GEOTECHNICAL ENGINEERING REPORT

FOR

PROPOSED IMPROVEMENTS TO ATHLETIC FIELDS AT

OXNARD HIGH SCHOOL,

3400 WEST GONZALES ROAD

OXNARD, CALIFORNIA

PROJECT NO.: 303278-001

AUGUST 27, 2019

PREPARED FOR

OXNARD UNION HIGH SCHOOL DISTRICT

BY

EARTH SYSTEMS PACIFIC

1731-A WALTER STREET

VENTURA, CALIFORNIA

Earth Systems

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

August 27, 2019

Project No.: 303278-001
Report No.: 19-8-7

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

Project: Improvements to Athletic Field Surfaces
Oxnard High School
3400 West Gonzales Road
Oxnard, California

As authorized, we have performed a geotechnical study for proposed improvements to the athletic field surfaces at Oxnard High School in the City of Oxnard, California. The accompanying Geotechnical Engineering Report presents the results of our subsurface exploration and laboratory testing programs, as well as our conclusions and recommendations pertaining to geotechnical aspects of project design. This report completes the scope of services described within our Proposal No. VEN-19-05-016 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,

EARTH SYSTEMS PACIFIC

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

Patrick V. Boales 8-27-19
Engineering Geologist

Anthony P. Mazzei
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
ANTHONY P. MAZZEI
GE 2823
Exp. 6-30-21
GEOTECHNICAL
STATE OF CALIFORNIA

8/27/19

Copies:
2 - Oxnard Union High School District (1 via US mail, 1 via email)
1 - LuEllen Benjamins, Little Diversified Architectural Consulting (via email)
1 - Barsin Bet Govargez, Little Diversified Architectural Consulting (via email)
1 - Project File

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 Oxnard 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 adjacent to the baseball field, and two ticket booths with attached entry gates and plaza concrete paving at the northwest and southeast entrances to the football field. New concrete sidewalks will be constructed around the east end of the track/football field, and around the south and west ends of the baseball, softball, and soccer fields. New asphalt service roads will be added along the southern campus boundary and between the softball field and the football field. The existing tennis courts, which currently have cracked asphalt paving, will be replaced with sport surfacing. 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 booths are expected to range from 50 to 70 square feet, and to have attached 10-foot tall entry gates supported by steel tube columns on pier footings. The one-story ticket booths will be constructed with reinforced CMU block, and will utilize conventional foundation systems with slab-on-grade floors. There will be 8-foot high freestanding reinforced CMU walls adjacent to the ticket booths at the entry gates.

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 7 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 storm water infiltration feasibility.

GENERAL GEOLOGY

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

Although there are several faults located within the region, the nearest known fault of significant activity the Oak Ridge Fault is located approximately 2.3 miles north of the subject site. The project area is not located within any of the “Fault Rupture Hazard Zones” that have been specified by the State of California (CDMG. 1972, Revised 1999).

The site is underlain by alluvial sediments consisting of loose to very dense silty sands to sandy silts, fine to coarse sands, clayey sands, and firm to stiff clayey silt.

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

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

SEISMICITY AND SEISMIC DESIGN

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

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 (34.2157˚ North Latitude and -119.2135° 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	2.510g
Spectral Response Acceleration at 1 sec. – S_1	0.935g
Site Coefficient – F_a	1.00
Site Coefficient – F_v	1.50
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.510g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.403g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.673g
One Second Spectral Response – S_{D1}	0.935g
Site Modified Peak Ground Acceleration - PGA_M	0.978g
Values appropriate for a 2% probability of exceedance in 50 years	

SOIL CONDITIONS

Evaluation of the subsurface indicates that soils are generally alluvium that consists of loose to very dense silty sands to sandy silts, fine to coarse sands, clayey sands, and firm to stiff clayey silt. Near-surface soils encountered in Borings B-1, B-2, B-5, and B-7 are characterized by high blow counts and in-place densities, and low compressibilities. Near-surface soils encountered in Borings B-3, B-4, and B-6 are characterized by moderate blow counts and in-place densities. Testing indicates that anticipated bearing soils lie in the "medium" expansion range because the expansion index equals 65. [A version of this classification of soil expansion, Table 18-I-D, is included in Appendix B of this report.] It appears that soils can be cut by normal grading equipment.

Groundwater was encountered at a depth of approximately 27 feet below existing site grades in Boring B-5. However, mapping of historically high groundwater levels by the California Geological Survey (CGS, 2002a) indicates that groundwater has risen in the past to about 10 below the ground surface near the subject site. Furthermore, borings advanced by Earth Systems Southern California in March 2007 encountered water at depths as shallow as 6 to 7 feet below the baseball field area, which appears to be about the lowest point of the campus.

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

Samples of near-surface soils were tested for pH, resistivity, soluble sulfates, and soluble chlorides. The test results provided in Appendix B should be distributed to the design team for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils. It should be noted that sulfate contents (1,300 mg/Kg) are in the "S1" ("moderate") exposure class of Table 19.3.1.1 of ACI 318-14; therefore, it appears that special concrete designs will be necessary for the measured sulfate contents. The typical concrete would be Type II with a maximum water to cement ratio of 0.5 and a minimum unconfined compressive strength of 4,000 psi.

Based on criteria established by the County of Los Angeles (2013), measurements of resistivity of near-surface soils (820 ohms-cm) indicate that they are "severely corrosive" to ferrous metal (i.e. cast iron, etc.) pipes.

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.

Given that the historically shallowest depths to groundwater are less than 10 feet, it appears that devices to infiltrate stormwater into the subsurface would not be able to maintain current vertical setback regulations, and for that reason would not be feasible.

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.

Because the subgrade soils underlying the panel drains are expected to be fine-grained and expansive, an impermeable liner should be installed over the entire drain layout before placing the drains.

Once 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.

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.

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% based on an anticipated average compaction of 92%. 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% 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% 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 and fences that are 18 feet high will also be necessary. Soils should be overexcavated to a depth of 4.5 feet below finished subgrade elevation, and to a distance of 3 feet on either side of the footing edges. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted to at least 90% of the maximum dry density.

Areas outside of the building area to receive fill, exterior slabs-on-grade, sidewalks, or paving should be overexcavated to a depth of 1.5 feet below finished subgrade elevation. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted. Because the expansion index of on-site soils is in the “medium” range, 4 inches of compacted aggregate base should be used 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% of the maximum dry density obtainable by the ASTM D 1557 test method.

The upper one foot of subgrade below areas to be paved should be compacted to a minimum of 95% of the maximum dry density.

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

If pumping soils or otherwise unstable soils are encountered during the overexcavation, stabilization of the excavation bottom will be required prior to placing fill. This can be accomplished by various means. The first method would include drying the soils as much as possible through scarification, and working thin lifts of “6-inch minus” crushed angular rock into the excavation bottom with small equipment (such as a D-4) until stabilization is achieved. Use of a geotextile fabric such as Mirafi 500X, or Tensar TX-160, or an approved equivalent, is another possible means of stabilizing the bottom. If this material is used, it should be laid on the excavation bottom and covered with approximately 12 inches of “3-inch minus” crushed angular rock prior to placement of filter fabric (until the bottom is stabilized). The rock should then be covered with a geotextile filter fabric before placing fill above. It is anticipated that stabilization will probably be necessary due to the existing high moistures of the soils, and due to the shallow groundwater depth. Unit prices should be obtained from the Contractor in advance for this work.

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

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

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

It is understood that the tennis courts may be resurfaced as part of this project. Records of grading from 1993 indicate that the tennis courts were originally overexcavated to a minimum depth of one foot below original grade. The resulting surface was scarified to a depth of one foot, moisture conditioned, and recompacted prior to replacing the overexcavated soils as compacted fill, and bringing the pad up to finished subgrade elevation. Recent exploration indicates that the tennis court surface consists of approximately 5.5 inches of asphalt supported by 3 inches of aggregate base on compacted silty sand. However, because the existing tennis courts will be demolished, which will disturb near-surface soils, the grading recommendations provided above for flatwork should be performed within the new tennis court facility.

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 Footings

Conventional continuous footings and/or isolated pad footings may be used to support structures. For one-story buildings, perimeter footings should have a minimum depth of 21 inches, and interior footings should have a minimum depth 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 that are 21 inches deep and 12 inches wide may be designed based on an allowable bearing value of 1,800 psf. This value has a factor of safety of greater than 3.

Isolated pad footings with an assumed size of 24 inches by 24 inches by 12 inches deep may be designed based on an allowable bearing value of 2,000 psf. This value has a factor of safety of greater than 3.

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

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 0.58 may be applied to dead load forces. This value does not include a factor of safety.

Passive resistance acting on the sides of foundation stems equal to 340 pcf of equivalent fluid weight may be included for resistance to lateral load. This value does not include a factor of safety.

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

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

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

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

Drilled Pier Foundations

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

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

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

For vertical (axial compression) and uplift capacity, the attached pile capacity graphs may be used. Drilled pier diameters of 1.5, 2.0, and 2.5 feet were analyzed, and the results are presented on the attached charts. Side resistance is not allowed to increase beyond a depth equal to 20 pile diameters. Upward resistance is taken as two-thirds of the downward resistance. The downward and upward capacity graphs for drilled piers are presented in Appendix 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 340 psf per foot of penetration in the compacted fill (upper 5 feet) and an EFW of 265 pcf in the firm native soils above the groundwater table may be used for lateral load design. An EFW of 170 pcf may be used for lateral load design in the firm native soils below the groundwater table. These resisting pressures are ultimate values. The maximum passive pressure used for design should not exceed 3,300 psf. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended).

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

Assuming 18-inch diameter piers of reinforced concrete that are fixed against rotation at the head, the "point of fixity" was estimated to be located at least 6 feet below the final ground elevation based on commonly accepted engineering procedures (Lee, 1968). If 24-inch diameter piers are used, the "point of fixity" was estimated to be located at least 7.5 feet below the final ground elevation. If 30-inch diameter piers are used, the "point of fixity" was estimated to be located at least 9 feet below the final ground elevation.

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

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

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

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

Slabs-on-Grade

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

It is recommended that perimeter slabs (sidewalks, plaza pavements, etc.) be designed relatively independent of footing stems (i.e. free floating) so foundation adjustment will be less likely to cause cracking. Because near-surface soils are in the "medium" expansion range, sidewalks and plaza pavements should be underlaid with 4 inches of aggregate base materials compacted to a minimum of 95% of the maximum dry density obtainable by the ASTM D 1557 test method. Current plans call for 4-inch thick concrete reinforced with No. 3 bars on 18-inch centers. These specifications are considered appropriate for the soil conditions. (Note that structural paving sections for areas to be exposed to vehicular traffic are presented elsewhere in this report.)

Interior slab designs should be provided by the Structural Engineer, but the reinforcement and slab thicknesses should not be less than the criteria set forth in Table 18-I-D for the "medium" expansion range.

