# LINAC Coherent Light Source Project -

## Tunnel Subcontract

## Geotechnical Baseline Report

March 16, 2006

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**Stanford Linear Accelerator Center**

Purchase Order No. 55608

# GEOTECHNICAL BASELINE REPORT

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# 1.0 Introduction

The Stanford Linear Accelerator Center (SLAC) is a national research facility operated by Stanford University for the United States Department of Energy (USDOE). SLAC is responsible for construction of the LINAC Coherent Light Source (LCLS) Project, which will involve the projection of a new type of x-ray through a series of tunnels and underground chambers. The project is located at the Stanford Linear Accelerator Center in Menlo Park, California, as shown on Figure 1. The new facility is a series of tunnels and underground caverns about 2200 feet in length extending east of the existing research facility. A plan of the underground facilities is shown on Figure 2. Hatch Mott MacDonald prepared this Geotechnical Baseline Report (GBR) for the tunneling portion of the LCLS project.

The design and preparation of Drawings and Specifications were carried out by Jacobs Engineering. Rutherford & Chekene (R&C) carried out a subsurface exploration program, prepared geotechnical data reports, and provided geotechnical recommendations for design purposes. Turner Construction is the Construction Manager and General Contractor (CM/GC) for the LCLS Project. This GBR is applicable to underground construction to be undertaken by a tunneling subcontractor to Turner.

This Geotechnical Baseline Report addresses the portals, tunnels, and mined underground cavern portions of the LCLS Project. The geotechnical issues associated with bored shafts completed in the Tunneling Subcontract, and the open cut portions of construction completed by others are not included in this GBR.

This GBR summarizes the geotechnical basis for design of the underground elements of the LCLS Project, and presents a baseline of subsurface conditions anticipated to be encountered during portal, tunnel and cavern construction. This report includes: a summary of the project and descriptions of the portals, tunnels and mined caverns to be constructed; interpretations of the geological and geotechnical information obtained for the project; a summary of anticipated ground and groundwater conditions to be encountered; a summary of how these anticipated conditions have been reflected in the project design; and discussions of other design and construction considerations that will impact the work.

The baselines contained in this GBR have been developed using judgment to interpolate between borings and extrapolate beyond the boring logs and laboratory test data, and are not necessarily geotechnical fact. The judgments applied reflect the view of the Owner and the Owner's consultants with regard to the anticipated conditions, and assume the use of appropriate means, methods, and levels of workmanship. Ultimately, the behavior of the geologic materials present in the surface and subsurface excavations will be influenced by the Subcontractor's selected equipment, means, and methods.

This GBR is a Tunneling Subcontract Document, and is intended to assist bidders in evaluating the requirements for excavating and supporting the ground, and in preparing their bids. The Tunneling Subcontract Documents for this project have been developed considering the risks inherent in underground construction. The baseline conditions presented in this report, along with other project information, will be utilized by the CM/GC to evaluate any differing site condition claims that may be submitted during the course of construction.

The GBR should be read in conjunction with the following Geotechnical Reports:

- Geotechnical Data Report, Proposed LINAC Coherent Light Source Tunnel Project, Stanford Linear Accelerator Center, Menlo Park CA, August, 2003, prepared by Rutherford & Chekene; and

- Geotechnical Data and Investigation Report, Volume 1, LINAC Coherent Light Source Project, Stanford Linear Accelerator Center, Menlo Park CA, prepared for the Stanford Linear Accelerator Center 21 January, 2005 by Rutherford & Chekene.

These Reports contain factual geotechnical data resulting from the explorations and testing completed for this project. The August 2003 Report and Volume 1 of the January 2005 Report are a part of the Tunneling Subcontract.

Volume 2 of the Geotechnical Data and Investigation Report (R&C, 2005b) contains a summary of geotechnical recommendations for use in design of the project facilities. Volume 2 is superceded by this GBR, and is a "Reference Document" for information only. It is not a part of the Tunneling Subcontract.

In the event of apparent conflicts, discrepancies, or inconsistencies with the reference documents or any other geotechnical data available in any way to the Tunneling Subcontractor, the GBR takes precedence in reconciliation of the conflict.

Certain elements of the work are based on requirements that cannot be varied. These include but are not limited to:

- Tunnel geometries – horizontal and vertical alignment and minimum excavated dimensions are fixed;
- Excavation by blasting is not allowed;
- Portals are required to be excavated and supported before tunneling begins or ends at a portal;
- Tunnel excavation sequences and support designs shown on the Drawings will not be altered unless ground conditions permit or require, and prior approval from the CM/GC is granted;
- Open cut excavation in between the Undulator and X-Ray tunnels (See Section 2) will be completed by others before tunnel excavation commences at the Undulator Hall East Portal and X-Ray Tunnel West Portal; and
- All construction water must be treated in accordance with permits and local regulatory requirements before being discharged.

A number of elements of the project are flexible and afford the Tunneling Subcontractor latitude in the selection of layout geometries, construction methods, equipment, procedures and sequences, subject to approval of the CM/GC. These include, but are not limited to the following:

- Excavations outside the minimum excavation lines for the Subcontractor's convenience;
- Ground support in addition to that called for in the design;
- The size, type and variety of mechanical excavation methods including roadheader, hoe-ram, excavator, and tunnel boring machine; and
- Sequence of developing full span of excavation within project constraints.

Some of the technical concepts, terms, and descriptions in this GBR may not be fully understood by bidders. It is highly recommended that bidders have a geotechnical engineer or engineering geologist who is familiar with all topics of this report carefully review and explain this information so that a complete understanding of the information presented in this report can be developed prior to submitting a bid.

Certain drawings and figures contained in other documents in the Tunneling Subcontract are referenced by the GBR as an aid to bidders in understanding the elements of the work. Such drawings are not reproduced in the GBR.


