ENGINEERING GEOLOGY AND GEOTECHNICAL ENGINEERING REPORT

FOR PROPOSED REMODELING OF
LOCKER ROOM AND KITCHEN AT
MATILIJA JUNIOR HIGH SCHOOL,
703 EL PASEO ROAD,
OJAI, CALIFORNIA

PROJECT NO.: 302294-001
JULY 12, 2018

PREPARED FOR
OJAI UNIFIED SCHOOL DISTRICT

BY

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Project No.: 302294-001
Report No.: 18-7-25

Attention: David Rogers
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Project: Matilija Junior High School Locker Room and Kitchen Remodels
703 El Paseo Road
Ojai, California

As authorized, we have performed an engineering geology and geotechnical engineering study for the proposed remodeling of the locker room and kitchen at Matilija Junior High School in the City of Ojai, California. The accompanying Engineering Geology and Geotechnical Engineering Report presents the results of studies, 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-18-002-025 (Revised) dated June 28, 2018, and authorized by you on July 2, 2018.

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

Respectfully submitted,

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INTRODUCTION

This report presents results of an Engineering Geology and Geotechnical Engineering study performed for proposed remodeling of the locker room and kitchen on the campus of Matilija Junior High School in the City of Ojai, California. The campus is located at 703 El Paseo Road (see Vicinity Map in Appendix A). The locker room is situated at the south end of the gymnasium, and the kitchen is located approximately 300 feet east of the locker room (see Geologic Map in Appendix A). The coordinates of the southeastern corner of the locker room are 34.4453° north latitude and 119.2552° west longitude. There are no springs or seeps on the property.

Current plans indicate that the proposed remodel will be include removal of one shear wall, addition of some new framed and/or shear walls, installing some new footings below new walls, adding some new slabs-on-grade to the existing buildings, and adding new anchors and hold downs to existing footings.

No significant grading is expected to be required to complete the project, but although some compaction will be required under new slabs and footings.

PURPOSE AND SCOPE OF WORK

The purpose of the geotechnical study that led to this report was to analyze the geology and soil conditions of the site with respect to the proposed improvements. These conditions include potential geohazards, 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. Reviewing pertinent geologic and geotechnical literature, including a 2004 preliminary engineering geology and geotechnical report prepared for the locker room on the campus.
2. Reviewing aerial photographs taken of the site in 1945 by Fairchild Aerial Surveys, Inc.
3. Consulting with owner representatives and design professionals.
4. Analyzing the geotechnical data obtained.
5. Preparing this report.

Contained in this report are:

1. Descriptions and results of field and laboratory tests that were performed in 2004 for the locker room.
2. Discussions pertaining to the local geologic, soil, and groundwater conditions.
3. Conclusions pertaining to geohazards that could affect the site.
4. Conclusions and recommendations pertaining to foundation construction and structural design.

GEOLOGY

The site lies within the northern Ventura basin in the western portion of the Transverse Ranges geologic province. Numerous east-west trending folds and reverse faults indicative of ongoing north-south transpressional tectonics characterize the region. The school site is situated within the Ojai Valley. Ongoing folding and uplift has tilted Pleistocene to Tertiary age sedimentary rocks in the region.

The campus is not within any of the Fault Rupture Hazard Zones that have been delineated by the California Division of Mines and Geology (CDMG, 1986). The Santa Ana fault is the fault that is nearest to the site. At its closest position to the school site (approximately 0.8 miles to the south of the campus), it is mapped as buried by alluvium. This fault is connected to the Arroyo Parida and Mission Ridge faults to the west, and these three faults are considered "potentially active" by the State (CDMG, 1977a, 1977b).

Mapping by Rockwell (1984) indicates that the area within which the campus is located is underlain by Late Pleistocene Older Alluvium (Qf5b) ranging in age from about 25,000 to 30,000 years. The units are described as coarse clastic fan deposits.

Bedding attitudes were not measured within the underlying fan deposits, but it is considered likely that bedding is oriented nearly parallel to the natural ground surface, and that the units are lenticular.

No faults were observed to be located on or trending into the subject property during reviews of the referenced geologic literature, or during review of the aerial photographs taken of the site.

No landslides were observed to be located on or trending into the subject property during reviews of the referenced geologic literature, or during review of the aerial photographs taken of the site.

GEOLOGIC HAZARDS

Geologic hazards that may impact a site include seismic shaking, fault rupture, landsliding, rock fall, liquefaction, seismic-induced settlement of dry sands, and flooding.

A. Seismic Shaking

1. Southern California is a seismically active region where the potential for significant ground shaking is universal. Earthquakes of a size large enough to cause structural damage are relatively common in the region. Per the State of California guidelines for these types of reports, when evaluating the seismicity potential of a specific site, it is general practice to look at the historical seismic record of the area and also review the site location with respect to mapped potentially active and active faults. By using this procedure, estimates of maximum ground accelerations are determined for consideration in structural design for buildings. The geotechnical community uses the method even though most are aware of its shortcomings. The most significant shortcomings relate to the presence of unknown seismogenic faults well below the surface, and the amount of uncertainty regarding the time intervals between earthquake events on many of the recognized faults. The 1983 Coalinga and 1994 Northridge Earthquakes are examples of relatively large events that occurred on previously unrecognized faults. Man has only been using instruments to monitor earthquakes since the 1930's, which is a relatively short time span considering that the intervals between large earthquakes on some of the regional faults are on the order of thousands of years. Considering the above, an evaluation of site acceleration potential will lead to a value that must be considered an approximation. The structural designers must be aware that there are inherent uncertainties in the determined value or range.

2. The Ojai area has not experienced any local large earthquakes since records have been kept; however, regional earthquakes have led to significant ground shaking and structural damage. Notable regional earthquakes include the 1812 Santa Barbara Channel and 1857 Fort Tejon events. The epicenter of the 1812 earthquake is thought to have been in the western part of the Santa Barbara channel. Associated

with this earthquake, a tsunami with a disputed run up height of up to 15 feet impacted the Ventura coastal area. On January 9, 1857, the Fort Tejon earthquake with an estimated Richter magnitude of 8.25 impacted the region. According to C.D.M.G. (1975), the earthquake caused the roof of the Mission San Buenaventura to fall in.

3. One measure of ground shaking is intensity. The Modified Mercalli Intensity Scale of ground shaking ranges from I to XII with XII indicating the maximum possible intensity of ground movement. Structural damage begins to occur when the intensity exceeds a value of VI. Southern Ventura County has been mapped by the California Division of Mines and Geology to delineate areas of varying predicted seismic response. The "Older Valley Fill" that underlies the subject area is mapped as having a probable maximum intensity of earthquake response of approximately VIII on the Modified Mercalli Scale. Historically, the highest estimated intensity in the Ojai area has been VII (CDMG, 1975, 1995).

4. The school site, like any other site in the region, is subject to relatively severe ground shaking in the event of a maximum earthquake on a nearby fault. In Appendix C is a regional fault location map that shows the site's relationship to the identified faults in the region. Also in Appendix C is a summary table listing well-identified faults within a 53-km radius of the school, the distance between each fault and the school, and mean earthquake magnitudes that could occur on each of the listed faults. A proprietary program utilizing the State of California's fault model (CGS and USGS, 2008) was used to prepare the list.

5. 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.

An analysis was conducted to determine the site class of the Older Alluvium encountered in Boring B-2. California modified sampler blow counts were reviewed, and because of the gravel and cobble content of those soils, the lowest six-inch blow count was used to provide a conservative evaluation. California sampler blow counts were converted to SPT blow counts using a factor of 0.63. The analysis (included in Appendix C) indicated that on-site soils are within Site Class D. For the remaining analyses to determine seismic design parameters, the velocity (Vs30) was assumed to be 259 to 270 meters per second when adjusting for site class.

The subject site is within Seismic Design Category E. For the" general procedure" (i.e. code value, or probabilistic) analysis, the Short Period Spectral Response (S_S) for the Maximum Considered Earthquake (MCE) was found to be 2.219 g, and the 1-Second Spectral Response (S_1) was found to be 0.828 g. Site Coefficients F_a and F_v were found to be 1.00 and 1.50, respectively. The spectral Response Parameter S_{MS} was found to be 2.219 g, and S_{M1} was found to be 1.242 g. The Short Period Spectral

Response (S_{DS}) was found to be 1.479 g, and the 1 Second Spectral Response (S_{D1}) was found to be 0.828 g.

(Because S1 is greater than or equal to 0.75 g, and the Seismic Design Category is "E", a site-specific seismic analysis was performed in addition to the "general procedure". For the Site-Specific Analysis, the Short Period Spectral Response (S_{DS}) was found to be 1.238 g, and the 1 Second Spectral Response (S_{D1}) was found to be 1.090 g.

The peak ground accelerations for the "site specific" and "general procedure" were both found to be 0.804 g.

6. California has had several large earthquakes in this century, and studies on the structural effects of the ground shaking have led to changes in the building codes. After the 1933 Long Beach Earthquake, the State of California Field Act was written with the intention of making public schools more earthquake resistant. The intent of the act, as is the intent of the most modern codes, is as follows: "School buildings constructed pursuant to these regulations are expected to resist earthquake forces generated by major earthquakes in California without catastrophic collapse, but may experience some repairable architectural or structural damage". Following the 1971 San Fernando Earthquake, many changes were made to the public school building codes. After the 1994 Northridge Earthquake, a study of 127 public schools in the Los Angeles area by the State of California Division of the State Architect (1994a) revealed that the intent of the Field Act was being met even when buildings were subjected to horizontal accelerations approaching 0.9 g (much higher than expected) over a large area. None of the schools collapsed and most of the damage that would have caused injury to students, had school been in session, was from failures of non-structural items such as light fixtures, florescent bulbs, suspended ceilings, etc. Most of the schools that experienced these non-structural failures were built before the changes to the building code that applied to these non-structural items. The study also resulted in recommended changes to building codes regarding steel framed school buildings, (State of Calif. Div. of State Architect, 1994b).

B. Fault Rupture

Surficial displacement along a fault trace is known as fault rupture. Fault rupture typically occurs along previously existing fault traces. As mentioned in the "Structure" section

above, no existing fault traces were determined to be crossing the site. As a result, it is the opinion of this firm that the potential for fault rupture on this site is low.

C. Landsliding and Rock Fall

As mentioned previously, relief across the subject site is only a few feet. As a result, landsliding and rock fall do not pose a hazard to this project.

D. Liquefaction, Cyclic Softening, and Lateral Spreading

Earthquake-induced cyclic loading 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. Cyclic softening in clays during earthquakes has resulted in buildings experiencing foundation failure and ground surface deformation similar to that resultant from liquefaction. If liquefaction or cyclic softening occurs beneath sloping ground, a phenomenon known as lateral spreading can occur. Liquefaction and cyclic softening is typically limited to the upper 50 feet of the subsurface soils. There are a number of conditions that need to be satisfied for liquefaction or cyclic softening to occur. Of primary importance is that groundwater, perched or otherwise, usually must be within the upper 50 feet of soils. Furthermore, soils that are sufficiently dense are not susceptible to liquefaction.

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

1. Perched groundwater was found at a depth of 15 feet below the ground surface. A sample taken from 20 feet indicates that the soils were "moist", not saturated.
2. Hydrometer analyses for soil samples obtained from Boring B-2 at a depth of 15 feet indicate that these soils are sandy gravels (see Appendix B).
3. Penetration tests conducted in the borings indicate that most soils within the tested depth are in a relatively dense state, even when the lowest of the blow counts for 6-inch increments are used, and the higher blow count intervals are ignored because of potential gravel influence.

Based on the above, cyclic mobility analyses were undertaken to analyze the liquefaction potentials of the various soil layers. The analyses were performed in general accordance

with the methods proposed by NCEER (1997). In the analyses, the design earthquake was considered to be a 7.2 moment magnitude event, and a peak ground acceleration of 0.804 g was used, as per the discussion in the Seismic Shaking section of this report. It was also assumed that the upper 5 feet of soil had been compacted during grading for the structure. The analysis assumed perched groundwater at 15 feet, and unsaturated soils below 20 feet. The lowest 6-inch blow count increment from each sample blow count was doubled, then multiplied by 0.63 to convert from California Modified Sampler blow count to Standard Penetration Test blow count.

The analysis indicated that the soil layer between depths of 15 and 17.5 feet had a factor of safety that exceeded 1.3 (see Appendix D for calculations), but the soil horizon between 17.5 and 20 feet had a factor of safety of 0.35. Zones with factors of safety less than 1.3 are considered potentially liquefiable (C.G.S., 2008, and SCEC, 1999).

The volumetric strain for the potentially liquefiable zone was estimated using a chart derived by Tokimatsu and Seed (1987) after reducing the $N_{1(60)}$ values by the calculated "FC Delta" value, then making adjustments for fines content as per Seed (1987) and SCEC (1999). Using this methodology, the volumetric strain was found to be approximately 0.6 inches.

According to data generated by Ishihara (National Academy Press, 1985), no "ground" damage would be expected related to the zones encountered in the borings because of the thickness of soils overlying the 2.5-foot thick potentially liquefiable layer. (Examples of ground damage are sand boils and ground cracks.)

Although the analysis predicts that there will be no ground damage, there is a potential for a small amount of differential areal settlement suggested by the findings. As mentioned previously, the total liquefaction-related settlement could potentially range up to about 0.6 inches. 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 0.3 inches at the ground surface.

"Free-face" lateral spreading does not appear to pose a potential hazard because there are no nearby sloped areas or canyons (Bartlett and Youd, 1995). However, "ground slope" lateral spreading, sometimes referred to as "ground oscillation", can occur when

adjusted blow counts ($N_{1(60)}$) measured within potentially liquefiable zones are less than 15, which includes the potentially liquefiable zone between 17.5 and 20 feet. Basing the calculation on the lowest 6-inch blow count increment from the sample taken at 15 feet, the adjusted blow count calculates to 14.6. However, it should be noted that these fan deposits include large percentages of gravels and cobbles that, when deposited, tend to erode through previously deposited units, thus making the stratigraphy lenticular, and laterally discontinuous. Based on the conservative blow count calculation, and on the stratigraphy, it is the opinion of this firm that "ground slope" lateral spreading does not pose a hazard to this project.

Based on the above, it is the opinion of this firm that a potential for liquefaction exists at this site, although potential settlements are relatively minor.

E. Seismic-Induced Settlement of Dry Sands

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. A procedure to evaluate this type of settlement was 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).

Older alluvium is generally fairly dense, although there is some possibility that it is susceptible to a limited amount of seismic induced settlement. An analysis was performed using the same parameters described above for the liquefaction analysis, including the conservative use of the lowest 6-inch blow count from each sampling interval, but assuming that no groundwater existed in the upper 23 feet of the soil profile.

The analysis indicated that the soil layers between depths of 15 and 23 feet could experience a combined seismic-induced settlement of approximately 0.2 inches (see Appendix D for calculations). If these data were doubled to account for the non-explored zone between 23 and 50 feet, a seismic-induced settlement of 0.4 inches could be estimated.

F. Hyrdoconsolidation

Hydroconsolidation is a phenomenon in which naturally occurring soil deposits, or non-engineered fill, collapse when wetted. Natural soils that are susceptible to this phenomenon are typically aeolian, debris flow, alluvial, or colluvial deposits with high apparent strength when dry. The dry strength is attributed to salts, clays, silts, and in some cases capillary tension, "bonding" larger soil grains together. As long as these soils remain dry, their strength and resistance to compression are retained. However, when wetted, the salt, clay, or silt bonding agent is weakened or dissolved, or capillary tension reduced, eventually leading to collapse.

The site is underlain by alluvial fan deposits that are dense to very dense. This type of deposit is typically not susceptible to hydroconsolidation, and the hazard posed by hydroconsolidation is considered "low".

G. Flooding

Earthquake-induced flooding types include tsunamis, seiches, and reservoir failure. Due to the inland location of the site, hazards from tsunamis and seiches are considered extremely unlikely.

Any nearby reservoir that may fail would normally drain into established major drainage channels. According to the Safety Element of the Ojai General Plan (1991), "there are currently no dams within, adjacent to, or upstream from the City of Ojai which are large enough to endanger lives and property in the event of a failure". In addition, the Ventura County General Plan Hazards Appendix does not show the site within any of the dam inundation hazard zones. As a result, flooding due to dam failure should not be considered a potential hazard.

