GEOTECHNICAL ENGINEERING REPORT
FOR
PROPOSED IMPROVEMENTS TO ATHLETIC FIELDS AT
ADOLFO CAMARILLO HIGH SCHOOL,
4660 MISSION OAKS BOULEVARD
CAMARILLO, CALIFORNIA

PROJECT NO.: 303275-001
AUGUST 28, 2019

PREPARED FOR
OXNARD UNION HIGH SCHOOL DISTRICT

BY

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August 28, 2019

Project No.: 303275-001
Report No.: 19-8-3 (Revised)

Attention: Poul Hanson
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Oxnard, CA 93030

Project: Improvements to Athletic Field Surfaces
Adolfo Camarillo High School
4660 Mission Oaks Boulevard
Camarillo, California

As authorized, we have performed a geotechnical study for proposed improvements to the athletic field surfaces at Adolfo Camarillo High School in the City of Camarillo, 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-19-05-014 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

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INTRODUCTION

This report presents results of a Geotechnical Engineering study performed for proposed improvements to the athletic fields at Adolfo Camarillo High School in the City of Camarillo (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 three ticket booths with attached entry gates at the entrances to the football field. Existing asphalt walkways around the football field will be replaced with concrete sidewalks ranging in width from 6 to 12 feet. New parking spaces will be added southeast of the eastern end of the track, including some in an area where leach lines reportedly currently exist. An existing asphalt service road will be replaced with new asphalt paving, and new sidewalk will run between the western parking lot and the baseball/softball fields. 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 either infiltrate into the subsurface or 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 5 hollow-stem-auger borings to study soil and groundwater conditions.
3. Drilling and logging 2 hollow-stem-auger borings for infiltration testing.
4. Laboratory testing soil samples obtained from the subsurface exploration to determine their physical and engineering properties.
5. Consulting with owner representatives and design professionals.
6. Analyzing the geotechnical data obtained.
7. Preparing this report.

Contained in this report are:

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

There are several faults located within the region, including the Camarillo Fault that is mapped along an east-west trend through the athletic field areas. As such, the project area is located within the "Fault Rupture Hazard Zone" delineated by the State of California (CDMG. 1972, Revised 1999) for the Camarillo Fault. However, the Camarillo Fault is not considered capable of generating a large seismic event. The nearest known fault capable of generating significant earthquakes is the Simi-Santa Rosa Fault, which is located approximately 1.4 miles north of the subject site.

The site is underlain by alluvial sediments consisting of loose to medium dense silty sands, fine to medium sands, and firm to very stiff sandy clays. Boring No. B-4 encountered artificial fill consisting of stiff silty clay with varying sand content. In addition to the artificial fill, bedrock consisting of the Saugus Formation was encountered and consisted of silty fine-grained sandstone. Boring B-2 from 2009 site studies for the aquatic center also encountered fill when advanced from the main campus level near the top of the walkway down to the football field.

The site is not within any of the Liquefaction Hazard 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 (34.2156˚ North Latitude and -119.0102° 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.)

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	2.146g
Spectral Response Acceleration at 1 sec. – S_1	0.787g
Site Coefficient – F_a	1.00
Site Coefficient – F_v	1.50
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.146g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.181g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.430g
One Second Spectral Response – S_{D1}	0.787g
Site Modified Peak Ground Acceleration - PGA_M	0.809g
Values appropriate for a 2% probability of exceedance in 50 years	

The Fault Parameters table in Appendix C lists the significant "active" and "potentially active" faults within a radius of about 34 miles from the subject site. The distance between the site and the nearest portion of each fault is shown, as well as the respective estimated maximum earthquake magnitudes, and the deterministic mean site peak ground accelerations.

SOIL CONDITIONS

Evaluation of the subsurface indicates that soils are generally alluvium consisting of loose to medium dense silty sands, fine to medium sands, and firm to very stiff sandy clays. Boring No. B-4, which was located near the northeast corner of the football field, encountered approximately 7 feet of artificial fill consisting of stiff silty clay with varying sand content. Artificial fill was also encountered to a depth of 8.5 feet in Boring B-2, which was drilled during 2009 studies for the aquatic center at the main level of the campus. Saugus Formation bedrock was encountered below the fill in Boring B-4, and consisted of silty fine-grained sandstone. Saugus Formation was also encountered below the fill in the 2009 boring (B-2), and consisted of interbeds of clayey silty sands with caliche, silty sands with gravels, and silty clay.

Near-surface alluvial soils encountered within the fields in Boring Nos. B-1 through B-4 are generally characterized by low blow counts and in-place densities, but low compressibilities. Testing indicates that near-surface soils within the field area lie in the "very low" expansion range because the expansion index equals 0. [A locally adopted version of the 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 not encountered to a depth of 51.5 feet during drilling for a feasibility study conducted for a proposed pool complex (see Site-Specific Bibliography). Mapping of historically high groundwater levels by the California Geological Survey (CGS, 2002a) indicates that groundwater has been at least 55 feet below the ground surface near the subject site.

The subject site is not located within any of the Liquefaction Hazard Zones delineated by the California Division of Mines and Geology (2002b). As a result, it appears that the hazard posed by liquefaction to the proposed improvements is low.

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 (61 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 (6,000 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.

Infiltration of storm water may be feasible for this campus. More detailed findings after infiltration testing is completed.

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% 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

It should be noted that the location provided to Earth Systems for the future 498 square-foot restroom building is within the Fault Rupture Hazard Zone for the Camarillo Fault, and an evaluation of the fault rupture hazard may be required. However, if the size precludes the requirement for hazard evaluation, or an acceptable location for the restroom is located outside the fault zone, a conventional foundation system would be acceptable.

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. Non-complying fill would include the gravel and piping of the leach lines that reportedly exist southeast of the eastern end of the track around the perimeter of the football field. 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.

Once the gravel and piping is completely removed from the existing leach lines, the excavations should be deepened and widened until firm native soils are encountered in each direction.

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

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

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.

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 2,000 psf. This value has a factor of safety of 3.

Isolated pad footings may be designed based on an allowable bearing value of 2,300 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 bases of foundations. A coefficient of friction of 0.60 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 380 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 “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 380 psf per foot of penetration in the compacted fill (upper 5 feet) and an EFW of 300 pcf in the underlying firm native soils. These resisting pressures are ultimate values. The maximum passive pressure used for design should not exceed 4,200 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).

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.

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. 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.)

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. 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

Conventional cantilever retaining walls backfilled with compacted on-site soils may be designed for active pressures of 40 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 60 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.

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 TRACK RESURFACING

Assuming a Traffic Index of 5 for areas to be used for asphalt below track resurfacing, and using the measured R-Value of 29, paving sections should have a minimum gravel equivalent of 1.14 feet. This can be achieved by using 3 inches of asphaltic concrete on 6 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 new fire lanes or drive lanes in parking areas with a Traffic Index of 6.5, paving sections should have a minimum gravel equivalent of 1.48 feet. This can be achieved by using 4 inches of asphaltic concrete on 9 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 29 (approximately equivalent to a coefficient of subgrade reaction of k = 150 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), the following minimum unreinforced paving section was determined:

1. Concrete thickness = 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= 12.5 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 = 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.

STORM WATER INFILTRATION FEASIBILITY TESTING

On August 22, 2019, a set of two 8-inch diameter infiltration borings (P-1 and P-2) were drilled to depths of about 7 and 18 feet below the existing ground surface to determine the soil profile and allow installation of plastic casing for infiltration testing (see Site Plan in Appendix A for infiltration boring locations). All infiltration borings were bottomed into native Alluvium (see Logs of Borings in Appendix A).

After drilling was completed, 3-inch diameter slotted PVC casings were lowered into the boreholes. The annuli between the casings and boring walls were then filled with pea gravel. The falling-head borehole infiltration test procedure was used for infiltration testing. Approximately 2 feet of water was added to the bottom of each of the holes to start the tests, and the drop in the water surface monitored by taking periodic measurements. Readings were taken at reasonable time intervals based on infiltrating rate, and after each of these intervals, water was added to return the water level to its original depth above the hole bottom for the next test interval. The tests were run until the infiltration rates were reasonably stable.

It should be noted that the rate the water surface drops in a borehole is a percolation rate, which is related to, but is not an infiltration rate. Percolation rate ignores the wetted soil surface area into which the water is infiltrating and does not account for the volume of water infiltrated. An infiltration rate considers both factors. Hence, percolation rates (in unit length per unit time) are an overestimation of infiltration rates (also in unit length per unit time).

Earth Systems uses the Porchet equation to account for the wetted surface area and volume of water infiltrated to estimate an infiltration rate. Forms of the equation can be found in the Riverside County - Low Impact Development BMP Design Handbook (2001), the South Orange County Version, Technical Guidance Documents Appendices (2017), or in a paper by J.W. Van Hoorn, "Determining Hydraulic Conductivity with the Inversed Auger Hole and Infiltrometer Methods." The Porchet equation in its most simple form is the volume of water infiltrated divided by the product of the change in time and the wetted surface area. By substitution, the equation can be shown to be equal to:

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

where: ΔH = Change in water level (inches)
Δt = Change in time (minutes)
r = Radius of test hole (inches)
H_{avg} = Average height of water in test hole (inches)

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

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

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

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

There are many factors that influence the infiltration rate. Clear water was used in our tests, whereas deleterious material will likely be contained in the storm water. Variations in soil conditions within the limits of the proposed infiltration system will likely affect infiltration characteristics. The designer who utilizes the infiltration results should consider these factors, as well as apply a factor-of-safety to the infiltration rate to account for future disposal bed siltation.

Based on the infiltration testing results in Appendix E, the measured test infiltration rates for the depths tested and boring locations are summarized in the following table:

Boring	Boring Depth (feet)	Infiltration Rate (inch/hour)	Infiltration Rate (cm/sec)
P-1	7	1.58	$1.115X10^{-3}$
P-2	18	0.14	$9.878X10^{-5}$

The designer of the proposed infiltration system beneath the synthetic turf should also consider that a minimum of 2 feet of compacted soil will be present below the bottom of the synthetic turf system. The infiltration rates provided above are for the native soils at the depths tested. Compaction of the native soils will reduce the infiltration rate of the upper 2 feet of soils underlying the 6-inch thick layer of Class II Permeable Base. The designer of the proposed infiltration system should consider the use of gravel-filled drains that extend below the compacted native soils to allow the storm water to infiltrate into the underlying native soils.

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.