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

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

Retaining Walls

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

Conventional cantilever retaining walls that are backfilled at a 1:1 projection upward from the heels of the wall footings with crushed rock or non-expansive sand may be designed for active pressures of 30 pcf of equivalent fluid weight for well-drained, level backfill. An 18-inch thick cap of compacted native soils should be placed above the rock or sand. Filter fabric should be placed between the rock or sand and native soils and/or backfill over the top.

Restrained retaining walls that are backfilled at a 1:1 projection upward from the heels of the wall footings with crushed rock or non-expansive sand may be designed for at-rest pressures of 52 pcf of equivalent fluid weight for well-drained, level backfill. An 18-inch thick cap of compacted native soils should be placed above the rock or sand. Filter fabric should be placed between the rock or sand and native soils and/or backfill over the top.

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.

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

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 24, paving sections should have a minimum gravel equivalent of 1.22 feet. This can be achieved by using 3 inches of asphaltic concrete on 6.5 inches of Processed Miscellaneous Base (PMB) compacted to a minimum of 95% of the maximum dry density on subgrade soils compacted to a minimum of 95% 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 1.58 feet. This can be achieved by using 4 inches of asphaltic concrete on 9.5 inches of Processed Miscellaneous Base (PMB) compacted to a minimum of 95% of the maximum dry density on subgrade soils compacted to a minimum of 95% 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 24 (approximately equivalent to a coefficient of subgrade reaction of k = 140 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 fire lanes and other heavy traffic areas), the following minimum unreinforced paving section was determined:

1.	Concrete thickness =	6 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=	15 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 Consultants Southern California, June 25, 1992, Geotechnical Engineering Report for the New Oxnard High School, Gonzales Road between Patterson Road and Victoria Avenue, Oxnard, California (Job No. B-19531-V1).

Earth Systems Consultants Southern California, July 27, 1993, Interim Grading Report for the New Oxnard High School, Gonzales Road between Patterson Road and Victoria Avenue, Oxnard, California (Job No. B-19531-V1).

Earth Systems Consultants Southern California, March 5, 1998, Re-Evaluation of Groundwater Levels, and New Recommendations, Oxnard High School Pool Facility, Oxnard, California (Job No. SS-19531-V9).

Earth Systems Southern California, March 2, 2007, Exploratory Borings for Oxnard H.S. Baseball Scoreboard, Oxnard, California (Job No. VT-19531-11).

Earth Systems Southern California, July 31, 2007, Engineering Geology and Geotechnical Engineering Report for Proposed Addition to Building K at Oxnard High School, Oxnard, California (Job No. VT-19531-12).

GENERAL BIBLIOGRAPHY

American Concrete Institute (ACI), 2009, ACI 318-14.

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.

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.

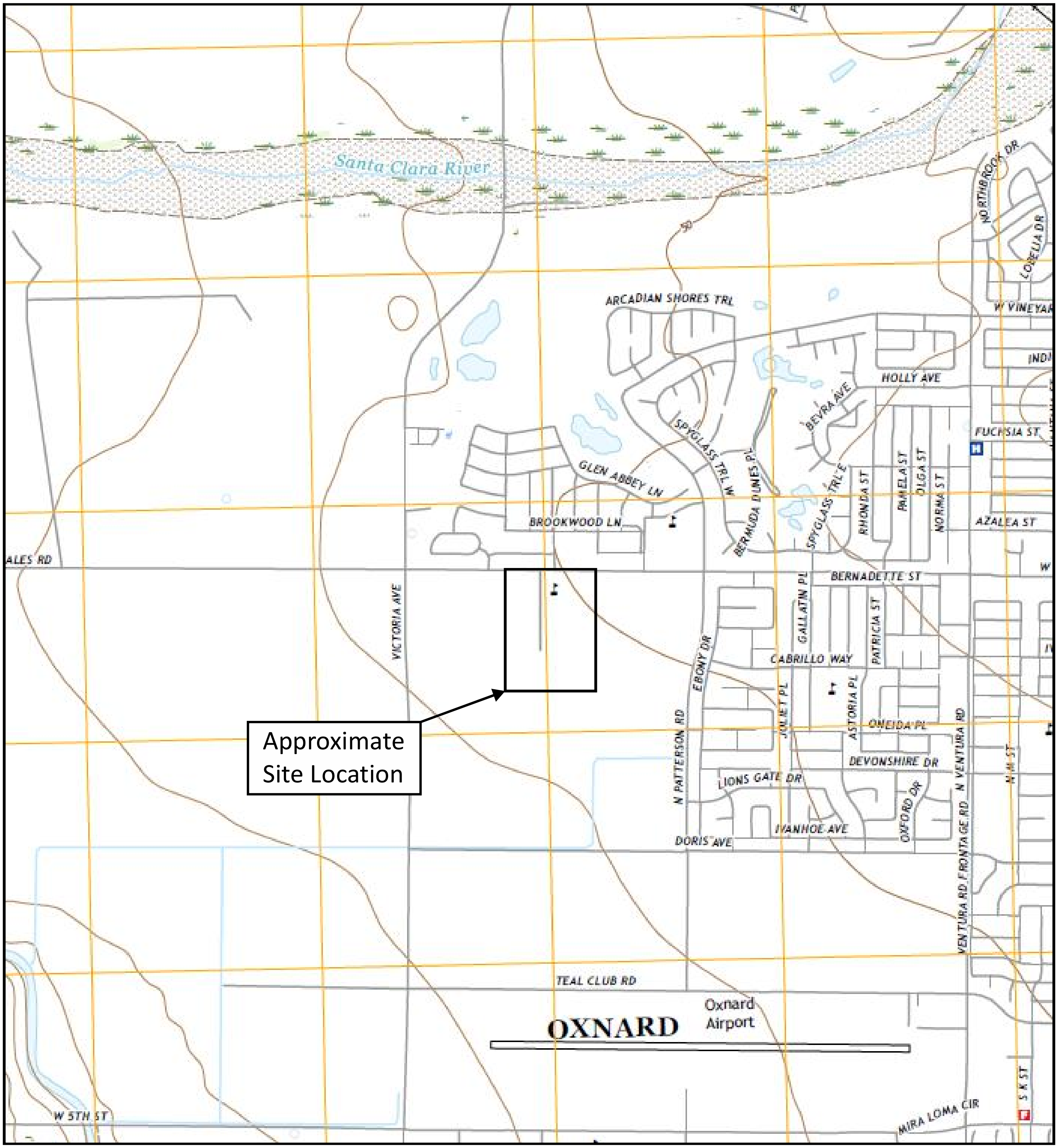
Ventura County Planning Department, October 22, 2013, Ventura County General Plan Hazards Appendix.

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

Vicinity Map
Regional Geologic Map
Seismic Hazard Zones Map
Historically Shallowest Groundwater Map
Field Study
Site Plan
Logs of Exploratory Borings
Boring Log Symbols
Unified Soil Classification System



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

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

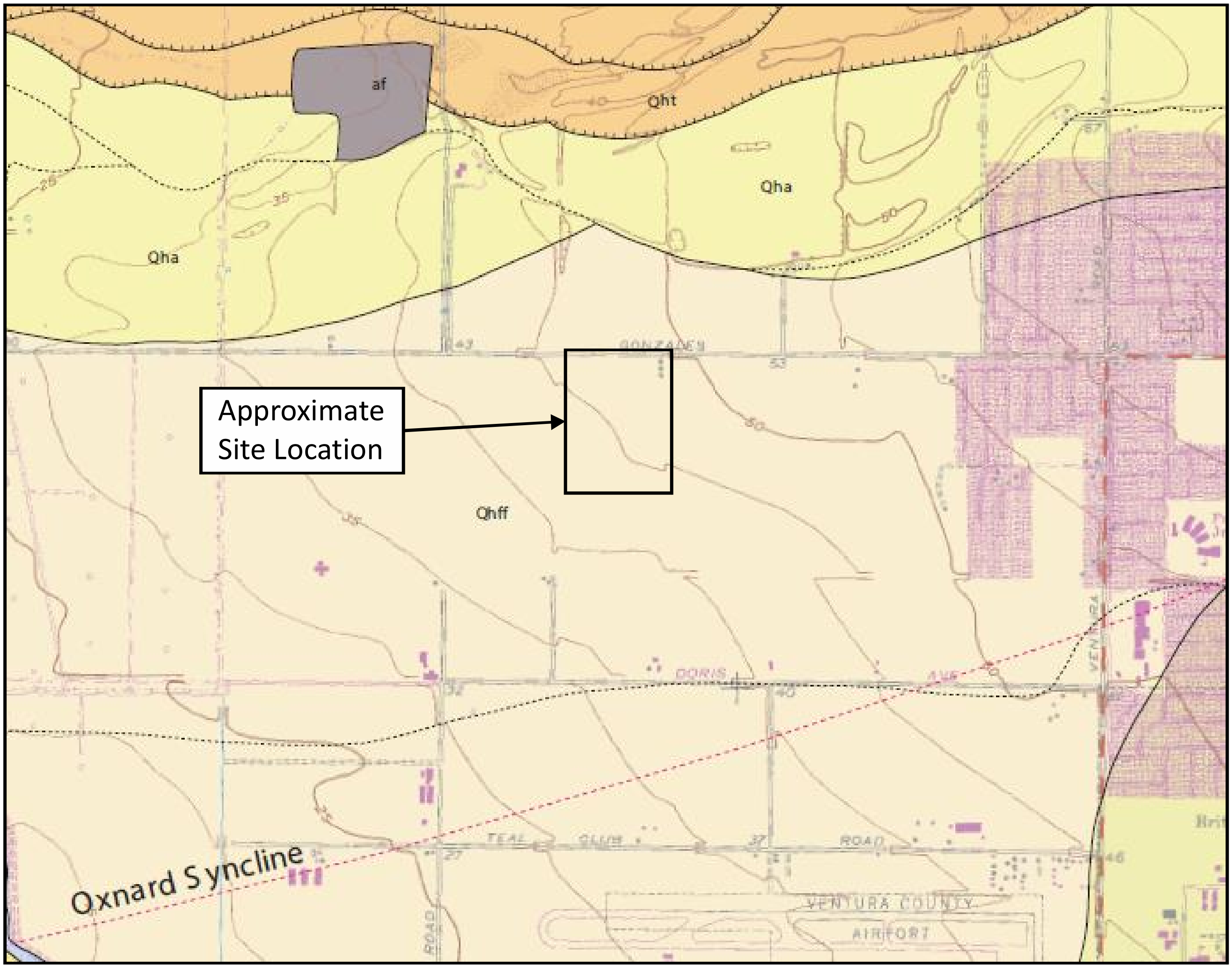
N

VICINITY MAP

Oxnard High School Synthetic Field
Oxnard, California

Earth Systems

August 2019	303278-001



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

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

N

MAP SYMBOLS

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

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

Contact between similar map units; generally approximately located.

Fault; dotted where concealed.

Axis of anticline; dotted where concealed.

Axis of syncline; dotted where concealed.

