GEOTECHNICAL ENGINEERING REPORT
FOR PROPOSED
TICKET BOOTHS AND GATEWAYS
TO STADIUM COMPLEX AT
CHANNEL ISLANDS HIGH SCHOOL,
1400 RAIDERS WAY,
OXNARD, CALIFORNIA

PROJECT NO.: 303514-002
NOVEMBER 26, 2019

PREPARED FOR
OXNARD UNION HIGH SCHOOL DISTRICT

BY
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November 26, 2019

Project No.: 303514-002
Report No.: 19-11-68

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Project: Ticket Booths and Gateways to Stadium Complex
Channel Islands High School
1400 Raiders Way
Oxnard, California

As authorized, we have performed geotechnical studies for proposed ticket booths and gateways to the stadium complex at Channel Islands High School in the City of Oxnard, California. The accompanying Geotechnical Engineering Report presents the results of our subsurface exploration and laboratory testing programs, as well as our conclusions and recommendations pertaining to geotechnical aspects of project design. This report completes the scope of services described within our Proposal No. VEN-19-09-004 dated September 5, 2019, and authorized by Purchase Order A20-01436 on October 22, 2019.

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,

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INTRODUCTION

This report presents results of a geotechnical engineering study performed for three structures that will serve as ticket booths and gateways to the athletic field complex at Channel Islands High School in the City of Oxnard, California (see Vicinity Map in Appendix A). Current plans indicate that the ticket booths will range from 50 to 70 square feet and will have attached 10-foot tall entry gates supported by steel tube columns on pier footings. The one-story ticket booths will be constructed with reinforced CMU block, and will utilize conventional foundation systems with slab-on-grade floors. There will be 8-foot high freestanding reinforced CMU walls adjacent to the ticket booths at the entry gates.

Structural considerations for building column loads of up to 10 kips with maximum wall loads of 1.5 kips per lineal foot were used as a basis for the recommendations of this report. If actual loads vary significantly from these assumed loads, Earth Systems should be notified since reevaluation of the recommendations contained in this report may be required.

The site of each proposed structure is currently essentially level. As a result, grading for the proposed project is expected to be limited to preparing near-surface soils to support the new loads.

PURPOSE AND SCOPE OF WORK

The purpose of the geotechnical study that led to this report was to supplement previous geotechnical studies done for currently proposed improvements to the athletic field complex at the high school by focusing on evaluating the hazards posed by liquefaction and related phenomenon. The scope of work included:

1. Performing a reconnaissance of the site.
2. Reviewing geotechnical data presented in previous campus-specific geotechnical reports generated by Earth Systems in 2009, 2010, and 2019.
3. Drilling, sampling, and logging two additional mud rotary borings to study soil and groundwater conditions.
4. Laboratory testing soil samples obtained from the new subsurface exploration to determine physical and engineering properties.
5. Consulting with owner representatives and design professionals.

6. Analyzing the geotechnical data obtained.
7. Preparing this report.

Contained in this report are:

1. Descriptions and results of field and laboratory tests that were performed.
2. Conclusions and recommendations pertaining to site grading and structural design.

GENERAL GEOLOGY

The site lies within the Oxnard Plain, which in turn lies within the western Transverse Ranges geomorphic province. The Oxnard Plain and the Transverse Ranges are characterized by ongoing tectonic activity. In the vicinity of the subject site, Tertiary and Quaternary sediments have been folded and faulted along predominant east-west structural trends.

Although there are several faults located within the region, the nearest known surface fault trace of significant activity, the Simi-Santa Rosa Fault, is located approximately 4.9 miles northeast of the subject site. (For the purposes of the site-specific seismic analysis and the liquefaction evaluation, it has been assumed that the fault plane of the Oak Ridge Fault projects downward toward the site at depth, and that the potential earthquake could happen 2 miles from the campus.) The project area is not located within any of the “Fault Rupture Hazard Zones” that have been specified by the State of California (CDMG. 1972, Revised 1999).

The site is underlain by deltaic (alluvial) sediments consisting of loose to very dense silty and clayey sands, fine to coarse sands, and soft to firm sandy to silty clays.

The site is within one of the Liquefaction Hazard Zones designated by the California Geological Survey (CGS, 2002).

No 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.

SEISMICITY AND SEISMIC DESIGN

Although the site is not within a State-designated “fault rupture hazard zone”, it is located in an active seismic region where large numbers of earthquakes are recorded each year. Historically, major earthquakes felt in the vicinity of the subject site have originated from faults outside the area. These include the December 21, 1812 "Santa Barbara Region" earthquake, that was presumably centered in the Santa Barbara Channel, the 1857 Fort Tejon earthquake, the 1872 Owens Valley earthquake, and the 1952 Arvin-Tehachapi earthquake.

Southern Ventura County was mapped by the California Division of Mines and Geology in 1975 to delineate areas of varying predicted seismic response. The deltaic (alluvial) deposits that underlie the campus are mapped as having a probable maximum intensity of earthquake response of approximately IX on the Modified Mercalli Scale. Historically, the highest observed intensity of ground response has been VII in the Oxnard area (C.D.M.G., 1975).

For school projects, the 2016 California Building Code (CBC) specifies that peak ground acceleration for design purposes can be determined from a site-specific study taking into account soil amplification effects. The United States Geological Survey (USGS, 2009) has undertaken a probabilistic earthquake analyses that covers the continental United States. A reasonable site-specific spectral response curve may be developed from USGS Unified Hazard Tool web page, which adjusts for site-specific ground factors. The interactive webpage appears to be a precise calculation based on site coordinates. The program incorporates the 2008 USGS/CGS working group consensus methodologies, and the output for base ground motion is a smooth curve based on seven spectral ordinates ranging from 0 to 2 seconds. The USGS interactive deaggregation spectral values are generally within about 5% of the precise site-specific values obtained from other programs such as OpenSHA or EZ-FRISK for the same model and attenuation relationships.

The NGA (Next Generation Attenuation) relationships for spectral response have been used in the analyses. A principal advantage in the NGA relationships is that the estimated site-specific soil velocity (Vs30) is used directly for site specific analysis rather than the NEHRP site corrections. The analysis also includes amplification factors (Idriss, 1993) to model the maximum rotated component of the ground motion.

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 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 2013 and 2016 California Building Code response spectrums.

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 (34.1691˚ North Latitude and -119.1632° West Longitude). The calculator adjusts for Soil Site Class E, and for Occupancy (Risk) Category III (for public school structures). (A listing of the calculated 2016 CBC and ASCE 7-10 Seismic Parameters is presented below and in Appendix C.)

Summary of Seismic Parameters – 2016 CBC

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	2.333 g
Spectral Response Acceleration at 1 sec. – S_1	0.828 g
Site Coefficient – F_a	0.90
Site Coefficient – F_v	2.40
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.100 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.987 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.400g
One Second Spectral Response – S_{D1}	1.325 g
Site Modified Peak Ground Acceleration - PGA_M	0.794 g
Values appropriate for a 2% probability of exceedance in 50 years	

Because the Seismic Design Category is "E", a site-specific seismic analysis must be performed in addition to the "general procedure". For the purposes of the site-specific evaluation, it has been assumed that the fault plane of the Oak Ridge Fault projects downward toward the site at depth, and that the potential earthquake could happen within 2 miles of the campus. For the Site-Specific Analysis, the Short Period Spectral Response (S_{DS}) was found to be 1.120 g, and the 1 Second Spectral Response (S_{D1}) was found to be 1.199 g. Both the "site specific" and "general procedure yielded peak ground accelerations of 0.794 g.

The Fault Parameters table in Appendix C lists the significant “active” and “potentially active” faults within a radius of about 36 miles from the subject site. The distance between the site and the nearest portion of each fault is shown, as well as the respective estimated maximum earthquake magnitudes, and the deterministic mean site peak ground accelerations.

SOIL CONDITIONS

Evaluation of the subsurface indicates that soils are generally alluvial sands, silty sands, clayey silts, and silty clays. Near-surface soils encountered are generally characterized by low blow counts and in-place densities, and relatively high compressibilities. Testing indicates that anticipated bearing soils lie in the “very low” expansion range because the expansion index equals 3. [A version of this classification of soil expansion, Table 18-I-D, is included in Appendix B of this report.] It appears that soils can be cut by normal grading equipment.

Groundwater was encountered in the test borings at depths ranging from approximately 9.5 to 11.5 feet below existing site grades. Mapping of historically high groundwater levels by the California Geological Survey (CGS, 2002a) indicates that groundwater has been 5 below the ground surface near the subject site.

As mentioned previously, the campus is within one of the Liquefaction Hazard Zones designated by the California Geological Survey (CGS, 2002).

Samples of near-surface soils were 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 sulfate contents (510 mg/Kg) are in the “S0” (“negligible”) exposure class of Table 19.3.1.1 of ACI 318-14; therefore, it appears that special concrete designs will not be necessary for the measured sulfate contents.

Based on criteria established by the County of Los Angeles (2013), measurements of resistivity of near-surface soils (2,200 ohms-cm) indicate that they are “moderately corrosive” to ferrous metal (i.e. cast iron, etc.) pipes.

ANALYSIS OF LIQUEFACTION POTENTIAL

As mentioned previously, the campus is located within one of the Liquefaction Hazard Zones designated by CGS (2002b).

Earthquake-induced vibrations 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. Liquefaction is typically limited to the upper 50 feet of soils underlying a site.

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, sufficiently dense soils, soils that have plasticity indices greater than 7, 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:

Groundwater was encountered in the test borings at depths ranging from approximately 9.5 to 11.5 feet below existing site grades, and historically high groundwater levels have been about 5 below the ground surface near the subject site.

The proposed gateways will be located near the northwest, northeast, and southeast corners of the football field/track complex. Cyclic mobility analyses were performed to analyze the liquefaction potentials of the various soil layers at each gateway location. The analyses were performed in general accordance with the methods proposed by NCEER (1997).

The surface trace of the Simi-Santa Rosa Fault is the nearest to the campus, and the surface trace of the Oak Ridge Fault is approximately 6 miles north of the site. Because the Oak Ridge Fault is a south-dipping reverse fault, for the purposes of the liquefaction study it has been assumed that the fault plane is two miles from the site after projecting downward and southward from the surface trace. The analyses used the calculated site-modified peak ground acceleration of 0.794 g, as per the discussion in the “Seismicity and Seismic Design” section of this report.

Northwest Gateway Analysis

Exploration that was performed at the northwest entry between the campus and the stadium complex included Boring B-1 from the athletic field studies of 2019 and a new boring (Boring B-7) advanced to a depth of 51.5 feet. (The boring was terminated at that depth due to sands plugging the annulus.) Data from these two borings indicates that geotechnical conditions in this area include:

1. Soils are generally alluvial interbeds of sands with minor interbeds of sands, silty sands, clayey silts, and fine to silty clays.
2. Groundwater was encountered at a depth of 10 feet in Boring B-7, but historically shallowest groundwater has been at a depth of about 5 feet.
3. Atterberg limit evaluations indicate that the finer grained soils at depths of 7, 24.5, 32, and 46.5 feet below the ground surface have plasticity indices (PIs) ranging from 10 to 24 and classify as clays (CL). (PI and hydrometer test results are presented in Appendix B.) These soil horizons in Boring B-7 would be expected to exhibit clay-like behavior during earthquake cyclic loading and would not be expected to be susceptible to liquefaction.
4. Standard penetration tests conducted in the borings indicate that soils within the tested depth are in a variably dense state.

Two analyses were performed: one assuming groundwater at a depth of 5 feet, and another assuming groundwater at a depth of 10 feet. The analysis assuming groundwater at 5 feet indicated that a soil layer between the depths of 5 and 7 feet and 22 and 24.5 feet had a factor of safety below 1.3 (see Appendix D for calculations). The layer between the depths of 22 and 24.5 feet was found to have a factor of safety below 1.3 when groundwater was assumed to be at a depth of 10 feet. Those 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 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 approximately 0.6 inches when groundwater was assumed to be at 5 feet, and 0.4 inches when groundwater was assumed to be at 10 feet.

There is also a potential for differential areal settlement suggested by the findings. 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 0.3 inches at the ground surface.

Because the potentially liquefiable zone is only 2 feet thick and is below 5 feet of non-liquefiable soils, “ground” damage could potentially occur if this zone was to liquefy. (Examples of ground damage are sand boils and ground cracks.) Some additional seismic-induced settlement may result from the volume of soil removed as a result of a volume of soil being ejected to the ground surface from sand boils.

To evaluate the potential for a bearing capacity failure, Earth Systems used the residual undrained shear strength of the liquefiable soil between the depths of about 5 and 7 feet below the ground surface. The residual undrained shear strength of the liquefiable soil was estimated using the equivalent clean sand SPT blow count $(N_1)_{60\text{-CS}}$ within this liquefiable zone and the lower bound of the Seed & Harder (1990) plot. The $(N_1)_{60\text{-CS}}$ for the liquefiable soil between the depths of about 5 and 7 feet is 23.1. Using the lower bound of the Seed & Harder (1990) plot and a $(N_1)_{60\text{-CS}}$ of 17.8, the residual undrained shear strength of this upper liquefiable zone is about 1,200 psf.

Based on a recommended bearing pressure of 1,500 psf for continuous foundations, the stress at the top of the liquefiable zone at a depth of 5 feet below the ground surface for a 15-inch wide continuous footing is 240 psf. Based on a recommended bearing pressure of 1,700 psf for isolated pad foundations, the stress at the top of the liquefiable zone at a depth of 5 feet below the ground surface for a 2-foot wide pad footing is 119 psf. Given the residual undrained shear strength of the liquefiable zone between 5 and 7 feet below the ground surface and the stress that will be imposed to the top of this layer, a bearing capacity failure is not anticipated to occur from structural loading.

”Free-face” lateral spreading does not appear to pose a potential hazard because there are no nearby sloped areas or canyons (Bartlett and Youd, 1995). “Ground slope” lateral spreading, sometimes referred to as “ground oscillation”, can occur when adjusted blow counts $(N_{1(60)})$ measured within potentially liquefiable zones are less than 15. The calculated $N_{1(60)}$ value is 18.1 for the potentially liquefiable layer between depths of 5 and 7 feet and 26.6 for the potentially liquefiable layer between depths of 22 and 24.5 feet. As a result, it does not appear that this area of the site is susceptible to lateral spreading.

Based on the measured liquidity indices, the majority of the clay layers encountered in Boring B-7 have sensitivities of about 6, and do not appear to be susceptible to significant strength loss and post-liquefaction consolidation. There is a clay layer of low plasticity at depths between 46.5 and 51.5 feet that has a liquidity index of about 0.74 and a sensitivity of about 8.5. This layer is only a few feet thick, and by itself, cannot lead to much post-liquefaction consolidation. Therefore, 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 liquefaction exists at the northeast gateway site.

Southeast Gateway Analysis

Exploration that was performed at the southeast entry to the stadium complex from Gary Drive included Boring B-4 from the athletic field studies of August 2019 and a new mud rotary boring (B-6) advanced to a depth of 46.5 feet. Data from those borings indicates that conditions in this area:

1. Soils are generally alluvial interbeds of sands with minor interbeds of sands, silty sands, clayey silts, and fine to silty clays.
2. Groundwater was encountered at a depth of 10 feet in Boring B-6, but historically shallowest groundwater has been at a depth of about 5 feet.
3. An Atterberg limit evaluation indicate that the finer grained soils at a depth of 25 feet below the ground surface have a plasticity index (PI) of 2 and classify as a silty sand (SM). No other soils within the upper 46.5 feet of the soil profile. (PI and hydrometer test results are presented in Appendix B.) None of the soils in Boring B-7 would be expected to exhibit clay-like behavior during earthquake cyclic loading, and all soils in the boring require further evaluation with respect to liquefaction.
4. Standard penetration tests conducted in the borings indicate that soils within the tested depth are in a variably dense state.

Two analyses were performed: one assuming groundwater at a depth of 5 feet, and another assuming groundwater at a depth of 10 feet. The analysis assuming groundwater at 5 feet indicated that soil layers between depths of 5 and 10 feet and 24.5 and 27 feet had factors of safety below 1.3 (see Appendix D for calculations). When groundwater was assumed to be at 10 feet, layers between 10 and 14.5 feet, and between 24.5 and 27 feet were found to have factors of safety below 1.3. Those 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 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 approximately 1.8 inches when groundwater was at 5 feet, and 1.5 inches when groundwater was at 10 feet.

There is also a potential for differential areal settlement suggested by the findings. 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 0.9 inches at the ground surface when the worst case is assumed.

The top of the shallowest potentially liquefiable zone is at a depth of 5 feet below the ground surface and extends down to a depth of 10 feet. The SPT blow count measured in Boring B-6 (2019) in the shallowest potentially liquefiable zone was 9. According to data generated by Ishihara (National Academy Press, 1985), "ground" damage could potentially occur if this zone was to liquefy. (Examples of ground damage are sand boils and ground cracks.) Some additional seismic-induced settlement may result from the volume of soil removed as a result of a volume of soil being ejected to the ground surface from sand boils.

To evaluate the potential for a bearing capacity failure, Earth Systems used the residual undrained shear strength of the liquefiable soil between the depths of about 5 and 10 feet below the ground surface. The residual undrained shear strength of the liquefiable soil was estimated using the equivalent clean sand SPT blow count $(N_1)_{60\text{-}CS}$ within this liquefiable zone and the lower bound of the Seed & Harder (1990) plot. The $(N_1)_{60\text{-}CS}$ for the liquefiable soil between the depths of about 5 and 10 feet is 17.8. Using the lower bound of the Seed & Harder (1990) plot and a $(N_1)_{60\text{-}CS}$ of 17.8, the residual undrained shear strength of this upper liquefiable zone is about 683 psf.

Based on a recommended bearing pressure of 1,500 psf for continuous foundations, the stress at the top of the liquefiable zone at a depth of 5 feet below the ground surface for a 15-inch wide continuous footing is 240 psf. Based on a recommended bearing pressure of 1,700 psf for isolated pad foundations, the stress at the top of the liquefiable zone at a depth of 5 feet below the ground surface for a 2-foot wide pad footing is 119 psf. Given the residual undrained shear strength of the liquefiable zone between 5 and 10 feet below the ground surface and the stress that will be imposed to the top of this layer, a bearing capacity failure is not anticipated to occur from structural loading.

”Free-face” lateral spreading does not appear to pose a potential hazard because there are no nearby sloped areas or canyons (Bartlett and Youd, 1995). However, “ground slope” lateral spreading, sometimes referred to as “ground oscillation”, can occur when adjusted blow counts ($N_{1(60)}$) measured within potentially liquefiable zones are less than 15. Although the shallowest of the potentially liquefiable zones have $N_{1(60)}$ values greater than 15, the value for the zone between 24.5 and 27 feet is less than 15. The thickness of this layer is about 0.77 meters. 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 0.25%, which is equivalent to about 5 feet of fall in 2,000 feet, as shown on the Oxnard Quadrangle near the subject site. Fine contents were assumed to be 43% based on hydrometer testing performed on a sample gathered from that layer. The cumulative displacement was calculated to be about 1.3 feet if the entire zone liquefied. (Calculations are included within Appendix D of this report.)

Because none of the soils exhibit significant plasticity, they are not considered to be sensitive. Hence, strength loss and post-liquefaction consolidation are not thought to be significant concerns for the southeast gateway.

Based on the above, it is the opinion of this firm that a potential for liquefaction and lateral spreading exists at the southwest gateway site.

<u>Northeast Gateway Analysis</u>

Exploration that was performed near the northeast entry from the campus included Boring B-3 and CPT-2 from studies performed in 2010 for an addition to the auto shop, which is immediately adjacent to the proposed northeast gateway. Data from those points of exploration indicate that conditions in this area include:

1. Soils are generally alluvial interbeds of sands with minor interbeds of sands, silty sands, clayey silts, and fine to silty clays.
2. Groundwater was encountered at a depth of 10 feet in Boring B-6, but historically shallowest groundwater has been at a depth of about 5 feet.
3. Interpretations of the CPT data indicate that the upper 50 feet of the soil profile includes several layers with I_c values greater than 2.6. Those layers are located at depths between 11.5 and 12 feet, 25 and 28 feet, 33 and 35 feet, and 47.5 and 50 feet. Soils with I_c values greater than 2.6 are not considered prone to liquefaction (see CPT Interpretations in Appendix A).
3. Atterberg limit evaluations include a Plasticity Index (PI) of 6 between 25 and 28 feet, and greater than 7 between 33 and 35 feet, and between 47.5 and 50 feet. (PI and hydrometer test results are presented in Appendix B.) Soils with PIs greater than 7 would be expected to exhibit clay-like behavior during earthquake cyclic loading. Although the PI is only 6 in the layer between 25 and 28 feet, because it has an I_c value greater than 2.6, it was also considered non-liquefiable.
4. Standard penetration tests conducted in the borings indicate that soils within the tested depth are in a variably dense state.

Two analyses were performed: one assuming groundwater at a depth of 5 feet, and another assuming groundwater at a depth of 8.5 feet. The analysis assuming groundwater at 5 feet indicated that a soil layer between the depths of 5 and 15 feet had a factor of safety below 1.3 (see Appendix D for calculations). The layer between the depths of 8.5 and 15 feet was found to have a factor of safety below 1.3 when groundwater was assumed to be at a depth of 8.5 feet. Those 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 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 approximately 1.9 inches when groundwater was at 5 feet, and 1.4 inches when groundwater was at 8.5 feet.

There is also a potential for differential areal settlement suggested by the findings. 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 1 inch at the ground surface when the worst case is assumed.

The top of the shallowest potentially liquefiable zone is at a depth of 5 feet below the ground surface and extends down to a depth of 15 feet. According to data generated by Ishihara (National Academy Press, 1985), “ground” damage could potentially occur if this zone was to liquefy. (Examples of ground damage are sand boils and ground cracks.) Some additional seismic-induced settlement may result from the volume of soil removed as a result of a volume of soil being ejected to the ground surface from sand boils.

To evaluate the potential for a bearing capacity failure, Earth Systems used the residual undrained shear strength of the liquefiable soil between the depths of about 5 and 15 feet below the ground surface. The residual undrained shear strength of the liquefiable soil was estimated using the equivalent clean sand SPT blow count $(N_1)_{60\text{-CS}}$ within this liquefiable zone and the lower bound of the Seed & Harder (1990) plot. The $(N_1)_{60\text{-CS}}$ for the liquefiable soil between the depths of about 5 and 15 feet is 18.3. Using the lower bound of the Seed & Harder (1990) plot and a $(N_1)_{60\text{-CS}}$ of 18.3, the residual undrained shear strength of this upper liquefiable zone is about 800 psf.

Based on a recommended bearing pressure of 1,500 psf for continuous foundations, the stress at the top of the liquefiable zone at a depth of 5 feet below the ground surface for a 15-inch wide continuous footing is 240 psf. Based on a recommended bearing pressure of 1,700 psf for isolated pad foundations, the stress at the top of the liquefiable zone at a depth of 5 feet below the ground surface for a 2-foot wide pad footing is 119 psf. Given the residual undrained shear strength of the liquefiable zone between 5 and 10 feet below the ground surface and the stress that will be imposed to the top of this layer, a bearing capacity failure is not anticipated to occur from structural loading.

”Free face” lateral spreading does not appear to pose a potential hazard because there are no nearby sloped areas or canyons (Bartlett and Youd, 1995). However, “ground slope” lateral spreading, sometimes referred to as “ground oscillation”, can occur when adjusted blow counts $(N_{1(60)})$ measured within potentially liquefiable zones are less than 15. The calculated $N_{1(60)}$ value is 18.3 for the potentially liquefiable layer between depths of 8.5 and 15 feet. As a result, it does not appear that this area of the site is susceptible to lateral spreading.

Based on the measured liquidity indices, the majority of the clay layers at the site do not appear to be sensitive. Hence, strength loss and post-liquefaction consolidation are not thought to be significant concerns. In-place moisture contents were not measured within the finer grained units; thus, liquidity indices cannot be calculated. However, even if cyclic softening of clays and post-liquefaction settlement from consolidation of clays disturbed by a design level earthquake could occur, measures provided elsewhere in this report for mitigation of liquefaction related effects will also mitigate settlement related to sensitive clays.

Based on the above, it is the opinion of this firm that a potential for liquefaction exists at the southeast gateway site.

CONCLUSIONS AND RECOMMENDATIONS

The site is suitable for the proposed development from a Geotechnical Engineering standpoint provided that the recommendations contained in this report are successfully implemented into the project. The grading recommendations provided herein should supersede those presented in the referenced Geotechnical Engineering Report dated August 27, 2019.

GRADING RECOMMENDATIONS FOR TICKET BOOTHS AND ENTRY GATES

Grading at a minimum should conform to the 2016 California Building Code, 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.

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.

To mitigate the anticipated liquefaction-related effects, Earth Systems recommends that a geogrid reinforced mat be constructed beneath the proposed structures (bathroom building, ticket booth, gateway walls). The intent of the geogrid reinforced mat is to stiffen the soils underlying and outside of the structure so that they act as a block that would move as a unit. The geogrid reinforced mat will mitigate the potential for lateral displacements and ground damage by providing a 5-foot thick mat of geogrid reinforced aggregate and compacted engineered fill beneath the structure, and will reduce the differential settlement by providing a more uniform settlement to occur beneath the structures.

To create the geogrid reinforced aggregate mat, native soils beneath the proposed buildings should be excavated a minimum of 5 feet below existing grade. The limits of overexcavation should be also extended laterally to a distance of at least 5 feet beyond the outside edges of the foundation systems. Where adjacent structures are within 10 feet, the overexcavation width could be reduced to 3 feet outside the building perimeter in that direction only. The bases of the overexcavations should be at relatively level elevations.

The bottoms of the remedial excavations should be scarified to depths of 6 inches, uniformly moisture conditioned to above optimum moisture content; and compacted to achieve a relative compaction of at least 90 percent of the ASTM D 1557 maximum dry density. Following compaction of each bottom, a layer of geogrid should be placed on the prepared subgrade that extends across the entire area of overexcavation and up the sidewalls of the remedial excavation. The reinforcing geogrids should consist of Tensar Tri-Axial TX190, or equivalent as approved by the Geotechnical Engineer. The bottom layers or sheets of geogrid should be overlapped at least 3 feet. A 1-foot layer of 1-inch minus aggregate base material should be placed and compacted over the bottom layer of geogrid. The aggregate base material should be uniformly moisture conditioned to at or above optimum moisture content and compacted to achieve a relative compaction of at least 95 percent of the ASTM D 1557 maximum dry density. A second layer of geogrid should be placed over the compacted aggregate base material. The second layer of geogrid should be overlapped 1-foot and extend across the entire excavation; however, it does not need to extend up the sidewalls. An additional foot of aggregate base material should be placed and compacted on top of the second geogrid layer. Once the second lift of aggregate base material has been compacted to achieve a minimum relative compaction of 95% of the ASTM D 1557 maximum dry density, the bottom layer of geogrid extending up the sidewall of the remedial excavation should be folded back onto the compacted surface to create an 8-foot overlap onto the compacted base material. The remedial excavation may then be brought up to finished grade using the excavated soil compacted to at least 95 percent of the ASTM D 1557 maximum dry density. The geogrid should be installed in accordance with the manufacturer's recommendations.

Overexcavation and recompaction of soils under and around pier footings and site walls near the entry gates will also be necessary. Soils should be overexcavated to a depth of 4.5 feet below finished subgrade elevation, and to a distance of 3 feet on either side of the footing edges. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted to at least 90% of the maximum dry density.

Areas outside of the building area to receive fill, exterior slabs-on-grade, sidewalks, or paving should be overexcavated to a depth of 1.5 feet below finished subgrade elevation. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted. Because the expansion index of on-site soils is in the “very low” range, no aggregate base will be required below sidewalks.

The bottoms of all excavations should be observed by a representative of this firm prior to processing or placing fill.

On-site soils may be used for fill once they are cleaned of all organic material, rock, debris, and irreducible material larger than 8 inches.

Fill and backfill should be placed at, or slightly above optimum moisture in layers with loose thickness not greater than 8 inches. Each layer should be compacted to a minimum of 90% of the maximum dry density obtainable by the ASTM D 1557 test method. The upper one foot of subgrade below areas to be paved should be compacted to a minimum of 95% of the maximum dry density.

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.

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 TX-160, or an approved 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 “3-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.

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% of the maximum dry density. Backfill of offsite service lines will be subject to the specifications of the approved project plans or this report, whichever are greater.

Utility trenches running parallel to footings should be located at least 5 feet outside the footing line, or above a 2:1 (horizontal to vertical) projection downward from a point 9 inches above the outside edge of the bottom of the footing.

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.

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

GEOTECHNICAL DESIGN PARAMETERS FOR BUILDINGS AND SITE WALLS

Conventional Spread Foundations

Conventional continuous footings and/or isolated pad footings may be used to support structures. It should be noted that if isolated pad footings are to be used, they must be restrained laterally in both directions by means of grade beams, structural slab, or other approved method.

For one-story buildings bearing in soils within the “very low” expansion range, perimeter and interior footings should have minimum depths of 12 inches.

Footings for the proposed structures should bear into the geogrid reinforced engineered fill pad prepared as recommended in the rough grading recommendations above. Foundation excavations should be observed by a representative of this firm after excavation, but prior to placing of reinforcing steel or concrete, to verify bearing conditions.

Conventional continuous footings that are 12 inches deep and a minimum of 15 inches wide may be designed based on an allowable bearing value of 1,500 psf. This value has a factor of safety of greater than 3.

Isolated pad footings that are 12 inches deep and a minimum of 2 feet wide may be designed based on an allowable bearing value of 1,700 psf. This value has a factor of safety of greater than 3.

Allowable bearing values are net (weight of footing and soil surcharge may be neglected) and are applicable for dead plus reasonable live loads.

A one-third increase is permitted for use with the alternative load combinations given in Section 1605.3.2 of the 2016 CBC.

Lateral loads may be resisted by soil friction on floor slabs and foundations and by passive resistance of the soils acting on foundation stem walls. Lateral capacity is based on the assumption that any required backfill adjacent to foundations and grade beams is properly compacted.

Resistance to lateral loading may be provided by friction acting on the base of foundations. A coefficient of friction of 0.62 may be applied to dead load forces. This value does not include a factor of safety.

Passive resistance acting on the sides of foundation stems equal to 390 pcf of equivalent fluid weight may be included for resistance to lateral load. This value does not include a factor of safety.

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

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

For retaining wall foundations, passive resistance may be combined with frictional resistance without reduction to the coefficient of friction.

Footing designs should be provided by the Structural Engineer, but the dimensions and reinforcement he recommends should not be less than the criteria set forth in Table 18-I-D for the “very low” expansion range.

Soils should be lightly moistened prior to placing concrete. Testing of premoistening is not required.

Drilled Pier Foundations for Entry Gates and Site Walls

A pier and grade-beam foundation system may be used to support the proposed entry gates and site walls. Foundation piers should be designed as friction piles. No allowance should be taken for end bearing.

Piers may consist of drilled, reinforced cast-in-place concrete caissons (cast-in-drilled-hole “CIDH” piles). Piers may be drilled or hand-dug. Steel reinforcing may consist of “rebar cages” or structural steel sections.

As a minimum, the new piers should be at least eighteen inches (18”) in diameter and embedded into compacted fill, firm native soil, or a combination of both. The geotechnical engineer should be consulted during pier installation to determine compliance with the geotechnical recommendations.

For vertical (axial compression) and uplift capacity, the attached pile capacity graphs may be used. For support of the proposed entry gates and site walls, drilled pier diameters of 1.5, 2.0, and 2.5 feet were analyzed. 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.

The strength loss in the liquefiable zones was taken into account in our analysis to represent the downward capacity during seismic loading conditions. For the reduced skin friction in the liquefiable zones, the residual undrained shear strength of the liquefiable soils was estimated using the equivalent clean sand SPT blow counts $(N_1)_{60\text{-CS}}$ for the liquefiable zones and the lower bound of the attached Seed & Harder (1990) plot.

The load capacities shown on the charts presented in Appendix E 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.

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

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.