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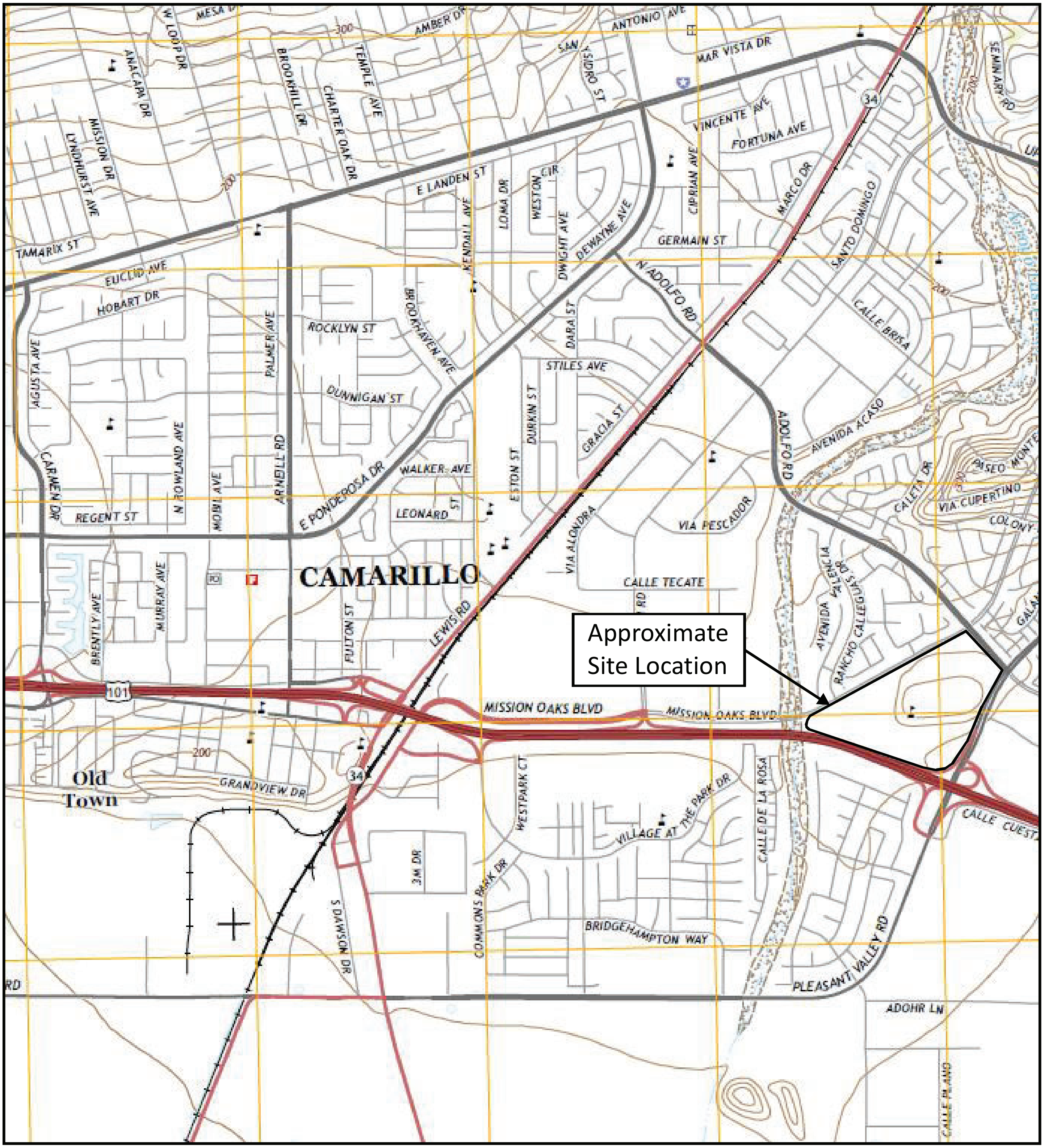
APPENDIX A

Vicinity Map
Regional Geologic Map
Seismic Hazard Zones Map
Historically Shallowest Groundwater Map
Field Study
Site Plan
Logs of Exploratory Borings (2019)
Log of Exploratory Boring B-2 (2009)
Boring Log Symbols
Unified Soil Classification System



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

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

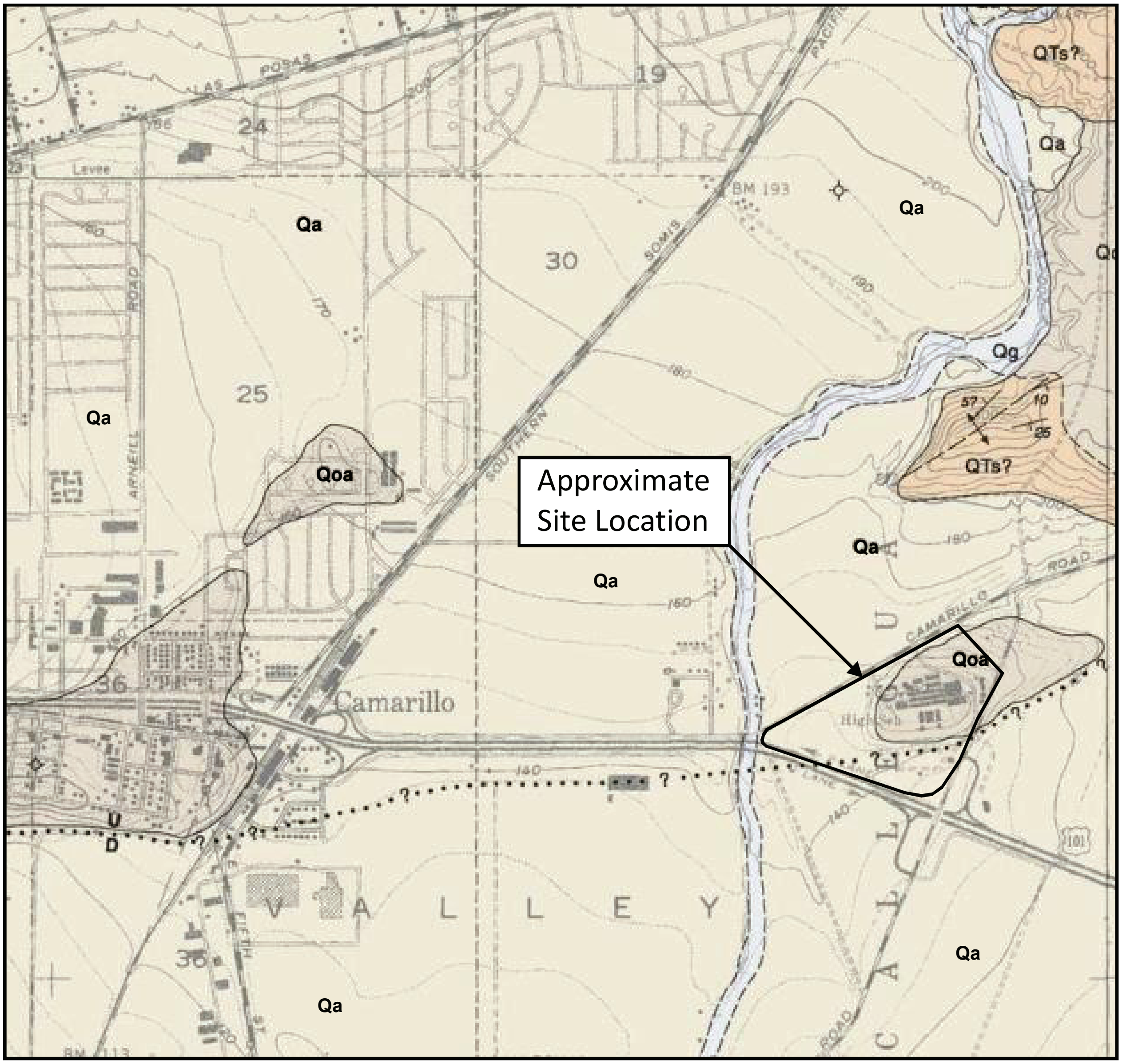
N

VICINITY MAP

Camarillo High School Synthetic Field
Camarillo, California

Earth Systems

August 2019 | 303275-001



Approximate Site Location

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

Qoa

OLDER DISSECTED SURFICIAL SEDIMENTS

Qoa *Dissected, weakly indurated alluvial gravel, sand and clay*

af

Qg

Qf

Qa

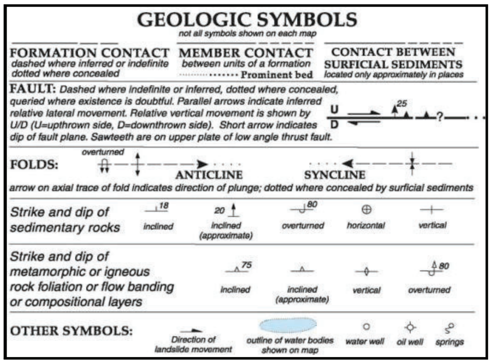
SURFICIAL SEDIMENTS

af *Artificial fill*

Qg *Stream channel sand and gravel*

Qf *Alluvial fan gravel and sand, locally slightly indurated*

Qa *Alluvium: gravel, sand and clay of flatlands*



GEOLOGIC SYMBOLS

not all symbols shown on each map

FORMATION CONTACT *dashed where inferred or indefinite dotted where concealed*

MEMBER CONTACT *between units of a formation* **Prominent bed**

CONTACT BETWEEN SURFICIAL SEDIMENTS *located only approximately in places*

FAULT: *Dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful. Parallel arrows indicate inferred relative lateral movement. Relative vertical movement is shown by U/D (U=upthrown side, D=downthrown side). Short arrow indicates dip of fault plane. Sawteeth are on upper plate of low angle thrust fault.*

FOLDS: *overturned* **ANTICLINE** **SYNCLINE**

arrow on axial trace of fold indicates direction of plunge; dotted where concealed by surficial sediments

Strike and dip of sedimentary rocks: *inclined*, *inclined (approximate)*, *overturned*, *horizontal*, *vertical*

Strike and dip of metamorphic or igneous rock foliation or flow banding or compositional layers: *inclined*, *inclined (approximate)*, *vertical*, *overturned*

OTHER SYMBOLS: *Direction of landslide movement*, *outline of water bodies shown on map*, *water well*, *oil well*, *springs*

