Catalina Ridge	27.4	44.1	90	38	na	137	B'	**7.3**		
Compton	29.3	47.2	20	34	90	65	B'	**7.5**		
Pitas Point (Upper)	30.0	48.3	42	15	90	35	B	**6.8**		1
San Pedro Escarpment	31.6	50.8	17	38	na	27	B'	**7.3**		
Santa Monica, alt 1	32.0	51.6	75	343	30	14	B	**6.5**		1
San Gabriel	32.4	52.1	61	39	180	71	B	**7.3**		1
Santa Monica, alt 2	32.5	52.3	50	338	30	28	B	**6.7**		1
Palos Verdes	33.8	54.4	90	53	180	99	B	**7.3**		3
Oak Ridge (Offshore), west extension	34.6	55.7	67	195	na	28	B'	**6.1**		

Reference: USGS OFR 2007-1437 (CGS SP 203) Based on Site Coordinates of 34.2077 Latitude, -119.0733 Longitude

Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2007-1437). Mean magntude is average of Ellworths-B and Hanks & Bakun moment area relationship.

APPENDIX D

Total Seismically-Induced Settlement Calculations

Prediction of Liquefaction Induced Lateral Spreading

CPT-LIQUEFY.XLS - A SPREADSHEET FOR EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL USING CPT DATA
Developed 2003 by Shelton L. Stringer, GE, Earth Systems Southwest

Project: Oxnard College Fire Academy
Job No: 302245-001
Date: 5/22/2019
Sounding: CPT-1

Liquefaction Analysis using 1998 NCEER (Robertson & Wride) method
Settlement Analysis using Tokimatsu & Seed (1987), clean sand Qc1n/N1(60) ratio =5

EARTHQUAKE INFORMATION:
Magnitude: 7.2 7.5
PGA, g: 0.79 0.71
MSF: 1.11
GWT, feet: 8.0
Calc GWT, feet: 8.0

Plot: 1
Method Used: 1 1998 NCEER (Robertson & Wride)
Averaging Increment: 2 0.10 m
Induced CSR (M=7.5) = 0.65*PGA*(po/p'o)*rd/MSF
Clean Sand Qc1n = C_Q*K_C*K_H*Qc
SF = $CRR_{7.5}$*Kσ/CSR

Ignore 1st/last increment into sand/silt soils: 1 yes
Ignore/remediate upper: 1.0 m
Unit Weight of unsaturated soils: 115 pcf
Unit Weight of saturated soils: 130 pcf
Limiting Ic for liquefiable soils: 2.60 Limiting Ic for K_H: 2.6

Use Moss @ P_L: 15%
Use Tokimatsu & Seed (0) or Ishihara &Yoshmine (1): 0
Required SF: 1.30
Min SF of Liquefiable Layers: 0.18
Avg SF of Liquefiable Layers: 0.40
Max $\Delta N_{1(60)}$ - post liquefied: 5.5
Max $\Delta N_{1(60)}$ - non liquefied: 5.0

Total Liquefied Thickness (feet)	Total Induced Subsidence (inches)
6.2	1.0

Depth (feet)	(m)	Tip Qc (tsf)	Friction Fs (tsf)	Friction Ratio Rf %	qc MPa	Total Unit Wt. (pcf)	Total Stress po (tsf)	Eff. Stress p'o (tsf)	rd	F %	n	Max 1.70 Cq	Q	Moss qc1 MPa	Moss Δqc MPa	Moss qc_{1mod} MPa	Moss eff K_C	Qc1n	Ic	Overide	Liquef. Suscept. (0 or 1)	Rel. Dens. Dr (%)	K_C	K_H	Clean Sand Qc1n	Sand 1.0 Kσ	CRR	Induced M=7.5 CSR	Liquefac. Safety Factor	Qc1n $N_{1(60)}$ Ratio	Equiv. $N_{1(60)}$	FC Adj. $\Delta N_{1(60)}$	Equiv. $N_{1(60)cs}$	Volumetric Strain (%)
0.33	0.10	3.95	0.35	8.86	0.38	115	0.019	0.019	1.000	8.90	1.00	1.70	6.32	0.64	4.56	5.20		6.35	3.44		0			1.00		1.00		0.309	Non-Liq.	2.3	2.8			0.00
0.66	0.20	12.55	0.31	2.49	1.20	115	0.038	0.038	1.000	2.50	0.82	1.70	20.10	2.04	2.02	4.06		20.17	2.70		0			1.00		1.00		0.309	Non-Liq.	3.7	5.4			0.00
0.98	0.30	16.50	0.83	5.00	1.58	115	0.057	0.057	1.000	5.02	0.85	1.70	26.42	2.69	4.56	7.25		26.51	2.81		0			1.00		1.00		0.309	Non-Liq.	3.5	7.6			0.00
1.31	0.40	16.40	0.83	5.03	1.57	115	0.075	0.075	0.999	5.05	0.85	1.70	26.23	2.67	4.56	7.23		26.35	2.81		0			1.00		1.00		0.309	Non-Liq.	3.5	7.5			0.00
1.64	0.50	13.10	0.80	6.11	1.25	115	0.094	0.094	0.998	6.15	0.89	1.70	20.90	2.13	4.56	6.69		21.05	2.94		0			1.00		1.00		0.309	Non-Liq.	3.2	6.5			0.00
1.97	0.60	23.60	0.65	2.75	2.26	115	0.113	0.113	0.997	2.77	0.76	1.70	37.74	6.32	2.28	8.60		62.35	2.52		0			1.64		1.00		0.308	Non-Liq.	4.1	15.3			0.00
2.30	0.70	12.50	0.68	5.40	1.20	115	0.132	0.132	0.997	5.46	0.89	1.70	19.87	2.03	4.56	6.59		20.09	2.92		0			1.00		1.00		0.308	Non-Liq.	3.3	6.1			0.00
2.62	0.80	12.65	0.83	6.52	1.21	115	0.151	0.151	0.996	6.60	0.90	1.70	20.08	2.06	4.55	6.61		20.33	2.98		0			1.00		1.00		0.308	Non-Liq.	3.2	6.4			0.00
2.95	0.90	13.10	0.90	6.87	1.25	115	0.170	0.170	0.995	6.96	0.90	1.70	20.78	2.13	4.55	6.69		21.05	2.98		0			1.00		1.00		0.308	Non-Liq.	3.2	6.7			0.00
3.28	1.00	12.55	0.88	6.97	1.20	115	0.189	0.189	0.994	7.08	0.91	1.70	19.86	2.04	4.55	6.59		20.17	3.00		0			1.00		1.00		0.307	Non-Liq.	3.2	6.4			0.00
3.61	1.10	11.00	0.75	6.82	1.05	115	0.208	0.208	0.994	6.95	0.92	1.70	17.34	1.79	4.55	6.34		17.67	3.04		0			1.00		1.00		0.307	Non-Liq.	3.1	5.8			0.00
3.94	1.20	9.45	0.60	6.35	0.90	115	0.226	0.226	0.993	6.51	0.93	1.70	14.82	1.54	4.55	6.09		15.18	3.07		0			1.00		1.00		0.307	Non-Liq.	3.0	5.1			0.00
4.27	1.30	8.15	0.53	6.44	0.78	115	0.245	0.245	0.992	6.64	0.95	1.70	12.70	1.33	4.55	5.87		13.10	3.13		0			1.00		1.00		0.306	Non-Liq.	2.9	4.5			0.00
4.59	1.40	9.35	0.45	4.81	0.90	115	0.264	0.264	0.991	4.95	0.91	1.70	14.60	1.52	4.36	5.88		15.02	3.00		0			1.00		1.00		0.306	Non-Liq.	3.1	4.8			0.00
4.92	1.50	11.40	0.30	2.63	1.09	115	0.283	0.283	0.990	2.70	0.84	1.70	17.86	1.86	2.15	4.01		18.32	2.77		0			1.00		1.00		0.306	Non-Liq.	3.6	5.1			0.00
5.25	1.60	8.90	0.15	1.69	0.85	130	0.304	0.304	0.990	1.75	0.83	1.70	13.81	1.45	1.20	2.65		14.30	2.75		0			1.00		1.00		0.306	Non-Liq.	3.6	4.0			0.00
5.58	1.70	4.95	0.10	2.02	0.47	130	0.326	0.326	0.989	2.16	0.92	1.70	7.43	0.81	1.53	2.34		7.95	3.03		0			1.00		1.00		0.305	Non-Liq.	3.1	2.6			0.00
5.91	1.80	2.55	0.10	3.92	0.24	130	0.347	0.347	0.988	4.54	1.00	1.70	3.54	0.42	3.45	3.87		4.10	3.47		0			1.00		1.00		0.305	Non-Liq.	2.2	1.9			0.00
6.23	1.90	2.60	0.10	3.85	0.25	130	0.368	0.368	0.987	4.48	1.00	1.70	3.59	0.42	3.38	3.80		4.18	3.46		0			1.00		1.00		0.305	Non-Liq.	2.2	1.9			0.00
6.56	2.00	3.40	0.10	2.94	0.33	130	0.390	0.390	0.987	3.32	1.00	1.70	4.84	0.55	2.46	3.02		5.46	3.28		0			1.00		1.00		0.305	Non-Liq.	2.6	2.1			0.00
6.89	2.10	3.50	0.10	2.86	0.34	130	0.411	0.411	0.986	3.24	1.00	1.70	4.96	0.57	2.38	2.95		5.62	3.27		0			1.00		1.00		0.305	Non-Liq.	2.6	2.2			0.00
7.22	2.20	3.55	0.10	2.82	0.34	130	0.432	0.432	0.985	3.21	1.00	1.70	5.01	0.58	2.34	2.91		5.70	3.26		0			1.00		1.00		0.305	Non-Liq.	2.6	2.2			0.00
7.55	2.30	3.60	0.10	2.78	0.34	130	0.454	0.454	0.984	3.18	1.00	1.70	5.06	0.59	2.30	2.88		5.78	3.26		0			1.00		1.00		0.304	Non-Liq.	2.6	2.2			0.00
7.87	2.40	3.60	0.10	2.78	0.34	130	0.475	0.475	0.984	3.20	1.00	1.70	5.02	0.59	2.29	2.88		5.78	3.26		0			1.00		1.00		0.304	Non-Liq.	2.6	2.2			0.00
8.20	2.50	3.35	0.10	2.99	0.32	130	0.496	0.495	0.983	3.50	1.00	1.70	4.59	0.55	2.89	3.43		5.38	3.32		0			1.00		1.00		0.457	Non-Liq.	2.5	2.1			0.00
8.53	2.60	3.10	0.10	3.23	0.30	130	0.518	0.506	0.982	3.86	1.00	1.70	4.17	0.50	3.19	3.69		4.98	3.37		0			1.00		1.00		0.465	Non-Liq.	2.4	2.1			0.00
8.86	2.70	2.95	0.10	3.39	0.28	130	0.539	0.517	0.982	4.11	1.00	1.70	3.91	0.48	3.40	3.88		4.74	3.41		0			1.00		1.00		0.474	Non-Liq.	2.3	2.0			0.00
9.19	2.80	3.40	0.10	2.94	0.33	130	0.560	0.528	0.981	3.48	1.00	1.70	4.61	0.55	2.89	3.44		5.46	3.31		0			1.00		1.00		0.482	Non-Liq.	2.5	2.2			0.00
9.51	2.90	3.85	0.10	2.60	0.37	130	0.582	0.539	0.980	3.02	1.00	1.70	5.32	0.63	2.49	3.12		6.19	3.23		0			1.00		1.00		0.490	Non-Liq.	2.7	2.3			0.00
9.84	3.00	3.95	0.15	3.80	0.38	130	0.603	0.550	0.979	4.41	1.00	1.70	5.46	0.64	3.94	4.58		6.35	3.31		0			1.00		1.00		0.497	Non-Liq.	2.5	2.5			0.00
10.17	3.10	5.20	0.20	3.85	0.50	130	0.624	0.562	0.979	4.31	0.99	1.70	7.45	0.85	4.01	4.86		8.36	3.19		0			1.00		1.00		0.504	Non-Liq.	2.8	3.0			0.00
10.50	3.20	9.20	0.20	2.17	0.88	130	0.646	0.573	0.978	2.32	0.88	1.70	13.86	1.38	2.02	3.39		14.78	2.82		0			1.00		1.00		0.511	Non-Liq.	3.5	4.2			0.00
10.83	3.30	5.65	0.20	3.54	0.54	130	0.667	0.584	0.977	3.95	0.97	1.70	8.14	0.90	3.68	4.57		9.08	3.14		0			1.00		1.00		0.517	Non-Liq.	2.9	3.2			0.00
11.15	3.40	5.30	0.20	3.77	0.51	130	0.688	0.595	0.977	4.25	0.99	1.70	7.56	0.84	3.97	4.81		8.52	3.18		0			1.00		1.00		0.523	Non-Liq.	2.8	3.1			0.00
11.48	3.50	5.15	0.20	3.88	0.49	130	0.709	0.606	0.976	4.40	1.00	1.70	7.30	0.81	4.12	4.93		8.28	3.20		0			1.00		1.00		0.529	Non-Liq.	2.7	3.0			0.00
11.81	3.60	4.15	0.20	4.82	0.40	130	0.731	0.617	0.975	5.66	1.00	1.70	5.68	0.67	5.28	5.95		6.67	3.36		0			1.00		1.00		0.535	Non-Liq.	2.4	2.7			0.00
12.14	3.70	4.20	0.20	4.76	0.40	130	0.752	0.628	0.975	5.60	1.00	1.68	5.69	0.67	5.23	5.90		6.69	3.35		0			1.00		1.00		0.541	Non-Liq.	2.4	2.7			0.00
12.47	3.80	4.40	0.20	4.55	0.42	130	0.773	0.639	0.974	5.32	1.00	1.66	5.88	0.68	4.98	5.66		6.88	3.33		0			1.00		1.00		0.546	Non-Liq.	2.5	2.8			0.00
12.80	3.90	4.80	0.20	4.17	0.46	130	0.795	0.650	0.973	4.82	1.00	1.63	6.38	0.72	4.52	5.24		7.38	3.27		0			1.00		1.00		0.551	Non-Liq.	2.6	2.8			0.00
13.12	4.00	4.40	0.20	4.55	0.42	130	0.816	0.661	0.973	5.35	1.00	1.60	5.65	0.66	5.00	5.66		6.65	3.34		0			1.00		1.00		0.556	Non-Liq.	2.5	2.7			0.00
13.45	4.10	4.70	0.20	4.26	0.45	130	0.837	0.672	0.972	4.97	1.00	1.57	5.99	0.68	4.66	5.34		6.99	3.30		0			1.00		1.00		0.561	Non-Liq.	2.5	2.8			0.00
13.78	4.20	5.65	0.20	3.54	0.54	130	0.859	0.684	0.971	4.03	0.99	1.54	7.22	0.78	3.78	4.56		8.22	3.19		0			1.00		1.00		0.565	Non-Liq.	2.8	3.0			0.00
14.11	4.30	5.90	0.20	3.39	0.56	130	0.880	0.695	0.970	3.84	0.98	1.51	7.43	0.80	3.60	4.40		8.42	3.16		0			1.00		1.00		0.570	Non-Liq.	2.8	3.0			0.00
14.44	4.40	5.85	0.20	3.42	0.56	130	0.901	0.706	0.970	3.89	0.98	1.49	7.24	0.79	3.65	4.43		8.23	3.18		0			1.00		1.00		0.574	Non-Liq.	2.8	3.0			0.00
14.76	4.50	6.30	0.20	3.17	0.60	130	0.923	0.717	0.969	3.58	0.97	1.46	7.69	0.83	3.35	4.18		8.68	3.13		0			1.00		1.00		0.578	Non-Liq.	2.9	3.0			0.00
15.09	4.60	7.25	0.25	3.45	0.69	130	0.944	0.728	0.968	3.83	0.96	1.43	8.82	0.92	3.70	4.62		9.80	3.10		0			1.00		1.00		0.582	Non-Liq.	2.9	3.3			0.00
15.42	4.70	36.45	0.40	1.10	3.49	130	0.965	0.739	0.968	1.12	0.68	1.28	43.15	6.40	0.75	7.15		66.89	2.23		0			1.52		1.00		0.586	Non-Liq.	4.6	14.4			0.00
15.75	4.80	42.25	0.60	1.42	4.05	130	0.987	0.750	0.967	1.45	0.69	1.27	49.71	7.21	1.16	8.37		76.86	2.25		0			1.52		1.00		0.589	Non-Liq.	4.6	16.7			0.00
16.08	4.90	14.65	0.55	3.75	1.40	130	1.008	0.761	0.966	3.96	0.88	1.34	17.56	1.68	4.10	5.78		18.52	2.87		0			1.00		1.00		0.593	Non-Liq.	3.4	5.5			0.00