Because the drilled piers for the proposed entry gates and site walls will most likely extend below a depth of 5 feet, downdrag loads will need to be considered in the design of the piers. For the southeast gateway, a negative skin friction value of 1.1 kips/foot should be used for drilled piers that extend into the non-liquefiable soils between the depths of 10 and 24 feet below the existing ground surface. For the northeast gateway, a negative skin friction value of 2.2 kips/foot should be used for drilled piers that extend into the non-liquefiable soils below a depth of 15 feet below the existing ground surface. For the northwest gateway, a negative skin friction value of 0.6 kips/foot should be used for drilled piers that extend into the non-liquefiable soils between the depths of 7 and 22 feet below the existing ground surface. The downdrag force to be carried by the drilled piers, in addition to the structural loads, can be determined by multiplying the circumference/perimeter of the drilled piers (in feet) by these negative skin friction values. 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 zone undergoing downdrag shown on the downward capacity graphs for drilled piers presented in Appendix E. For the southeast gateway, the upper 10 feet should be neglected. For the northeast and northwest gateways, the upper 15 and 7 feet, respectively, should be neglected

Lateral (horizontal) loads may be resisted by passive resistance of the soil against the piers. An equivalent fluid weight (EFW) of 390 psf per foot of penetration in the compacted fill (upper 5 feet) and an EFW of 300 pcf should be used in the the native alluvium for the portion of the drilled pier above the groundwater table. An EFW of 165 pcf may be used for lateral load design in the native soils below the groundwater table. These resisting pressures are ultimate values.

The maximum passive pressure used for design should not exceed 2,900 psf. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended).

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

Assuming 18-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 6.5 feet below the final ground elevation based on commonly accepted engineering procedures (Lee, 1968). If 24-inch diameter piers are used, the "point of fixity" was estimated to be located at least 8 feet below the final ground elevation. If 30-inch diameter piers are used, the "point of fixity" was estimated to be located at least 9.5 feet below the final ground elevation.

The geotechnical engineer, or their representative, should be present during excavation and installation of all piers to observe subsurface conditions, and to document penetration into load supporting materials (i.e. either compacted fill or firm native soil).

Due to the presence of relatively shallow groundwater and "clean" sands, temporary casing may be necessary to minimize borehole 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.

Since the piers are designed to rely completely on intimate frictional contact with the soil, any casing (if used) should be removed during placement of concrete. The bottoms of pier excavations should be relatively clean of loose soils and debris prior to placement of concrete.

Installed piers should not be more than two percent (2%) from the plumb position.

Pier footings to support fence posts that are drilled into native soils may be designed for passive pressures of 100 psf per foot below natural grade. This value is based on presumptive parameters provided in the California Building Code for clay soils.

Slabs-on-Grade

Concrete slabs should be supported by compacted structural fill as recommended elsewhere in this report.

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. Current plans call for 4-inch thick concrete reinforced with No. 3 bars on 18-inch centers. These specifications are considered appropriate for the soil conditions.

Slab designs should be provided by the Structural Engineer, but the reinforcement and slab thicknesses should not be less than the criteria set forth in Table 18-I-D for the “very low” expansion range.

Areas where floor wetness would be undesirable should be underlaid with a vapor retarder (as specified by the Project Architect or Civil Engineer) to reduce moisture transmission from the subgrade soils to the slab. The retarder should be placed as specified by the structural designer.

Soils should be lightly moistened prior to placing concrete. Testing of premoistening is not required.

Retaining Walls

Conventional cantilever retaining walls backfilled with compacted on-site soils may be designed for active pressures of 38 pcf of equivalent fluid weight for well-drained, level backfill.

Restrained retaining walls backfilled with compacted on-site soils may be designed for at-rest pressures of 58 pcf of equivalent fluid weight for well-drained, level backfill.

These pressures are based on the assumption that backfill soils will be compacted to 90% of the maximum dry density determined by the ASTM D 1557 Test Method.

Conventional spread foundations for retaining walls should be designed per the recommendations provided in this report.

Because walls will not retain more than 6 feet, seismic forces do not need to be added to the design.

The lateral earth pressure to be resisted by the retaining walls or similar structures should also be increased to allow for any other applicable surcharge loads. The surcharges considered should include forces generated by any structures or temporary loads that would influence the wall design.

A system of backfill drainage should be incorporated into retaining wall designs. Backfill comprising the drainage system immediately behind retaining structures should be free-draining granular material with a filter fabric between it and the rest of the backfill soils. As an alternative, the backs of walls could be lined with geodrain systems. The backdrains should extend from the bottoms of the walls to about 18 inches from finished backfill grade. Waterproofing may aid in reducing the potential for efflorescence on the faces of retaining walls.

Compaction on the uphill sides of walls within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential “locked-in” lateral pressures caused by compaction with heavy grading equipment.

SETTLEMENT CONSIDERATIONS

Maximum static settlements of about one inch are anticipated for conventional spread foundations and floor slabs supported on the recommended geogrid reinforced mat. Differential settlement between adjacent load bearing members should be expected to range up to about one-third the total settlement over a distance of 40 feet.

If the preliminary recommendations for foundation design and construction are followed, settlement of the piers should not exceed approximately 0.5 inch under static conditions. Differential settlement of neighboring pier footings of varying loads, depths or sizes may be as high as fifty percent of the total static settlement over a distance of about 30 feet.

Analyses of seismically-induced settlement potential indicate that approximately 0.6 inches of settlement could occur near the proposed northwest gateway as a result of a significant earthquake, approximately 1.8 inches could occur near the southeast gateway site, and approximately 1.9 inches could occur near the northeast gateway.

For structures supported on conventional spread foundations underlain by the recommended geogrid reinforced mat, approximately one-third of each of these total seismically-induced settlements could potentially be experienced as differential settlement over a distance of 40 feet.

For structures supported on drilled piers, the piers will not be subjected to the estimated liquefaction settlement of the liquefiable zone the pier extends through. For the southeast gateway, piers bottomed in the non-liquefiable soils between the depths of 10 and 24 feet will be subjected to 1 inch of seismically-induced settlement. For the northeast gateway, piers bottomed in the non-liquefiable soils below a depth of 15 feet will not be subjected to seismically-induced settlement. For the northwest gateway, piers bottomed in the non-liquefiable soils between the depths of 7 and 22 feet will be subjected to 0.3 inch of seismically-induced settlement. Differential settlement of neighboring pier footings of varying loads, depths or sizes may be as high as fifty percent of the total static settlement over a distance of about 30 feet.

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 exploratory borings advanced within the site for various studies. 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 structure 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 ensure that 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 contained herein.

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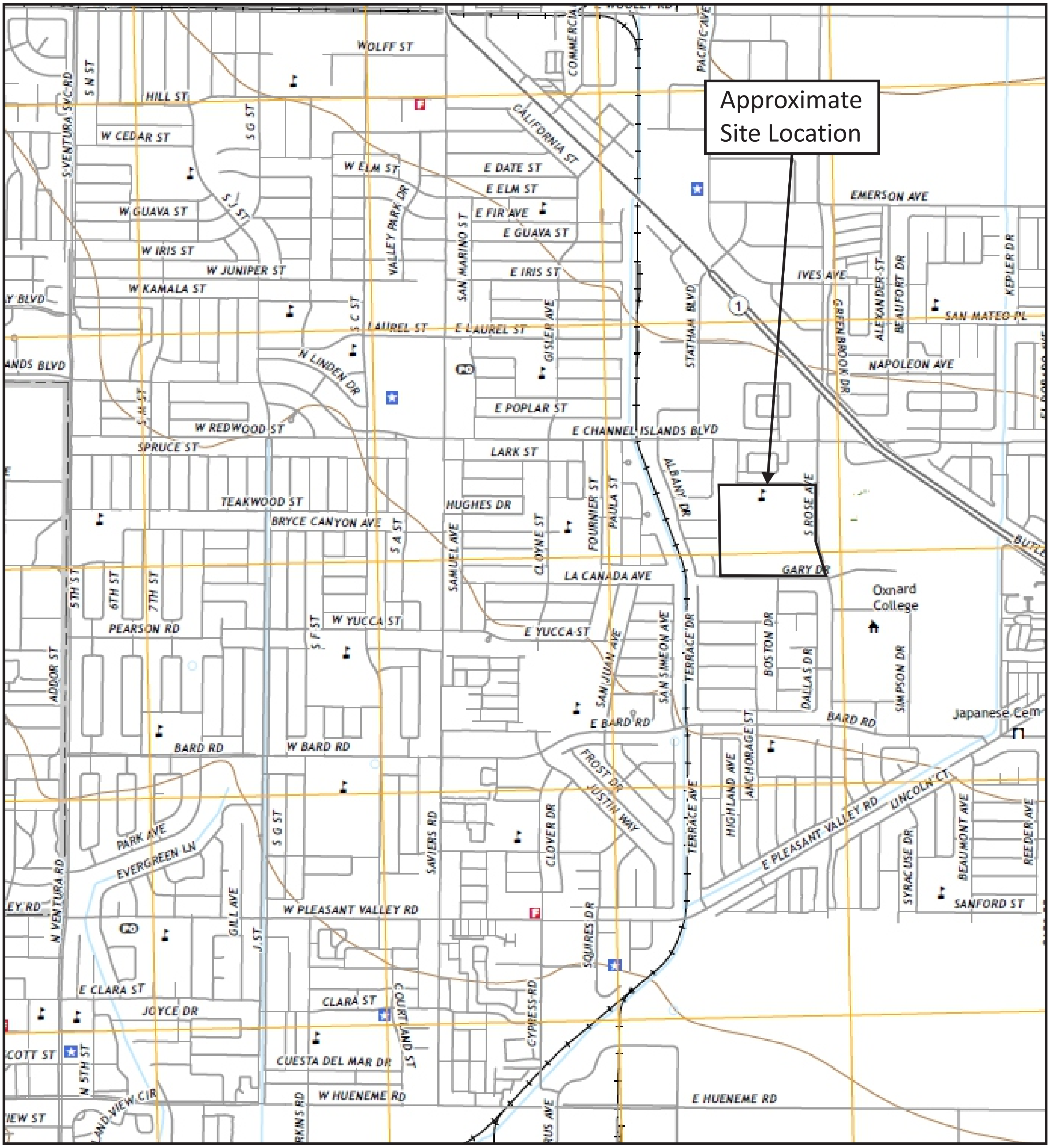
APPENDIX A

Vicinity Map
Regional Geologic Map
Seismic Hazard Zones Map
Historically High Groundwater Map
Field Study
Site Plan
Logs of Exploratory Borings (2019)
Logs of Boring B-3 and CPT-2 (2010)
Boring Log Symbols
Unified Soil Classification System



*Taken from USGS Topo Map, Oxnard Quadrangle, California, 2018.

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

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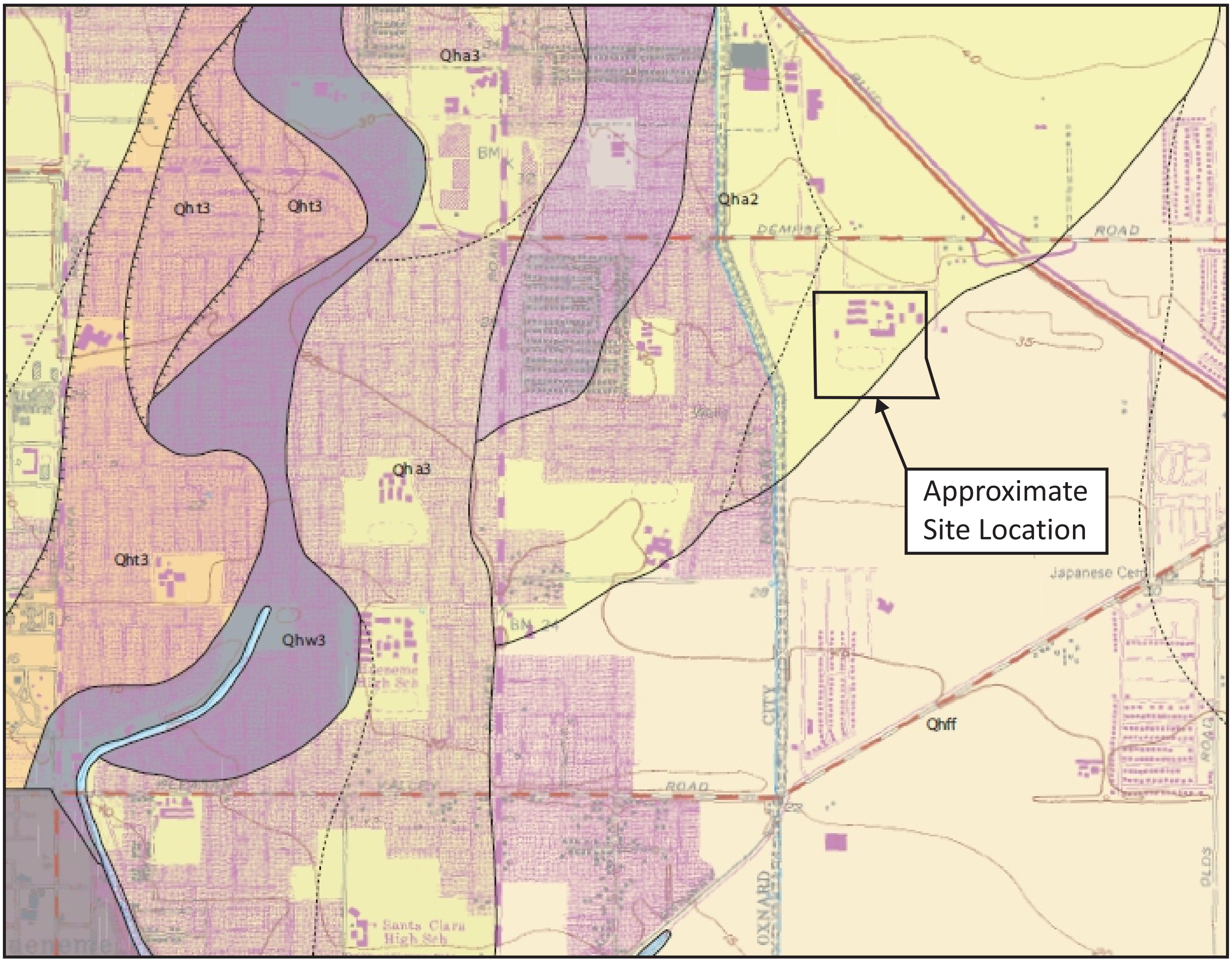
VICINITY MAP

Channel Islands High School Synthetic Field
Oxnard, California

Earth Systems

November 2019

303514-002



*Taken from USGS, SCAMP Geologic Map of the Ventura 7.5' Quadrangle, Ventura County, California, 2003.

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

N

MAP SYMBOLS

Contact between map units of different relative age; generally approximately located.

Contact between terraced alluvial units; hachures point towards topographically lower surface.

Contact between similar map units; generally approximately located.

Fault; dotted where concealed.

Axis of anticline; dotted where concealed.

Axis of syncline; dotted where concealed.

Qha2: Holocene alluvial deposits

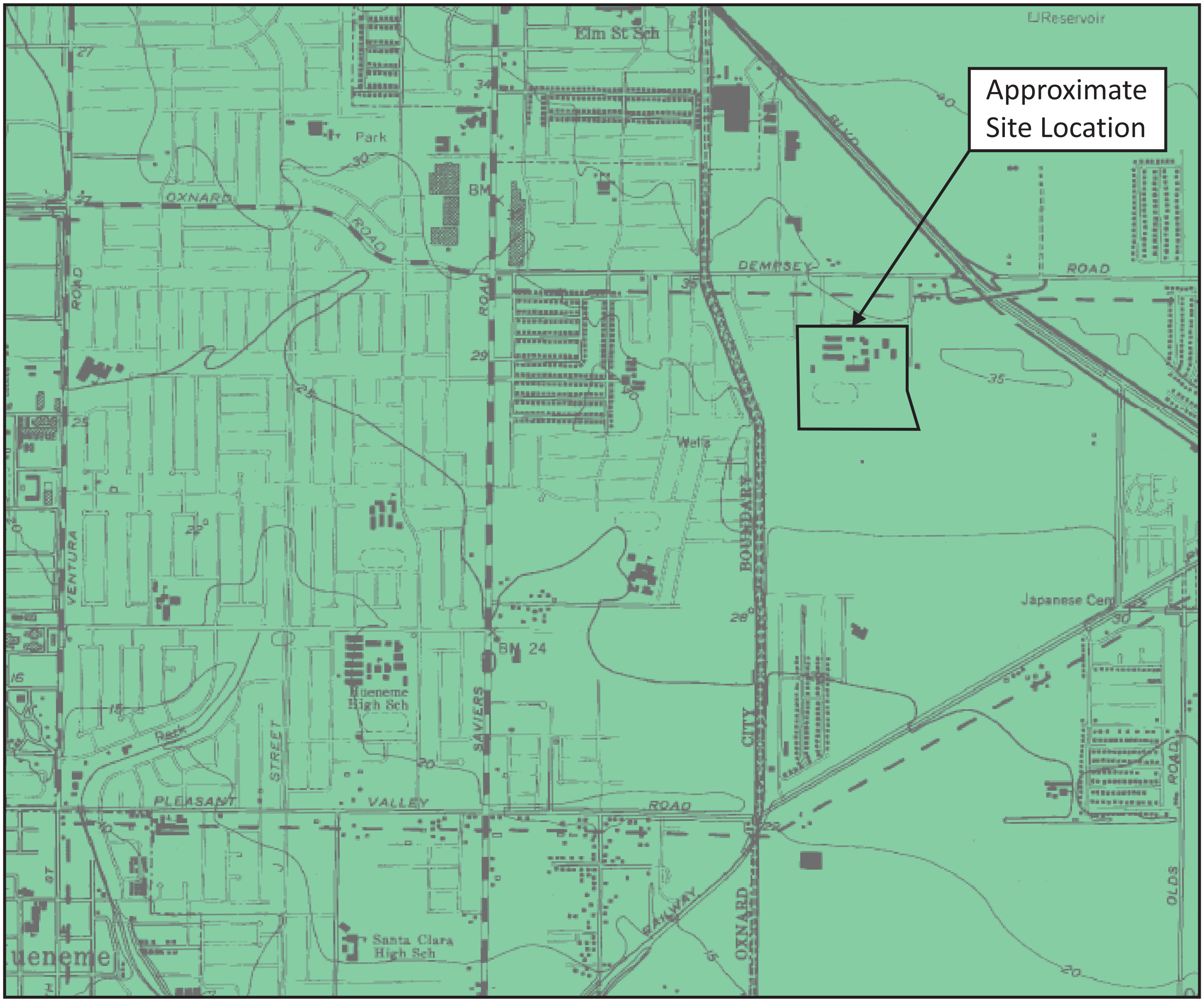
Qhff: Holocene alluvial fan deposits

REGIONAL GEOLOGIC MAP

Channel Islands High School Synthetic Field
Oxnard, California

Earth Systems

November 2019 303514-002



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.

Within the Oxnard Quadrangle, no areas have been designated as "zones of required investigation for earthquake-induced landslides." However, the potential for landslides may exist locally, particularly along stream banks, margins of drainage channels, and similar settings where steep banks or slopes occur. Such occurrences are of limited lateral extent, or are too small and discontinuous to be depicted at 1:24,000 scale (the scale of Seismic Hazard Zone Maps). Within the liquefaction zones, some geologic settings may be susceptible to lateral-spreading (a condition wherein low-angle landsliding is associated with liquefaction). Also, landslide hazards can be created during excavation and grading unless appropriate techniques are used.

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)

OXNARD QUADRANGLE

REVISED OFFICIAL MAP

Released: December 20, 2002

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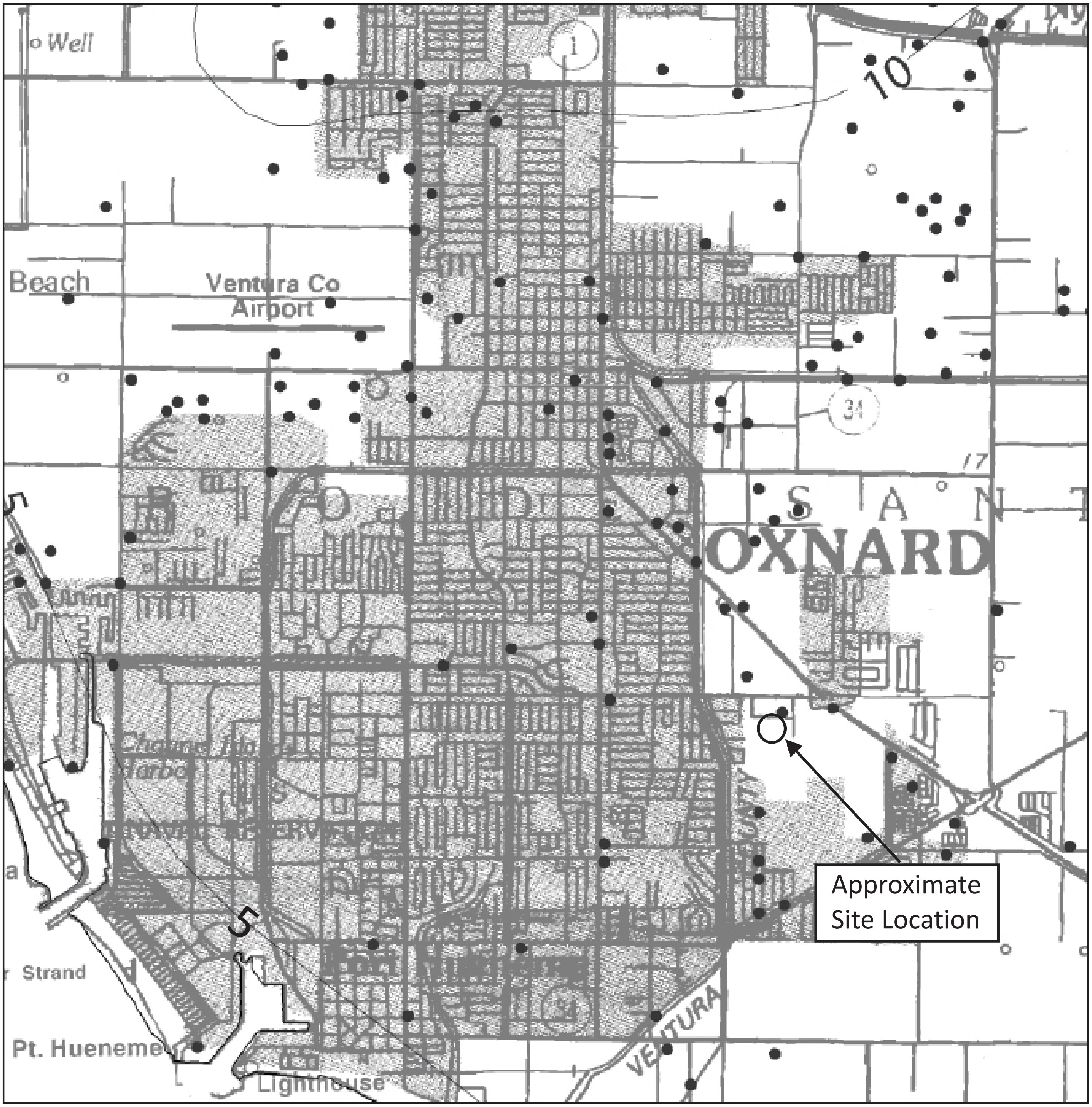
SEISMIC HAZARD ZONES MAP

Channel Islands High School Synthetic Field
Oxnard, California

Earth Systems

November 2019	303514-002



*Taken from CGS, Seismic Hazard Zone Report For The Oxnard 7.5-Minute Quadrangle, Ventura County, California, 2003.

30 Depth to ground water in feet

● Borehole Site

Approximate Scale: 1" = 4,000’

0 4,000’ 8,000’

N

HISTORICAL HIGH GROUNDWATER MAP

Channel Islands High School Synthetic Field
Oxnard, California

Earth Systems

November 2019	303514-002

FIELD STUDY

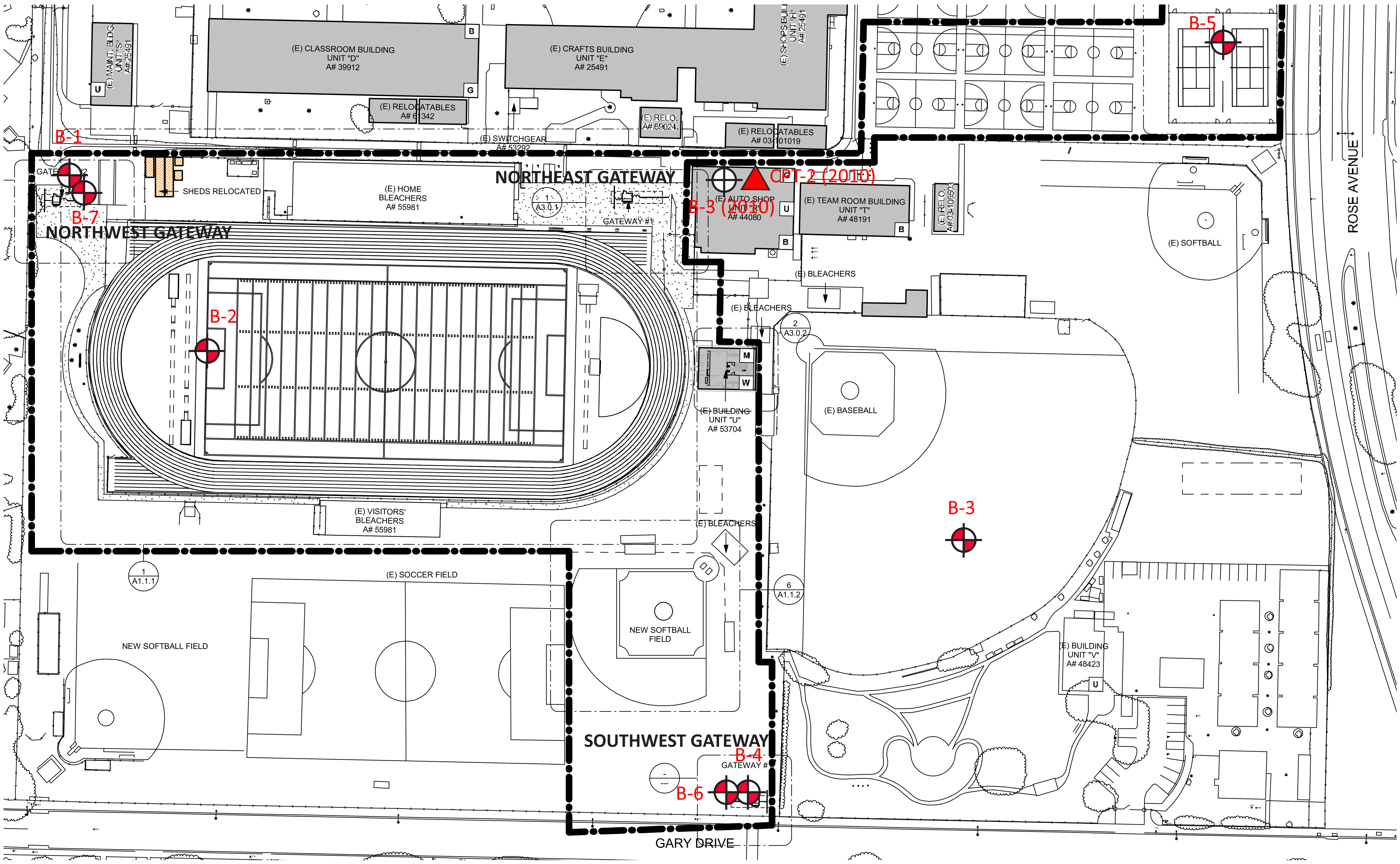
A. Five borings were drilled to a maximum depth of 15 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory analysis. The borings were drilled on June 26, 2019, using an 8-inch diameter hollow stem auger powered by a track-mounted CME-75 drilling rig. 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.

B. The initial five borings were supplemented by two borings (B-6 and B-7) drilled to a maximum depth of 51.5 feet below the existing ground surface. The supplemental borings were drilled on October 23, 2019, using a 4-inch diameter mud rotary system powered by a GTech 8 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. 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 2.00-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 automatic trip hammer dropping 30 inches in accordance with ASTM D 1586.

D. Bulk samples of the soils encountered in the upper 5 feet of Borings B-2 and B-3 were gathered from the cuttings.

E. The final logs of the 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 logs are included in this Appendix.



B-7 : Approximate boring locations

B-1 : Approximate boring location (2010 Site Studies)

CPT-2 : Approximate Cone Penetrometer Test (CPT) sounding (2010 Site Studies)

N

Approximate Scale: 1" = 100’

0 100’ 200’

SITE PLAN

Channel Islands High School
Oxnard, California

Earth Systems

October 2019

303514-002

Earth Systems

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

BORING NO: B-1	DRILLING DATE: June 26, 2019
PROJECT NAME: Channel Islands HS Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303276-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type Bulk	 SPT	 Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
			■	18/30/29	▥	SM	109.0	7.1	**ALLUVIUM:** Light brown Silty fine Sand, very dense, damp
5			■	4/7/7	░	SP	95.3	5.5	**ALLUVIUM:** Light Yellow Brown fine Sand, little Silt, loose, dry to damp
			■	2/3/3	▨	CL			**ALLUVIUM:** Dark Gray Silty Clay, trace iron oxide staining, soft to firm, very moist
10	▼		■	2/2/2	▨				
15									Total Depth: 15 feet Groundwater Depth: 11.5 feet.
20									
25									
30									
35									

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

BORING NO: B-2	DRILLING DATE: June 26, 2019
PROJECT NAME: Channel Islands HS Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303276-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type Bulk	 SPT	 Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0	☒		■	4/9/14	▥	SM	115.9	12.8	**ALLUVIUM:** Dark Brown Silty fine Sand, medium dense, damp to moist
5			■	2/4/3	▨	SC	91.8	31.5	**ALLUVIUM:** Dark Brown Clayey fine Sand, loose, moist
			■	6/8/15	░	SW	102.6	4.5	**ALLUVIUM:** Yellow Brown fine to coarse Sand, trace Silt, trace fine Gravel, medium dense, moist
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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-3	DRILLING DATE: June 26, 2019
PROJECT NAME: Channel Islands HS Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303276-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

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	☒		■	5/7/10	▥	ML	97.1	7.0	**ALLUVIUM:** Light Yellow Brown fine Sandy Silt, stiff, dry to damp
5			■	3/3/3	▥▨	ML/ CL	99.1	20.4	**ALLUVIUM:** Dark Gray Brown Clayey Silt, soft to firm, moist
			■	5/8/10	▦	SM	106.4	11.9	**ALLUVIUM:** Dark Gray Brown Silty fine Sand, medium dense, damp to moist
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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

Earth Systems

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PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-4
PROJECT NAME: Channel Islands HS Synthetic Field
PROJECT NUMBER: 303276-001
BORING LOCATION: Per Plan

DRILLING DATE: June 26, 2019
DRILL RIG: CME-75
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: A. Luna

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			■	3/5/7	▒	SP	99.2	3.7	**ALLUVIUM:** Light Yellow Brown fine Sand, trace Silt, medium dense, dry to damp
5			■	5/7/8	▥	ML	93.2	10.8	**ALLUVIUM:** Dark Gray Brown Clayey Silt, soft to firm, moist
			■	2/3/5	▨	CL			**ALLUVIUM:** Dark Brown fine Sandy Clay, firm, moist
					▥	SM			**ALLUVIUM:** Dark Brown Silty fine Sand, loose, moist
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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-5
PROJECT NAME: Channel Islands HS Synthetic Field
PROJECT NUMBER: 303276-001
BORING LOCATION: Per Plan

DRILLING DATE: June 26, 2019
DRILL RIG: CME-75
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: A. Luna

Vertical Depth	Sample Type Bulk	 SPT	 Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
			■	5/8/12	▨	SP	98.6	7.2	**ALLUVIUM:** Light Brown fine Sand, medium dense, dry to damp
5			■	4/8/12	▨	SP	95.9	3.4	
	▼		■	4/9/14	▨	SP			**ALLUVIUM:** Gray Brown fine Sand, trace to little medium Sand, medium dense, wet
10									Total Depth: 10 feet Groundwater Depth: 9.5 feet.
15									
20									
25									
30									
35									

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-6	DRILLING DATE: October 23, 2019
PROJECT NAME: Channel Islands HS	DRILL RIG: Gtech 8
PROJECT NUMBER: 303514-002	DRILLING METHOD: Mud Rotary
BORING LOCATION: Per Plan	LOGGED BY: AL

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						SP			**ALLUVIUM:** Light yellow brown fine Sand, trace Silt, medium dense,dry to damp.
5						ML			**ALLUVIUM:** Dark grayish brown Clayey Silt,soft to firm, moist.
10				3/4/5 4/6/8		SM			**ALLUVIUM:** Dark Brown to Dark Gray Brown Silty fine Sand, loose to medium dense, wet.
15				10/7/8 8/10/14		SW			**ALLUVIUM:** Gray Brown fine to medium Sand, trace to little Silt, medium dense, wet.
20				10/11/14 6/15/16 8/4/31		SW			**ALLUVIUM:** Gray Brown fine to medium Sand, trace coarse Sand, trace to little Silt, medium dense to dense, wet.
25				2/2/2		SM	-	24.2	**ALLUVIUM:** Dark Gray to Black Silty Sand, little Clay, very loose to loose, wet.
30				4/6/11 4/8/12		ML			**ALLUVIUM:** Gray fine Sandy Silt, trace Clay, very stiff, very moist.
				8/9/10		SM			**ALLUVIUM:** Gray Silty fine Sand, medium dense, wet.
35				12/16/18		SP			**ALLUVIUM:** Light Gray fine Sand, little Silt, dense, wet.
				15/18/23		SW			**ALLUVIUM:** Gray fine to medium Sand, trace to little Silt, dense, wet.