N

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

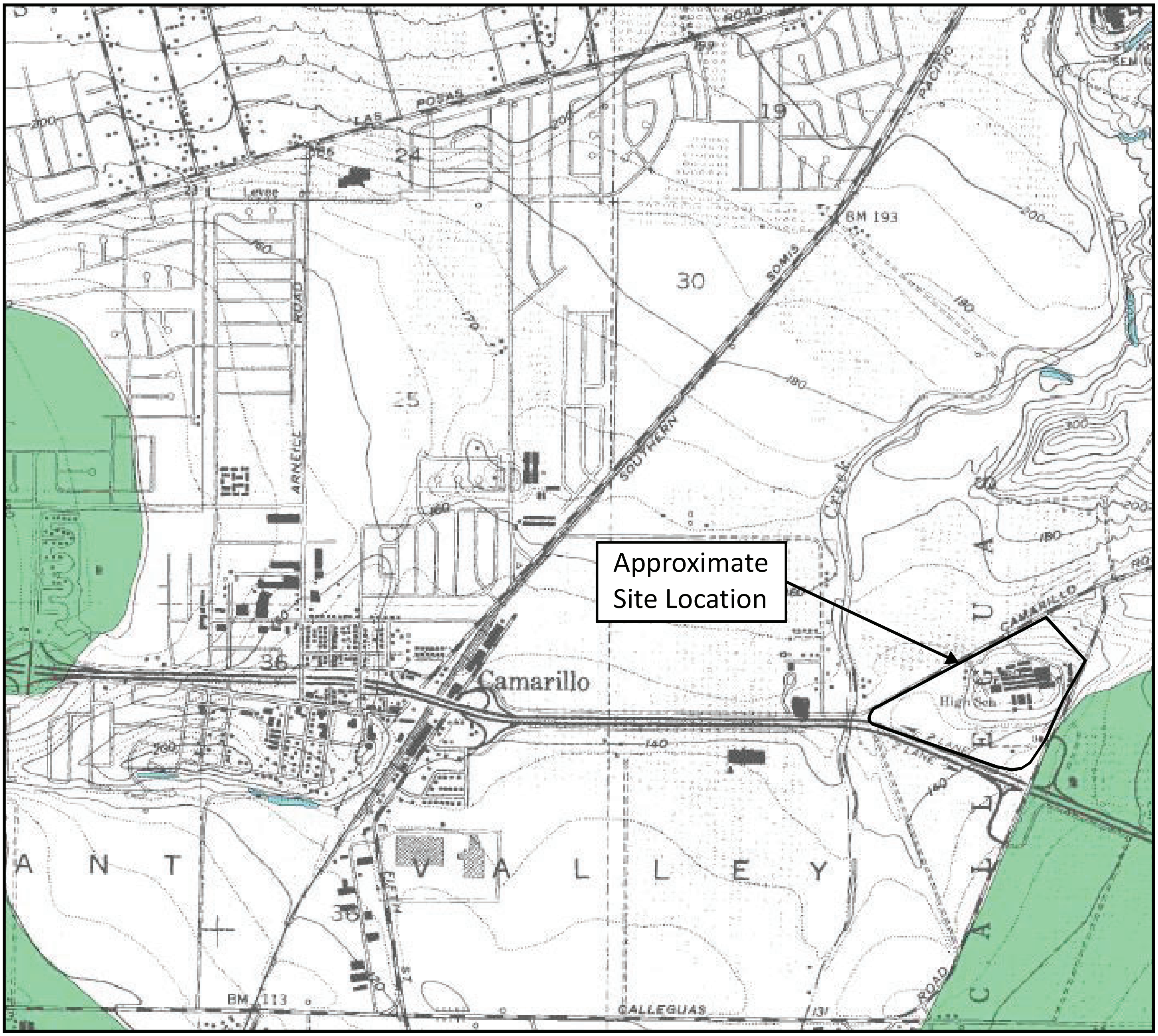
REGIONAL GEOLOGIC MAP

Camarillo High School Synthetic Field
Camarillo, California

Earth Systems

August 2019

303275-001



MAP EXPLANATION

Zones of Required Investigation:

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

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

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

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

STATE OF CALIFORNIA

SEISMIC HAZARD ZONES

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

CAMARILLO QUADRANGLE

OFFICIAL MAP

Released: February 7, 2002

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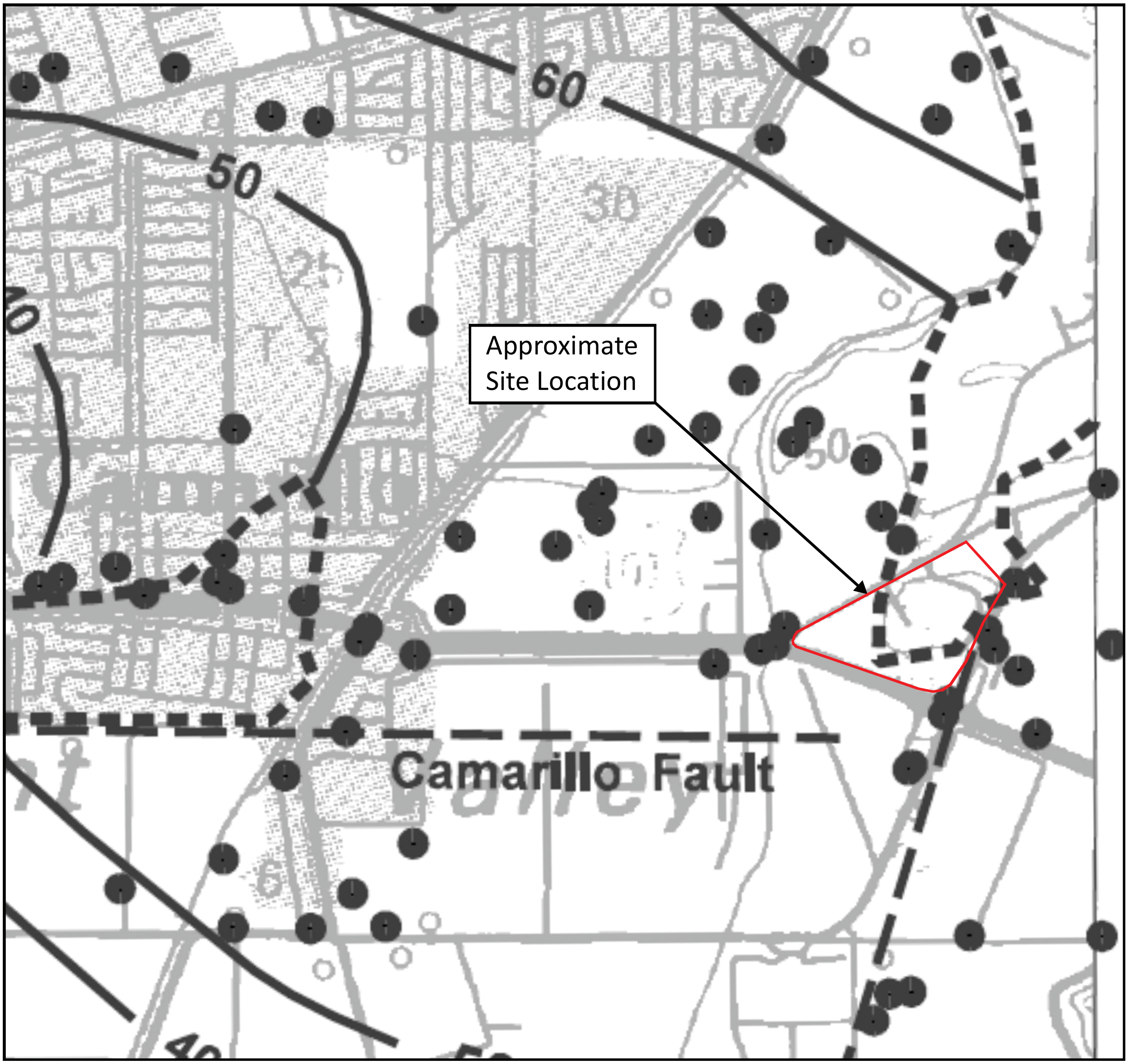
SEISMIC HAZARD ZONES MAP

Camarillo High School Synthetic Field
Camarillo, California

Earth Systems

August 2019	303275-001



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

30	Depth to historically highest ground water level in feet
▪▪▪▪	Valley / Mountains Boundary
— — —	Groundwater Barrier
●	Borehole Site
▲	Approximate location of sand boils and lurch cracks observed following the Point Mugu Earthquake of February 21, 1973.

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

N

SEISMIC HAZARD ZONES MAP

Camarillo High School Synthetic Field
Camarillo, California

Earth Systems

August 2019 | 303275-001

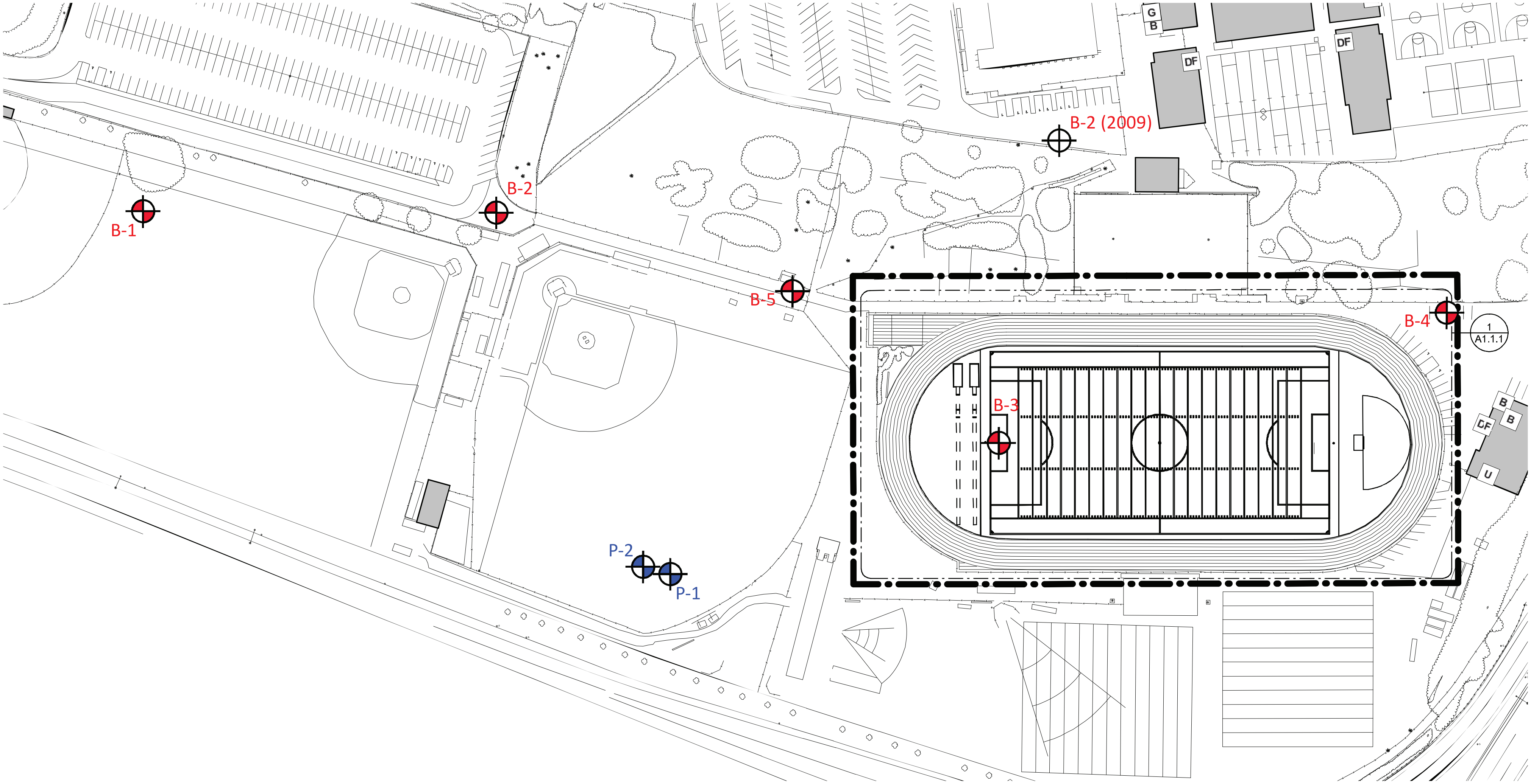
FIELD STUDY

A. Five soil 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. In addition, 2 borings were drilled for infiltration testing to depths of 7 and 18 feet below existing ground surface. The borings were drilled on June 27, 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). 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 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-1, B-3, and P-1 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 location

P-1 : Approximate infiltration test location

B-2 (2009) : Approximate boring location (2009 Studies)

N

Approximate Scale: 1" = 100'

0 100' 200'

SITE PLAN

Camarillo High School
Oxnard, California

Earth Systems

August 2019 | 303275-001

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

BORING NO: B-1	DRILLING DATE: June 27, 2019
PROJECT NAME: Camarillo High School Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303275-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
	Bulk	SPT	Mod. Calif.						
0	☒		⧄	4/6/8	▥	SM			**ALLUVIUM:** Light Gray Brown Silty fine Sand, trace medium Sand, loose, damp
5			■	4/5/5	░	SP	100.1	10.5	**ALLUVIUM:** Light Brown fine to medium Sand, loose, dry to damp
			■	3/4/5	▨	CL	83.9	21.4	**ALLUVIUM:** Dark Brown fine Sandy Clay, firm, moist
10									Total Depth: 10 feet No Groundwater Encountered
15									
20									
25									
30									
35									

