SOILS ENGINEERING AND GEOLOGIC HAZARDS REPORT
MANTECA COURTHOUSE ADDITION
EAST CENTER STREET
MANTECA, CALIFORNIA

May 3, 2011

Prepared for

Tetra Design, Inc.

Prepared by

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May 3, 2011

FILE NO.: SL-16437-SA

Mr. Jerrold R. Penrose, AIA
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PROJECT: MANTECA COURTHOUSE ADDITION
EAST CENTER STREET
MANTECA, CALIFORNIA

SUBJECT: Soils Engineering and Geologic Hazards Report

REF: Proposal to Provide a Soils Engineering and Geologic Hazards Report, San Joaquin County Courts, Manteca Courthouse Addition, East Center Street, Manteca, California, by Earth Systems Pacific, dated November 22, 2010, Doc. No. 1011-075.PRP

Dear Mr. Penrose:

In accordance with your authorization of the referenced proposal, this soils engineering and geologic hazards report has been prepared for use in the development of plans and specifications for the proposed additions to, and remodel of, the Manteca Courthouse in Manteca, California. Preliminary geotechnical recommendations for site preparation, grading, utility trenches, foundations, interior slabs-on-grade and exterior flatwork, retaining walls, pavement sections, drainage and maintenance, and observation and testing, as well as geologic recommendations, are presented herein. Per your request, copies of the report have been distributed as shown below.

We appreciate the opportunity to have provided services for this project and look forward to working with you again in the future. If there are any questions concerning this report, please do not hesitate to contact the undersigned.

Sincerely,

Earth Systems Pacific

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Doc. No.: 1105-007.SER/rw

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1.0 INTRODUCTION

The remodel of the Manteca Courthouse and the addition of approximately 7,500 square feet are planned in two phases. The first phase will consist of some interior remodeling, adding holding cells to the northeast corner, and access improvements to the main entrance on the south side of the existing building. A portion of two existing waterlines that currently crosses the site will also be removed as part of Phase 1. Phase 2 will include a major addition to the north side of the building and a corridor along the west side. With the exception of the holding cells, which are expected to be masonry, stud construction will be used. We understand that line loads for the stud portions of the structure will not exceed 1,000 plf dead load and 1,000 plf live loads; line loads for the masonry portions of the structure, i.e. the holding cells, will not exceed 2,500 plf dead load and 400 plf live loads. Maximum concentrated loads will not exceed 18 kips dead load, 15 kips live load, and 5 kips seismic load.

Where the major addition to the north side of the building is planned, grade will need to be raised approximately 2 feet to allow the finish floor elevation of the addition to match that of the existing building. This may be accomplished by placing fill across most of the site and retaining it with a perimeter wall, by extending and backfilling the stemwall footings of the addition, or a combination of both approaches.

The area to the north of the courthouse addition will be paved with asphalt concrete and used for secure parking. A drive aisle will extend down the east side of the building between the building and an alley, with egress to Center Street. A new asphalt concrete parking lot will be constructed on the triangular piece of the property at the southeast corner of the site.

Two waterlines currently cross the addition part of the site, trending from the northwest corner to the southeast corner. Where the pipes enter the northwest corner of the property, they are believed to consist of two 36-inch diameter reinforced concrete pipes. The flowlines of these pipes are believed to lie about 6 feet below existing grade. Where the pipes cross the end of an alley in the east region of the site, there is a manhole and the pipes are believed to transition to a single 48-inch diameter pipe. This pipe continues in a southeasterly direction, exiting the new parking lot area of the site to the southeast. As part of the project, the 36-inch pipes will be removed; the 48-inch pipe will remain in place.

2.0 SCOPE OF SERVICES

The scope of work for this soils engineering and geologic hazards report included a general site reconnaissance by a Registered Geotechnical Engineer and a Certified Engineering Geologist, field exploration, laboratory testing, geotechnical and geologic analysis of data, and the preparation of this report. The analysis and subsequent recommendations were based in part upon project plans and information provided by the client.

This report and recommendations are intended to comply with applicable requirements of Sections 1803A.1 through 1803A.5.4, 1803A.5.7, 1803A.5.8, 1803A.5.11 through 1803A.7, and J104.3 of the 2010 California Building Code (CBC), and common geotechnical engineering and engineering geology practice in this area under similar conditions at this time.

Preliminary geotechnical engineering recommendations for site preparation, grading, utility trenches, foundations, interior slabs-on-grade and exterior flatwork, retaining walls, pavement sections, drainage and maintenance, and observation and testing are presented herein. As there may be geotechnical issues yet to be resolved, the soils engineer and engineering geologist should be retained to provide consultation as the design progresses, and to review project plans as they near completion to assist in verifying that pertinent geotechnical and geologic issues have been addressed and to aid in conformance with the intent of this report.

It is our intent that this report be used exclusively by the client to form the geologic and geotechnical basis of the design of the identified project and in the preparation of plans and specifications. Application beyond this intent is strictly at the user's risk. If future property owners wish to use this report, such use will be allowed to the extent the report is applicable, only if the user agrees to be bound by the same contractual conditions as the original client, or contractual conditions that may be applicable at the time of the report's use.

This report does not address issues in the domain of contractors such as, but not limited to, site safety, loss of volume due to stripping of the site, shrinkage of soils during compaction, excavatability, temporary slope angles, construction means and methods, etc. Analyses of lead or mold potential, radioisotopes, hydrocarbons, or other chemical properties (with the exception of geotechnical corrosivity) are beyond the scope of this report. Evaluation of ancillary features such as temporary access roads, fences, light and flag poles, signage, and nonstructural fills are all not within our scope and are also not addressed.

In the event that there are any changes in the nature, design, or location of improvements, or if any assumptions used in the preparation of this report prove to be incorrect, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report verified or modified in writing by the soils engineer and/or engineering geologist. The geotechnical criteria presented in this report are considered preliminary until such time as any peer review or review by any jurisdiction has been completed, conditions are observed by the soils engineer and/or engineering geologist in the field during construction, and the recommendations have been verified as appropriate, or modified in writing by the soils engineer and/or engineering geologist.

3.0 SITE SETTING

The subject site is located on the northeast corner of Center Street and Lincoln Avenue in Manteca, California. The surrounding district consists of residential and commercial properties. Currently, the site is occupied by the existing courthouse, which is surrounded with asphalt concrete pavement that supports parking areas and drive aisles. The elevation of the existing courthouse and the area surrounding it is a couple of feet higher than that of the rest of the site; retaining walls are incorporated into the building but transition to site walls along the south and west sides of the site. There are several office trailers, fences, and other ancillary improvements on the site. Existing waterlines crosses the site as described previously.

4.0 INVESTIGATION METHODOLOGY

Site Reconnaissance and Literature Review

Site reconnaissance was performed by a Registered Geotechnical Engineer and a Certified Engineering Geologist to observe the geotechnical and geologic conditions visible on and adjacent to the site. Pertinent geologic literature and maps were reviewed to assess the underlying geologic units, faulting, and potential geologic hazards that might affect the proposed project.

Subsurface Exploration

On February 23, 2011, five exploratory borings were drilled in the proposed improvement areas with a truck-mounted Mobile Drill, Model B-53 drill rig, equipped with an 8-inch outside diameter hollow stem auger and a free-fall safety hammer for sampling. The borings were extended to depths of 15 to 50 feet below the existing ground surface. As they were drilled, soil samples were taken using a ring-lined barrel sampler (ASTM D 3550-01/07, with shoe similar to D 2937-04), and Standard Penetration Tests (ASTM D 1586-08a) were conducted at selected depths. According to calibration data provided by the driller, his sampling hammer operates at about 40 percent efficiency. The approximate locations of the borings are shown on the Boring Location Map in Appendix A.

Soils encountered in the borings were categorized and logged in general accordance with the Unified Soil Classification System and ASTM D 2488-09a. Copies of the boring logs, along with a Boring Log Legend, can also be found in Appendix A. In reviewing the boring logs and legend, the reader should realize that the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics as observed during drilling. These include, but are not limited to, the presence of cobbles or boulders, cementation, variations in soil moisture, presence of groundwater, and other factors. Consequently, the logger must exercise judgment in interpreting soil characteristic, possibly resulting in soil descriptions that vary somewhat from the legend.

5.0 LABORATORY ANALYSIS

The ring samples were tested for unit weight and moisture (ASTM D 2937-04, as modified for ring liners). Soil classifications were based in part upon sieve analysis (ASTM D 422-63/07, D 1140-06) and plasticity index (ASTM D 4318-05). Selected bulk samples were tested for maximum density and optimum moisture (ASTM D 1557-09), expansion index (ASTM D 4829-08a), and R-value (ASTM D 2844-07). An unconfined compressive strength test (ASTM D 2166-06) was performed on a ring sample. Two ring samples were tested for one-dimensional consolidation (ASTM D 2435-04). A direct shear test (ASTM D 3080-04) was performed on a bulk sample that was remolded to 90 percent of maximum dry density. The physical laboratory test results can be found in Appendix B.

A soil sample was submitted to Schiff Associates for geotechnical corrosivity testing. The corrosivity test results are presented in Appendix C for use by others in determining corrosion protection needs.

6.0 GENERAL SUBSURFACE PROFILE

Beneath a surficial cover of asphalt concrete and aggregate base, the upper soils at the site consisted of light brown silty sand alluvium. The silty sand was in a loose to medium dense condition and, in some borings, contained a zone of cementation at a depth of about 4 feet. The silty sand was underlain by light yellow brown to yellow gray poorly graded sand. The poorly graded sand was typically medium dense, although dense conditions were found in Boring 4. In Boring 3, which was drilled to a depth of 50 feet, the poorly graded sand transitioned to lean clay at a depth of 14.5 feet. The lean clay was gray and contained thin zones of silty sand. Conditions varied from medium stiff to hard. At a depth of 43 feet, yellow brown clayey sand was encountered. The clayey sand was dense and contained thin zones of sandy lean clay.

At the time of drilling, the soils ranged from slightly moist to moist; free subsurface water was observed in Boring 3 perched between depths of 22 and 29.5 feet.

7.0 GEOLOGY

Regional Geologic Setting

The site lies in the northern portion of the San Joaquin Valley. The filling of a large structural trough or downwarp in the underlying bedrock formed the Great Valley province of California. The trough is situated between the Sierra Nevada on the east and the Coast Range on the west. Both of these mountain ranges were initially formed by uplifts, which occurred during the Jurassic and Cretaceous periods of geologic time (greater than 65 million years ago). Renewed uplift began in the Sierra Nevada during late Tertiary time, and is continuing today. The trough, which underlies the Valley, is asymmetrical with the greatest depth of sediments near the western margin. The sediments, which fill the trough, originated as erosional material from the adjacent mountains and foothills. The upper and youngest sediments in the basin are continental deposits consisting of alluvial fan deposits and flood-basin, lake, and marsh deposits.

Local Geologic Setting

The geologic map of the San Francisco-San Jose Quadrangle by Wagner and others (1991) indicates that the site lies along the contact between Holocene dune sand deposits and Pleistocene alluvial fan deposits of the Modesto Formation (see Geologic Map in Appendix D). However, the results of our subsurface investigation suggest that the site is underlain by alluvial fan deposits of the Modesto Formation. This conclusion is due to the presence of the silt in the surficial sandy layer and the medium to coarse grained consistency of this soil.

Faulting

The project site and its vicinity are located in an area traditionally characterized by low to moderate seismic activity. The site is not located in an Alquist-Priolo Earthquake Fault Zone as established by the Alquist-Priolo Fault Zoning Act. The California Geological Survey Fault Activity Map of California (2010) was reviewed to determine if identified active faults are located on or near the subject site. According to the map, no identified active faults are located on or near the site. Locations of the active and Late Quaternary faults in the area of the site are shown on the Regional Fault Map in the Appendix D. Segment 7 of the Great Valley Fault, the Greenville Fault, and the Marsh Creek Fault are considered, at this time, to be the most significant regional active faults that could affect the site. The buried Vernalis fault is the closest mapped fault to the site, located approximately 12 miles to the west, though it is not considered to be active.

The Great Valley Fault is considered to be the closest active fault to the site, and is believed to be located approximately 15 miles west of the site though its exact location is unknown. This fault is a blind thrust fault and Segment 7 of this fault may be part of the buried San Joaquin Fault, located approximately 25 miles southwest of the site. As the location of the Great Valley Fault is not known, it is not shown on the Regional Fault Map.

Groundwater

Groundwater was encountered in Boring 3 at a depth 22 feet. The elevation of the groundwater in the Manteca area has lowered in the last 30 years, although fluctuations of water level have stabilized to some degree over the last 10 years, (DWR, July 31, 2005). This overall trend in water levels has been recorded throughout the county and results from over-pumping of the groundwater basin (San Joaquin County, September 2004). Water levels have declined significantly during drought cycles (e.g., 1987 to 1992) and recovered during times of increased precipitation (e.g., 1978 to 1986).

Slope Stability

The site consists of flat terrain with no significant natural slopes on or immediately adjacent to it; therefore, instability of slopes is not of concern.

Flooding

According to the Flood Insurance Rate Map, Map Number 06077C0640F, dated October 16, 2009, published by the Federal Emergency Management Agency (FEMA), the site is located in

Flood Zone X. Areas designated as Flood Zone X have a 0.20 percent chance of being flooded in any given year. The average depth of flooding is less than 1 foot.

8.0 SEISMICITY

Earthquake History

The historic seismicity in the site region was researched using EQSEARCH (Blake, updated 2010) and the Boore and others (1997) ground attenuation method for a CBC Site Class "D," a stiff soil profile. EQSEARCH is a computer program that performs automated searches of a custom catalog of historical California earthquakes. As the program searches the catalog, it computes and prints the epicentral distance from the selected site to each of the earthquakes within the specified search area. The epicentral distances should be considered estimated distances, particularly for earthquake data information that dates prior to 1932, before instruments were used to record earthquake data. The parameters used for the search consisted of earthquake Richter magnitudes ranging from 5.0 to 9.0 that occurred in a 100-mile radius from the site from 1800 to 2010.

Results of the search indicated that within the search parameters, 116 earthquakes have occurred. The highest peak horizontal ground acceleration (PGA) estimated to have occurred at the site from those historical earthquakes is 0.11g. This earthquake had a 7.8 magnitude, occurred in 1906, and was located approximately 70 miles southwest of the site. This earthquake occurred on the San Andreas Fault and is known as the "Great San Francisco Earthquake." The closest earthquake to the site was a 5.8 magnitude earthquake that occurred approximately 21 miles south of the site. It is estimated to have produced a 0.08g PGA at the site.

Site Specific Earthquake Ground Motion Analysis

General

The site is in a region of generally low to moderate seismicity and has the potential to experience ground shaking from earthquakes on regional or local causative faults. Based on a depth to groundwater of 22 feet in Boring 3, and the presence of sandy soils underlying the site, in our opinion, there is a sufficient potential for liquefaction to occur that analysis of liquefaction is justified. Our analysis is intended to be in conformance with sections 1615A.1.2 and 1803.6.2 of the 2010 CBC, and ASCE Standard 7-05, Chapter 21, "Site Specific Ground Motion Procedures for Seismic Design," Sections 21.2.1 through 21.2.4. That chapter requires that the Probabilistic Maximum Considered Earthquake (MCE) response curve, the Site Specific Deterministic MCE response curve, the Site Design response curve (2/3 Site Specific Deterministic MCE), and the Design Acceleration Parameters be developed for the site. The MCE is defined as having a 2 percent chance of exceedance in 50 years, with a return period of approximately 2,475 years.

The deterministic and probabilistic ground motions were calculated using the next generation attenuation (NGA) curves that were obtained from the computer software program EZFRISK (ver. 7.51) by Risk Engineering, Inc. The NGA curves used for this ground motion analysis were: Campell & Bozorgnia (2008) NGA USGS 2008; Boore & Atkinson (2008) NGA USGS 2008; Chiou & Young (2007) NGA USGS 2008. The average spectral curve of these three NGAs was used as the Probabilistic MCE Response Spectrum and Deterministic MCE Response Spectrum. Using the EZFRISK software, we included a maximum rotated component of ground motion in the analysis. The method used for the rotated component was per Huang, Whittaker, and Luco, (2008).

Probabilistic MCE

To develop the probabilistic MCE Response Spectrum for the site, the EZFRISK computer program was again used. Seventy seismic sources from the 2008 USGS database within a 65-mile radius of the site were selected using the EZFRISK program. The PGA and spectral accelerations, at 5 percent damped for a CBC soil Site Class "D," were estimated using the NGAs mentioned above. The soil class was derived from the subsurface investigation performed at the site. The average of these three curves was used as the Probabilistic MCE Response Spectrum (see the Average NGA Probabilistic MCE Response Spectrum in Appendix F).

Deterministic MCE

A deterministic analysis using the NGAs was performed in accordance with DSA Bulletin 09-01 (2009) which generally requires that the 84TH percentile of the deterministic ground motion be used in lieu of using 150 percent of the median value. Values for the three spectral NGA curves were computed from the EZFRISK computer program. The results of the deterministic analysis indicated that Segment 7 of the Great Valley Fault, 25.19 km from the site, yielded the highest ground motion of all the seismic sources. The value was a (Mw) magnitude of 6.9. The values from the above three attenuation models were then averaged and used as the basis for calculating the Average 84th Percentile Deterministic Response Spectrum (see the Average NGA 84TH Percentile Deterministic Response Spectrum in Appendix F).

