GEOTECHNICAL ENGINEERING REPORT

FOR PROPOSED TICKET BOOTH AND GATEWAY
TO STADIUM COMPLEX AT
RIO MESA HIGH SCHOOL,
545 CENTRAL AVENUE,
OXNARD AREA, VENTURA COUNTY, CALIFORNIA

PROJECT NO.: 303514-002
NOVEMBER 14, 2019

PREPARED FOR
OXNARD UNION HIGH SCHOOL DISTRICT

BY

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November 14, 2019

Project No.: 303514-002
Report No.: 19-11-30

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

Project: Ticket Booth and Gateway to Stadium Complex
Rio Mesa High School
545 Central Avenue
Oxnard Area Ventura County, California

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

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

Respectfully submitted,

EARTH SYSTEMS PACIFIC

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

INTRODUCTION

This report presents results of a geotechnical engineering study performed for a proposed structure that will serve as a ticket booth and gateway to the athletic field complex at Rio Mesa High School in the Oxnard area of Ventura County, California (see Vicinity Map in Appendix A). Current plans indicate that the ticket booth will have a structural footprint of about 70 square feet, and will have attached 10-foot tall entry gates supported by steel tube columns on pier footings. The one-story ticket booth will be constructed with reinforced CMU block, and will utilize conventional foundation systems with a slab-on-grade floor. There will be 8-foot high freestanding reinforced CMU walls adjacent to the ticket booths at the entry gates.

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

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

PURPOSE AND SCOPE OF WORK

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

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

7. Preparing this report.

Contained in this report are:

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

GENERAL GEOLOGY

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

Although there are several faults located within the region, the nearest known fault of significant activity the Oak Ridge Fault is located approximately 0.9 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, and gravelly sands.

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

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

SEISMICITY AND SEISMIC DESIGN

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

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

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

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

Seismic design values are referenced to the Maximum Considered Earthquake (MCE) and, by definition, the MCE has a 2% probability of occurrence in a 50-year period. This equates to a return rate of 2,475 years. Spectral acceleration parameters that are applicable to seismic design are presented in Appendix C. It should be noted that the school project carries a seismic importance factor I of 1.25 and that factor has been incorporated into the 2013 and 2016 California Building Code response spectrums.

It is assumed that the 2016 CBC and ASCE 7-10 guidelines will apply for the seismic design parameters. The 2016 CBC includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The seismic design parameters presented herein were determined by the U.S. Seismic Design Maps "risk-targeted" calculator on the USGS website for the jobsite coordinates (34.2556˚ North Latitude and -119.1443° West Longitude). The calculator adjusts for Soil Site Class D, and for Occupancy (Risk) Category III (for public school structures). (A listing of the calculated 2016 CBC and ASCE 7-10 Seismic Parameters is presented below and in Appendix C.)

Summary of Seismic Parameters – 2016 CBC

Site Class (Table 20.3-1 of ASCE 7-10 with 2016 update)	D
Occupancy (Risk) Category	III
Seismic Design Category	E
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	2.803 g
Spectral Response Acceleration at 1 sec. – S_1	1.077 g
Site Coefficient – F_a	1.00
Site Coefficient – F_v	1.50
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.803 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.616 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.869 g
One Second Spectral Response – S_{D1}	1.077 g
Site Modified Peak Ground Acceleration - PGA_M	1.13 g
Values appropriate for a 2% probability of exceedance in 50 years	

Because the Seismic Design Category is "E", a site-specific seismic analysis must be performed in addition to the "general procedure". For the Site-Specific Analysis, the Short Period Spectral Response (S_{DS}) was found to be 1.495 g, and the 1 Second Spectral Response (S_{D1}) was found to be 1.148 g. Both the "site specific" and "general procedure yielded peak ground accelerations of 1.130 g.

The Fault Parameters table in Appendix C lists the significant "active" and "potentially active" faults within a radius of about 35 miles from the subject site. The distance between the site and the nearest portion of each fault is shown, as well as the respective estimated maximum earthquake magnitudes, and the deterministic mean site peak ground accelerations.

SOIL CONDITIONS

Evaluation of the subsurface indicates that soils are generally alluvium that consists of loose to very dense silty sands to sandy silts, fine to coarse sands, and gravelly sands. Near-surface soils encountered below the fields are generally characterized by high blow counts and in-place densities, and low compressibilities. However, near-surface soils encountered in Boring B-4 had low blow counts and in-place densities. Testing indicates that anticipated bearing soils lie in the “very low” expansion range because the expansion index equals 0. [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 not encountered during drilling. Mapping of historically high groundwater levels by the California Geological Survey (CGS, 2002a) indicates that groundwater has been about 25 feet below the ground surface near the subject site.

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

Samples of near-surface soils were tested for pH, resistivity, soluble sulfates, and soluble chlorides. The test results provided in Appendix B should be distributed to the design team for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils. It should be noted that sulfate contents (1,700 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 (810 ohms-cm) indicate that they are “severely corrosive” to ferrous metal (i.e. cast iron, etc.) pipes.

ANALYSIS OF LIQUEFACTION POTENTIAL

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

Earthquake-induced vibrations can be the cause of several significant phenomena, including liquefaction in fine sands and silty sands. Liquefaction results in a loss of strength and can cause structures to settle or even overturn if it occurs in the bearing zone. Liquefaction is typically limited to the upper 50 feet of soils underlying a site.

Fine sands and silty sands that are poorly graded and lie below the groundwater table are the soils most susceptible to liquefaction. Soils that have I_C values greater than 2.6, sufficiently dense soils, soils that have plasticity indices greater than 7, and/or soils located above the groundwater table are not generally susceptible to liquefaction.

An examination of the conditions existing at the site, in relation to the criteria listed above, indicates the following:

Groundwater was not encountered during the drilling performed for the current study, which included a boring advanced to a depth of 52 feet below the ground surface. Mapping by the California Geological Survey (CGS, 2002a) indicates that historical high groundwater levels have been about 25 feet below the ground surface near the subject site. As a result, this depth was utilized in the analysis.

A cyclic mobility analysis was performed to analyze the liquefaction potentials of the various soil layers at the proposed gateway location near the southwest corner of the stadium complex. The analysis was performed in general accordance with the methods proposed by NCEER (1997). The analyses used the calculated site-modified peak ground acceleration of 1.13 g, as per the discussion in the "Seismicity and Seismic Design" section of this report.

Exploration that was performed near the proposed gateway included Boring B-3 from the athletic fields studies of 2019 and a new boring (B-5) drilled on October 24, 2019. Data from those borings indicates that conditions in this area:

1. Soils are generally sands with variable, but usually minor quantities of gravels. As such, none of the soils encountered to a depth of 52 feet were considered to have plasticity or I_c values greater than 2.6.
2. Standard penetration tests conducted in the borings indicate that soils within the tested depth are in a fairly dense state.
3. Two soil zones were identified as being gravelly sands. Those zones were between depths of 17 and 25 feet, and between 27 and 29.5 feet. For the analysis, it was assumed that the lowest of the last two 6-inch blow counts would be doubled to be more conservative. Thus, where blow counts in the 17- to 25-foot zone were 9/15/20 and 12/19/27, the 15 blow count in the first set of numbers was doubled to 30 for use in the entire zone. For the zone between 27 and 29.5 feet, where the blow counts were 9/19/21, the 19 blow count was doubled to 38 blows.

The analysis indicated that all soil layers had factors of safety that exceeded 1.3 (see Appendix D for calculations). Those zones with factors of safety greater than 1.3 are not considered potentially liquefiable (C.G.S., 2008, and SCEC, 1999).

No settlement is predicted within those soils below a water table assumed to be at a depth of 25 feet. However, there is some potential for settlement of dry sands, as discussed below.

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

ANALYSIS OF SEISMIC-INDUCED SETTLEMENT OF DRY SANDS

Dry sands tend to settle and densify when subjected to earthquake shaking. The amount of settlement is a function of relative density, cyclic shear strain magnitude, and the number of strain cycles. Procedures to evaluate this type of settlement were developed by Seed and Silver (1972) and later modified by Pyke, et al. (1975). Tokimatsu and Seed (1987) presented a simplified procedure that has been reduced to a series of equations by Pradel (1998).

For this project, the Tokimatsu and Seed procedure, as implemented by Pradel, has been used to evaluate seismic-induced settlement at this site. Two-thirds of the site-modified peak ground acceleration of 1.13 g (i.e. 0.76 g) and an earthquake magnitude of 7.4 were used in the analysis. Calculations (see Appendix D) using this procedure, the stated seismic data, and the data presented in the report for Borings B-3 and B-5 indicate that seismically-induced settlement could be about 0.7 inches if groundwater levels are deeper than 52 feet, or 0.4 inches if groundwater is at a depth of 25 feet.

The effect of the estimated seismically-induced settlement at the ground surface should be minor aerial settlement. According to SCEC (1999), up to about half of the total settlement could be realized as differential settlement. As a result, differential settlement could range up to about 0.4 inches at the ground surface.

CONCLUSIONS AND RECOMMENDATIONS

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

GRADING RECOMMENDATIONS FOR TICKET BOOTH AND ENTRY GATE

Grading at a minimum should conform to the 2016 California Building Code, and with the recommendations of the Geotechnical Engineer during construction. Where the recommendations of this report and the cited section of the 2016 CBC are in conflict, the Owner should request clarification from the Geotechnical Engineer.

The existing ground surface should be initially prepared for grading by removing all vegetation, trees, large roots, debris, other organic material and non-complying fill. Organics and debris should be stockpiled away from areas to be graded, and ultimately removed from the site to prevent their inclusion in fills. Voids created by removal of such material should be properly backfilled and compacted. No compacted fill should be placed unless the underlying soil has been observed by the Geotechnical Engineer.

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 the 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 to provide lateral passive resistance against lateral loads. 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.

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

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

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

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

GEOTECHNICAL DESIGN PARAMETERS FOR TICKET BOOTH, GATEWAY, 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 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 more than 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.

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

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

Resistance to lateral loading may be provided by friction acting on the base of foundations. A coefficient of friction of 0.58 may be applied to dead load forces. This value does not include a factor of safety.

Passive resistance acting on the sides of foundation stems equal to 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.

<u>Drilled Pier Foundations</u>

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

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

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

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

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

Reduction in axial capacity due to group effects should be considered for piers spaced at 3 diameters on-center or closer.

All piers should be tied together laterally (in both directions) at the top with grade beams. The size, spacing, and reinforcing of grade beams should be determined by the Structural Engineer.

Lateral (horizontal) loads may be resisted by passive resistance of the soil against the piers. An equivalent fluid weight (EFW) of 380 psf per foot of penetration in the compacted fill (upper 5 feet) and an EFW of 400 pcf in the underlying firm native soils may be used for lateral load design. 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 5.5 feet below the final ground elevation based on commonly accepted engineering procedures (Lee, 1968). If 24-inch diameter piers are used, the “point of fixity” was estimated to be located at least 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 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.

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

Slabs-on-Grade

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

It is recommended that perimeter slabs (walks, patios, etc.) be designed relatively independent of footing stems (i.e. free floating) so foundation adjustment will be less likely to cause cracking. Because near-surface soils are in the “very low” expansion range, no sand or aggregate base will be necessary below sidewalks. 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 “very low” expansion range.

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

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

Retaining Walls

Conventional cantilever retaining walls backfilled with compacted on-site soils may be designed for active pressures of 44 pcf of equivalent fluid weight for well-drained, level backfill.

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

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

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

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

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

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

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

SETTLEMENT CONSIDERATIONS

Maximum static settlements of about one inch are anticipated for foundations and floor slabs designed as recommended. Differential settlement between adjacent load bearing members should be expected to range up to about one-half the total settlement.

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

Analyses of potential seismic-induced settlement of dry sand indicate that approximately 0.7 inches of settlement could occur near the proposed ticket booth and gateway as a result of a significant earthquake. Approximately one-half of this total (i.e. 0.4 inches) could potentially be experienced as differential settlement.

ADDITIONAL SERVICES

This report is based on the assumption that an adequate program of monitoring and testing will be performed by Earth Systems during construction to check compliance with the recommendations given in this report. The recommended tests and observations include, but are not necessarily limited to the following:

1. Review of the building and grading plans during the design phase of the project.
2. Observation and testing during site preparation, grading, placing of engineered fill, and foundation construction.
3. Consultation as required during construction.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analysis and recommendations submitted in this report are based in part upon the data obtained from the borings advanced within the site. The nature and extent of variations between and beyond the sounding and borings may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

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

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

In the event that any changes in the nature, design, or location of the structure and other improvements are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

This report is issued with the understanding that it is the responsibility of the Owner, or of his representative to ensure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

As the Geotechnical Engineers for this project, Earth Systems has striven to provide services in accordance with generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the exclusive use of the Client for the purposes stated in this document for the referenced project only. No third party may use or rely on this report without express written authorization from Earth Systems for such use or reliance.

It is recommended that Earth Systems be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems is not accorded the privilege of making this recommended review, it can assume no responsibility for misinterpretation of the recommendations contained herein.

SITE-SPECIFIC BIBLIOGRAPHY

Earth Systems Southern California, December 3, 2010, Engineering Geology and Geotechnical Engineering Report for Proposed Solar Array at Rio Mesa High School, 545 Central Avenue, Oxnard Area, Ventura County, California (Job No. VT-24499-01).

Earth Systems Pacific, August 27, 2019, Geotechnical Engineering Report for Proposed Improvements to Stadium Complex at Rio Mesa High School, 545 Central Avenue, Oxnard Area, Ventura County, California (Project No. 303280-001).

GENERAL BIBLIOGRAPHY

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

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

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

California Geological Survey (C.G.S.), 2003a, Seismic Hazard Zone Report for the Saticoy 7.5 Minute Quadrangle, Ventura County, California, Seismic Hazard Zone Report 066.

C.G.S., 2003b, State of California Seismic Hazard Zones, Saticoy Quadrangle, Official Map, February 14, 2003.

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

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

Dibblee, Jr., Thomas W., 1992, Geologic Map of the Saticoy Quadrangle, Ventura County, California, Dibblee Foundation Map No. DF-42.

