INFORMATION HANDOUT

For Contract No. 02-0E0904
At 02-Sha-5-R29.3/R31.1, R45.0/R45.8

Identified by
Project ID 0200000016

PERMITS

United States Army Corps of Engineers

Non-Reporting Nationwide 404

WATER QUALITY

California Regional Water Quality Control Board, Central Valley Region

Board Order No. WDID No. 5A45CR00466

AGREEMENTS

California Department of Fish and Wildlife

Notification No. 1600-2014-0191-R1

RAILROAD RELATIONS

Railroad Relations and Insurance Requirements

Railroad’s Minimum Requirements

Railroad's Guidelines for Temporary Shoring

Railroad's Demolition Guidelines for Overpass Structures

MATERIALS INFORMATION

Revised Foundation Report for Shasta Viaduct (Replace), Dated March 12, 2015

Addendum to Revised Foundation Report for Shasta Viaduct (replace), Dated June 22, 2015

Suspension P-S Logging

Results of Tomographic Seismic Survey for Sidehill Viaduct Bridge Replacement Project, Shasta County, California, Dated December 12, 2014

Results of Seismic Refraction Survey for Sidehill Viaduct Bridge Replacement Project, Shasta County, California, Dated December 23, 2014

Laboratory Unconfined Compressive Strength and Point load Test Data, Dated April 16, 2015

Geotechnical Engineering Evaluation of Permanent Loadings on the Union Pacific Railroad (UPRR) Tunnel, Dated August 7, 2015

Geotechnical Memorandum, Dated June 1, 2015

Asbestos and Lead-Containing Paint Survey Report, Dated March 2015

Division of Occupational Safety and Health, Mining and Tunneling Unit, Underground Classification, Dated March 2, 2015

Optional Disposal Site Information

Water Source Information

PERMITS

United States Army Corps of Engineers

Non-Reporting Nationwide 404

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

REPLY TO
ATTENTION OF

August 7, 2014

Regulatory Division (SPK-2014-00648)

California Department of Transportation
Attn: Mr. Phil Baker
1657 Riverside Drive, MS#93
Redding, California 96001

Dear Mr. Baker:

We are responding to your agency's July 3, 2014, request for a Department of the Army Nationwide Permit 14 verification for the Interstate 5 (I-5) Sidehill Viaduct Replacement (EA 02-0E090, PM 29.5 to 30.5) project. This approximately 1.5-acre project involves activities, including discharges of dredged or fill material, in waters of the United States to place a 48" corrugated steel pipe (CSP) in an intermittent stream to accommodate the new roadway alignment. The project is located along southbound I-5, Section 21, Township 34 North, Range 4 West, Mount Diablo Meridian, Latitude 40.7816202°, Longitude -122.322085°, near Lake Shasta, Shasta County, California.

Furthermore, we understand the California Department of Transportation (Caltrans) is the National Environmental Policy Act (NEPA) lead Federal agency for this project, and as such, will ensure the authorized work complies with the NEPA, the Endangered Species Act, the National Historical Preservation Act, and any other applicable federal laws. As the applicant, you shall include this office in any future consultations involving compliance with the Endangered Species Act, the National Historic Preservation Act, and any other applicable federal laws, as they pertain to the activities authorized herein, so that we may consult as appropriate or designate you to consult on our behalf.

Based on the information you provided, the proposed activity, resulting in the permanent loss of approximately 0.043 acre of intermittent stream, is authorized by Nationwide Permit Number 14, Linear Transportation Projects. However, until Section 401 Water Quality Certification for the activity has been issued or waived, our authorization is denied without prejudice. Once you have provided us evidence of water quality certification, the activity is authorized and the work may proceed subject to the conditions of certification and the Nationwide Permit. Your work must comply with the general terms and conditions listed on the enclosed Nationwide Permit 14 information sheets (enclosure 1), our Final Sacramento District NWP Regional Conditions for California, specifically numbers 6c, 6d, 8a, 8b, 9, 10, 11, 12, 13, 14, and 19 (enclosure 2), and the following special conditions:

Special Conditions

1. To mitigate for the loss of 0.043 acres of waters of the United States, you shall construct 0.50 acres of riparian habitat within the off-site mitigation area (Jelly's Ferry) as shown in the document entitled *Bella Ditty Roadway Improvement Project Off-site Mitigation Planting and Monitoring Plan*, dated February 6, 2014, prepared by Western Shasta Resources Conservation District and Caltrans Stewardship Branch.

2. The enclosed plan drawing entitled *Preliminary Drainage Plan, PDP-1* (enclosure 3), plotted June 17, 2014, by Brett Ditzler of Caltrans, is incorporated by reference as a condition of this authorization. Any deviations from the work as authorized, which result in additional impacts to waters of the U.S., including wetlands, must be coordinated with this office prior to impacts.

3. No construction activities shall occur within standing or flowing waters. In perennial streams, this may be accomplished through dewatering the work area. Dewatering plans must be approved, in writing, by this office prior to commencement of construction activities. Plans, maps and/or drawings may be submitted electronically to regulatory-info@usace.army.mil.

4. Excavated materials from the permit area shall not be stockpiled or disposed of outside the permit area. Disposal and stockpile areas must be reviewed and approved by this office prior to commencement of construction activities. Plans, maps and/or drawings may be submitted electronically to regulatory-info@usace.army.mil.

5. If any of the above conditions are violated or unauthorized activities occur, you shall stop work immediately and notify this office. You shall provide us with a detailed description of the unauthorized activity(s), photo documentation, and any measures taken to remedy the violation.

6. This permit is contingent upon the permittee applying for and being issued a Section 401 Water Quality Certification. Evidence of a water quality certification must be submitted to this office, prior to commencing work in Waters of the U.S. All terms and conditions of the Section 401 Water Quality Certification are expressly incorporated as conditions of this permit.

This verification is valid until March 18, 2017, when the existing Nationwide Permits are scheduled to be modified, reissued, or revoked. Furthermore, if you commence or are under contract to commence this activity before the date that the relevant NWP is modified, reissued or revoked, you will have twelve (12) months from the date of the modification, reissuance or revocation of the NWP to complete the activity under the present terms and conditions. Failure to comply with the General and Regional Conditions of this Nationwide Permit, or the project-specific Special Conditions of this authorization, may result in the suspension or revocation of your authorization.

We would appreciate your feedback. At your earliest convenience, please tell us how we are doing by completing the customer survey on our website under *Customer Service Survey.*

Please refer to identification number SPK-2014-00648 in any correspondence concerning this project. If you have any questions, please contact Ms. Leah Fisher at California North Branch Office, Regulatory Division, Sacramento District, U.S. Army Corps of Engineers, 1325 J Street, Room 1350, Sacramento, California 95814-2922, by email at *Leah.M.Fisher@usace.army.mil*, or telephone at 916-557-6639. For more information regarding our program, please visit our website at *www.spk.usace.army.mil/Missions/Regulatory.aspx*.

Sincerely,

Nancy Arcady Haley

Nancy Arcady Haley
Chief, California North Branch

Enclosures

cc: (w/o encls)

Paul Jones, U.S. Environmental Protection Agency, Region 9, Jones.Paul@eps.gov
CA Department of Fish and Wildlife, North Region, askregion1@dfg.ca.gov
Danni Berchtold, CA Regional Water Quality Control Board, DBerchtold@waterboards.ca.gov
Robert Meade, California Department of Transportation, Robert.meade@dot.ca.gov
Matt Mitchell, California Department of Transportation, matt.mitchell@dot.ca.gov

COMPLIANCE CERTIFICATION

Permit File Name: I-5 Sidehill Viaduct Replacement (EA 02-0E090, PM 29.5 to 30.5)

Permit File Number: SPK-2014-00648

Nationwide Permit Number: 14

Permittee: California Department of Transportation
North Region, District 2
1657 Riverside Drive, MS#93
Redding, California 96001

County: Shasta

Date of Verification: August 7, 2014

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
1325 J Street, Room 1350
Sacramento, California 95814-2922
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.

Signature of Permittee　　　　Date

(Enclosure 1)

Nationwide Permit Summary

U S Army Corps of Engineers
Sacramento District

33 CFR Part 330; Issuance of Nationwide Permits – March 19, 2012

14. Linear Transportation Projects. Activities required for the construction, expansion, modification, or improvement of linear transportation projects (e.g., roads, highways, railways, trails, airport runways, and taxiways) in waters of the United States. For linear transportation projects in non-tidal waters, the discharge cannot cause the loss of greater than 1/2-acre of waters of the United States. For linear transportation projects in tidal waters, the discharge cannot cause the loss of greater than 1/3-acre of waters of the United States. Any stream channel modification, including bank stabilization, is limited to the minimum necessary to construct or protect the linear transportation project; such modifications must be in the immediate vicinity of the project.

This NWP also authorizes temporary structures, fills, and work necessary to construct the linear transportation project. Appropriate measures must be taken to maintain normal downstream flows and minimize flooding to the maximum extent practicable, when temporary structures, work, and discharges, including cofferdams, are necessary for construction activities, access fills, or dewatering of construction sites. Temporary fills must consist of materials, and be placed in a manner, that will not be eroded by expected high flows. Temporary fills must be removed in their entirety and the affected areas returned to pre-construction elevations. The areas affected by temporary fills must be revegetated, as appropriate.

This NWP cannot be used to authorize non-linear features commonly associated with transportation projects, such as vehicle maintenance or storage buildings, parking lots, train stations, or aircraft hangars.

Notification: The permittee must submit a pre-construction notification to the district engineer prior to commencing the activity if: (1) the loss of waters of the United States exceeds 1/10-acre; or (2) there is a discharge in a special aquatic site, including wetlands. (See general condition 31.) (Sections 10 and 404)

Note: Some discharges for the construction of farm roads or forest roads, or temporary roads for moving mining equipment, may qualify for an exemption under Section 404(f) of the Clean Water Act (see 33 CFR 323.4).

A. **Regional Conditions**

1. **Regional Conditions for California, excluding the Tahoe Basin**

http://www.spk.usace.army.mil/Portals/12/documents/regulatory/nwp/2012_nwps/2012-NWP-RC-CA.pdf

2. **Regional Conditions for Nevada, including the Tahoe Basin**

http://www.spk.usace.army.mil/Portals/12/documents/regulatory/nwp/2012_nwps/2012-NWP-RC-NV.pdf

3. **Regional Conditions for Utah**

http://www.spk.usace.army.mil/Portals/12/documents/regulatory/nwp/2012_nwps/2012-NWP-RC-UT.pdf

4. **Regional Conditions for Colorado.**

http://www.spk.usace.army.mil/Portals/12/documents/regulatory/nwp/2012_nwps//2012-NWP-RC-CO.pdf

B. **Nationwide Permit General Conditions**

Note: To qualify for NWP authorization, the prospective permittee must comply with the following general conditions, as applicable, in addition to any regional or case-specific conditions imposed by the division engineer or district engineer. Prospective permittees should contact the appropriate Corps district office to determine if regional conditions have been imposed on an NWP. Prospective permittees should also contact the appropriate Corps district office to determine the status of Clean Water Act Section 401 water quality certification and/or Coastal Zone Management Act consistency for an NWP. Every person who may wish to obtain permit authorization under one or more NWPs, or who is currently relying on an existing or prior permit authorization under one or more NWPs, has been and is on notice that all of the provisions of 33 CFR §§ 330.1 through 330.6 apply to every NWP authorization. Note especially 33 CFR § 330.5 relating to the modification, suspension, or revocation of any NWP authorization.

☐ 1. **Navigation.**

☐ (a) No activity may cause more than a minimal adverse effect on navigation.

☐ (b) Any safety lights and signals prescribed by the U.S. Coast Guard, through regulations or otherwise, must be installed and maintained at the permittee's expense on authorized facilities in navigable waters of the United States.

☐ (c) The permittee understands and agrees that, if future operations by the United States require the removal, relocation, or other alteration, of the structure or work herein authorized, or if, in the opinion of the Secretary of the Army or his authorized representative, said structure or work shall cause unreasonable obstruction to the free navigation of the navigable waters, the permittee will be required, upon due notice from the Corps of Engineers, to remove, relocate, or alter the structural work or obstructions caused thereby, without expense to the United States. No claim shall be made against the United States on account of any such removal or alteration.

☐ 2. **Aquatic Life Movements.** No activity may substantially disrupt the necessary life cycle movements of those species of aquatic life indigenous to the waterbody, including those species that normally migrate through the area, unless the activity's primary purpose is to impound water. All permanent and temporary crossings of waterbodies shall be suitably culverted, bridged, or otherwise designed and constructed to maintain low flows to sustain the movement of those aquatic species.

☐ 3. **Spawning Areas.** Activities in spawning areas during spawning seasons must be avoided to the maximum extent practicable. Activities that result in the physical destruction (e.g., through excavation, fill, or downstream smothering by substantial turbidity) of an important spawning area are not authorized.

☐ 4. **Migratory Bird Breeding Areas.** Activities in waters of the United States that serve as breeding areas for migratory birds must be avoided to the maximum extent practicable.

☐ 5. **Shellfish Beds.** No activity may occur in areas of concentrated shellfish populations, unless the activity is directly related to a shellfish harvesting activity authorized by NWPs 4 and 48, or is a shellfish seeding or habitat restoration activity authorized by NWP 27.

☐ 6. **Suitable Material.** No activity may use unsuitable material (e.g., trash, debris, car bodies, asphalt, etc.). Material used for construction or discharged must be free from toxic pollutants in toxic amounts (see Section 307 of the Clean Water Act).

☐ 7. **Water Supply Intakes.** No activity may occur in the proximity of a public water supply intake, except where the activity is for the repair or improvement of public water supply intake structures or adjacent bank stabilization.

☐ 8. **Adverse Effects From Impoundments.** If the activity creates an impoundment of water, adverse effects to the aquatic system due to accelerating the passage of water, and/or restricting its flow must be minimized to the maximum extent practicable.

☐ 9. **Management of Water Flows.** To the maximum extent practicable, the pre-construction course, condition, capacity, and location of open waters must be maintained for each activity, including stream channelization and storm water management activities, except as provided below. The activity must be constructed to withstand expected high flows. The activity must not restrict or impede the passage of normal or high flows, unless the primary purpose of the activity is to impound water or manage high flows. The activity may alter the pre-construction course, condition, capacity, and location of open waters if it benefits the aquatic environment (e.g., stream restoration or relocation activities).

☐ 10. **Fills Within 100-Year Floodplains.** The activity must comply with applicable FEMA-approved state or local floodplain management requirements.

☐ 11. **Equipment.** Heavy equipment working in wetlands or mudflats must be placed on mats, or other measures must be taken to minimize soil disturbance.

☐ 12. **Soil Erosion and Sediment Controls.** Appropriate soil erosion and sediment controls must be used and maintained in effective operating condition during construction, and all exposed soil and other fills, as well as any work below the ordinary high water mark or high tide line, must be permanently stabilized at the earliest practicable date. Permittees are encouraged to perform work within waters of the United States during periods of low-flow or no-flow.

☐ 13. **Removal of Temporary Fills.** Temporary fills must be removed in their entirety and the affected areas returned to pre-construction elevations. The affected areas must be revegetated, as appropriate.

☐ 14. **Proper Maintenance.** Any authorized structure or fill shall be properly maintained, including maintenance to ensure public safety and compliance with applicable NWP general conditions, as well as any activity-specific conditions added by the district engineer to an NWP authorization.

☐ 15. **Single and Complete Project.** The activity must be a single and complete project. The same NWP cannot be used more than once for the same single and complete project.

☐ 16. **Wild and Scenic Rivers.** No activity may occur in a component of the National Wild and Scenic River System, or in a river officially designated by Congress as a "study river" for possible inclusion in the system while the river is in an official study status, unless the appropriate Federal agency with direct management responsibility for such river, has determined in writing that the proposed activity will not adversely affect the Wild and Scenic River designation or study status. Information on Wild and Scenic Rivers may be obtained from the appropriate Federal land management agency responsible for the designated Wild and Scenic River or study river (e.g., National Park Service, U.S. Forest Service, Bureau of Land Management, U.S. Fish and Wildlife Service).

☐ 17. **Tribal Rights.** No activity or its operation may impair reserved tribal rights, including, but not limited to, reserved water rights and treaty fishing and hunting rights.

☐ 18. **Endangered Species.**

☐ (a) No activity is authorized under any NWP which is likely to directly or indirectly jeopardize the continued existence of a threatened or endangered species or a species proposed for such designation, as identified under the Federal Endangered Species Act (ESA), or which will directly or indirectly destroy or adversely modify the critical habitat of such species. No activity is authorized under any NWP which "may affect" a listed species or critical habitat, unless Section 7 consultation addressing the effects of the proposed activity has been completed.

☐ (b) Federal agencies should follow their own procedures for complying with the requirements of the ESA. Federal permittees must provide the district engineer with the appropriate documentation to demonstrate compliance with those requirements. The district engineer will review the documentation and determine whether it is sufficient to address ESA compliance for the NWP activity, or whether additional ESA consultation is necessary.

☐ (c) Non-federal permittees must submit a pre-construction notification to the district engineer if any listed species or designated critical habitat might be affected or is in the vicinity of the project, or if the project is located in designated critical habitat, and shall not begin work on the activity until notified by the district engineer that the requirements of the ESA have been satisfied and that the activity is authorized. For activities that might affect Federally-listed endangered or threatened species or designated critical habitat, the pre-construction notification must include the name(s) of the endangered or threatened species that might be affected by the proposed work or that utilize the designated critical habitat that might be affected by the proposed work. The district engineer will determine whether the proposed activity "may affect" or will have "no effect" to listed species and designated critical habitat and will notify the non-Federal applicant of the Corps' determination within 45 days of receipt of a complete pre-construction notification. In cases where the non-Federal applicant has identified listed species or critical habitat that might be affected or is in the vicinity of the project, and has so notified the Corps, the applicant shall not begin work until the Corps has provided notification the proposed activities will have "no effect" on listed species or critical habitat, or until Section 7 consultation has been completed. If the non-Federal applicant has not heard back from the Corps within 45 days, the applicant must still wait for notification from the Corps.

☐ (d)As a result of formal or informal consultation with the FWS or NMFS the district engineer may add species-specific regional endangered species conditions to the NWPs.

☐ (e) Authorization of an activity by a NWP does not authorize the "take" of a threatened or endangered species as defined under the ESA. In the absence of separate authorization (e.g., an ESA Section 10 Permit, a Biological Opinion with "incidental take" provisions, etc.) from the U.S. FWS or the NMFS, The Endangered Species Act prohibits any person subject to the jurisdiction of the United States to take a listed species, where "take" means to harass, harm, pursue, hunt, shoot, wound, kill, trap, capture, or collect, or to attempt to engage in any such conduct. The word "harm" in the definition of "take" means an act which actually kills or injures wildlife. Such an act may include significant habitat modification or degradation where it actually kills or injures wildlife by significantly impairing essential behavioral patterns, including breeding, feeding or sheltering.

☐ (f) Information on the location of threatened and endangered species and their critical habitat can be obtained directly from the offices of the U.S. FWS and NMFS or their world wide web pages at http://www.fws.gov/ or http://www.fws.gov/ipac and http://www.noaa.gov/fisheries.html respectively.

☐ 19. **Migratory Birds and Bald and Golden Eagles.** The permittee is responsible for obtaining any "take" permits required under the U.S. Fish and Wildlife Service's regulations governing compliance with the Migratory Bird Treaty Act or the Bald and Golden Eagle Protection Act. The permittee should contact the appropriate local office of the U.S. Fish and Wildlife Service to determine if such "take" permits are required for a particular activity.

☐ 20. **Historic Properties.**

☐ (a) In cases where the district engineer determines that the activity may affect properties listed, or eligible for listing, in the National Register of Historic Places, the activity is not authorized, until the requirements of Section 106 of the National Historic Preservation Act (NHPA) have been satisfied.

☐ (b) Federal permittees should follow their own procedures for complying with the requirements of Section 106 of the National Historic Preservation Act. Federal permittees must provide the district engineer with the appropriate documentation to demonstrate compliance with those requirements. The district engineer will review the documentation and determine whether it is sufficient to address section 106 compliance for the NWP activity, or whether additional section 106 consultation is necessary.

☐ (c) Non-federal permittees must submit a pre-construction notification to the district engineer if the authorized activity may have the potential to cause effects to any historic properties listed on, determined to be eligible for listing on, or potentially eligible for listing on the National Register of Historic Places, including previously unidentified properties. For such activities, the pre-construction notification must state which historic properties may be affected by the proposed work or include a vicinity map indicating the location of the historic properties or the potential for the presence of historic properties. Assistance regarding information on the location of or potential for the presence of historic resources can be sought from the State Historic Preservation Officer or Tribal Historic Preservation Officer, as appropriate, and the National Register of Historic Places (see 33 CFR 330.4(g)). When reviewing pre-construction notifications, district engineers will comply with the current procedures for addressing the requirements of Section 106 of the National Historic Preservation Act. The district engineer shall make a reasonable and good faith effort to carry out appropriate identification efforts, which may include background research, consultation, oral history interviews, sample field investigation, and field survey. Based on the information submitted and these efforts, the district engineer shall determine whether the proposed activity has the potential to cause an effect on the historic properties. Where the non-Federal applicant has identified historic properties on which the activity may have the potential to cause effects and so notified the Corps, the non-Federal applicant shall not begin the activity until notified by the district engineer either that the activity has no potential to cause effects or that consultation under Section 106 of the NHPA has been completed.

☐ (d) The district engineer will notify the prospective permittee within 45 days of receipt of a complete pre-construction notification whether NHPA Section 106 consultation is required. Section 106 consultation is not required when the Corps determines that the activity does not have the potential to cause effects on historic properties (see 36 CFR §800.3(a)). If NHPA section 106 consultation is required and will occur, the district engineer will notify the non- Federal applicant that he or she cannot begin work until Section 106 consultation is completed. If the non-Federal applicant has not heard back from the Corps within 45 days, the applicant must still wait for notification from the Corps.

☐ (e) Prospective permittees should be aware that section 110k of the NHPA (16 U.S.C. 470h-2(k)) prevents the Corps from granting a permit or other assistance to an applicant who, with intent to avoid the requirements of Section 106 of the NHPA, has intentionally significantly adversely affected a historic property to which the permit would relate, or having legal power to prevent it, allowed such significant adverse effect to occur, unless the Corps, after consultation with the Advisory Council on Historic Preservation (ACHP), determines that circumstances justify granting such assistance despite the adverse effect created or permitted by the applicant. If circumstances justify granting the assistance, the Corps is required to notify the ACHP and provide documentation specifying the circumstances, the degree of damage to the integrity of any historic properties affected, and proposed mitigation. This documentation must include any views obtained from the applicant, SHPO/THPO, appropriate Indian tribes if the undertaking occurs on or affects historic properties on tribal lands or affects properties of interest to those tribes, and other parties known to have a legitimate interest in the impacts to the permitted activity on historic properties.

☐ 21. **Discovery of Previously Unknown Remains and Artifacts.** If you discover any previously unknown historic, cultural or archeological remains and artifacts while accomplishing the activity authorized by this permit, you must immediately notify the district engineer of what you have found, and to the maximum extent practicable, avoid construction activities that may affect the remains and artifacts until the required coordination has been completed. The district engineer will initiate the Federal, Tribal and state coordination required to determine if the items or remains warrant a recovery effort or if the site is eligible for listing in the National Register of Historic Places.

☐ **22. Designated Critical Resource Waters.** Critical resource waters include, NOAA-managed marine sanctuaries and marine monuments, and National Estuarine Research Reserves. The district engineer may designate, after notice and opportunity for public comment, additional waters officially designated by a state as having particular environmental or ecological significance, such as outstanding national resource waters or state natural heritage sites. The district engineer may also designate additional critical resource waters after notice and opportunity for public comment.

☐ (a) Discharges of dredged or fill material into waters of the United States are not authorized by NWPs 7, 12, 14, 16, 17, 21, 29, 31, 35, 39, 40, 42, 43, 44, 49, 50, 51, and 52 for any activity within, or directly affecting, critical resource waters, including wetlands adjacent to such waters.

☐ (b) For NWPs 3, 8, 10, 13, 15, 18, 19, 22, 23, 25, 27, 28, 30, 33, 34, 36, 37, and 38, notification is required in accordance with general condition 31, for any activity proposed in the designated critical resource waters including wetlands adjacent to those waters. The district engineer may authorize activities under these NWPs only after it is determined that the impacts to the critical resource waters will be no more than minimal.

☐ **23. Mitigation.** The district engineer will consider the following factors when determining appropriate and practicable mitigation necessary to ensure that adverse effects on the aquatic environment are minimal:

☐ (a) The activity must be designed and constructed to avoid and minimize adverse effects, both temporary and permanent, to waters of the United States to the maximum extent practicable at the project site (i.e., on site).

☐ (b) Mitigation in all its forms (avoiding, minimizing, rectifying, reducing, or compensating for resource losses) will be required to the extent necessary to ensure that the adverse effects to the aquatic environment are minimal.

☐ (c) Compensatory mitigation at a minimum one-for-one ratio will be required for all wetland losses that exceed 1/10-acre and require pre-construction notification, unless the district engineer determines in writing that either some other form of mitigation would be more environmentally appropriate or the adverse effects of the proposed activity are minimal, and provides a project-specific waiver of this requirement. For wetland losses of 1/10-acre or less that require pre-construction notification, the district engineer may determine on a case-by-case basis that compensatory mitigation is required to ensure that the activity results in minimal adverse effects on the aquatic environment. Compensatory mitigation projects provided to offset losses of aquatic resources must comply with the applicable provisions of 33 CFR part 332.

☐ (1) The prospective permittee is responsible for proposing an appropriate compensatory mitigation option if compensatory mitigation is necessary to ensure that the activity results in minimal adverse effects on the aquatic environment.

☐ (2) Since the likelihood of success is greater and the impacts to potentially valuable uplands are reduced, wetland restoration should be the first compensatory mitigation option considered.

☐ (3) If permittee-responsible mitigation is the proposed option, the prospective permittee is responsible for submitting a mitigation plan. A conceptual or detailed mitigation plan may be used by the district engineer to make the decision on the NWP verification request, but a final mitigation plan that addresses the applicable requirements of 33 CFR 332.4(c)(2) – (14) must be approved by the district engineer before the permittee begins work in waters of the United States, unless the district engineer determines that prior approval of the final mitigation plan is not practicable or not necessary to ensure timely completion of the required compensatory mitigation (see 33 CFR 332.3(k)(3)).

☐ (4) If mitigation bank or in-lieu fee program credits are the proposed option, the mitigation plan only needs to address the baseline conditions at the impact site and the number of credits to be provided.

☐ (5) Compensatory mitigation requirements (e.g., resource type and amount to be provided as compensatory mitigation, site protection, ecological performance standards, monitoring requirements) may be addressed through conditions added to the NWP authorization, instead of components of a compensatory mitigation plan.

☐ (d) For losses of streams or other open waters that require pre-construction notification, the district engineer may require compensatory mitigation, such as stream rehabilitation, enhancement, or preservation, to ensure that the activity results in minimal adverse effects on the aquatic environment.

☐ (e) Compensatory mitigation will not be used to increase the acreage losses allowed by the acreage limits of the NWPs. For example, if an NWP has an acreage limit of 1/2-acre, it cannot be used to authorize any project resulting in the loss of greater than 1/2-acre of waters of the United States, even if compensatory mitigation is provided that replaces or restores some of the lost waters. However, compensatory mitigation can and should be used, as necessary, to ensure that a project already meeting the established acreage limits also satisfies the minimal impact requirement associated with the NWPs.

☐ (f) Compensatory mitigation plans for projects in or near streams or other open waters will normally include a requirement for the restoration or establishment, maintenance, and legal protection (e.g., conservation easements) of riparian areas next to open waters. In some cases, riparian areas may be the only compensatory mitigation required. Riparian areas should consist of native species. The width of the required riparian area will address documented water quality or aquatic habitat loss concerns. Normally, the riparian area will be 25 to 50 feet wide on each side of the stream, but the district engineer may require slightly wider riparian areas to address documented water quality or habitat loss concerns. If it is not possible to establish a riparian area on both sides of a stream, or if the waterbody is a lake or coastal waters, then restoring or establishing a riparian area along a single bank or shoreline may be sufficient. Where both wetlands and open waters exist on the project site, the district engineer will determine the appropriate compensatory mitigation (e.g., riparian areas and/or wetlands compensation) based on what is best for the aquatic environment on a watershed basis. In cases where riparian areas are determined to be the most appropriate form of compensatory mitigation, the district engineer may waive or reduce the requirement to provide wetland compensatory mitigation for wetland losses.

☐ (g) Permittees may propose the use of mitigation banks, in-lieu fee programs, or separate permittee-responsible mitigation. For activities resulting in the loss of marine or estuarine resources, permittee-responsible compensatory mitigation may be environmentally preferable if there are no mitigation banks or in-lieu fee programs in the area that have marine or estuarine credits available for sale or transfer to the permittee. For permittee-responsible mitigation, the special conditions of the NWP verification must clearly indicate the party or parties responsible for the implementation and performance of the compensatory mitigation project, and, if required, its long-term management.

(h) Where certain functions and services of waters of the United States are permanently adversely affected, such as the conversion of a forested or scrub-shrub wetland to a herbaceous wetland in a permanently maintained utility line right-of-way, mitigation may be required to reduce the adverse effects of the project to the minimal level.

☐ **24. Safety of Impoundment Structures.** To ensure that all impoundment structures are safely designed, the district engineer may require non-Federal applicants to demonstrate that the structures comply with established state dam safety criteria or have been designed by qualified persons. The district engineer may also require documentation that the design has been independently reviewed by similarly qualified persons, and appropriate modifications made to ensure safety.

☐ **25. Water Quality**. Where States and authorized Tribes, or EPA where applicable, have not previously certified compliance of an NWP with CWA Section 401, individual 401 Water Quality Certification must be obtained or waived (see 33 CFR 330.4(c)). The district engineer or State or Tribe may require additional water quality management measures to ensure that the authorized activity does not result in more than minimal degradation of water quality.

☐ **26. Coastal Zone Management.** In coastal states where an NWP has not previously received a state coastal zone management consistency concurrence, an individual state coastal zone management consistency concurrence must be obtained, or a presumption of concurrence must occur (see 33 CFR 330.4(d)). The district engineer or a State may require additional measures to ensure that the authorized activity is consistent with state coastal zone management requirements.

☐ **27. Regional and Case-By-Case Conditions.** The activity must comply with any regional conditions that may have been added by the Division Engineer (see 33 CFR 330.4(e)) and with any case specific conditions added by the Corps or by the state, Indian Tribe, or U.S. EPA in its section 401 Water Quality Certification, or by the state in its Coastal Zone Management Act consistency determination.

☐ **28. Use of Multiple Nationwide Permits**. The use of more than one NWP for a single and complete project is prohibited, except when the acreage loss of waters of the United States authorized by the NWPs does not exceed the acreage limit of the NWP with the highest specified acreage limit. For example, if a road crossing over tidal waters is constructed under NWP 14, with associated bank stabilization authorized by NWP 13, the maximum acreage loss of waters of the United States for the total project cannot exceed 1/3-acre.

☐ **29. Transfer of Nationwide Permit Verifications**. If the permittee sells the property associated with a nationwide permit verification, the permittee may transfer the nationwide permit verification to the new owner by submitting a letter to the appropriate Corps district office to validate the transfer. A copy of the nationwide permit verification must be attached to the letter, and the letter must contain the following statement and signature:

"When the structures or work authorized by this nationwide permit are still in existence at the time the property is transferred, the terms and conditions of this nationwide permit, including any special conditions, will continue to be binding on the new owner(s) of the property. To validate the transfer of this nationwide permit and the associated liabilities associated with compliance with its terms and conditions, have the transferee sign and date below."

(Transferee)

(Date)

☐ **30. Compliance Certification**. Each permittee who receives an NWP verification letter from the Corps must provide a signed certification documenting completion of the authorized activity and any required compensatory mitigation. The success of any required permittee responsible mitigation, including the achievement of ecological performance standards, will be addressed separately by the district engineer. The Corps will provide the permittee the certification document with the NWP verification letter. The certification document will include:

☐ (a) A statement that the authorized work was done in accordance with the NWP authorization, including any general, regional, or activity-specific conditions;

☐ (b) A statement that the implementation of any required compensatory mitigation was completed in accordance with the permit conditions. If credits from a mitigation bank or in-lieu fee program are used to satisfy the compensatory mitigation requirements, the certification must include the documentation required by 33 CFR 332.3(l)(3) to confirm that the permittee secured the appropriate number and resource type of credits; and

☐ (c) The signature of the permittee certifying the completion of the work and mitigation.

☐ **31. Pre-Construction Notification.**

☐ (a) **Timing**. Where required by the terms of the NWP, the prospective permittee must notify the district engineer by submitting a pre-construction notification (PCN) as early as possible. The district engineer must determine if the PCN is complete within 30 calendar days of the date of receipt and, if the PCN is determined to be incomplete, notify the prospective permittee within that 30 day period to request the additional information necessary to make the PCN complete. The request must specify the information needed to make the PCN complete. As a general rule, district engineers will request additional information necessary to make the PCN complete only once. However, if the prospective permittee does not provide all of the requested information, then the district engineer will notify the prospective permittee that the PCN is still incomplete and the PCN review process will not commence until all of the requested information has been received by the district engineer. The prospective permittee shall not begin the activity until either:

☐ (1) He or she is notified in writing by the district engineer that the activity may proceed under the NWP with any special conditions imposed by the district or division engineer; or

☐ (2) 45 calendar days have passed from the district engineer's receipt of the complete PCN and the prospective permittee has not received written notice from the district or division engineer. However, if the permittee was required to notify the Corps pursuant to general condition 18 that listed species or critical habitat might be affected or in the vicinity of the project, or to notify the Corps pursuant to general condition 20 that the activity may have the potential to cause effects to historic properties, the permittee cannot begin the activity until receiving written notification from the Corps that there is "no effect" on listed species or "no potential to cause effects" on historic properties, or that any consultation required under Section 7 of the Endangered Species Act (see 33 CFR 330.4(f)) and/or Section 106 of the National Historic Preservation (see 33 CFR 330.4(g)) has been completed. Also, work cannot begin under NWPs 21, 49, or 50 until the permittee has received written approval from the Corps. If the proposed activity requires a written waiver to exceed specified limits of an NWP, the permittee may not begin the activity until the district engineer issues the waiver. If the district or division engineer notifies the permittee in writing that an individual permit is required within 45 calendar days of receipt of a complete PCN, the permittee cannot begin the activity until an individual permit has been obtained. Subsequently, the permittee's right to proceed under the NWP may be modified, suspended, or revoked only in accordance with the procedure set forth in 33 CFR 330.5(d)(2)..

☐ (b) <u>Contents of Pre-Construction Notification</u>: The PCN must be in writing and include the following information:

☐ (1) Name, address and telephone numbers of the prospective permittee;

☐ (2) Location of the proposed project;

☐ (3) A description of the proposed project; the project's purpose; direct and indirect adverse environmental effects the project would cause, including the anticipated amount of loss of water of the United States expected to result from the NWP activity, in acres, linear feet, or other appropriate unit of measure; any other NWP(s), regional general permit(s), or individual permit(s) used or intended to be used to authorize any part of the proposed project or any related activity. The description should be sufficiently detailed to allow the district engineer to determine that the adverse effects of the project will be minimal and to determine the need for compensatory mitigation. Sketches should be provided when necessary to show that the activity complies with the terms of the NWP. (Sketches usually clarify the project and when provided results in a quicker decision. Sketches should contain sufficient detail to provide an illustrative description of the proposed activity (e.g., a conceptual plan), but do not need to be detailed engineering plans);

☐ (4) The PCN must include a delineation of wetlands, other special aquatic sites, and other waters, such as lakes and ponds, and perennial, intermittent, and ephemeral streams, on the project site. Wetland delineations must be prepared in accordance with the current method required by the Corps. The permittee may ask the Corps to delineate the special aquatic sites and other waters on the project site, but there may be a delay if the Corps does the delineation, especially if the project site is large or contains many waters of the United States. Furthermore, the 45 day period will not start until the delineation has been submitted to or completed by the Corps, as appropriate;

☐ (5) If the proposed activity will result in the loss of greater than 1/10-acre of wetlands and a PCN is required, the prospective permittee must submit a statement describing how the mitigation requirement will be satisfied, or explaining why the adverse effects are minimal and why compensatory mitigation should not be required. As an alternative, the prospective permittee may submit a conceptual or detailed mitigation plan.

☐ (6) If any listed species or designated critical habitat might be affected or is in the vicinity of the project, or if the project is located in designated critical habitat, for non-Federal applicants the PCN must include the name(s) of those endangered or threatened species that might be affected by the proposed work or utilize the designated critical habitat that may be affected by the proposed work. Federal applicants must provide documentation demonstrating compliance with the Endangered Species Act; and

☐ (7) For an activity that may affect a historic property listed on, determined to be eligible for listing on, or potentially eligible for listing on, the National Register of Historic Places, for non-Federal applicants the PCN must state which historic property may be affected by the proposed work or include a vicinity map indicating the location of the historic property. Federal applicants must provide documentation demonstrating compliance with Section 106 of the National Historic Preservation Act.

☐ (c) Form of Pre-Construction Notification: he standard individual permit application form (Form ENG 4345) may be used, but the completed application form must clearly indicate that it is a PCN and must include all of the information required in paragraphs (b)(1) through (7) of this general condition. A letter containing the required information may also be used.

☐ (d) Agency Coordination:

☐ (1) The district engineer will consider any comments from Federal and state agencies concerning the proposed activity's compliance with the terms and conditions of the NWPs and the need for mitigation to reduce the project's adverse environmental effects to a minimal level.

☐ (2) For all NWP activities that require pre-construction notification and result in the loss of greater than 1/2-acre of waters of the United States, for NWP 21, 29, 39, 40, 42, 43, 44, 50, 51, and 52 activities that require pre-construction notification and will result in the loss of greater than 300 linear feet of intermittent and ephemeral stream bed, and for all NWP 48 activities that require pre-construction notification, the district engineer will immediately provide (e.g., via email, facsimile transmission, overnight mail, or other expeditious manner) a copy of the complete PCN to the appropriate Federal or state offices (U.S. FWS, state natural resource or water quality agency, EPA, State Historic Preservation Officer (SHPO) or Tribal Historic Preservation Office (THPO), and, if appropriate, the NMFS). With the exception of NWP 37, these agencies will have 10 calendar days from the date the material is transmitted to telephone or fax the district engineer notice that they intend to provide substantive, site-specific comments. The comments must explain why the agency believes the adverse effects will be more than minimal. If so contacted by an agency, the district engineer will wait an additional 15 calendar days before making a decision on the pre-construction notification. The district engineer will fully consider agency comments received within the specified time frame concerning the proposed activity's compliance with the terms and conditions of the NWPs, including the need for mitigation to ensure the net adverse environmental effects to the aquatic environment of the proposed activity are minimal. The district engineer will provide no response to the resource agency, except as provided below. The district engineer will indicate in the administrative record associated with each pre-construction notification that the resource agencies' concerns were considered. For NWP 37, the emergency watershed protection and rehabilitation activity may proceed immediately in cases where there is an unacceptable hazard to life or a significant loss of property or economic hardship will occur. The district engineer will consider any comments received to decide whether the NWP 37 authorization should be modified, suspended, or revoked in accordance with the procedures at 33 CFR 330.5.

☐ (3) In cases of where the prospective permittee is not a Federal agency, the district engineer will provide a response to NMFS within 30 calendar days of receipt of any Essential Fish Habitat conservation recommendations, as required by Section 305(b)(4)(B) of the Magnuson-Stevens Fishery Conservation and Management Act.

☐ (4) Applicants are encouraged to provide the Corps with either electronic files or multiple copies of pre-construction notifications to expedite agency coordination.

C. District Engineer's Decision

☐ 1. In reviewing the PCN for the proposed activity, the district engineer will determine whether the activity authorized by the NWP will result in more than minimal individual or cumulative adverse environmental effects or may be contrary to the public interest. For a linear project, this determination will include an evaluation of the individual crossings to determine whether they individually satisfy the terms and conditions of the NWP(s), as well as the cumulative effects caused by all of the crossings authorized by NWP. If an applicant requests a waiver of the 300 linear foot limit on impacts to intermittent or ephemeral streams or of an otherwise applicable limit, as provided for in NWPs 13, 21, 29, 36, 39, 40, 42, 43, 44, 50, 51 or 52, the district engineer will only grant the waiver upon a written determination that the NWP activity will result in minimal adverse effects. When making minimal effects determinations the district engineer will consider the direct and indirect effects caused by the NWP activity. The district engineer will also consider site specific factors, such as the environmental setting in the vicinity of the NWP activity, the type of resource that will be affected by the NWP activity, the functions provided by the aquatic resources that will be affected by the NWP activity, the degree or magnitude to which the aquatic resources perform those functions, the extent that aquatic resource functions will be lost as a result of the NWP activity (e.g., partial or complete loss), the duration of the adverse effects (temporary or permanent), the importance of the aquatic resource functions to the region (e.g., watershed or ecoregion), and mitigation required by the district engineer. If an appropriate functional assessment method is available and practicable to use, that assessment method may be used by the district engineer to assist in the minimal adverse effects determination. The district engineer may add case-specific special conditions to the NWP authorization to address site-specific environmental concerns.

☐ 2. If the proposed activity requires a PCN and will result in a loss of greater than 1/10- acre of wetlands, the prospective permittee should submit a mitigation proposal with the PCN. Applicants may also propose compensatory mitigation for projects with smaller impacts. The district engineer will consider any proposed compensatory mitigation the applicant has included in the proposal in determining whether the net adverse environmental effects to the aquatic environment of the proposed activity are minimal. The compensatory mitigation proposal may be either conceptual or detailed. If the district engineer determines that the activity complies with the terms and conditions of the NWP and that the adverse effects on the aquatic environment are minimal, after considering mitigation, the district engineer will notify the permittee and include any activity-specific conditions in the NWP verification the district engineer deems necessary. Conditions for compensatory mitigation requirements must comply with the appropriate provisions at 33 CFR 332.3(k). The district engineer must approve the final mitigation plan before the permittee commences work in waters of the United States, unless the district engineer determines that prior approval of the final mitigation plan is not practicable or not necessary to ensure timely completion of the required compensatory mitigation. If the prospective permittee elects to submit a compensatory mitigation plan with the PCN, the district engineer will expeditiously review the proposed compensatory mitigation plan. The district engineer must review the proposed compensatory mitigation plan within 45 calendar days of receiving a complete PCN and determine whether the proposed mitigation would ensure no more than minimal adverse effects on the aquatic environment. If the net adverse effects of the project on the aquatic environment (after consideration of the compensatory mitigation proposal) are determined by the district engineer to be minimal, the district engineer will provide a timely written response to the applicant. The response will state that the project can proceed under the terms and conditions of the NWP, including any activity-specific conditions added to the NWP authorization by the district engineer.

☐ 3. If the district engineer determines that the adverse effects of the proposed work are more than minimal, then the district engineer will notify the applicant either: (a) That the project does not qualify for authorization under the NWP and instruct the applicant on the procedures to seek authorization under an individual permit; (b) that the project is authorized under the NWP subject to the applicant's submission of a mitigation plan that would reduce the adverse effects on the aquatic environment to the minimal level; or (c) that the project is authorized under the NWP with specific modifications or conditions. Where the district engineer determines that mitigation is required to ensure no more than minimal adverse effects occur to the aquatic environment, the activity will be authorized within the 45-day PCN period, with activity-specific conditions that state the mitigation requirements. The authorization will include the necessary conceptual or detailed mitigation or a requirement that the applicant submit a mitigation plan that would reduce the adverse effects on the aquatic environment to the minimal level. When mitigation is required, no work in waters of the United States may occur until the district engineer has approved a specific mitigation plan or has determined that prior approval of a final mitigation plan is not practicable or not necessary to ensure timely completion of the required compensatory mitigation.

D. Further Information

1. District Engineers have authority to determine if an activity complies with the terms and conditions of an NWP.

2. NWPs do not obviate the need to obtain other federal, state, or local permits, approvals, or authorizations required by law.

3. NWPs do not grant any property rights or exclusive privileges.

4. NWPs do not authorize any injury to the property or rights of others.

5. NWPs do not authorize interference with any existing or proposed Federal project.

E. **Definitions**

Best management practices (BMPs): Policies, practices, procedures, or structures implemented to mitigate the adverse environmental effects on surface water quality resulting from development. BMPs are categorized as structural or non-structural.

Compensatory mitigation: The restoration (re-establishment or rehabilitation), establishment (creation), enhancement, and/or in certain circumstances preservation of aquatic resources for the purposes of offsetting unavoidable adverse impacts which remain after all appropriate and practicable avoidance and minimization has been achieved.

Currently serviceable: Useable as is or with some maintenance, but not so degraded as to essentially require reconstruction.

Direct effects: Effects that are caused by the activity and occur at the same time and place.

Discharge: The term "discharge" means any discharge of dredged or fill material.

Enhancement: The manipulation of the physical, chemical, or biological characteristics of an aquatic resource to heighten, intensify, or improve a specific aquatic resource function(s). Enhancement results in the gain of selected aquatic resource function(s), but may also lead to a decline in other aquatic resource function(s). Enhancement does not result in a gain in aquatic resource area.

Ephemeral stream: An ephemeral stream has flowing water only during, and for a short duration after, precipitation events in a typical year. Ephemeral stream beds are located above the water table year-round. Groundwater is not a source of water for the stream. Runoff from rainfall is the primary source of water for stream flow.

Establishment (creation): The manipulation of the physical, chemical, or biological characteristics present to develop an aquatic resource that did not previously exist at an upland site. Establishment results in a gain in aquatic resource area.

High Tide Line: The line of intersection of the land with the water's surface at the maximum height reached by a rising tide. The high tide line may be determined, in the absence of actual data, by a line of oil or scum along shore objects, a more or less continuous deposit of fine shell or debris on the foreshore or berm, other physical markings or characteristics, vegetation lines, tidal gages, or other suitable means that delineate the general height reached by a rising tide. The line encompasses spring high tides and other high tides that occur with periodic frequency but does not include storm surges in which there is a departure from the normal or predicted reach of the tide due to the piling up of water against a coast by strong winds such as those accompanying a hurricane or other intense storm.

Historic Property: Any prehistoric or historic district, site (including archaeological site), building, structure, or other object included in, or eligible for inclusion in, the National Register of Historic Places maintained by the Secretary of the Interior. This term includes artifacts, records, and remains that are related to and located within such properties. The term includes properties of traditional religious and cultural importance to an Indian tribe or Native Hawaiian organization and that meet the National Register criteria (36 CFR part 60).

Independent utility: A test to determine what constitutes a single and complete non-linear project in the Corps regulatory program. A project is considered to have independent utility if it would be constructed absent the construction of other projects in the project area. Portions of a multi-phase project that depend upon other phases of the project do not have independent utility. Phases of a project that would be constructed even if the other phases were not built can be considered as separate single and complete projects with independent utility.

Indirect effects: Effects that are caused by the activity and are later in time or farther removed in distance, but are still reasonably foreseeable.

Intermittent stream: An intermittent stream has flowing water during certain times of the year, when groundwater provides water for stream flow. During dry periods, intermittent streams may not have flowing water. Runoff from rainfall is a supplemental source of water for stream flow.

Loss of waters of the United States: Waters of the United States that are permanently adversely affected by filling, flooding, excavation, or drainage because of the regulated activity. Permanent adverse effects include permanent discharges of dredged or fill material that change an aquatic area to dry land, increase the bottom elevation of a waterbody, or change the use of a waterbody. The acreage of loss of waters of the United States is a threshold measurement of the impact to jurisdictional waters for determining whether a project may qualify for an NWP; it is not a net threshold that is calculated after considering compensatory mitigation that may be used to offset losses of aquatic functions and services. The loss of stream bed includes the linear feet of stream bed that is filled or excavated. Waters of the United States temporarily filled, flooded, excavated, or drained, but restored to pre-construction contours and elevations after construction, are not included in the measurement of loss of waters of the United States. Impacts resulting from activities eligible for exemptions under Section 404(f) of the Clean Water Act are not considered when calculating the loss of waters of the United States.

Non-tidal wetland: A non-tidal wetland is a wetland that is not subject to the ebb and flow of tidal waters. The definition of a wetland can be found at 33 CFR 328.3(b). Non-tidal wetlands contiguous to tidal waters are located landward of the high tide line (i.e., spring high tide line).

Open water: For purposes of the NWPs, an open water is any area that in a year with normal patterns of precipitation has water flowing or standing above ground to the extent that an ordinary high water mark can be determined. Aquatic vegetation within the area of standing or flowing water is either non-emergent, sparse, or absent. Vegetated shallows are considered to be open waters. Examples of "open waters" include rivers, streams, lakes, and ponds.

Ordinary High Water Mark: An ordinary high water mark is a line on the shore established by the fluctuations of water and indicated by physical characteristics, or by other appropriate means that consider the characteristics of the surrounding areas (see 33 CFR 328.3(e)).

Perennial stream: A perennial stream has flowing water year-round during a typical year. The water table is located above the stream bed for most of the year. Groundwater is the primary source of water for stream flow. Runoff from rainfall is a supplemental source of water for stream flow.

Practicable: Available and capable of being done after taking into consideration cost, existing technology, and logistics in light of overall project purposes.

Pre-construction notification: A request submitted by the project proponent to the Corps for confirmation that a particular activity is authorized by nationwide permit. The request may be a permit application, letter, or similar document that includes information about the proposed work and its anticipated environmental effects. Pre-construction notification may be required by the terms and conditions of a nationwide permit, or by regional conditions. A pre-construction notification may be voluntarily submitted in cases where pre-construction notification is not required and the project proponent wants confirmation that the activity is authorized by nationwide permit.

Preservation: The removal of a threat to, or preventing the decline of, aquatic resources by an action in or near those aquatic resources. This term includes activities commonly associated with the protection and maintenance of aquatic resources through the implementation of appropriate legal and physical mechanisms. Preservation does not result in a gain of aquatic resource area or functions.

Re-establishment: The manipulation of the physical, chemical, or biological characteristics of a site with the goal of returning natural/historic functions to a former aquatic resource. Re-establishment results in rebuilding a former aquatic resource and results in a gain in aquatic resource area and functions.

Rehabilitation: The manipulation of the physical, chemical, or biological characteristics of a site with the goal of repairing natural/historic functions to a degraded aquatic resource. Rehabilitation results in a gain in aquatic resource function, but does not result in a gain in aquatic resource area.

Restoration: The manipulation of the physical, chemical, or biological characteristics of a site with the goal of returning natural/historic functions to a former or degraded aquatic resource. For the purpose of tracking net gains in aquatic resource area, restoration is divided into two categories: re-establishment and rehabilitation.

Riffle and pool complex: Riffle and pool complexes are special aquatic sites under the 404(b)(1) Guidelines. Riffle and pool complexes sometimes characterize steep gradient sections of streams. Such stream sections are recognizable by their hydraulic characteristics. The rapid movement of water over a course substrate in riffles results in a rough flow, a turbulent surface, and high dissolved oxygen levels in the water. Pools are deeper areas associated with riffles. A slower stream velocity, a streaming flow, a smooth surface, and a finer substrate characterize pools.

Riparian areas: Riparian areas are lands adjacent to streams, lakes, and estuarine-marine shorelines. Riparian areas are transitional between terrestrial and aquatic ecosystems, through which surface and subsurface hydrology connects riverine, lacustrine, estuarine, and marine waters with their adjacent wetlands, non-wetland waters, or uplands. Riparian areas provide a variety of ecological functions and services and help improve or maintain local water quality. (See general condition 23.)

Shellfish seeding: The placement of shellfish seed and/or suitable substrate to increase shellfish production. Shellfish seed consists of immature individual shellfish or individual shellfish attached to shells or shell fragments (i.e., spat on shell). Suitable substrate may consist of shellfish shells, shell fragments, or other appropriate materials placed into waters for shellfish habitat.

Single and complete linear project: A linear project is a project constructed for the purpose of getting people, goods, or services from a point of origin to a terminal point, which often involves multiple crossings of one or more waterbodies at separate and distant locations. The term "single and complete project" is defined as that portion of the total linear project proposed or accomplished by one owner/developer or partnership or other association of owners/developers that includes all crossings of a single water of the United States (i.e., a single waterbody) at a specific location. For linear projects crossing a single or multiple waterbodies several times at separate and distant locations, each crossing is considered a single and complete project for purposes of NWP authorization. However, individual channels in a braided stream or river, or individual arms of a large, irregularly shaped wetland or lake, etc., are not separate waterbodies, and crossings of such features cannot be considered separately.

Single and complete non-linear project: For non-linear projects, the term "single and complete project" is defined at 33 CFR 330.2(i) as the total project proposed or accomplished by one owner/developer or partnership or other association of owners/developers. A single and complete non-linear project must have independent utility (see definition of "independent utility"). Single and complete non-linear projects may not be "piecemealed" to avoid the limits in an NWP authorization.

Stormwater management: Stormwater management is the mechanism for controlling stormwater runoff for the purposes of reducing downstream erosion, water quality degradation, and flooding and mitigating the adverse effects of changes in land use on the aquatic environment.

Stormwater management facilities: Stormwater management facilities are those facilities, including but not limited to, stormwater retention and detention ponds and best management practices, which retain water for a period of time to control runoff and/or improve the quality (i.e., by reducing the concentration of nutrients, sediments, hazardous substances and other pollutants) of stormwater runoff.

Stream bed: The substrate of the stream channel between the ordinary high water marks. The substrate may be bedrock or inorganic particles that range in size from clay to boulders. Wetlands contiguous to the stream bed, but outside of the ordinary high water marks, are not considered part of the stream bed.

Stream channelization: The manipulation of a stream's course, condition, capacity, or location that causes more than minimal interruption of normal stream processes. A channelized stream remains a water of the United States.

Structure: An object that is arranged in a definite pattern of organization. Examples of structures include, without limitation, any pier, boat dock, boat ramp, wharf, dolphin, weir, boom, breakwater, bulkhead, revetment, riprap, jetty, artificial island, artificial reef, permanent mooring structure, power transmission line, permanently moored floating vessel, piling, aid to navigation, or any other manmade obstacle or obstruction.

Tidal wetland: A tidal wetland is a wetland (i.e., water of the United States) that is inundated by tidal waters. The definitions of a wetland and tidal waters can be found at 33 CFR 328.3(b) and 33 CFR 328.3(f), respectively. Tidal waters rise and fall in a predictable and measurable rhythm or cycle due to the gravitational pulls of the moon and sun. Tidal waters end where the rise and fall of the water surface can no longer be practically measured in a predictable rhythm due to masking by other waters, wind, or other effects. Tidal wetlands are located channelward of the high tide line, which is defined at 33 CFR 328.3(d).

Vegetated shallows: Vegetated shallows are special aquatic sites under the 404(b)(1) Guidelines. They are areas that are permanently inundated and under normal circumstances have rooted aquatic vegetation, such as seagrasses in marine and estuarine systems and a variety of vascular rooted plants in freshwater systems.

Waterbody: For purposes of the NWPs, a waterbody is a jurisdictional water of the United States. If a jurisdictional wetland is adjacent – meaning bordering, contiguous, or neighboring – to a waterbody determined to be a water of the United States under 33 CFR 328.3(a)(1)-(6), that waterbody and its adjacent wetlands are considered together as a single aquatic unit (see 33 CFR 328.4(c)(2)). Examples of "waterbodies" include streams, rivers, lakes, ponds, and wetlands.

(Enclosure 2)

Final Sacramento District Nationwide Permit
Regional Conditions for California, excluding the Lake Tahoe Basin
(Effective March 19, 2012 until March 18, 2017)

1.* When pre-construction notification (PCN) is required, the permittee shall notify the U.S. Army Corps of Engineers, Sacramento District (Corps) in accordance with General Condition 31 using either the South Pacific Division Preconstruction Notification (PCN) Checklist or a signed application form (ENG Form 4345) with an attachment providing information on compliance with all of the General and Regional Conditions. In addition, the PCN shall include:

a. A written statement describing how the activity has been designed to avoid and minimize adverse effects, both temporary and permanent, to waters of the United States;

b. Drawings, including plan and cross-section views, clearly depicting the location, size and dimensions of the proposed activity, as well as the location of delineated waters of the U.S. on the site. The drawings shall contain a title block, legend and scale, amount (in cubic yards) and area (in acres) of fill in Corps jurisdiction, including both permanent and temporary fills/structures. The ordinary high water mark or, if tidal waters, the mean high water mark and high tide line, should be shown (in feet), based on National Geodetic Vertical Datum (NGVD) or other appropriate referenced elevation. All drawings for activities located within the boundaries of the Los Angeles District shall comply with the September 15, 2010 Special Public Notice: *Map and Drawing Standards for the Los Angeles District Regulatory Division*, (available on the Los Angeles District Regulatory Division website at: www.spl.usace.army.mil/regulatory/); and

c. Numbered and dated pre-project color photographs showing a representative sample of waters proposed to be impacted on the site, and all waters of the U.S. proposed to be avoided on and immediately adjacent to the project site. The compass angle and position of each photograph shall be identified on the plan-view drawing(s) required in subpart b of this Regional Condition.

2. For all Nationwide Permits (NWPs), the permittee shall submit a PCN in accordance with General Condition 31 and Regional Condition 1, in the following circumstances:

a. For all activities that would result in the discharge of fill material into any vernal pool;

b. For any activity in the Primary and Secondary Zones of the Legal Delta, the Sacramento River, the San Joaquin River, and the immediate tributaries of these waters;

c. For all crossings of perennial waters and intermittent waters;

d. For all activities proposed within 100 feet of the point of discharge of a known natural spring source, which is any location where ground water emanates from a point in the ground excluding seeps or other discharges which lack a defined channel; and

e.* For all activities located in areas designated as Essential Fish Habitat (EFH) by the Pacific Fishery Management Council (i.e., all tidally influenced areas - Federal Register dated March 12, 2007 (72 FR 11092)), in which case the PCN shall include an EFH assessment and extent of proposed impacts to EFH. Examples of EFH habitat assessments can be found at: http://www.swr.noaa.gov/efh.htm.

3. The permittee shall record the NWP verification with the Registrar of Deeds or other appropriate official charged with the responsibility for maintaining records of title to or interest in real property for areas (1) designated to be preserved as part of compensatory mitigation for authorized impacts, including any associated covenants or restrictions, or (2) where boat ramps or docks, marinas, piers, and permanently moored vessels will be constructed or placed in or adjacent to navigable waters. The recordation shall also include a map showing the surveyed location of the preserved area or authorized structure.

* Regional Condition developed jointly between Sacramento District, Los Angeles District, and San Francisco District.

4. For all waters of the U.S. proposed to be avoided on a site, unless determined to be impracticable by the Corps, the permittee shall:

a. Establish and maintain, in perpetuity, a preserve containing all avoided waters of the U.S. to ensure that the functions of the aquatic environment are protected;

b. Place all avoided waters of the U.S. and any upland buffers into a separate parcel prior to discharging dredge or fill material into waters of the U.S., and

c. Establish permanent legal protection for all preserve parcels, following Corps approval of the legal instrument;

If the Corps determines that it is impracticable to require permanent preservation of the avoided waters, additional mitigation may be required in order to compensate for indirect impacts to the waters of the U.S.

5. For all temporary fills, the PCN shall include a description of the proposed temporary fill, including the type and amount of material to be placed, the area proposed to be impacted, and the proposed plan for restoration of the temporary fill area to pre-project contours and conditions, including a plan for the re-vegetation of the temporary fill area, if necessary. In addition, the PCN shall include the reason(s) why avoidance of temporary impacts is not practicable.

In addition, for all activities resulting in temporary fill within waters of the U.S., the permittee shall:

a. Utilize material consisting of clean and washed gravel. For temporary fills within waters of the U.S. supporting anadromous fisheries, spawning quality gravel shall be used, where practicable, as determined by the Corps, after consultation with appropriate Federal and state fish and wildlife agencies;

b. Place a horizontal marker (e.g. fabric, certified weed free straw, etc.) to delineate the existing ground elevation of the waters temporarily filled during construction; and

c. Remove all temporary fill within 30 days following completion of construction activities.

6. In addition to the requirements of General Condition 2, unless determined to be impracticable by the Corps, the following criteria shall apply to all road crossings:

a.* For all activities in waters of the U.S. that are suitable habitat for Federally-listed fish species, the permittee shall design all road crossings to ensure that the passage and/or spawning of fish is not hindered. In these areas, the permittee shall employ bridge designs that span the stream or river, including pier- or pile-supported spans, or designs that use a bottomless arch culvert with a natural stream bed;

b. Road crossings shall be designed to ensure that no more than minor impacts would occur to fish and wildlife passage or expected high flows, following the criteria listed in Regional Condition 6(a). Culverted crossings that do not utilize a bottomless arch culvert with a natural stream bed may be authorized for waters that do not contain suitable habitat for Federally listed fish species, if it can be demonstrated and is specifically determined by the Corps, that such crossing will result in no more than minor impacts to fish and wildlife passage or expected high flows;

c. No construction activities shall occur within standing or flowing waters. For ephemeral or intermittent streams, this may be accomplished through construction during the dry season. In perennial streams, this may be accomplished through dewatering of the work area. Any proposed dewatering plans must be approved, in writing, by the Corps prior to commencement of construction activities; and

* Regional Condition developed jointly between Sacramento District, Los Angeles District, and San Francisco District.

d. All bank stabilization activities associated with a road crossing shall comply with Regional Condition 19.

In no case shall stream crossings result in a reduction in the pre-construction bankfull width or depth of perennial streams or negatively alter the flood control capacity of perennial streams.

7.* For activities in which the Corps designates another Federal agency as the lead for compliance with Section 7 of the Endangered Species Act (ESA) of 1973 as amended, pursuant to 50 CFR Part 402.07, Section 305(b)(4)(B) of the Magnuson-Stevens Fishery Conservation and Management Act (EFH), pursuant to 50 CFR 600.920(b) and/or Section 106 of the National Historic Preservation Act (NHPA) of 1966, as amended, pursuant to 36 CFR 800.2(a)(2), the lead Federal agency shall provide all relevant documentation to the Corps demonstrating any previous consultation efforts, as it pertains to the Corps Regulatory permit area (for Section 7 and EFH compliance) and the Corps Regulatory area of potential effect (APE) (for Section 106 compliance). For activities requiring a PCN, this information shall be submitted with the PCN. If the Corps does not designate another Federal agency as the lead for ESA, EFH and/or NHPA, the Corps will initiate consultation for compliance, as appropriate.

8. For all NWPs which require a PCN, the permittee shall submit the following additional information with the compliance certificate required under General Condition 30:

a. As-built drawings of the work conducted on the project site and any on-site and/or off-site compensatory mitigation, preservation, and/or avoidance area(s). The as-builts shall include a plan-view drawing of the location of the authorized work footprint (as shown on the permit drawings), with an overlay of the work as constructed in the same scale as the permit drawings. The drawing shall show all areas of ground disturbance, wetland impacts, structures, and the boundaries of any on-site and/or off-site mitigation or avoidance areas. Please note that any deviations from the work as authorized, which result in additional impacts to waters of the U.S., must be coordinated with the appropriate Corps office prior to impacts; and

b. Numbered and dated post-construction color photographs of the work conducted within a representative sample of the impacted waters of the U.S., and within all avoided waters of the U.S. on and immediately adjacent to the proposed project area. The compass angle and position of all photographs shall be similar to the pre-construction color photographs required in Regional Condition 1(c) and shall be identified on the plan-view drawing(s) required in subpart a of this Regional Condition.

9. For all activities requiring permittee responsible mitigation, the permittee shall develop and submit to the Corps for review and approval, a final comprehensive mitigation and monitoring plan for all permittee responsible mitigation prior to commencement of construction activities within waters of the U.S. The plan shall include the mitigation location and design drawings, vegetation plans, including target species to be planted, and final success criteria, presented in the format of the *Sacramento District's Habitat Mitigation and Monitoring Proposal Guidelines*, dated December 30, 2004, and in compliance with the requirements of 33 CFR 332.

10.* The permittee shall complete the construction of any compensatory mitigation required by special condition(s) of the NWP verification before or concurrent with commencement of construction of the authorized activity, except when specifically determined to be impracticable by the Corps. When mitigation involves use of a mitigation bank or in-lieu fee program, the permittee shall submit proof of payment to the Corps prior to commencement of construction of the authorized activity.

11. The permittee is responsible for all authorized work and ensuring that all contractors and workers are made aware and adhere to the terms and conditions of the permit authorization. The permittee shall ensure

* Regional Condition developed jointly between Sacramento District, Los Angeles District, and San Francisco District.

that a copy of the permit authorization and associated drawings are available and visible for quick reference at the site until all construction activities are completed.

12. The permittee shall clearly identify the limits of disturbance in the field with highly visible markers (e.g. construction fencing, flagging, silt barriers, etc.) prior to commencement of construction activities within waters of the U.S. The permittee shall maintain such identification properly until construction is completed and the soils have been stabilized. The permittee is prohibited from any activity (e.g. equipment usage or materials storage) that impacts waters of the U.S. outside of the permit limits (as shown on the permit drawings).

13. For all activities in which a PCN is required, the permittee shall notify the appropriate district office of the start date for the authorized work within 10 days prior to initiation of construction activities.

14. The permittee shall allow Corps representatives to inspect the authorized activity and any mitigation areas at any time deemed necessary to determine compliance with the terms and conditions of the NWP verification. The permittee will be notified in advance of an inspection.

15. For all activities located in the Mather Core Recovery Area in Sacramento County, as identified in the U.S. Fish and Wildlife Service's *Recovery Plan for Vernal Pool Ecosystems of California and Southern Oregon* dated December 15, 2005, NWPs 14, 18, 23, 29, 39, 40, 42, 43 and 44 are revoked from use in vernal pools that may contain habitat for Federally-listed threatened and/or endangered vernal pool species.

16. For activities located in the Primary or Secondary Zone of the Legal Delta, NWPs 29 and 39 are revoked.

17. For all activities within the Secondary Zone of the Legal Delta, the permittee shall conduct compensatory mitigation for unavoidable impacts within the Secondary Zone of the Legal Delta.

18. For NWP 12: Permittees shall ensure the construction of utility lines does not result in the draining of any water of the U.S., including wetlands. This may be accomplished through the use of clay blocks, bentonite, or other suitable material (as approved by the Corps) to seal the trench. For utility line trenches, during construction, the permittee shall remove and stockpile, separately, the top 6 – 12 inches of topsoil. Following installation of the utility line(s), the permittee shall replace the stockpiled topsoil on top and seed the area with native vegetation. The permittee shall submit a PCN for utility line activities in the following circumstances:

a. The utility line crossing would result in a discharge of dredged and/or fill material into perennial waters, intermittent waters, wetlands, mudflats, vegetated shallows, riffle and pool complexes, sanctuaries and refuges or coral reefs;

b. The utility line activity would result in a discharge of dredged and/or fill material into greater than 100 linear feet of ephemeral waters of the U.S.;

c. The utility line installation would include the construction of a temporary or permanent access road, substation or foundation within waters of the U.S.; or

d. The proposed activity would not involve the restoration of all utility line trenches to pre-project contours and conditions within 30 days following completion of construction activities.

19. For NWP 13 and 14: All bank stabilization activities shall involve either the sole use of native vegetation or other bioengineered design techniques (e.g. willow plantings, root wads, large woody debris, etc.), or a combination of hard-armoring (e.g. rip-rap) and native vegetation or bioengineered design

* Regional Condition developed jointly between Sacramento District, Los Angeles District, and San Francisco District.

techniques, unless specifically determined to be impracticable by the Corps. The permittee shall submit a PCN for any bank stabilization activity that involves hard-armoring or the placement of any non-vegetated or non-bioengineered technique below the ordinary high water mark or, if tidal waters, the high tide line of waters of the U.S. The request to utilize non-vegetated techniques must include information on why the sole use of vegetated techniques is not practicable.

20. For NWP 23: The permittee shall submit a PCN for all activities proposed for this NWP, in accordance with General Condition 31 and Regional Condition 1. The PCN shall include a copy of the signed Categorical Exclusion document and final agency determinations regarding compliance with ESA, EFH and NHPA, in accordance with General Conditions 18 and 20 and Regional Condition 7.

21. For NWP 27: The permittee shall submit a PCN for aquatic habitat restoration, establishment, and enhancement activities in the following circumstances:

a. The restoration, establishment or enhancement activity would result in a discharge of dredged and/or fill material into perennial waters, intermittent waters, wetlands, mudflats, vegetated shallows, riffle and pool complexes, sanctuaries and refuges or coral reefs; or

b. The restoration, establishment or enhancement activity would result in a discharge of dredged and/or fill material into greater than 100 linear feet of ephemeral waters of the U.S.

22. For NWPs 29 and 39: The channelization or relocation of intermittent or perennial drainages is not authorized, except when, as determined by the Corps, the relocation would result in a net increase in functions of the aquatic ecosystem within the watershed.

23.* Any requests to waive the 300 linear foot limitation for intermittent and ephemeral streams for NWPs 21, 29, 39, 40, 42, 43, 44, 50, 51 and 52, or to waive the 500 linear foot limitation along the bank for NWP 13, must include the following:

a. A narrative description of the stream. This should include known information on: volume and duration of flow; the approximate length, width, and depth of the waterbody and characteristics observed associated with an Ordinary High Water Mark (e.g. bed and bank, wrack line or scour marks); a description of the adjacent vegetation community and a statement regarding the wetland status of the adjacent areas (i.e. wetland, non-wetland); surrounding land use; water quality; issues related to cumulative impacts in the watershed, and; any other relevant information;

b. An analysis of the proposed impacts to the waterbody, in accordance with General Condition 31 and Regional Condition 1;

c. Measures taken to avoid and minimize losses to waters of the U.S., including other methods of constructing the proposed activity(s); and

d. A compensatory mitigation plan describing how the unavoidable losses are proposed to be offset, in accordance with 33 CFR 332.

24. For NWPs 29, 39, 40, 42, and 43: The permittee shall establish and maintain upland vegetated buffers in perpetuity, unless specifically determined to be impracticable by the Corps, next to all preserved open waters, streams and wetlands including created, restored, enhanced or preserved waters of the U.S., consistent with General Condition 23(f). Except in unusual circumstances, as determined by the Corps, vegetated buffers shall be at least 50 feet in width.

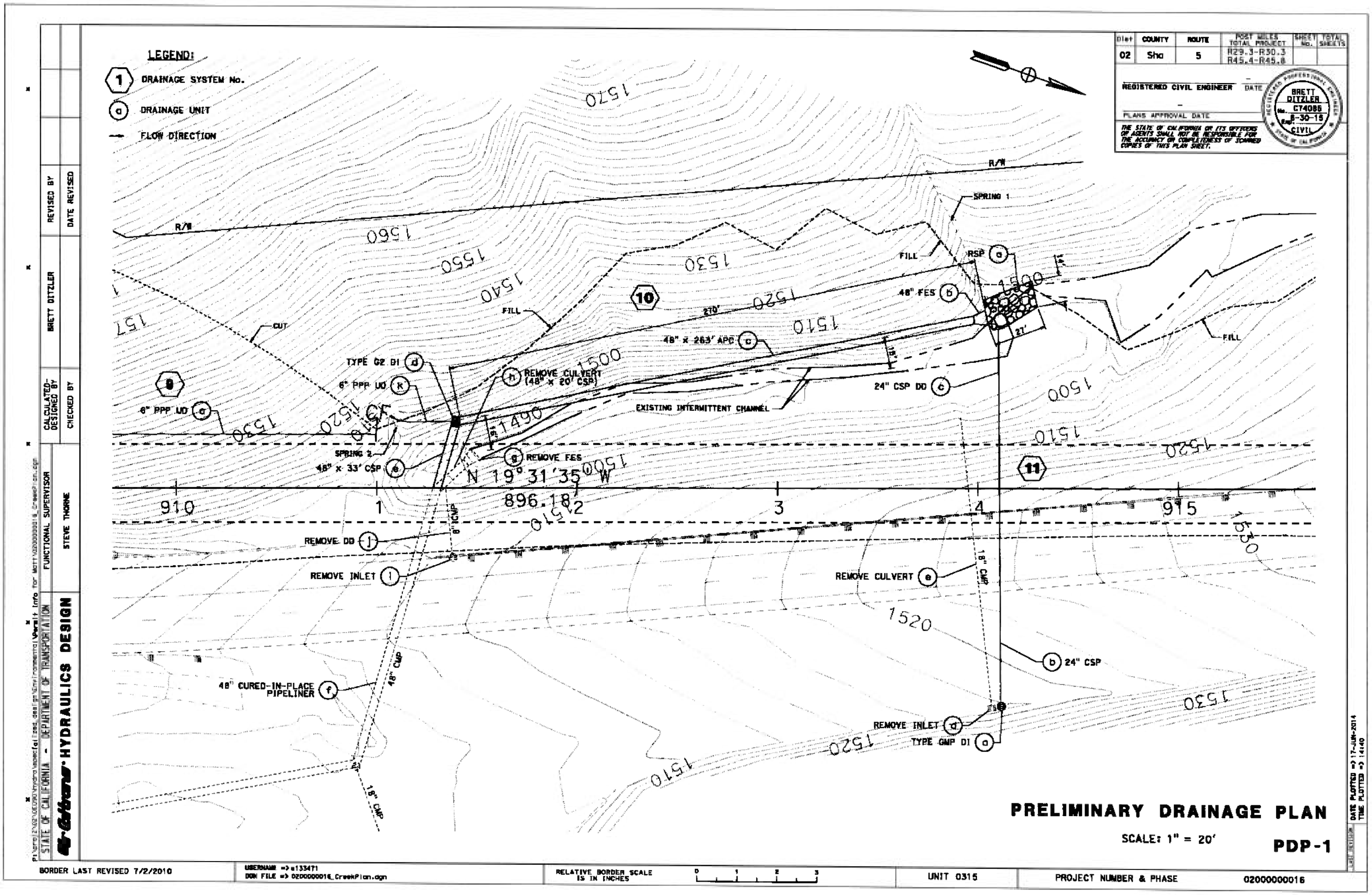
* Regional Condition developed jointly between Sacramento District, Los Angeles District, and San Francisco District.

25. For NWP 46: The discharge shall not cause the loss of greater than 0.5 acres of waters of the United States or the loss of more than 300 linear feet of ditch, unless specifically waived in writing by the Corps.

26. All NWPs except 3, 6, 20, 27, 32, and 38 are revoked for activities in histosols, fens, bogs and peatlands and in wetlands contiguous with fens. Fens are defined as slope wetlands with a histic epipedon that are hydrologically supported by groundwater. Fens are normally saturated throughout the growing season, although they may not be during drought conditions. For NWPs 3, 6, 20, 27, 32, and 38, the permittee shall submit a PCN to the Corps in accordance with General Condition 31 and Regional Condition 1. This condition does not apply to NWPs 1, 2, 8, 9, 10, 11, 24, 28, 35 or 36, as these NWPs either apply to Section 10 only activities or do not authorize impacts to special aquatic sites.

* Regional Condition developed jointly between Sacramento District, Los Angeles District, and San Francisco District.

(Enclosure 3)


LEGEND:
1 DRAINAGE SYSTEM No.
a DRAINAGE UNIT
FLOW DIRECTION
Dist 02 COUNTY Sha ROUTE 5 POST MILES TOTAL PROJECT R29.3-R30.3 R45.4-R45.8 SHEET No. TOTAL SHEETS
REGISTERED CIVIL ENGINEER DATE
PLANS APPROVAL DATE
THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.
BRETT DITZLER No. C74086 Exp. 6-30-15 CIVIL STATE OF CALIFORNIA
R/W
SPRING 1
RSP a
FILL
48" FES b
27'
14'
210'
48" x 263' APC c
24" CSP DD c
EXISTING INTERMITTENT CHANNEL
TYPE G2 DI d
6" PPP UD k
REMOVE CULVERT (48" x 20' CSP) h
6" PPP UD a
SPRING 2
48" x 33' CSP e
REMOVE FES g
CUT
N 19° 31' 35" W
896.18
910
1
2
3
4
915
REMOVE DD j
REMOVE INLET i
8" CMP
REMOVE CULVERT e
18" CMP
b 24" CSP
48" CURED-IN-PLACE PIPELINER f
48" CMP
18" CMP
REMOVE INLET d
TYPE GMP DI a
1490
1500
1510
1520
1530
1540
1550
1560
1570
10
11
9
PRELIMINARY DRAINAGE PLAN
SCALE: 1" = 20'
PDP-1
STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans HYDRAULICS DESIGN
FUNCTIONAL SUPERVISOR STEVE THORNE
CALCULATED-DESIGNED BY BRETT DITZLER
CHECKED BY
REVISED BY
DATE REVISED
BORDER LAST REVISED 7/2/2010
USERNAME => s133471
DGN FILE => 0200000016_CreekPlan.dgn
RELATIVE BORDER SCALE IS IN INCHES
0 1 2 3
UNIT 0315
PROJECT NUMBER & PHASE 02000000016
DATE PLOTTED => 17-JUN-2014
TIME PLOTTED => 14:40

WATER QUALITY

California Regional Water Quality Control Board, North Coast Region

Board Order No. WDID No. 5A45CR00466

CALIFORNIA
Water Boards

Edmund G. Brown Jr.
GOVERNOR

Matthew Rodriquez
SECRETARY FOR ENVIRONMENTAL PROTECTION

Central Valley Regional Water Quality Control Board

20 August 2014

Mr. Matt Mitchell
Caltrans
1657 Riverside Drive, MS #30
Redding, CA 96001

CLEAN WATER ACT §401 TECHNICALLY CONDITIONED WATER QUALITY CERTIFICATION FOR DISCHARGE OF DREDGED AND/OR FILL MATERIALS FOR THE SIDEHILL VIADUCT REPLACEMENT PROJECT (WDID#5A45CR00466), SHASTA 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 this project.

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.

REGIONAL WATER QUALITY CONTROL BOARD CONTACT PERSON:

Dannas J. Berchtold, Engineering Associate, Redding Branch Office, 364 Knollcrest Drive, Suite 205, Redding, California 96002, dberchtold@waterboards.ca.gov, (530) 224-4783

WATER QUALITY CERTIFICATION:

I hereby issue an order certifying that any discharge from Caltrans, Sidehill Viaduct Replacement Project (WDID# 5A45CR00466) 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.

George D Day

(for) PAMELA C. CREEDON
Executive Officer

DB:lmw

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

R:\RB5\R5RSection\N Central Valley\aCross Section\Clerical\Storm_water\DBerchtold\2014\401 5A45CR00466 Sidehill Viaduct Replacement Project, Caltrans.doc

PROJECT INFORMATION

Application Date: 10 July 2014

Application Complete Date: 19 August 2014

Applicant: Caltrans, Attn: Mr. Matt Mitchell

Project Name: Sidehill Viaduct Replacement Project

Application Number: WDID No. 5A45CR00466

U.S. Army Corps File Number: SPK-2014-00648

Type of Project: Replacement of the Sidehill Viaduct at post mile 29.7 on I-5 in Shasta County

Project Location: Section 21/27/28, Township 34 North, Range 4 West, MDB&M.
Latitude: 40°46'52" and Longitude: -122°19'18"

County: Shasta County

Receiving Water(s) (hydrologic unit): Unnamed intermittent tributary to Shasta Lake. Shasta Dam Hydrologic Unit-Lake Shasta Drainage Hydrologic Area No. 506.20

Water Body Type: Riparian, Streambed

Designated Beneficial Uses: The Basin Plan for the Central Valley Water Board has designated beneficial uses for surface and ground waters within the region. Beneficial uses that could be impacted by the project include: Municipal and Domestic Water Supply (MUN); Agricultural Supply (AGR); Industrial Supply (IND), Hydropower Generation (POW); Groundwater Recharge, Water Contact Recreation (REC-1); Non-Contact Water Recreation (REC-2); Warm Freshwater Habitat (WARM); Cold Freshwater Habitat (COLD); Spawning, Reproduction, and /or Early Development (SPWN); and Wildlife Habitat (WILD).

Project Description (purpose/goal): The Sidehill Viaduct Replacement Project consists of replacing the existing Sidehill Viaduct Bridge at post mile 29.7 on Interstate 5 because it does not meet current seismic standards. The bridge will be replaced with a new structure, which will require a new highway alignment. As part of the realignment, approximately 270' of intermittent stream will be relocated approximately 13' to the west from its current flow path and channeled into approximately 270' of 48" diameter corrugated steel pipe (CSP). The project requires stream relocation to accommodate fill required for the new alignment. Approximately 30' (10.4 cubic yards) of rock slope protection (RSP) will be placed at the inlet of the new CSP. Approximately 20' of natural perennial spring channel will be permanently impacted by the placement of RSP, while 25' of developed perennial spring channel will be filled with earthen materials and its flow placed in a pipe. Approximately 9,630 square feet of riparian forest will be permanently impacted for access, culvert placement, and the new highway alignment. The relocation and piping of the intermittent stream will require a clean water diversion for work within the channel.

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 0.221 acre of riparian and 0.043 acre of un-vegetated streambed.

Dredge Volume: Not Applicable

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

Department of Fish and Wildlife Streambed Alteration Agreement: Caltrans applied for a Streambed Alteration Agreement in July 2014. Lake & Streambed Alteration Agreement Number: 1600-2014-0191-R1

Possible Listed Species: None

Status of CEQA Compliance: The California Department of Transportation signed a CEQA Determination on 28 May 2014 approving a Categorical Exemption pursuant Class 1, §15301 which states exemption for activities resulting in replacement or reconstruction of existing bridges.

Compensatory Mitigation: Caltrans will purchase streambed mitigation credits from the Stillwater Mitigation Bank for the unavoidable impacts to 0.043 acres of streambed, for the purchase price of $6,450. Caltrans will purchase riparian habitat mitigation credits from the Jelly's Ferry mitigation site for the unavoidable impacts to 0.221 acres of riparian habitat, for the purchase price of $33,150.

Application Fee Provided: On 10 July 2014 a certification application fee of $9,490.00 was submitted as required by 23 CCR §3833b(3)(A) and by 23 CCR §2200(e).

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)

The State Water Resources Control Board (SWRCB) finds that:

1. Discharges eligible for coverage under these General WDRs are discharges of dredged or fill material that have received State Water Quality Certification (Certification) pursuant to federal Clean Water Act (CWA) section 401.

2. Discharges of dredged or fill material are commonly associated with port development, stream channelization, utility crossing land development, transportation water resource, and flood control projects. Other activities, such as land clearing, may also involve discharges of dredged or fill materials (e.g., soil) into waters of the United States.

3. CWA section 404 establishes a permit program under which the U.S. Army Corps of Engineers (ACOE) regulates the discharge of dredged or fill material into waters of the United States.

4. CWA section 401 requires every applicant for a federal permit or license for an activity that may result in a discharge of pollutants to a water of the United States (including permits under section 404) to obtain Certification that the proposed activity will comply with State water quality standards. In California, Certifications are issued by the Regional Water Quality Control Boards (RWQCB) or for multi-Region discharges, the SWRCB, in accordance with the requirements of California Code of Regulations (CCR) section 3830 et seq. The SWRCB's water quality regulations do not authorize the SWRCB or RWQCBs to waive certification, and therefore, these General WDRs do not apply to any discharge authorized by federal license or permit that was issued based on a determination by the issuing agency that certification has been waived. Certifications are issued by the RWQCB or SWRCB before the ACOE may issue CWA section 404 permits. Any conditions set forth in a Certification become conditions of the federal permit or license if and when it is ultimately issued.

5. Article 4, of Chapter 4 of Division 7 of the California Water Code (CWC), commencing with section 13260(a), requires that any person discharging or proposing to discharge waste, other than to a community sewer system, that could affect the quality of the waters of the State,[1] file a report of waste discharge (ROWD). Pursuant to Article 4, the RWQCBs are required to prescribe waste discharge requirements (WDRs) for any proposed or existing discharge unless WDRs are waived pursuant to CWC section 13269. These General WDRs fulfill the requirements of Article 4 for proposed dredge or fill discharges to waters of the United States that are regulated under the State's CWA section 401 authority.

[1] "Waters of the State" as defined in CWC Section 13050(e)

IT IS HEREBY ORDERED that WDRs are issued to all persons proposing to discharge dredged or fill material to waters of the United States where such discharge is also subject to the water quality certification requirements of CWA section 401 of the federal Clean Water Act (Title 33 United States Code section 1341), and such certification has been issued by the applicable RWQCB or the SWRCB, unless the applicable RWQCB notifies the applicant that its discharge will be regulated through WDRs or waivers of WDRs issued by the RWQCB. In order to meet the provisions contained in Division 7 of CWC and regulations adopted thereunder, dischargers shall comply with the following:

1. Dischargers shall implement all the terms and conditions of the applicable CWA section 401 Certification issued for the discharge. This provision shall apply irrespective of whether the federal license or permit for which the Certification was obtained is subsequently deemed invalid because the water body subject to the discharge has been deemed outside of federal jurisdiction.

2. Dischargers are prohibited from discharging dredged of fill material to waters of the United States without first obtaining Certification from the applicable RWQCB or SWRCB.

CERTIFICATION

The undersigned, Clerk to the Board, does hereby certify that the foregoing is a full, true, and correct copy of an order duly and regularly adopted at a meeting of the State Water Resources Control Board held on November 19, 2003.

AYE: Arthur G. Baggett, Jr.
Peter S. Silva
Richard Katz
Gary M. Carlton
Nancy H. Sutley

NO: None.

ABSENT: None.

ABSTAIN: None.

Debbie Irvin

Debbie Irvin
Clerk to the Board

AGREEMENTS

California Department of Fish and Wildlife

Notification No. 1600-2014-0191-R1

California Department of Fish and Wildlife
Northern Region
601 Locust Street
Redding, CA 96001

Streambed Alteration Agreement
Notification No. 1600-2014-0191-R1
Unnamed intermittent stream tributary to Shasta Lake.

California Department of Transportation
Sidehill Viaduct Bridge Replacement

This Streambed Alteration Agreement (Agreement) is entered into between the California Department of Fish and Wildlife (DFW) and the California Department of Transportation (Permittee) as represented by Mr. Phil Baker.

RECITALS

WHEREAS, pursuant to Fish and Game Code (FGC) section 1602, Permittee notified DFW on July 11, 2014 that Permittee intends to complete the project described herein;

WHEREAS, pursuant to FGC section 1603, DFW has determined that the project could substantially adversely affect existing fish or wildlife resources and has included measures in the Agreement necessary to protect those resources;

WHEREAS, Permittee has reviewed the Agreement and accepts its terms and conditions, including the measures to protect fish and wildlife resources;

NOW THEREFORE, Permittee agrees to complete the project in accordance with the Agreement.

PROJECT LOCATION

The project is located along Interstate 5, between post mile 29.5 and 30.5, approximately 11 miles south of the town of Lakehead and 10 miles north of the town of Shasta Lake, in the County of Shasta, State of California. Location is at T34N, R4W, sec 28, SW1/4, Mt. Diablo Meridian.

PROJECT DESCRIPTION

The project will involve replacing the Sidehill Viaduct Bridge at post mile 29.7 on Interstate 5 with a new bridge and realigning adjoing portions of the highway. As part of the road realignment work approximately 300 feet of an unnamed intermittent stream channel will be filled and moved 13 feet to the west from it current location and placed into a corrugated steel pipe 48 inches in diameter and 270 feet long. At the inlet to the culvert pipe rock slope protection will be placed approximately 30 feet in length near post mile 30.04. Approximately 9,630 square feet of riparian forest will be permanently lost. Additional impacts will be the loss of 20 feet of perennial spring channel that will be filled with the placement of the rock slope protection at the culvert inlet near post mile 30.04; and the loss of 25 feet of another spring channel near the existing inlet at 30.04 that will be placed into a pipe and routed to a new junction box as part of the drainage system.

PROJECT IMPACTS

The project will result in the permanent loss of 9,630 square feet of white alder riparian vegetation and the permanent loss of 45 feet of open spring channel habitat.

MEASURES TO PROTECT FISH AND WILDLIFE RESOURCES

1 Administrative Measures

Permittee shall meet each administrative requirement described below.

1.1 Documentation at Project Site. Permittee shall make the Agreement, any extensions and amendments to the Agreement, and all related notification materials and California Environmental Quality Act (CEQA) documents, readily available at the project site at all times and shall be presented to DFW personnel, or personnel from another state, federal, or local agency upon request.

1.2 Providing Agreement to Persons at Project Site. Permittee shall provide copies of the Agreement and any extensions and amendments to the Agreement to all persons who will be working on the project at the project site on behalf of Permittee, including but not limited to contractors, subcontractors, inspectors, and monitors.

1.3 Notification of Conflicting Provisions. Permittee shall notify DFW if Permittee determines or learns that a provision in the Agreement might conflict with a provision imposed on the project by another local, state, or federal agency. In that event, DFW shall contact Permittee to resolve any conflict.

1.4 Project Site Entry. Permittee agrees that DFW personnel may enter the project site at any time, after notifying the Resident Engineer, to verify compliance with the Agreement.

2 Avoidance and Minimization Measures

To avoid or minimize adverse impacts to fish and wildlife resources identified above, Permittee shall implement each measure listed below.

PROJECT TIMING

2.1 General Work Period for Stream Channel and Banks All work on the stream banks or within the stream channel, shall be confined to the period commencing May 15, and ending October 15, of any year in which this Agreement is valid when there is little or no stream flow. If weather conditions permit, and the stream remains in low flow conditions or dry, the Permittee may perform work within the stream channel or on the banks after October 15, provided a written request is made to the Department at least 5 days before the proposed work period variance. Written approval from the Department for the proposed work period variance must be received by the Permittee prior to the start or continuation of work after October 15.

2.2 Required Measures for Work after October 15. If work is performed within the stream channel or on the banks after October 15, the Permittee shall do all of the following:

a. Stage erosion and sediment control materials at the work site.

b. Monitor the seventy-two (72) hour forecast from the National Weather Service.

c. When the 72-hour forecast indicates a probability of precipitation of 60% or greater, or at the onset of any precipitation, ground disturbing activities shall cease and erosion control measures shall be implemented to stabilize exposed soils and prevent the mobilization of sediment into the stream channel or adjacent wetland or riparian areas.

HABITAT AND SPECIES PROTECTION

2.3 Delineating Limits of Work. Prior to initiating vegetation- or ground-disturbing Project activities, Permittee shall clearly delineate the construction limits in the stream and riparian areas. Permittee shall restrict all Project activities to the designated work area and shall maintain all fencing, stakes and flags until the completion of Project activities.

2.4 Minimize Loss of Riparian Vegetation. Removal of existing riparian vegetation shall not exceed the minimum necessary to complete operations.

2.5 Take of State Threatened or Endangered Species. This Agreement does not authorize the take of any State threatened or endangered species. If the project could result in the "take" of a state listed threatened or endangered species, the Permittee has the responsibility to obtain from the Department, a California Endangered Species Act Permit (CESA 2081 Permit). The Department may formulate a management plan that will avoid or mitigate take. If appropriate, contact the Department CESA coordinator at (530) 225-2300.

2.6 <u>Take of Special Status Mollusk Species and Salamanders.</u> The Permittee found two Federally Sensitive mollusk species on the proposed construction site, *Helminthoglypta hertleini* (Oregon shoulderband snail; Bureau of Land Management (BLM) Sensitive), and *Vespericola shasta* (Shasta hersperian snail; U.S. Forest Service (USFS) Sensitive species). The Permittee also identified two salamanders on site *Aneides flavipunctatus* (black salamander) and *Ensatina* spp., (ensatina salamander) specimens located on site were relocated to suitable off site habitat, adjacent to the proposed construction site, during surveys in 2014. The special status mollusk species, *Ancotrema voyanum voyanum* (hooded lancetooth snail; BLM Sensitive), has the potential to occur in the project area.

If *Hydromantes shastae* (Shasta salamander) is located during the relocation effort, DFW shall be immediately notified, and the project put on hold while determinations are made of the necessity for an Incidental Take Permit as defined in Section 2.5 of this agreement.

To avoid impacts to the special status species listed above, the Permittee shall develop a survey and relocation plan for review and approval by DFW at least 60 days prior to the start of construction activities, and shall survey the springs, streams, riparian areas, and any other sites with permanent ground cover and moisture. The relocation plan shall include the following:

(1) Completion of surveys at least two weeks prior to, and no more than two months prior to initiation of construction activities;

(2) Provide aerial photographs or maps with the survey area(s) outlined and identified;

(3) Provide an appropriate number of surveyors and days to adequately survey the area outlined with survey dates, during weather conditions conducive to locating and identifying individuals, and then to re-locate them to suitable habitat off-site;

(4) Identify relocation sites for each species on aerial photographs or topographic maps;

(5) Submit a post-survey report of the identification of the species, and number collected and relocated.

Implementation of these surveys and relocations of species will result in the project minimizing deleterious effects to these species.

2.7 Environmentally Sensitive Areas. All riparian areas, ephemeral and intermittent channels beyond the construction limits shall be identified on map plans and in the field as protected Environmentally Sensitive Areas (ESAs) and shall be off limits to construction equipment and personnel except as specifically authorized in this Agreement.

2.8 Installation of Environmentally Sensitive Area (ESA) Fencing. ESA fencing shall be installed as the first order of work. ESA fencing shall consist of temporary orange construction fence, other highly visible material, or a permanently installed pre-existing fence, that clearly delineates the limits of the work area.The placement of ESA fencing shall be inspected and approved by DFW prior to the initiation of work. Permittee shall provide written notification for inspection a minimum of 5 working days prior to beginning work. If DFW is unable to conduct a site inspection during this period, the inspection may be conducted by the Environmental Construction Liaison and the results forwarded to DFW for approval.

2.9 ESA Fencing Education. The Permittee shall ensure that the contractor, subcontractors, and all personnel working on the Project are instructed on the purpose of the ESA fencing and understand the limits of the work area.

2.10 Vegetation Removal Period and Nesting Birds. Removal of trees and shrubs from the work area shall take place between September 1 and February 15 to avoid impacts to nesting birds. Take of migratory birds will be avoided during construction activities. In no case shall active nests with eggs or young be removed during construction.

2.11 Materials for Rock Slope Protection. RSP and energy dissipation materials shall consist of clean (quarry run or equivalent) rock, competent for the application, sized and properly installed to resist washout. RSP slopes shall be supported with competent boulders keyed into a footing trench with a depth sufficient to properly seat the footing course boulders and prevent instability (typically at least 1/3 diameter of footing course boulders). Excavation spoils shall not be side-cast into the channel nor is any manipulation of the substrate of the channel authorized except as herein expressly provided.

2.12 Executive Order 13112 requires Federal agencies to prevent and control the introduction and spread of invasive species, therefore all equipment shall be washed pre – and post - construction to prevent the spread of any noxious weeds. All areas left disturbed at the end of construction will be seeded and mulched to help prevent the establishment of invasive weeds.

PETROLEUM, CHEMICAL AND OTHER POLLUTANTS

2.13 Storage of Materials All construction-related materials and equipment shall be stored at a local Caltrans maintenance yard or in designated staging areas located outside of the floodplain in upland areas located within Caltrans right-of-way.

2.14 Work Adjacent to Watercourses and Refueling. As part of the proposed construction activities, heavy equipment (drilling trucks, backhoe, excavator, etc.) may be required to work within and/or adjacent to perennial watercourses. Therefore, there is potential for chemical contamination as a result of a leak or spill of petroleum or hydraulic products into a channel. Measures will be taken to avoid or minimize potential chemical contamination, which will include no staging, storage and re-fueling of vehicles and equipment within 100 feet of any watercourse. In the event of a leak or spill, the project shall cease immediately and the Regional Water Quality Control Board (RWQCB) and CDFW shall be notified.

2.15 Equipment Washing. If equipment must be washed, washing will occur where the water cannot flow into a creek channel.

2.16 Maintenance and Inspection of Equipment to Prevent Leaks. Any equipment or vehicles driven and/or operated within or adjacent to the stream channel shall be checked and maintained daily to prevent leaks of materials that, if introduced to water, could be deleterious to aquatic life, wildlife, or riparian habitat.

2.17 Drip Pans. Stationary equipment such as motors, pumps, generators, and welders that contain deleterious materials, located within 100 feet of any watercourse shall be positioned over drip pans, for any part of the equipment that drips fluids.

2.18 Pollution of Waters of the State Prohibited. No debris, soil, silt, sand, bark, slash, sawdust, rubbish, cement or concrete or washings thereof, asphalt, paint or other coating material, oil or petroleum products or other organic or earthen material from any construction, or associated activity of whatever nature shall be allowed to enter into, or placed where it may be washed by rainfall or runoff into, waters of the State. When operations are completed, any excess materials or debris shall be removed from the work area. No rubbish shall be deposited within 150 feet of the high water mark of any stream or lake.

2.19 Disposal of Concrete Water. Water that has been in contact with uncured concrete shall be contained in a sealed concrete washout facility or other impervious container and shall not be discharged to surface or ground waters.

2.20 Fluid Spill Response. In the event of an unexpected fluid spill, the equipment operator will immediately stop work and contain the escaping fluids and mitigate any further potential fluid loss. Any fluid that should leak onto the ground will be collected by placing absorbent pads and absorbent material. These used pads and material will then be placed in 55 gallon drums for disposal.

2.21 Spill Containment, Clean up and Discharge Notification. All construction activities performed in or near the stream shall have absorbent materials designated for spill containment and clean up activities on-site for use in an accidental spill. In the event of a discharge, the Permittee shall immediately notify the California Emergency Management Agency at 1-800-852-7550 and immediately initiate clean up activities. DFW shall be notified by the Permittee and consulted regarding clean-up procedures.

EROSION AND SEDIMENT CONTROL

2.22 Erosion Control Measures Required. The project shall, at all times, feature adequate erosion and sediment control devices to prevent the degradation of water quality.

2.23 Installation and Maintenance of Best Management Practices. Soils exposed by project operations shall be treated to prevent sediment runoff and transport. Erosion control measures shall include the proper installation and maintenance of approved Best Management Practices (BMPs) and may include applications of seed, certified weed-free straw, compost, fiber, stabilizing emulsion and mulch, or combinations thereof.

2.24 Soil Stabilization and Sediment Prevention. Soils adjacent to the stream channel that are exposed by project operations shall be adequately stabilized when rainfall is reasonably expected during construction, and immediately upon completion of construction, to prevent the mobilization of such sediment into the stream channel or adjacent riparian areas. National Weather Service forecasts shall be monitored by the Permittee to determine the chance of precipitation.

2.25 Erosion Control Seeding. Prior to the end of construction, all disturbed areas shall be stabilized and reseeded with a suitable cover crop (such as winter wheat) that will not persist on site. A regionally appropriate California native seed mix shall be applied during the first or second year to provide succession from the erosion control cover crop for the establishment of native plants and will be subject to review and approval by DFW.

2.26 Spill Prevention Plan. Temporary construction site BMPs shall be implemented using a Spill Prevention Plan (SPP) which will be kept on site, along with all materials and equipment necessary to implement the SPP should it be needed. The temporary BMPs are aimed at reducing erosion and subsequent sediment transport, and preventing accidental spills during construction and may include check dams, straw bales, hydraulic mulch, sediment traps, concrete washouts, fiber rolls, and temporary Hot Mix Asphalt (HMA) dikes.

2.27 Temporary Fill. Temporary fills, placed in watercourses, will be removed within 30 days after completion of work at a given location and/or prior to the onset of the rain season, and in accordance with the Section 401 and 404 Clean Water Act requirements. These areas will be returned to their pre-construction contours, and treated with erosion control seed mix.

3 Compensatory Measures

To compensate for adverse impacts to fish and wildlife resources identified above that cannot be avoided or minimized, Permittee shall implement each measure listed below.

3.1 Off-site Riparian Restoration. To compensate for the permanent loss of 0.221 acre of white alder riparian habitat and the loss of 45 linear feet of perennial spring channel, Permittee shall implement one of the following options a or b. There shall be no temporal loss penalty that would increase the acreage or requirements for either option a or b due to problems beyond the requirements set forth in this agreement.

