Prepared for **San Lorenzo Unified School District**

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GEOTECHNICAL INVESTIGATION
MEASURE "O" CAMPUS ADDITIONS
SAN LORENZO SCHOOL
SAN LORENZO, CALIFORNIA

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September 2, 2009
File No.: 105356/pwgeo

KLEINFELDER
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September 2, 2009
File No.: 105356/pwgeo

Mr. David Estrada
Construction Manager
San Lorenzo Unified School District
15510 Usher Street
San Lorenzo, California 94580

SUBJECT: Geotechnical Investigation Report for Measure "O" Campus Additions at San Lorenzo High School in San Lorenzo, California

Dear Mr. Estrada:

We are pleased to submit our geotechnical investigation report for the proposed Measure "O" Campus Additions at San Lorenzo High School in San Lorenzo, California. Additional copies of our report have been transmitted to your architect as indicated at the end of this letter. The enclosed report provides a description of the investigation performed and geotechnical recommendations for design and construction of foundations, slabs-on-grade, and earthwork for this project. We are concurrently preparing a Geologic and Seismic Hazard report for this school that will be submitted under separate cover.

In summary, it is our opinion that the site is suitable for the proposed development provided the recommendations of our report are incorporated into the design and construction of the project. The main geotechnical concern for the project site is the moderate expansion potential of the near-surface soils across the site and the low to moderate collapse potential of near-surface soils in area of Building S2. The new buildings can be supported on spread footings, slightly deepened to mitigate the moderately expansive soils at the site. Localized over-excavation and recompaction should mitigate the collapse potential of the porous soils at Building S2. The building floor slabs will need to be supported on "non-expansive" imported soil or lime-treated soils to reduce the impact of expansive soils at the site.

These and other geotechnical recommendations pertaining to the proposed project are discussed in the report. The apparent geologic hazards at the site, other than the expansive soils mentioned above, are the potential for minor liquefaction-induced settlement and for strong ground shaking, which is typical for most of the San Francisco Bay Area. A summary of the geologic hazards is presented in this report; however, a

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separate detailed Geologic and Seismic Hazards Assessment report is being prepared that addresses geologic and seismic hazards at the site.

The conclusions and recommendations presented in this report are based on limited subsurface geotechnical exploration and laboratory testing programs. Consequently, variations between anticipated and actual subsurface soil conditions may be found in localized areas during construction. If significant variations in the subsurface conditions are encountered during construction, Kleinfelder should review the recommendations presented herein and provide supplemental recommendations, if necessary.

Additionally, design plans and specifications should be reviewed by our office prior to their issuance for conformance with the general intent of the recommendations presented in the enclosed report.

We appreciate the opportunity to provide our services to you on this project, and we trust this report meets your needs at this time. If you have any questions concerning the information presented in this report, or related project matters, please contact us at (925) 484-1700.

Sincerely,

KLEINFELDER WEST, INC.

Bradley E. Steen, C.E., G.E. #2839
GeoSciences Group Manager

REGISTERED PROFESSIONAL ENGINEER
BRADLEY E. STEEN
No. 2839
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STATE OF CALIFORNIA

BES/DGG/jmk

cc: Rachel Adams – A4E Architects (2 copies)

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MEASURE "O" CAMPUS ADDITIONS
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PLATES AND APPENDICES

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one - not even you* - should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from alight industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes - even minor ones - and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ-sometimes significantly from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in-this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer For Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
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GEOTECHNICAL INVESTIGATION
MEASURE "O" CAMPUS ADDITIONS
SAN LORENZO HIGH SCHOOL
SAN LORENZO, CALIFORNIA

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Measure "O" Campus Additions at San Lorenzo High School in San Lorenzo, California. The campus is located at 50 E. Lewelling Boulevard in San Lorenzo, California. A Vicinity Map showing the campus location is presented on Plate 1. The project architect is Architecture 4 Education (A4E) of Pasadena, California.

1.1 PROJECT DESCRIPTION

We have based our project description on a digital CAD file we received from A4E (undated) and our discussions with the project architect. The high school campus is an approximately 38-acre site which is essentially flat. A 2-acre parcel in the northeastern corner of the high school campus is currently occupied by a National Guard Armory facility. We understand that this property will be returned to campus use and the Measure "O" campus additions will built in the locations shown on Plate 2, Site Plan.

These additions will consist of a new Digital Arts Building within the current armory facility and an adjacent classroom addition (Building S2) overlapping into the existing school campus. The Digital Arts Building will be of wood-frame construction measuring about 14,000 square feet (sq ft) in plan area. The existing armory building will be renovated but is not included in the scope of our work. Building S2 will be a new, rectangular shaped classroom building, immediately north of Building S. This structure will also be of wood-frame construction, measuring about 3,000 sq ft in area.

We anticipate that grading for the project will include cuts and fills of one to two feet for building pads and to facilitate surface drainage. If actual structural details or grading differ significantly from those assumed we should be contacted to review and/or revise our recommendations.

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation is to explore and evaluate the subsurface soils at the location of the new buildings to provide geotechnical input for the design and construction of foundations and earthwork for this project. The scope of services, as outlined in our July 10, 2009 proposal (File Number: 01002prop-revised), consists of field exploration, laboratory testing, engineering analysis, and preparation of this report.

This report specifically excludes the assessment of site environmental characteristics, particularly those involving hazardous substances. A Phase I Environmental Site Assessment is being prepared under a separate cover.

1.3 PREVIOUS REPORTS

Kleinfelder has previously performed a geotechnical investigation and geologic hazard study for the school. The results of these previous investigations, along with design recommendations, were presented in the following reports:

1. *"Geotechnical Investigation Report, Buildings G and K, San Lorenzo High School, San Lorenzo, California"*, dated February 17,2006 (File No.: 64583/PWGEO).

2. *"Geologic and Seismic Hazards Assessment Report, San Lorenzo High School Campus, San Lorenzo, California"*, dated February 21, 2006 (File No. 64583/PWHAZ).

These reports were also reviewed and considered when preparing our geotechnical recommendations.

2 SITE DESCRIPTION

The San Lorenzo High School Campus is located at 50 East Lewelling Boulevard in San Lorenzo, California. The school was built in 1950 and includes existing school buildings, parking, asphalt play areas and athletic fields. The campus is approximately 38 acres, irregular in shape and generally level. The site is bounded on the north by Highway 238, the west by the railroad right-of-way, and the south and east by East Lewelling Boulevard and Ashland Avenue, respectively. The northwestern half of the campus is primarily grass playing fields. This existing campus configuration is shown on the Site Plan, Plate 2.

3 FIELD EXPLORATION

A field investigation, consisting of test borings and a Cone Penetration Test (CPT) was performed on August 13 and 14, 2009[1]. Note 48 of the California Geological Survey require a minimum of two borings per building and at least one per every 5,000 square feet of building footprint area. Four (4) borings and one CPT were performed for a total of five points of exploration (total building footprint area approximately 17,000 SF).

3.1 SOIL BORINGS

Exploration Geoservices of San Jose, California was subcontracted to provide drilling services. The investigation included drilling four soil borings, B-1 through B-4, located as shown on the Site Plan, Plate 2. These four borings extended to depths of approximately 20 to 25 feet below the ground surface. The borings were drilled using a truck-mounted drill-rig equipped with 8-inch diameter, hollow-stem augers. The borings were logged by a geologist from our offices.

Disturbed and relatively undisturbed samples were taken at the direction of the geologist during drilling. Relatively undisturbed samples of the subsurface materials were obtained using a California sampler with a 2.5-inch inside diameter (I.D.) and a 3-inch outside diameter (O.D.). The samplers were driven 18 inches using a 140-pound hammer falling 30 inches, and blow counts for successive 6-inch penetration intervals were recorded. The blow counts corresponding to the last 12 inches of penetration were reported on the boring logs. After the sampler was withdrawn from the borehole, the samples were removed, sealed to reduce moisture loss, labeled, and returned to our laboratory. Prior to sealing the samples, strength characteristics of the cohesive soil samples recovered were evaluated using a hand-held pocket penetrometer. The results of these tests are shown adjacent to the samples on the boring logs.

[1] Additional borings were performed for the Phase I environmental investigation that have not been included in this report.

Soil classifications made in the field from auger cuttings and samples, were re-evaluated in the laboratory after further examination and testing. The soils were classified in general accordance with the Unified Soil Classification System presented on Plate B-1, Boring Log Legend. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the soil boring logs. The boring logs for borings B-1 through B-4 for this site are presented on Plates B-2 through B-5 in Appendix B.

3.2 CONE PENETROMETER TESTS

We also advanced one Cone Penetration Test (CPT). CPT-1 was performed to assess subsurface conditions and liquefaction potential at the site. The CPT was performed by California Push Technologies of Menlo Park, California using an integrated electronic cone system. The CPT results are presented in Appendix D.

The CPT was performed in accordance with ASTM D 3441. A standard cone mounted on a special rig was used to perform the soundings. The cone has a tip area of 10 square centimeters, a friction sleeve area of 150 square centimeters, and a ratio of end area friction sleeve to tip end area equal to 0.85. The cone bearing (Q_c) and sleeve friction (F_s) were measured and recorded during the tests at every 5 centimeter (about 2 inch) depth intervals.

The cone was pushed using a special rig, having a down pressure capacity of approximately 20 tons. The penetration test was advanced to a depth of approximately 50 feet below the existing ground surface. Our field geologist specified the CPT location. The information gathered from the CPT was used for identifying potential liquefiable and soft soils, and for foundation design. The CPT data (cone bearing, sleeve friction, friction ratio, and equivalent Standard Penetration Test blow counts, (N) versus penetration depth below the existing ground surface are presented on the attached CPT log (Appendix D).

The stratigraphic interpretation of the CPT data was performed based on relationships between cone bearing and sleeve friction versus penetration depth. The friction ratio (R_f), which is sleeve friction divided by cone bearing, is a calculated parameter which is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone bearing and generate large excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate small excess pore water pressures. The interpretation of soil properties from the cone data has been carried out using correlations developed by Robertson et al, 1986 and Olsen and Malone, 1988. It should be noted that it is not always possible to clearly identify a soil type based on cone bearing (Q_c) and sleeve friction (F_s). In these situations, experience and judgment and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type. The soil classification chart used to interpret soil types based on cone bearing (Q_r) and friction ratio (R) is also attached in Appendix D.

Prior to our subsurface exploration, Underground Service Alert (USA) was contacted to provide utility clearance. A private utility locator previously met with the facilities maintenance staff of the San Lorenzo Unified School District (SLZUSD) and marked the known underground utilities. As required by the Alameda County Public Works Agency, a permit was obtained prior to drilling. Upon completion, the borings and CPTs were backfilled with cement grout and patched with cold asphaltic concrete patch near the surface where necessary. Excess soil cuttings were drummed into 55-gallon drums which were labeled and properly disposed by NRC Environmental Services.

The locations of the borings and CPT were estimated by our field engineer based on rough measurements from existing features at the site. Elevations shown on the boring logs were estimated based on the U.S.G.S. Hayward 7½ Minute Series Topographic Quadrangle revised 1980. As such the elevations and locations of the borings and CPTs will be considered approximate to the degree implied by the methods used.

4 LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program included unit weight and moisture content, Atterberg Limits, and unconsolidated-undrained triaxial compression strength tests. Most of the laboratory test results are presented on the boring logs. The results of the Atterberg Limits and unconsolidated-undrained triaxial compression tests are presented graphically on Plates C-1 through C-3 in Appendix C. A laboratory test was also attempted to evaluate the collapse potential of the porous surface soils in Boring B-3. However, the samples proved to be too brittle for appropriate trimming; therefore, only moisture content/dry density was measured.

A chemical analysis was performed by CERCO Analytical on a sample of the upper soils of the campus in a previous investigation to evaluate the corrosive potential of the near-surface soil. The results of the corrosion testing (utilizing ASTM Methods) are discussed in Section 6.8, and presented in Appendix E of this report.

5 SUBSURFACE CONDITIONS

The underlying stratigraphy is interbedded alluvial soils. Based on the past studies and current exploration, this alluvium consists predominantly of alternating layers of clays, silts and sands. The upper 20 feet of the site soils are sandy and silty clays, which are typically firm to hard layers exhibiting low to medium plasticity. The surface clays in Boring 3 exhibited a porous structure at shallow depths (in the area of Building S2). Beneath 20 feet, we encountered clays and silts with interbedded layers of loose to dense sands.

In the previous investigation we have performed at San Lorenzo High School, we encountered interbedded layers of potentially liquefiable silts and clays within the upper 20 feet of the existing ground surface. These shallower layers of potentially liquefiable soils are likely associated their proximity to San Lorenzo Creek (about 350 feet). The current campus additions lie about 1000 feet from the creek and these near-surface (upper 20 feet), potentially liquefiable deposits were not encountered in CPT-1.

Groundwater was encountered at a depth of about 12 feet below the ground surface within the borings. Our previous exploration and historical data[2] suggest groundwater can be as high as 6 feet below the ground surface. Note that groundwater levels can fluctuate depending on factors such as seasonal rainfall, irrigation, groundwater withdrawal, and construction activities on this or adjacent properties, and may rise several feet during a normal rainy season.

The above is a general description of soil and groundwater conditions encountered at the site in the borings and CPTs for this investigation. A more detailed description of the encountered soil and groundwater conditions is presented on the Log of the Borings, Plates B-2 through B-5 of Appendix B and the CPT results presented in Appendix D.

[2] California Geological Survey Seismic Hazard Zone maps for the Hayward quadrangle

Soil and groundwater conditions can deviate from those conditions encountered at the boring locations. If significant variations in the subsurface conditions are encountered during construction, it may be necessary for Kleinfelder to review the recommendations presented herein, and recommend adjustments as necessary.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Based on the results of our field investigation, it is our opinion that the proposed improvements are geotechnically feasible and that the site may be developed as presently planned. This conclusion is based on the assumption that the recommendations presented in this report will be incorporated into the design and construction of this project. The significant geotechnical issue for the planned structures is the presence of moderately expansive near-surface soil. In addition, the surface soils in area of Building S2 appear to exhibit a porous structure which could manifest a low to moderate collapse potential.

