GEOTECHNICAL ENGINEERING REPORT
FOR PROPOSED BATHROOM BUILDING,
TICKET BOOTHS AND GATEWAYS
TO STADIUM COMPLEX AT
OXNARD HIGH SCHOOL,
3400 WEST GONZALES ROAD,
OXNARD, CALIFORNIA

PROJECT NO.: 303514-002
NOVEMBER 25, 2019

PREPARED FOR
OXNARD UNION HIGH SCHOOL DISTRICT

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

Earth Systems

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

November 25, 2019

Project No.: 303514-002
Report No.: 19-11-67

Attention: Poul Hanson
Oxnard Union High School District
309 South K Street
Oxnard, CA 93030

Project: Bathroom Building, Ticket Booths and Gateways to Stadium Complex
Oxnard High School
3400 Gonzales Road
Oxnard, California

As authorized, we have performed geotechnical studies for proposed ticket booths and gateways to the stadium complex at Oxnard 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,

EARTH SYSTEMS PACIFIC

PATRICK V. BOALES
No. 1346
CERTIFIED ENGINEERING GEOLOGIST

Patrick V Boales

Patrick V. Boales 11-25-19
Engineering Geologist

Anthony P. Mazzei
Geotechnical Engineer

ANTHONY P. MAZZEI
GE 2823
Exp. 6-30-21

11/25/19

Copies:
- 2 - Oxnard Union High School District (1 via US mail, 1 via email)
- 1 - LuEllen Benjamins, Farnaz Mahjoob, Jay Tittle (via email)
- 1 - Sylvia Wallis, Architecture 4 Education (via email)
- 1 - Project File

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INTRODUCTION

This report presents results of a geotechnical engineering study performed for a proposed bathroom building and two structures that will serve as ticket booths and gateways to the athletic field complex at Oxnard High School in the City of Oxnard, California (see Vicinity Map in Appendix A). Current plans indicate that the one-story bathroom building will be a reinforced CMU block structure that will be approximately 498 square feet in plan dimensions. It is proposed to support it with a conventional foundation system and a slab-on-grade floor. 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 is currently essentially level. As a result, grading for the proposed structures 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 at various times between 1992 and 2019.
3. Drilling, sampling, and logging three 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 fault of significant activity the Oak Ridge Fault is located approximately 2.3 miles north of the subject site. (For the purposes of the liquefaction evaluation, it has been assumed that the fault plane of the Oak Ridge Fault projects below the site at depth, and that the potential earthquake could happen directly below the campus, as required by CGS.) 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 alluvial sediments consisting of loose to very dense silty sands to sandy silts, fine to coarse sands, clayey sands, and firm to stiff clayey silt.

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 (34.2155˚ North Latitude and -119.2143° West Longitude). The calculator adjusts for Soil Site Class D, 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)	D
Occupancy (Risk) Category	III
Seismic Design Category	E
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	2.507 g
Spectral Response Acceleration at 1 sec. – S_1	0.933 g
Site Coefficient – F_a	1.00
Site Coefficient – F_v	1.50
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.507 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.400 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.671g
One Second Spectral Response – S_{D1}	0.933 g
Site Modified Peak Ground Acceleration - PGA_M	0.976 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 below the site at depth, and that the potential earthquake could happen directly below the campus, as required by CGS. For the Site-Specific Analysis, the Short Period Spectral Response (S_{DS}) was found to be 1.337 g, and the 1 Second Spectral Response (S_{D1}) was found to be 1.097 g. Both the "site specific" and "general procedure yielded site modified peak ground accelerations of 0.976 g.

The Fault Parameters table in Appendix C lists the significant "active" and "potentially active" faults within a radius of about 37 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 alluvium that consists of loose to very dense silty sands to sandy silts, fine to coarse sands, clayey sands, and firm to stiff clayey silt. Near-surface soils encountered in Borings B-1, B-2, B-5, B-7, and B-8 are characterized by high blow counts and in-place densities, and low compressibilities. Near-surface soils encountered in Borings B-3, B-4, B-6, B-9, and B-10 are characterized by moderate blow counts and in-place densities. Testing indicates that anticipated bearing soils lie in the "medium" expansion range because the expansion index equals 65. [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 at a depth of approximately 20 feet below existing site grades in Boring B-9. However, mapping of historically high groundwater levels by the California Geological Survey (CGS, 2002a) indicates that groundwater has risen in the past to about 10 below the ground surface near the subject site. Furthermore, borings advanced by Earth Systems Southern California in March 2007 encountered water at depths as shallow as 6 to 7 feet below the baseball field area, which appears to be about the lowest point of the campus.

As mentioned previously, the site 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

(1,300 mg/Kg) are in the “S1” (“moderate”) exposure class of Table 19.3.1.1 of ACI 318-14; therefore, it appears that special concrete designs will be necessary for the measured sulfate contents. The typical concrete would be Type II with a maximum water to cement ratio of 0.5 and a minimum unconfined compressive strength of 4,000 psi.

Based on criteria established by the County of Los Angeles (2013), measurements of resistivity of near-surface soils (820 ohms-cm) indicate that they are “severely 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:

The proposed bathroom building will be located immediately south of the baseball field. The proposed gateways will be located near the northwest 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 proposed location. The analyses were performed in general accordance with the methods proposed by NCEER (1997).

The surface trace of the Oak Ridge Fault is approximately 2.3 miles north of the campus. However, 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 projects directly below the site, and is at a distance of 0 kilometers.

Baseball Field Bathroom Building Analysis

The proposed bathroom building will be located near the baseball field at the southwest corner of the campus property. Exploration that was performed at the northeast corner included Boring B-2 from the athletic field studies of 2019 and a new boring (Boring B-8) that was advanced to a depth of 51.5 feet. Data from those studies indicates that soil conditions in this area:

1. Soils are generally alluvial sands with minor interbeds of sandy silts.
2. Groundwater was encountered at a depth of 22 feet in Boring B-8, but historically shallowest groundwater has been at a depth of about 10 feet.
3. Atterberg limit evaluations indicate that the finer grained soils at a depth of 15 feet below the ground surface have a plasticity index (PI) of 6 and classify as a silty sand (SM). Soils at a depth of 27.5 feet were found to have a PI of 2 and classify as a sandy silt (ML). (PI test results and hydrometer tests are presented in Appendix B.) These soils would not be expected to exhibit clay-like behavior during earthquake cyclic loading, and could potentially be liquefiable if below groundwater.
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 10 feet, and another assuming groundwater at a depth of 22 feet. The analysis assuming groundwater at 22 feet indicated that no soil layers had factors of safety below 1.3 (see Appendix D for calculations). However, when groundwater was assumed to be at a depth of 10 feet, a 2.5-foot thick zone between depths of 14.5 and 17 feet had a factor 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 zone 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 0.7 inches.

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 0.4 inches at the ground surface.

Because the potentially liquefiable zone is only 2.5 feet thick and is below 14.5 feet of non-liquefiable soils, ground damage, such as sand boils are ground cracks, would not be expected.

”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, which is true for the potentially liquefiable zone described above. The cumulative 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 0.36%, which is equivalent to about 5 feet of fall in 1,400 feet, as shown on the Oxnard Quadrangle near the subject site. Fine contents were assumed to be 23% based on hydrometer testing performed on a sample gathered from that soil layer during subsurface studies. The cumulative displacement was calculated to be about 3.2 feet, if the zone was to liquefy. (Calculations are included within Appendix D of this report.)

Based on the measured liquidity indices, the finer-grained layers at the site do not appear to be sensitive. Hence, strength loss and post-liquefaction consolidation are not thought to be significant concerns.

Based on the above, it is the opinion of this firm that a potential for liquefaction and horizontal displacement (lateral spreading) exists at the baseball field bathroom site.

Northwest Gateway Analysis

Exploration that was performed at the proposed northwest entry between the rest of the campus and the stadium complex included Boring B-3 from the athletic field studies of 2019 and a new boring (Boring B-9) that was advanced to a depth of 51.5 feet. Data from those studies indicates that conditions in this area:

1. Soils are generally alluvial sands with minor interbeds of sandy silts.
2. Groundwater was encountered at a depth of 20 feet in Boring B-9, but historically shallowest groundwater has been at a depth of about 10 feet.
3. An Atterberg limit evaluation indicates that the finer grained soils at a depth of 27.5 feet below the ground surface have a plasticity index (PI) of 18 and classify as a sandy silt (ML). (PI test results and hydrometer tests are presented in Appendix B.) These soils would be expected to exhibit clay-like behavior during earthquake cyclic loading, would not be liquefiable if below groundwater.
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 10 feet, and another assuming groundwater at a depth of 20 feet. The analysis assuming groundwater at 20 feet indicated that three soil layers between depths of 20.0 and 22 feet, 29.5 and 32 feet and 47 and 49.5 feet had factors of safety below 1.3 (see Appendix D for calculations). When groundwater was assumed to be at a depth of 10 feet, two layers had factors of safety below 1.3. The soil layer between depths of 20.0 and 22 feet was not susceptible to liquefaction when the 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 1.5 inches when groundwater was assumed to be at a depth of 20 feet, and 1.1 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 0.8 inches at the ground surface if the worst case is assumed.

Because the shallowest potentially liquefiable zone is only 2.5 feet thick and is below 29.5 feet of non-liquefiable soils, ground damage, such as sand boils are ground cracks, would not be expected.

”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 shallower of the two potentially liquefiable zones has an $N_{1(60)}$ of 14.5, and thus may be potentially susceptible to ground slope lateral spreading. The cumulative 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 0.36%, which is equivalent to about 5 feet of fall in 1,400 feet, as shown on the Oxnard Quadrangle near the subject site. Fine contents were assumed to be 5% based on the soil description of “fine to coarse sand, trace to little silt”. The cumulative displacement was calculated to be about 3.9 feet, if the entire zone was to liquefy. (Calculations are included within Appendix D of this report.)

There is a fine-grained layer of low plasticity silt between the depths of 27 and 29.5 feet that has a liquidity index of about 0.67 and a sensitivity of about 6. This layer is only a few feet thick, and by itself, cannot lead to much post-liquefaction consolidation. Therefore, seismic-induced cyclic softening and post-liquefaction settlement from consolidation 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 and horizontal displacement (lateral spreading) exists at the northwest gateway site.

Southeast Gateway Analysis

Exploration that was performed at the proposed entry at the southeastern end of the stadium complex included Boring B-4 from the athletic field studies of 2019 and a new boring (Boring B-10) that was advanced to a depth of 51.5 feet. Data from those studies indicates that conditions in this area:

1. Soils are generally alluvial sands with minor interbeds of sandy silts.
2. Groundwater was encountered at a depth of 25 feet in Boring B-10, but historically shallowest groundwater has been at a depth of about 10 feet.
3. No soil layers were found to be plastic.

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 10 feet, and another assuming groundwater at a depth of 25 feet. The analysis assuming groundwater at 25 feet indicated one soil layer encountered at depths between 32 and 34.5 feet had a factor of safety below 1.3 (see Appendix D for calculations). When groundwater was assumed to be at a depth of 10 feet, none of the soils in the upper 50 feet had 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).

Because the shallowest potentially liquefiable zone is only 2.5 feet thick and is below 32 feet of non-liquefiable soils, ground damage, such as sand boils are ground cracks, would not be expected.

”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 can occur when adjusted blow counts ($N_{1(60)}$) measured within potentially liquefiable zones are less than 15. However, the $N_{1(60)}$ of the potentially liquefiable zone was calculated to be 25.9. As a result, lateral spreading does not appear to pose a hazard

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 RESTROOM BUILDING, 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 and lateral spreading effects, Earth Systems recommends that a geogrid reinforced aggregate mat be constructed beneath the proposed structures (bathroom building, ticket booths, and 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 to provide more uniform bearing conditions and additional lateral support in the upper soils. 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.

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 BATHROOM BUILDING, TICKET BOOTHS, GATEWAYS, 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 “medium” expansion range, perimeter footings should have a minimum embedment depth of 21 inches below lowest adjacent subgrade. Interior footings should have a minimum embedment depth of 18 inches below lowest adjacent subgrade.

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 21 inches deep and a minimum 15 inches wide may be designed based on an allowable bearing value of 2,000 psf. This value has a factor of safety of greater than 3.

Isolated pad footings with an assumed size of 24 inches by 24 inches by 18 inches deep may be designed based on an allowable bearing value of 2,500 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 0.58 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 340 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 “medium” expansion range.

Bearing soils in the “medium” expansion range should be premoistened to 130% of optimum moisture content to a depth of 27 inches below lowest adjacent grade. Premoistening should be confirmed by testing.

Drilled Pier Foundations

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. Drilled pier diameters of 1.5, 2.0, and 2.5 feet were analyzed, and the results are presented on the attached charts. Side resistance is not allowed to increase beyond a depth equal to 20 pile diameters. Upward resistance is taken as two-thirds of the downward resistance. The downward and upward capacity graphs for drilled piers are presented in Appendix E.

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

Reduction in axial capacity due to group effects should be considered for piers spaced at 3 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.

Lateral (horizontal) loads may be resisted by passive resistance of the soil against the piers. An equivalent fluid weight (EFW) of 340 psf per foot of penetration in the compacted fill (upper 5 feet) and an EFW of 250 pcf in the firm native soils above the groundwater table may be used for lateral load design. An EFW of 175 pcf may be used for lateral load design in the firm native soils below the groundwater table. These resisting pressures are ultimate values. The maximum passive pressure used for design should not exceed 3,500 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 2 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 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 7.5 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 feet below the final ground elevation.

The geotechnical engineers, or their representatives, 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 (sidewalks, plaza pavements, etc.) be designed relatively independent of footing stems (i.e. free floating) so foundation adjustment will be less likely to cause cracking. Because near-surface soils are in the "medium" expansion range, sidewalks and plaza pavements should be underlaid with 4 inches of aggregate base materials compacted to a minimum of 95% of the maximum dry density obtainable by the ASTM D 1557 test method. 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. (Note that structural paving sections for areas to be exposed to vehicular traffic are presented elsewhere in this report.)

Interior 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 "medium" 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 underlying slabs that are in the "medium" expansion range should be premoistened to 130% of optimum moisture content to a depth of 27 inches below lowest adjacent grade. Premoistening of slab areas should be observed and tested by this firm for compliance with these recommendations prior to placing of sand, reinforcing steel, or concrete.

Retaining Walls

Retaining walls should not be backfilled with on-site soils because of the expansive characteristics of those soils.

Conventional cantilever retaining walls that are backfilled at a 1:1 projection upward from the heels of the wall footings with crushed rock or non-expansive sand may be designed for active pressures of 35 pcf of equivalent fluid weight for well-drained, level backfill. An 18-inch thick cap of compacted native soils should be placed above the rock or sand. Filter fabric should be placed between the rock or sand and native soils and/or backfill over the top.

Restrained retaining walls that are backfilled at a 1:1 projection upward from the heels of the wall footings with crushed rock or non-expansive sand may be designed for at-rest pressures of 54 pcf of equivalent fluid weight for well-drained, level backfill. An 18-inch thick cap of compacted native soils should be placed above the rock or sand. Filter fabric should be placed between the rock or sand and native soils and/or backfill over the top.

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 foundations and floor slabs designed as recommended. Differential settlement between adjacent load bearing members should be expected to range up to about one-half the total settlement over a distance of 30 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 liquefaction potential indicate that as much as 1.5 inches of settlement could occur as a result of a significant earthquake. Approximately 0.8 inches of this total could potentially be experienced as differential settlement.

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 borings advanced within the site. The nature and extent of variations between and beyond the sounding and borings may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

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

Findings of this report are valid as of this date; however, changes in conditions of a property can occur with passage of time whether they are due to natural processes or works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of 1 year.

In the event that any changes in the nature, design, or location of the 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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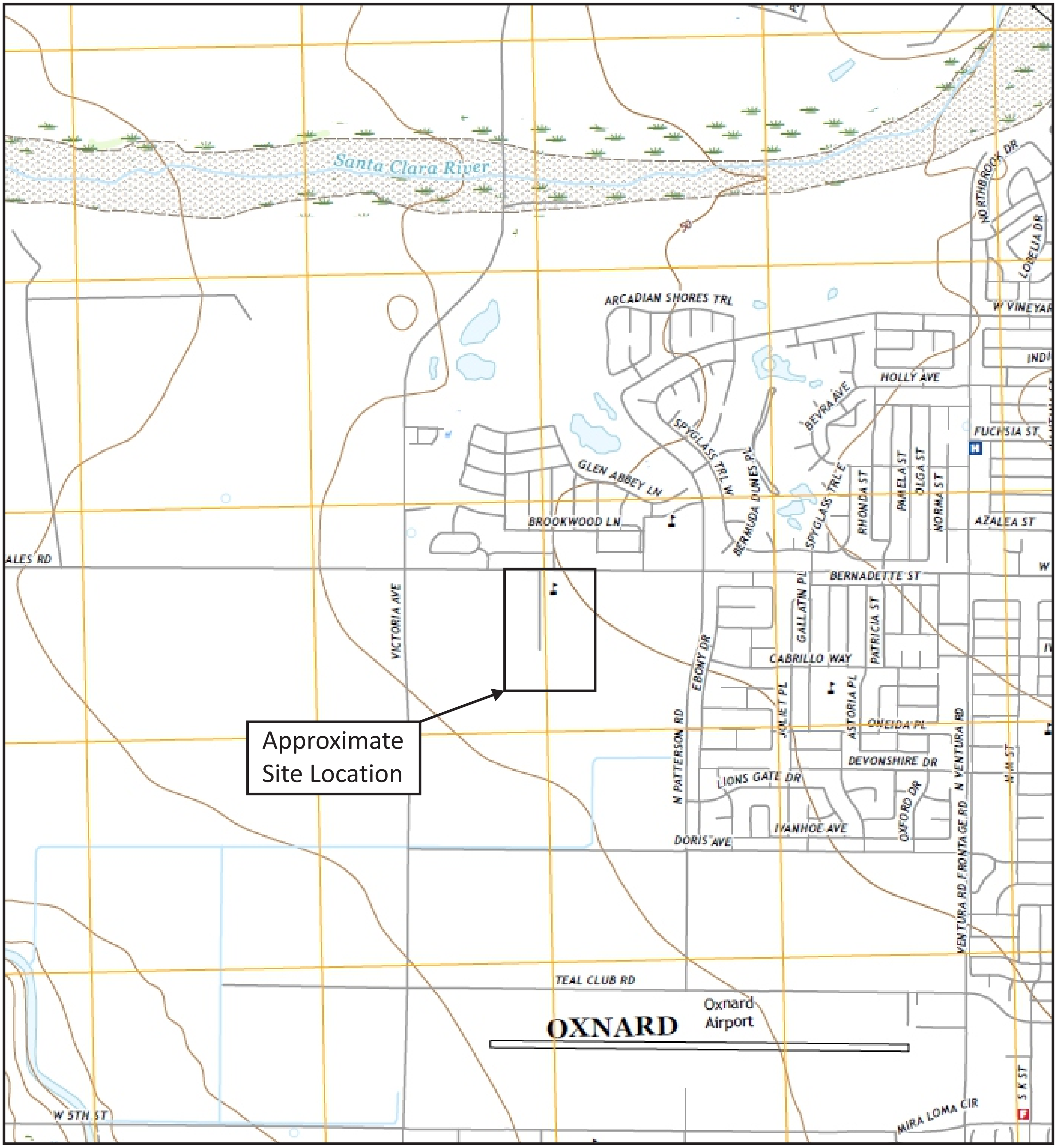
APPENDIX A



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

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

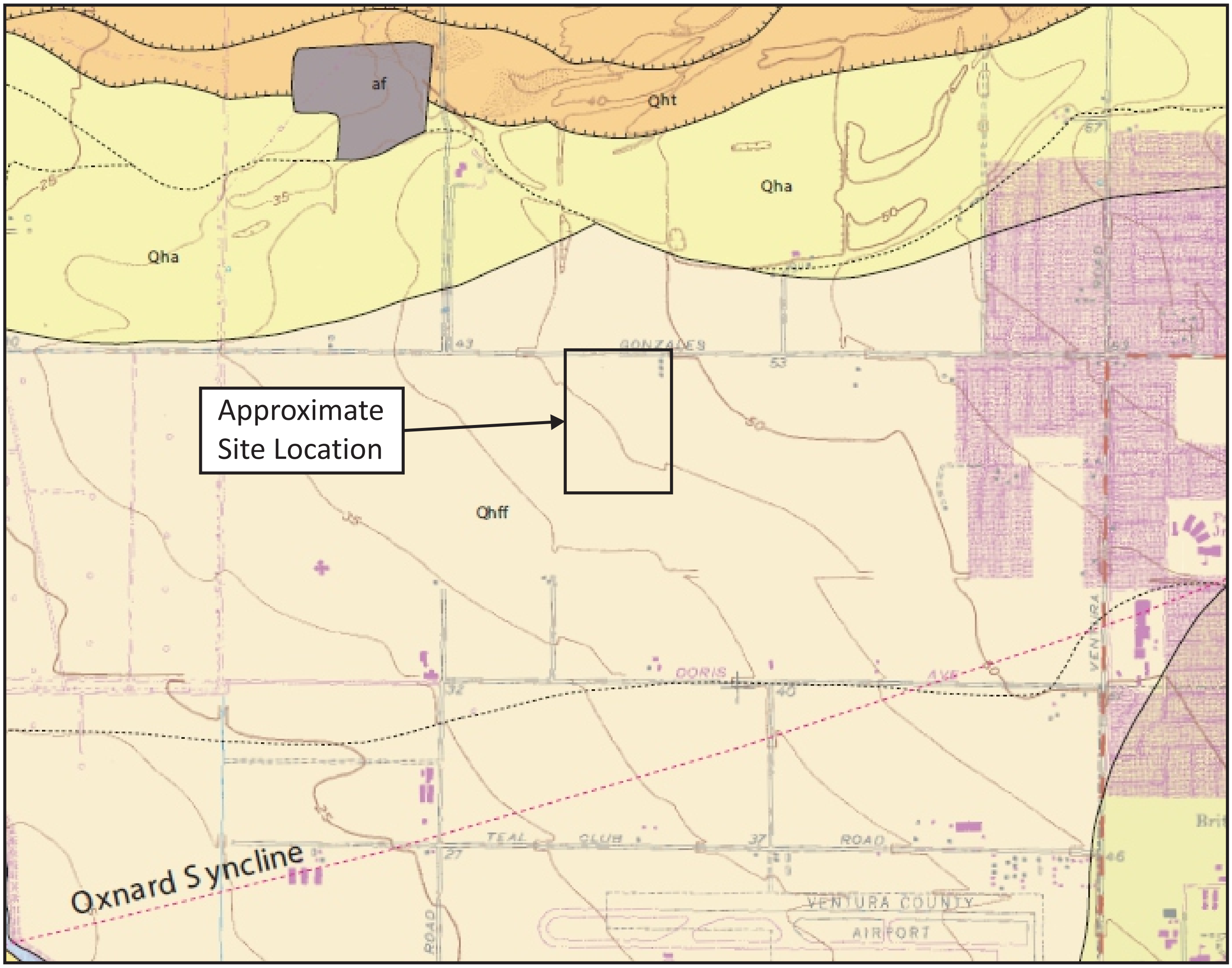
N

VICINITY MAP

Oxnard 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.