Option (a). Coordinate with DFW to enhance riparian vegetation at the Cantara Loop Fishing Access south of Mt. Shasta in Siskiyou County by (i) bringing in boulders, or installation of other mutally agreed upon trespass control devices such as (but not limited to), K-rail, posts and cable, gate and other fencing, or some combination of the aforementioned devices; (ii) soil restoration and contouring to eliminate ruts, rills and other erosive features, installation of erosion control measures such as certified weed free straw, willow waddles, silt fence or other appropriate measures; (iii) stabilized and reseeded with a suitable cover crop (such as winter wheat) that will not persist on site and a regionally appropriate California native seed mix shall be applied together in a mix of approximately 30% winter wheat and 70% native seed mix which will provide succession from the erosion control cover crop and the establishment of native plants; in addition a fertilizer with nutrient bacteria and mycorrhizae may be used; the winter wheat cover crop, native species seed mix, and the nutrient fertilizer shall will be subject to review and approval by DFW; (iv) planting of native riparian species that may include *Alnus rubra*, *Salix* spp., *Calocedrus decurrens*, or orther species agreed upon by DFW and the Permittee (total planting will be determined based upon costs associated with items i to iii, and will not exceed 0.221 acres. DFW will coordinate with the Permittee to examine this potential and determine feasibility, costs, and time frame for implementation.

Option (b). If option (a) is infeasible due to problems in negotiating with the Union Pacific Railroad, or other reasons that will impede implementation in a timely manner, or at the discretion of DFW, then, 1.105 acres (a 5:1 mitigation ratio) of riparian vegetation will be established at the Jelly's Ferry Mitigation Site in Tehama County; because the mitigation is off-site and out-of-kind. The mitigation shall occur prior to or concurrently with project construction. A draft riparian restoration plan shall be reviewed and approved by DFW prior to conducting planting activities at the site

4 Reporting Measures

Permittee shall meet each reporting requirement described below.

4.1 Post-Construction Monitoring and Annual Reporting.

Option (a). Permittee shall coordinate with DFW to develop a suitable monitoring plan that will be used to determine that the site has stabilized, plants have become established, and incorporates the following conditions: (a) site shall be monitored for three years following initial seeding; (b) native grasses and herbaceous species in seed mix are established to a mean of 40% absolute cover with standard error of 20% when measured with 15 stratified randomly placed circular plots of 3.14 sq. meter by year 3, if there have been no access and trespass problems that have substantially degraded the seeding effort; (c) planting of riparian tree or shrub species shall be measured by the fraction of total survival if there have been no access or trespass problems, with a minimum target of 50% survival; (d) placement of three signs which inform the public that this is a sensitive area that is undergoing restoration. Reduced survival percentages are being established for revegetation at this site given that there have been access and trespass control problems for many years perceeding this restoration effort. If vandalism results in the destruction of the tree and shrub plantings, or seeding, the Permittee is not responsible for additional plantings and will not be held to success criteria for these either seeding or planting. Control of vandalism is the responsibility of the DFW and not the Permittee. The Permittee may elect to conduct additional restoration actions at this site, beyond those required to meet the terms of this agreement, if so, the Permittee shall discuss the options with DFW and upon mutual agreement and completion of the restoration action(s), the additional conservation values will be documented, and then may be used for future project mitigation by the Permittee.

Option (b). Permittee shall monitor riparian plantings at the Jelly's Ferry Mitigation Site for a minimum of five years following planting. Annual reports shall be submitted to DFW by December 31 of each year. Reports shall document the survival, growth and vigor of riparian plantings, including any replacement plantings required.

CONTACT INFORMATION

Any communication that Permittee or DFW submits to the other shall be in writing and any communication or documentation shall be delivered to the address below by U.S. mail, fax, or email, or to such other address as Permittee or DFW specifies by written notice to the other.

<u>To Permittee:</u>

Mr. Phil Baker
Project Manager
Department of Transportation
1657 Riverside Drive
Redding, CA 96001
Fax: (530) 225-3019
Email: phil.baker@dot.ca.gov
ec: matt.mitchell@dot.ca.gov

<u>To DFW</u>:

Department of Fish and Wildlife
Northern Region
601 Locust Street
Redding, CA 96001
Attn: Lake and Streambed Alteration Program – Dr. Richard Lis
Notification #1600-2014-0191-R1
Fax: (530) 225-2267
Email: richard.lis@wildlife.ca.gov

LIABILITY

Permittee shall be solely liable for any violations of the Agreement, whether committed by Permittee or any person acting on behalf of Permittee, including its officers, employees, representatives, agents or contractors and subcontractors, to complete the project or any activity related to it that the Agreement authorizes.

This Agreement does not constitute DFW's endorsement of, or require Permittee to proceed with the project. The decision to proceed with the project is Permittee's alone.

SUSPENSION AND REVOCATION

DFW may suspend or revoke in its entirety the Agreement if it determines that Permittee or any person acting on behalf of Permittee, including its officers, employees, representatives, agents, or contractors and subcontractors, is not in compliance with the Agreement.

Before DFW suspends or revokes the Agreement, it shall provide Permittee written notice by certified or registered mail that it intends to suspend or revoke. The notice shall state the reason(s) for the proposed suspension or revocation, provide Permittee an opportunity to correct any deficiency before DFW suspends or revokes the Agreement, and include instructions to Permittee, if necessary, including but not limited to a directive to immediately cease the specific activity or activities that caused DFW to issue the notice.

ENFORCEMENT

Nothing in the Agreement precludes DFW from pursuing an enforcement action against Permittee instead of, or in addition to, suspending or revoking the Agreement.

Nothing in the Agreement limits or otherwise affects DFW's enforcement authority or that of its enforcement personnel.

OTHER LEGAL OBLIGATIONS

This Agreement does not relieve Permittee or any person acting on behalf of Permittee, including its officers, employees, representatives, agents, or contractors and subcontractors, from obtaining any other permits or authorizations that might be required under other federal, state, or local laws or regulations before beginning the project or an activity related to it.

This Agreement does not relieve Permittee or any person acting on behalf of Permittee, including its officers, employees, representatives, agents, or contractors and subcontractors, from complying with other applicable statutes in the FGC including, but not limited to, FGC sections 2050 et seq. (threatened and endangered species), 3503 (bird nests and eggs), 3503.5 (birds of prey), 5650 (water pollution), 5652 (refuse disposal into water), 5901 (fish passage), 5937 (sufficient water for fish), and 5948 (obstruction of stream).

Nothing in the Agreement authorizes Permittee or any person acting on behalf of Permittee, including its officers, employees, representatives, agents, or contractors and subcontractors, to trespass.

AMENDMENT

DFW may amend the Agreement at any time during its term if DFW determines the amendment is necessary to protect an existing fish or wildlife resource.

Permittee may amend the Agreement at any time during its term, provided the amendment is mutually agreed to in writing by DFW and Permittee. To request an amendment, Permittee shall submit to DFW a completed DFW "Request to Amend Lake or Streambed Alteration" form and include with the completed form payment of the corresponding amendment fee identified in DFW's current fee schedule (see Cal. Code Regs., tit. 14, § 699.5).

TRANSFER AND ASSIGNMENT

This Agreement may not be transferred or assigned to another entity, and any purported transfer or assignment of the Agreement to another entity shall not be valid or effective, unless the transfer or assignment is requested by Permittee in writing, as specified below, and thereafter DFW approves the transfer or assignment in writing.

The transfer or assignment of the Agreement to another entity shall constitute a minor amendment, and therefore to request a transfer or assignment, Permittee shall submit to DFW a completed DFW "Request to Amend Lake or Streambed Alteration" form and include with the completed form payment of the minor amendment fee identified in DFW's current fee schedule (see Cal. Code Regs., tit. 14, § 699.5).

EXTENSIONS

In accordance with FGC section 1605(b), Permittee may request one extension of the Agreement, provided the request is made prior to the expiration of the Agreement's term. To request an extension, Permittee shall submit to DFW a completed DFW "Request to Extend Lake or Streambed Alteration" form and include with the completed form payment of the extension fee identified in DFW's current fee schedule (see Cal. Code Regs., tit. 14, § 699.5). DFW shall process the extension request in accordance with FGC 1605(b) through (e).

If Permittee fails to submit a request to extend the Agreement prior to its expiration, Permittee must submit a new notification and notification fee before beginning or continuing the project the Agreement covers (Fish & G. Code, § 1605, subd. (f)). .

EFFECTIVE DATE

The Agreement becomes effective on the date of DFW's signature, which shall be: 1) after Permittee's signature; 2) after DFW complies with all applicable requirements under the California Environmental Quality Act (CEQA); and 3) after payment of the applicable FGC section 711.4 filing fee listed at:
http://www.dfg.ca.gov/habcon/ceqa/ceqa_changes.html.

TERM

This Agreement shall expire on December 31, 2019, unless it is terminated or extended before then. All provisions in the Agreement shall remain in force throughout its term. Permittee shall remain responsible for implementing any provisions specified herein to protect fish and wildlife resources after the Agreement expires or is terminated, as FGC section 1605(a) (2) requires.

AUTHORITY

If the person signing the Agreement (signatory) is doing so as a representative of Permittee, the signatory hereby acknowledges that he or she is doing so on Permittee's behalf and represents and warrants that he or she has the authority to legally bind Permittee to the provisions herein.

AUTHORIZATION

This Agreement authorizes only the project described herein. If Permittee begins or completes a project different from the project the Agreement authorizes, Permittee may be subject to civil or criminal prosecution for failing to notify DFW in accordance with FGC section 1602.

CONCURRENCE

The undersigned accepts and agrees to comply with all provisions contained herein.

FOR DEPARTMENT OF TRANSPORTATION

Phil Baker Project Manager	3-3-15 Date

FOR DEPARTMENT OF FISH AND WILDLIFE

Michael R. Harris Habitat Conservation Planning Supervisor	3-4-15 Date

Prepared by: Richard Lis, Ph.D.
Senior Environmental Scientist

RAILROAD RELATIONS

Railroad Relations and Insurance Requirements

EXHIBIT D

Information Handout Document to Project Construction Contract Documents:

"RAILROAD RELATIONS AND INSURANCE REQUIREMENTS"

1.01 GENERAL

The term "Railroad" shall mean Union Pacific Railroad Company.

It is expected that the Railroad will cooperate with the Contractor to the end that the work may be handled in an efficient manner. However, except for the additional compensation provided for hereinafter for delays in completion of specific unit of work to be performed by the Railroad, and except as provided in Public Contracts Code Section 7102, the Contractor shall have no claim for damages, extension of time, or extra compensation in the event his work is held up by railroad train operations or other work performed by the Railroad.

The Contractor must understand the Contractor's right to enter the Railroad's property is subject to the absolute right of the Railroad to cause the Contractor's work on the Railroad's property to cease if, in the opinion of the Railroad, the Contractor's activities create a hazard to the Railroad's property, employees, and operations.

The Contractor acknowledges its receipt from the State of a copy of the Contractor's Right of Entry Agreement that has been executed by the Railroad and the State. The Contractor agrees to execute and deliver to the Railroad the Contractor's Endorsement that is attached hereto as Appendix 1 and to provide to the State and/or the Railroad all insurance policies, binders, certificates or endorsements that are set forth in Exhibits B and C of the Contractor's Right of Entry Agreement.

1.02 RAILROAD REQUIREMENTS

The Contractor shall provide to Kevin Yoder, Railroad's Manager, Industry and Public Projects, 9451 Atkinson Boulevard, Roseville, California 95747, and State's Resident Engineer (Engineer) in writing, the advance notice requirements set forth in Section 1 of **Exhibit B** of the Right of Entry Agreement before performing any work on, or adjacent to the property or tracks of the Railroad.

Contractor shall cooperate with the Railroad where work is over or under the tracks, or within the limits of the Railroad property to expedite the work and avoid interference with the operation of railroad equipment.

Contractor shall comply with the rules and regulations of the Railroad or the instructions of its representatives in relation to protecting the tracks and property of the Railroad and the traffic moving on such tracks, as well as the wires, signals and other property of the Railroad, its tenants or licensees, at and in the vicinity of the work during the period of construction. The responsibility of the Contractor for safe conduct and adequate policing and supervision of its work at the job site shall not be lessened or otherwise affected by the presence at the work site of the Railroad representatives, or by the Contractor's compliance with any requests or recommendations made by the Railroad representatives.

Contractor shall perform work so as not to endanger or interfere with the safe operation of the tracks and property of the Railroad and traffic moving on such tracks, as well as wires, signals and other property of the Railroad, its tenants or licensees, at or in the vicinity of the work.

Contractor shall take protective measures to keep the Railroad facilities, including track ballast, free of sand or debris resulting from his operations. Damage to the Railroad facilities resulting from the Contractor's operations will be repaired or replaced by the Railroad and the cost of such repairs or replacement shall be deducted from the Contractor's progress and final pay estimates.

Contractor shall contact the Railroad's "Call Before You Dig" at least forty-eight (48) hours prior to commencing work, at 1-800-336-9193 during normal business hours (7:00 a.m. to 9:00 p.m. Central Time, Monday through Friday, except holidays – also a 24-hour, 7-day number for emergency calls) to determine location of fiber optics. If a telecommunications system is buried anywhere on or near the Railroad property, the Contractor will coordinate with the Railroad and the Telecommunication Company (ies) to arrange for relocation or other protection of the system prior to beginning any work on or near Railroad property.

Contractor shall not pile or store any materials nor park any equipment closer than 25'-0" to the centerline of the nearest track, unless directed by the Railroad's representative.

Contractor shall also abide by the following temporary clearances during the course of construction:

- 12'-0" horizontally from centerline of track
- 21'-6" vertically above top of rail

The temporary vertical construction clearance above provided will not be permitted until authorized by the California Public Utilities Commission. It is anticipated that authorization will be received not later than fifteen (15) days after the approval of the highway contract by the Attorney General. In the event authorization is not received by the time specified, and, if in the opinion of the Engineer, the Contractor's operations are delayed or interfered with by reason of authorization not being received by the said time, the Licensee will compensate the Contractor for such delay to the extent provided in Section 8-1.07, "Delays," of the 2010 Standard Specifications and not otherwise.

Walkways with railing shall be constructed by the Contractor over open excavation areas when in close proximity of tracks, and railings shall not be closer than 8'-6" horizontally from centerline of the nearest track, if tangent, or 9'-6" if curved.

Infringement on the above temporary construction clearances by the Contractor's operations shall be submitted to the Railroad by the Engineer, and shall not be undertaken until approved by the Railroad, and until the Engineer has obtained any necessary authorization from any governmental body or bodies having jurisdiction there-over. No extension of time or extra compensation will be allowed in the event the Contractor's work is delayed pending Railroad approval and governmental authorization.

When the temporary vertical clearance is less than 22'-6" above top of rail, the Railroad shall have the option of installing tell-tales or other protective devices the Railroad deems necessary for protection of the Railroad trainmen or rail traffic.

Four (4) sets of plans, in 11" x 17" format, and two (2) sets of calculations showing details of construction affecting the Railroad's tracks and property not included in the contract plans, including but not limited to shoring and falsework, shall be submitted to the Engineer for review prior to submittal to the Railroad for final approval. At Railroad's option, these plans and calculations may be submitted electronically via email or be mailed with appropriate CD or DVD disk. Falsework shall comply with the Railroad guidelines. Demolition of existing structures shall comply with the Railroad guidelines. Shoring shall be designed in accordance with the Railroad's shoring requirement of UPRR Drawing No. 106613 and guidelines for shoring and falsework, latest edition, issued by the Railroad's Office of Chief Engineer. Shoring and falsework plans and calculations shall be prepared and signed by a professional engineer registered in California. This work shall not be undertaken until such time as the Railroad has given such approval, review by the Railroad may take up to six (6) weeks after receipt of necessary information.

Contractor shall notify the Engineer in writing, at least twenty-five (25) calendar days but not more than forty (40) days in advance of the starting date of installing temporary work with less than permanent clearance at each structure site. The Contractor shall not be permitted to proceed with work across railroad tracks until this requirement has been met. No extension of time or extra compensation will be allowed if the Contractor's work is delayed due to failure to comply with the requirements in this paragraph.

Blasting will be permitted only when approved by the Railroad.

Contractor shall, upon completion of the work covered by this Contract to be performed by the Contractor upon the premises or over or beneath the tracks of the Railroad, promptly remove from the premises of the Railroad, the Contractor's tools, implements and other materials, whether brought upon said premises and cause said premises to be left in a clean and presentable condition.

Under track pipeline installations (if required) shall be constructed in accordance with the Railroad's current standards that may be obtained from the Railroad. The general guidelines are as follows:

> Edges of jacking or boring pit excavations shall be a minimum of 25 feet from the centerline of the nearest track.

If the pipe to be installed under the track is 4 inches in diameter or less, the top of the pipe shall be at least 42 inches below base of rail.

If the pipe diameter is greater than 4 inches in diameter, it shall be encased and the top of the steel pipe casing shall be at least 66 inches below base of rail.

Installation of pipe or conduit under the Railroad's tracks shall be done by dry bore and jack method.

Hydraulic jacking or boring will not be permitted.

Safety of personnel, property, rail operations and the public is of paramount importance. As reinforcement and in furtherance of overall safety measures to be observed by the Contractor (and not by way of limitation), the following special safety rules shall be followed:

(a) Contractor shall keep the job site free from safety and health hazards and ensure that its employees are competent and adequately trained in all safety and health aspects of the job. Contractor shall have proper first aid supplies available on the job site so that prompt first aid services can be provided to any person that may be injured on the job site. Contractor shall promptly notify the Railroad of any U.S. Occupational Safety and Health Administration reportable injuries occurring to any person that may arise during the work performed on the job site. Contractor shall have a non-delegable duty to control its employees while they are on the job site or any other property of the Railroad to be certain they do not use, be under the influence of, or have in their possession any alcoholic beverage, drug, narcotic or other substance that may inhibit the safe performance of work by the employee.

(b) Employees of the Contractor shall be suitably dressed to perform their duties safely and in a manner that will not interfere with their vision, hearing or free use of their hands or feet. Only waist length shirts with sleeves and trousers that cover the entire leg are to be worn. If flare-legged trousers are worn, the trouser bottoms must be tied to prevent catching. Employees should wear sturdy and protective work boots and at least the following protective equipment:

(1) Protective headgear that meets American National Standard-Z89.l-latest revision. It is suggested that all hardhats be affixed with the Contractor's or the subcontractor's company logo or name.

(2) Eye protection that meets American National Standard for occupational and educational eye and face protection, Z87.1-latest revision. Additional eye protection must be provided to meet specific job situations such as welding, grinding, burning, etc.; and

(3) Hearing protection that affords enough attenuation to give protection from noise levels that will be occurring on the job site.

(c) All heavy equipment provided or leased by the Contractor shall be equipped with audible back-up warning devices. If in the opinion of the Railroad Representative any of the Contractor's or the subcontractor's equipment is unsafe for use on the Railroad's right-of-way, the Contractor, at the request of the Railroad representative, shall remove such equipment from the Railroad's right-of-way.

1.03 PROTECTION OF RAILROAD FACILITIES

Upon the advance notification provided to the Railroad as set forth in Section 1 of Exhibit B of the Contractor's Right of Entry Agreement, the Railroad representatives, conductors, flagmen or watchmen will be provided by the Railroad to protect its facilities, property and movements of its trains or engines. Notice shall be made to the Railroad's Manager of Track Maintenance at ***(530) 235-0300, ext. 5080.*** At the time of notification, the Contractor shall provide the Railroad with a schedule of dates that flagging services will be needed, as well as times, if outside normal working hours. Subsequent deviation from the schedule shall require ten (10) working days' advance notice from the first affected date. The Railroad will furnish such personnel or other protective devices:

(a) When equipment is standing or being operated within 25 feet, measured horizontally, from centerline of any track on which trains may operate, or when any erection or construction activities are in progress within such limits, regardless of elevation above or below track.

(b) For any excavation below elevation of track subgrade if, in the opinion of the Railroad's representative, track or other Railroad facilities may be subject to settlement or movement.

(c) During any clearing, grubbing, grading or blasting in proximity to the Railroad which, in the opinion of the Railroad's representative, may endanger the Railroad facilities or operations.

(d) During any of the Contractor's operations when, in the opinion of the Railroad's representatives, the Railroad facilities, including, but not limited to, tracks, buildings, signals, wire lines or pipe lines, may be endangered.

The cost of flagging and inspection provided by the Railroad during the period of constructing that portion of the project located on or near the Railroad property, as deemed necessary for the protection of the Railroad's facilities and trains, will be borne by the State. The Railroad has indicated that its estimated flagging rate will be around One Thousand Three Hundred Dollars ($1,300.00) per day and that the State has estimated a total of four hundred (500) days of flagging. The State shall pay the Railroad for all actual flagging costs incurred by the Railroad under this Project.

1.04 WORK BY RAILROAD

The following work by the Railroad will be performed by Railroad forces and is not a part of the work under this Contract.

(a) Railroad will perform preliminary engineering and inspection (if any) construction inspection, plan change review (if any) and falsework plan review and flagging as specified in Section 1.03 "Protection of Railroad Facilities," of these special provisions.

(b) Temporary crossings at grade over tracks of Railroad for the purpose of hauling earth, rock, paving or other materials will require approval by railroad in advance. If the Contractor, for the purpose of constructing highway-railway grade separation structures, including construction ramps thereto, desires to move equipment or materials across Railroad's tracks, the Contractor shall first obtain permission from Railroad (at possible additional cost) via the State Engineer. Should Railroad approve the temporary crossings, State shall execute a Service Contract with Railroad for Railroad to construct the temporary crossing. Under the Service Contract, State shall bear the cost of the crossing surface, warning devices and other components that might be required. Notwithstanding State's Service Contract with Railroad, the Contractor is required to execute Railroad's form of Contractor's Haul Road Crossing Agreement. Railroad, at State's expense, shall provide flagmen to control movements of vehicles across the temporary crossing. State and its Contractor shall prevent the use of such temporary crossing by unauthorized persons and vehicles.

1.05 DELAYS DUE TO WORK BY RAILROAD.

If delays due to work by the Railroad occur, and the Contractor sustains loss which, in the opinion of the Engineer, could not have been avoided by the judicious handling of forces, equipment and plant, the amount of said loss shall be determined as provided in Section 8-1.07, " Delays," of the 2010 Standard Specifications.

If a delay due to work by the Railroad occurs, an extension of time determined pursuant to the provisions in Section 8-1.10, "Liquidated Damages," of the 2010 Standard Specifications will be granted.

1.06 LEGAL RELATIONS

The provisions of Section 1, "Relations with Railroad Company," and the provisions of Section 2, "Railroad Protective Insurance," of these special provisions shall inure directly to the benefit of the Railroad.

2.0 RAILROAD PROTECTIVE INSURANCE

In addition to any other form of insurance or bonds required under the terms of the contract and specifications, the Contractor will be required to carry insurance of the kinds and in the amounts hereinafter specified.

Such insurance shall be approved by the Railroad before any work is performed on the Railroad's property and shall be carried until all work required to be performed on or adjacent to the Railroad's property under the terms of the contract is satisfactorily completed as determined by the Engineer, and thereafter until all tools, equipment and materials have been removed from the Railroad's property and such property is left in a clean and presentable condition.

Full compensation for all premiums which the Contractor is required to pay on all the insurance described hereinafter shall be considered as included in the prices paid for the various items of work to be performed under the contract, and no additional allowance will be made thereof or for additional premiums which may be required by extensions of the policies of insurance.

The following insurance coverage will be required:

A. **Commercial General Liability** insurance. Commercial general liability (CGL) with a limit of not less than $5,000,000 each occurrence and an aggregate limit of not less than $10,000,000. CGL insurance must be written on ISO occurrence form CG 00 01 12 04 (or a substitute form providing equivalent coverage).

The policy must also contain the following endorsement, which must be stated on the certificate of insurance:

- Contractual Liability Railroads ISO form CG 24 17 10 01 (or a substitute form providing equivalent coverage) showing "Union Pacific Railroad Company Property" as the Designated Job Site.
- Designated Construction Project(s) General Aggregate Limit ISO Form CG 25 03 03 97 (or a substitute form providing equivalent coverage) showing the project on the form schedule.

B. **Business Automobile Coverage** insurance. Business auto coverage written on ISO form CA 00 01 (or a substitute form providing equivalent liability coverage) with a combined single limit of not less $5,000,000 for each accident. The policy must contain the following endorsements, which must be stated on the certificate of insurance:

- Coverage For Certain Operations In Connection With Railroads ISO form CA 20 70 10 01 (or a substitute form providing equivalent coverage) showing "Union Pacific Property" as the Designated Job Site.
- Motor Carrier Act Endorsement - Hazardous materials clean up (MCS-90) if required by law.

C. **Workers' Compensation and Employers' Liability** insurance. Coverage must include but not be limited to:

- Contractor's statutory liability under the workers' compensation laws of the State of California.
- Employers' Liability (Part B) with limits of at least $500,000 each accident, $500,000 disease policy limit $500,000 each employee.

If Contractor is self-insured, evidence of state approval and excess workers compensation coverage must be provided. Coverage must include liability arising out of the U. S. Longshoremen's and Harbor Workers' Act, the Jones Act, and the Outer Continental Shelf Land Act, if applicable.

The policy must contain the following endorsement, which must be stated on the certificate of insurance:

- Alternate Employer endorsement ISO form WC 00 03 01 A (or a substitute form providing equivalent coverage) showing Railroad in the schedule as the alternate employer (or a substitute form providing equivalent coverage).

D. **Railroad Protective Liability Insurance.** Contractor must maintain Railroad Protective Liability insurance written on ISO occurrence form CG 00 35 12 04 (or a substitute form providing equivalent coverage) on behalf of Railroad as named insured, with a limit of not less than $2,000,000 per occurrence and an aggregate of $6,000,000. A binder stating the policy is in place must be submitted to Railroad before the work may be commenced and until the original policy is forwarded to Railroad.

E. **Umbrella or Excess** insurance. If Contractor utilizes umbrella or excess policies, these policies must "follow form" and afford no less coverage than the primary policy.

F. **Pollution Liability** insurance. Pollution liability coverage must be written on ISO form Pollution Liability Coverage Form Designated Sites CG 00 39 12 04 (or a substitute form providing equivalent liability coverage), with limits of at least $5,000,000 per occurrence and an aggregate limit of $10,000,000.

If the scope of work as defined in this Agreement includes the disposal of any hazardous or non-hazardous materials from the job site, Contractor must furnish to Railroad evidence of pollution legal liability insurance maintained by the disposal site operator for losses arising from the insured facility accepting the materials, with coverage in minimum amounts of $1,000,000 per loss, and an annual aggregate of $2,000,000.

Other Requirements

G. All policy (ies) required above (except worker's compensation and employers liability) must include Railroad as "Additional Insured" using ISO Additional Insured Endorsements CG 20 26, and CA 20 48 (or substitute forms providing equivalent coverage). The coverage provided to Railroad as additional insured shall, to the extent provided under ISO Additional Insured

Endorsement CG 20 26, and CA 20 48 provide coverage for Railroad's negligence whether sole or partial, active or passive, and shall not be limited by Contractor's liability under the indemnity provisions of this Agreement.

H. Punitive damages exclusion, if any, must be deleted (and the deletion indicated on the certificate of insurance), unless the law governing this Agreement prohibits all punitive damages that might arise under this Agreement.

I. Contractor waives all rights of recovery, and its insurers also waive all rights of subrogation of damages against Railroad and its agents, officers, directors and employees. This waiver must be stated on the certificate of insurance.

J. Prior to commencing the work, Contractor shall furnish Railroad with a certificate(s) of insurance, executed by a duly authorized representative of each insurer, showing compliance with the insurance requirements in this Agreement.

K. All insurance policies must be written by a reputable insurance company acceptable to Railroad or with a current Best's Insurance Guide Rating of A- and Class VII or better, and authorized to do business in the State of California.

L. The fact that insurance is obtained by Contractor or by Railroad on behalf of Contractor will not be deemed to release or diminish the liability of Contractor, including, without limitation, liability under the indemnity provisions of this Agreement. Damages recoverable by Railroad from Contractor or any third party will not be limited by the amount of the required insurance coverage.

APPENDIX 1

CONTRACTOR'S ENDORSEMENT

A. As a condition to entering upon the Railroad's right-of-way to perform Work pursuant to this agreement, State's contractor, ______________________________
(Name of Contractor)
whose address is ____,
(Contractor's Mailing Address)
(hereinafter "Contractor"), agrees to comply with and be bound by all the terms and provisions of the attached Caltrans Right of Entry Agreement that was signed by Union Pacific Railroad Company ("Railroad") and the State of California, Department of Transportation ("State") relating to the Work to be performed and the insurance requirements set forth in Exhibits B and C of the Right of Entry Agreement. The Contractor further acknowledges and agrees that the reference to Cal. Gov. Code §14662.5 in Sections 5.b) and 8.b) of Exhibit B to the Right of Entry Agreement does not apply to the Contractor and in no way limits the indemnities set forth in those provisions, to which the Contractor agrees to be bound.

B. Before the Contractor commences any Work, the Contractor will provide the Railroad with (i) a binder of insurance for the Railroad Protective Liability Insurance described in Section 2 of the Contract Special Provisions, hereto attached, and the original policy, or a certified duplicate original policy when available, and (ii) a certificate issued by its insurance carrier providing the other insurance coverage and endorsements required pursuant to Section 2 of the Contract Special Provisions.

C. All insurance correspondence, binders or originals shall be directed to:

Union Pacific Railroad Company
Attn: Real Estate Department
1400 Douglas Street, MS 1690
Omaha, Nebraska
Attn.: Senior Manager-Contracts
Folder No. 2901-00

D. Please note that fiber optic cable may be buried on the Railroad's property. **Prior to commencing any work, the Contractor agrees to contact the Railroad's Telecommunications Operation Center as provided in Section 5 of Exhibit A of the Right of Entry Agreement to determine if any fiber optic cable is located on the Railroad's property on or near the location where the work is to be performed.** If there is, the Contractor must comply with the terms and conditions of Section 5 of Exhibit A before commencing any work on the Railroad's property.

E. **The Contractor agrees to also provide to the Railroad's Manager-Track Maintenance at (530) 235-0300, ext. 5080** the advance notice required in Section 1 of Exhibit B of the Right of Entry Agreement prior to working on the Railroad's property in order for the Railroad to coordinate the Contractor's work with the Railroad's operations and to make arrangements for flagging protection (if applicable).

This endorsement shall be completed and sent to the person named in Paragraph C above.

____
(Name of Contractor)

By__________________________________

Title: ________________________________

Date: ________________________________

RAILROAD RELATIONS

Railroad’s Minimum Requirements

EXHIBIT E

Information Handout Document to Project Construction Contract Documents:

UNION PACIFIC RAILROAD
MINIMUM REQUIREMENTS

PART 1 – GENERAL

1.01 DESCRIPTION

This project includes construction work within the Right-of-Way and/or properties of the Union Pacific Railroad "UPRR" and adjacent to tracks, wire lines and other facilities. This section describes the special requirements for coordination with railroad when work by the Contractor will be performed upon, over or under the railroad right-of-way or may impact current or future railroad operations. The Contractor will coordinate with railroad while performing the work outlined in this Contract, and shall afford the same cooperation with railroad as it does with the Agency. The Railroad Designated Representative shall complete all submittals and work in accordance with railroad guidelines and AREMA recommendations as modified by these minimum special requirements or as directed in writing.

For purposes of this project, the Railroad Designated Representative shall be the person or persons designated by railroad to handle specific tasks related to the project.

1.02 DEFINITION OF AGENCY AND CONTRACTOR

As used in these railroad requirements, the term "Agency" shall mean the State of California, by and through its Department of Transportation.

As used in these railroad requirements, the term "Contractor" shall mean the contractor or contractor's hired by the Agency to perform any project work on any portion of railroad's property and shall also include the contractor's subcontractors and the contractor's and subcontractor's respective officer, agents and employees, and others acting under its or their authority.

1.03 UPRR CONTACTS

The primary point of contact for this project is:

Kevin Yoder
Manager Industry and Public Projects
Union Pacific Railroad Company
9451 Atkinson Street
Roseville, CA 95747
Phone: (916) 789-5054
E-mail: kayoder@up.com

For UPRR railroad flagging services and track work, contact:

Gene Womack
Manager of Track Maintenance
Union Pacific Railroad
(530) 235-0300, ext. 5080

1.04 REQUEST FOR INFORMATION / CLARIFICATION

All Requests for Information ("RFI") involving work within any railroad right-of-way shall be in accordance with the procedures listed elsewhere in these bid documents. All RFI's shall be submitted to the Engineer of Record. The Engineer of Record will submit the RFI to the Railroad Designated Representative for review and approval for corresponding to work within the railroad right-of-way. The Contractor shall allow four (4) weeks for the review and approval process by railroad.

1.05 PLANS / SPECIFICATIONS

The plans and specifications for this project, affecting the railroad, are subject to the written approval by the railroad and changes in the plans may be required after award of the Contract. Such changes are subject to the approval of the Agency and Railroad.

PART 2 – UTILITIES AND FIBER OPTIC

2.01 UTILITIES AND FIBER OPTIC

All installations shall be constructed in accordance with current AREMA recommendations and railroad specifications and requirements. UPRR general guidelines and the required application forms for utility installations can be found on the UPRR website at www.uprr.com.

3.01 GENERAL

A. Contractor shall perform all work in compliance with all applicable Railroad and FRA rules and regulations. Contractor shall arrange and conduct all work in such manner and at such times as shall not endanger or interfere with the safe operation of the tracks and property of railroad and the traffic moving on such tracks, or the wires, signals and other property of railroad, its tenants or licensees, at or in the vicinity of the work. Railroad shall be reimbursed by Contractor or Agency for train delay costs and lost revenue claims due to any delays or interruption of train operations resulting from Contractor's construction work or other activities.

B. Construction activities will be permitted within 12 feet of the centerline of operational tracks only if absolutely necessary and Railroad's Designated Representative grants approval. Construction activities within 12 feet of the operational track(s) must allow the tracks to stay operational.

C. Track protection is required for all work equipment (including rubber tired equipment) operating within 25 feet from nearest rail.

D. The Contractor is also advised that new railroad facilities within the project may be built by UPRR and that certain Contractor's activities cannot proceed until that work is completed. The Contractor shall be aware of the limits of responsibilities and allow sufficient time in the schedule for that work to be accomplished and shall coordinate its efforts with UPRR.

3.02 RAILROAD OPERATIONS

A. The Contractor shall be advised that trains and/or equipment are expected on any track, at any time, in either direction. Contractor shall become familiar with the train schedules in this location and structure its bid assuming intermittent track windows in this period, as defined in Paragraph B below.

B. All railroad tracks within and adjacent to the Contract Site are active, and rail traffic over these facilities shall be maintained throughout the Project. Activities may include both through moves and switching moves to local customers. Railroad traffic and operations will occur continuously throughout the day and night on these tracks and shall be maintained at all times as defined herein. The Contractor shall coordinate and schedule the work so that construction activities do not interfere with railroad operations.

C. Work windows for this Contract shall be coordinated with the Agency's and the UPRR Designated Representatives. Types of work windows include Conditional Work Windows and Absolute Work Windows, as defined below:

1. Conditional Work Window: A Conditional Work Window is a period of time that railroad operations have priority over construction activities. When construction activities may occur on and adjacent to the railroad tracks within 25 feet of the nearest track center line, an UPRR flag person will be required. At the direction of the UPRR flag person, upon approach of a train, and when trains are present on the tracks, the tracks must be cleared (i.e., no construction equipment, materials or personnel within 25 feet, or as directed by the UPRR Designated Representative, from the tracks). Conditional Work Windows are available for the Project.

2. Absolute Work Window: An Absolute Work Window is a period of time that construction activities are given priority over railroad operations. During this time frame the designated railroad track(s) will be inactive for train movements and may be fouled by the Contractor. At the end of an Absolute Work Window the railroad tracks and/or signals must be completely operational for train operations and all UPRR, Public Utilities Commission (PUC) and Federal Railroad Administration (FRA) requirements, codes and regulations for operational tracks must be complied with. In the situation where the operating tracks and/or signals have been affected, the UPRR will perform inspections of the work prior to placing that track back into service. UPRR flag persons will be

required for construction activities requiring an Absolute Work Window. **Absolute Work Windows will not generally be granted. Any request will require a detailed explanation for UPRR review.**

3.03 RIGHT OF ENTRY, ADVANCE NOTICE AND WORK STOPPAGES

A. Prior to beginning any work on or over the property of, or affecting the facilities of, UPRR, Contractor shall execute the Contractor's Endorsement that is a part of the Right of Entry Agreement to be signed by UPRR and Agency. There is a fee for processing the Right of Entry Agreement. This cost shall be borne by the Contractor. Contractor shall submit a copy of the executed agreement and the insurance policies, binders, certificates and endorsements set forth therein to the Agency prior to commencing work on UPRR property. The right of entry agreement shall specify working time frames, flagging and inspection requirements, and any other items specified by UPRR.

B. The Contractor shall give the advance notice to UPRR as required in the Right of Entry Agreement before commencing work in connection with construction upon or over UPRR Right-of-Way and shall observe UPRR rules and regulations with respect thereto.

C. All work upon UPRR Right-of-Way shall be done at such times and in such manner so as not to interfere with or endanger the operations of UPRR. Whenever work may affect the operations or safety of trains, the method of doing such work shall first be submitted to UPRR's Designated Representative for approval, but such approval shall not relieve the Contractor from liability. Any work to be performed by the Contractor, which requires flagging and/or inspection service, shall be deferred until the flagging protection required by UPRR is available at the job site. See Section 3.18 for railroad flagging requirements.

D. The Contractor shall make requests in writing for both Absolute and Conditional Work Windows, at least two weeks in advance of any work. The written request must include:

1. Exactly what the work entails.
2. The days and hours that work will be performed.
3. The exact location of work, and proximity to the tracks.
4. The type of window requested and the amount of time requested.
5. The designated contact person.

Contractor shall provide a written confirmation notice to UPRR at least 48 hours before commencing work in connection with approved work windows when work will be performed within **25 feet of any track center line.** All work shall be performed in accordance with previously approved work plans.

E. Should a condition arising from, or in connection with the work, require that immediate and unusual provisions be made to protect operations and property of UPRR, Contractor shall make such provisions. If in the judgment of UPRR's Designated Representative such provisions are insufficient, the UPRR Designated Representative may require or provide such provisions as deemed necessary. In any event, such provisions shall be at the Contractor's expense and without cost to UPRR. UPRR or the Agency shall have the right to order Contractor to temporarily cease operations in the event of an emergency or, if in the opinion of the UPRR's Designated Representative, the Contractor's operations could endanger UPRR operations. In the event such an order is given, Contractor shall immediately notify the Agency of the order.

3.04 INSURANCE

Contractor shall not begin work upon or over UPRR Right-of-Way until UPRR has been furnished the insurance policies, binders, certificates and endorsements required by the Right-of-Entry Agreement and UPRR's Designated Representative has advised the Agency that such insurance is in accordance with the Agreement. The required insurance shall be kept in full force and effect during the performance of work and thereafter until Contractor removes all tools, equipment, and material from UPRR's property and cleans the premises in a manner reasonably satisfactory to UPRR.

3.05 RAILROAD SAFETY ORIENTATION

All personnel employed by the Contractor and all subcontractors must complete the UPRR or equivalent course "Orientation for Contractor's Safety", and be registered prior to working on UPRR property. This orientation is available at www.contractororientation.com. This course is required to be completed annually.

3.06 COOPERATION

UPRR will cooperate with Contractor so that work may be conducted in an efficient manner, and will cooperate with Contractor in enabling use of UPRR's right-of-way in performing the work.

3.07 MINIMUM CONSTRUCTION CLEARANCES FOR FALSEWORK AND OTHER TEMPORARY STRUCTURES

The Contractor shall abide by the following minimum temporary clearances during the course of construction:

A. 12' – 0" horizontal from centerline of track
B. 21' – 6" vertically above top of rail.

For construction clearance less than listed above, local Operating Unit review and approval is required.

3.08 APPROVAL OF REDUCED CLEARANCES

A. The minimum track clearances to be maintained by the Contractor during construction are specified in Section 3.07 herein.

B. Any proposed infringement on the specified minimum clearances due to the Contractor's operations shall be submitted to UPRR's Designated Representative through the Agency at least 30 days in advance of the work and shall not be undertaken until approved in writing by UPRR's Designated Representative.

C. No work shall commence until the Contractor receives in writing assurance from UPRR's Designated Representative that arrangements have been made for flagging service, as may be necessary and receives permission from UPRR's Designated Representative to proceed with the work.

3.09 CONSTRUCTION AND AS-BUILT SUBMITTALS

A. Submittals are required for construction materials and procedures as outlined below. The submittals shall include all review comments from the Agency and the Engineer of Record. All design submittals shall be stamped and signed by a Professional Engineer registered in the State of California.

B. The tables below provide UPRR's minimum submittal requirements for the construction items noted. Submittal requirements are in addition to those specified elsewhere in these bid documents. The minimum review times indicated below represent UPRR's requirements only. The Contractor shall allow additional time for the Agency's review time as stated elsewhere in these bid documents.

C. Submittals shall be made by the Agency to UPRR's designated representative unless otherwise directed by the Railroad. Items in Table 1 shall be submitted for both railroad overpass and underpass projects, as applicable. Items in Table 2 shall be submitted for railroad underpass projects only.

TABLE 1

ITEM	DESCRIPTION	SETS REQD.	UPRR's Minimum Review Time
1	Shoring design and details	4	4 weeks
2	**Falsework design and details**	4	4 weeks
3	Drainage design provisions	4	4 weeks
4	Erection diagrams and sequence	4	4 weeks
5	Demolition diagram and sequence	4	4 weeks

Prior to or during construction of railroad's viaduct bridge structure, UPRR requires the review of drawings, reports, test data and material data sheets to determine compliance with the specifications. Product information for items noted in Table 2 be submitted to UPRR's Designated Representative through the Agency for their own review and approval of the material. UPRR or their consultant will review the

signed submittal and the Agency's review comments. If a consultant performs the reviews, the consultant may reply directly to the Agency or its Designated Representative after consultation with UPRR. Review of the submittals will not be conducted until after review by the Agency or its Designated Representative. Review of the submittal items will require a minimum of four (4) weeks after receipt from the Agency.

TABLE 2

ITEM	DESCRIPTION	SETS REQD.	NOTES
1	Shop drawings	4	Steel and Concrete members
2	**Bearings**	4	For entire structures
3	Concrete Mix Designs	4	For entire structures
4	Rebar & Strand certifications	4	For superstructure only
5	28 day concrete strength	4	For superstructure only
6	Waterproofing material certifications and installation procedure	4	Waterproofing & protective boards
7	Structural steel certifications	4	All fracture critical members & other members requiring improved notch toughness
8	Fabrication and Test reports	4	All fracture critical members & other members requiring improved notch toughness
9	Welding Procedures and Welder Certification	4	AWS requirements
10	Foundation Construction Reports	4	Pile driving, drilled shaft construction, bearing pressure test reports for spread footings
11	Compaction testing reports for backfill at abutments	4	Must meet 95% maximum dry density, Modified Proctor ASTM D1557

D. As-Built Records shall be submitted to UPRR within 60 days of completion of the structures. These records shall consist of the following items:

<u>Overpass Projects</u>

1. Electronic files of all structure design drawings with as-constructed modifications shown, in Microstation J or Acrobat .PDF format.
2. Hard copies of all structure design drawings with as-constructed modifications shown.

<u>Underpass Projects</u>

1. Electronic files of all structure design drawings with as-constructed modifications shown, in Microstation J or Acrobat .PDF format.
2. Hard copies of all structure design drawings with as-constructed modifications shown.

3. Final approved copies of shop drawings for concrete and steel members.
4. Foundation Construction Reports
5. Compaction testing reports for backfill at abutments

3.10 APPROVAL OF DETAILS

The details of the construction affecting UPRR tracks and property not already included in the Contract Plans shall be submitted to UPRR's Designated Representative through the Agency for UPRR's review and written approval before such work is undertaken. Review and approval of these submittals will require a minimum of four (4) weeks in addition to the Agency's review time as stated elsewhere in these bid documents.

3.11 MAINTENANCE OF RAILROAD FACILITIES

A. The Contractor shall be required to maintain all ditches and drainage structures free of silt or other obstructions which may result from Contractor's operations; to promptly repair eroded areas within UPRR's right of way and to repair any other damage to the property of UPRR, or its tenants.

B. All such maintenance and repair of damages due to the Contractor's operations shall be done at the Contractor's expense.

C. The Contractor must submit a proposed method of erosion control and have the method reviewed by UPRR prior to beginning any grading on the Project Site. Erosion control methods must comply with all applicable local, state and federal regulations.

3.12 SITE INSPECTIONS BY UPRR's DESIGNATED REPRESENTATIVE

A. In addition to the office reviews of construction submittals, site inspections may be performed by UPRR's Designated Representative at significant points during construction, including but not limited to the following:

   1. Preconstruction meetings.
   2. Pile driving, drilling of caissons or drilled shafts.
   3. Reinforcement and concrete placement for railroad bridge substructure and/or superstructure.
   4. Erection of precast concrete or steel bridge superstructure.
   5. Placement of waterproofing (prior to placing ballast on bridge deck).
   6. Completion of the bridge structure.

B. Site inspection is not limited to the milestone events listed above. Site visits to check progress of the work may be performed at any time throughout the construction as deemed necessary by UPRR.

C. A detailed construction schedule, including the proposed temporary horizontal and vertical clearances and construction sequence for all work to be performed, shall be provided to the Agency for submittal to UPRR's

Designated Representative for review prior to commencement of work. This schedule shall also include the anticipated dates when the above listed events will occur. This schedule shall be updated for the above listed events as necessary, but at least monthly so that site visits may be scheduled.

3.13 UPRR's REPRESENTATIVES

A. UPRR representatives, conductors, flag person or watch person will be provided by UPRR at expense of the Agency or Contractor (as stated elsewhere in these bid documents) to protect UPRR facilities, property and movements of its trains or engines. In general, UPRR will furnish such personnel or other protective services as follows:

1. When any part of any equipment is standing or being operated within 25 feet, measured horizontally, from centerline of any track on which trains may operate, or when any object is off the ground and any dimension thereof could extend inside the 25 foot limit, or when any erection or construction activities are in progress within such limits, regardless of elevation above or below track.

2. For any excavation below elevation of track subgrade if, in the opinion of UPRR's Designated Representative, track or other UPRR facilities may be subject to settlement or movement.

3. During any clearing, grubbing, excavation or grading in proximity to UPRR facilities, which, in the opinion of UPRR's Designated Representative, may endanger UPRR facilities or operations.

4. During any contractor's operations when, in the opinion of UPRR's Designated Representative, UPRR facilities, including, but not limited to, tracks, buildings, signals, wire lines, or pipe lines, may be endangered.

5. The Contractor shall arrange with the UPRR Designated Representative to provide the adequate number of flag persons to accomplish the work.

3.14 WALKWAYS REQUIRED

Along the outer side of each exterior track of multiple operated track, and on each side of single operated track, an unobstructed continuous space suitable for trainman's use in walking along trains, extending to a line not less than twelve feet (12') from centerline of track, shall be maintained. Any temporary impediments to walkways and track drainage encroachments or obstructions allowed during work hours while UPRR's flagman service is provided shall be removed before the close of each work day. Walkways with railings shall be constructed by Contractor over open excavation areas when in close proximity of track, and railings shall not be closer than 8' – 6" horizontally from centerline of tangent track or 9' – 6" horizontally from centerline of curved track.

3.15 COMMUNICATIONS AND SIGNAL LINES

If required, UPRR will rearrange its communications and signal lines, its grade crossing warning devices, train signals and tracks, and facilities that are in use and maintained by UPRR's forces in connection with its operation at expense of the Agency. This work by UPRR will be done by its own forces and it is not a part of the Work under this Contract.

3.16 TRAFFIC CONTROL

Contractor's operations that control traffic across or around UPRR facilities shall be coordinated with and approved by the UPRR's Designated Representative.

3.17 CONSTRUCTION EXCAVATIONS

A. The Contractor shall be required to take special precaution and care in connection with excavating and shoring. Excavations for construction of footings, piers, columns, walls or other facilities that require shoring shall comply with requirements of OSHA, AREMA and "Guidelines for Temporary Shoring".

B. The Contractor shall contact UPRR's "Call Before Your Dig" at least 48 hours prior to commencing work at 1-800-336-9193 during normal business hours (6:30 a.m. to 8:00 p.m. central time, Monday through Friday, except holidays - also a 24 hour, 7 day a week number for emergency calls) to determine location of fiber optics. If a telecommunications system is buried anywhere on or near UPRR property, the Contractor will co-ordinate with UPRR and the Telecommunication Company (ies) to arrange for relocation or other protection of the system prior to beginning any work on or near UPRR property.

3.18 RAILROAD FLAGGING

Performance of any work by the Contractor in which person(s) or equipment will be within twenty-five (25) feet of any track, or will be near enough to any track that any equipment extension (such as, but not limited to, a crane boom) will reach within twenty-five (25) feet of any track, may require railroad flagging services or other protective measures. Contractor shall give the advance notice to the UPRR as required in the "Contractor's Right of Entry Agreement" before commencing any such work, so that the UPRR may determine the need for flagging or other protective measures to ensure the safety of the railroad's operations. Contractor shall comply with all other requirements regarding flagging services covered by the "Contractor's Right of Entry Agreement". Any costs associated with failure to abide by these requirements will be borne by the Contractor.

3.19 CLEANING OF RIGHT-OF-WAY

Contractor shall, upon completion of the work to be performed by Contractor upon the premises, over or beneath the tracks of UPRR, promptly remove from the Right-of-Way of UPRR all of Contractor's tools, implements, and other materials whether brought upon the Right-of-Way by Contractor or any subcontractors, employee or agent of Contractor or of any subcontractor, and leave the Right-of-Way in a clean and presentable condition to satisfaction of UPRR.

3.20 GUIDELINES FOR RAILROAD GRADE SEPARATION PROJECTS

The "Guidelines for Railroad Grade Separation Project" shall be followed and is a part of the Railroad's minimum requirements by reference to the following website address:

http://www.uprr.com/aboutup/operations/specs/attachments/grade_separation.pdf

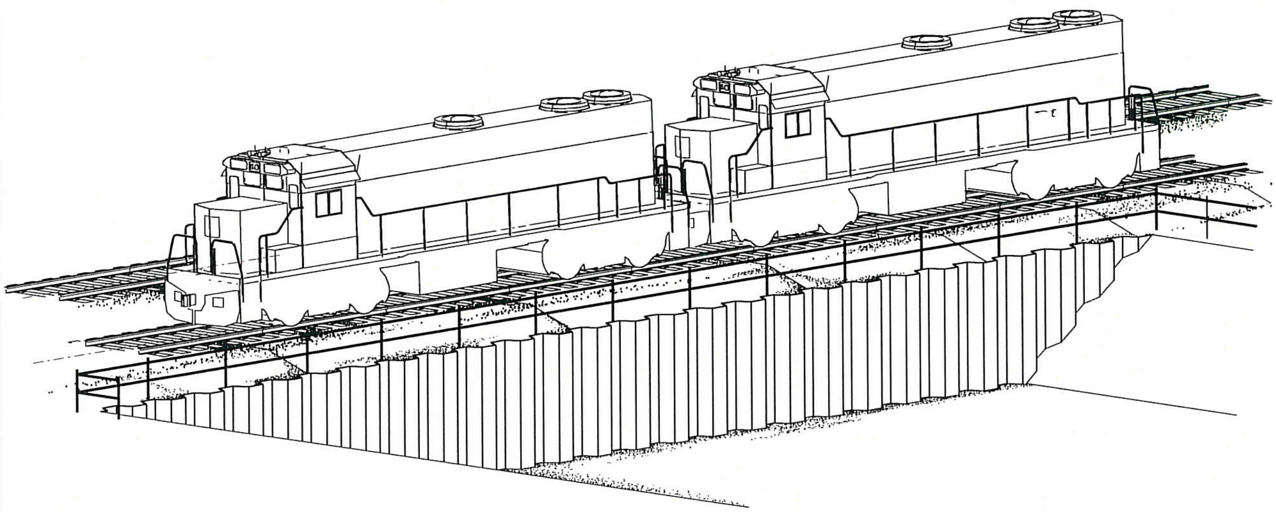
RAILROAD RELATIONS

Railroad's Guidelines for Temporary Shoring

<u>EXHIBIT F</u>

Railroad's Guidelines for Temporary Shoring

GUIDELINES FOR TEMPORARY SHORING



BUILDING AMERICA℠

"CALL BEFORE YOU DIG!"
1-800-533-2891

ASSISTANT DIRECTOR STRUCTURE DESIGN
4515 KANSAS AVE
KANSAS CITY, KS 66106-1124

"CALL BEFORE YOU DIG!"
1-800-336-9193

OFFICE AVP ENGINEERING - DESIGN
1400 DOUGLAS ST. STOP 0910
OMAHA, NE 68179-0910

INDEX

GUIDELINES FOR TEMPORARY SHORING

1. SCOPE

The scope of these guidelines is to inform public agencies, design engineers, contractors and inspectors of current Railroad standards and requirements concerning design and construction of temporary shoring.

1. The term **Railroad** refers to the Burlington Northern & Santa Fe Railway (BNSF) and/or the Union Pacific Railroad (UPRR). The term **Contractor** is defined as any party gaining access to work on Railroad right-of-way or other Railroad operating locations.

2. These guidelines are provided as a reference and may not be taken as authority to construct without prior review and written approval of the Railroad. These guidelines supersede all previous guidelines for temporary shoring and are subject to revision without notice.

3. These guidelines supplement the current, American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual of Recommended Practice. The 2002 AREMA Manual was utilized in developing this guideline. The AREMA Manual is available from:

   American Railway Engineering and Maintenance-of-Way Association
   8201 Corporate Drive, Suite 1125
   Landover, MD 20785-2230
   Phone: (301) 459-3200
   FAX: (301) 459-8077
   www.arema.org

4. The specific requirements for temporary shoring addressed in this document shall be followed for all locations where the Railroad operates, regardless of track ownership.

5. Any items not covered specifically herein shall be in accordance with the AREMA Manual and subject to the review and approval of the Railroad. Where conflicts exist, the most stringent specification should be applied.

6. All excavations shall also be governed by Railroad requirements, Federal, State and Local laws, rules, and regulations concerning construction safety.

7. Safe rail operations shall be required for the duration of the project. All personnel, railroad tracks and property shall be protected at all times.

8. To expedite the review process of the temporary shoring plans, drawings submitted by the Contractors are required to adhere to the project specifications, AREMA and other Railroad requirements.

2. GENERAL CRITERIA

The Contractor must not begin construction of any component of the shoring system affecting the Railroad right-of-way until written Railroad approval has been received.

1. All excavations shall be in compliance with applicable OSHA regulations and shall be shored where there is any danger to tracks, structures or personnel regardless of depth.

2. The Contractor is responsible for planning and executing all procedures necessary to construct, maintain and remove the temporary shoring system in a safe and controlled manner.

3. Emergency Railroad phone numbers are to be obtained from the Railroad representative in charge of the project prior to the start of any work and shall be posted at the job site.

4. The Contractor must obtain a valid right of entry permit from the Railroad and comply with all Railroad requirements when working on Railroad property.

5. The Contractor is required to meet minimum safety standards as defined by the Railroad.

6. All temporary shoring systems that support or impact the Railroad's tracks or operations shall be designed and constructed to provide safe and adequate rigidity.

7. The Railroad requirements, construction submittal review times and review criteria should be discussed at the pre-construction meeting with the Contractor.

8. A flagman is required when any work is performed within 25 feet of track centerline. If the Railroad provides flagging or other services, the Contractor shall not be relieved of any responsibilities or liabilities as set forth in any document authorizing the work. No work is allowed within 50 feet of track centerline when a train passes the work site and all personnel must clear the area within 25 feet of track centerline and secure all equipment when trains are present.

9. Appropriate measures for the installation and protection of fiber optic cables shall be addressed in the plans and contract documents. For specific Railroad requirements and additional information refer to:

   www.bnsf.com or call 1-800-533-2891.

   www.uprr.com, call 1-800-336-9193 or refer to UPRR Fiber Optic Engineering, Construction and Maintenance Standards.

10. Relocation of utilities or communication lines not owned by the Railroad shall be coordinated with the utility owners. The utility relocation plans must then be submitted to the Railroad utility representative for approval. The shoring plans must include the correct contact for the Railroad, State or Local utility locating service provider. The Railroad will not be responsible for cost associated with any utility, signal, or communication line relocation or adjustments.

3. CONTRACTOR RESPONSIBILITIES

The Contractor shall be solely responsible for the design, construction and performance of the temporary structure. **(AREMA 8.28.1.3)**

1. The Contractor's work shall in no way impede the train operations of the Railroad and must be coordinated with the local Railroad operating department.

2. The Contractor shall develop a work plan that enables the track(s) to remain open to train traffic at all times.

3. The Contractor shall comply with all State and Federal Laws, county or municipal ordinances and regulations which in any manner affect the work.

4. All removed soils will become the responsibility of the Contractor and shall be disposed of outside the Railroad right-of-way according to the applicable Federal, State and Local regulations.

5. The Project Engineer and the Contractor shall evaluate the quality of materials furnished and work performed.

6. The Contractor is responsible to protect the Railroad ballast and subballast from contamination.

7. The Contractor must monitor and record top of rail elevations and track alignment for the duration of the project. The movement shall be within the limits defined in **Table 1, Deflection Criteria** on page 10. Displacements exceeding the limits defined in **Table 1** must be immediately reported to the Railroad. All work on the project must stop and the Railroad may take any action necessary to ensure safe passage of trains. The Contractor must immediately submit a corrective action plan to the Railroad for review and approval. The Railroad must review and approve the proposed repair procedure. The repair must be inspected by the Railroad before the track can be placed back in service.

8. Any damage to Railroad property such as track, signal equipment or structure could result in a train derailment. All damage must be reported immediately to the Railroad representative in charge of the project and to the Railroad Manager of Track Maintenance (MTM).

4. INFORMATION REQUIRED

Plans and calculations shall be submitted, signed and stamped by a Registered Professional Engineer familiar with Railroad loadings and who is licensed in the state where the shoring system is intended for use. Shoring design plans and calculations shall be in English units. If Metric units are used, all controlling dimensions, elevations, design criteria assumptions, and material stresses shall be expressed in dual units, with English units to be in parentheses. Information shall be assembled concerning right-of-way boundary, clearances, proposed grades of tracks and roads, and all other factors that may influence the controlling dimensions of the proposed shoring system. See section 10 for additional requirements.

1. Field Survey.

   Sufficient information shall be shown on the plans in the form of profiles, cross sections and topographical maps to determine general design and structural requirements. Field survey information of critical or key dimensions shall be referenced to the centerline of track(s) and top of rail elevations. Existing and proposed grades and alignment of tracks and roads shall be indicated together with a record of controlling elevation of water surfaces or ground water. Show the location of existing/proposed utilities and construction history of the area which might hamper proper installation of the piling, soldier beams, or ground anchors.

2. Geotechnical Report shall provide:
   a. Elevation and location of soil boring in reference to the track(s) centerline and top of rail elevations.
   b. Classification of all soils encountered.
   c. Internal angle of soil friction.
   d. Dry and wet unit weights of soil.
   e. Active and passive soil coefficients, pressure diagram for multiple soil strata.
   f. Bearing capacity and unconfined compression strength of soil.
   g. Backfill and compaction recommendations.
   h. Optimum moisture content of fill material.
   i. Maximum density of fill material.
   j. Minimum recommended factor of safety.
   k. Water table elevation on both sides of the shoring system.
   l. Dewatering wells and proposed flownets or zones of influence.
   m. In seismic areas, evaluation of liquefaction potential of various soil strata.

3. Loads.

All design criteria, temporary and permanent loading must be clearly stated in the design calculations and on the contract and record plans. Temporary loads include, but are not limited to: construction equipment, construction materials and lower water levels adjoining the bulkhead causing unbalanced hydrostatic pressure. Permanent loads include, but are not limited to: future grading and paving, Railroads or highways, structures, material storage piles, snow and earthquake. The allowable live load after construction should be clearly shown in the plans and painted on the pavements behind the bulkheads or shown on signs at the site and also recorded on the record plans. Some of the loads are:

a. Live load pressure due to E80 loading for track parallel to shoring system.
b. Live load pressure due to E80 loading for track at right angle to shoring system.
c. Other live loads.
d. Active earth pressure due to soil.
e. Passive earth pressure due to soil.
f. Active earth pressure due to surcharge loads.
g. Active pressure due to sloped embankment.
h. Dead load.
i. Buoyancy.
j. Longitudinal force from live load.
k. Centrifugal forces.
l. Shrinkage.
m. Temperature.
n. Earthquake.
o. Stream flow pressure.
p. Ice pressure.

4. Drainage. **(AREMA 8.20.2.4)**

   a. The drainage pattern of the site before and after construction should be analyzed and adequate drainage provisions should be incorporated into the plans and specifications. Consideration should be given to groundwater as well as surface drainage.

   b. Drainage provisions for backfill should be compatible with the assumed water conditions in design.

5. Structural design calculations.

   a. List all assumptions used to design the temporary shoring system.

   b. Determine E80 live load lateral pressure using the Boussinesq strip load equation. See **Figure 2** which illustrates Plan Number **710001 "LIVE LOAD PRESSURE DUE TO COOPER E80**".

   c. Computerized calculations and programs must clearly indicate the input and output data. List all equations used in determining the output.

   d. Example calculations with values must be provided to support computerized output and match the calculated computer result.

   e. Provide a simple free body diagram showing all controlling dimensions and applied loads on the temporary shoring system.

   f. Calculated lateral deflections of the shoring and effects to the rail system must be included. See section 8, Part 6. Include the elastic deflection of the wall as well as the deflection due to the passive deflection of the resisting soil mass.

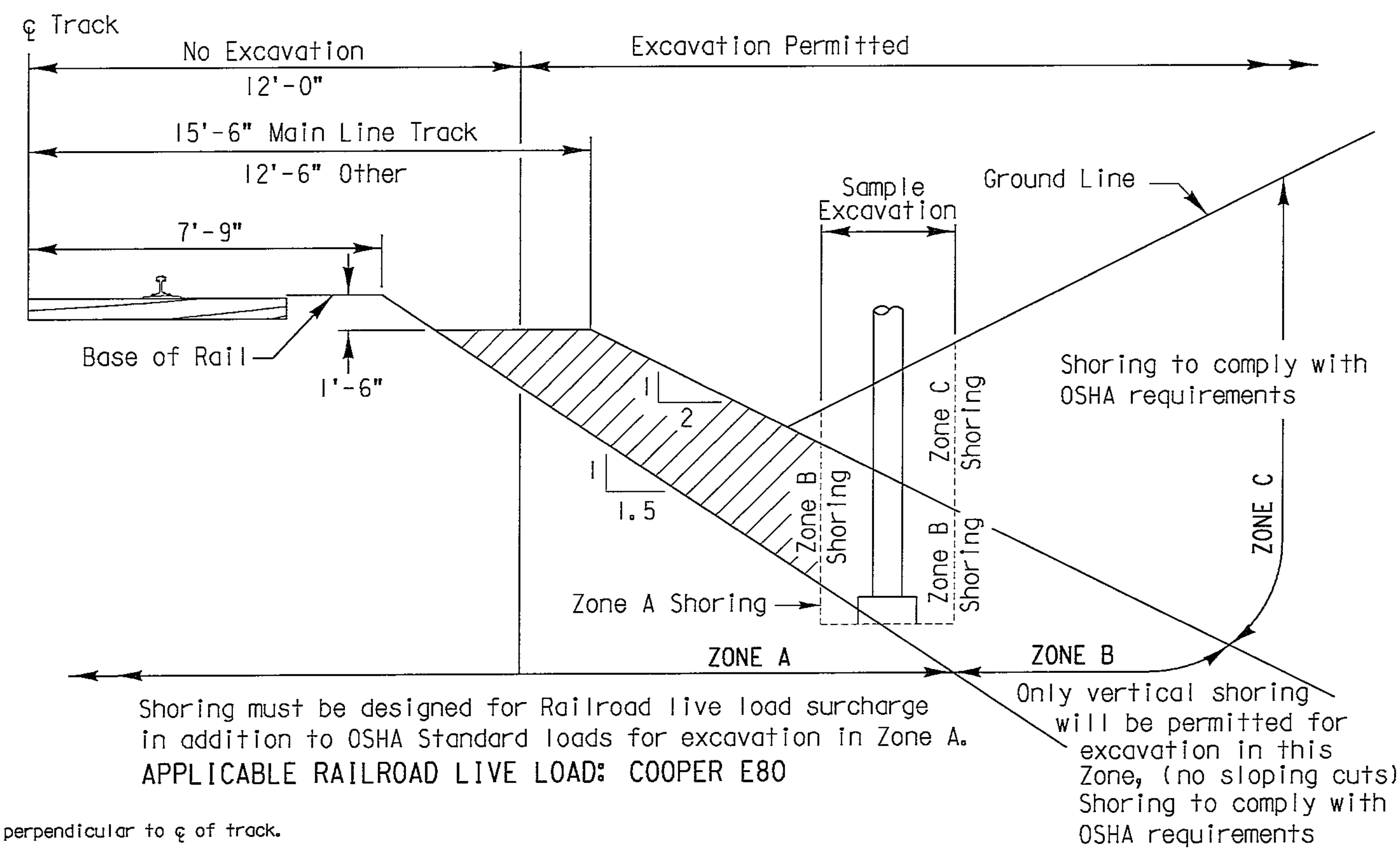
   g. Documents and manufacturer's recommendations which support the design assumptions must be included with the calculations.

5. TYPES OF TEMPORARY SHORING

1. A shoring box is a prefabricated shoring system which is installed as the excavation progresses. This shoring system is not accepted by the Railroad. This system is allowed in special applications only, typically where Railroad live load surcharge is not present. The shoring box is moved down into the excavation by gravity or by applying vertical loading from excavation equipment.
2. Anchored systems with tiebacks are discouraged. The tiebacks will be an obstruction to future utility installations and may also damage existing utilities. Tiebacks must be removed per Railroad direction. Removal of tieback assemblies is problematic.
3. An anchored sheet pile wall is a structure designed to provide lateral support for a soil mass and derives stability from passive resistance of the soil in which the sheet pile is embedded and the tensile resistance of the anchors.
   a. For purposes of these guidelines, ground anchors shall be cement-grouted tiebacks designed, furnished, installed, tested and stressed in accordance with the project specifications and AREMA requirements.
4. An anchored soldier beam with lagging wall is a structure designed to provide lateral support for a soil mass and derives stability from passive resistance of the soil in which the soldier beam is embedded and from the tensile resistance of the ground anchors.
   a. Anchored soldier beam with lagging walls are generally designed as flexible structures which have sufficient lateral movement to mobilize active earth pressures and a portion of the passive pressure.
   b. For purposes of these specifications, soldier beams include steel H-piles, wide flange sections or other fabricated sections that are driven or set in drilled holes. Lagging refers to the members spanning between soldier beams.
5. A cantilever sheet pile wall is a structure designed to provide lateral support for a soil mass and derives stability from passive resistance of the soil in which the sheet pile is embedded. If cantilever sheet pile is used for shoring adjacent to an operating track, the shoring system shall be at least 12'-0" away from the centerline of track. Cantilever sheet pile walls shall be used only in granular soils or stiff clays.
6. A cantilever soldier beam with lagging wall is a structure designed to provide lateral support for a soil mass and derives stability from passive resistance of the soil in which the soldier beam is embedded.
7. A braced excavation is a structure designed to provide lateral support for a soil mass and derives stability from passive resistance of the soil in which the vertical members are embedded and from the structural capacity of the bracing members.
   a. For purposes of these guidelines, the vertical members of the braced excavation system include steel sheet piling or soldier beams comprised of steel H-piles, wide flange sections, or other fabricated sections that are driven or installed in drilled holes. Wales are horizontal structural members designed to transfer lateral loads from the vertical members to the struts. Struts are structural compression members that support the lateral loads from the wales.
8. A cofferdam is an enclosed temporary structure used to keep water and soil out of an excavation for a permanent structure such as a bridge pier or abutment or similar structure. Cofferdams may be constructed of timber, steel, concrete or a combination of these. These guidelines consider cofferdams primarily constructed with steel sheet piles.

6. GENERAL SHORING REQUIREMENTS

For general shoring requirements and specific applications of the following items refer to **Figure 1** on the next page which illustrates Plan Number **710000 "GENERAL SHORING REQUIREMENTS"**.



GENERAL NOTES:

All dimensions are measured perpendicular to ℄ of track.

Prior to commencing any work, the contractor shall submit for approval by the Railroad detailed plans indicating the nature and extent of the track protection shoring proposed. The contractor shall install the temporary shoring system per the approved plans. Design of the temporary shoring system to comply with **GUIDELINES FOR TEMPORARY SHORING.**

For excavations which encroach into zone A or B, shoring plans shall be accompanied by design calculations. Plans and calculations must be signed and stamped by a Professional Engineer registered in the state where the work will be performed.

GENERAL EXCAVATION ZONES

SCALE: (NOT TO SCALE)

REVISIONS		
DATE	LTR.	DESCRIPTION
5/03	1	FORMERLY UPRR C.E. 106613
/		
/		
/		
/		
/		

DESIGN BY: PGP | DRAWN BY: JFS | CHECKED BY: AA

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J. Mayer 9-1-04

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BRIDGE STANDARDS

UNION PACIFIC

GENERAL SHORING REQUIREMENTS

FILE OWNER: UPRR | DATE: 5-8-03

PLAN NO.: 710000 1 | SHEET: 1 OF 1

PLOTTED: $$$$DATE$$$$ $TIME

Figure 1

1. No excavation shall be permitted closer than 12'-0" measured at a right angle from the centerline of track to the trackside of shoring system. If existing conditions preclude the installation of shoring at the required minimum distance, the shifting of tracks or temporary removal of tracks shall be investigated prior to any approval. All costs associated with track shifting or traffic interruption shall be at Contractor's expense.

2. Evaluate slope and stability conditions to ensure the Railroad embankment will not be adversely affected. Local and global stability conditions must also be evaluated.

3. All shoring within the limits of Zone A or Zone B must be placed prior to the start of excavation.

4. Lateral clearances must provide sufficient space for construction of the required ditches parallel to the standard roadbed section. The size of ditches will vary depending upon the flow and terrain and should be designed accordingly.

5. The shoring system must be designed to support the theoretical embankment shown for zones A and B.

6. Any excavation, holes or trenches on the Railroad property shall be covered, guarded and/or protected. Handrails, fence, or other barrier methods must meet OSHA and FRA requirements. Temporary lighting may also be required by the Railroad to identify tripping hazards to train crewmen and other Railroad personnel.

7. The most stringent project specifications of the Public Utilities Commission Orders, Department of Industrial Safety, OSHA, FRA, AREMA, BNSF, UPRR or other governmental agencies shall be used.

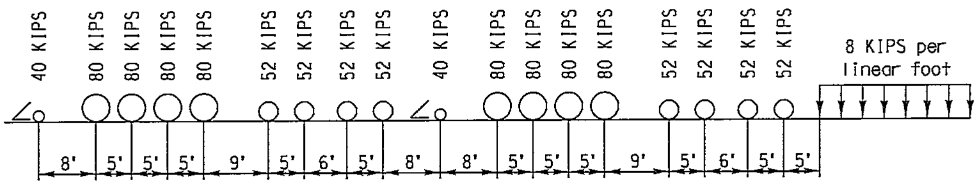
8. Secondhand material is not acceptable unless the Engineer of Record submits a full inspection report which verifies the material properties and condition of the secondhand material. The report must be signed and sealed by the Engineer of Record.

9. All components of the shoring system are to be removed when the shoring is no longer needed. All voids must be filled and drainage facilities restored. See compaction requirements section 9, Part 4.

10. Slurry type materials are not acceptable as fill for soldier piles in drilled holes. Concrete and flowable backfill may prevent removal of the shoring system. Use compacted peagravel material.

7. COMPUTATION OF APPLIED FORCES

1. Railroad live load and lateral forces.
   a. For specific applications of the Coopers E80 live load refer to **Figure 2** on the next page which illustrates Plan Number **710001 "LIVE LOAD PRESSURE DUE TO COOPER E80"**. Supplemental information and sample calculations are provided in the Appendix pages A-1 through A-4.
2. Dead load.
   a. Spoil pile: must be included assuming a minimum height of two feet of soil adjacent to the excavation.
   b. Track: use 200 lbs/linear ft for rails, inside guardrails and fasteners.
   c. Roadbed: ballast, including track ties, use 120 lb per cubic foot.



COOPER E80 LOAD

SCALE: (NOT TO SCALE)

Vertical pressure q shall be based on a distribution width Ld.
Ld Is the length of tie plus H_1.
H_1 is the height from the bottom of tie to the top of shoring.
H_2 is the depth of point being evaluated with the Boussinesq equation.
S is a distance perpendicular from centerline of track to the face of shoring.
D is from top of shoring to one foot below dredge line.
Zp is the minimum embedment depth.
Length of tie is 9 feet
q is the intensity of strip load due to E80 Railroad live load and shall be calculated as follows:

For $H_1 = 0$ Ld = length of tie; therefore, $q = \dfrac{80{,}000 \text{ lb}}{(5 \text{ feet})(9 \text{ feet})} = 1{,}778$ psf

For $H_1 > 0$ Ld = length of tie + H_1; therefore, $q = \dfrac{80{,}000 \text{ lb}}{(5 \text{ feet})(Ld)}$

CASE 1: Lateral live load pressure Ps, due to E80 loading for track **parallel** to shoring system is calculated using the Boussinesq Strip Load Equation.

$$Ps = \frac{2q}{\pi}(\beta + \sin\beta \sin^2\alpha - \sin\beta \cos^2\alpha)$$

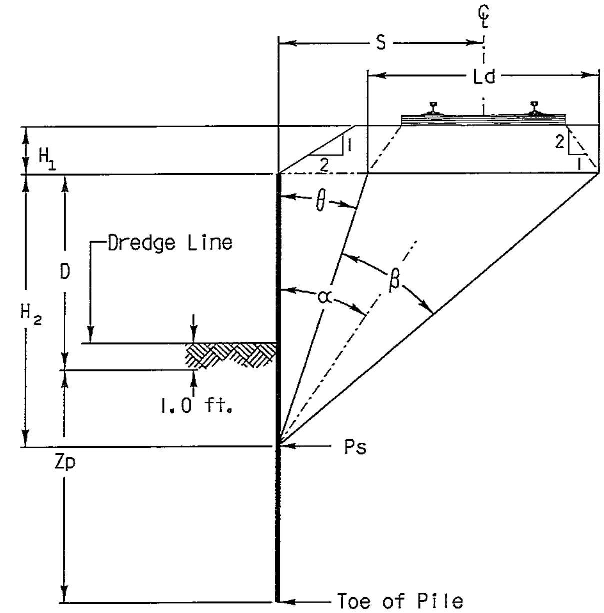
The above equation can be simplified into the following equivalent form:

$$Ps = \frac{2q}{\pi}[\beta - \sin\beta \cos(2\alpha)]$$

α and β are angles measured in radians, $\alpha = \theta + \dfrac{\beta}{2}$

CASE 2: Live load pressure due to E80 loading for track at a **right angle** to the shoring system can be calcuated using the following equation.

$Ps = K_A q$
where $K_A = \tan^2(45 - \frac{\phi}{2})$
ϕ is the angle of internal friction in degrees



PLAN

SCALE: (NOT TO SCALE)

REVISIONS		
DATE	LTR.	DESCRIPTION
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DESIGN BY: PGP | DRAWN BY: JFS | CHECKED BY: AA

APPROVED:

K.H. Jennison
BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J. Meyer 9-1-04
UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BRIDGE SHORING STANDARDS

UNION PACIFIC

LIVE LOAD PRESSURE DUE TO COOPER E80

FILE OWNER: UPRR | DATE:
PLAN NO.: 710001 | SHEET: 1 OF 1

PLOTTED: $$$DATE$$$ $TIME

Figure 2

3. Active earth pressure.

   a. The active earth pressure due to the soil may be computed by the Coulomb Theory or other approved method.

   b. The active earth pressure at depth "z_a" is:

      $P_A = K_A \gamma z_a$, where $K_A = \tan^2(45 - \frac{\phi}{2})$

      z_a = depth of soil influencing the active pressure.

4. Active earth pressure due to unbalanced water pressure.

   a. When bulkheads are used for waterfront construction, the bulkhead is subjected to a maximum earth pressure at the low water stage. During a rainstorm or a rapidly receding high water, the water level behind the bulkhead may be several feet higher than in front of the bulkhead.

   b. Drained conditions in backfill apply when clean sand or clean sand and gravel are used and adequate permanent drainage outlets are provided. Where drained conditions exist, the design water level may be assumed at the drainage outlet elevation.

5. Active earth pressure due to surcharge load.

   The active earth pressure due to surcharge load q':

   $P_U = K_A q'$, where $K_A = \tan^2(45 - \frac{\phi}{2})$

6. Passive earth pressure.

   The passive earth pressure, P_p, in front of the bulkhead may also be computed by the Coulomb Theory.

   $P_p = K_p \gamma z_p$, where $K_P = \tan^2(45 + \frac{\phi}{2})$

   z_p = vertical distance beginning one foot below dredge line but not to exceed embedment depth

7. Pressure due to embankment surcharges.

   Conventional analysis (Rankine, Coulomb, or Log-Spiral) should be used to determine the additional surcharge from embankment slopes.

8. Additional analysis for centrifugal force calculations as described in **AREMA Chapter 15, Part 1, Section 1.3, Article 1.3.6** Centrifugal Loads are required where track curvature exceeds three degrees.

9. Include and compute all other loads that are impacting the shoring system such as a typical Railroad service vehicle (HS-20 truck).

8. STRUCTURAL INTEGRITY

Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the loads and forces in such combinations as stipulated in **AREMA Chapter 8 Part 2 Article 2.2.4b**, which represents various combinations of loads and forces to which a structure may be subjected. Each part of the structure shall be proportioned for the group loads that are applicable, and the maximum design required shall be used.

1. Embedment depth.

   a. Calculated depth of embedment is the embedment depth required to maintain static equilibrium.

b. Minimum depth of embedment is the total depth of embedment required to provide static equilibrium plus additional embedment due to the minimum factor of safety.

   1. Embedment depth factor of safety for well-defined loading conditions and thoroughly determined soil parameters is generally 1.3 for most temporary shoring systems. (See **AREMA 8.20.4.1.c**)

   2. All anchored shoring systems require a minimum embedment depth of 1.5 times the calculated depth of embedment. Shallow penetration into strong soil layers is not acceptable. (See **AREMA 8.20.5.1**)

2. The allowable stresses based on AREMA requirements are as follows:

   Structural Steel: 0.55Fy for Compression in extreme fiber. **(AREMA Ch.15 Table 1-11)**

   Structural Steel: 0.35Fy for Shear. **(AREMA Ch.15 Table 1-11)**

   Sheet Pile Sections: 2/3 of yield strength for steel. **(AREMA 8.20.5.7)**

   Concrete: 1/3 of Compressive strength. **(AREMA 8.20.5.7)**

   Anchor Rods: 1/2 of yield strength for steel. **(AREMA 8.20.5.7)**

3. AISC allowances for increasing allowable stress due to temporary loading conditions are not acceptable.

4. Gravity type temporary shoring systems must also be analyzed for overturning, sliding and global stability.

5. The contractor is responsible for providing an approved test method to verify the capacity of anchored or tieback systems. The manufacturers recommendations for testing must be satisfied. Systems which support the Railroad embankment will be considered high risk in determining the percentage of elements to be proof tested.

6. Calculated deflections of temporary shoring system and top of rail elevation shall not exceed the criteria outlined in **Table 1 Deflection Criteria**.

Table 1 Deflection Criteria

Horizontal distance from shoring to track C/L measured at a right angle from track	Maximum horizontal movement of shoring system	Maximum acceptable horizontal or vertical movement of rail
12' < S < 18'	3/8"	1/4"
18' < S < 24'	1/2"	1/4"

9. SOIL CHARACTERISTICS

1. Subsurface Exploration. **(AREMA 8.5.2.2)**

   a. Sufficient borings shall be made along the length of the structure to determine, with a reasonable degree of certainty, the subsurface conditions. Irregularities found during the initial soil boring program may dictate that additional borings be taken.

   b. The subsurface investigation shall be made in accordance with the provisions of **AREMA Chapter 8 Part 22, Geotechnical Subsurface Investigation.**

2. Type of backfill.

   a. Backfill is defined as material behind the wall, whether undisturbed ground or fill, that contributes to the pressure against the wall.

b. The backfill shall be investigated and classified with reference to the soil types described in **AREMA Table 8-5-1**.

c. Types 4 and 5 backfill shall be used only with the permission of the Engineer. In all cases the wall design shall be based on the type of backfill used.

Table 8-5-1 (AREMA) Types of Backfill for Retaining Walls

Backfill Type	Backfill Description
1	Coarse-grained soil without admixture of fine soil particles, very free-draining (clean sand, gravel or broken stone).
2	Coarse-grained soil of low permeability due to admixture of particles of silt size.
3	Fine silty sand; granular materials with conspicuous clay content; or residual soil with stones.
4	Soft or very soft clay, organic silt; or soft silty clay.
5	Medium or stiff clay that may be placed in such a way that a negligible amount of water will enter the spaces between the chunks during floods or heavy rains.

3. Computation of backfill pressure. **(AREMA 8.5.3.2a)**

   a. Values of the unit weight, cohesion, and angle of internal friction of the backfill material shall be determined directly by means of soil tests or, if the expense of such tests is not justifiable, by means of **AREMA Table 8-5-2** referring to the soil types defined in **AREMA Table 8-5-1**. Unless the minimum cohesive strength of the backfill material can be evaluated reliably, the cohesion shall be neglected and only the internal friction considered. See Appendix page A-6 for AREMA generic soil properties.

Table 8-5-2 (AREMA) Properties of Backfill Materials

Type of Backfill	Unit Weight Lb. Per Cu. Ft.	Cohesion "c"	Angle of Internal Friction
1	105	0	33°-42°(38°for broken stone)
2	110	0	30°
3	125	0	28°
4	100	0	0°
5	120	240	0°

4. Compaction.

   a. The backfill shall preferably be placed in loose layers not to exceed 8 inches in thickness. Each layer shall be compacted before placing the next, but over compaction shall be avoided.

   b. It is required that backfill be compacted to no less than 95% of maximum dry density at a moisture content within 2% of optimum and tested using Modified Proctor ASTM D1557.

   c. Fill within 100 feet of bridge ends or 20 feet outside culverts shall be placed and compacted to not less than 100% of maximum.

   d. No dumping of backfill material shall be permitted in such a way that the successive layers slope downward toward the wall. The layers shall be horizontal or shall slope downward away from the wall.

10. PLANS

The shoring plans must completely identify the site constraints and the shoring system. Use the design templates provided in the appendix as an example to show the required information, specifications and drawings. The specific requirements of the plan submittals are as follows:

1. General plan view should show:
   a. Railroad right-of-way and North arrow.
   b. Position of all Railroad tracks and identify each track as mainline, siding, spur, etc.
   c. Spacing between all existing tracks.
   d. Location of all access roadways, drainage ditches and direction of flow.
   e. Footprint of proposed structure, proposed shoring system and any existing structures if applicable.
   f. Proposed horizontal construction clearances. The minimum allowable is 12 feet measured at a right angle from centerline of track.
   g. Location of existing and proposed utilities.
   h. Drawings must be signed and stamped by a Licensed Professional Engineer, registered in the state where the work will be performed.
   i. Railroad and other "CALL BEFORE YOU DIG" numbers.
   j. Detailed view of shoring along with controlling elevations and dimensions.

2. Typical section and elevation should show:
   a. Top of rail elevations for all tracks.
   b. Offset from the face of shoring system to the centerline of all tracks at all changes in horizontal alignment.
   c. All structural components, controlling elevations and dimensions of shoring system.
   d. All drainage ditches and controlling dimensions.
   e. All slopes, existing structures and other facilities which may surcharge the shoring system.
   f. Location of all existing and proposed utilities.
   g. Total depth of shoring system.

3. General criteria
   a. Design loads to be based on the AREMA manual and Cooper E80 loading.
   b. Pressure due to embankment surcharges.
   c. ASTM designation and yield strength for each material.
   d. Maximum allowable bending stress for structural steel is 0.55Fy.
   e. Temporary overstress allowances are not acceptable.
   f. All timber members shall be Douglas Fir grade 2 or better.
   g. Insitu soil classification.
   h. Backfill soil classification.
   i. Internal angle of friction and unit weight of the soil.
   j. Active and passive soil coefficients.
   k. Fill within 100 feet of bridge ends or 20 feet outside culverts shall be placed and compacted to a minimum of 100% of maximum dry density tested per Modified Proctor ASTM D1557.
   l. Slopes without shoring shall not be steeper than 2 horizontal to 1 vertical

m. Dredge line elevation.

n. Shoring deflection to be calculated and meet Railroad requirements.

4. Miscellaneous:

a. Project name, location, GPS coordinates, track owner, Railroad line segment, milepost and subdivision in the title block.

b. Procedure outlining the installation and removal of the temporary shoring system.

c. General notes specifying material requirements, design data, details, dimensions, cross-sections, sequence of construction etc.

d. A description of the tieback installation including drilling, grouting, stressing information and testing procedures, anchor capacity, type of tendon, anchorage hardware, minimum unbonded lengths, minimum anchor lengths, angle of installation, tieback locations and spacing.

e. All details for construction of drainage facilities associated with the shoring system shall be clearly indicated.

f. Details and descriptions of all shoring system members and connection details.

g. Settlement and displacement calculations.

h. Handrail and protective fence details along the excavation.

i. Drawings must be signed and stamped by a Licensed Professional Engineer, registered in the state where the work will be performed.

j. Call before you dig number.

k. Construction clearance diagram.

11. SUBMITTALS

The Contractor will be responsible for any and all cost associated with the review of plans by the Railroad. Review of design submittals by the Railroad will require a minimum of four (4) weeks. To avoid impacting the construction schedule, the Contractor must schedule submittals well in advance. Partial, incomplete or inadequate designs will be rejected, thus delaying the approval. Revised submittals will follow the same procedure as the initial submittal until all issues are resolved. Submit a minimum of three sets of shoring plans and two sets of calculations with manufacturers' specifications. Drawings and calculations must be signed and stamped by a Registered Professional Engineer familiar with Railway loadings and who is licensed in the state where the shoring system is intended for use. Drawings accompanying the shoring plans shall be submitted on 11" x 17" or 8½" x 11" sized paper.

1. Contractor review.

   The Contractor must review the temporary shoring plans to ensure that the proposed method of construction is compatible with the existing site and soil conditions. The Contractor's work plan must be developed to allow train traffic to remain in service. Removal of the shoring system must also be addressed.

2. Applicant and or Engineer of Record review.

   The applicant and or Engineer of Record must review and approve the submittal for compliance with the project specifications, AREMA Manual, these guidelines and structural capacity before forwarding the submittal to the Railroad.

3. Review process.

All design submittals shall be forwarded to the Railroad Representative who will send them to the Structures Design Department. The Structures Design Department shall review or have an outside consultant review said submittals. If a Railroad consultant performs said review, the consultant may reply directly to the applicant or their representative after consultation with the Structures Design Department. A copy of the reply will be mailed to the Railroad Representative. During the review process the Railroad Representative is the point of contact to resolve outstanding issues.

12. APPENDIX

13. BIBLIOGRAPHY

The following list of references used in these guidelines are placed here in alphabetical order for your convenience.

1. *Manual for Railway Engineering*, 2002 American Railway Engineering and Maintenance-of-Way Association.

2. *TRENCHING AND SHORING MANUAL,* January 1990, Revision 11/12/96. State of California Department of Transportation, Office of Structures Construction.

SAMPLE PROBLEM

Point in question: S = 12 ft H = 6 ft

$$q = \frac{80{,}000\text{ lbs}}{(5\text{ ft})(9\text{ ft})} = 1778\text{ psf}$$ for E80 loading, axle spacing = 5 ft, tie length b = 9 ft

Solve for $X_1 = S - b/2 = 7.5$ ft

Solve for $X_2 = S + b/2 = 16.5$ ft

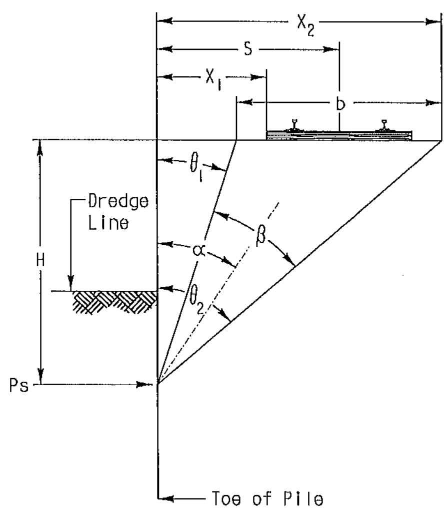
Solve for $\theta_1 = \arctan\left(\frac{X_1}{H}\right) = 0.896$ radians

Solve for $\theta_2 = \arctan\left(\frac{X2}{H}\right) = 1.222$ radians

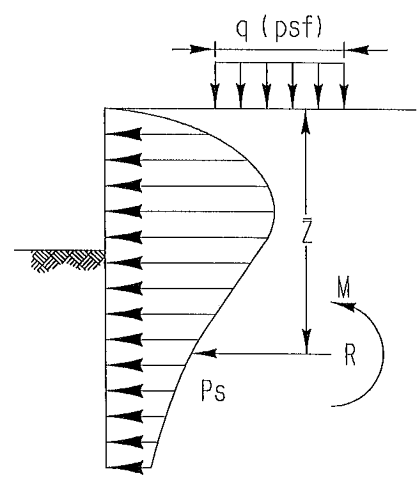
Solve for $\beta = \theta_2 - \theta_1 = 0.326$ radians

Solve for $\alpha = \frac{\theta_1 + \theta_2}{2} = 1.059$ radians

Note: $\tan\alpha \neq \frac{S}{H}$



PRESSURE DISTRIBUTION FOR STRIP LOAD



EQUIVALENT LOADING

- Pressure, Ps due to E80 liveload at the above-identified point:

$$Ps = \frac{2q}{\pi}(\beta - \sin\beta\cos 2\alpha) = \frac{2 * 1778}{\pi}(0.326 - \sin(0.326)\cos(2 * 1.059)) = 558.\text{psf}$$

- Shear due to E80 liveload at the above-identified point:

$$R_x = \frac{2qH\beta}{\pi} = \frac{2 * 1778 * 6 * 0.326}{\pi} = 2214.lbs/\text{ft}$$

- Depth $\bar{z}$ from base of tie:

$$\bar{z} = \frac{H^2\beta - bH + x_2^2\left(\frac{\pi}{2} - \theta_2\right) - x_1^2\left(\frac{\pi}{2} - \theta_1\right)}{2H\beta} = \frac{6^2 * 0.326 - 9 * 6 + 16.5^2\left(\frac{\pi}{2} - 1.222\right) - 7.5^2\left(\frac{\pi}{2} - 0.896\right)}{2 * 6 * 0.326} = 3.77\text{ft}$$

- Moment due to E80 liveload at the above identified point:

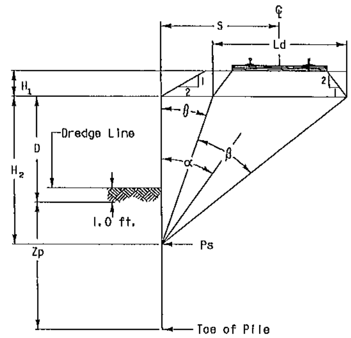
$$M = R_x\left(H - \bar{z}\right) = 2214 * \left(6 - 3.77\right) = 4940 \text{ ft-lbs/ft}$$

Use the above equations to determine Ps, M, R_x & $\bar{z}$ due to the E80 liveload along the **entire** depth of the shoring system. Typically the equations are evaluated on 6" increments to determine the maximum values along the depth of the shoring system. The resultants must be combined with other applicable pressures and loads to evaluate the total loading on the shoring system for the entire depth of the system. Determine the minimum embedment depth required and the minimum cross sectional properties of the shoring system based on the allowable stresses and the required factors of safety.

CHART A

This chart identifies the active pressure and resulting forces due to E80 liveload.
See "SAMPLE PROBLEM" sheet for definitions of variables and equations.

1. Select distance S from track centerline to face of shoring.
2. Select depth H_2 below base of tie.
3. Read Ps, M, R and $\bar{z}$ from the table.
4. Use the procedure outlined in the sample problem to determine values at non-tabulated points.



$$\boxed{Ps = \frac{2q}{\pi}\left[\beta - \sin\beta\cos(2\alpha)\right]}$$

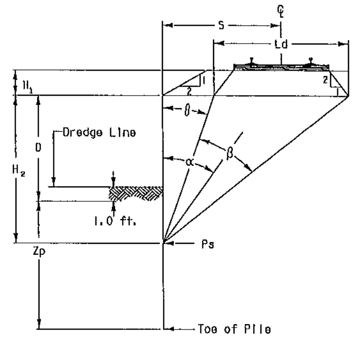
where q = 1778 psf

Boussinesq surcharge pressure E80 live load for H_1=0

Depth below top of shoring H_2 (ft)	Variables		Horizontal distance (S) from shoring to track CL measured at a right angle									
			12	14	16	18	20	22	24	26	28	30
2	**Ps**	(psf)	**305**	**220**	**166**	**130**	**105**	**86**	**72**	**61**	**53**	**46**
	α	(radians)	1.38	1.41	1.44	1.45	1.47	1.48	1.48	1.49	1.50	1.50
	β	(radians)	0.14	0.10	0.07	0.06	0.05	0.04	0.03	0.03	0.02	0.02
	$\bar{z}$	(ft)	1.32	1.33	1.33	1.33	1.33	1.33	1.33	1.33	1.33	1.33
	M	(ft-lbs/ft)	215	152	114	89	71	58	49	41	36	31
	R	(lbs/ft)	317	226	170	132	106	87	73	62	53	46
4	**Ps**	(psf)	**496**	**381**	**299**	**240**	**197**	**164**	**138**	**118**	**102**	**89**
	α	(radians)	1.21	1.27	1.31	1.34	1.36	1.38	1.40	1.41	1.43	1.44
	β	(radians)	0.25	0.19	0.14	0.11	0.09	0.07	0.06	0.05	0.05	0.04
	$\bar{z}$	(ft)	2.59	2.61	2.63	2.64	2.64	2.65	2.65	2.65	2.65	2.66
	M	(ft-lbs/ft)	1,609	1,165	882	692	557	459	384	327	281	244
	R	(lbs/ft)	1,141	840	643	508	411	339	285	242	209	182
6	**Ps**	(psf)	**558**	**461**	**381**	**317**	**266**	**225**	**193**	**167**	**146**	**128**
	α	(radians)	1.06	1.13	1.19	1.23	1.27	1.29	1.32	1.34	1.35	1.37
	β	(radians)	0.33	0.25	0.20	0.16	0.13	0.11	0.09	0.08	0.07	0.06
	$\bar{z}$	(ft)	3.77	3.83	3.88	3.90	3.92	3.94	3.95	3.96	3.96	3.97
	M	(ft-lbs/ft)	4,944	3,674	2,830	2,244	1,822	1,508	1,269	1,082	933	813
	R	(lbs/ft)	2,214	1,696	1,332	1,070	877	731	618	529	458	400
8	**Ps**	(psf)	**535**	**476**	**414**	**358**	**309**	**268**	**234**	**205**	**181**	**160**
	α	(radians)	0.94	1.02	1.08	1.13	1.17	1.21	1.24	1.26	1.29	1.30
	β	(radians)	0.37	0.29	0.24	0.19	0.16	0.14	0.12	0.10	0.09	0.08
	$\bar{z}$	(ft)	4.84	4.97	5.06	5.11	5.16	5.19	5.21	5.23	5.24	5.26
	M	(ft-lbs/ft)	10,481	8,006	6,286	5,051	4,141	3,452	2,920	2,501	2,165	1,892
	R	(lbs/ft)	3,316	2,641	2,134	1,751	1,456	1,228	1,047	903	786	689
10	**Ps**	(psf)	**474**	**449**	**411**	**370**	**329**	**293**	**260**	**232**	**207**	**186**
	α	(radians)	0.83	0.92	0.99	1.04	1.09	1.13	1.17	1.19	1.22	1.24
	β	(radians)	0.38	0.32	0.26	0.22	0.19	0.16	0.14	0.12	0.10	0.09
	$\bar{z}$	(ft)	5.81	6.02	6.16	6.26	6.34	6.39	6.44	6.47	6.50	6.52
	M	(ft-lbs/ft)	18,145	14,227	11,385	9,280	7,689	6,463	5,502	4,736	4,117	3,610
	R	(lbs/ft)	4,328	3,571	2,964	2,482	2,099	1,792	1,544	1,341	1,175	1,037
12	**Ps**	(psf)	**404**	**403**	**386**	**360**	**331**	**302**	**274**	**248**	**225**	**204**
	α	(radians)	0.75	0.83	0.90	0.96	1.01	1.06	1.10	1.13	1.16	1.18
	β	(radians)	0.38	0.33	0.28	0.24	0.20	0.18	0.15	0.13	0.12	0.11
	$\bar{z}$	(ft)	6.68	6.97	7.18	7.34	7.46	7.55	7.61	7.67	7.71	7.75
	M	(ft-lbs/ft)	27,703	22,237	18,121	14,980	12,550	10,641	9,121	7,895	6,894	6,068
	R	(lbs/ft)	5,207	4,424	3,763	3,214	2,762	2,389	2,080	1,823	1,608	1,427
14	**Ps**	(psf)	**338**	**351**	**349**	**337**	**319**	**298**	**276**	**255**	**234**	**215**
	α	(radians)	0.68	0.76	0.83	0.89	0.94	0.99	1.03	1.07	1.10	1.13
	β	(radians)	0.38	0.33	0.28	0.25	0.22	0.19	0.17	0.15	0.13	0.12
	$\bar{z}$	(ft)	7.46	7.85	8.13	8.35	8.51	8.64	8.74	8.82	8.89	8.94
	M	(ft-lbs/ft)	38,880	31,856	26,395	22,116	18,729	16,021	13,831	12,043	10,568	9,339
	R	(lbs/ft)	5,948	5,178	4,499	3,913	3,414	2,990	2,631	2,327	2,068	1,847
16	**Ps**	(psf)	**280**	**301**	**310**	**308**	**300**	**286**	**271**	**254**	**237**	**220**
	α	(radians)	0.62	0.70	0.77	0.83	0.88	0.93	0.97	1.01	1.04	1.07
	β	(radians)	0.36	0.32	0.28	0.25	0.22	0.20	0.18	0.16	0.14	0.13
	$\bar{z}$	(ft)	8.17	8.64	9.01	9.29	9.51	9.68	9.82	9.93	10.03	10.10
	M	(ft-lbs/ft)	51,411	42,880	36,066	30,598	26,183	22,590	19,644	17,207	15,175	13,468
	R	(lbs/ft)	6,563	5,829	5,158	4,560	4,034	3,576	3,179	2,837	2,540	2,284

This chart identifies the active pressure and resulting forces due to E80 liveload. See "SAMPLE PROBLEM" sheet for definitions of variables and equations.

