GEOTECHNICAL ENGINEERING REPORT
FOR
PROPOSED IMPROVEMENTS TO ATHLETIC FIELDS AT
HUENEME HIGH SCHOOL,
500 WEST BARD ROAD
OXNARD, CALIFORNIA

PROJECT NO.: 303277-001
AUGUST 27, 2019

PREPARED FOR
OXNARD UNION HIGH SCHOOL DISTRICT

BY
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August 27, 2019

Project No.: 303277-001
Report No.: 19-8-4 (Revised)

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

Project: Improvements to Athletic Field Surfaces
Hueneme High School
500 West Bard Road
Oxnard, California

As authorized, we have performed a geotechnical study for proposed improvements to the athletic field surfaces at Hueneme 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-19-05-012 dated May 20, 2019, and authorized by Purchase Order A19-03284 on June 19, 2019.

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

Respectfully submitted,

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

INTRODUCTION

This report presents results of a Geotechnical Engineering study performed for proposed improvements to the athletic fields at Hueneme High School in the City of Oxnard (see Vicinity Map in Appendix A). Proposed improvements will include installation of synthetic turf surfaces and subdrainage systems to replace natural turf surfaces on the athletic fields, a new bathroom building, and a new ticket booth with attached entry gate at the north entrance to the football field. A new entry plaza will be installed between the new entry gate and the northeast end of the football field. New concrete walkways will replace existing sidewalks around the football field. New basketball courts will be installed between reconfigured layouts of softball and baseball fields. An existing asphalt parking lot will be converted to a new asphalt-paved basketball court. An existing asphalt service road running along the northern edge of the playfields will be replaced with new asphalt paving. Water and sewer lines will connect the new restroom near the baseball field to existing utilities.

Current plans indicate that a minimum of 12 inches of soils are to be compacted below the drain system that will underlie the fields. Where flat panel drains will be located within the drainage grid, a trench about 18 inches wide will be cut about 3 to 4 inches deeper than adjacent subgrade soils. Subgrade soil elevation will be 6 inches below the finished base grade elevation (before synthetic turf is placed). The panel drains are 12 inches wide and approximately 2 inches high, and are to be wrapped with a filter sock and backfilled with a minimum of 0.5 inches of clean washed sand.

The panel drains are to flow at a gradient of 0.6% toward the perimeter of the field where they will be collected within a trench with a depth and design that will depend on the soil characteristics and groundwater conditions at the site. The trench will run parallel to and under the sidelines toward a storm drain outlet.

The synthetic turf will be supported by 6 inches of permeable base (rock) material on the subgrade soils and panel drain sand cover.

The all-weather track surface will be underlain by asphalt pavement above compacted aggregate base materials and compacted subgrade soils. Surface flow will be directed inward to a drain running parallel to the track edge. Storm water will flow from the track edge drain at a 2%

gradient toward and into the larger trench that gathers the athletic field flat panel drain waters. The water gathered within the trench will be piped to a storm drain system.

The one-story bathroom building will be a reinforced CMU block structure that will be approximately 498 feet in plan view. It is proposed to support it with a conventional foundation system and a slab-on-grade floor.

The ticket booth is expected to comprise approximately 70 square feet, and to have attached 10-foot tall entry gates supported by steel tube columns on pier footings. The one-story ticket booth will be constructed with reinforced CMU block, and will utilize a conventional foundation system with a slab-on-grade floor. There will be 8-foot high freestanding reinforced CMU walls adjacent to the ticket booths at the entry gates.

It is understood that there may be 6-foot high CMU and/or concrete site walls, some of which may be retaining, but none that retain more than 6 feet. There may also be fences that range in height from 8 to 18 feet high in various areas of the site.

PURPOSE AND SCOPE OF WORK

The purpose of the geotechnical study that led to this report was to analyze the soil conditions of the site with respect to the proposed improvements. These conditions include surface and subsurface soil types, expansion potential, settlement potential, bearing capacity, and the presence or absence of subsurface water. The scope of work included:

1. Performing a reconnaissance of the site.
2. Drilling, sampling, and logging 3 hollow-stem-auger borings to study soil and groundwater conditions.
3. Laboratory testing soil samples obtained from the subsurface exploration to determine their physical and engineering properties.
4. Consulting with owner representatives and design professionals.
5. Analyzing the geotechnical data obtained.
6. 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 infiltration potential.

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 Simi-Santa Rosa Fault is located approximately 6.2 miles northeast of the subject site. 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 medium dense silty sands, fine to medium sands, and clayey sands.

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.

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 (34.1578˚ North Latitude and -119.1820° West Longitude). The calculator adjusts for Soil Site Class D, and for Occupancy (Risk) Category I (for non-habitable structures). (A listing of the calculated 2016 CBC and ASCE 7-10 Seismic Parameters is presented below and in Appendix C.)

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.

Summary of Seismic Parameters – 2016 CBC

Site Class (Table 20.3-1 of ASCE 7-10 with 2016 update)	D
Occupancy (Risk) Category	I
Seismic Design Category	E
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	2.254 g
Spectral Response Acceleration at 1 sec. – S_1	0.799 g
Site Coefficient – F_a	1.00
Site Coefficient – F_v	1.50
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.254 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.199 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.503 g
One Second Spectral Response – S_{D1}	0.799 g
Site Modified Peak Ground Acceleration - PGA_M	0.844 g
Values appropriate for a 2% probability of exceedance in 50 years	

SOIL CONDITIONS

Evaluation of the subsurface indicates that soils are generally alluvial sands, silty sands, and clayey sands. Near-surface soils encountered below the fields are characterized by low blow counts and in-place densities, and moderate compressibilities. Testing indicates that anticipated bearing soils lie in the "very low" expansion range because the expansion index equals 20. [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 depths ranging from 7 to 10 feet below existing site grades. Mapping of historically high groundwater levels by the California Geological Survey (CGS, 2002a) indicates that groundwater has been 10 below the ground surface near the subject site.

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

Samples of near-surface soils were tested for pH, resistivity, soluble sulfates, and soluble chlorides. The test results provided in Appendix B should be distributed to the design team for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils. It should be noted that sulfate contents (530 mg/Kg) are in the "S0" ("negligible") exposure class of Table 19.3.1.1 of ACI 318-14; therefore, it appears that special concrete designs will not be necessary for the measured sulfate contents.

Based on criteria established by the County of Los Angeles (2013), measurements of resistivity of near-surface soils (2,100 ohms-cm) indicate that they are "moderately corrosive" to ferrous metal (i.e. cast iron, etc.) pipes.

GEOTECHNICAL CONCLUSIONS

The site is suitable for the proposed athletic field improvements from a Geotechnical Engineering standpoint provided that the recommendations contained in this report are successfully implemented into the project.

GEOTECHNICAL RECOMMENDATIONS FOR FIELD AND TRACK SURFACE IMPROVEMENTS

All proposed grading should conform to the 2016 California Building Code.

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

The existing ground surface should be initially prepared for grading by removing all grass and vegetation, 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.

Proposed areas of athletic field improvements or areas to receive fill should be overexcavated to a depth of one foot. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted. This will result in at least 12 inches of compacted fill below the flat panel drains, and 18 inches of compacted fill below the areas between the drains. Compaction should be verified to be a minimum of 90% of the maximum dry density obtained by the ASTM D 1557 test method.

Proposed areas of track surface replacements (and underlying asphaltic concrete pavement), exterior slabs-on-grade, or sidewalks should be overexcavated to a depth of one foot. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted. Compaction should be verified to be a minimum of 95% of the maximum dry density obtained by the ASTM D 1557 test method.

Once subgrade elevations are achieved and flat panel drains are installed, a permeable filter fabric, such as Mirafi 140N, should be placed over the subgrade soils and panel drains. Permeable base should be placed over the filter fabric and compacted to a minimum of 95% of the maximum dry density obtained by the ASTM D 1557 test method.

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.

Shrinkage of soils affected by compaction is estimated to be about 10 percent based on an anticipated average compaction of 92 percent. Shrinkage from removal of any existing subsurface structures is not included in these figures.

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

Compaction tests shall be made to determine the relative compaction of the fills, subgrade soils, and utility trench backfills in accordance with the following minimum guidelines: one test for each two-foot vertical lift, one test for each 1,000 cubic yards of material placed, one test per two-foot vertical lift per 250 lineal feet of utility trench backfill, and four tests at finished subgrade elevation of each field.

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

GRADING RECOMMENDATIONS FOR BUILDINGS, ENTRY GATES, AND PAVEMENTS

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

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.

Overexcavation and recompaction of soils in the building area will be necessary to decrease the potential for differential settlement and provide more uniform bearing conditions. Soils should be overexcavated to a depth of 4.5 feet below finished subgrade elevation throughout the entire building area, and to a distance of 5 feet beyond the perimeter of each building. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted to at least 90 percent of the maximum dry density. The intent of these recommendations is to have a minimum of 5 feet of compacted soil below the building.

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

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

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

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

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

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

GEOTECHNICAL DESIGN PARAMETERS FOR BUILDINGS AND SITE WALLS

Conventional Spread Foundations

Conventional continuous footings and/or isolated pad footings may be used to support structures. For one-story buildings, perimeter and interior footings should have minimum depths of 12 inches.

Footings should bear into firm recompacted soils. as recommended elsewhere in this report. 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 may be designed based on an allowable bearing value of 1,400 psf. This value has a factor of safety of 3.

Isolated pad footings may be designed based on an allowable bearing value of 1,700 psf. This value has a factor of safety of 3.

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

Bearing values may be increased by one-third when transient loads such as wind and/or seismicity are included.

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

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

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

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

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

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

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

Drilled Pier Foundations

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 D.

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 390 psf per foot of penetration in the compacted fill (upper 5 feet) and an EFW of 300 pcf in the firm native soils above the groundwater table may be used for lateral load design. An EFW of 165 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 2,900 psf. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended).

For piers spaced at least three diameters apart, an effective width of 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 feet below the final ground elevation. If 30-inch diameter piers are used, the “point of fixity” was estimated to be located at least 8.5 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.

Slabs-on-Grade

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

It is recommended that perimeter slabs (walks, patios, etc.) be designed relatively independent of footing stems (i.e. free floating) so foundation adjustment will be less likely to cause cracking. Current plans call for 4-inch thick concrete reinforced with No. 3 bars on 18-inch centers. These specifications are considered appropriate for the soil conditions. (Note that structural paving sections for areas to be exposed to vehicular traffic are presented elsewhere in this report.)

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

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

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

Retaining Walls

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

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

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

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

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 settlements of about one inch are anticipated for foundations and floor slabs designed as recommended. (It should be noted that these values do not include potential seismic- or liquefaction-induced settlements.)

Differential settlement between adjacent load bearing members should be expected to range up to about one-half the total settlement.

DESIGN VALUES FOR FENCEPOST PIER FOOTINGS IN NON-COMPACTED AREAS

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.

PRELIMINARY ASPHALT PAVING SECTIONS FOR VEHICULAR PAVEMENTS

Assuming a Traffic Index of 5 for areas to be used for parking stalls and other light vehicular duty uses, and using the measured R-Value of 64, paving sections should have a minimum gravel equivalent of 0.58 feet. This can be achieved by using 3 inches of asphaltic concrete on 4 inches of Processed Miscellaneous Base (PMB) compacted to a minimum of 95 percent of the maximum dry density on subgrade soils compacted to a minimum of 95 percent of the maximum dry density.

For fire lanes or drive lanes in new pavement areas with an assumed Traffic Index of 6.5, paving sections should have a minimum gravel equivalent of -/85 feet. This can be achieved by using 3 inches of asphaltic concrete on 5 inches of Processed Miscellaneous Base (PMB) compacted to a minimum of 95 percent of the maximum dry density on subgrade soils compacted to a minimum of 95 percent of the maximum dry density.

The preliminary paving sections provided above have been designed for the type of traffic indicated. If the pavement is placed before construction on the project is complete, construction loads, which could increase the Traffic Indices above those assumed above, should be taken into account.

PRELIMINARY CONCRETE PAVING SECTIONS

Concrete paving sections provided below have been based on an assumed design life of 20 years and have been calculated for the measured R-Value of 64 (approximately equivalent to a coefficient of subgrade reaction of k = 240 pounds per cubic inch) using design methods presented by the American Concrete Institute (ACI 330R-87). For an assumed Traffic Index of 5 (for light traffic with the heaviest vehicles limited to UPS type trucks), the following minimum unreinforced paving section was determined:

1. Concrete thickness = 4.5 inches
2. Aggregate base thickness under concrete = 4 inches
3. Compressive strength of concrete, fc = 3,500 psi at 28 days
4. Modulus of flexural strength of 3,500 psi concrete = 530 psi
5. Maximum spacing of contraction joints, each way= 11 feet

For an assumed Traffic Index of 6.5 (for traffic that includes fire trucks), the following minimum unreinforced paving section was determined:

1. Concrete thickness = 5.5 inches
2. Aggregate base thickness under concrete = 4 inches
3. Compressive strength of concrete, fc = 3,500 psi at 28 days
4. Modulus of flexural strength of 3,500 psi concrete = 530 psi
5. Maximum spacing of contraction joints, each way= 13.5 feet

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

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 grading plans during the design phase of the project.
2. Observation and testing during site preparation, grading, placing of subdrainage systems and engineered fill, and permeable base.
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 drilled on the site. The nature and extent of variations between and beyond the 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 locations of the 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.

SITE-SPECIFIC BIBLIOGRAPHY

Earth Systems Southern California, November 14, 2011, Engineering Geology and Geotechnical Engineering Report for Proposed Aquatic Center at Hueneme High School, 500 West Bard Road, Oxnard, California (Job No. VT-24627-01).

Earth Systems Southern California, December 14, 2011, Addendum to Engineering Geology and Geotechnical Engineering Report for Proposed Aquatic Center at Hueneme High School, 500 West Bard Road, Oxnard, California (Job No. VT-24627-01).
American Concrete Institute (ACI), 2009, ACI 318-14.

GENERAL BIBLIOGRAPHY

California Building Standards Commission, 2016, California Building Code, California Code of Regulations Title 24.

California Division of Mines and Geology (C.D.M.G.), 1972 (Revised 1999), Fault Rupture Hazard Zones in California, Special Publication 42.

C.D.M.G., 1975, Seismic Hazards Study of Ventura County, California, Open File Report 76-5-LA.

California Geological Survey (C.G.S.), 2002a, Seismic Hazard Zone Report for the Oxnard 7.5-Minute Quadrangle, Ventura County, California, Seismic Hazard Zone Report 052.

C.G.S., 2002b, State of California Seismic Hazard Zones, Oxnard Quadrangle, Official Map, December 20, 2002.

C.G.S., 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A.

Clahan, Kevin B., 2003, Geologic Map of the Oxnard 7.5' Quadrangle, Ventura County, California: A Digital Database, Version 1.0, U.S.G.S., S.C.A.M.P., and C.G.S. Map.

County of Los Angeles Department of Public Works, July 1, 2013, Manual for Preparation of Geotechnical Reports.

Idriss, I.M., and Boulanger, R.W., 2008, Soil Liquefaction during Earthquakes, Earthquake Engineering Research Institute, MNO-12.

Jennings, C.W. and W.A. Bryant, 2010, Fault Activity Map of California, C.G.S. Geologic Data Map No. 6.