Qhff: Holocene alluvial fan deposits

Qha: Latest Holocene alluvial deposits

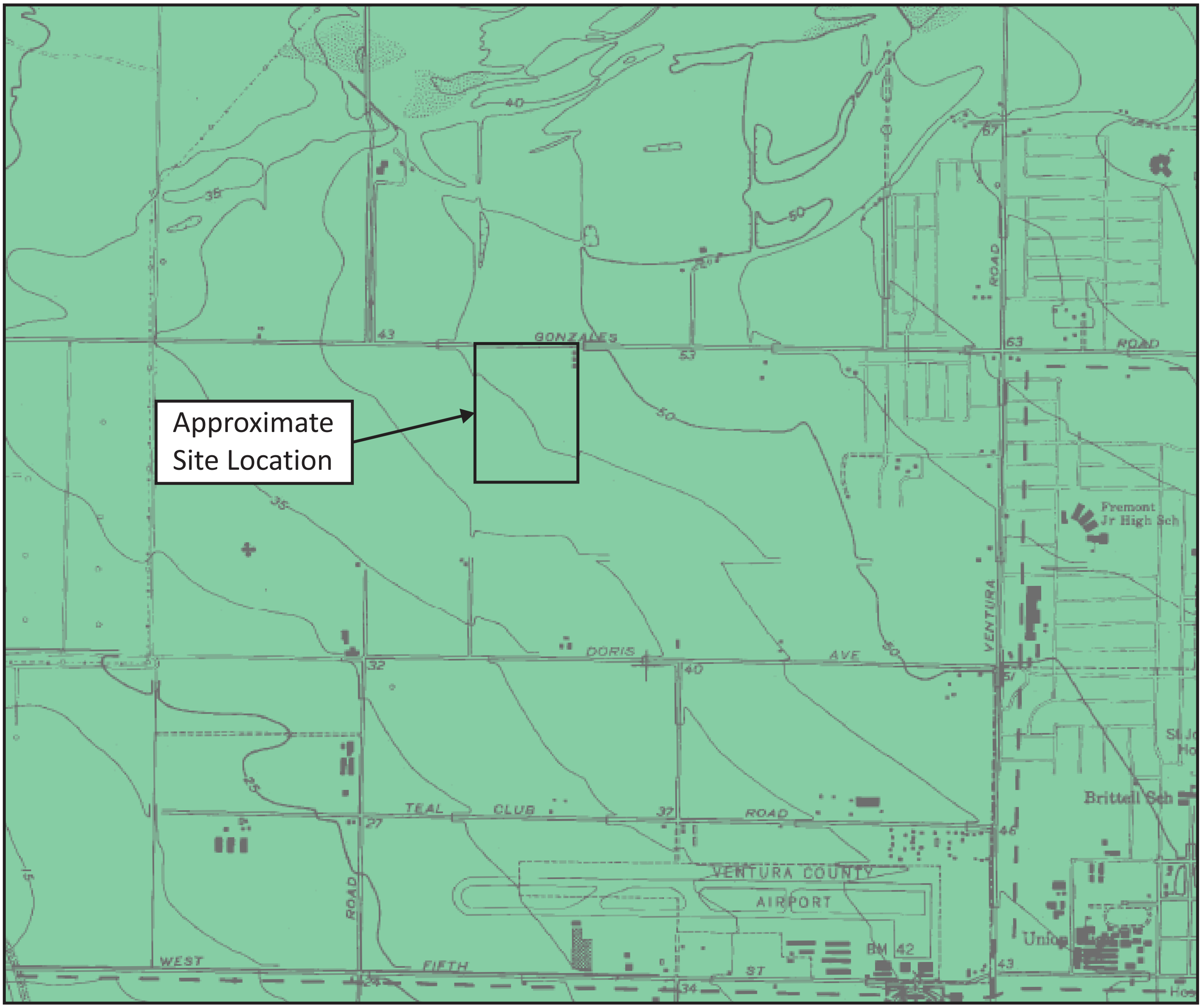
REGIONAL GEOLOGIC MAP

Oxnard 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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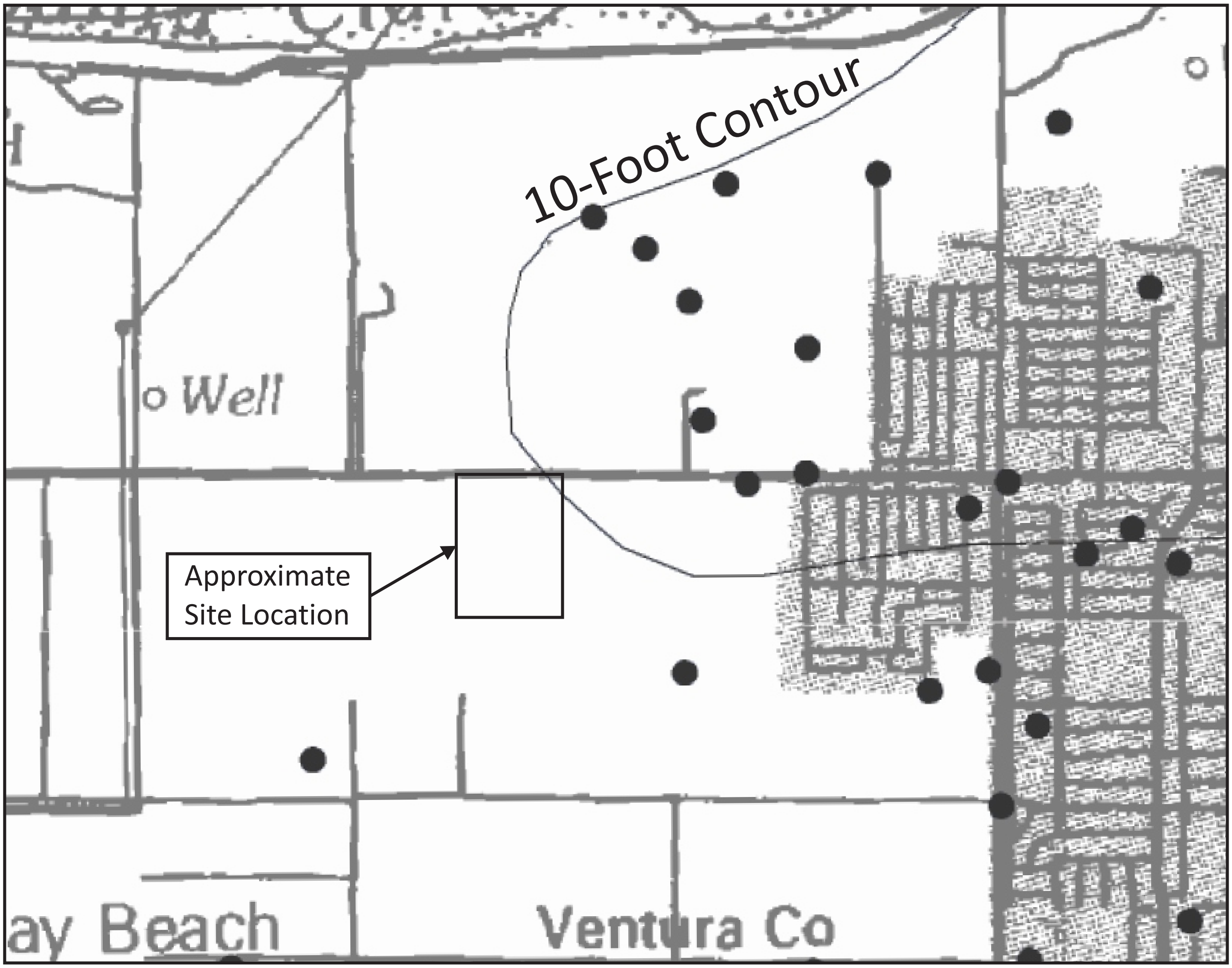
SEISMIC HAZARD ZONES MAP

Oxnard 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, 2002.

30 Depth to ground water in feet

● Borehole Site

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

N

HISTORICAL HIGH GROUNDWATER MAP

Oxnard High School Synthetic Field
Oxnard, California

Earth Systems

November 2019 | 303514-002

FIELD STUDY

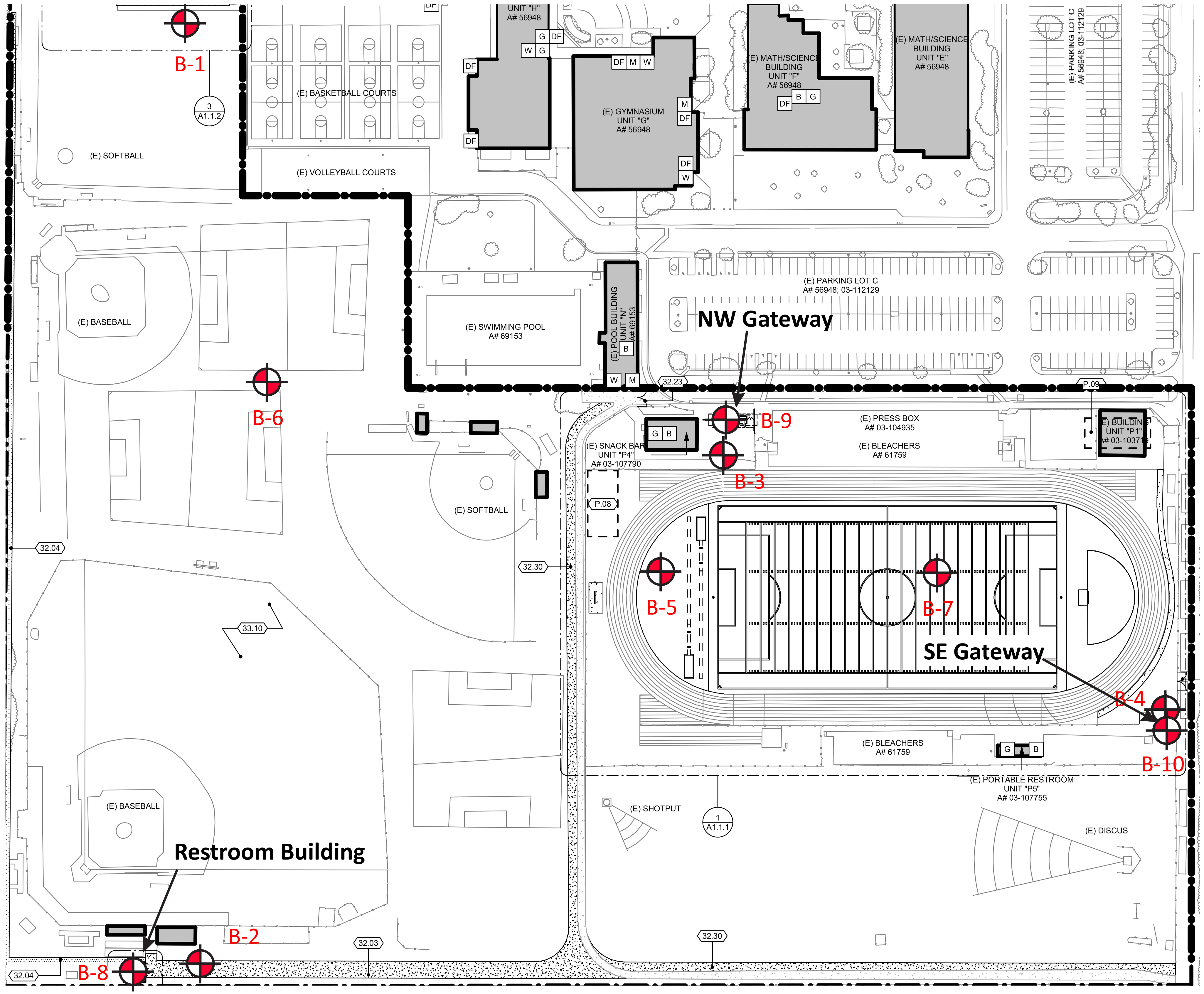
A. Seven borings were drilled to a maximum depth of 30 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory analysis. The borings were drilled on June 28, 2019, using an 8-inch diameter hollow stem auger powered by a track-mounted CME-75 drilling rig. 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 seven initial borings were supplemented by three additional borings. The supplemental borings were drilled between October 25 and 29, 2019, using a 4-inch diameter mud rotary system powered by a GTech 8 drilling rig. The borings were drilled to a maximum depth of 51.5 feet below the ground surface.

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-6 and B-7 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-1

: Approximate boring locations

N

Approximate Scale: 1" = 120’

0 120’ 240’

SITE PLAN

Oxnard High School
Oxnard, California

Earth Systems

November 2019

303514-002

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

BORING NO: B-1
PROJECT NAME: Oxnard HS Synthetic Field
PROJECT NUMBER: 303278-001
BORING LOCATION: Per Plan

DRILLING DATE: June 28, 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			■	20/25/21	▥	SM / ML	121.2	12.4	**ALLUVIUM:** Gray Brown Silty fine Sand to fine Sandy Silt, dense, damp
5			■	5/6/7	▦	SM	113.6	12.5	**ALLUVIUM:** Light Gray Brown Silty fine Sand, loose, damp
			⧄	6/11/15	⁘	SW			**ALLUVIUM:** Light Gray fine to medium Sand, trace coarse Sand, medium dense, dry to damp
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-2	DRILLING DATE: June 28, 2019
PROJECT NAME: Oxnard HS Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303278-001	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			■	16/28/37	▥	SM / ML	121.5	8.1	**ALLUVIUM:** Light Brown Silty fine Sand to fine Sandy Silt, trace Clay, trace calcareous veins, very dense, damp
5			■	8/16/23	⁘	SW	108.0	2.0	**ALLUVIUM:** Light Yellow Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, dense, dry to damp
			⧅	10/14/14	⁘	SW			**ALLUVIUM:** Light Yellow Brown fine to coarse Sand, trace fine Gravel, medium dense, dry to damp
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.

BORING NO: B-3
PROJECT NAME: Oxnard HS Synthetic Field
PROJECT NUMBER: 303278-001
BORING LOCATION: Per Plan

DRILLING DATE: June 28, 2019
DRILL RIG: CME-75
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: A. Luna

Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
	Bulk	SPT	Mod. Calif.						
0			■	12/12/11	▥	SM	111.8	9.8	**ALLUVIUM:** Light Brown Silty fine Sand, trace Clay, medium dense, damp
5			■	4/4/8	▥▨	ML-CL	102.1	16.1	**ALLUVIUM:** Dark Gray Brown Clayey Silt, firm to stiff, moist
			⧄	6/9/12	░	SP			**ALLUVIUM:** Light Yellow Brown fine Sand, little medium Sand, medium dense, dry to damp
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-4	DRILLING DATE: June 28, 2019
PROJECT NAME: Oxnard HS Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303278-001	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			■	6/9/12	▥	ML	105.9	11.9	**ALLUVIUM:** Dark Gray Brown fine Sandy Silt, trace calcareous veins, medium dense, damp
5			■	5/9/20	▦	SM	117.5	9.8	**ALLUVIUM:** Brown Silty fine Sand, trace Clay, medium dense, damp
			■	17/50-6"	▩	SC			**ALLUVIUM:** Light Brown Clayey fine Sand, very dense, damp
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: Oxnard HS Synthetic Field
PROJECT NUMBER: 303278-001
BORING LOCATION: Per Plan

DRILLING DATE: June 28, 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									
			■	15/28/31		ML	122.7	12.4	**ALLUVIUM:** Dark Gray Brown fine Sandy Silt, trace calcareous veins, medium dense, damp
5			■	12/18/22		SM / ML	120.2	15.1	**ALLUVIUM:** Brown Silty fine Sand to fine Sandy Silt, trace Clay, medium dense, damp
			■	6/11/16			117.7	14.6	
10			■			SP			**ALLUVIUM:** Light Yellow Brown fine Sand, trace medium to coarse Sand, medium dense, dry to damp
15									
20						SW			**ALLUVIUM:** Brown fine to medium Sand, trace coarse Sand, little fine Gravel, medium dense, dry to damp
25	▼								
30									Total Depth: 30 feet Groundwater Depth: 27 feet
35									

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

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BORING NO: B-6	DRILLING DATE: June 28, 2019
PROJECT NAME: Oxnard HS Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303278-001	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	☒				▥				
	☒		■	10/14/15	▥	ML	106.6	1.8	**ALLUVIUM:** Light Yellow Brown fine to medium Sandy Silt, trace coarse Sand, trace fine Gravel, very stiff, dry to damp
5			■	8/12/17	▥		108.9	2.7	
			■	8/16/26	⁘	SW	108.5	2.5	**ALLUVIUM:** Light Yellow Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, dense, dry to damp
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.

BORING NO: B-7
PROJECT NAME: Oxnard HS Synthetic Field
PROJECT NUMBER: 303278-001
BORING LOCATION: Per Plan

DRILLING DATE: June 28, 2019
DRILL RIG: CME-75
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: A. Luna

Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
	Bulk	SPT	Mod. Calif.						
0	☒		■	12/30/33	▥	ML	119.0	14.4	**ARTIFICIAL FILL:** Black fine Sandy Silt, trace medium to coarse Sand, trace fine Gravel, dense, damp
5			■	7/7/12	▥	ML	111.2	19.7	**ALLUVIUM:** Dark Gray to Black fine Sandy Silt, little Clay, medium dense, damp
			■	4/6/6	▥	ML		46.4	**ALLUVIUM:** Brown fine Sandy Silt, trace Clay, loose, 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.

BORING NO: B-8
PROJECT NAME: Oxnard HS
PROJECT NUMBER: 303514-002
BORING LOCATION: Per Plan

DRILLING DATE: October 28, 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		■		16/28/37		SM / ML			**ALLUVIUM:** Light Brown Silty fine Sand to fine Sandy Silt, trace Clay, trace calcareous veins, very dense, damp
5		■		8/16/23		SW			**ALLUVIUM:** Light Yellow Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, dense, dry to damp
		■		7/11/12		SW			**ALLUVIUM:** Light Brown fine to medium Sand, little Silt, medium dense-damp
10		■ ■		7/11/15 12/17/20		SW			**ALLUVIUM:** Light Brown fine to medium Sand, little Silt, trace coarse Sand, trace fine Gravel, medium dense to dense-damp
15		■		8/3/4		SM	-	15.0	**ALLUVIUM:** Dark Gray Brown Silty fine to coarse Sand, some Gravel, 2in Clay layer, medium dense-moist
20 ▼		■ ■		11/13/15 14/16/16		SP			**ALLUVIUM:** Gray Brown fine Sand, little Silt, medium dense-moist to very moist
25		■ ■		16/18/17 10/11/14		SW			**ALLUVIUM:** Gray Brown fine to coarse Sand, trace fine Gravel, little Silt, medium dense to dense-wet
		■		6/9/7		ML	-	28.1	**ALLUVIUM:** Dark Gray Sandy Silt, little Clay, very stiff-moist to very moist
30		■		12/16/19		SP			**ALLUVIUM:** Dark Gray fine Sand, little Silt, dense-very dense
35		■ ■ ■		12/15/17 16/20/24 13/14/17		SW			**ALLUVIUM:** Gray fine to medium Sand, little coarse Sand, trace 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-8 (Continued)	DRILLING DATE: October 28, 2019
PROJECT NAME: Oxnard 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			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	**DESCRIPTION OF UNITS**
	Bulk	SPT	Mod. Calif.						
40		■		16/17/18	░	SW			**ALLUVIUM:** Gray fine to medium Sand, little coarse Sand, trace Silt, dense-wet
		■		14/14/16	░	SW			**ALLUVIUM:** Gray fine to coase Sand, trace Silt, little fine to coarse Gravel, medium dense to dense-wet
45		■		11/21/17	░				
		■		17/20/21	▒	SP			**ALLUVIUM:** Gray fine Sand, trace Silt, medium dense-wet
50		■		16/12/10	░	SW			**ALLUVIUM:** Gray fine to medium Sand, little Silt, trace coarse Sand, medium dense-wet
									Total Depth: 51.5 feet **Groundwater Depth: 22.0 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.

BORING NO: B-9
PROJECT NAME: Oxnard HS
PROJECT NUMBER: 303514-002
BORING LOCATION: Per Plan

DRILLING DATE: October 25, 2019
DRILL RIG: Gtech 8
DRILLING METHOD: Mud Rotary
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		■		12/12/11	⁘	SM			**ALLUVIUM:** Light Brown Silty fine Sand, trace Clay, medium dense, damp
5		■		4/4/8	▨	ML-CL			**ALLUVIUM:** Dark Gray Brown Clayey Silt, firm to stiff, moist
		■		5/7/11	⁘	SW			**ALLUVIUM:** Light Gray Brown fine to medium Sand, trace to little Silt, medium dense to dense-damp to moist
10		■		9/14/16	⁘				
		■		8/10/21	⁘	SW			**ALLUVIUM:** Light Gray Brown fine to medium Sand, trace Silt, trace coarse Sand, trace fine Gravel, medium dense-damp to moist
15		■		15/22/27	⁘	SW			**ALLUVIUM:** Gray fine to coarse Sand, trace Silt, trace fine Gravel, dense-dry to damp
▼ 20		■		10/12/7	⁘	SW			**ALLUVIUM:** Dark Gray Gravelly fine to coarse Sand, trace Silt, medium dense-wet
		■		10/20/27	⁘	SW			**ALLUVIUM:** Dark Gray Brown fine to medium Sand, little coarse Sand, little fine Gravel, dense-very moist to wet
25		■		12/18/20	⁘				
		■		10/11/11	▨	ML - CL	-	40.0	**ALLUVIUM:** Dark Gray fine Sandy Silt, very stiff-very moist
30		■		11/6/4	⁘	SW			**ALLUVIUM:** Gray fine to coarse Sand, trace to little Silt, loose to medium dense-very moist
		■		16/20/24	⁘	SW			**ALLUVIUM:** Gray fine to medium Sand, trace Silt, dense-very moist to wet
35		■		15/17/20	⁘	SW			**ALLUVIUM:** Gray fine to medium Sand, little coarse Sand, trace Silt, trace fine Gravel, dense-very moist to 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-9 (Continued)
PROJECT NAME: Oxnard HS
PROJECT NUMBER: 303514-002
BORING LOCATION: Per Plan

DRILLING DATE: October 25, 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
40		■		18/25/29	░	SW			**ALLUVIUM:** Gray fine to medium Sand, little coarse Sand, trace Silt, trace fine Gravel, very dense-very moist to wet
		■		17/20/21	░	SW			**ALLUVIUM:** Gray fine to coarse Sand, little fine Gravel, trace Silt, dense-very moist to wet
45		■		20/17/16	░				
		■		17/12/6	░	SW			**ALLUVIUM:** Gray fine to medium Sand, trace to little Silt, medium dense-wet
50		■		15/18/18	░	SW			**ALLUVIUM:** Gray fine to coarse Sand, some medium Sand, dense-very moist to wet
									Total Depth: 51.5 feet **Groundwater Depth: 20.0 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.

