RUTHERFORD & CHEKENE

report

VOLUME 2

GEOTECHNICAL DATA AND INVESTIGATION REPORT

Linac Coherent Light Source (LCLS) Project
Stanford Linear Accelerator Center
Menlo Park, California

Prepared for
Stanford Linear Accelerator Center

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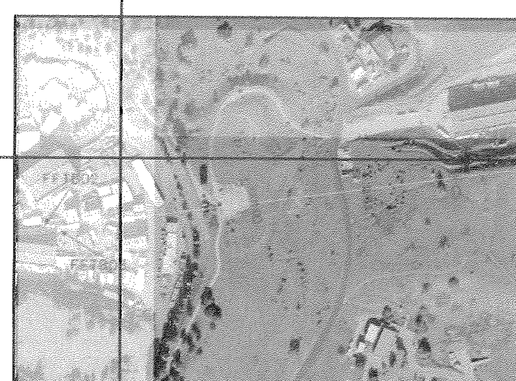

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INTRODUCTION

INTRODUCTION

General

This report summarizes the results of the engineering analysis of geotechnical data that were presented in Volume 1 of this report. The report also summarizes the recommendations that were developed for design of the proposed Linac Coherent Light Source (LCLS) Project, to be located at the Stanford Linear Accelerator Center (SLAC) at 2575 Sand Hill Road in Menlo Park, California.

The geotechnical investigation program consists of the following two components:

1. Obtain Geotechnical Data from Field Exploration and Laboratory Test Programs. The findings from that component of the investigation are presented in Volume 1 of the report.

2. Develop Geotechnical Recommendations based on the analysis of Geotechnical Data. The results of this component of the investigation are presented in this volume of the report.

We note that all elevations in this report are based on the MSL Datum, unless otherwise noted.

Project Description

For ease of reference, we are providing a slightly abridged version of the project description contained in Volume 1 of the report for inclusion in this volume of the report.

We understand that the proposed LCLS facility is aimed at creating a new type of x-ray light source from a single pass free electron laser (FEL), which would have a peak brightness ten orders of magnitude greater and with faster pulses than the most intense synchrotrons currently available. The higher peak brightness would make it possible to examine much smaller particles, and the faster pulses would make it possible to evaluate changes within a very short time frame.

The new LCLS facility will use the last one-third of the Linac, which will be modified by adding two magnetic bunch compressors. Most of the Linac and its infrastructure will however remain unchanged. The existing components in the FFTB tunnel will be removed and replaced by a new 426-foot (130-meter) undulator and associated equipment.

The key elements of the new construction will include the following:

1. SLAC Research Yard Modifications – The Beam Transport Hall (BTH) will be built in the Research Yard. To facilitate this construction, a number of existing buildings and underground utilities will be modified and the Final Focus Test Beam (FFTB) Facility will be demolished and removed. A north-south overpass roadway will be constructed on

top of the cut slope on the east side of the research yard to allow vehicular access to the north and south sides of the research yard.

2. Beam Transport Hall (BTH) – A tunnel-like structure crossing the research yard that will house the LCLS electron beam line.

3. Undulator Hall (UH) – the UH is a tunnel-like section designed to house about thirty-three specially-designed undulator magnets, which extends from the BTH to the Front End Enclosure (FEE). The Undulator Hall has tight alignment constraints, which places stringent requirements on its foundation and temperature stability.

4. Front End Enclosure (FEE) – The FEE will house varied optics and electron beam separation components and is located immediately downstream of the Undulator Hall. The FEE is 131 feet (40 meters) in length and has environmental requirements similar to those for the Undulator Hall.

5. Electron Beam Dump Enclosure (BDE) – The BDE is a high radiation enclosure that contains a beam dump designed to absorb the energy of the LCLS electron beam. The BDE is immediately downstream of the FEE and is approximately 26 feet (eight meters) square in cross section.

6. Near Experimental Hall (NEH) – The NEH is the first of two experimental facilities, and is located immediately downstream of the BDE. The proposed building site is approximately 108 feet (33 meters) in length with a ground floor level approximately 23 feet (seven meters) below existing grade.

7. X-Ray Transport and Diagnostics Tunnel (XDT) – The XDT is a 951-foot (290-meter) long tunnel section that extends from the NEH to the Far Experimental Hall (FEH), which will house specially-designed optical equipment.

8. Far Experimental Hall (FEH) – The FEH is the second experimental facility, and is located approximately 951 feet (290 meters) downstream from the NEH. The FEH extends approximately 98 feet (30 meters) in the direction of the beam and is approximately 59 feet (18 meters) wide. One of the principal design challenges of the FEH is that its finished floor will be approximately 107 feet (33 meters) below existing grade.

9. Free Electron Laser (FEL) Center – The FEL Center will be constructed to house the research offices and laboratory space for the LCLS users, scientific and support staff. The FEL Center footprint is envisioned to be approximately 180 feet (55 meters) long and 115 feet (35 meters) wide, but is not constrained by LCLS technical requirements. The FEL is planned to be architecturally attractive and moderately landscaped, with parking provided adjacent to it. The proposed location of the FEL is on grade and adjacent to the east edge of PEP Ring Road.

10. Central Lab Office Complex (CLOC) – An office complex shaped in the form of a segment of a circle and a northeast-southwest longitudinal axis will be built over the eastern third of the NEH. The CLOC is envisioned to have two floors above grade and one floor below grade. A plaza will be built on the east and west sides of the CLOC. Because of the existing topography, construction of the CLOC will lead to a portion of the building being supported on a cut pad, a portion on the NEH, and the remaining portion on engineered fill.

11. Service Stations – New service stations are to be built at various locations on the site. Service Stations A, B, and C are to be built at the west end, mid-point, and east end of the BTH, respectively. Service Station D will be built near the east end of the UH, at the location of borings LCLS-3 and LCLS-3A. Service Station F will be located over the XDT near the location of boring SB-D. Service Station G will be built to the immediate east of the existing SLC Experimental Hall.

12. Access Tunnel – An. access tunnel will be constructed from Service Station G, on the east side of the existing SLC Experimental Hall, to the FEH.

13. Substations – New substations are to be built at various locations on the site. Substations 1, 2, and 3 will be built to the north of CLOC and to the southwest of the existing SLC Experimental Hall. Substation 4 will be built to the immediate north of Service Station G.

14. Sighting Holes – A 24-inch diameter sighting hole, extending from the ground surface to the UH, will be installed to the east of the existing survey tower. Another sighting hole extending to the UH will be located just to the west of Service Station D. The last sighting hole, which will extend from the ground surface to the XDT, is to be located adjacent to Service Station F.

Based on the proposed invert elevation, the LCLS tunnel will traverse the alignment of the existing PEP tunnel about 34 feet above the invert elevation of 213.9 feet for that tunnel. It will also traverse the alignment of the existing SLC tunnel about 33 feet above that tunnel's invert elevation of 214.89 feet.

Proposed site work includes the following:

1. Crossover Road – A crossover road, which will involve the widening and upgrading of an existing roadway, will be built to the east of the existing survey tower. The Crossover Road will facilitate access to the north and south sides of the new BTH, which will extend above grade in the Research Yard.

2. Parking Areas and Access Roads – New parking areas will be built on the north side of Service Station D and on the west side of the CLOC. Access roads will be provided from

the PEP Ring Road to the new parking areas. The section of the existing PEP Ring Road within the projection of the CLOC and the new parking area on the western side of the CLOC will be reconstructed by raising the existing grade along the subject section of the roadway. The raising of the grade of the subject section of the PEP Ring Road will help minimize the gradients of new access roads from the roadway to the new parking area.

Organization of Report

General: The geotechnical data and investigation report has been organized into two volumes as follows:

Volume 1 of the Report – Constitutes the Geotechnical Data portion of the report.

Volume 2 of the Report – Constitutes the Analysis and Recommendations portion of the report.

Volume 1 of the Report: Volume 1 of the report is divided into six sections, covering background and general information regarding the project, geology and seismicity, the field exploration program, the laboratory testing program, and a description of the subsurface conditions encountered. That volume of the report is under a separate cover.

Volume 2 of the Report: Volume 2 of the report is divided into the following sections:

Section 1: Introduction – Contains a summary of background information culled from Volume 1 of the report.

Section 2: Surface Structures – Covers recommendations relating to above-grade structures without basement space.

Section 3: Site and Below-Grade Walls – Covers recommendations relating to site retaining walls and below-grade walls.

Section 4: Tunnel and Caverns – Covers recommendations relating to the tunnel and proposed caverns.

Section 5: Site Response Analysis – Provides a summary of the findings from the site response analysis performed by Dr. Robert Pyke, including criteria for seismic design of the tunnel and caverns.

Section 6: Pavement Design – Covers recommendations relating to the design of proposed pavements.

Section 7: Miscellaneous – Covers recommendations relating to temporary excavation support, earthwork, grading, and surface drainage, and construction observation.

Section 8: Constructibility Issues – Discusses potential constructibility issues.

Section 9: References – Contains a list of references used in the course of the investigation.

Volume 2 of the report contains the following appendices:

Appendix A: Contains figures relating to design recommendations.

Appendix B: Contains site response analysis report by Dr. Robert Pyke.

Limitations

1. This report has been prepared for the exclusive use of the Stanford Linear Accelerator Center and its consultants for specific application to the LCLS Project as described herein. In the event that there are any changes in ownership, nature, location or design of the project, the information contained in this report shall not be considered valid unless the project changes are reviewed by Rutherford & Chekene.

2. Any conclusions contained in this report are based in part upon the data obtained from exploratory borings and laboratory testing performed as part of this investigation. The nature and extent of variations between the borings may not become evident until construction. If variations are discovered, it will be necessary to re-evaluate any conclusions contained in this report.

3. We cannot be responsible for the impacts of any changes in geotechnical or geologic standards, practices, or regulations subsequent to the performance of our services if we are not consulted subsequent to the changes.

4. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for use of segregated portions of this report without consultation with our office.

SECTION 2
SURFACE STRUCTURES

STRUCTURAL AND FOUNDATION DESIGN

Seismic Design Criteria

The seismic zone map of the United States of America shows that the site is located in Zone 4. Based on maps of known active fault near-source zones by the ICBO (1998), the fault with the greatest potential impact on the site is the San Andreas fault, located about 2.5 miles (about four kilometers) to the southwest of the site. Since it is located at a distance of less than 10 kilometers from the site, seismic activity on the nearby segment of the San Andreas Fault will have near-source effects on the site. Based on its seismic activity, the San Andreas Fault is classified as a Type A fault.

Using the shear wave velocity measurements from borings LCLS-3 and LCLS-5, we have determined that the site soil profile can be classified as S_C.

From the above discussion, the seismic design parameters in Table 1 as defined in the 2001 California Building Code (CBC) are applicable to the project site.

Table 1
2001 CBC Seismic Design Parameters

Seismic Zone	4
Seismic Zone Factor, Z	0.40
Soil Profile Type	S_C
Seismic Coefficient, Ca	1.08
Seismic Coefficient, Cv	0.58
Near-Source Factor, Na	1.45
Near-Source Factor, Nv	1.93

Foundation Systems

General: We evaluated two different foundation systems for the following proposed surface structures:

1. The BTH

2. The CLOC

3. Service Stations A, B, C, D, F and G

4. Substations 1, 2, 3, and 4

The two foundation systems that we considered are: 1) a drilled pier foundation system, and 2) a shallow foundation system.

The results of our evaluation are as follows:

1. Drilled Pier Foundation

   A drilled pier foundation system is ideal for structures subjected to both substantial compressive and/or tensile loading. Drilled piers will also be ideal where the need to minimize differential settlement is important. Compared to shallow foundations, drilled piers are more expensive.

2. Shallow Foundations

   A shallow foundation system, such as spread footings or mat foundation, is ideal for locations where the bearing layer is shallow. This system would be more economical than a drilled pier foundation system.

<u>Foundation System for BTH</u>: Thick concrete walls are required for the BTH for radiation-shielding purposes hence the structural loads on the foundations are expected to be high. If a shallow foundation system were used, a mat or footing extending beyond the footprint of the structure would be required. A drilled pier foundation system is therefore recommended for this structure. Criteria for the design of drilled piers are given below.

<u>Foundation System for CLOC, Service Stations and Sub-stations</u>: We recommend that the listed structures be supported on a shallow foundation system for economical reasons. There are however various special factors associated with the use of spread footings that must be accounted for in the design. These factors are discussed below.

In the case of the CLOC, the design must account for the presence of fill below the building on the south side of NEH. The design should also account for the potential impact of new CLOC footings on the walls and foundations of the NEH. If the bearing elevations of new CLOC footings and Sub-station 1, 2, and 3 footings don't match, the potential impact of one set of footings on the other should be considered in the design.

It is not clear if the footprints of Service Stations A, B, C, D, and F match the footprints of the underlying buried structures or not, as depicted in Figure 1a and Figure 1b. If they do match as shown in Figure 1b, there is no special design requirement other than considering the loads from the service stations in the design of the foundations of the underlying structures. Where a service station extends beyond the underlying structure as shown in Figure 1a, the design should account for the impact of the footings for the service station on adjacent below-grade walls and footings. In the case of Service Station G, the impact of new service station footings on any below-grade walls and footings for the existing SLC Experimental Hall should be considered in the design.

If the footings for Substation 1, 2, and 3 and those of the north portion of CLOC bear at the same elevation, there are no special factors to consider. Similarly, if the footings for Substation 4 and the footings for Station G bear at the same elevation, there are no special factors to consider in designing the footings for the substation.

Measures for mitigating specific impacts of one set of footings on another are discussed below.

Interface Between Adjacent Structures

<u>Surcharge Effects of Service Station A, B, and C Footings on Foundation of BTH:</u> If the footprint of Station A, B, and C extend beyond that of the BTH, the surcharge from footings of the extended portions of the stations should be included in the design of the BTH. The surcharge can be estimated using Boussinesq's charts (NAVFAC, 1982, page 7.2-74).

In order to minimize the impact of the higher bearing footings of the stations on the below-grade walls and footings of the UH or XDT, the criterion shown in Figure 2 between adjacent footings bearing at different elevations should be satisfied.

<u>Surcharge Effects of Service Station D and G Footings on Below-Grade UH and XDT Walls and Footings, Respectively:</u> If the footprint of Station D extends beyond that of the below-grade UH, the footings of the extended portion of the station would surcharge the walls and footings of the UH. The same situation will apply if the footprint of Station F extends beyond the footprint of the XDT.

In order to minimize the impact of the higher bearing footings of the stations on the below-grade walls and footings of the UH or XDT, the criterion shown in Figure 1 between adjacent footings bearing at different elevations should be satisfied.

<u>Surcharge Effects of CLOC Footings on Below-Grade NEH Walls and Footings:</u> The footings of the portion of the CLOC extending beyond the footprint of the NEH would surcharge the walls and footings of the NEH.

In order to minimize the impact of the higher bearing footings of the CLOC on the walls and footings of the UH, the criterion shown in Figure 2 between adjacent footings bearing at different elevations should be satisfied.

<u>Surcharge Effects of CLOC Footings on Substation 1, 2, and 3 Walls and Footings or Vice Versa:</u> Depending on the relative bearing elevations of these neighboring structures, there could be surcharge effects of one set of footings on the other set of footings and walls. If the relative bearing elevations of the subject footings indicate a potential for surcharge effects, the criterion in Figure 2 should be satisfied to minimize the potential for such surcharge effects.

<u>Surcharge Effects of Station G Footings on Below-Grade Walls and Footings of SLC Experimental Hall:</u> We currently do not know the details of the elevation of the existing SLC Experimental Hall footings relative to the bearing elevation of the proposed Station G footings.

We recommend that the Structural Engineer verify that the criterion in Figure 2 is satisfied considering the relative bearing elevations of the two sets of footings. If this criterion cannot be satisfied, then the surcharge effects from the Station G footings on the adjacent SLC Experimental Wall footings should be estimated using Boussinesq's charts (NAVFAC, 1982, page 7.2-74) and the effects on the below-grade experimental hall walls should be estimated using the equations in Figure 3. The ability of the existing experimental hall to accommodate these surcharge effects should be verified.

Criteria for Footings at CLOC, Service Stations, and Substations

If new footings were to bear at the same elevation, footings for the portion of CLOC outside of the footprint of NEH will bear in Ladera Sandstone in some areas and in engineered fill in other areas as depicted in Figure 4. We recommend that all CLOC footings bear in Ladera Sandstone in order to minimize the potential for detrimental total and/or differential settlements. We note however that at Service station D and other locations, new footings can bear in artificial fill or residual soil.

Pending settlement calculations, allowable bearing pressures for footings presented in Table 2 should be used.

Table 2
Allowable Bearing Pressures for Footings Bearing in Indicated Material

Load Condition	**Allowable Bearing Pressure (ksf)**		
	Artificial Fill	**Residual Soil**	**Ladera Sandstone**
Dead + Live Loads	2.5	3.0	9.0
Total Loads (Including Wind or Seismic)	3.3	4.0	12.0

To obtain ultimate bearing pressures from the allowable values given in the table above, multiply the allowable "Dead + Live Loads" bearing pressure by two. Allowable dead load pressures will be provided after settlement has been evaluated.

Lateral loads on building foundations can be resisted by 1) passive pressure on the side of the footing perpendicular to the applied force, and 2) friction at the base of the footing.

The lateral resistance of a mat foundation or footing is presented below.

1. *Passive soil pressure against the side of the footing:* Passive pressure against a mat foundation or footing perpendicular to the applied force should be taken as a pressure of P_t psf at the top of the footing and increasing as an equivalent fluid pressure of E_{fp} pcf. To obtain the ultimate passive soil pressure, multiply the allowable value by 1.5.

2. *Friction at the base of the mat or footing:* If the mat foundation or footing is not underlain by a waterproofing element, the allowable horizontal frictional resistance, F_{base}, at the interface between the mat and the underlying native soils may be taken as:

$$F_{base} = f_s \text{ x Actual Dead Load Pressure at Base of Mat or Footing (psf)}$$

If a waterproofing element is present, the friction factor, f_{swp}, should be substituted for f_s. The value of f_{swp} will be dependent on the type of waterproofing element used and is likely to have a value less than f_s. After a waterproofing element is selected, the manufacturer should be consulted for guidance on an appropriate friction factor. f_{swp}.

The values of P_t, E_{fp} and f_s at different bearing elevations are presented in Table 13.

Table 3
Parameters for Calculating Lateral Load Resistance

Footing Bearing in Indicated Earth Material	**P_t (psf)**	**E_{fp} (pcf)**	**f_s**
Existing Artificial Fill	0	250	0.25
Residual Soil	0	300	0.3
Ladera Sandstone	0	750	0.4

Impact of Slope on Adjacent Footings

Based on the current architectural plans, footings for some of the walls of the CLOC and Service Station D will be located adjacent to the cut slope near the SLC Experimental Hall. The configuration of the footing relative to the slope will fall into one of the two categories depicted in Figure 5. The presence of the slope will impact both the bearing capacity and the lateral resistance of the footing.

Where the slope impacts are likely to occur, the factors given below should be used to estimate the allowable capacity and lateral resistance for footings adjacent to the slope.

Table 4
Reduction Factors for Slope Effects

Slope-Footing Identification	Reduction Factor to Correct Bearing Capacity Values	Reduction Factor to Passive Pressure Values
Case (a)	0.75	0.25
Case (b)	0.9	0.3

The following procedure should be followed to obtain corrected bearing capacity and/or passive pressure vales:

1. Establish which of the two cases apply.
2. Determine the bearing capacity reduction factor that is applicable to the case.
3. Obtain the appropriate bearing capacity values from Table 2.
4. Multiply the bearing capacity by the reduction factor.
5. Determine passive pressure reduction factor.
6. Obtain the passive pressure parameter, E_{fp}.
7. Multiply the passive pressure parameter by the reduction factor.

Construction of Footings

To assure that the recommended passive and frictional resistances are developed from all footings, they should be cast directly against firm native materials.

The following measures are recommended to minimize the potential detrimental impacts of mat or footing excavations on foundation performance:

1. Footing excavations should not be left open for a long time period, especially during the rainy season, and water should not be introduced into the excavations. The intent of this recommendation is to avoid the softening of the native material, as well as the introduction of soft materials into the bottoms of excavations by erosion. If necessary, the excavations should be covered to minimize ponding or infiltration of rainwater.

2. Footing excavations should be thoroughly cleaned of all loose and soft materials immediately prior to concrete placement. The effort to clean the excavations is usually hampered by the presence of reinforcing bars in the excavations, making this a less

preferred approach than the option described below for creating acceptable bearing conditions.

3. If footing construction occurs during the rainy season, a two-inch thick lean concrete layer could be placed at the bottom of the foundation excavations after suitable bearing conditions have been established. This lean concrete layer would ensure that the bearing conditions are maintained, provide a firm bearing surface for the footing reinforcement cage, and ensure adequate concrete cover on the bottom reinforcing bars. Also, any loose materials that accumulate in the excavation can be easily removed using air blowing techniques. This approach can also be adopted by the Contractor if he chooses, even if foundation construction occurs outside of the rainy season.

The Geotechnical Engineer should be given the opportunity to observe the bearing conditions prior to the placement of reinforcement and immediately before concrete placement. Remedial work should be performed, if necessary, until the bearing conditions are deemed to be satisfactory by the Geotechnical Engineer. The responsibility to maintain suitable bearing conditions and control sloughing of the sides of the excavation should remain however with the Contractor.

Settlement

General: Foundation settlements can be grouped into three main components: elastic settlement, inelastic settlement, and settlement induced by construction activities. The sum of these components can be considered as the total settlement.

Elastic and Inelastic: We do not have estimates of structural loads, but we estimate that the sum of the elastic and inelastic settlements of footings bearing in Ladera sandstone is expected to be relatively small compared to the settlement in fill or residual soil.

Actual magnitudes of the total settlement depend on the degree of disturbance existing in the foundation soils at the time of the footing placement and on the final dead load pressure.

Effects of Construction Activities: In addition to the above causes, settlement may also result from construction activities and practices. This settlement is impossible to estimate because the actual construction conditions, activities and practices cannot be anticipated. To minimize the impact of these unknowns on settlement, we recommend that the requirements contained in the subsection titled "Construction of Footings" be satisfied.

Criteria for Drilled Piers at BTH

Axial Capacity: Drilled piers for the support of BTH should be designed according to the design recommendations presented below. The drilled piers should be designed to resist axial compressive loads through friction between the shaft walls and the surrounding soft rock. The end-bearing capacity of the drilled piers should be neglected, since it is assumed to be engaged at unacceptably high settlements.

We recommend using drilled piers with a minimum diameter of 24 inches. Drilled piers should have a minimum center-to-center spacing of three times the pier diameter to minimize group effects on the axial load. That spacing does not however prevent group effects under lateral loads. The piers should have a minimum center to center spacing of 6 times the pier diameter to minimize group effect on that lateral load.

The axial compressive capacity of drilled piers can be estimated by using an average friction resistance of 1,500 psf for dead plus live loads in the Ladera Sandstone. This friction resistance value can be increased by one-third for dead plus live plus seismic loads. For ultimate load case, use a friction value equal to twice the allowable friction for dead plus live loads. Friction resistance for dead load only will be provided after settlement evaluation has been completed.

We estimate that the Ladera Sandstone is overlain by 12 inches of pavement section in the Research Yard. Friction contribution from the pavement section should be neglected.

Lateral Capacity: Lateral loads on the BTH will be resisted by flexure in the drilled piers and by passive pressure acting against the side of the grade beam or mat connecting the drilled piers, assuming those connecting structural elements are embedded below grade.

We have performed lateral load analyses on the drilled piers using the computer program LPILE based on lateral load estimates that we developed. We analyzed the piers assuming the following diameters, lengths, and applied lateral loads at the top of the pier:

1. Diameters: 4 feet and 6 feet
2. Lengths: 20 feet and 30 feet
3. Applied Lateral Loads: 10, 20, 30, 40, and 50 kips

The results of the lateral load analysis are shown in Figures 6 – 17.

Installation: The drilled pier installation process should be observed by the Geotechnical Engineer to verify the subsurface conditions assumed in developing the pier design recommendations. Allowance should be made in the specifications through unit pricing to account for pier length adjustments that may need to be made in the field. Drilled pier cages should be designed to allow for similar length adjustments in the field. Additional installation issues are discussed in the section titled “Constructibility Issues”.

DESIGN OF SLABS-ON-GRADE

Summary

The design requirements for slab-on-grade sections in surface structures can be summarized as follows: 1) preventing dampness and efflorescence in the floor and 2) supporting anticipated floor loads. To fulfill these objectives, the following section is recommended for slab-on-grade floors:

1. Reinforced concrete slab of minimum five-inch thickness.

2. Impervious membrane of good quality, per ASTM E1745, Class C. The membrane should be Stego Wrap or approved equal.

3. Granular cushion, with a minimum nominal thickness of four-inches. The granular cushion should consist of broken stone or crushed or uncrushed gravel. It should also be angular and free of deleterious matter. The gradation should conform to the following:

U.S. Series Sieve Size	Percentage Passing Sieve (Dry Weight Composition)
¾-inch	100
No. 4	0-10
No. 200	0-2

The granular cushion should be compacted with a vibro-plate before subsequent construction.

SECTION 3
RETAINING STRUCTURES

RETAINING STRUCTURES

Lateral Pressures on Basement and Other Permanent Retaining Walls

General: Basement and other retaining walls that are part of the proposed facility should be designed to resist static lateral earth pressures plus additional lateral pressures that may be caused by earthquakes and/or surcharge loads behind them, as detailed below. The design criteria for these walls are for permanent loading conditions. Temporary excavation walls should be designed according to the guidelines provided in the subsection of this report titled "Miscellaneous Issues" under the subheading "Temporary Excavation Support".

It should be noted that the design lateral earth pressures described below do not include contributions from hydrostatic pressures. Thus, a subdrain system, as described in "Subdrainage and Moisture Protection for Basement and Other Permanent Retaining Walls", should be provided behind the subject walls.

The basement walls and other permanent retaining walls can be constructed in one of two ways:

1. Provide temporary excavations and subsequently construct conventional retaining walls.

2. Provide temporary shoring, using either soil nail walls or soldier beam and lagging, and subsequently construct conventional retaining walls.

Design Lateral Earth Pressures: Although a variety of retention conditions will require specific analyses and design recommendations, the following pressures can be used for design of basement and other permanent retaining walls. The design should consider whether the static case or static plus dynamic cases govern.

1. *Static Forces*: The following recommendations apply for walls retaining backfill material with a horizontal surface. For top-restrained walls or cantilever site walls with footings bearing on Ladera sandstone, a lateral earth pressure equal to a 65 pcf equivalent fluid pressure should be used. For cantilever site walls with footings bearing on artificial fill or residual soil, a lateral earth pressure equal to an equivalent fluid pressure of 45 pcf should be used.

2. *Dynamic Case*: For a wall of height H feet, the dynamic earth pressure increment induced by an earthquake should be assumed to be about 15H psf. The associated static earth pressure for top-restrained walls or cantilever site walls with footings bearing on Ladera sandstone should be equal to an equivalent fluid pressure of 55 pcf. For cantilever site walls with footings bearing on artificial fill or residual soil, the associated static earth pressure is still equal to an equivalent fluid pressure of 45 pcf. The total lateral earth pressure is equal to the sum of the dynamic earth pressure increment and the static earth pressure.

3. *Surcharge Loads*: A uniform lateral pressure equal to 0.5 times any surcharge loads (uniform vertical pressures) behind the walls should be used to account for surcharge directly behind top-restrained walls or cantilever site walls with footings bearing on Ladera sandstone. The corresponding coefficient for cantilever site walls with footings bearing on artificial fill or residual soil is 0.33.

Subdrainage and Moisture Protection for Basement & Other Permanent Retaining Walls

General: The design lateral earth pressures presented above do not include contributions from hydrostatic pressures behind the walls. Thus, a subdrain system should be provided behind the walls to prevent an accumulation of water from perched groundwater derived from water percolating through uncemented permeable zones in the Ladera sandstone, leaking pipes, or sewer pipes, if such utility lines exist in the vicinity of the walls.

All Retaining Walls: To prevent hydrostatic pressures against any retaining structure, a continuous drain should be installed. The drain should consist of prefabricated drainage panels (Miradrain, or equal) with filter fabric on the side facing the earth, draining into a perforated collector pipe placed on top of the footing. As an alternative to prefabricated drainage panels, permeable backfill material at least one foot thick may be used. For this alternative, the permeable backfill material should conform to the gradation requirements for Class 2 Permeable Material as specified by the California Department of Transportation (Caltrans) Standard Specifications, Section 68.

The drainage panels or permeable blanket should extend from the bottom of the wall up to a depth of two feet below the finish grade level. The uppermost two feet of backfill should be composed of soils of lower permeability, placed as a "cap" to prevent direct flow of surface runoff water into the subdrain system.

Water accumulating from this subdrainage system should be connected to the storm drain system. For site walls, weep holes can be used to discharge water accumulating from the subdrainage system.

Moisture Protection for Basement and Below-Grade Walls: If moisture vapor penetration into the basement or below-grade walls is an issue, an impervious membrane of good quality should be placed directly against the outside of the basement wall, between the wall and the drainage panels/permeable blanket. The purpose of this membrane is to provide a moisture-resistant barrier between the soil and the basement walls.

SECTION 4
TUNNELS AND CAVERNS

TUNNEL DESIGN AND CONSTRUCTION

Exposed and Cut-and-Cover Tunnel Sections

Exposed Tunnel: The exposed tunnel section is constituted by the BTH. Design of this section of the larger tunnel was considered under Section 2: "Surface Structures".

Design of Cut-and-Cover Tunnel: The cut-and-cover tunnel section includes the BDE, FEE, and the NEH. The design of this tunnel section is can be accomplished by using the criteria for footings presented under Section 2: "Surface Structures". In this case, the foundation will be considered a mat foundation. The walls of the tunnel can be designed using the criteria presented under Section 3: "Retaining Structures".

The roof load on the cut and cover tunnel section can be estimated as follows:

1. Determine the thickness of backfill cover on the tunnel roof.
2. Use an assumed unit weight of 130 pcf for the backfill to estimate the overburden on the roof.
3. Account for the downdrag effect of the backfill on the walls of the tunnel, which will be supported on a rigid foundation by multiplying the overburden by a factor of 1.25.
4. Include the effect of traffic surcharge from PEP Ring Road and the new parking areas, based on an assumed design truck loads.
5. Discuss with Landscape Architect the potential for surcharge from trees to be planted along the alignment of this section of tunnel. The discussion should also include the potential impact of deep-rooted trees on tunnel subdrainage system.

Tunneling Sections and Caverns

Static Design: Geotechnical parameters obtained from the current investigation for use in the design of the subject tunneling sections and caverns include pressuremeter test results summarized in Volume 1 of the report. The following are the recommended values for the identified geotechnical parameters:

1. Unit Weight = 130 pcf
2. Friction Angle = 34 degrees
3. Cohesion = 0 psf
4. Poisson's Ratio = 0.075 to 0.12
5. Minimum RQD = 0

6. Average RQD = 5

7. Maximum RQD = 52

The average modulus from the pressuremeter test results are as follows:

1. Undulator Hall Area: 1900 bars between elevations 260 and 245 feet

2. FEH Area:

   2,500 bars between elevations 310 and 285 feet

   6,000 bars between elevations 285 and 250 feet

   3,000 bars between elevations 250 and 245 feet.

Because of the variability in the nature of the Ladera Sandstone, we recommend that parametric studies be performed using the values above as well as values greater or smaller than the ones above to determine the sensitivity of these parameters to the design. The design should err on the conservative side.

Seismic Design: Geotechnical parameters for seismic design of the tunnel are presented in Section 5: "Site Response Analyses".

Junctions: There are a number of junctions in the UH section of the facility. An even more complex set of junctions occurs at the FEH where it connects to the XDT and to the link to Service Station G. Although it could be rationalized that load reduction occur at these junctions due to arching, we recommend that the full overburden be assumed to act on each junction.

Preparation of Tunnel Invert Subgrade: One of the essential factors in minimizing differential movement of the completed tunnel invert is adequate preparation of the subgrade prior to the placement of reinforcement and concrete for the invert. We recommend that muck, loose materials and soft spots should be removed under the direction of the Geotechnical Engineer to ensure that the subgrade is adequately prepared.

Subdrainage

Although historical groundwater data indicate that the proposed LCLS facility will be above the groundwater table, the potential for water to percolate through porous zones in the Ladera Sandstone exists. The buried structure will serve as a lateral hydraulic barrier to the percolating or perched water. Moisture from this and other sources must be kept out of the buried structures. A subdrainage system should therefore be provided around buried tunnels and caverns to intercept this percolating or perched water. A waterproofing element should also be provided around the tunnel lining.

We also recommend that a separate drainage system be provided to collect water originating from inside the tunnels and caverns.

Interface with Existing Buried Structures

The new LCLS facility will be above the existing PEP tunnel, which has a finished invert elevation of 213.9 feet and a ceiling elevation of 224.4 feet. The invert elevation for the SLC is 214.89 feet. This suggests that where the XDT tunnel and the PEP tunnel cross, the crown of the PEP tunnel will be about 23 feet below the finished invert of the LCLS tunnel.

Based on the relative proximity at the point of intersection, we are of the opinion that there is a slight potential for some heave of the PEP tunnel during the construction phase of the LCLS tunnel. Deformation in the intersection area of the two tunnels should be monitored during the construction phase.

We also recommend that as-built drawings of the PEP tunnel be reviewed for potential presence of rock bolts that might extend into the cross-section of the XDT tunnel and interfere with the construction of the latter tunnel.

Construction of Tunnels and Caverns

The proposed facility would have tunnel cross sections that vary in invert-to-crown heights of about 12 to 22 feet and widths varying from about 12 to 31 feet. The relatively soft nature of the Ladera Sandstone would allow for the use of a roadheader for tunnel excavation and reinforced shotcrete for initial support.

Based on generalized subsurface information derived from discrete boring locations that are far apart along the tunnel alignment, conditions are relatively uniform. There were however evidences of some variation that gives cause to expect some unknowns. The three major factors or hazards that are likely to affect the construction of the tunnels are:

1. Unexpected changes in the behavior of the ground due to changes in the geotechnical properties of the materials encountered.
2. Instability of the ground to be excavated due to non cohesive layers or zones, or due to fissures.
3. Unexpected hard knobs or knockers within the larger rock mass.

The first two factors can lead to face stability, convergence, and settlement problems whereas the last factor is more likely to reduce advance rate.

Based on the preceding discussion, we recommend that tunnel sections with larger cross sections than has been attempted on the SLAC campus in the past should not be advanced full-face. Instead, they should be subdivided to achieve one or more of the following objectives:

1. A reduction of the exposed face to minimize the potential for the three problems listed in the preceding paragraph.
2. A reduction of the excavation, reinforcement, and shotcrete quantities per each increment of advance. This results in the provision of support earlier or minimizes stand-up time.
3. Early invert closure in each of the subdivisions. We note that each subdivision face can be stepped at heading, bench and invert if desired.

The segmental approach and the sequencing that is established at the design stage should be validated by instrumentation and monitoring during the construction phase.

SECTION 5
SITE RESPONSE ANALYSIS

SUMMARY

Methodology

We commissioned Dr. Pyke to perform a nonlinear site response analysis across four subsurface profile sections shown in Figure 18. Dr. Pyke developed three three-component acceleration histories that served as input to the site response analyses. The time histories were fitted to the SLAC spectra contained in ES&H (2000). Details regarding the methodology are contained in Dr. Pyke's report in Appendix B.

Results

The following conclusions were developed from the analysis:

1. Shear wave velocity = 1,850 ft/sec
2. P-Wave velocity = 4,000 ft/sec
3. Rayleigh wave velocity similar to shear wave, but not applicable at the project site.
4. Peak displacement = 0.04 feet horizontal, 0.006 feet vertical
5. Modulus of Elasticity = 64,000,000 psf
6. Poisson's Ratio = 0 to 0.1
7. Racking Deformation = 0.004 feet per 10 feet (upper bound, refer to Dr. Pyke's report)
8. Peak acceleration = 0.6g horizontal and vertical
9. Peak velocity = 2.6 feet/sec. horizontal, 1.3 feet/sec. vertical.

Additional details regarding these conclusions are contained in Dr. Pyke's report in Appendix B.

SECTION 6
PAVEMENT DESIGN

SUMMARY

Asphalt Concrete and Other Paving

General: Asphalt concrete paved areas will include the 1) new Crossover Road, 2) regraded section of PEP Ring Road, 3) new Parking Area on the north side of Service Station D, and 4) new parking Area on the west side of CLOC.

Crossover Road and PEP Ring Road: For the new and regraded roadways, we recommend using a section consisting of a 3-inch thick asphalt concrete layer placed on a 9-inch thick base composed of Class 2 aggregate base. This paving section is based on the California State Flexible Paving Design Method, using a Traffic Index (TI) of 7 and R-value of about 50. Selection of the TI value was based on assumed use and not on a detailed equivalent wheel load analysis or traffic study. A TI of 7.0 was assumed for the pavement sections to account for potentially large truck loads.

Parking Areas Near Service Station D and CLOC: For the new parking areas, we recommend using a section consisting of either a 2.5-inch thick asphalt concrete layer placed on a 9-inch thick base composed of Class 2 aggregate base. This paving section is based on the California State Flexible Paving Design Method, using a Traffic Index (TI) of 5.0 and R-value of 18. Selection of the TI value was based on assumed use and not on a detailed equivalent wheel load analysis or traffic study.

Concrete Sidewalks: For concrete sidewalks subjected to pedestrian traffic only, a minimum four-inch thick nominally reinforced concrete slab on prepared subgrade should be adequate.

Pavement Subgrade Preparation and Drainage

Subgrade-All Paving Types: The subgrade for all paving types should consist of existing non-organic site soils (after stripping) scarified to a depth of six inches, moisture-conditioned, and recompacted to a minimum 95 percent relative compaction (based on ASTM Test Method D1557).

Pavement Drainage: Our observations of pavement performance indicate that there is a strong correlation between poor pavement drainage conditions and the amount of pavement failures (potholes, settlement bowls, alligator cracks, etc.) observed. For this reason, we recommend that new pavement sections should be adequately drained by providing swales, culverts, or subdrains, as deemed necessary.

Aggregate Base Materials

Where aggregate base material is specified, the furnished material should meet the requirements of Class 2 Aggregate Base as described in the California Department of Transportation (Caltrans) Standard Specifications. Aggregate base materials should consist of virgin rock aggregates only, unless the Contractor can provide certification that any proposed recycled materials are free of hazardous and/or deleterious contaminants. The Contractor should provide written certification from the quarry stating that aggregate base materials meet all the requirements of Caltrans Class 2 Aggregate Base.

SECTION 7
MISCELLANEOUS ISSUES

TEMPORARY EXCAVATION SUPPORT

Summary

General: In theory, the walls of an excavation can either be sloped using the recommended gradient or temporarily shored. However, the constraints imposed by the adjacent buildings and other conditions may dictate that all or parts of the excavation be shored.

The recommendations provided in this subsection apply to temporary shoring walls, which are designed to retain soil under short-term (non-permanent) conditions. Criteria for long-term (permanent) retaining walls are presented later in this report in a previous subsection titled "Basement and Other Permanent Retaining Walls". Recommendations for temporary and permanent cut slopes are presented under the "Excavation and Slopes" subheading of this subsection titled "Miscellaneous Issues".

Design Guidelines: We considered using either a soldier beam and lagging system or a soil nail wall system and concluded on the basis of cost that the latter is more economical. If a soil nail wall is constructed, it should satisfy the following:

We recommend that the soil nail wall be designed in accordance with the provisions of the Federal Highway Administration (FHWA), *Manual for Design and Construction Monitoring of Soil Nail Walls, Publication No. FHWA-SA-96-069.*

The soil nail wall should be designed using the following average strength parameters:

Friction Angle = 34 degrees

Cohesion = 0 psf

These strength parameters are based on averaging our lab data from our 2003 geotechnical study and the current investigation. To account for uncertainties associated with the limited data, we have neglected the cohesion component from some of our test data.

An average unit weight of 130 pcf should be assumed for the earth materials.

Assume a design groundwater elevation of 242 feet, based on historical groundwater data compiled by SLAC near boring SB-A. The design groundwater elevation was obtained by allowing for a two-foot rise in the measured water level.

Design Seismic Event: We recommend that a design seismic event with a peak ground acceleration of 0.6g should be assumed for the dynamic analysis of the soil nail wall.

Drainage: We recommend that a drainage system be provided between the facing and the exposed earth materials behind the facing. This system should as a minimum consist of strips of drainage panel (MIRADRAIN 6000 or approved equal) with a collector pipe at the base of the wall. The collector pipe can then be connected to the storm drain system.

EARTHWORK, GRADING AND SURFACE DRAINAGE

Site Preparation and Demolition

Existing structures within the Research Yard should be completely demolished to make room for new construction. Demolition should include the removal of footings and existing utility lines that traverse the footprint of new construction. The resulting excavations should be backfilled with concrete density fill.

If an existing below-grade structural element such as a utility structure is encountered within the footprint of proposed construction, it should be removed at least two feet below the subgrade for new concrete slabs and other flatwork, and the pit should be properly backfilled with site-derived or imported materials in accordance with "Fill and Backfill Materials" and "Engineered Fill and Backfill Placement".

In the areas of new improvements, the unpaved portions of the site should be stripped at least six inches below the existing grade. Concrete, wood, and other debris should be hauled off the site. In the existing paved areas, the asphalt and subgrade should be stripped to the depth of undisturbed native soils.

Stripping should extend at least five feet beyond the footprints of the proposed service stations and substations. Unless the stripped materials are considered suitable for landscaping purposes, they should be hauled off the site.

Excavation and Slopes

Conventional excavation and earthwork equipment should be satisfactory for mass grading, foundation and basement excavations, and utility trenching on this site.

During the excavation operations, temporary cut slopes should be used, where feasible, to prevent movement of materials exposed on the excavation walls. A temporary slope gradient of 1.5:1 (horizontal:vertical) or flatter should be used except in well-cemented sandstone where the gradient can be increased to 1:1.

Permanent cut and fill slopes, if any, should not exceed a gradient of 2:1 (horizontal:vertical) in order to ensure stability, encourage plant growth, and minimize erosion. To provide erosion protection, permanent slopes should be initially stabilized with straw plugs and then planted with native plants, grasses, and shrubs consistent with the approved landscaping plan. For slopes 20 feet or higher, we recommend that a minimum six-foot wide bench should be provided for every 20 feet of height. A swale should be provided on the bench to minimize the potential for erosion of the slope surface by surface runoff.

The Contractor should be aware that slope height, slope inclination, and excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, or federal safety regulations; e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations.

Subgrade Preparation

Unless otherwise stated in this report, any exposed subgrade that will receive fill should be prepared by scarifying to a depth of six inches and moisture-conditioning to a moisture content of about two percent above optimum moisture content. The moisture-conditioned material should then be compacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). The moisture conditions are to be maintained until fill is placed.

Directly under slabs-on-grade or granular cushion, the exposed subgrade should be scarified to a depth of six inches, moisture-conditioned as described above, and compacted to at least 95 percent relative compaction. Where the slab will bear on engineered fill, the top six inches of the fill should be compacted to a minimum 95 percent relative compaction.

Where materials exposed in footing excavations or at tunnel inverts are disturbed (as determined by the Geotechnical Engineer) by the excavation operations, a reasonably smooth surface should be prepared for foundation placement by removal of loose materials as directed by the Geotechnical Engineer.

Engineered Fill and Backfill Placement

General: In areas designated to receive fill, the subgrade-to-receive-fill should be prepared as described in the preceding subsection. Approved fill material should then be placed in lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to a moisture content of about two percent above the optimum moisture content of the material, and compacted to at least 90 percent relative compaction (ASTM D1557).

In areas to be overlain by a slab-on-grade, each lift should be compacted, at a suitable moisture content, to a minimum relative compaction of 95 percent in the uppermost six inches of all fill and backfill, and a minimum 90 percent at other depths.

In addition to being compacted to the required relative compaction, the engineered fill should also be stable, i. e., not exhibit "pumping" behavior. Ponding or jetting should not be used to densify fill or backfill.

Fill and Backfill Materials

Imported Fill: If imported material is required for fill and backfill, the imported material must be granular soil, free of organic matter, which does not exhibit excessive shrinkage or swelling behavior when subjected to changes in water content. Imported fill should contain no environmental contaminants or construction debris. The material should conform to the following:

1. Satisfy the following grading requirements:

U.S. Sieve Size	Percentage Passing (Dry Weight Composition)
2½-inch	100
No. 8	25-45
No. 200	0-10

2. Be thoroughly compactable without excessive voids.

3. Meet the following plasticity requirements:

   a. Maximum Plasticity Index of 12 (ASTM D4318)

   b. Maximum Liquid Limit of 35 (ASTM D4318)

Selective Stockpiling of Site-Derived Fill Materials: During the excavation operations, the Geotechnical Engineer should be given the opportunity to identify native soils to be selectively stockpiled for use as fill or backfill.

Drain Rock and Filter Fabric

Drain rock, if required, should consist of Class 2 Permeable Material, meeting gradation and other requirements contained in the California Standard Specifications. Alternatively, three-quarter-inch crushed rock encapsulated in filter fabric (Mirafi 140N or equivalent) can be used instead of Class 2 Permeable Material. The Contractor should provide written certification to the Geotechnical Engineer stating that drain rock materials meet all the requirements of Caltrans Class 2 Permeable Material.

Surface Drainage

Finished grading for surface drainage should be designed to direct surface runoff away from the CLOC, the new service stations and substations toward discharge facilities. Ponding of surface water should not be allowed adjacent to the new structures. Downspouts and gutters should be provided and water from downspouts should be directed through unperforated pipes to storm drains. Alternatively, drainage culverts may be used to direct water from downspouts to storm drains.

Winter Construction

If earthwork operations are performed during the winter or the rainy season, long delays may result from the Contractor's inability to properly moisture-condition the mostly clayey site soils to achieve the required relative compaction. In that case, lime treatment could be employed to make the site soils workable and compactable.

Once the subgrade soils have been properly compacted, a six-inch layer of Caltrans Class 2 Aggregate Base can be placed over the subgrade as a cap to maintain suitable working conditions, if necessary. Also, a gravelly surface course may be required on construction traffic routes to improve working conditions. Provisions should be made to dewater the excavations and to minimize the flow of surface runoff into the excavations if earthwork is performed during the rainy season.

We must note that the moisture content shown in this report for native materials reflects the moisture conditions at the time of the field exploration. The moisture content of those materials should be expected to be much higher if earthwork is performed during the winter or rainy season.

CONSTRUCTION OBSERVATION

Summary

The following items should be reviewed or observed by the geotechnical engineer during the construction phase.

Our construction observation services will include (but will not necessarily be limited to) the following:

A. Review any proposed earthwork materials, both on-site and imported, to determine their acceptability.

B. Observe drilled pier installation continuously.

C. Review subgrade conditions at the tunnel invert prior to the placement of reinforcing and again immediately before the placement of concrete.

D. Observe bearing conditions in footing excavations, prior to placement of reinforcing and again immediately prior to the placement of concrete.

SECTION 8
CONSTRUCTIBILITY ISSUES

IMPACT OF SITE CONDITIONS ON CONSTRUCTION

General

Although this investigation was performed primarily for design purposes, a brief discussion of the impact of the site conditions on construction is presented for information purposes only. The discussion must not be considered as a presentation of every possible impact of site conditions on construction. Construction issues relating to tunnels and caverns are presented.

Mass Excavation

General: Rock as defined in this report encompasses materials ranging from partially to fully weathered and fractured material with shear wave velocities ranging from about 1,500 fps in the upper 22 feet to values of about 2,400 fps. We can also define the rock characteristics by drilling rates. From our recently completed field investigation, we found the coring rates to be on the order of 0.4 to two minutes per foot of core, using a CME-55 drill rig equipped with an HQ-size core barrel. In compensation for drilling or excavation work, no differentiation will be made between rock of various hardness.

Excavation: The rate of drilling through the rock encountered is one of many indicators of the ease with which the rock that will be removed. Representative drilling rates are shown on the boring logs. The drilling rates suggest that the bedrock formation could be excavated with slight to moderate effort using conventional construction equipment. For tunnel and cavern excavations, a roadheader can be used. Based on the boring logs and the laboratory tests results, hard zones (at least 5 feet in dimension) or knockers of bedrock, requiring the use of jack-hammers, hoe-rams, etc, should however be expected.

Drilled Pier Installation

Knockers might be encountered during the installation of drilled piers based on observations made during the drilling and pressuremeter testing. Anecdotal information supporting this observation was provided to us by a contractor, who encountered very hard pockets of rock in an excavation that he performed in the vicinity of Boring SB-C.

Based on observations from rock core samples, zones of poorly cemented sands should be anticipated during construction. These zones, where under water, would be susceptible to significant sloughing.

Other Issues

Dust, noise and vibration control may be necessary to minimize the impact of construction activities on nearby buildings.

SECTION 9
REFERENCES

REFERENCES

The following reports and publications were used for information in the course of this investigation:

1. Department of the Navy Naval Facilities Engineering Command (NAVFAC), (1982), *Foundations and Earth Structures, Design Manual 7.2,* May.

2. ES&H Division, (2000), *Specification for Seismic Design of Buildings, Structures, Equipment, and Systems at the Stanford Linear Acceleration Center*, December 4.

3. Institution of Civil Engineers (1996), *Design and Practice Guides, Sprayed Concrete Linings (NATM) for Tunnels in Soft Ground,* Thomas Telford, London.

4. International Conference of Building Officials (ICBO), (1997), *Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada.*

5. International Conference of Building Officials (ICBO), (1997), *Uniform Building Code.*