1. Select distance S from track centerline to face of shoring.
2. Select depth H_2 below base of tie.
3. Read Ps, M, R and $\bar{z}$ from the table.
4. Use the procedure outlined in the sample problem to determine values at non-tabulated points.



$$\boxed{Ps = \frac{2q}{\pi}[\beta - \sin\beta\cos(2\alpha)]}$$

where q = 1778 psf

Boussinesq surcharge pressure E80 live load for H_1=0

Depth below top of shoring H_2 (ft)	Variables		Horizontal distance (S) from shoring to track CL measured at a right angle									
			12	14	16	18	20	22	24	26	28	30
18	**Ps**	(psf)	**231**	**256**	**271**	**277**	**276**	**269**	**259**	**247**	**234**	**220**
	α	(radians)	0.57	0.64	0.71	0.77	0.82	0.87	0.92	0.96	0.99	1.02
	β	(radians)	0.35	0.31	0.28	0.25	0.23	0.20	0.18	0.16	0.15	0.13
	$\bar{z}$	(ft)	8.80	9.37	9.81	10.16	10.44	10.67	10.85	11.00	11.12	11.22
	M	(ft-lbs/ft)	65,062	55,110	46,976	40,313	34,834	30,304	26,536	23,384	20,728	18,477
	R	(lbs/ft)	7,072	6,386	5,739	5,145	4,609	4,132	3,710	3,338	3,012	2,725
20	**Ps**	(psf)	**191**	**217**	**236**	**246**	**250**	**249**	**244**	**237**	**227**	**217**
	α	(radians)	0.52	0.59	0.66	0.72	0.77	0.82	0.87	0.91	0.94	0.98
	β	(radians)	0.33	0.30	0.28	0.25	0.23	0.21	0.19	0.17	0.15	0.14
	$\bar{z}$	(ft)	9.37	10.03	10.56	10.98	11.32	11.59	11.82	12.01	12.16	12.30
	M	(ft-lbs/ft)	79,641	68,368	58,973	51,137	44,586	39,093	34,465	30,548	27,216	24,367
	R	(lbs/ft)	7,493	6,859	6,245	5,668	5,135	4,651	4,214	3,822	3,474	3,163
22	**Ps**	(psf)	**159**	**184**	**204**	**217**	**225**	**228**	**227**	**223**	**217**	**210**
	α	(radians)	0.49	0.55	0.62	0.67	0.73	0.77	0.82	0.86	0.90	0.93
	β	(radians)	0.31	0.29	0.27	0.25	0.23	0.21	0.19	0.17	0.16	0.14
	$\bar{z}$	(ft)	9.89	10.64	11.24	11.73	12.14	12.47	12.74	12.97	13.17	13.33
	M	(ft-lbs/ft)	94,986	82,497	71,913	62,945	55,341	48,878	43,370	38,658	34,611	31,122
	R	(lbs/ft)	7,842	7,260	6,684	6,131	5,611	5,128	4,685	4,283	3,918	3,590
24	**Ps**	(psf)	**133**	**157**	**176**	**191**	**202**	**207**	**210**	**209**	**206**	**201**
	α	(radians)	0.45	0.52	0.58	0.63	0.68	0.73	0.78	0.82	0.85	0.89
	β	(radians)	0.30	0.28	0.26	0.24	0.22	0.20	0.19	0.17	0.16	0.15
	$\bar{z}$	(ft)	10.35	11.19	11.87	12.44	12.90	13.29	13.62	13.89	14.13	14.32
	M	(ft-lbs/ft)	110,969	97,366	85,670	75,625	66,997	59,577	53,183	47,661	42,875	38,716
	R	(lbs/ft)	8,132	7,600	7,064	6,540	6,037	5,564	5,122	4,715	4,342	4,001
26	**Ps**	(psf)	**112**	**134**	**153**	**168**	**180**	**188**	**192**	**194**	**193**	**191**
	α	(radians)	0.42	0.48	0.54	0.60	0.65	0.69	0.74	0.78	0.82	0.85
	β	(radians)	0.28	0.27	0.25	0.23	0.22	0.20	0.19	0.17	0.16	0.15
	$\bar{z}$	(ft)	10.78	11.69	12.45	13.09	13.62	14.07	14.44	14.77	15.04	15.28
	M	(ft-lbs/ft)	127,485	112,863	100,135	89,071	79,460	71,105	63,836	57,499	51,963	47,113
	R	(lbs/ft)	8,376	7,890	7,393	6,899	6,418	5,959	5,524	5,118	4,741	4,393
28	**Ps**	(psf)	**94**	**114**	**132**	**148**	**160**	**169**	**175**	**179**	**180**	**180**
	α	(radians)	0.40	0.46	0.51	0.56	0.61	0.66	0.70	0.74	0.78	0.81
	β	(radians)	0.27	0.26	0.24	0.23	0.21	0.20	0.19	0.17	0.16	0.15
	$\bar{z}$	(ft)	11.17	12.16	12.99	13.70	14.29	14.80	15.23	15.60	15.91	16.19
	M	(ft-lbs/ft)	144,448	128,896	115,211	103,191	92,642	83,385	75,258	68,113	61,823	56,274
	R	(lbs/ft)	8,581	8,137	7,677	7,214	6,758	6,315	5,892	5,491	5,115	4,764
30	**Ps**	(psf)	**80**	**98**	**115**	**130**	**142**	**152**	**160**	**165**	**167**	**168**
	α	(radians)	0.37	0.43	0.48	0.53	0.58	0.63	0.67	0.71	0.74	0.78
	β	(radians)	0.26	0.25	0.23	0.22	0.21	0.20	0.18	0.17	0.16	0.15
	$\bar{z}$	(ft)	11.52	12.59	13.49	14.26	14.92	15.48	15.97	16.38	16.75	17.06
	M	(ft-lbs/ft)	161,789	145,388	130,819	117,903	106,466	96,343	87,381	79,443	72,404	66,153
	R	(lbs/ft)	8,755	8,349	7,925	7,492	7,060	6,636	6,227	5,834	5,462	5,112
32	**Ps**	(psf)	**69**	**85**	**101**	**115**	**127**	**137**	**145**	**151**	**155**	**157**
	α	(radians)	0.35	0.41	0.46	0.51	0.55	0.60	0.64	0.68	0.71	0.75
	β	(radians)	0.25	0.24	0.22	0.21	0.20	0.19	0.18	0.17	0.16	0.15
	$\bar{z}$	(ft)	11.85	12.98	13.95	14.79	15.51	16.13	16.67	17.13	17.54	17.89
	M	(ft-lbs/ft)	179,452	162,274	146,888	133,136	120,859	109,909	100,144	91,432	83,655	76,706
	R	(lbs/ft)	8,904	8,532	8,140	7,736	7,329	6,925	6,531	6,150	5,785	5,438

GUIDELINE & WEBSITE DIRECTORY

BNSF guidelines are as follows:

a. Guidelines for Design and Construction of Grade Separation Structures.

UPRR guidelines are as follows:

a. **Underpass Structures** – "Guidelines for Design and Construction of Grade Separation Underpass Structures."
b. **Overhead Grade Separation** – "Guidelines for Design of Highway Separation Structures Over Railroad (Overhead Grade Separation)."
c. **Demolition** – "Guidelines for Preparation of a Bridge Demolition and Removal Plan for Structures Over Railroad."
d. **Shoofly** – "Guidelines for Design and Construction of Shoofly (Detour) Tracks."
e. **Fiber Optic** – "UPRR Fiber Optic Engineering, Construction And Maintenance Standards." 1/1/2002
f. **Pipeline** – "Pipeline Installation" available at www.uprr.com.
g. **Industry Track** – "Technical Specification for Construction of Industrial Tracks"

WEBSITE DIRECTORY:

1. www.astm.org
2. www.arema.org
3. www.bnsf.com
4. www.pilespecs.com
5. www.uprr.com

AREMA Table 8-20-1. Granular Soils

Descriptive Term for Relative Density	Standard Penetration Test Blows per Foot "N"
Very Loose	0 - 4
Loose	4 - 10
Medium	10 - 30
Dense	30 - 50
Very Dense	Over 50

AREMA Table 8-20-2. Silt and Clay Soils

Descriptive Term for Consistency	Unconfined Compressive Strength Tons per Square Foot
Very Soft	Less than 0.25
Soft	0.25 - 0.50
Medium	0.50 - 1.00
Stiff	1.00 - 2.00
Very Stiff	2.00 - 4.00
Hard	Over 4.00

AREMA Table 8-20-3. Unit Weights of Soils, and Coefficients of Earth Pressure

Type of Soil	Unit Weight of Moist Soil, γ (Note 1)		Unit Weight of Submerged Soil, γ' (Note 1)		Coefficient of Active Earth Pressure, K_A				Coefficient of Passive Earth Pressure, Kp		
					For Backfill	For Soils in Place	Friction Angles (Note 2)		For Soils in Place	Friction Angles (Note 2)	
	Minimum	Maximum	Minimum	Maximum			ϕ	δ		ϕ	δ
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Clean Sand:											
Dense	110	140	65	78		0.20	38	20	9.0	38	25
Medium	110	130	60	68		0.25	34	17	7.0	34	23
Loose	90	125	56	63	0.35	0.30	30	15	5.0	30	20
Silty Sand:											
Dense	110	150	70	88		0.25			7.0		
Medium	95	130	60	68		0.30			5.0		
Loose	80	125	50	63	0.50	0.35			3.0		
Silt and Clay (Note 3)	$\frac{165(1+w)}{1+2.65w}$		$\frac{103}{1+2.65w}$		1.00	$1-\frac{q_u}{\bar{p}+\gamma\bar{z}}$			$1+\frac{q_u}{\bar{p}+\gamma\bar{z}}$		

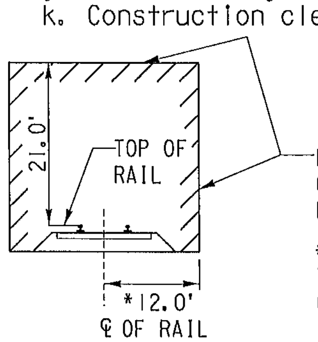
Note 1: In pounds per cubic foot.
Note 2: These angles, expressed in degrees, are ϕ, the angle of internal friction, and δ, the angle of wall friction, and are used in estimating the coefficients under which they are listed.
Note 3: The symbol γ represents γ or γ', whichever is applicable; $\bar{p}$ is the effective unit pressure on the top surface of the stratum; q_u is the unconfined compressive strength; w is the natural water content, in percentage of dry weight; and z is the depth below the top surface of the stratum.

General criteria:

a. Design loads to be based on the AREMA manual and Cooper E80 loading.
b. Pressure due to embankment surcharges.
c. ASTM designation and yield strength for each material.
d. Maximum allowable bending stress for steel is 0.55Fy.
e. Temporary overstress allowances are not acceptable.
f. All timber members shall be Douglas Fir Grade 2 or better.
g. Insitu soil classification.
h. Backfill soil classification.
i. Internal angle of friction and unit weight of soil.
j. Active and passive soil coefficients.
k. Backfill compacted to a minimum of 95% Proctor density per ASTM D-1557.
l. Slopes without shoring shall not be steeper than 2 horizontal to 1 vertical.
m. Dredge line elevation.
n. Shoring deflection to be calculated and meet Railroad requirements.

Miscellaneous:

a. Project name, location, GPS coordinates, track owner, Railroad line segment, milepost and subdivision in the title block.
b. Procedure outlining the installation and removal of the temporary shoring system.
c. General notes specifying material requirements, design data, details, dimensions and cross-sections, sequence of construction etc.
d. A description of tieback installation including drilling, grouting, stressing information and testing procedures, anchor capacity, type of tendon, anchorage hardware, minimum unbonded lengths, minimum anchor lengths, angle of installation, tieback locations and spacing.
e. All details for construction of drainage facilities associated with the shoring system shall be clearly indicated.
f. Details and descriptions of all shoring system members and connection details.
g. Settlement and displacement calculations.
h. Handrail and protective fence details along the excavations.
i. Drawings must be signed and stamped by a Licensed Professional Engineer, registered in the state where the work will be performed.
j. Call before you dig number.
k. Construction clearances diagram as shown below.



NO CONSTRUCTION ACTIVITIES OR OTHER OBSTRUCTIONS MAY BE PLACED WITHIN THESE LIMITS.

*ADD 1.5 INCHES PER DEGREE OF TRACK CURVATURE TO THE HORIZONTAL CLEARANCE DISTANCE.

MINIMUM CONSTRUCTION CLEARANCES

(NORMAL TO RAILROAD)

Not to scale

DESIGN BY:	NAME & LOGO OF ENGINEERING FIRM OR PROJECT OWNER		
DRAWN BY:			
SCALE:	GENERAL CRITERIA AND MISCELLANEOUS		
DRAWING NO:			
SHEET: 1 of 3	RR M.P.		SUBDIVISION
DOT#:	CITY	COUNTY	STATE
DATE:	PROJECT NAME & LOCATION		

General plan view should show:

a. Railroad right-of-way and North arrow.
b. Position of all Railroad tracks and identify each track as mainline, siding, spur, etc.
c. Spacing between all existing tracks.
d. Location of all access roadways, drainage ditches and direction of flow.
e. Footprint of proposed structure, proposed shoring system and any existing structures if applicable.
f. Proposed horizontal construction clearances. The minimum allowable is 12 feet measured at a right angle from centerline of track.
g. Location of existing and proposed utilities.
h. Drawings must be signed and stamped by a Licensed Professional Engineer, registered in the state where the work will be performed.
i. Railroad and other "CALL BEFORE YOU DIG" numbers.
j. Detailed view of shoring along with controlling elevations and dimensions.

DESIGN BY:	NAME & LOGO OF ENGINEERING FIRM OR PROJECT OWNER
DRAWN BY:	
SCALE:	GENERAL PLAN VIEW
DRAWING NO:	
SHEET: 2 OF 3	RR M.P. SUBDIVISION
DOT#:	city COUNTY STATE
DATE:	PROJECT NAME & LOCATION

Typical section and elevation should show:

a. Top of rail elevations for all tracks.
b. Offset from the face of shoring system to the centerline of all tracks at all changes in horizontal alignment.
c. All structural components, controlling elevations and dimensions of shoring system.
d. All drainage ditches and controlling dimensions.
e. All slopes, existing structures and other facilities which may surcharge the shoring system.
f. Location of all existing and proposed utilities.
g. Total depth of shoring system.

DESIGN BY:	NAME & LOGO OF ENGINEERING FIRM OR PROJECT OWNER	
DRAWN BY:		
SCALE:	TYPICAL SECTION & ELEVATION VIEW	
DRAWING NO:		
SHEET: 3 of 3	RR M.P.	SUBDIVISION
DOT#:	CITY COUNTY	STATE
DATE:	PROJECT NAME & LOCATION	

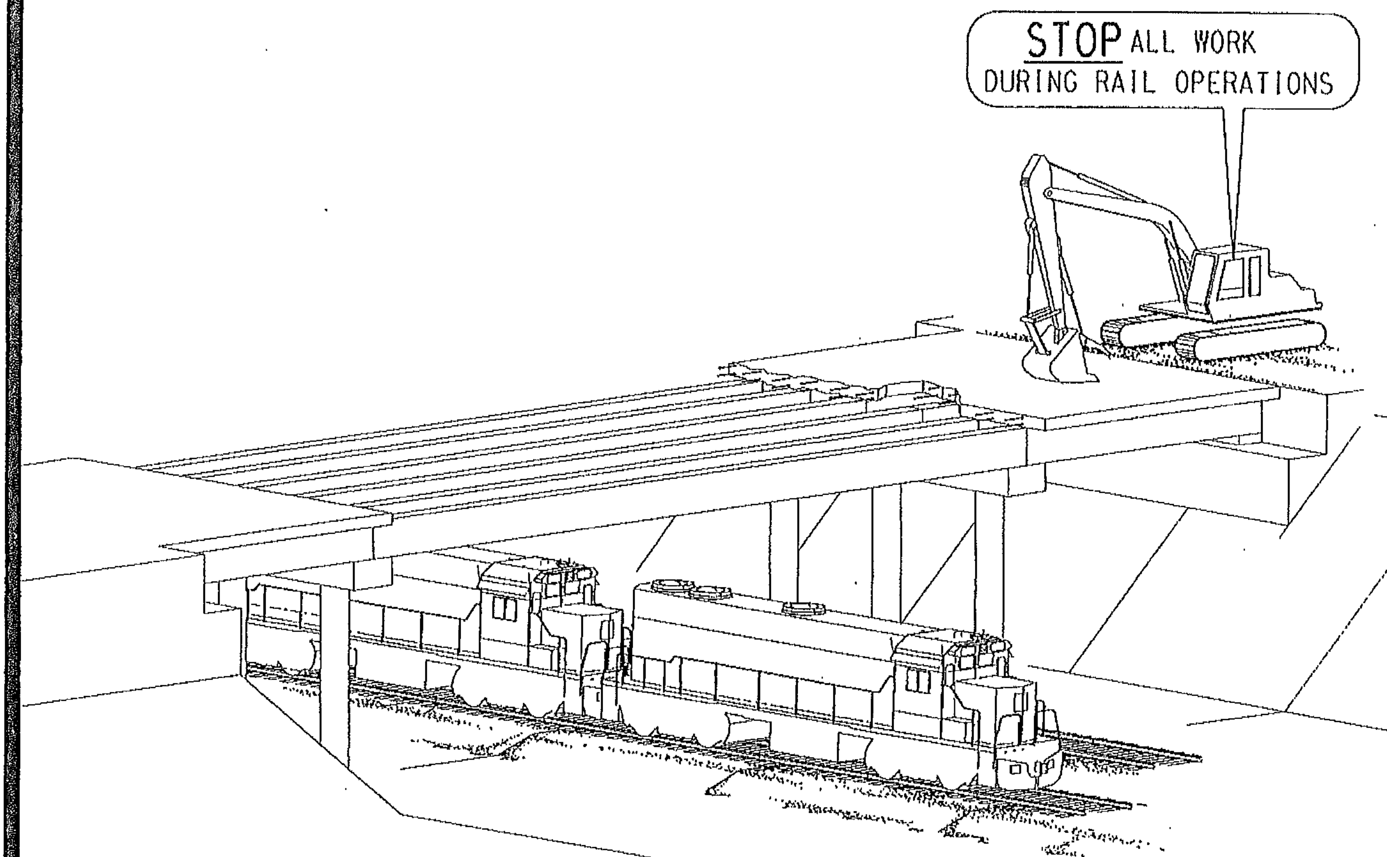
RAILROAD RELATIONS

Railroad's Demolition Guidelines for Overpass Structures

EXHIBIT G

Railroad's Demolition Guidelines for Overpass Structure

GUIDELINES FOR PREPARATION OF A BRIDGE DEMOLITION AND REMOVAL PLAN FOR STRUCTURES OVER RAILROAD



UNION PACIFIC

UNION PACIFIC RAILROAD

OFFICE OF CHIEF ENGINEER DESIGN
1416 DODGE ST.
OMAHA. NE 68179

INDEX

I. GENERAL

A. The Contractor's work shall in no way impede the train operations of the Union Pacific Railroad.

B. The Contractor shall develop a work plan assuming that minimal track windows will be available.

C. The Contractor shall be responsible for planning and executing all procedures necessary to remove the overhead bridge in a safe and controlled manner.

D. The Railroad's tracks and property shall be protected at all times.

E. The contractor shall ensure the area immediately adjacent to operational tracks shall remain free from stumble or like hazards to the ground Railroad personnel to prevent injuries. Open excavations shall be in accordance with current CE Drawing 106613 and shall be protected by appropriate fencing.

F. The words "demolition" and "removal" will be used interchangeably.

G. All removed materials shall be disposed of outside the Railroad right-of-way at no expense to the Railroad.

H. No work is allowed within 50 feet of the nearest rail when trains pass the work site.

I. Staged demolition of the portions of structure immediately adjacent to operational tracks will not jeopardize the integrity of the structure over said tracks until actual removal of the portion of the structure over the tracks is being done.

J. A flagman is required when any work is performed within 25 feet of the nearest rail.

K. No blasting will be permitted on Railroad's right-of-way.

II. BRIDGE REMOVAL PLAN

A. The Contractor shall submit a complete Bridge Removal Plan to the Railroad. The Bridge Removal Plan shall include details, procedures and the sequence of staged removal of the bridge, including all steps necessary to remove the bridge in a safe and controlled manner.

B. The Contractor shall submit to the Railroad; three (3) complete sets of the Bridge Removal Plan for review and comments. The Plan shall be sealed by a Civil or Structural Engineer registered in the state where the proposed demolition will take place. A minimum of three (3) weeks shall be allowed for the Railroad's review after the complete submittal is received. No removal operations will be permitted over the Railroad right of way until the submitted material has been reviewed and comments provided.

C. Review and comment of the Removal Plan by the Railroad will not relieve the Contractor of the ultimate responsibility and liability for the demolition of the structure.

D. The Removal Plan shall include the following:

1) Plan, elevation and location of the bridge, and the locations of any access roads needed for movement of the equipment. The as-built drawings may be used for the submittal provided the removal steps are clearly marked and legible.

2) Indicate the position of all railroad tracks below the bridge and identify each track as mainline, siding, spur, etc.

3) Bridge removal sequence and procedures for entire bridge including the staging for the removal of the superstructure and substructure.

4) List type and number of equipment required and their locations during demolition operations.

5) Locations and types of temporary supports, shoring or bracing required. These members shall be designed to meet Union Pacific Railroad current standard drawing 106613 "General Shoring Requirements", "Guidelines for Design and Construction of Falsework for Structures Over Union Pacific Railroad", "Guidelines for Design and Construction of Shoring Adjacent to Active Railroad Tracks", and the appropriate local and national building and design code requirements.

6) The proposed vertical and horizontal clearance from all tracks to the temporary and permanent supports. The minimum vertical and horizontal clearances shall be as per attached frame protection details.

7) If any temporary supports interfere with the natural drainage along the Railroad right-of-way, a temporary drainage plan shall be submitted for review and comment prior to constructing temporary supports. The proposed drainage plan shall route all drainage away from the railroad tracks.

8) Details, limits, and locations of protective covers or other measures proposed to be used to protect the tracks. This includes any shields or other measures that will protect the tracks from falling debris during removal of the overhead bridge and from any debris rolling down the side slopes or otherwise coming into the area around the tracks which could affect train operations. Design loads, including impact loads, shall be noted. In addition equipment should be on site capable of removing debris and track shield from operational tracks.

9) All procedures necessary to remove the bridge in a safe and controlled manner. The estimated time for complete removal over the tracks shall be noted.

10) All overhead and underground utilities in the area affected by removal of the bridge shall be located on the drawings, including any fiber optic, railroad signal, and communication lines.

11) The location and details of track crossings required for moving of the equipment across the railroad tracks.

12) Limits of demolition of substructures.

13) Details of on-site fire suppression.

III. PROCEDURE

A. During removal operations the remaining structure shall be stable during all stages of the removal operations.

B. Prior to proceeding with bridge removal the sealing Civil or Structural Engineer, or his authorized representative working for the Contractor, shall inspect the temporary support shoring, including temporary bracing and protective coverings, for conformity with the working drawings. The Engineer shall certify in writing to the Railroad that the work is in conformance with the drawings and that the materials and workmanship are satisfactory. A copy of this certification shall be available at the site of work at all times.

C. Coordinate the removal schedule with the Railroad. All the removal work within the track area shall be performed during the time windows when the trains are not passing the work site.

D. All substructures shall be removed to at least 3 feet below the final finished grade or at least 2 feet below base of rail whichever is lower, unless otherwise specified by the Railroad.

E. All debris and refuse resulting from the work shall be removed from the right of way by the contractor and the premises left in a neat and presentable condition.

F. The work progress shall be reviewed and logged by the Contractor's Engineer. Should an unplanned event occur, the Contractor shall inform the Railroad and submit procedure to correct or remedy the occurrence.

G. Preferably all demolition and beam removal shall be from above. In the case that the beams require removal from below, the beams may temporarily straddle the tracks. The following steps shall be taken:

1) The work shall be scheduled with the Railroad's Service Unit Superintendent subject to the Railroad's operational requirements for continuos train operations. The beams removed in sufficient time for train passage.

2) The tracks shall be protected and no equipment placed on the tracks.

3) The beams shall be blocked and not come in contact with the tracks. Blocking shall not be placed on the tracks.

4) The beams and all equipment will be moved a minimum of 15 feet from the nearest rail of the tracks when a train is passing.

IV. TRACK PROTECTION

A. The track protective cover shall be constructed before beginning bridge removal work and may be supported by falsework or members of the existing structure. See the attached Track Shield Detail and Frame Protection Detail for additional requirements. Types of protective covers that may be acceptable methods for protecting the tracks are:

1) A decking supported by the bridge or a suspended cover from the bridge above the track clearance envelope.

2) A track shield cover over the tracks per the attached detail.

3) A framed cover outside the track clearance envelope.

4) A catcher box or loader bucket under decking and parapets overhanging the exterior girders.

B. Construction equipment shall not be placed on the tracks unless tracks are protected.

C. Temporary haul road crossings shall be either Section Timbers or Precast Concrete Panels. The type of crossing shall be determined by the Manager of Industry and Public Projects. Solid timbers or ballast with timber headers shall be used between multiple tracks. If temporary crossing is accessible to public crossing shall be protected with barricades or locked gates when contractor is not actively working at the site or weekends.

D. Track protection is required for all equipment including rubber tired equipment operating within 25 ft. or over the tracks.

V. CRANES

A. When cranes are operated near the tracks the following is required:

1) Only cranes with the capacity to handle the loads may be used. Front end loaders and backhoes cannot be used to lift over the tracks.

2) The Contractor shall verify that the foundations under the crane can support the loads.

3) The size and material type of crane mats shall be submitted to the Railroad for review and comment. No mat substitution will be allowed. The mats shall be rigid and of sufficient capacity to distribute the crane loads and prevent tipping of the crane.

4) Installation of temporary track crossings for equipment shall be scheduled with the Manager of Industry and Public Projects .

5) Additional track protection is required when crossing with a crane. The protection methods shall be submitted to the Railroad for review and comment.

6) Equipment shall not place outriggers on the tracks or ballast.

7) Cranes shall not be placed within the track clearance envelope without flagman protection.

VI. CUTTING TORCHES

A. When a cutting torch is used near the tracks or any timber, the following steps shall be taken:

1) Fire suppression equipment is required on-site.

2) Do not use a torch over, between, or adjacent to the tracks unless a steel plate protective cover is used. Care shall be taken to make certain the use of a steel plate does not come in contact with the rails. See "Track Shield Details" for other requirements. Details of the shield shall be submitted to the Railroad for approval.

3) Wet the ties and other timber below the cutting area.

4) Monitor the work site for at least three hours after cutting for a smoldering fire.

B. Extensive overhead cutting will not be performed over the track area without the proper fire suppression equipment on-site and proper protection.

VII. UTILITIES

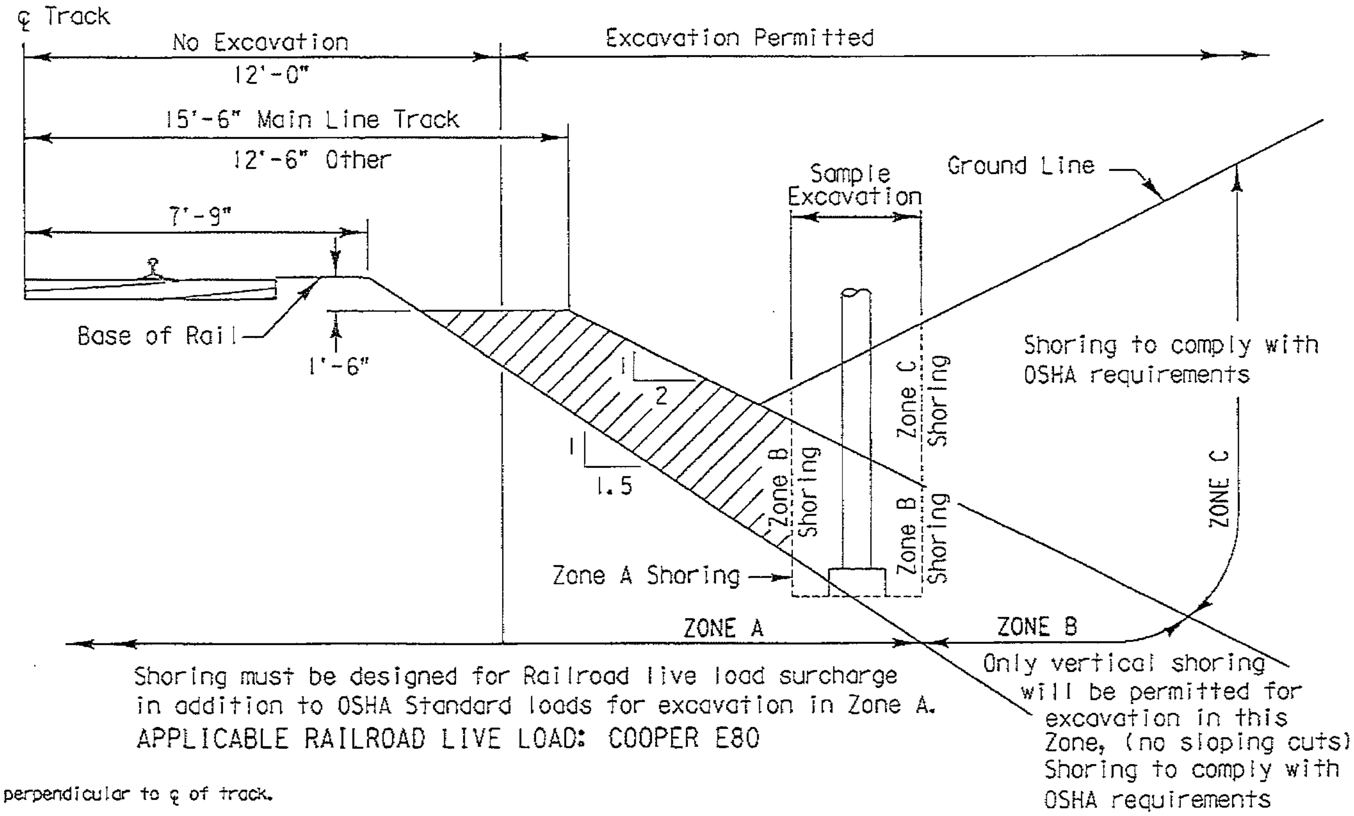
A. The demolition operations shall be planned such that the utility lines are operating safely at all times. The utility lines shall be protected if affected by demolition operations. All the work associated with utility lines should be coordinated by the contractor with the respective utility companies.

VIII. HAZARDOUS MATERIAL

A. If any hazardous materials are found, provide material protection as specified in local hazardous material codes and immediately contact the Railroad.

APPENDIX

- GENERAL SHORING REQUIREMENTS
- LIVE LOAD PRESSURE DUE TO COOPER E80
- TRACK SHIELD DETAIL
- FRAME PROTECTION DETAILS, sheet 1 of 2
- FRAME PROTECTION DETAILS, sheet 2 of 2



GENERAL NOTES:

All dimensions are measured perpendicular to ℄ of track.

Prior to commencing any work, the contractor shall submit for approval by the Railroad detailed plans indicating the nature and extent of the track protection shoring proposed. The contractor shall install the temporary shoring system per the approved plans. Design of the temporary shoring system to comply with **GUIDELINES FOR TEMPORARY SHORING.**

For excavations which encroach into zone A or B, shoring plans shall be accompanied by design calculations. Plans and calculations must be signed and stamped by a Professional Engineer registered in the state where the work will be performed.

GENERAL EXCAVATION ZONES

SCALE: (NOT TO SCALE)

REVISIONS		
DATE	LTR.	DESCRIPTION
5/03	1	FORMERLY UPRR C.E. 106613
/		
/		
/		
/		
/		

DESIGN BY: PGP | DRAWN BY: JFS | CHECKED BY: AA

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

9-1-04

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

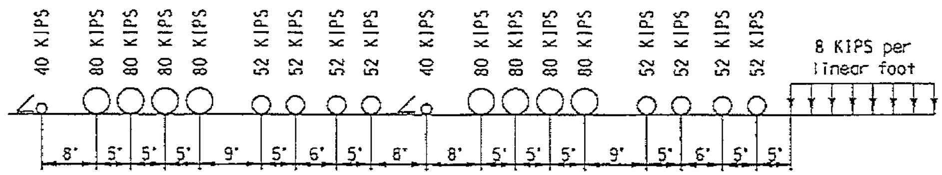
BRIDGE STANDARDS

GENERAL SHORING REQUIREMENTS

FILE OWNER: UPRR | DATE: 5-6-03

PLAN NO.: 710000 | SHEET: 1 OF 1

Figure 1



COOPER E80 LOAD

SCALE: (NOT TO SCALE)

Vertical pressure q shall be based on a distribution width Ld.
Ld is the length of tie plus H_1.
H_1 is the height from the bottom of tie to the top of shoring.
H_2 is the depth of point being evaluated with the Boussinesq equation.
S is a distance perpendicular from centerline of track to the face of shoring.
D is from top of shoring to one foot below dredge line.
Zp is the minimum embedment depth.
Length of tie is 9 feet
q is the intensity of strip load due to E80 Railroad live load and shall be calculated as follows:

For $H_1 = 0$ Ld = length of tie; therefore, $q = \frac{80,000 \text{ lb}}{(5 \text{ feet})(9 \text{ feet})} = 1,778$ psf

For $H_1 > 0$ Ld = length of tie + H_1; therefore, $q = \frac{80,000 \text{ lb}}{(5 \text{ feet})(Ld)}$

CASE 1: Lateral live load pressure Ps, due to E80 loading for track **parallel** to shoring system is calculated using the Boussinesq Strip Load Equation.

$$Ps = \frac{2q}{\pi}(\beta + \sin\beta \sin^2\alpha - \sin\beta \cos^2\alpha)$$

The above equation can be simplified into the following equivalent form:

$$Ps = \frac{2q}{\pi}[\beta - \sin\beta \cos(2\alpha)]$$

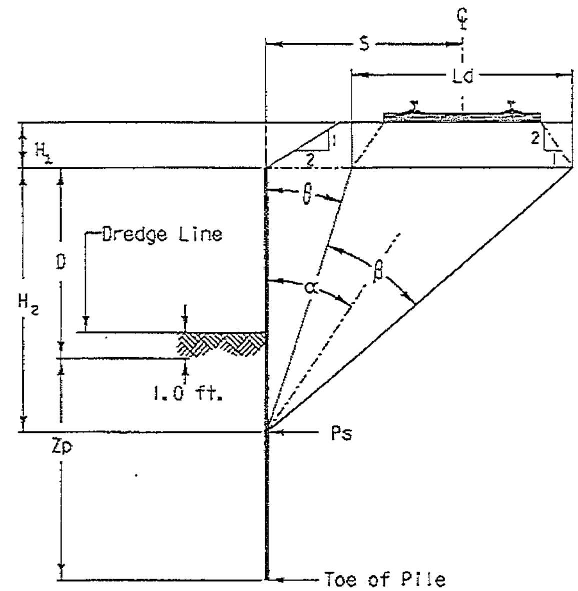
α and β are angles measured in radians, $\alpha = \theta + \frac{\beta}{2}$

CASE 2: Live load pressure due to E80 loading for track at a **right angle** to the shoring system can be calcuated using the following equation.

$Ps = K_A q$

where $K_A = \tan^2(45 - \frac{\phi}{2})$

ϕ is the angle of internal friction in degrees



PLAN

SCALE: (NOT TO SCALE)

REVISIONS		
DATE	LTR.	DESCRIPTION
/		
/		
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/		
/		

DESIGN BY: PGP | DRAWN BY: JFS | CHECKED BY: AA

APPROVED:

K.H. Jennison
BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

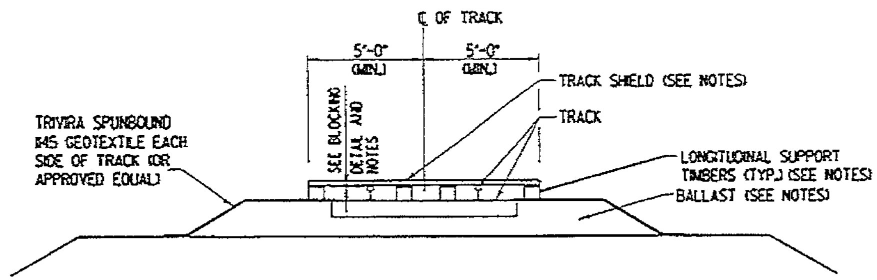
9-1-04
UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BRIDGE SHORING STANDARDS

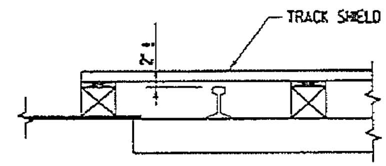
LIVE LOAD PRESSURE DUE TO COOPER E80

FILE OWNER: UPRR | DATE:
PLAN NO.: 710001 | SHEET: 1 OF 1

Figure 2



TRACK SHIELD DETAIL
FOR DEBRIS FALLING FROM BRIDGE DECK REMOVAL
(WHEN TRACK TIME WINDOW IS AVAILABLE)



BLOCKING DETAIL

NOTES:

1. A FLAGMAN IS REQUIRED AT ALL TIMES DURING THE USE OF A TRACK SHIELD.
2. THE TRACK SHIELD SHALL BE DESIGNED BY THE CONTRACTOR AND SHALL BE OF SUFFICIENT STRENGTH TO SUPPORT THE ANTICIPATED LOADS, INCLUDING IMPACT. THE SHIELD SHALL PREVENT ANY MATERIALS, EQUIPMENT OR DEBRIS FROM FALLING ONTO THE RAILROAD TRACK. ADDITIONAL LAYERS OF MATERIALS SHALL BE FURNISHED AS NECESSARY TO PREVENT FINE MATERIALS OR DEBRIS FROM SIFTING DOWN UPON THE TRACK.
3. THE SHIELD SHOULD PREFERABLY BE PREFABRICATED AND FURNISHED WITH LIFTING HOOKS TO SIMPLIFY REMOVAL.
4. THE SHIELD SHALL BE OF SUFFICIENT STRENGTH TO SPAN BETWEEN IT'S SUPPORTS WITHOUT BEARING UPON THE RAILS AND TO WITHSTAND DROPPING RUBBLE.
5. BEFORE REMOVAL, THE SHIELD SHALL BE CLEANED OF ALL DEBRIS AND FINE MATERIAL.
6. THE TRACK SHIELD SHALL EXTEND AT LEAST 20 FEET BEYOND THE LIMITS OF DEMOLITION TRANSVERSE TO THE EDGE OF THE BRIDGE.
7. LONGITUDINAL SUPPORT TIMBERS FOR THE SHIELD SHALL NOT EXTEND ABOVE THE TOP OF RAIL WHEN THE SHIELD IS REMOVED. BLOCKING FROM THE TOP OF RAIL TO THE BOTTOM OF THE SHIELD MAY BE ATTACHED TO THE SHIELD. REMAINING TIMBERS SHALL BE ANCHORED.
8. FOR TRAIN PASSAGE, THE RUBBLE SHALL BE REMOVED TO A MINIMUM OF 8'6" FROM THE NEAREST RAIL AND TO AN ELEVATION NO HIGHER THAN THE TOP OF RAIL.
9. AT THE END OF THE DAY, THE RUBBLE SHALL BE REMOVED COMPLETELY TO A MINIMUM OF 10'0" FROM THE NEAREST RAIL AND DOWN TO ORIGINAL GRADE.
10. CARE SHALL BE TAKEN TO NOT PLACE METAL ACROSS THE TRACK RAILS. RAILROAD COMMUNICATIONS ARE SENT THROUGH THE RAILS AND WILL BE DISRUPTED BY A SHORT BETWEEN RAILS.
11. DETAILS SHOWN APPLY FOR TIMBER TIES. SPECIAL DETAILS ARE REQUIRED FOR CONCRETE TIES.

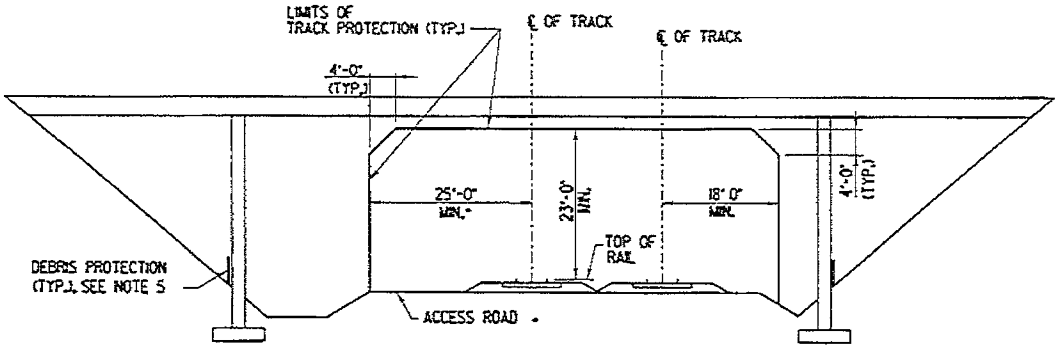
UNION PACIFIC RAILROAD

TRACK SHIELD DETAIL

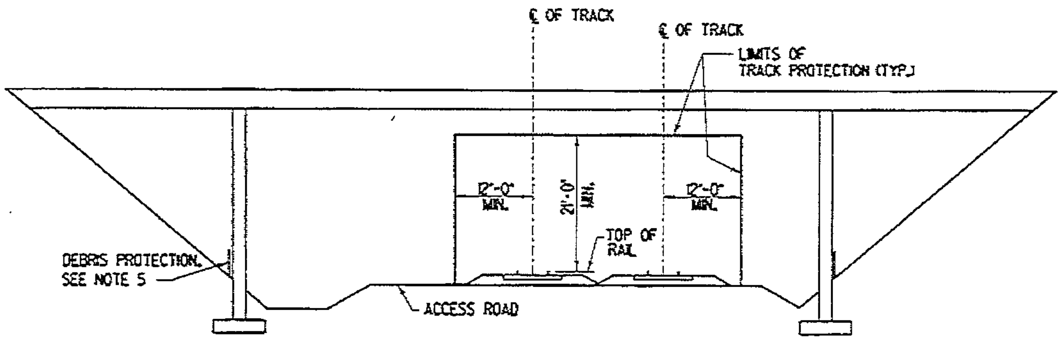
OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98

SHEET 1 OF 1



BRIDGE ELEVATION
STANDARD LIMITS OF PROTECTION FOR FRAME PROTECTION



BRIDGE ELEVATION
MINIMUM LIMITS OF PROTECTION FOR FRAME PROTECTION
(SPECIAL PERMISSION REQUIRED, SEE NOTE 1)

NOTES:

1. THE STANDARD LIMITS OF PROTECTION NOTED ARE THE MIN. CLEARANCES ALLOWED WITHOUT SPECIAL PERMISSION FROM THE RAILROAD. THE REDUCED CLEARANCES NOTED MAY BE ALLOWED BY THE RAILROAD. SPECIAL PERMISSION FOR THE REDUCED CLEARANCES IS REQUIRED FROM THE RAILROAD SERVICE UNIT SUPERINTENDENT.
2. THE PROTECTION FRAME SHALL AS A MINIMUM MATCH THE DEMOLITION LIMITS SHOWN AND EXTEND PAST THE BRIDGE WIDTH AS SHOWN ON THE ATTACHED DEMOLITION PLAN SHEET.
3. FOR ADDITIONAL CLEARANCE AND PROTECTION INFORMATION, SEE UNION PACIFIC RAILROAD STANDARD DRAWING NO. 0035
4. THE PROTECTION FRAME SHALL PREVENT DEMOLITION DEBRIS, DUST AND FINE MATERIAL FROM FALLING ONTO THE RAILROAD TRACKS, ACCESS ROAD OR TRAINS. THE FRAME SHALL BE DESIGNED BY THE CONTRACTOR TO SUPPORT THE ANTICIPATED DEMOLITION LOADS, AND IN ACCORDANCE WITH UNION PACIFIC GUIDELINES FOR DESIGN OF FALSEWORK FOR STRUCTURES OVER THE RAILROAD.
5. DEBRIS PROTECTION IS REQUIRED NEAR THE BASE OF THE SIDE SLOPES AND ADJACENT TO ROADS USED BY DEMOLITION EQUIPMENT TO PREVENT DEBRIS FROM ROLLING ONTO THE TRACK, ACCESS ROAD OR DITCH. USE TIMBERS AS REQUIRED TO STOP LARGE PIECES OF ROLLING DEBRIS.
6. ANY ACTIVITY WITHIN 25 FEET OF THE NEAREST RAIL OF A TRACK REQUIRES A FLAGMAN.

* IF NO ACCESS ROAD, USE MIN. DIMENSION FROM OTHER SIDE OF DETAIL.

UNION PACIFIC RAILROAD

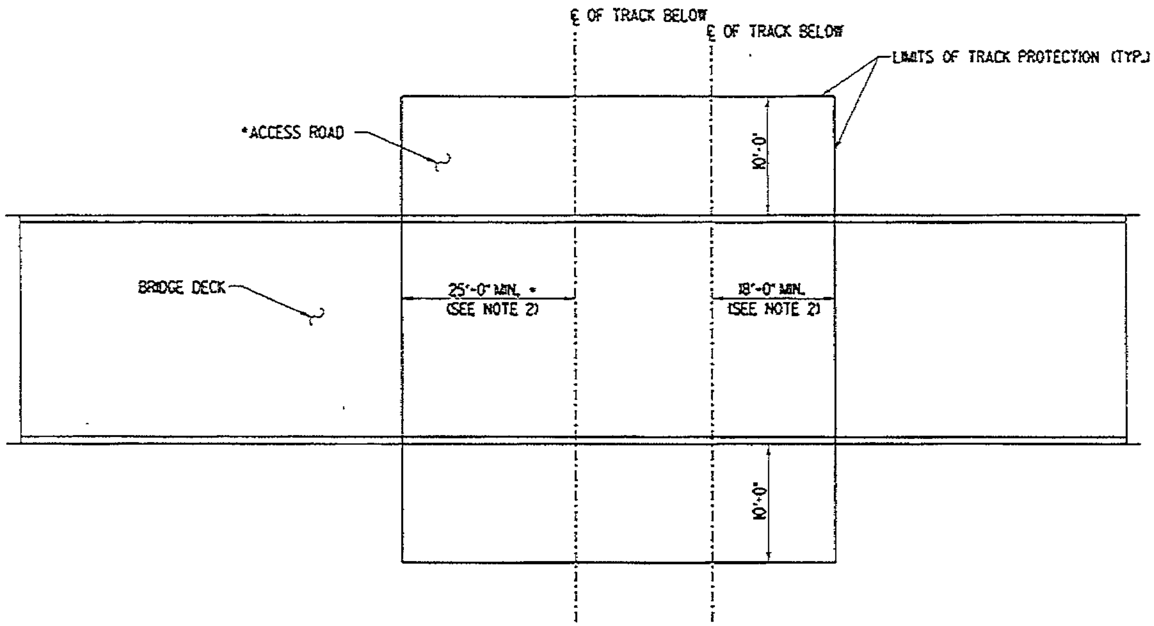
FRAME PROTECTION DETAILS

OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98

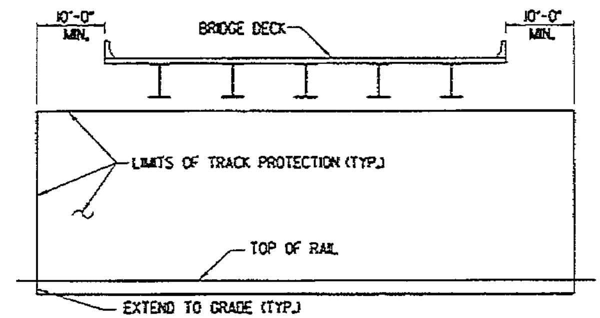
SHEET 1 OF 2

NOTES:
1. SEE GENERAL NOTES ON BRIDGE ELEVATION SHEET
2. STANDARD LIMITS OF PROTECTION ARE SHOWN. FOR MIN. LIMITS OF PROTECTION DIMENSIONS, SEE BRIDGE ELEVATION, MINIMUM LIMITS OF PROTECTION.



BRIDGE PLAN
STANDARD LIMITS OF PROTECTION FOR FRAME PROTECTION

* IF NO ACCESS ROAD, USE MIN. DIMENSION FROM OTHER SIDE



BRIDGE DECK CROSS SECTION
STANDARD LIMITS OF PROTECTION

* IF NO ACCESS ROAD, USE MIN. DIMENSION FROM OTHER SIDE

UNION PACIFIC RAILROAD

FRAME PROTECTION DETAILS

OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98

SHEET 2 OF 2

MATERIALS INFORMATION

Revised Foundation Report for Shasta Viaduct (Replace)

Dated march 12, 2015

M e m o r a n d u m

Serious Drought.
Help Save Water!

To: MR. JOSEPH E. DOWNING
Chief, Bridge Design Branch 3
Office of Bridge Design West
Division of Structure Design

Attention: Mr. Jose M. Aquino III, P.E.

Date: March 12, 2015

File: 02-SHA-5-PM 29.72
EA 02-0E0901
ID 0200000016
Shasta Viaduct
(Replace)
Br. No. 06-0212L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES – MS 5

Subject: Revised Foundation Report for Shasta Viaduct (Replace)

Per your request, the Office of Geotechnical Design North (OGD-N) has prepared this revised Foundation Report (FR) for the proposed Shasta Viaduct (Br. No. 06-0212L), which will replace the existing Sidehill Viaduct (Br. No. 06-0042L). The bridge is located on Interstate 5 approximately 15 miles north of Redding in Shasta County. This revised report supersedes the previous report dated November 21, 2014.

SCOPE OF WORK

This report summarizes the published geologic map and site geologic information, subsurface geologic information from the 2014 subsurface investigation, and seismic surveys. The 2014 subsurface investigation included eight drilled test borings, two suspension P-S velocity logs, a seismic refraction survey, and a tomographic survey. An additional six drilled test borings were drilled in January and February 2015. This report also provides seismic and foundation recommendations. The elevations used in this FR are based on NAVD 88.

PROJECT DESCRIPTION

The proposed new Shasta Viaduct (Br. No. 06-0212L) will replace the existing Sidehill Viaduct (Br. No. 06-0042L). The proposed bridge will shift northeast (downslope) a distance of approximately 90 to 140 feet (existing bridge center line to the proposed new bridge center line). The General Plan (GP) dated October 31, 2014 indicates that the proposed new Shasta Viaduct will be a 8-span cast-in-place/prestressed concrete structure, which includes open spandrel arch spans, supported by two-column bents. Six supports (Abutment 1, Arch Footing 2, Arch Footing 6, Bent 7, Bent 8 and Abutment 9) are on foundations and three supports (Spandrel Bent 3, Isolation Bent 4, and Spandrel Bent 5) are on arches. The proposed new bridge will be approximately 810 feet in length and 42 feet in width.

A retaining wall (Abutment 1 Right Wingwall) is proposed at the south end of the bridge along the downslope side of Abutment 1. The Abutment 1 Layout and Detail Sheets indicate that the retaining wall will be 61.5 feet in length and have a maximum height of 20 feet. The proposed wall type is a soldier pile wall using 24 inch diameter CIDH piles with concrete panel lagging.

FIELD INVESTIGATION AND TESTING PROGRAM

Field Investigation

The field investigation for this project was conducted in 2014 and 2015. The field investigation consisted of fourteen test borings, two suspension P-S velocity logs, a seismic tomographic survey, and a seismic refraction survey.

The test borings were drilled to various depths from 17.5 feet to 150 feet below ground surface (bgs). P-S (Primary-Shear Wave) logging was performed in Borings RC-14-004 and RC-14-002 located at Abutment 1 and Abutment 9, respectively. The tomographic survey was performed in Boring RC-14-004 at Abutment 1. The seismic refraction survey was performed at fifteen locations within the project limits.

The seismic field survey (refraction/tomography) was planned and designed primarily for the purpose of characterizing the site for design and construction of a construction access road and was not specifically performed for foundation design.

A summary of the test borings is presented in **Table 1**.

Table 1. Test Boring Summary

Boring No.	Completion Date	Drill Rig	Hammer Type	Hammer Efficiency (%)	Ground Surface Elev. (ft)	Boring Depth (ft)
RC-14-001	7/14/2014	Acker 3711	Safety	58	1454.7	17.5
RC-14-002	7/17/2014	Acker 3711	Safety	58	1454.5	120.0
RC-14-003	7/15/2014	Acker 1974	Safety	67	1402.1	45.0
RC-14-004	7/28/2014	Acker 1974	Safety	67	1403.6	150.0
RC-14-005	7/24/2014	Model 45	N/A	N/A	1321.0	70.0
RC-14-006	7/29/2014	Model 45	N/A	N/A	1366.0	69.7
RC-14-007	8/2/2014	Model 45	N/A	N/A	1355.6	69.7
RC-14-008	8/6/2014	Acker 3711	Safety	58	1402.4	67.5
RC-15-001	1/23/2015	Viper	N/A	N/A	1390.0	70.0
RC-15-002	1/26/2015	Viper	N/A	N/A	1372.6	80.0
RC-15-003	1/28/2015	Viper	N/A	N/A	1374.4	70.0
RC-15-004	1/30/2015	Viper	N/A	N/A	1335.8	70.0
RC-15-005	2/4/2015	Viper	N/A	N/A	1359.2	70.0
RC-15-006	2/8/2015	Viper	N/A	N/A	1384.7	70.0

Laboratory Testing Program

Laboratory testing was performed on selected soil and rock samples obtained from the 2014 and 2015 subsurface field investigation. Samples were selected for corrosion evaluation and rock core samples were selected for rock strength testing.

Since the bedrock at the project site has undergone severe tectonic stress in the past and is intensely folded, fractured, sheared, and weathered, it was difficult to select rock core samples without discontinuities for laboratory testing. Therefore, only those samples that remained intact during the sample selection process and preparation were tested. Even the samples that remained intact during the sample selection process and sample preparation contain numerous preexisting healed visible and non-visible discontinuities.

A total of 17 samples were tested for unconfined compressive strength (UCS). The tested UCS values range from 3489 psi to 15,367 psi. A total of 21 samples were tested using the point load (PL) test method. These results indicated the Rock Strength

Index $I_{s(50)}$ ranged from 12 psi to 1751 psi. The majority of the tested samples failed along preexisting, healed discontinuities.

Please note that these tested rock sample UCS and PL $I_{s(50)}$ values only represent the samples tested. These test results should not be used without incorporating knowledge of the local geology, observation of the site geology, and review of the subsurface drilling results including core samples. Directly using these test results without appropriate interpretation and study may result in misleading conclusions.

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Geology

The proposed bridge site lies within the Klamath Mountains geomorphic province of California. Within the project limits, the topography consists of rolling terrain with occasional areas of steep slopes due to natural drainage features.

The California Geological Survey, Geologic Map of California, Olaf P. Jenkins Edition, Redding Sheet, compilation by Rudolph G. Strand 1962, third printing 1977, scale 1:250,000, indicates the site is underlain by Undivided Carboniferous marine rocks (C). These undivided marine sediments are described as the Baird Formation; fossiliferous mudstone, undifferentiated tuff and small limestone lenses (deposits probably confined to Mississippian time). Also in proximity lies the Mississippian Bragdon Formation. It is described on the Redding Sheet as dark greenish-gray to black thinly bedded meta-shale, interstratified metamorphosed siltstone, sandstone, grit and conglomerate in the upper part and metamorphosed local thin-bedded chert, rhyolitic tuff and mafic volcanic rocks in the lower part.

Subsurface descriptions from the 2014 and 2015 field investigations describe the material as moderately fractured to very intensely fractured metasedimentary rock that include metasandstone, metasiltstone, metachert and metashale.

The proposed bridge will be constructed along a very steep slope with a predominant slope ratio of 1:1 or steeper. The slope is covered with trees, bushes, loose soils, and rock debris. Bedrock is exposed in the slope face in some areas. Gullies with various widths and depths are present in the slope, which were created by runoff from the existing highway drainage.

Subsurface Conditions

The foundation material encountered during the 2014 and 2015 subsurface investigations for the proposed Shasta Viaduct consist of fill material, colluvium and bedrock. The descriptions used in the following sections are based on visual field observations and Standard Penetration Test (SPT) sampling.

The fill material encountered at Abutment 1 and Abutment 9 generally consists of well-graded gravel with sand and well-graded sand with gravel. The apparent density is very dense based on SPT blow counts. This fill material was placed during construction of the original Sidehill Viaduct (Br. No. 06-0042L) in 1941 and widening in 1968.

At Abutment 1, the material below the fill consists of colluvium (well-graded gravel with sand and silt). The apparent density of the colluvium is very dense based on SPT blow counts.

Bedrock was encountered in all borings. The bedrock encountered was visually identified as metamorphic rock and consists of metasandstone, metasiltstone, metachert and metashale. The bedrock at the project site has undergone severe tectonic stress in the past and is intensely folded, fractured, and sheared. The quality of the bedrock encountered at the site varies greatly. In general, the fracture density of the bedrock ranges from moderately to very intensely fractured. The variability of the bedrock weathering ranges from fresh to decomposed. The hardness of the bedrock ranges from soft to very hard.

For detailed subsurface data and boring locations, please refer to the Log-of-Test-Borings.

Groundwater

Groundwater was not encountered in the 2014 and 2015 field investigation. Water seepage was not observed on the slope above or below the existing highway. However, seepage water may be encountered in various amounts and locations during construction due to the fractured nature of the bedrock.

SCOUR EVALUATION

The proposed Shasta Viaduct does not cross a waterway. Therefore, scour will not impact the foundation design.

CORROSIVITY

Four (4) corrosion samples were collected during the August 2014 field investigation and analyzed for corrosivity, as shown in **Table 2**. The test results show that the pH is less than 5.5 for two of the samples, which indicates corrosivity to foundation elements. The corrosive samples were taken from Boring RC-14-008 located at the south end of the proposed Abutment 1 Right Wingwall. The sample elevations for these two samples are below the specified pile tip elevations for Abutment 1 and Abutment 1 Right Wingwall. From Bent 2 to Abutment 9, all foundations are founded in bedrock which is unable to be tested for corrosion by the Corrosion Laboratory (solid rock). However, due to the presence of bedrock, the area from Bent 2 to Abutment 9 is believed to be non-corrosive.

The corrosive samples are only located near Abutment 1 Right Wingwall, below the specified pile tip elevations for Abutment 1 and Abutment 1 Right Wingwall and the area from Bent 2 to Abutment 9 is considered to be non-corrosive. Therefore, it is the opinion of this Office that there should be no impact of corrosive soils on the foundation elements.

Due to the location of this project, it is expected that deicing salts will be used on the roadway and bridge deck. As such, appropriate corrosion protection measures should be considered. **Table 2** below presents the summary of these test results.

Table 2. Corrosion Test Summary Report-Soil

Sample Location	Sample Depth (ft)	Depth/ Elev. (ft)	pH	Minimum Resistivity (ohm-cm)	Sulfate Content (ppm)	Chloride Content (ppm)	Lab Report Date
Abut 1	RC-14-008	15/1387	6.42	9087			12/23/2014
Abut 1	RC-14-008	59/1343	6.42	10098			03/12/2015
Abut 1	RC-14-008	67/1335	4.14	1617	819	0	12/23/2014
Abut 1	RC-14-008	67.5/1334.5	4.3	1721	836	2	03/12/2015

Note:
The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content are not performed.

SEISMIC RECOMMENDATIONS

Based on the Seismic Refraction Survey and Borehole P-S Suspension Logging performed at the bridge site, a V_{S30} (the weighted average shear wave velocity for the top 100 feet of foundation material) of 4500 ft/s is considered applicable for the proposed bridge site.

The deterministic spectrum from the Caltrans ARS Online Tool (version 2.3.06) is based on the nearest active fault which controls ground motion. For the proposed bridge site, this fault is the Keswick fault (ID No. 35) with MMax of 6.0. The fault is located southwest of the proposed bridge site and has a dip angle of 65 degrees to the southeast. The closest distance from the bridge site to the fault rupture plane is approximately 3 miles. The Caltrans ARS Online Tool refers to this fault as reverse.

Based on the “Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012”, the governing design Acceleration Response Spectrum (ARS) curve is obtained by any or a combination of the following three methods for the proposed Shasta Viaduct:

1. Statewide minimum deterministic spectrum with MMax of 6.5, vertical strike-slip event with a rupture distance of 7.5 miles.
2. Deterministic Seismic Hazard spectrum from the Caltrans ARS Online Tool (version 2.3.06).
3. The USGS Interactive Deaggregation procedure with a 5% Probability of Exceedance in 50 years (975 years return period).

For the proposed Shasta Viaduct, the recommended ARS curve is an envelope of methods 2 and 3 stated above. The peak ground acceleration is estimated to be 0.34g. The recommended ARS curve is presented on **Plate No. 1**.

Soil liquefaction occurs when loose, water-saturated soils lose shear strength in response to sudden shaking from an earthquake, reducing their ability to support embankments and structures. Due to the presence of shallow rock at the bridge site, the potential for liquefaction is considered non-existent.

The potential for surface fault rupture at the site is absent as there are no known faults Holocene or younger in age that fall within 1000 feet of the structure and the structure does not fall within an Alquist-Priolo fault zone.

AS-BUILT FOUNDATION DATA

The existing Sidehill Viaduct (Br. No. 06-0042L) and retaining walls are supported by spread footings, which are founded on competent bedrock. As the footings are founded on a steep slope (approximate slope ratio of 1:1), the bottom of footing elevations vary considerably and some footings are stepped.

Bearing capacities are not provided on the “As-Built” plans from the 1941 construction. A “footing pressure” of 4 tsf (tons per square foot) is shown on the “As-Built” General Plan (GP) dated November 18, 1968 and was recommended in the “Foundation Recommendations” from 1965.

FOUNDATION RECOMMENDATIONS

According to the GP dated October 31, 2014, Abutment 1, Arch Footing 2, Arch Footing 6, Bent 7, Bent 8 and Abutment 9 are supported on foundations and Spandrel Bent 3, Isolation Bent 4, and Spandrel Bent 5 are supported on arches. Arch Footing 2 and Arch Footing 6 are founded on footings supported by 6-foot diameter CIDH piles and Bent 7 and 8 are founded on 8-foot diameter CIDH piles. Abutment 1 and Abutment 1 Right Wingwall are founded on 24-inch diameter CIDH piles and Abutment 9 is founded on a spread footing.

The design method for this project is Load and Resistance Factor Design (LRFD).

Pile foundation recommendations for Abutment 1 and the Bents are presented in **Table 3**.

Table 3. Pile Foundation Recommendations

Supp Loc	Pile Type	Cut-off Elev (ft)	Service-I Limit State Load Per Support (kips)		Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elev (ft)	Specified Tip Elev (ft)
						Strength/ Construction		Extreme Event			
			Total	Perma -nent		Comp $\varphi=0.5$	Tension $\varphi=0.7$	Comp $\varphi=1.0$	Tension $\varphi=1.0$		
Abut 1 Left	24” ϕ CIDH	1388.25	3591	3260	1	280[5]	0	0	0	1355 (a-I)	1355
Abut 1 Middle	24” ϕ CIDH	1383.25	3591	3260	1	280[5]	0	0	0	1350 (a-I)	1350
Abut 1 Right	24” ϕ CIDH	1378.25	3591	3260	1	280[5]	0	0	0	1345 (a-I)	1345

Table 3. Pile Foundation Recommendations (continued)

Supp Loc	Pile Type	Cut-off Elev (ft)	Service-I Limit State Load Per Support (kips)		Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elev (ft)	Specified Tip Elev (ft)
						Strength/ Construction		Extreme Event			
			Total	Perma -nent		Comp φ=0.5	Tension φ=0.7	Comp φ=1.0	Tension φ=1.0		
Arch Footing 2 Left, Pile A	72” ϕ CIDH	1355.96	1110	1025	1	1434	0	460	0	1329 (a-I) 1344 (a-II)	1329
Arch Footing 2 Left, Pile B	72” ϕ CIDH	1355.96	1110	1025	1	1434	0	460	0	1329 (a-I) 1344 (a-II)	1329
Arch Footing 2 Left, Pile C	72” ϕ CIDH	1355.96	1110	1025	1	1434	0	460	0	1327 (a-I) 1342 (a-II)	1327
Arch Footing 2 Left, Pile D	72” ϕ CIDH	1355.96	1110	1025	1	1434	0	460	0	1323 (a-I) 1340 (a-II)	1323
Arch Footing 2 Right, Pile A	72” ϕ CIDH	1347.02	1156	1059	1	1577	0	400	0	1318 (a-I) 1344 (a-II)	1318
Arch Footing 2 Right, Pile B	72” ϕ CIDH	1347.02	1156	1059	1	1577	0	400	0	1315 (a-I) 1332 (a-II)	1315
Arch Footing 2 Right, Pile C	72” ϕ CIDH	1347.02	1156	1059	1	1577	0	400	0	1312 (a-I) 1329 (a-II)	1312
Arch Footing 2 Right, Pile D	72” ϕ CIDH	1347.02	1156	1059	1	1577	0	400	0	1305 (a-I) 1322 (a-II)	1305
Arch Footing 6 Left, Pile A	72” ϕ CIDH	1327.62	1317	1228	1	1574	0	434	0	1300 (a-I) 1316 (a-II)	1300
Arch Footing 6 Left,Pile B	72” ϕ CIDH	1327.62	1317	1228	1	1574	0	434	0	1300 (a-I) 1316 (a-II)	1300
Arch Footing 6 Left, Pile C	72” ϕ CIDH	1327.62	1317	1228	1	1574	0	434	0	1290 (a-I) 1310 (a-II)	1290

Table 3. Pile Foundation Recommendations (continued)

Supp Loc	Pile Type	Cut-off Elev (ft)	Service-I Limit State Load Per Support (kips)		Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elev (ft)	Specified Tip Elev (ft)
						Strength/ Construction		Extreme Event			
			Total	Perma -nent		Comp φ=0.5	Tension φ=0.7	Comp φ=1.0	Tension φ=1.0		
Arch Footing 6 Left, Pile D	72” ϕ CIDH	1327.62	1317	1228	1	1574	0	434	0	1295 (a-I) 1312 (a-II)	1295
Arch Footing 6 Right, Pile A	72” ϕ CIDH	1311.61	1436	1436	1	1925	0	433	0	1283 (a-I) 1300 (a-II)	1283
Arch Footing 6 Right, Pile B	72” ϕ CIDH	1311.61	1436	1436	1	1925	0	433	0	1283 (a-I) 1300 (a-II)	1283
Arch Footing 6 Right, Pile C	72” ϕ CIDH	1311.61	1436	1436	1	1925	0	433	0	1276 (a-I) 1295 (a-II)	1276
Arch Footing 6 Right, Pile D	72” ϕ CIDH	1311.61	1436	1436	1	1925	0	433	0	1283 (a-I) 1300 (a-II)	1283
Bent 7 Left	96” ϕ CIDH	1373.00	2400	2030	1	3315	0	337	0	1307 (a-I) 1363 (a-II)	1307
Bent 7 Right	96” ϕ CIDH	1356.00	2375	2000	1	3315	0	318	0	1289 (a-I) 1346 (a-II)	1289
Bent 8 Left	96” ϕ CIDH	1391.00	2363	1996	1	3420	0	300	0	1322 (a-I) 1381 (a-II)	1322
Bent 8 Right	96” ϕ CIDH	1374.00	2340	1980	1	3420	0	295	0	1305 (a-I) 1364 (a-II)	1305

Notes:
1. Design tip elevations are controlled by (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event).
2. Design tip elevation controlled by Settlement is not applicable.
3. Design tip elevation for Lateral Load is provided by SD.
4. Specified tip elevation shall not be raised if controlled by Lateral Load.
5. A resistance factor of 0.7 was used for Abutment 1 piles.

The pile data table is presented in **Table 4**.

Table 4. Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (ft)	Specified Tip Elevation (ft)
		Compression	Tension		
Abut 1 Left	24” ϕ CIDH	400	0	1355 (a)	1355
Abut 1 Middle	24” ϕ CIDH	400	0	1350 (a)	1350
Abut 1 Right	24” ϕ CIDH	400	0	1345 (a)	1345
Arch Footing 2 Left, Pile A	72” ϕ CIDH	2870	0	1329 (a)	1329
Arch Footing 2 Left, Pile B	72” ϕ CIDH	2870	0	1329 (a)	1329
Arch Footing 2 Left, Pile C	72” ϕ CIDH	2870	0	1327 (a)	1327
Arch Footing 2 Left, Pile D	72” ϕ CIDH	2870	0	1323 (a)	1323
Arch Footing 2 Right, Pile A	72” ϕ CIDH	3160	0	1318 (a)	1318
Arch Footing 2 Right, Pile B	72” ϕ CIDH	3160	0	1315 (a)	1315
Arch Footing 2 Right, Pile C	72” ϕ CIDH	3160	0	1312 (a)	1312
Arch Footing 2 Right, Pile D	72” ϕ CIDH	3160	0	1305 (a)	1305
Arch Footing 6 Left, Pile A	72” ϕ CIDH	3150	0	1300 (a)	1300
Arch Footing 6 Left, Pile B	72” ϕ CIDH	3150	0	1300 (a)	1300
Arch Footing 6 Left, Pile C	72” ϕ CIDH	3150	0	1290 (a)	1290
Arch Footing 6 Left, Pile D	72” ϕ CIDH	3150	0	1295 (a)	1295
Arch Footing 6 Right, Pile A	72” ϕ CIDH	3850	0	1283 (a)	1283
Arch Footing 6 Right, Pile B	72” ϕ CIDH	3850	0	1283 (a)	1283
Arch Footing 6 Right, Pile C	72” ϕ CIDH	3850	0	1276 (a)	1276
Arch Footing 6 Right, Pile D	72” ϕ CIDH	3850	0	1283 (a)	1283
Bent 7 Left	96” ϕ CIDH	6630	0	1307 (a)	1307
Bent 7 Right	96” ϕ CIDH	6630	0	1289 (a)	1289
Bent 8 Left	96” ϕ CIDH	6840	0	1322 (a)	1322
Bent 8 Right	96” ϕ CIDH	6840	0	1305 (a)	1305

Notes:
1. Design tip elevations for abutments and bents are controlled by (a) Compression.

The nominal bearing resistance of the soil underlying the spread footing was calculated using the formulation provided by AASHTO Bridge Design Specifications, 2012. Spread footing recommendations for Abutment 9 are presented in **Table 5**.

Table 5. Spread Footing Recommendations for Abutment 9

Support Location	Footing Size (ft)		Bottom of Footing Elev. (ft)	Minimum Footing Embedment Depth (ft)	Total Permissible Support Settlement (inches)	Service Limit State	Strength $\varphi = 0.45$	Extreme Event $\varphi = 1.00$
	L	B				Permissible Net Contact Stress[2] (ksf)	Factored Gross Nominal Bearing Resistance[3] (ksf)	Factored Gross Nominal Bearing Resistance[3] (ksf)
Abut 9	44	11	1443.0	5	1	16	30	67

Notes:
1. Recommendations are based on the foundation geometry and the load data provided by Structure Design.
2. See MTD 4-1 for definitions and applications of the recommended design parameters.

Abutment 1 Right Wingwall

Soil properties and earth pressure coefficients for Abutment 1 Right Wingwall are presented in **Table 6**. The active and passive earth pressures were determined using Section 5 of the Caltrans Bridge Design Specifications, August 2004.

Table 6. Soil Properties and Earth Pressure Coefficients

Soil Type	Effective unit weight (pcf)	Internal friction angle (deg)	Undrained shear strength, c (psf)	Active earth pressure, Ka	Passive earth pressure, Kp
Backfill	120	34	0	0.25	N/A
Native Inside	130	38	300	0.21	6.1
Native Outside	130	38	300	N/A	0.35

Notes:
1. Soil type “native inside” denotes soil on upslope side of wall.
2. Soil type “native outside” denotes soil on downslope side of wall.
3. The friction angle between backfill material and back of retaining wall is assumed to be 11.3°.
4. The friction angle between native inside soil and back of retaining wall is assumed to be 12.7°.

Notes to Designer

1. The structural design engineer shall indicate on the Plans, in the pile data table, the minimum pile tip elevation required to meet the lateral load demand.

2. Should the specified pile tip elevation required to meet lateral load demand exceed the specified pile tip elevation provided in this report, the Office of Geotechnical Design North should be contacted for further recommendations.

3. All support locations are to be plotted in Plan View on the Log of Test Borings as stated in the "Memo to Designers" 4-2. The plotting of the support locations should be made prior to requesting a final foundation review.

Construction Considerations

1. The proposed bridge will cross over a steep slope. Potential rock fall may occur during construction and should be prepared for accordingly. Precautionary planning is recommended to protect workers' safety and maintain the construction schedule. Also, precautions should be taken due to the railroad and railroad tunnel below the site.

2. There is potential for seepage or spring water flow into the drilled shaft foundations through fractures and shear zones at various depths and locations. The flow rate and the amount of water will depend on seasonal precipitation and other factors.

3. The proposed bridge site is located on a very steep slope. If the Contractor proposes any earthwork, such as cuts or excavations, a temporary access road on the slope, or any kind of earthwork involving excavation or removal of earth materials, the Contractor shall submit a work plan prior to starting the work. The work plan shall be submitted with sufficient time to provide the Engineer time to review, evaluate, and approve. The work plan should include, but not be limited to; the area and depth of the cut and/or excavation, methods and sequences of the cut and/or excavation, and methods and sequence of stabilizing the exposed surface caused by the cut and/or excavation.

4. Due to the presence of intensely weathered and intensely fractured rock, the Contractor should anticipate cut slope stability issues during the construction of Arch Footing 2 and Arch Footing 6 and be prepared to design shoring systems.

5. If water is encountered in the CIDH piles and can be controlled by dewatering methods during the CIDH pile construction, the concrete should be placed under "dry" conditions. Otherwise, the "wet" method should be used. Caution must be taken during CIDH pile construction to avoid pile anomalies.

6. The bottom of all the CIDH piles must be clean and free from any rock debris and cuttings prior to placement of concrete, with the exception of Abutment 1 and Abutment 1 Right Wingwall piles. Verification of a clean drilled shaft bottom is required for all piles at Arch Footing 2, Arch Footing 6, Bent 7, and Bent 8. If the "wet" method is used, the Contractor must submit a cleaning and inspection plan that fully describes the cleaning and inspection methods and procedures to be used. Additionally, the bottom of the CIDH piles must be inspected by Caltrans Foundation Testing Branch's Shaft Inspection Device (SID) and/or other device before the CIDH piles may be considered acceptable.

7. Drilling of the CIDH piles, placement of the rebar cage, and concrete pour must be completed in one continuous operation.

8. It is anticipated that drilling of the CIDH piles will be difficult when drilling through the very hard and moderately to very intensely fractured bedrock. Zones of intensely fractured bedrock with open fractures may cause the drilled shafts to cave or collapse during drilling.

9. Loss of drilling mud circulation was experienced during the 2014 foundation investigation. If drilling fluid is used for CIDH pile construction, drilling fluid loss should be anticipated.

10. Temporary casing may be used to stabilize the hole if necessary. The temporary casing must be removed from the bedrock during concrete placement. Precautions should be made to avoid pile anomalies. If the Contractor proposes to oscillate or rotate the temporary casing into the bedrock section, this Office must be notified and further recommendations will be provided.

11. It is anticipated that the temporary casing installation at Bent 7 and Bent 8 will be difficult due to the intensely weathered and intensely fractured rock. The Contractor should take all necessary precautions to avoid the temporary casing from sinking and/or tilting during drilling of the shaft below. If the Contractor proposes to leave the temporary casing in the upper portion of drilled hole, this Office must be notified and a minimum casing tip elevation and further recommendations will be provided.

12. If temporary casing is used, care should be taken during construction of the CIDH piles to not disturb the material surrounding the bottom of the temporary casing. Equipment and methods used for constructing the CIDH piles shall not cause scouring or caving around or below the tip of the temporary casing.

13. The bedrock at the site is predominantly hard to very hard. Drilling and excavation may be very difficult and may require specialized equipment and/or specialized methods.

14. It is highly recommended that the Contractor inspect/observe the core samples at the Translab facility before bidding. This inspection/observation will give the prospective bidders an understanding of the potentially unstable subsurface material, and the soft to very hard, fresh to decomposed, and moderately to very intensely fractured bedrock.

Project Information

"Project Information," discloses to bidders and contractors a list of pertinent information available for their inspection prior to bid opening. The following is information originating from Geotechnical Services.

Data and information attached with the project plans are:

A. Log of Test Borings, Shasta Viaduct (Replace), Br. No. 06-0212L.

Data and information included in the Information Handout provided to the bidders and contractors are:

A. Foundation Report, Shasta Viaduct (Replace), dated March12, 2015.
B. Suspension P-S Logging.
C. Seismic Refraction Survey.
D. Tomographic Survey.
E. Laboratory Unconfined Compressive Strength and Point Load Test Data.

Data and information available for inspection at the District Office:

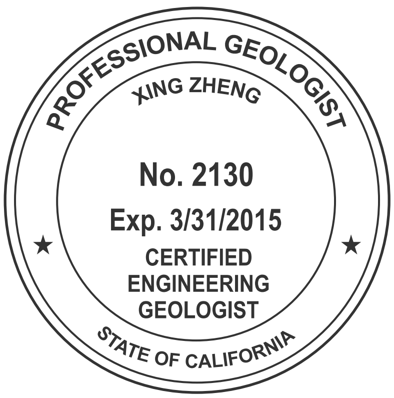
A. None.

Data and information available for inspection at the Transportation Laboratory:

A. Core samples.

If you have any questions regarding this report, please contact Xing Zheng at 227-1036, Ben Barnes at 227-1039, or Reid Buell at 916-227-1012.

Xing Zheng, C.E.G. No. 2130
Engineering Geologist
Geotechnical Design – North



Benjamin M. Barnes, P.E. No. 66090
Transportation Engineer
Geotechnical Design – North

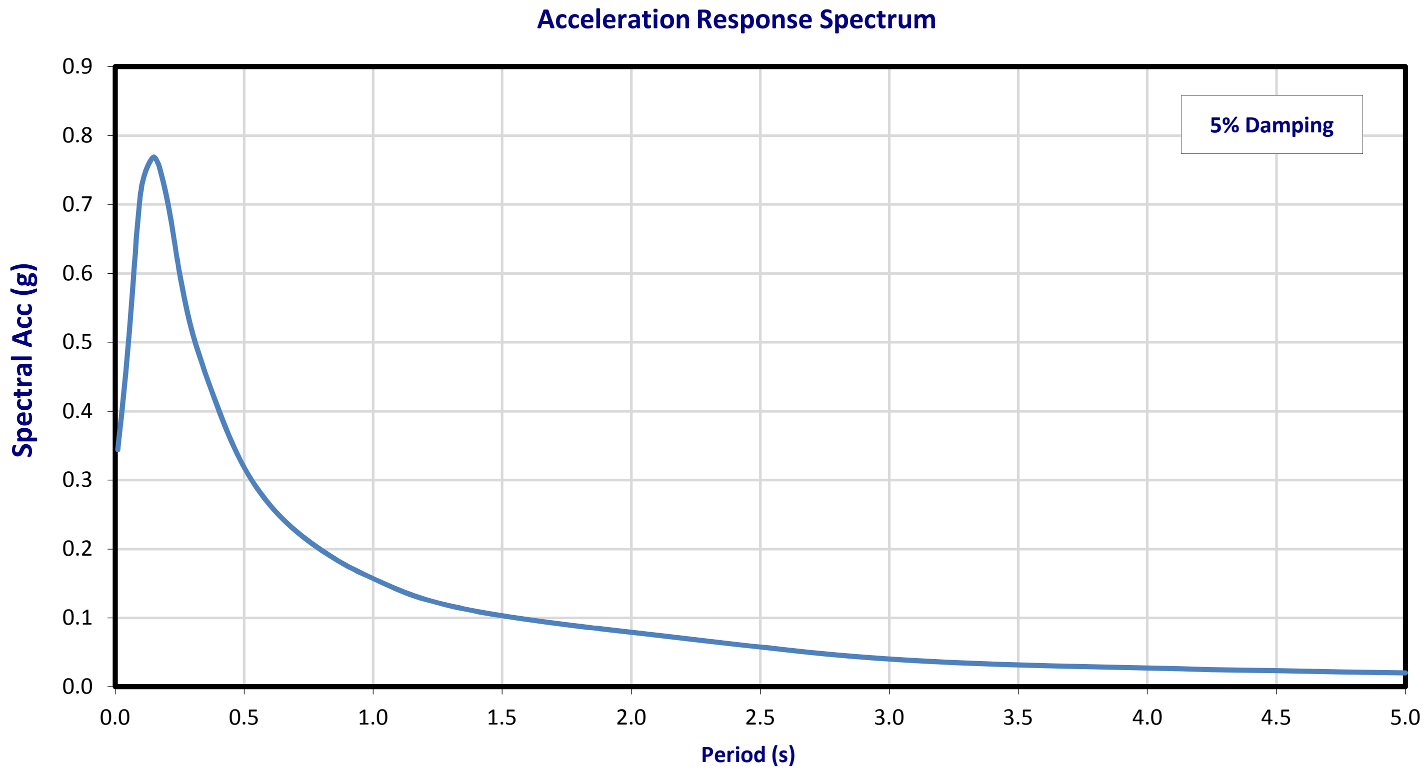
Attachment
Plate No. 1: ARS Curve

Report Copy List
Reid Buell, Branch Chief, OGDN-A
Phil Baker, Project Manager
Structure Construction R.E. Pending File (RE_Pending_File@dot.ca.gov)
Mohsen Sultan (DES Office Engineer, Office of PS&E)
Byron Berger, District Material Engineer
Geotechnical Archive (http://svgcgeodog.dot.ca.gov/)

Shasta Viaduct (Replace)

Bridge No. 06-0212L **Latitude** 40.7771 **Longitude** -122.3187 **Control** Envelope

Period (s)	Sa(g)
0.01	0.34
0.05	0.49
0.10	0.72
0.15	0.77
0.20	0.71
0.25	0.60
0.30	0.51
0.40	0.40
0.50	0.32
0.60	0.26
0.70	0.23
0.85	0.19
1.00	0.16
1.20	0.13
1.50	0.10
2.00	0.08
3.00	0.04
4.00	0.03
5.00	0.02



Deterministic Procedure Data

Fault	Keswick fault	
Fault ID	35	
Mmax	6	
Style	Rev	
Dip (deg)	65	
Dip Dir	SE	
Z_{TOR}	3.1	miles
Z_{BOT}	15.5	miles

R_{rup}	3.2	miles
R_{jb}	0.8	miles
R_x	0.006	miles
V_{S30}	4500	ft/s
$Z_{1.0}$	N/A	ft
$Z_{2.5}$	N/A	miles

Note: The ARS curve above is an envelope of the deterministic procedure from the ARS Online Tool and the USGS 5% Probability of Exceedance in 50 years (975 years return period).

Division of Engineering Services Geotechnical Services Office of Geotechnical Design - North	ID 0200000016	ARS Curve	Plate No. 1
	EA 02-0E0901	02-SHA-5 PM R29.72	

MATERIALS INFORMATION

Addendum to Revised Foundation Report for Shasta Viaduct (Replace)

Dated June 22, 2015

M e m o r a n d u m

Serious Drought.
Help Save Water!

To: MR. JOSEPH E. DOWNING
Chief, Bridge Design Branch 3
Office of Bridge Design West
Division of Structure Design

Attention: Mr. Jose M. Aquino III, P.E.

Date: June 22, 2015

File: 02-SHA-5-PM 29.72
EA 02-0E0901
ID 0200000016
Shasta Viaduct
(Replace)
Br. No. 06-0212L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES – MS 5

Subject: Addendum to Revised Foundation Report for Shasta Viaduct (Replace)

The Office of Geotechnical Design North (OGD-N) has prepared this addendum to revised Foundation Report (FR) for the proposed Shasta Viaduct (Br. No. 06-0212L), which will replace the existing Sidehill Viaduct (Br. No. 06-0042L). The bridge is located on Interstate 5 approximately 15 miles north of Redding in Shasta County. This addendum provides clarification to the Revised Foundation Report dated March 12, 2015.

The following paragraphs in the Construction Consideration section of the revised Foundation Report dated March 12, 2015,

6. The bottom of all the CIDH piles must be clean and free from any rock debris and cuttings prior to placement of concrete, with the exception of Abutment 1 and Abutment 1 Right Wingwall piles. Verification of a clean drilled shaft bottom is required for all piles at Arch Footing 2, Arch Footing 6, Bent 7, and Bent 8. If the "wet" method is used, the Contractor must submit a cleaning and inspection plan that fully describes the cleaning and inspection methods and procedures to be used. Additionally, the bottom of the CIDH piles must be inspected by Caltrans Foundation Testing Branch's Shaft Inspection Device (SID) and/or other device before the CIDH piles may be considered acceptable.

11. It is anticipated that the temporary casing installation at Bent 7 and Bent 8 will be difficult due to the intensely weathered and intensely fractured rock. The Contractor should take all necessary precautions to avoid the temporary casing from sinking and/or tilting during drilling of the shaft below. If the Contractor proposes to leave the temporary casing in the upper portion of drilled hole, this Office must be notified and a minimum casing tip elevation and further recommendations will be provided.

12. If temporary casing is used, care should be taken during construction of the CIDH piles to not disturb the material surrounding the bottom of the temporary casing. Equipment and methods used for constructing the CIDH piles shall not cause scouring or caving around or below the tip of the temporary casing.

Are amended as follows,

6. The bottom of all the CIDH piles must be clean and free from any rock debris and cuttings prior to placement of concrete. Verification of a clean drilled shaft bottom is required for all piles at Arch Footing 2, Arch Footing 6, Bent 7, and Bent 8. If the "wet" method is used, the Contractor must submit a cleaning and inspection plan that fully describes the cleaning and inspection methods and procedures to be used. Additionally, the bottom of the CIDH piles must be inspected by Caltrans Foundation Testing Branch's Shaft Inspection Device (SID) and/or other device before the CIDH piles may be considered acceptable.

11. It is anticipated that casing installation will be difficult due to the intensely weathered and intensely fractured rock. The Contractor should take all necessary precautions to prevent any casing from sinking and/or tilting during drilling of the shaft below. If the Contractor proposes to leave any temporary casing in the upper portion of a drilled hole, this Office must be notified and a minimum casing tip elevation and further recommendations will be provided.

12. If casing is used, care should be taken during construction of the CIDH piles to not disturb the material surrounding the bottom of the casing. Equipment and methods used for constructing the CIDH piles shall not cause scouring or caving around or below the tip of the casing.

This addendum only makes changes to the items shown above. All other information and recommendations in the March 12, 2015 Revised Foundation Report remain applicable.

Project Information

"Project Information," discloses to bidders and contractors a list of pertinent information available for their inspection prior to bid opening. The following is information originating from Geotechnical Services.

Data and information attached with the project plans are:

A. Log of Test Borings, Shasta Viaduct (Replace), Br. No. 06-0212L.

Data and information included in the Information Handout provided to the bidders and contractors are:

A. Revised Foundation Report, Shasta Viaduct (Replace), dated March 12, 2015.
B. Suspension P-S Logging.
C. Seismic Refraction Survey.
D. Tomographic Survey.
E. Laboratory Unconfined Compressive Strength and Point Load Test Data.
F. Addendum to Revised Foundation Report, Shasta Viaduct (Replace), dated June 22, 2015.

Data and information available for inspection at the District Office:

A. None.

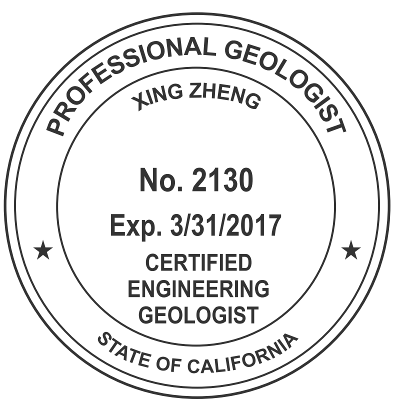
Data and information available for inspection at the Transportation Laboratory:

A. Core samples.

If you have any questions regarding this report, please contact Xing Zheng at 227-1036, Ben Barnes at 227-1039, or Reid Buell at 916-227-1012.

Xing Zheng, C.E.G. No. 2130
Engineering Geologist
Geotechnical Design – North

Benjamin M. Barnes, P.E. No. 66090
Transportation Engineer
Geotechnical Design – North



Report Copy List

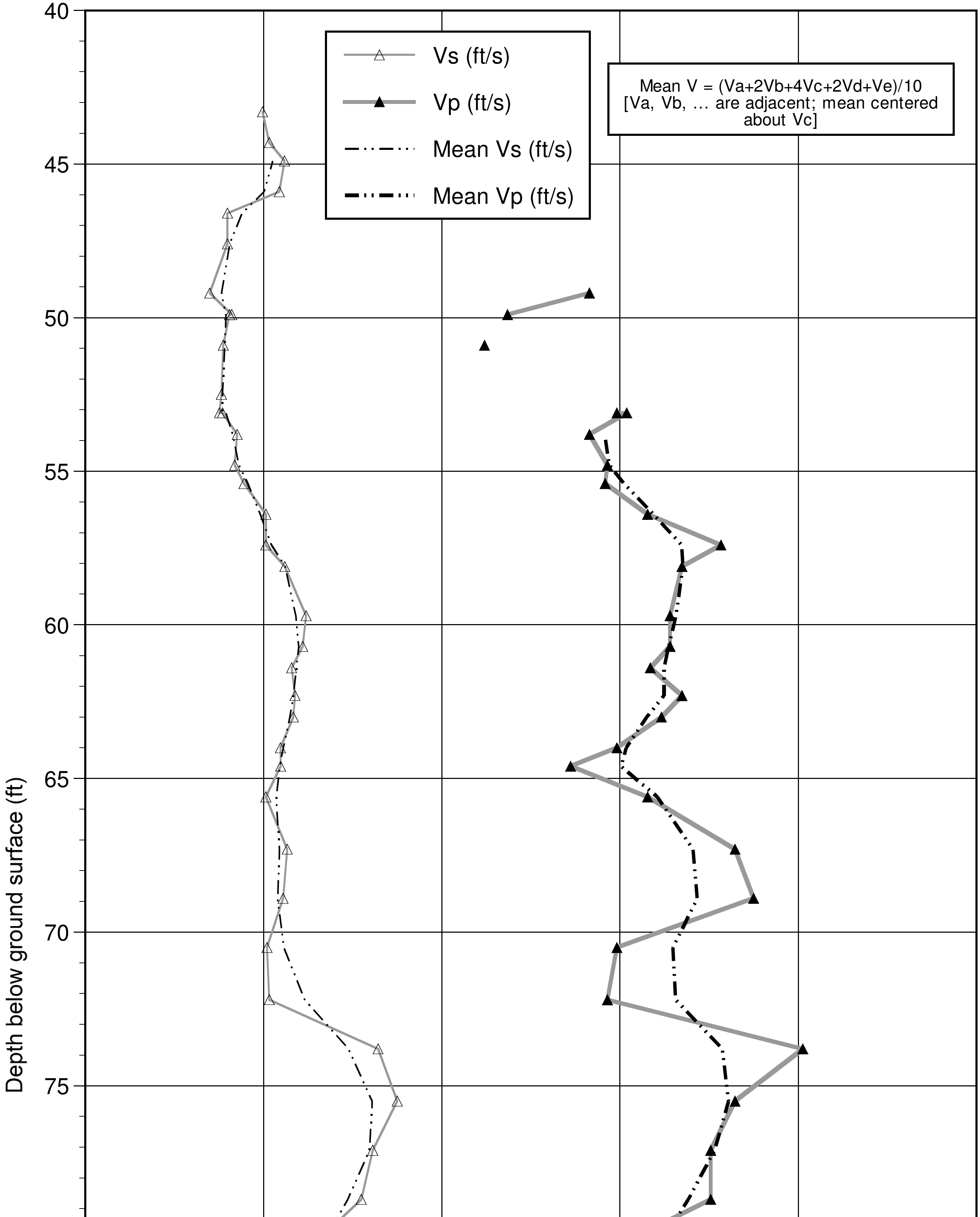
Reid Buell, Branch Chief, OGDN-A
Phil Baker, Project Manager
Byron Berger, District Material Engineer
Structure Construction R.E. Pending File
Mohsen Sultan (DES Office Engineer)
Geotechnical Archive

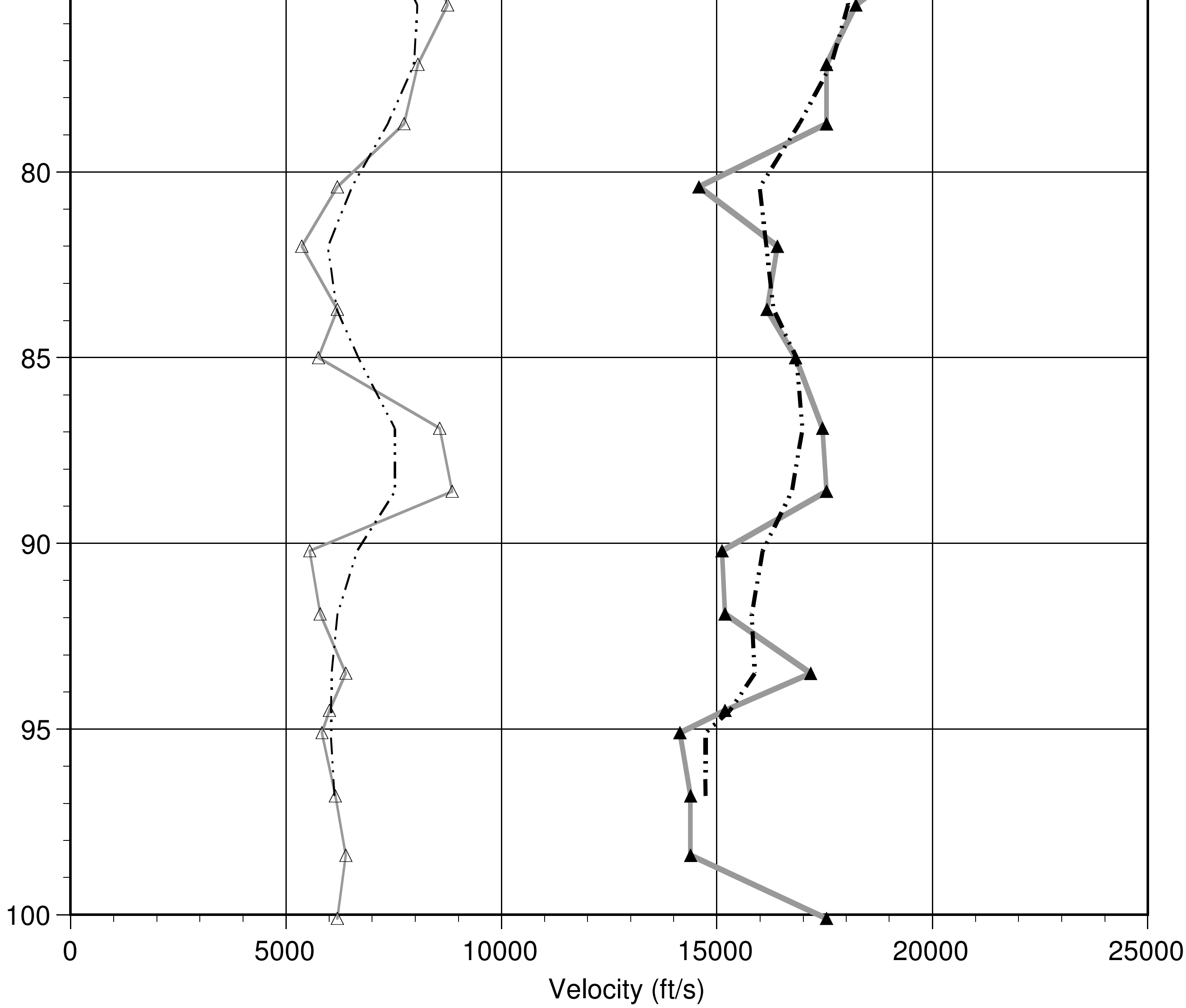
MATERIALS INFORMATION

Suspension P-S Logging

SHA-5 Sidehill Viaduct
Bridge No. 06-0212L, boring RC-14-002 (Abutment 9)
Downhole Interval Velocities

Top hole elevation: Not provided





80
85
90
95
100
0
5000
10000
15000
20000
25000
Velocity (ft/s)

Top Hole Elevation: Not provided

SHA-5 PM 29.72 Shasta Viaduct
Bridge No. 06-0212L, Boring RC-14-002
PS Suspension Log Data Summary

Top of Hole Elevation (m)	Depth (m)	Depth (ft)	Vs (m/s)	Vs (ft/s)	Vp (m/s)	Vp (ft/s)	γ	ρ(Vp) (g/cc)	ρ(Vp) (lb/ft^3)	G (GPa)	E (GPa)	K (GPa)	G (10^{3}lb/ft^2)	E (10^{3}lb/ft^2)	K (10^{3}lb/ft^2)
Not provided	13.2	43.3	1515	4971		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	13.5	44.3	1572	5159		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	13.7	44.9	1704	5589		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	14	45.9	1658	5441		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	14.2	46.6	1215	3986		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	14.5	47.6	1215	3986		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	15	49.2	1062	3485	4310	14142	0.47	2.75	171.67	3.102	9.106	46.953	64789	190180	980625
	15.2	49.9	1252	4106	3610	11844	0.43	2.65	165.56	4.154	11.895	29.025	86763	248438	606205
	15.2	49.9	1232	4043		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	15.5	50.9	1179	3869	3413	11197	0.43	2.62	163.41	3.640	10.427	25.637	76024	217766	535442
	16	52.5	1163	3815		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	16.2	53.1	1147	3762	4545	14913	0.47	2.78	173.30	3.651	10.704	52.489	76249	223564	1096240
	16.2	53.1	1173	3848	4630	15189	0.47	2.78	173.85	3.832	11.233	54.579	80029	234597	1139899
	16.4	53.8	1298	4258	4310	14142	0.45	2.75	171.67	4.632	13.434	44.913	96739	280571	938026
	16.7	54.8	1277	4190	4464	14647	0.46	2.77	172.76	4.514	13.139	49.134	94270	274407	1026169
	16.9	55.4	1356	4449	4444	14582	0.45	2.77	172.62	5.084	14.730	47.841	106177	307633	999175
	17.2	56.4	1543	5063	4808	15773	0.44	2.80	174.94	6.674	19.255	55.875	139384	402142	1166964
	17.5	57.4	1543	5063	5435	17831	0.46	2.86	178.24	6.800	19.802	75.267	142011	413580	1571970
	17.7	58.1	1705	5594	5102	16739	0.44	2.83	176.59	8.223	23.637	62.669	171750	493656	1308870
	18.2	59.7	1887	6190	5000	16404	0.42	2.82	176.04	10.039	28.450	57.113	209665	594182	1192822
	18.5	60.7	1859	6098	5000	16404	0.42	2.82	176.04	9.743	27.665	57.508	203476	577800	1201073
	18.7	61.4	1764	5786	4831	15849	0.42	2.80	175.08	8.724	24.829	53.820	182195	518568	1124055
	19	62.3	1792	5880	5102	16739	0.43	2.83	176.59	9.085	25.976	61.521	189742	542521	1284880
	19.2	63.0	1778	5833	4926	16162	0.43	2.81	175.63	8.892	25.343	56.414	185702	529299	1178233
	19.5	64.0	1668	5473	4545	14913	0.42	2.78	173.30	7.724	21.970	47.058	161321	458856	982811
	19.7	64.6	1669	5477	4149	13613	0.40	2.73	170.44	7.609	21.358	36.861	158921	446068	769854
	20	65.6	1543	5063	4808	15773	0.44	2.80	174.94	6.674	19.255	55.875	139384	402142	1166964
	20.5	67.3	1726	5662	5556	18227	0.45	2.86	178.79	8.528	24.675	77.024	178118	515334	1608671
	21	68.9	1691	5547	5714	18748	0.45	2.88	179.48	8.217	23.864	82.923	171624	498408	1731869
	21.5	70.5	1553	5094	4545	14913	0.43	2.78	173.30	6.694	19.196	48.432	139796	400919	1011510
	22	72.2	1572	5159	4464	14647	0.43	2.77	172.76	6.841	19.555	46.030	142884	408417	961351
	22.5	73.8	2503	8212	6135	20128	0.40	2.90	181.14	18.181	50.911	84.970	379708	1063289	1774633
	23	75.5	2667	8749	5556	18227	0.35	2.86	178.79	20.366	55.002	61.240	425356	1148726	1279021
	23.5	77.1	2457	8061	5348	17545	0.37	2.85	177.83	17.196	46.987	58.531	359150	981346	1222438
	24	78.7	2358	7738	5348	17545	0.38	2.85	177.83	15.845	43.709	60.333	330928	912870	1260068
	24.5	80.4	1887	6190	4444	14582	0.39	2.77	172.62	9.844	27.367	41.494	205590	571571	866624
	25	82.0	1635	5365	5000	16404	0.44	2.82	176.04	7.541	21.720	60.443	157502	453639	1262373

γ = Poisson's Ratio ρ(Vp) = Vp-derived density G = Shear Modulus E = Young's Modulus K = Bulk Modulus

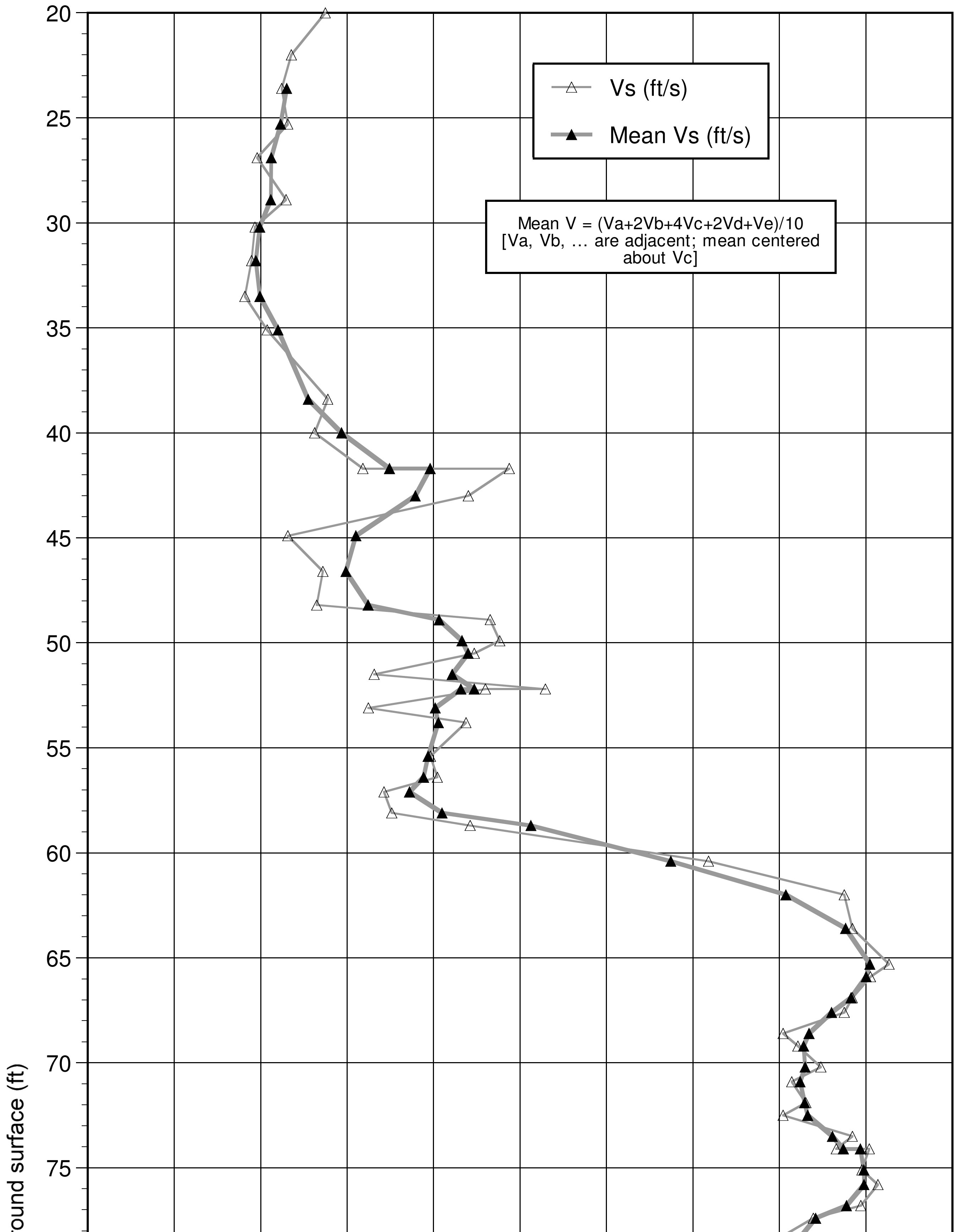
Shaded cells denote questionable data.