Expansive Surface Soils

The surficial clays are moderately expansive and are subject to shrinking and swelling with changes in moisture content. To reduce the potentially adverse effects of these moderately expansive soils, we recommend that shallow foundations should be founded slightly deeper than usual to help reduce foundation movement induced by shrinking and swelling of these soils and that concrete floor slabs and exterior flatwork be underlain by 18 inches and 6 inches of imported "non-expansive" engineered fill, respectively. Lime treatment of the in-situ soils can be considered as an alternative to "non-expansive" fill and is discussed in the recommendations below.

Soil Collapse Potential

The porous structure observed in the near-surface clays suggest a low to moderate collapse potential. In order to support the anticipated building loads and reduce total and differential settlements, we believe the appropriate mitigation of this variable condition is excavation and recompaction. This excavation should extend 12 inches below the planned bottom of shallow foundations and concrete slabs-on-grade. The exposed subgrade should then be scarified for a depth of 12 inches, moisture-conditioned, and recompacted. The excavated soil may be replaced as fill with proper moisture conditioning and recompaction (exclusive of the required “non-expansive” fill beneath the floor slab).

If the grading recommendations presented in this report are followed, the proposed buildings may be supported on shallow foundations consisting of continuous and isolated footings bearing on native soils or properly compacted fill. Any planned covered walkways may be supported on spread footings or drilled cast-in-place piers, depending on the required resistance to lateral and vertical loads. Building foundation settlements should be primarily elastic with the majority of the settlement occurring relatively soon after application of the load. We estimate that total elastic and consolidation settlements for footings should be less than 1 inch, and differential settlements over a 50 foot distance should be less than ½ inch. Post-construction total and differential settlements for drilled piers should be less than ½ inch.

6.2 GEOLOGIC HAZARDS SUMMARY

As required by State of California in Title 24 of the California Building Code (CBC), a geologic and seismic hazard evaluation for the entire school was performed. The results of this evaluation will be presented in a separate report, along with a discussion of the geology and seismicity of the site. To summarize, we have concluded that the proposed new buildings are essentially free of geologic and seismic hazards except for 1) strong ground shaking from earthquakes, which is typical of the entire San Francisco Bay Area, 2) the presence of moderately expansive soils and 3) the potential for liquefaction-induced settlement. This site is not within an Alquist-Priolo Earthquake Fault Zone, and there are no known faults that cross the campus. Seismic design parameters are discussed in the Geologic and Seismic Hazard Assessment report as noted above and are provided in Section 6.4 of this report for reference. Based on the Seismic Hazard Map by the California Geological Society (CGS) for the Hayward quadrangle, the site lies within a region mapped as potentially susceptible to liquefaction.

Our previous investigation at the southern edge of the campus had encountered multiple sand layers which we concluded could result in seismic-induced settlements (liquefaction and dynamic compaction) of up to 4½ inches. These same near-surface layers were not encountered in the northeast corner of the campus where the current improvements are planned.

Our current subsurface exploration encountered primarily firm to hard sandy and silty clays. The near-surface clays have a medium plasticity and are considered to have a moderate expansion potential. Below about 23 feet, there are interbedded layers of potentially liquefiable sands. Although current stabilized groundwater was measured at about 12 feet below the ground surface, our previous exploration and historical data[3] suggest groundwater can be as high as 6 feet below the ground surface. Based on that information, we performed liquefaction analyses using the methods proposed in Youd et. al. (2001) and developed into the software CLiq (version 1.1.1 by GeoLogismiki Geotechnical Software and Dr. Peter Robertson). For our analyses, we used peak ground accelerations of 0.55g associated with an earthquake magnitude of M6.9. We conservatively used a higher than measured groundwater of 6 feet (below ground surface) for our liquefaction analysis. Our liquefaction analyses results from the CPT indicate that layers below a depth of about 23 feet may liquefy during an earthquake. Based on Tokimatsu and Seed, (1987), and the referenced software, total liquefaction induced settlements for the CPTs are estimated to be about 1 inch. These potentially liquefiable layers range from about ½ to 3 feet in thickness. Based on Ishihara (1985) and Youd and Garris (1995), we believe that the potential for ground surface disruption (such as sand boils, ground fissures, etc.) to occur at site is low due to the presence of the non-liquefiable clayey soils above the sandy layers. The results of our liquefaction analyses are presented in Appendix C.

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or poorly compacted fill soils. The subsurface conditions encountered in our borings above the groundwater table are not conducive to such seismically induced ground failures. For this reason we conclude that the potential for dynamic compaction to occur at this site is very low.

[3] California Geological Survey Seismic Hazard Zone maps for the Hayward quadrangle

6.3 FOUNDATIONS

6.3.1 General

If the grading recommendations presented in this report are followed, the proposed buildings may be supported on shallow foundations consisting of continuous and isolated footings bearing on native soils or properly compacted fill. Any planned covered walkways may be supported on spread footings or drilled cast-in-place piers, depending on the required resistance to lateral and vertical loads.

6.3.2 Shallow Foundations

The recommended allowable soil bearing pressures, depth of embedment, and width of footings are presented below. The allowable bearing values provided have been estimated assuming that all footings uniformly bear on undisturbed native soils.

FOUNDATION BEARING CAPACITY RECOMMENDATIONS

Footing Type	Allowable Bearing Pressure (psf)*	Minimum Embedment (in)**	Minimum Width (in)
Exterior Continuous Footing***	2,500	18	18
Interior Continuous Footing	2,500	18	18
Isolated Interior Footing	2,500	18	18x18
Isolated Exterior Footing	2,500	18	18x18

* Dead plus live load
** Below lowest adjacent grade defined as bottom of slab on the interior and finish grade at the exterior.
*** Includes perimeter footing around building.

Footings within Building S2 should be over-excavated to 12 inches below planned bottom of footing. The exposed surface should be scarified to a depth of 12 inches, moisture-conditioned to above optimum, and recompacted as discussed in Section 6.7, Earthwork. The excavated soil may be replaced as engineered fill up to the planned footing bottom elevation.

Allowable soil bearing pressures may be increased by one-third for transient applications such as wind and seismic loads. It should be noted that because of the moderate expansive nature of the near-surface soils, the actual bearing capacities should not be less than 1,000 psf to reduce potential movement of the foundations as a result of the expansion potential of the near-surface soils. Footings with bearing pressures below 1,000 psf will need to be extended 6 inches deeper than indicated in the table above to compensate for the lower pressure. Alternatively, this bottom 6 inches could be filled with sand/cement slurry.

Where footings are located behind retaining walls or near and parallel to major underground utilities, the footings should extend below a plane projected at a slope of 2:1 upward from the bottom of the retaining wall or the underground utility to avoid surcharging the retaining wall or underground utility with building loads.

To help reduce fluctuations in moisture content beneath the buildings and the shrink/swell cycle associated with expansive soils, continuous footings should be used around the perimeter of the buildings to provide a barrier against changes in moisture of the soils beneath the interior floor slabs. Where utilities cross perimeter footing lines, the trench backfill should consist of a vertical barrier of impervious type material or lean concrete extending about 2 feet either side of the perimeter footing.

Lateral loads may be resisted by a combination of friction between the footing bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable friction coefficient of 0.30 between the foundation and supporting subgrade may be used. For passive resistance, an allowable equivalent fluid pressure of 300 pounds per cubic foot acting against the footing may be used. The friction coefficient and passive resistance may be used concurrently, and can be increased by one-third for wind and/or seismic loading. We recommend that the first foot of soil cover be neglected in the passive resistance calculations if the ground surface above is not confined by a slab, pavement or in some similar manner. These values include a factor of safety of about 1½.

Concrete for footings should be placed neat against undisturbed soil. It is important that footing excavations in clayey soils not be allowed to dry before placing concrete. If shrinkage cracks appear in the footing excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The footing excavations should be monitored by a representative of Kleinfelder for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials. If soft or loose soils are encountered at the bottom of footing excavations, they should be removed and replaced with lean concrete.

6.3.3 Drilled Piers

Drilled piers should be at least 12 inches in diameter, spaced at least three pier diameters center to center, and extend at least six feet below the finished ground surface. Drilled piers may be designed using an allowable skin friction of 600 and 500 psf for axial compressive and uplift loads, respectively. These values include a factor of safety of at least two, and may be increased by one-third for resisting total loads, including wind and seismic. The upper two feet of soil should be ignored for calculation of skin friction, unless the ground surface around the pier is covered with a slab or pavement.

Piers will provide lateral resistance from passive pressure acting on the upper portion of the piers and from their structural rigidity. Lateral resistance of piers depends on the pier diameter, pier head fixity (restrained or unrestrained), allowable deflection of the pier top, and the bending moment resistance of the piers. Piers may be designed to resist lateral loads using an equivalent fluid weight of 300 pounds per cubic foot to a maximum pressure of 3,000 pounds per square foot, acting over an equivalent width of two times the pier diameter, to compute the allowable passive resistance. The top two feet of soil should be ignored for calculating lateral resistance, unless the ground surfaces adjacent to piers are covered with flatwork.

Soil exposed in pier excavations should be maintained in a moist to wet condition until concrete is placed. Also, concrete overpour at the top of piers should be avoided, because overpour may create horizontal surfaces over which swelling of expansive soil could impose uplift. Cardboard forms may be used at the tops of pier to control this condition.

We recommend steel reinforcement and concrete be placed within about 4 to 6 hours upon completion of each drilled pier hole; as a minimum, the holes should be poured the same day they are drilled. The steel reinforcement should be centered in the drilled hole. Concrete used for pier construction should be discharged vertically into the holes to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during construction. Water was encountered at about 12 feet below ground surface during our investigation; therefore groundwater will probably not be encountered within shallow drilled pier holes; however, seasonal variations and irrigation can result in higher groundwater water. If more than 10 inches of water is present during concrete placement, either the water needs to be pumped out or the concrete placed into the hole using tremie methods. In order to develop the design skin friction value previously provided, concrete used for pier construction should have a slump of 6 to 8 inches. Our borings indicate that the drilled piers can be drilled with a standard flight auger using a standard rig, subject to access restrictions.

The bottom of the drilled holes should be cleaned such that no more than 2 inches of loose soil remains in the hole prior to placement of concrete. A representative from Kleinfelder should be present to observe drilled holes to confirm bottom conditions prior to placing steel reinforcement.

6.4 2007 CBC SEISMIC DESIGN PARAMETERS

The seismicity of the region surrounding the site is discussed in the previously referenced Geologic and Seismic Hazards Assessment Report. From that discussion it is important to note that the site is in a region of high seismic activity and will likely be subjected to major shaking during the life of the project. As a result, structures to be constructed on the site should be designed in accordance with applicable seismic provisions of the building codes.

Due to potential earthquake motion resulting from nearby faults, seismic design factors should be considered in the structural design of the proposed facility. Structures with strength discontinuities, soft stories, plan irregularities, discontinuous shear walls and ductile moment frames are particularly vulnerable to these types of motions and should either be avoided or properly evaluated.

Based on the 2007 CBC, the seismic ground motion parameters S_s and S_1 for the site are 1.865g and 0.707g, respectively. Note that these parameters were developed based on Table 20.3-1 and Figures 22-3 and 22-4 of ASCE 7-05 (Java ground motion parameter calculator developed by the U.S. Geological Survey, 2007), which is the basis for the selection process of seismic parameters in the 2007 CBC. These factors are for a Type B soil profile. However, these factors need to be adjusted by the structural engineer based on the site-specific soil profile.

Based on the results of our liquefaction analyses, some of the sand layers underlying the site may liquefy. Therefore, according to Table 1613.5.2 of the 2007 California Building Code (CBC), the site should be classified as Site Class F, which requires site response analysis. However, the site-specific modifying factors, F_a and F_v, for Class F references Section 11.4.7 of ASCE 7. Sections 11.4.7 and 20.3.1 of ASCE 7 state that for a short period (less than ½ second) structure on liquefiable soils, these factors may be based on the assessment of the site class assuming no liquefaction. Since the proposed building will consist of structures with a period of less than ½ second, we recommend using Site Class D (stiff soil site) and modifying F_a and F_v as shown in the Table 6.4-1 shown below.

Because the site lies within 10 km of a mapped active fault, Kleinfelder has performed a ground motion hazard analysis using Probabilistic and Deterministic Seismic Hazard Analysis (PSHA and DSHA) methods in accordance with the requirements of Section 21.2 of ASCE 7. The purpose of this study was to develop the site-specific ground motion criteria in terms of spectral accelerations by using a seismic source model and subsurface soil conditions encountered at the site. The recommended seismic design parameters developed from our analyses are presented as follows:

Table 6.4-1
Recommended 2007 CBC Seismic Design Parameters

Design Parameter	Symbol	Recommended Value	2007 CBC (ASCE 7) Reference(s)
Site Class	--	D	Section 1613A.5.2
Mapped Spectral Acceleration for Short Periods (Unfactored Type B Site)	S_s	1.865g	Section 1613A.5.1
Mapped Spectral Acceleration for a 1-Second Period (Unfactored Type B Site)	S_1	0.707g	Section 1613A.5.1
Site Coefficient	F_a	1.0	Table 1613A.5.3(1)
Site Coefficient	F_v	1.5	Table 1613A.5.3(2)
MCE* Peak Ground Acceleration (S_M at T=0)	PGA_M	0.824g	(Section 21.2)
MCE* Spectral Response Acceleration for Short Periods	S_{MS}	1.818g	Section 1613A.5.3 (Section 21.4)
MCE* Spectral Response Acceleration at 1-Second Period	S_{M1}	1.818g(1)	Section 1613A.5.3 (Section 21.4)
Design Peak Ground Acceleration (S_D at T=0)	PGA_D	0.550g	(Section 21.2)
Design Spectral Response Acceleration (5% damped) at Short Periods	S_{DS}	1.212g	Section 1613A.5.4 (Sections 21.3, 21.4)
Design Spectral Response Acceleration (5% damped) at 1-Second Period	S_{D1}	1.212g (2)	Section 1613A.5.4 (Sections 21.3, 21.4)

*MCE: Maximum Considered Earthquake
(1) This value is 2.493 but was matched to S_{MS}
(2) This value is 1.662 but was matched to S_{DS}

6.5 SLABS-ON-GRADE

Slabs-on-grade for this project will consist of concrete floor slabs and exterior flatwork. As previously discussed, the near-surface soils are moderately expansive, and will be subject to shrink/swell cycles with fluctuations in moisture content. To reduce these potentially adverse effects, we recommend that concrete floor slabs be underlain by 18 inches of imported "non-expansive" engineered fill placed on subgrade prepared as described in the "Earthwork" section of this report. The properties of this “non-expansive” fill should also meet the criteria listed in the "Earthwork" section of this report.