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

BORING NO: B-6 (Continued)
PROJECT NAME: Channel Islands HS
PROJECT NUMBER: 303514-002
BORING LOCATION: Per Plan

DRILLING DATE: October 23, 2019
DRILL RIG: Gtech 8
DRILLING METHOD: Mud Rotary
LOGGED BY: AL

Vertical Depth	Sample Type Bulk	 SPT	 Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6"	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
40		■		14/15/17	░	SW			**ALLUVIUM:** Gray fine to medium Sand, little coarse Sand, trace to little Silt, dense, wet.
		■		12/16/20	░				
45		■		13/15/17	░				
									Total Depth: 46.5 feet (Refusal for Flowing Sands) **Groundwater Depth: 10.0 feet**
50									
55									
60									
65									
70									
75									

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

BORING NO: B-7
PROJECT NAME: Channel Islands HS
PROJECT NUMBER: 303514-002
BORING LOCATION: Per Plan

DRILLING DATE: October 23, 2019
DRILL RIG: Gtech 8
DRILLING METHOD: Mud Rotary
LOGGED BY: A. Luna

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					▥	SM			**ALLUVIUM:** Light Brown Silty fine Sand, very dense, damp.
5					░	SP			**ALLUVIUM:** Light yellow Brown fine Sand, little Silt, loose, dry to damp.
▼		■		1/1/1	▨	CH	-	49.6	**ALLUVIUM:** Gray Silty Clay, trace iron oxide staining, soft, very moist.
10		■		1/1/1					
		■		9/10/13	░	SP			**ALLUVIUM:** Gray fine Sand, little Silt, medium dense, wet.
15		■		9/13/14	░	SW			**ALLUVIUM:** Gray fine to medium Sand, trace to little Silt, medium dense to dense, wet.
		■		9/14/18					
20		■		8/10/10	░	SP			**ALLUVIUM:** Gray fine Sand, little Silt, medium dense, wet.
		■		2/6/8	░	SW			**ALLUVIUM:** Gray fine to medium Sand, trace coarse Sand, little Silt, medium dense, wet.
25		■		2/2/3	▨	CL	-	32.0	**ALLUVIUM:** Gray Silty Clay, medium stiff, very moist.
		■		9/12/6	▥	SM			**ALLUVIUM:** Gray Silty fine Sand, medium dense to dense, wet.
30		■		9/17/15					
		■		3/4/5	▨	CL	-	28.4	**ALLUVIUM:** Gray Brown Silty Clay to Clayey Silt, stiff, very moist.
35		■		8/10/11	▥	ML			**ALLUVIUM:** Dark Gray fine Sandy Silt, medium dense to dense, very moist.
		■		10/8/23					

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

BORING NO: B-7 (Continued)	DRILLING DATE: October 23, 2019
PROJECT NAME: Channel Islands HS	DRILL RIG: Gtech 8
PROJECT NUMBER: 303514-002	DRILLING METHOD: Mud Rotary
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type Bulk	 SPT	 Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	**DESCRIPTION OF UNITS**
40		■		15/18/22	▥	ML			**ALLUVIUM:** Gray Sandy Silt, hard, wet.
		■		4/4/15	▥	ML			**ALLUVIUM:** Dark Gray fine Sandy Silt, trace Clay, very stiff, moist.
45		■		10/13/7	▒	SM			**ALLUVIUM:** Gray Silty fine Sand, trace Clay, medium dense, very moist.
		■		3/3/6	▨	CL	-	32.6	**ALLUVIUM:** Gray Silty Clay, stiff to medium stiff, very moist.
50		■		2/2/5	▨				
									Total Depth: 51.5 feet **Groundwater Depth: 10.0 feet**
55									
60									
65									
70									
75									

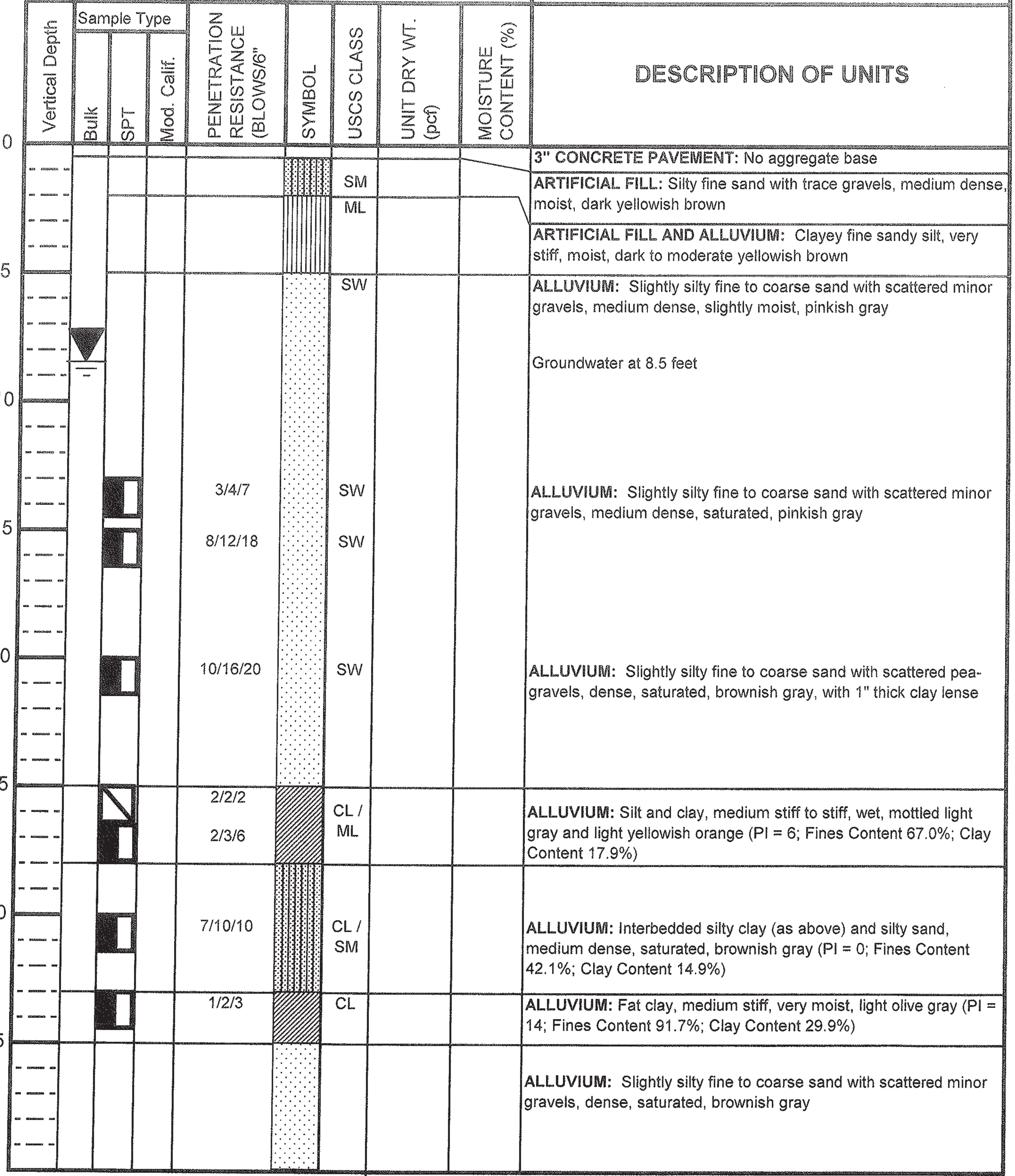
Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems Southern California

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

BORING NO: 3
PROJECT NAME: Channel Islands HS Auto Shop Addition
PROJECT NUMBER: VT-24349-01
BORING LOCATION: Per Plan

DRILLING DATE: June 24, 2010
DRILL RIG: Mobile Drill B-61
DRILLING METHOD: 4" Mud Rotary
LOGGED BY: P. Boales



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									**3" CONCRETE PAVEMENT:** No aggregate base
					▦	SM			**ARTIFICIAL FILL:** Silty fine sand with trace gravels, medium dense, moist, dark yellowish brown
					▥	ML			**ARTIFICIAL FILL AND ALLUVIUM:** Clayey fine sandy silt, very stiff, moist, dark to moderate yellowish brown
5					⋯	SW			**ALLUVIUM:** Slightly silty fine to coarse sand with scattered minor gravels, medium dense, slightly moist, pinkish gray Groundwater at 8.5 feet
		■		3/4/7	⋯	SW			**ALLUVIUM:** Slightly silty fine to coarse sand with scattered minor gravels, medium dense, saturated, pinkish gray
		■		8/12/18	⋯	SW			
		■		10/16/20	⋯	SW			**ALLUVIUM:** Slightly silty fine to coarse sand with scattered pea-gravels, dense, saturated, brownish gray, with 1" thick clay lense
25		■		2/2/2 2/3/6	▨	CL / ML			**ALLUVIUM:** Silt and clay, medium stiff to stiff, wet, mottled light gray and light yellowish orange (PI = 6; Fines Content 67.0%; Clay Content 17.9%)
		■		7/10/10	▦	CL / SM			**ALLUVIUM:** Interbedded silty clay (as above) and silty sand, medium dense, saturated, brownish gray (PI = 0; Fines Content 42.1%; Clay Content 14.9%)
		■		1/2/3	▨	CL			**ALLUVIUM:** Fat clay, medium stiff, very moist, light olive gray (PI = 14; Fines Content 91.7%; Clay Content 29.9%)
35					⋯				**ALLUVIUM:** Slightly silty fine to coarse sand with scattered minor gravels, dense, saturated, brownish gray

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

BORING NO: 3 (Continued)
PROJECT NAME: Channel Islands HS Auto Shop Addition
PROJECT NUMBER: VT-24349-01
BORING LOCATION: Per Plan

DRILLING DATE: June 24, 2010
DRILL RIG: Mobile Drill B-61
DRILLING METHOD: 4" Mud Rotary
LOGGED BY: P. Boales

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
40		■		14/20/24	░	SW			**ALLUVIUM:** Slightly silty fine to coarse sand with scattered minor gravels, dense, saturated, brownish gray
		■		1/7/8	▨	CL			**ALLUVIUM:** Silty clay, very moist, stiff, light olive gray (PI = 11; Fines Content 81.0%; Clay Content 25.3%)
45									
50									
55									
60									
65									
70									
75									

TOTAL DEPTH: 49.0 Feet

Groundwater Encountered at 8.5 Feet

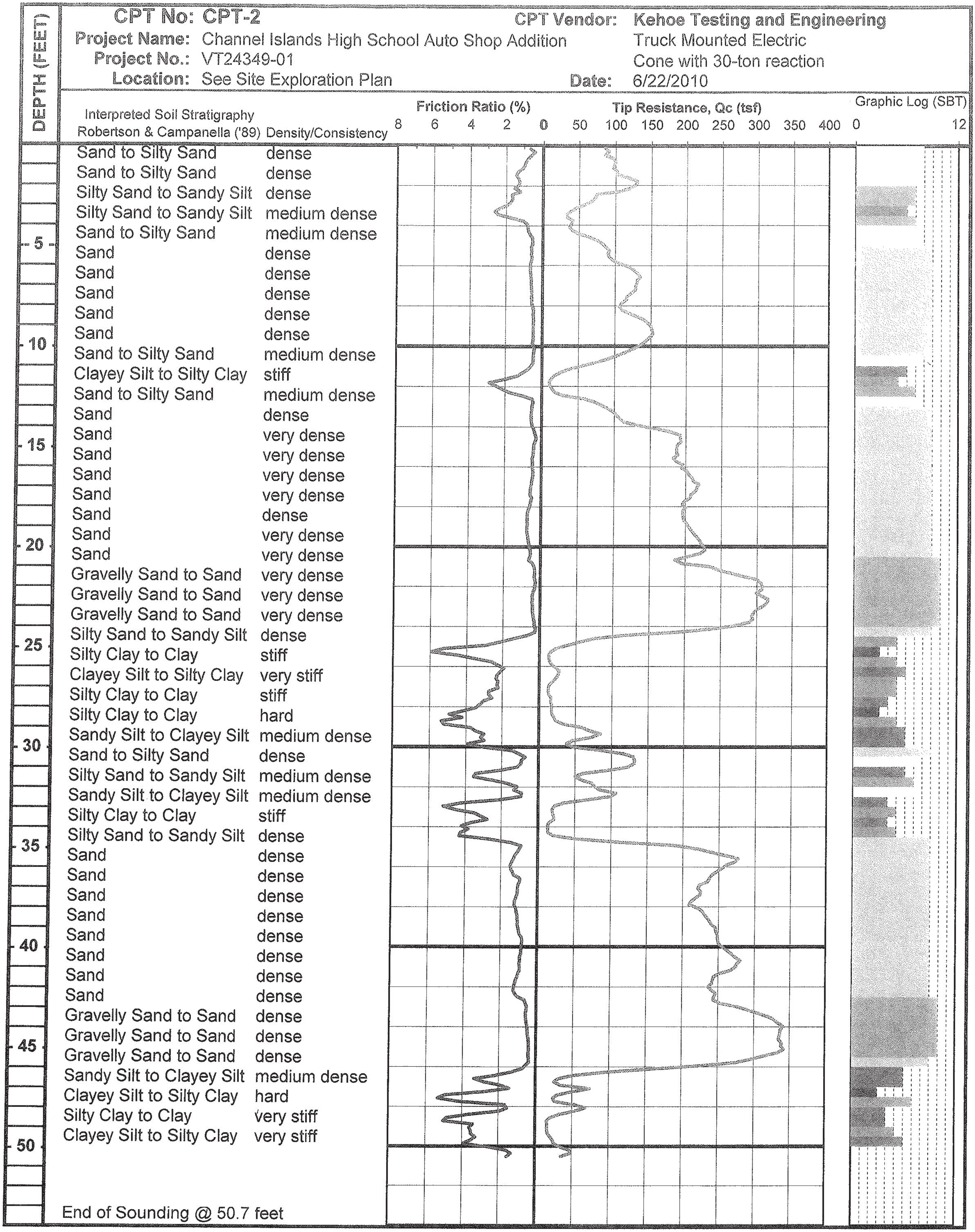
Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

CPT No: CPT-2
Project Name: Channel Islands High School Auto Shop Addition
Project No.: VT24349-01
Location: See Site Exploration Plan

CPT Vendor: Kehoe Testing and Engineering
Truck Mounted Electric
Cone with 30-ton reaction
Date: 6/22/2010

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

End of Sounding @ 50.7 feet



Earth Systems
Southern California

CONE PENETROMETER INTERPRETATION

(based on Robertson & Campanella, 1989)

Project: Channel Islands High School Auto Shop Addition

Project No: VT24349-01

Date: 06/22/10

CPT SOUNDING: CPT-2
Plot: 1
Density: 1 SPT N
Program developed 2003 by Shelton L. Stringer, GE, Earth Systems Southwest
Est. GWT (feet): **5.0**
Dr correlation: 0 Baldi
Qc/N: 0 Jefferies & Davies
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 $N_{1(60)}$	Est. % Fines	Rel. Dens. Dr (%)	Phi (deg.)	Nk: Su (tsf)	17 OCR
0.15	0.5	90.68	0.62	Sand to Silty Sand	SP/SM	medium dense	100	5.8	16	0.013	0.013	0.62	0.50	1.70	145.7	1.65	146.4	27	29	15	92	35		
0.30	1.0	98.34	0.99	Sand to Silty Sand	SP/SM	dense	100	5.6	18	0.038	0.038	0.99	0.54	1.70	158.0	1.76	170.2	30	34	20	96	36		
0.46	1.5	108.27	1.36	Sand to Silty Sand	SP/SM	dense	100	5.4	20	0.063	0.063	1.36	0.56	1.70	174.0	1.83	196.4	34	39	20	100	37		
0.61	2.0	126.74	1.36	Sand to Silty Sand	SP/SM	dense	120	5.5	23	0.090	0.090	1.36	0.54	1.70	203.6	1.78	222.8	39	45	20	100	38		
0.76	2.5	85.06	1.51	Silty Sand to Sandy Silt	SM/ML	medium dense	120	5.2	16	0.120	0.120	1.51	0.59	1.70	136.7	1.93	167.0	28	33	25	90	35		
0.91	3.0	57.87	1.98	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.8	12	0.150	0.150	1.98	0.65	1.70	93.0	2.13	141.5	20	28	40	74	33		
1.07	3.5	35.10	2.29	Sandy Silt to Clayey Silt	ML	medium dense	120	4.4	8	0.180	0.180	2.30	0.71	1.70	56.4	2.34	116.7	13	23	55	53	31		
1.22	4.0	38.85	0.87	Silty Sand to Sandy Silt	SM/ML	medium dense	120	5.0	8	0.210	0.210	0.87	0.62	1.70	62.4	2.04	84.4	13	17	35	57	31		
1.37	4.5	51.56	0.65	Sand to Silty Sand	SP/SM	medium dense	120	5.3	10	0.240	0.240	0.65	0.57	1.70	82.9	1.86	95.9	16	19	25	69	32		
1.52	5.0	82.01	0.54	Sand to Silty Sand	SP/SM	medium dense	120	5.8	14	0.270	0.270	0.54	0.50	1.70	131.8	1.65	132.6	24	27	15	88	34		
1.68	5.5	92.73	0.57	Sand	SP	medium dense	120	5.8	16	0.300	0.284	0.57	0.50	1.70	149.0	1.62	149.0	27	30	10	93	35		
1.83	6.0	117.16	0.64	Sand	SP	dense	120	5.9	20	0.330	0.299	0.64	0.50	1.70	188.3	1.58	188.3	34	38	10	100	37		
1.98	6.5	132.72	0.60	Sand	SP	dense	120	6.0	22	0.360	0.313	0.60	0.50	1.70	213.2	1.52	213.2	37	43	10	100	38		
2.13	7.0	129.28	0.57	Sand	SP	dense	120	6.0	21	0.390	0.328	0.58	0.50	1.70	207.7	1.51	207.7	36	42	10	100	38		
2.29	7.5	118.26	0.55	Sand	SP	dense	120	6.0	20	0.420	0.342	0.56	0.50	1.70	190.0	1.53	190.0	34	38	10	100	37		
2.44	8.0	107.81	0.50	Sand	SP	dense	120	6.0	18	0.450	0.356	0.50	0.50	1.70	173.2	1.53	173.2	30	35	10	100	36		
2.59	8.5	129.73	0.46	Sand	SP	dense	120	6.1	21	0.480	0.371	0.47	0.50	1.69	207.1	1.46	207.1	35	41	5	100	37		
2.74	9.0	149.13	0.46	Sand	SP	dense	120	6.2	24	0.510	0.385	0.46	0.50	1.66	233.6	1.41	233.6	39	47	5	100	38		
2.90	9.5	149.57	0.46	Sand	SP	dense	120	6.2	24	0.540	0.400	0.46	0.50	1.63	230.0	1.42	230.0	38	46	5	100	38		
3.05	10.0	132.76	0.44	Sand	SP	dense	120	6.2	22	0.570	0.414	0.44	0.50	1.60	200.6	1.45	200.6	34	40	5	100	37		
3.20	10.5	101.80	0.45	Sand	SP	medium dense	120	5.9	17	0.600	0.428	0.45	0.50	1.57	151.2	1.56	151.2	26	30	10	94	35		
3.35	11.0	55.87	0.60	Sand to Silty Sand	SP/SM	medium dense	120	5.4	10	0.630	0.443	0.61	0.56	1.63	86.0	1.83	97.4	16	19	20	71	32		
3.51	11.5	18.05	1.46	Sandy Silt to Clayey Silt	ML	loose	120	4.2	4	0.660	0.457	1.51	0.74	1.70	29.0	2.45	73.0	6	15	60	25	29		
3.66	12.0	11.21	2.51	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.6	3	0.690	0.472	2.68	0.84	1.70	18.0	2.76		3		95			0.63	6.7
3.81	12.5	34.28	1.08	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.8	7	0.720	0.486	1.10	0.65	1.66	53.9	2.15	83.4	10	17	40	51	30		
3.96	13.0	82.22	0.48	Sand to Silty Sand	SP/SM	medium dense	120	5.7	14	0.750	0.500	0.49	0.51	1.47	114.1	1.68	114.1	20	23	15	82	33		
4.11	13.5	103.25	0.53	Sand	SP	medium dense	120	5.8	18	0.780	0.515	0.54	0.50	1.43	139.9	1.63	139.9	25	28	15	91	35		
4.27	14.0	133.49	0.47	Sand	SP	dense	120	6.0	22	0.810	0.529	0.47	0.50	1.41	178.4	1.51	178.4	30	36	10	100	36		
4.42	14.5	187.43	0.30	Sand	SP	dense	120	6.5	29	0.840	0.544	0.30	0.50	1.40	247.2	1.29	247.2	39	49	0	100	38		
4.57	15.0	189.65	0.37	Sand	SP	dense	120	6.4	30	0.870	0.558	0.37	0.50	1.38	246.8	1.33	246.8	40	49	5	100	38		
4.72	15.5	186.05	0.43	Sand	SP	dense	120	6.3	30	0.900	0.572	0.43	0.50	1.36	239.1	1.39	239.1	39	48	5	100	38		
4.88	16.0	194.69	0.44	Sand	SP	dense	120	6.3	31	0.930	0.587	0.44	0.50	1.34	247.1	1.38	247.1	40	49	5	100	39		
5.03	16.5	208.06	0.45	Sand	SP	dense	120	6.3	33	0.960	0.601	0.46	0.50	1.33	260.9	1.37	260.9	43	52	5	100	39		
5.18	17.0	215.93	0.54	Sand	SP	dense	120	6.2	35	0.990	0.616	0.54	0.50	1.31	267.6	1.41	267.6	44	54	5	100	39		
5.33	17.5	204.41	0.52	Sand	SP	dense	120	6.2	33	1.020	0.630	0.52	0.50	1.30	250.4	1.42	250.4	41	50	5	100	39		
5.49	18.0	197.18	0.60	Sand	SP	dense	120	6.1	32	1.050	0.644	0.61	0.50	1.28	238.8	1.48	238.8	40	48	10	100	39		
5.64	18.5	197.02	0.71	Sand	SP	dense	120	6.0	33	1.080	0.659	0.71	0.50	1.27	236.0	1.53	236.0	40	47	10	100	39		
5.79	19.0	204.94	0.72	Sand	SP	dense	120	6.0	34	1.110	0.673	0.72	0.50	1.25	242.8	1.53	242.8	42	49	10	100	39		
5.94	19.5	215.73	0.67	Sand	SP	dense	120	6.1	36	1.140	0.688	0.67	0.50	1.24	252.9	1.50	252.9	43	51	10	100	39		
6.10	20.0	225.17	0.57	Sand	SP	dense	120	6.2	36	1.170	0.702	0.57	0.50	1.23	261.3	1.44	261.3	43	52	5	100	39		
6.25	20.5	202.93	0.56	Sand	SP	dense	120	6.1	33	1.200	0.716	0.56	0.50	1.22	233.1	1.47	233.1	39	47	5	100	38		
6.40	21.0	225.35	0.33	Gravelly Sand to Sand	SW	dense	120	6.5	35	1.230	0.731	0.33	0.50	1.20	256.3	1.29	256.3	41	51	0	100	39		
6.55	21.5	281.24	0.28	Gravelly Sand to Sand	SW	dense	120	6.7	42	1.260	0.745	0.28	0.50	1.19	316.7	1.18	316.7	49	63	0	100	40		
6.71	22.0	306.84	0.27	Gravelly Sand to Sand	SW	very dense	120	6.8	45	1.290	0.760	0.27	0.50	1.18	342.3	1.14	342.3	52	68	0	100	41		
6.86	22.5	307.36	0.39	Gravelly Sand to Sand	SW	very dense	120	6.6	47	1.320	0.774	0.39	0.50	1.17	339.7	1.24	339.7	53	68	0	100	41		
7.01	23.0	308.42	0.39	Gravelly Sand to Sand	SW	very dense	120	6.6	47	1.350	0.788	0.39	0.50	1.16	337.7	1.24	337.7	53	68	0	100	41		
7.16	23.5	294.46	0.35	Gravelly Sand to Sand	SW	dense	120	6.6	45	1.380	0.803	0.35	0.50	1.15	319.5	1.23	319.5	50	64	0	100	41		
7.32	24.0	265.50	0.25	Gravelly Sand to Sand	SW	dense	120	6.7	40	1.410	0.817	0.25	0.50	1.14	285.5	1.19	285.5	44	57	0	100	39		
7.47	24.5	127.28	0.75	Sand	SP	medium dense	120	5.6	23	1.440	0.832	0.76	0.53	1.14	136.6	1.73	144.6	25	29	15	90	35		
7.62	25.0	39.37	3.40	Clayey Silt to Silty Clay	ML/CL	medium dense	120	4.0	10	1.470	0.846	3.53	0.77	1.19	44.2	2.54	131.9	11	26	70	43	30		
7.77	25.5	13.01	4.66	Clay	CL/CH	stiff	120	3.1	4	1.500	0.860	5.27	0.92	1.21	14.9	3.01		4		100			0.71	4.0
7.92	26.0	17.62	2.29	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.7	5	1.530	0.875	2.50	0.83	1.17	19.5	2.72		5		90			0.99	5.5
8.08	26.5	23.60	2.25	Sandy Silt to Clayey Silt	ML	very stiff	120	3.9	6	1.560	0.889	2.41	0.80	1.15	25.6	2.61		6		75			1.34	7.4
8.23	27.0	12.26	2.43	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.4	4	1.590	0.904	2.79	0.88	1.15	13.3	2.88		4		100			0.67	3.5
8.38	27.5	14.32	2.79	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.4	4	1.620	0.918	3.15	0.87	1.13	15.3	2.86		4		100			0.79	4.2
8.53	28.0	16.54	3.62	Silty Clay to Clay	CL	stiff	120	3.4	5	1.650	0.932	4.02	0.88	1.12	17.5	2.88		5		100			0.92	4.8
8.69	28.5	20.02	4.82	Clay	CL/CH	very stiff	120	3.3	6	1.680	0.947	5.27	0.88	1.10	20.9	2.90		6		100			1.12	5.8
8.84	29.0	52.93	4.16	Clayey Silt to Silty Clay	ML/CL	medium dense	120	4.0	13	1.710	0.961	4.30	0.77	1.08	53.9	2.54	160.8	13	32	70	51	31		
8.99	29.5	68.07	3.04	Sandy Silt to Clayey Silt	ML	medium dense	120	4.4	16	1.740	0.976	3.12	0.72	1.06	68.2	2.37	149.7	16	30	55	61	32		
9.14	30.0	56.73	2.79	Sandy Silt to Clayey Silt	ML	medium dense	120	4.3	13	1.770	0.990	2.88	0.73	1.05	56.3	2.40	130.9	13	26	60	53	31		
9.30	30.5	126.15	0.87	Sand	SP	medium dense	120	5.5	23	1.800	1.004	0.89	0.55	1.03	122.7	1.81	136.6	23	27	20	85	34		
9.45	31.0	117.33	1.35	Sand to Silty Sand	SP/SM	medium dense	120	5.2	23	1.830	1.019	1.37	0.60	1.02	113.4	1.96	142.1	23	28	30	82	34		
9.60	31.5	56.47	3.18	Sandy Silt to Clayey Silt	ML	medium dense	120	4.2	13	1.860	1.033	3.28	0.75	1.02	54.3	2.45	138.5	13	28	65	52	31		
9.75	32.0	78.02	1.39	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.9	16	1.890	1.048	1.42	0.64	1.01	74.2	2.11	109.0	16	22	35	64	32		
9.91	32.5	91.34	1.27	Sand to Silty Sand	SP/SM	medium dense	120	5.0	18	1.920	1.062	1.30	0.62	1.00	86.1	2.03	116.0	18	23	30	71	33		
10.06	33.0	25.26	4.66	Silty Clay to Clay	CL	very stiff	120	3.4	7	1.950	1.076	5.05	0.87	0.99	23.5	2.85		7		100			1.42	6.5
10.21	33.5	20.60	3.13	Clayey Silt to Silty Clay	ML/CL	very stiff	120	3.5	6	1.980	1.091	3.47	0.86	0.97	19.0	2.81		6		100			1.15	5.1
10.36	34.0	13.04	3.89	Silty Clay to Clay	CL	stiff	120	3.0	4	2.010	1.105	4.60	0.93	0.96	11.8	3.05		4		100			0.70	3.0
10.52	34.5	28.31	3.73	Clayey Silt to Silty Clay	ML/CL	very stiff	120	3.6	8	2.040	1.120	4.02	0.84	0.95	25.5	2.75		8		90			1.60	7.0
10.67	35.0	174.45	1.12	Sand	SP	medium dense	120	5.5	32	2.070	1.134	1.13	0.55	0.96	158.7	1.80	175.4	30	35	20	96	36		
10.82	35.5	258.12	1.20	Sand	SP	dense	120	5.7	46	2.100	1.148	1.21	0.52	0.96	233.8	1.71	243.7	43	49	15	100	39		
10.97	36.0	261.94	1.44	Sand	SP	dense	120	5.5	47	2.130	1.163	1.45	0.54	0.95	235.3	1.77	254.6	44	51	20	100	39		
11.13	36.5	239.56	1.12	Sand	SP	dense	120	5.7	42	2.160	1.177	1.13	0.52	0.95	214.2	1.71	223.6	39	45	15	100	38		
11.28	37.0	222.75	1.05	Sand	SP	dense	120	5.6	39	2.190	1.192	1.06	0.52	0.94	197.9	1.71	206.8	36	41	15	100	38		
11.43	37.5	217.50	1.16	Sand	SP	dense	120	5.6	39	2.220	1.206	1.17	0.53	0.93	191.7	1.75	205.7	36	41	20	100	37		

Earth Systems
Southern California

CONE PENETROMETER INTERPRETATION (based on Robertson & Campanella, 1989)

Project: Channel Islands High School Auto Shop Addition Project No: VT24349-01 Date: 06/22/10

CPT SOUNDING: CPT-2 Plot: 1 Density: 1 SPT N Program developed 2003 by Shelton L. Stringer, GE, Earth Systems Southwest