The ordinates of the 84TH percentile Deterministic MCE Response Spectrum are lower than the corresponding ordinates of the response spectrum (Deterministic MCE Lower Limit) calculated in accordance with Fig. 21.2.1 in ASCE Standard 7-05 (see the Site Specific Design Response Spectrum in Appendix F). The Deterministic MCE Lower Limit was calculated using an F_a value of 1.14, an F_v value of 1.78. It is also plotted on the Site Specific Design Response Spectra graph in Appendix F. As the 84TH percentile Deterministic MCE Response Spectrum is lower than the Deterministic MCE Lower Limit, it is our opinion that the Deterministic MCE Lower Limit should be used as the Site Deterministic MCE curve.

Site Specific MCE

The Site Specific MCE is defined by ASCE 7-05, Section 21.2.3 as the lesser of the Probabilistic MCE and the Site Deterministic MCE (Deterministic MCE Lower Limit). Review of the plots on the Site Specific Design Response Spectra graph indicates that the Probabilistic MCE is the lesser of the two curves; therefore, it should be used as the Site Specific MCE.

Site Specific Design Response Spectrum

Per ASCE 7-05 Section 21.3, the Site Design Response Spectra are obtained by taking 2/3 of the Site Specific MCE (Probabilistic MCE); this information is also plotted on the Site Specific Design Response Spectra. The Site Design Response Spectra accelerations for 2/3 of the Site Specific MCE were higher than the accelerations from the 80 percent of the general procedure Design Response Curve as shown on the Site Specific Design Response Spectra graph. Therefore, it is recommended that the Site Specific MCE should be used for design purposes.

Design Acceleration Parameters

The following design acceleration parameters are based on two separate analytical techniques; 1) the USGS Java Motion Parameter Calculator (USGS, 2010) using the 2005 ASCE 7 Standard setting (see USGS Java Motion Parameter Calculator Values in the following tables) and 2) the procedures described above that were used to determine the site specific design ground motion values. Parameters from both of these methods are shown for comparison; however; the use of the appropriate design acceleration parameters is left to the discretion of the architect/engineer. Note that ASCE 7-05-21.4 requires that the parameter S_{D1} shall be taken as the greater of the spectral acceleration, S_a, at a period of 1 second (0.384g), or two times the spectral acceleration, S_a, at a period of 2 seconds (0.226g). In this case, the value at two times the spectral acceleration, S_a, at a period of 2 seconds was higher (0.452g).

Mapped Acceleration Values for Site Class B		**2010 CBC Site Coefficients and General Procedure Adjusted MCE Spectral Response Acceleration Parameters For Site Class D (PGA = $S_{DS}/2.5 = 0.690/2.5 = 0.276g$)**					
Seismic Parameters	Values (g)	Site Coefficients	Values	Seismic Parameters	Values (g)	Seismic Parameters	Values (g)
S_S	0.911	F_a	1.14	S_{MS}	1.04	S_{DS}	0.690
S_1	0.313	F_v	1.78	S_{M1}	0.555	S_{D1}	0.370

80% of the General Procedure Design Response Spectrum Acceleration Values for Site Class D			
Seismic Parameters	Values (g)	Seismic Parameters	Values (g)
S_{MS}	0.828	S_{DS}	0.552
S_{M1}	0.444	S_{D1}	0.296

Site Specific Design Response Spectrum Acceleration Values for Site Class D (PGA = 0.268g)			
Seismic Parameters	Values (g)	Seismic Parameters	Values (g)
S_{MS}	0.916	S_{DS}	0.611
S_{M1}	0.678	S_{D1}	0.452

Design Earthquake Level

California Geologic Survey, Note 48 indicates the design peak ground acceleration for evaluation of liquefaction may be based on a site specific study or taken by $S_{DS}/2.5$, where S_{DS} is defined in Section 1613A.5.4 of the 2010 CBC as the Design Spectral Response Acceleration Parameter. Because the site is located within a potential liquefaction hazard area, a site specific design peak ground acceleration of 0.268g is recommended.

A probabilistic seismic hazards analysis (PSHA) procedure was used to estimate the relative contribution for different magnitude-distance combinations for the earthquake (model) magnitude associated with the site specific design peak ground acceleration of 0.268g. This magnitude is necessary to assess the potential for liquefaction at the site. The computer program EZFRISK Version 7.51 (Risk Engineering, Inc, 2009) was utilized, which allows the user to input the project site coordinates and peak ground motion amplitude (site specific design peak ground acceleration) for the site.

The analysis was deaggregated to obtain the model magnitude associated with the site specific design peak ground acceleration of 0.268g. The results of the analysis indicated a model magnitude of 6.55.

Seismic Design Category

Section 1613A.5.6 of the 2010 CBC indicates that structures shall be assigned to Category D unless $S_1 \geq 0.75$. The S_1 calculated for the site is 0.313g; therefore, the site would be a Category D.

9.0 CONCLUSIONS

In our opinion, the site is suitable, from a geotechnical engineering and geologic standpoint, for the proposed courthouse addition.

With respect to the geologic conditions at the site, it is our opinion that there are no significant local geologic conditions that would preclude development of the site as described in the "Introduction" section of this report. The site is underlain by alluvial fan deposits of the Modesto Formation; groundwater was encountered in one boring at a depth of 22 feet. Due to the site's level nature, there is no potential for landsliding to impact the site.

The project site and its vicinity are located in an area traditionally characterized by low to moderate seismic activity. Earthquakes have occurred very infrequently in this area during historic time (since 1800). The largest historical mean peak horizontal acceleration estimated to have occurred on the site within the last 211 years was 0.11g. The site is not located in any State Earthquake Fault Zones and there are no mapped faults crossing the site. Segment 7 of the Great Valley Fault is considered to be the closet active fault to the site, located approximately 15 miles west of the site. Therefore, the potential for surface fault rupture to occur at the site is considered to be very low.

The potential for liquefaction to occur at the site was assessed. The term liquefaction refers to a phenomenon that tends to occur in saturated soils of low density and that have grain sizes within a certain range, usually fine- to medium-grained poorly graded sands, silty sands, and silts. A sufficiently strong earthquake is also required to cause liquefaction. During liquefaction, the energy from the earthquake causes the water pressure within the pores of the soil to increase. The increase in water pressure decreases the friction between the soil grains, allowing the soil grains to move relative to one another. During this state, the soil will behave as a viscous liquid, temporarily losing its ability to support foundations and other improvements. The high pressure water will flow through the soil along the path of least resistance. As the pressure is released, the soils typically settle in a process called "dynamic settlement." Dynamic settlement can cause damage to structures and other surface and subsurface improvements.

To assess the potential for liquefaction, subsurface data from Boring 3 were used as input for a computer-generated analysis. Groundwater was encountered at a depth of 22 feet in the borings. Groundwater is a required element for liquefaction, and as groundwater rises, liquefaction becomes more likely. While the trend in the area of the site is for the water level to get lower with time, for the sake of conservatism, we assumed a 5-foot rise in the groundwater to a depth of 17 feet.

Another of the input parameters for the liquefaction analysis is whether or not a particular stratum is considered to have a sufficient liquefaction potential that additional analysis is warranted. Based upon the information from Boring 3, soils from the surface to a depth of 14.5 feet were judged to be susceptible to liquefaction. The clay soils from 14.5 feet to 43 feet were considered to be non-liquefiable. This is due to their being categorized as a cohesive, clay soil, and due to the unconfined compressive strength of over 8,300 psf that resulted from a test on this material. Below 43 feet the clayey sand soils were again considered to be liquefiable.

Liquefaction analysis also requires both the earthquake magnitude and the Peak Ground Acceleration (PGA). A discussion of these parameters was presented previously; an earthquake magnitude of 6.55 and a PGA of 0.27 were used in the analysis. With these values, liquefaction potential at the site was analyzed following the guidelines of Special Publication 117 (CDMG, 1997, Revised 2008), and recommended procedures for analyzing liquefaction potential (Martin and others, 1999) using the "Simplified Procedure" as presented at the NCEER workshop and summarized by Youd and others (2001). The analysis also considered recent information presented by Seed and others (2003), and Idriss and others (2004). A factor of safety of 1.25 was used to determine the liquefaction potential.

Based upon the discussed input parameters and our analysis, no layers of the site are predicted to liquefy; the minimum factor of safety against liquefaction was 1.70. Dynamic settlement of unliquefied dry sand layers was estimated at 0.06 inch. Consequently, liquefaction and dry sand settlement are simply not a concern.

The primary concern from a geotechnical engineering standpoint is the potential for differential settlement. Specifically, differential settlement will occur between the new structure, (which has yet to settle) and the old structure (that probably has undergone virtually all of its settlement), as well as the differential settlement that may be created due to non-uniformity of the site. The non-uniformity will come from disturbance of the soils as the existing buildings, and service utilities are removed, and due to the removal of the 36-inch waterlines.

As the existing buildings and service utilities are removed, the site will become disturbed. However, this disturbance is expected to be relatively shallow and could be mitigated by a shallow overexcavation and recompaction program. Removal of the 36-inch water pipes will result in disturbance to a much greater depth, possibly up to a depth of 6 feet in a narrow zone that crosses diagonally through the site. The trench from which the pipes are removed will be backfilled with compacted soil but this zone will then be firmer to a greater depth than the remainder of the site. Accordingly, it would still behave somewhat differently than the remainder of the site. To mitigate this effect, a stepped overexcavation and recompaction program is recommended. The overexcavation will begin relatively shallow near the existing building and will step down about half way between the building and the waterline trench, stepping down again at the trench. On the far side of the trench, the earthwork program will step up in two increments. This should spread out the non-uniform soil conditions that would otherwise influence the site.

10.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

These preliminary geotechnical recommendations are applicable for the proposed improvements as described in the "Introduction" section of this report. If improvements not previously noted are included in the project, or if locations, elevations, structural loads, etc., change, the recommendations contained herein may require modification. If taller retaining walls, mechanically stabilized earth walls, or other such features are incorporated into the project, the soils engineer should be contacted for individual assessment.

The "building area" is defined as the area within and extending a minimum of 5 feet beyond the perimeter of the new structure. The term "foundation area" as used in conjunction with new footings within the remodeled existing structure or site retaining walls includes the footprint of the foundation. The "grading area" is defined as the entire area to be graded; it includes the building area, site wall foundation areas, and any areas where surface improvements will be constructed or fill will be placed.

Site Preparation

1. The ground surface in the grading area should be prepared for construction by removing all existing fill and backfill, vegetation, large roots, debris, existing foundations, asphalt concrete (AC), and other deleterious materials. Existing service utility lines that will not remain in service should be either removed or properly abandoned. The appropriate method of service utility abandonment will depend upon the type and depth of the utility. Recommendations for abandonment of service utilities can be made as necessary

2. The existing 36-inch waterlines should be removed as discussed in the following "Grading" section.

3. Voids created by the removal of materials or utilities should be called to the attention of the soils engineer. No fill should be placed unless the underlying soil has been observed by the soils engineer.

Grading

Pipe Removal Excavation

1. The 36-inch water pipes should be removed and the ends sealed in accordance with the specifications of the architect/engineer or the utility company.

2. The bottom of the trench should then be cut to a uniform grade and observed by the soils engineer. The trench should be cut no wider than necessary to allow removal of the pipes.

3. The soil exposed in the bottom of the excavation should be scarified a minimum of 12 inches, moisture conditioned, and recompacted to a minimum of 90 percent of maximum dry density.

4. The excavation should be backfilled exclusively with nonexpansive materials. Nonexpansive materials are defined as belonging in the GM, GC, SP, SW, SC and SM categories per ASTM D 2487-10, and that have an expansion index of 10 or less (ASTM D 4829-08a). The surficial on-site materials are considered nonexpansive. Proposed imported nonexpansive materials should be reviewed by the soils engineer before being brought to the site, and on an intermittent basis during placement.

North Addition

1. Where the addition to the north side of the courthouse is planned, the existing soil should be removed to a depth of 2 feet below the proposed bottom-of-footing elevation. This elevation is referred to as the "main overexcavation elevation." The main overexcavation elevation should extend from the existing building to a distance approximately midway between the building and the limits of the pipe removal trench. At this mid-point, the earthwork should step down from the main overexcavation elevation to an elevation that is approximately half way to the elevation of the bottom of the pipe removal trench. This is referred to as the "intermediate overexcavation elevation." For example, if the main overexcavation elevation is 4 feet below existing grade, and the depth of the pipe removal trench is 6 feet below existing grade, the intermediate overexcavation elevation starting at the midpoint between the building and the edge of the excavation would be 5 feet below existing grade.

2. On the far side of the pipe removal trench, the overexcavation should extend to the intermediate overexcavation elevation for a distance of 10 feet from the far edge of the trench.

3. At 10 feet beyond the pipe removal trench, the overexcavation program should step up to the main overexcavation elevation for the remainder of the building area.

4. Adjacent to the existing building, the overexcavation program should be accomplished in short segments or the existing footings should be protected from settling by other means. While protection of the existing improvements is the responsibility of the contractor, the width of each overexcavation segment should probably be no greater than 8 feet. Each

segment should be properly backfilled with compacted soil before moving to the next segment.

5. Use of vibratory compaction equipment is not recommended within 10 feet of the existing structure. During compaction within this 10-foot zone, and whenever vibratory compaction equipment is being used on the site, the building and interior furnishings and contents should be monitored for damage. If any damage is noted, the operation should be immediately discontinued.

6. Throughout the building area, the surfaces exposed by overexcavation should be scarified to a minimum depth of 1 foot, moisture conditioned to optimum moisture content or just above, and recompacted to a minimum of 90 percent of maximum dry density.

7. A typical diagram of this recommended earthwork program is included in Appendix E.

West Corridor and Interior Footing Areas for Remodeled Existing Building

1. In the foundation area for the addition planned along the west side of the courthouse and in any foundation areas for new footings in the existing building, the existing soil should be removed to a depth of 2 feet below the proposed bottom-of-footing elevation. The exposed soil surface should be moisture conditioned to optimum moisture content or just above, and recompacted to a minimum of 90 percent of maximum dry density.

2. The excavation should then be backfilled to bottom-of-footing elevation with moisture conditioned, properly compacted nonexpansive soil. The backfill should be placed in maximum 6-inch thick lifts.

Site Retaining Walls

1. In site retaining wall foundation areas, the soil should be removed to bottom-of-footing elevation (not including any keyway). The resulting surface should be moisture conditioned to optimum moisture content or just above, and recompacted.

2. Please note that the recommendations presented above apply *only to site retaining walls*. Where retaining walls will be rigidly attached to, or will form part of, the building, the earthwork should conform to the recommendations for the building area.

Parking Lots and Drive Aisles

1. Where asphalt concrete (AC) or Portland cement concrete (PCC) pavement is planned, the existing soil should be removed to a uniform plane below the level of disturbance resulting from removal of existing improvements. The exposed soil surface should be scarified to a minimum depth of 1 foot, moisture conditioned to optimum moisture content or just above, and recompacted to a minimum of 95 percent of maximum dry density.

2. Please see "Pavement Design Criteria" for additional recommendations regarding AC and PCC pavement.

General

1. Beyond the building and pavement areas, surfaces to receive fill or surface improvements should be scarified to a minimum depth of 1 foot, moisture conditioned to optimum moisture content or just above, and recompacted.

2. Any materials to be used as general fill should be nonexpansive. Nonexpansive materials are defined as belonging in the GM, GC, SP, SW, SC and SM categories per ASTM D 2487-10, and that have an expansion index of 10 or less (ASTM D 4829-08a). The on-site soils and appropriate imported soils, once cleared of any vegetation and deleterious materials, may be used as general fill material. Proposed imported nonexpansive materials should be reviewed by the soils engineer before being brought to the site, and on an intermittent basis during placement.

3. All materials used as fill should be cleaned of all debris, and any rocks larger than 3 inches in diameter. If fill material includes rocks, the rocks should be placed in a sufficient soil matrix to ensure that voids caused by nesting of the rocks will not occur and that the fill can be properly compacted.

4. All fill should be placed with moisture contents slightly above optimum moisture content. Moisture contents well in excess of optimum should be avoided, as unstable conditions could result and mitigating measures (as noted in the following paragraph) could be needed.

5. Depending on *in situ* soil moisture content at the time of construction, there is a potential for the site soils to become unstable during grading. Unstable soils are difficult to properly compact and are unsuitable for the placement of additional lifts of fill. Methods to correct instability include scarification and aeration of the soils in place, or the placement of gravel layers or geotextiles. The appropriate method to be utilized will depend on the conditions observed at the time of construction.

6. In general, fill should be placed in maximum lifts of 8 inches in loose thickness, however, in small areas such as footing excavations and trenches, lift thickness should be decreased to 6 inches. Fill and backfill should be compacted to a minimum of 90 percent of the maximum dry density unless otherwise recommended. The upper 12 inches of subgrade and all aggregate base in areas to be paved with AC or PCC should be compacted to a minimum of 95 percent of maximum dry density.

7. Aggregate base and subgrade should be firm and unyielding when proofrolled by heavy rubber-tired equipment prior to paving.

8. The recommended soil moisture content should be maintained throughout construction. Soils that have been disturbed should be removed, moisture conditioned, and recompacted. To reduce the potential for disruption of drainage patterns, rodent activity should be aggressively controlled.

Utility Trenches

1. Unless otherwise recommended, utility trenches adjacent to footings should not be excavated within the zone of foundation influence, as shown in Typical Detail A in Appendix E.

2. Utilities that must pass beneath a footing should be placed with properly compacted utility trench backfill and the foundation should be designed to span the trench.

3. A select, noncorrosive, granular, easily compacted material should be used as bedding and shading immediately around utilities. The site soil or imported nonexpansive soil may be used for trench backfill above the select material.

