INFORMATION HANDOUT

For Contract No. 02-4G03u4
At Teh-36-R75.1/78.4

Identified by
Project ID 0215000056

PERMITS

United States Army Corps of Engineers NWP 14
Dated October 30, 2015

WATER QUALITY

California Regional Water Quality Control Board
Central Valley Region 401 Certification
Dated November 4, 2015

MATERIALS INFORMATION

Geotechnical Report
Dated June 10, 2016

Water Source Information
Non-Potable information dated May 3, 2016
Potable information dated May 4, 2016

Site Investigation Report of Soils Summary
Dated February 2016

Specifications for Rock Excavation (Controlled Blasting)

INFORMATION HANDOUT

For Contract No. 02-4G03u4
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Project ID 0215000056

PERMITS

United States Army Corps of Engineers Nation Wide Permit (NWP) 14
Dated October 30, 2015

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS, SACRAMENTO DISTRICT
1325 J STREET
SACRAMENTO CA 95814-2922

REPLY TO
ATTENTION OF

October 30, 2015

Regulatory Division (SPK-2015-00808)

Mr. Rob Burnett
California Dept of Transportation
1031 Butte Street, MS-30
Redding, California 96001

Dear Mr. Burnett:

We are responding to your September 17, 2015, request for a Department of the Army permit for the Tehama 36 Lassen Lodge Curve Realignment (EA 02-4G03U, PM 75.1-78.4) project. The approximately 3.2 mile long project site is located near South Fork Battle Creek, Latitude 40.3460601658214°, Longitude -121.707467267098°, Tehama County, California.

Based on the information you provided to this office, the Lassen Lodge Curve Realignment project involves the construction of culvert upgrades and replacements in accordance with the Sheets 1-13 of the Tehama 36 Preliminary plans dated August 17, 2015. The specific activities that require the discharge of dredged or fill material in waters of the United States are the extension or realignment of culverts for the realignment of Highway 36. These activities will result in the permanent loss of approximately 0.07 acres of intermittent stream and wetland seep habitat.

We have determined activities in waters of the U.S. associated with the project are authorized by Nationwide Permit Number (NWP) NWP 14 Linear Transportation Projects. However, this authorization is denied without prejudice until water quality certification under Section 401 of the Clean Water Act has been issued or waived for the activities requiring a permit from this office. Once you receive water quality certification or waiver thereof, the activities are authorized and the work may proceed subject to the any conditions of water quality certification, and the terms and conditions of the NWP.

You must comply with all terms and conditions of the NWP, applicable regional conditions, and project-specific special conditions. Information about the NWP and regional conditions are available on our website at *www.spk.usace.army.mil/Missions/Regulatory/Permitting/NationwidePermits.aspx*. In addition, your work must comply with the following special conditions:

1. To mitigate for the loss of 0.07 acres of intermittent stream and seasonal wetland habitat, you shall purchase 0.14 aquatic resource credits from the National Fish and Wildlife Foundation's (NFWF) Sacramento District California In-Lieu Fee Program for the Northeast Sacramento River Service Area. Contact information for NFWF can be found on their website at: www.nfwf.org/ilf. Evidence of this purchase shall be provided to the Corps prior to initiation of construction activities within waters of the U.S.

2. Within 30 days after completion of the authorized work, you must sign the enclosed Compliance Certification and return it to this office.

This verification is valid until March 18, 2017, when the existing NWPs are scheduled to be modified, reissued, or revoked. Furthermore, if you commence or are under contract to commence this activity before the date the NWP is modified, reissued, or revoked, you will have 12 months from the date of the modification, reissuance or revocation to complete the activity under the present terms and conditions. Failure to comply with the general and regional conditions of this NWP, or the project-specific special conditions of this authorization, may result in the suspension or revocation of your authorization.

We would appreciate your feedback on this permit action including your interaction with our staff. At your earliest convenience, please tell us how we are doing by completing the Corps' Regulatory Program national customer service survey found on our website at *www.spk.usace.army.mil/Missions/Regulatory.aspx.*

Please refer to identification number SPK-2015-00808 in any correspondence concerning this project. If you have any questions, please contact Matthew Kelley at Redding Regulatory Office, 310 Hemsted Drive, Suite 310, Redding, California 96002, by email at *Matthew.P.Kelley@usace.army.mil*, or telephone at 530-223-9534.

Sincerely,

Matthew Kelley
Chief, Redding Regulatory Office

Enclosures

cc: (w/o encls)

Ms. Leana Rosetti, U.S. Environmental Protection Agency, Rosetti.Leana@epa.gov
Mr. Scott Zaitz, Regional Water Quality Control Board, Scott.Zaitz@waterboards.ca.gov

COMPLIANCE CERTIFICATION

Permit File Name: TEH036 Lassen Lodge Curve Realignment (EA 02-4G03U, PM 75.1-78.4)

Action ID: SPK-2015-00808

Nationwide Permit Number: NWP 14 Linear Transportation Projects.

Permittee: Mr. Rob Burnett
California Department of Transportation
1031 Butte Street, MS-30
Redding, California 96001

County: Tehama

Date of Verification: October 30, 2015

Within 30 days after completion of the activity authorized by this permit, sign this certification and return it to the following address:

U.S. Army Corps of Engineers
Sacramento District
310 Hemsted Drive, Suite 310
Redding, California 96002
DLL-CESPK-RD-Compliance@usace.army.mil

Please note that your permitted activity is subject to a compliance inspection by a U.S. Army Corps of Engineers representative. If you fail to comply with the terms and conditions of the permit your authorization may be suspended, modified, or revoked. If you have any questions about this certification, please contact the U.S. Army Corps of Engineers.

* * * * * * * * *

I hereby certify that the work authorized by the above-referenced permit, including all the required mitigation, was completed in accordance with the terms and conditions of the permit verification.

_____ _____
Permittee Signature Date

PRELIMINARY JURISDICTIONAL DETERMINATION FORM
Sacramento District

This preliminary JD finds that there *"may be"* waters of the United States on the subject project site, and identifies all aquatic features on the site that could be affected by the proposed activity, based on the following information:

Regulatory Branch: **California North** File/ORM #: **SPK-2015-00808** PJD Date: **October 30, 2015**

State: **CA** City/County: **Tehama County**
Nearest Waterbody: **South Fork Battle Creek**

Location (Lat/Long): **40.3460601658214°, -121.707467267098°**

Size of Review Area: **3.2 miles** acres

Name/Address Of Property Owner/ Potential Applicant: **Attn: Rob Burnett**
Attn: Mr. State of California Department of Transportation, District 2
1031 Butte Street, MS-30
Redding, California 96001

Identify (Estimate) Amount of Waters in the Review Area

Non-Wetland Waters:
linear feet ft wide **0.095** acre(s)
Stream Flow: **Intermittent**

Wetlands: **0.005** acre(s)
Cowardin Class: **Palustrine, emergent**

Name of any Water Bodies on the site identified as Section 10 Waters: Tidal: Non-Tidal:

☒ Office (Desk) Determination
☐ Field Determination:
Date(s) of Site Visit(s):

SUPPORTING DATA: Data reviewed for preliminary JD (check all that apply – checked items should be included in case file and, where checked and requested, appropriately reference sources below)

☒ Maps, plans, plots or plat submitted by or on behalf of the applicant/consultant:
☐ Data sheets prepared/submitted by or on behalf of the applicant/consultant.
☐ Data sheets prepared by the Corps.
☐ Corps navigable waters' study.
☒ U.S. Geological Survey Hydrologic Atlas:
  ☒ USGS NHD data.
  ☐ USGS HUC maps.
☒ U.S. Geological Survey map(s). Cite scale & quad name: **1:24K; CA-LYONSVILLE**
☐ USDA Natural Resources Conservation Service Soil Survey.
☐ National wetlands inventory map(s).
☐ State/Local wetland inventory map(s).
☐ FEMA/FIRM maps.
☐ 100-year Floodplain Elevation (if known):
☒ Photographs: ☒ Aerial
  ☐ Other
☐ Previous determination(s). File no. and date of response letter:
☐ Other information (please specify):

IMPORTANT NOTE: The information recorded on this form has not necessarily been verified by the Corps and should not be relied upon for later jurisdictional determinations.

10-30-15

Signature and Date of Regulatory Project Manager (REQUIRED)

Signature and Date of Person Requesting Preliminary JD (REQUIRED, unless obtaining the signature is impracticable)

EXPLANATION OF PRELIMINARY AND APPROVED JURISDICTIONAL DETERMINATIONS:

1. The Corps of Engineers believes that there may be jurisdictional waters of the United States on the subject site, and the permit applicant or other affected party who requested this preliminary JD is hereby advised of his or her option to request and obtain an approved jurisdictional determination (JD) for that site. Nevertheless, the permit applicant or other person who requested this preliminary JD has declined to exercise the option to obtain an approved JD in this instance and at this time.

2. In any circumstance where a permit applicant obtains an individual permit, or a Nationwide General Permit (NWP) or other general permit verification requiring "preconstruction notification" (PCN), or requests verification for a non-reporting NWP or other general permit, and the permit applicant has not requested an approved JD for the activity, the permit applicant is hereby made aware of the following: (1) the permit applicant has elected to seek a permit authorization based on a preliminary JD, which does not make an official determination of jurisdictional waters; (2) that the applicant has the option to request an approved JD before accepting the terms and conditions of the permit authorization, and that basing a permit authorization on an approved JD could possibly result in less compensatory mitigation being required or different special conditions; (3) that the applicant has the right to request an individual permit rather than accepting the terms and conditions of the NWP or other general permit authorization; (4) that the applicant can accept a permit authorization and thereby agree to comply with all the terms and conditions of that permit, including whatever mitigation requirements the Corps has determined to be necessary; (5) that undertaking any activity in reliance upon the subject permit authorization without requesting an approved JD constitutes the applicant's acceptance of the use of the preliminary JD, but that either form of JD will be processed as soon as is practicable; (6) accepting a permit authorization (e.g., signing a proffered individual permit) or undertaking any activity in reliance on any form of Corps permit authorization based on a preliminary JD constitutes agreement that all wetlands and other water bodies on the site affected in any way by that activity are jurisdictional waters of the United States, and precludes any challenge to such jurisdiction in any administrative or judicial compliance or enforcement action, or in any administrative appeal or in any Federal court; and (7) whether the applicant elects to use either an approved JD or a preliminary JD, that JD will be processed as soon as is practicable. Further, an approved JD, a proffered individual permit (and all terms and conditions contained therein), or individual permit denial can be administratively appealed pursuant to 33 C.F.R. Part 331, and that in any administrative appeal, jurisdictional issues can be raised (see 33 C.F.R. 331.5(a)(2)). If, during that administrative appeal, it becomes necessary to make an official determination whether CWA jurisdiction exists over a site, or to provide an official delineation of jurisdictional waters on the site, the Corps will provide an approved JD to accomplish that result, as soon as is practicable.

NOTIFICATION OF ADMINISTRATIVE APPEAL OPTIONS AND PROCESS AND REQUEST FOR APPEAL

Applicant: Mr. Rob Burnett, California Dept of Transportation:	File No.: SPK-2015-00808	Date: October 30, 2015
Attached is:		See Section below
	INITIAL PROFFERED PERMIT (Standard Permit or Letter of permission)	A
	PROFFERED PERMIT (Standard Permit or Letter of permission)	B
	PERMIT DENIAL	C
	APPROVED JURISDICTIONAL DETERMINATION	D
✓	PRELIMINARY JURISDICTIONAL DETERMINATION	E

SECTION I - The following identifies your rights and options regarding an administrative appeal of the above decision. Additional information may be found at *http://www.usace.army.mil/cecw/pages/reg_materials.aspx* or Corps regulations at 33 CFR Part 331.

A: INITIAL PROFFERED PERMIT: You may accept or object to the permit.

- ACCEPT: If you received a Standard Permit, you may sign the permit document and return it to the district engineer for final authorization. If you received a Letter of Permission (LOP), you may accept the LOP and your work is authorized. Your signature on the Standard Permit or acceptance of the LOP means that you accept the permit in its entirety, and waive all rights to appeal the permit, including its terms and conditions, and approved jurisdictional determinations associated with the permit.

- OBJECT: If you object to the permit (Standard or LOP) because of certain terms and conditions therein, you may request that the permit be modified accordingly. You must complete Section II of this form and return the form to the district engineer. Your objections must be received by the district engineer within 60 days of the date of this notice, or you will forfeit your right to appeal the permit in the future. Upon receipt of your letter, the district engineer will evaluate your objections and may: (a) modify the permit to address all of your concerns, (b) modify the permit to address some of your objections, or (c) not modify the permit having determined that the permit should be issued as previously written. After evaluating your objections, the district engineer will send you a proffered permit for your reconsideration, as indicated in Section B below.

B: PROFFERED PERMIT: You may accept or appeal the permit

- ACCEPT: If you received a Standard Permit, you may sign the permit document and return it to the district engineer for final authorization. If you received a Letter of Permission (LOP), you may accept the LOP and your work is authorized. Your signature on the Standard Permit or acceptance of the LOP means that you accept the permit in its entirety, and waive all rights to appeal the permit, including its terms and conditions, and approved jurisdictional determinations associated with the permit.

- APPEAL: If you choose to decline the proffered permit (Standard or LOP) because of certain terms and conditions therein, you may appeal the declined permit under the Corps of Engineers Administrative Appeal Process by completing Section II of this form and sending the form to the division engineer (address on reverse). This form must be received by the division engineer within 60 days of the date of this notice.

C: PERMIT DENIAL: You may appeal the denial of a permit under the Corps of Engineers Administrative Appeal Process by completing Section II of this form and sending the form to the division engineer (address on reverse). This form must be received by the division engineer within 60 days of the date of this notice.

D: APPROVED JURISDICTIONAL DETERMINATION: You may accept or appeal the approved JD or provide new information.

- ACCEPT: You do not need to notify the Corps to accept an approved JD. Failure to notify the Corps within 60 days of the date of this notice, means that you accept the approved JD in its entirety, and waive all rights to appeal the approved JD.

- APPEAL: If you disagree with the approved JD, you may appeal the approved JD under the Corps of Engineers Administrative Appeal Process by completing Section II of this form and sending the form to the division engineer (address on reverse). This form must be received by the division engineer within 60 days of the date of this notice.

E: PRELIMINARY JURISDICTIONAL DETERMINATION: You do not need to respond to the Corps regarding the preliminary JD. The Preliminary JD is not appealable. If you wish, you may request an approved JD (which may be appealed), by contacting the Corps district for further instruction. Also you may provide new information for further consideration by the Corps to reevaluate the JD.

SECTION II - REQUEST FOR APPEAL or OBJECTIONS TO AN INITIAL PROFFERED PERMIT

REASONS FOR APPEAL OR OBJECTIONS: (Describe your reasons for appealing the decision or your objections to an initial proffered permit in clear concise statements. You may attach additional information to this form to clarify where your reasons or objections are addressed in the administrative record.)

ADDITIONAL INFORMATION: The appeal is limited to a review of the administrative record, the Corps memorandum for the record of the appeal conference or meeting, and any supplemental information that the review officer has determined is needed to clarify the administrative record. Neither the appellant nor the Corps may add new information or analyses to the record. However, you may provide additional information to clarify the location of information that is already in the administrative record.

POINT OF CONTACT FOR QUESTIONS OR INFORMATION:

If you have questions regarding this decision and/or the appeal process you may contact:

Matthew Kelley
Chief, Redding Regulatory Office
U.S. Army Corps of Engineers
310 Hemsted Drive Suite 310
Redding, California 96002
Phone: 530-223-9534, FAX 916-557-7803
Email: Matthew.P.Kelley@usace.army.mil

If you only have questions regarding the appeal process you may also contact:

Thomas J. Cavanaugh
Administrative Appeal Review Officer
U.S. Army Corps of Engineers
South Pacific Division
1455 Market Street, 2052B
San Francisco, California 94103-1399
Phone: 415-503-6574, FAX 415-503-6646)
Email: Thomas.J.Cavanaugh@usace.army.mil

RIGHT OF ENTRY: Your signature below grants the right of entry to Corps of Engineers personnel, and any government consultants, to conduct investigations of the project site during the course of the appeal process. You will be provided a 15 day notice of any site investigation, and will have the opportunity to participate in all site investigations.

Signature of appellant or agent.	Date:	Telephone number:

INFORMATION HANDOUT

For Contract No. 02-4G03u4
At Teh-36-R75.1/78.4

Identified by
Project ID 0215000056

WATER QUALITY

California Regional Water Quality Control Board
Central Valley Region 401 Certification

Dated November 4, 2015

04 November 2015

Mr. Matt Mitchell
Caltrans
1657 Riverside Drive
Redding, CA 96001

CLEAN WATER ACT §401 TECHNICALLY CONDITIONED WATER QUALITY CERTIFICATION FOR DISCHARGE OF DREDGED AND/OR FILL MATERIALS FOR THE LASSEN LODGE CURVE REALIGNMENT PROJECT (WDID#5A52CR00143), CHILDS MEADOW, TEHAMA COUNTY

ACTION:

1. ☐ Order for Standard Certification
2. ■ Order for Technically-conditioned Certification
3. ☐ Order for Denial of Certification

WATER QUALITY CERTIFICATION STANDARD CONDITIONS:

1. This certification action is subject to modification or revocation upon administrative or judicial review, including review and amendment pursuant to §13330 of the California Water Code and §3867 of Title 23 of the California Code of Regulations (23 CCR).

2. This certification action is not intended and shall not be construed to apply to any discharge from any activity involving a hydroelectric facility requiring a Federal Energy Regulatory Commission (FERC) license or an amendment to a FERC license unless the pertinent certification application was filed pursuant to 23 CCR subsection 3855(b) and the application specifically identified that a FERC license or amendment to a FERC license for a hydroelectric facility was being sought.

3. The validity of any non-denial certification action shall be conditioned upon total payment of the full fee required under 23 CCR §3833, unless otherwise stated in writing by the certifying agency.

4. Certification is valid for the duration of the described project. Caltrans shall notify the Central Valley Water Board in writing within 7 days of project completion.

ADDITIONAL TECHNICALLY CONDITIONED CERTIFICATION CONDITIONS:

In addition to the four standard conditions, Caltrans shall satisfy the following:

1. Caltrans shall notify the Central Valley Water Board in writing 7 days in advance of the start of any in-water activities.

2. Except for activities permitted by the U.S. Army Corps under §404 of the Clean Water Act, soil, silt, or other organic materials shall not be placed where such materials could pass into surface water or surface water drainage courses.

3. All areas disturbed by project activities shall be protected from washout or erosion.

4. Caltrans shall maintain a copy of this Certification and supporting documentation (Project Information Sheet) at the Project site during construction for review by site personnel and agencies. All personnel (employees, contractors, and subcontractors) performing work on the proposed project shall be adequately informed and trained regarding the conditions of this Certification.

5. An effective combination of erosion and sediment control Best Management Practices (BMPs) must be implemented and adequately working during all phases of construction.

6. All temporarily affected areas will be restored to pre-construction contours and conditions upon completion of construction activities.

7. Caltrans shall perform surface water sampling: 1) When performing any in-water work; 2) In the event that project activities result in any materials reaching surface waters or; 3) When any activities result in the creation of a visible plume in surface waters. The following monitoring shall be conducted immediately upstream out of the influence of the project and 300 feet downstream of the active work area. Sampling results shall be submitted to this office within two weeks of initiation of sampling and every two weeks thereafter. The sampling frequency may be modified for certain projects with written permission from the Central Valley Water Board.

Parameter	Unit	Type of Sample	Frequency of Sample
Turbidity	NTU	Grab	Every 4 hours during in water work
Settleable Material	ml/l	Grab	Same as above.
Visible construction related pollutants	Observations	Visible Inspections	Continuous throughout the construction period

8. Activities shall not cause turbidity increases in surface water to exceed:

   (a) where natural turbidity is less than 1 Nephelometric Turbidity Units (NTUs), controllable factors shall not cause downstream turbidity to exceed 2 NTU;
   (b) where natural turbidity is between 1 and 5 NTUs, increases shall not exceed 1 NTU;
   (c) where natural turbidity is between 5 and 50 NTUs, increases shall not exceed 20 percent;
   (d) where natural turbidity is between 50 and 100 NTUs, increases shall not exceed 10 NTUs;
   (e) where natural turbidity is greater than 100 NTUs, increases shall not exceed 10 percent.

   Except that these limits will be eased during in-water working periods to allow a turbidity increase of 15 NTU over background turbidity as measured in surface waters 300 feet downstream from the working area. In determining compliance with the above limits, appropriate averaging periods may be applied provided that beneficial uses will be fully protected. Averaging periods may only be assessed by prior permission of the Central Valley Water Board.

9. Activities shall not cause settleable matter to exceed 0.1 ml/l in surface waters as measured in surface waters 300 feet downstream from the project.

10. The discharge of petroleum products or other excavated materials to surface water is prohibited. Activities shall not cause visible oil, grease, or foam in the work area or downstream. Caltrans shall notify the Central Valley Water Board immediately of any spill of petroleum products or other organic or earthen materials.

11. Caltrans shall notify the Central Valley Water Board immediately if the above criteria for turbidity, settleable matter, oil/grease, or foam are exceeded.

12. Caltrans shall comply with all Department of Fish and Wildlife 1600 requirements for the project.

13. Caltrans must obtain coverage under the NPDES General Permit for Storm Water Discharges Associated with Construction and Land Disturbance Activities issued by the State Water Resources Control Board for any project disturbing an area of 1 acre or greater.

14. The Conditions in this water quality certification are based on the information in the attached "Project Information." If the information in the attached Project Information is modified or the project changes, this water quality certification is no longer valid until amended by the Central Valley Water Board.

15. In the event of any violation or threatened violation of the conditions of this Order, the violation or threatened violation shall be subject to any remedies, penalties, process, or sanctions as provided for under State law and section 401 (d) of the federal Clean Water Act. The applicability of any State law authorizing remedies, penalties, process, or

sanctions for the violation or threatened violation constitutes a limitation necessary to ensure compliance into this Order.

a. If Caltrans or a duly authorized representative of the project fails or refuses to furnish technical or monitoring reports, as required under this Order, or falsifies any information provided in the monitoring reports, the applicant is subject to civil monetary liabilities, for each day of violation, or criminal liability.

b. In response to a suspected violation of any condition of this Order, the Central Valley Water Board may require Caltrans to furnish, under penalty of perjury, any technical or monitoring reports the Central Valley Water Board deems appropriate, provided that the burden, including cost of the reports, shall be in reasonable relationship to the need for the reports and the benefits to be obtained from the reports.

c. Caltrans shall allow the staff(s) of the Central Valley Water Board, or an authorized representative(s), upon the presentation of credentials and other documents, as may be required by law, to enter the project premises for inspection, including taking photographs and securing copies of project-related records, for the purpose of assuring compliance with this certification and determining the ecological success of the project.

16. Staff of the Central Valley Water Board has prepared total maximum daily load (TMDL) allocations that, once approved, would limit methylmercury in storm water discharges to the Sacramento-San Joaquin Delta. The Central Valley Water Board has scheduled these proposed allocations to be considered for adoption. When the Central Valley Water Board adopts the TMDL and once approved by the Environmental Protection Agency, the discharge of methylmercury may be limited from the proposed project. The purpose of this condition is to provide notice to Caltrans that methylmercury discharge limitations and monitoring requirements may apply to this project in the future and also to provide notice of the Central Valley Water Board's TMDL process and that elements of the planned construction may be subject to a TMDL allocation.

ADDITIONAL STORM WATER QUALITY CONDITIONS:

Caltrans shall also satisfy the following additional storm water quality conditions:

1. During the construction phase, Caltrans must employ strategies to minimize erosion and the introduction of pollutants into storm water runoff. These strategies must include the following:

   (a) the Storm Water Pollution Prevention Plan (SWPPP) must be prepared during the project planning and design phases and before construction;
   (b) an effective combination of erosion and sediment control Best Management Practices (BMPs) must be implemented and adequately working prior to the rainy season and during all phases of construction.

2. Caltrans must minimize the short and long-term impacts on receiving water quality from the Lassen Lodge Curve Realignment Project by implementing the following post-construction storm water management practices:
   (a) minimize the amount of impervious surface;
   (b) reduce peak runoff flows;
   (c) provide treatment BMPs to reduce pollutants in runoff;
   (d) ensure existing waters of the State (e.g., wetlands, vernal pools, or creeks) are not used as pollutant source controls and/or treatment controls;
   (e) preserve and, where possible, create or restore areas that provide important water quality benefits, such as riparian corridors, wetlands, and buffer zones;
   (f) limit disturbances of natural water bodies and natural drainage systems caused by development (including development of roads, highways, and bridges);
   (g) use existing drainage master plans or studies to estimate increases in pollutant loads and flows resulting from projected future development and require incorporation of structural and non-structural BMPs to mitigate the projected pollutant load increases in surface water runoff;
   (h) identify and avoid development in areas that are particularly susceptible to erosion and sediment loss, or establish development guidance that protects areas from erosion/ sediment loss;
   (i) control post-development peak storm water run-off discharge rates and velocities to prevent or reduce downstream erosion, and to protect stream habitat.

3. Caltrans must ensure that all development within the project provides verification of maintenance provisions for post-construction structural and treatment control BMPs. Verification shall include one or more of the following, as applicable:

   (a) the developer's signed statement accepting responsibility for maintenance until the maintenance responsibility is legally transferred to another party; or
   (b) written conditions in the sales or lease agreement that require the recipient to assume responsibility for maintenance; or
   (c) written text in project conditions, covenants and restrictions for residential properties assigning maintenance responsibilities to a home owner's association, or other appropriate group, for maintenance of structural and treatment control BMPs; or
   (d) any other legally enforceable agreement that assigns responsibility for storm water BMP maintenance.

REGIONAL WATER QUALITY CONTROL BOARD CONTACT PERSON:

Scott A. Zaitz, R.E.H.S., Redding Branch Office, 364 Knollcrest Drive, Suite 205, Redding, California 96002, Scott.Zaitz@waterboards.ca.gov, (530) 224-4784

WATER QUALITY CERTIFICATION:

I hereby issue an order certifying that any discharge from Caltrans, Lassen Lodge Curve Realignment Project (WDID# 5A52CR00143) will comply with the applicable provisions of §301 ("Effluent Limitations"), §302 ("Water Quality Related Effluent Limitations"), §303 ("Water Quality Standards and Implementation Plans"), §306 ("National Standards of Performance"),

and §307 ("Toxic and Pretreatment Effluent Standards") of the Clean Water Act. This discharge is also regulated under State Water Resources Control Board Water Quality Order No. 2003-0017 DWQ "Statewide General Waste Discharge Requirements For Dredged Or Fill Discharges That Have Received State Water Quality Certification (General WDRs)."

Except insofar as may be modified by any preceding conditions, all certification actions are contingent on (a) the discharge being limited and all proposed mitigation being completed in strict compliance with Caltrans's project description and the attached Project Information Sheet, and (b) compliance with all applicable requirements of the Water Quality Control Plan *for the Sacramento River and San Joaquin River*, Fourth Edition, revised October 2011 (Basin Plan).

Any person aggrieved by this action may petition the State Water Quality Control Board to review the action in accordance with California Water Code § 13320 and California Code of Regulations, title 23, § 2050 and following. The State Water Quality Control Board must receive the petition by 5:00 p.m., 30 days after the date of this action, except that if the thirtieth day following the date of this action falls on a Saturday, Sunday, or state holiday, the petition must be received by the State Water Quality Control Board by 5:00 p.m. on the next business day. Copies of the law and regulations applicable to filing petitions may be found on the Internet at: http://www.waterboards.ca.gov/public_notices/petitions/water_quality or will be provided upon request.

(for) Pamela C. Creedon
Executive Officer

SAZ:wrb:reb

Enclosure: Water Quality Order No. 2003-0017 DWQ

cc w/o enclosures: Mr. Matt Kelley, U.S. Army Corp of Engineers, Redding
Ms. Donna Cobb, Department of Fish and Wildlife, Region 1, Redding
U.S. Fish and Wildlife Service, Sacramento
Mr. Bill Jennings, CALSPA, Stockton

cc w/o enclosures by email: U.S. EPA, Region 9, San Francisco
Mr. Bill Orme, SWRCB, Certification Unit, Sacramento

PROJECT INFORMATION

Application Date: 28 September 2015

Application Complete Date: 3 November 2015

Applicant: Caltrans, Attn: Mr. Matt Mitchell

Project Name: Lassen Lodge Curve Realignment Project

Application Number: WDID No. 5A52CR00143

U.S. Army Corps File Number: SPK-2015-00808

Type of Project: Highway curve realignment.

Project Location: State Route 36E Post Mile 75.1 to 78.4.
Latitude: 40°20'45" and Longitude: -121°42'26"

County: Tehama County

Receiving Water(s) (hydrologic unit): Hillside seeps and unnamed intermittent drainages, which are tributary to Sacramento River. Whitmore Hydrologic Unit-Battle Creek Hydrologic Subarea No. 507.12

Water Body Type: Streambed

Designated Beneficial Uses: The Water Quality Control Plan *for the Sacramento River and San Joaquin River*, Fourth Edition, revised September 2009, has designated beneficial uses for surface and ground waters within the region. Beneficial uses that could be impacted by the project include: Agricultural Supply (AGR); Industrial Supply (IND), Hydropower Generation (POW); Groundwater Recharge, Water Contact Recreation (REC-1); Non-Contact Water Recreation (REC-2); Cold Freshwater Habitat (COLD); Migration of Aquatic Organisms (MIGR); Spawning, Reproduction, and /or Early Development (SPWN); and Wildlife Habitat (WILD).

Project Description (purpose/goal): The Lassen Lodge Curve Realignment Project consists of a highway curve realignment project intended to reduce the frequency and severity of traffic accidents between post miles 75.1 and 78.4 on State Route 36E in Tehama County. This area experiences three times the accident rate compared to similar routes. The proposed project will improve roadway geometrics by realigning the roadway. At various locations, portions of the adjacent hillside will either be cut away or fill slopes will be constructed to accommodate critical structural support for the realignment. The placement of fill requires the re-routing or relocation of four U.S. Clean Water Act jurisdictional features (two seeps and three intermittent drainages). The area is very steep, and therefore structural fill will be extensive. Placing the intermittent drainages in culverts within the current natural flow path is infeasible as culverts would need to be placed deep in the fill profile, which creates maintenance difficulties.

Preliminary Water Quality Concerns: Construction activities may impact surface waters with increased turbidity and settleable matter.

Proposed Mitigation to Address Concerns: Caltrans will implement Best Management Practices (BMPs) to control sedimentation and erosion. All temporary affected areas will be restored to pre-construction contours and conditions upon completion of construction activities. Caltrans will conduct turbidity and settleable matter testing during in-water work, stopping work if Basin Plan criteria are exceeded or are observed.

Fill/Excavation Area Project implementation will permanently impact 0.066 acre/593 linear feet of un-vegetated streambed and temporarily impact 0.005 acre/45 linear feet of un-vegetated streambed.

Dredge Volume: 3 cubic yards of soil.

U.S. Army Corps of Engineers Permit Number: Nationwide Permit #14 (Linear Transportation Projects)

Department of Fish and Wildlife Streambed Alteration Agreement: Based on a determination reached by the Department of Fish and Wildlife a Streambed Alteration Agreement is not required.

Possible Listed Species: None

Status of CEQA Compliance: Caltrans signed a Categorical Exclusion Determination Form on 31 July 2015 stating the project is Categorically Exempt pursuant to Class 1 (PRC 21084; 14 CCR 15300 et seq.).

Compensatory Mitigation: The Central Valley Water Board is not requesting compensatory mitigation.

Application Fee Provided: On 28 September 2015 a certification application fee of $9,180.00 was submitted as required by 23 CCR §3833b(3)(A) and by 23 CCR §2200(e).

INFORMATION HANDOUT

For Contract No. 02-4G03u4
At Teh-36-R75.1/78.4

Identified by
Project ID 0215000056

MATERIALS INFORMATION

Geotechnical Report

Dated June 10, 2016

GEOTECHNICAL DESIGN REPORT

Lassen Lodge Curve Improvement Safety Project

State Highway 36

02-TEH-36-PM 75.1/78.4
EFIS ID: 0215000056
EA# 02-4G03U

PREPARED FOR DISTRICT 02
SAFETY DESIGN TEAM
DESIGN R-6

By

J. Scott Lewis P.G., C.E.G., R.G.P.
Associate Engineering Geologist

Shawn Wei P.E.
Senior transportation Engineer

DIVISION of ENGINEERING SERVICES-GEOTECHNICAL SERVICES
OFFICE of GEOTECHNICAL DESIGN-NORTH
June 10, 2016

State of California
Department of Transportation

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: **MR. AL TRUJILLO**
District 2 Safety Team Senior Engineer

Date: June 10, 2016

File: 02-TEH-36-PM 75.1/78.4
EFIS ID: 0215000056
EA 02-4G03U
Lassen Lodge Safety Project

Attn: Mr. Shaun Alexander
Transportation Engineer

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services

Subject: Geotechnical Design Report for Lassen Lodge Curve Improvement Safety Project

Per your request, we are providing a Geotechnical Design Report for the Lassen Lodge Curve Improvement Safety Project on State Highway 36 from PM 75.4 to PM 78.4 in Tehama County, California. This report defines the geotechnical conditions used in the development of the geotechnical design as evaluated from field observations and measurements, laboratory test data, and geophysical surveys. It provides recommendations and specifications for project design and construction.

Specific geotechnical aspects of this project that are addressed in this report include cut slopes, fill embankments, zoning of fills, select material, and project risks resulting from geotechnical conditions and project constraints. This project involves no structures.

If you have any questions or require further assistance, please call Mr. Lewis at (530) 225-3516.

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1. Introduction

This Geotechnical Design Report (GDR) is for the Lassen Lodge Realignment and Widening Safety Project on State Highway 36 from PM 75.4 to 78.4 in Tehama County, California. Plate 1 presents a map showing the project location. Plate 2 presents an aerial view of the entire project site with the locations of seismic lines and lab samples. Plate 3 presents a topographic map of the project area and nearby surroundings. For ease of discussion and reference in this report, the project area is separated into western and eastern halves, with the small cluster of structures of the Lassen Lodge area located around station 1645+00 being the dividing point.

The Tehama County Wagon Road, predecessor to the present day highway 36 route in this area, was first created in 1862 to facilitate access to the gold rush in Idaho. The following year it became established as part of the Red Bluff to Susanville wagon-stagecoach road. The presence of water from several springs in the area led to the Lassen Lodge area becoming a convenient rest stop first for cattle drives and later for auto travelers in the 1920's when car camping began to be popular and the route was first transformed into an actual roadway, albeit unpaved. With the official establishment of Lassen Lodge in 1939, traffic to and through the area increased significantly and the highway was improved. Improvements to the highway, including pavement and realignment work, was performed between 1956 and 1958. Additional improvements were performed in the 1970's.

2. Existing Facilities and Proposed Improvements

At the time of our investigation Highway 36 within the project area consisted primarily of a 2-lane roadway with a few small pullouts, little to no shoulders, and no passing lanes.

Existing cut slopes in the western half of the project area have slope ratios varying from about 1:1 to 1.2:1, with heights reaching no more than about 20 feet (ft). Existing fills in the west are predominantly 1.5:1 with heights no greater than about 25 ft. Proposed cuts in the west have slope ratios of 1:1 with heights up to 25 ft. Proposed fills have slope ratios of 2:1 and 1.5:1, with a maximum height of about 25 ft.

Existing cut and fill slopes in the eastern half of the project area are generally steeper and higher than those in the western half. Existing cut slope ratios vary from about 1:1 to about 0.4:1, with the most prevalent ratio standing somewhere around 0.8:1 and the maximum height extending up to about 70 ft. A majority of the existing fill slopes in the eastern project area stand at approximately 1.25:1 and have a maximum height of about 35 ft. Other existing fill slopes vary between 2:1 and 1.5:1. The majority of proposed cuts in the eastern project area have slope ratios of 0.75:1 with heights up to 90 ft. Lesser amounts of cuts with slope ratios of 0.5:1 and maximum heights of about 60 ft are also proposed, these being located in hard andesite and basalt outcrops.

Proposed fills in the eastern project area have slope ratios of either 1.55:1 or 1.25:1. Three considerably large (height and volume) fills are proposed for the eastern end of the eastern project area with slope ratios of 1.55:1 and heights of about 150 ft, 90 ft, and 60 ft, respectively. These large fills have some special design and construction criteria that involve zoning of material within

the fills in order to increase stability and provide a high level of erosion protection. Lesser amounts of fills with 1.25:1 slope ratios with heights up to about 35 ft are proposed primarily as sliver fills in order to provide widening to the roadway in areas where existing 1.25:1 fill slopes restrict such widening. Due to the relative steepness and sliver nature of most of these fills, special design and material criterion are called for.

The proposed alignment improvements generally follow the existing route, with some curves being eliminated entirely and others being smoothed (wider radius). Smoothing is achieved typically by pushing the roadway alignment deeper into existing cut slopes and by moving the roadway alignment over fills that bridge valleys. Besides smoothing the alignment, these new fills and cuts are also used to create additional space for wider shoulders and new passing lanes at both ends of the project.

3. Pertinent Reports and Investigations

This report includes a review of Caltrans, state, federal, and private publications. A search on the Caltrans Bridge Inspection Records Information System (BIRIS) Site yielded no information considered pertinent to the project investigation or report. A search on the Caltrans Intranet Document Retrieval System (DRS) site yielded As-Builts and Plans that were reviewed for information pertinent to this project.

Caltrans literature, tools, and websites reviewed and/or utilized pertaining to seismic issues include the *Caltrans Seismic Design Criteria website, Appendix B*, the *Caltrans Fault Database* (Merriam, 2009), and the internal Caltrans website for calculating acceleration response spectra (ARS) curves at http://dap3.dot.ca.gov/ARS_Online/index.php.

Geologic literature reviewed include the *Geologic Map of California, Westwood Sheet* (Lydon, Gay, and Jennings, 1960), the *Fault Activity Map of California and Adjacent Areas* (Jennings, 1994),

Soil information was obtained from the Natural Resources Conservation Service (NRCS) Web Soil Survey Website (http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx) and the Soil Survey of Tehama County, California (1967).

4. Physical Setting

The physical setting of the project and the surrounding area was reviewed to provide information that might aid the Design, Construction, and Environmental Offices on climate, topography, drainage, and man-made and natural features. The project work is located on the gently sloping western limb of the southern Cascade volcanic fields on State Highway 36 at an elevation ranging from about 3800 ft above mean sea level (western limit of project) to about 4300 ft above mean sea level (eastern limit of project).

4.1. Climate

Climate information was obtained from the Western Regional Climate Data Center (http://www.wrcc.dri.edu/) for the weather station located at Mineral, which is located approximately 10 miles east of the project area at an elevation of about 4900 ft above mean sea level, which is about 900 ft higher than the average elevation of the project area. Data is for the period of record from 1909 to 2015. The average annual precipitation is about 53 inches, and the average annual snowfall is about 152 inches. The majority of the precipitation falls between October and May; the snowfall between November and April. The average annual maximum and minimum air temperatures are 59.2°F and 31.5°F, respectively. The average monthly extremes are 21.6°F in January and 81.5°F in July.

4.2. Topography and Drainage

The present highway climbs gradually but steadily as it moves eastward (up station) through the western half of the project area to the Lassen Lodge location in the middle of the project. Elevation remains fairly level, with relatively small variations in the vertical roadway profile no greater than 20 ft. The highway is notched into the slopes above and to the south of the South Fork of Battle Creek, which roughly parallels the roadway about half a mile to the north. Throughout the entire project the highway generally follows a course atop high ground between the watershed basins of the South Fork of Battle Creek to the north and Paynes Creek to the south. Moderate to gentle cross slopes that descend to the south exist above and below the road in the western half of the project. Moderately steep to steep cross slopes that descend to the north exist above and below the road in the eastern half of the project.

Surface drainage in the western half of the project flows crossways to the road, generally from north to south. Surface drainage in the eastern half of the project flows crossways to the road, generally from south to north. The topographic map in Plate 3 shows these surface flow orientations.

No perennial streams exist within the boundaries of the project area. One intermittent stream flows through the project area, running from south to north, crossing the road at about station 1710+00 where it flows beneath the existing road, down the thalweg that lies beneath the center of proposed Fill 1 (see section 8.3), and down into the valley below where it enters the South Fork of Battle Creek, a perennial stream that flows to the west approximately 0.5 to 0.8 miles north of the project area. An intermittent tributary stream to Paynes Creek runs roughly parallel to the roadway for the entire length of the project at a distance of 0.1 to 0.5 miles to the south, though only flow in the western half of the project area enters this tributary. The headwaters of three other intermittent streams that flow into the South Fork of Battle Creek approach within a few hundred feet of the roadway. Over a dozen other topographic drainages run generally perpendicular to the length of the project and the roadway, drainages that likely only see surface flow during or shortly after large storm events. All drainages, however, act to some degree as funneling pathways for ground water

to travel down gradient to the South Fork of Battle Creek (eastern half of project), or the intermittent drainages of Paynes Creek to the south (western half of project area).

4.3. Man-made and Natural Features of Engineering and Construction Significance

Man-made features that may potentially have an impact on the project, or be impacted by the project, include drainage inlets and culverts, a horizontal drain that functions as a source of spring water to some locals (at approximately station 1679), a water pipeline located south of the existing highway above existing cuts in the eastern half of the project, and an overhead high-voltage transmission line that crosses the roadway overhead once in the western part of the project area at about station 1571+75 and twice in the eastern part of the project at about station 1663+00 and 1713+10. The water pipeline begins at approximately station 1712+00 above the roadway (to the south) in the drainage that flows south to north. This pipeline runs parallel to the roadway above the existing top of cut until it reaches the cluster of structures at the Lassen Lodge area located at the middle of the project.

4.4. Regional Geology and Seismicity

The project lies within the Cascade Range Geomorphic/Physiographic Province of California (CGS, 2002). The project lies on the western flanks of the volcanic pediment formed by Mt. Lassen, the southernmost volcano in the Cascades, and earlier predecessors to Lassen, particularly the ancient Mt. Tehama. Rocks within the region are composed of a wide suite of volcanic rocks including andesite, basalt, rhyolite, and various pyroclastics, dating from Miocene (approximately 23 million years ago (Mya) to 5 Mya) to Holocene (10,000 years ago to present) times.

No known faults are present within the project area. A cluster of short (less than 1 mile in length) faults that cut Pliocene (5.3 Mya to 2.6 Mya) rocks lies less than 5 miles to the west of the project. These faults are not considered to be active. They are most likely a set of cooling fractures that were created as the volcanics cooled. The nearest active fault is the Battle Creek Fault (Merriam, 2009; Jennings, 1996), which trends west-southwest / north-northeast and is located northwest of the project area about 10 miles.

4.5 Soil Survey Mapping

Four soil types from three different soil series, as classified by the USDA Soil Conservation Service (Soil Survey Tehama County California; Gowans, et al, 1967) comprise the soils in the project area. All soils are derived from volcanic parent material and are typically stony. The most common soil type, which covers a majority of the western half of the project, is the Lyonsville and Jiggs stony sandy loam (Soil Survey abbreviation: LyE) on 30% to 50% slopes. The second most prevalent soil is the Cohassett loam, very deep, on 10% to 30% slopes (Soil Survey abbreviation: CeD), which in typical section is 4 to 6 feet thick atop fractured bedrock. The Windy Rocky Sandy Loam (Soil Survey abbreviation: WnF) is present through the eastern fourth of the project. The last soil type is the Lyonsville and Jiggs Sandy Loam (Soil Survey abbreviation: LyF) on 50% to 65% slopes.

Based on these descriptions provided in the soil survey, together with field examination, these soils are categorized, according to the Caltrans Soil and Rock Classification Logging Manual (2013), predominantly as silty sand with gravel (SM), silty sand with gravel and cobbles (SM), and well-graded sand with silt and gravel and cobbles (SW-SM). Minor amounts of clay exist in some local areas resulting in well-graded sand with clay and well-graded sand with clay and gravel (SW-SC).

4.6 Naturally Occurring Asbestos (NOA)

Geologic units mapped (Lydon, Gay, and Jennings, 1960) in the project area are not known to typically harbor naturally occurring asbestos (NOA) deposits. According to the map contained within the report referenced by the State of California Air Resources Board (California Dept of Conservation, 2000), the project site is not mapped as an area likely to contain NOA. No native serpentine exists within the project area. Non-native serpentine (dumped or imported as fill material) was not observed anywhere within the project area.

5. Exploration

5.1 Drilling and Sampling

No borings were performed for this report.

Six surface samples were collected for laboratory testing from existing cut slopes within the project area, four bag samples and two tube samples. The locations of the sample sites are shown in Plate 2.

Dozens of additional hand samples were examined and assessed with field methods for gradation, angularity (of fines, gravel, and cobbles), and plasticity during field reconnaissance.

5.2. Geologic Mapping

A portion of the geologic map produced by Lydon, Gay, and Jennings (1960) that includes the project area and neighboring terrain is shown in Plate 4. An aerial photograph with the approximate boundaries of the same geologic units delineated is shown in Plate 5.

Analysis of aerial photos of the project area and nearby surroundings was performed prior to, during, and after field work.

Geologic reconnaissance and mapping of lithology and soil types was conducted along the road, above the existing cut faces, and in select locations below the road in the vicinity of proposed fills. Information obtained from these efforts, together with information garnered through a literature search, was plotted on aerial photographs taken from the Caltrans Digital Highway Inventory Photography Program (DHIPP) and Google Earth, as well as draft design layouts of the proposed realignment. This information has been incorporated into the design work and recommendations provided within this report.

5.3. Geophysical Studies

Eighteen seismic refraction survey lines were shot for this project for the evaluation of subsurface conditions in the areas of both proposed cuts and fills. Field work, processing, and report preparation for this work was performed under task order 83480 by Norcal Geophysical. Depth sections for these lines are shown in Appendix 13.2. Travel-time curves, velocity models, and depth sections for these lines are shown at the end of Appendix 13.2. The locations of all eighteen seismic refraction lines are shown on Plate 2.

Multichannel analysis of surface waves (MASW) was also performed at each refraction line location with the refraction layout in place. This method does not require any extra field time or field work, it merely requires that the shot records be recorded for a longer period of time so that the surface waves (primarily the Rayleigh waves) have time to fully sample the deeper material. The MASW technique does, however, require its own additional processing and analysis time. The result is a one-dimensional vertical profiling of the shear wave velocity of the subsurface material. Because refraction analysis is usually unable to see low-velocity layers or beneath them, MASW can be highly complementary to refraction analysis in areas like those in the project site where, due to the nature of typical volcanic terrain, the potential for low-velocity layers is significant. The 1-D V_s profiles for each seismic line are also provided in Appendix 13.2.

5.4 Instrumentation

No monitoring instrumentation was installed or utilized during the field investigation.

6. Geotechnical Testing

6.1 In situ Testing

No in-situ testing geotechnical testing was performed.

6.2 Laboratory Testing

Geotechnical laboratory tests performed include:
- Four corrosion (4 pH and 4 resistivity tests[Caltrans Test Method 643]). Sulphate testing was deemed unnecessary based on the initial results of the pH and resistivity tests.
- Ten moisture content tests (Caltrans Test Method 226)
- Six mechanical analyses (Caltrans Test Method 203)
- Six Atterberg Limits (Caltrans Test Method 204),
- Four compaction curve analyses (Caltrans Test Method 216)
- Eighteen consolidated undrained (CU) triaxial tests (as per ASTM D4767).

Mechanical analysis and Atterberg Limit testing were performed on each of the six samples (four bag samples and two tube samples) collected in the field. Three triaxial tests were performed per sample, so that Mohr's circles could be constructed and phi angles (φ) and

cohesion values (C) could be determined for each bag sample. The four compaction tests were performed to construct compaction curves for each of the four bag samples. Ten moisture density tests were performed for each of the tube samples (three tubes per sample equals six tests) and for each of the four bag samples. The triaxial tests were all performed at the same three confining pressures (1500, 4500, & 9600 psf) in order to establish the Mohr strength curve from which the C and φ parameters were determined: The confining pressures chosen were based on the range of confining pressures the fills might be expected to encounter according to preliminary designs.

Table 1. Soil Sample Laboratory Testing Results.

SAMPLE ID	Source Location & Comments	Test Results								CT Soil Name / Symbol
		LL	PI	Φ´ °	C	γ pcf	Resistivity (ohm-m)	pH	corrosive ?	
LLSSB1	Station 1624+75, existing cut slope above road. Surface fines. Andesite-derived. Bag Sample	40	10	38.5	-4.97	96.5 - 98.3	12936	5.82	no	Silty Sand/ SM
LLSSP1	Station 1701+50. From erosion pile beneath highly welded pyroclastic tuff w/ cobbles & boulders. Bag Sample.	no	non-plastic	39.4	0.315	106.6-111.8	10438	6.35	no	Well-graded Sand w Silt & Gravel/ SW-SM
LLSS-S1	Station 1664+50, immediately east of large rock prominitory. Seismic line 2. Surface colluvial fines. Bag Sample.	no	non-plastic	31.8	16.8	111.2-113.1	9868	6.67	no	Silty Sand w Gravel/ SM
LLSS-T1	Station 1691+00. Seismic Line 7. Excavated mtl and loose mtl representing 3 weathered welded ashes on cut slope. Bag Sample.	49	5	33	7.4	81.58-84.13	9204	5.82	no	Silty Sand/ SM
LLT-1	Station 1718+80 cut slope above road. Representative of surface soil beneath Large Fill 2? Brass.	38	1	42.2	-3.69	62.7-91.95	NA	NA	NA	Silty Sand w Gravel/ SM
LLT-2	Station 1711+00, existing cut slope above road. Representative of surface soil beneath big fill1? Brass.	no	non-plastic	40.1	0.159	59.7-80.3	NA	NA	NA	Silty Sand w Gravel/ SM

Geotechnical laboratory tests and results are presented in Appendix 13.3.

Table 1 lists the samples, with a brief description of their locations, the test results most directly pertinent to the geotechnical analyses, comments, and the soil classification acronym that the tests results indicate for each sample.

7. Geotechnical Conditions

7.1 Site Geology

7.1.1 Lithology

Lithology in the project area is solely of volcanic origin. All rocks and deposits are of Pliocene age (5.3 Mya to 2.6 Mya), with the exception of recent colluvial deposits, the deeper ones probably being from the Pleistocene Epoch (2.6 Mya to 10,000 years ago) and the majority being likely of the more recent Holocene Epoch (10,000 years ago to present). Andesite is the more prevalent hard rock found throughout most of the project area, with a small amount of basalt found at the eastern end of the project. Pyroclastic rocks are also highly prevalent, though solely in the eastern half of the project. These consist of at least three different welded ash tuffs of fine, medium and coarse texture, along with a fine mildly welded to non-welded ash no more than 1 foot thick. Additional tuffs and ashes may be present, but evidence of their existence may be shrouded beneath colluvium deposits and the tuffs and ash mentioned above.

The coarse highly welded tuff (CHWT) consists almost entirely of well-graded angular to subangular clasts from gravel to cobble size in a relatively small amount of hard coarse sand matrix. Where exposed it is brown to gray brown in color and unweathered. It is highly resistant to erosion and weathering.

A medium coarse welded tuff (MCWT) containing predominantly gravel-sized clasts of various compositions set in a medium sand matrix of pale to chalky white color is found in several outcrops in the eastern half of the project, with the cut centered around station 1685+00 being the most prominent. This tuff contains a very small amount of local cobbles and occasional boulders that appear to be andesitic in composition.

A welded tuff (FWT) composed almost entirely of a medium and fine sand matrix with minor gravel is exposed in several high slope cuts within the eastern project area, most notably at around station 1691+00. It is red-orange to brown orange and contains a very incidental amount of local cobbles and an occasional boulder or two that appear to be andesitic in composition. This tuff is highly weathered and exists primarily as a saprolite.

A fine-grained dirty creamy white ash (WA) less than 1 foot in thickness is exposed at about station 1684+50. It may continue eastward, quite possibly in thicker section, but other overlying rocks and deposits obscure any verification of this.

Stratigraphic Relationships and Geomaterial Distribution in the Project Area

Andesite bedrock underlies, and crops out in several locations, in the western half of the project and slightly into the eastern half up to approximately station 1663+25, where a large massive tower of andesite looms about 130 vertical ft above the existing roadway. Andesite gravel, cobbles, and boulders up to about 6 ft in diameter sit atop and within the soil overburden throughout this area.

Eastward from the andesitic tower at station 1663+25 the andesite exposure veers sharply upslope away from the roadway before roughly paralleling the roadway as a near vertical 100- to 200-foot high wall located about 100 to 250 ft upslope from the road. This andesite wall diminishes in stature further eastward as it extends well beyond the eastern project limits.

East of station 1663+50, to about station 1712+00, the pyroclastic rocks described above are intermittingly exposed in the cut slopes above the existing roadway. Based on exposed field relations and interpretations of seismic data it is believed that these pyroclastics lap unconformably onto the andesite, which exists at depths varying from about 10 to 40 ft below the surface. The upper 2 to 10 ft of the buried andesite is weathered to varying degrees. Resting uncomformably (likely but not certain about this contact) atop the andesite is the fine orange welded saprolitic tuff (FWT). Above the FWT lies the white ash (WA) uncomformably, which likely represents a precursor eruptive phase of the overlying medium coarse welded tuff (MCWT) and the coarse hard welded tuff (CHWT), which appear to be coeval. These pyroclastics contain andesite cobbles and boulders, a few as large as 40 ft in diameter that were shed from the large andesitic massif upslope to the south. The native slopes within this stretch of pyroclastic rocks are covered with a 3- to 10-foot thick mantle of andesite cobble and boulder colluvium in a sandy to silty sand matrix composed of andesite derived materials with lesser contributions from the tuffs. Subsurface and surface boulders as large as 40 feet in diameter exist within and atop this colluvium.

Basalt bedrock begins somewhere between stations 1711+50 and 1713+00, with the first outcrop appearing in the existing cut slope at station 1713+00. The basalt bedrock continues eastward beyond the eastern project limits, with another prominent outcrop located from about station 1720+00 to 1723+00.

Colluvial deposits overlying the basalt are also composed of andesitic gravel, cobbles, and boulders in a sandy to silty sand matrix consisting of andesitic and lesser basaltic derived materials.

As mentioned previously, Plate 4 presents a portion of a geologic map published by Lydon, Gay, and Jennings (1960) that covers the project area. Plate 5 shows the geology from Plate 4, together with minor field corrections by OGDN, overlain on an aerial photo of the project area.

7.1.2 Structure

Internal geological structure is absent in the pyroclastic deposits. Horizontal tabular and platy structure, likely related to cooling, is found in much, though not all, of the andesite and basalt rocks exposed within the project area. Some andesite outcrops lack structure beyond some simple localized random jointing and are best described as massive.

7.1.3 Natural Slope Stability

Natural slopes within the project area are considered stable in their current morphology. No large or small scale sloughing or sliding was observed within the native uncut surfaces. Most slope surfaces consist of colluvium with slope ratios ranging from about 2.5:1 to about 1.3:1. The thick accumulations of surface and buried colluvium appear to indicate that the colluvium is fairly stable and slow-moving at the slope ratios present.

Some slopes consist of exposed andesite or lesser amounts of basalt bedrock, with slopes as steep as vertical.

7.2 Soils and Ground Water Conditions

Field reconnaissance and evaluation of soils in the project area, together with laboratory tests performed on samples collected from existing cut slopes, produced soil descriptions with engineering properties and characteristics described. Most soils in the eastern project area are colluvial in nature, with the fines predominantly composed of sands and silty sands, with lesser percentages of the fines in some areas being composed of silts, fine silts and fine sands derived from the weathering of a couple of the welded tuffs and ashes. Most soils are generally well- to excessively-drained, non-plastic, non-corrosive soils with no shrink-swell potential. A few soils demonstrate mild plasticity.

All of the above soils are underlain by volcanic bedrock (andesite, basalt, tuff, pyroclastic flows) at depths varying from about 1.5 ft to 15 ft.

In a few locations the silty sandy and sandy nature of the soils combined with some local ground water seepage may contribute to the collapse or failure of a cut slope.

7.3 Water

7.3.1 Surface Water

7.3.1.1 Scour

Only one small roadway structure, a short (less than three feet in height) can wall, is present within the project limits, and there does not appear to be any noticeable scour at its base. This structure is to be removed during the construction of this project.

Since no structures are proposed for this project, scour is of no concern.

7.3.1.2 Erosion

Erosion potential varies throughout the project area depending upon geologic and topographic conditions.

Native ground surfaces show no signs of any significant erosion, with the exception of the thalwegs below the road within the ephemeral and intermittent stream drainages (station 1662+75, 1711+75, and 1719+00) that descend to the north. Erosion here is limited, however, constrained by the gravel, cobbles, and boulders that remained after some of the sands and silty sands were eroded from the native ground and fill ground (closer to the road).

Existing fills of 1.5:1 or flatter show little erosive damage, with the exception of a few areas where surface flow had become concentrated and rills had been allowed to develop. Existing fills of 1.25:1 that line the roadway downslope from the highway in several places within the eastern project area show moderate erosion damage only in a few locations where water appears to have been allowed to concentrate. Otherwise, these steep fills appear to have stood up well to normal surface flow in most areas. It should be noted, however, that tree and vegetation cover in almost all locations have protected these fill slopes from particle mobility created by direct raindrop impact.

The most noticeable erosion damage on these 1.25:1 fills seems to have occurred downslope of the highway in the vicinity of the curve centered on station 1662+75. This is likely the result of a leaky and failed downdrain and culvert system that fed a significant amount of concentrated flow onto the fill slope and native slope, and possibly into the fill itself. This erosion is likely partly responsible (see additional causality discussion below in section 7.3.2) for undermining the stability of the fill slope, which shows signs of having slipped slightly, including a 100-foot long arcuate crack that reaches the centerline.

Existing cut slopes in the western half of the project show considerable resistance to erosion, despite a relative lack of solid vegetative cover. This is likely due to a combination of the properties of the andesite-derived overburden soil and the fact that surface flow does not seem to concentrate on these slopes.