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

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

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

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

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

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

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

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

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

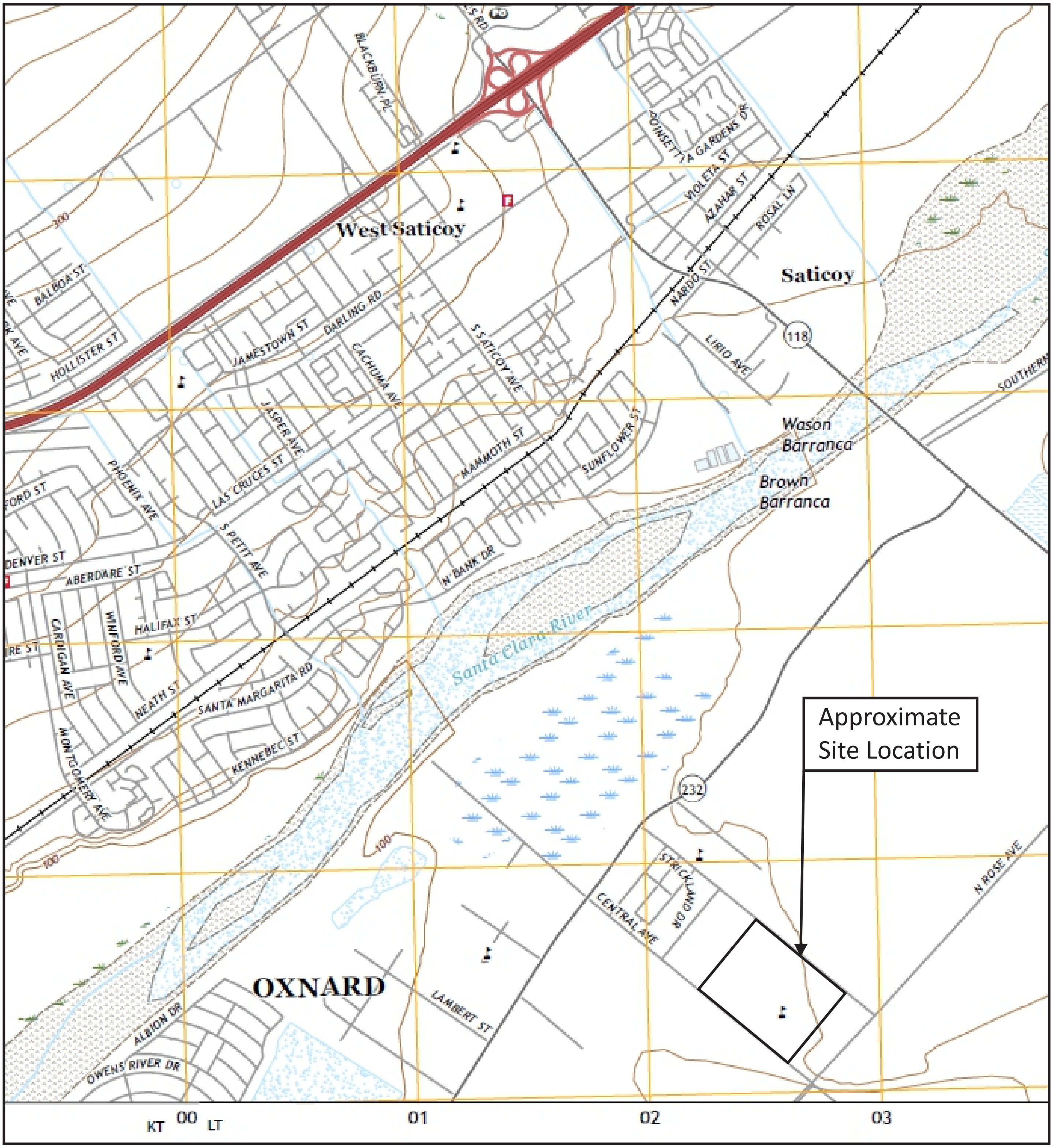
Ventura County Planning Department, October 22, 2013, Ventura County General Plan Hazards Appendix.

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

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

APPENDIX A

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, Saticoy Quadrangle, California, 2018.

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

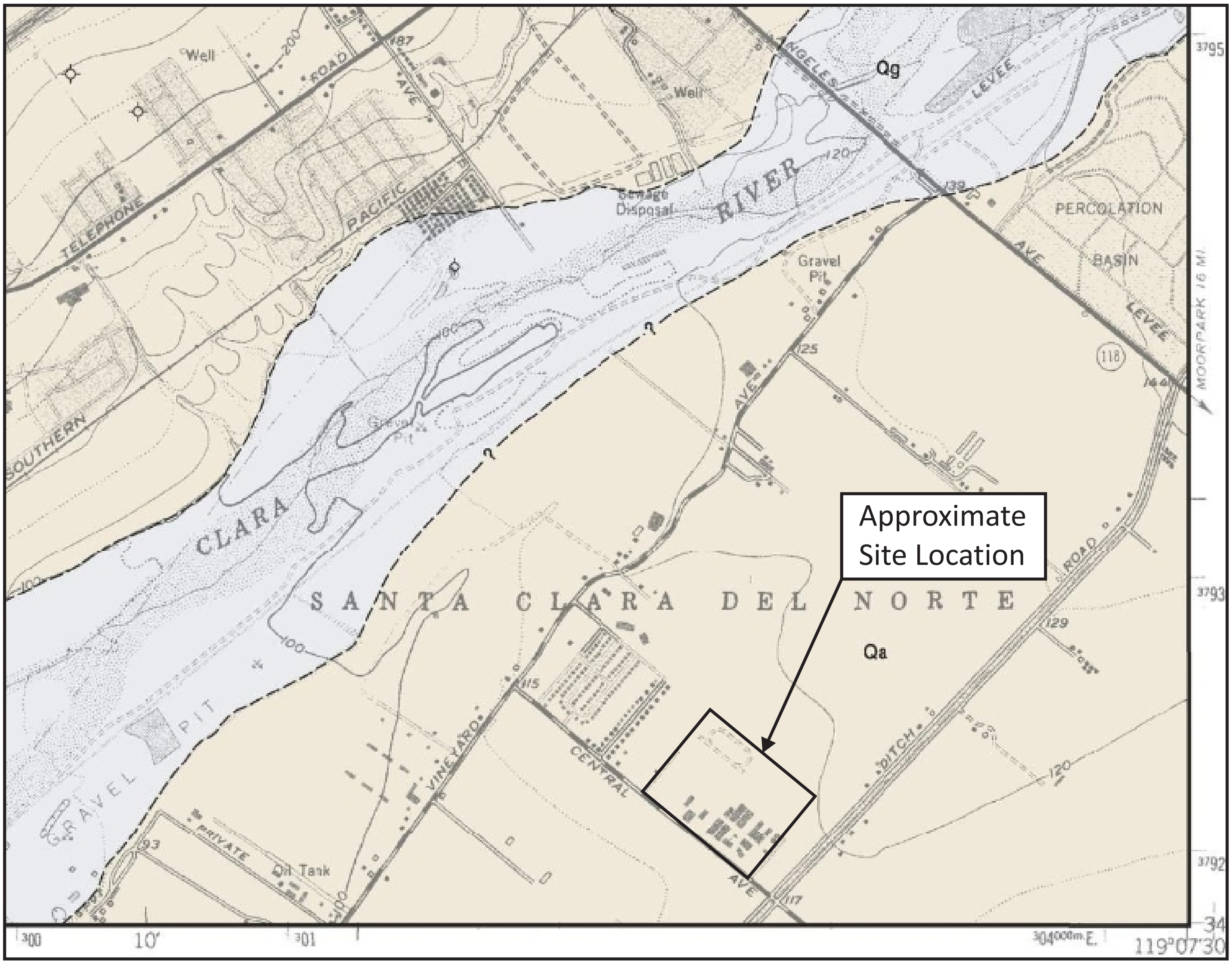
N

VICINITY MAP

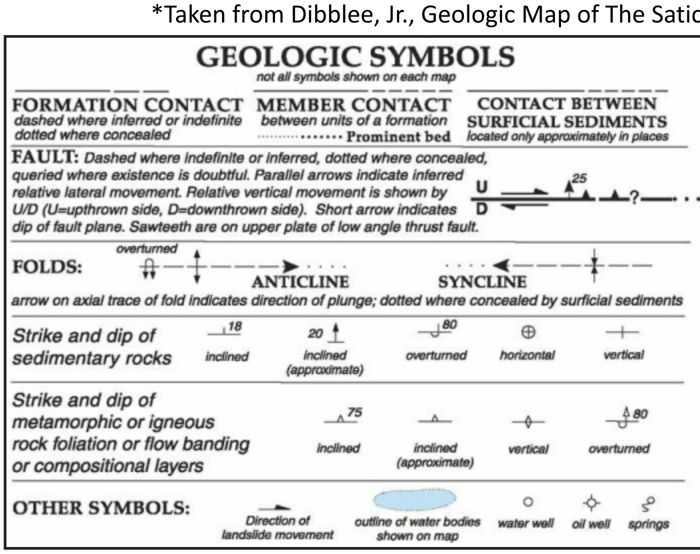
Rio Mesa High School Athletic Field Improvements
Oxnard, California

Earth Systems

November 2019	303514-001



*Taken from Dibblee, Jr., Geologic Map of The Saticoy Quadrangle, Ventura County, California, 1992, DF-42.



LEGEND

Qg	Qa

SURFICIAL SEDIMENTS

Unconsolidated alluvial deposits, generally undissected

Qg *Gravel, sand and silt of major stream channels*
Qa *Alluvium: silt, sand and gravel of valley and floodplain areas*

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

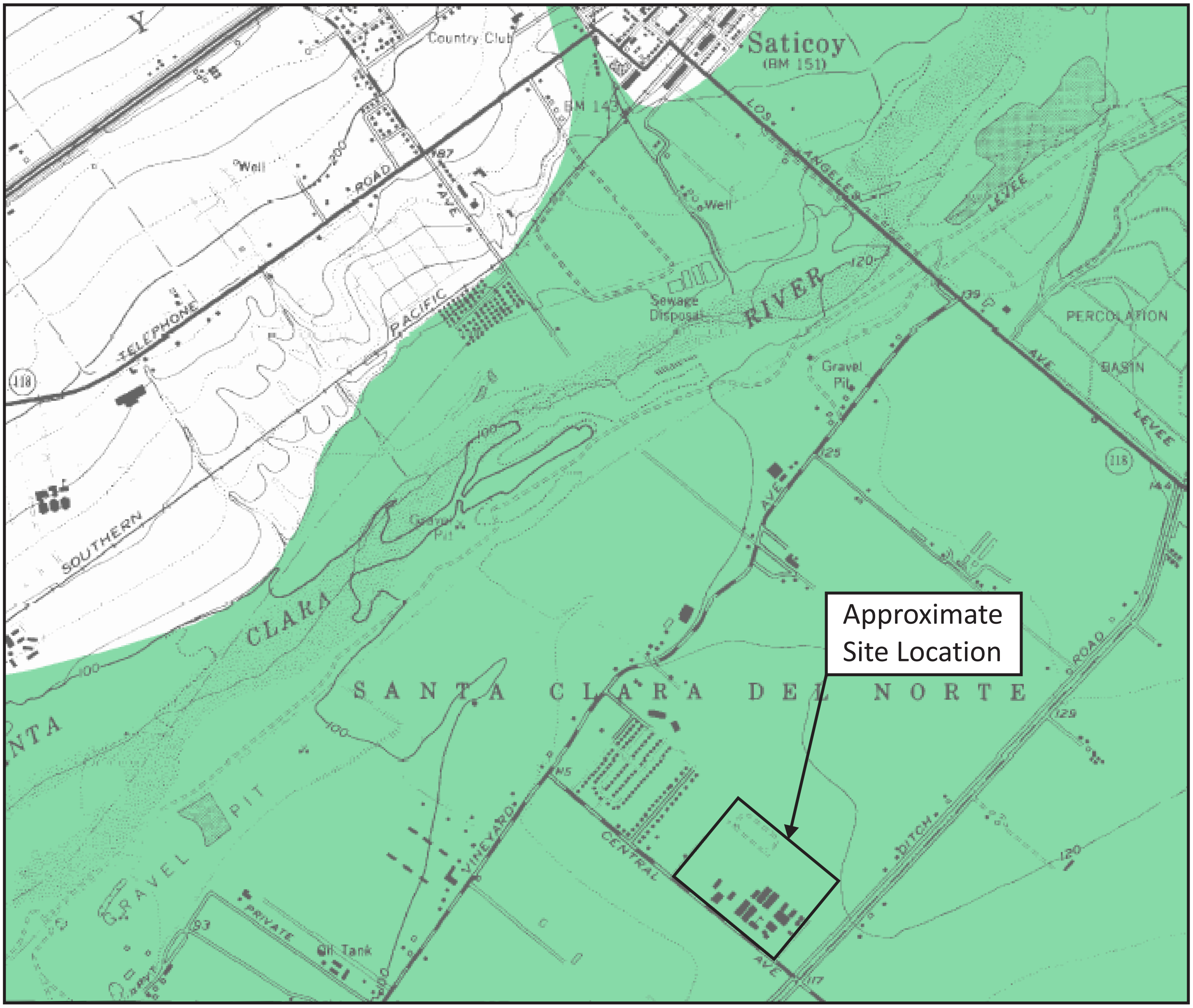
N

REGIONAL GEOLOGIC MAP

Rio Mesa High School Athletic Field Improvements
Oxnard, California

Earth Systems

November 2019	303514-002



MAP EXPLANATION

Zones of Required Investigation:

Liquefaction

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

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)