Lime treatment of the in-situ soils can be considered as an alternative to "non-expansive" fill. If this alternative is desired, extensive quality control is needed as well as laboratory testing to evaluate the appropriate lime treatment mixture. The client needs to understand the risk of this approach if selected. For estimating purposes, approximately 18 inches of soil would be needed to be treated provided that the moisture content of the soils below that to be treated is at least 2 percent over optimum moisture. Otherwise, the thickness of the lime treatment needs to be increased to 21 inches. Our experience with similar soils has indicated that about 3 to 5 percent lime by weight is needed for the treatment. If lime treatment is selected, we will need to perform additional laboratory tests to refine this estimate prior to construction.

The "non-expansive" fill or lime treated native soil should extend a minimum horizontal distance of 5 feet beyond all building areas, including the outer edge of perimeter footings and footings extending beyond perimeter walls, where flatwork is planned, and 3 feet elsewhere. It is important that placement of this material be done as soon as possible after compaction of the subgrade to prevent drying of the native subgrade soils and that slabs be constructed as soon as possible after "non-expansive" material is placed, as subgrades will dry out even through "non-expansive" fills. A representative of Kleinfelder should be present to observe the condition of the subgrade, and observe and test the installation of the "non-expansive" engineered fill prior to slab construction.

The negative impact of lime treatment on future vegetation should be considered in whether it should be used, and what mitigation measures are needed. We do not recommend lime-treating in future landscaping areas. If lime-treating extends into future plantings for overbuild purposes, it may significantly raise the pH levels of the soils. This may be remediated through use of sulfur or commercial fertilizers containing ammonium-N. Consideration must be given to the impact of such soil-reworking adjacent to a structural element. For example, tilling (which negates the applied compactive effort and strength gain through lime treatment) and application of amendments adjacent to a foundation element could result in a reduced ability to resist lateral loads.

6.5.1 Concrete Floor Slabs

To provide uniform slab support over the existing clays within Building S2 (only), the near-surface soils will require recompaction and proper moisture conditioning to mitigate the collapse potential. This moisture conditioning and recompaction should extend a minimum horizontal distance of 5 feet beyond all building areas, including the outer edge of perimeter footings and footings extending beyond perimeter walls, where flatwork is planned, and 3 feet elsewhere. It is recommended that the upper 12 inches (measured from the bottom of capillary-break slab rock) of the building pad be excavated. The exposed subgrade should then be scarified to a depth of 12 inches, moisture-conditioned to above optimum moisture content, and recompacted. The excavated soil may then be replaced as fill with the same proper moisture conditioning and recompaction, as described in the "Earthwork" Section of this report. If lime-treatment of on-site soils is not selected as the underslab treatment, then non-expansive, imported soils will be used within upper 12 inches beneath slab rock.

All concrete floors should be supported on at least 6 inches of angular gravel or crushed rock to enhance subgrade support for the slab. This material may be considered part of the required minimum of 18 inches of "non-expansive" engineered fill or lime treated soil. The capillary break material should be 3/4-inch maximum size with no more than 10 percent by weight passing the #4 sieve. It is important that placement of this material and concrete be done as soon as possible after compaction of the "non-expansive" subgrade materials to reduce drying of the subgrade.

Floor slabs should have a minimum thickness of 5 inches. A Structural Engineer should design reinforcing and slab thickness. Because the floor slabs are to be supported on imported "non-expansive" granular material or lime-treated soil, a modulus of subgrade reaction is difficult to estimate. For estimating purposes, a value of 200 pounds per cubic inch may be used. If necessary, this value may be confirmed when the source and type of the imported soil has been determined.

Special care should be taken to ensure that reinforcement is placed at the slab mid-height. The floor slab should be separated from footings, structural walls, and utilities and provisions made to allow for settlement or swelling movements at these interfaces. If this is not possible from a structural or architectural design standpoint, it is recommended that the slab connection to footings be reinforced such that there will be resistance to potential differential movement.

Subsurface moisture and moisture vapor naturally migrate upward through the soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of the subsurface moisture and potential impact of future introduced moisture (such as landscape irrigation or precipitation) the current industry standard is to place a vapor retarder on the compacted crushed rock layer. This membrane typically consists of visqueen or polyvinyl plastic sheeting at least 10 mils in thickness. It should be noted that although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building and all elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils affect slab moisture and can control future performance. In many cases, floor moisture problems are the result of either improper curing of floors slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) manual.

It is emphasized that we are not floor moisture proofing experts. We make no guarantee nor provide any assurance that use of capillary break/vapor retarder system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for floor slab moisture protection.

Exterior grading will have an impact on potential moisture beneath the floor slab. Recommendations for exterior draining are provided in the "Site Drainage" section of this report.

6.5.2 Exterior Flatwork

Concrete exterior flatwork at grade will be constructed on soils subject to swell/shrink cycles. Some of the adverse effect of swelling and shrinking can be reduced with proper moisture treatment. The intent is to reduce the fluctuations in moisture content by moisture conditioning the soils, sealing the moisture in, and controlling it. Near-surface soils should be moisture conditioned according to the recommendations in Exhibit 1, Appendix A. In addition, all exterior concrete slabs should be supported on a minimum of 6 inches of "non-expansive" soil, lime-treated soil, Class 2 Aggregate Sub-Base (ASB), or Class 2 Aggregate Base (AB). Even with the 6 inches of "non-expansive" material, some movement of exterior slabs may occur. Where concrete flatwork is to be exposed to vehicle traffic, this 6 inches of "non-expansive" fill should be Class 2 Aggregate Base as specified in the current California of Transportation Standard Specifications. Exterior flatwork will be subjected to edge effects due to the drying out of subgrade soils. Because of the expansive soils, flatwork should have control joints on no greater than 8 feet centers. To protect against edge effects adjacent to unprotected areas, such as vacant or landscaped areas, lateral cutoffs such as inverted curbs are recommended. Prior to construction of the flatwork, the 6 inches of "non-expansive" fill, ASB or AB, should be moisture conditioned to near optimum moisture content. If the "non-expansive" fill, ASB or AB is not covered within 30 days after placement, the soils below this material will need to be checked for appropriate moisture of at least 2 percent over optimum. If the moisture is found to be below this level, the flatwork areas will need to be soaked until the proper moisture content is reached. Where flatwork is adjacent to curbs, reinforcing bars should be placed between the flatwork and the curbs. Expansion joint material should be used between flatwork and curbs, and flatwork and buildings.

6.6 SITE DEMOLITION

6.6.1 Existing Improvements

As part of the demolition process, excavations for the removal of foundations, underground utilities or other below-ground obstructions should be cleaned of loose soil and deleterious material, and backfilled with compacted engineered fill. Compaction requirements for this backfill should conform to the recommendations presented in Exhibit 1, Appendix A.

6.6.2 Existing Utilities

In addition, active or inactive utilities within the construction area should be protected, relocated, or abandoned. Pipelines, 2 inches in diameter or less, may be left in place beneath the planned building. Pipelines between 2 and 6 inches in diameter may be left in place within the limits of the building provided they are filled with sand/cement slurry and capped at both ends. Pipelines larger than 6 inches in diameter within the planned building should be removed. Active utilities to be reused should be carefully located and protected during demolition and construction.

6.6.3 Re-Use of On-site Material

Existing asphalt concrete may be pulverized and mixed with the underlying base for use as engineered fill if it meets the following requirements:

Gradation	
Sieve Size	**Percentage Passing**
3 inch	100 min.
1½ inch	85 min.
No. 200	8 – 40

The processed asphalt concrete/base material may be used as “non-expansive” fill if the material meets the gradation requirements above and has a plasticity index of 15 or less. Site Portland cement concrete (PCC) may be processed and reused as engineered fill, “non-expansive” fill beneath the floor slab if it meets the requirements presented in this report for the specific materials.

6.7 EARTHWORK

6.7.1 General

Grading plans were not available to us at the time this report was prepared. However, based on the existing topography and the nature of the proposed development, we anticipate that grading will consist of minor cuts and fills on the order of one to two feet to create subgrades for the new development and to achieve proper site drainage. Final grading plans should be reviewed by Kleinfelder for conformance to our design recommendations prior to construction bidding. In addition, it is important that a representative of Kleinfelder observe and evaluate the competency of existing soils or new fills underlying structures, slabs-on-grade, and pavements. In general, if soft or unsuitable materials are encountered, these should be overexcavated, removed and replaced with compacted engineered fill material.

6.7.2 Site Preparation and Grading

In general, site preparation and grading should be performed in accordance with the site specific recommendations which follow. A brief summary of compaction recommendations is presented in Exhibit 1, Appendix A. Additional earthwork recommendations are presented in related sections of this report.

Initial site preparation will include removal of the asphalt and concrete capped areas. In these capped areas, the soil may be significantly above optimum moisture content when exposed. As a result, the subgrade soils may need to be dried prior to preparation. It is recommended that any landscape watering in the area be stopped at least two weeks prior to the start of stripping. Areas which are to receive fill or are at subgrade for pavements or exterior slabs should be stripped of existing surface vegetation, organic topsoil, and debris prior to preparation and recompaction. The stripped materials should not be reused as engineered fill and should be removed from the site, or used in landscaped areas, as appropriate. The depth of stripping related to the removal of surface vegetation is anticipated to be 3 inches or less. Stripping should extend a minimum of 5 feet and 2 feet laterally, outside the buildings and pavements, and the back of curbs and the sides of flatwork, respectively.

Tree stumps and roots over 1 inch in diameter and over 3 feet in length should be removed within the building footprint and areas for planned improvements. Additional grubbing may be required if high concentrations of smaller roots or other organics are encountered. From a geotechnical standpoint, existing landscaping may be left in place as landscaping provided that it is outside of the area to be graded. In order to address the presence of expansive soil, we have recommended that deepened curbs be installed where landscape areas abut pavement or sidewalks. If the existing trees are to be left in place, then deepened curbs will be needed around these trees. Also, leaving the trees in place may require additional cautionary measures during grading that may create problems (such as additional setback, observation by an arborist), which should be evaluated by the design team.

Prior to placement of engineered fill or following excavation to reach desired subgrade levels in proposed buildings, the exposed subgrades should be scarified to a minimum depth of 12 inches. Scarification should extend laterally a minimum of 5 feet beyond building limits and 2 feet beyond pavement and flatwork areas, where achievable. Scarified areas should be moisture conditioned to above 2 percent over optimum, and recompacted as specified in Exhibit 1. Fills should be placed in lifts no greater than 8 inches in uncompacted thickness. Laboratory maximum density and optimum moisture relationships should be determined by ASTM Test Designation D1557. Finished fill slopes should not exceed a slope of 2:1 (horizontal to vertical). Soft areas and areas of loose soil may be encountered which may require overexcavation and recompaction. Unit prices for overexcavation and replacement with compacted fills should be obtained during bidding.

The on-site soils, if free of organic matter or other deleterious materials, are suitable for use as general engineered fill; however, the near-surface clayey soils are moderately expansive and should not be re-used where "non-expansive" fill is required. Maximum particle size for fill material should be limited to 3 inches, with at least 90 percent by weight passing the 1 inch sieve.

Where "non-expansive" imported material is required, it is recommended that it be granular in nature, adhere to the above gradation recommendations and conform to the following minimum criteria:

Plasticity Index	15 or less
Liquid Limit	30% or less
Percent Soil Passing #200 Sieve	8% to 40%

Highly pervious materials such as pea gravel or clean sands are not recommended because these permit transmission of water to the underlying soils.

All on-site or import fill material should be compacted to the general recommendations provided for engineered fill. "Non-expansive" import material should be compacted at or slightly above the optimum moisture content, and on-site soils should be compacted to minimum moisture content of above 2 percent over the optimum moisture content. Grading operations during the wet season or in areas where the soils are saturated may require provisions for drying of soil prior to compaction. If the project necessitates fill placement and compaction in wet conditions, we could provide alternatives for drying the soil. Conversely, additional moisture may be required during the dry months. Water trucks should be available in sufficient number to provide adequate water during compaction.

All site preparation and fill placement should be observed by Kleinfelder. It is important that during the stripping and scarification process, a representative of Kleinfelder be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during the geotechnical site exploration.

6.7.3 Excavation and Backfill

Excavations for footings, utility trenches, or other excavations are anticipated to be made with either a backhoe or trencher. We expect the walls of trenches less than 5 feet deep to stand near vertical without support for a period of several days.

Where trenches or other excavations are extended deeper than 5 feet, the excavation may become unstable and should be evaluated to monitor stability prior to personnel entering the trenches. Shoring or sloping of any deep trench wall may be necessary to protect personnel and to provide stability. All trenches should conform to the current OSHA requirements for work safety. Excavations should be located so that no structures, foundations, or slabs are located above a plane projected at 2:1 (horizontal to vertical) upward from any point in an excavation, regardless of whether it is shored or unshored.