NCEER, 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Technical Report NCEER-97-0022.

Pradel, D., 1998 Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 4, April.

Pyke, R., Seed, H. B. And Chan, C. K., 1975, Settlement of Sands Under Multidirectional Shaking, ASCE, Journal of Geotechnical Engineering, Vol. 101, No. 4, April, 1975.

Seed, H. B., and Silver, M. L., 1972, Settlement of Dry Sands During Earthquakes, ASCE, Journal of Geotechnical Engineering, Vol. 98, No. 4.

Seed, H.B., 1987, Design Problems in Soil Liquefaction, Journal of the Geotechnical Engineering Division, ASCE, Volume 113, No. 8.

Southern California Earthquake Center (SCEC), 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California.

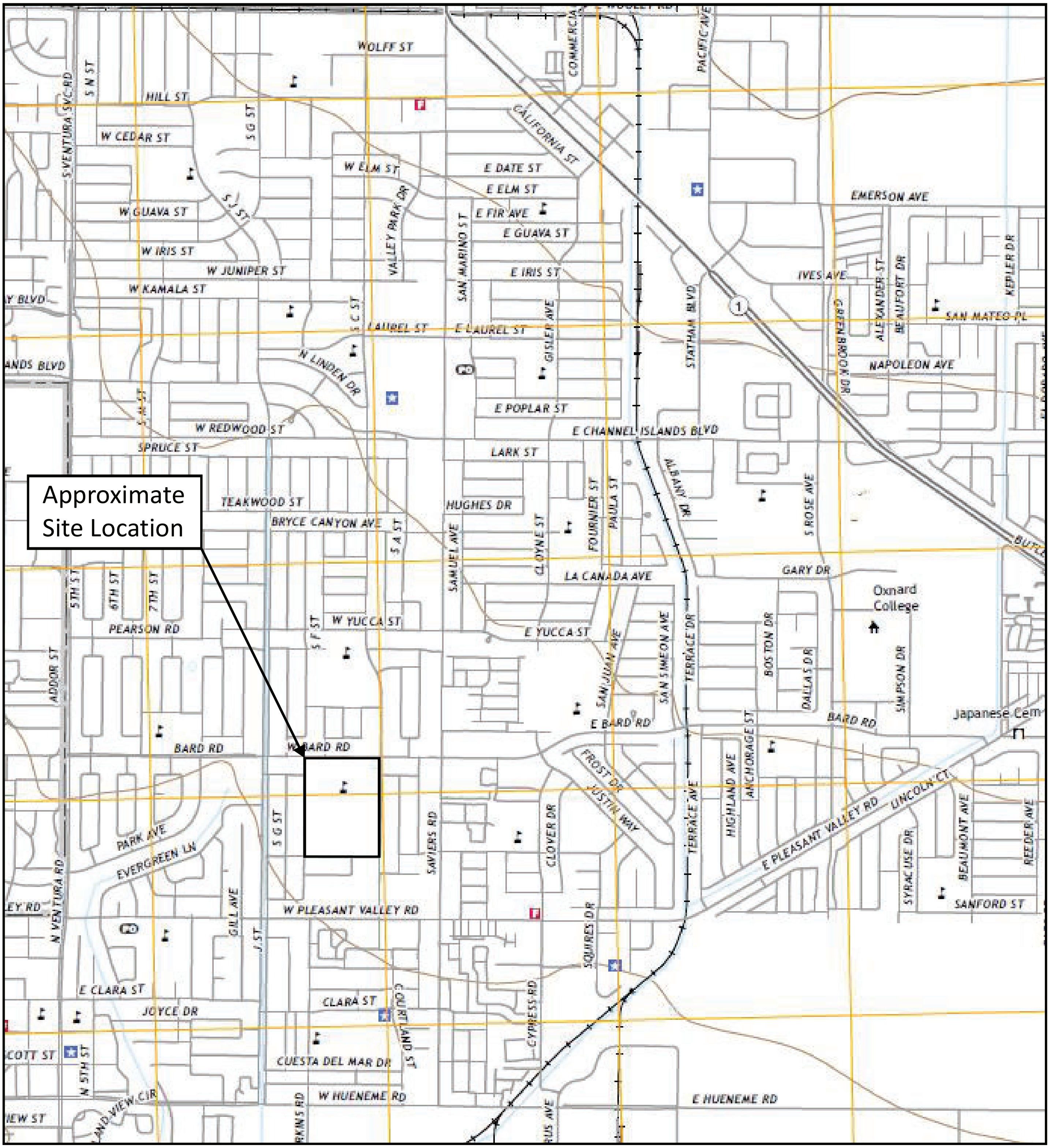
Tokimatsu, Kohji and H. Bolton Seed, 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, August 1987, New York, New York.

Ventura County Planning Department, October 22, 2013, Ventura County General Plan Hazards Appendix.

Weber, F. Harold, Jr. and others, 1973, Geology and Mineral Resources of Southern Ventura County, California, C.D.M.G., Preliminary Report 14.

Youd, T.L., C.M. Hansen, and S.F. Bartlett, 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement, in Journal of Geotechnical and Geoenvironmental Engineering, December 2002.

APPENDIX A



Approximate Site Location

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

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

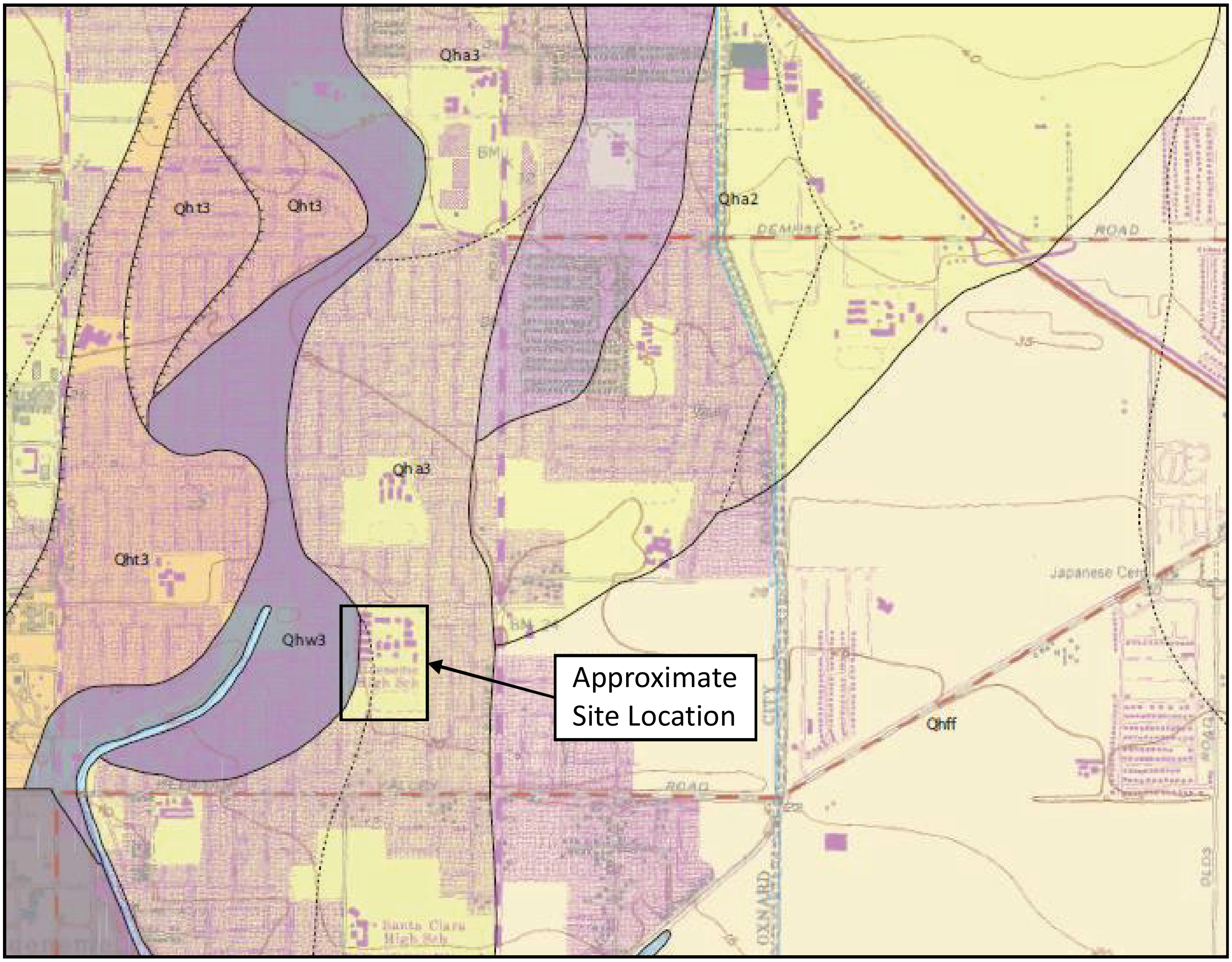
N

VICINITY MAP

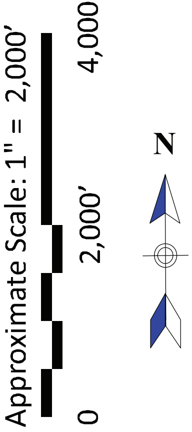
Hueneme High School Synthetic Field
Oxnard, California

Earth Systems

August 2019	303277-001



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



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.

Qha3: Holocene alluvial deposits

Qhw3: Holocene wash deposit

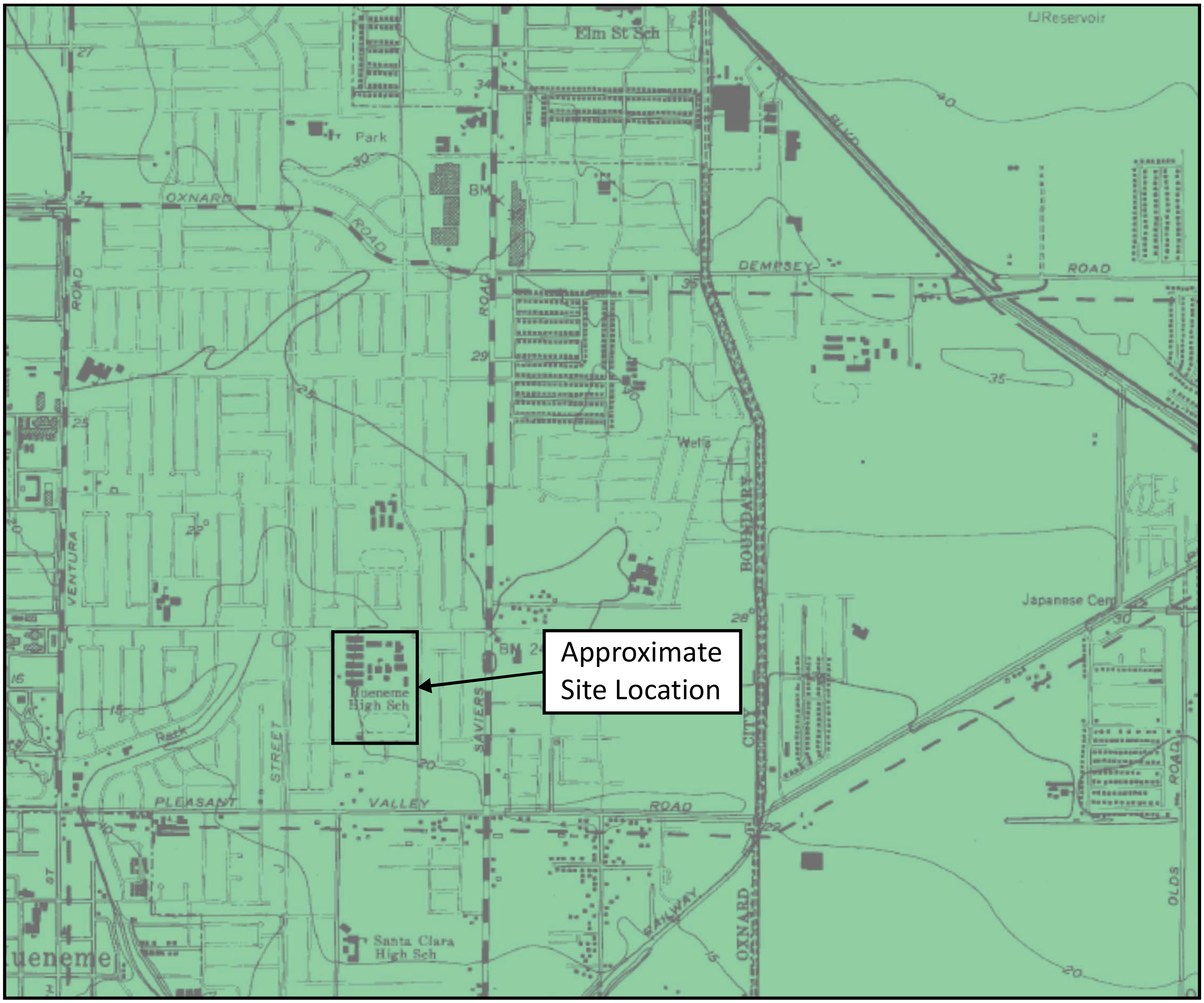
REGIONAL GEOLOGIC MAP

Hueneme High School Synthetic Field
Oxnard, California

Earth Systems

August 2019

303277-001



MAP EXPLANATION

Zones of Required Investigation:

Liquefaction

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

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

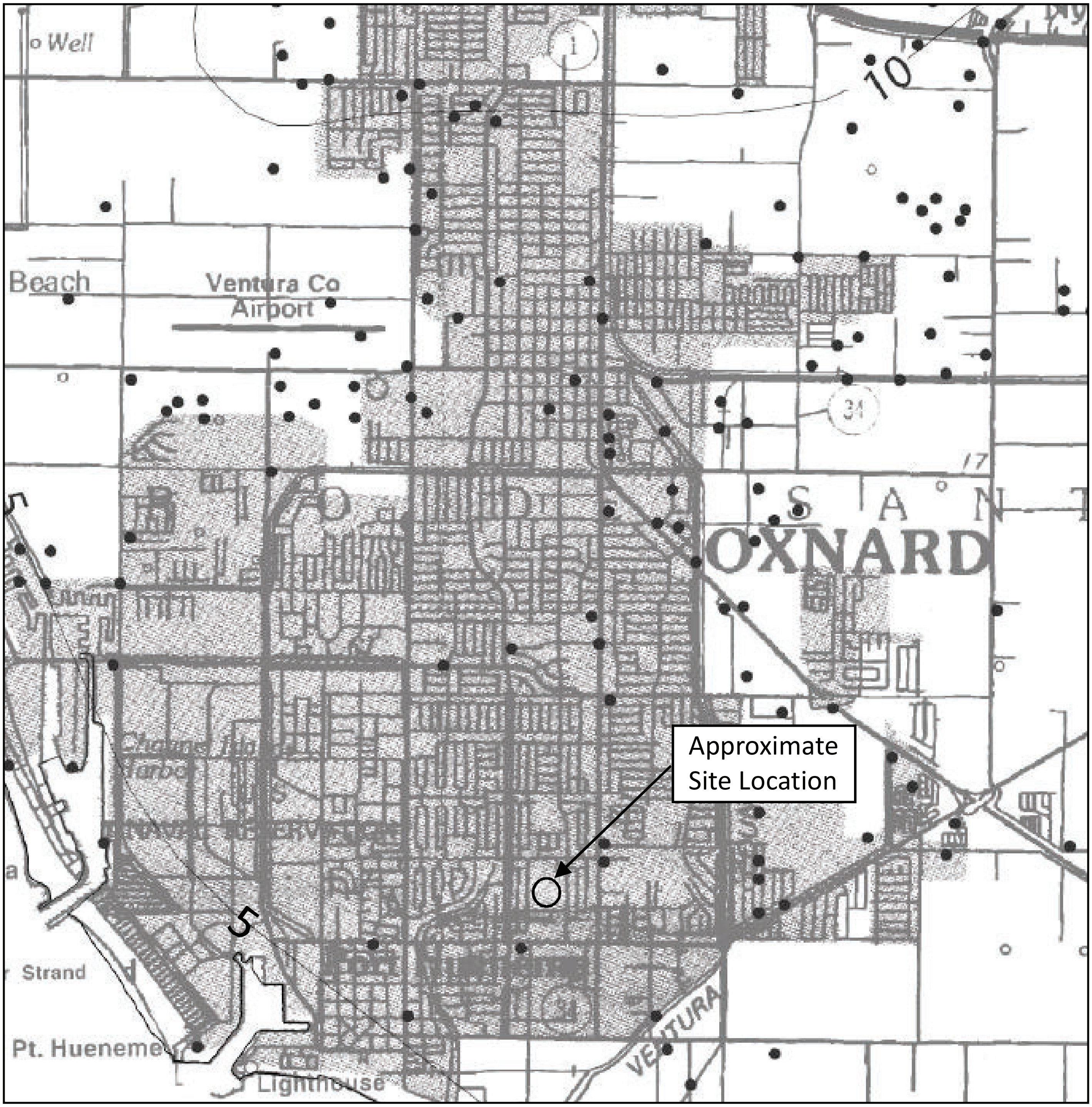
SEISMIC HAZARD ZONES MAP

Hueneme High School Synthetic Field
Oxnard, California

Earth Systems

August 2019

303277-001



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

30 Depth to ground water in feet

● Borehole Site

Approximate Scale: 1" = 4,000’

0 4,000’ 8,000’

N

HISTORICAL HIGH GROUNDWATER MAP

Hueneme High School Synthetic Field
Oxnard, California

Earth Systems

August 2019

303277-001

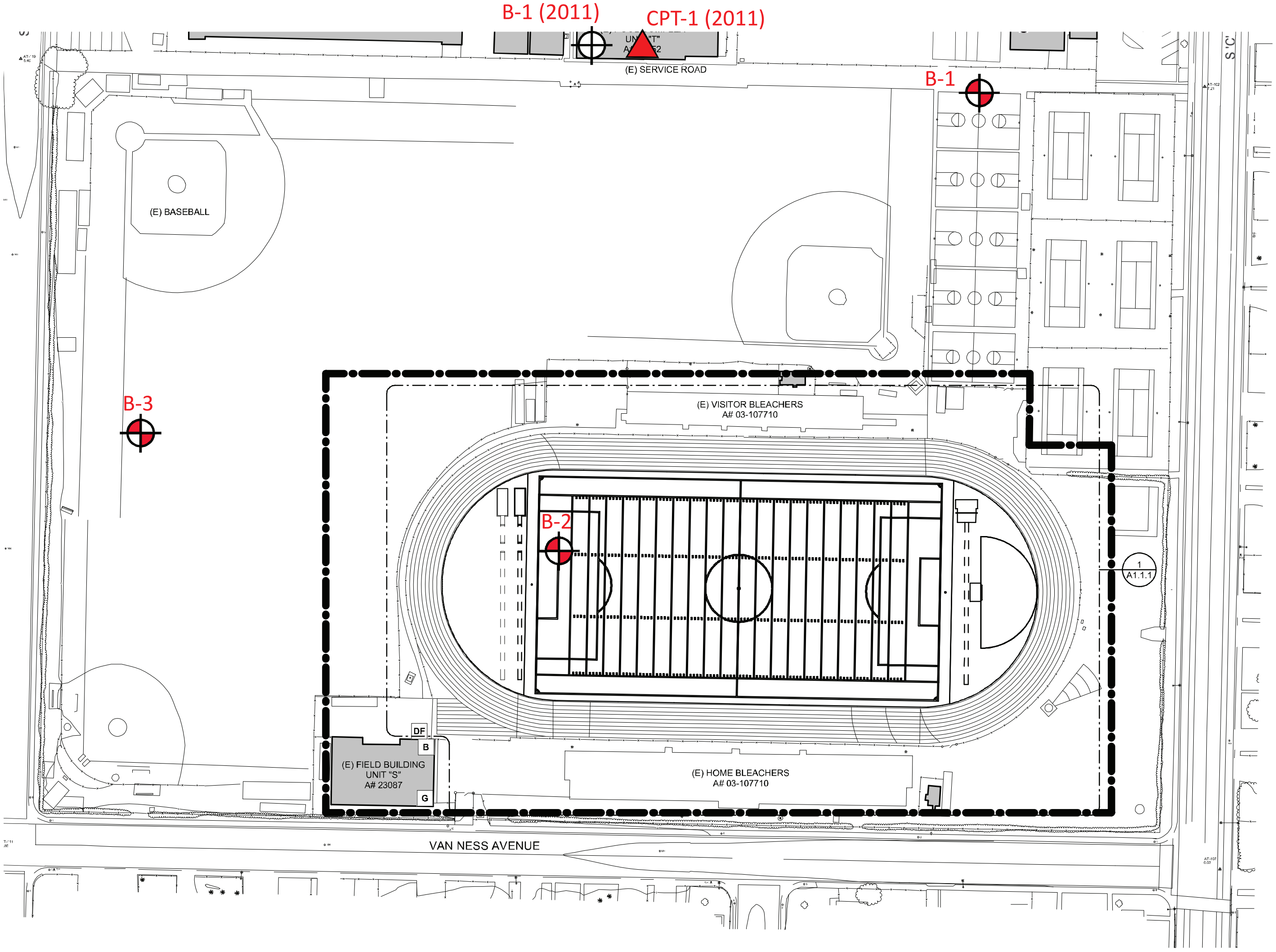
FIELD STUDY

A. Three borings were drilled to a maximum depth of 10 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory analysis. The borings were drilled on June 26, 2019, using an 8-inch diameter hollow stem auger powered by a track-mounted CME-75 drilling rig. The approximate locations of the test borings were determined in the field by pacing and sighting, and are shown on the Site Plan in this Appendix.

B. 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.

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

D. 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

B-1 ⊕ : Approximate boring location (2011 Site Studies)

CPT-1 ▲ : Approximate Cone Penetrometer Test (CPT) sounding (2011 Site Studies)

N

Approximate Scale: 1" = 100’

0 100’ 200’

SITE PLAN

Hueneme High School
Oxnard, California

Earth Systems

August 2019	303277-001

Earth Systems

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

BORING NO: B-1
PROJECT NAME: Hueneme HS Synthetic Field
PROJECT NUMBER: 303277-001
BORING LOCATION: Per Plan

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

Vertical Depth	Sample Type Bulk	 SPT	 Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0					▒				**Asphalt: 3.0", Base Material: 2.0"**
			■	4/8/10	▒	SP	97.0	4.7	**ALLUVIUM:** Light brown fine Sand, little medium Sand, trace iron oxide staining, medium dense, dry to damp
5			■	5/8/10	░	SW	89.2	8.1	**ALLUVIUM:** Gray Brown fine to medium Sand, little Silt, trace coarse Sand, medium dense, moist
	▼		■	3/5/5	░	SW	92.3	25.0	**ALLUVIUM:** Gray Brown fine to medium Sand, little Silt, trace coarse Sand, loose, moist
10									Total Depth: 10 feet Groundwater Depth: 7.5 feet
15									
20									
25									
30									
35									

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

Earth Systems

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

BORING NO: B-2
PROJECT NAME: Hueneme HS Synthetic Field
PROJECT NUMBER: 303277-001
BORING LOCATION: Per Plan

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

Vertical Depth	Sample Type Bulk	SPT	Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0	☒		■	7/11/14	▦	SM	103.5	3.4	**ALLUVIUM:** Brown Silty fine Sand, medium dense, damp
	☒				▒	SP			**ALLUVIUM:** Yellow Gray fine Sand, medium dense, dry to damp
5			■	5/8/8	░	SW	97.1	3.5	**ALLUVIUM:** Light Yellow Gray fine to medium Sand, little coarse Sand, trace fine Gravel, medium dense, dry to damp
	▼		■	5/7/6	░	SW	101.1	18.5	**ALLUVIUM:** Light Yellow Gray fine to medium Sand, little coarse Sand, trace fine Gravel, loose, dry to damp
10									Total Depth: 10 feet Groundwater Depth: 10 feet
15									
20									
25									
30									
35									

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

Earth Systems

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

BORING NO: B-3
PROJECT NAME: Hueneme HS Synthetic Field
PROJECT NUMBER: 303277-001
BORING LOCATION: Per Plan

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

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0	☒		■	4/4/4	▥	SM	104.1	10.1	**ALLUVIUM:** Gray Brown Silty fine Sand, loose, damp
5	▼		■	3/3/5	▨	SC	90.8	32.7	**ALLUVIUM:** Gray Clayey fine Sand, trace iron oxide staining, loose, moist to very moist
			■	3/4/5	⁘	SW			**ALLUVIUM:** Gray fine to medium Sand, loose, wet
10									Total Depth: 10 feet Groundwater Depth: 7 feet
15									
20									
25									
30									
35									

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

Earth Systems Southern California

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

BORING NO: 1	DRILLING DATE: October 13, 2011
PROJECT NAME: Hueneme High School Aquatics Center	DRILL RIG: CME 75
PROJECT NUMBER: VT-24627-01	DRILLING METHOD: 8" Hollow Stem
BORING LOCATION: Per Plan	LOGGED BY: G. Olin

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	☒				▥	SM			**ARTIFICIAL FILL:** 1" AC over silty sand with some pebbles; slightly moist; medium dense; brown
	☒		■	2/4/5	░	SP - SM	92.3	14.0	**ALLUVIUM:** Poorly graded medium sand with silt; slightly moist; loose; yellowish brown
5			■	7/7/6	░	SP - SM	102.3	20.9	**ALLUVIUM:** Poorly graded medium to coarse sand with silt and some gravel; wet; loose; orange yellow gray
		■		1/3/7	░	SP - SM			**ALLUVIUM:** Poorly graded medium to coarse sand with silt and some gravel; wet; medium dense; orange yellow gray
10		■		3/5/7	░	SP - SM			**ALLUVIUM:** Poorly graded medium to coarse sand with silt and some gravel; wet; medium dense; orange yellow gray
15		■		5/4/6	░	SP - SM			**ALLUVIUM:** Poorly graded medium to coarse sand with silt and some gravel; wet; medium dense; orange yellow gray
					░	SP - SM			**ALLUVIUM:** Poorly graded medium sand with silt; wet; medium dense; purple gray
					▨	CL			**ALLUVIUM:** Silty clay; wet; medium stiff; gray
20		■		9/10/9	⁘	SW			**ALLUVIUM:** Well graded medium to coarse sand with some gravel; medium dense; medium dense; purple gray
25		■		3/4/6	▨	CL			**ALLUVIUM:** Sandy silty clay; wet; stiff; mottled gray and yellow brown
					▥	SM - ML			**ALLUVIUM:** Clayey silt & sand;wet;med. dense;gray & yel. brown
30		■		5/2/4	▨	CL			**ALLUVIUM:** Sandy silty clay; wet; medium stiff; gray
		■		3/3/5	▨	CL			**ALLUVIUM:** Sandy silty clay; wet; medium stiff; gray
35					▥	SM			**ALLUVIUM:** Silty sand; wet; medium dense; gray
		■		8/6/5	▥	ML			**ALLUVIUM:** Clayey sandy silt; wet; stiff; gray

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

BORING NO: 1 (Continued)	DRILLING DATE: October 13, 2011
PROJECT NAME: Hueneme High School Aquatics Center	DRILL RIG: CME 75
PROJECT NUMBER: VT-24627-01	DRILLING METHOD: 8" Hollow Stem
BORING LOCATION: Per Plan	LOGGED BY: G. Olin

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					▥	ML			**ALLUVIUM:** Clayey sandy silt; wet; stiff; gray
		■		4/3/3	▨	CL			**ALLUVIUM:** Silty clay with sand; wet; medium stiff; gray
45									
		■		1/2/3	▨	CH			**ALLUVIUM:** Fat clay with sand; wet; medium stiff; dark gray
50									
									TOTAL DEPTH: 49.5 Feet
55									**Groundwater Encountered At 6 Feet**
60									
65									
70									
75									

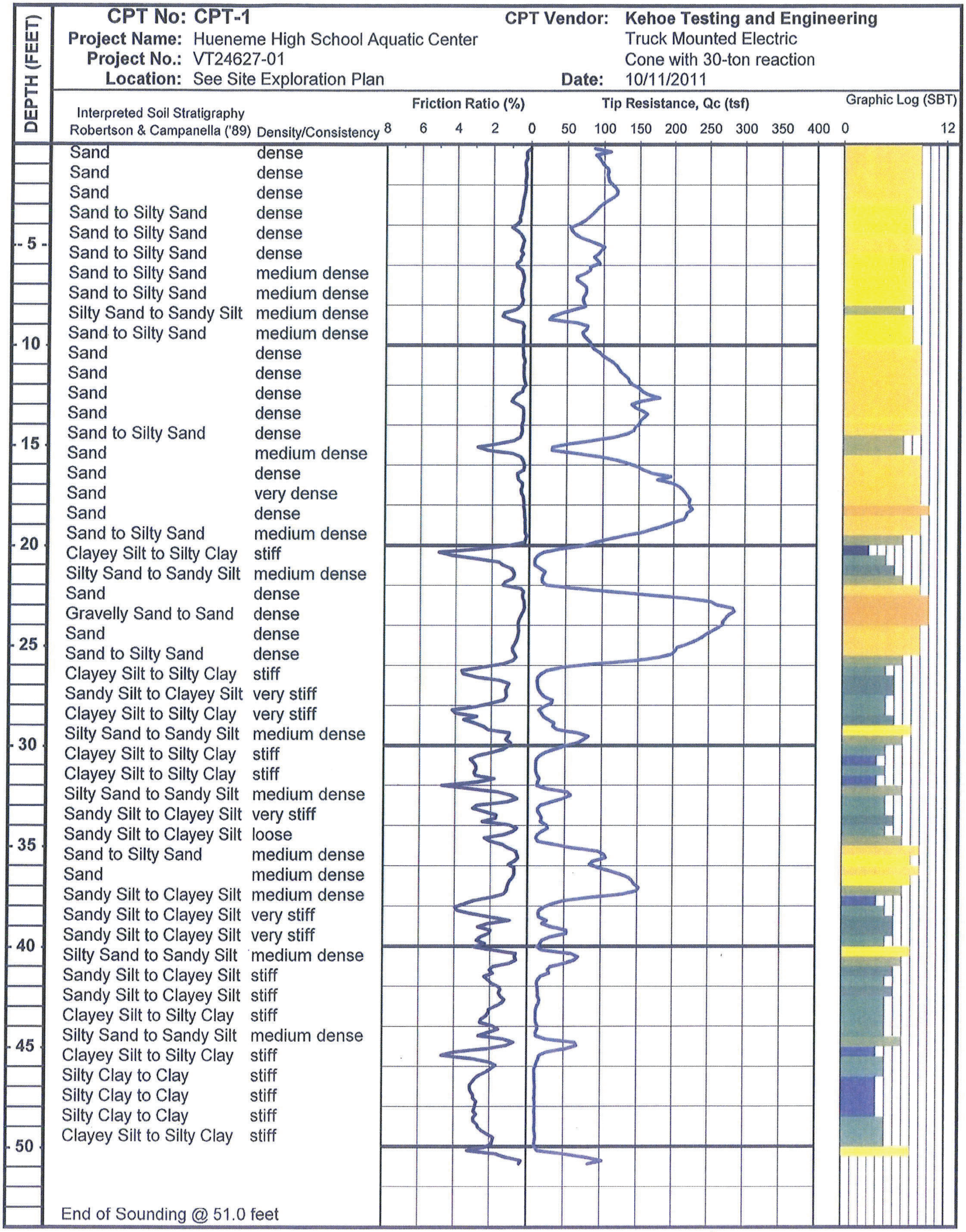
Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

