Prepared for **San Lorenzo Unified School District**

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GEOLOGIC AND SEISMIC HAZARDS ASSESSMENT REPORT
SAN LORENZO HIGH SCHOOL CAMPUS
50 EAST LEWELLING BOULEVARD
SAN LORENZO, CALIFORNIA

August 29, 2009
File No.: 105356

KLEINFELDER
Bright People. Right Solutions.

August 29, 2009
File No.: 105356

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

SUBJECT: Geologic and Seismic Hazards Assessment Report for San Lorenzo High School Campus located at 50 East Lewelling Boulevard in San Lorenzo, California

Dear Mr. Estrada:

Kleinfelder is pleased to submit our geologic and seismic hazard assessment report for San Lorenzo High School campus in San Lorenzo, California. This report has been prepared to address the entire school campus, and can be used for future improvements and additions as well, provided that appropriate updates to the information presented herein are performed. This study was conducted concurrently with a geotechnical investigation, which will be presented in a separate report. This geologic and seismic report is based on our previous geologic hazards assessment and geotechnical reports prepared for the school in February 2006.

This report has been prepared in accordance with the scope of work described in our proposal to you dated July 10, 2009 (File No. 01002PROP/PLE9P196). The enclosed report provides a description of our field reconnaissance and investigation, geologic and seismic literature review results, and our assessment of potential seismic and geologic hazards that can adversely affect this site.

Based on the results of our assessment, it is our opinion that the site is suitable for the proposed improvements, from a geological viewpoint, provided that our conclusions and recommendations presented herein are adhered to and incorporated into the design and construction of the proposed development. The primary geological issues of concern are the potential for strong ground shaking as a result of future seismic events along one of the Bay Area active earthquake faults and the potential for underlying layers of sand to silt to liquefy during a major seismic event.

The conclusions and recommendations presented in this report are based upon a limited site reconnaissance, surficial mapping, review of published geologic and seismic

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literature, and subsurface data obtained during our previous and concurrent geotechnical investigations at the campus. These investigations included a total of eight soil borings (to depths ranging from 20 to 50 feet) and two Cone Penetration Tests (CPT) advanced to a depth of 50 feet.

We appreciate the opportunity of providing our services to the San Lorenzo Unified School District on this project and trust that this report meets your needs at this time. If you have any questions concerning the information presented, please contact this office at (925) 484-1700.

Sincerely,

KLEINFELDER, INC.

for

Jorge Meneses, Ph.D, P.E. # 72544
Project Engineer

David C. Seymour, PG, CEG #1574
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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
The Best People on Earth

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GEOLOGIC AND SEISMIC HAZARDS ASSESSMENT REPORT
SAN LORENZO HIGH SCHOOL CAMPUS
50 EAST LEWELLING BOULEVARD
SAN LORENZO, CALIFORNIA

1. INTRODUCTION

This report presents the results of Kleinfelder's geologic and seismic hazards assessment for San Lorenzo High School, located at 50 East Lewelling Boulevard in San Lorenzo, California. The location of the site is approximately shown on the Site Vicinity Map, Plate 1.

This report has been prepared for submittal with supporting design documents to the Division of the State Architect, as required for new construction of public schools and essential services buildings. This report is intended to be an Engineering Geology and Seismology Report for the entire school campus as required by the 2007 California Building Code (CBC) and may be incorporated into future projects with appropriate updates of the information presented herein. As a minimum, the updates should include site-specific borings/Cone Penetrometer Tests (CPTs) and reconnaissance for individual projects, and evaluation of that data to confirm that it is consistent with this report.

Kleinfelder performed a geotechnical investigation at the school campus concurrently with this geologic and seismic hazards assessment. The concurrent geotechnical investigation was performed for a proposed digital arts building and classroom addition, which will be located in the northeast portion of the campus (see Plate 2). In addition to the concurrent study, Kleinfelder also performed other investigations and geologic hazard evaluations for the campus in 2006 (see References). Since that time, the CBC has been modified, most recently in 2007. Many of the modifications are related to seismic design parameters, which have updated for this study in accordance with the 2007 CBC.

1.1 Site Location

San Lorenzo High School is located at 50 East Lewelling Boulevard in San Lorenzo, California. The location is encompassed on the east by Ashland Avenue, on the west by Union Pacific Railroad, and on the north by Interstate 238. Based on the U. S. Geological Survey (USGS, 1993) 7½-Minute Hayward Topographic Quadrangle Map, the existing ground elevation at the site is about 35 to 45 feet above Mean Sea Level. The coordinates at the center of the site are approximately:

Latitude: 37.6884° N Longitude: 122.1223° W

1.2 Purpose and Scope of Services

The purpose of this report is to identify and assess potential geologic and seismic hazards at the site in accordance with the requirements for such studies set forth by the California Education Code (Chapter 1, Section 39002) and the California Code of Regulations, Title 24, 2007 California Building Code (CBC). In addition to these documents, this report is prepared in accordance with the guidelines established in the following documents:

- California Department of Conservation, California Geological Survey (CGS, previously known as the California Division of Mines and Geology [CDMG]), Special Publication 117 (*Guidelines for Evaluating and Mitigating Seismic Hazards*);
- CDMG Special Publication 42 (*Fault-Rupture Hazard Zones in California*);
- CDMG Note 42 (*Guidelines to Geologic/Seismic Reports*);
- CDMG Note 44 (*Recommended Guidelines for Preparing Engineering Geologic Reports*); and
- CGS Note 48 (*Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings*).

Specifically, our scope of services as outlined in our proposal dated July 10, 2009 (File No. 01002PROP/PLE9P196):

- Research and review of available geologic, geotechnical, and seismologic publications and maps covering the site and vicinity;

- A geologic reconnaissance of the site by a Certified Engineering Geologist (CEG) to observe and document pertinent surface features indicative of possible geologic hazards;
- Discussion of significant faults and assessment of site seismicity;
- Estimation of peak horizontal ground surface accelerations and ground motion parameters for the Maximum Considered Earthquake (MCE) and the Design Earthquake (DE). MCE is defined as the lesser of the 5 percent damped spectral response accelerations obtained from a ground motion having a 2% probability of exceedance in a 50-year period (return period of about 2,475 years) or the deterministic ground motion (the greater of the 150 percent of the median deterministic values obtained from the controlling faults or the deterministic lower limit per ASCE 7-05, Section 21.2.2). The DE is defined as 2/3 of the MCE, but cannot be taken as less than 80 percent of the Code Spectrum obtained from ASCE 7-05, Section 11.4.5;
- Analysis based on our review of the subsurface data obtained during our concurrent geotechnical engineering investigation; and
- Evaluation of the researched data and preparation of this written report with conclusions regarding possible geologic and seismic hazards affecting the site and the proposed project.

The references reviewed for compilation of this report are listed in the "References" section of this report. We have utilized the subsurface information obtained from our soil borings drilled as part of our concurrent geotechnical engineering investigation in developing our conclusions for this study. Our observations and conclusions presented herein specifically exclude the assessment of environmental characteristics, particularly those involving hazardous substances.

2. GEOLOGIC SETTING

2.1 Regional Geology

The San Francisco Bay Area lies within the Coast Range geomorphic province, a series of discontinuous northwest trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The general geologic framework of the San Francisco Bay Area is illustrated in studies by Schlocker (1970), as well as in studies by Wagner et al. (1991), Chin et al. (1993), Helley & Graymer et al. (1997), and Graymer et al. (1996 and 2000).

Geologic and geomorphic structures within the San Francisco Bay Area are dominated by the San Andreas fault (SAF), a right-lateral strike-slip fault that extends from the Gulf of California in Mexico, to Cape Mendocino, on the coast of Humboldt County in northern California. It forms a portion of the boundary between two independent tectonic plates on the surface of the earth. To the west of the SAF is the Pacific plate, which moves north relative to the North American plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF; however, it is also distributed, to a lesser extent across a number of other faults that include the Hayward, Calaveras, and Concord among others. Together, these faults are referred to as the SAF system. Movement along the SAF system has been ongoing for about the last 25 million years. The northwest trend of the faults within this fault system is largely responsible for the strong northwest structural orientation of geologic and geomorphic features in the San Francisco Bay Area. Currently, active compressional forces normal to the northwest structural trend of the Coast Range province are also partially responsible for the strong northwest structural trend and uplift of the mountains within the province (Brown 1990). These compressional forces are responsible for the movements associated with the Great Valley fault system, a series of blind (no surface expressions of the faults are evident) thrust faults along the eastern margin of the Coast Range province and folding of the younger rocks within the region. Regional faulting and seismicity are discussed in detail in Section 3.0 of this report.

Basement rocks west of the SAF are generally granitic, while to the east it consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (205 to 65 million years old). Overlying the basement rocks are Cretaceous (about 140 to 65 million years old) marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted as a result of Late Tertiary and Quaternary regional compressional forces. The inland valleys as well as the structural depression within which the San Francisco Bay is located are filled with unconsolidated to semi-consolidated deposits of Quaternary age (about the last 1.6 million years). Continental surficial deposits (alluvium, colluvium, and landslide deposits) consist of unconsolidated to semi-consolidated sand, silt, clay, and gravel while the Bay deposits typically consist of very soft organic rich silt and clay (Bay mud) or sand. The regional geologic conditions of this portion of California are depicted on Plate 3.

2.2 Area and Site Geology

The project site is situated on an alluvial plain that lies between the eastern shore of San Francisco Bay and the East Bay Hills of the Diablo Range. Several researchers have mapped the geology in the vicinity of the school campus including Robinson (1956), Dibblee (1980), Graymer et al. (1996), Knudsen et al. (1997), and Graymer (2000). Based on these maps, Mesozoic and Cenozoic bedrock formations underlie the nearby East Bay Hills and are composed of various types of igneous, metamorphic and sedimentary rocks. Geologic structures within the foothills trend to the northwest, and are strongly influenced by the active Hayward fault, which transects the western boundary of the hills.

Localized studies by the United States Geological Survey (USGS) that describe the Quaternary alluvial and Bay deposits in the vicinity of the school include Helley et al. (1979), Helley and Graymer (1997), Knudsen et al. (1997), and Witter et al. (2006). Based on Knudsen et al. (1997), from which the Area Geologic Map (Quaternary Units), Plate 4 is derived, the northeastern and northwestern portions of the campus are underlain by Holocene age alluvial fan deposits, while Holocene age natural levee deposits underlie the southern portion. Knudsen et al. (1997) defines alluvial fan deposits as sediment deposited by streams from adjacent mountain and hillside canyons onto adjoining valley floors and plains. These sediments generally vary in composition from poorly graded sand to silt and clay and generally decrease in grain size down gradient of the top, or apex, of the fan. Natural levee deposits commonly border stream channels and are composed of fine-grained deposits of silt and clay with variable amounts of sand.

Plate 5, Area Geologic Map (Bedrock Units) is based on Graymer et al. (1996). Their mapping includes their interpretation of other regional geologic maps, most notably those by Robinson (1956) and Dibblee (1980). According to Graymer et al. (1996), the western slopes of the foothills expose Jurassic age deposits of keratophyre and gabbro,

2.3 Aerial Photographic Review

Stereoscopic pairs of historical aerial photographs (1947 through 1996) reviewed for the site reveal that several changes have occurred over the past fifty years. The 1947 and 1957 photographs indicate that the school campus was constructed sometime before 1957. Prior to construction of the campus, the site was part of a rural residential area, with portions of site under cultivation. By 1957 the school campus was constructed as well as a commercial facility in the northeast portion of the site, where the proposed digital arts building is proposed. From 1957 through 1996, further development and urbanization of the surrounding vicinity occurred with little change to the campus itself. The aerial photographic review revealed no linear offsets, distinct tonal lineaments, scarps or slides, erosion rills or distinct drainage patterns. The results of our aerial photographic review are in general agreement with the published reports and on-site geologic reconnaissance.

2.4 Site Reconnaissance

A Certified Engineering Geologist (CEG) with our firm performed a site reconnaissance of the site on February 13, 2006. During our reconnaissance, we observed the relatively flat surface area to be underlain by clayey soils that displayed shrinkage cracking implying that the near-surface soils may be expansive. The site is generally level with numerous structures and sports fields. No slopes or open creek channels were observed in the immediate vicinity of the school campus.

2.5 Subsurface Interpretation

The site is underlain by alluvial soils that vary in composition laterally and with depth. Our interpretation of the subsurface geologic conditions is limited to the exploratory borings and CPTs performed at the campus during our concurrent and previous geotechnical investigations. The points of exploration are located in the northeastern and southeastern portions of the campus (see Plate 2). In general, the underlying alluvial sediments are composed of silt and clay-rich deposits with occasional layers of fine-grained sand with variable amounts of gravel. In localized areas, the alluvium is overlain by one to two feet of artificial fill of variable composition. The silt and clay layers within the alluvium vary from firm to hard and are of low to medium plasticity. Interbedded within the silt and clay layers are occasional layers of loose to medium dense fine-grained sand with variable amounts of silt and gravel. The thickest sand layers were encountered in CPT-1 (2009) at depths of approximately 24 and 42 feet, with thickness of about 4 feet and over 8 feet, respectively. A geologic cross-section depicting the general subsurface conditions is included as Plate 6, which generally typifies the subsurface conditions across the school campus. The Quaternary age alluvial units are shown undivided on the cross-section. Logs of the borings and CPTs performed for our concurrent geotechnical investigation are included in Appendix A.

Free groundwater was encountered in each of the borings, varying from about 6 to 22 feet below the ground surface. According to the seismic hazard zone report prepared by the California Geological Survey (CGS) for the Hayward 7.5-Minute Quadrangle (CGS, 2003), the historical high groundwater depth for the campus is between 5 to 10 feet.

The above is a brief description of the subsurface soil encountered in our borings and CPTs. For a more detailed description of the site subsurface conditions, refer to the appended boring and CPT log sheets in Appendix A.

3. FAULTING AND SEISMICITY

3.1 Faulting

Based on the information provided in Bryant and Hart (2007) and CGS (DMG, 1993 and 2000), the site is not located within a State-designated, Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required and no known active faults traverse the site. The site area is situated within a region traditionally characterized by numerous active faults and moderate to high seismic activity.

An active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time (Hart and Bryant, 1997). Faults considered to be active are shown in orange or red on the Regional Fault Map, Plate 7 (Jennings, 1994). The definition of “potentially active” varies. A generally accepted definition of “potentially active” is a fault showing evidence of displacement that is older than 11,000 years (Holocene age) and younger than 1.6 million years (Pleistocene age). These “potentially active” faults are shown in green or purple on Plate 7. However, “potentially active” is no longer used as criteria for zoning by the CGS. The terms “sufficiently active” and “well-defined” are now used by the CGS as criteria for zoning faults under the Alquist-Priolo Earthquake Fault Act. A “sufficiently active fault” is a fault that shows evidence of Holocene surface displacement along one or more of its segments and branches, while a “well-defined fault” is a fault whose trace is clearly detectable by a trained geologist as a physical feature at or just below the ground surface. The definition “inactive” generally implies that a fault has not been active since the beginning of the Pleistocene Epoch (older than 1.6 million years old).

Locations of the significant active and potentially active faults are shown on Plate 7. The school site is located approximately 2.2 kilometers (km) to the southwest of the Hayward – Rodgers Creek fault, 15 km to the southwest of Calaveras fault, and 22 km to the southwest of the Mount Diablo Thrust. A major seismic event on these or other nearby faults may cause substantial ground shaking at the site.

3.2 Seismic Source Model

Our seismic model is based on the seismic source model used in developing probabilistic seismic hazard maps by CGS for the State of California (Cao and others, 2003) and by the Working Group on California Earthquake Probabilities (2003) for the San Francisco Bay Area. We have used faults within 200 km of the site in our analyses. However, faults within only 100 km and their seismic parameters are listed in Table 3.2-1. The locations of the faults and associated parameters presented on Table 3.2-1 are based on data presented by, Jennings (1994), Wakabayashi and Smith (1994), Frankel and others (1996, 2002), Petersen and others (1996), ICBO (1998), Cao and others (2003), and the Working Group on California Earthquake Probabilities (2003). The maximum earthquake magnitudes presented in this table are based on the moment magnitude scale developed by Kanamori (1977).

TABLE 3.2-1: SIGNIFICANT FAULTS

Fault Name	Fault Length (km)	Closest Distance to Site (km)	Magnitude of Maximum Earthquake *	Slip Rate (mm/yr)	Recurrence Interval (yr)
Hayward – Rodgers Creek (HS + HN + RC)	150	2.2	7.2	9	3524
Calaveras (CS + CC +CN)	123	15	6.9	6 – 15	1555
Mount Diablo Thrust	25	22**	6.6	2	389
Concord – Green Valley (CON + GVS + GVN)	56	26	6.7	4 – 5	580
San Andreas (SAS + SAP + SAN + SAO)	473	27	7.9	17 – 24	378
Monte Vista–Shannon	45	30	6.7	0.4	2410
Greenville (GS + GN)	51	33	6.9	2	1994
San Gregorio (SGS + SGN)	176	38	7.4	3 – 7	1202
Great Valley (segment 7)	45	47	6.7	1.5	622
Great Valley (segment 5)	28	47	6.5	1.5	501
West Napa	30	54	6.5	1	701
Great Valley (segment 4)	42	66	6.6	1.5	472

Fault Name	Fault Length (km)	Closest Distance to Site (km)	Magnitude of Maximum Earthquake *	Slip Rate (mm/yr)	Recurrence Interval (yr)
Zayante-Vergeles	58	68	7.0	0.1	8821
Point Reyes	47	68	7.0	0.3	3503
Great Valley (segment 8)	41	83	6.6	1.5	483
Monterey Bay – Tularcitos	84	85	7.3	0.5	2841
Hunting Creek-Berryessa	60	85	7.1	6	194
Ortigalita	70	87	7.1	1	1153

* *Moment magnitude*: An estimate of an earthquake's magnitude based on the seismic moment (measure of an earthquake's size utilizing rock rigidity, amount of slip, and area of rupture).
** Closest horizontal distance to the vertical projection of the potential rupture.

According to the Working Group on California Earthquake Probabilities (2003) study, characterizations of the Calaveras, Concord-Green Valley, Greenville, Hayward-Rodgers Creek, San Andreas, and San Gregorio faults are based on the following fault rupture segments and fault rupture scenarios.

- The Calaveras fault includes three segments and six rupture scenarios, plus a floating earthquake. The three segments are southern (CS), central (CC), and northern (CN).
- The Concord-Green Valley fault has been characterized by three segments and six rupture scenarios plus a floating earthquake. The three segments are the Concord fault (CON), the Green Valley South (GVS), and the Green Valley North (GVN).
- The Greenville fault has been characterized by two segments and three rupture scenarios plus a floating earthquake. The two segments are Greenville South (GS) and Greenville North (GS).
- The Hayward-Rodgers Creek fault has been characterized by three segments and six rupture scenarios plus a floating earthquake. The three segments are the Rodgers Creek fault (RC), the Hayward North (HN), and the Hayward South (HS).

- The San Andreas fault has been characterized by four segments and nine rupture scenarios, plus a floating earthquake. The four segments are Santa Cruz Mountains (SAS), North Coast (SAN), Peninsula (SAP), and Offshore (SAO).
- The San Gregorio fault has been characterized by two segments and three rupture scenarios, plus a floating earthquake. The two segments are San Gregorio South (SGS) and San Gregorio North (SGN).

The recurrence intervals for these faults are listed in Table 3.2-1 and represent a scenario of rupturing all the segments. Recurrence intervals for other scenarios can be found in the Working Group on California Earthquake Probabilities (2003).

3.3 Magnitude-Frequency Distribution

The earthquake probabilities for the faults and their segments were developed using a magnitude-frequency relationship derived from the seismicity catalogs and the fault activity based on their slip rates. In general, there are two models based on magnitude-frequency relationships. In the first, earthquake recurrence is modeled by a modified form of the Gutenberg-Richter (G-R) (Gutenberg and Richter, 1956) magnitude-frequency relation given by:

$$\log N = a - bM$$

where N(M) is the cumulative number of earthquakes of magnitude "M" or greater per year, and "a" and "b" are constants based on recurrence analyses. The relation is truncated at the maximum earthquake. In the G-R model, it is assumed that seismicity along a given fault or fault zones satisfies the above equation. This model generally implies that seismic events of all sizes occur continually on a fault during the interval between the occurrences of the maximum expected events along the fault zone.

The second model, generally referred to as a Characteristic model (Schwartz and Coppersmith, 1984), implies that the time between maximum size earthquakes along particular fault zones or fault segments is generally quiescent except for foreshocks, aftershocks, or low level background activity.

Wesnousky (1994) has suggested that for well defined seismic sources and for practical purposes, the Characteristic model is more appropriate. In the development of the Seismic Hazard Maps for the State of California (Petersen and others, 1996, Cao and others, 2003), the CGS categorizes the faults into two classes and applies different magnitude-frequency statistical distributions for each class. Class A faults generally have slip rates greater than 5 mm/yr and well constrained paleoseismic data (i.e., the San Andreas, San Jacinto, Elsinore, Imperial, Hayward, and Rodgers Creek faults). Class B faults include all the other faults lacking paleoseismic data necessary to constrain the recurrence intervals of large events. They use the Characteristic model for class A faults, and both the Characteristic and G-R models with 0.67 and 0.33 weights, respectively, for class B faults.

We have used the CGS approach in our analyses. A b-value of 0.8 is used for all the faults in California. The most likely a-values were estimated for each seismic source based on the recurrence rates of earthquakes and events per year associated with that seismic source as reported by Petersen and others (1996) and Cao and others (2003).

3.4 Background Seismicity

In addition to the individual seismogenic sources, our seismic analysis also includes background seismicity, which accounts for random earthquakes between M5 and M7 based on the methodology described by Frankel and others (1996, 2002). Some of the local seismic sources are not included in our analysis as independent seismogenic sources because they were not considered by the CGS as independent seismogenic sources during the development of hazard maps for California. However, the seismicity of these faults was incorporated into our analysis by including background seismicity in our model. The a-values are calculated using the method described in Weichert (1980). The hazard may then be calculated using this a-value, a b-value of 0.8 minimum and maximum magnitudes of M5 and M7, respectively, and by applying an exponential distribution as described by Hermann (1977).

3.5 Historical Seismicity

The project site and its vicinity are located in an area traditionally characterized by moderate to high seismic activity. A number of large earthquakes have occurred within the site vicinity during historic time (since 1800). Some of the significant regional earthquake events include: the 1868 (M7.0) Hayward earthquake, located approximately 2.5 km to the northeast; the 1838 (M7.0) San Francisco Peninsula earthquake, located about 26 km to the southwest, the 1955 (M5.4) earthquake near the Concord-Green Valley fault, located approximately 31 km to the northeast of the site, the 1980 (M5.8) Livermore earthquake, located about 34 km to the northeast; and the 1906 (M7.9) San Francisco earthquake, located approximately 34 km to the west of the site. Other significant regional earthquakes include: the 1889 (M6.3) Antioch earthquake, located about 40 km to the east; the 1911 (M6.5) Calaveras Fault earthquake, located about 59 km to the southeast, the 1898 (M6.5) Mare Island earthquake, located approximately 62 km to the northwest, the 1892 (M6.5) Vacaville earthquake, located about 79 km to the north, and the 1892 (M6.3) Winters earthquake, located approximately 92 km to the northeast of the site.