Depth (feet)	(m)	Tip Qc (tsf)	Friction Fs (tsf)	Friction Ratio Rf %	qc MPa	Total Unit Wt. (pcf)	Total Stress po (tsf)	Eff. Stress p'o (tsf)	rd	F %	n	Max 1.70 Cq	Q	Moss qc1 MPa	Moss Δqc MPa	Moss qc_{1mod} MPa	Moss eff K_C	Qc1n	Ic	Overide	Liquef. Suscept. (0 or 1)	Rel. Dens. Dr (%)	K_C	K_H	Clean Sand Qc1n	1.0 Kσ	CRR	Induced M=7.5 CSR	Liquefac. Safety Factor	Qc1n $N_{1(60)}$ Ratio	Equiv. $N_{1(60)}$	FC Adj. $\Delta N_{1(60)}$	Equiv. $N_{1(60)cs}$	Volumetric Strain (%)
16.40	5.00	7.90	0.30	3.80	0.76	130	1.029	0.772	0.965	4.21	0.96	1.35	9.11	0.95	4.17	5.11		10.09	3.12		0			1.00		1.00		0.596	Non-Liq.	2.9	3.5			0.00
16.73	5.10	7.05	0.15	2.13	0.68	130	1.051	0.783	0.965	2.39	0.93	1.32	7.84	0.86	2.06	2.92		8.82	3.03		0			1.00		1.00		0.599	Non-Liq.	3.1	2.9			0.00
17.06	5.20	7.90	0.18	2.22	0.76	130	1.072	0.794	0.964	2.46	0.92	1.30	8.74	0.94	2.17	3.12		9.72	3.00		0			1.00		1.00		0.603	Non-Liq.	3.1	3.1			0.00
17.39	5.30	14.65	0.20	1.37	1.40	130	1.093	0.806	0.963	1.44	0.81	1.25	16.32	1.68	1.10	2.78		17.27	2.65		0			1.00		1.00		0.606	Non-Liq.	3.8	4.5			0.00
17.72	5.40	11.00	0.15	1.36	1.05	130	1.115	0.817	0.962	1.47	0.85	1.25	11.99	1.27	1.10	2.37		12.95	2.77		0			1.00		1.00		0.609	Non-Liq.	3.6	3.6			0.00
18.04	5.50	7.30	0.18	2.40	0.70	130	1.136	0.828	0.962	2.70	0.94	1.26	7.70	0.85	2.42	3.26		8.69	3.07		0			1.00		1.00		0.612	Non-Liq.	3.0	2.9			0.00
18.37	5.60	7.50	0.20	2.67	0.72	130	1.157	0.839	0.961	3.00	0.94	1.25	7.84	0.86	2.76	3.62		8.83	3.08		0			1.00		1.00		0.614	Non-Liq.	3.0	3.0			0.00
18.70	5.70	7.30	0.20	2.74	0.70	130	1.179	0.850	0.960	3.10	0.95	1.23	7.51	0.83	2.86	3.69		8.50	3.11		0			1.00		1.00		0.617	Non-Liq.	2.9	2.9			0.00
19.03	5.80	7.20	0.18	2.43	0.69	130	1.200	0.861	0.959	2.76	0.95	1.22	7.28	0.81	2.47	3.28		8.27	3.09		0			1.00		1.00		0.619	Non-Liq.	2.9	2.8			0.00
19.36	5.90	7.35	0.15	2.04	0.70	130	1.221	0.872	0.959	2.32	0.93	1.20	7.33	0.82	1.97	2.79		8.32	3.05		0			1.00		1.00		0.622	Non-Liq.	3.0	2.7			0.00
19.69	6.00	8.55	0.18	2.05	0.82	130	1.243	0.883	0.958	2.28	0.91	1.18	8.55	0.94	1.98	2.92		9.53	2.99		0			1.00		1.00		0.624	Non-Liq.	3.1	3.0			0.00
20.01	6.10	9.00	0.23	2.50	0.86	130	1.264	0.894	0.957	2.78	0.92	1.17	8.94	0.97	2.56	3.54		9.93	3.02		0			1.00		1.00		0.627	Non-Liq.	3.1	3.2			0.00
20.34	6.20	11.20	0.45	4.02	1.07	130	1.285	0.905	0.956	4.37	0.93	1.16	11.25	1.18	4.52	5.69		12.24	3.05		0			1.00		1.00		0.629	Non-Liq.	3.0	4.0			0.00
20.67	6.30	22.95	1.03	4.47	2.20	130	1.307	0.916	0.955	4.65	0.86	1.13	23.56	2.34	5.10	7.43		24.54	2.82		0			1.00		1.00		0.631	Non-Liq.	3.5	7.1			0.00
21.00	6.40	37.00	1.53	4.12	3.54	130	1.328	0.928	0.954	4.23	0.80	1.11	37.90	3.71	4.66	8.37		38.88	2.64		0			1.00		1.00		0.633	Non-Liq.	3.8	10.2			0.00
21.33	6.50	39.75	1.68	4.21	3.81	130	1.349	0.939	0.953	4.32	0.80	1.10	40.37	3.97	4.78	8.75		41.35	2.63		0			1.00		1.00		0.635	Non-Liq.	3.9	10.7			0.00
21.65	6.60	39.90	1.80	4.51	3.82	130	1.371	0.950	0.952	4.62	0.81	1.09	40.17	3.96	5.17	9.13		41.15	2.65		0			1.00		1.00		0.637	Non-Liq.	3.8	10.8			0.00
21.98	6.70	79.45	2.18	2.74	7.61	130	1.392	0.961	0.951	2.77	0.70	1.07	79.33	11.03	2.89	13.92		112.97	2.29		0			1.41		1.00		0.639	Non-Liq.	4.5	25.0			0.00
22.31	6.80	70.70	2.43	3.43	6.77	130	1.413	0.972	0.951	3.48	0.73	1.06	70.11	9.77	3.78	13.55		100.01	2.40		0			1.41		1.00		0.640	Non-Liq.	4.3	23.2			0.00
22.64	6.90	31.10	2.10	6.75	2.98	130	1.435	0.983	0.950	6.97	0.87	1.07	30.35	3.05	5.81	8.86		31.34	2.87		0			1.00		1.00		0.642	Non-Liq.	3.4	9.2			0.00
22.97	7.00	53.80	1.78	3.30	5.15	130	1.456	0.994	0.949	3.36	0.75	1.05	52.31	6.06	3.62	9.68		61.42	2.47		0			1.15		1.00		0.644	Non-Liq.	4.2	14.8			0.00
23.29	7.10	23.60	1.43	6.04	2.26	130	1.477	1.005	0.948	6.31	0.89	1.05	22.35	2.31	5.82	8.13		23.35	2.93		0			1.00		1.00		0.645	Non-Liq.	3.3	7.1			0.00
23.62	7.20	15.45	1.15	7.44	1.48	130	1.499	1.016	0.946	7.97	0.95	1.04	14.18	1.51	5.82	7.33		15.17	3.14		0			1.00		1.00		0.647	Non-Liq.	2.8	5.3			0.00
23.95	7.30	32.10	1.35	4.21	3.07	130	1.520	1.027	0.945	4.34	0.83	1.02	30.09	3.11	4.80	7.91		31.09	2.72		0			1.00		1.00		0.648	Non-Liq.	3.7	8.5			0.00
24.28	7.40	49.90	1.70	3.41	4.78	130	1.541	1.038	0.944	3.48	0.76	1.01	46.85	4.83	3.77	8.59		48.02	2.52		0			1.00		1.00		0.649	Non-Liq.	4.1	11.8			0.00
24.61	7.50	51.30	1.98	3.85	4.91	130	1.563	1.050	0.943	3.93	0.77	1.01	47.79	4.93	4.34	9.27	1.88	48.79	2.55		1	47	3.03	1.00	147.8	1.00	0.380	0.651	**0.58**	4.0	12.2	5.5	17.7	**1.69**
24.93	7.60	66.95	1.90	2.84	6.41	130	1.584	1.061	0.942	2.88	0.72	1.00	62.17	6.41	3.03	9.44	1.47	63.17	2.37		1	58	2.21	1.00	139.4	1.00	0.332	0.652	**0.51**	4.4	14.5	5.5	20.0	**1.51**
25.26	7.70	111.50	1.40	1.26	10.68	130	1.605	1.072	0.941	1.27	0.60	0.99	103.58	10.63	0.98	11.61	1.09	104.58	1.97		1	79	1.26	1.00	131.6	0.99	0.292	0.653	**0.44**	5.1	20.3	5.5	25.8	**1.14**
25.59	7.80	133.60	1.10	0.82	12.79	130	1.626	1.083	0.940	0.83	0.54	0.99	123.69	12.67	0.42	13.09	1.03	124.70	1.79		1	86	1.10	1.00	136.9	0.99	0.319	0.654	**0.48**	5.5	22.7	4.7	27.4	**1.05**
25.92	7.90	140.75	1.05	0.75	13.48	130	1.648	1.094	0.938	0.75	0.53	0.98	129.68	13.29	0.32	13.61	1.02	130.70	1.74		1	88	1.07	1.00	139.6	0.99	0.333	0.655	**0.50**	5.6	23.4	4.5	27.9	**1.02**
26.25	8.00	134.45	0.93	0.69	12.87	130	1.669	1.105	0.937	0.69	0.53	0.98	123.17	12.63	0.24	12.88	1.02	124.19	1.74		1	86	1.07	1.00	132.3	0.98	0.295	0.656	**0.44**	5.6	22.2	4.2	26.5	**1.10**
26.57	8.10	115.20	0.90	0.78	11.03	130	1.690	1.116	0.936	0.79	0.56	0.97	104.67	10.78	0.37	11.14	1.03	105.69	1.83		1	79	1.13	1.00	119.4	0.98	0.238	0.657	**0.36**	5.4	19.5	4.4	23.9	**1.25**