Est. GWT (feet): **5.0** Dr correlation: 0 Baldi Qc/N: 0 Jefferies & Davies 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 $N_{1(60)}$	Est. % Fines	Rel. Dens. Dr (%)	Phi (deg.)	Nk: Su (tsf)	17 OCR
11.58	38.0	218.80	1.17	Sand	SP	dense	120	5.6	39	2.250	1.220	1.18	0.54	0.93	191.6	1.76	206.0	36	41	20	100	37		
11.73	38.5	234.01	1.06	Sand	SP	dense	120	5.7	41	2.280	1.235	1.07	0.52	0.92	204.1	1.71	212.7	37	43	15	100	38		
11.89	39.0	243.77	1.02	Sand	SP	dense	120	5.7	43	2.310	1.249	1.03	0.51	0.92	211.6	1.68	217.0	38	43	15	100	38		
12.04	39.5	248.59	0.84	Sand	SP	dense	120	5.8	43	2.340	1.264	0.85	0.50	0.92	215.0	1.62	215.0	38	43	10	100	38		
12.19	40.0	256.86	0.82	Sand	SP	dense	120	5.9	44	2.370	1.278	0.83	0.50	0.91	220.9	1.60	220.9	39	44	10	100	38		
12.34	40.5	274.04	0.88	Sand	SP	dense	120	5.9	47	2.400	1.292	0.89	0.50	0.90	234.4	1.61	234.4	41	47	10	100	39		
12.50	41.0	272.05	0.92	Sand	SP	dense	120	5.8	47	2.430	1.307	0.93	0.50	0.90	231.4	1.62	231.4	41	46	10	100	39		
12.65	41.5	245.30	1.05	Sand	SP	dense	120	5.7	43	2.460	1.321	1.06	0.52	0.89	206.6	1.70	214.4	38	43	15	100	38		
12.80	42.0	239.23	1.21	Sand	SP	dense	120	5.6	43	2.490	1.336	1.23	0.54	0.88	199.6	1.76	214.6	37	43	20	100	38		
12.95	42.5	244.48	0.93	Sand	SP	dense	120	5.7	43	2.520	1.350	0.94	0.51	0.88	204.2	1.67	207.0	37	41	15	100	38		
13.11	43.0	273.07	0.52	Gravelly Sand to Sand	SW	dense	120	6.1	44	2.550	1.364	0.53	0.50	0.88	227.3	1.46	227.3	38	45	5	100	38		
13.26	43.5	317.48	0.54	Gravelly Sand to Sand	SW	dense	120	6.2	51	2.580	1.379	0.54	0.50	0.88	262.9	1.42	262.9	43	53	5	100	39		
13.41	44.0	337.05	0.47	Gravelly Sand to Sand	SW	dense	120	6.3	53	2.610	1.393	0.47	0.50	0.87	277.6	1.36	277.6	45	56	5	100	40		
13.56	44.5	335.47	0.41	Gravelly Sand to Sand	SW	dense	120	6.4	52	2.640	1.408	0.41	0.50	0.87	274.9	1.33	274.9	44	55	5	100	39		
13.72	45.0	339.46	0.37	Gravelly Sand to Sand	SW	dense	120	6.5	53	2.670	1.422	0.37	0.50	0.86	276.8	1.30	276.8	44	55	0	100	39		
13.87	45.5	315.00	0.37	Gravelly Sand to Sand	SW	dense	120	6.4	49	2.700	1.436	0.38	0.50	0.86	255.5	1.33	255.5	41	51	5	100	39		
14.02	46.0	220.80	0.53	Sand	SP	dense	120	6.0	37	2.730	1.451	0.54	0.50	0.85	178.2	1.55	178.2	31	36	10	100	36		
14.17	46.5	59.79	2.27	Sandy Silt to Clayey Silt	ML	medium dense	120	4.3	14	2.760	1.465	2.38	0.74	0.79	44.4	2.42	107.0	12	21	60	43	31		
14.33	47.0	45.52	2.24	Sandy Silt to Clayey Silt	ML	loose	120	4.1	11	2.790	1.480	2.38	0.77	0.77	33.3	2.52	95.3	9	19	70	31	30		
14.48	47.5	26.18	4.81	Clay	CL/CH	very stiff	120	3.2	8	2.820	1.494	5.39	0.90	0.73	18.1	2.95		8		100			1.45	4.7
14.63	48.0	57.86	1.71	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.4	13	2.850	1.508	1.79	0.72	0.77	42.4	2.36	91.3	11	18	55	41	30		
14.78	48.5	22.39	4.48	Silty Clay to Clay	CL	very stiff	120	3.1	7	2.880	1.523	5.14	0.91	0.72	15.2	2.99		7		100			1.23	3.8
14.94	49.0	15.31	3.50	Silty Clay to Clay	CL	stiff	120	3.0	5	2.910	1.537	4.33	0.94	0.70	10.2	3.08		5		100			0.81	2.4
15.09	49.5	20.19	3.43	Clayey Silt to Silty Clay	ML/CL	very stiff	120	3.2	6	2.940	1.552	4.01	0.91	0.71	13.5	2.97		6		100			1.10	3.3
15.24	50.0	35.41	2.69	Sandy Silt to Clayey Silt	ML	very stiff	120	3.7	9	2.970	1.566	2.94	0.82	0.73	24.3	2.68		9		85			1.99	6.2

BORING LOG SYMBOLS

■	Modified California Split Barrel Sampler
⧅	Modified California Split Barrel Sampler - No Recovery
◨	Standard Penetration Test (SPT) Sampler
◫	Standard Penetration Test (SPT) Sampler - No Recovery
▽	Perched Water Level
▼	Water Level First Encountered
▼	Water Level After Drilling
⊙	Pocket Penetrometer (tsf)
⊕	Vane Shear (ksf)

1. The location of borings were approximately determined by pacing and/or siting from visible features. Elevations of borings are approximately determined by interpolating between plan contours. The location and elevation of the borings should be considered.

2. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

3. Water level readings have been made in the drill holes at times and under conditions stated on the boring logs. This data has been reviewed and interpretations made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, tides, temperature, and other factors at the time measurements were made.

BORING LOG SYMBOLS

Earth Systems

UNIFIED SOIL CLASSIFICATION SYSTEM

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

UNIFIED SOIL CLASSIFICATION SYSTEM

Earth Systems

APPENDIX B

Laboratory Testing
Tabulated Laboratory Test Results
Individual Laboratory Test Results
Table 18-I-D with Footnotes

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

D. The relative strength characteristics of soils were determined from the results of a Direct Shear test performed on remolded samples. 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.

E. 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% saturation. The sample was then submerged in water for 24 hours, and the amount of expansion was recorded with a dial indicator.

F. Settlement characteristics were developed from the results of a one-dimensional Consolidation test performed in general accordance with ASTM D 2435. The sample was loaded to 0.5 ksf, flooded with water, and then incrementally loaded to 1.0, 2.0, and 4.0 ksf. The sample was allowed to consolidate under each load increment. Rebound was measured under reverse alternate loading. Compression was measured by dial gauges accurate to 0.0001 inch. Results of the consolidation test are presented as a curve plotting percent consolidation versus log of pressure.

G. A portion of the bulk sample 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.

H. The gradation characteristics of selected 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 distribution of the minus No. 200 mesh material of the samples. The hydrometer portions of the tests were run using sodium hexametaphosphate as a dispersing agent.

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

J. A Resistance ("R") Value test was conducted on a bulk sample secured during the field study. The test was performed in accordance with California Method 301. Three specimens at different moisture contents were tested for each sample, and the R-Value at 300 psi exudation pressure was determined from the plotted results.

TABULATED LABORATORY TEST RESULTS

BORING AND DEPTH	B-2 @ 0-5’		B-3 @ 0-5’
USCS	SM		ML
MAXIMUM DENSITY (pcf)	122.0		--
OPTIMUM MOISTURE (%)	10.0		--
COHESION (psf)	0*	0**	--
ANGLE OF INTERNAL FRICTION	34°*	32°**	--
EXPANSION INDEX	3		--
RESISTANCE (“R”) VALUE	--		57
pH	8.0		--
SOLUBLE CHLORIDES (mg/Kg)	10		--
RESISTIVITY (ohms-cm)	2,200		--
SOLUBLE SULFATES (mg/Kg)	510		--
GRAIN SIZE DISTRIBUTION (%)			
GRAVEL	--		0
SAND	--		44
SILT AND CLAY	--		56

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

BORING AND DEPTH	B-6 @ 7.5’	B-6 @ 25’	B-6 @ 27.5’
USCS	SM	SM	ML
LIQUID LIMIT	--	23	--
PLASTIC LIMIT	--	21	--
PLASTICITY INDEX	--	2	--
GRAIN SIZE DISTRIBUTION (%)			
GRAVEL	0.0	0.0	0.1
SAND	75.2	56.9	42.0
SILT	18.3	23.3	33.2
CLAY (2ųm to 5ųm)	1.7	6.7	8.3
CLAY (≤2ųm)	4.8	13.1	16.4

TABULATED LABORATORY TEST RESULTS (Continued)

BORING AND DEPTH	B-7 @ 7.5’	B-7 @ 25’	B-7 @ 32.5’
USCS	CH	CL	CL
LIQUID LIMIT	53	39	32
PLASTIC LIMIT	29	19	22
PLASTICITY INDEX	24	20	10
GRAIN SIZE DISTRIBUTION (%)			
GRAVEL	0.0	0.0	0.0
SAND	10.4	20.3	25.2
SILT	43.2	43.2	48.3
CLAY (2ųm to 5ųm)	16.7	11.7	8.3
CLAY (≤2ųm)	29.7	24.8	18.2

BORING AND DEPTH	B-7 @ 35’	B-7 @ 42.5’	B-7 @ 47.5’
USCS	ML	ML	CL
LIQUID LIMIT	--	--	36
PLASTIC LIMIT	--	--	23
PLASTICITY INDEX	--	--	13
GRAIN SIZE DISTRIBUTION (%)			
GRAVEL	0.0	0.0	0.0
SAND	38.5	10.2	11.8
SILT	48.3	65.0	58.4
CLAY (2ųm to 5ųm)	3.4	8.3	11.6
CLAY (≤2ųm)	9.8	16.5	18.2

File Number: 303276-001 Lab Number: 098209

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Channel Island High School Synthetic Turf Field
Sample ID: B 2 @ 0-5'

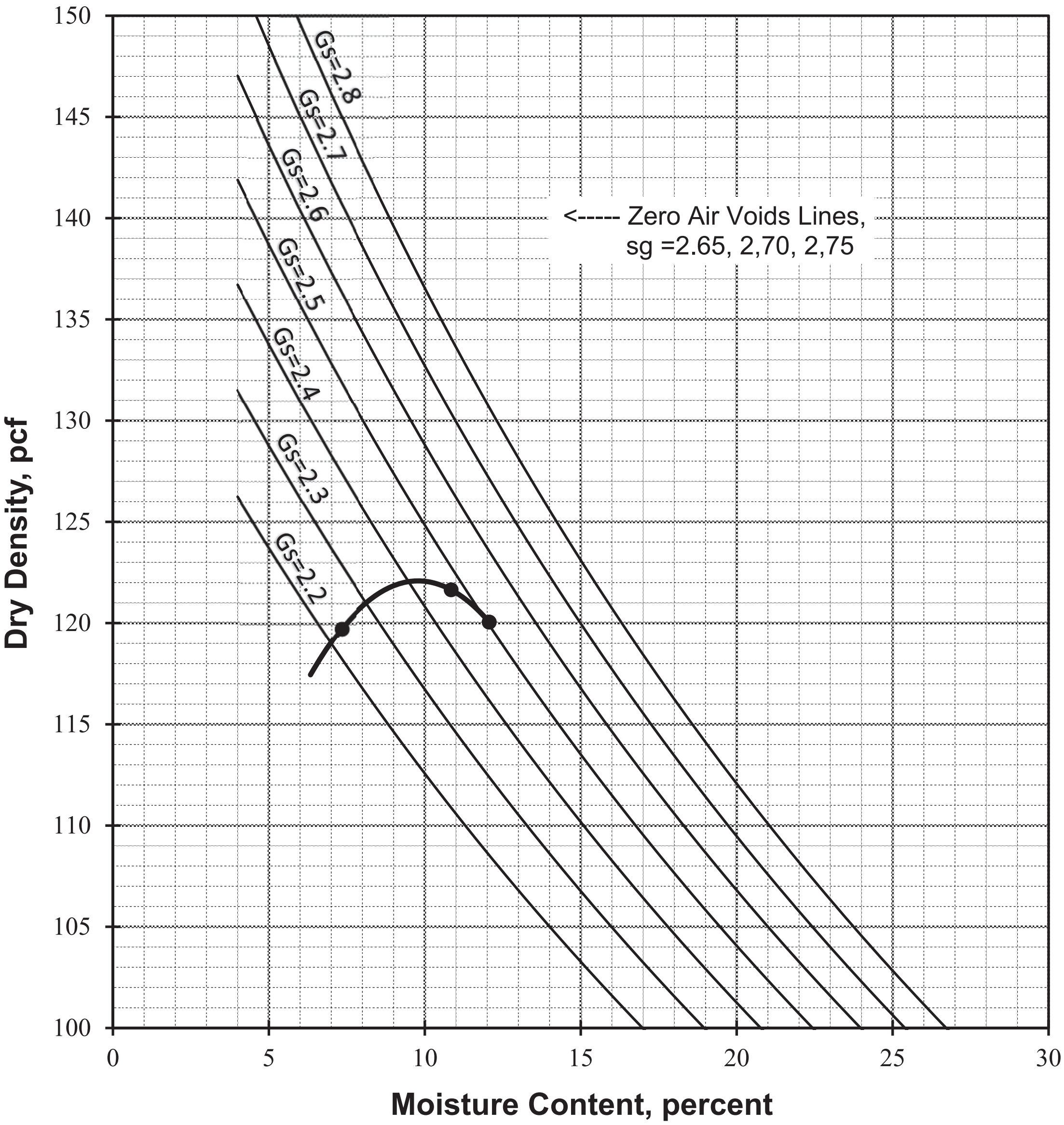
Procedure Used: A
Prep. Method: Moist

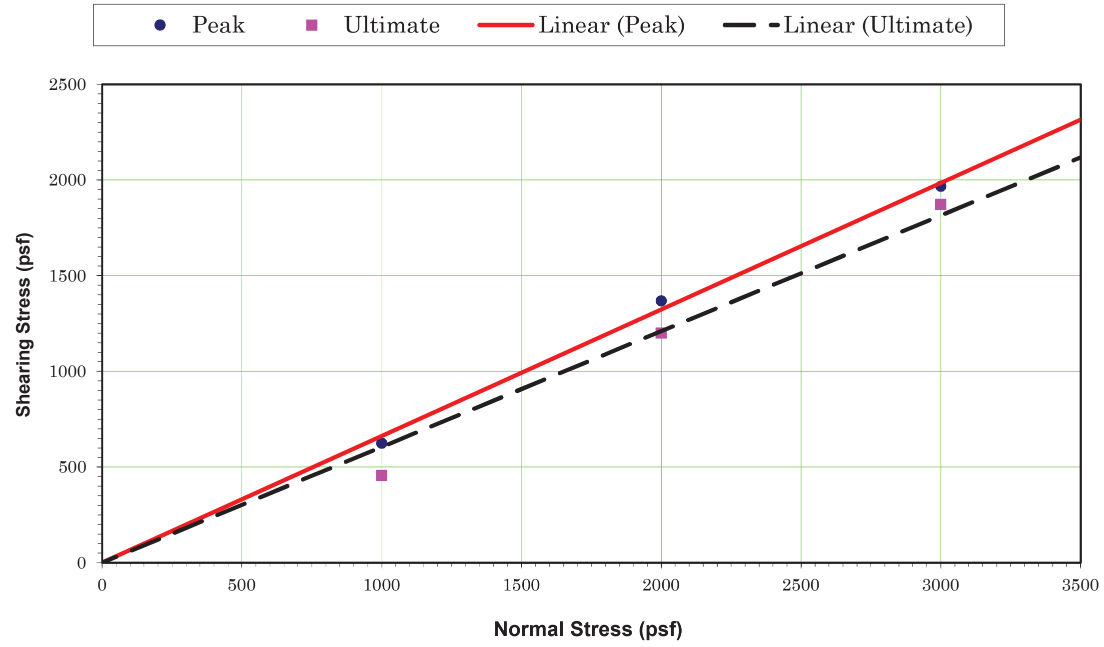
Date: 7/29/2019
Description: Olive Brown Silty Sand
SG: 2.42

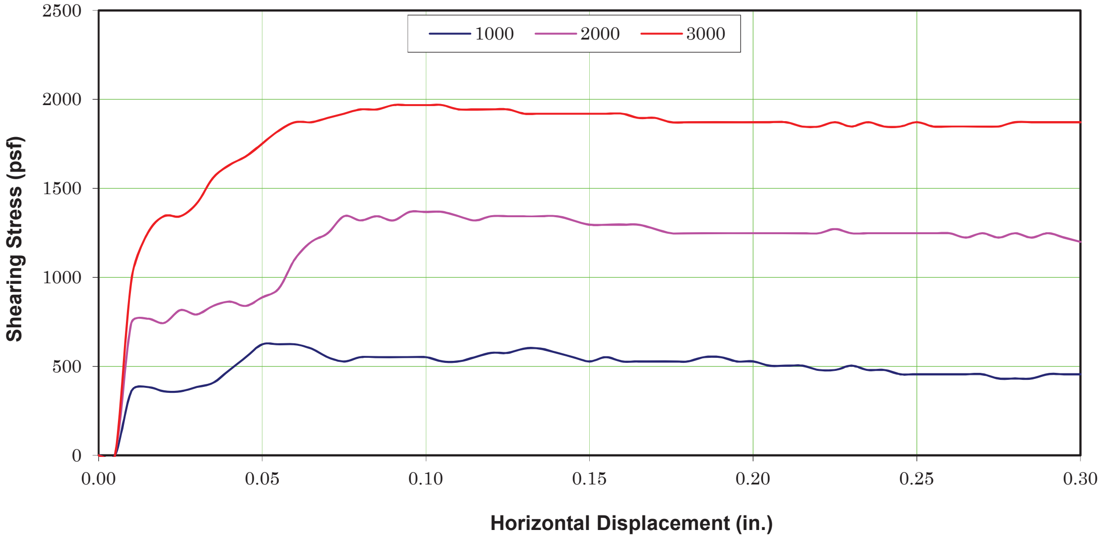
Rammer Type: Automatic

Maximum Density: **122 pcf**
Optimum Moisture: **10%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	0.4







DIRECT SHEAR DATA*

Sample Location: B 2 @ 0-5'
Sample Description: Silty Sand
Dry Density (pcf): 109.9
Intial % Moisture: 9.8
Average Degree of Saturation: 87.2
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	624	1368	1968
Ultimate stress (psf)	456	1200	1872

	Peak	Ultimate
ϕ Angle of Friction (degrees):	34	32
c Cohesive Strength (psf):	0	0

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Channel Island High School Synthetic Turf Field

Earth Systems

8/27/2019 303276-001

File No.: 303276-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

Job Name: Channel Island High School Synthetic Turf Field
Sample ID: B 2 @ 0-5'
Soil Description: SM

Initial Moisture, %:	9.0
Initial Compacted Dry Density, pcf:	113.7
Initial Saturation, %:	51
Final Moisture, %:	17.9
Volumetric Swell, %:	0.3

Expansion Index: **3** **Very Low**

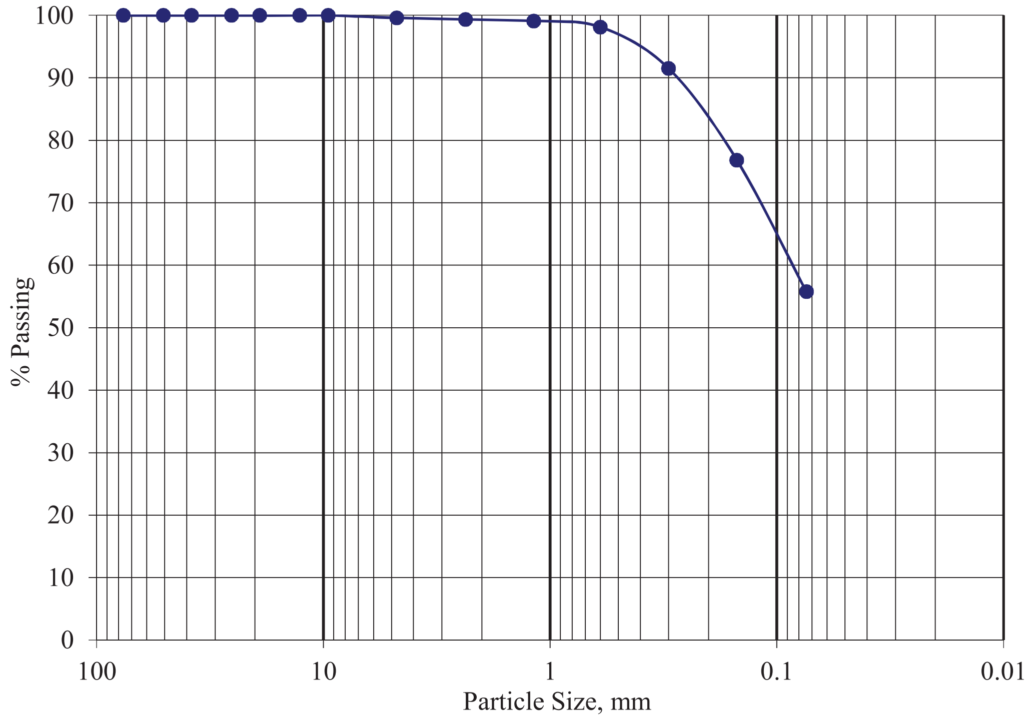
EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
130+	Very High

SIEVE ANALYSIS

ASTM C-136

Job Name: 303276-001
Sample ID: B 3 @ 0-5'
Description: ML

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	100
#4	100
#8	99
#16	99
#30	98
#50	92
#100	77
#200	56



RESISTANCE '*R* ' VALUE AND EXPANSION PRESSURE

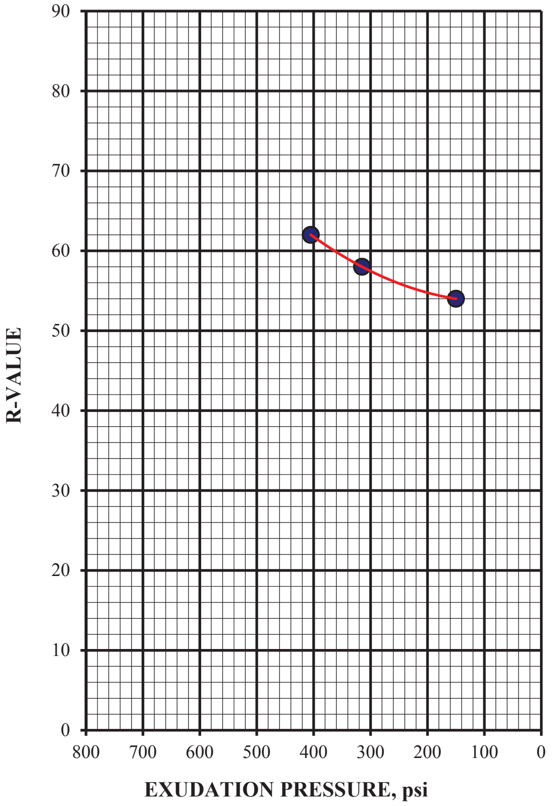
ASTM D 2844/D2844M-13

August 7, 2019

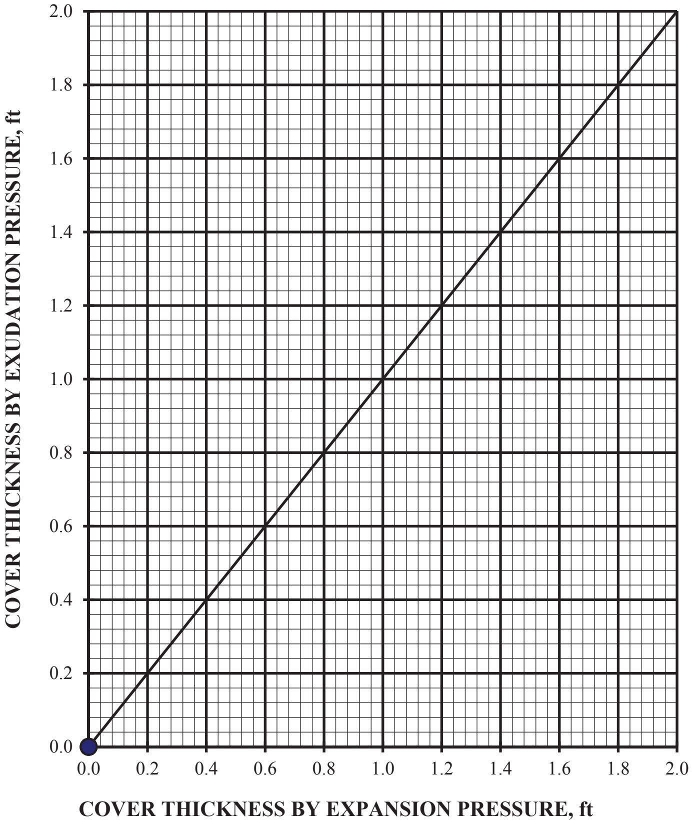
Boring #3 @ 0.0 - 5.0'
Light Brown Sandy Silt (ML)

Dry Density @ 300 psi Exudation Pressure: 117.8-pcf
%Moisture @ 300 psi Exudation Pressure: 12.3%
R-Value - Exudation Pressure: 57
R-Value - Expansion Pressure: N/A
R-Value @ Equilibrium: 57

EXUDATION PRESSURE CHART



EXPANSION PRESSURE CHART

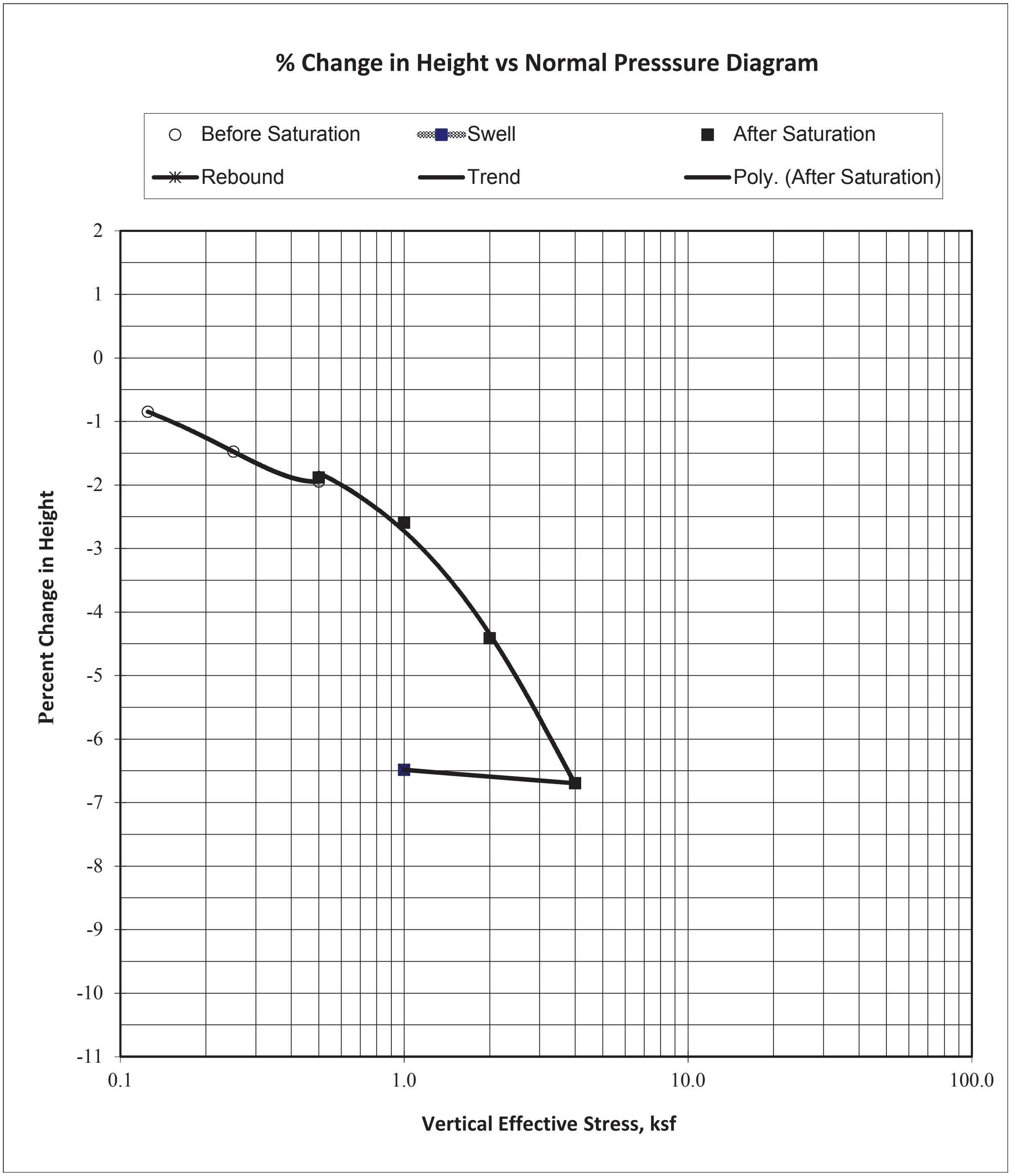


CONSOLIDATION TEST

ASTM D 2435-90

Channel Island High School Synthetic Turf Field
B 2 @ 5'
Clayey Sand
Ring Sample

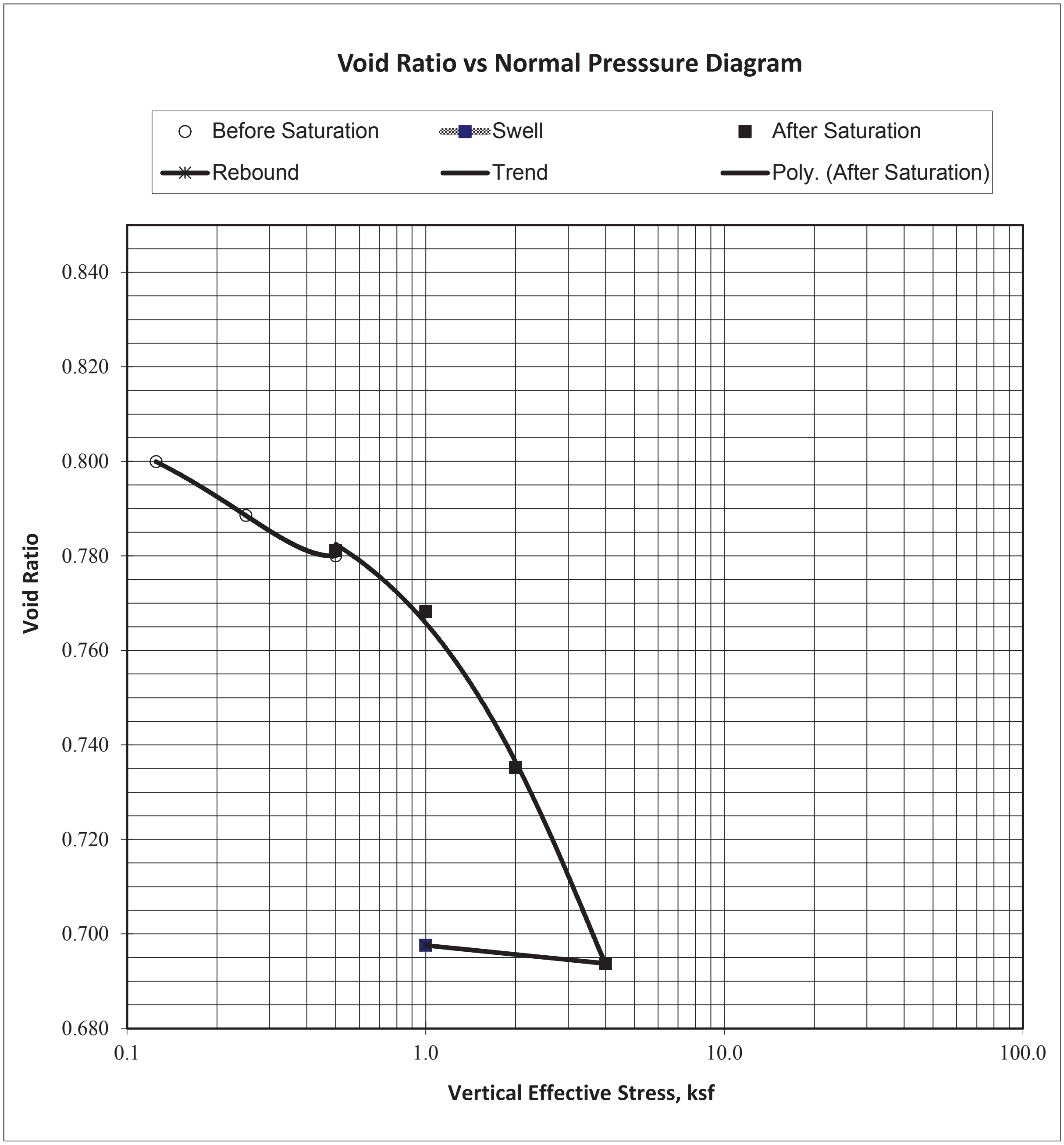
Initial Dry Density:	91.8 pcf
Initial Moisture, %:	31.5%
Specific Gravity:	2.67 (assume
Initial Void Ratio:	0.815