4. In general, trench backfill should be compacted to a minimum of 90 percent of maximum dry density. In areas to be paved with AC or PCC pavement, a minimum of 95 percent of maximum dry density should be obtained for the 12 inches below subgrade and in all aggregate base.

5. Trench backfill should be placed in level lifts not exceeding 6 inches in loose thickness and compacted to the minimums noted above. Trench backfill should be moisture conditioned to optimum moisture content or just above prior to application of compactive effort.

6. Compaction of trench backfill by jetting or flooding is not recommended except under extraordinary circumstances. However, to aid in *encasing* utility conduits, particularly corrugated drain pipes, and multiple, closely-spaced conduits in a single trench, jetting or flooding may be useful. Flooding or jetting should only be attempted with extreme caution, and any jetting operation should be subject to review by the soils engineer.

7. The recommendations of this section are minimums only, and may be superseded by the architect/engineer based upon soil corrosivity or the requirements of pipe manufacturers, utility companies or the governing jurisdiction. Soil corrosivity test results and recommendations for mitigation of soil corrosivity are included in Appendix C for use by the architect/engineer in specifying corrosion protection measures.

Foundations

1. The remodeled structure and addition should be supported by continuous and spread footings. Footings should penetrate a minimum depth of 18 inches below pad grade or the lowest adjacent grade, whichever is deeper.

2. Provided that the building and foundation areas have been graded as recommended, footings should be designed using maximum allowable bearing capacities of 1,200 psf dead load and 1,800 psf dead plus live loads. Using these criteria, maximum settlement and differential settlement are expected to be on the order of 1/2 inch and 3/8 inch in 25 feet, respectively.

3. Where new footings will abut existing footings, the two should be doweled together unless there is a structural reason not to do so.

4. Allowable bearing and friction capacities may be increased by one-third when transient loads such as wind or seismicity are included. Foundations may be designed using the seismic parameters provided previously.

5. Foundations supporting the heavier loads of the holding cells may be designed as individual footings or mat foundations, utilizing the same criteria as above. A subgrade modulus (K_{30}) of 150 pci (psi/in) may be used in the design of mat foundations for the holding cells.

6. Continuous footings and grade beams should be reinforced, at a minimum, by two No. 4 rebar, one at the top and one at the bottom, or as required by the architect/engineer. Specification of reinforcement for spread footings and mat foundations is left to the architect/engineer.

7. Lateral loads may be resisted by friction and by passive resistance of the soil acting on foundations. Lateral capacity is based on the assumption that backfill adjacent to foundations is properly compacted. Please see the "Retaining Walls" section for lateral parameters to be used for design purposes.

8. Footing excavations should be observed by the soils engineer prior to placement of reinforcing steel or concrete. Footing excavations should be lightly moistened prior to concrete placement.

Interior Slabs-on-Grade and Pedestrian Flatwork

1. Interior slabs-on-grade and exterior pedestrian flatwork should have a minimum thickness of 4 full inches. Reinforcement size, placement, and dowels should be as directed by the architect/engineer; minimum interior slab and flatwork reinforcement should consist of No. 3 rebar placed at 24 inches on-center each way.

2. Due to the current use of impermeable floor coverings, water-soluble flooring adhesives, and the speed at which buildings are now constructed, moisture vapor transmission

through slabs is a much more common problem than in past years. Where moisture vapor transmitted from the underlying soil would be undesirable, the slabs should be protected from subsurface moisture vapor. A number of options for vapor protection are discussed below; however, the means of vapor protection, including the type and thickness of the vapor retarder, if specified, are left to the discretion of the architect/engineer.

3. Several recent studies, including those of American Concrete Institute (ACI) Committees 302 and 306, have concluded that excess water above the vapor retarder increases the potential for moisture damage to floor coverings and could increase the potential for mold growth or other microbial contamination. The studies also concluded that it is preferable to eliminate the typical sand layer beneath the slab and place the slab concrete in direct contact with a "Class A" vapor retarder, particularly during wet weather construction. However, placing the concrete directly on the vapor retarder requires special attention to using the proper vapor retarder (see discussion below), a very low water-cement ratio in the concrete mix, and special finishing and curing techniques.

4. Probably the next most effective option would be vapor-inhibiting admixtures and/or surface sealers. This would also require special concrete mixes and placement procedures, depending upon the recommendations of the admixture or sealer manufacturer.

5. Another option that may be a reasonable compromise between effectiveness and cost considerations is the use of a subslab vapor retarder protected by a sand layer. If a "Class A" vapor retarder (see discussion below) is specified, the barrier can be placed directly on the prepared subgrade. The retarder should be covered with a minimum 2 inches of *clean* sand. If a less durable vapor retarder is specified (Class B or C), a minimum of 4 inches of clean sand should be provided on top of the prepared subgrade, and the retarder should be placed in the center of the clean sand layer. Clean sand is defined as a well or poorly graded sand (ASTM D 2487-06) of which less than 3 percent passes the No. 200 sieve. The clean sand layer, if utilized, is considered to be part of the nonexpansive layer recommended in the "Grading" section of this report to be placed below slabs-on-grade, not in addition to it.

6. Regardless of the underslab vapor retarder selected, proper installation of the retarder is critical for optimum performance. All seams must be properly lapped, and all seams and utility penetrations properly sealed in accordance with the vapor retarder manufacturer's recommendations.

7. If sand is used between the vapor retarder and the slab, it should be moistened only as necessary to promote concrete curing; saturation of the sand should be avoided, as the excess moisture would be on top of the vapor retarder, potentially resulting in vapor transmission through the slab for months or years.

8. Positive drainage away from the building should be maintained, see the "Drainage Around and Maintenance" section for additional discussion of this issue. If water is allowed to pond near the structure, it may seep into the ground and migrate laterally through cracks or utility penetrations in the foundation, ultimately gaining access above the barrier.

9. To reduce shrinkage cracks in concrete, the concrete aggregates should be of appropriate size and proportion, the water/cement ratio should be low, the concrete should be properly placed and finished, contraction joints should be installed, and the concrete should be properly cured. This is particularly applicable to slabs that will be cast directly upon a vapor retarder and those that will be protected from transmission of vapor by use of admixtures or surface sealers. Concrete materials, placement, and curing specifications should be at the direction of the architect/engineer; ACI 302.1R-04 and ACI 302.2R-04 are suggested as resources for the architect/engineer in preparing such specifications.

Retaining Walls

1. Retaining walls should be supported by soil that has been overexcavated or recompacted per the "Grading" section of this report.

2. Foundations for retaining walls should have a minimum depth (not including any keyway) of 18 inches below the lowest grade.

3. The on-site silty sand soil or imported sand, or gravel may be used as retaining wall backfill.

4. Retaining wall design should be based on the following parameters:

Active equivalent fluid pressure	35 pcf
At rest equivalent fluid pressure	50 pcf
Passive equivalent fluid pressure	300 pcf
Maximum allowable toe pressure	1,800 psf
Coefficient of sliding friction	0.45

5. No surcharges are taken into consideration in the values presented above. The maximum toe pressure is an *allowable* value; no factors of safety, load factors or other factors have been applied to the remaining values. With the exception of the maximum toe pressure, these values will require application of appropriate factors of safety, load factors, and/or other factors as deemed appropriate by the architect/engineer.

6. Due to the height of the walls (maximum of 3 feet) and the low seismic accelerations expected at the site, design of walls to accommodate seismic loads should not be necessary.

7. The above pressures are applicable to a horizontal retained surface behind the wall. Walls having a retained surface that slopes upward from the wall should be designed for an additional equivalent fluid pressure of 1 pcf for the active case and 1.5 pcf for the at-rest case, for every two degrees of slope inclination.

8. Foundations for the addition should not bear in retaining wall backfill without special consideration by the soils engineer.

9. Long-term settlement of properly compacted site soils, imported sand or gravel retaining wall backfill should be assumed to be about 0.25 to 0.5 percent of the depth of the backfill. Improvements that are constructed near the tops of retaining walls should be designed to accommodate the potential for settlement.

10. The above active and at-rest values are for drained conditions. Consequently, all retaining walls should be drained with perforated pipe encased in a free-draining gravel blanket. The pipe should be placed perforations downward, and should discharge in a nonerosive manner away from foundations and other improvements. The gravel blanket

should have a width of approximately 1 foot and should extend upward to approximately 1 foot from the top of the wall backfill. The upper foot should be backfilled with native soil, except in areas where pavement or flatwork will abut the top of the wall. In such cases, the gravel should extend to the aggregate base or other material as appropriate. To reduce infiltration of the soil into the gravel, a permeable synthetic fabric conforming to Caltrans Standard Specifications, Section 88-1.03 for Underdrains, should be placed between the two. Manufactured synthetic drains, such as Miradrain or Enkadrain are acceptable alternatives to the use of gravel, provided that they are installed in accordance with the recommendations of the manufacturer.

11. Where weep hole drainage can be properly discharged, the perforated pipe may be omitted in lieu of weep holes on maximum 4-foot centers. A filter fabric as described above should be placed between the weep holes and the drain gravel.

12. Walls facing habitable areas or areas where moisture transmission through the wall would be undesirable should be *thoroughly* waterproofed in accordance with the specifications of the architect/engineer.

13. The architect/engineer should bear in mind that retaining walls by their nature are flexible structures, and that surface treatments on walls often crack. Where walls are to be plastered or otherwise have a finish applied, the flexibility should be considered in determining the suitability of the surfacing material, spacing of horizontal and vertical control joints, etc. The flexibility should also be considered where a retaining wall will abut or be connected to a rigid structure, and where the geometry of the wall is such that its flexibility will vary along its length.

Pavement Design Criteria

Pavement Sections, AC

The following AC pavement sections are based upon a tested R-value, or resistance to deformation under repeated loading, of 49. The pavement sections are based on assumed Traffic Indices (TI) of 4.5 through 8.0. Determination of the appropriate TI for specific areas of the project is left to others. The AC sections were calculated in accordance with the method presented in the *Caltrans Highway Design Manual*. This method includes a safety factor. Normal Caltrans construction tolerances should apply. The calculated aggregate base and AC thicknesses are for compacted material. Final AC recommendations should be based upon the R-value of the soil near the subgrade level during construction.

R-value	**Traffic Index**	**AC**	**Class 2 Base**
49	4.5	2.50"	4.0"
49	5.0	2.75"	4.0"
49	5.5	3.00"	4.0"
49	6.0	3.25"	4.0"
49	6.5	3.75"	4.5"
49	7.0	4.00"	5.0"
49	7.5	4.25"	5.5"
49	8.0	4.50"	6.0"

Pavement Sections, PCC

1. PCC pavement should be considered where bus traffic is anticipated and at trash dumpsters, and other locations where trucks or busses will maneuver.

2. All PCC that will be subject to traffic loads is considered to be PCC pavement, and should be underlain by a minimum of 8 inches of Class 2 aggregate base. Design may be based on a modulus of subgrade reaction of 250 pci (psi/in.)

Pavement Sections, General

1. The upper 12 inches of subgrade and all aggregate base should be compacted to a minimum of 95 percent of maximum dry density. Subgrade and aggregate base should be firm and unyielding when proofrolled with heavy, rubber-tired grading equipment prior to continuing construction.

2. Finished AC and PCC pavement surfaces should be sloped to freely drain toward appropriate drainage facilities. Water should not be allowed to stand or pond on or adjacent to pavement or other improvements as it could infiltrate into the aggregate base and/or subgrade, causing premature pavement deterioration.

3. To reduce migration of surface drainage into the subgrade, maintenance of pavement areas is critical. Any cracks that develop in the pavement should be promptly sealed.

4. The local jurisdiction may have additional requirements for pavement that could take precedence over the above recommendations.

Drainage and Maintenance

1. Unpaved ground surfaces should be *graded during construction,* and *finish graded* to direct surface runoff away from foundations, retaining walls, and other improvements at a minimum 5 percent grade for a minimum distance of 10 feet (per CBC Section 1804A.3). If this is not feasible due to the terrain, property lines, or other factors, swales with improved surfaces, area drains, or other drainage facilities should be provided to divert drainage away from these areas. Paved surfaces should provide positive drainage away from foundations and other improvements.

2. To reduce the potential for planter drainage gaining access to subslab areas, any raised planter boxes adjacent to the structure should be installed with drains, and sealed sides and bottoms. Drains should also be provided for areas adjacent to structure that would not otherwise freely drain away from the building.

3. Eaves of the building should be provided with roof gutters. Runoff from roof gutters, downspouts, area drains, weep holes, etc., should discharge to an appropriate outlet in a nonerosive manner away from foundations and other improvements in accordance with the requirements of the governing agencies. Erosion protection should be placed at all discharge points unless the discharge is to a storm drain or to an AC or PCC surface.

4. The site soils are highly erodible. To reduce erosion damage, it is essential to stabilize surface soils, particularly those disturbed during construction. Soils should be stabilized by vegetation or other means *during and following construction.* Care should be taken to establish and maintain vegetation. The landscaping and exterior flatwork should be installed to maintain the surface drainage recommended above.

5. To reduce the potential for disruption of drainage patterns and undermining of foundations and any slopes, rodent activity should be aggressively controlled.

Observation and Testing

1. It must be recognized that the recommendations contained in this report are based, in part, on a limited number of borings drilled at the site and rely on continuity of the subsurface conditions encountered.

2. Unless otherwise stated, the terms "compacted" and "recompacted" refer to soils placed in level lifts not exceeding 8 inches in loose thickness and compacted to a minimum of 90 percent of maximum dry density.

3. Unless otherwise stated, "moisture conditioning" refers to the moistening or drying of soils to optimum moisture content or just above, prior to application of compactive effort.

4. The standard tests used to define maximum dry density and field density should be ASTM D 1557-09 and ASTM D 6938-10, respectively, or other methods acceptable to the soils engineer and jurisdiction.

5. At a minimum, the soils engineer should be retained to provide:
   - Review of grading, retaining wall, and foundation plans and details
   - Professional observation during grading
   - Oversight of soil special inspection and testing during grading and backfill

6. Special inspection of grading and backfill should be provided as per Section 1704A.7 and Table 1704.7 of the CBC; the soil special inspector should be under the direction of the soils engineer. The following should be inspected by the soil special inspector:
   - Stripping and clearing of vegetation
   - Verification of preparation of the bottom of the pipe removal trench
   - Verification of stepped overexcavation to the correct depths and areas
   - Scarification, moisture conditioning and recompaction of the bottoms of the overexcavation areas
   - Service utility and pipe removal trench backfill
   - Retaining wall backfill
   - Fill quality, placement, moisture conditioning, and compaction
   - Foundation excavations

7. A program of quality control should be developed prior to the beginning of the project. The contractor or project manager should determine any additional inspection items required by the architect/engineer or the governing jurisdiction.

8. Locations and frequency of compaction tests should be as per the recommendation of the soils engineer at the time of construction. The recommended test location and frequency may be subject to modification by the soils engineer, based upon soil and moisture conditions encountered, size and type of equipment used by the contractor, the general trend of the results of compaction tests, or other factors.

9. A preconstruction conference among the owner, the jurisdiction, the soils engineer, the soil special inspector, the architect/engineer, and contractors is recommended to discuss planned construction procedures and quality control requirements.

10. The soils engineer should be notified at least 48 hours prior to beginning construction operations. If Earth Systems Pacific is not retained to provide construction observation and testing services, it shall not be responsible for the interpretation of the information by others or any consequences arising there from.

11.0 CLOSURE

Our intent was to perform the investigation in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in the locality of this project and under similar conditions. No representation, warranty, or guarantee is either expressed or implied. This report is intended for the exclusive use by the client as discussed in the "Scope of Services" section. Application beyond the stated intent is strictly at the user's risk.

This report is valid for conditions as they exist at this time for the type of project described herein. The conclusions and recommendations contained in this report could be rendered invalid, either in whole or in part, due to changes in building codes, regulations, standards of geotechnical or construction practice, changes in physical conditions, or the broadening of knowledge.

If changes with respect to project type or location become necessary, if items not addressed in this report are incorporated into plans, or if any of the assumptions used in the preparation of this report are not correct, the soils engineer shall be notified for modifications to this report. Any items not specifically addressed in this report should comply with the CBC and the requirements of the governing jurisdiction.

The preliminary recommendations of this soils and geologic hazards report are based upon the geotechnical and geologic conditions encountered at the site and may be augmented by additional requirements of the architect/engineer, or by additional recommendations provided by this firm based on conditions exposed at the time of construction.

This document, the data, conclusions, and recommendations contained herein are the property of Earth Systems Pacific. This report shall be used in its entirety, with no individual sections reproduced or used out of context. Copies may be made only by Earth Systems Pacific, the client, and the client's authorized agents for use exclusively on the subject project. Any other use is subject to federal copyright laws and the written approval of Earth Systems Pacific.

Thank you for this opportunity to have been of service. If you have any questions, please feel free to contact this office at your convenience.

End of Text.

REFERENCES

Blake, T. F., EQSEARCH, "A Computer Program for the Estimation of Peak Horizontal Acceleration from California Historical Earthquake Catalogs," User's Manual, Newbury Park, California, 2000, database updated 2010.

Boore, D. M., and Atkinson G. M. (2008), "Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods between 0.01 s and 10.0 s," Earthquake Spectra, Volume 24, No. 1, pages 99–138, February 2008; © 2008, Earthquake Engineering Research Institute.

Boore, D. M., W.B. Joyner, and T.E. Fumal, "Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of Recent Work," *United States Geological Survey Open File Report,* 1997 (*93-509).*

California Building Code, 2010.