CPT No: CPT-1
Project Name: Hueneme High School Aquatic Center
Project No.: VT24627-01
Location: See Site Exploration Plan

CPT Vendor: Kehoe Testing and Engineering
Truck Mounted Electric
Cone with 30-ton reaction
Date: 10/11/2011

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

End of Sounding @ 51.0 feet



Earth Systems
Southern California

CONE PENETROMETER INTERPRETATION

(based on Robertson & Campanella, 1989)

Project: Hueneme High School Aquatic Center | Project No: VT24627-01 | Date: 10/11/11

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

Est. GWT (feet): 6.9 | Dr correlation: 0 Baldi | Qc/N: 0 Jefferies & Davies | Phi Correlation: 4 SPT N

													Initial			Second															Nk: 17	
Base Depth meters	Base Depth feet	Avg Tip Qc, tsf	Avg Friction Ratio, %	Soil Classification	USCS	Density or Consistency	Est. Density (pcf)	Qc to N	SPT N(60)	Total po tsf	p'o tsf	F	Cq	Q	Ic	n	Cq	Q	Ic	n	Cq	Norm. Qc1n	2.6 Ic	Kc	Clean Sand Qc1n	$N_{1(60)}$	Clean Sand $N_{1(60)}$	Est. % Fines	Rel Dens. Dr (%)	Phi (deg.)	Su (tsf)	OCR
0.15	0.5	96.40	0.24	Sand	SP	medium dense	100	6.2	15	0.013	0.013	0.24	1.70	154.9	1.42	0.50	1.70	154.9	1.42	0.50	1.70	154.9	1.42	1.00	154.9	26	31	5	95	35		
0.30	1.0	99.50	0.32	Sand	SP	medium dense	100	6.1	16	0.038	0.038	0.32	1.70	159.8	1.48	0.50	1.70	159.8	1.48	0.50	1.70	159.9	1.48	1.00	159.9	28	32	5	96	35		
0.46	1.5	104.73	0.29	Sand	SP	medium dense	100	6.2	17	0.063	0.063	0.29	1.70	168.2	1.42	0.50	1.70	168.2	1.42	0.50	1.70	168.3	1.42	1.00	168.3	29	34	5	98	35		
0.61	2.0	110.67	0.33	Sand	SP	dense	100	6.2	18	0.088	0.088	0.33	1.70	177.7	1.42	0.50	1.70	177.7	1.42	0.50	1.70	177.8	1.42	1.00	177.8	30	36	5	100	36		
0.76	2.5	115.40	0.37	Sand	SP	dense	100	6.2	19	0.113	0.113	0.37	1.70	185.2	1.44	0.50	1.70	185.2	1.44	0.50	1.70	185.4	1.44	1.00	185.4	32	37	5	100	36		
0.91	3.0	97.47	0.47	Sand	SP	medium dense	100	6.0	16	0.138	0.138	0.47	1.70	156.4	1.56	0.50	1.70	156.4	1.56	0.50	1.70	156.6	1.55	1.00	156.6	28	31	10	95	35		
1.07	3.5	82.90	0.57	Sand to Silty Sand	SP/SM	medium dense	100	5.7	14	0.163	0.163	0.57	1.70	132.9	1.66	0.51	1.70	132.9	1.66	0.51	1.70	133.2	1.66	1.01	135.0	25	27	15	89	35		
1.22	4.0	63.23	0.60	Sand to Silty Sand	SP/SM	medium dense	120	5.4	12	0.190	0.190	0.60	1.70	101.3	1.85	0.56	1.70	101.3	1.85	0.56	1.70	101.6	1.85	1.14	116.0	20	23	20	77	33		
1.37	4.5	60.00	0.60	Sand to Silty Sand	SP/SM	medium dense	120	5.5	11	0.220	0.220	0.60	1.70	96.1	1.79	0.55	1.70	96.1	1.79	0.55	1.70	96.4	1.79	1.10	106.0	19	21	20	75	33		
1.52	5.0	89.27	0.43	Sand	SP	medium dense	120	5.9	15	0.250	0.250	0.43	1.70	143.0	1.57	0.50	1.70	143.0	1.57	0.50	1.70	143.4	1.56	1.00	143.4	26	29	10	92	35		
1.68	5.5	91.00	0.53	Sand	SP	medium dense	120	5.8	16	0.280	0.280	0.53	1.70	145.8	1.61	0.50	1.70	145.8	1.61	0.50	1.70	146.2	1.61	1.00	146.2	26	29	10	93	35		
1.83	6.0	84.73	0.68	Sand to Silty Sand	SP/SM	medium dense	120	5.7	15	0.310	0.310	0.68	1.70	135.7	1.70	0.52	1.70	135.7	1.70	0.52	1.70	136.1	1.70	1.04	141.3	25	28	15	90	35		
1.98	6.5	66.00	0.40	Sand to Silty Sand	SP/SM	medium dense	120	5.7	11	0.340	0.340	0.40	1.70	105.5	1.67	0.51	1.70	105.5	1.67	0.51	1.70	106.0	1.66	1.00	106.0	20	21	15	79	33		
2.13	7.0	72.83	0.48	Sand to Silty Sand	SP/SM	medium dense	120	5.7	13	0.370	0.367	0.48	1.70	116.4	1.67	0.51	1.70	116.4	1.67	0.51	1.70	117.0	1.67	1.00	117.0	21	23	15	83	34		
2.29	7.5	72.23	0.47	Sand to Silty Sand	SP/SM	medium dense	120	5.7	13	0.400	0.381	0.47	1.70	115.5	1.67	0.51	1.68	114.1	1.67	0.51	1.68	114.9	1.67	1.00	114.9	20	23	15	83	33		
2.44	8.0	68.23	0.61	Sand to Silty Sand	SP/SM	medium dense	120	5.6	12	0.430	0.396	0.62	1.70	109.0	1.75	0.53	1.69	108.4	1.75	0.53	1.69	109.1	1.75	1.07	117.1	19	23	20	80	33		
2.59	8.5	32.27	1.35	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.7	7	0.460	0.410	1.37	1.70	51.2	2.22	0.68	1.70	51.2	2.22	0.68	1.70	51.8	2.22	1.72	88.9	11	18	45	50	30		
2.74	9.0	71.83	0.41	Sand to Silty Sand	SP/SM	medium dense	120	5.7	12	0.490	0.424	0.41	1.70	114.7	1.64	0.50	1.58	106.6	1.67	0.51	1.59	107.9	1.66	1.00	107.9	19	22	15	80	33		
2.90	9.5	72.60	0.40	Sand to Silty Sand	SP/SM	medium dense	120	5.7	13	0.520	0.439	0.41	1.70	115.9	1.63	0.50	1.55	105.9	1.67	0.51	1.56	107.3	1.66	1.00	107.3	19	21	15	80	33		
3.05	10.0	82.90	0.42	Sand to Silty Sand	SP/SM	medium dense	120	5.8	14	0.550	0.453	0.42	1.70	132.5	1.59	0.50	1.53	119.1	1.63	0.50	1.53	119.7	1.63	1.00	119.7	21	24	15	84	34		
3.20	10.5	98.50	0.38	Sand	SP	medium dense	120	6.0	16	0.580	0.468	0.38	1.70	157.5	1.50	0.50	1.50	139.4	1.55	0.50	1.50	140.0	1.55	1.00	140.0	24	28	10	91	34		
3.35	11.0	116.47	0.35	Sand	SP	medium dense	120	6.1	19	0.610	0.482	0.35	1.70	186.4	1.42	0.50	1.48	162.4	1.47	0.50	1.48	163.1	1.47	1.00	163.1	27	33	5	97	35		
3.51	11.5	128.50	0.36	Sand	SP	medium dense	120	6.2	21	0.640	0.496	0.37	1.70	205.7	1.40	0.50	1.46	176.6	1.45	0.50	1.46	177.3	1.45	1.00	177.3	30	35	5	100	36		
3.66	12.0	141.63	0.32	Sand	SP	dense	120	6.3	23	0.670	0.511	0.32	1.70	226.8	1.33	0.50	1.44	192.0	1.39	0.50	1.44	192.6	1.39	1.00	192.6	32	39	5	100	36		
3.81	12.5	165.10	0.70	Sand	SP	dense	120	6.0	28	0.700	0.525	0.70	1.70	264.4	1.50	0.50	1.42	220.8	1.55	0.50	1.42	221.5	1.55	1.00	221.5	38	44	10	100	38		
3.96	13.0	144.90	0.69	Sand	SP	dense	120	5.9	25	0.730	0.540	0.69	1.70	232.0	1.53	0.50	1.40	191.0	1.59	0.50	1.40	191.8	1.59	1.00	191.8	34	39	10	100	37		
4.11	13.5	157.60	0.38	Sand	SP	dense	120	6.2	25	0.760	0.554	0.38	1.70	252.3	1.33	0.50	1.38	205.1	1.41	0.50	1.38	205.8	1.41	1.00	205.8	34	41	5	100	37		
4.27	14.0	148.13	0.43	Sand	SP	dense	120	6.1	24	0.790	0.568	0.43	1.70	237.1	1.39	0.50	1.36	190.3	1.47	0.50	1.36	191.0	1.47	1.00	191.0	32	38	5	100	37		
4.42	14.5	129.50	0.49	Sand	SP	medium dense	120	6.0	22	0.820	0.583	0.50	1.70	207.1	1.47	0.50	1.35	164.2	1.56	0.50	1.35	164.9	1.56	1.00	164.9	28	33	10	98	36		
4.57	15.0	55.83	1.83	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.7	12	0.850	0.597	1.85	1.70	88.8	2.13	0.65	1.45	75.6	2.18	0.66	1.46	77.0	2.17	1.60	123.5	15	25	40	66	32		
4.72	15.5	62.87	1.25	Silty Sand to Sandy Silt	SM/ML	medium dense	120	5.0	13	0.880	0.612	1.27	1.70	100.0	1.98	0.60	1.39	81.8	2.04	0.62	1.41	83.5	2.04	1.35	112.7	16	23	35	69	32		
4.88	16.0	139.25	0.35	Sand	SP	medium dense	120	6.1	23	0.910	0.626	0.35	1.69	221.4	1.36	0.50	1.30	170.3	1.46	0.50	1.30	171.1	1.45	1.00	171.1	29	34	5	99	36		
5.03	16.5	179.70	0.67	Sand	SP	dense	120	6.0	30	0.940	0.640	0.67	1.65	279.8	1.45	0.50	1.29	217.5	1.54	0.50	1.29	218.3	1.54	1.00	218.3	38	44	10	100	38		
5.18	17.0	204.93	0.44	Sand	SP	dense	120	6.3	33	0.970	0.655	0.45	1.62	311.9	1.31	0.50	1.27	245.4	1.39	0.50	1.27	246.2	1.38	1.00	246.2	40	49	5	100	39		
5.33	17.5	219.00	0.39	Sand	SP	dense	120	6.4	34	1.000	0.669	0.39	1.58	326.2	1.26	0.50	1.26	259.5	1.33	0.50	1.26	260.3	1.33	1.00	260.3	42	52	5	100	39		
5.49	18.0	222.50	0.33	Sand	SP	dense	120	6.5	34	1.030	0.684	0.33	1.55	324.4	1.21	0.50	1.24	260.8	1.28	0.50	1.24	261.6	1.28	1.00	261.6	41	52	0	100	39		
5.64	18.5	216.17	0.28	Gravelly Sand to Sand	SW	dense	120	6.5	33	1.060	0.698	0.28	1.52	308.7	1.19	0.50	1.23	250.7	1.26	0.50	1.23	251.5	1.26	1.00	251.5	40	50	0	100	38		
5.79	19.0	183.63	0.25	Sand	SP	dense	120	6.4	29	1.090	0.712	0.25	1.48	256.7	1.23	0.50	1.22	210.7	1.31	0.50	1.22	211.5	1.30	1.00	211.5	34	42	5	100	37		
5.94	19.5	130.43	0.21	Sand	SP	medium dense	120	6.2	21	1.120	0.727	0.21	1.46	178.4	1.33	0.50	1.21	147.9	1.41	0.50	1.21	148.7	1.41	1.00	148.7	24	30	5	93	35		
6.10	20.0	68.07	1.37	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.9	14	1.150	0.741	1.39	1.43	90.8	2.04	0.62	1.25	79.3	2.08	0.63	1.25	80.6	2.07	1.41	113.6	16	23	35	63	32		
6.25	20.5	12.50	4.02	Clay	CL/CH	stiff	120	3.3	4	1.180	0.756	4.44	1.40	15.5	2.96	0.89	1.35	15.0	2.96	0.90	1.35	16.0	2.94			4		100			0.69	4.5
6.40	21.0	9.33	1.35	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.5	3	1.210	0.770	1.55	1.37	11.1	2.80	0.85	1.31	10.6	2.82	0.85	1.31	11.6	2.79			3		95			0.50	3.2
6.55	21.5	18.73	0.83	Sandy Silt to Clayey Silt	ML	loose	120	4.2	4	1.240	0.784	0.92	1.35	22.9	2.42	0.73	1.25	21.1	2.45	0.74	1.25	22.1	2.43	2.45	54.1	5	11	60	14	28		
6.71	22.0	34.83	1.07	Silty Sand to Sandy Silt	SM/ML	loose	120	4.6	8	1.270	0.799	1.11	1.32	42.6	2.23	0.68	1.21	38.9	2.27	0.69	1.21	39.9	2.26	1.82	72.7	9	15	45	39	29		
6.86	22.5	163.93	0.35	Sand	SP	medium dense	120	6.2	27	1.300	0.813	0.35	1.30	200.6	1.40	0.50	1.14	175.9	1.44	0.50	1.14	176.7	1.44	1.00	176.7	29	35	5	100	36		
7.01	23.0	264.90	0.27	Gravelly Sand to Sand	SW	dense	120	6.6	40	1.330	0.828	0.28	1.28	319.1	1.17	0.50	1.13	282.2	1.22	0.50	1.13	283.1	1.21	1.00	283.1	44	57	0	100	39		
7.16	23.5	278.50	0.42	Gravelly Sand to Sand	SW	dense	120	6.4	43	1.360	0.842	0.42	1.26	329.7	1.27	0.50	1.12	294.2	1.31	0.50	1.12	295.1	1.31	1.00	295.1	47	59	5	100	40		
7.32	24.0	266.67	0.59	Gravelly Sand to Sand	SW	dense	120	6.2	43	1.390	0.856	0.59	1.24	310.4	1.39	0.50	1.11	279.2	1.43	0.50	1.11	280.1	1.43	1.00	280.1	46	56	5	100	40		
7.47	24.5	246.07	0.60	Sand	SP	dense	120	6.1	40	1.420	0.871	0.60	1.21	281.5	1.43	0.50	1.10	255.4	1.46	0.50	1.10	256.3	1.46	1.00	256.3	43	51	5	100	39		
7.62	25.0	217.53	0.79	Sand	SP	dense	120	5.9	37	1.450	0.885	0.80	1.20	244.7	1.56	0.50	1.09	223.9	1.59	0.50	1.09	224.8	1.58	1.00	224.8	39	45	10	100	38		