The site is within a "Zone X" flood zone, as indicated on the National Flood Hazard Layer Firmette generated by the Federal Emergency Management Agency website (2018). The Zone X flood zone is defined as "Area of minimal flood hazard". From this, it appears that storm-induced flooding does not pose a significant hazard to the proposed project.

SOIL CONDITIONS

Evaluation of the subsurface in 2004 indicated that soils included old fill over alluvial fan soils. The depth of fill encountered in the test borings drilled in 2004 ranged from approximately 1.5

feet in Boring B-2 to approximately 4.5 feet in Boring B-1. The fill material consisted of medium stiff fine to coarse sandy clay and sandy silts. A dense to very dense clayey fine to coarse sand with some gravels and scattered cobbles was encountered below the fill to depths of about 7 to 8.5 feet. This stratum was underlain by very dense silty gravel with sand and cobbles that extended to the maximum depths explored. Both borings met refusal on large cobbles in a dense matrix of soil.

Expansion determination indicated that bearing soils lie in the "very low" expansion range because the expansion index was found to be 1. Table 1809.7 provides minimum foundation and slab requirements as a function of expansion index, and is included in Appendix B of this report.

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 (130 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 (4,740 ohms-cm) indicate that they are "moderately corrosive" to ferrous metal (i.e. cast iron, etc.) pipes.

Perched groundwater was encountered at a depth of about 15 feet. True groundwater was not encountered to the maximum depth explored, which was 23 feet. (For instance, the description of a sample taken from 20 feet was "moist", not "saturated".) A map of historical high groundwater from the Seismic Hazard report for the Matilija Quadrangle (CGS, 2003b) shows historical high groundwater to be about 50 feet below the ground surface.

GEOTECHNICAL ENGINEERING
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. As mentioned in the introduction to this report, no grading is expected to be necessary for the remodels as proposed.

Excavations for new footings and slab areas should be observed by a representative of this firm to check for firmness. The bottoms should be tested to determine if bearing soils are at a minimum of 90 percent of the maximum dry density, as determined by the ASTM D 1557 test method.

Current plans indicate that new footings will be 4.5 feet wide and bottomed 3.5 feet below top of slab. If soils tested at footing bottom elevations have relative compactions less than 90 percent, soils should be overexcavated to a depth of 2 feet below footing bottom. Because the new footing excavations will cut through existing slab areas, the overexcavation need not extend laterally outside the footing width. Excavated soils should be replaced in thin, moisture conditioned lifts, and compacted into place. Given the interior location for the new footings, it is likely that a "jumping jack" will be the recompaction tool. As such, lifts should probably be no more than 3 inches thick.

If soils with relative compactions less than 90 percent are encountered at slab subgrade elevations below slabs, soils should be overexcavated to a depth of 1 foot below slab subgrade elevation. Excavated soils should be replaced in thin, moisture conditioned lifts, and compacted into place. Given the interior location for the new footings, it is likely that a "jumping jack" will be the recompaction tool. As such, lifts should probably be no more than 3 inches thick.

Conventional continuous and isolated pad footings may be designed based on a bearing value of 1,500 psf, which is the presumptive bearing value presented in the 2016 California Building Code (CBC) for silt and sandy silt.

Resistance to lateral loading may be provided by cohesion of 130 psf. This is a presumptive value presented in the 2016 CBC.

Passive resistance acting on the sides of foundation stems equal to 100 pcf of equivalent fluid weight may be included for resistance to lateral load. This is a presumptive value presented in the 2016 CBC.

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 less than one-half the total settlement.

As mentioned previously, liquefaction-induced settlements are estimated to be approximately 0.6 inches, and related differential settlements could range up to about 0.3 inches.

Seismic-induced settlements of soils above the groundwater level are estimated to be approximately 0.4 inches, and related differential settlements could range up to about 0.2 inches.

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 drilled on the site in 2004. 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 be 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 the control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of one year.

In the event that any changes in the nature, design, or location 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.

AERIAL PHOTOGRAPHS REVIEWED

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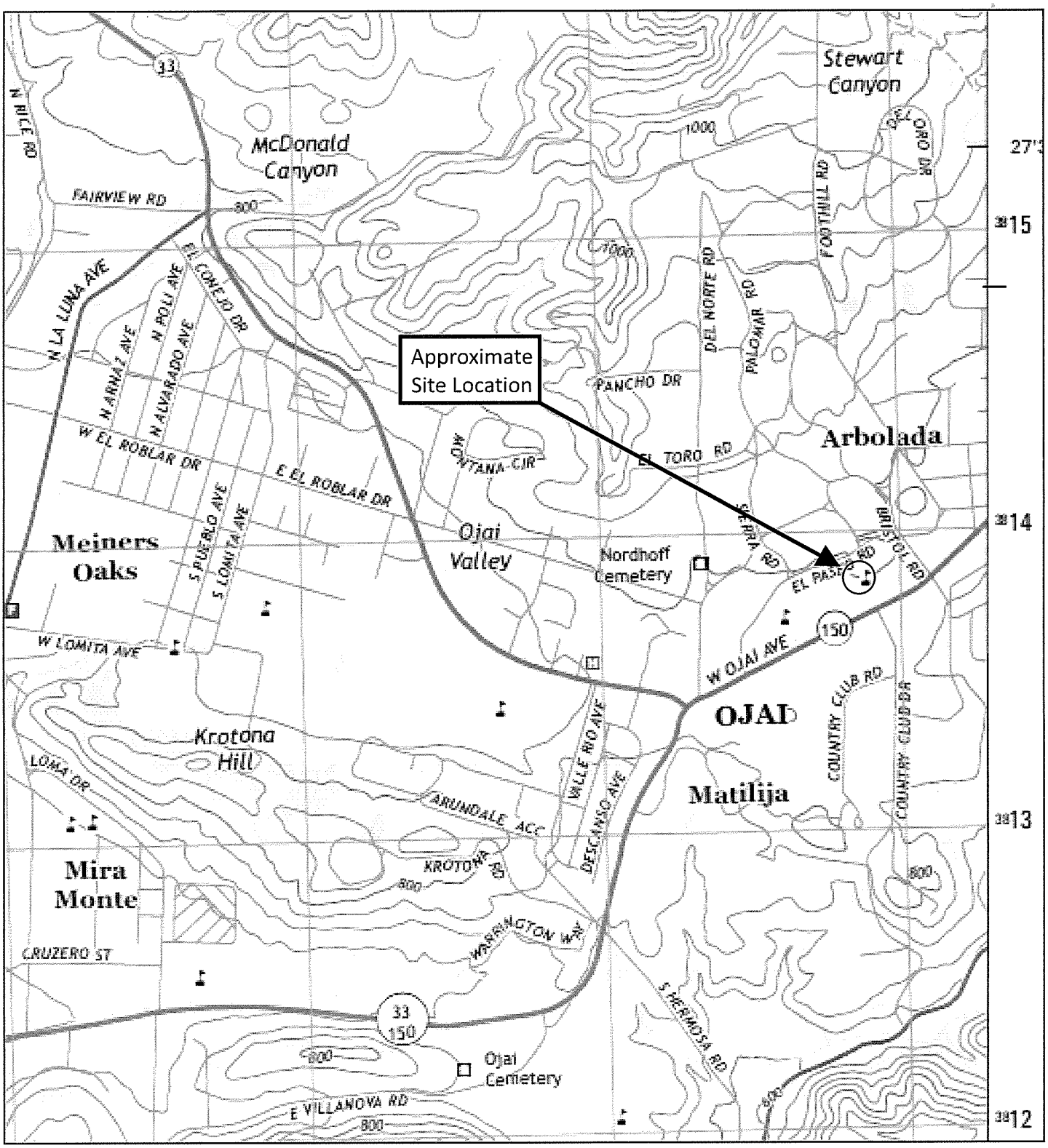
APPENDIX A

Vicinity Map
Regional Fault Map
Regional Geologic Map 1
Regional Geologic Map 2
Seismic Hazard Zones Map
Historical High Groundwater Map
Field Study (2004)
Geologic Map
Geologic Cross-Section
Logs of Borings (2004)
Boring Log Symbols
Unified Soil Classification System



*Taken from USGS Topo Map, Matilija Quadrangle, Ventura County, California, 2015.

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

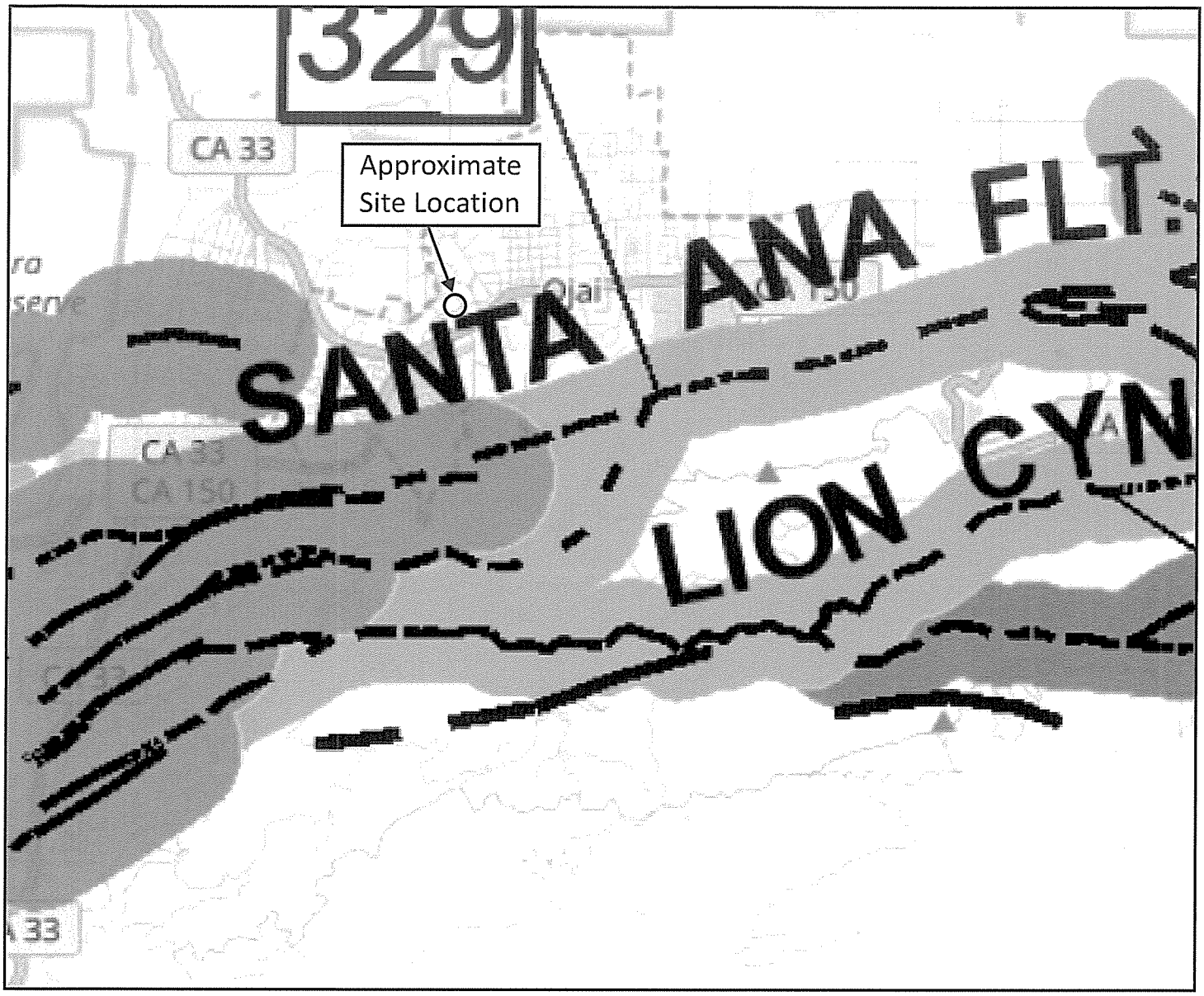
N

VICINITY MAP

MATILIJA JR. HIGH SCHOOL
LOCKER ROOM AND KITCHEN REMODELS
OJAI, CALIFORNIA

Earth Systems

July 2018	302294-001



*Taken from California Geological Survey, Geologic Data Map No. 6, Fault Activity Map of California, 2010

Legend

Holocene fault displacement (during past 11,700 years) without historic record.

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

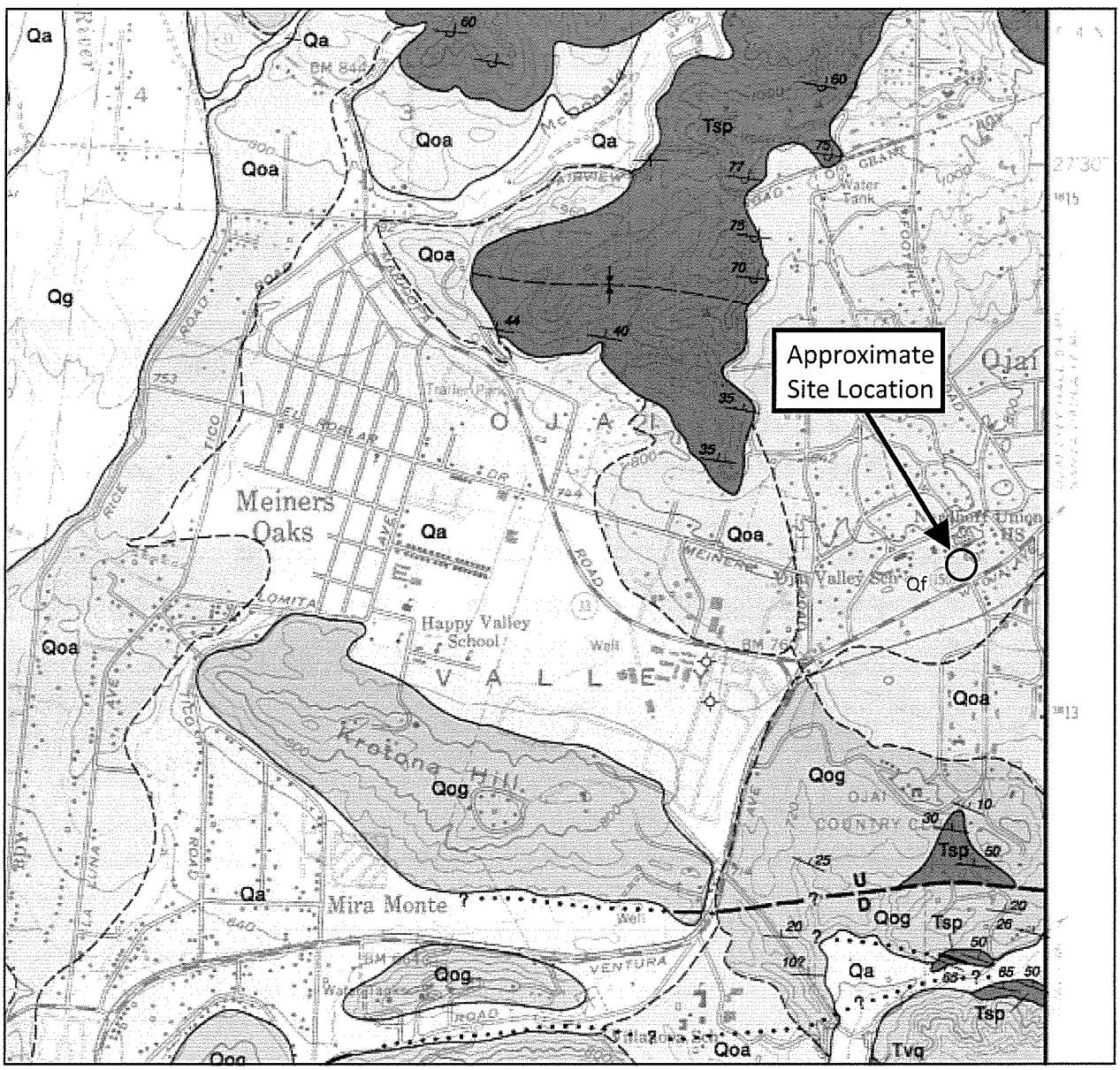
Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate).