California Department of Water Resources (DWR), Unpublished water level data, downloaded from website, http://well.water.ca.gov , July 31, 2005.

California Division of Mines and Geology, Guidelines for Evaluating and Mitigating Seismic Hazards in California: California Division of Mines and Geology Special Publication 117, 1997.

California Geological Survey, "Fault Activity Map of California," Compiled by Charles W. Jennings and William A. Bryant, 2010

Campbell, K.W., "Empirical Near-Source Attenuation Relationships for Horizontal and Vertical Components of Peak Ground Acceleration, Peak Ground Velocity, and Pseudo-Absolute Acceleration Response Spectra," Seismological Research Letters, Vol. 68, No. 1, pp. 154-179, 1997.

Campbell, K.W., Bozorgnia, Y., "NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10.0 s," Earthquake Spectra, Volume 24, No. 1, pages 139–171, February 2008; © 2008, Earthquake Engineering Research Institute.

Chiou, Brian S. J. and Robert R. Youngs, "A NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra," Earthquake Spectra, Volume 24, No. 1, pages 173–215, February 2008; © 2008, Earthquake Engineering Research Institute.

Federal Emergency Management Agency, "Flood Insurance Rate Map," Map Number 06077C0640F, dated October 16, 2009.

REFERENCES CONTINUED

Huang, Yin-Nan, Andrew S. Whittaker, and Nicolas Luco, Maximum Spectral Demands in the Near-Fault Region, *Earthquake Spectra,* Volume 24, No. 1, pages 319–341, February 2008; Earthquake Engineering Research Institute.

Idriss, I. M., and R.W. Boulanger, "Semi-Empirical Procedures for Evaluating Liquefaction Potential during Earthquakes," 11th International Conference on Soil Dynamics & Earthquake Engineering (ICSDEE) and 3rd International Conference on Earthquake Geotechnical Engineering (ICEGE), 2004.

Martin, G. R. and Lew, M. eds., 1999, Recommended procedures for implementation of DMG Special Publication 117 guidelines for analyzing and mitigating liquefaction hazard in California: Southern California Earthquake Center, 63 p.

Risk Engineering, "EZ-FRISK, Version 7.51," 2009.

San Joaquin County, Eastern San Joaquin Groundwater Basin, Groundwater Management Plan, Northeastern San Joaquin County Groundwater Banking Authority, September 2004.

Seed, R.B., K. O. Cetin, R. E. S. Moss, A. M. Kammerer, J. Wu., J. M. Pestana, M. F. Riemer, R. B. Sancio, J. D. Bray, R. E. Yayen, and A. Faris, "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework," 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, April 2003.

Wagner, D.L., Bortugno, E.J., and McJunkin, R.D., Geologic Map of the San Francisco-San Jose Quadrangle, California Department of Conservation, Division of Mines and Geology, Scale 1:250,000, 1991.

Youd, T. L., and I. M. Idriss, *Liquefaction Resistance of Soils: Summary Report from the 1996 and 1998 NCEER/NSF Workshops on Evalation of Liquefaction of Soils*, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 4, 2001.

United States Geological Survey, Earthquake Hazards Program Website, 2010.

APPENDIX A

Boring Location Map

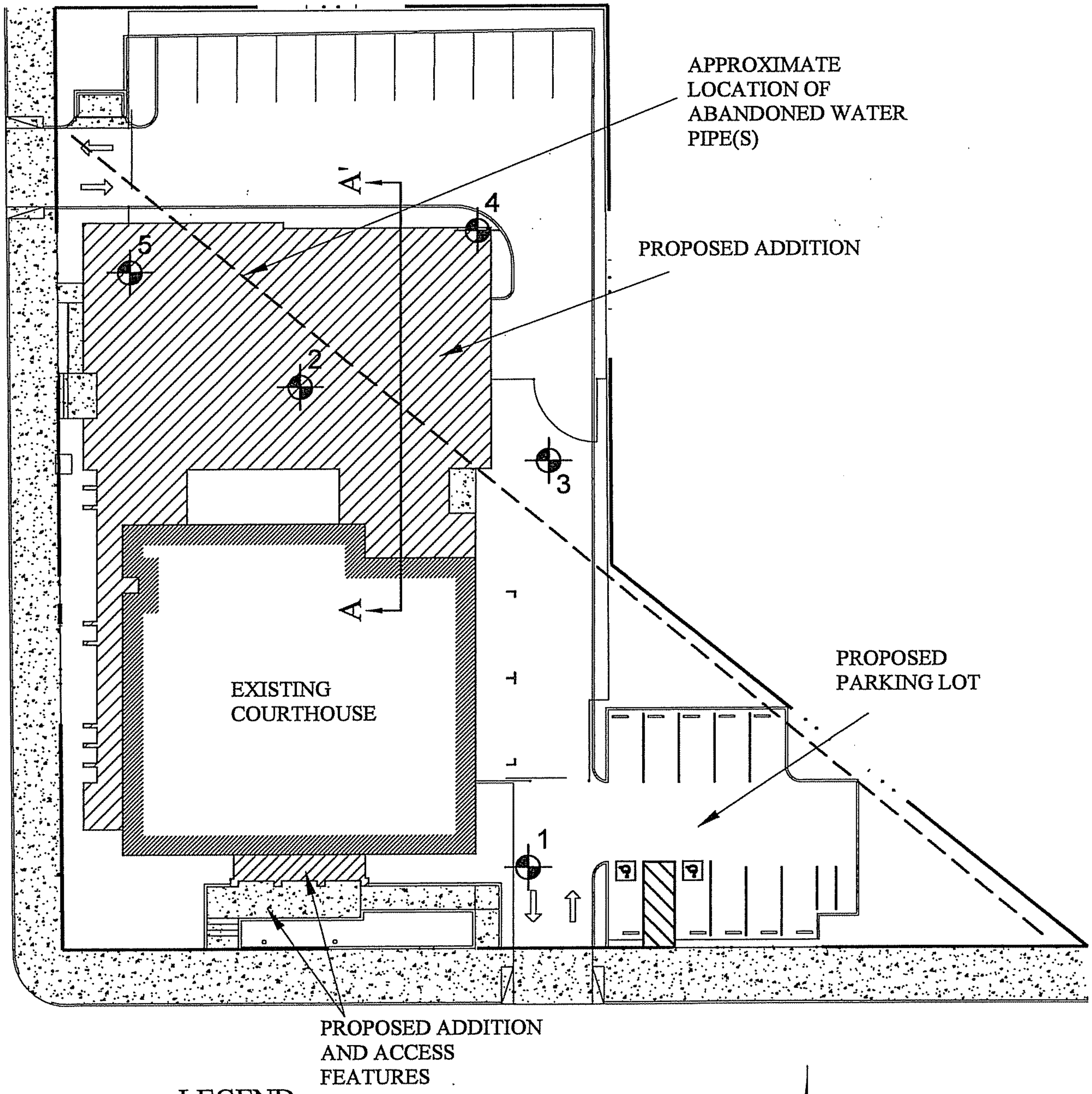
Boring Log Legend

Boring Logs

BORING LOCATION MAP

MANTECA COURTHOUSE ADDITION

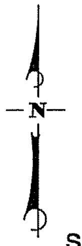
East Center Street
Manteca, California



LEGEND

5 ⊕ Boring Location (Approx.)

A └─┘ A' Cross Section



NOT TO SCALE

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May 3, 2011

SB

SL-16437-SA

Earth Systems Pacific

BORING LOG LEGEND

SAMPLE / SUBSURFACE WATER SYMBOLS

SAMPLE / SUBSURFACE WATER SYMBOLS	GRAPH. SYMBOL
CALIFORNIA MODIFIED	■
STANDARD PENETRATION TEST (SPT)	●
SHELBY TUBE	▭
BULK	○
SUBSURFACE WATER DURING DRILLING	▼
SUBSURFACE WATER AFTER DRILLING	▽

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTIONS	GRAPH. SYMBOL
COARSE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS TESTED OR JUDGED TO BE LARGER THAN #200 SIEVE SIZE	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	○
		GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	⬡
		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, NON-PLASTIC FINES	▦
		GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, PLASTIC FINES	⬡
		SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	▒
		SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	░
		SM	SILTY SANDS, SAND-SILT MIXTURES, NON-PLASTIC FINES	▥
		SC	CLAYEY SANDS, SAND-CLAY MIXTURES, PLASTIC FINES	▩
FINE GRAINED SOILS	HALF OR MORE OF MATERIAL IS TESTED OR JUDGED TO BE SMALLER THAN #200 SIEVE SIZE	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY, CLAYEY FINE SANDS, 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	┄
		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY, SILTY SOILS, ELASTIC SILTS	▥
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	▧
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS	┅
		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	▽

OBSERVED MOISTURE CONDITION

DRY	SLIGHTLY MOIST	MOIST	VERY MOIST	WET
LITTLE/NO MOISTURE	JUDGED BELOW OPTIMUM	JUDGED ABOUT OPTIMUM	JUDGED OVER OPTIMUM	SATURATED

TYPICAL CONSISTENCY

COARSE GRAINED SOILS

BLOWS/FOOT		DESCRIPTIVE TERM
SPT	CA SAMPLER	
0-10	0-16	LOOSE
11-30	17-50	MEDIUM DENSE
31-50	51-83	DENSE
OVER 50	OVER 83	VERY DENSE

FINE GRAINED SOILS

BLOWS/FOOT		DESCRIPTIVE TERM
SPT	CA SAMPLER	
0-2	0-3	VERY SOFT
3-4	4-7	SOFT
5-8	8-13	MEDIUM STIFF
9-15	14-25	STIFF
16-30	26-50	VERY STIFF
OVER 30	OVER 50	HARD

GRAIN SIZES

U.S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENING		
# 200	# 40	# 10	# 4	3/4"	3"	12"

SILT & CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

TYPICAL ROCK HARDNESS

MAJOR DIVISIONS	TYPICAL DESCRIPTIONS
EXTREMELY HARD	CORE, FRAGMENT, OR EXPOSURE CANNOT BE SCRATCHED WITH KNIFE OR SHARP PICK; CAN ONLY BE CHIPPED WITH REPEATED HEAVY HAMMER BLOWS
VERY HARD	CANNOT BE SCRATCHED WITH KNIFE OR SHARP PICK; CORE OR FRAGMENT BREAKS WITH REPEATED HEAVY HAMMER BLOWS
HARD	CAN BE SCRATCHED WITH KNIFE OR SHARP PICK WITH DIFFICULTY (HEAVY PRESSURE); HEAVY HAMMER BLOW REQUIRED TO BREAK SPECIMEN
MODERATELY HARD	CAN BE GROOVED 1/16 INCH DEEP BY KNIFE OR SHARP PICK WITH MODERATE OR HEAVY PRESSURE; CORE OR FRAGMENT BREAKS WITH LIGHT HAMMER BLOW OR HEAVY MANUAL PRESSURE
SOFT	CAN BE GROOVED OR GOUGED EASILY BY KNIFE OR SHARP PICK WITH LIGHT PRESSURE, CAN BE SCRATCHED WITH FINGERNAIL; BREAKS WITH LIGHT TO MODERATE MANUAL PRESSURE
VERY SOFT	CAN BE READILY INDENTED, GROOVED OR GOUGED WITH FINGERNAIL, OR CARVED WITH KNIFE; BREAKS WITH LIGHT MANUAL PRESSURE

TYPICAL ROCK WEATHERING

MAJOR DIVISIONS	TYPICAL DESCRIPTIONS
FRESH	NO DISCOLORATION, NOT OXIDIZED
SLIGHTLY WEATHERED	DISCOLORATION OR OXIDATION IS LIMITED TO SURFACE OF, OR SHORT DISTANCE FROM; SOME FRACTURES PRESENT; FELDSPAR CRYSTALS ARE DULL
MODERATELY WEATHERED	DISCOLORATION OR OXIDATION EXTENDS FROM FRACTURES, USUALLY THROUGHOUT; Fe-Mg MINERALS ARE "RUSTY"; FELDSPAR CRYSTALS ARE "CLOUDY"
INTENSELY WEATHERED	DISCOLORATION OR OXIDATION THROUGHOUT; FELDSPAR AND Fe-Mg MINERALS ARE ALTERED TO CLAY TO SOME EXTENT OR CHEMICAL ALTERATION PRODUCES IN SITU DISAGGREGATION
DECOMPOSED	DISCOLORATION OR OXIDATION THROUGHOUT, BUT RESISTANT MINERALS SUCH AS QUARTZ MAY BE UNALTERED; FELDSPAR AND Fe-Mg MINERALS ARE COMPLETELY ALTERED TO CLAY

drafting/masters/Boring Log Legend091608.dwg

Earth Systems Pacific

Boring No. 1

PAGE 1 OF 1
JOB NO.: SL-16437-SA
DATE: 02/23/11

LOGGED BY: B. Faust
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

MANTECA COURTHOUSE ADDITION
East Center Street
Manteca, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0			2" ASPHALT CONCRETE OVER 4" AGGREGATE BASE					
1	SM		SILTY SAND: light brown, moist, loose, medium coarse grained (Alluvium)	1.0–4.0	○			
2				2.0–3.5	■	113.9	9.5	6 5 6
4			cemented	4.5–6.0	■	113.9	13.4	7 33 50–4.0"
8				8.5–10.0	●			7 8 11
9			medium dense					
13	SP		POORLY GRADED SAND: yellow gray, moist, medium dense, medium coarse grained	13.5–15.0	●			10 15 13
15			End of Boring @15.0' No subsurface water encountered					
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								

LEGEND: ■ Ring Sample ○ Grab Sample ☐ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 2

PAGE 1 OF 1
JOB NO.: SL-16437-SA
DATE: 02/23/11

LOGGED BY: B. Faust
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

MANTECA COURTHOUSE ADDITION
East Center Street
Manteca, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA: INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0			2.5" ASPHALT CONCRETE OVER 3" AGGREGATE BASE					
1	SM		SILTY SAND: light brown, moist, medium dense, mostly medium coarse grained (Alluvium)	2.0–3.5	▬	117.4	7.3	21 21 16
4				4.5–6.0	▬	117.7	9.8	6 8 14
8			loose, less silt	8.5–10.0	●			6 5 5
12	SP		POORLY GRADED SAND: yellow gray, moist, medium dense, medium coarse grained	13.5–15.0	●			15 15 20
15			End of Boring @15.0' No subsurface water encountered					

LEGEND: ▬ Ring Sample ○ Grab Sample ☐ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Boring No. 3

PAGE 1 OF 2
JOB NO.: SL-16437-SA
DATE: 02/23/11

LOGGED BY: B. Faust
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

MANTECA COURTHOUSE ADDITION
East Center Street
Manteca, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0			2.0" ASPHALT CONCRETE OVER 2.5" AGGREGATE BASE					
1	SM		SILTY SAND: light brown, moist, medium dense, fine to medium coarse grained (Alluvium)					13
2				2.0–3.5	▬	113.1	2.3	17 19
3								
4			weakly cemented	4.5–6.0	▬	114.8	3.7	14 26
5			cementation ends					28
6								
7	SP		POORLY GRADED SAND: light yellow brown, slightly moist, medium dense, medium coarse grained					
8				8.5–10.0	●			6 7
9								9
10								
11								
12								
13			predominantly coarse grained	13.5–15.0	●			12 14
14								18
15	CL		LEAN CLAY: gray, moist, medium stiff, trace fine sand					
16			thin zones of silty sand					
17								
18				18.5–20.0	●			18 28
19								30
20								
21								
22			wet ▼					
23				23.5–25.0	●			14 11
24								23
25								
26								

LEGEND: ▬ Ring Sample ○ Grab Sample ☐ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 3

PAGE 2 OF 2
JOB NO.: SL-16437-SA
DATE: 02/23/11

LOGGED BY: B. Faust
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

MANTECA COURTHOUSE ADDITION
East Center Street
Manteca, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA: INTERVAL (feet)	SAMPLE DATA: SAMPLE TYPE	SAMPLE DATA: DRY DENSITY (pcf)	SAMPLE DATA: MOISTURE (%)	SAMPLE DATA: BLOWS PER 6 IN.
27	CL	▨	LEAN CLAY: as above					
28		▨						
29		▨	moist, very stiff to hard	29.5–31.0	▬	95.6	28.8	13 21 50
30		▨						
31		▨						
32		▨						
33		▨		33.5–35.0	●			14 12 16
34		▨	medium stiff, thin zones of silty sand					
35		▨						
36		▨						
37		▨						
38		▨	yellow brown	38.5–40.0	●			4 9 9
39		▨						
40		▨	yellow brown to gray, very moist					
41		▨						
42		▨						
43	SC	▩	CLAYEY SAND: yellow brown, very moist, dense, fine to coarse grained	43.5–45.0	●			8 14 24
44		▩						
45		▩						
46		▩	thin zones of sandy lean clay					
47		▩						
48		▩		48.5–50.0	●			12 16 19
49		▩						
50			End of Boring @ 50.0' Subsurface water encountered @ 22.0'					
51								
52								
53								

LEGEND: ▬ Ring Sample ○ Grab Sample ☐ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Boring No. 4