7.77	25.5	192.53	0.79	Sand	SP	dense	120	5.8	33	1.480	0.900	0.80	1.18	213.0	1.60	0.50	1.08	196.4	1.63	0.50	1.08	197.3	1.63	1.00	197.3	35	39	15	100	37		
7.92	26.0	92.27	1.60	Silty Sand to Sandy Silt	SM/ML	medium dense	120	5.0	19	1.510	0.914	1.63	1.16	99.9	2.05	0.62	1.10	94.6	2.07	0.63	1.10	95.6	2.07	1.40	133.5	19	27	35	75	33		
8.08	26.5	15.85	2.92	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.5	5	1.540	0.928	3.23	1.14	16.1	2.65	0.86	1.12	15.8	2.65	0.86	1.12	16.8	2.63			5		100			0.68	4.6
8.23	27.0	12.27	1.16	Sandy Silt to Clayey Silt	ML	stiff	120	3.7	3	1.570	0.943	1.33	1.12	12.0	2.74	0.83	1.10	11.8	2.75	0.83	1.10	12.8	2.72			3		90			0.67	3.4
8.38	27.5	22.87	1.26	Sandy Silt to Clayey Silt	ML	loose	120	4.1	6	1.600	0.957	1.36	1.11	22.9	2.51	0.76	1.08	22.3	2.52	0.76	1.08	23.3	2.50	2.77	64.6	6	13	65	16	28		
8.53	28.0	21.27	3.04	Clayey Silt to Silty Clay	ML/CL	very stiff	120	3.6	6	1.630	0.972	3.30	1.09	20.9	2.76	0.84	1.07	20.6	2.77	0.84	1.07	21.6	2.75			6		90			1.19	6.1
8.69	28.5	21.03	3.45	Clayey Silt to Silty Clay	ML/CL	very stiff	120	3.5	6	1.660	0.986	3.75	1.07	20.3	2.81	0.85	1.06	20.1	2.81	0.85	1.06	21.1	2.80			6		95			1.18	5.9
8.84	29.0	35.03	2.55	Sandy Silt to Clayey Silt	ML	loose	120	4.0	9	1.690	1.000	2.68	1.06	34.0	2.54	0.77	1.04	33.6	2.55	0.77	1.04	34.6	2.54	2.97	102.8	9	21	70	33	30		
8.99	29.5	74.20	1.12	Sand to Silty Sand	SP/SM	medium dense	120	5.0	15	1.720	1.015	1.14	1.04	72.1	2.06	0.63	1.03	71.0	2.06	0.63	1.03	72.0	2.06	1.39	99.6	15	20	35	63	32		
9.14	30.0	50.37	1.13	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.7	11	1.750	1.029	1.17	1.03	47.9	2.20	0.67	1.02	47.5	2.21	0.67	1.02	48.5	2.20	1.67	80.9	11	16	45	47	30		
9.30	30.5	16.00	2.65	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.4	5	1.780	1.044	2.98	1.01	14.3	2.87	0.87	1.01	14.3	2.87	0.87	1.01	15.3	2.85			5		100			0.88	4.1
9.45	31.0	9.77	2.90	Silty Clay to Clay	CL	stiff	120	3.0	3	1.810	1.058	3.56	1.00	8.2	3.11	0.94	1.00	8.2	3.11	0.94	1.00	9.2	3.07			3		100			0.51	2.3
9.60	31.5	12.20	2.56	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.2	4	1.840	1.072	3.02	0.99	10.4	2.99	0.90	0.99	10.4	2.98	0.90	0.99	11.4	2.95			4		100			0.65	2.9
9.75	32.0	15.90	3.58	Silty Clay to Clay	CL	stiff	120	3.2	5	1.870	1.087	4.05	0.97	13.6	2.97	0.90	0.98	13.7	2.97	0.90	0.98	14.7	2.94			5		100			0.87	3.9
9.91	32.5	52.23	1.06	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.7	11	1.900	1.101	1.10	0.96	46.4	2.20	0.67	0.97	47.1	2.19	0.67	0.97	48.1	2.19	1.64	78.7	11	16	40	46	30		
10.06	33.0	20.00	2.32	Clayey Silt to Silty Clay	ML/CL	very stiff	120	3.6	6	1.930	1.116	2.67	0.96	16.9	2.77	0.84	0.96	17.1	2.77	0.84	0.96	18.1	2.75			6		90			1.11	4.9
10.21	33.5	15.80	1.95	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.5	4	1.960	1.130	2.23	0.94	13.0	2.83	0.86	0.95	13.1	2.83	0.86	0.95	14.1	2.80			4		95			0.86	3.7
10.36	34.0	22.33	1.38	Sandy Silt to Clayey Silt	ML	loose	120	3.9	6	1.990	1.144	1.51	0.92	18.5	2.61	0.79	0.94	18.8	2.60	0.79	0.94	19.8	2.58	3.23	64.2	6	13	75	10	28		
10.52	34.5	13.47	1.51	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.5	4	2.020	1.159	1.77	0.91	10.6	2.85	0.88	0.92	10.8	2.85	0.88	0.92	11.8	2.81			4		100			0.72	3.0
10.67	35.0	36.40	1.51	Silty Sand to Sandy Silt	SM/ML	loose	120	4.2	9	2.050	1.173	1.60	0.90	30.0	2.45	0.74	0.93	30.8	2.44	0.74	0.93	31.9	2.43	2.43	77.5	8	15	60	29	29		
10.82	35.5	101.40	0.60	Sand	SP	medium dense	120	5.4	19	2.080	1.188	0.62	0.89	84.4	1.84	0.56	0.94	88.8	1.83	0.56	0.94	89.9	1.82	1.12	100.9	17	20	20	72	32		
10.97	36.0	89.03	0.85	Sand to Silty Sand	SP/SM	medium dense	120	5.2	17	2.110	1.202	0.87	0.88	73.1	1.98	0.60	0.93	76.9	1.96	0.60	0.93	78.0	1.96	1.25	97.4	16	19	30	67	32		
11.13	36.5	129.43	0.83	Sand	SP	medium dense	120	5.4	24	2.140	1.216	0.84	0.87	105.4	1.85	0.56	0.92	112.0	1.83	0.56	0.93	113.2	1.82	1.12	127.1	22	25	20	82	34		
11.28	37.0	149.63	1.12	Sand to Silty Sand	SP/SM	medium dense	120	5.3	28	2.170	1.231	1.14	0.86	119.8	1.89	0.57	0.92	127.7	1.87	0.57	0.92	128.9	1.87	1.16	149.2	25	30	25	87	36		
11.43	37.5	87.67	2.08	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.6	19	2.200	1.245	2.13	0.85	69.6	2.25	0.68	0.89	73.3	2.23	0.68	0.90	74.4	2.23	1.73	128.8	17	26	45	65	32		
11.58	38.0	22.97	3.77	Silty Clay to Clay	CL	very stiff	120	3.4	7	2.230	1.260	4.18	0.84	17.2	2.89	0.88	0.86	17.6	2.89	0.87	0.86	18.6	2.87			7		100			1.28	4.9
11.73	38.5	18.20	2.01	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.5	5	2.260	1.274	2.29	0.83	13.3	2.83	0.86	0.85	13.6	2.82	0.85	0.85	14.7	2.79			5		95			1.00	3.8
11.89	39.0	34.47	2.30	Sandy Silt to Clayey Silt	ML	loose	120	3.9	9	2.290	1.288	2.45	0.82	25.7	2.61	0.79	0.86	26.8	2.60	0.79	0.86	27.9	2.59	3.25	90.7	8	18	75	24	29		
12.04	39.5	34.33	2.47	Sandy Silt to Clayey Silt	ML	very stiff	120	3.9	9	2.320	1.303	2.65	0.81	25.4	2.64	0.80	0.85	26.4	2.63	0.80	0.85	27.5	2.61			9		80			1.94	7.4
12.19	40.0	16.50	2.36	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.4	5	2.350	1.317	2.76	0.80	11.5	2.92	0.89	0.82	11.8	2.92	0.88	0.82	12.9	2.89			5		100			0.89	3.2
12.34	40.5	63.73	0.65	Sand to Silty Sand	SP/SM	medium dense	120	5.0	13	2.380	1.332	0.69	0.79	46.9	2.08	0.63	0.86	51.0	2.05	0.62	0.87	52.2	2.04	1.36	71.0	11	14	35	50	30		
12.50	41.0	38.03	1.63	Silty Sand to Sandy Silt	SM/ML	loose	120	4.1	9	2.410	1.346	1.74	0.79	25.8	2.52	0.77	0.83	27.3	2.50	0.76	0.83	28.4	2.49	2.72	77.2	8	15	65	25	29		
12.65	41.5	20.20	2.11	Sandy Silt to Clayey Silt	ML	very stiff	120	3.6	6	2.440	1.360	2.40	0.78	13.8	2.83	0.86	0.81	14.4	2.81	0.85	0.81	15.4	2.79			6		95			1.11	3.9
12.80	42.0	13.20	1.65	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.3	4	2.470	1.375	2.03	0.77	8.6	2.96	0.90	0.79	8.8	2.95	0.89	0.79	9.9	2.91			4		100			0.70	2.3
12.95	42.5	14.70	1.41	Sandy Silt to Clayey Silt	ML	stiff	120	3.5	4	2.500	1.389	1.70	0.76	9.8	2.88	0.87	0.79	9.9	2.87	0.87	0.79	11.0	2.83			4		100			0.78	2.6
13.11	43.0	13.30	1.72	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.3	4	2.530	1.404	2.12	0.75	8.5	2.98	0.90	0.78	8.7	2.96	0.90	0.78	9.8	2.92			4		100			0.70	2.3
13.26	43.5	11.27	2.28	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.0	4	2.560	1.418	2.95	0.75	6.9	3.12	0.95	0.76	7.1	3.12	0.94	0.76	8.1	3.07			4		100			0.58	1.8
13.41	44.0	13.03	2.00	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.2	4	2.590	1.432	2.49	0.74	8.1	3.03	0.92	0.76	8.3	3.02	0.91	0.76	9.3	2.98			4		100			0.68	2.2
13.56	44.5	16.07	2.01	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.3	5	2.620	1.447	2.40	0.73	10.1	2.94	0.89	0.76	10.5	2.93	0.89	0.76	11.5	2.89			5		100			0.86	2.8
13.72	45.0	61.30	1.35	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.6	13	2.650	1.461	1.41	0.72	40.9	2.31	0.70	0.80	45.1	2.27	0.69	0.80	46.4	2.26	1.84	85.3	11	17	50	45	30		
13.87	45.5	19.20	3.91	Silty Clay to Clay	CL	very stiff	120	3.1	6	2.680	1.476	4.55	0.72	12.0	3.04	0.92	0.74	12.3	3.03	0.92	0.74	13.4	3.00			6		100			1.04	3.4
14.02	46.0	11.77	1.89	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.1	4	2.710	1.490	2.46	0.71	6.9	3.09	0.93	0.73	7.1	3.08	0.93	0.73	8.1	3.03			4		100			0.60	1.8
14.17	46.5	10.13	2.53	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.8	4	2.740	1.504	3.47	0.70	5.7	3.23	0.98	0.71	5.8	3.23	0.98	0.71	6.8	3.17			4		100			0.51	1.5
14.33	47.0	10.55	3.06	Silty Clay to Clay	CL	stiff	120	2.7	4	2.770	1.519	4.14	0.70	5.9	3.26	0.99	0.70	6.0	3.26	0.99	0.70	7.0	3.21			4		100			0.53	1.5
14.48	47.5	10.37	2.86	Silty Clay to Clay	CL	stiff	120	2.7	4	2.800	1.533	3.92	0.69	5.8	3.26	0.99	0.69	5.8	3.26	0.99	0.69	6.8	3.20			4		100			0.52	1.5
14.63	48.0	10.53	2.85	Silty Clay to Clay	CL	stiff	120	2.7	4	2.830	1.548	3.90	0.68	5.8	3.26	0.98	0.69	5.8	3.25	0.98	0.69	6.8	3.20			4		100			0.53	1.5
14.78	48.5	10.63	2.85	Silty Clay to Clay	CL	stiff	120	2.7	4	2.860	1.562	3.90	0.68	5.8	3.26	0.98	0.68	5.8	3.25	0.98	0.68	6.8	3.20			4		100			0.53	1.5
14.94	49.0	10.50	2.57	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.8	4	2.890	1.576	3.55	0.67	5.7	3.24	0.98	0.68	5.7	3.24	0.98	0.68	6.7	3.18			4		100			0.52	1.4
15.09	49.5	11.30	1.92	Clayey Silt to Silty Clay	ML/CL	stiff	120	3.0	4	2.920	1.591	2.59	0.67	6.1	3.14	0.95	0.68	6.2	3.13	0.95	0.68	7.3	3.09			4		100			0.57	1.6
15.24	50.0	12.13	2.43	Clayey Silt to Silty Clay	ML/CL	stiff	120	2.9	4	2.950	1.605	3.27	0.66	6.6	3.17	0.96	0.67	6.7	3.16	0.96	0.67	7.7	3.11			4		100			0.62	1.7

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

TABULATED LABORATORY TEST RESULTS

BORING AND DEPTH	B-2 @ 0-5’	B-3 @ 0-5’
USCS	SM	SM
MAXIMUM DENSITY (pcf)	--	121.0
OPTIMUM MOISTURE (%)	--	10.5
COHESION (psf)	--	0* 0**
ANGLE OF INTERNAL FRICTION	--	35°* 32°**
EXPANSION INDEX	--	20
RESISTANCE (“R”) VALUE	64	--
pH	--	8.2
SOLUBLE CHLORIDES (mg/Kg)	--	12
RESISTIVITY (ohms-cm)	--	2,100
SOLUBLE SULFATES (mg/Kg)	--	530
GRAIN SIZE DISTRIBUTION (%)		
GRAVEL	0	--
SAND	74	--
SILT AND CLAY	26	--

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

File Number: 303277-001 Lab Number: 098210

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Hueneme High School Synthetic Turf Field
Sample ID: B 3 @ 0-5'