26.90	8.20	53.05	1.20	2.26	5.08	130	1.712	1.127	0.935	2.31	0.73	0.96	46.87	4.96	2.29	7.25	1.46	47.89	2.40		1	46	2.30	1.00	110.1	0.99	0.204	0.658	**0.31**	4.3	11.1	5.5	16.6	**1.78**
27.23	8.30	116.95	1.78	1.52	11.20	130	1.733	1.138	0.933	1.53	0.61	0.96	104.67	10.92	1.32	12.25	1.12	105.70	2.02		1	79	1.33	1.00	140.2	0.97	0.336	0.658	**0.50**	5.0	21.0	5.5	26.5	**1.10**
27.56	8.40	191.45	2.13	1.11	18.33	130	1.754	1.149	0.932	1.12	0.54	0.96	172.03	17.82	0.79	18.62	1.04	173.06	1.77		1	100	1.09	1.00	187.8	0.97	Infin.	0.659	Non-Liq.	5.5	31.3	5.0	36.3	0.00
27.89	8.50	207.65	2.38	1.14	19.88	130	1.776	1.160	0.930	1.15	0.53	0.95	185.77	19.29	0.84	20.13	1.04	186.81	1.76		1	100	1.08	1.00	200.9	0.96	Infin.	0.660	Non-Liq.	5.6	33.6	5.0	38.6	0.00
28.22	8.60	221.40	2.33	1.05	21.20	130	1.797	1.171	0.929	1.06	0.52	0.95	197.39	20.49	0.72	21.20	1.03	198.44	1.71		1	100	1.05	1.00	207.4	0.96	Infin.	0.660	Non-Liq.	5.6	35.1	5.0	40.1	0.00
28.54	8.70	224.95	2.15	0.96	21.54	130	1.818	1.183	0.928	0.96	0.51	0.94	199.80	20.72	0.59	21.32	1.03	200.86	1.68		1	100	1.02	1.00	205.4	0.96	Infin.	0.661	Non-Liq.	5.7	35.2	5.0	40.2	0.00
28.87	8.80	215.80	2.10	0.97	20.67	130	1.840	1.194	0.926	0.98	0.52	0.94	190.57	19.82	0.62	20.43	1.03	191.63	1.70		1	100	1.04	1.00	198.6	0.95	Infin.	0.661	Non-Liq.	5.7	33.8	5.0	38.8	0.00
29.20	8.90	178.80	2.35	1.31	17.12	130	1.861	1.205	0.925	1.32	0.56	0.93	156.02	16.42	1.06	17.48	1.06	157.08	1.85		1	96	1.15	1.00	180.1	0.95	Infin.	0.662	Non-Liq.	5.4	29.2	5.0	34.2	0.00
29.53	9.00	205.90	2.78	1.35	19.72	130	1.882	1.216	0.923	1.36	0.55	0.93	179.14	18.88	1.10	19.99	1.06	180.21	1.82		1	100	1.12	1.00	202.0	0.95	Infin.	0.662	Non-Liq.	5.4	33.1	5.0	38.1	0.00
29.86	9.10	236.35	2.63	1.11	22.63	130	1.904	1.227	0.921	1.12	0.52	0.93	205.68	21.57	0.80	22.37	1.04	206.75	1.72		1	100	1.05	1.00	216.8	0.94	Infin.	0.662	Non-Liq.	5.6	36.7	5.0	41.7	0.00
30.18	9.20	188.65	2.00	1.06	18.07	130	1.925	1.238	0.920	1.07	0.54	0.92	162.75	17.11	0.73	17.84	1.04	163.82	1.77		1	97	1.09	1.00	178.1	0.94	Infin.	0.663	Non-Liq.	5.5	29.6	5.0	34.6	0.00
30.51	9.30	99.95	1.90	1.90	9.57	130	1.946	1.249	0.918	1.93	0.65	0.90	83.70	9.06	1.83	10.89		84.76	2.16		0			1.00		0.97		0.663	Non-Liq.	4.8	17.8			0.00
30.84	9.40	48.75	1.98	4.05	4.67	130	1.968	1.260	0.916	4.16	0.79	0.87	39.08	4.41	4.63	9.04		40.12	2.63		0			1.00		0.97		0.663	Non-Liq.	3.9	10.4			0.00
31.17	9.50	163.75	2.00	1.22	15.68	130	1.989	1.271	0.914	1.23	0.57	0.90	138.39	19.28	0.94	20.22		182.51	1.87		0			1.31		0.96		0.663	Non-Liq.	5.3	34.1			0.00
31.50	9.60	144.00	2.03	1.41	13.79	130	2.010	1.282	0.913	1.42	0.59	0.89	120.37	16.92	1.18	18.10	1.07	158.93	1.95		1	96	1.24	1.31	197.5	0.93	Infin.	0.663	Non-Liq.	5.2	30.7	5.0	35.7	0.00
31.82	9.70	73.30	1.60	2.18	7.02	130	2.032	1.293	0.911	2.22	0.70	0.87	59.16	8.57	2.19	10.76	1.26	78.80	2.31		1	67	1.98	1.31	156.2	0.94	0.435	0.663	**0.62**	4.5	17.6	5.5	23.1	**1.30**
32.15	9.80	64.30	1.05	1.63	6.16	130	2.053	1.305	0.909	1.67	0.69	0.87	51.57	7.42	1.48	8.90		68.88	2.27		0			1.31		0.96		0.663	Non-Liq.	4.5	15.1			0.00
32.48	9.90	23.45	0.65	2.77	2.25	130	2.074	1.316	0.907	2.94	0.84	0.83	17.42	2.02	2.96	4.99		18.46	2.80		0			1.00		0.96		0.662	Non-Liq.	3.5	5.2			0.00
32.81	10.00	18.25	0.55	3.01	1.75	130	2.096	1.327	0.905	3.25	0.88	0.82	13.12	1.56	3.28	4.83		14.15	2.92		0			1.00		0.96		0.662	Non-Liq.	3.3	4.3			0.00
33.14	10.10	15.75	0.58	3.65	1.51	130	2.117	1.338	0.903	3.99	0.91	0.81	11.01	1.33	4.11	5.44		12.03	3.04		0			1.00		0.95		0.662	Non-Liq.	3.1	3.9			0.00
33.46	10.20	23.90	0.70	2.93	2.29	130	2.138	1.349	0.901	3.10	0.84	0.81	17.37	2.04	3.16	5.21		18.41	2.81		0			1.00		0.95		0.662	Non-Liq.	3.5	5.3			0.00
33.79	10.30	14.50	0.60	4.14	1.39	130	2.160	1.360	0.899	4.57	0.93	0.79	9.83	1.22	4.74	5.95		10.85	3.11		0			1.00		0.95		0.661	Non-Liq.	2.9	3.7			0.00
34.12	10.40	12.65	0.43	3.36	1.21	130	2.181	1.371	0.897	3.77	0.93	0.79	8.37	1.04	3.72	4.77		9.39	3.12		0			1.00		0.95		0.661	Non-Liq.	2.9	3.2			0.00
34.45	10.50	13.00	0.40	3.08	1.24	130	2.202	1.382	0.894	3.44	0.92	0.78	8.58	1.06	3.36	4.42		9.60	3.09		0			1.00		0.95		0.660	Non-Liq.	3.0	3.2			0.00
34.78	10.60	13.75	0.40	2.91	1.32	130	2.224	1.393	0.892	3.24	0.91	0.78	9.09	1.12	3.14	4.26		10.11	3.05		0			1.00		0.95		0.660	Non-Liq.	3.0	3.3			0.00
35.10	10.70	14.50	0.40	2.76	1.39	130	2.245	1.404	0.890	3.05	0.90	0.77	9.59	1.18	2.94	4.12		10.62	3.02		0			1.00		0.94		0.659	Non-Liq.	3.1	3.4			0.00
35.43	10.80	12.70	0.38	2.95	1.22	130	2.266	1.415	0.888	3.32	0.92	0.76	8.15	1.02	3.19	4.21		9.17	3.10		0			1.00		0.94		0.658	Non-Liq.	2.9	3.1			0.00
35.76	10.90	16.45	0.48	2.89	1.58	130	2.288	1.427	0.885	3.16	0.89	0.77	10.88	1.34	3.11	4.45		11.92	2.98		0			1.00		0.94		0.658	Non-Liq.	3.2	3.8			0.00
36.09	11.00	84.25	0.75	0.89	8.07	130	2.309	1.438	0.883	0.91	0.61	0.83	64.83	7.06	0.51	7.57		66.21	2.03		0			1.00		0.94		0.657	Non-Liq.	5.0	13.2			0.00
36.42	11.10	94.85	0.88	0.92	9.08	130	2.330	1.449	0.880	0.94	0.60	0.83	73.00	7.94	0.55	8.49	1.07	74.13	2.00		1	64	1.30	1.00	96.4	0.91	0.163	0.656	**0.23**	5.1	14.6	4.7	19.3	**1.57**
36.75	11.20	117.20	0.65	0.55	11.22	130	2.352	1.460	0.878	0.56	0.54	0.84	91.82	9.59	0.07	9.66	1.01	92.98	1.79		1	74	1.10	1.00	102.4	0.88	0.180	0.655	**0.24**	5.5	16.9	3.5	20.5	**1.48**
37.07	11.30	75.15	0.68	0.90	7.20	130	2.373	1.471	0.875	0.92	0.63	0.81	56.61	6.19	0.52	6.70	1.08	57.74	2.08		1	54	1.43	1.00	82.4	0.91	0.132	0.654	**0.18**	4.9	11.7	4.7	16.5	**1.80**
37.40	11.40	45.75	0.83	1.80	4.38	130	2.394	1.482	0.873	1.86	0.74	0.78	32.63	3.80	1.69	5.49	1.45	33.73	2.46		1	32	2.57	1.00	86.7	0.93	0.141	0.653	**0.20**	4.2	8.1	5.5	13.6	**2.09**