PAGE 1 OF 1
JOB NO.: SL-16437-SA
DATE: 02/23/11

LOGGED BY: B. Faust
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

MANTECA COURTHOUSE ADDITION
East Center Street
Manteca, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0			2.0" ASPHALT CONCRETE OVER 2.5" AGGREGATE BASE					
1	SM		SILTY SAND: light brown, moist, medium dense, medium coarse grained (Alluvium)					
2				2.0–3.5	▬	115.1	6.8	20 21 14
3								
4			cemented	4.0–6.5	▬	107.1	17.1	12 50
5								
6								
7								
8				8.5–10.0	●			11 14 13
9			light gray with orange staining, fine grained					
10								
11								
12								
13	SP		POORLY GRADED SAND: yellow brown to gray, moist, dense, medium coarse grained	13.5–15.0	●			13 18 23
14								
15			End of Boring @15.0' No subsurface water encountered					
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 5

PAGE 1 OF 1
JOB NO.: SL-16437-SA
DATE: 02/23/11

LOGGED BY: B. Faust
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

MANTECA COURTHOUSE ADDITION
East Center Street
Manteca, California

				SAMPLE DATA				
DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0			2.5" ASPHALT CONCRETE OVER 5" AGGREGATE BASE					
1	SM		SILTY SAND: light brown, moist, medium dense, fine to medium coarse grained (Alluvium)	1.0–5.0	○			
2				2.0–3.5	■	113.6	5.3	8 13 14
4				4.5–6.0	■	114.5	5.7	10 11 14
8				8.5–10.0	●			5 5 6
9			less silt					
12	SP		POORLY GRADED SAND: yellow gray, moist, medium dense, medium coarse grained					
13				13.5–15.0	●			8 14 17
14			dark yellow brown					
19	SC		CLAYEY SAND: gray, moist, dense, medium coarse grained	19.0–20.0	■	110.2	19.6	19 50
20			End of Boring @19.5' No subsurface water encountered					

LEGEND: ■ Ring Sample ○ Grab Sample ☐ Shelby Tube Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

APPENDIX B

Physical Laboratory Test Results

BULK DENSITY TEST RESULTS

ASTM D 2937-04 (modified for ring liners)

March 9, 2011

BORING NO.	DEPTH feet	MOISTURE CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
1	2.0 - 2.5	9.5	124.7	113.9
1	4.5 - 5.0	13.4	129.2	113.9
2	2.0 - 2.5	7.3	126.0	117.4
2	4.5 - 5.0	9.8	129.3	117.7
3	2.0 - 2.5	2.3	115.7	113.1
3	4.5 - 5.0	3.7	119.0	114.8
3	29.5 - 30.0	28.8	123.1	95.6
4	2.0 - 2.5	6.8	122.9	115.1
4	4.0 - 4.5	17.1	125.4	107.1
5	2.0 - 2.5	5.3	119.5	113.6
5	4.5 - 5.0	5.7	121.0	114.5
5	19.0 - 19.5	19.6	131.8	110.2

EXPANSION INDEX TEST RESULTS

ASTM D 4829-08a

BORING NO.	DEPTH feet	EXPANSION INDEX
5	1.0 - 5.0	0

PARTICLE SIZE ANALYSIS

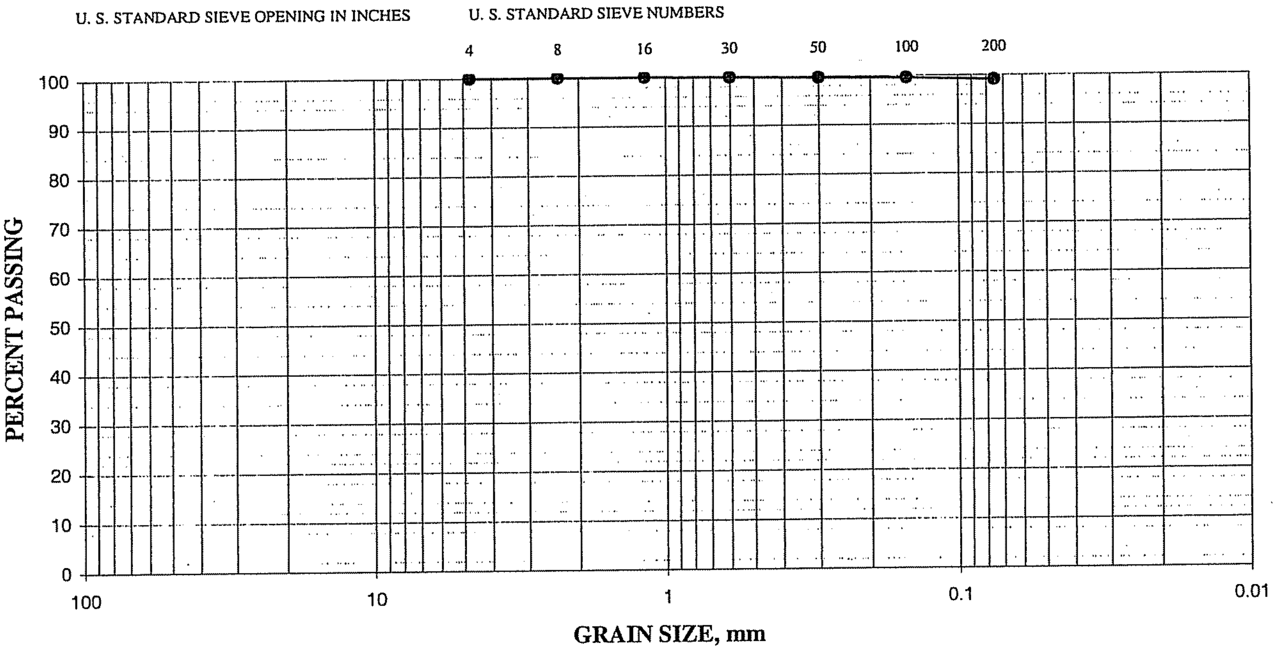
ASTM D 422-63/07; D 1140-06

Boring #3 @ 29.0 - 30.0'
Lean Clay (CL)

March 9, 2011

LL = 40; PL = 23; PI = 17

Sieve size	% Retained	% Passing
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600-μm)	0	100
#50 (300-μm)	0	100
#100 (150-μm)	0	100
#200 (75-μm)	1	99



Manteca Courthouse Addition

SL-16437-SA

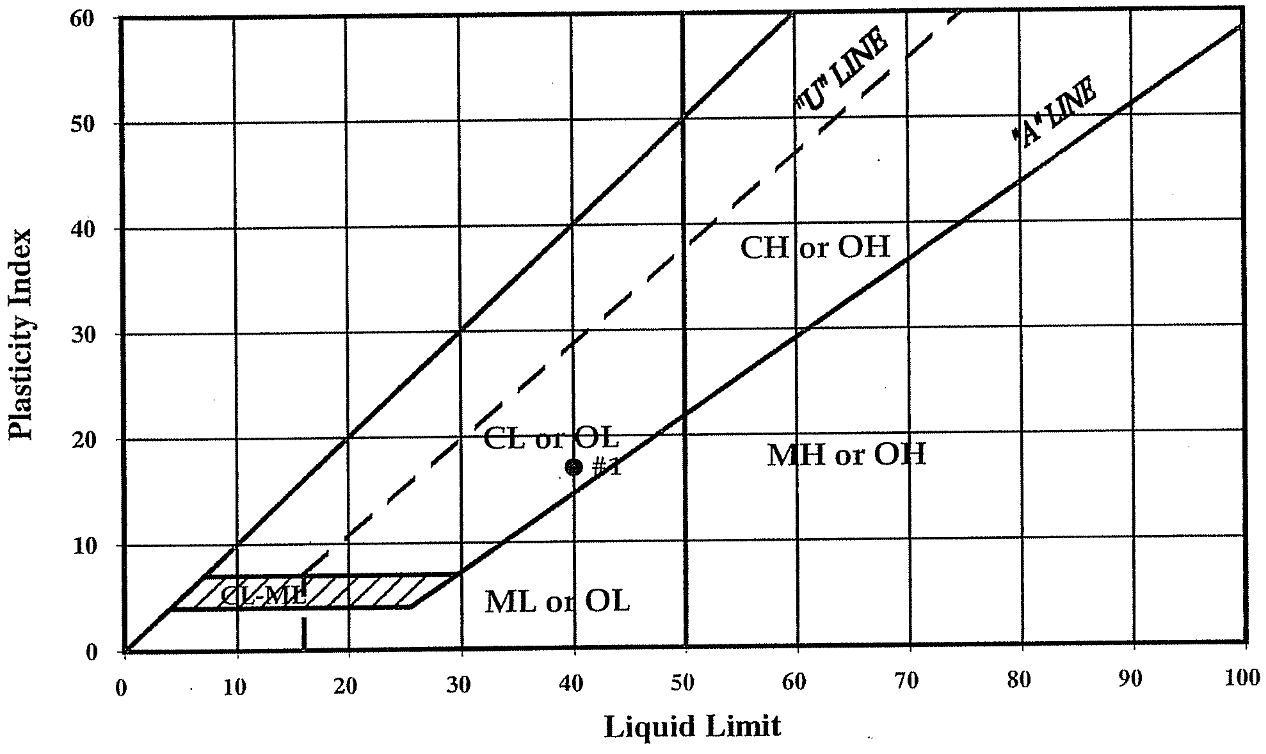
PLASTICITY INDEX

ASTM D 4318-05

March 9, 2011

Test No.:	1	2	3	4	5
Boring No.:	3				
Sample Depth:	29.5 - 30.0'				
Liquid Limit:	40				
Plastic Limit:	23				
Plasticity Index:	17				

Plasticity Chart



Manteca Courthouse Addition

SL-16437-SA

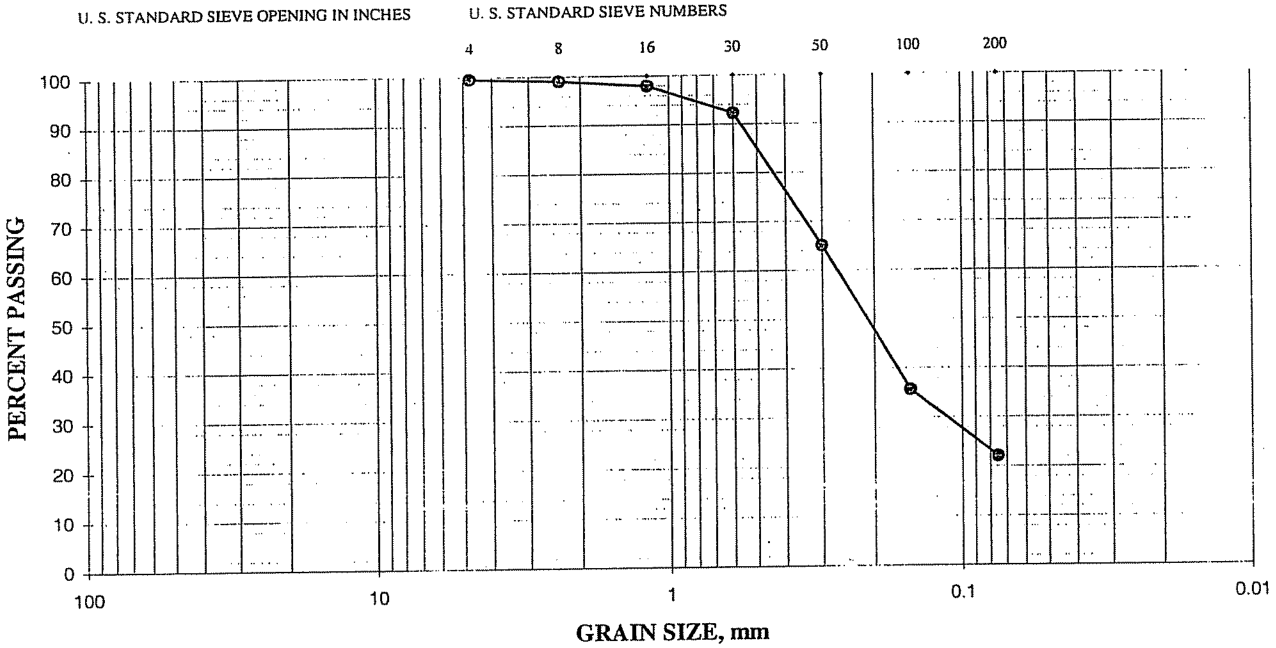
PARTICLE SIZE ANALYSIS

ASTM D 422-63/07; D 1140-06

Boring #5 @ 1.0 - 5.0'
Silty Sand (SM)

March 9, 2011

Sieve size	% Retained	% Passing
#4 (4.75-mm)	0	100
#8 (2.36-mm)	1	99
#16 (1.18-mm)	2	98
#30 (600-μm)	8	92
#50 (300-μm)	35	65
#100 (150-μm)	64	36
#200 (75-μm)	78	22



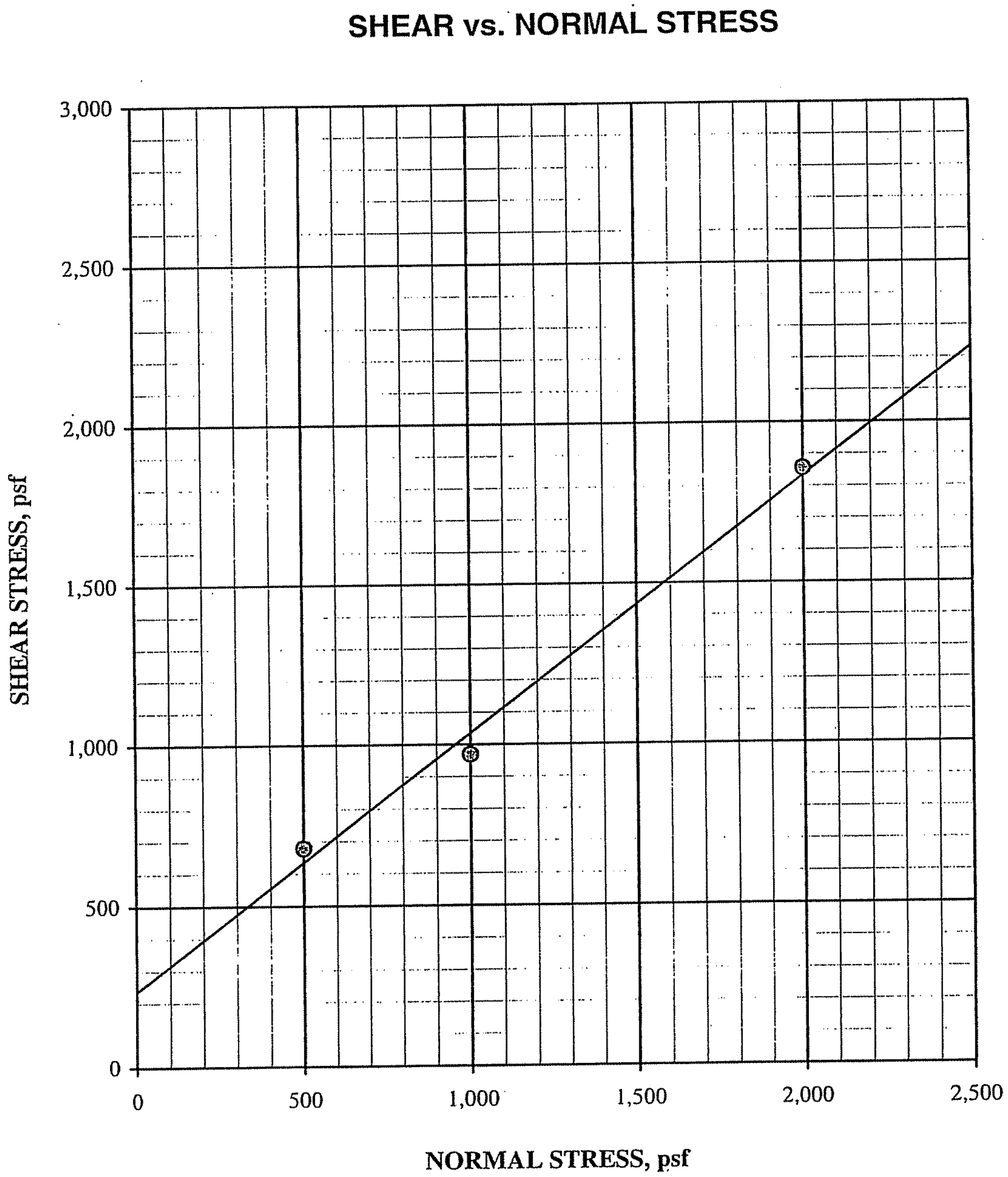
DIRECT SHEAR

ASTM D 3080-04 (modified for consolidated, undrained conditions)

March 9, 2011

Boring #5 @ 1.0 - 5.0'
Silty Sand (SM)
Compacted to 90% RC, saturated

INITIAL DRY DENSITY: 112.7 pcf
INITIAL MOISTURE CONTENT: 8.6 %
PEAK SHEAR ANGLE (Ø): 39°
COHESION (C): 235 psf



DIRECT SHEAR continued

ASTM D 3080-04 (modified for consolidated, undrained conditions)

Boring #5 @ 1.0 - 5.0'