CONSOLIDATION TEST

ASTM D 2435-90

Channel Island High School Synthetic Turf Field		Initial Dry Density:	91.8
B 2 @ 5'		Initial Moisture, %:	31.5
Clayey Sand		Specific Gravity:	2.67 (assume
Ring Sample		Initial Void Ratio:	0.815



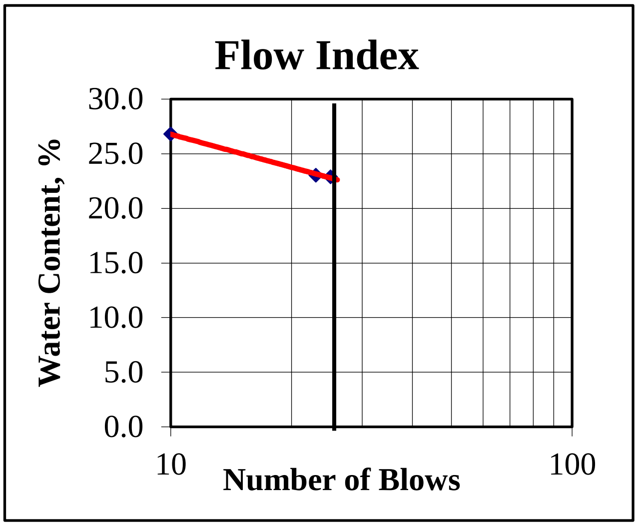
Earth Systems Pacific

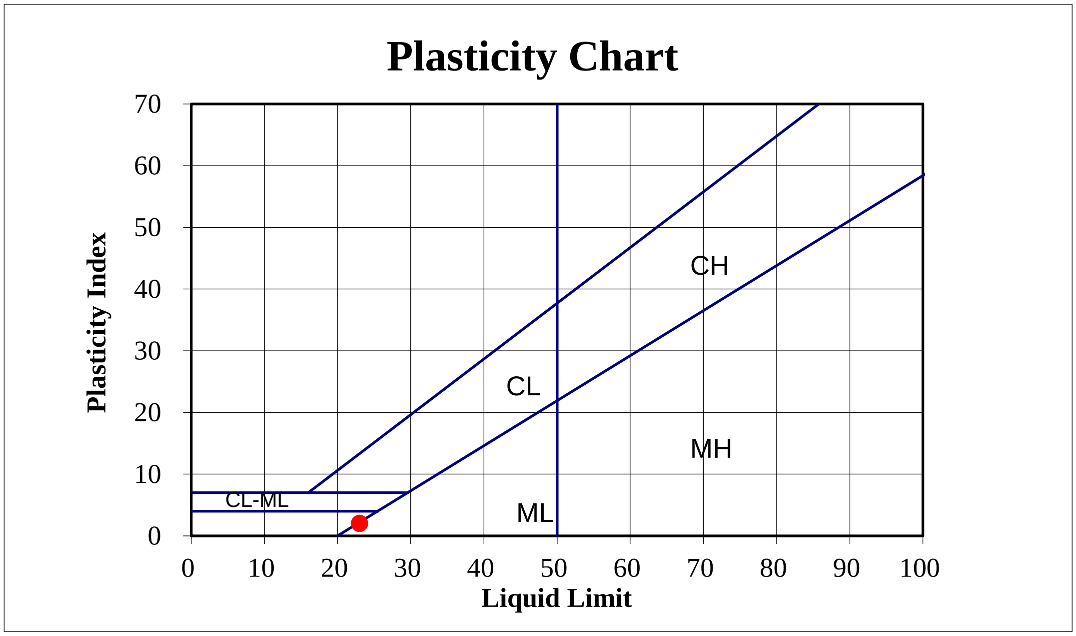
PLASTICITY INDEX

ASTM D-4318

Job Name: 3 High Schools (CIHS)
Sample ID: B 6 @ 25'
Soil Description: SM

DATA SUMMARY				TEST RESULTS	
Number of Blows:	10	23	25	**LIQUID LIMIT**	**23**
Water Content, %	26.8	23.0	22.9	**PLASTIC LIMIT**	**21**
Plastic Limit:	21.0	20.9		**PLASTICITY INDEX**	**2**



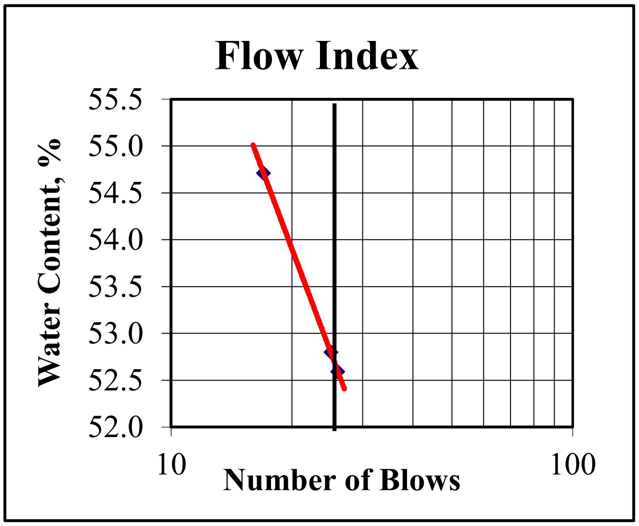


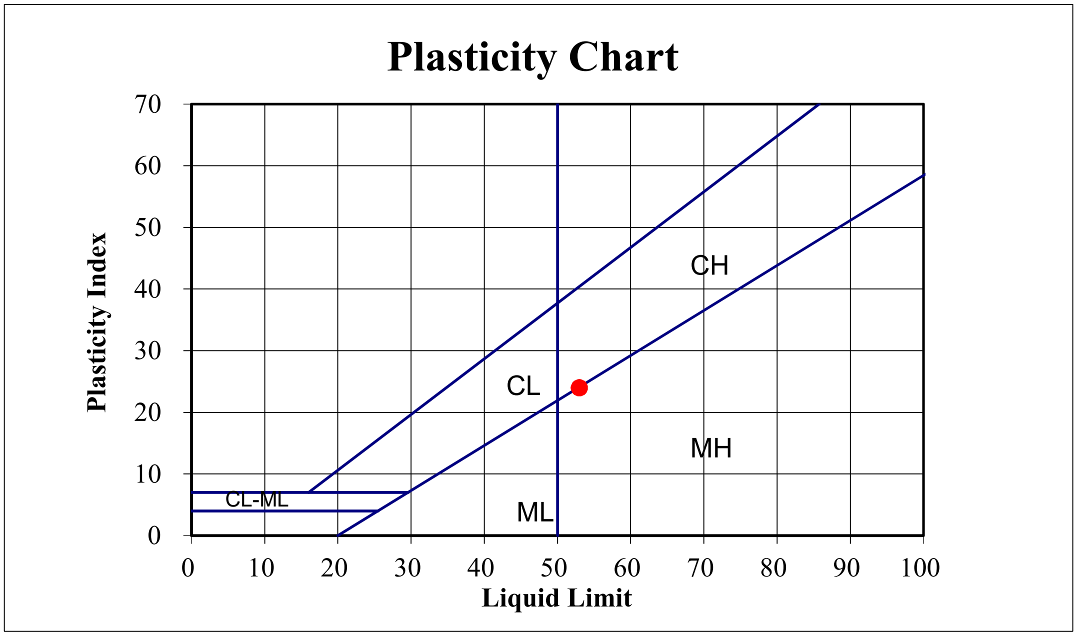
PLASTICITY INDEX

ASTM D-4318

Job Name: 3 High Schools (CIHS)
Sample ID: B 7 @ 7.5'
Soil Description: CH

DATA SUMMARY				**TEST RESULTS**	
Number of Blows:	17	25	26	**LIQUID LIMIT**	**53**
Water Content, %	54.7	52.8	52.6	**PLASTIC LIMIT**	**29**
Plastic Limit:	28.8	28.7		**PLASTICITY INDEX**	**24**





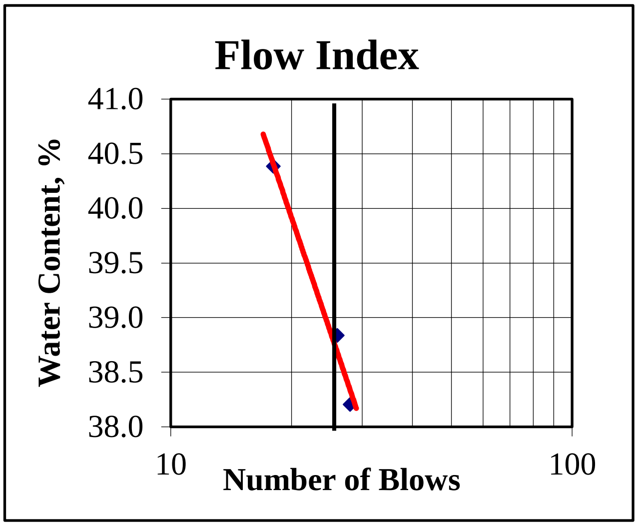
File No.: 303514-002 November 19, 2019

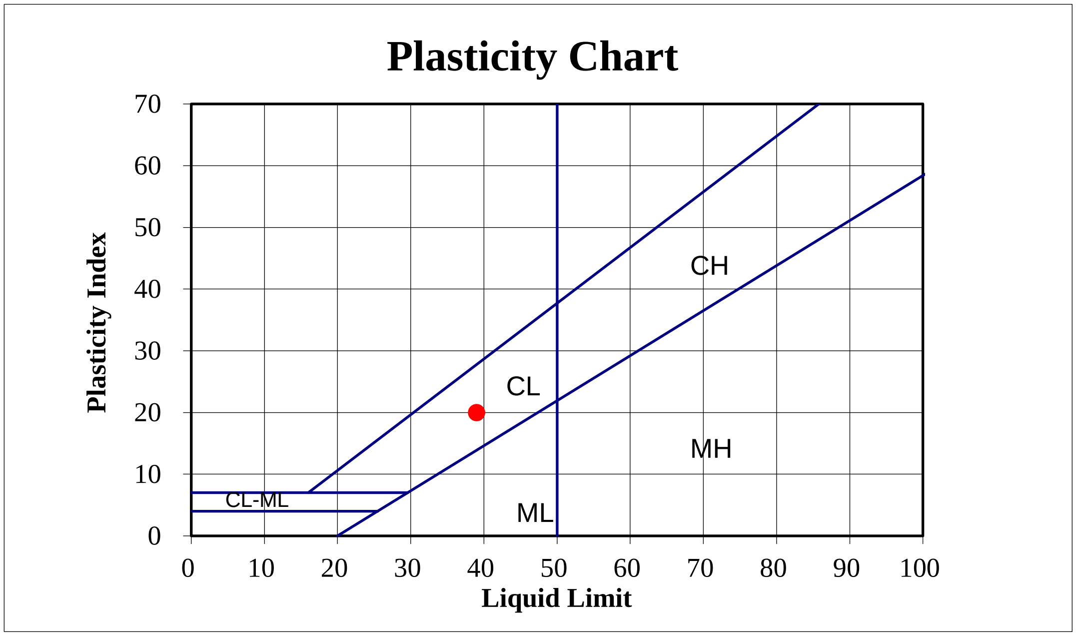
PLASTICITY INDEX

ASTM D-4318

Job Name: 3 High Schools (CIHS)
Sample ID: B 7 @ 25'
Soil Description: CL

DATA SUMMARY				TEST RESULTS	
Number of Blows:	18	26	28	**LIQUID LIMIT**	**39**
Water Content, %	40.4	38.8	38.2	**PLASTIC LIMIT**	**19**
Plastic Limit:	19.4	19.5		**PLASTICITY INDEX**	**20**





File No.: 303514-002 November 19, 2019

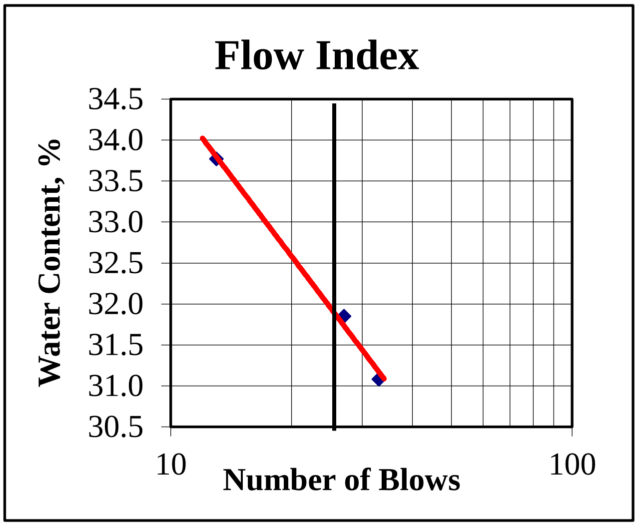
PLASTICITY INDEX

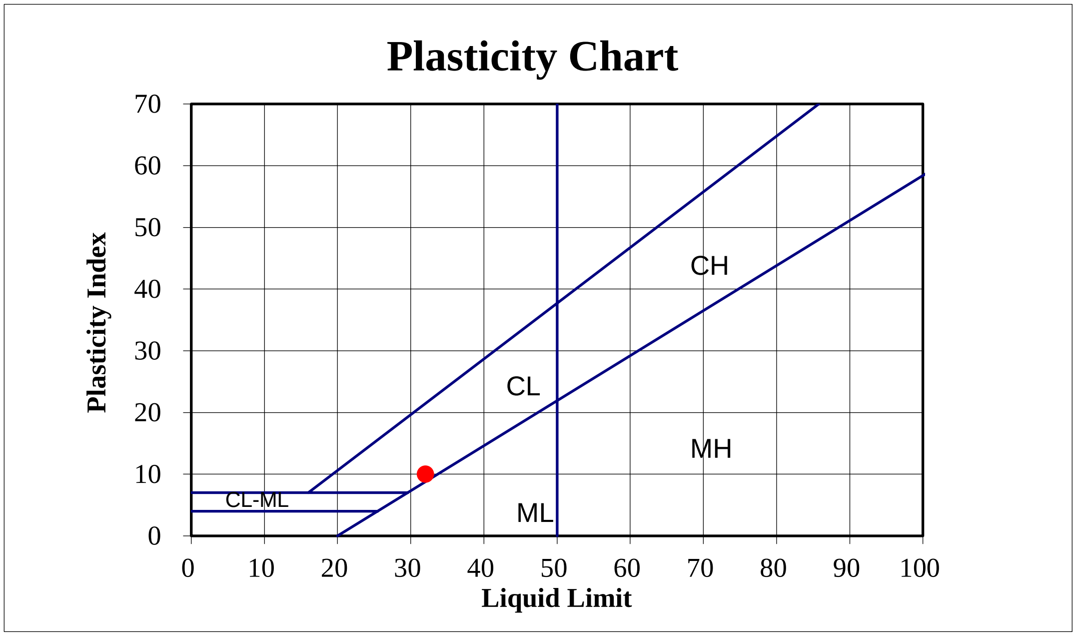
ASTM D-4318

Job Name: 3 High Schools (CIHS)
Sample ID: B 7 @ 32.5'
Soil Description: CL

DATA SUMMARY **TEST RESULTS**

Number of Blows:	13	27	33	**LIQUID LIMIT**	**32**
Water Content, %	33.8	31.9	31.1	**PLASTIC LIMIT**	**22**
Plastic Limit:	22.1	21.9		**PLASTICITY INDEX**	**10**



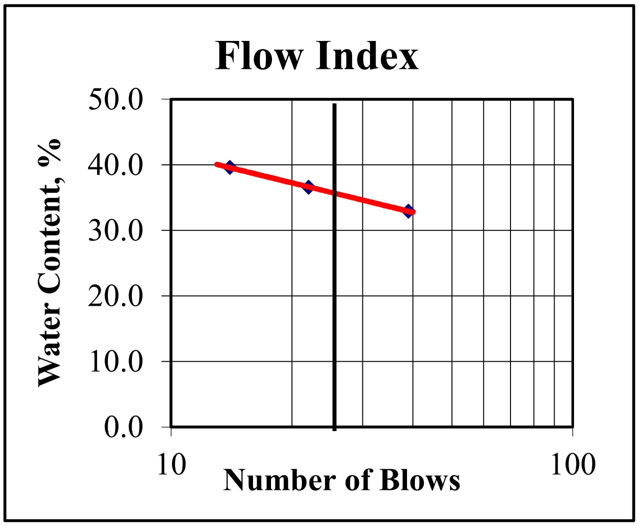


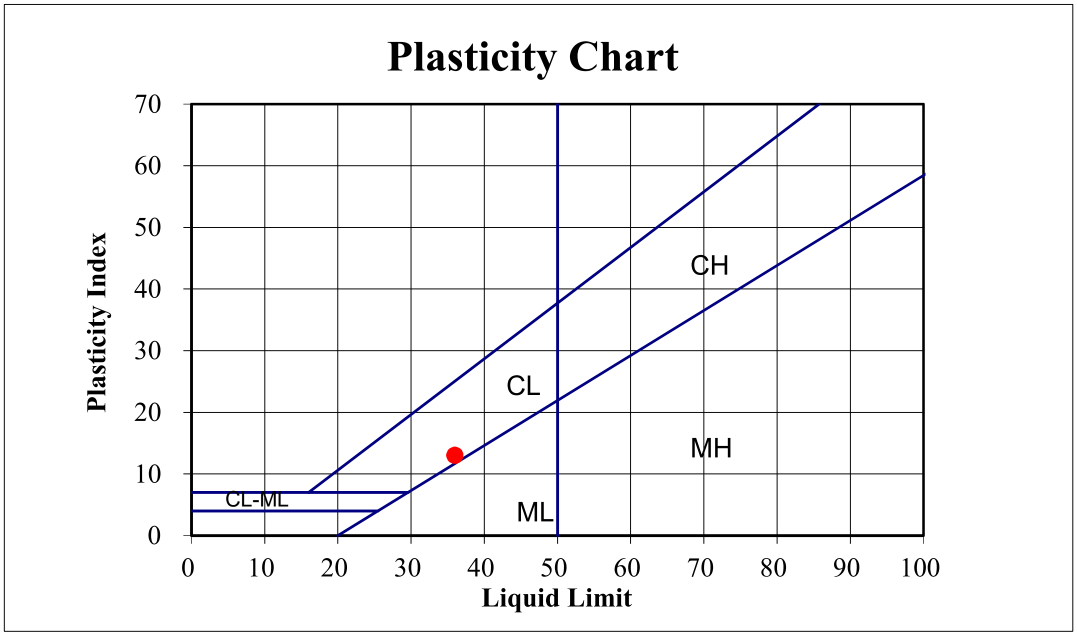
PLASTICITY INDEX

ASTM D-4318

Job Name: 3 High Schools (CIHS)
Sample ID: B 7 @ 47.5'
Soil Description: CL

DATA SUMMARY				TEST RESULTS	
Number of Blows:	14	22	39	**LIQUID LIMIT**	**36**
Water Content, %	39.6	36.6	32.9	**PLASTIC LIMIT**	**23**
Plastic Limit:	22.9	22.4		**PLASTICITY INDEX**	**13**





MECHANICAL ANALYSIS

CTM 203-08

Job Name: 3 High Schools (CIHS)
Job No.: 303514-002
Sample ID: **B 6 @ 7.5'**
Soil Description: **SM**
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:	374.4
Corrected Wt., g:	374.4

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	0.3	0.08	99.92
#10	0.4	0.11	99.89

Air Dry Hydro Sample Wt., g:	60.1
Corrected Wt., g:	60.1
Calculation Factor	0.6017

Hydrometer Analysis for <#10 Material

Start time: 9:52:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	9:52:20 AM	20	20	5.1	14.9
1 hour	10:52:00 AM	9	20	5.1	3.9
6 hour	3:52:00 PM	8	20	5.1	2.9

% Gravel:	0.0
% Sand(2mm - 74μm):	75.2
% Silt(74μm- 5μm):	18.3
% Clay(5μm - 2μm):	1.7
% Clay(≤2μm):	4.8

MECHANICAL ANALYSIS

CTM 203-08

Job Name: 3 High Schools (CIHS)
Job No.: 303514-002
Sample ID: **B 6 @ 25'**
Soil Description: **SM**
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:	456.9
Corrected Wt., g:	456.9

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:	60.1
Corrected Wt., g:	60.1
Calculation Factor	0.6010

Hydrometer Analysis for <#10 Material

Start time: 9:50:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	9:50:20 AM	31	20	5.1	25.9
1 hour	10:50:00 AM	17	20	5.1	11.9
6 hour	3:50:00 PM	13	20	5.1	7.9

% Gravel:	0.0
% Sand(2mm - 74µm):	56.9
% Silt(74µm- 5µm):	23.3
% Clay(5µm - 2µm):	6.7
% Clay(≤2µm):	13.1

MECHANICAL ANALYSIS

CTM 203-08

Job Name: 3 High Schools (CIHS)
Job No.: 303514-002
Sample ID: **B 6 @ 27.5'**
Soil Description: **ML**
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: 155.7
Corrected Wt., g: 155.7

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	0.2	0.13	99.87
#8	0.5	0.32	99.68
#10	0.5	0.32	99.68

Air Dry Hydro Sample Wt., g: 60.1
Corrected Wt., g: 60.1
Calculation Factor 0.6029

Hydrometer Analysis for <#10 Material

Start time: 9:36:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	9:36:20 AM	40	20	5.1	34.9
1 hour	10:36:00 AM	20	20	5.1	14.9
6 hour	3:36:00 PM	15	20	5.1	9.9

% Gravel:	0.1
% Sand(2mm - 74µm):	42.0
% Silt(74µm- 5µm):	33.2
% Clay(5µm - 2µm):	8.3
% Clay(≤2µm):	16.4

MECHANICAL ANALYSIS

CTM 203-08

Job Name: 3 High Schools (CIHS)
Job No.: 303514-002
Sample ID: **B 7 @ 7.5'**
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: 269
Corrected Wt., g: 269.0

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	0.2	0.07	99.93
#10	0.3	0.11	99.89

Air Dry Hydro Sample Wt., g: 60.1
Corrected Wt., g: 60.1
Calculation Factor 0.6017

Hydrometer Analysis for <#10 Material

Start time: 10:22:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	10:22:20 AM	59	20	5.1	53.9
1 hour	11:22:00 AM	33	20	5.1	27.9
6 hour	4:22:00 PM	23	20	5.1	17.9

% Gravel: 0.0
% Sand(2mm - 74µm): 10.4
% Silt(74µm- 5µm): 43.2
% Clay(5µm - 2µm): 16.7
% Clay(≤2µm): 29.7

MECHANICAL ANALYSIS

CTM 203-08

Job Name: 3 High Schools (CIHS)
Job No.: 303514-002
Sample ID: **B 7 @25'**
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:	274.4
Corrected Wt., g:	274.4

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	0.3	0.11	99.89
#10	0.3	0.11	99.89

Air Dry Hydro Sample Wt., g:	60
Corrected Wt., g:	60.0
Calculation Factor	0.6007

Hydrometer Analysis for <#10 Material

Start time: 9:36:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	9:36:20 AM	53	20	5.1	47.9
1 hour	10:36:00 AM	27	20	5.1	21.9
6 hour	3:36:00 PM	20	20	5.1	14.9

% Gravel:	0.0
% Sand(2mm - 74µm):	20.3
% Silt(74µm- 5µm):	43.2
% Clay(5µm - 2µm):	11.7
% Clay(≤2µm):	24.8

MECHANICAL ANALYSIS

CTM 203-08

Job Name: 3 High Schools (CIHS)
Job No.: 303514-002
Sample ID: **B 7 @32.5'**
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: 655
Corrected Wt., g: 655.0

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: 60
Corrected Wt., g: 60.0
Calculation Factor 0.6000

Hydrometer Analysis for <#10 Material

Start time: 9:43:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	9:43:20 AM	50	20	5.1	44.9
1 hour	10:43:00 AM	21	20	5.1	15.9
6 hour	3:43:00 PM	16	20	5.1	10.9

% Gravel:	0.0
% Sand(2mm - 74µm):	25.2
% Silt(74µm- 5µm):	48.3
% Clay(5µm - 2µm):	8.3
% Clay(≤2µm):	18.2

MECHANICAL ANALYSIS

CTM 203-08

Job Name:	3 High Schools (CIHS)
Job No.:	303514-002
Sample ID:	**B 7 @35'**
Soil Description:	**ML**
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:	448.8
Corrected Wt., g:	448.8

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	0.0	0.00	100.00
#10	0.0	0.00	100.00

Air Dry Hydro Sample Wt., g:	60
Corrected Wt., g:	60.0
Calculation Factor	0.6000

Hydrometer Analysis for <#10 Material

Start time: 9:43:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	9:43:20 AM	42	20	5.1	36.9
1 hour	10:43:00 AM	13	20	5.1	7.9
6 hour	3:43:00 PM	11	20	5.1	5.9

% Gravel:	0.0
% Sand(2mm - 74µm):	38.5
% Silt(74µm- 5µm):	48.3
% Clay(5µm - 2µm):	3.4
% Clay(≤2µm):	9.8

MECHANICAL ANALYSIS

CTM 203-08

Job Name: 3 High Schools (CIHS)
Job No.: 303514-002
Sample ID: **B 7 @42.5'**
Soil Description: **ML**
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: 422.2
Corrected Wt., g: 422.2

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: 60
Corrected Wt., g: 60.0
Calculation Factor 0.6000

Hydrometer Analysis for <#10 Material

Start time: 9:59:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	9:59:20 AM	59	20	5.1	53.9
1 hour	10:59:00 AM	20	20	5.1	14.9
6 hour	3:59:00 PM	15	20	5.1	9.9

% Gravel:	0.0
% Sand(2mm - 74µm):	10.2
% Silt(74µm- 5µm):	65.0
% Clay(5µm - 2µm):	8.3
% Clay(≤2µm):	16.5

MECHANICAL ANALYSIS

CTM 203-08

Job Name: 3 High Schools (CIHS)
Job No.: 303514-002
Sample ID: **B 7 @47.5'**
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: 588.8
Corrected Wt., g: 588.8

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	0.0	0.00	100.00
#10	0.0	0.00	100.00

Air Dry Hydro Sample Wt., g: 60
Corrected Wt., g: 60.0
Calculation Factor 0.6000

Hydrometer Analysis for <#10 Material

Start time: 9:27:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	9:27:20 AM	58	20	5.1	52.9
1 hour	10:27:00 AM	23	20	5.1	17.9
6 hour	3:27:00 PM	16	20	5.1	10.9

% Gravel:	0.0
% Sand(2mm - 74µm):	11.8
% Silt(74µm- 5µm):	58.4
% Clay(5µm - 2µm):	11.6
% Clay(≤2µm):	18.2

CAPCO Analytical Services, Inc.