Qhff: Holocene alluvial fan deposits

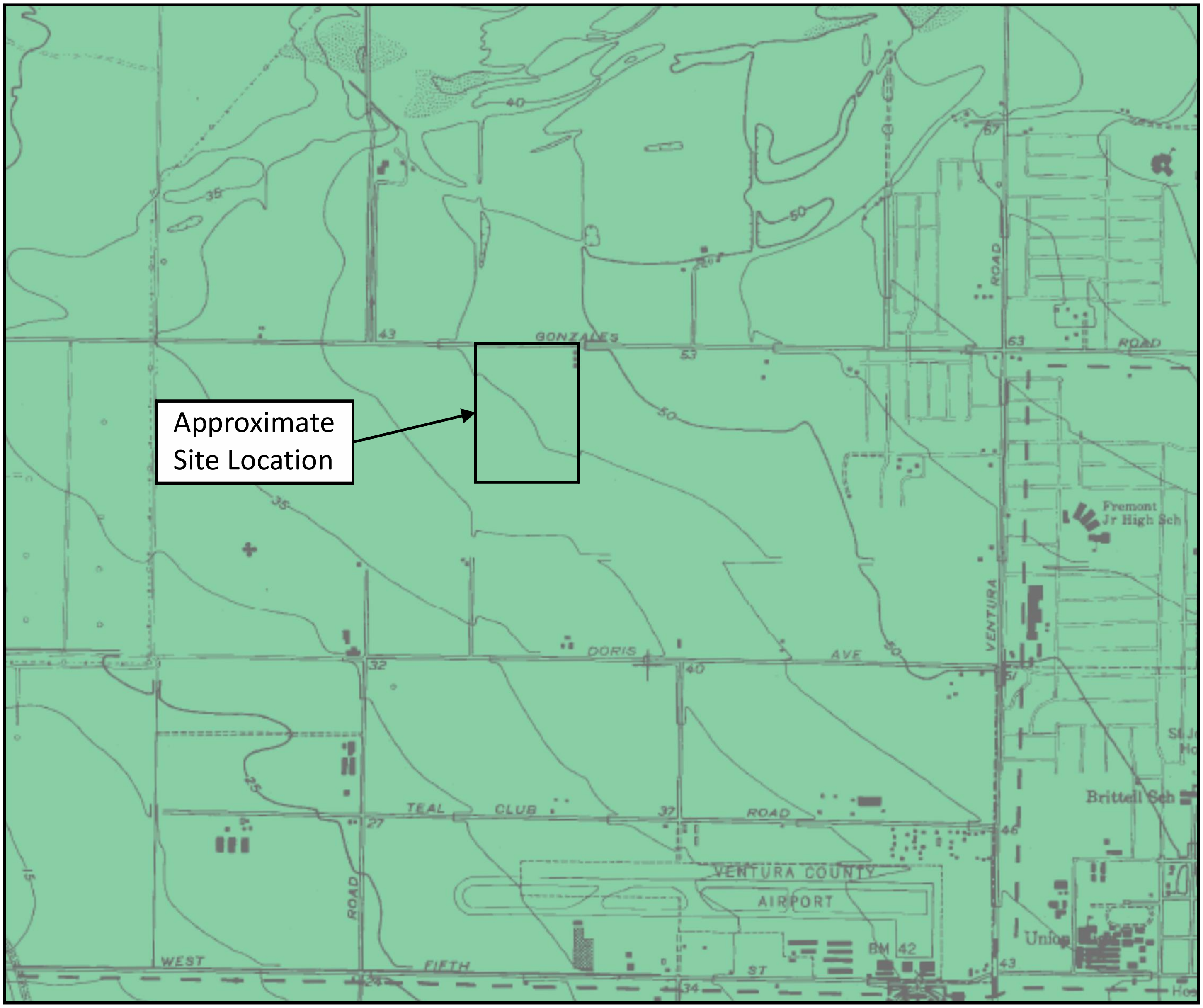
Qha: Latest Holocene alluvial deposits

REGIONAL GEOLOGIC MAP

Oxnard High School Synthetic Field
Oxnard, California

Earth Systems

August 2019	303278-001



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

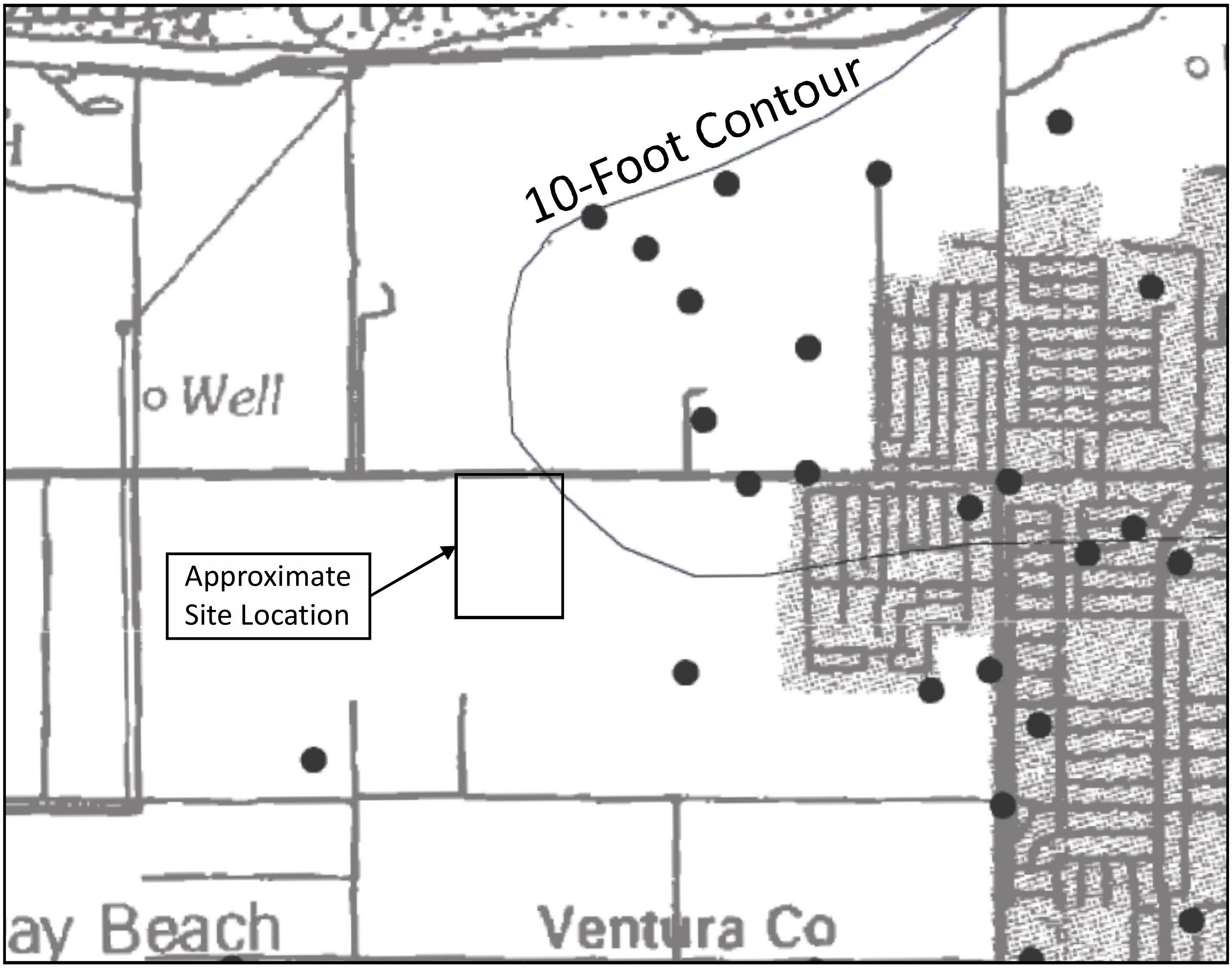
N

SEISMIC HAZARD ZONES MAP

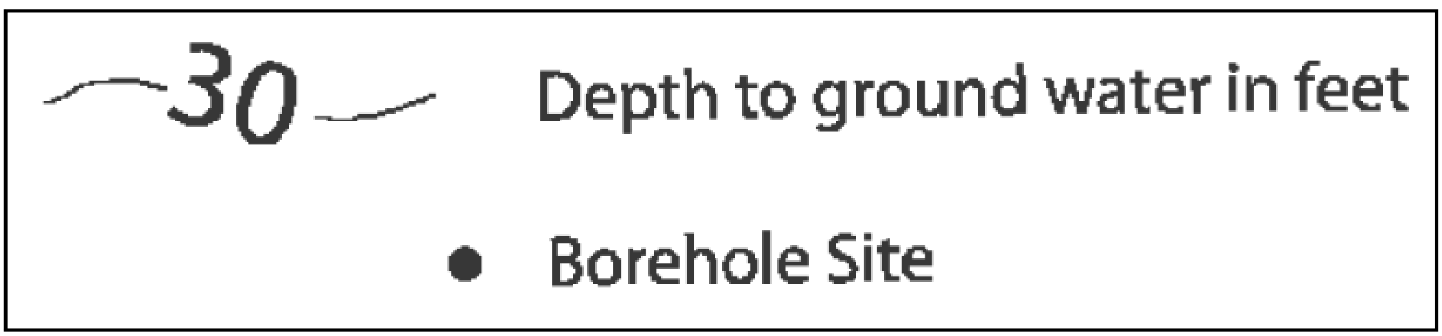
Oxnard High School Synthetic Field
Oxnard, California