Cut slopes in the eastern half of the project are both highly resistant to erosion and fairly erosive, depending upon the geological composition of the particular slope. Cut slopes in andesite and basalt bedrock (station intervals 1649+50 to 1663+50; 1713+00 to 1717+75, and 1720+00 to 1723+25) are highly resistant to erosion. Many of the slopes cut in the pyroclastic material (station interval 1663+75 to 1708+35) are moderately erosive, particularly those in the softer tuff (FWT) and ash (WA) deposits. The CHWT, the most erosion resistant pyroclastic deposit, has suffered only very minor erosive effects since the existing slopes were cut. Differential erosion is considerable in multiple locations within this station interval, with the softer tuffs eroding out from beneath the considerably more resistant CHWT and cobble-rich colluvial overburden deposits that often top the cut slopes.

The MCWT tuff also erodes differentially beneath these more resistant deposits, though not nearly as substantially as the softer FWT and WA. The CHWT and MCWT both erode to globally stable vertical slopes when capped by cobble rich colluvium, while the softer FWT and WA deposits erode from their original 0.75:1 slope ratios to slope ratios possibly as flat as 1.8:1 when water is present chronically.

7.3.2 Ground Water

There is no evidence of the presence of ground water near the ground surface or within reach of any of the proposed cuts within the western half of the project area, so it is highly unlikely that ground water will be encountered during construction of the proposed western cut slopes.

Ground water appears to be present near the surface in only one location where the andesite or basalt bedrock is observed at the surface. That location is centered on station 1662+75 where an intermittent stream runs above and below the road and where shallow ground water likely flows year-round through fractures in the bedrock. This flow likely contributes to the slope failure and erosion below the road at this location that was discussed above in section 7.3.1.2.

Ground water is present near the surface in several locations in the middle of the pyroclastic deposits. This ground water is responsible for several seeps and springs either at or near road level, or further upslope from the road. Based on the geology, topography, elevations, and the locations and volumes of water observed, it appears that this ground water is flowing from a year-round source at higher elevations somewhere to the east through fractures in the underlying andesite. This artesian ground water begins to surface in the vicinity of station 1710+00 and in several locations further down station, likely making its way from the andesite fractures and then through the pyroclastic deposits to the surface. Some of these seeps have created a few chronically wet cut slopes that are flattened at their lower parts to slope ratios as flat as 1.5:1.

The collection of structures and cabins known as Lassen Lodge at the center of the project obtains its water from one of these springs via a metal pipe that taps into a spring upslope from the highway in the vicinity of the intermittent stream and thalweg located around station 1711+50. This pipe runs above the road and above the top of cut all the way to the eastern edge of Lassen Lodge where it flows into multiple collecting water tanks. This water flows year-round, even during the low snow and drought years, indicating that it is coming from a fairly large underground source.

A PVC horizontal drain pipe protrudes several feet from the cut slope at about station 1679+00, issuing water year round with a flow of at least 1 gallon per minute (gpm), even during the drought summer of 2015. This pipe has apparently become a source of water to many residents, according to the local maintenance forces. This pipe should be reinstalled after the slopes are cut in this area, to maintain a good relationship with the local public. It may also help to relieve pore water pressure buildup in the slope and prevent sliding.

Water seeps from existing slopes in several locations between stations 1676+00 and

1680+50. OGDN recommends installing underdrains through this section of roadway at the toe of the new cuts.

7.4 Project Site Seismicity

7.4.1 Ground Motions

Based on Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations (November 2012), the subsurface conditions discussed above, and several MASW (Multichannel Analysis of Shear Waves) 1-D vertical profiles, an average shear wave velocity (Vs_{30}) of about 2000 ft per second (fps) (600 meters/second) was used for the upper 100 ft (30 meters) of the rock/soil at the project site. The particular location within the project chosen for the Vs_{30} profile was in the area beneath the largest proposed fill, since there are no structures in this project. This means that MASW profile 11 and 12 were combined together to determine Vs_{30}.

Table 2. Details for Controlling Faults for Project Seismicity.

Fault Name	USGS Fault Number	Fault Type	Dip °	Dip Direction	Maximum Magnitude	Fault Distance (miles)	Fault Length (miles)
Battle Creek Fault	20	Normal	75	S	6.5	12.8	20.5
Butt Creek fault zone	189	Right-Lateral Strike Slip	90	Vertical	6.8	18.0	19.9
Hat Creek fault zone	9	Normal	60	W	7.2	24.2	36.0

Based on Caltrans ARS Online (2.3.07), an acceleration spectrum taken by combining the probabilistic acceleration spectrum developed from the USGS 2008 Interactive Deaggregation (Beta) model and the statewide minimum deterministic spectrum controls at the site.

The active controlling faults having the greatest potential seismic impact on the project site are the Battle Creek Fault, the Butt Creek fault zone, and the Hat Creek fault zone. Details on these faults are given in Table 2 above.

With the Vs_{30} value above, the peak ground acceleration (PGA) of 0.20 g is estimated to be applicable at the site for a period of 1 sec.

Liquefaction potential is considered to be very low for most locations in the project area. All bedrock, which is often near or at the surface, is not liquefiable. Although the fines in the soils and overburden material consist primarily of sands and silty sands, which are generally susceptible to liquefaction, they also contain considerable gravel, cobbles, and even boulders

that will help to maintain the structure of the material during ground shaking. Most, though not all, of the soils and overburden material, is above the ground water table and unsaturated; hence, these materials are not going to liquefy with ground shaking. Those local cuts that contain seeps from ground water sources are slightly susceptible to liquefaction, but the gravel, cobble, and boulder content of these local cuts will serve to provide at least partial resistance to liquefaction. Lastly, the founding material beneath the fills is mostly above the ground water table and will not liquefy. The thalweg portions of these founding materials beneath Fill 1 and Fill 2 are the exception to this paucity of water. These thalweg areas, however, have an increased proportion of rocky material due to the stripping of fines caused by the long term water flow. In addition, these thalweg areas represent a fairly small portion of the founding surface beneath the fills. In summary, liquefaction generally poses no threat in the project area.

7.4.2 Ground Rupture

No known fault is projected towards or passing directly through the project site. Therefore, potential for surface rupture due to fault movement is considered nil.

8. Geotechnical Analysis and Design

8.1 Dynamic Analysis

Simplified pseudo-static slope analysis was performed on the fills primarily to gain a general idea of their expected response to the maximum credible earthquake. Cut slopes were not analyzed for seismic stability for the same reasons (discussed in section 8.2.1) that limit equilibrium stability analyses were not performed on them.

Choosing the correct value for the horizontal seismic coefficient is an uncertain process, given the breadth of recommendations in the literature, with values typically ranging from 0.05 to 0.2, and/or 0.2 to 0.5 of the peak horizontal acceleration (PHA). Seed (1979) recommends k_h values ranging from 0.1 to 0.2 with a required FOS$\geq$ 1.15, while the Army Corps of Engineers (1982) requires a FOS $\geq$ 1.0 with a recommended k_h of 0.1 for a major earthquake, which is what the maximum magnitude 6.5 for the nearby Battle Creek Fault would be classified as. OGDN followed the recommendations postulated by the Corps of Engineers in this report for performing pseudo-static analysis, since their work is geared more towards general embankments while Seed's work is focused on earthen and rock filled dams, which require a higher factor of safety standard. Analyses results for earthquake response are briefly discussed in section 8.3.

Liquefaction analyses were not performed because there are no structures involved in this project.

8.2. Cuts and Excavations

8.2.1 Stability

Existing cut slopes provide empirical evidence regarding stability. They are considered here as the best single source of geotechnical information available for determining stability and behavior of the future proposed cuts, since proposed cuts will be cut in essentially the same materials, and at or close to the same slope ratio as the existing slopes.

Stability analysis of the cuts in the andesite, basalt, or pyroclastics by limit equilibrium methods that assume the cut slope material acts as a strict Mohr-Coulomb mass is inappropriate and was not performed. Attempting to define Mohr-Coulomb parameter values (φ, C) for the pyroclastics results in the use of significant assumptions that render results wrought with great uncertainties. Cuts in the andesite or basalt bedrock outcrops, which possess some structural features, are more amenable to kinematic analysis, but that was not considered necessary as the simple geologic structure (explained below) did not warrant that level of effort.

While the geomaterial in the cuts of the western half of the project is the most conducive of all project geomaterials to limit equilibrium analysis as a Mohr-Coulomb mass, such an analysis is still hampered by the need to make some considerable assumptions in assigning parameter values. Laboratory test values indicate that the matrix material, which has a general φ value somewhere around 40° to 42°, will produce unsatisfactory FOS values if assumptions are not made for the gravel, cobbles, and, in some locations, in-place andesite bedrock, especially if cohesion (C) is ignored, as is often the common engineering practice for embankment analyses, due to the unreliability of cohesion, at least in some instances. Increasing the φ angle slightly, to account for the embedded gravel and rock, and adding a moderate amount of cohesion can produce a FOS of 1.3 for a back-analysis of an existing stable 15-foot high cut. Identical parameter values for a proposed 30-foot cut, however, results in a less than adequate FOS of about 1.2. Arbitrarily raising either the φ value a bit more, as well as the cohesion value brings the 30-foot cut FOS to an acceptable 1.3, but does this really provide certainty when it comes to assessing real-world stability? These academic exercises were performed for these slopes, using φ (raised to account for subangular rock present) and C values that are judged (engineering judgment) to be realistic, but the preferred and more reliable approach is simply applying good engineering judgment, based on experience and existing cut slopes, in extrapolating the condition and performance of the existing slopes to the proposed slopes.

Existing cut slopes in the western part of the project have slope ratios varying from 1.7:1 to 0.6:1, with the majority having slope ratios around 1:1. Heights of these existing cuts are typically between 10 and 15 ft, with the highest cut reaching 22 ft. These existing slopes have proven to be highly stable, with little to no signs of failure since they were cut decades ago.

The proposed cut slopes in the western half of the project have slope ratios of 1:1 with maximum heights of up to 30 ft. These proposed slopes push the existing empirical bounds

of stability slightly by their increased height over the height of the existing cuts, but this is judged to pose no significant risk problem considering the lab φ angle and subangular nature of the fines, the general rocky nature of most of the material, and the presence of buried cobbles, and boulders, some with horizontal to sub horizontal tabular and platy structure. A few cuts even appear to involve bedrock. Pushing the risk boundaries ever slightly is in line with the project (and PDT) approach to reduce excavation quantities to meet the tight budget, though the risk here is considered to be very small. Should a failure occur, it will likely be relatively insignificant and only require cleanup of the mobilized material.

Existing cut slopes in the andesite and basalt bedrock outcrops generally have slope ratios of about 0.5:1, with some flattening towards 0.75:1 at the bottom due to rockfall. Existing andesite bedrock cut slopes in the middle of the project (station 1649+50 to 1661+00) generally average about 15 ft in height, with a maximum height of about 30 ft. A few andesite cut slopes standing at or near vertical, including the near vertical to vertical 70-foot cut bounding the quarried area located around station 1658+00, have all demonstrated long-term stability. While massive in some locations, a majority of these rocks possess some structural fabric, which primarily consists of cooling joints and fractures that generally trend horizontal or near horizontal and do not contribute to global instability. These rocks are completely unsuited to Mohr-Coulomb analysis, while a first-pass kinematic structural assessment indicates that detailed kinematic analyses are not necessary for these rocks, given their either massive or general horizontal to sub-horizontal structural fabric. OGDN proposes cut slope ratios of 0.5:1 for the new cuts in these locations, with maximum heights up to 70 ft. These cut slopes should be fairly stable with a very low risk of large failure and a small risk of a few small local failures peeling off during or soon after construction.

Existing basalt bedrock cut slopes in the east end of the project located from station 1713+00 to 1717+75 generally average about 30 ft in height, with a maximum height of about 45 ft. These cuts are fairly stable, excluding rockfall. These rocks are more akin to the andesite rocks described above with regards to structural fabric and stability. Therefore, OGDN recommends similar cut slope ratios of 0.5:1 for the new cuts in this location with maximum cut heights up to 70 ft. Risks are similar to the andesite rocks mentioned above.

Existing cut slopes in the basalt bedrock outcrop further east from about station 1720+00 to 1723+00 generally average about 12 to 15 ft in height, with a maximum height of about 20 ft. These cuts are all fairly stable, excluding rockfall. The great majority of these rocks possess considerable structural fabric primarily consisting of cooling joints and fractures that generally trend horizontal or near horizontal. Depending upon their continuity and tightness deeper inside the existing slope, these structural features may or may not contribute to some degree of global instability at a 0.5:1 slope ratio, due to the high density of fractures and the looseness and relaxation observed on these fractures exposed at the existing surface. It is uncertain whether this looseness continues deeper into the rock mass. These rocks are also completely unsuited to Mohr-Coulomb analysis. Because of the density and looseness of the sub horizontal to horizontal fractures in these rocks, OGDN proposes cut slope ratios of 0.75:1 for the new cuts in these locations, with maximum heights up to 60 ft. These cut slopes should be fairly stable with a low risk of large failure and a moderate risk of a few small local failures peeling off during or after construction.

Existing cuts from station interval 1730+25 to 1733+00, which is composed of basalt and andesite derived soil with some colluvium, are generally about 1:1 with maximum heights of about 25 ft. These are highly stable. Although the realignment proposed by Design requires only small shifts into the existing slopes, even small 1:1 cuts would require substantial cut heights and volumes to achieve this; consequently, OGDN recommends cut slope ratios of 0.75:1 here with maximum cut heights of 10 ft. These cut slope ratios carry a moderate risk of some local sloughing, but the proposed catchment (section 8.2.4) will be capable of containing such sloughs, thereby mitigating the major impact stemming from such a risk.

Existing cut slopes in the pyroclastic deposits, roughly from station 1663+50 to about 1708+00, have a predominant slope ratio of 0.75:1. A few of these have sloughed or eroded to slope ratios closer to 1:1, while a smaller percentage of cut slopes here appear to have been originally cut at about 1:1. A couple local cut slopes, in areas where ground water seepage appears to be chronic, have slumped, creating slopes with bottom sections as flat as 1.3:1. Shallow sloughing or erosion in a few cut slopes, involving mostly FWT deposits, has resulted in a few local 1.2:1 slopes. Aprons of material eroded from the MCWT lie at the bottom of some slopes with slope ratios from 1:1 to 1.2:1. Existing cut slopes in the pyroclastics predominantly range in height from about 15 to 40 ft, with a few outliers standing fairly stable at about 85 ft.

In an effort to keep excavation quantities down, the PDT similarly agreed to take a certain amount of risk here in the pyroclastics and design the slope cuts all at 0.75:1, rather than take the more costly approach of laying back all slopes in this interval to 1:1 or possibly even flatter. Laying slopes back here even a small amount results in a substantial increase in excavation quantities because of the steep native ground (0.9:1 to 1.8:1) above the existing cuts. The great majority of proposed cuts range in height from about 20 to 50 ft, and have a maximum height around 95 ft.

Due to the significant presence of ground water here and there in the pyroclastics section, the weaker nature of some local material, the moderate prevalence of some shallow sloughing and surficial movement, and the increased cut slope heights relative to existing cuts, there is a moderate risk that this station interval will see a few global failures during construction. This risk is at its highest during the first post-construction winter season. At least a few shallow sloughs and/or minor failures are likely to occur here during the first post-construction wet season.

Proposed cut slope recommendations discussed above are summarized below in Table 3.

Table 3. Recommended Proposed Cut Slope Ratios and Maximum Cut Height.

Station Interval	Proposed Cut Slope Ratio	Maximum Cut Height (ft)
1592+75 to 1649+00	1:1	30
1649+50 to 1662+25	0.5:1	70
1662+25 to 1709+25	0.75:1	100
1713+00 to 1717+00	0.5:1	70
1720+25 to 1723+00	0.75:1	60
1730+25 to 1733+00	0.75:1	10

Post-Construction Erosion and Instability

The risk of shallow sloughs, small failures, and increased rockfall occurring also increases elsewhere throughout the project area during the first winter season, though the risk is highest in the pyroclastics section (station 1662+25 to 1709+25). Originally, during the initial design phases of this project, these risks were discussed with the PDT and considered acceptable. A primary reason for accepting these risks was because the project was supposed to have two construction seasons, meaning that during the first overwintering period, when the great majority of post-construction instabilities typically act out, the travelled way would be a substantial distance away from the new slopes as they underwent their post-construction adjustment, and the failed material would almost certainly come to rest outside of the travelled way where the contractor, who would be available and under contract, could clean up any potential failed material if necessary. Near the very end of the design phase, however, the project was shortened to one season due to budgetary reasons, thereby eliminating the overwintering period. The risks mentioned above remain, however, because to correct them by making the cut slope designs more conservative would break the project budget. The PDT is aware of this and is party to the decision to keep the initial slope designs.

Due to these post-construction risks being unaccounted for or unmitigated by an overwintering period under contract, OGDN strongly recommends that a post-construction period of at least 8 months wherein the contractor is still responsible for erosion, storm water, and slide issues, be inserted into the contract via an nSSP. This should require that the contractor be responsible for responding immediately to slides and erosion problems during the first overwintering period following construction. This nSSP has been created collaboratively by the Office Engineer and OGDN and it refers to this work as permanent erosion control establishment work. This work consists of identifying deficiencies and maintaining permanent erosion control, including slide and slipout repair, re-application of materials, and adjusting and repairing erosion control features. This work begins after permanent erosion control work has been completed, which typically occurs at the very end of construction.

8.2.2 Rippability and Excavation

Rippability assessments are made based on seismic velocities (P waves), rock type, and rock fracture and joint characteristics. Seismic velocity correlations are based on two different scales, each with differing rippability assessments depending upon ripping equipment and rock type. Caltrans has its own internal non-rock-type specific correlation scale between seismic velocity and rippability based on a Caterpillar D9 Series bulldozer with a single-toothed ripper:

Velocity (fps)	**(Caltrans)**	**Rippability**
< 3445		Easily Ripped
3446 – 4921		Moderately Difficult
4922 – 6562		Difficult
> 6563		Not Rippable

A rock-type specific seismic velocity scale based on a larger bulldozer (Caterpillar D10 with a single shank ripper) taken from a handbook published by Caterpillar (2010) is also presented here to provide the contractor with a wider range of rippability and excavation information. Choosing from the limited rock types offered in the Caterpillar table, basalt is clearly the most appropriate rock type to choose for matching the hard extrusive igneous rock types (andesite, basalt) observed within the project area. Similarly, breccia, though not a tuff, is probably the best rock type offered in the Caterpillar table most similar in rock texture and hardness characteristics to those of the CHWT tuff, which is the hardest rock of all the pyroclastics within the project area.

For basaltic rocks, and for a D10 with a single shank ripper, the Caterpillar manual presents the following rippability assessment:

Velocity (fps)	**(Basalt)**	**Rippability**
≤ 8,000		Rippable
8,000 – 9,000		Marginally Rippable
> 9,000		Non-Rippable

For breccia rocks, and for a D10 with a single shank ripper, the Caterpillar manual presents the following rippability assessment:

Velocity (fps)	**(Breccia)**	**Rippability**
≤ 8,300		Rippable
8,300 – 10,400		Marginally Rippable
> 10,400		Non-Rippable

It is important to note that rippability in this situation is synonymous with ease of excavation, and it says nothing about the efficacy of boulder size reduction. Rippable means that large boulders will be removable from the slope. The ease with which boulders can be reduced by a ripper is a completely different issue. Boulders in the western half of the project area are

mostly fresh to slightly weathered, and probably have seismic velocities ranging from 4500 fps to 9000 fps. Given the predominant lack of structural grain within a majority of these boulders, a characteristic that would otherwise allow a ripping tooth to bite into the rock, these boulders should be considered unrippable (incapable of being reduced in size) by a ripping tool. A large excavator-mounted hoe-ram should be fairly successful in reducing these boulders, though the harder and larger boulders will obviously require more blows than the more weathered and/or smaller ones. Of course, drilling and blasting these boulders will also be successful in reducing boulder size, though it is left to the contractor to decide which approach is most effective and economical. OGDN estimates that boulders greater than 18 inches in diameter comprise at least 10 percent by volume of the excavation quantities in the western half of the project.

Cut slopes in the western half of the project area occur predominantly in the soil overburden and upper weathered zone of the andesite bedrock. A few specific locations (such as station intervals 1604+50 to 1607+00; 1610+00 to 1612+75; and 1620+50 to 1624+00) in this half of the project area contain a larger percentage of bedrock, with approximately 10 percent of it extending down into rock that is fresh or only slightly weathered. Based on Caltrans' rippability scale, OGDN estimates that 85 percent of the cuts in the western half of the project area are easily ripped, with the remaining 15 percent being considered moderately difficult to rip. Based on the Caterpillar scale, 100 percent of the cuts in this area are considered rippable.

The seismic velocities for the andesitic bedrock outcrops from station 1650+00 to 1663+25 are determined from seismic line 1, as well as velocities taken in other seismic lines elsewhere in the project where similar andesite is found at the surface or interpreted to be at depth. The velocities for these outcrops generally range from about 2000 fps to 7000 fps, although some massive blocks within this station interval that were not specifically surveyed seismically likely have velocities upwards of 9000 fps or more, based on field examination, comparisons with other locations and velocities, and experience. Much of the andesite within this stretch has platy, tabular, and/or fairly fractured structure, which makes the rock more amenable to being ripped. Based on the Caltrans' rippability scale, approximately 20 percent of the material here is considered to be easily ripped, 20 percent is considered to be moderately difficult to rip, 20 percent is considered to be difficult to rip, and 40 percent is considered not rippable. Based on the Caterpillar rippability scale, 70 percent is considered to be rippable, 10 percent is considered marginally rippable, and 20 percent is considered non-rippable.

Seismic velocities range from 2000 fps to 9000 fps for pyroclastic cuts from station 1663+50 to 1708+50, though the great majority (90 percent) of the material velocities are below 6500 fps. It is also possible that the underlying andesite may possibly be encountered in a few local areas in the deeper cuts. Based on the Caltrans' rippability scale, approximately 50 percent of the material here is considered to be easily ripped, 30 percent is considered to be moderately difficult to rip, 10 percent is considered to be difficult to rip, and 10 percent is considered non-rippable. Based on the Caterpillar rippability scale, 90 percent is considered to be rippable, and 10 percent is considered marginally rippable. It is likely, though not a certainty, that the marginally rippable material is underlying andesite.

Cuts from station 1713+00 to 1717+75 are in basalt bedrock with a slight amount of soil overburden. Seismic velocities range from 2000 fps to 10000 fps. Based on the Caltrans' rippability scale, approximately 35 percent of the material here is considered to be easily ripped, 15 percent is considered to be moderately difficult to rip, 20 percent is considered to be difficult to rip, and 30 percent is considered non-rippable. Based on the Caterpillar rippability scale, 75 percent is considered to be rippable, 15 percent is considered marginally rippable, and 10 percent is considered non-rippable.

Cuts from station 1720+00 to 1723+25 are in basalt bedrock with a slight amount of soil overburden. Seismic velocities range from 2000 fps to 8000 fps. Based on the Caltrans' rippability scale, approximately 35 percent of the material here is considered to be easily ripped, 25 percent is considered to be moderately difficult to rip, 25 percent is considered to be difficult to rip, and 15 percent is considered non-rippable. Based on the Caterpillar rippability scale, 95 percent is considered to be rippable, and 5 percent is considered marginally rippable.

Rippability estimations discussed above are summarized in Table 4 below.

Table 4. Rippability Estimations by Station Interval. Station Intervals not listed are considered easily ripped. See text for further explanation.

Station Interval	Caltrans Scale				Caterpillar Scale		
	Easily Ripped	Moderately Difficult	Difficult	Not Rippable	Rippable	Marginally Rippable	Non-Rippable
	PERCENTAGES						
1590+00 to 1640+00 (Andesite Bedrock)	85	15	0	0	100	0	0
1650+00 to 1663+00 (Andesite Bedrock)	20	20	20	40	70	10	20
1663+50 to 1708+50 (Pyroclastic Deposits)	50	30	10	10	90	10	0
1713+00 to 1717+75 (Basalt Bedrock)	35	15	20	30	75	15	10
1720+00 to 1723+25 (Basalt Bedrock)	35	25	25	15	95	5	0

Boulders are to be found throughout the project, some with diameters as large as 40 ft. A majority of these are rounded to subrounded, and will need to be fractured to create material that is angular to subangular before being placed in fills. Some boulders sit atop the general surveyed datum for original ground, and as such represent at least (and probably more) 3000 yards^3 of material not accounted for within the volumes calculated by Design. Some of these boulders may lie outside of the designated cut line, but close enough to it that their removal will be necessary for safety due to the possibility that the boulder will roll or fall over the new hinge point either during or after construction. Other boulders that are embedded within

the soil or pyroclastic deposits may be exposed during cutting, with a portion of the boulder extending into the cut area and the remainder lying outside of the proposed cut boundary. These situations may require the removal of the entire boulder and the consequent removal of additional material in order to accomplish the task.

Based on the rippability descriptions and boulder issues discussed above, some method of excavation beyond standard ripping and cutting with an excavator or dozer blade will be necessary. If blasting is chosen, it should be performed following Caltrans specifications for pre-splitting and/or controlled blasting as specified.

8.2.3 Grading Factor

Grading factors are presented in Table 5 below by station interval.

Table 5. Fill Material Group and Grading Factors by Station Interval

Station Interval	Fill Material Group (section 8.3.1)	Grading Factor		Station Interval	Fill Material Group (section 8.3.1)	Grading Factor
1592+75 to 1604+50	F1	0.93		1662+25 to 1682+00	F4	0.92
1604+50 to 1607+00	F2	1.05		1682+00 to 1686+00	F5	0.94
1607+00 to 1610+00	F3	0.94		1686+00 to 1695+00	F4	0.92
1610+00 to 1612+75	F2	1.05		1695+00 to 1709+25	F5	0.94
1612+75 to 1620+50	F3	0.94		1713+00 to 1717+00	E	1.15
1620+50 to 1624+00	F2	1.05		1717+00 to 1720+25	F6	0.94
1624+00 to 1650+00	F3	0.94		1720+25 to 1723+00	E	1.15
1650+00 to 1662+25	E	1.15		1730+25 to 1733+00	F6	0.94

The above table classifies particular station intervals of excavation material into Fill Material Groups (F1 thru F6, and E) and assigns grading factors for each Fill Material Group. These Fill Material Groups, their descriptions, and parameter values are discussed and explained further in the section (8.3.1) on embankment material. Grading factor assignment is discussed there also, rather than in this section, because these factors are directly tied to the material properties and the relative quantities of particular materials present in any particular Fill Material Group.

Grading factors less than unity (1.0) represent material quantities that are expected to shrink in volume when going from their in-situ location to their placement and compaction in a fill. Grading factors greater than unity represent material quantities that are expected to expand in volume during the same process. The factor is a simple multiplier of the original in-situ quantity.

8.2.4 Rockfall and Catchment

Minor rockfall presently occurs from many cut slopes throughout the project area, primarily in the eastern half of the project. For the most part it is well contained by the existing catchment present at the base of existing cut slopes. The primary sources are colluvial deposits above hinge points, rocks weathering out of the cut slopes, and fractured rock coming loose from andesite and basalt cut slopes at the middle and eastern end of the project.

The proposed cuts for this project are generally higher than existing cuts, and in some places are slightly steeper, changes that will increase both the amount of rockfall and the trajectory of rockfall run-out (post impact rolling and/or bouncing away from the slope towards the travelled way). The optimal solution is the expansion of existing catchment dimensions in the design of the new cut slopes.

Catchment is generally defined as the unpaved shoulder beneath a cut slope that is graded either flat or sloping away from the pavement back towards the toe of the cut slope. This catchment serves to provide an impact zone for falling rocks and to contain rockfall run-out from reaching the travelled way. This catchment area also provides a location for surficial sloughs to collect, thereby keeping them off the roadway and allowing maintenance to address the material at a time of their choosing rather than on an immediate emergency basis.

Because every foot of catchment requires the cut slope above it to be pushed back one foot deeper into the slope, it is not feasible economically to provide catchment capable of containing 99 percent of all rockfall run-out, as well as any and all large slides. Therefore, it is necessary to balance risk, cost, and functionality in the design recommendations for catchment. It was a joint decision by the project development team (PDT) to take this approach. The optimal catchment design based on a balance of these guiding factors (risk, cost, functionality) should attempt to contain at least 95 percent of impacts and about 75 percent of run-out. 70 percent containment for runout is acceptable only for fringe cuts (cuts that make up only a very small percentage of the total). These numbers are considered to be somewhat equivalent with what has been occurring beneath the existing slopes, so there should be no decline in catchment efficacy with the new cut slopes and catchments, and in many cases there will be an improvement. Tolerance for direct impacts must be kept extremely low because they can result in rocks striking a vehicle from above, including through the windshield. Run-outs can still create road hazards, but typically not as severe as impacts.

Catchment analysis and design was done integrally in conjunction with cut slope design, and was based on field observations of present catchments and their performance, present geological conditions, and catchment tables and charts created from over 10,000 rockfall

simulations by Pierson, et al (2001). The frequency of rockfall events was used as a weighting factor on a slope by slope basis. In addition, for evaluating catchment widths necessary for run-out containment, the shoulder backing adjacent to the edge of pavement and the paved shoulder space outside of the edge of traveled way (ETW) were included in the evaluation using a correction factor for their differing surfaces and slopes. The results of previous work using a rockfall simulation program (CRSP, Jones, et al, 2000) to evaluate the value of the shoulder backing and paved shoulder for run-out containment was incorporated here. The 6:1 backslope is recommended for all catchment within the project as it is significantly more effective at containment than a flat surface and, though not as effective for containment as a 4:1 backslope, it provides a far gentler transition onto the shoulder for vehicle recovery.

Cut slopes in the western half of the project (from station 1592+00 to 1640+00) have proposed slope ratios of 1:1, maximum heights of 25 ft, and proposed paved shoulders 8 ft in width. Recommended catchment (unpaved surface with backslope of 6:1) widths of 6 ft will provide 99 percent containment of impacts and 95 percent containment of run-out (into travelled way).

For all other slopes within the project area a minimum catchment width of 6 ft is recommended.

Cut slopes in the beginning of the eastern half of the project area from station 1647+00 to 1662+50 have proposed slope ratios of 0.5:1, maximum heights of 42 ft, and proposed paved shoulders 4 ft in width. Recommended catchment widths range from 8 ft to 12 ft in width.

Cut slopes in the eastern half of the project area from station 1663+75 to 1708+75 have proposed slope ratios of 0.75:1, maximum heights of 105 ft, and proposed paved shoulders 4 ft in width. Recommended catchment widths range from 8 ft to 15 ft in width.

Cut slopes in the eastern half of the project area from station 1713+00 to 1717+50 have proposed slope ratios of 0.5:1, maximum heights of 59 ft, and proposed paved shoulders 4 ft in width. Recommended catchment widths range from 6 ft to 14 ft

Cut slopes in the eastern half of the project area from station 1720+00 to 1723+25 have proposed slope ratios of 0.75:1, maximum heights of 58 ft, and proposed paved shoulders 4 ft in width. Recommended catchment widths range from 8 ft to 14 ft

Table 6 below provides recommended catchment widths together with corresponding cut heights by stationing. The percent (of rocks) containment for both impacts and run-outs is given for each cut slope height interval.

No alternative rockfall mitigation method is recommended.

It should be noted that the first post-construction winter season is likely to see a moderate to significant increase in rockfall relative to existing rates as the tops of the new slopes adjust and stabilize to their new configuration.

Table 6. Recommended Catchment Widths by Station Interval and Cut Slope Height, with Percent Containment for Impacts and Run-Outs.

Station Interval	Cut Slope Height Range (H)	Recommended Catchment Width (ft)	Impact Containment (%)	Run-Out Containment (%)
1592+00 to 1640+00	H ≤ 25 ft	6	99	95
1647+00 to 1662+50	H ≤ 25 ft	8	99	80
	25 ft > H ≤ 30 ft	10	99	80
	30 ft > H ≤ 35 ft	12	99	80
	35 ft > H ≤ 45 ft	12	99	75
1663+75 to 1708+75	H ≤ 20 ft	8	99	95
	20 ft > H ≤ 30 ft	10	99	90
	30 ft > H ≤ 40 ft	11	99	90
	40 ft > H ≤ 50 ft	12	99	90
	50 ft > H ≤ 55 ft	13	99	80
	55ft > H ≤ 65ft	14	99	80
	65ft > H ≤ 75ft	14	95	75
	75ft > H ≤ 100 ft	15	90	70
1713+00 to 1717+50	H ≤ 10 ft	6	99	98
	10 ft > H ≤ 20 ft	8	99	95
	20 ft > H ≤ 30 ft	10	99	90
	30 ft > H ≤ 40 ft	12	99	80
	40 ft > H ≤ 50 ft	13	99	80
	50 ft > H ≤ 60 ft	14	95	75
1720+00 to 1723+25	H ≤ 25 ft	8	99	95
	25 ft > H ≤ 35 ft	10	99	95
	35 ft > H ≤ 45 ft	12	99	85
	45 ft > H ≤ 55 ft	14	99	75
	H > 55 ft	14	99	70

8.3 Embankments

Slope ratios for the proposed fills vary from 1.25:1 to 4:1, depending upon multiple factors and parameters, including material properties of the material obtained from the cuts, steepness of the founding terrain and original ground, a need in some locations to limit spatially excessive fill footprints (environmental and right-of-way concerns), budgetary constraints to reduce both cut and fill volumes, and other factors.

Proposed fills equal to or flatter than 2:1 are located in several locations through the project, primarily in the westernmost part or beginning of the project. Their relatively flat slopes (26° or less) and the general quality of the fill material available from the cuts (the lowest lab φ angle being 31.8° and the φ angle for most material being considerably larger, especially due to incorporated gravel and rock) indicates that these fills are highly stable and do not require stability analysis.

Three large fills with slope ratios of 1.55:1 are proposed for the eastern end of the project. These fills shall be referred to here as Fill 1 (station interval 1708+30 to 1713+00 LT), Fill 2 (station interval 1717+75 to 1720+10 LT), and Fill 3 (station interval 1723+00 to 1729+00 LT). These fills are all higher and larger than any existing fills in the project area. Fill 1 is about 150 ft in height from top to toe, with a maximum vertical column of about 60 ft. Fill 2 is about 92 ft in height from top to toe, with a maximum vertical column of about 30 ft. Fill 3 is 65 ft in height from top to toe, with a maximum vertical column of 30 ft.

Fills 1, 2, and 3 are founded on fairly steep native ground. Fill 1 is founded on native ground with slope ratios varying from 2.6:1 to 1.5:1. Fill 2 is founded on native ground with slope ratios varying from 2.3:1 to 1.4:1. Fill 3 is founded on native ground with slope ratios varying from 3:1 to 1.7:1.

Because of their considerable size, their location in valleys that descend steeply to the north away from their toes, and the presence of at least some surface water exiting the bottom of the fills via modified underdrains, Fills 1 and 2 have been designed with 60-foot wide (30 ft on either side of the thalweg) toe buttresses constructed of 2 ton RSP (Method A placement). A cross-section of this buttress is shown in Plate 6. Fill 3 does not sit centered in a thalweg like Fill 1 and Fill 2, and has considerably less height; therefore, it does not require a toe buttress. These 3 fills are all to be encapsulated with Selected E material (a rocky material discussed below in section 8.3.1), both to increase global stability and to provide erosion protection. Fill 1 and Fill 2 have 10 horizontal ft of this encapsulation, while Fill 3 has 6 horizontal ft of Selected E. Because of its considerable height, Fill 1 requires high quality material at its base in order to meet factor of safety requirements (section 8.3.2). Therefore, it has a bottom zoned portion requiring material taken only from a specific station interval where the material is known to have qualities that meet stability requirements. This interval is located between station 1695+00 and 1709+25. These zoning designs and stability analyses are discussed more in section 8.3.2. Plate 7 shows cross-section details for Fill 1, including the zoning of the Selected E encapsulation material, the zoning of the bottom of the fill, and an additional detail showing the utilization of a subgrade enhancement geotextile to separate the subgrade fill material above from being stripped into the highly porous (due to its rocky nature) Selected E material. Plate 8 shows cross-section details of Fill 2 and Fill 3, helping to clarify the differences between Fills 2 and 3, and between them and Fill 1.

Existing fills in the western half of the project have slope ratios from about 1.9:1 to 1.5:1, the large majority of them being 1.5:1. Existing fills have a maximum height of 20 ft. The proposed fills in the western half of the project have slope ratios of 1.5:1, with the exception

of a single large double-sided embankment fill near the beginning of the project with slope ratios of 2:1 and 4:1. Proposed fills in the western half of the project area have a maximum height of about 30 ft and do not have Selected E encapsulation.

Right-of-way issues and steep topography below the roadway in several locations in the eastern half of the project limit fill options to steep (1.25:1) 'sliver' fills. These 1.25:1 fills are to be constructed entirely of Selected E material, and must have a minimum width of 6 ft perpendicular to the slope face. This means that even at the periphery or lateral edge of these 1.25:1 fills, the Selected E material forms a 6-foot wide or thick embedded section that abruptly terminates.

Disposal fills should be built no steeper than 1.5:1. These may be constructed anywhere within the project area for the purpose of disposing of excess material. These shall be constructed in areas occupied currently by existing roadway or pullouts that will be abandoned once the new alignment is constructed. These fills are to be constructed at the direction of the engineer only when all other fills are constructed and the engineer can be certain that the material is truly excess. These fills should be no taller than 30 ft.

Selected E material shall not be placed on the fill slope as a later veneer after the fill has been constructed. Selected E material shall be placed in horizontal lifts simultaneously with the general fill material, lift for lift, so that the individual lifts of the general fill material and the horizontally juxtaposed Selected E material are integrally intertongued and compacted simultaneously. The objective of such simultaneous construction is 1) to create a well-integrated, slightly rough, mildly irregular contact between the two materials rather than a relatively smooth detachable contact surface more prone to sliding, and 2) to assure that the fill is continuously protected during construction from possible rainstorms. A schematic showing the encapsulation and its lift-by-lift placement and compaction along with the general fill material is shown in Plate 9.

Most fills should be compacted to the standard specification of 90%, with the exception of the three larger 1.55:1 fills (Fill 1, 2, and 3) located at the eastern end of the project, where the compaction requirement for all fill material, with the exception of the Selected E, shall be 93%. The purpose behind the higher compaction designation is to enhance stability, since these fills are substantially large, as well as to reduce or eliminate as much as possible post-construction settlement.

8.3.1 Embankment Material

The fills are to be built from material excavated within the project limits. The project is roughly a balanced job (between cuts and fills) that includes the following geomaterials: solid rock outcrops and boulders of andesite and basalt; pyroclastic flows consisting of fairly to strongly welded/consolidated tuffs, andesite rock, and volcanic ash; colluvial overburden composed of subangular to angular andesite pieces in a matrix of varying amounts of sand and silt, and very minor to no clay; and soils derived from andesite and basalt bedrock with varying amounts of parent bedrock.

Small to extremely large boulders are found in various locations throughout the project area, both below ground surface within some of the above deposits, and above ground resting on, or partially embedded in, the surface. The extremely large boulders are mainly found in the eastern half of the project area, primarily in the pyroclastics section (1662+25 to 1709+00). Many of these boulders are fresh and may be utilized for making the 2 ton RSP required in the toe buttresses of Fill 1 and 2. These boulders may also be placed within the fills; however, the maximum size for rocks used for general embankment fill should be 5 ft in any direction. Boulders and cobbles placed in the fill must also not be rounded. Rock placed in a fill that is 18 inches or greater in any direction must have a subangular to angular shape with at least one fractured face. Flat or needle shaped rock must not be used unless the individual rock thickness is greater than 0.33 times the length.

Table 7 below presents a tabulation of Fill Material Groups by station intervals, with general material descriptions, and parameter values or ranges. Table 5 above, listed these Fill Material Groups in an up station direction by station intervals, along with the grading factor assigned to each group. These groups are denoted by an 'F' for fill, followed by a number, except for the group that is the source area for Selected E material (discussed below), which is simply denoted by 'E'. These groups are basically continuous sections, delimited by station intervals, wherein the collection of geomaterials is consistent. Each individual Fill material Group may contain multiple materials, such as andesite colluvium, intact welded tuff, and residual andesite soil, for example, because they will be excavated together as a material group and mixed somewhat by virtue of the excavation, placement, and compaction process. Each Fill Material Group has its own parameter values, and by direct corollary, its own grading factor, which were listed in Table 5 in section 8.2.3. A group can repeat itself if found further on in the project, and, indeed this does happen, as can be seen in Table 5, where F2 and F3 alternate back and forth several times in the western half of the project area before an E group comes up. Each of these 7 groups was assigned parameter values (C, φ, γ), so embankments constructed with them could be analyzed for stability, and a lump grading factor was estimated for earthwork balance calculations. Parameter values were estimated partly based on lab tests, which represent some local fines. At least equally important in the estimation of parameter values were the type and quality of bedrock, and its likely relative abundance in the quantity to be excavated; the quality, angularity, and relative abundance of colluvium; and both the seismic refraction and MASW, which helped determine rough depths to bedrock within the proposed cuts and which qualitatively assisted in assessing the density of overburden material and the quality or strength of the bedrock. Quality bedrock (including some of the hard pyroclastic such as CHWT and MCWT) and sufficient quantities of angular colluvial rock (typically andesite) could typically raise the φ angle of the Fill Material Group up to 5° over that of the fines' lab test value, based on experience and geologic engineering judgment. Lab tests were performed only for a relative few (5) of the fines. Close to fifty field hand sample evaluations (angularity, relative fines content, plasticity, dilatancy (none)…) also contributed to the assessments, assessments that were to some point 'calibrated' by the lab tests. Cohesion values for the groups were based primarily on the lab tests and less on the hand sample evaluations, with the C for the actual fines being reduced up to 80% from lab values for the group. Unit weights (γ) for the groups were based on estimates of the relative proportions of fines (and lab values for unit weights),

bedrock, colluvium, and incorporated cobbles and boulders, and the appropriate unit weights for these geomaterials. The resultant φ values are considered to be moderately conservative educated estimates from a safety and stability analysis perspective. ɣ values were estimated as a range and tabulated as such in Table 7, so that they could be applied conservatively during analysis (explained in section 8.3.3).

Grading factors for the groups were estimated in a somewhat similar fashion as the parameters. Typically loosely packed, overburden was assigned a factor between 0.8 and 0.9, depending upon the amount of colluvial rock present. Bedrock varied widely for assigned factors, from 0.9 for the ash and FWT tuff to 1.15 for the basalt bedrock. Seismic velocities of both overburdens and bedrocks contributed to the factor assessment, although the greatest weight was given to field interpretations and observations. Each constituent of the group and its assigned factor were then weighted according to the estimated relative proportions for that group.

The seventh group, E, which is the source for Selected E material, was evaluated for all three parameters and grading factor using similar methodology. The Selected E ɣ range was estimated by taking published ɣ values for andesite and basalt and reducing them based on the expected pore spaces expected between the rock pieces.

Selected E material (E) consists of excavated andesite (station 1650 to 1662+25) and basalt (station 1713+00 to 1717+00 and 1720+25 to 1723+00) that is intended to serve as encapsulation for the outer surfaces of some fills, and the entirety of some others (1.25:1) in order to provide both stability and erosion protection. Typically these rocks will be subangular to angular as a result of excavation processes. Specifications should limit Selected E material to a maximum size of 18 inches in any dimension. Selected E material must provide a stable structure for its required purpose. Selected E material must not contain rounded boulders or cobbles, and it must be subangular to angular with at least one fractured face. Flat or needle shaped rock cannot be included as Selected E material unless the individual rock thickness is greater than 0.33 times the length. Material excavated from the station intervals mentioned above for Selected E may require additional breakage beyond that occurring during excavation in order to meet these requirements. Generally, rocky material like that described as Selected E above typically has an angle of repose (φ angle) varying from about 44° to sometimes as high as 63°, depending upon the degree of angularity and 'compaction'. Uncompacted side-cast rocks are typically lower, while rocks placed in 'compacted' lifts (meaning essentially a couple of passes per lift with the appropriate compactor) usually start at about 50° and can often range up to the low 60's.

One additional material required for fill construction, specifically for Fills 1 and 2, is the 2 ton RSP required for the construction of the toe buttresses. The contractor may choose to manufacture this material from boulders and rock found within the project area, or import it from elsewhere. For purposes of stability analysis, it is assumed that this material will be composed of fractured boulders of either andesite or basalt taken from the project. Averaging the unit weight of both of these rocks and applying a reduction to account for the voids

between individual rock pieces results in a ɣ of 150 pcf. A φ angle of 56 is chosen for this material, based on the method A placement and the angularity of the rock. C is set at 0.

Table 7. Fill Material Groups, Descriptions, Parameter Values, and Station Intervals

Material Type	General Description *	ϕ°	C (psf)	ɣ (pcf)	Station Intervals
F1	Soil derived from andesite bedrock; mostly soil, includes andesite minor gravel, cobbles, and boulders.	38	400	95 - 105	1592+75to1604+50
F2	Primarily andesite bedrock, boulders, ,cobbles, and gravel with fines.	45	0	130-140	1604+50to1607+00 1610+00to1612+75 1620+50to1624+00
F3	Soil derived from andesite bedrock; typically possessing more rocky material (andesite gravel, cobbles, and boulders) than F1	40	400	100 - 115	1607+00to1610+00 1612+75to1620+50 1624+00to1650+00
F4	Soil derived primarily from fine pyroclastic ash with some gravel and rock embedded; some locations have considerable subangular andesite colluvium	36	1000	90-120	1662+25to1682+00 1686+00to1695+00
F5	Predominantly composed of coarse pyroclastic welded tuffs (CHWT and MCWT) beneath lesser overburden soil derived from andesite colluvium	42	50	110-125	1682+00to1686+00 1695+00to1709+25
F6	Soil derived from andesite colluvium and weathered basalt. Colluvial andesitic/basaltic gravel, cobbles, and boulders	42	20 - 100	95 - 110	1717+00to1720+25 1730+25to1732+50
E	Andesite and basalt bedrock outcrops. Selected E material source. When excavated and 'processed" to specification quality and 'compacted' these parameter values apply	50 - 56	0	140 - 160	1650+00to1662+25 1713+00to1717+00 1720+25to1723+00

Notes: * ~95% of non-rock Fill (F#) material is classified as either silty sand (SM), Silty Sand w/ Gravel (SM), silty sand w/ gravel and cobbles (SM), or sand with silt and gravel (SW-SM).

Any of the 7 Fill Material Groups could technically be used as general fill material. The Selected E material (E), however, must be reserved for encapsulation purposes and for the construction of steeper fills as described and specified in section 8.3.3. In addition, a section of Fill Material Group F5, from station 1695+00 to 1709+00, must also be reserved for the bottom of Fill 1, as will be further discussed in section 8.3.3. General fill material shall be used to build the core of all fills with slope ratios equal to or flatter than 1.5:1. General fill material shall not be used to construct any fills steeper than 1.5:1.

8.3.2 Embankment Stability Analysis

Slope stability analyses were performed using the computer software Slope/W. The Morgenstern-Price limit equilibrium method was employed in all analyses. A value of 0.1 was used for the horizontal coefficient k_h when performing pseudo-static analysis to assess the stability of the fills under seismic loading.

Parameter values used in the stability analyses were taken from Table 7. A φ angle of 36° was used for general fill material because it is the lowest φ angle of all the Fill Material Groups, and as such is the most conservative choice. Cohesion (C) was generally not applied in the analyses, as is often typical engineering practice, because of the unreliability of C in certain situations such as long term saturation from heavy rains. Alternate analyses were performed using some C, strictly to obtain some idea of how stable the fills would likely be in reality, most, if not all, of the time. Unit weight (ɣ) values were tested with alternate analyses in order to determine which produced the more conservative results. Heavier material above should create more driving force, so applying the larger ɣ for material above should result in more conservative results. Similarly, utilizing a smaller ɣ from the range in Table 7 for material located near the bottom should also produce conservative results. After examining the alternate analyses, one value for ɣ was chosen.

<u>Fills 1, 2, & 3 (1.55:1)</u>

Table 8 provides the parameter values used for the stability analyses of Fill 1, together with the FOS results for non-dynamic and dynamic (pseudo-static horizontal seismic loading) analyses. Plate 10-A and 10-B show the cross-sections from the Slope/w program, with 10-A depicting the model and 10-B showing the model with the critical slip area. As mentioned earlier, stability analysis indicated that Fill 1 required a more stable material (basically with a higher φ angle) than the general fill material in the lower core of the fill, in order to achieve a FOS of 1.3. This stable base material was assigned the parameter values of Fill Material Group F5 (to be excavated between station 1695+00 and 1709+00), as mentioned above. This station interval was chosen because, besides being sufficiently strong (high enough φ angle), it is located immediately to the west of Fill 1, which makes for efficient construction.

The piezometric surface for all three fills was modelled within the colluvial original ground, which is generally about 10 to 15 ft thick and sits on bedrock. This piezometric location is probably present only within the general thalweg area for Fill 1 and Fill 2, and is likely not present in this location for Fill 3, because the water table is likely deeper within the fractures

of the bedrock outside of the thalweg for Fill 1 and 2 and for the entire area covered by Fill 3. Modelling the ground water table this way lends conservatism to the analysis. None of the ground water sources and seeps observed elsewhere in the project were observed within the footprint of the fills during field investigations. The effect of surface waters should be fairly mild for all three fills, coming only from the roadway runoff atop the fill and precipitation striking the encapsulation surface. Given the AC roadway cap atop the fills, the 95% compaction of the subgrade, and the 93% compaction for the fill material, saturation of the fills is not expected to occur during times of high precipitation, except perhaps within the outer two or three feet of general fill material that lies beneath the encapsulation.

Table 8. Parameter Values and FOS Results For Stability Analysis of Fill 1.

Material	ϕ°	C (psf)	γ (pcf)	FOS	FOS w/Cohesion	Seismic Analysis (k_h=0.1)
General Fill	36	0 (10)	115	1.3 (1.296)	1.3 (1.307)	1.1 (1.064)
E	52	0	150			
Fill Base	40	0	120			
Toe Buttress	56	0	150			
Original Ground (overburden)	42	0	115			
Original Ground (Bedrock)	na	na	165			

Stability analysis of Fill 1 produced a FOS of 1.3 (1.296 before rounding) for cohesionless general fill material, and a FOS of 1.3 (1.307 before rounding) for general fill material with a meager 10 psf of C. When modelling with C the outer 3 horizontal feet of both the general fill material and the F5 material (fill base material) is left without cohesion to simulate potential loss of C brought on by saturation due to heavy rains. This outer 3 feet is similarly modelled for Fill 2 and Fill 3, and is shown in the respective plates. The FOS produced for pseudo-static horizontal seismic loading for Fill 1 was 1.1 (1.064 before rounding), which meets the Corps of Engineers requirement calling for a FOS > 1.

Table 9 provides the parameter values used for the stability analyses of Fill 2, together with the FOS results for non-dynamic and dynamic analyses. Plate 11-A and 11-B show the cross-sections from the Slope/w program, with 11-A depicting the model and 11-B showing the model with the critical slip area. Stability analysis of Fill 2 produced a FOS of 1.3 (1.289 before rounding) for cohesionless general fill material, and a FOS of 1.3 (1.296 before rounding) for general fill material with just 10 psf of C. Much the same as for Fill 1, when modelling with C the outer 3 horizontal feet of the general fill material is left without cohesion to simulate potential loss of C brought on by saturation due to heavy rains. The

FOS produced for pseudo-static horizontal seismic loading was 1.1 (1.057 before rounding), which meets the Corps of Engineers requirement calling for a FOS > 1.

Table 9. Parameter Values and FOS Results For Stability Analysis of Fill 2.

Material	ϕ°	C (psf)	ɣ (pcf)	FOS	FOS w/Cohesion	Seismic Analysis (k_h=0.1)
General Fill	36	0 (10)	115	1.3 (1.289)	1.3 (1.296)	1.1 (1.057)
E	52	0	150			
Toe Buttress	56	0	150			
Original Ground (overburden)	42	0	115			
Original Ground (Bedrock)	na	na	165			

Table 10 provides the parameter values used for the stability analyses of Fill 3, together with the Factor of Safety (FOS) results for non-dynamic and dynamic analyses. Plate 12-A and 12-B show the cross-sections from the Slope/w program, with 12-A depicting the model and 12-B showing the model with the critical slip area. Stability analysis of Fill 3 produced a FOS of 1.3 (1.278 before rounding) for cohesionless general fill material, and a FOS of 1.3 (1.296 before rounding) for general fill material with just 10 psf of C. Much the same as for Fills 1 and 2, when modelling with C the outer 3 horizontal feet of both the general fill material and the F5 material is left without cohesion to simulate potential loss of C brought on by saturation due to heavy rains. The FOS produced for pseudo-static horizontal seismic loading was 1.00 (1.041 before rounding), which barely meets the Corps of Engineers requirement calling for a FOS > 1.

Table 10. Parameter Values and FOS Results For Stability Analysis of Fill 3.

Material	ϕ°	C (psf)	ɣ (pcf)	FOS	FOS w/Cohesion	Seismic Analysis (k_h=0.1)
General Fill	36	0 (10)	115	1.3 (1.278)	1.3 (1.296)	1.0 (1.041)
E	52	0	150			
Original Ground (overburden)	42	0	115			
Original Ground (Bedrock)	na	na	165			

Based on the above results, designs for Fills 1, 2, and 3 are all judged to be sufficiently stable.

1.25:1 Fills

Table 11 provides the parameter values used for the stability analyses of the two main geometric variants of the proposed 1.25:1 fills, together with the FOS results for both non-dynamic and dynamic analyses. Plate 13-A and 13-B show the cross-sections from the Slope/w program for these two models, one a thin 50-foot sliver fill, and the second, an 85-foot slightly thicker sliver fill. There is no piezometric surface shown, since OGDN believes that the water table is well below the area influencing stability. Before analyzing the stability of the additional E fill material, it was first necessary to back-analyze the existing slopes, which, with a slope ratio of 1.25:1 and the material parameters previously defined, could not produce a FOS of 1.3 without utilizing cohesion and adding a couple of degrees to the φ angle for original ground, despite their having proven their stability empirically for decades. This discrepancy is likely the product of conservative modelling, including the non-utilization of cohesion in the fill models. Consequently, the φ angle of the original ground was raised to 42° (a reasonable adjustment given lab results for some material and field examination of the material in these areas) and cohesion was added during the back-analysis until a FOS of 1.3 was reached. Then the same φ angle and cohesion value was utilized in modelling the addition of the 1.25:1 sliver fills. The resultant cohesion values utilized for original ground in the modelling of the 50-foot and 80-foot sliver fills were 30 psf and 44 psf, respectively, values which are relatively small and considered very reasonable and mildly conservative given some of the values measured in the lab tests. Stability analysis of the 50-foot 1.25:1 fill produced a FOS of 1.3 (1.312 before rounding), which was essentially the same (or slightly better if rounding is not performed) as the native slopes, demonstrating that the addition of the sliver fill did not reduce stability. The FOS produced for the 50-foot sliver fill model with pseudo-static horizontal seismic loading was 1.2 (1.241 before rounding), which easily meets the Corps of Engineers requirement calling for a FOS > 1. It should be noted that cohesion was used only to allow the existing ground to meet the required FOS of 1.3. No cohesion was used in the sliver fill material (Selected E material). The same general results were produced for the 80-foot sliver fill, which saw a FOS of 1.3 (1.316 before rounding) and a pseudo-static FOS of 1.2 (1.187 before rounding). Based on these results, designs for these 1.25:1 sliver fills are judged to be sufficiently globally stable.

Surficial stability of these 1.25:1 sliver fills needs to be addressed, however, as sliver fills have a typical proclivity to slip from the original ground surface unless well-constructed according with a 6-foot key-in as called for by the standard specifications. OGDN believes that the steepness of these fills, combined with their considerable heights, calls for additional design efforts to insure these problems due not arise, particularly since the travelled way actually sits above some of these fills. Therefore, these fills should have a minimum of 6 ft of Selected E perpendicular to the slope face in all locations. This means that the keys for the bottom of these fills shall be excavated 6 ft deep and filled with 6 ft Selected E material, not original ground material. This requirement also applies to the peripheral edges of the fills, where they should terminate in a blunt 6-foot thick section (perpendicular to slope face) of Selected E material.

Table 11. Parameter Values and FOS Results for Stability Analysis of 1.25:1 Fills.

Fill	Material	ϕ°	C (psf)	ɣ (pcf)	FOS Existing Slope	FOS	Seismic Analysis (k_h=0.1)
50ft sliver@1.25:1	E	52	0	150	1.3 (1.300)	1.3 (1.312)	1.2 (1.240)
	Original Ground	42	30	110			
85 ft @ 1.25:1	E	52	0	150	1.3 (1.300)	1.3 (1.316)	1.2 (1.187)
	Orignal Ground	42	44	110			

8.2.4 Embankments - Founding and Settlement

Fill materials throughout the project area are generally classified as cohesionless, and as such, should undergo internal settlement very quickly within the fills they are placed, essentially during the construction process. The three large fills (Fill 1, Fill 2, and Fill 3) have the largest maximum internal vertical columns of material of 60 ft, 30 ft, and 30 ft, respectively, but these still create relatively insignificant forces to create any noticeable internal settlement to fills that have been compacted to 93%, based on previous experience with other large, or larger cohesionless fills. The 1.5:1 fills in the western half of the project area have maximum internal soil columns of about 8 ft, which would create internal forces also incapable of driving any noticeable settlement for fills compacted to 90%. The 1.25:1 fills in the eastern half of the project area are fairly thin and are composed entirely of rock, which, once compacted (rolled by a compactor) will be essentially non-compressible internally. So OGDN expects negligible post-construction settlement internally from the fills post.

Based on seismic refraction surveys, lab test results, and geological field relationships, founding conditions generally consist of a silty sand overburden, with varying amounts of gravel, cobbles, and boulders, all overlying competent bedrock. These overburden deposits are generally no more than 10 to 15 feet thick in most fill locations. Though not compacted lift by lift by mechanical means, these deposits, by virtue of time and gravity, have likely achieved an average relative compaction somewhere between 77% and 88%, based on experience. These deposits generally have considerable amounts of rock, which are virtually incompressible at the relatively small loads created by even the largest fill (Fill 1) in this project.