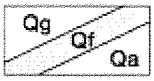
N

APPROXIMATE SCALE
1 Inch = 1 Mile

REGIONAL FAULT MAP	
MATILIJA JR. HIGH SCHOOL LOCKER ROOM AND KITCHEN REMODELS OJAI, CALIFORNIA	
Earth Systems	
July 2018	302294-001



*Taken from Dibblee, T.W., Geologic Map of the Matilija Quadrangle, Ventura County, California, 1987, DF-12.

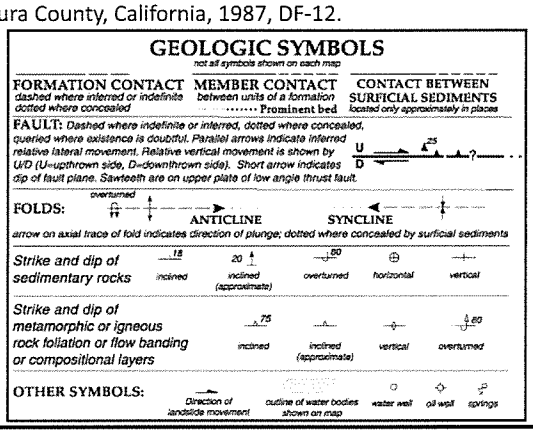


SURFICIAL SEDIMENTS

Qg *Stream channel deposits, mostly gravel and sand*
Qa *Alluvial fan boulder gravel*
Qf *Alluvium: uncolsolidated floodplain deposits of silt, sand and gravel*

OLDER DISSECTED SURFICIAL SEDIMENTS

Qoa *Remnants of weakly consolidated older alluvial deposits of gravel, sand and silt*
Qog *Cobble-boulder fan gravel and fanglomerate deposits composed largely of sandstone detritus*



Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

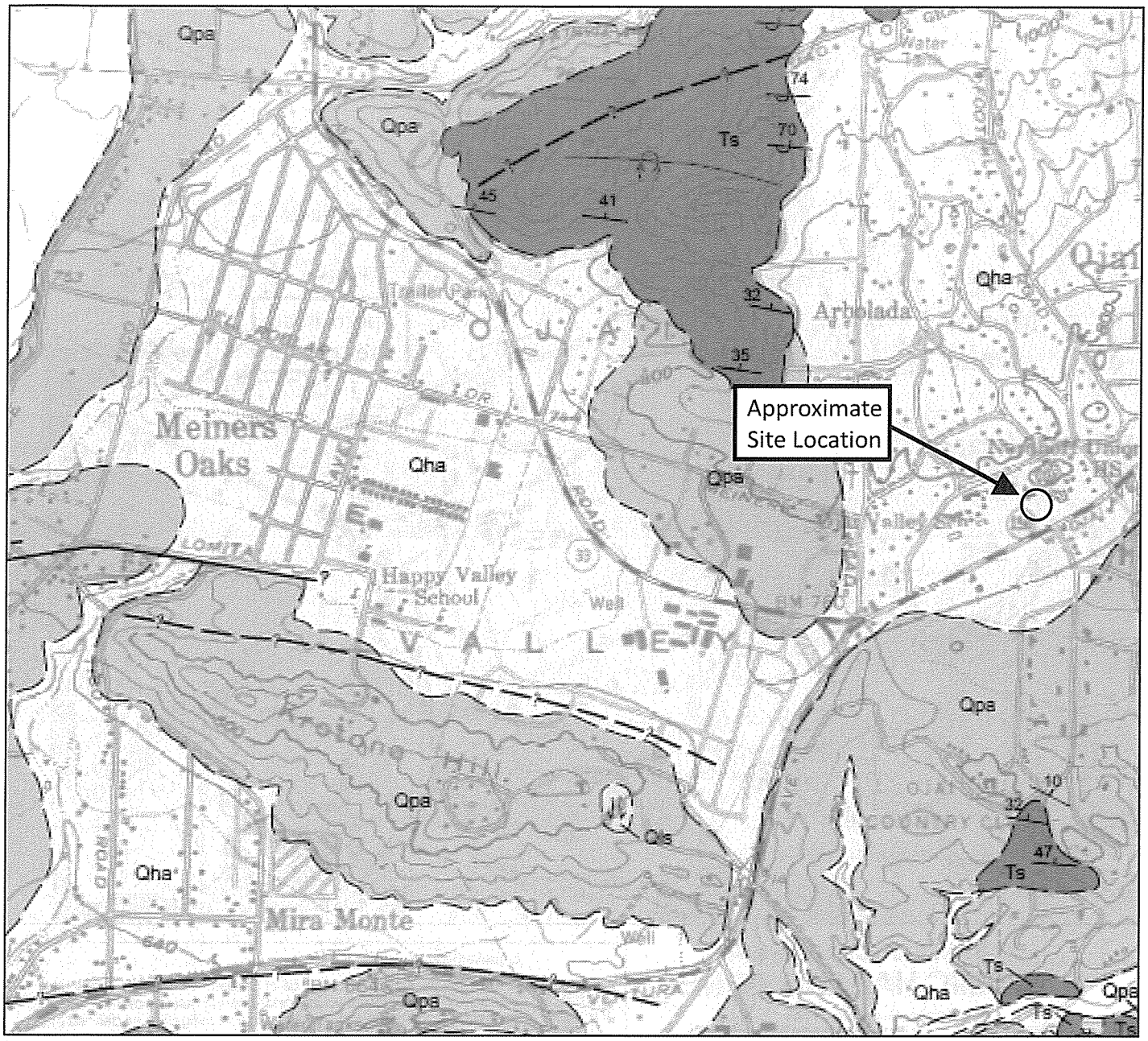
N

REGIONAL GEOLOGIC MAP 1

MATILIJA JR. HIGH SCHOOL
LOCKER ROOM AND KITCHEN REMODELS
OJAI, CALIFORNIA

Earth Systems

July 2018	302294-001



*Taken from USGS, SCAMP Geologic Map of the Matilija 7.5' Quadrangle, California, 2006.

Qha: Alluvial and colluvial deposits, undivided (Holocene) - Located on the floors of valleys; includes active stream deposits in hill slope areas; composed of unconsolidated sandy clay with some gravel.

Qpa: Alluvial deposits, undivided (late Pleistocene) - Consists of semi-consolidated silt, sand, clay, and gravel.

Contact between map units - Dashed where approximately located or inferred; dotted where concealed.

Fault - Solid where accurately located; long dash where approximately located; short dash where inferred; dotted where concealed; queried where location is uncertain. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

Axis of anticline - Approximately located; dotted where concealed. Arrow indicates direction of plunge.

Axis of syncline - Approximately located; dotted where concealed.

Axis of overturned syncline - Approximately located; dotted where concealed.

Strike and dip of bedding.

Vertical bedding.

Strike and dip of overturned bedding.

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

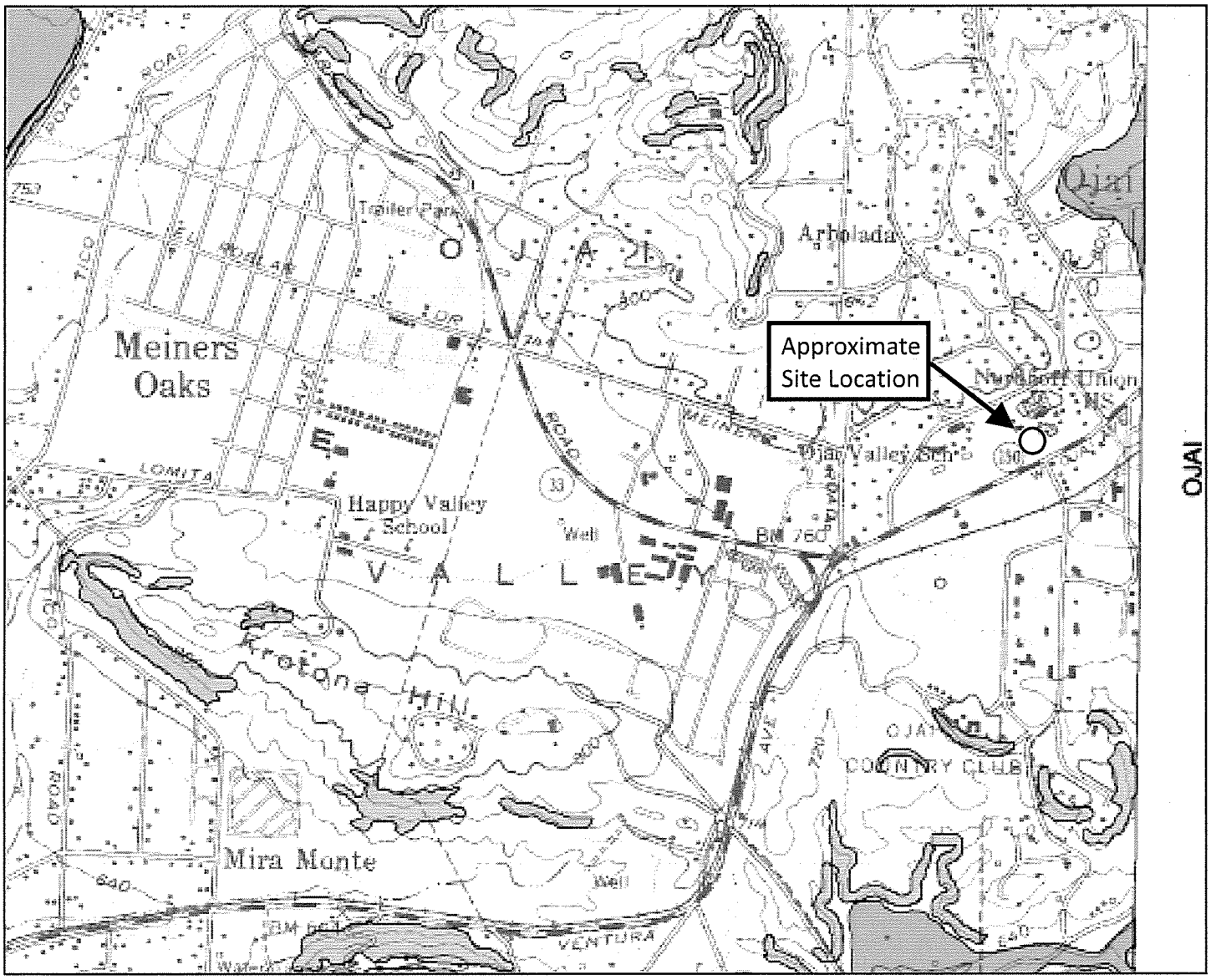
N

REGIONAL GEOLOGIC MAP 2

MATILIJA JR. HIGH SCHOOL
LOCKER ROOM AND KITCHEN REMODELS
OJAI, CALIFORNIA

Earth Systems

July 2018	302294-001



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)

MATILIJA QUADRANGLE

OFFICIAL MAP

Released: April 17, 2003

STATE GEOLOGIST

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

N

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.

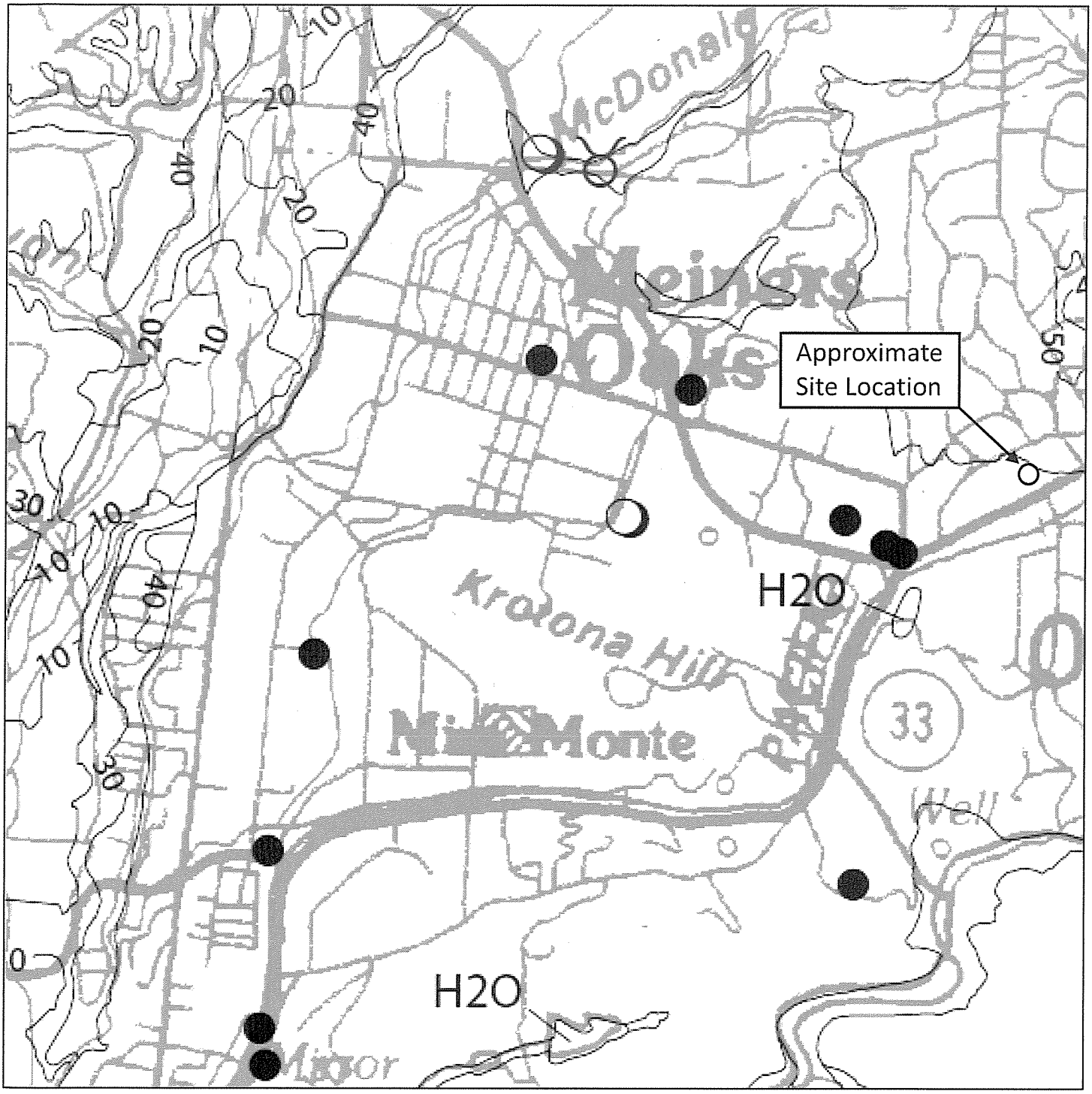
NOTE: Seismic Hazard Zones identified on this map may include developed land where delineated hazards have already been mitigated to city or county standards. Check with your local building/planning department for information regarding the location of such mitigated areas.