Depth (feet)	(m)	Tip Qc (tsf)	Friction Fs (tsf)	Friction Ratio Rf %	qc MPa	Total Unit Wt. (pcf)	Total Stress po (tsf)	Eff. Stress p'o (tsf)	rd	F %	n	Max 1.70 Cq	Q	Moss qc1 MPa	Moss Δqc MPa	Moss qc_{1mod} MPa	Moss eff K_C	Qc1n	Ic	Overide	Liquef. Suscept. (0 or 1)	Rel. Dens. Dr (%)	K_C	K_H	Clean Sand Qc1n	1.0 Kσ	CRR	Induced M=7.5 CSR	Liquefac. Safety Factor	Qc1n $N_{1(60)}$ Ratio	Equiv. $N_{1(60)}$	FC Adj. $\Delta N_{1(60)}$	Equiv. $N_{1(60)cs}$	Volumetric Strain (%)
37.73	11.50	116.75	0.65	0.56	11.18	130	2.416	1.493	0.870	0.56	0.55	0.83	90.26	9.45	0.07	9.53	1.01	91.43	1.80		1	73	1.11	1.00	101.1	0.87	0.176	0.652	**0.24**	5.5	16.7	3.5	20.2	**1.50**
38.06	11.60	158.80	0.63	0.39	15.21	130	2.437	1.504	0.868	0.40	0.50	0.84	124.69	12.67	0.00	12.67	1.00	125.88	1.60		1	86	1.00	1.00	125.9	0.87	0.266	0.651	**0.35**	5.9	21.5	3.7	25.2	**1.17**
38.39	11.70	165.70	0.78	0.47	15.87	130	2.458	1.515	0.865	0.47	0.50	0.84	129.67	13.34	0.00	13.34	1.00	130.87	1.63		1	88	1.00	1.00	130.9	0.87	0.288	0.650	**0.38**	5.8	22.5	3.7	26.2	**1.12**
38.71	11.80	169.20	1.00	0.59	16.20	130	2.479	1.526	0.862	0.60	0.51	0.83	131.41	13.78	0.12	13.90	1.01	132.61	1.68		1	89	1.02	1.00	135.7	0.86	0.312	0.649	**0.42**	5.7	23.2	3.9	27.1	**1.06**
39.04	11.90	180.65	1.58	0.87	17.30	130	2.501	1.537	0.859	0.88	0.54	0.82	138.53	15.03	0.48	15.51	1.03	139.72	1.77		1	91	1.08	1.00	151.3	0.86	0.402	0.648	**0.53**	5.5	25.2	5.0	30.3	**0.76**
39.37	12.00	225.10	2.25	1.00	21.56	130	2.522	1.549	0.857	1.01	0.53	0.82	172.81	18.93	0.65	19.57	1.03	174.01	1.74		1	100	1.06	1.00	184.8	0.86	Infin.	0.646	**Non-Liq.**	5.6	31.1	5.0	36.1	**0.00**
39.70	12.10	240.25	2.40	1.00	23.01	130	2.543	1.560	0.854	1.01	0.52	0.82	184.23	20.19	0.65	20.84	1.03	185.44	1.72		1	100	1.05	1.00	194.5	0.86	Infin.	0.645	**Non-Liq.**	5.6	32.9	5.0	37.9	**0.00**
40.03	12.20	244.25	2.43	0.99	23.39	130	2.565	1.571	0.851	1.00	0.52	0.81	186.75	20.48	0.64	21.12	1.03	187.96	1.71		1	100	1.04	1.00	196.4	0.85	Infin.	0.644	**Non-Liq.**	5.6	33.3	5.0	38.3	**0.00**
40.35	12.30	248.70	2.78	1.12	23.82	130	2.586	1.582	0.848	1.12	0.53	0.81	188.74	20.94	0.80	21.73	1.04	189.94	1.74		1	100	1.07	1.00	202.7	0.85	Infin.	0.642	**Non-Liq.**	5.6	34.0	5.0	39.0	**0.00**
40.68	12.40	218.75	3.35	1.53	20.95	130	2.607	1.593	0.845	1.54	0.57	0.79	162.41	18.59	1.33	19.92	1.07	163.60	1.89		1	97	1.18	1.00	192.9	0.85	Infin.	0.641	**Non-Liq.**	5.3	30.9	5.0	35.9	**0.00**
41.01	12.50	117.45	3.48	2.96	11.25	130	2.629	1.604	0.842	3.00	0.69	0.75	82.11	10.05	3.17	13.22		83.25	2.30		0			1.00		0.92		0.639	**Non-Liq.**	4.5	18.5			**0.00**
41.34	12.60	58.30	2.28	3.90	5.58	130	2.650	1.615	0.839	4.01	0.78	0.72	38.46	4.90	4.39	9.29		39.56	2.62		0			1.00		0.92		0.638	**Non-Liq.**	3.9	10.2			**0.00**
41.67	12.70	23.45	1.03	4.37	2.25	130	2.671	1.626	0.836	4.70	0.89	0.68	14.07	1.87	4.99	6.86		15.12	2.99		0			1.00		0.92		0.636	**Non-Liq.**	3.1	4.8			**0.00**
41.99	12.80	16.85	0.68	4.01	1.61	130	2.693	1.637	0.833	4.44	0.92	0.67	9.61	1.30	4.51	5.81		10.65	3.11		0			1.00		0.92		0.635	**Non-Liq.**	2.9	3.7			**0.00**
42.32	12.90	15.65	0.60	3.83	1.50	130	2.714	1.648	0.830	4.29	0.93	0.66	8.77	1.19	4.29	5.48		9.80	3.13		0			1.00		0.92		0.633	**Non-Liq.**	2.9	3.4			**0.00**
42.65	13.00	17.65	0.60	3.40	1.69	130	2.735	1.659	0.827	3.75	0.90	0.67	10.06	1.34	3.73	5.07		11.11	3.05		0			1.00		0.91		0.631	**Non-Liq.**	3.0	3.7			**0.00**
42.98	13.10	18.60	0.58	3.09	1.78	130	2.757	1.670	0.823	3.40	0.89	0.67	10.65	1.41	3.33	4.74		11.70	3.01		0			1.00		0.91		0.630	**Non-Liq.**	3.1	3.8			**0.00**
43.31	13.20	15.30	0.53	3.43	1.47	130	2.778	1.682	0.820	3.86	0.92	0.65	8.39	1.14	3.76	4.90		9.43	3.12		0			1.00		0.91		0.628	**Non-Liq.**	2.9	3.3			**0.00**
43.64	13.30	14.75	0.48	3.22	1.41	130	2.799	1.693	0.817	3.64	0.92	0.65	7.99	1.09	3.49	4.58		9.03	3.12		0			1.00		0.91		0.626	**Non-Liq.**	2.9	3.1			**0.00**
43.96	13.40	14.10	0.48	3.37	1.35	130	2.821	1.704	0.814	3.83	0.93	0.64	7.51	1.03	3.67	4.71		8.54	3.16		0			1.00		0.91		0.624	**Non-Liq.**	2.8	3.0			**0.00**
44.29	13.50	14.90	0.45	3.02	1.43	130	2.842	1.715	0.811	3.41	0.92	0.64	8.00	1.09	3.23	4.31		9.04	3.11		0			1.00		0.91		0.622	**Non-Liq.**	2.9	3.1			**0.00**
44.62	13.60	13.70	0.40	2.92	1.31	130	2.863	1.726	0.807	3.34	0.93	0.63	7.19	0.99	3.09	4.08		8.22	3.14		0			1.00		0.91		0.621	**Non-Liq.**	2.9	2.9			**0.00**
44.95	13.70	14.45	0.43	2.94	1.38	130	2.885	1.737	0.804	3.34	0.92	0.63	7.61	1.04	3.12	4.16		8.65	3.12		0			1.00		0.91		0.619	**Non-Liq.**	2.9	3.0			**0.00**
45.28	13.80	16.25	0.50	3.08	1.56	130	2.906	1.748	0.801	3.45	0.91	0.63	8.68	1.19	3.29	4.48		9.72	3.08		0			1.00		0.90		0.617	**Non-Liq.**	3.0	3.3			**0.00**
45.60	13.90	18.65	0.70	3.75	1.79	130	2.927	1.759	0.798	4.14	0.91	0.63	10.05	1.40	4.15	5.54		11.10	3.08		0			1.00		0.90		0.615	**Non-Liq.**	3.0	3.7			**0.00**
45.93	14.00	26.10	0.78	2.97	2.50	130	2.949	1.770	0.794	3.19	0.85	0.65	14.84	1.98	3.15	5.13		15.92	2.87		0			1.00		0.90		0.613	**Non-Liq.**	3.4	4.7			**0.00**
46.26	14.10	16.75	0.63	3.73	1.60	130	2.970	1.781	0.791	4.18	0.92	0.62	8.75	1.23	4.11	5.34		9.79	3.13		0			1.00		0.90		0.611	**Non-Liq.**	2.9	3.4			**0.00**
46.59	14.20	15.85	0.53	3.31	1.52	130	2.991	1.792	0.788	3.73	0.92	0.62	8.17	1.15	3.58	4.72		9.22	3.12		0			1.00		0.90		0.609	**Non-Liq.**	2.9	3.2			**0.00**
46.92	14.30	17.50	0.70	4.00	1.68	130	3.013	1.804	0.784	4.46	0.92	0.61	9.06	1.29	4.45	5.74		10.10	3.13		0			1.00		0.90		0.607	**Non-Liq.**	2.9	3.5			**0.00**
47.24	14.40	46.95	1.00	2.13	4.50	130	3.034	1.815	0.781	2.22	0.76	0.66	28.31	5.96	2.07	8.03		48.42	2.55		0			1.64		0.90		0.605	**Non-Liq.**	4.0	12.1			**0.00**
47.57	14.50	22.90	0.85	3.71	2.19	130	3.055	1.826	0.778	4.03	0.89	0.62	12.28	1.72	4.07	5.79		13.34	3.00		0			1.00		0.90		0.603	**Non-Liq.**	3.1	4.3			**0.00**
47.90	14.60	16.85	0.50	2.97	1.61	130	3.077	1.837	0.774	3.33	0.91	0.61	8.60	1.20	3.12	4.33		9.66	3.08		0			1.00		0.90		0.601	**Non-Liq.**	3.0	3.2			**0.00**
48.23	14.70	13.75	0.40	2.91	1.32	130	3.098	1.848	0.771	3.36	0.93	0.59	6.69	0.95	3.05	4.00		7.73	3.17		0			1.00		0.89		0.599	**Non-Liq.**	2.8	2.8			**0.00**
48.56	14.80	13.40	0.40	2.99	1.28	130	3.119	1.859	0.767	3.47	0.94	0.59	6.43	0.92	3.14	4.06		7.46	3.19		0			1.00		0.89		0.597	**Non-Liq.**	2.8	2.7			**0.00**
48.88	14.90	15.20	0.45	2.96	1.46	130	3.141	1.870	0.764	3.38	0.92	0.59	7.45	1.06	3.11	4.17		8.50	3.13		0			1.00		0.89		0.595	**Non-Liq.**	2.9	3.0			**0.00**
49.21	15.00	15.05	0.50	3.32	1.44	130	3.162	1.881	0.761	3.80	0.93	0.58	7.28	1.05	3.56	4.61		8.32	3.17		0			1.00		0.89		0.592	**Non-Liq.**	2.8	3.0			**0.00**
49.54	15.10	15.10	0.55	3.64	1.45	130	3.183	1.892	0.757	4.16	0.94	0.58	7.23	1.06	3.96	5.02		8.27	3.19		0			1.00		0.89		0.590	**Non-Liq.**	2.7	3.0			**0.00**
49.87	15.20	19.15	0.70	3.66	1.83	130	3.205	1.903	0.754	4.06	0.91	0.59	9.55	1.38	3.97	5.35		10.61	3.09		0			1.00		0.89		0.588	**Non-Liq.**	3.0	3.6			**0.00**
50.20	15.30	17.60	0.80	4.55	1.69	130	3.226	1.914	0.751	5.10	0.94	0.57	8.50	1.27	5.08	6.36		9.54	3.19		0			1.00		0.89		0.586	**Non-Liq.**	2.8	3.5			**0.00**