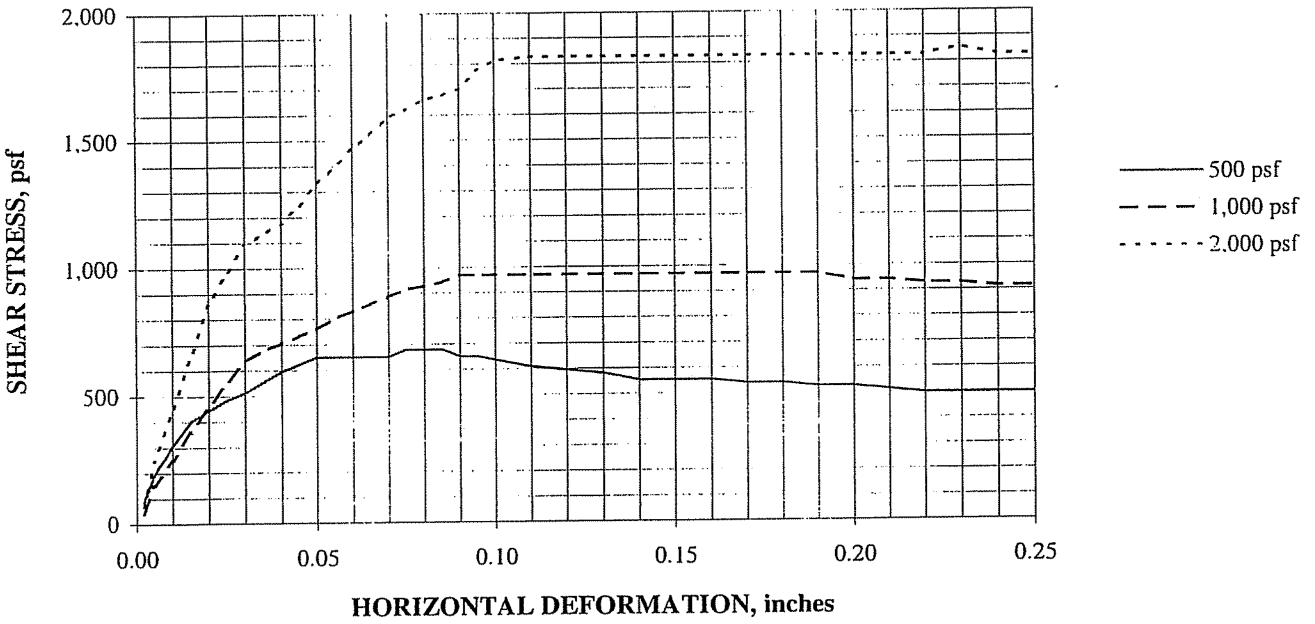
March 9, 2011

Silty Sand (SM)

Compacted to 90% RC, saturated

SPECIFIC GRAVITY: 2.65 (assumed)

SAMPLE NO.:	1	2	3	AVERAGE
INITIAL				
WATER CONTENT, %	8.6	8.6	8.6	8.6
DRY DENSITY, pcf	112.7	112.7	112.7	112.7
SATURATION, %	48.7	48.7	48.7	48.7
VOID RATIO	0.467	0.467	0.467	0.467
DIAMETER, inches	2.375	2.375	2.375	
HEIGHT, inches	1.00	1.00	1.00	
AT TEST				
WATER CONTENT, %	18.8	16.3	17.1	
DRY DENSITY, pcf	114.3	115.6	115.8	
SATURATION, %	100.0	100.0	100.0	
VOID RATIO	0.447	0.431	0.428	
HEIGHT, inches	0.99	0.98	0.97	



Manteca Courthouse Addition

SL-16437-SA

MOISTURE-DENSITY COMPACTION TEST

ASTM D 1557-09 (Modified)

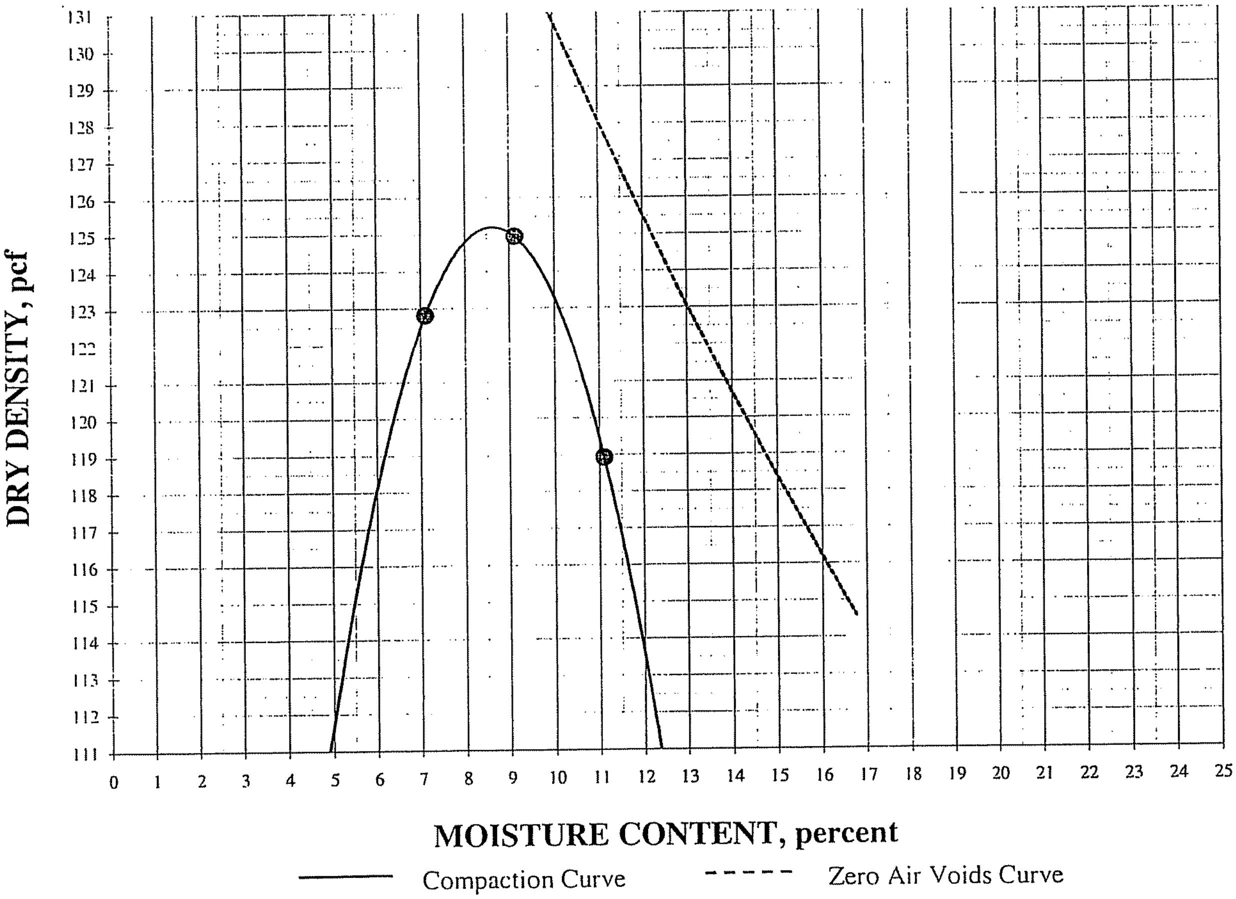
PROCEDURE USED: A
PREPARATION METHOD: Moist
RAMMER TYPE: Mechanical
SPECIFIC GRAVITY: 2.65 (assumed)

March 9, 2011
Boring #5 @ 1.0 - 5.0'
Light Brown Silty Sand (SM)

SIEVE DATA:

Sieve Size	% Retained
3/4"	0
3/8"	0
#4	0

MAXIMUM DRY DENSITY: 125.2 pcf
OPTIMUM MOISTURE: 8.6%



Manteca Courthouse Addition

SL-16437-SA

CONSOLIDATION TEST

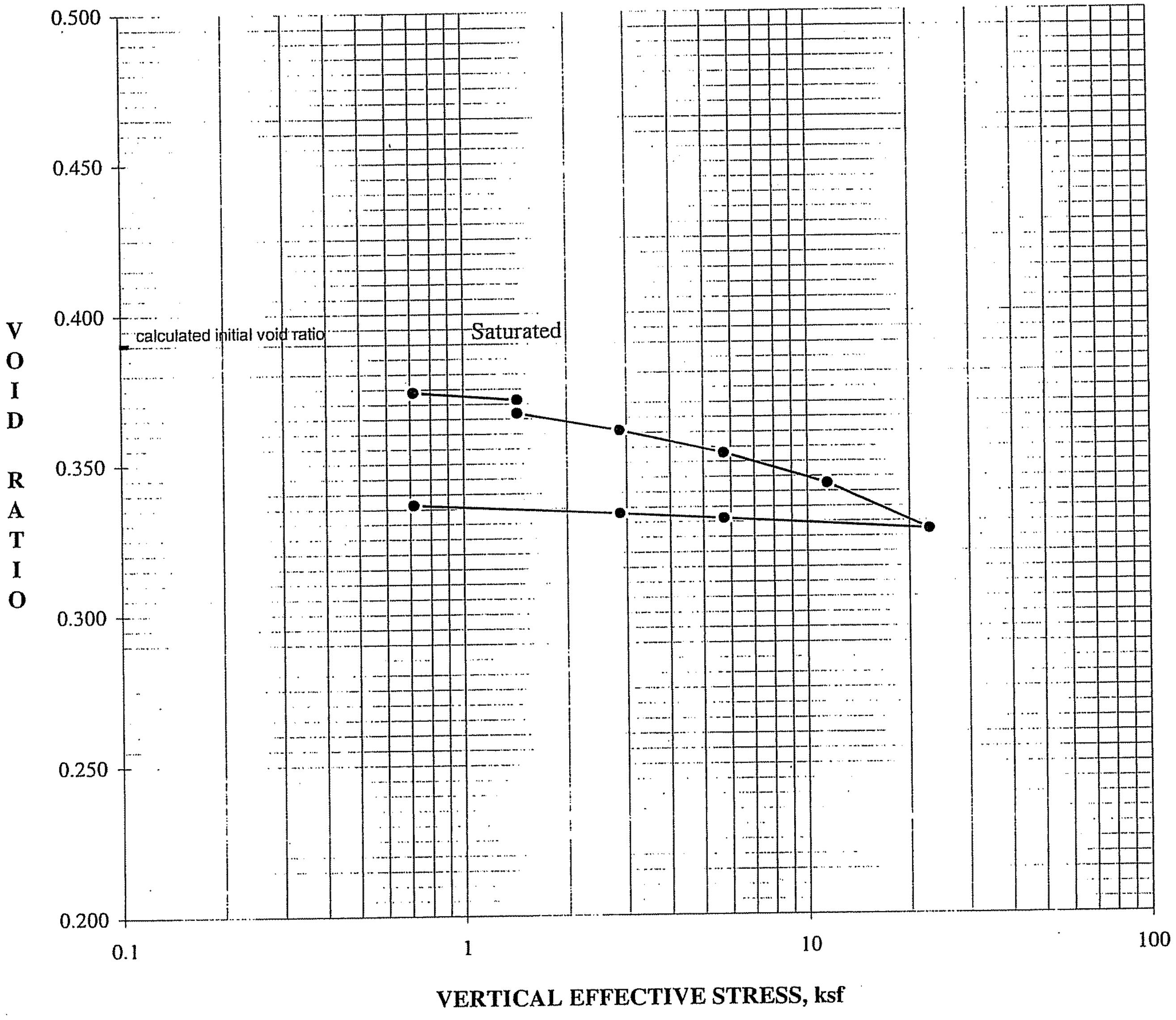
ASTM D 2435-04

March 9, 2011

Boring #2 @ 2.0 - 2.5'
Silty Sand (SM)
Ring Sample

DRY DENSITY: 119.0 pcf
MOISTURE CONTENT: 7.3%
SPECIFIC GRAVITY: 2.65 (assumed)
INITIAL VOID RATIO: 0.390

VOID RATIO vs. NORMAL PRESSURE DIAGRAM



Manteca Courthouse Addition

SL-16437-SA

CONSOLIDATION TEST

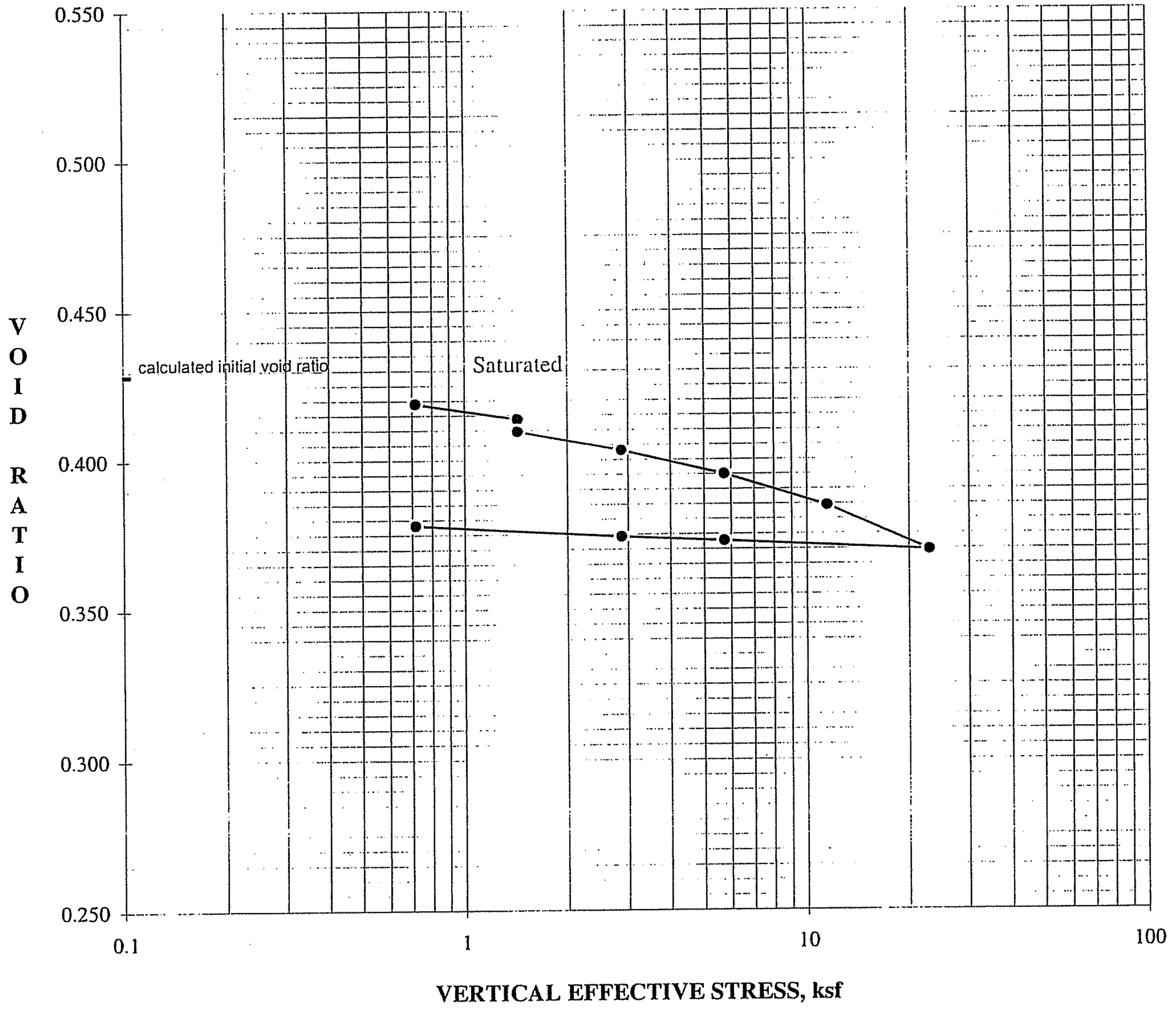
ASTM D 2435-04

March 9, 2011

Boring #2 @ 4.5 - 5.0'
Silty Sand (SM)
Ring Sample

DRY DENSITY: 115.8 pcf
MOISTURE CONTENT: 9.8%
SPECIFIC GRAVITY: 2.65 (assumed)
INITIAL VOID RATIO: 0.428

VOID RATIO vs. NORMAL PRESSURE DIAGRAM



UNCONFINED COMPRESSION ON COHESIVE SOIL

ASTM D 2166-06

March 9, 2011

Boring #3 @ 29.5 - 30'
Lean Clay (CL)
Ring Sample
COMPRESSIVE STRENGTH: 58 psi (8,362 psf)

Dry Density: 97.2 pcf
Moisture Content: 28.8%
Degree Saturation: 100%
Specific Gravity: 2.70 (assumed)
H/D Ratio: 2.11

TIME (MINUTES)	DEFORM, in (X 1000)	AXIAL STRAIN	AREA (SQ. IN.)	APPLIED LOAD (LBS)	STRENGTH (PSI)	STRENGTH (PSF)
0.5	30	0.0060	4.46	20	4	646
1.0	61	0.0122	4.48	41	9	1,316
1.5	93	0.0186	4.51	58	13	1,850
2.0	125	0.0250	4.54	70	15	2,218
2.5	158	0.0316	4.57	85	19	2,676
3.0	189	0.0378	4.60	101	22	3,159
3.5	224	0.0448	4.64	116	25	3,602
4.0	255	0.0510	4.67	131	28	4,041
4.5	290	0.0580	4.70	152	32	4,654
5.0	328	0.0656	4.74	176	37	5,346
5.5	367	0.0734	4.78	205	43	6,174
6.0	405	0.0810	4.82	225	47	6,721
6.5	440	0.0880	4.86	248	51	7,352
7.0	477	0.0954	4.90	266	54	7,821
7.5	518	0.1036	4.94	287	58	8,362
8.0	557	0.1114	4.99	278	56	8,030
8.5	590	0.1180	5.02	166	33	4,759
9.0	627	0.1254	5.07	97	19	2,758
9.5	666	0.1332	5.11	65	13	1,831
10.0	702	0.1404	5.15	50	10	1,397