SEISMIC HAZARD ZONES MAP

MATILIJA JR. HIGH SCHOOL
LOCKER ROOM AND KITCHEN REMODELS
OJAI, CALIFORNIA

Earth Systems

July 2018	302294-001



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

20 Historically shallowest ground-water depth contours (in feet)

>40 Historically shallowest ground-water depth greater than 40 feet over a broad area

Alluviated Valley Boundary

Borehole Site

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

N

HISTORICAL HIGH GROUNDWATER MAP

MATILIJA JR. HIGH SCHOOL
LOCKER ROOM AND KITCHEN REMODELS
OJAI, CALIFORNIA

Earth Systems

July 2018	302294-001

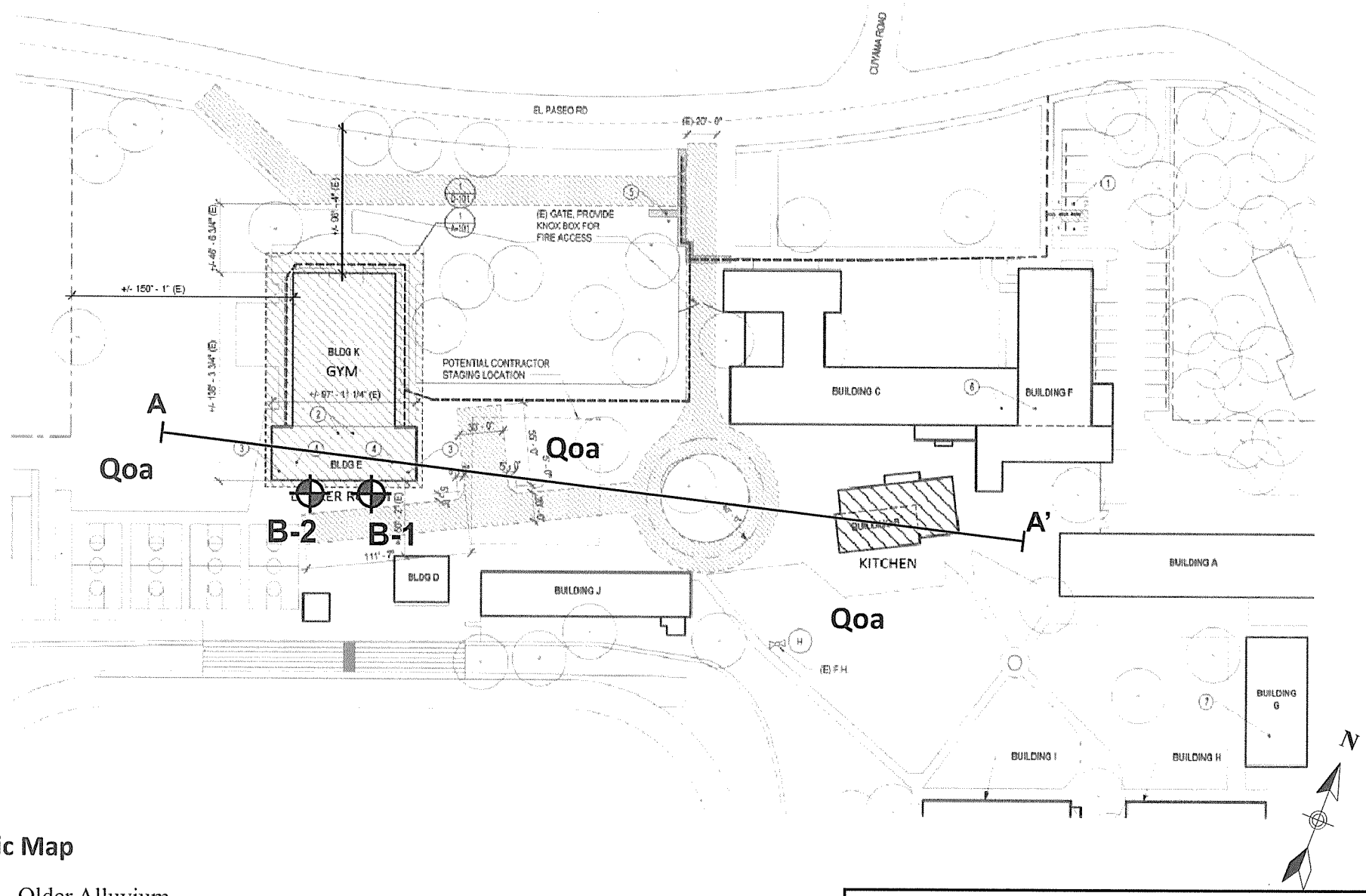
FIELD INVESTIGATION

A. Two borings were drilled to a maximum depth of 23 feet below the existing pavement surface to observe the soil profile and to obtain samples for laboratory analysis. The borings were drilled on July 27, 2004, using a 6-inch outside diameter hollow stem auger powered by a Mobile B80 truck mounted 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 (MC) ring sampler (ASTM D 3550 with shoe similar to ASTM D 1586). The MC sampler has a 3-inch outside diameter and a 2.37-inch inside diameter. The samples were obtained by driving the sampler with a 140-pound hammer dropping 30 inches in accordance with ASTM D 1586. The hammer was a downhole safety type, connected by a 1/2-inch diameter steel cable to a power reversing hydraulic winch that was used to lift and drop the hammer.

C. Bulk samples of the soils encountered were gathered from the auger cuttings.

D. The final logs of the borings represent our interpretation of the contents of the field logs and the results of laboratory testing performed on the samples obtained during the subsurface investigation. The final logs are included in this Appendix.



Geologic Map

Qoa Older Alluvium

B-2 Exploratory Boring Locations

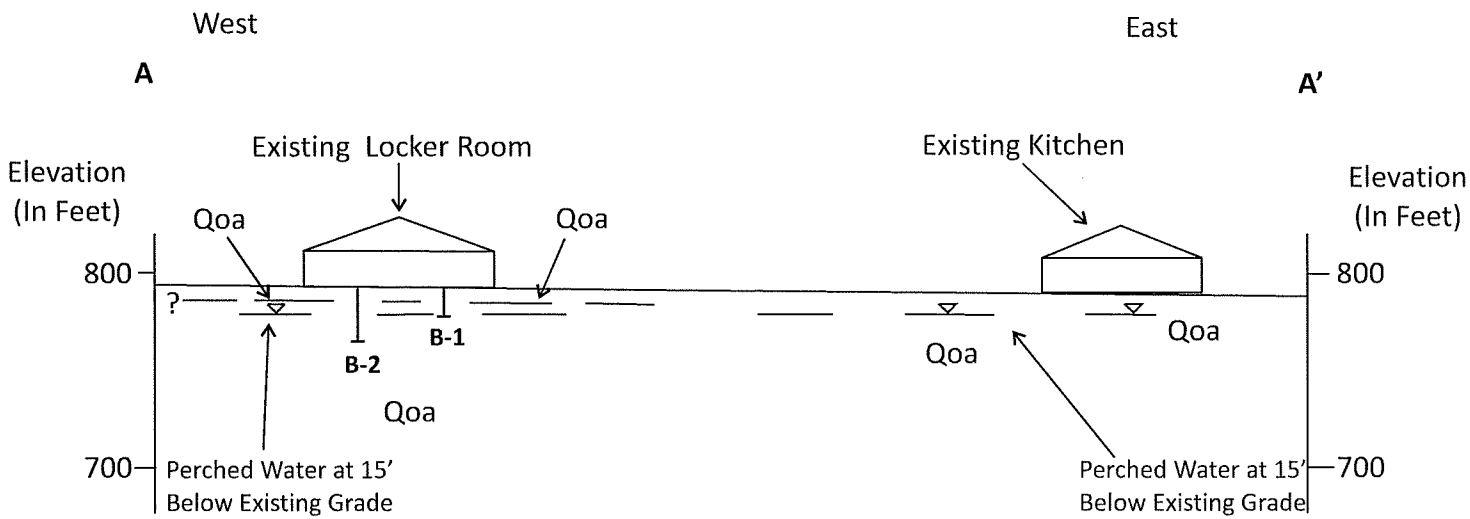
A A' Line Of Cross-section

Approximate Scale: 1" = 100'

0 100' 200'

GEOLOGIC MAP	
Matilija Jr. High School Locker Room and Kitchen Remodels Ojai, California	
Earth Systems	
July 2018	302294-001

GEOLOGIC CROSS-SECTION A-A'



Approximate Scale: 1" = 100'

SITE PLAN

Matilija Jr. High School
Locker Room And Kitchen Remodels
Ojai, California

Earth Systems

July 2018	302294-001

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

BORING NO: 1
PROJECT NAME: Matilija Junior H. S. Locker Rm Expansion
PROJECT NUMBER: VT-23241-01
BORING LOCATION: Per Plan

DRILLING DATE: July 27, 2004
DRILL RIG: Mobile B-80
DRILLING METHOD:6" Hollow Stem Auger
LOGGED BY: Wesley Smith

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6" N)	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									SURFACE: 3" of Asphalt over 4.5" of Base.
	X			3/3/6		CL	111	15	**ARTIFICIAL FILL:** Silty fine sandy clay, low plasticity, moist, medium stiff, dark to moderate reddish brown.
	X			8/13/16		CL	111	17	**ARTIFICIAL FILL:** Same as above to fine to coarse sandy silt, moist, very stiff, dark reddish brown to moderate reddish brown to dark yellowish brown.
5				14/35/48		SM	116	15	**ALLUVIUM:** Very silty slightly clayey fine to coarse sand, some fine to coarse gravel, trace cobbles, moist, very dense, moderate reddish brown to dark yellowish brown.
10				38/50		GM	131	9	**ALLUVIUM:** Very silty slightly clayey fine to coarse gravel with cobbles, slightly moist, very dense, dark reddish brown to dark
15									Refusal at 12.5 feet due to cobbles in a dense matrix. Groundwater was not 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.

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

BORING NO: 2
PROJECT NAME: Matilija Junior H. S. Locker Rm Expansion
PROJECT NUMBER: VT-23241-01
BORING LOCATION: Per Plan

DRILLING DATE: July 27, 2004
DRILL RIG: Mobile B-80
DRILLING METHOD:6" Hollow Stem Auger
LOGGED BY: Wesley Smith

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									**SURFACE:** 3" of Asphalt over 4" of base.
						CL			**ARTIFICIAL FILL:** Fine to coarse sandy clay, reddish brown.
				7/14/22		SC	116	13	**ALLUVIUM:** Clayey silty fine to coarse sand, some fine to coarse gravel, trace cobbles, moist, medium dense, dark reddish brown to moderate yellowish brown.
5				38/45		SC	119	7	Same as above, except more gravel and cobbles.
				13/29/42		SM	116	14	**ALLUVIUM:** Silty slightly clayey fine to coarse sand with fine to coarse gravel and cobbles, slightly moist, dense, moderate reddish brown to moderate yellowish brown.
10				35/50 for 5"		GM	119	13	**ALLUVIUM:** Silty slightly clayey fine to coarse gravel and cobbles, angular to subangular, slightly moist, very dense, moderate reddish brown to moderate yellowish brown.
15				13/29/32		GM	--	8	Same as above, except perched water in sample.
20				19/32/50 for 4"		GM	--	--	ALLUVIUM: Slightly clayey silty fine to coarse gravel and cobbles with fine to coarse sand, angular to subrounded, moist, very dense, dark reddish brown.
25									Refusal at 23.0 feet due to large cobbles in a dense matrix.
									Perched water encountered around 15.0 feet.
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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE				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

Tabulated Laboratory Test Results from 2004
Individual Laboratory Test Results from 2004
Table 1809.7 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 the proposed structure. 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. The relative strength characteristics of the soils were determined from the results of Direct Shear tests on remolded and relatively undisturbed ring samples. Specimens were placed in contact with water at least 24 hours before testing, and were then sheared under normal loads ranging from 0.5 to 2.0 kips per square foot in general accordance with ASTM D 3080.

D. Settlement characteristics were developed from the results of one-dimensional Consolidation tests performed in general accordance with ASTM D 2435. The samples were typically incrementally loaded form 0.125 ksf, flooded with water, and then incrementally loaded to 0.25, 0.50, 1.0, 2.0, 4.0, and 8.0 ksf. The samples were 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 tests in the form of percent consolidation versus log of pressure curves are presented in this Appendix.

E. An expansion index test was performed on 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. A maximum density test was performed to estimate the moisture-density relationship of typical soil materials. The test was performed in accordance with ASTM designation D 1557.

TEST RESULTS

BORING AND DEPTH	1 @ 0'-5'
USCS	ML
MAXIMUM DENSITY (pcf)	129
OPTIMUM MOISTURE (%)	10
COHESION (psf)	150
ANGLE. OF INT. FRICTION (°)	36
EXPANSION INDEX	1
GRAVEL (%)	5
SAND (%)	45
SILT (%)	29
CLAY (%)	21
pH	--
RESISTIVITY (ohms/cm)	--
SOLUBLE CHLORIDES ((mg/kg)	--
SOLUBLE SULFATES (mg/kg)	--

"UNDISTURBED" DIRECT SHEAR RESULTS

BORING AND DEPTH	2 @ 2'
IN-PLACE DENSITY (pcf)	114
IN-PLACE MOISTURE (%)	3
COHESION (psf)	1090
ANGLE. OF INT. FRICTION (°)	38

GRAIN SIZE DISTRIBUTION (%)

BORING AND DEPTH 2 @ 15'

GRAVEL	49
SAND	39
SILT	10
CLAY	2

VT-23241-01

MAXIMUM DENSITY / OPTIMUM MOISTURE

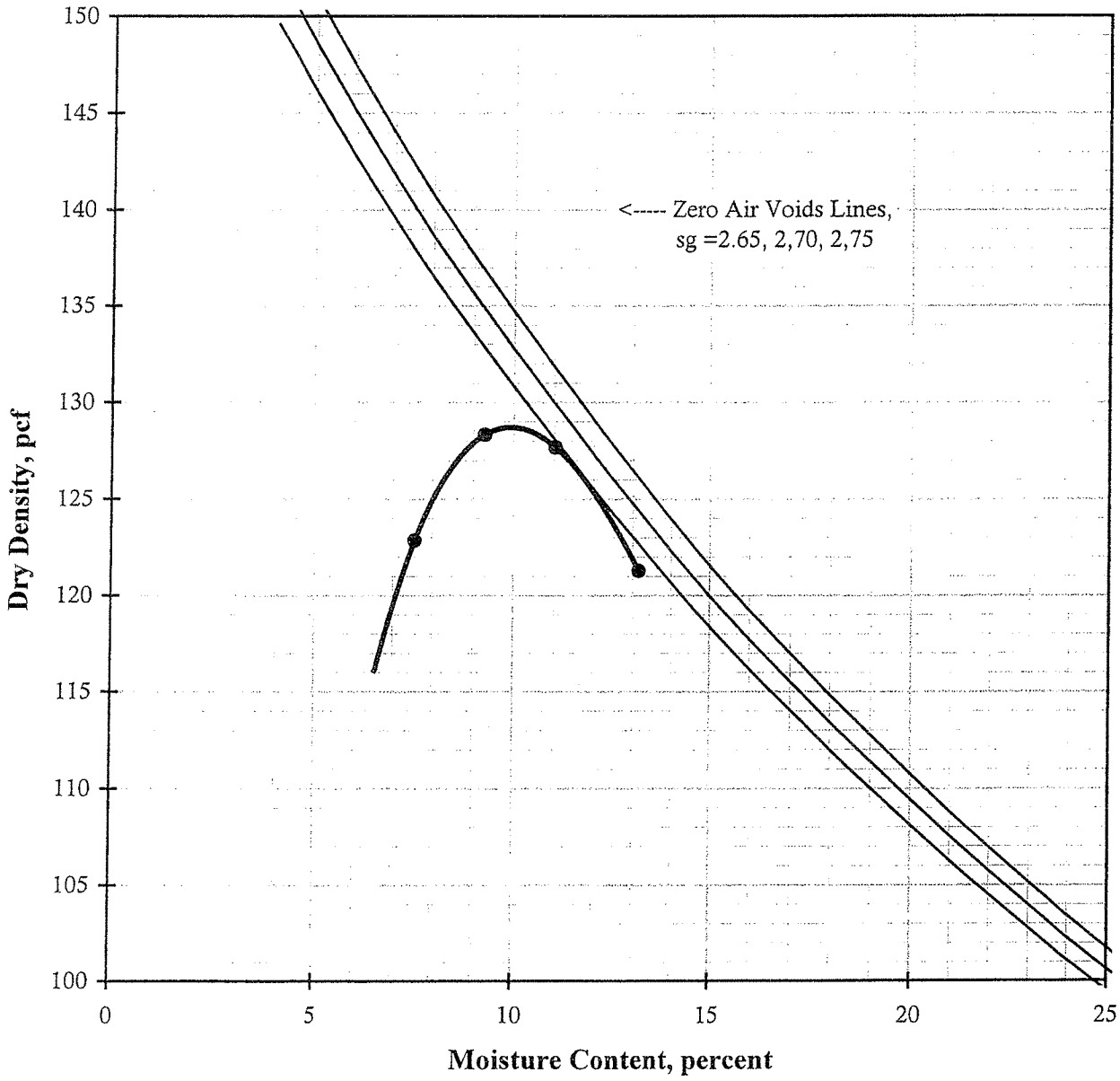
ASTM D 1557-91 (Modified)

