SOILS ENGINEERING REPORT
SAN LUIS OBISPO COUNTY
FIRE STATION 43
HIGHWAY 229 AT IRONGATE ROAD
CRESTON, CALIFORNIA

May 1, 2009

Prepared for

Ms. Kathy MacNeill
San Luis Obispo County
Department of General Services

Prepared by

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May 1, 2009

File No.: SL-15969-SA

Ms. Kathy MacNeill
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1087 Santa Rosa Street
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PROJECT: SAN LUIS OBISPO COUNTY FIRE STATION 43
HIGHWAY 229 AT IRONGATE ROAD
CRESTON, CALIFORNIA

SUBJECT: Soils Engineering Report

CONTRACT REF.: Purchase Order 25005062 to Provide a Soils Engineering Investigation for San Luis Obispo County Fire Station 43, Highway 229, Creston, California, by San Luis Obispo County, dated April 1, 2009

Dear Ms. MacNeill:

In accordance with the terms of the referenced purchase order, this Soils Engineering Report has been prepared for use in the development of plans and specifications for the proposed Fire Station 43 to be constructed in Creston, California. Preliminary geotechnical recommendations for site preparation, grading, utility trench backfill, foundations, exterior flatwork, retaining walls, pavement sections, drainage around improvements, and observation and testing are presented herein. Three hard copies and one electronic copy of this report are being furnished for your use.

Our geologic hazards report is submitted under separate cover.

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

Sincerely,

Earth Systems Pacific

REGISTERED PROFESSIONAL ENGINEER
FREDERICK J. POTTHAST
No. GE 2369
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Doc. No.: 0905-003.SER/tl

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1.0 INTRODUCTION

The proposed San Luis Obispo County Fire Station 43 on Highway 229 in Creston, California, will consist of an apparatus bay structure, and a separate administration/living quarters structure. The total footprint of the two structures is expected to be approximately 6,000 square feet, and they are to be constructed in the center of the site. The two-bay high-roof apparatus structure will be a single-story, and designed to accommodate a possible expansion for a third bay. The administration/living quarters may be one or two-story. Both structures are anticipated to be of steel and/or wood frame, and possible masonry, construction. Conventional continuous and spread foundations, with concrete slabs-on-grade, have been planned. Maximum continuous loads of 3 klf and maximum isolated loads of 100 kips have been assumed for the purposes of this report. The project is expected to include exterior flatwork and possibly landscaping improvements between and around the structures. Retaining walls for sitework, or connected to or forming part of a structure, and a maximum of 5 feet tall, may also be constructed. To accommodate local flood conditions, finish floor elevation of the structures may be 3 to 4 feet above existing grade. The site will be served by the existing utility systems in the area; an on-site well and drainage retention/detention basins may be located on the south side of the building area, and an on-site effluent disposal system is anticipated for the north side of the building area. Driveways and parking area improvements composed of either asphalt concrete (AC) or Portland cement concrete (PCC) over aggregate base (AB) are planned for the site, as are typical frontage improvements along Highway 229.

2.0 SCOPE OF SERVICES

The scope of work for the soils engineering report included a general site reconnaissance, subsurface exploration, laboratory testing of selected soil samples, geotechnical evaluation of the data collected, and preparation of this report. The report and subsequent recommendations were based on information and a topographic map of the site with an approximate building area provided by the client.

This soils engineering report is intended to fulfill the requirements of Sections 1802.1 through 1802.7 of the 2007 California Building Code (CBC), Chapter 33 of the 2001 CBC, and common soils engineering practice in this area under similar conditions at this time. The test procedures were accomplished in general conformance with the standards noted, as modified by common geotechnical practice in this area under similar conditions at this time.

Preliminary geotechnical recommendations for site preparation, grading, utility trench backfill, foundations, exterior flatwork, retaining walls, pavement sections, drainage around improvements, and observation and testing are presented to guide the development of project plans and specifications. The soils engineer 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 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 in the preparation of plans and specifications. Application beyond this intent is strictly at the user's risk.

This report does not address issues in the domain of contractors such as, but not limited to, site safety, subsidence of the site due to compaction, loss of volume due to stripping of the site, shrinkage of soils during compaction, excavatability, shoring, temporary slope angles, construction means and methods, etc. Analysis of the soil for radioisotopes, hydrocarbons, corrosivity, chemical properties, or toxic substances is beyond the scope of this report. Evaluations of lead or mold potential, or of the potential for asbestos (either naturally occurring or man-made) were not part of the scope of work authorized for this project. Evaluations of percolation potential, on-site effluent disposal, and ancillary features such as fences, flag and light poles, signage, and nonstructural fills, are also not within our scope and are not addressed. Analyses of areal and site geology are contained in our geologic hazards report (ESP, 2009) which is submitted under separate cover.

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 preliminary conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions presented in this report are verified or modified in writing by the soils engineer. The criteria presented in this report are considered preliminary until such time as any peer review or review by any jurisdiction has been completed, conditions have been observed by the soils engineer in the field during construction, and the recommendations have been verified as appropriate or modified in writing.

3.0 SITE SETTING

The site is on the east side of Highway 229, approximately ½-mile north of the community of Creston. Irongate Road extends to the west off Highway 229 near the north property line of the site. The site is flat, and slopes to the north at 1 percent or less. The surrounding properties are large acreage parcels that are utilized for hay, row crops or livestock grazing. The Huer Huero Creek, a seasonal drainage, is approximately 1,000 feet east of the site. At the time of our field investigation, the site was planted with a winter cover crop, and the perimeters had been disced. Aerial power and buried telephone lines are present along the Highway 229 frontage. The locations of other utility lines on the site are unknown.

4.0 FIELD INVESTIGATION

On April 3, 2009, seven exploratory borings were drilled on the site using a Mobile Drill Model B-53 truck-mounted rig, equipped with an 8-inch outside diameter hollow stem auger. The approximate locations of the borings are shown on the Boring Location Map in Appendix A. As the borings were drilled, standard penetration tests (SPT) were performed (ASTM D 1586-99), and soil samples were obtained using a ring-lined barrel sampler (ASTM D 3550-01, reapproved 07, with shoe similar to ASTM D 2937-04). The samplers were driven with an automatic trip hammer. Bulk soil samples were obtained from the auger cuttings.

Soils encountered in the borings were categorized and logged in general accordance with the Unified Soil Classification System and ASTM D 2488-06. Logs of the borings and a Boring Log Legend are also presented in Appendix A. In reviewing the boring logs and legend, the reader should recognize 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 cementation, variations in soil moisture, the presence of groundwater, and other factors. Consequently, the logger must exercise judgment in interpreting soil characteristics, possibly resulting in soil descriptions that vary somewhat from the legend.

5.0 LABORATORY ANALYSIS

Selected ring samples were tested for unit weight and moisture (ASTM D 2937-04), and for one-dimensional consolidation (ASTM D 2435-04). Bulk samples were tested for maximum density and optimum moisture (ASTM D 1557-07), expansion index (ASTM D 4829-07), angle of shearing resistance and cohesion (ASTM D 3080-04, remolded to 90 percent of maximum dry density), and R-value (ASTM D 2844-07). An SPT sample was tested for particle size distribution (ASTM D 1140-06, D422-63,-07). The laboratory test results are presented in Appendix B.

6.0 GENERAL SOIL PROFILE

In all seven borings, the upper soil encountered was alluvium consisting of loose, moist clayey sand. At 14 to 15.5 feet in Borings 1 through 3, loose to medium dense, moist to wet, well-graded sand was encountered. The well-graded sand contained a trace of coarse gravel below 15.5 feet in Boring 1, between 30 and 35 feet in Boring 2, and below 14 feet in Boring 3. Thin layers of fine grained clayey sand were found below 15.5 feet in Boring 3. The well-graded sand continued to the boring termination depths of 16.5 feet in Boring 1, 51.5 feet in Boring 2, and 21.5 feet in Boring 3. The upper clayey sand was encountered for the entire depth (5 feet) of Borings 4 through 7. Subsurface water was found in Boring 2 at 25 feet. Subsurface water was not encountered in the other six borings.

7.0 CONCLUSIONS

In our opinion, the site is suitable, from a soils engineering standpoint, for the proposed fire station and other site improvements, provided the recommendations contained herein are implemented in the design and construction. This opinion does not extend to percolation potential or suitability for on-site effluent disposal, which are not a part of our current work scope. The primary soils engineering concerns are the potentials for liquefaction and associated dynamic settlement, and the soil's erosion potential. The result of expansion index testing on a sample of the upper soils was 6; therefore, per CBC 1802.3.2, the site soils are considered to be nonexpansive and no special measures with respect to expansive soils are considered necessary.

The term "liquefaction" refers to the liquefied condition and subsequent loss of soil strength that can occur in soils when they are subjected to a sudden shock, such as that generated during an earthquake. Studies of areas where liquefaction has occurred have led to the general conclusion that saturated soil conditions, low soil density, grain sizes within a certain range, and a sufficiently strong earthquake are factors that, in combination, create a potential for 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 along the path of least resistance, which may be to the ground surface. As it flows, the water carries sand and silt in suspension, often releasing these materials on the surface in cone-shaped deposits called "sand boils."

To assess the potential for liquefaction, soil and SPT data from Boring 2 were used as input for a computer generated analysis. Groundwater was encountered at a depth of 25 feet in this boring. A potential rise in the groundwater to a depth of 14.5 feet below the existing ground surface, the gradational contact between clayey sand and well graded sand, was used in the liquefaction analysis. Based upon the surrounding topography and the distance that the Huer Huero Creek is from the site, a groundwater surface shallower than 14.5 feet was considered unlikely.

The analysis also requires both the earthquake magnitude and the Peak Ground Acceleration (PGA). The PGA value of 0.33g was taken from the Site-Specific Design Response Spectrum in the geologic hazards report (ESP, 2009). The seismicity of nearby faults was then deaggregated to determine the statistical mean and modal earthquake magnitudes that contributed to the site PGA of 0.33g. These magnitudes were estimated by performing a probabilistic seismic hazard deaggregation using the United States Geologic Survey Website (USGS, 2008). The mean and the modal magnitudes were then compared and the higher of the two was used in the analysis. In this case, the modal magnitude was higher than the mean magnitude; this value was a maximum moment magnitude of 7.78.

Using the developed seismic values, liquefaction potential at the site was analyzed following the guidelines of Special Publication 117 (CGS, 1997, revised 2008), and the 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).

The analyses indicated that there is a potential for liquefaction at the site in all of the soils below the groundwater table, with maximum dynamic settlement that would occur under the parameters analyzed on the order of 8.2 to 8.5 inches (see the Liquefaction Analysis spreadsheets in Appendix C). It should be noted that the methods of analysis available to date are largely thought to overestimate the magnitude of dynamic settlement that would actually occur. Our judgment is that the maximum dynamic settlement at the site due to a future earthquake would on the order of 4 to 6 inches with maximum differential settlement of up to 4 inches.