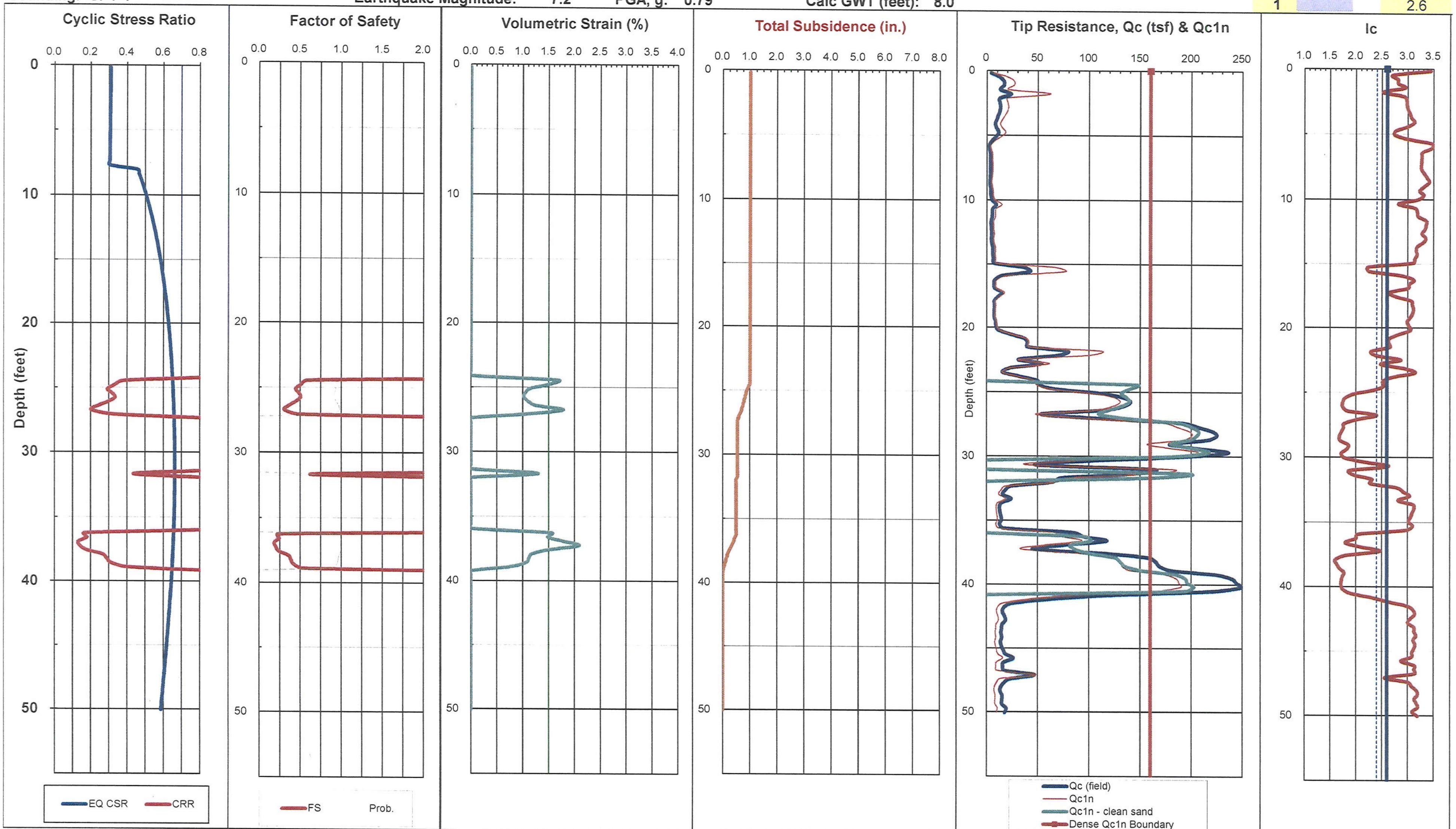
EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Oxnard College Fire Academy | Project No: 302245-001 | Method Used: 1 1998 NCEER (Robertson & Wride)