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

Earth Systems

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

BORING NO: B-2
PROJECT NAME: Camarillo High School Synthetic Field
PROJECT NUMBER: 303275-001
BORING LOCATION: Per Plan

DRILLING DATE: June 27, 2019
DRILL RIG: CME-75
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: A. Luna

Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
	Bulk	SPT	Mod. Calif.						
0					▨				**Asphalt: 2.0", Base Material: 3.0"**
			■	9/12/11	▨	SP	102.3	4.9	**ALLUVIUM:** Light Yellow Brown fine to medium Sand, trace coarse Sand, medium dense, dry to damp
5			■	6/12/22	▨	SP	110.0	3.6	
			■	10/15/22	▨	SP			
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 27, 2019
PROJECT NAME: Camarillo High School Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303275-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type Bulk	 SPT	 Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6"	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0	☒		■	4/5/6	▥	SM	100.6	8.5	**Asphalt: 2.0", Base Material: 3.0"** **ALLUVIUM:** Light Yellow Brown Silty fine Sand, loose, damp
5			■	3/7/9	░	SW	99.4	4.5	**ALLUVIUM:** Light Yellow Brown fine to medium Sand, medium dense, dry to damp
			■	4/3/4	░	SW	97.8	4.4	**ALLUVIUM:** Light Yellow Brown fine to medium Sand, loose, 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
PROJECT NAME: Camarillo High School Synthetic Field
PROJECT NUMBER: 303275-001
BORING LOCATION: Per Plan

DRILLING DATE: June 27, 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: 5.0"**
			■	5/8/11	▨	CL	106.8	14.5	**ARTIFICIAL FILL:** Black Silty Clay, trace medium to coarse Sand, little fine Sand, stiff, damp to moist
5			■	4/6/9	▨	CL	105.4	14.8	
			■	6/16/36	▒	QTs			**SAUGUS FORMATION:** Light Yellow Brown Silty fine grained Sandstone, friable, weakly cemented, 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-5	DRILLING DATE: June 27, 2019
PROJECT NAME: Camarillo High School Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303275-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
	Bulk	SPT	Mod. Calif.						
0					▨				**Asphalt: 2.0", Base Material: 6.0"**
			■	6/11/18	▨	CL	111.1	17.8	**ALLUVIUM:** Dark Brown fine Sandy Clay, trace to little medium Sand, very stiff, moist
5			■	10/16/17	░	SP	123.7	9.6	**ALLUVIUM:** :Light Orange Brown fine Sand, little medium Sand, trace fine Gravel, medium dense, damp
			■	5/8/12	░	SP	94.8	15.6	
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: P-1
PROJECT NAME: Rio Mesa High School Synthetic Field
PROJECT NUMBER: 303280-001
BORING LOCATION: Per Plan

DRILLING DATE: June 27, 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	☒				▥	SM			**ALLUVIUM:** Light Gray Silty fine Sand, medium dense-damp
5					░	SP			**ALLUVIUM:** Light Brown fine Sand, trace medium Sand, trace Silt, medium dense-dry to damp
									Total Depth: 7 feet No Groundwater Encountered
10									
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: P-2	DRILLING DATE: June 27, 2019
PROJECT NAME: Camarillo High School Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303275-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
	Bulk	SPT	Mod. Calif.						
0 5 10					▥	SM			**ALLUVIUM:** Light Gray Brown Silty fine Sand, trace medium Sand, trace fine Gravel, medium dense-damp
					░	SP			**ALLUVIUM:** Light Yellow Brown fine Sand, little medium Sand, trace Silt, medium dense-dry to damp
15					▨	CL			**ALLUVIUM:** Dark Gray fine Sandy Clay, stiff-damp
					▥	SP			**ALLUVIUM:** Light Brown Silty fine Sand, trace Clay, medium dense-damp
20 25 30 35									Total Depth: 18 feet No Groundwater Encountered

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

BORING NO: 2
PROJECT NAME: Camarillo High School Pool
PROJECT NUMBER: VT-24393-01
BORING LOCATION: Per Plan (N34.2164, W119.0090)

DRILLING DATE: November 23, 2009
DRILL RIG: Mobile B-80
DRILLING METHOD: 6" Hollow Stem
LOGGED BY: P. Boales

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					■				**SURFACE:** 3.5" Asphalt on 9" aggregate base
					▥	ML			**ARTIFICIAL FILL:** Sandy clayey silt, moist, stiff, dk. yel. brown
			■	9/11/15	▒	SC	110.5	12.8	**ARTIFICIAL FILL:** Silty clayey sand with some roots, moist, medium dense, dark yellowish orange
5			■	8/15/18	▨	CL	96.7	16.5	**ARTIFICIAL FILL:** Sandy clay with trace pinhole voids, very stiff, moist, dark yellowish brown
			■	11/15/21	▨	CL	113.6	15.0	
10			■	7/9/11	▥	SM	102.1	13.3	**SAUGUS FM.:** Clayey silty sand with minor caliche, medium dense, moist, dark yellowish orange
15			☒	18/27/40	░	SP			**SAUGUS FM.:** Slightly silty sand with some gravels, dry, dense, very pale orange
20			■	8/13/19	▨	CL	93.2	28.5	**SAUGUS FM.:** Silty clay, moist, very stiff, dark yellow orange
25									**TOTAL DEPTH: 21.5 Feet** **Groundwater Was Not Encountered**
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: 3	DRILLING DATE: November 23, 2009
PROJECT NAME: Camarillo High School Pool	DRILL RIG: Mobile B-80
PROJECT NUMBER: VT-24393-01	DRILLING METHOD: 6" Hollow Stem
BORING LOCATION: Per Plan (N34.2165, W119.0096)	LOGGED BY: P. Boales

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					■				**SURFACE:** 3.5" Asphalt on 9" aggregate base
					▥	ML			**ARTIFICAL FILL:** Clay sand silt w/ roots, moist, stiff, mod yel bn
			■	10/21/43	▥	SM	116.2	7.2	**ARTIFICIAL FILL:** Slightly clayey silty sand with trace roots, slightly moist, dense, mottled dark yellowish orange
5			■	28/40/50	▥	ML	97.8	23.3	**SAUGUS FM.:** Siltstone with caliche veins, slightly moist, hard, moderate yellowish brown
			■	12/28/33	▥	ML / CL	108.0	15.1	**SAUGUS FM.:** Sandy clayey silt, moist, hard, moderate yellowish brown
10			■	9/11/16	▥	SM	95.2	15.2	**SAUGUS FM.:** Silty sand with trace gravels, moist, medium dense, moderate yellowish brown
15			■	12/13/26	░	SP	101.1	13.2	**SAUGUS FM.:** Slightly silty sand with trace gravels, dry, medium dense, very pale orange
20			■	11/14/18	░	SP	97.3	23.2	**SAUGUS FM.:** Slightly silty sand with trace gravels, dry, medium dense, very pale orange
25									**TOTAL DEPTH: 21.5 Feet** **Groundwater Was Not Encountered**
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.

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-1 @ 0-5’	B-3 @ 0-5’
USCS	SM	SM
MAXIMUM DENSITY (pcf)	--	122.0
OPTIMUM MOISTURE (%)	--	11.0
COHESION (psf)	--	250* 110**
ANGLE OF INTERNAL FRICTION	--	33°* 31°**
EXPANSION INDEX	--	0
RESISTANCE (“R”) VALUE	29	--
pH	--	9.3
SOLUBLE CHLORIDES (mg/Kg)	--	13
RESISTIVITY (ohms-cm)	--	6,000
SOLUBLE SULFATES (mg/Kg)	--	61
GRAIN SIZE DISTRIBUTION (%)		
GRAVEL	--	0
SAND	--	58
SILT AND CLAY	--	42