SATICOY QUADRANGLE

OFFICIAL MAP

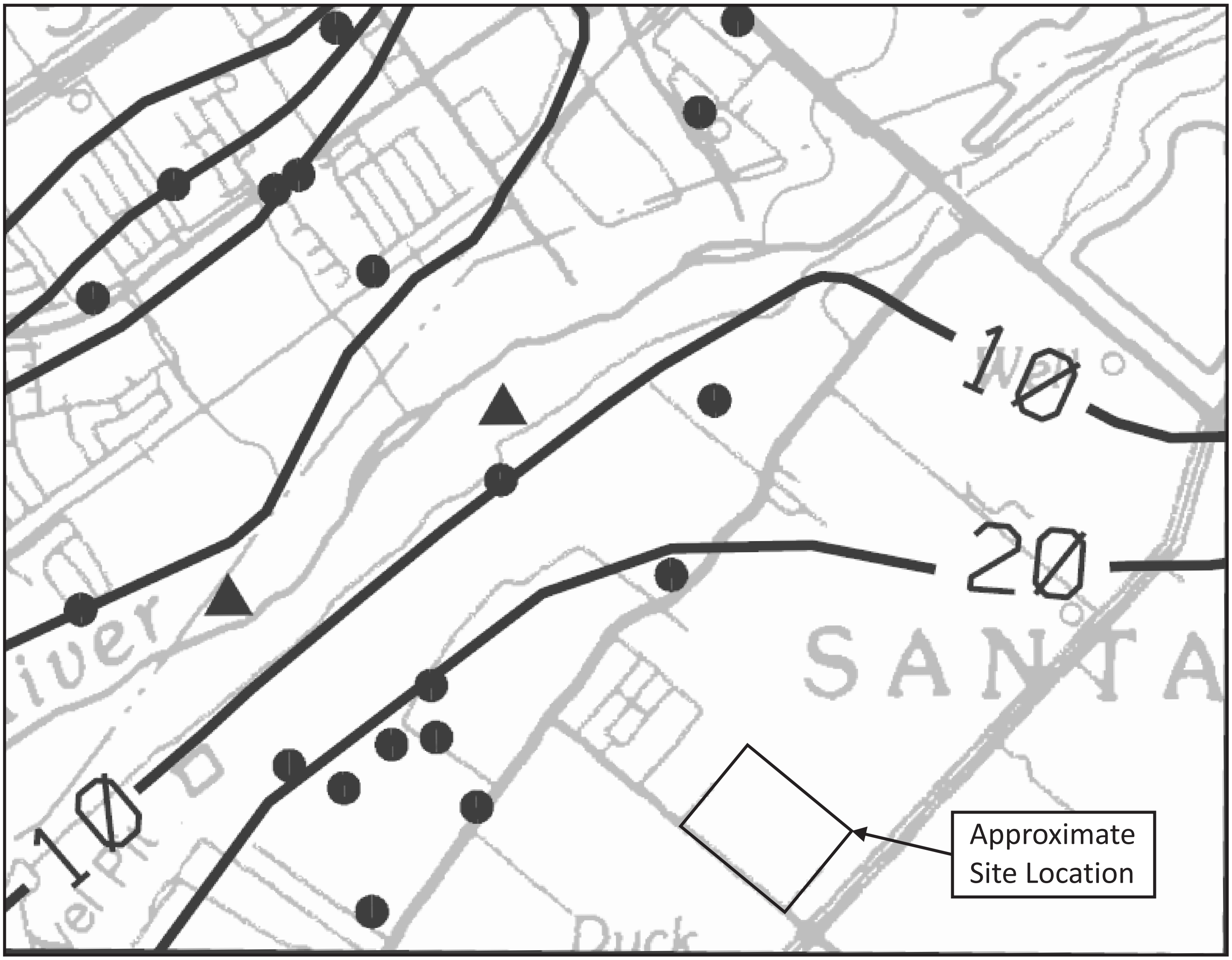
Released: February 14, 2003

SEISMIC HAZARD ZONES MAP

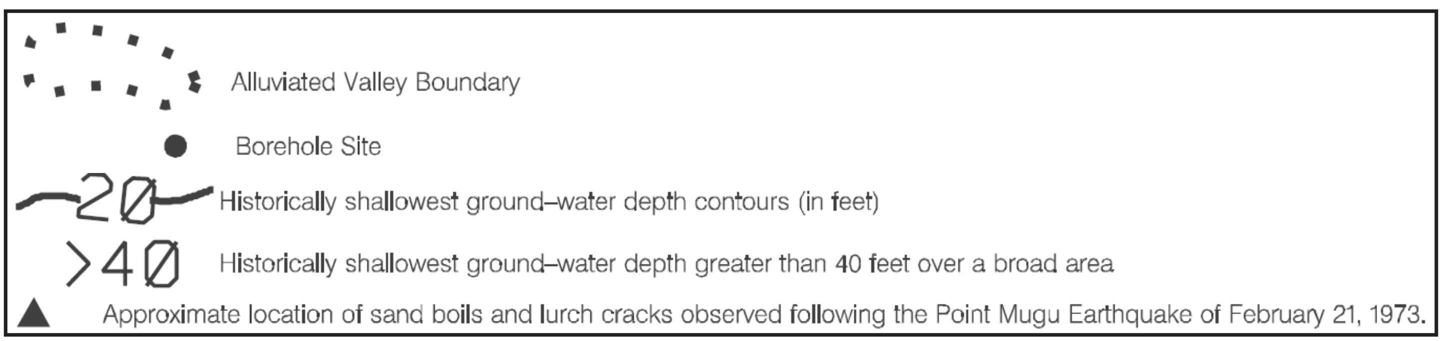
Rio Mesa High School Athletic Field Improvments
Oxnard, California

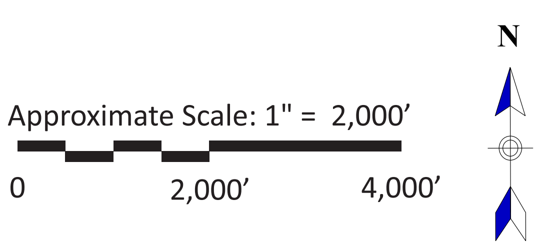
Earth Systems

November 2019	303514-002



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





HISTORICAL HIGH GROUNDWATER MAP

Rio Mesa High School Athletic Field Improvements
Oxnard, California

Earth Systems

November 2019	303514-002

FIELD STUDY

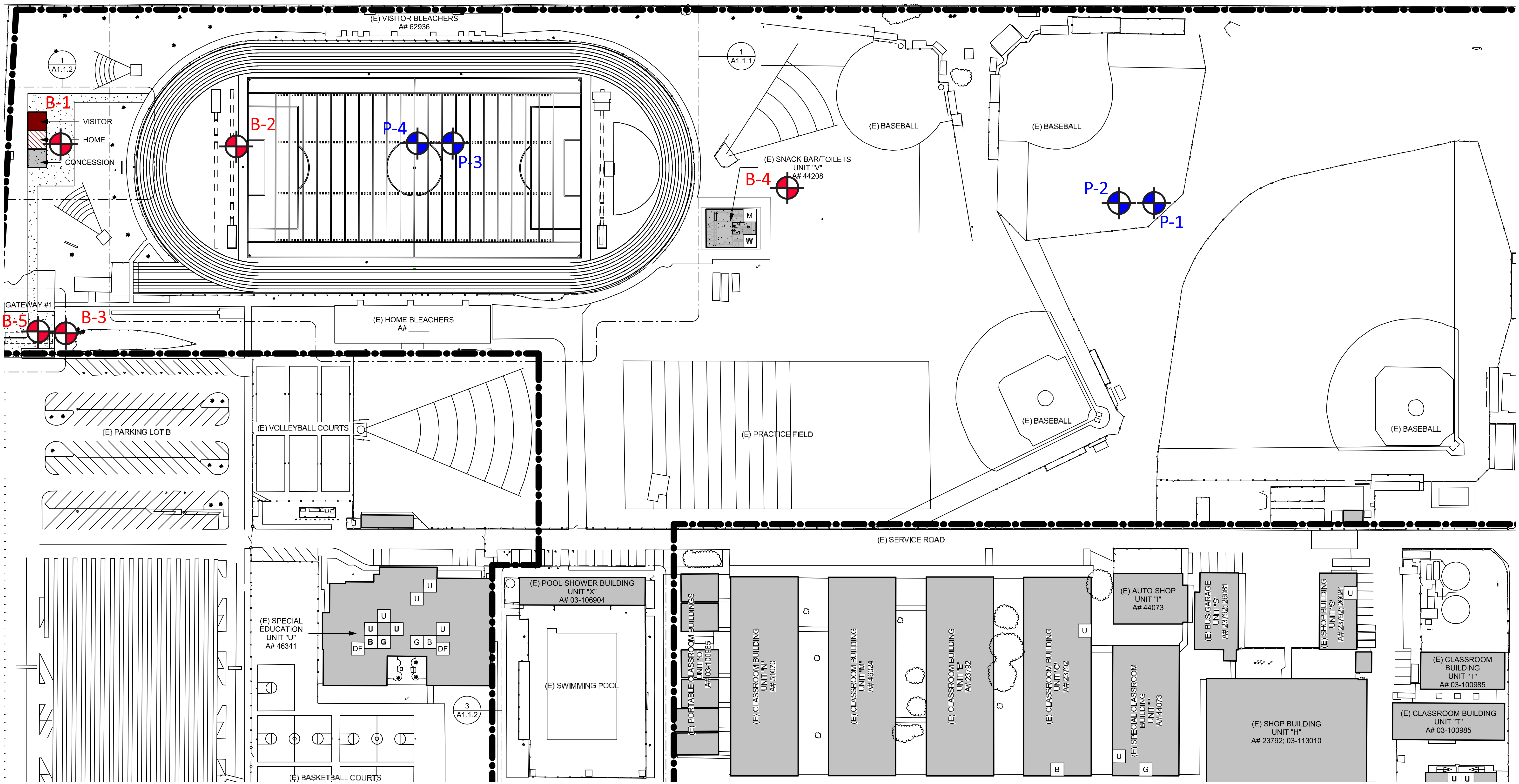
A. Four soil borings were drilled to a maximum depth of 10 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory analysis. In addition, 4 borings were drilled for infiltration testing to depths of 7 and 18 feet below existing ground surface. The borings 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. The first four borings were supplemented by an additional boring. The supplemental boring was drilled on October 24, 2019 using a 6-inch diameter hollow-stem auger powered by a GTech 8 drilling rig. The boring was advanced to a depth of 51.5 feet below the ground surface.

C. Samples were obtained within the test borings with a Modified California (M.C.) ring sampler (ASTM D 3550 with shoe similar to ASTM D 1586). 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.

D. Bulk samples of the soils encountered in the upper 5 feet of Borings B-2 and P-1 were gathered from the cuttings.

E. The final logs of the borings represent interpretations of the contents of the field logs and the results of laboratory testing performed on the samples obtained during the subsurface study. The final logs are included in this Appendix.



B-5 : Approximate boring location

P-4 : Approximate infiltration test location

N

Approximate Scale: 1" = 100'

0 100' 200'

SITE PLAN

Rio Mesa High School Athletic Field Improvements
Oxnard, California

Earth Systems

November 2019 | 303514-002

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

BORING NO: B-1
PROJECT NAME: 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	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
				5/18/18		SM	122.5	9.6	**ALLUVIUM:** Light Gray Silty fine to coarse Sand with Gravel, medium dense, dry to damp
5				18/32/18		SM / GM	124.2	5.4	**ALLUVIUM:** Light Gray SIlty fine to coarse Sand and Gravel, dense, dry to damp
				12/17/48		SW	98.1	6.0	**ALLUVIUM:** Light Yellow Brown fine to medium Sand, little 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-2	DRILLING DATE: June 27, 2019
PROJECT NAME: Rio Mesa High School Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303280-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: A. Luna

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0–5	X		■	25/30/25		SP	124.6	6.1	**ALLUVIUM:** Brown fine Sand, little medium Sand, trace Silt, trace fine to coarse Gravel, dense, dry to damp
5			■	10/7/6		SW-SM	121.8	10.7	**ALLUVIUM:** Dark Brown Slightly Silty fine to medium Sand, trace fine Gravel, loose, damp to moist
			■	6/14/16		SW	101.7	4.6	**ALLUVIUM:** Light Yellow Brown fine to 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-3
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	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
				9/16/20		SW	111.5	7.3	**ALLUVIUM:** Brown fine to coarse Sand, trace Silt, trace fine to coarse Gravel, medium dense, damp
5				10/12/14		SW			
				12/16/23		SW	102.6	4.5	**ALLUVIUM:** Light Yellow Brown fine to 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-4
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	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									
				4/4/4		ML			**ALLUVIUM:** Dark Brown fine Sandy Silt, little Clay, loose, moist
5				8/12/15		SW		5.9	**ALLUVIUM:** Light Yellow Brown fine to coarse Sand, little fine Gravel, trace coarse Gravel, medium dense, dry to damp
				5/14/16		SW	105.3	3.1	
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: October 24, 2019
PROJECT NAME: Rio Mesa HS	DRILL RIG: Gtech 8
PROJECT NUMBER: 303514-002	DRILLING METHOD: 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						SW			**ALLUVIUM:** Brown fine to coarse sand, trace Silt, trace fine to coarse Gravel, medum dense, damp
5									
10									
15		X		15/16/12		SW			**ALLUVIUM:** Light Gray Brown fine to medium Sand, little coarse Sand, little fine to coarse Gravel, trace Silt, medium dense, damp
		X		9/15/20		SW			**ALLUVIUM:** Light Gray Brown Gravelly fine to coase Sand, dense, dry to damp
20		X		12/19/27					
		X		12/18/20		SW			**ALLUVIUM:** Light Gray Brown fine to coarse Sand, little fine Gravel, trace Silt, dense, dry to damp
25		X		8/16/19		SP			**ALLUVIUM:** Gray Brown fine Sand, trace Silt, dense, dry to damp
		X		9/19/21		SW			**ALLUVIUM:** Dark Brown Gravelly fine to coarse Sand, dense, dry to damp
30		X		20/21/23		SP			**ALLUVIUM:** Light Gray Brown fine Sand, trace to little Silt, dense, damp
		X		12/18/14		SW			**ALLUVIUM:** Brown fine to coarse Sand, little fine to coarse Gravel, trace Silt, dense, damp
35		X		12/18/20		SW			**ALLUVIUM:** Light Gray Brown fine to medium Sand, little coarse Sand, little fine Gravel, dense, damp
		X		10/15/20					

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 (Continued)
PROJECT NAME: Rio Mesa HS
PROJECT NUMBER: 303514-002
BORING LOCATION: Per Plan

DRILLING DATE: October 24, 2019
DRILL RIG: Gtech 8
DRILLING METHOD: 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
40		X		14/19/21		SW			**ALLUVIUM:** Light Gray Brown fine to medium Sand, little coarse Sand, little fine Gravel, dense, damp
		X		13/20/21					
45		X		10/15/20		SP			**ALLUVIUM:** Brown fine Sand, trace to little Silt, dense to very dense, damp
		X		16/23/27					
50		X		15/23/30					
									Total Depth: 51.5 feet
									No Groundwater Encountered
55									
60									
65									
70									
75									

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

Earth Systems
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	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					SM			**ALLUVIUM:** Light Gray Brown Silty fine Sand, trace Clay, loose to medium dense, damp
5						SM			**ALLUVIUM:** Dark Brown Silty fine Sand, little Clay, medium dense, damp
10									Total Depth: 7 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-2	DRILLING DATE: June 27, 2019
PROJECT NAME: Rio Mesa High School Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303280-001	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						SM			**ALLUVIUM:** Light Gray Brown Silty fine Sand, trace Clay, medium dense, damp
5									
10						SW			**ALLUVIUM:** Brown fine to coarse Sand, trace fine Gravel, trace Silt, medium dense to dense, dry to damp
15						GW			**ALLUVIUM:** Brown Gravelly fine to coarse Sand, occasional Cobbles, very dense, dry to damp
20									Total Depth: 18 feet No Groundwater Encountered
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-3	DRILLING DATE: June 27, 2019
PROJECT NAME: Rio Mesa High School Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303280-001	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						SW-SM			**ALLUVIUM:** Dark Brown Silty fine to medium Sand, little Clay, medium dense, damp to moist
5						SW-SM			**ALLUVIUM:** Brown Silty fine to coarse Sand, little fine to coarse Gravel, medium dense, 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-4	DRILLING DATE: June 27, 2019
PROJECT NAME: Rio Mesa High School Synthetic Field	DRILL RIG: CME-75
PROJECT NUMBER: 303280-001	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						SW-SM			**ALLUVIUM:** Dark Brown Silty fine to medium Sand, little Clay, medium dense, moist
5						SW			**ALLUVIUM:** Brown fine to coarse Sand, little Silt, trace to little fine to coarse Gravel, medium dense, damp
10									
15									
									Total Depth: 18 feet No Groundwater Encountered
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	GRAVEL AND GRAVELLY SOILS	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
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	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	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
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNTOF FINES)		**SM**	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE				**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-2 @ 0-5’		P-1 @ 0-5’
USCS	SM		SM
MAXIMUM DENSITY (pcf)	128.0	131.0^	--
OPTIMUM MOISTURE (%)	9.0	8.5^	--
COHESION (psf)	320*	160**	--
ANGLE OF INTERNAL FRICTION	30°*	30°**	--
EXPANSION INDEX	0		--
RESISTANCE (“R”) VALUE	--		61
pH	8.0		--
SOLUBLE CHLORIDES (mg/Kg)	14		--
RESISTIVITY (ohms-cm)	810		--
SOLUBLE SULFATES (mg/Kg)	1,700		--
GRAIN SIZE DISTRIBUTION (%)			
GRAVEL	14		--
SAND	65		--
SILT AND CLAY	21		--