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BORING NO: B-10
PROJECT NAME: Oxnard HS
PROJECT NUMBER: 303514-002
BORING LOCATION: Per Plan

DRILLING DATE: October 29, 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				6/9/12		ML			**ALLUVIUM:** Dark Gray Brown fine Sandy Silt, trace calcareous veins, medium dense, damp
5				5/9/20		SM			**ALLUVIUM:** Brown Silty fine Sand, trace Clay, medium dense, damp
				8/12/13		SW			**ALLUVIUM:** Light Gray Brown Silty fine Sand, trace iron oxide staining, medium dense-dry to damp
10				7/11/12		SW			**ALLUVIUM:** Brown fine to medium Sand, trace coarse Sand, trace Silt, medium dense-very moist to wet
				8/12/13		SW			**ALLUVIUM:** Light Gray Brown fine to medium Sand, trace Silt, medium dense-dry to damp
15				10/18/21 13/15/16		SW			**ALLUVIUM:** Light Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, trace to little Silt, dense-very moist to wet
20				10/6/10		SM			**ALLUVIUM:** Dark Gray Silty fine Sand, little Clay, medium dense-very moist to wet
				10/13/15		SW			**ALLUVIUM:** Gray fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense-moist to very moist
25 30				16/20/21 22/16/21 14/17/18 14/7/13		SW			**ALLUVIUM:** Gray Brown fine to coarse Sand, trace to little Silt, medium dense to dense-wet
35				8/20/26 18/22/19		SW			**ALLUVIUM:** Gray fine to medium Sand, little coarse Sand, trace 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-10 (Continued)
PROJECT NAME: Oxnard HS
PROJECT NUMBER: 303514-002
BORING LOCATION: Per Plan

DRILLING DATE: October 29, 2019
DRILL RIG: Gtech 8
DRILLING METHOD: Mud Rotary
LOGGED BY: A. Luna

Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
	Bulk	SPT	Mod. Calif.						
40		■		19/20/22	▦	SM			**ALLUVIUM:** Gray Brown Silty fine to medium Sand, little coarse Sand, little fine to coarse Gravel, dense-wet
		■		16/17/20	░	SW			**ALLUVIUM:** Gray Brown fine to coarse Sand, little Silt, trace to little fine to coarse Gravel, dense-wet
45		■		14/18/19	░				
		■		21/33/30	░				
50		■		22/28/34	░	SW			**ALLUVIUM:** Gray Brown fine to medium Sand, little Silt, trace coarse Sand, trace fine to coarse Gravel, very dense-wet
									Total Depth: 51.5 feet **Groundwater Depth: 25.0 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.

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 a selected sample was evaluated by hydrometer (in accordance with ASTM D 422) and sieve analysis procedures. The sample was 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, a hydrometer analysis was performed to assess the distribution of the minus No. 200 mesh material of the sample. The hydrometer portion of the test was run using sodium hexametaphosphate as a dispersing agent.

LABORATORY TESTING (Continued)

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-6 @ 0-5’	B-7 @ 0-5’
USCS	ML	ML
MAXIMUM DENSITY (pcf)	115.0	--
OPTIMUM MOISTURE (%)	12.0	--
COHESION (psf)	260* 180**	--
ANGLE OF INTERNAL FRICTION	28°* 30°**	--
EXPANSION INDEX	65	--
RESISTANCE (“R”) VALUE	--	24
pH	8.4	--
SOLUBLE CHLORIDES (mg/Kg)	190	--
RESISTIVITY (ohms-cm)	820	--
SOLUBLE SULFATES (mg/Kg)	1,300	--
GRAIN SIZE DISTRIBUTION (%)		
GRAVEL	0	--
SAND	43	--
SILT AND CLAY	57	--

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

BORING AND DEPTH	B-8 @ 15’	B-8 @ 27.5’
USCS	SM	ML
LIQUID LIMIT	24	29
PLASTIC LIMIT	18	27
PLASTICITY INDEX	6	2
GRAIN SIZE DISTRIBUTION (%)		
GRAVEL	17.8	0.1
SAND	59.3	20.4
SILT	13.8	63.1
CLAY (2ųm to 5ųm)	3.5	4.9
CLAY (≤2ųm)	5.6	11.5

BORING AND DEPTH	B-9 @ 27.5	B-9 @ 50’	B-10 @ 20’
USCS	ML	SP	SM
LIQUID LIMIT	46	--	--
PLASTIC LIMIT	28	--	--
PLASTICITY INDEX	18	--	--
GRAIN SIZE DISTRIBUTION (%)			
GRAVEL	0.8	0.1	0.1
SAND	14.0	88.4	56.9
SILT	49.3	6.7	26.5
CLAY (2ųm to 5ųm)	11.4	1.6	5.0
CLAY (≤2ųm)	24.5	3.2	11.5

File Number: 303277-001　　Lab Number: 098208

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Oxnard High School Synthetic Turf Field
Sample ID: B 6 @ 0-5'

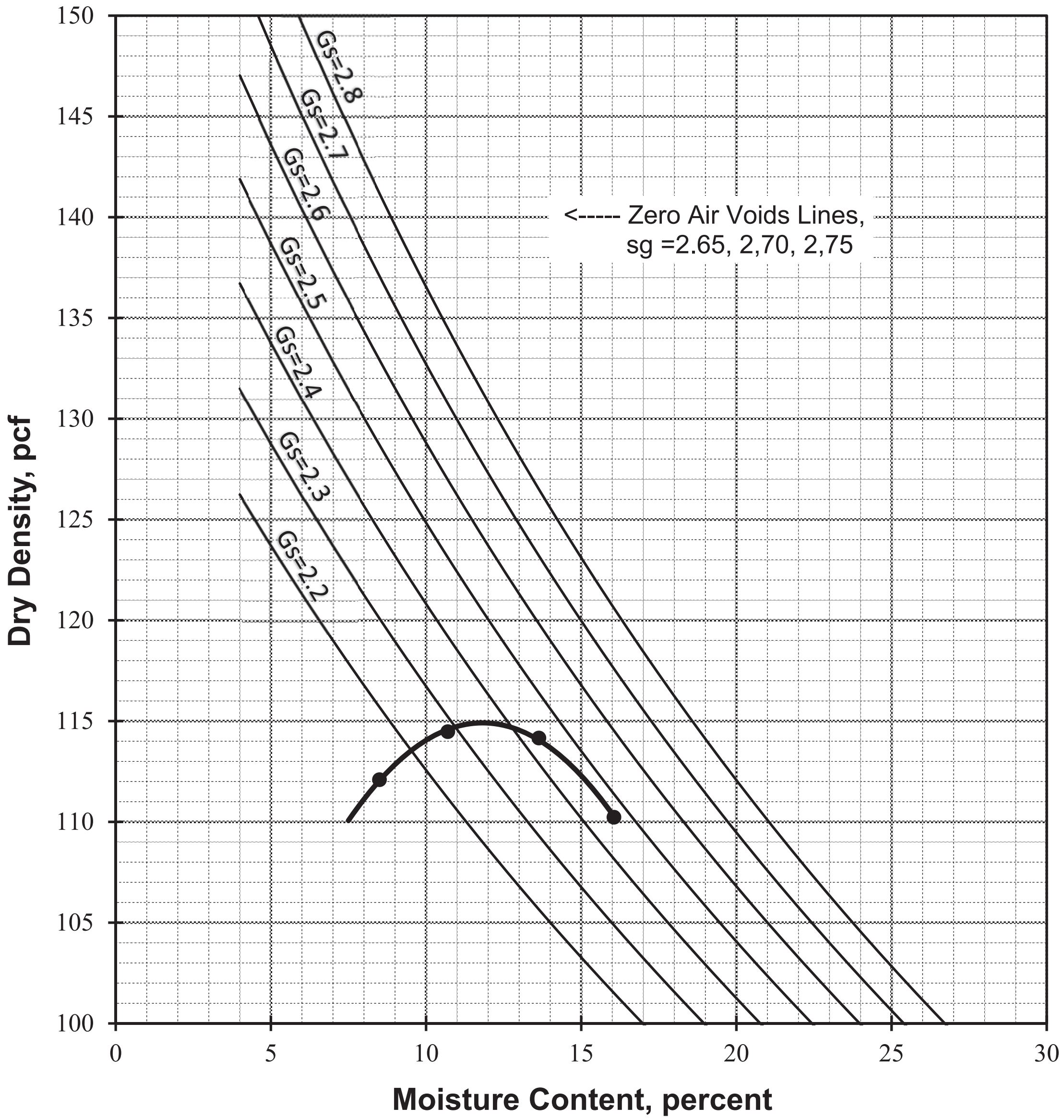
Procedure Used: A
Prep. Method: Moist

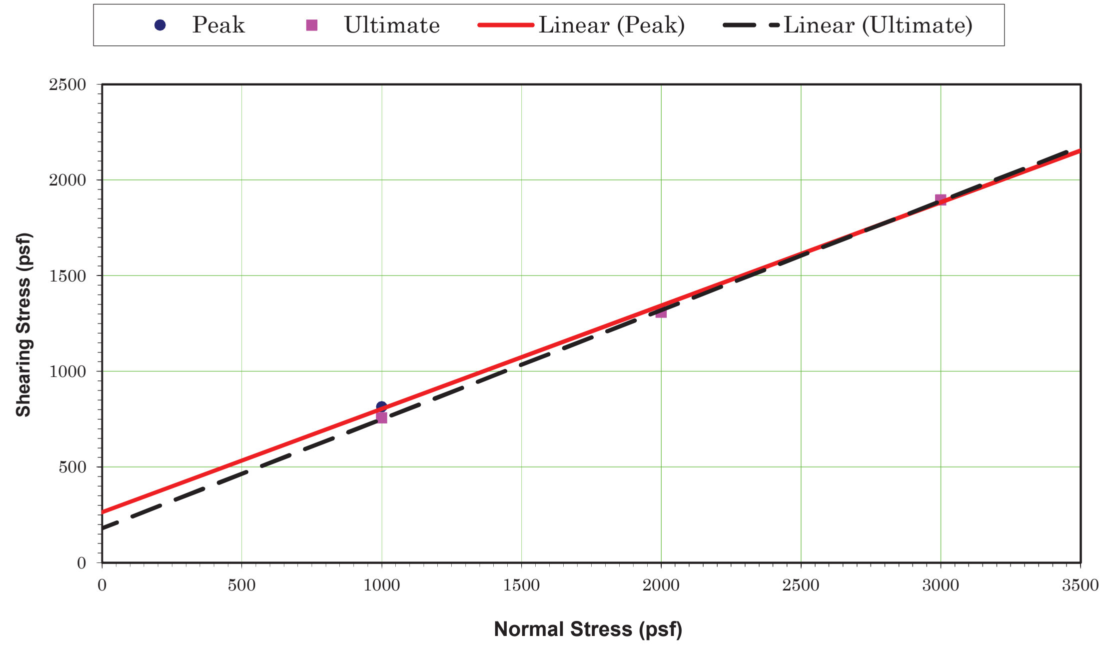
Date: 7/29/2019
Description: Dark Brown Sandy Silt
SG: 2.35

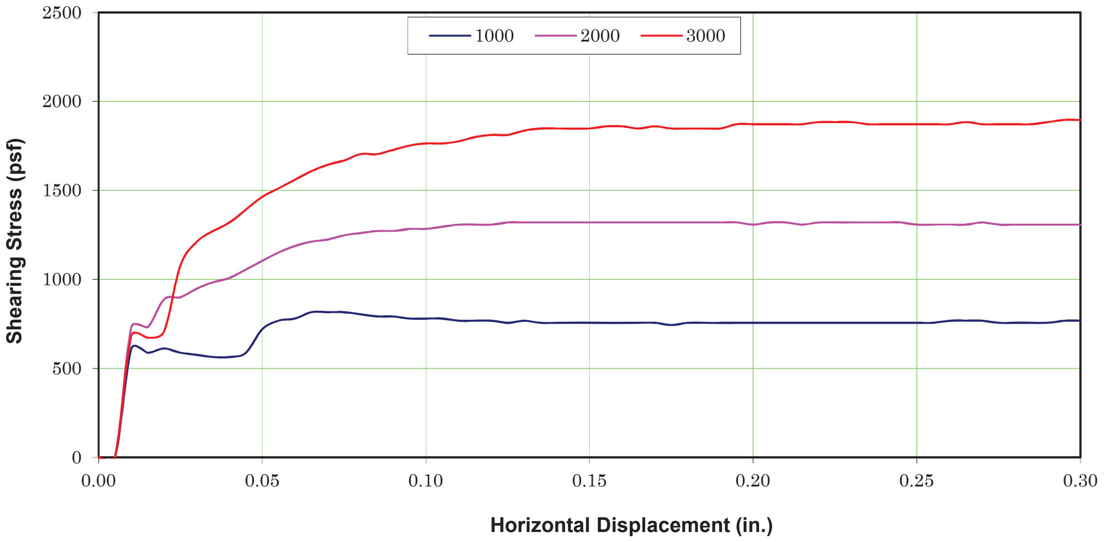
Rammer Type: Automatic

Maximum Density: **115 pcf**
Optimum Moisture: **12%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	0.5







DIRECT SHEAR DATA*

Sample Location: B 6 @ 0-5'
Sample Description: Sandy Silt
Dry Density (pcf): 103.8
Intial % Moisture: 11.8
Average Degree of Saturation: 100.0
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	816	1320	1896
Ultimate stress (psf)	756	1308	1896

	Peak	Ultimate
ϕ Angle of Friction (degrees):	28	30
c Cohesive Strength (psf):	260	180

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Oxnard High School Synthetic Turf Field

Earth Systems

8/27/2019 303278-001

File No.: 303278-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

Job Name: Oxnard High School Synthetic Turf Field
Sample ID: B 6 @ 0-5'
Soil Description: ML

Initial Moisture, %:	10.0
Initial Compacted Dry Density, pcf:	108.2
Initial Saturation, %:	49
Final Moisture, %:	21.7
Volumetric Swell, %:	6.5

Expansion Index: 65 Medium

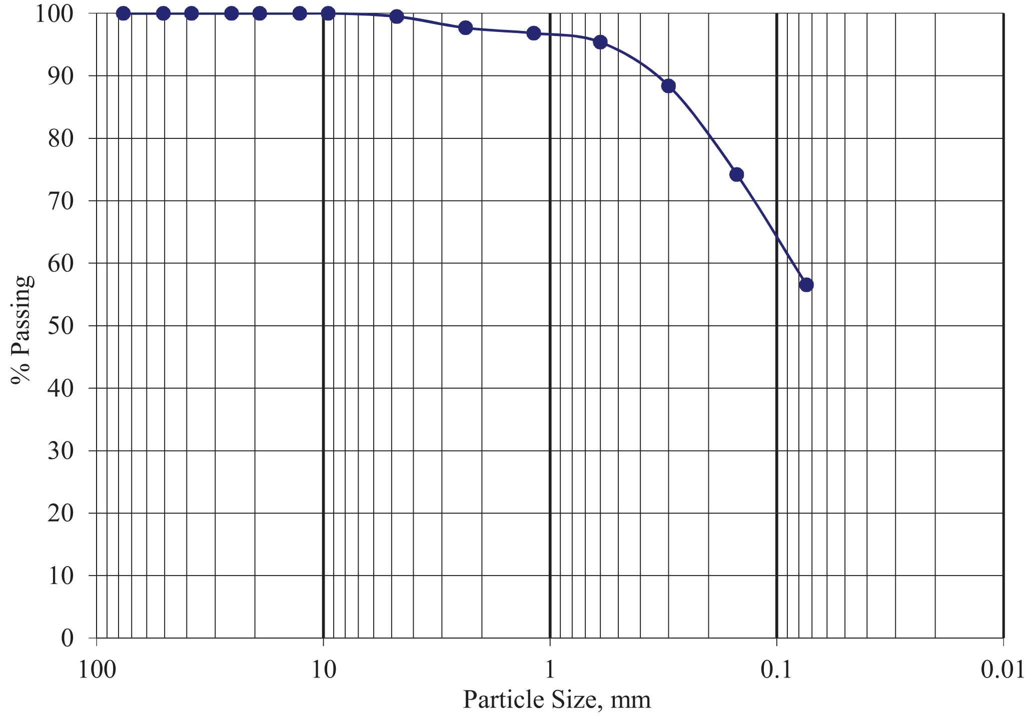
EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
130+	Very High

SIEVE ANALYSIS

ASTM C-136

Job Name: 303278-001
Sample ID: B 7 @ 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	98
#16	97
#30	95
#50	88
#100	74
#200	57



Oxnard High School
Synthetic Turf Field

303278-001

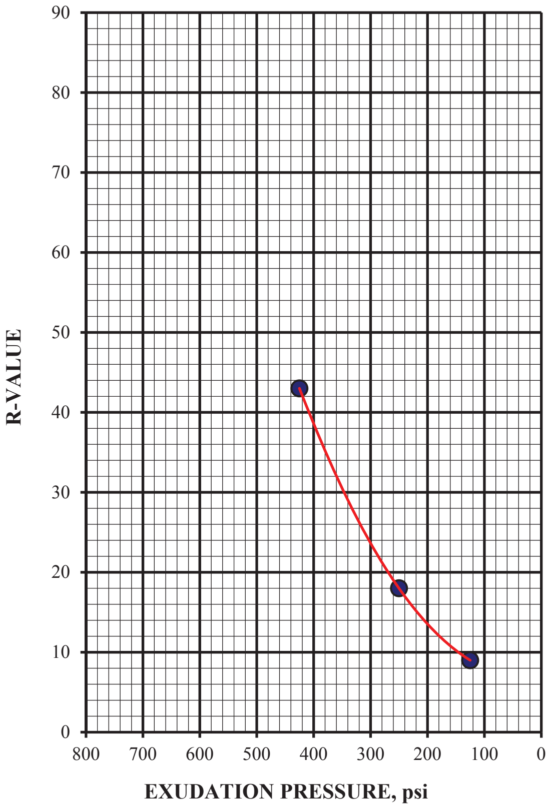
RESISTANCE '*R* ' VALUE AND EXPANSION PRESSURE

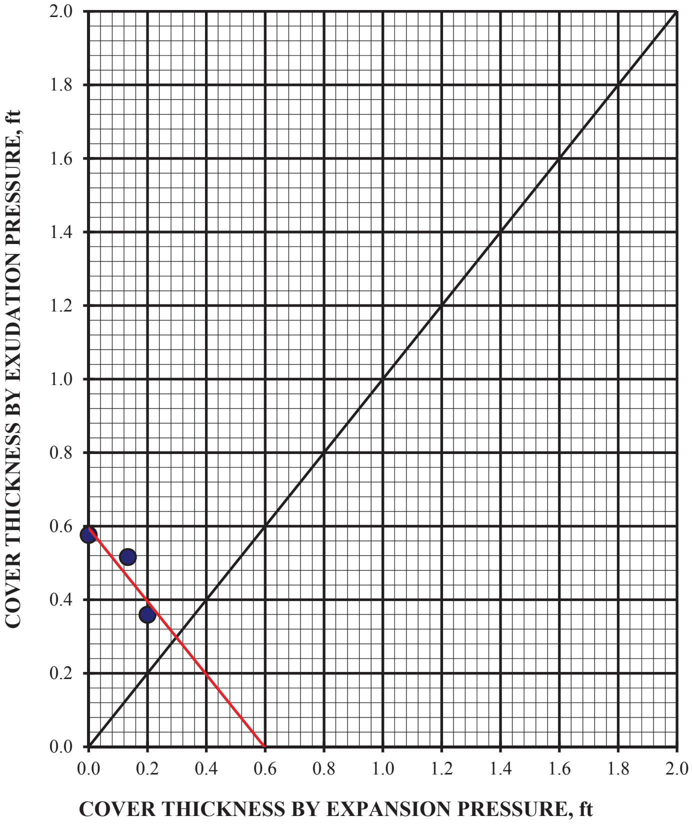
ASTM D 2844/D2844M-13

August 9, 2019

Boring #7 @ 0.0 - 5.0'
Dark Gray Sandy Silt (ML)
Specified Traffic Index: 5.0

Dry Density @ 300 psi Exudation Pressure: 119.9-pcf
%Moisture @ 300 psi Exudation Pressure: 16.1%
R-Value - Exudation Pressure: 24
R-Value - Expansion Pressure: 53
R-Value @ Equilibrium: 24