A publication prepared by the U.S. Geological Survey regarding earthquake probabilities in the Bay Area (Working Group on California Earthquake Probabilities, 2003) concludes that there is a 62 percent chance that one of the major faults within the Bay Area will experience a major (M6.7+) earthquake during the period of 2003-2032. As has been demonstrated recently by the 1989 M6.9 Loma Prieta earthquake, the 1994 M6.7 Northridge earthquake, and the 1995 M6.9 Kobe earthquake, earthquakes of this magnitude range can cause severe ground shaking and significant damage to modern urban areas.

Epicenters of some significant earthquakes ($M \geq 4.0$) within the vicinity of the site are shown on Plate 8. The earthquake database used in our search contains in excess of 5,500 seismic events and covers the period from 1800 through June 2009. The earthquake database is primarily comprised of an earthquake catalog for the State of California prepared by the CGS. The original CGS catalog (Real and others, 1978) is a merger of the University of California at Berkeley and the California Institute of Technology instrumental catalogs (Hileman and others, 1973). The combined catalog contains earthquake records from January 1, 1900 through December 31, 1974. Updates prepared by the CGS in 1979 and 1982 extend the coverage through 1982. In addition to the CGS updates, the data for earthquakes that occurred during the period between 1910 through June 2009 has been obtained from a composite catalog by the Advanced National Seismic System (ANSS). The ANSS catalog is a worldwide earthquake catalog which is created by merging the master earthquake catalogs from contributing ANSS member networks and then removing duplicate events, or non-unique solutions from the same event. The ANSS network includes the Northern and Southern California Seismic Networks, the Pacific Northwest Seismic Network, the University of Nevada, Reno Seismic Network, the University of Utah Seismographic Stations, and the United States National Earthquake Information Service. The earthquake database also consists of earthquake records between 1800 and 1900 from Seeburger and Bolt (1976) and Toppozada and others (1978, 1981). In addition, we have also utilized the data from DMG Map Sheet 49 (Toppozada and others, 2000).

The parameters used to define the limits of the historical earthquake search include geographical limits (within 100 km of the site), dates (1800 through June 2009), and magnitudes ($M \geq 4$). A summary of the results of the historical search is presented below.

Time Period (1800 to June 2009)	209+ years
Maximum Magnitude*	7.9
Approximate distance to nearest historical $M \geq 4$ earthquake	2 km
Number of events exceeding magnitude 4 within search area	251

*Moment magnitude

3.6 Site Class

In developing site-specific ground motions, the characteristics of the soils underlying the site are an important input to evaluate the site response at a given site. Based on the preliminary boring logs from our recent geotechnical investigation at the school campus, the site is underlain by clayey, silty, and sandy soils. The silty and sandy layers below groundwater are potentially liquefiable. Historic ground water table was shallower than what we encountered during our subsurface investigation. Therefore, historic ground water table was used for our liquefaction analysis.

Based on the results of our liquefaction analyses, some of the sand and silt layers underlying the site may liquefy. Therefore, according to Table 1613A-5.2 of the 2007 CBC, the site should be classified as Site Class F, which requires site response analysis. However, as per ASCE 7-05, 20.3.1, if the structure has a fundamental period of vibration equal to or less than 0.5 seconds, site response analysis is not required, and the site class can be assigned using the standard methods assuming no liquefaction. We anticipate that future buildings at the project site will consist of one- to two-story structures. Therefore, the period of such structure(s) should be less than ½ second and the selection of site class is based on the assessment of the site soil profile type assuming no liquefaction.

Considering the above information the site can be classified as Site Class D, as presented in Table 1613A.5.2 of the 2007 CBC. Site Class D is defined as stiff soil profile with shear wave velocities between 180 m/s (600 feet/sec) and 366 m/s (1,200 feet/sec), SPT-N = 15 to 50 blows/foot, or Su = 50 - 100 kPa (1,000 - 2,000 psf) for the upper 30 meters (100 feet).

3.7 Design Level Earthquake

According to Section 1614A.1.2.2.b of the 2007 CBC, a site-specific ground motion hazard analysis is required for sites within 10 km of an active fault and allowed for other sites. It should be noted that the seismic provisions contained in the 2007 CBC are based on and refer (for more requirements) to the Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7-05 (referred to herein as ASCE 7-05). We estimated ground motion parameters using a site-specific ground motion hazard analysis per Section 1614A.1.2 of the 2007 CBC and Chapter 21 of ASCE 7-05. In addition, we also estimated ground motion parameters using the mapped values per Section 1613A.5.1 of the 2007 CBC.

According to the 2007 CBC, peak ground and spectral accelerations are to be developed for the Maximum Considered Earthquake (MCE). According to 2007 CBC and ASCE 7-05, the MCE is defined as the lesser of the (1) 2 percent probability of being exceeded in 50 years (return period of about 2,475 years) and (2) greater of 150 percent of the median deterministic values from the controlling fault and lower limit of the Figure 21.2-1 of ASCE 7-05. In addition, for site-specific parameters, procedures provided in Chapter 21 of ASCE7-05 should be used and the spectral accelerations at any period from site-specific analyses should not be less than the 80% of the code spectrum based on S_{MS} and S_{M1} values from Chapter 11. According to the 2007 CBC, the Design Earthquake (DE) may be taken as two thirds of the MCE.

Per code requirements, both probabilistic and deterministic seismic hazard analyses were used to estimate the peak ground and spectral accelerations for the MCE discussed above. These analyses involve the selection of appropriate predictive relationships to estimate the ground motion parameters, and, through probabilistic and deterministic methods, determination of peak ground and spectral accelerations.

3.8 Attenuation Relationships

Site-specific ground motions can be influenced by the types of faulting, magnitudes of the earthquakes, and the local soil conditions. The attenuation relationships used to estimate ground motion from an earthquake source at some distance from the site need to consider these effects.

Many attenuation relationships have been developed to estimate the variation of peak ground surface acceleration with respect to earthquake magnitude and distance from the site to the source of an earthquake. Of these relationships, we have selected the relationships presented by Abrahamson and Silva (1997), Boore and others (1997), Campbell and Bozorgnia (2003), and Sadigh and others (1997), because of their wide acceptance by seismologists. Our results were obtained by averaging the individual hazard results. These relationships have also been used in developing National Seismic Hazard Maps (Frankel and others, 1996, 2002) and for the State of California (Petersen and others, 1996; Cao and others, 2003). The relationship by Boore and others (1997) uses an estimate of the average shear wave velocity (V_S) of the soil profile in the analysis. Since the site can be classified as Site Class D, per the 2007 CBC, we have used this attenuation relationship with a V_S of 250 m/s, as recommended by Boore and others (1997) for Site Class D. We have used deep soil, firm soil, and soil relationships for Abrahamson and Silva (1997), Campbell and Bozorgnia (2003), and Sadigh and others (1997), respectively. The predictive relationships were developed from statistical analyses of recorded earthquakes from Western North America, including the records from the 1989 Loma Prieta, the 1992 Landers, and the 1994

Northridge earthquakes. These attenuation relationships provide mean values of ground motions associated with one set of parameters: magnitude, distance, site soil conditions, and mechanism of faulting. The uncertainty in the predicted ground motion is taken into consideration by including a magnitude dependent standard error in the probabilistic analysis.

3.9 Probabilistic Analysis

We developed a response spectrum for the Maximum Considered Earthquake (MCE) ground motion (2007 CBC 1614A.1.2.2.b; ASCE 7-05, Section 21.2.1), using a probabilistic seismic hazard analysis (PSHA). The probabilistic MCE is defined as the ground motion that has a 2% probability of being exceeded in 50 years (return period of about 2,475 years). The PSHA analysis involves the selection of an appropriate predictive relationship to estimate the ground motion parameters, and through probabilistic methods, development of spectral accelerations.

The theory behind the probabilistic seismic hazard analysis has been developed over many years (Cornell 1968, 1971, Merz and Cornell 1973, McGuire 2004) and is based on the "total probability theorem" and on the assumption that earthquakes are events that are independent of time and space from one another. According to this approach and assuming a Poisson process for ground motion occurrences, the probability of an event, P, is related to the annual frequency of exceedance of the ground motion γ and the exposure time t through

$$P = 1 - \exp(-\gamma \times t)$$

The probabilistic MCE is defined to have a 2 percent probability of exceedance in 50 years, which corresponds to an exposure time or return period of about 2,475 years and an annual frequency of exceedance of 0.00040/year.

The PSHA can be explained through a four-step procedure as follows:

1. The first step involves identification and characterization of seismic sources and probability distribution of potential rupture within the source. Usually, uniform probability distributions are assigned to each source. The probability distribution

of site distance is obtained by combining potential rupture distributions with source geometry.

2. The second step involves characterization of seismicity distribution of earthquake recurrence. An earthquake recurrence relationship such as Gutenberg-Richter recurrence is used to characterize the seismicity of each source.
3. The third step involves the use of predictive or attenuation relationships in assessing the ground motion produced at the site by considering the applicable sources and the distance of the sources to site. The variability of attenuation relationships is also included in the analysis. The effects of site soil conditions and mechanism of faulting are accounted for in these attenuation relationships.
4. The fourth and the last step involve combining all of these uncertainties to obtain the probability of ground motion exceedance during a particular time period.

We used the commercially available computer program EZ-FRISK Version 7.32 (Risk Engineering, 2009) for our analysis.

Response spectral values were calculated for a damping factor of 5 percent of critical per ASCE 7-05, Section 21.2.1. Because the site is located within the near-source zone of the Hayward Rodgers Creek fault, we also included the rupture directivity and near-source effects in our design spectra using the methods proposed by Somerville et al. (1997) and Abrahamson (2000). Plate 9 presents the MCE probabilistic response spectrum.

3.10 Deterministic Seismic Hazard Analysis

Deterministic seismic hazard analysis (DSHA) is based on the characteristics of the earthquake and of the causative fault associated with the earthquake. These characteristics include such items as magnitude of the earthquake, distance from the site to the causative fault, and maximum magnitude of earthquake associated with that fault. The effects of local soil conditions and mechanism of faulting are accounted for in the attenuation relationships for the project site.

The DSHA can be explained through a four-step procedure as follows:

1. The first step involves identification and characterization of all seismic sources capable of producing significant ground motions at the site.
2. The second step involves estimating maximum magnitude of earthquake associated with the known seismic sources and establishing site to source distance. The distance may be expressed as epicentral or hypocentral distance depending on the attenuation relationship.
3. The third step involves identifying the *controlling earthquake(s)* and use of predictive or attenuation relationships to estimate the ground motion produced at the site by considering the size of the earthquake occurring at the source and the distance of the source to site. The effects of the soil conditions and mechanism of faulting are accounted for in these relationships.
4. The fourth and last step involves formally defining the hazard in terms of spectral accelerations.

A deterministic procedure was used to estimate the median (50^{th} percentile) peak and spectral ground motions for the nearby sources to the project site. In calculating the median spectral accelerations, we used the same attenuation relationships and forward directivity modifications as in our PSHA. The median deterministic response spectrum was calculated based on the average of the various attenuation relationships used. Due to its proximity to the site and maximum magnitude of M7.2, the Hayward fault had the highest calculated spectral accelerations for all periods up to 4 seconds. It should be noted that we have included the rupture directivity and near-source effects per Somerville et al. (2003) and Abrahamson (2000) in our deterministic analysis. The site-specific deterministic spectral accelerations are presented for the fault normal conditions.

Per ASCE 7-05 Section 21.2.2, the deterministic MCE response acceleration at each period is calculated as 150 percent of the largest median 5 percent damped spectral response acceleration computed at that period for characteristic earthquakes on all known active faults within the region. The deterministic MCE response acceleration spectrum should not be lower than the Deterministic Lower Limit (DLL) on MCE

Response Spectrum presented on Figure 21.2.1 of ASCE 7-05. Plate 10 presents the deterministic response spectrum.

3.11 Determination of Site-Specific MCE Response Spectrum

The site-specific MCE response spectrum was determined according to ASCE 7 Section 21.2.3 as the lesser spectral acceleration from the probabilistic MCE response spectrum and deterministic MCE response spectrum. The MCE response spectrum for this site is the same as the probabilistic response spectrum up to a period of 2 seconds, and is the same as the deterministic response spectrum after the period of 2 seconds. The site-specific MCE response spectrum is presented in Plate 11.

3.12 Design Response Spectrum and Acceleration Parameters

As stipulated by ASCE 7 Section 21.3, the design response spectral accelerations are calculated as two-thirds of the MCE spectral accelerations except that the design spectral accelerations shall not be taken as less than 80 percent of spectral accelerations determined in accordance with ASCE 7 Section 11.4.5 using the mapped values of S_S and S_1. The design response spectrum determination is presented graphically in Plate 12. The design response spectrum is presented graphically in Plate 13, and the corresponding design spectral accelerations are presented numerically in Table 4.4.1-2, Section 4.4.1 of this report.

The short period design spectral acceleration (S_{DS}) and 1-second period design spectral acceleration (S_{D1}) parameters were determined in accordance with ASCE 7 Section 21.4. The parameter S_{DS} is taken as the spectral acceleration at a period of 0.2 seconds or 90 percent of the highest spectral acceleration at periods larger than 0.2 seconds, which ever is greater. The parameter S_{D1} is taken as the design spectral acceleration at a period of 1 second or two times the spectral acceleration at the 2 second period, whichever is greater. The design S_{DS} and S_{D1} parameters are presented in Section 4.4.1 of this report.

4. CONCLUSIONS AND RECOMMENDATIONS

Discussion and conclusions regarding specific geologic hazards, which could impact the site, are included below. The hazards considered include: surface fault rupture, landslides, expansive soils, seismic shaking, seismically induced ground failures (liquefaction, lateral spreading, dynamic compaction/seismic settlement, and seismically induced landslides and slope failures), and tsunami/seiches, and flooding (seismically induced or otherwise).

4.1 Surface Fault Rupture

The school site is not situated within an Alquist-Priolo Earthquake Fault Zone established by the State around active fault traces and it is located approximately 1.5 kilometers (km) to the southwest of the Hayward – Rodgers Creek fault, 15 km to the southwest of Calaveras fault, and 22 km to the southwest of the Mount Diablo Thrust. Based on the reviewed geologic/seismologic reports and maps, no known active, or potentially active faults cross or project toward the site. Therefore, it is our opinion that the potential for fault-related surface rupture at the school site is very low.

4.2 Landslides

The proposed school site is relatively flat, with little to no topographic relief. Therefore, it is our opinion that the potential for seismically induced (or otherwise) landslides and slope failures to occur at the site is considered low.

4.3 Expansive Soils

The near-surface soils encountered within our borings drilled as part of our geotechnical report are moderately expansive. Pertinent mitigation measures addressing the potential presence of expansive soils at the site is presented in our concurrent geotechnical report for the proposed development.

4.4 Seismic Ground Shaking

4.4.1 Peak Ground Acceleration

We understand that the proposed structures will be designed in accordance with the requirements of the latest 2007 edition of the California Building Code (CBC). It should be noted that the seismic provision of the 2007 CBC are based on and refer to (for more requirements) "Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7" (referred to herein as "ASCE 7").

Based on our understanding of the proposed structures, and on definitions provided in Table 1604A.5 and Section 1613A.5.6 of the 2007 CBC, we understand that the Occupancy Category for the proposed structures are IV and the Seismic Design Category is D.

According to the 2007 CBC, a site specific ground motion hazard analysis (See Section 2) should be performed since the site lies within 10 kilometers of an active fault (2007 CBC Section 1614A.1.2). In the case where a ground motion hazard analysis is performed, the design response spectrum and the design short and long period spectral parameters, S_{DS} and S_{D1}, are based on the ground motion hazard analysis results rather than on the mapped values of S_S and S_1 from the code, except that the design response spectrum shall not be less than 80 percent of the code-based spectrum developed using mapped values of S_S and S_1 (ASCE 7 Section 21.3).

Kleinfelder 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 in Table 4.4.1-1.

Table 4.4.1-1
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	S_s	1.865 g	Section 1613A.5.1
Mapped Spectral Acceleration for a 1-Second Period	S_1	0.707 g	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	0.824 g	N/A
MCE* Spectral Response Acceleration for Short Periods	S_{MS}	1.818 g	Section 1614A.1.1 (Section 21.4)
MCE* Spectral Response Acceleration at 1-Second Period	S_{M1}	1.818 g (1)	Section 1613A.5.3 (Section 21.4)
Design Peak Ground Acceleration (S_D at T=0)	PGA_D	0.55 g	Section 1802A.2.7
Design Spectral Response Acceleration (5 percent damped) at Short Periods	S_{DS}	1.212 g	Section 1613A.5.4 (Section 21.4)
Design Spectral Response Acceleration (5 percent damped) at 1-Second Period	S_{D1}	1.212 g (2)	Section 1613A.5.4 (Section 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}

The design response spectrum is presented graphically in Plate 13 and numerically in Table 4.4.1-2 below.

Table 4.4.1-2
DESIGN SPECTRAL ACCELERATIONS (5 PERCENT DAMPING)

Period (sec)	DE S_a(g)
0.010 (PGA)	0.550
0.05	0.712
0.06	0.744
0.08	0.823
0.10	0.910
0.114	1.000
0.15	1.057
0.20	1.202
0.25	1.273
0.30	1.327
0.40	1.347
0.50	1.312
0.60	1.282
0.75	1.280
1.00	1.201
1.50	1.011
2.00	0.831
2.50	0.699
3.00	0.609
4.00	0.473

Table 4.4.1-3 below shows a summary of the deaggregation analysis for the design PGA.

TABLE 4.4.1-3:

PGA DEAGGREGATION ANALYSIS RESULTS

Event	Mean Distance (km)	Mean Magnitude*	Mode Distance (km)	Mode Magnitude*
Design Earthquake	4.8	6.8	1.25	6.8

*Moment Magnitude

4.5 Seismically Induced Ground Failures

4.5.1 Liquefaction and Lateral Spreading

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits. If liquefaction occurs, foundations resting on or within the liquefiable layer may undergo settlements. This will result in reduction of foundation stiffness and capacities.

The site lies within the Hayward quadrangle, which has been mapped by the CGS as part of its ongoing effort to map landslide and liquefaction related hazards throughout the San Francisco Bay Area. According to the CGS, the school campus is located within an area where historical occurrence of liquefaction, or local geological, and ground-water conditions indicate a potential for permanent displacements such that mitigation as defined in California Public Resources Code Section 2693© would be required (CGS, 2003). According to Youd and Hoose (1978), ground cracks associated with sand boils and miscellaneous effects were recorded about 2 km to the southeast of the site during 1868 (m7.0) Hayward earthquake. This feature was described as the opening of fissures in the earth and appearance of new spring of water. No historic ground failures were reported within approximately 8 km of the site by Holzer (1998) as a result of the 1989 M6.9 Loma Prieta earthquake.

The project site lies within a seismic hazard zone for potential liquefaction as determined by the California Geological Survey (2003; see Plate 14). Based on the borings performed in 2006 in the southeast corner of the site and a recent CPT performed in the northeast corner of the site as part of the concurrent geotechnical investigation, the site is underlain by sandy silt and silty lean clay, with a few approximately 3-foot thick layers of sand. Groundwater was encountered within the borings as shallow as 6 feet, even though historical groundwater levels are about 10 feet according to the Seismic Hazard Zone report by the CGS for the Hayward Quadrangle. The silt and low plasticity lean clay are considered to be potentially liquefiable. Based on that information, we performed liquefaction analysis using the methods proposed in Youd et al. (2001). For our analysis, we used a peak ground acceleration of 0.55 g, associated with an earthquake magnitude of M6.8. We assumed groundwater to be at a depth of 6 feet below ground surface for our analysis. Our results indicate that the silts and low plasticity lean clays may liquefy during an earthquake. Based on Tokimatsu and Seed, (1987), the estimated total liquefaction induced settlement at the southeast corner of the site is on the order of about 3½ inches, and at the northeast corner of the site on the order of about 1 inch. Based on Martin and Lew (1999), differential settlements may be taken as half of the total settlements between adjacent supports.

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 the southeast corner of the site is high due to relatively thin non-liquefiable layers over relatively thick liquefiable layers. The potential at the northeast corner of the site appears to be low.

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. These phenomena typically occur adjacent to free faces such as slopes and creek channels. No such features are present in the immediate vicinity of the campus. Therefore, we believe that the potential for lateral spreading to take place at the site is negligible.

4.5.2 Dynamic Compaction

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 uncompacted fill soils. Our borings encountered about 2½ feet of loose silty sands above the water table at the southeast corner of the site. We estimate approximately 1/2 inch of densification from these layers during a major seismic event based on the method presented in Tokimatsu and Seed (1987). Based on Martin and Lew (1999), differential settlements may be taken as half of the total settlements between adjacent supports. At the northeast corner of the site densification appears to be negligible.

Total estimated seismic settlement due to liquefaction and densification during a major earthquake at the southeast corner of the site is 4 inches and the resulting estimated differential settlement between adjacent supports is about 2 inches. At the northeast corner of the site, the total estimated seismic settlement is about 1 inch and the resulting estimated differential settlement between adjacent supports is about ½ inch. These settlement estimates are in addition to the ½-inch and ¼-inch of total and differential consolidation settlements discussed above. Because of the relatively large differential settlement at the southeast corner of the site and due to the potential for liquefaction-induced sand boils and ground fissures, we recommend tying footings together with grade beams or using a mat foundation in order to provide a stiff enough structure to withstand this differential settlement. Also, due to the relatively large total settlement, it may be prudent to provide flexible utility connections at the perimeter of the buildings.

Further discussion of the information presented above with design recommendations for foundations, slabs-on-grade, and earthwork, is presented in the Geotechnical Engineering Report.

4.6 Tsunami, Seiche, and Flooding

Flood hazards are generally considered from three sources that include tsunami and seiche, seismically related dam failure and 100 to 500-year storm events. The site is located approximately 5 km east of San Francisco Bay. Ritter and Dupre (1972) indicate that the coastal lowland areas, immediately adjacent to San Francisco Bay, are subject to possible inundation from a tsunami with a run up height of 20 feet at the Golden Gate Bridge. Ritter and Dupre's 1972 map does not show the site area to be within an area that could become inundated by tsunami waves. The closest area that could become inundated by flooding resulting from tsunami to the site is located along the southeastern margin of the Bay. Based on that information, we judge that the potential for tsunamis or seiche flooding to impact the site is low.