Manteca Courthouse Addition

SL-16437-SA

RESISTANCE '*R*' VALUE AND EXPANSION PRESSURE

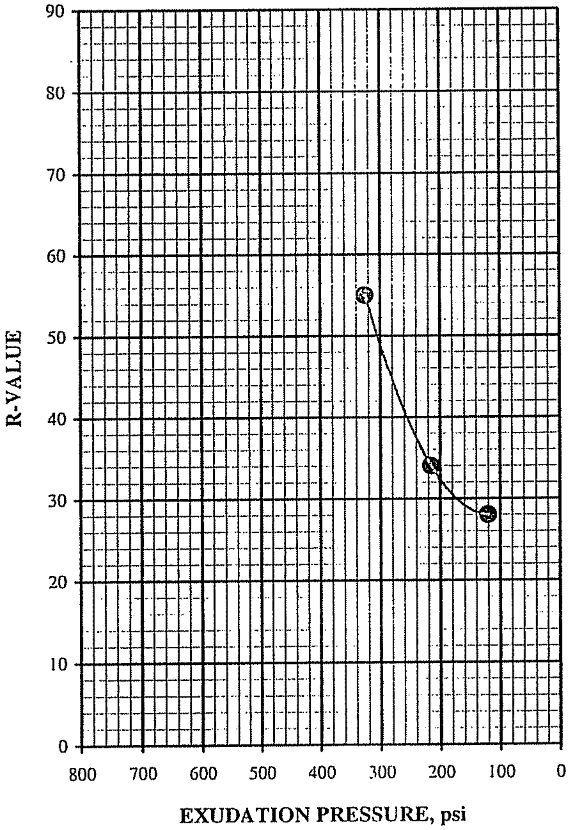
ASTM D 2844-07

March 9, 2011

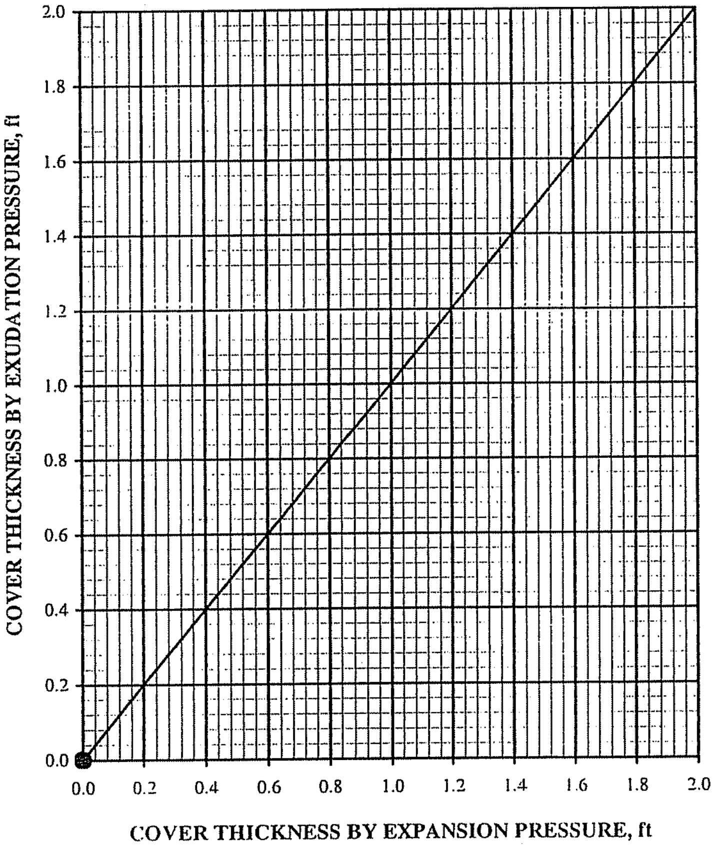
Boring #1 @ 1.0 - 4.0'
Light Brown Silty Sand (SM)

Dry Density @ 300 psi Exudation Pressure: 117.6-pcf
%Moisture @ 300 psi Exudation Pressure: 12.8%
R-Value - Exudation Pressure: 49
R-Value - Expansion Pressure: N/A
R-Value @ Equilibrium: 49

EXUDATION PRESSURE CHART



EXPANSION PRESSURE CHART



APPENDIX C

Soil Corrosivity Test Results

HDR | SCHIFF
www.hdrinc.com
Corrosion Control and Condition Assessment (C3A) Department

TRANSMITTAL LETTER

DATE: March 10, 2011

ATTENTION: **Dennis Shallenberger**

TO:
Earth Systems Pacific
4378 Old Santa Fe Road
San Luis Obispo, CA 93401

SUBJECT:
Laboratory Test Data
Manteca Courthouse Addition
Your #SL-16437-SA, SA #11-0221LAB

COMMENTS: Enclosed are the results for the subject project.

Leo Solis
Laboratory Manager

431 West Baseline Road · Claremont, CA 91711
Phone: 909.626.0967 · Fax: 909.626.3316

HDR | SCHIFF

www.hdrinc.com
Corrosion Control and Condition Assessment (C3A) Department

Table 1 - Laboratory Tests on Soil Sample(s)

Earth Systems Pacific
Manteca Courthouse Addition
Your #SL-16437-SA, SA #11-0221LAB
1-Mar-11

Sample ID			5 @ 1-5' Silty Sand SM
Resistivity		**Units**	
as-received		ohm-cm	60,000
saturated		ohm-cm	4,000
pH			8.2
Electrical Conductivity		mS/cm	0.08
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	23
magnesium	Mg^{2+}	mg/kg	3.3
sodium	Na^{1+}	mg/kg	82
potassium	K^{1+}	mg/kg	5.9
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	116
fluoride	F^{1-}	mg/kg	3.5
chloride	Cl^{1-}	mg/kg	21
sulfate	SO_4^{2-}	mg/kg	31
phosphate	PO_4^{3-}	mg/kg	16
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	16
sulfide	S^{2-}	qual	na
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

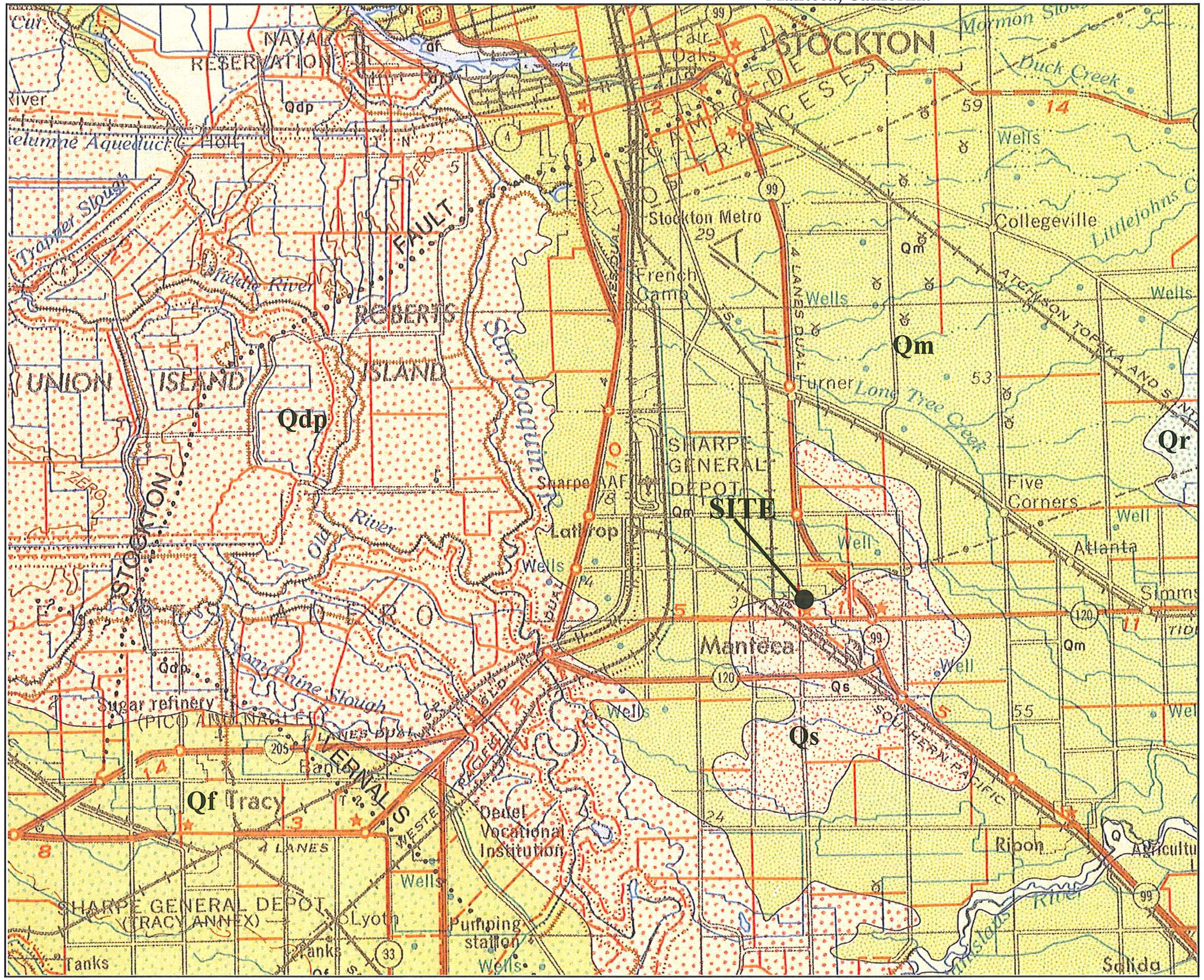
APPENDIX D

Geologic Map

Regional Fault Map

GEOLOGIC MAP

MANTECA COURTHOUSE ADDITION
East Center Street
Manteca, California



Extract from: Geologic Map of the San Francisco-San Jose Quadrangle, Wagner, Bortugno, and McJunkin, 1991

EXPLANATION

Geologic Units

Qs
Dune Sand

Qf
Alluvial Fan Deposits

Qdp
Dos Palos Alluvium

Qm
Modesto Formation

Qr
Riverbank Formation

Geologic Symbols

Contact
Dashed where approximately located or inferred

High-angle fault
Dashed where approximately located or inferred; dotted where concealed

Thrust or reverse fault
Dashed where approximately located or inferred; dotted where concealed. Saw-teeth on upper plate. Dip of fault plane between 30° and 80°

Anticline
Showing axis at surface. Dashed where approximately located; dotted where concealed

Syncline
Showing axis at surface. Dashed where approximately located; dotted where concealed

Horizontal | 30 Inclined | 90 Vertical
Strike and dip of beds

N

Approx. Scale: 1" = 3 miles

Earth Systems Pacific
4378 Old Santa Fe Road, San Luis Obispo, CA 93401
March 2011

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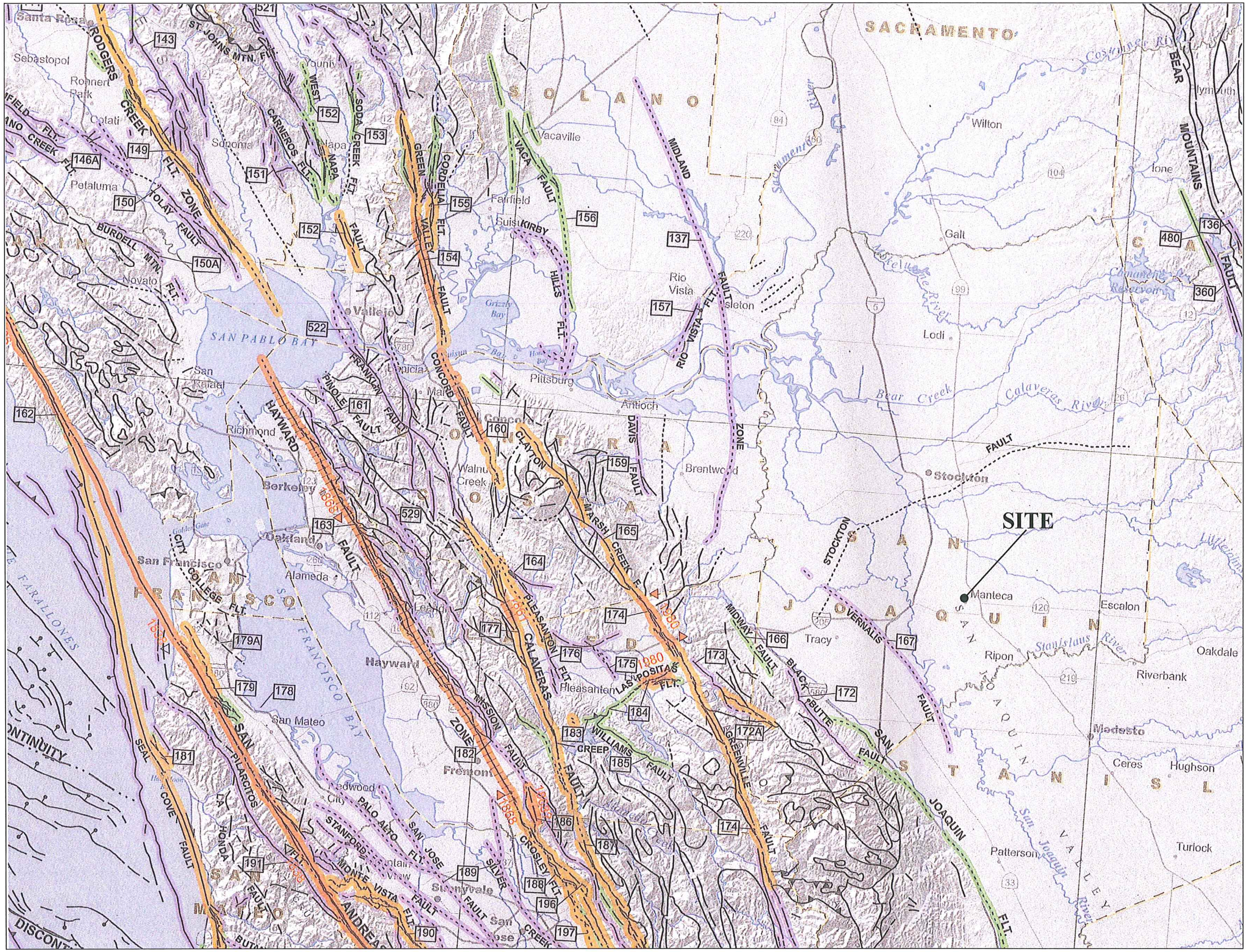
www.earthsys.com - e-mail: esc@earthsystems.com
SL-16437-SA

RG



REGIONAL FAULT MAP

MANTECA COURTHOUSE ADDITION
East Center Street
Manteca, California

LEGEND

🟨 Historic rupture (<200 years)
🟧 Holocene fault (<10,000 years)
🟩 Late Quaternary (<700,000 years)
🟪 Quaternary fault (<1.6 million)

REFERENCES

Jennings, C.W, and Bryant, W.A., 2010

N

(Approximate Scale: 1" = 15 miles)

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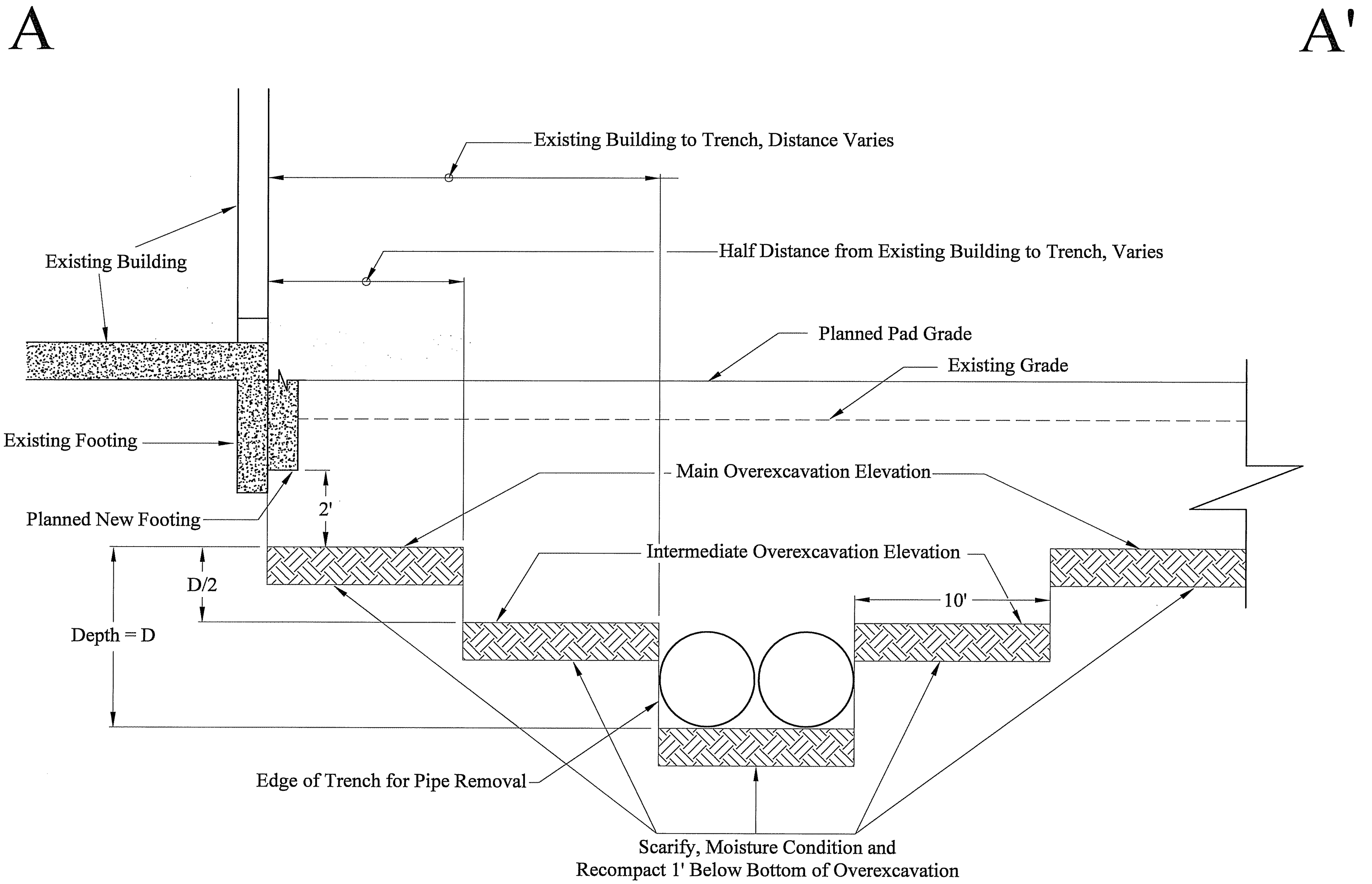
SL-16437-SA