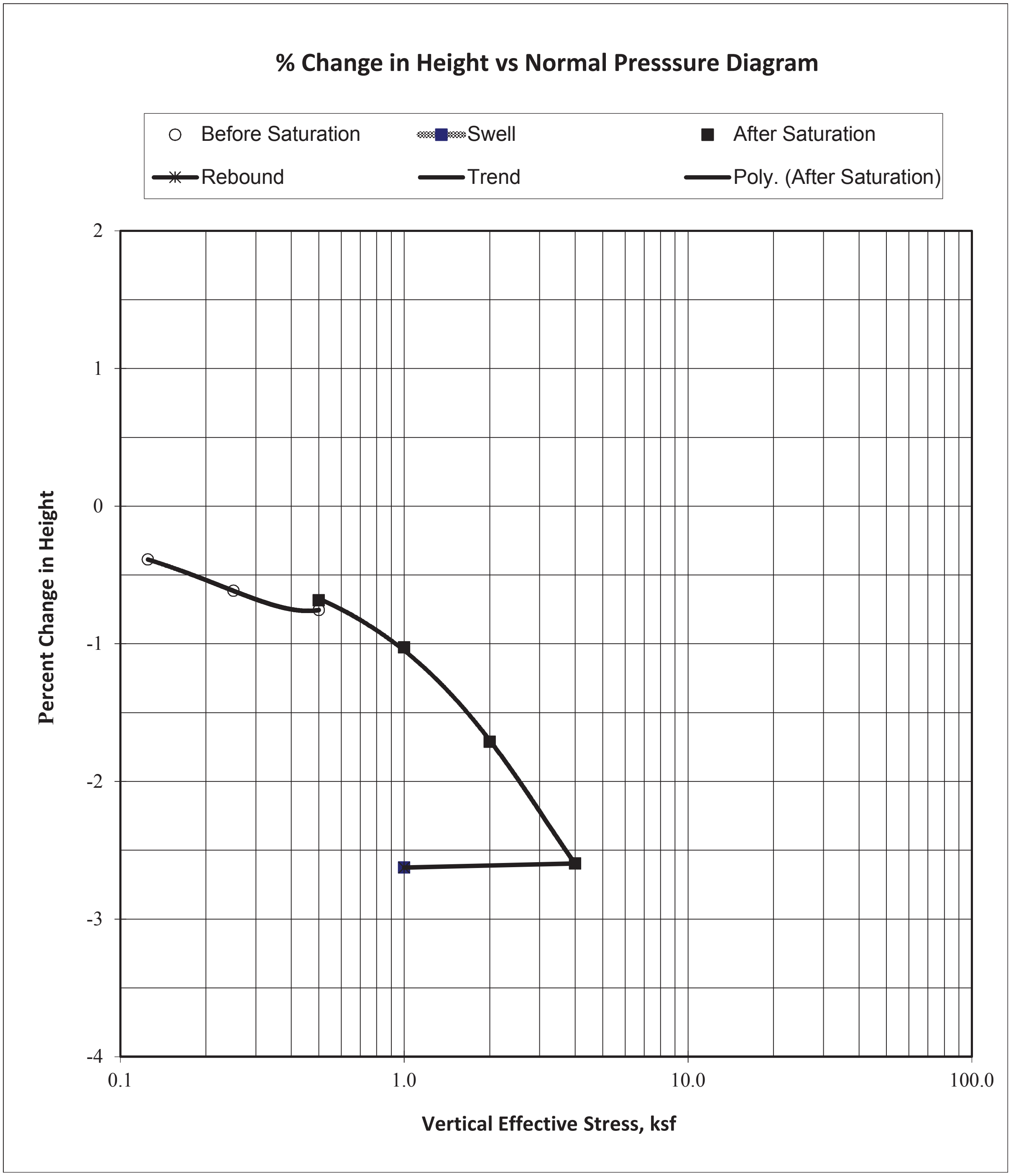

EXUDATION PRESSURE CHART


EXPANSION PRESSURE CHART

CONSOLIDATION TEST

ASTM D 2435-90

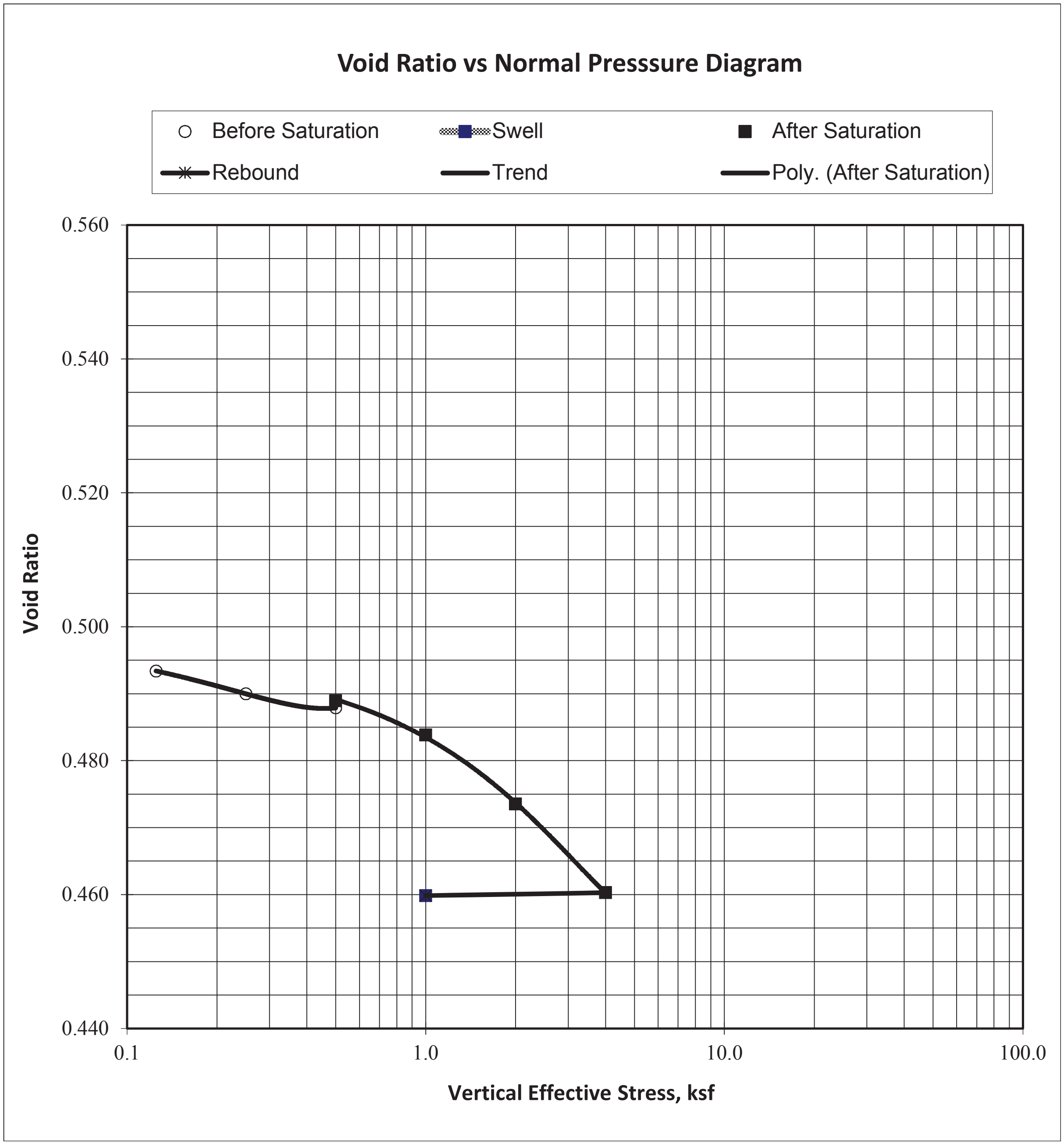
Oxnard High School Synthetic Turf Field		Initial Dry Density:	111.2 pcf
B 7 @5'		Initial Moisture, %:	19.7%
Silty Sand		Specific Gravity:	2.67 (assume
Ring Sample		Initial Void Ratio:	0.499



CONSOLIDATION TEST

ASTM D 2435-90

Oxnard High School Synthetic Turf Field		Initial Dry Density:	111.2
B 7 @5'		Initial Moisture, %:	19.7
Silty Sand		Specific Gravity:	2.67 (assume
Ring Sample		Initial Void Ratio:	0.499

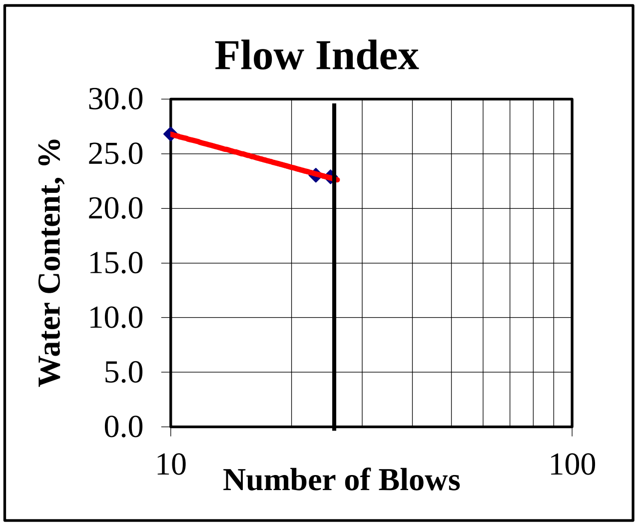


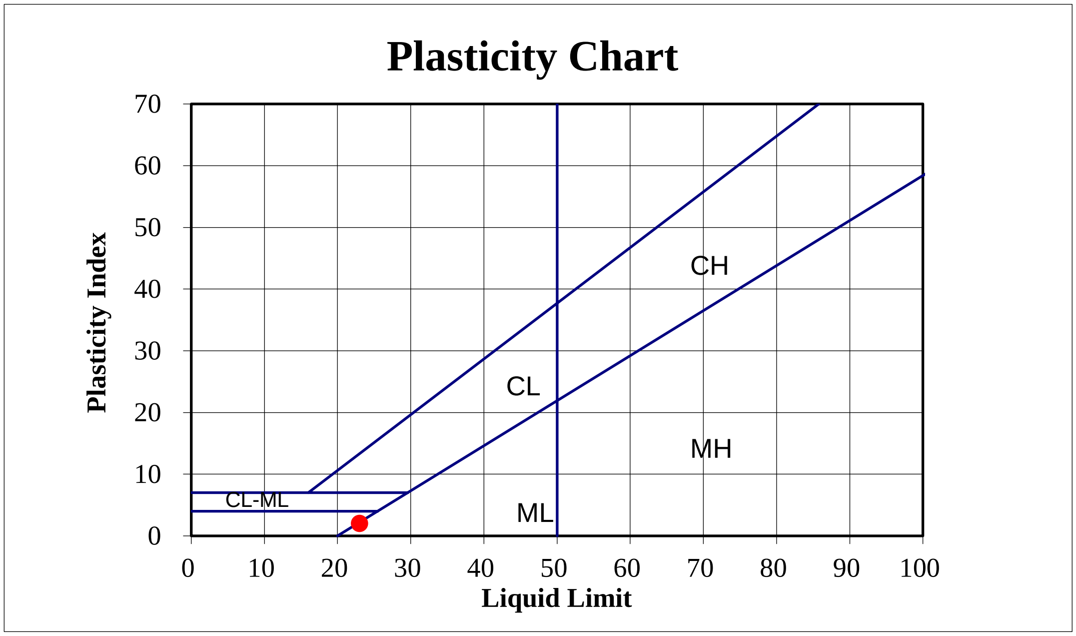
PLASTICITY INDEX

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**





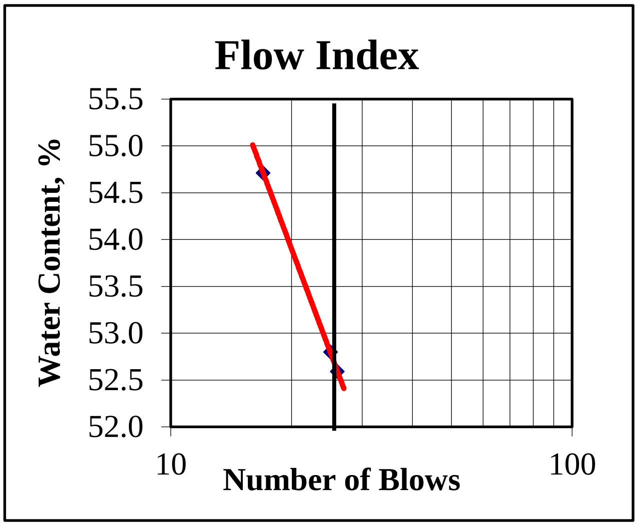
File No.: 303514-002 November 19, 2019

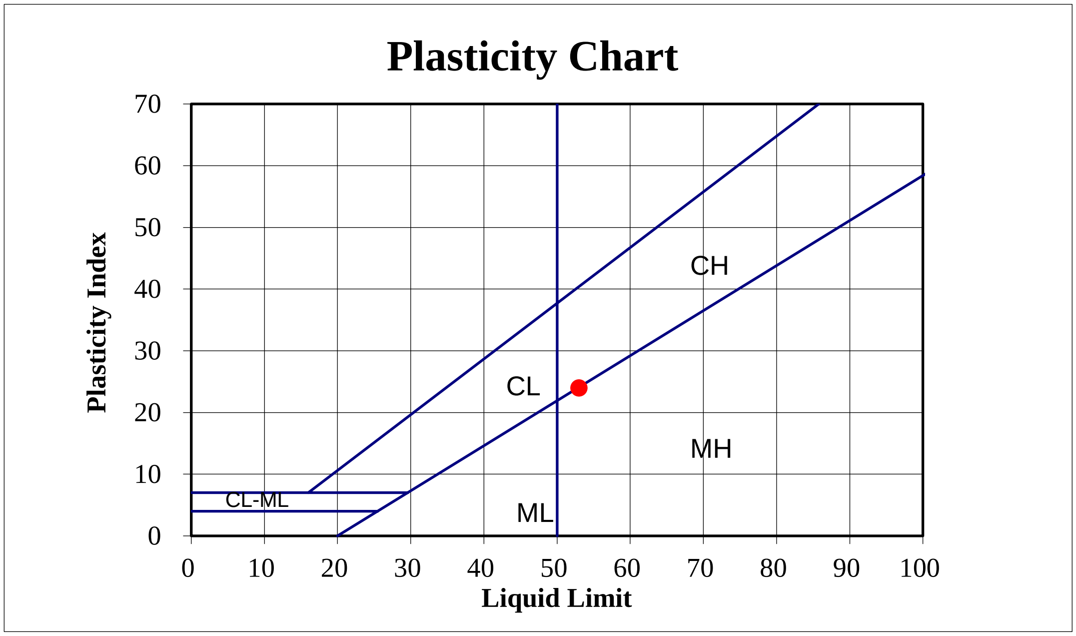
PLASTICITY INDEX

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**





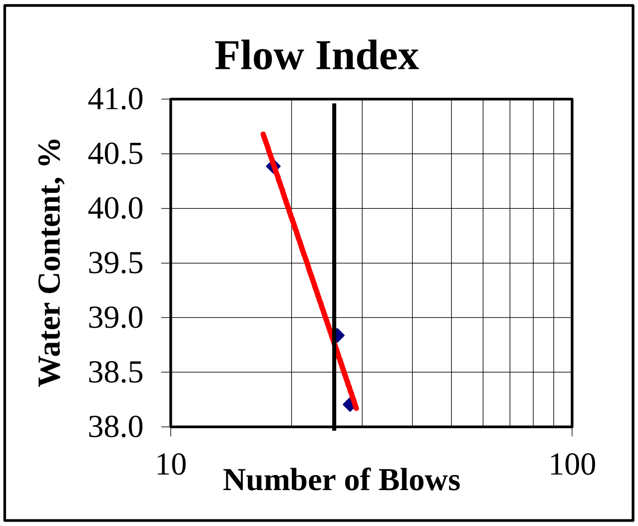
File No.: 303514-002 November 19, 2019

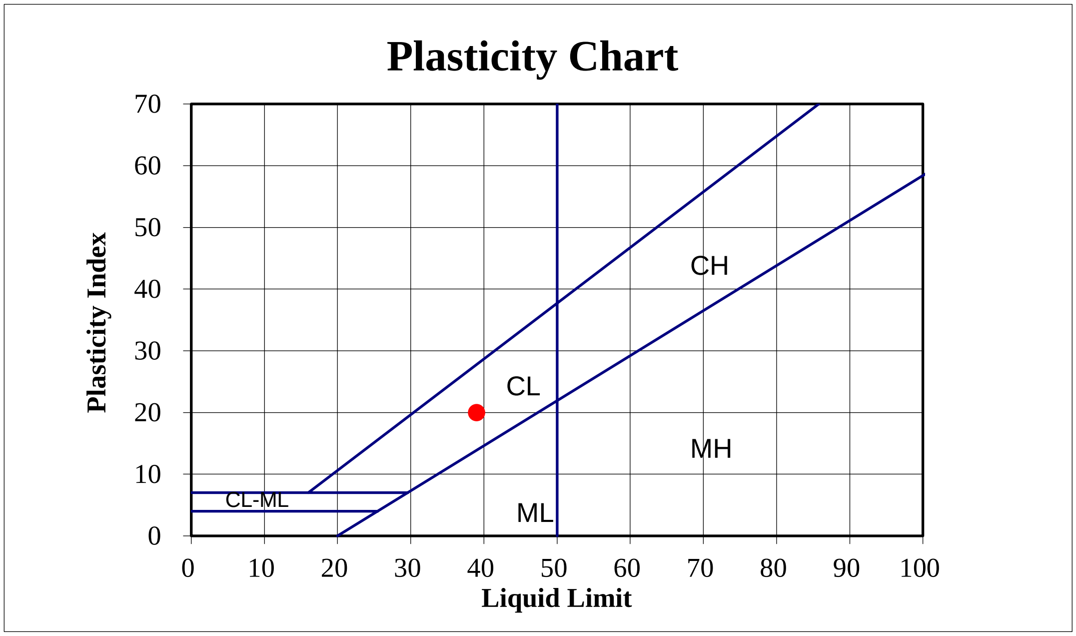
PLASTICITY INDEX

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**





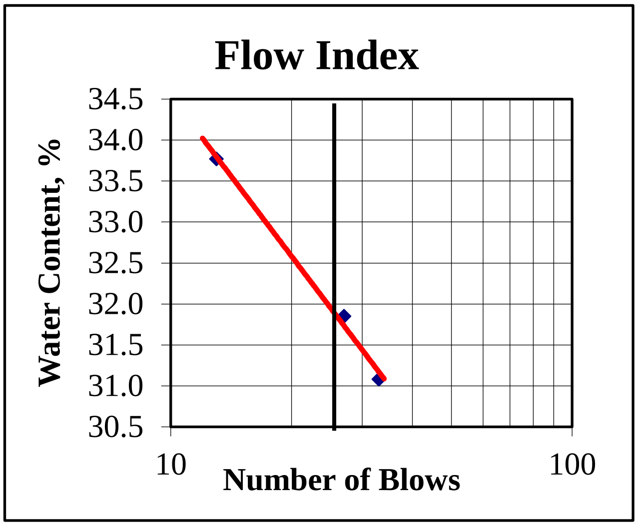
File No.: 303514-002 November 19, 2019

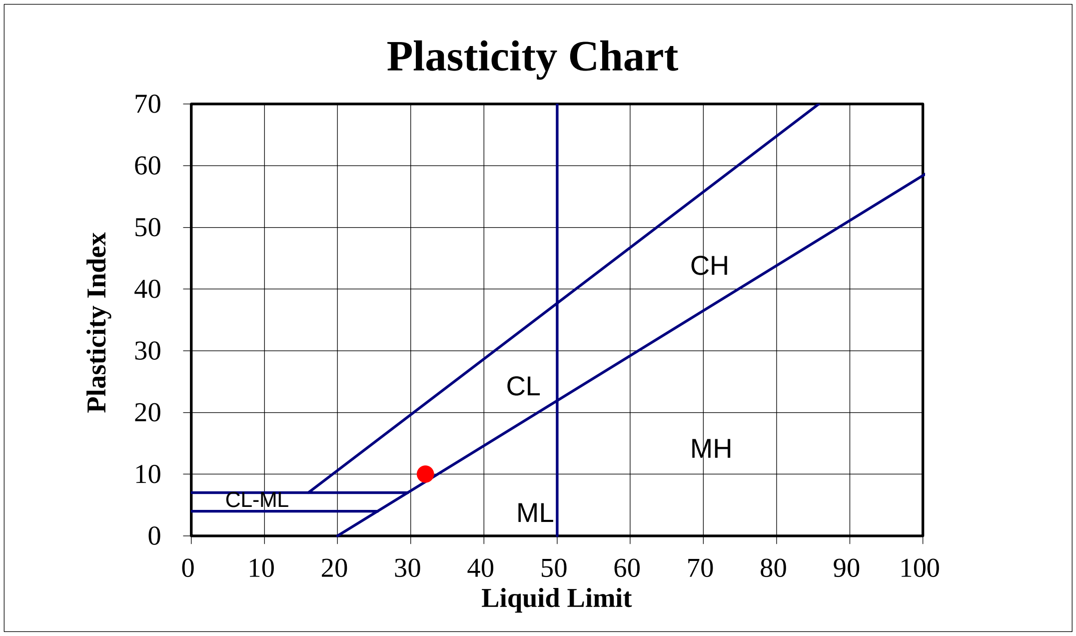
PLASTICITY INDEX

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

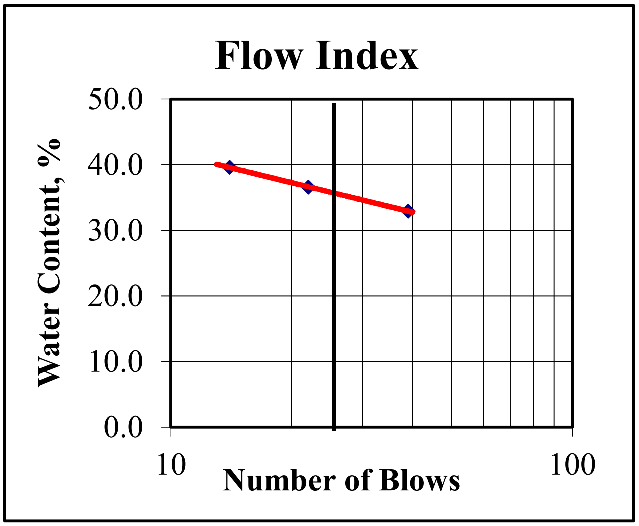
Job Name: 3 High Schools (CIHS)
Sample ID: B 7 @ 47.5'
Soil Description: CL