^ = Values Corrected for Oversized Material

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

File Number: 303280-001 Lab Number: 098207

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

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

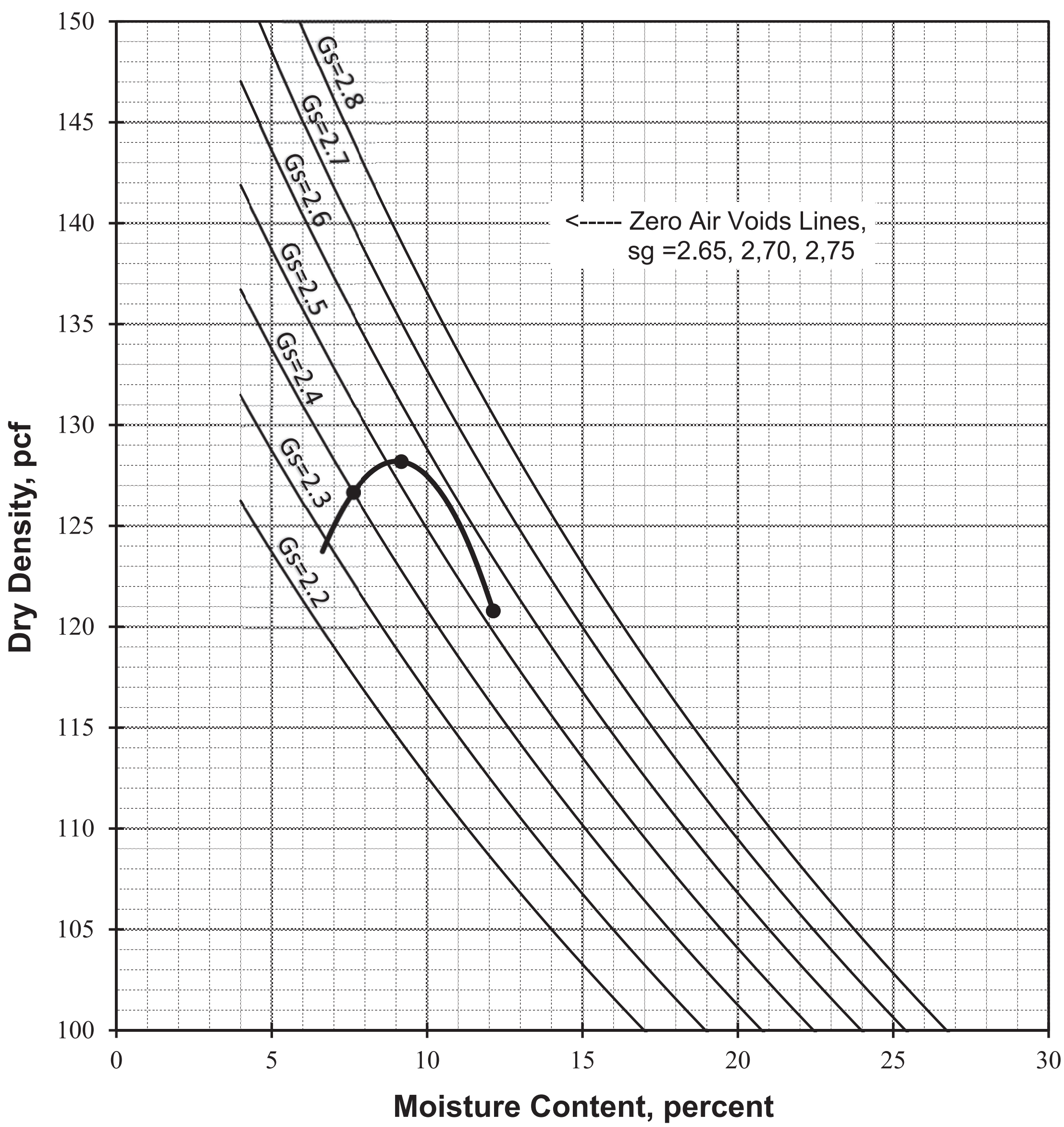
Procedure Used: B
Prep. Method: Moist

Date: 7/29/2019
Description: Very Dark Grayish Brown Silty Sand
SG: 2.52

Rammer Type: Automatic

Maximum Density: **128 pcf**
Optimum Moisture: **9%**

Sieve Size	% Retained
3/4"	0.0
3/8"	10.2
#4	0.0



File Number: 303280-001 Lab Number: 098207

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

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

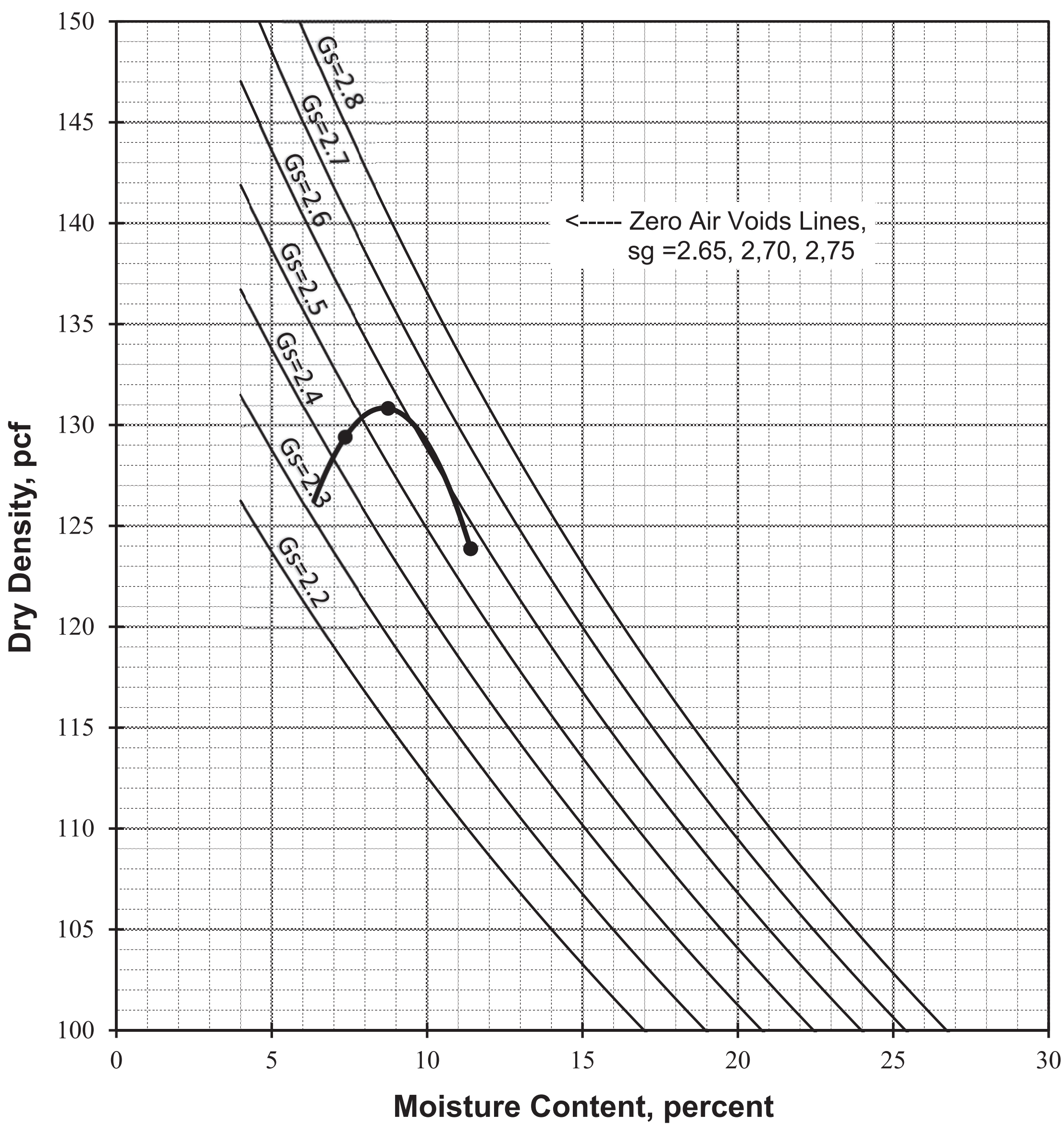
Procedure Used: B
Prep. Method: Moist

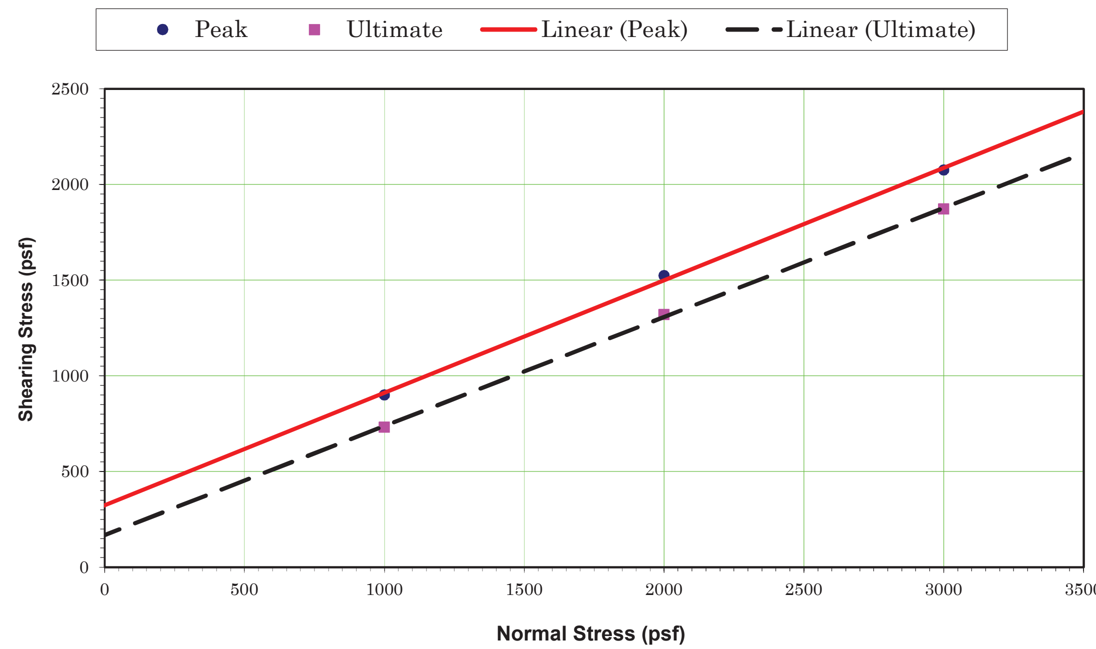
Date: 7/29/2019
Description: Very Dark Grayish Brown Silty Sand
SG: 2.56

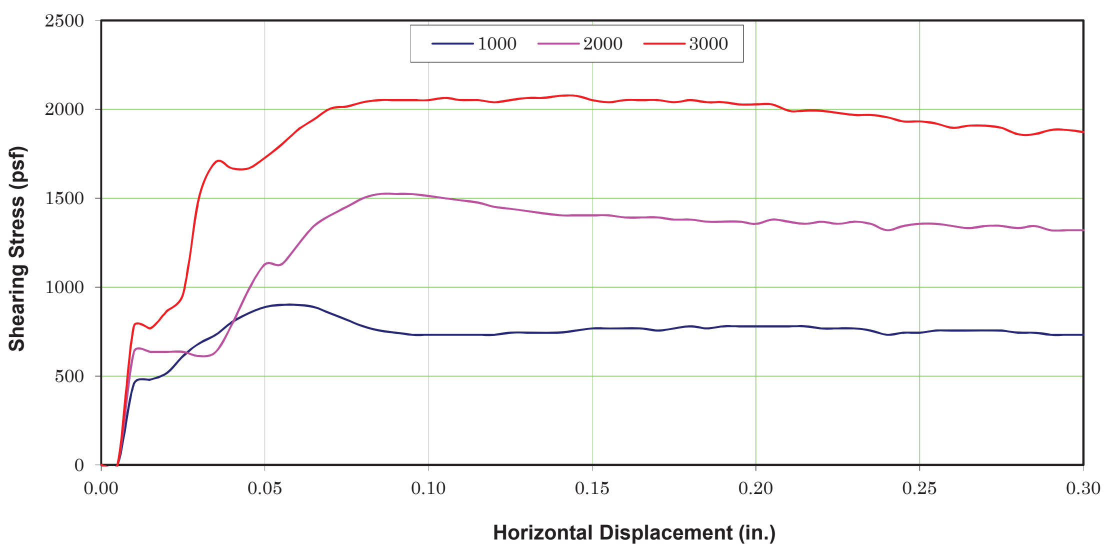
Rammer Type: Automatic

Maximum Density: **131 pcf**
Optimum Moisture: **8.5%**
Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained
3/4"	0.0
3/8"	10.2
#4	0.0







DIRECT SHEAR DATA*

Sample Location: B 2 @ 0-5'
Sample Description: Silty Sand
Dry Density (pcf): 115.2
Intial % Moisture: 9
Average Degree of Saturation: 89.6
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	900	1524	2076
Ultimate stress (psf)	732	1320	1872

	Peak	Ultimate
ϕ Angle of Friction (degrees):	30	30
c Cohesive Strength (psf):	320	160

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Rio Mesa High School Synthetic Turf Field

Earth Systems

8/27/2019	303280-001

File No.: 303280-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

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

Initial Moisture, %:	8.3
Initial Compacted Dry Density, pcf:	117.1
Initial Saturation, %:	52
Final Moisture, %:	14.6
Volumetric Swell, %:	0.0

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

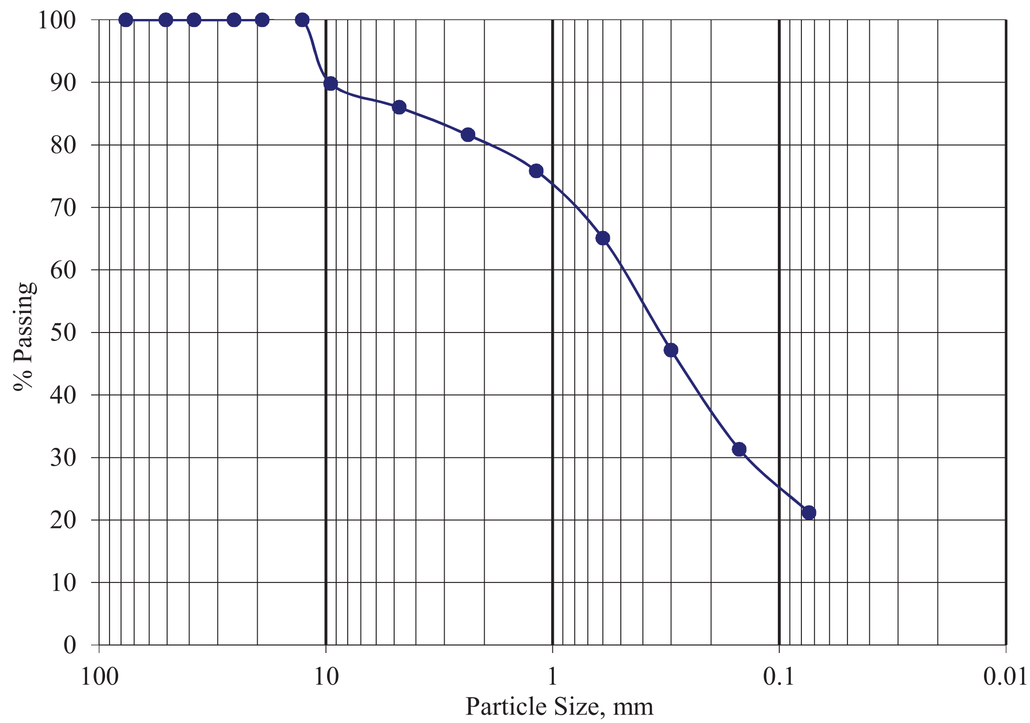
EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
130+	Very High