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

File Number: 303275-001 Lab Number: 098211

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

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

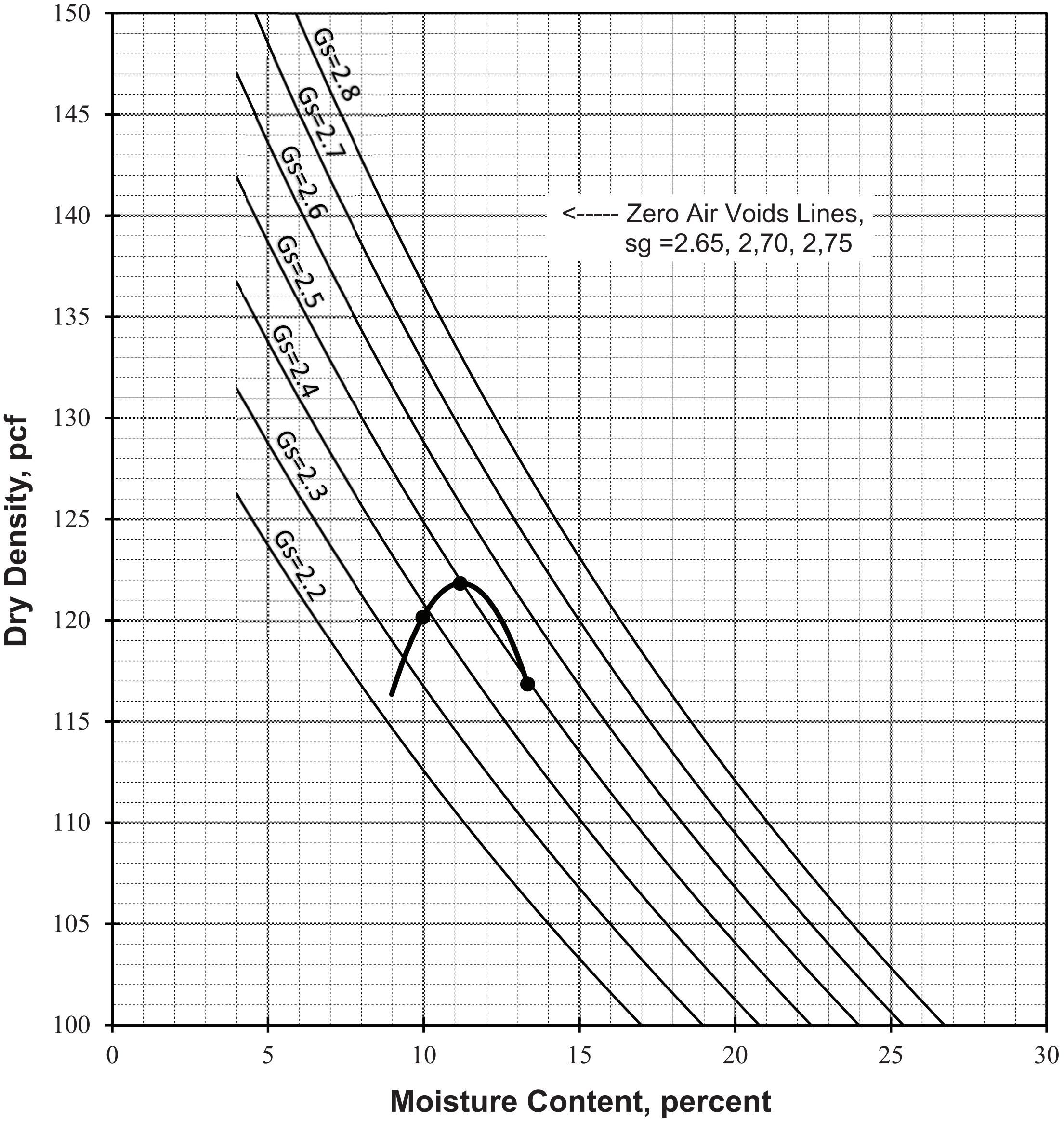
Procedure Used: A
Prep. Method: Moist

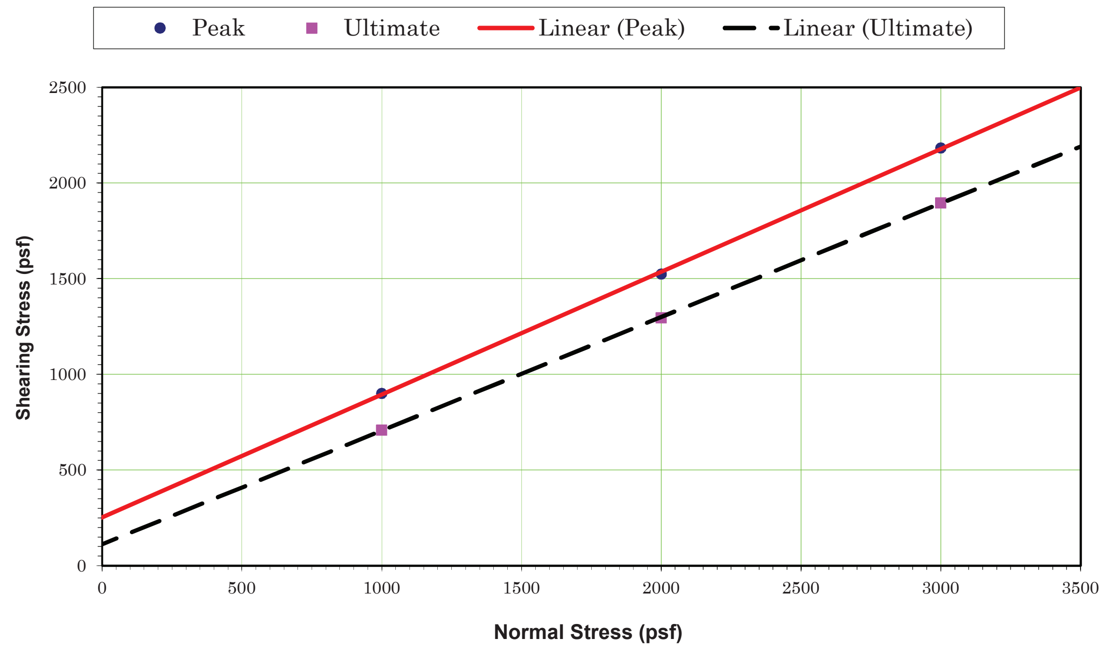
Date: 7/29/2019
Description: Dark Yellowish Brown Silty Sand
SG: 2.50

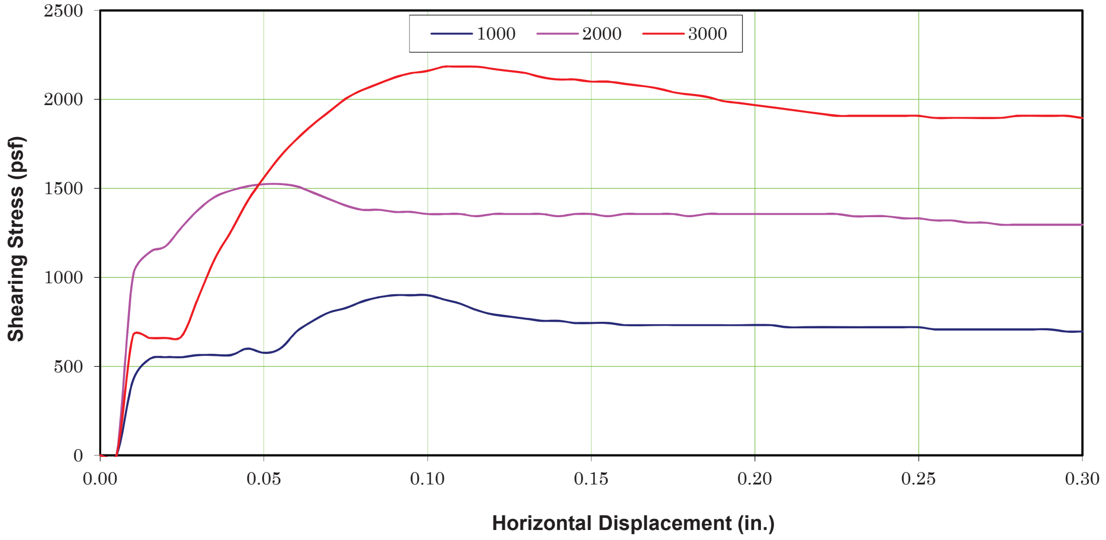
Rammer Type: Automatic

Maximum Density: **122 pcf**
Optimum Moisture: **11%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	0.1







DIRECT SHEAR DATA*

Sample Location: B 3 @ 0-5'
Sample Description: Silty Sand
Dry Density (pcf): 109.2
Intial % Moisture: 11.2
Average Degree of Saturation: 91.3
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	900	1524	2184
Ultimate stress (psf)	708	1296	1896

	Peak	Ultimate
ϕ Angle of Friction (degrees):	33	31
c Cohesive Strength (psf):	250	110

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Camarillo High School Synthetic Turf Field

Earth Systems

7/31/2019 | 303275-001

File No.: 303275-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

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

Initial Moisture, %:	9.0
Initial Compacted Dry Density, pcf:	113.1
Initial Saturation, %:	50
Final Moisture, %:	16.6
Volumetric Swell, %:	0.0

Expansion Index: **0** **Very Low**

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

Camarillo High School
Synthetic Turf Field

303275-001

RESISTANCE '*R*' VALUE AND EXPANSION PRESSURE

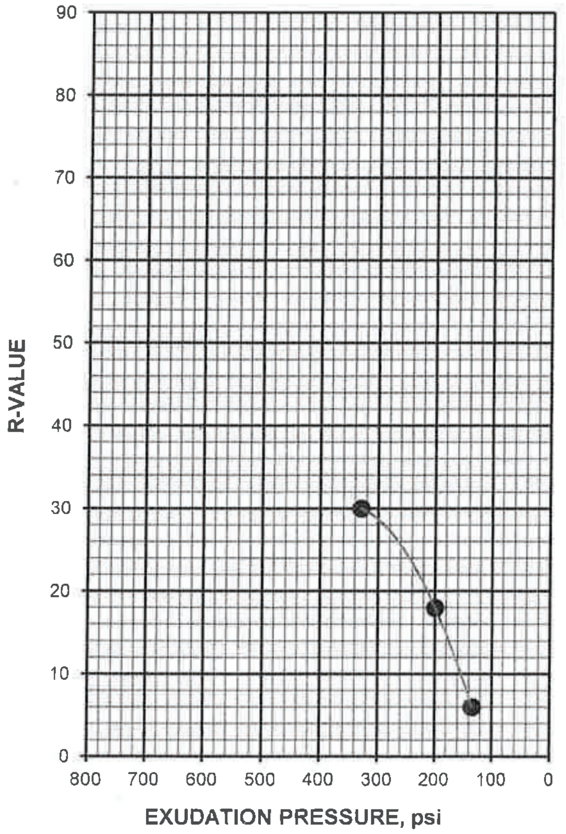
ASTM D 2844/D2844M-13

August 1, 2019

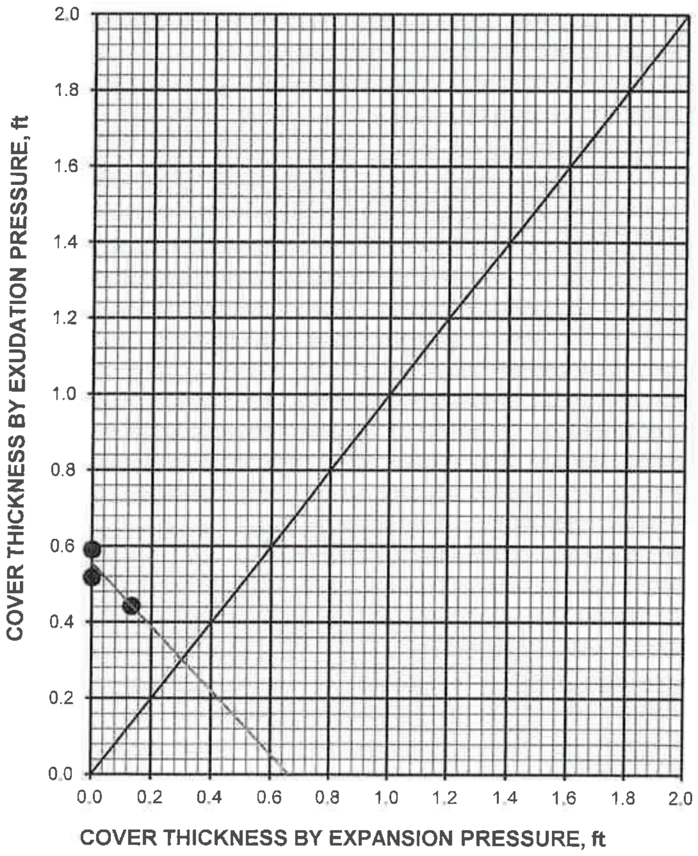
Boring #1 @ 1.0 - 5.0'
Brown Silty Sand (SM)
Specified Traffic Index: 5.0

Dry Density @ 300 psi Exudation Pressure: 120.6-pcf
%Moisture @ 300 psi Exudation Pressure: 14.9%
R-Value - Exudation Pressure: 29
R-Value - Expansion Pressure: 52
R-Value @ Equilibrium: 29