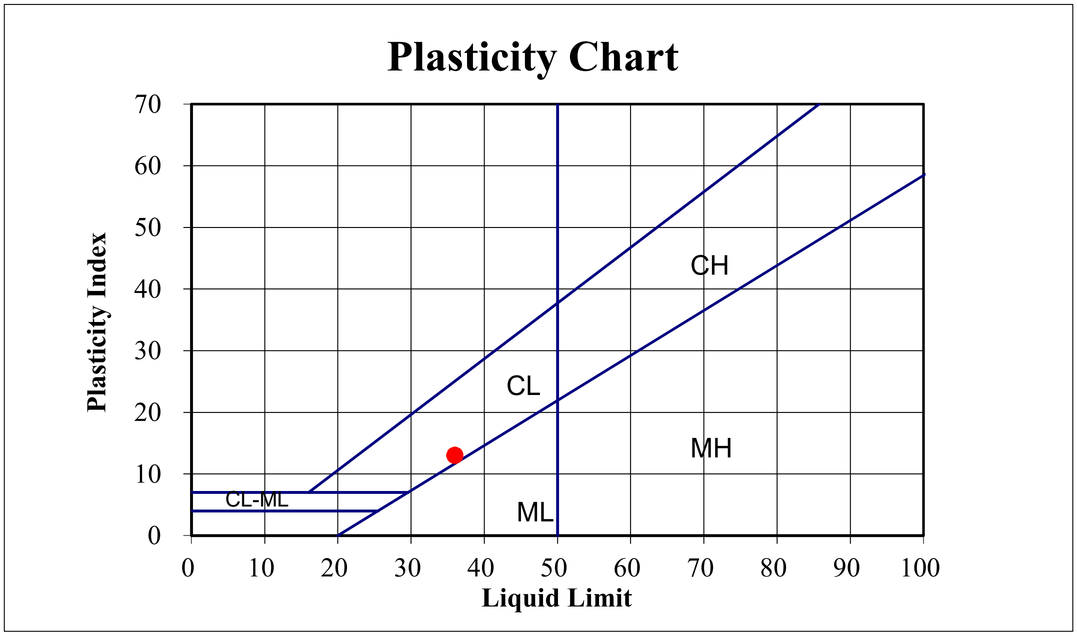
DATA SUMMARY

Number of Blows:	14	22	39
Water Content, %	39.6	36.6	32.9
Plastic Limit:	22.9	22.4	

TEST RESULTS

LIQUID LIMIT	**36**
PLASTIC LIMIT	**23**
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

CERTIFICATE OF ANALYSIS

Client: Earth Systems Pacific
CAS LAB NO: 191290-01
Sample ID: B6@0-5'
Analyst: GP

Date Sampled: 07/15/19
Date Received: 07/17/19
Sample Matrix: Soil

WET CHEMISTRY SUMMARY

COMPOUND	RESULTS	UNITS	DF	PQL	METHOD	ANALYZED
pH (Corrosivity)	8.4	S.U.	1	---	9045	07/24/19
Resistivity*	820	Ohms-cm	1	---	SM 120.1M	07/24/19
Chloride	190	mg/Kg	1	0.3	300.0M	07/24/19
Sulfate	1300	mg/Kg	4	1.2	300.0M	07/24/19

*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)**

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

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: Oxnard HS Athletic Field Improvemen
Calc Date: 11/20/2019
CPT/Boring ID: B-8

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

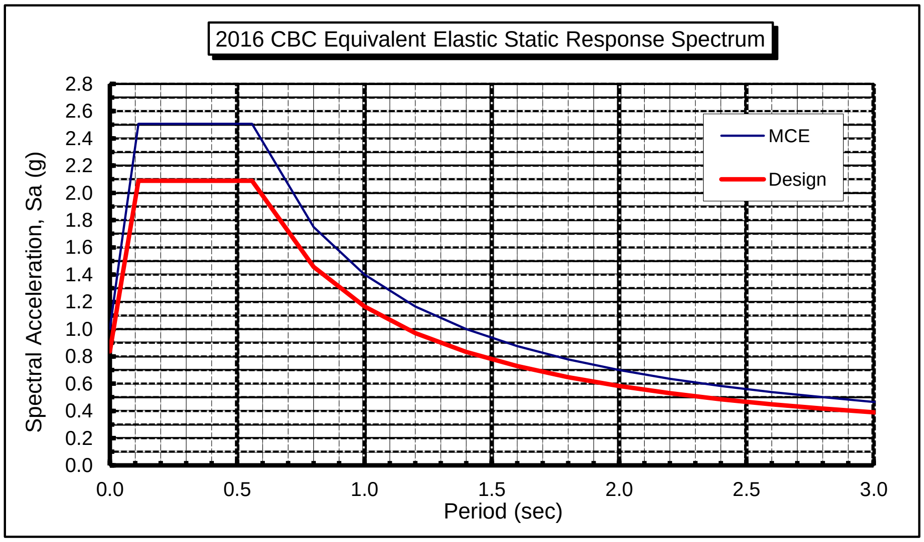
Depth (ft)	SPT N	Sublayer Thick (ft)	Sublayer Thick/N
9.5	23.0	9.5	0.413
10.0	24.0	0.5	0.021
12.0	26.0	2.0	0.077
14.5	37.0	2.5	0.068
17.0	7.0	2.5	0.357
19.5	28.0	2.5	0.089
22.0	32.0	2.5	0.078
24.5	35.0	2.5	0.071
27.0	25.0	2.5	0.100
29.5	16.0	2.5	0.156
32.0	35.0	2.5	0.071
34.5	32.0	2.5	0.078
37.0	44.0	2.5	0.057
39.5	31.0	2.5	0.081
42.0	32.0	2.5	0.078
44.5	30.0	2.5	0.083
47.0	38.0	2.5	0.066
49.5	41.0	2.5	0.061
51.5	22.0	2.0	0.091
100.0	30.0	48.5	1.617

Total Thickness of Soil =	100.00	ft
N-bar Value =	26.9	*
Site Classification =	Class D	

*Equation 20.4-2 of ASCE 7-10

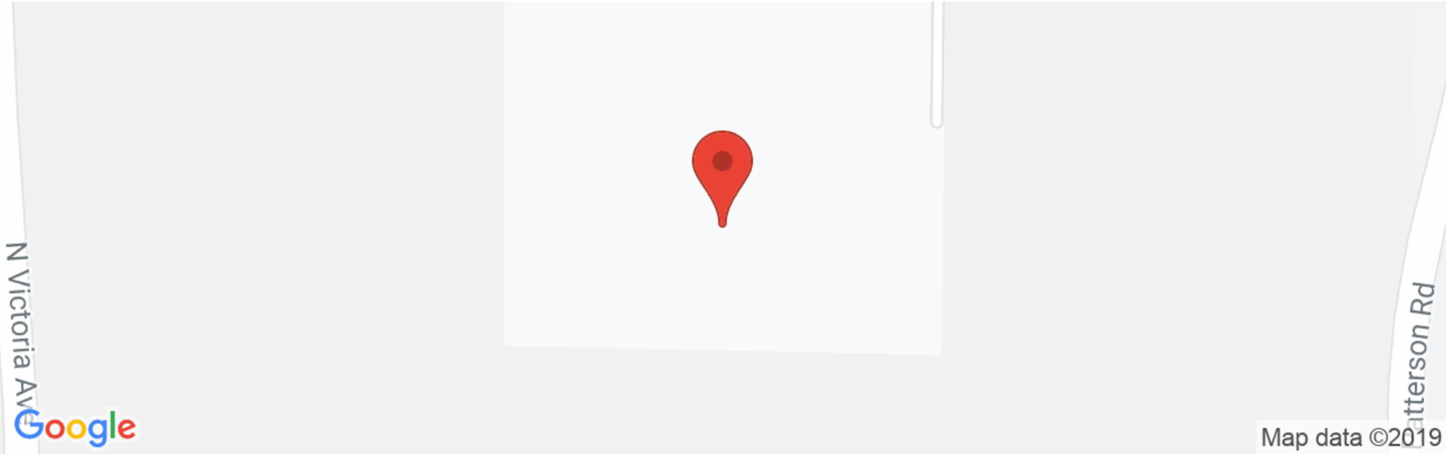
2016 California Building Code (CBC) (ASCE 7-10) Seismic Design Parameters

				CBC Reference	ASCE 7-10 Reference
Seismic Design Category		**E**		Table 1613.5.6	Table 11.6-2
Site Class		**D**		Table 1613.5.2	Table 20.3-1
Latitude:		34.215	N		
Longitude:		-119.214	W		
Maximum Considered Earthquake (MCE) Ground Motion					
Short Period Spectral Reponse	**S_S**	**2.507**	**g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	**S_1**	**0.933**	**g**	Figure 1613.5	Figure 22.4
Site Coefficient	F_a	1.00		Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F_v	1.50		Table 1613.5.3(2)	Table 11-4.2
	S_{MS}	2.507	g	$= F_a*S_S$	
	S_{M1}	1.400	g	$= F_v*S_1$	
Design Earthquake Ground Motion					
Short Period Spectral Reponse	**S_{DS}**	**1.671**	**g**	$= 2/3*S_{MS}$	
1 second Spectral Response	**S_{D1}**	**0.933**	**g**	$= 2/3*S_{M1}$	
	To	0.11	sec	$= 0.2*S_{D1}/S_{DS}$	
	Ts	0.56	sec	$= S_{D1}/S_{DS}$	
Seismic Importance Factor	I	1.25		Table 1604.5	Table 11.5-1
	F_{PGA}	1.00			



Period T (sec)	Design Sa (g)
0.00	0.836
0.05	1.397
0.11	2.089
0.56	2.089
0.80	1.458
1.00	1.166
1.20	0.972
1.40	0.833
1.60	0.729
1.80	0.648
2.00	0.583
2.20	0.530
2.40	0.486
2.60	0.449
2.80	0.417
3.00	0.389

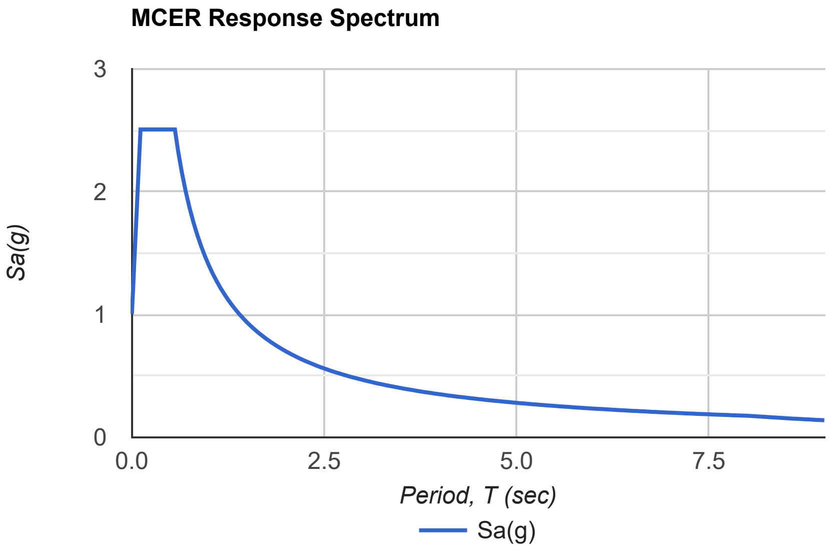
Latitude, Longitude: 34.215471, -119.214277

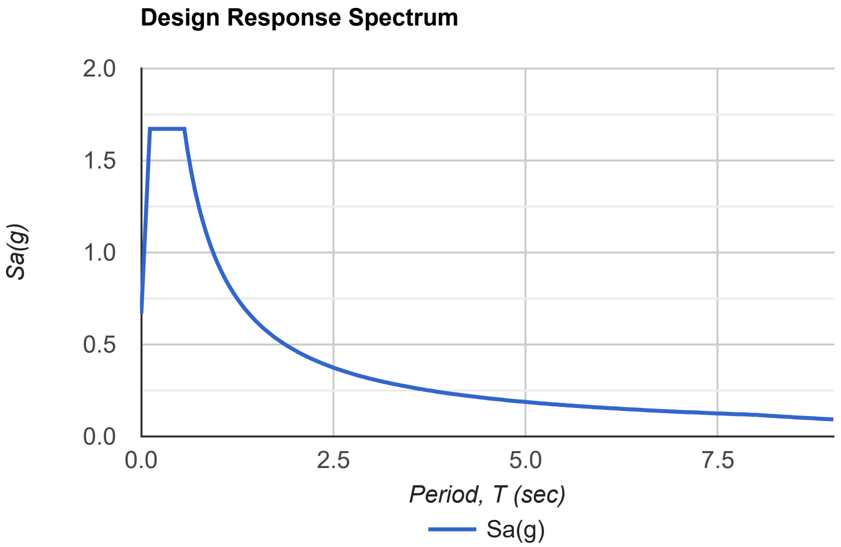


Date	11/20/2019, 2:26:55 PM
Design Code Reference Document	ASCE7-10
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.507	MCE_R ground motion. (for 0.2 second period)
S_1	0.933	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.507	Site-modified spectral acceleration value
S_{M1}	1.4	Site-modified spectral acceleration value
S_{DS}	1.671	Numeric seismic design value at 0.2 second SA
S_{D1}	0.933	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	E	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.976	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.976	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.507	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.728	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.633	Factored deterministic acceleration value. (0.2 second)
S1RT	0.933	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.023	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.008	Factored deterministic acceleration value. (1.0 second)
PGAd	0.998	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.919	Mapped value of the risk coefficient at short periods
C_{R1}	0.912	Mapped value of the risk coefficient at a period of 1 s


MCER Response Spectrum
3
2
1
0
Sa(g)
0.0
2.5
5.0
7.5
Period, T (sec)
Sa(g)


Design Response Spectrum
2.0
1.5
1.0
0.5
0.0
Sa(g)
0.0
2.5
5.0
7.5
Period, T (sec)
Sa(g)

Spectral Response Values

Probabilistic and Deterministic Response Spectra for MCE compared to Code Spectra

for 5% Viscous Damping Ratio

Natural Period T (seconds)	GeoMean Probab. 2% in 50 yr MCE Spectrum (1) 2475-yr	Max Rotated Probab. 2% in 50 yr MCEr (2) 2475-yr	Max 84th Percentile Determ. MCE Spectrum (3)	Determ. Lower Limit MCE Spectrum (4)	Determ. MCE Spectrum (5) max(3,4)	**Site Specific MCE Spectrum** (6) min(2.5)	**2016 CBC MCE Spectrum** (7)	Site Specific Design Spectrum (8) 2/3*(6)*	2016 CBC Design Spectrum (9) 2/3*(7)
0.00	0.877	0.887	1.047	0.600	1.047	**0.887**	**1.003**	0.591	0.669
0.05	1.147	1.159	1.230	0.975	1.230	**1.159**	**1.676**	0.894	1.118
0.10	1.417	1.432	1.608	1.350	1.608	**1.432**	**2.350**	1.253	1.567
0.15	1.617	1.635	1.914	1.500	1.914	**1.635**	**2.507**	1.337	1.671
0.20	1.817	1.837	2.086	1.500	2.086	**1.837**	**2.507**	1.337	1.671
0.30	1.928	1.947	2.254	1.500	2.254	**1.947**	**2.507**	1.337	1.671
0.40	1.890	1.994	2.330	1.500	2.330	**1.994**	**2.507**	1.337	1.671
0.50	1.852	2.037	2.385	1.500	2.385	**2.037**	**2.507**	1.358	1.671
0.75	1.598	1.826	2.284	1.200	2.284	**1.826**	**1.866**	1.217	1.244
1.00	1.344	1.593	1.981	0.900	1.981	**1.593**	**1.400**	1.062	0.933
1.50	1.019	1.208	1.539	0.600	1.539	**1.208**	**0.933**	0.805	0.622
2.00	0.694	0.823	1.221	0.450	1.221	**0.823**	**0.700**	0.549	0.467

Crs: 0.919

Cr1: 0.912

* > 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

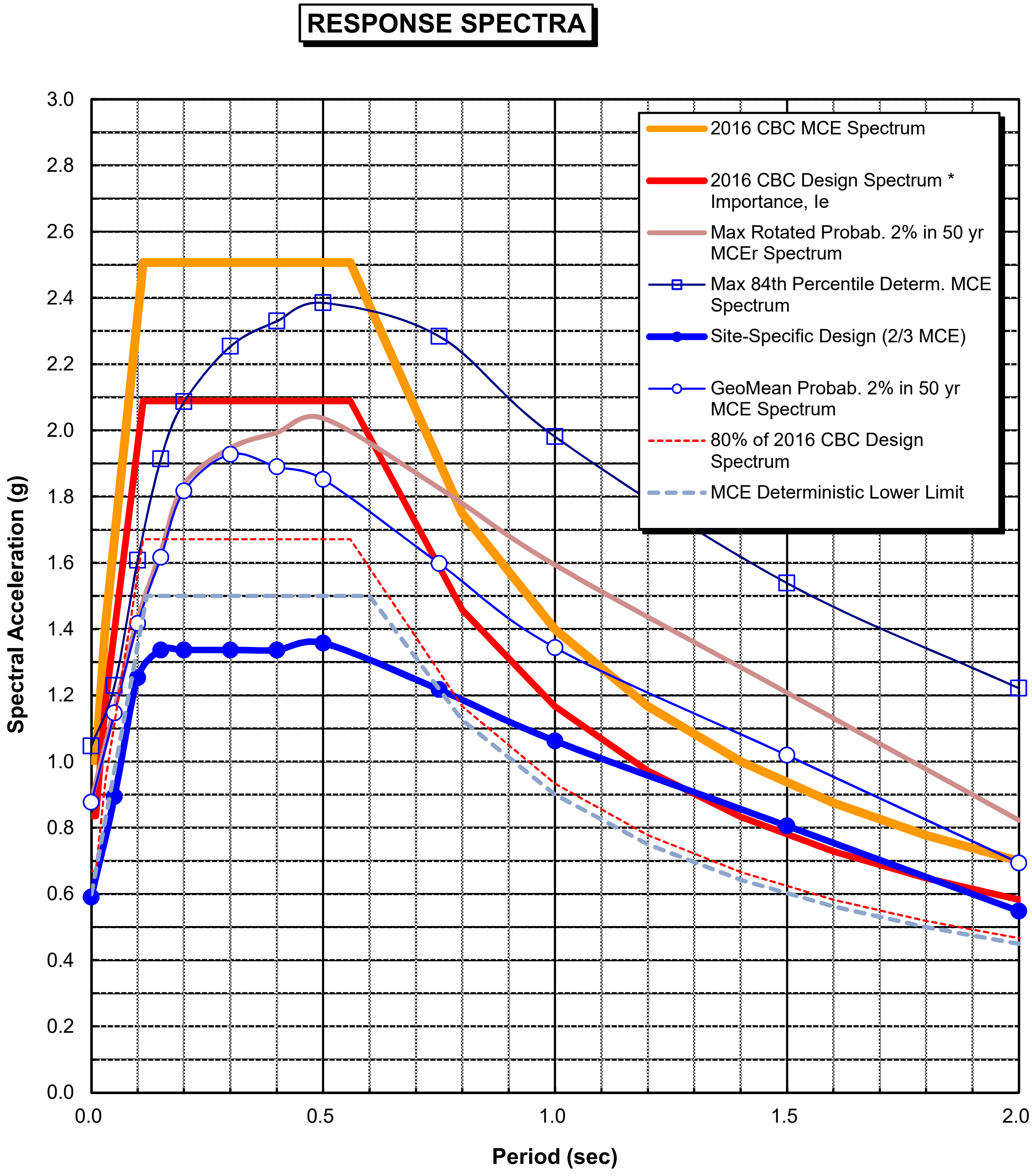
Mapped MCE Acceleration Values			Site Coefficients		**Site-Specific Design Acceleration Values**		
PGA	0.976	g	F_{PGA}	1.00	**PGA_M**	**0.976**	**g**
Ss	2.507	g	F_a	1.00	**S_{DS}**	**1.337**	**g**
S_1	0.933	g	F_v	1.50	**S_{D1}**	**1.097**	**g**

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: D
Latitude: 34.215471
Longitude: -119.214277

Spectral Response Curves

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

Earth Systems

Fault Parameters

Fault Section Name	**Distance** (miles)	(km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
Oak Ridge (Onshore)	0.0	0.0	65	159	90	49	B	**7.4**		4
Oak Ridge (Offshore)	4.2	6.7	32	180	90	38	B	**6.9**		3
Ventura-Pitas Point	5.2	8.4	64	353	60	44	B	**6.9**		1
Simi-Santa Rosa	6.5	10.5	60	346	30	39	B	**6.8**		1
Red Mountain	9.9	16.0	56	2	90	101	B	**7.4**		2
Malibu Coast (Extension), alt 1	10.1	16.3	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	10.1	16.3	74	4	30	35	B'	**6.9**		
Sisar	13.2	21.3	29	168	na	20	B'	**7.0**		
Channel Islands Thrust	13.4	21.5	20	354	90	59	B	**7.3**		1.5
North Channel	14.0	22.5	26	10	90	51	B	**6.7**		1
Channel Islands Western Deep Ramp	14.9	24.1	21	204	90	62	B'	**7.3**		
Pitas Point (Lower)-Montalvo	15.3	24.7	16	359	90	30	B	**7.3**		2.5
Mission Ridge-Arroyo Parida-Santa Ana	15.4	24.8	70	176	90	69	B	**6.8**		0.4
Santa Cruz Island	16.3	26.2	90	188	30	69	B	**7.1**		1
San Cayetano	16.3	26.2	42	3	90	42	B	**7.2**		6
Anacapa-Dume, alt 1	17.1	27.5	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	17.1	27.5	41	352	60	65	B	**7.2**		3
Malibu Coast, alt 1	19.9	32.0	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	19.9	32.0	74	3	30	38	B	**6.9**		0.3
Santa Ynez (East)	21.0	33.8	70	172	0	68	B	**7.2**		2
Pitas Point (Upper)	22.1	35.5	42	15	90	35	B	**6.8**		1
Shelf (Projection)	22.2	35.7	17	21	na	70	B'	**7.8**		
Santa Cruz Catalina Ridge	22.6	36.4	90	38	na	137	B'	**7.3**		
Pine Mtn	25.4	40.9	45	5	na	62	B'	**7.3**		
Oak Ridge (Offshore), west extension	26.5	42.7	67	195	na	28	B'	**6.1**		
Santa Susana, alt 1	27.4	44.1	55	9	90	27	B	**6.8**		5
Santa Susana, alt 2	27.5	44.2	53	10	90	43	B'	**6.8**		
Santa Monica Bay	29.1	46.9	20	44	na	17	B'	**7.0**		
Northridge Hills	29.6	47.7	31	19	90	25	B'	**7.0**		
Del Valle	30.1	48.5	73	195	90	9	B'	**6.3**		
Holser, alt 1	30.5	49.0	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	30.5	49.0	58	182	90	17	B'	**6.7**		
San Pedro Basin	30.5	49.0	88	51	na	69	B'	**7.0**		
Santa Ynez (West)	31.0	49.9	70	182	0	63	B	**6.9**		2
Northridge	31.9	51.3	35	201	90	33	B	**6.8**		1.5
Pitas Point (Lower, West)	32.1	51.6	13	3	90	35	B	**7.2**		2.5
Big Pine (Central)	32.3	51.9	76	167	na	23	B'	**6.3**		
Big Pine (West)	33.3	53.6	50	2	na	18	B'	**6.5**		
Big Pine (East)	35.9	57.8	73	338	na	23	B'	**6.6**		
Compton	36.8	59.2	20	34	90	65	B'	**7.5**		