APPENDIX E

Typical Grading Detail, Cross Section A – A'

Typical Detail A



NOT TO SCALE

Earth Systems Pacific

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San Luis Obispo, CA 93401-8116
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E-mail: esc@earthsys.com

March 28, 2011 KM SL-16437-SA

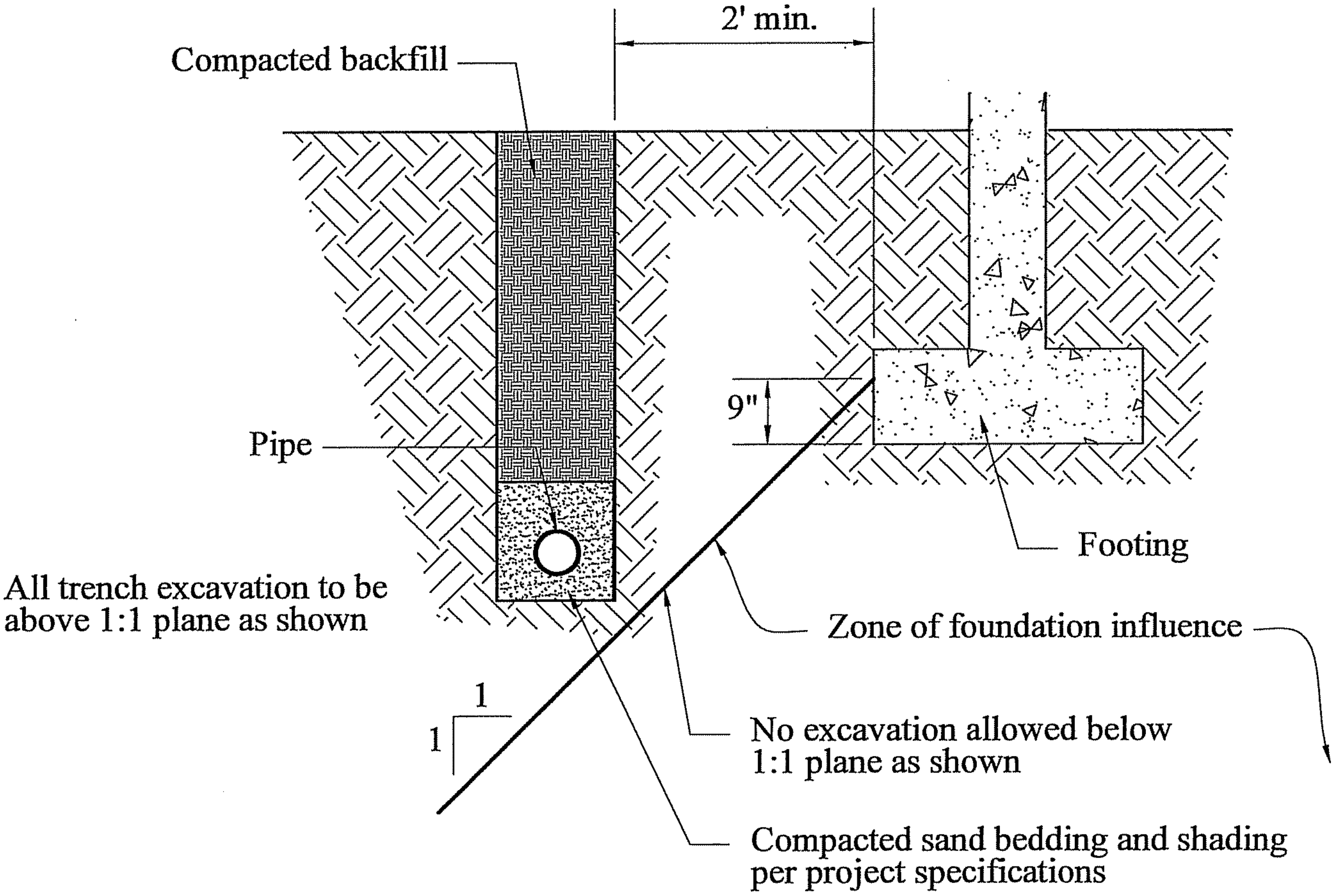
TYPICAL GRADING DETAIL CROSS SECTION A - A'

MANTECA COURTHOUSE ADDITION

East Center Street
Manteca, California

MANTECA COURTHOUSE ADDITION-032811Cross Section

TYPICAL DETAIL A
PIPE PLACE PARALLEL TO FOOTING



SCHEMATIC ONLY
NOT TO SCALE

APPENDIX F

Site Specific Design Response Spectra

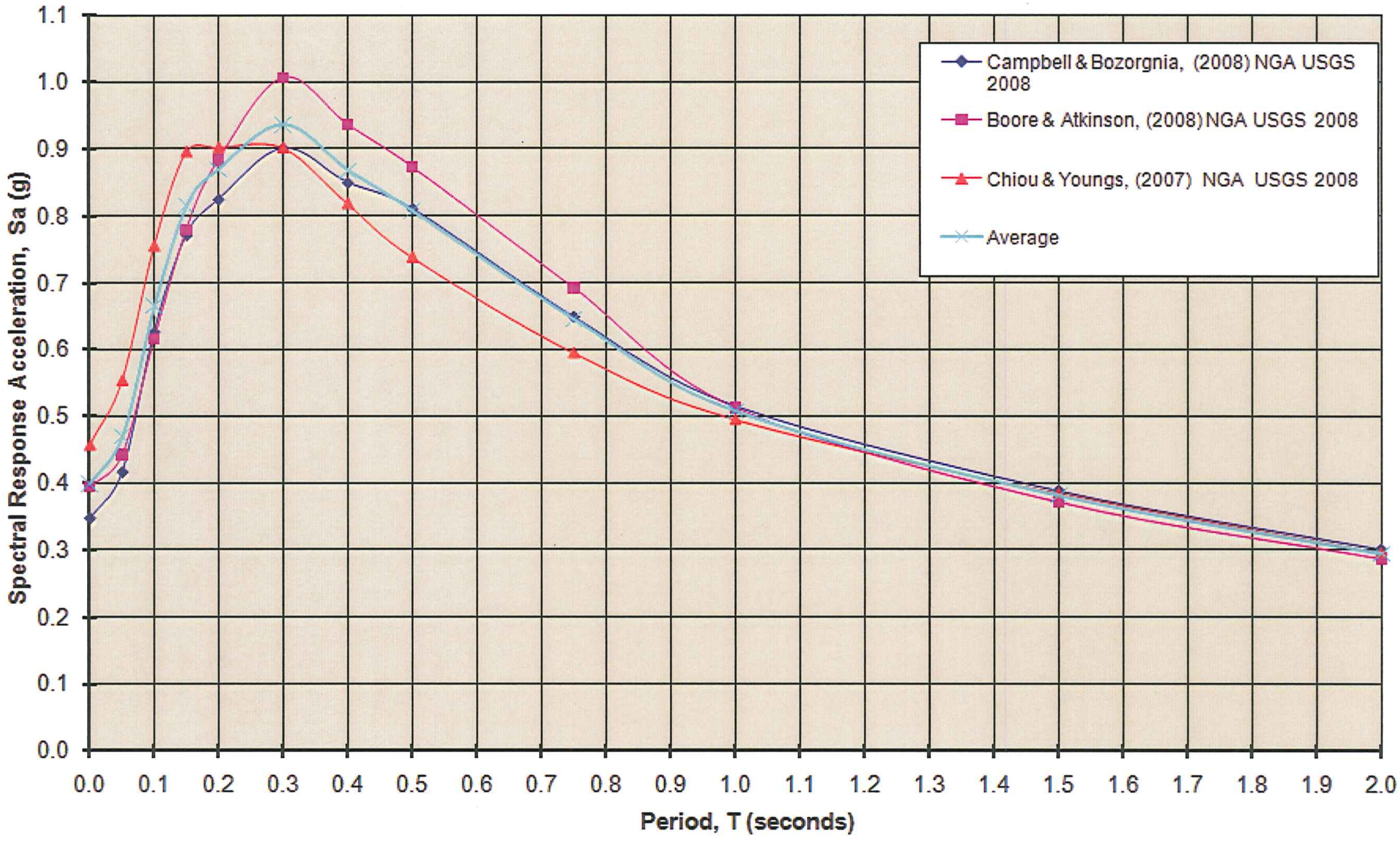
Average NGA Probabilistic MCE Response Spectra

Average NGA 84th Percentile Probabilistic MCE Response Spectra

AVERAGE NGA 84th PERCENTILE DETERMINISTIC RESPONSE SPECTRA

MANTECA COURTHOUSE ADDITION

East Center Street
Manteca, California



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March 2011

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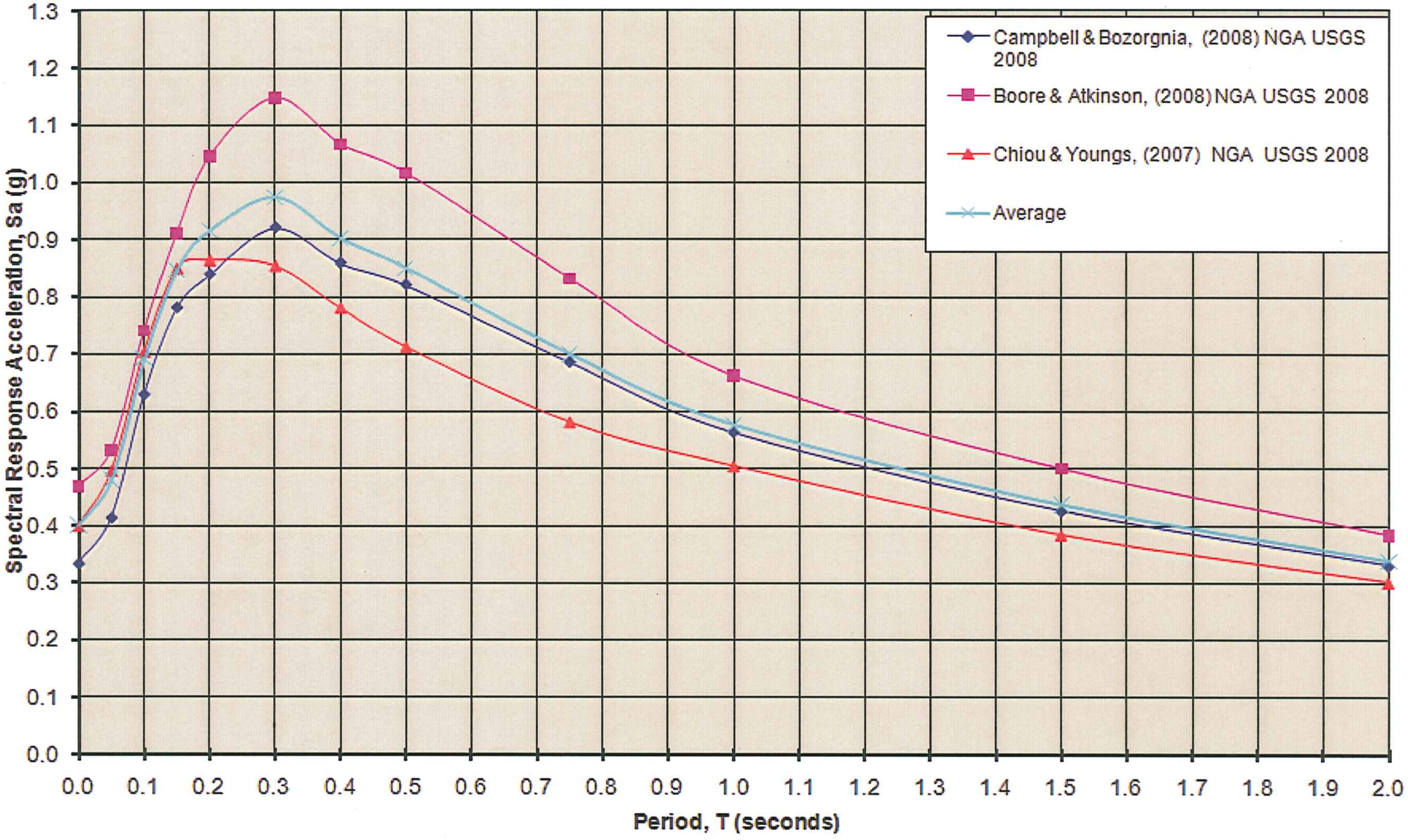
AVERAGE NGA PROBABILISTIC MCE RESPONSE SPECTRA

MANTECA COURTHOUSE ADDITION
East Center Street
Manteca, California



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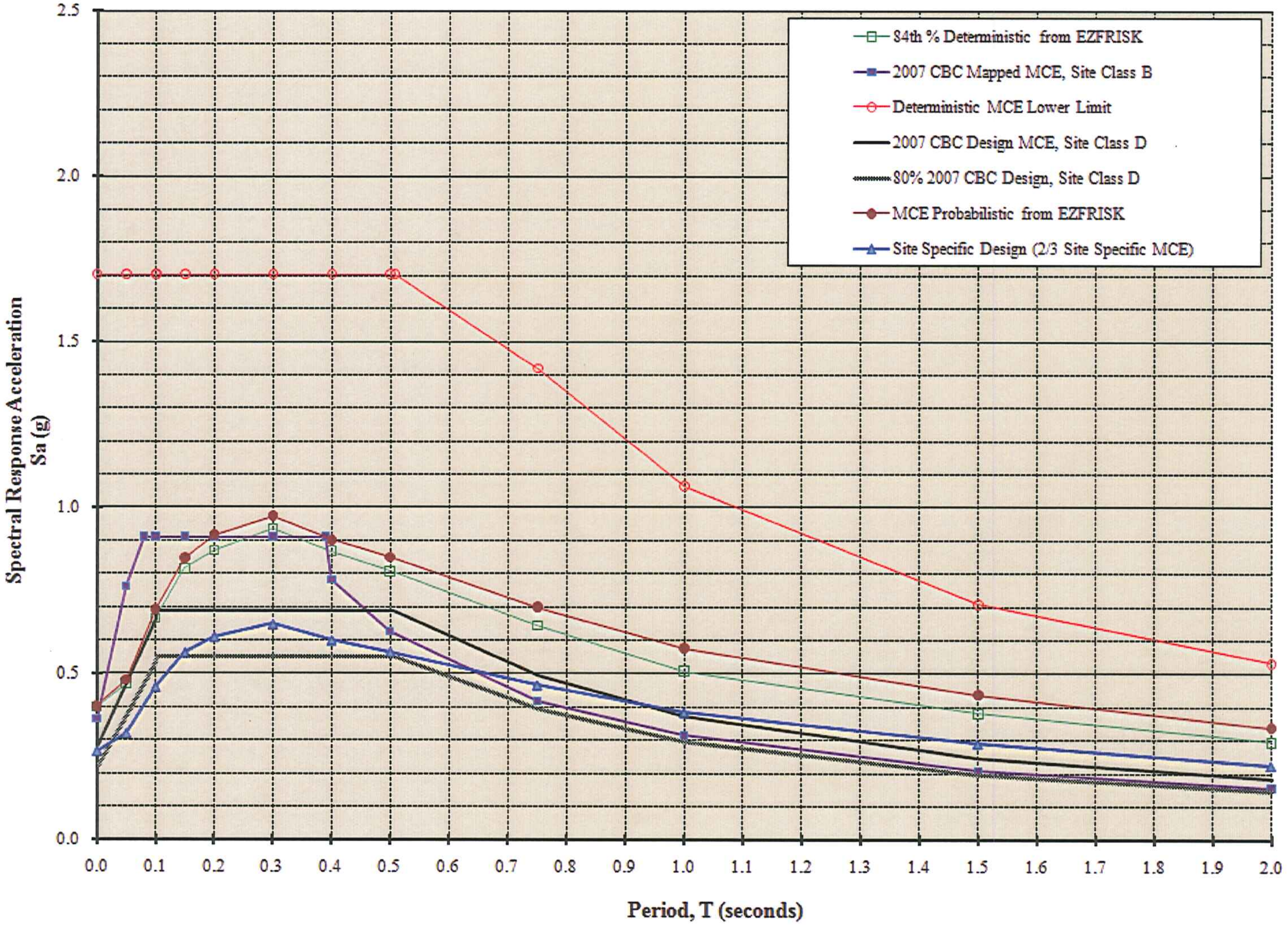
SL-16437-SA

SITE SPECIFIC DESIGN RESPONSE SPECTRUM

MANTECA COURTHOUSE ADDITION

East Center Street
Manteca, California



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