These overburden deposits are essentially classified as coarse grained to very coarse grained deposits when it comes to compaction/settlement analysis. Utilizing estimation methods provided in Corps of Engineers (1982) for coarse grained soils, OGDN estimates the maximum total settlement for the largest fill (Fill 1) to be about 1 inch. Based on previous

experience in areas where the founding conditions also consisted of coarse-grained soils compacted primarily during Quaternary time (2 Mya to present), we would estimate a maximum post-construction settlement of less than 1 inch for the largest fill and considerably less for the others.

In conclusion, total post-construction settlement for Fill 1 should not exceed 1 inch, while Fill 2 and 3 should likely not exceed 0.5 inches. The 1.25:1 fills should demonstrate negligible total post-construction settlement. The 1.5:1 fills should exhibit no more than 0.3 inches of total post-construction settlement.

8.2.5 Embankments-Erosion

Fill 1, 2, and 3 are all encapsulated with rocky Selected E material and should be highly resistant to erosion, both during – due to the synchronous placement and compaction of fill material and the encapsulating Selected E material -- and after construction.

1.25:1 fills are all constructed entirely of Selected E material and, as such, should be highly resistant to erosion.

Fills in the western half of the project constructed at slope ratios of 1.5:1 or flatter are not encapsulated so they will not possess the high resistance to erosion that the fills discussed above do. However, based on the performance of existing fills in the western half of the project area, the proposed fills are likely to stand up fairly well to erosion provided they are not exposed to concentrated sheet flow and they are protected during their firs overwintering period with some type of protective mulch.

8.2.6 Embankments - Drainage

No seeps were discovered within the footprint area of the proposed 1.55:1 fills, 1.25:1 fills or 1.5:1 fills, so no specific drainage blanket systems need be designed for seeps. Fill 1 and Fill 2 are located in drainage basins roughly centered atop central thalwegs that intermittently carry surface water, and almost certainly serve as year-round subsurface ground water conduits. An underdrain system must be constructed within these thalwegs beneath the proposed fills.

No additional drainage systems are recommended for the embankments. It should be noted, however, that seeps might still be discovered in the founding area(s) of one or more of the proposed fills during construction when the area has been cleared and grubbed, which would then require the design and construction of the appropriate drainage system(s) at the point in time.

Existing culverts in the locations of the future fills should either be properly abandoned (concreted and plugged or removed), or else replaced and modified to work with the new proposed fills.

9. Construction Considerations

9.1 Construction Advisories

Prior to cutting slopes, vegetation should be completely cleared and grubbed within the excavation and fill slope lines to prevent organics from being included in fill material. Trees larger than 6" at chest height that are situated within 5 feet outside of the excavation lines should also be cut. Smaller vegetation may remain in place in this area outside of the cut zone. Stumps from the trees cut within this 5-foot zone shall be left in place at a height of 10 inches to 24 inches above the surrounding ground.

Cut slopes in the eastern half of the project area are likely to shed some rockfall during construction; therefore, appropriate caution around these cuts should be exercised by construction forces.

Boulders of various sizes throughout the project that lie slightly outside of the cut line may still require removal if they are judged to be a potential falling/rolling hazard to either construction forces or the travelling public after construction is completed.

Once ground has been cleared and prepped for fill construction, the areas to be buried beneath fills should be examined by the engineer (and, possibly a representative from OGDN) to assess the ground for any possible signs of ground water seeps that might have been missed during the investigation period when heavy brush may have concealed them. Should any such wet areas be found drainage should be designed and installed that allows water from these areas to escape unimpeded from beneath the planned fill. Such systems should include an underdrain, similar to that installed in the thalwegs beneath Fill 1 and Fill 2. These systems might also entail a broader permeable blanket, consisting of angular drain rock enveloped above and below by non-woven Type B RSP fabric. Should any such seeps be encountered, please contact Mr. Lewis of OGDN.

9.2 Construction Considerations that Influence Design

The considerable likelihood of some failed cut slopes and increased rockfall impacting the completed constructed highway during the first winter following construction, due to the change from a two-year construction period to a single construction season as explained previously in section 8.2.1, should be mitigated wherever possible by Design. OGDN strongly recommends that the contractor be made available for cleanup, repair, and related storm water issues through the primary construction contract for a period of up to one year following the completion of construction. This will allow a swift response to any rockfall and slide problems, prevent maintenance from being unduly burdened by these problems, and prevent the more costly and far less effective approach of utilizing director's orders to address said problems.

OGDN also recommends that excavation within the pyroclastics section (station 1663+00 to 1709+00) be staged as early as possible in the construction period so that there is more time,

albeit in the dry season, to assess the performance of the cut slopes and to respond with any needed mitigation or corrective efforts as early as possible.

9.3 Construction Monitoring

No instrument monitoring of the cuts or fills is recommended.

9.4 Differing or Problematic Site Conditions

Should differing site conditions arise during construction please contact Mr. Lewis of OGDN.

10. Recommendations and Specifications

10.1. Cut Slopes

- Prior to cutting slopes, it is recommended that trees larger than 6" at chest height that are situated within 5 feet outside of the excavation lines be cut. Stumps from trees cut within this 5-foot zone shall be left at a height of 10 inches to 24 inches above surrounding ground.

- Recommended cut slope ratios vary from 0.5:1 to 1:1. Recommended cut slope ratios and maximum cut slope heights are listed in Table 3, section 8.2.1.

- No benches are recommended for any cuts.

- Recommended catchment widths are listed by station interval and cut slope height in Table 6, section 8.2.4. Catchments shall be unpaved and have a backslope of 6:1, except where the surface is composed of shoulder backing, in which case the slope ratio may be flatter.

- If blasting is chosen by the contractor to construct final rock cut slope faces, presplitting methods and specifications should apply, and federal, state, and local blasting regulations must be complied with. If blasting is chosen for any purpose other than presplitting, or if blasting is to occur within 500 feet of any structure, the contractor must comply with controlled blasting specifications. All rock excavation shall be paid for as roadway excavation; the contractor must adjust his bid accordingly. All specifications regarding blasting mentioned in this bullet shall be included as part of the supplemental project information as an nSSP (written collaboratively by the Office of the Engineer and OGDN) replacing section 19.4 in the standard specifications.

- OGDN strongly recommends that a post-construction period be established and inserted into the contract by an nSSP wherein the contractor remains responsible for erosion, storm water, and slide and slipout repair work for a period lasting into the construction period of the following year. This nSSP has been created collaboratively by the Office

Engineer, the Office of Design, and OGDN, and it refers to this work as permanent erosion control establishment work. This work begins after permanent erosion control work has been completed, which typically occurs at the very end of construction.

- OGDN recommends replacing the existing horizontal drain pvc pipe at station 1679+00 after cutting the existing slope. The pipe should extrude 3 to 4 ft from the cut face surface, much as it does from the existing slope.

- OGDN recommends installing underdrains beneath the new cut slopes from station 1676+00 to 1680+50.

10.2 Embankments

- OGDN recommends that the fills proposed for the western half of the project area (station 1590+00 to 1645+00) be constructed at a slope ratio of 1.5:1 or flatter, and that they be compacted to at least 90% compaction (CTM216).

- Three large fills, referred to in this report as Fill 1, Fill 2, and Fill3, located in the eastern half of the project area at station station interval 1708+30 to 1713+00 LT (Fill 1), 1717+75 to 1720+10 LT (Fill 2), and 1723+00 to 1729+00 LT (Fill 3) shall be constructed with a slope ratio of 1.55:1.

- Fill 1, Fill 2, and Fill 3 shall be encapsulated with Selected E material. Fill 1 and Fill 2 shall have 10 horizontal ft of Selected E encapsulation. Fill 3 shall have 6 horizontal ft of encapsulation. Drawings of these recommended encapsulation designs are shown in Plates 7 and 8.

- The lower portion of fill material for Fill 1, up to a minimum elevation datum of 4155 ft, shall be constructed from material excavated from station 1695+00 to 1709+00. No material excavated from this station interval may be used elsewhere until this minimum elevation datum is reached for Fill 1.

- Fill material in Fill 1, Fill 2, and Fill 3 shall be compacted to at least 93%.

- Selected E material shall be compacted to standard specifications of 90% relative compaction, but because of its rocky nature it will not be subject to compaction tests. Selected E material shall be deemed ‘compacted’ by the engineer after two passes by the compactor; the engineer retains the right to request a third pass.

- The remaining proposed fills between station 1648+00 and 1700+00 are primarily thin fills or sliver fills. These shall all be constructed at a slope ratio of 1.25:1.

- Fills constructed at 1.25:1 shall be constructed entirely with Selected E material.

- 1.25:1 fills shall be constructed to the standard specifications of 90% relative compaction.

- Selected E material shall be comprised of all material excavated between station intervals 1650+00 to 1662+25, 1713+00 to 1717+00, and 1720+25 to 1723+00. Selected E material shall be used solely for encapsulation of Fill 1, Fill 2, and Fill 3, and the construction of all 1.25:1 fills. Selected E material may only be used elsewhere when Fills 1, 2, and 3, and all 1.25:1 fills have been completed.

- Specifications should limit Selected E material to a maximum size of 18 inches in any dimension. Selected E material must provide a stable structure for its required purpose. Selected E material must not contain rounded boulders or cobbles, and must be subangular to angular with at least one fractured face. Flat or needle shaped rock cannot be included as Selected E material unless the individual rock thickness is greater than 0.33 times the length.

- All fills designed at slope ratios of 1.25:1 shall have a minimum thickness of 6 ft (perpendicular to the slope face) of Selected E material). This will require the removal of material excavated during the keying-in process and its replacement by Selected E material.

- Buttresses shall be constructed at the toe of Fill 1 and Fill 2. These shall have a cross-sectional design as shown in Plate 6 and have a width of 60 ft (30 ft on either side of the thalweg). These shall be constructed with 2-Ton RSP, method A placement.

- 2-Ton RSP used for buttresses shall be manufactured from fresh boulders within the project limits or imported.

- In the case of Fill 1, 2, and 3, Selected E material shall be placed in horizontal lifts simultaneously with the general fill material, lift for lift, so that the individual lifts of the general fill material and the horizontally juxtaposed Selected E material are compacted simultaneously. Selected E material shall not be placed on the slopes of these fills as a later veneer after the fill has been constructed.

- Maximum size for rocks used for general fill shall be 5 ft in any direction. Boulders and cobbles placed in the fill must also not be rounded. Rock that is 18 inches or greater in any direction must have a subangular to angular shape with at least one fractured face before being placed in a fill. Flat or needle shaped rock must not be used unless the individual rock thickness is greater than 0.33 times the length.

- It is recommended that all fill faces not encapsulated with Selected E material be protected with some type of erosion protective mulch sometime after the fill is completed and before the first rains occur.

- It is recommended that underdrains be constructed within the thalwegs that lie beneath proposed Fill 1 and Fill 2.

- Present culverts in the locations of the future fills should be properly abandoned (concreted and plugged) to prevent ground water from finding them and focusing water directly at the foundation of the future fills.

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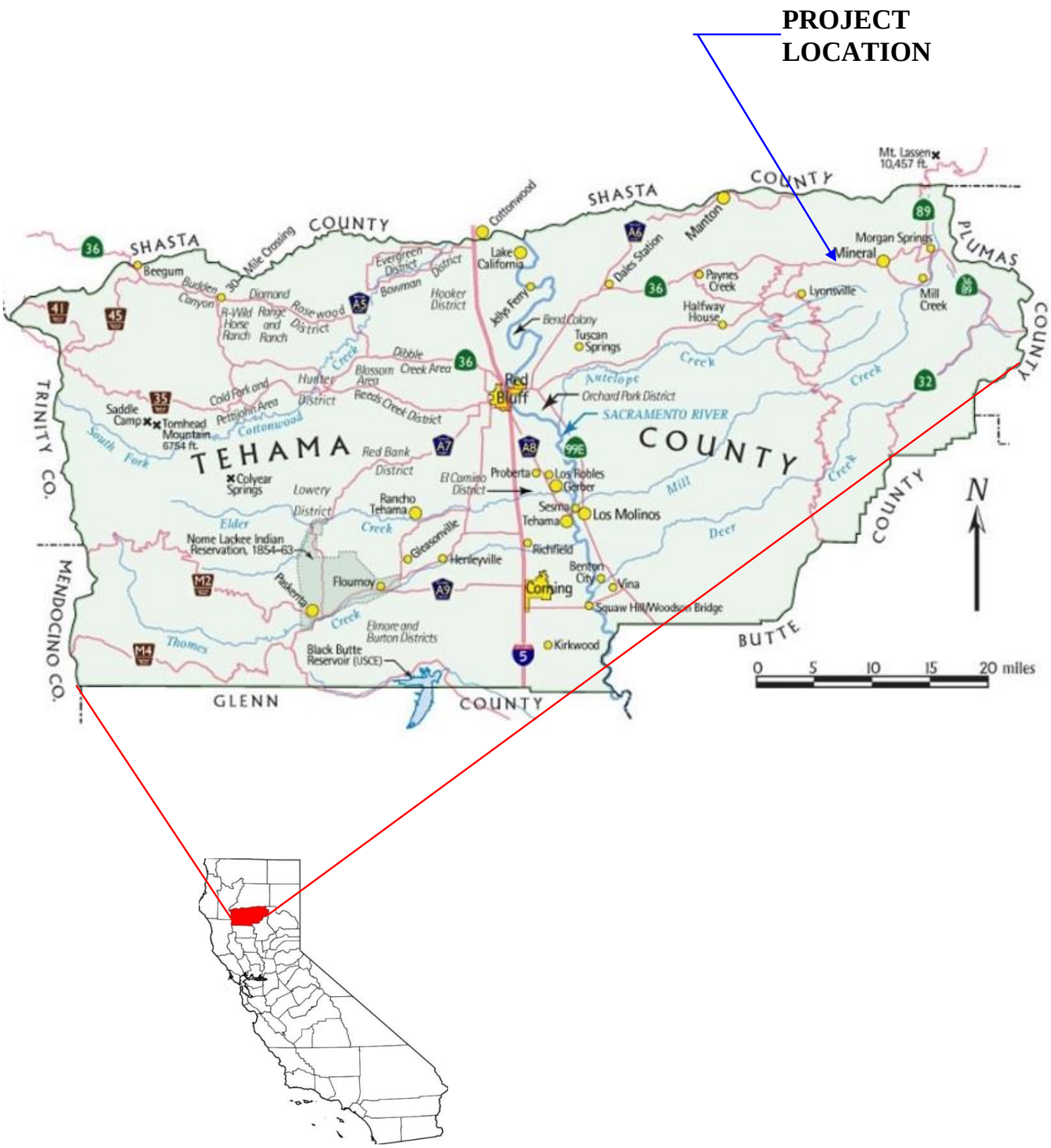
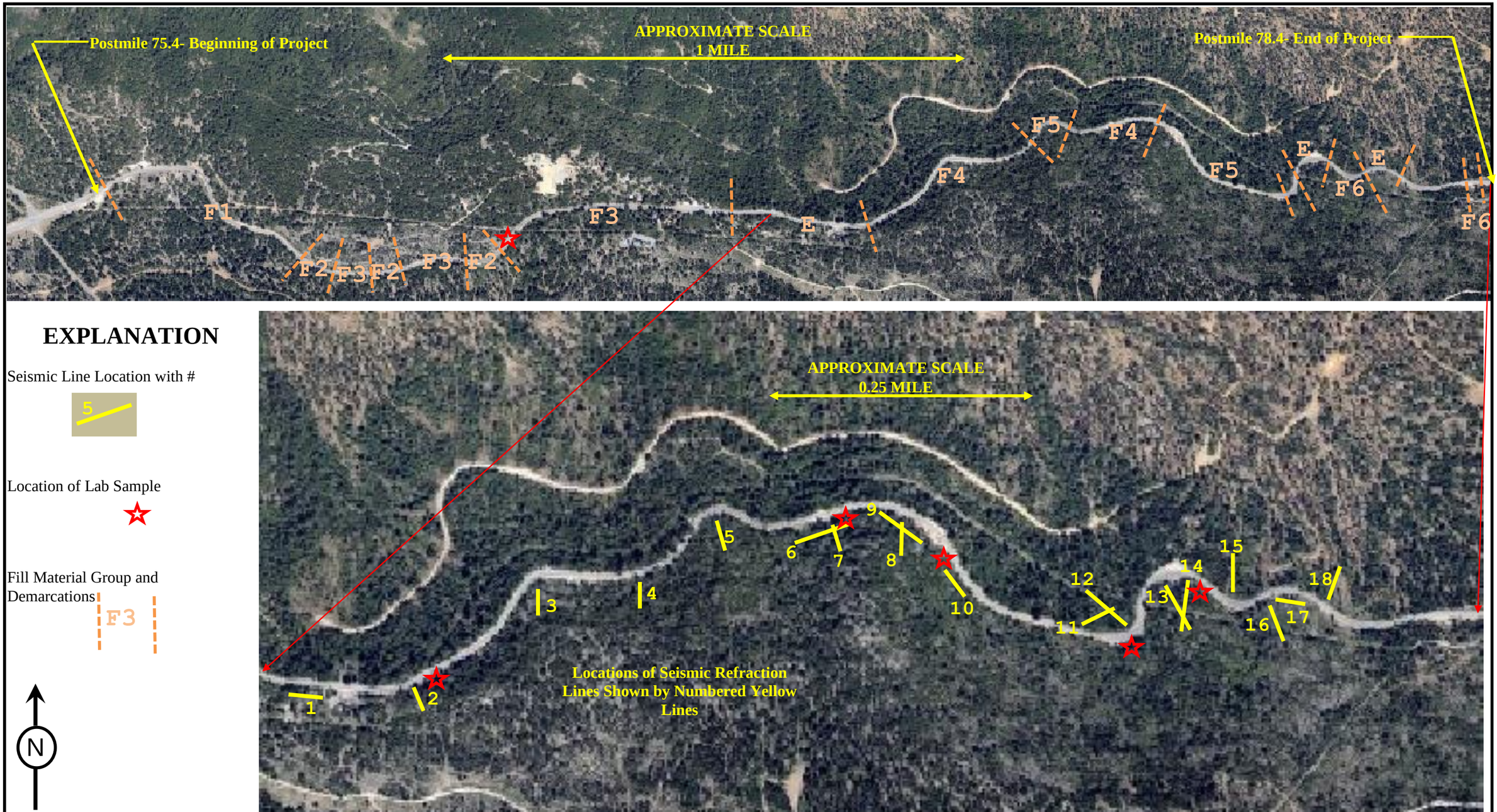
12. APPENDICES

12.1. Plates

Plate 1. Project Location
Plate 2. Aerial Photo of Lassen Lodge Project Area with Seismic Lines and Laboratory Sample Locations.
Plate 3. Topographic Map of Lassen Lodge Project Area
Plate 4. Geologic Map of Lassen Lodge Project Area
Plate 5. Geology Overlain on Aerial Photo of Lassen Lodge Project Area
Plate 6. Cross-Section of Toe Buttress for Fill 1 & 2.
Plate 7. Fill 1 Cross-Section Details (showing base zoning and separation geotextile)
Plate 8. Fill 2 and 3 Cross-Sections
Plate 9. Placement and Compaction of Fill and Selected E
Plate 10. Cross-Sections of Slope/W Models for Fill 1 without and with Critical Slip Area
Plate 11. Cross-Sections of Slope/W Models for Fill 2 without and with Critical Slip Area
Plate 12. Cross-Sections of Slope/W Models for Fill 3 without and with Critical Slip Area
Plate 13. Cross-Sections of Slope/W Models for 1.25:1 Fills



CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**PROJECT LOCATION**
	02-TEH-36 PM 75.1 /78.4 GEOTECHNICAL DESIGN REPORT	**Plate No. 1**



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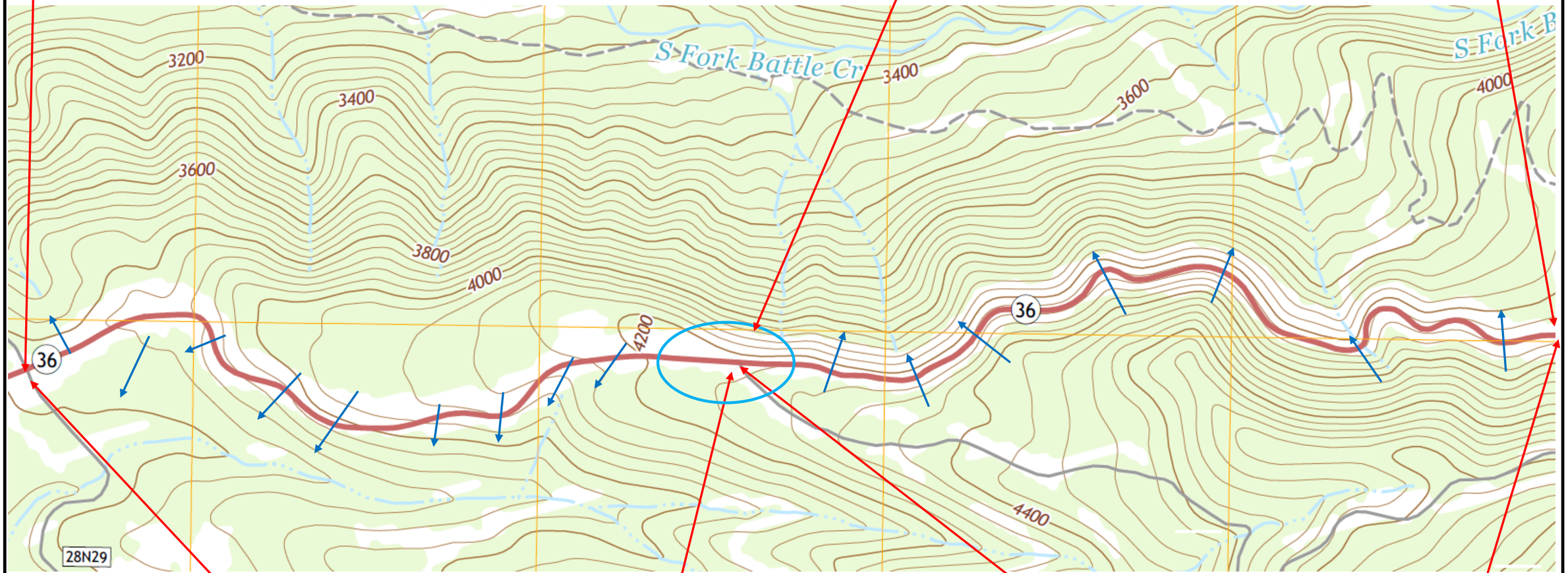
Date: June 2016

AERIAL OF PROJECT AREA w/ LOCATION OF SEISMIC LINES, LAB SAMPLES & FILL MATERIAL GROUPS

02-TEH-36 PM 75.1 /78.4
GEOTECHNICAL DESIGN REPORT

Plate No.2



APPROXIMATE SCALE
1 MILE

Blue arrows show general flow direction of surface water. Generally flow goes to the south in the western half of the project area and to the north in the eastern half of the project.

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TOPOGRAPHIC MAP OF LASSEN LODGE PROJECT AREA w/SURFACE FLOW DIRECTIONS

02-TEH-36 PM 75.1/78.4
GEOTECHNICAL DESIGN REPORT

Plate No.3

EXPLANATION
Geologic Units

Pv^a. Pliocene Volcanic Andesite.

Pv^b. Pliocene Volcanic Basalt.

Pv^p Pliocene Volcanic Pyroclastics.

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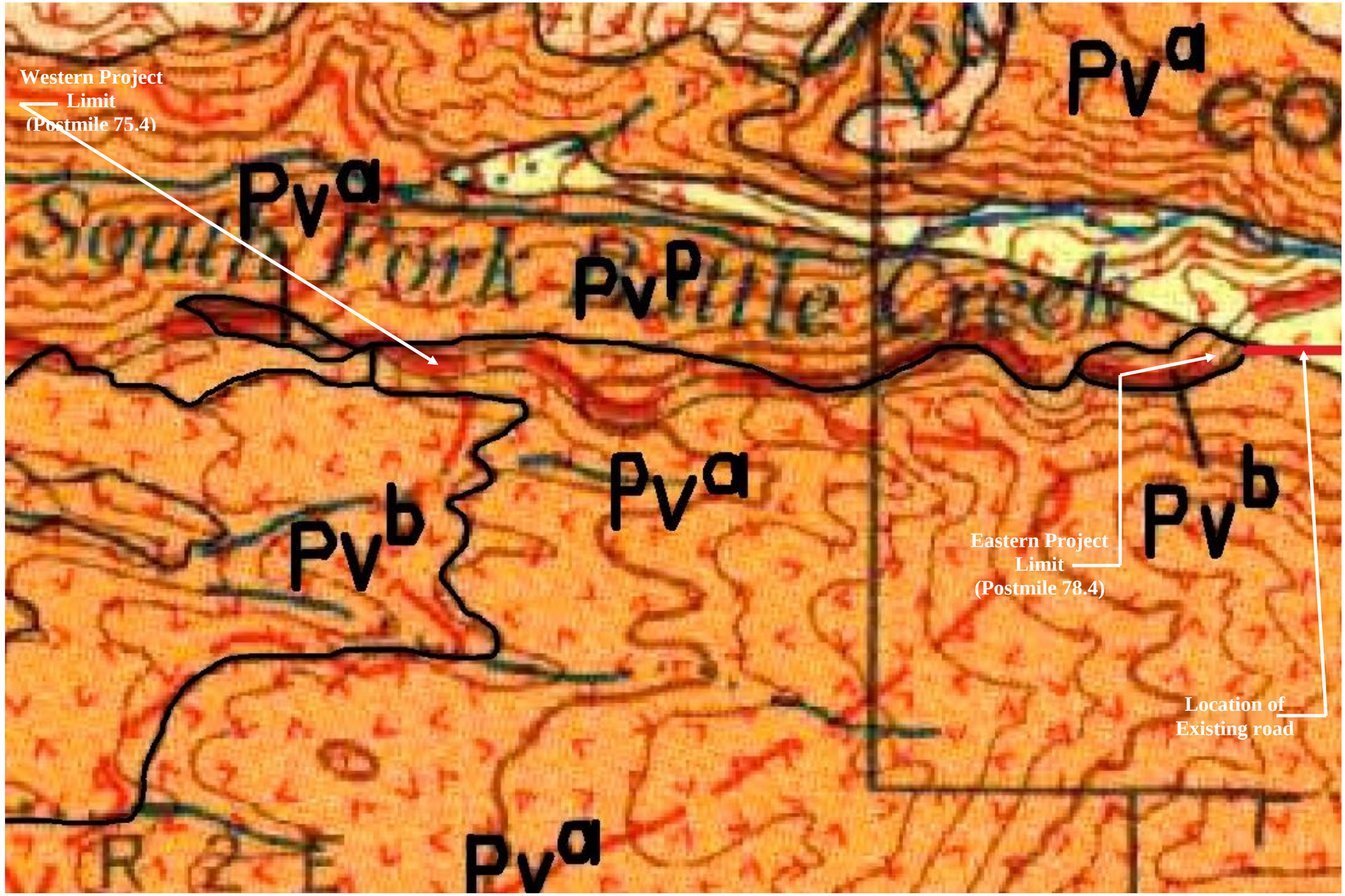
APPROXIMATE SCALE
1 MILE

From *Geologic Map of California, Westwood Sheet, California* by Lydon, Gay, and Jennings, 1960.



CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**GEOLOGIC MAP OF LASSEN LODGE PROJECT AREA**
	02-TEH-36 PM 75.1 /78.4 **GEOTECHNICAL DESIGN REPORT**	**Plate No.4**

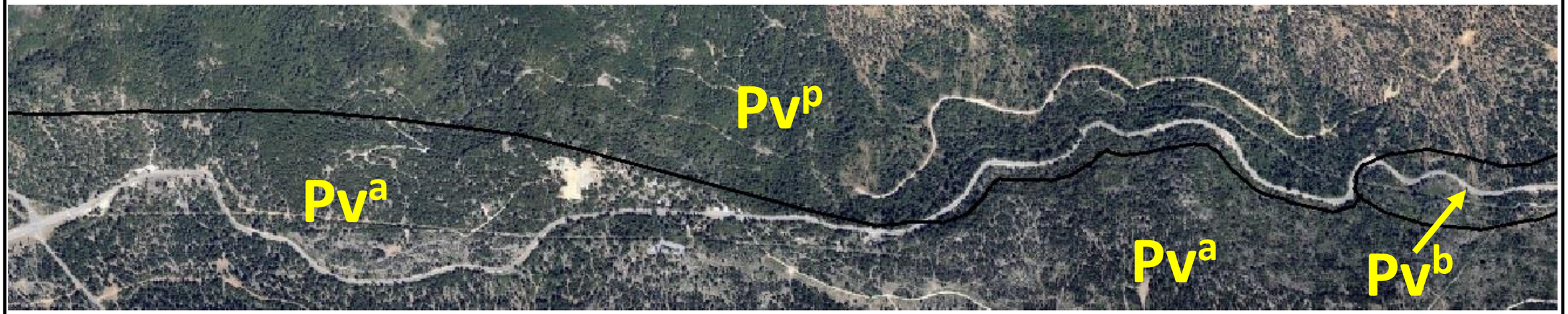
EXPLANATION
Geologic Units

Pv^p Pliocene Volcanic Pyroclastics

Pv^a. Pliocene Volcanic Andesite.

Pv^b. Pliocene Volcanic Basalt.

Pv^p typically overlies **Pv^a**. Depositional relationship between **Pv^b** and both **Pv^a** & **Pv^p** is uncertain.



The contact between Pv^p and Pv^a or Pv^b in the eastern half of project area is a close approximation based on field relations/observations, mapping taken from *Geologic Map of California, Westwood Sheet, California* (Lydon, Gay, and Jennings, 1960), and results of seismic work. The same contact in the western part of the project area (between Pv^p and Pv^a) is an approximation taken solely from the *Geologic Map of California, Westwood Sheet, California* (Lydon, Gay, and Jennings, 1960).

APPROXIMATE SCALE
1 MILE

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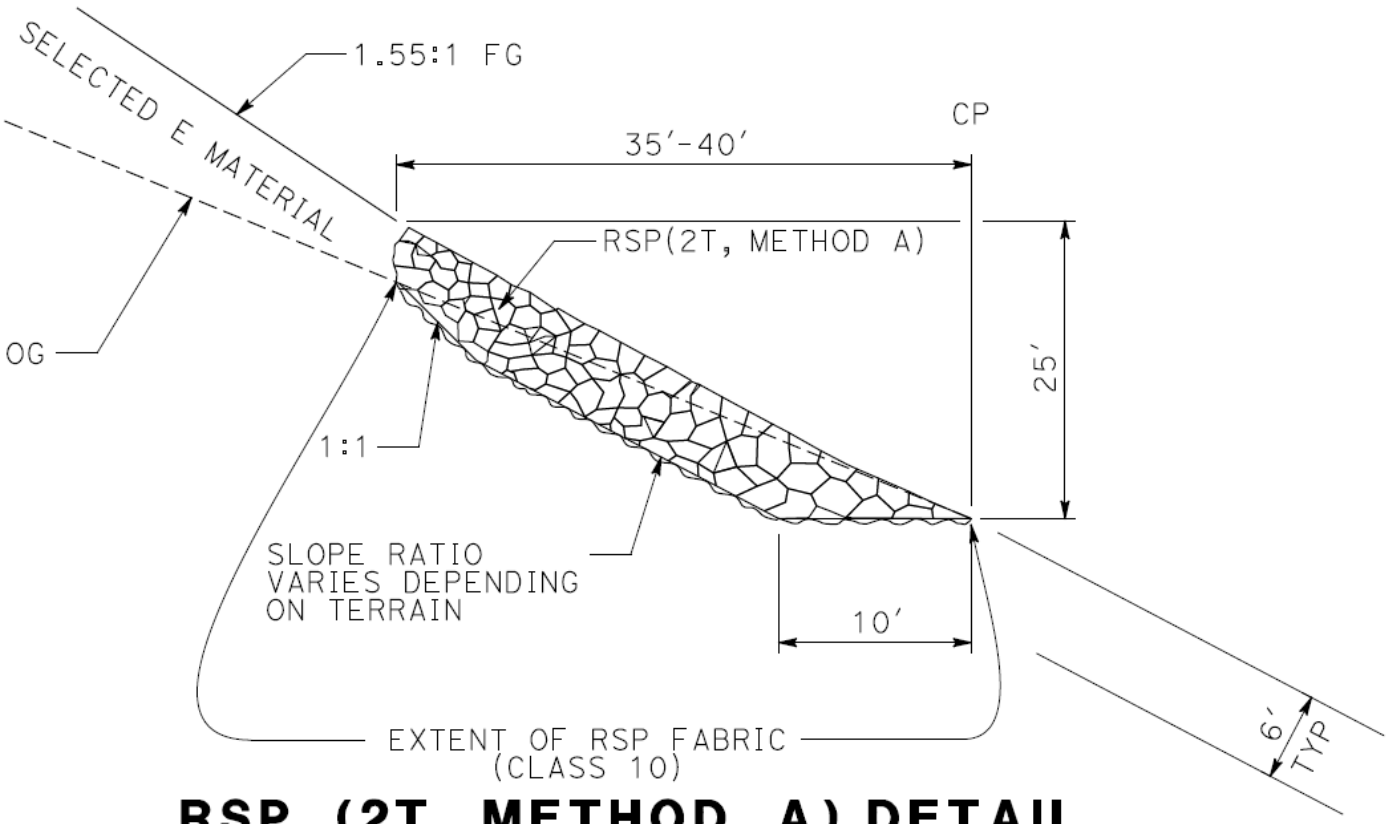
EA: 02-4G03U

Date: June 2016

GEOLOGY OVERLAIN ON AERIAL PHOTO OF LASSEN LODGE PROJECT AREA

02-TEH-36 PM 75.1/78.4
GEOTECHNICAL DESIGN REPORT

Plate No.5

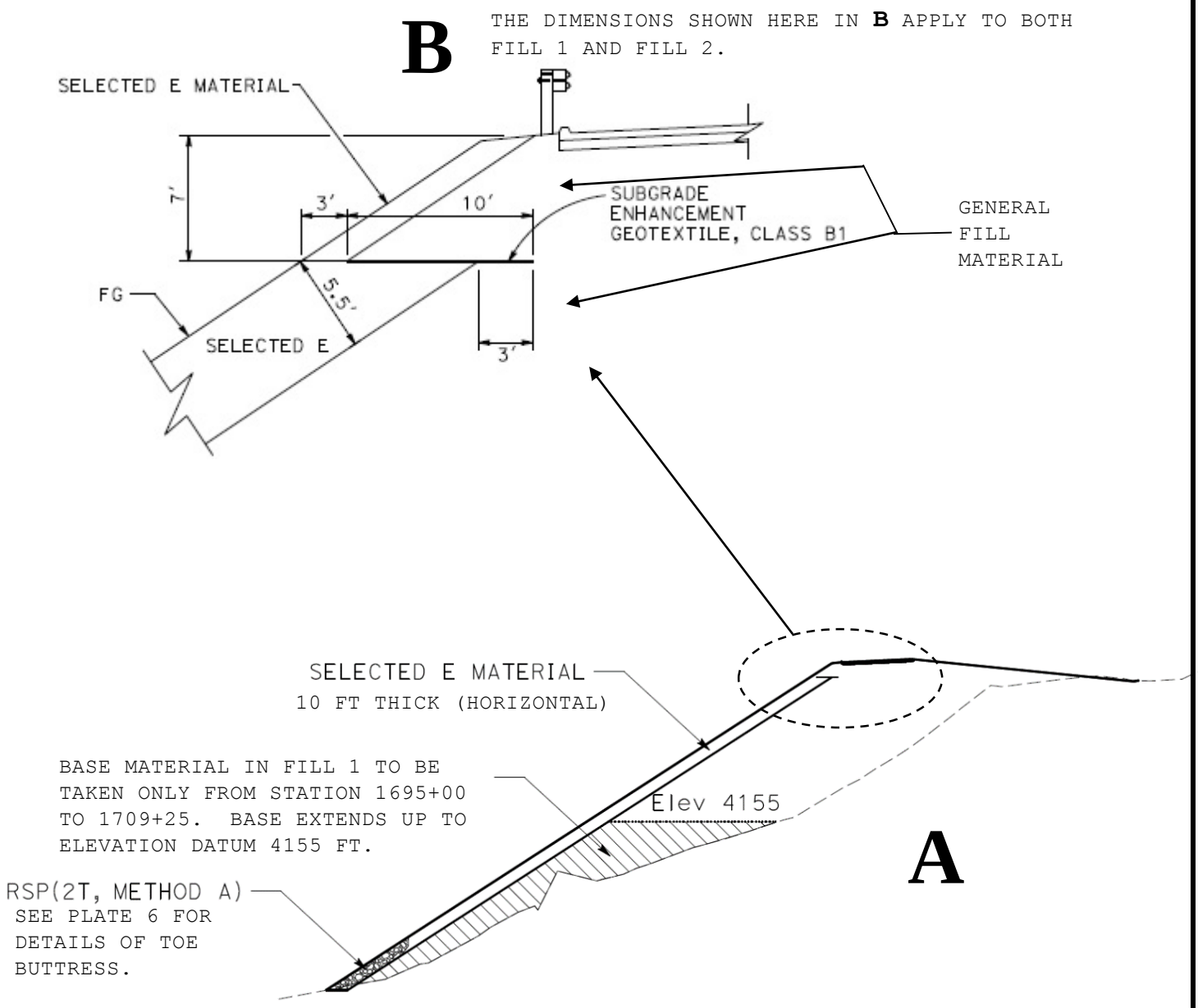


RSP (2T, METHOD A) DETAIL

TO BE INSTALLED FOR TWO
FILLS LOCATED WITHIN THESE STATION
INTERVALS.
"A1" 1711+09 TO "A1" 1712+12 L+
"A1" 1718+45 TO "A1" 1719+75 L+

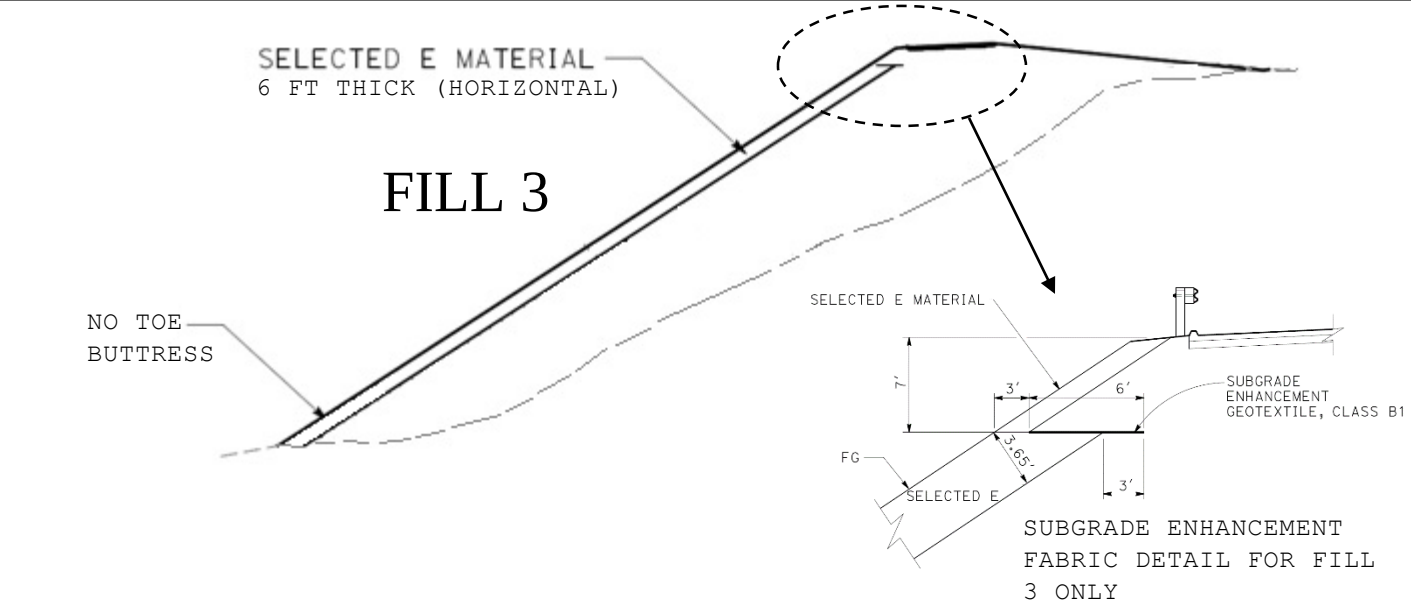
Cross-Sectional Drawing of Toe-Buttress. Cross-section drawing of toe buttress to be installed at the toe of Fill 1 and Fill 2.

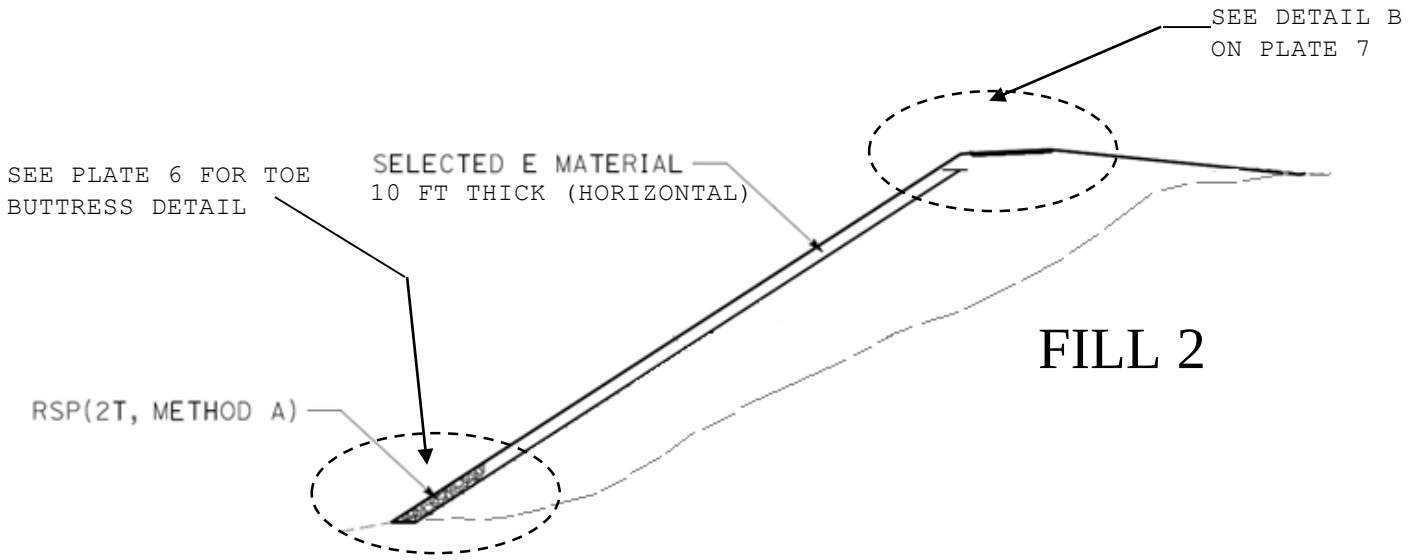
CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**FILL 1 & 2 TOE BUTTRESS CROSS-SECTION**
	02-TEH-36 PM 75.1/78.4 GEOTECHNICAL DESIGN REPORT	Plate No. 6



Cross-Section Detail Drawings of Fill 1. A. Cross-section drawing of Fill 1 showing Selected E encapsulation layer, toe buttress, and base material zoning. **B.** Enlarged cross-section detail showing use of subgrade enhancement geotextile fabric to separate Selected E material from overlying subgrade fill material. The dimensions shown here in B apply to Fill 1 and Fill 2. For Fill 3 see Plate 8.

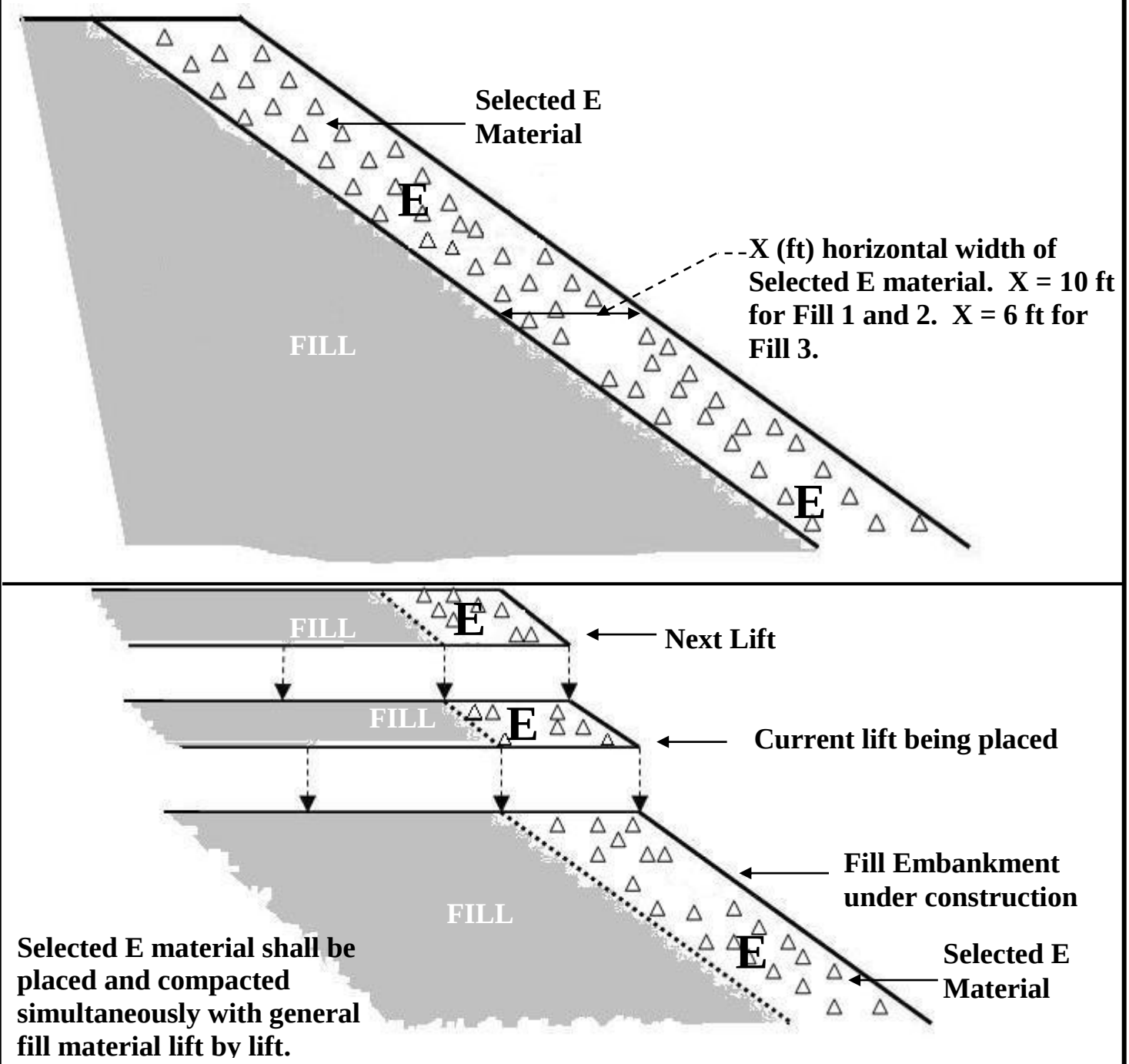
CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**FILL 1 CROSS-SECTION DETAILS**
	02-TEH-36 PM 75.1 /78.4 GEOTECHNICAL DESIGN REPORT	Plate No. 7





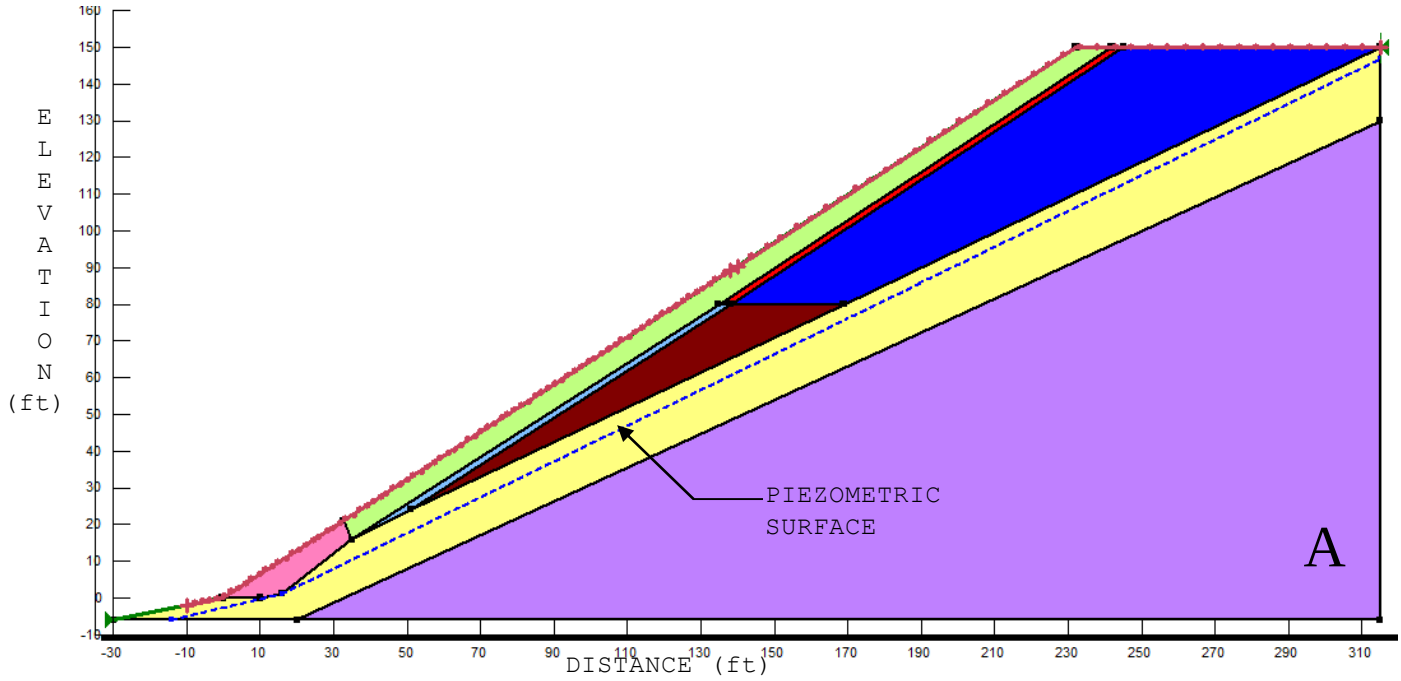
Cross-Section Drawings of Fills 1 and 2. Cross-sections of Fill 2 and Fill 3 showing the encapsulation on both fills with Selected E material. Note that only Fill 3 does not have a toe buttress. See Plate 7 for detail showing use of subgrade enhancement geotextile to separate Selected E material from subgrade fill material for Fill 1 & 2. A slightly modified version of this detail for Fill 3 is shown above. See Plate 6 for detail of Toe Buttress for Fill 2.

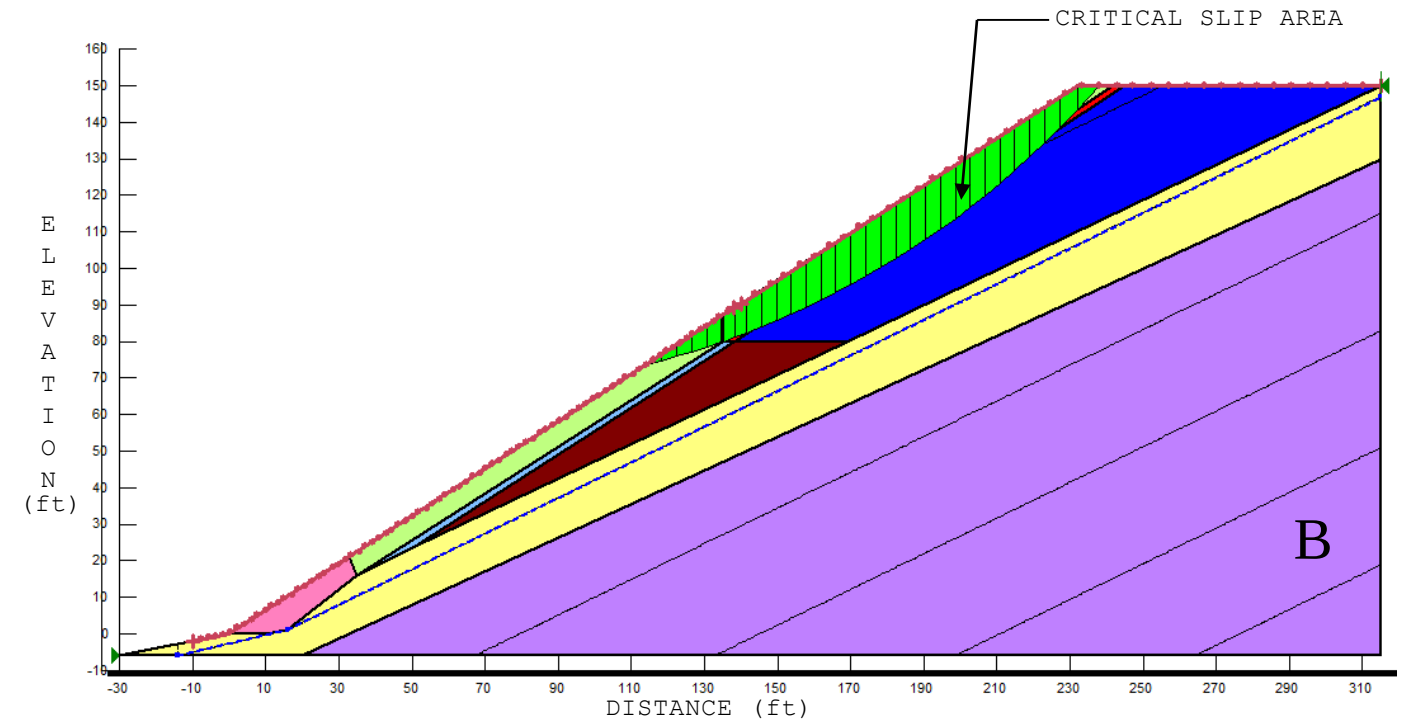
CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**FILL 2 & 3 CROSS-SECTIONS**
	02-TEH-36 PM 75.1 /78.4 GEOTECHNICAL DESIGN REPORT	Plate No. 8



Fill Encapsulated by Selected E Material. Upper drawing depicts a fill embankment constructed with fill material encapsulated by X horizontal ft of compacted Select E material. Lower drawing shows schematically that Select E material should be placed and compacted simultaneously with fill material lift by lift. Select E shall not be applied as a later veneer. The faces of Fill 1, 2, and 3 shall be encapsulated as shown. X equals 10 ft for Fill 1 and 2. X equals 6 ft for Fill 3.