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

Based on Site Coordinates of 34.215471 Latitude, -119.214277 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 Curves
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: Oxnard HS Bathroom & Gateways
Job No: 303514-002
Date: 11/25/2019
Boring: B-8 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: 0.98 0.94
MSF: 1.03
GWT: 22.0 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: **0.23**

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

Total (in.) Induced Subsidence: **0.7**

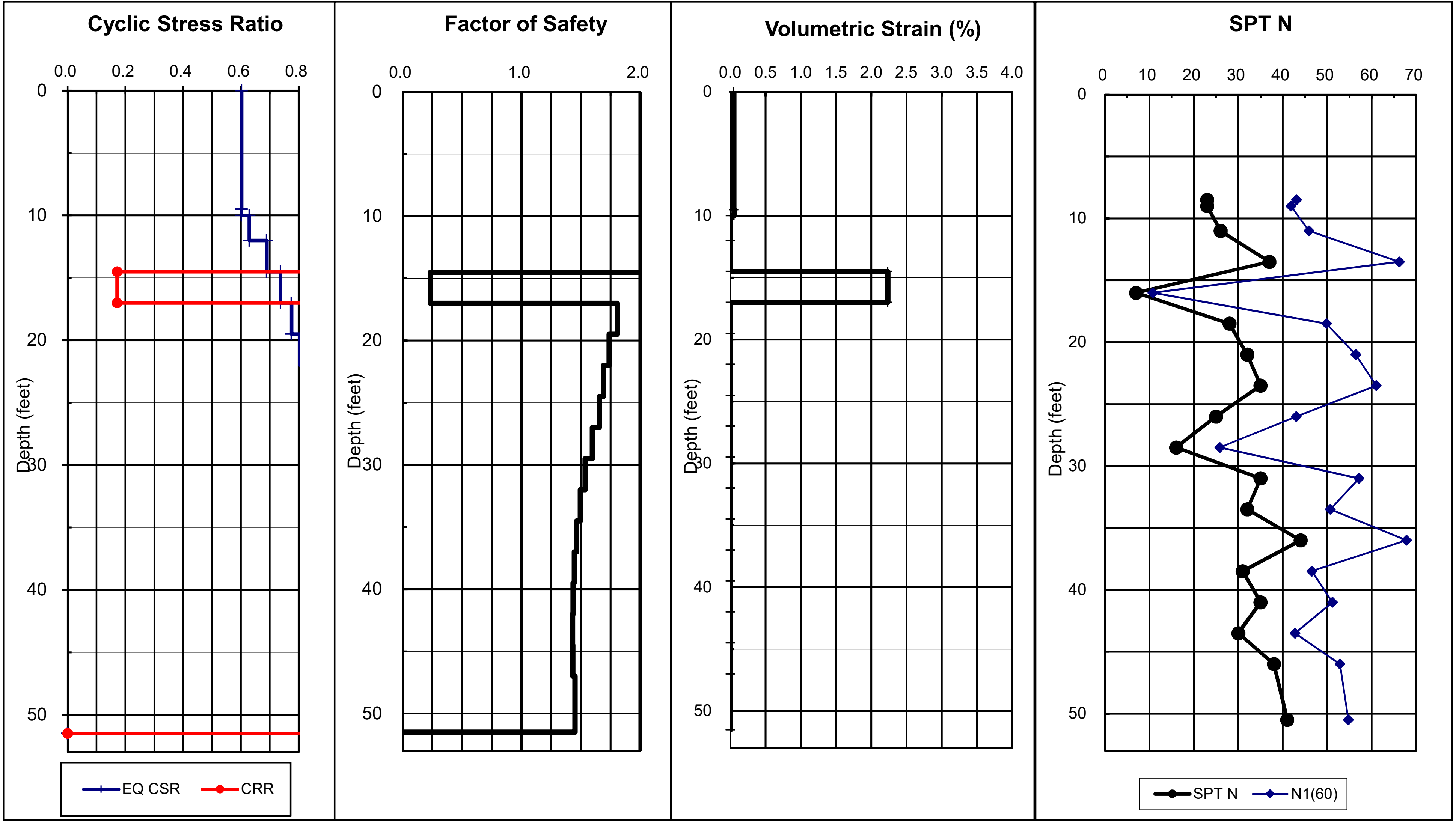
Required SF: 1.30
Minimum Calculated SF: 0.23

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																	
9.5	0	**23**	1	120	10	8.5	11.5	0.510	0.510	0.98	1.44	0.75	1.30	43.1	78	1.8	44.9	1.00	1.400	0.602	**Non-Liq.**	1.8	44.9	0.05	**0.05**
10.0	0	**23**	1	120	10	9.0	12.0	0.540	0.540	0.98	1.40	0.75	1.30	41.9	77	1.8	43.6	1.00	1.400	0.602	**Non-Liq.**	1.8	43.6	0.05	**0.00**
12.0	0	**26**	1	115	10	11.0	14.0	0.658	0.626	0.98	1.30	0.78	1.30	45.9	81	1.9	47.7	1.00	1.400	0.629	**2.23**	1.9	47.7	0.00	**0.00**
14.5	0	**37**	1	120	10	13.5	16.5	0.805	0.696	0.97	1.23	0.84	1.30	66.2	97	2.3	68.5	1.00	1.400	0.689	**2.03**	2.3	68.5	0.00	**0.00**
17.0	0	**7**	1	120	23	16.0	19.0	0.955	0.768	0.97	1.17	0.88	1.12	10.8	39	5.1	15.9	1.00	0.172	0.737	**0.23**	1.9	12.6	2.23	**0.67**
19.5	0	**28**	1	120	10	18.5	21.5	1.105	0.840	0.96	1.12	0.92	1.30	49.9	84	1.9	51.8	1.00	1.400	0.775	**1.81**	1.9	51.8	0.00	**0.00**
22.0	0	**32**	1	120	10	21.0	24.0	1.255	0.912	0.95	1.08	0.94	1.30	56.4	90	2.1	58.5	1.00	1.400	0.805	**1.74**	2.1	58.5	0.00	**0.00**
24.5	0	**35**	1	120	10	23.5	26.5	1.405	0.984	0.95	1.04	0.97	1.30	61.0	93	2.2	63.2	1.00	1.400	0.829	**1.69**	2.2	63.2	0.00	**0.00**
27.0	0	**25**	1	120	10	26.0	29.0	1.555	1.056	0.94	1.00	0.99	1.30	43.0	78	1.8	44.8	1.00	1.400	0.847	**1.65**	1.8	44.8	0.00	**0.00**
29.5	0	**16**	1	115	80	28.5	31.5	1.701	1.124	0.93	0.97	1.00	1.25	25.8	61	10.0	35.8	0.98	1.400	0.877	**1.60**	10.0	35.8	0.00	**0.00**
32.0	0	**35**	1	120	10	31.0	34.0	1.849	1.194	0.92	0.94	1.00	1.30	57.1	90	2.1	59.2	0.95	1.400	0.912	**1.53**	2.1	59.2	0.00	**0.00**
34.5	0	**32**	1	120	5	33.5	36.5	1.999	1.266	0.90	0.91	1.00	1.30	50.7	85	0.0	50.7	0.93	1.400	0.937	**1.49**	0.0	50.7	0.00	**0.00**
37.0	0	**44**	1	120	5	36.0	39.0	2.149	1.338	0.88	0.89	1.00	1.30	67.8	98	0.0	67.8	0.91	1.400	0.956	**1.46**	0.0	67.8	0.00	**0.00**
39.5	0	**31**	1	120	5	38.5	41.5	2.299	1.410	0.86	0.87	1.00	1.30	46.6	82	0.0	46.6	0.89	1.400	0.969	**1.44**	0.0	46.6	0.00	**0.00**
42.0	0	**35**	1	125	5	41.0	44.0	2.453	1.485	0.84	0.84	1.00	1.30	51.2	86	0.0	51.2	0.87	1.400	0.976	**1.43**	0.0	51.2	0.00	**0.00**
44.5	0	**30**	1	125	5	43.5	46.5	2.609	1.564	0.82	0.82	1.00	1.30	42.8	78	0.0	42.8	0.86	1.400	0.979	**1.43**	0.0	42.8	0.00	**0.00**
47.0	0	**38**	1	125	5	46.0	49.0	2.765	1.642	0.79	0.80	1.00	1.30	52.9	87	0.0	52.9	0.84	1.400	0.977	**1.43**	0.0	52.9	0.00	**0.00**
51.5	0	**41**	1	125	5	50.5	53.5	3.046	1.783	0.75	0.77	1.00	1.30	54.7	88	0.0	54.8	0.81	1.400	0.965	**1.45**	0.0	54.8	0.00	**0.00**
0.0																									

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Oxnard HS Bathroom & Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-8 **Earthquake Magnitude: 7.4** **PGA, g: 0.98** **Calc GWT (feet): 10**



Total Thickness of Liquefiable Layers: 2.5 feet

Estimated Total Ground Subsidence: 0.7 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: Oxnard HS Bathroom & Gateways
Job No: 303514-002
Date: 11/25/2019
Boring: B-8 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: 0.98 0.94
MSF: 1.03
GWT: 22.0 feet
Calc GWT: 22.0 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

Total (ft) Liquefied Thickness: 0
Total (in.) Induced Subsidence: 0.4

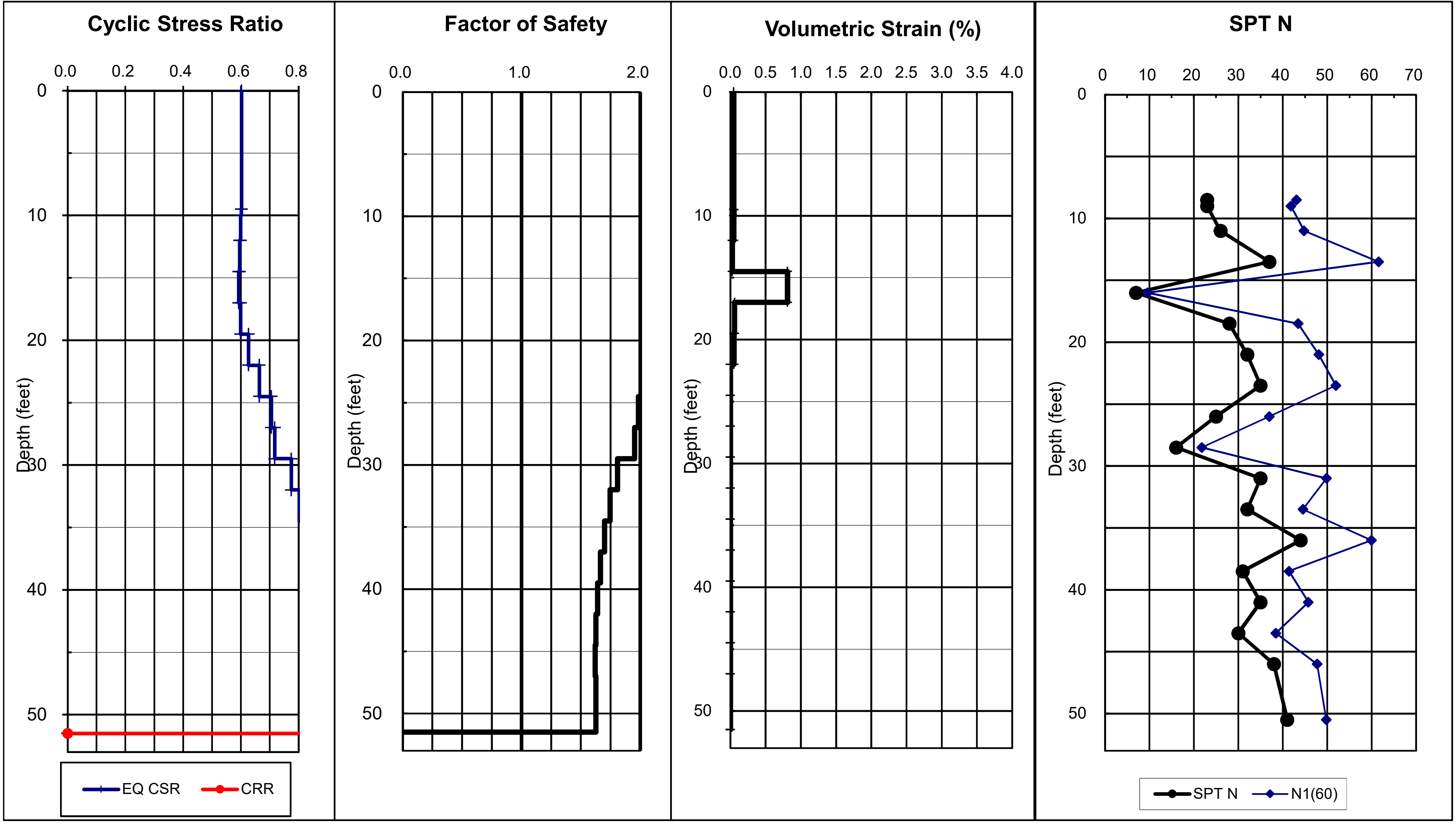
Threshold Acceler., g: 1.58
Required SF: 1.30
Minimum Calculated SF: 1.62

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* $N_{1(60)CS}$	$K\sigma$	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																	
9.5	0	**23**	1	120	10	8.5	11.5	0.510	0.510	0.98	1.44	0.75	1.30	43.1	78	1.8	44.9	1.00	1.400	0.602	**Non-Liq.**	1.8	44.9	0.05	**0.05**
10.0	0	**23**	1	120	10	9.0	12.0	0.540	0.540	0.98	1.40	0.75	1.30	41.9	77	1.8	43.6	1.00	1.400	0.602	**Non-Liq.**	1.8	43.6	0.05	**0.00**
12.0	0	**26**	1	115	10	11.0	14.0	0.658	0.658	0.98	1.27	0.78	1.30	44.8	80	1.8	46.6	1.00	1.400	0.599	**Non-Liq.**	1.8	46.6	0.05	**0.01**
14.5	0	**37**	1	120	10	13.5	16.5	0.805	0.805	0.97	1.15	0.84	1.30	61.6	94	2.2	63.8	1.00	1.400	0.596	**Non-Liq.**	2.2	63.8	0.03	**0.01**
17.0	0	**7**	1	120	23	16.0	19.0	0.955	0.955	0.97	1.05	0.88	1.10	9.5	37	5.0	14.5	1.00	0.157	0.593	**Non-Liq.**	5.0	14.5	0.81	**0.24**
19.5	0	**28**	1	120	10	18.5	21.5	1.105	1.105	0.96	0.98	0.92	1.30	43.5	79	1.8	45.3	0.98	1.400	0.599	**Non-Liq.**	1.8	45.3	0.06	**0.02**
22.0	0	**32**	1	120	10	21.0	24.0	1.255	1.255	0.95	0.92	0.94	1.30	48.1	83	1.9	50.0	0.93	1.400	0.626	**Non-Liq.**	1.9	50.0	0.05	**0.02**
24.5	0	**35**	1	120	10	23.5	26.5	1.405	1.358	0.95	0.88	0.97	1.30	51.9	86	2.0	53.9	0.90	1.400	0.664	**2.11**	2.0	53.9	0.00	**0.00**
27.0	0	**25**	1	120	10	26.0	29.0	1.555	1.430	0.94	0.86	0.99	1.30	37.0	73	1.7	38.6	0.89	1.400	0.706	**1.98**	1.7	38.6	0.00	**0.00**
29.5	0	**16**	1	115	80	28.5	31.5	1.701	1.498	0.93	0.84	1.00	1.22	21.8	56	9.4	31.1	0.90	1.400	0.717	**1.95**	9.4	31.1	0.00	**0.00**
32.0	0	**35**	1	120	10	31.0	34.0	1.849	1.568	0.92	0.82	1.00	1.30	49.8	84	1.9	51.8	0.85	1.400	0.775	**1.81**	1.9	51.8	0.00	**0.00**
34.5	0	**32**	1	120	5	33.5	36.5	1.999	1.640	0.90	0.80	1.00	1.30	44.6	80	0.0	44.6	0.84	1.400	0.802	**1.75**	0.0	44.6	0.00	**0.00**
37.0	0	**44**	1	120	5	36.0	39.0	2.149	1.712	0.88	0.79	1.00	1.30	60.0	93	0.0	60.0	0.82	1.400	0.824	**1.70**	0.0	60.0	0.00	**0.00**
39.5	0	**31**	1	120	5	38.5	41.5	2.299	1.784	0.86	0.77	1.00	1.30	41.4	77	0.0	41.4	0.81	1.400	0.841	**1.66**	0.0	41.4	0.00	**0.00**
42.0	0	**35**	1	125	5	41.0	44.0	2.453	1.860	0.84	0.75	1.00	1.30	45.8	81	0.0	45.8	0.80	1.400	0.853	**1.64**	0.0	45.8	0.00	**0.00**
44.5	0	**30**	1	125	5	43.5	46.5	2.609	1.938	0.82	0.74	1.00	1.30	38.4	74	0.0	38.4	0.78	1.400	0.861	**1.63**	0.0	38.4	0.00	**0.00**
47.0	0	**38**	1	125	5	46.0	49.0	2.765	2.016	0.79	0.72	1.00	1.30	47.7	83	0.0	47.7	0.77	1.400	0.864	**1.62**	0.0	47.7	0.00	**0.00**
51.5	0	**41**	1	125	5	50.5	53.5	3.046	2.157	0.75	0.70	1.00	1.30	49.8	84	0.0	49.8	0.75	1.400	0.861	**1.63**	0.0	49.8	0.00	**0.00**
0.0																									

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Oxnard HS Bathroom & Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-8 **Earthquake Magnitude: 7.4** **PGA, g: 0.98** **Calc GWT (feet): 22**



Total Thickness of Liquefiable Layers: 0.0 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: Oxnard HS Bathroom & Gateways
Job No: 303514-002
Date: 11/25/2019
Boring: B-9 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: 0.98 0.94
MSF: 1.03
GWT: 20.0 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

Total (ft) Liquefied Thickness 5

Total (in.) Induced Subsidence 1.1

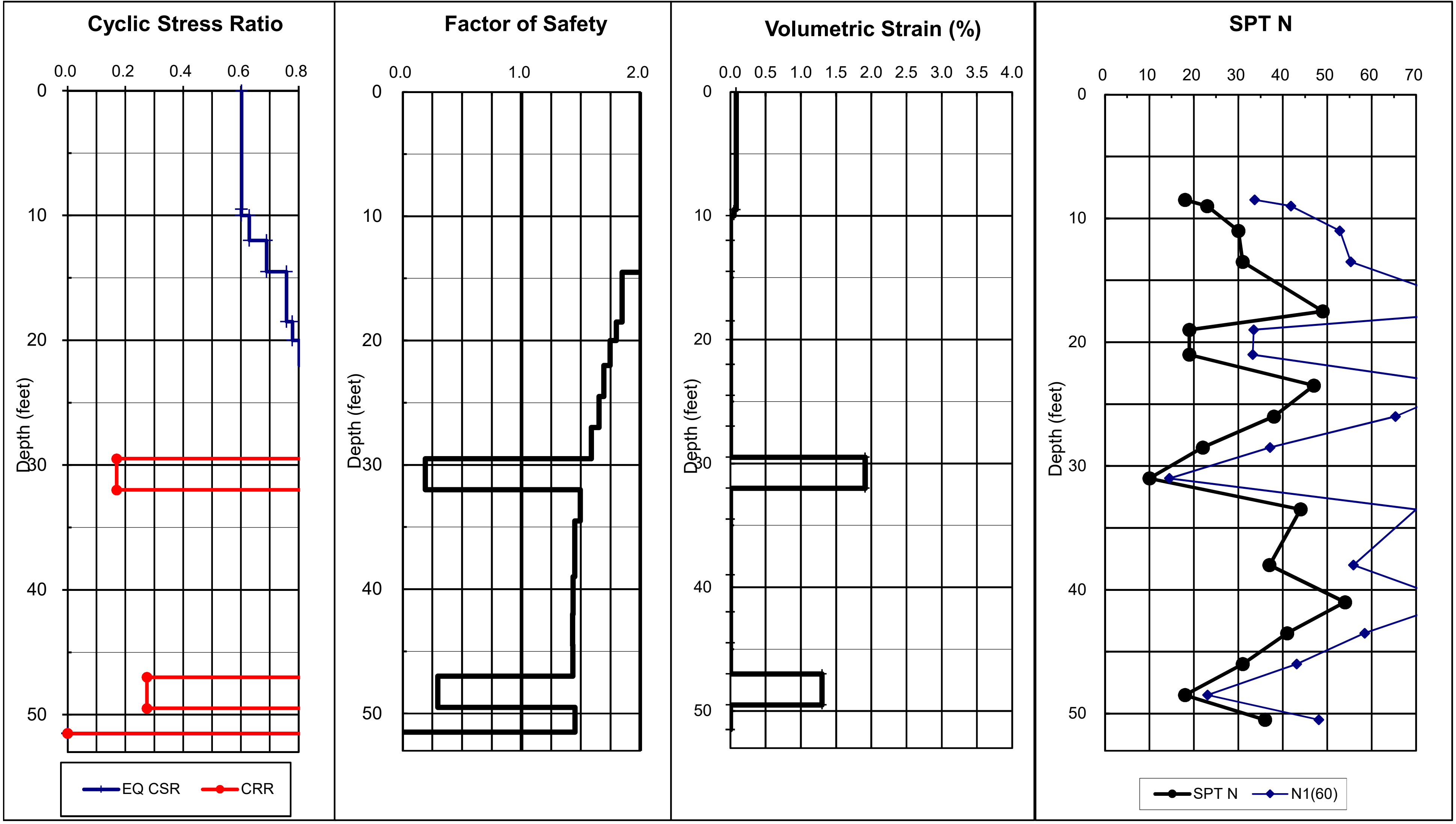
Threshold Acceler., g: 0.19
Required SF: 1.30
Minimum Calculated SF: 0.19