As previously noted, liquefaction at depth typically manifests itself in the form of isolated "sand boils" at the surface, rather than as an area-wide phenomena. If continuous or spread footings were to bear across one of these sand boil zones of liquefaction, the loss of bearing capacity and subsequent settlement can damage the structure. One option to resist liquefaction is to utilize deep foundations (i.e., piles) for structure support; the piles bear through the upper potentially liquefiable zone and into more dense, nonliquefiable materials at depth. However, deep foundations are often an inordinately expensive foundation option, especially considering that liquefiable soils were encountered to the maximum depth explored of 52.5 feet below existing grade. On this site, it is our opinion that a hybrid solution involving grading and a rigid mat foundations can be utilized, provided the client understands and accepts the risks involved.

To reduce the potential effects of dynamic settlement, the soils supporting the foundation should be reinforced with geotextile stabilization fabric and a layer of crushed gravel. Mat foundations distribute the structural loads over a wider area of the soil, and can be designed to be sufficiently rigid such that the foundation will act as an integral unit in the event of liquefaction. The foundation should be designed to accommodate the shear and bending stresses that would result from the anticipated differential dynamic settlement that could result due to liquefaction. A relatively low bearing value is also recommended, as is a design of the foundations to accommodate a span of lost bearing at any point within the foundation. The owner must recognize, however, that if liquefaction occurs as a result of a major earthquake, there will be a risk of movement and some damage to the structures and their foundations.

Another scenario would be if an earthquake slightly smaller than the earthquake with the design parameters analyzed were to occur. Such an earthquake could produce dynamic settlement of a lesser magnitude than that calculated using the previously mentioned design parameters.

Regardless of the magnitude of the causative earthquake, the result of the ensuing dynamic settlement would be that the structures would probably no longer be level. Our intent in recommending the mat foundations is to provide a system that would remain sufficiently intact such that re-leveling would be feasible. Re-leveling would most likely be accomplished by mud-jacking or pressure-grouting the foundations back to their original elevation.

The on-site soils are highly erodible. Stabilization of surface soils, particularly those disturbed during construction, by vegetation or other means *during and following construction* is essential to protect the site from erosion damage. Care should be taken to establish and maintain vegetation. To reduce the potential for disruption of drainage patterns and undermining of structures and other site improvements, all rodent activity should be aggressively controlled.

8.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

The following recommendations are for improvements constructed as described in the "Introduction" section of this report. If locations, elevations, structural loads, etc., change, the recommendations contained herein may require modification.

The "building areas" are defined as the areas within and extending to a minimum of 5 feet beyond the perimeter of each proposed structure. The building area for the apparatus structure should include the area that will be encompassed by the future third bay expansion, so that all earthwork for the structure is completed in a relatively uniform manner. The building areas include the footprint of any retaining walls, exterior stairways, breezeways, canopies, or other features that are attached to a structure and are expected to perform in a manner similar to it. The "foundation area" for a sitework retaining wall is defined as the entire foundation footprint, plus 3 feet to the front and rear. The "grading area" is the *entire* area to be graded; it includes all building and foundation areas, and all areas where surface improvements will be constructed or where fill will be placed.

Site Preparation

1. The existing ground surface in the grading area should be prepared for construction by removing all vegetation, large roots, debris, and all other deleterious material. Any existing utilities that will not be serving the new site should be removed, relocated, or properly abandoned. The appropriate method of utility abandonment will depend upon the type and depth of the utility. Recommendations for abandonment can be made as necessary.

2. Voids created by the removal of materials or utilities should be immediately 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

1. Following site preparation, the existing soils in the building areas should be removed to a minimum of 5 feet below lowest existing grade in the building area, or 5 feet below the lowest foundation element, whichever is deeper. The exposed surface should be scarified a minimum of 1 foot, moisture conditioned to, or to just above, optimum moisture content and recompacted.

2. Following recompaction of the excavated surfaces in the building areas, geotextile stabilization fabric (Caltrans Standard Specifications, Section 88-1.04, Type A, Nonwoven Rock Slope Protection Fabric) should be placed in the excavation. The fabric should be stretched as tightly as practicable, and held in place using pins or other methods recommended by the manufacturer. The fabric should also be overlapped on the sides as recommended by the manufacturer, and extended up the sidewalls of the excavation with a minimum of 10 feet of extra material above the excavation bottom.

3. A minimum of 2 feet of 0.75-inch by 1.5-inch crushed gravel should be placed over the fabric throughout the entire excavation. The gravel should be placed in at least 2 lifts, and it should be compacted in at least two directions using a vibrating steel-drum compactor.

4. Following placement of the gravel layer, the fabric that was extended up the sidewalls should be pulled over the top of the gravel and stretched as tightly as practicable. Additional fabric should be placed over the top of the gravel, with overlaps and pins (or other methods) to hold it in place as per the manufacturer. The intent is to completely encase the gravel layer in fabric.

5. Following placement of the geotextile/gravel layer in the building areas, previously removed soils and appropriate imported soils may be replaced over the geotextile-encased gravel in thin, moisture conditioned lifts and compacted to finish pad grade.

6. The first lift of fill above the geotextile should be placed by end-dumping and spreading ahead of the earthmoving equipment. No equipment should be allowed to travel over the geotextile until at least 6 inches of fill has been placed over it. The first lift of soil over the geotextile should be compacted using heavy rubber-tired equipment; subsequent lifts of fill may be compacted using static or vibratory sheepsfoot compactors.

7. As recommended in the "Utility Trench Backfill" section of this report, all utility lines below structures on this site should be placed in the zone of compacted backfill between the top layer of geotextile encasing the gravel and finish pad elevation. Utility lines should not extend into the gravel layer, and the geotextile should not be interrupted by placement of utilities. If a utility line must be placed at an elevation deeper than three feet below bottom-of-foundation elevation, then the overexcavation elevation

recommended in Paragraph 1 of this section should also be deepened, so that the utility line can be placed in the fill placed above the gravel/geotextile layer.

8. Following site preparation, the existing soils in sitework retaining wall foundation areas should be overexcavated to a minimum of 2 feet below existing grade or 2 feet below bottom-of-footing elevation, whichever is deeper. The exposed surface should be scarified a minimum of 1 foot, moisture conditioned to, or to just above, optimum moisture content and recompacted.

9. Following site preparation, the existing soils in areas to receive fill, pavement, flatwork or other improvements should be should be scarified a minimum of 1 foot, moisture conditioned to, or to just above, optimum moisture content and recompacted.

10. Following overexcavation and/or scarification as recommended above, previously removed site soils may be used for fill to finish grade throughout the grading area.

11. Imported soils to be used within the building area should be nonexpansive. Nonexpansive materials are defined as falling in the GW, GM, GC, SP, SW, SC, and SM categories per ASTM D 2487-06, and having an expansion index of 10 or less (per ASTM D 4829-07). In addition, nonexpansive materials used within the building area should have an angle of internal friction and a cohesion (per ASTM D 3080-04, modified for consolidated, undrained conditions) to meet the allowable bearing values and lateral parameters recommended in the "Foundations" section of this report.

12. Imported soil to be used within pavement improvement areas should have an R-value (ASTM D 2844-07) to meet the minimum sections recommended in the "Pavement Sections" portion of this report.

13. Proposed imported materials for building and pavement improvement areas should be reviewed by the soils engineer before being brought to the site, and on an intermittent basis during placement.

14. In landscape areas, the soil type and/or any amendments should be in accordance with the requirements of the architect/engineer.

15. Prior to placement of any fill, the underlying soil surface should be moistened to, or just above, optimum moisture content and no desiccation cracks should be present.

16. Voids created by dislodging rocks and/or debris during scarification should be backfilled and recompacted, and the dislodged materials should be removed from the work area.

17. All materials used as fill should be cleaned of all debris and any rocks larger than 3 inches in diameter. When 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.

18. 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. In areas to receive pavement improvements, the top 12 inches of subgrade and all AB should be compacted to a minimum of 95 percent of maximum dry density. Areas to receive pavement improvements should also be firm and unyielding when proofrolled with heavy rubber-tired equipment prior to paving.

19. Moisture conditioning refers to adjusting the soil moisture to, or just above, optimum moisture content prior to application of compactive effort. If the soils are overly moist so that instability occurs, or if the minimum recommended compaction cannot be readily achieved, drying the soil to optimum moisture content, or just above, may be necessary. Placement of gravel layers or geotextiles may also be necessary to help stabilize unstable soils. Additional overexcavation may also be recommended to correct unstable conditions or if soft or loose conditions are encountered during grading.

20. If soils near improvements such as foundations, flatwork, AC, curbs, etc. are otherwise disturbed, damage to those improvements may result. Soils that have cracked due to desiccation or are disturbed should be removed, moisture conditioned, and compacted.

Utility Trench Backfill

1. Utility trenches adjacent to foundations should not be excavated within the zone of foundation influence, as shown in Typical Detail A in Appendix D.

2. Utilities that will pass below a foundation should be placed with properly compacted utility trench backfill, and the foundation should be designed to span the trench.

3. All utility lines below the structures on this site should be placed in the zone of compacted backfill between the top layer of geotextile encasing the gravel and finish pad

elevation. Utility lines should not extend into the gravel layer, and the geotextile should not be interrupted by placement of utilities.

4. A select, noncorrosive, granular, easily compacted material should be used as bedding and shading immediately around utilities and as trench backfill above utilities. The site soils may be used as trench backfill above the bedding and shading material.

5. In general, trench backfill should be compacted a minimum of 90 percent of maximum dry density. A minimum of 95 percent of maximum dry density should be maintained in the backfill for all AB and the upper 12 inches of subgrade where trenches will cross below areas to receive pavement improvements. A minimum of 85 percent of maximum dry density will generally be sufficient where trench backfill is located in landscaped or other unimproved areas where settlement of trench backfill would not be detrimental.

6. For compaction of trench backfill soils by jetting to be successful, the water must have a free drainage path that will allow the water to dissipate very rapidly without causing erosion within the trench. Although the site soils were classified as being nonexpansive, their minor clay contents could cause drainage of trench backfill to occur slowly on this site. Therefore, jetting of utility trench backfill should only be attempted with extreme caution, and only for utilities such as joint trenches with multiple, closely spaced pipes and trenches for corrugated storm drains, where compaction by conventional means would be difficult. Any jetting operation should be subject to review by the soils engineer.

7. To reduce the potential for damage in the event of a seismic event, flexible, articulating connections should be provided for utilities that span between sitework areas and the building pads.

8. The local jurisdiction, utility companies and/or pipe manufacturers may have additional requirements for utility trench backfill that could take precedence over the above recommendations.

Foundations

1. Mat foundations should be utilized for support of the structures. The mat foundations can be either conventionally reinforced or post-tensioned as per the architect/engineer, and should be supported by firm soils recompacted per the "Grading" section of this report.

2. The mat foundation can be a "waffle" design, i.e., a structural slab that spans between grade beams with the grade beams providing primary shear and moment resistance, or it can be designed with a uniform thickness. The term "uniform thickness" refers to the design concept for the foundation, and differing thicknesses of the mat may be appropriate for where the foundation loads differ.

3. The mat should be designed using maximum allowable bearing capacities of 1,200 psf dead load and 1,500 psf dead plus live load, and a subgrade modulus (K_{30}) value of 250 pci (psi/inch). To accommodate the potential for sand boils in the event of liquefaction, the mat should also be designed to accommodate a 5-foot diameter of lost bearing at any point.

4. Maximum and differential settlement of a rigid mat foundation under static conditions are expected to be less than 3/4-inch, and less than 3/8-inch in 25 feet, respectively. Under the seismic conditions estimated to occur from the design earthquake parameters, as previously noted, the maximum settlement of the site with subsurface water at 14.5 feet is estimated to be approximately 8.5 inches, although our opinion is that a maximum settlement of 4 to 6 inches is more likely. Differential settlement of the site under the design seismic conditions is estimated to be a maximum of 4 inches in 25 feet. In our opinion, assuming the structures are supported by properly designed mat foundations bearing on recompacted soil/geotextile-encased gravel, total and differential of the structures should be a maximum of 2 inches.

5. Regardless of the reinforcement utilized for the mat foundation (either conventional or post-tensioned), the perimeter grade beams should penetrate a minimum of 18 inches below lowest adjacent grade.

6. Allowable bearing capacities may be increased by one-third when transient loads such as wind or seismicity are included. Foundations may be designed using the following site-specific design response acceleration parameters; please refer to the geologic hazards report for this project (ESP, 2009) for a discussion of the method used to develop these parameters:

Site Classification (CBC Table 1613A.5.2)	E
Site-Specific Modified Acceleration Parameters	
0.2 second period – S_{MS}	1.078g
1.0 second period – S_{M1}	1.422g
Site-Specific Design Spectral Response Acceleration Parameters	
0.2 second period - S_{DS}	0.719g
1.0 second period – S_{D1}	0.948g

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 refer to the "Retaining Walls" section of this report for criteria pertaining to lateral resistance.

8. 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 in the following paragraphs, however, the means of vapor protection, including the type and thickness of the vapor barrier, if specified, are left to the discretion of the architect/engineer.

9. Several recent studies, including those of 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.

10. Probably the next most effective option would be the use of vapor-inhibiting admixtures in the slab concrete mix and/or application of a sealer to the surface of the slab. This would also require special concrete mixes and placement procedures, depending upon the recommendations of the admixture or sealer manufacturer.

11. 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 nonexpansive soil layer. The barrier 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 nonexpansive soil, 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.

12. Where specified, vapor retarders should conform to ASTM Standard E 1745-97 (Reapproved 2004). This standard specifies properties for three performance classes; Class A, B and C. The appropriate class should be selected based on the sensitivity of floor coverings to moisture intrusion and the potential for damage to the vapor retarder during placement of slab reinforcement and concrete.

13. 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.

14. If the 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.

15. Foundation excavations should be observed by the soils engineer during excavation, and prior to placement of reinforcing steel. Soils in foundation excavations should be lightly moistened prior to concrete placement.

Exterior Flatwork

1. Exterior pedestrian flatwork should be constructed over compacted soil as per the "Grading" section of this report. 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 flatwork reinforcement should consist of No. 3 rebar placed at 24 inches on-center each way.

2. Where exterior flatwork will support vehicular traffic (i.e. at building or trash enclosure aprons), the flatwork should be cast over a minimum of 12 inches of Class 2 aggregate base. Design of flatwork to support vehicles may be based upon a modulus of subgrade reaction (K_{30}) of 400 pci (psi/in).

3. Exterior flatwork supporting vehicles that will be washed on a regular basis should be provided with sealed joints, to reduce the potential for migration of drainage into the subgrade.

4. A minimum of 4 inches of sand should be placed beneath exterior pedestrian flatwork. Exterior pedestrian flatwork should be constructed with frequent joints to allow articulation as the flatwork moves in response to seasonal temperature and soil moisture variations.

5. Where it is desired to maintain the elevation of flatwork at doorways and other areas, the flatwork should be doweled to the perimeter foundations of the structures, at a minimum, by No. 3 dowels lapped to the flatwork rebar at a 24-inch spacing. In other areas the flatwork may be doweled to the foundation or the flatwork may be allowed to "float free," at the discretion of the architect/engineer. Flatwork that is intended to float free should be separated from foundations by a felt joint or other means.

6. 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. Concrete materials, placement, and curing specifications should be at the direction of the architect/engineer; ACI 302.1R-04 is suggested as a resource for the architect/engineer in preparing such specifications.

Retaining Walls

1. Foundations for retaining walls rigidly attached to or forming part of a structure should be supported by firm soils and the gravel/geotextile layer, as per the "Grading" section of this report. Sitework retaining walls that are not rigidly attached to a structure should be supported by firm soils recompacted per the "Grading" section of this report.

2. Footings for all retaining walls should penetrate a minimum of 18 inches below lowest adjacent grade (not including any keyways).

3. Design of retaining walls should be based on the following parameters:

   Active equivalent fluid pressure (native soil or imported sand or gravel backfill) .... 35 pcf
   At-rest equivalent fluid pressure (native soil or imported sand or gravel backfill) .... 50 pcf
   Passive equivalent fluid pressure .... 400 pcf
   Maximum toe pressure .... 1,500 psf
   Coefficient of sliding friction 0.45

4. To accommodate seismic loads on retaining walls, a uniformly distributed pressure of 64 psf should be used for the active case, or 95 psf should be used for the at-rest case. The pressure due to seismicity may be analyzed like any other uniform load with the resultant of the uniformly distributed pressure acting at one-half the height of the retaining wall.

5. No surcharges are taken into consideration in the values indicated in Paragraphs 3 and 4. The maximum toe pressure in Paragraph 3 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. The above pressures are applicable to a retained surface that is horizontal at the top of the wall. Walls having a retained surface that slopes upward from the top of 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. It is assumed that retaining wall heights will not exceed 5 feet.

7. Long-term settlement of properly compacted retaining wall backfill should be assumed to be 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 estimated settlement.

8. The active and at-rest pressures contained in Paragraph 3 are for fully drained conditions; therefore, 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 zone encasing the pipe should have a width of approximately 1 foot and should

extend upward to 1 foot from the top of the backfill of any wall that does not have a slab-on-grade or pavement section abutting the top of the wall. The upper 1 foot of backfill should consist of native soils to reduce the flow of surface drainage into the wall drain system. If a slab-on-grade or pavement section abuts the top of the wall, the gravel zone should extend to the layer below the slab-on-grade, or to the AB below the pavement section. To reduce infiltration of the backfill soil into the gravel, a permeable synthetic filter fabric, conforming to Caltrans Section 88-1.03 for Underdrains, should be placed between the two. Manufactured synthetic drains, such as Miradrain and Enkadrain, are acceptable alternatives to the use of gravel, provided they are installed in accordance with the manufacturer's recommendations. Where seepage can be properly controlled, the perforated pipe may be omitted in lieu of weep holes on maximum 4-foot centers placed at the lowest point in the wall that will still provide drainage. A filter fabric as described above should be placed between the weep holes and the drain gravel.

9. The final foot of all retaining wall backfill should consist of native soil, to reduce the potential for surface drainage to enter the retaining wall drain system. If a slab-on-grade or pavement will abut the top of the wall, the backfill should extend to the sand or nonexpansive material layer below the slab-on-grade, or to the AB below the pavement section.

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

11. Retaining walls by their nature are flexible structures, and surface treatments on walls often crack. Where walls are to be plastered or will otherwise have a finish surface applied, the flexibility should be considered in determining the suitability of the surfacing material, spacing of horizontal and vertical 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 Sections

1. The following flexible pavement sections are based on the tested R-value of 23 for the site soils, and assumed Traffic Indices (TIs) of 5.0 through 8.0. Determination of the appropriate TI for specific areas of the project is left to the engineer. The AC sections were calculated in accordance with the *Caltrans Highway Design Manual*, which

incorporates a safety factor. The calculated base and AC thickness are for compacted material. Normal Caltrans construction tolerances should apply.

Traffic Index	AC	Class 2 Aggregate Base
5.0	2.75"	7.0"
5.5	3.00"	8.5"
6.0	3.25"	9.5"
6.5	3.75"	10.0"
7.0	4.00"	11.0"

2. The upper 12 inches of subgrade and all AB below pavement should be compacted to a minimum of 95 percent of maximum dry density.

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

4. Finished 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 as it could infiltrate into the aggregate base, nonexpansive material and subgrade, causing premature pavement deterioration.

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

6. Where trucks will maneuver at slow speeds on the site, PCC pavement should be provided. PCC pavement should be underlain by a minimum of 12 inches of Class 2 AB (Caltrans Standard Specifications), compacted to a minimum of 95 percent of maximum dry density. A modulus of subgrade reaction (K_{30}) of 400 psi/inch (pci) may be used for design of PCC pavement. Design of PCC pavement thickness and reinforcement are left to the architect/engineer.

Drainage Around Improvements

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 of 2 percent grade for a minimum distance of 5 feet. 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. Exterior pedestrian flatwork should be sloped to freely drain toward appropriate drainage facilities.

2. To reduce the potential for planter drainage to gain access below foundations, any raised planter boxes adjacent to foundations should be installed with drains and sealed sides and bottoms. Drains should also be provided for areas adjacent to structures that would not otherwise freely drain.

3. The eaves of the structures 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 drainage outlets unless discharge is to an AC or PCC surface.

4. The site soils are erodible. To reduce erosion damage, it is essential to stabilize surface soils, particularly those disturbed during construction, 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. Maintenance of drainage and other improvements is critical to the long-term stability of the site and the integrity of the structures. Site improvements should be inspected and maintained on a regular basis.

6. All exterior drains and retaining wall drains and their outlets should be cleaned and repaired as necessary to maintain free-flowing conditions.

7. Vegetation and erosion matting (where utilized) should be maintained and repaired or augmented as needed. Irrigation systems should be adjusted and/or repaired so that soils around structures and on slopes are maintained at a relatively uniform year-round moisture content, and are neither over-watered nor allowed to dry and desiccate.

8. To reduce the potential for disruption of drainage patterns and undermining of structures, retaining walls, pavement, fill areas, etc., all rodent activity should be aggressively controlled.

Observation and Testing

1. It must be recognized that the recommendations contained in this report are based on a limited number of borings 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, or just above, optimum moisture content, prior to application of compactive effort.

4. The standard tests used to define maximum dry density and field density should be ASTM D 1557-07 and ASTM D 6938-07b, 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 final grading, utility, and foundation plans
    - Professional observation during grading, foundation excavation, and trench backfill
    - Oversight of special inspection during utility trench backfill
    - Oversight of special inspection during grading

6. Backfill of excavations and trenches should be considered to fall under Section 1704.7 "Soils" of the CBC. Special inspection of backfill should be provided as per Section 1704.7 and Table 1704.7 of the CBC. The special inspector should be under the direction of the soils engineer.

7. In our opinion, considering the relatively minor nature of the earthwork anticipated for this site, the following operations *should not* require *continuous* special inspection; *periodic* special inspection should suffice, subject to approval by the Building Official:

- Verification of use of proper materials, densities and lift thicknesses during placement and compaction of controlled fill

8. In accordance with CBC Section 1803.5, the following locations and frequency of tests are recommended. At a minimum, the special inspector should verify that:

   - A minimum of three compaction tests are taken in the building area at the bottom of the overexcavation.
   - A minimum of three compaction tests are taken for every 1.5 feet of fill placed over the gravel/geotextile layer, for every 2,000 square feet of building area or fraction thereof.
   - A minimum of one compaction test is taken in each site utility trench for every 2 feet above the pipe, for every 75 linear feet of trench, or fraction thereof.
   - A minimum of two compaction tests are taken within each retaining wall area for every 1.5 feet of backfill or recompacted soil.
   - A minimum of three compaction tests are taken at subgrade and aggregate base grade, for every 2,000 square feet of pavement area or fraction thereof.

   The soil engineer may elect to increase or decrease the frequency of testing at the time of construction, depending on the actual soil conditions exposed, the compaction equipment being utilized, the initial test results, or other factors.

9. A program of quality assurance should be developed prior to beginning construction. At a minimum, the program should include all geotechnical items shown on the testing and inspection schedule of the approved plans. It should also include any additional inspection items required by the architect/engineer or the governing jurisdiction. These items should be discussed at a preconstruction meeting among a representative of the owner, the jurisdiction, the soils engineer, the architect/engineer, and contractors. The soils engineer should be notified at least 48 hours prior to beginning construction operations.

9.0 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 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 of this report. 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 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.

If changes with respect to the 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, this firm 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 report are based upon the geotechnical 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 peer or jurisdiction reviews, or 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.

TECHNICAL REFERENCES

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APPENDIX A

Boring Location Map
Boring Log Legend
Boring Logs

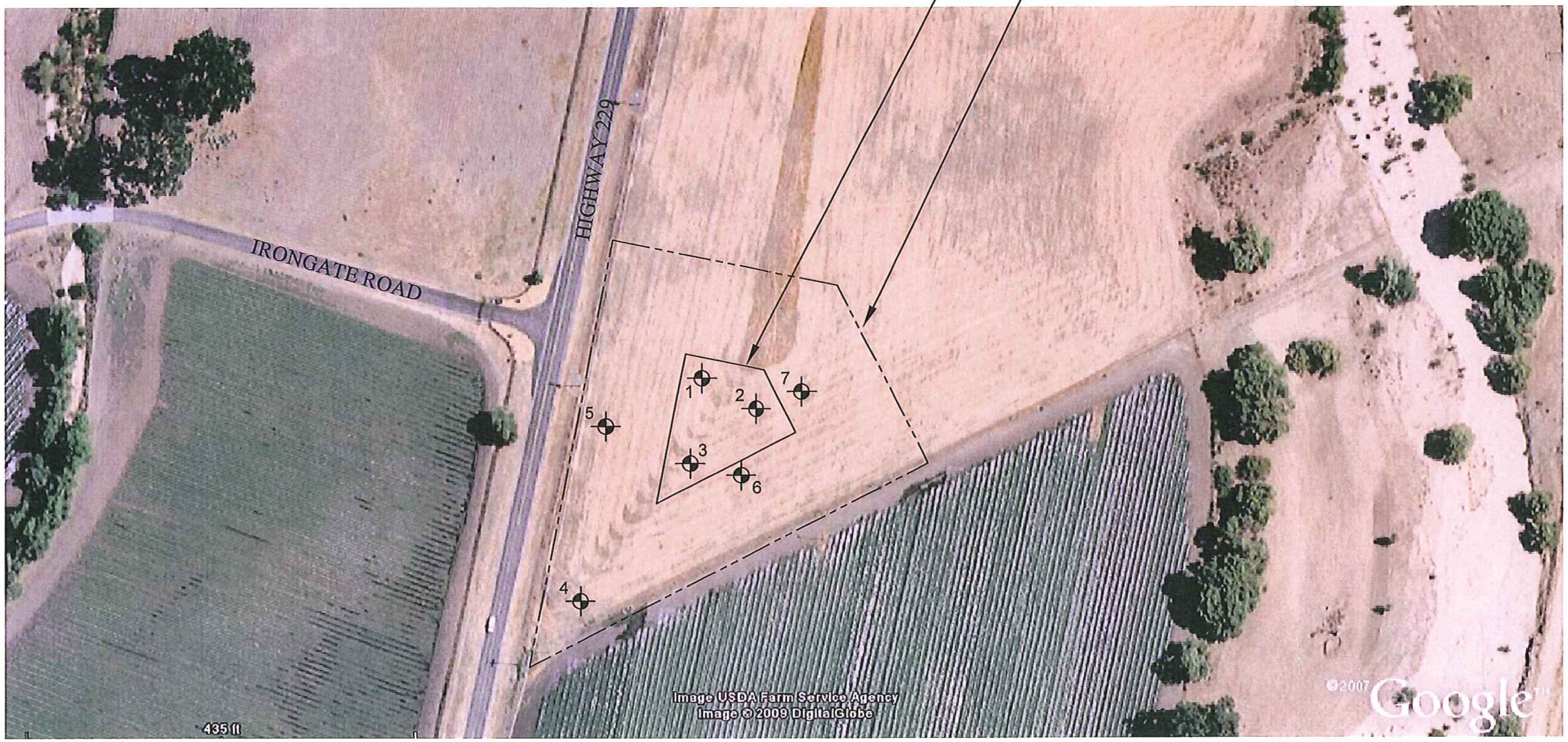

Image provided by Google Earth, 2007

N

LEGEND

7 Boring Location (Approx.)

NOT TO SCALE

Earth Systems Pacific

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April 15, 2009 KM SL-15969-SA

BORING LOCATION MAP

SAN LUIS OBISPO COUNTY FIRE STATION 43

Highway 229 at Irongate Road
Creston, California

SAN LUIS OBISPO COUNTY FIRE STATION 43–041509Borings

Earth Systems Pacific

BORING LOG LEGEND

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

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Boring No. 1
PAGE 1 OF 1
JOB NO.: SL-15969-SA
DATE: 4/3/09

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

SAN LUIS OBISPO COUNTY
FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SC		CLAYEY SAND: dark brown, loose, moist (alluvium)					
3				3.0–4.5	Ring Sample	110.6	14.2	1 3 5
5				5.0–6.5	Ring Sample	114.0	9.9	2 7 8
7			brown					
10				10.0–11.5	SPT			2 2 3
15				15.0–16.5	SPT			3 6 5
15.5	SW		WELL GRADED SAND: light brown, medium dense, moist, trace coarse gravel					
16.5			End of Boring @ 16.5' 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.

Earth Systems Pacific

Boring No. 2

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

JOB NO.: SL-15969-SA
DATE: 4/3/09

SAN LUIS OBISPO COUNTY FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SC		CLAYEY SAND: dark brown, loose, moist (alluvium)					
1				1.0–2.5	Ring Sample	118.0	9.1	3 6 8
5				5.0–6.5	SPT			2 4 5
8			brown					
10				10.0–11.5	SPT			2 3 5
14.5	SW		WELL GRADED SAND: light brown, medium dense, moist					
15				15.0–16.5	SPT			4 6 8
20				20.0–21.5	SPT			4 6 7
25			wet	25.0–26.5	SPT			3 7 7

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

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

Boring No. 2

JOB NO.: SL-15969-SA
DATE: 4/3/09

SAN LUIS OBISPO COUNTY FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
27	SW		WELL GRADED SAND: as above					
30			loose, 5 to 15% coarse gravel	30.0–31.5	SPT			2, 2, 6
35			medium dense	35.0–36.5	SPT			1, 4, 7
40				40.0–41.5	SPT			1, 1, 1
45				45.0–46.5	SPT			3, 6, 15
50				50.0–51.5	SPT			2, 3, 11
51.5			End of Boring @ 51.5' Subsurface water encountered @ 25.0'					

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.

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Boring No. 3

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

PAGE 1 OF 1
JOB NO.: SL-15969-SA
DATE: 4/3/09

SAN LUIS OBISPO COUNTY
FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

DEPTH (feet)	USCS CLASS	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SC	CLAYEY SAND: dark brown, loose, moist (alluvium)					
2			2.0–3.5	Ring Sample	117.6	8.4	4 7 9
5			5.0–6.5	Ring Sample	110.4	18.0	3 3 5
9		brown					
10			10.0–11.5	Ring Sample	115.5	2.7	4 7 9
14	SW	WELL GRADED SAND: light brown, loose, moist, trace coarse gravel					
15			15.0–16.5	SPT			2 2 3
15.5		thin layers of fine grained clayey sand					
20		medium dense	20.0–21.5	SPT			5 9 10
21.5		End of Boring @ 21.5' 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.

Earth Systems Pacific

Boring No. 4

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

JOB NO.: SL-15969-SA
DATE: 4/3/09

SAN LUIS OBISPO COUNTY
FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0–5	SC		CLAYEY SAND: dark brown, loose, moist (alluvium)	1.0–2.5	Ring Sample	112.2	11.8	4, 7, 9
				3.5–5.0	SPT			1, 1, 2
				3.0–5.0	Grab Sample			
5			End of Boring @ 5.0' No subsurface water encountered					

Depth scale: 0 through 26 feet.

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.

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Boring No. 5

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

PAGE 1 OF 1
JOB NO.: SL-15969-SA
DATE: 4/3/09

SAN LUIS OBISPO COUNTY
FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA: INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0–5	SC		CLAYEY SAND: dark brown, loose, moist (alluvium)	2.0–3.5	Ring Sample	117.1	7.4	3 7 6
				3.5–5.0	SPT			1 2 4
5			End of Boring @ 5.0' No subsurface water encountered					

Depth scale: 0 to 26 feet.

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. 6

PAGE 1 OF 1
JOB NO.: SL-15969-SA
DATE: 4/3/09

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

SAN LUIS OBISPO COUNTY
FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SC		CLAYEY SAND: dark brown, medium dense, moist (alluvium)	1.0–2.5	Ring Sample	118.5	9.3	6 10 10
3			loose	3.5–5.0	SPT			5 6 6
5			End of Boring @ 5.0' 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.

Earth Systems Pacific

Boring No. 7

PAGE 1 OF 1
JOB NO.: SL-15969-SA
DATE: 4/3/09

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

SAN LUIS OBISPO COUNTY
FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA INTERVAL (feet)	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0–5	SC		CLAYEY SAND: dark brown, loose, moist (alluvium)	1.5–3.0	Ring Sample	111.7	12.7	3 4 4
				0.0–3.0	Grab Sample			
				3.5–5.0	SPT			2 4 6
5			End of Boring @ 5.0' 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.

APPENDIX B

Laboratory Test Results

San Luis Obispo County Fire Station 43
Creston, California

SL-15969-SA

BULK DENSITY TEST RESULTS

ASTM D 2937-04 (modified for ring liners)

April 21, 2009

BORING NO.	DEPTH feet	MOISTURE CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
1	4.0 - 4.5	14.2	126.3	110.6
1	6.0 - 6.5	9.9	125.3	114.0
2	2.0 - 2.5	9.1	128.7	118.0
3	3.0 - 3.5	8.4	127.5	117.6
3	6.0 - 6.5	18.0	130.3	110.4
3	11.0 - 11.5	7.7	124.4	115.5
4	2.0 - 2.5	11.8	125.4	112.2
5	3.0 - 3.5	7.4	125.8	117.1
6	2.0 - 2.5	9.3	129.4	118.5
7	2.5 - 3.0	12.7	125.9	111.7

EXPANSION INDEX TEST RESULTS

ASTM D 4829-07

BORING NO.	DEPTH feet	EXPANSION INDEX
7	0.0 - 3.0	6

San Luis Obispo County Fire Station 43
Creston, California

SL-15969-SA

MOISTURE-DENSITY COMPACTION TEST

ASTM D 1557-07 (Modified)

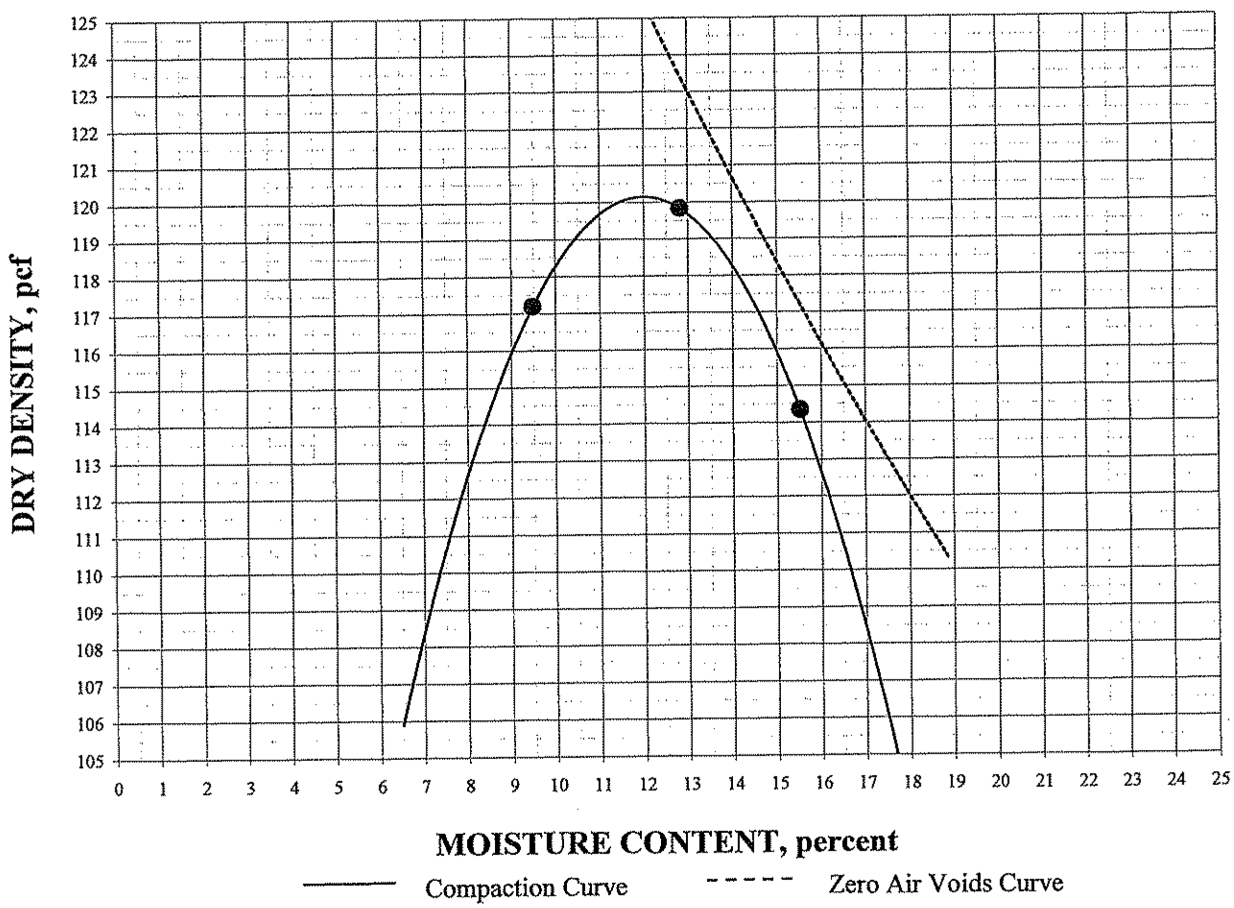
PROCEDURE USED: A
PREPARATION METHOD: Moist
RAMMER TYPE: Mechanical
SPECIFIC GRAVITY: 2.65 (assumed)

April 20, 2005
Boring #7 @ 0.0 - 3.0'
Dark Brown Clayey Sand (SC)

SIEVE DATA:

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

MAXIMUM DRY DENSITY: 120.2 pcf
OPTIMUM MOISTURE: 12.0%



San Luis Obispo County Fire Station 43
Creston, California

SL-15969-SA

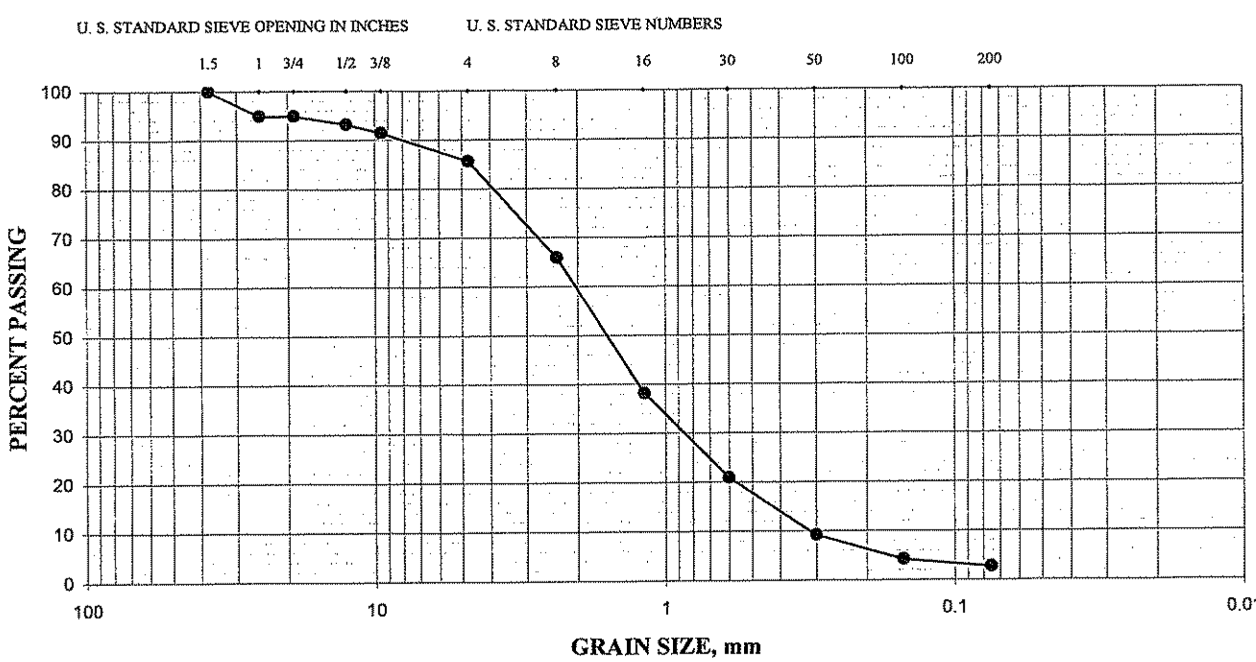
PARTICLE SIZE ANALYSIS

ASTM D 422-07; D 1140-06

Boring #2 @ 30.0 - 31.5'
Well-Graded Sand (SW)
Cu = 6.5; Cc = 1.1

April 21, 2009

Sieve size	% Retained	% Passing
1.5" (37.5-mm)	0	100
1" (25-mm)	5	95
3/4" (19-mm)	5	95
1/2" (12.5-mm)	7	93
3/8" (9.5-mm)	9	91
#4 (4.75-mm)	14	86
#8 (2.36-mm)	34	66
#16 (1.18-mm)	62	38
#30 (600-µm)	79	21
#50 (300-µm)	91	9
#100 (150-µm)	96	4
#200 (75-µm)	97	3



San Luis Obispo County Fire Station 43
Creston, California

SL-15969-SA

CONSOLIDATION TEST

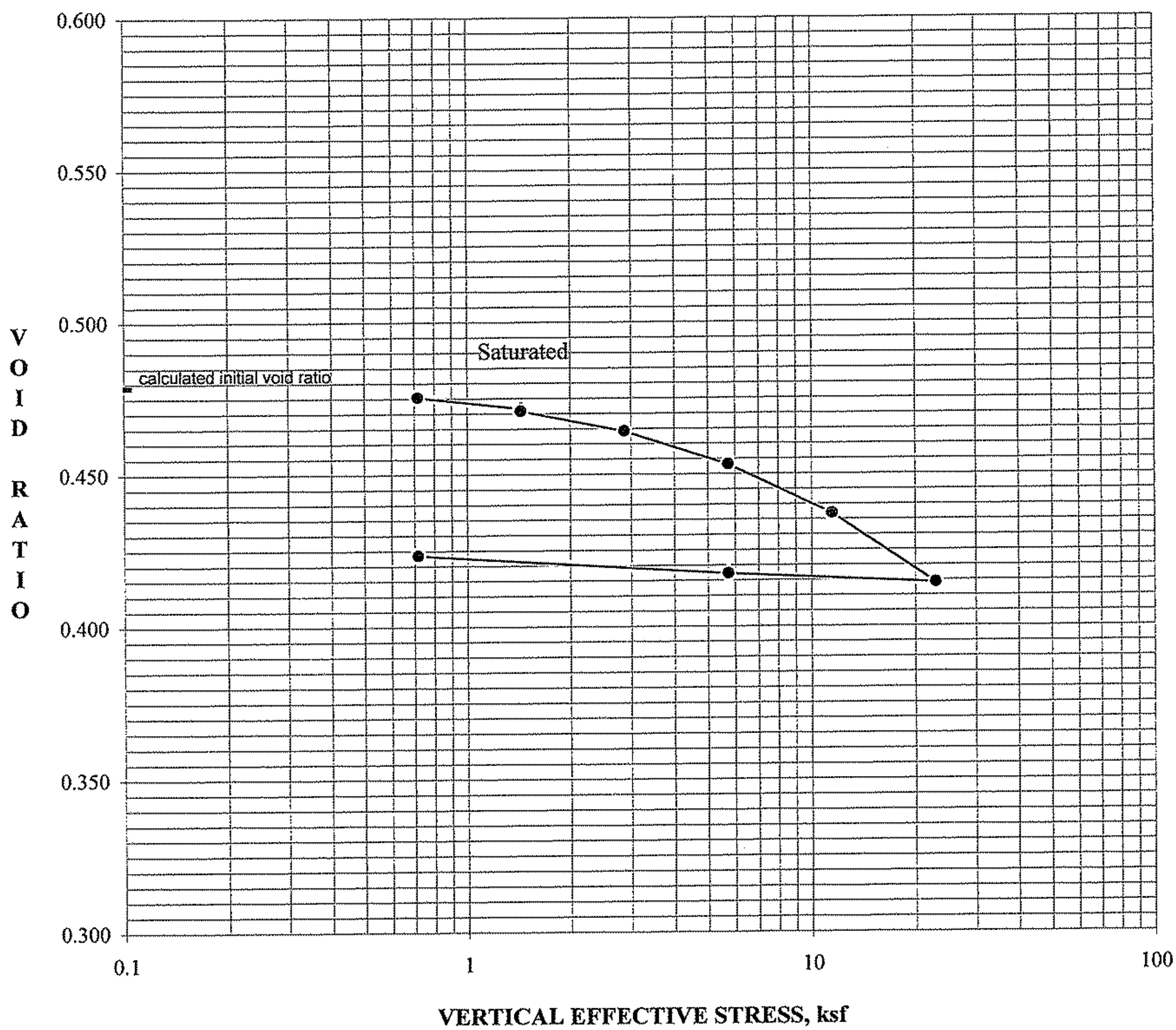
ASTM D 2435-04

April 21, 2009

Boring #1 @ 6.0 - 6.5'
Clayey Sand (SC)
Ring Sample

DRY DENSITY: 111.9 pcf
MOISTURE CONTENT: 9.9%
SPECIFIC GRAVITY: 2.65 (assumed)
INITIAL VOID RATIO: 0.479

VOID RATIO vs. NORMAL PRESSURE DIAGRAM



San Luis Obispo County Fire Station 43
Creston, California

SL-15969-SA

CONSOLIDATION TEST

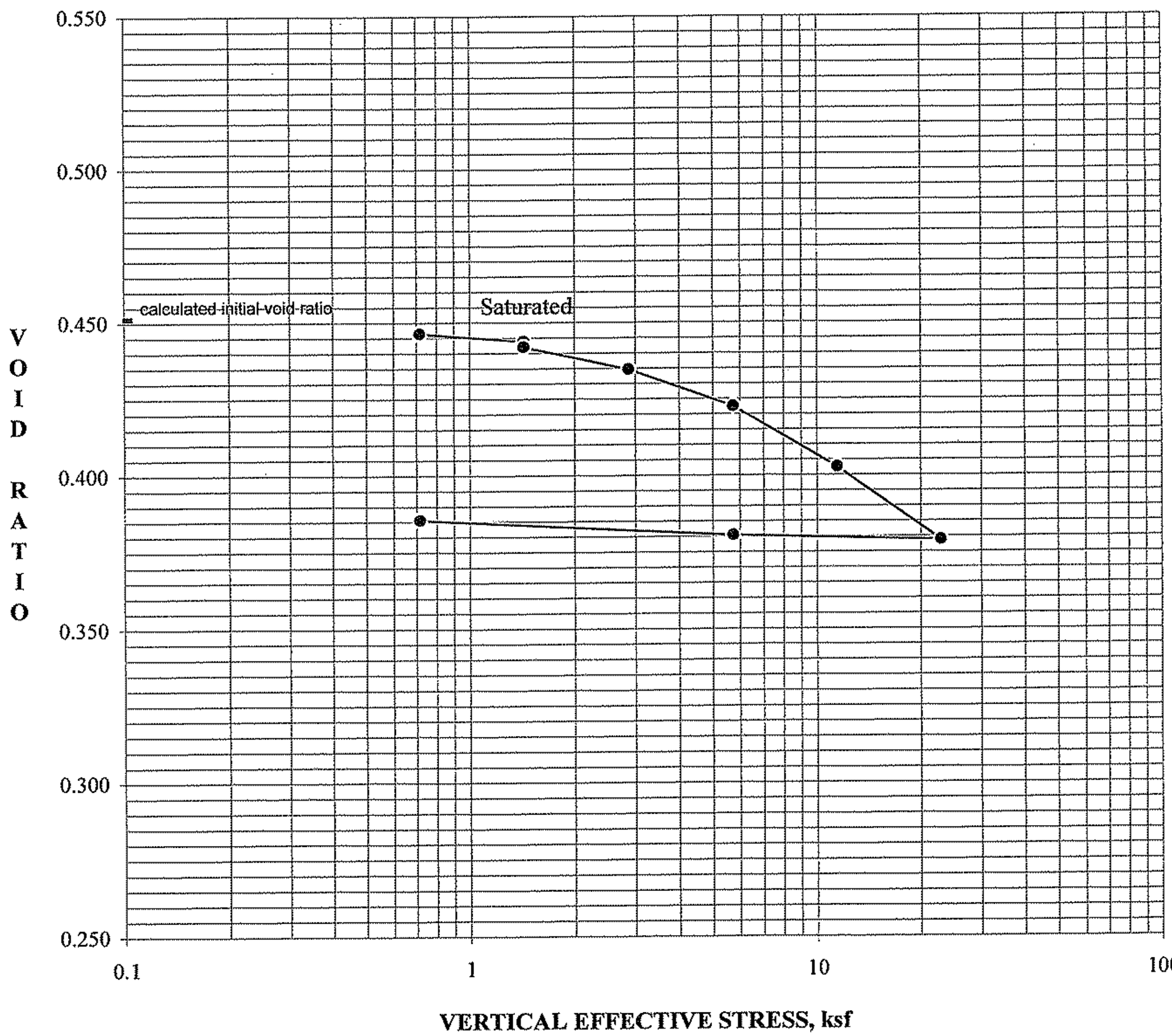
ASTM D 2435-04

April 20, 2005

Boring #3 @ 3.0 - 3.5'
Clayey Sand (SC)
Ring Sample

DRY DENSITY: 114.0 pcf
MOISTURE CONTENT: 8.4%
SPECIFIC GRAVITY: 2.65 (assumed)
INITIAL VOID RATIO: 0.451

VOID RATIO vs. NORMAL PRESSURE DIAGRAM



San Luis Obispo County Fire Station 43
Creston, California

SL-15969-SA

DIRECT SHEAR

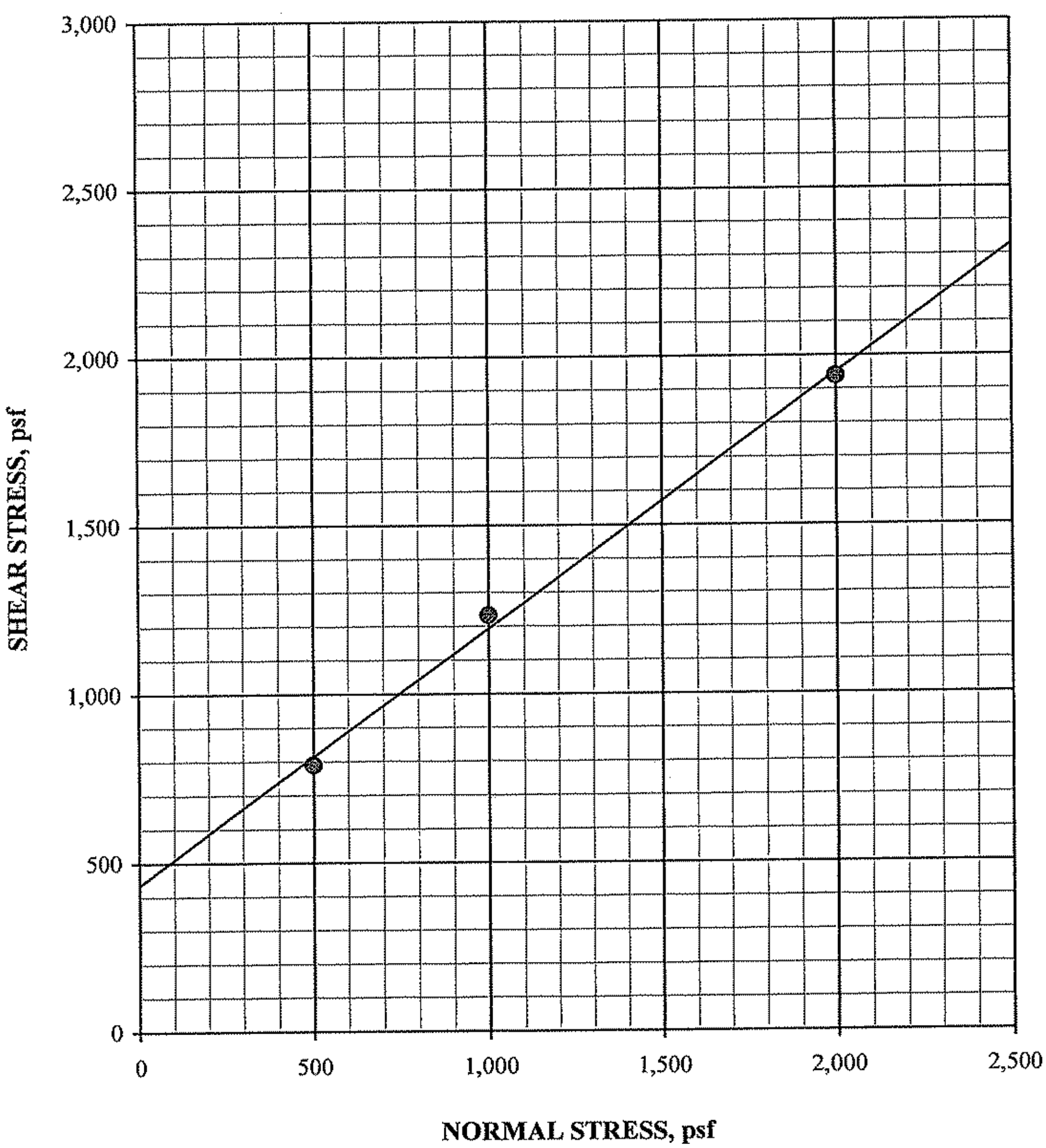
ASTM D 3080-04 (modified for consolidated, undrained conditions)