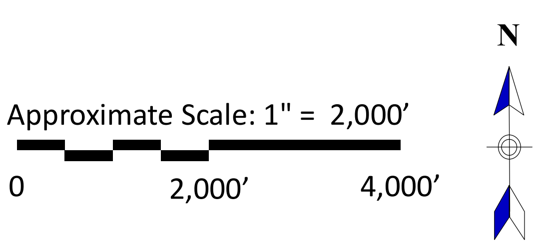
Earth Systems

August 2019	303278-001



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





HISTORICAL HIGH GROUNDWATER MAP

Oxnard High School Synthetic Field
Oxnard, California

Earth Systems

August 2019 | 303278-001

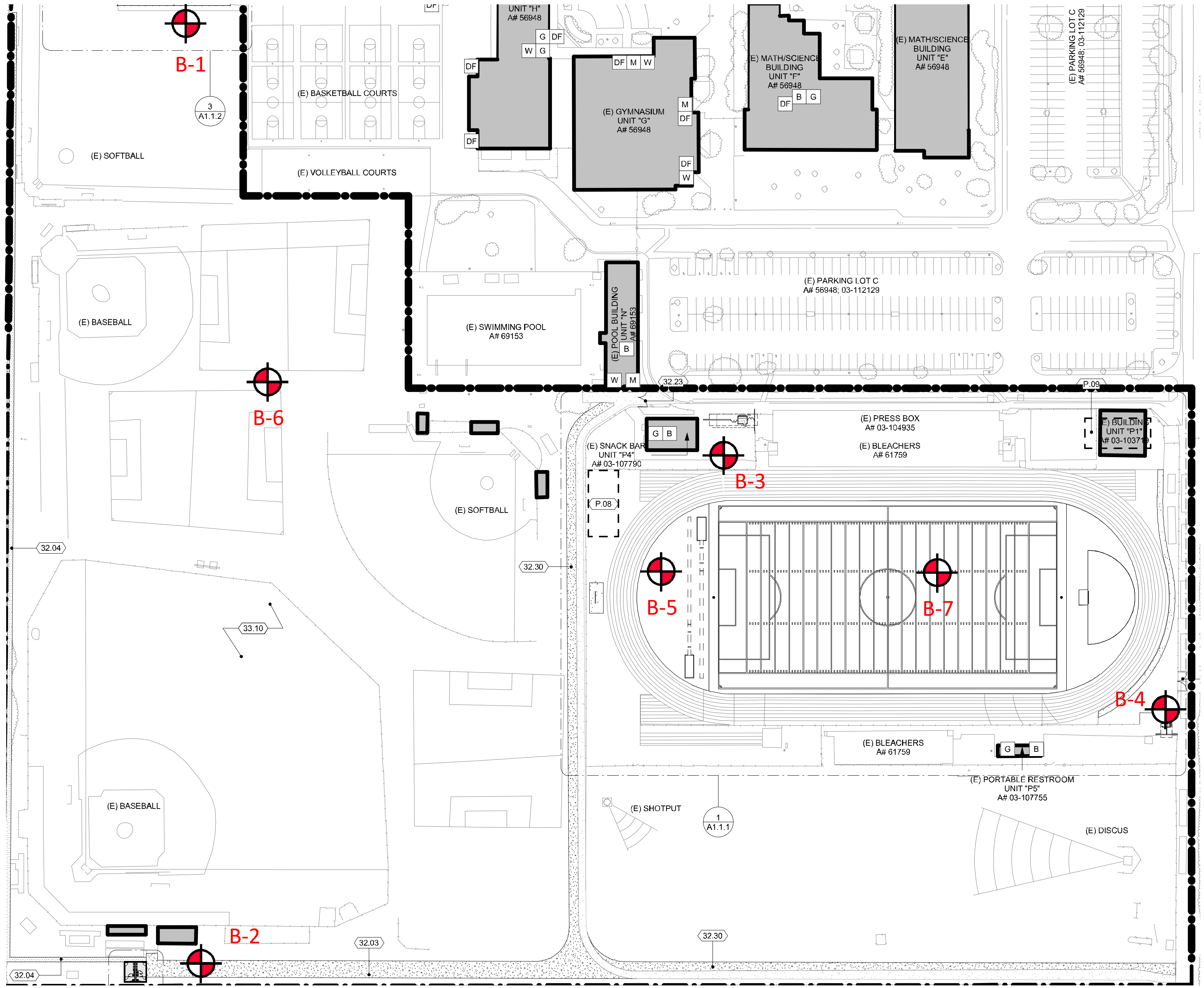
FIELD STUDY

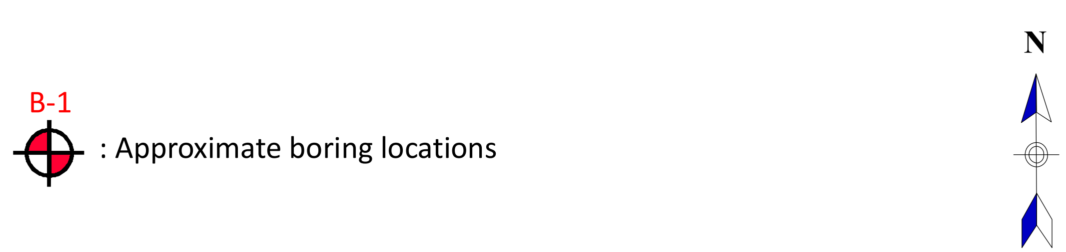
A. Seven borings were drilled to a maximum depth of 30 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory analysis. The borings were drilled on June 28, 2019, using an 8-inch diameter hollow stem auger powered by a track-mounted CME-75 drilling rig. The approximate locations of the test borings were determined in the field by pacing and sighting, and are shown on the Site Plan in this Appendix.

B. 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-6 and B-7 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.





: Approximate boring locations

Approximate Scale: 1" = 120'

0 120' 240'

SITE PLAN

Oxnard High School
Oxnard, California

Earth Systems

August 2019 | 303278-001

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

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

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

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
			X	20/25/21		SM / ML	121.2	12.4	**ALLUVIUM:** Gray Brown Silty fine Sand to fine Sandy Silt, dense, damp
5			X	5/6/7		SM	113.6	12.5	**ALLUVIUM:** Light Gray Brown Silty fine Sand, loose, damp
			X	6/11/15		SW			**ALLUVIUM:** Light Gray fine to medium Sand, trace coarse Sand, medium dense, dry to damp
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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

Earth Systems

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

BORING NO: B-2
PROJECT NAME: Oxnard HS Synthetic Field
PROJECT NUMBER: 303278-001
BORING LOCATION: Per Plan

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

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
			■	16/28/37		SM / ML	121.5	8.1	**ALLUVIUM:** Light Brown Silty fine Sand to fine Sandy Silt, trace Clay, trace calcareous veins, very dense, damp
5			■	8/16/23		SW	108.0	2.0	**ALLUVIUM:** Light Yellow Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, dense, dry to damp
			☒	10/14/14		SW			**ALLUVIUM:** Light Yellow Brown fine to coarse Sand, trace fine Gravel, medium dense, dry to damp
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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

Earth Systems

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

BORING NO: B-3	DRILLING DATE: June 28, 2019
PROJECT NAME: Oxnard HS Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303278-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
				12/12/11		SM	111.8	9.8	**ALLUVIUM:** Light Brown Silty fine Sand, trace Clay, medium dense, damp
5				4/4/8		ML-CL	102.1	16.1	**ALLUVIUM:** Dark Gray Brown Clayey Silt, firm to stiff, moist
				6/9/12		SP			**ALLUVIUM:** Light Yellow Brown fine Sand, little medium Sand, medium dense, dry to damp
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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

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

BORING NO: B-4	DRILLING DATE: June 28, 2019
PROJECT NAME: Oxnard HS Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303278-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
			■	6/9/12		ML	105.9	11.9	**ALLUVIUM:** Dark Gray Brown fine Sandy Silt, trace calcareous veins, medium dense, damp
5			■	5/9/20		SM	117.5	9.8	**ALLUVIUM:** Brown Silty fine Sand, trace Clay, medium dense, damp
			■	17/50-6"		SC			**ALLUVIUM:** Light Brown Clayey fine Sand, very dense, damp
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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

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

BORING NO: B-5	DRILLING DATE: June 28, 2019
PROJECT NAME: Oxnard HS Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303278-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
			■	15/28/31		ML	122.7	12.4	**ALLUVIUM:** Dark Gray Brown fine Sandy Silt, trace calcareous veins, medium dense, damp
5			■	12/18/22		SM / ML	120.2	15.1	**ALLUVIUM:** Brown Silty fine Sand to fine Sandy Silt, trace Clay, medium dense, damp
			■	6/11/16			117.7	14.6	
10			■			SP			**ALLUVIUM:** Light Yellow Brown fine Sand, trace medium to coarse Sand, medium dense, dry to damp
15									
20						SW			**ALLUVIUM:** Brown fine to medium Sand, trace coarse Sand, little fine Gravel, medium dense, dry to damp
25	▼								
30									Total Depth: 30 feet Groundwater Depth: 27 feet
35									

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

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

BORING NO: B-6
PROJECT NAME: Oxnard HS Synthetic Field
PROJECT NUMBER: 303278-001
BORING LOCATION: Per Plan

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

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
				10/14/15		ML	106.6	1.8	**ALLUVIUM:** Light Yellow Brown fine to medium Sandy Silt, trace coarse Sand, trace fine Gravel, very stiff, dry to damp
5				8/12/17			108.9	2.7	
				8/16/26		SW	108.5	2.5	**ALLUVIUM:** Light Yellow Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, dense, dry to damp
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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

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

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

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

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0–5	X		■	12/30/33		ML	119.0	14.4	**ARTIFICIAL FILL:** Black fine Sandy Silt, trace medium to coarse Sand, trace fine Gravel, dense, damp
5			■	7/7/12		ML	111.2	19.7	**ALLUVIUM:** Dark Gray to Black fine Sandy Silt, little Clay, medium dense, damp
			■	4/6/6		ML		46.4	**ALLUVIUM:** Brown fine Sandy Silt, trace Clay, loose, damp to moist
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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