Job Name: Matilija Juinor High School
Sample ID: Clayey Silty Sand
Location:
Description: 1 @ 0 - 5

Procedure Used: A
Prep. Method: Moist
Rammer Type: Manual

Maximum Density: **129 pcf**
Optimum Moisture: **10%**

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

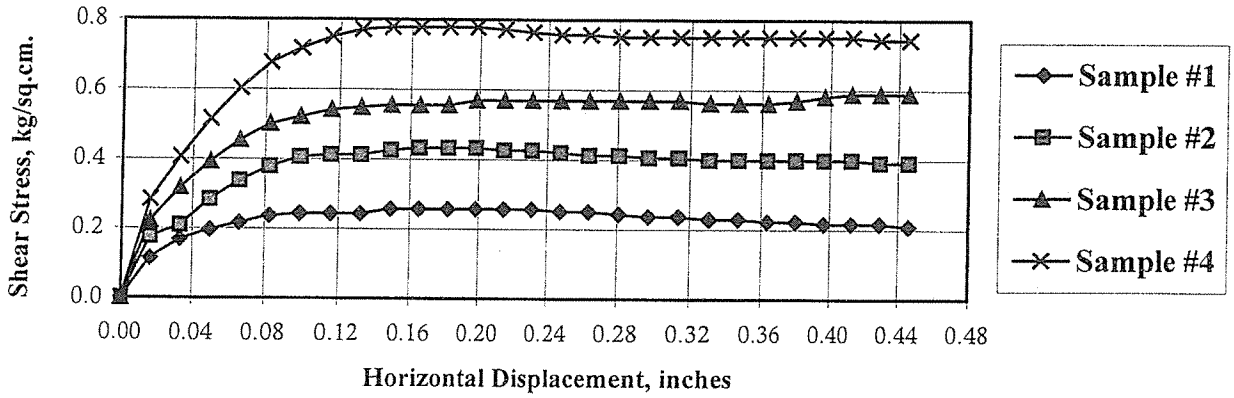


DIRECT SHEAR

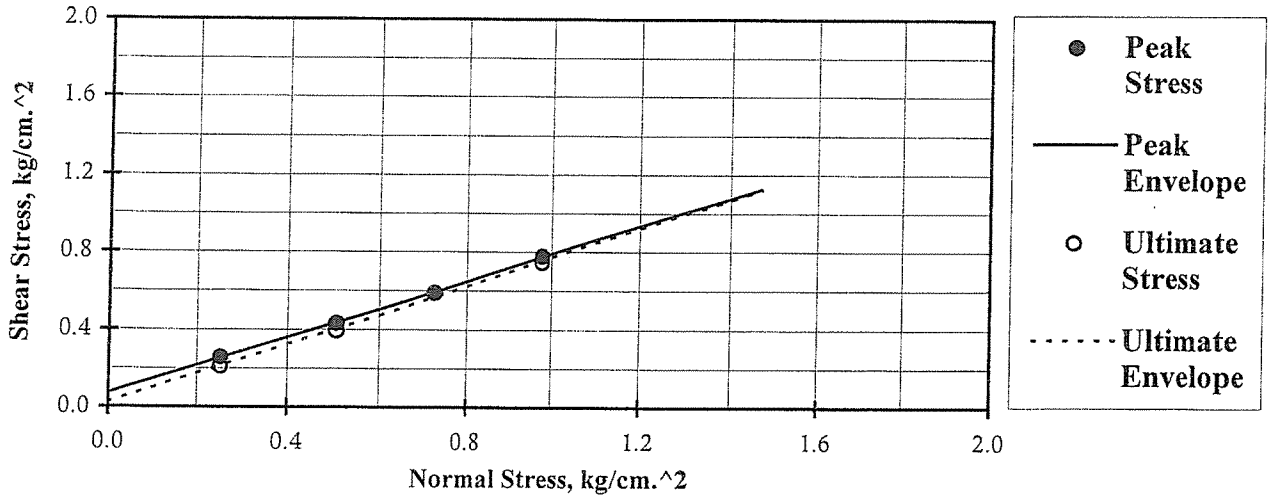
Matilija Juinor High School
1 @ 0 - 5
Clayey Silty Sand
Remolded

Initial Dry Density: 115.9 pcf
Initial Mosture Content: 10.0 %
Peak Friction Angle (Ø): 36°
Cohesion (c): 0.073 kg/cm^2 (150 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	116.0	115.9	115.4	116.4	115.9
Moisture Content, %	10.0	10.0	10.0	10.0	10.0
Saturation, %	61	61	60	62	61
At Test					
Moisture Content, %	15.4	15.4	16.1	15.3	15.5
Saturation, %	94	94	97	95	95
Normal Stress, kg/cm^2	0.25	0.51	0.73	0.98	
Peak Stress, kg/cm^2	0.26	0.43	0.59	0.78	
Ultimate Stress, kg/cm^2	0.21	0.39	0.59	0.74	



SHEAR vs. NORMAL STRESS DIAGRAM

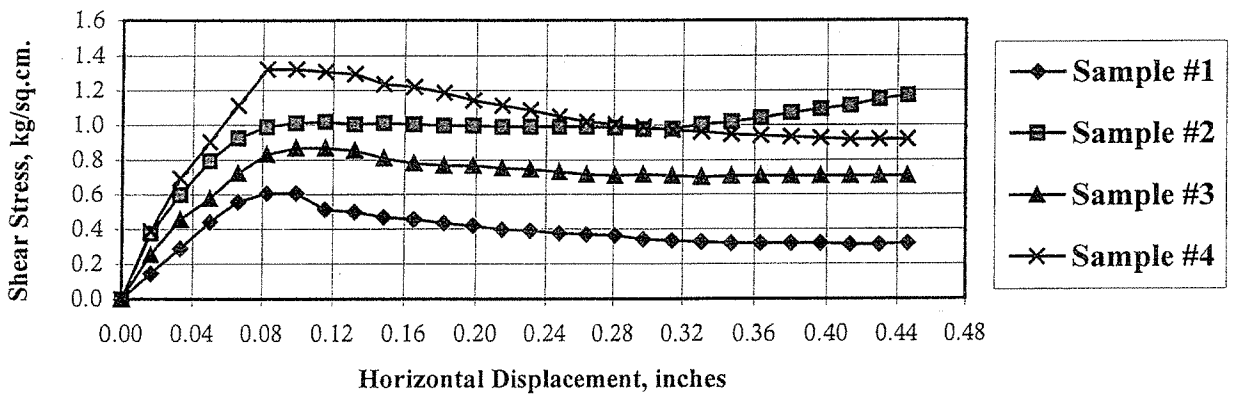


DIRECT SHEAR

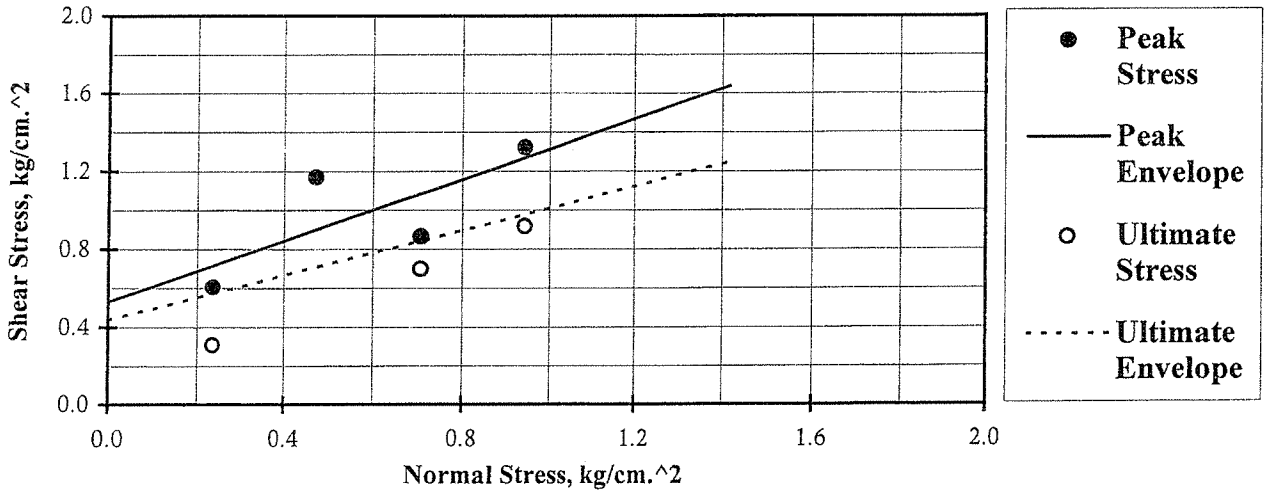
Matilija Juinor High School
2 @ 2
Sandy Clayey Silt
Remolded

Initial Dry Density: 114.4 pcf
Initial Mosture Content: 13.0 %
Peak Friction Angle (Ø): 38°
Cohesion (c): 0.531 kg/cm^2 (1090 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	113.6	115.6	113.5	115.1	114.4
Moisture Content, %	13.0	13.0	13.0	13.0	13.0
Saturation, %	74	79	74	78	76
At Test					
Moisture Content, %	16.5	15.7	16.9	15.7	16.2
Saturation, %	94	95	97	94	95
Normal Stress, kg/cm^2	0.24	0.47	0.71	0.94	
Peak Stress, kg/cm^2	0.61	1.17	0.87	1.32	
Ultimate Stress, kg/cm^2	0.31	1.17	0.70	0.92	



SHEAR vs. NORMAL STRESS DIAGRAM

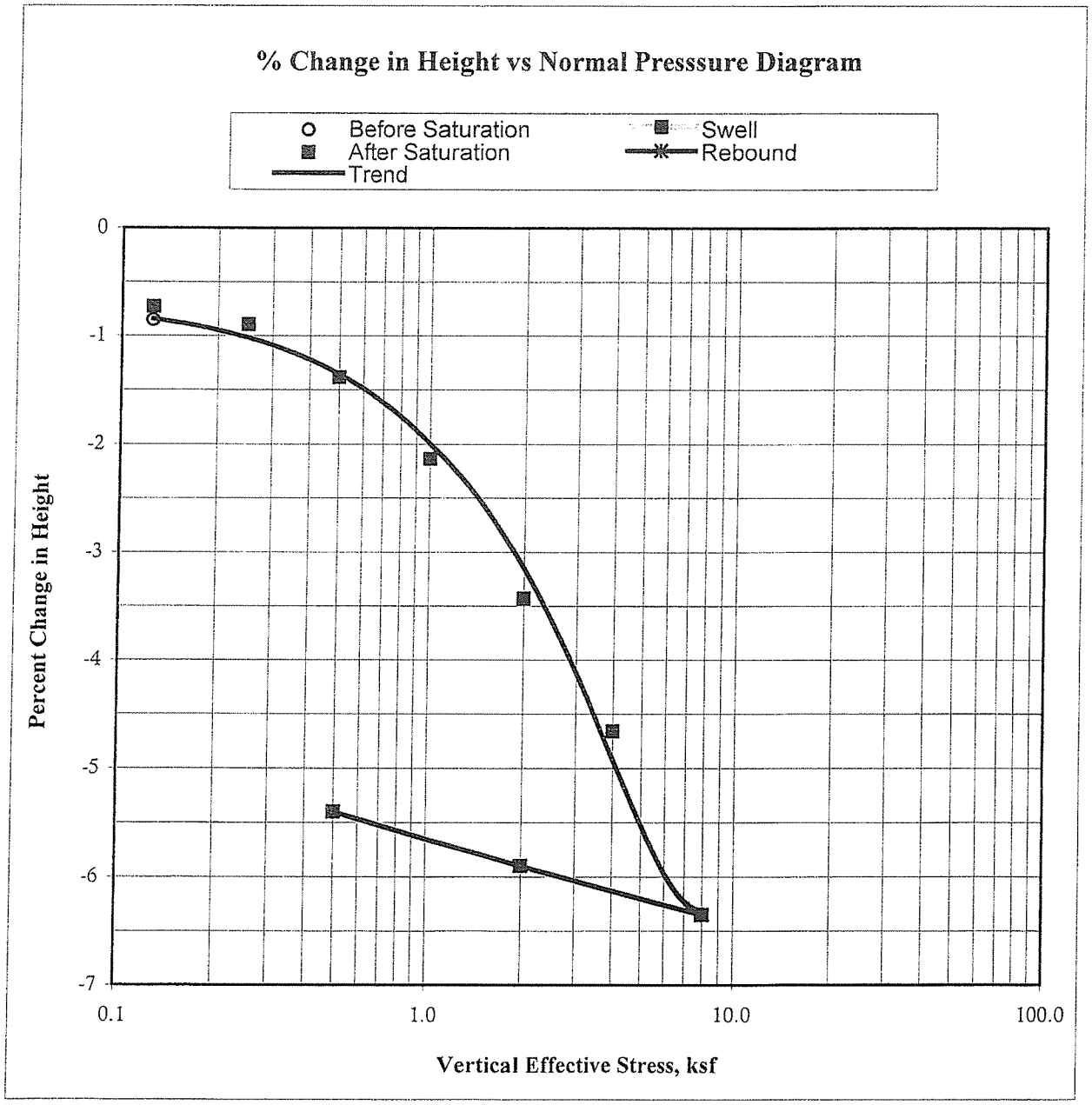


CONSOLIDATION TEST

ASTM D 2435-90

Matilija Juinor High School
1 @ 1
Clayey Sandy Silt
Ring Sample

Initial Dry Density: 110.4 pcf
Initial Moisture, %: 14.9%
Specific Gravity: 2.67 (assumed)
Initial Void Ratio: 0.510

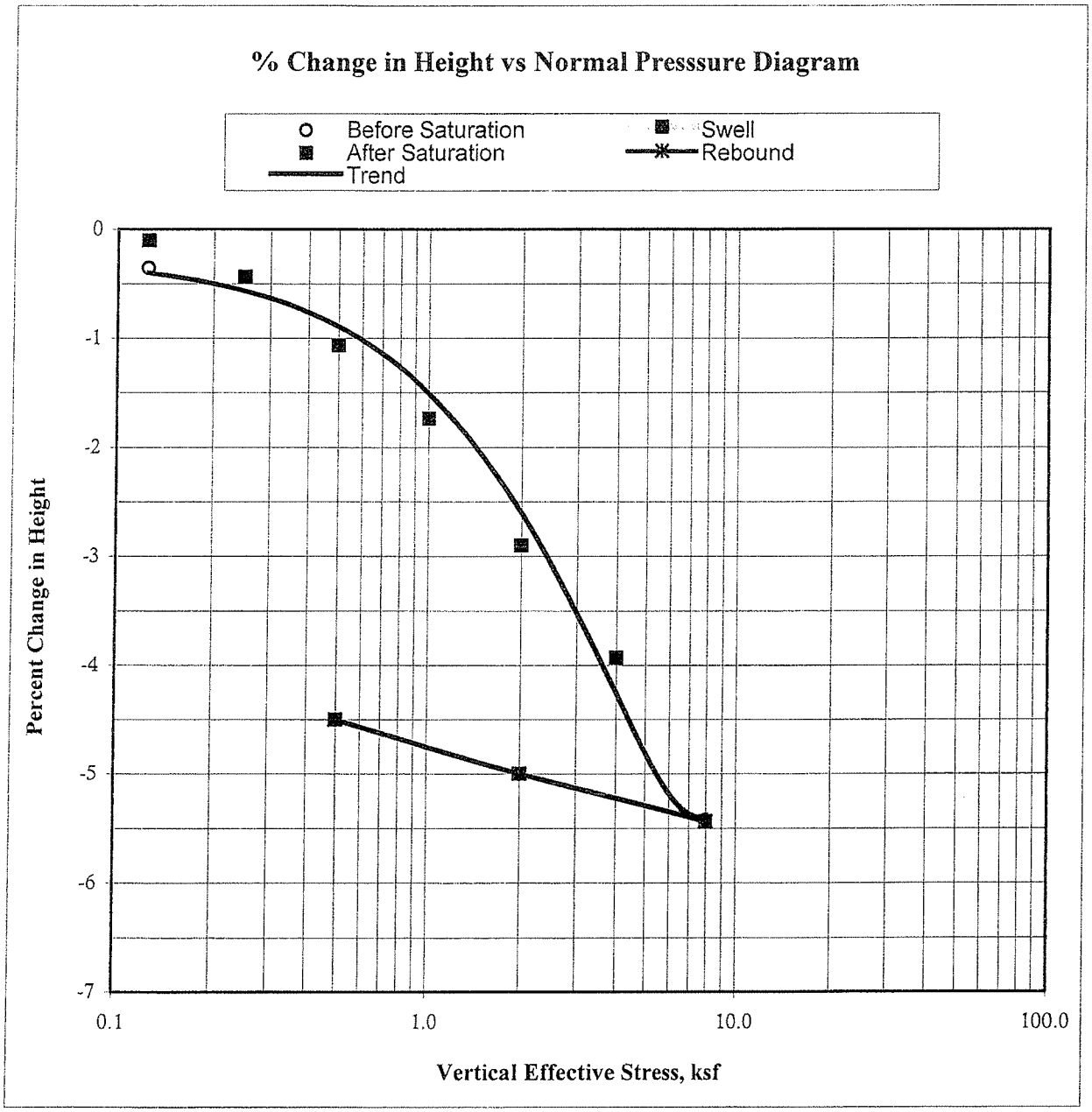


CONSOLIDATION TEST

ASTM D 2435-90

Matilija Juinor High School
1 @ 3
Clayey Sandy Silt
Ring Sample

Initial Dry Density: 111.7 pcf
Initial Moisture, %: 16.9%
Specific Gravity: 2.67 (assumed)
Initial Void Ratio: 0.493