Backfills for trenches or other small excavations beneath slabs should be compacted as noted in Exhibit 1. Special care should be taken in the control of utility trench backfilling under structural, pavement, and slab areas. Poor compaction may cause excessive settlements resulting in damage to overlying structures, slabs and pavements.

Where utility trenches extend from the exterior to the interior limits of the building, native clayey soils or lean concrete should be used as backfill material for a distance of approximately 2 feet laterally on each side of the exterior building line to reduce the potential for the trench acting as a conduit for the exterior surface water. Utility trenches located in landscaped areas should also be capped with a minimum 12 inches compacted on-site clayey soil.

Underground utilities should be located above a 2:1 (horizontal to vertical) plane projected downward from the bottom of the new footings to avoid undermining the footings during the excavation of the utility trench.

6.7.4 Site Drainage

Proper site drainage is important for the long-term performance of the planned structures. The site should be graded so as to carry surface water away from the building foundations at a minimum of 2 percent in paved areas and 3 percent in landscaped areas to a minimum of 5 feet laterally from the buildings. In addition, all roof gutters should be connected directly into the storm drainage system or drain onto impervious surfaces provided that a safety hazard is not created.

6.7.5 Storm Water Runoff Mitigation

Storm water runoff regulations require pretreatment of runoff and infiltration of storm water to the extent feasible. Typically, this results in the use of bioretention areas, vegetated swales, infiltration trenches, buried storm water detention/infiltration galleries, or permeable pavement near or within parking lots and at the location of roof run-off collection. These features are not well-suited to clay soils due to their relatively low permeability, which does not allow significant infiltration over short time periods. In addition, allowing water to pond on expansive clay soils can cause the soils to swell, which can cause distress to pavements, slabs, and lightly loaded structures.

Implementation of storm water infiltration criteria will likely result in increased distress and reduced service life of pavement and flatwork if not carefully designed in clay soils. In general, bioretention areas, vegetated swales and infiltration areas should be located in landscaped areas and well away from pavements, buildings, and slopes.

If it is not possible to locate these infiltration systems away from buildings and pavements, alternatives that isolate the infiltrated water, such as flow-through planters, could be considered. When using an infiltration system in clay soils, underdrains should be used. In addition, the top of the swales should be a laterally separated a minimum of 12 inches from the curbs. To reduce potential for rotation of the curbs, curbs adjacent to the swales should extend a minimum of 12 inches below the bottom of the aggregate base course.

Permeable pavements and pavers rely on the underlying aggregate to drain and/or store water. Aggregate base rock (such as Caltrans Class 2) does not drain readily. Open-graded bases or gravel drain faster and store water in the intergranular (i.e. pore) spaces, but still require a drainage outlet to prevent overflow (backup) of stored water where underlain by clay or low permeability soils.

Due to the potential adverse affects on project performance we should review the geotechnical aspect of the storm water infiltration system and its location. Backup or overflow of storm water systems may not be feasible to prevent, particularly in clay soil environments and during prolonged or intense storm events.

6.8 CORROSION ASSESSMENT

Corrosion testing was performed on the campus near-surface soils in conjunction with our 2006 geotechnical investigation for Building G and K at San Lorenzo High School. A copy of the letter transmitting the results is presented in Appendix E. Based upon the resistivity measurements, the sample was classified as “moderately corrosive”. All buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion. Since we are not corrosion specialists, a corrosion testing firm should be contacted for specific design details.

A more detailed investigation may include more or fewer concerns and should be directed by a corrosion expert. Soils actually in contact with concrete should be sampled and tested for sulfate content during construction and the concrete mixes used should comply with the requirements of the 2007 CBC based on these results. Consideration should also be given to soils in contact with concrete that will be imported to the site during construction, such as topsoil and landscaping materials. Also, on-site cutting and filling may result in soils contacting concrete that were not anticipated at the time of the investigation.

As an alternative or in addition to meeting CBC mix requirements, your Structural Engineer, architect or corrosion expert may choose to isolate the concrete from the corrosive soils or from ground or surface water that may leach corrosive materials from the soils and contact the concrete. These alternatives may include waterproofing, capillary breaks, vapor barriers and removal of corrosive soils.

7 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by San Lorenzo Unified School District and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The work performed was based on project information provided by Client. If Client does not retain Kleinfelder to review any plans and specifications, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, Client must obtain written approval from Kleinfelder's engineer that such changes do not affect our recommendations. Failure to do so will vitiate Kleinfelder's recommendations.

The scope of services was limited to four borings and one CPT, and restrictions on access to the equipment sites. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on subsurface exploration including borings drilled to a maximum depth of 35 feet, groundwater level measurements in borings, laboratory testing of soil plasticity, gradation density, and compressive strength, and engineering analyses].

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil, rock or groundwater conditions could vary between or beyond the points explored. If soil, rock or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated building loads, and the design depths or locations of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered. As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications;

- Observe foundation installation and supporting soils to confirm conditions are as anticipated.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

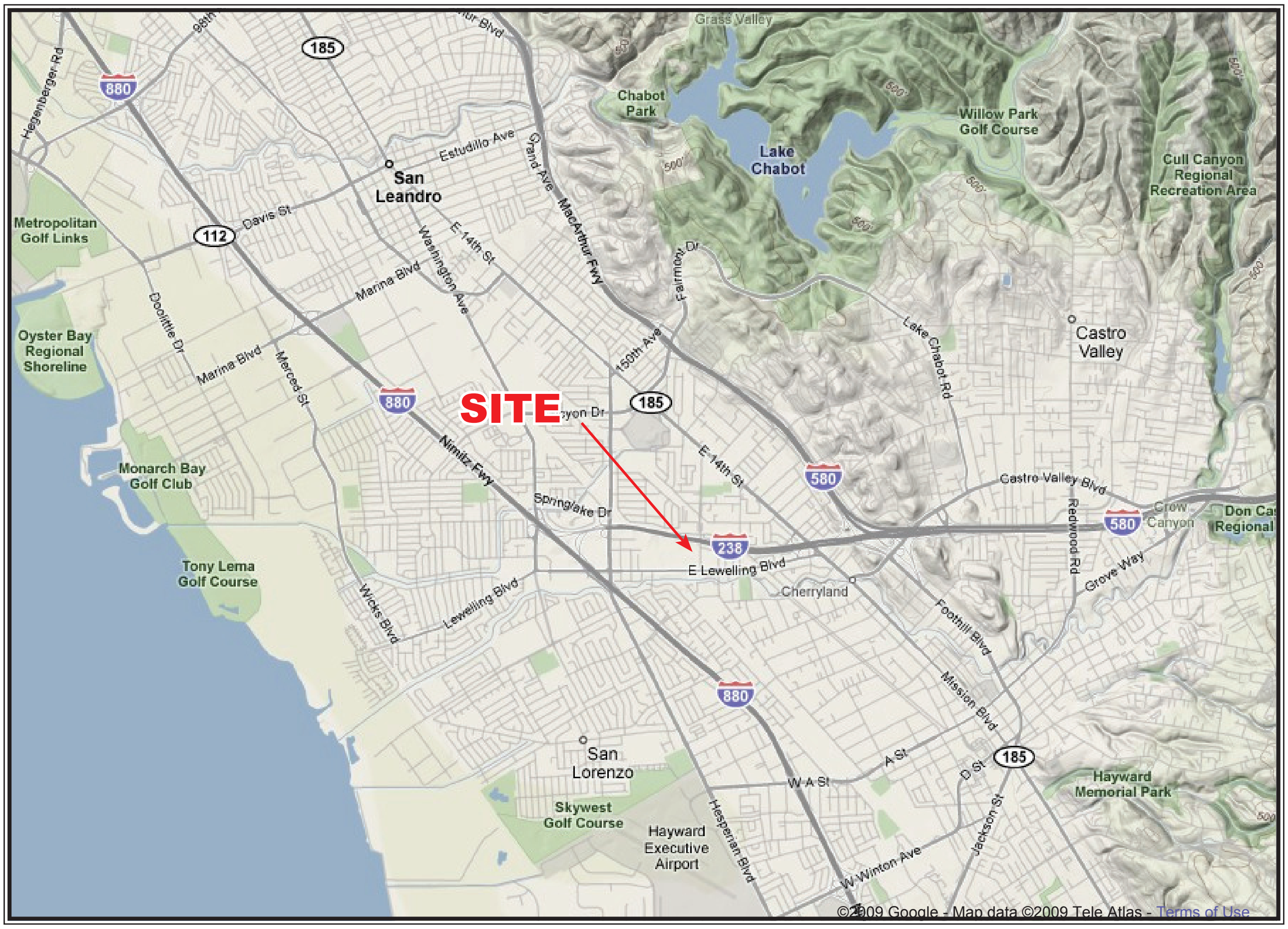
Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, preparation of foundations, installation of piers, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil, rock and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction. Furthermore, the contractor should be prepared to handle contamination conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

PLATES

Approximate Scale

Not to Scale



Reference: http://maps.google.com, 2009

The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. Kleinfelder makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.

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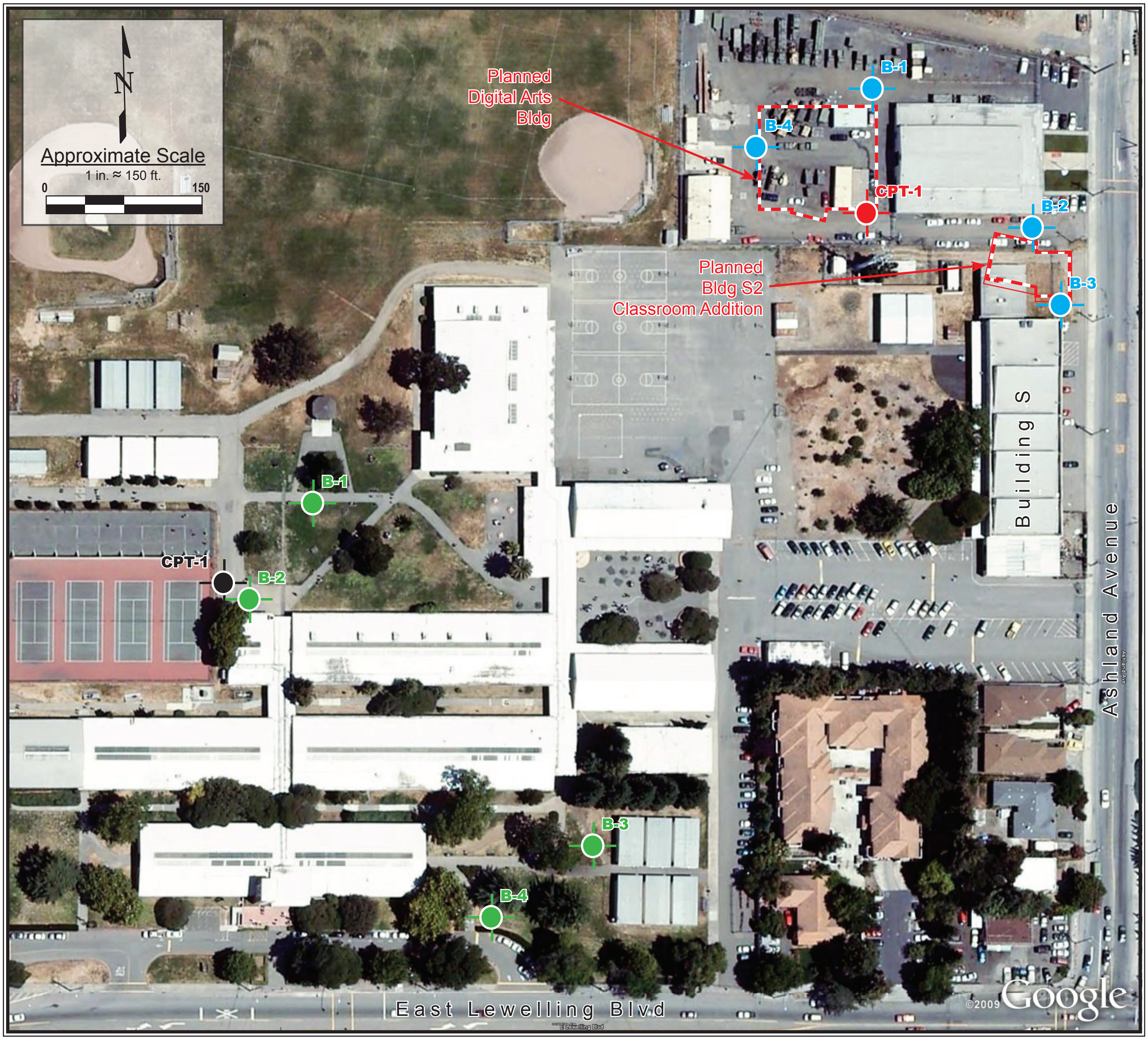
PROJECT NO.	105356
DRAWN:	08/18/09
DRAWN BY:	B. Steen
CHECKED BY:	D. Gray
FILE NAME:	VicMap.indd

VICINITY MAP

Measure O Campus Additions
San Lorenzo High School
50 East Lewelling Blvd.
San Lorenzo, California

PLATE

1



Legend

CPT-1 (red)	Approximate Location Cone Penetrometer Test (Kleinfelder, 2009)
B-3 (blue)	Approximate Boring Location (Kleinfelder, 2009)
CPT-1 (black)	Approximate Location Cone Penetrometer Test (Kleinfelder, 2006)
B-2 (green)	Approximate Boring Location (Kleinfelder, 2006)
(dashed red box)	Approximate Building Addition Location

Reference: http://earth.google.com, 2009

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PROJECT NO. 105356	**SITE PLAN**	PLATE
DRAWN: 08/18/09		
DRAWN BY: B. Steen		**2**
CHECKED BY: D. Gray	**Measure O Campus Additions** **San Lorenzo High School** **50 East Lewelling Blvd.** **San Lorenzo, California**	
FILE NAME: SitePlan.indd		

APPENDIX A

EXHIBIT 1
SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendation [3,4,5]
General Engineered Fill	Compact to a minimum of 90 percent compaction at least 2 percent over optimum moisture content for clayey soils and at near optimum moisture content for granular soils.
“Non-Expansive” Fill	Compact to a minimum of 90 percent compaction at near optimum moisture content.
Trenches [2]	Compact to a minimum of 90 percent compaction at least 2 percent over optimum moisture content for clayey soils and at near optimum moisture content for granular soils.
Exterior Flatwork [1]	Compact upper 12 inches to between 88 and 92 percent compaction and at least 2 percent over optimum moisture content for clayey soils and at near optimum moisture content for granular soils. Where exterior flatwork is exposed to vehicular traffic, compact baserock to a minimum of 95 percent compaction.
Lime treated Soils in Building Pad [1]	Compact lime-treated soils to a minimum of 90 percent relative compaction at least 2 percent over optimum moisture content.