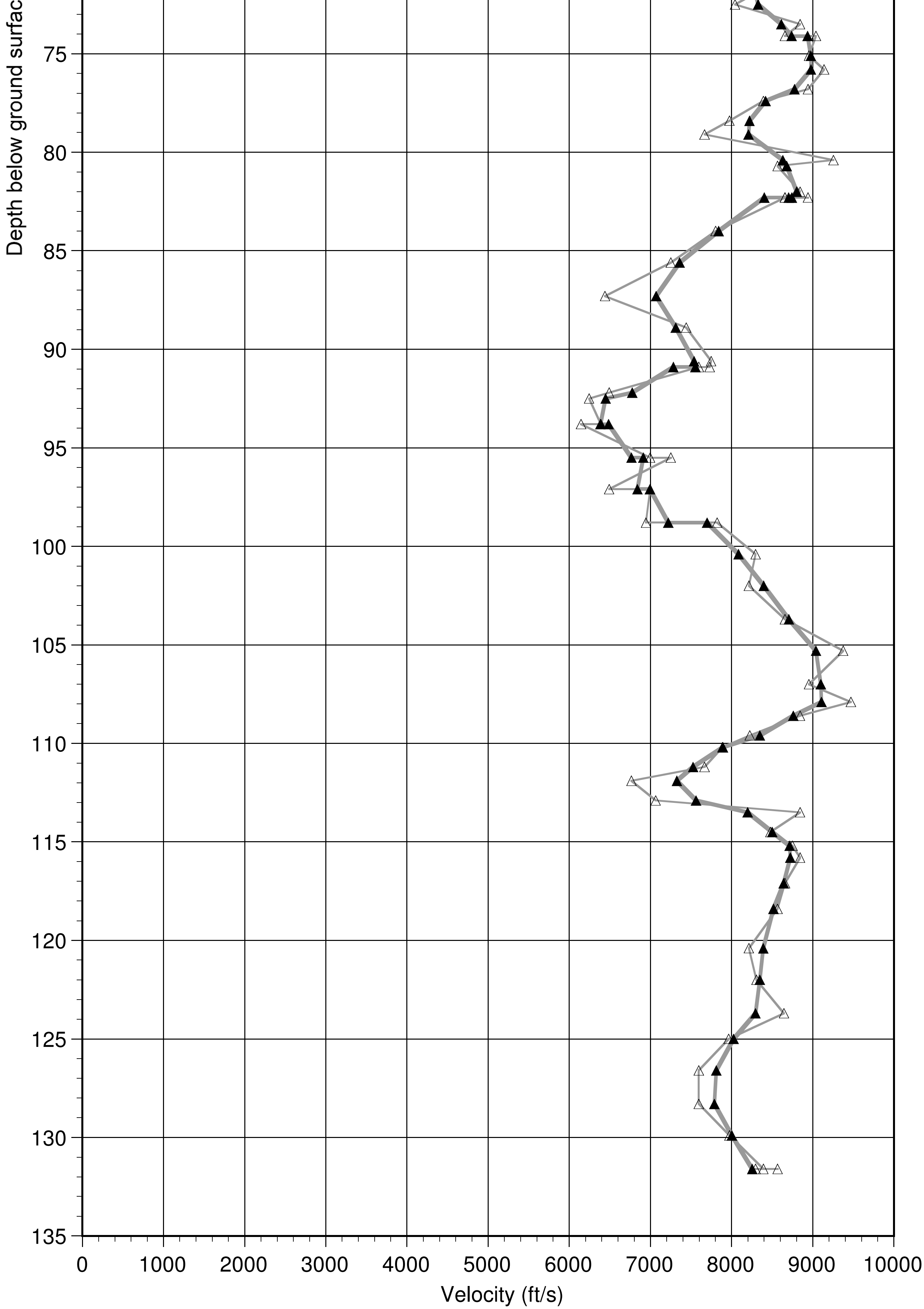
Top of Hole Elevation (m)	Depth (m)	Depth (ft)	Vs (m/s)	Vs (ft/s)	Vp (m/s)	Vp (ft/s)	γ	ρ(Vp) (g/cc)	ρ(Vp) (lb/ft^3)	G (GPa)	E (GPa)	K (GPa)	G (10^{3}lb/ft^2)	E (10^{3}lb/ft^2)	K (10^{3}lb/ft^2)
	25.5	83.7	1887	6190	4926	16162	0.41	2.81	175.63	10.015	28.324	54.916	209175	591562	1146936
	25.9	85.0	1753	5751	5128	16825	0.43	2.83	176.73	8.698	24.943	62.852	181659	520947	1312679
	26.5	86.9	2611	8566	5319	17451	0.34	2.85	177.69	19.404	52.053	54.661	405258	1087134	1141607
	27	88.6	2699	8855	5348	17545	0.33	2.85	177.83	20.751	55.161	53.791	433400	1152054	1123439
	27.5	90.2	1692	5551	4608	15119	0.42	2.78	173.71	7.967	22.659	48.470	166387	473233	1012320
	28	91.9	1765	5791	4630	15189	0.41	2.78	173.85	8.678	24.556	48.118	181233	512868	1004961
	28.5	93.5	1947	6389	5236	17177	0.42	2.84	177.28	10.770	30.579	63.482	224925	638660	1325844
	28.8	94.5	1832	6009	4630	15189	0.41	2.78	173.85	9.341	26.291	47.233	195097	549093	986475
	29	95.1	1778	5833	4310	14142	0.40	2.75	171.67	8.691	24.291	39.501	181510	507323	824998
	29.5	96.8	1873	6144	4386	14390	0.39	2.76	172.21	9.674	26.865	40.167	202041	561080	838900
	30	98.4	1947	6389	4386	14390	0.38	2.76	172.21	10.462	28.816	39.117	218495	601832	816961
	30.5	100.1	1887	6190	5348	17545	0.43	2.85	177.83	10.141	28.981	67.938	211794	605267	1418914

γ = Poisson's Ratio   ρ(Vp) = Vp-derived density   G = Shear Modulus   E = Young's Modulus   K = Bulk Modulus
Shaded cells denote questionable data.

SHA-5 Sidehill Viaduct
Bridge No. 06-0212L, boring RC-14-004 (Abutment 1)
Downhole Interval Velocities

Top hole elevation: Not provided




Depth below ground surfac
75
80
85
90
95
100
105
110
115
120
125
130
135
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Velocity (ft/s)

Top of Hole Elevation (m)	Depth (m)	Depth (ft)	Vs (m/s)	Vs (ft/s)	γ	ρ(Vp) (g/cc)	ρ(Vp) (lb/ft^3)	G (GPa)	E (GPa)	K (GPa)	G (10^3lb/ft^2)	E (10^3lb/ft^2)	K (10^3lb/ft^2)
Not provided	6.1	20.0	837	2747	NA	NA	NA	NA	NA	NA	NA	NA	NA
	6.7	22.0	717	2354	NA	NA	NA	NA	NA	NA	NA	NA	NA
	7.2	23.6	683	2242	NA	NA	NA	NA	NA	NA	NA	NA	NA
	7.7	25.3	705	2313	NA	NA	NA	NA	NA	NA	NA	NA	NA
	8.2	26.9	597	1958	NA	NA	NA	NA	NA	NA	NA	NA	NA
	8.8	28.9	699	2293	NA	NA	NA	NA	NA	NA	NA	NA	NA
	9.2	30.2	589	1934	NA	NA	NA	NA	NA	NA	NA	NA	NA
	9.7	31.8	577	1894	NA	NA	NA	NA	NA	NA	NA	NA	NA
	10.2	33.5	554	1817	NA	NA	NA	NA	NA	NA	NA	NA	NA
	10.7	35.1	632	2075	NA	NA	NA	NA	NA	NA	NA	NA	NA
	11.7	38.4	846	2776	NA	NA	NA	NA	NA	NA	NA	NA	NA
	12.2	40.0	799	2623	NA	NA	NA	NA	NA	NA	NA	NA	NA
	12.7	41.7	969	3181	NA	NA	NA	NA	NA	NA	NA	NA	NA
	12.7	41.7	1486	4875	NA	NA	NA	NA	NA	NA	NA	NA	NA
	13.1	43.0	1341	4401	NA	NA	NA	NA	NA	NA	NA	NA	NA
	13.7	44.9	705	2313	NA	NA	NA	NA	NA	NA	NA	NA	NA
	14.2	46.6	829	2719	NA	NA	NA	NA	NA	NA	NA	NA	NA
	14.7	48.2	807	2648	NA	NA	NA	NA	NA	NA	NA	NA	NA
	14.9	48.9	1418	4654	NA	NA	NA	NA	NA	NA	NA	NA	NA
	15.2	49.9	1452	4765	NA	NA	NA	NA	NA	NA	NA	NA	NA
	15.4	50.5	1362	4470	NA	NA	NA	NA	NA	NA	NA	NA	NA
	15.7	51.5	1010	3312	NA	NA	NA	NA	NA	NA	NA	NA	NA
	15.9	52.2	1613	5292	NA	NA	NA	NA	NA	NA	NA	NA	NA
	15.9	52.2	1402	4598	NA	NA	NA	NA	NA	NA	NA	NA	NA
	16.2	53.1	989	3245	NA	NA	NA	NA	NA	NA	NA	NA	NA
	16.4	53.8	1333	4374	NA	NA	NA	NA	NA	NA	NA	NA	NA
	16.9	55.4	1208	3962	NA	NA	NA	NA	NA	NA	NA	NA	NA
	17.2	56.4	1232	4043	NA	NA	NA	NA	NA	NA	NA	NA	NA
	17.4	57.1	1044	3425	NA	NA	NA	NA	NA	NA	NA	NA	NA
	17.7	58.1	1071	3515	NA	NA	NA	NA	NA	NA	NA	NA	NA
	17.9	58.7	1348	4422	NA	NA	NA	NA	NA	NA	NA	NA	NA

γ = Poisson's Ratio ρ(Vp) = Vp-derived density G = Shear Modulus E = Young's Modulus K = Bulk Modulus
Shaded cells denote questionable data.

Top of Hole Elevation (m)	Depth (m)	Depth (ft)	Vs (m/s)	Vs (ft/s)	γ	ρ(Vp) (g/cc)	ρ(Vp) (lb/ft^3)	G (GPa)	E (GPa)	K (GPa)	G (10^3lb/ft^2)	E (10^3lb/ft^2)	K (10^3lb/ft^2)
	18.4	60.4	2188	7179	NA	NA	NA	NA	NA	NA	NA	NA	NA
	18.9	62.0	2667	8749	NA	NA	NA	NA	NA	NA	NA	NA	NA
	19.4	63.6	2695	8843	NA	NA	NA	NA	NA	NA	NA	NA	NA
	19.9	65.3	2825	9268	NA	NA	NA	NA	NA	NA	NA	NA	NA
	20.1	65.9	2759	9051	NA	NA	NA	NA	NA	NA	NA	NA	NA
	20.4	66.9	2695	8843	NA	NA	NA	NA	NA	NA	NA	NA	NA
	20.6	67.6	2667	8749	NA	NA	NA	NA	NA	NA	NA	NA	NA
	20.9	68.6	2451	8041	NA	NA	NA	NA	NA	NA	NA	NA	NA
	21.1	69.2	2503	8212	NA	NA	NA	NA	NA	NA	NA	NA	NA
	21.4	70.2	2584	8478	NA	NA	NA	NA	NA	NA	NA	NA	NA
	21.6	70.9	2481	8141	NA	NA	NA	NA	NA	NA	NA	NA	NA
	21.9	71.9	2532	8306	NA	NA	NA	NA	NA	NA	NA	NA	NA
	22.1	72.5	2451	8041	NA	NA	NA	NA	NA	NA	NA	NA	NA
	22.4	73.5	2695	8843	NA	NA	NA	NA	NA	NA	NA	NA	NA
	22.6	74.1	2639	8657	NA	NA	NA	NA	NA	NA	NA	NA	NA
	22.6	74.1	2755	9038	NA	NA	NA	NA	NA	NA	NA	NA	NA
	22.9	75.1	2729	8952	NA	NA	NA	NA	NA	NA	NA	NA	NA
	23.1	75.8	2786	9139	NA	NA	NA	NA	NA	NA	NA	NA	NA
	23.4	76.8	2725	8940	NA	NA	NA	NA	NA	NA	NA	NA	NA
	23.6	77.4	2558	8391	NA	NA	NA	NA	NA	NA	NA	NA	NA
	23.9	78.4	2430	7973	NA	NA	NA	NA	NA	NA	NA	NA	NA
	24.1	79.1	2336	7666	NA	NA	NA	NA	NA	NA	NA	NA	NA
	24.5	80.4	2821	9255	NA	NA	NA	NA	NA	NA	NA	NA	NA
	24.6	80.7	2611	8566	NA	NA	NA	NA	NA	NA	NA	NA	NA
	25	82.0	2695	8843	NA	NA	NA	NA	NA	NA	NA	NA	NA
	25.1	82.3	2639	8657	NA	NA	NA	NA	NA	NA	NA	NA	NA
	25.1	82.3	2725	8940	NA	NA	NA	NA	NA	NA	NA	NA	NA
	25.1	82.3	2639	8657	NA	NA	NA	NA	NA	NA	NA	NA	NA
	25.6	84.0	2378	7802	NA	NA	NA	NA	NA	NA	NA	NA	NA
	26.1	85.6	2210	7250	NA	NA	NA	NA	NA	NA	NA	NA	NA
	26.6	87.3	1963	6439	NA	NA	NA	NA	NA	NA	NA	NA	NA

γ = Poisson's Ratio ρ(Vp) = Vp-derived density G = Shear Modulus E = Young's Modulus K = Bulk Modulus
Shaded cells denote questionable data.

Top of Hole Elevation (m)	Depth (m)	Depth (ft)	Vs (m/s)	Vs (ft/s)	γ	ρ(Vp) (g/cc)	ρ(Vp) (lb/ft^3)	G (GPa)	E (GPa)	K (GPa)	G (10^3lb/ft^2)	E (10^3lb/ft^2)	K (10^3lb/ft^2)
	27.1	88.9	2268	7440	NA	NA	NA	NA	NA	NA	NA	NA	NA
	27.6	90.6	2361	7747	NA	NA	NA	NA	NA	NA	NA	NA	NA
	27.7	90.9	2356	7729	NA	NA	NA	NA	NA	NA	NA	NA	NA
	27.7	90.9	2315	7595	NA	NA	NA	NA	NA	NA	NA	NA	NA
	28.1	92.2	1978	6490	NA	NA	NA	NA	NA	NA	NA	NA	NA
	28.2	92.5	1903	6243	NA	NA	NA	NA	NA	NA	NA	NA	NA
	28.6	93.8	1947	6389	NA	NA	NA	NA	NA	NA	NA	NA	NA
	28.6	93.8	1873	6144	NA	NA	NA	NA	NA	NA	NA	NA	NA
	29.1	95.5	2132	6995	NA	NA	NA	NA	NA	NA	NA	NA	NA
	29.1	95.5	2210	7250	NA	NA	NA	NA	NA	NA	NA	NA	NA
	29.6	97.1	1978	6490	NA	NA	NA	NA	NA	NA	NA	NA	NA
	29.6	97.1	2132	6995	NA	NA	NA	NA	NA	NA	NA	NA	NA
	30.1	98.8	2116	6944	NA	NA	NA	NA	NA	NA	NA	NA	NA
	30.1	98.8	2384	7821	NA	NA	NA	NA	NA	NA	NA	NA	NA
	30.6	100.4	2528	8295	NA	NA	NA	NA	NA	NA	NA	NA	NA
	31.1	102.0	2503	8212	NA	NA	NA	NA	NA	NA	NA	NA	NA
	31.6	103.7	2639	8657	NA	NA	NA	NA	NA	NA	NA	NA	NA
	32.1	105.3	2857	9374	NA	NA	NA	NA	NA	NA	NA	NA	NA
	32.6	107.0	2729	8952	NA	NA	NA	NA	NA	NA	NA	NA	NA
	32.9	107.9	2886	9469	NA	NA	NA	NA	NA	NA	NA	NA	NA
	33.1	108.6	2695	8843	NA	NA	NA	NA	NA	NA	NA	NA	NA
	33.4	109.6	2506	8223	NA	NA	NA	NA	NA	NA	NA	NA	NA
	33.6	110.2	2404	7887	NA	NA	NA	NA	NA	NA	NA	NA	NA
	33.9	111.2	2336	7666	NA	NA	NA	NA	NA	NA	NA	NA	NA
	34.1	111.9	2062	6765	NA	NA	NA	NA	NA	NA	NA	NA	NA
	34.4	112.9	2153	7063	NA	NA	NA	NA	NA	NA	NA	NA	NA
	34.6	113.5	2695	8843	NA	NA	NA	NA	NA	NA	NA	NA	NA
	34.9	114.5	2584	8478	NA	NA	NA	NA	NA	NA	NA	NA	NA
	35.1	115.2	2667	8749	NA	NA	NA	NA	NA	NA	NA	NA	NA
	35.3	115.8	2695	8843	NA	NA	NA	NA	NA	NA	NA	NA	NA
	35.7	117.1	2639	8657	NA	NA	NA	NA	NA	NA	NA	NA	NA

γ = Poisson's Ratio ρ(Vp) = Vp-derived density G = Shear Modulus E = Young's Modulus K = Bulk Modulus
Shaded cells denote questionable data.

Top of Hole Elevation (m)	Depth (m)	Depth (ft)	Vs (m/s)	Vs (ft/s)	γ	ρ(Vp) (g/cc)	ρ(Vp) (lb/ft^3)	G (GPa)	E (GPa)	K (GPa)	G (10^3lb/ft^2)	E (10^3lb/ft^2)	K (10^3lb/ft^2)
	36.1	118.4	2611	8566	NA	NA	NA	NA	NA	NA	NA	NA	NA
	36.7	120.4	2503	8212	NA	NA	NA	NA	NA	NA	NA	NA	NA
	37.2	122.0	2532	8306	NA	NA	NA	NA	NA	NA	NA	NA	NA
	37.7	123.7	2635	8645	NA	NA	NA	NA	NA	NA	NA	NA	NA
	38.1	125.0	2427	7963	NA	NA	NA	NA	NA	NA	NA	NA	NA
	38.6	126.6	2315	7595	NA	NA	NA	NA	NA	NA	NA	NA	NA
	39.1	128.3	2315	7595	NA	NA	NA	NA	NA	NA	NA	NA	NA
	39.6	129.9	2430	7973	NA	NA	NA	NA	NA	NA	NA	NA	NA
	40.1	131.6	2558	8391	NA	NA	NA	NA	NA	NA	NA	NA	NA
	40.1	131.6	2611	8566	NA	NA	NA	NA	NA	NA	NA	NA	NA
	40.1	131.6	2528	8295	NA	NA	NA	NA	NA	NA	NA	NA	NA

γ = Poisson's Ratio ρ(Vp) = Vp-derived density G = Shear Modulus E = Young's Modulus K = Bulk Modulus
Shaded cells denote questionable data.

MATERIALS INFORMATION

Results of Tomographic Seismic Survey for Sidehill Viaduct Bridge Replacement Project, Shasta County, California

Dated December 12, 2014

Memorandum

Serious drought.
Help save water!

To: **Charlie Narwold**
Senior Engineering Geologist
Office of Geotechnical Design North
Division of Engineering Services

Date: December 12, 2014

File: 02-SHA-5-30
Project: 0200000016

Attn: Scott Lewis

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES-MS#5

Subject: Results of Tomographic Seismic Survey for Sidehill Viaduct Bridge Replacement Project, Shasta County, California

Introduction

This memorandum summarizes the results of a tomographic seismic survey to assist in the design of a bridge replacement project at the above referenced project. At this site tomographic seismic, P/S Suspension logging and seismic refraction methods were employed to evaluate the engineering characteristics of the material. The use of three different geophysical methods to supplement the borehole data maximized the drilling effort and knowledge gained from this project. The seismic refraction data and the P/S logging data are presented in separate reports. This report addresses the tomographic seismic portion of the project.

Results and Discussion

The survey employed a network of closely-spaced sources and receivers to provide redundancy and detail. Receivers were placed both down hole and at the surface. This resulted in a level of detail much greater than that achieved using normal refraction profiling. At the surface, sources and receivers were employed every 6.6 ft (2 m) along the 187 ft (57 m) profile. Solid PVC pipe was grouted into test boring RC-14-004. Hydrophones were placed in this water-filled pipe at 6.6 ft (2 m) intervals. The field logs from test borings RC-14-004 and RC-14-006 were used to evaluate the results of the tomographic survey.

Plate 2 shows the processed result of our findings. The field log for RC-14-004 indicates approximately 40 ft (12.2 m) of fill material lying above metamorphic rock (metasediment) described as intensely to very intensely fractured. Top of rock elevation is identified at 1362.1 ft (415.2 m). RC-14-006 is located 105 ft (32 m) along the profile. The field log for RC-14-006 indicates 10.8 ft (3.3 m) of fill and colluvium over intensely to very intensely fractured metamorphic rock. Top of rock elevation at this location is 1355.17 ft. The tomographic seismic model is consistent with the available borehole information. At the boreholes, the velocity model indicates the top of rock is at or near the limit of rippability (Plate 2).

Data Acquisition and Processing

Seismic refraction data were recorded using an EG&G Smartseis 24-channel seismograph with 14-Hz geophones. The profile was 187 ft (57 m) long. The energy source employed was a hammer and striker plate. Seismic waveform data from each shot were stored in the seismograph's memory. Both profile geometry and refraction data were backed up to external memory upon completion of the survey.

SeisOpt Pro, a commercially available computer application, was used to generate the tomographic seismic models. The program utilizes a controlled Monte-Carlo inversion to develop a globally optimized velocity model of the subsurface (Pullammanappallil and Louie, 1994). The methodology uses only first arrival time data and profile geometry as input. No initial assumptions of velocity structure or layering are required. As such, the method is well suited for investigation of areas dominated by complex shallow structure, significant velocity gradients and variable topography. In general, seismic tomographic inversion techniques develop “best-fit” velocity models by iteratively comparing observed arrival data to calculated arrival times derived from generated velocity structures. A final model is produced when the calculated times match observed data within a specified error limit. The velocity models produced by tomographic inversion differ from traditional refraction profiles in that velocities are presented by cell rather than by layer. An advantage of tomography is that the minimum-curve envelope in the inversion (the boundary defined by those ray paths traversing the maximum shot-receiver distances in the shortest time) defines a maximum depth of investigation. No information is available below that envelope. Only estimates of investigation depth are possible using traditional layer analysis. The inversion process may generate false results in cases where data coverage is lacking. Therefore, as with any seismic method, multiple shot points and receivers along a profile provide greater data coverage and redundancy that aid analysis and help generate a more accurate model.

The primary limitation of tomographic inversion is that the resulting solution is non-unique. Non-uniqueness dictates that, if all we can observe are the effects of a causative body (in this case, seismic travel time differences caused by a geologic formation), many models are possible and we are unlikely to ever determine the one, true solution. So we must approximate the true solution by applying suitable constraints to the model universe. Fortunately, nature provides a number of constraints for us: finite dimensions, density ranges and limits on seismic velocity are prime examples. Most models will be invalid for anticipated geologic conditions, and with careful consideration we can pare down our solution universe to those models that are geologically plausible (which can still be a large number) and then use actual geologic observations to provide further discrimination. By those means, we attempt to arrive at a single model, or at least a very small set of models, that best represents actual geologic conditions.

Observed P-wave arrivals and profile geometry were input to SeisOpt Pro, and velocity models were generated using different initial input parameters (i.e., grid size and cell dimensions). Multiple model runs were made at different levels of detail to evaluate appropriate resolution limits. The final velocity model was derived from those results using the procedure described below.

For this investigation, a final model was derived using a statistical analysis of a subset of model runs. Appropriate resolution limits were established by performing multiple model runs with different cell sizes. Six model runs were subsequently made using different grid dimensions, but with the same cell size. The resulting models were evaluated for validity. None of the model runs were invalidated (i.e., yielded implausible results), so all of the runs were included for analysis. The six models were then averaged to produce the final model shown in Plate 2. Additional evaluation of model variance was performed to define uncertainty limits within the final model. The standard deviation of velocity was calculated for each cell within the final model and used to illustrate regions of the model where the defined tolerance is exceeded. For this investigation, tolerance limits are exceeded at any cell where the average seismic velocity exceeds 2000 m/s (the limit of rippability), standard deviation of velocity equals or exceeds 30%, and at least one model yielded a velocity *below* the 2000 m/s cutoff. Regions of the model where tolerance is exceeded are shown as faded areas on Plate 3. It is important to note that these results do not invalidate the entire model. Rather, they indicate regions of the model where the solution is not well-constrained. If any of those regions represent critical locations for the design of engineered remedies, we recommend limited additional investigation to verify conditions at those locations.

Reference

Pullammanappallil, S. K. and J. N. Louie, 1994, A generalized simulated-annealing optimization for inversion of first arrival times, Bulletin of the Seismological Society of America, v. 84, 5 p. 1397-1409.

Thank you for the opportunity to work on this project. If you have any questions or need additional assistance, please contact Dennison Leeds at (916) 227-1307 or William Owen at (916) 227-0227.

Sincerely,

Dennison Leeds
Engineering Geologist
Geophysics and Geology Branch

William Owen, PGP 1031
Chief, Geophysics and Geology Branch

PROFESSIONAL GEOPHYSICIST
W.P. OWEN
No. 1031
Exp. 3/31/2016
STATE OF CALIFORNIA

c: Project File.

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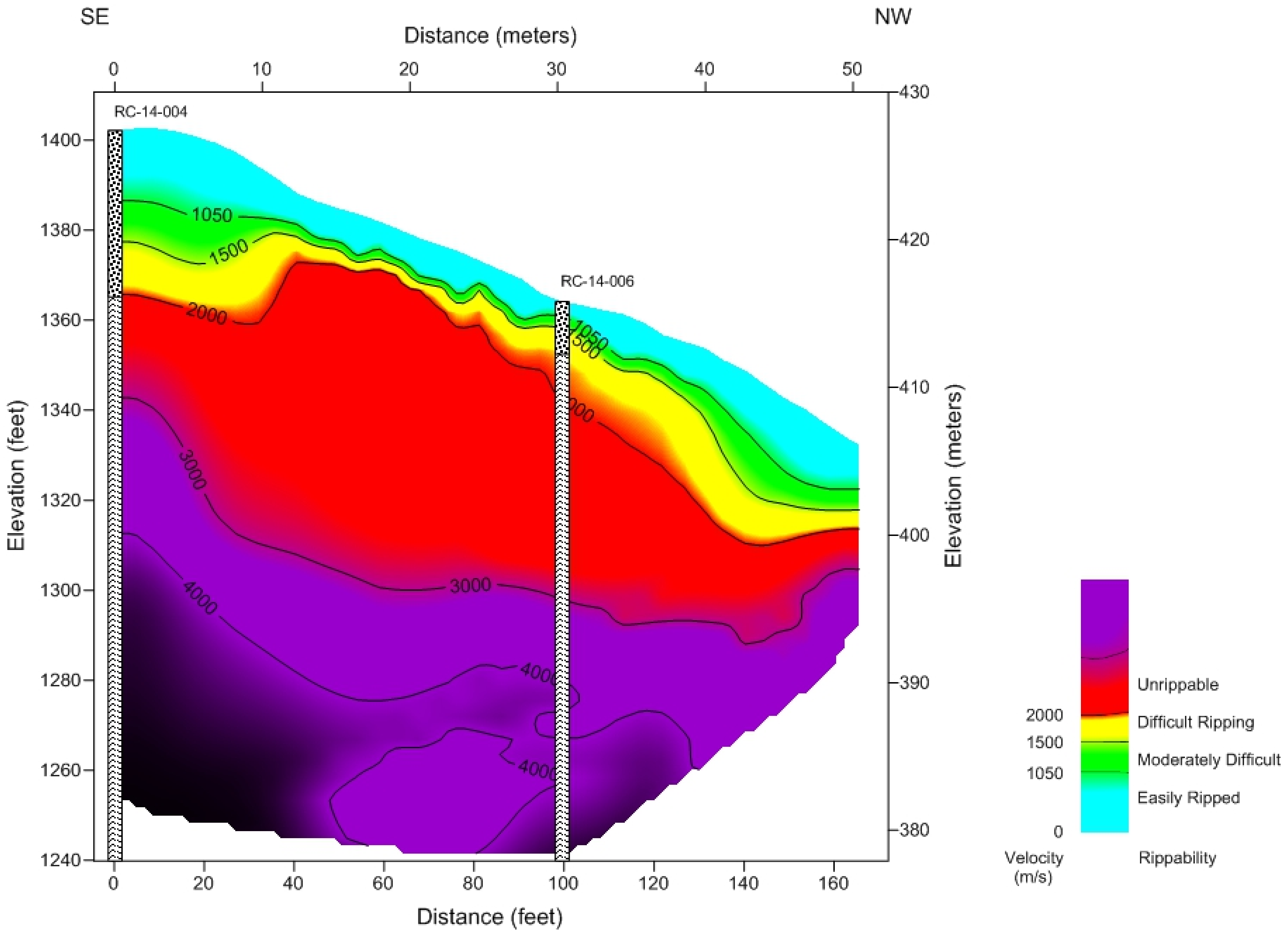
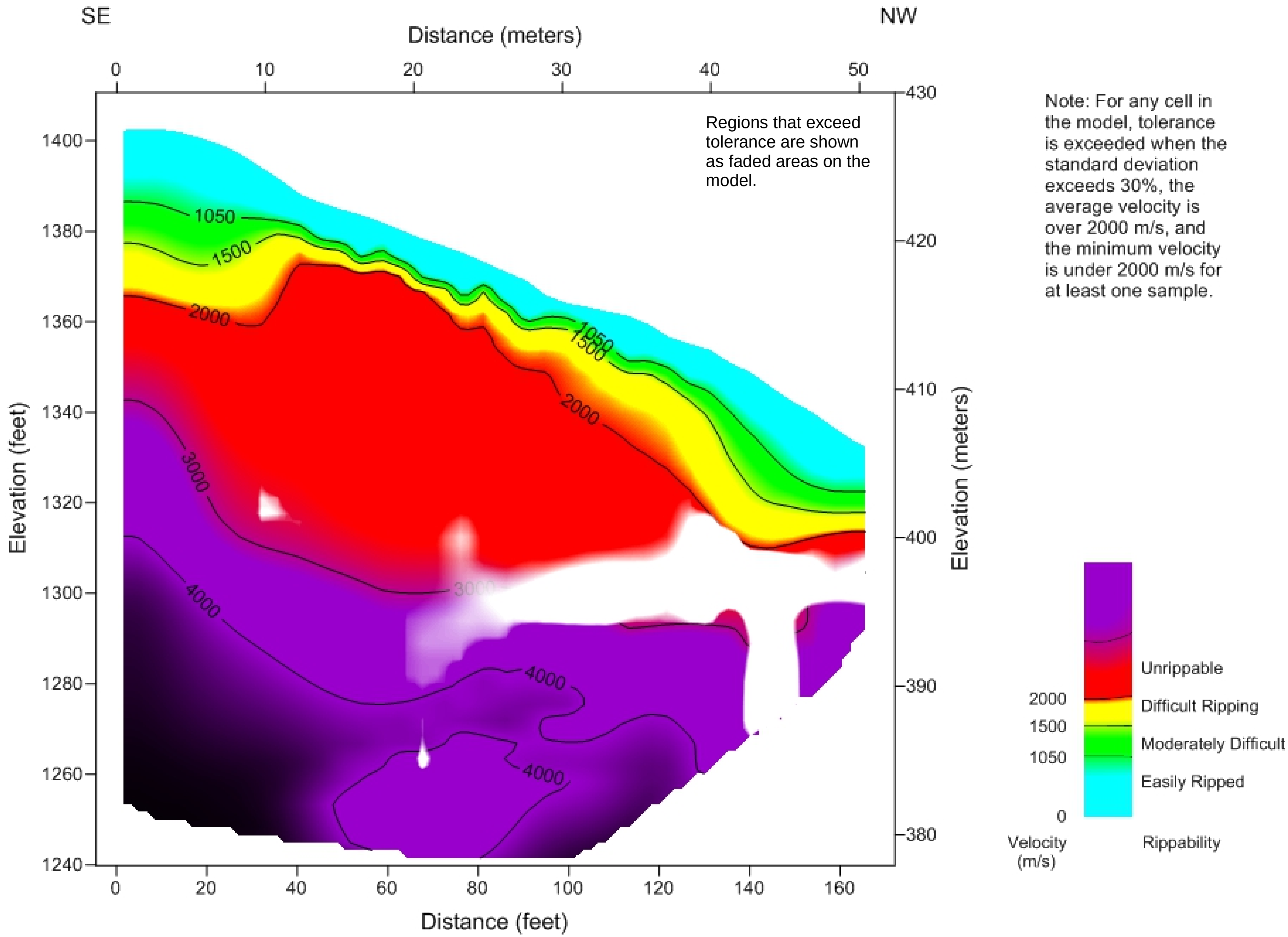
Caltrans Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	02-SHA-5-30, Sidehill Viaduct Bridge Replacement	Plate 1
	0200000016	Location Map of Tomographic Line	



Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	02-SHA-5-30, Sidehill Viaduct Bridge Replacement	Plate 2
	0200000016	Average Velocity Model	



Caltrans — **Division of Engineering Services** **Office of Geotechnical Support** **Geophysics and Geology Branch**	EA 02-0E090	02-SHA-5-30, Sidehill Viaduct Bridge Replacement	Plate 3
	0200000016	Tolerance Plot	

MATERIALS INFORMATION

Results of Seismic Refraction Survey for Sidehill Viaduct Bridge Replacement Project, Shasta County, California

Dated December 23, 2014

M e m o r a n d u m

Serious drought.
Help save water!

To: **Charlie Narwold**
Senior Engineering Geologist
Office of Geotechnical Design North
Division of Engineering Services

Date: December 23, 2014

File: 02-SHA-05-30
Project: 0200000016

Attn: Scott Lewis

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES-MS#5

Subject: Results of Seismic Refraction Survey for Sidehill Viaduct Bridge Replacement Project, Shasta County, California

Introduction

This memorandum summarizes the results of a seismic survey at the above-referenced project. We performed the work to assist in the design of the bridge replacement. We used the seismic refraction method to evaluate the rippability of material to be excavated for construction of access roads, platforms and shallow footings associated with the new bridge. Fifteen seismic refraction lines were acquired in May 2014. Plate 1 shows the approximate locations of the seismic lines.

Geology

The site is located on the eastern side of the Klamath Mountains geomorphic province of California. Site geology consists of metamorphosed sedimentary and volcanic rocks of the Eastern Klamath Belt (Harden, 1998), overlain by unconsolidated landslide deposits. The site is located on steep and rugged terrain with dense brush, making access difficult. The steep slopes and loose footing at times required the use of fall protection equipment to perform field work.

Results and Discussion

The seismic refraction profiles varied in length and had geophones placed every 5 ft (1.5 m). The results of the survey were limited in depth, because the rugged topography and dense growth limited the profile lengths and shot offsets that are required for deeper investigation. We were able to calculate seismic velocities in the shallow subsurface at each seismic line, but modeling of the refractor surface at the top of rock was often incomplete. Our results are summarized in Table 1. Additional discussion is also provided below.

Table 1. Results of Seismic Refraction Study for Sidehill Viaduct Bridge Replacement

Seismic Line	Project Station	Layer	Average Thickness	Average Velocity	Line Length	Inferred Material	Rippability[1]
SH 1	899+70	1	20 ft (6.1 m)	1680 ft/s (512 m/s)	118 ft (36 m)	Colluvium	ER
		2	N/A	8400 ft/s (2560 m/s)		Slightly Weathered Metasediments	NR
SH 2	900+35 to 899+16	1	22 ft (6.7 m)	1900 ft/s (580 m/s)	118 ft (36 m)	Colluvium	ER
		2	N/A	7050 ft/s (2150 m/s)		Slightly Weathered Metasediments	NR
SH 3	910+83	1	13 ft (4.0 m)	1200 ft/s (366 m/s)	118 ft (36 m)	Colluvium	ER
		2	23 ft (7 m)	2160 ft/s (658 m/s)		Moderately Weathered Metasediments	ER
		3	N/A	6700 ft/s (2040 m/s)		Slightly Weathered Metasediments	NR
SH 4	911+34 to 910+15	1	5 ft (1.5 m)	990 ft/s (302 m/s)	118 ft (36 m)	Colluvium	ER
		2	28 ft (8.5 m)	1900 ft/s (579 m/s)		Moderately Weathered Metasediments	ER
		3	N/A	7800 ft/s (2380 m/s)		Slightly Weathered Metasediments	NR
SH 5	901+57 to 900+65	1	10 ft (3.0 m)	1250 ft/s (381 m/s)	118 ft (36 m)	Colluvium	ER
		2	N/A	9150 ft/s (2790 m/s)		Slightly Weathered Metasediments	NR
SH 6	902+56	1	24 ft (7.3 m)	1700 ft/s (518 m/s)	118 ft (36 m)	Colluvium	ER
		2	N/A	11,400 ft/s (3470 m/s)		Metasediments	NR
SH 7	904+14 to 902+56	1	27 ft (8.2 m)	5000 ft/s (1520 m/s)	157 ft (48 m)	Highly Weathered Metasediments	DR
		2	N/A	10,800 ft/s (3290 m/s)		Metasediments	NR
SH 8	896+88	1	17 ft (5.2 m)	1200 ft/s	157 ft	Colluvium	ER

Table 1. Results of Seismic Refraction Study for Sidehill Viaduct Bridge Replacement

Seismic Line	Project Station	Layer	Average Thickness	Average Velocity	Line Length	Inferred Material	Rippability[1]
				(366 m/s)	(48 m)		
		2	N/A	7400 ft/s (2260 m/s)		Moderately Weathered Metasediments	NR
SH 9	897+73 to 896+55	1	11 to 22 ft (3.4 to 6.7 m)	1633 ft/s (498 m/s)	118 ft (36 m)	Colluvium	ER
		2	N/A	6670 ft/s (2030 m/s)		Unconsolidated west side	NR
SH 10	898+23	1	15 ft (4.6 m)	1250 ft/s (381 m/s)	118 ft (36 m)	Colluvium	ER
		2	N/A	11,500 ft/s (3500 m/s)		Metasediments	NR
SH 11	895+02	1	18 ft (5.5 m)	1800 ft/s (549 m/s)	79 ft (24 m)	Fill	ER
		2	N/A	4680 ft/s (1426 m/s)		Highly Weathered Metasediments	MD
SH 12	896+20 to 895+02	1	19 ft (5.8 m)	2300 ft/s (701 m/s)	118 ft (36 m)	Colluvium	ER
		2	N/A	4920 ft/s (1500 m/s)		Highly Weathered Metasediments	MD
SH 13	892+50	1	15 ft (4.6 m)	1300 ft/s (396 m/s)	118 ft (36 m)	Colluvium	ER
		2	N/A	3200 ft/s (975 m/s)		Completely Weathered Metasediment?	ER
SH 14	892+80 to 891+24	1	18 to 33 ft (5.5 to 10.1 m)	2450 ft/s (747 m/s)	157 ft (48 m)	Fill	ER
		2	N/A	4900 ft/s (1494 m/s)		Weathered Metasediments	MD
SH 15	887+33 to 888+18	1	12 ft (3.7 m)	1450 ft/s (442 m/s)	118 ft (36 m)	Colluvium	ER
		2	N/A	2700 ft/s (823 m/s)		Completely Weathered Metasediment?	ER

[1] ER = Easily Ripped, MD = Moderately Difficult, DR = Difficult Ripping, NR = Not Rippable

Seismic Line SH 1 is located at project station 899+70 and was surveyed normal to project stationing. The seismic line is 118 ft (36 m) in length and has two velocity units. The upper layer is approximately 20 ft (6.1 m) thick, has a velocity of 1680 ft/s (512 m/s), and will be easily ripped. The lower layer has a velocity of 8400 ft/s (2560 m/s) and is not rippable. The processed model is shown on Plate 2.

Seismic Line SH 2 runs from project station 900+35 to 899+16 and was surveyed parallel to project stationing. The seismic line is 118 ft (36 m) in length and has two velocity units. The upper layer is approximately 22 ft (6.7 m) thick, has a velocity of 1900 ft/s (580 m/s), and will be easily ripped. The lower layer has a velocity of 7050 ft/s (2150 m/s) and is not rippable. The processed model is shown on Plate 3. This profile exhibited a poor tie with SH 1, a conclusive explanation for the mis-tie is not available.

Seismic Line SH 3 is located at project station 910+83 and was surveyed normal to project stationing. The seismic line is 118 ft (36 m) in length and has three velocity units. The upper layer is approximately 13 ft (4.0 m) thick, has a velocity of 1200 ft/s (366 m/s), and will be easily ripped. The middle layer is approximately 23 ft (7 m) thick, has a velocity of 2160 ft/s (658 m/s), and will also be easily ripped. The lower layer has a velocity of 6700 ft/s (2040 m/s) and is not rippable. The processed model is shown on Plate 4. This profile exhibits a good tie with Line SH 4 at the top of the middle layer. However, because only limited GRM coverage was possible at the base of SH 3, a lower-layer tie between SH3 and SH 4 could not be made.

Seismic Line SH 4 runs from project station 911+34 to 910+15 and was surveyed parallel to project stationing. The seismic line is 118 ft (36 m) in length and has three velocity units. The upper layer is approximately 5 ft (1.5 m) thick, has a velocity of 990 ft/s (302 m/s), and will be easily ripped. The middle layer is approximately 28 ft (8.5 m) thick, has a velocity of 1900 ft/s (579 m/s), and will also be easily ripped. The lower layer has a velocity of 7800 ft/s (2380 m/s) and is not rippable. The processed model is shown on Plate 5.

Seismic Line SH 5 runs from project station 901+57 to 900+65 and was surveyed oblique to project stationing. The seismic line is 118 ft (36 m) in length and has two velocity units. The upper layer is approximately 10 ft (3.0 m) thick, has a velocity of 1250 ft/s (381 m/s), and will be easily ripped. The lower layer has a velocity of 9150 ft/s (2790 m/s) and is not rippable. The processed model is shown on Plate 6.

Seismic Line SH 6 is located at project station 902+56 and was surveyed normal to project stationing. The seismic line is 118 ft (36 m) in length has two velocity units. The upper layer is approximately 24 ft (7.3 m) thick, has a velocity of 1700 ft/s (518 m/s), and will be easily ripped. The lower layer has a velocity of 11,400 ft/s (3470 m/s) and is not rippable. The processed model is shown on Plate 7.

Seismic Line SH 7 runs from project station 904+14 to 902+56 and was surveyed parallel to project stationing. The seismic line is 157 ft (48 m) in length and has two velocity units. The upper layer is approximately 27 ft (8.2 m) thick, has a velocity of 5000 ft/s (1520 m/s), and will experience difficult ripping. The lower layer has a velocity of 10,800 ft/s (3290 m/s) and is not rippable. The processed model is shown on Plate 8.

Seismic Line SH 8 is located at project station 896+88 and was surveyed normal to project stationing. The seismic line is 157 ft (48 m) in length and has two velocity units. The upper layer is approximately 17 ft (5.2 m) thick, has a velocity of 1200 ft/s (366 m/s), and will be easily ripped. The lower layer has a velocity of 7400 ft/s (2260 m/s) and is not rippable. The processed model is shown on Plate 9.

Seismic Line SH 9 runs from project station 897+73 to 896+55 and was surveyed parallel to project stationing. The seismic line is 118 ft (36 m) in length and has two velocity units. The upper layer varies in thickness from 11 ft (3.4 m) in the west to 22 ft (6.7 m) in the east; it has a velocity of 1633 ft/s (498 m/s) and is easily ripped. The lower layer has a velocity of 6670 ft/s (2030 m/s) and is not rippable. The processed model is shown on Plate 10. This profile exhibits an excellent tie with Line SH 8.

Seismic Line SH 10 is located at project station 898+23 and was surveyed normal to project stationing. The seismic line is 118 ft (36 m) in length and has two velocity units. The upper layer is approximately 15 ft (4.6 m) thick, has a velocity of 1250 ft/s (381 m/s), and is easily ripped. The lower layer has a velocity of 11,500 ft/s (3500 m/s) and is not rippable. The processed model is shown on Plate 11.

Seismic Line SH 11 is located at project station 895+02 and was surveyed normal to project stationing. The seismic line is 79 ft (24 m) in length and has two velocity units. The upper layer is approximately 18 ft (5.5 m) thick, has a velocity of 1800 ft/s (549 m/s), and is easily ripped. The lower layer has a velocity of 4680 ft/s (1426 m/s) and will experience moderately difficult ripping. The processed model is shown on Plate 12.

Seismic Line SH 12 runs from project station 896+20 to 895+02 and was surveyed parallel to project stationing. The seismic line is 118 ft (36 m) in length and has two velocity units. The upper layer is approximately 19 ft (5.8 m) thick, has a velocity of 2300 ft/s (701 m/s), and is easily ripped. The lower layer has a velocity of 4920 ft/s (1500 m/s) and will experience moderately difficult ripping. The processed model is shown on Plate 13. This profile exhibits a good tie with Line SH 11.

Seismic Line SH 13 is located at project station 892+50 and was surveyed normal to project stationing. The seismic line is 118 ft (36 m) in length and has two velocity units. The upper layer is approximately 15 ft (4.6 m) thick, has a velocity of 1300 ft/s (396 m/s), and is easily ripped. The lower layer has a velocity of 3200 ft/s (975 m/s) and is also easily ripped. The processed model is shown on Plate 14.

Seismic Line SH 14 runs from project station 892+80 to 891+24 and was surveyed parallel to project stationing. The seismic line is 157 ft (48 m) in length and has two velocity units. The upper layer is fill ranging in thickness from 18 ft (5.5 m) in the west to 33 ft (10.1 m) in the east; it has a velocity of 2450 ft/s (747 m/s) and is easily ripped. The lower layer is also fill having a velocity of 4900 ft/s (1494 m/s) and will experience moderately difficult ripping. The processed model is shown on Plate 15.

Seismic Line SH 15 runs from project station 887+33 to 888+18 and was surveyed oblique to project stationing. The seismic line is 118 ft (36 m) in length and has two velocity units. The upper layer is approximately 12 ft (3.7 m) thick, has a velocity of 1450 ft/s (442 m/s), and is easily ripped. The lower layer is an unconsolidated unit having a velocity of 2700 ft/s (823 m/s) and is easily ripped. The processed model is shown on Plate 16.

Ripping ability is based on unpublished Caltrans data for the Caterpillar D9 series tractor with a single-tooth ripper. These values are as follows:

Velocity	**Rippability**
<3440 ft/s (<1050 m/s)	Easily Ripped
3440-4920 ft/s (1050-1500 m/s)	Moderately Difficult
4920-6560 ft/s (1500-2000 m/s)	Difficult Ripping
>6560 ft/s (>2000 m/s)	Not Rippable

Different excavation equipment may experience different results. Penetrating efficacy of the ripping tooth is often more important in predicting ripping success than seismic velocity alone. Undetected blocks or lenses of high-velocity material may exist within rippable zones, requiring blasting or other means of mechanical breakage for excavation.

Data Acquisition and Processing

Seismic refraction data were recorded using an EG&G Smartseis 24-channel seismograph with 14-Hz geophones. The profiles varied in length. The energy source employed was a hammer and striker plate. Refraction data from each shot were stored in the seismograph's memory. Both profile geometry and refraction data were backed up to external memory upon completion of the survey.

Refraction data were processed using GeoGiga Refractor, a commercially available computer application. Initial P-wave arrivals were picked from the seismic refraction records, refractor layers were evaluated and assigned, and analysis was performed using the Generalized Reciprocal Method (GRM).

The GRM is described by Palmer (1980). The GRM calculates refractor depths for each geophone location using overlapping refraction arrival times from both forward and reverse shots. The method can accommodate variation in refractor velocity and depth along the seismic line, is relatively insensitive to refractor dips up to 20 degrees, and can accommodate hidden layer conditions when coupled with borehole data. Where refraction data are insufficient for GRM interpretation, the refractor can be modeled using the standard intercept-time method of interpretation (ITM), though with comparatively reduced accuracy and resolution (for ITM, layer depths are calculated from each shot point, not from each geophone, and far fewer shot points are available on a typical refraction profile). Each GRM model is shown as a profile view of the velocity section. The velocity section is a layered representation of the subsurface velocity distribution. GRM interpretations are drawn as a series of arcs, with each arc representing the solution set for individual refractor locations beneath each geophone. The envelope formed by the locus of interconnecting points near the base of these arcs represents the best-fit model for the refractor. Velocity sections in this report are presented in terms of velocity units. A velocity unit is a three-dimensional unit which, due to its elastic properties and density, propagates seismic waves at a characteristic velocity or within a characteristic velocity range. A velocity unit may not correspond to a discrete stratigraphic unit. Stratigraphic units may contain more than one seismic velocity, and each zone of weathering or fracturing within a stratigraphic unit can constitute its own velocity unit. Conversely, when two or more juxtaposed stratigraphic units propagate seismic waves at the same velocity, such as saturated gravel and moderately weathered rock, both units would appear as the same velocity unit. Those potential effects on the velocity section should be considered in the interpretation of the data.

References

Harden, D., 1998, California Geology: Prentice Hall, Inc., Upper Saddle River, New Jersey, p. 196-97.

Palmer, D., 1980, The generalized reciprocal method of seismic refraction interpretation: Society of Exploration Geophysicists, Tulsa, Oklahoma, 104 p.

Thank you for the opportunity to work on this project. If you have any questions or need additional assistance, please contact Dennison Leeds at (916) 227-1307 or William Owen at (916) 227-0227.

Sincerely,

Dennison Leeds
Engineering Geologist
Geophysics and Geology Branch

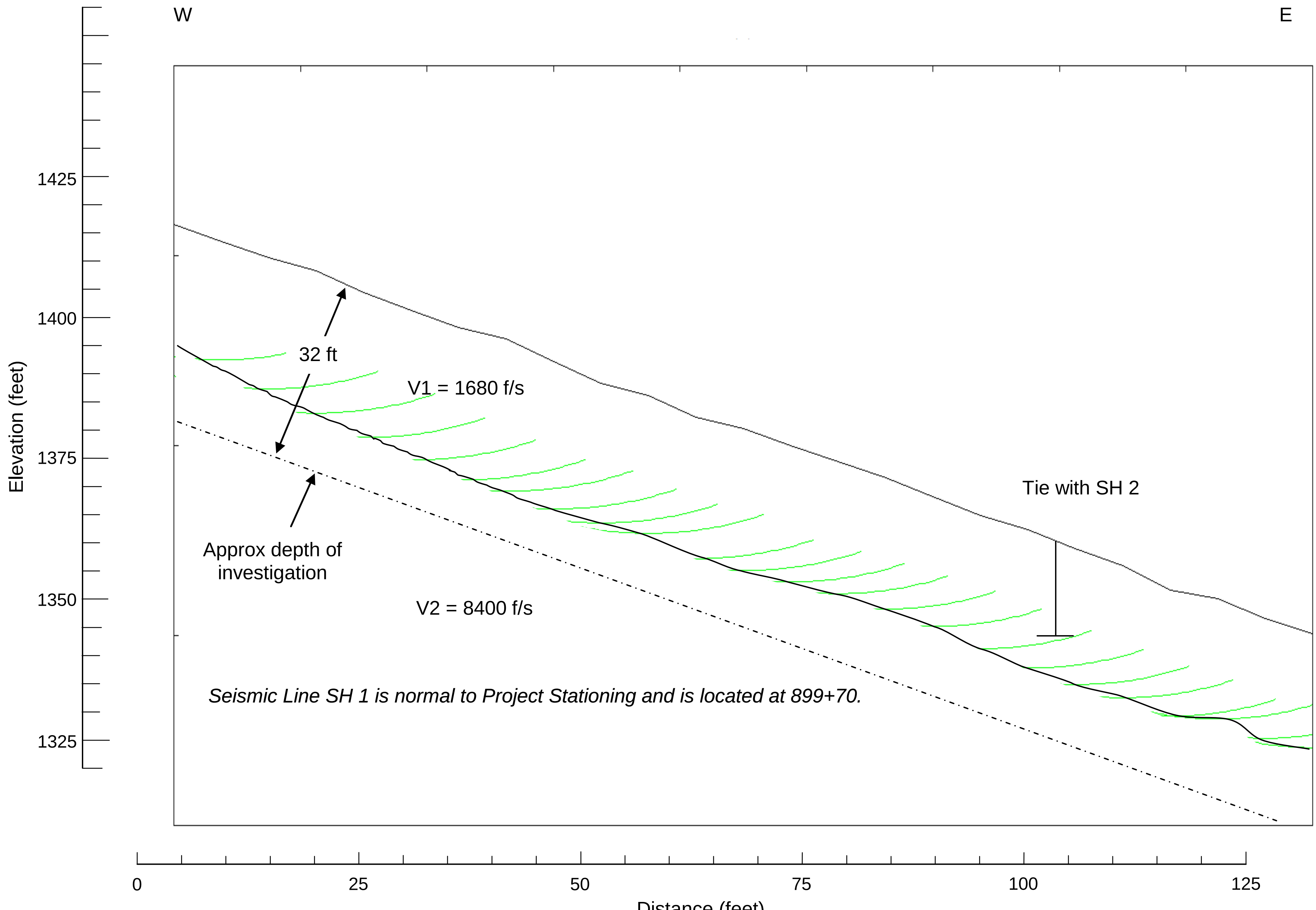
William Owen, PGP 1031
Chief, Geophysics and Geology Branch

c: Project File.

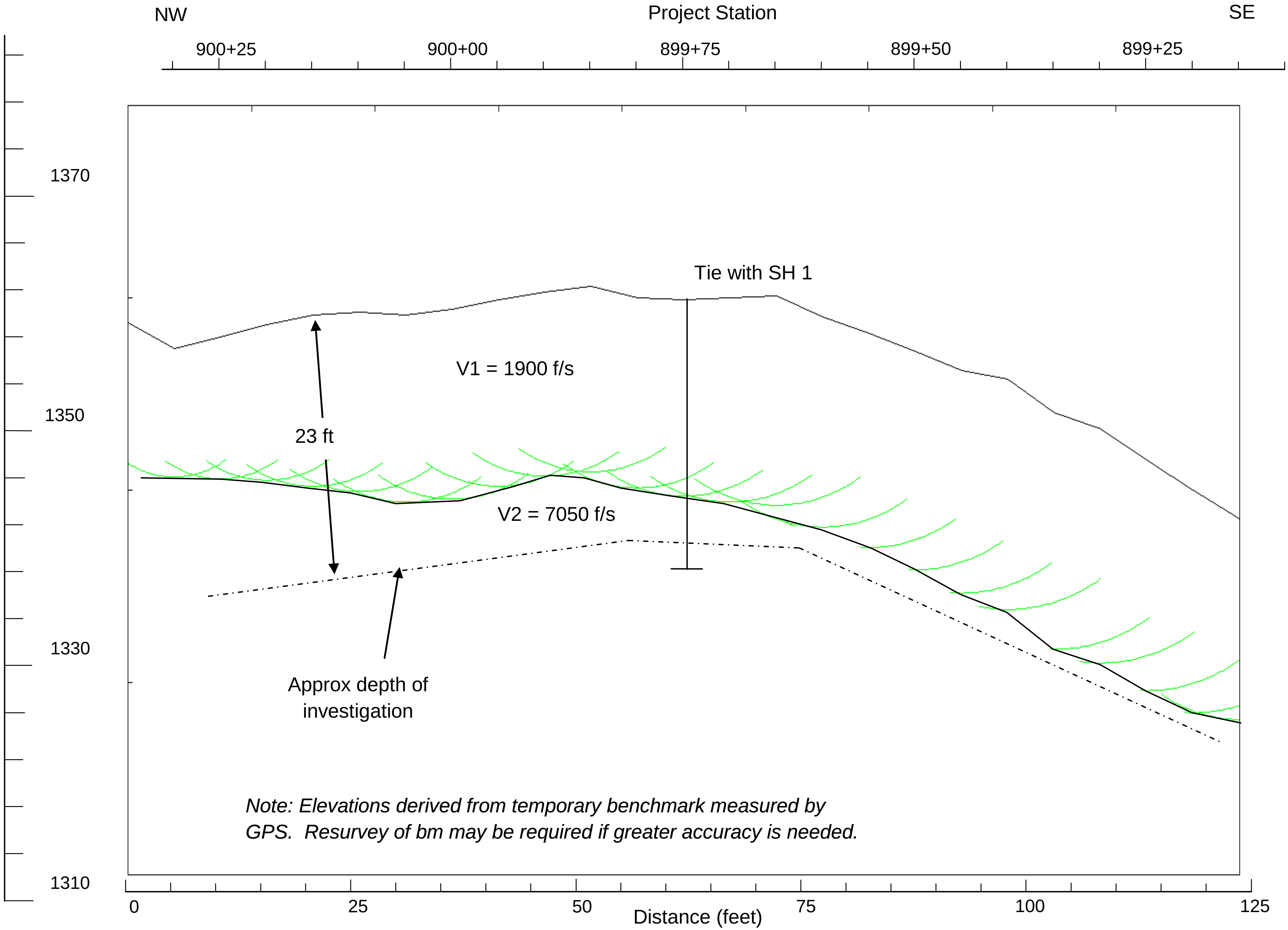
DL/WO/sm

02_SHA_5_30_2014_SEI.doc

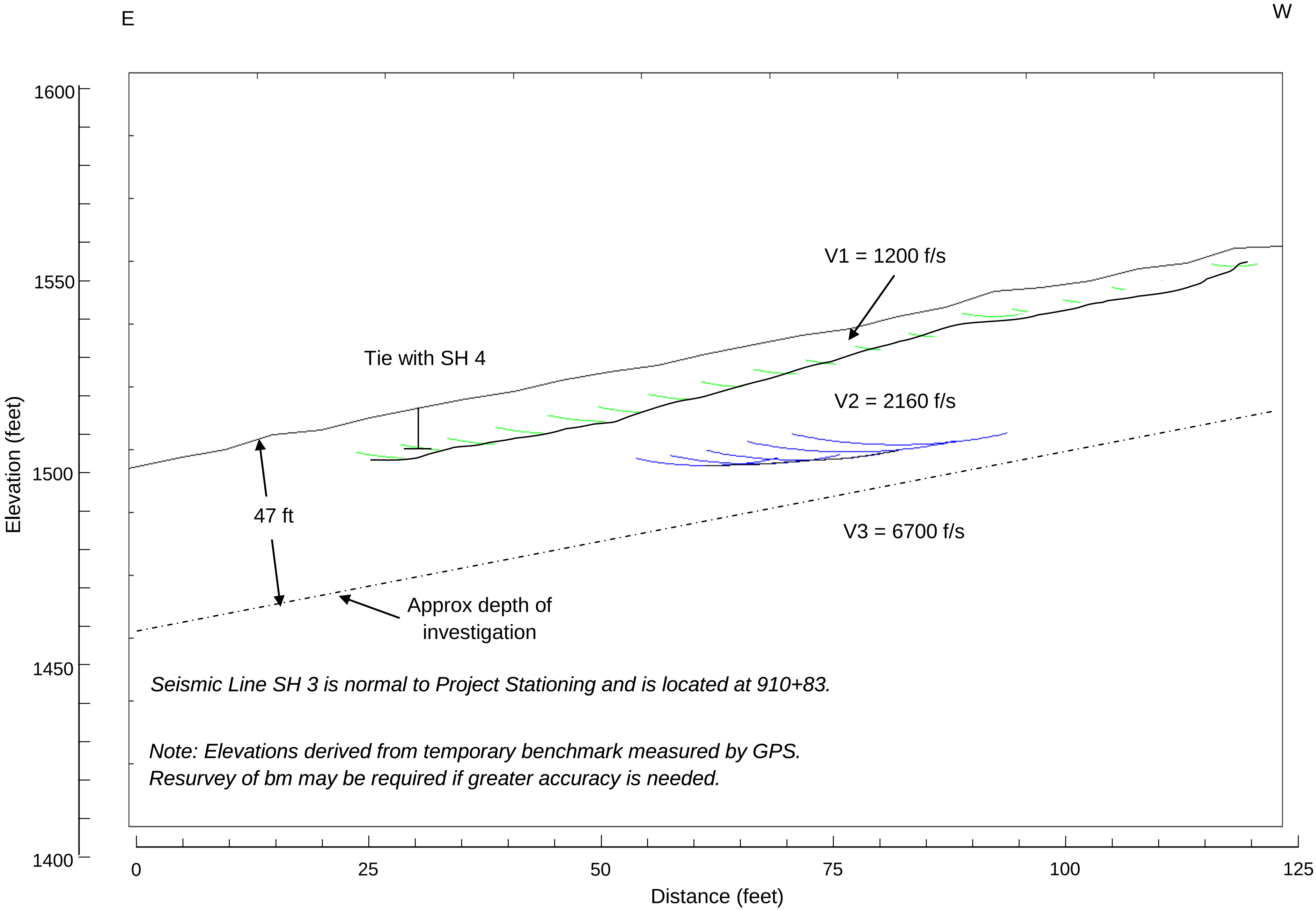
Caltrans Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.1
	ID 0200000016	Location Map of Seismic Lines	



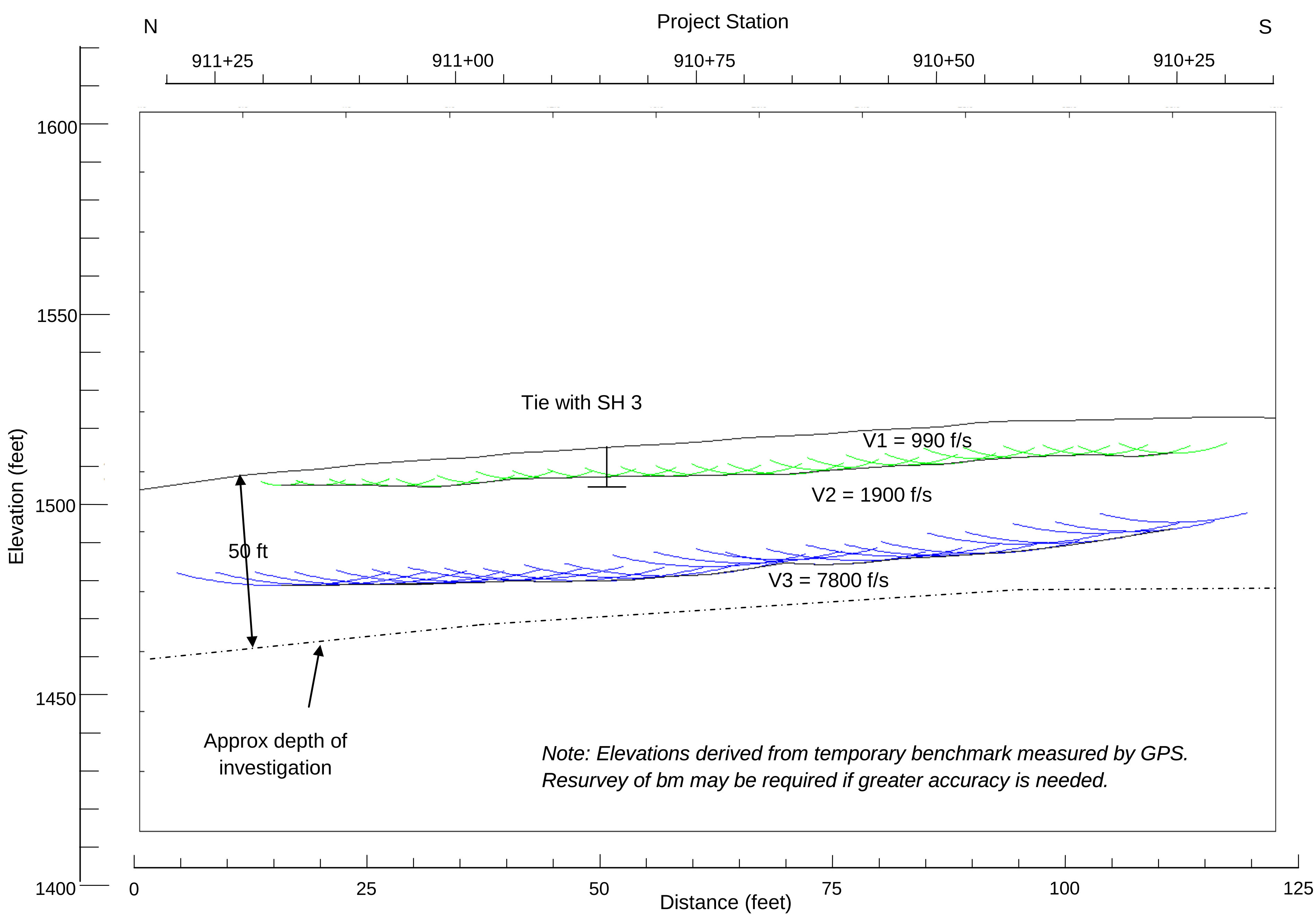
Caltrans	Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No. 2
		0200000016	Seismic Line SH 1	



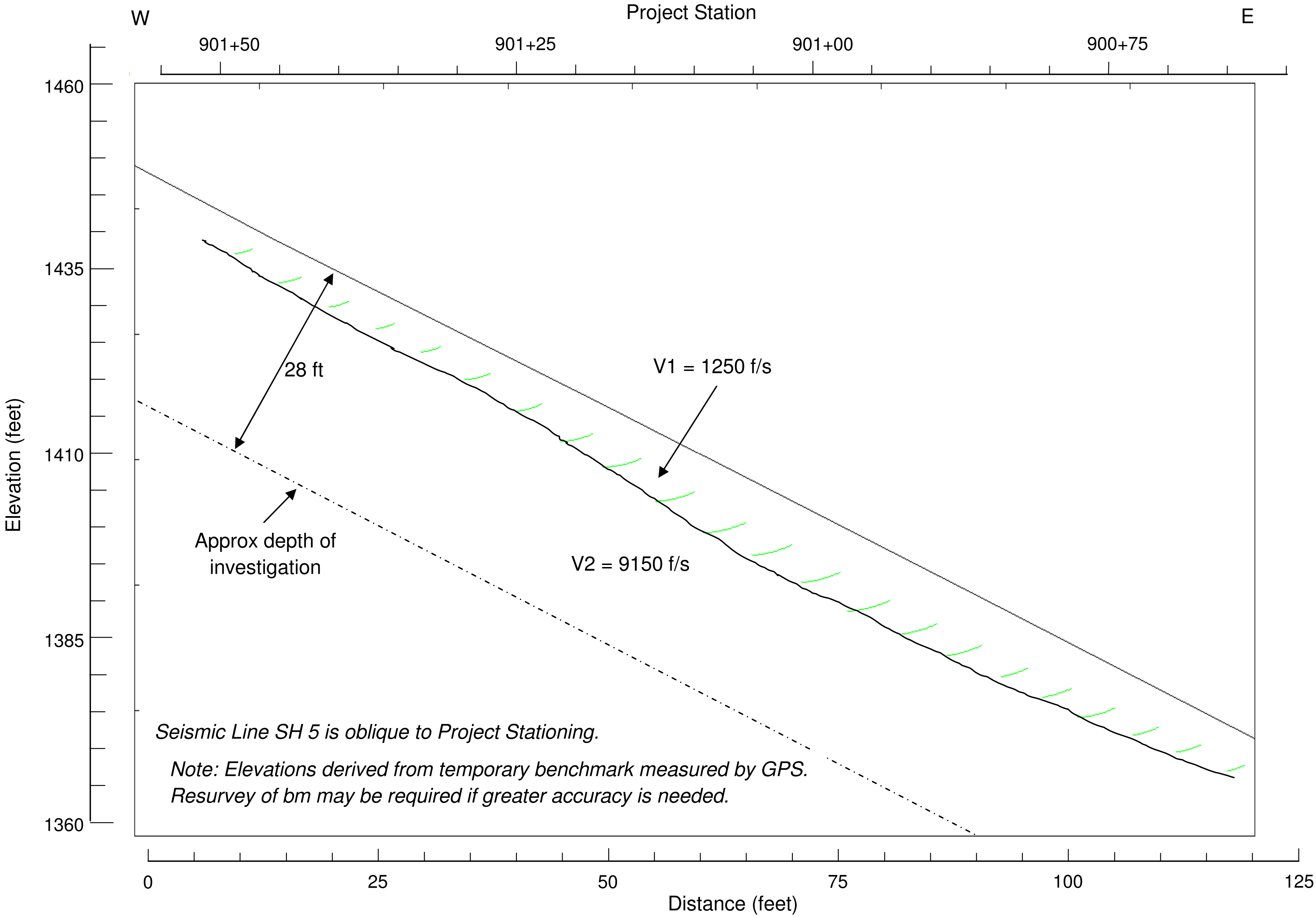
Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No. 3
	0200000016	Seismic Line SH 2	



Caltrans — Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No. 4
	0200000016	Seismic Line SH 3	

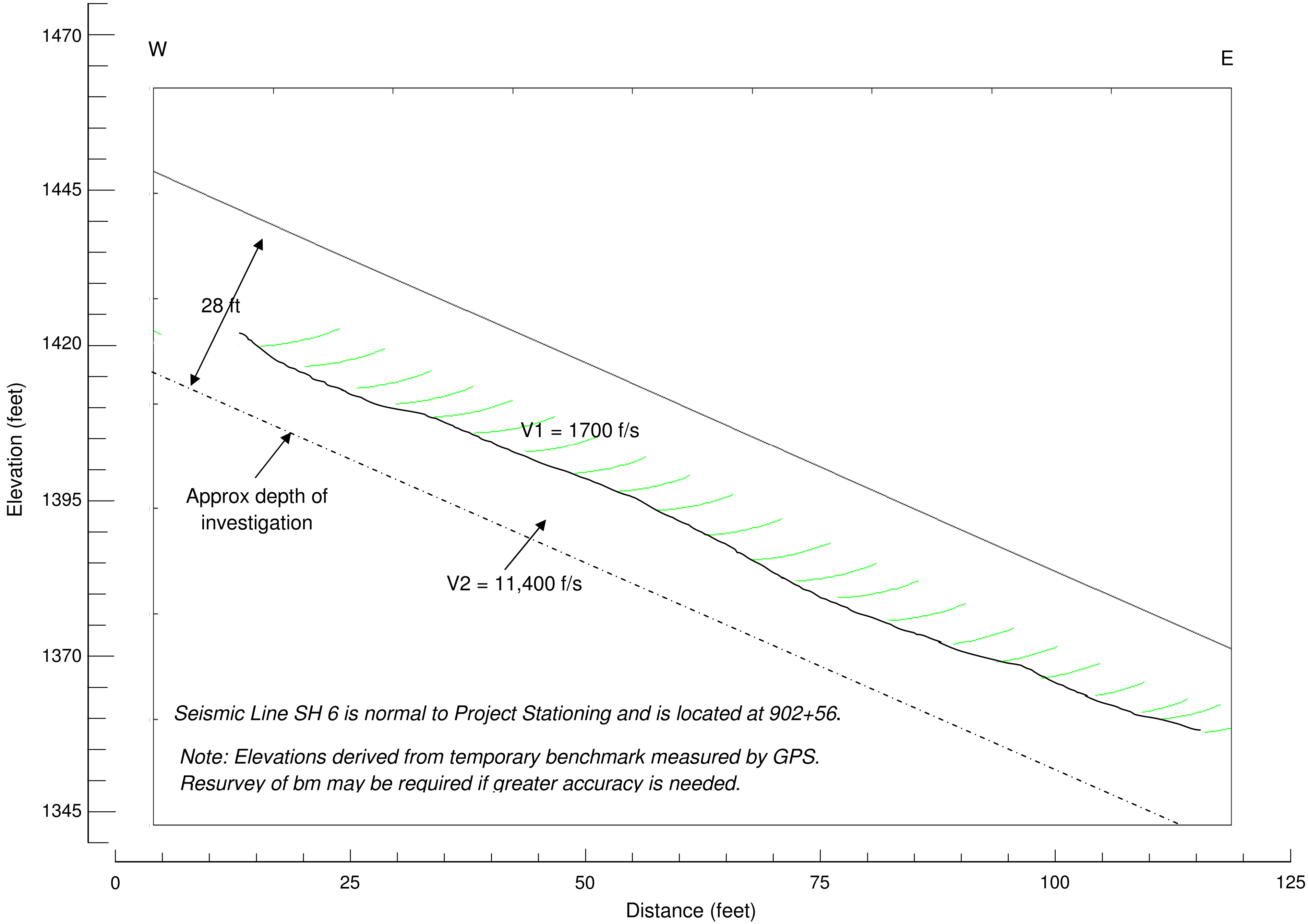


Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.5
	0200000016	Seismic Line SH 4	

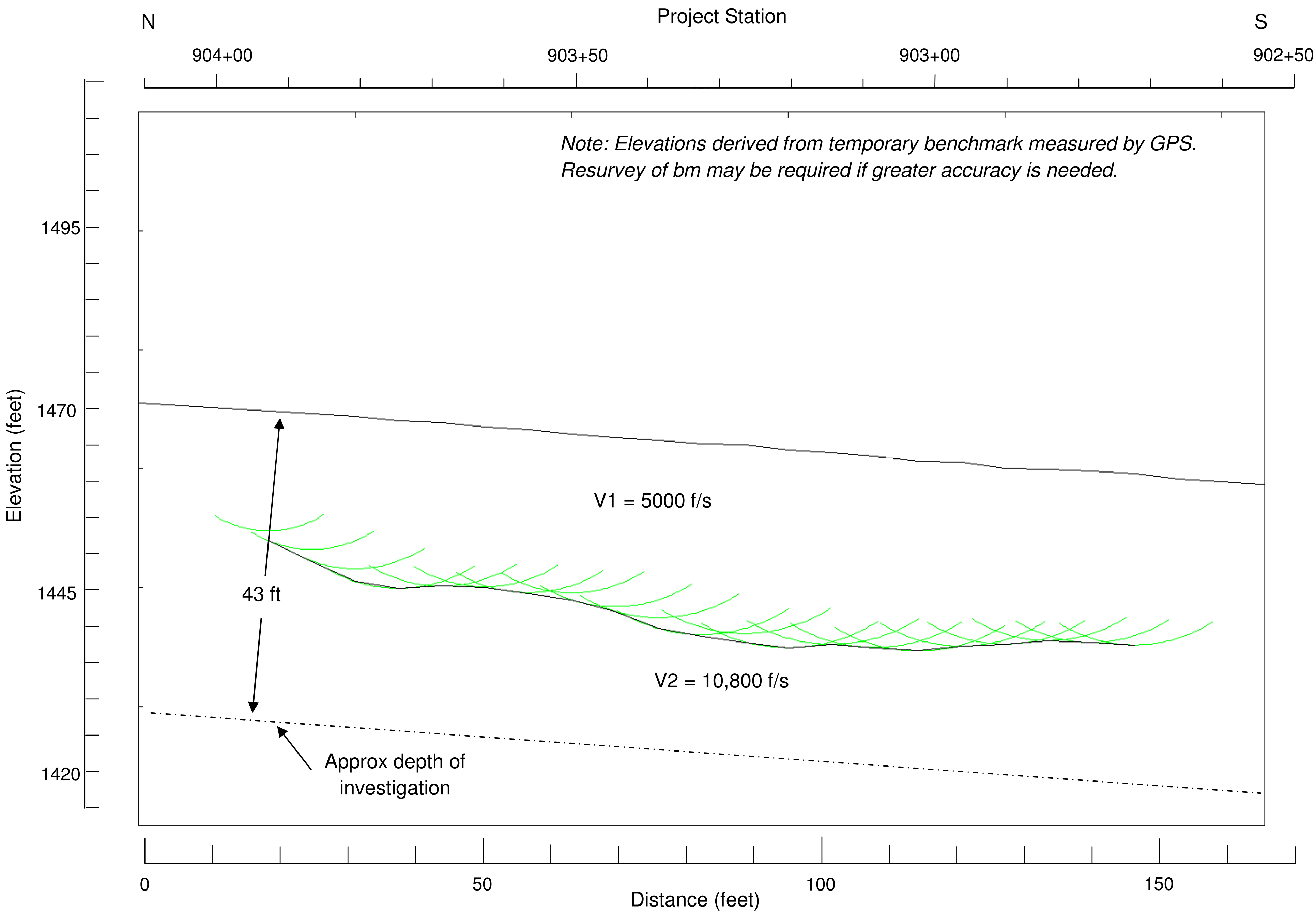


Division of Engineering Services
Office of Geotechnical Support
Geophysics and Geology Branch

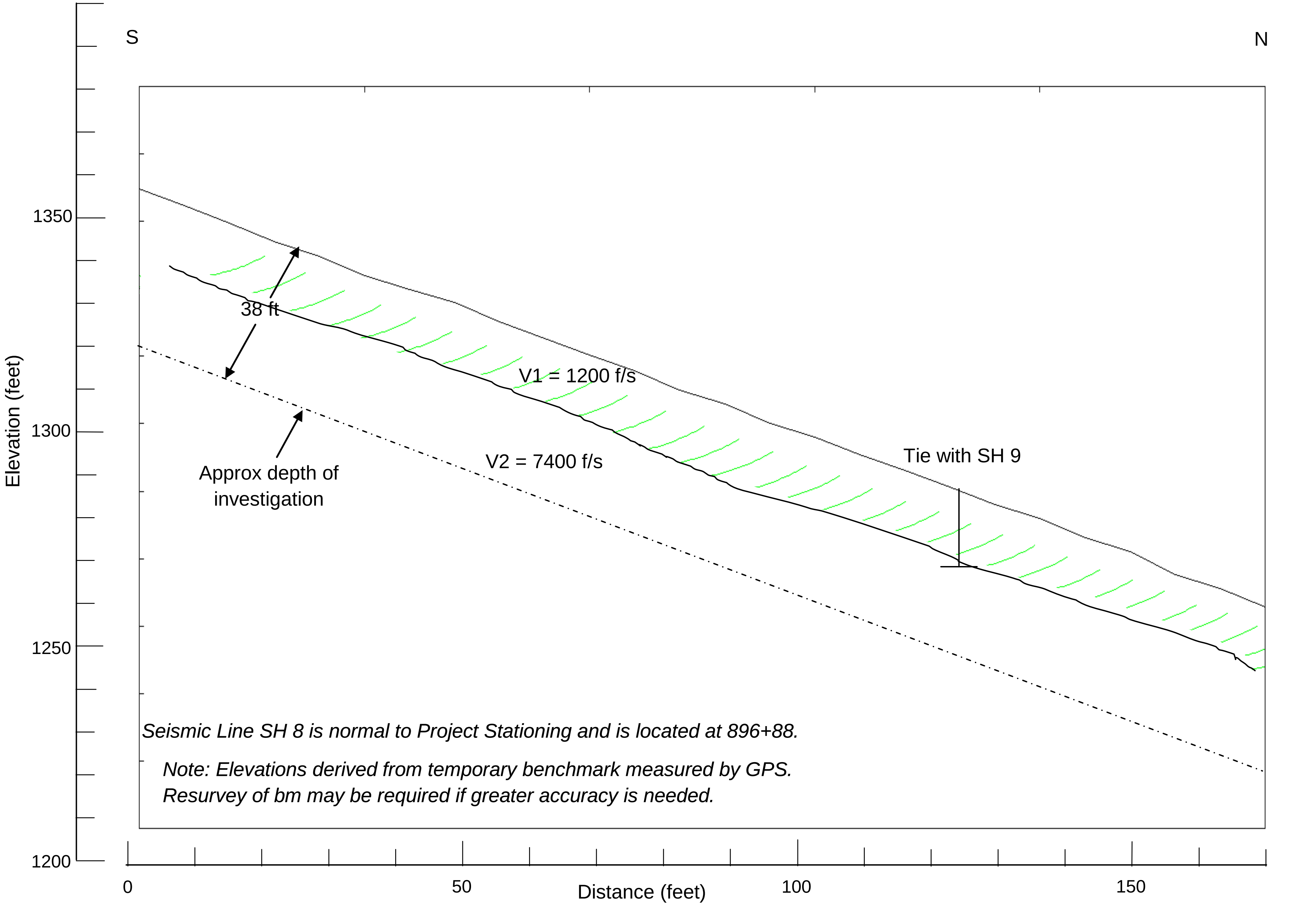
EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No. 6
0200000016	Seismic Line SH 5	



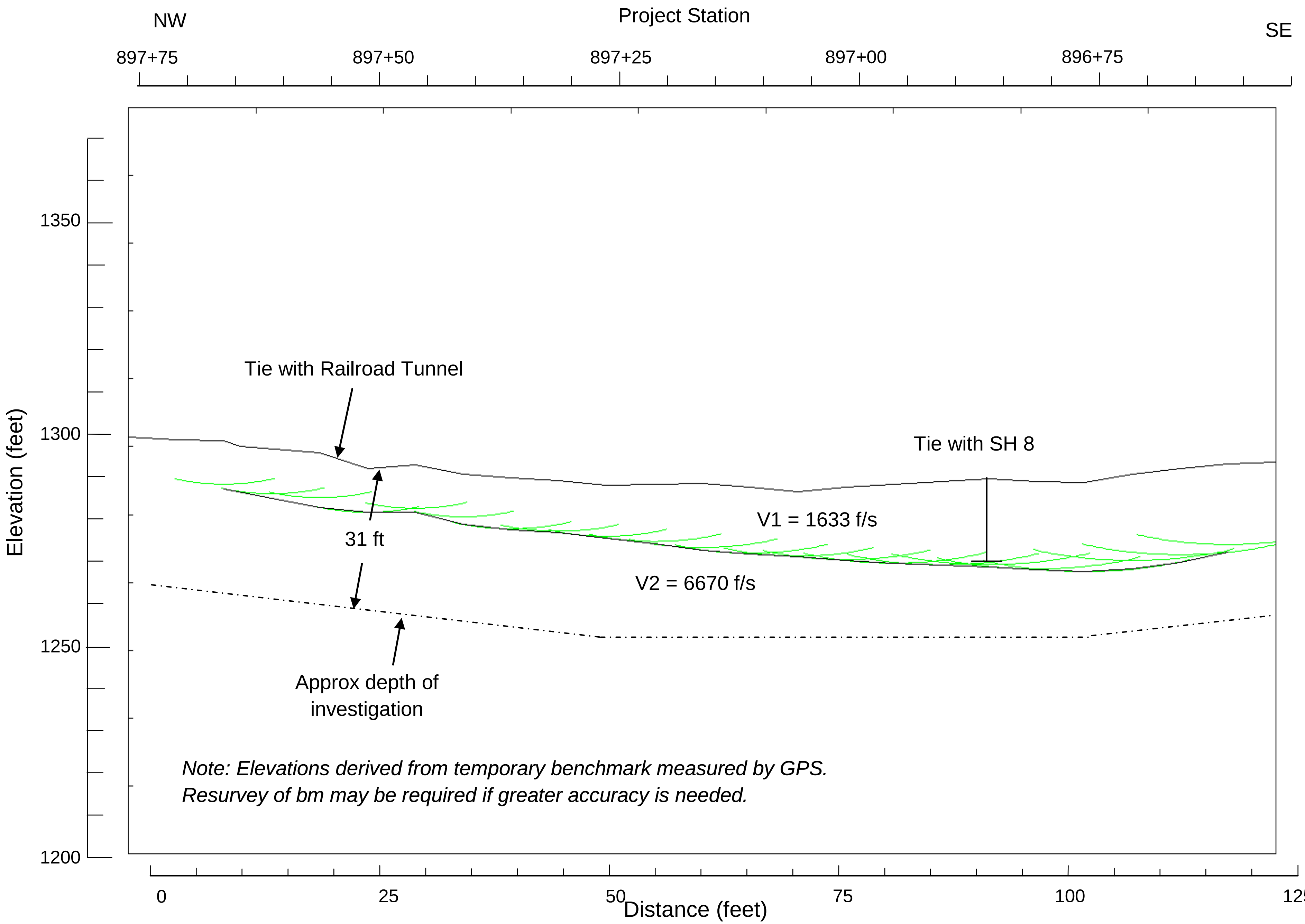
Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No. 7
	0200000016	Seismic Line SH 6 (Construction Road)	



Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.8
	0200000016	Seismic Line SH 7	

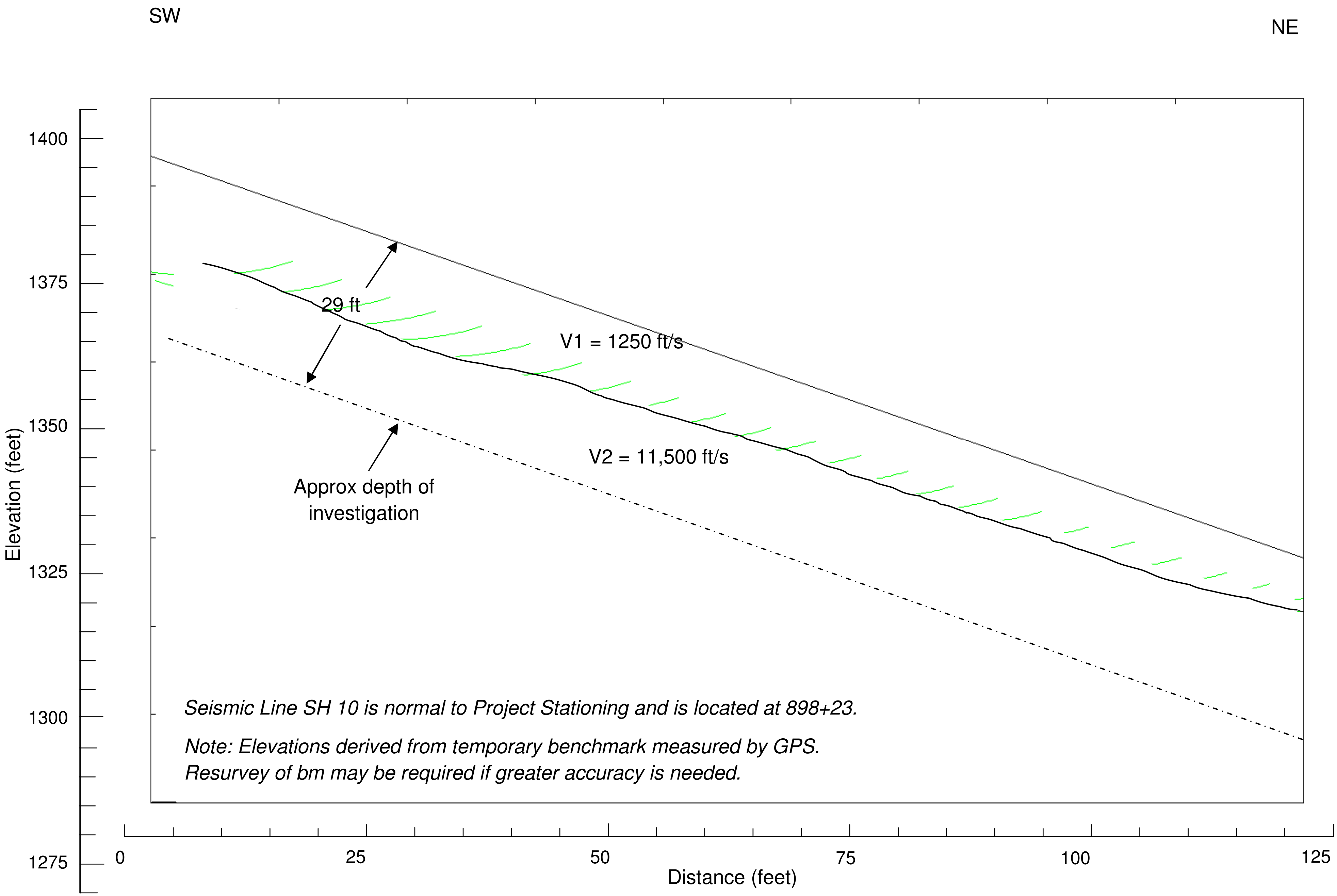


Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.9
	0200000016	Seismic Line SH 8	

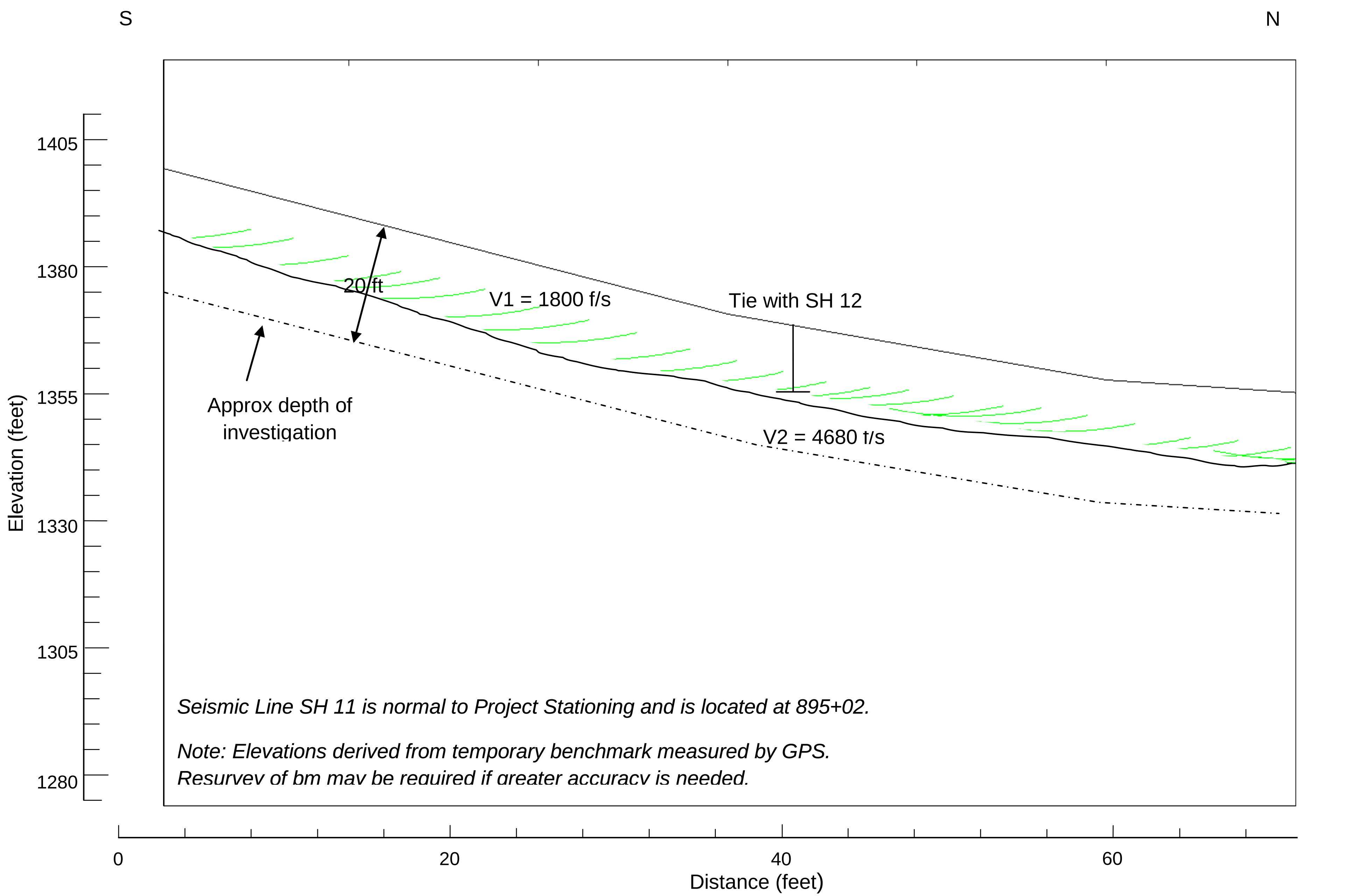


Note: Elevations derived from temporary benchmark measured by GPS.
Resurvey of bm may be required if greater accuracy is needed.