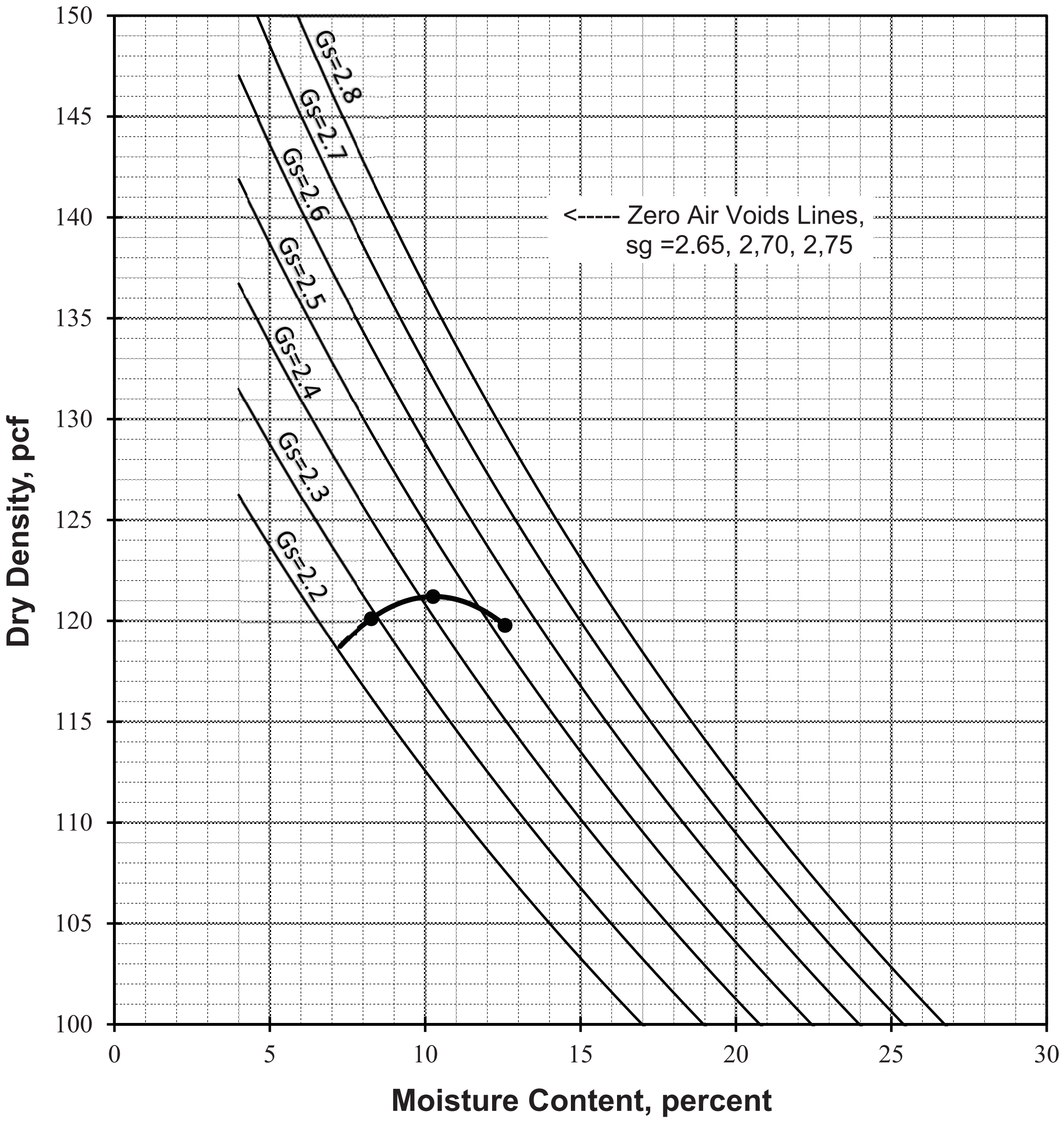
Procedure Used: A
Prep. Method: Moist

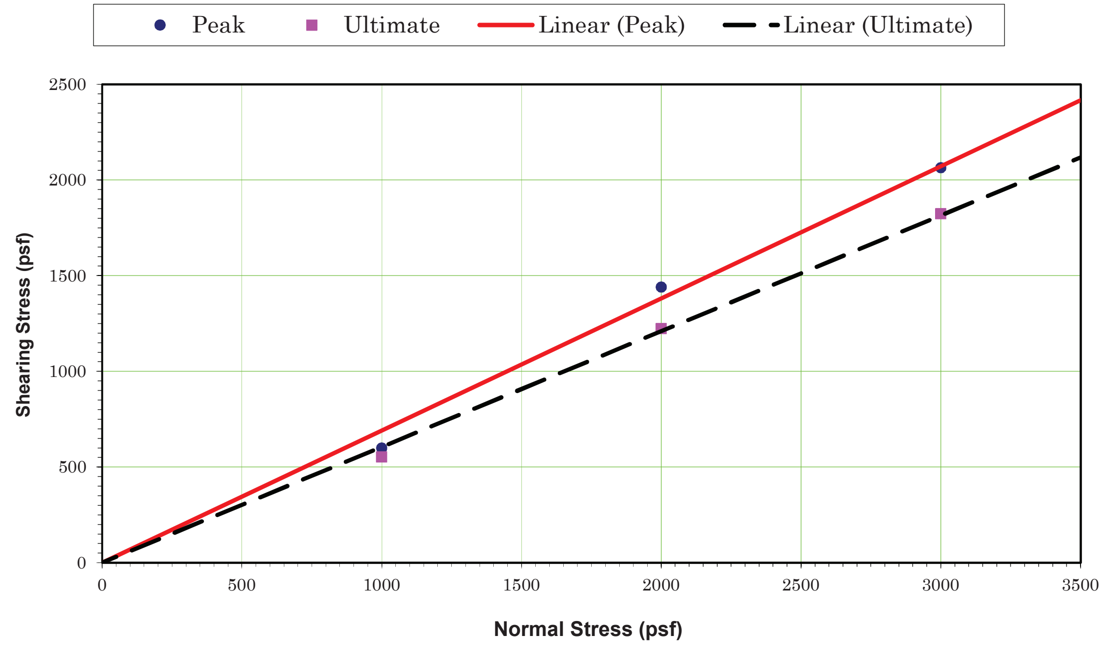
Date: 7/29/2019
Description: Dark Brown Silty Sand
SG: 2.43

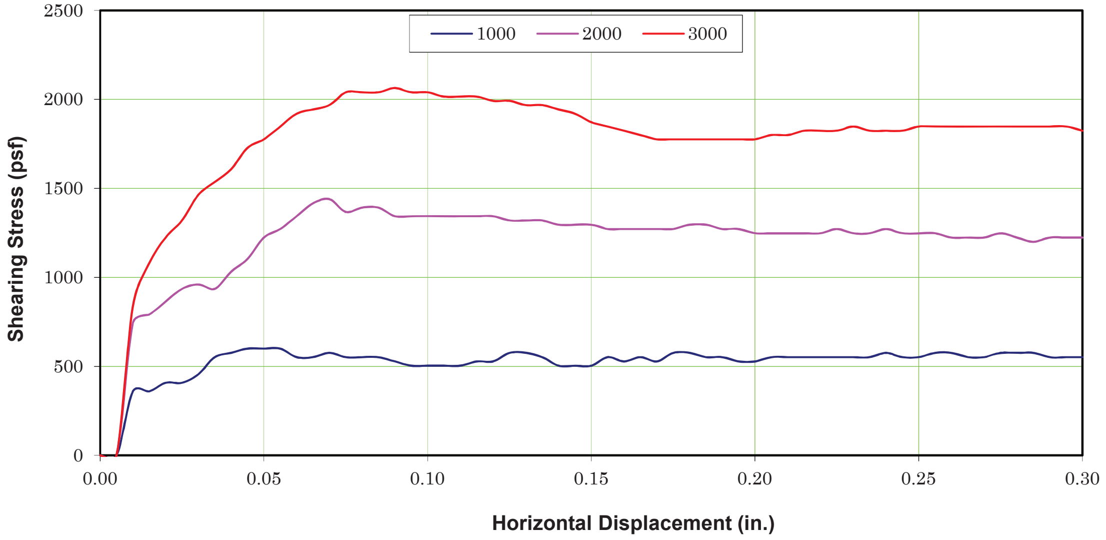
Rammer Type: Automatic

Maximum Density: **121 pcf**
Optimum Moisture: **10.5%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	0.2







DIRECT SHEAR DATA*

Sample Location: B 3 @ 0-5'
Sample Description: Silty Sand
Dry Density (pcf): 109.1
Intial % Moisture: 10.3
Average Degree of Saturation: 98.7
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	600	1440	2064
Ultimate stress (psf)	552	1224	1824

	Peak	Ultimate
ϕ Angle of Friction (degrees):	35	32
c Cohesive Strength (psf):	0	0

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Hueneme High School Synthetic Turf Field

Earth Systems

8/5/2019 303277-001

File No.: 303277-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

Job Name: Hueneme High School Synthetic Turf Field
Sample ID: B 3 @ 0-5'
Soil Description: SM

Initial Moisture, %:	9.5
Initial Compacted Dry Density, pcf:	110.6
Initial Saturation, %:	49
Final Moisture, %:	20.6
Volumetric Swell, %:	2.0

Expansion Index: **20** **Very Low**

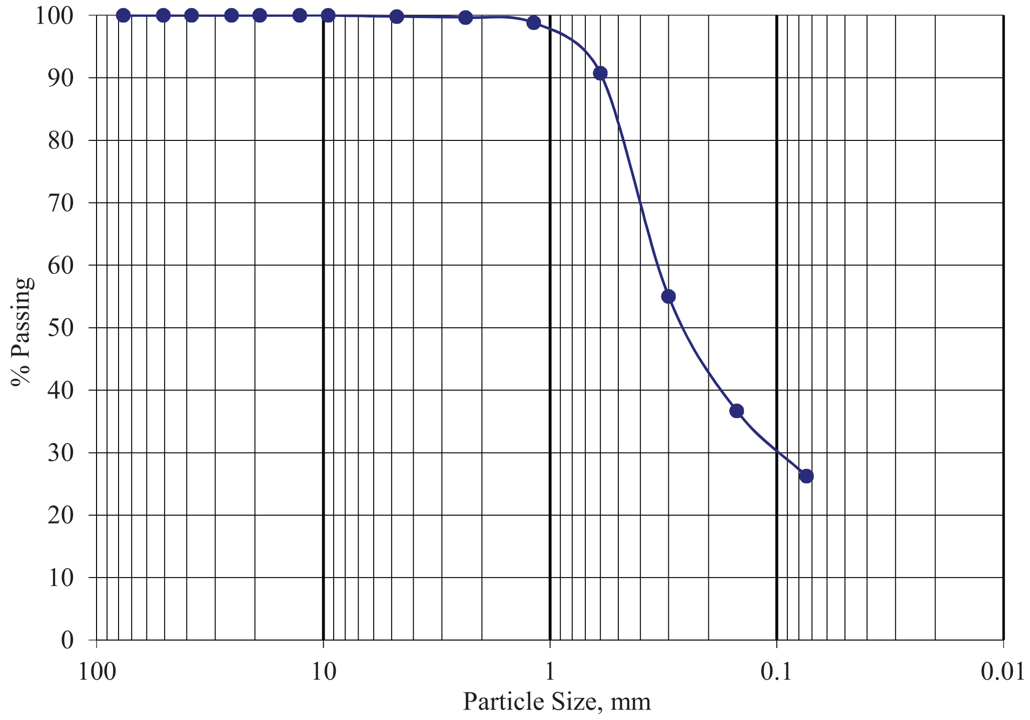
EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
130+	Very High

SIEVE ANALYSIS

ASTM C-136

Job Name: 303277-001
Sample ID: B 2 @ 0-5'
Description: SM

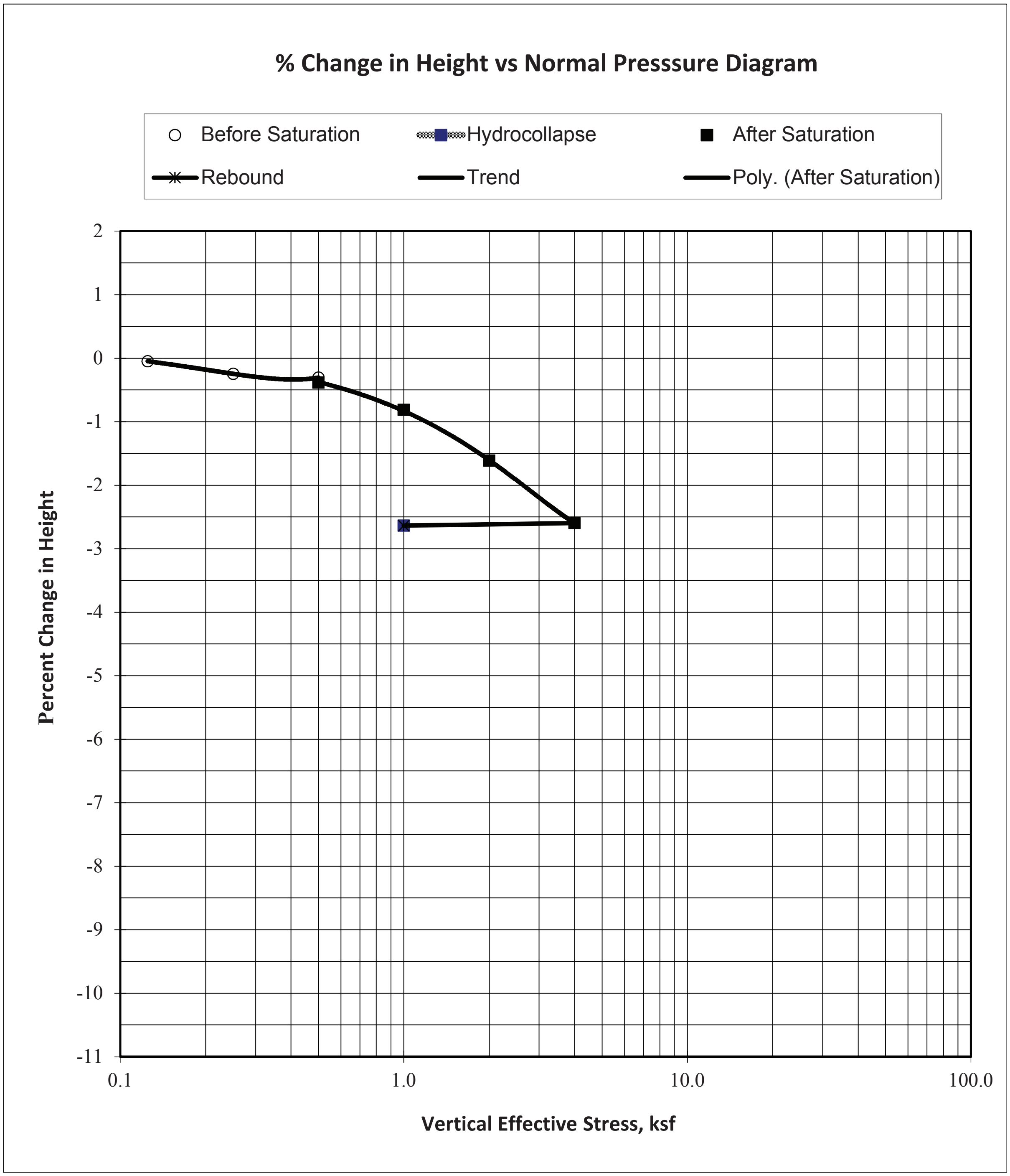
Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	100
#4	100
#8	100
#16	99
#30	91
#50	55
#100	37
#200	26