Capco Analytical Services INC. (CAS)
1536 Eastman Avenue, Suite B
Ventura CA 93003
(805) 644-1095

Client: Earth Systems
Sample ID: B 1 @ 0-5
Date Received: 08/20/04
Date Sampled: 08/20/04

Sample Matrix: Soil
CAS LAB NO: 04156601

WET CHEMISTRY ANALYSIS SUMMARY

COMPOUND	RESULT	UNITS	DF	PQL	METHOD	ANALYZED
*Chloride	BQL	mg/Kg	1	10	300.0M	08/24/04
pH	7.5	S.U.	1	---	9045	08/24/04
*Resistivity	4740	ohms-cm	1	3	CA test 424	08/24/04
*Sulfate	130	mg/Kg	1	10	300.0M	08/24/04

*Sample was analyzed on a 1:3 soil/water extract. Results were reported based on the original soil sample weight.
PQL: Practical Quantitation Limit
BQL: Below Practical Quantitation Limit

Principal Analyst

CAPCO Analytical Services, Inc.

TABLE 1809.7
PRESCRIPTIVE FOOTINGS FOR SUPPORTING WALLS OF LIGHT FRAME CONSTRUCTION*

WEIGHTED EXPANSION INDEX (13)	FOUNDATION FOR SLAB & RAISED FLOOR SYSTEM (4) (8)							CONCRETE SLABS (8) (12)		PREMOISTENING OF SOILS UNDER FOOTINGS, PIERS AND SLABS (4) (5)	RESTRICTION ON PIERS UNDER RAISED FLOORS
	NUMBER OF STORIES	STEM THICKNESS	FOOTING WIDTH	FOOTING THICKNESS	ALL PERIMETER FOOTINGS (5)	INTERIOR FOOTINGS FOR SLAB AND RAISED FLOORS (5)	REINFORCEMENT FOR CONTINUOUS FOUNDATIONS (2) (6)	3-1/2" MINIMUM THICKNESS			
					DEPTH BELOW NATURAL SURFACE OF GROUND AND FINISH GRADE			REINFORCEMENT (3)	TOTAL THICKNESS OF SAND (10)		
		(INCHES)									
0 - 20 Very Low (non-expansive)	1 2 3	6 8 10	12 15 18	6 6 8	12 18 24	12 18 24	1-#4 top and bottom	#4 @ 48" o.c. each way, or #3 @ 36" o.c. each way	2"	Moistening of ground recommended prior to placing concrete	Piers allowed for single floor loads only
21-50 Low	1 2 3	6 8 10	12 15 18	6 6 8	15 18 24	12 18 24	1-#4 top and bottom	#4 @ 48" o.c. each way, or #3 @ 36" o.c. each way	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	6 8	12 15	6 6	21 21	12 18	1-#4 top and bottom	#3 @ 24" o.c. each way	4"	130% of optimum moisture required to a depth of 27" below lowest adjacent grade. Testing required	Piers not allowed
	3	10	18	8	24	24	#3 bars @ 24" in ext. footing Bend 3' into slab (7)				
91-130 High	1 2	6 8	12 15	6 6	27 27	12 18	2-#4 Top and Bottom	#3 @ 24" o.c. each way	4"	140% of optimum moisture required to a depth of 33" below lowest adjacent grade. Testing required.	Piers not allowed
	3	10	18	8	27	24	#3 bars @ 24" in ext. footing Bend 3' into slab (7)				
Above 130 Very High	Special design by licensed engineer/architect										

*Refer to next page for footnotes (1) through (14).

APPENDIX C

Site Class Analysis
2016 CBC & ASCE 7-10 Seismic Parameters
USGS Design Maps Reports
Spectral Response Values
Response Spectra Curves
Fault Parameters

EARTH SYSTEMS PACIFIC	Job Number: 302294-001 Job Name: Matilija JHS Locker Rm & Kitchen Calc Date: 7/9/2018 CPT/Boring ID: B-2

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	44.1	5.0	0.113
15.0	16.4	5.0	0.305
20.0	23.9	5.0	0.209
100.0	23.9	80.0	3.347

Total Thickness of Soil =	100.00	ft
N-bar Value =	23.4	*
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:		34.445 N		
Longitude:		-119.255 W		
Maximum Considered Earthquake (MCE) Ground Motion				
Short Period Spectral Reponse	S_S	**2.219 g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	S_1	**0.828 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}	2.219 g	$= F_a*S_S$	
	S_{M1}	1.242 g	$= F_v*S_1$	
Design Earthquake Ground Motion				
Short Period Spectral Reponse	S_{DS}	**1.479 g**	$= 2/3*S_{MS}$	
1 second Spectral Response	S_{D1}	**0.828 g**	$= 2/3*S_{M1}$	
	To	0.11 sec	$= 0.2*S_{D1}/S_{DS}$	
	Ts	0.56 sec	$= S_{D1}/S_{DS}$	
Seismic Importance Factor	I	1.25	Table 1604.5	
	F_{PGA}	1.00		

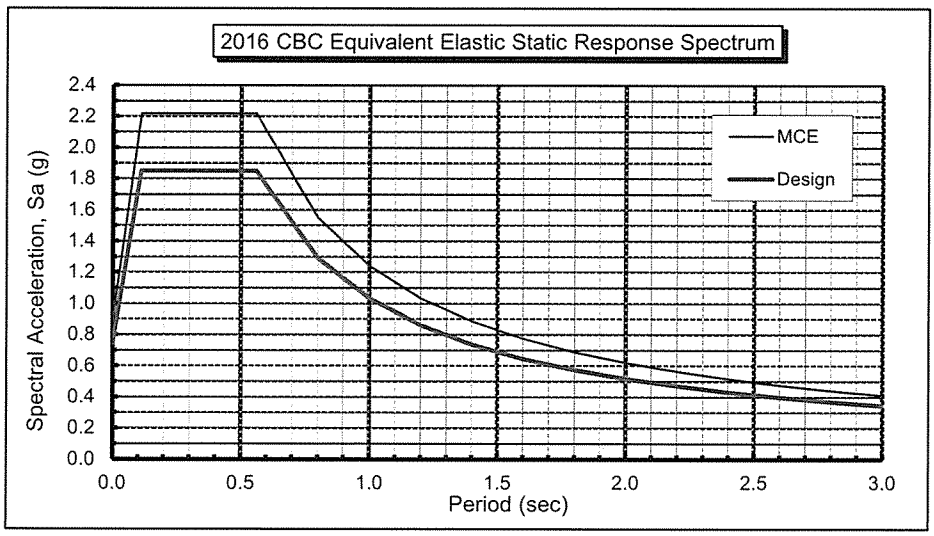


Table 11.5-1

Period T (sec)	Design Sa (g)
0.00	0.740
0.05	1.235
0.11	1.849
0.56	1.849
0.80	1.294
1.00	1.035
1.20	0.863
1.40	0.739
1.60	0.647
1.80	0.575
2.00	0.518
2.20	0.470
2.40	0.431
2.60	0.398
2.80	0.370
3.00	0.345

USGS Design Maps Summary Report

User–Specified Input

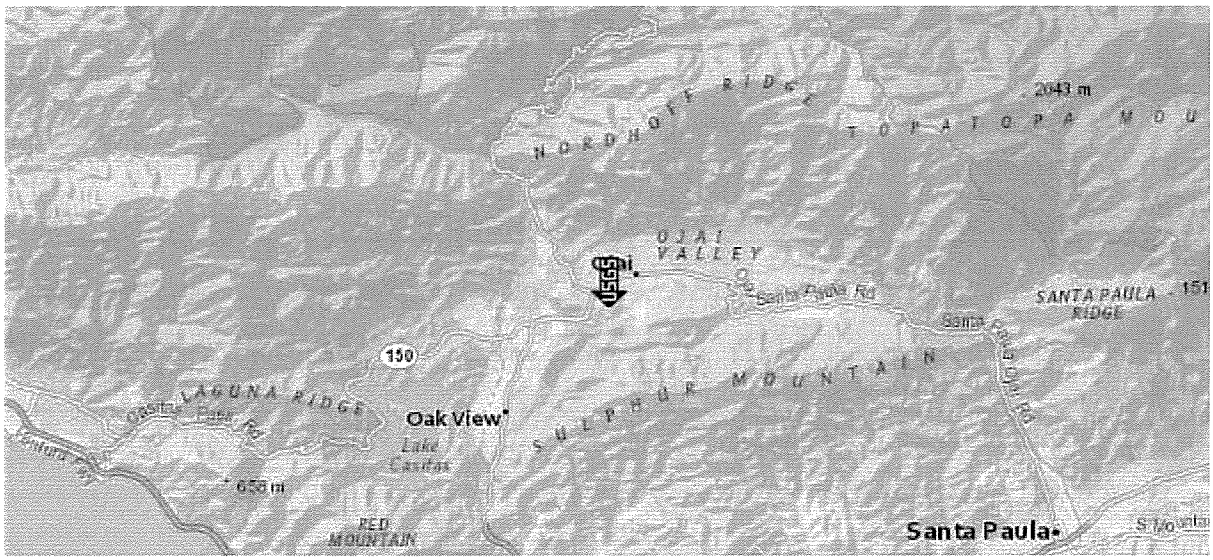
Report Title Matilija Jr. HS Locker Room & Kitchen Remodels
Mon July 9, 2018 16:52:35 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 34.4453°N, 119.2552°W

Site Soil Classification Site Class D – "Stiff Soil"

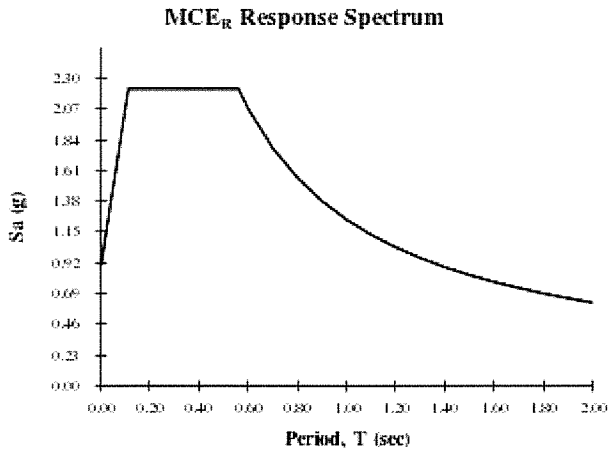
Risk Category I/II/III

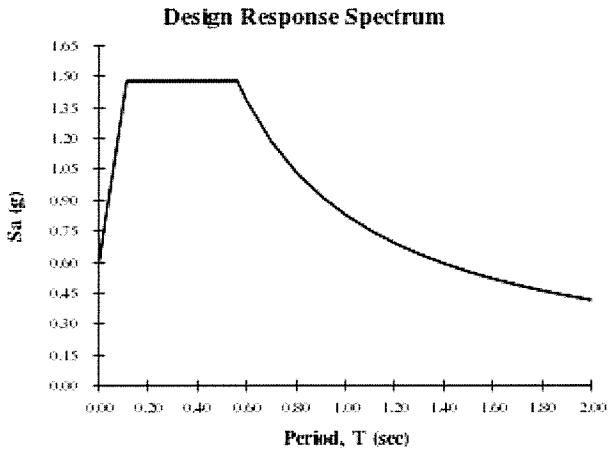


USGS–Provided Output

S_S =	2.219 g	S_{MS} =	2.219 g	S_{DS} =	1.479 g
S_1 =	0.828 g	S_{M1} =	1.242 g	S_{D1} =	0.828 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.4453°N, 119.2552°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 2.219$ g

From Figure 22-2 [2] $S_1 = 0.828$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 2.219 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.828 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_a S_S = 1.000 \times 2.219 = 2.219$ g

Equation (11.4–2): $S_{M1} = F_v S_1 = 1.500 \times 0.828 = 1.242$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

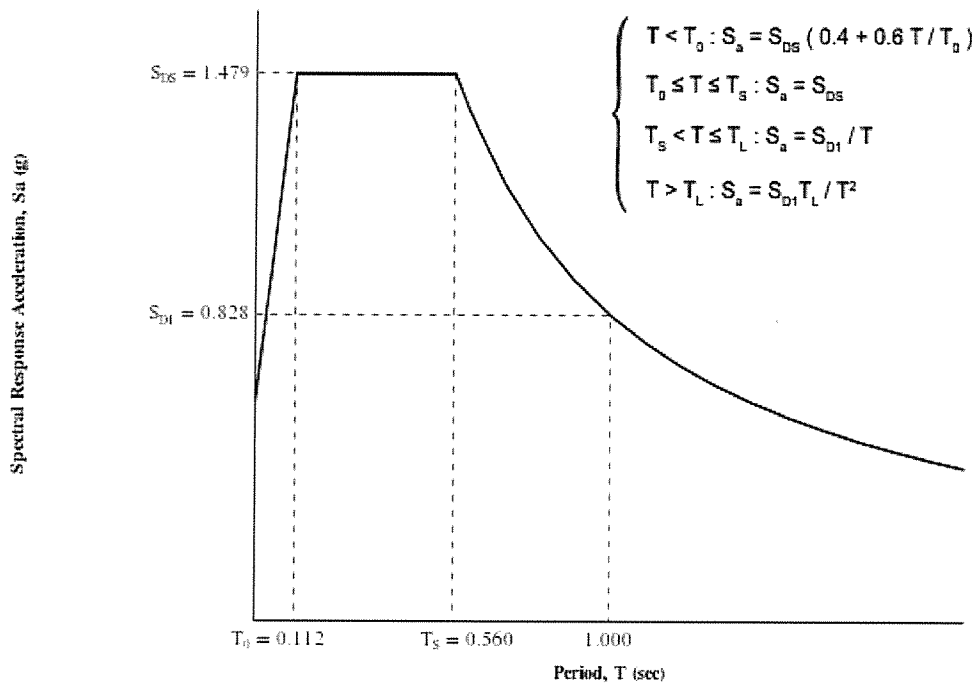
Equation (11.4–3): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 2.219 = 1.479$ g

Equation (11.4–4): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 1.242 = 0.828$ g