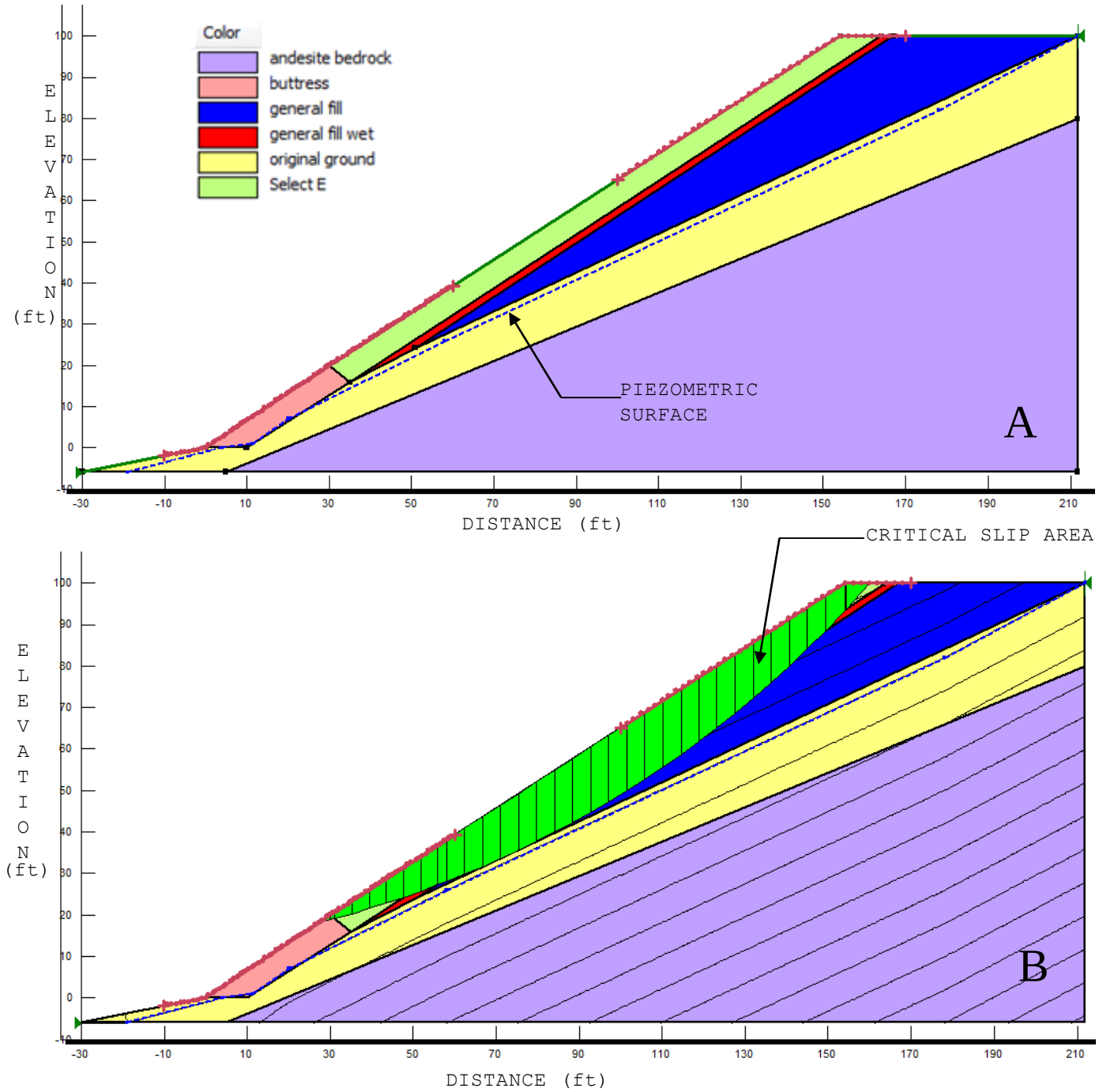
CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**PLACEMENT & COMPACTION OF FILL & SELECTED E**
	02-TEH-36 PM 75.1/78.4 GEOTECHNICAL DESIGN REPORT	**Plate No.9**





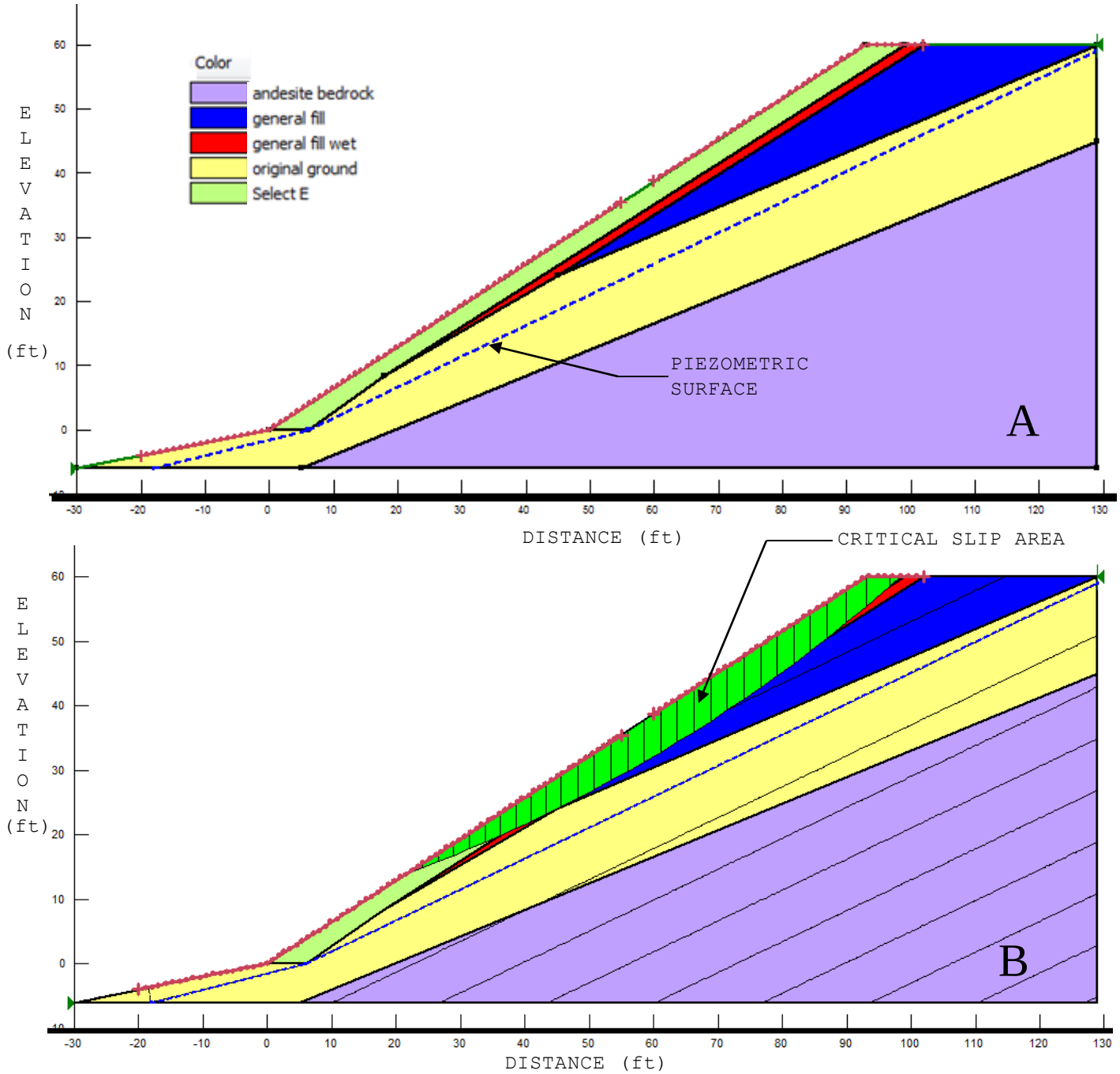
Slope/W Cross-sections for Fill 1. Cross-sections of Slope/W models for Fill 1 stability analyses, showing the model (A) and the model with the critical slip area (B). Note: General fill wet is the same as general fill in this model. General fill base-wet and general fill base are also the same for this model. These materials are distinguished only when applying cohesion to the non-wet materials, while C remains zero for the wet materials in order to represent a wet season wetting front beneath the encapsulating rock (E).

CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**SLOPE/W CROSS-SECTIONS FOR FILL 1**
	02-TEH-36 PM 75.1 /78.4 GEOTECHNICAL DESIGN REPORT	Plate No. 10



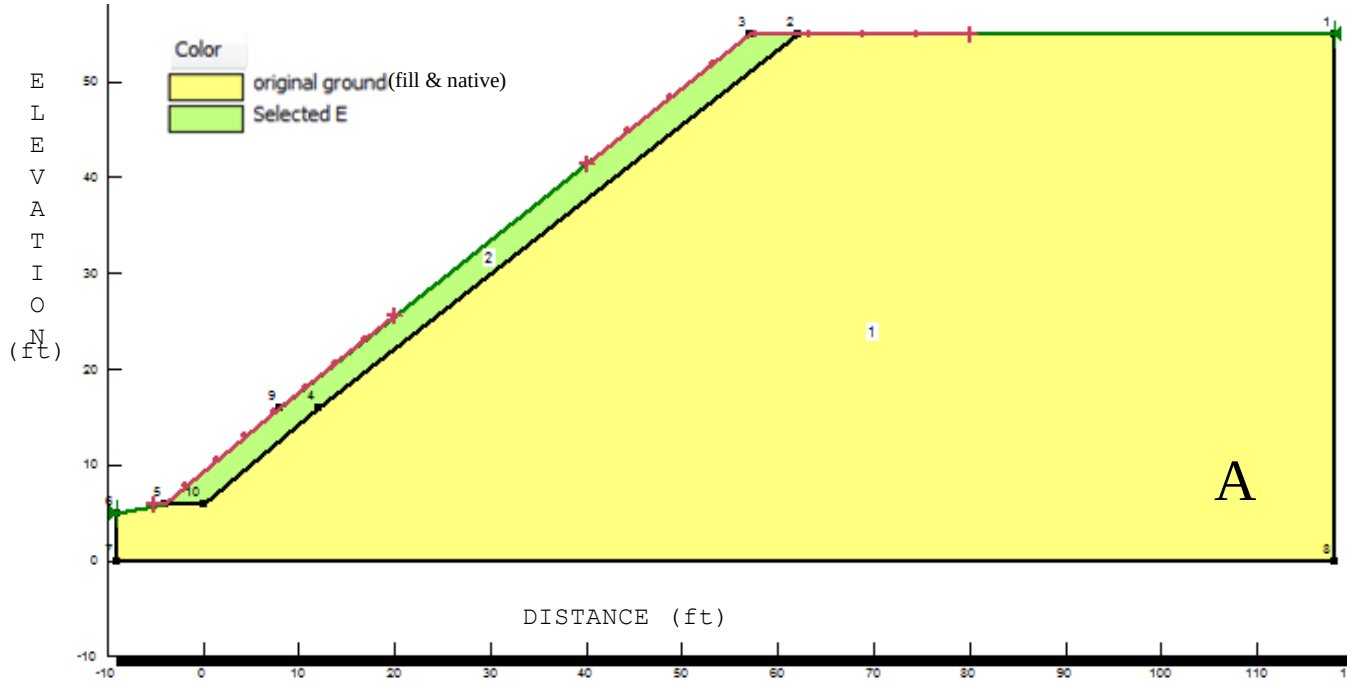
Slope/W Cross-sections for Fill 2. Cross-sections of Slope/W models for Fill 2 stability analyses, showing the basic model (A) and the model with the critical slip area (B). Note: General fill wet is the same as general fill here. These materials are distinguished only when applying cohesion to the non-wet general fill, while C remains zero for the general fill wet material in order to represent a wet season wetting front beneath the encapsulating rock (E).

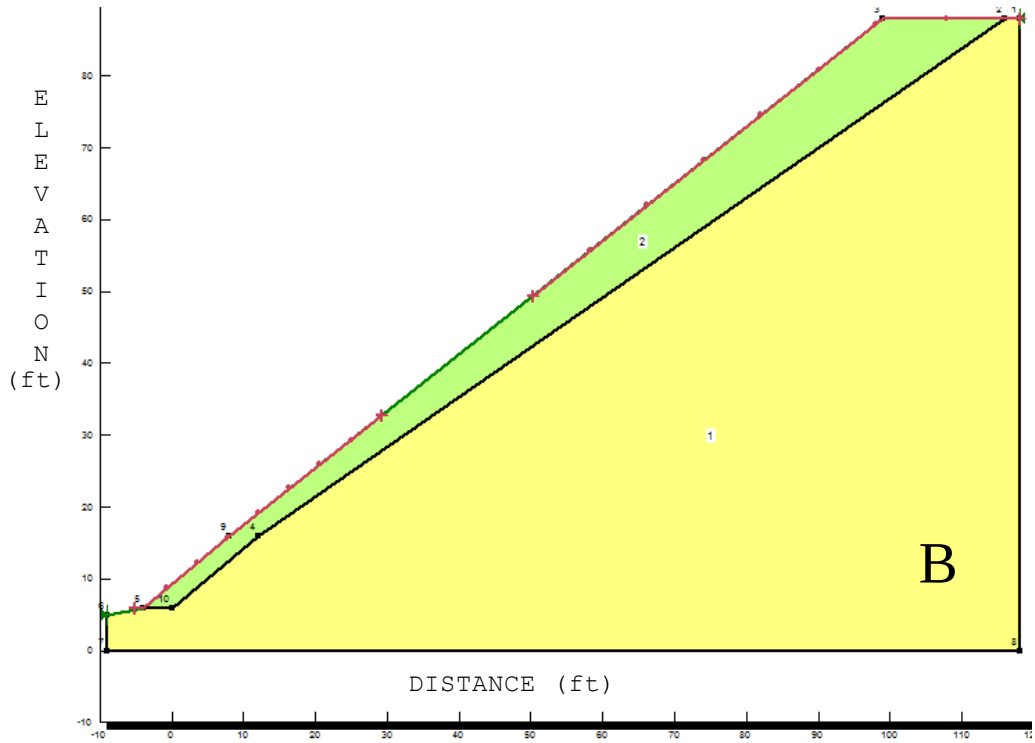
CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**SLOPE/W CROSS-SECTIONS FOR FILL 2**
	02-TEH-36 PM 75.1 /78.4 GEOTECHNICAL DESIGN REPORT	Plate No. 11



Slope/W Cross-sections for Fill 3. Cross-sections of Slope/W models for Fill 3stability analyses, showing the basic model (A) and the model with the critical slip surface (B). Note: General fill wet is the same as general fill here. These materials are distinguished only when applying cohesion to the non-wet general fill, while C remains zero for the general fill wet material in order to represent a wet season wetting front beneath the encapsulating rock (E).

CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**SLOPE/W CROSS-SECTIONS FOR FILL 3**
	02-TEH-36 PM 75.1/78.4 GEOTECHNICAL DESIGN REPORT	Plate No. 12





Slope/W Cross-sections for 1.25:1 Fills. Cross-sections of Slope/W models for 1.25:1 fills stability analyses, showing the 50-foot sliver model (A) and the 80-foot thicker model (B).

CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-4G03U Date: June 2016	**SLOPE/W CROSS-SECTIONS FOR 1.25:1 FILLS**
	02-TEH-36 PM 75.1/78.4 GEOTECHNICAL DESIGN REPORT	Plate No. 13

12.2. Seismic Refraction and MASW Results

GEOPHYSICAL INVESTIGATION
STATE ROAD 36
TEHAMA COUNTY, CALIFORNIA

NORCAL Project Number 15-177.216

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July 30, 2015

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1.0 INTRODUCTION

This report presents the findings of a geophysical survey performed by NORCAL Geophysical Consultants, Inc. along a portion of Highway 36 in Tehama County, CA. The survey was performed during the period of June 8-12, and June 15-19, 2015 by NORCAL Professional Geophysicists William E. Black (PGp No. 843) and Donald J. Kirker (PGp No. 997), and geophysical technicians Travis Black, Chris Bissiri and Tom Ogasawara. Logistical support was provided by Chase White of Kleinfelder, Inc and Scott Lewis of the State of California Department of Transportation (Caltrans). This investigation was performed for Kleinfelder, Inc and the State of California Department of Transportation under Caltrans Contract No.: 59A0834; Task Order No.: 83480; Project ID: 02-THE-36 PM 75.4/78.4; PIN: 02-1500-0056-; EA: 02-4G03U.

1.1 SITE DESCRIPTION

State Road 36 traverses California's Cascade Range between Interstate 5 at Red Bluff in the central valley and Highway 395 near Susanville. The project site is a 1.5-mile stretch of the road located 35-miles east of Red Bluff and 5-miles west of Mineral, as shown on Plate 1. In this area the highway winds along an east trending ridge above the south fork of Battle Creek at elevations between 4,250- and 4,270-ft. The terrain comprises rocky covered slopes that are moderately to heavily vegetated and wooded.

1.2 GENERAL GEOLOGY

Geologic information, obtained from Caltrans, indicates that the project site lies within an area that consists solely of volcanic rocks. This is characterized by a 3- to 10-foot surficial layer of colluvium that contains andesite cobbles and boulders over bedrock that consists of andesite, welded tuff and basalt. Rock outcrops and cut-slopes along the road indicate that the west end of the project site consists primarily of andesite. To the east, the andesite progresses up slope and to the south. In this area, the exposed bedrock along the roadway consists of pyroclastic rock that comprises at least four different welded tuffs. Near the center of the project site, a rock outcrop reveals a moderately weathered andesite beneath the tuff. From this location to the east end of the project site, bedrock consists primarily of basalt.

1.3 OBJECTIVE AND SCOPE OF WORK

Most of State Road 36 between Red Bluff and Mineral is relatively straight with gentle curves. However, the section of roadway that lies within the project site is relatively windy. Because of the sharper curves, this section of highway is prone to an increase in auto accidents. To improve the safety of the road, Caltrans is planning to reconstruct this portion of highway. Specifically, they will realign portions of the road to improve roadway geometry, improve drainages and create new embankments and cut slopes. Therefore, the objective of the geophysical investigation is to provide subsurface information and characterizations that will aid in determining 1) the depth to and rippability of bedrock along proposed cut slopes and 2) the foundation characterizations for large embankment fills. We understand that Caltrans will use this information, along with data obtained from their geotechnical studies, to plan for the reconstruction of the roadway.

To achieve this objective we conducted high resolution 2D seismic refraction (SR) and 1-D multi-channel analysis of surface waves (MASW) as directed by Caltrans and Kleinfelder, Inc. The scope of work also includes analyzing and interpreting the geophysical data and presenting the results in a written report. At the request of Caltrans, this report does not include specific interpretations of the SR and MASW results.

1.4 SURVEY COVERAGE

The scope of work includes obtaining geophysical data along 18-lines designated as LLSL1 through LLSL18. The initial locations of the lines were predetermined by Caltrans and marked in the field by a Caltrans representative at the start of the investigation. However, the final line locations, lengths, and orientations, as shown on Plates 2 through 4, were dictated by access, terrain, and vegetation. Table 1 below lists each line designation (LLSL1 through LLSL18), the approximate survey stationing along the proposed highway, the final line length, the geophysical method used along each line and the respective objective for each line.

Table 1: Profile Number, Survey Station, Line Length, Method and Objective

PROFILE	APPROXIMATE STATIONING	LINE LENGTH (ft)*	METHOD **	OBJECTIVE
LLSL1	1656+30 to 1657+80	150	SR/MASW	Rippability/Cut
LLSL2	1663+80	150	SR/MASW	Rippability/Cut
LLSL3	1672+50	125	SR/MASW	Rippability/Cut
LLSL4	1678+40	150	SR/MASW	Rippability/Cut
LLSL5	1684+60	150	SR/MASW	Rippability/Cut
LLSL6	1689+30 to 1692+00	270	SR/MASW	Rippability/Cut
LLSL7	1690+80	150	SR/MASW	Rippability/Cut
LLSL8	1695+10	125	SR/MASW	Rippability/Cut
LLSL9	1693+90 to 1696+50	250		Rippability/Cut
LLSL10	1700+30 to 1702+05	175	SR/MASW	Rippability/Cut
LLSL11	1711+80	150	SR/MASW	Fill
LLSL12	1710+40	175	SR/MASW	Fill
LLSL13	1714+60	175	SR/MASW	Rippability/Cut
LLSL14	1717+00	175	SR/MASW	Rippability/Cut
LLSL15	1719+10	175	SR/MASW	Fill
LLSL16	1722+20	175	SR/MASW	Rippability/Cut
LLSL17	1723+90 to 1725+10	125	SR/MASW	Rippability/Cut
LLSL18	1725+60	150	SR/MASW	Fill

*Surface distance between near offset shotpoints
**SR=Seismic Refraction
MASW= Multi-channel Analysis of Surface Waves

As shown above, we obtained SR and MASW data along 18-lines. The SR method was used to characterize the thickness of overburden, the depth to rock and the apparent rippability of the rock based on the measured rock velocities. The MASW method was used to further characterize the shear (s) wave velocity with depth.

We used a Trimble global positioning system (GPS) with sub-foot accuracy to measure the geographical coordinates of the beginning and end geophone of each line. These positions

were differentially corrected and exported for data analysis. We also hand leveled each line to determine the change in elevation of each geophone with respect to one end of the line. To determine the elevation profile along each SR line, we superimposed the GPS data onto a topography map provided by Caltrans to obtain the initial starting elevation. We then added this value to our hand-level data to determine the specific elevations of each geophone. Since the elevations of the SR lines are based on specific changes defined by hand-leveling and the Caltrans provided contour map is based on lidar imagery, there may be discrepancies between the respective SR lines and contours shown on Plates 2 through 4.

2.0 SEISMIC REFRACTION

The seismic refraction (SR) method was used to determine the compressional wave velocity of subsurface materials. The seismic velocity of fill, sediments, and rock are dependent on physical properties such as compaction, density, hardness, and induration. However, other factors such as bedding, fracturing, and saturation also affect seismic velocity. Typically, low velocities of less than 1,000-ft/s to 4,000-ft/s are indicative of loose soil, poorly compacted fill, poorly to semi-consolidated sediments, and deeply weathered and highly fractured rock. Moderate velocities of 4,000- to 8,000-ft/s are usually indicative of dense and highly compacted sediments and fill, and highly to moderately weathered and/or fractured rock. High velocities of 8,000- to over 11,000-ft/s are indicative of slightly weathered to unweathered rock with little fracturing.

2.1 DATA ACQUISITION PROCEDURES AND EQUIPMENT

We obtained SR data along 18 lines (LLSL1 through LLSL18) as noted above in Table 1. Each line ranged in length from 125- to 270-ft and consisted of one spread. Each spread consisted of 24 geophones and nine shot points distributed in a collinear array. The geophones were coupled to the ground surface at 5- to 10-ft intervals. Five shot points were evenly spaced within the interior of the spread. The remaining four were used as near- and far-offset shots at each end. The near-offset shots were located approximately 5- to 10-ft beyond the end geophones. The far-offset shots were located approximately 20- to 48-ft beyond the end geophones.

The SR data were recorded using a Geometrics ***Geode***, 24-bit digital seismic recording system and Oyo Geospace digital-grade geophones with a natural frequency of 8-Hz. We produced seismic energy at each shot point by striking an aluminum plate, placed on the ground surface, with a 16-pound sledge hammer. An accelerometer attached to the hammer transmitted a triggering pulse to the seismograph with each impact using the GISCO seismic radio trigger link. The analog signals transmitted by the geophones were digitized by the ***Geodes*** 24-bit analog to digital converters, amplified, conditioned and processed, and then transmitted via a network cable to a Panasonic ***Toughbook*** field computer where the signals were displayed and recorded.

2.2 DATA ANALYSIS

The recorded SR data were downloaded to a computer and processed using the program ***Seisimager*** by Geometrics, Inc. and ***Rayfract*** by Intelligent Resources Inc. ***Seisimager*** is an interactive program that we used to determine the shot point to geophone travel times (first arrivals) and to compute 2D models based on those travel times. Once the travel times for a given line were determined, the programs time-term algorithm was used to compute a preliminary 2D seismic model. This model was then used as input for ***Rayfract's*** tomographic routine.

Rayfract uses a tomography method to model multiple signal propagation paths contributing to one first break, based on the Fresnel volume approach. As a consequence the velocity anomaly imaging capability is enhanced compared to conventional ray tomography. ***Rayfract*** also includes a smooth inversion tomographic method that is based on physically realistic modeling of first break propagation for P-wave and S-wave surveys. It forward models refraction, transmission and diffraction (Lecomte, 2000) and back-projects traveltime residuals along wave paths, also known as Fresnel volumes (Watanabe, 1999) instead of conventional rays. This increases the numerical robustness of the inversion.

The time-term model computed by ***Seisimager*** was as a smooth starting model for ***Rayfract***. The program then refined the model using 2DWET inversion (Schuster, 1993). ***Rayfract*** uses an adapted SIRT algorithm for velocity update of grid cells, when back-projecting traveltime residuals along wave paths (Schuster, 1993) and (Watanabe, 1999). The result of these features is a seismic velocity model that is geologically reasonable even in cases involving rugged topography and strong lateral velocity variations. The software then contours the model velocities to produce seismic velocity vs. depth and distance cross-sections (profiles) using the computer program ***Surfer 12.0*** by Golden Software.

The seismograms showing the first arrivals for every shot point along Lines LLSL1 through LLSL18 are included in this report in pdf format (Appendix B). These seismograms show that the overall quality of the first arrivals was very good. Specifically, almost 90% of the first arrivals along each trace were clean and not affected by significant noise. Approximately 10% of the first arrivals were marginal to poor. These are located on three of the shot points from LLSL2, one shot point from LLSL3, seven of the shot points from LLSL5 and eight from

LLSL9. In most cases, the poor quality was a result of poor coupling due to very loose surface materials. An effort was made to improve the quality by digging through the material at both the geophone and shot point locations. However, the material was too thick for this to significantly improve the coupling. In spite of this, we were able to make good first arrival picks that were consistent at every shot point along the respective lines, as shown on the time-distance graphs also included.

2.3 EXCAVATION CHARACTERISTICS (RIPPABILITY)

Seismic velocity charts relating seismic compressional (P) velocities and excavation characteristics have been developed from field tests by others. These charts list the seismic velocity of various types of rock and their relative ease of excavation using different types of rippers. Caterpillar Tractor Company publishes a performance manual that lists ripper performance charts for the D8L, D9L, and D11L tractors. The information listed in Table 2, below, was obtained from a performance chart for a Caterpillar D9L Ripper:

Table 2: D9L Ripper Performance Chart

PERFORMANCE	ROCK TYPE	VELOCITY RANGE (ft/s)
Rippable	Sedimentary	< 6,400 to 7,800
	Igneous	< 6,700 to 7,600
	Metamorphic	< 7,200 to 7,300
Marginally Rippable	Sedimentary	6,400 to 9,700
	Igneous	6,700 to 8,600
	Metamorphic	7,200 to 9,200
Non-rippable	Sedimentary	> 8,600 to 9,700
	Igneous	> 8,000 to 8,700
	Metamorphic	> 9,000 to 9,200

The information listed above should only be used as a general guide, however, as many other factors should also be considered. These factors include rock jointing and fracture patterns, the experience of the equipment operator, and the equipment and excavation methods selected. Also, the computed velocities measured along each profile are an average for each layer. Therefore, there may be localized zones where the velocities may be higher or lower than indicated. Since the accuracy of our findings is subject to these limitations, it should be noted that subsurface conditions may vary slightly from those depicted in the final results. A more detailed discussion of the seismic refraction method is presented in Appendix A.

2.4 RESULTS

The results of the seismic refraction surveys along LLSL1 through LLSL18 are presented on the 2-D seismic velocity profiles shown on Plates 5 through 16. On each profile the vertical axis represents elevation above mean sea level in feet and the horizontal axis represents distance (station) in feet. Each profile depicts the ground surface, and shows color contours representing the respective range in seismic compressional velocities (SR) according to the color scale shown at the bottom of each plate.

The following sections provide a general description of each survey area, the objectives of the SR survey in each area and a description of the SR results based on each seismic profile.

2.4.1 LLSL1 (PLATE 5)

LLSL1 is 180-ft long and is located on the south side of State Road 36 between Stations 1656+30 to 1657+80, as shown on Plate 2. It is positioned at the edge of a cut-slope above the present highway. It trends east and parallels the proposed road alignment approximately 50- to 65-ft to its south (upslope). The survey area is covered with pines, shrubbery, and surface debris (downed trees, etc.). Caltrans plans to remove some of this material and create a larger cut-slope. Therefore, the objective of the geophysical survey is to obtain SR data to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock.

The SR profile for LLSL1 indicates seismic velocities of less than 1,000- to over 7,000-ft/s within the upper 20- to 37-ft. The highest velocity material (6,000- to 7,000-ft/s) ranges in depth from 10- to 27-ft. It is shallowest near the center and increases in depth at each end. Within

this layer is a zone of lower velocity material (5,000-ft/s). It is located in the center of the profile from Stations 60- through 90-ft at a depth of 14-ft.

2.4.2 LLSL2 (PLATE 5)

LLSL2 is 150-ft long and is located at Station 1663+80, as shown on Plate 2. It is located on a large cut-slope above the present highway. It trends northwest to southeast and is positioned perpendicular to the proposed road. The survey area is very steep and free of vegetation in the steepest sections of the cut and covered with pines, shrubbery, and surface debris (downed trees, etc.) near the top of the cut. Caltrans plans to remove some of this material and create a larger cut-slope. Therefore, the objective of the geophysical survey is to obtain SR data to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock.

The SR profile for LLSL2 indicates seismic velocities of 1,000- to over 9,000-ft/s within the upper 17- to 70-ft. The highest velocities are located in two zones that occupy the southeast end and northwest half of the profile. At the southeast end (up slope) there is a relatively small zone of 6,000- to 9,000-ft/s material that is relatively shallow, ranging in depth from 10- to 13-ft. Along the northwest half, there is a much deeper zone. It is characterized by velocities of 6,000- to over 7,000-ft/s at depths of 17- to 40-ft.

2.4.3 LLSL3 THROUGH LLSL5 (PLATES 6 & 7)

LLSL3 through LLSL5 are 125- to 150-ft long and are located on a slope above the south side of the present highway at Stations 1672+50, 1678+40 and 1684+60, respectively, as shown on Plates 2 and 3. All three lines trend perpendicular to the highway and elevation contours and end at the top of a cut where the proposed road will be constructed. The terrain along each line is relatively steep and is covered by pines, shrubbery, and surface debris. Caltrans plans to remove a portion of the slope at each area and create a larger cut-slope. Therefore, the objective of the geophysical survey at LLSL3 through LLSL5 is to obtain SR data to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock.

The velocities beneath LLSL3 through LLSL5 are all relatively low. LLSL3 is characterized by seismic velocities ranging from 1,000- to 4,000-ft/s at depths of 10- to 52-ft. LLSL4 and LLSL5 indicate velocities ranging from 1,000- to slightly more than 5,000-ft/s within the upper 50-ft. The highest velocity material (5,000-ft/s) is deepest beneath LLSL4 (56-

ft) and shallowest beneath LLSL5, where it is defined at depths of 10- to 20-ft. The depth of investigation along LLSL5 was limited to about 20 feet according to the processed profile.

2.4.4 LLSL6 & LLSL7 (PLATES 8 & 9)

LLSL6 is 270-ft long and positioned at the edge of a cut-slope along relatively level terrain above (south of) the present highway, as shown on Plate 3. It trends east and parallels the highway from Stations 1689+30 to 1692+00. LLSL7 (150-ft long) crosses LLSL6 and trends upslope from the present highway at Station 1690+80. The terrain along this line is relatively steep, especially over the cut adjacent to the road. The area is covered by pines, shrubbery, and surface debris. Caltrans plans to remove a portion of this slope and create a larger cut-slope. Therefore, the objective of the geophysical survey is to obtain SR data to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock.

The seismic velocities beneath LLSL6 and LLSL7 are also relatively low and similar to the previous three lines. LLSL6 defines seismic velocities of less than 1,000- to over 4,000-ft/s to depths of at least 80-ft. LLSL7 (Plate 9) indicates velocities of less than 2,000- to over 3,000-ft/s to depths of at least 54-ft.

2.4.5 LLSL8 & LLSL9 (PLATE 10)

LLSL8 is 125-ft long and is located on the slope above the south side of the present highway at Station 1695+10. It trends north (down slope) and ends at the top of a cut-slope where the proposed road will be constructed, as shown on Plate 3. LLSL9 (250-ft long) crosses LLSL8 above the cut-slope along relatively level terrain. It trends east and parallels the highway from Stations 1693+90 to 1696+50. The ground surface along the line increases slightly in elevation to the southeast and is covered with pines and surface debris. Caltrans plans to remove a portion of this slope and create a larger cut. Therefore, the objective of the geophysical survey is to obtain SR data to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock.

The SR profile along LLSL8 indicates seismic velocities of less than 1,000- to over 7,000-ft/s within the upper 60-ft. The highest velocities (6,000- to over 7,000-ft/s) are defined at depths of 46- to 52-ft. There is also a small isolated zone of 5,000-ft/s material at a depth of 15-ft from Stations 45- to 70-ft.

LLSL9 defines velocities of less than 1,000- to over 4,000-ft/s within the upper 28- to 42-ft. High values of 6,000- to over 7,000-ft/s were not defined. As a result, these crossing lines indicate that the subsurface consists primarily of lower velocity material (1,000- to 5,000-ft/s) to depths of 46- to 52-ft.

2.4.6 LLSL10 (PLATE 11)

LLSL10 is 175-ft long and is positioned above the cut-slope parallel to the present highway. It trends southeast over relatively level terrain from Stations 1700+30 to 1702+05. The survey area is covered with pines, vegetation and surface debris. Caltrans plans to remove a portion of this slope and create a larger cut. Therefore, the objective of the geophysical survey is to obtain SR data to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock.

The SR profile indicates seismic velocities of less than 1,000- to over 8,000-ft/s within the upper 20- to 54-ft. The highest velocity material (6,000- to over 8,000-ft/s) ranges in depth from 16- to 34-ft. The top of this unit is shallowest in the center of the profile and at the southeast end and forms a depression from Stations 105- to 150-ft.

2.4.7 LLSL11 & LLSL12 (PLATE 12)

LLSL11 and -12 are positioned below (north of) the proposed road and cross at their north ends, as shown on Plate 4. Both trend down slope and are 150- and 175-ft long, respectively. The southeast end of LLSL11 is approximately 15-ft from Station 1711+80. The southwest end of LLSL12 is approximately 30-ft from Station 1710+40. A northwest trending drainage is located between the lines. The survey area along the two lines is covered with pines, shrubbery, and surface debris (downed trees, etc.). Caltrans plans to fill in this area to accommodate the construction of the proposed road. Therefore, the objective of the geophysical surveys is to obtain SR data to aid in characterizing the subsurface materials that will act as a foundation to the imported fill.

The SR profile along LLSL11 indicates seismic velocities of less than 2,000- to over 5,000-ft/s within the upper 44-ft. LLSL12 defines similar velocities in the same approximate depth range. However, the seismic velocity structure beneath LLSL12 was such that the model for that line extended to greater depths (over 80-ft) and consequently defined much higher velocities (up to 10,000-ft/s). The highest velocities, e.g. 6,000- to 10,000-ft/s, occur at depths of

only 8- to 30-ft beneath the northeast end of the line and 50- to 80-ft beneath the remainder of the line.

2.4.8 LLSL13 & LLSL14 (PLATE 13)

LLSL13 and LLSL14 are each 175-ft long and are located on a slope above the south side of the proposed road at Stations 1714+60 and 1717+00, respectively. They both trend down slope and end at the top of a cut where the proposed road will be constructed, as shown on Plate 4. The southern ends of each line are within approximately 15-ft of each other. The northern ends are over 210-ft apart. The survey area along each line is moderately sloping and consists of pines, shrubbery, and surface debris. Caltrans plans to remove a portion of the slope at each area and create a larger cut-slope. Therefore, the objective of the geophysical survey is to obtain SR data to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock.

The SR profile along LLSL13 indicates seismic velocities of less than 1,000- to over 5,000-ft/s within the upper 45-ft. LLSL14 defines velocities of less than 1,000- to over 9,000-ft/s within the upper 50-ft. The highest velocities of 6,000- to 9,000-ft/s are located at depths of 20- to over 46-ft. The fact that LLSL14 detected velocities in excess of 6,000-ft/s within its depth range but LLSL13 did not, suggests that the high velocity materials increases in depth to the west.

2.4.9 LLSL15 (PLATE 14)

LLSL15 is 175-ft long and is located at Station 1719+10, as shown on Plate 4. It is positioned north of and down slope of the proposed road. The survey area is very steep and covered with pines, shrubbery, and surface debris (downed trees, etc.). Caltrans plans to fill in this area to accommodate the construction of the proposed road. Therefore, the objective of the geophysical surveys is to obtain SR data to aid in characterizing the subsurface materials that will act as a foundation to the imported fill.

The SR profile for this line indicates seismic velocities ranging from 1,000- to over 6,000-ft/s within the upper 10- to 82-ft of the subsurface. The highest velocity of 6,000-ft/s is located at a depth of over 60- to 80-ft.

2.4.10 LLSL16 (PLATE 15)

LLSL16 is 175-ft long and is located above a cut-slope on the south side of the present highway. It trends north down a relatively steep slope and ends at Station 1722+50 of the proposed road. The survey area is covered with dense shrubbery, scattered pines and surface debris (downed trees, etc.). Caltrans plans to remove some of this material and create a larger cut-slope. Therefore, the objective of the geophysical survey is to obtain SR data to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock.

The SR profile indicates seismic velocities of less than 1,000- to over 7,000-ft/s within the upper 10- to 54-ft. The highest velocities (6,000- to over 7,000-ft/s) are defined at depths of 22- to over 50-ft.

2.4.11 LLSL17 (PLATE 16)

LLSL17 is 125-ft long and is located between Stations 1723+90 to 1725+10, as shown on Plate 4. It is positioned at the edge of a cut-slope above the present highway. It trends east and parallels the proposed road alignment approximately 90- to 110-ft to its south (upslope). The survey area is covered with pines, shrubbery, and surface debris (downed trees, etc.). Caltrans plans to remove some of this material and create a larger cut-slope. Therefore, the objective of the geophysical survey is to obtain SR data to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock.

The SR profile for LLSL17 indicates seismic velocities of less than 1,000- to over 5,000-ft/s within the upper 37-ft. The highest velocity of 5,000-ft/s is defined at depths of 26- to over 32-ft.

2.4.12 LLSL18 (PLATE 16)

LLSL18 is 150-ft long and is located at Station 1719+10, as shown on Plate 4. It is positioned north of and down slope of the proposed road. The survey area is steep and covered with pines, shrubbery, and surface debris (downed trees, etc.). Caltrans plans to fill in this area to accommodate the construction of the proposed road. Therefore, the objective of the geophysical surveys is to obtain SR data to aid in characterizing the subsurface materials that will act as a foundation to the imported fill.

The velocities along LLSL18 are relatively low and similar to those defined by LLSL17. The SR profile indicates seismic velocities of less than 2,000- to over 4,000-ft/s within the upper 32-ft.

3.0 MULTI CHANNEL ANALYSIS OF SURFACE WAVES (MASW)

3.1 RATIONAL AND METHODOLOGY

When seismic waves are generated at or near the ground surface, both body and surface waves are generated. When a vertical impact device is used to produce the seismic energy surface waves, commonly referred to as ground roll in seismic surveys, account for more than two-thirds of the energy. As a result, surface waves are the most prominent signal on multi-channel seismic records. In addition, surface waves have dispersion properties that body waves lack. That is, different wavelengths have different penetration depths and, therefore, propagate at different velocities. By analyzing the dispersion of surface waves it is possible to obtain a near-surface S-wave velocity (Vs) profile. Since Vs is directly proportional to the shear modulus, this provides a direct indication in the variation of stiffness (or rigidity) of subsurface materials. A more detailed description of the MASW methodology is provided in Appendix A.

3.2 DATA ACQUISITION PROCEDURES

We recorded the SR data in such a way that they could also be analyzed using the MASW method to indicate the variation in shear wave velocity (Vs) beneath the center of each seismic line. The only requirement was that we used a record length of 1-second and a sample interval of 0.25 m/sec. The record length is longer than what we would typically use for refraction but does not diminish the refraction data in any way. Furthermore, an MASW analysis can only be conducted using end-on shots. Consequently, we used the data from the near and far offset shot points for the MASW analysis. As a result, having four MASW shot points for each line provided a high level of enhancement and a good signal to noise ratio.

3.3 DATA ANALYSIS

We processed the surface wave data using the software package ***SurfSeis 3.0*** which was developed by the University of Kansas. The data from each MASW shot point were processed to develop a dispersion curve representing variations in surface wave velocity with frequency. Each dispersion curve was then inverted to compute a 1D model representing Vs versus depth beneath the center of the 24-geophone array used to record the respective data.

Since MASW data are highly dependent on local geologic conditions, an optimal shot point to geophone array offset distance can vary from one location to another. Consequently, we analyzed the data from each shot and merged the best data sets to produce a dispersion curve

based on the highest possible signal to noise ratio. We then inverted the resulting curve to produce a single (S-) wave velocity plot. It should be noted that the data quality of the dispersion curves and inversion process was consistent and repetitive. A more detailed description of data acquisition and analysis procedures are also provided in Appendix A.

3.4 RESULTS

The results of the MASW analysis are presented by the S-wave velocity versus depth plots shown on Plates 17 through 25. The one-dimensional S-wave model is represented by a dark blue vertically stepped line. The step changes in S-wave velocities are based on the trends in the MASW model. Each stepped change in velocity versus depth probably represents a change in consolidation and/or lithology. The depth ranges to various velocity zones are also listed in the following tables:

Table 3: MASW S-WAVE VELOCITY DATA

LLSL1		LLSL2		LLSL3		LLSL4	
Depth Range (ft)	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**
0-7	1,820	0-6	1,940	0-5	1,200	0-4	560
7-15	1,320	6-14	2,190	5-9	1,110	4-15	820
15-25	2,150	14-35	1,500	9-14	920	15-22	1,380
25-38	3,030	35+	2,060	14-20	1,100	22-31	1,190
38-54	2,580			20-37	1,620	31-43	1,450
54+	2,450			37-48	1,160	43+	2,100
				48-63	1,550		
				63+	2,210		

Table 4: MASW S-WAVE VELOCITY DATA

LLSL5		LLSL6		LLSL7		LLSL8	
Depth Range (ft)	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**
0-4	1,860	0-6	1,620	0-3	1,100	0-9	1,200
4-8	1,680	6-15	1,300	3-11	1,320	9-16	830
8-21	1,820	15-37	2,000	11-16	1,010	16-24	1,570
21-30	1,500	37-53	1,270	16-23	680	24-34	1,740
30-41	1,240	53+	2,650	23-32	1,130	34-47	1,460
41-54	2,190			32-57	1,400	47+	2,310
54+	2,590			57+	1,550		

Table 5: MASW S-WAVE VELOCITY DATA

LLSL9		LLSL10		LLSL11		LLSL12	
Depth Range (ft)	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**
0-5	1,410	0-7	2,120	0-13	1,750	0-15	1,220
5-12	1,320	7-28	2,000	13-22	1,930	15-26	1,700
12-20	1,120	28-42	2,410	22-33	1,570	26-56	1,900
20-30	1,490	42+	2,640	33-47	2,400	56+	2,190
30-43	1,960			47+	2,930		
43-59	2,100						
59+	1,950						

Table 6: MASW S-WAVE VELOCITY DATA

LLSL13		LLSL14		LLSL15		LLSL16	
Depth Range (ft)	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**
0-6	2,080	0-10	1,800	0-10	600	0-5	1,580
6-15	2,360	10-17	1,280	10-15	530	5-11	1,280
15-25	2,630	17-26	1,790	15-21	680	11-18	1,970
25-37	2,470	26-51	2,300	21-29	870	18-27	1,790
37-54	1,860	51+	2,620	29-39	960	27-39	1,870
54+	1,680			39-51	1,100	39-53	2,360
				51+	1,280	53+	2,500

Table 7: MASW S-WAVE VELOCITY DATA

LLSL17		LLSL18	
Depth Range (ft)	**Vs (ft/s)**	**Depth Range (ft)**	**Vs (ft/s)**
0-4	1,110	0-3	1,010
4-9	860	3-13	940
9-15	1,250	13-19	1,050
15-23	1,580	19-38	1,200
23-32	1,710	38-51	1,440
32-59	2,100	51+	1,530
59+	1,680		

It should be noted that the variations in S-wave velocity with depth typically do not correlate with the SR P-wave velocity interfaces since the S-wave velocities are due to different rock properties. In particular the resolution of the S-wave velocities within the upper 10-20 feet is poor and velocities can be unrealistically high.

Appendix A

GEOPHYSICAL METHODOLOGY

Appendix A

SEISMIC REFRACTION (SR)

METHODOLOGY

The seismic refraction method provides information regarding the seismic velocity structure of the subsurface. An impulsive (mechanical or explosive) source is used to produce compressional (P) wave seismic energy. The P-waves propagate into the earth and are refracted along interfaces caused by an increase in velocity. A portion of the P-wave energy is refracted back to the surface where it is detected by sensors (geophones) that are coupled to the ground surface in a collinear array (spread). The detected signals are recorded on a multi-channel seismograph and are analyzed to determine the shot point-to-geophone travel times. These data can be used along with the corresponding shot point-to-geophone distances to determine the depth, thickness, and velocity of subsurface seismic layers.

The seismic refraction technique is based on several assumptions. Paramount among these are:

1) that seismic velocity increases with depth, and,

2) that the velocity of each seismic layer is uniform over the length of the given spread.

In cases where these assumptions do not hold, the accuracy of the technique decreases. For example, if a low velocity layer occurs between two layers of higher velocity, the low velocity layer will not be detected and the depth to the underlying high velocity layer will be erroneously large. Also, if the velocity of a seismic layer varies laterally within a spread, those variations will be interpreted as fluctuations in the elevation of the underlying seismic layer.

INSTRUMENTATION

Data acquisition is initiated along each SR line by producing seismic energy using a mechanical source. Mechanical sources produce energy by impacting a metal strike plate on the ground surface with either a 12-16 pound sledge hammer or an elastic-band driven weight drop. The resulting seismic wave forms are recorded using a Geometrics 24-channel engineering

seismograph and Mark Products geophones with a natural frequency of 10 Hz. The data are recorded on hard copy records (seismograms) as well as on computer disks for future processing. The seismograms display the amount of time it takes for a compression (P) wave to travel from a given shot point to each geophone in a spread.

DATA ANALYSIS

Preliminary seismic refraction models were computed using the software package ***SeisImager*** which was written by Oyo Corporation (Japan) and distributed by Geometrics Inc. The first stage of seismic processing included compilation and identification of first arriving P-wave energy at each geophone from each shot point. This process was conducted using ***Pickwin, Version 3.2.0.1*** (2004), which is part of the ***SeisImager*** package. A second interactive program ***Plotrefa, Version 2.8.0.1*** (2006) was used to assign surface elevations to each geophone and velocity layer assignments to travel time. We then used ***SeisImager's*** time-term routine to compute a 2D seismic velocity model based on these inputs.

We used the resulting 2D seismic velocity model as input to the computer program ***Rayfract*** by Intelligent Resources, Lt. ***Rayfract*** uses wavepath eikonal travel time (WET) tomography to model multiple signal propagation paths contributing to one first break, based on the Fresnel volume approach. Conventional ray tracing tomography is limited to the modeling of just one ray per first break. ***Rayfract*** also uses an Eikonal solver (Lecomte, Gjoystdal et al. Geophysical Prospecting May 2000) for traveltime field computation to explicitly model diffraction besides refraction and transmission of acoustic waves. As a consequence the velocity anomaly imaging capability is enhanced with the WET tomographic inversion compared to conventional ray tomography. ***Rayfract*** also includes a smooth inversion tomographic method that is based on physically realistic modeling of first break propagation, for P-wave and S-wave surveys. It forward models refraction, transmission and diffraction (Lecomte, 2000) and back-projects traveltime residuals along wave paths, also known as Fresnel volumes (Watanabe, 1999) instead of conventional rays. This increases the numerical robustness of the inversion. A smooth minimum-structure and artifact-free 1D starting model is determined automatically, directly from the seismic traveltime data, by horizontally averaging DeltaV (Wiechert-Herglotz) method 1D

velocity-depth profiles along the seismic line. The starting model is then refined with 2DWET inversion (Schuster, 1993). ***Rayfract*** uses an adapted SIRT algorithm for velocity update of grid cells, when back-projecting traveltime residuals along wave paths (Schuster, 1993) and (Watanabe, 1999). The result of these features is a seismic velocity model that is geologically reasonable even in cases involving rugged topography and strong lateral velocity variations. The software then contours the model velocities to produce seismic velocity vs. depth and distance cross-sections (profiles).

MULTI-CHANNEL ANALYSIS OF SURFACE WAVES (MASW) METHODOLOGY

When seismic waves are generated at or near the ground surface, both body and surface waves are generated. Body waves consist of both compressional (P) and shear (S) waves. Surface waves (e.g., Rayleigh, Love, etc.) propagate at velocities that are proportional to shear wave velocity (Vs). If a vertical energy source is used, Rayleigh type surface waves are produced. These are commonly referred to as ground roll in seismic surveys. Rayleigh waves are retrograde elliptical and travel at approximately 0.9 times the velocity of S-waves.

MASW data are gathered in much the same way as high-resolution reflection data. Seismic energy - generated by vertical impacts on the ground surface - is detected by an array of closely spaced geophones. The primary differences are that the surface wave technique requires an energy source that is capable of producing ground roll and geophones that are capable of detecting low frequency (<10 Hz) signals.

Surface waves account for more than two-thirds of the energy produced by vertical seismic energy sources. As a result, surface waves are the most prominent signal on multi-channel seismic records. In addition, surface waves have dispersion properties that body waves lack. That is, different wavelengths have different penetration depths and, therefore, propagate at different velocities. By analyzing the dispersion of surface waves it is possible to obtain a near-surface S-wave velocity profile. Since S-wave velocity is directly proportional to shear modulus, this provides a direct indication in the variation of stiffness (or rigidity) of subsurface materials.

DATA ACQUISITION

The spectral analysis of surface waves (SASW) is not a new technique. This method has been used by engineers and geophysicists for many years. However, the conventional method involves using two receivers (geophones) to record multiple impacts at a fixed point. The survey is initiated with the geophones at close proximity to the impact (shot) point. With succeeding impacts the geophones are moved further and further away. This is done so that the surface wave can be sampled at numerous locations. The data from all of the geophone locations are then analyzed to determine the variation in the velocity of the surface wave with respect to frequency. This results in what is referred to as a dispersion curve. With this technique, it may take several hours to obtain a dispersion curve at a single point.

Recent advances in computer software and processing techniques developed by researchers at the University of Kansas have made it possible to analyze surface waves using a large number of shot points and receivers. This is referred to as the multi-channel surface wave (MASW) technique. Dispersion curves at dozens of points distributed along a profile can now be obtained with MASW in the same amount of time it previously took to obtain a single dispersion curve using SASW. The surface wave data are gathered in much the same way as high resolution seismic reflection data. Seismic waves generated by vertical impacts on the ground surface are detected by an array of closely spaced geophones (spread). The energy source and the geophones are sequentially moved along a profile as the survey progresses.

DATA ANALYSIS

We analyze MASW data using the computer program ***Surfseis***, which was developed at the University of Kansas. The software analyzes the data by identifying the ground-roll portion of the seismic wave traces, computing the frequency and velocity of the wavelets, and constructing a dispersion curve representing the variation in surface wave velocity versus frequency. The program then inverts the dispersion curve to compute a one-dimensional (1D) layered model indicating shear-wave velocity (Vs) versus depth beneath the center of the geophone array for each shot gather. In all cases the MASW modeling are iterated until the dispersion curve generated from the S-wave velocity model closely matched that calculated from the shot gathers.

Since MASW data is highly dependent on local geologic conditions, and thus dictated by an optimal shot to geophone distance, we analyze the data from each shot gather and use the highest quality dispersion curve to generate an S-wave velocity plot.

The inversion of the dispersion curve into a shear wave velocity profile is a non-unique process. By default, the software will produce a shear wave profile containing 10 distinct subsurface velocity intervals at various depths. Typically, the MASW technique resolves velocity interfaces at greater depths than the refraction analysis given the same geophone spread length. Furthermore, the MASW technique is capable of resolving velocity reversals (decrease in velocity with depth) that cannot be detected by the refraction method.

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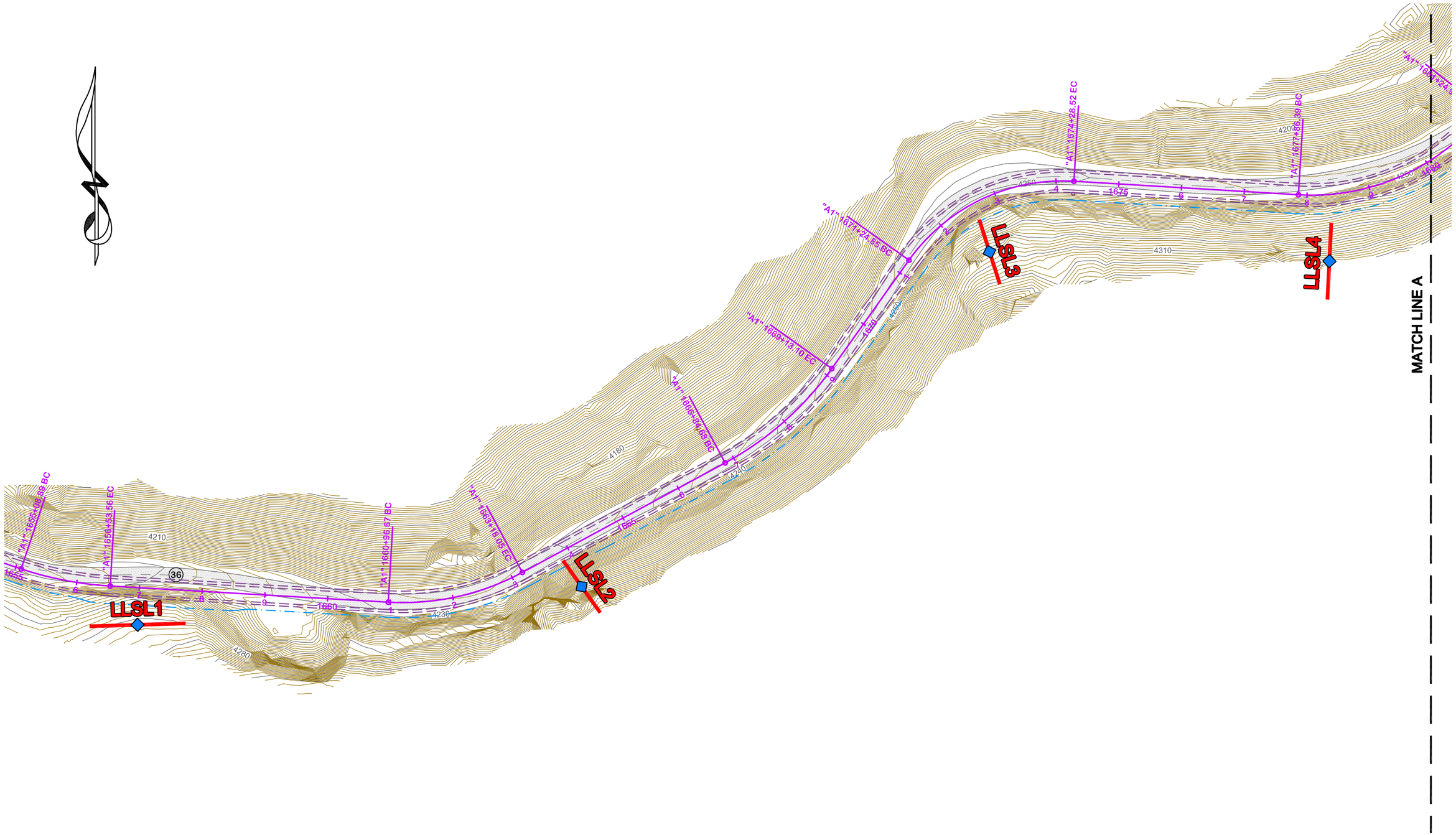
SCALE

0 2500 5000 10000

(1 inch = 5000 feet)

NORCAL

SITE LOCATION MAP SEISMIC REFRACTION SURVEY LASSEN LODGE - HIGHWAY 36	
LOCATION: MINERAL, CALIFORNIA	
CLIENT: KLEINFELDER	PLATE 1
JOB #: 15-177.216	NORCAL GEOPHYSICAL CONSULTANTS INC.
DATE: JUL. 2015	DRAWN BY: G.RANDALL / APPROVED BY: DJK



LEGEND

	SEISMIC REFRACTION LINE	1645	PROPOSED ALIGNMENT
◆	MASW SOUNDING		PROPOSED ROAD PAVEMENT EDGE
	EXISTING ROADWAY		PROPOSED DRAINAGE ALIGNMENT

SCALE

0 150 300

(1 inch = 150 feet)

NOTE: SITE FEATURES PROVIDED BY KLEINFELDER

NORCAL

SITE MAP - LINES LLSL1-4
SEISMIC REFRACTION SURVEY
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: JUL. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 2

LEGEND

	SEISMIC REFRACTION LINE	1645	PROPOSED ALIGNMENT
	MASW SOUNDING		PROPOSED ROAD PAVEMENT EDGE
	EXISTING ROADWAY		PROPOSED DRAINAGE ALIGNMENT

SCALE

0 150 300

(1 inch = 150 feet)

NOTE: SITE FEATURES PROVIDED BY KLEINFELDER

NORCAL

SITE MAP - LINES LLSL5-10
SEISMIC REFRACTION SURVEY
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

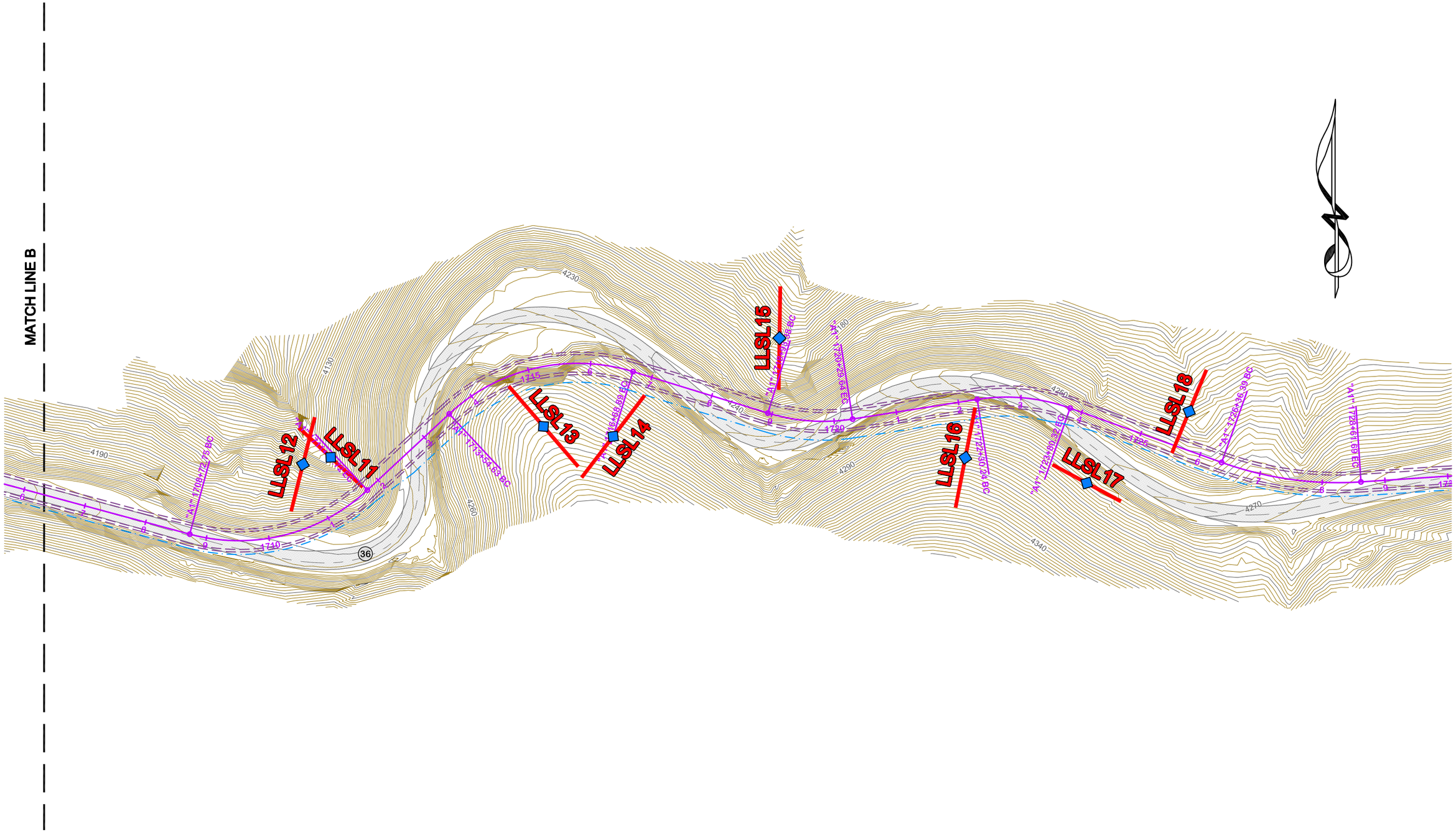
NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: JUL. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 3



LEGEND

	SEISMIC REFRACTION LINE	1645	PROPOSED ALIGNMENT
	MASW SOUNDING		PROPOSED ROAD PAVEMENT EDGE
	EXISTING ROADWAY		PROPOSED DRAINAGE ALIGNMENT

SCALE

0 150 300

(1 inch = 150 feet)

NOTE: SITE FEATURES PROVIDED BY KLEINFELDER

NORCAL

SITE MAP - LINES LLSL11-18
SEISMIC REFRACTION SURVEY
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

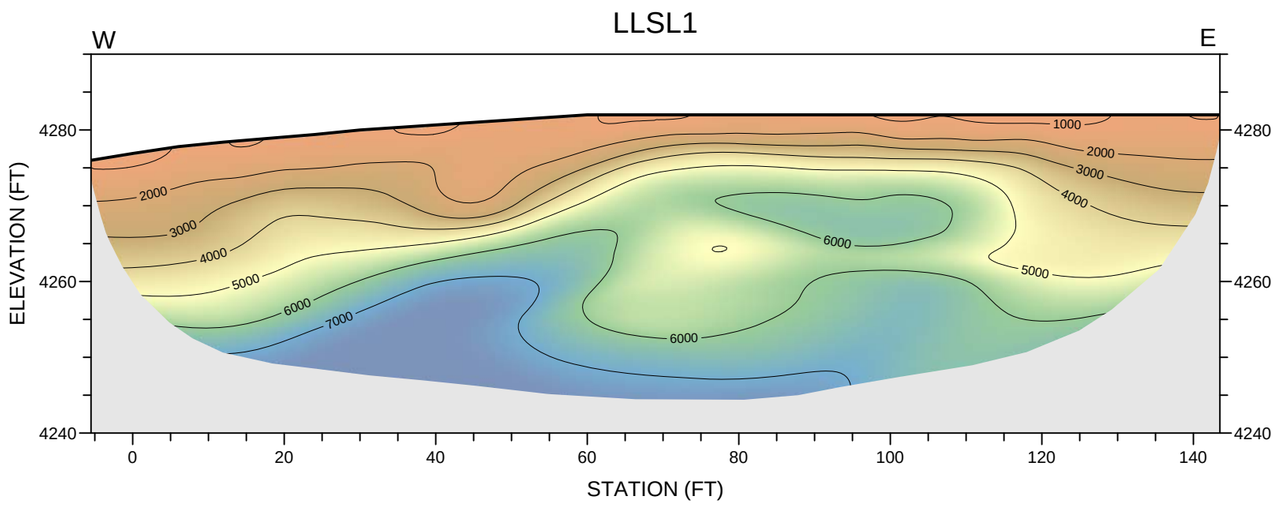
NORCAL GEOPHYSICAL CONSULTANTS INC.