With respect to flooding from both 100 and 500-year flood events, Flood Insurance Rate Maps prepared by FEMA (2000) place the site within Zone A9, which they define as follows:

> **Zone A9:** "Area of 100 year flood; base flood elevations and flood hazard factors determined."

With respect to flooding related to dam failure, the site is situated outside of the potential inundation wave/zone resulting from failure of Cull Creek/San Lorenzo Creek (ABAG, 2004).

4.7 Naturally Occurring Asbestos

According to State of California guidelines established by the Department of Toxic Substances and Control (2004 and 2005), a Preliminary Environmental Assessment (PEA) is required for school sites that are located within a 10-mile radius of any rock formation that may contain naturally occurring asbestos (NOA). The nearest mapped locations of rock outcrops from which NOA may be found are approximately two miles northeast of the school campus (Plate 6). The potential source rocks are composed of keratophyre and gabbro (Graymer et al., 1996), and limited exposures of serpentine based on mapping by Robinson (1956) and Dibblee (1980). This same area is also identified on the California Geological Survey's map of areas most likely to contain naturally occurring asbestos (Churchill and Hill, 2000). The surficial sediments underlying the campus could contain naturally occurring asbestos minerals that were eroded from the rock outcrops from the nearby hills and deposited in the alluvial sediments.

In accordance with California Department of Toxic Substances and Control (DTSC) requirements, we recommend that the possible presence of NOA in the soils underlying the location of the proposed improvements be further evaluated. If a PEA is required, then we recommend that the PEA be conducted in accordance with DTSC (2004, 2005) and California Geological Survey (2002) guidelines.

5. 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 the 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 the San Lorenzo Unified School District. If the San Lorenzo Unified School District 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 to the plans and specifications during design or construction, the San Lorenzo Unified School District 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.

6. REFERENCES

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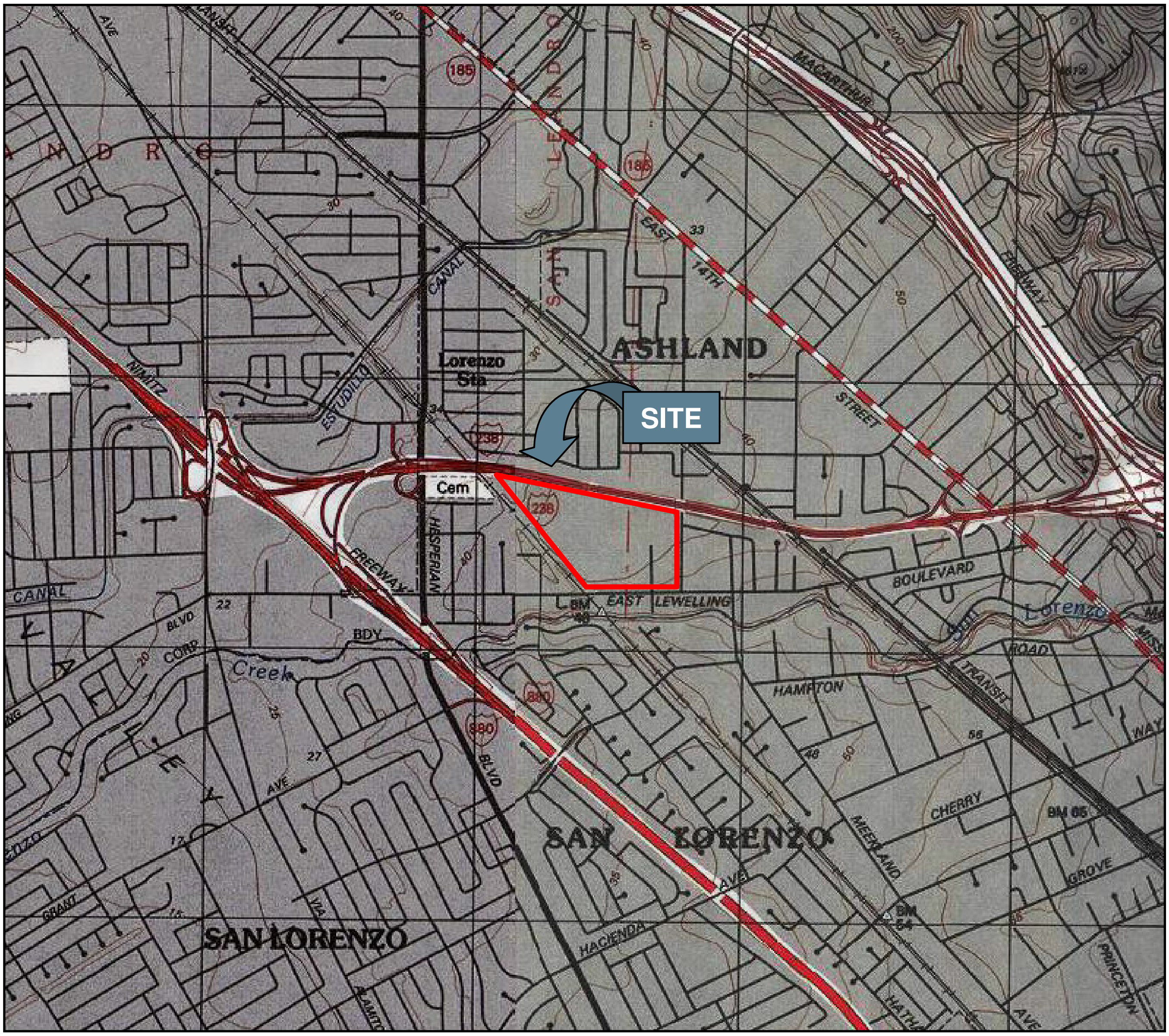
Historical Aerial Photographs
Source: Pacific Aerial Surveys
Black & White Photographs

Date	Flight No.	Line No.	Photo No.
10-08-1996	AV5200	16	37
06-22-1981	AV2040	7	38,39
05-19-1971	AV995	5	35,36
07-26-1963	AV550	11	25,26
05-03-1957	AV253	15	37
03-24-1947	AV11	6	14,15,16

PLATES



2000 1000 0 2000

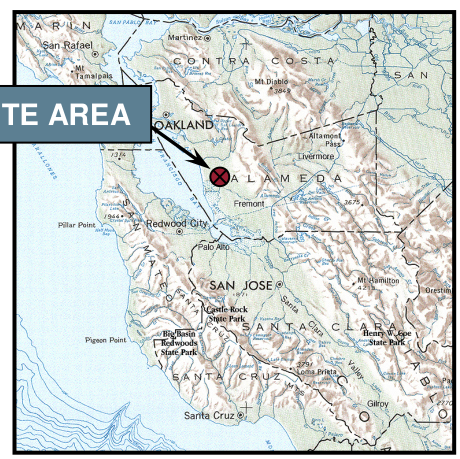
Approximate Scale (feet)

N

Site Coordinates:

Latitude: 37.6884° N

Longitude: 122.1223° W



Base: U.S. Geological Survey, Hayward and San Leandro 7½-minute quadrangles

Original in Color

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KLEINFELDER
Bright People. Right Solutions.

PROJECT NO:	105356
DRAWN:	August 2009
DRAWN BY:	DS
CHECKED BY:	
FILE NAME:	

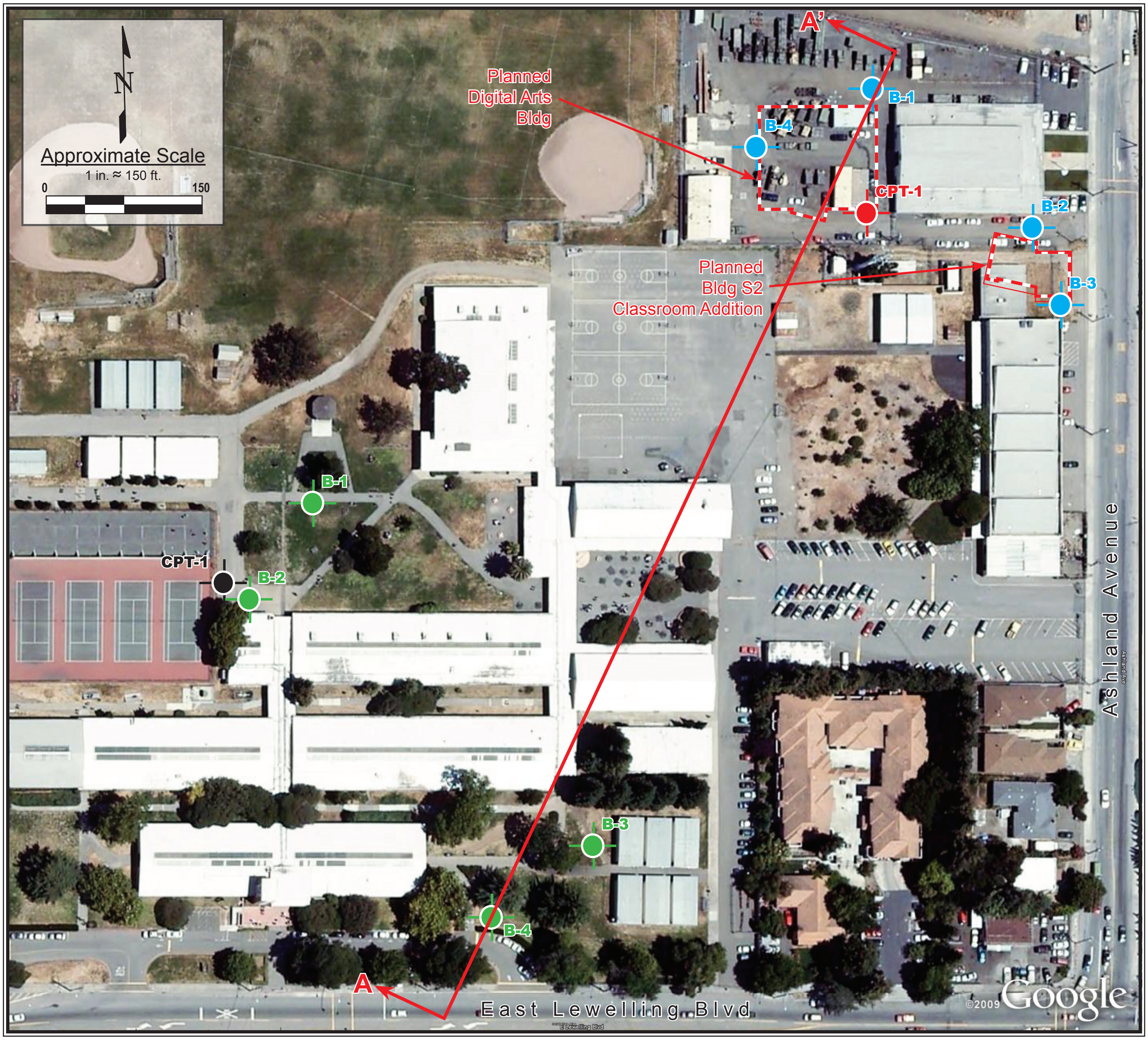
SITE VICINITY MAP

San Lorenzo High School
50 East Lewelling Boulevard
San Lorenzo, California

PLATE

1

PLOTTED:28 August 2009, 10:27 AM



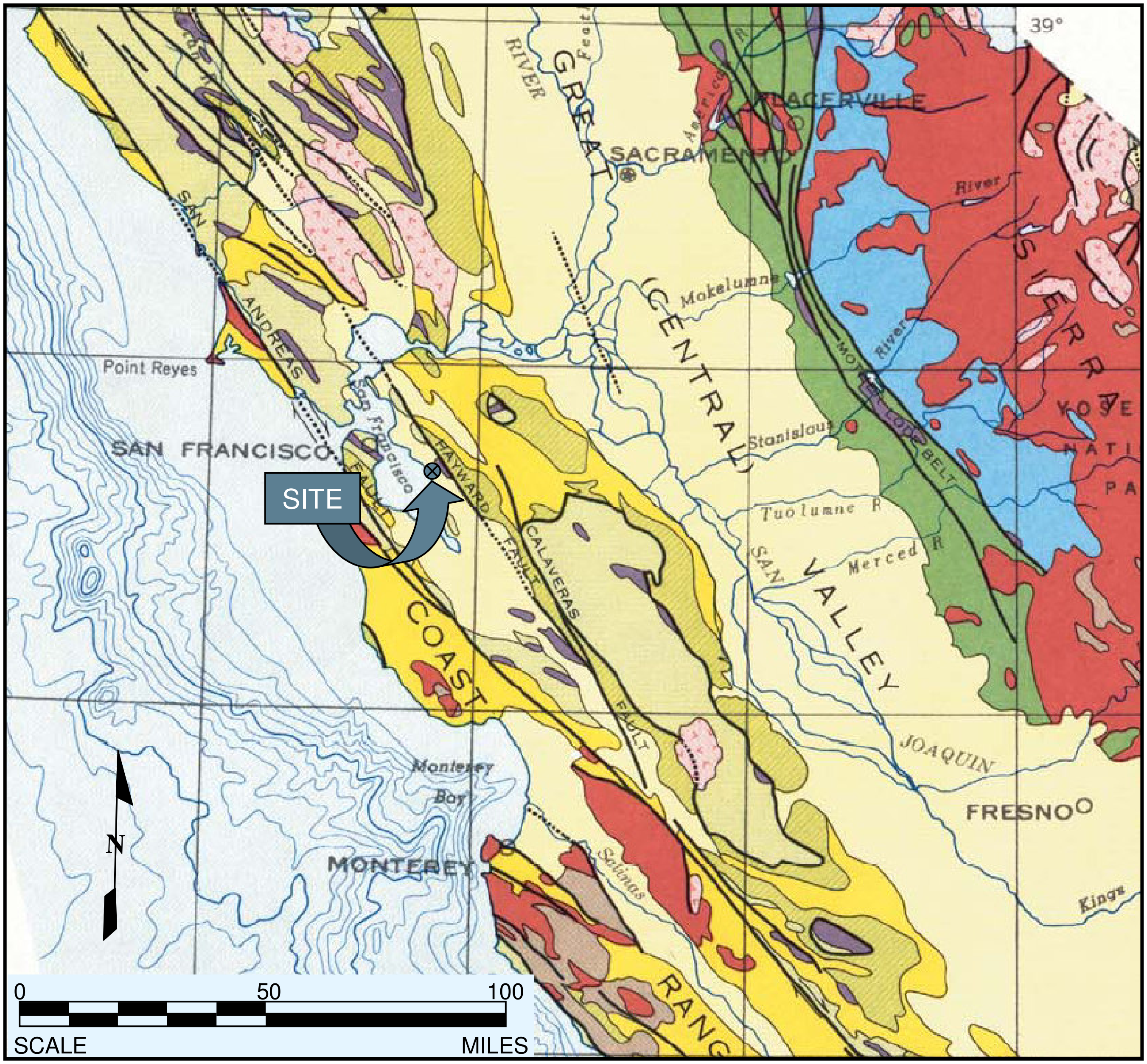
Legend

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
A — A'	Geologic Cross-Section

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

KLEINFELDER
Bright People. Right Solutions.

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



EXPLANATION

Cenozoic nonmarine	Mesozoic Granitic rocks
Cenozoic marine	Mesozoic Ultramafic rocks
Late Mesozoic shelf and slope	
Late Mesozoic of the Franciscan Formation	
Fault, dotted where concealed, arrows indicate direction of movement	

Map source: California Geological Survey, 2002, Geologic Map of California, California Department of Conservation, used with permission.

Original in Color

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KLEINFELDER
Bright People. Right Solutions.

PROJECT NO:	105356
DRAWN:	August 2009
COMPILED BY:	DS
CHECKED BY:	
FILE NAME:	

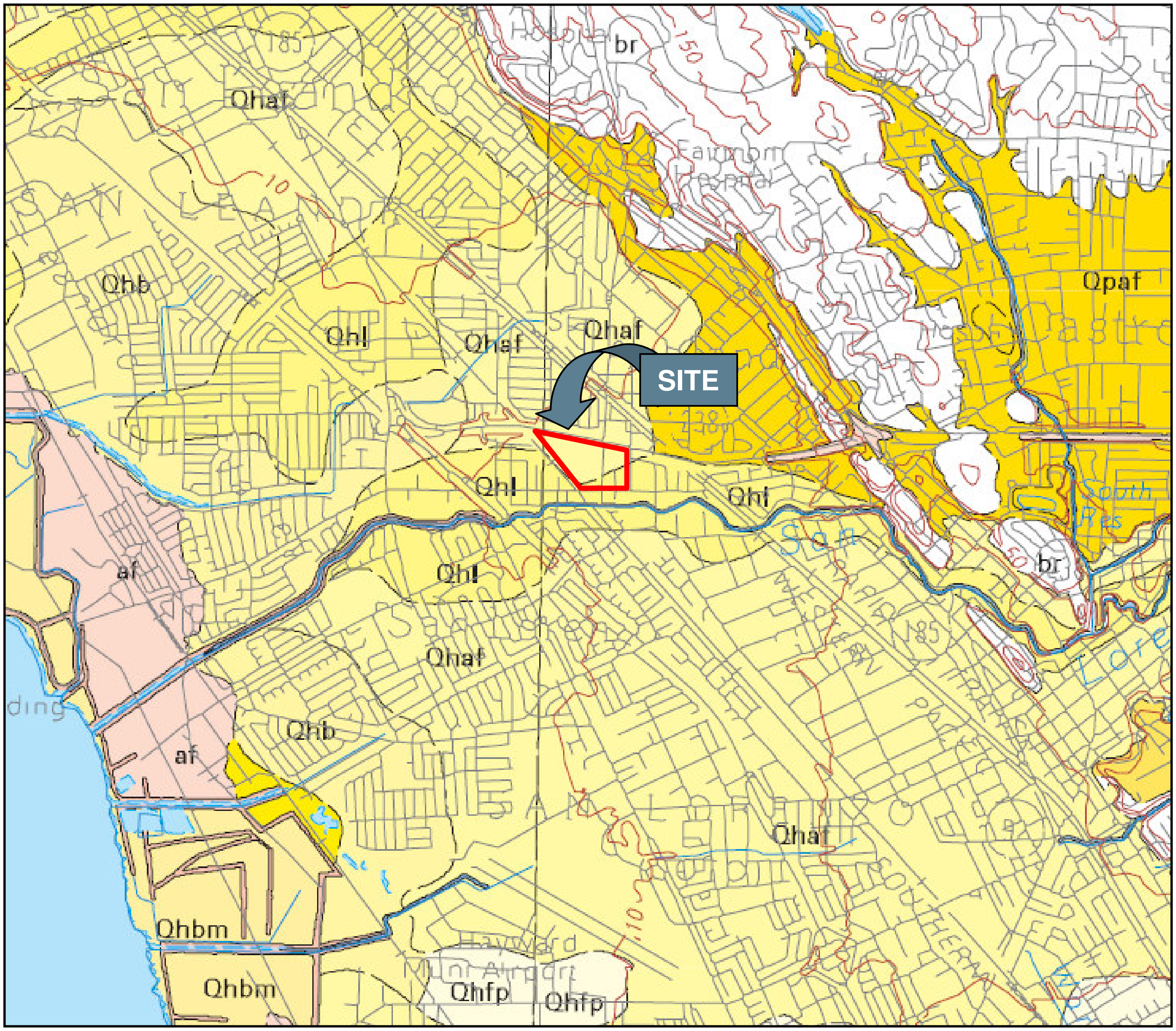
REGIONAL GEOLOGIC MAP

San Lorenzo High School
50 East Lewelling Boulevard
San Lorenzo, California

PLATE

3

PLOTTED: 28 August 2009, 10:27 AM



4000 2000 0 4000

Approximate Scale (feet)

N

EXPLANATION

Quaternary Units

(h – Holocene, p – Pleistocene)

af	Artificial Fill
Qhbm	Bay Mud Deposits
Qhb	Alluvial Basin Deposits
Qhaf	Alluvial Fan Deposits
Qhl	Alluvial Fan Levee Deposits
Qpaf	Alluvial Fan Deposits

Map Source: Knudsen et al., 1997, Quaternary Geology and Liquefaction Susceptibility Maps, San Francisco, California, 1:100,000 Quadrangle: USGS OFR 97-715.