Section 11.4.5 — Design Response Spectrum

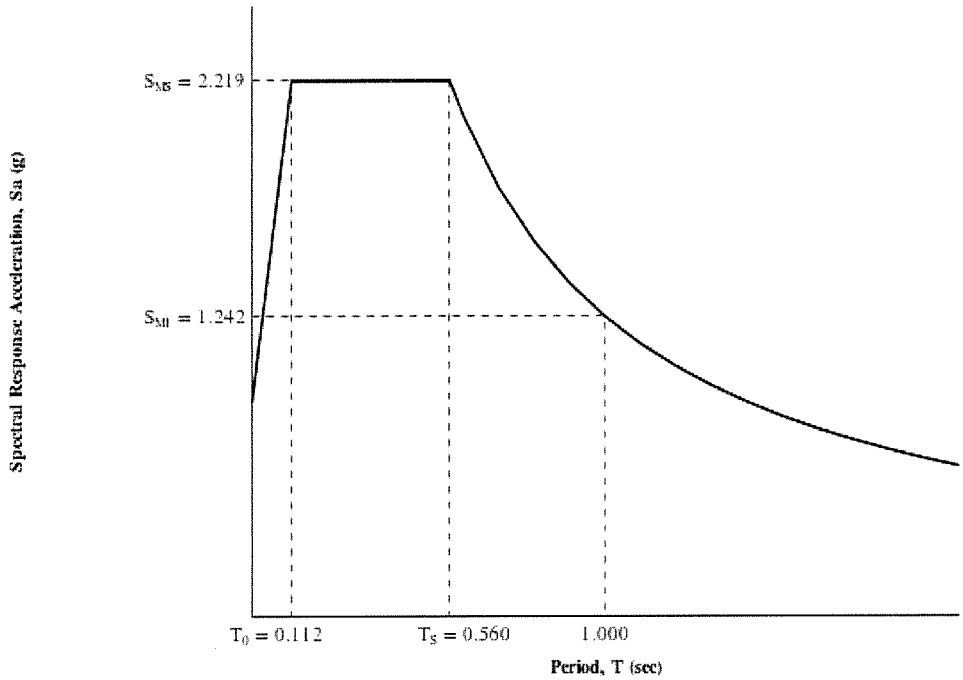
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.804

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.804 = 0.804$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.804 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.977

From Figure 22-18 [6] C_{R1} = 0.978

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.479 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.828 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

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	0.819	0.880	0.984	0.600	0.984	**0.880**	**0.888**	0.587	0.592
0.05	1.082	1.163	1.155	0.975	1.155	**1.155**	**1.482**	0.791	0.988
0.10	1.346	1.447	1.516	1.350	1.516	**1.447**	**2.077**	1.108	1.385
0.15	1.537	1.652	1.806	1.500	1.806	**1.652**	**2.219**	1.183	1.479
0.20	1.728	1.857	1.984	1.500	1.984	**1.857**	**2.219**	1.238	1.479
0.30	1.797	1.931	2.089	1.500	2.089	**1.931**	**2.219**	1.288	1.479
0.40	1.733	1.947	2.174	1.500	2.174	**1.947**	**2.219**	1.298	1.479
0.50	1.668	1.956	2.183	1.500	2.183	**1.956**	**2.219**	1.304	1.479
0.75	1.426	1.742	1.951	1.200	1.951	**1.742**	**1.656**	1.161	1.104
1.00	1.183	1.504	1.638	0.900	1.638	**1.504**	**1.242**	1.003	0.828
1.50	0.913	1.161	1.178	0.600	1.178	**1.161**	**0.828**	0.774	0.552
2.00	0.643	0.818	0.887	0.450	0.887	**0.818**	**0.621**	0.545	0.414

Crs: 0.977

Cr1: 0.978

* > 80% of (9)

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

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

Mapped MCE Acceleration Values			Site Coefficients		**Site-Specific Design Acceleration Values**		
PGA	0.804	g	F_{PGA}	1.00	**PGA_M**	**0.804**	**g**
Ss	2.219	g	F_a	1.00	**S_{DS}**	**1.238**	**g**
S_1	0.828	g	F_v	1.50	**S_{D1}**	**1.090**	**g**

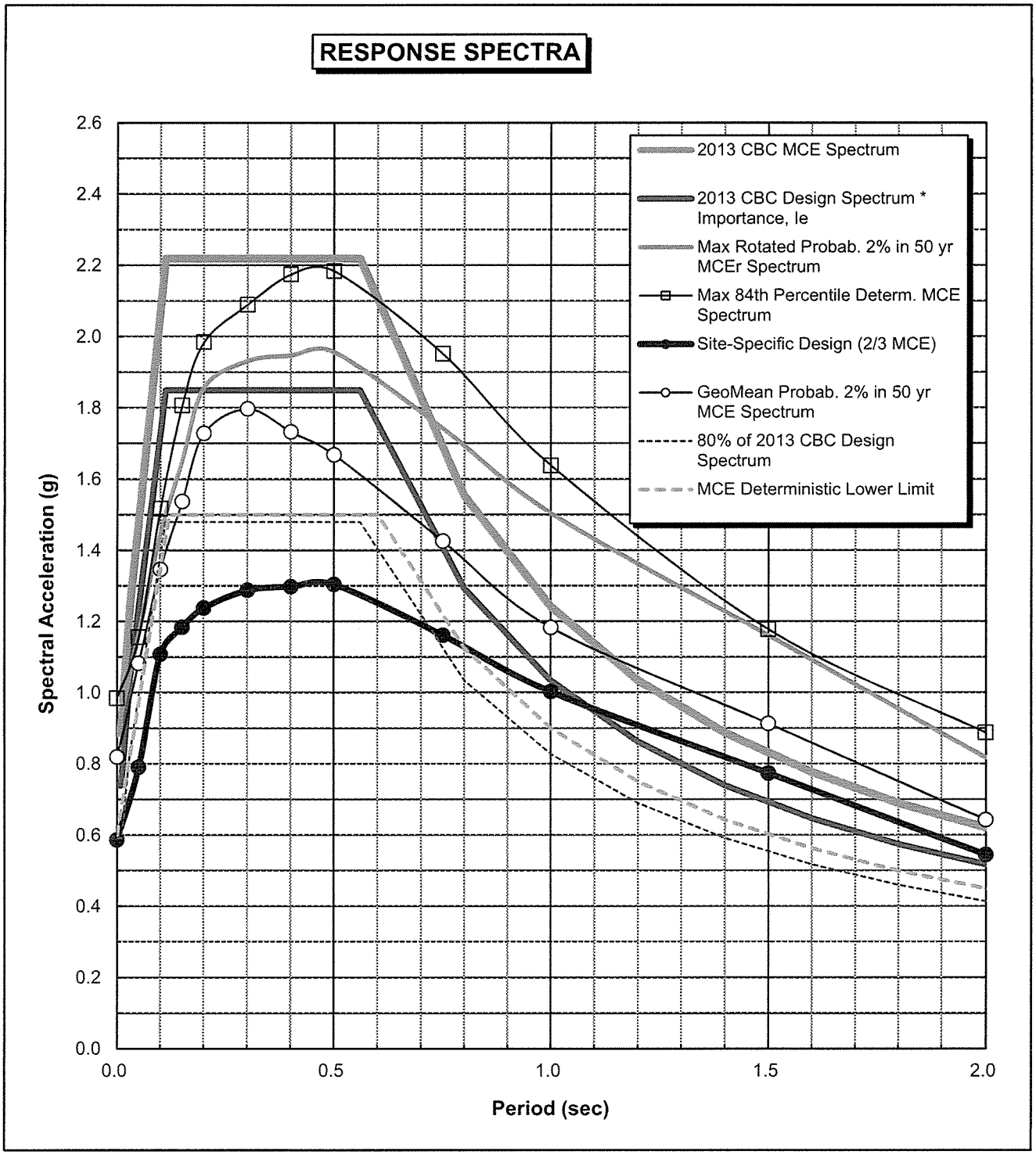
Spectral Amplification Factor for different viscous damping, D (%):

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: 34.4453
Longitude: -119.2552

Response Spectra

Matilija Jr. High School Locker Room and Kitchen Remodels
File No.: 302294-001

Earth Systems

Table 1
Fault Parameters

Fault Section Name	Distance (miles)	Distance (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)
Mission Ridge-Arroyo Parida-Santa Ana	0.8	1.3	70	176	90	69	B	**6.8**		0.4
Sisar	2.8	4.5	29	168	na	20	B'	**7.0**		
Santa Ynez (East)	5.2	8.3	70	172	0	68	B	**7.2**		2
San Cayetano	5.5	8.8	42	3	90	42	B	**7.2**		6
Oak Ridge (Onshore)	7.4	11.9	65	159	90	49	B	**7.4**		4
Red Mountain	7.9	12.7	56	2	90	101	B	**7.4**		2
Ventura-Pitas Point	9.7	15.6	64	353	60	44	B	**6.9**		1
Pine Mtn	10.6	17.1	45	5	na	62	B'	**7.3**		
North Channel	13.0	21.0	26	10	90	51	B	**6.7**		1
Oak Ridge (Offshore)	13.4	21.6	32	180	90	38	B	**6.9**		3
Big Pine (Central)	16.5	26.6	76	167	na	23	B'	**6.3**		
Big Pine (West)	18.1	29.1	50	2	na	18	B'	**6.5**		
Simi-Santa Rosa	18.1	29.1	60	346	30	39	B	**6.8**		1
Big Pine (East)	20.4	32.9	73	338	na	23	B'	**6.6**		
Pitas Point (Upper)	21.8	35.1	42	15	90	35	B	**6.8**		1
Santa Ynez (West)	22.0	35.3	70	182	0	63	B	**6.9**		2
Pitas Point (Lower)-Montalvo	22.8	36.6	16	359	90	30	B	**7.3**		2.5
Nacimiento	24.4	39.3	66	40	na	113	B'	**7.1**		
Malibu Coast (Extension), alt 1	25.9	41.7	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	25.9	41.7	74	4	30	35	B'	**6.9**		
Channel Islands Western Deep Ramp	26.4	42.5	21	204	90	62	B'	**7.3**		
Oak Ridge (Offshore), west extension	27.5	44.2	67	195	na	28	B'	**6.1**		
Santa Susana, alt 2	28.0	45.1	53	10	90	43	B'	**6.8**		
Santa Susana, alt 1	28.4	45.8	55	9	90	27	B	**6.8**		5
San Gabriel	28.5	45.8	61	39	180	71	B	**7.3**		1
Del Valle	28.6	46.0	73	195	90	9	B'	**6.3**		
Holser, alt 1	28.6	46.0	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	28.6	46.0	58	182	90	17	B'	**6.7**		
San Andreas (Big Bend)	28.8	46.3	90	198	180	50	A	**7.8**	108	34
Channel Islands Thrust	29.0	46.6	20	354	90	59	B	**7.3**		1.5
Northridge	31.0	49.9	35	201	90	33	B	**6.8**		1.5
Santa Cruz Island	31.9	51.4	90	188	30	69	B	**7.1**		1
Garlock (West)	32.4	52.1	90	149	0	98	A	**7.6**	493	6
San Andreas (Mojave N)	32.5	52.3	90	199	180	37	A	**7.8**	106	27
Northridge Hills	32.9	52.9	31	19	90	25	B'	**7.0**		
Pitas Point (Lower, West)	33.1	53.2	13	3	90	35	B	**7.2**		2.5
Anacapa-Dume, alt 1	33.1	53.3	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	33.1	53.3	41	352	60	65	B	**7.2**		3
Malibu Coast, alt 1	33.1	53.3	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	33.1	53.3	74	3	30	38	B	**6.9**		0.3

Reference: USGS OFR 2007-1437 (CGS SP 203) Based on Site Coordinates of 34.4453 Latitude, -119.2552 Longitude

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

APPENDIX D

Liquefaction Analysis Printout
Seismic-Induced Settlement Analysis Printout

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: Matilija Jr. HS Locker Room & Kitchen Remod
Job No: 302294-001
Date: 7/10/2018
Boring: B2 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: **7.2** 7.5
PGA, g: **0.80** 0.72
MSF: 1.11
GWT: **15.0** feet
Calc GWT: **15.0** feet
Remediate to: **5.0** **feet**

SPT N VALUE CORRECTIONS:

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

Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
2.5	0.6

Required SF: 1.30
Threshold Acceler., g: 0.28 **Minimum Calculated SF:** 0.35

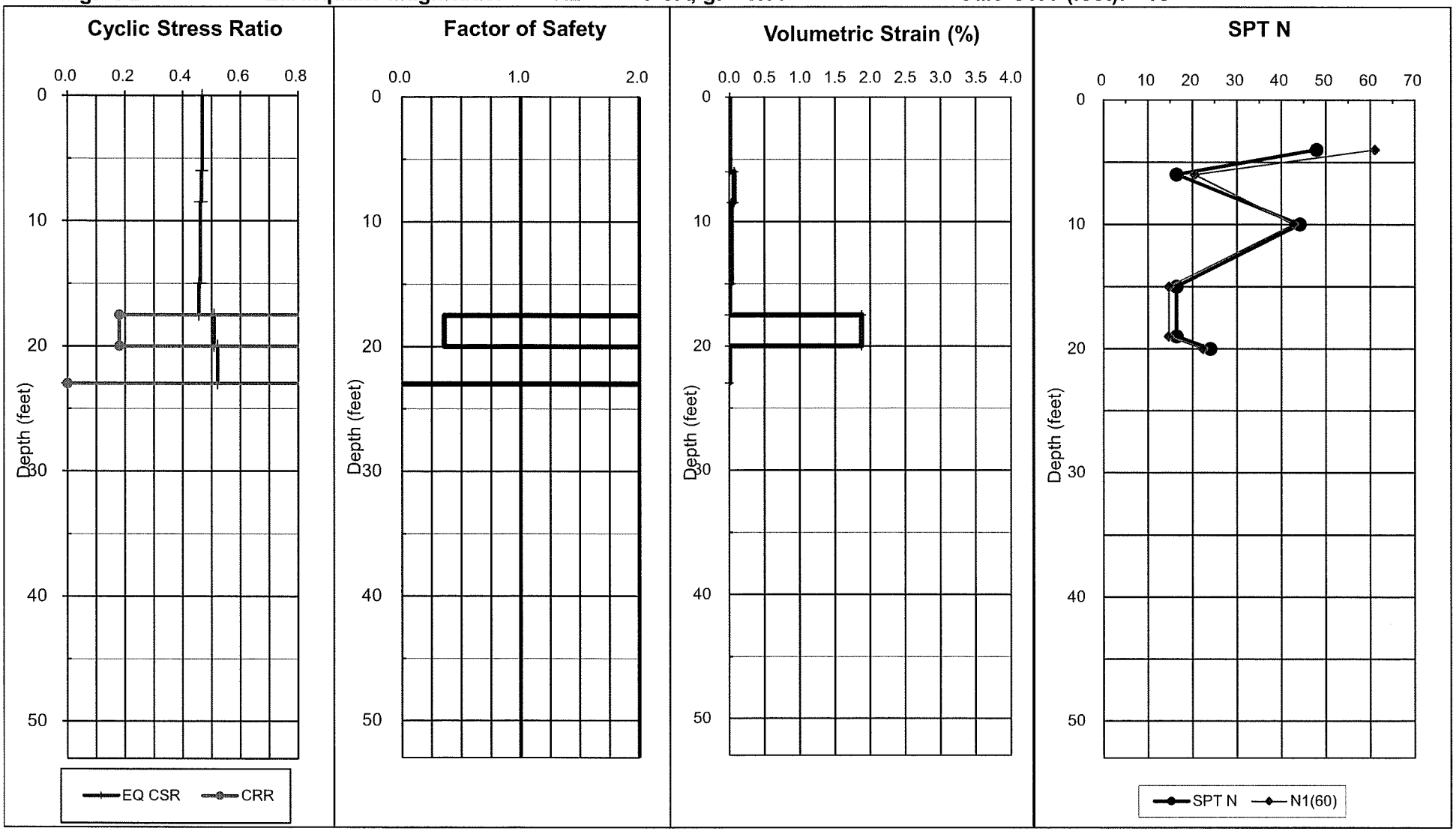
Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	$K\sigma$	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
								0.000																	
6.0	76	**48**	1	127	50	4.0	7.0	0.254	0.254	0.99	1.70	0.75	1.00	61.0	93	10.0	71.0	1.00	1.400	0.467	**Non-Liq.**	10.0	71.0	0.01	**0.01**
8.5	26	**16**	1	132	25	6.0	9.0	0.381	0.381	0.99	1.67	0.75	1.00	20.5	54	6.6	27.1	1.00	0.322	0.465	**Non-Liq.**	6.6	27.1	0.07	**0.02**
15.0	70	44	1	134	12	10.0	13.0	0.647	0.647	0.98	1.28	0.76	1.00	42.7	78	2.9	45.6	1.00	1.400	0.461	**Non-Liq.**	2.9	45.6	0.03	**0.02**
17.5	26	16	1	134	12	15.0	18.0	0.982	0.982	0.97	1.04	0.86	1.00	14.7	46	2.0	16.7	1.00	0.180	0.456	**Non-Liq.**	2.0	16.7	0.00	**0.00**
20.0	26	16	1	134	12	19.0	22.0	1.250	1.125	0.96	0.97	0.92	1.00	14.6	46	2.0	16.7	0.99	0.180	0.508	**0.35**	1.1	15.8	1.88	**0.56**
23.0	38	24		134	12	20.0	23.0	1.317	1.161	0.96	1.00	0.93	1.00	22.3				0.98	Infin.	0.521	**Non-Liq.**		22.3	0.00	**0.00**