Notes:
(1) Depths are below finished subgrade elevation.
(2) In landscaping areas, this percent compaction in trenches may be reduced to 85 percent.
(3) All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D-1557. All lifts to be compacted shall be a maximum of 8 inches loose thickness.
(4) All compacted surfaces should be unyielding under compaction equipment.
(5) Where fills are deeper than 7 feet, the portion below 7 feet should be compacted to a minimum of 95 percent.

APPENDIX B

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	DESCRIPTION	MAJOR DIVISIONS		LTR	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW	Well-graded gravels or gravel with sand, little or no fines.	FINE GRAINED SOILS	SILTS AND CLAYS	ML	Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		GP	Poorly-graded gravels or gravel with sand, little or no fines.			CL	Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		GM	Silty gravels, silty gravel with sand mixture.			OL	Organic silts and organic silt-clays of low plasticity.
		GC	Clayey gravels, clayey gravel with sand mixture		SILTS AND CLAYS	MH	Inorganic elastic silts, micaceous or diatomaceous or silty soils.
	SAND AND SANDY	SW	Well-graded sands or gravelly sands, little or no fines.			CH	Inorganic fat clays (high plasticity).
		SP	Poorly-graded sands or gravelly sands, little or no fines.			OH	Organic clays of medium high to high plasticity.
		SM	Silty sand.	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
		SC	Clayey sand.				

Standard Penetration Split Spoon Sampler 2.0 inch O.D., 1.4 inch I.D.

Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.

Bulk Sample

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Shelby Tube 3.0 inch O.D.

0745, 5/31 Approximate water level first observed in boring. Time recorded in reference to a 24 hour clock.

0800, 5/31 Approximate water level observed in boring following drilling

PEN	Pocket Penetrometer reading, in tsf
TV:Su	Torvane shear strength, in ksf

LL	Liquid Limit	UC	Unconfined Compression
PI	Plasticity Index	TxUU	Triaxial Shear
%-#200	Sieve Analysis (#200 Screen)	CONSOL	Consolidation
DS	Direct Shear	R-Value	Resistance Value
C	Cohesion (psf)	SE	Sand Equivalent
PHI	Friction Angle	EI	Expansion Index
		FS	Free Swell (U.S.B.R.)

Notes: Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.

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PROJECT NO. *105356*

BORING LOG LEGEND

MEASURE O CAMPUS ADDITIONS
SAN LORENZO HIGH SCHOOL
16501 ASHLAND AVENUE
SAN LORENZO, CALIFORNIA

PLATE

B-1

8/28/2009 1:37:21 PM

Date Completed:	**8/13/09**	*Drilling method:*	**8" Hollow Stem Auger**
Logged By:	**N. Berner**	*Hammer Wt:*	**140 lbs., 30" drop**
Total Depth:	**Approximately 20.5 ft**	*Notes:*	**Drilled on Asphaltic-Concrete pavement**

Surface Elevation: **Estimated 38.0 feet (MSL)**

Depth, ft	Sample (FIELD)	Blows/ft (FIELD)	Dry Density pcf (LABORATORY)	Moisture Content % (LABORATORY)	Compress. Strength tsf (LABORATORY)	Other Tests (LABORATORY)	Pen, tsf	DESCRIPTION
								ASPHALT CONCRETE - approximately 2-inches thick
								AGGREGATE BASEROCK - approximately 6-inches thick
		12	99	21.7			1.3-1.8	**SILTY LEAN CLAY (CL)** - olive-gray, moist, firm to hard, medium plasticity, trace fine grained sand
5		16					2.3	- hard
10		17					1.8-2.0	- hard
		12						
15		12	▽ 11:40				0.5-1.5	- carbonate nodules
20		14					0.5	- yellow, wet, soft to firm, increase in carbonate nodules
								Boring terminated at approx. 20.5 feet below ground surface. Backfilled with cement grout
25								
30								

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PROJECT NO. *105356*

LOG OF BORING NO. B-1

MEASURE O CAMPUS ADDITIONS
SAN LORENZO HIGH SCHOOL
16501 ASHLAND AVENUE
SAN LORENZO, CALIFORNIA

PLATE

B-2

Date Completed: 8/13/09

Logged By: N. Berner

Total Depth: Approximately 20.0 ft

Drilling method: 8" Hollow Stem Auger

Hammer Wt: 140 lbs., 30" drop

Notes: Drilled on Asphaltic-Concrete pavement

Surface Elevation: Estimated 38.0 feet (MSL)

Depth, ft	Sample (FIELD)	Blows/ft (FIELD)	Dry Density pcf (LABORATORY)	Moisture Content % (LABORATORY)	Compress. Strength tsf (LABORATORY)	Other Tests (LABORATORY)	Pen, tsf	DESCRIPTION
								ASPHALT CONCRETE - approximately 1-inch thick
								AGGREGATE BASEROCK - approximately 6-inches thick
	■	6	100	19.6			1.0-1.8	**SANDY LEAN CLAY (CL)** - olive-yellow, moist, firm, low plasticity, fine grained sand
5	■	11	94	24.9	1.17 @ 15.0%		1.5	**SILTY LEAN CLAY (CL)** - olive-gray, moist, hard, low to medium plasticity
10	■	10					1.0	- increase in fine grained sand content
			7:37					
	■	18					2.0	- olive, decrease in sand content
15	■	12					1.5	- gray, firm to hard, carbonate traces
20	■	15					2.0	- yellow, wet, firm to hard, trace fine grained sand, carbonate traces
								Boring terminated at approx. 20 feet below ground surface. Backfilled with cement grout
25								
30								

KLEINFELDER
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PROJECT NO. 105356

LOG OF BORING NO. B-2

MEASURE O CAMPUS ADDITIONS
SAN LORENZO HIGH SCHOOL
16501 ASHLAND AVENUE
SAN LORENZO, CALIFORNIA

PLATE

B-3

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8/28/2009 9:06:04 AM

Date Completed: **8/13/09**

Logged By: **N. Berner**

Total Depth: **Approximately 25.0 ft**

Drilling method: **8" Hollow Stem Auger**

Hammer Wt: **140 lbs., 30" drop**

Notes: **Drilled On Soil and Grass**

Surface Elevation: **Estimated 38.0 feet (MSL)**

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								SILTY LEAN CLAY (CL) - yellow, dry to moist, hard, low plasticity, trace fine grained sand, rootlets, porous structure
		12	80	10.6		LL=29; PI=13	1.8	
5		12	95	17.0			4.2	
								- dark brown, moist, firm to hard
10		14					1.4	
			8:30					
15		16					1.5	- gray-brown
		11					1.3-1.5	
20								
								- olive-yellow, soft, iron oxide staining
		18					0.3	
							2.1	- hard, iron oxide staining
25		21						Boring terminated at approx. 25 feet below ground surface. Backfilled with cement grout
30								

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PROJECT NO. 105356

LOG OF BORING NO. B-3

MEASURE O CAMPUS ADDITIONS
SAN LORENZO HIGH SCHOOL
16501 ASHLAND AVENUE
SAN LORENZO, CALIFORNIA

PLATE

B-4

Date Completed: 8/13/09

Logged By: N. Berner

Total Depth: Approximately 25.0 ft

Drilling method: 8" Hollow Stem Auger

Hammer Wt: 140 lbs., 30" drop

Notes: Drilled on Asphaltic-Concrete pavement

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: Estimated 38.0 feet (MSL))
								ASPHALT CONCRETE - approximately 2-inch thick
								AGGREGATE BASEROCK - approximately 6-inches thick
		11	100	20.9	1.65 @ 12.4%	LL=38; PI=22	2.0	**SANDY LEAN CLAY (CL)** - olive-yellow, moist, firm, low plasticity, fine grained sand
5		17					1.0	
10		11					0.5-1.3	- brown, firm
		12					1.5-2.0	- olive
15		13					1.0	
20		17					2.0	- olive-yellow, manganese oxide staining, iron oxide staining, trace fine grained sand
		9:00					0.8	
		35						**SILTY SAND (SM)** - olive-yellow, wet, medium dense to dense, fine grained sand
25								**POORLY GRADED SAND with GRAVEL (SP)** - brown, wet, medium dense, coarse grained sand, fine subrounded gravel
								Boring terminated at approx. 25 feet below ground surface. Backfilled with cement grout
30								

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PROJECT NO. 105356

LOG OF BORING NO. B-4

MEASURE O CAMPUS ADDITIONS
SAN LORENZO HIGH SCHOOL
16501 ASHLAND AVENUE
SAN LORENZO, CALIFORNIA

PLATE

B-5

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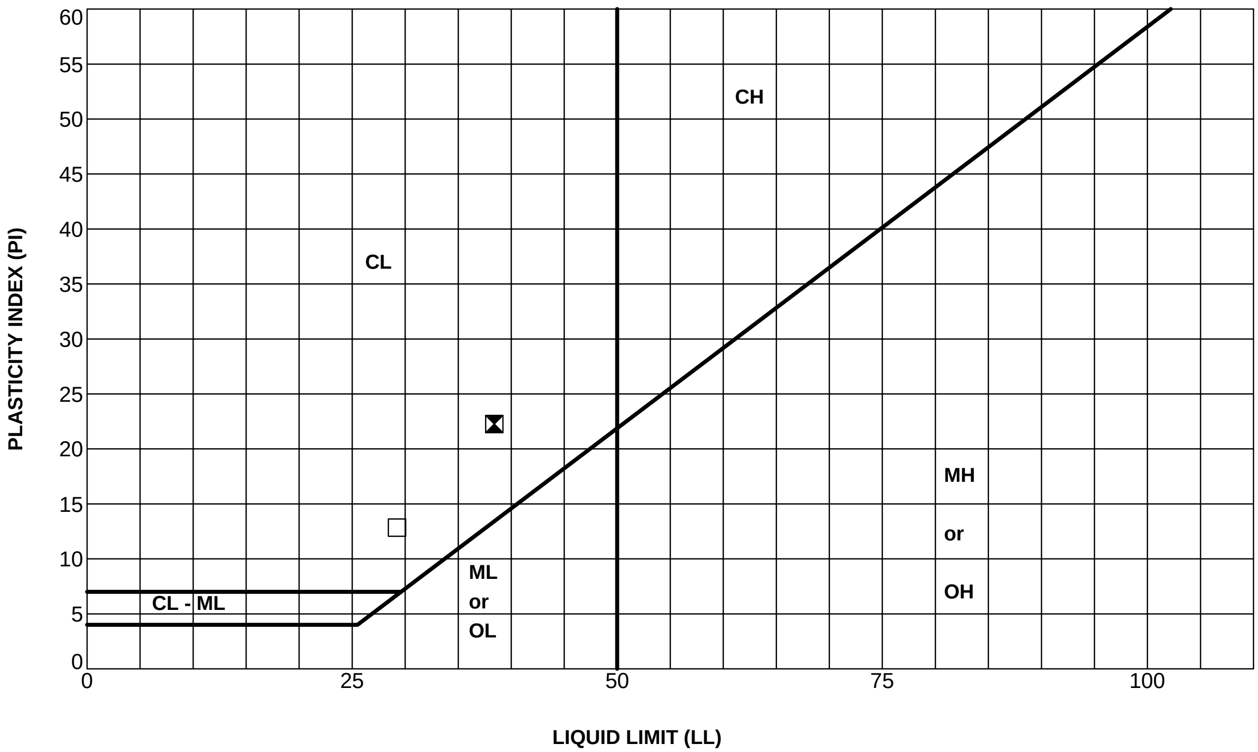
APPENDIX C



SYMBOL	BORING	DEPTH, ft	LL	PL	PI	SAMPLE DESCRIPTION
□	B-3	2.0	29	16	13	Light Olive Brown Sandy Lean Clay (CL)
⊠	B-4	1.5	38	16	22	Very Dark Grayish Brown Sandy Lean Clay (CL)

Unified Soil Classification

Fine Grained Soil Groups

Symbol	LL < 50	Symbol	LL > 50
ML	Inorganic clayey silts to very fine sands of slight plasticity	MH	Inorganic silts and clayey silts of high plasticity
CL	Inorganic clays of low to medium plasticity	CH	Inorganic clays of high plasticity
OL	Organic silts and organic silty clays of low plasticity	OH	Organic clays of medium to high plasticity, organic silts

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318 (DRY PREP)
Pursuant to 2006 IBC Section 1704, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail (meets/does not meet), if provided.