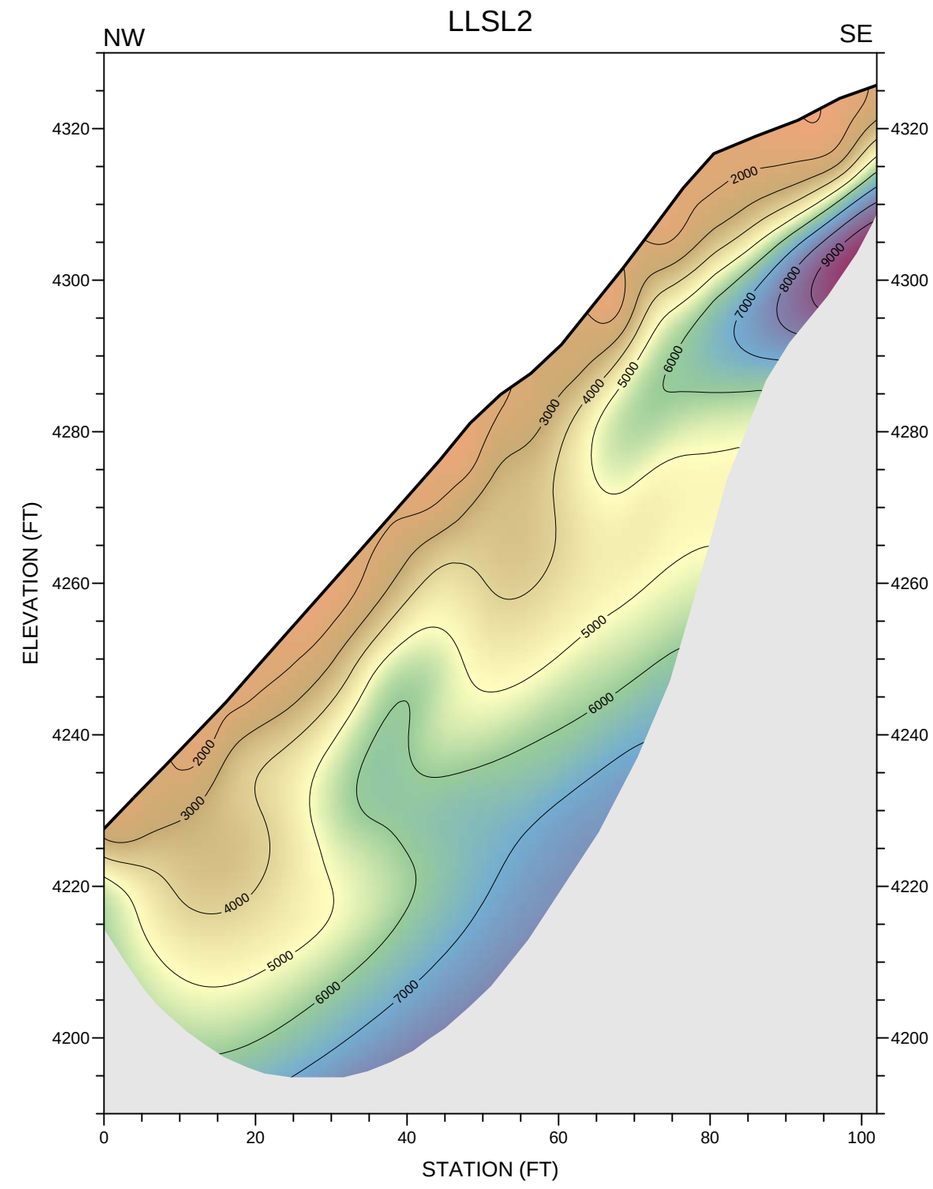
DATE: JUL. 2015

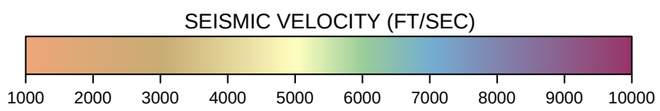
DRAWN BY: G.RANDALL

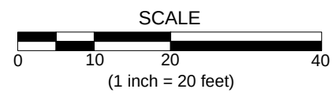
APPROVED BY: DJK

PLATE 4









NORCAL

SEISMIC REFRACTION PROFILES
LLSL1 & LLSL2
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

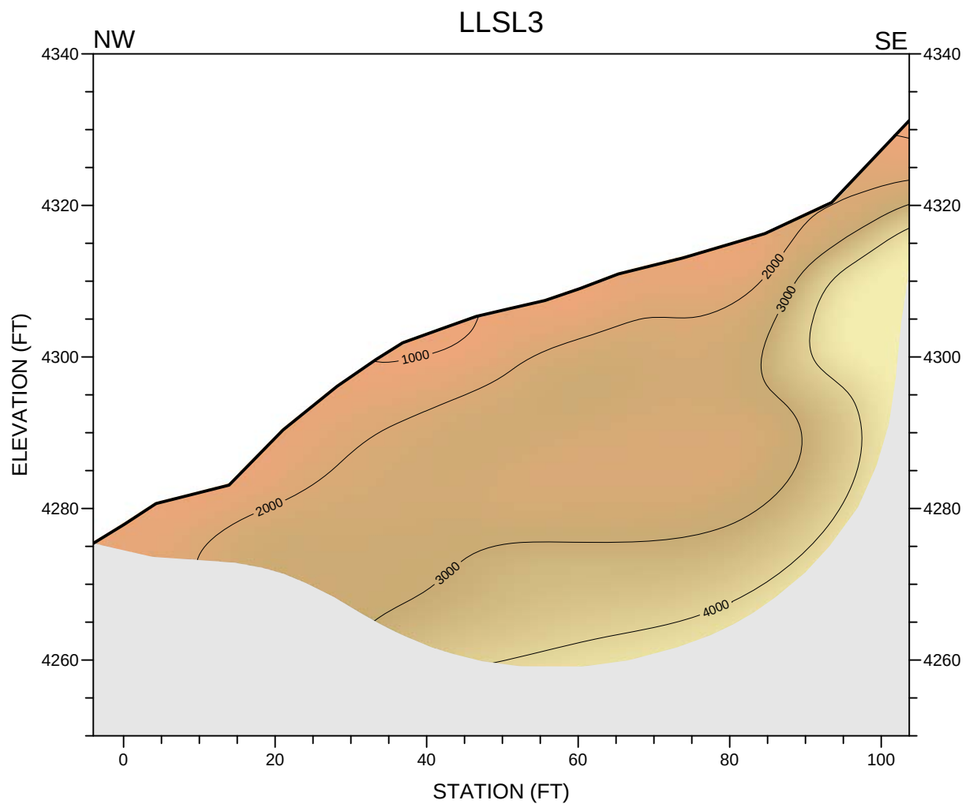
NORCAL GEOPHYSICAL CONSULTANTS INC.

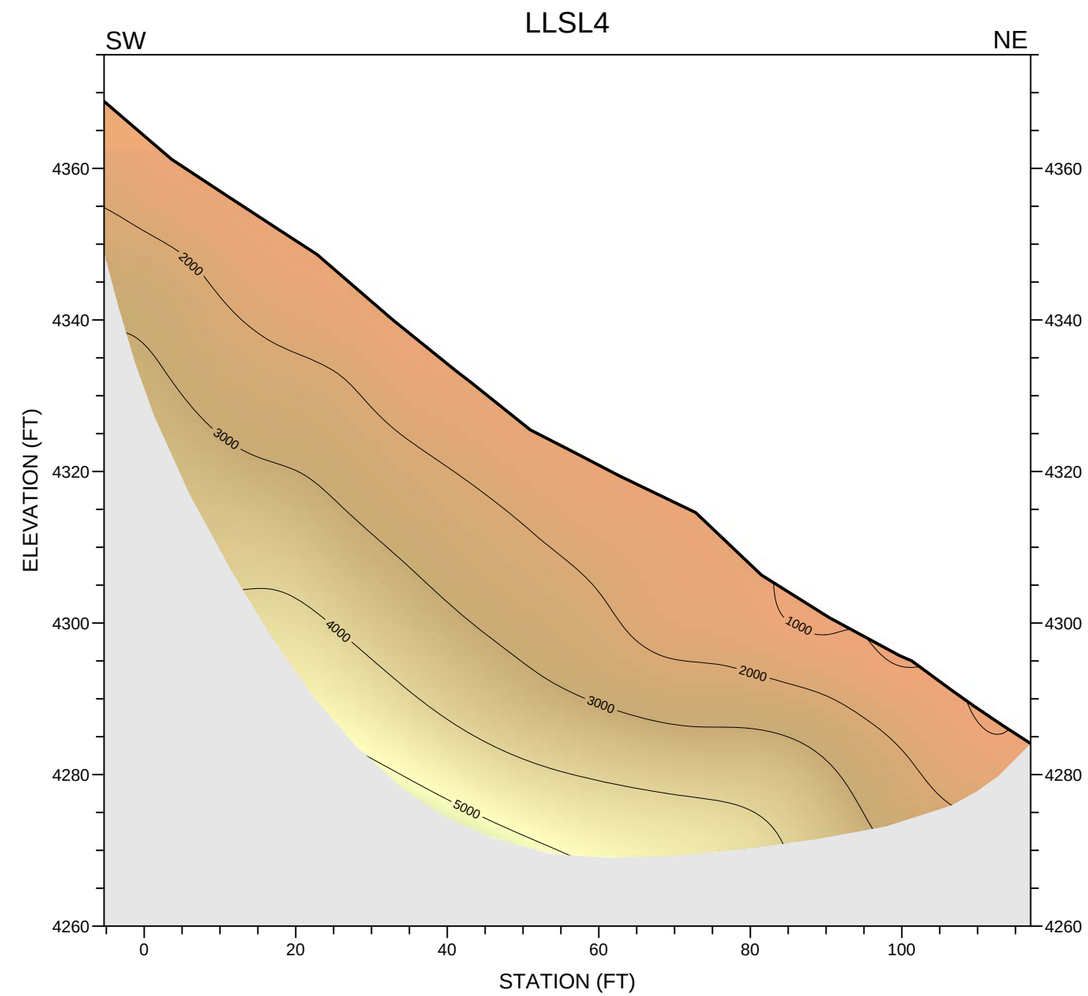
DATE: JUL. 2015

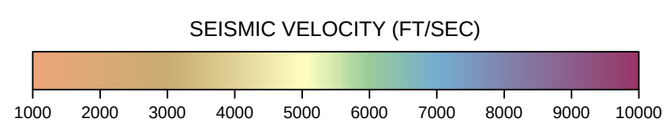
DRAWN BY: G.RANDALL

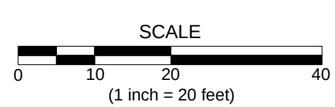
APPROVED BY: DJK

PLATE 5









NORCAL

SEISMIC REFRACTION PROFILES
LLSL3 & LLSL4
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

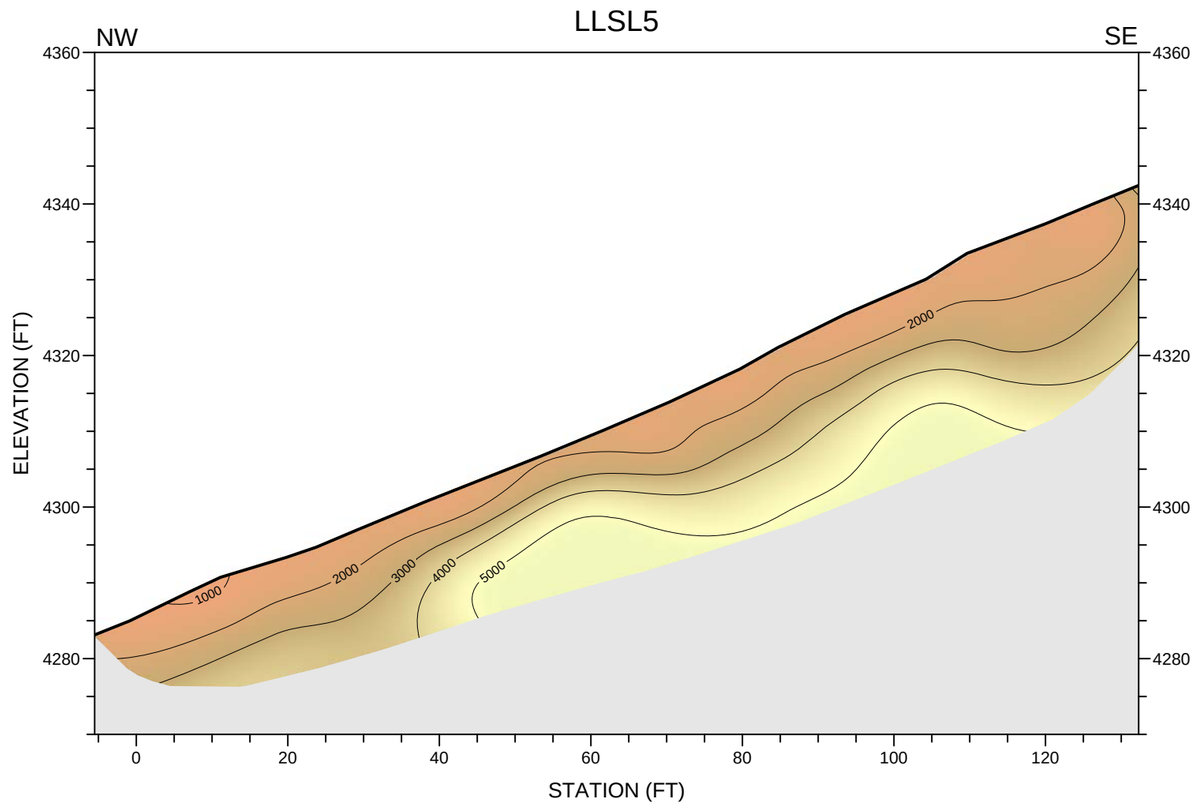
NORCAL GEOPHYSICAL CONSULTANTS INC.

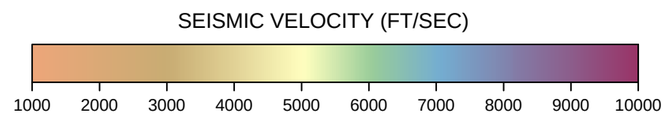
DATE: JUL. 2015

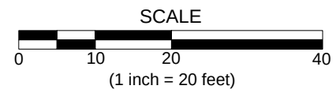
DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 6







NORCAL

SEISMIC REFRACTION PROFILE
LLSL5
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

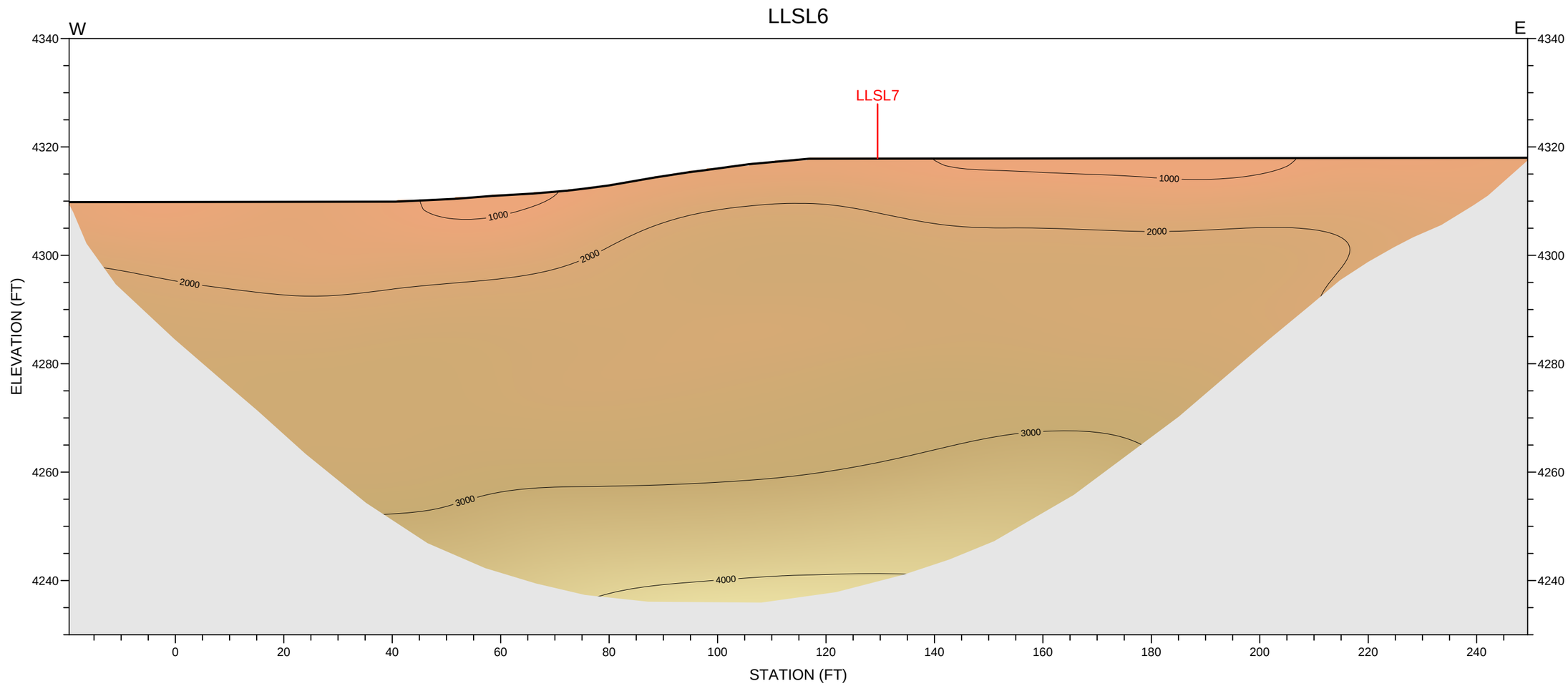
NORCAL GEOPHYSICAL CONSULTANTS INC.

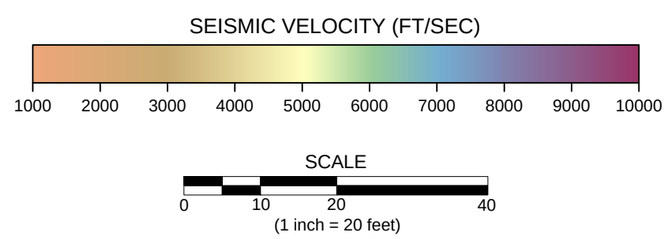
DATE: JUL. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 7





NORCAL

SEISMIC REFRACTION PROFILE
LLSL6
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

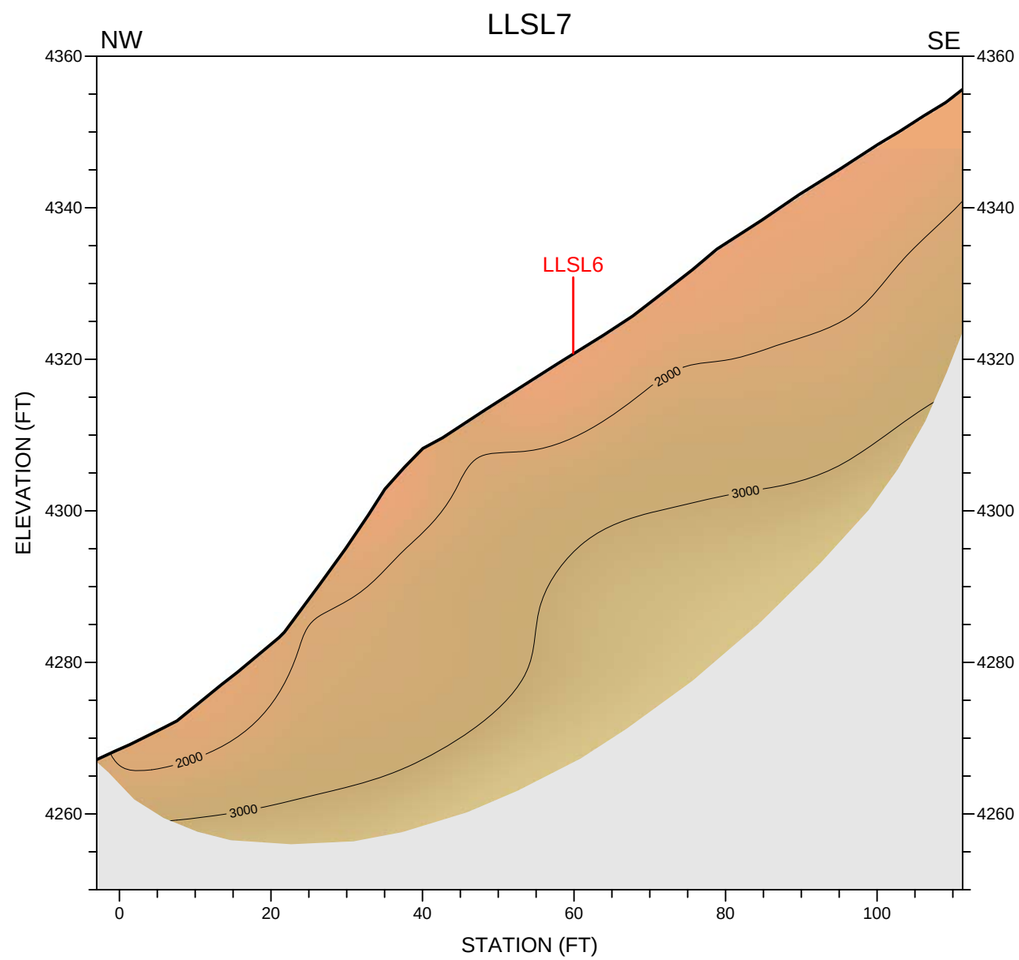
NORCAL GEOPHYSICAL CONSULTANTS INC.

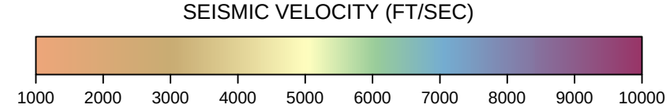
DATE: JUL. 2015

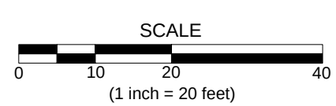
DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 8







NORCAL

SEISMIC REFRACTION PROFILE
LLSL7
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

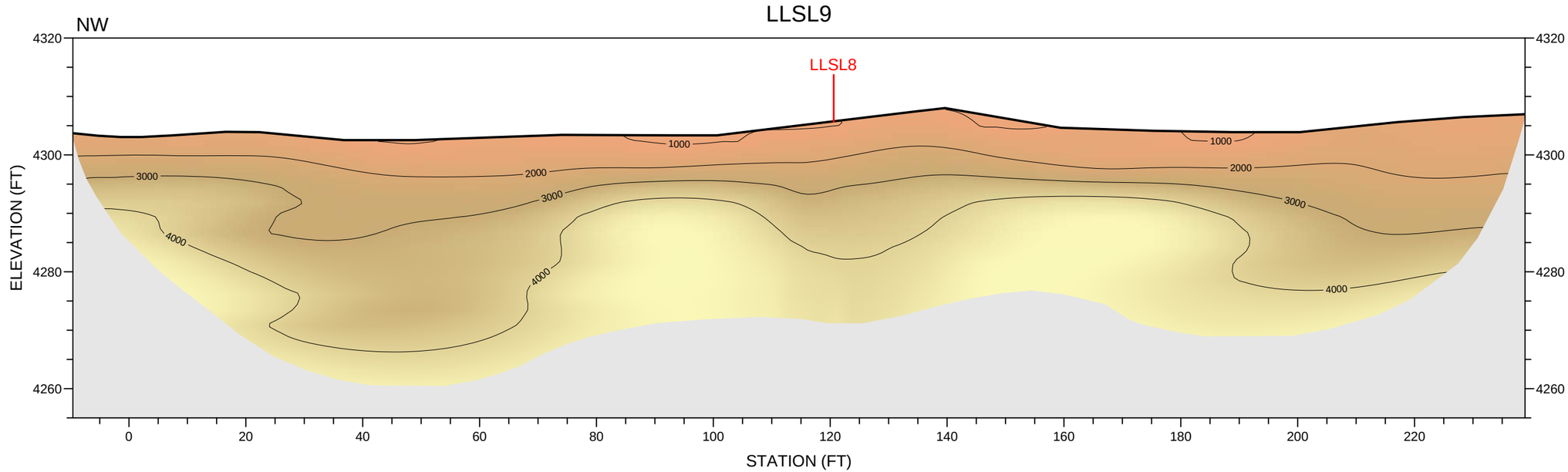
NORCAL GEOPHYSICAL CONSULTANTS INC.

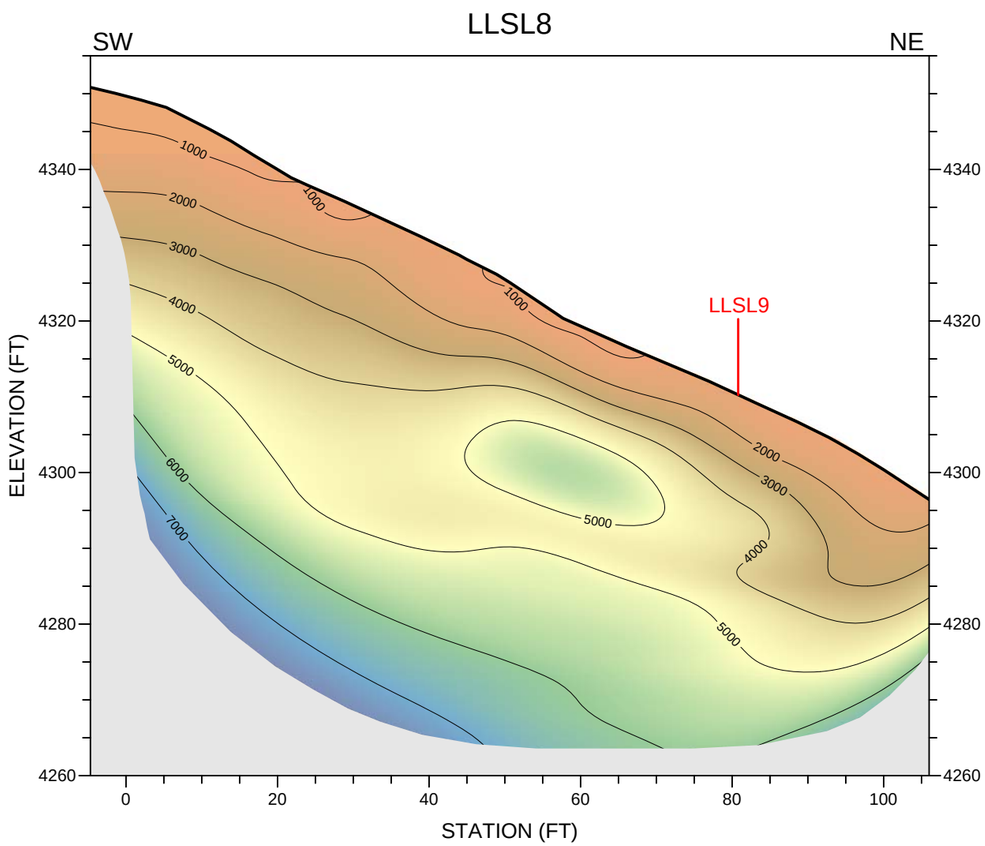
DATE: JUL. 2015

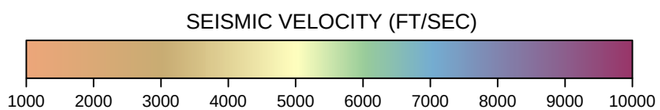
DRAWN BY: G.RANDALL

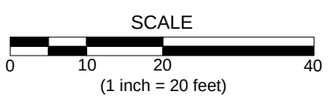
APPROVED BY: DJK

PLATE
9







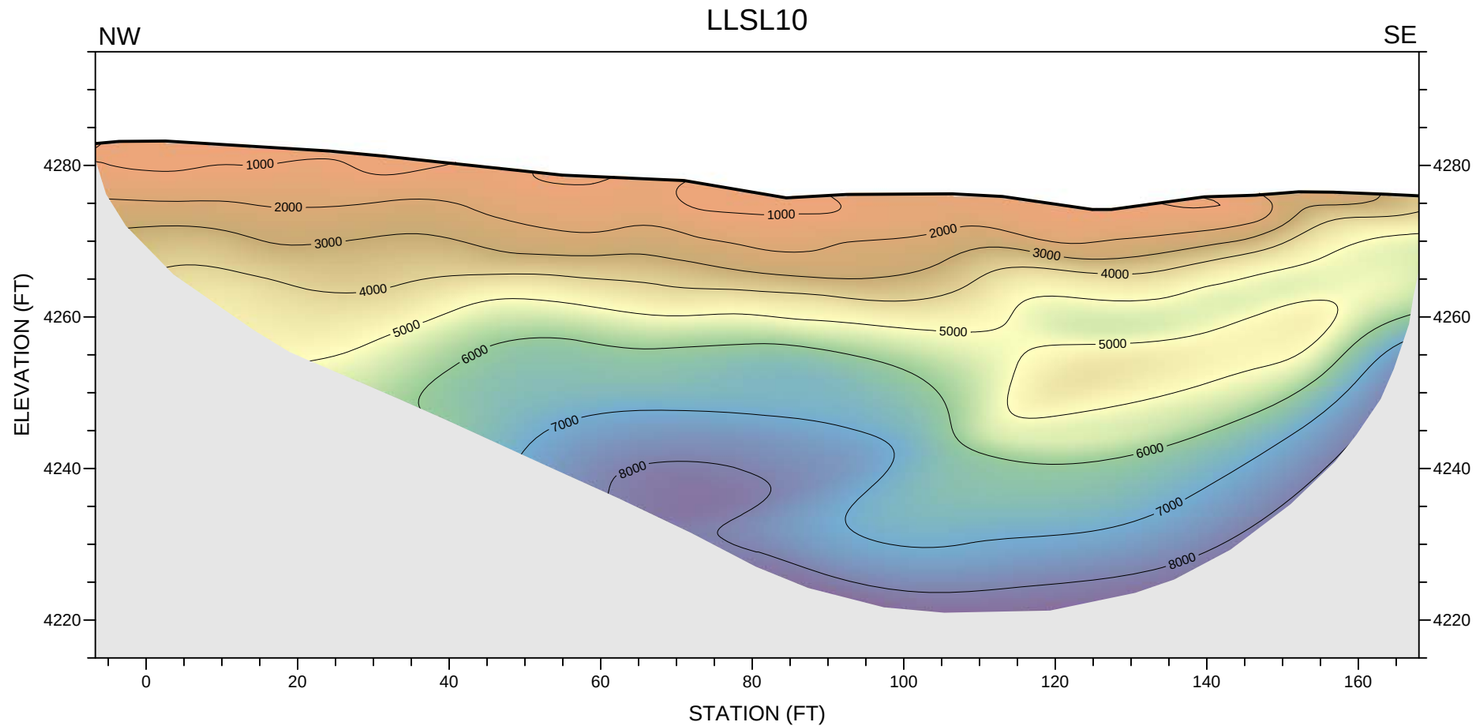


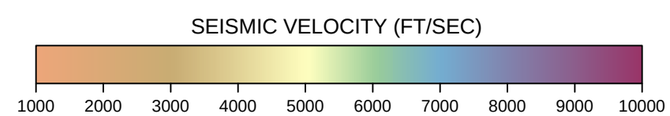
NORCAL

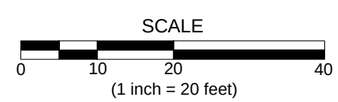
SEISMIC REFRACTION PROFILES
LLSL8 & LLSL9
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
JOB #: 15-177.216
NORCAL GEOPHYSICAL CONSULTANTS INC.
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE
10







NORCAL

SEISMIC REFRACTION PROFILE
LLSL10
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

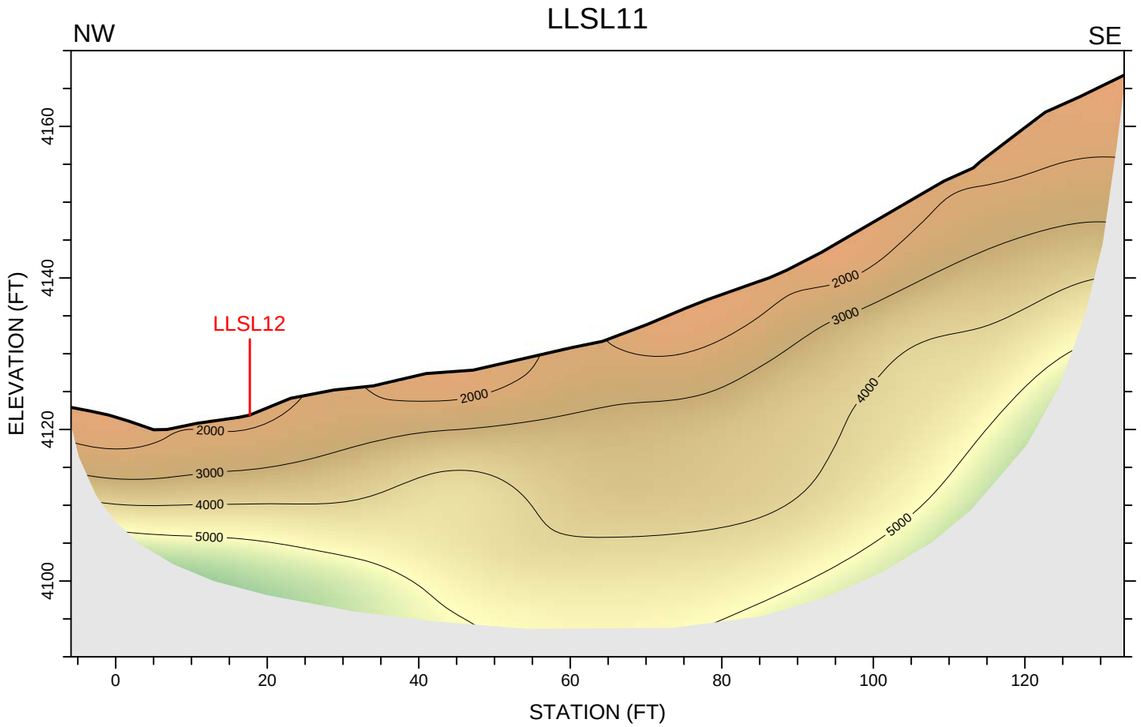
NORCAL GEOPHYSICAL CONSULTANTS INC.

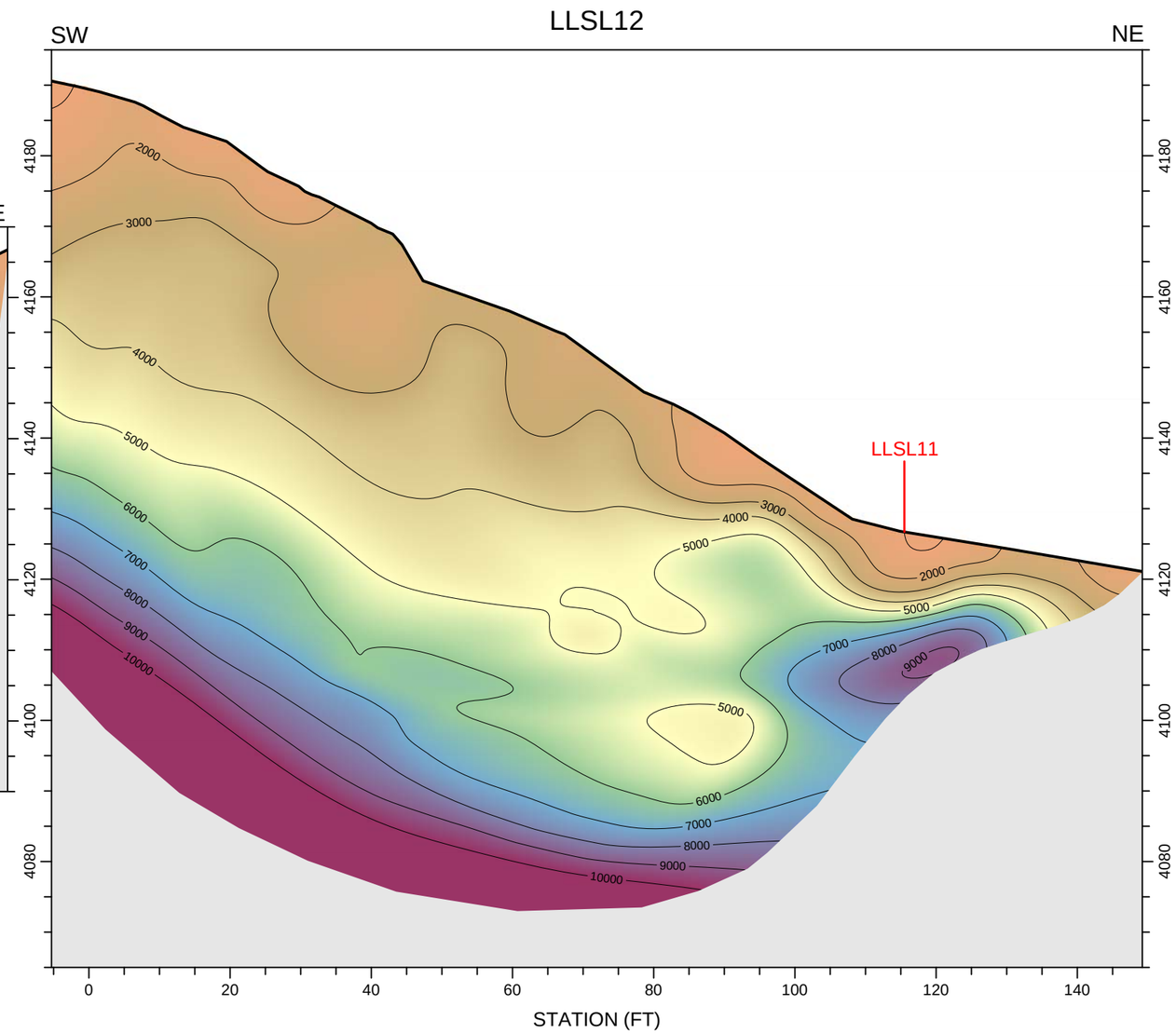
DATE: JUL. 2015

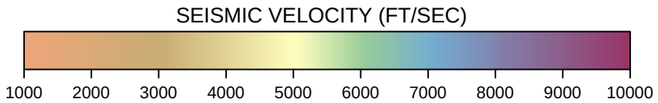
DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 11







SCALE

0 10 20 40

(1 inch = 20 feet)

NORCAL

SEISMIC REFRACTION PROFILES
LLSL11 & LLSL12
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

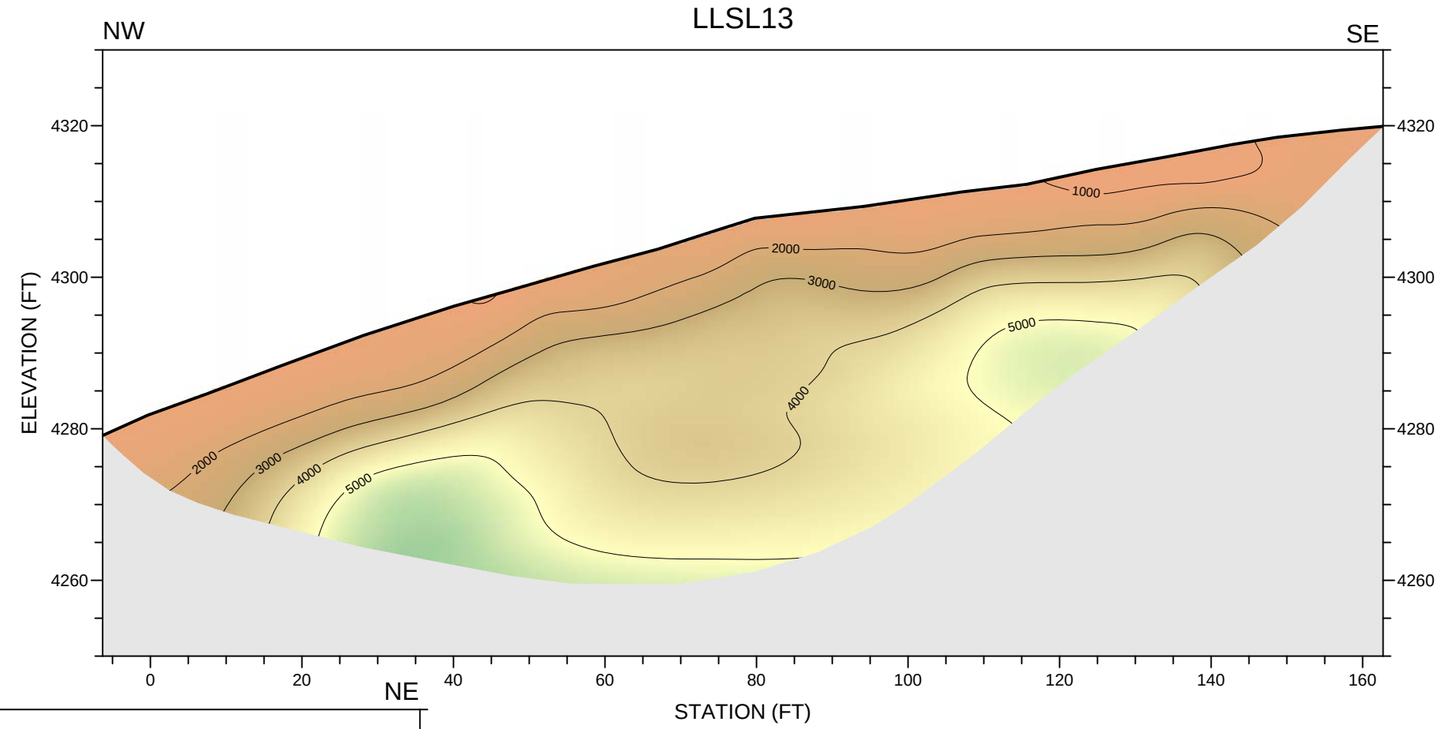
NORCAL GEOPHYSICAL CONSULTANTS INC.

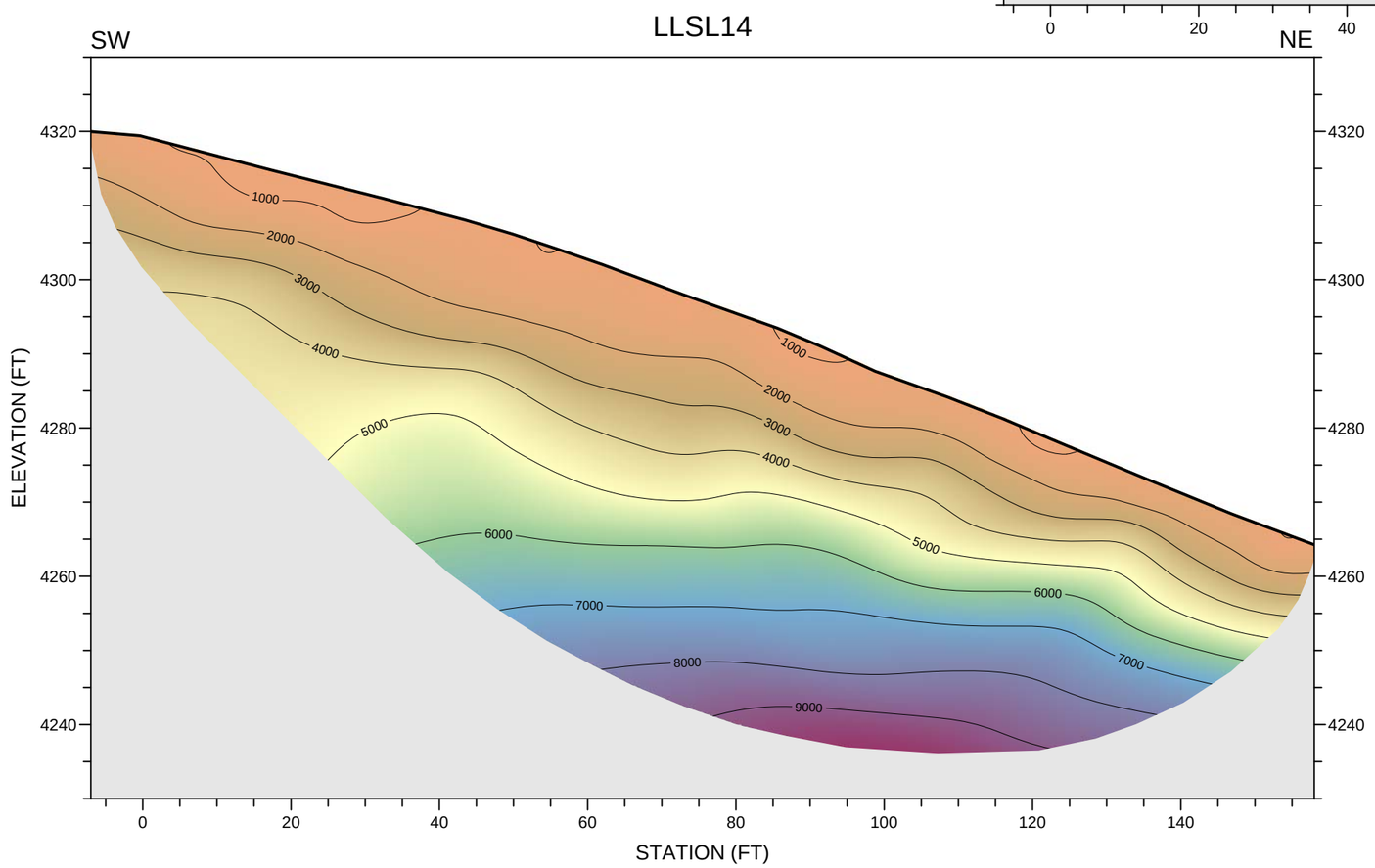
DATE: JUL. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 12





SEISMIC VELOCITY (FT/SEC)

1000 2000 3000 4000 5000 6000 7000 8000 9000 10000

SCALE

0 10 20 40

(1 inch = 20 feet)

NORCAL

SEISMIC REFRACTION PROFILES
LLSL13 & LLSL14
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

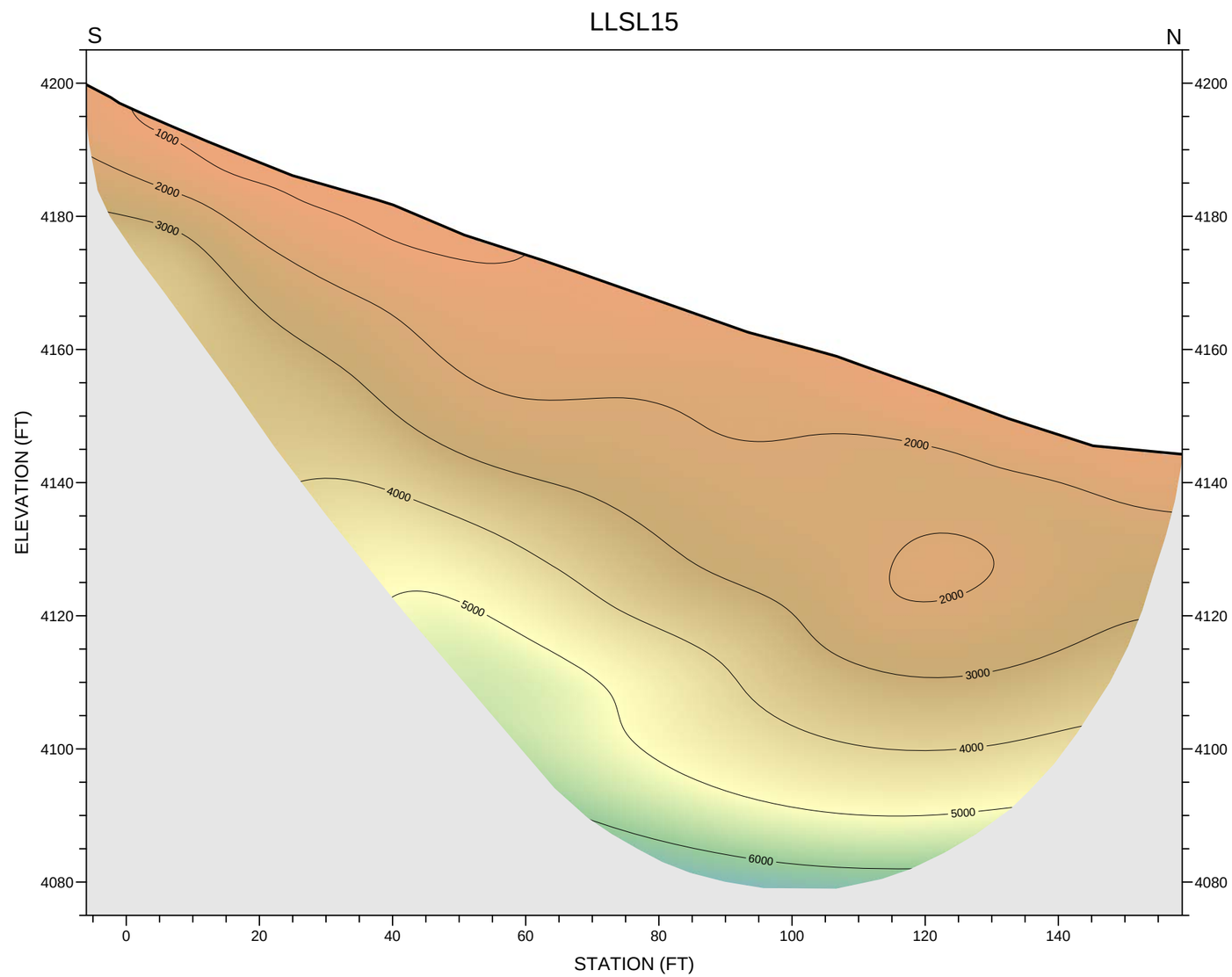
NORCAL GEOPHYSICAL CONSULTANTS INC.

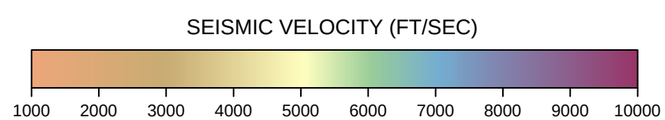
DATE: JUL. 2015

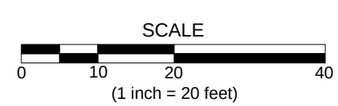
DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 13







NORCAL

SEISMIC REFRACTION PROFILE
LLSL15
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

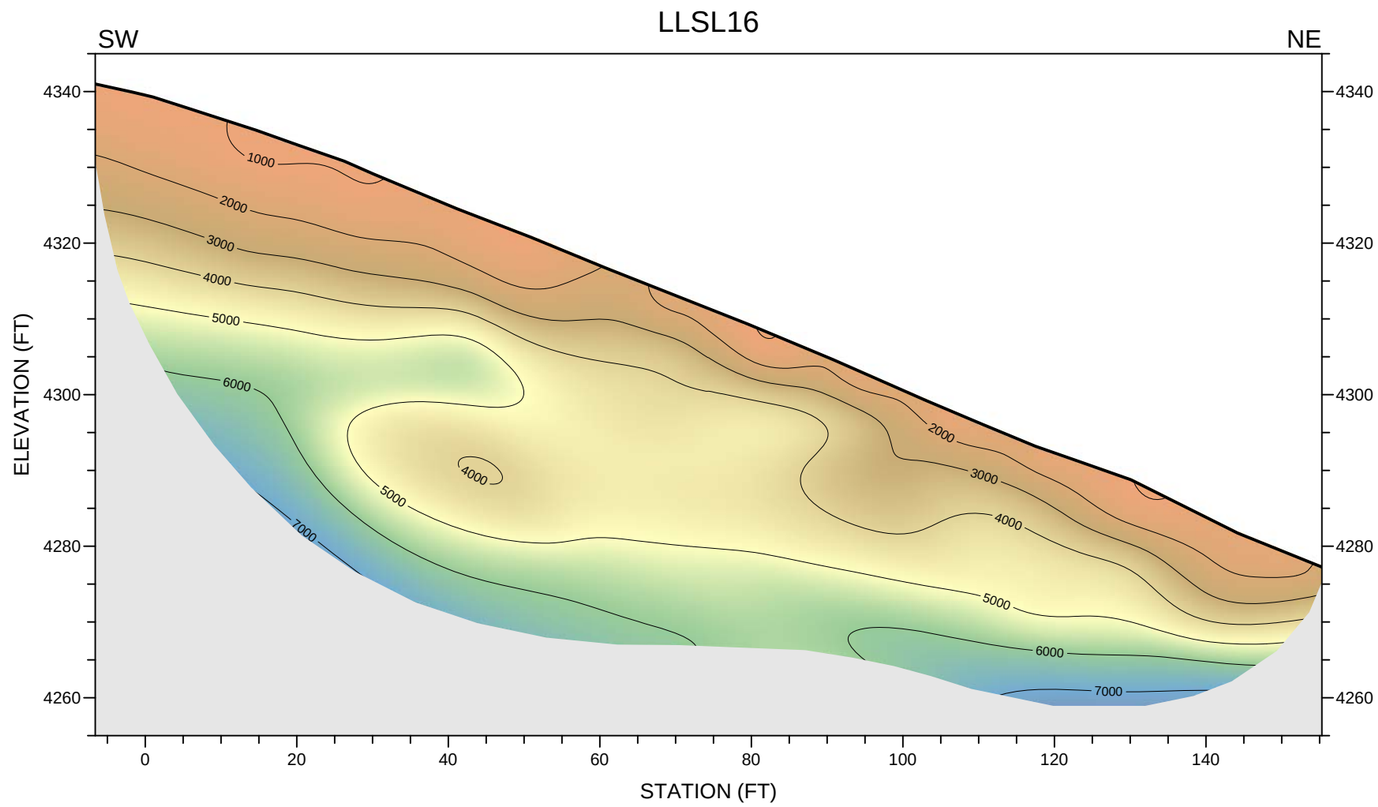
NORCAL GEOPHYSICAL CONSULTANTS INC.

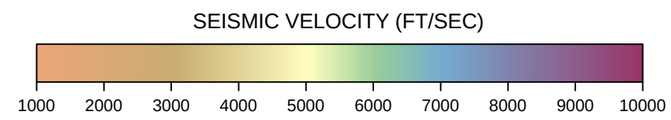
DATE: JUL. 2015

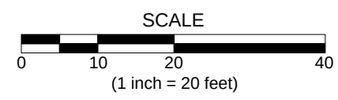
DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 14







NORCAL

SEISMIC REFRACTION PROFILE
LLSL16
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

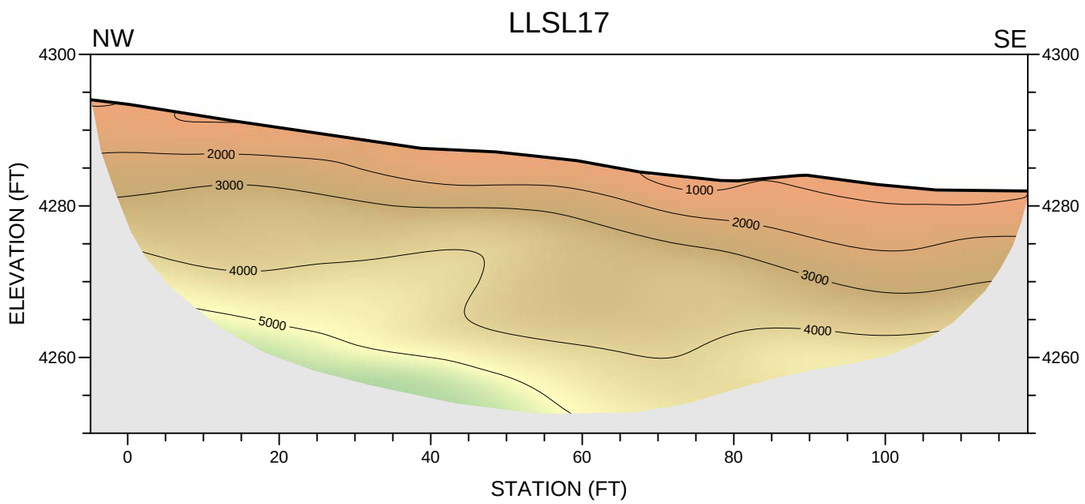
NORCAL GEOPHYSICAL CONSULTANTS INC.