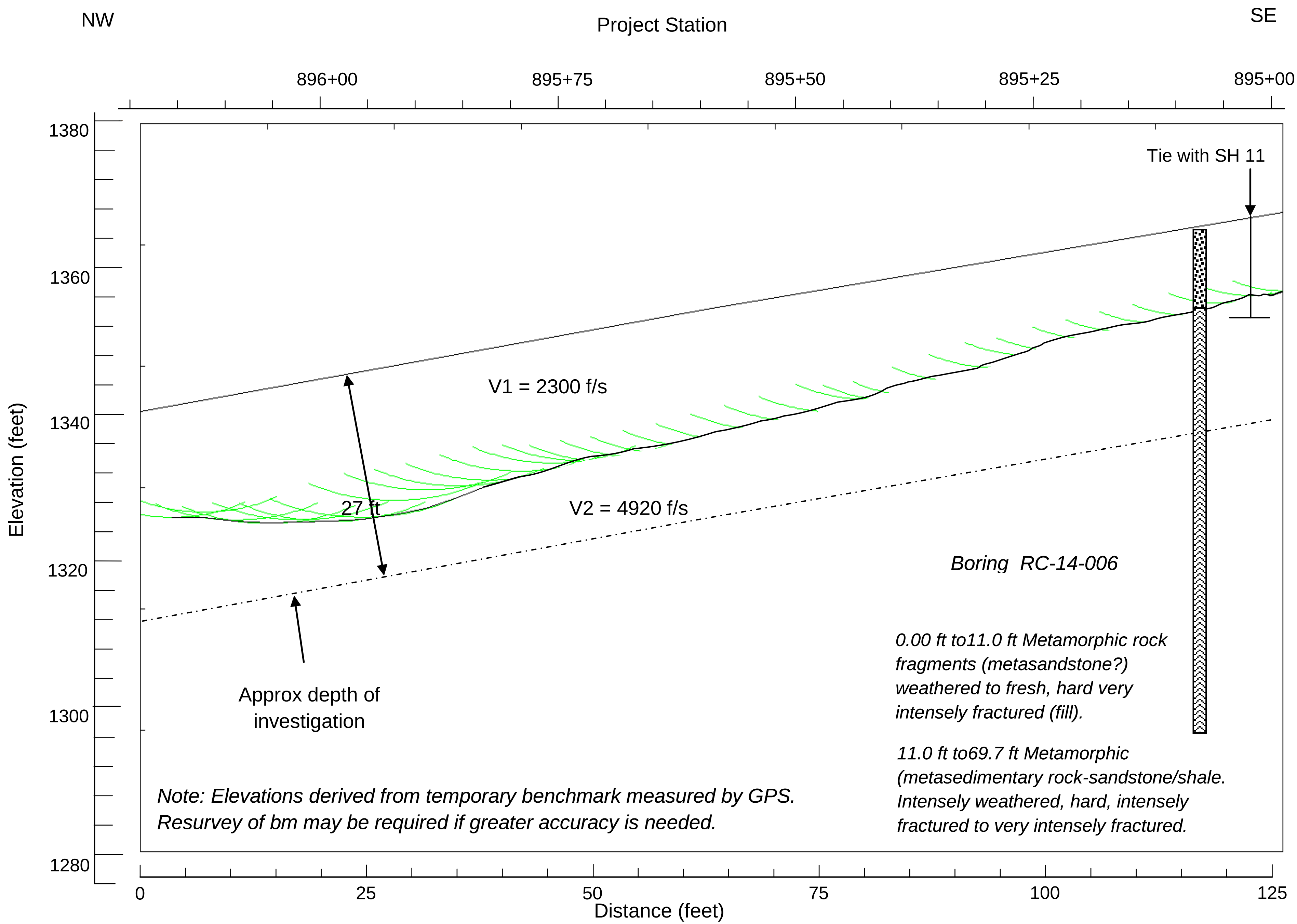
Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.10
	0200000016	Seismic Line SH 9	



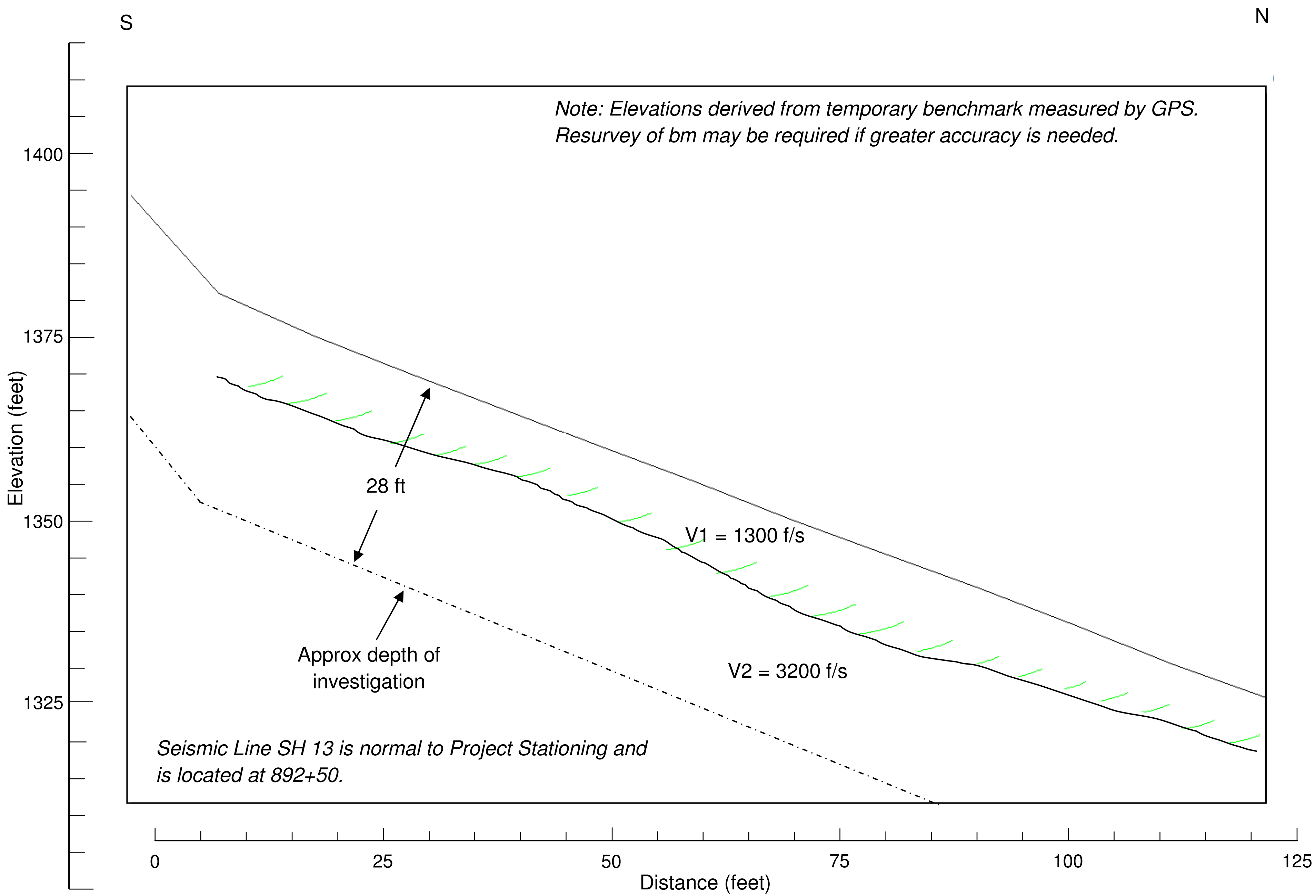
Division of Engineering Services **Office of Geotechnical Support** **Geophysics and Geology Branch**	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.11
	0200000016	Seismic Line SH 10	



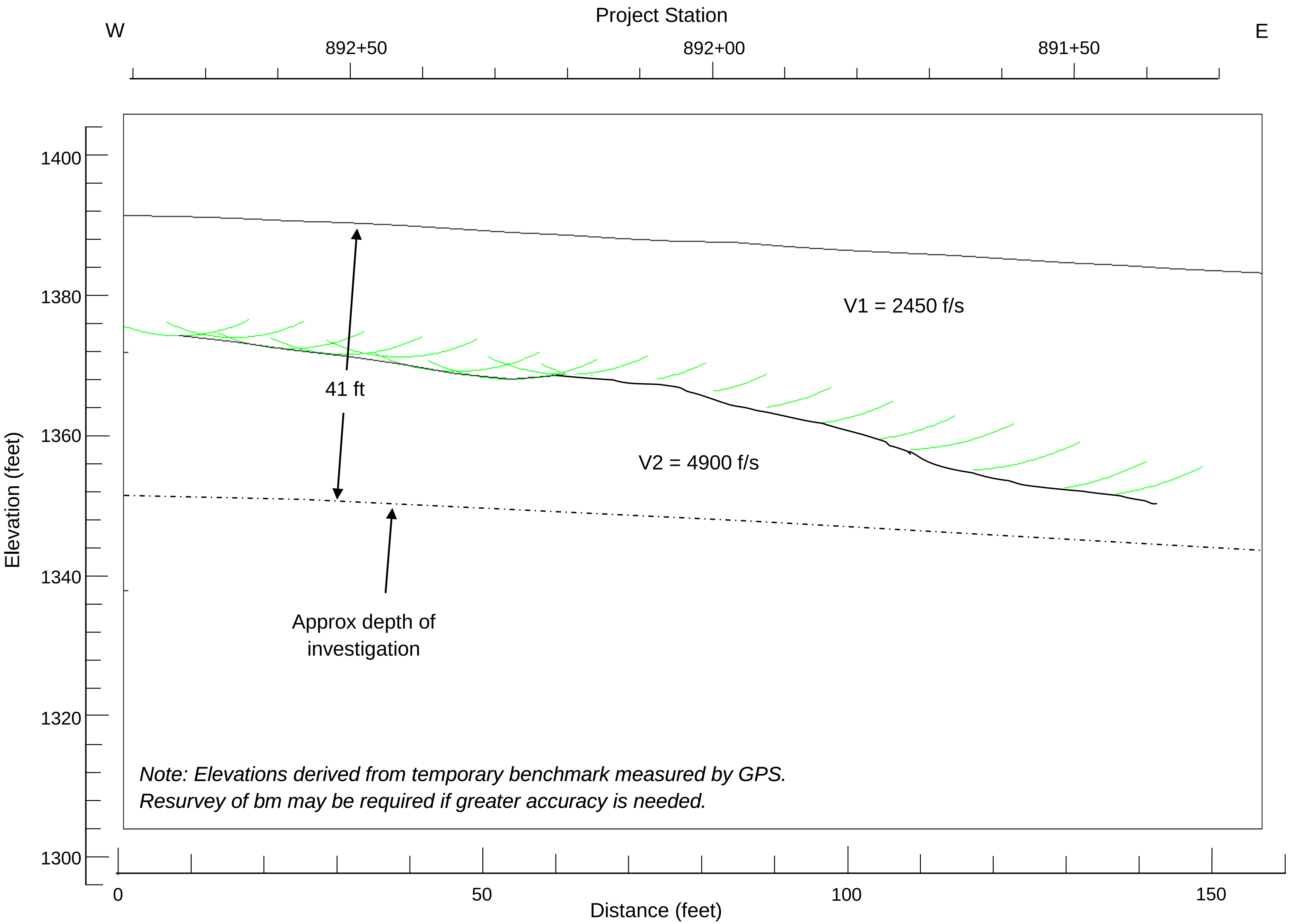
Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.12
	0200000016	Seismic Line SH 11	



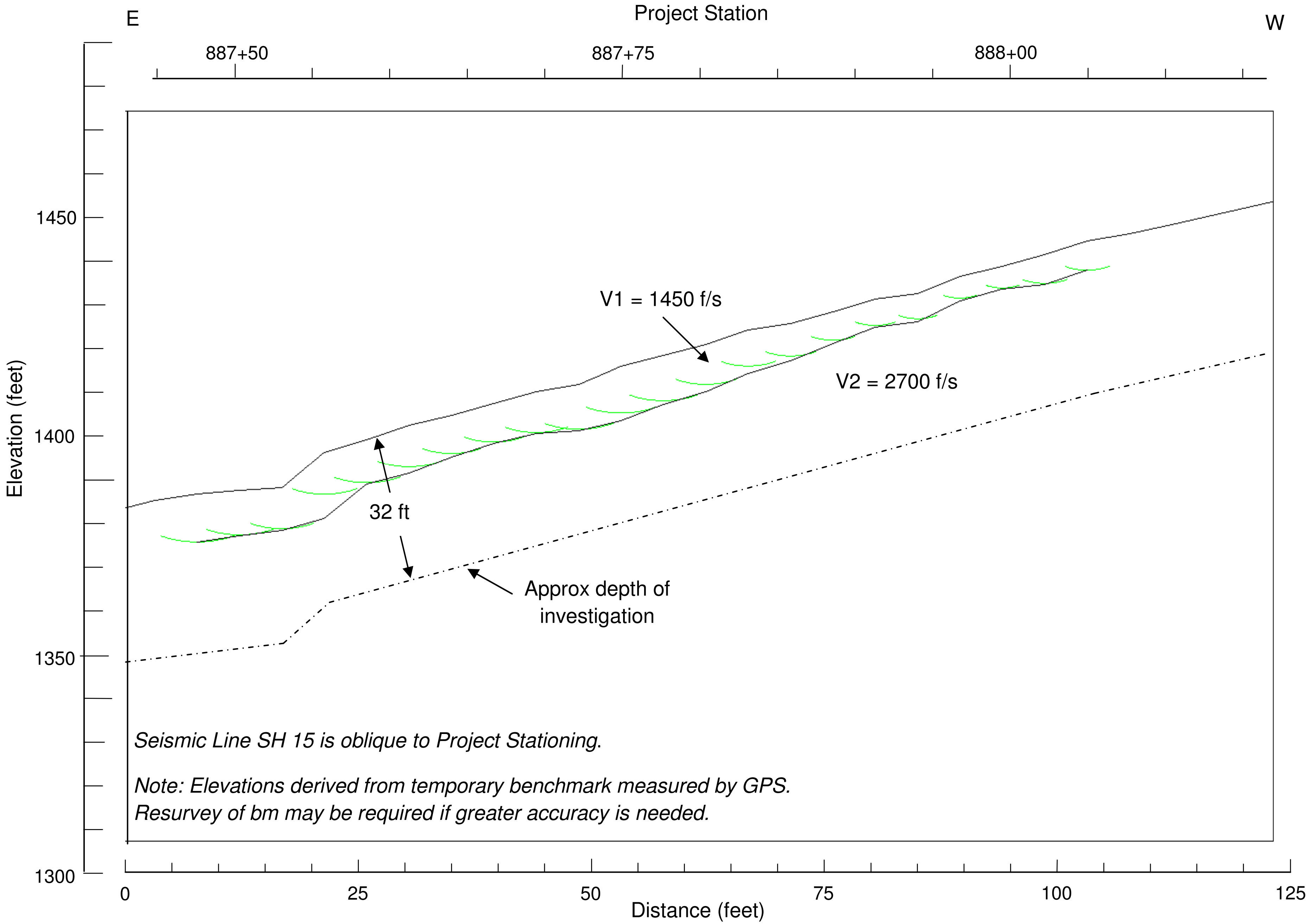
Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.13
	0200000016	Seismic Line SH 12	



Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.14
	0200000016	Seismic Line SH 13	



Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.15
	0200000016	Seismic Line SH 14	



Seismic Line SH 15 is oblique to Project Stationing.

Note: Elevations derived from temporary benchmark measured by GPS.
Resurvey of bm may be required if greater accuracy is needed.

Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project 02-SHA-5-30	Plate No.16
	0200000016	Seismic Line SH 15	

MATERIALS INFORMATION

Laboratory Unconfined Compressive Strength and Point Load Test Data, Dated April 16, 2015

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

Date: 4/16/2015

To: Xing Zheng / GDN

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

RE: Laboratory Test Report -- EA: 02-0E0901
Project: 0200000016
GL 14-070

Final test results. (Revised)

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

AASHTO R18

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL SUPPORT
GEOTECHNICAL LABORATORY

UNCONFINED COMPRESSION TEST SUMMARY (ASTM D7012 Method C)

JOB LOCATION **02-SHA-5** **PM 29.72** GL No. **14-070** DATE **3/9/2015**

TEST BY **KSH**

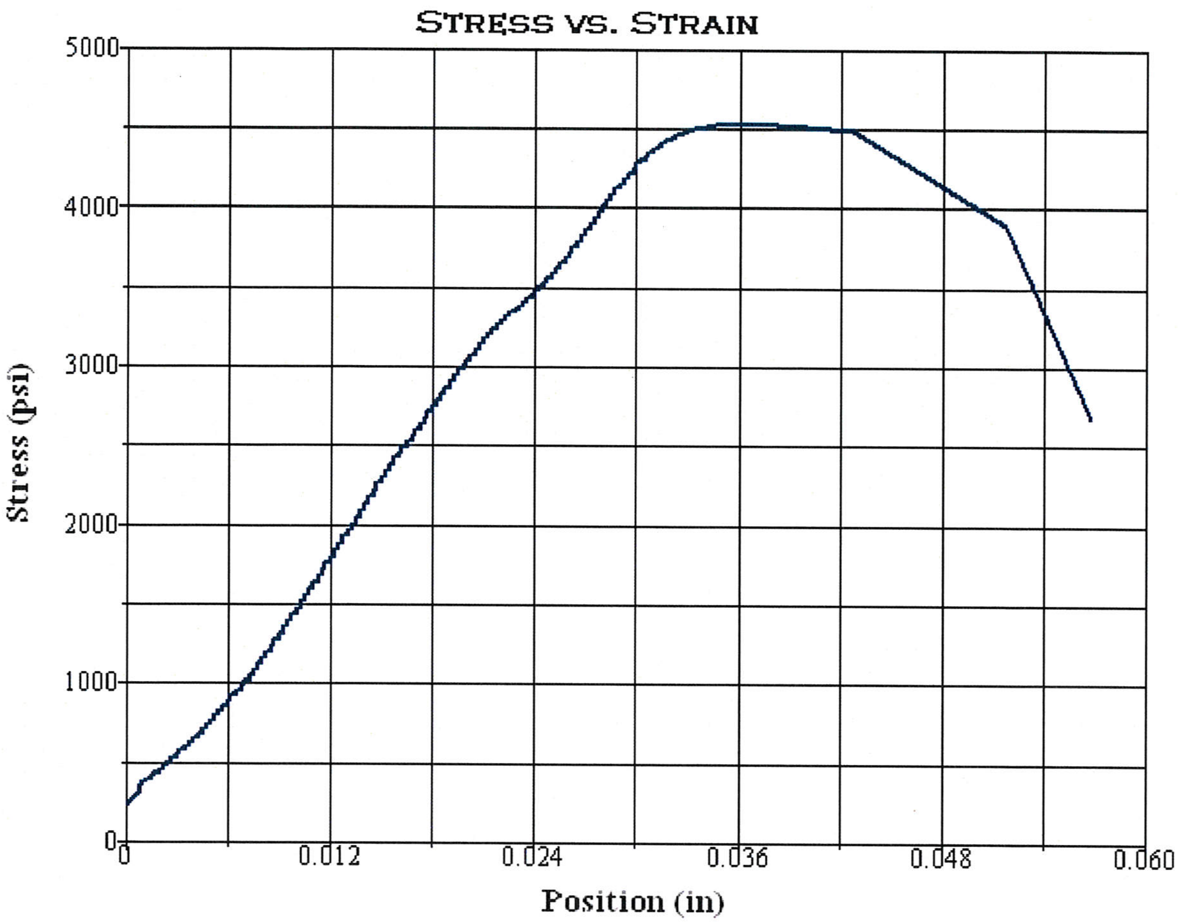
JOB NUMBER **02-0E0901** **Shasta Viaduct** Bridge No. **06-0212L** CHECKED BY **LP 3/10/15**

SAMPLE NO.	DEPTH FT.	DIA. IN.	LENGTH IN.	L/D RATIO	WEIGHT LBS.	LOAD LBS.	DENSITY PCF	STRENGTH PSI	REMARKS
RC-14-004_1404-02	49.0	2.38	3.94	1.65	1.7	20187	164	4527	**
RC-14-004_1404-04	62.0	2.39	5.65	2.36	2.5	46826	169	10416	
RC-14-004_1404-05	68.0	2.40	4.92	2.05	2.2	24535	169	5433	
RC-14-004_1404-08	96.0	2.40	3.58	1.49	1.5	53167	164	11769	**
RC-14-004_1404-09	99.0	2.40	5.72	2.39	2.5	27189	167	6023	
RC-14-004_1404-10	105.5	2.41	3.94	1.63	1.7	15919	167	3489	**
RC-14-004_1404-11	113.5	2.41	4.23	1.76	1.9	18475	169	4053	**
RC-14-004_1404-12	120.0	2.41	5.42	2.25	2.4	41346	171	9084	
RC-14-004_1404-13	128.5	2.40	4.07	1.69	1.8	69779	171	15367	**
RC-14-004_1404-15	140.1	2.41	5.69	2.37	2.5	40745	170	8960	
RC-14-005_1405-06	57.5	2.40	5.39	2.24	2.3	44422	165	9795	
RC-14-005_1405-07	66.5	2.40	4.36	1.81	1.9	54210	166	11945	**
RC-14-006_1406-02	65.0	2.27	4.16	1.83	1.7	45717	173	11306	**
RC-14-007_1407-05	57.0	2.39	4.05	1.70	1.8	31250	173	6957	**

Note: No moistures recorded

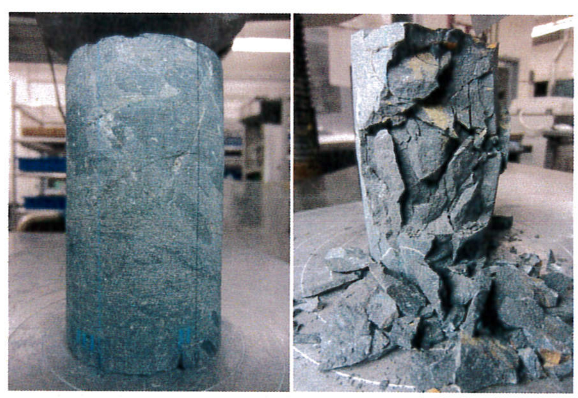
* Sample fell apart while preparing for testing -- Not suitable for testing

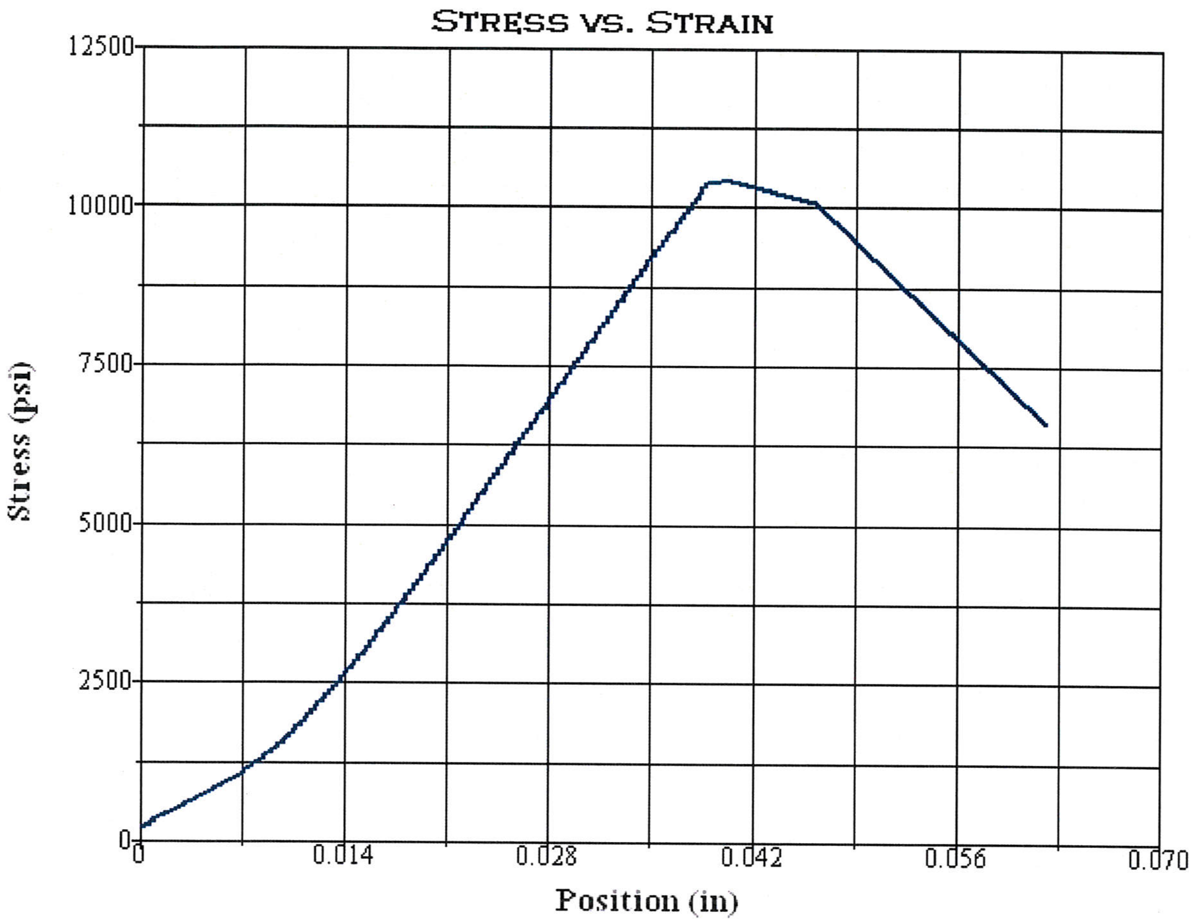
** The test specimen length/diameter ratio was not in compliance with the test method



Test Summary	
Ticket:	Q15-036
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-27.92; 06-0212L
Sample:	RC-14-004_1404-02
Depth:	49'
Procedure Name:	ASTM D7012 Method C
Start Date:	3/8/2015
Elapsed Time:	00:02:01
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metasiltstone
Load Direction:	No visible plane
Non-Conformances:	Flatness, Perpendicularity, Straightness; L/D<2
Moisture Cond:	As-Received; Not Tested
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results	
Specimen Gage Length:	3.9385 in
Diameter:	2.3828 in
Area:	4.4593 in²
Maximum Load:	20187 lbf
Compressive Strength:	4527 psi
Peak Load Rate:	1.31e+005 lbf/min



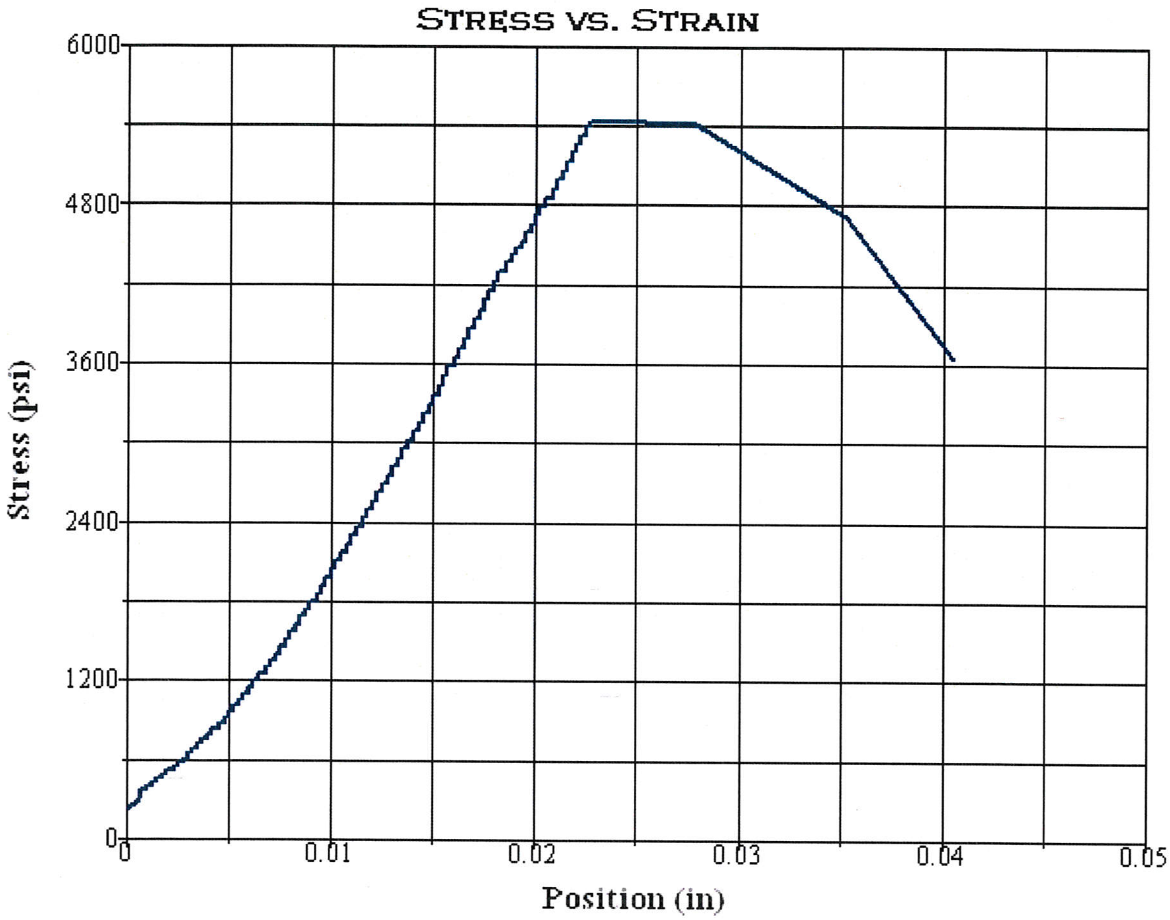


Test Summary

Ticket:	Q15-023
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-004_1404-04
Depth:	62'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/13/2015
Elapsed Time:	00:04:48
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metasiltstone
Load Direction:	At angle to Visual Discontinuity
Non-Conformances:	Flatness; Perpendicularity
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Near vertical through the top of the core; Preexisting healed fractures

Test Results

Specimen Gage Length:	5.6490 in
Diameter:	2.3925 in
Area:	4.4957 in²
Maximum Load:	46826 lbf
Compressive Strength:	10416 psi
Peak Load Rate:	1.43e+005 lbf/min

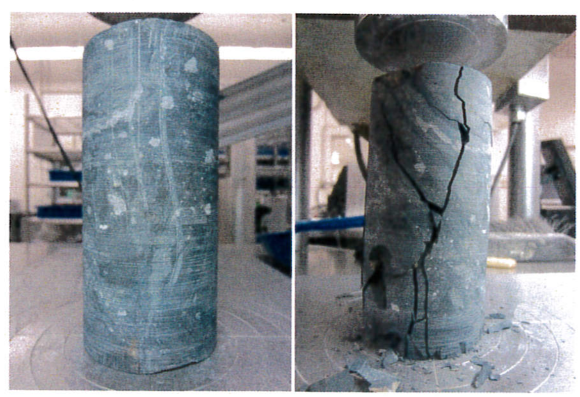


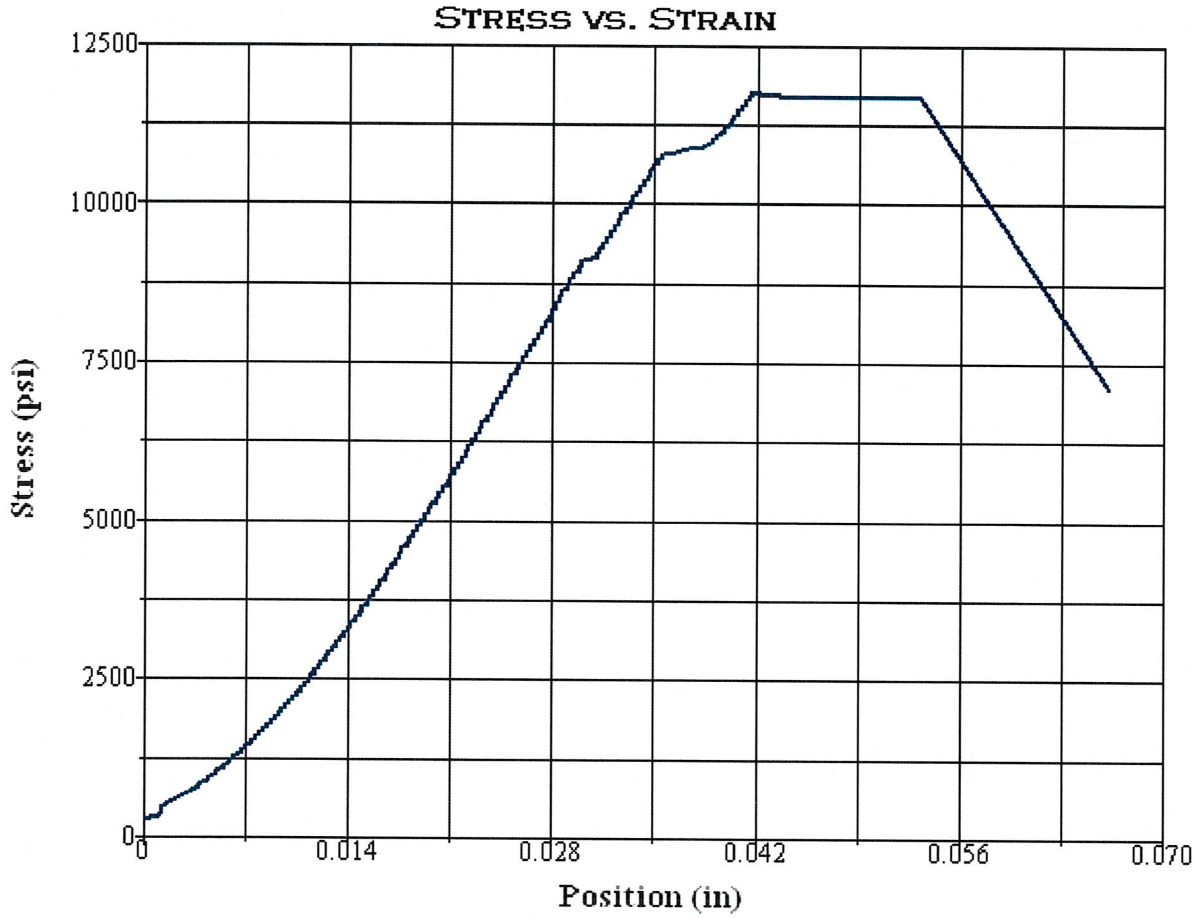
Test Summary

Ticket:	Q15-024
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-004_1404-05
Depth:	68'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/17/2015
Elapsed Time:	00:02:26
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metasiltstone
Load Direction:	At angle to Visual Discontinuity
Non-Conformances:	Flatness; Perpendicularity
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results

Specimen Gage Length:	4.9220 in
Diameter:	2.3978 in
Area:	4.5156 in^2
Maximum Load:	24535 lbf
Compressive Strength:	5433 psi
Peak Load Rate:	1.63e+005 lbf/min



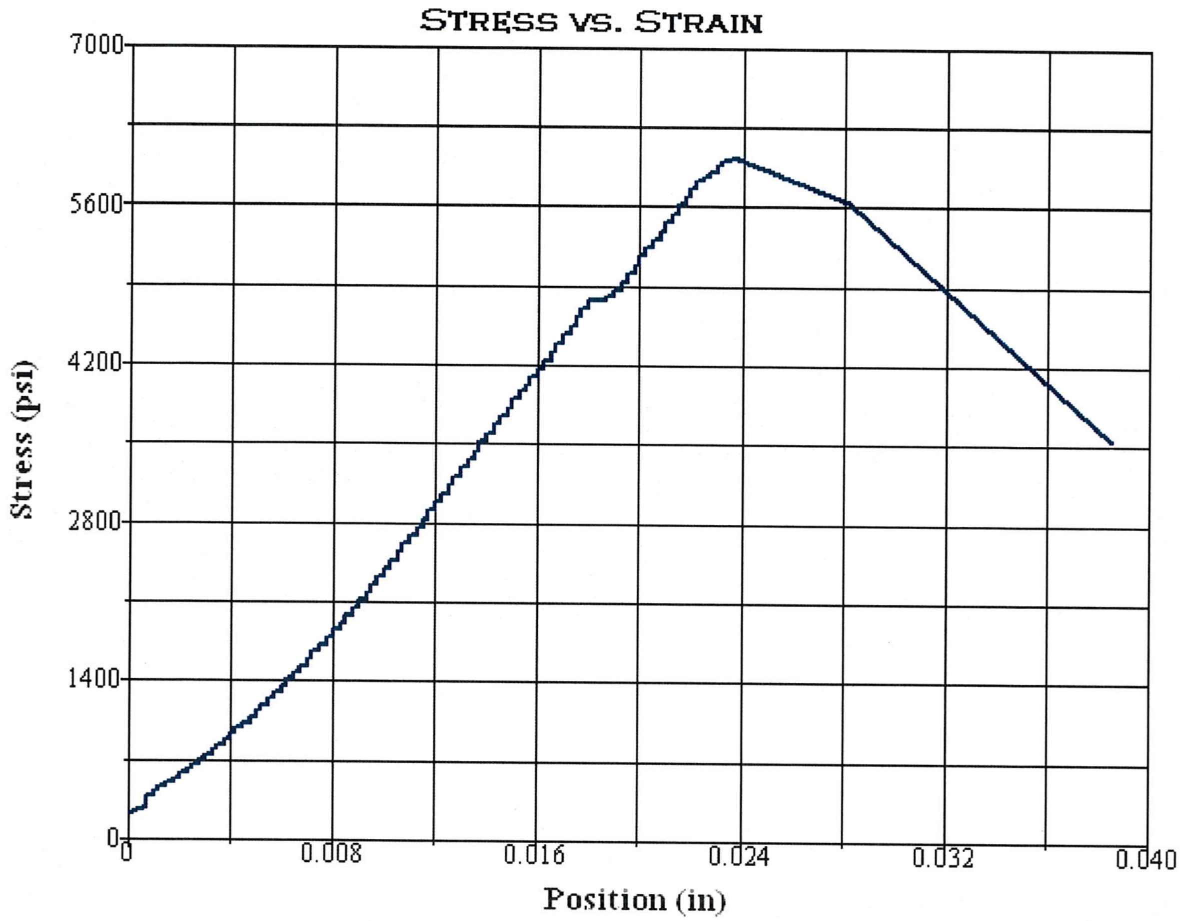


Test Summary

Ticket:	Q15-037
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-27.92; 06-0212L
Sample:	RC-14-004_1404-08
Depth:	96'
Procedure Name:	ASTM D7012 Method C
Start Date:	3/8/2015
Elapsed Time:	00:05:24
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metasandstone/Metashale
Load Direction:	At angle to Visual Discontinuity
Non-Conformances:	Flatness, Perpendicularity; L/D<2
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results

Specimen Gage Length:	3.5750 in
Diameter:	2.3983 in
Area:	4.5175 in²
Maximum Load:	53167 lbf
Compressive Strength:	11769 psi
Peak Load Rate:	1.88e+005 lbf/min

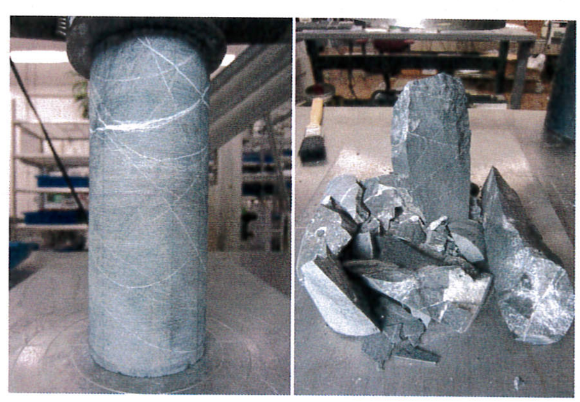


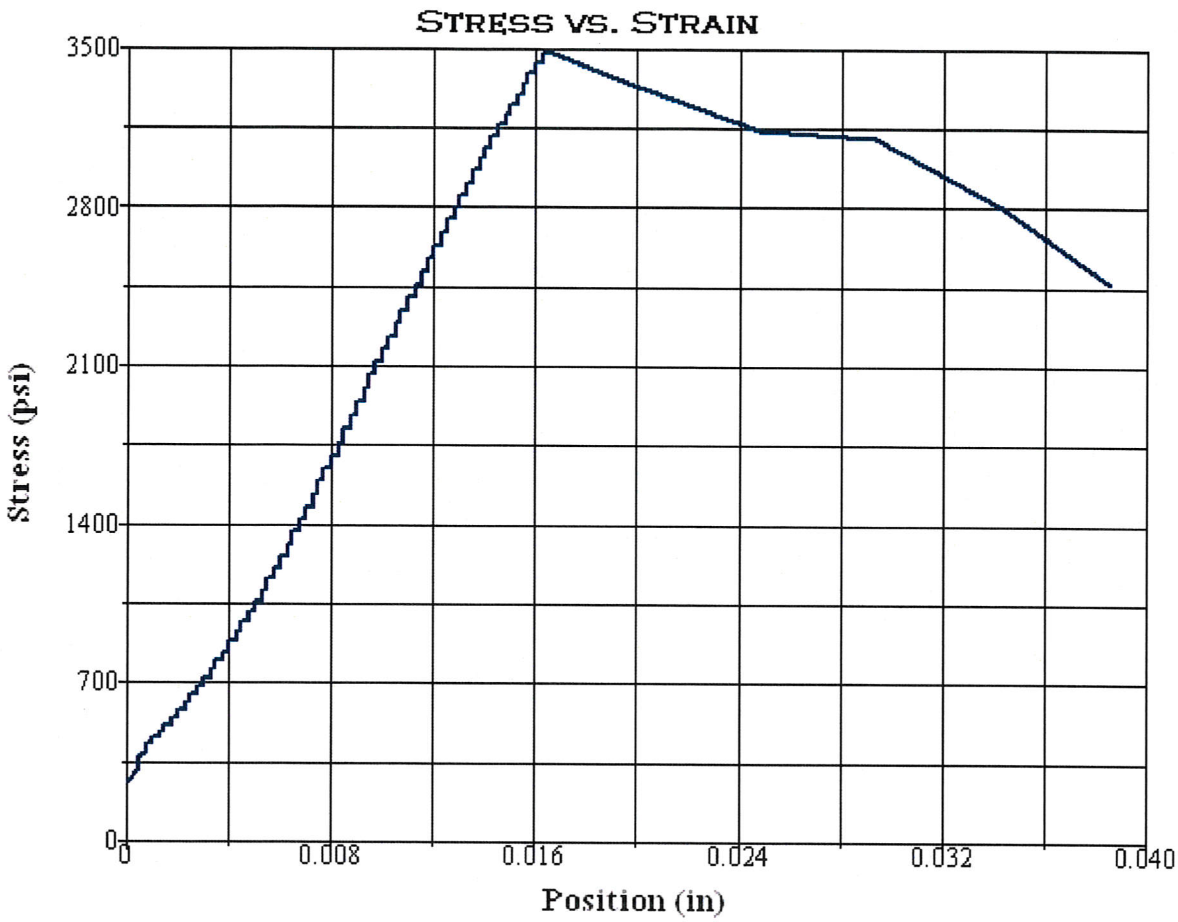
Test Summary

Ticket:	Q15-025
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-004_1404-09
Depth:	99'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/17/2015
Elapsed Time:	00:02:41
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metasiltstone
Load Direction:	45 deg to discontinuities
Non-Conformances:	Flatness; Perpendicularity
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results

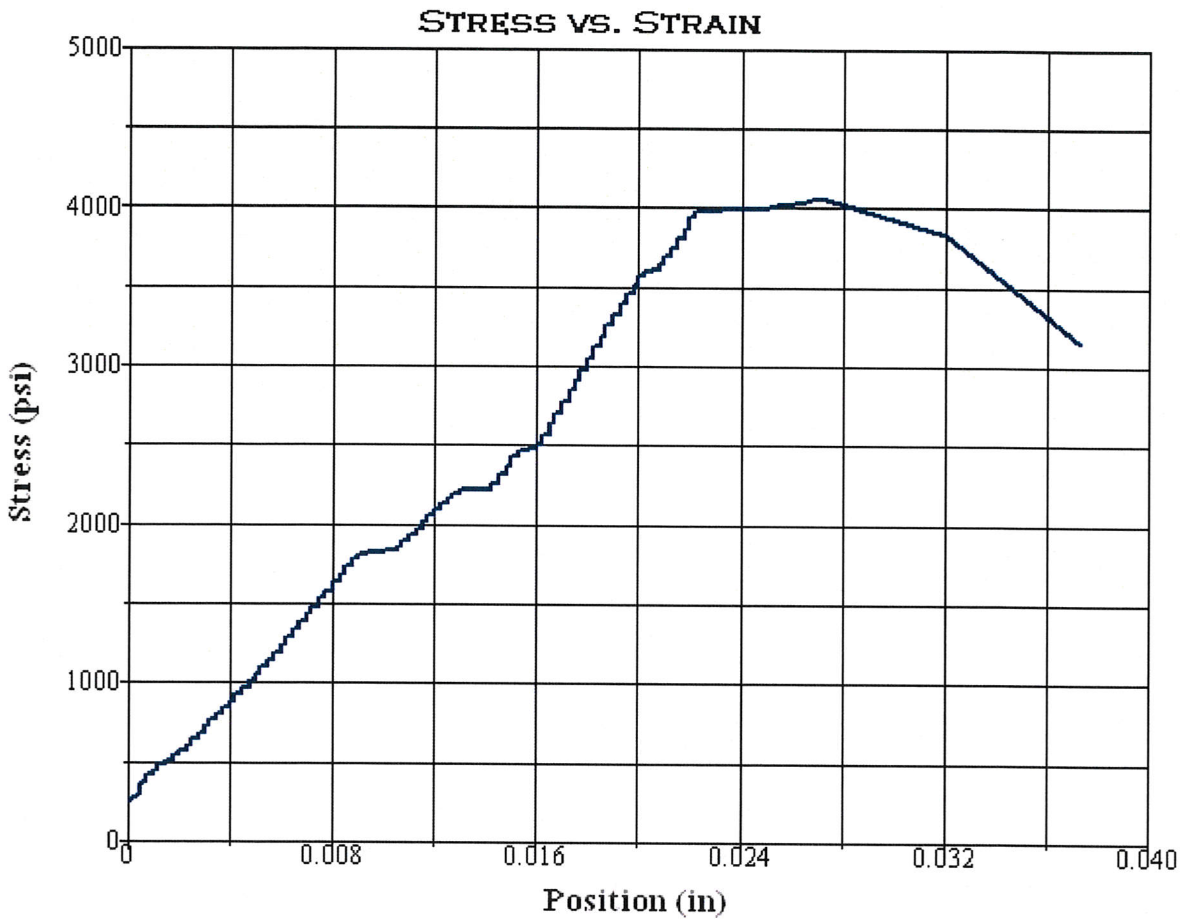
Specimen Gage Length:	5.7220 in
Diameter:	2.3975 in
Area:	4.5145 in^2
Maximum Load:	27189 lbf
Compressive Strength:	6023 psi
Peak Load Rate:	1.84e+005 lbf/min





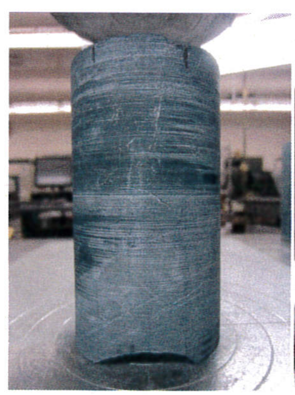
Test Summary	
Ticket:	Q15-026
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-004_1404-10
Depth:	105.5'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/17/2015
Elapsed Time:	00:01:28
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metashale
Load Direction:	45deg to planes, healed fractures
Non-Conformances:	Flatness; Perpendicularity; L/D<2
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures/beddings

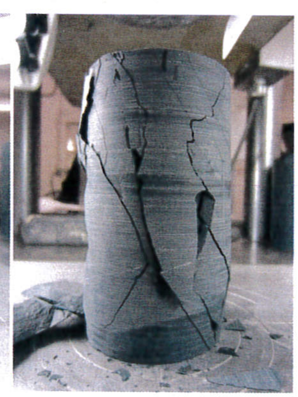
Test Results	
Specimen Gage Length:	3.9355 in
Diameter:	2.4103 in
Area:	4.5628 in²
Maximum Load:	15919 lbf
Compressive Strength:	3489 psi
Peak Load Rate:	1.77e+005 lbf/min

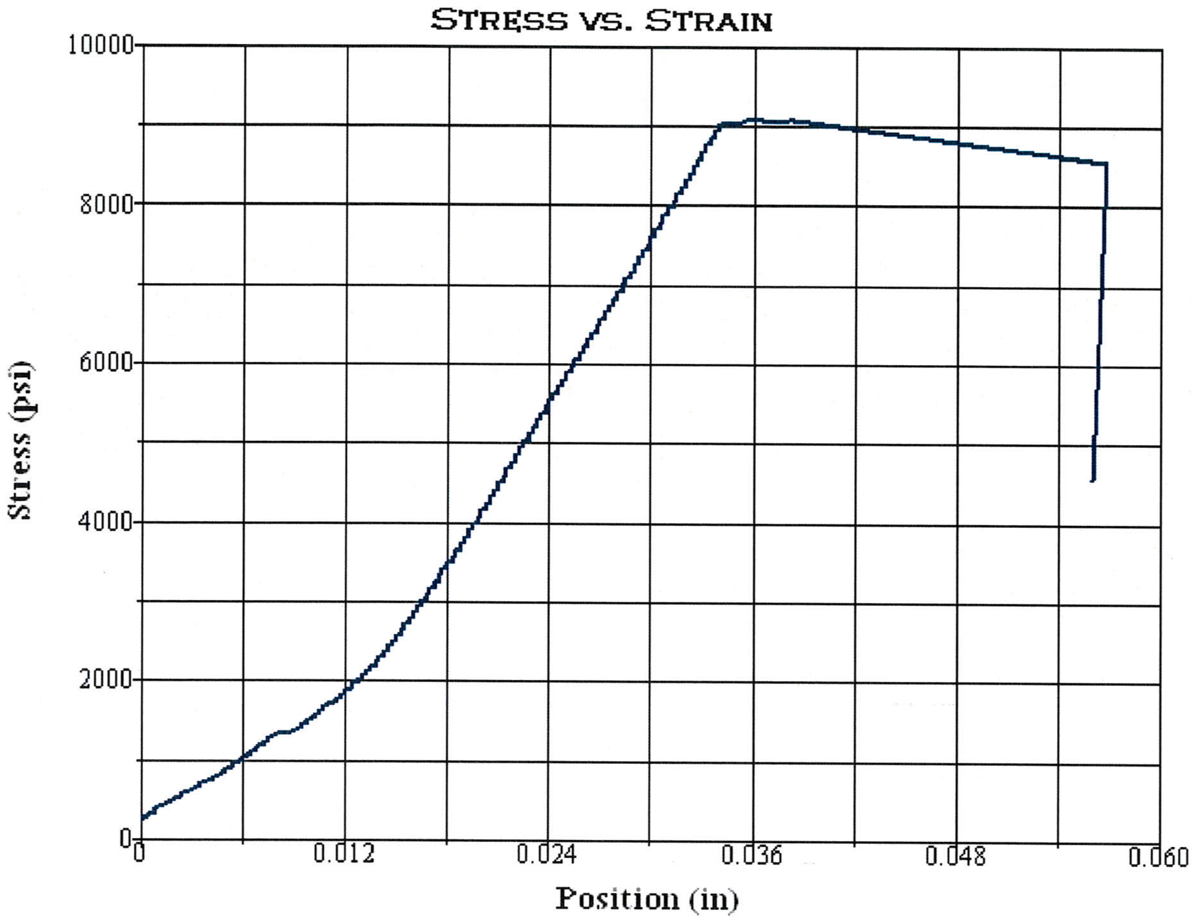



Test Summary	
Ticket:	Q15-027
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-004_1404-11
Depth:	113.5'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/17/2015
Elapsed Time:	00:01:45
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metashale
Load Direction:	Across multiple planes
Non-Conformances:	Flatness; Perpendicularity; L/D<2
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results	
Specimen Gage Length:	4.2295 in
Diameter:	2.4090 in
Area:	4.5579 in^2
Maximum Load:	18475 lbf
Compressive Strength:	4053 psi
Peak Load Rate:	1.74e+005 lbf/min

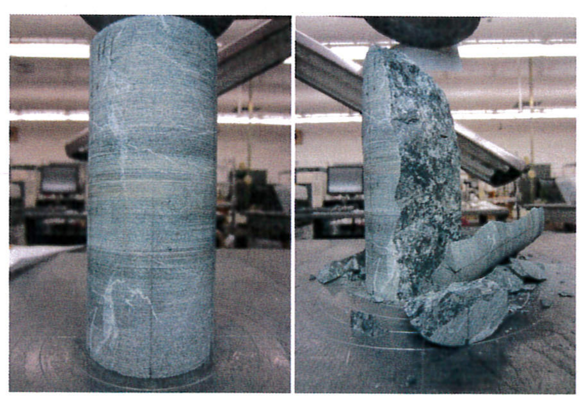


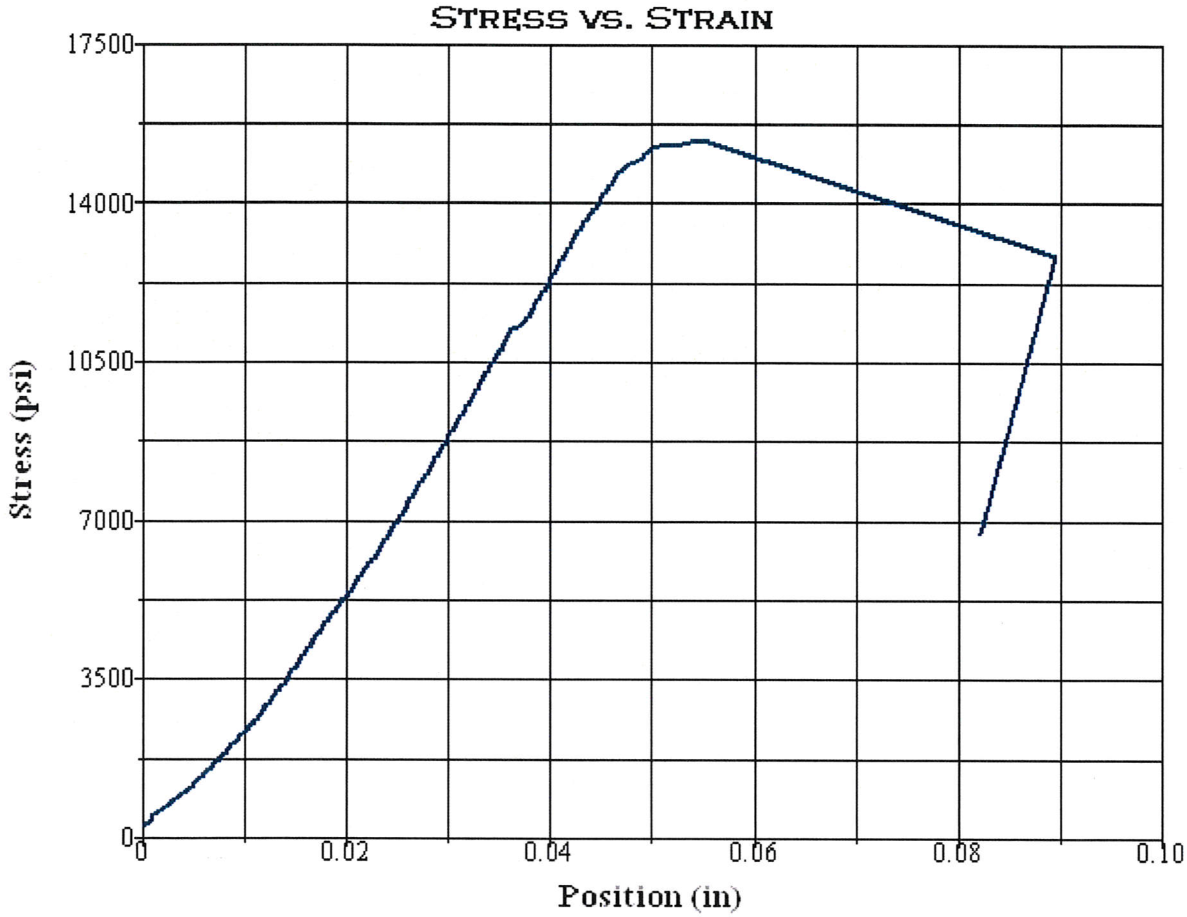




Test Summary	
Ticket:	Q15-028
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-004_1404-12
Depth:	120'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/17/2015
Elapsed Time:	00:04:09
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metasandstone
Load Direction:	Vertical & 45deg to discontinuities
Non-Conformances:	Straightness; Flatness; Perpendicularity
Moisture Cond:	As-Received; Not Tested
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results	
Specimen Gage Length:	5.4215 in
Diameter:	2.4073 in
Area:	4.5515 in²
Maximum Load:	41346 lbf
Compressive Strength:	9084 psi
Peak Load Rate:	1.58e+005 lbf/min



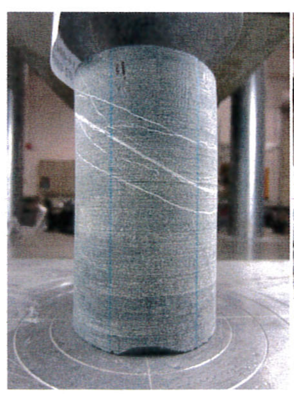


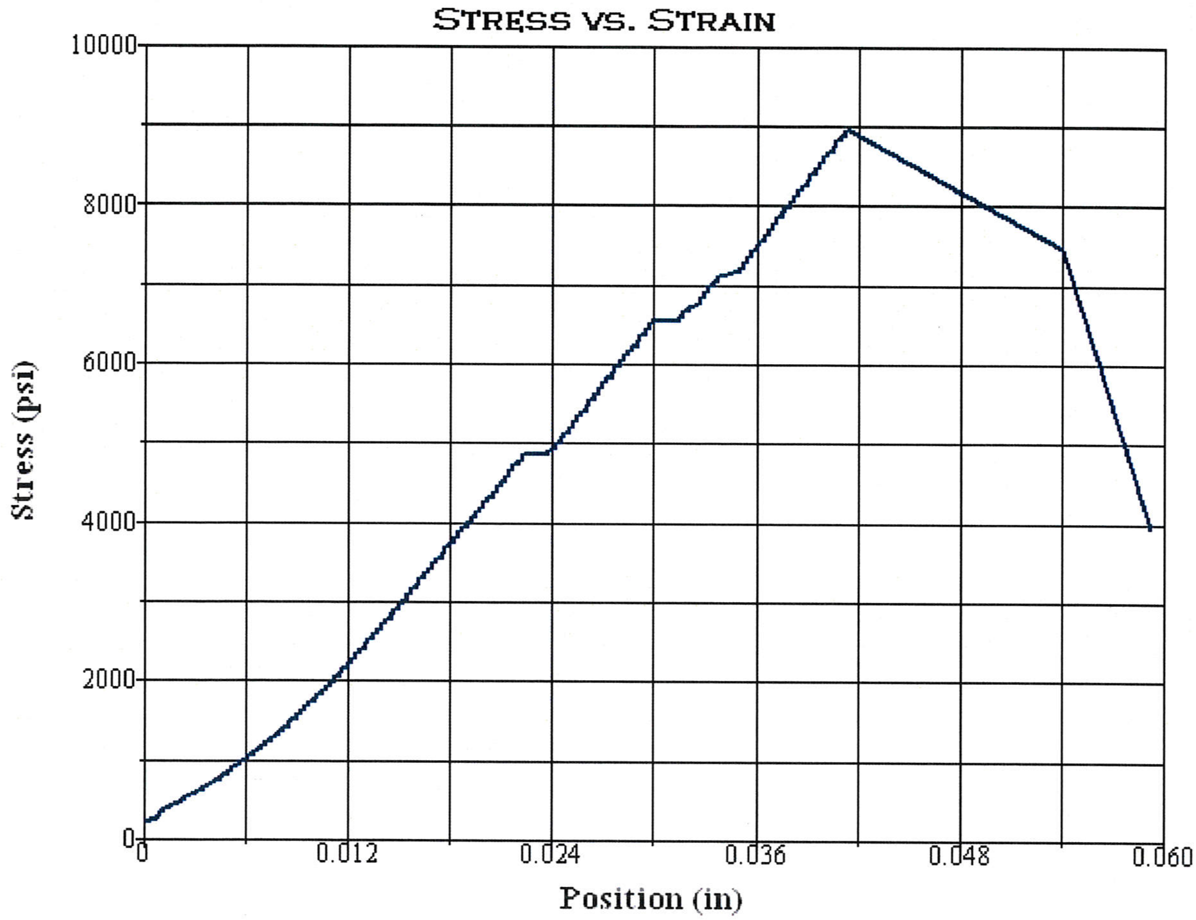
Test Summary

Ticket:	Q15-029
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-004_1404-13
Depth:	128.5'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/18/2015
Elapsed Time:	00:07:05
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metasandstone
Load Direction:	45deg to discontinuities
Non-Conformances:	Straightness, Flatness, Perpendicularity, L/D<2
Moisture Cond:	As-Received; Not Tested
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Near vertical through the top of the core

Test Results

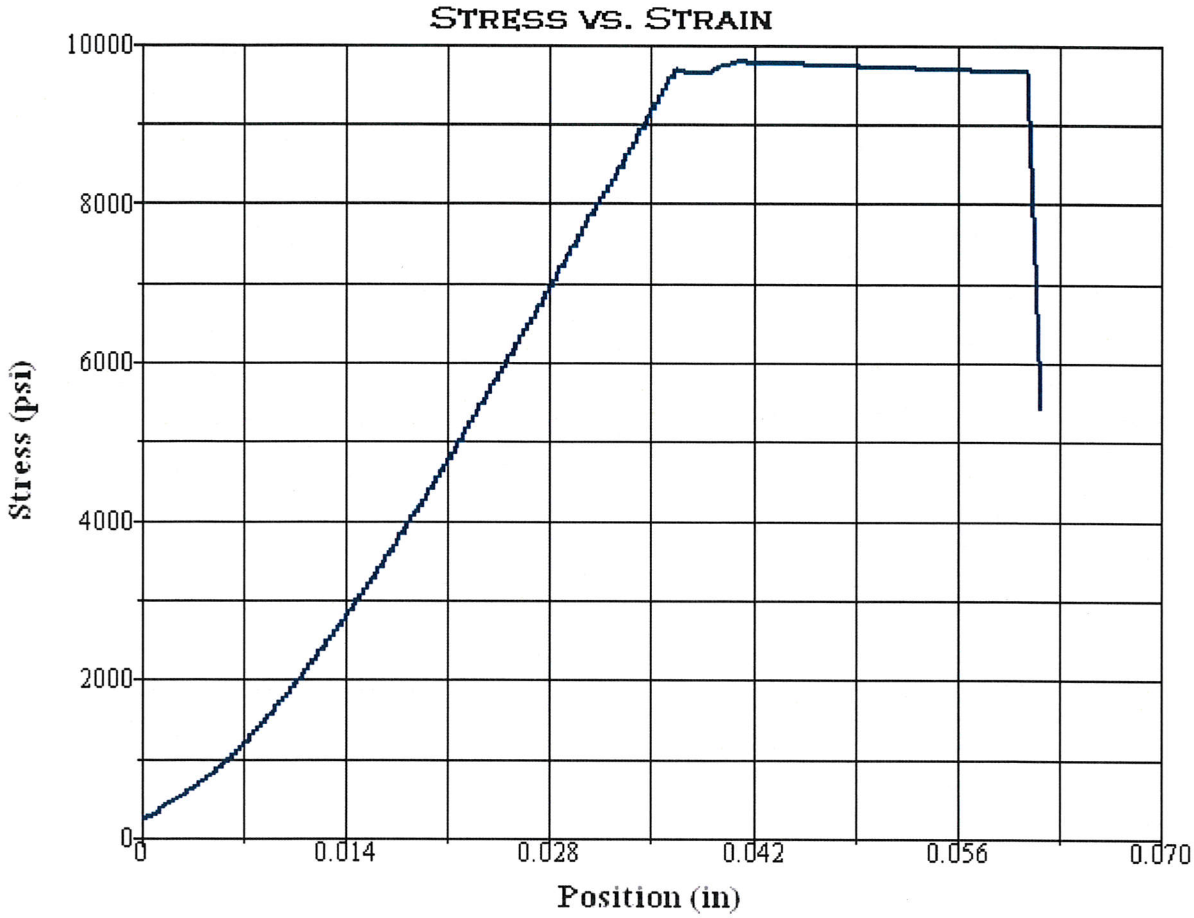
Specimen Gage Length:	4.0660 in
Diameter:	2.4045 in
Area:	4.5409 in²
Maximum Load:	69779 lbf
Compressive Strength:	15367 psi
Peak Load Rate:	2.06e+005 lbf/min





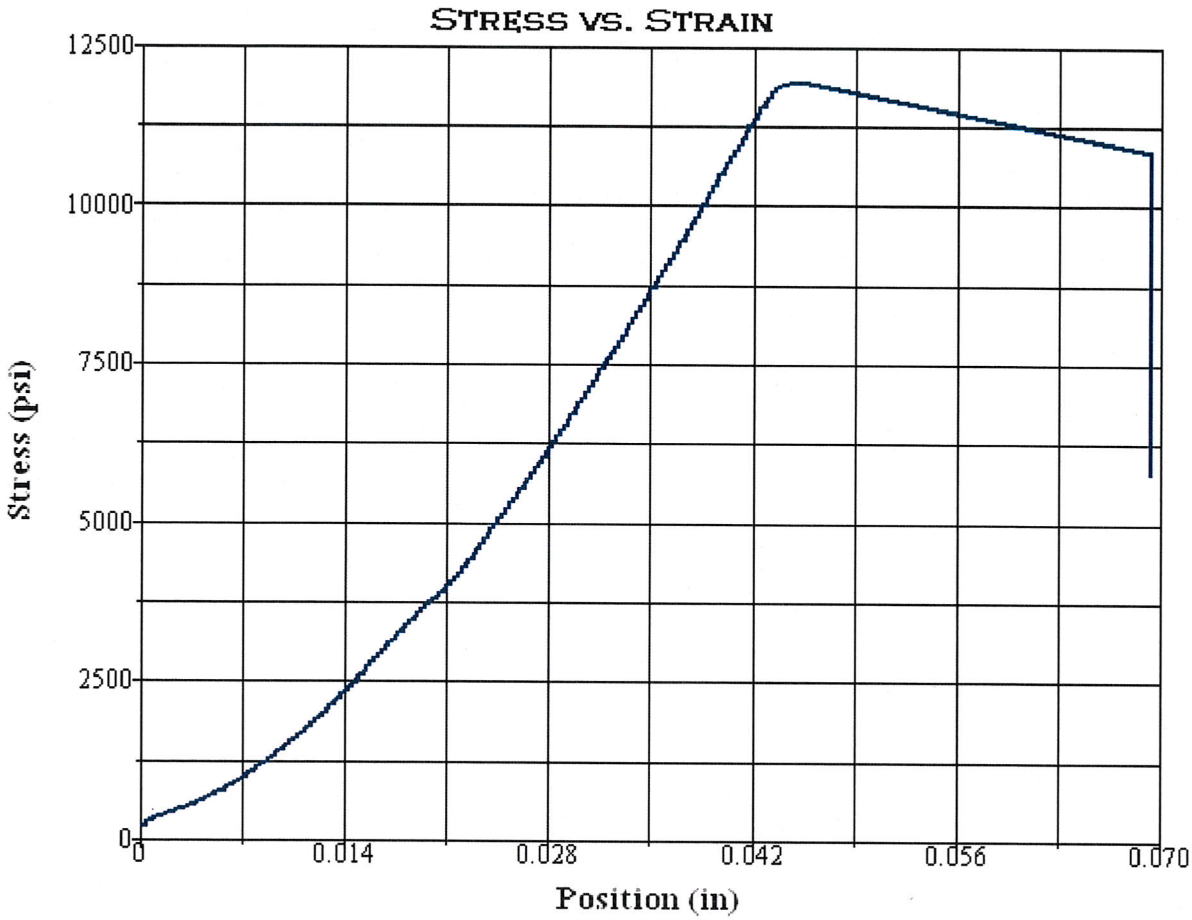
Test Summary	
Ticket:	Q15-030
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-004_1404-15
Depth:	140'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/18/2015
Elapsed Time:	00:04:06
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metashale/Metasiltstone
Load Direction:	45deg & 60deg discontinuities
Non-Conformances:	Flatness, Perpendicularity
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results	
Specimen Gage Length:	5.6935 in
Diameter:	2.4063 in
Area:	4.5477 in^2
Maximum Load:	40745 lbf
Compressive Strength:	8959 psi
Peak Load Rate:	1.43e+005 lbf/min

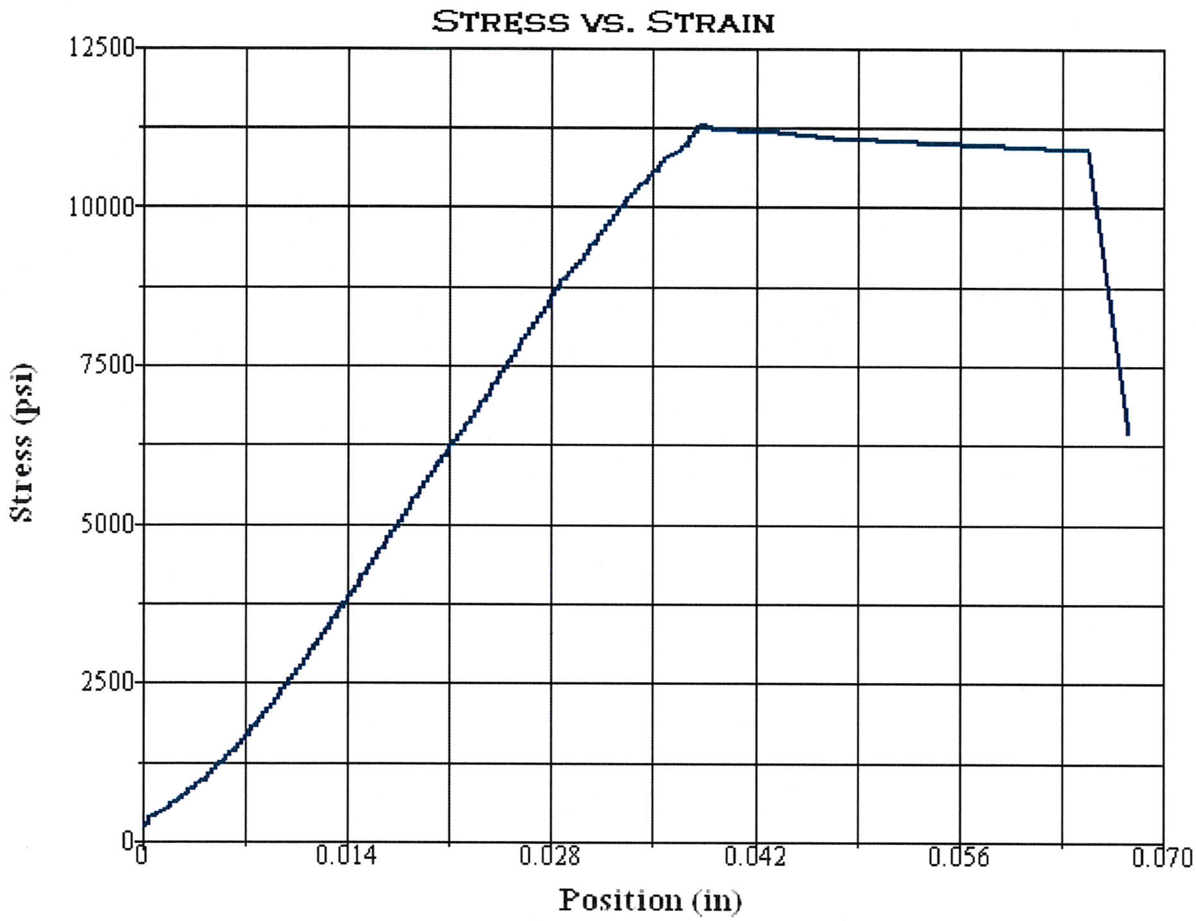
Test Summary	
Ticket:	Q15-031
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-005_1405-06
Depth:	57.5'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/18/2015
Elapsed Time:	00:04:30
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metashale
Load Direction:	Lines in all directions
Non-Conformances:	Flatness, Perpendicularity
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results	
Specimen Gage Length:	5.3900 in
Diameter:	2.4030 in
Area:	4.5352 in²
Maximum Load:	44422 lbf
Compressive Strength:	9795 psi
Peak Load Rate:	1.46e+005 lbf/min



Test Summary	
Ticket:	Q15-032
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-29.72; 06-0212L
Sample:	RC-14-005_1405-07
Depth:	66.5'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/18/2015
Elapsed Time:	00:05:33
Workstation:	D1K00YB1
Operator:	KSH
Description:	Metashale
Load Direction:	Lines in all directions
Non-Conformances:	Flatness, Perpendicularity; L/D<2
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results	
Specimen Gage Length:	4.3550 in
Diameter:	2.4038 in
Area:	4.5382 in²
Maximum Load:	54210 lbf
Compressive Strength:	11945 psi
Peak Load Rate:	1.29e+005 lbf/min

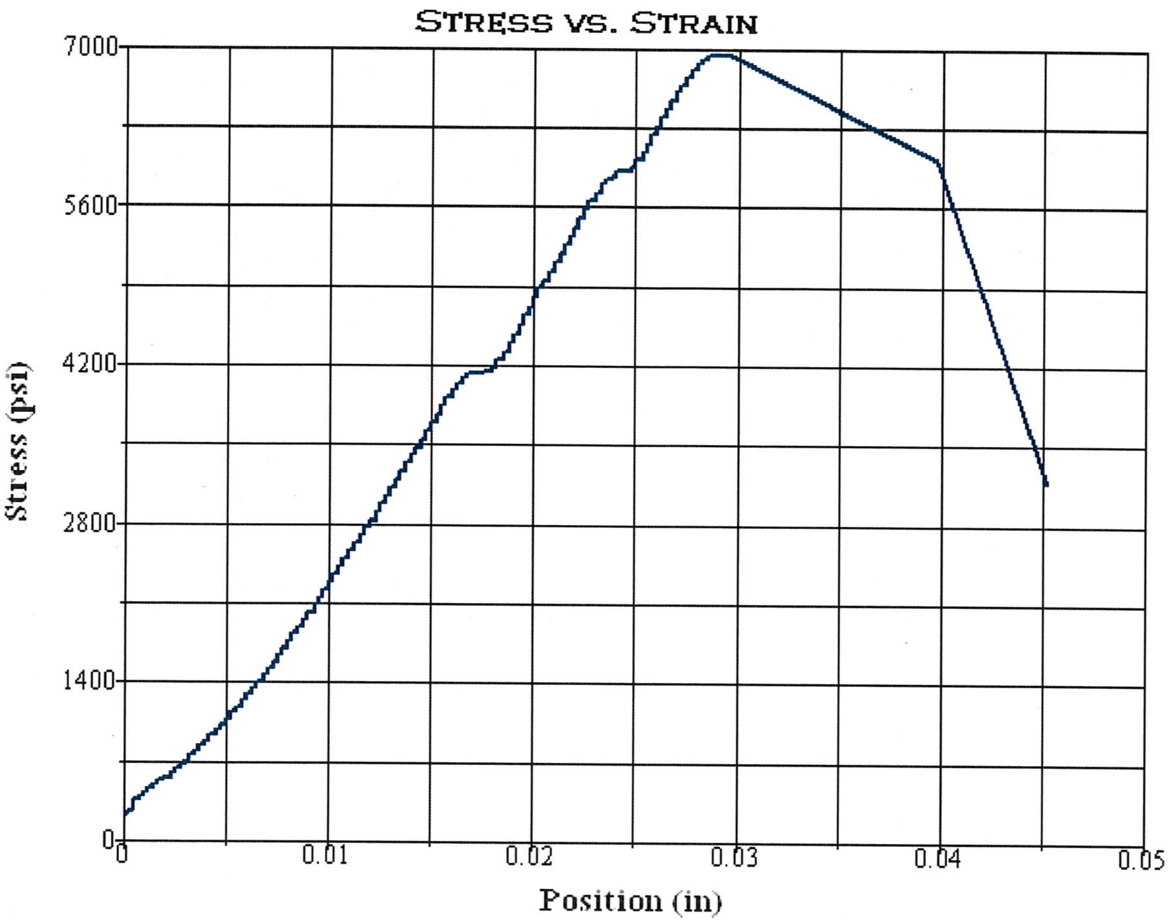


Test Summary

Ticket:	Q15-038
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-27.92; 06-0212L
Sample:	RC-14-006_1406-02
Depth:	65'
Procedure Name:	ASTM D7012 Method C
Start Date:	3/8/2015
Elapsed Time:	00:05:13
Workstation:	D1K00YB1
Operator:	KSH
Description:	METASILTSTONE
Load Direction:	Along Visual Discontinuity
Non-Conformances:	Straightness, Flatness, Perpendicularity; L/D<2
Moisture Cond:	As-Received; Not Tested
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results

Specimen Gage Length:	4.1635 in
Diameter:	2.2690 in
Area:	4.0435 in²
Maximum Load:	45717 lbf
Compressive Strength:	11306 psi
Peak Load Rate:	1.57e+005 lbf/min



Test Summary	
Ticket:	Q15-039
GL #:	14-070
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-27.92; 06-0212L
Sample:	RC-14-007_1407-05
Depth:	57'
Procedure Name:	ASTM D7012 Method C
Start Date:	3/8/2015
Elapsed Time:	00:03:09
Workstation:	D1K00YB1
Operator:	KSH
Description:	METASILTSTONE
Load Direction:	Along Visual Discontinuity
Non-Conformances:	Flatness, Perpendicularity; L/D<2
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results	
Specimen Gage Length:	4.0545 in
Diameter:	2.3915 in
Area:	4.4919 in²
Maximum Load:	31250 lbf
Compressive Strength:	6957 psi
Peak Load Rate:	1.65e+005 lbf/min

Division of Engineering Services
Geotechnical Laboratory

Point Load Strength Index

Dist-EA: 02-0E0901
Dist-Co-Rte-PM: SHA-5-29.72/

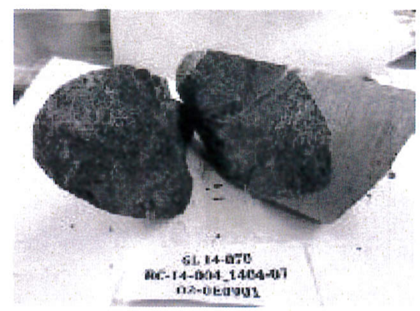
GI Tracking No.: 14-070
Report Date: April 9, 2015

Sample ID	Test Type	Length , L (mm)	Width, W (mm)	Initial Distance Between Contact Points, D(mm)	Final Distance Between Contact Points, D'(mm)	Equivalent Diameter, De (mm)	Failure Load, P (lbs)	Uncorrected Point Load Strength Index Is (psi)	Point Load Strength Index Is (50) (psi)	Remarks
RC-14-004_1404-01-42'	D-L	30.67		57	54	55.48	633.6	132.8	139	
RC-14-004_1404-03-55.5'	A-L		61	35.5	30.5	48.67	1392.16	379.16	375	
RC-14-004_1404-06-74.5'	D-L	50.94		60	55.5	57.71	3555.2	688.79	735	
RC-14-004_1404-07-86'	D-L	32.56		57	54	55.48	4275.04	896.06	939	

RC-14-004_1404-01-42'

RC-14-004_1404-03-55.5'


RC-14-004_1404-06-74.5'


RC-14-004_1404-07-86'

Test Type Abbreviations: D- Diametral, A - Axial, B - Block, I - Irregular Lump

Orientation of Load Direction (if anisotropic): P - Perpendicular to plane of weakness, L - Parallel to plane of weakness

Division of Engineering Services
Geotechnical Laboratory

Point Load Strength Index

Dist-EA: 02-0E0901
Dist-Co-Rte-PM: SHA-5-29.72/

GI Tracking No.: 14-070
Report Date: April 9, 2015

Sample ID	Test Type	Length , L (mm)	Width, W (mm)	Initial Distance Between Contact Points, D(mm)	Final Distance Between Contact Points, D'(mm)	Equivalent Diameter, De (mm)	Failure Load, P (lbs)	Uncorrected Point Load Strength Index Is (psi)	Point Load Strength Index Is (50) (psi)	Remarks
RC-14-004_1404-08-96'	A-P		61.3	43					Invalid test	• Fracture Surface Passes Through Only One Loading Point
RC-14-005_1405-01-15.5'	D-L	33.04		59.5	56	57.72	339.68	65.77	70	
RC-14-005_1405-02-24.7'	I-L	26	47.97	26	22.5	37.07	1195.04	561.03	490	
RC-14-005_1405-03-28.7'	I-L	26	50.25	26	23	38.36	1077.12	472.23	419	

No Image Available

RC-14-004_1404-08-96'

RC-14-005_1405-01-15.5'

RC-14-005_1405-02-24.7'

RC-14-005_1405-03-28.7'

Test Type Abbreviations: D- Diametral, A - Axial, B - Block, I - Irregular Lump

Orientation of Load Direction (if anisotropic): P - Perpendicular to plane of weakness, L - Parallel to plane of weakness

Division of Engineering Services
Geotechnical Laboratory

Point Load Strength Index

Dist-EA: 02-0E0901
Dist-Co-Rte-PM: SHA-5-29.72/

GI Tracking No.: 14-070
Report Date: April 9, 2015

Sample ID	Test Type	Length , L (mm)	Width, W (mm)	Initial Distance Between Contact Points, D(mm)	Final Distance Between Contact Points, D'(mm)	Equivalent Diameter, De (mm)	Failure Load, P (lbs)	Uncorrected Point Load Strength Index Is (psi)	Point Load Strength Index Is (50) (psi)	Remarks
RC-14-005_1405-04-38.8'	I	49	60.82	49					Invalid test	• Fracture Surface Passes Through Only One Loading Point
RC-14-005_1405-05-49'	I-L	32	61.4	32	30	48.43	45.76	12.59	12	
RC-14-006_1406-01-43.5'	I-L	35	60	35	31.5	49.06	607.2	162.79	161	
RC-14-007_1407-01-28.9'	D-L	46		58	50.5	54.12	2548.48	561.34	582	

No Image Available

RC-14-005_1405-04-38.8'

RC-14-005_1405-05-49'

RC-14-006_1406-01-43.5'

RC-14-007_1407-01-28.9'

Test Type Abbreviations: D- Diametral, A - Axial, B - Block, I - Irregular Lump

Orientation of Load Direction (if anisotropic): P - Perpendicular to plane of weakness, L - Parallel to plane of weakness

Division of Engineering Services
Geotechnical Laboratory

Point Load Strength Index

Dist-EA: 02-0E0901
Dist-Co-Rte-PM: SHA-5-29.72/

GI Tracking No.: 14-070
Report Date: April 9, 2015

Sample ID	Test Type	Length , L (mm)	Width, W (mm)	Initial Distance Between Contact Points, D(mm)	Final Distance Between Contact Points, D'(mm)	Equivalent Diameter, De (mm)	Failure Load, P (lbs)	Uncorrected Point Load Strength Index Is (psi)	Point Load Strength Index Is (50) (psi)	Remarks
RC-14-007_1407-02-38.3'	I	40	53.2	40					Invalid test	• Fracture Surface Passes Through Only One Loading Point
RC-14-007_1407-03-45.7'	D	45.24		57.5					Invalid test	• Fracture Surface Passes Through Only One Loading Point
RC-14-007_1407-04-51.7'	D-L	32.79		58.5	56	57.24	2472.8	486.98	518	
RC-14-007_1407-06-64'	D-L	42.13		59	56.5	57.74	1562.88	302.48	323	

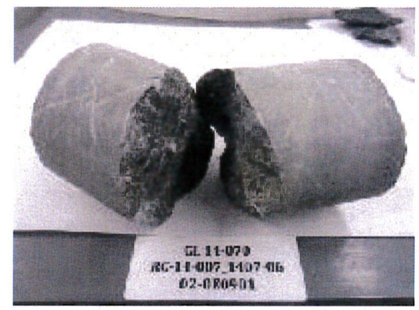
No Image Available

RC-14-007_1407-02-38.3'

No Image Available

RC-14-007_1407-03-45.7'

RC-14-007_1407-04-51.7'



RC-14-007_1407-06-64'

Test Type Abbreviations: D- Diametral, A - Axial, B - Block, I - Irregular Lump

Orientation of Load Direction (if anisotropic): P - Perpendicular to plane of weakness, L - Parallel to plane of weakness

Division of Engineering Services
Geotechnical Laboratory

Point Load Strength Index

Dist-EA: 02-0E0901
Dist-Co-Rte-PM: SHA-5-29.72/

GI Tracking No.: 14-070
Report Date: April 9, 2015

Sample ID	Test Type	Length , L (mm)	Width, W (mm)	Initial Distance Between Contact Points, D(mm)	Final Distance Between Contact Points, D'(mm)	Equivalent Diameter, De (mm)	Failure Load, P (lbs)	Uncorrected Point Load Strength Index Is (psi)	Point Load Strength Index Is (50) (psi)	Remarks
RC-14-007_1407-07-67'	D-L	35.13		59	56	57.48	2948	575.65	613	

RC-14-007_1407-07-67'

No Image Available

No Image Available

No Image Available

Test Type Abbreviations: D- Diametral, A - Axial, B - Block, I - Irregular Lump

Orientation of Load Direction (if anisotropic): P - Perpendicular to plane of weakness, L - Parallel to plane of weakness

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

Date: 4/15/2015

To: Xing Zheng / GDN

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

RE: Laboratory Test Report -- EA: 02-0E0901
Project: 0200000016
GL 15-012

Final test results. (Revised)

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

AASHTO R18

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL SUPPORT
GEOTECHNICAL LABORATORY

UNCONFINED COMPRESSION TEST RESULTS (ASTM D7012 Method C)

JOB LOCATION **02-SHA-5** **PM 27.92** GL No. **15-012** DATE **2/27/2015**

TEST BY **AZM**

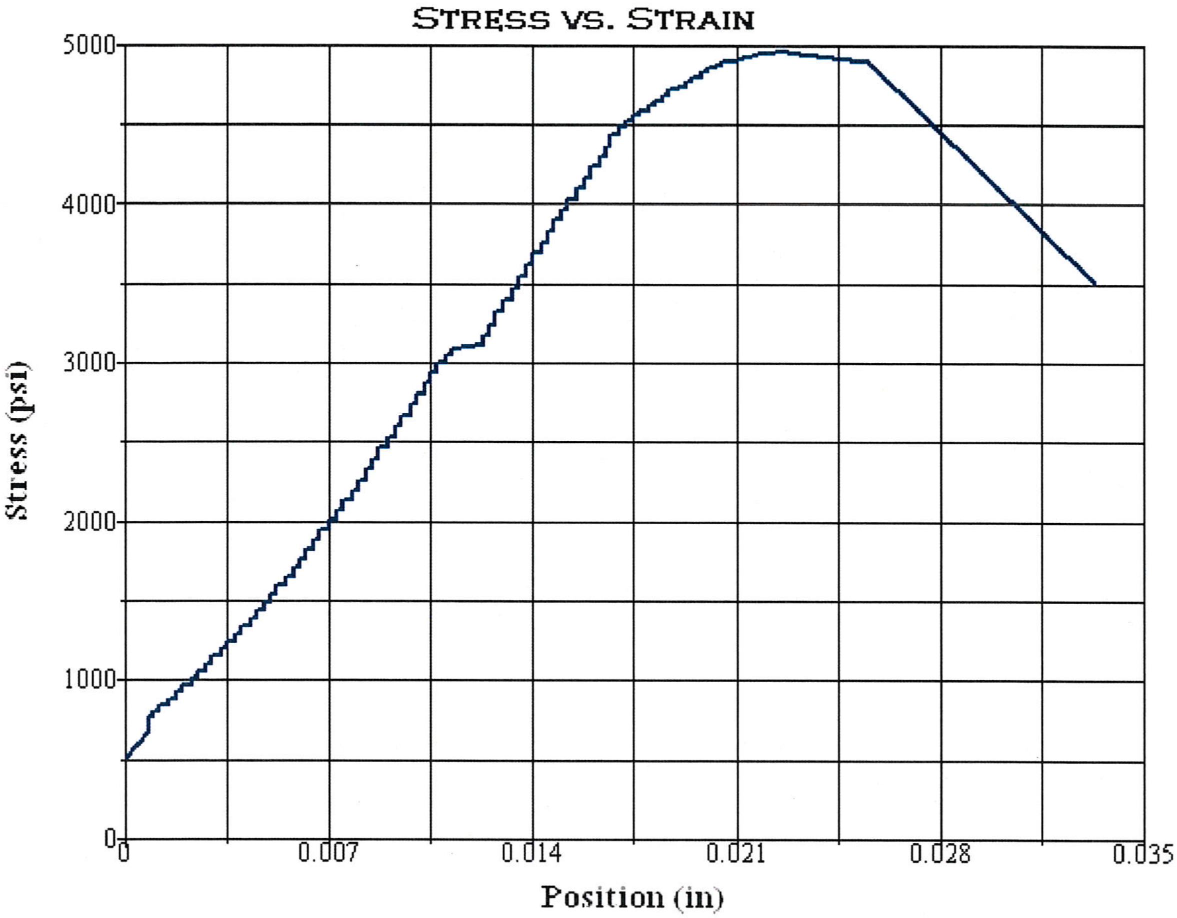
JOB NUMBER **02-0E0901** **Shasta Viaduct** Bridge No. **06-0212L** CHECKED BY **LP 3/3/15**

SAMPLE NO.	DEPTH FT.	DIA. IN.	LENGTH IN.	L/D RATIO	WEIGHT LBS.	LOAD LBS.	DENSITY PCF	STRENGTH PSI	REMARKS
RC-15-001-50.5'	50.5	1.65	3.10	1.88	0.6	10602	161	4958	**
RC-15-004-63-63.6'	63-63.6	1.66	3.40	2.05	0.7	15176	169	7029	
RC-15-006-57.5'	57.5	1.66	2.54	1.53	0.5	22351	167	10377	**

Note: No moistures recorded

* Sample fell apart while preparing for testing -- Not suitable for testing

** The test specimen length/diameter ratio was not in compliance with the test method

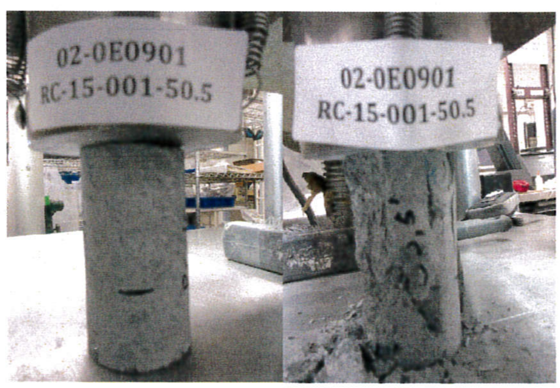


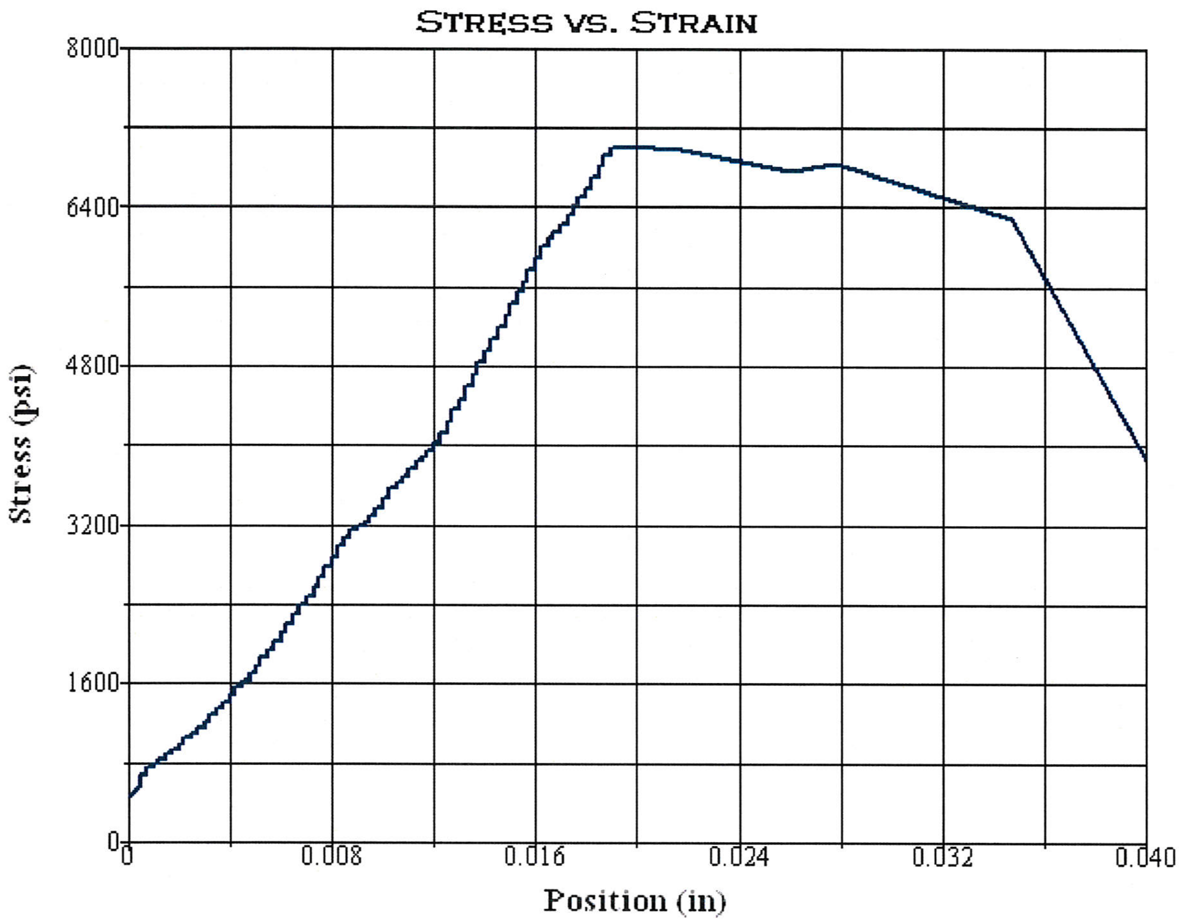
Test Summary

Ticket:	Q15-033
GL #:	15-012
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-27.92; 06-0212L
Sample:	RC-15-001-50.5
Depth:	50.5'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/27/2015
Elapsed Time:	00:02:01
Workstation:	D1K00YB1
Operator:	AZM
Description:	Metabreccia
Load Direction:	No visible plane
Non-Conformances:	Flatness, Perpendicularity; L/D<2
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Near vertical through the top of the core

Test Results

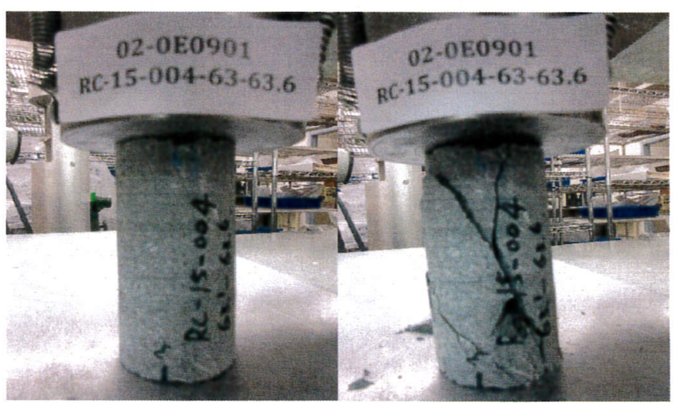
Specimen Gage Length:	3.0970 in
Diameter:	1.6500 in
Area:	2.1382 in²
Maximum Load:	10602 lbf
Compressive Strength:	4958 psi
Peak Load Rate:	1.30e+005 lbf/min

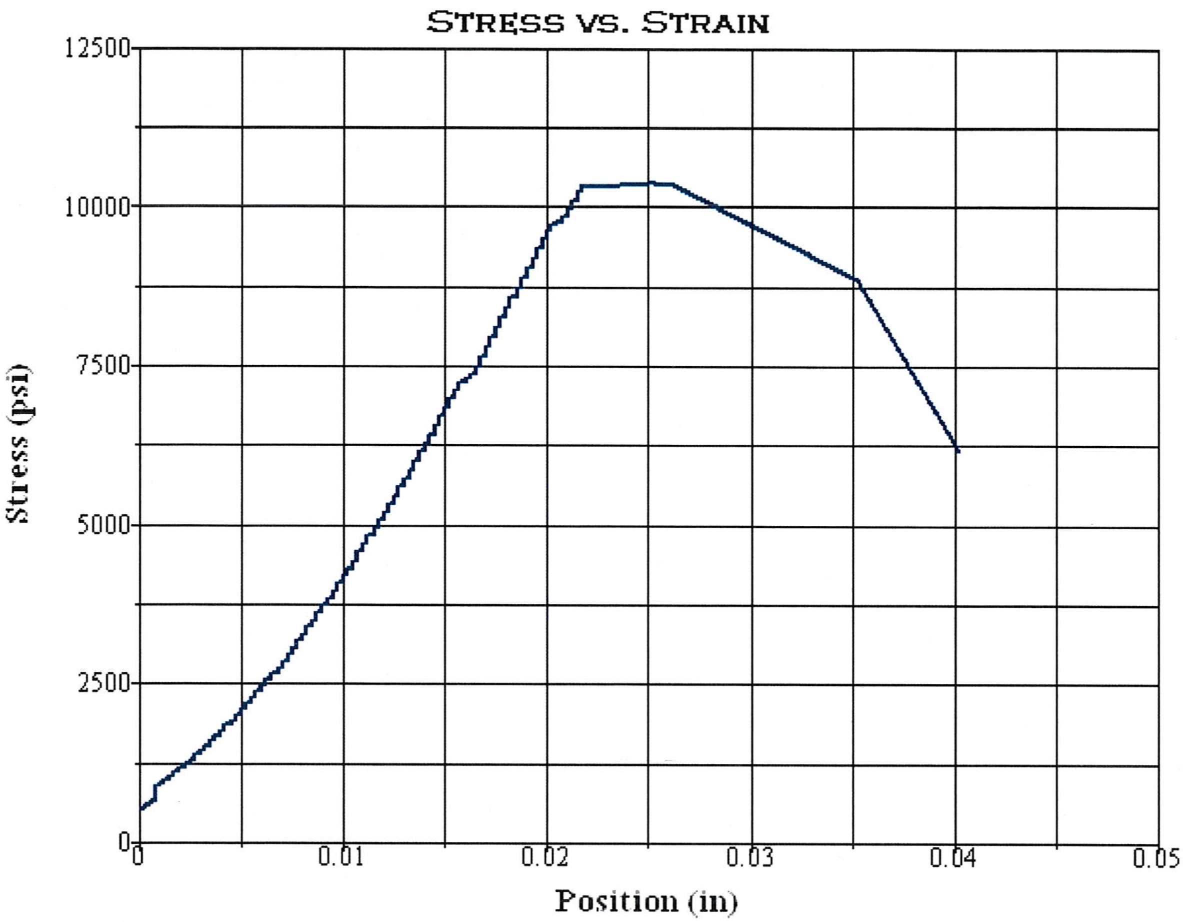




Test Summary	
Ticket:	Q15-034
GL #:	15-012
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-27.92; 06-0212L
Sample:	RC-15-004-63-63.6
Depth:	63-63.6'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/27/2015
Elapsed Time:	00:03:01
Workstation:	D1K00YB1
Operator:	AZM
Description:	Metasandstone
Load Direction:	horizontal to plane
Non-Conformances:	Flatness, Perpendicularity;
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results	
Specimen Gage Length:	3.4000 in
Diameter:	1.6580 in
Area:	2.1590 in²
Maximum Load:	15176 lbf
Compressive Strength:	7029 psi
Peak Load Rate:	1.50e+005 lbf/min





Test Summary	
Ticket:	Q15-035
GL #:	15-012
Project:	Shasta Viaduct
Dist-EA:	02-0E0901
Location:	02-SHA-5-27.92; 06-0212L
Sample:	RC-15-006-57.5
Depth:	57.5'
Procedure Name:	ASTM D7012 Method C
Start Date:	2/27/2015
Elapsed Time:	00:04:32
Workstation:	D1K00YB1
Operator:	AZM
Description:	Metashale
Load Direction:	45degree visible plane
Non-Conformances:	Flatness, Perpendicularity; L/D<2
Moisture Condition:	As-Received
Comments:	Best Effort Conform, Test As-Is
Failure Description:	Preexisting healed fractures

Test Results	
Specimen Gage Length:	2.5400 in
Diameter:	1.6560 in
Area:	2.1538 in²
Maximum Load:	22351 lbf
Compressive Strength:	10377 psi
Peak Load Rate:	1.87e+005 lbf/min

Division of Engineering Services
Geotechnical Laboratory

Point Load Strength Index

Dist-EA: 02-0E0901
Dist-Co-Rte-PM: SHA-5-27.92/

GI Tracking No.: 15-012
Report Date: April 9, 2015

Sample ID	Test Type	Length , L (mm)	Width, W (mm)	Initial Distance Between Contact Points, D(mm)	Final Distance Between Contact Points, D'(mm)	Equivalent Diameter, De (mm)	Failure Load, P (lbs)	Uncorrected Point Load Strength Index Is (psi)	Point Load Strength Index Is (50) (psi)	Remarks
RC-15-001_31.3'	I-L	20.45	41.7	19.5	17.5	30.48	325.6	226.08	181	
RC-15-001_39'	D-L	29.41		39.5	38	38.74	932.8	400.94	357	
RC-15-001_50.5'	D-L	22.41		39	33.5	36.15	1207.36	596.2	515	
RC-15-004_16.1'	A-L		42.4	31.5	29	39.57	869.44	358.29	322	

RC-15-001_31.3'

RC-15-001_39'

RC-15-001_50.5'

RC-15-004_16.1'

Test Type Abbreviations: D- Diametral, A - Axial, B - Block, I - Irregular Lump

Orientation of Load Direction (if anisotropic): P - Perpendicular to plane of weakness, L - Parallel to plane of weakness

Division of Engineering Services
Geotechnical Laboratory

Point Load Strength Index

Dist-EA: 02-0E0901
Dist-Co-Rte-PM: SHA-5-27.92/

GI Tracking No.: 15-012
Report Date: April 9, 2015

Sample ID	Test Type	Length , L (mm)	Width, W (mm)	Initial Distance Between Contact Points, D(mm)	Final Distance Between Contact Points, D'(mm)	Equivalent Diameter, De (mm)	Failure Load, P (lbs)	Uncorrected Point Load Strength Index Is (psi)	Point Load Strength Index Is (50) (psi)	Remarks
RC-15-004_49.5'	D-L	26.85		40.5	35	37.65	4371.84	1989.8	1751	
RC-15-004_59.3'	D-L	25.96		38	35	36.47	109.12	52.93	46	
RC-15-004_70'	A-L		42.3	40	36	44.03	681.12	226.64	214	
RC-15-006_69.8'	A-L		42.2	36	35	43.37	1268.96	435.34	408	

RC-15-004_49.5'

RC-15-004_59.3'

RC-15-004_70'

RC-15-006_69.8'

Test Type Abbreviations: D- Diametral, A - Axial, B - Block, I - Irregular Lump

Orientation of Load Direction (if anisotropic): P - Perpendicular to plane of weakness, L - Parallel to plane of weakness

MATERIALS INFORMATION

Geotechnical Engineering Evaluation of Permanent Loadings on the Union Pacific Railroad (UPRR) Tunnel, Dated August 7, 2015

Memorandum

Serious drought.
Help save water!

To: MR. JOHN MARTIN
Design Chief RI District 2

Attn: Mr. Travis Gurney, PE

Date: August 7, 2015

File: 02-SHA-5 PM R29.3/R30.3
Sidehill Viaduct Replacement
EA# 02-0E090
EFIS ID# 0200000016

From: **DEPARTMENT OF TRANSPORTATION DIVISION OF ENGINEERING SERVICES GEOTECHNICAL SERVICES – MS 5**

Subject: **Geotechnical Engineering Evaluation of Permanent Loadings on the Union Pacific Railroad (UPRR) Tunnel**

Based on a request, the Office of Geotechnical Design North (OGDN) has prepared this memorandum to provide a geotechnical engineering evaluation of potential impact by the proposed bridge foundation loadings on the UPRR tunnel that exists at the project site. Work performed by OGDN for this memo includes reviewing available publications and specific project design information, performing engineering studies and preparing this memo. Site investigation is not performed for this report. However, the recent subsurface exploration was utilized to assign engineering properties to the rock formation.