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PROJECT NO. *105356*

ATTERBERG LIMITS*

MEASURE O CAMPUS ADDITIONS
SAN LORENZO HIGH SCHOOL
16501 ASHLAND AVENUE
SAN LORENZO, CALIFORNIA

PLATE

C-1

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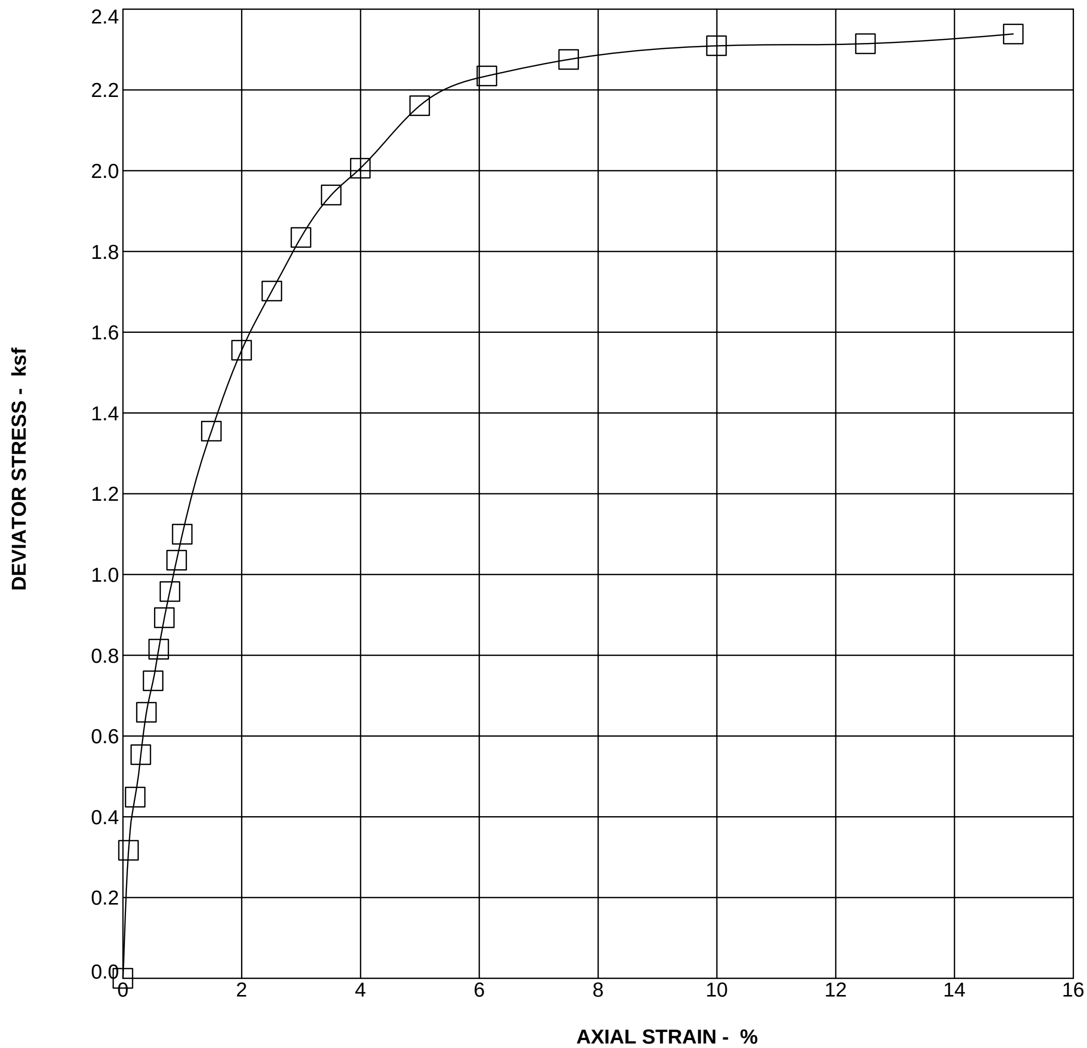

BORING NO.	B-2	**DRY DENSITY - pcf**	94
DEPTH - ft	6.0	**WATER CONTENT - %**	24.9
SAMPLE DESCRIPTION	Dark Grayish Brown Lean Clay (CL)		
CONFINING STRESS - ksf	0.5		

MAXIMUM DEVIATOR STRESS= 2.34 ksf at 15.0 % STRAIN

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850
Pursuant to 2006 IBC Section 1704, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail (meets/does not meet), if provided.

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PROJECT NO. *105356*

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION*
MEASURE O CAMPUS ADDITIONS
SAN LORENZO HIGH SCHOOL
16501 ASHLAND AVENUE
SAN LORENZO, CALIFORNIA

PLATE
C-2

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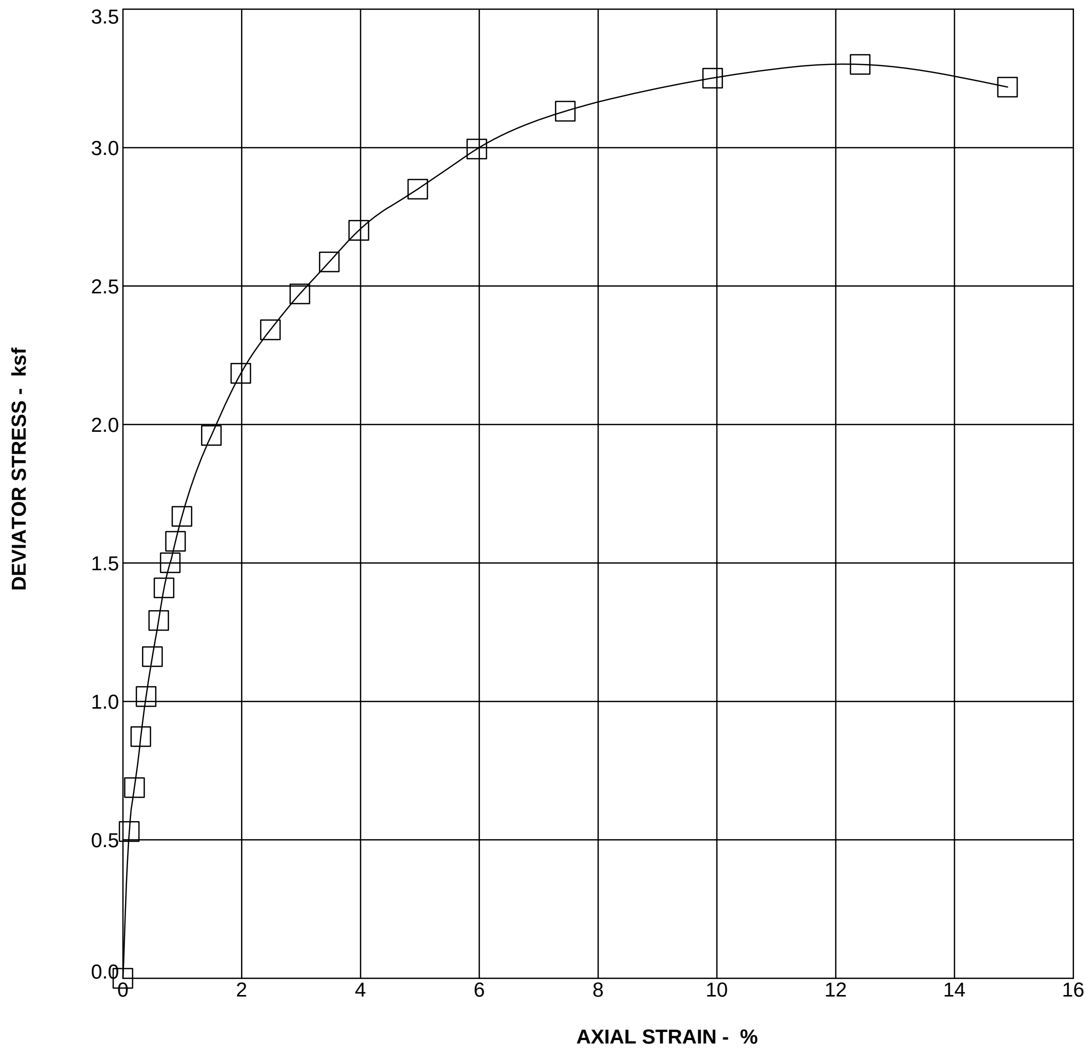

BORING NO.	B-4	**DRY DENSITY - pcf**	100
DEPTH - ft	2.5	**WATER CONTENT - %**	20.9
SAMPLE DESCRIPTION	Dark Grayish Brown Sandy Lean Clay (CL)		
CONFINING STRESS - ksf	0.5		

MAXIMUM DEVIATOR STRESS= 3.30 ksf at 12.4 % STRAIN

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850
Pursuant to 2006 IBC Section 1704, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail (meets/does not meet), if provided.

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PROJECT NO. *105356*

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION*

MEASURE O CAMPUS ADDITIONS
SAN LORENZO HIGH SCHOOL
16501 ASHLAND AVENUE
SAN LORENZO, CALIFORNIA

PLATE

C-3

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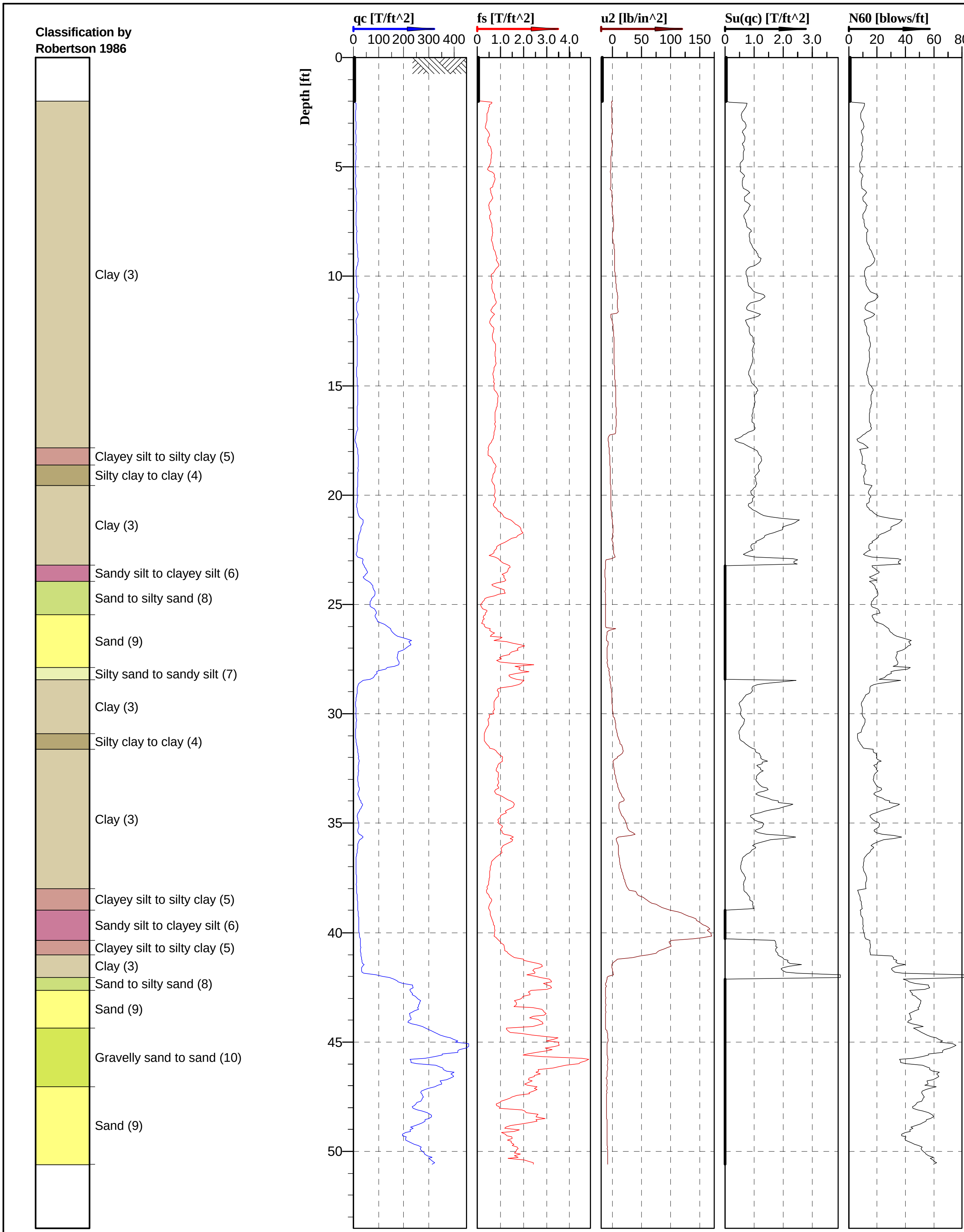
APPENDIX D



U2

Cone No: 3794
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: San Lorenzo, California	Position:	Ground level:	Test no: CPT-1
Project ID: 105356	Client: Kleinfelder	Date: 8/13/2009	Scale: 1 : 75
Project: San Lorenzo High School		Page: 1/1	Fig:
		File: CPT-1.cpd	

CALIFORNIA PUSH
TECHNOLOGIES
INCORPORATED

California Push Technologies Inc.
104 Constitution Drive Suite 2
Menlo Park CA 94025
(C-57 License #884827)

office: 650 854 0300
fax: 650 854 0301

www.cptinc.com

Cone penetration testing and soil sampling methods description.

Rig Description

Our services are based on the state-of-the-art, Geoprobe Model 6625CPT rig, a limited-access, self-anchoring, 20-ton push capacity, track-mounted push platform for dedicated Geotechnical CPT applications with the unique and valuable added ability to quickly perform intermittent or continuous soil sampling.