Original in Color

KLEINFELDER
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PROJECT NO:	105356
DRAWN:	August 2009
COMPILED BY:	DS
CHECKED BY:	
FILE NAME:	

AREA GEOLOGIC MAP

(Quaternary Units)

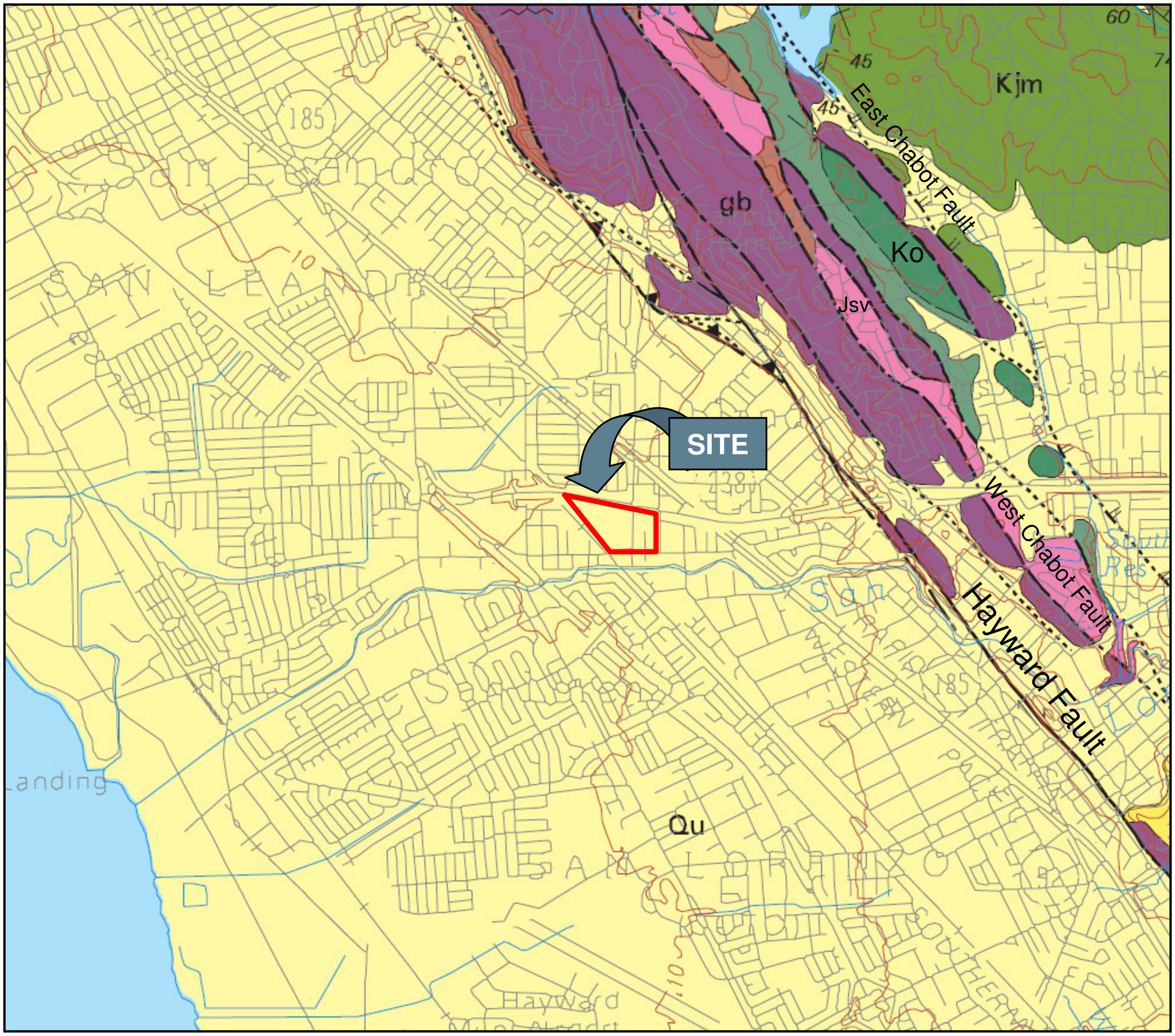
San Lorenzo High School
50 East Lewelling Boulevard
San Lorenzo, California

PLATE

4

Library file: L:\2009\library\projects\105356***.ppt

PLOTTED: 28 August 2009, 10:27 AM



4000 2000 0 4000

Approximate Scale (feet)

N

EXPLANATION

Selected Units

Qu Alluvial Deposits, Undifferentiated (Quaternary)

Kjm Joaquin Miller Formation (Cretaceous)
Ko Oakland Formation (Cretaceous)
Jsv Keratophyre and Quartz Keratphyre (Jurrasic)
gb Gabbro (Jurassic)

Map Source: Graymer, Jones, and Brabb, 1996, Preliminary Geologic Map Emphasizing Bedrock Formations in Alameda County, California: USGS OFR 96-252, Original Map Scale 1:75,000

Original in Color

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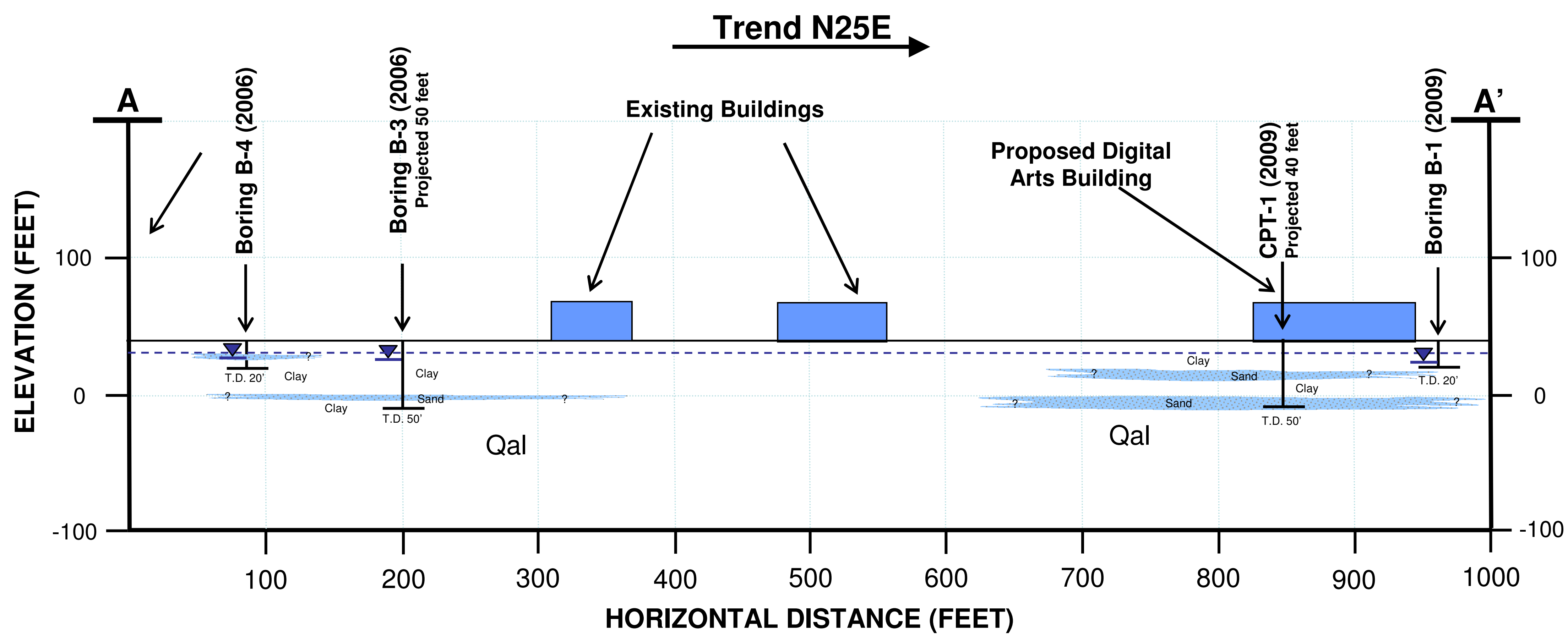
PROJECT NO: 105356	**AREA GEOLOGIC MAP**	**PLATE**
DRAWN: August 2009	**(Bedrock Units)**	**5**
COMPILED BY: DS		
CHECKED BY:	**San Lorenzo High School** **50 East Lewelling Boulevard** **San Lorenzo, California**	
FILE NAME:		

PLOTTED: 28 August 2009, 10:27 AM



EXPLANATION

Qal Alluvial Fan Deposits (Quaternary)

Boring or CPT by Kleinfelder

Groundwater Level (Measured in Borehole)

Dashed blue line represents historical high groundwater level per CGS SHZR 091 (2003)

No Vertical Exaggeration

NOTES:

1. Locations are approximate.
2. This cross-section is for illustration purposes only and is based on extrapolation and interpolation between and beyond the borings performed at the site. As such, the cross-section should be considered approximate. Actual subsurface conditions may vary.

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PROJECT NO:	105356
DRAWN:	August 2009
DRAWN BY:	DS
CHECKED BY:	BS
FILE NAME:	

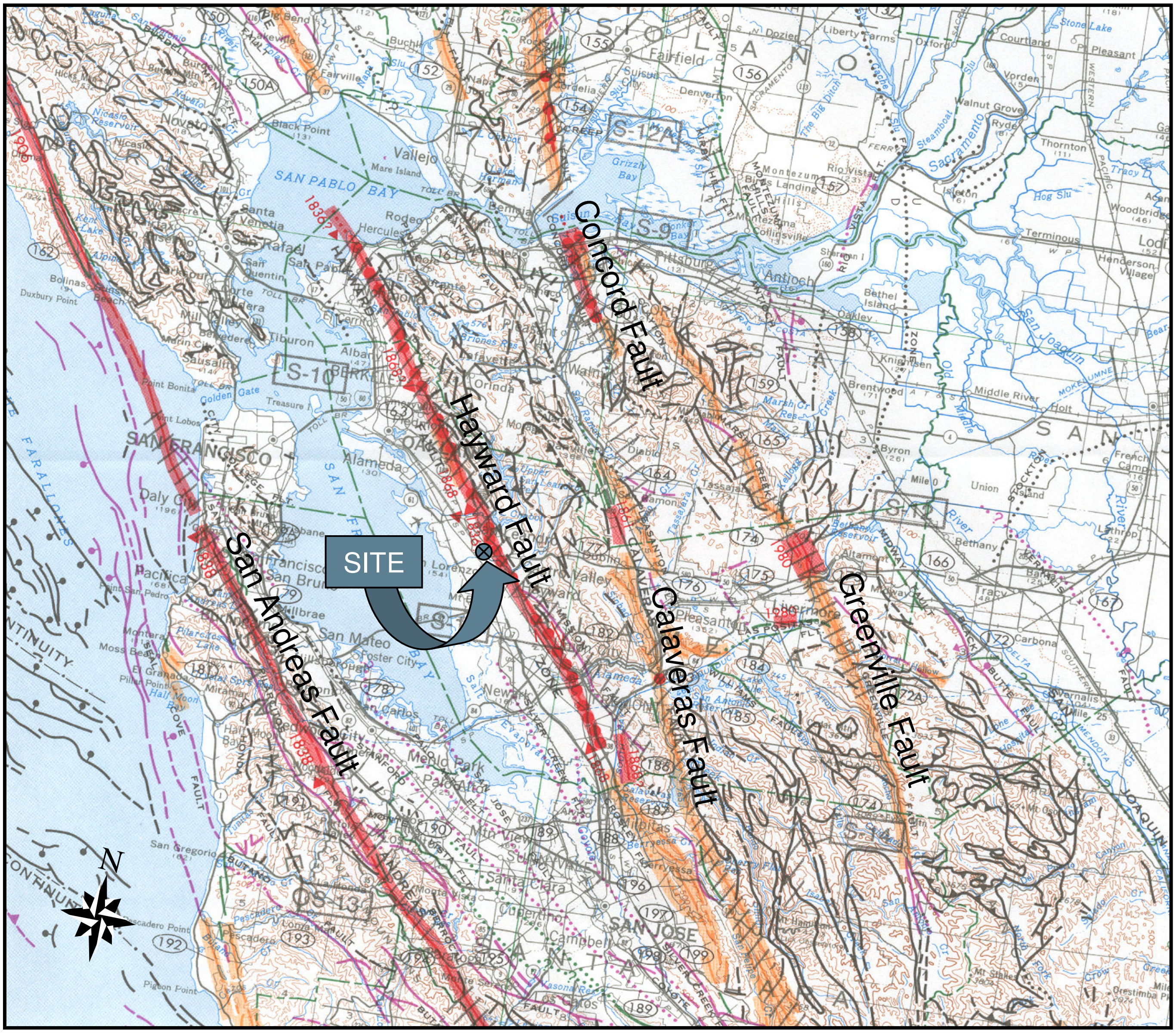
GEOLOGIC CROSS-SECTION A-A'

San Lorenzo High School
50 East Lewelling Boulevard
San Lorenzo, California

PLATE

6

PLOTTED:28 August 2009, 10:27 AM



10 5 0 10

Approximate Scale (feet)

EXPLANATION

- Fault with historic movement
- Fault with within last 10,000 years
- Faults with movement older than 10,000 years or undifferentiated

Reference
Jennings, C.W., 1994, Fault activity map of California and adjacent area, with locations and ages or Recent volcanic eruptions; California geologic data map series, map no. 6, Department of Conservation, Division of Mines and Geology

Original in Color

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DRAWN:	August 2009
DRAWN BY:	DS
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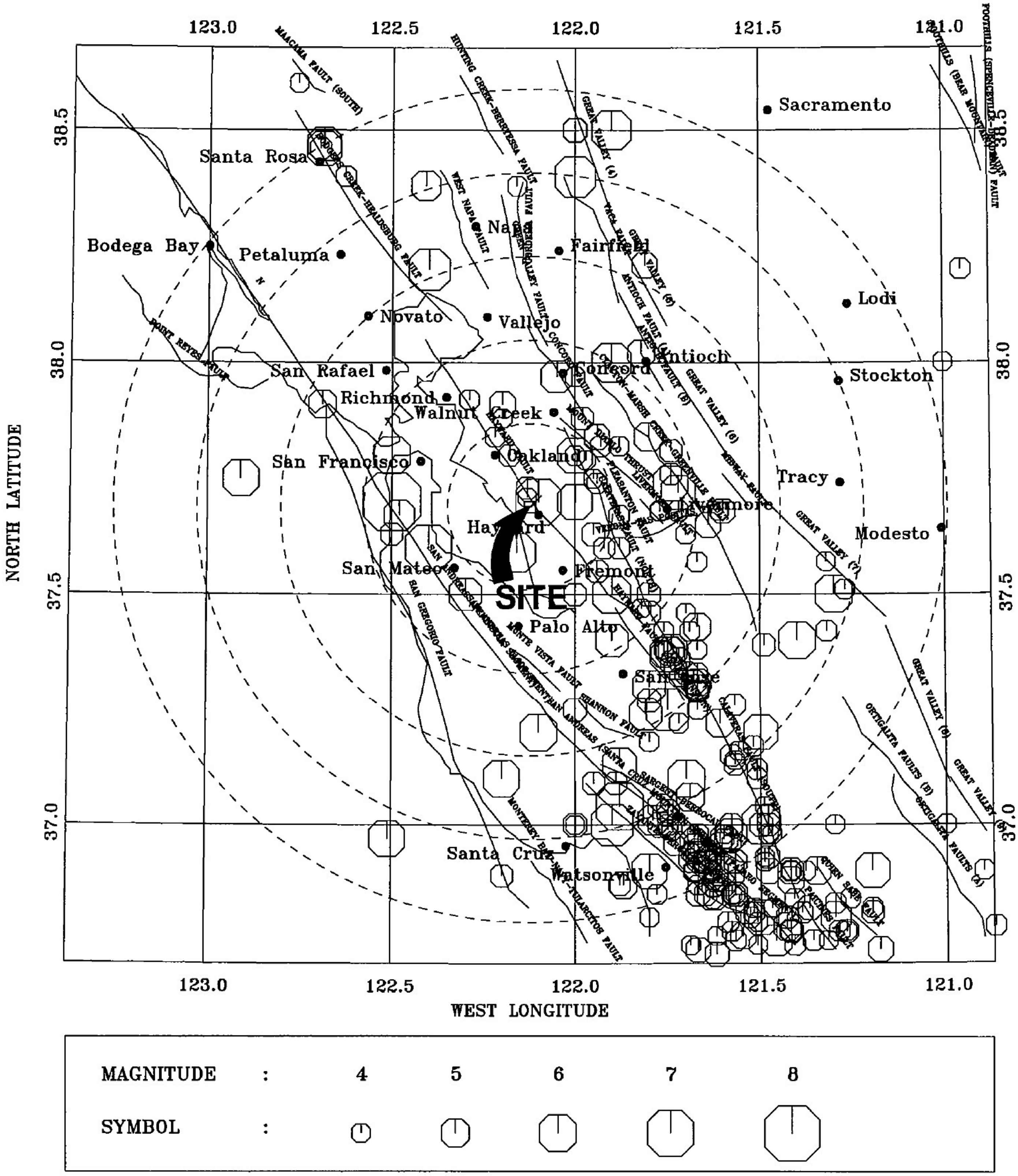
REGIONAL FAULT MAP

San Lorenzo High School
50 East Lewelling Boulevard
San Lorenzo, California

PLATE

7

PLOTTED:28 August 2009, 10:27 AM



RADIUS OF LARGEST CIRCLE IS 100 KM

KLEINFELDER
Bright People. Right Solutions.

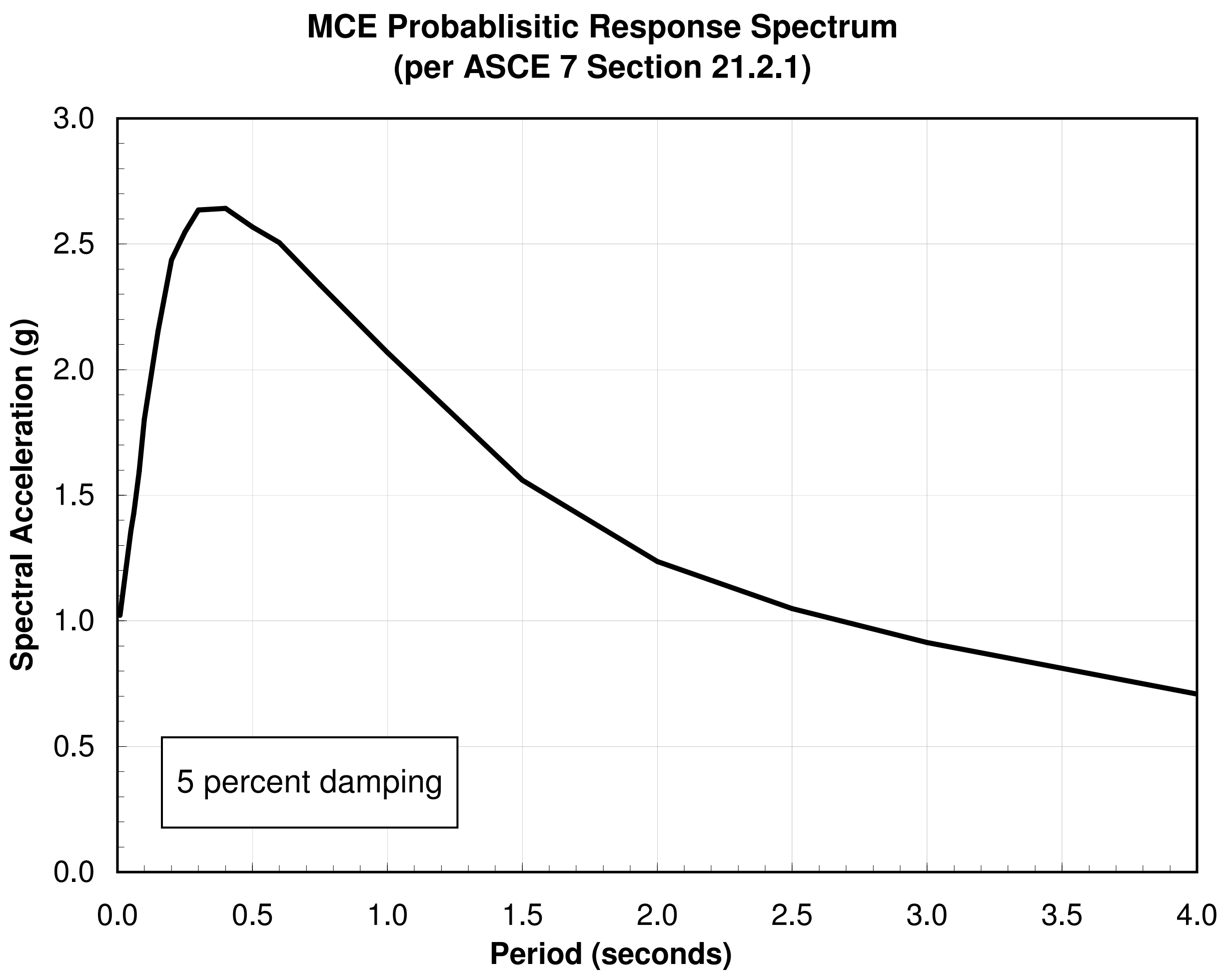
PROJECT NO:	105356
DRAWN:	August 2009
COMPILED BY:	DS
CHECKED BY:	
FILE NAME:	

Fault Map and Earthquake Epicenters (1800 to December 2005)

San Lorenzo High School
50 East Lewelling Boulevard
San Lorenzo, California

PLATE

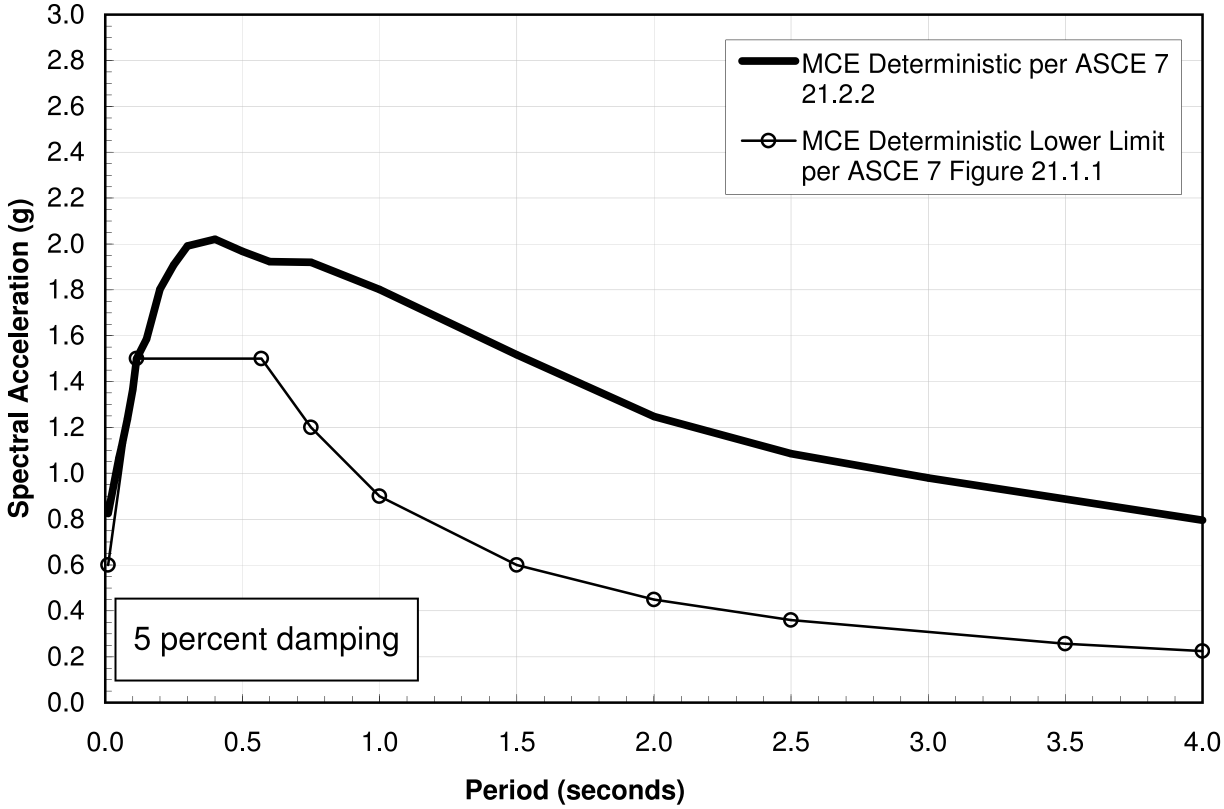
8



THIS FIGURE DOES NOT PRESENT THE RECOMMENDED DESIGN SPECTRUM.
PLEASE REFER TO PLATE 12 FOR A PRESENTATION OF THE DESIGN SPECTRUM

KLEINFELDER Bright People. Right Solutions.		**MCE Probabilistic Response Spectrum**	**PLATE**
By: JFM	Reviewed by: JRG	**San Lorenzo High School** **50 East Lewelling Boulevard** **San Lorenzo, California**	**9**
Project # 105356	July 2009		