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

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

File Number: 303277-001 Lab Number: 098208

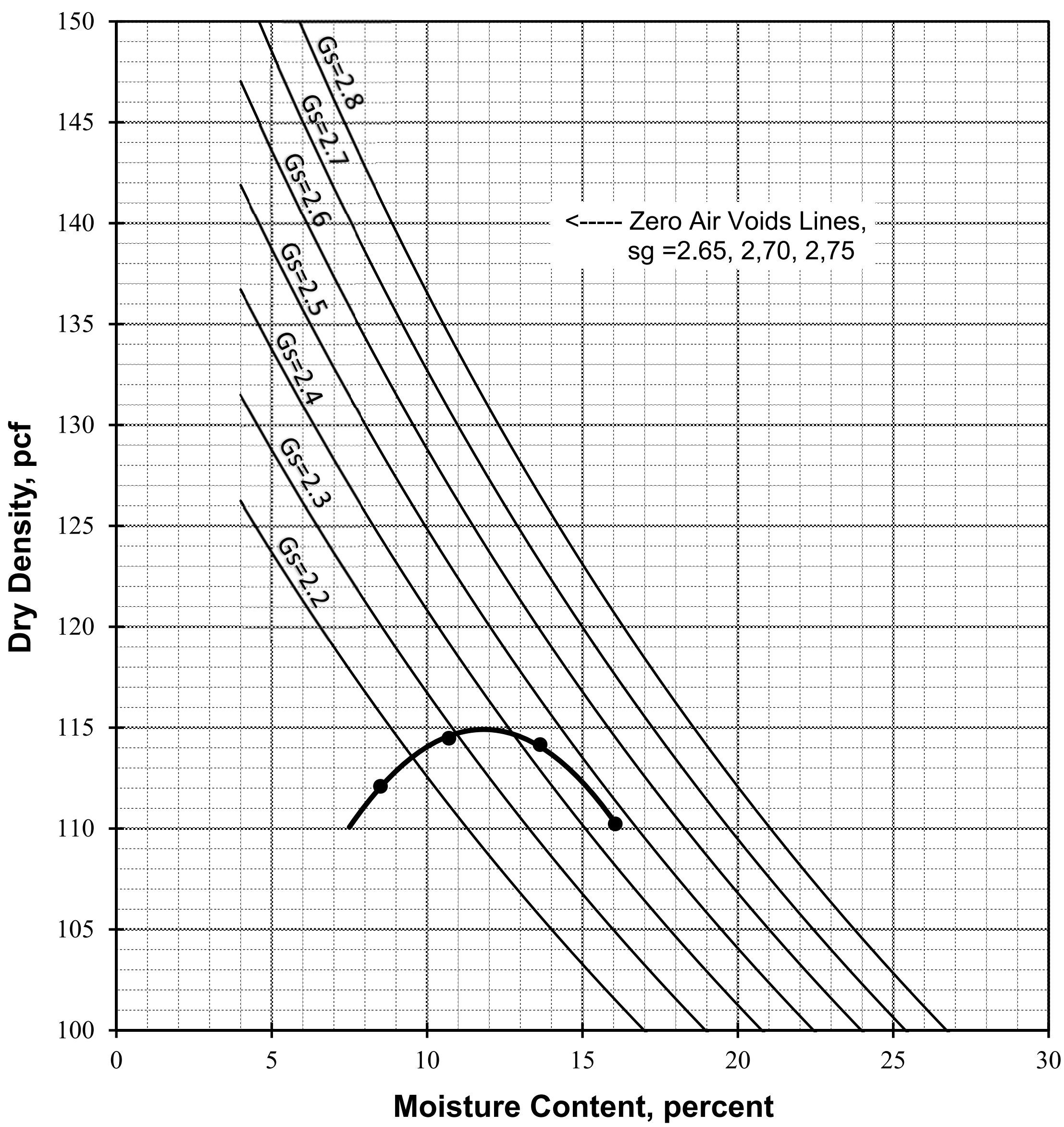
MAXIMUM DENSITY / OPTIMUM MOISTURE

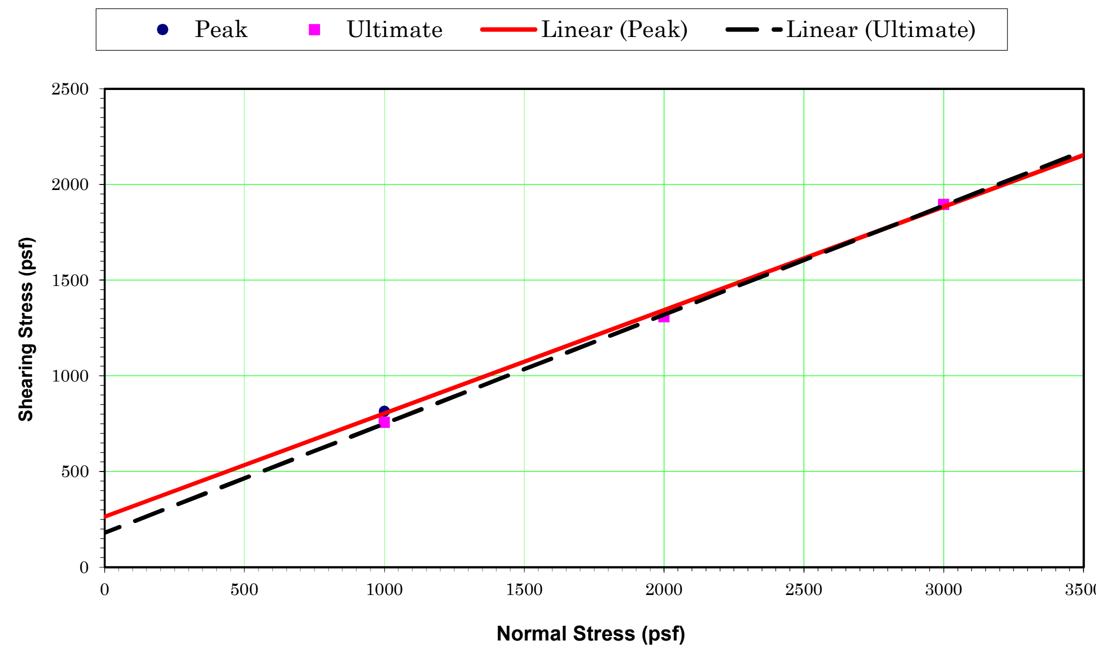
ASTM D 1557-12 (Modified)

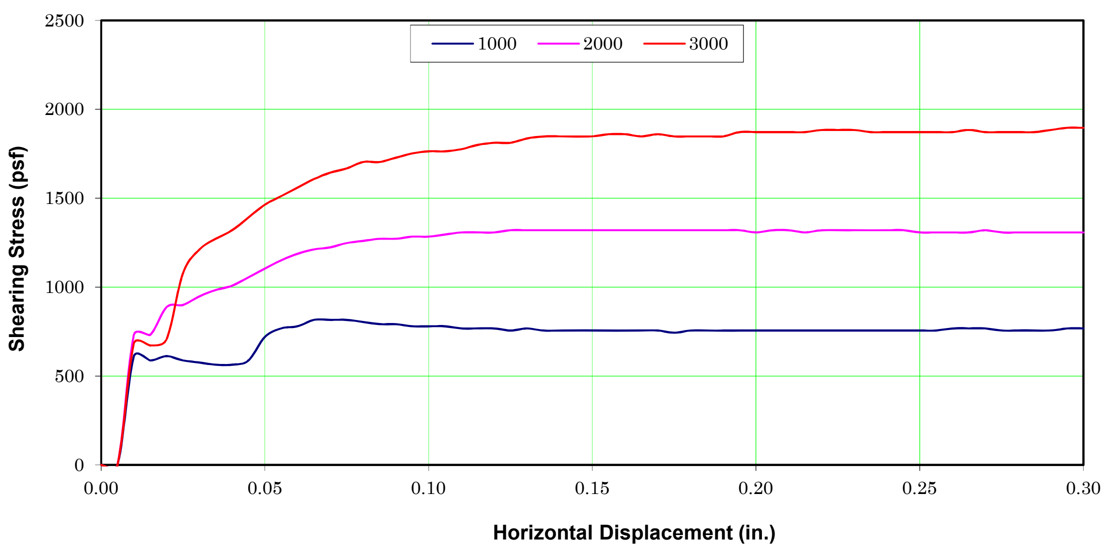
Job Name:	Oxnard High School Synthetic Turf Field	Procedure Used: A
Sample ID:	B 6 @ 0-5'	Prep. Method: Moist
Date:	7/29/2019	Rammer Type: Automatic
Description:	Dark Brown Sandy Silt	
SG:	2.35	

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

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







DIRECT SHEAR DATA*

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

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

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

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Oxnard High School Synthetic Turf Field

Earth Systems

8/27/2019 | 303278-001

File No.: 303278-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

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

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

Expansion Index: 65 Medium

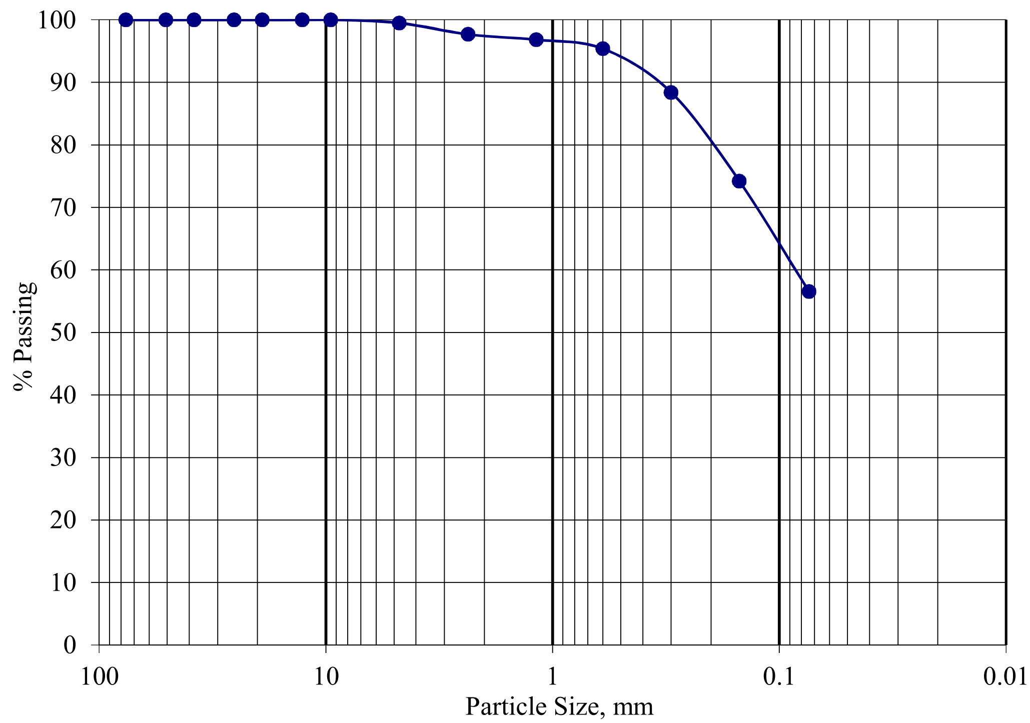
EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
130+	Very High

SIEVE ANALYSIS

ASTM C-136

Job Name: 303278-001
Sample ID: B 7 @ 0-5'
Description: ML

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	100
#4	100
#8	98
#16	97
#30	95
#50	88
#100	74
#200	57



Oxnard High School
Synthetic Turf Field

303278-001

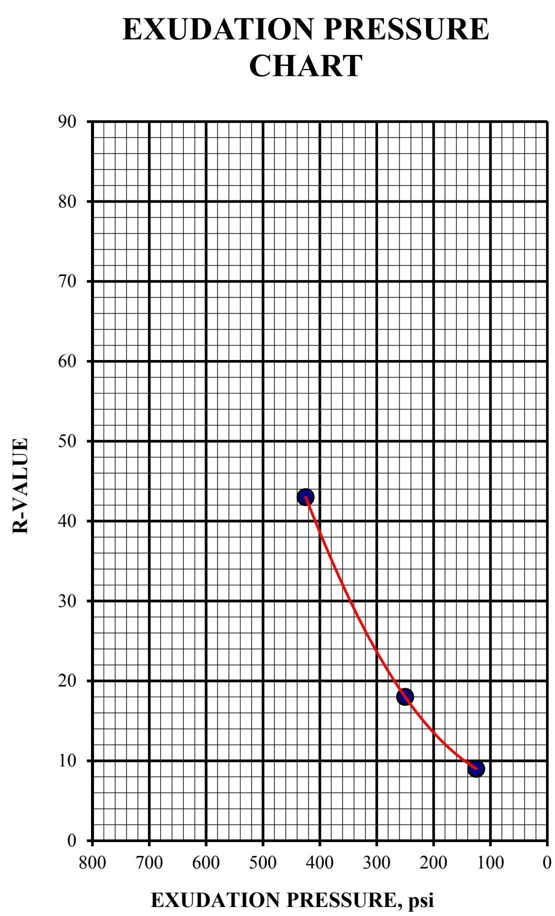
RESISTANCE '*R* ' VALUE AND EXPANSION PRESSURE