Please note our office has already prepared a Geotechnical Design Report (GDR) and a Foundation Report (FR) for the proposed project.

Existing UPRR Tunnel

Based on the bridge General Plan dated October 31, 2014 and the Foundation Plan (FP) dated November 16, 2014, the UPRR tunnel is located at approximately Station 897+20 on Al- Line. Based on an email communication dated July 7, 2015 with Mr. Travis Gurney, the Design Engineer from D02, top of tunnel is at approximate elevation of 1171.6 ft. and the track elevation is about 1136.0 ft.

Proposed Development

We understand that the proposed new bridge will be an arch bridge consisting of two abutments (Abutments 1 and 9) and 7 bents (Bents 2 to 8). The arch will support the bridge crossing over the UPRR tunnel. Bents 2 and 6 are the two thrust blocks supporting the arch. Bents 3, 4 and 5 are the intermediate bents supported by the arch. Bents 7 and 8 are individual supports and are discrete from the arch. All bents will be supported by Cast-In-Drilled-Hole (CIDH) piles of various diameters and lengths. The existing viaduct will be demolished after the new bridge is completed.

Loading Impact on Tunnel

Based on the FP, the arch footings at Bents 2 and 6 will be the nearest support locations to the tunnel. The footings are proposed to be 22 by 22 feet in plan dimension. Each footing will be supported by four 72-inch diameter CIDH piles.

To estimate the loading impact on the tunnel, the 4 piles at each footing are considered together and simulated as a "super pile" that produces a matrix of uniform downward stresses in the surrounding foundation materials. The "super pile" has a square cross section of 22 by 22 feet and a pile length equal to the average length of the 4 piles at each arch footing. The stresses are determined based on "Service – I Limit State Load" as shown in Table 3 of the FR dated March 12, 2015. The downward stresses were analyzed using computer program PLAXIS 2D which utilizes finite element method to analyze deformation and stability of the UPRR tunnel. The results of the analyses indicate an insignificant impact of permanent loading on top of the tunnel.

If you have any questions regarding this memorandum, please contact me at (916) 227-1033.

Reza Mahallati, PE
Senior Materials and Research Engineer
Office of Geotechnical Design - North

REGISTERED PROFESSIONAL ENGINEER
REZA MAHALLATI
No. 49374
Exp. 9/30/16
CIVIL
STATE OF CALIFORNIA

c: District Project Engineer
Structure Construction RE Pending File
Structure Office Engineer
District Material Engineer
Geotechnical Archive

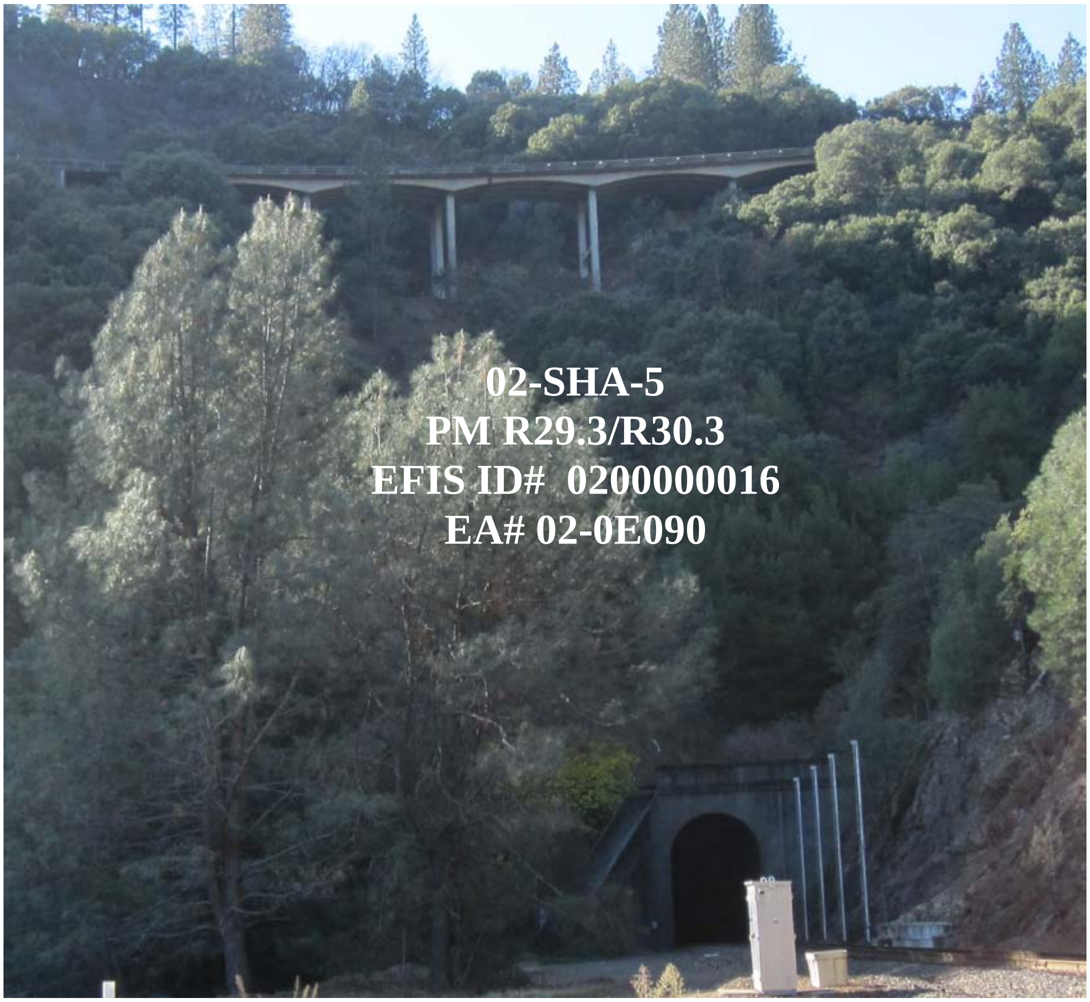
MATERIALS INFORMATION

Geotechnical Memorandum, Dated June 1, 2015

GEOTECHNICAL MEMORANDUM

DOG HILL PROJECT
(Sidehill Viaduct Replacement Portion)

State Highway 5



PREPARED FOR DISTRICT 02 DESIGN TEAM R-1

By:

J. Scott Lewis
Certified Engineering Geologist-2257
Registered Geophysicist-1032

DIVISION of ENGINEERING SERVICES-GEOTECHNICAL SERVICES
OFFICE of GEOTECHNICAL DESIGN-NORTH

June 1, 2015

Memorandum

Flex your power!
Be energy efficient!

To: **MR. JOHN MARTIN**
District 2 Design Chief R1

Attn: Mr. Travis Gurney
Transportation Engineer

Date: June 1, 2015

File: 02-SHA-5-PM R29.3/R30.3
EFIS ID# 0200000016
EA 02-0E090
Sidehill Viaduct Replacement
Part of Dog Hill Project

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

Subject: Geotechnical Memorandum Addressing Union Pacific Railroad Questions Regarding Sidehill Viaduct Replacement Project

Per your request, we are providing a Geotechnical Design Report (GDR) specifically addressing comments and requests posed by the Union Pacific Railroad (UPRR) regarding geotechnical aspects of the Sidehill Viaduct replacement portion of the Dog Hill Project on the southbound section of Interstate Highway 5 from PM 29.3 to PM 30.3 in Shasta County, California. Those comments and requests are contained within the following statement parsed from correspondence from Ms. Peggy J. Ygbuhay of UPRR to Mr. Michael Guzman with Caltrans:

> *1. Please provide foundation, debris and geotechnical reports for review. The reports shall consider and describe proposed risk and/or mitigation for the following aspects of the construction;*
> *- Expected impact, vibration, or risk to the existing Railroad tunnel caused by installation of new foundations at arch supports (drilling, excavating, machinery, etc.)*
> *- Expected impact, vibration, or risk to the existing Railroad tunnel caused by installation of new foundations to support falsework. (Falsework is not currently shown but may be necessary for temporary support at Bent 3, 4, 5 for arch construction).*
> *- Expected risk of debris (size, probability), and mitigation (debris drapes, permanent or temporary).*
> *- Expected path of debris at arch supports (will debris be directed toward either tunnel portal?)*
>
> *If the design or proposed construction methods pose a risk for tunnel damage or debris directed toward portals, alternate methods shall be coordinated with and may be required by the Railroad.*

The Foundation Report (FR) and attached Geotechnical design Report (GDR) are included with this submittal. A separate Geotechnical Memorandum (GM) is being submitted regarding loading conditions. There is no separate debris report, as this topic is covered in detail in the GDR.

The GDR focuses primarily on five geotechnical aspects of the project. The first two foci involve the analysis and design of two proposed cut slopes north and south of the new bridge, and rockfall mitigation for the northern slope. A third aspect is the description of the geotechnical conditions present in the areas where the contractor might consider constructing a construction access road for the new bridge. The remaining two focal aspects of this GDR involve potential affects the project might have upon the railroad infrastructure due to debris/rockfall and construction vibrations. These include the analysis and design of a rockfall and construction debris protection system and the potential effect of construction vibrations upon railroad tunnel #3. The GDR does not cover load and its effects upon the tunnel; these topics are addressed in the separate GM.

The FR, GM (loading), and GDR collectively consider, discuss, and address all of the topics listed above in Ms. Ygbuhay's requests, hopefully to the degree desired by the railroad personnel and/or representatives reviewing it. Abbreviated responses are provided below, together with a citation of the particular section(s) in the GDR where the topic is covered in detail.

Ms. Ygbuhay requested that these issues be addressed:

-The expected impact, vibration, or risk to the existing Railroad tunnel caused by installation of new foundations at arch supports (drilling, excavating, machinery, etc.)

The GM addresses loads upon the tunnel. The GDR addresses vibrations. By contract and specifications, the contractor will be required to submit to Caltrans and UPRR plans, drawings, and specifications for all access and falsework construction prior to beginning work. Work will not be permitted to begin until these plans, drawings, and specifications have been approved by both Caltrans and UPRR.

- *The expected impact, vibration, or risk to the existing Railroad tunnel caused by installation of new foundations to support falsework. (Falsework is not currently shown but may be necessary for temporary support at Bent 3, 4, 5 for arch construction).*

The GM addresses loads upon the tunnel. The GDR addresses vibrations in section 8.4.2.

Falsework and construction access plans are not available at this time, because these features will be designed by the contractor during the construction phase.

By contract and specifications, the contractor will be required to submit to Caltrans and UPRR plans, drawings, and specifications for all access and falsework construction prior to beginning work. Work will not be permitted to begin until these plans, drawings, and specifications have been approved by both Caltrans and UPRR.

Ms. Ygbuhay also requested that two rockfall and debris-fall related concerns be addressed including -

- *Expected risk of debris (size, probability), and mitigation (debris drapes, permanent or temporary).*

The GDR addresses debris and rockfall and its potential impact upon UPRR infrastructure. The GDR presents mitigation designs involving rockfall protection systems, including a double-twisted wire mesh (DTWM) attenuator and a Cable Mesh Attenuator (CMA) in section 8.4.2.

Ms. Ygbuhay also requested - *Expected path of debris at arch supports (will debris be directed toward either tunnel portal?)*

The GDR addresses debris-fall and rockfall paths in section 8.4.2.

The cable mesh attenuator system is one of the first orders of work for the project and shall be completed prior to bridge work. This rockfall and debris fall mitigation system can be left in place or removed following construction, depending upon the wishes of the UPRR. Our Office recommends that the system be left in place to provide extra long-term protection for the tunnel.

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

J. SCOTT LEWIS, P.G., C.E.G., R.G.P.
Associate Engineering Geologist
Office of Geotechnical Design - North

ec: John Martin – Senior Engineer R1 Design Team
Phil Baker – Project Manager
Charlie Narwold – Branch Chief, OGDN Branch B
R.E. Pending File (RE_Pending_File@dot.ca.gov
Travis Gurney- Project Engineer
Deena Matagulay – District 2 Office Engineer
Byron Berger, District Materials Engineer

Table of Contents

LIST OF TABLES

LIST OF PLATES

1. INTRODUCTION

This Geotechnical Design Report (GDR) is for the Sidehill Viaduct Replacement portion of the Doghill Project. The Sidehill portion is located on southbound Interstate Highway 5 from PM R29.3 to R30.3 in Shasta County, California. For ease of writing and reading the Sidehill portion of the project will hereafter simply be referred to as the Sidehill project in this GDR. Plate 1 presents a vicinity map showing the general location of the Sidehill project. Plate 2 presents an aerial photo of the Sidehill project.

The new bridge and its bents and abutments are addressed in a separate Foundation Report (FR) *Foundation Report for Shasta Viaduct (Replace)* (Zheng & Barnes, 2014). A separate memorandum (Song, 2015) provides a geotechnical engineering evaluation of potential impact by the proposed bridge foundation loadings on the UPRR tunnel (tunnel # 3). This GDR addresses other geotechnical issues of the project, including new cut slopes north and south of the new bridge that allow a smooth conform between the existing roadway and the new travelled way created by the new bridge. Additional topics addressed in this report include a rockfall and debris fall protection system to protect the railroad infrastructure, a discussion and analysis of the effects of construction vibrations that might impact the existing viaduct or the UPRR railroad tunnel (tunnel # 3) passing beneath the project area, and a description of the geotechnical conditions present in areas potentially utilized for construction access and falsework.

2. EXISTING FACILITIES AND PROPOSED IMPROVEMENTS

Currently southbound Interstate Highway 5 within the areas of proposed construction consists of a 2-lane roadway that travels atop the Sidehill Viaduct through a super-elevated turn that is rated considerably below the speed limit (65 MPH) of the remainder of the highway in the area. Besides being over 70 years old the viaduct structure was not sufficiently supported to meet current seismic standards and had been programmed by the California Department of Transportation (Caltrans) for a seismic upgrade. An evaluation by the project development team (PDT) and further discussion with management resulted in the altering of the initial project programming from seismic retrofit to replacement. The replacement bridge is located to the east of the existing viaduct and possesses geometrics (much larger radius turn with less super–elevation) more in line with the 65 mph speeds of the roadway north and south of the new bridge. This location shift brings the new bridge and highway into closer proximity to the railroad tracks and tunnel portal structure (Tunnel #3) of the Union Pacific Railroad that is situated downslope from both the old and new structures. The new bridge is founded on two abutments (north and south) and 7 intermediate bents. Bents 2 and 6 are thrust blocks at the base of an arch that spans the area directly over the ground wherein the railroad tunnel passes. Bents 3, 4, and 5 are not founded in the earth; these are supported by the arch.

Railroad tunnel # 3 is an arched back tunnel. The reinforced concrete lining is partially exposed for a distance of about 50 ft before it disappears completely inside the mountain. The tunnel has an interior height (from ballast surface to apex of ceiling) of about 27 ft and an interior width (from concrete wall to concrete wall) of about 18 ft. The thickness of the tunnel lining was indirectly estimated from exterior observations and photographs to be about 22 inches. This puts the tunnel width (B) at about 22 ft and the tunnel height (from top of ballast surface to top of concrete tunnel lining) at about 29 ft. Because the ballast surface does not truly represent the tunnel base from the standpoint of a rock mechanics analysis of the tunnel excavation, an effort was made to estimate the true tunnel bottom by examining the ground surface near the entrance and extrapolating it to the tunnel interior. This leads to an estimated tunnel height (H_t) of 30 ft, which includes the tunnel lining. The top of the portal facing, which surveys place at an elevation of 1171.6 ft, is estimated to be an additional 5 ft above the top of the concrete tunnel lining. This places the top of the concrete tunnel lining at an elevation of about 1167 ft.

Existing cut slope ratios north and south of the existing viaduct are predominantly 1:1, with some portions of the southern cut slope steepening to as much as 0.75:1 in a few locations. The cut north of the viaduct has a mid-slope bench; the southern cut is not benched. Existing cut slope heights exceed 150 ft.

The proposed cut slopes south of the new bridge have slope ratios of 1:1 and 0.75:1, with much of the slope possessing a dual slope ratio of 1:1 above 0.75:1 (see section 8.1). The proposed cut slopes north of the new bridge have slope ratios of 1:1 and a dual slope ratio of 1:1 above 0.75:1. The proposed new cuts slopes are not benched, except for a small remnant of the existing bench that is outside the limits of the north cut. Maximum height for the proposed cut slopes is approximately 100 ft, with some of the new top of cuts merging with the upper portions of the higher existing cuts on the north side.

Existing fill slopes situated immediately to the north and south of the viaduct have a maximum slope ratio of approximately 1.4:1. These fills are likely not higher, or perhaps more appropriately stated, thicker (height of vertical column), than about 25 ft, as they merge with natural slopes beneath them that extend further down the hill below. Proposed new fills on the north end of the project have slope ratios of 1.6:1 or flatter with maximum heights of about 40 ft. A proposed fill on the south end of the project in the vicinity of the bridge abutment has a slope ratio of 1.5:1 with a maximum height of about 20 ft.

Due to the potential for unacceptable rockfall risk on Interstate 5, combined with right-of-way (ROW) and excavation cost concerns that push back against the creation of an adequate rockfall catchment to address rockfall concerns (see section 8.1.4), a rockfall mitigation system (double twisted wire mesh (DTWM) attenuator) is proposed for the northern cut slope.

Finally, the demolition and removal of the existing viaduct is planned following the construction of the new bridge.

3. PERTINENT REPORTS AND INVESTIGATIONS

Background and research work for this GDR includes a review of Caltrans, state, federal, and private publications. The FR (Zheng & Barnes, 2014) and the geotechnical engineering evaluation memorandum (Song, 2015) cited earlier were reviewed. A search on the Caltrans Bridge Inspection Records Information System (BIRIS) intranet site yielded no geotechnically pertinent information for this project. A search on the Caltrans Intranet Document Retrieval

System (DRS) site yielded As-Builts and Plans on the existing viaduct that were reviewed for information pertinent to this report.

Foundation Report (FR) *Foundation Report for Shasta Viaduct (Replace)* (Zheng & Barnes, 2014). memorandum (Song, 2015) provides a geotechnical engineering evaluation of potential impact by the proposed bridge foundation loadings on the UPRR tunnel (tunnel # 3) that exists at the project site.

Caltrans research and publications on construction related vibrations and their impacts upon structures were reviewed and utilized in an effort to define and address their potential impact upon the railroad infrastructure and existing viaduct. The salient documents on this topic are cited in the text and listed in the references at the end of this report, together with pertinent non-Caltrans literature.

A Caltrans map (*District 2 Areas Likely to Contain Asbestos*; 2004) was utilized in corroborating field evidence that indicated no Naturally Occurring Asbestos (NOA) existed in the project area.

Caltrans literature, tools, and websites reviewed and/or utilized pertaining to seismic issues include the *Caltrans Fault Database* (Merriam, 2012), and the internal Caltrans website for calculating acceleration response spectra (ARS 2.2.06) curves.

Government published geologic literature reviewed include, but are not limited to, the *Geologic Map of California, Redding Sheet* (Strand, 1962); the *Fault Activity Map of California and Adjacent Areas* (Jennings, 1994); *Geology and Ore Deposits of East Shasta Copper-Zinc District Shasta County, California* (Albers and Robertson, 1961); *General Location Guide for Ultramafic Rocks in California - Areas Likely to Contain Naturally Occuring Asbestos,* (2000); *Shasta and Keswick Dams Central Valley Project, California, Issue Evaluation-Screening Level Ground Motion Analysis* (USBR, 2004); and *Geology of the Klamath Mountains Province, California division of Mines and Geology Bulletin 190* (Irwin, 1966).

Soil information was investigated on the Natural Resources Conservation Service (NRCS) Web Soil Survey Website (http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx) and in the Soil Survey of Shasta County Area, California (1974), although neither source proved to be helpful for the project area.

Other non-government produced literature related to multiple engineering and geological topics are cited throughout this report and listed in the references at the end of the text.

4. PHYSICAL SETTING

The physical setting of the project and the surrounding area was reviewed to provide information that might aid the Offices of Design, Construction, and Environmental Services on climate, topography, drainage, and man-made (including the railroad infrastructure) and natural features. The following is a discussion of the above review.

4.1. Climate

Historical climate information was obtained from the Western Regional Climate Data Center (http://www.wrcc.dri.edu/) weather stations located at Turntable Creek (station 049083) and Lakeshore 2 (station 044709). These two weather stations are the nearest available stations to the project, with one (Turntable Creek) being located approximately 4 miles south of the project and the other (Lakehead 2) being located approximately 10 miles north of the project. The average maximum temperatures (F°) for both stations are 94.1° and 95.3°, occurring in July. The average minimum temperatures for both stations are 51.0° and 52.7°. The average annual precipitation is about 64 inches and 69 inches at the Turntable Creek Station and Lakehead Station 2, respectively.

4.2. Topography and Drainage

The existing highway descends very gradually as it proceeds southward in this area; the new bridge will not change this. In the area of the project, the terrain slopes steeply down towards the east, towards the railroad tunnel portal and tracks, the northbound section of Interstate Highway 5, and finally Shasta Lake, in that order. The viaduct portion of the highway runs across (perpendicular to) this steep cross-slope, which is concave on contour, forming about 120° of a funnel-shaped slope that is focused upon the railroad portal below.

The new bridge will span a greater portion of the cross slope, as it will be founded lower down the slope to the east of the existing viaduct. The terrain below the new bridge includes the same stretch of funnel-like concave slope, plus additional terrain to the south that is convex and terrain to the north that is flat on contour. The terrain to the south drops into a 20- to 50-foot wide rock-lined drainage ditch that is at least 20 ft deep. The terrain to the north drops down into a steeply incised gully that runs generally north-northwest and is bounded on its eastern side by a very ridge up to 300 ft high. For the purpose of ease of discussion, all the sloping terrain beneath the new bridge described above shall hereafter be referred to as the *bridge slope*. Terrain north of the bridge slope and east of the highway that might potentially be utilized for construction access egress-ingress shall be referred to as the north terrain, while terrain up to 300 ft south of the bridge slope that lies east and downslope of the highway shall be referred to as the south terrain. The bridge slope, north terrain, and south terrain are demarcated on the aerial photo presented in Plate 2.

Surface flows slightly north and west of the existing viaduct flow down into this basin where they are collected by a concrete surface flow diversion structure constructed by the railroad atop the exposed portion of the concrete tunnel lining. This diversion directs the water coming off the northern part of the bridge slope into a large, deep (greater than 20 ft) rock-lined channel that sits at an elevation well below the grade of the railroad tracks and flows off to the east-northeast and Shasta Lake. This rock-lined channel also collects surface water coming off the central and southern part of the bridge slope.

Plate 3 shows the location of the main drainages marked in blue atop an aerial photo of part of the project area. All such drainages are ephemeral, with most flowing only during and shortly after precipitation. The surface flow diversion structure, which eventually receives all water from the bridge slope north of about station 897+50, generally has running water about 6 to 10 months per year, depending upon the amount and seasonal timing of precipitation for any particular year. The primary contributor allowing this nearly year-round flow is the drainage flowing into the project area from the north, beneath the highway, and down towards the diversion structure through the steeply incised gully mentioned above.

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

The primary man-made features within the project limits that could potentially be impacted by the project are the railroad tunnel and portal, the railroad tracks, and the existing viaduct. Design parameters and construction activities should be performed with these features in mind. The analysis and discussion of possible effects that the new bridge and its construction might have upon the railroad infrastructure is presented in sections 8.3 and 8.4.

4.4. Regional Geology and Seismicity

The project lies within the Eastern Klamath Belt in the southeastern portion of the Klamath Mountains Geologic Province (Irwin, 1966). Within the project region the rocks are mapped (Albers and Roberson, 1961) with unconsolidated Quaternary deposits overlying mafic quartz diorite, augite quartz diorite and birdseye dacite porphyry dikes and sills that intruded older pre-Jurassic (before 200 million years ago) rocks. These older rocks include marine sedimentary and volcanic rocks of the Hosselkus Limestone and Pit Formation. These rocks conformably overlie the Bully Hill Rhyolite of Triassic age (250 to 200 million years ago), which conformably overlies the Permian (299 to 251 million years ago) Dekkas Andesite. The Dekkas unconformably overlies the early Permian McCloud Limestone which (unconformably?) overlies the Carboniferous (359 to 299 million years ago) Baird Formation. The marine Baird Formation is composed of shale, sandstone, tuff, tuffaceous sandstone, greenstone, limestones, cherts, and mafic pyroclastics, and (unconformably?) overlies the Carboniferous Bragdon Formation, which is also of marine origin and is composed of shale, mudstone, siltstone, conglomerate, sandstone, and tuff breccias. The Kennett Formation and the Balaklala Rhyolite of Devonian age (about 400 to 345 million years ago) conformably underlie the Bragdon. The Balaklala Rhyolite intertongues with, and unconformably overlies, the Copley Greenstone, and is composed of porphyritic and non-porphyritic quartz keratophyre with some minor tuff, tuffaceous shale, and breccia. The Devonian (416 to 359 million years ago) Copley Greenstone, which unconformably overlies the Trinity Ultramafic sheet, is composed of keratophyre, spilite, and meta-andesite with a few localized lenses of tuff and shale.

The nearest active faults are the Keswick Fault (Caltrans fault ID number 35), which is less than 5 miles south-southwest of the project area, and the Battle Creek Fault (Caltrans fault ID number 20), which is located approximately 40 miles south of the project area. The Keswick Fault, a fairly recent discovery (USBR, 2004) that was located seismogenically, is located at depth on the subducting oceanic plate that dips into the earth beneath the project area and the area to the east of the project. The Keswick Fault has no surface expression or ground rupture.

4.5 Soil Survey Mapping

The entire project area is mapped as the Marpa-Holland deep families soil complex, 40 to 60 percent slopes by the USDA Natural Resources Conservation Service (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm). The typical profile for this complex is 13 inches of gravelly loam overlying another 13 inches of very gravelly clay loam on top of unweathered bedrock. This description is highly generic and has no practical use in predicting soil types, soil profiles, and/or depths to bedrock. The combined/synergized findings of field reconnaissance mapping, seismic refraction and seismic tomographic surveys, and foundation drilling notes have produced much more practical and representative information on soil types, soil profiles, and bedrock depths.

4.6 Naturally Occurring or Imported Asbestos (NOA)

Geologic units mapped (Albers and Roberson, 1961) in the project area are not known to typically contain 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) and the map published by Caltrans (2004), the project site is not mapped as an area likely to contain NOA. No native or non-native (such as imported serpentine-bearing RSP) serpentine was observed on site. These publications and the aforementioned field observations indicate that there is no native serpentine within the project area.

5. EXPLORATION

5.1 Drilling and Sampling

No borings were performed strictly for this GDR. At the time of this report writing, eight borings had been drilled specifically for the foundation report, with at least one boring located for each bent or twin sets of bents (two bents for the two different sides of the arch). The locations of these borings are shown in Plate 4. Information gleaned from these eight foundation borings was utilized in this report. Based on decisions by the PDT, additional foundation borings are planned, but these borings will have no influence upon this report, since they will be performed after this report is published. OGDN does not expect this exclusion to have any significant impact upon the quality or accuracy of the assessments or presented in this report, since the new borings are located in fairly close proximity to existing borings and surface field mapping does not indicate any apparent or obvious dramatic changes over these short intervals.

Surface rock samples and outcrops were examined in the field by hand lens as part of the geological reconnaissance of the project area. Surface soil was excavated with a small digging tool (ice ax) in over 50 locations of the project area and examined via methods explained in the Caltrans Soil and Rock Logging Manual (2012).

5.2. Geologic Mapping

A portion of a geologic map produced by Albers and Robertson (1961) that includes the project area and neighboring terrain 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 was conducted throughout the entire project area from north to south, from the northbound section of Interstate 5 to west of the top of the slopes west of the southbound section of Interstate 5. Reconnaissance was partially limited by the extensive brush and steep topography that exists in many locations within the project area, but the area was still reconnoitered fairly thoroughly using climbing gear, fixed or swinging ropes, and a lot of bushwhacking. Observations were made of the surficial geology, soils, petrology, geologic structure, stratigraphy, slope ratios and heights. Linear swaths of brush were cleared by Cal Fire for Caltrans where seismic refraction work was planned by OGDN. These swaths, which were about 4 ft wide and up to 300 ft in length, permitted additional detailed evaluation of the soils and surface geology in some critical areas. Information obtained from these field mapping efforts was noted 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.

5.3 Geophysical Studies

Fifteen seismic refraction survey lines were performed specifically for this report. A two-dimensional seismic tomography (downhole and surface source) survey was also performed in the vicinity of the southern thrust block and abutment in order to provide additional subsurface information for both the foundation work and the geotechnical assessment of construction access conditions. Three of the refraction lines were located specifically for the evaluation of proposed cut slopes north (SHRL3 & SHRL4) and south (SHRL15) of the new bridge. The remainder of the lines (SHRL 1, 2, 5, 6, 7, 8, 9, 10, 11, 12, 13, and 14) was shot along the alignment of the new bridge, as well as downslope of the roadway portions immediately to the north and south of the new bridge, in order to obtain subsurface information that, in conjunction with the drilling logs and surface mapping, would geotechnically characterize the subsurface sufficiently to allow the development and design of bridge falsework and construction access.

Seismic refraction and tomography depth sections are presented in Appendix 1. The locations of all 15 seismic refraction survey lines and the seismic tomography survey are shown together with boring locations on Plate 4.

6. GEOTECHNICAL TESTING

Geotechnical testing was not performed specifically for this report.

7. GEOTECHNICAL CONDITIONS

Geological and geotechnical conditions are described in this section for the project area, with some site specific information for the south and north cut slopes. Specific areas defined as construction access geotechnical zones (CAGZ) are discussed later in detail in section 8.3, where as much specific pertinent information as is available is provided, together with some geological interpretations, on a zone by zone basis. These zones, which are shown in Plate 6, also serve as location references from this point onward in this report for ease of discussion.

7.1 Site Geology

7.1.1 Lithology

With the exception of a very small ovoid area of mafic quartz diorite of Jurassic (about 208 to 144 million years ago) or Cretaceous (about 144 to 65 million years ago) age, the lithology of the entire project area is mapped (Albers and Robertson, 1961) as undifferentiated shale, sandstone, tuff, and tuffaceous sandstone of the Carboniferous (359 to 299 million years ago) Baird Formation. Plate 5 presents a portion of a Geologic Map published by Albers and Robertson (1961) that covers the entire Sidehill project area.

Based on the geological reconnaissance of the entire project area and an examination of the drilling cores obtained during the foundation drilling effort, the lithology of the project area consists, in order of decreasing prevalence, of sandstone, tuffaceous sandstone, shale, and tuff. Original fine sand texture is evident in the sandstones, and some of these rocks also had occasional small inclusions or larger grains within the finer grain texture. The rock identified as tuffaceous sandstone is primarily a sandstone that appears to be slightly metamorphosed, with the original grain structure of the sandstone being slightly to completely obscured or smeared out by welding caused presumably by the heat of the tuff component. The degree of welding varies gradationally, and, in samples where welding appears to have completely replaced the fine-grained to medium-grained texture of the tuffaceous sandstone, the rock becomes a welded tuff with a non-clastic aphanitic microcrystalline texture.

The sandstone and tuffaceous sandstone are fine- to medium-grained and vary from thinly bedded to massive. The tuff is aphanitic to fine-grained and varies from thinly bedded to very thickly bedded. Most sandstone outcrops and cores lack clear bedding structures while tuffaceous sandstone and tuff appear to demonstrate bedding slightly more often. The color of all three rocks varies from generally very light gray to medium gray, with some very fine lithic shale material possibly being responsible for the darkening or 'contaminating' of the otherwise lighter color. The sandstone, tuffaceous sandstone, and tuff vary from fresh (typically at depths greater than 30 to 50 ft, although some is located at or near the surface) to intensely weathered in the bedrock zone, and from slightly weathered to decomposed in the overburden colluvium or soil zone. The sandstone, tuffaceous sandstone, and tuff are typically very hard when fresh, hard when slightly weathered, moderately soft to moderately hard when moderately weathered, soft when intensely weathered, and very soft when decomposed. These three rock types are typically moderately fractured to slightly fractured in surface outcrops of fresh to slightly weathered rock, while core samples of similarly weathered rock appear to be more moderately fractured. The discrepancy between the surface and core samples are believed to be due to the drill bit chattering and creating greater fractures in the core samples. Lightly to moderately weathered sandstone, tuffaceous sandstone, and tuff all demonstrate degrees of iron oxide staining. Calcite infilling is found in some fractures and monocrystalline quartz has been injected in other fractures.

The shale is aphanitic and varies from laminated to very thickly bedded, with a majority varying from very thinly bedded to moderately bedded. Some minor shale is thickly bedded to very thickly bedded. Color varies from gray black to jet black, with weathered outcrops showing reddish brown iron oxide stains. Weathering varies from fresh (typically at depth) to intensely weathered, with most surface exposures varying from slightly weathered to intensely weathered. Shale hardness varies from moderately soft to hard, while fracture density varies from intensely fractured (in some intensely weathered outcrops) to slightly fractured (typically at depth in cores). Some shale outcrops and cores clearly demonstrate depositional features and textures, including darker or lighter shale clasts entrained inside a darker or lighter shale matrix, and flowing curving laminations.

The very minor mafic quartz diorite within the project area is the surface exposure of a dike that extends to some undefined depth, according to mapping and diagrams in Albers and Roberson (1961).

7.1.2 Structure

The evident structure in the Baird Formation rocks in the project area consists of joints or fractures, and either bedding or relict bedding structures. No broad trend or clustering (on a stereonet) of joint/fracture orientations was found, as the dips and azimuths are widely dispersed. Surface exposures of shale vary from very intensely fractured (intensely weathered samples) to moderately fractured. Fractures in some shale outcrops are roughly perpendicular to bedding but do not appear to be continuous across more than a few beds. Fresh to moderately weathered surface exposures of sandstones, tuffaceous sandstones, and tuff vary primarily from moderately fractured to very slightly fractured.

Spacing or thickness of bedding structures exposed in outcrops, varies from less than 1 inch to greater than three ft, with the average and predominant thickness likely somewhere between 2 to 6 inches. Bedding is much more widespread in the shales than the sandstones, tuffaceous sandstones, and tuffs. All exposed bedrock discontinuities observed in fresh to moderately weathered rock within the project area and along its periphery are tight and lack infilling, with the exception of iron oxide stains. Discontinuities in intensely weathered rock are sometimes infilled locally with thin layers of soil. Layer surfaces are smooth to slightly rough, depending upon lithology and location. Large scale (wavelength) undulations in planarity were not observed. Slight to moderate variations in bedding orientation, however, did change spatially over distances as small as ten ft.

Structural orientation of the Baird Formation bedding/layering is shown in Plate 5 by the strike and dip symbols placed on this map by Albers and Roberson (1961). These structural data have been supplemented by structural measurements taken for this project by OGDN in the form of dip angle and dip azimuth (data from a Clar compass or structural geology compass). These additional data are shown on Plate 5 as fractions, with the numerator in parentheses representing a range of dip angles and the denominator in parentheses representing a range of dip azimuths for the general location where the fraction is located. Single values indicate that only a single measurement was available. The orientations obtained by Albers and Roberson (A&R) and OGDN form the basis for the kinematic analysis and design of the south and north cut slopes, and should also be used for designing construction access in areas where cuts might be considered.

Primary (F1; bedding) structural orientations measured by A&R and OGDN pertinent to the northern cut slope dip between 50° and 67° at azimuths between about 105° and 122°. Secondary structural orientations (F2) consist of fractures/joints are not considered as kinematically important as the F1 (bedding) structures, because of their relatively short lengths, their localized nature, and their varying or inconsistent orientation from locale to locale.

No bedding structures or measurable rock outcrops were found within the location of the proposed southern cut. Structural orientations located at least four hundred ft to the northwest of the proposed southern cut are considered likely to be primary bedding (F1) structures and, therefore, possibly pertinent to the kinematics of the southern cut slope. These have orientations dipping between 13° and 32° at azimuths between 100° and 140°.

Structural orientations were measured elsewhere within the project area where viable outcrops were available. Such outcrops were found on the bridge slope, beneath the existing viaduct, and north of the bridge slope within the Caltrans ROW where construction access egress-ingress may potentially be developed. These measurements were taken on primary (F1) bedding structures in outcrops interpreted to represent in-situ rock and not colluvial float.

Albers and Roberson (1961) mapped several geological structures in the periphery of the project that can be seen on Plate 5. The most prominent such feature is the long anticlinal structure trending northwest along the western boundary of the project. The other such feature is a southerly plunging anticline mapped about 800 ft to the east of the project somewhere in the vicinity of the existing northbound Interstate 5 (Northbound Interstate 5 is not shown on Plate 5 since it did not exist at the time of mapping and publication (1961) by Albers and Roberson). No work was performed by OGDN to map or confirm these structures.

Seismic Anisotropy and Geologic Structure

Bedrock seismic anisotropy is clearly evident in the seismic refraction data and is most likely due to the nearly ubiquitous presence of bedding structure throughout the Baird formation. This anisotropy was observed in all areas of the project, with the exception of the south cut, where only one refraction line was shot, thereby making it impossible to confirm or deny anisotropy (a cross-line was not considered feasible due to the uneven lay of the topography). This seismic anisotropy can be used to corroborate inferred structural orientation of buried bedrock.

Bedrock in the portion of the bridge slope demarcated as CAGZ zones 8 thru 16 and 19 is covered entirely by overburden. No bedrock outcrops exist within this area, while the few outcrops of weathered rock exposed here do not contain any clear bedding structure, nor can it be said with any certainty that these outcrops are even representative of in-situ bedrock, as opposed to simply being large pieces of floating colluvium. Bedding, which generally dips to the east and southeast in nearby areas upslope (to the west and southwest) and downslope (to the east and northeast) of the CAGZ zones mentioned above, would likely have orientations somewhere between the values found in these upslope and downslope areas. No direct geological evidence exists, however, of such bedding orientations beyond that of interpolated inference from these nearby areas. The strike of such inferred structure, when converted to the apparent ‘rake’ on top of the bedrock surface (essentially where the structure or a bed traces along the dipping bedrock surface), generally trends fairly close to the alignment of seismic refraction lines 1, 8, and 10. The dip azimuths for these inferred bedding structures also align fairly close to the alignments of seismic lines 2 and 9, which are more or less perpendicular to lines 1, 8, and 10.

In cases of structurally induced seismic anisotropy, the higher velocity anisotropic direction is typically the one where the seismic raypaths travel along the strike of a single, or just a few, fast beds. The slower velocity direction is that where the raypaths travel perpendicular to this structure. The slower velocity is produced by the numerous bedding contacts, as well as the lesser velocity of the beds of slower material, that the raypaths must traverse. In our case, bedrock velocity for line 1 is 8400 ft per second (fps), while the bedrock velocity in the cross line (line 2) is 7050 fps, which is a 16% decrease in velocity. Bedrock velocity for the upsloping line 8 is 7400 fps while its cross line, line 9, has a bedrock velocity of 6670 fps, which is a 10% velocity decrease. Bedrock velocity for the upsloping line 10 is a very fast 11,500 fps. No cross-line exists for line 10, although the relatively high speed of this material makes it likely that this is also the high velocity direction. The northern end of line 9, which is oriented perpendicularly to line 10, is less than 50 ft from intersecting line 10, while the southern end of line 2, which is also oriented perpendicularly to line 10 is located about 90 ft north of line 10. Since both of these lines are considered to be oriented in the slow direction, and because of their relatively close proximity to line 10, their velocities might be used for comparisons with the bedrock velocity of line 10, which results in velocity decreases of 42% (when compared to line 9) or 39% (when compared to line 2). Though by no means can this anisotropy be considered to be definitive proof of the structural orientation, it does provide strong corroboration to the inference of southeast- to south-dipping bedrock structure existing beneath the overburden in CAGZ zones 8 through 16 and 19.

Similar lines of reasoning can be applied to the seismic anisotropy observed between lines 5, 6 and 7, which are located in CAGZ zones 4 and 5. Structural data from clearly defined bedding in the north cut around station 906+25, which is fairly close to these CAGZ zones, infer that bedding structure in the bedrock likely dips about 40° to 55° at an azimuth between 100° and 125°.

Bedrock structure beneath CAGZ zones 20 through 28 is inferred by combining neighboring structure data obtained on outcrops, coring data, and an analysis of the anisotropy and general velocity structure shown in seismic lines 11 through 14. Bedrock structure (bedding) in CAGZ zones 20 through 28 likely dips somewhere between 15° and 35° at azimuths between 75° and 140°. Narrower estimates of dip and azimuth ranges are provided, where possible, in the description of each individual CAGZ zone in section 8.3.

7.1.3 Natural Slope Stability

Natural slopes within the project area are considered stable in their current morphology, with the exception of some active colluvial areas. No significant sloughing or sliding of uncut intact bedrock or soil was observed, except in active colluvium.

While performing field reconnaissance, Factor of Safety (FOS) estimates were assigned to slopes based on a combination of personal experience examining slopes and experience performing global stability analyses via several different limit equilibrium methods within computer software packages. Field observations of slope ratio, surface and subsurface material type (observed in ravines), angularity, gradation, cohesion, and density contributed to these estimates. These FOS estimates are intended to be approximate qualitative stability evaluations that carry less ambiguity than generalized adjectival descriptions of stability.

The rock and colluvial slopes of the bridge slope have slopes predominantly between 35° (1.43:1) and 45° (1:1), with the average being about 40°, which is slightly steeper than a 1.2:1 slope ratio. Local areas of these slopes are considerably steeper, however, in the range of 55° (0.7:1) to 65° (0.47:1), with a few local near-vertical drops up to 20 ft in height. These steeper areas are essentially rock slopes consisting of moderately weathered to fresh rock. Slopes on the southern part of the bridge slope vary between 29° (1.8:1) and 40°. Bridge slope areas that are primarily composed of active colluvial material, essentially loose talus slopes, lay predominantly at around 40°, which is basically slightly below the angle of repose (friction angle) of the colluvium, while some lesser areas of established colluvial slopes recline at angles between 30° (1.73:1) and 40°, and an even smaller percentage reach angles up to about 45°, probably due to localized bedrock protrusions that extend into the overburden and provide support. Most areas of colluvial overburden have an estimated global stability FOS from slightly above 1.0 to about 1.3. Many surface deposits in the center of CAGZ 19 consist of loose unconsolidated talus with an FOS barely over 1.0. In contrast, overburden deposits in the northern portion of CAGZ 10 are composed of subangular to angular boulders and cobbles in a semi-compacted clayey sandy matrix resting atop a steep but likely irregular bedrock surface, all of which results in an estimated FOS of about 1.2 for the overburden zone. A key fact limiting the upper bound for any estimated FOS for most of these slopes is that underlying all of these slopes is a typically steep competent bedrock surface that can typically provide only limited stability for the overburden.

Slopes south of the bridge slope that lie within the ROW and might possibly be used as egress-ingress for construction access vary in slope from about 28°(2.1:1) to 36°(1.7:1). Overburden on these slopes, with an average thickness of about 20 ft, is composed of a mixture of subangular colluvium and intensely weathered bedrock. This overlies moderately weathered bedrock that transitions into slightly weathered and even fresh bedrock only at considerable depths often exceeding 40 to 50 ft from the surface. Based on these conditions, these slopes have an estimated FOS varying from about 1.2 (steeper slopes approaching 35°) to about 1.4 (flatter slopes). Further downslope, outside of the ROW, portions of these slopes steepen to as much as 50° where stable shallow bedrock is moderately to slightly weathered.

The slope north of the bridge slope that makes up the major portion of the terrain in CAGZ zones 2, 3, and 4, descends eastward into a gully at an average slope angle of about 40°, though slope angles generally vary between about 36° and 45°, and reach angles above 45° (in CAGZ zone 3) where overburden has been almost entirely removed by erosion and bedrock structure becomes the predominant factor influencing slope angle. Overburden here has a relatively high seismic velocity and is likely composed of a mixture of remnant bedrock protrusions and subangular colluvium of boulders and cobbles in a matrix of well-graded gravel with clay and sand, and well-graded gravel with silt and sand. Bedrock underlying the moderately compacted overburden is very competent, based on neighboring outcrops and high seismic velocities. The majority of this slope is considered stable and has an estimated FOS of about 1.2 to 1.3 where the overburden is established, while some lesser areas where the top material is loose has an estimated FOS ranging from 1.0 to about 1.1.

Natural slopes located north and south of the existing viaduct and west of the highway that have an overburden of soil and weathered rock vary between 26° (2.1:1) and 40° and are considered stable. A few native local rock slopes on the west side of the existing viaduct and highway vary from 45° (1:1) to 90° (vertical) and are also considered stable.

7.2 Soils

Soil information was obtained during field reconnaissance by hand sampling and testing, in accordance with the Caltrans Logging Manual, of surface material and cut slopes. Soil information was also obtained from examination of the upper material obtained during the foundation drilling. Extrapolated and/or interpreted soil profile thicknesses and lateral extents were estimated based on correlations between exposed soil profiles, cut slopes, eroded soil slopes, boring samples, and seismic refraction results.

Soil profiles in the areas of the proposed north and south cuts vary in depth from about 8 to 15 ft, with the lower reaches of the profiles consisting of remnant gravel- and cobble-sized pieces of weathered bedrock in a matrix of decomposed bedrock weathered to a clayey gravel with sand (GC) or gravelly lean clay with sand (CL). The upper reaches of these profiles appear to be fairly loamy, with varied engineering descriptions likely including sandy lean clay with gravel (CL), sandy silt with gravel (ML), gravelly lean clay with sand (CL), and gravelly silt with sand (ML). These soils are generally moderately to well-drained, of low plasticity or non-plastic, with no apparent shrink-swell potential.

As mentioned before, soils of the bridge slope are derived from a mixture of colluvium and weathered bedrock. These soils vary from a thin veneer to as much as 28 ft in thickness, and are sometimes interwoven and laced between remnant bedrock outcrops that may protrude above the surface or remain covered at varying depths below the surface. Soils derived strictly from in-situ bedrock without any contribution from colluvium are far less common than soils involving colluvium in the terrain of the bridge slope. The weathered bedrock soils are generally far thinner than most of the colluvial soils. In some areas the colluvial soils have minimal development and are essentially undeveloped colluvium that is migrating gradually downslope, while in other areas the colluvium has at least partially stabilized due to irregularities in the bedrock topography, together with vegetative, shrub and arboreal growth. The colluvial derived soils are primarily gravels in the upper reaches of profiles, and cobbles with some boulders in the lower parts of profiles, although this varies quite considerably. The following soils, in likely order of prevalence, can be found on the bridge slope:

- Well-graded gravel with silt and sand (GW-GM) with and without cobbles
- Silty gravel with sand (GM) with and without cobbles, some with boulders
- Silty sand with gravel (SM) with and without cobbles, some with boulders
- Clayey gravel with sand (GC) with and without cobbles, some with boulders
- Well-graded gravel with clay and sand (GW-GC) with and without cobbles, some with boulders
- Cobbles, or cobbles and boulders, with a matrix of:

well-graded gravel with silt and sand (GW-GM) or
silty gravel with sand (GM) or
silty sand with gravel (SM) or
Clayey gravel with sand (GC) or
Well-graded gravel with silt and sand (GW-GM)

The majority of soils found on the slope north of the bridge slope (CAGZ zones 2, 3, and 4) consist of well-graded gravel with sand (GW), well-graded gravel with silt and sand (GW-GM), well-graded gravel with clay and sand (GW-GC), and cobbles with a matrix of well-graded gravel with silt and sand (GW-GM).

Soils in the level area above this slope (in CAGZ 2, 3, and 4) consist primarily of poorly graded gravels with sand (GP).

7.3 Surface Water and Groundwater

No year-round streams are present within the project area. Plate 3 shows the drainage locations and patterns for the immediate project area. The slightly thinner blue lines show drainages that carry surface water runoff primarily only during and shortly after storm events. The heavier blue line shows the drainage coming from the north-northwest that carries water for the longest period of time every year, probably 6 to 9 months per year on average.

Groundwater exists in some fracture zones and at some local contacts between colluvial and/or soil overburden and bedrock. This presence is sporadic spatially throughout the project area. It also varies seasonally between the wet (typically winter through spring) and dry seasons (typically summer into fall). These water areas are demarcated with blue lines on Plate 6.

The existing cut slope at station 905+00 and 906+00, which is within the area of the north cut, supports a considerable growth of highly hydrophilic (water-loving) plants, a growth indicative of the presence of groundwater fracture zones. These water areas are demarcated with blue lines on Plate 6. Smaller but similar clusters of plants can be found in a few scattered locations across the bridge slope. It is considered highly unlikely that these fracture zone locations carry water quantities capable of causing construction difficulties, such as significant slope instabilities or gushing water.

There are additional water producing fracture zones downslope of the Caltrans ROW, including one on the cut slope immediately north of the railroad tracks behind the pole and wire rockfall alert system belonging to the railroad. Several low-volume water-bearing fracture zones exist near the toe of the bridge slope where it meets the drainage demarcated with the heavier blue line (mentioned above).

Groundwater also travels at a relatively rapid rate atop the bedrock beneath the colluvial overburden. During rainfall and storm runoff a considerable amount of groundwater likely travels thusly atop the bedrock surface directly above the tunnel until it encounters the tunnel lining at the location where the bedrock-overburden interface intersects the tunnel lining. This location is likely about 100 ft from the tunnel entrance. A considerable volume of groundwater likely flows around the tunnel lining in this location. This flow has possibly created some localized weakness in the bedrock, and quite possibly in the tunnel lining. Water may possibly issue from any cracks or holes in the tunnel lining, should they exist (this could not be verified because access was not permitted).

7.4 Erosion

Erosion is low in the project area except in those colluvial areas where several gullies have formed primarily due to surface flow and storm runoff stripping the loose unconsolidated colluvial material. Erosion is low on the native slopes north and south of the viaduct. Many parts of the project area consist of exposed bedrock, which is highly resistant to erosive forces.

7.5 Project Site Seismicity

The Caltrans ARS Online Tool (version 2.3.06) bases its deterministic spectrum on the nearest active fault that controls ground motion, which, in this case, is the Keswick fault (ID No. 35) with a Maximum Moment (MMax) of 6.0 and a dip of 65° to the southeast. The project area is approximately 3 to 4 miles from what is thought to be the fault's northern terminus. The Caltrans Fault Database (Merriam, 2010) classifies the Keswick Fault as a reverse fault.

Although the sandy and silty nature of some of the surface material might initially indicate a susceptibility to liquefaction, the potential for liquefaction is considered very low due to the lack of any stable groundwater table within the surface deposits, the steep topography, and the low water production, if any, of any possible buried fracture zones in the underlying bedrock.

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

Small localized, inactive ancient faults are mapped (Albers & Roberson, 1961) in the rocks near the project area, but none are shown to be present within the confines of the Sidehill Project. These faults are old and not considered active.

8. GEOTECHNICAL ANALYSIS AND DESIGN

8.1. Cuts and Excavations

8.1.1 Cut Slopes

Existing rock cut slopes along the west side of the highway lay predominantly at a 1:1(45°) slope ratio, with some areas steepening to 0.75:1(53°), and a few localized areas being as steep as 0.25:1(63°). A majority of these cuts are at least over 50 years old and do not appear to have undergone any significant instability beyond some localized surficial rockfall. Existing natural rock slopes that show vertical drops of up to 25 ft strongly suggest that steeper cuts may be feasible where kinematic analysis and rock quality are favorable.

Steepening up the north and south cut slopes, if possible, would serve to reduce excavation costs considerably, especially since the net earthwork balance for the project has considerably more excavation than fill in the initial design, which has cut slopes at 1:1.

Due to a limited amount of ROW in the vicinity of the north cut slope and the inability of the project to procure additional ROW because of the project's tight schedule (according to the PDT) there is a need and value in investigating the possibility of steepening the cut slope design on the north end of the project.

Kinematic analysis of the exposed bedding (or relict bedding) structures indicates that the rocks on the north end of the project could conceivably be cut at a triple slope ratio with 1:1 slopes above 0.75:1 slopes above 0.5:1 slopes, with the slope ratio transitions occurring a few ft below where the refraction velocities increase noticeably (indicating stronger more competent rock). Such a design keeps the slopes outside the kinematic area of instability, but not by more than about ten degrees in places. Should highly weathered zones be encountered at depth during construction due to groundwater having been restrained there by harder bedrock below, local failure in such rocks could create a cavity in the cut slope that would then exceed the kinematic stability indicated by the observed structure and failure could ensue that could run all the way up the slope to the top of cut. A more prudent dual slope ratio of 1:1 over 0.75:1, with the steeper slopes being cut at depths where the rock hardness and competence increases noticeably as indicated by seismic refraction results, should be able to handle potential weak zones without being destabilized. With P wave velocities of about 6700 fps at depths between 30 and 37 ft (determined perpendicular to the original ground surface), the transition into the 0.75:1 slope ratio can safely begin at a depth of 40 ft (this provides a few extra ft above the 37 ft mentioned above to conservatively handle potential variation). The more competent deep rock has more than enough strength to stand steeply, given the favorable kinematics. This analysis results in the cut slope ratios listed by station and depth perpendicular to the original ground surface (for different slope ratios) in Table 1 for the north cut slope. Plate 7 offers an example of a double cut slope ratio to explain how the depths given in Table 1 are determined on the cross-section. The basis for depths being determined perpendicular to the original ground is due to the fact that this is how the seismic information samples the subsurface, and this is typically how soil and rock weathering fronts usually develop (perpendicular to original ground surface).

Table 1. Cut slope ratios, transition depths between differing slope ratios, minimum catchment widths, and approximate locations of DTWM Attenuator posts and lateral cable anchors (see section 8.1.4) for the proposed north cut slope (station 905+50 to 910+75). (L-LCA: left lateral cable anchor. R-LCA: right lateral cable anchor.)

STATION	CATCHMENT WIDTH (MIN)	SLOPE RATIO 1	SLOPE RATIO 2	Depth to Slope Break (ft)	DTWM ATTENUATOR (see section 8.1.4)
	(ft)	(S1)	(S2)	(S1/S2)	
905+50	6	1:1	NO	NO	Post1, L-LCA
905+75	6	1:1	NO	NO	
906+00	6	1:1	NO	NO	Post2
906+25	6	1:1	NO	NO	
906+50	6	1:1	NO	NO	Post3
906+75	6	1:1	NO	NO	
907+00	6	1:1	NO	NO	Post4, L,R-LCA
907+25	6	1:1	NO	NO	
907+50	6	1:1	NO	NO	Post5
907+75	6	1:1	NO	NO	
908+00	6	1:1	NO	NO	Post6
908+25	6	1:1	0.75:1	40	
908+50	6	1:1	0.75:1	40	Post7, L,R-LCA
908+75	6	1:1	0.75:1	40	
909+00	6	1:1	0.75:1	40	Post8
909+25	6	1:1	0.75:1	40	
909+50	6	1:1	NO	NO	Post9
909+75	6	1:1	NO	NO	
910+00	6	1:1	NO	NO	Post10,L,R-LCA
910+25	6	1:1	NO	NO	
910+50	6	1:1	NO	NO	
910+75	6	1:1	NO	NO	Post11, L-LCA

There are no structural outcrops within the area of the proposed southern cut slope so steepening the cut based strictly on kinematic analysis is not possible, at least directly. Design has a large amount of shoulder in this area in order to create sufficient sight distance, so steepening the initial 1:1 design for the purposes of rockfall catchment is not necessary. Steepening the cut slope design would, however, reduce excavation without impacting the needed sight distance, at least in most situations, so the investigation of cut slope steepening is still warranted on the basis of cost reduction.

Table 2. Cut slope ratios, transition depths between differing slope ratios, and minimum catchment widths for the proposed south cut slope (station 884+75 to 891+25).

STATION	CATCHMENT WIDTH (MIN)	SLOPE RATIO 1	SLOPE RATIO 2	Depth to Slope Break (ft)
	(ft)	(S1)	(S2)	(S1/S2)
884+75	15	1:1	NO	NO
885+00	15	1:1	NO	NO
885+25	15	1:1	NO	NO
885+50	15	1:1	NO	NO
885+75	15	1:1	NO	NO
886+00	15	1:1	NO	NO
886+25	15	1:1	NO	NO
886+50	15	1:1	NO	NO
886+75	15	1:1	NO	NO
887+00	15	1:1	0.75:1	15
887+25	20	1:1	0.75:1	15
887+50	20	1:1	0.75:1	15
887+75	20	1:1	0.75:1	15
888+00	20	1:1	0.75:1	15
888+25	20	1:1	0.75:1	15
888+50	25	1:1	0.75:1	15
888+75	25	1:1	0.75:1	15
889+00	28	1:1	0.75:1	15
889+25	28	1:1	0.75:1	15
889+50	28	1:1	0.75:1	15
889+75	28	1:1	0.75:1	20
890+00	28	1:1	0.75:1	20
890+25	28	1:1	0.75:1	20
890+50	25	1:1	0.75:1	20
890+75	22	1:1	0.75:1	20
891+00	20	1:1	0.75:1	20
891+25	20	1:1	0.75:1	20

Information that supports cut steepening and assists in delineating depths to transitions for multiple cut slope ratios is provided by the following:

1) structural field data obtained several hundred ft north of the proposed southern cut are kinematically favorable and are likely safely extrapolated to the south cut area;
2) existing cut slope measurements in the proposed south cut slope area showing that a small portion of the existing cut approaches a steepness of 0.75:1; and
3) seismic refraction results indicate a maximum depth (in the area of the refraction line only) of only 10 ft to the transition between the top overburden of soil and weak rock to a stronger rock with a velocity of about 2700 fps. This information and subsequent field analysis leads to the cut slope ratios listed by station and depth (again, perpendicular to the original ground surface) for the south cut slope listed in Table 2.

Transition depths between different slope ratios have been assigned considerably deeper than 10 ft due to field observations and an effort to provide some conservatism. Conservatism is less in this south cut design than the design in the north cut, because of the significantly greater catchment available at the base of the south cut (albeit paved catchment), catchment that can handle sloughs and slides should they occur.

8.1.2 Rippability

Rippability assessments are made based on P wave seismic velocity (V_p) correlations to empirical data, rock type, and rock fracture and joint characteristics. Seismic velocity correlations in this report are based on two different scales, each with differing rippability assessments depending upon ripping equipment and degree of rock type specification. The Caltrans Geophysics Group has its own internal non-rock-type specific correlation scale between seismic velocity and rippability based generally on a Caterpillar D9 Series bulldozer with a single-toothed ripper. The Caltrans scale is considerably more conservative.

V_p (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 (1982; 2010) is also presented here to provide the contractor with a wider range of rippability information. For metamorphic schist (and a D10 with a single shank ripper) the following scale applies:

V_p (fps)	(Schist)	Rippability
≤ 8000		Rippable
8001 – 10,000		Marginally Rippable
> 10,000		Non-Rippable

For sandstone (and a D10 with a single shank ripper) the following scale applies:

V_p (fps)	**(Sandstone)**	**Rippability**
≤ 9300		Rippable
9301 – 11,500		Marginally Rippable
> 11,500		Non-Rippable

For shale (and a D10 with a single shank ripper) the following scale applies:

V_p (fps)	**(Shale)**	**Rippability**
≤ 10,000		Rippable
10,000 – 12,000		Marginally Rippable
> 12,000		Non-Rippable

No scale is presented directly for tuff, but an average of the scales presented for breccia and conglomerate is probably the best approximation (for a D10 with a single shank ripper):

V_p (fps)	**(Tuff)**	**Rippability**
≤ 9,000		Rippable
9,000 – 11,000		Marginally Rippable
> 11,000		Non-Rippable

Given the V_p values observed in Line 15 and the exposures in the existing and nearby slopes, the south cut is considered to be completely and easily rippable within the limits of both rippability scales.

Based on the non-rock-type specific and more conservative Caltrans scale (smaller dozer (D9) and single shank ripper), approximately 85% to 90% of the material proposed for cutting in the north cut is easily ripped, while the remaining 10% to 15% is deemed not rippable. In contrast, the north cut is considered completely rippable based on the Caterpillar seismic velocity rippability scales for the larger D10 with a single shank ripper. For the high velocity rock (V_p = 6700 fps across the structural grain) present in the lower reaches of a portion of the north cut the bedding layers are distinct, though tight, and should offer a structural grain amenable to ripping with the single shank ripper.

Bedrock in the bridge slope, the south access terrain, and the north access terrain has V_p values ranging from about 3200 fps to 11,500 fps, which indicates easily rippable to non-rippable rock based on both rippability scales. Rippability of these areas is discussed later in greater detail in the section (section 8.3) on construction access geotechnical zones.

Based on the rippability descriptions discussed above, some method of excavation beyond standard ripping and cutting with an excavator or dozer blade may be necessary in some locations if the contractor decides not to utilize the larger D10 size equipment. This need for an alternate method may also apply in some areas of the bridge slope and the northern access terrain, regardless of excavation equipment size, depending upon how the contractor decides to create access in these areas.

8.1.3 Grading Factor

A cumulative grading factor of 1.0 is recommended for material excavated from the north and south cuts that is to be utilized in constructing the fill to the north of the northern cut and station 111+00. This factor is based on an assumed relative compaction of 90% for embankments. This grading factor was estimated based on the approximate relative percentages of soil, weathered rock, and unweathered rock expected to be excavated from the cuts; seismic refraction velocities and their correlative earthwork factor to similar rocks according to Smith et al. (1972) and Stephens (1978); and engineering judgment based on previous experience with similar material.

8.1.4 Rockfall

This section discusses rockfall potential and mitigation issues for design cut slopes north and south of the bridge replacement where the new roadway conforms to the existing highway. The potential for construction-spawned rockfall and debris to impact the railroad infrastructure below the new bridge is examined and analyzed separately in section 8.4.1.

Existing cut slopes south, north, and west of the existing viaduct all demonstrate signs of rockfall production, primarily in the form of rocks lying on the paved shoulders. Currently, the size of the existing shoulders, the roadway super-elevation sloping away from the roadway and towards the base of the slopes, the existence of a mid-slope bench (north cut slope), the existence of a 12-foot paved shoulder, and the relatively moderate cut slope heights all serve to adequately prevent the rockfall from noticeably interfering with the travelled way. According to maintenance forces, a few small rocks occasionally make it to the travelled way, and once every few years a cobble-sized rock has done so as well, but generally rockfall has not been considered to be a significant problem in this location due to the mitigating factors described above.

North Cut.
The relative locations of the proposed north cut and new roadway alignment will dramatically increase the velocity and volume of rocks that reach the travelled way, both by direct impact and run-out (rocks that roll or bounce out into the highway after the first direct impact). Horizontally shifting the new cut slope away from the new alignment approximately 20 feet would allow the creation of a catchment area at the toe of the slope that would mitigate the problem. Unfortunately, ROW in the vicinity of

the proposed north cut slope is very restricted, with the top of the proposed north cut coming within a few ft of the ROW boundary. The ROW restriction also resulted in the removal from the new cut of the mid-slope bench that is in place on the existing cut slope. This elimination and the significantly greater height of the new cut (section 8.1) relative to the existing one contribute substantially to the increased rockfall risk to the travelled way. The change of the new roadway alignment super-elevation, from one sloping inwards toward the slope toe to one sloping away from the slope toe and towards the traffic, greatly increases the run-out distance to a point where rocks would likely be all over both lanes of traffic. This situation requires rockfall mitigation without relying solely on catchment. Based on an estimated average rockfall diameter of about 4 to 6 inches, a Double Twisted Wire Mesh (DTWM) Attenuator is considered to be the optimal option for mitigation. This involves a DTWM mesh fastened to a wire rope cable that is supported on the tops of posts that are founded on the slope a short distance above the top of cut. The posts serve to elevate the mesh sufficiently above the ground in the vicinity of the top of cut so that rocks falling or rolling from the native slopes above are caught by the mesh and kept beneath it. These rocks, and others that may come loose from the new cut slope face, descend along the cut face behind the mesh at a greatly reduced speed until exiting the restraint of the mesh near the bottom of the slope where they are dropped into a small unpaved catchment that retains them and prevents them from rolling onto the paved shoulder.

The recommended design for this system involves 6-foot high posts placed approximately every 50 ft above the top of cut between stations 905+50 and 910+75. The DTWM mesh extends from the top wire rope to 2 vertical ft above the toe of the cut slope. The minimum catchment width at the bottom of the slope should be at least 6 ft in width, which includes 3 ft of unpaved shoulder sloping back towards the cut slope at 4:1 and 3 ft of shoulder backing at 20:1. Approximate locations of posts and lateral cable anchors are listed in Table 1. Plan sheets showing design drawings and details for the DTWM Attenuator were provided by OGDN.

South Cut.
The relative location of the south cut slope and the new roadway alignment created a substantial amount of catchment at the toe of the cut slope, because of a design standards mandate to create sufficient sight distance for the concave turn on which it is located. The maximum amount of catchment required for the highest portion of the south cut slope is about 28 ft, based on tables and charts used to determine catchment needs (Pierson et. al., 2001). The initial catchment design for the south cut slope provided by design surpasses all rockfall catchment needs determined according to these tables and charts. Therefore, no additional rockfall mitigation measures are required for this slope beyond catchment. Table 2 above shows minimum catchment widths for the south cut slope by stationing.

8.1.5 Post-Construction Erosion

Erosion from a majority of the cut surfaces of both the south and north cut slopes is expected to be minimal to non-existent, because of the rocky nature of these areas. The upper reaches of these cut slopes, which are likely composed of soil and weathered rock material, may be slightly susceptible to erosion and should be protected during the first post-cutting wet season by some type of fiber mulch or other similar erosion protection substance.

8.2 Embankments

Fills are proposed in the southern and northern areas of the project. Near the southern abutments a fill is proposed primarily as backfill to partially bury the southern abutment of the new bridge. This fill is proposed at a slope ratio of 1.5:1 (33.7°) or flatter, with a maximum height of about 25 ft. In the northern end of the project beyond the north cut, between stations 911+00 and 918+25, proposed fills of 1.8:1 (29.0°) or flatter are proposed to complete the conform of the new alignment to the existing highway. These fills have a maximum height of 33 ft and a maximum vertical column thickness of 26 ft.

8.2.1 Embankment Material

The northern fills are likely to be built from material excavated from the north cut, due to their proximity. This cut will produce fresh rock, weathered rock, and soil in roughly equal proportions. The angularity of this material will easily give it a φ angle of at least 36° to 38°, while the non-fat clay will provide a moderate amount of cohesion to the mass. Fills constructed of this material at 90% relative compaction or greater will be stable at slope ratios of 1.5:1 or flatter.

The proposed 1.5:1 backfill around the southern bridge abutment will likely be constructed either from material excavated during the construction of the abutment or else material excavated from the southern cut, due again to the relative proximity of these sources. The angularity of the rocky portion of this material will easily give it a φ angle between 35° to 38°, depending upon the relative percentages of excavated rock and soil used. The non-fat clay will provide a moderate amount of cohesion to the mass. Both of these material sources will be stable when compacted at 90% relative compaction or greater at a slope ratio of 1.5:1 or flatter.

8.2.2 Embankment Stability Analysis

No formal slope stability analyses were performed for the proposed fills because of their relatively flat slope ratios, moderate heights, the quality of the material to be used, and engineering judgment based on experience with these types of materials and fills.

8.2.3 Embankments - Founding and Settlement

The founding conditions beneath the proposed northern fills consist of 2 to about 15 ft of soil overlying competent bedrock of the Baird Formation, the surface of which lies fairly flat to gently sloping. The soils vary from local creek bed deposits composed of gravels and cobbles with silty sands to clayey sands primarily of colluvial origin. These soils pose no liquefaction danger due to the gravel and/or cobble components. Most of these soils will settle simultaneously with the loading of the fill construction, with post-construction settlement expected to be minimal (maximum of 1.0 inches).

The southern fill is essentially backfilling areas that will have been excavated during construction and have proven to be stable prior to the excavation. Some fill material will be constructed against the abutment structure, which is founded on piles and stable.

8.2.4 Embankments-Erosion

The soil portions of the fill material will be slightly susceptible to erosion, particularly during the first post-construction season, until vegetation takes hold, after which the risk should dissipate.

8.2.5 Embankments - Drainage

Drainage should not be needed for the southern fill around the southern abutment, as there is no water source other than the local surface runoff, all of which will shed off the fill and continue downslope as surface runoff.

The northern fills will be constructed, at least partly, in an area of an ephemeral creek and will require an extension of the existing culvert to maintain flow.

8.3 Construction Access Geotechnical Zones

Construction access geotechnical zones (CAGZ) have been delimited on Plate 6 to provide specific localized geological and geotechnical information to assist the contractor in the development and design of construction access for the bridge bents, as well as founding conditions for bridge falsework.

Twenty-eight CAGZ zones are shown on Plate 6 and described below. Boundaries do not necessarily imply a definitive, sharp, or absolute change in any or all properties described between one zone and its neighboring zone. Zone boundaries were drawn and areas designated based on general changes in one or more of the following: topography, slope ratio, aspect, overburden type, soil type, bedrock geology, bedrock strength, bedrock structure, relation to bridge foundations and abutments, relation to viaduct structure, and likely potential for use in construction access. Depths are defined as perpendicular to the original existing ground surface (D_{og}) as shown in the example presented in Plate 7, or as depths in the vertical direction (D_v). Seismic velocities (V_p) described are compressional P wave velocities determined from refraction and/or tomographic surveys, either in the CAGZ zone being discussed or extrapolated/interpolated from a neighboring zone or zones. Two V_p values separated by a dash indicate a range of possible velocities. Two V_p values given as a fraction indicate a description of anisotropy, with the numerator representing the faster direction and the denominator the slower direction. The fraction is followed by an orientation in parentheses approximating the fast direction. Rippabilities are based on both the conservative scale utilized by the Caltrans Geophysics Branch (R_{ct}) and the less conservative Caterpillar Scale (R_{cp}), which is rock-specific and based on a larger dozer. Both rippability scales are explained in section 8.1.2. Estimated allowable bearing capacities (ABC) of bedrock provided are based on direct observations of outcrops and drilling cores, seismic refraction and tomography data, published estimates of ABC values and the application, when deemed suitable, of correction factors for parameters such as RQD, bedding, and weathering (Wyllie, 1999; Goodman, 1989; Peck et al, 1974; Woodward, et al, 1972; and Thorburn, 1966). ABC estimates in some zones are based on reasonable geological or geophysical extrapolation or interpolation from other CAGZ zones. Rock types, structural geology, soil types and thicknesses provided are based as much as possible on direct observations (surface exposure, incised faces in ravines, and drill cores). but reasonable extrapolation/interpolation of outcrop and seismic information from neighboring CAGZ zones was employed to estimate these characteristics in many zones. Subsurface data (drill cores, seismic data) was obviously not procured for every patch of terrain, as that would have been nearly impossible logistically and technically, as well as economically prohibitive. As mentioned, geologic and geophysical interpolation and extrapolation was employed to extend coverage of available subsurface and outcrop (including soil profiles in incised gullies) information to nearby areas, and because of this some subsurface conditions described here may vary from conditions discovered during construction.

CAGZ-1.

Topography:

Level ground giving way to steeply sloping ground (40° to 65° to the west) in the southeast part of zone.

Geo Profile:

Level area – Poorly graded gravel with sand (GP) (fill), and well-graded gravel with sand (GW) over moderately weathered bedrock (R_{ct} = Difficult, R_{cp} = rippable) at unknown depths over slightly weathered to fresh bedrock (R_{ct} = unrippable, R_{cp} = marginally-rippable) likely at least 15 ft to 20 ft D_v.

Bedrock:

Dipping slightly to the south. Likely slightly metamorphosed sandstone, tuffaceous sandstone, welded tuff (Likely V_p = 7800 fps – 10,800 fps). Beds, when present, likely at (40-65/115-135). ABC (for slightly weathered to fresh) = 20 ksf.

CAGZ-2.

Topography:

West side is level ground; East side sloping to the east, slope is moderate (20° -30°) in north side increasing to 40-45°+ moving south.

Geo Profile:

Level area – Shallow poorly graded gravel with sand (GP), and well-graded gravel with sand (GW) over moderately weathered bedrock (R_{ct} = Difficult, R_{cp} = rippable) to 15 ft to 30 ft D_v; Slightly weathered to fresh bedrock (R_{ct} = unrippable, R_{cp} = non-rippable) below.

Sloping area – Shallow well-graded gravel with sand (GW), well-graded gravel with silt and sand (GW-GM), and well-graded gravel with clay and sand (GW-GC), possibly grading into cobbles with a matrix of well-graded gravel with silt and sand (GW-GM) near the bottom of slope. Intensely to moderately weathered bedrock and/or boulder-cobble colluvium (R_{ct} = easily ripped, R_{cp} = rippable) at D_{og} = 3 ft to 10 ft; Slightly weathered to fresh bedrock (R_{ct} = unrippable, R_{cp} = non-rippable) at 15 ft to 25 ft D_{og}, likely shallowing near bottom of slope.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone, welded tuff. Beds, when present, likely at (40-65/102-140). ABC (for slightly weathered to fresh) = 25 ksf.

CAGZ-3.

Topography:

West side is level ground; East side sloping to the east at 40°-50° in steep eroded ravine.

Geo Profile:

Level area – Shallow poorly graded gravel with sand (GP), and well-graded gravel with sand (GW) over moderately weathered bedrock (R_{ct} = Difficult, R_{cp} = rippable) to 15 ft to 30 ft D_v; Slightly weathered to fresh bedrock (V_p = 11,400fps/10,800 fps; EW; R_{ct} = unrippable, R_{cp} = non- rippable) below.

Sloping area – Over burden mostly removed. Very shallow well-graded gravel with sand (GW) grading into cobbles with a matrix of well-graded gravel with silt and sand (GW-GM) near the bottom of slope. Intensely to moderately weathered bedrock and/or boulder-cobble colluvium (R_{ct} = easily ripped, R_{cp} = rippable) at D_{og} = 0.5 ft to 5 ft; Slightly weathered to fresh bedrock (V_p = 11,400fps/10,800 fps; EW; R_{ct} = unrippable, R_{cp} = non-rippable) at 10 ft to 20 ft D_{og}, likely shallowing near bottom of slope.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone, welded tuff. Beds, when present, likely at (40-65/102-140). ABC (for slightly weathered to fresh) = 25 ksf.

CAGZ-4.

Topography:

West side is level ground; East side sloping to the east at 40°-50°.

Geo Profile:

Level area – Shallow poorly graded gravel with sand (GP), and well-graded gravel with sand (GW) over moderately weathered bedrock (R_{ct} = Difficult, R_{cp} = rippable) to 15 ft to 30 ft D_v; Slightly weathered to fresh bedrock (R_{ct} = unrippable, R_{cp} = non-rippable) below.

Sloping area – Shallow well-graded gravel with sand (GW), well-graded gravel with silt and sand (GW-GM), and well-graded gravel with clay and sand (GW-GC), possibly grading into cobbles with a matrix of well-graded gravel with silt and sand (GW-GM) near the bottom of slope. Intensely to moderately weathered bedrock and/or boulder-cobble colluvium (R_{ct} = easily ripped, R_{cp} = rippable) at D_{og} = 3 ft to 10 ft; Slightly weathered to fresh bedrock (V_p = 11,400fps/10,800 fps; EW; R_{ct} = unrippable, R_{cp} = non-rippable) at 15 ft to 25 ft D_{og}, likely shallowing near bottom of slope.

Bedrock:

Likely slightly metamorphosed sandstone. Beds, when present, likely at (40-65/102-140). ABC (for slightly weathered to fresh) = 25 ksf.

CAGZ-5.

Topography:

Slope dipping (40° - 50°) generally at about 110° azimuth.

Geo Profile:

Soil shallow to non-existent, well-graded gravel with sand (GW) over thin zone of intensely weathered bedrock (R_{ct} = easily ripped, R_{cp} = rippable) over slightly weathered to fresh bedrock (V_p = 9,150fps; R_{ct} = unrippable, R_{cp} = rippable) at 7 ft to 12 ft D_{og}, likely shallowing near bottom of slope.

Bedrock:

Likely slightly metamorphosed sandstone. Beds, when present, likely at (40-50/102-115). ABC (for slightly weathered to fresh) = 20 ksf.

CAGZ-6.

Topography:

Slopes towards the east from bottom of wall at 40° to 60°.

Geo Profile:

Fill over native soil over moderately to intensely weathered bedrock over east sloping slightly weathered to fresh bedrock (V_p = 7,050fps - 9,150fps; R_{ct} = unrippable, R_{cp} = rippable) at D_{og} of 8 ft to 15 ft. Fill/native soil likely composed of cobbles and boulders with a matrix of poorly graded gravel with sand (GP) and well-graded gravel with sand (GW).

Bedrock:

Likely slightly metamorphosed sandstone. Beds, when present, likely at (40-56/102-115). Bedrock likely slopes eastward at 30° to 40°. ABC (for slightly weathered to fresh) = 15 ksf.

CAGZ-7.

Gravity retaining wall likely founded on slightly metamorphosed sandstone bedrock. This are will not be excavated or drilled until after completion of bridge, after which this wall shall be removed.

CAGZ-8.

Topography:

Slopes to the east-northeast at 40° to 45°.

Geo Profile:

Colluvium with local moderately to intensely weathered bedrock below surface: Well-graded gravel with silt and sand (GW-GM), poorly graded sand (SP), well graded sand with gravel (SW), and well-graded gravel with clay and sand (GW-GC), with cobbles and boulders locally to 50%; sand to boulders are subangular to angular. Slightly to moderately weathered bedrock (V_p = 8400fps/7050fps; R_{ct} = unrippable, R_{cp} = rippable) at 12 ft to 30 ft D_{og},

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone. Beds, when present, likely at (40-65/102-140). Bedrock dips 40° to 45° predominantly at approximately 70° azimuth. ABC (for slightly to moderately weathered bedrock) = 18 ksf.

CAGZ-9.

Topography:

Center of zone slopes to the east-northeast at 40° to 45°; north part of zone slopes to the north at 40°; south part of zone slopes to the southeast at 45°.

Geo Profile:

Colluvium with local moderately to intensely weathered bedrock below surface: Well-graded gravel with silt and sand (GW-GM), poorly graded sand (SP), well graded sand with gravel (SW), and well-graded gravel with clay and sand (GW-GC), with cobbles and boulders locally to 50%; sand to boulders are subangular to angular. Slightly to moderately weathered bedrock (V_p = 8400fps/7050fps; R_{ct} = unrippable, R_{cp} = rippable) at 10 ft (northeast corner of zone) to 12 ft (center line station 900+25) to 25 ft (center line station 899+25 to 899+50) D_{og}.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone. Beds, when present, likely at (40-65/102-140). Bedrock dips predominantly 40° to 45° at about 70° azimuth. ABC (for slightly to moderately weathered bedrock) = 18 ksf.

CAGZ-10.

Topography:

North half of zone slopes primarily to the east-northeast at 40° to 45° with the northern edge sloping down into gully; southern half of zone slopes to the east at 40°;

Geo Profile:

Colluvium with local moderately to intensely weathered bedrock below surface. Northern end of zone: 1 ft to 2 ft of well-graded gravel with silt and sand (GW-GM) and some cobbles over 8 to 12 ft of well-graded gravel with clay and sand (GW-GC) over cobbles and boulders with well-graded gravel with silt and sand (GW-GM) matrix; sand to boulders subangular to angular; remainder of zone is likely similar. Slightly to moderately weathered bedrock (V_p = 8400fps/7050fps; R_{ct} = unrippable, R_{cp} = rippable) at 10 ft to 14 ft D_{og} on north end of zone. Bedrock shallows generally at east side of zone with small outcrops present at surface.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone. Beds, when present, likely at (40-65/102-140). Bedrock dips 35° to 40° predominantly at about 70° azimuth. ABC (for slightly to moderately weathered bedrock) = 18 ksf.

CAGZ-11.

Topography:

Center of zone slopes to the east-northeast (about 70° azimuth) at about 30°.

Geo Profile:

Colluvium with local moderately to intensely weathered bedrock. Well-graded gravel with silt and sand (GW-GM), poorly graded sand (SP), and well graded sand with gravel (SW), with cobbles and boulders locally from 5% to 20%; sand to boulders are subangular to angular. Fresh to moderately weathered bedrock (V_p = (11,000 fps-8400fps) / 7050fps; R_{ct} = unrippable, R_{cp} = some areas marginally rippable, some areas non-rippable) at 15 ft to 30 ft D_{og} across western part of zone thinning to 5ft to 20 ft D_{og} across the eastern part of zone.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone. Beds, when present, likely at (40-65/102-140). Bedrock dips 27° to 30° predominantly at about 70° azimuth. ABC (for fresh to moderately weathered bedrock) = 20 ksf.

CAGZ-12.

Topography:

Center of zone slopes to the east-northeast (about 70° azimuth) at about 30°.

Geo Profile:

Colluvium with local moderately to intensely weathered bedrock below surface and at surface. Well-graded gravel with silt and sand (GW-GM), and well graded sand with gravel (SW), with cobbles and boulders locally; sand to boulders are subangular to angular. Fresh to slightly weathered bedrock (V_p = (11,000-8400fps) / 7050fps; R_{ct} = unrippable, R_{cp} = some areas marginally rippable, some areas non-rippable) at 5 ft to 18 ft D_{og}.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone. Beds, when present, likely at (40-65/102-140). Bedrock dips 27° to 30° predominantly at about 70° azimuth. ABC (for fresh to slightly weathered bedrock) = 20 ksf.

CAGZ-13.

Topography:

Center of zone slopes to the east-northeast (about 70° azimuth) at about 30°.

Geo Profile:

Colluvium with local moderately to intensely weathered bedrock below surface and at surface. Well-graded gravel with silt and sand (GW-GM), and well graded sand with gravel (SW), with cobbles and boulders locally; sand to boulders are subangular to angular. Fresh to slightly weathered bedrock (V_p = (11,000-8400fps) / 7050fps; R_{ct} = unrippable, R_{cp} = some areas marginally rippable, some areas non-rippable) at 3 ft to 12 ft D_{og}.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone, possible thin shale interbeds. Beds, when present, likely at (50-65/102-120). Bedrock dips 27° to 30°

predominantly at about 70° azimuth. ABC (for fresh to slightly weathered bedrock) = 20 ksf.

CAGZ-14.

Topography:

Slopes to the east-northeast at 29° to 33°.

Geo Profile:

Colluvium: Well-graded gravel with silt and sand (GW-GM), poorly graded sand (SP), well graded sand with gravel (SW), and well-graded gravel with clay and sand (GW-GC), with cobbles and boulders locally to 50%; sand to boulders are subangular to angular. Fresh to slightly weathered bedrock (V_p = (11,500 fps-8400fps) / 7050fps; R_{ct} = unrippable, R_{cp} = some areas marginally rippable, some areas non-rippable) at 15 ft to 30 ft D_{og} across western part of zone thinning to about 8 ft to 20 ft D_{og} across the eastern part of zone.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone with minor thinly and moderately bedded shale interbeds. Beds, when present, likely at (40-65/102-140). Bedrock dips about 27° predominantly at approximately 70° azimuth. ABC (for fresh to slightly weathered bedrock) = 25 ksf.

CAGZ-15.

Topography:

Slopes to the east-northeast (about 80° azimuth) at 29° to 33°.

Geo Profile:

Colluvium: Well-graded gravel with silt and sand (GW-GM), poorly graded sand (SP), well graded sand with gravel (SW), and well-graded gravel with clay and sand (GW-GC), with cobbles and boulders locally to 50%; sand to boulders are subangular to angular. Slightly to moderately weathered bedrock (V_p = (7400fps-8400fps) / 6670fps; R_{ct} = unrippable, R_{cp} = rippable) at 5 ft to 8 ft D_{og} in northern part of zone and 8 ft to 15 ft D_{og} in southern half of zone.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone with lesser thinly and moderately bedded shale interbeds. Beds, when present, likely at (40-65/102-140). Bedrock dips about 27° predominantly at approximately 80° azimuth. ABC (for slightly to moderately weathered bedrock) = 15 ksf.

CAGZ-16.

Topography:

Slopes to the east-northeast (about 80° azimuth) at 25° to 33°.

Geo Profile:

Colluvium with local moderately to intensely weathered bedrock below and at surface. Well-graded gravel with silt and sand (GW-GM) and cobbles and boulders over well-graded gravel with clay and sand (GW-GC); sand to boulders subangular to angular. Slightly to moderately weathered bedrock (V_p = 7400fps/6670fps; R_{ct} =

unrippable, R_{cp} = rippable) at 3 ft to 7 ft D_{og} in northern part of zone and 8 ft to 18 ft D_{og} in southern half of zone.

Bedrock:

Likely slightly metamorphosed sandstone, tuffaceous sandstone with lesser thinly and moderately bedded shale interbeds. Beds, when present, likely at (40-65/102-140). Bedrock surface dips about 27° to 30° predominantly at approximately 80° azimuth. ABC (for slightly to moderately weathered bedrock) = 15 ksf.

CAGZ-17.

Topography:

Slopes/faces to the east, northeast, and north at 50° to 90°.

Geo Profile:

Slightly to intensely weathered bedrock cut slopes and native slopes. R_{ct} = Easily ripped to moderately difficult to rip, R_{cp} = rippable. At D_{og} between 5ft and 10 ft R_{ct} = moderately difficult to rip to not rippable, R_{cp} = rippable.

Bedrock:

Slightly metamorphosed sandstone, and tuffaceous sandstone in northern part of zone, giving way to shale in southern part of zone. Shale beds dip at 25-30/155-165. ABC for northern part = 15 ksf and 10 ksf for southern part.

CAGZ-18.

Topography:

Sloping about 25° to the north-northwest beneath southern abutment; sloping about 18° to the southeast in the northern part; Level to gradually sloping to the northeast in central and southern part.

Geo Profile:

Colluvium consisting of well-graded gravel with silt and sand (GW-GM) and well-graded sand with gravel (SW) 0 ft to 6 ft D_v. 80% of ground cover; slightly to intensely weathered bedrock 20% of surface cover.

Bedrock:

Slightly metamorphosed sandstone, and tuffaceous sandstone in northern part of zone, giving way to shale in southernmost part of zone. Shale beds dip at 25-30/155-165. Northern part R_{ct} = moderately difficult to rip to not rippable, R_{cp} = rippable to marginally rippable. Central part R_{ct} = moderately difficult to rip to not rippable; R_{cp} = rippable to non-rippable. Southern part R_{ct} = easily ripped to moderately difficult to rip; R_{cp} = rippable. ABC = 20 ksf for northern part, 18 ksf for central part, and 10 ksf for southern part.

CAGZ-19.

Topography:

Slopes to the northeast at 36° to 38°. Contains gully 3 ft to 7 ft D_v.

Geo Profile:

Colluvium: Well-graded gravel with silt and sand (GW-GM), poorly graded sand (SP), well graded sand with gravel (SW), and well-graded gravel with clay and sand (GW-GC), all with cobbles and boulders locally to 50%; sand to boulders are subangular to angular. Slightly weathered to moderately weathered bedrock (V_p = 7400fps/6670fps; R_{ct} = unrippable, R_{cp} = rippable) at 18 ft to 25 ft D_{og} across upper third of slope, at 8 ft to 24 ft D_{og} across center of slope, at 5 ft to 22 ft D_{og} across lower slope.

Bedrock:

Northern half to two-thirds of zone: likely slightly metamorphosed sandstone, tuffaceous sandstone with thinly and moderately bedded shale interbeds. Beds, when present, likely at (40-65/102-140). Southern third of zone: Likely predominantly shale with minor slightly metamorphosed sandstone and tuffaceous sandstone. Beds, when present, likely at (40-65/102-140) or at 25-30/155-165. Bedrock dips about 35° predominantly at approximately 40° to 50° azimuth. ABC (for slightly to moderately weathered bedrock) = 15 ksf.

CAGZ-20.

Topography:

Slopes to the northwest at 10° to 30°. Easternmost part of zone slopes to the north and northeast at 60° to 85°.

Geo Profile:

Shallow soil overburden of well-graded gravel with silt and sand (GW-GM) and well-graded gravel with clay and sand (GW-GC). Sand and gravel subangular to angular. Moderately weathered to intensely weathered bedrock (V_p = 4920fps/4680fps; R_{ct} = moderately difficult to rip, R_{cp} = rippable) at 1ft to 15 ft D_{og}.

Bedrock:

Likely all thinly and moderately bedded shale. Beds likely at (25-30)/(155-165). Bedrock surface dips about 10° to 30° to the northwest, except in easternmost part of zone where it likely dips about 30° to 85° to the north and northeast. ABC (for moderately weathered to intensely weathered bedrock) = 10 ksf.

CAGZ-21.

Topography:

Slopes to the north at 32° to 38°.

Geo Profile:

Well-graded gravel with silt and sand (GW-GM) and minor cobbles. Sand, gravel, and cobbles subangular to angular. Moderately weathered to intensely weathered bedrock (V_p = 4920fps/4680fps; R_{ct} = moderately difficult to rip, R_{cp} = rippable) at 12 ft to 20 ft D_{og}. Slightly to moderately weathered bedrock (V_p = 6565fps; R_{ct} = not-rippable, R_{cp} = rippable) at 16 ft to 30 ft D_{og}.

Bedrock:

Likely thinly to thickly bedded metashale with minor metasandstone interbeds. Beds likely at (10-30)/(135-165). Moderately weathered bedrock surface dips about 30° to 60° to the north in southern part of zone and about 10° to 20° to the north in northern part of zone. Fresh to slightly weathered bedrock surface dips about 30° to 60° to the north in southern part of zone and about 10° to 20° to the north in northern part of zone. ABC (for moderately weathered to intensely weathered bedrock) = 10 ksf. ABC for slightly to moderately weathered bedrock =17 ksf.

CAGZ-22.

Topography:

Slopes to the north at 37° to 42°.

Geo Profile:

Well-graded gravel with silt and sand (GW-GM) and cobbles and boulders. Sand, gravel, cobbles, and boulders subangular to angular. Moderately weathered to intensely weathered bedrock (V_p = 4920fps/4680fps; R_{ct} = moderately difficult to rip, R_{cp} = rippable) likely at 12 ft to 20 ft D_{og}. Slightly weathered bedrock (V_p = 6565fps; R_{ct} = not-rippable, R_{cp} = rippable) likely at 15 ft to 40 ft D_{og}.

Bedrock:

Likely thinly to thickly bedded shale, some minor sandstone interbeds possible. Beds likely at (25-30)/(110-165). Bedrock surface likely dips about 10° to 30° to the north. ABC (for moderately weathered to intensely weathered bedrock) = 10 ksf. ABC for slightly weathered bedrock =20 ksf.

CAGZ-23.

Topography:

Slopes to the north at 10° to 40°.

Geo Profile:

Shallow soil overburden of silty sand with gravel and cobbles (SM) over well-graded gravel with silt and sand (GW-GM) and cobbles. Sand and gravel subangular to angular. Moderately weathered to intensely weathered bedrock (V_p = 4920fps/4680fps; R_{ct} = moderately difficult to rip, R_{cp} = rippable) likely at 5 ft to 20 ft D_{og}.

Bedrock:

Likely thinly and moderately bedded shale, some minor sandstone interbeds possible. Beds likely at (25-30)/(135-165 and/or 70-85). Bedrock surface dips about 10° to 30° to the north, except in northwesternmost part of zone, where it likely dips about 30° to 85° to the north. ABC (for moderately weathered to intensely weathered bedrock) = 10 ksf.

CAGZ-24.

Topography:

Slopes to the north at 35° to 40°.

Geo Profile:

Well-graded gravel with silt and sand (GW-GM) and minor cobbles. Sand, gravel, and cobbles subangular to angular. Moderately weathered to intensely weathered bedrock (V_p = 4920fps/4680fps; R_{ct} = moderately difficult to rip, R_{cp} = rippable) at 10 ft to 20 ft D_{og}. Slightly to moderately weathered bedrock (V_p = 6565fps; R_{ct} = not-rippable, R_{cp} = rippable) at 18 ft to 30 ft D_{og}.

Bedrock:

Likely thinly to thickly bedded metashale with minor metasandstone interbeds. Beds likely at (10-30)/(135-165). Moderately weathered bedrock surface dips about 30° to the north. Fresh to slightly weathered bedrock surface dips about 30° to the north.

ABC (for moderately weathered to intensely weathered bedrock) = 10 ksf. ABC for slightly to moderately weathered bedrock =15 ksf.

CAGZ-25.
Topography:
Slopes to the north at 35° to 40°.
Geo Profile:
Well-graded gravel with silt and sand (GW-GM) and minor cobbles. Sand, gravel, and cobbles subangular to angular. Moderately weathered to intensely weathered bedrock (V_p = 4920fps/4680fps; R_{ct} = moderately difficult to rip, R_{cp} = rippable) likely at 12 ft to 24 ft D_{og}. Slightly weathered bedrock (V_p = 6565fps; R_{ct} = not-rippable, R_{cp} = rippable) likely at 18 ft to 40 ft D_{og}. Bedrock generally shallows at downslope end of zone.
Bedrock:
Likely thinly to thickly bedded metashale with minor metasandstone interbeds. Beds likely at (10-30)/(135-165). Moderately weathered bedrock surface likely dips about 30° to 60° to the north in southern part of zone and about 10° to 20° to the north in northern part of zone. Fresh to slightly weathered bedrock surface likely dips about 30° to 60° to the north in southern part of zone and about 10° to 20° to the north in northern part of zone. ABC (for moderately weathered to intensely weathered bedrock) = 10 ksf. ABC for slightly weathered bedrock = 17 ksf.

CAGZ-26. (continues southeastward out of field of view in Plate 6 to station 891+50)
Topography:
Slopes to the south at about 5°.
Geo Profile:
Silty sand with gravel and cobbles (SM), well graded gravel with sand and clay (GW-GC), well-graded gravel with silt and sand (GW-GM), well-graded sand with clay and gravel (GW-SC), and minor cobbles. Sand, gravel, and cobbles subangular. Moderately to intensely weathered bedrock (V_p = 4500fps-49fps; R_{ct} = moderately difficult to rip, R_{cp} = rippable) at 20 ft to 35 ft D_v Slightly weathered bedrock (V_p = 6565fps; R_{ct} = not-rippable, R_{cp} = rippable) at 35 ft to 60 ft D_{og}.
Bedrock:
Thinly bedded to massive metashale. Beds likely at (10-35)/(100-155). Intensely weathered bedrock surface dipping gently to the north in northern part of zone and to the south in the southern part of zone, reaching a D_v of about 36 ft at station 891+50. Slightly weathered bedrock surface likely dipping gently to the north in northern part of zone and gently to the south in the southern part of zone. ABC for slightly weathered bedrock =18 ksf.

CAGZ-27.
Topography:
Slopes to the north at 24° to 32°.

Geo Profile:

Silty sand with gravel (SM), well-graded gravel with silt and sand (GW-GM), well-graded gravel with sand (GW), well-graded gravel with sand and clay (GW-GC) and minor cobbles. Sand, gravel, and cobbles angular to subrounded. Moderately weathered to decomposed bedrock (V_p = 3200fps-4680fps; R_{ct} = easily ripped to moderately difficult to rip, R_{cp} = rippable) at 10 ft to 18 ft D_{og}. Fresh to slightly weathered bedrock (V_p = 6565fps; R_{ct} = not-rippable, R_{cp} = rippable) at 60 ft D_v.

Bedrock:

Thinly bedded to massive shale. Beds likely at (10-30)/(100-155). Moderately weathered bedrock surface dips about 30° to the north. Fresh to slightly weathered bedrock surface dips about 30° to the north. ABC (for moderately weathered to intensely weathered bedrock) = 10 ksf. ABC for fresh to slightly weathered bedrock =18 ksf.

CAGZ-28. (continues southeastward out of field of view in Plate 6 to station 892+50)

Topography:

Slopes to the north at 24° to 27°.

Geo Profile:

Silty sand with gravel (SM), well-graded gravel with silt and sand (GW-GM), well-graded gravel with sand (GW), well-graded gravel with sand and clay (GW-GC) and minor cobbles. Sand, gravel, and cobbles angular to subrounded. Intensely weathered to decomposed bedrock (V_p = 3200fps; R_{ct} = easily ripped, R_{cp} = rippable) at 10 ft to 15 ft D_{og}. Fresh to slightly weathered bedrock (V_p = 6565fps; R_{ct} = not-rippable, R_{cp} = rippable) likely at 60+ ft D_v.

Bedrock:

Thinly bedded to massive shale. Beds likely at (10-30)/(100-155). Intensely weathered to decomposed bedrock surface dips about 26° to the north. ABC (for intensely weathered to decomposed bedrock) = 6 ksf.

8.4 Impact of Bridge Construction and Completion upon the Railroad Infrastructure and Existing Viaduct - Analysis and Design

Section 8.4 presents discussion and analyses of two possible impacts that the construction of the new bridge could have upon the railroad infrastructure and/or the existing viaduct, together with some mitigation measures where necessary and/or feasible. The first possible impact is construction-induced rockfall and debris falling, or being accidentally jettisoned, down the steep slopes below the proposed bridge where it might contact the tunnel portal structure or the tracks. The second potential impact involves construction vibrations and their effects, if any, upon the railroad tunnel and existing viaduct. Potential loading effects upon the railroad tunnel created by the construction and presence of the new bridge are addressed in the separate memo by Song (2015).

8.4.1 Analysis and Design of Construction-Induced Rockfall and Debris Mitigation

The first step taken in the assessment of possible construction-induced rockfall and debris fall and its effect upon the railroad infrastructure was a thorough field reconnaissance of the terrain defined earlier as the bridge slope, as well as the terrain downslope of the bridge slope, including the railroad tunnel portal and the exposed exterior concrete tunnel structure. This reconnoiter extended laterally across the bridge slope.

The topographic map of the site produced internally by Caltrans' Office of Surveying Engineering was used as a base map upon which observations and refinements were made involving directions, slopes, and locations of representative potential fall lines. Cross-section profiles of representative potential fall lines were produced based on the initial survey work and then refined based on field observations and measurements of local topographic features. In addition, other data pertinent to rockfall was collected, including average and maximum rock size and shape, surface roughness (of the slope) and natural source areas. A few semi-rounded boulders with maximum diameters of about 2.5 ft were observed during field work about 10 ft from the concrete tunnel structure in the area where fall line 5 (on Plate 8) intersects the primary surface flow channel. No clear evidence was found of any rocks having made it further down towards the railroad infrastructure.

Plate 8 shows topography, a layout view of the site, potential representative fall lines, and a layout of a Cable Mesh Attenuator (CMA) system, which is the recommended system for preventing construction-induced rockfall and debris fall from impacting the railroad infrastructure. Plate 9 shows cross-section profiles of the fall lines demarcated on Plate 8.

Some basic rockfall analysis utilizing CRSP (Colorado Rockfall Simulation Program) in two-dimensional mode corroborated the findings developed in the field that no natural rockfall coming off the bridge slope could possibly reach the railroad tracks, due to the topography, the concrete tunnel lining, and the portal structure. Construction-induced rockfall, where equipment accidentally mobilizes a rock with a high initial velocity, was simulated. With a very high velocity surcharge (surcharged above gravitational initiation) a few rocks eventually made it to the tracks, primarily through fall line 5. In this case, the rocks rolled upward across the south-facing slope situated north of the tunnel and tracks, before coming down in the vicinity of the railroad rockfall detection installation and tracks. Given enough artificially surcharged velocity a few rocks also made it to the tracks via fall line 4 by bouncing perfectly on to the top of the exposed portion of the concrete tunnel lining and then over the concrete walls of the concrete surface flow diversion structure and onto the tracks below. Even with substantial velocity surcharge, no rocks travelling down fall lines 2 or 3 made it to the tracks due to the large catchment formed at the bottom of the slope by the wide rock-lined drainage ditch. Surcharged rocks on fall line 3 were all stopped by the high concrete wall of the surface flow diversion structure. All rockfall events that travel the slope between fall lines 3 and 4 would fail to reach the tracks as they would be stopped by impact with the exposed portion of the railroad concrete tunnel structure. Even with a large velocity surcharge fall lines 6 and 7 saw no rocks reaching the tracks due to the high ridge located immediately to the north of the tracks. With significant velocities, however, the turn that occurs in fall line 6 may not occur, since the rock may avoid the topographic feature that would cause most rocks to veer left. In this case the trajectory may intersect and cross fall line 5 and hit the concrete infrastructure (stopping short of the tracks). It should be noted that in those cases above where significant velocity surcharges allowed a few rocks to reach the tracks (fall lines 5 and 4), such surcharges are considered possible only in the case of a rock or piece of debris being projected off the new bridge; no natural rockfall or construction-induced incident at or near the existing surface here could reach the tracks.