**Figure 1 - Site Vicinity**

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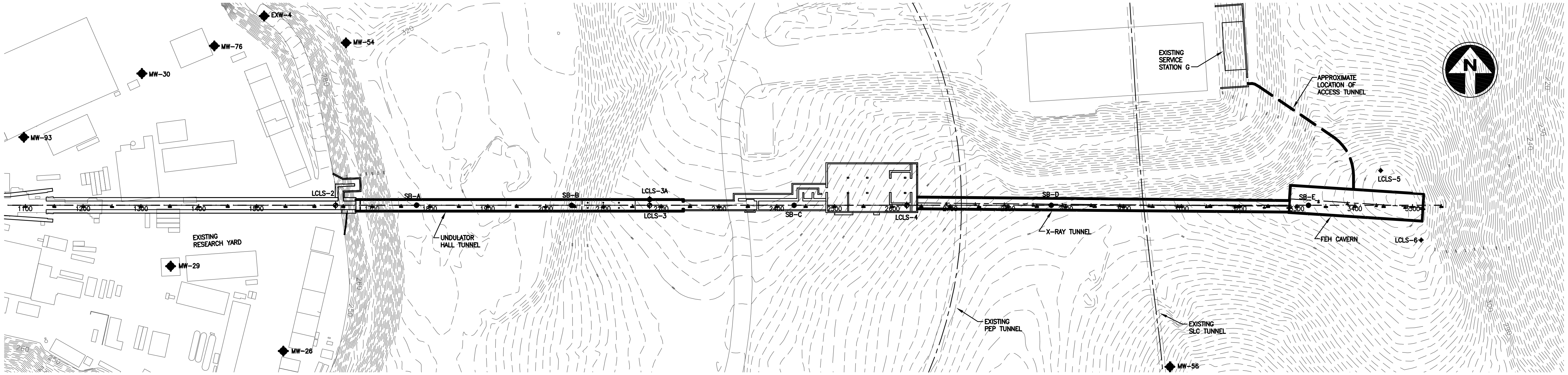
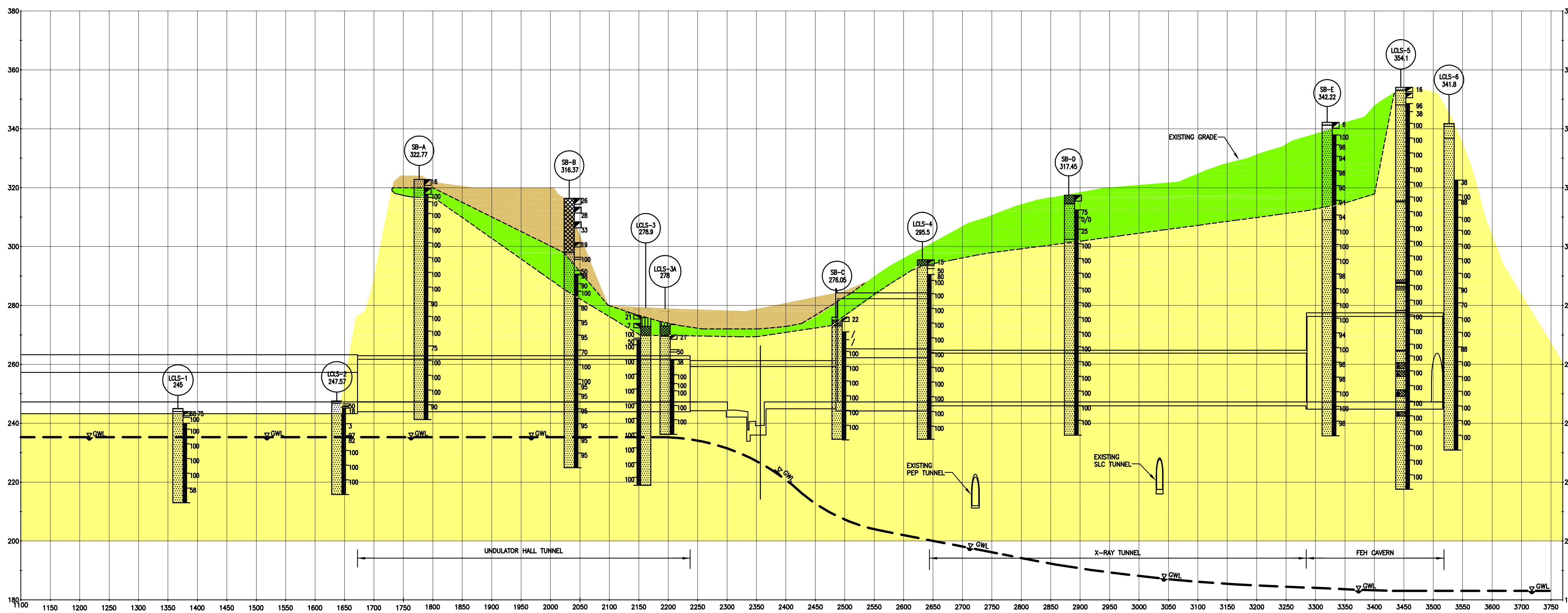



## GENERAL PLAN (REFER TO CONTRACT DRAWINGS FOR DIMENSIONS)

APPROXIMATE SCALE H: 1" = 20'




LEGEND:

SB – BORING LOCATIONS FROM 2003 BORING PROGRAM

LCLS – BORING LOCATIONS FROM 2004 BORING PROGRAM

MONITORING WELL (LOCATION APPROXIMATE)

LOGGED BY RUTHERFORD AND CHEKENE AS SOIL

LOGGED BY RUTHERFORD AND CHEKENE AS SANDSTONE

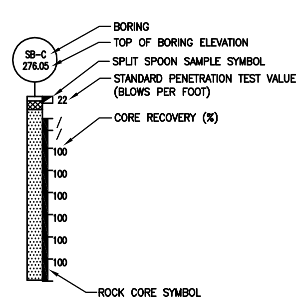
NATIVE FILL: GRAVEL, SAND, SILT AND ROCK FRAGMENTS (POORLY TO WELL CONSOLIDATED)

SANTA CLARA FORMATION: WELL GRADED DEPOSITS OF GRAVEL, SAND, SILT, GREYWACKE DEBRIS (NOTE–RESIDUAL SOILS OF THIS FORMATION ARE HIGHLY WEATHERED MIXTURES OF SAND, SILT AND CLAY)

LADERA SANDSTONE: A VARIABLY CEMENTED AND VARIABLY WEATHERED INTERMEDIATE GEO-MATERIAL CLASSIFIED AS BOTH HARD SOIL AND EXTREMELY WEAK TO VERY WEAK ROCK; FINE TO MEDIUM GRAINED WITH SILT, THIN TO MASSIVE BEDDED, WITH INTERBEDS OF SILTSTONE.

GWL–GROUND WATER LEVEL (INFERRED) BASED ON MONITORING WELL MEASUREMENTS FROM YEARS 2002 TO 2005.

LITHOLOGICAL CONTACT, DASHED WHERE INFERRED




## SUBSURFACE PROFILE ALONG ALIGNMENT

APPROXIMATE SCALE H: 1" = 20'
V: 1" = 40'

NOTES:

1. FOR A DETAILED DESCRIPTION OF CONDITIONS AT EACH BORING, REFER TO BORING LOGS INCLUDED IN THE GEOTECHNICAL REPORT PREPARED BY RUTHERFORD AND CHEKENE, DATED 21 JANUARY 2005.
2. GROUND STRATIFICATION AND GROUND WATER LEVEL SHOWN REPRESENTS NECESSARY EXTRAPOLATION FROM AVAILABLE INFORMATION AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
3. LOCATION OF EXISTING PEP TUNNEL IS APPROXIMATE. REFER TO POSITION ELECTRON PROJECT, BEAM HOUSING PLAN AND PROFILE, REGIONS 3-4-5, SHEET C5, DRAWING ID 323-001-09-R0 AS-BUIILT, FROM PBQ & D, 1979.
4. LOCATION OF EXISTING SLC TUNNEL IS APPROXIMATE. REFER TO STANFORD LINEAR COLLIDER, NORTH AND SOUTH ARC TUNNEL, SOUTH ARC TUNNEL PLAN AND PROFILE, SHEET C-11, DRAWING ID 372-011-08-R2, REV 2, FROM TUDOR ENGINEERING, 1986.

| Hatch Mott MacDonald | LCLS – GEOTECHNICAL BASELINE REPORT<br>TUNNEL PLAN AND SUBSURFACE PROFILE | DATE: 3/16/06<br>FIGURE 2 |
|---|---|---|

P:\SLAC\22042A\6.0 Drawings\31606\TUNNEL SECTION.dwg

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# 2.0 Project Description

SLAC is a national research facility operated by Stanford University for the USDOE. Research at the facility centers around experimental and theoretical particle physics using accelerated electron beams. The LCLS facility is intended to create a new type of x-ray light source from a single pass free electron laser.

The new facility is a series of tunnels and underground caverns about 2200 feet in length extending east of the existing research facility. An approximate plan of the underground facilities is shown on Figure 2. Details are shown on the Subcontract Drawings. The underground construction will occur in a formation called the Ladera Sandstone, which is an intermediate geo-material that can exhibit the properties of soil and rock. The new facility will be connected to the existing facility with a new Beam Transport Hall, which is a box structure constructed across the existing Research Yard.

The key elements of the new underground construction that are addressed in this GBR are included below. Approximate excavation sizes are provided. Refer to the Drawings for exact dimensions.

1. Undulator Hall Tunnel – a tunnel designed to house about thirty-three specially designed undulator magnets, which extends approximately 600 feet east from the new Beam Transport Hall to the Front End Enclosure. The excavated cross section is a 'D' shape about 21 feet wide and about 19 feet high measured at the tunnel centerline.

2. X-Ray Tunnel – a tunnel that extends approximately 650 feet from the Near Experimental Hall (NEH) to the Far Experimental Hall (FEH), which will house specially designed optical equipment. The excavated cross section is a 'D' shape about 21 feet wide and about 19 feet high measured at the tunnel centerline.

3. Far Experimental Hall (FEH) – the FEH is an underground experimental facility approximately 250 feet long. The excavated cross section is a 'D' shape about 49 feet wide and about 33 feet high measured at the tunnel centerline.

4. Access Tunnel – an access tunnel will extend approximately 350 feet from Service Station G, on the east side of the existing Stanford Linear Collider (SLC) Experimental Hall, to the FEH. The excavated cross section is a 'D' shape about 21 feet wide and about 19 feet high measured at the tunnel centerline.

Portals are required at both ends of the Undulator Hall Tunnel, at the west end of the X-Ray Tunnel, and at the north end of the Access Tunnel. Portal excavation will be completed by others prior to tunnel excavation under this Subcontract. The Tunneling Subcontractor will be responsible for stabilizing the portal as detailed in the tunnel scope.

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# 3.0 Sources of Geologic and Geotechnical Information

The sources of geologic and geotechnical information used to develop this GBR are two geotechnical data reports (Rutherford & Chekene, 2003 and 2005a) for the LCLS Project and information from two previously constructed tunnels, the Positron-Electron Project (PEP) and the SLC tunnels in the vicinity of the LCLS facilities.

## 3.1 Overview of Investigation Program

Two subsurface investigations were carried out by Rutherford & Chekene in 2003 (May 27 to June 2) and 2004 (July 19 to August 18). The explorations consisted of drilling, logging, and sampling borings, conducting in-situ testing, and performing laboratory testing of the borehole samples.

### 3.1.1 Subsurface Drilling

In 2003, Rutherford & Chekene oversaw the drilling of five boreholes performed by Taber Drilling of West Sacramento, California. The borings (SB-A to SB-E) ranged in depth from 41.5 feet below ground surface (bgs) to 106.5 feet bgs and are summarized in Table 1 of the 2003 Geotechnical Data Report (R&C, 2003). The detailed boring logs are presented in Appendix A of the 2003 Geotechnical Data Report.

In 2004, Rutherford & Chekene oversaw seven additional borings (LCLS-1, LCLS-2, LCLS-3, LCLS-3A, LCLS-4, LCLS-5, and LCLS-6) drilled by Taber Drilling. Table 1 of the 2005 Geotechnical Data Report summarizes the borings that range in depth from 31.8 feet to 136.5 feet bgs. The boring logs are presented in Appendix C of the 2005 Geotechnical Data Report, Volume 1 (R&C, 2005a).

### 3.1.2 Environmental Testing

Soil samples were collected for environmental testing at the ground surface, at approximately zero to one foot, and subsequently at one-foot intervals below grade through the fill and/or residual soil at five boring locations.

### 3.1.3 In-Situ Testing and Measurement

Specialty in-situ geophysical and pressuremeter testing were conducted in the borings during the 2004 field exploration. GEOVision Geophysical Services of Corona, California logged exploratory borings LCLS-5 and LCLS-3, using P-S Suspension Logging techniques. Geophysical logging allowed Rutherford & Chekene to estimate the P- and S-wave velocities of the various soil strata in the borehole. P-waves are compression waves, whereas S-waves are shear waves. The shear wave velocities obtained from geophysical logging were used in a ground motion study and in developing seismic design criteria.

The results of the geophysical logging for exploratory borings LCLS-3 and LCLS-5 are presented in Appendix F of the 2005 Geotechnical Data Report, Volume 1 (R&C, 2005a).

Twenty-one pressuremeter tests were performed by Fugro Geosciences of Oakland, California in exploratory borings LCLS-3A and LCLS-6. Six pressuremeter tests were performed in LCLS-

3A, starting at a depth of about 15 feet, and 15 tests were performed in LCLS-6, starting at a depth of about 39 feet. Tests were performed at approximately 5-foot depth intervals. The tests were performed in accordance with ASTM D-4719. The results of the pressuremeter tests are presented in Appendix G of the 2005 Geotechnical Data Report, Volume 1 (R&C, 2005a).

### 3.1.4 Geotechnical Laboratory Tests

Rutherford & Chekene commissioned Cooper Testing Laboratory (CTL) of Mountain View and Geo Test Unlimited (GTU) to perform laboratory index and strength characteristic testing of selected core samples from the borings. John Wakabayashi was commissioned by Rutherford & Chekene to perform mineralogical analysis of selected core samples. Laboratory tests were performed on selected samples to characterize the pertinent engineering properties of the earth materials to be encountered along the tunnel alignment. The tests, which are summarized in Table 1, were performed according to ASTM specifications where applicable. The test results are contained in the 2003 Geotechnical Data Report (R&C 2003) and Appendix D and E of the 2005 Geotechnical Data Report, Volume 1 (R&C, 2005a).

**Table 1 Summary of Laboratory Tests**

| Type of Test | Applicable Test Method | Purpose | Total No. of Tests (2003) | Total No. of Tests (2005) |
|---|---|---|---|---|
| Slake Durability | ASTM 4644-87 | Slake durability | 7 | 2 |
| Dry Density | ASTM D 2937 | In-situ density | 3 | 7 |
| Moisture Content | ASTM D 2216 | Iin-situ moisture content | 3 | 7 |
| Unconfined Compression | ASTM D 2166 | Unconfined strength | 13 | 3 |
| Particle Size Analysis with Hydrometer | ASTM D 422 | Particle size distribution and clay/ silt content | - | 2 |
| Unconsolidated - Undrained Triaxial | ASTM D 2850 | Undrained shear strength | - | 8 |
| Creep Test | ASTM D 5202 | Creep potential | - | 2 |
| Shink-Swell / Expansion Pressure | ASTM D 3877 | Shrink and swell potential | - | 4 |
| Cerchar Abrasion | - | Abrasiveness | - | 2 |
| Rock Modulus and Poisson's Ratio | ASTM D 3148 | Elastic modulus and Poisson's ratio | - | 2 |
| Unconfined Compression of Rock Core | ASTM D 2938 | Unconfined strength | - | 2 |
| Resistance Value (R-value) | Caltrans 301 | Resistance | - | 3 |
| Consolidated Undrained Triaxial | ASTM D-4767-mod | Undrained shear strength | 9 | |
| Petrographic Analysis | | Mineral content | 8 | |

The R&C data (2003 and 2005a) have been combined into one dataset and statistical summaries for the combined dataset are presented in Section 5.0.

### 3.1.5 Rock and Groundwater Chemistry

Rutherford & Chekene commissioned CERCO Analytical of Pleasanton to perform a corrosivity analysis of eight samples of Ladera Sandstone taken from the exploratory borings using ASTM methods. Tests were performed to measure resistivity, pH, chloride, and sulfate ion concentrations, and redox potentials of the samples. No groundwater chemistry test data was reported.

CERCO concluded based on the resistivity measurements, that two samples are classified as severely corrosive, one sample is classified as corrosive, and the remainder of the samples are classified as moderately corrosive. CERCO also concluded that, based on chloride ion concentrations, two samples are classified as being able to attack steel embedded in a concrete mortar coating.

R&C report that water quality analysis of samples obtained from the SLAC area indicate fresh to brackish groundwater occurs within the site. The sulfate content in tested groundwater samples is reportedly corrosive and may affect concrete.

## 3.2 Geologic Exposures

The ground surface elevation at the Research Yard located immediately west of boring SB-A is near the proposed tunnel invert elevation of 245 feet. The west-facing slope at the Research Yard, near Boring SB-A, is cut at approximately one (horizontal) to one (vertical) (or 1H: 1V) and provides observable exposures of the Ladera Sandstone. The experimental hall for the SLC, located north of Boring SB-D and SB-E, is constructed in a 2H:1V cut with a maximum slope height of about 80 feet. The Santa Clara Formation and the Ladera Sandstone are exposed in the cut slopes at this building location.

## 3.3 Groundwater Conditions

Groundwater monitoring data has been collected for over 30 years from monitoring wells at SLAC and monitoring wells close to the alignment are shown on Figure 2. Earth Sciences Associates (ESA) completed a review of the data and prepared a report in 1994 entitled 'Stanford Linear Accelerator Center, Hydrogeologic Review' (ESA 1994). The report provides estimates of groundwater levels in the vicinity of the proposed LCLS tunnels. Further monitoring from years 2002 to 2005 (SLAC, 2006) have been reviewed during preparation of this GBR.

Groundwater was reportedly not encountered to the maximum depth of exploration in borings SB-A through SB-E or in borings LCLS-1 to LCLS-6 from the 2003 and 2005 exploration programs.

Groundwater information is also available from the Positron-Electron Project (PEP) and Stanford Linear Collider (SLC) tunnels previously constructed on the site below the elevation of the proposed LINAC tunnels.

## 3.4 Local Construction Experience

There are existing tunnels in the project vicinity that are part of the Stanford Linear Accelerator Center facilities. The location of these tunnels in relation to the LCLS project is shown on Figure 2. The PEP tunnel ring and SLC South Arc tunnel pass below the X-Ray Tunnel at a near perpendicular orientation with as little as 15 feet clearance. Refer to the Subcontract Drawings for the best information on this clearance. The information available regarding construction of these tunnels that was used to guide the design of the LCLS project follows.

### 3.4.1 Positron-Electron Project (PEP)

The PEP ring tunnel was investigated and designed in the late 1970s by Parsons Brinkerhoff Quade and Douglas (PBQ&D) and Kaiser Engineers, joint venture. Construction occurred in 1977 (Elioff, 2005). Region 3 of the PEP ring was constructed in Ladera Sandstone by tunneling methods near the present LCLS alignment with cover height varying from 35 to 95 feet. The

Region 3 tunnel was excavated in a D shape with a width of about 12 to 13 feet and a height of about 11 to 12 feet based on the Typical Tunnel Section on As-Built Drawing S151 (PBQ&D, 1979). About 85% of the ground was described as indurated sandstone, while the remaining 15% was of poorer quality and encountered water (Elioff, 2005). Ground support shown on As-Built Drawing S152 (PBQ&D, 1979) consisted of shotcrete, shotcrete plus steel sets and shotcrete plus rock reinforcement. The final lining consisted of shotcrete. The PEP tunnel is currently accessible, and portions of the PEP ring are experiencing seepage inflows.

### 3.4.2 Stanford Linear Collider (SLC Project)

In 1983 – 84, the 10 feet diameter, 9448 feet long SLC Arc tunnels were constructed at SLAC. The eastern extent of the SLC South Arc tunnel is located immediately below and perpendicular to the proposed X-Ray Tunnel. The SLC tunnels were excavated by roadheader and supported by steel ribs and fiber reinforced shotcrete in Miocene Sandstone with an average strength reported to be 50 pounds per square inch (psi) (Rose, 1985).

The following are excerpts from personal communications regarding the SLC tunnel project (Colzani 2005, and Kaneshiro 2005) describing ground conditions as follows:

*"Geology - Gray to Black (Tan to Buff when weathered) dense silty, fine-grained Miocene sandstone with intermittently dispersed large cobble sized dense (limestone?) concretions".*

*"In general, good to excellent stand up time, firm, but probably less than 10% of the ground is slow raveling"*

*"Loose (running) saturated fine-grained sands with interbedded gravels."*

*"Black to Gray/Green hard siltstone interbedded with squeezing clay."*

*"Tan to Buff dense sands and gravels (Water Bearing)"*

*"Groundwater - Generally above water table except in the area of the South Cut and Cover Box area and a portion of the N. Arc Tunnel East of the N. Adit Structure. Random seeping and weeping of groundwater was experienced along the tunnel alignment."*

Based on these assessments, it appeared that the Miocene Sandstone was generally a good tunneling medium and exhibited stand-up times in excess of days. However, there were some notable exceptions, summarized as follows:

*" a series of randomly spaced cross-joints created unpredictable wedge failures…" … " ranging from a cubic foot to a cubic yard in size."*

*" a formation of interbedded dense siltstone" and "squeezing clays."*

*" groundwater inflows described as seeping to dripping"*

*" running sands"…. " underlain by gravels"*

*" Softening of the tunnel invert due to nuisance water seepage and movement of construction equipment"*

# 4.0 Site Setting

## 4.1 Geologic Setting

The SLAC is located in the central part of the Coast Range's physiographic province of California. The geology of the Coast Ranges is extremely complex in the variety of rock formations as well as structure. The project site lies in the eastern foothills of the Santa Cruz Mountains, which form part of the Coast Ranges, in an area underlain mainly by Tertiary-age marine sedimentary rocks and Mesozoic-age sedimentary, volcanic, and metamorphic rocks of the Franciscan Complex. The Tertiary-age sedimentary rocks are covered in places by younger formations, such as the Plio-Pleistocene-age Santa Clara Formation and Quaternary-age alluvial and terrace deposits. Depositional contacts between these various units are generally angular unconformities resulting from periods of erosion, folding, and faulting. The project Geotechnical Data Report (R&C, 2005a) addresses geologic setting in detail and should be reviewed for more information.

## 4.2 Hydrogeologic Setting

Available groundwater information summarized below is used to establish baselines for groundwater in Section 5.

The ESA 1994 Report estimates the following groundwater levels based on review of 30 years of groundwater data at SLAC: higher than elevation 240 feet near boring SB-A; elevation 230 feet near boring SB-B; about elevation 210 feet near boring SB-C; and about elevation 190 feet near boring SB-D.

SLAC (2006) groundwater level monitoring shows that groundwater depth in the Research Yard varies from 5 to 10 feet, and that groundwater levels fluctuate seasonally. Seasonal groundwater level fluctuations during the 2002 to 2005 period east of the research yard were less than 20 feet.

R&C did not report encountering groundwater to the maximum depths of exploration in either borings SB-A through SB-E during the 2003 geotechnical study or in borings LCLS-1 to LCLS-6 in their 2005 investigation; however, groundwater should have been encountered at the depths drilled based on the ESA Report (ESA 1994) and more recent monitoring (SLAC 2006). It is believed that the presence of groundwater could have been masked by the use of the rotary method of drilling, which involves the use of drilling mud.

R&C indicate in the 2005 Report that minor free water is expected within fractures and some of the cleaner sandstone strata within the Ladera Formation. Observation of drainage system flows in the existing PEP and SLC tunnels confirms this assessment. The PEP tunnel, which lies below the groundwater table in parts of the ring, has a drainage system that reportedly produces less than two gallons per minute.

## 4.3 Seismologic Setting

### 4.3.1 Faulting

Although numerous minor shear zones cut the Tertiary-age rocks, no major faults have been discovered or mapped previously on the SLAC site nor have any active faults been located. Known active faults that are considered capable of causing strong ground shaking at the site include the San Andreas and Monte Vista-Shannon faults. The nearest of these, the peninsula-section of the San Andreas fault, passes approximately 2.5 miles southwest of the site and has the potential to generate a maximum earthquake of $M_w = 7.1$. The location of this segment of the San Andreas Fault relative to the site is shown in Figure 6 of the 2005 Geotechnical Data Report.

Several minor faults were mapped in the SLAC and PEP excavations and in the cuts for Alpine Road east of the site. The strike of the faults are randomly oriented with a common orientation striking northeasterly, and dipping from 35 to 90 degrees. None of the faults are accompanied by shear zones and gouge greater than 8 feet wide measured normal to the fault plane. The site is not located within an Earthquake Fault Hazard Zone.

### 4.3.2 Ground Shaking

The site is located within the Uniform Building Code (UBC) Zone 4, in the seismically active San Francisco Bay region. During a major earthquake on any one of the nearby active faults, the site may experience strong ground shaking. The intensity of the earthquake ground motion at the site will depend on the characteristics of the generating fault, the distance to the earthquake epicenter, the magnitude and duration of the earthquake, and specific site geologic conditions.

A number of historical earthquakes have affected the area, including the 1906 earthquake and the more recent Loma Prieta earthquake. According to Rose (1990), during the October 17, 1989 Loma Prieta earthquake, movements of more than 0.4 inch vertically and about 0.2 inch horizontally occurred in the LINAC concrete housing near its junction with the SLC tunnel. During that earthquake, accelerations at SLAC were reported to have reached 0.3g where g is the acceleration of gravity.

### 4.3.3 Future Earthquakes

The United States Geological Survey estimates the chance of one or more major (M6.7 or larger) earthquakes on the San Andreas Fault between 2003 and 2032 to be 21 percent. The corresponding probability for major faults in the whole San Francisco Bay region within the same time period is estimated to be about 62 percent.

## 4.4 Ladera Sandstone

The Miocene-age Ladera Sandstone is what the underground excavations will encounter. The Ladera Sandstone is an intermediate geo-material that can be classified as both soil and rock. It is fine to medium grained silty, friable sandstone that includes interbeds of siltstone, although it was logged as soil in some instances by R&C (2003, 2005a). Strength varies with degree of cementation. Bedding thickness is reportedly thin to massive, but bedding planes are not a source of weakness in the rock mass. Bedding of the Ladera Sandstone exposed in the Research Yard strikes northeast and dips from 9 to 25 degrees toward the southeast.

The degree of weathering of the Ladera Sandstone is indicated by stained joints, discoloration of the rock mass and reduction in the degree of cementation, which affects strength. The discoloration in the Ladera Sandstone is not related to depth below ground surface. In general, the sandstone near the ground surface was either uncemented or weakly cemented, while the cementation was weak to moderate as depth increased (R&C, 2005a). Occasionally, thin to moderately thick beds that are strongly cemented with calcium carbonate are present locally. Minor occurrences of thin calcium carbonate and gypsum filled fractures are present.

Core recovery in the Ladera Sandstone was typically greater than 80 percent, and three quarters of the core runs had 100% recovery. Recovery is shown in Figure 2.

The sandstone grains are made up of 25 to 35 percent quartz, 25 to 45 percent feldspar and 25 to 50 percent lithic (rock) grains, which are a combination of minerals. A total quartz content considering the quartz content of lithic grains was not obtained. Serpentinite reportedly constitutes about 5 to 20 percent of the sample. The serpentinite is reported to be composed mainly of Lizardite, but also includes Chrysotile, a potentially hazardous asbestos mineral if encountered in sufficient quantity. The specifications provide direction on handling hazardous materials.

The cementing agent in the sandstone is reportedly a combination of calcium carbonate, silicate and clay.

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# 5.0 Ground Characterization

## 5.1 Rock Type and Intact Rock Strength

The anticipated occurrence of Ladera Sandstone along the alignment is shown in Figure 2. The rock mass is described as variably weathered and variably cemented. Both weathering and degree of cementation have a strong influence on the intact rock strength. The use of rock terminology to describe the properties of the anticipated ground is not a baseline. Weathering and/or lack of cementation are anticipated to affect the behavior of the Ladera Sandstone such that soil tunneling conditions are encountered in limited areas, as described in Section 5.3

The sandstone is described by various authors using terms like weakly to strongly cemented, or uncemented to well cemented. Note that there is no consistent classification system to describe degree of cementation in sandstone. For baseline purposes, assume the least cemented Ladera Sandstone will be uncemented and described using soil classification terminology as a very dense granular soil. This material is also described as friable and during laboratory sample preparation, the material can be cut and trimmed with a trowel, yet it is competent enough to be recovered using rock core drilling techniques, with 100 percent core recovery being common.

The strongest rock samples are described as well-cemented or strongly-cemented and frequently occurred in core intervals less than 3 feet thick. Occasionally, 20-foot thick intervals of well- or strongly cemented rock are reported in boring logs near the alignment. Samples of this material had to be cut using a power saw for trimming in the laboratory. For baseline purposes, assume the most cemented Ladera Sandstone is classified as weak rock using terminology defined by the International Society of Rock Mechanics (ISRM, 1981).

There does not appear to be a trend in degree of weathering or cementation within the length and cross-sectional dimensions of the LCLS tunnels. For baseline purposes, it should be expected that weathering and lack of cementation appear randomly throughout the rock mass.

### 5.1.1 Intact Rock Strength Data

Intact rock strength was measured in the laboratory using the unconfined compressive strength test and the triaxial compression test. Figure 3 summarizes the unconfined compressive strength data from R&C (2003 and 2005a). Intact rock strength of the Ladera Sandstone is expected to be dependant on moisture condition. The strengths presented in Figure 3 are for rock at field moisture condition as described in Section 5.4. The intact rock strength is expected to increase when the rock is dry, based on observations of the Ladera Sanstone and laboratory tests of other weakly cemented sandstones. The laboratory tests represent the strength of the intact rock but not the rock mass, which contains discontinuities and other defects.

Two creep tests were performed to investigate the time dependant effects of an applied load on intact rock strength. Neither of the two tests reached creep failure. Rutherford & Chekene state that in general the creep test results showed that rock deformation is minimal when considering the long-term effect of an applied load.

Intact rock strength was also tested in the laboratory using the triaxial compression test, whereby a confining pressure is applied to the sample as it is loaded axially. Based on the R&C data (R&C 2003 and 2005a), the average Mohr-Coulomb strength parameters are a cohesion of 31 psi and a friction angle of 44 degrees for the range of minor principal stress up to 40 psi, commensurate with a maximum tunnel depth (to springline) of about 90 feet.

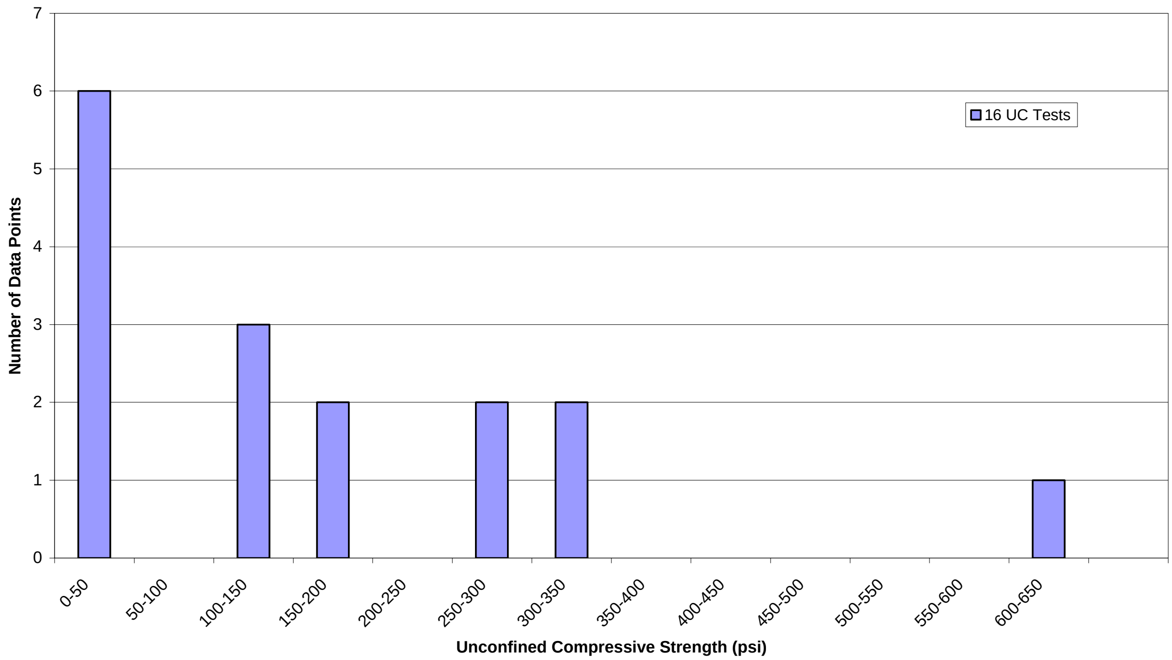


**Figure 3 - Histogram of Unconfined Compressive Strength from Combined 2003 and 2005 Dataset**

### 5.1.2 Baseline Values

The baseline strengths for intact rock are expressed in terms of uniaxial compressive strength and Mohr-Coulomb strength parameters.

For baseline purposes, the average uniaxial compressive strength of intact rock is 167 psi. The strengths are representative of the ground at its natural moisture content. Drying tends to increase the intact strength. For baseline purposes, assume that the unconfined compressive strength of intact rock increases by 10% when the rock is dried out.

For baseline purposes, the average Mohr-Coulomb strength parameters are a cohesion of 31 psi and a friction angle of 44 degrees for the range of minor principal stress up to 40 psi, commensurate with the maximum tunnel depth (to springline) of about 90 feet.

These baseline rock strengths are representative for intact rock that could be sampled from rock cores recovered from the geotechnical investigation and that were robust enough to test. The average intact rock strength was based on all of the available laboratory unconfined compressive and triaxial strength test data.

It is expected that 5 percent of the rock mass exposed in the portal and tunnel excavations will consist of weak sandstone layers up to 6 inches in thickness. These layers have not been sampled nor tested, but can be expected to have a uniaxial compressive strength of up to 1,000 psi. Excavation of these hard layers should be expected to cause significant vibrations to roadheader excavation equipment.

There does not appear to be a trend in intact rock strength within the length and cross-section of the various tunnels. It should be expected that hard layers will appear randomly within the rock mass.

## 5.2 Fractures and Faults

Fracturing is the term used by R&C to describe rock mass defects. Fracturing was observed in rock cores and recorded on field logs. Field core logs indicate between zero and five natural fractures per foot depth of vertical drill hole where cemented rock core was recovered. The orientation of fractures appears to be primarily horizontal to sub-horizontal with the occasional sub-vertical fracture.

It should be noted that there were lengths of uncemented rock cores which were classified as ‘sand’ in the boring logs but which have been assigned zero fractures per foot in the Geotechnical Data Reports (R&C 2003 and 2005a). For purposes of discussing fracture spacing in this GBR, materials classified as uncemented or as sand are not considered to be intact rock. Similarly, material that was not recovered by coring is not considered intact rock.

Baseline values for fracture spacing of intact rock have been developed following an evaluation of the core logs (R&C 2003, 2005a). For baseline purposes, an average of one fracture per vertical foot can be expected in excavations in intact rock, with a range of 0 to 5 fractures per foot in intact rock. Fracture orientations can be expected to be primarily horizontal to sub-horizontal with infrequent sub-vertical fractures. There does not appear to be a trend in fracturing within the lengths and cross-sections of the various tunnels. It should be expected that fractures will appear randomly.

Excavations for the SLC and PEP tunnels encountered faults up to eight feet wide measured normal to the fault plane that contained closely spaced fractures and gouge. The faults were frequently oriented with a northeasterly strike and a dip of 35 degrees to 90 degrees. For baseline purposes, assume two of these faults will be encountered in the underground excavations.

## 5.3 Rock Mass Quality

Rock mass quality was initially assessed using the Geomechanics or Rock Mass Rating (RMR) System originally proposed by Bieniawski, (1974) and the “Q” Rock Mass Rating System originally proposed by Barton Lien and Lunde (1974). Both of these rating systems consider intact rock and rock mass properties, including RQD (Deere, 1989). During core logging, Rutherford & Chekene assessed the RQD of the rock to be essentially zero due to the rock’s low strength. Based on this assumption, RMR ratings resulted in values between 50 and 30, indicating generally fair to poor rock. The resulting Q evaluations ranged from 0.5 to 0.042, indicating very poor rock. These RMR and Q ratings are inconsistent with observed ground behavior in the PEP and SLC tunnel excavations described in Section 3.4. Both sets of rock mass ratings are believed to be over-penalized by the low RQD values, underestimate the rock mass quality, and are considered non-representative of the conditions to be encountered. Based on experience during previous tunnel excavations for the PEP and SLC Tunnels, the ground behavior during tunnel excavation can be expected to be firm to slow raveling for 85% of the project, and fast-raveling to running for 15% of the project. Descriptions of the terms firm, raveling, flowing and running are based on the Tunnelman's Classification System. This classification system was developed for classifying tunneling conditions in soil, and it has been used to describe the ground behavior during previous tunneling at SLAC. For baseline purposes, it is the classification system adopted for this GBR. Descriptions of these terms are provided in Appendix A.

## 5.4 Density and Moisture Content

For baseline purposes, the dry unit weight can be expected to average 112 pounds per cubic foot (pcf). The moisture content can be expected to average 13% and the porosity is expected to average 36%.

Histograms of the dry unit weights and moisture contents from the combined set of R&C data (R&C 2003, 2005a) are shown in Figures 4 and 5, respectively.

## 5.5 Durability

The Ladera sandstone is generally considered a friable sandstone which indicates poor durability. Slaking is the deterioration and breakdown of a rock after exposure to either air or water as the result of excavation. The slake durability test measures a material's resistance to erosion when tumbled and washed with water. Seven slake durability tests were presented in the 2003 report and two in the 2005 report. All of the test results fall into the "Type III" category, which indicates that the samples will slake easily. Slake Durability tests indicated indices of from 0 to 15 with 85% of test results indicating indices less than 10.

Based on the results of these tests, the Ladera Sandstone is considered highly susceptible to slaking.

## 5.6 Abrasivity

Two Cerchar Abrasion tests were performed from samples at tunnel depth; one from a sample from Test Hole LCLS-3, the other from in Test Hole LCLS-6. The Cerchar Indices were 0.8 and 2.2 respectively. Also, based on petrographic thin section analyses summarized in Appendix D of the R&C 2005 Report, it was determined that 65% of the minerals that comprise the Ladera Sandstone have a Moh's hardness value of six or higher.

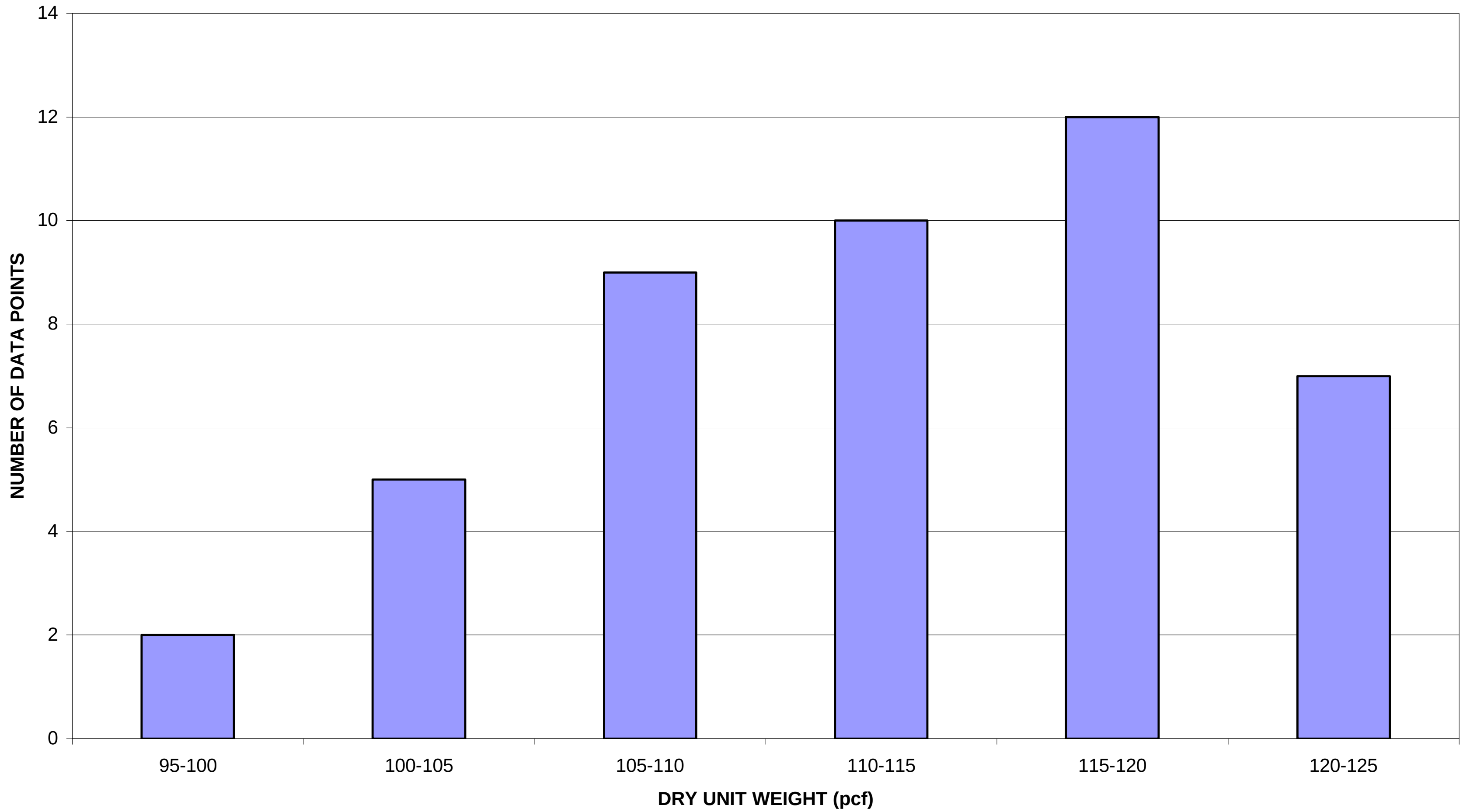


**Figure 4 - Histogram of Dry Unit Weights from Combined Dataset**

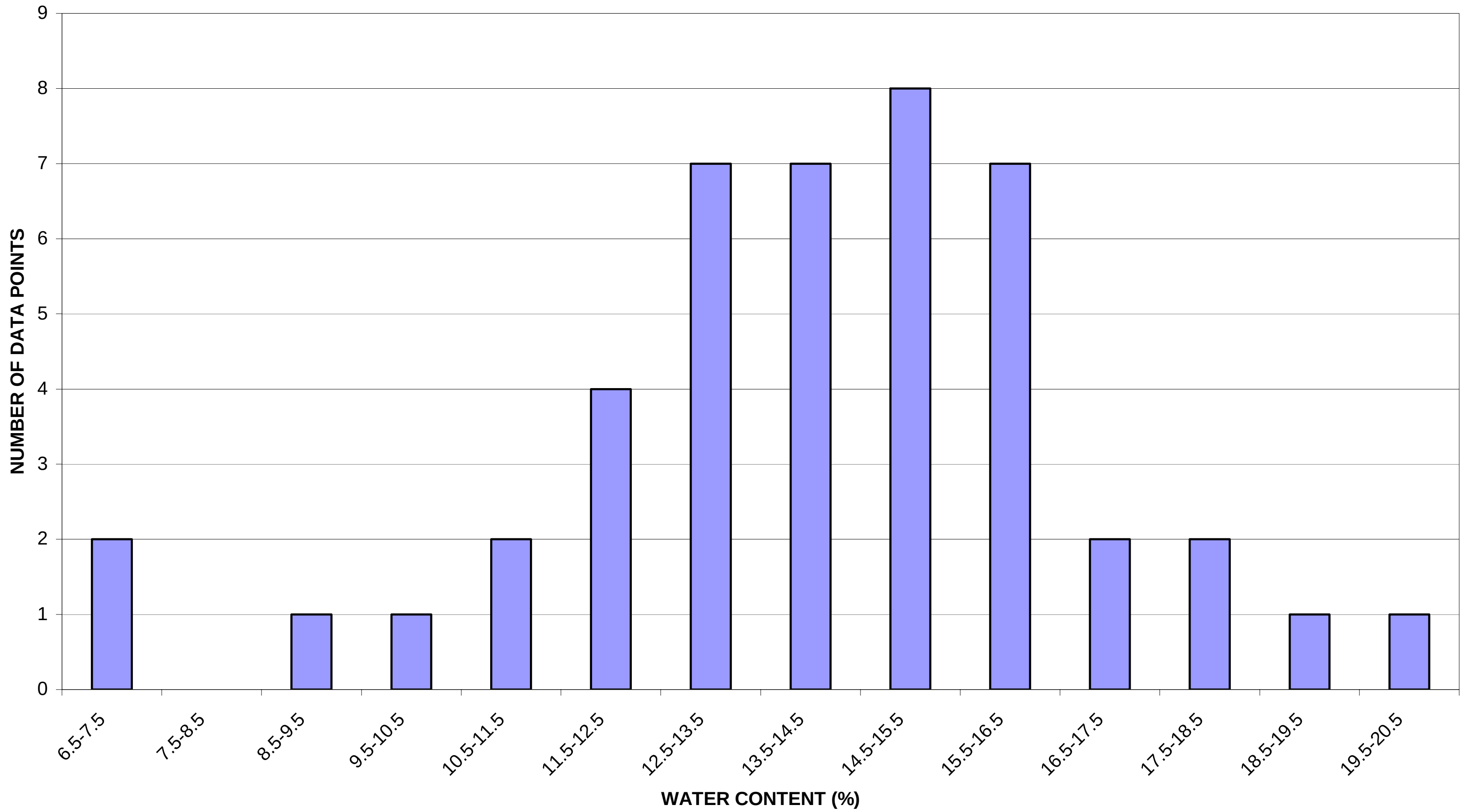


**Figure 5 - Histogram of Moisture Contents from Combined Dataset.**

Abrasivity is dependant on the hardness of grains, grain size, shape, and strength of cementation between the grains. The Cerchar Abrasivity Test is believed to be more representative of abrasivity for this project than hard mineral content because basing abrasivity on hard mineral content alone ignores the effect of cementation, which is relatively low in the Ladera Sandstone. The Cerchar Abrasivity Test and resulting Cerchar Abrasivity Index (CAI) can be used to predict wear on equipment and cutter tools. For baseline purposes, assume that the average CAI value for the Ladera Sandstone is 2.2. It is important to use the same test procedures reported by R&C (2005a) for any future CAI testing used for comparison with this baseline because of the variability in CAI test methods at different laboratories performing this test. Based on the baseline CAI, the Ladera Sandstone should be considered abrasive.

## 5.7 Groundwater

Existing groundwater data is not entirely consistent due to a number of factors. Although no groundwater was encountered to the maximum depths of exploration in the 2003 or 2005 borings there is ample evidence that groundwater exists immediately below the proposed tunnel and possibly in the tunnel profile near the West Portal.

For baseline purposes, the groundwater level should be assumed to be as shown in Figure 2. Free groundwater is expected above the groundwater level shown in Figure 2, within fractures and some of the sandstone strata, creating dampness and minor seeps measuring less than 2 gallons per minute locally.

## 5.8 Hazardous Gas

No gas occurrences were reported during the geotechnical site investigations; however, gas is often encountered in rock similar in age to Ladera Sandstone in the bay area. As a baseline, assume that traces of gas will be encountered, but will not exceed 10% of the lower explosive limit while ventilation is provided.

## 5.9 Pre-Existing Hazardous or Contaminated Substances

Serpentinite is known to occur in the Ladera Sandstone. The serpentinite is reported to be composed mainly of the mineral Lizardite, but also includes Chrysotile, a naturally occurring asbestos mineral. No pre-existing hazardous or contaminated substances were encountered in any of the boreholes drilled for the project that would affect underground construction.

For baseline purposes, assume that Chyrsotile asbestos will be encountered, but not in sufficient concentrations to warrant special handling or disposal measures beyond controlling dust and monitoring worker exposure to asbestos, as required by law.

## 5.10 Surface Debris or Obstructions

No surface debris or obstructions are known to exist at the site. For baseline purposes, assume that no surface debris or obstructions will be encountered in excavations or deter construction except those that may develop during construction due to the construction activity of the project. Such obstructions are dealt with in the General Conditions.

Shafts constructed from ground surface into the crown of the Undulator Tunnel, as shown on the Drawings, will cause an obstruction to tunneling.

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# 6.0 Design Considerations

## 6.1 Ground Stabilization

### 6.1.1 Portals

Open cut excavations will be completed by others prior to the Tunneling Subcontractor taking possession of the site. The Tunneling Subcontractor will be required to complete final trimming and stabilization of the portal as detailed in the tunnel scope. A vertical face extending 2 feet above and around the tunnel opening will be excavated. The 2 feet zone will be covered by 6 inches of shotcrete and twenty feet long grouted canopy tubes or self-drilled pipe will be installed in the portal face in a pattern extending 60 degrees on either side of tunnel centerline, as shown on the Drawings.

### 6.1.2 Tunnels

All tunnels (Undulator, X-Ray, Far Experimental Hall and Access tunnels) will be a 'D' shape, and range in excavated height from 18 to 32 feet and in excavated width from 21 to 49 feet. Tunnels have been designed recognizing the variability of the ground. Recognizing the relatively short lengths of each tunnel, the design consists of a single, 'standard' support system with one exception discussed below, with the provision of contingency support as needed to manage variable ground conditions. The exception is a section at the east end of the Undulator Hall where the single 'standard' support is augmented with pre-support as shown on the Drawings to address a low cover condition.

The excavation of the Undulator, X-Ray and Access Tunnels, which are approximately the same size and shape, is required to be completed in a heading and bench sequence as shown on the Drawings. Restrictions have been placed on the dimensions and sequencing of the headings and benches.

The standard support system consists of:

- 6 inches of synthetic fiber reinforced shotcrete applied in two-3 inch layers; and
- lattice girders installed between the first and second layer of shotcrete.

Contingency support elements are included in the Tunneling Subcontract to supplement the standard support systems at any location where ground behavior warrants contingency measures. Contingency support elements in the Subcontract include rock dowels/bolts, self drilled and grouted pipe spiles, fiberglass rock bolts, shotcrete and lattice girders. The design of the standard support system and the types and quantities of contingency support measures are based on the ground conditions and geotechnical parameters as described in Section 5. The anticipated ground condition intended by the design to warrant contingency support is noted in the Drawings on Sheet T9001.

A final layer of un-reinforced shotcrete ranging from 6 to 12 inches completes the support in the crown and walls of the tunnels.

Tunnel invert will be protected by a working concrete slab. A reinforced concrete base slab will complete the invert.

The following outlines specific design considerations for the various tunnels.

## 6.2 Undulator Tunnel

Special provisions have been made in the Drawings and Specifications to minimize the post-construction movement of the base of the Undulator Tunnel. These include deepening the excavation of the invert and forming a concave base; placing restrictions on the excavation sequence; and installing a thick reinforced concrete base slab. Pre-support is required to address a low cover condition near the east portal as shown in the Drawings on Sheet T2300.

## 6.3 X-Ray Tunnel

Special rock bolting and face support measures are required at the junction of the X-Ray Tunnel and the Far Experimental Hall as shown on the Drawings.

## 6.4 Far Experimental Hall Cavern

The Far Experimental Hall Cavern with excavated dimensions of 33 feet height and 48.5 feet width is the largest underground opening on the project and has design considerations related to its size that the other tunnels on the project do not have. In addition to the standard support system elements described in Section 6.1.2, cement grouted rock dowels are also used for permanent support. The Drawings and Specifications require the full excavation width to be completed in a series of smaller headings to control ground movement and preserve the integrity of the ground around the cavern. The number and location of headings will be selected by the Subcontractor subject to approval of the CM/GC

## 6.5 Access Tunnel

Special rock bolting and face support measures are required at the junction of the Access Tunnel and the Far Experimental Hall as shown on the Drawings.

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# 7.0 Construction Considerations

## 7.1 Tunnel Excavation

Because of the relatively low strength of the rock mass and its low durability, the Drawings and Specifications contain a number of requirements related to tunnel excavation and installation of initial support:

- The tunnels are required to be excavated using a top heading, bench and invert sequence, although the bench is only required for the larger FEH excavation.
- Top heading, bench and invert advance lengths, and distances between the advancing top heading, invert, and bench, are controlled. The following restrictions apply:
    - Maximum excavation length for top heading: 4 feet
    - Maximum excavation length for bench and invert : 8 feet
    - Minimum distance from heading to invert excavation (applies to Undulator, Xray and Access Tunnels): 60 feet
- Placement of a working slab, initial shotcrete, and secondary shotcrete in conjunction with lattice girders, will need to be accomplished according to prescribed sequences and within set time limits.

## 7.2 Contingency Measures

The standard support system shown on the Drawings shall be considered a minimum and shall be supplemented by Contingency Support elements as required. The requirement for Contingency Support elements will be determined by the Subcontractor and will be subject to approval of the CM/GC.

## 7.3 Over-break and Over-excavation

Over-break is anticipated where jointing creates unstable wedge fall-out, where the ground breaks back to canopy tubes and pipe spiles, and where lack of cementation results in running or raveling ground with stand-up time less than 4 hours. Where wedge fall-out occurs, it is expected to be up to a cubic yard in size for each occurrence.

Over-excavation is defined in the Subcontract Specifications as excavation of ground that occurs beyond the Excavation Line as shown on the Subcontract Drawings.

The excavated rock surface depicted in the Drawings is not intended to be a baseline description of anticipated over-break or over-excavation. It is the Subcontractor’s responsibility to control the excavated cross-section to accommodate its anticipated construction tolerances and any tunnel deformation. There will be no measurement and no payment for over-excavation or over-break nor measurement or payment for shotcrete to infill over-excavation or over-break.

## 7.4 Tunnel Convergence

It is expected that the maximum convergence of the crown and walls of tunnels will be between 1 and 3 inches during construction. For baseline purposes to provide adequate clearance inside the final lining design line shown on the Drawings, the Tunneling Subcontractor should layout the work on the baseline assumption that 2 inches of convergence will occur. The actual amount of convergence experienced is expected to be variable.

## 7.5 Invert Protection

Invert protection is addressed in detail in the subcontract because of the susceptibility of the Ladera Sandstone to deterioration. A limited duration of exposure of ground beneath the invert excavation is specified. Special requirements are in place for the Undulator Hall tunnel because of the tight tolerances required for the Undulator Hall Concrete Invert.

## 7.6 Drilling and Grouting of Rock Dowels, Spiles and Canopy Tubes

Due to the susceptibility of the Ladera Sandsone to deterioration, normal water flushing of drill cuttings from precussion drill holes is anticipated to result in excessive enlargement of drill holes. Drilling with water shall be avoided in accordance with the Subcontract. Regardless of drilling method, there is a likelihood of hole enlargement beyond the drill bit size in uncemented sandstone areas and this must be considered when planning the design of mechanical anchors or other mechanism to fix rock dowels in the hole prior to grouting and for estimating grout take for grouted canopy tubes, pipe spiles, rock dowels and bolts.

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## Appendix A – TUNNELMAN'S CLASSIFICATION (After Heuer, 1974)

| Classification | | Behavior | Typical Soil Type |
|---|---|---|---|
| Firm | | Heading can be advanced without initial support, and final lining can be constructed before ground starts to move. | Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed. |
| Raveling | Slow Raveling | Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling | Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress. |
| | Fast Raveling | | |
| Squeezing | | Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress. | Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at execution surface and squeezing at depth behind surface. |
| Running | Cohesive Running | Granular materials without cohesion are unstable at a slope greater than their angle of repose. When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose. | Clean dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-raveling. |
| | Running | | |
| Flowing | | A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter from the invert as well as the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases. | Below the water table in silt, sand or gravel without enough clay content to give significant cohesion and plasticity. May also occur in sensitive clay when such material is disturbed. |
| Swelling | | Ground absorbs water, increases in volume, and expands slowly into the tunnel. | Highly pre-consolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillite. |