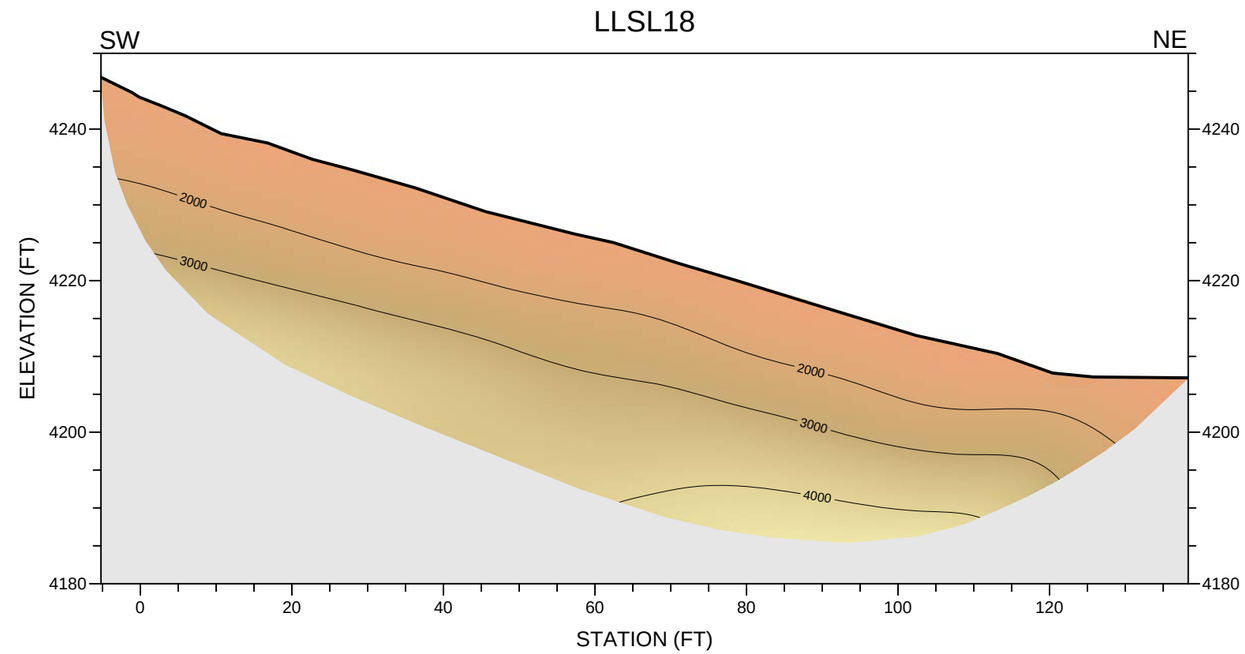
DATE: JUL. 2015

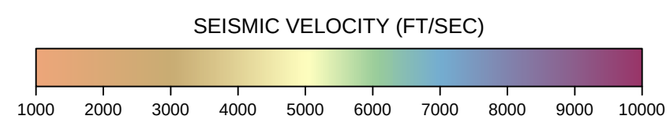
DRAWN BY: G.RANDALL

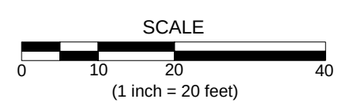
APPROVED BY: DJK

PLATE 15









NORCAL

SEISMIC REFRACTION PROFILES
LLSL17 & LLSL18
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA

CLIENT: KLEINFELDER

JOB #: 15-177.216

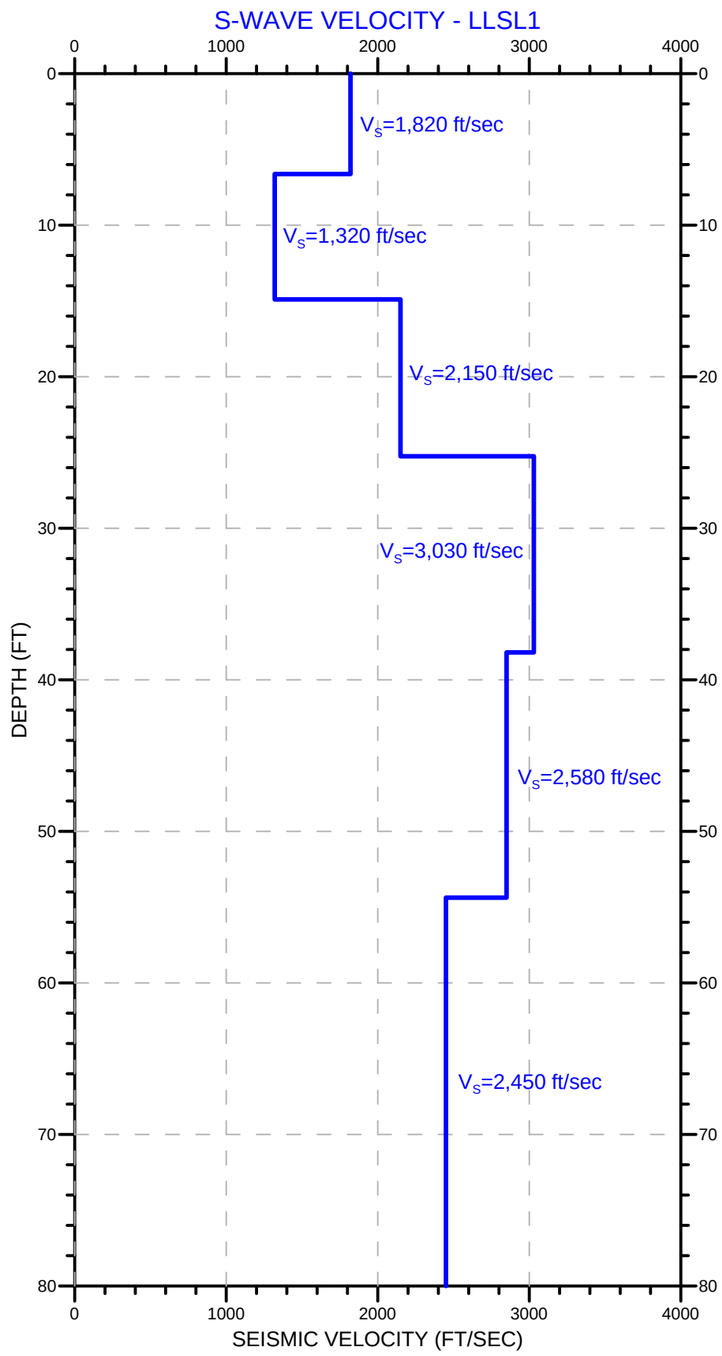
NORCAL GEOPHYSICAL CONSULTANTS INC.

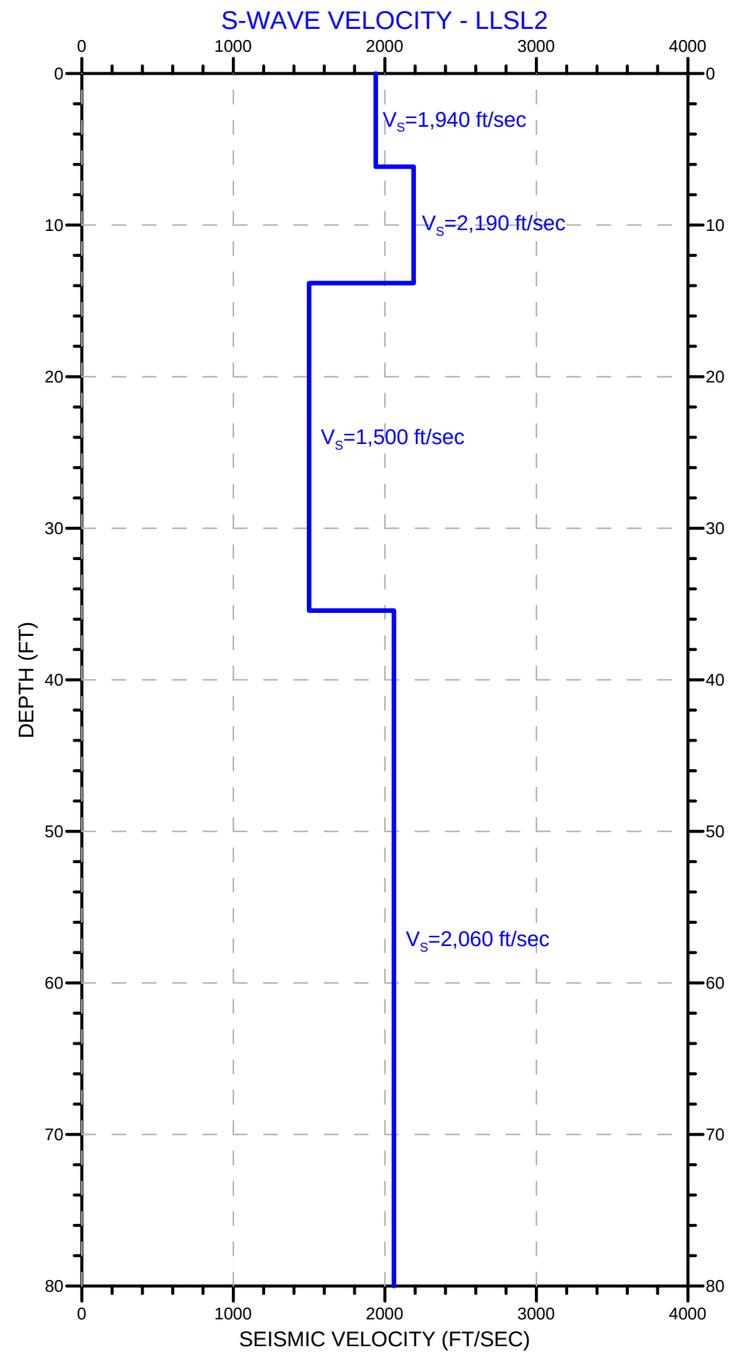
DATE: JUL. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 16





LEGEND	
———	S-WAVE VELOCITY (MASW)

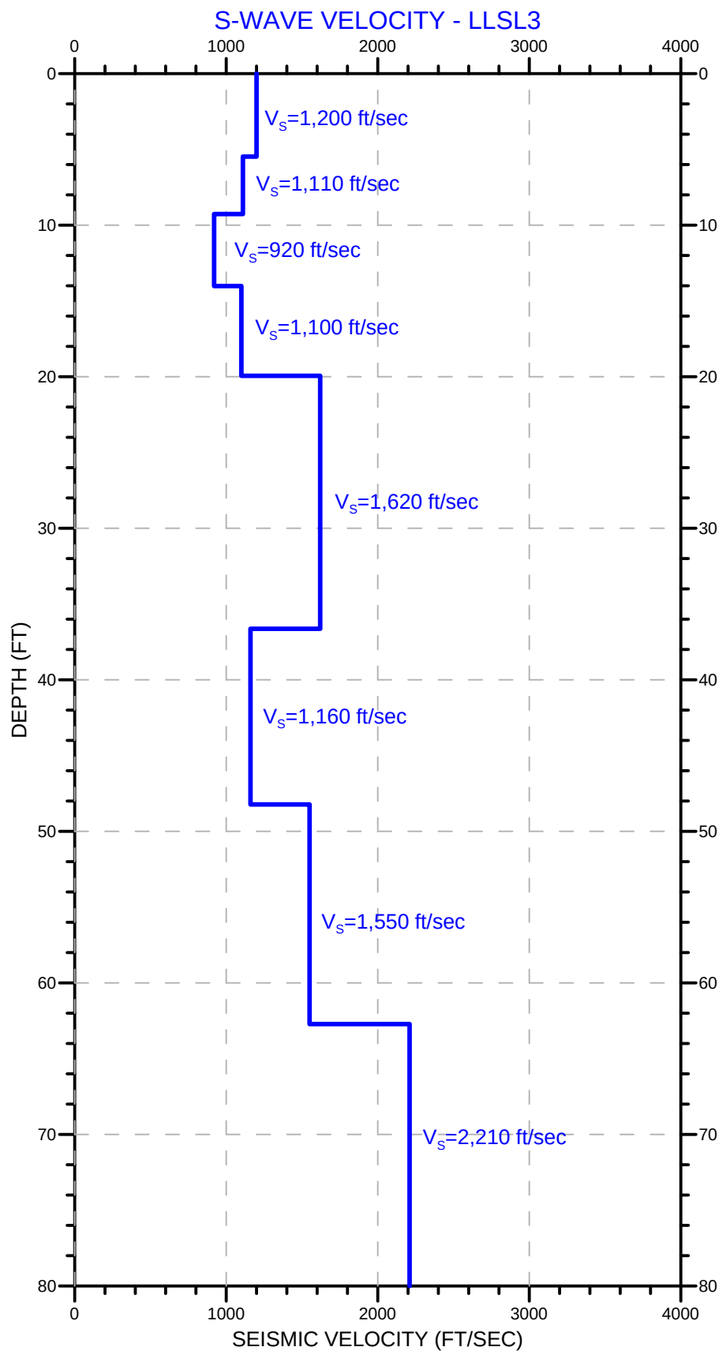
NORCAL

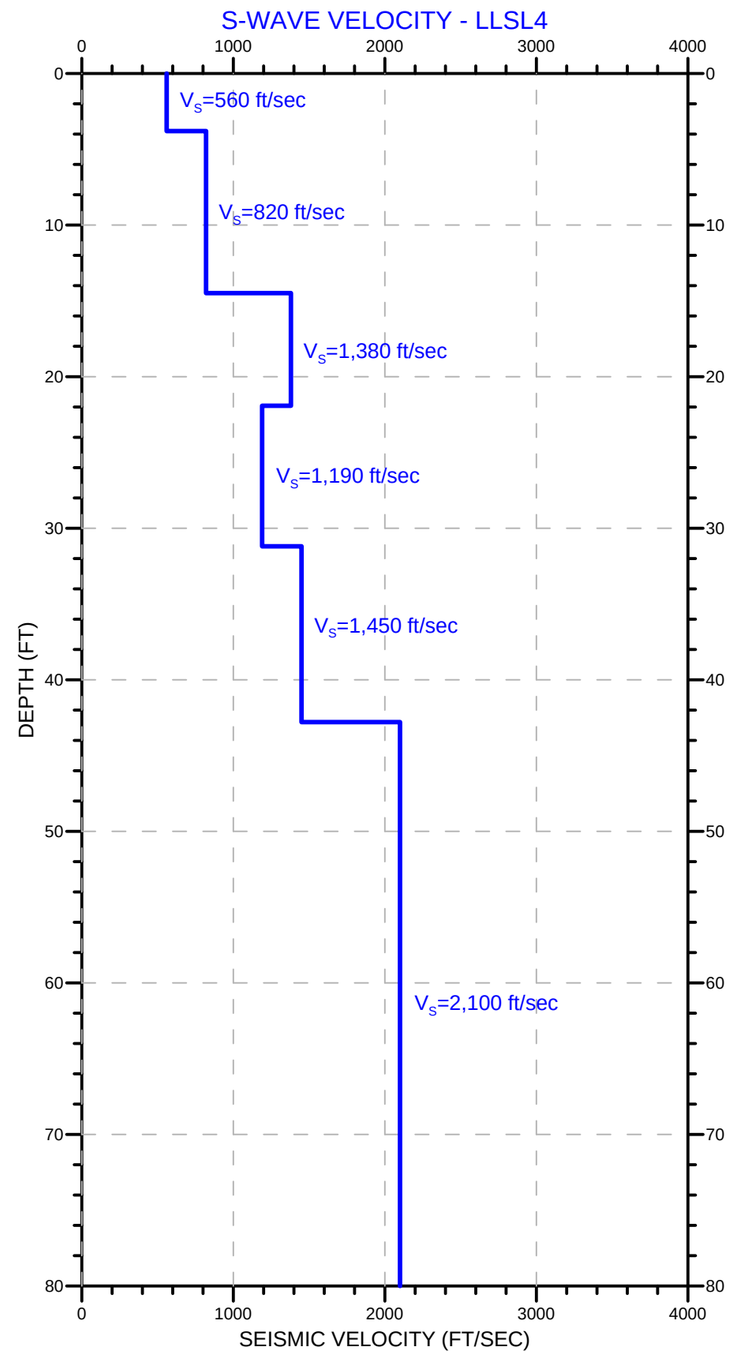
MASW 1D S-WAVE VELOCITY SOUNDINGS
LLSL1 & LLSL2
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-177.216
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE 17





LEGEND	
———	S-WAVE VELOCITY (MASW)

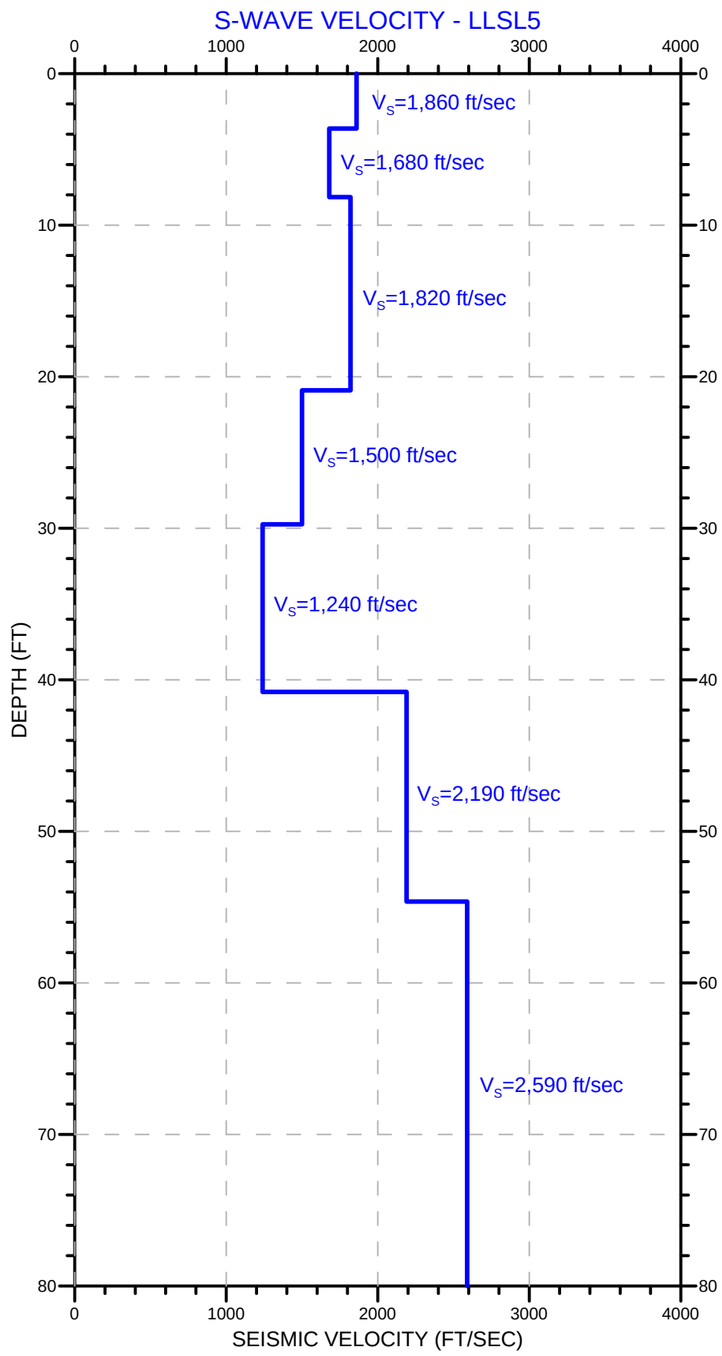
NORCAL

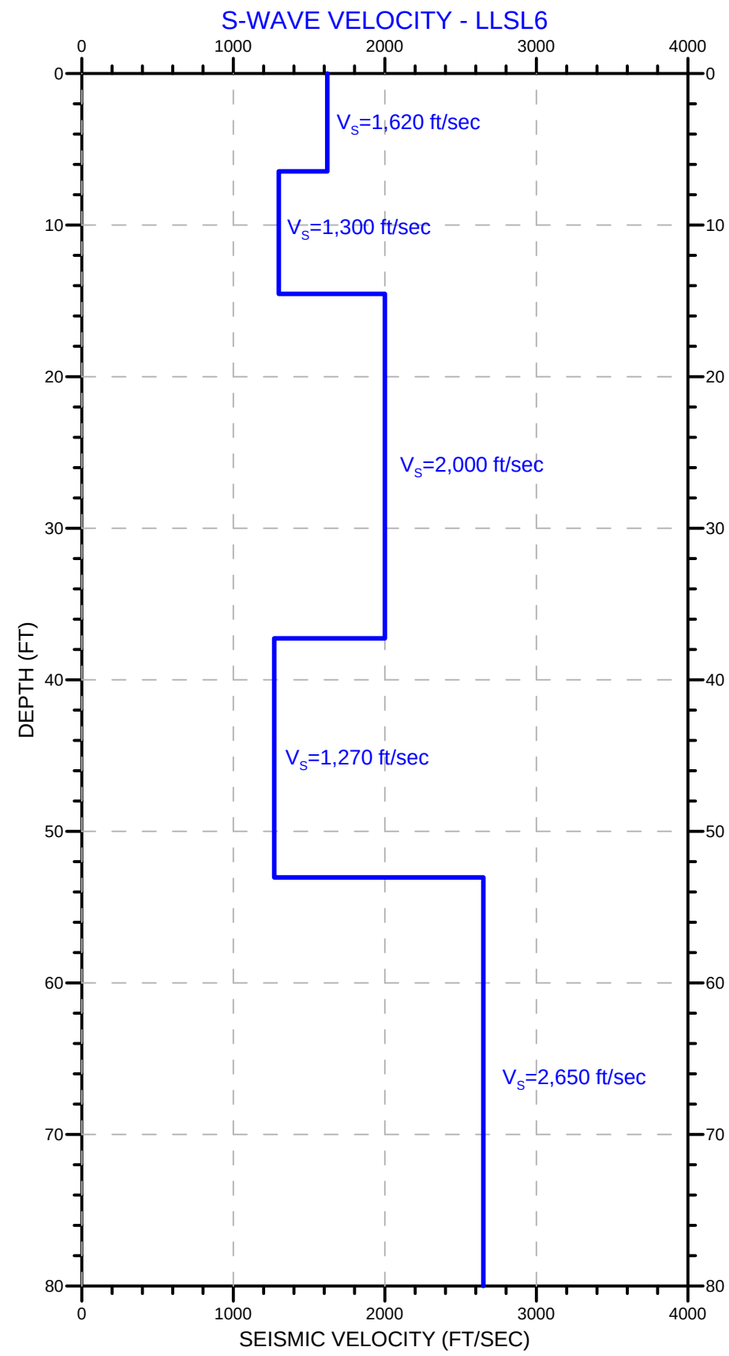
MASW 1D S-WAVE VELOCITY SOUNDINGS
LLSL3 & LLSL4
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-177.216
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE 18





LEGEND	
———	S-WAVE VELOCITY (MASW)

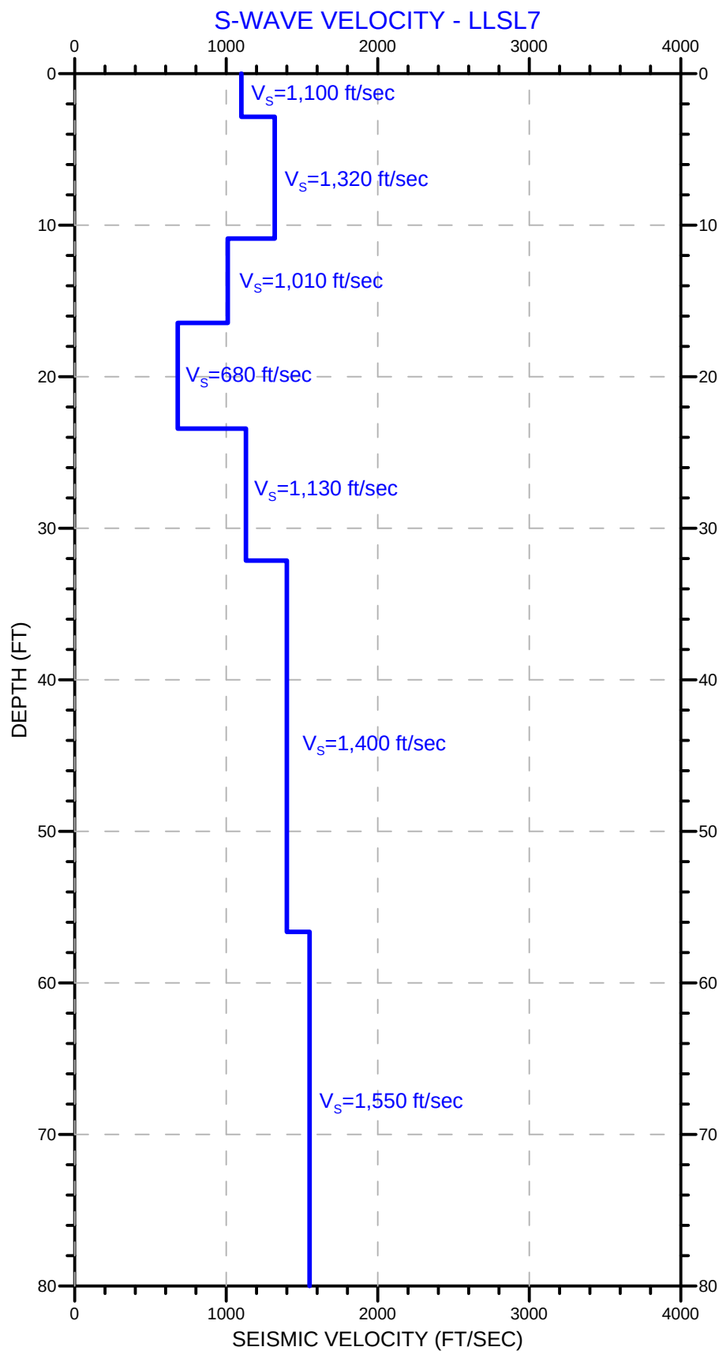
NORCAL

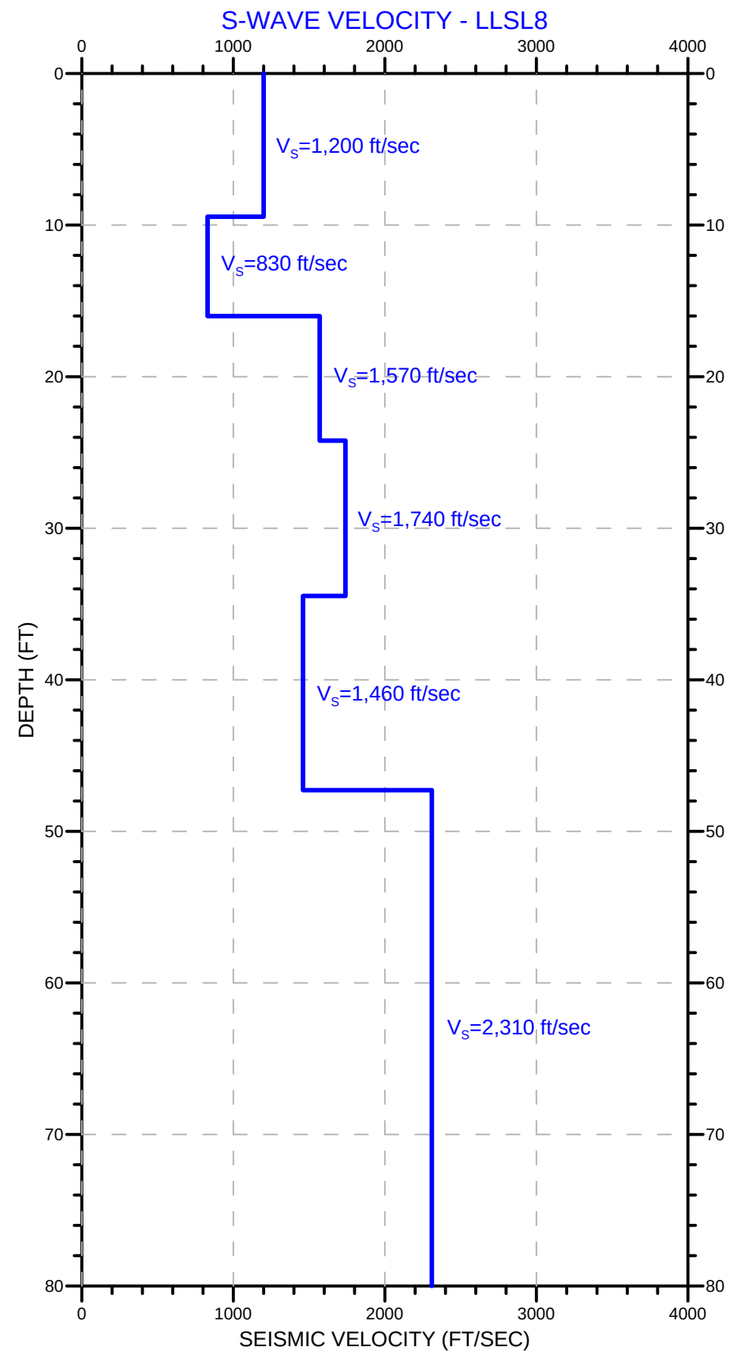
MASW 1D S-WAVE VELOCITY SOUNDINGS
LLSL5 & LLSL6
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-177.216
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE 19





LEGEND	
———	S-WAVE VELOCITY (MASW)

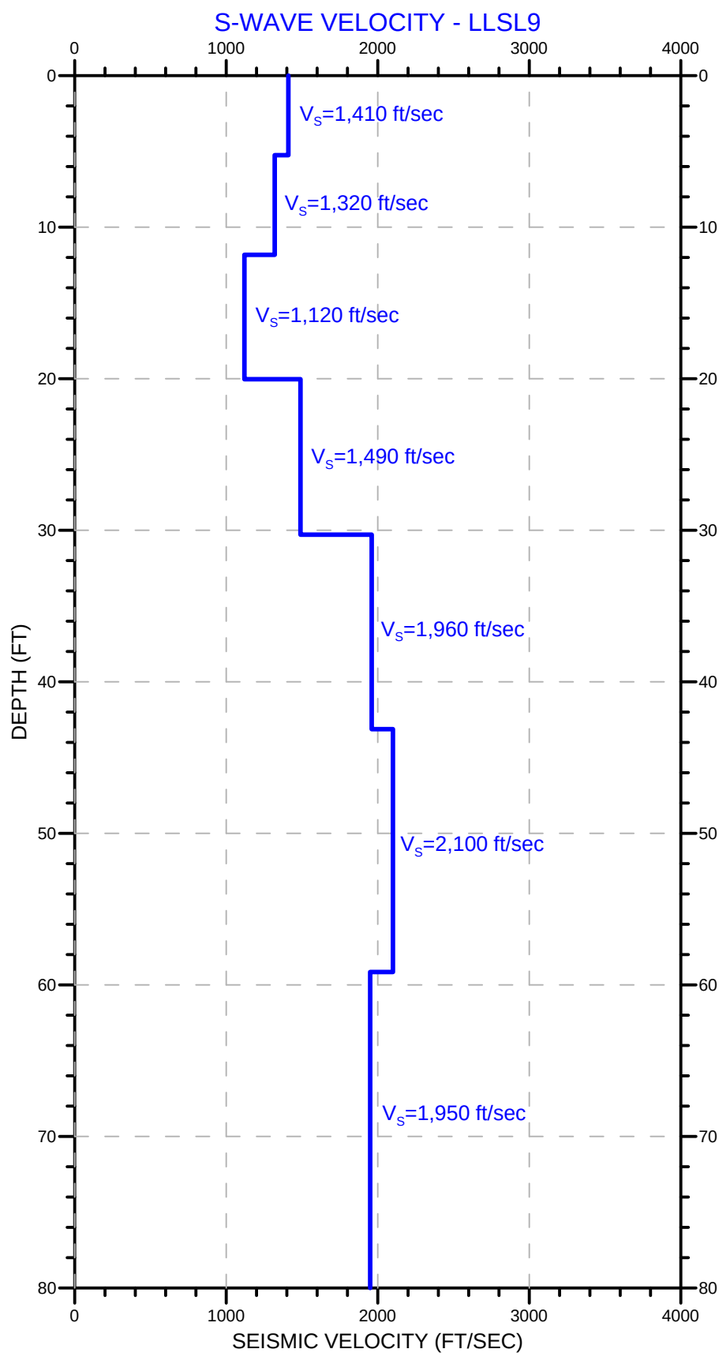
NORCAL

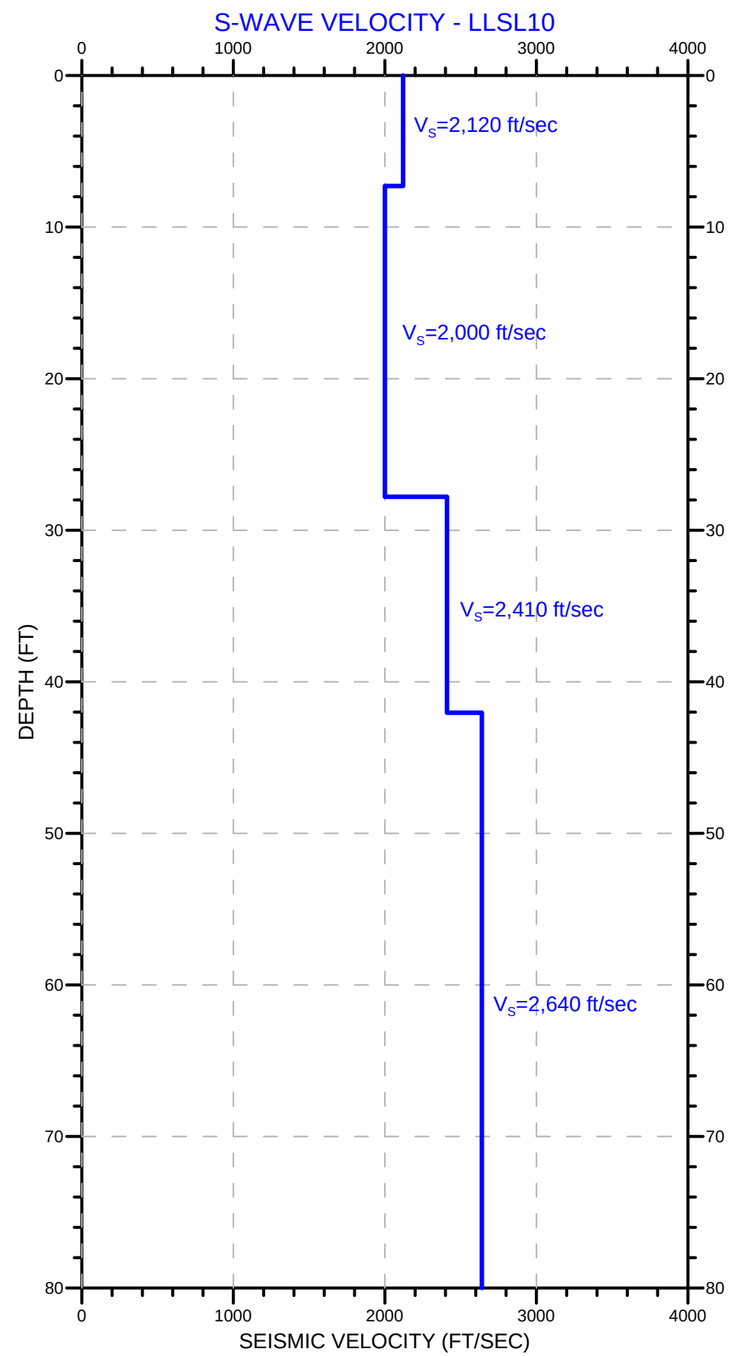
MASW 1D S-WAVE VELOCITY SOUNDINGS
LLSL7 & LLSL8
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-177.216
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE 20





LEGEND	
———	S-WAVE VELOCITY (MASW)

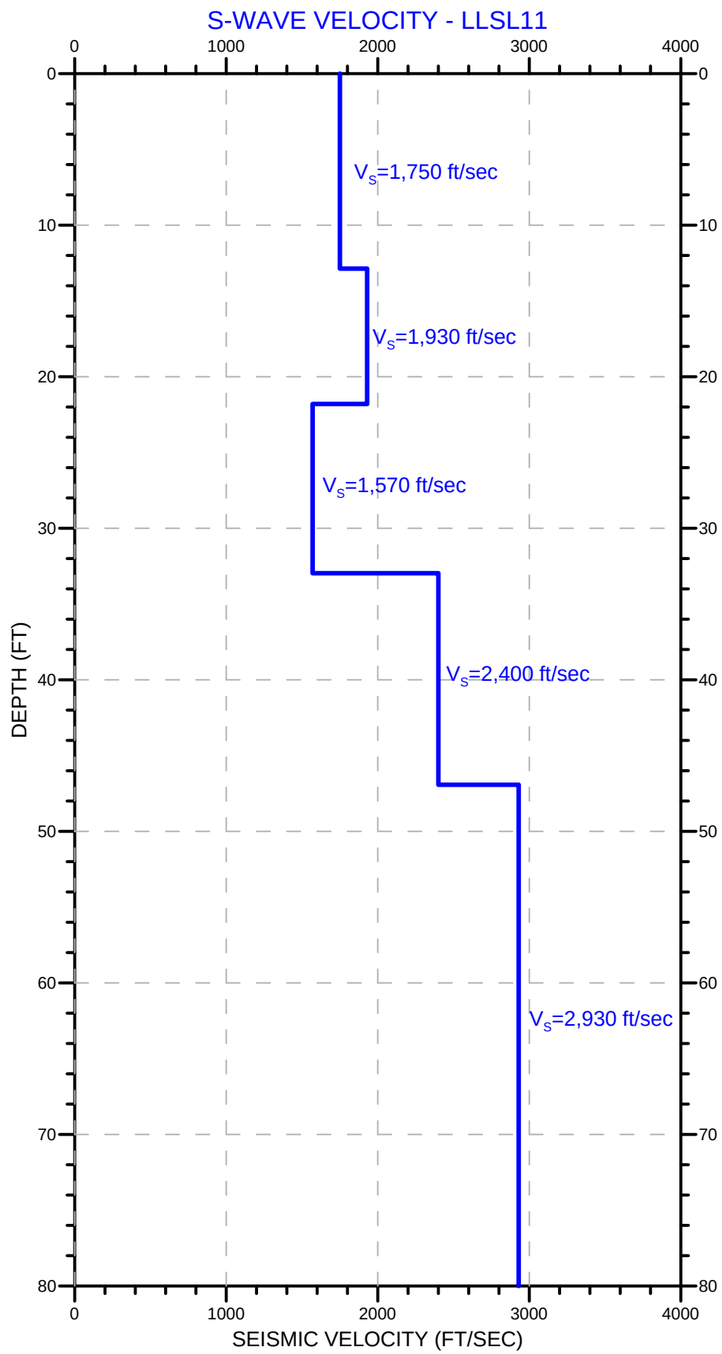
NORCAL

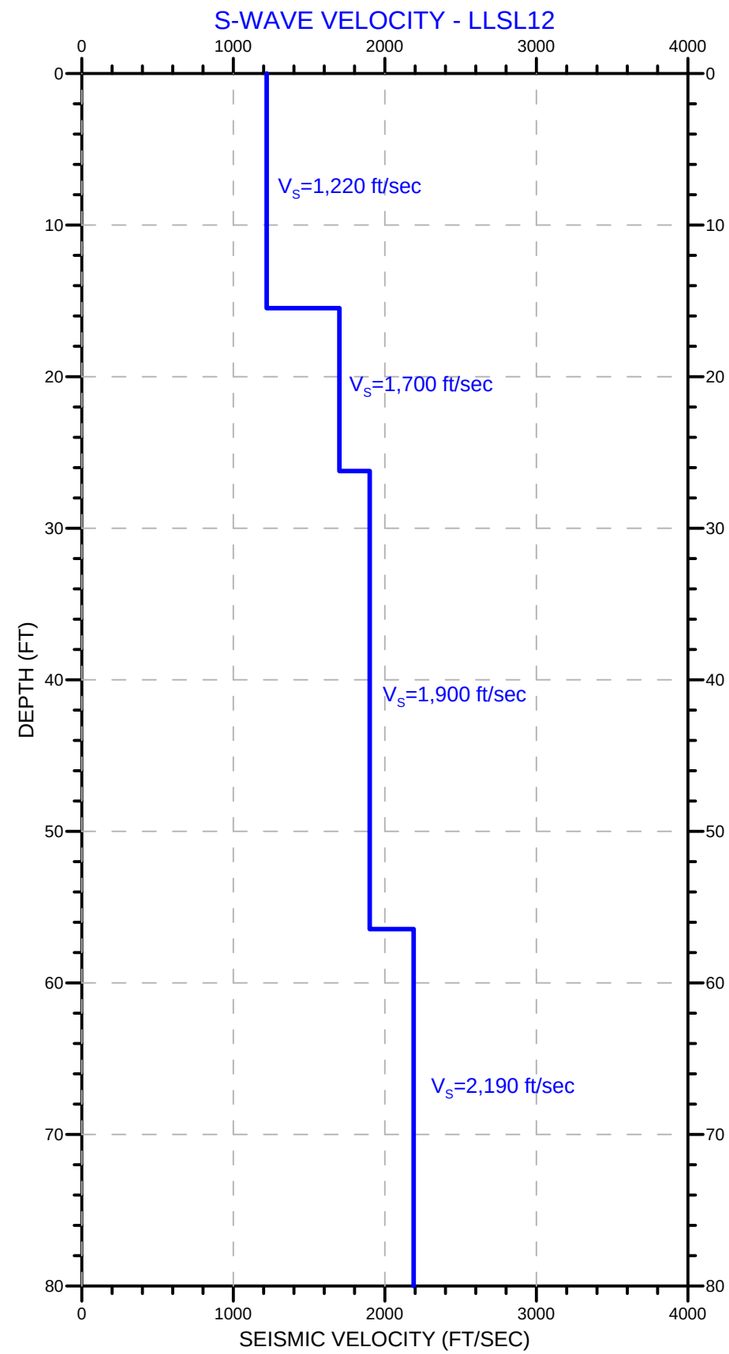
MASW 1D S-WAVE VELOCITY SOUNDINGS
LLSL9 & LLSL10
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-177.216
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE 21





LEGEND	
———	S-WAVE VELOCITY (MASW)

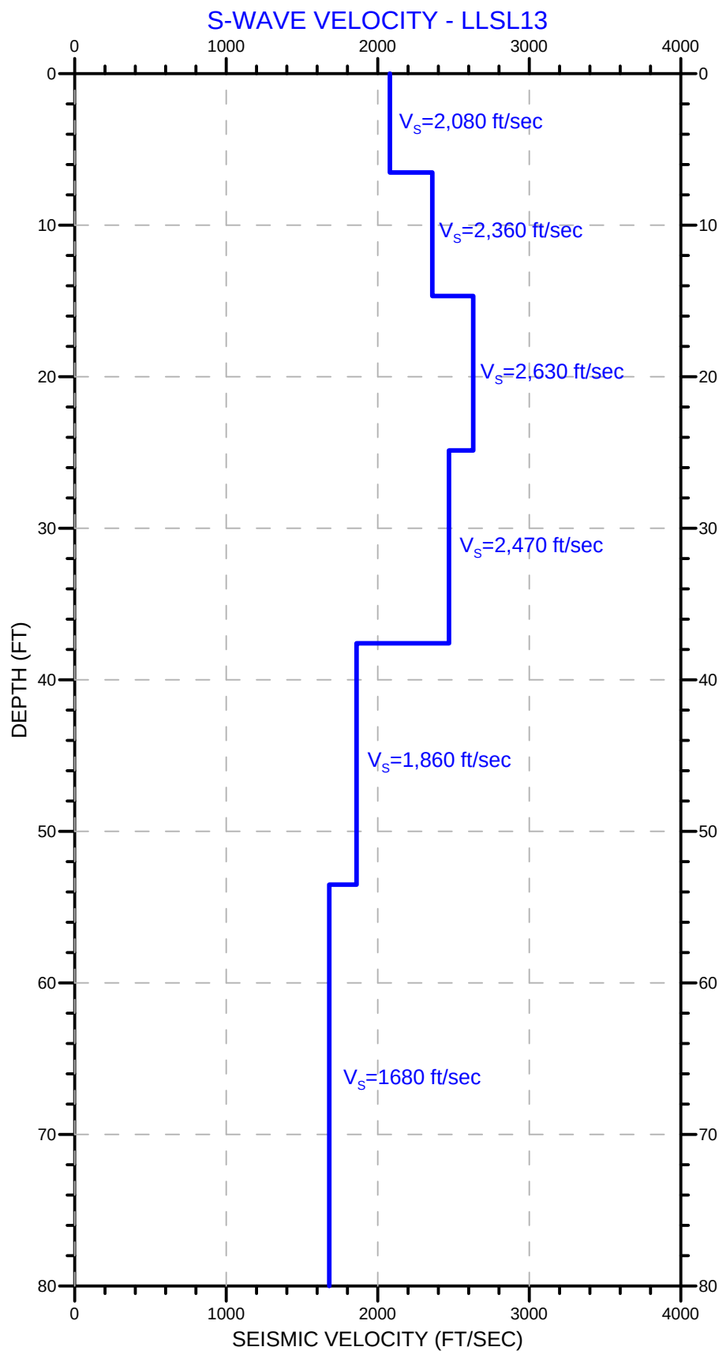
NORCAL

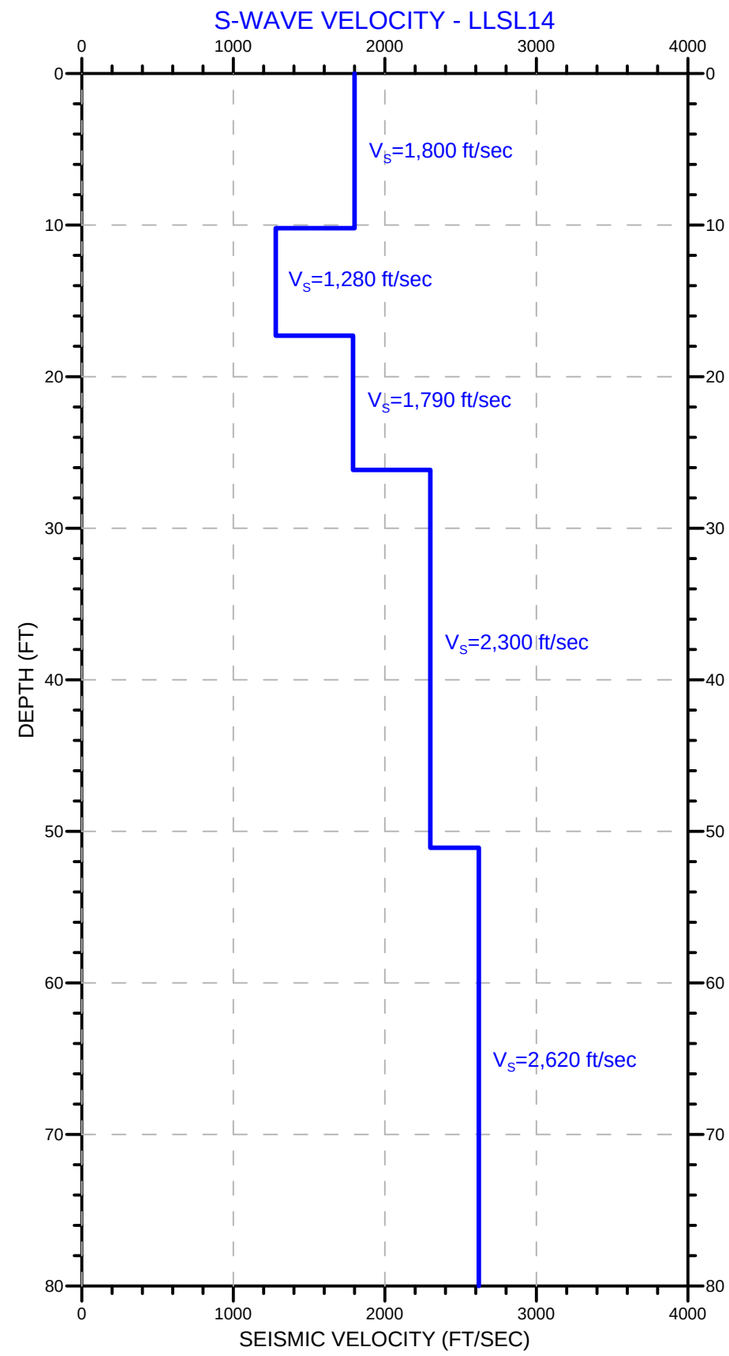
MASW 1D S-WAVE VELOCITY SOUNDINGS
LLSL11 & LLSL12
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-177.216
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE 22





LEGEND	
———	S-WAVE VELOCITY (MASW)

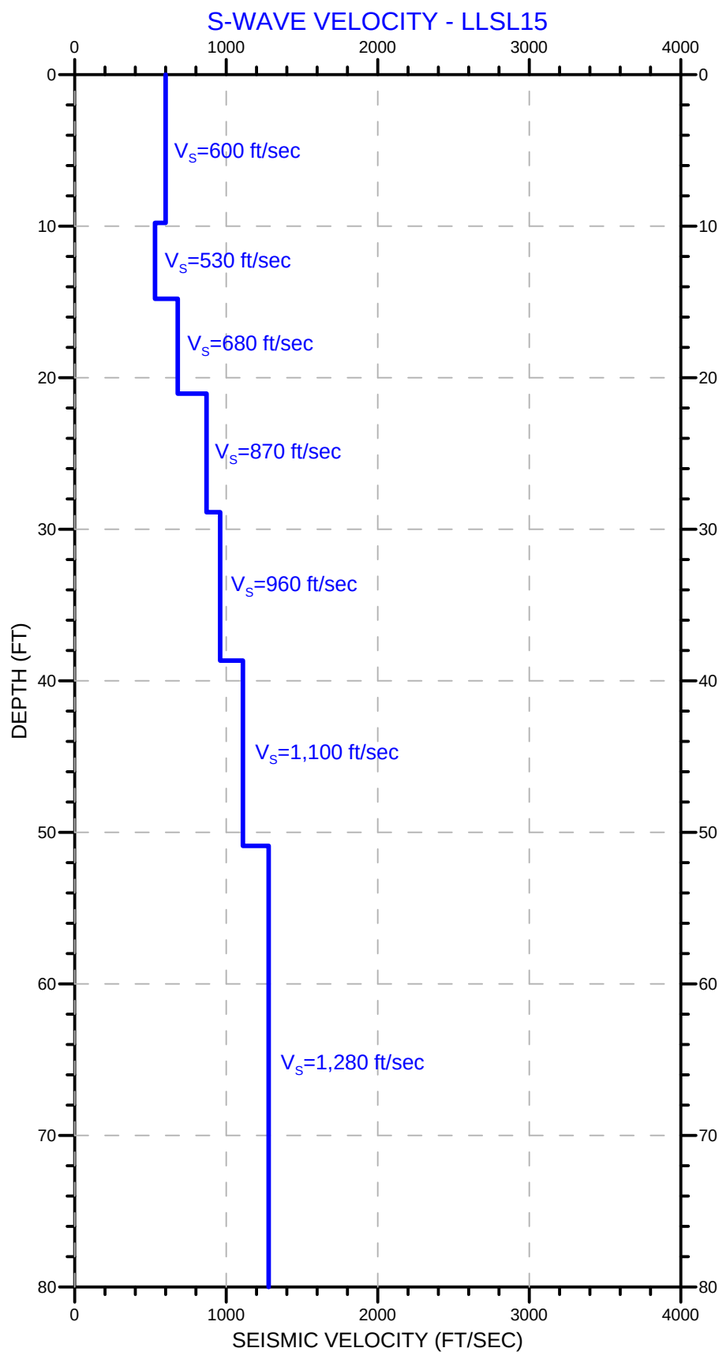
NORCAL

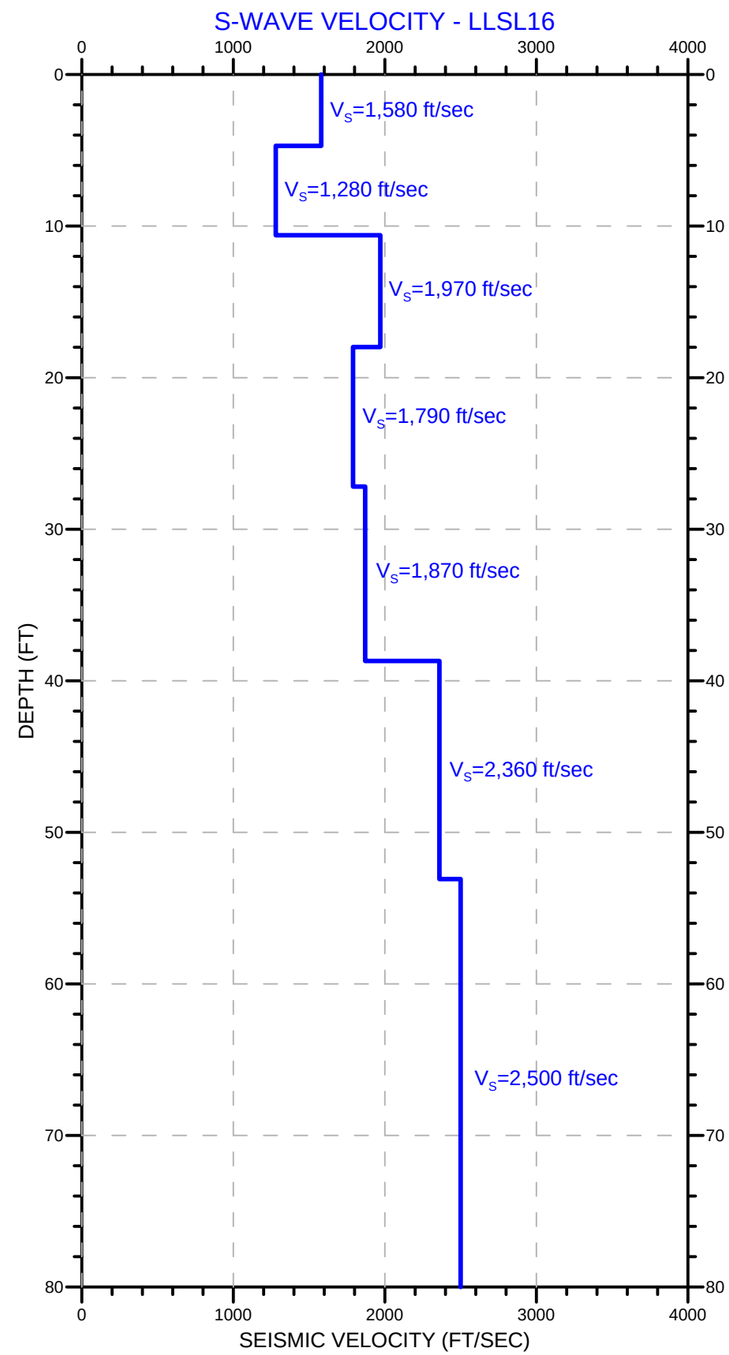
MASW 1D S-WAVE VELOCITY SOUNDINGS
LLSL13 & LLSL14
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-177.216
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE 23





LEGEND	
———	S-WAVE VELOCITY (MASW)

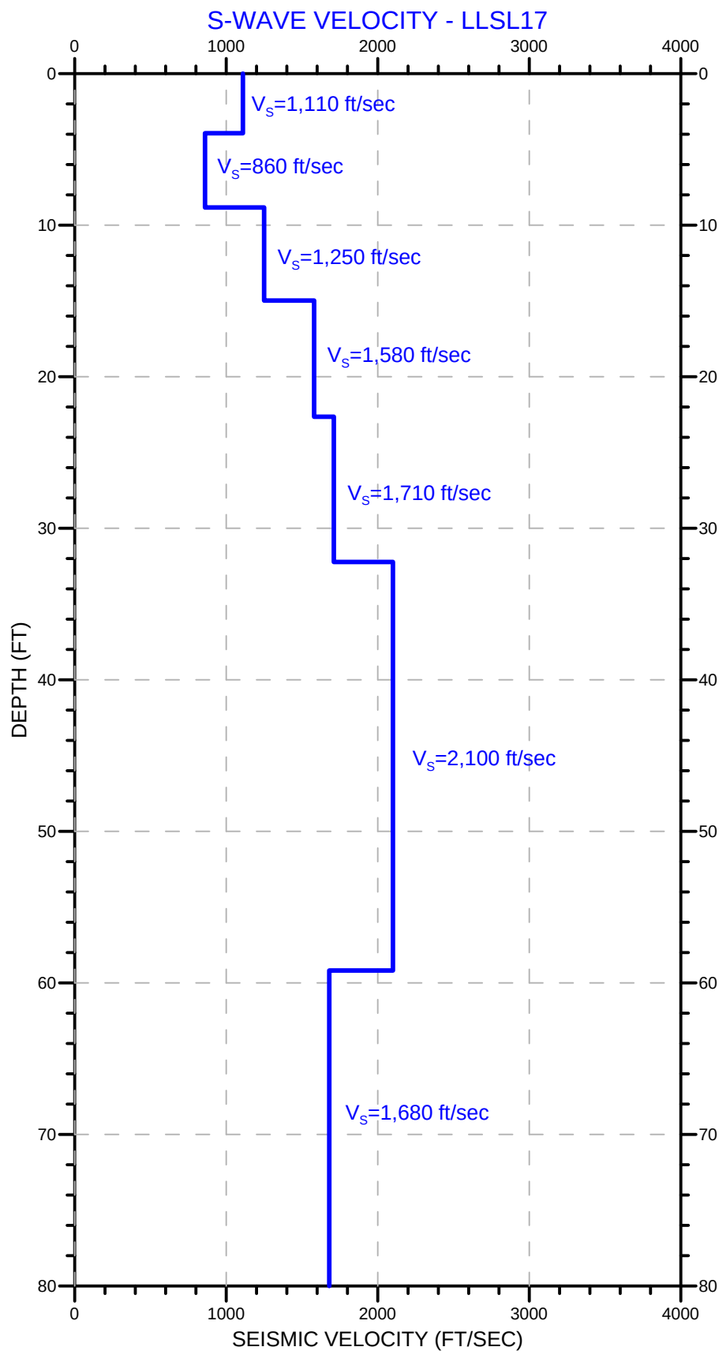
NORCAL

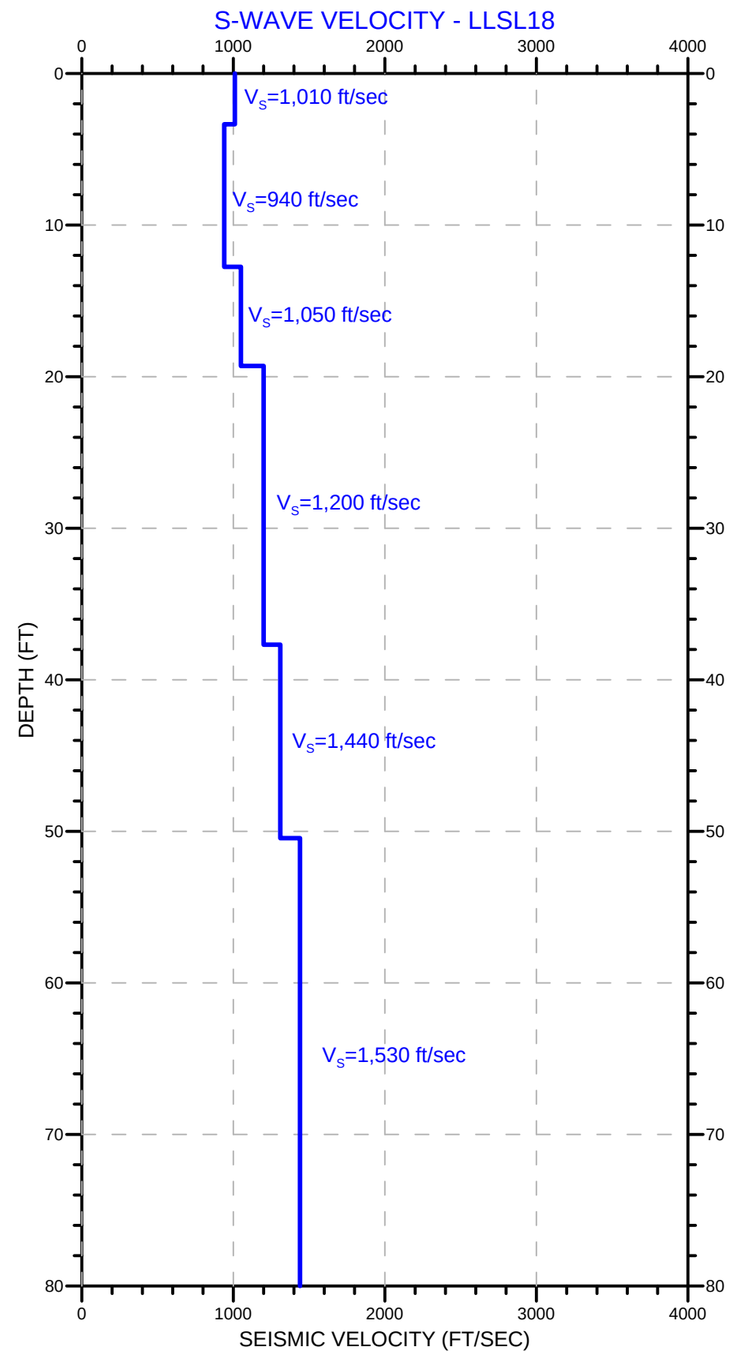
MASW 1D S-WAVE VELOCITY SOUNDINGS
LLSL15 & LLSL16
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-177.216
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE 24





LEGEND	
———	S-WAVE VELOCITY (MASW)

NORCAL

MASW 1D S-WAVE VELOCITY SOUNDINGS
LLSL17 & LLSL18
LASSEN LODGE - HIGHWAY 36

LOCATION: MINERAL, CALIFORNIA
CLIENT: KLEINFELDER
NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-177.216
DATE: JUL. 2015
DRAWN BY: G.RANDALL
APPROVED BY: DJK

PLATE 25

12.3. Laboratory Tests

DIVISION OF
ENGINEERING SERVICES
OFFICE OF GEOTECHNICAL SUPPORT
GEOTECHNICAL LABORATORY
5900 Folsom Boulevard
Sacramento, CA 95819

Date: 8/4/2015

To: Scott Lewis / GDN

From: Lilibeth C. Purta / (916) 227-5239

RE: Laboratory Test Report -- EA: 02-4G03U0
Project: 0215000056
GL 15-030

Final test results.

Note: All remaining test specimens will be disposed of in 30 calendar days from the release date of the final test results.

AASHTO R18

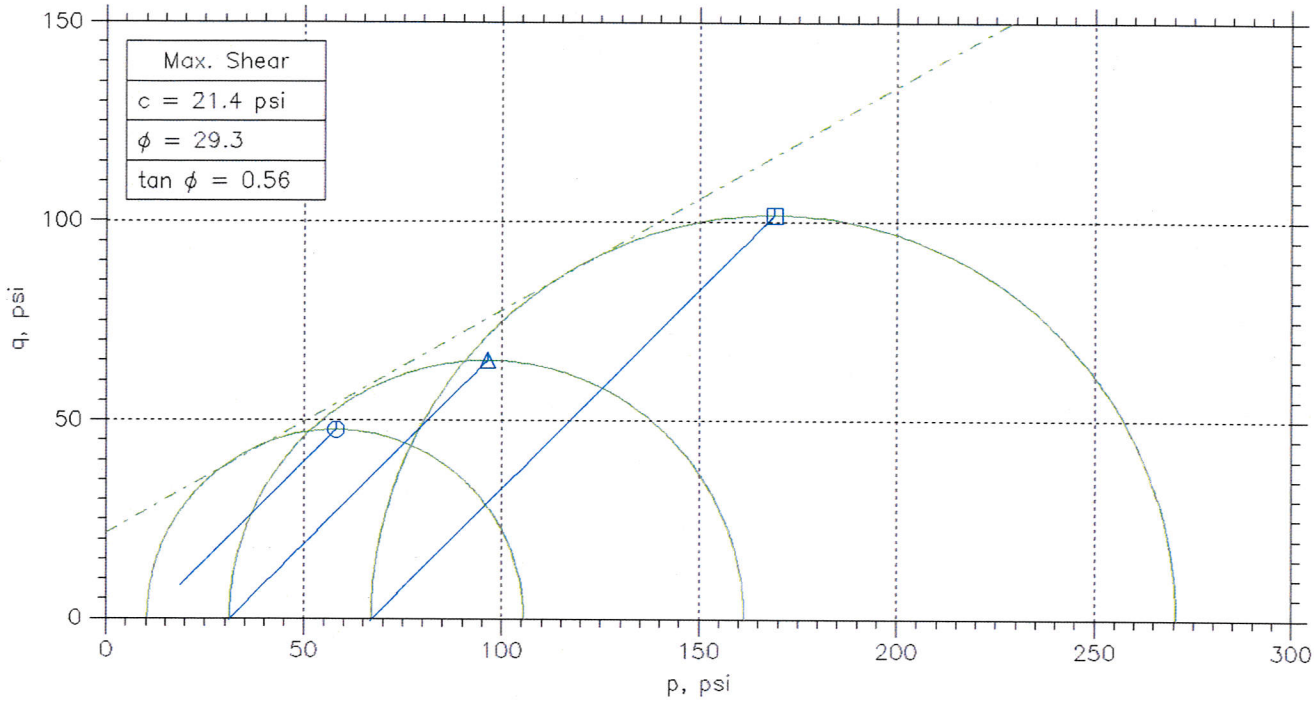
CALIFORNIA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL LABORATORY

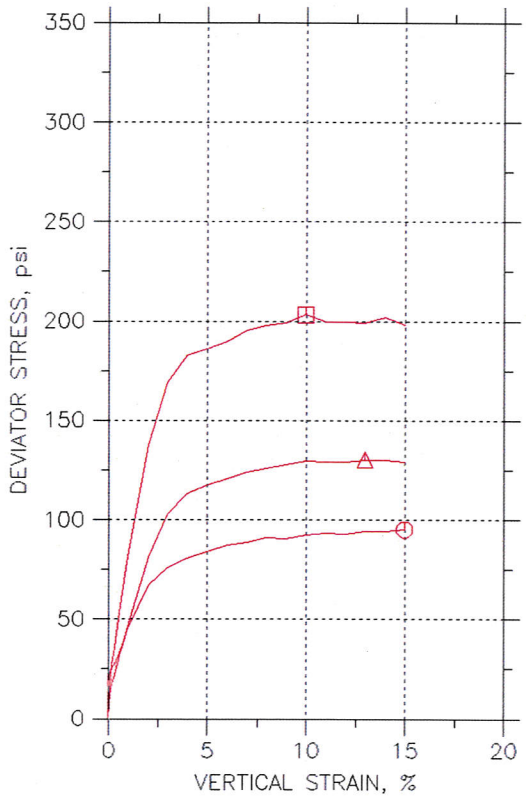
GL TRACKING NO : 15-030
Dist - EA: 02-4G03U0
Report Date: August 4, 2015
Page: 1/1

CLASSIFICATION TEST SUMMARY

SAMPLE ID	% FINER THAN																	ATTERBERG LIMITS		AS RECEIVED		Gs
	3"	2 1/2"	2"	1 1/2"	1"	3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	5μ	1μ	LL	PI	γd (pcf)	%m	
LLSS_B1		100	99	99	98	97	96	96	93	90	87	80	70	59	49	27	15	40	10			
LLSS_B1-A																						
LLSS_B1-B																						
LLSS_B1-C																						
LLSS_P1				100	98	97	94	90	78	72	63	51	37	23	14	5	3		NP			
LLSS_P1-A																						
LLSS_P1-B																						
LLSS_P1-C																						
LLSS_S1		100	99	99	97	95	91	88	77	72	65	58	49	36	26	11	6		NP			
LLSS_S1-A																						
LLSS_S1-B																						
LLSS_S1-C																						
LLSS_T1					100	99	98	98	97	94	88	78	62	49	38	16	7	49	5			2.82
LLSS_T1-A																						
LLSS_T1-B																						
LLSS_T1-C																						
LLT_1-A						100	92	87	70	62	54	47	39	32	26	8	5	38	1		22.9	
LLT_1-B																					22.8	
LLT_1-C																					23.8	
LLT_2-A						100	93	90	78	76	70	61	50	39	30	11	4		NP		31.6	
LLT_2-B																					35.7	
LLT_2-C																					38.9	

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⦶	△	⊡	
Sample No.		B1-A	B1-B	B1-C	
Test No.		C15-08A	C15-08B	C15-08C	
Depth		0.5	0.5	0.5	
Initial	Diameter, in	1.942	1.941	1.944	
	Height, in	4.117	4.096	4.05	
	Water Content, %	22.2	22.3	22.4	
	Dry Density, pcf	96.87	97.97	98.67	
	Saturation, %	79.2	81.5	83.1	
	Void Ratio	0.772	0.752	0.74	
Before Shear	Water Content, %	28.0	28.3	27.1	
	Dry Density, pcf	97.01	96.5	98.29	
	Saturation*, %	100.0	100.0	99.9	
	Void Ratio	0.77	0.779	0.747	
	Back Press., psi	101.	101.	100.	
Ver. Eff. Cons. Stress, psi		10.42	31.25	67.58	
Shear Strength, psi		47.63	65.1	101.7	
Strain at Failure, %		15	13	10	
Strain Rate, %/min		0.1	0.1	0.1	
B-Value		0.91	0.95	0.93	
Implied Specific Gravity		2.75	2.75	2.75	
Liquid Limit		---	---	---	
Plastic Limit		---	---	---	

Caltrans

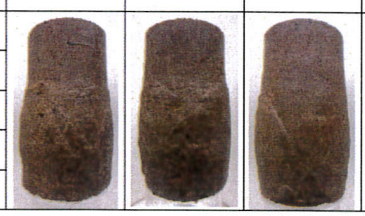
Project: Lassen Lodge

Location: 02-TEH-36-75.4-78.4

Project No.: 02-4G03U0

Boring No.: LLSS

Sample Type: REMOLD



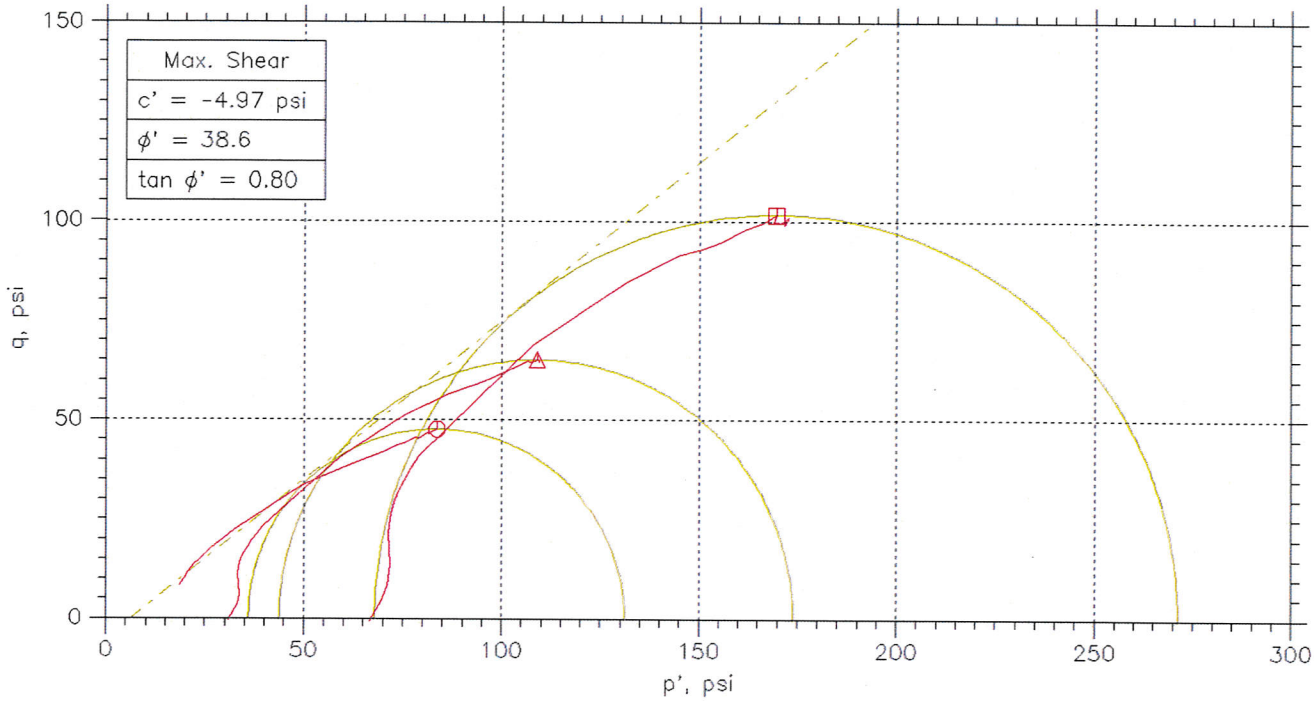
Description: Moist, Stiff, Reddish Brown, Silt w/Few Clay/Sand to 93%RC (94.49 PCF @22.1%MC).

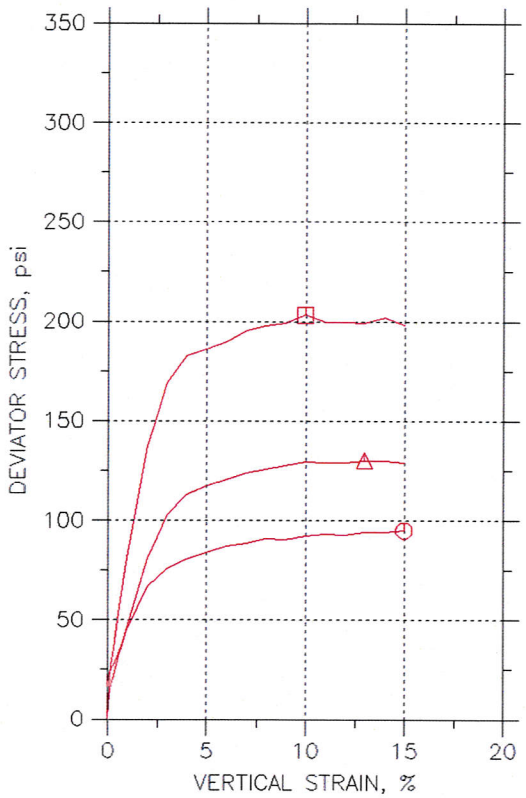
Remarks: TRIAX#8. Desc is not a Classification. Bulge deformation w/shear bands. B-Value<0.95

Phase calculations based on start and end of test.

7/30/15

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⦶	△	⊡	
Sample No.		B1-A	B1-B	B1-C	
Test No.		C15-08A	C15-08B	C15-08C	
Depth		0.5	0.5	0.5	
Initial	Diameter, in	1.942	1.941	1.944	
	Height, in	4.117	4.096	4.05	
	Water Content, %	22.2	22.3	22.4	
	Dry Density, pcf	96.87	97.97	98.67	
	Saturation, %	79.2	81.5	83.1	
	Void Ratio	0.772	0.752	0.74	
Before Shear	Water Content, %	28.0	28.3	27.1	
	Dry Density, pcf	97.01	96.5	98.29	
	Saturation*, %	100.0	100.0	99.9	
	Void Ratio	0.77	0.779	0.747	
	Back Press., psi	101.	101.	100.	
Ver. Eff. Cons. Stress, psi		10.42	31.25	67.58	
Shear Strength, psi		47.63	65.1	101.7	
Strain at Failure, %		15	13	10	
Strain Rate, %/min		0.1	0.1	0.1	
B-Value		0.91	0.95	0.93	
Implied Specific Gravity		2.75	2.75	2.75	
Liquid Limit		---	---	---	
Plastic Limit		---	---	---	

Caltrans

Project: Lassen Lodge

Location: 02-TEH-36-75.4-78.4

Project No.: 02-4G03U0

Boring No.: LLSS

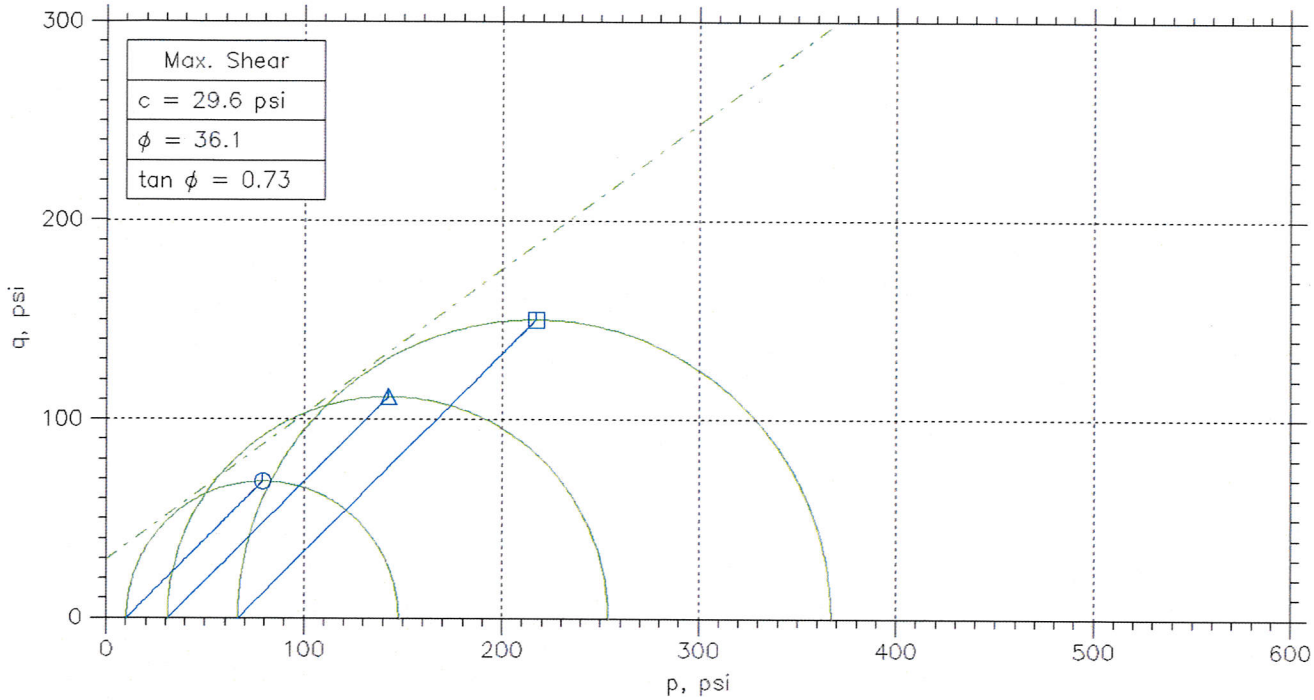
Sample Type: REMOLD

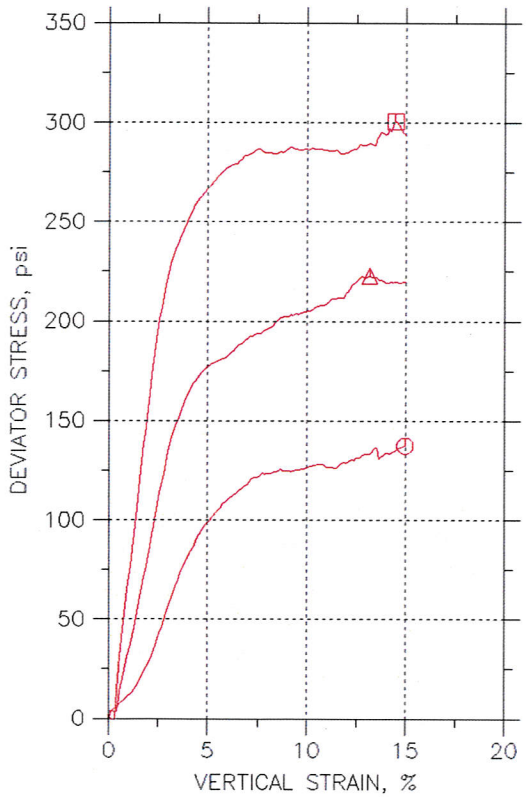
Description: Moist, Stiff, Reddish Brown, Silt w/Few Clay/Sand to 93%RC (94.49 PCF @22.1%MC).

Remarks: TRIAX#8. Desc is not a Classification. Bulge deformation w/shear bands. B-Value<0.95

Phase calculations based on start and end of test.

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⦶	△	⊡	
Sample No.		P1-A	P1-B	P1-C	
Test No.		C15-09A	C15-09B	C15-09C	
Depth		0.5'	0.5'	0.5'	
Initial	Diameter, in	1.947	1.948	1.949	
	Height, in	4.13	4.095	4.073	
	Water Content, %	10.5	10.3	10.4	
	Dry Density, pcf	106.6	107.5	107.8	
	Saturation, %	47.5	47.6	48.3	
	Void Ratio	0.61	0.597	0.592	
Before Shear	Water Content, %	20.6	20.2	19.5	
	Dry Density, pcf	109.7	110.4	111.8	
	Saturation*, %	100.0	100.0	100.0	
	Void Ratio	0.566	0.556	0.535	
	Back Press., psi	99.	101.	101.	
Ver. Eff. Cons. Stress, psi		10.41	31.25	66.66	
Shear Strength, psi		68.79	111.4	150.3	
Strain at Failure, %		15	13.2	14.5	
Strain Rate, %/min		0.1	0.1	0.1	
B-Value		0.78	0.83	0.87	
Implied Specific Gravity		2.75	2.75	2.75	
Liquid Limit		---	---	---	
Plastic Limit		---	---	---	

Caltrans

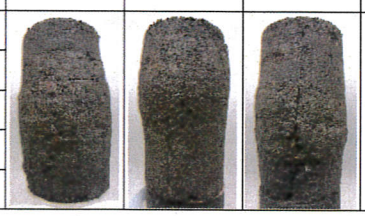
Project: Lassen Lodge

Location: 02-TEH-36-75.4-78.4

Project No.: 02-4G03U0

Boring No.: LLSS

Sample Type: REMOLD



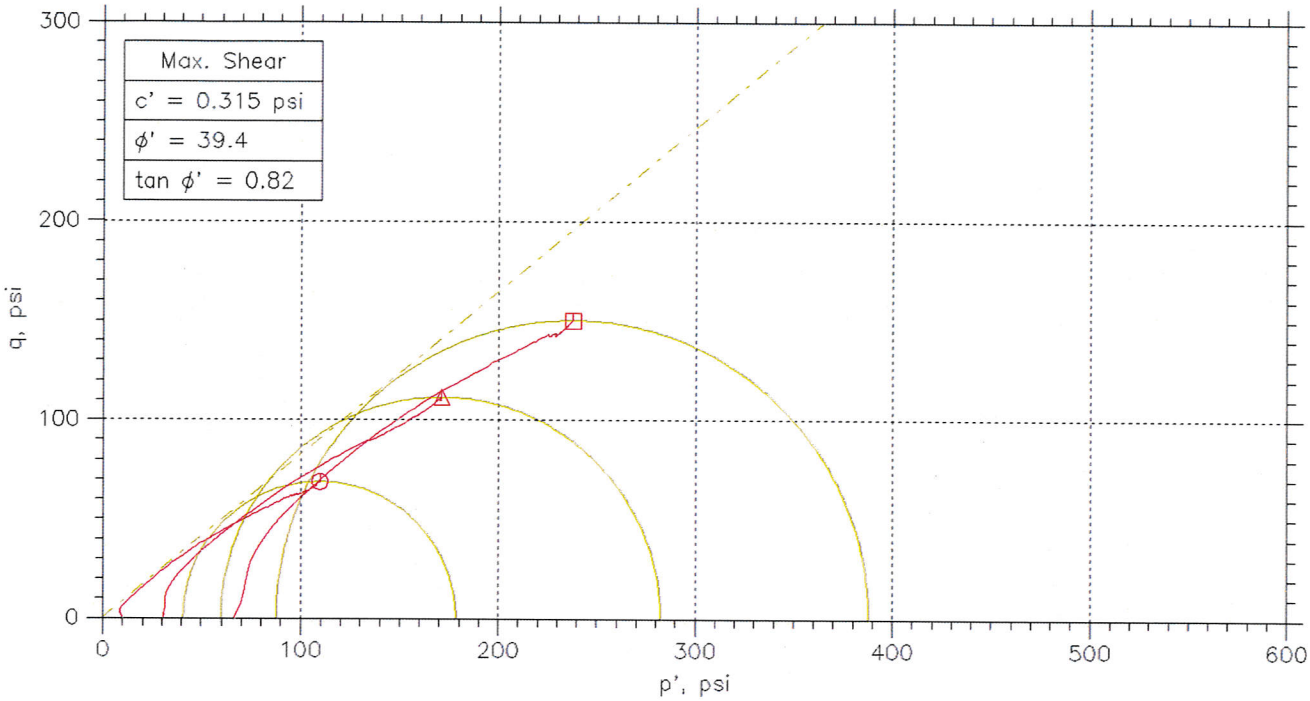
Description: Moist, Dense, Brown, Silty Sand w/Organics to 93%RC (104.25PCF @10.50%MC)

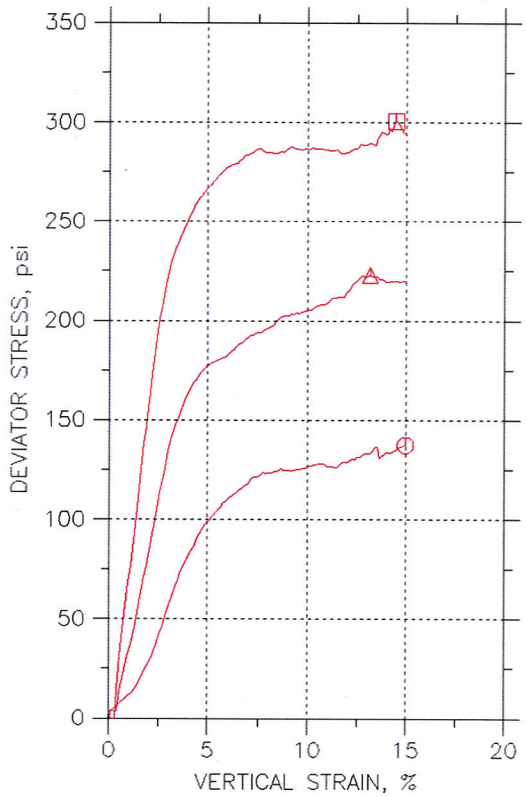
Remarks: Triax#7. Desc is not a Classification. Bulge deformation w/light Shear Bands. B-Value<0.95

Phase calculations based on start and end of test.

8/7/15

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⦶	△	⊡	
Sample No.		P1-A	P1-B	P1-C	
Test No.		C15-09A	C15-09B	C15-09C	
Depth		0.5'	0.5'	0.5'	
Initial	Diameter, in	1.947	1.948	1.949	
	Height, in	4.13	4.095	4.073	
	Water Content, %	10.5	10.3	10.4	
	Dry Density, pcf	106.6	107.5	107.8	
	Saturation, %	47.5	47.6	48.3	
	Void Ratio	0.61	0.597	0.592	
Before Shear	Water Content, %	20.6	20.2	19.5	
	Dry Density, pcf	109.7	110.4	111.8	
	Saturation*, %	100.0	100.0	100.0	
	Void Ratio	0.566	0.556	0.535	
	Back Press., psi	99.	101.	101.	
Ver. Eff. Cons. Stress, psi		10.41	31.25	66.66	
Shear Strength, psi		68.79	111.4	150.3	
Strain at Failure, %		15	13.2	14.5	
Strain Rate, %/min		0.1	0.1	0.1	
B-Value		0.78	0.83	0.87	
Implied Specific Gravity		2.75	2.75	2.75	
Liquid Limit		---	---	---	
Plastic Limit		---	---	---	

Caltrans

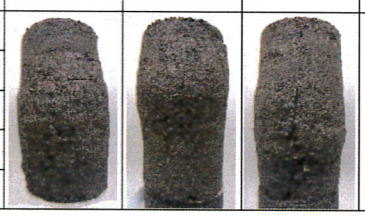
Project: Lassen Lodge

Location: 02-TEH-36-75.4-78.4

Project No.: 02-4G03U0

Boring No.: LLSS

Sample Type: REMOLD



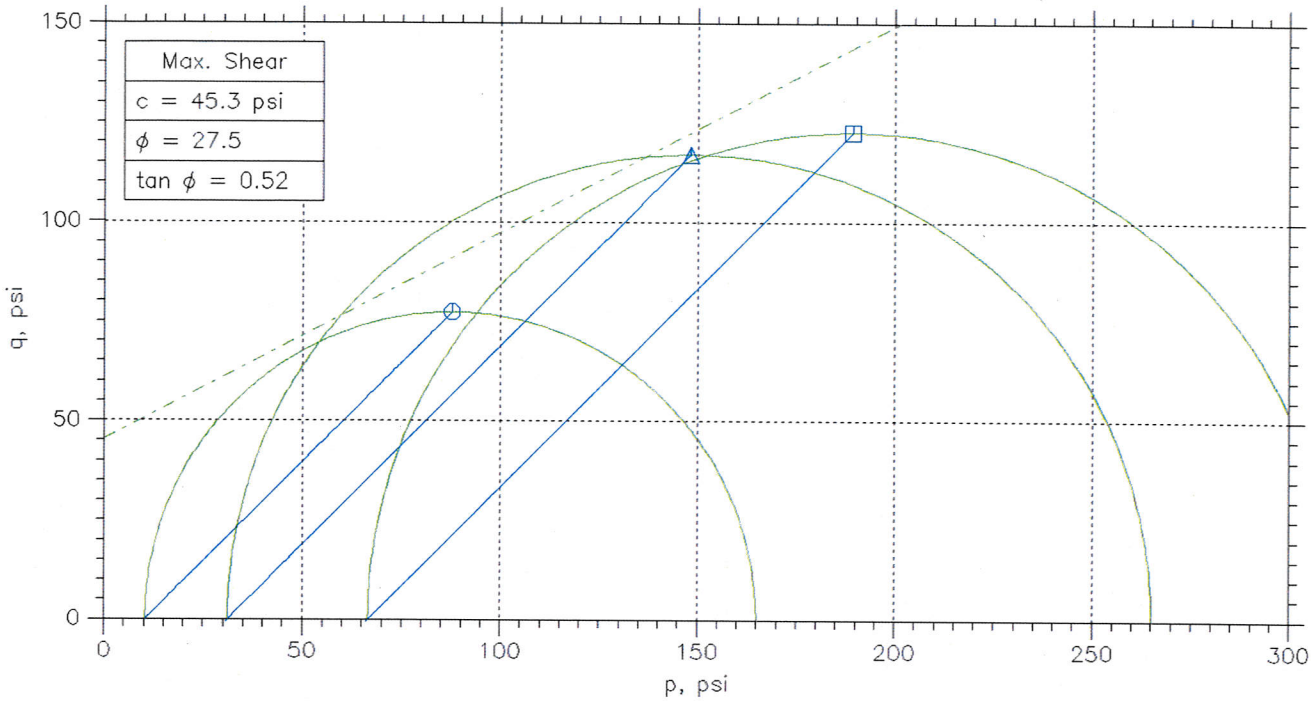
Description: Moist, Dense, Brown, Silty Sand w/Organics to 93%RC (104.25PCF @10.50%MC)

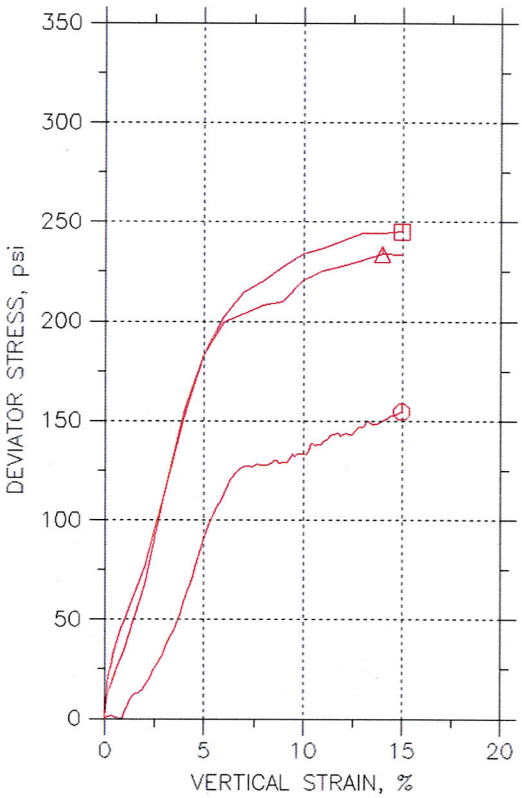
Remarks: Triax#7. Desc is not a Classification. Bulge deformation w/light Shear Bands. B-Value<0.95

Phase calculations based on start and end of test.

8/3/15

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⦶	△	⊡	
Sample No.		S1-A	S1-B	S1-C	
Test No.		C15-10A	C15-10B	C15-10C	
Depth		0.5'	0.5	0.5	
Initial	Diameter, in	1.937	1.935	1.927	
	Height, in	4.248	4.231	4.31	
	Water Content, %	12.1	11.5	11.8	
	Dry Density, pcf	113.1	113.1	112.	
	Saturation, %	64.2	61.1	61.1	
	Void Ratio	0.517	0.517	0.533	
Before Shear	Water Content, %	18.6	19.8	16.7	
	Dry Density, pcf	113.5	111.2	117.6	
	Saturation*, %	100.0	100.0	100.0	
	Void Ratio	0.513	0.544	0.46	
	Back Press., psi	101.	101.	101.	
Ver. Eff. Cons. Stress, psi		10.42	31.25	66.66	
Shear Strength, psi		77.32	117.	122.6	
Strain at Failure, %		15	14	15	
Strain Rate, %/min		0.1	0.1	0.1	
B-Value		0.93	0.94	0.94	
Implied Specific Gravity		2.75	2.75	2.75	
Liquid Limit		---	---	---	
Plastic Limit		---	---	---	

Caltrans

Project: Lassen Lodge

Location: 02-TEH-36-75.4-78.4

Project No.: 02-4G03U0

Boring No.: LLSS

Sample Type: REMOLD

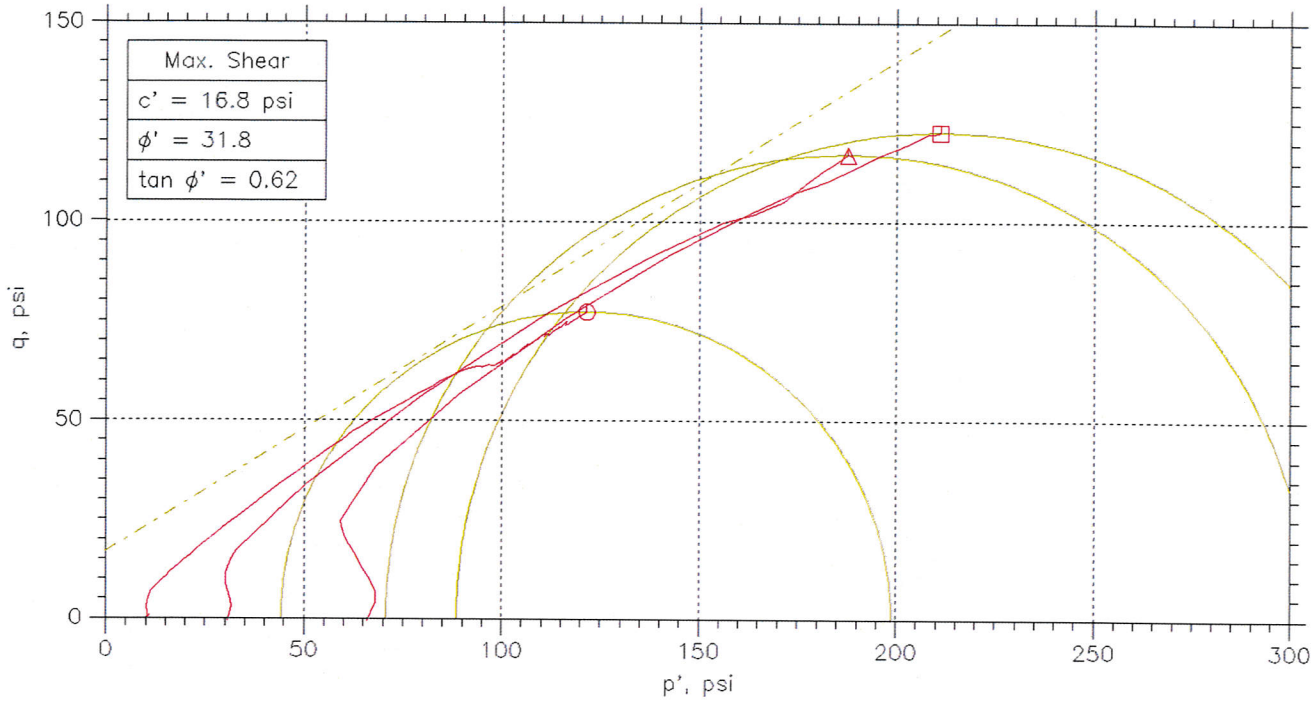
Description: Moist, Stiff, Brown, Silt w/Few Sand & Organics to 93%RC (112.34PCF @12.1%MC)

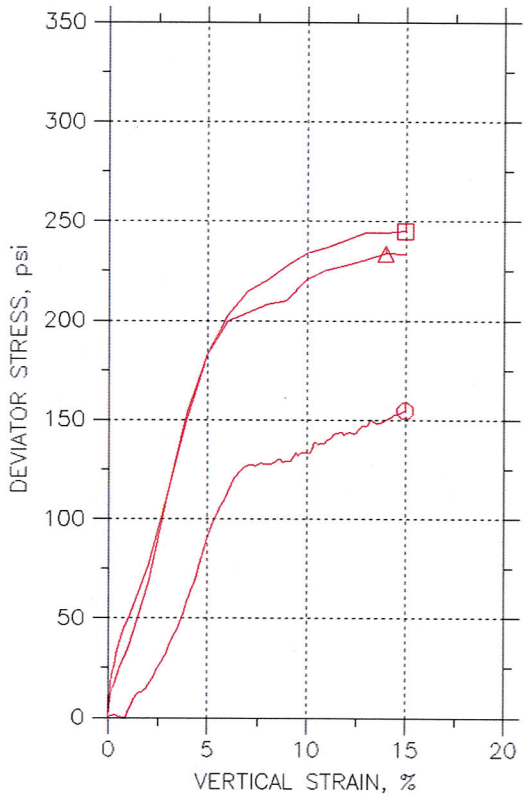
Remarks: Triax#7. Desc is not a Classification. Shear deformation, some bulge. B-Value < 0.95

8/3/15

Phase calculations based on start and end of test.

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⦶	△	⊡	
Sample No.		S1-A	S1-B	S1-C	
Test No.		C15-10A	C15-10B	C15-10C	
Depth		0.5'	0.5	0.5	
Initial	Diameter, in	1.937	1.935	1.927	
	Height, in	4.248	4.231	4.31	
	Water Content, %	12.1	11.5	11.8	
	Dry Density, pcf	113.1	113.1	112.	
	Saturation, %	64.2	61.1	61.1	
	Void Ratio	0.517	0.517	0.533	
Before Shear	Water Content, %	18.6	19.8	16.7	
	Dry Density, pcf	113.5	111.2	117.6	
	Saturation*, %	100.0	100.0	100.0	
	Void Ratio	0.513	0.544	0.46	
	Back Press., psi	101.	101.	101.	
Ver. Eff. Cons. Stress, psi		10.42	31.25	66.66	
Shear Strength, psi		77.32	117.	122.6	
Strain at Failure, %		15	14	15	
Strain Rate, %/min		0.1	0.1	0.1	
B-Value		0.93	0.94	0.94	
Implied Specific Gravity		2.75	2.75	2.75	
Liquid Limit		---	---	---	
Plastic Limit		---	---	---	

Caltrans

Project: Lassen Lodge

Location: 02-TEH-36-75.4-78.4

Project No.: 02-4G03U0

Boring No.: LLSS

Sample Type: REMOLD

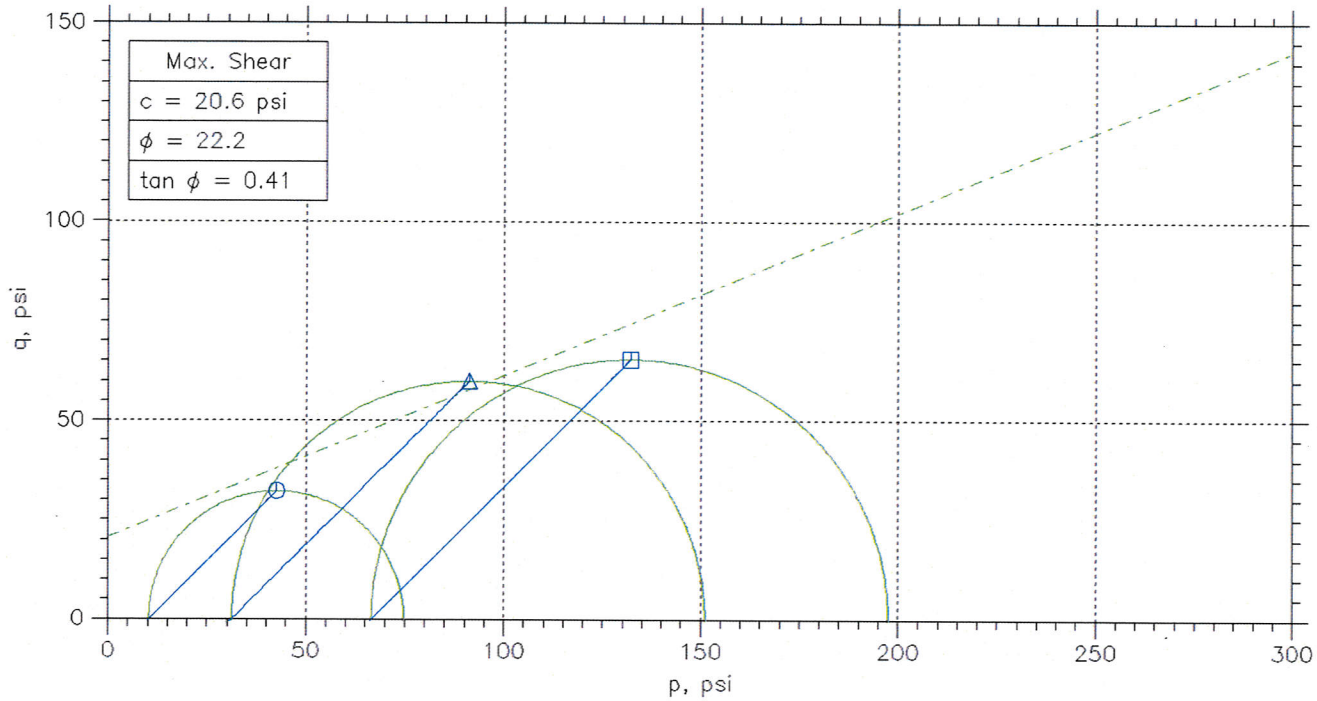
Description: Moist, Stiff, Brown, Silt w/Few Sand & Organics to 93%RC (112.34PCF @12.1%MC)

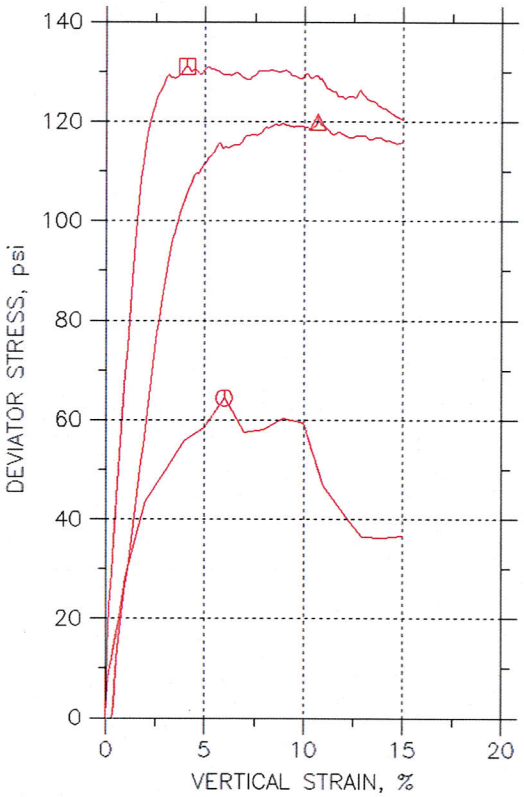
Remarks: Triax#7. Desc is not a Classification. Shear deformation, some bulge. B-Value < 0.95

Phase calculations based on start and end of test.

8/31/15

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⦶	△	⊡	
Sample No.		T1-A	T1-B	T1-C	
Test No.		C15-11A	C15-11B	C15-11C	
Depth		0.5	0.5'	0.5'	
Initial	Diameter, in	1.937	1.942	1.938	
	Height, in	4.085	4.034	4.092	
	Water Content, %	29.1	29.1	28.7	
	Dry Density, pcf	82.18	82.74	82.43	
	Saturation, %	71.8	72.8	71.3	
	Void Ratio	1.14	1.13	1.13	
Before Shear	Water Content, %	41.6	39.2	38.7	
	Dry Density, pcf	81.58	83.64	84.13	
	Saturation*, %	100.0	100.0	100.0	
	Void Ratio	1.16	1.1	1.09	
	Back Press., psi	101.	101.	101.	
Ver. Eff. Cons. Stress, psi		10.42	31.24	66.66	
Shear Strength, psi		32.23	59.97	65.48	
Strain at Failure, %		6	10.7	4.1	
Strain Rate, %/min		0.1	0.1	0.1	
B-Value		0.96	0.88	0.87	
Measured Specific Gravity		2.82	2.82	2.82	
Liquid Limit		---	---	---	
Plastic Limit		---	---	---	

Caltrans

Project: Lassen Lodge

Location: 02-TEH-36-75.4-78.4

Project No.: 02-4G03U0

Boring No.: LLSS

Sample Type: REMOLD

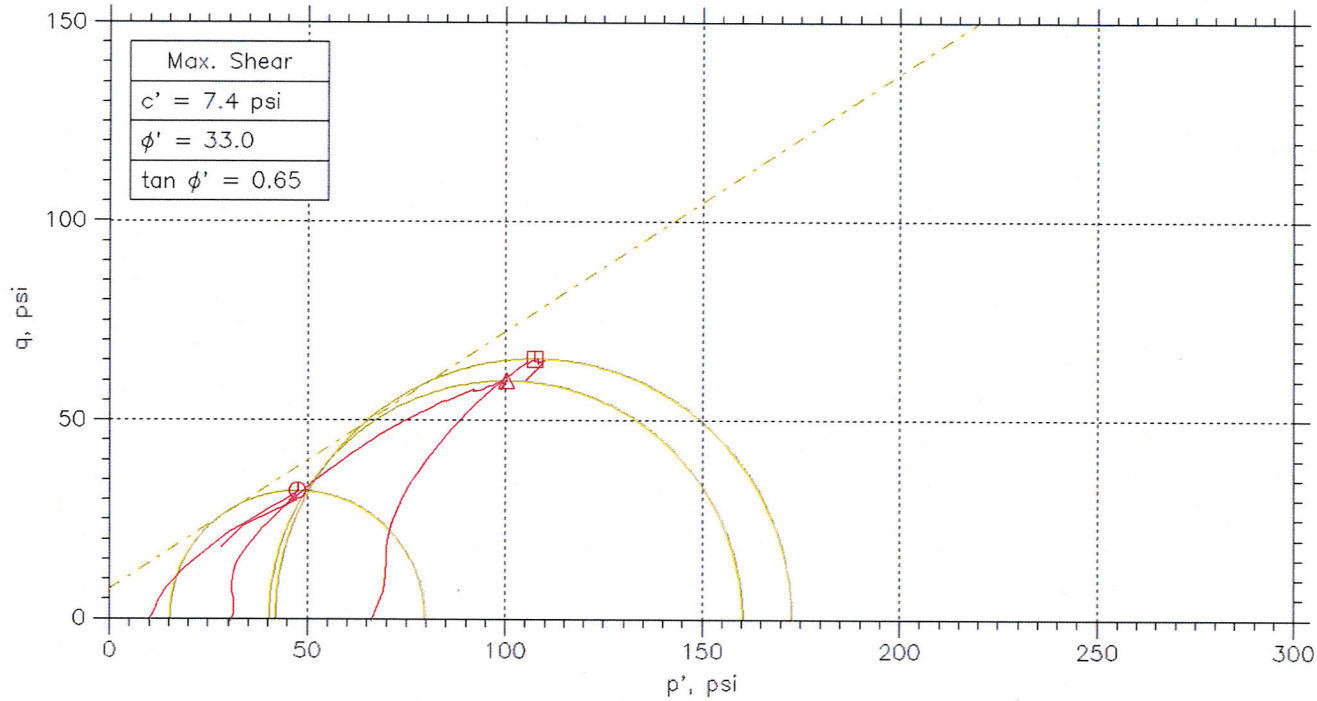
Description: Moist, Very Stiff, Reddish-Brown, Silt w/Clay, Few Sand, to 93%RC 79.42PCF @28.10%MC

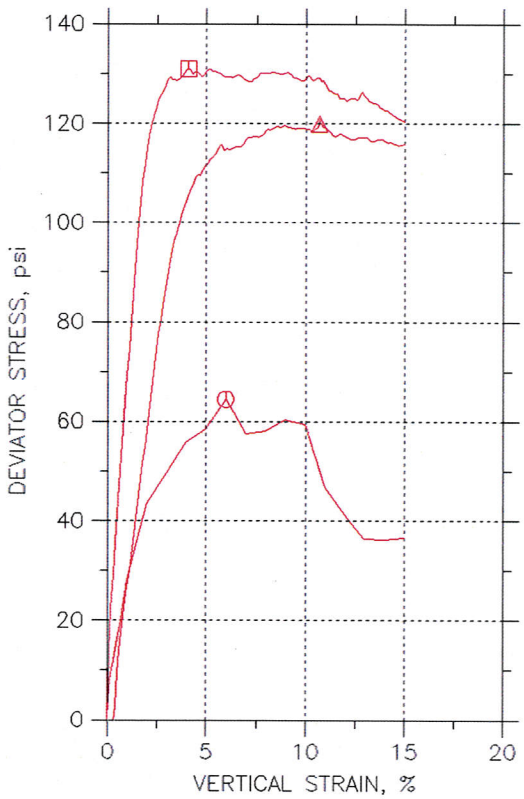
Remarks: TRIAX#8. Description is not a Classification. B-Value < 0.95. Shear Deformation w/Bulge.

Phase calculations based on start and end of test.

8/12/15

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		◯	△	□	
Sample No.		T1-A	T1-B	T1-C	
Test No.		C15-11A	C15-11B	C15-11C	
Depth		0.5	0.5'	0.5'	
Initial	Diameter, in	1.937	1.942	1.938	
	Height, in	4.085	4.034	4.092	
	Water Content, %	29.1	29.1	28.7	
	Dry Density, pcf	82.18	82.74	82.43	
	Saturation, %	71.8	72.8	71.3	
	Void Ratio	1.14	1.13	1.13	
Before Shear	Water Content, %	41.6	39.2	38.7	
	Dry Density, pcf	81.58	83.64	84.13	
	Saturation*, %	100.0	100.0	100.0	
	Void Ratio	1.16	1.1	1.09	
	Back Press., psi	101.	101.	101.	
Ver. Eff. Cons. Stress, psi		10.42	31.24	66.66	
Shear Strength, psi		32.23	59.97	65.48	
Strain at Failure, %		6	10.7	4.1	
Strain Rate, %/min		0.1	0.1	0.1	
B-Value		0.96	0.88	0.87	
Measured Specific Gravity		2.82	2.82	2.82	
Liquid Limit		---	---	---	
Plastic Limit		---	---	---	

Project: Lassen Lodge

Location: 02-TEH-36-75.4-78.4

Project No.: 02-4G03U0

Boring No.: LLSS

Sample Type: REMOLD

Description: Moist, Very Stiff, Reddish-Brown, Silt w/Clay, Few Sand, to 93%RC 79.42PCF @28.10%MC

Remarks: TRIAX#8. Description is not a Classification. B-Value < 0.95. Shear Deformation w/Bulge.

Phase calculations based on start and end of test.

2/2/15

Division of Engineering Services
Geotechnical Laboratory

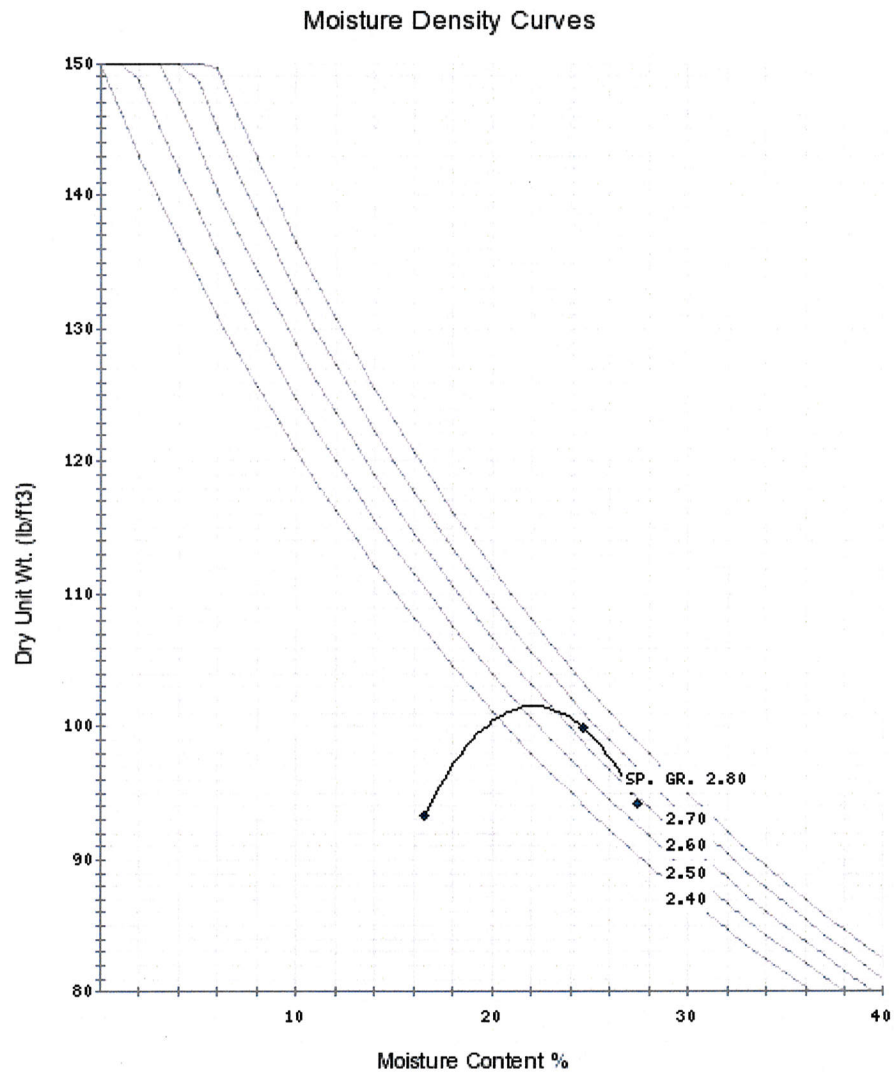
Compaction Curve

CTM 216

Dist-EA: 02-4G03U0
Dist-Co-Rte-PM: TEH-36-75.4/78.4
Sample ID: LLSS_B1
GI Tracking No.: 15-030

Maximum Dry Density: 101.6 pcf
Optimum Moisture: 22.1 %
Moisture (as Received:) 10.0 %
Approved: June 25, 2015

Trial No.	Moisture Adjustment	Tamper Reading	Wet + Tare Weight (g)	Dry + Tare Weight (g)	Tare (g)	Moisture Content (%)	Dry Unit Weight (pcf)
1	200	11.90	3011	2699	811	16.5	93.3
2	350	11.10	3126	2662	776	24.6	99.9
3	400	11.75	3140	2624	743	27.4	94.2
4							
5							
6							



Soil Description : MOIST CLAY WITH SILT AND GRAVEL

Remarks:

Division of Engineering Services
Geotechnical Laboratory

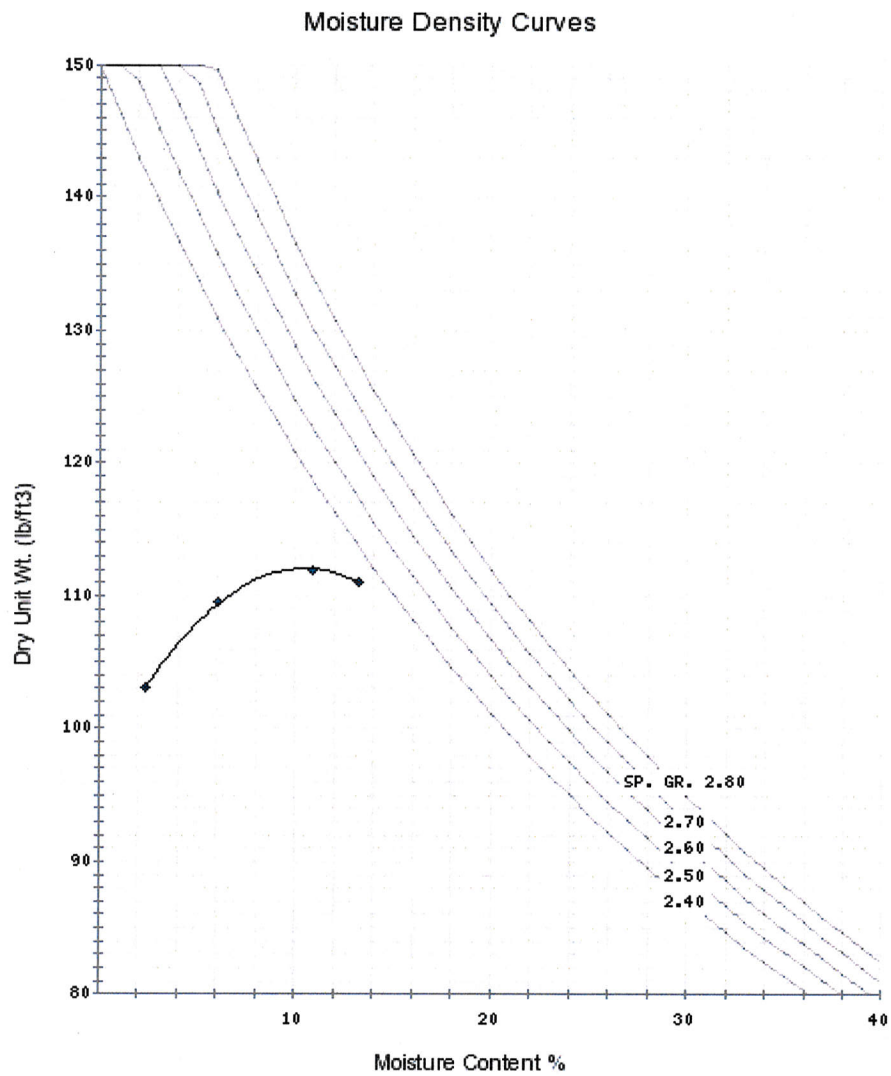
Compaction Curve

CTM 216

Dist-EA: 02-4G03U0
Dist-Co-Rte-PM: TEH-36-75.4/78.4
Sample ID: LLSS_P1
GI Tracking No.: 15-030

Maximum Dry Density: 112.1 pcf
Optimum Moisture: 10.5 %
Moisture (as Received:) 5.5 %
Approved: June 26, 2015

Trial No.	Moisture Adjustment	Tamper Reading	Wet + Tare Weight (g)	Dry + Tare Weight (g)	Tare (g)	Moisture Content (%)	Dry Unit Weight (pcf)
1	25	11.85	2870	2820	744	2.4	103.1
2	100	11.15	2943	2817	742	6.1	109.5
3	200	10.90	3047	2821	746	10.9	112.0
4	250	11.00	3097	2821	745	13.3	111.0
5							
6							



Soil Description : SILT WITH GRAVEL

Remarks:

Division of Engineering Services
Geotechnical Laboratory

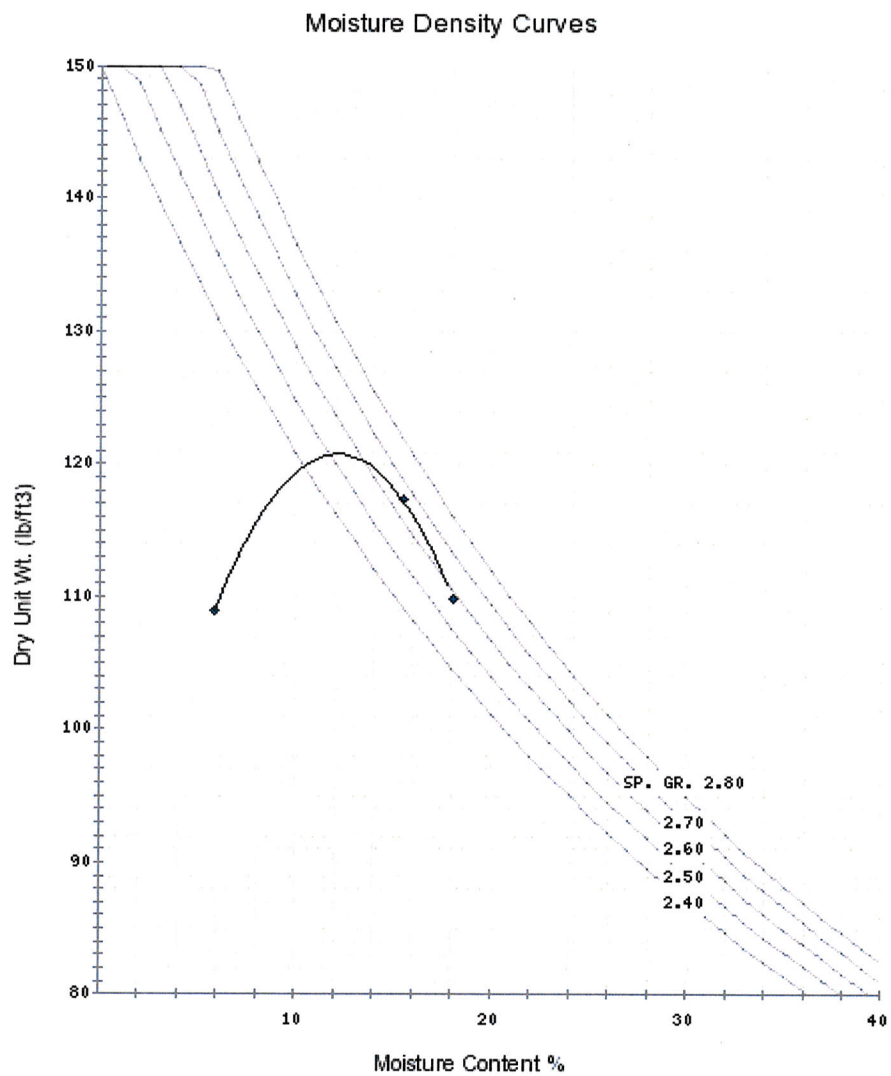
Compaction Curve

CTM 216

Dist-EA: 02-4G03U0
Dist-Co-Rte-PM: TEH-36-75.4/78.4
Sample ID: LLSS_S1
GI Tracking No.: 15-030

Maximum Dry Density: 120.8 pcf
Optimum Moisture: 12.1 %
Moisture (as Received:) 2.6 %
Approved: June 29, 2015

Trial No.	Moisture Adjustment	Tamper Reading	Wet + Tare Weight (g)	Dry + Tare Weight (g)	Tare (g)	Moisture Content (%)	Dry Unit Weight (pcf)
1	100	11.20	2944	2822	748	5.9	108.9
2	300	10.40	3140	2819	745	15.5	117.3
3	350	11.10	3247	2872	799	18.1	109.9
4							
5							
6							



Soil Description : GREY SILT SAND AND GRAVEL

Remarks:

Division of Engineering Services
Geotechnical Laboratory

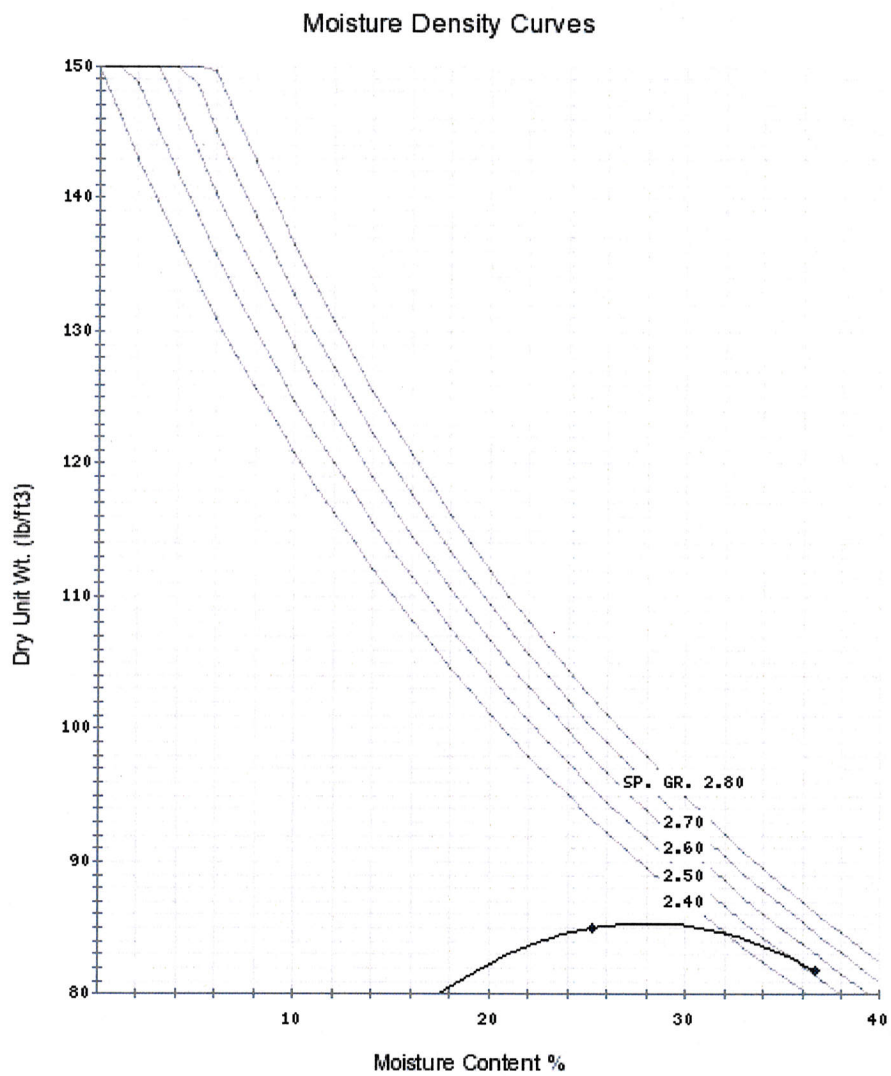
Compaction Curve

CTM 216

Dist-EA: 02-4G03U0
Dist-Co-Rte-PM: TEH-36-75.4/78.4
Sample ID: LLSS_T1
GI Tracking No.: 15-030

Maximum Dry Density: 85.4 pcf
Optimum Moisture: 28.1 %
Moisture (as Received:) 16.4 %
Approved: July 1, 2015

Trial No.	Moisture Adjustment	Tamper Reading	Wet + Tare Weight (g)	Dry + Tare Weight (g)	Tare (g)	Moisture Content (%)	Dry Unit Weight (pcf)
1	150	13.00	2793	2498	744	16.8	79.4
2	300	12.15	2943	2499	743	25.3	85.0
3	500	12.60	3140	2497	744	36.7	81.8
4							
5							
6							



Soil Description : BROWN SILTY CLAY WITH SAND AND GRAVEL

Remarks:

Results sent to: SCOTT LEWIS

Division of Engineering Services
Materials Engineering and Testing Services
Corrosion and Structural Concrete Field Investigation Branch

Report Date: 6/15/2015
Reported by Michael Mifkovic

CORROSION TEST SUMMARY REPORT - SOIL

EFIS: **0215000056**

Dist/Co/Rte/PM **02 / TEH /036/ / 75.4-78.4 PM**

CORROSION LAB #	TL101 #	BORE #	DEPTH (FT) START	DEPTH (FT) END	MINIMUM RESISTIVITY[1] (ohm-cm)	pH[1]	CHLORIDE CONTENT[2] (ppm)	SULFATE CONTENT[3] (ppm)	IS SAMPLE CORROSIVE?
SOIL SAMPLE FROM: CUT SLOPE									
CR20150172	**C734205**		0	0.5	9204	5.82			NO
CR20150173	**C734204**		0	0.5	10438	6.35			NO
CR20150174	**C734203**		0	0.5	12936	5.82			NO
CR20150175	**C734206**		0	0.5	9868	6.67			NO

This site is not corrosive to foundation elements (see note below).

Note: For Structural Elements, the Department considers a site corrosive if one or more of the following conditions exist: pH is 5.5 or less, chloride concentration is 500 ppm or greater, sulfate concentration is 2000 ppm or greater. Resistivity is not considered for Structural Elements. MSE backfill shall conform to the requirements of section 47-2.02C Structure Backfill in the 2010 Standard Specifications.

[1]CT 643, [2]CT 422, [3]CT 417

INFORMATION HANDOUT

For Contract No. 02-4G03u4
At Teh-36-R75.1/78.4

Identified by
Project ID 0215000056

MATERIALS INFORMATION

Water Source Information
Non-Potable information dated May 3, 2016
Potable information dated May 4, 2016

Water Conservation, Non-Potable-Potential Sources

Due to water shortages, water conservation efforts are in place for this project. The use of non-potable or recycled water will be required for this project. Non-potable water sources in this area are limited to open bodies of water such as nearby Battle Creek, Mill Creek and Lake Almanor which are within 50 miles from the project and the following:

Sierra Pacific Industries
Forestry Division
3950 Carson Road
Camino, CA 95709
(530) 644-2311

The contractor will need to obtain any necessary permits or agreements as needed. If the sources listed above cannot be ~~reasonability obtain~~, then this should be documented and submitted for an exception.

reasonably obtained

Water Conservation, Potable-Potential Sources

It is the responsibility of the contractor to acquire an adequate and reliable source of water for construction activities for this project. Potential sources of water near the project location are included in, but not limited to, the following list.

Chester Public Utility District

(530) 258-2171

Red Bluff Public Works - Water Department

(530) 527-4300 ext. 2

INFORMATION HANDOUT

For Contract No. 02-4G03u4
At Teh-36-R75.1/78.4

Identified by
Project ID 0215000056

MATERIALS INFORMATION

Site Investigation Report of Soils Summary

Dated February 2016

SITE INVESTIGATION REPORT

State Route 36 Realignment Project
Post Mile 75.4 to 78.4
Tehama County, California

PREPARED FOR:

CALIFORNIA DEPARTMENT OF TRANSPORTATION – DISTRICT 3
ENVIRONMENTAL ENGINEERING OFFICE
703 B STREET
MARYSVILLE, CALIFORNIA 95901

Caltrans®

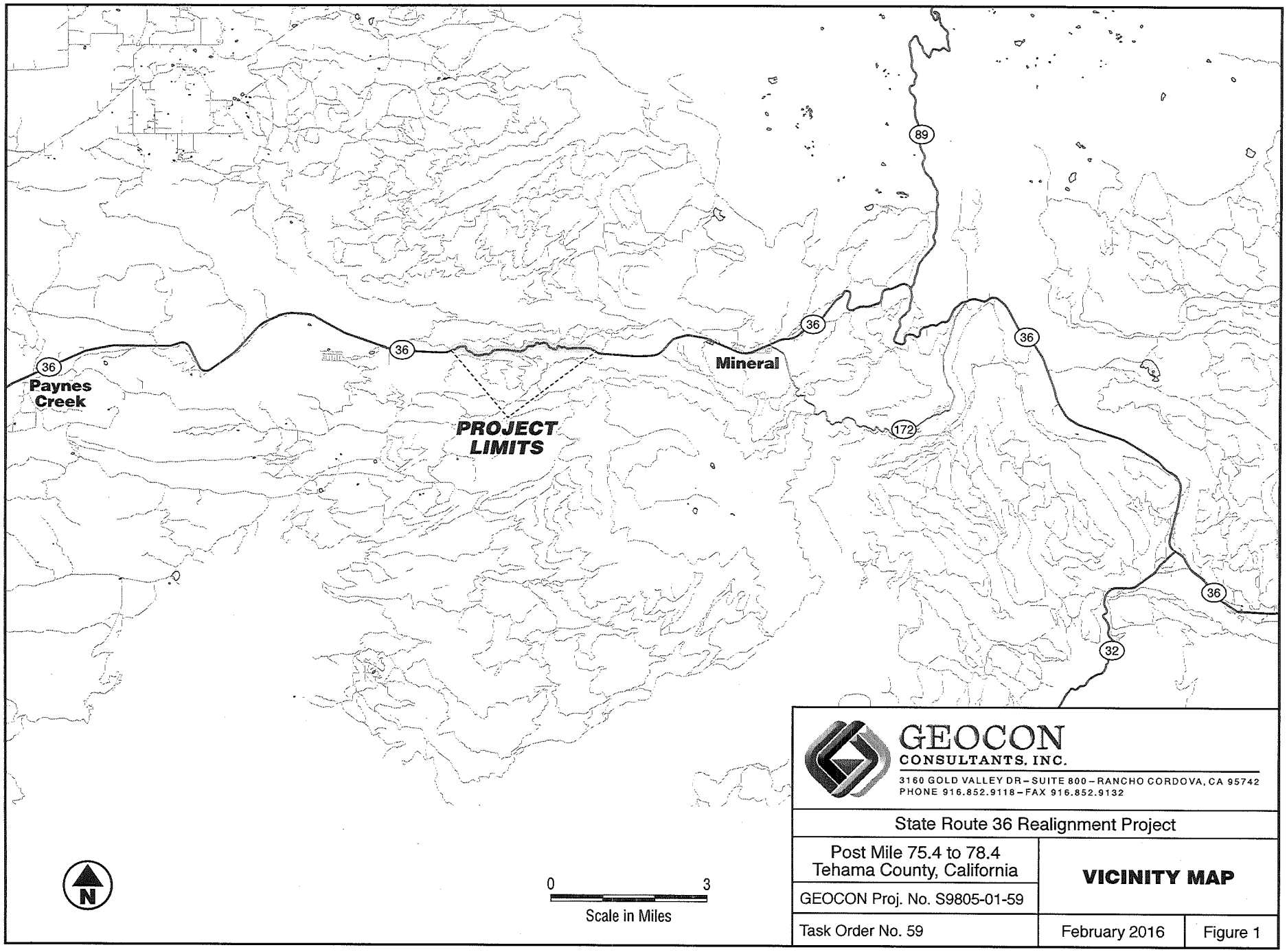
PREPARED BY:

GEOCON CONSULTANTS, INC.
3160 GOLD VALLEY DRIVE, SUITE 800
RANCHO CORDOVA, CALIFORNIA 95742

GEOCON

GEOCON PROJECT NO. S9805-01-59
TASK ORDER NO. 59
EA 02-4G03U1, CONTRACT NO. 03A2132

FEBRUARY 2016



GEOCON
CONSULTANTS, INC.
3160 GOLD VALLEY DR – SUITE 800 – RANCHO CORDOVA, CA 95742
PHONE 916.852.9118 – FAX 916.852.9132

State Route 36 Realignment Project

Post Mile 75.4 to 78.4
Tehama County, California

GEOCON Proj. No. S9805-01-59

Task Order No. 59

VICINITY MAP

February 2016

Figure 1

TABLE 1
SUMMARY OF SOIL BORING COORDINATES
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DATE	LATITUDE	LONGITUDE
EB SR-36 SHOULDER			
EB1	1/26/2016	40.346043763	-121.731741194
EB2	1/26/2016	40.346600544	-121.730170614
EB3	1/26/2016	40.346653296	-121.728396716
EB4	1/26/2016	40.345553077	-121.727480748
EB5	1/26/2016	40.344956322	-121.726011471
EB6	1/26/2016	40.344250667	-121.724571517
EB7	1/26/2016	40.343759539	-121.722911560
EB8	1/26/2016	40.343767902	-121.721156412
EB9	1/26/2016	40.344109619	-121.719462914
EB10	1/26/2016	40.344104745	-121.717730227
EB11	1/26/2016	40.345059933	-121.716436924
EB12	1/26/2016	40.345484006	-121.714869517
EB13	1/26/2016	40.345621961	-121.713053493
EB14	1/26/2016	40.345533514	-121.711257963
EB15	1/26/2016	40.345451599	-121.709566544
EB31	1/26/2016	40.345376276	-121.707810251
EB32	1/26/2016	40.345103120	-121.706066852
EB33	1/26/2016	40.344994235	-121.704272302
EB34	1/26/2016	40.345507445	-121.702731323
EB35	1/26/2016	40.346500230	-121.701397074
EB36	1/26/2016	40.346820871	-121.699789396
EB37	1/26/2016	40.347134778	-121.698109110
EB38	1/26/2016	40.347707369	-121.696632250
EB39	1/26/2016	40.347736749	-121.695093436
EB40	1/26/2016	40.347760878	-121.693224996
EB41	1/26/2016	40.346902242	-121.692170819
EB42	1/26/2016	40.346130883	-121.690792565
EB43	1/26/2016	40.345768665	-121.688859857
EB44	1/26/2016	40.346752097	-121.688137135
EB45	1/26/2016	40.346323381	-121.686577732
EB46	1/26/2016	40.346514471	-121.685116959
EB47	1/26/2016	40.345975891	-121.683495681
EB48	1/26/2016	40.346133714	-121.681722223
EB49	1/26/2016	40.346162228	-121.679893593
EB50	1/26/2016	40.346235004	-121.677999760
EB51	1/26/2016	40.346125353	-121.676285641
EB52	1/26/2016	40.346113430	-121.674530794
EB53	1/26/2016	40.345589018	-121.672932877

TABLE 1
SUMMARY OF SOIL BORING COORDINATES
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DATE	LATITUDE	LONGITUDE
WB SR-36 SHOULDER			
WB16	1/26/2016	40.345603718	-121.710859941
WB17	1/26/2016	40.345634391	-121.712176581
WB18	1/26/2016	40.345668646	-121.713748940
WB19	1/26/2016	40.345495008	-121.715654186
WB20	1/26/2016	40.344662249	-121.717003615
WB21	1/26/2016	40.344192501	-121.718547766
WB22	1/26/2016	40.344065276	-121.720324013
WB23	1/26/2016	40.343827771	-121.722112371
WB24	1/26/2016	40.344042666	-121.723849318
WB25	1/26/2016	40.344856626	-121.725249536
WB26	1/26/2016	40.345239821	-121.726821814
WB27	1/26/2016	40.346429615	-121.727750027
WB28	1/26/2016	40.346748757	-121.729403871
WB29	1/26/2016	40.346435674	-121.731108921
WB30	1/26/2016	40.345816226	-121.732748161
WB54	1/26/2016	40.345289926	-121.672028536
WB55	1/26/2016	40.346016443	-121.673576277
WB56	1/26/2016	40.346258820	-121.675276041
WB57	1/26/2016	40.346258101	-121.677109458
WB58	1/26/2016	40.346233506	-121.678776738
WB59	1/26/2016	40.346263489	-121.680629110
WB60	1/26/2016	40.346259213	-121.682413888
WB61	1/26/2016	40.346223259	-121.684254650
WB62	1/26/2016	NA	NA
WB63	1/26/2016	40.346924024	-121.687312863
WB64	1/26/2016	40.346117083	-121.688509368
WB65	1/26/2016	40.346046512	-121.689968443
WB66	1/26/2016	40.346530249	-121.691675483
WB67	1/26/2016	40.347605498	-121.692629500
WB68	1/26/2016	40.347853547	-121.694267691
WB69	1/26/2016	40.347668542	-121.696116882
WB70	1/26/2016	40.347908304	-121.697608773
WB71	1/26/2016	40.346906028	-121.698958364
WB72	1/26/2016	40.346972903	-121.700676909
WB73	1/26/2016	40.345935569	-121.702081901
WB74	1/26/2016	40.345283236	-121.703558313
WB75	1/26/2016	40.345189428	-121.705274174
WB76	1/26/2016	40.345435409	-121.707016434
WB77	1/26/2016	40.345485504	-121.708790709

TABLE 1
SUMMARY OF SOIL BORING COORDINATES
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DATE	LATITUDE	LONGITUDE
FORMER SERVICE STATION ON THE NORTH SIDE OF SR-36			
B78	1/27/2016	40.345642745	-121.710679923
B79	1/27/2016	40.345653406	-121.710516217
B80	1/27/2016	40.345643091	-121.710396328
B81	1/27/2016	40.345649641	-121.710305083
B82	1/27/2016	40.345644797	-121.710242252
B83	1/27/2016	40.345633348	-121.710204340
B84	1/27/2016	40.345647640	-121.710176757
B85	1/27/2016	40.345694048	-121.710157518
B86	1/27/2016	40.345634464	-121.710106026
B87	1/27/2016	40.345623210	-121.710144747
B88	1/27/2016	40.345633903	-121.710121311
B89	1/27/2016	40.345616512	-121.710126053
B90	1/27/2016	40.345639053	-121.710100724
B91	1/27/2016	40.345647701	-121.710088029
B92	1/27/2016	40.345647470	-121.710059065
B93	1/27/2016	40.345631969	-121.710020082
B94	1/27/2016	40.345607033	-121.710201390

Notes: NA = GPS data not available

TABLE 2
SUMMARY OF SOIL ANALYTICAL RESULTS - TITLE 22 METALS
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

Sample ID	Antimony	Arsenic	Barium	Beryllium	Cadmium	Chromium	Cobalt	Copper	Lead	Molybdenum	Nickel	Selenium	Silver	Thallium	Vanadium	Zinc	Mercury
B78-4'	<2.0	2.6	280	<1.0	<1.0	5.4	2.8	9.5	2.8	<1.0	4.8	<1.0	<1.0	<1.0	23	9.2	<0.10
B79-4'	<2.0	6.2	270	1.1	<1.0	51	15	16	9.8	<1.0	20	<1.0	<1.0	<1.0	95	14	<0.10
B80-4'	2.4	6.6	290	1.2	<1.0	62	12	20	12	<1.0	28	<1.0	<1.0	<1.0	110	12	<0.10
B81-4'	<2.0	4.7	910	1.0	<1.0	21	7.1	17	6.5	<1.0	16	<1.0	<1.0	<1.0	47	34	<0.10
B83-4'	<2.0	5.4	510	<1.0	<1.0	38	8.5	20	9.3	<1.0	23	<1.0	<1.0	<1.0	62	8.8	<0.10
B85-4'	<2.0	3.8	840	<1.0	<1.0	12	5.2	12	5.9	<1.0	9.3	<1.0	<1.0	<1.0	34	17	<0.10
B88-4'	<2.0	6.0	640	1.2	<1.0	31	24	18	14	<1.0	20	<1.0	<1.0	<1.0	68	16	<0.10
TTLC	500	500	10,000	75	100	2,500	8,000	2,500	1,000	3,500	2,000	100	500	700	2,400	5,000	20
10 x STLC	150	50	1,000	7.5	10	50	800	250	50	3,500	200	10	50	70	240	2,500	2.0
CHHSLs																	
Industrial	380	0.24	63,000	190	7.5	100,000	3,200	38,000	320	4,800	16,000	4,800	4,800	63	6,700	100,000	180
Residential	30	0.07	5,200	16	1.7	100,000	660	3,000	80	380	1,600	380	380	5.0	530	23,000	18
ESLs																	
Table A, Industrial	40	1.6	1,500	8.0	12	750*	80	230	320	40	150	10	40	10	200	600	10
Table A, Residential	20	0.39	750	4.0	12	750*	23	230	80	40	150	10	20	0.78	200	600	6.7
Table K-3, Construction Exposure	120	10	61,000	180	110	460,000*	49	12,000	320	1,500	6,100	1,500	1,500	3.1	1,500	93,000	27

TABLE 2
SUMMARY OF SOIL ANALYTICAL RESULTS - TITLE 22 METALS
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

Sample ID	Antimony	Arsenic	Barium	Beryllium	Cadmium	Chromium	Cobalt	Copper	Lead	Molybdenum	Nickel	Selenium	Silver	Thallium	Vanadium	Zinc	Mercury
Background Concentrations [1]																	
Minimum	0.15	0.6	133	0.25	0.05	23	2.7	9.1	12.4	0.1	9.0	0.015	0.10	0.17	39	88	0.10
Maximum	1.95	11	1,400	2.70	1.70	1,579	46.9	96.4	97.1	9.6	509	0.430	8.30	1.10	288	236	0.90
Mean	0.60	3.5	509	1.28	0.36	122	14.9	28.7	23.9	1.3	57	0.058	0.80	0.56	112	149	0.26

Notes: B78-4
- Top of 1-foot sample depth interval in feet below ground surface
- Boring identification

Concentrations are shown in milligrams per kilogram unless otherwise noted
< = Less than laboratory reporting limits
TTLC = Total Threshold Limit Concentration
STLC = Soluble Threshold Limit Concentration
CHHSLs = California Environmental Protection Agency, California Human Health Screening Levels for industrial and residential land use
TTLC, STLC, and CHHSLs shown for chromium are for chromium III
* = for Chromium or Chromium III compounds
ESLs = Environmental Screening Levels, Table A and Table K-3, San Francisco Bay Regional Water Quality Control Board, December 2013
[1] Background Concentrations of Trace and Major Elements in California Soils (Kearney Foundation of Soil Science, Division of Agricultural and Natural Resources, University of California, March 1
Concentration(s) in parenthesis is/are WET or WET/TCLP soluble metal concentrations

TABLE 3
SUMMARY OF SOIL ANALYTICAL RESULTS - LEAD
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DEPTH INTERVAL (feet)	TOTAL LEAD (mg/kg)
EB SR-36 SHOULDER		
EB1-0'	0-1	4.3
EB1-1'	1-2	1.3
EB1-2'	2-3	<1.0
EB2-0'	0-1	8.7
EB2-1'	1-2	20
EB2-2'	2-3	5.7
EB3-0'	0-1	17
EB3-1'	1-2	21
EB3-2'	2-3	7.5
EB4-0'	0-1	14
EB4-1'	1-2	8.1
EB4-2'	2-3	6.3
EB5-0'	0-1	2.2
EB5-1'	1-2	48
EB5-2'	2-3	9.7
EB6-0'	0-1	22
EB6-1'	1-2	6.3
EB6-2'	2-3	7.8
EB7-0'	0-1	2.7
EB7-1'	1-2	26
EB7-2'	2-3	7.7
EB8-0'	0-1	4.7
EB8-1'	1-2	3.9
EB8-2'	2-3	12
EB9-0'	0-1	2.4
EB9-1'	1-2	30
EB9-2'	2-3	8.0
EB10-0'	0-1	2.9
EB10-1'	1-2	14
EB10-2'	2-3	6.0
EB11-0'	0-1	10
EB11-1'	1-2	9.9
EB11-2'	2-3	5.9
EB12-0'	0-1	6.7
EB12-1'	1-2	5.8
EB12-2'	2-3	10

TABLE 3
SUMMARY OF SOIL ANALYTICAL RESULTS - LEAD
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DEPTH INTERVAL (feet)	TOTAL LEAD (mg/kg)
EB13-0'	0-1	17
EB13-1'	1-2	17
EB13-2'	2-3	11
EB14-0'	0-1	15
EB14-1'	1-2	8.8
EB14-2'	2-3	9.4
EB15-0'	0-1	19
EB15-1'	1-2	15
EB15-2'	2-3	9.1
EB31-0'	0-1	4.7
EB31-1'	1-2	5.8
EB31-2'	2-3	6.0
EB32-0'	0-1	18
EB32-1'	1-2	4.4
EB32-2'	2-3	6.2
EB33-0'	0-1	13
EB33-1'	1-2	7.7
EB33-2'	2-3	6.1
EB34-0'	0-1	12
EB34-1'	1-2	1.9
EB34-2'	2-3	1.6
EB35-0'	0-1	36
EB35-1'	1-2	3.0
EB35-2'	2-3	2.6
EB36-0'	0-1	7.6
EB36-1'	1-2	4.7
EB36-2'	2-3	3.3
EB37-0'	0-1	7.1
EB37-1'	1-2	4.7
EB37-2'	2-3	5.5
EB38-0'	0-1	2.5
EB38-1'	1-2	2.0
EB39-0'	0-1	12
EB39-1'	1-2	5.0
EB39-2'	2-3	6.3
EB40-0'	0-1	8.3
EB40-1'	1-2	3.4
EB40-2'	2-3	3.6

TABLE 3
SUMMARY OF SOIL ANALYTICAL RESULTS - LEAD
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DEPTH INTERVAL (feet)	TOTAL LEAD (mg/kg)
EB41-0'	0-1	3.0
EB41-1'	1-2	12
EB41-2'	2-3	8.0
EB42-0'	0-1	2.8
EB42-1'	1-2	9.6
EB42-2'	2-3	11
EB43-0'	0-1	24
EB43-1'	1-2	9.4
EB43-2'	2-3	4.2
EB44-0'	0-1	5.1
EB44-1'	1-2	18
EB45-0'	0-1	15
EB45-1'	1-2	21
EB45-2'	2-3	5.7
EB46-0'	0-1	9.1
EB46-1'	1-2	42
EB47-0'	0-1	6.1
EB47-1'	1-2	6.3
EB47-2'	2-3	7.4
EB48-0'	0-1	26
EB48-1'	1-2	4.7
EB48-2'	2-3	4.4
EB49-0'	0-1	4.7
EB49-1'	1-2	8.7
EB49-2'	2-3	4.5
EB50-0'	0-1	6.5
EB50-1'	1-2	6.4
EB50-2'	2-3	5.3
EB51-0'	0-1	15
EB51-1'	1-2	5.1
EB51-2'	2-3	3.5
EB52-0'	0-1	19
EB52-1'	1-2	3.9
EB53-0'	0-1	19
EB53-1'	1-2	11
EB53-2'	2-3	7.5

TABLE 3
SUMMARY OF SOIL ANALYTICAL RESULTS - LEAD
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DEPTH INTERVAL (feet)	TOTAL LEAD (mg/kg)
WB SR-36 SHOULDER		
WB16-0'	0-1	49
WB16-1'	1-2	8.0
WB16-2'	2-3	7.9
WB17-0'	0-1	2.3
WB17-1'	1-2	47
WB17-2'	2-3	9.7
WB18-0'	0-1	1.9
WB18-1'	1-2	36
WB18-2'	2-3	20
WB19-0'	0-1	25
WB19-1'	1-2	13
WB19-2'	2-3	8.1
WB20-0'	0-1	6.0
WB20-1'	1-2	110
WB21-0'	0-1	2.1
WB21-1'	1-2	5.7
WB21-2'	2-3	4.3
WB22-0'	0-1	2.9
WB22-1'	1-2	21
WB22-2'	2-3	3.3
WB23-0'	0-1	4.1
WB23-1'	1-2	31
WB23-2'	2-3	12
WB24-0'	0-1	16
WB24-1'	1-2	12
WB24-2'	2-3	14
WB25-0'	0-1	2.7
WB25-1'	1-2	7.4
WB25-2'	2-3	10
WB26-0'	0-1	3.7
WB26-1'	1-2	7.7
WB26-2'	2-3	11
WB27-0'	0-1	16
WB27-1'	1-2	7.1
WB27-2'	2-3	7.2

TABLE 3
SUMMARY OF SOIL ANALYTICAL RESULTS - LEAD
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DEPTH INTERVAL (feet)	TOTAL LEAD (mg/kg)
WB28-0'	0-1	2.8
WB28-1'	1-2	13
WB28-2'	2-3	4.0
WB29-0'	0-1	18
WB29-1'	1-2	38
WB29-2'	2-3	25
WB30-0'	0-1	8.7
WB30-1'	1-2	<1.0
WB30-2'	2-3	1.1
WB54-0'	0-1	7.5
WB54-1'	1-2	4.9
WB54-2'	2-3	6.4
WB55-0'	0-1	17
WB55-1'	1-2	2.5
WB55-2'	2-3	4.2
WB56-0'	0-1	40
WB56-1'	1-2	1.9
WB56-2'	2-3	1.0
WB57-0'	0-1	11
WB57-1'	1-2	4.6
WB57-2'	2-3	1.5
WB58-0'	0-1	24
WB58-1'	1-2	2.0
WB59-0'	0-1	28
WB59-1'	1-2	6.9
WB59-2'	2-3	6.3
WB60-0'	0-1	30
WB60-1'	1-2	1.9
WB60-2'	2-3	5.1
WB61-0'	0-1	17
WB61-1'	1-2	11
WB61-2'	2-3	6.2
WB62-0'	0-1	1.3
WB62-1'	1-2	4.3
WB62-2'	2-3	7.4
WB63-0'	0-1	13
WB63-1'	1-2	31

TABLE 3
SUMMARY OF SOIL ANALYTICAL RESULTS - LEAD
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DEPTH INTERVAL (feet)	TOTAL LEAD (mg/kg)
WB64-0'	0-1	13
WB64-1'	1-2	1.1
WB64-2'	2-3	4.7
WB65-0'	0-1	1.9
WB65-1'	1-2	35
WB65-2'	2-3	6.4
WB66-0'	0-1	18
WB66-1'	1-2	5.3
WB66-2'	2-3	5.7
WB67-0'	0-1	4.5
WB67-1'	1-2	3.0
WB67-2'	2-3	5.0
WB68-0'	0-1	2.8
WB68-1'	1-2	4.4
WB68-2'	2-3	6.4
WB69-0'	0-1	16
WB69-1'	1-2	7.7
WB70-0'	0-1	3.3
WB70-1'	1-2	11
WB70-2'	2-3	3.4
WB71-0'	0-1	5.9
WB71-1'	1-2	5.4
WB72-0'	0-1	4.4
WB72-1'	1-2	12
WB72-2'	2-3	2.9
WB73-0'	0-1	16
WB73-1'	1-2	5.7
WB74-0'	0-1	14
WB74-1'	1-2	17
WB74-2'	2-3	21
WB75-0'	0-1	2.8
WB75-1'	1-2	18
WB75-2'	2-3	8.2
WB76-0'	0-1	9.3
WB76-1'	1-2	8.5
WB77-0'	0-1	5.4
WB77-1'	1-2	13
WB77-2'	2-3	7.8

TABLE 3
SUMMARY OF SOIL ANALYTICAL RESULTS - LEAD
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

BORING ID	SAMPLE DEPTH INTERVAL (feet)	TOTAL LEAD (mg/kg)
TTLC		1,000
10 x STLC		50
CHHSLs		
Industrial		320
Residential		80
ESLs		
Table A, Industrial		320
Table A, Residential		80
Table K-3, Construction Exposure		320
Background Concentrations [1]		
Minimum		12.4
Maximum		97.1
Mean		23.9

Notes:
EB1-0
Top of 1-foot sampling depth interval in feet below ground surface
Boring identification
mg/kg = Milligrams per kilogram
< = Less than the laboratory reporting limit
TTLC = Total Threshold Limit Concentration
STLC = Soluble Threshold Limit Concentration
CHHSLs = California Environmental Protection Agency, California Human Health Screening Levels
ESLs = Environmental Screening Levels, Table A and Table K-3, San Francisco Bay Regional Water Quality Control Board, Dec. 2013
[1] Background Concentrations of Trace and Major Elements in California Soils (Kearney Foundation of Soil Science, Division of Agricultural and Natural Resources, University of California, March 1996)

TABLE 4
SUMMARY OF SOIL ANALYTICAL RESULTS - PETROLEUM HYDROCARBONS
EA 02-4G03U1
STATE ROUTE 36 (02-TEH-36) POST MILE 75.4 TO 78.4 REALIGNMENT PROJECT
TEHAMA COUNTY, CALIFORNIA

SAMPLE ID	GRO (mg/kg)	DRO (mg/kg)	ORO (mg/kg)	BENZENE (µg/kg)	TOLUENE (µg/kg)	ETHYL BENZENE (µg/kg)	XYLENES (µg/kg)	VOCs (µg/kg)
B78-4'	<1.0	3.9	2.9	<5.0	<5.0	<5.0	<15	ND
B79-4'	<1.0	60	210	<5.0	<5.0	<5.0	<15	ND
B80-4'	<1.0	2.3	2.1	<5.0	<5.0	<5.0	<15	ND
B81-4'	<1.0	2.4	1.9	<5.0	<5.0	<5.0	<15	ND
B83-4'	<1.0	3.2	2.0	<5.0	<5.0	<5.0	<15	ND
B85-4'	<1.0	5.4	4.4	<5.0	<5.0	<5.0	<15	ND
B88-4'	<1.0	3.7	2.7	<5.0	<5.0	<5.0	<15	ND
ESLs								
Table A, Residential	100	100	100	44	2,900	3,300	2,300	
Table A, Commercial/Industrial	500	110	500	44	2,900	3,300	2,300	
Table K-3, Construction Exposure	2,700	900	28,000	71,000	4,300,000	490,000	2,500,000	

Notes:
B78-4'
Top of 1-foot sample depth interval in feet below ground surface
Boring Identification
GRO = Gasoline-range organics
DRO = Diesel-range organics
ORO = Oil-range organics
VOCs = Volatile organic compounds
mg/kg = Milligrams per kilogram
µg/kg = Micrograms per kilogram
< = Less than the laboratory reporting limit
ND = Not detected
ESLs = Environmental Screening Levels, Table A and Table K-3, San Francisco Bay Regional Water Quality Control Board, December 2013

INFORMATION HANDOUT

For Contract No. 02- 4G03U4
At 02-TEH-36-75.1/78.4

Identified by
Project ID 0215000056

MATERIALS INFORMATION

Specifications for Rock Excavation (Controlled Blasting)

Replace section 19-4 with:

19-4 ROCK EXCAVATION (CONTROLLED BLASTING)

19-4.01 GENERAL

19-4.01A Summary

Section 19-4 includes specifications for performing rock excavation with controlled blasting and presplitting rock to form rock excavation slopes.

You may use hydraulic splitters, pneumatic hammers, controlled blasting, or other roadway excavation techniques authorized to fracture rock and construct stable final rock cut faces.

Comply with section 12.

Comply with federal, state, and local blasting regulations. Regulations containing specific Cal-OSHA requirements for blasting activities include 8 CA Code of Regs, Ch 4, Subchapter 7, Group 18, "Explosive Materials." Regulations for explosives containing percholate materials include 22 CA Code of Regs, Division 4.5, Ch 33, "Best Management Practices for Percholate Materials."

You are liable for damages resulting from blasting activities.

19-4.01B Definitions

controlled blasting: Use of explosives and blasting accessories in predetermined spaced and aligned drill holes to limit blast vibrations, noise from airblast overpressure, and flyrock.

flyrock: Rock that becomes airborne due to blasting.

near field blasting: Blasting within 30 feet of a critical structure.

presplitting: Establishment of a free surface or shear plane in rock along the specified excavation slope by the controlled use of explosives and blasting accessories in appropriately aligned and spaced drill holes.

19-4.01C Submittals

19-4.01C(1) General

Submit 3 copies of the blasting safety plan and each controlled blasting plan. After each plan is authorized, submit 3 additional copies of each authorized plan.

19-4.01C(2) Blasting Safety Plan

Submit a blasting safety plan. The plan must include:

1. References to applicable federal, state, and local codes and regulations
2. Copies of permits required for blasting activities
3. Business name, contractor license number, address, and telephone number of the blasting subcontractor
4. Proof of current liability insurance and bonding
5. Name, address, telephone number, copies of applicable licenses, and resume of:
   - 5.1. Blaster-in-charge
   - 5.2. Personnel responsible for controlled blast design, loading, and conducting the blasting operation
   - 5.3. Safety officer for blasting subcontractor
   - 5.4. Blast monitoring consultant
   - 5.5. Blasting consultant
6. Name, address, and telephone number of the local fire station and law enforcement agencies
7. Detailed description of:
   - 7.1. Location where explosives will be stored
   - 7.2. Security measures to protect and limit access to the explosives
   - 7.3. Transportation means for explosives
   - 7.4. List of personnel permitted to handle the explosives
8. Exclusion zone and limited-entry zone for nonblast related operations and personnel surrounding loading and blasting operations

9. Details of warning signals used to alert employees on the job site of an impending blast and to indicate the blast is completed and the area is safe to enter
10. How blasting operations will be conducted
11. Measures to protect blasting operations and personnel from lightning
12. Emergency evacuation procedures for areas where explosives may be present
13. How misfires will be recognized, handled, and resolved including:
    13.1. Who will be notified
    13.2. How blast zone will be secured until misfire is resolved
    13.3. Identification of equipment that may be needed to resolve misfires
14. Details of signs to be used around blasting zones including:
    14.1. Timing of when signs will be posted relative to a specific blast
    14.2. Name and telephone number of person responsible for placing signs
    14.3. Roadway signs for compliance with Chapter 6, Typical Application 2, of the California MUTCD.
15. Traffic control details for:
    15.1. Loading and blasting operations
    15.2. Misfire event or other blast related phenomenon that causes a transportation corridor to remain closed to the public
16. Description of possible noxious gas generation and details of safeguards to be used to protect employees, work zones adjacent to the shot, private property, and the public
17. Procedure to report and resolve complaints for blast related accidents
18. Copies of each MSDS and manufacturer data sheets of explosives, caps, primers, initiators, and other compounds

19-4.01C(3) Controlled Blasting Plan

Submit a controlled blasting plan for each blast. The plan must include details on how each blast will be controlled and the following:

1. Blast identification by numerical and chronological sequence
2. Location, referenced to stationing, offset distance, date, and time of blast
3. Drawings showing drill hole pattern, spacing, burden, and initiation sequence
4. Typical cross-sections through zone to be blasted
5. Groundwater level, if present, within the prism to be blasted
6. Initiation-sequence diagram showing the actual firing time of each delay
7. Type of material to be blasted
8. Number of drill holes
9. Diameter, depth, and spacing of holes
10. Height or length of stemming
11. Types and characteristics of explosives used, including explosive's density, relative strength, and date of manufacture
12. Type of caps and delay periods used and their date of manufacture
13. Total amount of explosives used
14. Total amount of explosives detonating within any 8 millisecond period
15. Powder factor (pounds of explosive per cubic yard of material blasted)
16. Method of firing
17. Direction and distance to nearest building or structure
18. Type and method of instrumentation
19. Location and placement of instruments
20. Measures to limit air noise and flyrock
21. Measures to limit overbreak
22. Name of blasting subcontractor
23. Name and signature of blaster-in-charge
24. Drawings showing spacing and proximity of shot guards to blast location

Changes to the controlled blasting plan made to adjust for site conditions must be submitted for review before implementing.

19-4.01D Quality Contol and Assurance

19-4.01D(1) General

Not Used

19-4.01D(2) Blaster-In-Charge

Assign a blaster-in-charge responsible for supervising all blasting activities. The blaster-in-charge must have 10 years of experience in performing or supervising similar blasting activities and must be a licensed blaster.

19-4.01D(3) Blast Monitoring Consultant

Assign a blast monitoring consultant to monitor blasting generated vibrations and noise near buildings and structures that may be subject to damage. The monitoring consultant must be responsible for collecting and interpreting vibration and noise data. The blast monitoring consultant must:

1. Not be employed by the blasting contractor or other subcontractor on the project
2. Have a minimum of a 2-year Associate's Degree in science or engineering
3. Have at least 5 years of documented experience in collecting and interpreting ground vibrations and noise data

19-4.01D(4) Blasting Consultant

Assign a blasting consultant to oversee near field blasting activities. The blasting consultant must:

1. Be an engineer or geologist who is licensed in the State
2. Have 10 years of experience providing specialized blasting services in near field blasting
3. Not be employed by the blasting contractor, explosive manufacturer, or explosive distributor
4. Submit a resume of credentials and a list of projects worked on

19-4.01D(5) Preblast Surveys

At least 15 days before starting blasting activities, prepare a preblast survey of all buildings and structures within 330 feet of blasting activities and submit it with the controlled blasting plan. The preblast survey must include a written report, sketches, and photos or a videotape with date and time displayed on the image. The preblast survey must include:

1. Name of the person making the inspection
2. Name of property owner and occupants
3. Property address
4. Date and time of the inspection
5. Description of the structure or other improvement including culverts and bridges
6. Detailed description of existing condition of walls, ceiling, and floor of each interior room including attic and basement
7. Detailed description of existing condition of foundations, exterior walls, roofs, doors, windows, and porches
8. Detailed description of existing condition of garages, outbuildings, sidewalks, driveways, and swimming pools
9. Detailed listing of highway sign posts, light fixtures, and overhead power lines
10. Survey of wells or other private water supplies including total depth and existing water surface levels
11. Identification of sites conducting procedures, processes, or operations that may be sensitive to blasting activities
12. Scaled map or aerial photo showing the location of structures and properties surveyed and location of all proposed blasting sites

If blasting activities are suspended for a period of 45 days or more, perform another preblast survey and submit it at least 15 days before resuming blasting activities.

After blasting activities are completed, prepare and submit a postblast survey of the same buildings and structures as in the preblast survey. The postblast survey must include all items included in the preblast survey.

19-4.01D(6) Vibration and Noise Monitoring

Vibration levels must be kept below peak particle velocity of 2 inches per second at the nearest building or structure.

Noise from airblast overpressure levels must be kept below 128 dB (C-network or Linear network) at the nearest building

Ground vibrations and noise created from blasting must be controlled by using properly designed delay sequencing and charge weights for shots.

Provide 3 seismographs to be available for deployment that are appropriate for controlled blasting activities and capable of:

1. Recording particle velocities for 3 mutually perpendicular components of vibration and instantaneous resultant peak vector sum in the range generally found with controlled blasting.
2. Continuously measuring, recording, and reporting vibrations along 3 primary axes.
3. Measuring and recording vibration frequencies ranging from 2 to 300 Hz.
4. Providing a printed record of each event showing a plot of peak particle velocity versus vibration frequencies.
5. Measuring and recording airblast noise levels. The noise transducer must be detachable from the main unit to allow placing at elevations with a clear line of sight between transducer and blast.

Record each blast shot using approved seismographs and prepare a vibration and noise monitoring report. The report must include:

1. Identification of instruments used
2. Name of blast monitoring consultant
3. Distance and direction of recording stations from blast area
4. Type of ground at recording station and material on which instrument sits
5. Maximum particle velocity in each component and resultant peak particle velocity of each shot
6. Copy of seismograph readings with date and signature of blast monitoring consultant
7. Noise levels recorded in dB (C-network or Linear network) units

19-4.01D(7) Video Recording of Blasts

Video-record each blast. The video-recording must be taken from a safe location with a clear view of the blast area, activities, and progression. Identify each video or section of video with an index to identify each blast. Submit a copy of each video in DVD-Video format.

19-4.01D(8) Blasting Complaints

Accurately document each complaint. Notify the Engineer immediately of a complaint received or at the start of the next day's work shift. Complaint documentation must include:

1. Name and address of complainant
2. Date, time, and nature of complaint
3. Dated photo or videotape of physical damage
4. Name of person receiving complaint
5. Record of complaint investigation conducted
6. Resolution of complaint

19-4.01D(9) Postblast Reports

Document each shot in a postblast report. The postblast report must include all data required in the controlled blasting plan for that shot and the following:

1. Description of site conditions, loading, and time of blast
2. Description of weather conditions at time of blast including wind direction and cloud cover
3. Drillers boring record
4. Copy of vibration and noise monitoring report
5. Copy of documented complaints arising from the blast

Submit the postblast report within 48 hours of the blast.

19-4.02 MATERIALS

The maximum diameter of explosives used in presplit holes must not be greater than 50 percent of the diameter of the presplit hole.

Only standard cartridge explosives prepared and packaged by explosive manufacturing firms must be used in the presplit holes. These must consist of one of the following:

1. Fractional portions of standard cartridges to be affixed to the detonating cord in the field
2. Solid column explosives joined and affixed to the detonating cord in the field

Stemming materials must be dry, free-running material meeting the grading requirements in the following table when tested under California Test 202:

Sieve sizes	Percentage passing
3/8"	100
No. 8	90

19-4.03 CONSTRUCTION

At least 7 days before starting or resuming blasting activities, notify occupants of the local buildings within 330 feet of the blasting area in writing. Verbally notify occupants of pending blasting activities on the day of blasting.

Do not perform blasts within 1,200 feet of concrete placed within 72 hours.

Before firing any blast, confirm that groundwater conditions are consistent with shot design and explosive type to be used.

Before firing any blast in areas where flyrock may result in personal injury or damage to property or the work, cover the rock to be blasted with blasting mats, soil, or other equally serviceable material to prevent flyrock.

If blasting causes flyrock, suspend blasting activities. The blasting consultant must review the site to determine the cause of the flyrock problem and provide an amendment to the controlled blasting plan that prevents flyrock.

Do not use drill cuttings as stemming in controlled blasting operations.

Before drilling the presplitting holes, remove overburden soil and weathered rock along the top of the excavation for a distance of at least 50 feet beyond the drilling limits or to the end of the excavation. Ensure removal of overburden soil and weathered rock and expose fresh rock to an elevation equal to the bottom of the adjacent lift of the presplitting holes being drilled.

Drill slope holes for presplitting along the line of the planned slope within the tolerances specified. The drill holes must be at least 2-1/2 inches, but not more than 3 inches in diameter. Control the drilling operations by using proper equipment and techniques. Ensure no hole deviates from the plane of the planned slope by more than 12 inches or from parallel to an adjacent hole by more than 67 percent of the planned horizontal spacing between holes.

The length of presplit holes for an individual lift must not exceed 30 feet, unless you can demonstrate to the Engineer that you can stay within the above tolerances and produce a uniform slope. The length of holes may then be increased to a maximum of 60 feet if authorized.

The spacing of presplit holes must not exceed 3 feet on centers and must be adjusted to produce a uniform shear face between holes.

The Engineer may order you to drill auxiliary holes along the presplit line. These holes must not be loaded or stemmed. Except for spacing, auxiliary drill holes must comply with the specifications for presplit holes. Place the adjacent line of production holes inside the presplit lines in such a manner that avoids damage to the presplit face.

If necessary to reduce shatter and overbreak of the presplit surface, the 1st line of production holes must be drilled parallel to the slope line at the top of the cut and at each bench level thereafter.

Blasting techniques that result in damage to the presplit surface must be discontinued immediately.

No portion of the production holes must be drilled within 8 feet of a presplit plane unless authorized. The bottom of the production holes must not be lower than the bottom of the presplit holes.

A maximum offset of 24 inches will be permitted for a construction working bench at the bottom of each lift for use in drilling the next lower presplitting pattern.

Adjust the drilling operations to compensate for drift of previous levels and for the offset at the start of new levels to maintain the specified slope plane.

If the methods of drilling and blasting do not produce the desired result of a uniform slope and shear face without overbreak and within the tolerances specified, drill, blast, and excavate in short sections, up to 100 feet, until a technique produces desired results.

If a fractional portion of a standard explosive cartridge is used, the cartridge must be firmly affixed to a length of detonating cord equal to the depth of the drill hole so that the cartridge does not slip down the detonating cord nor cock across the hole and bridge the flow of stemming material. Spacing of cartridges along the length of the detonating cord must not exceed 30 inches center to center and must be adjusted to give the desired results.

If a solid column type explosive is used, the column must be assembled and affixed to the detonating cord to comply with the explosive manufacturer's instructions. Submit as an informational submittal a copy of the explosive manufacturer's instruction before using the column type explosive.

The bottom charge of a presplit hole may be larger than the line charges but must not cause overbreak. The top charge of the presplitting hole must be placed far enough below the collar to avoid overbreaking the surface.

Before placing the charge, the hole must be free of obstructions for the hole's entire depth. Ensure placing of the charge does not cause caving of material from the walls of the holes.

The Engineer may order the use of stemming materials as necessary to achieve a satisfactory presplit face. Stemmed presplit holes must be completely filled to the collar.

Detonate charges in each presplitting pattern simultaneously.

The tolerances in section 19-2.03G do not apply to presplit surfaces of excavation slopes where presplitting is required. The presplit face must not deviate more than 1 foot from the plane passing through adjacent drill holes, except where the character of the rock is such that irregularities are unavoidable. The average plane of the completed slopes must not deviate more than 1 foot from the plan slopes. These tolerances are measured perpendicular to the plane of the slope. No portion of the slope may encroach on the roadbed.

If equally satisfactory presplit slopes are obtained, you may either presplit the slope face before drilling for production blasting or presplit the slope face and production blast at the same time, provided that the presplitting drill holes are fired with zero delay. The production holes must be delayed by at least 50 milliseconds starting at the row of holes farthest from the slope and progressing in steps to the row of holes nearest the presplit line. The presplitting holes must extend either to the end of the excavation or for a distance of not less than 50 feet beyond the limits of the production holes to be detonated.

19-4.04 PAYMENT

Rock excavation (controlled blasting) is paid for as roadway excavation.The payment quantity for roadway excavation includes the length of presplitting and any auxiliary drilled holes used for rock excavation (controlled blasting).