The decision to design and construct a CMA system in the location shown on Plate 8 was based on the fall line analysis. The potential size of the falling rocks and construction debris was the deciding factor for choosing a cable net mesh versus a lighter double-twisted wire mesh system (lacking the stronger cable net).

The analysis of fall lines has lead to the following conclusions:

1) Natural rockfall from the bridge slope is incapable of reaching the tracks, but can impact the concrete tunnel lining and water diversion structure if there is no CMA system in place. If the CMA system is in place, natural rockfall will not be capable of reaching any part of the railroad infrastructure, either during construction (with restraining cable) or after construction (without restraining cable).
2) Rockfall or debris fall at or slightly above the ground level that occurs during, and as a result of, construction, but does not begin with any substantial initial velocity boost, would fail to reach the tracks even if the CMA system were not in place. If the CMA system is in place, construction-induced rockfall and debris lacking any substantial initial velocity boost would fail to reach the railroad infrastructure (tracks and concrete structure).
3) Rockfall or debris fall at or slightly above the ground level that occurs during, and as a result of, construction, and is given a significant initial artificial velocity boost by the equipment, does not have the potential to reach the tracks if the CMA were not in place. It does, however, have potential to impact the concrete tunnel and water diversion structure without the CMA system in place. With the CMA system in place rockfall will have no potential of impacting any part of the railroad infrastructure.
4) Debris falling (with little to no horizontal velocity boost) off the new bridge, during or after construction, would behave as a moderately velocity surcharged object. It would not be capable of reaching the tracks but could possibly impact the concrete railroad structures if the CMA system were not in place. With the CMA system in place, this debris would have no potential to reach any part of the railroad infrastructure.
5) Debris coming off the new bridge with a significant horizontal velocity boost would behave as a high velocity surcharged object moving both horizontally and

vertically through space. It would have a low but feasible chance to reach the tracks and a very high chance of impacting the concrete railroad infrastructure if the CMA system were not in place. With the CMA system in place, the chances of the tracks and/or the concrete structures being impacted is reduced, but not eliminated, as this debris might potentially fly over the CMA system before hitting the ground.

A CMA system with 8-foot posts (high enough to capture any potential bounce heights) that is aligned along the slope as shown in Plate 8 will conservatively meet the needs of the rockfall protection system mentioned above. Unlike most attenuators that Caltrans has designed and built, this attenuator shall be restrained along the ground surface from post to post by a ½" wire cable (shown in the drawings as restraining cable), in order to completely retain material during the construction period. This may require periodic removal of collected material during construction by the contractor, depending upon the amount of material mobilized and collected by the CMA. Once construction is completed the contractor shall at least remove the restraining cable so that the attenuator can function as a true attenuator rather than a barrier that will collect rocks indefinitely. Following completion of the project, OGDN recommends that the unrestrained CMA system should be left in place, where it will provide substantial protection against natural rockfall. As an unrestrained attenuator the cable mesh absorbs the energy of falling rocks, dropping them to the ground where they can migrate downslope slowly beneath the hanging mesh that drapes the slope below. The removal of the restraint is imperative for the attenuator to function as designed. Failure to remove the restraining cable could adversely affect the railroad infrastructure in the long term.

The contractor shall be held responsible for cleaning and maintaining the cable mesh attenuator system during construction as needed.

The contractor shall still remain responsible for erecting protection to prevent objects from coming off the elevated bridge structure during its construction that might possibly be launched in a horizontal direction so as to pass directly over the cable mesh attenuator.

8.4.2 Construction Vibrations and Potential Effects Upon the Tunnel and Viaduct

Vibrations created from all possible constructions activities that might take place during this project are examined and discussed, with a focus on their potential effect upon the tunnel and the existing viaduct. Possible sources of such vibrations include trucks, tracked vehicles (bulldozers, excavators…), static and vibratory compaction equipment, impact and vibratory pile drivers, caisson and anchor drilling, and blasting.

This section will first introduce the universally accepted parameter for quantifying vibration and vibration damage, the peak particle velocity (PPV). This will be followed by a discussion and determination of the critical threshold PPV values that the tunnel and viaduct can be conservatively expected to tolerate without incurring any damage. Typical PPV values measured at certain set distances for the various construction sources that might possibly be utilized on this project are cited from the literature and discussed, including a brief review of the science and practice of construction blasting as it relates to PPV thresholds and monitoring. Then it will be shown how the set distances for these PPV values can be adjusted or scaled by the use of fairly simple established equations to determine the PPV values expected to occur at the tunnel and/or viaduct for each of the construction sources. A short discussion follows on seismic energy (vibrations) propagation through the ground. Using this background information, distances between the tunnel and each particular construction source will be determined that will not produce PPV values in excess of the threshold PPV values established for the tunnel. The same determination will also be performed for the viaduct. Finally, a brief discussion of vibration monitoring and a recommended monitoring approach for this project will be presented.

Peak Particle Velocity (PPV)

The PPV, which is essentially the peak velocity that a theoretical point in an elastic medium (such as the earth) moves during a vibration in question, is generally accepted as the most appropriate descriptor for evaluating the potential for building or structural damage (Jones and Stokes, 2004; Konya and Walter, 2002; Siskind, et. Al., 1980). This has been the universally accepted parameter for such evaluation for decades, having been adopted by highly respected organizations for over fifty years (Duvall and Fogelson, 1962). This parameter is the basis of the discussion in this section regarding the potential effects, risks, and safety of construction vibrations with respect to the railroad tunnel and existing viaduct.

Critical PPV Threshold Values for the Tunnel

Defining local critical threshold PPV values for both the tunnel and the existing viaduct is necessary in order to determine safe vibration levels for this project and how the PPV values for equipment and blasting can be applied, specified, and monitored. When examining the tunnel and its environment, there are three areas that are considered: the rock immediately surrounding the tunnel, the concrete tunnel lining structure, and resonance effects.

The most critical portion of rock surrounding the tunnel in regards to its strength and integrity, and the impact of construction vibrations upon it, is the rock that essentially forms the bridging or arching area around the top of the tunnel. Examination of the rock around the portal, in conjunction with the fairly detailed reconnoiter of the rocks within the project area by OGDN and an examination of rock cores obtained during the foundation drilling, leads to a fairly safe geologic extrapolation that the rock comprising this perimeter is strong, hard, and competent rock. Bedding structures, though certainly present, are tight and the contacts between individual beds are strongly connected. Fractures are likely present, but neither open nor continuous for any significant length.

Given the quality of rock described above, PPV values corresponding to various levels of damage to rock may be utilized to determine the appropriate PPV threshold that will protect the rock immediately surrounding the tunnel. Blasts in the near proximity of the free surface of hard competent rock that create PPV values between 12 in/s and 25 in/s at the rock surface were found to cause minor tensile slabbing or tensile fracturing (Bauer and Calder, 1978; Oriard, 1970). Bauer and Calder (1978) also concluded that PPV values less than 10 in/s do not cause surface damage to intact rock. PPV values over 12 in/s may cause rockfalls in unlined tunnels (Langefors et al, 1948), which corroborates the 12 to 25 in/s range for minor tensile slabbing or fracturing mentioned above. PPV values responsible for the fracturing of confined rock would certainly be higher than the values cited above for rock surfaces, but using these surface values as guides for threshold PPV values lends a safe conservatism to their usage here. In addition, reducing the value for undamaged rock of 10 in/s by 20% to 8 in/s provides additional conservatism. Therefore, a critical threshold PPV of 8 in/s is here defined for the rock portion of the tunnel environment.

The tunnel structure itself is essentially a 2-foot thick steel reinforced concrete tunnel lining as discussed in section 8.4.2. Steel reinforcement is considered a safe assumption, given the size and importance of the structure. Tart et al (1980) found that varying levels of explosives inside concrete cause a complete blowout at a PPV of 600 in/s, radial cracks at 375 in/s, spalling of a loose weathered concrete outer layer at 200 in/s, and spalling of fresh grout at 100 in/s. Explosives near concrete caused no damage to concrete while vibrating at a PPV of 100 in/s (Oriard and Coulson, 1988). Explosives inside a concrete block caused cracks at a PPV of 8.0 in/s (Bauer and Calder, 1977). According to Konya and Walter (2002), reinforced concrete bridges, which are unconfined by rock like a tunnel, would be protected quite conservatively by a threshold PPV of 2.0 in/s, a value that they state could likely safely be extended to about 5 in/s in most cases. Considering that the tunnel is confined and constrained by strong rock, it is highly likely that this upper level for concrete bridges would be applicable. Major cracking occurred in wall plaster at a PPV of 8.0 in/s (Northwood et. al., 1963), while a PPV of 7.6 in/s had a 50% probability of major plaster damage and a PPV of 5.4 in/s had a 50% probability of minor plaster damage, according to a study by E.I. du Pont de Nemours & Co. (1977). Based on these empirical data and the above discussion, a threshold PPV of 5.0 in/s might be defined here for the concrete tunnel lining, a value that might still be considered conservative considering that this is the threshold at which plaster might simply begin to crack and that vibrations with a PPV of 100 in/s were cited above as having caused no damage to concrete from nearby explosives. None-the-less, a far more conservative threshold PPV of 2.0 in/s shall be chosen here, in order to provide extremely safe standards for the tunnel and to remain in line and in accordance with past Caltrans projects and specifications, where PPV were defined at 2 in/s.

The limiting PPV for the tunnel environment is therefore 2.0 in/s, since the 2.0 in/s value chosen for the concrete lining is lower than the 5.0 decided upon for the rock.

The threat of resonant amplification of the construction vibrations affecting the tunnel is likely non-existent. Typically resonant effects instigated by earthborne vibrations occur with surface structures that have a natural vibratory frequency that is the same, or nearly the same, as the surface geological medium. Even if the natural frequency of the tunnel were the same as the surface soils in which vibrations are created during construction, the tunnel is heavily constrained by the surrounding rock and is not free to move in an amplified fashion. It is highly unlikely they could move to any significant extent independently of the medium or be subjected to vibration amplification.

Critical PPV Threshold Values for the Viaduct

When examining the viaduct and its environment, there are two areas to consider: the concrete of the structure itself, and resonance effects. Based on the discussion above regarding concrete PPV values and the statement by Konya and Walter (2004) that reinforced concrete bridges would minimally be covered by a PPV of 2.0 in/s and might still be covered by a PPV of 5.0 in/s, the PPV threshold for the viaduct is here defined as 2.0 in/s, which is still considered fairly conservative.

Potential resonance effects cannot be dismissed for the viaduct as easily as they were for the tunnel, primarily because of its being on the surface and not constrained by any surrounding medium. Blasting and other single impact types of construction sources do not possess the potential to set up a resonant effect because they are not repetitive, at least not on the time scale necessary to instigate harmonic cyclic amplification. Most construction sources that create continuous vibrations do not create vibrations of sufficient strength and of sufficiently low frequency to instigate widespread resonance with most soils, with the possible exception of some pile drivers.

Based on the above discussion a critical threshold PPV for both the tunnel and viaduct shall be defined for this project as 2.0 in/s. The higher PPV value of 5 in/s for the arch rock is neglected in lieu of the lower 2 in/s for the concrete tunnel. How this is applied and monitored is discussed later in this section.

Typical PPV Values for Construction Sources

Abundant research literature is available with empirical data regarding typical PPV values for various sources at given distances, as well as critical damage threshold values for various structures. The Federal Transit Administration (1995) lists PPV values in inches per second (in/s) produced by various pieces of construction equipment at a distance of 25 ft. Some of these are shown below in Table 3.

Table 3. Construction Equipment and PPV values at a distance of 25 ft (FTA, 1995).

Equipment	PPV (in/s)

Vibratory Roller	0.21
Large Bulldozer	0.089
Caisson Drilling	0.089
Loaded Trucks	0.076
Small Bulldozer	0.0003
Crack and Seat Operations	2.4

A Caltrans authorized study by Ames et. al (1976) found that at a distance of 10 ft a large dozer (D9 size or greater) and a large earthmover produced PPV values of 0.09 in/s and 0.03 in/s, respectively.

Although crack and seat operations are not part of this project, examining the vibrations generated by these operations and the equipment provides a perspective on relatively large PPV magnitudes seen during some construction operations. A crack and seat operation that used a Walker Megabreaker model 8-13000, which drops a 13,000 pound steel plate a distance of 4 ft, produced PPV values of 1.25 in/s at 40 ft and 0.62 in/s at 89 ft (Caltrans, 2000). A similar crack and seat operation produced a PPV of 2.99 in/s at 10 ft and 0.275 in/s at 38 ft (Ames et. Al, 1976). This same study found that an EMSCO pavement breaking machine produced PPV values of 2.88 in/s at 10 ft and 0.275 in/s at 38 ft.

Numerous PPV measurements taken on trains moving at different velocities (Ames et. al., 1976) revealed that train vibrations are generally greater than most, though not all, construction equipment. The greatest PPV observed was 0.17 in/s at a distance of 5 ft. Hendricks (2002) found PPV values for trains of 0.28 in/s at 15 ft and 0.18 in/s at 33 ft. Based on these and other findings it is assumed that the lining structure for tunnel # 3 is exposed daily to PPV values potentially as great as 0.28 in/s.

Impact and vibratory pile drivers, which might possibly be utilized on this project, are capable of producing considerable vibrations. Vibration levels generated by a 50,000 foot-pound impact pile driver produced a PPV of 0.24 in/s at 53 ft, and 0.079 in/s at 99 ft (Ames,et. al., 1976). Vibratory pile drivers demonstrate similarly high values as impact drivers, with the single caveat that resonance, which can increase the vibration amplitudes, is not allowed to set up between the object being driven and the soil in which it is being driven (Morris, 1991). Though the smallest and largest pile driver energy ratings are somewhere around 300 and 1,800,000 foot-pounds (Woods, 1997), respectively, the rated energies of most pile drivers are in the range of about 20,000 to 300,000 foot-pounds (Woods, 1997), which is up to 6 times the energy of the 50,000 foot-pound impact pile driver cited above. Multiplying this 0.24 in/s by 6 yields a potential PPV of about 1.44 in/s at 53 ft for what should likely be the largest possible pile driver to be used on this project.

Blasting is another important source of vibrations that might possibly be utilized by the contractor on the Sidehill project. The maximum PPV noted in a Caltrans study (Egan et. al., 2001) that recorded 27 construction blasts over a ten month period performed in different geology was 2.76 in/s at distances of 59 ft and 98 ft, with lesser PPV values observed at similar distances when the blast size was less and/or the blast was designed differently. Modern construction blasting is both a developed science and an art that, when practiced by a licensed blaster, can successfully and repetitively be conducted while maintaining PPV values below threshold defined levels at reasonably defined distances. Based on numerous studies and empirical data from multiple projects discussed by Konya and Walter (2004) it is quite feasible to perform construction blasting within 50 feet, or even less, of the tunnel or viaduct and confidently keep the PPV values at these structures below the 2.0 in/s defined earlier. Caltrans controlled blasting specifications define near field blasting as being within 30 ft of a structure. These controlled blasting specifications also typically define the critical threshold PPV at 2 in/s for the nearest building or structure. This 2.0 in/s PPV threshold is based on much of the same literature discussed earlier in this section and the fact that it has been an accepted and widely used standard for many years by many government agencies (Edwards and Northwood, 1959; Nichols, Johnson, and Duvall, 1971; Siskind, et al, 1980).

Seismic Wave Propagation

In order to better and more fully evaluate potential vibration risks to the tunnel and viaduct posed by construction sources, the mechanics of wave energy propagation through the ground from these sources to the tunnel and/or viaduct must be briefly discussed. Shock waves, or vibrations, from relatively nearby sources (as opposed to earthquakes) are transmitted through the earth almost entirely as body waves or as a form of surface wave known as a Rayleigh wave. Body waves, which propagate through the body of a material, are primarily of two types, P waves and S waves. P waves are compressional waves, which means that they travel through an elastic medium (earth materials are elastic) by alternate dilatational and compressional pulses oriented in the direction of propagation. S waves are shear waves, which means that they propagate by means of a side-by-side (or up and down) sort of whipping or snaking motion that is perpendicular to the direction of propagation. P waves can travel through a liquid or a solid; an S wave can only travel though a solid, as a liquid is incapable of shearing. Rayleigh waves are a form of surface wave that propagates in a half-space, which is basically a boundary between the earth and a medium incapable of shearing such as air, water, or a vacuum. Rayleigh waves propagate as a sort of combination of compressional and shear wave in a form of retrograde motion, where the motion is best described as a backward spinning circle or ovoid that moves forward with the top of the motion bounded along the half-space boundary. The amplitude of body waves attenuates with distance in direct proportion to the distance from the source, except along the surface where they attenuate in direct proportion to the square of the distance (Jones & Stokes, 2004). The amplitude of Rayleigh waves attenuates in direct proportion to the square root of the distance to the source when travelling along the ground surface. Rayleigh waves, however, attenuate quite rapidly with depth, within just a few wavelengths, because of the fact that they are a surface wave confined to the half-space boundary. Along the surface of the ground Rayleigh waves carry about 67% of the energy of a nearby shock or vibrating source, while the S wave carries about 24% and the P wave about 7% (Richart, 1970). As the distance increases the body waves attenuate much more rapidly (distance squared versus square root of the distance) than the Rayleigh wave, so that the Rayleigh wave carries an increasingly greater percentage of the energy with increasing distance. These facts together indicate that the Rayleigh wave is by far the most significant vibratory wave along the ground surface. They indicate that for any given distance from a source the energy reaching the location of interest (the structure of concern, or perhaps a seismometer to record the wave, which can then yield the particle velocity) is greater when the location is on the surface, as opposed to it being located straight down into the ground below the source. The difference in energy is that between the reciprocal of the square root of the distance (Rayleigh wave along the surface) versus the reciprocal of the distance squared (body waves going down). This difference has some pertinence to this project because most PPV data is based on Rayleigh waves, yet the vibrations of concern here with regards to the tunnel are body waves travelling downward.

Effect of Frequency

Investigation of the damage on structures caused by mining blasting done by Siskind et al for the U.S. Bureau of Mines (1980) reintroduced the dependence of damage on frequency. Though valid, this work was focused on mining blasting, not construction blasting or other construction sources. Furthermore, it was directed at preventing very low probability damage and cosmetic damage (drywall and plaster on walls). These criteria are not accepted by all concerned with assessing blasting damage (Konya & Walter, 1993) and are not considered realistic for most structures, certainly not stout concrete structures. Typical vibrations from transportation and construction sources typically fall in the range of 10-30 Hz and usually center around 15 Hz (Hendricks, 2002). Within the narrow range of frequencies associated with most construction sources, including construction blasting, frequency independence is a reasonable assumption (Jones and Stokes, 2004).

Resonance, however, is a special situation where frequency can pose a problem for both vibratory and impact pile drivers. If the driving (impact) or oscillatory (vibratory) frequency is the same as, or close to, the resonant frequency of the soil in which the driving is to be performed, resonance will set up and, if allowed to continue, will result in considerable amplification of the vibrations. This can begin to occur with a vibratory driver as it is starting up and its dominant frequency gradually passes up through the resonant frequency range of the soil. Typically, once vibratory drivers are in full operation they will operate at a higher frequency than the resonant frequency of most soils, so once they are running at full speed this problem no longer exists, until, that is, they have to shut down and the vibration frequency once again passes through the natural resonance frequency of the soil. Should this start up and shut down be problematic in terms of the PPV values created and the duration of the amplification, there are resonance-free or variable eccentric moment vibratory pile drivers that do not impart energy into the ground until they are running at full speed and centrifugal force.

Minimum Allowable Distances from Tunnel and Viaduct for Construction Equipment

Most studies discussed above regarding threshold PPV values from construction sources and blasting provided PPV values at specific distances. In order to determine the approximate distance from either the tunnel or the viaduct that equipment can be operated at while maintaining PPV values beneath the specified 2.0 in/s threshold levels several equations from Jones and Stokes (2004) can be used.

The following equation, applicable to most construction equipment, can be used to determine the distance (D) at which the PPV_{site} (the PPV at the tunnel or viaduct) will approach 2.0 in/s:

$PPV_{site} = PPV_{Ref}\,(25/D)^{n}$ *(in/sec)* **Equation 1.**

Where:
PPV_{Ref} = reference PPV at 25 ft. (from FTA, 1995; listed above earlier in this section)
D = distance from equipment to the site in ft.
n = 1.0 to 1.4.

The *n* value is related to the attenuation rate through ground with 1.0 being solid rock and 1.4 being weak, soft, or loose soils. In the case of the environment above tunnel # 3 values varying from 1.0 to 1.4 may apply because of the variation from the hard rock at depth and the soft loose soils seen in some of the colluvium. An *n* value of 1.0 is used in this GDR in order to assure that conservative distances (greater distance than necessary to stay below threshold) are obtained.

In order to utilize other PPV references that list different distances, simply replace the PPV_{ref} value with the other reference value and change the value of 25 in the equation with the new reference distance.

Table 4. Minimum Vertical Distances Between Different Ground and Bedrock Surfaces, and the Tunnel Top.

LOCATION	TUNNEL TOP (1167 FT)	
HWY EASEMENT GROUND SURFACE (1225 FT)	58 ft	

HWY EASEMENT BEDROCK SURFACE (1207 FT)	40 ft	
RIGHT EDGE OF BRIDGE EXTRAPOLATED TO GROUND SURFACE (1273 FT)	106 ft	
RIGHT EDGE OF BRIDGE EXTRAPOLATED TO BEDROCK SURFACE (1248 FT)	81 ft	
CENTER LINE EXTRAPOLATED TO GROUND SURFACE (1287 FT)	120 ft	
CENTER LINE EXTRAPOLATED TO BEDROCK SURFACE (1260 FT)	93 ft	
LEFT EDGE OF BRIDGE EXTRAPOLATED TO GROUND SURFACE (1305 FT)	138 ft	
LEFT EDGE OF BRIDGE EXTRAPOLATED TO BEDROCK SURFACE (1272 FT)	105 ft	

Table 4 presents minimum vertical distances to the tunnel and rock arch from the ground surface and bedrock surface. The minimum distance on the entire list is 27 ft, which is the distance between the top of the rock arch and the bedrock surface at the boundary between the Caltrans highway easement and the UPRR ROW. The distance directly above the top of the tunnel to the top of the bedrock surface at the same location (boundary between the Caltrans highway easement and the UPRR ROW) is 40 ft. If these two distances are extended to the ground surface they become 45 ft and 58 ft, respectively. Table 3 above gives PPV values at 25 ft, all but one of which has PPV values well below the PPV threshold of 2 in/s established for the tunnel. When adjusted using equation 1, the PPV value of 0.079 in/s for caisson drilling at 25 ft yields a PPV value of 2 in/s at a distance less than 1 foot. This essentially means that any drilling near the Caltrans Highway easement/UPRR ROW in the area directly above the tunnel (such as drilling for the CMA system posts and anchors), or anywhere else within the Caltrans easement or ROW for that matter, will fall short of the critical PPV threshold by multiple orders of magnitude. The PPV value for the crack and seat operations listed in Table 3 is 2.4 in/s, which is above the critical threshold. Using the above equation iteratively results in the determination that the critical distance for these crack and seat operations would be 30 ft, which is 10 ft shy of the minimum tunnel top to bedrock distance and 28 ft shy of the minimum tunnel top to ground surface distance. Other PPV values cited above for other crack and seat operations using different equipment at different distances can be reduced using the above equation to similarly determine their minimum critical distance. The minimum distance thusly derived was 25 ft for the Walker Megabreaker 8-13000 and 14.5 ft for the EMSCO pavement breaking machine. These distances are clearly closer than any distance construction equipment working within the Caltrans Easement or ROW might approach. Although there should be absolutely no real reason for performing crack and seat operations on this project, the exercise of determining these distances was performed to demonstrate that even fairly high energy construction equipment should be safe to use, from a vibration standpoint, essentially anywhere within the potential construction area.

Pile drivers, both impact and vibratory, present a special case due to their high energy and their continuous (versus single or intermittent impact) nature. Slightly different equations than Equation 1 are used to determine critical distances, with the main difference being the inclusion of energy ratings. Equation 2 is based on over 10 sources referenced by Jones and Stokes (2004) and is used for impact pile drivers using a reference pile driver:

$$PPV_{Impact\ Pile\ Driver} = PPV_{Ref}\,(25/D)^{n} \quad x \quad (E_{equip}/E_{Ref})^{0.5} \qquad (in/sec) \qquad \textbf{Equation 2.}$$

Where:
PPV_{Ref} = 0.65 in/sec for a reference pile driver at 25 ft.
D = distance from pile driver to the receiver in ft.
n = 1.0 (most conservative option for this case)
This value was explained under equation 1.
E_{Ref} = 36,000 ft-lb (rated energy of reference pile driver)
E_{equip} = rated energy of impact pile driver to be used in project in ft-lbs.

The above 36,000 ft-lb pile driver would produce PPV values of 2 in/s at a distance of about 8 ft based on equation 2. As discussed earlier in this section, the majority of pile drivers have energy ratings between 20,000 and 300,000 ft-lbs. A 300,000 ft-lb impact pile driver would produce PPV values of 2 in/s at a distance of about 23.5 ft, based on equation 2. This value is still less than the closest direct distance (58 ft at the ground surface or 40 ft if excavated down to bedrock at the highway easement boundary directly above the tunnel) to the tunnel top that construction equipment might conceivably approach within the construction zone.

Similar data for vibratory pile drivers is not as widespread or available as it is for impact pile drivers, but research by Jones and Stokes (2004) indicate that Equation 2 should be similarly applicable. While Wood and Thiesen (1982) state that vibration damage from vibratory pile drivers may possibly exceed that from impact pile drivers due to more sustained vibration levels, data supplied by Morris (1991, 1996, 1997) support the idea that the vibration levels between the two are fairly similar. Vibratory pile drivers may produce resonant effects more often than impact pile drivers, however, so this should be guarded against.

Throughout the discussion above regarding PPV values and distances for all types of construction equipment, calculations and analyses were performed using conservative values when possible so that errors or estimates would tilt in favor of protecting the tunnel and viaduct. An additional layer of conservatism is integrated into these estimates for the tunnel due to the differences in energy, attenuation, and wave propagation between Rayleigh waves and body waves as discussed earlier, and the fact that most PPV data is based on Rayleigh wave data, when, in the case of the Sidehill project, the vibrations reaching the tunnel will be transmitted via body waves because Rayleigh waves attenuate so quickly with depth. Propagation of vibrations from construction sources, including pile drivers, is typically modelled in terms of Rayleigh waves (Jones and Stokes, 2004). In fact most, if not all, PPV studies and research involve ground surface source to receiver paths, which means that Rayleigh waves are almost the sole vehicle of vibrational energy propagation in these studies. This means that PPV values taken from published tables for a set distance are likely to overestimate the vibration effect on the tunnel, because of the greater attenuation of the body wave energy relative to that of the Rayleigh wave energy per equal distance.

Vibration Monitoring

The information presented and discussed in this section indicates that vibrations created by construction equipment should not pose a damage threat to the structural integrity of the tunnel. Nor should they pose a damage threat to the integrity of the viaduct, provided that some minimal distances between larger sources and the viaduct are maintained. The sources that could come closest to producing problematic PPV levels at the tunnel are the larger pile drivers, although PPV magnitudes would likely not reach critical magnitude. These larger pile drivers are also the only sources that might exceed critical PPV values for the viaduct if minimum distances are not met. Pile driving efforts in these areas are not likely to achieve great success due to the colluvial nature of the surface material and its proclivity for boulders. Based on these facts and a PDT discussions of these issues, OGDN recommends that pile driving be prohibited in the area bounded by the arches, between stations 895+00 and 899+00, from the easement boundary downslope to the west side of the bridge and that all piles or posts the contractor might want to install within this area for falsework or other purposes shall require pre-drilling of the holes and pile placement therein.

Should blasting be chosen as an excavation method by the contractor, blasting specifications for pre-splitting and controlled blasting shall be followed, which include PPV monitoring and safe distances. All blasting within the project area must be monitored to confirm and document that the critical threshold is not exceeded at either structure.

8.4.3 Pile or Ground Anchor Insertion into Terrain Directly above the Tunnel Structure

Installing piles or ground anchors into the overburden material above the bedrock should have no detrimental effect upon the integrity of the tunnel rock arch or the load support capacity of the rock mass above the arch. As stipulated in the previous section, such installation shall not include driving as a means of installation. Installing piles and/or ground anchors (by drilling) into the bedrock below the overburden should also have no detrimental effect upon the integrity of the tunnel roof or rock arch provided that these installations do not extend within 25 ft of the top of the rock arch, or within 38 ft of the top of the concrete tunnel lining.

9. CONSTRUCTION CONSIDERATIONS

9.1 Construction Advisories

Cut slopes may shed some minor rockfall during construction, so appropriate caution around these cuts should be exercised.

Prior to installation of the rockfall and debris CMA system vegetation should be completely cleared in a 15-foot swath over the entire area of installation between CMA post 1 and CMA post 10 with the upslope edge of the swath coincident with the line of CMA posts.

9.2 Construction Considerations that Influence Design

If possible, project staging, and possibly specifications and plans, should be formulated so that the North Cut Slope is excavated early in the construction schedule. This will allow more time for the new cut slope face to be exposed to precipitation, which will help to expose any weaknesses in the new cut slope so that it can be addressed during the final construction season before traffic is put beneath it. In addition, a minimum of two months should be allowed to pass following the completion of the north cut before the construction of anchor locations for the DTWM attenuator begins. This delay will help to prevent the placement of anchors in potentially unstable or unsuitable locations. Due to staging issues, the portion of this cut from station 905+00 to 907+00 is exempt from this delay.

9.3 Construction Monitoring

The north cut slope, and the terrain immediately above it, should be monitored during and after construction to insure that the DTWM attenuator posts and anchors are located on stable ground.

9.4 Differing or Problematic Site Conditions

Should differing site conditions arise during construction please contact ODGN.

10. RECOMMENDATIONS AND SPECIFICATIONS

10.1. Cut Slopes

- Prior to cutting slopes (south cut and north cut), vegetation should be completely cleared and grubbed. Trees larger than 6" at chest height that are situated within 5 ft 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 unless they are in the clear recovery zone where they will be removed completely within 30 ft of the edge of travelled way (ETW).

- Recommended cut slope ratios for both the south and north cut slopes are primarily 1:1 with some portions being 0.75:1 and other parts being constructed with a dual slope ratio of 1:1 over 0.75:1. Station by station cut slope ratio recommendations are provided in Tables 1 and 2 (section 8.1.1)

- An unpaved shoulder with a 6:1 backslope (towards the cut slope) and a width varying from 15 to 28 ft (including shoulder backing) is recommended at the base of the south cut slope for rockfall catchment. Catchment widths are provided by stationing in Table 1 (section 8.1.1).

- A minimum 3-foot wide unpaved shoulder with a 4:1 backslope (towards the cut slope), together with an additional 3 ft of shoulder backing, is recommended at the base of the north cut slope (stations 905+50 to 911+00).

- The construction of a double-twisted wire mesh (DTWM) attenuator is recommended to mitigate potential rockfall for most of the northern cut slope. The general design for this attenuator is described in section 8.1.4. OGDN supplied drawings and details are part of the contract package.

- The DTWM attenuator shall be constructed no sooner than two months after the construction of the north cut slope from station 907+00 to 911+00 is completed. This is intended to allow assessment of the new cut face and top of cut hinge point so that final attenuator post locations may be properly located by the Residential Engineer (RE) with the assistance of OGDN. Due to staging considerations, the portion of the North Cut from station 905+00 to 907+00 is exempt from this stipulation.

- Blasting, if chosen by the contractor as a means of excavating the north and/or the south cut slope, must be performed subject to controlled blasting and presplitting specifications.

10.2 Embankments

- It is recommended that the fill slope constructed around the southern abutment be constructed at a slope ratio of 1.5:1 or flatter. It is recommended that the fill slopes constructed north of the north cut between stations 911+00 and 918+25 be constructed at a slope ratio of 1.5:1 or flatter.

- It is recommended that all fill faces be sprayed with some type of hydro seed mulch or other erosion protective measure sometime after their completion.

10.3 Railroad Protection from Construction-Induced Rockfall and Debris

- A cable mesh attenuator (CMA) shall be constructed according to plans and specs to protect the railroad infrastructure from construction-induced rockfall and debris-fall. This construction shall be completed prior to any work on the slope regarding the construction access and bridge.

- The contractor shall be responsible for repairing and cleaning/maintaining the CMA system to assure that it remains completely functional.

- The contractor must add further protection against rockfall and debris protection if the contractor deems it necessary. The presence of the CMA system does not relieve the contractor of liability for any damages incurred to the UPRR infrastructure or for any shipping time lost by UPRR due to any line shutdown caused by construction-induced rockfall or debris.

10.4 Construction Vibrations - Limits, Constraints, and Monitoring

- It is recommended that pile driving be prohibited as a means of pile installation inside the area between the arch blocks, roughly between stations 895+00 and 899+00, from the downslope easement boundary to the west side of the existing viaduct.

- Any construction blasting performed within the project area shall follow the final controlled blasting specifications. These specifications require vibration monitoring for both the tunnel and the viaduct.

- The critical vibration threshold for both the tunnel and viaduct shall be defined as a Peak Particle Velocity (PPV) of 2.0 inches per second (in/s).

10.5 Access for Bridge Construction

- Any and all construction access planned by the contractor must first be approved by the engineer. Any and all construction access planned by the contractor that occurs within the UPRR ROW must also first be approved by UPRR. Prior to any work on construction access, the contractor must submit plans and specifications to the resident engineer regarding any excavations, fills, and structures intended to provide access to bridge foundations and falsework. Such plans should take into account the geotechnical and geological conditions described in this report and be designed in accordance with the conditions described. Such plans shall include details regarding equipment and methodology to be used. Any plans for access construction deemed to pose a risk to the existing viaduct, the future bridge, or the UPRR tunnel shall not be approved. Any change to these plans during the course of the access construction must be approved by the resident engineer prior to the initiation of said change. Any change to these plans during the course of the access construction that occur or effect the UPRR ROW must also be approved by UPRR prior to the initiation of said change.

- The contractor shall be responsible for removing the access road(s) as well as any fills or structures built for access, unless an exemption is granted for a particular feature by the resident engineer and by UPRR for any features within the UPRR ROW. Initial access plans must also contain details of the methodology and plan for such removal.

- The contractor shall comply with, and be bound by, the limitations on load placed on the tunnel as described in section 8.4.2, with regards to any construction access and/or falsework.

- In building construction access, the contractor shall comply with, and be bound by, the limitations on construction vibration and blasting discussed in section 8.4.3 and 10.4, and submit as part of the access plans information detailing how this compliance will be done.

11. REFERENCES

Albers, J.P. & Robertson, J.F., 1961, *Geology and Ore Deposits of East Shasta Copper-Zinc District Shasta County, California*. USGS Professional Paper 338.

American Association of State Highway and Transportation Officials. 1990. *Standard recommended practice for evaluation of transportation-related earthborne vibrations.* Washington, DC.

Ames, W.H., Chow, W, Sequiera, A., and Johnson, R., 1976, *Survey of earthborne vibrations due to highway construction and highway traffic.* (Report CA-DOT-TL-6391-1-76-20.) Sacramento, CA

Barton, N.R., Lien, B., Lunde, J., 1974, Engineering classification of rock masses for the design of tunnel support, Rock Mechanics 6(4), 189-236.

Bieniawski, Z.T., 1974, Geomechanics classification of rock masses and its application in tunnelling, Proc 3rd Int. Cong. Rock Mech., Denver 2(2), 27-32.

Brady, B.H.G., and Brown, E.T., *Rock Mechanics for Underground Mining,* 3rd edition, Springer. 2004

Baur, P, Caulder, P.N., 1977, *Pit Slope Manual*. Ottawa, CANMET.

Baur, P. Caulder, P.N., 1978, *Open Pit and Blast Seminar*. Kingston, Ontario, Canada.

Boussinesq, M.J., 1885, *Application Des Potentiels, a' l'Etude de l'Equilibre et du Movvment Des Solides Elastiques,* Gauthier-Villars, Paris

California Department of Conservation, Division of Mines and Geology, 2000, *General Location Guide for Ultramafic Rocks in California - Areas Likely to Contain Naturally Occuring Asbestos,* 2000, Map scale 1:1,100,000, Open-File Report 2000-19

California Department of Transportation, Division of Maintenance GIS in coordination with Department of Environmental Analysis (2004), District 2 Areas Likely to Contain Asbestos, Map.

California Department of Transportation, 2000, *State Route 101 crack-and-seat vibration study.* Sacramento, CA.
———. 2002. *Transportation-related earthborne vibrations.* (Technical Advisory TAV-02-01-R9601.) Sacramento, CA.

California Department of Transportation, 2011, Bridge Design Specifications, October.

Caterpillar Tractor Co., 1982, October, Caterpillar Performance Handbook, edition 13.

Chae, Y.S., 1978, *Design of excavation blasts to prevent damage*, Civil Engineering- American Society of Civil Engineers 48(4):77-79.

Chai, J., 2000, Shallow Foundations Chapter 31, Bridge Engineering Handbook, boca Raton, CRC Press.

Dowding, C.H., 1996, Construction Vibrations, Prentice-Hall, Englewood Cliffs, NJ.

Duncan, J.M., Buchignani, A.L., 1976, An Engineering Manual for Settlement Studies, Department of Civil Engineering, University of California, Berkeley, June, 94 p.

Duvall, W.I., and Fogelson, D.E., 1962, *Review of criteria for estimating damage to residences from blasting activities*: U.S. Department of Interior Bureau of Mines, Report of Investigations 5968, 19 p.

Edwards and Northwood, 1959, *Experimental Blasting Studies on Structures*, National Research Council, Ottawa, Canada

Egan, J., J. Kermode, M. Skyrman, and L. Turner. 2001. *Ground vibration monitoring for construction blasting in urban areas*. (Report FHWA/CA/OR-2001/03.) California Department of Transportation. Sacramento, CA.

Fang, H.Y., ed. (1991) Foundation Engineering Handbook, 2nd ed, Chapman and Hall, New York, 923 p

Federal Transit Administration, 1995, *Transit Noise and Vibration Impact Assessment*, DOT-T-95-16. Office of Planning, Washington, D.C. Prepared by Harris Miller, Miller and Hanson Inc., Burlington, MA.

Goodman, R.E., 1989, *Introduction to Rock Mechanics*, John Wiley & Sons, New York 2nd ed.

Heidbach, O., Tingay, M., Barth, A., Reinecker, J., Kurfeß, D., and Müller, B., The World Stress Map database release 2008 doi:10.1594/GFZ.WSM.Rel2008, 2008

Hendriks, R. 2002. *Transportation related earthborne vibrations (Caltrans experience).* California Department of Transportation. Sacramento, CA.

Hoek, E., and Bray, J., 1977, *Rock Slope Engineering*, 1st edition, IMM, London.

Hoek, E., and Brown, E.T., 1980, *Empirical Rock Strength Criterion for Rock Masses*, J. Geotechnical Engineering Division ASCE 106 (GT9), 1013-1035.

Hoek, E., and Brown, E.T., 1980, *Underground Excavation in Rock*, p 137-150, The Institution of Mining and Metallurgy, London, Hoek, E., 1983, Strength of jointed rock masses, *Geotechnique* 33, 3, 187-223.

Hoek, E., and Brown, E.T., 1988, The Hoek-Brown Failure Criterion – a 1988 update. *15th Canadian Rock Mechanics Symposium*, Toronto, Canada.

Hung, J.C., Monsees, J. Munfah, N, Wisniewski, J, 2008, Technical Manual for Design and Construction of Road Tunnels, FHWA NHI-09-010, November, 700pp.

International Society for Rock Mechanics (ISRM), Committee on Laboratory Testing, 1979, Suggested methods for determining the uniaxial compressive strength and deformability of rock materials, Int. Jour of Rock Mechanics, 162(2), 137-140.

International Society for Rock Mechanics (ISRM), 1981, Suggested Methods for the Quantitative Description of discontinuities in Rock Masses. Pergammon Press, UK.

Irwin, W.P., 1966, *Geology of the Klamath Mountains Province*, California Division of Mines and Geology Bulletin 190, p. 19-38.

Jones and Stokes, June, 2004, *Transportation- and Construction-Induced Vibration Guidance Manual* (J&S 02-039). Prepared for California Department of Transportation Environmental Program, Environmental Engineering; Noise, Vibration, and Hazardous Waste Management Office.

Konya, C.J., & Walter, E.J., 2002, *Rock Blasting and Overbreak Control*, 2nd Edition, National highway Institute, FHWA-HI-92-001. 430 p.

Kulhawy, F.H., Goodman, R.E., 1980, Design of Foundations on Discontinuous Rock, International conference on Structural Foundations on Rock, Sydney, 7-9 May, 1980, P. 209-220.

Kulhawy, F.H., 1978, Geomechnical Model for Rock Foundation Settlement, Journal of the Geotechnical Engineering division, Feb 1979, pp. 211-227.

Langefors, U., Kihlstrom, B., 1958, *Ground vibrations in blasting*. Water Power. September.

Merriam, Martha, 2012, Caltrans Fault Database.

Morris, R.S., 1991, *Surface vibration measurements for vibratory pile drivers*, Tests conducted by International Construction Equipment.

NAVFAC, DM-7.2, 1982, Foundations and Earth Structures, Department of the Navy – Naval Facilities Engineering Command, Alexandria, VA

Nelson, P.M. (ed), 1987, *Transportation Noise Reference Book*. London: Butterworths.

Nichols, Johnson, and Duvall, 1971, *Blasting Vibration and Their Effects on Structures*. U. S. Bureau of Mines, Bulletin 656.

Oriard, L.L., 1970, *Dynamic effect on rock masses from blasting operations*, Slope Stability Seminar, University of Nevada, Reno.

Oriard, L.L. Coulson, J.H., 1980, *TVA blast vibration criteria for mass concrete*. Reston, Virginia: American Society of Civil Engineers

Pariseau, William, G., 2012, *Design Analysis in Rock Mechanics,* 2nd edition, CRC press

Peck, R.B., Hanson, W.E., and Thornburn, T.H., 1974, *Foundation Engineering*, Wiley, New York.

Pierson, L.A., Gullixson, C.F., and Chassie, R.G., 2001, *Rockfall Catchment Area Design Guide*, Final Report SPR-3(032), Oregon Dept. Transportation, FHWA-OR-RD-02-04. 91 p.

Richart,F.E., 1970, *Vibrations of soil and foundations*. Englewood Cliffs, NJ: Prentice Hall.

Sawyer, T.L., 2013, The Northern California Shear Zone- Missing Link in the Pacific North American Plate Transform Margin? Geological Society of America, Abstracts 23-10, Cordilleran Section, May 20-22, Fresno.

Sawyer, T.L., 2014. Personal communication.

Siskind, D.E., Stagg, M.S., and Kopp, J.W., and Dowding, C.H., 1980, *Structure response and damage produced by ground vibration from surface mine blasting:* U.S. Department of Interior Bureau of Mines Report of Investigations 8507, 74 p.

Smith, T., McCauley, M., Mearns, R., & Baumeister, K., November, 1972, *Correlation of Seismic Velocities with Earthwork Factors – Final Report*, California Department of Transportation Research Report No. M&R 632103.

Sofianos, A.I. (1996) *Analysis and design of an underground hard rock Voussoir beam roof*. Int. Jnl. Rock Mechanics and Mining Sciences, Vol. 33, No. 2, pp. 153-166.

Song, Thomas, 2015, Geotechnical Engineering Analysis of Potential Impact by the Proposed Bridge Foundation Loadings on the UPRR Tunnel, Caltrans document, May, 2015.

Stephens, E., 1978, *Calculating Earthwork Factors Using Seismic Velocities*: California Department of Transportation, Report No. FHWA-CA-TL-78-23.

Strand, R.G., 1962, *Geologic Map of California, Redding Sheet*, California Division of Mines and Geology, scale 1:250,000

Tart, R.G., Oriard, L.L., Plump, J.H., 1980, *Blast Damage for Massive Concrete Structure* (American Society of Civil Engineers Preprint 80-175.) Portland OR: ASCE National Meeting, Specialty Session on Minimizing Detrimental Construction Vibrations

Terzaghi, K., 1946, Rock Defects and Loads on Tunnel Supports, Rock Tunneling with Steel Support, R. V. Proctor and T. White, Commercial Shearing Co., Youngstown, OH: 15-99.

Thorburn, S.H., 1966, Large Diameter Piles Founded in Bedrock, *Proc. Of symposium on Large Bored Piles, Inst. Of Civil Engineering*, London, pp. 95-103.

USBR, 2004, *Shasta and Keswick Dams Central Valley Project, California, Issue Evaluation-Screening Level Ground Motion Analysis*, Technical Memorandum no. D-8330-2004-12.

U.S. Dept of Transportation, 1982, *Urban rail noise and vibration control*, Washington, D.C.

Wang, J.N., 1993, *Seismic Design of Tunnels*, Parsons Brinckerhoff Quade & Douglas Inc., June

Winterkorn, H.F., and Fang, H-F. 1975, *Foundation Engineering Handbook*, Van Nostrand Reinhold, New York, pp. 148-166.

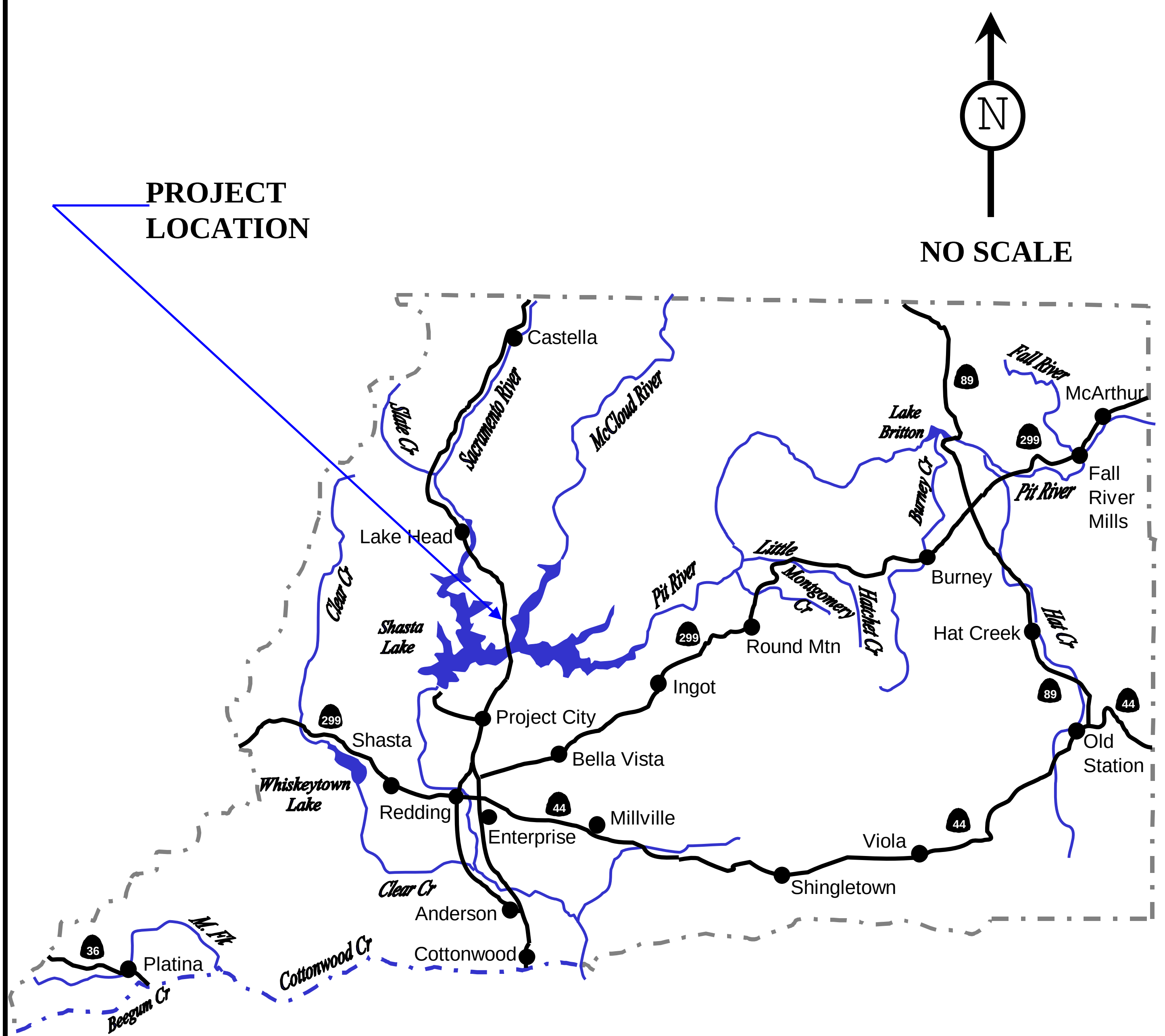
Woods, R. S., and L. P. Jedele. 1985. *Energy-attenuation relationships from construction vibrations*. Pages 187–202 in G. Gazetas and E. T. Selig (eds.), *Vibration Problems in Geotechnical Engineering: Proceedings of a Symposium*. Sponsored by the Geotechnical Engineering Division, in conjunction with the American Society of Civil Engineers Convention, Detroit, MI, October 22, 1985. New York: American Society of Civil Engineers.

Woods, R.D., 1997, *Dynamic effects of pile installations on adjacent structures* (Synthesis of Highway Practice 253.) Washington, D.C.: National Academy Press

Woodward, R.J., Gardner, W.S., and Greer, D.M., 1972, *Drilled Pier Foundations*, McGraw-Hill, New York.

Wyllie, D.C., 1999, *Foundations on Rock*, 2nd Edition, E & FN Spon, London, 401 p.

LIST OF PLATES



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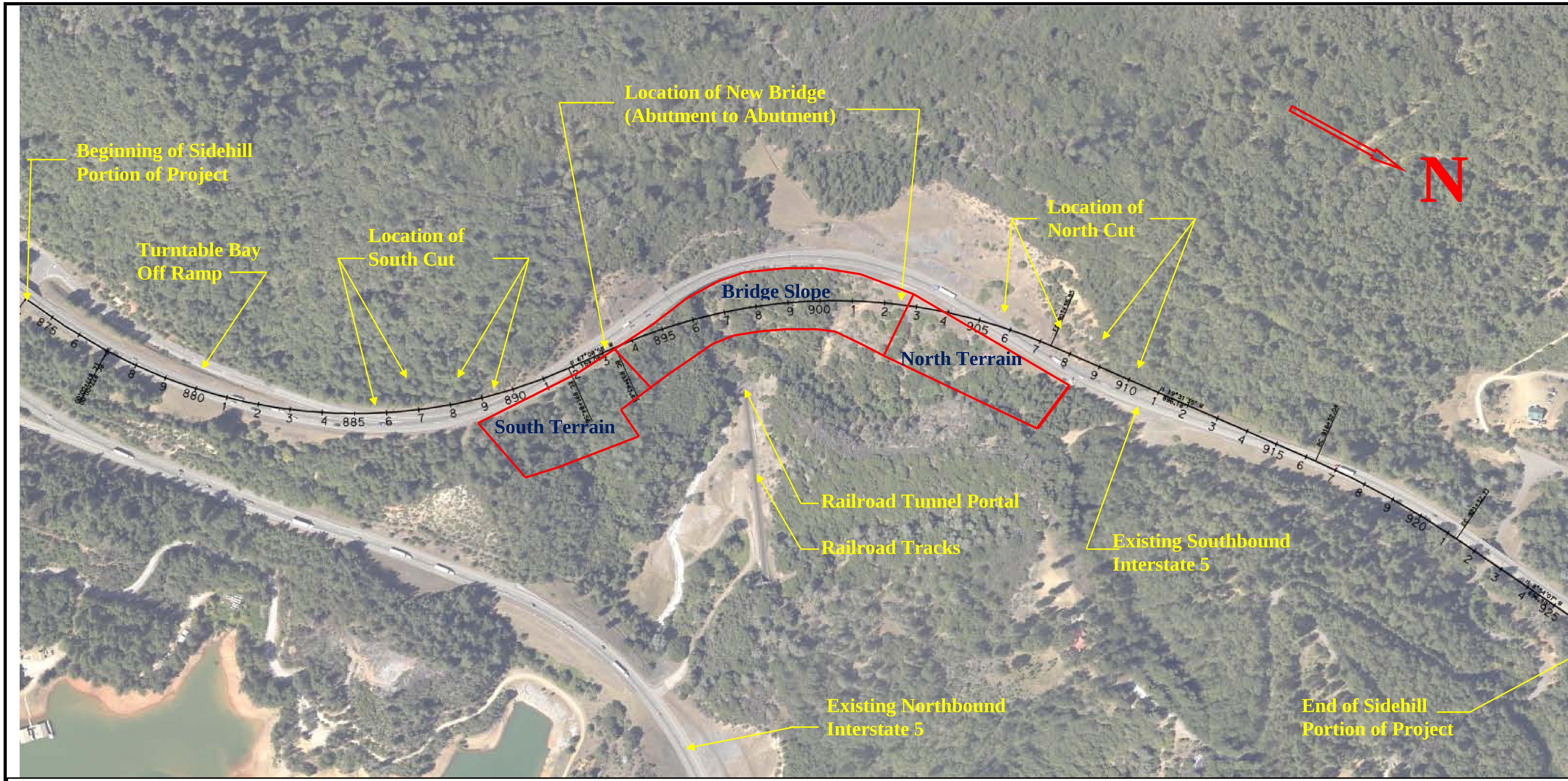
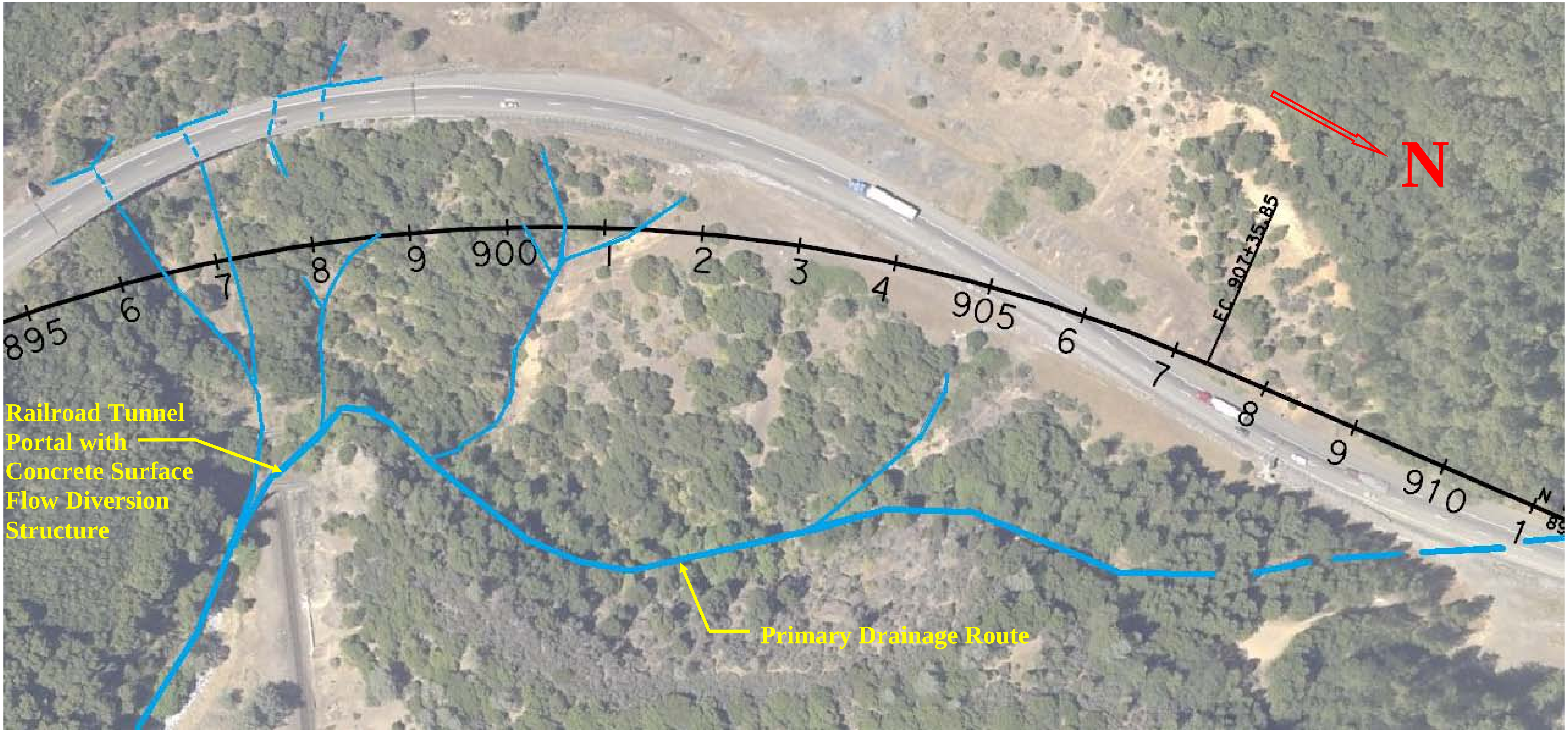
Office of Geotechnical Design-North

EA: 02-0E090	**PROJECT LOCATION**	
Date: June 2015		
02-SHA-5 PM R29.3/R30.3 GEOTECHNICAL DESIGN REPORT		Plate No. 1



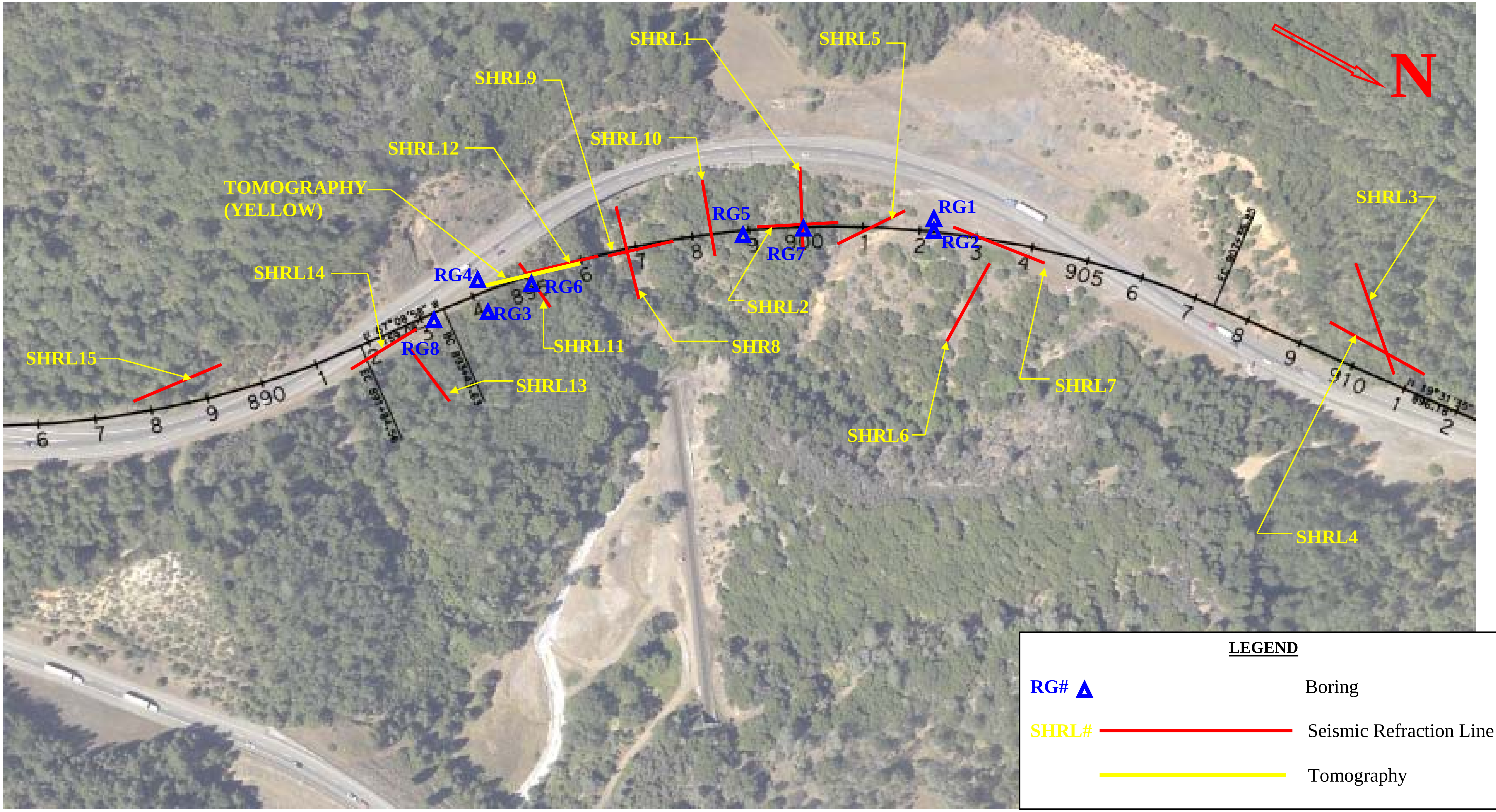
Aerial Photo of the Sidehill Viaduct Replacement Portion of the Doghill Project, Showing Features, Landmarks, and Defined Areas. Project stationing is shown on new alignment, which is shown as solid black line. Feature locations shown include the start and end of the Sidehill Portion of the Project, existing southbound and northbound Interstate 5, the railroad tunnel portal and tracks, the location of the new bridge, the south cut and north cut. Areas defined in the text for ease of discussion and demarcated on the photo include the bridge slope, north terrain, and south terrain.

Caltrans	CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-0E090 Date: June 2015	AERIAL PHOTO OF SIDEHILL PROJECT AREA
		02-SHA-5 PM R29.3/R30.3 GEOTECHNICAL DESIGN REPORT	Plate No. 2



Aerial Photo Showing the Approximate Drainage Locations of the Sidehill Project. Blue lines indicate the approximate location of drainage channels, gullies, rills or areas where ephemeral flow is funneled and transported. Lines are dashed where the drainage passes beneath the existing viaduct or the highway. The primary drainage route, which comes from the north-northwest end of the project, is indicated by the thicker blue line. Stationing is shown for project reference.

Caltrans	CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-0E090 Date: June 2015	**AERIAL PHOTO SHOWING DRAINAGES**
		02-SHA-5 PM R29.3/R30.3 GEOTECHNICAL DESIGN REPORT	Plate No.3



Aerial Photo of the Sidehill Project Area with Centerline Stationing Showing Locations of Refraction Lines, Tomography, and Borings.

CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-0E090 Date: June 2015	**SIDEHILL PROJECT AERIAL PHOTO W/ REFRACTION LINE, TOMOGRAPHY & BORING LOCATIONS**
	02-SHA-5 PM 29.3/30.3 **GEOTECHNICAL DESIGN REPORT**	Plate No. 4

EXPLANATION

Geologic Units

Jurassic - Cretaceous

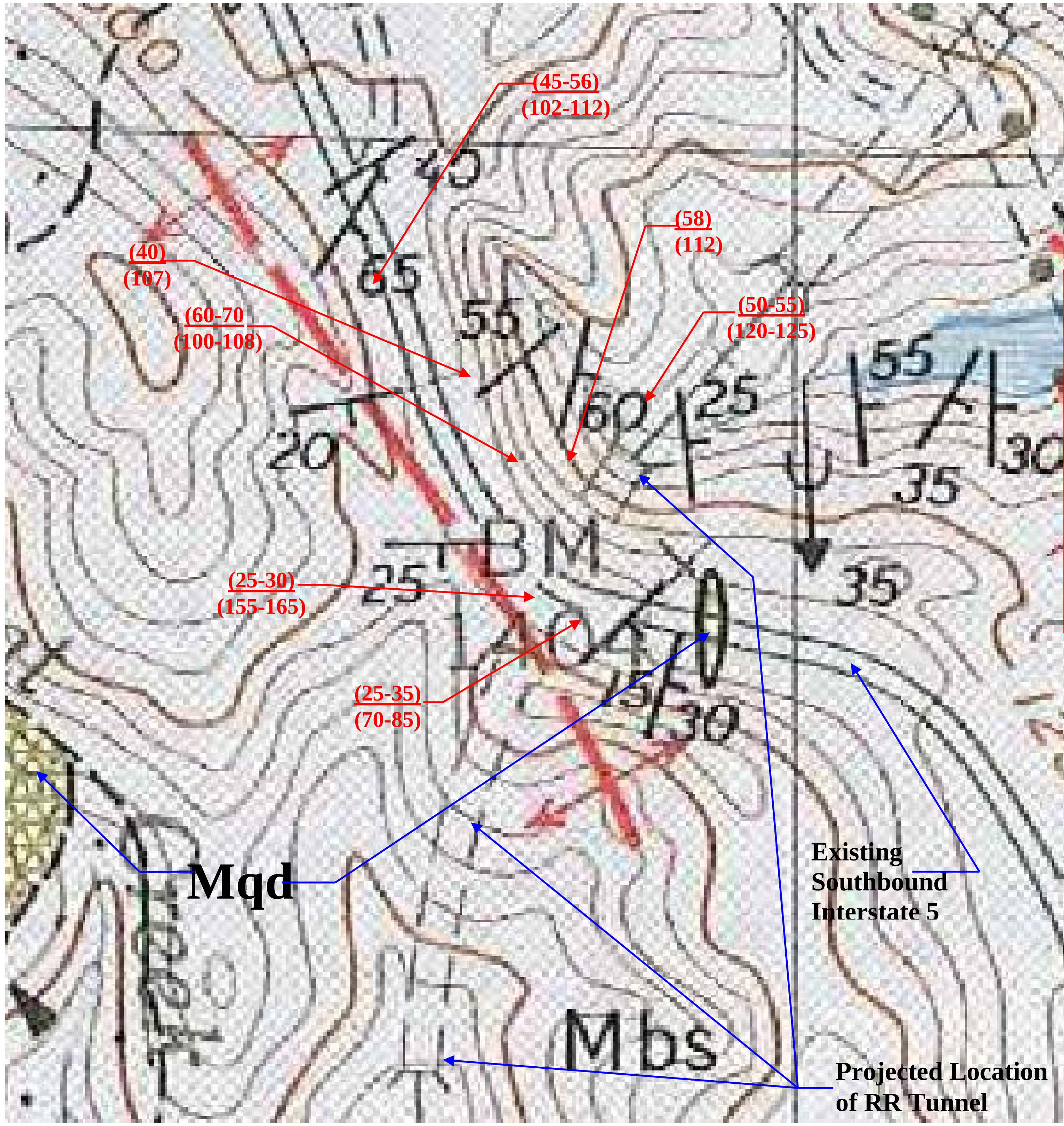
Mqd. Mafic quartz diorite, diorite, and metadiabase, undifferentiated.

Mississippian-Carboniferous

Baird formation

Mbs. Shale, sandstone, tuff, and tuffaceous sandstone, undifferentiated.

From *Geologic Map and Sections of the East Shasta Copper-Zinc District, Shasta County, California* in *Geology and Ore deposits of East Shasta Copper-Zinc District Shasta County, California, U.S.G.S. Professional Paper 338*, by J.P. Albers and J.F. Robertson, 1961.



Bedding Structural Data. Dip Angle over Dip Azimuth. Pair in Parentheses indicates range of values from multiple measurements. Single value indicates single measurement

(25-30)
(155-165)

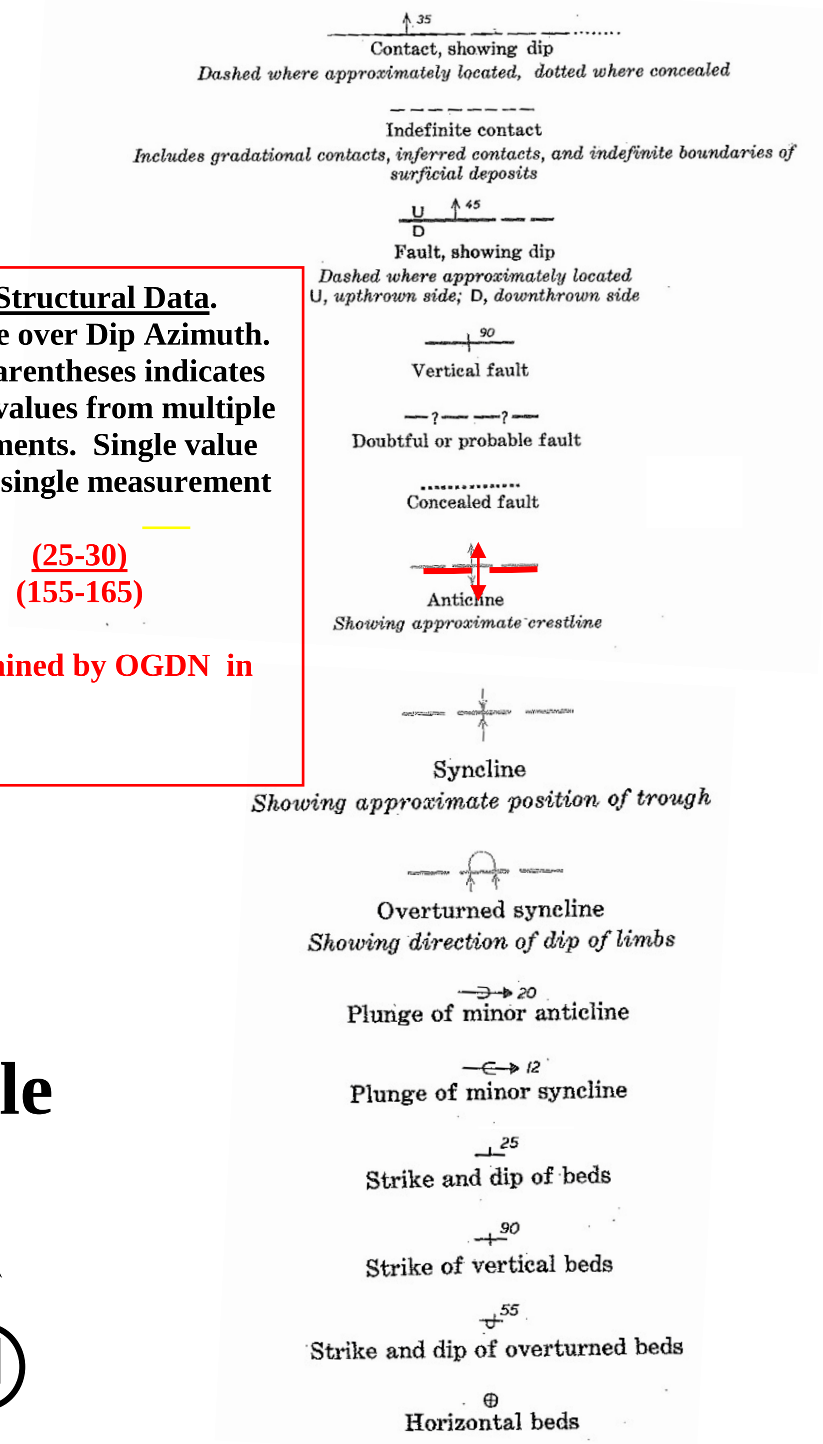
Data obtained by OGDN in 2014.

No Scale

N

EXPLANATION

Structural Features



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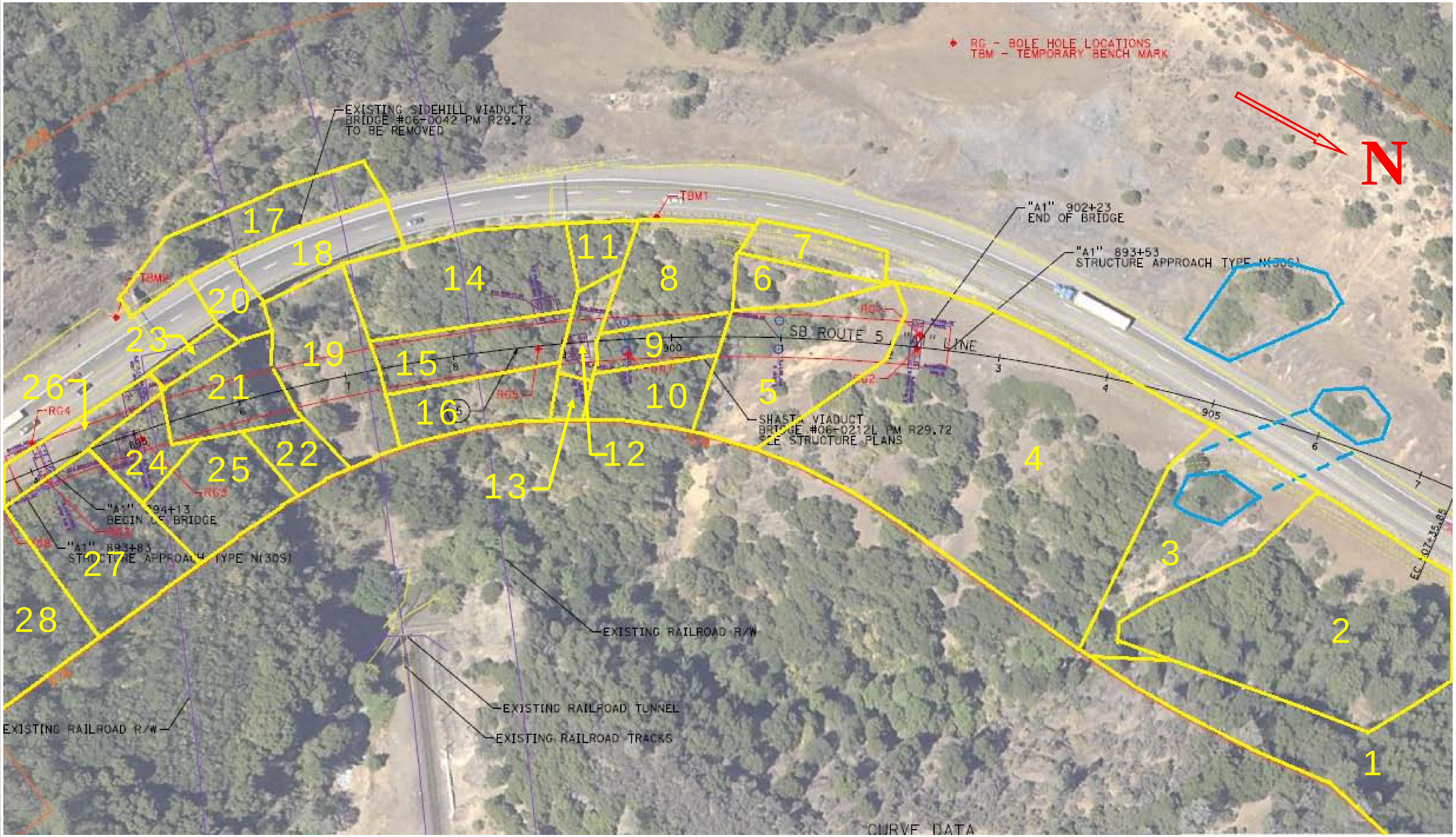
EA: 02-0E090	GEOLOGIC MAP OF SIDEHILL PROJECT AREA
Date: June 2015	
02-SHA-5 PM R29.3/R30.3 GEOTECHNICAL DESIGN REPORT	Plate No. 5



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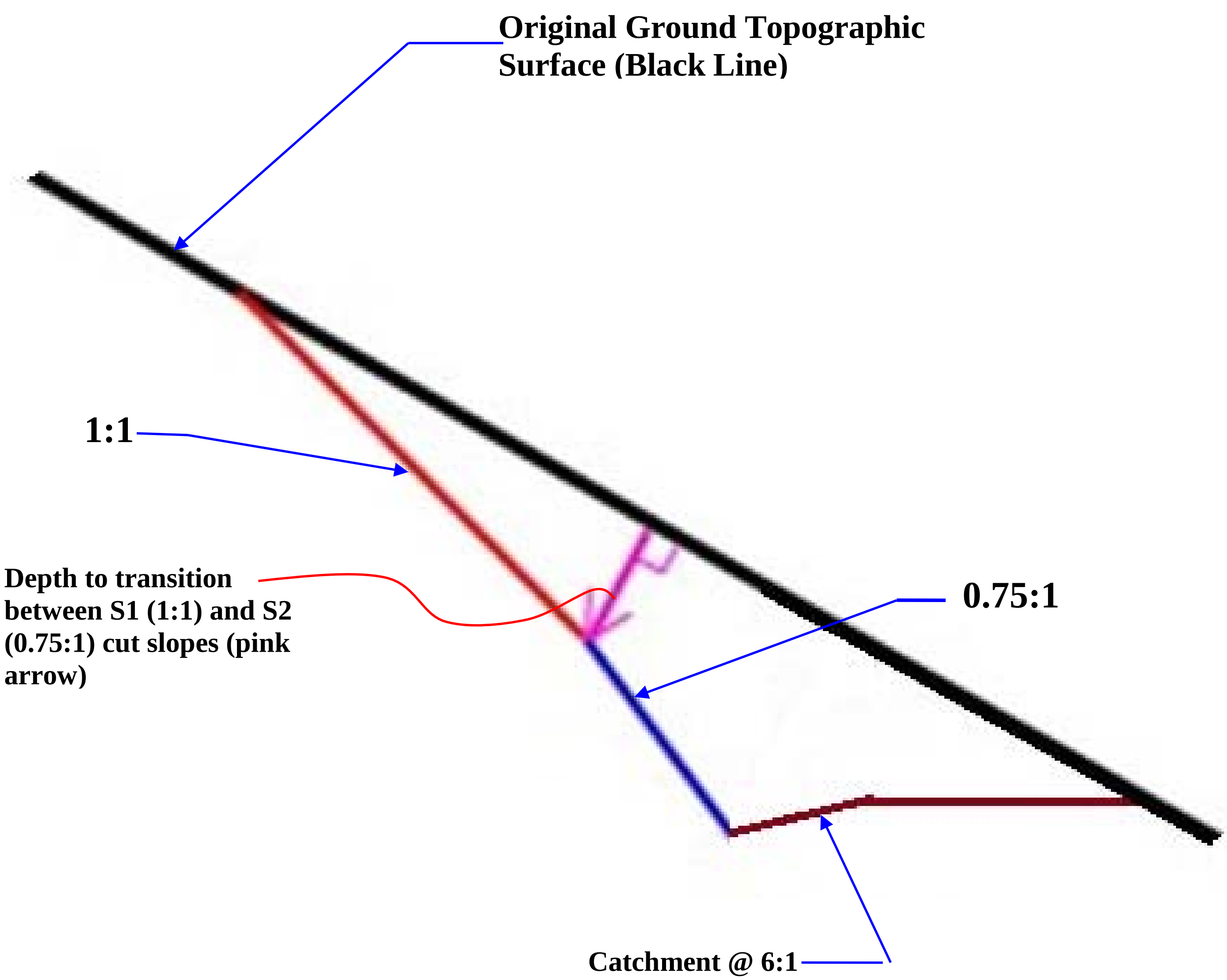
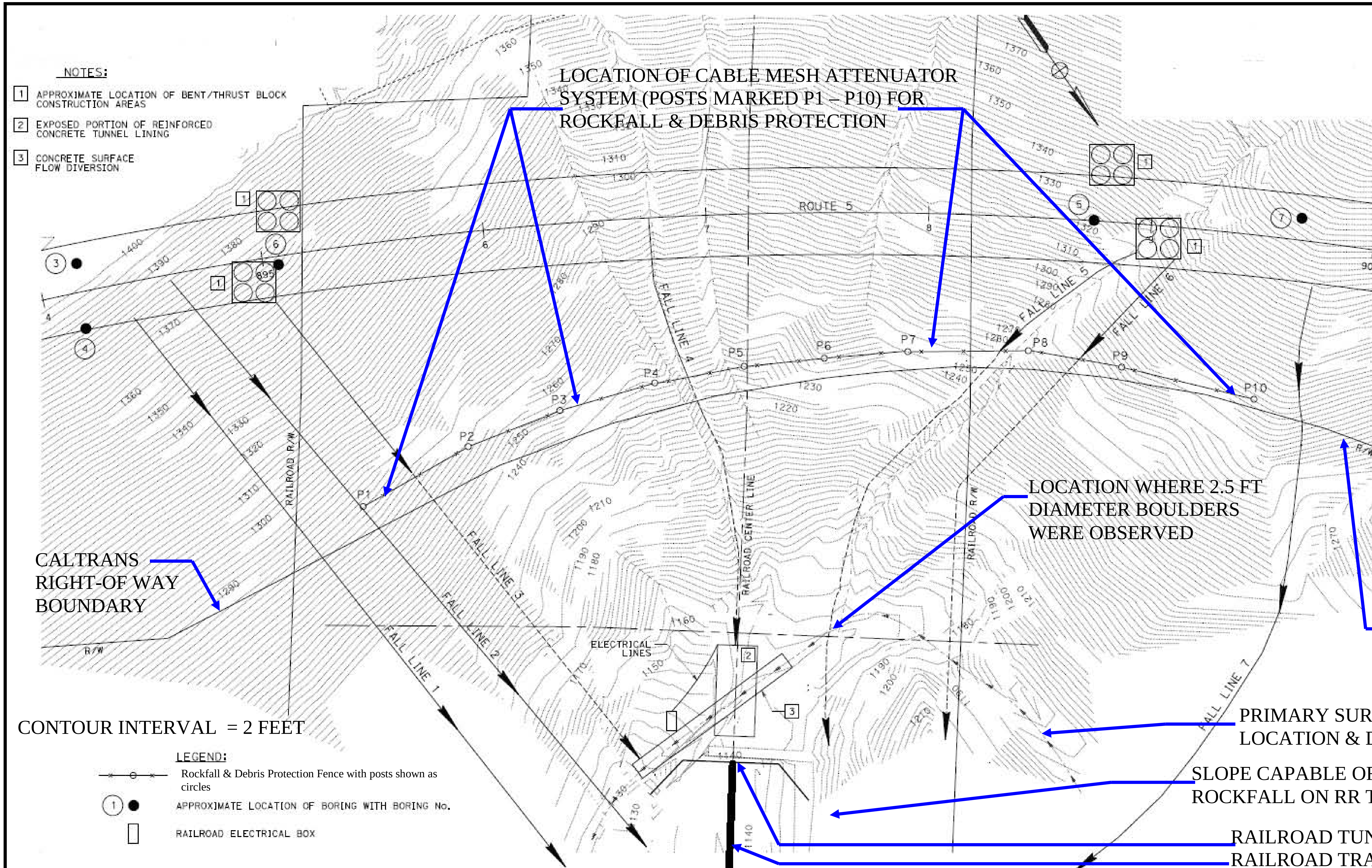
Office of Geotechnical Design-North

EA: 02-0E090	**CONSTRUCTION ACCESS GEOTECHNICAL ZONES (CAGZ)**
Date: June 2015	
02-SHA-5 PM R29.3/R30.3 GEOTECHNICAL DESIGN REPORT	Plate No. 6



Cross-Sectional Drawing Showing a Double Slope Ratio Cut Slope and the Determination of the Depth to Slope Ratio Transition. The top part of the cut has a slope ratio of 1:1 (red), and the bottom of the cut has a 0.75:1 slope ratio (blue). The catchment has a backslope of 6:1. The depths given in Table 1 (North Cut Slope) and Table 2 (South Cut Slope) at which a transition between slope ratios occurs is determined perpendicular to the native topographic surface, as is shown by the pink arrows. This is based on the fact that weathering generally decreases and rock quality generally increases with depth parallel to the native topographic surface in the project area. This has been verified in seismic refraction profiles and field observations of existing cut slopes.

CALTRANS Division of Engineering Services Geotechnical Services Office of Geotechnical Design-North	EA: 02-0E090 Date: June 2015	**DOUBLE SLOPE RATIO CUT SLOPE**
	02-SHA-5 PM R29.3/R30.3 GEOTECHNICAL DESIGN REPORT	Plate No. 7



FALL LINES

These are the likely paths objects would take under the force of gravity when starting at the top of a fall line with no initial inertia. Nearly countless similar fall lines are possible, with the point of origin being the only single determining factor. The select ones shown here were chosen to represent the different areas or locales from which rocks and or debris might originate and possibly (or not possibly) impinge upon the railroad infrastructure. Fall lines are solid where unimpeded by protection system; dashed where impeded by protection system.

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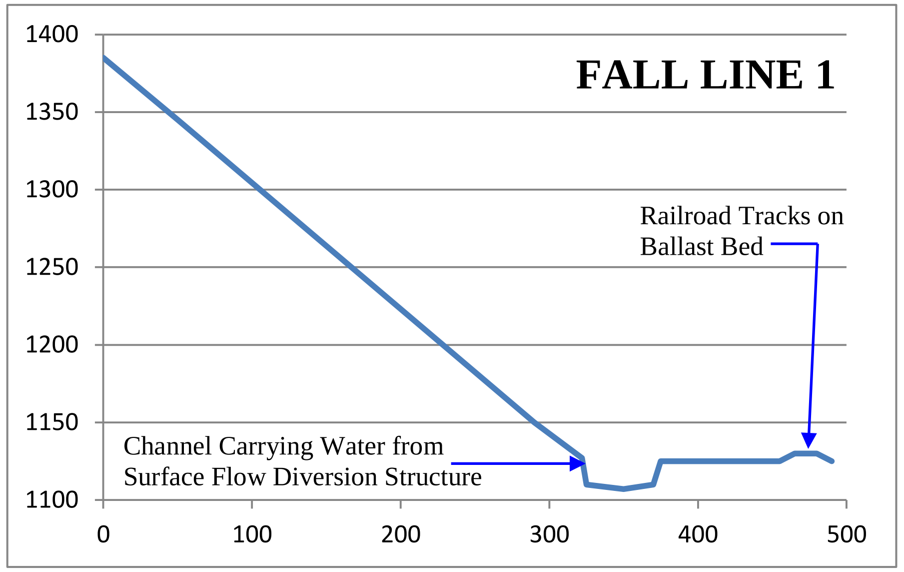
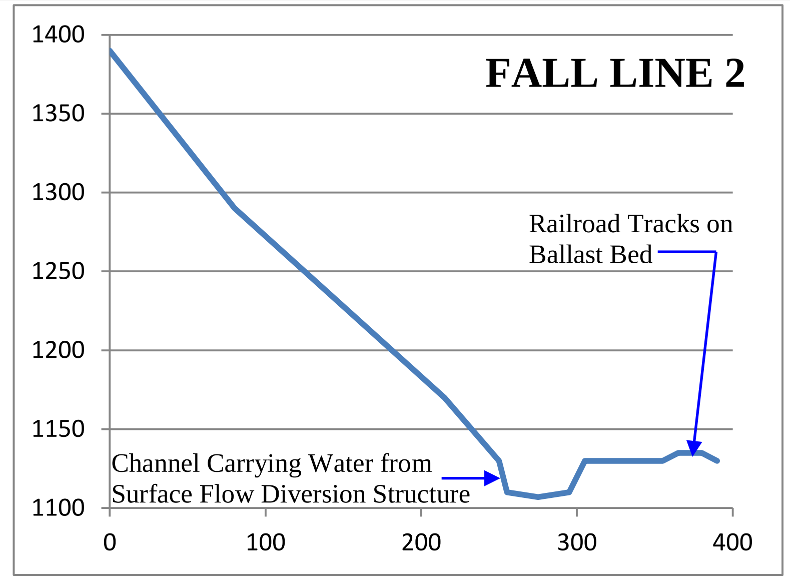
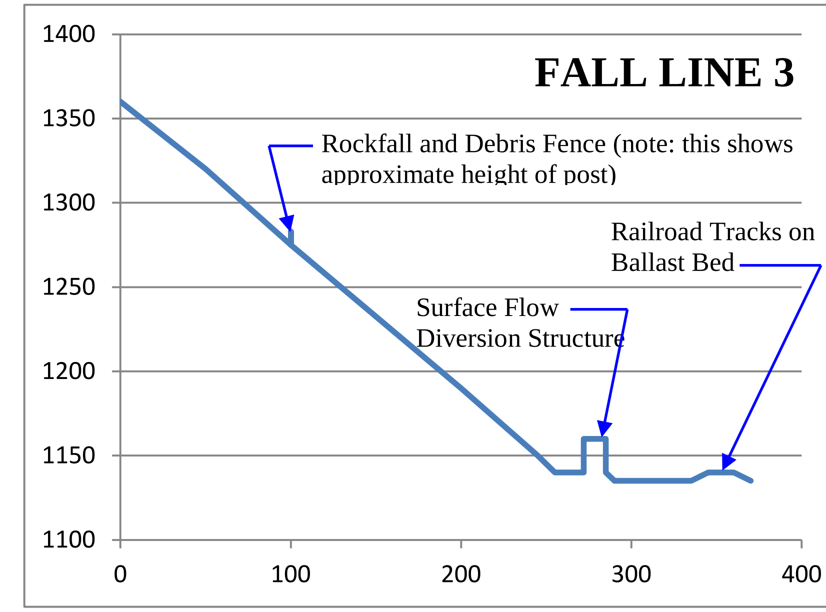
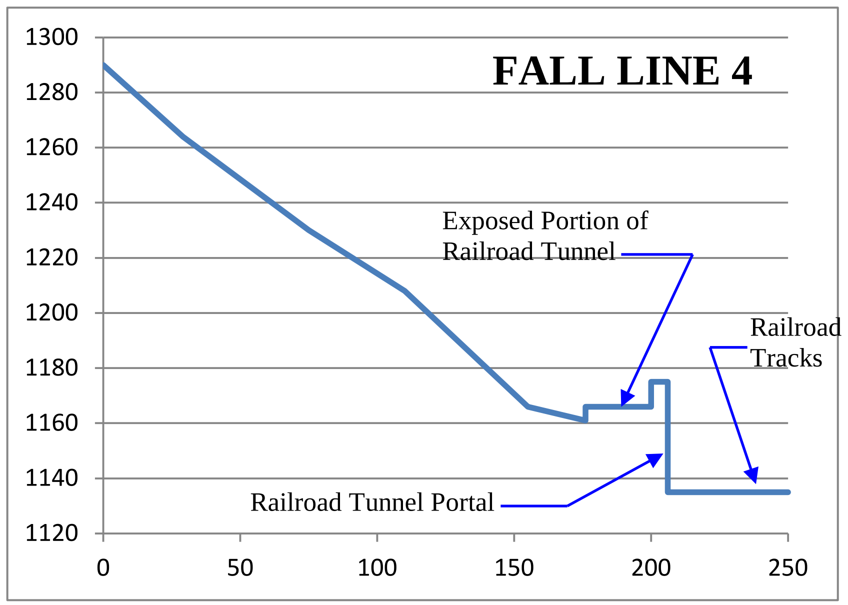
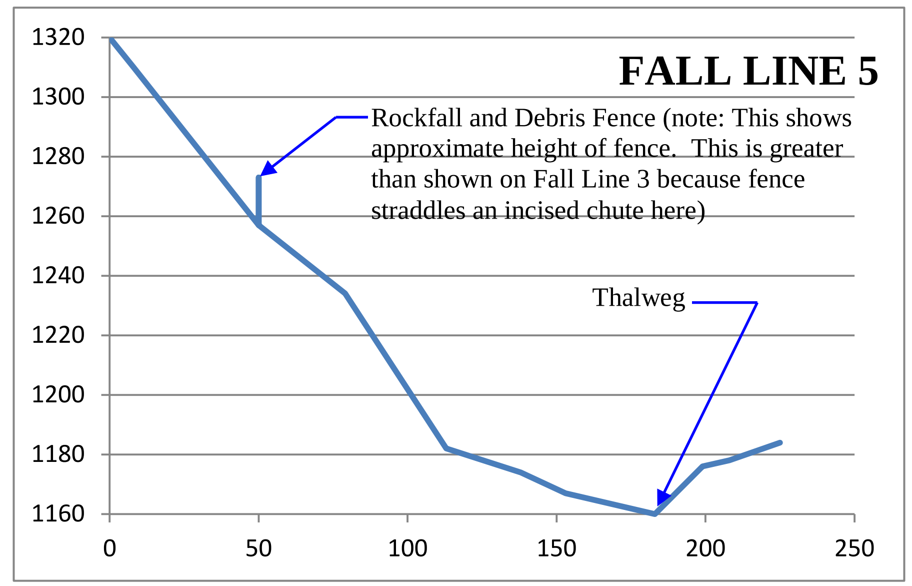
EA: 02-0E090	**LAYOUT OF ROCKFALL & DEBRIS PROTECTION FENCE w/ FALL LINES**
Date: June 2014	
02-SHA-5 PM R29.3/R30.3 GEOTECHNICAL DESIGN REPORT	Plate No.8

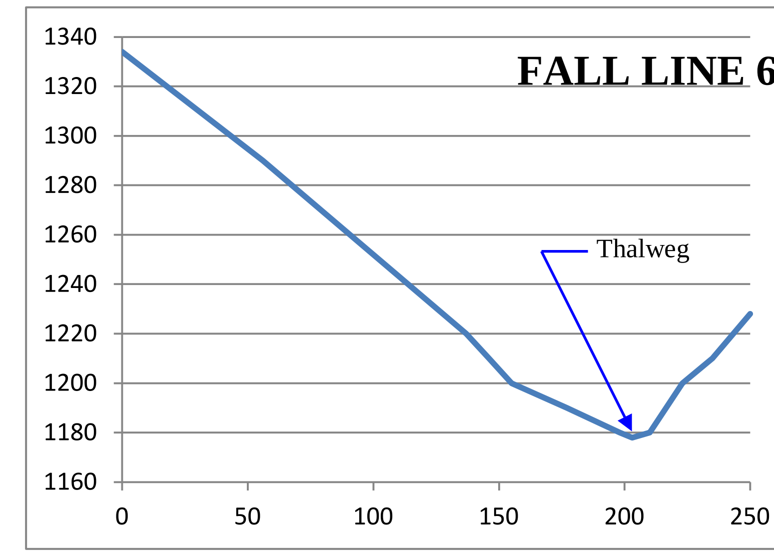


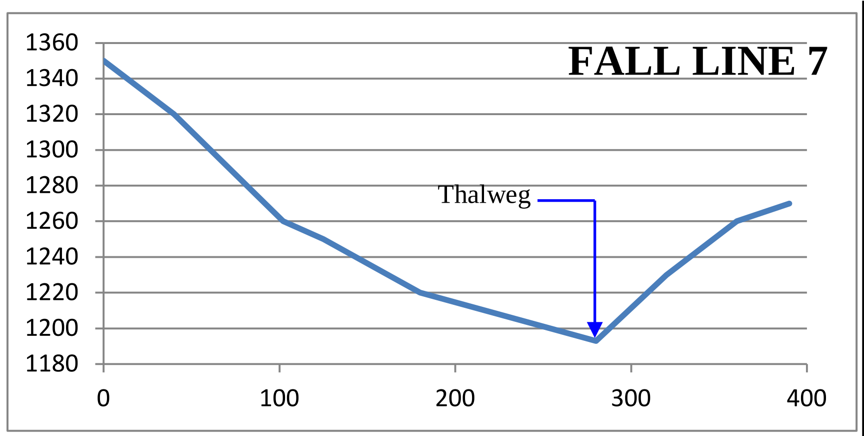












Fall Line Profiles for Rockfall and Debris. Fall line profiles for lines designated on Plate X. Horizontal axes represent horizontal distances in feet. Vertical axes represent elevations in feet. The “0” distance represents the starting point as shown on Plate 8 for that particular Fall Line. The horizontal terminations of the fall line profiles extend beyond the end of the fall lines as shown on Plate X for fall lines 1, 2, 3, 4, and 7. This extension was necessary because the topographic coverage on Plate 8 was insufficient. This extension was created by additional field measurements by OGDN beyond those done by the survey crew. The railroad bed and tracks are shown on fall lines 1, 2, and 3 for the benefit of the reader since the continuation of the fall line direction would eventually encounter the tracks if the rockfall could make it that far (but doesn’t as explained in the text). The tracks only (no bed) are shown on fall line 4 because the continuation of the fall line direction lands directly on the tracks without first encountering the bed. Neither the tracks nor the bed are shown on fall lines 5, 6, or 7 because of the large topographic rise that would clearly impede the further travel needed to reach the tracks.