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Matilija Jr. HS Locker Room & Kitchen Remodels **Project No: 302294-001** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B2 **Earthquake Magnitude: 7.2** **PGA, g: 0.80** **Calc GWT (feet): 15**



Total Thickness of Liquefiable Layers: 2.5 feet **Estimated Total Ground Subsidence: 0.6 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: Matilija Jr. HS Locker Room & Kitchen Remod
Job No: 302294-001
Date: 7/10/2018
Boring: B2 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:	**7.2**	7.5
PGA, g:	**0.80**	0.72
MSF:	1.11	
GWT:	**23.0**	feet
Calc GWT:	**23.0**	feet
Remediate to:	**5.0**	**feet**

SPT N VALUE CORRECTIONS:

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

Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
0	0.2

Required SF: 1.30
Threshold Acceler., g: #N/A **Minimum Calculated SF:** #N/A

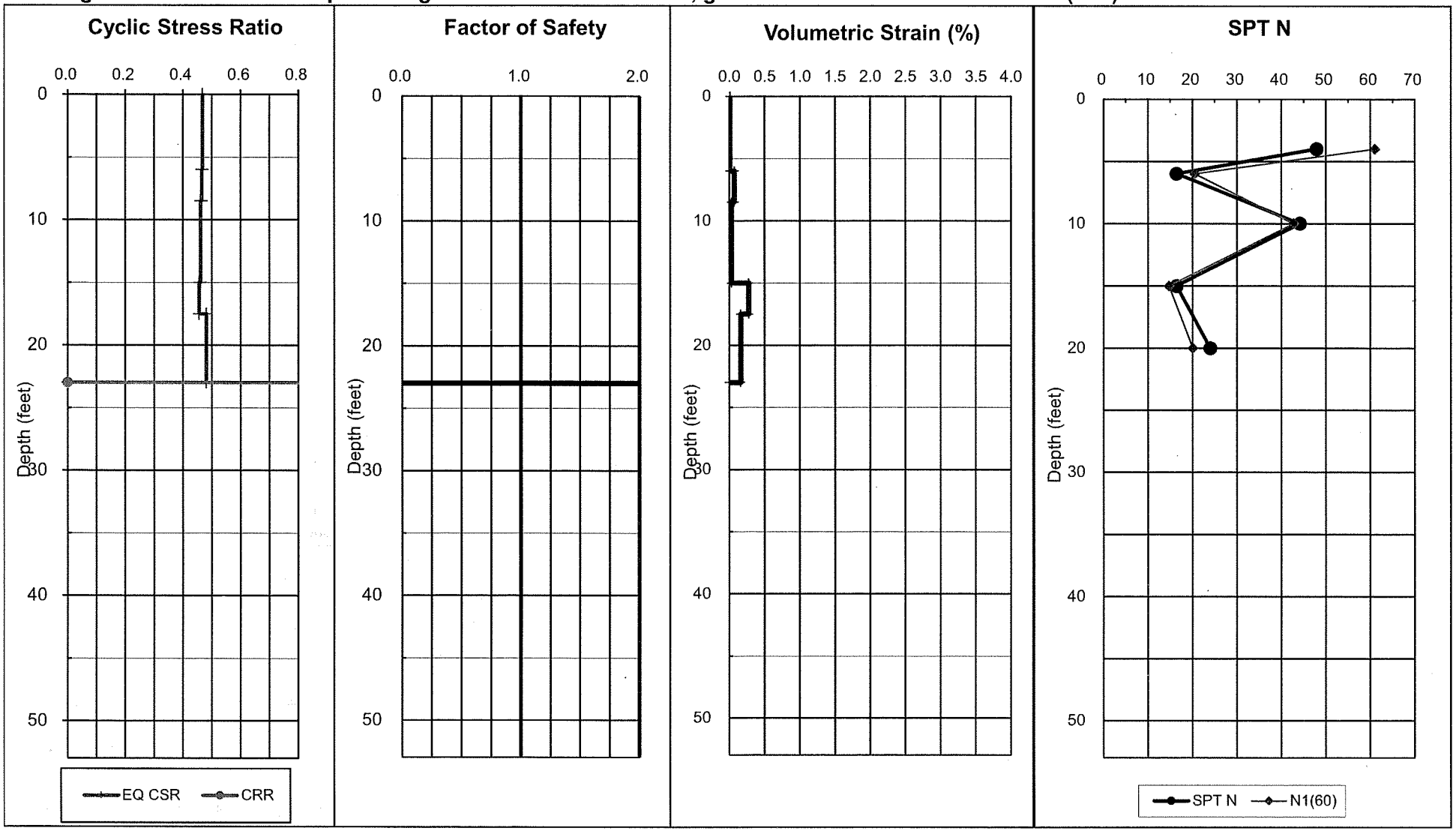
Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	$K\sigma$	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
								0.000																	
6.0	76	**48**	1	127	50	4.0	7.0	0.254	0.254	0.99	1.70	0.75	1.00	61.0	93	10.0	71.0	1.00	1.400	0.467	**Non-Liq.**	10.0	71.0	0.01	**0.01**
8.5	26	**16**	1	132	25	6.0	9.0	0.381	0.381	0.99	1.67	0.75	1.00	20.5	54	6.6	27.1	1.00	0.322	0.465	**Non-Liq.**	6.6	27.1	0.07	**0.02**
15.0	70	44	1	134	12	10.0	13.0	0.647	0.647	0.98	1.28	0.76	1.00	42.7	78	2.9	45.6	1.00	1.400	0.461	**Non-Liq.**	2.9	45.6	0.03	**0.02**
17.5	26	16	1	134	12	15.0	18.0	0.982	0.982	0.97	1.04	0.86	1.00	14.7	46	2.0	16.7	1.00	0.180	0.456	**Non-Liq.**	2.0	16.7	0.27	**0.08**
23.0	38	24	1	134	12	20.0	23.0	1.317	1.317	0.96	0.90	0.93	1.00	20.0	53	2.2	22.2	0.94	0.243	0.481	**Non-Liq.**	2.2	22.2	0.16	**0.11**

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Matilija Jr. HS Locker Room & Kitchen Remodels **Project No: 302294-001** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B2 **Earthquake Magnitude: 7.2** **PGA, g: 0.80** **Calc GWT (feet): 23**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 0.2 inches**

State of California • Natural Resources Agency
Department of Conservation
California Geological Survey
801 K Street • MS 12-31
Sacramento, CA 95814
(916) 324-7324 • FAX (916) 445-3334

Edmund G. Brown Jr., *Governor*
John G. Parrish, Ph.D., *State Geologist*

Mr. David Rogers
District Director of Facilities
Ojai Unified School District
414 E Ojai Avenue
Ojai, CA 93023

October 2, 2018

Soil Report
A #03-118467

Subject: **Engineering Geology and Seismology Review for Matilija Junior High School –Locker Room Remodel 703 El Paseo Road, Ojai, CA CGS Application No. 03-CGS3561 DSA Application No. 03-118467**

Dear Mr. Rogers:

In accordance with your request and transmittal of documents received on August 7, 2018, the California Geological Survey has reviewed the engineering geology and seismology aspects of the consulting report prepared for Matilija Junior High School in Ojai. It is our understanding that this project involves Gym reroofing and a Locker Room remodel. This review was performed in accordance with Title 24, California Code of Regulations, 2016 California Building Code (CBC) and followed CGS Note 48 guidelines. We reviewed the following report:

> **Engineering Geology and Geotechnical Engineering Report for Proposed Remodeling of Locker Room and Kitchen at Matilija Junior High School, 703 El Paseo Road, Ojai, California:** Earth Systems Pacific, 1731-A Walter Street, Ventura, California 93003; company Project No. 302294-001, report dated July 12, 2018, 20 pages, 4 appendices.

Based on our review, the consultants provide a reasonable and mostly well-documented assessment of engineering geology and seismology issues with respect to the proposed improvements. The principal concerns identified by the consultants are the potential for strong ground shaking and moderately corrosive soils to ferrous metals. The consultants recommend site-specific design spectral acceleration parameters of S_{DS} = **1.238*g*** and S_{D1} = **1.090*g***, which are considered reasonable. Their evaluation indicates liquefaction, surface fault rupture, dynamic settlement, and deep-seated slope instability are not design concerns for the project.

In conclusion, ***the engineering geology and seismology issues at this site are adequately assessed in the referenced report.*** If you have any further questions about this review letter, please contact the reviewer at Jacqueline.Bott@conservation.ca.gov.

Respectfully submitted,

Jacqueline Bott
Engineering Geologist
PG 7459, CEG 2382

CERTIFIED ENGINEERING GEOLOGIST
Jacqueline Bott
No. 2382
STATE OF CALIFORNIA

Concur:

Jennifer Thornburg
Senior Engineering Geologist
PG 5476, CEG 2240

CERTIFIED ENGINEERING GEOLOGIST
Jennifer Thornburg
No. 2240
STATE OF CALIFORNIA

Enclosures:

Note 48 Checklist Review Comments
Keyed to: *Note 48 - Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings*

Copies to:

Patrick V. Boales, *Certified Engineering Geologist*, and Anthony P. Mazzei, *Registered Geotechnical Engineer*
Earth Systems Pacific, 1731Walter Street, Suite A, Ventura, CA 93003

Tyson Cline, *Architect*
RNT Architects, 285 N Ventura Avenue, Suite 102, Ventura, CA 93001

Ted Beckwith, *Senior Structural Engineer*
Division of State Architect, 700 North Alameda Street, Suite 5-500, Los Angeles, CA 90012

Note 48 Checklist Review Comments

In the numbered paragraphs below, this review is keyed to the paragraph numbers of California Geological Survey Note 48 (October, 2013 edition), *Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings.*

Project Location

1. Site Location Map, Street Address, County Name: Adequately addressed.
2. Plot Plan with Exploration Data with Building Footprint: Adequately addressed.
3. Site Coordinates: Adequately addressed. Latitude and Longitude provided in report: 34.4453°N, 119.2552°W

Engineering Geology/Site Characterization

4. Regional Geology and Regional Fault Maps: Adequately addressed.
5. Geologic Map of Site: Adequately addressed.
6. Subsurface Geology: Adequately addressed. The consultants report the site is underlain by Late Pleistocene Older Alluvium described as coarse clastic fan deposits, of age between 25,000 and 30,000 years according to mapping by Rockwell (1984). The consultants report finding perched groundwater at a depth of 15 feet below ground surface.
7. Geologic Cross Sections: Adequately addressed.
8. Active Faulting & Coseismic Deformation Across Site: Adequately addressed. The consultants report the site is not located within any of the Fault Rupture Hazard Zones that have been delineated by CDMG, and no faults were observed to be located on or trending into the subject property during reviews of the literature and aerial photographs.
9. Geologic Hazard Zones (Liquefaction & Landslides): Not addressed by consultants, and therefore not reviewed.
10. Geotechnical Testing of Representative Samples: Adequately addressed.
11. Geological Consideration of Grading Plans and Foundation Plans: Adequately addressed.

Seismology & Calculation of Earthquake Ground Motion

12. Evaluation of Historic Seismicity: Adequately addressed.
13. Classify the Geologic Subgrade (Site Class): Adequately addressed. The consultants classify the site soil profile as Site Class D, Stiff Soil, based on average SPT N-value in the upper 23 feet, and assuming the N-value is the same or greater with depth below the depth of exploration. The data presented appears to support this conclusion.
14. General Procedure Seismic Parameters: Adequately addressed. The consultants report the following parameters derived from a map-based analysis:
    $S_S = 2.219$ and $S_1 = 0.828$
    $S_{DS} = 1.479$ and $S_{D1} = 0.828$

15. Seismic Design Category: The consultants report Seismic Design Category of E for the site as S_1>0.75.
16. Site-Specific Ground Motion Analysis: Adequately addressed. The consultants' deterministic MCE spectrum is significantly lower than CGS would expect at periods 0.3 seconds and greater. It is not clear from the consultants' report how the deterministic analysis was performed, but CGS reminds them that the full, unsegmented, rupture length should be considered in a deterministic analysis. However, the consultants' results are controlled by **the probabilistic MCE spectrum, which appears reasonable** based on comparison with results from the State-Wide Model (from Petersen and others, 2008). Therefore, their analysis is acceptable. The consultants report their site-specific seismic design parameters are; $\mathbf{S_{DS} = 1.238}\boldsymbol{g}$ and $\mathbf{S_{D1} = 1.090}\boldsymbol{g}$. Alternatively, S_a values presented in the 9th column of Table labelled "Spectra Response Values" in Appendix C may be used with the equivalent lateral force procedure, per ASCE 7, Section 21.4. The site-specific ground motion analysis presented appears to be reasonable and in accordance with ASCE 7-10.
17. Deaggregated Seismic Source Parameters: Not applicable.
18. Time-Histories of Earthquake Ground Motion: Not applicable.

Liquefaction/Seismic Settlement Analysis

19. Geologic Setting for Occurrence of Seismically Induced Liquefaction: Adequately addressed. The consultants report perched water at 15 feet depth, but note the deposits below this were moist and not saturated. The consultants report regional historical high groundwater is about 50 feet below the ground surface as shown in the groundwater map for the Matilija Quadrangle (CGS, 2003). The consultants report their penetration tests indicate the site soils are in a relatively dense state, even when using the lowest of the blow counts for any 6-inch increment and ignoring higher blow counts because of potential gravel influence. However, the consultants performed liquefaction analyses and report that the potential for liquefaction exists at the site due to the perched water.
20. Seismic Settlement Calculations: Adequately addressed. The consultants report a soil layer between 17.5 and 20 feet had a factor of safety of less than 1.3 and so is considered potentially liquefiable. The consultants computed total liquefaction settlement of 0.6 inches for this layer and estimate differential settlement to be half this value, or 0.3 inches. The consultants report 0.2 inches of potential dry seismic settlement. The data presented appear to support this conclusion.
21. Other Liquefaction Effects: Adequately addressed. The consultants report "free-face" lateral spreading does not appear to pose a potential hazard as there are no nearby sloped areas or canyons.
22. Mitigation Options for Liquefaction: Not applicable.

Slope Stability Analysis

23. Geologic Setting for Occurrence of Landslides: Adequately addressed. The consultants report landsliding and rock fall do not pose a hazard to this project based on the relief across the site of only a few feet, and no landslides were observed to be located on or trending into the subject site.

24. Determination of Static and Dynamic Strength Parameters: Not applicable.
25. Determination of Pseudo-Static Coefficient (Keq): Not applicable.
26. Identify Critical Slip Surfaces for Static and Dynamic Analyses: Not applicable.
27. Dynamic Site Conditions: Not applicable.
28. Mitigation Options/Other Slope Failure: Not applicable.

Other Geologic Hazards or Adverse Site Conditions

29. Expansive Soils: Adequately addressed. The consultants report the bearing soils lie in the "very low" expansion range with an expansion index of 1.
30. Corrosive/Reactive Geochemistry of the Geologic Subgrade: Adequately addressed. The consultants report the site soils are "moderately corrosive" to ferrous metal pipes based on the resistivity measurements.
31. Conditional Geologic Assessment: Adequately addressed. No significant conditional hazards of potential concern were identified by the consultants.

Report Documentation

32. Geology, Seismology, and Geotechnical References: Adequately addressed.
33. Certified Engineering Geologist: Adequately addressed.
    Patrick V. Boales, Certified Engineering Geologist #1346
34. Registered Geotechnical Engineer: Adequately addressed.
    Anthony P. Mazzei, Registered Geotechnical Engineer #2823