CONSOLIDATION TEST

ASTM D 2435-90

Hueneme High School Synthetic Turf Field		Initial Dry Density:	104.1 pcf
B 3 @ 2.5'		Initial Moisture, %:	10.1%
Silty Sand		Specific Gravity:	2.67 (assume
Ring Sample		Initial Void Ratio:	0.601

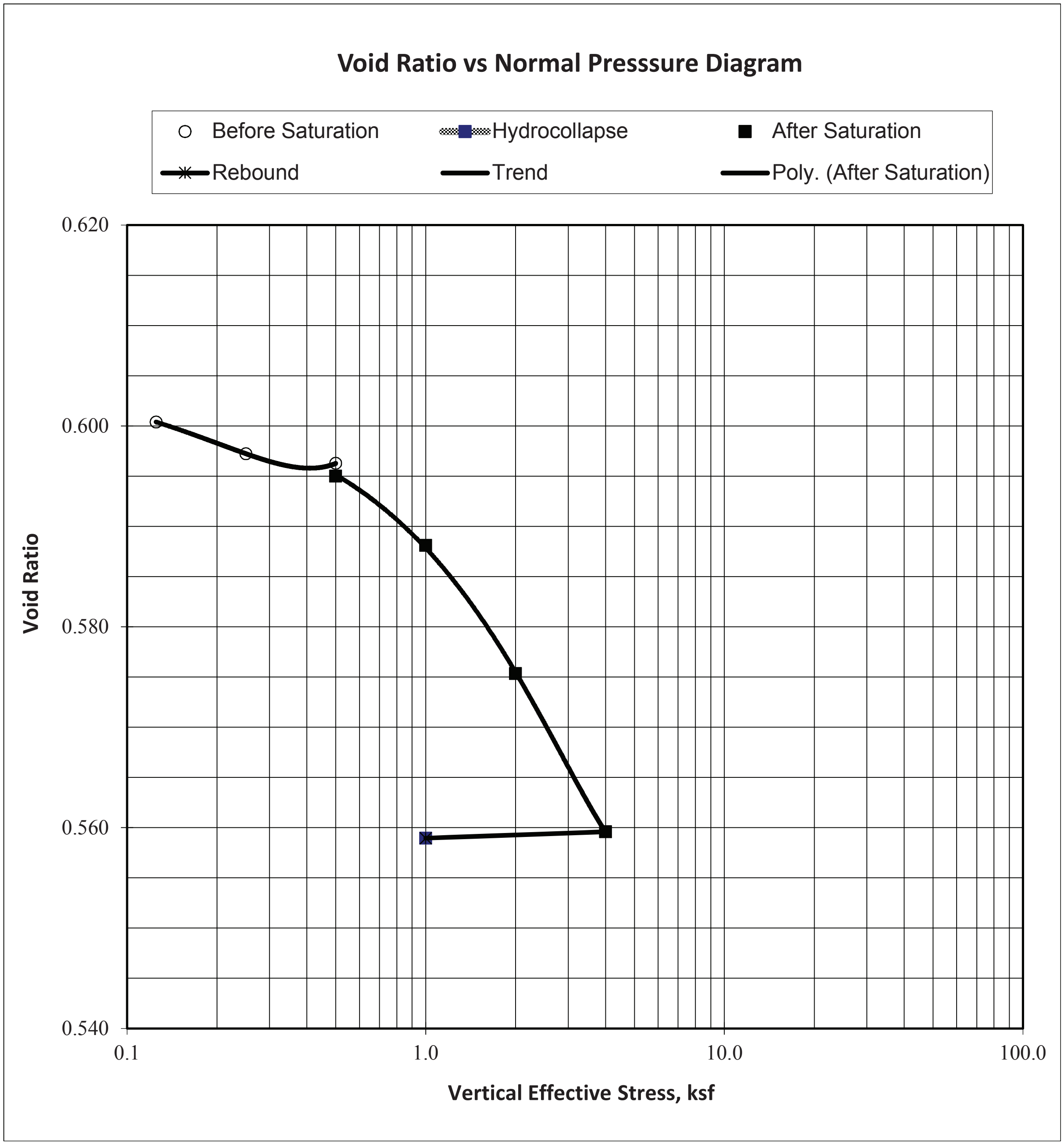


CONSOLIDATION TEST

ASTM D 2435-90

Hueneme High School Synthetic Turf Field
B 3 @ 2.5'
Silty Sand
Ring Sample

Initial Dry Density:	104.1
Initial Moisture, %:	10.1
Specific Gravity:	2.67 (assume
Initial Void Ratio:	0.601



Hueneme High School
Synthetic Turf Field

303277-001

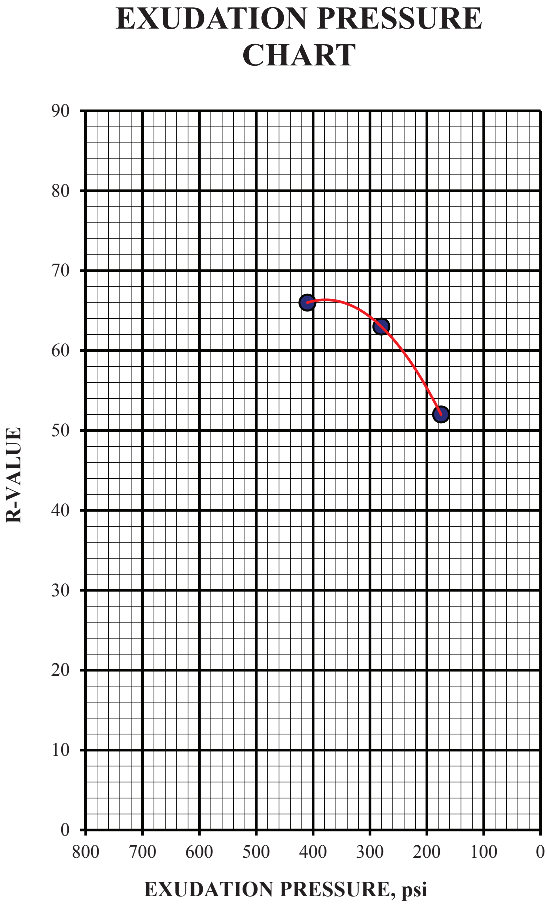
RESISTANCE '*R* ' VALUE AND EXPANSION PRESSURE

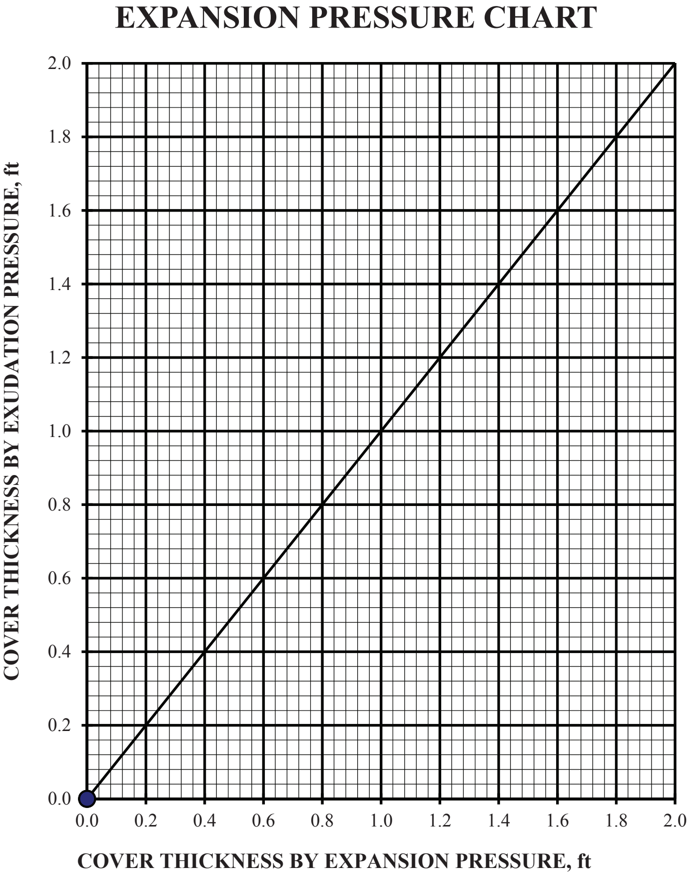
ASTM D 2844/D2844M-13

July 31, 2019

Boring #2 @ 0.0 - 5.0'
Brown Poorly Graded Sand (SP)

Dry Density @ 300 psi Exudation Pressure: 121.7-pcf
%Moisture @ 300 psi Exudation Pressure: 12.9%
R-Value - Exudation Pressure: 64
R-Value - Expansion Pressure: N/A
R-Value @ Equilibrium: 64





CERTIFICATE OF ANALYSIS

Client: Earth Systems Pacific
CAS LAB NO: 191289-01
Sample ID: B3@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)	8.2	S.U.	1	---	9045	07/24/19
Resistivity*	2100	Ohms-cm	1	---	SM 120.1M	07/24/19
Chloride	12	mg/Kg	1	0.3	300.0M	07/24/19
Sulfate	530	mg/Kg	1	0.6	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)

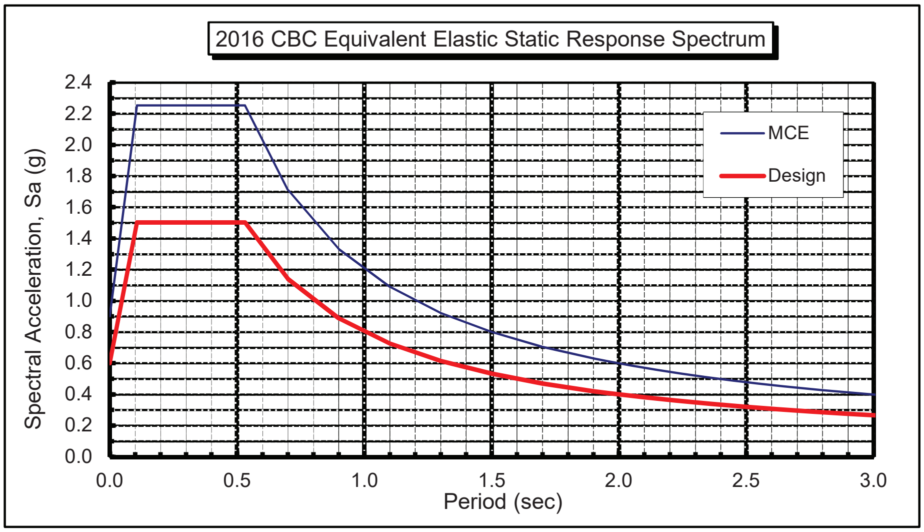
WEIGHTED EXPANSION INDEX	FOUNDATIONS FOR SLAB AND RAISED FLOOR SYSTEM (4) (5)							CONCRETE SLABS		**PREMOISTENING OF SOILS UNDER FOOTINGS, PIERS AND SLABS (1)**	**RESTRICTIONS ON PIERS UNDER RAISED FLOORS** A design by a registered structural engineer may be excepted when approved by the Building Official
	NUMBER OF FLOORS	STEM THICKNESS	FOOTING WIDTH	FOOTING THICKNESS	ALL PERIMETER FOOTINGS (5)	INTERIOR FOOTINGS FOR SLAB AND RAISED FLOORS (5)	REINFORCEMENT FOR CONTINUOUS FOUNDATIONS (2)	3 ½ " MINIMUM THICKNESS			
					DEPTH BELOW NATURAL SURFACE OF GROUND AND FINISH GRADE (3) (8)			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	10	18	8	24	24					
							#3 BARS @ 24” IN EXT. FOOTING BEND3’ INTO SLAB (7)				
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	10	18	8	24	24					
							#3 BARS @ 24” IN EXT. FOOTING BEND 3’ INTO SLAB (7)				
Above 130 Very High	Special design by licensed engineer/architect										

APPENDIX C

2016 CBC & ASCE 7-10 Seismic Parameters
US Seismic Design Maps
Fault Parameters

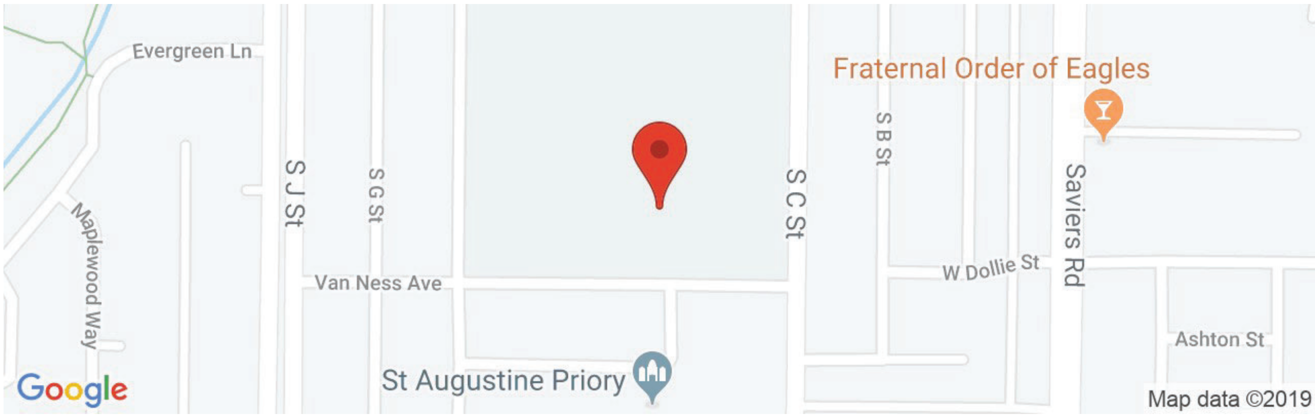
2016 California Building Code (CBC) (ASCE 7-10) Seismic Design Parameters

				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:		34.158	N		
Longitude:		-119.182	W		
Maximum Considered Earthquake (MCE) Ground Motion					
Short Period Spectral Reponse	$\mathbf{S_S}$	**2.254**	**g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	$\mathbf{S_1}$	**0.799**	**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}	2.254	g	$= F_a*S_S$	
	S_{M1}	1.199	g	$= F_v*S_1$	
Design Earthquake Ground Motion					
Short Period Spectral Reponse	$\mathbf{S_{DS}}$	**1.503**	**g**	$= 2/3*S_{MS}$	
1 second Spectral Response	$\mathbf{S_{D1}}$	**0.799**	**g**	$= 2/3*S_{M1}$	
	To	0.11	sec	$= 0.2*S_{D1}/S_{DS}$	
	Ts	0.53	sec	$= S_{D1}/S_{DS}$	
Seismic Importance Factor	I	1.00		Table 1604.5	Table 11.5-1
	F_{PGA}	1.00			



Period T (sec)	Design Sa (g)
0.00	0.601
0.05	1.025
0.11	1.503
0.53	1.503
0.70	1.141
0.90	0.888
1.10	0.726
1.30	0.615
1.50	0.533
1.70	0.470
1.90	0.421
2.10	0.380
2.30	0.347
2.50	0.320
2.70	0.296
2.90	0.276