April 21, 2009

Boring #7 @ 0.0 - 3.0'
Clayey Sand (SC)
Compacted to 90% RC, saturated

INITIAL DRY DENSITY: 108.1 pcf
INITIAL MOISTURE CONTENT: 12.0 %
PEAK SHEAR ANGLE (Ø): 37°
COHESION (C): 436 psf

SHEAR vs. NORMAL STRESS



San Luis Obispo County Fire Station 43
Creston, California

SL-15969-SA

DIRECT SHEAR continued

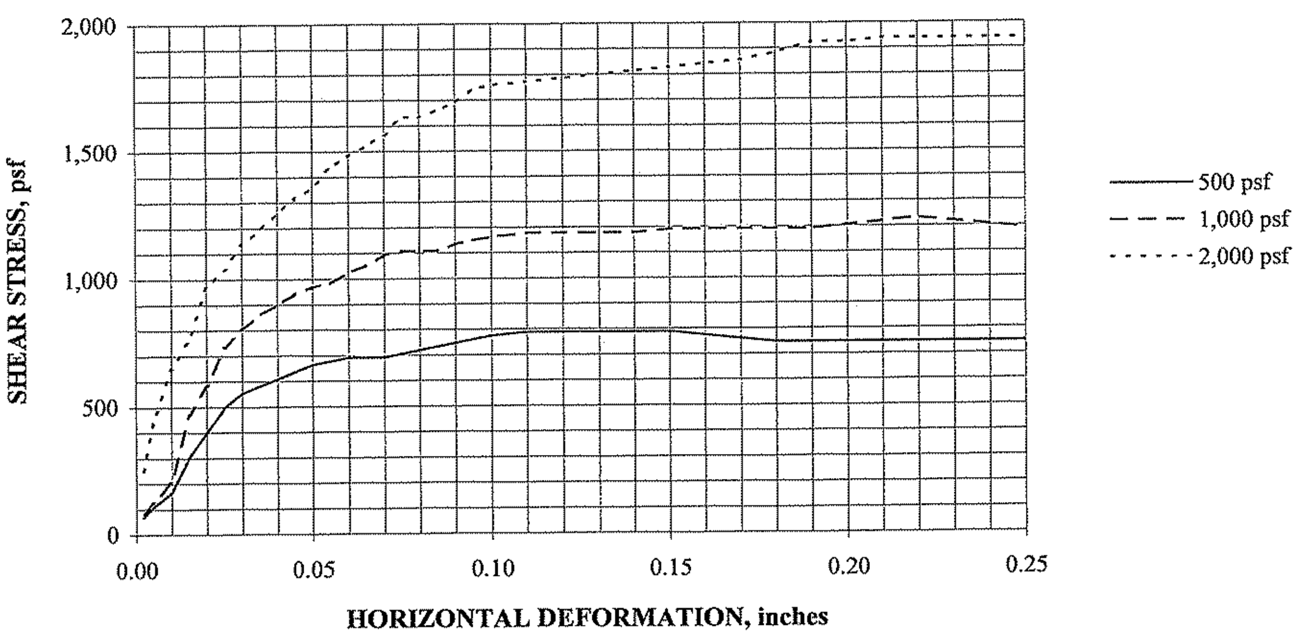
ASTM D 3080-04 (modified for consolidated, undrained conditions)

Boring #7 @ 0.0 - 3.0'
Clayey Sand (SC)
Compacted to 90% RC, saturated

April 21, 2009

SPECIFIC GRAVITY: 2.65 (assumed)

SAMPLE NO.:	1	2	3	AVERAGE
INITIAL				
WATER CONTENT, %	12.0	12.0	12.0	12.0
DRY DENSITY, pcf	108.1	108.1	108.1	108.1
SATURATION, %	60.1	60.1	60.1	60.1
VOID RATIO	0.529	0.529	0.529	0.529
DIAMETER, inches	2.375	2.375	2.375	
HEIGHT, inches	1.00	1.00	1.00	
AT TEST				
WATER CONTENT, %	19.8	19.3	17.9	
DRY DENSITY, pcf	108.5	110.0	113.5	
SATURATION, %	100.0	100.0	100.0	
VOID RATIO	0.525	0.503	0.457	
HEIGHT, inches	1.00	0.98	0.95	



San Luis Obispo County Fire Station 43
Creston, California

SL-15969-SA

RESISTANCE '*R*' VALUE AND EXPANSION PRESSURE

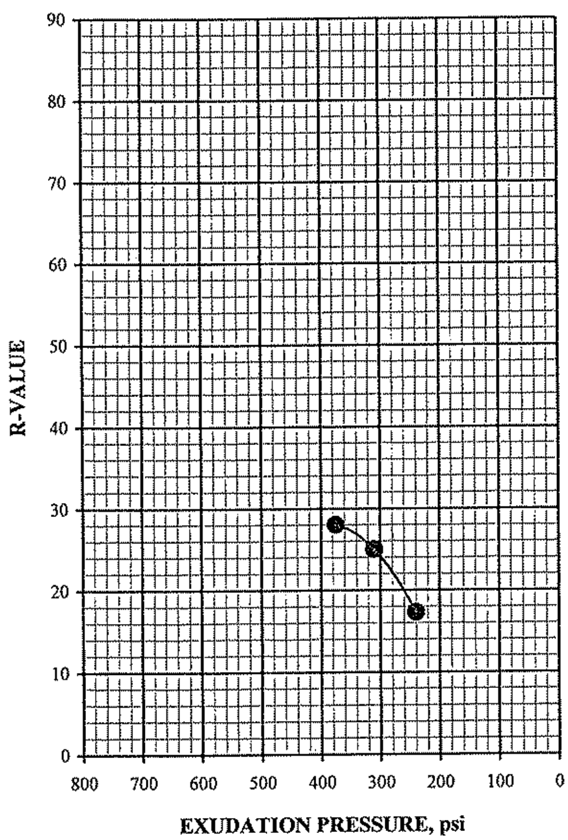
ASTM D 2844-07

April 21, 2009

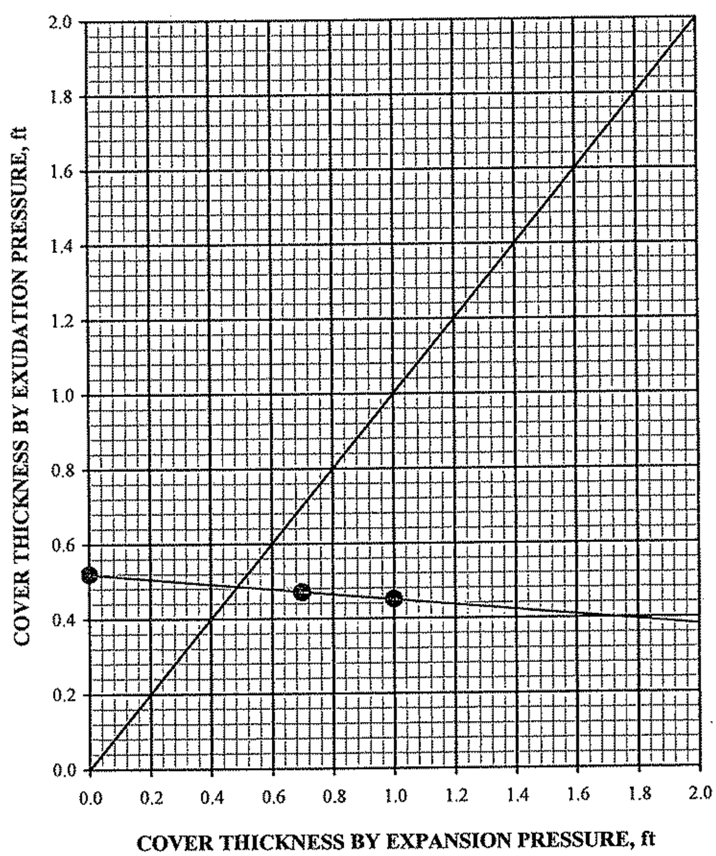
Boring #4 @ 3.0 - 5.0'
Dark Brown Clayey Sand (SC)
Specified Traffic Index: 5.0

Dry Density @ 300 psi Exudation Pressure: 119.6-pcf
%Moisture @ 300 psi Exudation Pressure: 9.6%
R-Value - Exudation Pressure: 24
R-Value - Expansion Pressure: 23
R-Value @ Equilibrium: 23

EXUDATION PRESSURE CHART



EXPANSION PRESSURE CHART



APPENDIX C

Liquefaction Analysis Spreadsheets

Project: SLO County Fire Station 43, Creston, CA
Job No: SL-15969-SA
Date: 4/27/2009
Boring: B-2 Data Set: 1

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

EARTHQUAKE INFORMATION:

Magnitude:	7.78	7.5
PGA, g:	0.33	0.36
MSF:	0.91	
GWT:	25.0	feet
Calc GWT:	25.0	feet
Remediate to:	0.0	feet

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E):	1.25	
Drive Rod Corr. (C_R):	1	Default
Rod Length above ground (feet):	3.0	
Borehole Dia. Corr. (C_B):	1.15	
Sampler Liner Correction for SPT?:	1	Yes
Cal Mod/ SPT Ratio:	0.63	

Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
26.5	8.2

Required SF: 1.30
Threshold Acceler., g: 0.06 **Minimum Calculated SF:** 0.19

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	K_σ	M = 7.5 Available CRR	M =7.5 Induced CSR*	Liquefac. Safety Factor	Post FC Adj. $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)
								0.000																	
2.5	14	9	1	115	20	1.5	4.5	0.086	0.086	1.00	1.70	0.75	1.00	16.2	48	4.9	21.1	1.00	0.229	0.235	Non-Liq.	4.9	21.1	0.02	0.01
7.5		9	1	115	20	5.5	8.5	0.316	0.316	0.99	1.70	0.75	1.20	19.8	53	5.2	24.9	1.00	0.282	0.233	Non-Liq.	5.2	24.9	0.03	0.02
14.5		8	1	115	20	10.5	13.5	0.604	0.604	0.98	1.32	0.77	1.14	13.4	44	4.7	18.1	1.00	0.195	0.231	Non-Liq.	4.7	18.1	0.07	0.06
17.5		14	1	115	5	15.5	18.5	0.891	0.891	0.97	1.09	0.87	1.23	23.5	58	0.0	23.5	1.00	0.260	0.228	Non-Liq.	0.0	23.5	0.05	0.02
25.0		13	1	115	5	20.5	23.5	1.179	1.179	0.96	0.95	0.94	1.20	19.9	53	0.0	19.9	0.97	0.216	0.233	Non-Liq.	0.0	19.9	0.08	0.07
27.5		14	1	115	5	25.5	28.5	1.466	1.451	0.94	0.85	0.99	1.20	20.4	54	0.0	20.4	0.91	0.221	0.246	0.90	0.0	20.4	1.15	0.34
32.5		8	1	115	5	30.5	33.5	1.754	1.582	0.92	0.82	1.00	1.11	10.5	39	0.0	10.5	0.92	0.113	0.260	0.44	0.0	10.5	2.53	1.52
37.5		11	1	115	5	35.5	38.5	2.041	1.714	0.89	0.79	1.00	1.15	14.3	45	0.0	14.3	0.91	0.155	0.274	0.56	0.0	14.3	1.97	1.18
42.5		2	1	115	5	40.5	43.5	2.329	1.845	0.85	0.76	1.00	1.10	2.4	18	0.0	2.4	0.89	0.053	0.281	0.19	0.0	2.4	6.89	4.13
47.5		21	1	115	5	45.5	48.5	2.616	1.977	0.80	0.73	1.00	1.27	27.9	63	0.0	27.9	0.83	0.341	0.301	1.14	0.0	27.9	0.10	0.06
51.5		14	1	115	5	50.5	53.5	2.904	2.108	0.75	0.71	1.00	1.17	16.7	49	0.0	16.7	0.87	0.180	0.279	0.65	0.0	16.7	1.71	0.82

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April 27, 2009 KM SL-15969-SA

LIQUEFACTION ANALYSIS - BORING 2
GROUNDWATER AT 25.0'

SAN LUIS OBISPO COUNTY FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

SAN LUIS OBISPO COUNTY FIRE STATION 43-042709LiquefactionB2,GW25

Project: SLO County Fire Station 43, Creston, CA
Job No: SL-15969-SA
Date: 4/27/2009
Boring: B-2 Data Set: 1

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

EARTHQUAKE INFORMATION:

Magnitude:	7.78	7.5
PGA, g:	0.33	0.36
MSF:	0.91	
GWT:	14.5	feet
Calc GWT:	14.5	feet
Remediate to:	0.0	feet

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E):	1.25	
Drive Rod Corr. (C_R):	1	Default
Rod Length above ground (feet):	3.0	
Borehole Dia. Corr. (C_B):	1.15	
Sampler Liner Correction for SPT?:	1	Yes
Cal Mod/ SPT Ratio:	0.63	

Threshold Acceler., g: 0.05

Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
32	8.5

Required SF: 1.30
Minimum Calculated SF: 0.16

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)
2.5	14	9	1	115	20	1.5	4.5
7.5		9	1	115	20	5.5	8.5
14.5		8	1	115	20	10.5	13.5
17.5		14	1	115	5	15.5	18.5
25.0		13	1	115	5	20.5	23.5
27.5		14	1	115	5	25.5	28.5
32.5		8	1	115	5	30.5	33.5
37.5		11	1	115	5	35.5	38.5
42.5		2	1	115	5	40.5	43.5
47.5		21	1	115	5	45.5	48.5
51.5		14	1	115	5	50.5	53.5

Base Depth (feet)	Tot. Stress at SPT po (tsf)	Eff. Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	K_σ	M = 7.5 Available CRR	M = 7.5 Induced CSR*	Liquefac. Safety Factor	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	Induced Subsidence (in.)
	0.000																	
2.5	0.086	0.086	1.00	1.70	0.75	1.00	16.2	48	4.9	21.1	1.00	0.229	0.235	Non-Liq.	4.9	21.1	0.02	0.01
7.5	0.316	0.316	0.99	1.70	0.75	1.20	19.8	53	5.2	24.9	1.00	0.282	0.233	Non-Liq.	5.2	24.9	0.03	0.02
14.5	0.604	0.604	0.98	1.32	0.77	1.14	13.4	44	4.7	18.1	1.00	0.195	0.231	Non-Liq.	4.7	18.1	0.07	0.06
17.5	0.891	0.860	0.97	1.11	0.87	1.23	24.0	59	0.0	24.0	1.00	0.268	0.236	1.13	0.0	24.0	0.14	0.05
25.0	1.179	0.992	0.96	1.03	0.94	1.22	22.1	56	0.0	22.1	1.00	0.241	0.268	0.90	0.0	22.1	1.09	0.98
27.5	1.466	1.123	0.94	0.97	0.99	1.23	23.8	58	0.0	23.8	0.98	0.264	0.294	0.90	0.0	23.8	1.04	0.31
32.5	1.754	1.255	0.92	0.92	1.00	1.13	11.9	41	0.0	11.9	0.97	0.129	0.313	0.41	0.0	11.9	2.32	1.39
37.5	2.041	1.386	0.89	0.87	1.00	1.17	16.1	48	0.0	16.1	0.95	0.174	0.325	0.54	0.0	16.1	1.82	1.09
42.5	2.329	1.518	0.85	0.83	1.00	1.10	2.6	19	0.0	2.6	0.93	0.054	0.329	0.16	0.0	2.6	6.34	3.80
47.5	2.616	1.649	0.80	0.80	1.00	1.29	31.2	67	0.0	31.2	0.88	1.200	0.341	3.52	0.0	31.2	0.00	0.00
51.5	2.904	1.781	0.75	0.77	1.00	1.19	18.4	51	0.0	18.4	0.86	0.199	0.336	0.59	0.0	18.4	1.63	0.78

LIQUEFACTION ANALYSIS - BORING 2
GROUNDWATER AT 14.5'

SAN LUIS OBISPO COUNTY FIRE STATION 43
Highway 229 at Irongate Road
Creston, California

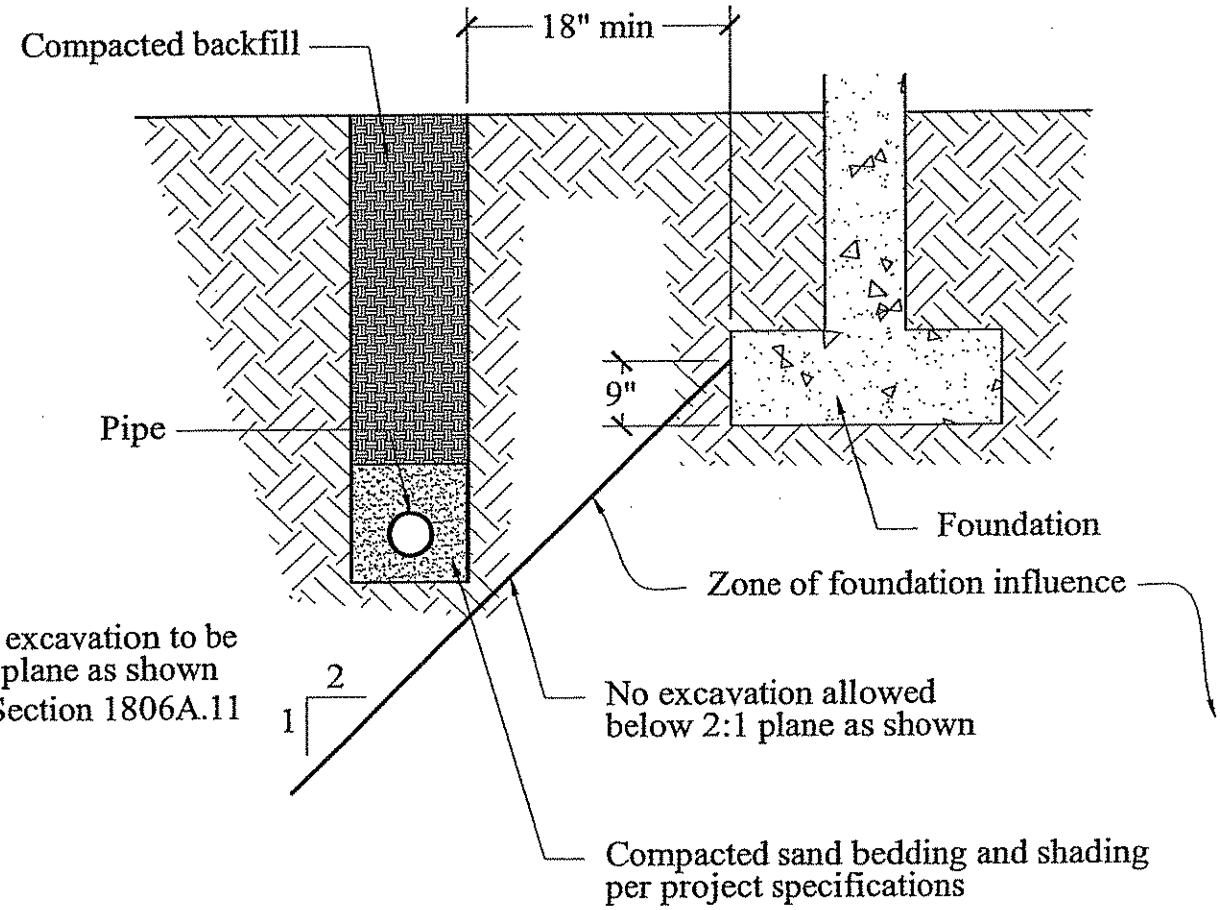
Earth Systems Pacific
4378 Old Santa Fe Road
San Luis Obispo, CA 93401-8116
(805) 544-3276 • FAX (805) 544-1786
E-mail: esc@earthsys.com

April 27, 2009 KM SL-15969-SA

APPENDIX D

Typical Detail A: Pipe Placed Parallel to Foundations

TYPICAL DETAIL A:
TITLE 24
PIPE PLACED PARALLEL TO FOUNDATIONS



All trench excavation to be above 2:1 plane as shown per CBC Section 1806A.11

SCHEMATIC ONLY

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

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