SIEVE ANALYSIS

ASTM C-136

Job Name: 303280-001
Sample ID: B 2 @ 0-5'
Description: SM

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	90
#4	86
#8	82
#16	76
#30	65
#50	47
#100	31
#200	21



Rio Mesa High School
Synthetic Turf Field

303280-001

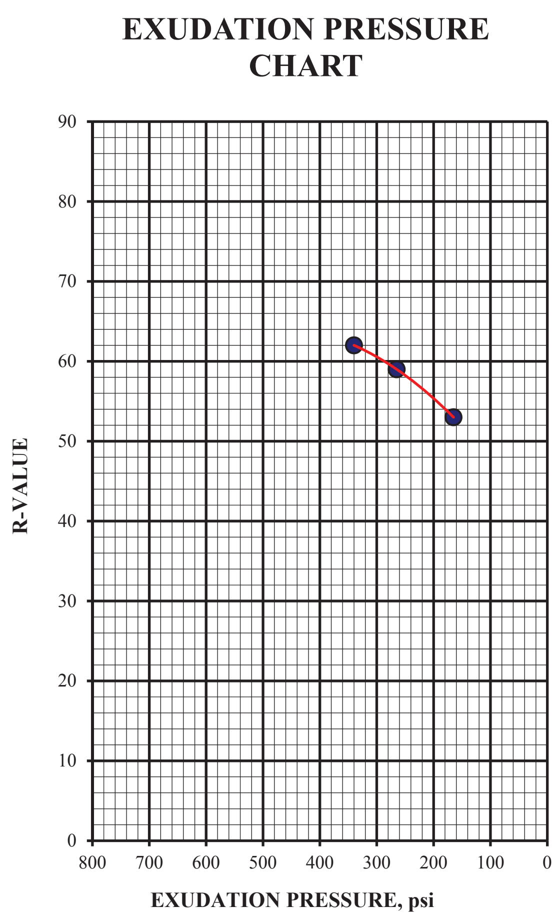
RESISTANCE '*R* ' VALUE AND EXPANSION PRESSURE

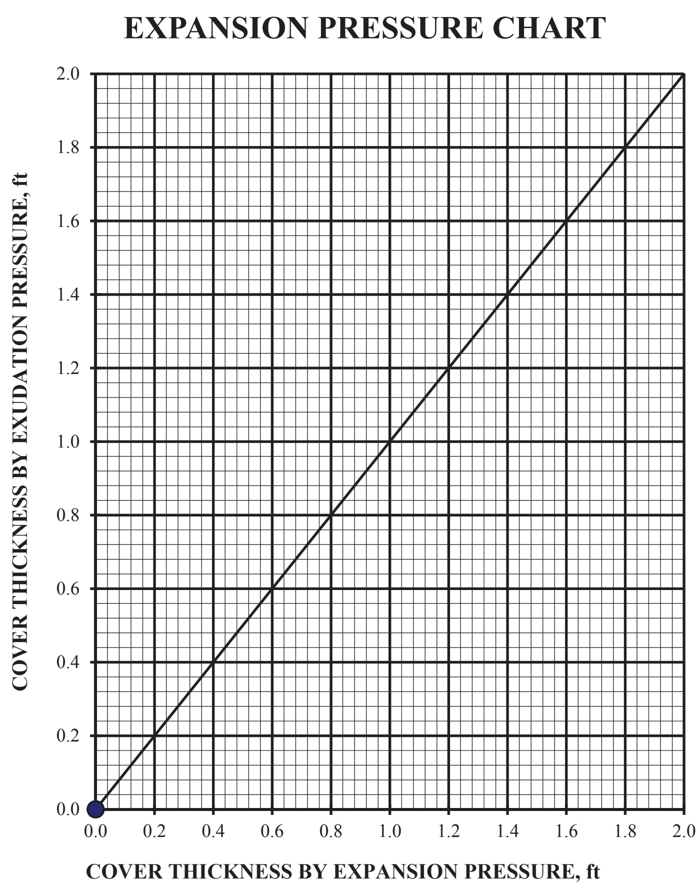
ASTM D 2844/D2844M-13

August 9, 2019

Boring #1 @ 0.0 - 5.0'
Light Gray Silty Sand (SM)

Dry Density @ 300 psi Exudation Pressure: 133.5-pcf
%Moisture @ 300 psi Exudation Pressure: 8.0%
R-Value - Exudation Pressure: 61
R-Value - Expansion Pressure: N/A
R-Value @ Equilibrium: 61



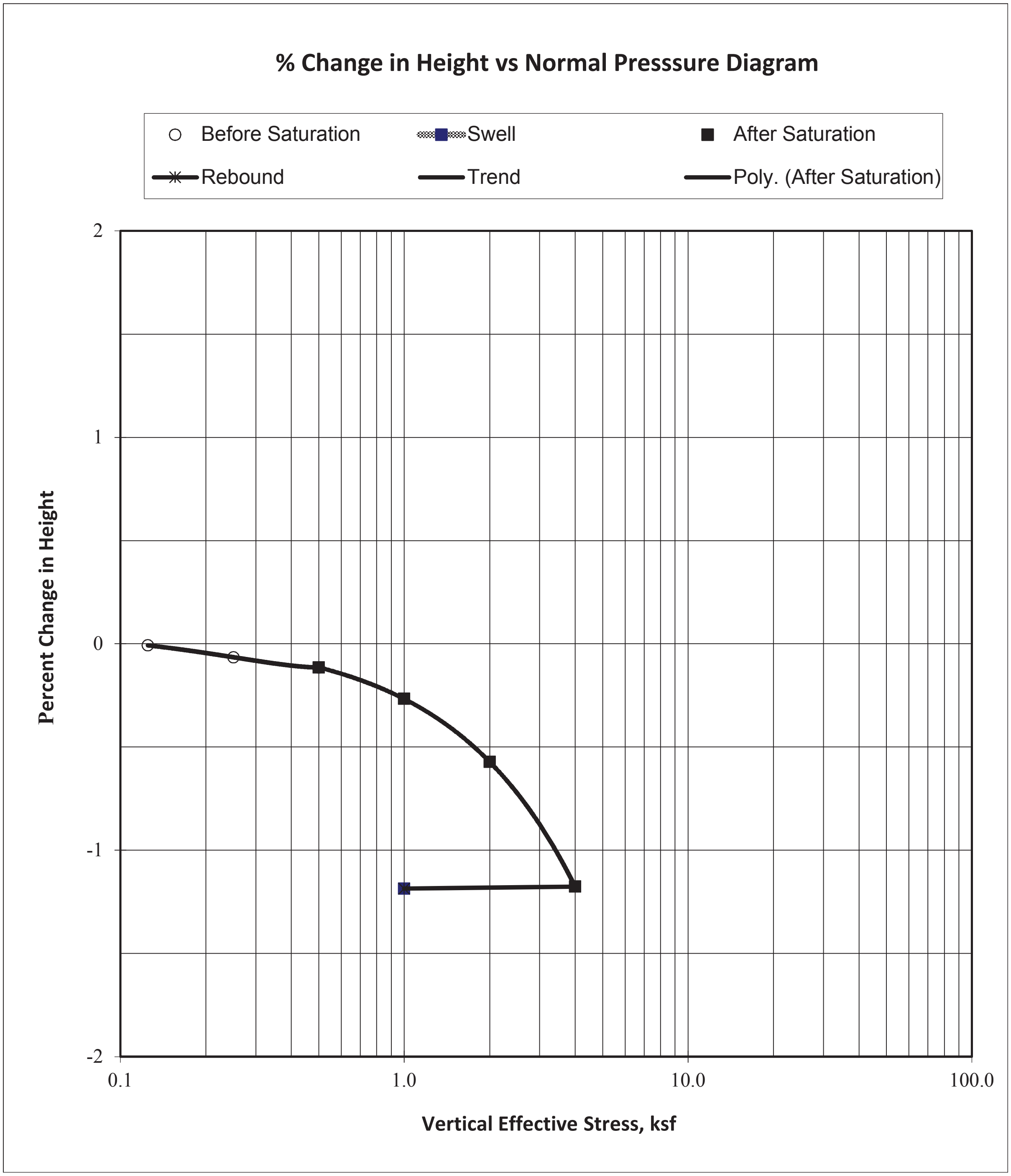


CONSOLIDATION TEST

ASTM D 2435-90

Rio Mesa High School Synthetic Turf Field
B 2 @ 5'
Silty Sand
Ring Sample

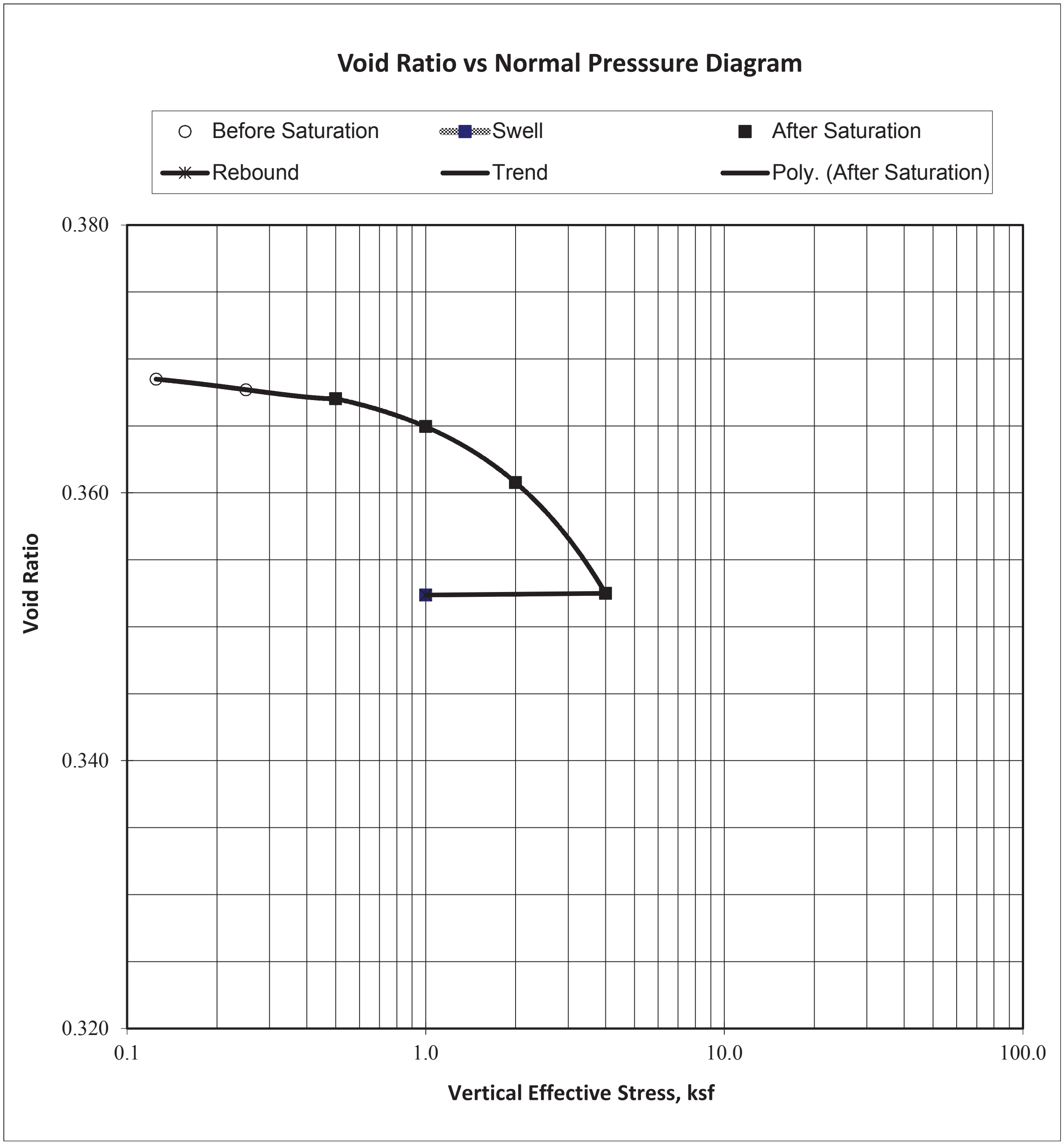
Initial Dry Density: 121.8 pcf
Initial Moisture, %: 10.7%
Specific Gravity: 2.67 (assume
Initial Void Ratio: 0.369



CONSOLIDATION TEST

ASTM D 2435-90

Rio Mesa High School Synthetic Turf Field		Initial Dry Density:	121.8
B 2 @ 5'		Initial Moisture, %:	10.7
Silty Sand		Specific Gravity:	2.67 (assume
Ring Sample		Initial Void Ratio:	0.369



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California State Accredited Laboratory in Accordance with ELAP Certificate # 2332

CERTIFICATE OF ANALYSIS

Client: Earth Systems Pacific
CAS LAB NO: 191284-01
Sample ID: B2@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)	8.0	S.U.	1	---	9045	07/24/19
Resistivity*	810	Ohms-cm	1	---	SM 120.1M	07/24/19
Chloride	14	mg/Kg	1	0.3	300.0M	07/24/19
Sulfate	1700	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)

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www.capcoenv.com

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	FOOTING WIDTH	FOOTING THICKNESS	ALL PERIMETER FOOTINGS (5)	INTERIOR FOOTINGS FOR SLAB AND RAISED FLOORS (5)	REINFORCEMENT FOR CONTINUOUS FOUNDATIONS (2)	CONCRETE SLABS 3 ½ " MINIMUM THICKNESS: REINFORCEMENT (3)	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
		INCHES			DEPTH BELOW NATURAL SURFACE OF GROUND AND FINISH GRADE (3) (8)						
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

Site Classification

2016 CBC & ASCE 7-10 Seismic Parameters

US Seismic Design Maps

Spectral Response Values

Spectral Response Curves

Fault Parameters

EARTH SYSTEMS	Job Number: 303514-002 Job Name: Rio Mesa HS Gateway Liquefaction Calc Date: 11/11/2019 CPT/Boring ID: B-3/B-5