Weight = ~ 9,500 pounds
Surface load = ~ 4.5 psi
Push capacity = ~ 20 tons; self-anchoring achieved using 10- or 15-inch diameter helical soil anchors driven 4- to 10-feet into the soil
Sampling hammer percussion rate = 32 Hz & 20,000 lbs force/blow
Length = ~ 12 feet; Width = ~ 7 feet
Height (folded) = 7 feet; Height (unfolded) = 14 feet

CPT Description

Our Geoprobe 6625CPT incorporates the Swedish-made Geotech AB Cone Penetration Testing tools which meet the ASTM D-5778 Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils. Cones have 10 cm^2 tips and 150 cm^2 friction sleeves, and include a porous filter and pressure sensor located in the u_2 position directly behind the cone. The cone and porous filter are saturated under vacuum with glycerin to promote rapid equilibration with in-situ pore pressures. Cones are advanced at the ASTM standard rate of 2 cm/second. Baseline readings are performed both before and after each push to check for load cell drift. The cone measures bearing (max load = 100 MPa ~ 1044 TSF), friction sleeve (max load = 1.0 MPa ~ 10.4 TSF), and dynamic pore pressure (max load = 2.5 MPa ~ 363 psi) at 2 cm or 4 cm intervals (client's choice) and this data is plotted in real-time and recorded on a laptop computer adjacent to the push platform. Holes are grouted upon completion of each push, or at the end of each day, as site conditions and regulations warrant.

The basic equation to determine the depth to the free water surface from the pore pressure dissipation test is;

Depth to phreatic surface = [Dissipation depth] – [equilibrium pore pressure / unit weight of H2O x unit conversation factor]

…where;

1) Surface elevation is always assumed to be 0 feet
2) Dissipation depth = the depth (feet) below surface elevation where the cone advancement was paused while waiting for equilibrium pore pressure to be achieved
3) Equilibrium pore pressure = the pore pressure after an elapsed time where no increase or decrease in pore pressure is occurring, in pounds per square inch (psi)
4) Unit weight of water = 62.3 pounds per cubic foot (lb/ft^3)
5) Unit conversion factor (for dimensional analysis): 1 psi = 144 lb/ft^3

From the dissipation plots, simply read the dissipation depth and dissipated pressure for the values to plug into the equation above. On the plots, pore pressure (psi) is on the abscissa and log time (seconds) is on the ordinate.

Sampling Description

Geoprobe® brand Dual Tube Sampling Systems are efficient methods of collecting continuous soil cores with the added benefit of a cased hole. Dual tube sampling uses two sets of probe rods to collect continuous soil cores. One set of rods is driven into the ground as an outer casing (2.2 or 3.25 inches in diameter). These rods receive the driving force from the hammer and provide a sealed hole from which soil samples may be recovered without the threat of cross contamination. The second, smaller set of rods are placed inside the outer casing. The smaller rods hold a sample liner in place as the outer casing is driven one sampling interval. The small rods are then retracted to retrieve the filled liner. Soil samples are collected in 1.85-inch diameter or 1.125-inch diameter clear PVC sample sheaths.

Interpretations

Soil behavior type (SBT), SPT N60 energy ratio, undrained shear strength, OCR, and unit weights are calculated and/or are interpretations generated by the CPT-Pro software based on empirical relationships derived in the following references;

P.K. Robertson, R.G. Campanella, D. Gillespie, and J. Greig, 1986, Use of Piezometer Cone Data, Proceedings of the ASCE Specialty Conference In Situ '86: Use of In Situ Tests in Geotechnical Engineering; pp. 1263-1280.

P.K. Roberston, 1990, Soil Classification Using the Cone Penetration Test, Canadian Geotechnical Journal, 27(1), pp. 151-158.

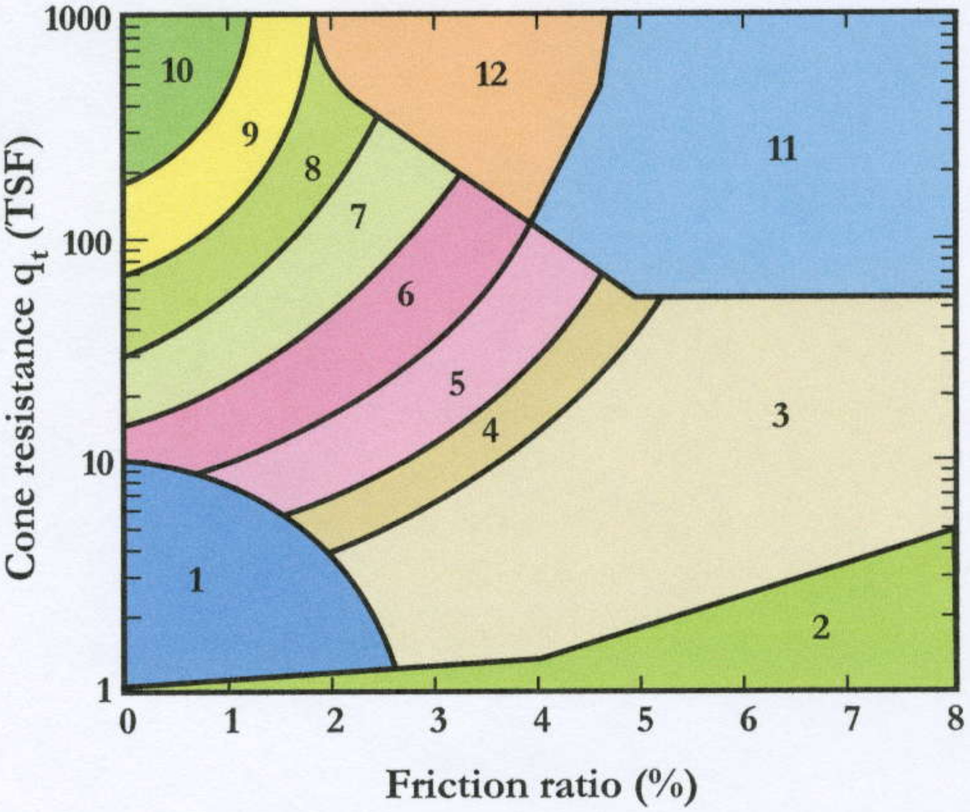
T. Lunne, P.K. Robertson, and J.J.M. Powell, 1997, Cone Penetration in Geotechnical Practice, Taylor and Francis Publishing.

CPT Inc. makes no recommendation on which soil behavior type analysis is "most-correct". The engineer should be aware of the limitations of using CPT data to derive soil behavior type and other engineering parameters and is encouraged to review the above references to better understand the applicability and limitations of CPT data. It is sometimes not possible to determine soil type based solely on tip resistance, sleeve friction, and dynamic pore pressure response, and confirmatory samples may be required.

Please do not hesitate to contact CPT Inc. if you have questions.

Sincerely,
John Rogie

President
California Push Technologies, Inc.



Zone	Soil Behavior Type
1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (overconsolidated or cemented)
12	sand to clayey sand (overconsolidated or cemented)

Source: Robertson, P.K., Campanella, R.G., Gillespie, D., and Greig, J., 1986, Use of Piezometer Cone Data. Proceedings of the ASCE Specialty Conference In Situ 86: Use of In Situ Tests in Geotechnical Engineering.

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Soil Behavior Type (SBT) Model

GEOLOGISMIKI
Geotechnical Software

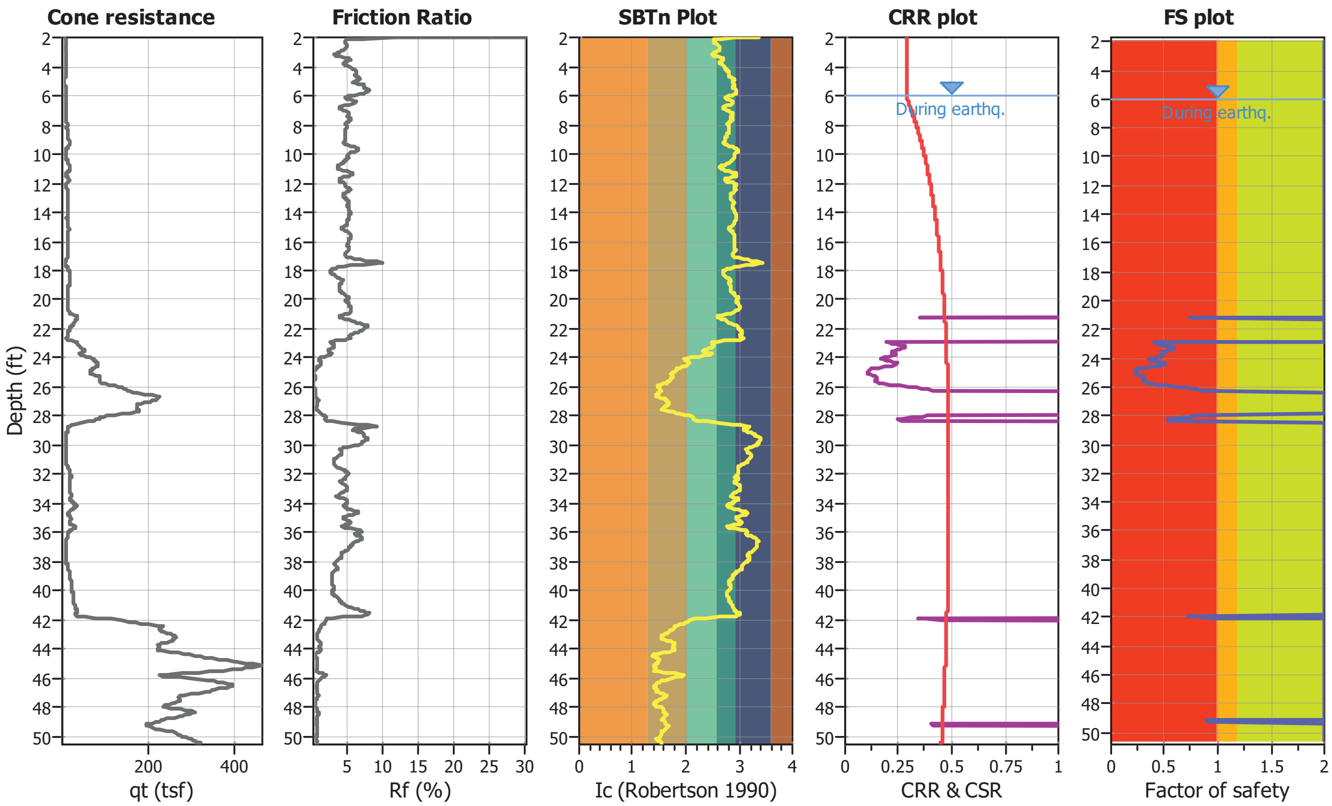
LIQUEFACTION ANALYSIS REPORT

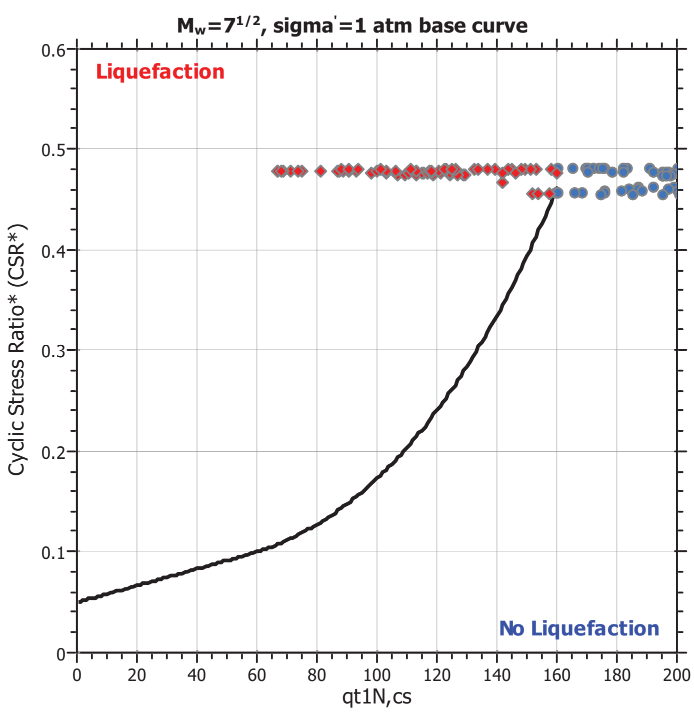
Project title : SLZHS Measure O Additions **Location :**

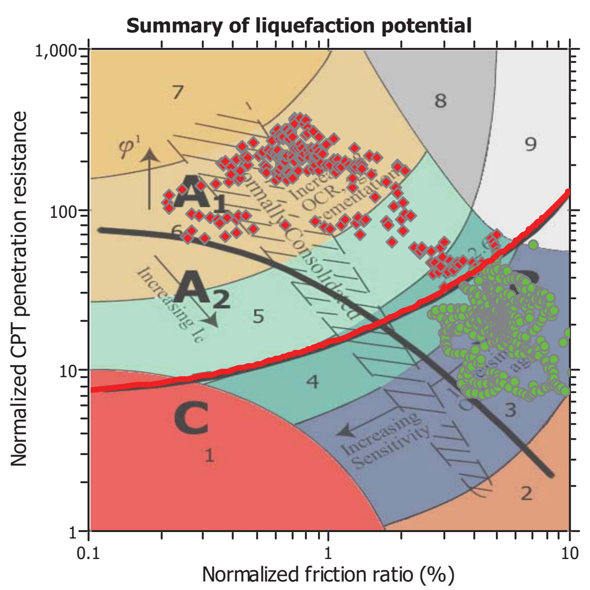
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER 1998	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	No
Fines correction method:	Robertson & Wride	G.W.T. (earthq.):	6.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.55	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