Settlement Analysis using Tokimatsu & Seed (1987), clean sand Qc1n/N1(60) ratio =5

Sounding: CPT-1 | Earthquake Magnitude: 7.2 | PGA, g: 0.79 | Calc GWT (feet): 8.0

Plot 1 | Limiting Ic: 2.6



Total Thickness of Liquefiable Layers: 6.2 feet

Estimated Total Ground Subsidence (Settlement): 1.0 inches

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

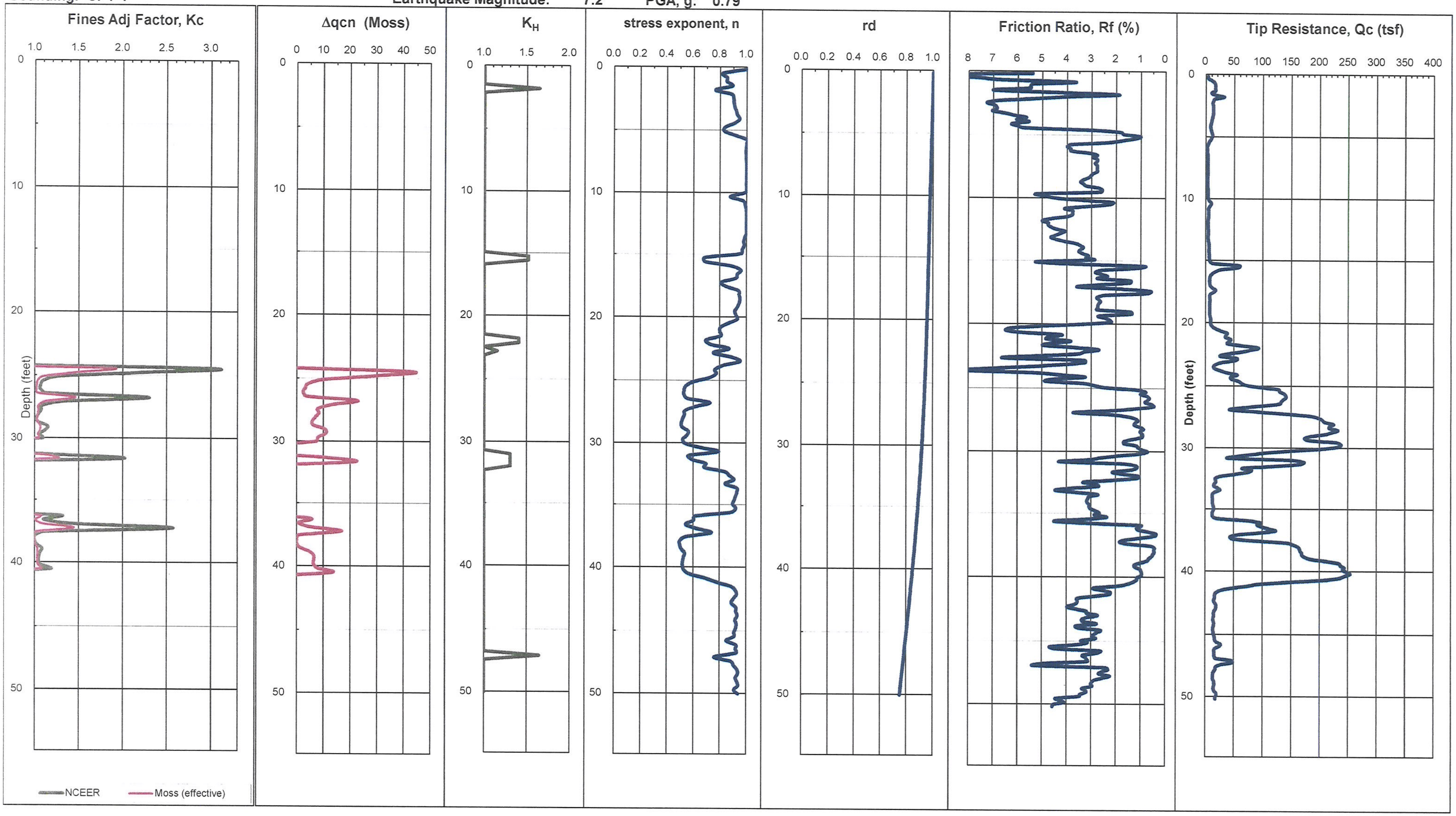
3 avg increment =0.10m Qc1n/N1(60): 5

Ignore 1st/last increment into sand/silt soils: 0

Method Used: 1998 NCEER (Robertson & Wride)

Sounding: CPT-1

Earthquake Magnitude: 7.2 PGA, g: 0.79



Job Number:	302245-001
Job Name:	Oxnard College Fire Academy
Boring Number:	CPT-1
Date:	April 16, 2019
Calculated By:	A. Mazzei

Prediction of Liquefaction Induced Lateral Spreading with Ground Slope Conditions

Based on Data Published in the ASCE Journal of Geotechnicial and Geoenvironmental Engineering December 2002
(Youd, Hansen and Bartlett, 2002)

Variables Used in Calculation Defined

Earthquake Magnitude (M)
Horizontal Distance to Nearest Seismic Energy Source, km (R)
Percent Slope (S)
Cumulative Thickness in Meters of Saturated Cohesionless Sediments with SPT $(N1)_{60}$ Values <= 15 (T_{15})
Average Fines Content in Percent (F_{15})
Mean Grain size in milimeters $(D50_{15})$

$\text{Log } D_H=-16.213+1.532M-1.406\text{Log}(R+10^{(0.89M-5.64)})-0.012R+0.338\text{Log}S+0.540\text{Log}T_{15}+3.413\text{Log}(100-F_{15})-0.795\text{Log}(D50_{15}+0.1\text{mm})$

Requirements and Limitations Used to Develop this Model

Soils must be Liquefiable
Saturated Cohesionless Sediments with SPT $(N1)_{60}$ less than 15
Earthquake Magnitude (M) must be between 6 and 8
Percent Slope (S) must be between 0.1% and 6%
Cumulative Thickness (T15) must be between 1 and 15 meters
Depth to top of Liquefied layer must be between 1 and 10 meters
Distance to Fault Rupture (R_{eq}) must be determined using Figure 10 if soft soils are present.
F_{15} and $D50_{15}$ must be within bounds shown in Fig. 5.
If R or R_{eq} < 0.5 km use 0.5; otherwise use R or R_{eq}.

Input Values		
M =	7.2	
R =	9.9	km
S =	0.5	%
T_{15} =	0.6	m
F_{15} =	30	%
$D50_{15}$ =	0.7	mm

Horizontal Ground Displacement in meters (D_H) = 0.15
Horizontal Ground Displacement in feet (D_H) = 0.5

Displacements should be between 0.1 and 6 meters and should be multiplied by a FOS of 2 for a conservative estimate. Any displacement greater than 6 meters is outside of the data set used in the analysis and may not be an accurate estimate.

Earth Systems

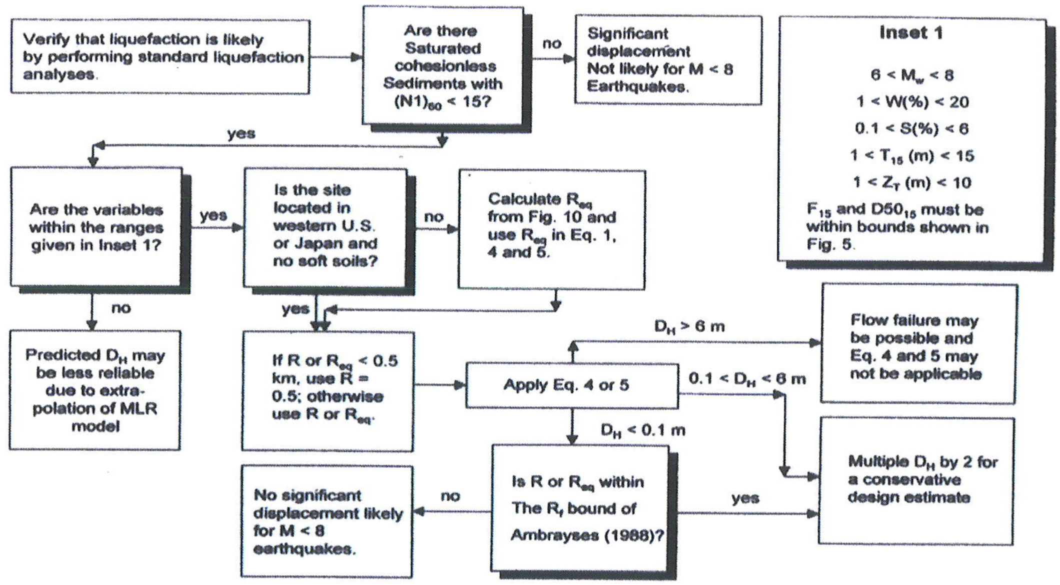


Fig. 9. Flow chart [for application of Eq. (6)]

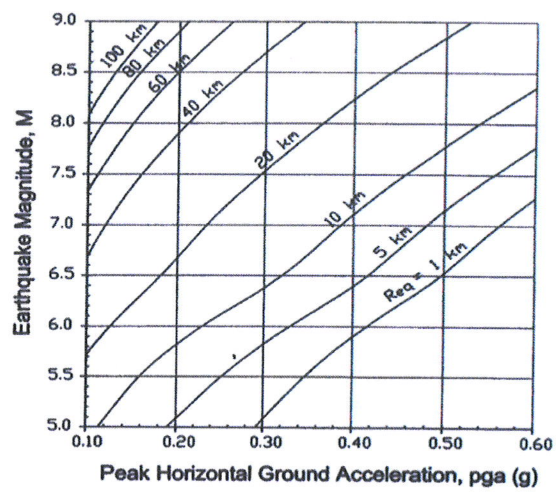


Fig. 10. Graph for determining equivalent source distance, R_{eq}, from magnitude, M, and peak acceleration, a_{max} (revised from Bartlett and Youd 1992, 1995). The above curves are the averages of pga from three different attenuation relations: Abrahamson and Silva (1997); Boore et al. (1997); and Campbell (1997). For the Abrahamson and Silva (1997) relation, the following parameters were used in the regression equation: a) R equals the distance to the fault rupture, b) fault type was set to "otherwise", c) HW=hanging wall factor was set to 1, which implies that sites are found on the hanging wall, d) site classification was set to 1 for deep soil sites. For the Boore, Joyner and Fumal (1997) relation, the following parameters were used in the regression equation: a) R is the closest horizontal distance (km) to a vertical projection of fault rupture surface (km), b) Vs in the upper 30 meters was set to 270 m/s, which is the mid range for a medium stiff soil (site class D), c) fault type was set to "fault mechanism not specified." For the Campbell (1997) relation, the following parameters were used in the regression equation: a) R is the closest distance to the seismogenic rupture surface (km), b) fault style factor was set to "otherwise", c) soft rock and hard rock site factors were set to "otherwise", which implies a stiff soil site.

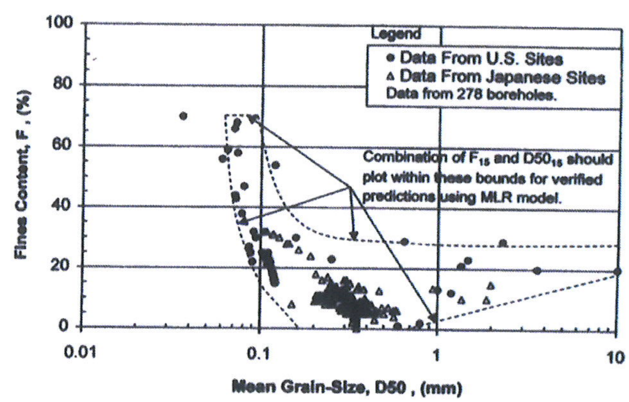
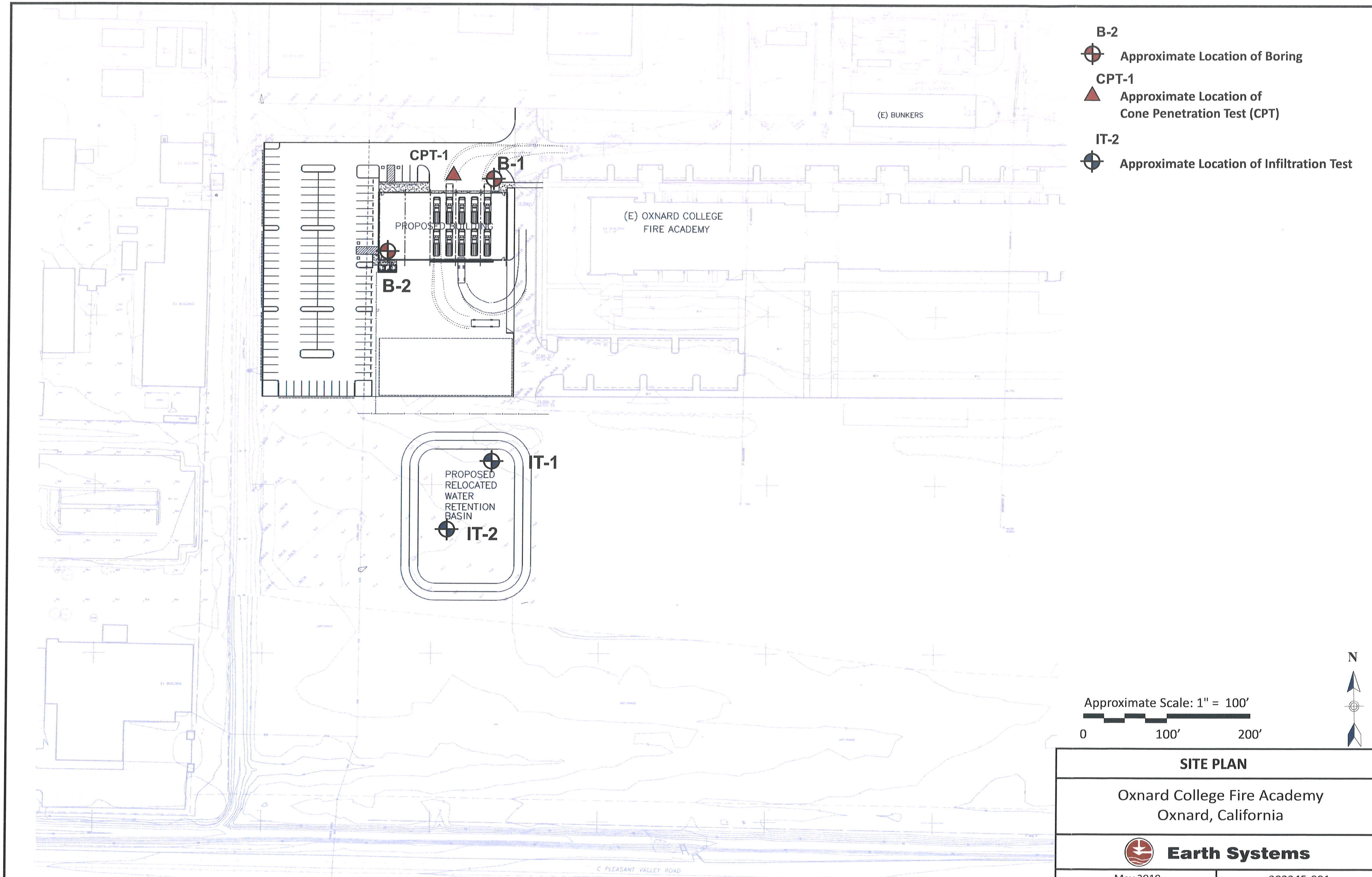


Fig. 5. Compiled grain-size data with ranges of F_{15} and $D50_{15}$ [for which Eq. (6) is applicable]



SITE PLAN

Oxnard College Fire Academy
Oxnard, California

Earth Systems

May 2019 | 302245-001

INFILTRATION RATE BY THE BOREHOLE PERCOLATION TEST METHOD

This workbook calculates an adjusted infiltration rate from a borehole percolation test. The percolation rate is adjusted for sidewall area according to the Porchet method, and then re-adjusted for the effect of the gravel placed in annulus between the borehole wall and a pipe placed in the borehole by a method presented in Caltrans Test 750.

Project Name	Oxnard College Fire Academy
Project Number	302245-001
Test Hole No.	IT-2
Tester	Scott Calvert
Pre-Soak Date	3/28/2019
Test Date	3/29/2019

Test Hole Radius, r (inches)	2
Total Depth of Test Hole, D_T (feet)	3.0
Inside Diameter of Pipe, I (inches)	2.00
Outside Diameter of Pipe, O (inches)	2.38
Pipe Stick-Up (feet)	0.0
Porosity of Gravel, n	0.41
Porosity Correction Factor, C	0.51
Factor of Safety (FOS), F	N/A

Interval No.	Delta Time, Δt (min.)	Initial Depth to Water from TOP, D_o (in.)	Final Depth to Water from TOP, D_f (in.)	Initial Water Height, H_o (in.)	Final Water Height, H_f (in.)	Change in Water Height, ΔH (in.)	Perc Rate, (in/hr)	Infiltration Rate (in./hr.)	Corrected Infiltration Rate (in/hr)
1	30.00	1.10	1.15	1.90	1.85	0.05	0.10	0.03	0.02
2	30.00	1.15	1.22	1.85	1.78	0.07	0.14	0.05	0.03
3	30.00	1.22	1.28	1.78	1.72	0.06	0.12	0.04	0.02
4	30.00	1.28	1.32	1.72	1.68	0.04	0.08	0.03	0.02
5	30.00	1.32	1.35	1.68	1.65	0.03	0.06	0.02	0.01
6	30.00	1.35	1.40	1.65	1.60	0.05	0.10	0.04	0.02
7	30.00	1.40	1.45	1.60	1.55	0.05	0.10	0.04	0.02
8	30.00	1.10	1.14	1.90	1.86	0.04	0.08	0.03	0.01
9									
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18									
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23									
24									
25									

INFILTRATION RATE BY THE BOREHOLE PERCOLATION TEST METHOD

This workbook calculates an adjusted infiltration rate from a borehole percolation test. The percolation rate is adjusted for sidewall area according to the Porchet method, and then re-adjusted for the effect of the gravel placed in annulus between the borehole wall and a pipe placed in the borehole by a method presented in Caltrans Test 750.