Deterministic Response Spectra



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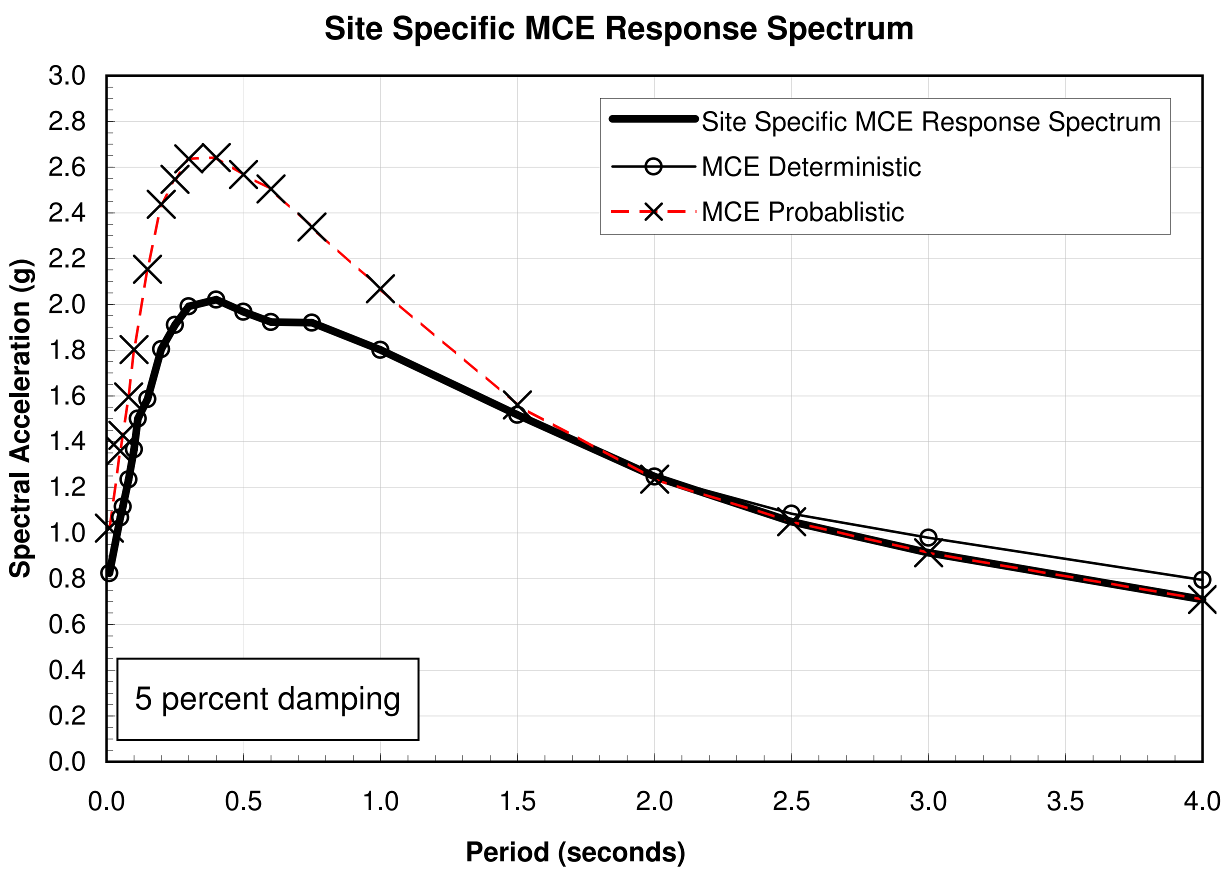
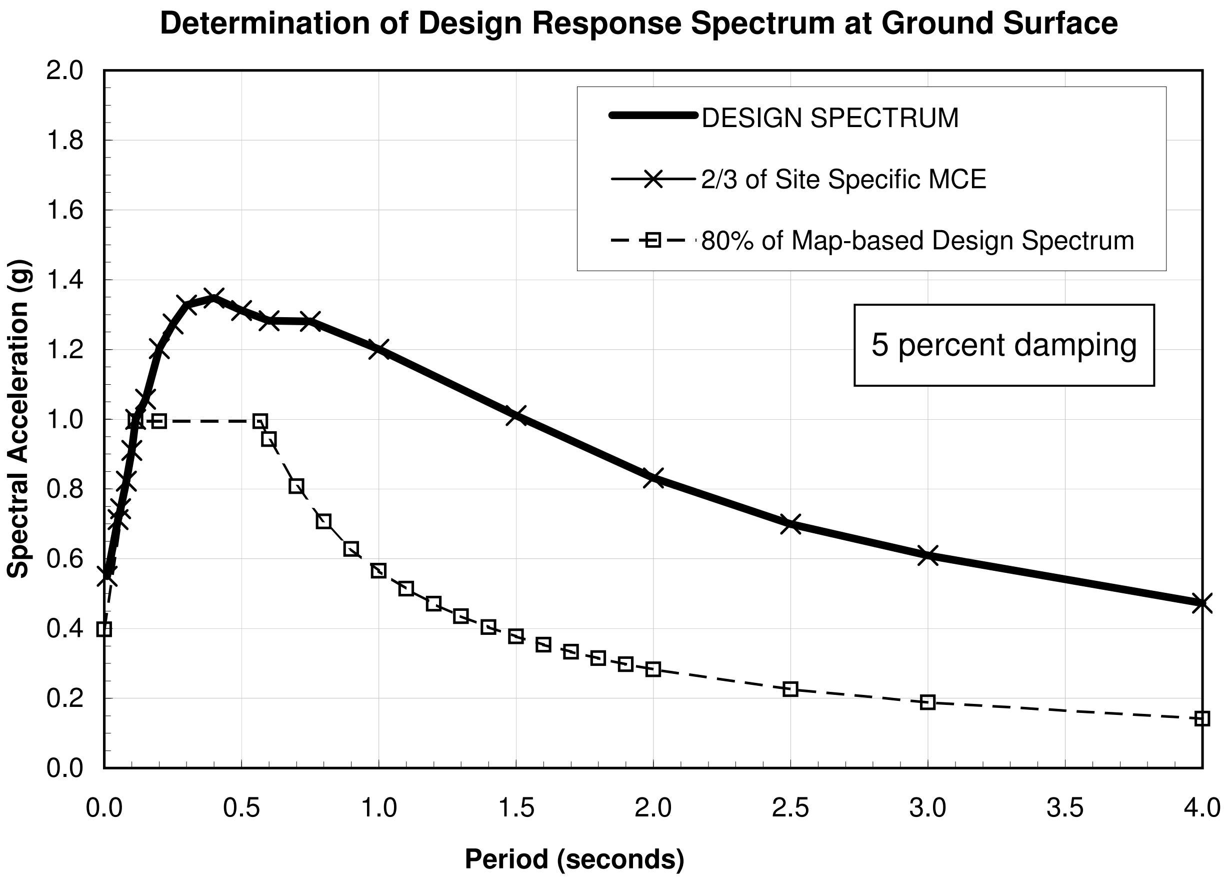
PLEASE REFER TO PLATE 12 FOR A PRESENTATION OF THE DESIGN SPECTRUM

KLEINFELDER Bright People. Right Solutions.		MCE Deterministic Response Spectra	PLATE
By: JFM	Reviewed by: JRG	San Lorenzo High School 50 East Lewelling Boulevard San Lorenzo, California	10
Project # 105356	July 2009		



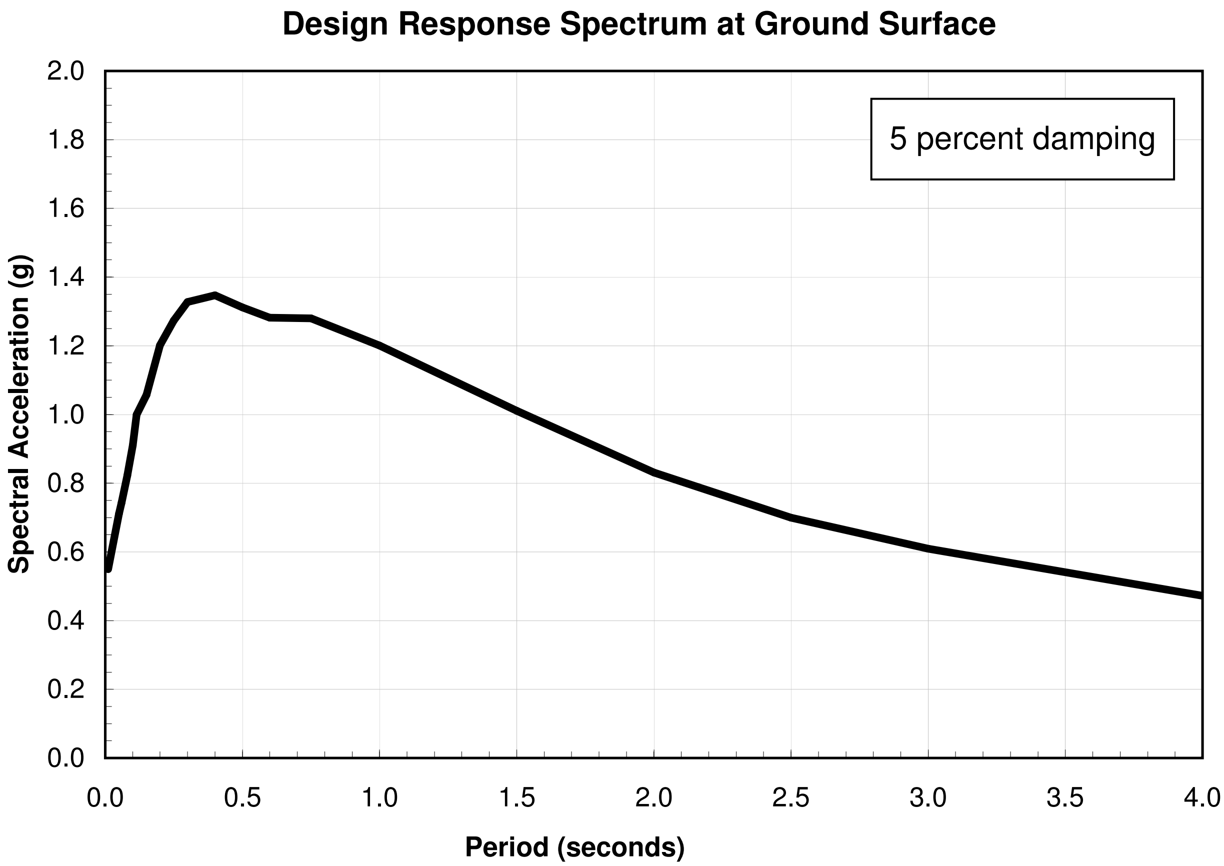
THIS FIGURE DOES NOT PRESENT THE RECOMMENDED DESIGN SPECTRUM.
PLEASE REFER TO PLATE 12 FOR A PRESENTATION OF THE DESIGN SPECTRUM

KLEINFELDER Bright People. Right Solutions.		Site Specific MCE Response Spectrum Determination	PLATE
By: JFM	Reviewed by: JRG	San Lorenzo High School 50 East Lewelling Boulevard San Lorenzo, California	11
Project # 105356	July 2009		



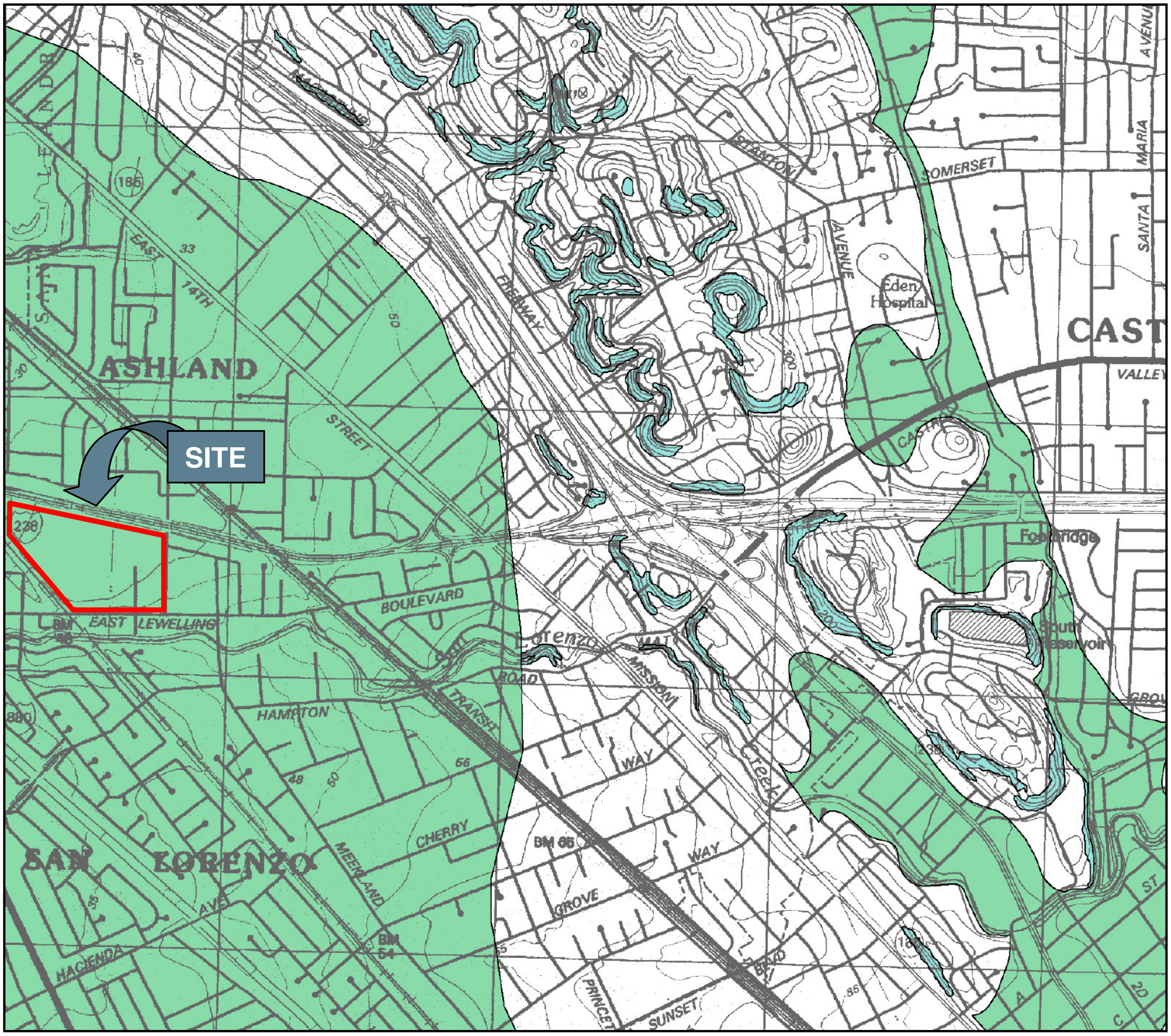
THIS FIGURE DOES NOT PRESENT THE RECOMMENDED DESIGN SPECTRUM.
PLEASE REFER TO PLATE 12 FOR A PRESENTATION OF THE DESIGN SPECTRUM

KLEINFELDER Bright People. Right Solutions.		**Design Response Spectrum Determination**	**PLATE**
By: JFM	Reviewed by: JRG	**San Lorenzo High School** **50 East Lewelling Boulevard** **San Lorenzo, California**	**12**
Project # 105356	July 2009		



Design Acceleration Parameters (ASCE 7 Section 21.4) S_{DS}, S_{D1}, S_{MS} and S_{M1} are determined from this spectrum and presented in Table 4.4.1-1 of the report.

KLEINFELDER Bright People. Right Solutions.		Design Response Spectrum	PLATE
By: JFM	Reviewed by: JRG	San Lorenzo High School 50 East Lewelling Boulevard San Lorenzo, California	13
Project # 105356	July 2009		



2000 1000 0 2000

Approximate Scale (feet)

N

EXPLANATION

Liquefaction Zone – Areas where liquefaction has occurred or has the potential to occur

Earthquake-Induced Landslide Zone – Areas where landslides have occurred or have the potential to occur

Map Source: California Geological Survey, 2003, Seismic Hazard Zones Map, Hayward 7.5-minute Quadrangle

Original in Color

KLEINFELDER
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PROJECT NO:	105356
DRAWN:	August 2009
COMPILED BY:	DS
CHECKED BY:	
FILE NAME:	

SEISMIC HAZARD ZONES MAP

San Lorenzo High School
50 East Lewelling Boulevard
San Lorenzo, California

PLATE

14

PLOTTED: 28 August 2009, 10:27 AM

Library file: L:\2009\library\projects\105356***.ppt

APPENDIX A

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

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

A-1

8/28/2009 1:39:04 PM

Date Completed: 8/13/09

Logged By: N. Berner

Total Depth: Approximately 20.5 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-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

A-2

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8/28/2009 1:40:25 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 20.0 ft	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)**)
0								**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
							1.0	- increase in fine grained sand content
10		10						
		7:37						
		18					2.0	- olive, decrease in sand content
15		12					1.5	- gray, firm to hard, carbonate traces
							2.0	- yellow, wet, firm to hard, trace fine grained sand, carbonate traces
20		15						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

A-3

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**

Depth, ft	Sample	Blows/ft (FIELD)	Dry Density pcf (LABORATORY)	Moisture Content %	Compress. Strength tsf	Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: **Estimated 38.0 feet (MSL)**)
								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	
25		21					2.1	- hard, iron oxide staining
								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 **A-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**

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

Depth, ft	FIELD: Blows/ft	LABORATORY: Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests	Pen, tsf	DESCRIPTION
0							**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							

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture.
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

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	TX	TRIAXIAL SHEAR
PI	PLASTICITY INDEX	CONSOL	CONSOLIDATION
%-#200	SIEVE ANALYSIS (#200 SCREEN)	R-Value	RESISTANCE VALUE
DS	DIRECT SHEAR	SE	SAND EQUIVALENT
C	COHESION (PSF)	EI	EXPANSION INDEX
PHI	FRICTION ANGLE	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.

KLEINFELDER	BORING LOG LEGEND	PLATE
PROJECT NO. 64583-PWGEO	SAN LORENZO HIGH SCHOOL 50 EAST LEWELLING BLVD SAN LORENZO, CALIFORNIA	B-1

1/20/06 11:35:52 AM

Date Completed:	1/5/06	Drilling method:	8" Hollow Stem Auger
Logged By:	D. Daily	Hammer Wt:	140 lbs., 30" drop
Total Depth:	Approximately 30.0 ft	Notes:	Drilled in grass

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 37 feet (MSL))
								TOPSOIL - approximately 7 inches thick
							1.0	**CLAYEY SILT (ML)** - olive-brown, moist, medium stiff, low to medium plasticity
		7	93.4	27.0	0.6 @ 15.0%			
5			92.9	28.2				- black
		5					1.5	**LEAN CLAY (CL)** - olive-brown, moist, medium stiff, medium plasticity
			14:15					
			13:39					**SILTY SAND (SW)** - gray-brown, wet, loose, no cementation
10		5	91.3	31.5			1.5 1.0	**SILTY LEAN CLAY (CL)** - brown, moist, medium stiff, medium plasticity, some sand
15		20	104.5	22.3			1.5 3.5	**LEAN CLAY (CL)** - dark brown, moist, stiff, medium plasticity
20		18					4.5 2.5	**SILTY LEAN CLAY (CL)** - orange-brown, moist, stiff, medium plasticity - brownish-gray with caliche deposits, very stiff - red-brown, no caliche deposits
25		15		28.7		LL=33; PI=9	1.0	**CLAYEY SILT (ML)** - light brown, moist, stiff, low plasticity
								SILTY SAND with CLAY DEPOSITS (SM) - gray-brown, wet, medium dense, fine grained sand, variable clay content
30		21						
								Boring terminated at 30 feet Backfilled with portland cement grout

KLEINFELDER

PROJECT NO. 64583-PWGEO

LOG OF BORING NO. B-1

SAN LORENZO HIGH SCHOOL
50 EAST LEWELLING BLVD
SAN LORENZO, CALIFORNIA

PLATE **B-2**

Date Completed:	1/5/06	Drilling method:	8" Hollow Stem Auger
Logged By:	D. Daily	Hammer Wt:	140 lbs., 30" drop
Total Depth:	Approximately 20.0 ft	Notes:	Drilled in asphalt

DESCRIPTION

Surface Elevation: **Estimated 37 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
								ASPHALT - approximately 3 inches thick
								AGREGATE BASE - approximately 3 inches thick
								GRAVELLY SANDY SILT (ML) - dark brown, moist, fine to coarse grained sand, fine gravel (Fill)
	■	5	96.7	21.6				**SILTY SAND (SM)** - dark gray-brown, moist, loose, fine grained sand
5	■	4	86.8	31.3			<0.5	**SILTY LEAN CLAY (CL)** - gray-brown, moist, soft, low plasticity
								SANDY SILT (ML) - gray-brown, wet, soft, fine grained sand
10	■	3	▽ 14:40 83.9	38.3			0.5	**SANDY SILT with CLAY (ML)** - dark olive-brown, moist, soft, low plasticity
15	■	19	93.6	30.4			0.5	**LEAN CLAY (CL)** - dark olive-brown, moist, medium stiff to stiff, medium plasticity
20	■	21					3.0	- becoming very stiff, increasing sand
								Boring terminated at 20 feet Backfilled with portland cement grout
25								
30								

KLEINFELDER

PROJECT NO. 64583-PWGEO

LOG OF BORING NO. B-2

SAN LORENZO HIGH SCHOOL
50 EAST LEWELLING BLVD
SAN LORENZO, CALIFORNIA

PLATE

B-3

Date Completed:	1/5/06
Logged By:	D. Daily
Total Depth:	Approximately 50.0 ft
Drilling method:	8" Hollow Stem Auger
Hammer Wt:	140 lbs., 30" drop
Notes:	Drilled in grass

Surface Elevation: **Estimated 40 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
								TOPSOIL and HIGH ORGANICS - approximately 6 inches thick
							1.5	**LEAN CLAY (CL)** - black, moist, stiff, medium plasticity
	■	13	100.9	20.2	1.1 @ 15.0%	LL=35; PI=18	0.5 1.5	**SILTY LEAN CLAY (CL)** - brown, moist to wet, medium stiff to stiff, low to medium plasticity
5	■	9	102.6	22.0			0.5	
10	■	12					1.0	- increasing sand, decreasing clay content
			15:50				0.5	**GRAVELLY SAND (SW)** - gray-brown, wet, loose, fine grained sand, coarse angular gravels
15	■	6					0.5	**SILT (ML)** - olive-brown, moist, soft, low plasticity
			86.5	38.4				**SILTY LEAN CLAY (CL)** - olive-brown, moist, soft, medium plasticity
20	■	15		27.5		LL=31; PI=12	1.0 2.5 2.5	- increasing silt content, dark brown, becoming stiff
25	■	23	108.2	20.5			3.5	**SILTY LEAN CLAY (CL)** - light brown, moist, very stiff, low plasticity, trace caliche deposits and oxidation staining
30	■	25				LL=27; PI=13	1.0	

KLEINFELDER

PROJECT NO. 64583-PWGEO

LOG OF BORING NO. B-3

SAN LORENZO HIGH SCHOOL
50 EAST LEWELLING BLVD
SAN LORENZO, CALIFORNIA

PLATE **B-4**

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Depth, ft	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests	Pen, tsf	DESCRIPTION
								(Continued from previous plate)
35		33						**SILTY LEAN CLAY (CL)** - continued
40		16						- stiff
								SILTY SAND (SM) - brown, wet, medium dense, fine grained sand
								SILTY LEAN CLAY (CL) - olive-brown, moist, stiff, medium plasticity
45		18					1.0	
							2.0	**CLAYEY SILT (ML)** - olive-brown, moist, very stiff, low plasticity, trace caliche deposits
50		30						Boring terminated at 50 feet Backfilled with portland cement grout
55								
60								
65								
70								

KLEINFELDER

PROJECT NO. 64583-PWGEO

LOG OF BORING NO. B-3

SAN LORENZO HIGH SCHOOL
50 EAST LEWELLING BLVD
SAN LORENZO, CALIFORNIA

PLATE **B-4** (cont'd)

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2/16/06 10:49:26 AM

Date Completed:	1/6/06	Drilling method:	8" Hollow Stem Auger
Logged By:	C. Varela	Hammer Wt:	140 lbs., 30" drop
Total Depth:	Approximately 20.0 ft	Notes:	Drilled in grass

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 40 feet (MSL))
								TOPSOIL - approximately 6 inches thick
							1.5-2.0	**SILTY LEAN CLAY (CL)** - dark brown, moist, medium stiff, medium plasticity, angular gravel (Fill)
		64	111.9	14.7			>4.5	**CLAYEY SILT (ML)** - brown, moist, hard, medium plasticity (Fill)
5		10	80.6	19.1				**SANDY SILT (ML)** - brown, moist, medium stiff, low plasticity
								SILTY SAND (SM) - olive-brown, moist, loose, slight cementation, fine grained sand
10		10	95.3	11.3			>4.0	**CLAYEY SILT (ML)** - dark brown, moist, medium stiff, medium plasticity
			8:30					
15		5				Passing -#200=66%	<0.5	**SANDY SILT (ML)** - olive brown, wet, soft, low plasticity, fine grained sand
							1.5	**SILTY LEAN CLAY (CL)** - gray, moist, stiff, medium plasticity, varying sand content
20		12						Boring terminated at 20 feet Backfilled with portland cement grout
25								
30								

KLEINFELDER

PROJECT NO. 64583-PWGEO

LOG OF BORING NO. B-4

SAN LORENZO HIGH SCHOOL
50 EAST LEWELLING BLVD
SAN LORENZO, CALIFORNIA

PLATE **B-5**

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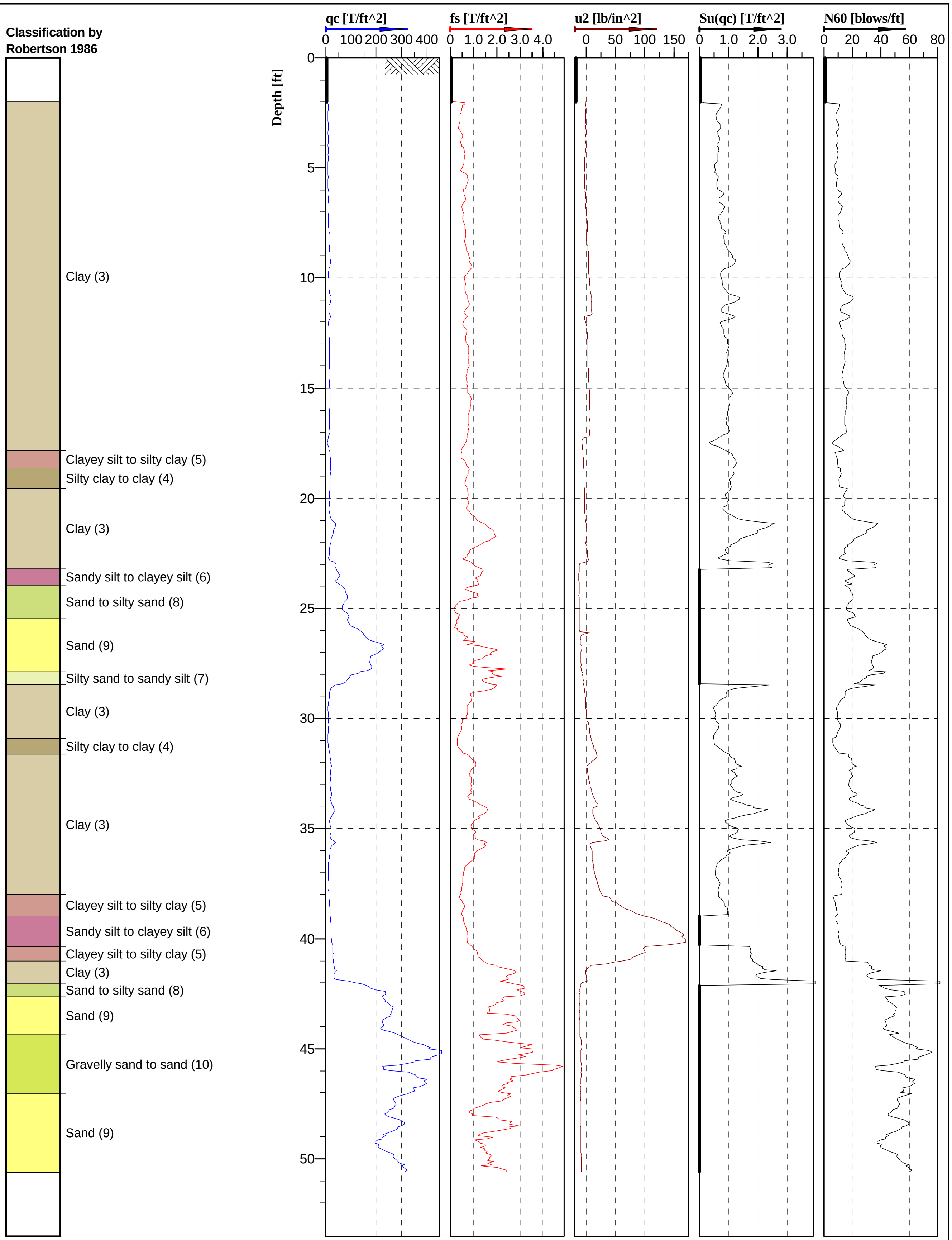


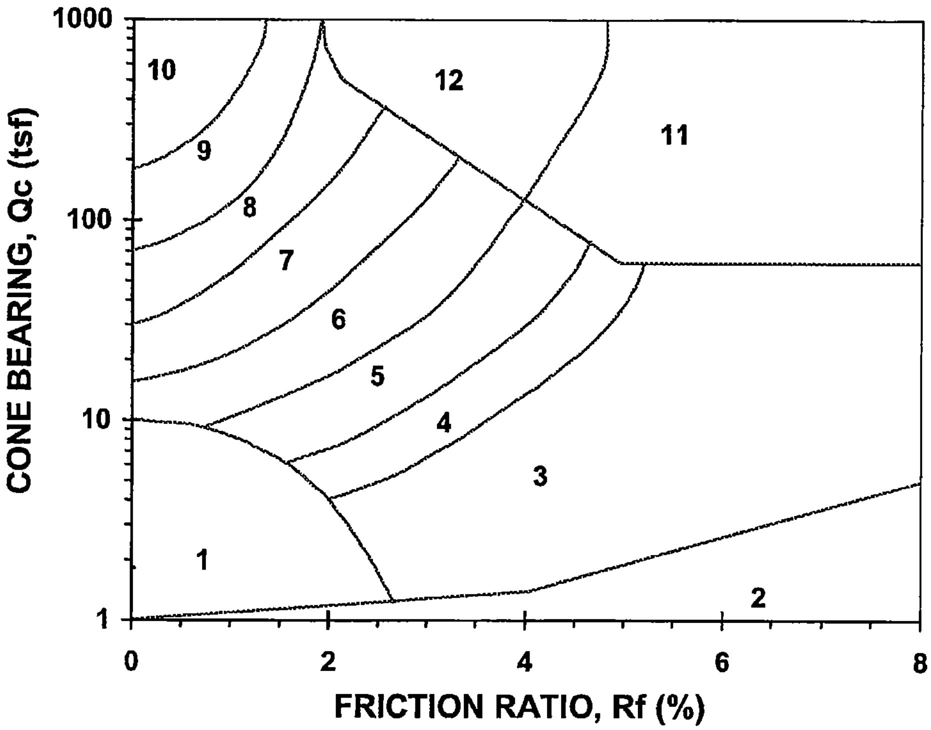
CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

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	

SIMPLIFIED SOIL BEHAVIOR TYPE CLASSIFICATION FOR STANDARD ELECTRONIC CONE PENETROMETER



ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2		Sensitive Fine Grained
2	1	for Zones 1 to 6	Organic Material
3	1	10 for Qc <= 9 tsf	CLAY
4	1.5	12 for Qc = 9 to 12 tsf	Silty CLAY to CLAY
5	2	15 for Qc > 12 tsf	Clayey SILT to Silty CLAY
6	2.5		Sandy SILT to Clayey SILT
7	3	---	Silty SAND to Sandy SILT
8	4	---	SAND to Silty SAND
9	5	---	SAND
10	6	---	Gravelly SAND to SAND
11	1	15	Very Stiff Fine Grained (*)
12	2	---	SAND to Clayey SAND (*)

(*) Overconsolidated or Cemented

Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc*100 = Friction Ratio

References: [1]Robertson, 1986, Olsen, 1988
[2]Bonaparte & Mitchell, 1979 (young bay mud Qc <= 9)
[2]Estimated from local experience (fine grained soils Qc > 9)

Note: Testing performed in accordance with ASTM D3441

John Sarmiento & Associates
Cone Penetrometer Testing Services

PROJECT: San Lorenzo School District
LOCATION: San Lorenzo CA
PROJ. NO.: 64583(KLF-115)

CPT NO.: LORENZO-1
DATE : 02-02-2006
Groundwater estimated at 10.0 feet
Terminated at 50.0 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
0.53	220.90	2.580	1.2	44	71	0.06	45	----	SAND	120-130
1.03	85.70	3.170	3.7	43	69	0.13	----	11.42	Clayey SILT to Silty CLAY	130-140
1.58	54.10	1.080	2.0	18	29	0.20	37	----	Silty SAND to Sandy SILT	''
2.03	16.40	0.540	3.3	11	17	0.26	----	2.17	Silty CLAY to CLAY	120-130
2.53	8.60	0.160	1.9	4	7	0.31	----	1.69	Clayey SILT to Silty CLAY	100-110
3.01	1.60	0.110	6.9	2	3	0.35	----	0.28	Organic Material	85-90
3.58	3.90	0.170	4.4	4	6	0.41	----	0.74	CLAY	90-100
4.08	10.80	0.160	1.5	5	9	0.46	----	1.76	Clayey SILT to Silty CLAY	100-110
4.58	2.70	0.130	4.8	3	4	0.51	----	0.49	CLAY	90-100
5.00	4.10	0.230	5.6	4	7	0.55	----	0.77	''	''
5.55	3.90	0.230	5.9	4	6	0.60	----	0.72	''	''
6.05	3.90	0.190	4.9	4	6	0.65	----	0.72	''	''
6.55	3.40	0.120	3.5	3	5	0.70	----	0.61	''	''
7.05	2.50	0.080	3.2	3	4	0.74	----	0.43	''	85-90
7.55	3.20	0.100	3.1	3	5	0.79	----	0.56	''	90-100
8.05	2.90	0.130	4.5	3	5	0.83	----	0.50	''	''
8.56	3.20	0.110	3.4	3	5	0.88	----	0.55	''	''
9.06	2.10	0.110	5.2	2	3	0.93	----	0.33	''	85-90
9.56	1.70	0.080	4.7	2	2	0.97	----	0.24	Organic Material	''
10.06	3.20	0.080	2.5	3	5	1.01	----	0.54	CLAY	''
10.56	3.70	0.160	4.3	4	5	1.06	----	0.63	''	90-100
11.05	4.60	0.200	4.3	5	6	1.11	----	0.81	''	''
11.55	7.50	0.150	2.0	5	7	1.15	----	1.38	Silty CLAY to CLAY	''
12.03	3.60	0.120	3.3	4	5	1.20	----	0.60	CLAY	''
12.53	3.60	0.150	4.2	4	5	1.25	----	0.60	''	''
13.03	4.80	0.170	3.5	5	7	1.30	----	0.83	''	''
13.53	8.40	0.450	5.4	8	11	1.35	----	1.54	''	110-120
14.02	11.60	0.570	4.9	12	15	1.41	----	1.82	''	120-130
14.51	10.30	0.560	5.4	10	14	1.48	----	1.59	''	''
15.01	8.90	0.480	5.4	9	12	1.53	----	1.63	''	110-120
15.50	11.20	0.390	3.5	7	10	1.59	----	1.73	Silty CLAY to CLAY	''
16.08	12.80	0.580	4.5	13	16	1.66	----	1.60	CLAY	120-130
16.57	15.20	0.810	5.3	15	19	1.72	----	1.91	''	''
17.06	16.50	0.750	4.5	17	20	1.78	----	2.08	''	''
17.55	16.60	0.870	5.2	17	20	1.85	----	2.09	''	''
18.04	17.70	0.970	5.5	18	21	1.91	----	2.23	''	130-140
18.56	20.70	0.930	4.5	21	25	1.98	----	2.63	''	''
19.04	23.10	1.120	4.8	23	27	2.05	----	2.94	''	''
19.52	21.70	1.270	5.9	22	25	2.11	----	2.75	''	''
20.07	22.90	1.080	4.7	23	26	2.19	----	2.91	''	''
20.55	26.90	1.060	3.9	18	20	2.25	----	3.44	Silty CLAY to CLAY	''
21.02	23.80	1.230	5.2	24	27	2.31	----	3.02	CLAY	''
21.56	16.10	1.060	6.6	16	18	2.39	----	1.99	''	''
22.06	22.80	1.060	4.6	23	25	2.46	----	2.88	''	''
22.54	26.60	0.780	2.9	13	14	2.52	----	3.38	Clayey SILT to Silty CLAY	''
23.02	17.20	0.870	5.1	17	18	2.58	----	2.12	CLAY	120-130
23.58	8.70	0.380	4.4	9	9	2.64	----	1.48	''	110-120
24.06	24.10	1.200	5.0	24	25	2.71	----	3.03	''	130-140
24.53	22.90	1.570	6.9	23	24	2.77	----	2.87	''	''
25.05	46.50	1.850	4.0	23	24	2.84	----	6.01	Clayey SILT to Silty CLAY	''
25.52	50.40	2.420	4.8	34	34	2.91	----	6.53	Silty CLAY to CLAY	''
26.01	88.10	2.140	2.4	29	30	2.97	37	----	Silty SAND to Sandy SILT	''
26.53	72.70	2.520	3.5	29	29	3.04	----	9.49	Sandy SILT to Clayey SILT	''
27.05	62.50	2.090	3.3	25	25	3.11	----	8.13	''	''
27.58	46.70	1.730	3.7	23	23	3.18	----	6.01	Clayey SILT to Silty CLAY	''
28.02	19.10	0.820	4.3	19	19	3.24	----	2.33	CLAY	120-130
28.51	12.40	0.430	3.5	8	8	3.30	----	1.43	Silty CLAY to CLAY	''
29.05	9.30	0.330	3.5	9	9	3.36	----	1.27	CLAY	110-120
29.51	8.60	0.360	4.2	9	9	3.42	----	1.38	''	''
30.06	41.60	1.240	3.0	17	17	3.49	----	5.31	Sandy SILT to Clayey SILT	130-140
30.53	52.90	1.120	2.1	21	21	3.55	----	6.82	''	''
31.00	29.30	1.860	6.3	29	29	3.62	----	3.67	CLAY	''
31.54	52.80	1.160	2.2	21	21	3.69	----	6.79	Sandy SILT to Clayey SILT	''
32.04	18.50	0.890	4.8	19	18	3.75	----	2.22	CLAY	120-130
32.51	13.40	0.470	3.5	9	9	3.81	----	1.53	Silty CLAY to CLAY	''

John Sarmiento & Associates
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PROJECT: San Lorenzo School District
LOCATION: San Lorenzo CA
PROJ. NO.: 64583(KLF-115)

CPT NO.: LORENZO-1
DATE : 02-02-2006
Groundwater estimated at 10.0 feet
Terminated at 50.0 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
33.04	16.70	0.700	4.2	17	17	3.88	----	1.97	CLAY	''
33.51	18.70	0.830	4.4	19	19	3.93	----	2.23	''	''
34.05	17.00	0.810	4.8	17	17	4.00	----	2.00	''	''
34.52	14.50	0.730	5.0	15	14	4.06	----	1.66	''	''
35.08	17.60	0.790	4.5	18	17	4.13	----	2.07	''	''
35.56	10.90	0.530	4.9	11	11	4.19	----	1.47	''	''
36.03	10.60	0.420	4.0	11	10	4.25	----	1.41	''	110-120
36.50	12.80	0.460	3.6	9	8	4.30	----	1.42	Silty CLAY to CLAY	120-130
37.06	15.50	0.610	3.9	10	10	4.37	----	1.78	''	''
37.53	14.20	0.590	4.2	14	13	4.43	----	1.60	CLAY	''
38.07	18.40	0.920	5.0	18	17	4.51	----	2.15	''	130-140
38.57	13.90	0.550	4.0	14	13	4.57	----	1.55	''	120-130
39.04	11.70	0.440	3.8	12	10	4.63	----	1.56	''	''
39.51	10.90	0.380	3.5	7	6	4.68	----	1.43	Silty CLAY to CLAY	110-120
40.06	10.20	0.340	3.3	7	6	4.74	----	1.30	''	''
40.53	10.30	0.320	3.1	7	6	4.80	----	1.32	''	''
41.08	13.10	0.420	3.2	9	7	4.87	----	1.42	''	120-130
41.50	15.50	0.570	3.7	10	9	4.92	----	1.74	''	''
42.02	13.80	0.420	3.0	9	8	4.98	----	1.51	''	''
42.57	18.90	0.670	3.5	13	10	5.05	----	2.18	''	''
43.04	17.70	0.600	3.4	12	10	5.11	----	2.02	''	''
43.51	17.90	1.180	6.6	18	15	5.18	----	2.04	CLAY	130-140
44.04	17.50	0.630	3.6	12	10	5.24	----	1.98	Silty CLAY to CLAY	120-130
44.51	18.10	0.630	3.5	12	10	5.30	----	2.06	''	''
45.05	23.00	0.770	3.3	12	9	5.37	----	2.71	Clayey SILT to Silty CLAY	130-140
45.51	20.60	0.790	3.8	14	11	5.43	----	2.38	Silty CLAY to CLAY	120-130
46.05	24.00	0.930	3.9	16	13	5.50	----	2.83	''	130-140
46.51	18.70	0.820	4.4	19	15	5.56	----	2.12	CLAY	120-130
47.06	18.70	0.750	4.0	12	10	5.63	----	2.12	Silty CLAY to CLAY	''
47.56	22.30	0.720	3.2	11	9	5.69	----	2.59	Clayey SILT to Silty CLAY	''
48.03	19.80	0.500	2.5	10	8	5.75	----	2.26	''	''
48.57	14.70	0.390	2.7	7	6	5.82	----	1.57	''	''
49.03	17.00	0.530	3.1	9	7	5.88	----	1.87	''	''
49.57	22.80	1.370	6.0	23	18	5.95	----	2.64	CLAY	130-140
50.01	124.10	2.300	1.9	41	32	6.01	38	----	Silty SAND to Sandy SILT	''

DEPTH = Sampling interval (2 inches)
Qc = Tip bearing resistance — TotStr = Total Stress using est. density**
Fs = Sleeve friction resistance — Phi = Soil friction angle*
Rf = Tip/Sleeve ratio — Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf)
SPT = Equivalent Standard Penetration Test* (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)
References: * Robertson and Campanella, 1988
** Olsen, 1989 *** Durgunoglu & Mitchell, 1975

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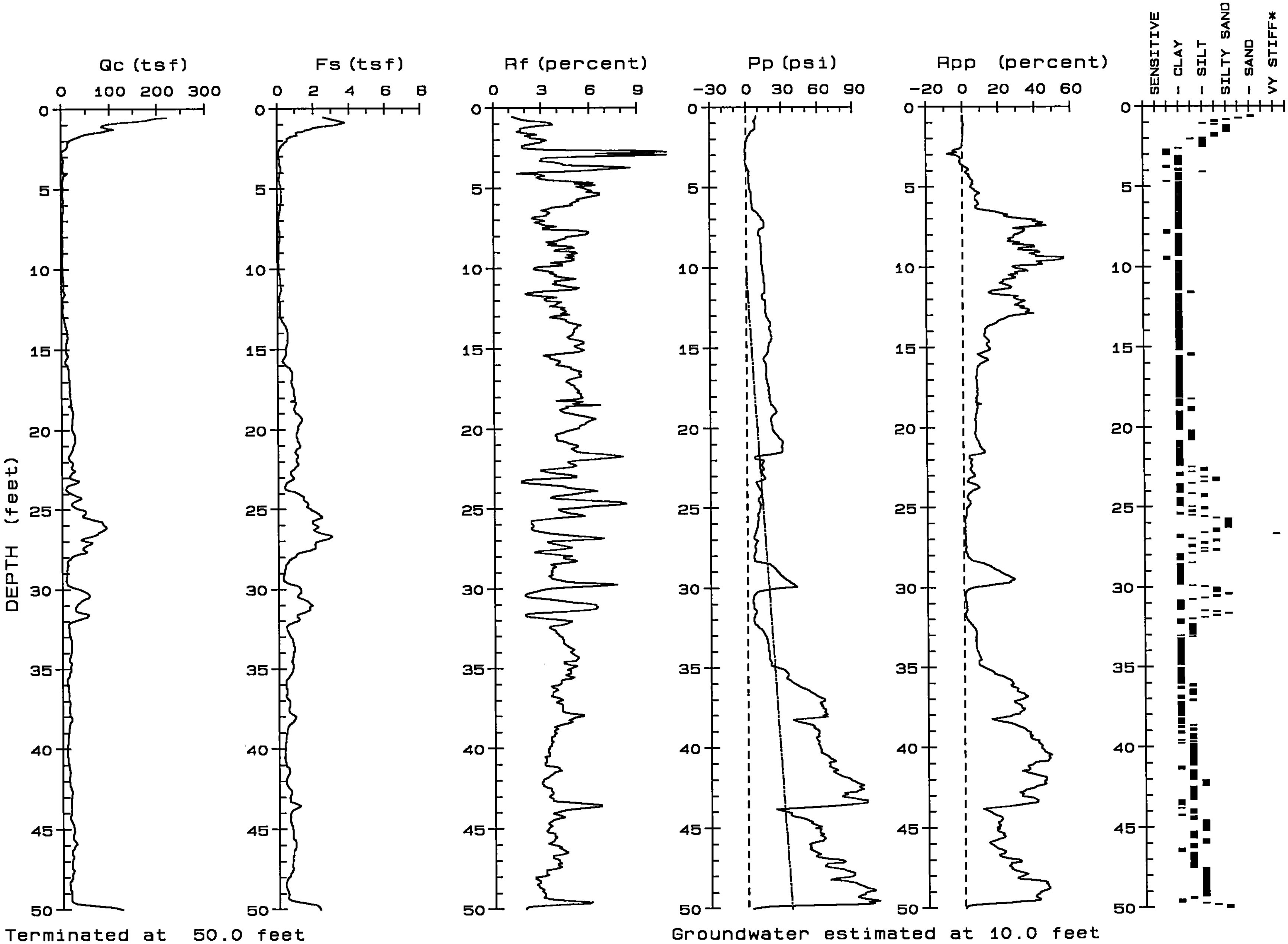

PROJECT: San Lorenzo School District
LOCATION: San Lorenzo CA
PROJ. NO.: 64583 (KLF-115)

CPT NO.: LORENZO-1
DATE : 02-02-2006

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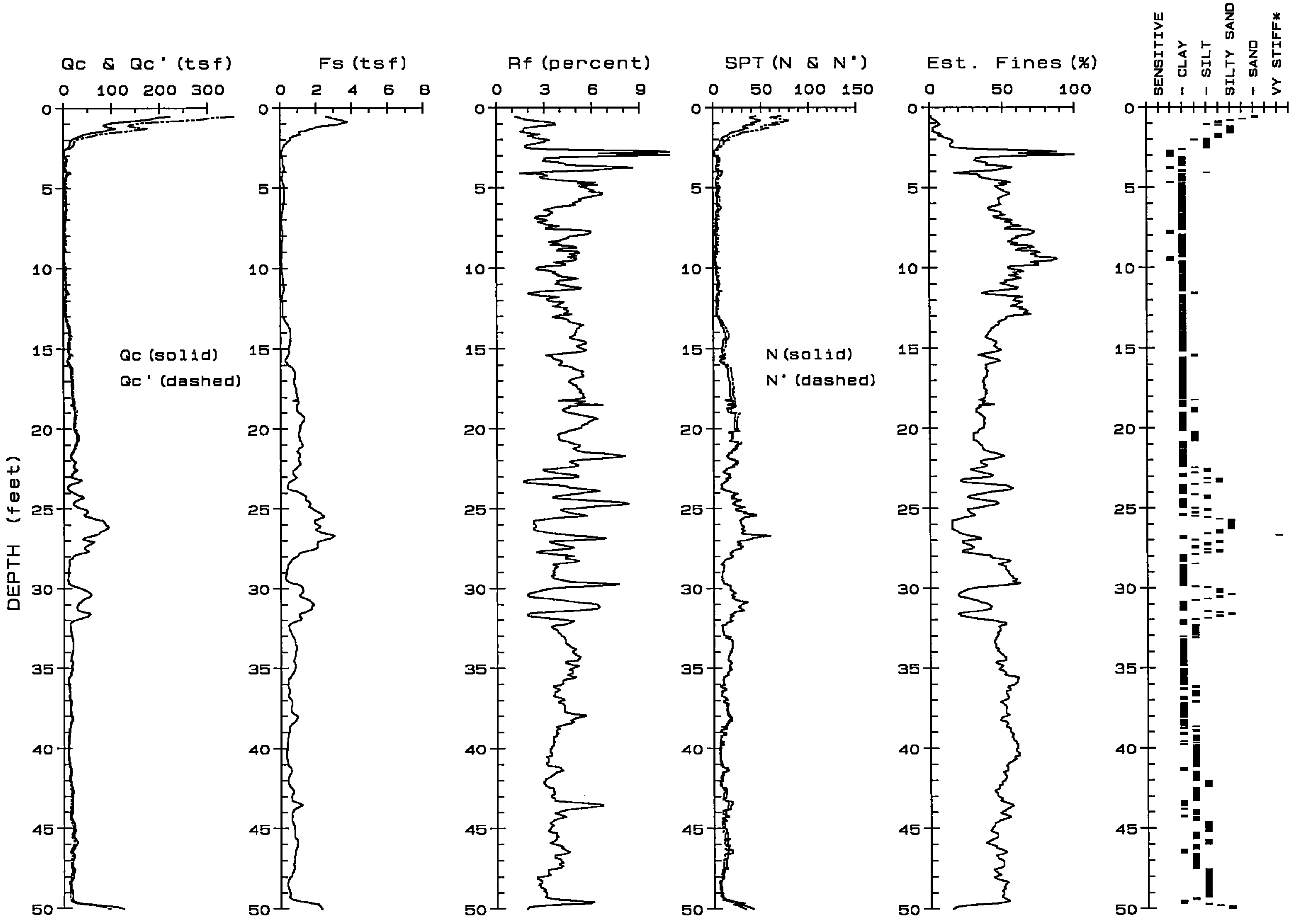

Terminated at 50.0 feet

Groundwater estimated at 10.0 feet

PROJECT: San Lorenzo School District
LOCATION: San Lorenzo CA
PROJ. NO.: 64583 (KLF-115)

CPT NO.: LORENZO-1
DATE : 02-02-2006

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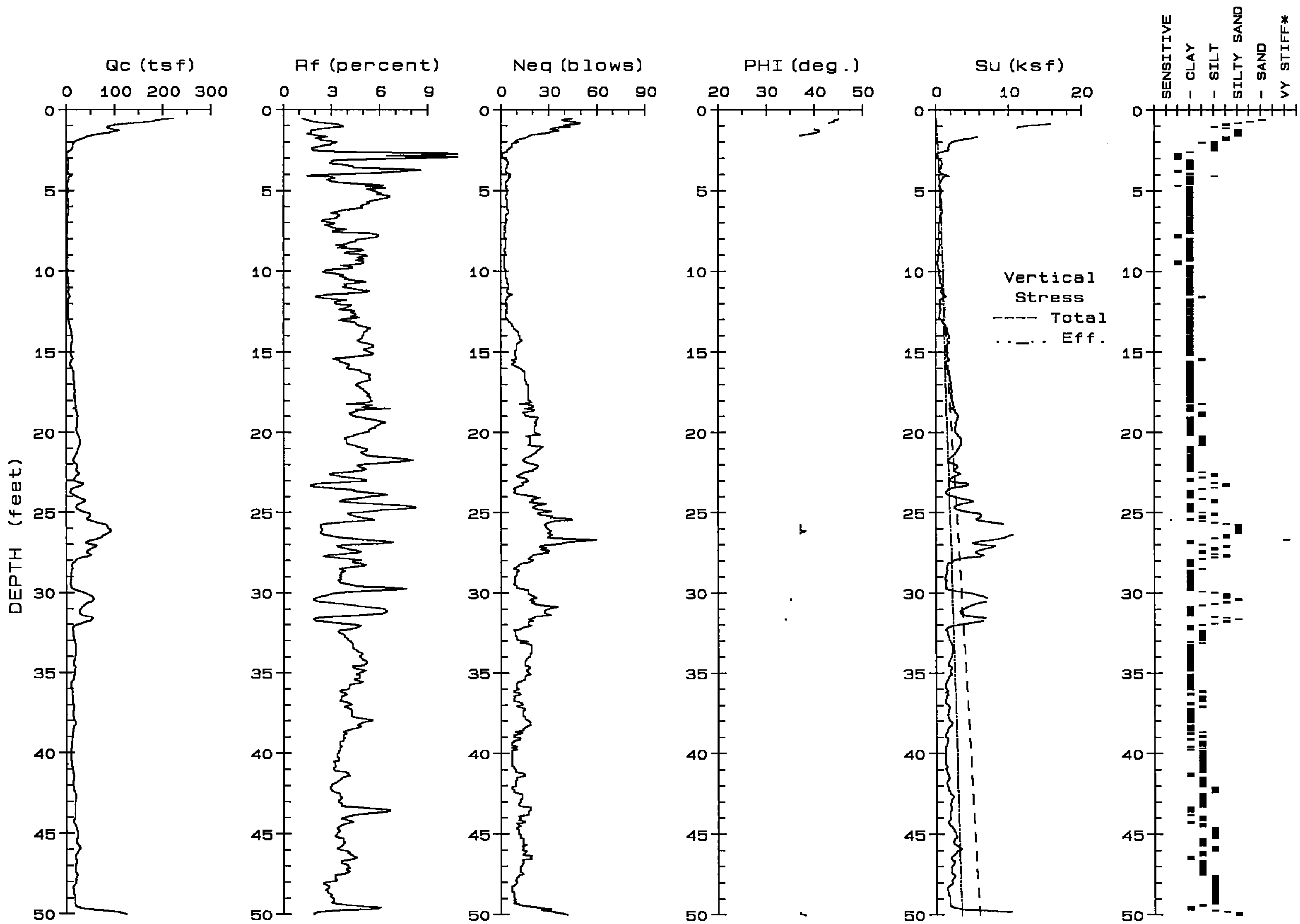

Terminated at 50.0 feet

Groundwater estimated at 10.0 feet

PROJECT: San Lorenzo School District
LOCATION: San Lorenzo CA
PROJ. NO.: 64583 (KLF-115)

CPT NO.: LORENZO-1
DATE : 02-02-2006

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APPENDIX B

Boring B-1

Mw= **6.8** MSF= **1.28**

a_{max}= **0.55g**

GW Depth at time of drilling = **6.0 ft**

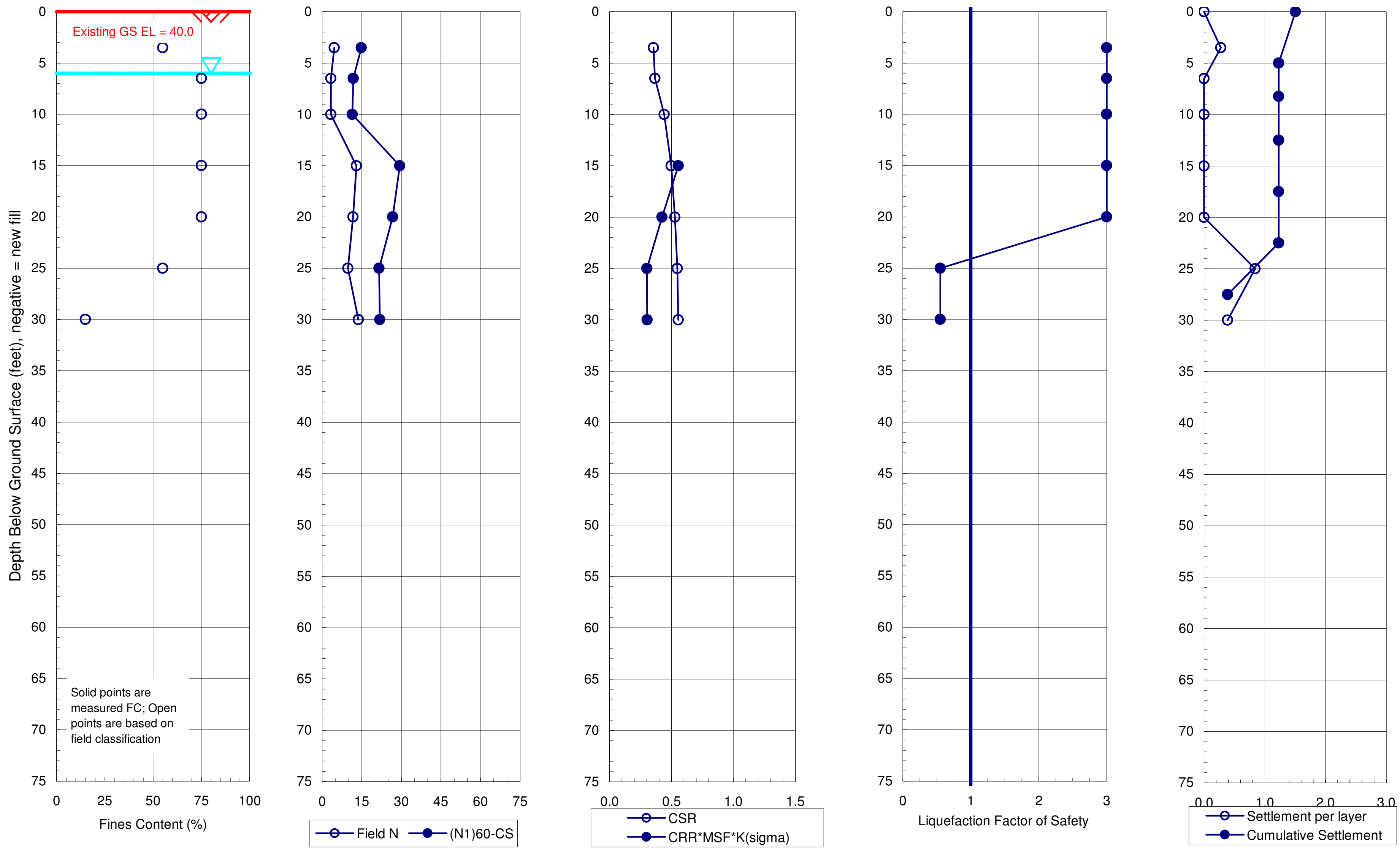
Design GW Depth = **6.0 ft**

Ground El. at Drilling = **40.0 ft**

Final Ground El. = **40.0 ft**

By: JFM

Checked:



Liquefaction calculations based on Youd, et al (2001) and NCEER (1997)

Settlement calculation based on Tokimatsu Seed (1987)

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San Lorenzo High School
San Lorenzo, California
Project No. 105356

LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT ANALYSIS

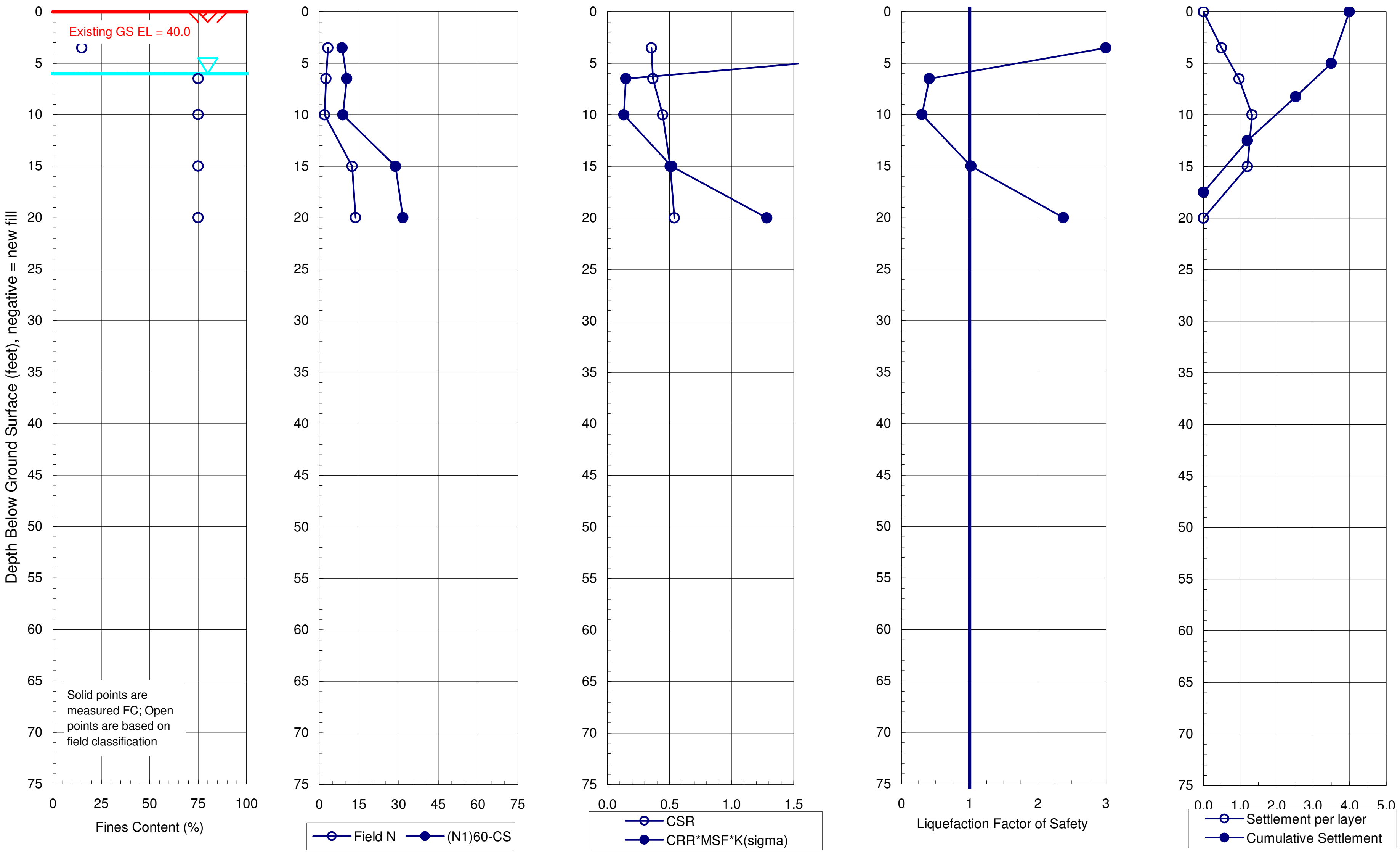
Date: July 2009

FIGURE B-1

Boring B-2

Mw= **6.8** MSF= **1.28** GW Depth at time of drilling = **6.0 ft** Ground El. at Drilling = **40.0 ft** By: JFM

a_{max}= **0.55g** Design GW Depth = **6.0 ft** Final Ground El. = **40.0 ft** Checked:



Liquefaction calculations based on Youd, et al (2001) and NCEER (1997)

Settlement calculation based on Tokimatsu Seed (1987)

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LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT ANALYSIS

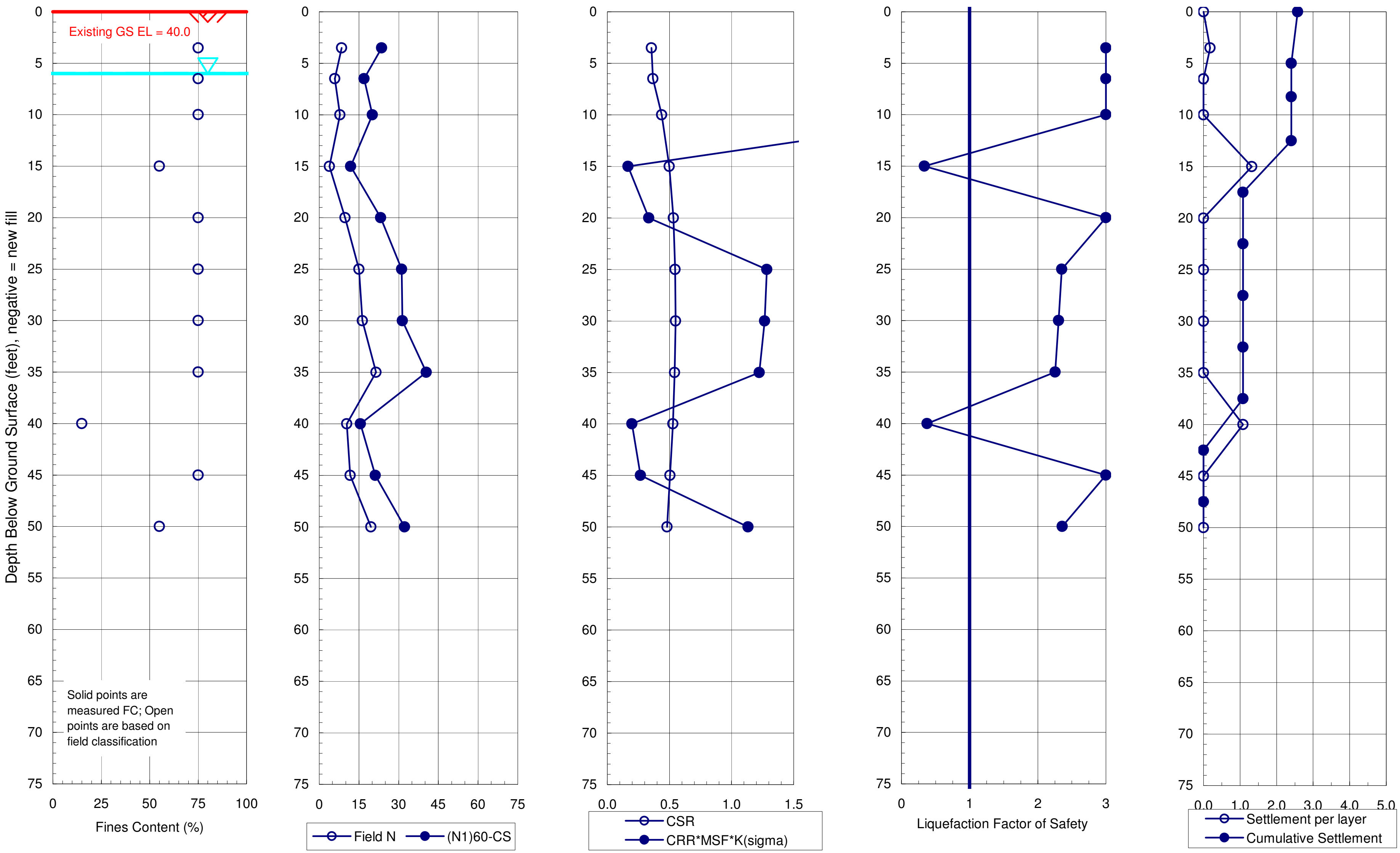
Date: July 2009

FIGURE B-2

Boring B-3

Mw= **6.8** MSF= **1.28** GW Depth at time of drilling = **6.0 ft** Ground El. at Drilling = **40.0 ft** By: JFM

a_{max}= **0.55g** Design GW Depth = **6.0 ft** Final Ground El. = **40.0 ft** Checked:



Liquefaction calculations based on Youd, et al (2001) and NCEER (1997)

Settlement calculation based on Tokimatsu Seed (1987)

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LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT ANALYSIS

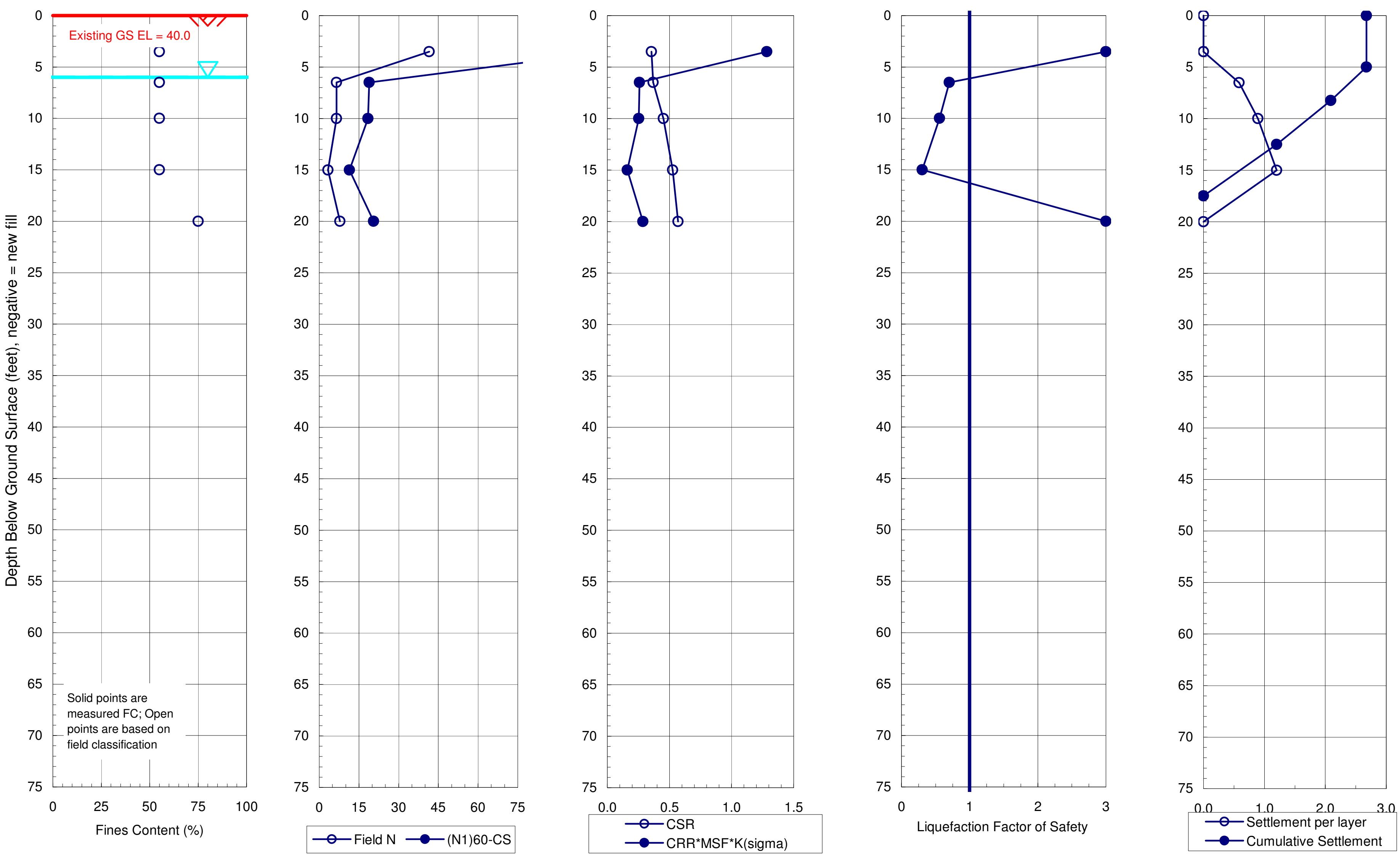
Date: July 2009

FIGURE B-3

Boring B-4

Mw= **6.8** MSF= **1.28** GW Depth at time of drilling = **6.0 ft** Ground El. at Drilling = **40.0 ft** By: JFM

a_{max}= **0.55g** Design GW Depth = **6.0 ft** Final Ground El. = **40.0 ft** Checked:



Liquefaction calculations based on Youd, et al (2001) and NCEER (1997)

Settlement calculation based on Tokimatsu Seed (1987)

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Project No. 105356

LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT ANALYSIS

Date: July 2009

FIGURE B-4

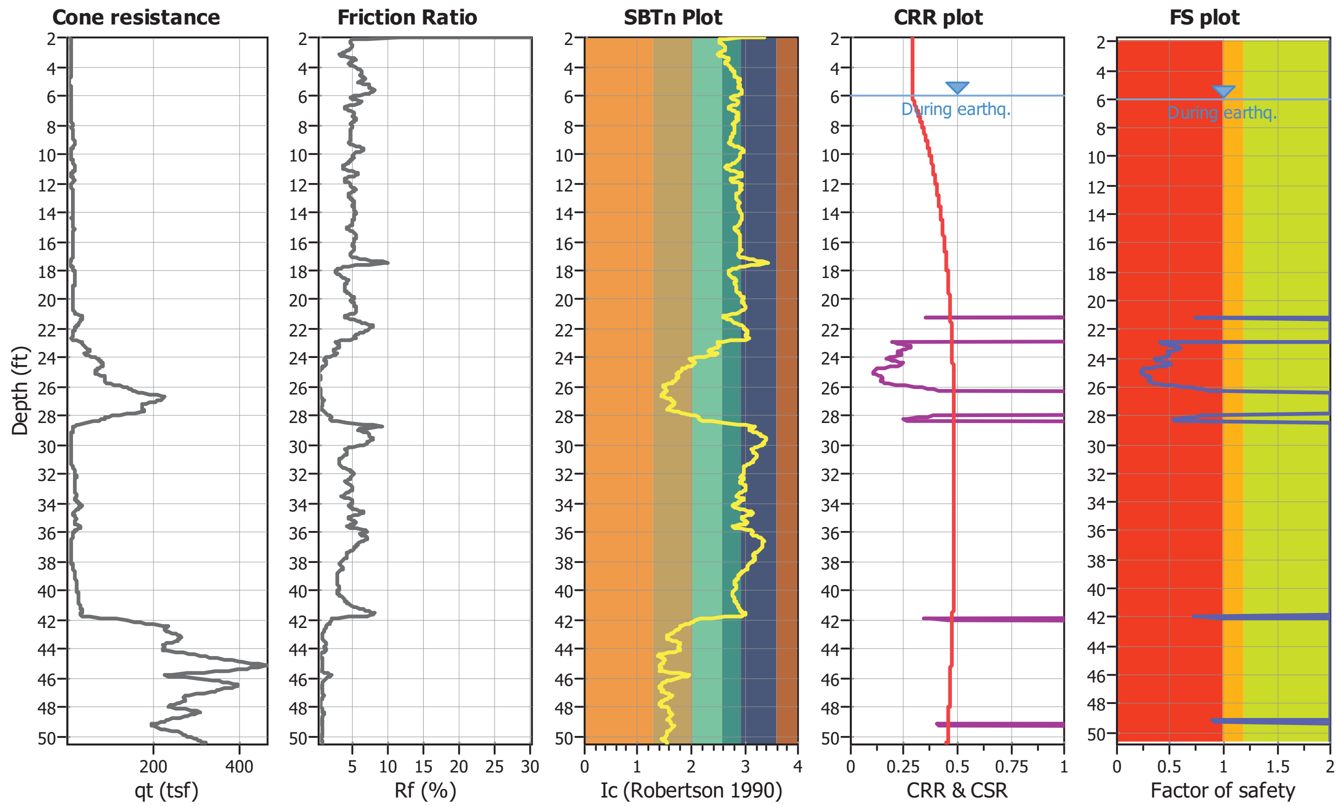
LIQUEFACTION ANALYSIS REPORT

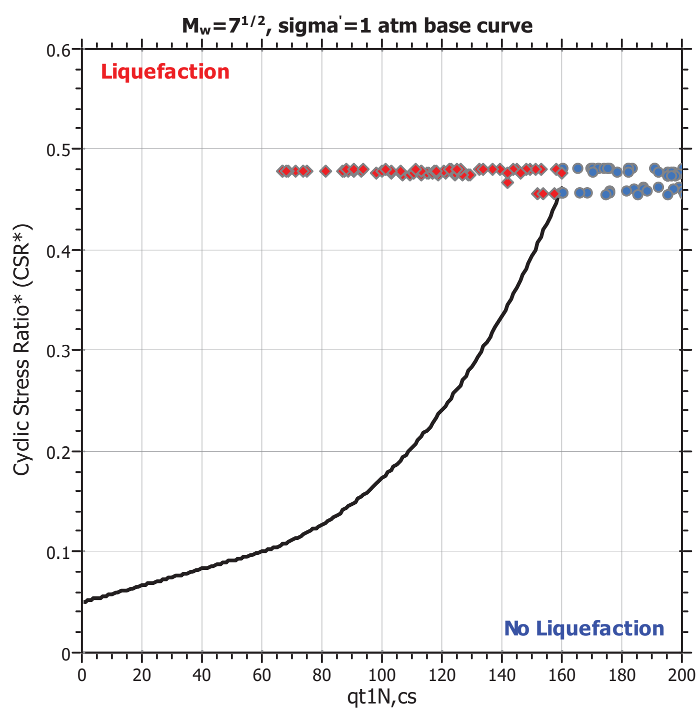
Project title : SLZHS Measure O Additions **Location :**

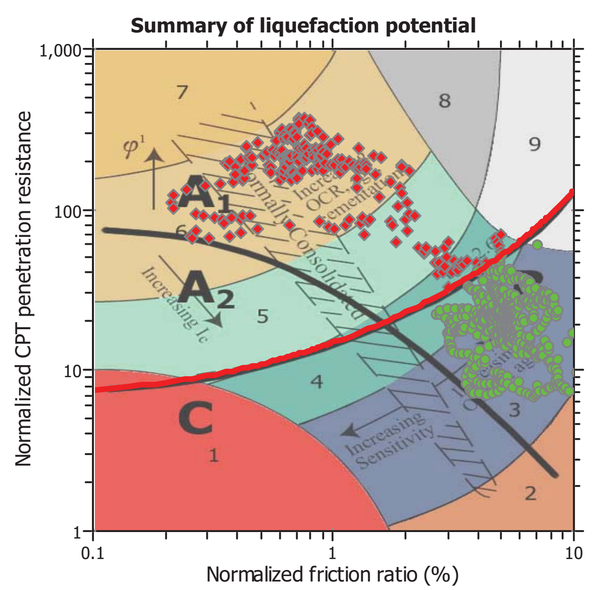
CPT file : CPT-01

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		

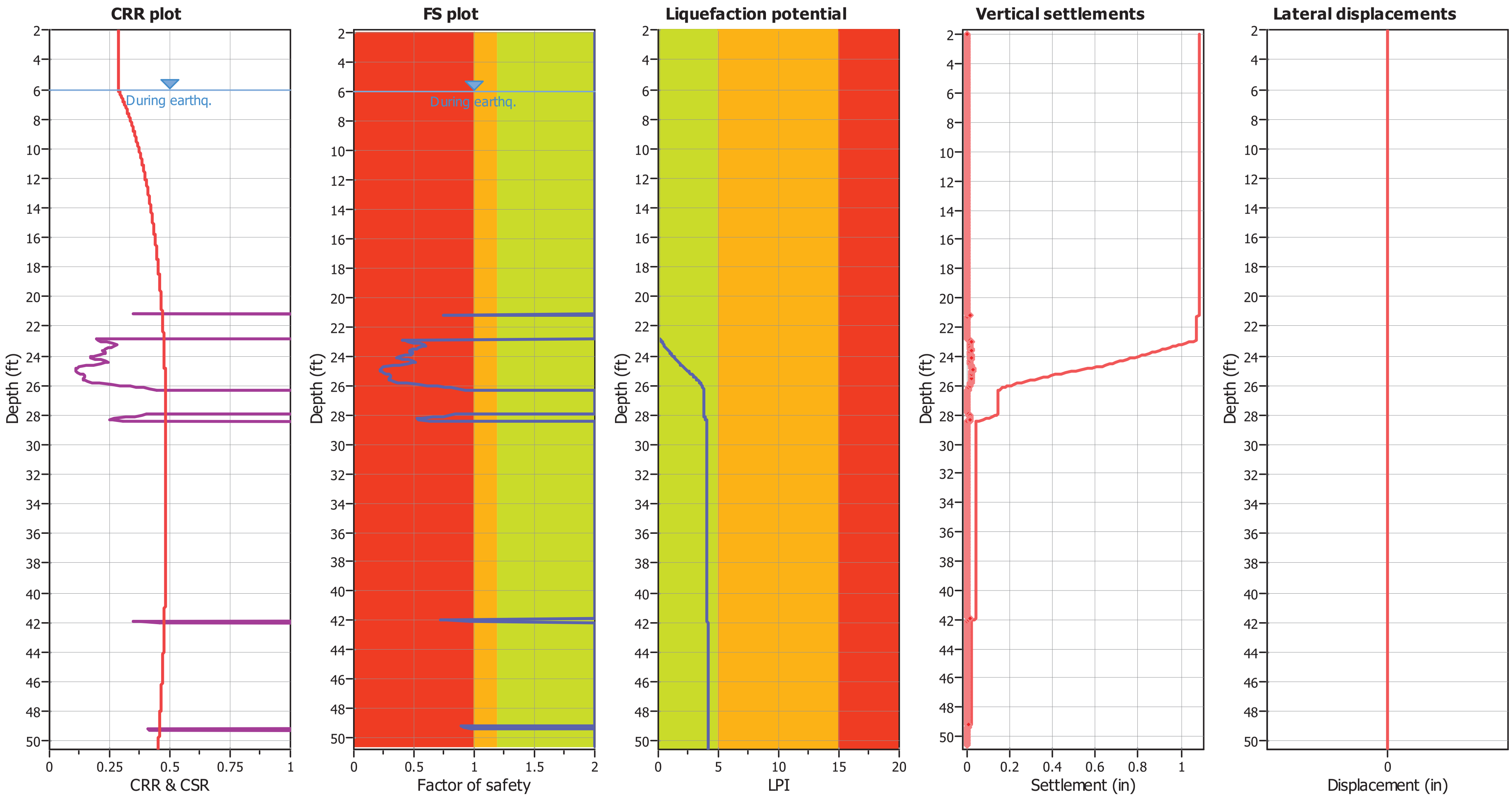






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and 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

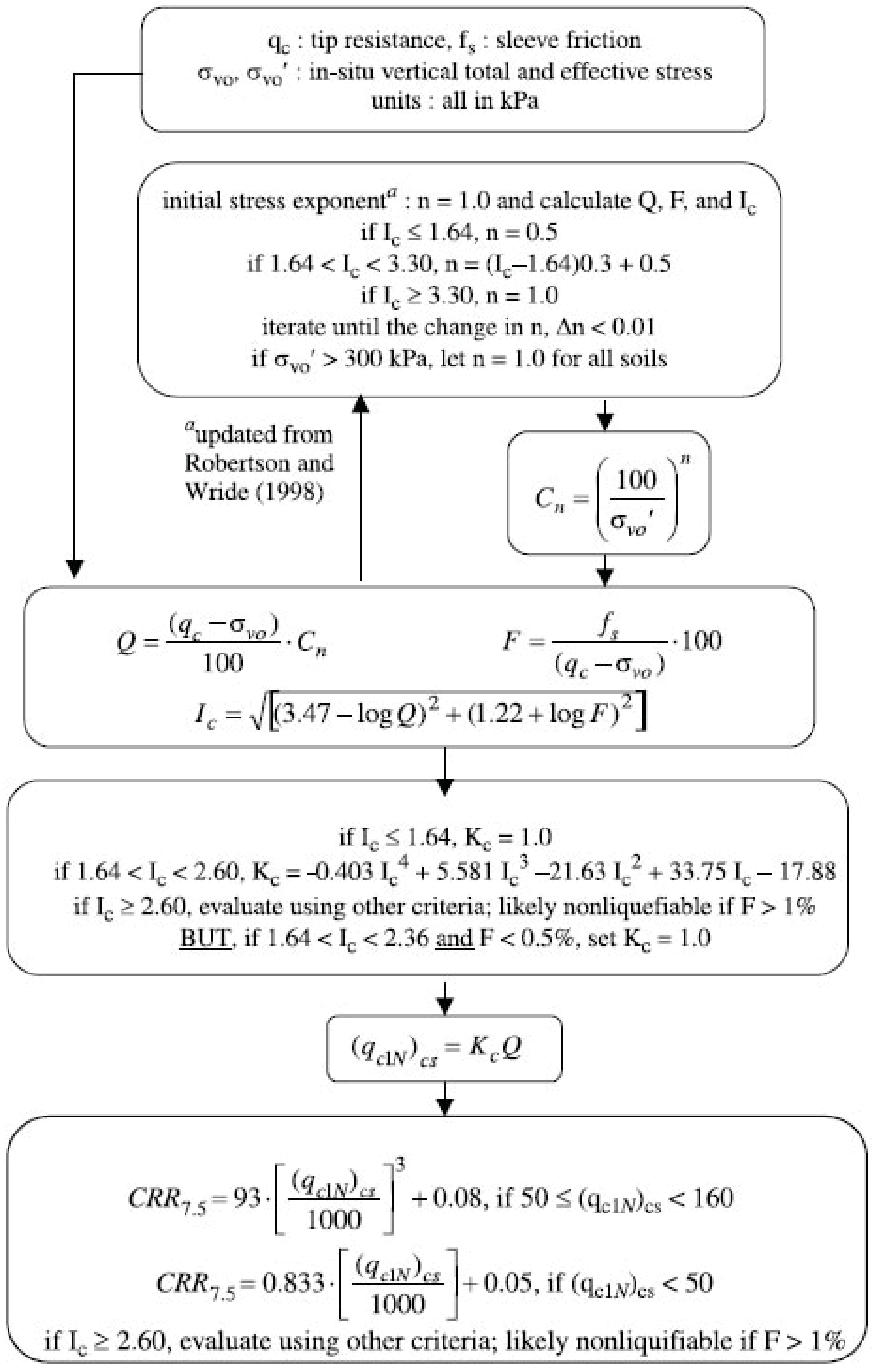
- Liquefied
- Marginally liquefied
- Non-liquefied

LPI color scheme

- Very high risk
- High risk
- Low risk

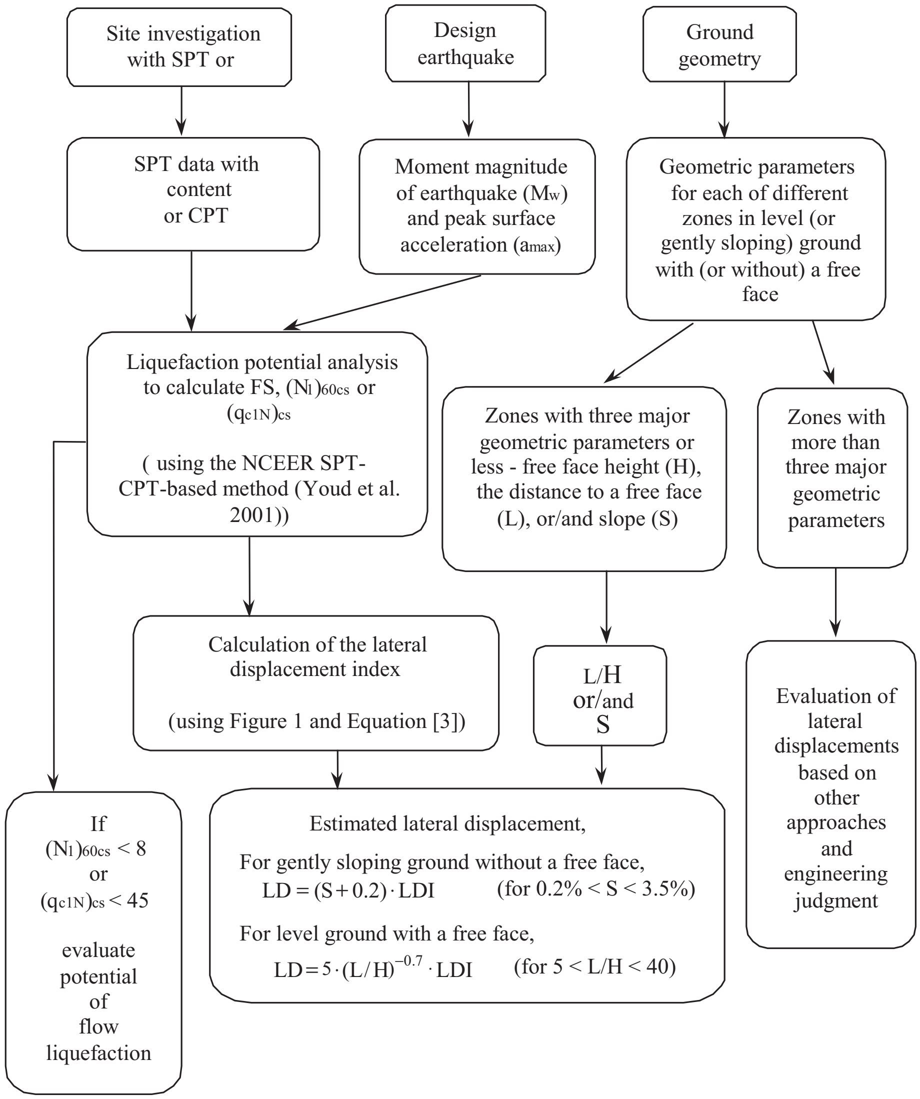
Procedure for the evaluation of soil liquefaction resistance

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]:

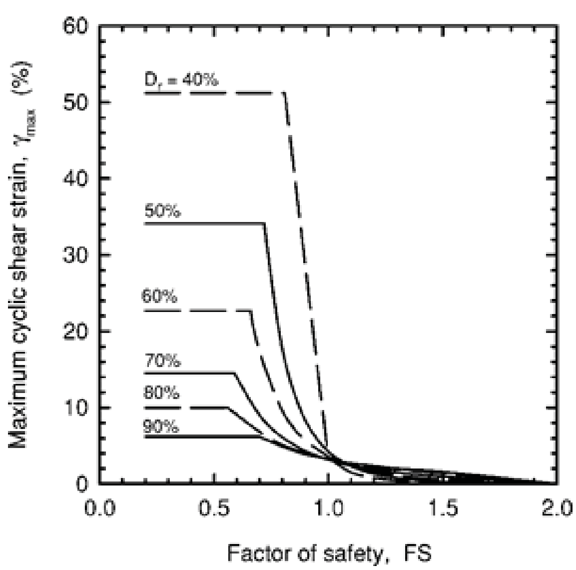


[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

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.

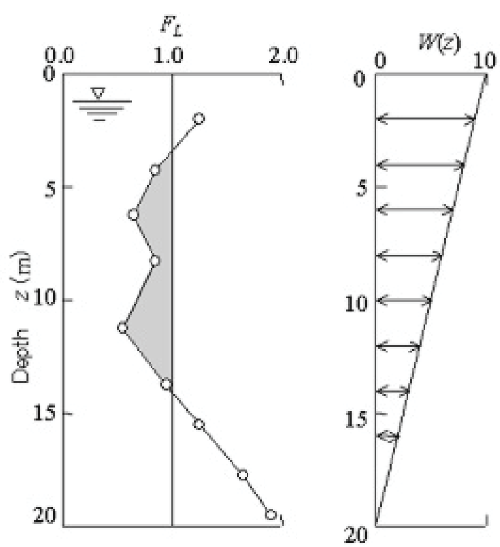
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

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

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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