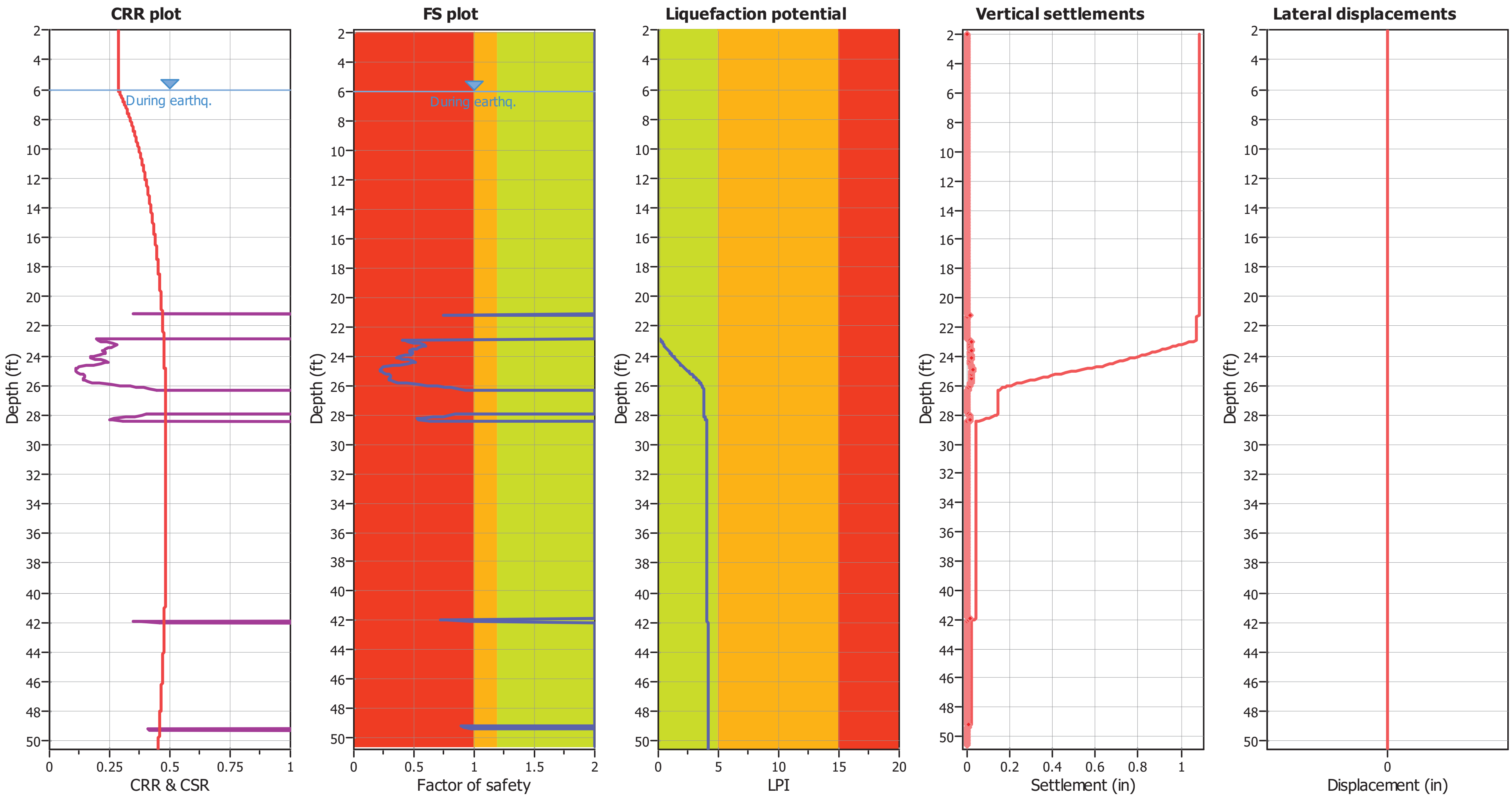






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER 1998	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	Robertson & Wride	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	No
Peak ground acceleration:	0.55	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

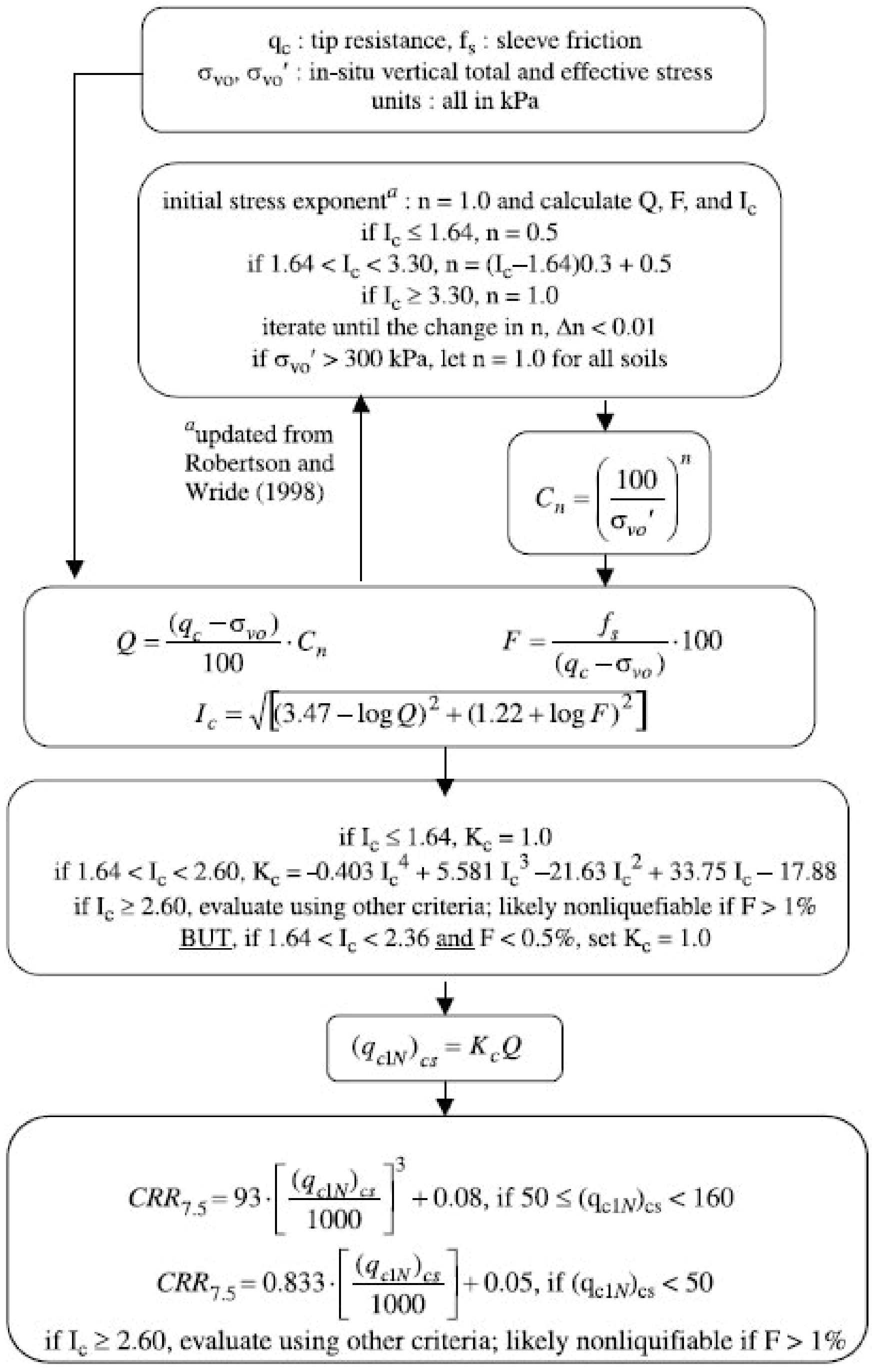
- Liquefied
- Marginally liquefied
- Non-liquefied

LPI color scheme

- Very high risk
- High risk
- Low risk

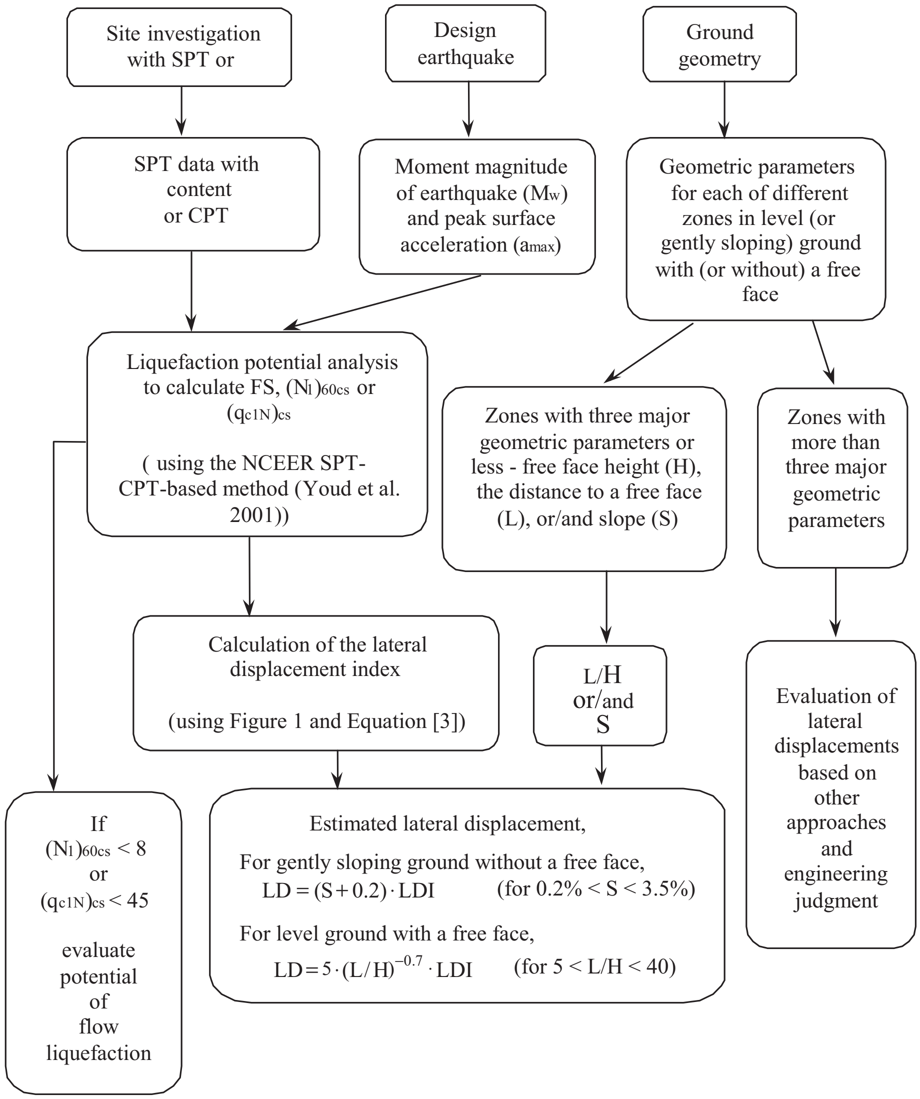
Procedure for the evaluation of soil liquefaction resistance

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:

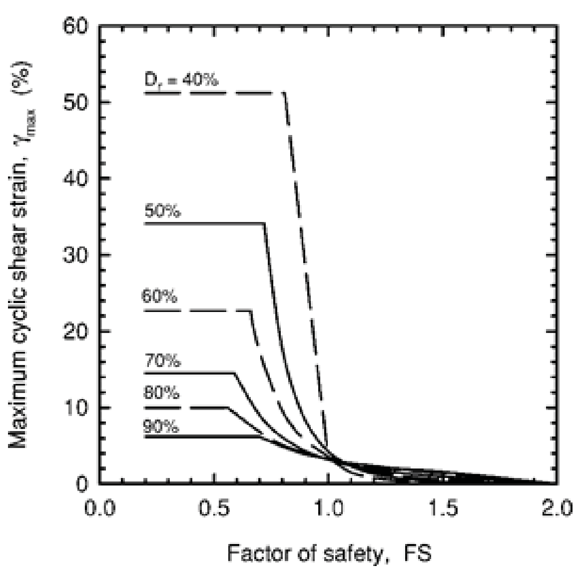


[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



[1] **Figure 1**

$$LDI = \int_0^{Z_{max}} \gamma_{max} \, dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

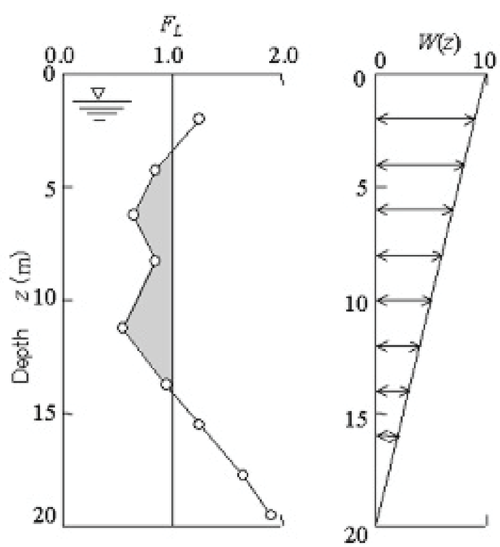
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_z) \times F_L \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

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APPENDIX E

California State Certified Laboratory No.2153

CERCO

analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel: 925.462.2771
Fax: 925.462.2775

25 January, 2006

Job No.0601061
Cust. No.10527

Ms. Carrie Foulk
Kleinfelder
7133 Koll Center Parkway
Pleasanton, CA 94566

Subject: Project No.: 64583/PWGEO
Project Name: San Lorenzo HS
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Foulk:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on January 09, 2006. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration ranges from none detected to 19 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration reflects none detected with a detection limit of 15 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 7.5 to 7.6 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials range from 480 to 490-mV, which are indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO Analytical, Inc.

3942-A Valley Avenue, Pleasanton, CA 94566-4715 (925) 462-2771 Fax (925) 462-2775

FINAL RESULTS

Client:	Kleinfelder
Client's Project No.:	64583/PWGEO
Client's Project Name:	San Lorenzo HS
Authorization:	P.O. No.R9373

Date Sampled:	Not Indicated
Date Received:	9-Jan-2006
Date of Report:	25-Jan-2006
Matrix:	Soil

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0601061-001	B-1,2B @ 2.5'	480	7.5	-	2,600	-	19	N.D.
0601061-002	B-3,2B @ 5.5'	490	7.6	-	2,800	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	13-Jan-2006	16-Jan-2006	-	23-Jan-2006	-	16-Jan-2006	16-Jan-2006

Cheryl McMillen
Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits