PRELIMINARY GEOTECHNICAL ENGINEERING REPORT
PROPOSED LASSEN SUBSTATION
MOUNT SHASTA, CALIFORNIA
PACIFICORP PROJECT #10038611

PSI PROJECT No: 0595148

March 11, 2011

Prepared For:
PacifiCorp
1407 West North Temple, Suite 230
Salt Lake City, UT 84116

Attn: Mr. Benjamin S. Fowler, PE

Prepared By:

PROFESSIONAL SERVICE INDUSTRIES, INC.
2779 South 600 West
Salt Lake City, Utah
(801) 484 - 8827

March 11, 2011

Mr. Benjamin S. Fowler, PE
PacifiCorp
1407 West North Temple, Suite 230
Salt Lake City, UT 84116

SUBJECT: Preliminary Geotechnical Engineering Report
                   Proposed Lassen Substation
                   Mount Shasta, California
                   PacifiCorp Contract Release #4600002357
                   PacifiCorp Project #10038611
                   PacifiCrop P.O. #3000074794
                   PSI Project No. 0595148

Dear Mr. Fowler:

Professional Service Industries, Inc. (PSI) is pleased to submit this report of a preliminary geotechnical investigation for the proposed Lassen Substation in Mount Shasta, California. This report summarizes the work accomplished and provides our preliminary recommendations for design and construction of the substation based on the three completed borings. We understand that there will be a phase 2 field investigation for Lassen Substation if PacifiCorp chooses this location.

Based on the results of our limited field investigation, laboratory testing, and engineering analysis, the proposed site is potentially suitable for the construction from a geotechnical standpoint provided the recommendations of this report are followed. PSI recommends that additional investigation takes place at the Lassen Substation that includes additional borings and test pits to cover the whole property. The primary geotechnical considerations with respect to the proposed construction include shallow groundwater conditions, cobbles and boulders at varying depths, potential near surface liquefiable soils, foundation subgrade preparation and excavation, potential caving of drilled pier excavations in granular soils, and surface drainage. Recommendations regarding the geotechnical aspects of project design and construction are presented in the attached preliminary report.

PSI is committed to providing quality services to its clients, commensurate with their wants, needs, and desires. PSI appreciates the opportunity to provide its services on this project. If you have questions pertaining to this project or if we may be of further assistance, please call the undersigned. We appreciate your business and look forward to our next project with you.

Respectfully Submitted,
PROFESSIONAL SERVICE INDUSTRIES, INC.

TYLER K. BERNIUS
Staff Engineer

CHRIS J. GARRIS
Vice President

Martin L. Skyrman, PE
Senior Geotechnical Engineer

KEVIN C. MILLER
Principal Consultant

TKB/CTG/MLS/KCM
P:\595\\\2011 Projects\0595148 Lassen Substation\Report\0595148 Lassen Substation FINAL Geotechnical Repot 3-11-2011

TABLE OF CONTENTS

APPENDIX A

APPENDIX B

APPENDIX C

APPENDIX D

1.0 INTRODUCTION

This report presents the results of PSI's preliminary geotechnical investigation for the proposed Lassen Substation near Mount Shasta, California. This report was prepared in accordance with PSI's Proposal No. 38001 dated January 31, 2011 and the Master Services agreement #4600002357 between PacifiCorp and PSI executed December, 2010.

The purpose of the preliminary investigation is to explore subsurface materials and conditions at the project site and to develop shallow foundation, drilled pier, and soil-related recommendations for the design and construction for the proposed Lassen Substation in Mount Shasta, California. This preliminary report describes the work accomplished and presents PSI's conclusions and recommendations for design and construction of the project based on three field borings with auger refusal depths of 20, 7 ½, and 5 feet below the ground surface. The field investigation also included four geophysical Refraction Microtremor (ReMi®) arrays.

2.0 PROJECT DESCRIPTION

Based on the information provided in an RFP for Lassen Substation January 21, 2011, PSI understands that PacifiCorp is proposing the construction of a new substation in close proximity to the existing Mt. Shasta Substation, which will consist of the following inside a 115 kV yard.

- Demolition of a residential house
- Transformer
- Bus supports
- Switch Supports Switchgear
- Dead-end Structures
- Installation of an interior access road
- Relocation of drainage pipes
- Construction of the new yard

The access road will have an aggregate base course with crushed road rock. The portions of the substation yard not designated as roadways will be surfaced with clean crushed yard rock. PSI understands that PacifiCorp typically uses drilled pier foundations for the majority of structures including transmission line structures, and slab-on-grade foundations for transformers, breakers and switchgears. Loading conditions for the structures are shown below. Loading criteria was not provided for transformer pads, capacitor banks, and the control building.

Table 1: Typical Loading Information for Substation Structures and Transmission Line Foundations.

Substation Item	Loading Criteria	Design Governing Design
Line Dead-End Structures	60.0 kips (vertical)	High Wind
Switch Structures	10.0 kips (vertical)	High Wind
Bus Supports	5.0 kips (vertical)	High Wind
Breaker Pads	35.0 kips (vertical)	Seismic
Transformer pads*	180.0 kips (vertical)	Seismic
Capacitor Banks*	15.0 kips (vertical)	High Wind
Control Building*	75.0 kips (vertical)	High Wind

Note: * indicates that loading criteria was not provided. Values in table come from similar projects.

PSI anticipates that if a drilled pier foundation is selected, it will be design using the computer program PLS-CAISSONS by Powerline Systems, Inc. and/or LPILE by Ensoft, Inc. Other deep foundation systems will also be considered as determined by the subsurface conditions at the site.

PSI was not informed of the amount of cut/fill at the proposed Lassen Substation. We anticipate less than 2 feet for the preliminary investigation as a result of the relatively flat surface.

The geotechnical recommendations presented herein are based on the available project information, proposed substation location, and the subsurface conditions described in this report. If the loads or any of the noted information is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate.

3.0 SITE DESCRIPTION

3.1 Site Conditions and Topography

3.1.1 Lassen Substation

The proposed Lassen Substation will be located about 0.2 miles east of the West A Barr Road and South Old Stage Road in Mount Shasta, California (Latitude N 41° 18' 18" Longitude W 122° 19' 15"). The proposed site (504 South Old Stage Road, Mount Shasta, California) is currently occupied by a resident with landscaping consisting of grass, trees, shrubs, and gardens. Based on the available topographic information, the ground surface at the project site generally slopes slightly downward (<2.5%) to the west. The substation site is bounded by residential properties on the east and north, existing Mount Shasta Substation to the west (about 150 feet from the western property boundry), and South Old Stage Road about 370 feet to the south. It should be noted that PSI was informed by PacifiCorp that proposed Lassen Substation will not be adjoining to the existing Mount Shasta Substation. Drilled borings SB-1, SB-2, and SB-3 were located within the proposed substation area on the gravel driveway in order to not tear up current landscaping.

These exploration locations are shown overlain on an aerial photograph image on Figure A-2. Photographs of the exploration locations are located in the Appendix A.

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 3

3.2 Regional Geology

The proposed Lassen Substation is located in a large region known as the Cascade Range geomorphic province. This geomorphic province is characterized by a chain of volcanic cones, which extend from northern California into Oregon and Washington. The volcanic cones include both Mount Shasta and Lassen Peak, which is located at the southern terminus of the province (CGS, 2002). Lava flows and other volcanic deposits compose much of the surface materials in this province.

According to the 2010 Fault Activity Map of California there are Pre-Quaternary faults (older than 1.6 million years) within ten (10) miles of the subject site. In addition, an unnamed fault, which lies about 7 miles west of the site on Mt. Shasta, shows evidence of displacement sometime during the past 1.6 million years (CGS, 2010). No known faults pass through the project site. Earthquake design parameters including the Site Class are provided in Section 7.10 of this report.

The proposed Lassen Substation location is situated between Rainbow Ridge and Mount Shasta, approximately 2½ miles north of the Sacramento River. The site is located within an alluvial floodplain, near the southern end of Shasta Valley, consisting of several converging tributaries including Cold Creek. Our observations and analysis of readily available, pertinent geologic literature indicate that the subject site is underlain by Quaternary-aged alluvium (Strand, 1963). Alluvial soils generally consist of clay, silt, sand, and gravel that were deposited as sediment, likely derived from the surrounding volcanic peaks.

Mount Shasta, located about 10 miles northeast of the proposed Lassen Substation, is considered a dormant volcano, which will erupt again. On average the volcano has erupted once per 600 years during the last 4,500 years. Based on radiocarbon dating, the last eruption occurred about 200 years ago (Miller 1980). It is impossible to predict the date of next eruption, but it will likely occur within the next several hundred years. If Mount Shasta were to erupt, lava flows, pyroclastic flows, and mud flows could adversely affect the subject site, possibly to include destruction of Lassen Substation.

4.0 FIELD EXPLORATIONS

4.1 Soil Borings

Subsurface conditions at the project sites were evaluated with three (3) borings, designated as SB-1, SB-2, and SB-3 at the proposed Lassen substation at the approximate locations indicated on Figures A-2 in Appendix A. Termination depths of the borings varied between about 5 ½ feet and 20 feet below the ground surface after grinding on a cobbles or boulders for at least 20 minutes. Drillers were able to break through cobbles around 17 feet below the ground surface in boring SB-1.

The explorations were located on the site by a PSI representative using a hand held GPS device, with coordinates chosen based on the site plan sent by PacifiCorp.   The approximate locations of the explorations are summarized in the following Table 2.

Table 2: Soil Boring Locations at Proposed Lassen Substation

Exploration ID	Latitude/Longitude
SB-1	41.3047/-122.3209
SB-2	41.3051/-122.3209
SB-3	41.3052/-122.3209

The exploration borings were drilled to observe and document the stratigraphy, density, and variability of subsurface soil conditions and the thickness of the subsurface soil layers beneath the proposed structures. The borings were drilled using a CME 55 drill rig equipped with 8" Hollow-Stem Auger equipment. PSI used a backup drilling subcontractor attempting to meet deadlines during the field investigation that encountered numerous weather delays. If this site is chosen for Lassen Substation the follow-up field investigation could include an ODEX drilling system or possibly rotary wash methods to penetrate the cobbles and boulders and reach greater depths. Drilling and sampling were performed under the direction of a PSI Geotechnical Engineer who maintained detailed logs of the subsurface materials and conditions encountered in the borings, and collected representative samples.

Soil samples were obtained at about 2½ to 5 foot intervals. Soil samples were obtained by driving a standard 2-inch (O.D.) split-spoon sampler and 3-inch (O.D) Modified California ring sampler into the soil a distance of eighteen (18) inches using a 140-lb manual hammer dropped from a height of thirty (30) inches. The number of blows required to drive the split-spoon sampler the last twelve (12) inches is known as the standard penetration resistance value or N-value. The N-value for the Modified California rings was adjusted using input energy correction because blow counts relate energy input versus the area of the sampler barrel and sample. Standard penetration resistance values along with laboratory testing provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as clay or silt.

Soil samples obtained in the field were examined in the field and representative portions were stored in sealable plastic bags. The samples were transported to PSI's laboratory for further examination and testing. The borings were backfilled up to the ground surface with soil cuttings and on-site soils.

4.2 Refraction Microtremor

Four (4) ReMi® arrays were conducted at the West Point Substation to obtain a two-dimensional profile of average shearwave velocities. The ReMi® method uses standard seismic refraction equipment and records microtremors (or background noise) in the area as the source. Ambient background noise consisted of traffic along S Old Stage Road and Interstate I-5 located about 800 feet to the east. The background noise generates surface waves (including Rayleigh wave energy) that are detected and recorded by the twenty-four (24) channel geophone array.

The maximum depth of sampling using the ReMi® method is a function of the array length, as well as, the subsurface velocities. The ReMi® approach uses array lengths long enough to penetrate

the site to depths beyond one-hundred (100) feet depth. Typically the dynamic properties of the upper 100 feet control the near surface response to dynamic motion.

Once collected, the data from the arrays around the proposed site was checked for accuracy and fidelity. Multiple data samples were recorded for each array at the site. To assure a robust profile is being made, both individual recordings and multiple summed recordings were evaluated. The first step in data reduction was to produce a velocity spectrum of the recorded data. This process involves computing a surface wave phase velocity dispersion spectral ratio image using p-tau (slant spectra) and Fourier transforms across the array. This process is described in Louie, 2001. The resulting spectrum is in the slowness-frequency (p-f) domain. The p-f transformation helps segregate the Rayleigh wave arrivals from the other P and S seismic wave arrivals.

The normal mode dispersion can be distinguished from the aliasing and wave-field transformation truncation artifact trends in the spectra. Picking of the surface wave dispersion curve is done along the envelope of the lowest phase velocities. The data processing includes interactively forward modeling of the normal mode dispersion data using the picks from the p-f plots. The modeling process iterates on phase velocity at each period (or frequency), to provide a shear velocity profile as a function of depth beneath the site. The process and resulting velocity profiles are able to identify velocity inversions within the subsurface profile, which allow multiple subsurface soils or bedrock layers if encountered.

The ReMi tests were performed at the proposed substation area as indicated on Figure A-2 in Appendix A. ReMi results are shown in Figure D-1 to D-12 in Appendix D. In general, the ReMi shows lower velocity material (approximately 1000 ft/s) most likely consisting of silty sand and gravel to a depth of 15 to 20 feet. Below 20 feet there is a fairly consistent layer of much higher shearwave velocity in the range of 2000 to 2400 ft/s. These velocities are in the range that would be expected for very dense silty sand with gravel intermixed with cobbles and boulders. At a depth of 40 feet below the ground surface it appears that the soil profile decreases in shearwave velocity with a material change to a silty sand with gravel. Shearwave velocities generally exceeded 600 ft/sec, thereby indicating that liquefiable soils generally are not present at the proposed Lassen substation location. Section 7.10.1 further addresses the liquefaction potential. The velocity changes in ReMi® shearwave velocities generally agree with the changes encountered in the deepest boring SB-1.

4.3 Field Reconnaissance

A field reconnaissance was conducted by PSI near the proposed substation area to obtain information relative to surficial soils and potential rock outcrops. There appears to be very large boulder (8 feet diameter) along the western side of the proposed substation area. In addition, there is a well or sewage lagoon that may need to be abandoned or backfilled if the site is chosen for Lassen substation. Pictures of these are shown in Appendix A. The surficial soils visible at the ground surface consisted of topsoil with landscaped grass, trees, shrubs and gardens. There is also a gravel driveway located on the western side of the residential house.

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 6

5.0 LABORATORY TESTING PROGRAM

A laboratory-testing program which supplemented field exploration was conducted to evaluate additional engineering characteristics and soil index properties of the subsurface soils encountered that are considered necessary to analyze the behavior of the soils as it relates to the construction of the proposed substation construction. The laboratory testing program conducted is summarized as follows:

5.1 Laboratory Determination of Water (Moisture) Content of Soil by Mass

The water content is a significant index property used in establishing a correlation between soil behavior and its index properties. The water content is used in expressing the phase relationship of air, water, and solids in a given volume of material. In fine grained cohesive soils, the behavior of a given soil type often depends on its water content. The water content of a soil along with its liquid and plastic limits as determined by Atterberg Limit testing is used to express its relative consistency or liquidity index. Moisture content ranged from 31 to 34 percent in the near surface sandy silt. Silty sand with gravel had moisture contents ranging from 34 to 41%. A poorly graded sand with silt and gravel layer encountered in boring SB-1 with roughly equal amounts of sand and gravel (39% each) had a laboratory moisture content of 12%.

5.2 Atterberg Limits

The Atterberg Limits are defined by the liquid limit (LL) and plastic limit (PL) states of a given soil. These limits are used to determine the moisture content limits where the soil characteristics changes from behaving more like a fluid on the liquid limit end to where the soil behaves more like individual soil particles on the plastic limit end. The liquid limit is often used to indicate if a soil is a low or high plasticity soil. The plasticity index (PI) is difference between the liquid limit and the plastic limit. The plasticity index is used in conjunction with the liquid limit to assess if the material will behave like a silt or clay. The material can also be classified as an organic material by comparing the liquid limit of the natural material to the liquid limit of the sample after being oven-dried.

The soil profile consisted of a cohesive soil in the upper 2 ½ to 3 feet. Laboratory test results for this layer had a liquid limit of 37 percent and plastic limit of 30 percent and resulting plasticity indices of 7 percent. The fine grained material for the substation and the transmission line classified as a silt.

5.3 Grain Size Analysis

The purpose of determining the grain or particle size distribution of a sample is to classify and characterize the density of materials, determine the packing arrangement of the particles and estimate the shear strength and permeability of the soil matrix. To determine the grain size of coarse particles, sieves of varying screen opening sizes are used. In addition to classification, the grain size distribution is an important for use in filter design between two materials, estimating the permeability of a soil, and liquefaction and swell potential of a soil.

Laboratory testing consisted of four (4) analyses to measure grain size. Results show that the soil profile consisted of sandy silt, silty sand with gravel, and poorly graded sand with silt and gravel. A fourth test on select Modified California rings after the direct shear test classified the soil as a silty sand.

5.4 Direct Shear Testing

Direct shear testing provides shear strength parameters for the samples of surface soils. Results of this test can be used to determine ultimate shear resistance of soil. Therefore, multiple driven samples including 2.5 inch diameter brass ring sample specimens were obtained at different depth in the subsurface profile in the borings and were tested general accordance with ASTM D 3080 procedures. Many of the collected ring specimens were not suitable for Direct Shear testing because of soil particle size and size of testing equipment. Four rings were obtained at a depth of 10 feet in Boring SB-1. A sieve analysis classified the soil as a silty sand with some gravel (10%). About 20% of the particles were retained on the No. 10 Sieve (2.0 mm). These particle sizes are considered to be too large for the 2.42 inch I.D. soil ring sample. However, we decided to run the test on the in-situ soil while looking at the vertical displacement to make sure a piece of course sand or fine gravel was not on the shear plane. The stress failure envelope was defined at 4% strain and data is located in the appendix. Results of the direct shear testing are presented in Table 3 below.

Table 3: Soil Boring Locations at Proposed Lassen Substation

Location	Depth, ft	Friction Angle (deg), Area Uncorrected	Cohesion, psf, Area Uncorrected	Friction Angle (deg), Area Corrected	Cohesion, psf, Area Corrected
SB-1	10.0	46	270	47.5	280

In providing parameters with engineering judgment for this phase of the project we are providing a friction and of 46 degrees and 0 psf cohesion for this material.

5.5 Chemical Reactivity Testing

Select soil samples obtained in the subsurface profile in the borings were tested to evaluate the chemical reactivity of the on-site soils. Chemical reactivity tests of soil pH, resistivity, and water-soluble sulfate ion contents. Soil pH was performed using method SW9045D from USEPA SW-846 standard. Resistivity (A2510B) and water soluble sulfates (A4500-S04-E) methods came from "Standard Methods for the Examination of Water and Wastewater".

Select soil samples obtained of the subsurface profile was tested to evaluate the chemical reactivity of the on-site soils. Table 4 summarizes the chemical reactivity test results conducted on selected soil samples obtained from the borings:

Table 4: Summary of Chemical Reactivity Testing

Boring ID	Depth (feet)	Sulfates (mg/kg-dry = ppm)	Sulfate Exposure	Resistivity (ohm-cm)	Corrosivity Rating	Soil pH @ 25°C (-)
SB-1	5.0	<7.32	Negligible	14,800	Mildly Corrosive	6.8
SB-1	7.5	<7.08	Negligible	13,000	Mildly Corrosive	7.4

Test results indicate that the soil at the proposed substation and proposed transmission line have a soluble sulfate concentration of less than 7.32 ppm. Based on the American Concrete Institute (ACI) Building Code, these concentrations represent a negligible degree of sulfate attack potential on concrete structures shown in Table 5. For negligible to moderate concentrations of sulfates in the subsurface soils, Type I or 2 Portland Cement Concrete can be used in accordance with PacifiCorp design requirements for concrete elements in contact with the on-site soils or granular fill. PSI recommends that a qualified corrosion engineer evaluate the test results to determine the appropriate concrete type and mix design specifications for the project.

Table 5. Sulfate Exposure Categories

Severity	Class	Water-soluble sulfate (SO_4) in soil, percent by weight	Dissolved sulfate (SO_4) in water, ppm
Negligible	SO	$SO_4 < 0.10$	$SO_4 < 150$
Moderate	S1	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 < 1,500$
Severe	S2	$0.20 \leq SO_4 \leq 2.00$	$1,500 \leq SO_4 < 10,000$
Very Severe	S3	$SO_4 > 2.00$	$SO_4 > 10,000$

*Source: ACI, Building Code and Commentary (ACI 318-08)

Test results also indicate that the soil has a pH value of about 6.8 and 7.4 and laboratory resistivity values of about 13,000 to 14,800 ohm-cm (130-148 ohm-m). These soil pH and resistivity values may be used to predict the corrosion attack on the underground steel structures. Based on these test results, the on-site soils pose a mildly corrosive risk of corrosion attack on steel structures in the proposed Lassen substation, as suggested by Table 4. It should be noted that the results from the laboratory resistivity tests are typically lower than the field resistivity results because the laboratory samples are saturated. Field resistivity was not conducted during this phase of the geotechnical investigation of Lassen substation.

For protection against corrosion to buried metals, polyethylene encasement may be considered. Consideration may also be given to providing cathodic protection for buried metals. However, these

are simply suggestions, and PSI recommends that an experienced corrosion engineer be retained to design a suitable corrosion protection system for underground metal structures or components. Table 6 shows the corrosivity ratings based on laboratory soil resistivity.

Table 6. Soil Resistivity Corrosivity Ratings

Corrosivity Ratings Based on Soil Resistivity*

Soil Resistivity (Ω -cm)	Corrosivity Rating
> 20,000	Non-corrosive
10,000 - 20,000	Mildly Corrosive
5,000 - 10,000	Moderately Corrosive
3,000 - 5,000	Corrosive
1,000 - 3,000	Highly Corrosive
< 1,000	Extremely Corrosive

* Source: ASTM STP 1013

6.0 SUBSURFACE CONDITIONS

6.1 Soils

6.1.1 Lassen Substation

Subsurface conditions within the proposed substation during this phase of investigation consisted of one (1) to two (2) inches of gravel roadway underlain by two (2) to three (3) feet of soft to stiff sandy silt. The sandy silt is underlain by medium dense to very dense silty sand with gravel and poorly graded sand with silt and gravel with cobbles and boulders resulting in auger refusal at depths ranging from 5 feet (northern end of site) to 20 feet (southern part of site) below the ground surface. Bedrock was not encountered during our field investigation at Lassen Substation. As our access was limited to the gravel roadway, we were unable to make a determination of topsoil depth at the site.

Standard penetration resistance values ranged from 3 blows per foot to 13 blows per foot in the upper native fine grained soils. Standard penetration resistance values ranged from about 3 blows per foot to about 27 blows per foot in the lower native coarse grained soils. It should be noted that the 3 blows per foot encountered in boring SB-1 at a depth of 2.5 feet most likely was affected by the groundwater interface and is probably not indicative of the true apparent density. The blows per foot increased to well over 50 per foot when cobbles and boulders were encountered. For a detailed description of the materials and conditions encountered at boring location within the proposed substation area, please refer to Figure B-1 through B-3 in Appendix B.

6.2 Free Groundwater

Free groundwater (interpreted as groundwater that freely flows into a borehole) was encountered in the drilled borings at the proposed Lassen substation during our field investigation. The results of the subsurface free water measurements are shown in Table 7. It should be noted that laboratory testing shows saturated soil condition at a depth of 1 to 2 ½ feet below the ground surface.

Table 7: Groundwater Levels during field exploration.

Boring ID	During Drilling	After Drilling
SB-1	1 to 2.5 ft	4.0 ft
SB-2	1 to 2.5 ft	4.0 ft
SB-3	1 to 2.5 ft	5.0 ft

Water level observations do not suggest flowing artesian conditions, i.e. pressure which raises the groundwater table above the ground surface at depths obtained in this phase of drilling. PSI recommends installing a vibrating wire piezometer to monitor groundwater over time.

It should be noted that it is possible for the groundwater levels to fluctuate during the year depending upon climatic and other factors. Additionally, discontinuous zones of perched water may exist at varying locations and depths beneath the ground surface. As a result, groundwater conditions during or after construction may be different than those observed during the field investigation.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Geotechnical Discussion

The following geotechnical-related recommendations have been developed based on the predominantly granular subsurface soil conditions encountered in the borings and PSI's understanding of the proposed construction at Lassen substation. In PSI's opinion, the proposed site is suitable for further consideration of constructing proposed substation structures, if the preliminary in nature geotechnical engineering recommendations in this report are followed. The primary geotechnical considerations with respect to the proposed construction of Lassen substation include the following.

- Shallow groundwater was encountered roughly 2 ½ to 5 feet below the ground surface. This could impact both shallow foundation excavations and drilling of pier foundations.

- Foundation subgrade preparation for disturbed soil near the groundwater elevation

- Silty subgrade soils may be sensitive to changes in moisture content. Stabilization measures may be required during site grading.

- Potential caving of deeper excavations in granular soils

- Isolated boulders and cobbles of unknown dimensions encountered in the upper 40 feet (estimated from the ReMi®) below ground surface. This will likely increase the difficulty of installing drilled pier foundations.

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 11

- Surface drainage away from constructed foundations

Further details are provided in the following sections of the report.

7.2 Site Preparation, Earthwork, and Demolition for Lassen Substation

7.2.1 Site Preparation and Earthwork

PSI recommends that the ground surface within the proposed substation area and other areas to receive structural fill be cleared of a residential house, garage, and sheds, topsoil, organics, and other unsuitable or deleterious material. During this phase of field investigation we only drilled on the gravel roadway. We anticipate that excavations to depths of about six (6) to twelve (12) inches will be required within the proposed substation site area to remove topsoil, organics, and other unsuitable material; however, borings or test pits in phase 2 subsurface investigation should be used to verify this. Deeper or shallower excavations may be required locally.

Upon completion of stripping the site and preparing foundation excavations, the exposed subgrade should be evaluated by the Geotechnical Engineer. Proof rolling with construction equipment may be a part of this evaluation. Subgrade soils that are observed to rut or deflect excessively (typically greater than 1-inch) under the moving load of a loaded rubber-tired dump truck (typically 50 –ton) or other suitable rubber-tired construction vehicle should be over-excavated to firm undisturbed native soils and backfilled with properly placed and compacted structural fill.

If the subgrade is disturbed during construction, loose, disturbed soils should be over-excavated to firm, undisturbed soil and backfilled with compacted granular materials as outlined in Section 7.4. PSI recommends that site preparation, earthwork, and pavement subgrade preparation be accomplished during warmer, drier months, typically extending from mid-May to mid-October of the year. Modifications to the grading plans should be reviewed by the Geotechnical Engineer.

7.2.2 Demolition and Backfill

If this site is chosen for the proposed Lassen Substation, a residential house in addition to a three car garage and sheds will need to be demolished. It appears that the house is supported on a concrete slab, which should be excavated and removed from the site. Excess material and concrete should be disposed of off-site according to prevailing laws, ordinances, regulations, and rules. Excavations should be backfilled with "Road Finish Rock" (Table 10) and compacted according to recommendations in Section 7.5 of this report.

7.3 Excavation Consideration

Excavations for foundations, or utility trenches within the proposed Lassen substation areas should be performed in accordance with OSHA regulations as stated in 29 CFR Part 1926. The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor should evaluate the soil exposed in the excavations as part of the required safety procedures. In no case should slope height, slope inclination, or

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 12

excavation depth, including utility trench excavation depth, exceed those specified by local, state, and federal safety regulations. PSI is providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

During wet weather, earthen berms or other methods should be used to prevent runoff water from entering the excavations. The bottom of the excavations should be sloped to a collection point. Collected water within the foundation and utility trench excavations should be discharged to a suitable location outside the construction limits. At the proposed Lassen substation the easiest area for discharge would be the southwest corner because of its lower elevation.

7.4 Fill Materials (General Site Grading-RG-2 and Aggregate Base Course)

Based on the results from the field and laboratory investigation, near surface on-site soils at the proposed Lassen substation contain significant amounts of silt, such that they are generally unsuitable for use as General Site Grading RG-2 or Aggregate Base Course materials but may be used in site grading or landscape areas outside structure limits. Imported structural fill should consist of well-graded sand and gravel materials that are free of organic or other deleterious materials. Imported fill material should be approved by the Geotechnical Engineer prior to its delivery to the project site. Structural fill material to be used for site grading within the substation pad area and other structural areas (RG-2) should meet the grading specifications presented in Table 8 below.

Table 8: Structural Fill Gradation Requirements (RG-2)

Sieve Size	Percent Passing by Weight
3 inch	80 - 100
¾ inch	70 - 90
No. 4	40 - 60
No. 40	20 – 40
No. 200	0 - 15
Liquid Limit (LL)	≤ 35
Plasticity Index (PI)	15 (Max.) - 4 (Min.)

Structural fill material to be used as aggregate base course within the substation pad area and other structural areas along with Road Finish Rock should meet the following specifications in Tables 9 and 10:

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 13

Table 9: Structural Fill Gradation Requirements (Aggregate Base Course)

Sieve Size	Percent Passing by Weight
3 inch	100
2 ½ inch	85 - 100
1 ¼ inch	55-75
¾ inch	>70
3/8 inch	30-45
No. 10	15-25
No 40	10-20
No 200	0-7

Table 10: Structural Fill Gradation Requirements (Road Finish Rock)

Sieve Size	Percent Passing by Weight
1 ½ inch	100
1 inch	95-100
3/4 inch	80-95
No. 4	40-55
No 16	20-30
No 200	0-7

Recommended gradations for Yard Rock are included in Section 7.11 of this report.

7.5 Compaction

Structural fill materials should be moisture conditioned to two (2) percent below optimum to two (2) percent above optimum moisture content. Structural fill should be placed in loose lifts not exceeding nine (9) inches thick for self-propelled compaction equipment and six (6) inches thick for hand-guided compaction equipment and compacted to at least 95 percent of the maximum dry density as determined by the ASTM D 1557 Test Method. Site grading fill or backfill placed beneath floor slabs or flat work should be compacted to at least 95 percent of the maximum dry density as determined using ASTM D 1557.

Placement and compaction of the fill materials should be observed, tested, and documented by a representative of the Geotechnical Engineer. Tested fill materials that do not achieve either the required dry density or moisture content requirements should be recorded, the location noted, and reported to the contractor and owner. A re-test of that area should be performed after the contractor has performed all necessary remedial measures including moisture conditioning (wetting or drying) and reworking the fill. Please note that foundation subgrade stabilization as described in Section 7.2 may be required before placement of fill. The Geotechnical Engineer should be retained to observe site stabilization, if required

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 14

7.6 Surface Drainage and Erosion Control

The potential for soil erosion is largely impacted by local soil characteristics, vegetative cover, topographic relief, and the frequency and intensity of rainfall and wind. Removal of vegetation and/or disturbance to surficial soil by construction activities may result in local increases of erosion rates in unprotected areas. As a result, sedimentation may increase in local drainages at site perimeters and slope intersections. Uncontrolled diversion of storm water runoff from the site to unlined drainage channels could result in erosion of the drainage channels due to concentrated flow. This is particularly true during and immediately following site grading.

PSI recommends the following surface drainage and erosion control practices be incorporated in this project:

- Control surface runoff from disturbed land on the project. Design and prepare drainage ways to divert and control concentrated runoff from disturbed areas by using rip-rap or other lining materials to control erosion;

- Trap sediment-laden runoff in basins to allow soil particles to settle out before flows are released from the site;

- Reduce erosion by limiting the area of exposed soil and time of exposure to wind and weather, and by the provision of diversion channels;

- Use temporary plant cover, mulching, and/or structures to control runoff and protect areas subject to erosion during construction;

- Minimize soil exposure during wet weather by proper timing of grading and construction and be prepared to shut down all earthworks if heavy precipitation occurs;

- Have erosion control equipment and materials on-site if needed in an emergency to quickly construct temporary collectors, diversion channels, intercept drains, berms, dikes, or filters after major precipitation events;

To reduce soil erosion and sediment transport, protective material such as gravel, crushed stone, pavement, and other effective erosion control materials should be used to stabilize exposed soil. Slopes should be protected with temporary drainage and erosion control measures during construction until permanent measures can be installed. Storm water runoff from construction areas should be conveyed to temporary dike detention areas for sediment deposition, then discharged to a suitable location with velocities slow enough to prevent further erosion in the drainage courses.

Control of erosion and sedimentation on recently graded construction sites may require both vegetative and structural measures. Vegetative species used to control erosion should be selected in accordance with the soil characteristics and climate at the site. Storm runoff control should be provided during and after completion of site grading by using diversion dikes and permanent drainage facilities. Sediment retention structures such as sediment basins, sediment traps, silt

fences should be used to keep eroded material from the site. Sediment control should be properly installed and maintained to ensure their desired performance level and reliability.

Site grading should be carefully planned to promote positive drainage away from the structures and to divert surface water away from the site. Landscaping irrigated areas should be kept as far from the proposed transformer pads or structures as possible. Drip irrigation and landscaping with low water usage landscaping is encouraged. Positive site drainage away from the proposed transformer pad areas and pavement sub-grades should be established during and after construction. Water should not be allowed to collect near the foundations, transformer pad areas or in pavement areas either during or after construction. Undercut or excavated areas should be sloped towards one corner to facilitate removal of any collected rainwater, groundwater seepage, or surface runoff.

7.7 Shallow Foundations

During our preliminary site investigation at a new Lassen substation the proposed transformers, circuit breakers, and control house within the proposed Lassen substation can be supported on shallow foundations. Loose or otherwise disturbed soils are not suitable for supporting foundations and slabs. Loose and/or disturbed soils should be removed down to firm, undisturbed soils and replaced with properly placed and compacted structural fill. The following design parameters in Table 11 are recommended for foundations or slab design and construction.

Table 11: Allowable Bearing Pressures at Proposed Lassen Substation

Footing Width, B (feet)	Allowable Bearing Pressure, q_a (psf)
B ≤ 2	1500
2 <B ≤ 3	1600
3 <B ≤ 4	1800
4 ≥ B	2000

The recommended allowable bearing pressure refers to the total dead load and can be increased by 1/3 to include the sum of all loads including wind and seismic.

- Footings should bear at a minimum depth of 12 inches below final grade for frost protection according to the Sisikyou County Building Department. For non-frost areas, such as interior footings, a minimum embedment depth of 6 inches is recommended. Non frost-susceptible soils such as free draining gravel may be used to reduce the depth of concrete footings.

- Crushed angular gravel with less than 5% passing No. 200 Sieve can be placed at the bottom of the excavation to help in achieving compaction near the groundwater

Preliminary Geotechnical Engineering Report   March 11, 2011
Proposed Lassen Substation   PSI Job No. 0595148
Mount Shasta, California   Page 16

surface.

- If the foundation dimensions provided in the table are different, then PSI should be contacted to re-evaluate the bearing capacity recommendations provided above.

- Structural fill, if required, should extend laterally a minimum of ½ the depth of foundation element beyond the outside edge of the foundation.

- PSI recommends that the foundations be designed in accordance with appropriate IEEE, ASCE, ACI standards.

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of spread footings and the underlying soil. The total frictional resistance between the footing and the soil is the sum of vertical forces (dead load) times the coefficient of friction between the soil and the base of the footing. PSI recommends a value of 0.45 for concrete placed on granular native soils, or properly placed and compacted granular structural fill. If additional lateral resistance is required for shallow foundations, passive soil resistance from embedded foundations may be evaluated on the basis of an equivalent fluid having a unit weight of 250 pcf to a depth of 15 feet. Lateral forces below 15 feet shall be examined for specific structures. If required, additional loads i.e. seismic lateral loads may be provided upon request.

For subgrade prepared as recommended and properly compacted fill, a modulus of subgrade reaction, *k* value, of 200 pounds per cubic inch (pci) may be used in the grade slab design based on values typically obtained from 1 ft. x 1 ft. plate load tests. However, depending on how the slab load is applied, the value will have to be geometrically modified. The value should be adjusted for larger areas using the following expression for cohesive and cohesionless soil:

Modulus of Subgrade Reaction, $k_s = \left(\dfrac{k}{B} \right)$ for cohesive soil and

$$k_s = k \left(\dfrac{B+1}{2B} \right)^2 \text{ for cohesionless soil}$$

where: k_s = coefficient of vertical subgrade reaction for loaded area,
                   k = coefficient of vertical subgrade reaction for 1x1 square foot area, and
                  B = width of area loaded, in feet

PSI recommends that the footing excavations be observed and documented by PSI's Geotechnical Engineer or designated technical representative prior to placement of structural fill, concrete or reinforcing steel to verify their suitability for foundation support.

7.7.1 Foundation Settlements

Total settlement of an individual shallow foundation will vary depending on the plan dimensions of the foundations and the actual load supported. For footings designed according to the recommendations described in Sections 7.7 above, under the static loading conditions, is not

expected to exceed one (1) inch. Differential settlement is expected to be about ½ to ¾ of the total settlement under static conditions.

7.8 Deep Foundations

PSI anticipates that drilled pier foundations will generally be considered to support heavier substation structures and dead-end transmission line tower structures. However, based on our subsurface exploration, constructability of drilled piers may be difficult in some areas of the substation due to the presence of groundwater levels which will affect the drilled pier construction. In addition, difficulties with drilled pier construction may be encountered due to the possibility of caving granular soils along with cobbles and boulders at varying depths throughout the site. Note that auger (8 – inch diameter) refusal on cobbles and boulders were encountered at depths between about 5 ½ and 7 ½ feet in borings SB-2 and SB-3. Drilled pier construction will require the judicious use of casing, drilling fluid, and other careful construction strategy and methodology to avoid disturbance to the supporting soil profile and to achieve the desired depths, geometry and construction specifications of the designed foundations. PSI has provided recommendations for drilled piers and also recommendations for alternate foundations for consideration.

7.8.1 Drilled Pier Foundations

Drilled pier foundations consist of an augured shaft having typical diameters ranging from about three (3) to more than ten (10) feet that is drilled to a design depth and filled with reinforced concrete. The axial load carrying capacity of a drilled pier can be computed using the static method of analysis. According to this method, axial capacity, Q, at a given penetration is taken as the sum of the skin friction on the side of the shaft, Q_s, and the end or point bearing at the shaft tip, Q_p, so that:

$$Q = Q_s + Q_p = fA_s + qA_p$$

Where A_s and A_p represent, respectively, the embedded surface area and the end area of the shaft; f and q represent, respectively, the unit skin friction and the unit end or point bearing.

Drilled pier foundations may be used to support the proposed tower. PSI anticipates that the computer program CAISSONS by Powerline Systems, Inc. or L-PILE by Ensoft, Inc. may be used to estimate the lateral capacity of drilled pier foundations.

Based on the subsurface conditions encountered during our field investigation, the soil parameters presented on the boring logs in the Appendix have been compiled to aid in evaluating the vertical and lateral capacity of drilled pier foundations.

7.8.1.1 Drilled Pier Design Parameters

Design parameters related to drilled pier foundations have been prepared based on the field, laboratory, and engineering judgment for each boring location. In general, the design parameters were developed from the establishment of the following for each material type.

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 18

- Density

- Friction angle

- Cohesion

Therefore, these three physical parameters were established first and the design parameters were developed secondary to these. The following paragraphs summarize the development and recommended use of the physical and design parameters.

Design Unit Weight

The term "Design Unit Weight" represents PSI's recommended parameter for use in the design process. Above the water table, this term represents the total moist unit weight for the designated material. Below the water table, this term represents the effective unit weight as defined as the saturated unit weight minus the unit weight of water. The elevation or depth to the "design water table" is PSI's recommended depth to groundwater based on our assessment of the field information.

<u>Unit Weight Evaluation of Sands (Cohesionless Soils)</u>

The dry unit weight evaluation of the sands, silty sands, sandy silts, sandy gravels and gravels were based upon an assessment of the relative density of the cohesionless soils based on the relationship developed by Holtz and Gibbs, 1979, that relates the standard penetration test (SPT) to the relative density (D_r) based on the overburden pressures. Once the relative density (D_r) was assessed for the cohesionless materials, the void ratio (e) was derived based on maximum and minimum void ratio values typical for the gradation classification of the cohesionless soil. The typical ranges were obtained from Kulhawy and Mayne, 1990, then were slightly lowered based on local experience, for the following gradations of cohesionless soils shown in Table 12 below.

Table 12: Typical Soil Unit Weights

Soil Description	Minimum Dry Unit Weight (pcf)	Maximum Dry Unit Weight (pcf)	Maximum Void Ratio (e_{max})	Minimum Void Ratio (e_{min})
Silty Sand and Gravel	89	140	0.85	0.18
Clean, Poorly graded Sand	85	138	1.00	0.40
Silty Sand	87	127	0.90	0.30
Gravel	92	110	0.8	0.5

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 19

The void ratio (e) for the soil was calculated with the following relationship:

$$e = e_{max} - D_r * (e_{max} - e_{min})$$

Once the void ratio for the soil layer was determined the dry unit weight (γ_{dry}) was calculated with the following relationship:

$$\gamma_{dry} = \left(\frac{1}{(1+e)}\right) * SpGr * 62.4$$

A specific gravity (SpGr) of 2.65 was used for this project.

The design unit weights (γ_{Design}, and γ'_{Design}) were calculated in the following manner:
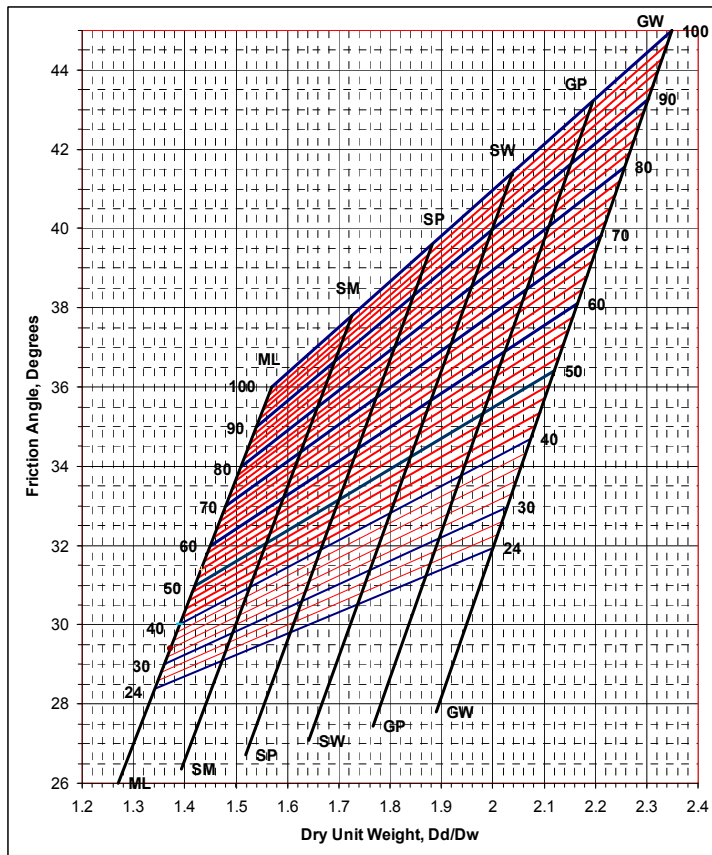
Above the water table:

$$\gamma_{Design} = \gamma_{dry} * (1 + w_n)$$

Below the water table:

$$\gamma_{Design} = (\gamma_{dry} * (1 + w_n)) - 62.4$$

Where w_n is the saturated moisture content.

<u>Unit Weight Evaluation of Clays (Cohesive Soils)</u>

The dry unit weight evaluation of the sandy silt encountered in the upper 3 feet of boring SB-1, SB-2, and SB-3 was based on an evaluation of the laboratory measured natural moisture content (w_n), and an assigned degree of saturation where a relatively undisturbed sample was not available. These cohesive soils encountered down to 3 feet were assigned saturation levels of 100%. Based on these relationships, the dry unit weight (γ_{dry}) was calculated as follows:

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 20

$$\gamma_{dry} = \frac{62.4 * Sat.}{\left(w_n + \dfrac{Sat.}{SpGr} \right)}$$

The design unit weights for the cohesive soils are calculated the same way as for the cohesionless soils listed above.

Friction Angle for Cohesionless Soils:

The friction angle for the cohesionless soils was derived from direct shear laboratory testing in conjunction the relationship given by NAVFAC DM 7.1 and Kulhawy and Mayne, 1990, where dry unit weight is normalized by the unit weight of water, then the friction angle is determined based on the relative density derived from the SPT testing as illustrated here. The resulting friction angles were rounded to the nearest 0.5º.

The N values were truncated at a value of 50 because it is believed that the reliability of N values over 50 decreases. As a result, there appears to be some reduction in strength with depth that is more closely associated with the limits of the field data rather than the in-situ soil conditions. Using this graph also assisted in assessing the consistency of the material descriptions. It was possible to compare the combined normalized dry unit weight and and relative density plot to the sample description and the plotted classification. This allowed the ability to check the consistency of the design processes. The friction angles derived from this graph were also used in the calculation of the Rankine active and passive earth pressures for the cohesionless materials encountered on this project. Some engineering judgment was used in the compiling of the friction angles used in the design parameter recommendations.

Friction Angle for Cohesive Soils:

Only the upper 3 feet of the soil profile in borings SB-1, SB-2, and SB-3 encountered fine graine sandy silt soil. The shear strength parameters for drilled piers are typically total stress parameter in which the cohesive soils would be represented by only a cohesion term and no friction angle. However, to provide Rankine active and passive lateral earth pressures, a residual drained (or effective) friction angle was derived for the cohesive soils based on liquid limits for an inplace or normally consolidated clay. This is believed to be a conservative estimate because it is a residual angle derived from large strains. The relationship used for residual friction angle in cohesive soils comes from Gibson, 1953. The maximum residual friction angle was capped at 35º in this relationship.

Cohesion for Clay (cohesive) Soils

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 21

The cohesion, which is defined as the total undrained shear strength, was derived from the SPT

results as modified by engineering judgment. Generally, the relationship between SPT and cohesion is a lower bound derivation from the relationship developed by Sowers, 1979. The relationship used in this project was derived from the lower limits of the SC-ML range as shown here. Cohesion terms were not used for the granular defined soils.
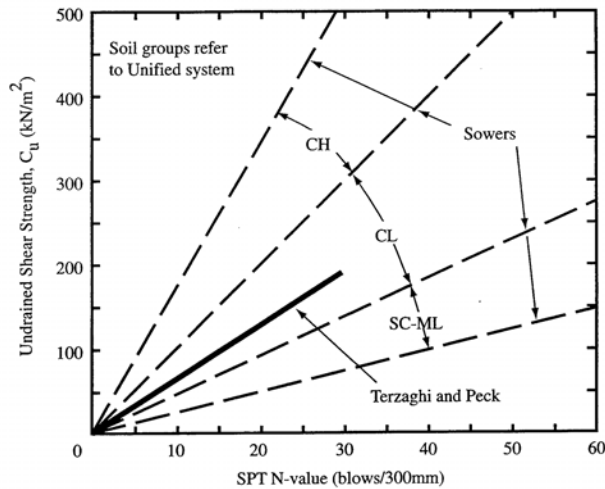
Ultimate End Bearing Capacity and Ultimate Skin Friction:

The end bearing capacities and skin friction capacities given in the drilled pier design parameters represent unfactored, ultimate strength capacities. No factor of safety has been applied to these values. The unfactored value is given so that the term can be used for either an ASD or LRFD approach to the design process. It is recommended that a higher factor of safety be used for the end bearing than the skin friction in cases where the piers do not extend to bedrock. In this case, it is recommended that a factor of safety of 3 be used for end bearing in combination with a factor of safety of 2 for the skin resistance.

The bearing capacity values should be used in consideration with the thickness of the bearing materials. There should be at least 3 diameters of the same bearing soil under the end of the drilled pier to use the design values given in this report. Where the bearing values within 3 diameters below the pier are less than what is immediately below the pier, a weighted average of the bearing capacities should be used. Where the values increase, it is recommended that the lower bearing capacity be used.

Table 13, below, lists the bearing capacity and skin friction relationships recommended for cohesive (upper 3 feet of Lassen substation) and cohesionless materials.

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 22

Table 13: Bearing Capacity and Skin Friction Calculations

Material Type	Location	Relationship	Source
Sands (cohesionless)	End Bearing	$Q_{ult} = 1.4 * N * 1000$ (psf)	Reese and O'Neill (FHWA)
	Skin Friction	$f_{s-ult} = \beta * \sigma_{vo}'$ $where$ $\beta = 1.5 - 0.135 * \sqrt{Depth}$ $\sigma_{vo}' = OverburdenStess$	Reese and O'Neill (FHWA)
Clays (cohesive)	Skin Friction	$f_{s-ult} = 0.55 * c$	Foundation Analysis and Design, Bowles, 1996

Lateral Earth Pressures:

The lateral earth pressures have been derived from the Rankine solution for active and passive earth pressures. The following gives the Rankine expressions for both the active and passive earth components:

$$K_A = Tan^2\left(45 - \frac{\phi}{2}\right)$$

$$K_P = Tan^2\left(45 + \frac{\phi}{2}\right)$$

In both of these cases the friction angle (ϕ), is the term generated as described above.

Horizontal Subgrade Modulus

The horizontal subgrade modulus has been determined based on the graphical representation of the soil type as given by Reese for L-Pile or COM624P. The cohesive terms are derived specifically from the undrained shear strength or c value and the cohesionless terms from the SPT "N" values and gradations.

Drilled Pier Construction Considerations:

PSI recommends that the drilling contractor review the boring logs of this report before starting excavations for the drilled piers. Please note that flowing sand conditions and diffucult drilling in

boulders and cobbles may be encountered at some locations at Lassen substation. The free water level at the substation should be considered variable and subject to fluctuation due to changes in climatic conditions, local irrigation practices and other factors. It should be expected that the advancement of casing and placement of concrete will be more difficult and special measures will be required where drilled piers are installed below the observed free water level. PSI anticipates it may be necessary to utilize a temporary steel casing and drilling fluid to support the walls of the drilled pier excavation during drilled pier construction. Ideally, the casing should be driven through the loose silty sand material into the medium dense silty sand with gravel a sufficient distance to mitigate heaving conditions. PSI recommends that an experienced drilling fluid Engineer be consulted to properly design a suitable weighted drilling fluid system to counter the anticipated hydrostatic and construction conditions at the site. Weighted drilling fluid where applicable should be used continuously from the start of excavation through concrete placement in drilled pier excavations. A representative of the Geotechnical Engineer should be on site to observe and document the installation of the deep foundation system. Based on PSI experience, even after using the drilling mud to drill holes for the piers, we anticipate that it may be difficult to install drilled piers due to potential caving, flowing sands, or large cobbles and boulders. Alternate foundation types may be considered to avoid some of these installation difficulties as discussed in section 7.9 of this report.

When the drilling processes are completed for the pier, the reinforcing steel and the concrete should be placed immediately. During simultaneous concrete placement and casing removal operations, sufficient concrete should be maintained inside the casing to offset the hydrostatic head of the ground water outside the casing and minimize the intrusion of soil into the pier concrete.

Concrete placed in the pier excavations should have a slump in the range of 7 to 9-inches for wet excavations to reduce the potential for the formation of voids as the temporary pier casing is extracted. The concrete mix should be designed to attain the required 28-day design strength when placed at this slump. PSI should be retained to observe and document the drilled pier construction and to evaluate whether the subsurface and pier bearing conditions are as anticipated in this report. The contractor should submit their procedures for drilled pier installation to the Geotechnical Engineer for approval prior to the start of the drilled pier construction.

Consideration should be given to non destructive testing of the drilled piers, including Cross Hole Sonic Logging (CSL). Access tubes for CSL testing should be considered in construction of the drilled pier reinforcing cage to help insure quality built drilled pier. These tubes allow post construction cross-hole sonic logging that will indicate the presence of either sound concrete or defective concrete. One tube per foot of shaft diameter should be placed around the inside

circumference of the rebar cage. Tubes should be made from Schedule 40 steel and filled with water soon after the concrete is placed.

7.8.2 Lateral Load Analysis

For the deep foundation types listed above, lateral loads are resisted by the surrounding soil profile and the internal resistance or strength of the foundation element. The 'p-y' parameters used in the LPILE analyses are provided on the boring logs in the Appendix.

Please note that the soil strength parameters including earth pressure and skin friction design parameters provided on the boring logs are ultimate values. Lateral resistance values should be neglected within 2 feet (24 inches) of the ground surface due to disturbance associated with frost action, construction activities, and movement of the drilled piers due to wind loads and other axial loads over time.

7.9 Alternate Foundation Recommendations

7.9.1 Pad and Pedestal Footing

As an alternative, a pad and pedestal type foundation may be considered to support all substation structures inside the proposed substation. This foundation option may be considered to bypass potential excavation difficulties due to groundwater and caving granular soils in some areas, and provides for over-excavation of potentially liquefiable materials encountered in the upper soil profile. A pad and pedestal foundation utilizes the dead weight of the concrete spread footing and soil backfill to resist lateral loads. The following design parameters are recommended for footing and anchor block design and construction:

Foundations bearing on undisturbed native silty sand with gravel soil 3 to 10 feet below the existing ground surface during the preliminary investigation at Lassen substation may be designed using the design parameters based on depth in the Table 14 assuming a width of 5 feet.

Table 14: Ultimate Soil Bearing Capacities for Pad and Pedestal Foundations at Lassen Substation.

Footing Width, B (feet)	Ultimate Bearing Pressure, q_a (psf)
$3 < D \leq 4$	6,500
$4 < D \leq 5$	7,700
$5 < D \leq 6$	9,000
$D \geq 6$	10,200

Allowable bearing capacities for pad and pedestal foundations may be obtained by dividing the values by a factor of 3.0.

Excavation considerations for pad and pedestal foundation alternatives include the potential for shallow groundwater levels and the subsequent need for dewatering and/or subgrade stabilization. Dewatering options are explained in Section 7.9.1.2.

The recommended allowable bearing pressure refers to the total dead load and can be increased by 1/3 to include the sum of all loads including wind and seismic.

Unit weights of 120 and 150 pounds per cubic foot (pcf) can be used to calculate the weight of soil backfill and concrete, respectively for uplift and lateral force resistance. These values should be reduced by 62.4 pcf if these materials are below the free water level. The volume of soil should be calculated by extending a vertical line upward from the outside edge of the foundation. Additional lateral resistance may be achieved through passive soil resistance against the side of the anchor block. A design passive earth pressure based on an equivalent fluid pressure having a unit weight of 225 pcf can be used to calculate lateral resistance for granular backfill compacted to at least 95% of the maximum dry density of ASTM D 1557 (Modified Proctor) adjacent to the concrete anchor block.

Structural fill, if needed, should extend a minimum of ½ the depth of fill laterally beyond the outside edge of the foundation.

PSI recommends that the foundations be designed in accordance with the California Building Code (CBC), 2010 edition.

For footings designed according to the recommendations described above, we estimate total settlement due to loads under static conditions will not exceed one (1) inch. Differential settlements beneath pad and pedestal foundations may be on the order of approximately ¼ to ½ inch. PSI should be allowed to review actual plans for pad and pedestal foundations including depth below existing ground surface and soil pressures to verify our assumptions used in these recommendations.

7.9.1.2 Dewatering Considerations

There are four basic methods for controlling groundwater on a construction project:

1. Permit the flow of water into an excavation, collect it in ditches and sumps, and pump it from there out of the excavation,
2. Predrain the soils before excavation begins by using pumped wells, wellpoints, ejectors, or drains,
3. Cut off the inflow of groundwater with steel sheet piling, diaphragm walls, ground freezing, tremie seals, or grout, or
4. Exclude the groundwater with compressed air, a slurry shield or an earth pressure shield.

To make a proper selection, the dewatering designer needs to have the following information on the many factors affecting the excavation:

1. The porosity, permeability, density and strength of the soils,
2. The groundwater hydrology including source, recharge, and areal extent of the aquifer,
3. Size and depth of the excavation,
4. Method of excavation and ground support systems,
5. Proximity of the excavation and dewatering to existing structures and their foundation systems,
6. Design and function of the structure to be built,
7. Schedule of construction, and
8. Existence and nature of contaminations at the site.

This report only concerns the dewatering method of permitting the flow of water into an excavation, collecting it in ditches and sumps, and pumping it from there out of the excavation. The key is to identify those conditions that are or are not favorable for open pumping and recognizing which conditions predominate in a given job situation. A decision to proceed by open pumping should be reached with a thorough knowledge of the job situation as generally described below:

Conditions favorable to Open Pumping:

- Soil characteristics
    o Dense, well-graded granular soils, especially those with some degree of cementation or cohesive binder.
    o Firm clays with no more than a few lenses of sand, which are not connected to a significant water source.
    o Hard fissured rock.
- Hydrology Characteristics
    o Low to moderate dewatering head
    o Remote source of recharge
    o Low to moderate permeability
    o Minor storage depletion
- Excavation Methods
    o Dragline
    o Backhoe
    o Clamshell
- Excavation Support
    o Steel sheetpile sheeting
    o Slurry concrete walls
- Miscellaneous
    o Open unobstructed site,
    o Large excavations,
    o Light foundation loads in surrounding areas.

Conversely, conditions that are unfavorable to open pumping which will typically require a predrainage or cutoff system generally take the form of the following and which are outside of the scope of this report:

- Soil characteristics
    - Loose, uniform granular soils without plastic fines
    - Soft granular silts, and clays with moisture contents near or above the liquid limit
    - Soft rock, rock with large fissures filled with soft materials or soluble precipitates, sandstone with uncemented sand layers
- Hydrology Characteristics
    - Moderate to high dewatering head
    - Close proximity to source of recharge
    - Moderate to High permeability
    - Large quantities of storage groundwater
    - Artesian pressure below subgrade
- Excavation Methods
    - Scrapers
    - Loaders and trucks (equipment that "pump" the soils)
- Excavation Support
    - Steep slopes
    - Soldier Beam and lagging
- Miscellaneous
    - Close adjacent structures
    - Small confined excavations
    - Heavy foundation loads

The installation of an open pumping system may need to be staged to avoid boils and blows in the base of the excavation resulting from too high of a groundwater table near the excavation. This being said, the sump pit for the pumping of open trenches should generally have the following characteristics:

- The sump must be excavated deep enough to drain the excavation. This may require multiple pump pits or points,
- The water flowing towards the sumps will likely carry fines, however, the amount of fines should me minimal or the excavation may become unstable. The approaches to the sumps should be paved with gravel to reduce the fines by sedimentation and filtrations.
- The size of the sump should be substantially larger than that necessary to physically accommodate the pumps. Ample size allow for a reduction in water velocity so that fines settle out and the space provides for storage for the sediment between cleanings.
- The sump should be arranged for convenient servicing of the pumps and so that accumulated sediments can be readily removed.

Based on the limited investigation of this preliminary report, a predrainage or cutoff system may be necessary for Lassen Substation. Granular soils encountered in SB-1 were loose and the granular silts were soft near the ground surface with moisture contents near the liquid limit. In addition, structures inside substations are often close in proximity to confined excavations.

7.9.2 Auger-Cast-in-Place Piles (ACIP)

Auger cast-in-place (ACIP) piles are typically slender foundation elements constructed by auger drilling to the design depth and placing concrete grout though the augers as they are removed. Steel reinforcement is then inserted into the grout after the auger is completely removed. Using

suitably sized equipment, ACIP elements can be constructed more rapidly and efficiently across a large site compared to drilled piers. The ultimate axial load carrying capacity of a pile can be computed using the static method of analysis. Based on PacifiCorp transmission foundation standards, ACIP piles shall be at least 16-inches diameter consisting of a 4-pile group. ACIP piers in group must be constructed at least five (5) pile diameters (center-to-center) apart. As with drilled piers, this foundation alternative may be adversely impacted by the presence of cobbles and or boulders. Additional recommendations and specifications for ACIP foundations may be provided upon request.

7.9.2.1 ACIP Construction Considerations

Auger-cast piles represent a specialized foundation system and require a high level of expertise for proper installation. Consequently, we recommend that only contractors experienced in the installation of this type of foundation and subsurface conditions be considered for this project.

A properly functioning pressure gauge and pump stroke counter should be provided on the grout pump to assist in monitoring auger-cast-in-place pile installation. The pump should be calibrated prior to its use. The pressure gauge is used to monitor the pressure of the grout to evaluate the rate at which the auger should be withdrawn and if the auger or hoses are plugged with grout. The auger should be withdrawn with slow positive rotation at slow steady pull and should not be pulled until the grout has been pumped several feet above the downhole tip.

A sufficient head of grout should be maintained in the auger system at all times during grouting. A quantity of grout equivalent to five feet of pile volume or more should be pumped prior to initiating the auger withdrawal. The volume of concrete grout placed per every 5-foot length of the pile should not be less than 1.15 times the calculated volume for every 5-foot length of pile. The total volume of concrete grout placed should be approximately 1.3 times the theoretical volume of the design pile dimensions. PSI recommends that no two adjacent piles that are located within four-pile diameters to each other be installed on the same day.

The concrete mix should be designed to attain the required 28-day design strength of 4,000 psi. Non-shrink grout should be fluid, with appropriate design mix and additives and should be placed at appropriate temperature and age. Fluidity of the grout should be checked frequently during installation using a ¾-inch flow cone in accordance with ASTM C 939. Flow rates of 15 to 25 seconds are typically considered acceptable when using ¾-inch flow cone. Compressive strength of the grout mix should be verified and the sampling and testing should be performed ASTM C 109.

Pile installation should be monitored on a full-time basis by the Geotechnical Engineer. The Engineer should monitor the progress of drilling, the overall depth of penetration, and grout takes and pressures.

Preliminary Geotechnical Engineering Report   March 11, 2011
Proposed Lassen Substation   PSI Job No. 0595148
Mount Shasta, California   Page 29

7.10 Earthquake and Seismic Design Parameters

A search of the U.S. Geological Survey National Earthquake Hazard Reduction Program (NEHRP) database resulted in the following probabilistic ground motion values at the bedrock elevation for Lassen substation with Latitude and Longitude indicated in Table 15.

Table 15: Earthquake and Seismic Design Parameters

Boring Location	Latitude (North)	Longitude (West)	PGA	2% in 50 Years S_S	S_1
Lassen Substation	41.3048	-122.3209	.293	0.693	0.258

Notes: **PGA** = Peak Ground Acceleration

       S_S = 0.2 sec Spectral Acceleration

       S_1 = 1.0 sec Spectral Acceleration

PacifiCorp requires equipment within substations be qualified according to the requirements presented in IEEE Std 693-2005, Recommended Practice for Seismic Design of Substations. Seismic qualification ensures electrical substation equipment will maintain the correct operational state during, and remain operational after, the maximum considered earthquake ground motion with 5% damping. The required qualification level was determined using procedures detailed in IEEE Std 693-2005. Utilizing the 0.2 second spectral acceleration based on 2% probability of exceedance in 50 years, the peak ground acceleration is, PGA is 0.293 for the Lassen Substation. Sites where PGA values are less than 0.1 receive a "low" qualification requirement. Values between 0.1 and 0.5 receive a "moderate" qualification requirement and values greater than 0.5 receive a "high" qualification requirement. The Lassen Substation site requires equipment be qualified to a moderate level.

7.10.1 Liquefaction Potential

In general, liquefaction is a condition where soils lose intergranular strength due to abrupt increases in pore water pressure. Pore water pressure increases typically occur during dynamic loading such as ground shaking during a seismic event. Liquefaction, should it occur on a site, can induce ground settlement and lateral spreading, which can result in damage to the structures. For liquefaction to occur, the following conditions must be present:

- The soil sediments must be in saturated or near-saturated conditions. At least 80-85 percent saturation is generally considered necessary for the liquefaction to occur.
- The soil must be predominately composed of non-plastic material such as sand or silt.
- The soil must be in a loose state.
- The soil must be subjected to dynamic loading, such as an earthquake.

Based on subsurface conditions encountered in the limited number and depth of borings, the granular soils encountered in the borings are in a loose to very dense state. Liquefaction should be

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 30

further evaluated in the second phase of investigation to better define soils that have the potential to liquefy during an earthquake. Blow counts at 2.5 and 5 feet below the ground surface appear to be affected by the groundwater and flowing sands, thereby indicating loose silty sand when in reality it is dense. With the exception of a thin soil zone at a depth between about 7 and 9 feet as noted on Array No. 4, ReMi results generally did not identify any layers with a velocity of less than 600 ft/sec in they upper 100 feet. A shearwave velocity of 600 ft/sec is generally considered the threshold limit of whether granular soil can liquefy during an earthquake.

7.11 Cable Trenches and Tray Systems

Utility trenches should be kept free from water during excavation, fine grading, pipe laying, jointing, and embedment operations. Where the trench bottom is disturbed or otherwise unstable because of the presence of groundwater, or where the static free water elevation is above the bottom of the trench, the free water level should be lowered to the extent necessary using a suitable dewatering system to keep the trench free from water and the trench bottom stable when the work within the trench is in progress. Surface water should be prevented from entering trenches. If unstable soils are encountered at invert elevations, it may be necessary to over-excavate and replace the unstable soils with free draining gravel backfill. The depth of over-excavation, if necessary, should be determined by field observation.

7.11.1 Cable Trench Backfill

The backfill placed in utility trench excavations within the limits of the proposed structures and yard areas should consist of sand, sand and gravel, or crushed rock with a maximum size of up to 1½ inches, and with not more that 5 percent passing the No. 200 sieve (washed analysis). This backfill should be uniformly moisture conditioned and firmly compacted for pipe support. The granular backfill should be placed in maximum 9 inches-thick lifts (loose) and compacted using vibratory compaction equipment to at least 95 percent of the maximum dry density as determined by ASTM D 1557. Flooding or jetting the backfilled trenches with water to attempt to achieve compaction should not be permitted.

Even when placed and compacted under optimum conditions, trench backfill may settle over time. Therefore, all improvements such as concrete foundations placed over trench backfill should be designed to span over localized irregularities or be designed to allow some differential movement.

8.0 GEOTECHNICAL RISK

The concept of risk is an important aspect of the geotechnical evaluation. The primary reason for this is that the analytical methods used to develop geotechnical recommendations do not comprise an exact science. The analytical tools which geotechnical engineers use are generally empirical and must be used in conjunction with engineering judgment and experience. Therefore, the solutions and recommendations presented in the geotechnical evaluation should not be considered risk-free and, more importantly, are not a guarantee that the interaction between the soils and the proposed structure will perform as planned. The engineering recommendations presented in the preceding sections constitute PSI's professional estimate of those measures that are necessary for

Preliminary Geotechnical Engineering Report
Proposed Lassen Substation
Mount Shasta, California

March 11, 2011
PSI Job No. 0595148
Page 31

the proposed structure to perform according to the proposed design based on the information generated and referenced during this evaluation, and PSI's experience in working with these conditions.

9.0 LIMITATIONS

The recommendations submitted are based on the available subsurface information obtained by PSI, and information provided by PacifiCorp, and their design consultants. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation and/or other recommendations are required. If PSI is not retained to perform these functions, PSI cannot be responsible for the impact of those conditions on the performance of the project. The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

The Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of PacifiCorp and their design consultants for the specific application to the proposed Lassen Substation in Mount Shasta, California.

List of References

California Geological Survey (CGS), April, 2002, "California Geomorphic Provinces" http://www.consrv.ca.gov/cgs/information/publications/cgs_notes/note_36/note_36.pdf (web document).

California Geological Survey (CGS), 2010, "Fault Activity Map of California" http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html

Gibson R.E., (1953), Experimental Determination of the True Cohesion and True Angle of Internal Friction in Clay.

Holtz W.G. and Gibbs H.J., 1979, Discussion of SPT and Relative Density in Coarse Sand.

Idriss, I.M, and Boulanger, R.W. (2008). *Soil liquefaction during earthquakes.* Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA, 261 pp.

Kulhawy F.H. and Mayne P.W., 1990, Manual on Estimating Soil Properties for Foundation Design.

Louie, J.N. (2001). Faster, better: shear-wave velocity to 100 meters depth from refraction microtremor arrays.

Miller C. D., (1980). Potential Hazards from Future Eruptions in the Vicinity of Mount Shasta Volcano, Northern California.

Naval Facilities Engineering Command, Soil Mechanics: NAVFAC DM 7.01.

Sowers G.F., (1979), Introductory Soil Mechanics and Foundations: Geotechnical Engineering

Strand, R.G., 1963, Geologic map of California: Weed sheet: California Division of Mines and Geology, scale 1:250,000.

APPENDIX A

Vicinity Map
Site Map with Boring and ReMi Locations
Photographs

Lassen Substation

Mt. Shasta, CA
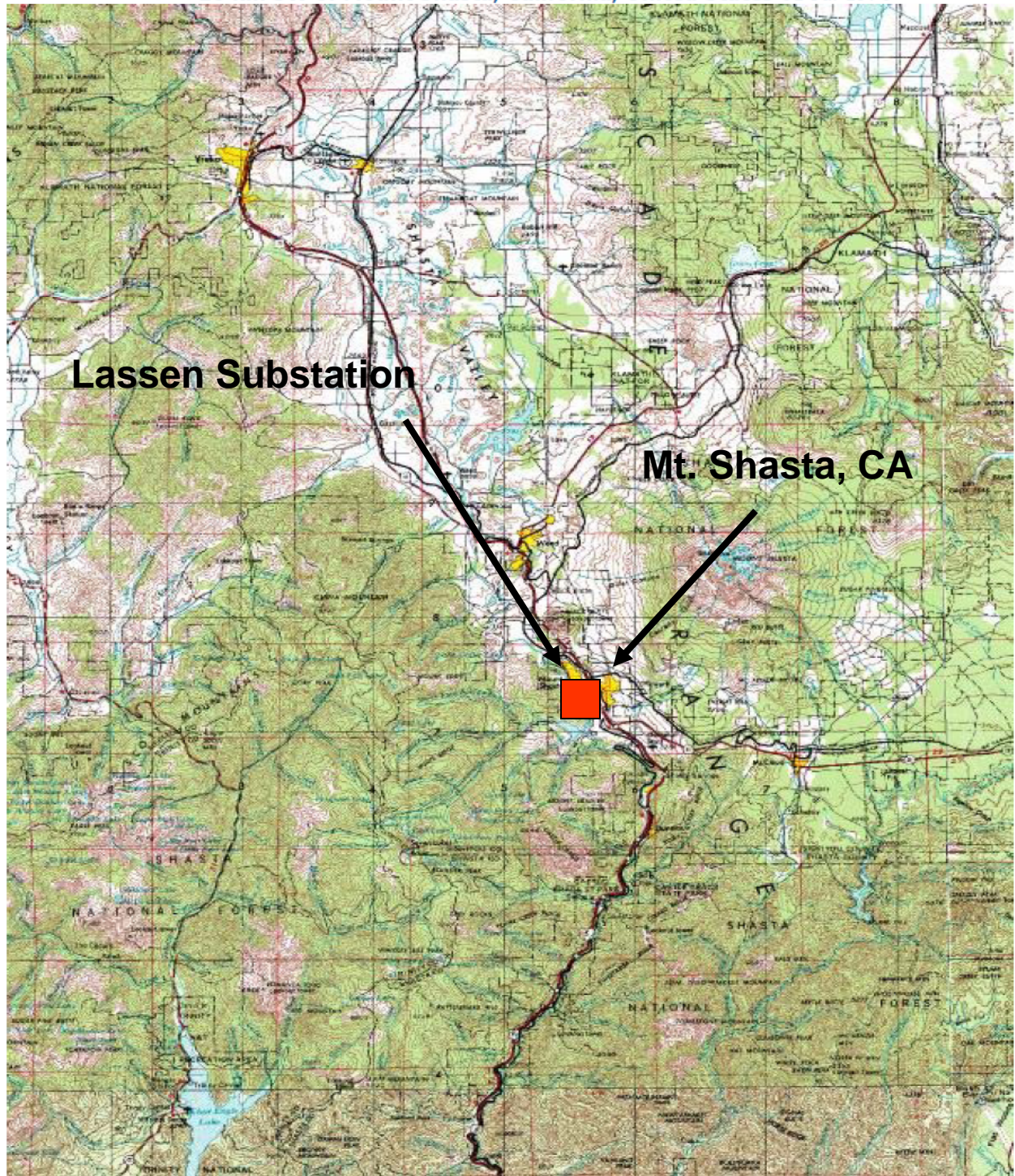
Image courtesy of the U.S. Geological Survey

© 2010 Microsoft Corporation.   **Terms of Use**   **Privacy Statement**

| psi *Information To Build On* Engineering • Consulting • Testing | Lassen Substation Vicinity map Job Number: 0595148 | **Figure A-1** |

Boring Location

ReMi Array
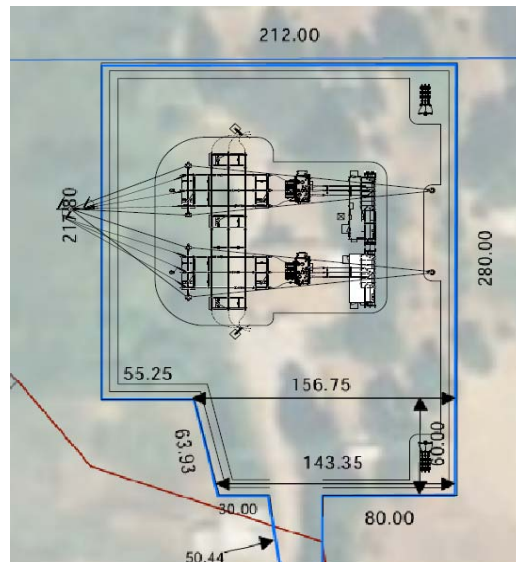
Lassen Substation Site Layout

Lassen Substation

Site Map Borings and Refraction
Microtremor Test Locations

Job Number: 0595148

Figure A-2

APPENDIX B

Logs of Borings

LOG OF BORING SB-1

PSI
2779 S 600 W
Salt Lake City, Utah, 84115
Telephone: 801-484-8827
Fax: 801-487-3312

Sheet 1 of 1

PSI Job No.:	0595148
Project:	Lassen Substation
Latitude:	41.3047
Longitude:	-122.3209

Drill Rig:	CME 55
Drilling Method:	HSA (8 inch)
Sampling Method:	Split Spoon/Modified Cal.
Hammer Type:	140 lb/30 inch (Manual)

Standard Penetration Test Data

N in blows/ft ◎
× Moisture ◣ PL ✚ LL
STRENGTH, tsf ▲ Qu ✳ Qp

DRILLED PIER DESIGN PARAMETERS

Depth (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows/6-in (SS)	Moisture, %	Additional Remarks	Foundation Profile	Ult End Bearing Capacity, psf	Ult Skin Friction, psf (comp/uplift)	Friction Angle, Degrees	Cohesion, psf	Total Unit Weight pcf	Active, K_a	Passive, K_p	LPile Soil Model	Hor Sub Mod K_h Static, pci	Hor Sub Mod K_h cyclic, pci	Strain, e_{50}
Elev 3438, 0			**Gravel Roadway** 2 inches **Sandy SILT (ML)** - trace organics, soft, very moist to wet, low plasticity, dark brown	ML		33	LL = 37 PL = 30	0.0				0		450		7			
	1	8			2-1-2 N=3	31													
3435	2	8	**Silty SAND with gravel (SM)** trace organics to 5 feet, loose, moist to wet, fine to coarse grained, brown		1-2-1 N=3		Gravel=18.1% Sand=44.2% Fines=37.6%	2.5		250	0	450	56.0	0.39	2.56	7	40	20	0.0175
								3.0		188 440 330	32	0	50.0	0.31	3.25	4	0	0	---
5	3	8	-- blowcounts may have been affected by groundwater -- wet		2-2-7 N=7	34	Resist. = 13000.0 ohm-cm pH = 6.8 SO$_4$ = <7.32 mg/Kg	5.0		550 413	32	0	50.0	0.31	3.25	4	20	20	---
3430	4	8	-- medium dense	SM	5-5-15 N=20	41	Resist. = 14800.0 ohm-cm pH = 7.4 SO$_4$ = <7.08 mg/Kg	7.5		610 458	42	0	56.0	0.20	5.04	4	110	110	---
10	5	6			6-8-9 N=17	35	DD = 88 pcf Gravel=39.5% Sand=39.9% Fines=20.7%	10.0	23800	640 480	46	0	57.0	0.16	6.13	4	90	90	---
3425			**Poorly graded SAND with silt and gravel (SP-SM)** - very dense, wet, fine to coarse grained sand, gray					13.0	70000	770 578	45	0	65.0	0.17	5.83	4	350	350	---
15	6	5			28-50(1")	12	DD = 126 pcf Gravel=24.4% Sand=64.1% Fines=11.5%												
3420			-- cobbles and boulders in a silty sand matrix	SP-SM				17.5	70000	940 705	45	0	65.0	0.17	5.83	4	350	350	---
20	7	4	Auger refusal on cobbles and boulders at 20 feet 4 inches		50(5")	22													

Date Boring Started: 2/28/11
Date Boring Completed: 2/28/11
Logged By: ER
Reviewed By: KCM

WATER LEVELS
▽ While Drilling — 4 ft
▼ Design Water Table — 2.5 ft
▽ Delay — N/A ft

Sample Types: ⊠ Split-Spoon ■ Shelby Tube ▌ Rock Core

L-Pile Models: 1-Soft Clay; 2-Stiff Clay w/ Water; 3-Stiff Clay w/o Water; 4-Sand; 5-Linear Interp.; 6- Vuggy Limestone; 7-Silt

Remarks: FIGURE B-1

The stratification lines represent approximate boundaries. The transition may be gradual.

LOG OF BORING SB-2

Sheet 1 of 1

PSI Job No.:	0595148
Project:	Lassen Substation
Latitude:	41.3051
Longitude:	-122.3209

Drill Rig:	CME 55
Drilling Method:	HAS (8 inch)
Sampling Method:	Split Spoon/Modified Cal.
Hammer Type:	140 lb/30 inch (Manual)

DRILLED PIER DESIGN PARAMETERS

Standard Penetration Test Data
N in blows/ft ◎
× Moisture   ◢ PL
＋ LL
0   25   50
STRENGTH, tsf
▲ Qu   ✳ Qp
0   2.0   4.0

Material Descriptions:

Gravel Roadway 1 inch
Sandy SILT (ML) - trace organics, stiff, very moist to wet, low plasticity, dark brown

Silty SAND with gravel (SM) trace organics to 4 feet, medium dense, very moist to wet, fine to coarse grained, brown

Auger refusal on cobbles and boulders at 5 feet 6 inches

Elevation (feet)	Depth (feet)	Sample No.	Recovery (in)	USCS	SPT Blows/6-in (SS)	Moisture %
3438	0					
	~2	1	5	ML	6-2-11 N=13	34
3435	~3	2	0	SM	4-5-5 N=10	
	5	3	0		50(4.5")	

Foundation Profile	Ult End Bearing Capacity, psf	Ult Skin Friction, psf (comp/uplift)	Friction Angle, Degrees	Cohesion, psf	Total Unit Weight pcf	Active, K_a	Passive, K_p	LPile Soil Model	Hor Sub Mod K_h Static, pci	Hor Sub Mod K_h cyclic, pci	Strain, e_{50}
0.0			0	450				7			
2.5		1070 / 803	0	1950	54.0	0.39	2.56	7	650	260	0.0061
3.0		450 / 338	37	0	55.0	0.25	4.02	4	40	40	---
5.0	70000	570 / 428	44	0	55.0	0.18	5.55	4	350	350	---

Additional Remarks

Date Boring Started:	2/28/11	
Date Boring Completed:	2/28/11	
Logged By:	ER	
Reviewed By:	KCM	

WATER LEVELS

▽ While Drilling	4	ft
▼ Design Water Table	2.5	ft
▽ Delay	N/A	ft

Sample Types: ⊠ Split-Spoon ■ Shelby Tube ▐ Rock Core

L-Pile Models 1-Soft Clay; 2-Stiff Clay w/ Water; 3-Stiff Clay w/o Water; 4-Sand; 5-Linear Interp.; 6- Vuggy Limestone; 7-Silt

Remarks: FIGURE B-2

The stratification lines represent approximate boundaries. The transition may be gradual.

PSI
2779 S 600 W
Salt Lake City, Utah, 84115
Telephone: 801-484-8827
Fax: 801-487-3312

LOG OF BORING SB-3

Sheet 1 of 1

PSI Job No.:	0595148
Project:	Lassen Substation
Latitude:	41.3052
Longitude:	-122.3209

Drill Rig:	CME 55
Drilling Method:	HAS (8 inch)
Sampling Method:	Split Spoon/Modified Cal.
Hammer Type:	140 lb/30 inch (Manual)

Standard Penetration Test Data

N in blows/ft ◎
× Moisture ◢ PL ➕ LL

STRENGTH, tsf
▲ Qu ✳ Qp

DRILLED PIER DESIGN PARAMETERS

Elevation 3439

Material Description

- **Gravel Roadway** 1 inch
- **Sandy SILT (ML)** - trace organics, stiff, very moist to wet, low plasticity, dark brown — ML
- **Silty SAND with gravel (SM)** trace organics to 4 feet, medium dense, very moist to wet, fine to coarse grained, brown — SM
- -- wet, cobbles at 5 feet
- Auger refusal on cobbles and boulders at 7 feet 8 inches

Sample No.	Recovery (inches)	SPT Blows/6-in (SS)	Moisture, %
1	6	2-2-20 **N=22**	31
2	0	12-6-10 **N=16**	
3	0	50(3.5")	

Elevation marks: 3435

Drilled Pier Design Parameters Table

Foundation Profile	Ult End Bearing Capacity, psf	Ult Skin Friction, psf (comp/uplift)	Friction Angle, Degrees	Cohesion, psf	Total Unit Weight pcf	Active, K_a	Passive, K_p	LPile Soil Model	Hor Sub Mod K_h Static, pci	Hor Sub Mod K_h cyclic, pci	Strain, e_{50}
0.0				0	450			7			
2.5		430 / 323	38	0	55.0	0.39	2.56	4	120	120	---
5.0		570 / 428	37	0	55.0	0.31	3.25	4	80	80	---
7.5	70000	690 / 518	44	0	55.0	0.31	3.25	4	350	350	---

Date Boring Started:	2/28/11	
Date Boring Completed:	2/28/11	
Logged By:	ER	
Reviewed By:	KCM	

WATER LEVELS

▽ While Drilling	5	ft
▼ Design Water Table	2.5	ft
▽ Delay	N/A	ft

Sample Types: ⊠ Split-Spoon ▮ Shelby Tube ▐ Rock Core

L-Pile Models: 1-Soft Clay; 2-Stiff Clay w/ Water; 3-Stiff Clay w/o Water; 4-Sand; 5-Linear Interp.; 6- Vuggy Limestone; 7-Silt

Remarks: FIGURE B-3

The stratification lines represent approximate boundaries. The transition may be gradual.

APPENDIX C

Summary of Laboratory Test Results
Direct Shear Results

Laboratory Summary Sheet

Borehole	Approx. Depth	Liquid Limit	Plastic Limit	Plasticity Index	Qu (tsf)	%<#200 Sieve	Est. Specific Gravity	Water Content (%)	Dry Density (pcf)	Satur- ation (%)	Void Ratio
SB-1	1.01	37	30	7				33			
SB-1	2.51					37.6		31			
SB-1	5.01							34			
SB-1	7.51							41			
SB-1	10.01					20.7		35	88	100%	0.88
SB-1	15.01					11.5		12	126	98%	0.31
SB-1	20.01							22			
SB-2	1.01							34			
SB-3	2.51							31			

Summary of Laboratory Results

PSI
2779 S 600 W
Salt Lake City, Utah, 84115
Telephone: 801-484-8827
Fax: 801-487-3312

PSI Job No.: 0595148
Project: Lassen Substation
Location: Mount Shasta, California

Figure C-1

Professional Service Industries, Inc.

Consolidated Drained Direct Shear Test (ASTM D5321)

psi Information To Build On
Engineering • Consulting • Testing

Date 3/3/11

T.Bernius

Checked By

Date 3/3/11

T. Bernius

Computed By

Date 3/3/11

T. Bernius

Tested By

Shear Stress Vs. Normal Stress
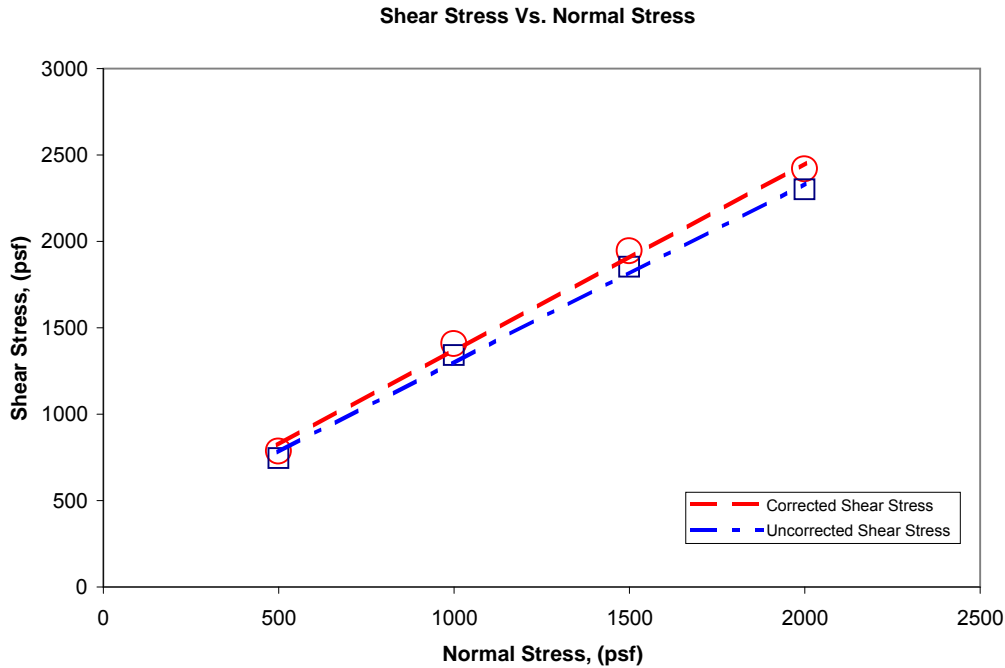
Mohr-Coulomb Stress Envelope				Parameter	Specimen			
					A	**B**	**C**	**D**
Area Corrected	Ø' (deg)	47.4		Initial Water Content (%)	≈35.2	≈35.2	≈35.2	≈35.2
	C' (psf)	280		Final Water Content (%)	37.5	36.8	34.4	N/A
	R^2	0.9962		Dry Density (psf)	88.7	87.4	92.3	85.4
	SSE	N/A		Diameter (in)	2.42			
Area Uncorrected	Ø' (deg)	46.0		Height (in)	1.00			
	C' (psf)	270		Strain Rate (in/min)	0.004			
	R^2	0.9962		Plastic Index	NP			
	SSE	N/A		Average T_{50} (min)	1.31			

Project:	Lassen Substation	Data Points		
Project Number:	595148	**Normal Stress**	**Corrected Shear Stress**	**Uncorrected Shear Stress**
Boring Number:	SB-1			
Sample Number:	5	500	784	746
Depth:	10 feet	1000	1408	1339
Sample Type:	In-Situ	2000	2420	2301
Rel. Compaction:	N/A	1500	1945	1850
Description:	Silty Sand with some gravel (10%)			
Remarks:	Some grains were larger than No. 10 Sieve (20%). Stress envelope taken at 4% strain.			

Information To Build On
Engineering • Consulting • Testing

Date 3/3/11

T.Bernius

Checked By

Date 3/3/11

T. Bernius

Computed By

Date 3/3/11

T. Bernius

Tested By

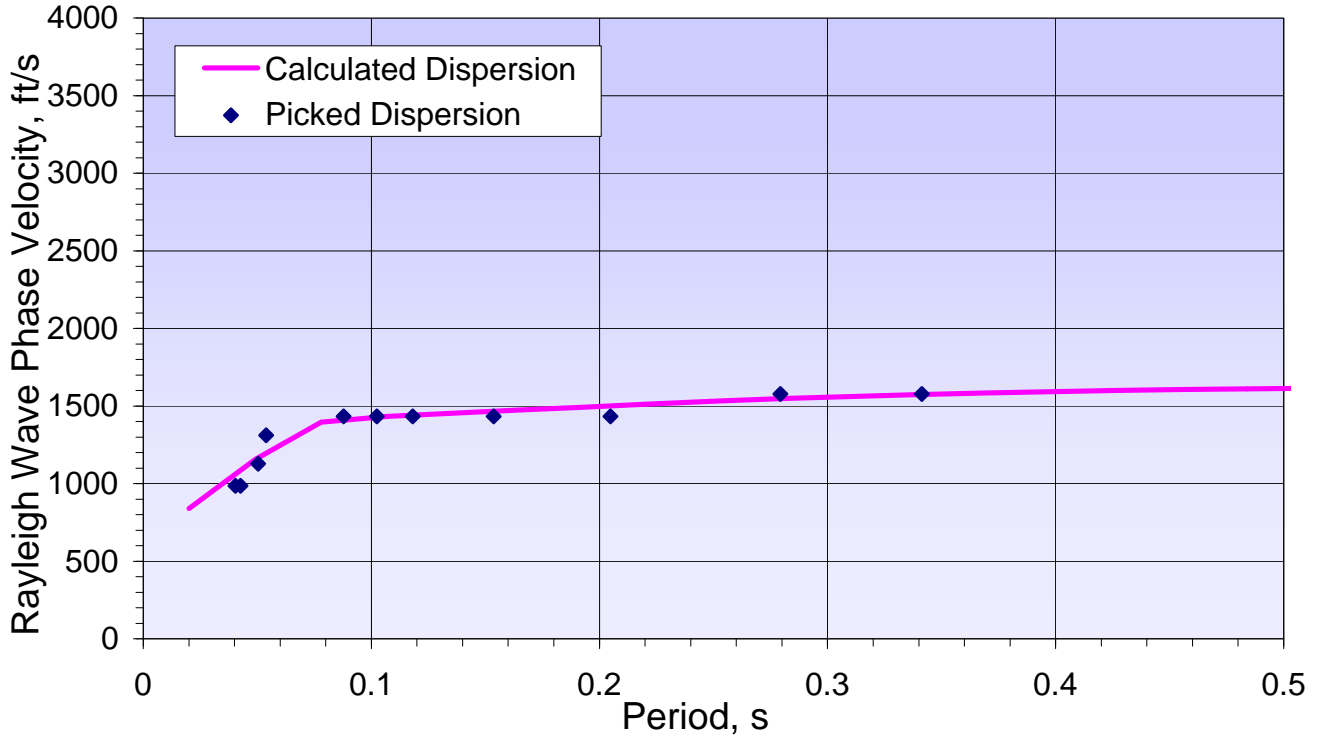**Corrected Shear Stress Vs. Horizontal Strain**

Stress Envelope

Legend:
- 500
- 1000
- 2000
- 1500

Uncorrected Shear Stress Vs. Horizontal Strain

Stress Envelope

Legend:
- 500
- 1000
- 2000
- 1500

CD Direct Shear - Results

Date 3/3/11

T.Bernius

Checked By

Date 3/3/11

T. Bernius

Computed By

Date 3/3/11

T. Bernius

Tested By

Vertical Strain Vs. Horizontal Strain

Legend: 500, 1000, 2000, 1500

Y-axis: Vertical Strain, (%)
X-axis: Horizontal Strain, (%)

Consolidation Curves

Legend: 500, 1000, 2000, 1500, 250

Y-axis: Vertical Strain, (%)
X-axis: Square Root of Time, (min^1/2)
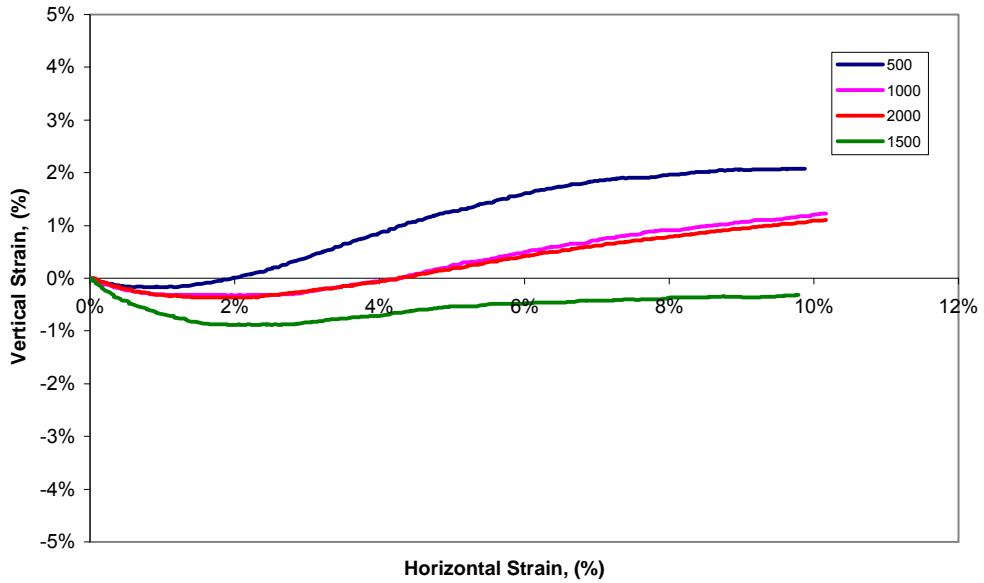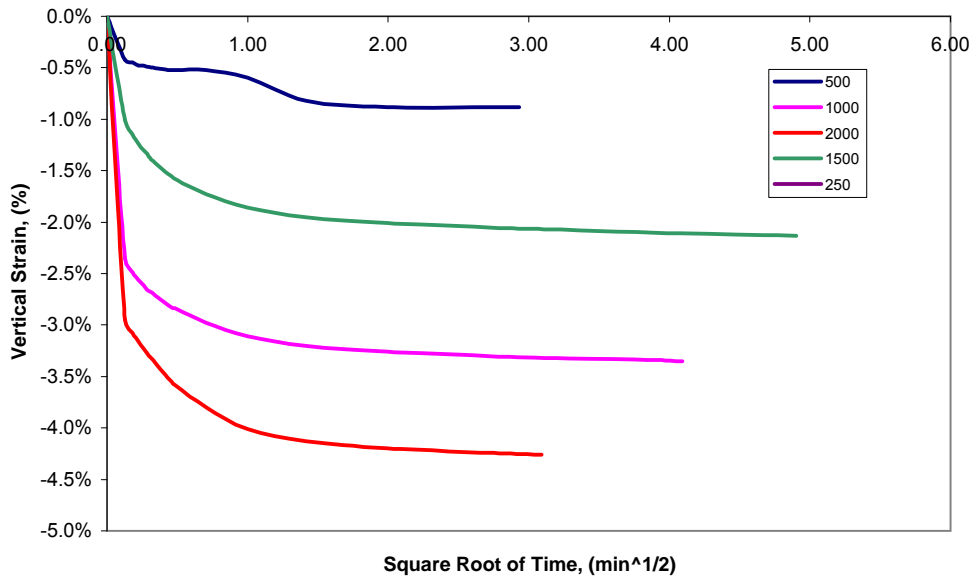
APPENDIX D

Refraction Microtremor Results

Dispersion Curve Showing Picks and Fit

Legend:
- Calculated Dispersion
- Picked Dispersion

Y-axis: Rayleigh Wave Phase Velocity, ft/s (0 to 4000)
X-axis: Period, s (0 to 0.5)

| Geophone Spacing (ft) | 15 | Sampling Interval (ms) | 2 |

p-f Image with Dispersion Modeling Picks

Frequency, Hz — 0.0 to 34.830
Slowness, sec/meter — 0.0 to 0.01
Averaged ReMi Spectral Ratio — 0.0 to 2.5

PSI Information To Build On
Engineering • Consulting • Testing
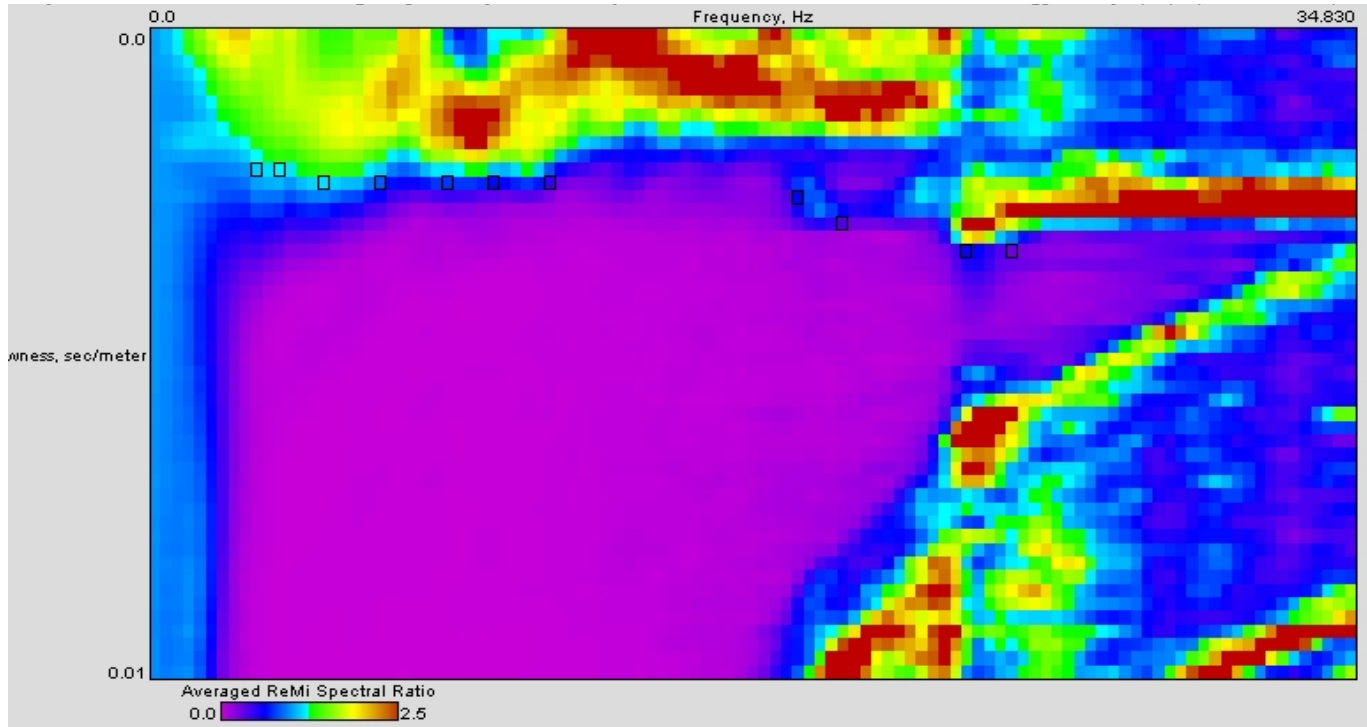
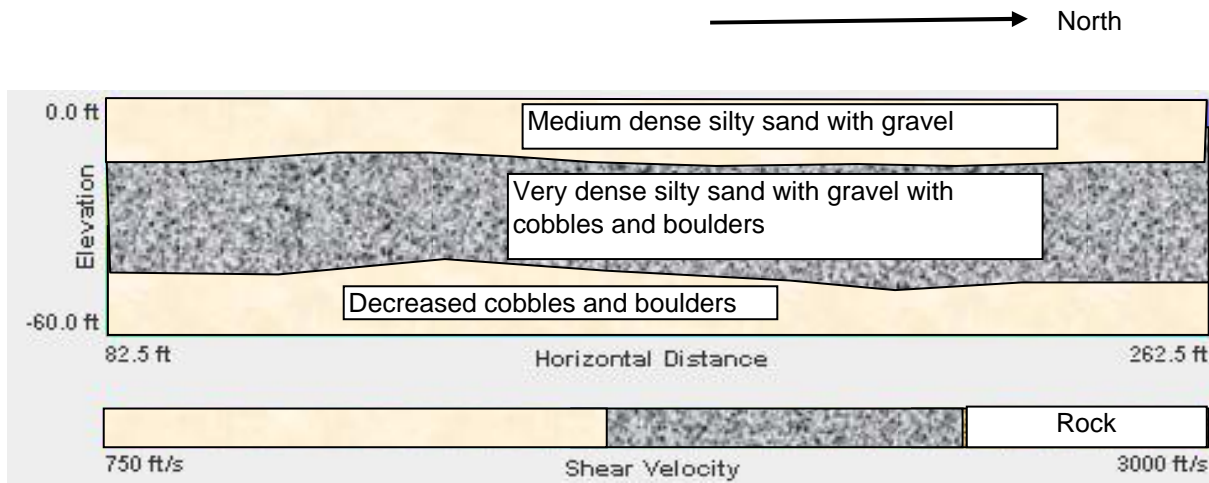| Line Number	Array 1 (Geophones 1-24)
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

Figure D-1

Shear Wave Velocity Profile Vs. Depth

Legend: Vs Refraction Microtremor

X-axis: Shear-Wave Velocity, Vs (ft/s) — 0, 500, 1000, 1500, 2000, 2500

Y-axis: Depth (ft) — 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100

IBC Site Class	C	Line Number	Array 1 (Geophones 1-24)
		Project Number	595148
Average Shearwave Velocity within 100 feet, V_s (ft/s)	1,533	Project Name	Lassen Substation
		Location	Mount Shasta, California

psi *Information To Build On*
Engineering • Consulting • Testing

Figure D-2

2-Dimensional Profile

2-Dimensional Profile (Interpretation)

North →

Medium dense silty sand with gravel

Very dense silty sand with gravel with cobbles and boulders

Decreased cobbles and boulders

Rock

Line Number	Array 1
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

PSI Information To Build On
Engineering • Consulting • Testing

Figure D-3

Dispersion Curve Showing Picks and Fit

Geophone Spacing (ft)　　15　　　　Sampling Interval (ms)　　2

p-f Image with Dispersion Modeling Picks

Line Number	Array 2 (Geophones 5-21)
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

Figure D-4

PSI **Information To Build On**

Engineering • Consulting • Testing

Shear Wave Velocity Profile Vs. Depth

Legend: — Vs Refraction Microtremor

X-axis: Shear-Wave Velocity, Vs (ft/s) — 0, 500, 1000, 1500, 2000, 2500, 3000

Y-axis: Depth (ft) — 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100

IBC Site Class	C	Line Number	Array 2 (Geophones 5-21)
		Project Number	595148
Average Shearwave Velocity within 100 feet, V_s (ft/s)	1,485	Project Name	Lassen Substation
		Location	Mount Shasta, California

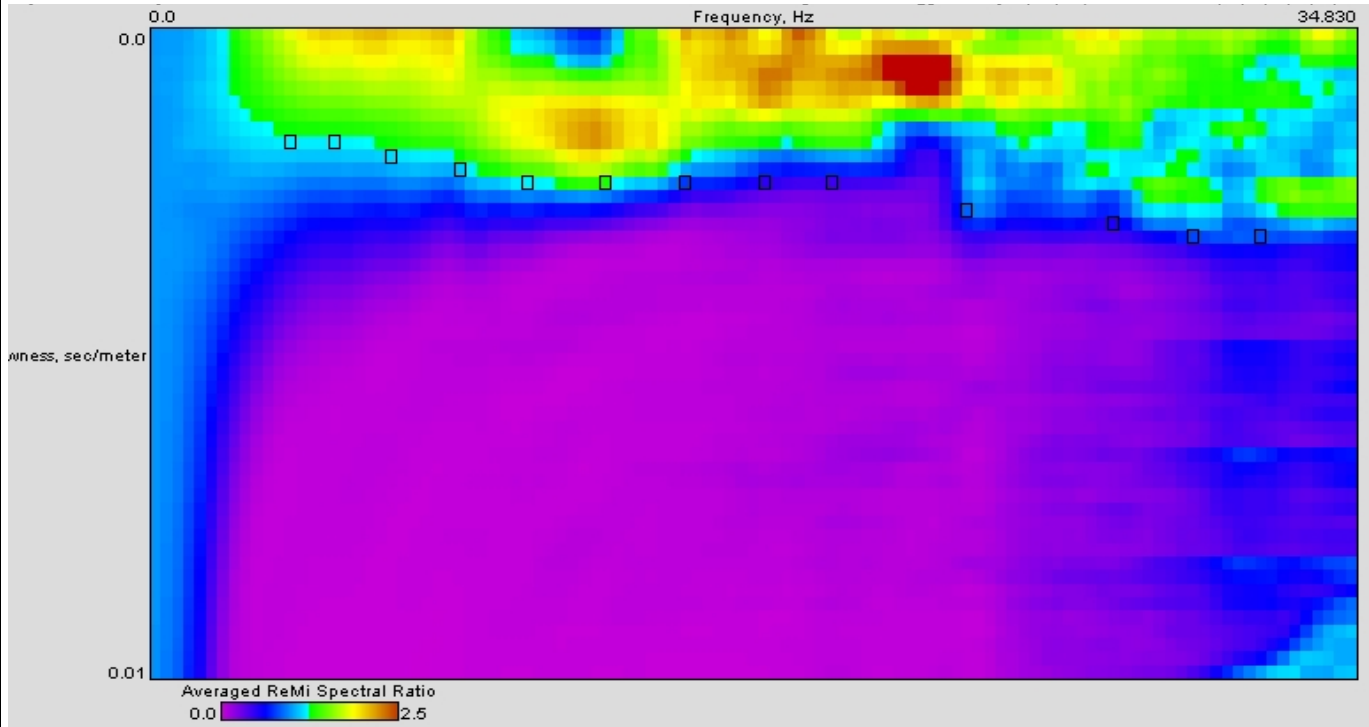**psi** *Information To Build On*
Engineering • Consulting • Testing

Figure D-5

2-Dimensional Profile
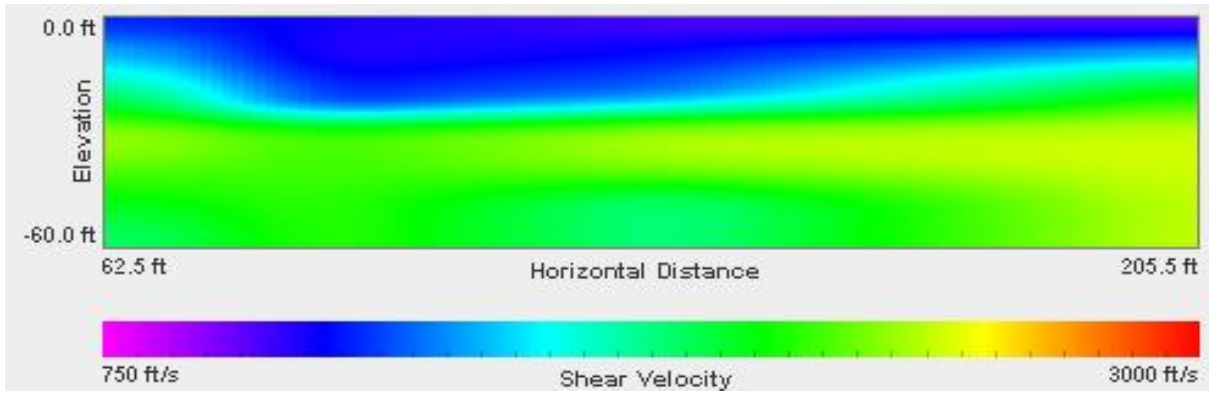
2-Dimensional Profile (Interpretation)

North

Medium dense silty sand with gravel

Very dense silty sand with gravel with cobbles and boulders

Rock

Line Number	Array 2
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

PSI Information To Build On

Engineering • Consulting • Testing

Figure D-6
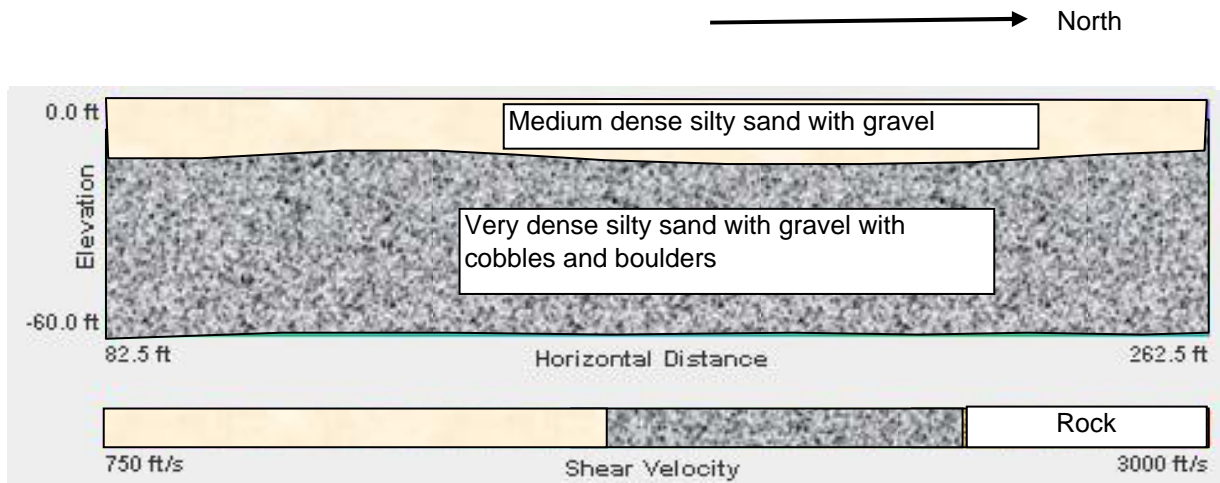
Dispersion Curve Showing Picks and Fit
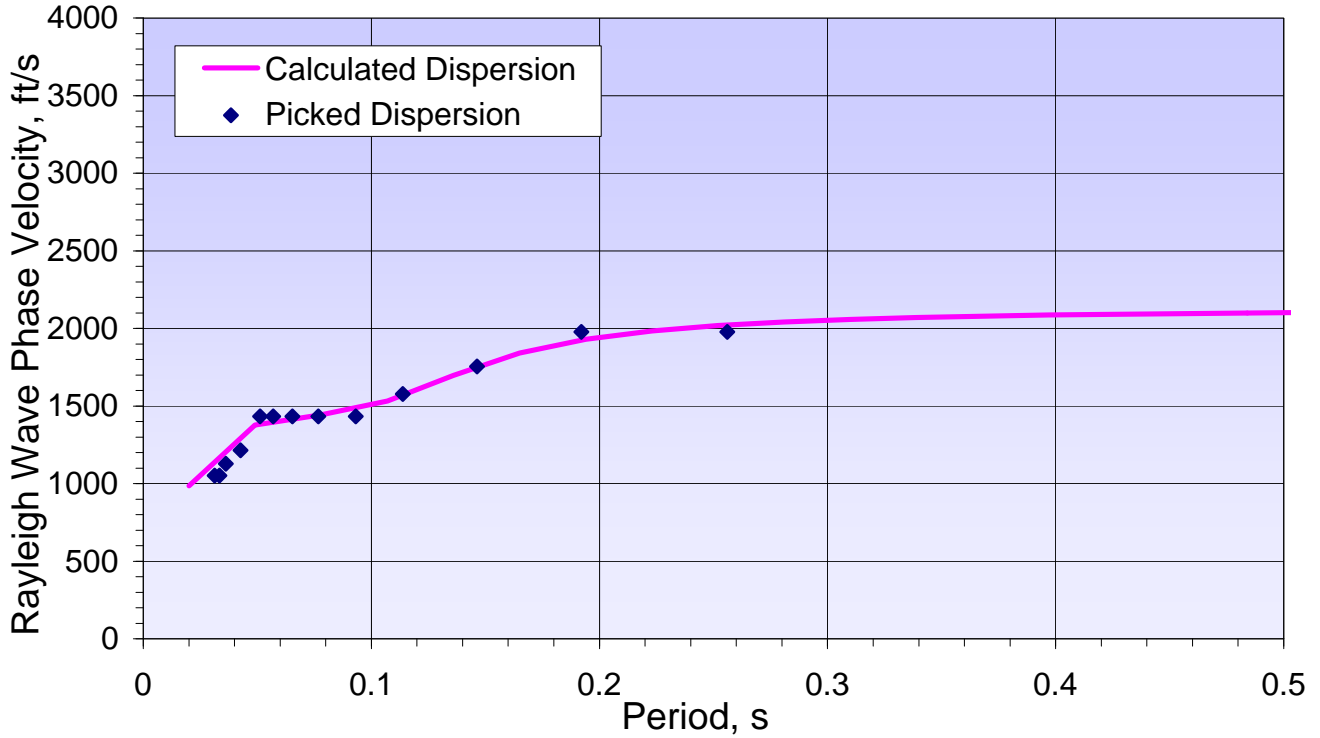
Geophone Spacing (ft)   8   Sampling Interval (ms)   2

p-f Image with Dispersion Modeling Picks

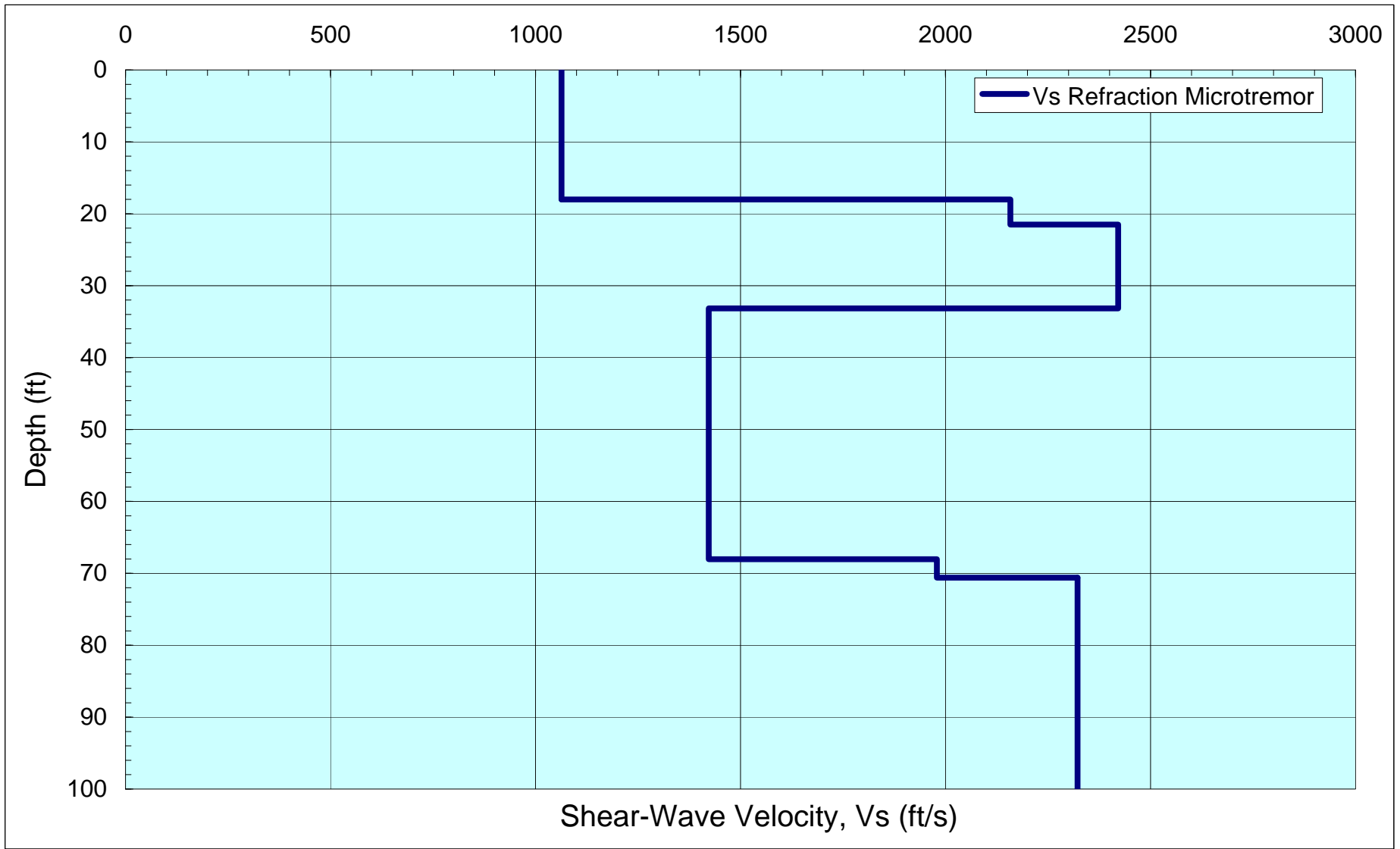| Line Number	Array 3 (Geophones 5-24)
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

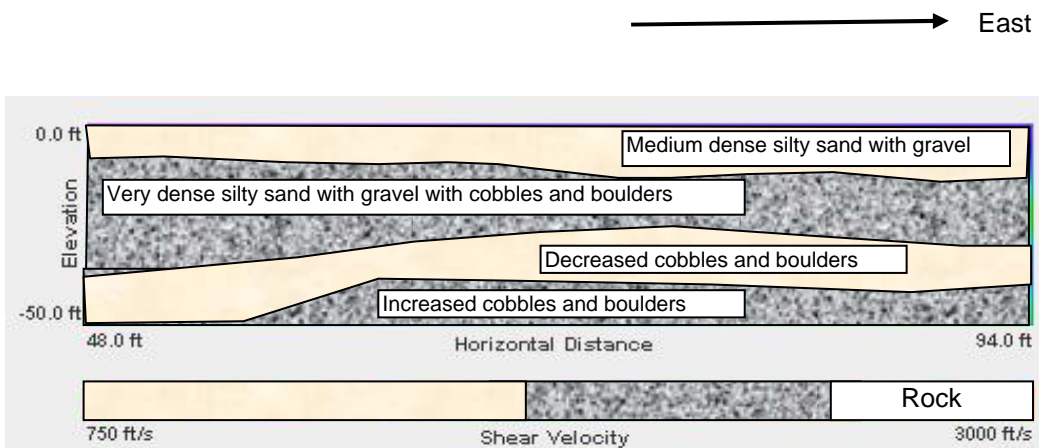Figure D-7

Shear Wave Velocity Profile Vs. Depth

Chart: Shear-Wave Velocity, Vs (ft/s) vs. Depth (ft), with legend "Vs Refraction Microtremor". X-axis: 0 to 3000. Y-axis (Depth): 0 to 100 ft.

IBC Site Class		**C**
Average Shearwave Velocity within 100 feet, V$_s$ (ft/s)		**1,617**

Line Number	Array 3 (Geophones 5-24)
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

Information To Build On
Engineering • Consulting • Testing

Figure D-8

2-Dimensional Profile

2-Dimensional Profile (Interpretation)

East →

Medium dense silty sand with gravel

Very dense silty sand with gravel with cobbles and boulders

Decreased cobbles and boulders

Increased cobbles and boulders

Rock

Line Number	Array 3
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

PSI Information To Build On
Engineering • Consulting • Testing
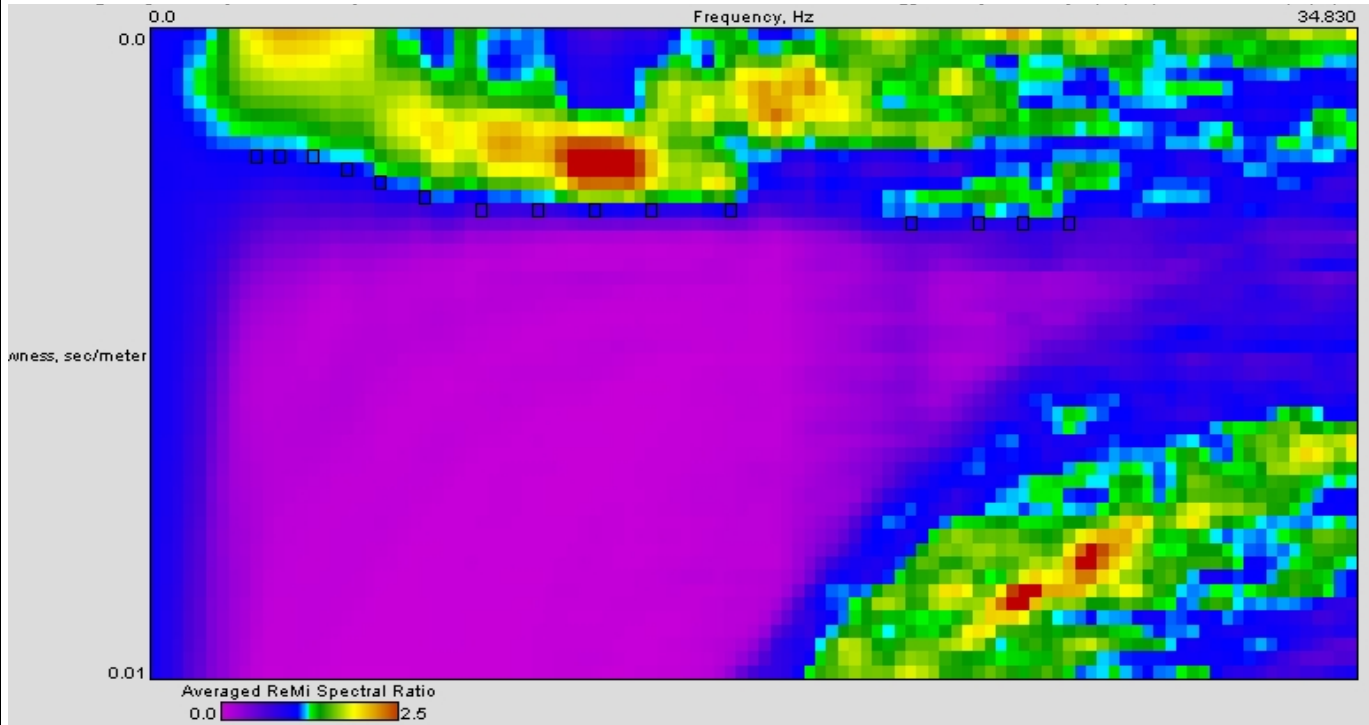
Figure D-9

Dispersion Curve Showing Picks and Fit

Geophone Spacing (ft)   15   Sampling Interval (ms)   2

p-f Image with Dispersion Modeling Picks

Line Number	Array 4 (Geophones 5-21)
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

Figure D-10

PSI
Information To Build On
Engineering • Consulting • Testing

Shear Wave Velocity Profile Vs. Depth

Legend: Vs Refraction Microtremor

Y-axis: Depth (ft), from 0 to 100

X-axis: Shear-Wave Velocity, Vs (ft/s), from 0 to 2500
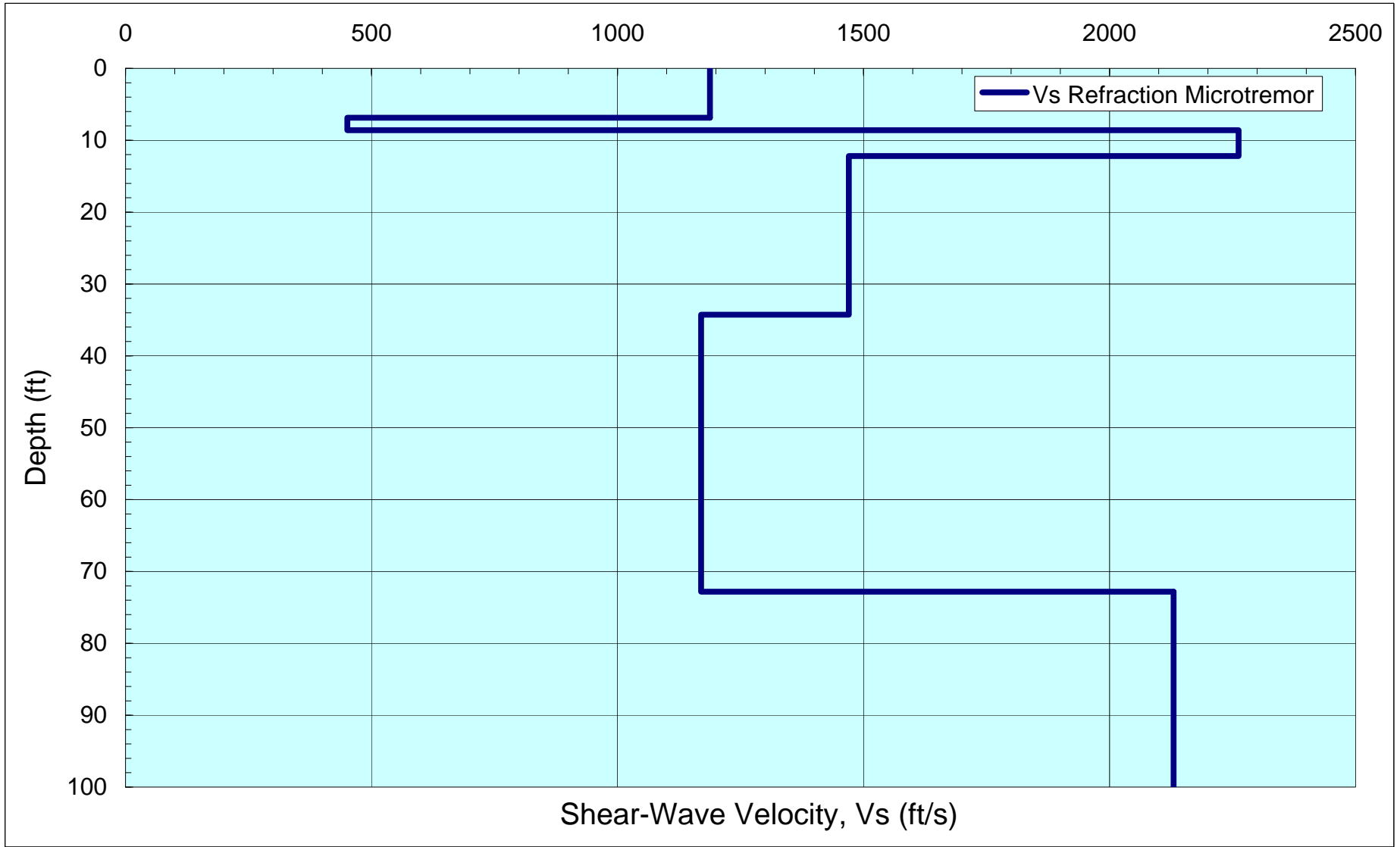
IBC Site Class	**C**
Average Shearwave Velocity within 100 feet, V_s (ft/s)	**1,390**

Line Number	Array 4 (Geophones 5-21)
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

PSi Information To Build On
Engineering • Consulting • Testing

Figure D-11

2-Dimensional Profile

2-Dimensional Profile (Interpretation)

East →

Very dense silty sand with gravel with cobbles and boulders

Decreased cobbles and boulders

Rock
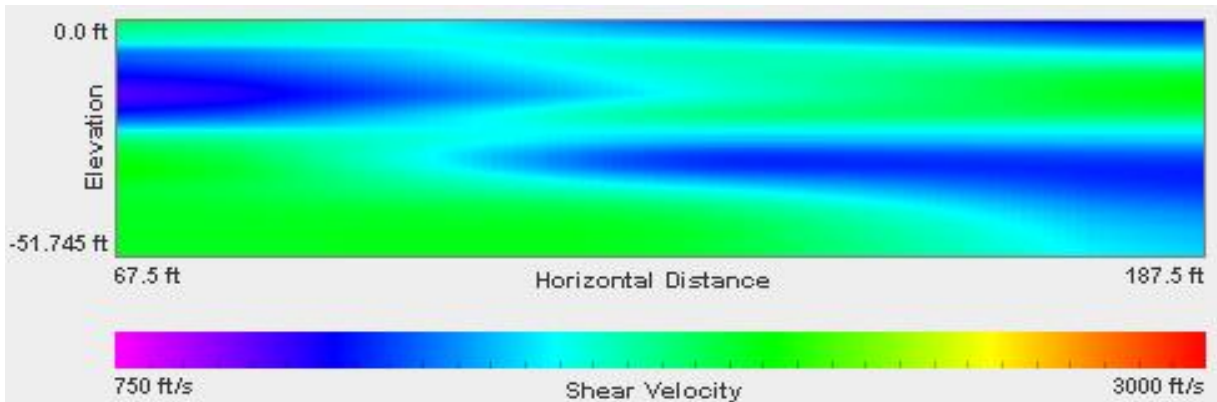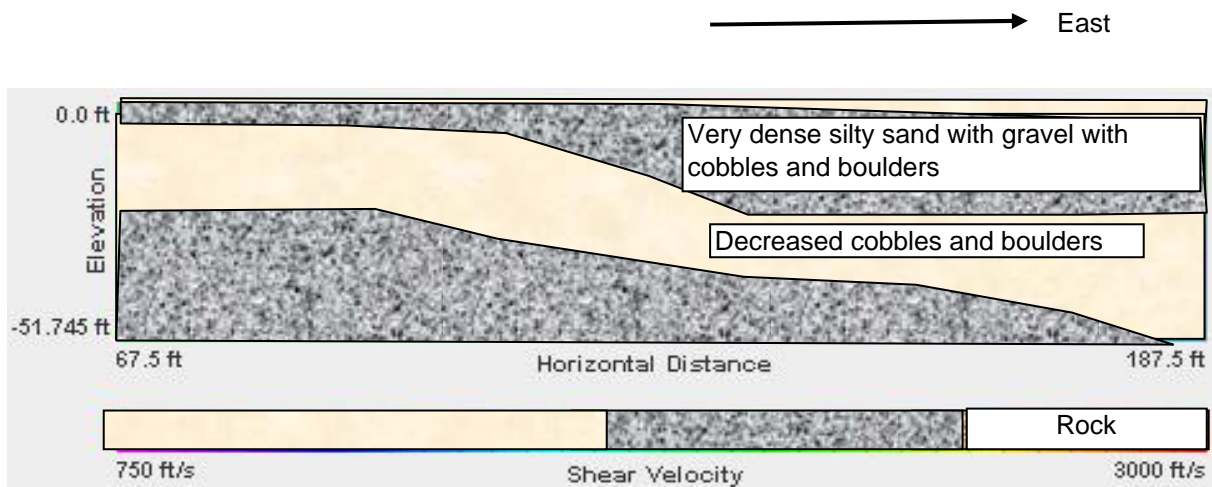
Line Number	Array 4
Project Number	595148
Project Name	Lassen Substation
Location	Mount Shasta, California

PSI **Information To Build On**
Engineering • Consulting • Testing

Figure D-12