EXUDATION PRESSURE CHART



EXPANSION PRESSURE CHART

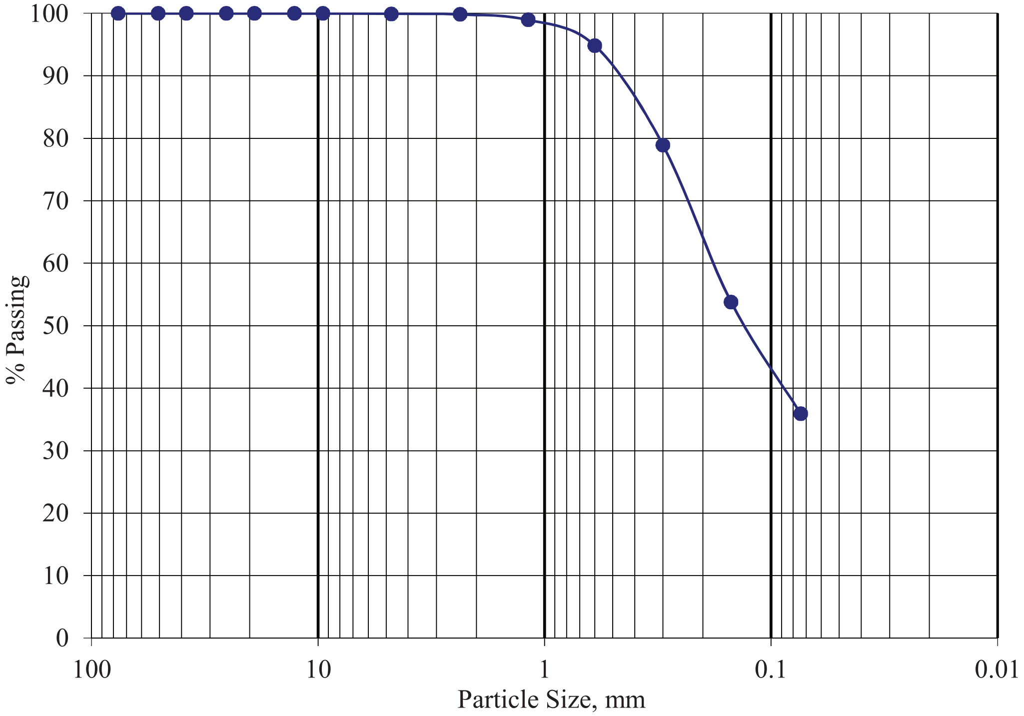


SIEVE ANALYSIS

ASTM C-136

Job Name: 303275-001
Sample ID: B 3 @ 0-5'
Description: SM

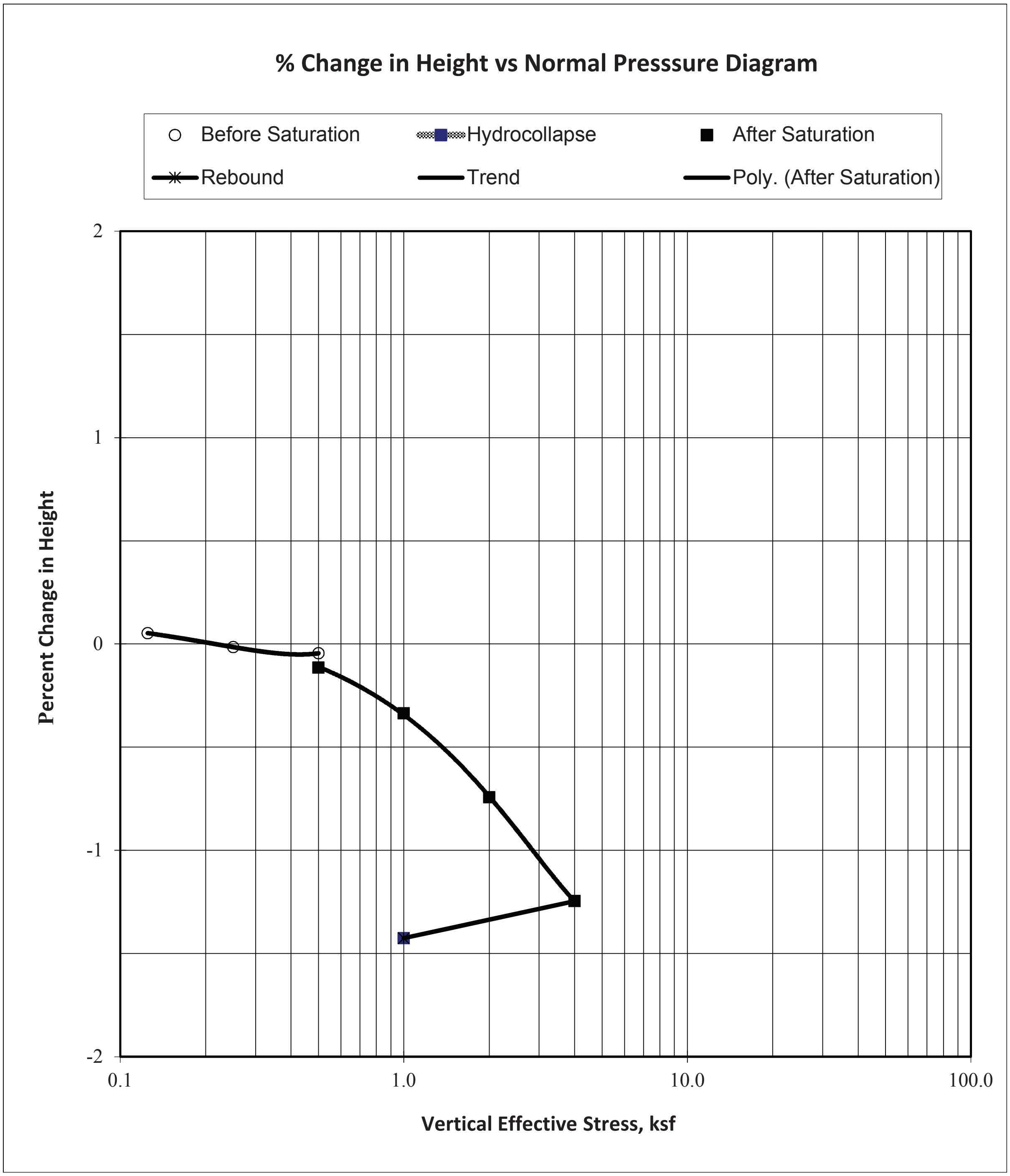
Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	100
#4	100
#8	100
#16	99
#30	95
#50	79
#100	54
#200	36



CONSOLIDATION TEST

ASTM D 2435-90

Camarillo High School Synthetic Turf Field		Initial Dry Density:	99.4 pcf
B 3 @ 5'		Initial Moisture, %:	4.5%
Sand		Specific Gravity:	2.67 (assume
Ring Sample		Initial Void Ratio:	0.677

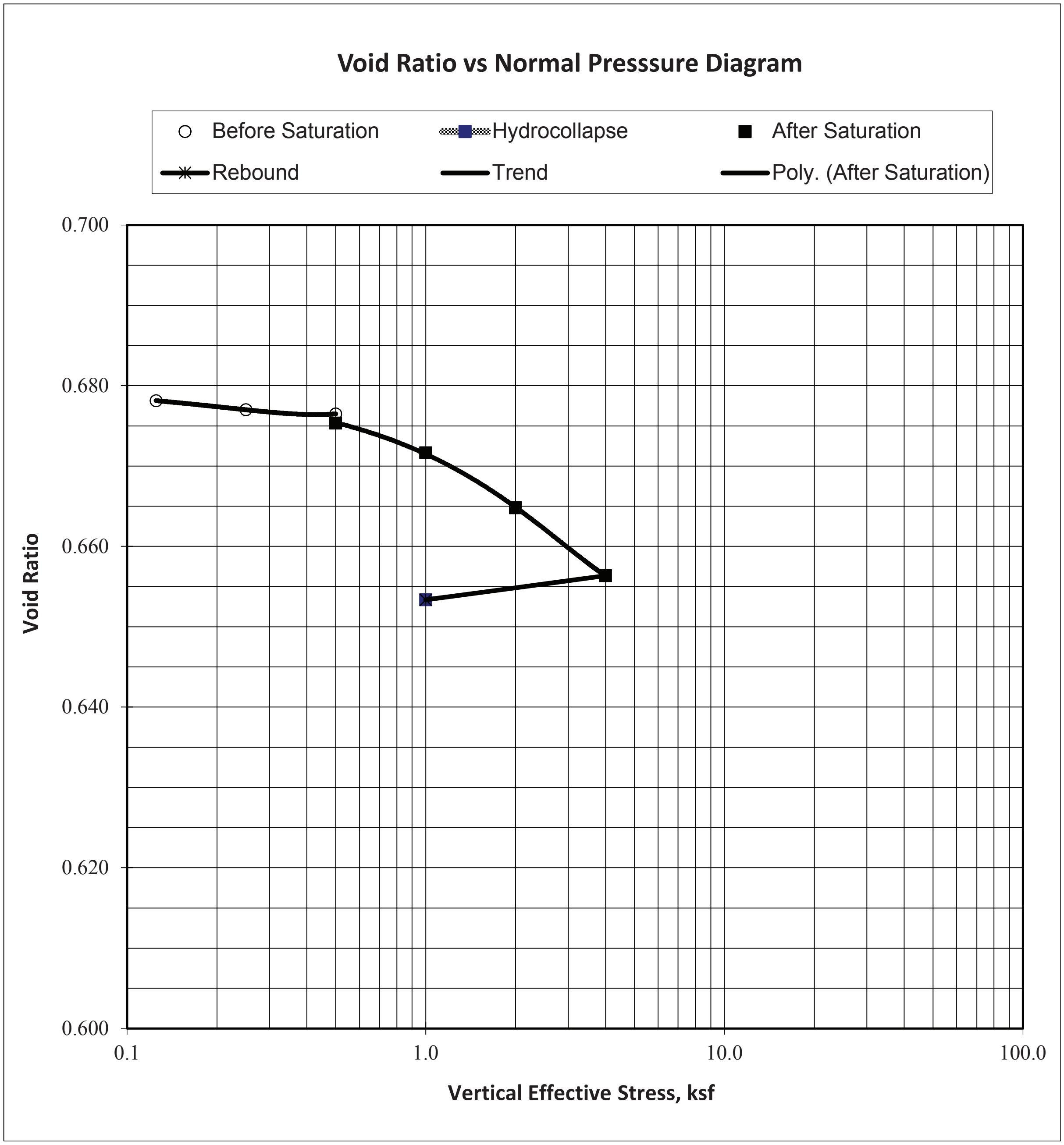


CONSOLIDATION TEST

ASTM D 2435-90

Camarillo High School Synthetic Turf Field
B 3 @ 5'
Sand
Ring Sample

Initial Dry Density:	99.4
Initial Moisture, %:	4.5
Specific Gravity:	2.67 (assume
Initial Void Ratio:	0.677



CERTIFICATE OF ANALYSIS

Client: Earth Systems Pacific
CAS LAB NO: 191288-01
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)	9.3	S.U.	1	---	9045	07/24/19
Resistivity*	6000	Ohms-cm	1	---	SM 120.1M	07/24/19
Chloride	13	mg/Kg	1	0.3	300.0M	07/24/19
Sulfate	61	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 UBC 18-1-D