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

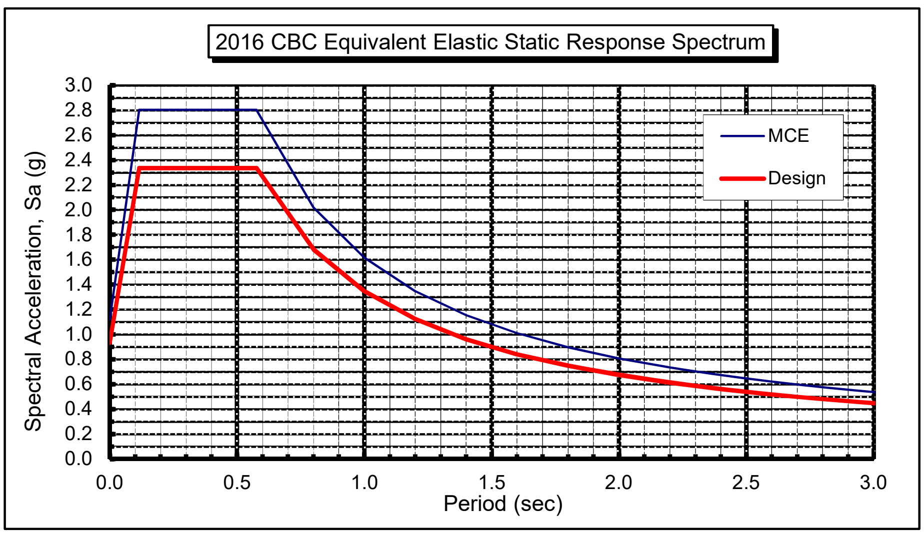
Depth (ft)	SPT N	Sublayer Thick (ft)	Sublayer Thick/N
5.0	16.4	5.0	0.305
10.0	24.6	5.0	0.203
15.0	24.6	5.0	0.203
17.0	28.0	2.0	0.071
20.0	35.0	3.0	0.086
22.0	46.0	2.0	0.043
24.5	38.0	2.5	0.066
27.0	35.0	2.5	0.071
29.5	40.0	2.5	0.063
32.0	44.0	2.5	0.057
34.5	32.0	2.5	0.078
37.0	38.0	2.5	0.066
39.5	35.0	2.5	0.071
42.0	40.0	2.5	0.063
44.5	41.0	2.5	0.061
47.0	35.0	2.5	0.071
49.5	50.0	2.5	0.050
52.0	53.0	2.5	0.047
100.0	50.0	48.0	0.960

Total Thickness of Soil =	100.00	ft
N-bar Value =	37.9	*
Site Classification =	Class D	

*Equation 20.4-2 of ASCE 7-10

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

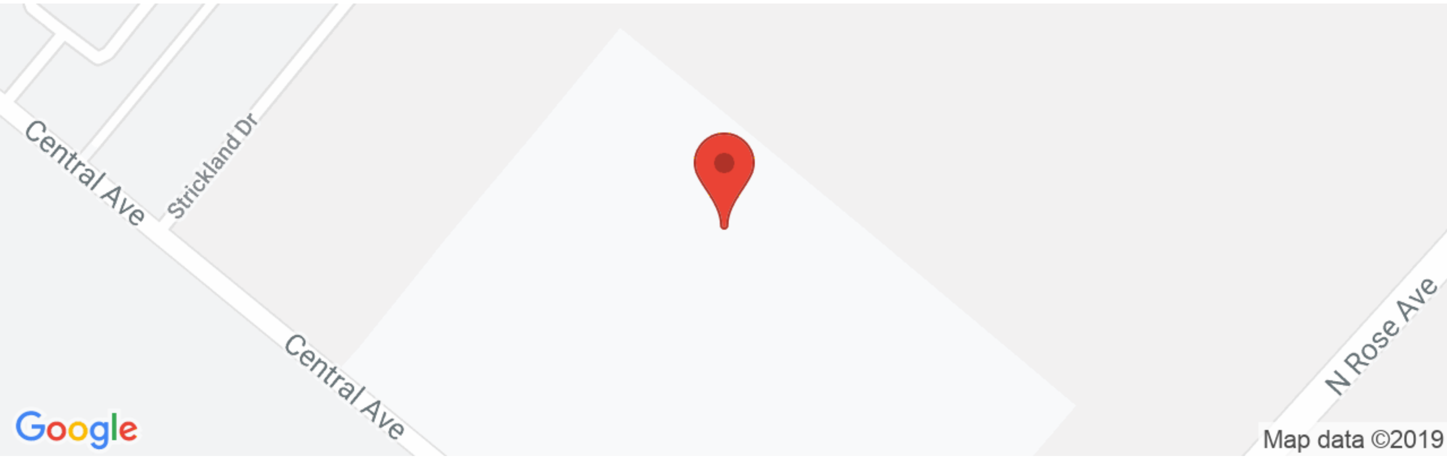
				CBC Reference	ASCE 7-10 Reference
Seismic Design Category		**E**		Table 1613.5.6	Table 11.6-2
Site Class		**D**		Table 1613.5.2	Table 20.3-1
Latitude:		34.256	N		
Longitude:		-119.144	W		
Maximum Considered Earthquake (MCE) Ground Motion					
Short Period Spectral Reponse	S_S	**2.803**	**g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	S_1	**1.077**	**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}	2.803	g	$= F_a*S_S$	
	S_{M1}	1.616	g	$= F_v*S_1$	
Design Earthquake Ground Motion					
Short Period Spectral Reponse	S_{DS}	**1.869**	**g**	$= 2/3*S_{MS}$	
1 second Spectral Response	S_{D1}	**1.077**	**g**	$= 2/3*S_{M1}$	
	To	0.12	sec	$= 0.2*S_{D1}/S_{DS}$	
	Ts	0.58	sec	$= S_{D1}/S_{DS}$	
Seismic Importance Factor	I	1.25		Table 1604.5	Table 11.5-1
	F_{PGA}	1.00			



Period T (sec)	Design Sa (g)
0.00	0.934
0.05	1.542
0.12	2.336
0.58	2.336
0.80	1.683
1.00	1.346
1.20	1.122
1.40	0.962
1.60	0.841
1.80	0.748
2.00	0.673
2.20	0.612
2.40	0.561
2.60	0.518
2.80	0.481
3.00	0.449

Rio Mesa High School Stadium Gateway

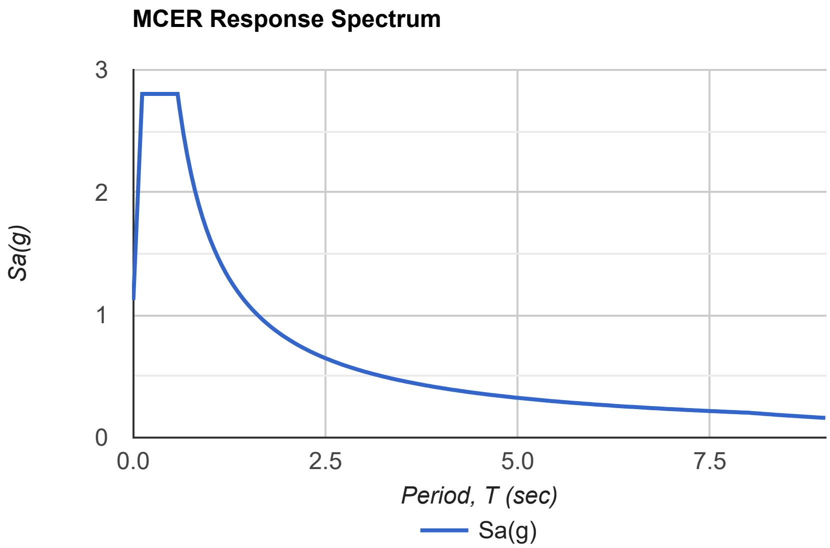
Latitude, Longitude: 34.2556, -119.1443

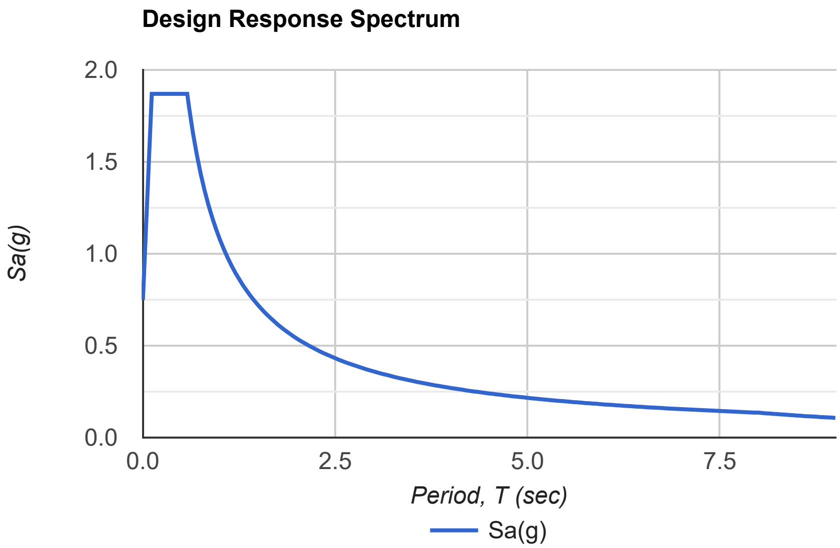


Date	11/12/2019, 1:23:45 PM
Design Code Reference Document	ASCE7-10
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.803	MCE_R ground motion. (for 0.2 second period)
S_1	1.077	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.803	Site-modified spectral acceleration value
S_{M1}	1.615	Site-modified spectral acceleration value
S_{DS}	1.869	Numeric seismic design value at 0.2 second SA
S_{D1}	1.077	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	1.13	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	1.13	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.803	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	3.109	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.31	Factored deterministic acceleration value. (0.2 second)
S1RT	1.077	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.205	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.331	Factored deterministic acceleration value. (1.0 second)
PGAd	1.277	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.902	Mapped value of the risk coefficient at short periods
C_{R1}	0.894	Mapped value of the risk coefficient at a period of 1 s





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Spectral Response Values

Probabilistic and Deterministic Response Spectra for MCE compared to Code Spectra

for 5% Viscous Damping Ratio

Natural Period T (seconds)	GeoMean Probab. 2% in 50 yr MCE Spectrum (1) 2475-yr	Max Rotated Probab. 2% in 50 yr MCEr (2) 2475-yr	Max 84th Percentile Determ. MCE Spectrum (3)	Determ. Lower Limit MCE Spectrum (4)	Determ. MCE Spectrum (5) max(3,4)	**Site Specific MCE Spectrum** (6) min(2.5)	**2013 CBC MCE Spectrum** (7)	Site Specific Design Spectrum (8) 2/3*(6)*	2013 CBC Design Spectrum (9) 2/3*(7)
0.00	0.926	0.919	0.965	0.600	0.965	**0.919**	**1.121**	0.613	0.748
0.05	1.204	1.195	1.131	0.975	1.131	**1.131**	**1.851**	0.987	1.234
0.10	1.483	1.471	1.497	1.350	1.497	**1.471**	**2.580**	1.376	1.720
0.15	1.688	1.674	1.790	1.500	1.790	**1.674**	**2.803**	1.495	1.869
0.20	1.892	1.877	1.950	1.500	1.950	**1.877**	**2.803**	1.495	1.869
0.30	2.016	1.998	2.100	1.500	2.100	**1.998**	**2.803**	1.495	1.869
0.40	1.989	2.059	2.161	1.500	2.161	**2.059**	**2.803**	1.495	1.869
0.50	1.962	2.117	2.203	1.500	2.203	**2.117**	**2.803**	1.495	1.869
0.75	1.698	1.903	2.088	1.200	2.088	**1.903**	**2.154**	1.269	1.436
1.00	1.434	1.667	1.802	0.900	1.802	**1.667**	**1.616**	1.111	1.077
1.50	1.088	1.264	1.393	0.600	1.393	**1.264**	**1.077**	0.843	0.718
2.00	0.741	0.861	1.105	0.450	1.105	**0.861**	**0.808**	0.574	0.539
	Crs:	0.902						* > 80% of (9)	
	Cr1:	0.894							

Probabilistic Spectrum from 2008 USGS Ground Motion Mapping Program adjusted for site conditions and maximum rotated component of ground motion using NGA, Column 2 has risk coefficients Cr applied.

Reference: ASCE 7-10, Chapters 21.2, 21.3, 21.4 and 11.4

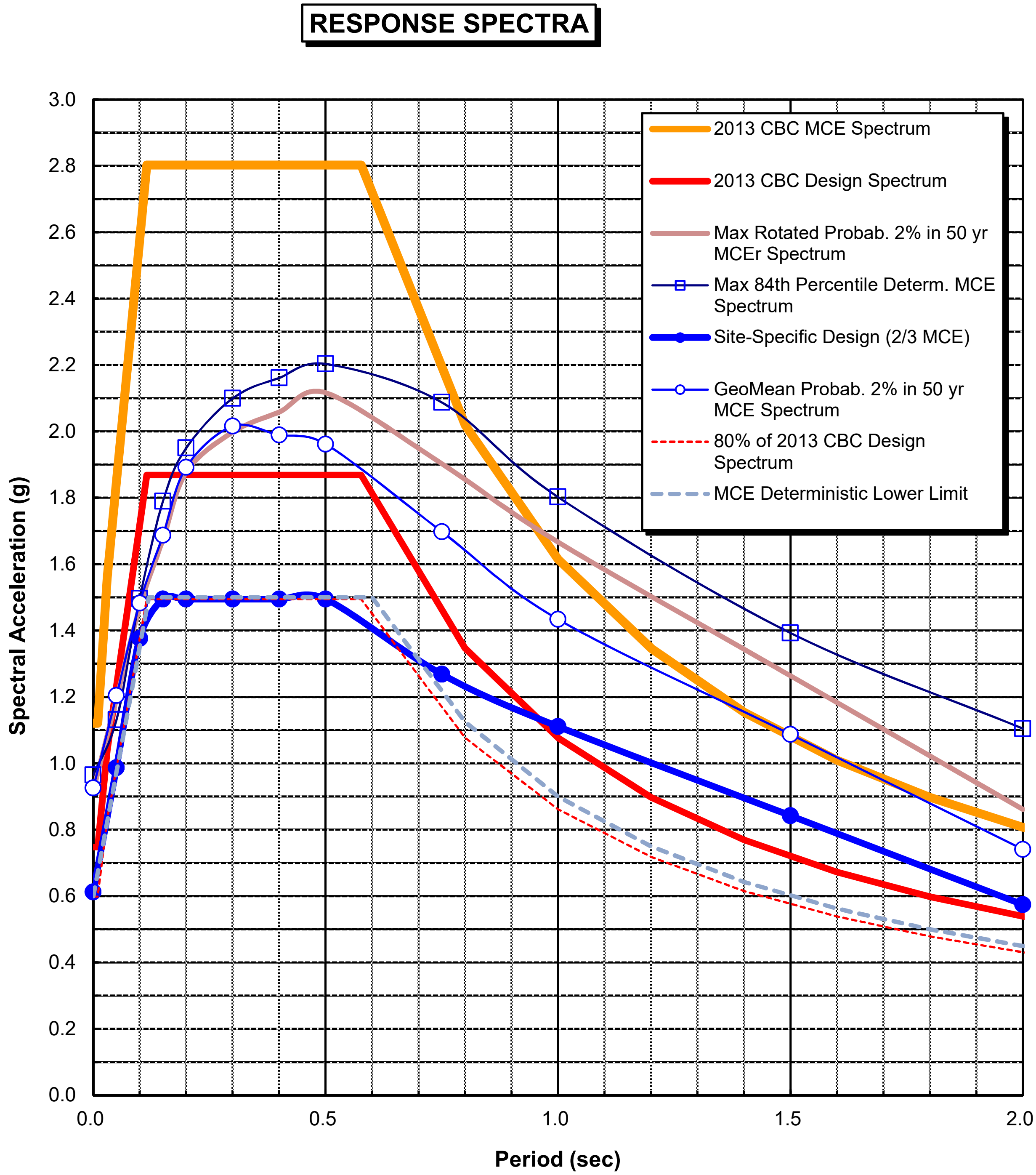
Mapped MCE Acceleration Values			Site Coefficients		**Site-Specific Design Acceleration Values**		
PGA	1.130	g	F_{PGA}	1.00	**PGA_M**	**1.130**	**g**
Ss	2.803	g	F_a	1.00	**S_{DS}**	**1.495**	**g**
S_1	1.077	g	F_v	1.50	**S_{D1}**	**1.148**	**g**