Environmental and Analytical Services-Since 1994
California State Accredited Laboratory in Accordance with ELAP Certificate # 2332

CERTIFICATE OF ANALYSIS

Client: Earth Systems Pacific
CAS LAB NO: 191342-01
Sample ID: B2@0-5'
Analyst: GP

Date Sampled: 07/29/19
Date Received: 07/29/19
Sample Matrix: Soil

WET CHEMISTRY SUMMARY

COMPOUND	RESULTS	UNITS	DF	PQL	METHOD	ANALYZED
pH (Corrosivity)	8.0	S.U.	1	---	9045	08/01/19
Resistivity*	2200	Ohms-cm	1	---	SM 120.1M	08/01/19
Chloride	10	mg/Kg	1	0.3	300.0M	08/01/19
Sulfate	510	mg/Kg	1	0.3	300.0M	08/01/19

*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)

TABLE 18-I-D

MINIMUM FOUNDATION REQUIREMENTS

(Numbers within parenthesis ( ) are footnotes.
Refer to the following pages footnotes (1) through (8)

<table>
<tr><th rowspan="4">WEIGHTED EXPANSION INDEX</th><th colspan="7">FOUNDATIONS FOR SLAB AND RAISED FLOOR SYSTEM (4) (5)</th><th colspan="2">CONCRETE SLABS</th><th rowspan="4">PREMOISTENING OF SOILS UNDER FOOTINGS, PIERS AND SLABS (1)</th><th rowspan="4">RESTRICTIONS ON PIERS UNDER RAISED FLOORS
A design by a registered structural engineer may be excepted when approved by the Building Official</th></tr>
<tr><th rowspan="3">NUMBER OF FLOORS</th><th rowspan="2">STEM THICKNESS</th><th rowspan="2">FOOTING WIDTH</th><th rowspan="2">FOOTING THICKNESS</th><th>ALL PERIMETER FOOTINGS (5)</th><th>INTERIOR FOOTINGS FOR SLAB AND RAISED FLOORS (5)</th><th rowspan="3">REINFORCEMENT FOR CONTINUOUS FOUNDATIONS (2)</th><th colspan="2">3 ½ “ MINIMUM THICKNESS</th></tr>
<tr><th colspan="2">DEPTH BELOW NATURAL SURFACE OF GROUND AND FINISH GRADE (3) (8)</th><th rowspan="2">REINFORCEMENT (3)</th><th rowspan="2">TOTAL THICKNESS OF SAND</th></tr>
<tr><th colspan="5">INCHES</th></tr>
<tr><td>0-20
Very low
(nonexpansive)</td><td>1
2
3</td><td>8
8
10</td><td>12
15
18</td><td>8
7
8</td><td>12
18
24</td><td>12
18
24</td><td>1-#4 top and bottom</td><td>6x6-10/10 WWF</td><td>2”</td><td>Moistening of ground recommended prior to placing concrete.</td><td>Piers allowed for single floor loads only</td></tr>
<tr><td>21-50
Low</td><td>1
2
3</td><td>8
8
10</td><td>12
15
18</td><td>6
7
8</td><td>15
18
24</td><td>12
18
24</td><td>1-#4 top and bottom</td><td>6x6-10/10 WWF</td><td>4”</td><td>120% of optimum moisture required to a depth of 21” below lowest adjacent grade. Testing required.</td><td>Piers allowed for single floor loads only.</td></tr>
<tr><td rowspan="2">51-90
Medium</td><td rowspan="2">1
2
3</td><td rowspan="2">8
8
10</td><td rowspan="2">12
15
18</td><td rowspan="2">8
8
8</td><td rowspan="2">21
21
24</td><td rowspan="2">12
18
24</td><td>1-#4 top and bottom</td><td>6x6-10/10 WWF</td><td rowspan="2">4”</td><td rowspan="2">130% of optimum moisture required to a depth of 27” below lowest adjacent grade. Testing required.</td><td rowspan="2">Piers not allowed.</td></tr>
<tr><td colspan="2">#3 BARS @ 24” IN EXT. FOOTING BEND3’ INTO SLAB (7)</td></tr>
<tr><td rowspan="2">91-130
High</td><td rowspan="2">1
2
3</td><td rowspan="2">8
8
10</td><td rowspan="2">12
15
18</td><td rowspan="2">8
8
8</td><td rowspan="2">27
27
24</td><td rowspan="2">12
18
24</td><td>1-#5 top and bottom</td><td>6x6-10/10 or #3 @ 24’ E.W.</td><td rowspan="2">4”</td><td rowspan="2">140% of optimum moisture required of a depth of 33” below lowest adjacent grade. Testing required</td><td rowspan="2">Piers not allowed.</td></tr>
<tr><td colspan="2">#3 BARS @ 24” IN EXT. FOOTING BEND 3’ INTO SLAB (7)</td></tr>
<tr><td>Above 130
Very High</td><td colspan="11">Special design by licensed engineer/architect</td></tr>
</table>

APPENDIX C

Site Classification
2016 CBC & ASCE 7-10 Seismic Parameters
US Seismic Design Maps
Spectral Response Values
Spectral Response Curves
Fault Parameters

EARTH SYSTEMS

Job Number: 303514-002
Job Name: Channel Islands HS Athletic Field Imp
Calc Date: 11/19/2019
CPT/Boring ID: B-7

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

↓

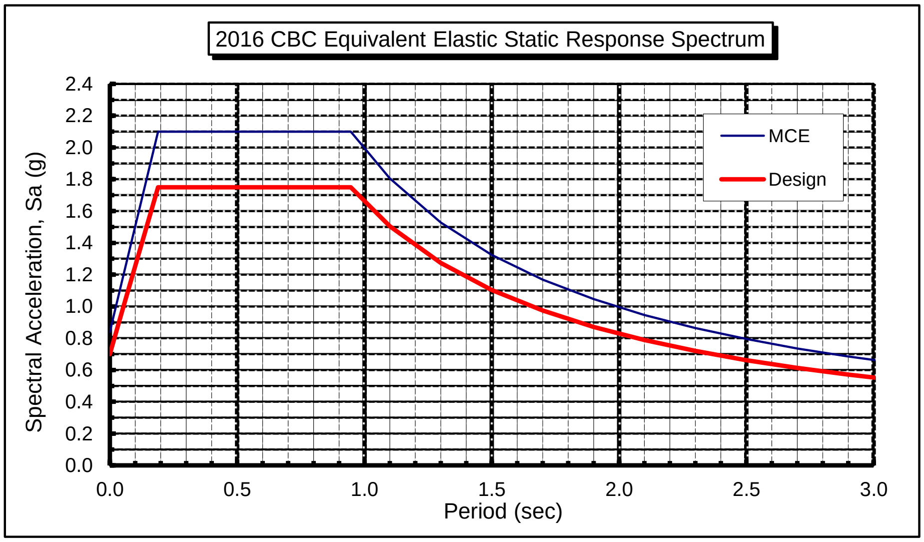
Depth (ft)	SPT N	Sublayer Thick (ft)	Sublayer Thick/N
9.5	2.0	9.5	4.750
10.0	2.0	0.5	0.250
12.0	2.0	2.0	1.000
14.5	23.0	2.5	0.109
17.0	27.0	2.5	0.093
19.5	32.0	2.5	0.078
22.0	20.0	2.5	0.125
24.5	14.0	2.5	0.179
27.0	5.0	2.5	0.500
29.5	18.0	2.5	0.139
32.0	32.0	2.5	0.078
34.5	9.0	2.5	0.278
37.0	21.0	2.5	0.119
39.5	31.0	2.5	0.081
42.0	40.0	2.5	0.063
44.5	19.0	2.5	0.132
47.0	20.0	2.5	0.125
49.5	9.0	2.5	0.278
51.5	7.0	2.0	0.286
100.0	20.0	48.5	2.425

Total Thickness of Soil =	100.00	ft
N-bar Value =	9.0	*
Site Classification =	Class E	

*Equation 20.4-2 of ASCE 7-10

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

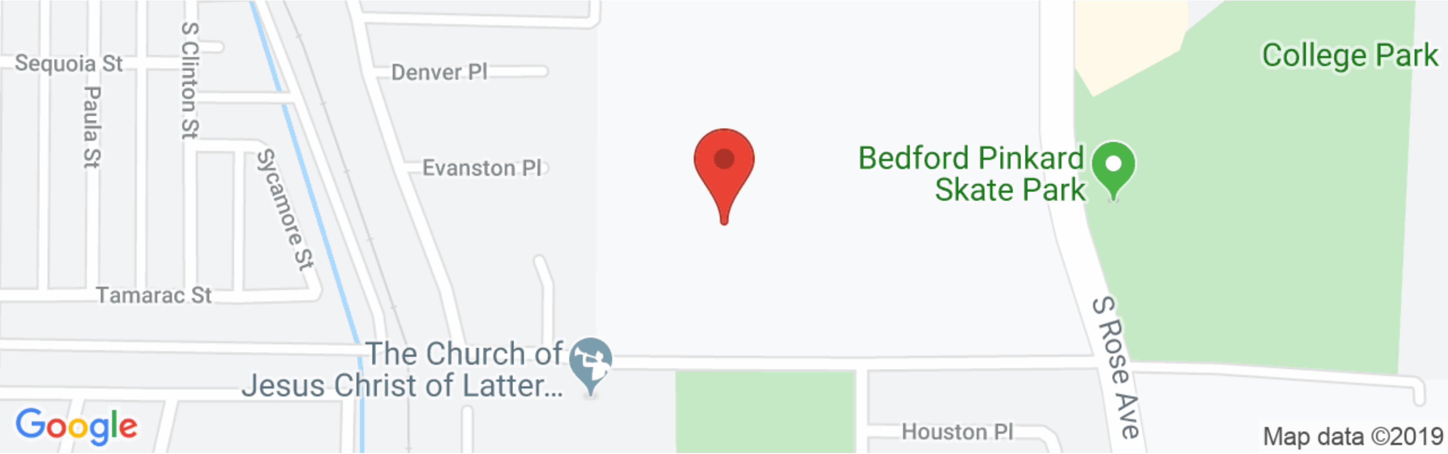
				CBC Reference	ASCE 7-10 Reference
Seismic Design Category		**E**		Table 1613.5.6	Table 11.6-2
Site Class		**E**		Table 1613.5.2	Table 20.3-1
Latitude:		34.169	N		
Longitude:		-119.163	W		
Maximum Considered Earthquake (MCE) Ground Motion					
Short Period Spectral Reponse	S_S	**2.333**	**g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	S_1	**0.828**	**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}	2.100	g	$= F_a*S_S$	
	S_{M1}	1.987	g	$= F_v*S_1$	
Design Earthquake Ground Motion					
Short Period Spectral Reponse	S_{DS}	**1.400**	**g**	$= 2/3*S_{MS}$	
1 second Spectral Response	S_{D1}	**1.325**	**g**	$= 2/3*S_{M1}$	
	To	0.19	sec	$= 0.2*S_{D1}/S_{DS}$	
	Ts	0.95	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	0.700
0.05	0.977
0.19	1.750
0.95	1.750
1.10	1.505
1.30	1.274
1.50	1.104
1.70	0.974
1.90	0.872
2.10	0.789
2.30	0.720
2.50	0.662
2.70	0.613
2.90	0.571
3.10	0.534
3.30	0.502

OSHPD

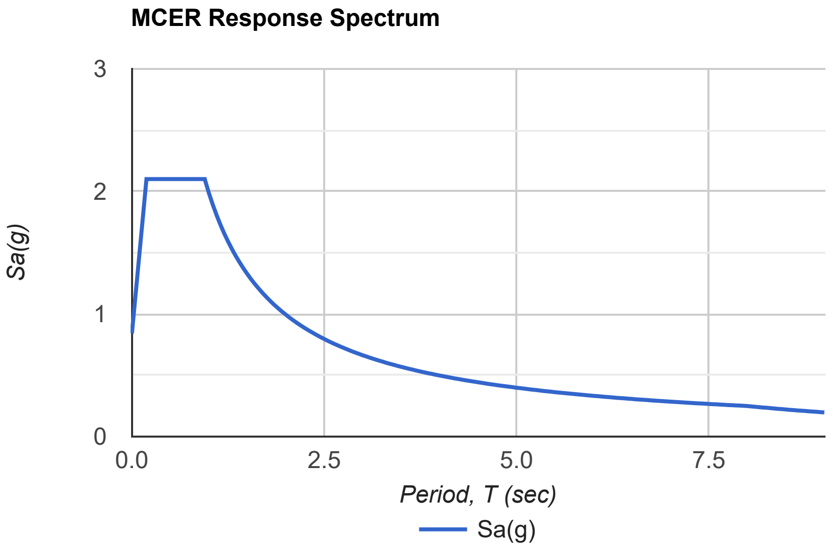
Latitude, Longitude: 34.169050, -119.163249



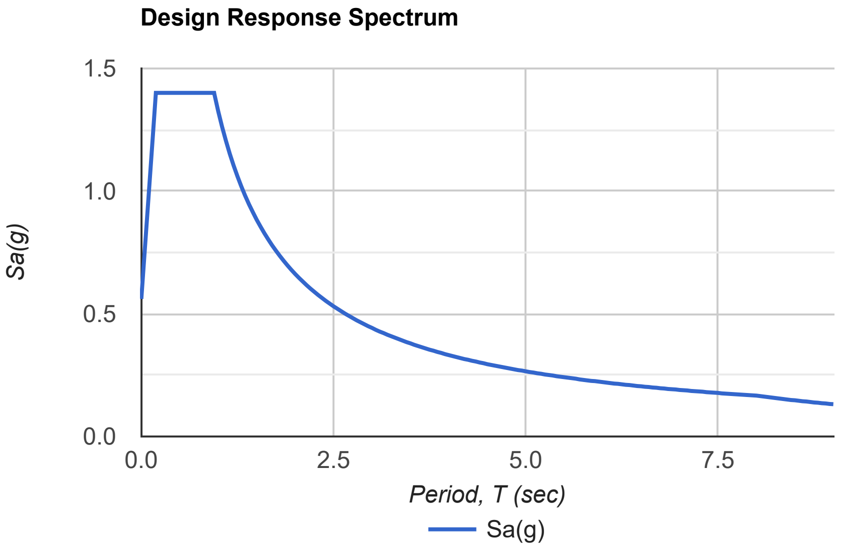
Date	11/19/2019, 5:05:16 PM
Design Code Reference Document	ASCE7-10
Risk Category	III
Site Class	E - Soft Clay Soil

Type	Value	Description
S_S	2.333	MCE_R ground motion. (for 0.2 second period)
S_1	0.828	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.1	Site-modified spectral acceleration value
S_{M1}	1.987	Site-modified spectral acceleration value
S_{DS}	1.4	Numeric seismic design value at 0.2 second SA
S_{D1}	1.325	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	0.882	MCE_G peak ground acceleration
F_{PGA}	0.9	Site amplification factor at PGA
PGA_M	0.794	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.333	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.515	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.621	Factored deterministic acceleration value. (0.2 second)
S1RT	0.828	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.885	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.882	Factored deterministic acceleration value. (1.0 second)
PGAd	0.973	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.928	Mapped value of the risk coefficient at short periods
C_{R1}	0.936	Mapped value of the risk coefficient at a period of 1 s



MCER Response Spectrum



Design Response Spectrum

DISCLAIMER

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	Max Rotated Probab. 2% in 50 yr MCEr	Max 84th Percentile Determ. MCE Spectrum	Determ. Lower Limit MCE Spectrum	Determ. MCE Spectrum	**Site Specific MCE Spectrum**	**2016 CBC MCE Spectrum**	Site Specific Design Spectrum	2016 CBC Design Spectrum
	(1) 2475-yr	(2) 2475-yr	(3)	(4)	(5) max(3,4)	(6) min(2.5)	(7)	(8) 2/3*(6)*	(9) 2/3*(7)
0.00	0.694	0.708	0.649	0.540	0.649	**0.649**	**0.840**	0.448	0.560
0.05	0.889	0.908	0.724	0.730	0.730	**0.730**	**1.173**	0.625	0.782
0.10	1.085	1.108	0.946	0.920	0.946	**0.946**	**1.505**	0.803	1.004
0.15	1.269	1.295	1.147	1.110	1.147	**1.147**	**1.838**	0.980	1.225
0.20	1.452	1.482	1.213	1.299	1.299	**1.299**	**2.100**	1.120	1.400
0.30	1.557	1.591	1.299	1.350	1.350	**1.350**	**2.100**	1.120	1.400
0.40	1.536	1.643	1.364	1.350	1.364	**1.364**	**2.100**	1.120	1.400
0.50	1.515	1.693	1.473	1.350	1.473	**1.473**	**2.100**	1.120	1.400
0.75	1.355	1.581	1.613	1.350	1.613	**1.581**	**2.100**	1.120	1.400
1.00	1.194	1.453	1.589	1.350	1.589	**1.453**	**1.987**	1.060	1.325
1.50	0.967	1.176	1.491	0.960	1.491	**1.176**	**1.325**	0.784	0.883
2.00	0.739	0.899	1.332	0.720	1.332	**0.899**	**0.994**	0.599	0.662

Crs: 0.928
Cr1: 0.936

* > 80% of (9)

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

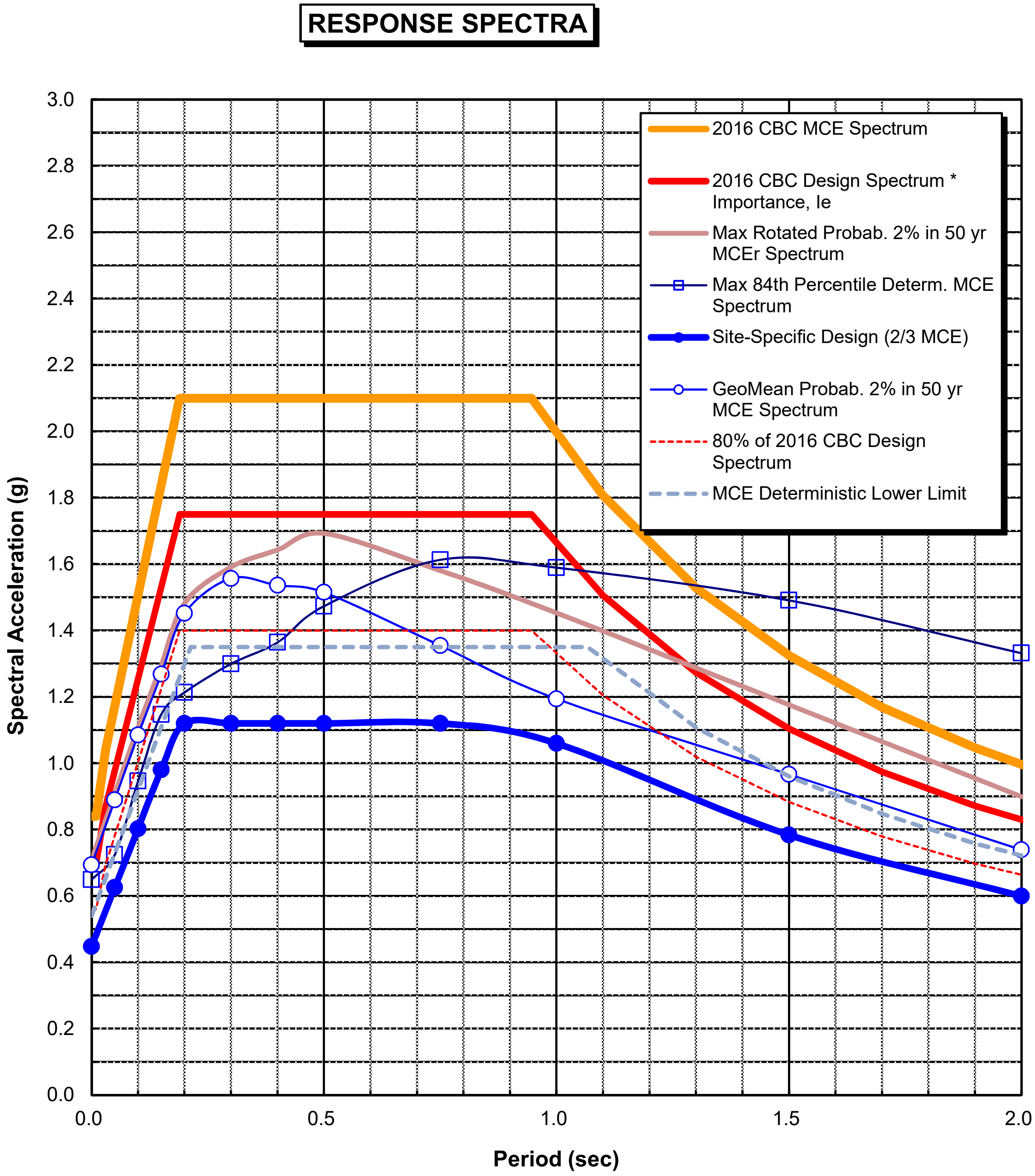
Mapped MCE Acceleration Values			Site Coefficients		**Site-Specific Design Acceleration Values**		
PGA	0.882	g	F_{PGA}	0.90	**PGA_M**	**0.794**	**g**
Ss	2.333	g	F_a	0.90	**S_{DS}**	**1.120**	**g**
S_1	0.828	g	F_v	2.40	**S_{D1}**	**1.199**	**g**

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



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

Site Class: E
Latitude: 34.16905
Longitude: -119.163249

Spectral Response Curves

Channel Islands High School Athletic Field Improvements
File No.: 303514-002

Earth Systems

Table 1
Fault Parameters

Fault Section Name	**Distance**		Avg Dip Angle	Avg Dip Direction	Avg Rake	Trace Length	Fault Type	**Mean Mag**	Mean Return Interval	Slip Rate
	(miles)	(km)	(deg.)	(deg.)	(deg.)	(km)			(years)	(mm/yr)
Oak Ridge (Onshore)	2.0	3.2	65	159	90	49	B	**7.4**		4
Simi-Santa Rosa	4.9	7.9	60	346	30	39	B	**6.8**		1
Malibu Coast (Extension), alt 1	7.2	11.5	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	7.2	11.5	74	4	30	35	B'	**6.9**		
Oak Ridge (Offshore)	8.5	13.6	32	180	90	38	B	**6.9**		3
Ventura-Pitas Point	9.1	14.7	64	353	60	44	B	**6.9**		1
Channel Islands Thrust	11.4	18.4	20	354	90	59	B	**7.3**		1.5
Anacapa-Dume, alt 1	13.5	21.7	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	13.5	21.7	41	352	60	65	B	**7.2**		3
Santa Cruz Island	14.0	22.5	90	188	30	69	B	**7.1**		1
Red Mountain	14.2	22.8	56	2	90	101	B	**7.4**		2
Channel Islands Western Deep Ramp	15.5	25.0	21	204	90	62	B'	**7.3**		
Malibu Coast, alt 1	15.7	25.2	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	15.7	25.2	74	3	30	38	B	**6.9**		0.3
Sisar	17.0	27.4	29	168	na	20	B'	**7.0**		
Pitas Point (Lower)-Montalvo	18.0	28.9	16	359	90	30	B	**7.3**		2.5
North Channel	18.2	29.2	26	10	90	51	B	**6.7**		1
Shelf (Projection)	18.4	29.5	17	21	na	70	B'	**7.8**		
San Cayetano	18.5	29.8	42	3	90	42	B	**7.2**		6
Mission Ridge-Arroyo Parida-Santa Ana	19.3	31.1	70	176	90	69	B	**6.8**		0.4
Santa Cruz Catalina Ridge	22.2	35.8	90	38	na	137	B'	**7.3**		
Santa Monica Bay	24.8	39.9	20	44	na	17	B'	**7.0**		
Santa Ynez (East)	25.0	40.2	70	172	0	68	B	**7.2**		2
Pitas Point (Upper)	25.8	41.5	42	15	90	35	B	**6.8**		1
Santa Susana, alt 1	26.1	42.1	55	9	90	27	B	**6.8**		5
San Pedro Basin	26.2	42.2	88	51	na	69	B'	**7.0**		
Santa Susana, alt 2	26.4	42.5	53	10	90	43	B'	**6.8**		
Northridge Hills	27.7	44.5	31	19	90	25	B'	**7.0**		
Pine Mtn	28.2	45.4	45	5	na	62	B'	**7.3**		
Del Valle	29.5	47.5	73	195	90	9	B'	**6.3**		
Oak Ridge (Offshore), west extension	29.9	48.1	67	195	na	28	B'	**6.1**		
Holser, alt 1	29.9	48.1	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	29.9	48.1	58	182	90	17	B'	**6.7**		
Northridge	30.9	49.8	35	201	90	33	B	**6.8**		1.5
Compton	32.9	52.9	20	34	90	65	B'	**7.5**		
San Pedro Escarpment	33.7	54.2	17	38	na	27	B'	**7.3**		
Pitas Point (Lower, West)	35.3	56.7	13	3	90	35	B	**7.2**		2.5
Santa Ynez (West)	35.3	56.8	70	182	0	63	B	**6.9**		2
Santa Monica, alt 1	36.1	58.0	75	343	30	14	B	**6.5**		1
Big Pine (Central)	36.3	58.4	76	167	na	23	B'	**6.3**		

Reference: USGS OFR 2007-1437 (CGS SP 203)

Based on Site Coordinates of 34.16905 Latitude, -119.163249 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

Liquefaction Analysis Calculations
Liquefaction Analysis Graphs
Lateral Spreading Analysis Printouts

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: Channel Islands HS Gateways
Job No: 303514-002
Date: 11/26/2019
Boring: B-7 Data Set: 2

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:

Magnitude: 7.4 7.5
PGA, g: 0.79 0.77
MSF: 1.03
GWT: 10.0 feet
Calc GWT: 10.0 feet
Remediate to: 0.0 **feet**

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E): 1.33 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: 0.33

Total (ft) Liquefied Thickness: **2.5**

Total (in.) Induced Subsidence: **0.4**

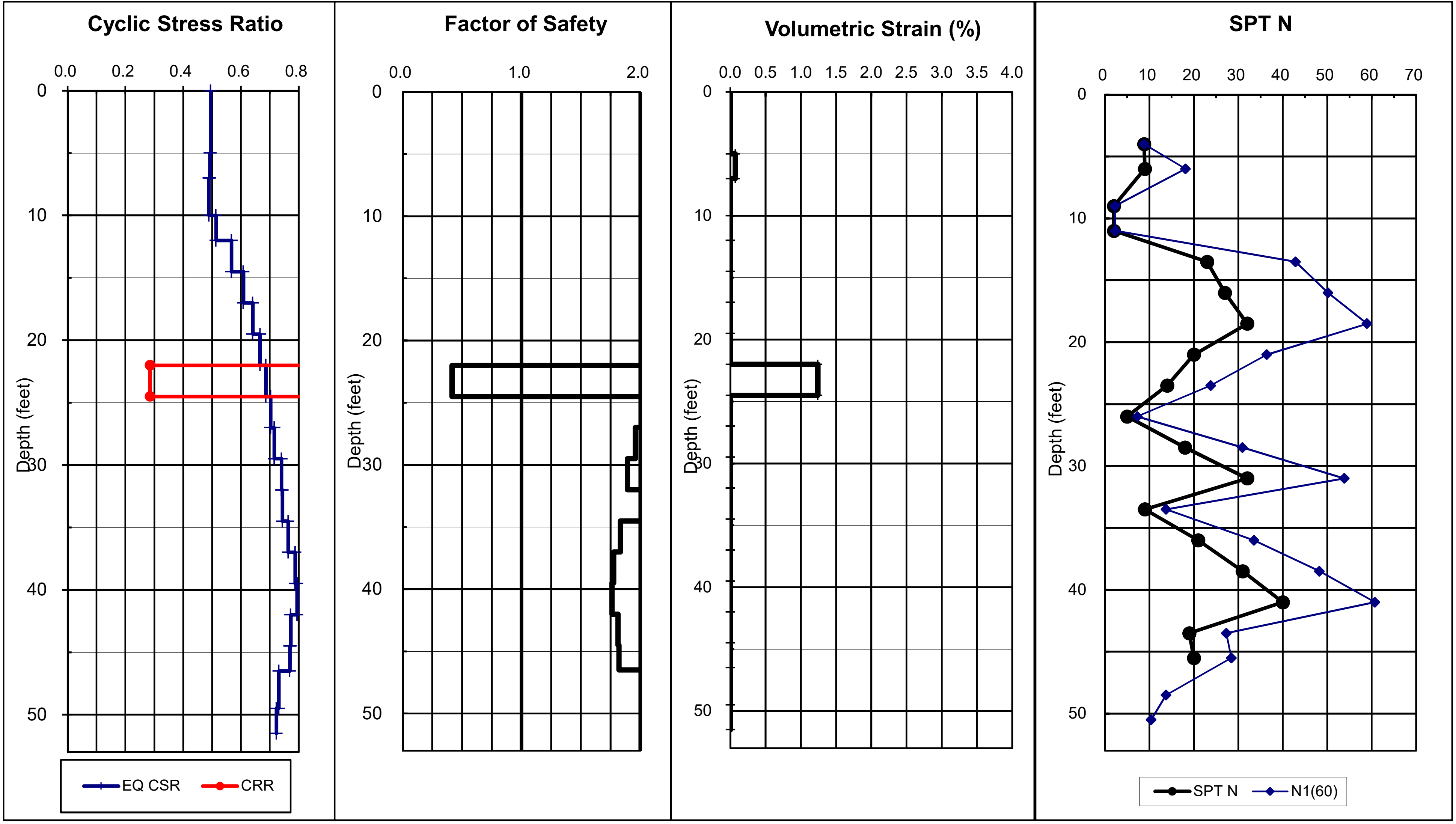
Required SF: 1.30
Minimum Calculated SF: 0.42

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain* *(%)*	**Induced Subsidence (in.)**
0.0				0				0.000																	
5.0	14	**9**	0	110	90	4.0	7.0	0.220	0.220	0.99	1.00	0.75	1.00	8.8				1.00	Infin.	0.495	**Non-Liq.**		8.8	0.00	**0.00**
7.0	0	**9**	1	110	90	6.0	9.0	0.330	0.330	0.99	1.70	0.75	1.18	18.1	51	8.6	26.7	1.00	0.314	0.493	**Non-Liq.**	8.6	26.7	0.07	**0.02**
10.0	0	**2**	0	110	90	9.0	12.0	0.495	0.495	0.98	1.00	0.75	1.10	2.2				1.00	Infin.	0.490	**Non-Liq.**		2.2	0.00	**0.00**
12.0	0	**2**	0	110	90	11.0	14.0	0.605	0.574	0.98	1.00	0.78	1.10	2.3				1.00	Infin.	0.514	**Non-Liq.**		2.3	0.00	**0.00**
14.5	0	**23**	1	120	10	13.5	16.5	0.750	0.641	0.97	1.28	0.84	1.30	42.9	78	1.8	44.7	1.00	1.400	0.567	**2.47**	1.8	44.7	0.00	**0.00**
17.0	0	**27**	1	120	10	16.0	19.0	0.900	0.713	0.97	1.22	0.88	1.30	50.2	85	2.0	52.1	1.00	1.400	0.609	**2.30**	2.0	52.1	0.00	**0.00**
19.5	0	**32**	1	120	10	18.5	21.5	1.050	0.785	0.96	1.16	0.92	1.30	58.9	92	2.1	61.1	1.00	1.400	0.641	**2.18**	2.1	61.1	0.00	**0.00**
22.0	0	**20**	1	120	10	21.0	24.0	1.200	0.857	0.95	1.11	0.94	1.30	36.4	72	1.7	38.1	1.00	1.400	0.667	**2.10**	1.7	38.1	0.00	**0.00**
24.5	0	**14**	1	120	10	23.5	26.5	1.350	0.929	0.95	1.07	0.97	1.23	23.8	58	1.4	25.2	1.00	0.286	0.687	**0.42**	1.0	24.8	1.24	**0.37**
27.0	0	**5**	0	110	80	26.0	29.0	1.493	0.993	0.94	1.00	0.99	1.10	7.3				1.00	Infin.	0.703	**Non-Liq.**		7.3	0.00	**0.00**
29.5	0	**18**	1	115	25	28.5	31.5	1.634	1.057	0.93	1.00	1.00	1.29	30.9	66	7.8	38.8	1.00	1.400	0.715	**1.96**	7.8	38.8	0.00	**0.00**
32.0	0	**32**	1	115	25	31.0	34.0	1.778	1.122	0.92	0.97	1.00	1.30	53.9	88	10.0	63.9	0.98	1.400	0.740	**1.89**	10.0	63.9	0.00	**0.00**
34.5	0	**9**	0	110	75	33.5	36.5	1.918	1.184	0.90	1.00	1.00	1.14	13.7				0.98	Infin.	0.744	**Non-Liq.**		13.7	0.00	**0.00**
37.0	0	**21**	1	115	62	36.0	39.0	2.059	1.248	0.88	0.92	1.00	1.30	33.5	69	10.0	43.5	0.95	1.400	0.764	**1.83**	10.0	43.5	0.00	**0.00**
39.5	0	**31**	1	115	55	38.5	41.5	2.203	1.313	0.86	0.90	1.00	1.30	48.2	83	10.0	58.2	0.92	1.400	0.788	**1.78**	10.0	58.2	0.00	**0.00**
42.0		**40**	1	115	55	41.0	44.0	2.346	1.379	0.84	0.88	1.00	1.30	60.7	93	10.0	70.7	0.90	1.400	0.795	**1.76**	10.0	70.7	0.00	**0.00**
44.5		**19**	1	115	90	43.5	46.5	2.490	1.445	0.82	0.86	1.00	1.26	27.3	62	10.0	37.3	0.91	1.400	0.773	**1.81**	10.0	37.3	0.00	**0.00**
46.5		**20**	1	115	25.0	45.5	48.5	2.605	1.497	0.80	0.84	1.00	1.27	28.4	64	7.6	36.0	0.90	1.400	0.769	**1.82**	7.6	36.0	0.00	**0.00**
49.5		**9**	0	110	88.2	48.5	51.5	2.773	1.571	0.77	1.00	1.00	1.14	13.7				0.92	Infin.	0.732	**Non-Liq.**		13.7	0.00	**0.00**
51.5		**7**	0	110	70.0	50.5	53.5	2.883	1.619	0.75	1.00	1.00	1.11	10.4				0.92	Infin.	0.723	**Non-Liq.**		10.4	0.00	**0.00**

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Channel Islands HS Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-7 **Earthquake Magnitude: 7.4** **PGA, g: 0.79** **Calc GWT (feet): 10**



Total Thickness of Liquefiable Layers: 2.5 feet

Estimated Total Ground Subsidence: 0.4 inches

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: **Channel Islands HS Gateways**
Job No: **303514-002**
Date: **11/26/2019**
Boring: **B-7** Data Set: 2

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: **7.4** 7.5
PGA, g: **0.79** 0.77
MSF: 1.03
GWT: **10.0** feet
Calc GWT: **5.0** feet
Remediate to: **0.0** **feet**

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.33 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: **0.33**