FOUNDATIONS FOR STUD BEARING WALLS – MINIMUM REQUIREMENTS

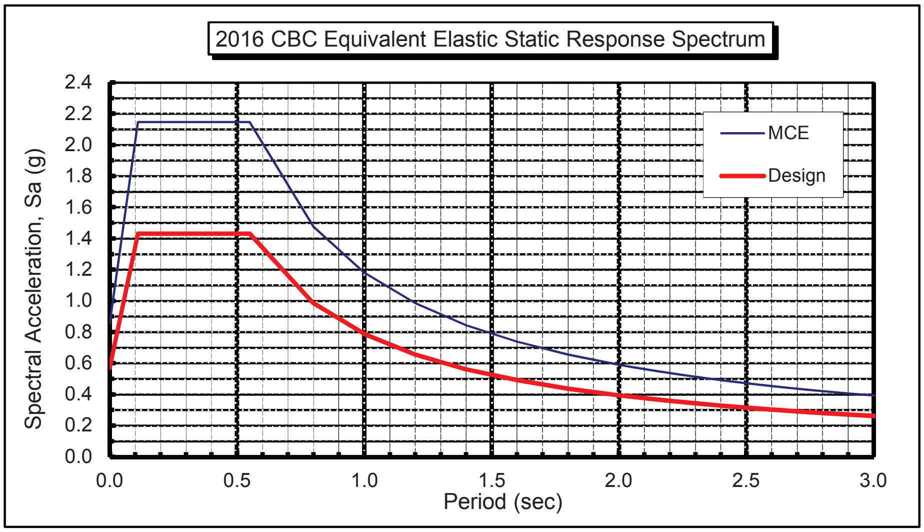
EXPANSION INDEX (E. I.)	FOUNDATIONS FOR SLAB AND RAISED FLOOR SYSTEM						REINFORCEMENT FOR FOUNDATIONS	CONCRETE SLAB		PREMOISTENING CONTROLS FOR SOILS UNDER FOOTINGS, PEIRS AND SLABS	PIERS UNDER RAISED FLOORS
	NUMBER OF STORIES	STEM THICKNESS	FOOTING WIDTH	FOOTING THICKNESS	ALL PERIMETER FOOTINGS	INTERIOR FOOTINGS FOR SLAB AND RAISED FLOORS		3-1/2" MINIMUM THICKNESS (4" WHEN OVER 51, E. I.)			
					DEPTH BELOW NATURAL SURFACE OF GROUND & FINISH GRADE			REINFORCEMENT	TOTAL THICKNESS OF SAND		
		INCHES									
0 -20 VERY LOW (NON-EXPANSIVE)	1	6	12	6	12	12	1 - #4 @ TOP AND BOTTOM	#3 @ 24" O.C. EACH WAY	2"	MOISTENING OF GROUND PRIOR TO PLACING CONCRETE IS RECOMMENDED	PIERS ALLOWED FOR SINGLE FLOOR LOADS ONLY
	2	8	15	7	18	18					
	3	10	18	8	24	24					
21-50 LOW	1	6	12	6	15	12	1 - #4 @ TOP AND BOTTOM	#3 @ 24" O.C. EACH WAY	4"	3% OVER OPTIMUM MOISTURE CONTENT TO A DEPTH OF 18" BELOW LOWEST ADJACENT GRADE TESTING REQ'D	PIERS ALLOWED FO SINGLE FLOOR LOADS ONLY
	2	8	15	7	21	18					
	3	10	18	8	24	24					
51-90 MEDIUM	1	6	12	6	21	12	1 - #4 @ TOP AND BOTTOM	#3 @ 24" O.C. EACH WAY	4"	3% OVER OPTIMUM MOISTURE CONTENT TO A DEPTH OF 18" BELOW LOWEST ADJACENT GRADE TESTING REQ'D	PIERS NOT ALLOWED
	2	8	15	8	21	18					
	3	10	18	8	24	24					
							#3 BARS @ 24" O.C. 12" INTO FOOTING AND BENT 3' INTO SLAB				
91-130 HIGH	1	6	12	8	27	12	2 - #4 @ TOP AND BOTTOM	#3 @ 24" O.C. EACH WAY	4"	3% OVER OPTIMUM MOISTURE CONTENT TO A DEPTH OF 24" BELOW LOWEST ADJACENT GRADE TESTING REQ'D	PIERS NOT ALLOWED
	2	8	15	8	27	18					
	3	10	18	8	27	24					
							#3 BARS @ 24" O.C. 12" INTO FOOTING AND BENT 3' INTO SLAB				
ABOVE 130 VERY HIGH	REQUIRES SPECIAL DESIGN BY A STATE LICENSED SOILS PROFESSIONAL										

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:		-119.010	W		
Maximum Considered Earthquake (MCE) Ground Motion					
Short Period Spectral Reponse	$\mathbf{S_S}$	**2.146**	**g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	$\mathbf{S_1}$	**0.787**	**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}	2.146	g	$= F_a{*}S_S$	
	S_{M1}	1.181	g	$= F_v{*}S_1$	
Design Earthquake Ground Motion					
Short Period Spectral Reponse	$\mathbf{S_{DS}}$	**1.431**	**g**	$= 2/3{*}S_{MS}$	
1 second Spectral Response	$\mathbf{S_{D1}}$	**0.787**	**g**	$= 2/3{*}S_{M1}$	
	To	0.11	sec	$= 0.2{*}S_{D1}/S_{DS}$	
	Ts	0.55	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	0.572
0.05	0.962
0.11	1.431
0.55	1.431
0.80	0.984
1.00	0.787
1.20	0.656
1.40	0.562
1.60	0.492
1.80	0.437
2.00	0.394
2.20	0.358
2.40	0.328
2.60	0.303
2.80	0.281
3.00	0.262

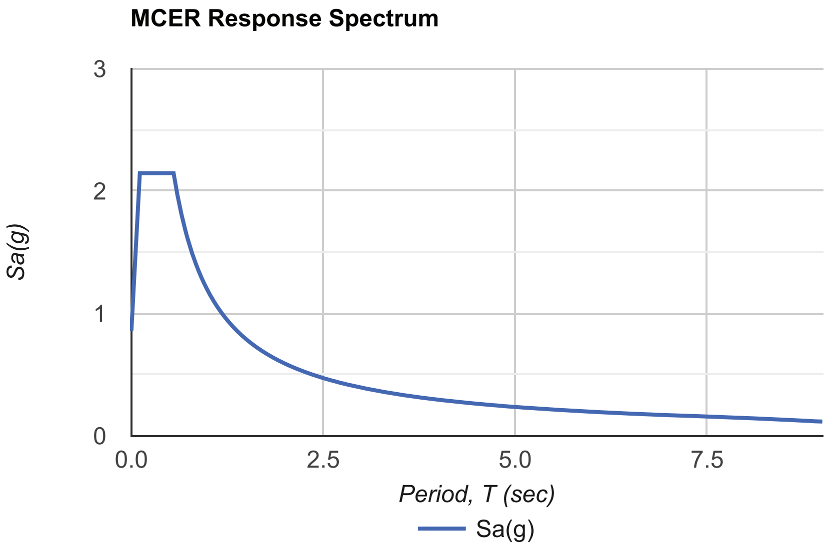
Camarillo High School Synthetic Turf Fields

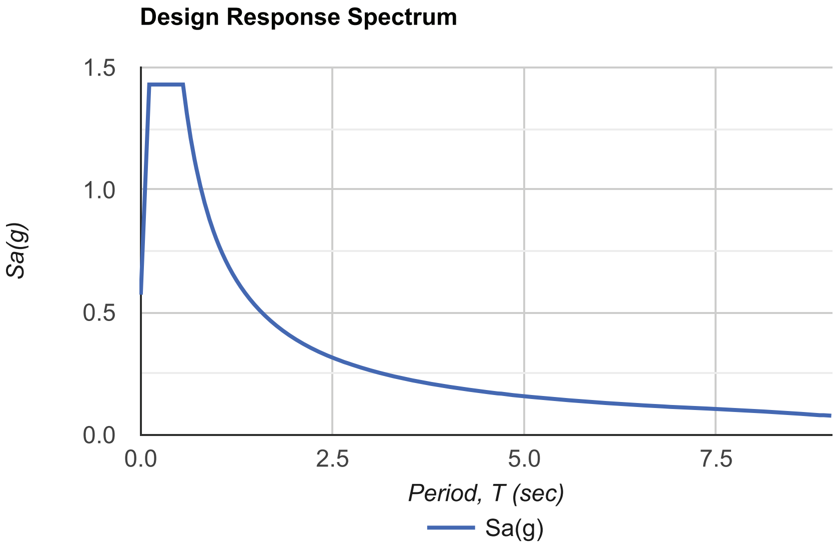
Latitude, Longitude: 34.2156, -119.0102

Date	7/31/2019, 10:16:28 AM
Design Code Reference Document	ASCE7-10
Risk Category	I
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.146	MCE_R ground motion. (for 0.2 second period)
S_1	0.787	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.146	Site-modified spectral acceleration value
S_{M1}	1.181	Site-modified spectral acceleration value
S_{DS}	1.43	Numeric seismic design value at 0.2 second SA
S_{D1}	0.787	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	0.809	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.809	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.2	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.28	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.146	Factored deterministic acceleration value. (0.2 second)
S1RT	0.787	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.811	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.814	Factored deterministic acceleration value. (1.0 second)
PGAd	0.822	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.965	Mapped value of the risk coefficient at short periods
C_{R1}	0.971	Mapped value of the risk coefficient at a period of 1 s


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


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

DISCLAIMER

Table 1
Fault Parameters

Fault Section Name	**Distance**		Avg Dip Angle	Avg Dip Direction	Avg Rake	Trace Length	Fault Type	**Mean Mag**	Mean Return Interval	Slip Rate
	(miles)	(km)	(deg.)	(deg.)	(deg.)	(km)			(years)	(mm/yr)
Simi-Santa Rosa	1.4	2.3	60	346	30	39	B	**6.8**		1
Oak Ridge (Onshore)	8.5	13.8	65	159	90	49	B	**7.2**		4
Malibu Coast (Extension), alt 1	11.1	17.9	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	11.1	17.9	74	4	30	35	B'	**6.9**		
Ventura-Pitas Point	11.6	18.6	64	353	60	44	B	**6.9**		1
Malibu Coast, alt 1	12.5	20.1	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	12.5	20.1	74	3	30	38	B	**6.9**		0.3
San Cayetano	14.2	22.9	42	3	90	42	B	**7.2**		6
Oak Ridge (Offshore)	15.2	24.5	32	180	90	38	B	**6.9**		3
Sisar	15.6	25.2	29	168	na	20	B'	**7.0**		
Anacapa-Dume, alt 1	17.0	27.3	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	17.0	27.3	41	352	60	65	B	**7.2**		3
Santa Susana, alt 1	17.1	27.4	55	9	90	27	B	**6.8**		5
Santa Susana, alt 2	17.4	28.0	53	10	90	43	B'	**6.8**		
Northridge Hills	18.3	29.5	31	19	90	25	B'	**7.0**		
Red Mountain	18.8	30.2	56	2	90	101	B	**7.4**		2
Channel Islands Thrust	19.5	31.4	20	354	90	59	B	**7.3**		1.5
Mission Ridge-Arroyo Parida-Santa Ana	19.5	31.4	70	176	90	69	B	**6.8**		0.4
Del Valle	20.8	33.5	73	195	90	9	B'	**6.3**		
Holser, alt 1	21.3	34.2	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	21.3	34.2	58	182	90	17	B'	**6.7**		
Santa Cruz Island	21.5	34.6	90	188	30	69	B	**7.1**		1
Shelf (Projection)	21.8	35.1	17	21	na	70	B'	**7.8**		
Northridge	22.0	35.4	35	201	90	33	B	**6.8**		1.5
San Pedro Basin	22.5	36.3	88	51	na	69	B'	**7.0**		
Santa Ynez (East)	23.2	37.4	70	172	0	68	B	**7.2**		2
Santa Monica Bay	24.2	39.0	20	44	na	17	B'	**7.0**		
North Channel	24.7	39.8	26	10	90	51	B	**6.7**		1
Channel Islands Western Deep Ramp	24.8	39.9	21	204	90	62	B'	**7.3**		
Pine Mtn	25.1	40.4	45	5	na	62	B'	**7.3**		
Compton	26.5	42.7	20	34	90	65	B'	**7.5**		
Pitas Point (Lower)-Montalvo	26.9	43.2	16	359	90	30	B	**7.3**		2.5
Santa Monica, alt 1	29.0	46.6	75	343	30	14	B	**6.5**		1
San Gabriel	29.3	47.2	61	39	180	71	B	**7.3**		1
Santa Monica, alt 2	29.5	47.4	50	338	30	28	B	**6.7**		1
San Pedro Escarpment	29.6	47.6	17	38	na	27	B'	**7.3**		
Santa Cruz Catalina Ridge	30.0	48.3	90	38	na	137	B'	**7.3**		
Palos Verdes	31.0	49.8	90	53	180	99	B	**7.3**		3
Sierra Madre (San Fernando)	31.0	49.8	45	9	90	18	B	**6.6**		2
Pitas Point (Upper)	33.5	53.9	42	15	90	35	B	**6.8**		1