Project Name	Oxnard College Fire Academy
Project Number	302245-001
Test Hole No.	IT-1
Tester	Scott Calvert
Pre-Soak Date	3/28/2019
Test Date	3/29/2019

Test Hole Radius, r (inches)	2
Total Depth of Test Hole, D_T (feet)	3.0
Inside Diameter of Pipe, I (inches)	2.00
Outside Diameter of Pipe, O (inches)	2.38
Pipe Stick-Up (feet)	0.0
Porosity of Gravel, n	0.41
Porosity Correction Factor, C	0.51
Factor of Safety (FOS), F	N/A

Interval No.	Delta Time, Δt (min.)	Initial Depth to Water from TOP, D_o (ft.)	Final Depth to Water from TOP, D_f (ft.)	Initial Water Height, H_o (in.)	Final Water Height, H_f (in.)	Change in Water Height, ΔH (in.)	Perc Rate, (in/hr)	Infiltration Rate (in./hr.)	Corrected Infiltration Rate (in/hr)
1	30.00	0.92	0.98	24.96	24.24	0.72	1.44	0.06	0.03
2	30.00	0.98	1.02	24.24	23.76	0.48	0.96	0.04	0.02
3	30.00	1.02	1.08	23.76	23.04	0.72	1.44	0.06	0.03
4	30.00	1.08	1.13	23.04	22.44	0.60	1.20	0.05	0.03
5	30.00	1.13	1.18	22.44	21.84	0.60	1.20	0.05	0.03
6	30.00	1.18	1.22	21.84	21.36	0.48	0.96	0.04	0.02
7	30.00	1.22	1.26	21.36	20.88	0.48	0.96	0.04	0.02
8	30.00	0.92	0.96	24.96	24.48	0.48	0.96	0.04	0.02
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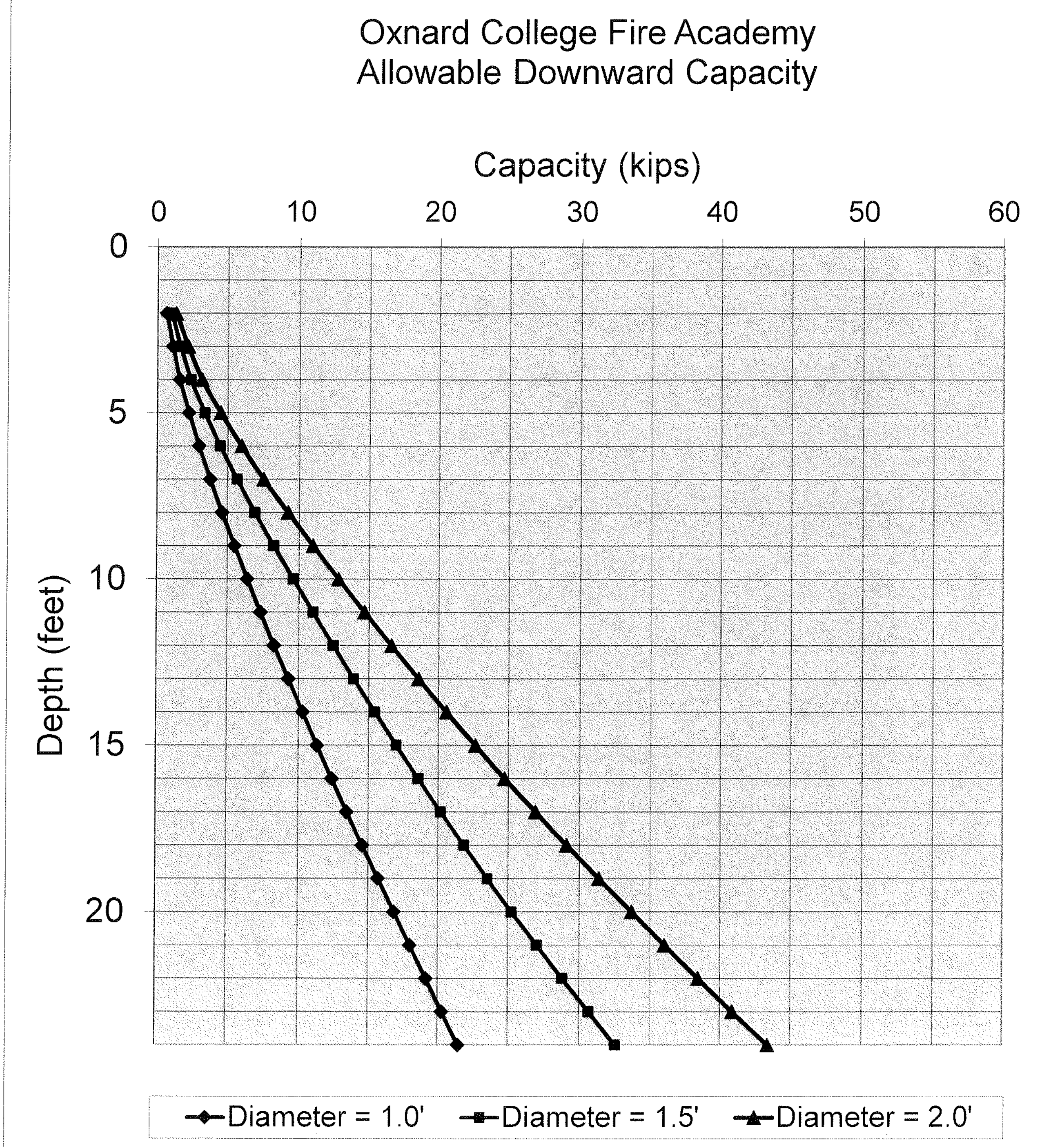
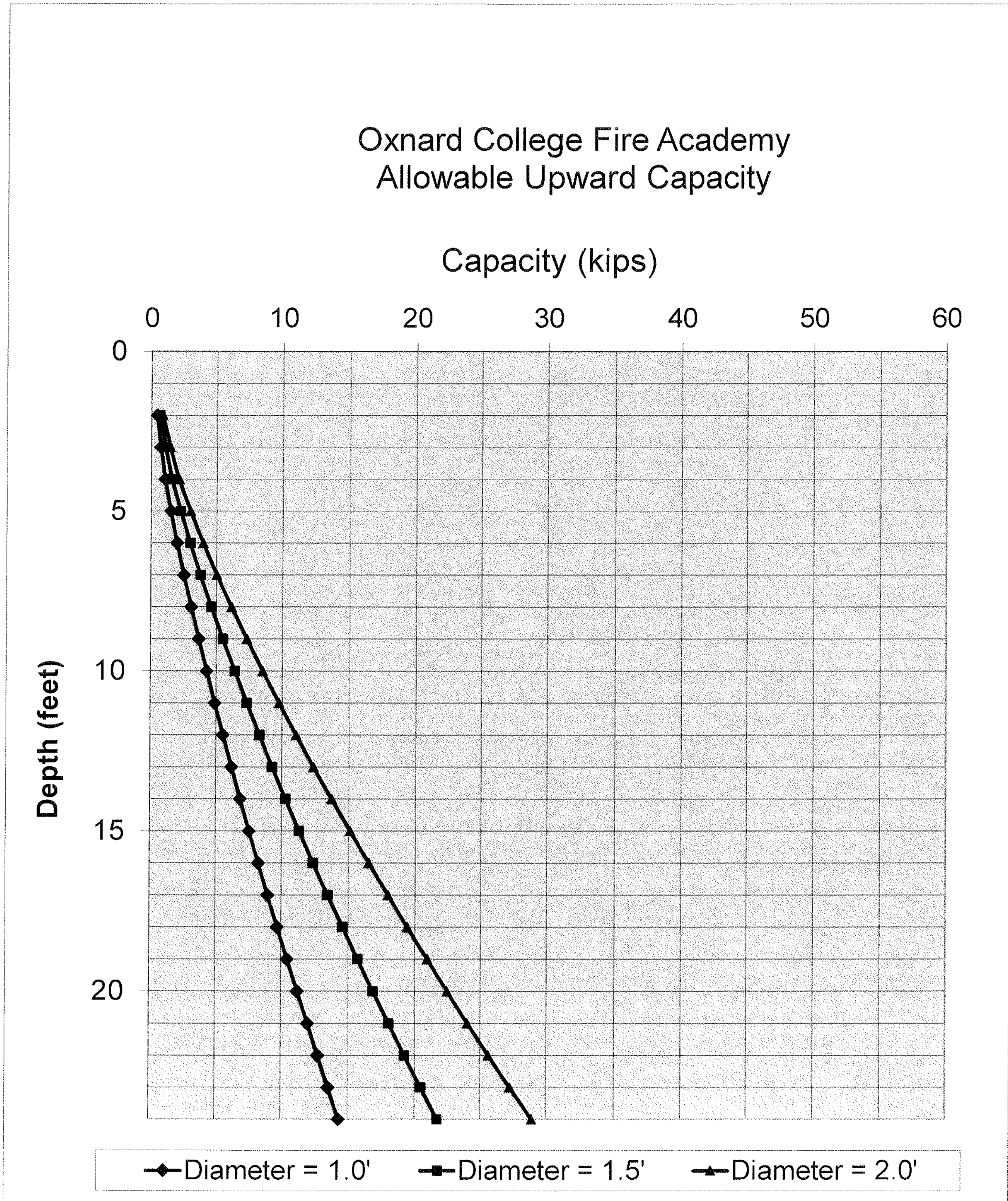
APPENDIX E

Pile Capacity Graphs



Oxnard College Fire Academy
Allowable Downward Capacity
Capacity (kips)
0
10
20
30
40
50
60
Depth (feet)
0
5
10
15
20
Diameter = 1.0'
Diameter = 1.5'
Diameter = 2.0'


Oxnard College Fire Academy
Allowable Upward Capacity
Capacity (kips)
0
10
20
30
40
50
60
0
5
10
15
20
Depth (feet)
Diameter = 1.0'
Diameter = 1.5'
Diameter = 2.0'