Spectral Amplification Factor for different viscous damping, D (%):

0.5%	2%	10%	20%
1.50	1.23	0.83	0.67

1 g = 980.6 cm/sec^2 =32.2 ft/sec^2
PSV (ft/sec) = 32.2(Sa)T/(2π)

Key: Probab. = Probabilistic, Determ. = Deterministic, MCE = Maximum Considered Earthquake



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

Site Class: D
Latitude: 34.2556
Longitude: -119.1443

Spectral Response Curves

Rio Mesa High School Athletic Fields
File No.: 303514-002

Earth Systems

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)	0.9	1.5	65	159	90	49	B	**7.4**		4
Simi-Santa Rosa	3.7	5.9	60	346	30	39	B	**6.8**		1
Ventura-Pitas Point	4.8	7.7	64	353	60	44	B	**6.9**		1
Oak Ridge (Offshore)	7.4	11.8	32	180	90	38	B	**6.9**		3
Red Mountain	10.8	17.3	56	2	90	101	B	**7.4**		2
Sisar	11.4	18.4	29	168	na	20	B'	**7.0**		
San Cayetano	12.5	20.0	42	3	90	42	B	**7.2**		6
Malibu Coast (Extension), alt 1	13.2	21.3	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	13.2	21.3	74	4	30	35	B'	**6.9**		
Mission Ridge-Arroyo Parida-Santa Ana	13.8	22.2	70	176	90	69	B	**6.8**		0.4
North Channel	16.6	26.7	26	10	90	51	B	**6.7**		1
Channel Islands Thrust	17.3	27.8	20	354	90	59	B	**7.3**		1.5
Malibu Coast, alt 1	18.8	30.3	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	18.8	30.3	74	3	30	38	B	**6.9**		0.3
Santa Ynez (East)	19.1	30.7	70	172	0	68	B	**7.2**		2
Anacapa-Dume, alt 1	19.4	31.2	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	19.4	31.2	41	352	60	65	B	**7.2**		3
Channel Islands Western Deep Ramp	19.8	31.8	21	204	90	62	B'	**7.3**		
Pitas Point (Lower)-Montalvo	19.9	32.0	16	359	90	30	B	**7.3**		2.5
Santa Cruz Island	20.0	32.1	90	188	30	69	B	**7.1**		1
Pine Mtn	22.2	35.7	45	5	na	62	B'	**7.3**		
Santa Susana, alt 1	22.7	36.5	55	9	90	27	B	**6.8**		5
Santa Susana, alt 2	22.7	36.6	53	10	90	43	B'	**6.8**		
Shelf (Projection)	24.1	38.8	17	21	na	70	B'	**7.8**		
Northridge Hills	25.2	40.6	31	19	90	25	B'	**7.0**		
Del Valle	25.3	40.7	73	195	90	9	B'	**6.3**		
Pitas Point (Upper)	25.5	41.1	42	15	90	35	B	**6.8**		1
Holser, alt 1	25.6	41.2	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	25.6	41.2	58	182	90	17	B'	**6.7**		
Northridge	27.1	43.5	35	201	90	33	B	**6.8**		1.5
Santa Cruz Catalina Ridge	27.5	44.2	90	38	na	137	B'	**7.3**		
Santa Monica Bay	29.4	47.4	20	44	na	17	B'	**7.0**		
San Pedro Basin	29.4	47.4	88	51	na	69	B'	**7.0**		
Oak Ridge (Offshore), west extension	30.4	48.9	67	195	na	28	B'	**6.1**		
Big Pine (Central)	31.0	50.0	76	167	na	23	B'	**6.3**		
Big Pine (West)	32.5	52.4	50	2	na	18	B'	**6.5**		
Santa Ynez (West)	32.7	52.6	70	182	0	63	B	**6.9**		2
San Gabriel	32.9	53.0	61	39	180	71	B	**7.3**		1
Big Pine (East)	33.1	53.3	73	338	na	23	B'	**6.6**		
Compton	34.5	55.5	20	34	90	65	B'	**7.5**		

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

Based on Site Coordinates of 34.2556 Latitude, -119.1443 Longitude

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

APPENDIX D

Liquefaction and Seismic-Induced Settlement Calculations
Liquefaction and Seismic-Induced Settlement Curves

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

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

Project: Rio Mesa High School Gateway
Job No: 303514-002
Date: 11/13/2019
Boring: B-5 Data Set: 1

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

EARTHQUAKE INFORMATION:

Magnitude: 7.4 7.5
PGA, g: 1.13 1.09
MSF: 1.03
GWT: 52.0 feet
Calc GWT: 25.0 feet
Remediate to: 5.0 feet

SPT N VALUE CORRECTIONS:

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

Threshold Acceler., g: **1.62**

Required SF: 1.30
Minimum Calculated SF: 1.43

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

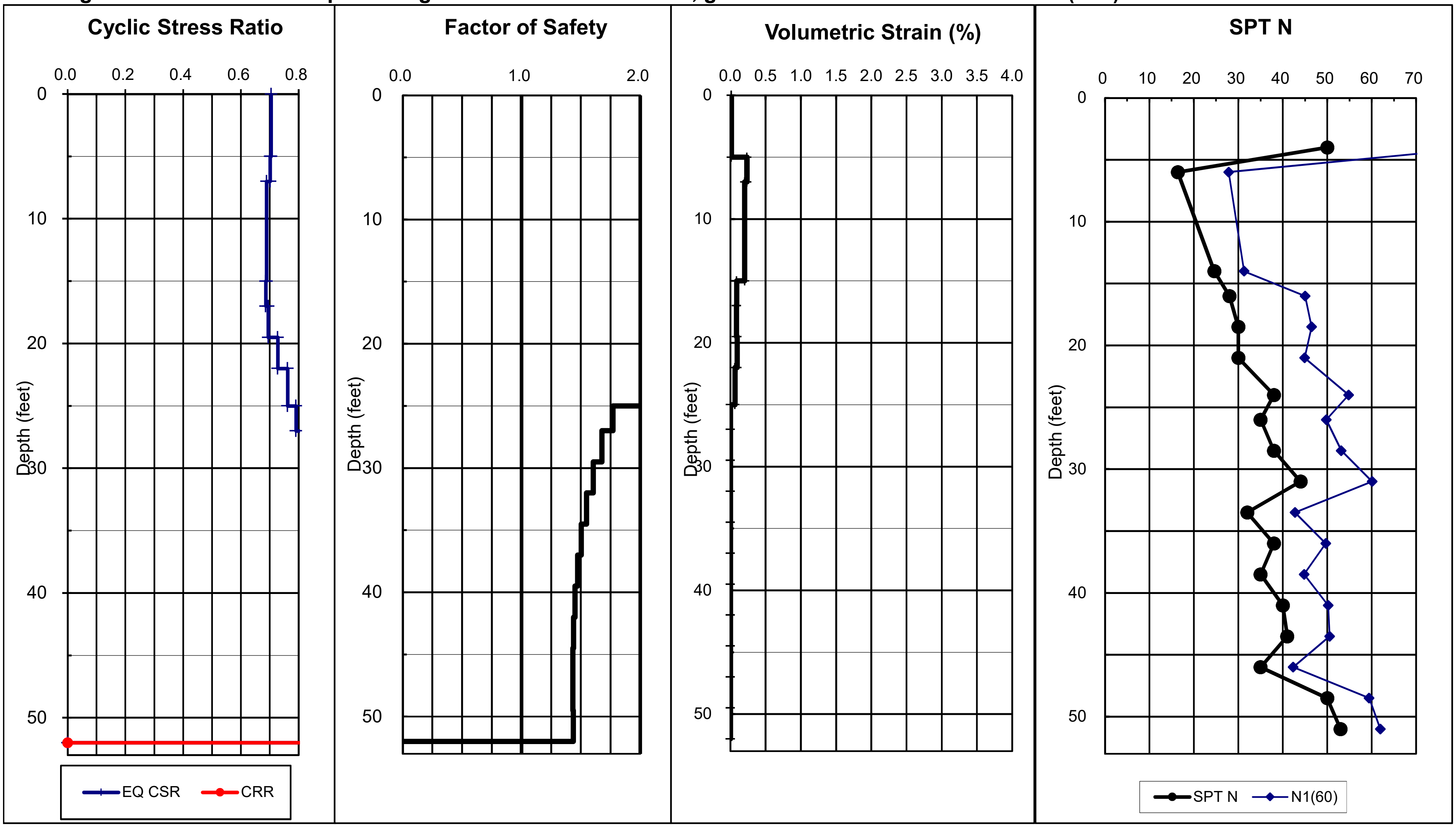
Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	Liquefac. Safety Factor	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	Induced Subsidence (in.)
0.0				0				0.000																	
5.0	36	**50**	1	117	5	4.0	7.0	0.234	0.234	0.99	1.70	0.75	1.00	85.0	###	0.0	85.0	1.00	1.400	0.705	**Non-Liq.**	0.0	85.0	0.01	**0.01**
7.0	26	**16**	1	117	5	6.0	9.0	0.351	0.351	0.99	1.70	0.75	1.00	27.8	63	0.0	27.8	1.00	0.339	0.701	**Non-Liq.**	0.0	27.8	0.24	**0.06**
15.0	39	25	1	120	5	14.0	17.0	0.830	0.830	0.97	1.13	0.85	1.00	31.3	67	0.0	31.3	1.00	1.400	0.689	**Non-Liq.**	0.0	31.3	0.20	**0.20**
17.0	0	28	1	125	5	16.0	19.0	0.952	0.952	0.97	1.05	0.88	1.30	45.0	80	0.0	45.0	1.00	1.400	0.686	**Non-Liq.**	0.0	45.0	0.09	**0.02**
19.5	0	30	1	125	5	18.5	21.5	1.108	1.108	0.96	0.98	0.92	1.30	46.5	82	0.0	46.5	0.98	1.400	0.695	**Non-Liq.**	0.0	46.5	0.09	**0.03**
22.0	0	30	1	125	5	21.0	24.0	1.265	1.265	0.95	0.91	0.94	1.30	44.9	80	0.0	44.9	0.93	1.400	0.728	**Non-Liq.**	0.0	44.9	0.10	**0.03**
25.0	0	38	1	125	5	24.0	27.0	1.452	1.452	0.95	0.85	0.97	1.30	54.8	88	0.0	54.8	0.88	1.400	0.762	**Non-Liq.**	0.0	54.8	0.06	**0.02**
27.0	0	35	1	125	5	26.0	29.0	1.577	1.546	0.94	0.83	0.99	1.30	49.8	84	0.0	49.8	0.86	1.400	0.791	**1.77**	0.0	49.8	0.00	**0.00**
29.5	0	38	1	125	5	28.5	31.5	1.733	1.624	0.93	0.81	1.00	1.30	53.2	87	0.0	53.2	0.84	1.400	0.834	**1.68**	0.0	53.2	0.00	**0.00**
32.0	0	44	1	125	5	31.0	34.0	1.890	1.702	0.92	0.79	1.00	1.30	60.1	93	0.0	60.1	0.83	1.400	0.872	**1.60**	0.0	60.1	0.00	**0.00**
34.5	0	32	1	125	5	33.5	36.5	2.046	1.781	0.90	0.77	1.00	1.30	42.8	78	0.0	42.8	0.81	1.400	0.905	**1.55**	0.0	42.8	0.00	**0.00**
37.0	0	38	1	125	5	36.0	39.0	2.202	1.859	0.88	0.75	1.00	1.30	49.7	84	0.0	49.7	0.80	1.400	0.931	**1.50**	0.0	49.7	0.00	**0.00**
39.5	0	35	1	125	5	38.5	41.5	2.358	1.937	0.86	0.74	1.00	1.30	44.8	80	0.0	44.8	0.79	1.400	0.951	**1.47**	0.0	44.8	0.00	**0.00**
42.0	0	40	1	125	5	41.0	44.0	2.515	2.015	0.84	0.72	1.00	1.30	50.2	85	0.0	50.2	0.77	1.400	0.965	**1.45**	0.0	50.2	0.00	**0.00**
44.5	0	41	1	125	5	43.5	46.5	2.671	2.094	0.82	0.71	1.00	1.30	50.5	85	0.0	50.5	0.76	1.400	0.974	**1.44**	0.0	50.5	0.00	**0.00**
47.0	0	35	1	125	5	46.0	49.0	2.827	2.172	0.79	0.70	1.00	1.30	42.3	78	0.0	42.3	0.75	1.400	0.978	**1.43**	0.0	42.3	0.00	**0.00**
49.5	0	50	1	125	5	48.5	51.5	2.983	2.250	0.77	0.69	1.00	1.30	59.4	92	0.0	59.4	0.74	1.400	0.978	**1.43**	0.0	59.4	0.00	**0.00**
52.0	0	53	1	125	5	51.0	54.0	3.140	2.328	0.74	0.67	1.00	1.30	61.9	94	0.0	61.9	0.73	1.400	0.975	**1.44**	0.0	61.9	0.00	**0.00**
				0																					

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Rio Mesa High School Gateway **Project No: 303514-002** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-5 **Earthquake Magnitude: 7.4** **PGA, g: 1.13** **Calc GWT (feet): 25**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.4 inches**

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

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

Project: Rio Mesa High School Gateway
Job No: 303514-002
Date: 11/13/2019
Boring: B-5 Data Set: 1

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

EARTHQUAKE INFORMATION:

Magnitude: 7.4 7.5
PGA, g: 1.13 1.09
MSF: 1.03
GWT: 52.0 feet
Calc GWT: 52.0 feet
Remediate to: 5.0 feet

SPT N VALUE CORRECTIONS:

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

Threshold Acceler., g: #N/A

Total (ft) Liquefied Thickness: 0

Total (in.) Induced Subsidence: 0.7

Required SF: 1.30
Minimum Calculated SF: #N/A

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M = 7.5 Induced CSR*	Liquefac. Safety Factor	Post FC Adj. $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)
0.0				0				0.000																	
5.0	36	**50**	1	117	5	4.0	7.0	0.234	0.234	0.99	1.70	0.75	1.00	85.0	###	0.0	85.0	1.00	1.400	0.705	**Non-Liq.**	0.0	85.0	0.01	**0.01**
7.0	26	**16**	1	117	5	6.0	9.0	0.351	0.351	0.99	1.70	0.75	1.00	27.8	63	0.0	27.8	1.00	0.339	0.701	**Non-Liq.**	0.0	27.8	0.24	**0.06**
15.0	39	25	1	120	5	14.0	17.0	0.830	0.830	0.97	1.13	0.85	1.00	31.3	67	0.0	31.3	1.00	1.400	0.689	**Non-Liq.**	0.0	31.3	0.20	**0.20**
17.0	0	28	1	125	5	16.0	19.0	0.952	0.952	0.97	1.05	0.88	1.30	45.0	80	0.0	45.0	1.00	1.400	0.686	**Non-Liq.**	0.0	45.0	0.09	**0.02**
19.5	0	30	1	125	5	18.5	21.5	1.108	1.108	0.96	0.98	0.92	1.30	46.5	82	0.0	46.5	0.98	1.400	0.695	**Non-Liq.**	0.0	46.5	0.09	**0.03**
22.0	0	30	1	125	5	21.0	24.0	1.265	1.265	0.95	0.91	0.94	1.30	44.9	80	0.0	44.9	0.93	1.400	0.728	**Non-Liq.**	0.0	44.9	0.10	**0.03**
25.0	0	38	1	125	5	24.0	27.0	1.452	1.452	0.95	0.85	0.97	1.30	54.8	88	0.0	54.8	0.88	1.400	0.762	**Non-Liq.**	0.0	54.8	0.06	**0.02**
27.0	0	35	1	125	5	26.0	29.0	1.577	1.577	0.94	0.82	0.99	1.30	49.3	84	0.0	49.3	0.85	1.400	0.781	**Non-Liq.**	0.0	49.3	0.08	**0.02**
29.5	0	38	1	125	5	28.5	31.5	1.733	1.733	0.93	0.78	1.00	1.30	51.5	86	0.0	51.5	0.82	1.400	0.802	**Non-Liq.**	0.0	51.5	0.08	**0.02**
32.0	0	44	1	125	5	31.0	34.0	1.890	1.890	0.92	0.75	1.00	1.30	57.1	90	0.0	57.1	0.79	1.400	0.820	**Non-Liq.**	0.0	57.1	0.06	**0.02**
34.5	0	32	1	125	5	33.5	36.5	2.046	2.046	0.90	0.72	1.00	1.30	39.9	75	0.0	39.9	0.77	1.400	0.832	**Non-Liq.**	0.0	39.9	0.13	**0.04**
37.0	0	38	1	125	5	36.0	39.0	2.202	2.202	0.88	0.69	1.00	1.30	45.7	81	0.0	45.7	0.75	1.400	0.841	**Non-Liq.**	0.0	45.7	0.10	**0.03**
39.5	0	35	1	125	5	38.5	41.5	2.358	2.358	0.86	0.67	1.00	1.30	40.6	76	0.0	40.6	0.73	1.400	0.845	**Non-Liq.**	0.0	40.6	0.12	**0.04**
42.0	0	40	1	125	5	41.0	44.0	2.515	2.515	0.84	0.65	1.00	1.30	45.0	80	0.0	45.0	0.71	1.400	0.845	**Non-Liq.**	0.0	45.0	0.09	**0.03**
44.5	0	41	1	125	5	43.5	46.5	2.671	2.671	0.82	0.63	1.00	1.30	44.7	80	0.0	44.7	0.69	1.400	0.842	**Non-Liq.**	0.0	44.7	0.09	**0.03**
47.0	0	35	1	125	5	46.0	49.0	2.827	2.827	0.79	0.61	1.00	1.30	37.1	73	0.0	37.1	0.67	1.400	0.835	**Non-Liq.**	0.0	37.1	0.13	**0.04**
49.5	0	50	1	125	5	48.5	51.5	2.983	2.983	0.77	0.60	1.00	1.30	51.6	86	0.0	51.6	0.66	1.400	0.825	**Non-Liq.**	0.0	51.6	0.06	**0.02**
52.0	0	53	1	125	5	51.0	54.0	3.140	3.140	0.74	0.58	1.00	1.30	53.3	87	0.0	53.3	0.65	1.400	0.815	**Non-Liq.**	0.0	53.3	0.06	**0.02**
				0																					

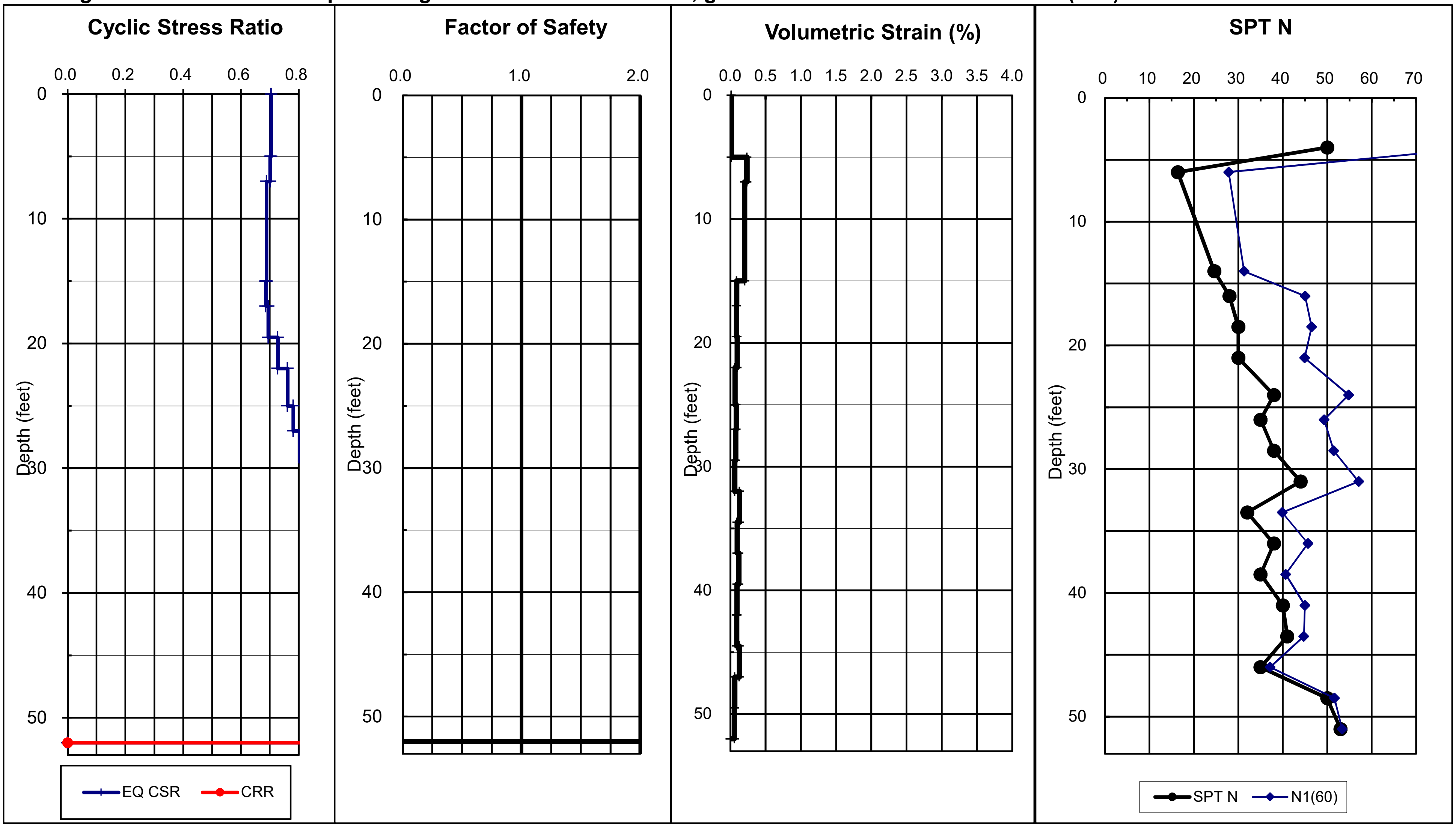
EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Rio Mesa High School Gateway **Project No: 303514-002**

1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B-5 **Earthquake Magnitude: 7.4** **PGA, g: 1.13** **Calc GWT (feet): 52**

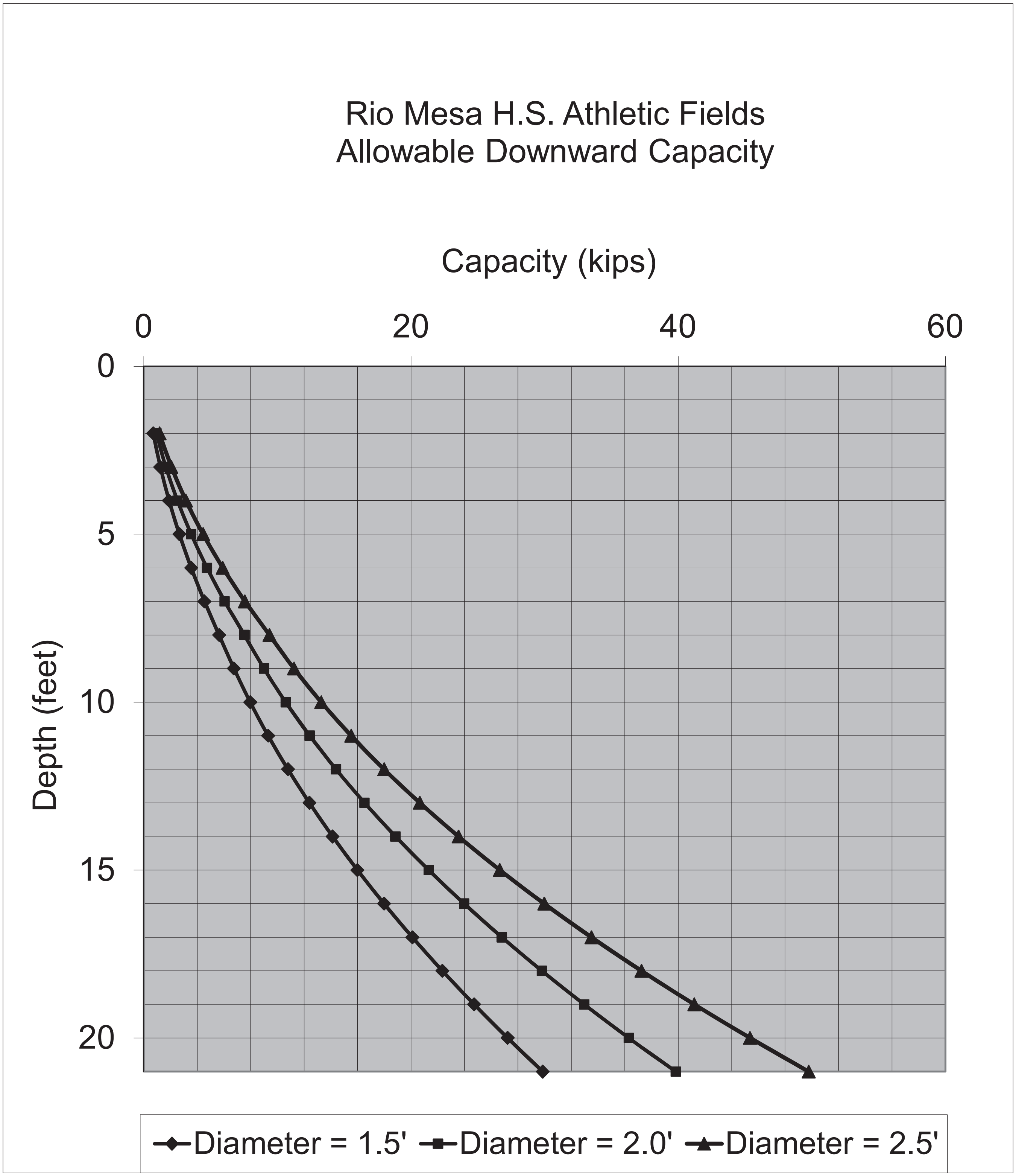


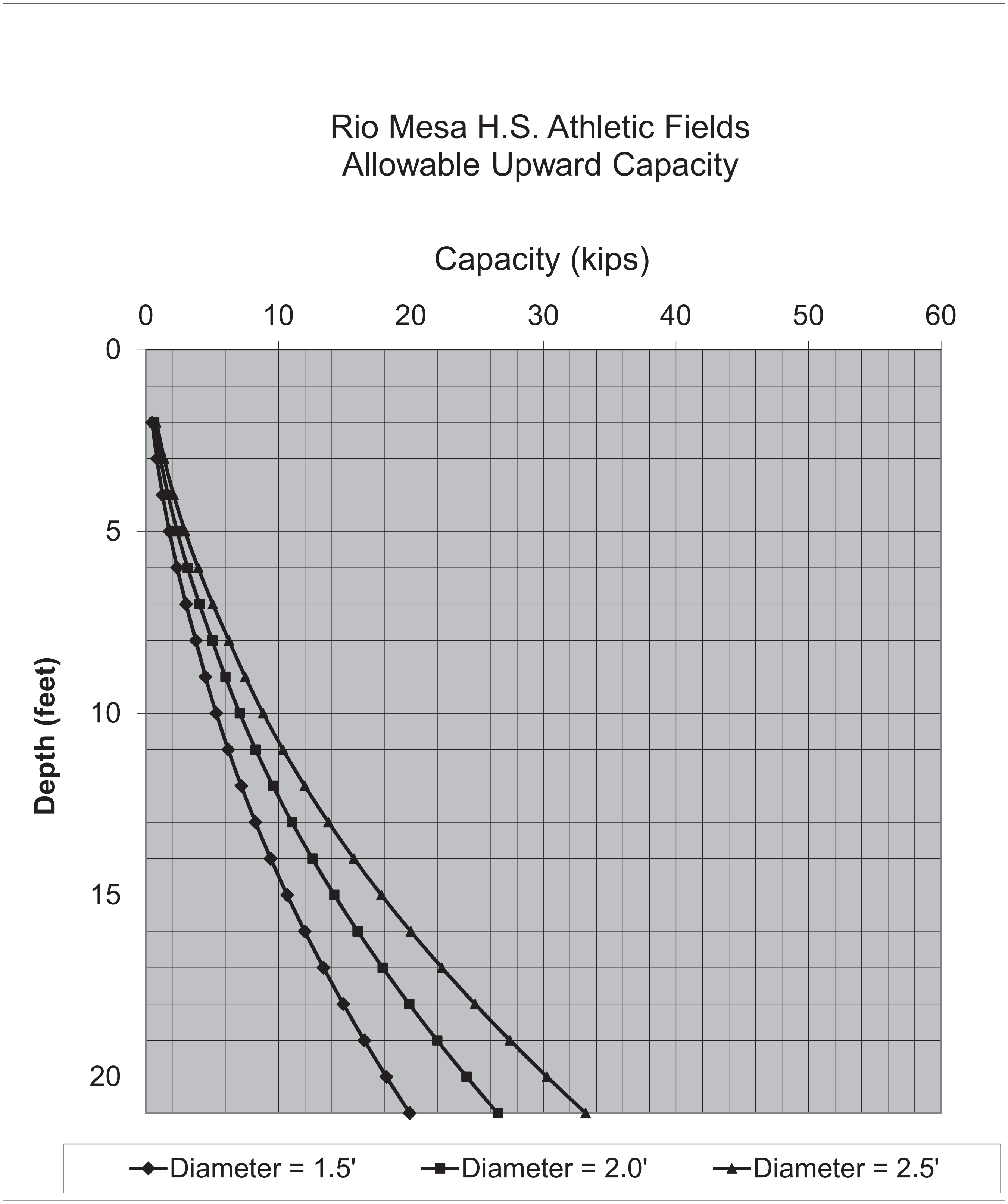
Total Thickness of Liquefiable Layers: 0.0 feet

Estimated Total Ground Subsidence: 0.7 inches

APPENDIX E

Pile Capacity Graphs


Rio Mesa H.S. Athletic Fields
Allowable Downward Capacity
Capacity (kips)
0
20
40
60
Depth (feet)
0
5
10
15
20
Diameter = 1.5'
Diameter = 2.0'
Diameter = 2.5'


Rio Mesa H.S. Athletic Fields
Allowable Upward Capacity
Capacity (kips)
0
10
20
30
40
50
60
0
5
10
15
20
Depth (feet)
Diameter = 1.5'
Diameter = 2.0'
Diameter = 2.5'