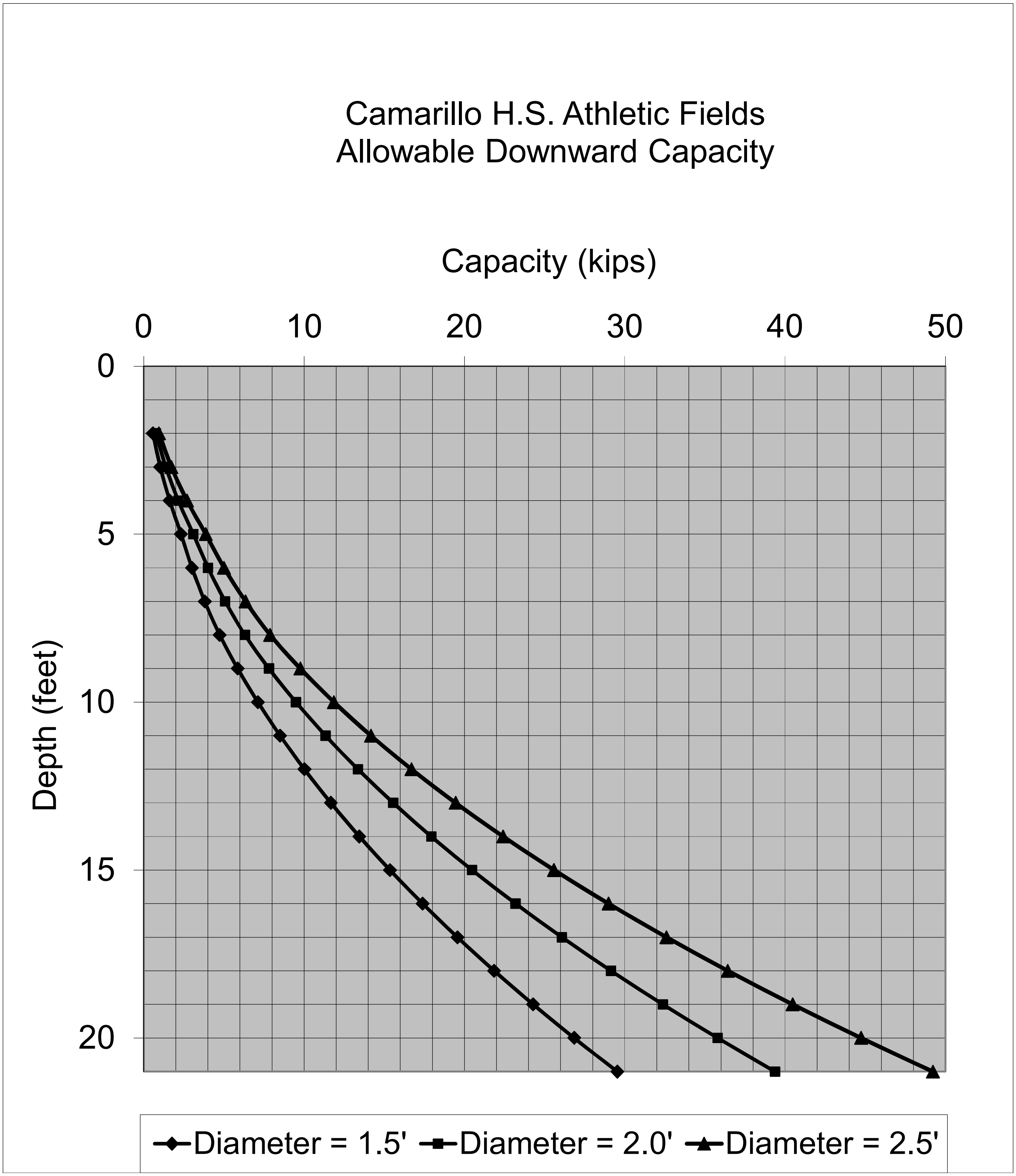
Reference: USGS OFR 2007-1437 (CGS SP 203)

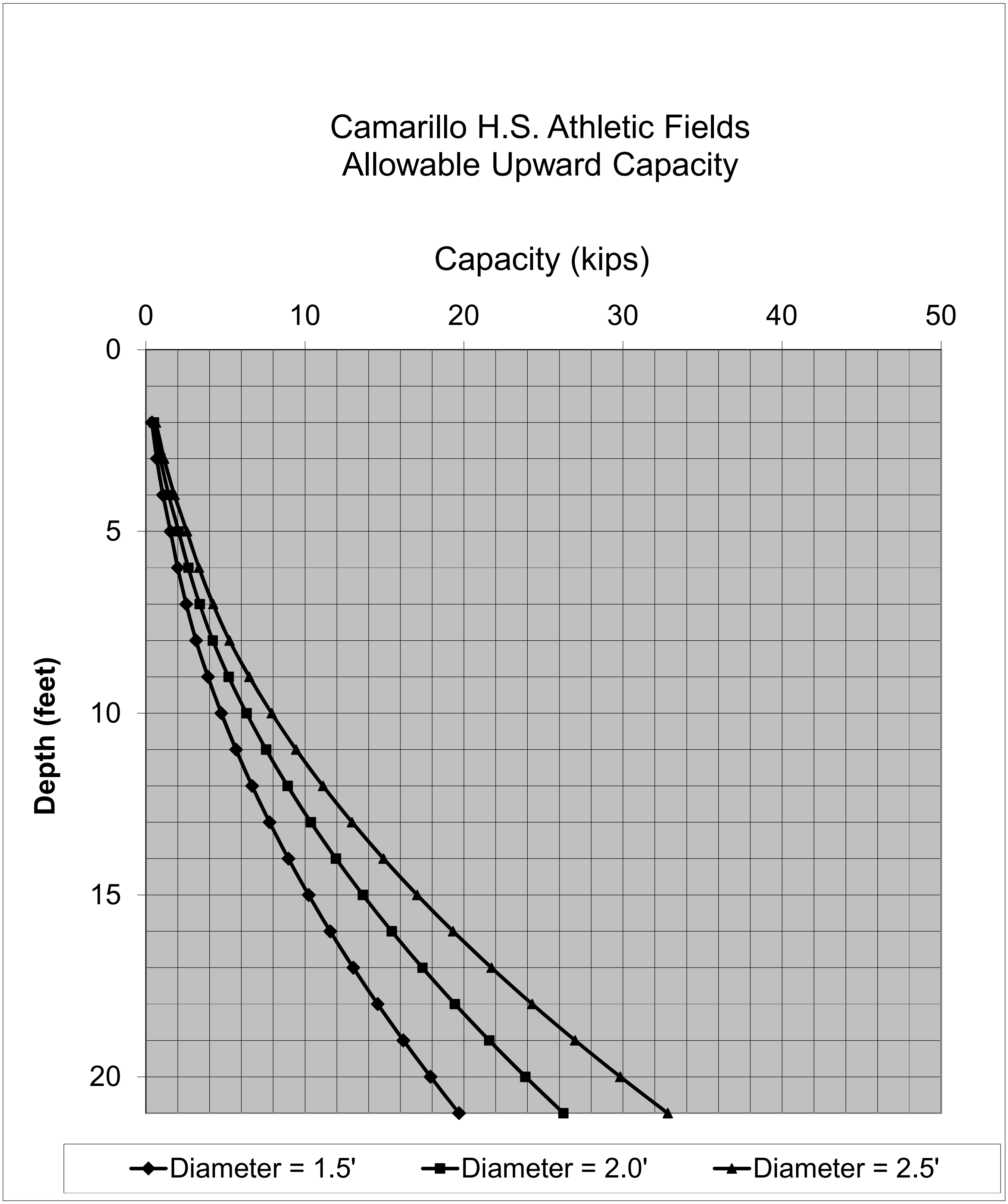
Based on Site Coordinates of 34.2156 Latitude, -119.0102 Longitude

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


Camarillo 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'



Camarillo H.S. Athletic Fields
Allowable Upward Capacity

APPENDIX E

Infiltration Test Data

INFILTRATION RATE BY THE BOREHOLE PERCOLATION TEST METHOD

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

Project Name	Camarillo High School Turf Field
Project Number	303275-001
Test Hole No.	P-1
Tester	A.Luna
Pre-Soak Date	8/22/2019
Test Date	8/23/2019

Test Hole Radius, r (inches)	4
Total Depth of Test Hole, D_T (feet)	7.0
Inside Diameter of Pipe, I (inches)	3.00
Outside Diameter of Pipe, O (inches)	3.25
Pipe Stick-Up (feet)	0.0
Porosity of Gravel, n	0.41
Porosity Correction Factor, C	0.48
Factor of Safety (FOS), F	N/A

Interval No.	Delta Time, Δt (min.)	Initial Depth to Water from TOP, D_o (ft.)	Final Depth to Water from TOP, D_f (ft.)	Initial Water Height, H_o (in.)	Final Water Height, H_f (in.)	Change in Water Height, ΔH (in.)	Perc Rate, (in/hr)	Infiltration Rate (in./hr.)	Corrected Infiltration Rate (in/hr)
1	10.00	5.00	5.60	24.00	16.80	7.20	43.20	3.86	1.86
2	10.00	5.00	5.60	24.00	16.80	7.20	43.20	3.86	1.86
3	10.00	5.00	5.55	24.00	17.40	6.60	39.60	3.49	1.69
4	10.00	5.00	5.55	24.00	17.40	6.60	39.60	3.49	1.69
5	10.00	5.00	5.52	24.00	17.76	6.24	37.44	3.27	1.58
6	10.00	5.00	5.52	24.00	17.76	6.24	37.44	3.27	1.58
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25									

INFILTRATION RATE BY THE BOREHOLE PERCOLATION TEST METHOD

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

Project Name	Camarillo High School Turf Field
Project Number	303275-001
Test Hole No.	P-2
Tester	A.Luna
Pre-Soak Date	8/22/2019
Test Date	8/23/2019

Test Hole Radius, r (inches)	4
Total Depth of Test Hole, D_T (feet)	18.0
Inside Diameter of Pipe, I (inches)	3.00
Outside Diameter of Pipe, O (inches)	3.25
Pipe Stick-Up (feet)	0.0
Porosity of Gravel, n	0.41
Porosity Correction Factor, C	0.48
Factor of Safety (FOS), F	N/A

Interval No.	Delta Time, Δt (min.)	Initial Depth to Water from TOP, D_o (ft.)	Final Depth to Water from TOP, D_f (ft.)	Initial Water Height, H_o (in.)	Final Water Height, H_f (in.)	Change in Water Height, ΔH (in.)	Perc Rate, (in/hr)	Infiltration Rate (in./hr.)	Corrected Infiltration Rate (in/hr)
1	22.00	15.85	16.00	25.80	24.00	1.80	4.91	0.36	0.18
2	30.00	16.00	16.15	24.00	22.20	1.80	3.60	0.29	0.14
3	30.00	16.00	16.15	24.00	22.20	1.80	3.60	0.29	0.14
4	30.00	16.00	16.14	24.00	22.32	1.68	3.36	0.27	0.13
5	30.00	16.00	16.15	24.00	22.20	1.80	3.60	0.29	0.14
6	30.00	16.00	16.15	24.00	22.20	1.80	3.60	0.29	0.14
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