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																	
9.5	0	**18**	1	120	10	8.5	11.5	0.510	0.510	0.98	1.44	0.75	1.30	33.7	69	1.6	35.3	1.00	1.400	0.602	**Non-Liq.**	1.6	35.3	0.08	**0.09**
10.0	0	**23**	1	120	10	9.0	12.0	0.540	0.540	0.98	1.40	0.75	1.30	41.9	77	1.8	43.6	1.00	1.400	0.602	**Non-Liq.**	1.8	43.6	0.05	**0.00**
12.0	0	**30**	1	120	10	11.0	14.0	0.660	0.629	0.98	1.30	0.78	1.30	52.8	87	2.0	54.9	1.00	1.400	0.629	**2.23**	2.0	54.9	0.00	**0.00**
14.5	0	**31**	1	120	5	13.5	16.5	0.810	0.701	0.97	1.23	0.84	1.30	55.3	89	0.0	55.3	1.00	1.400	0.689	**2.03**	0.0	55.3	0.00	**0.00**
18.5	0	**49**	1	125	5	17.5	20.5	1.058	0.824	0.96	1.13	0.90	1.30	86.8	100	0.0	86.8	1.00	1.400	0.758	**1.85**	0.0	86.8	0.00	**0.00**
20.0	0	**19**	1	120	5	19.0	22.0	1.150	0.869	0.96	1.10	0.92	1.30	33.5	69	0.0	33.5	1.00	1.400	0.778	**1.80**	0.0	33.5	0.00	**0.00**
22.0	0	**19**	1	120	5	21.0	24.0	1.270	0.927	0.95	1.07	0.94	1.30	33.2	69	0.0	33.2	1.00	1.400	0.802	**1.75**	0.0	33.2	0.00	**0.00**
24.5	0	**47**	1	115	5	23.5	26.5	1.416	0.995	0.95	1.03	0.97	1.30	81.5	100	0.0	81.5	1.00	1.400	0.826	**1.69**	0.0	81.5	0.00	**0.00**
27.0	0	**38**	1	110	5	26.0	29.0	1.556	1.057	0.94	1.00	0.99	1.30	65.4	97	0.0	65.4	1.00	1.400	0.847	**1.65**	0.0	65.4	0.00	**0.00**
29.5	0	**22**	1	110	85	28.5	31.5	1.694	1.117	0.93	0.97	1.00	1.30	37.1	73	10.0	47.1	0.98	1.400	0.882	**1.59**	10.0	47.1	0.00	**0.00**
32.0	0	**10**	1	120	10	31.0	34.0	1.839	1.184	0.92	0.95	1.00	1.15	14.5	46	1.2	15.7	0.98	0.170	0.892	**0.19**	1.0	15.5	1.91	**0.57**
34.5	0	**44**	1	120	5	33.5	36.5	1.989	1.256	0.90	0.92	1.00	1.30	70.0	100	0.0	70.0	0.93	1.400	0.937	**1.49**	0.0	70.0	0.00	**0.00**
39.0	0	**37**	1	125	5	38.0	41.0	2.268	1.394	0.87	0.87	1.00	1.30	55.9	89	0.0	55.9	0.90	1.400	0.967	**1.45**	0.0	55.9	0.00	**0.00**
42.0	0	**54**	1	125	5	41.0	44.0	2.455	1.488	0.84	0.84	1.00	1.30	78.9	100	0.0	78.9	0.87	1.400	0.977	**1.43**	0.0	78.9	0.00	**0.00**
44.5	0	**41**	1	125	5	43.5	46.5	2.611	1.566	0.82	0.82	1.00	1.30	58.4	91	0.0	58.4	0.85	1.400	0.979	**1.43**	0.0	58.4	0.00	**0.00**
47.0	0	**31**	1	120	5	46.0	49.0	2.764	1.641	0.79	0.80	1.00	1.30	43.2	79	0.0	43.2	0.84	1.400	0.977	**1.43**	0.0	43.2	0.00	**0.00**
49.5	0	**18**	1	125	10	48.5	51.5	2.918	1.716	0.77	0.79	1.00	1.23	23.1	57	1.4	24.5	0.86	0.274	0.926	**0.30**	1.0	24.1	1.30	**0.39**
51.5	0	**36**	1	125	12	50.5	53.5	3.043	1.779	0.75	0.77	1.00	1.30	48.1	83	3.1	51.2	0.81	1.400	0.965	**1.45**	3.1	51.2	0.00	**0.00**
0.0																									

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Oxnard HS Bathroom & Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-9 **Earthquake Magnitude: 7.4** **PGA, g: 0.98** **Calc GWT (feet): 10**



Total Thickness of Liquefiable Layers: 5.0 feet **Estimated Total Ground Subsidence: 1.1 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: Oxnard HS Bathroom & Gateways
Job No: 303514-002
Date: 11/25/2019
Boring: B-9 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: 0.98 0.94
MSF: 1.03
GWT: 20.0 feet
Calc GWT: 20.0 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

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

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

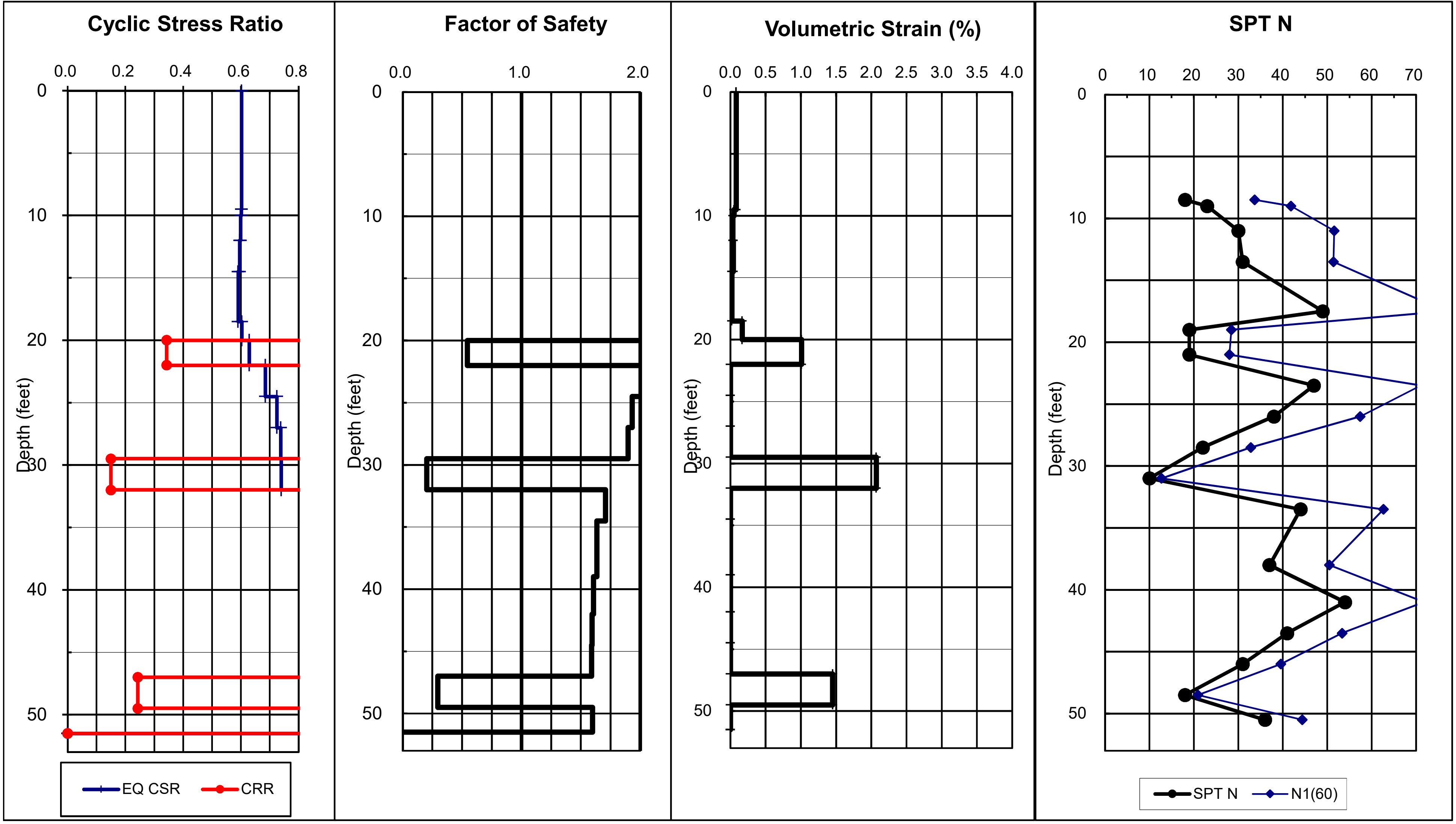
Required SF: 1.30
Threshold Acceler., g: **0.20**
Minimum Calculated SF: **0.20**

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																	
9.5	0	**18**	1	120	10	8.5	11.5	0.510	0.510	0.98	1.44	0.75	1.30	33.7	69	1.6	35.3	1.00	1.400	0.602	**Non-Liq.**	1.6	35.3	0.08	**0.09**
10.0	0	**23**	1	120	10	9.0	12.0	0.540	0.540	0.98	1.40	0.75	1.30	41.9	77	1.8	43.6	1.00	1.400	0.602	**Non-Liq.**	1.8	43.6	0.05	**0.00**
12.0	0	**30**	1	120	10	11.0	14.0	0.660	0.660	0.98	1.27	0.78	1.30	51.6	86	2.0	53.6	1.00	1.400	0.599	**Non-Liq.**	2.0	53.6	0.04	**0.01**
14.5	0	**31**	1	120	5	13.5	16.5	0.810	0.810	0.97	1.14	0.84	1.30	51.4	86	0.0	51.4	1.00	1.400	0.596	**Non-Liq.**	0.0	51.4	0.04	**0.01**
18.5	0	**49**	1	125	5	17.5	20.5	1.058	1.058	0.96	1.00	0.90	1.30	76.6	100	0.0	76.6	1.00	1.400	0.590	**Non-Liq.**	0.0	76.6	0.02	**0.01**
20.0	0	**19**	1	120	5	19.0	22.0	1.150	1.150	0.96	0.96	0.92	1.27	28.4	64	0.0	28.4	0.98	0.355	0.603	**Non-Liq.**	0.0	28.4	0.17	**0.03**
22.0	0	**19**	1	120	5	21.0	24.0	1.270	1.239	0.95	0.92	0.94	1.27	28.0	63	0.0	28.0	0.95	0.343	0.629	**0.55**	0.0	28.0	1.01	**0.24**
24.5	0	**47**	1	115	5	23.5	26.5	1.416	1.307	0.95	0.90	0.97	1.30	71.1	100	0.0	71.1	0.92	1.400	0.685	**2.04**	0.0	71.1	0.00	**0.00**
27.0	0	**38**	1	110	5	26.0	29.0	1.556	1.369	0.94	0.88	0.99	1.30	57.4	91	0.0	57.4	0.90	1.400	0.725	**1.93**	0.0	57.4	0.00	**0.00**
29.5	0	**22**	1	110	85	28.5	31.5	1.694	1.429	0.93	0.86	1.00	1.30	32.8	68	10.0	42.8	0.91	1.400	0.738	**1.90**	10.0	42.8	0.00	**0.00**
32.0	0	**10**	1	120	10	31.0	34.0	1.839	1.496	0.92	0.84	1.00	1.13	12.7	43	1.1	13.9	0.93	0.150	0.740	**0.20**	1.0	13.7	2.07	**0.62**
34.5	0	**44**	1	120	5	33.5	36.5	1.989	1.568	0.90	0.82	1.00	1.30	62.7	95	0.0	62.7	0.85	1.400	0.820	**1.71**	0.0	62.7	0.00	**0.00**
39.0	0	**37**	1	125	5	38.0	41.0	2.268	1.706	0.87	0.79	1.00	1.30	50.5	85	0.0	50.5	0.83	1.400	0.856	**1.63**	0.0	50.5	0.00	**0.00**
42.0	0	**54**	1	125	5	41.0	44.0	2.455	1.800	0.84	0.77	1.00	1.30	71.8	100	0.0	71.8	0.81	1.400	0.871	**1.61**	0.0	71.8	0.00	**0.00**
44.5	0	**41**	1	125	5	43.5	46.5	2.611	1.878	0.82	0.75	1.00	1.30	53.3	87	0.0	53.3	0.79	1.400	0.878	**1.59**	0.0	53.3	0.00	**0.00**
47.0	0	**31**	1	120	5	46.0	49.0	2.764	1.953	0.79	0.74	1.00	1.30	39.6	75	0.0	39.6	0.78	1.400	0.880	**1.59**	0.0	39.6	0.00	**0.00**
49.5	0	**18**	1	125	10	48.5	51.5	2.918	2.028	0.77	0.72	1.00	1.21	20.9	55	1.3	22.3	0.82	0.243	0.823	**0.30**	1.0	21.9	1.45	**0.44**
51.5	0	**36**	1	125	12	50.5	53.5	3.043	2.091	0.75	0.71	1.00	1.30	44.4	80	3.0	47.3	0.76	1.400	0.876	**1.60**	3.0	47.3	0.00	**0.00**
0.0																									

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Oxnard HS Bathroom & Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-9 **Earthquake Magnitude: 7.4** **PGA, g: 0.98** **Calc GWT (feet): 20**



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: Oxnard HS Bathroom & Gateways
Job No: 303514-002
Date: 11/25/2019
Boring: B-10 Data Set: 3

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: 0.98 0.94
MSF: 1.03
GWT: 25.0 feet
Calc GWT: 25.0 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: **0.39**

Total (ft) Liquefied Thickness	**Total (in.) Induced Subsidence**
2.5	**0.5**

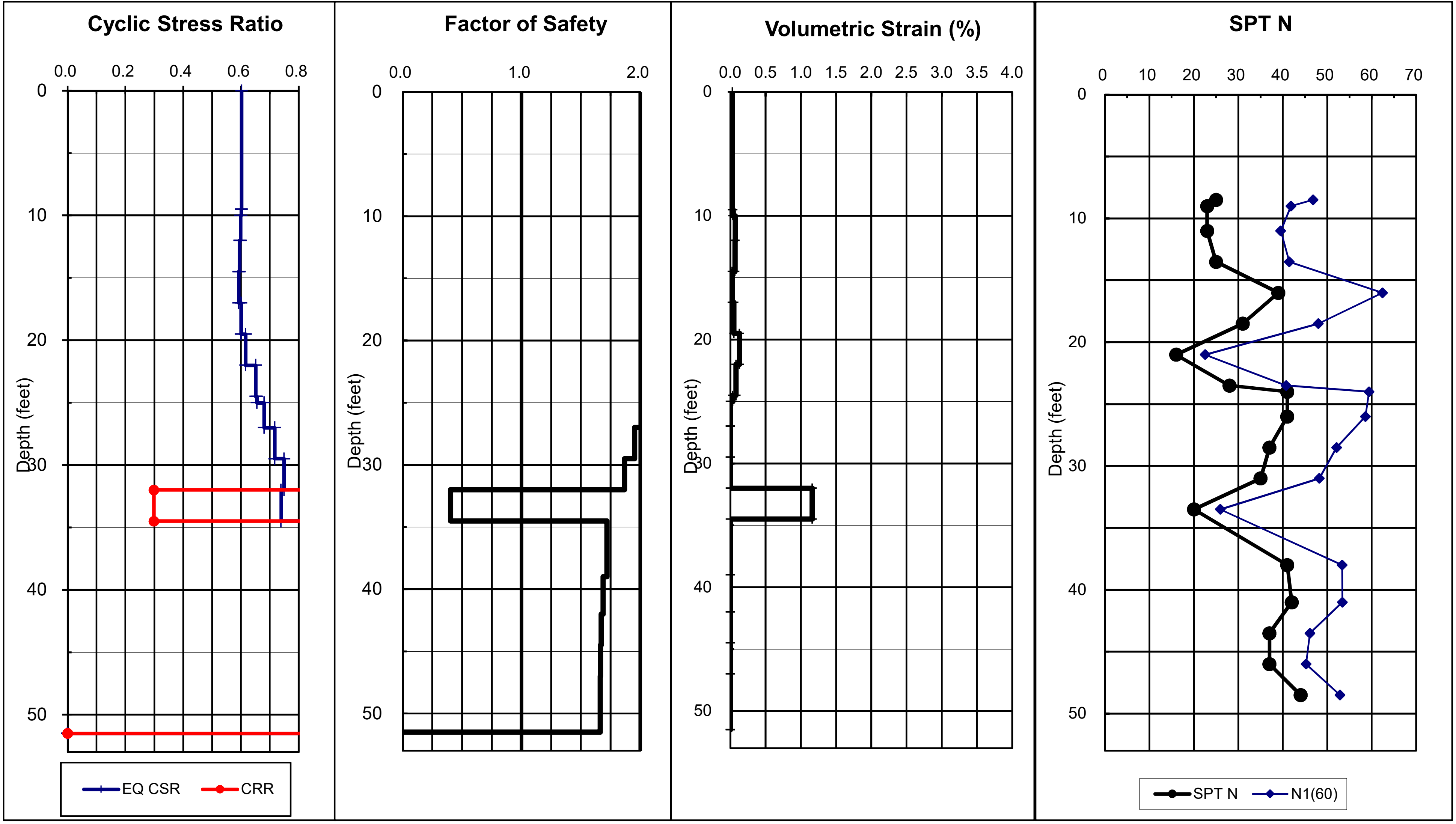
Required SF: 1.30
Minimum Calculated SF: **0.40**

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																	
9.5	0	**25**	1	120	25	8.5	11.5	0.510	0.510	0.98	1.44	0.75	1.30	46.8	82	9.7	56.5	1.00	1.400	0.602	**Non-Liq.**	9.7	56.5	0.03	**0.03**
10.0	0	**23**	1	120	25	9.0	12.0	0.540	0.540	0.98	1.40	0.75	1.30	41.9	77	9.1	51.0	1.00	1.400	0.602	**Non-Liq.**	9.1	51.0	0.04	**0.00**
12.0	0	**23**	1	120	5	11.0	14.0	0.660	0.660	0.98	1.27	0.78	1.30	39.5	75	0.0	39.5	1.00	1.400	0.599	**Non-Liq.**	0.0	39.5	0.07	**0.02**
14.5	0	**25**	1	120	5	13.5	16.5	0.810	0.810	0.97	1.14	0.84	1.30	41.5	77	0.0	41.5	1.00	1.400	0.596	**Non-Liq.**	0.0	41.5	0.07	**0.02**
17.0	0	**39**	1	120	10	16.0	19.0	0.960	0.960	0.97	1.05	0.88	1.30	62.5	94	2.2	64.7	1.00	1.400	0.593	**Non-Liq.**	2.2	64.7	0.03	**0.01**
19.5	0	**31**	1	120	10	18.5	21.5	1.110	1.110	0.96	0.98	0.92	1.30	48.0	83	1.9	49.9	0.98	1.400	0.600	**Non-Liq.**	1.9	49.9	0.05	**0.01**
22.0	0	**16**	1	120	43	21.0	24.0	1.260	1.260	0.95	0.92	0.94	1.22	22.6	57	9.5	32.1	0.95	1.400	0.617	**Non-Liq.**	9.5	32.1	0.13	**0.04**
24.5	0	**28**	1	120	5	23.5	26.5	1.410	1.410	0.95	0.87	0.97	1.30	40.8	76	0.0	40.8	0.89	1.400	0.651	**Non-Liq.**	0.0	40.8	0.08	**0.02**
25.0	0	**41**	1	125	10	24.0	27.0	1.439	1.439	0.95	0.86	0.97	1.30	59.4	92	2.2	61.5	0.88	1.400	0.655	**Non-Liq.**	2.2	61.5	0.04	**0.00**
27.0	0	**41**	1	120	10	26.0	29.0	1.561	1.530	0.94	0.83	0.99	1.30	58.6	92	2.1	60.8	0.86	1.400	0.680	**2.06**	2.1	60.8	0.00	**0.00**
29.5	0	**37**	1	120	10	28.5	31.5	1.711	1.602	0.93	0.81	1.00	1.30	52.1	86	2.0	54.1	0.85	1.400	0.717	**1.95**	2.0	54.1	0.00	**0.00**
32.0	0	**35**	1	120	10	31.0	34.0	1.861	1.674	0.92	0.79	1.00	1.30	48.2	83	1.9	50.1	0.83	1.400	0.750	**1.87**	1.9	50.1	0.00	**0.00**
34.5	0	**20**	1	120	5	33.5	36.5	2.011	1.746	0.90	0.78	1.00	1.25	25.9	61	0.0	25.9	0.86	0.299	0.739	**0.40**	0.0	25.9	1.16	**0.35**
39.0	0	**41**	1	120	10	38.0	41.0	2.281	1.876	0.87	0.75	1.00	1.30	53.4	87	2.0	55.4	0.80	1.400	0.814	**1.72**	2.0	55.4	0.00	**0.00**
42.0	0	**42**	1	125	25	41.0	44.0	2.466	1.967	0.84	0.73	1.00	1.30	53.4	87	10.0	63.4	0.78	1.400	0.830	**1.69**	10.0	63.4	0.00	**0.00**
44.5	0	**37**	1	125	10	43.5	46.5	2.623	2.045	0.82	0.72	1.00	1.30	46.1	81	1.9	48.0	0.77	1.400	0.838	**1.67**	1.9	48.0	0.00	**0.00**
47.0	0	**37**	1	125	10	46.0	49.0	2.779	2.124	0.79	0.71	1.00	1.30	45.3	80	1.8	47.1	0.76	1.400	0.841	**1.66**	1.8	47.1	0.00	**0.00**
51.5	0	**44**	1	125	10	48.5	51.5	2.935	2.202	0.77	0.69	1.00	1.30	52.9	87	2.0	54.9	0.75	1.400	0.842	**1.66**	2.0	54.9	0.00	**0.00**
51.5																									