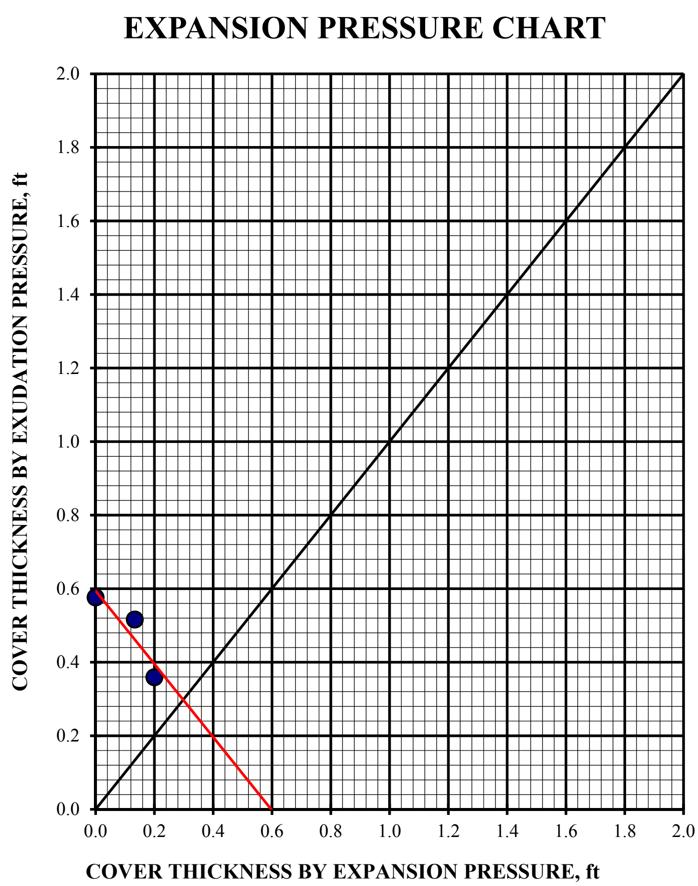
ASTM D 2844/D2844M-13

August 9, 2019

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

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



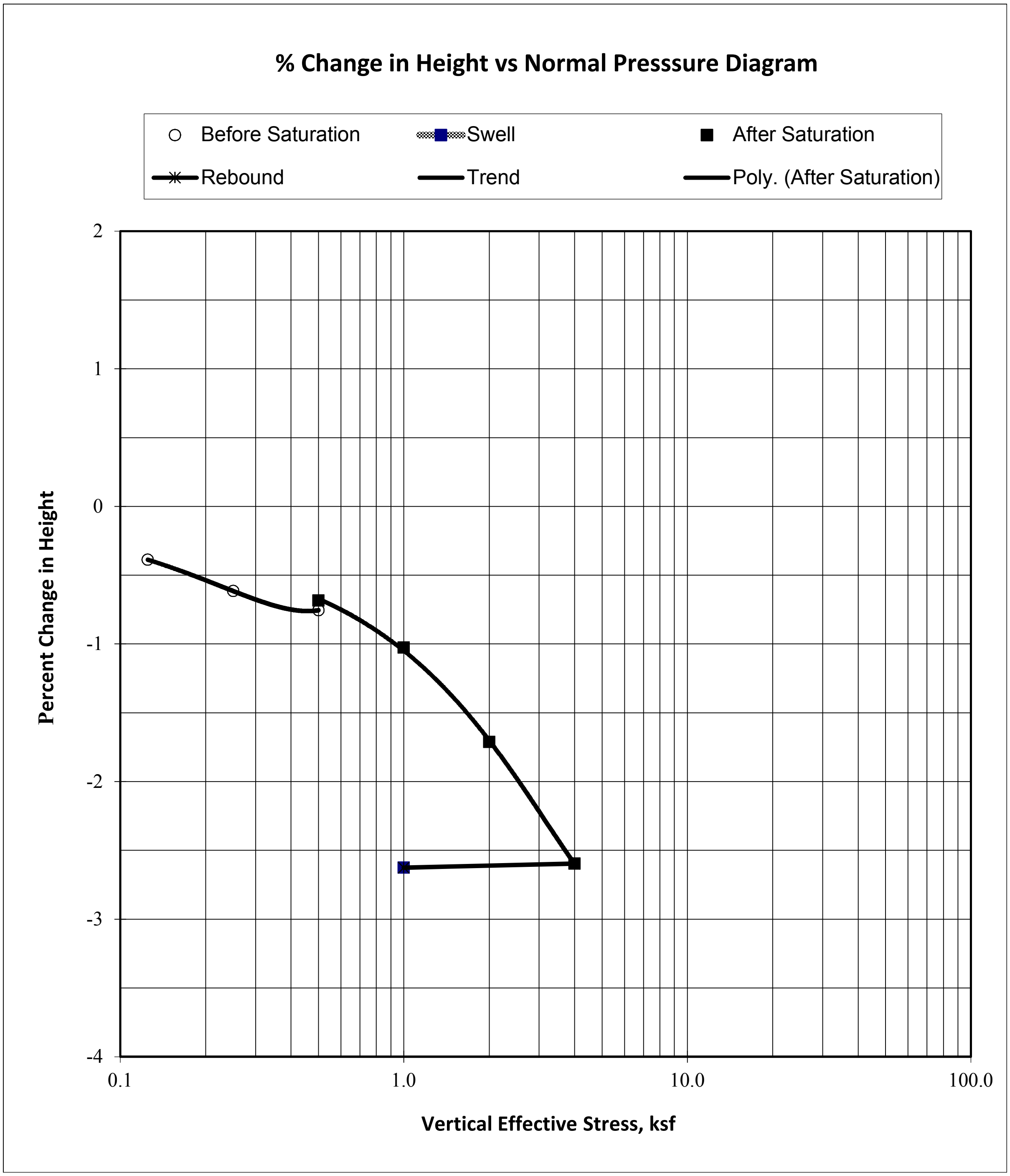


CONSOLIDATION TEST

ASTM D 2435-90

Oxnard High School Synthetic Turf Field
B 7 @5'
Silty Sand
Ring Sample

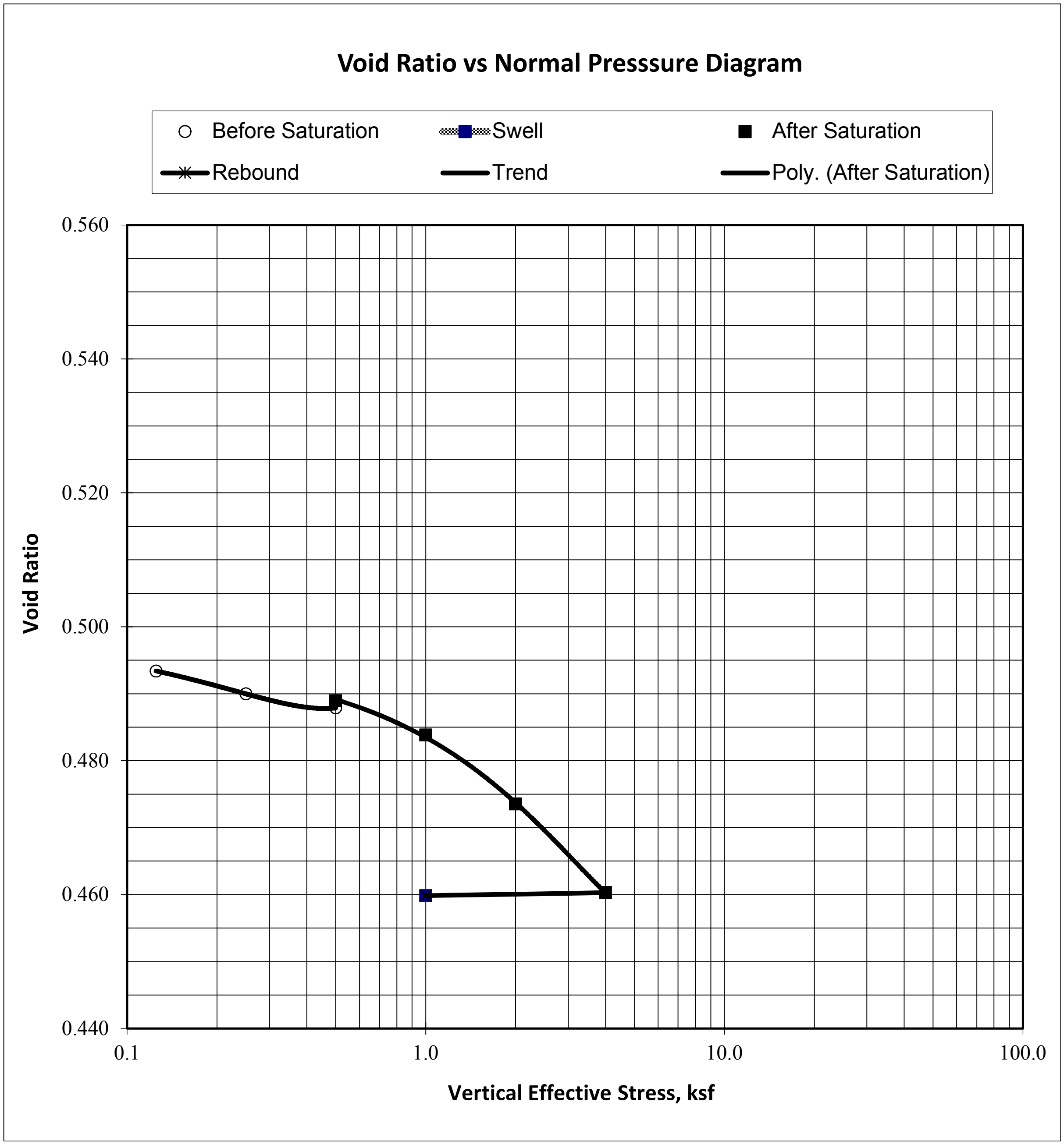
Initial Dry Density: 111.2 pcf
Initial Moisture, %: 19.7%
Specific Gravity: 2.67 (assume
Initial Void Ratio: 0.499



CONSOLIDATION TEST

ASTM D 2435-90

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



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

CERTIFICATE OF ANALYSIS

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

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

WET CHEMISTRY SUMMARY

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

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

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

TABLE 18-I-D

MINIMUM FOUNDATION REQUIREMENTS

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

WEIGHTED EXPANSION INDEX	FOUNDATIONS FOR SLAB AND RAISED FLOOR SYSTEM (4) (5): NUMBER OF FLOORS	STEM THICKNESS (INCHES)	FOOTING WIDTH (INCHES)	FOOTING THICKNESS (INCHES)	ALL PERIMETER FOOTINGS (5) — DEPTH BELOW NATURAL SURFACE OF GROUND AND FINISH GRADE (3) (8) (INCHES)	INTERIOR FOOTINGS FOR SLAB AND RAISED FLOORS (5) — DEPTH BELOW NATURAL SURFACE OF GROUND AND FINISH GRADE (3) (8) (INCHES)	REINFORCEMENT FOR CONTINUOUS FOUNDATIONS (2)	CONCRETE SLABS 3 ½ " MINIMUM THICKNESS: REINFORCEMENT (3)	CONCRETE SLABS 3 ½ " MINIMUM THICKNESS: TOTAL THICKNESS OF SAND	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)
0-20 Very low (nonexpansive)	1 2 3	8 8 10	12 15 18	8 7 8	12 18 24	12 18 24	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
21-50 Low	1 2 3	8 8 10	12 15 18	6 7 8	15 18 24	12 18 24	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.
51-90 Medium	1 2 3	8 8 10	12 15 18	8 8 8	21 21 24	12 18 24	1-#4 top and bottom #3 BARS @ 24" IN EXT. FOOTING BEND3' INTO SLAB (7)	6x6-10/10 WWF	4"	130% of optimum moisture required to a depth of 27" below lowest adjacent grade. Testing required.	Piers not allowed.
91-130 High	1 2 3	8 8 10	12 15 18	8 8 8	27 27 24	12 18 24	1-#5 top and bottom #3 BARS @ 24" IN EXT. FOOTING BEND 3' INTO SLAB (7)	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.
Above 130 Very High	Special design by licensed engineer/architect										