Latitude, Longitude: 34.1578, -119.1820

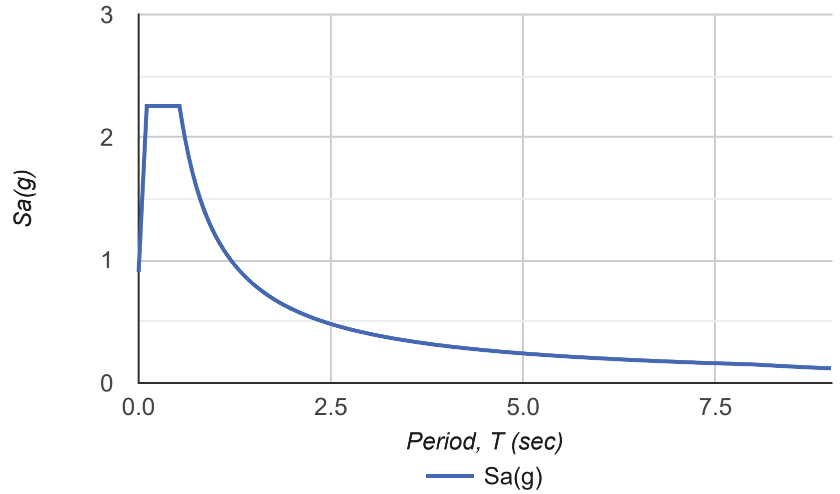


Date	7/5/2019, 11:46:56 AM
Design Code Reference Document	ASCE7-10
Risk Category	I
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.254	MCE_R ground motion. (for 0.2 second period)
S_1	0.799	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.254	Site-modified spectral acceleration value
S_{M1}	1.199	Site-modified spectral acceleration value
S_{DS}	1.502	Numeric seismic design value at 0.2 second SA
S_{D1}	0.799	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	0.844	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.844	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.254	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.418	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.357	Factored deterministic acceleration value. (0.2 second)
S1RT	0.799	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.848	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.802	Factored deterministic acceleration value. (1.0 second)
PGAd	0.872	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.932	Mapped value of the risk coefficient at short periods
C_{R1}	0.943	Mapped value of the risk coefficient at a period of 1 s

MCER Response Spectrum



Design Response Spectrum

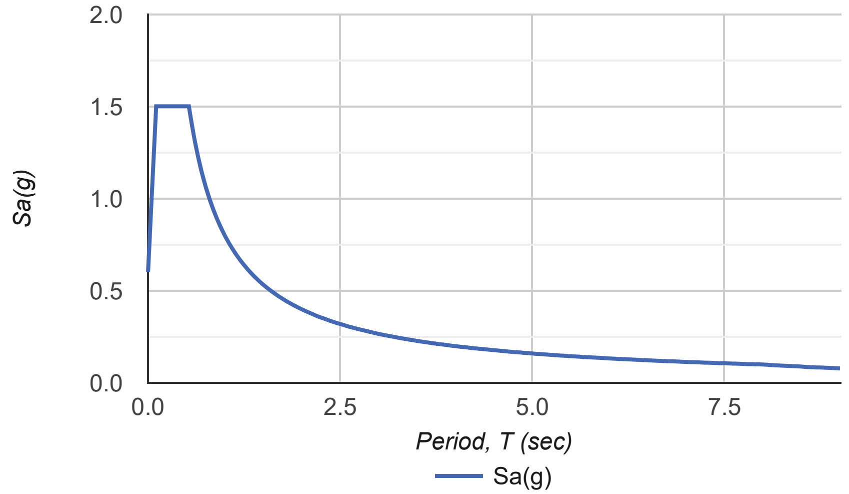


Table 1
Fault Parameters

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)
Simi-Santa Rosa	6.2	10.0	60	346	30	39	B	**6.8**		1
Malibu Coast (Extension), alt 1	6.3	10.1	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	6.3	10.1	74	4	30	35	B'	**6.9**		
Oak Ridge (Onshore)	6.4	10.3	65	159	90	49	B	**7.2**		4
Oak Ridge (Offshore)	8.3	13.4	32	180	90	38	B	**6.9**		3
Ventura-Pitas Point	9.5	15.3	64	353	60	44	B	**6.9**		1
Channel Islands Thrust	10.2	16.5	20	354	90	59	B	**7.3**		1.5
Anacapa-Dume, alt 1	12.8	20.6	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	12.8	20.6	41	352	60	65	B	**7.2**		3
Santa Cruz Island	12.9	20.7	90	188	30	69	B	**7.1**		1
Channel Islands Western Deep Ramp	14.2	22.9	21	204	90	62	B'	**7.3**		
Red Mountain	14.3	23.0	56	2	90	101	B	**7.4**		2
Malibu Coast, alt 1	16.2	26.0	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	16.2	26.0	74	3	30	38	B	**6.9**		0.3
Pitas Point (Lower)-Montalvo	16.9	27.2	16	359	90	30	B	**7.3**		2.5
Sisar	17.5	28.2	29	168	na	20	B'	**7.0**		
North Channel	17.8	28.6	26	10	90	51	B	**6.7**		1
Shelf (Projection)	17.8	28.7	17	21	na	70	B'	**7.8**		
San Cayetano	19.5	31.4	42	3	90	42	B	**7.2**		6
Mission Ridge-Arroyo Parida-Santa Ana	19.7	31.7	70	176	90	69	B	**6.8**		0.4
Santa Cruz Catalina Ridge	20.9	33.7	90	38	na	137	B'	**7.3**		
Santa Monica Bay	24.8	39.9	20	44	na	17	B'	**7.0**		
Pitas Point (Upper)	25.0	40.3	42	15	90	35	B	**6.8**		1
Santa Ynez (East)	25.3	40.8	70	172	0	68	B	**7.2**		2
San Pedro Basin	26.6	42.8	88	51	na	69	B'	**7.0**		
Santa Susana, alt 1	27.5	44.2	55	9	90	27	B	**6.8**		5
Santa Susana, alt 2	27.7	44.6	53	10	90	43	B'	**6.8**		
Northridge Hills	28.9	46.6	31	19	90	25	B'	**7.0**		
Oak Ridge (Offshore), west extension	29.0	46.7	67	195	na	28	B'	**6.1**		
Pine Mtn	29.1	46.8	45	5	na	62	B'	**7.3**		
Del Valle	30.8	49.6	73	195	90	9	B'	**6.3**		
Holser, alt 1	31.2	50.2	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	31.2	50.2	58	182	90	17	B'	**6.7**		
Northridge	32.2	51.9	35	201	90	33	B	**6.8**		1.5
Compton	33.6	54.1	20	34	90	65	B'	**7.5**		
San Pedro Escarpment	34.1	54.9	17	38	na	27	B'	**7.3**		
Pitas Point (Lower, West)	34.3	55.2	13	3	90	35	B	**7.2**		2.5
Santa Ynez (West)	35.0	56.3	70	182	0	63	B	**6.9**		2
Big Pine (Central)	36.6	59.0	76	167	na	23	B'	**6.3**		
Santa Monica, alt 1	36.9	59.4	75	343	30	14	B	**6.5**		1

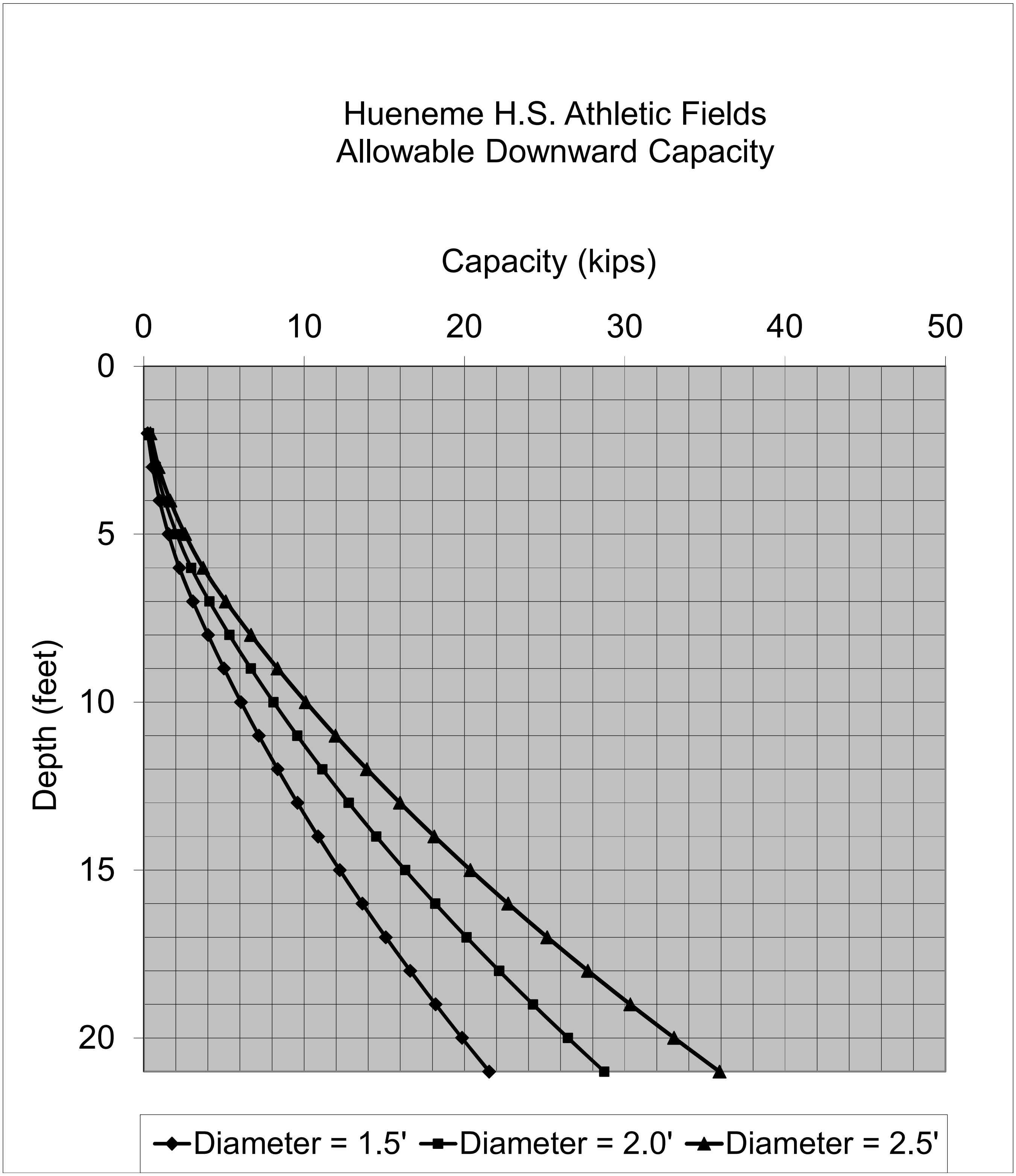
Reference: USGS OFR 2007-1437 (CGS SP 203)

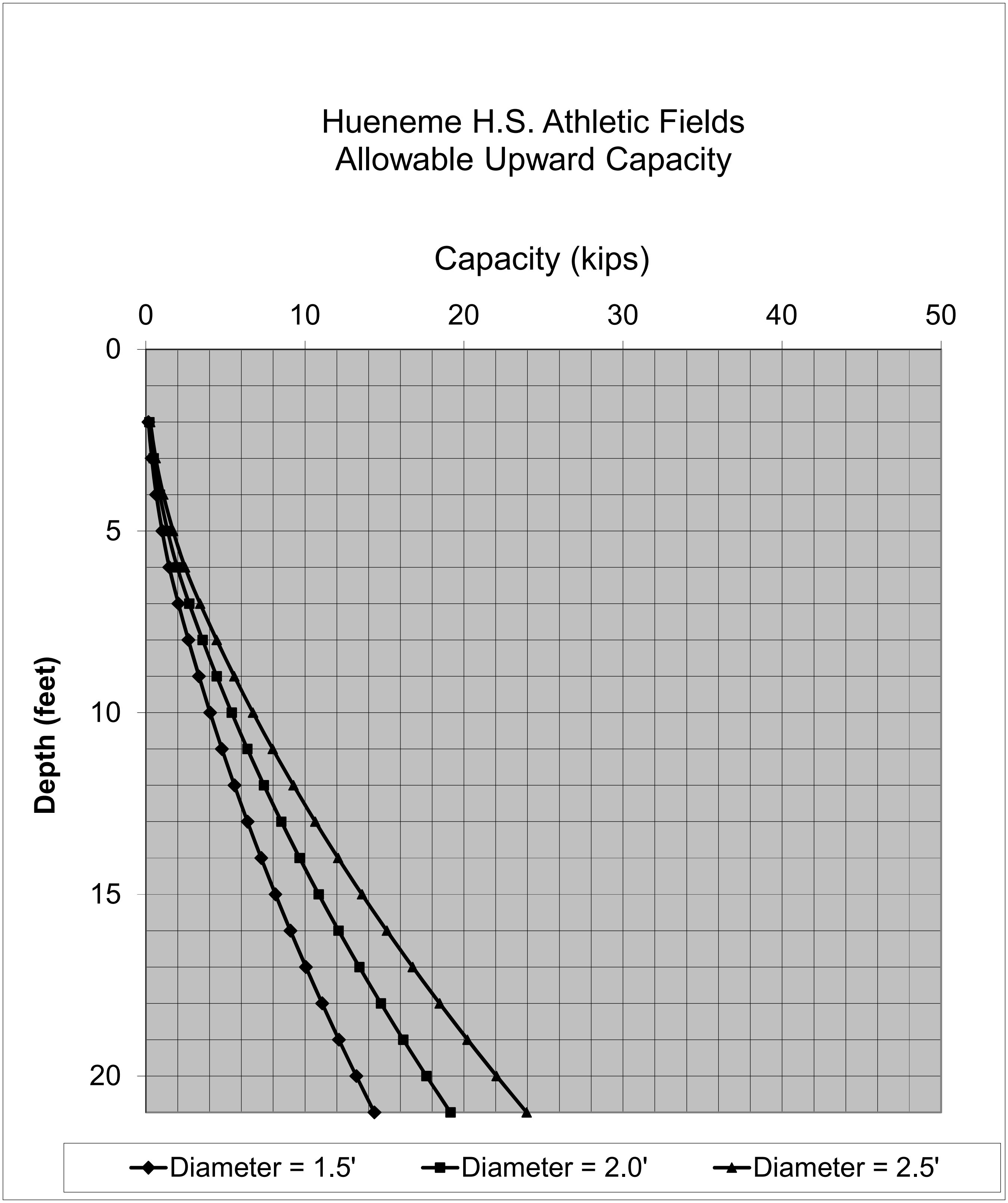
Based on Site Coordinates of 34.1578 Latitude, -119.182 Longitude

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

Pile Capacity Graphs


Hueneme H.S. Athletic Fields
Allowable Downward Capacity
Capacity (kips)
0
10
20
30
40
50
Depth (feet)
0
5
10
15
20
Diameter = 1.5'
Diameter = 2.0'
Diameter = 2.5'


Hueneme H.S. Athletic Fields
Allowable Upward Capacity
Capacity (kips)
0
10
20
30
40
50
Depth (feet)
0
5
10
15
20
Diameter = 1.5'
Diameter = 2.0'
Diameter = 2.5'