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Oxnard HS Bathroom & Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-10 **Earthquake Magnitude: 7.4** **PGA, g: 0.98** **Calc GWT (feet): 25**



Total Thickness of Liquefiable Layers: 2.5 feet

Estimated Total Ground Subsidence: 0.5 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: Oxnard HS Bathroom & Gateways
Job No: 303514-002
Date: 11/25/2019
Boring: B-10 Data Set: 3

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: 0.98 0.94
MSF: 1.03
GWT: 25.0 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: **1.40**

Total (ft) Liquefied Thickness	**Total (in.) Induced Subsidence**
0	**0.0**

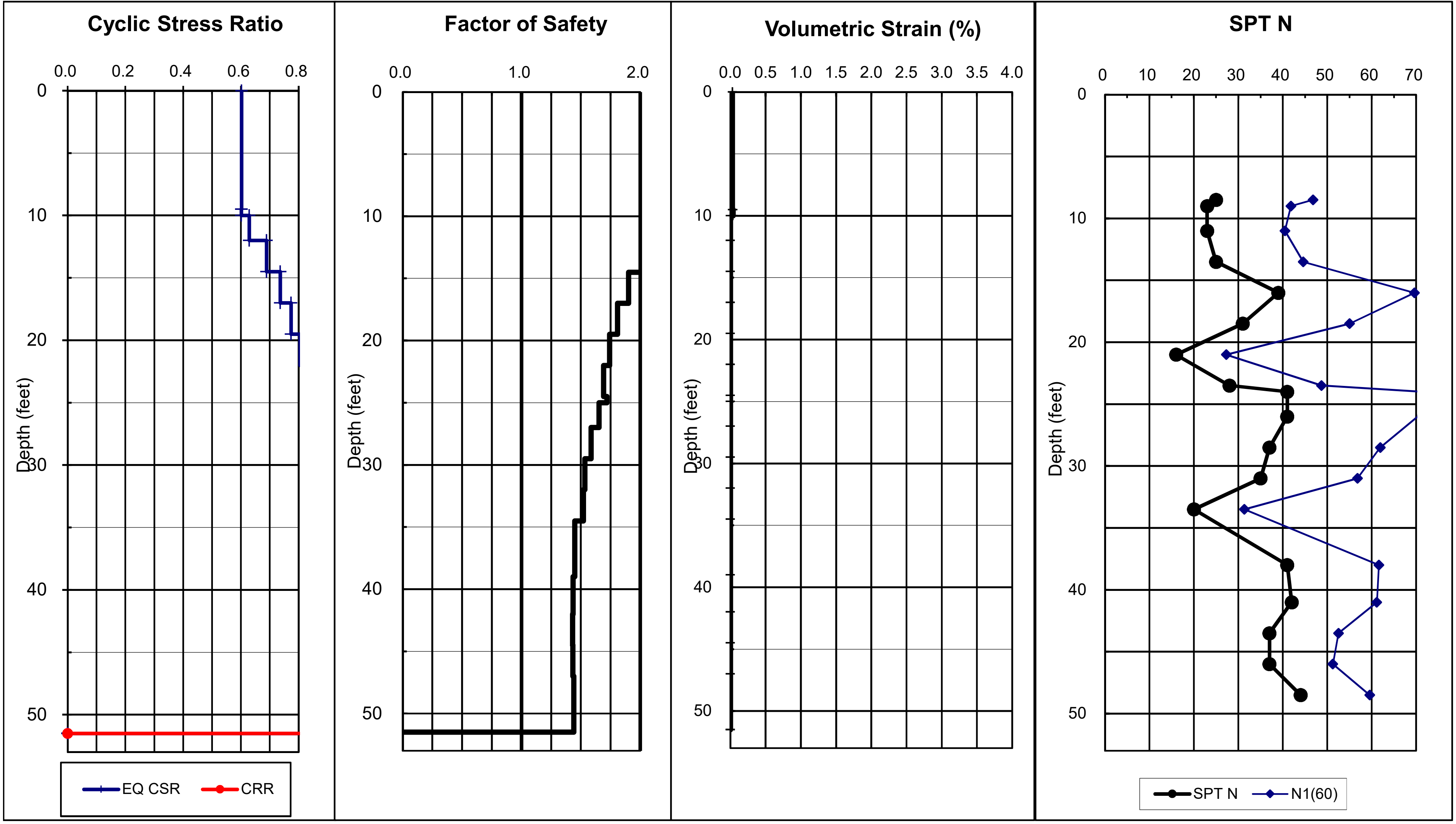
Required SF: 1.30
Minimum Calculated SF: **1.43**

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																	
9.5	0	**25**	1	120	25	8.5	11.5	0.510	0.510	0.98	1.44	0.75	1.30	46.8	82	9.7	56.5	1.00	1.400	0.602	**Non-Liq.**	9.7	56.5	0.03	**0.03**
10.0	0	**23**	1	120	25	9.0	12.0	0.540	0.540	0.98	1.40	0.75	1.30	41.9	77	9.1	51.0	1.00	1.400	0.602	**Non-Liq.**	9.1	51.0	0.04	**0.00**
12.0	0	**23**	1	120	5	11.0	14.0	0.660	0.629	0.98	1.30	0.78	1.30	40.5	76	0.0	40.5	1.00	1.400	0.629	**2.23**	0.0	40.5	0.00	**0.00**
14.5	0	**25**	1	120	5	13.5	16.5	0.810	0.701	0.97	1.23	0.84	1.30	44.6	80	0.0	44.6	1.00	1.400	0.689	**2.03**	0.0	44.6	0.00	**0.00**
17.0	0	**39**	1	120	10	16.0	19.0	0.960	0.773	0.97	1.17	0.88	1.30	69.6	100	2.4	72.0	1.00	1.400	0.736	**1.90**	2.4	72.0	0.00	**0.00**
19.5	0	**31**	1	120	10	18.5	21.5	1.110	0.845	0.96	1.12	0.92	1.30	55.0	89	2.1	57.1	1.00	1.400	0.774	**1.81**	2.1	57.1	0.00	**0.00**
22.0	0	**16**	1	120	43	21.0	24.0	1.260	0.917	0.95	1.07	0.94	1.26	27.3	62	10.0	37.3	1.00	1.400	0.804	**1.74**	10.0	37.3	0.00	**0.00**
24.5	0	**28**	1	120	5	23.5	26.5	1.410	0.989	0.95	1.03	0.97	1.30	48.7	83	0.0	48.7	1.00	1.400	0.828	**1.69**	0.0	48.7	0.00	**0.00**
25.0	0	**41**	1	125	10	24.0	27.0	1.439	1.002	0.95	1.03	0.97	1.30	71.2	100	2.4	73.6	1.02	1.400	0.814	**1.72**	2.4	73.6	0.00	**0.00**
27.0	0	**41**	1	120	10	26.0	29.0	1.561	1.062	0.94	1.00	0.99	1.30	70.4	100	2.4	72.7	1.00	1.400	0.847	**1.65**	2.4	72.7	0.00	**0.00**
29.5	0	**37**	1	120	10	28.5	31.5	1.711	1.134	0.93	0.97	1.00	1.30	61.9	94	2.2	64.2	0.97	1.400	0.883	**1.59**	2.2	64.2	0.00	**0.00**
32.0	0	**35**	1	120	10	31.0	34.0	1.861	1.206	0.92	0.94	1.00	1.30	56.8	90	2.1	58.9	0.95	1.400	0.913	**1.53**	2.1	58.9	0.00	**0.00**
34.5	0	**20**	1	120	5	33.5	36.5	2.011	1.278	0.90	0.91	1.00	1.29	31.3	67	0.0	31.3	0.94	1.400	0.920	**1.52**	0.0	31.3	0.00	**0.00**
39.0	0	**41**	1	120	10	38.0	41.0	2.281	1.408	0.87	0.87	1.00	1.30	61.6	94	2.2	63.8	0.89	1.400	0.967	**1.45**	2.2	63.8	0.00	**0.00**
42.0	0	**42**	1	125	25	41.0	44.0	2.466	1.499	0.84	0.84	1.00	1.30	61.2	93	10.0	71.2	0.87	1.400	0.977	**1.43**	10.0	71.2	0.00	**0.00**
44.5	0	**37**	1	125	10	43.5	46.5	2.623	1.577	0.82	0.82	1.00	1.30	52.5	87	2.0	54.5	0.85	1.400	0.979	**1.43**	2.0	54.5	0.00	**0.00**
47.0	0	**37**	1	125	10	46.0	49.0	2.779	1.656	0.79	0.80	1.00	1.30	51.3	86	2.0	53.2	0.84	1.400	0.977	**1.43**	2.0	53.2	0.00	**0.00**
51.5	0	**44**	1	125	10	48.5	51.5	2.935	1.734	0.77	0.78	1.00	1.30	59.6	92	2.2	61.7	0.82	1.400	0.971	**1.44**	2.2	61.7	0.00	**0.00**
51.5																									

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Oxnard HS Bathroom & Gateways **Project No: 303514-002** 1996/1998 NCEER Method

Boring: B-10 **Earthquake Magnitude: 7.4** **PGA, g: 0.98** **Calc GWT (feet): 10**



Total Thickness of Liquefiable Layers: 0.0 feet

Estimated Total Ground Subsidence: 0.0 inches

Job Number:	303514-002
Job Name:	Oxnard HS Bathroom Bldg
Boring Number:	B-8
Date:	November 22, 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 $(D50_{15})$

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

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 = 0	km
S = 0.36	%
T_{15} = 0.77	m
F_{15} = 23	%
$D50_{15}$ = 1	mm

Horizontal Ground Displacement in meters (D_H) = 0.97
Horizontal Ground Displacement in feet (D_H) = 3.2

Job Number: 303514-002
Job Name: Oxnard HS Northwest Gateway
Boring Number: B-9
Date: November 22, 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 ($D50_{15}$)

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

Requirements and Limitations Used to Develop this Model

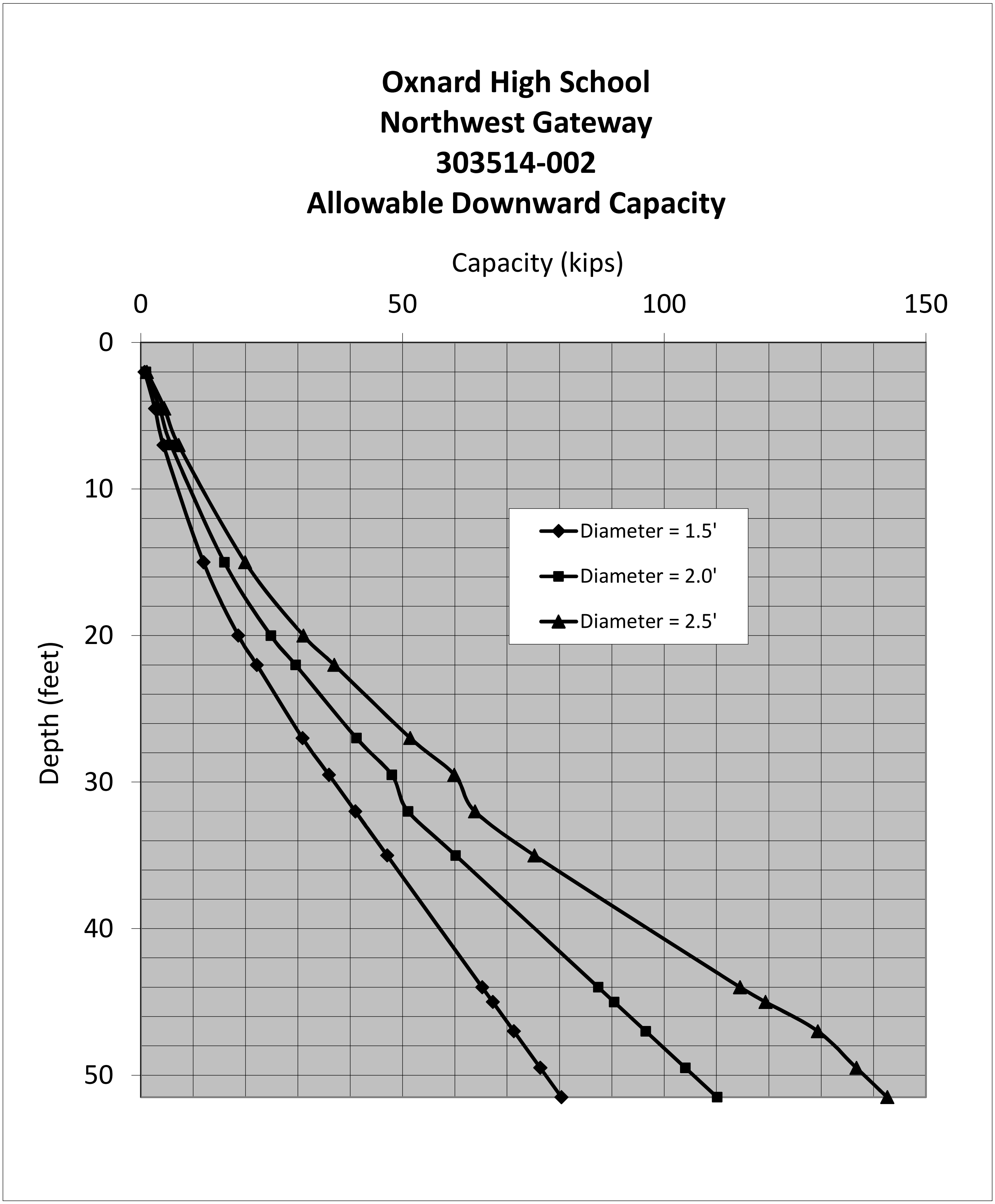
Soils must be Liquefiable
Saturated Cohesionless Sediments with SPT $(N1)_{60}$ less than 15
Earthquake Magnitude (M) must be between 6 and 8
Percent Slope (S) must be between 0.1% and 6%
Cumulative Thickness (T15) must be between 1 and 15 meters
Depth to top of Liquefied layer must be between 1 and 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 = 0	km
S = 0.36	%
T_{15} = 0.77	m
F_{15} = 5	%
$D50_{15}$ = 2	mm

Horizontal Ground Displacement in meters (D_H) = 1.19
Horizontal Ground Displacement in feet (D_H) = 3.9

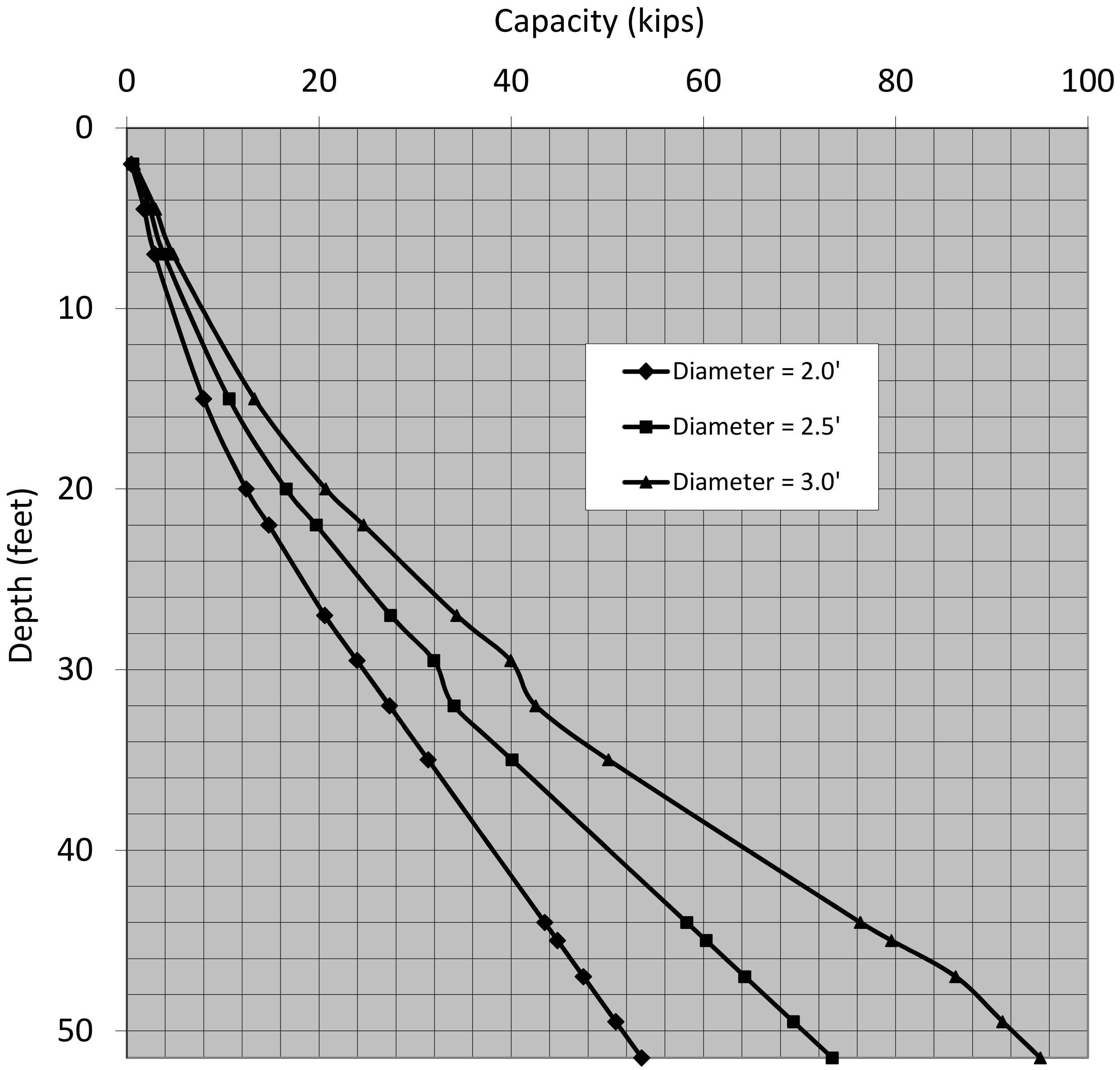
APPENDIX E

Pile Capacity Graphs

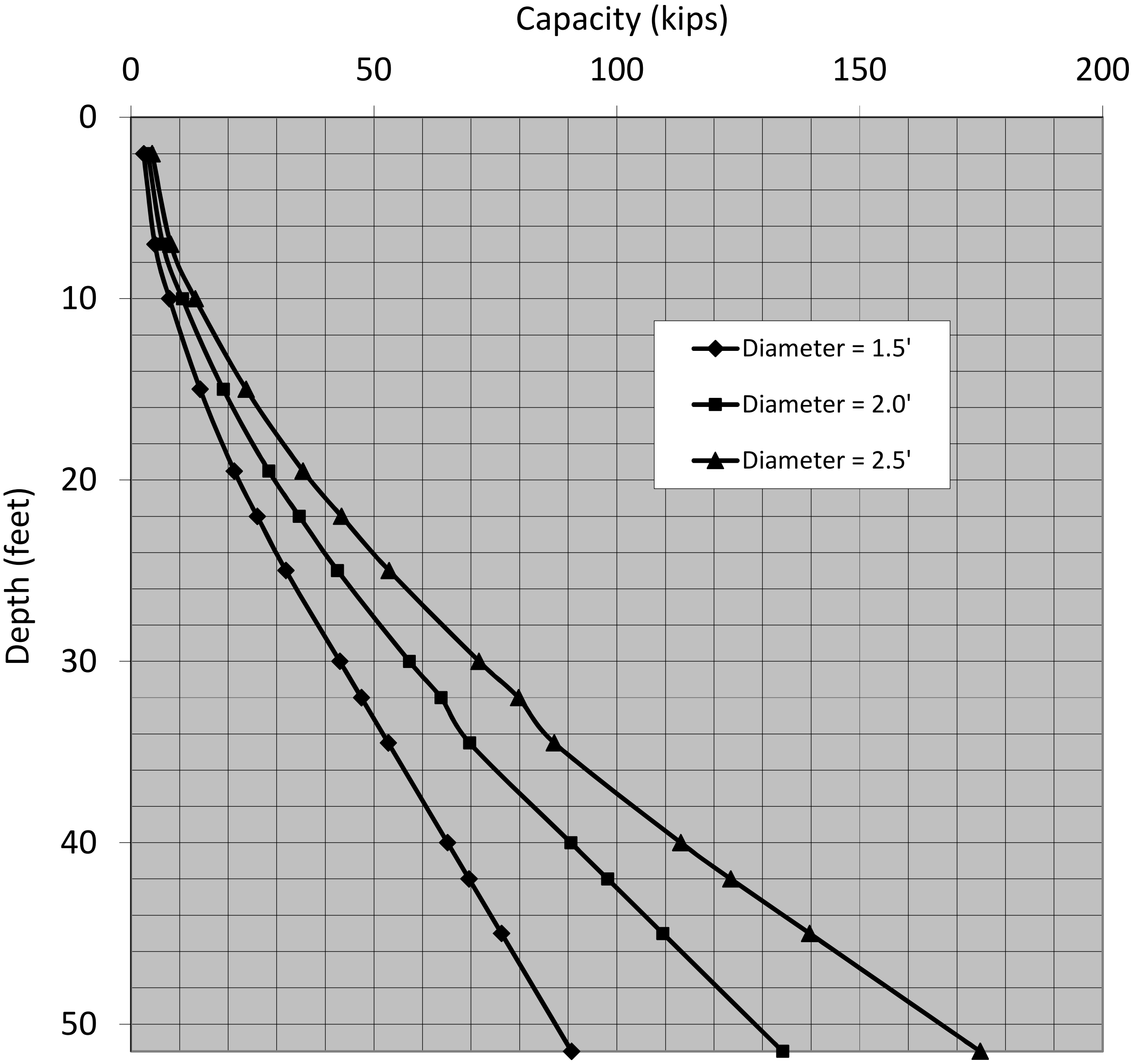


Oxnard High School
Northwest Gateway
303514-002
Allowable Downward Capacity

Oxnard High School
Northwest Gateway
303514-002
Allowable Upward Capacity



Oxnard High School
Southeast Gateway
303514-002
Allowable Downward Capacity



Oxnard High School
Southeast Gateway
303514-002
Allowable Upward Capacity