APPENDIX C

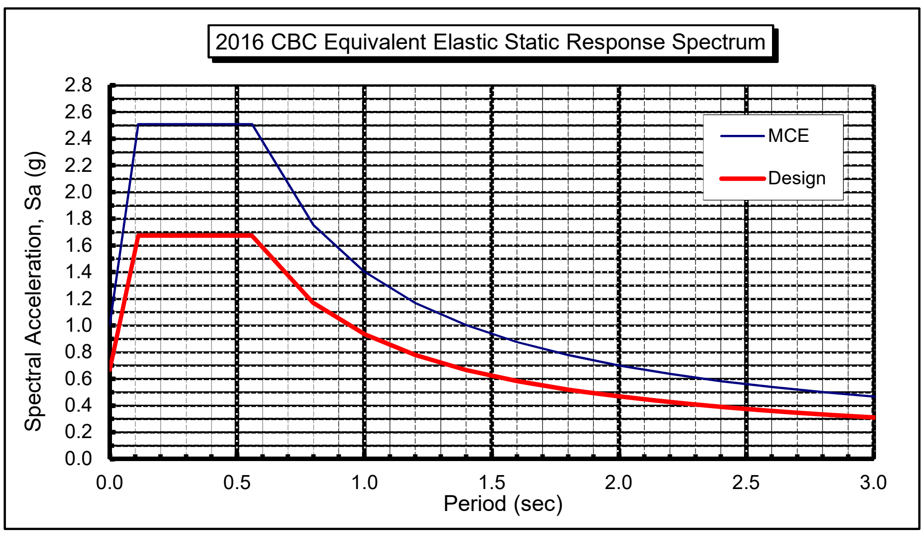
2016 CBC & ASCE 7-10 Seismic Parameters

US Seismic Design Maps

Fault Parameters

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

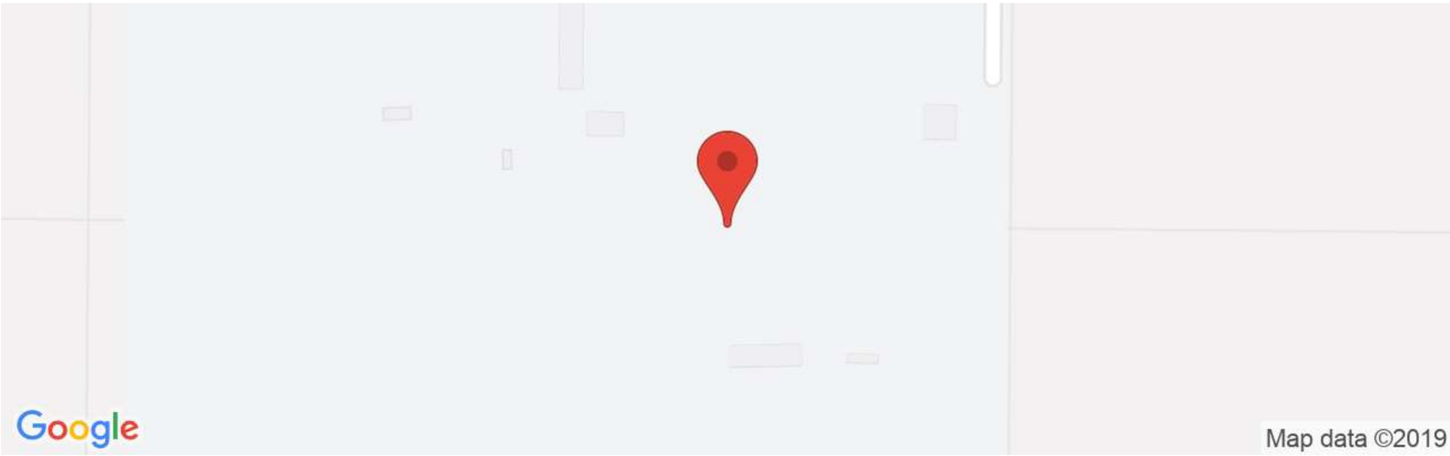
			CBC Reference	ASCE 7-10 Reference
Seismic Design Category		**E**	Table 1613.5.6	Table 11.6-2
Site Class		**D**	Table 1613.5.2	Table 20.3-1
Latitude:		34.216 N		
Longitude:		-119.213 W		
Maximum Considered Earthquake (MCE) Ground Motion				
Short Period Spectral Reponse	$\mathbf{S_S}$	**2.510 g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	$\mathbf{S_1}$	**0.935 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}	2.510 g	$= F_a*S_S$	
	S_{M1}	1.403 g	$= F_v*S_1$	
Design Earthquake Ground Motion				
Short Period Spectral Reponse	$\mathbf{S_{DS}}$	**1.673 g**	$= 2/3*S_{MS}$	
1 second Spectral Response	$\mathbf{S_{D1}}$	**0.935 g**	$= 2/3*S_{M1}$	
	To	0.11 sec	$= 0.2*S_{D1}/S_{DS}$	
	Ts	0.56 sec	$= S_{D1}/S_{DS}$	
Seismic Importance Factor	I	1.00	Table 1604.5	Table 11.5-1
	F_{PGA}	1.00		



Period T (sec)	Design Sa (g)
0.00	0.669
0.05	1.119
0.11	1.673
0.56	1.673
0.80	1.169
1.00	0.935
1.20	0.779
1.40	0.668
1.60	0.584
1.80	0.519
2.00	0.468
2.20	0.425
2.40	0.390
2.60	0.360
2.80	0.334
3.00	0.312

OSHPD

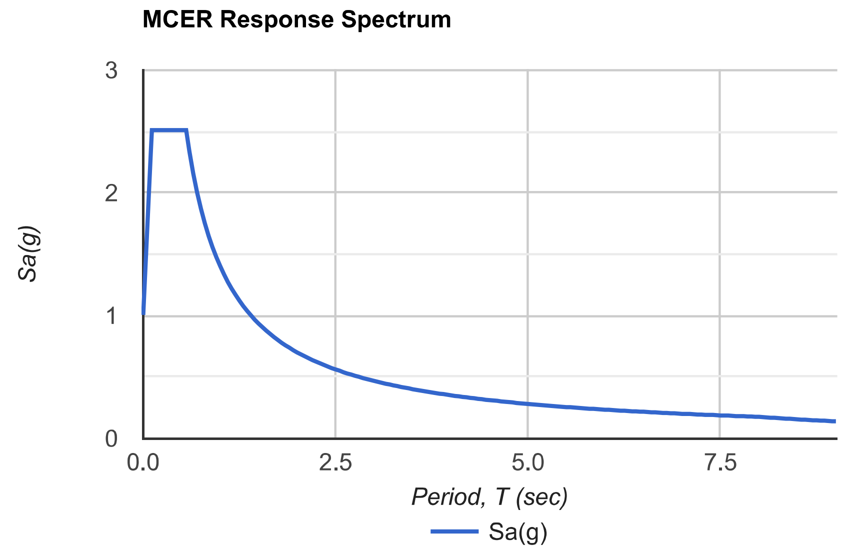
Latitude, Longitude: 34.2157, -119.2135

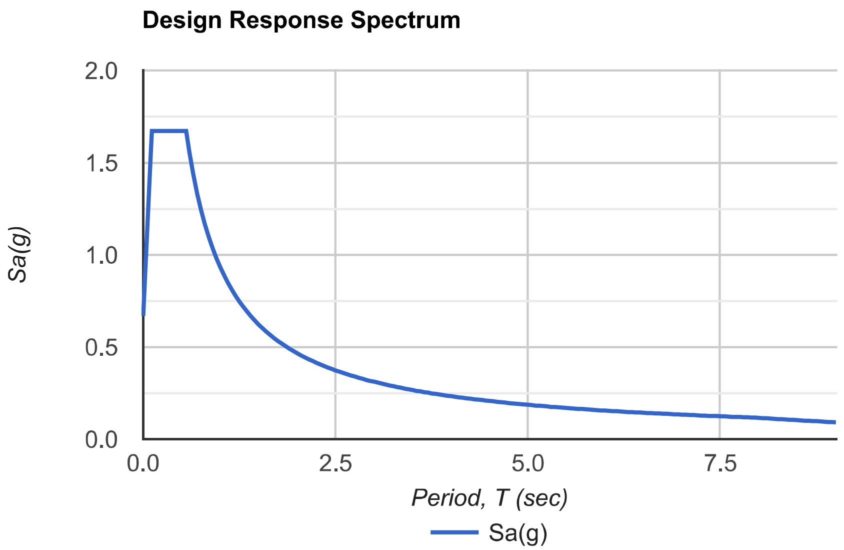


Date	7/8/2019, 1:52:59 PM
Design Code Reference Document	ASCE7-10
Risk Category	I
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.51	MCE_R ground motion. (for 0.2 second period)
S_1	0.935	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.51	Site-modified spectral acceleration value
S_{M1}	1.403	Site-modified spectral acceleration value
S_{DS}	1.673	Numeric seismic design value at 0.2 second SA
S_{D1}	0.935	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	0.978	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.978	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.51	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.733	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.654	Factored deterministic acceleration value. (0.2 second)
S1RT	0.935	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.025	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.016	Factored deterministic acceleration value. (1.0 second)
PGAd	1.007	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.918	Mapped value of the risk coefficient at short periods
C_{R1}	0.912	Mapped value of the risk coefficient at a period of 1 s





DISCLAIMER

While the information presented on this website is believed to be correct, SEAOC /OSHPD and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in this web application should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. SEAOC / OSHPD do not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the seismic data provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the search results of this webstie.