Total (ft) Liquefied Thickness **4.5**

Total (in.) Induced Subsidence **0.6**

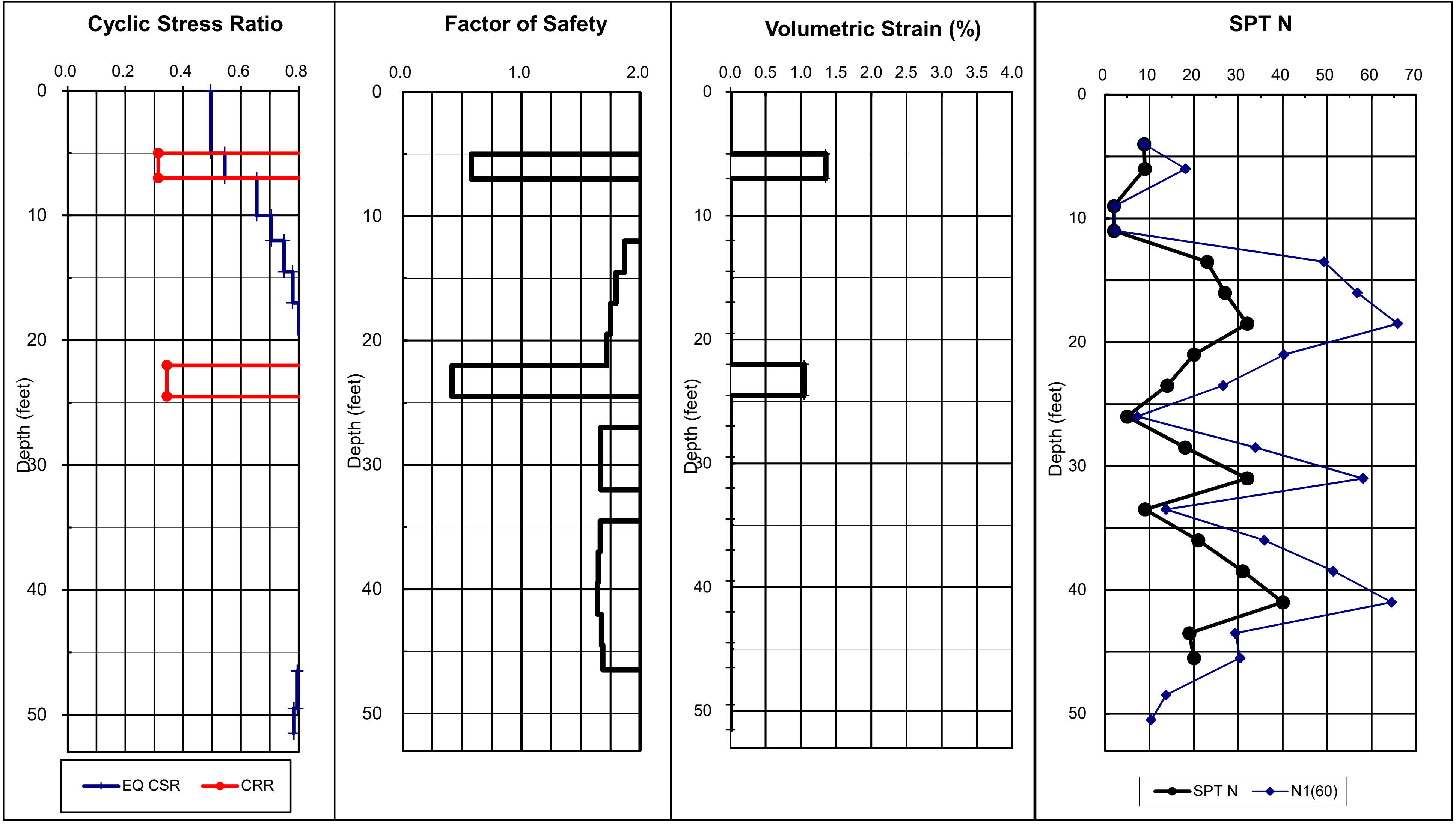
Required SF: **1.30**
Minimum Calculated SF: **0.42**

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
0.0				0				0.000																	
5.0	14	**9**	0	110	90	4.0	7.0	0.220	0.220	0.99	1.00	0.75	1.00	8.8				1.00	Infin.	0.495	**Non-Liq.**		8.8	0.00	**0.00**
7.0	0	**9**	1	110	90	6.0	9.0	0.330	0.299	0.99	1.70	0.75	1.18	18.1	51	8.6	26.7	1.00	0.314	0.544	**0.58**	5.0	23.1	1.35	**0.33**
10.0	0	**2**	0	110	90	9.0	12.0	0.495	0.370	0.98	1.00	0.75	1.10	2.2				1.00	Infin.	0.655	**Non-Liq.**		2.2	0.00	**0.00**
12.0	0	**2**	0	110	90	11.0	14.0	0.605	0.418	0.98	1.00	0.78	1.10	2.3				1.00	Infin.	0.706	**Non-Liq.**		2.3	0.00	**0.00**
14.5	0	**23**	1	120	10	13.5	16.5	0.750	0.485	0.97	1.48	0.84	1.30	49.3	84	1.9	51.2	1.00	1.400	0.750	**1.87**	1.9	51.2	0.00	**0.00**
17.0	0	**27**	1	120	10	16.0	19.0	0.900	0.557	0.97	1.38	0.88	1.30	56.8	90	2.1	58.9	1.00	1.400	0.779	**1.80**	2.1	58.9	0.00	**0.00**
19.5	0	**32**	1	120	10	18.5	21.5	1.050	0.629	0.96	1.30	0.92	1.30	65.9	97	2.3	68.1	1.00	1.400	0.800	**1.75**	2.3	68.1	0.00	**0.00**
22.0	0	**20**	1	120	10	21.0	24.0	1.200	0.701	0.95	1.23	0.94	1.30	40.2	76	1.7	42.0	1.00	1.400	0.815	**1.72**	1.7	42.0	0.00	**0.00**
24.5	0	**14**	1	120	10	23.5	26.5	1.350	0.773	0.95	1.17	0.97	1.25	26.6	62	1.4	28.0	1.00	0.343	0.825	**0.42**	1.0	27.6	1.05	**0.31**
27.0	0	**5**	0	110	80	26.0	29.0	1.493	0.837	0.94	1.00	0.99	1.10	7.3				1.00	Infin.	0.834	**Non-Liq.**		7.3	0.00	**0.00**
29.5	0	**18**	1	115	25	28.5	31.5	1.634	0.901	0.93	1.08	1.00	1.30	33.8	70	8.2	42.0	1.00	1.400	0.840	**1.67**	8.2	42.0	0.00	**0.00**
32.0	0	**32**	1	115	25	31.0	34.0	1.778	0.966	0.92	1.05	1.00	1.30	58.0	91	10.0	68.0	1.00	1.400	0.840	**1.67**	10.0	68.0	0.00	**0.00**
34.5	0	**9**	0	110	75	33.5	36.5	1.918	1.028	0.90	1.00	1.00	1.14	13.7				1.01	Infin.	0.833	**Non-Liq.**		13.7	0.00	**0.00**
37.0	0	**21**	1	115	62	36.0	39.0	2.059	1.092	0.88	0.98	1.00	1.30	35.8	72	10.0	45.8	0.99	1.400	0.842	**1.66**	10.0	45.8	0.00	**0.00**
39.5	0	**31**	1	115	55	38.5	41.5	2.203	1.157	0.86	0.96	1.00	1.30	51.4	86	10.0	61.4	0.96	1.400	0.850	**1.65**	10.0	61.4	0.00	**0.00**
42.0		**40**	1	115	55	41.0	44.0	2.346	1.223	0.84	0.93	1.00	1.30	64.5	96	10.0	74.5	0.94	1.400	0.854	**1.64**	10.0	74.5	0.00	**0.00**
44.5		**19**	1	115	90	43.5	46.5	2.490	1.289	0.82	0.91	1.00	1.28	29.3	65	10.0	39.3	0.94	1.400	0.837	**1.67**	10.0	39.3	0.00	**0.00**
46.5		**20**	1	115	25.0	45.5	48.5	2.605	1.341	0.80	0.89	1.00	1.28	30.4	66	7.8	38.2	0.93	1.400	0.831	**1.69**	7.8	38.2	0.00	**0.00**
49.5		**9**	0	110	88.2	48.5	51.5	2.773	1.415	0.77	1.00	1.00	1.14	13.7				0.94	Infin.	0.796	**Non-Liq.**		13.7	0.00	**0.00**
51.5		**7**	0	110	70.0	50.5	53.5	2.883	1.463	0.75	1.00	1.00	1.11	10.4				0.94	Infin.	0.784	**Non-Liq.**		10.4	0.00	**0.00**

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Channel Islands HS Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-7 **Earthquake Magnitude: 7.4** **PGA, g: 0.79** **Calc GWT (feet): 5**



Total Thickness of Liquefiable Layers: 4.5 feet **Estimated Total Ground Subsidence: 0.6 inches**

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: **Channel Islands HS Gateways**
Job No: **303514-002**
Date: **11/26/2019**
Boring: **B-6** Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: **7.4** 7.5
PGA, g: **0.79** 0.77
MSF: 1.03
GWT: **10.0** feet
Calc GWT: **10.0** feet
Remediate to: **0.0** **feet**

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.33 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: 0.15

Total (ft) Liquefied Thickness: **7**

Total (in.) Induced Subsidence: **1.5**

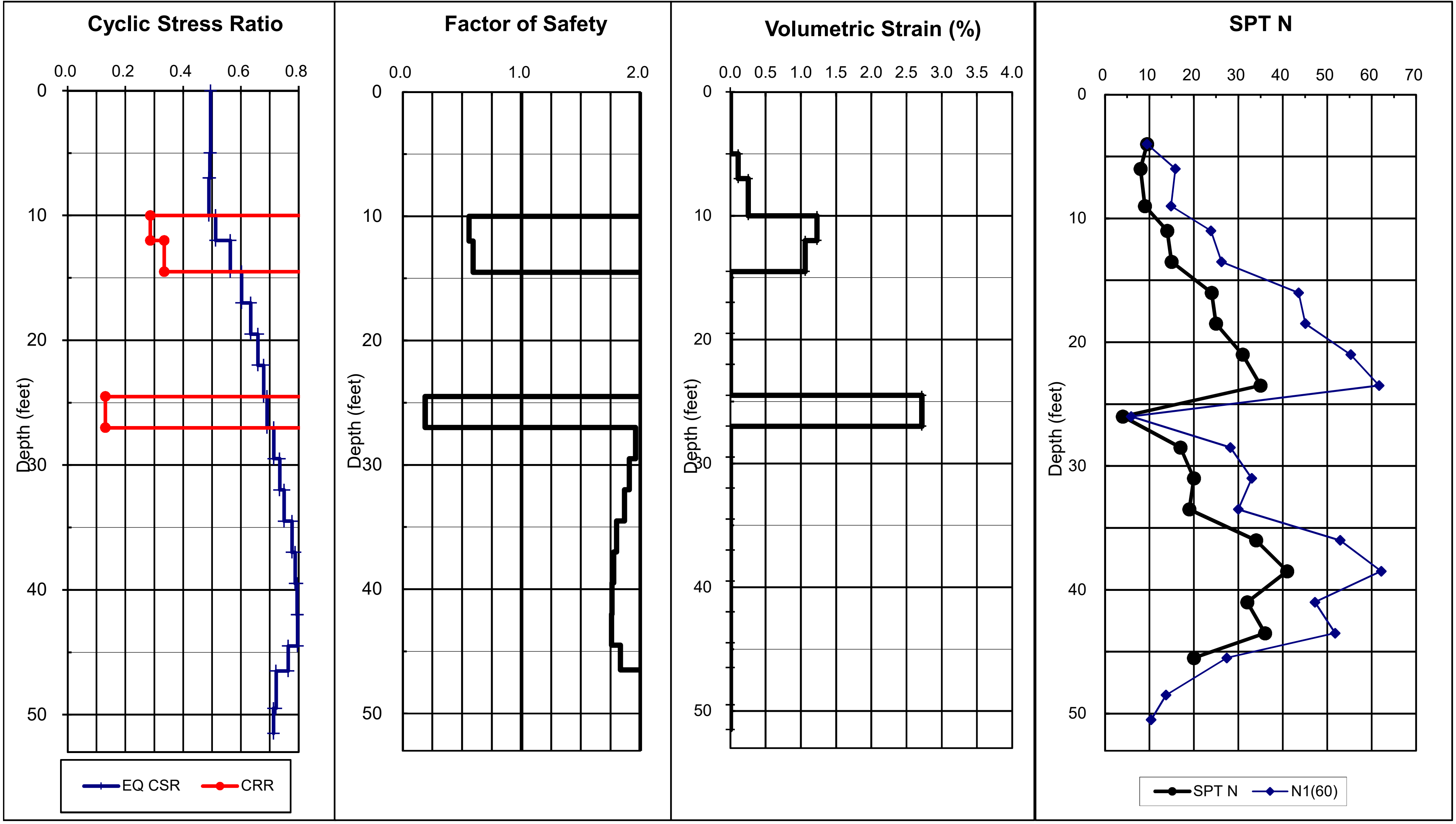
Required SF: **1.30**
Minimum Calculated SF: **0.19**

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
0.0				0				0.000																	
5.0	15	**9**	0	115	25	4.0	7.0	0.230	0.230	0.99	1.00	0.75	1.00	9.5				1.00	Infin.	0.495	**Non-Liq.**		9.5	0.00	**0.00**
7.0	0	**8**	1	115	25	6.0	9.0	0.345	0.345	0.99	1.70	0.75	1.16	15.8	48	6.1	21.9	1.00	0.239	0.493	**Non-Liq.**	6.1	21.9	0.11	**0.03**
10.0	0	**9**	1	115	10	9.0	12.0	0.518	0.518	0.98	1.43	0.75	1.15	14.9	46	1.2	16.0	1.00	0.173	0.490	**Non-Liq.**	1.2	16.0	0.25	**0.09**
12.0	0	**14**	1	120	10	11.0	14.0	0.635	0.604	0.98	1.32	0.78	1.23	23.9	58	1.4	25.2	1.00	0.287	0.513	**0.56**	1.0	24.9	1.23	**0.29**
14.5	0	**15**	1	120	10	13.5	16.5	0.785	0.676	0.97	1.25	0.84	1.25	26.2	61	1.4	27.7	1.00	0.334	0.563	**0.59**	1.0	27.2	1.06	**0.32**
17.0	0	**24**	1	120	10	16.0	19.0	0.935	0.748	0.97	1.19	0.88	1.30	43.5	79	1.8	45.4	1.00	1.400	0.603	**2.32**	1.8	45.4	0.00	**0.00**
19.5	0	**25**	1	120	10	18.5	21.5	1.085	0.820	0.96	1.14	0.92	1.30	45.1	80	1.8	46.9	1.00	1.400	0.634	**2.21**	1.8	46.9	0.00	**0.00**
22.0	0	**31**	1	120	10	21.0	24.0	1.235	0.892	0.95	1.09	0.94	1.30	55.3	89	2.1	57.4	1.00	1.400	0.659	**2.12**	2.1	57.4	0.00	**0.00**
24.5	0	**35**	1	120	10	23.5	26.5	1.385	0.964	0.95	1.05	0.97	1.30	61.7	94	2.2	63.9	1.00	1.400	0.679	**2.06**	2.2	63.9	0.00	**0.00**
27.0	0	**4**	1	115	43	26.0	29.0	1.531	1.032	0.94	1.01	0.99	1.10	5.9	29	6.2	12.1	1.00	0.131	0.691	**0.19**	3.4	9.3	2.72	**0.82**
29.5	0	**17**	1	115	58	28.5	31.5	1.675	1.098	0.93	0.98	1.00	1.27	28.2	63	10.0	38.2	0.99	1.400	0.714	**1.96**	10.0	38.2	0.00	**0.00**
32.0	0	**20**	1	120	55	31.0	34.0	1.823	1.167	0.92	0.95	1.00	1.30	33.0	69	10.0	43.0	0.97	1.400	0.734	**1.91**	10.0	43.0	0.00	**0.00**
34.5	0	**19**	1	120	25	33.5	36.5	1.973	1.239	0.90	0.92	1.00	1.28	30.0	65	7.7	37.7	0.95	1.400	0.750	**1.87**	7.7	37.7	0.00	**0.00**
37.0	0	**34**	1	120	10	36.0	39.0	2.123	1.311	0.88	0.90	1.00	1.30	52.9	87	2.0	55.0	0.92	1.400	0.777	**1.80**	2.0	55.0	0.00	**0.00**
39.5	0	**41**	1	120	10	38.5	41.5	2.273	1.383	0.86	0.87	1.00	1.30	62.2	94	2.2	64.4	0.90	1.400	0.788	**1.78**	2.2	64.4	0.00	**0.00**
42.0	0	**32**	1	125	10	41.0	44.0	2.426	1.459	0.84	0.85	1.00	1.30	47.2	82	1.9	49.1	0.88	1.400	0.794	**1.76**	1.9	49.1	0.00	**0.00**
44.5	0	**36**	1	125	10	43.5	46.5	2.583	1.537	0.82	0.83	1.00	1.30	51.8	86	2.0	53.8	0.86	1.400	0.796	**1.76**	2.0	53.8	0.00	**0.00**
46.5		**20**	1	115	25.0	45.5	48.5	2.703	1.595	0.80	0.81	1.00	1.26	27.4	63	7.4	34.8	0.88	1.400	0.764	**1.83**	7.4	34.8	0.00	**0.00**
49.5		**9**	0	110	88.2	48.5	51.5	2.870	1.669	0.77	1.00	1.00	1.14	13.7				0.91	Infin.	0.722	**Non-Liq.**		13.7	0.00	**0.00**
51.5		**7**	0	110	70.0	50.5	53.5	2.980	1.716	0.75	1.00	1.00	1.11	10.4				0.91	Infin.	0.713	**Non-Liq.**		10.4	0.00	**0.00**

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Channel Islands HS Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-6 **Earthquake Magnitude: 7.4** **PGA, g: 0.79** **Calc GWT (feet): 10**



Total Thickness of Liquefiable Layers: 7.0 feet

Estimated Total Ground Subsidence: 1.5 inches

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: Channel Islands HS Gateways
Job No: 303514-002
Date: 11/26/2019
Boring: B-6 Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:

Magnitude: 7.4 7.5
PGA, g: 0.79 0.77
MSF: 1.03
GWT: 10.0 feet
Calc GWT: 5.0 feet
Remediate to: 0.0 **feet**

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E): 1.33 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: 0.13

Total (ft) Liquefied Thickness 7.5

Total (in.) Induced Subsidence 1.8

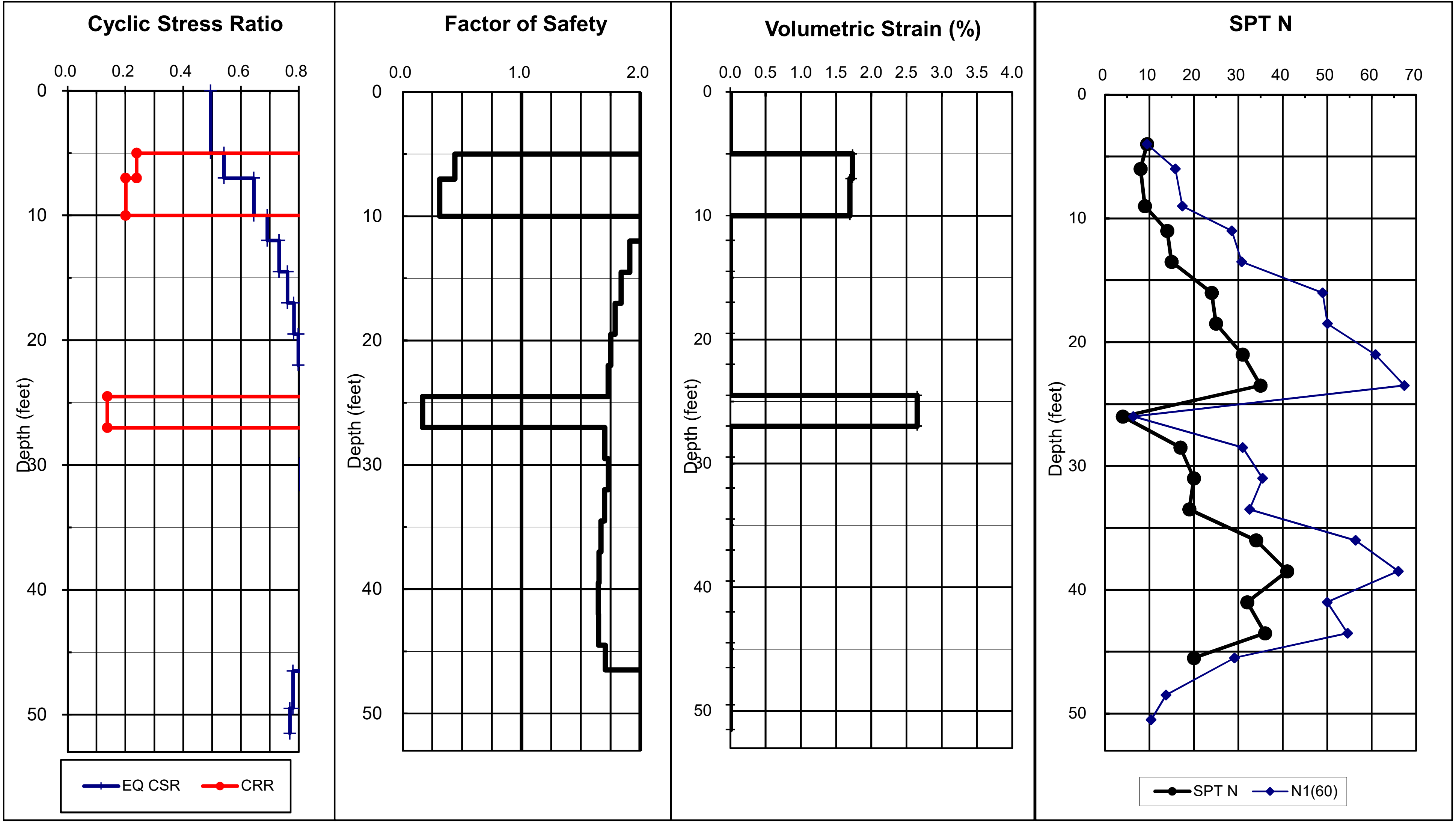
Required SF: 1.30
Minimum Calculated SF: 0.17

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
0.0				0				0.000																	
5.0	15	**9**	0	115	25	4.0	7.0	0.230	0.230	0.99	1.00	0.75	1.00	9.5				1.00	Infin.	0.495	**Non-Liq.**		9.5	0.00	**0.00**
7.0	0	**8**	1	115	25	6.0	9.0	0.345	0.314	0.99	1.70	0.75	1.16	15.8	48	6.1	21.9	1.00	0.239	0.542	**0.44**	2.0	17.8	1.73	**0.42**
10.0	0	**9**	1	115	10	9.0	12.0	0.518	0.393	0.98	1.64	0.75	1.18	17.4	50	1.2	18.6	1.00	0.201	0.645	**0.31**	1.0	18.4	1.70	**0.61**
12.0	0	**14**	1	120	10	11.0	14.0	0.635	0.448	0.98	1.54	0.78	1.27	28.5	64	1.5	30.0	1.00	1.400	0.691	**2.03**	1.5	30.0	0.00	**0.00**
14.5	0	**15**	1	120	10	13.5	16.5	0.785	0.520	0.97	1.43	0.84	1.29	30.7	66	1.5	32.3	1.00	1.400	0.732	**1.91**	1.5	32.3	0.00	**0.00**
17.0	0	**24**	1	120	10	16.0	19.0	0.935	0.592	0.97	1.34	0.88	1.30	49.0	84	1.9	50.9	1.00	1.400	0.762	**1.84**	1.9	50.9	0.00	**0.00**
19.5	0	**25**	1	120	10	18.5	21.5	1.085	0.664	0.96	1.26	0.92	1.30	50.1	85	2.0	52.0	1.00	1.400	0.783	**1.79**	2.0	52.0	0.00	**0.00**
22.0	0	**31**	1	120	10	21.0	24.0	1.235	0.736	0.95	1.20	0.94	1.30	60.9	93	2.2	63.1	1.00	1.400	0.799	**1.75**	2.2	63.1	0.00	**0.00**
24.5	0	**35**	1	120	10	23.5	26.5	1.385	0.808	0.95	1.14	0.97	1.30	67.3	98	2.3	69.7	1.00	1.400	0.810	**1.73**	2.3	69.7	0.00	**0.00**
27.0	0	**4**	1	115	43	26.0	29.0	1.531	0.876	0.94	1.10	0.99	1.10	6.4	30	6.3	12.7	1.00	0.137	0.818	**0.17**	3.4	9.8	2.65	**0.80**
29.5	0	**17**	1	115	58	28.5	31.5	1.675	0.942	0.93	1.06	1.00	1.29	31.0	66	10.0	41.0	1.00	1.400	0.823	**1.70**	10.0	41.0	0.00	**0.00**
32.0	0	**20**	1	120	55	31.0	34.0	1.823	1.011	0.92	1.02	1.00	1.30	35.5	71	10.0	45.5	1.02	1.400	0.808	**1.73**	10.0	45.5	0.00	**0.00**
34.5	0	**19**	1	120	25	33.5	36.5	1.973	1.083	0.90	0.99	1.00	1.30	32.5	68	8.0	40.6	0.99	1.400	0.824	**1.70**	8.0	40.6	0.00	**0.00**
37.0	0	**34**	1	120	10	36.0	39.0	2.123	1.155	0.88	0.96	1.00	1.30	56.4	90	2.1	58.5	0.97	1.400	0.839	**1.67**	2.1	58.5	0.00	**0.00**
39.5	0	**41**	1	120	10	38.5	41.5	2.273	1.227	0.86	0.93	1.00	1.30	66.0	97	2.3	68.3	0.94	1.400	0.847	**1.65**	2.3	68.3	0.00	**0.00**
42.0	0	**32**	1	125	10	41.0	44.0	2.426	1.303	0.84	0.90	1.00	1.30	50.0	84	2.0	51.9	0.92	1.400	0.850	**1.65**	2.0	51.9	0.00	**0.00**
44.5	0	**36**	1	125	10	43.5	46.5	2.583	1.381	0.82	0.88	1.00	1.30	54.6	88	2.1	56.7	0.90	1.400	0.849	**1.65**	2.1	56.7	0.00	**0.00**
46.5		**20**	1	115	25.0	45.5	48.5	2.703	1.439	0.80	0.86	1.00	1.27	29.1	65	7.6	36.8	0.91	1.400	0.821	**1.71**	7.6	36.8	0.00	**0.00**
49.5		**9**	0	110	88.2	48.5	51.5	2.870	1.513	0.77	1.00	1.00	1.14	13.7				0.93	Infin.	0.781	**Non-Liq.**		13.7	0.00	**0.00**
51.5		**7**	0	110	70.0	50.5	53.5	2.980	1.560	0.75	1.00	1.00	1.11	10.4				0.93	Infin.	0.770	**Non-Liq.**		10.4	0.00	**0.00**

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Channel Islands HS Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-6 **Earthquake Magnitude: 7.4** **PGA, g: 0.79** **Calc GWT (feet): 5**



Total Thickness of Liquefiable Layers: 7.5 feet

Estimated Total Ground Subsidence: 1.8 inches

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: Channel Islands HS NE Entry
Job No: 303514-002
Date: 11/26/2019
Boring: B3 Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 7.4 7.5
PGA, g: 0.79 0.77
MSF: 1.03
GWT: 8.5 feet
Calc GWT: 8.5 feet
Remediate to: 0.0 feet

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.33 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: 0.26

Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
6.5	1.4

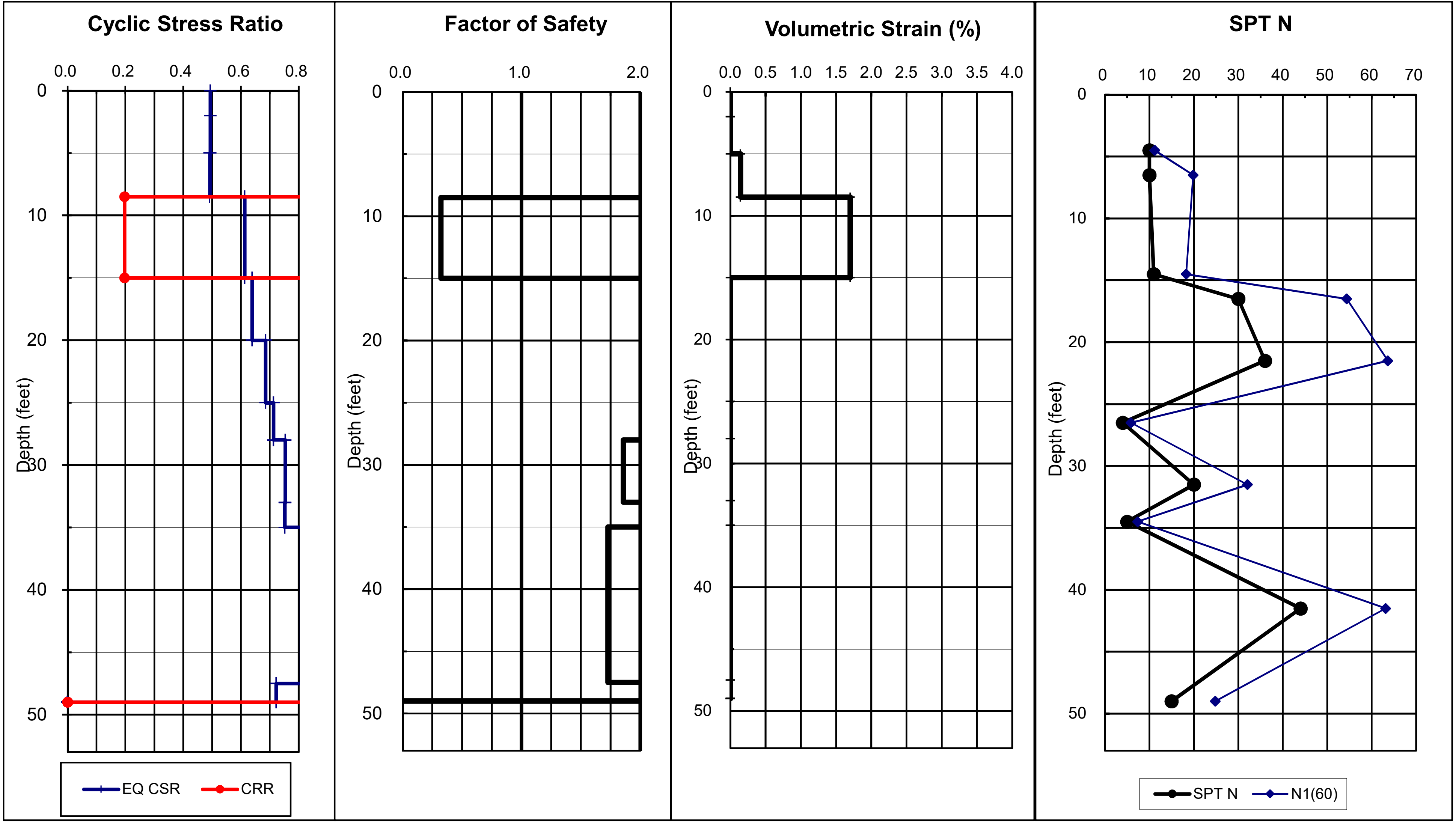
Required SF: 1.30
Minimum Calculated SF: 0.32

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
0.0				0				0.000																	
2.0	0	10	0	117	35	4.5	7.5	0.263	0.263	0.99	1.00	0.75	1.12	11.2				1.00	Infin.	0.495	**Non-Liq.**		11.2	0.00	**0.00**
5.0	0	10	0	117	55	4.5	7.5	0.263	0.263	0.99	1.00	0.75	1.12	11.2				1.00	Infin.	0.495	**Non-Liq.**		11.2	0.00	**0.00**
8.5	0	10	1	125	5	6.5	9.5	0.386	0.386	0.99	1.66	0.75	1.20	19.8	53	0.0	19.8	1.00	0.214	0.492	**Non-Liq.**	0.0	19.8	0.14	**0.06**
15.0	0	11	1	125	5	14.5	17.5	0.886	0.699	0.97	1.23	0.86	1.19	18.3	51	0.0	18.3	1.00	0.197	0.613	**0.32**	0.0	18.3	1.70	**1.33**
20.0	0	30	1	125	5	16.5	19.5	1.011	0.762	0.97	1.18	0.89	1.30	54.4	88	0.0	54.4	1.00	1.400	0.639	**2.19**	0.0	54.4	0.00	**0.00**
25.0	0	36	1	125	5	21.5	24.5	1.324	0.918	0.95	1.07	0.95	1.30	63.6	95	0.0	63.6	1.00	1.400	0.685	**2.04**	0.0	63.6	0.00	**0.00**
28.0	0	4	0	125	90	26.5	29.5	1.636	1.075	0.94	1.00	1.00	1.10	5.8				1.00	Infin.	0.713	**Non-Liq.**		5.8	0.00	**0.00**
33.0	0	20	1	125	42	31.5	34.5	1.949	1.231	0.91	0.93	1.00	1.30	32.1	68	10.0	42.1	0.96	1.400	0.754	**1.86**	10.0	42.1	0.00	**0.00**
35.0	0	5	0	125	92	34.5	37.5	2.136	1.325	0.89	1.00	1.00	1.10	7.3				0.96	Infin.	0.752	**Non-Liq.**		7.3	0.00	**0.00**
47.5	0	44	1	125	5	41.5	44.5	2.574	1.544	0.84	0.83	1.00	1.30	63.1	95	0.0	63.1	0.86	1.400	0.810	**1.73**	0.0	63.1	0.00	**0.00**
49.0	0	15	0	125	81	49.0	52.0	3.043	1.779	0.76	1.00	1.00	1.24	24.8				0.90	Infin.	0.722	**Non-Liq.**		24.8	0.00	**0.00**
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Channel Islands HS NE Entry **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B3 **Earthquake Magnitude: 7.4** **PGA, g: 0.79** **Calc GWT (feet): 9**