Table 1
Fault Parameters

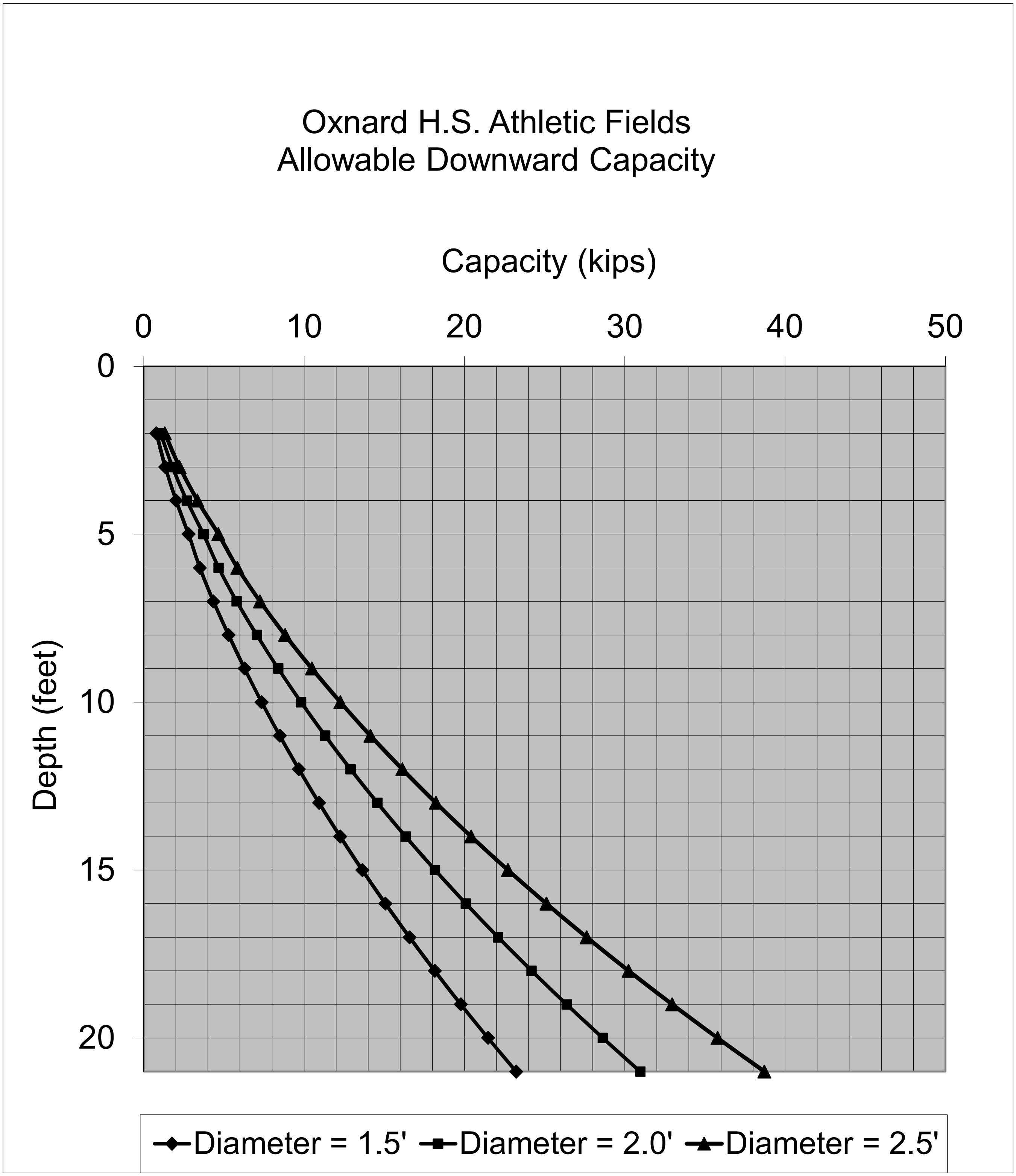
Fault Section Name	**Distance** (miles)	(km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
Oak Ridge (Onshore)	2.3	3.7	65	159	90	49	B	**7.2**		4
Oak Ridge (Offshore)	4.2	6.8	32	180	90	38	B	**6.9**		3
Ventura-Pitas Point	5.2	8.4	64	353	60	44	B	**6.9**		1
Simi-Santa Rosa	6.5	10.4	60	346	30	39	B	**6.8**		1
Red Mountain	9.9	16.0	56	2	90	101	B	**7.4**		2
Malibu Coast (Extension), alt 1	10.1	16.3	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	10.1	16.3	74	4	30	35	B'	**6.9**		
Sisar	13.2	21.3	29	168	na	20	B'	**7.0**		
Channel Islands Thrust	13.4	21.6	20	354	90	59	B	**7.3**		1.5
North Channel	14.0	22.6	26	10	90	51	B	**6.7**		1
Channel Islands Western Deep Ramp	15.0	24.1	21	204	90	62	B'	**7.3**		
Mission Ridge-Arroyo Parida-Santa Ana	15.4	24.8	70	176	90	69	B	**6.8**		0.4
Pitas Point (Lower)-Montalvo	15.4	24.8	16	359	90	30	B	**7.3**		2.5
San Cayetano	16.3	26.2	42	3	90	42	B	**7.2**		6
Santa Cruz Island	16.3	26.2	90	188	30	69	B	**7.1**		1
Anacapa-Dume, alt 1	17.1	27.5	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	17.1	27.5	41	352	60	65	B	**7.2**		3
Malibu Coast, alt 1	19.8	31.9	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	19.8	31.9	74	3	30	38	B	**6.9**		0.3
Santa Ynez (East)	21.0	33.8	70	172	0	68	B	**7.2**		2
Pitas Point (Upper)	22.1	35.6	42	15	90	35	B	**6.8**		1
Shelf (Projection)	22.2	35.7	17	21	na	70	B'	**7.8**		
Santa Cruz Catalina Ridge	22.7	36.5	90	38	na	137	B'	**7.3**		
Pine Mtn	25.4	40.9	45	5	na	62	B'	**7.3**		
Oak Ridge (Offshore), west extension	26.6	42.7	67	195	na	28	B'	**6.1**		
Santa Susana, alt 1	27.3	44.0	55	9	90	27	B	**6.8**		5
Santa Susana, alt 2	27.4	44.2	53	10	90	43	B'	**6.8**		
Santa Monica Bay	29.1	46.8	20	44	na	17	B'	**7.0**		
Northridge Hills	29.6	47.6	31	19	90	25	B'	**7.0**		
Del Valle	30.1	48.4	73	195	90	9	B'	**6.3**		
Holser, alt 1	30.4	49.0	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	30.4	49.0	58	182	90	17	B'	**6.7**		
San Pedro Basin	30.4	49.0	88	51	na	69	B'	**7.0**		
Santa Ynez (West)	31.0	49.9	70	182	0	63	B	**6.9**		2
Northridge	31.8	51.2	35	201	90	33	B	**6.8**		1.5
Pitas Point (Lower, West)	32.1	51.7	13	3	90	35	B	**7.2**		2.5
Big Pine (Central)	32.3	51.9	76	167	na	23	B'	**6.3**		
Big Pine (West)	33.3	53.6	50	2	na	18	B'	**6.5**		
Big Pine (East)	35.9	57.8	73	338	na	23	B'	**6.6**		
Compton	36.7	59.1	20	34	90	65	B'	**7.5**		

Reference: USGS OFR 2007-1437 (CGS SP 203) Based on Site Coordinates of 34.215682 Latitude, -119.213465 Longitude

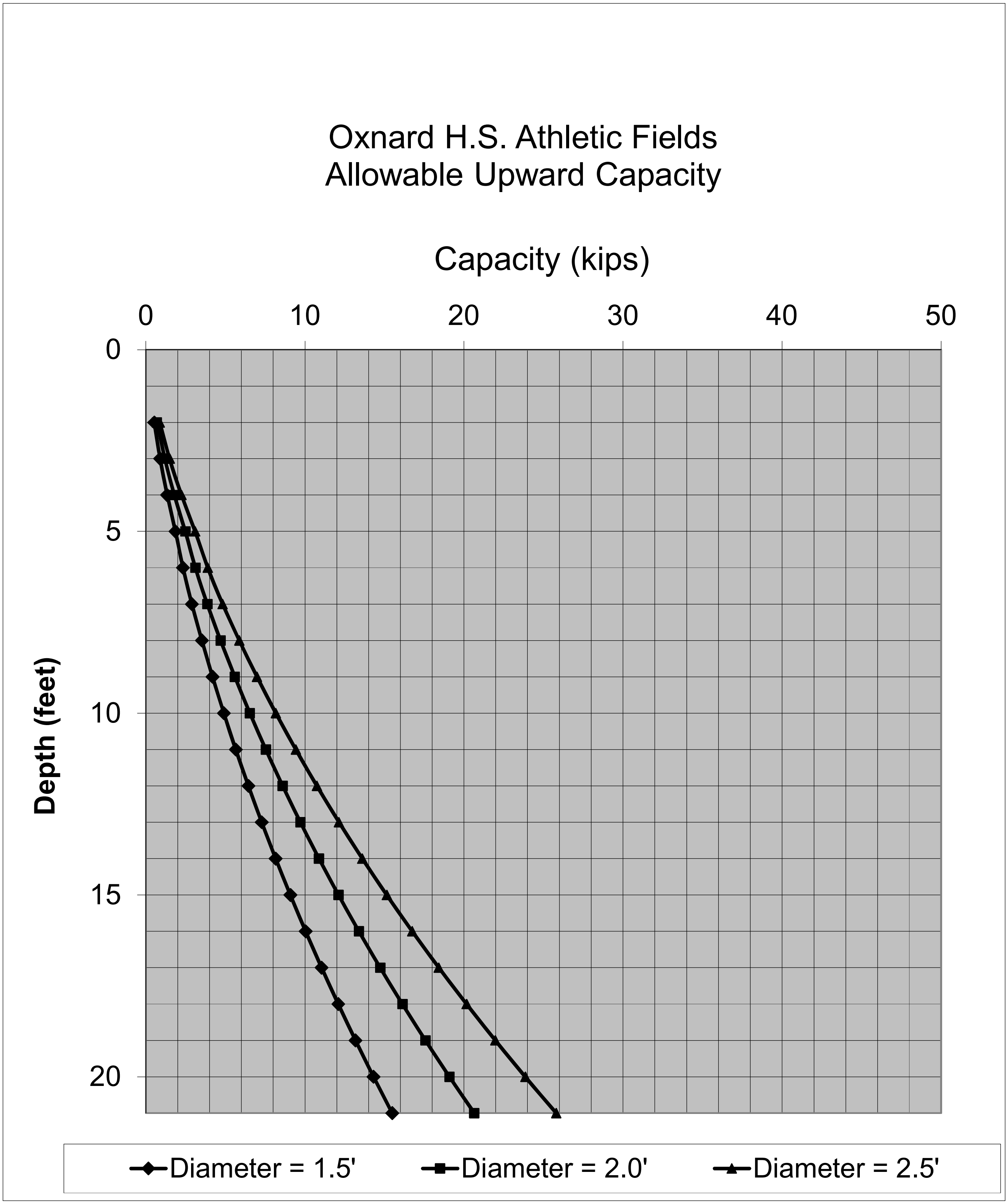
Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2007-1437). Mean magntude is average of Ellworths-B and Hanks & Bakun moment area relationship.

APPENDIX D

Pile Capacity Graphs



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



Oxnard 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'