Total Thickness of Liquefiable Layers: 6.5 feet **Estimated Total Ground Subsidence: 1.4 inches**

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: Channel Islands HS NE Entry
Job No: 303514-002
Date: 11/26/2019
Boring: B3 Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:

Magnitude: 7.4 7.5
PGA, g: 0.79 0.77
MSF: 1.03
GWT: 8.5 feet
Calc GWT: 5.0 feet
Remediate to: 0.0 **feet**

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E): 1.33 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: 0.24

Total (ft) Liquefied Thickness: 10

Total (in.) Induced Subsidence: 1.9

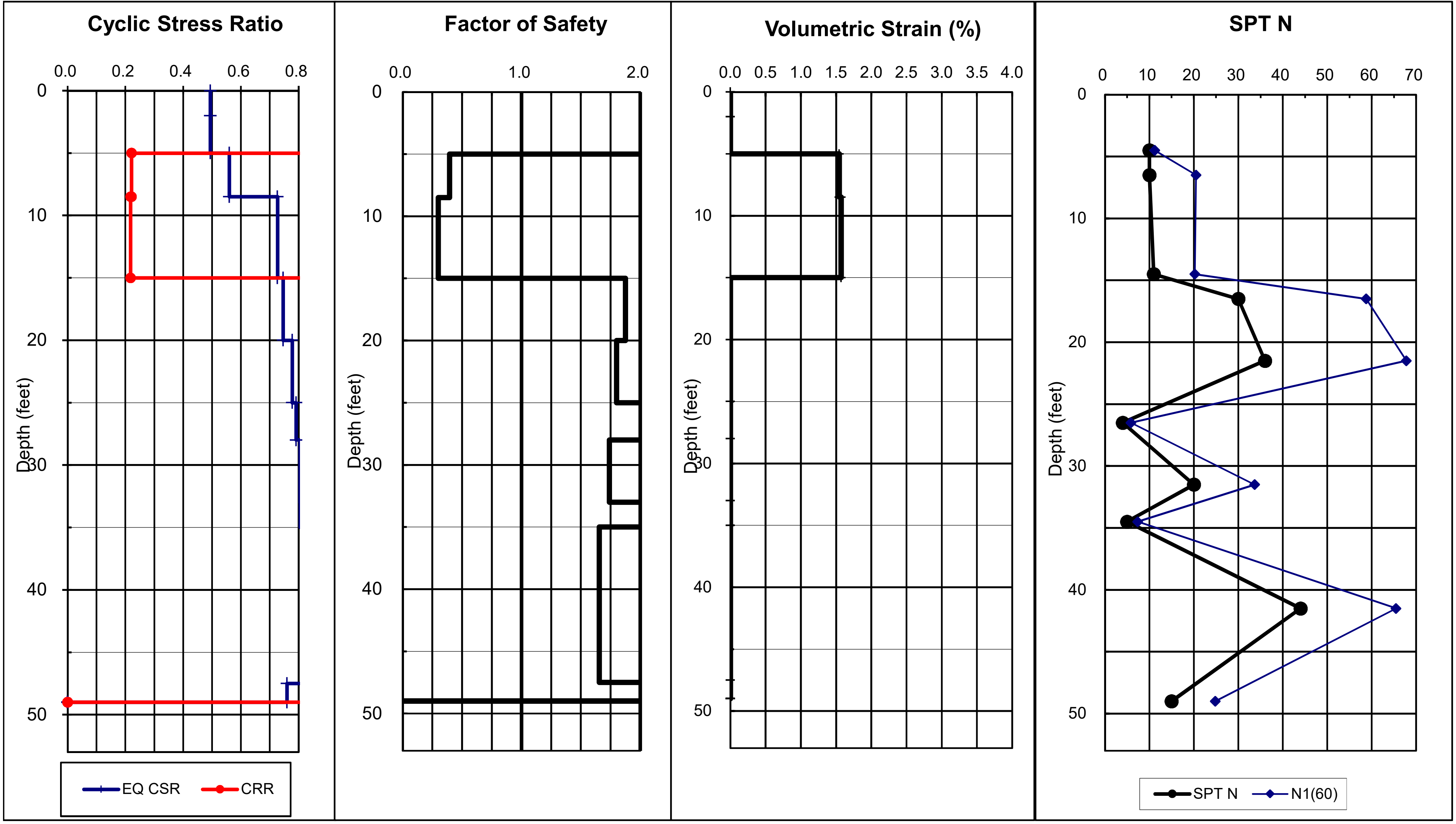
Required SF: 1.30
Minimum Calculated SF: 0.30

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
0.0				0				0.000																	
2.0	0	**10**	0	117	35	4.5	7.5	0.263	0.263	0.99	1.00	0.75	1.12	11.2				1.00	Infin.	0.495	**Non-Liq.**		11.2	0.00	**0.00**
5.0	0	**10**	0	117	55	4.5	7.5	0.263	0.263	0.99	1.00	0.75	1.12	11.2				1.00	Infin.	0.495	**Non-Liq.**		11.2	0.00	**0.00**
8.5	0	10	1	125	5	6.5	9.5	0.386	0.339	0.99	1.70	0.75	1.20	20.5	54	0.0	20.5	1.00	0.222	0.560	**0.40**	0.0	20.5	1.54	**0.65**
15.0	0	11	1	125	5	14.5	17.5	0.886	0.590	0.97	1.34	0.86	1.20	20.2	54	0.0	20.2	1.00	0.218	0.727	**0.30**	0.0	20.2	1.57	**1.23**
20.0	0	30	1	125	5	16.5	19.5	1.011	0.652	0.97	1.27	0.89	1.30	58.8	92	0.0	58.8	1.00	1.400	0.746	**1.88**	0.0	58.8	0.00	**0.00**
25.0	0	36	1	125	5	21.5	24.5	1.324	0.809	0.95	1.14	0.95	1.30	67.8	98	0.0	67.8	1.00	1.400	0.778	**1.80**	0.0	67.8	0.00	**0.00**
28.0	0	4	0	125	90	26.5	29.5	1.636	0.965	0.94	1.00	1.00	1.10	5.8				1.00	Infin.	0.792	**Non-Liq.**		5.8	0.00	**0.00**
33.0	0	20	1	125	42	31.5	34.5	1.949	1.122	0.91	0.97	1.00	1.30	33.7	69	10.0	43.7	0.98	1.400	0.805	**1.74**	10.0	43.7	0.00	**0.00**
35.0	0	5	0	125	92	34.5	37.5	2.136	1.216	0.89	1.00	1.00	1.10	7.3				0.97	Infin.	0.806	**Non-Liq.**		7.3	0.00	**0.00**
47.5	0	44	1	125	5	41.5	44.5	2.574	1.435	0.84	0.86	1.00	1.30	65.5	97	0.0	65.5	0.89	1.400	0.846	**1.65**	0.0	65.5	0.00	**0.00**
49.0	0	15	0	125	81	49.0	52.0	3.043	1.670	0.76	1.00	1.00	1.24	24.8				0.91	Infin.	0.760	**Non-Liq.**		24.8	0.00	**0.00**
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Channel Islands HS NE Entry **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B3 **Earthquake Magnitude: 7.4** **PGA, g: 0.79** **Calc GWT (feet): 5**



Total Thickness of Liquefiable Layers: 10.0 feet

Estimated Total Ground Subsidence: 1.9 inches

Job Number: 303514-002
Job Name: Channel Islands HS SE Gateway
Boring Number: B-6
Date: November 25, 2019
Calculated By: PVB

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
(Bartlett and Youd 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}.

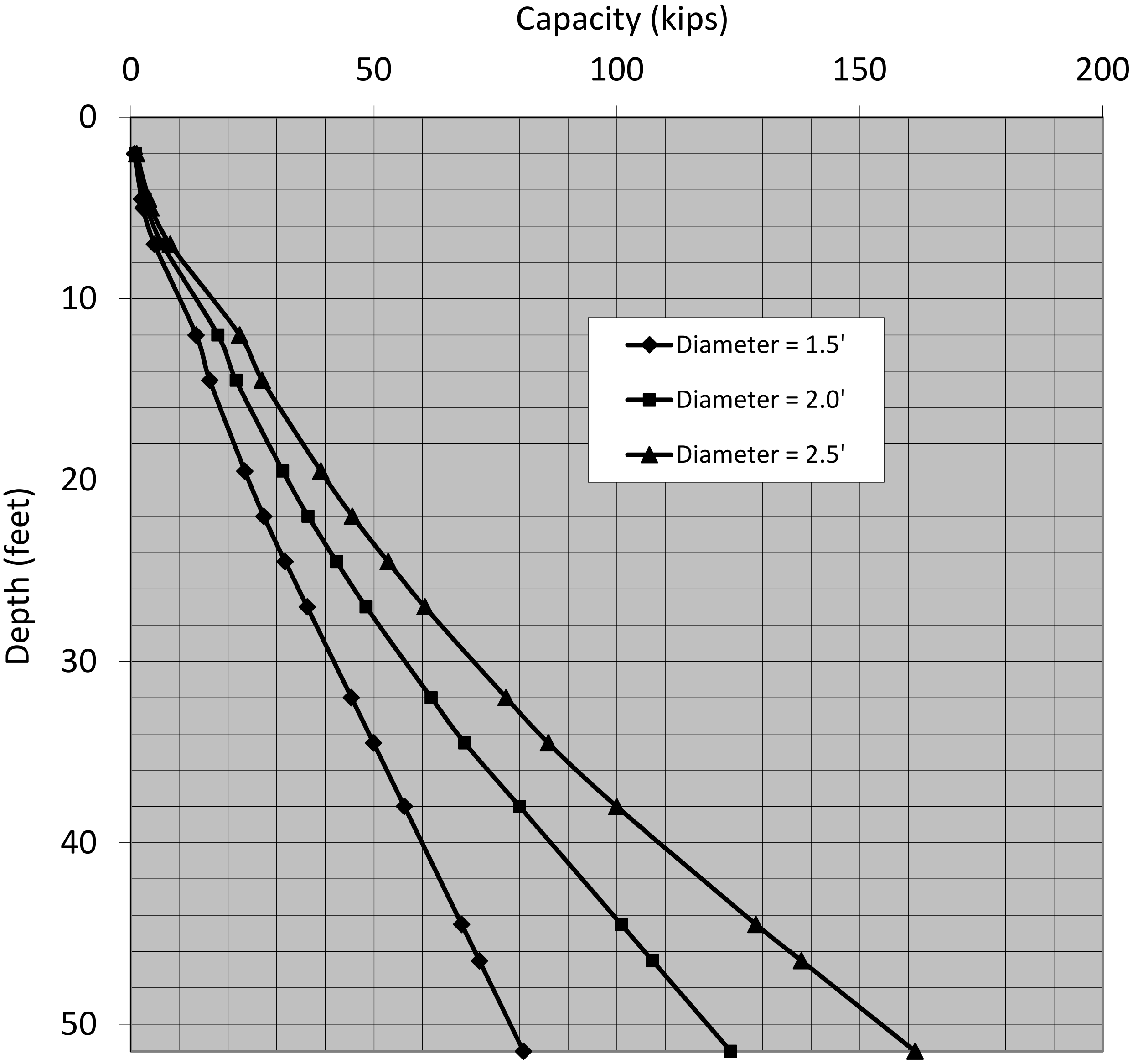
Input Values	
M = 7.4	
R = 3.2	km
S = 0.25	%
T_{15} = 0.77	m
F_{15} = 43	%
$D50_{15}$ = 0.3	mm

Horizontal Ground Displacement in meters (D_H) = 0.41
Horizontal Ground Displacement in feet (D_H) = 1.3

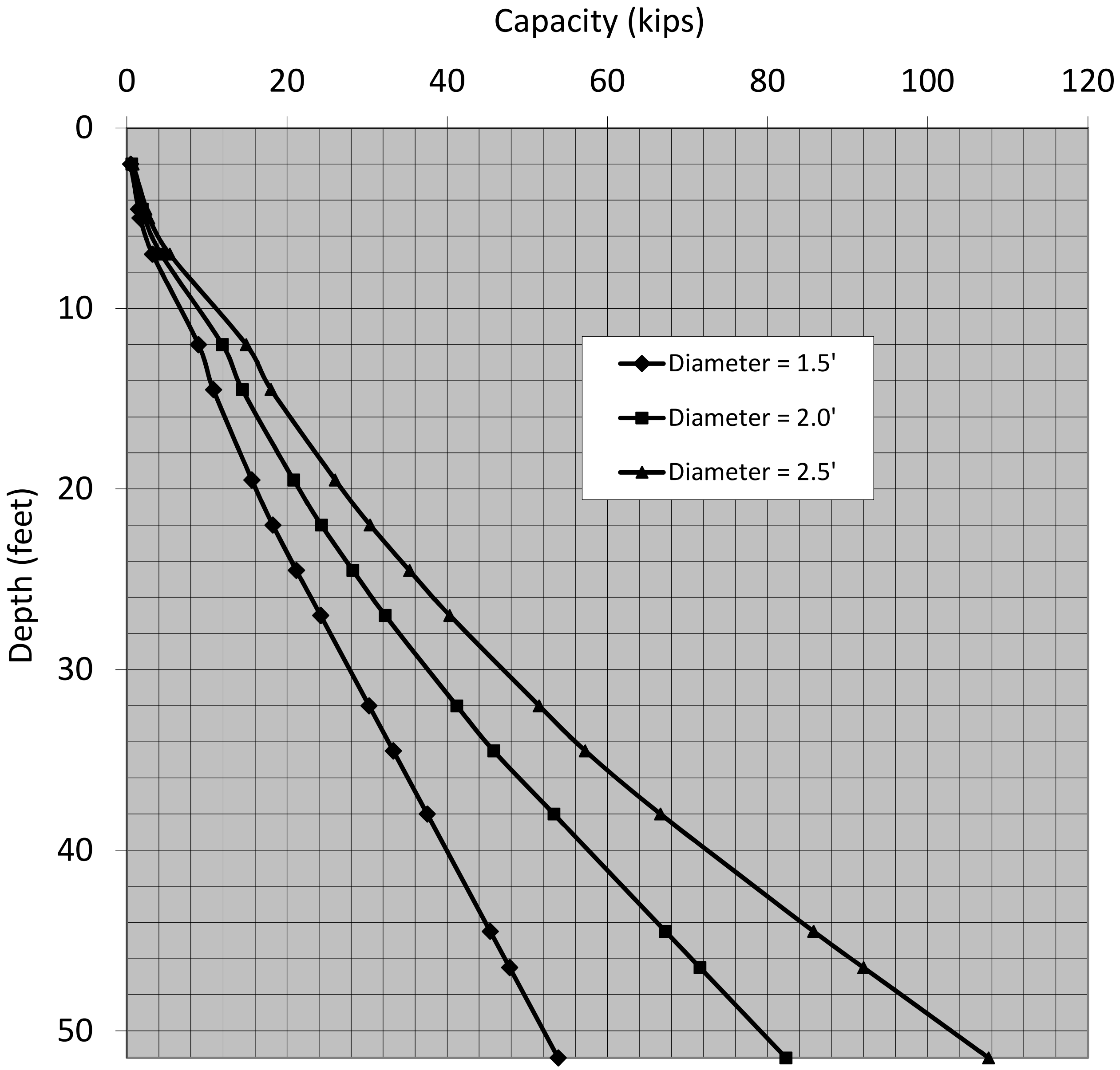
APPENDIX E

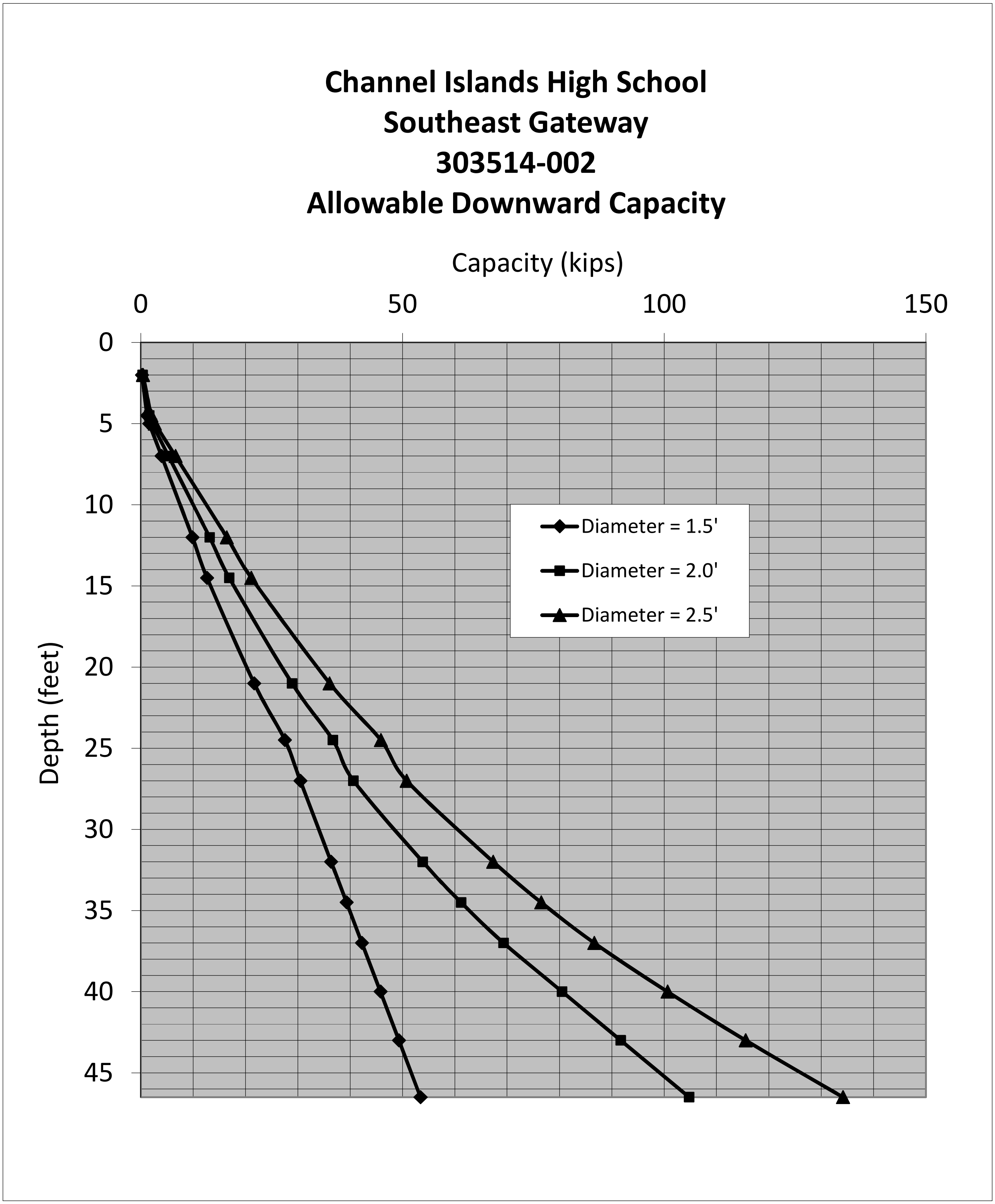
Pile Capacity Graphs

Channel Islands High School
Northwest Gateway
303514-002
Allowable Downward Capacity



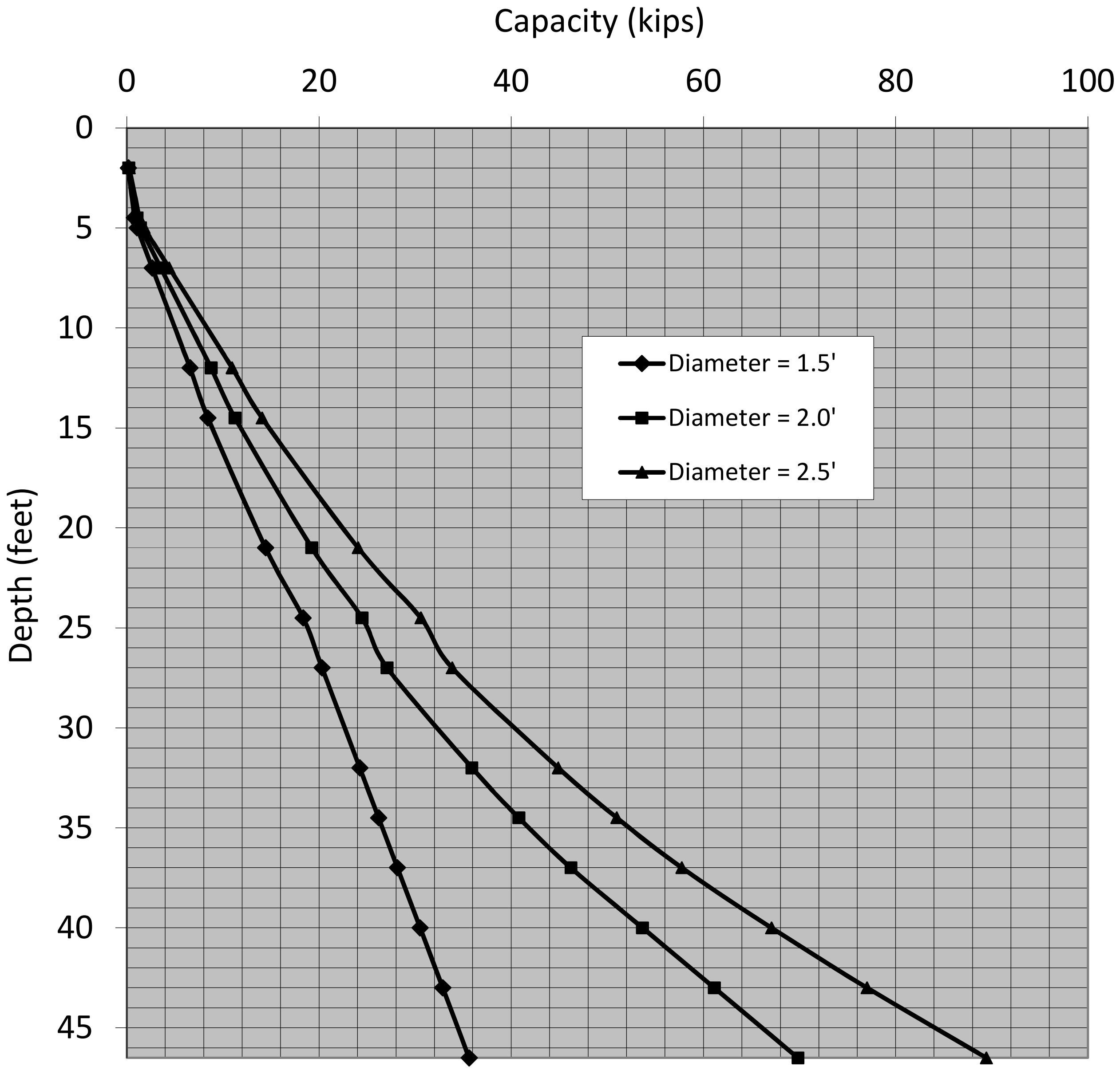
Channel Islands High School
Northwest Gateway
303514-002
Allowable Upward Capacity



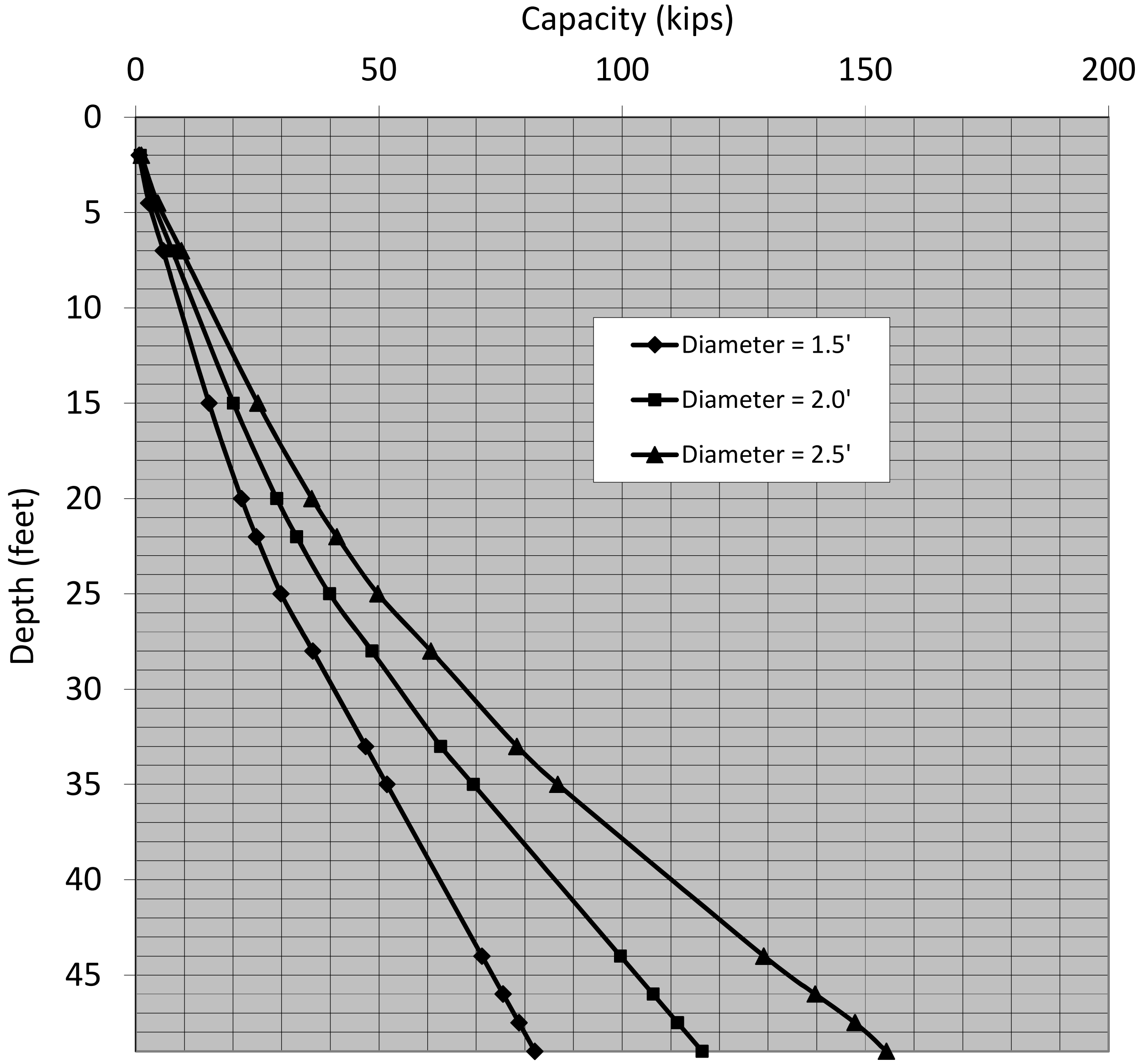


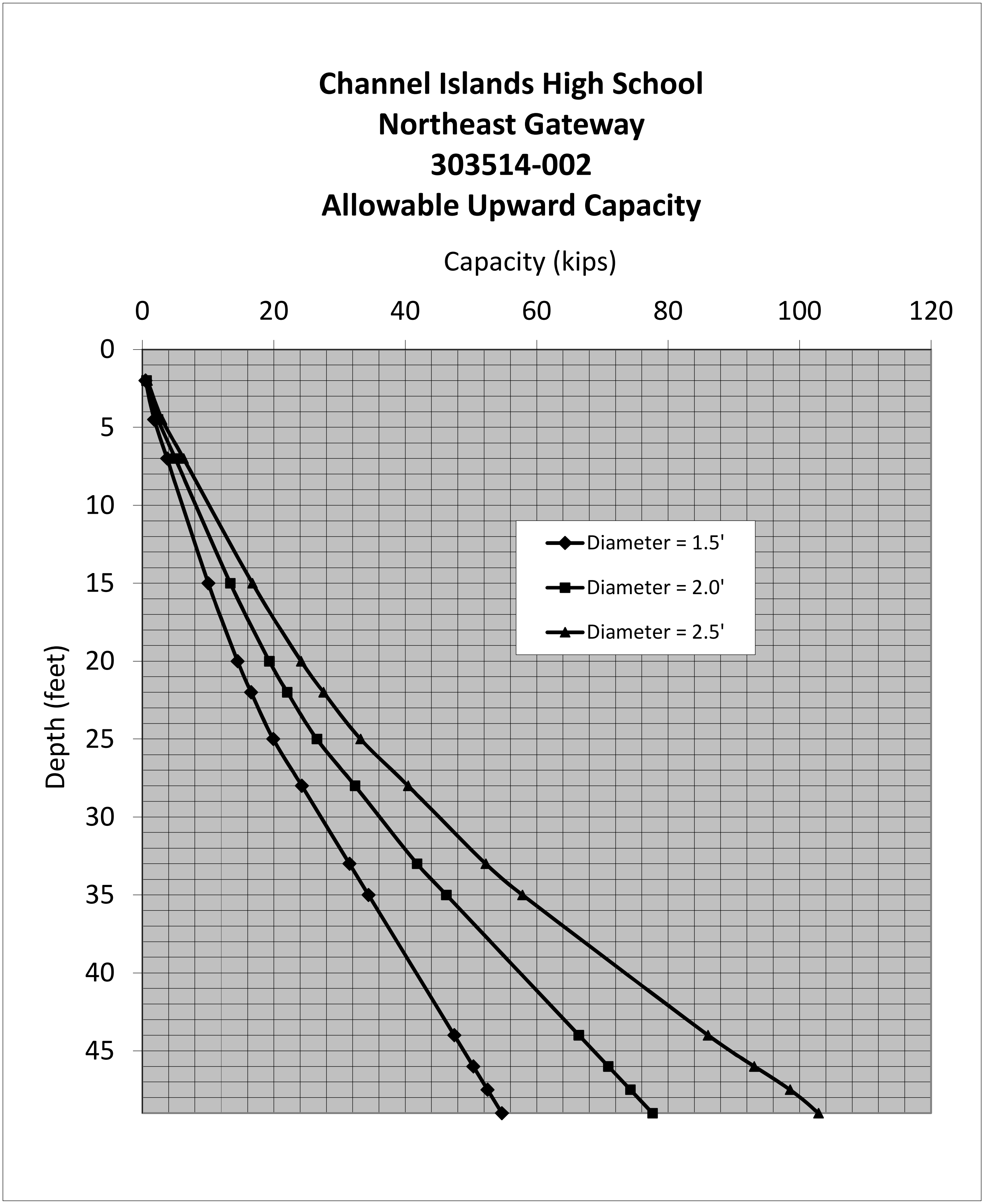
Channel Islands High School
Southeast Gateway
303514-002
Allowable Downward Capacity

Channel Islands High School
Southeast Gateway
303514-002
Allowable Upward Capacity



Channel Islands High School
Northeast Gateway
303514-002
Allowable Downward Capacity





Channel Islands High School
Northeast Gateway
303514-002
Allowable Upward Capacity