APPENDIX 1

Depth Section for Seismic Refraction Line 1

Depth Section for Seismic Refraction Line 2

Depth Section for Seismic Refraction Line 3

Depth Section for Seismic Refraction Line 4

Depth Section for Seismic Refraction Line 5

Depth Section for Seismic Refraction Line 6

Depth Section for Seismic Refraction Line 7

Depth Section for Seismic Refraction Line 8

Depth Section for Seismic Refraction Line 9

Depth Section for Seismic Refraction Line 10

Depth Section for Seismic Refraction Line 11

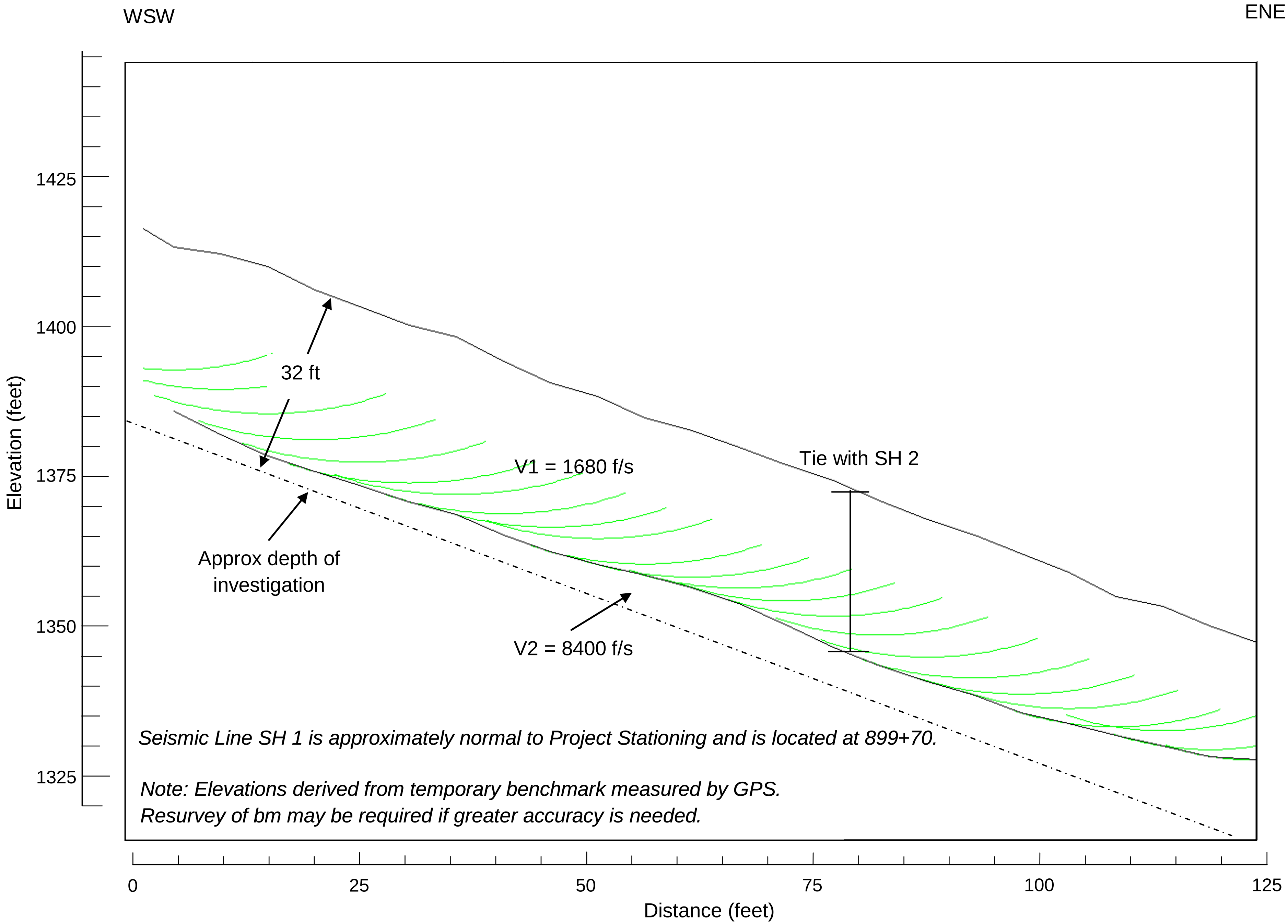
Depth Section for Seismic Refraction Line 12

Depth Section for Seismic Refraction Line 13

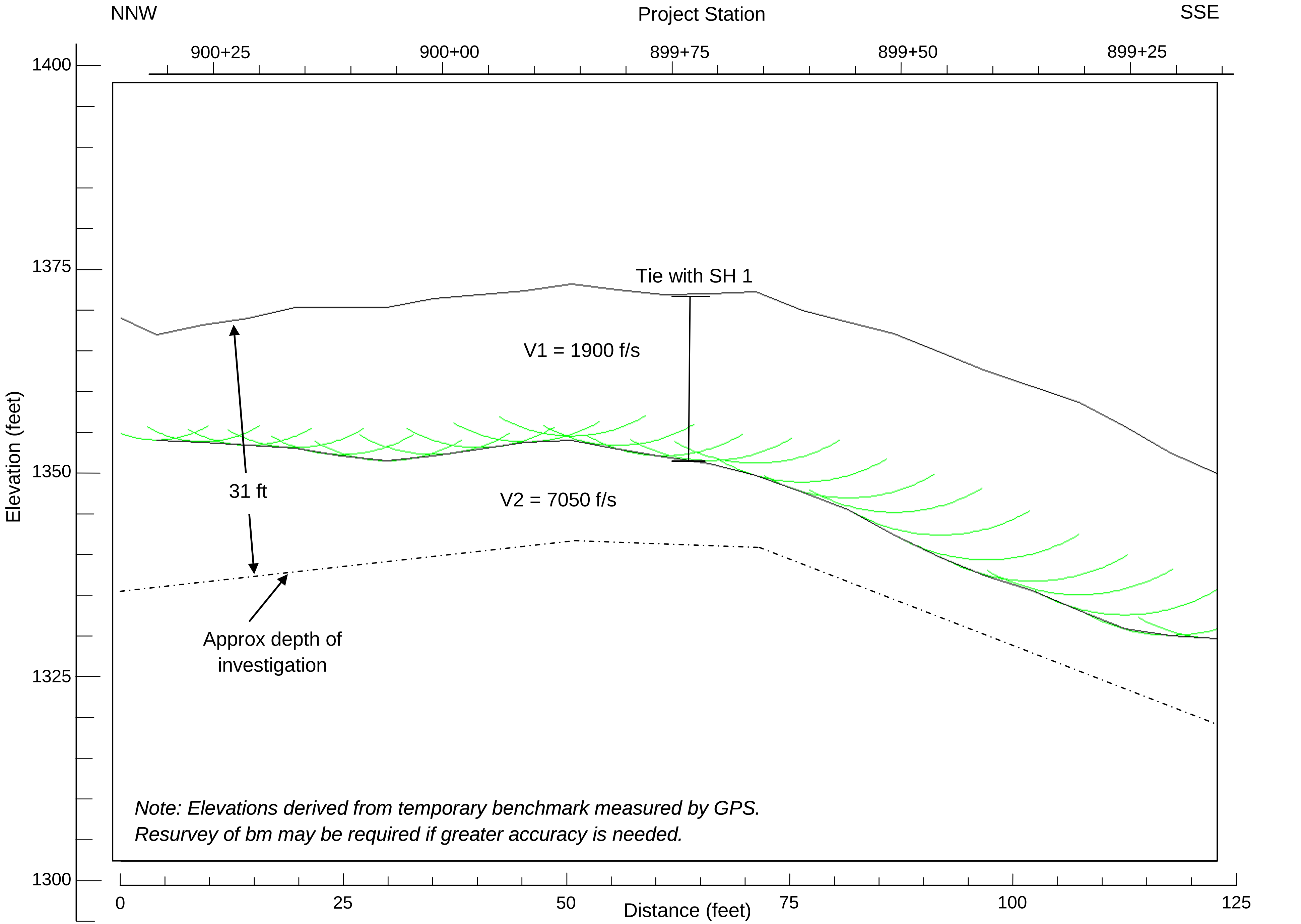
Depth Section for Seismic Refraction Line 14

Depth Section for Seismic Refraction Line 15

Tomography Survey Velocity Depth Section



Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project	APP1
	0200000016	Seismic Line SH 1	



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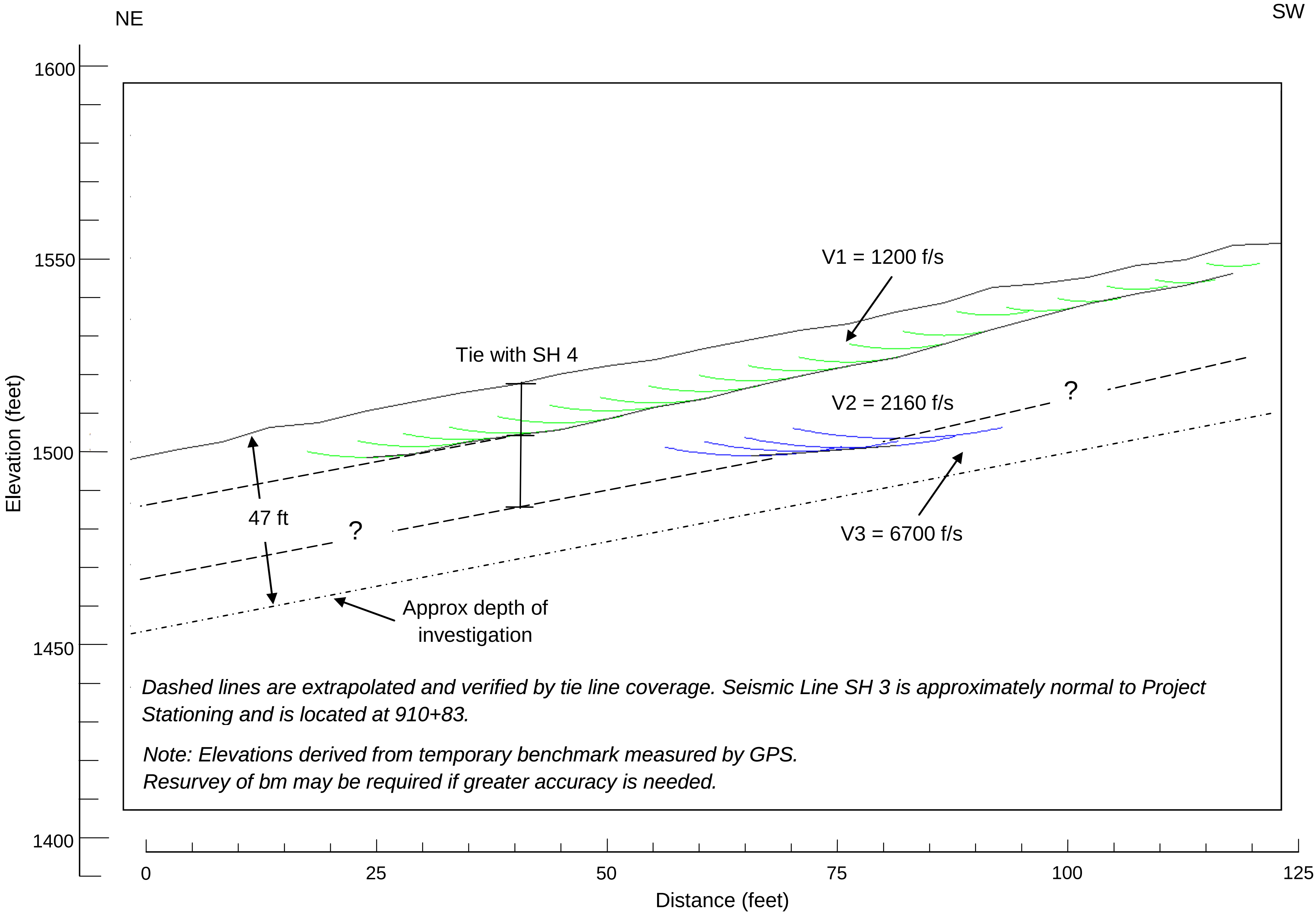
	0200000016	Seismic Line SH 2	



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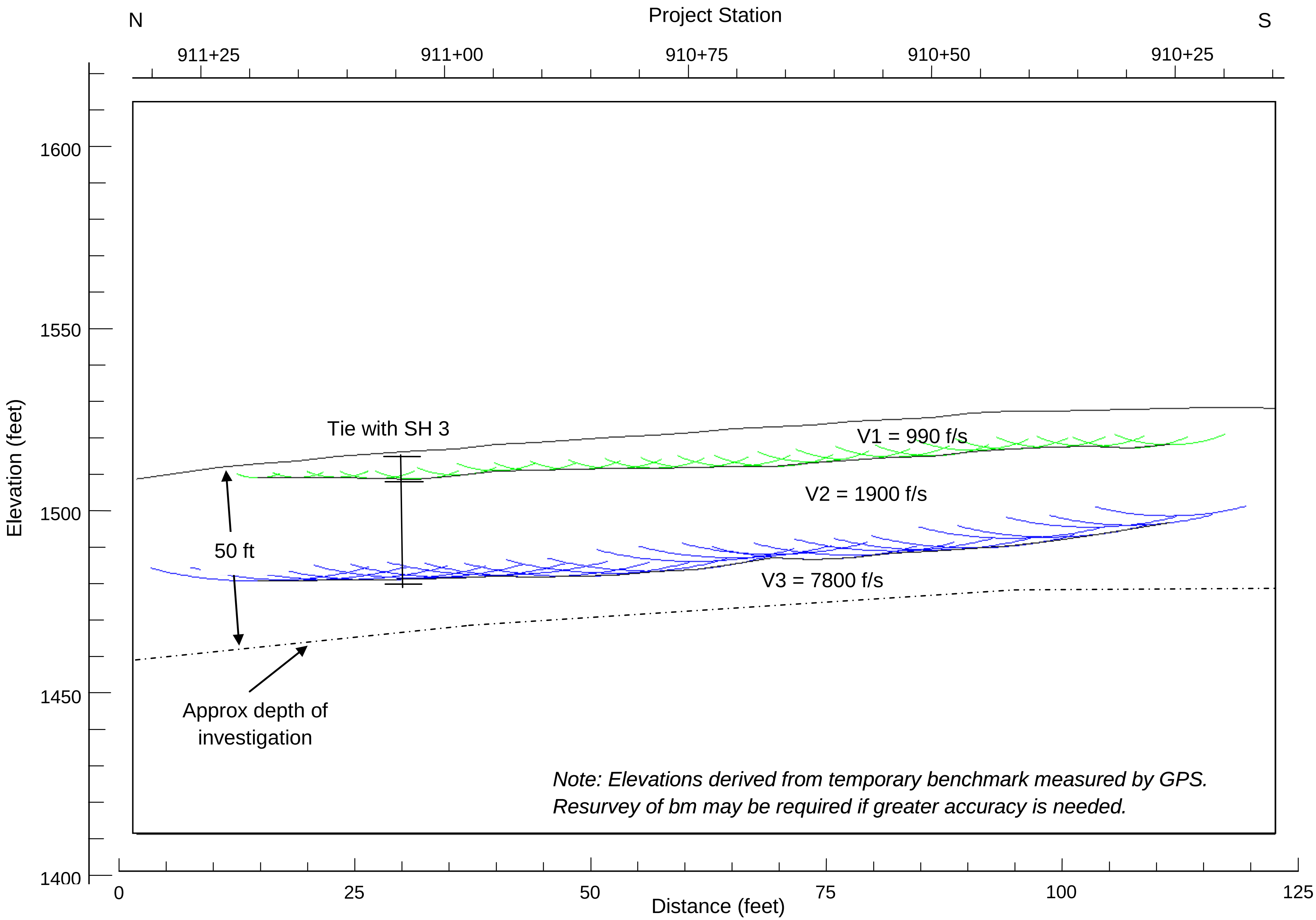
	0200000016	Seismic Line SH 3	



Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project	APP1
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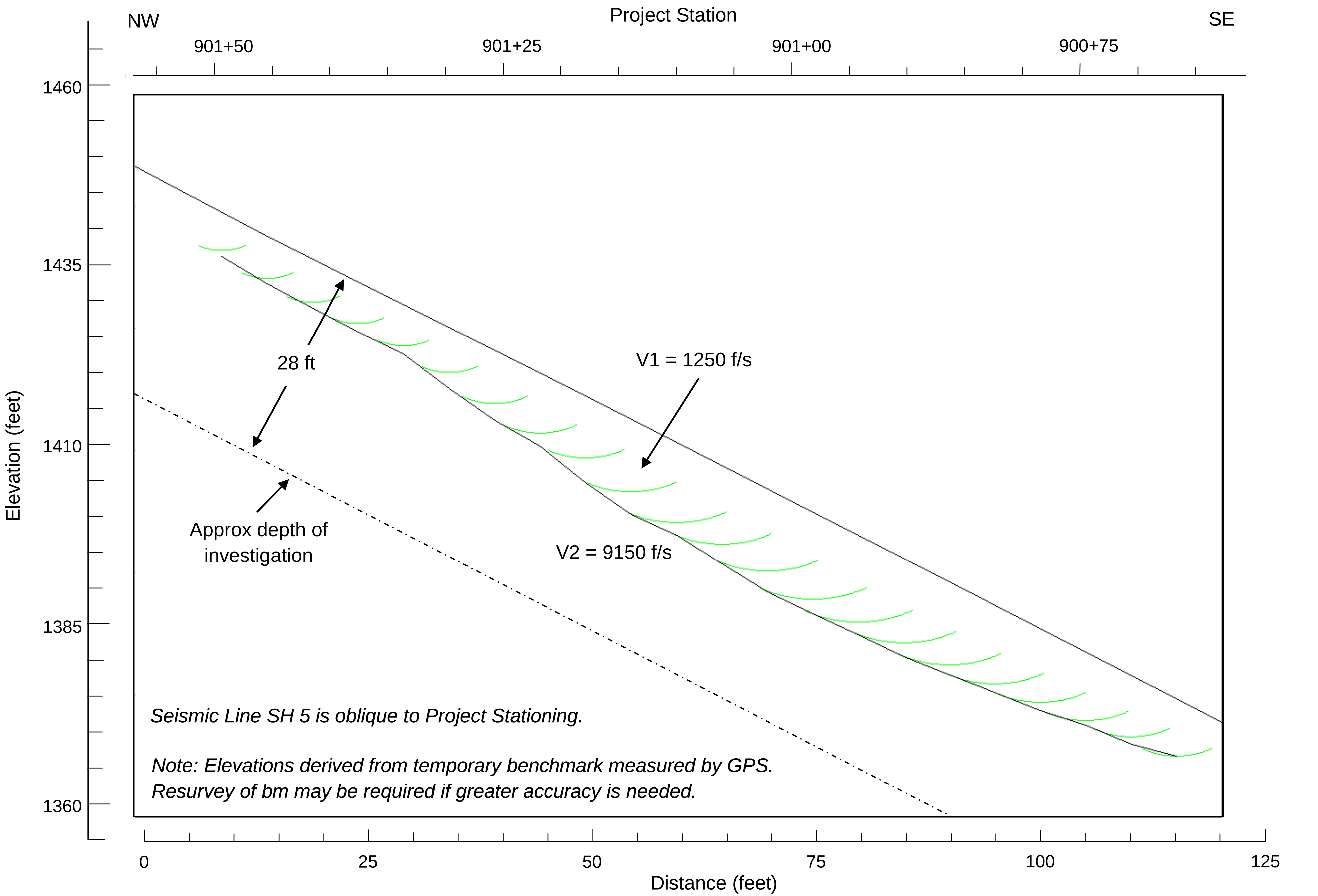

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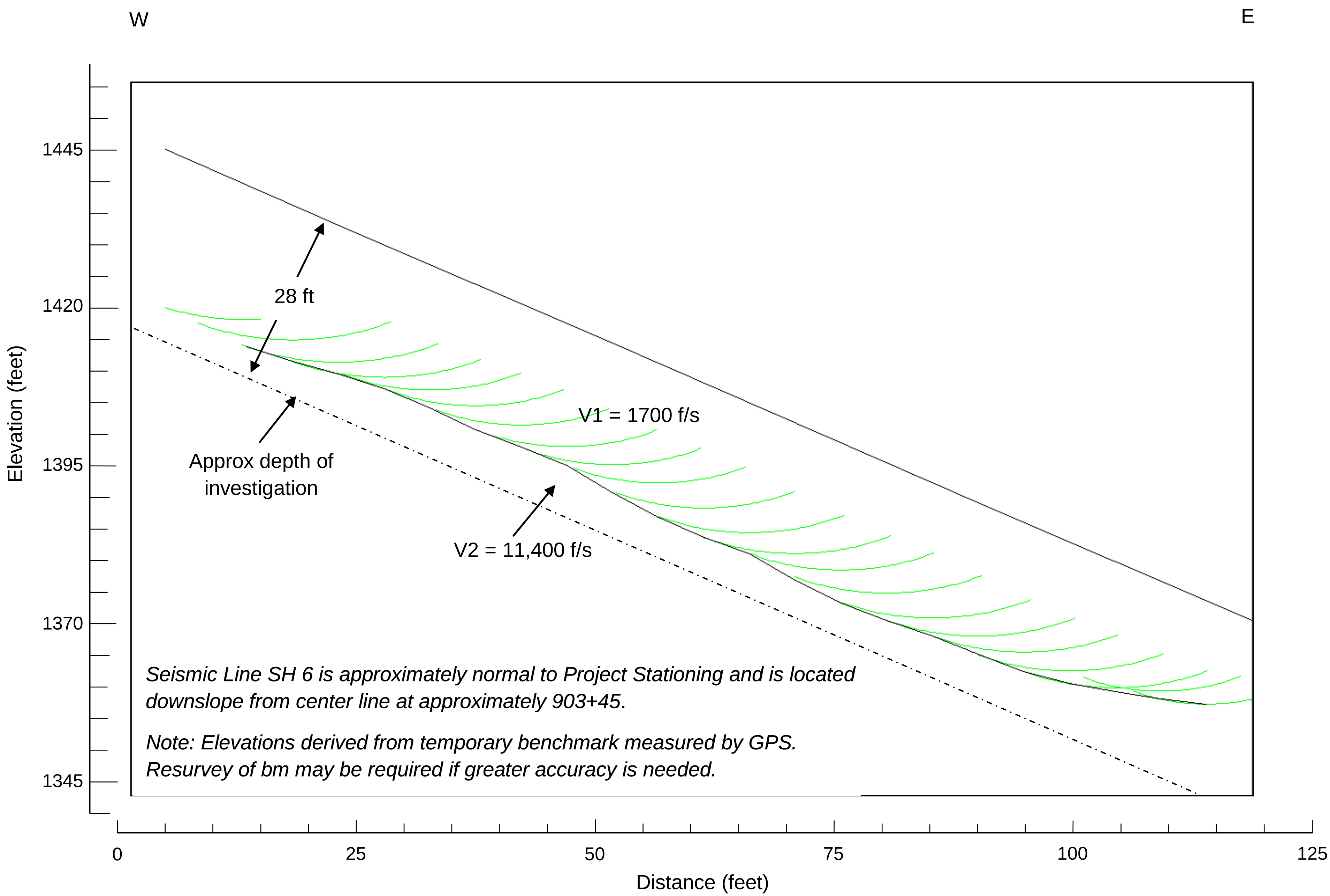

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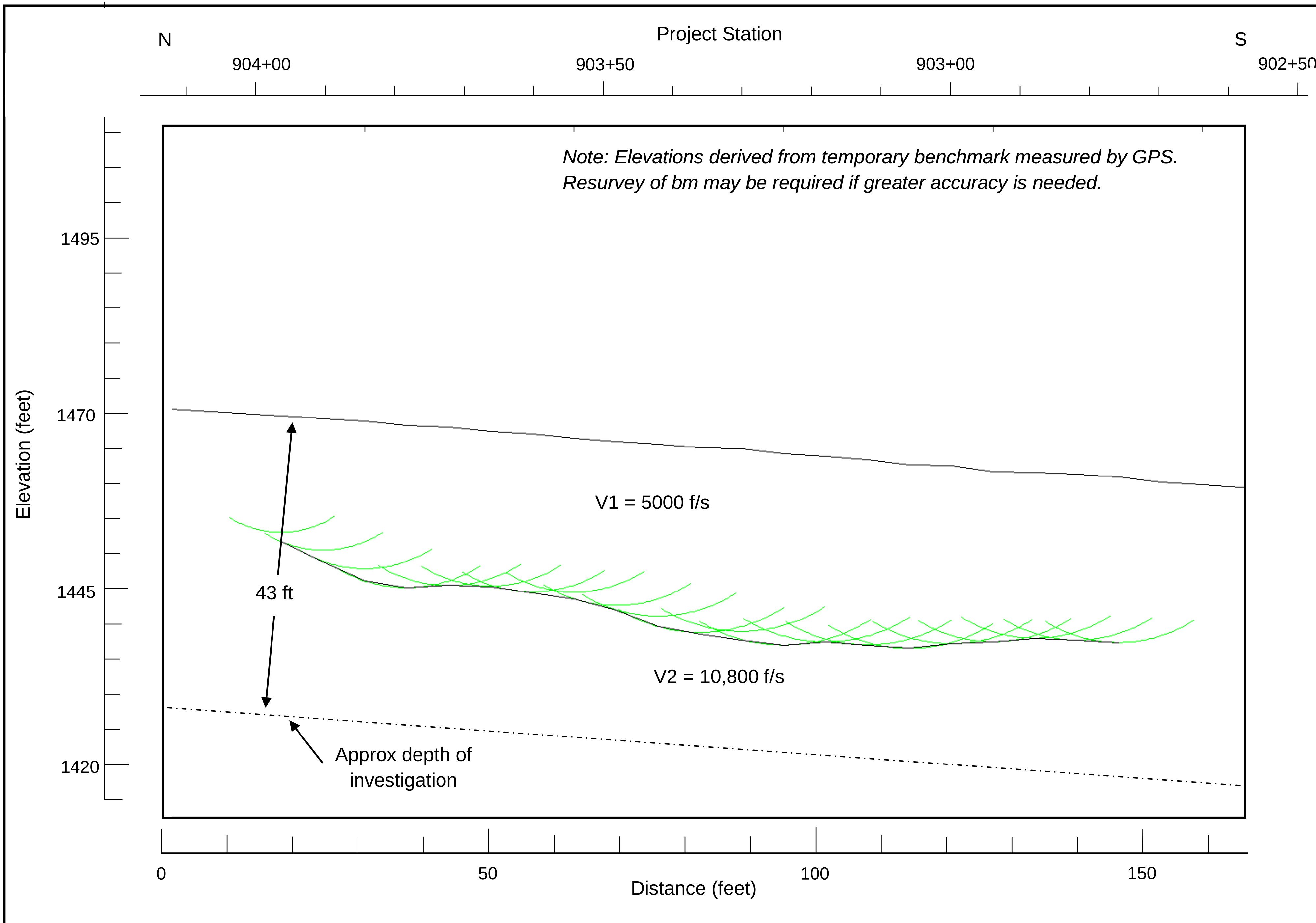
	0200000016	Seismic Line SH 6	



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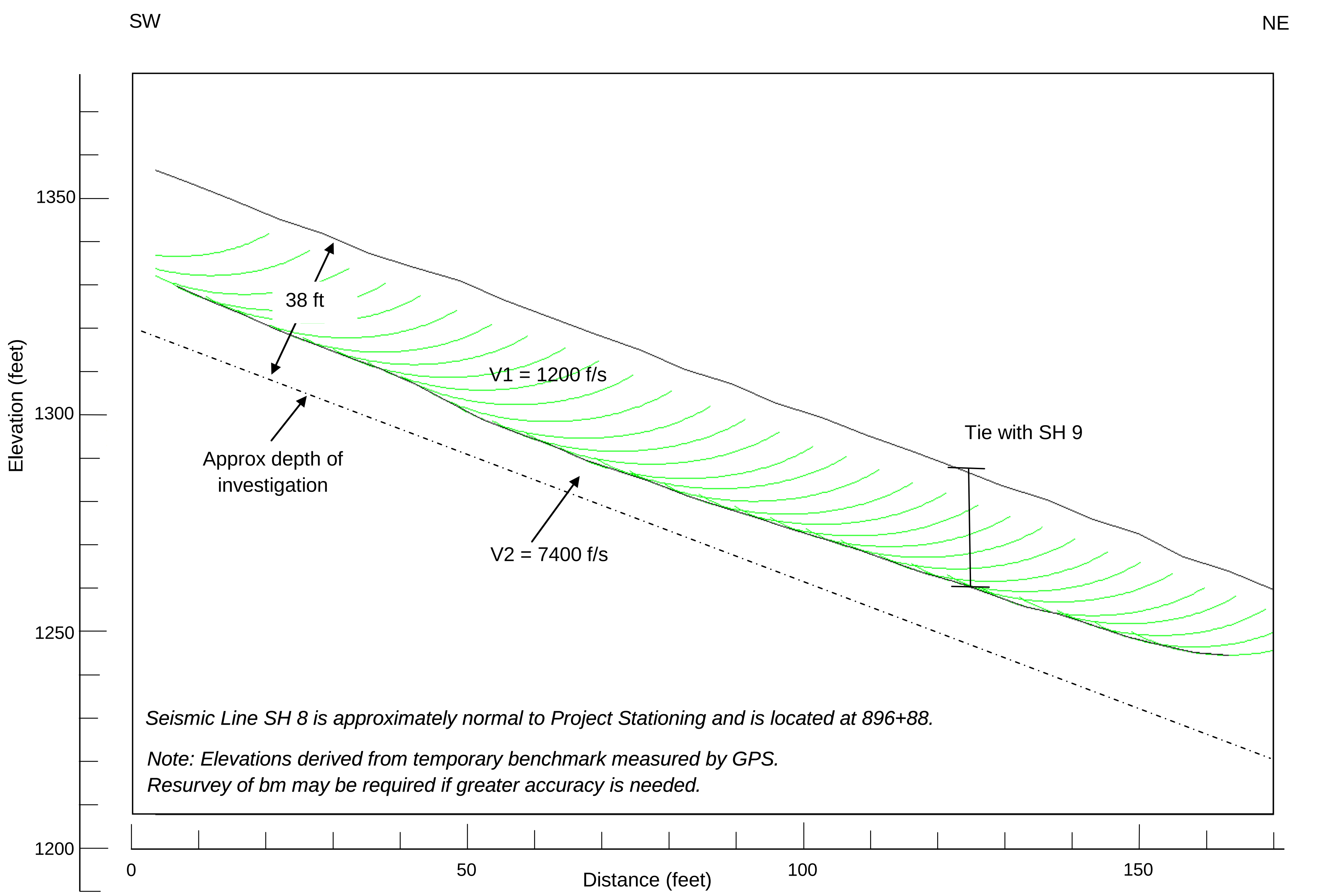

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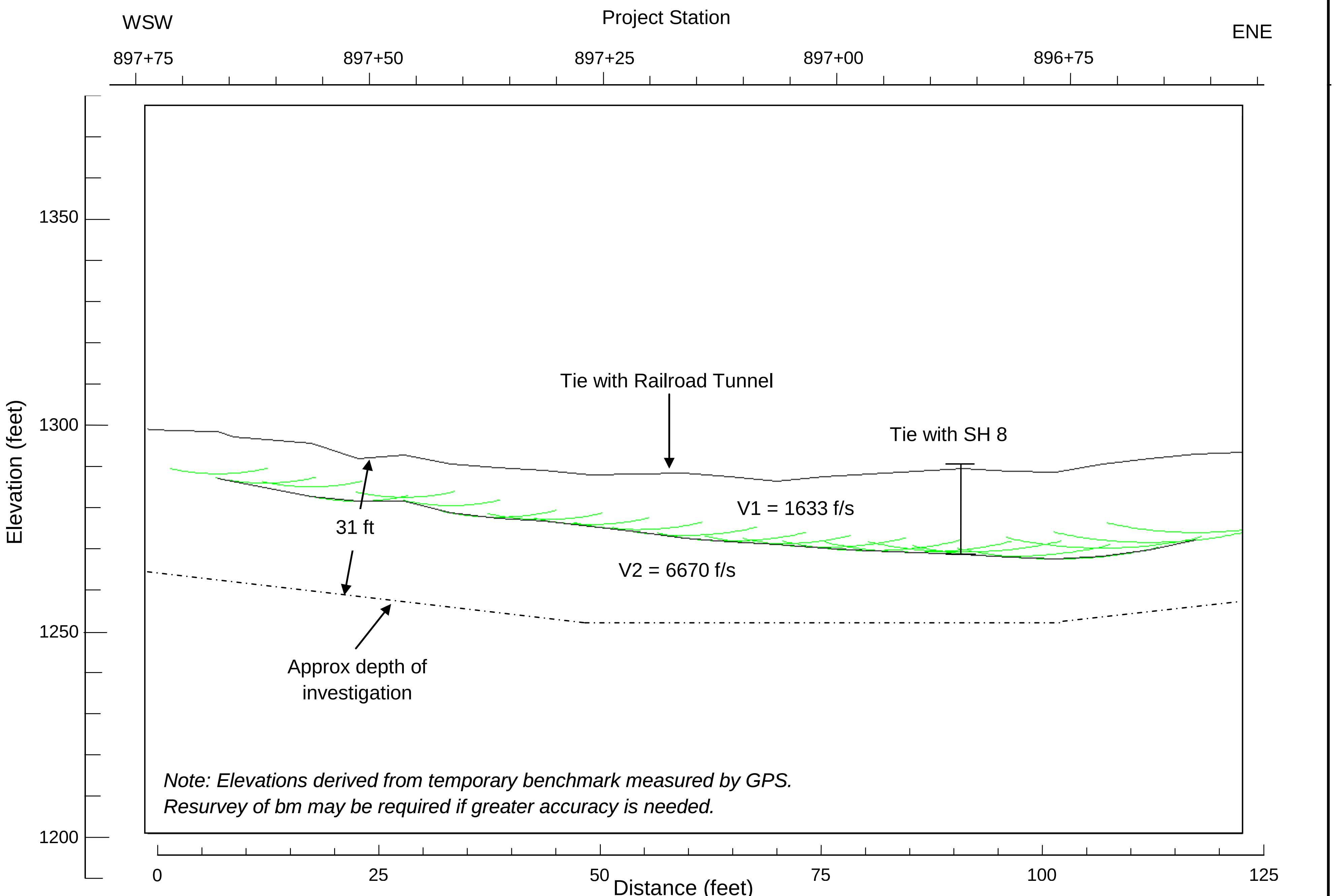
	0200000016	Seismic Line SH 8	



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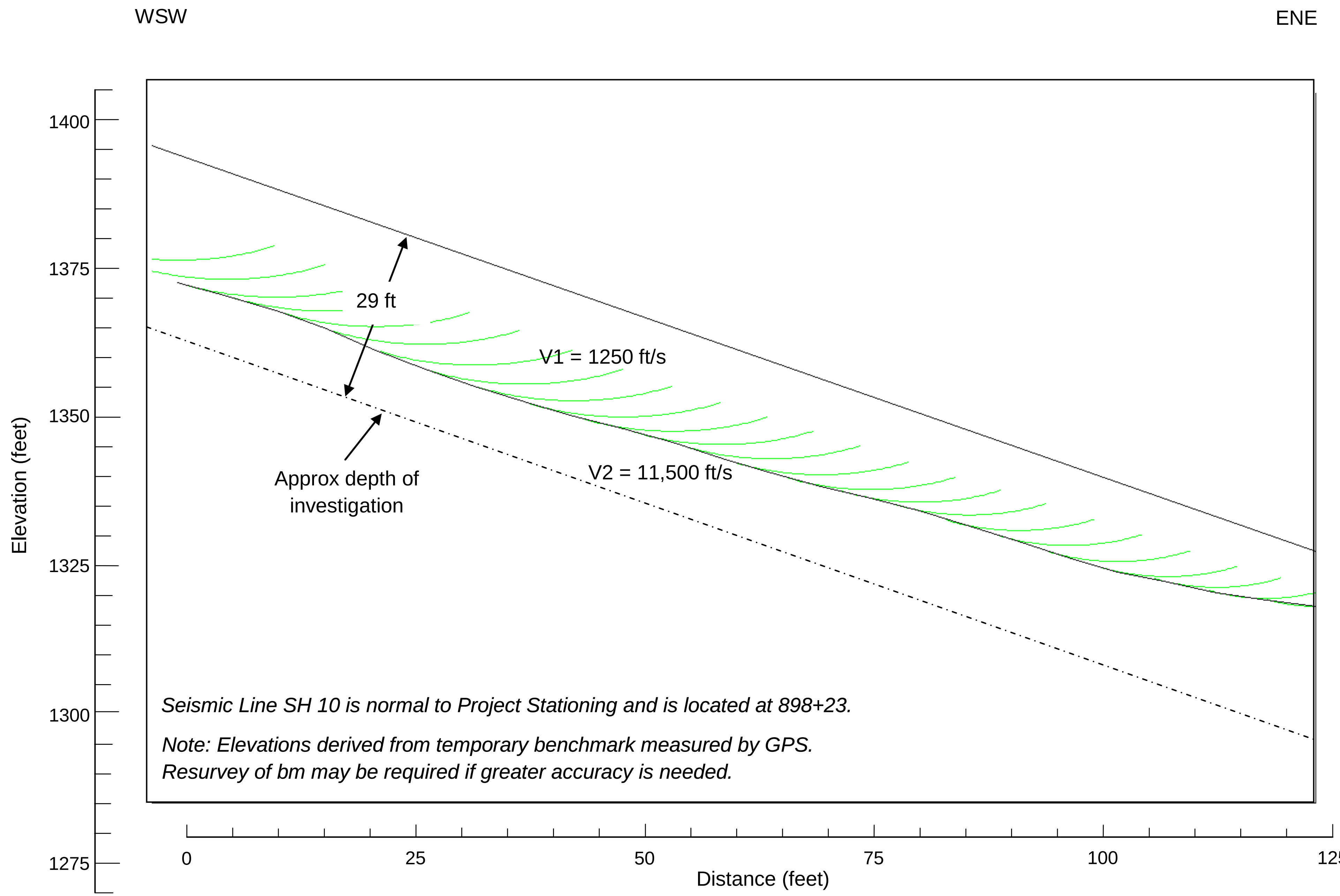
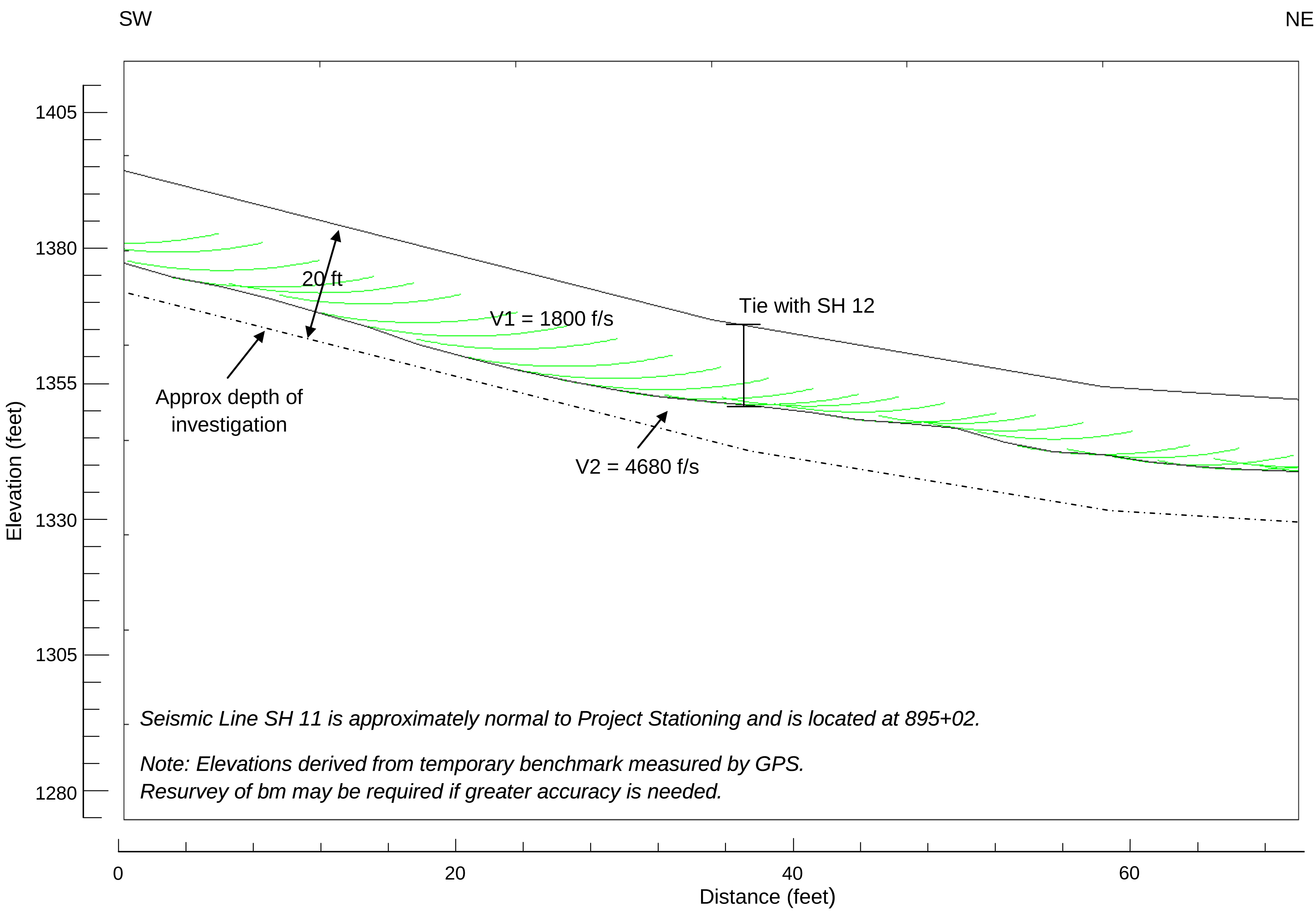
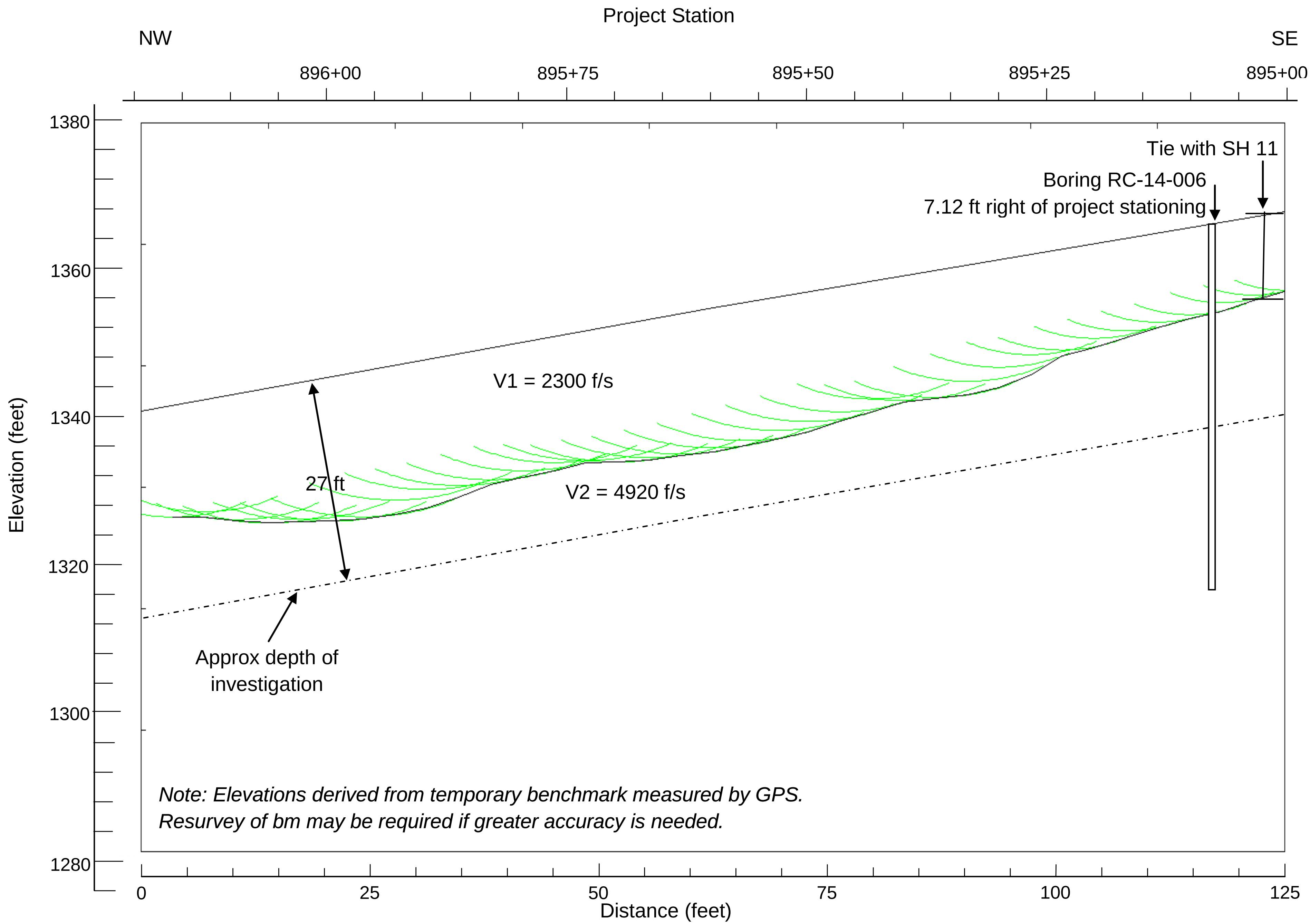


Division of Engineering Services **Office of Geotechnical Support** **Geophysics and Geology Branch**	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project	APP1
	0200000016	Seismic Line SH 10	



Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project	APP1
	0200000016	Seismic Line SH 11	



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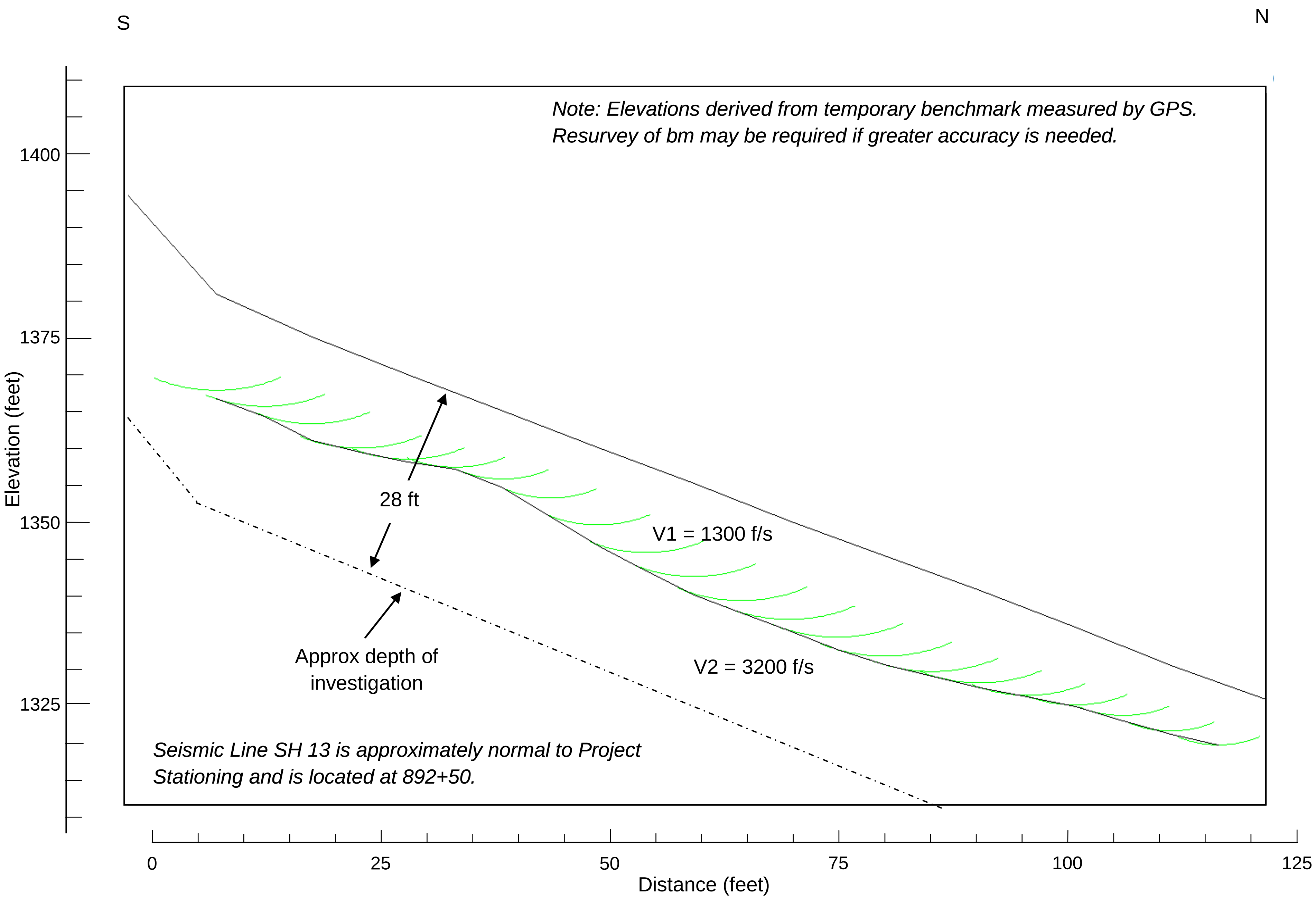

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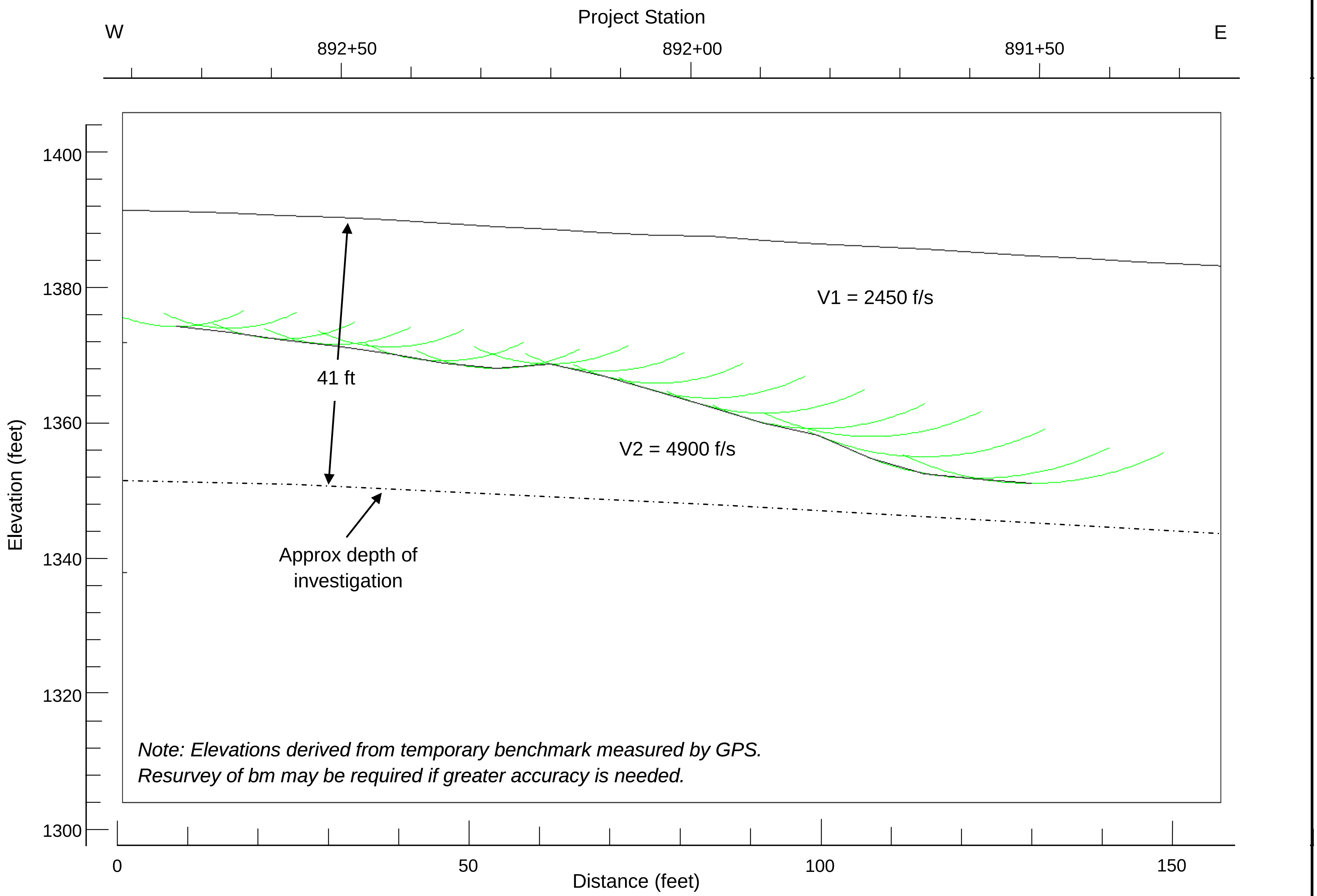
	0200000016	Seismic Line SH 13	



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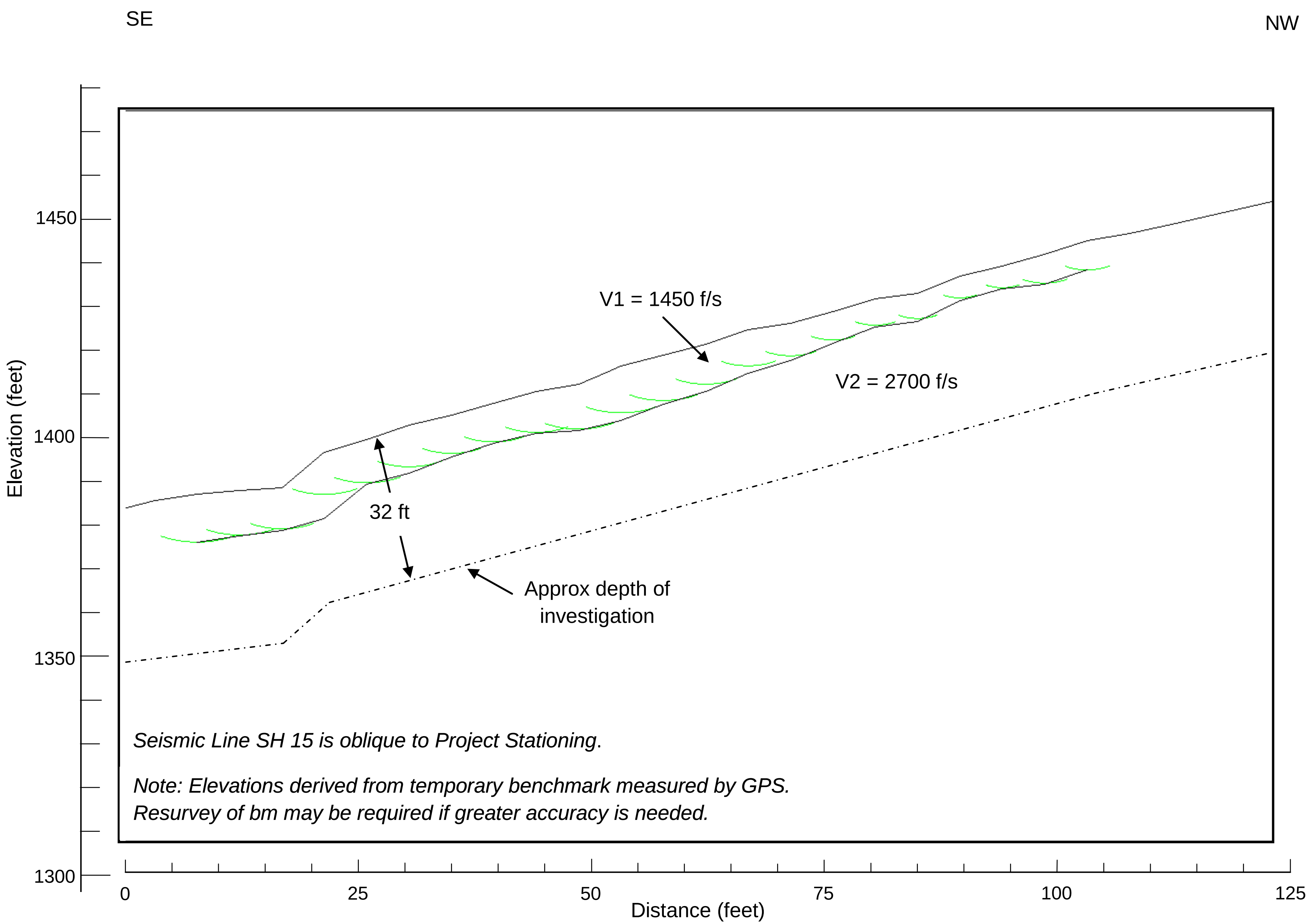
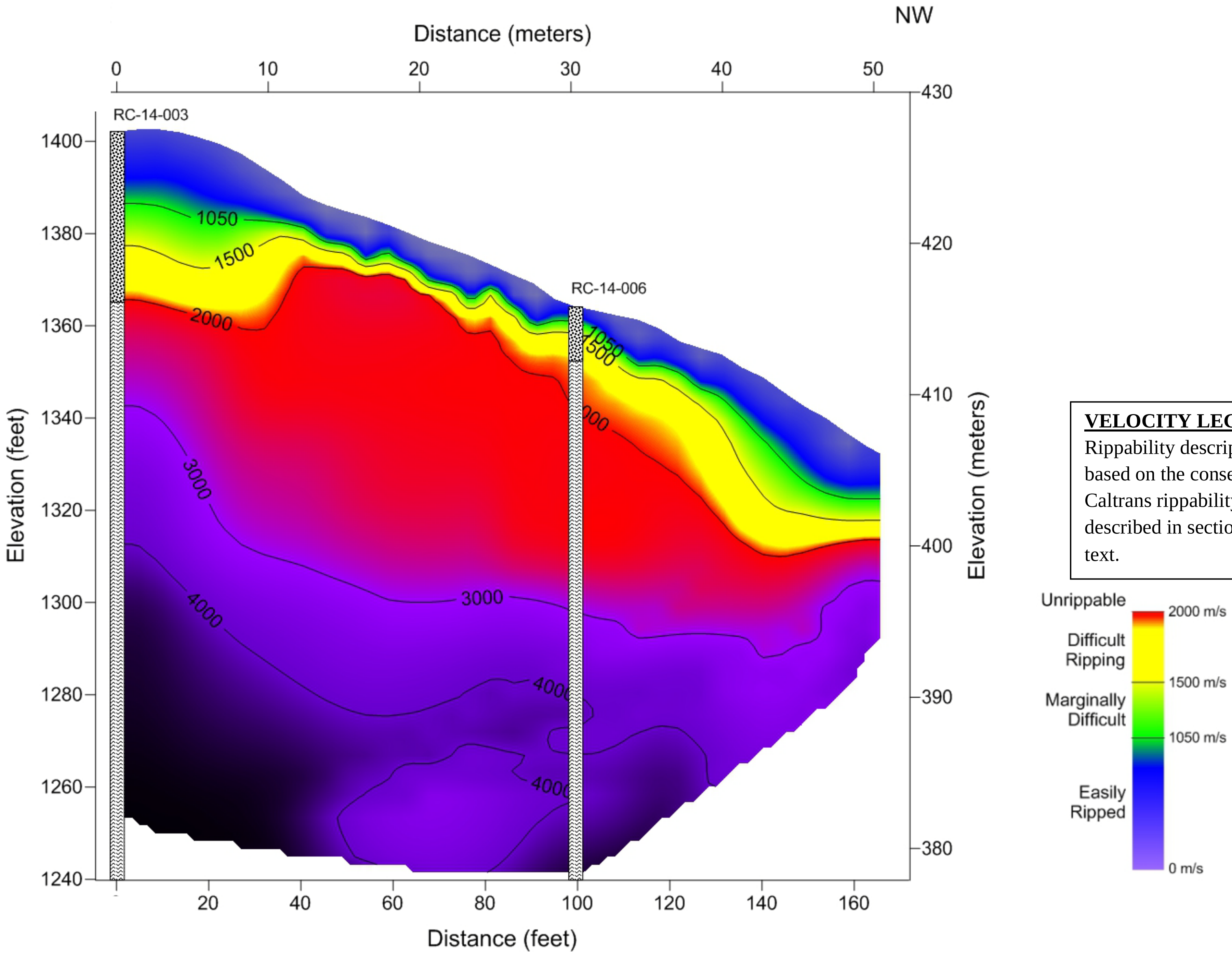
	0200000016	Seismic Line SH 14	



Seismic Line SH 15 is oblique to Project Stationing.

Note: Elevations derived from temporary benchmark measured by GPS.
Resurvey of bm may be required if greater accuracy is needed.

Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project	APP1
	0200000016	Seismic Line SH 15	



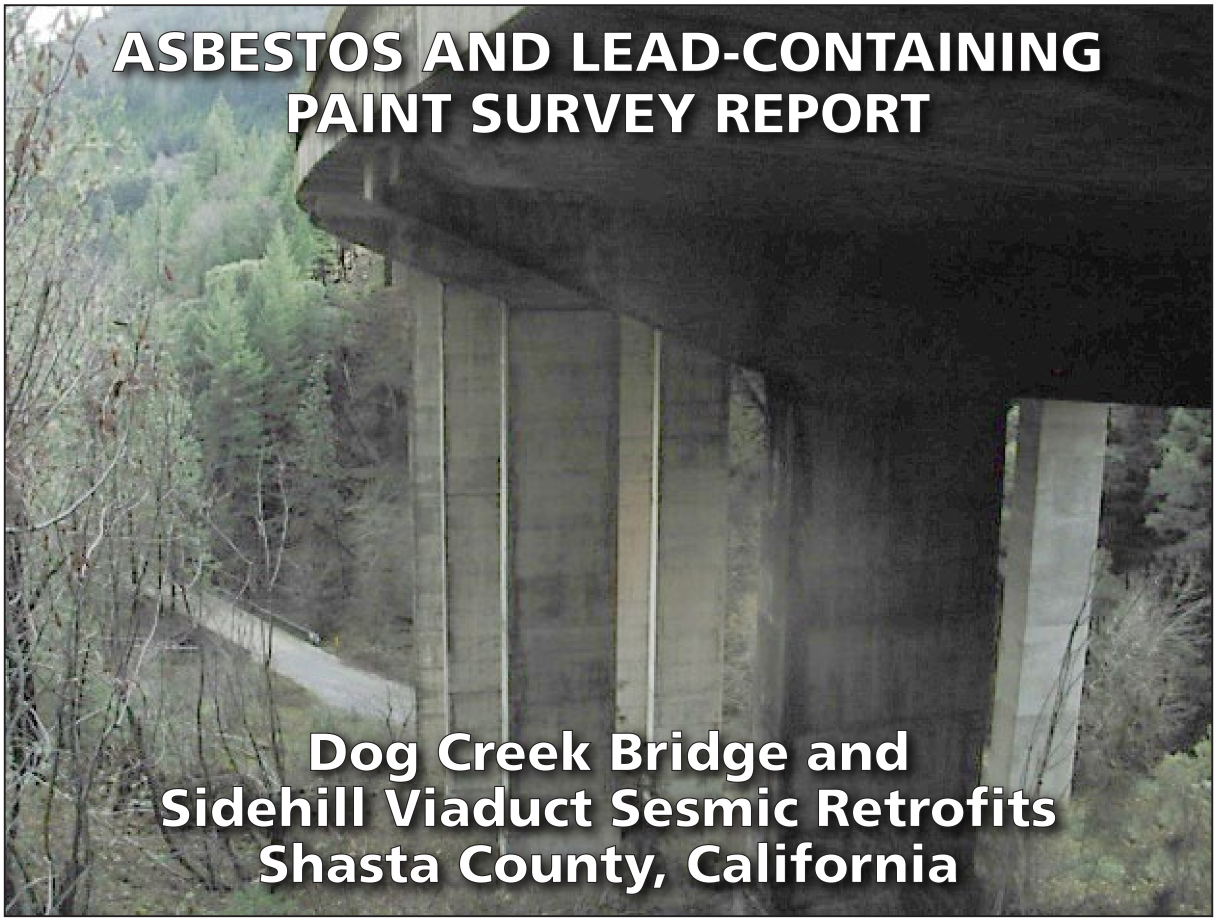
VELOCITY LEGEND.
Rippability descriptions are based on the conservative Caltrans rippability scale described in section 8.1.2 in text.

Division of Engineering Services Office of Geotechnical Support Geophysics and Geology Branch	EA 02-0E090	Sidehill Viaduct Bridge Replacement Project	APP1
	0200000016	Tomography Survey Velocity Depth Section	

MATERIALS INFORMATION

Asbestos and Lead-Containing Paint Survey Report,

Dated March 2015



ASBESTOS AND LEAD-CONTAINING PAINT SURVEY REPORT

Dog Creek Bridge and Sidehill Viaduct Sesmic Retrofits Shasta County, California

PREPARED FOR:

CALIFORNIA DEPARTMENT OF TRANSPORTATION
DISTRICT 3
703 B STREET, P.O. BOX 911
MARYSVILLE, CALIFORNIA 95901

PREPARED BY:

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

GEOCON PROJECT NO. S9805-01-46
TASK ORDER NO. 46
E-FIS 02-0000-0016 (EA 02-0E0901)
CONTRACT NO. 03A2132

MARCH 2015

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. S9805-01-46
March 23, 2015

Mr. Rajive Chadha
California Department of Transportation - District 3
Environmental Engineering Office
703 B Street
Marysville, California 95901

Subject: ASBESTOS AND LEAD-CONTAINING PAINT SURVEY REPORT
DOG CREEK BRIDGE AND SIDEHILL VIADUCT SEISMIC RETROFITS
SHASTA COUNTY, CALIFORNIA
CONTRACT NO. 03A2132, E-FIS 02-0000-0016, EA 02-0E0901
TASK ORDER NO. 46

Dear Mr. Chadha:

In accordance with California Department of Transportation (Caltrans) Contract No. 03A2132 and Task Order No. 46, we have performed an asbestos and lead-containing paint survey of the Dog Creek Bridge (06-0027) and Sidehill Viaduct (06-0042) in Shasta County, California. Our scope of services included surveying the structures for suspect asbestos-containing materials and lead-containing paint, collecting bulk samples, and submitting the samples to a laboratory for analysis.

The accompanying report summarizes the services performed and laboratory analysis.

The contents of this report reflect the views of Geocon Consultants, Inc., who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Please contact us if you have questions concerning the contents of this report or if we may be of further service.

Sincerely,

GEOCON CONSULTANTS, INC.

David A. Watts, CAC No. 98-2404
Senior Project Scientist

John E. Juhrend, PE, CEG
Principal/Senior Engineer

(2 + 2 CDs) Addressee

3160 Gold Valley Drive, Suite 800 ■ Rancho Cordova, CA 95742-7515 ■ Telephone 916.852.9118 ■ Fax 916.852.9132

TABLE OF CONTENTS

ASBESTOS AND LEAD-CONTAINING PAINT SURVEY REPORT

1.0 INTRODUCTION

This asbestos and lead-containing paint (LCP) survey report was prepared by Geocon Consultants, Inc. under Caltrans Contract No. 03A2132, Task Order No. 46 (TO-46).

1.1 Project Description

The project consists of the Dog Creek Bridge (06-0027) and Sidehill Viaduct (06-0042) at Post Miles (PM) 45.66 and 29.72, respectively, on Interstate 5 in Shasta County, California. We performed asbestos and LCP survey activities at the project locations. The project locations are depicted on the Vicinity Map, Figure 1, and Site Plan, Figure 2.

1.2 General Objectives

The purpose of the scope of services outlined in TO-46 was to determine the presence and quantity of asbestos and LCP at the project locations prior to various activities associated with seismic retrofits. The information obtained from this investigation will be used by Caltrans for waste profiling, determining California Occupational Safety and Health Administration (Cal/OSHA) applicability, and coordinating asbestos and LCP disturbance activities.

> *It was <u>not</u> Geocon's intent during this inspection to conduct an evaluation of lead-based paint hazards in accordance with U.S. Department of Housing and Urban Development (HUD) guidelines.*

2.0 BACKGROUND

2.1 Asbestos

The Code of Federal Regulations (CFR), 40 CFR 61, Subpart M, National Emissions Standards for Hazardous Air Pollutants (NESHAP) and Federal Occupational Safety and Health Administration (FED OSHA) classify asbestos-containing material (ACM) as any material or product that contains *greater than* 1% asbestos. Nonfriable ACM is classified by NESHAP as either Category I or Category II material defined as follows:

- **Category I** – asbestos-containing packings, gaskets, resilient floor coverings, and asphalt roofing products.
- **Category II** – all remaining types of nonfriable asbestos-containing material not included in Category I that when dry, cannot be crumbled, pulverized, or reduced to powder by hand pressure.

Regulated asbestos-containing material (RACM), a hazardous waste when friable, is classified as any manufactured material that contains *greater than* 1% asbestos by dry weight *and* is:

- Friable (can be crumbled, pulverized, or reduced to powder by hand pressure); or
- Category I material that has become friable; or
- Category I material that has been subjected to sanding, grinding, cutting, or abrading; or
- Category II nonfriable material that has a high probability of becoming crumbled, pulverized, or reduced to a powder during demolition or renovation activities.

Activities that disturb materials containing *any* amount of asbestos are subject to certain requirements of the Cal/OSHA asbestos standard contained in Title 8 of the California Code of Regulations (CCR) §1529. Typically, removal or disturbance of more than 100 square feet of material containing more than 0.1% asbestos must be performed by a registered asbestos abatement contractor, but associated waste labeling is not required if the material contains 1% or less asbestos. When the asbestos content of a material exceeds 1%, virtually all requirements of the standard become effective.

Materials containing more than 1% asbestos are also subject to NESHAP regulations (40 CFR Part 61, Subpart M). RACM (friable ACM and nonfriable ACM that will become friable during demolition operations) must be removed from structures prior to demolition. Certain nonfriable ACM and materials containing 1% or less asbestos may remain in structures during demolition; however, there are waste handling/disposal issues and Cal/OSHA work requirements that must be addressed. Contractors are responsible for segregating and characterizing waste streams prior to disposal.

With respect to potential worker exposure, notification, and registration requirements, Cal/OSHA defines asbestos-containing construction material (ACCM) as construction material that contains more than 0.1% asbestos (Title 8, CCR 341.6).

2.2 Lead Paint

Construction activities (including demolition) that disturb materials or paints containing *any* amount of lead are subject to certain requirements of the Cal/OSHA lead standard contained in Title 8, CCR, §1532.1. Deteriorated paint is defined by Title 17, CCR, Division 1, Chapter 8, §35022 as a surface coating that is cracking, chalking, flaking, chipping, peeling, non-intact, failed, or otherwise separating from a substrate. Demolition of a deteriorated LCP component would require waste characterization and appropriate disposal. Intact LCP on a component is currently accepted by most landfills and recycling facilities; however, contractors are responsible for segregating and characterizing waste streams prior to disposal.

For a solid waste containing lead, the waste is classified as California hazardous when: 1) the representative total lead content equals or exceeds the respective Total Threshold Limit Concentration

(TTLC) of 1,000 milligrams per kilogram (mg/kg); or 2) the representative soluble lead content equals or exceeds the respective Soluble Threshold Limit Concentration (STLC) of 5 milligrams per liter (mg/l) based on the standard Waste Extraction Test (WET). A waste has the potential for exceeding the lead STLC when the waste's total lead content is greater than or equal to ten times the respective STLC value since the WET uses a 1:10 dilution ratio. Hence, when total lead is detected at a concentration greater than or equal to 50 mg/kg, and assuming that 100 percent of the total lead is soluble, soluble lead analysis is required. Lead-containing waste is classified as "Resource, Conservation, and Recovery Act" (RCRA) hazardous, or Federal hazardous, when the representative soluble lead content equals or exceeds the Federal regulatory level of 5 mg/l based on the Toxicity Characteristic Leaching Procedure (TCLP).

The above regulatory criteria are based on chemical concentrations. Wastes may also be classified as hazardous based on other criteria such as ignitability; however, for the purposes of this investigation, toxicity (i.e., lead concentration) is the primary factor considered for waste classification since waste generated during the construction activities would not likely warrant testing for ignitability or other criteria. Waste that is classified as either California-hazardous or RCRA-hazardous requires management as a hazardous waste.

Potential hazards exist to workers who remove or cut through LCP coatings during demolition. Dust containing hazardous concentrations of lead may be generated during scraping or cutting materials coated with lead-containing paint. Torching of these materials may produce lead oxide fumes. Therefore, air monitoring and/or respiratory protection may be required during the demolition of materials coated with LCP. Guidelines regarding regulatory provisions for construction work where workers may be exposed to lead are presented in Title 8, CCR, §1532.1.

2.3 Architectural Drawings and Previous Survey Activities

We reviewed structure as-built plans provided by Caltrans prior to field activities. We did not identify specifications or notes regarding the use of asbestos-containing materials or lead paint in the architectural plans provided. Previous asbestos survey reports were not available for our review.

3.0 SCOPE OF SERVICES

Mr. David Watts, a California-Certified Asbestos Consultant (CAC), certification No. 98-2404 (expiration September 16, 2015), and Certified Lead Paint Inspector/Assessor and Project Monitor with the California Department of Public Health (DPH), certification numbers I-1734 and M-1734 (expiration December 4, 2015), performed the asbestos and LCP survey at the project locations on February 3, 2015.

3.1 Asbestos

Suspect ACM were grouped into homogeneous areas with representative samples randomly collected from each. In addition, each potential ACM was evaluated for friability. A total of twenty-four bulk asbestos samples representing nine suspect materials were collected.

Our procedures for inspection and sampling in accordance with TO-46 are discussed below:

- Collected bulk asbestos samples after first wetting friable materials with a light mist of water. The samples were then cut from the substrate and transferred to labeled containers. Note that when multiple samples were collected, the sampling locations were distributed throughout the homogeneous area (spaces where the material was observed).
- Relinquished bulk asbestos samples under standard chain-of-custody protocol to EMSL Analytical, Inc., a California-licensed and Caltrans-approved subcontractor, for asbestos analysis in accordance with United States Environmental Protection Agency (EPA) Test Method 600/R-93/116 using polarized light microscopy (PLM). EMSL Analytical, Inc. is a laboratory accredited by the National Institute of Standards and Technology National Voluntary Laboratory Accreditation Program (NIST-NVLAP) for bulk asbestos fiber analysis. The laboratory analyses were requested on a turnaround period of ten days.

Sample group identification numbers, material descriptions, approximate quantities, friability assessments, and photo references are summarized on Table 1. Approximate sample locations are presented on Figure 2. Materials represented by the samples collected are shown in the attached photographs.

3.2 Lead Paint

We observed no suspect LCP on structural members of the bridges. Consequently, we collected no paint samples.

4.0 INVESTIGATIVE RESULTS

Chrysotile asbestos at a concentration of 30% was detected in samples representing approximately 10 square feet of nonfriable asbestos sheet packing used as shims on the Sidehill Viaduct (06-0042) barrier rail systems.

No asbestos was detected in samples of the remaining suspect materials collected during our survey. A summary of the analytical laboratory test results for asbestos is presented on Table 1. Reproductions of the laboratory report and chain-of-custody documentation are presented in Appendix A.

5.0 RECOMMENDATIONS

Based on our findings, we recommend the following:

5.1 Asbestos

NESHAP regulations do not require that asbestos-containing sheet packing (a Category I nonfriable/nonhazardous material) identified during our survey be removed prior to demolition/renovation or be treated as hazardous waste. The sheet packing may also be reused or stored. However, activities causing *disturbance* of the sheet packing (i.e., cutting, abrading, sanding, grinding, etc.) would require compliance with the Cal/OSHA asbestos standard (Title 8, CCR §1529).

We also recommend the notification of contractors (that will be conducting demolition, renovation, or related activities) of the presence of asbestos in their work areas (i.e., provide the contractor[s] with a copy of this report and a list of asbestos removed by contractor[s] during subsequent activities). Personnel not trained for asbestos work should be instructed not to disturb asbestos. Written notification to the U.S. EPA Region IX and the California Air Resources Board is required ten working days prior to commencement of *any* demolition activity (whether asbestos is present or not).

5.2 Lead Paint

We recommend that all paints at the project location (graffiti, graffiti abatement, traffic striping, signage, etc.) be treated as lead-containing for purposes of determining the applicability of the Cal/OSHA lead standard during maintenance, renovation, and demolition activities. This recommendation is based on the fact that lead was a common ingredient of paints manufactured before 1978 and is still an ingredient of some paints. In accordance with Title 8, CCR, §1532.1(p), written notification to the nearest Cal/OSHA district office is required at least 24 hours prior to certain lead-related work. Compliance and training requirements regarding construction activities where workers may be exposed to lead are presented in Title 8, CCR, §1532.1, subsections (e) and (l), respectively. Contractors are responsible for segregating and characterizing waste streams prior to disposal.

6.0 REPORT LIMITATIONS

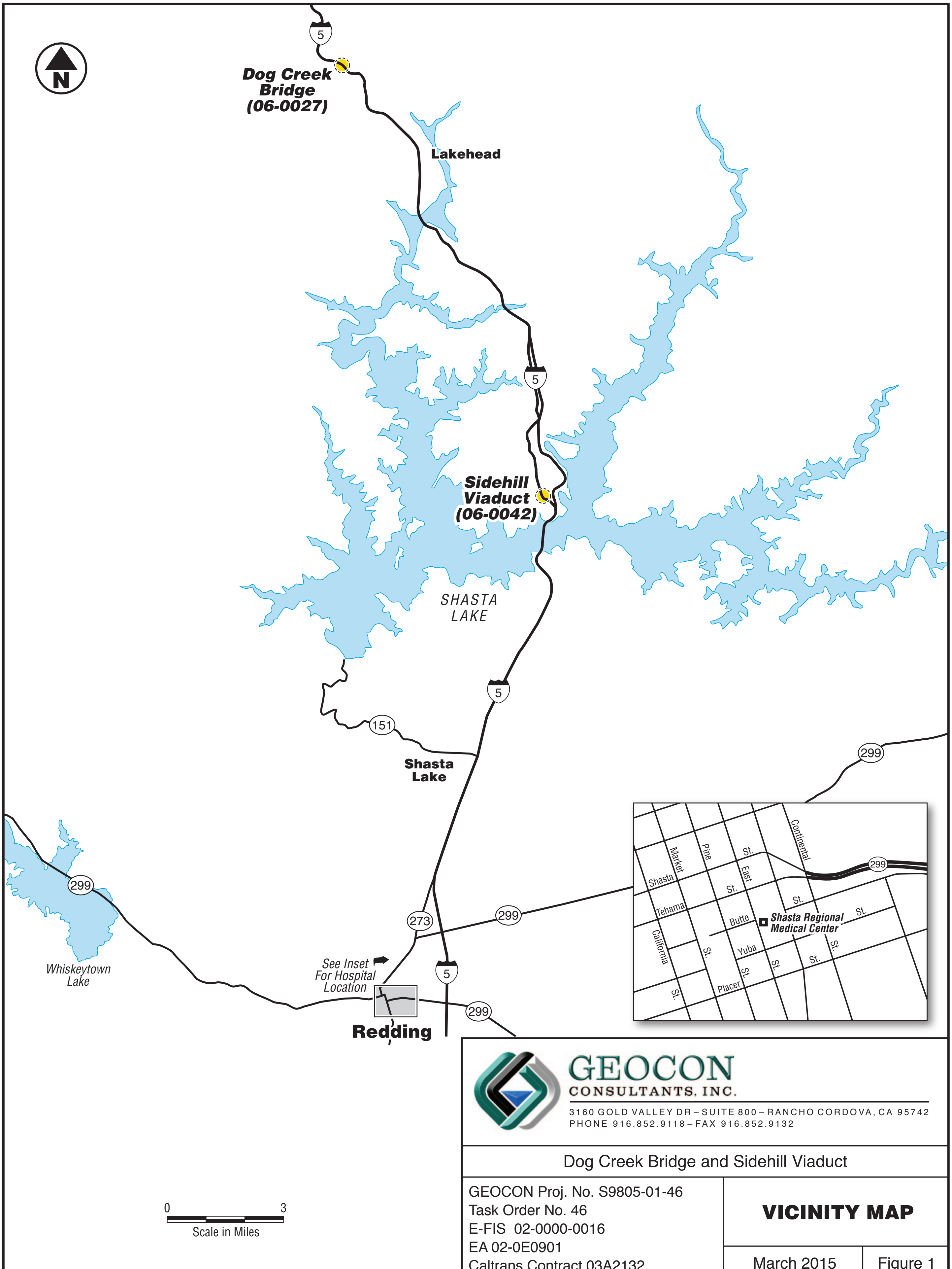
The asbestos and LCP survey was conducted in conformance with generally accepted standards of practice for identifying and evaluating asbestos and LCP in structures. The survey addressed only the structures identified in Section 1.1. Due to the nature of structure surveys, asbestos and LCP use, and laboratory analytical limitations, some ACM or LCP at the project location may not have been identified. Spaces such as cavities, voids, crawlspaces, and pipe chases may have been concealed to our investigator. Previous renovation work may have concealed or covered spaces or materials or may have partially demolished materials and left debris in inaccessible areas. Additionally, renovation activities may have partially replaced ACM with indistinguishable non-ACM. Asbestos and/or LCP may exist in areas of the structures that were not accessible or sampled in conjunction with this TO.

During renovation or demolition operations, suspect materials may be uncovered which are different from those accessible for sampling during this assessment. Personnel in charge of renovation/demolition should be alerted to note materials uncovered during such activities that differ substantially from those included in this or previous assessment reports. If suspect ACM and/or LCP are found, additional sampling and analysis should be performed to determine if the materials contain asbestos or lead.

This report has been prepared exclusively for Caltrans. The information contained herein is only valid as of the date of the report and will require an update to reflect additional information obtained.

This report is not a comprehensive site characterization and should not be construed as such. The findings as presented in this report are predicated on the results of the limited sampling and laboratory testing performed. In addition, the information obtained is not intended to address potential impacts related to sources other than those specified herein. Therefore, the report should be deemed conclusive with respect to only the information obtained. We make no warranty, express or implied, with respect to the content of this report or any subsequent reports, correspondence or consultation. Geocon strived to perform the services summarized herein in accordance with the local standard of care in the geographic region at the time the services were rendered.

The contents of this report reflect the views of the author who is responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.



GEOCON CONSULTANTS, INC.

3160 GOLD VALLEY DR – SUITE 800 – RANCHO CORDOVA, CA 95742
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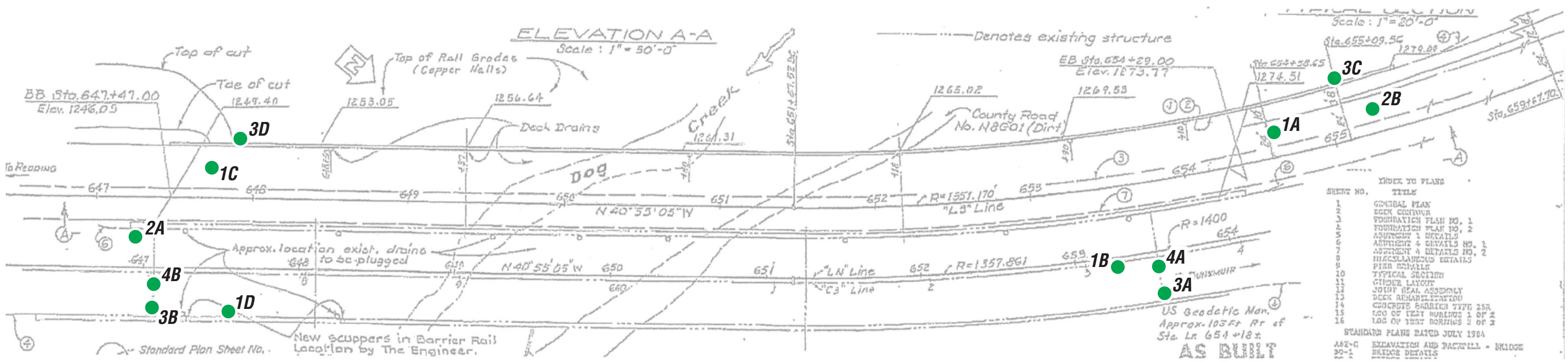
Dog Creek Bridge and Sidehill Viaduct

GEOCON Proj. No. S9805-01-46
Task Order No. 46
E-FIS 02-0000-0016
EA 02-0E0901
Caltrans Contract 03A2132

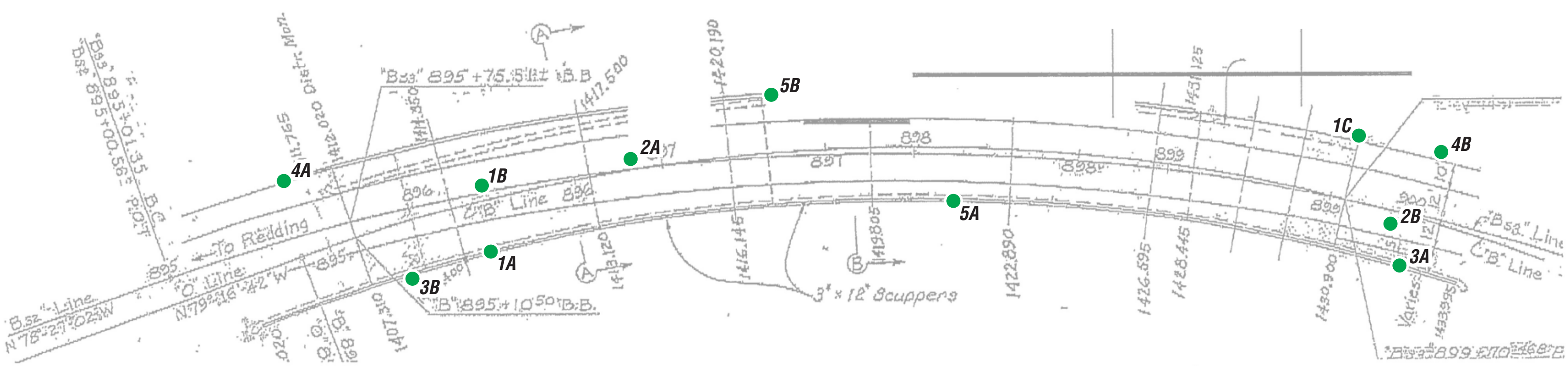
VICINITY MAP

March 2015

Figure 1



DOG CREEK BRIDGE (06-0027)



SIDEHILL VIADUCT (06-0042)

1D

GEOCON
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Dog Creek Bridge and Sidehill Viaduct

GEOCON Proj. No. S9805-01-46
Task Order No. 46
E-FIS 02-0000-0016
EA 02-0E0901
Caltrans Contract 03A2132

SITE PLAN

March 2015

Figure 2

LEGEND:

● Approximate Asbestos Sample Location

NO SCALE

Photo 1 – Dog Creek Bridge (06-0027) at PM 45.66 on Interstate 5 in Shasta County, California

Photo 2 – Dog Creek Bridge (deck)

Photo 3 – Dog Creek Bridge (south abutment)

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PHOTOGRAPHS 1, 2, & 3		
Dog Creek and Sidehill Seismic Retrofits Shasta County, California		
S9805-01-46		March 2015

Photo 4 – Dog Creek Bridge (joint fill material)

Photo 5 – Dog Creek Bridge (north abutment)

Photo 6 – Dog Creek Bridge (north abutment)

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PHOTOGRAPHS 4, 5, & 6		
Dog Creek and Sidehill Seismic Retrofits Shasta County, California		
S9805-01-46		March 2015

Photo 7 – Dog Creek Bridge (span and columns)

Photo 8 – Dog Creek Bridge (north abutment)

Photo 9 – Dog Creek Bridge (span and columns)

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PHOTOGRAPHS 7, 8, & 9		
Dog Creek and Sidehill Seismic Retrofits Shasta County, California		
S9805-01-46		March 2015

Photo 10 – Sidehill Viaduct (06-0042) at PM 29.72 on Interstate 5 in Shasta County, California

Photo 11 – Sidehill Viaduct (south abutment)

Photo 12 – Sidehill Viaduct (span and columns)

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

PHOTOGRAPHS 10, 11, & 12		
Dog Creek and Sidehill Seismic Retrofits Shasta County, California		
S9805-01-46		March 2015

Photo 13 – Sidehill Viaduct (joint fill material)

Photo 14 – Sidehill Viaduct (asbestos sheet packing used as barrier rail shims)

Photo 15 – Sidehill Viaduct (retaining wall)

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

PHOTOGRAPHS 13, 14, & 15		
Dog Creek and Sidehill Seismic Retrofits Shasta County, California		
S9805-01-46		March 2015

TABLE 1
SUMMARY OF ASBESTOS ANALYTICAL RESULTS
DOG CREEK BRIDGE AND SIDEHILL VIADUCT SEISMIC RETROFITS
CALTRANS CONTRACT 03A2132, TASK ORDER NO. 46, E-FIS 02-0000-0016 (EA 02-0E0901)
SHASTA COUNTY, CALIFORNIA

Polarized Light Microscopy (PLM) - EPA Test Method 600/R-93/116

Bridge No.	Sample Group No.	Material Description	Approximate Quantity	Friable	Site Photos	Asbestos Content
06-0027	1	Concrete	NA	NA	1 through 9	ND
	2	Asphalt	NA	NA		ND
	3	Joint fill material	NA	NA		ND
	4	Drainpipe	NA	NA		ND
06-0042	1	Concrete	NA	NA	10 through 15	ND
	2	Asphalt	NA	NA	10	ND
	3	Joint fill material	NA	NA	13	ND
	4	Drainpipe	NA	NA	11	ND
	5	**Sheet packing (shims)**	**10 square feet**	**No**	**14**	**30%**

Notes:

NA = Not applicable (no asbestos detected)

ND = Not detected

APPENDIX A

EMSL Analytical, Inc
464 McCormick Street, San Leandro, CA 94577
Phone/Fax: (510) 895-3675 / (510) 895-3680
http://www.EMSL.com sanleandrolab@emsl.com

EMSL Order:	091501674
CustomerID:	GECN21
CustomerPO:	S9805-01-46
ProjectID:	03A2132

Attn: **Dave Watts**
Geocon Consultants, Inc.
6671 Brisa Street
Livermore, CA 94550

Phone:	(925) 371-5900
Fax:	(925) 371-5915
Received:	02/04/15 9:00 AM
Analysis Date:	2/17/2015
Collected:	2/3/2015

Project: **S9805-01-46/ SHASTA CO. BRIDGES**

Test Report: Asbestos Analysis of Bulk Materials via EPA 600/R-93/116 Method using Polarized Light Microscopy

			Non-Asbestos				Asbestos
Sample	**Description**	**Appearance**	**%**	**Fibrous**	**%**	**Non-Fibrous**	**% Type**
0027-1A Concrete *091501674-0001*		Tan Non-Fibrous Homogeneous			10% 90%	Quartz Non-fibrous (other)	**None Detected**
0027-1B Concrete *091501674-0002*		Tan Non-Fibrous Homogeneous			10% 90%	Quartz Non-fibrous (other)	**None Detected**
0027-1C Concrete *091501674-0003*		Tan Non-Fibrous Homogeneous			10% 90%	Quartz Non-fibrous (other)	**None Detected**
0027-1D-Concrete *091501674-0004*		Gray/Tan Non-Fibrous Homogeneous			20% 80%	Quartz Non-fibrous (other)	**None Detected**
0027-2A Asphalt *091501674-0005*		Black Non-Fibrous Homogeneous			40% 60%	Matrix Non-fibrous (other)	**None Detected**
0027-2B Asphalt *091501674-0006*		Black Non-Fibrous Homogeneous			10% 40% 50%	Quartz Matrix Non-fibrous (other)	**None Detected**
0027-3A Joint Fill Material *091501674-0007*		Black Fibrous Homogeneous	20%	Glass	80%	Non-fibrous (other)	**None Detected**
0027-3B Joint Fill Material *091501674-0008*		Black Fibrous Homogeneous	5%	Glass	95%	Non-fibrous (other)	**None Detected**

Analyst(s)

Laura Torres (7)
Matthew Batongbacal (16)

Chris Dojlidko

Chris Dojlidko, Laboratory Manager
or other approved signatory

Initial report from 02/17/2015 12:17:50

EMSL Analytical, Inc
464 McCormick Street, San Leandro, CA 94577
Phone/Fax: (510) 895-3675 / (510) 895-3680
http://www.EMSL.com sanleandrolab@emsl.com

EMSL Order:	091501674
CustomerID:	GECN21
CustomerPO:	S9805-01-46
ProjectID:	03A2132

Attn: **Dave Watts**
Geocon Consultants, Inc.
6671 Brisa Street
Livermore, CA 94550

Phone:	(925) 371-5900
Fax:	(925) 371-5915
Received:	02/04/15 9:00 AM
Analysis Date:	2/17/2015
Collected:	2/3/2015

Project: **S9805-01-46/ SHASTA CO. BRIDGES**

Test Report: Asbestos Analysis of Bulk Materials via EPA 600/R-93/116 Method using Polarized Light Microscopy

			Non-Asbestos		Asbestos
Sample	**Description**	**Appearance**	**% Fibrous**	**% Non-Fibrous**	**% Type**
0027-3C-Joint Fill Material *091501674-0009*		Brown Fibrous Homogeneous	70% Cellulose	30% Non-fibrous (other)	**None Detected**
0027-3D-Joint Fill Material *091501674-0010*		Brown Fibrous Homogeneous	70% Cellulose	30% Non-fibrous (other)	**None Detected**
0027-4A-Drain Pipe *091501674-0011*		Black Fibrous Homogeneous	20% Cellulose	60% Matrix 20% Non-fibrous (other)	**None Detected**
0027-4B-Drain Pipe *091501674-0012*		Black Fibrous Homogeneous	20% Cellulose	60% Matrix 20% Non-fibrous (other)	**None Detected**
0042-1A-Concrete *091501674-0013*		Tan Non-Fibrous Homogeneous		10% Quartz 40% Ca Carbonate 50% Non-fibrous (other)	**None Detected**
0042-1B-Concrete *091501674-0014*		Gray/Tan Non-Fibrous Homogeneous		10% Quartz 40% Ca Carbonate 50% Non-fibrous (other)	**None Detected**
0042-1C-Concrete *091501674-0015*		Gray/Tan Non-Fibrous Homogeneous		10% Quartz 40% Ca Carbonate 50% Non-fibrous (other)	**None Detected**

Analyst(s)

Laura Torres (7)
Matthew Batongbacal (16)

Chris Dojlidko, Laboratory Manager
or other approved signatory

Initial report from 02/17/2015 12:17:50

EMSL Analytical, Inc
464 McCormick Street, San Leandro, CA 94577
Phone/Fax: (510) 895-3675 / (510) 895-3680
http://www.EMSL.com sanleandrolab@emsl.com

EMSL Order:	091501674
CustomerID:	GECN21
CustomerPO:	S9805-01-46
ProjectID:	03A2132

Attn: **Dave Watts**
Geocon Consultants, Inc.
6671 Brisa Street
Livermore, CA 94550

Phone:	(925) 371-5900
Fax:	(925) 371-5915
Received:	02/04/15 9:00 AM
Analysis Date:	2/17/2015
Collected:	2/3/2015

Project: **S9805-01-46/ SHASTA CO. BRIDGES**

Test Report: Asbestos Analysis of Bulk Materials via EPA 600/R-93/116 Method using Polarized Light Microscopy

Sample	Description	Appearance	Non-Asbestos % Fibrous	Non-Asbestos % Non-Fibrous	Asbestos % Type
0042-1D-Concrete *091501674-0016*		Gray/Tan Non-Fibrous Homogeneous		10% Quartz 40% Ca Carbonate 50% Non-fibrous (other)	**None Detected**
0042-2A-Asphalt *091501674-0017*		Black Non-Fibrous Homogeneous		20% Quartz 40% Matrix 40% Non-fibrous (other)	**None Detected**
0042-2B-Asphalt *091501674-0018*		Black Non-Fibrous Homogeneous		20% Quartz 40% Matrix 40% Non-fibrous (other)	**None Detected**
0042-3A-Joint Fill Material *091501674-0019*		Black Non-Fibrous Homogeneous	5% Glass	60% Matrix 35% Non-fibrous (other)	**None Detected**
0042-3B-Joint Fill Material *091501674-0020*		Black Non-Fibrous Homogeneous	5% Glass	60% Matrix 35% Non-fibrous (other)	**None Detected**
0042-4A-Drain Pipe *091501674-0021*		Red Non-Fibrous Homogeneous		30% Quartz 70% Non-fibrous (other)	**None Detected**
0042-4B-Drain Pipe *091501674-0022*		Tan Non-Fibrous Homogeneous		30% Quartz 70% Non-fibrous (other)	**None Detected**

Analyst(s)
Laura Torres (7)
Matthew Batongbacal (16)

Chris Dojlidko

Chris Dojlidko, Laboratory Manager
or other approved signatory

Initial report from 02/17/2015 12:17:50

EMSL Analytical, Inc
464 McCormick Street, San Leandro, CA 94577
Phone/Fax: (510) 895-3675 / (510) 895-3680
http://www.EMSL.com sanleandrolab@emsl.com

EMSL Order:	091501674
CustomerID:	GECN21
CustomerPO:	S9805-01-46
ProjectID:	03A2132

Attn: **Dave Watts**
Geocon Consultants, Inc.
6671 Brisa Street
Livermore, CA 94550

Phone:	(925) 371-5900
Fax:	(925) 371-5915
Received:	02/04/15 9:00 AM
Analysis Date:	2/17/2015
Collected:	2/3/2015

Project: **S9805-01-46/ SHASTA CO. BRIDGES**

Test Report: Asbestos Analysis of Bulk Materials via EPA 600/R-93/116 Method using Polarized Light Microscopy

			Non-Asbestos				Asbestos	
Sample	**Description**	**Appearance**	**%**	**Fibrous**	**%**	**Non-Fibrous**	**%**	**Type**
0042-5A-Sheet Packing *091501674-0023*		Black Fibrous Homogeneous			20% 50%	Ca Carbonate Non-fibrous (other)	**30%**	**Chrysotile**
0042-5B *091501674-0024*							**Stop Positive (Not Analyzed)**	

Analyst(s)

Laura Torres (7)
Matthew Batongbacal (16)

Chris Dojlidko

Chris Dojlidko, Laboratory Manager
or other approved signatory

Initial report from 02/17/2015 12:17:50

EMSL ANALYTICAL, INC.
LABORATORY • PRODUCTS • TRAINING

Asbestos Chain of Custody

EMSL Order Number *(Lab Use Only)*

#091501674

CT # 03A2132

EMSL ANALYTICAL, INC.
2235 POLVOROSA DR., STE. 230
SAN LEANDRO, CA 94577
PHONE: (510) 895-3675
FAX: (510) 895-3680

Company: GEOCON	EMSL-Bill to: ☒ Same ☐ Different (If Bill to is Different note instructions in Comments**)
Street: 6671 BRISA ST	*Third Party Billing requires written authorization from third party*
City: LIVERMORE — State/Province: CA	Zip/Postal Code: 94550 — Country: USA
Report To (Name): D. WATTS	Fax #: 925-371-5915
Telephone #: 925-371-5900	Email Address: WATTS@GEOCONINC.COM
Project Name/Number: SHASTA CO. BRIDGES 59805-01-46	
Please Provide Results: ☐ Fax ☒ Email — Purchase Order:	U.S. State Samples Taken: CA

Turnaround Time (TAT) Options* – Please Check

☐ 3 Hour ☐ 6 Hour ☐ 24 Hour ☐ 48 Hour ☐ 72 Hour ☐ 96 Hour ☐ 1 Week ☒ 2 Week

*For TEM Air 3 hours/6 hours, please call ahead to schedule.*There is a premium charge for 3 Hour TEM AHERA or EPA Level II TAT. You will be asked to sign an authorization form for this service. Analysis completed in accordance with EMSL's Terms and Conditions located in the Analytical Price Guide.*

PCM - Air
- ☐ NIOSH 7400
- ☐ w/ OSHA 8hr. TWA

PLM - Bulk (reporting limit)
- ☒ PLM EPA 600/R-93/116 (<1%)
- ☐ PLM EPA NOB (<1%)

Point Count
- ☐ 400 (<0.25%) ☐ 1000 (<0.1%)

Point Count w/Gravimetric
- ☐ 400 (<0.25%) ☐ 1000 (<0.1%)
- ☐ NYS 198.1 (friable in NY)
- ☐ NYS 198.6 NOB (non-friable-NY)
- ☐ NIOSH 9002 (<1%)

TEM – Air ☐ 4-4.5hr TAT (AHERA only)
- ☐ AHERA 40 CFR, Part 763
- ☐ NIOSH 7402
- ☐ EPA Level II
- ☐ ISO 10312

TEM - Bulk
- ☐ TEM EPA NOB
- ☐ NYS NOB 198.4 (non-friable-NY)
- ☐ Chatfield SOP
- ☐ TEM Mass Analysis-EPA 600 sec. 2.5

TEM – Water: EPA 100.2
- Fibers >10µm ☐ Waste ☐ Drinking
- All Fiber Sizes ☐ Waste ☐ Drinking

TEM- Dust
- ☐ Microvac - ASTM D 5755
- ☐ Wipe - ASTM D6480
- ☐ Carpet Sonication (EPA 600/J-93/167)

Soil/Rock/Vermiculite
- ☐ PLM CARB 435 - A (0.25% sensitivity)
- ☐ PLM CARB 435 - B (0.1% sensitivity)
- ☐ TEM CARB 435 - B (0.1% sensitivity)
- ☐ TEM CARB 435 - C (0.01% sensitivity)
- ☐ EPA Protocol (Semi-Quantitative)
- ☐ EPA Protocol (Quantitative)

Other:
- ☐

☒ **Check For Positive Stop – Clearly Identify Homogenous Group** >1%

Samplers Name: D. WATTS — Samplers Signature: Watt

Sample #	Sample Description	Volume/Area (Air) HA # (Bulk)	Date/Time Sampled
0027-1A-D	CONCRETE/(AB/COL/DK/BAR)	NA	3 FEB 2015
-2A/B	ASPHALT		
-3A-D	JOINT FILL MAT'L		
-4A/B	DRAIN PIPE		
0042-1A-D	CONCRETE (AB/DK/BAR/RW)		
-2A/B	ASPHALT		
-3A/B	JOINT FILL MAT'L		
-4A/B	DRAIN PIPE		
-5A/B	SHEET PACKING		

Client Sample # (s): — Total # of Samples: 24

Relinquished (Client): Watt — Date: 2/3/15 — Time: 1745

Received (Lab): FA FEDEX — Date: 2/4/15 2/3/15 — Time: 1745 FedEx

Comments/Special Instructions:

MATERIALS INFORMATION

Division of Occupational Safety and Health, Mining and Tunneling Unit,

Underground Classification, Dated March 2, 2015

STATE OF CALIFORNIA EDMUND G. BROWN, JR., Governor

DEPARTMENT OF INDUSTRIAL RELATIONS
DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT
2424 Arden Way, Suite 125
Sacramento, California 95825
doshM&Tsac@dir.ca.gov

Telephone (916) 574-2540
FAX (916) 574-2542

March 2, 2015

State of California Dept. of Transportation
North Region Design R1
District 02, Caltrans
1031 Butte Street
Redding, CA 96001

Attention: Travis Gurney, P.E.

Subject: Project: 15065 – Southbound Route 5 Shasta Viaduct
Classification: Potentially Gassy With Special Conditions
Number Attached: 5 (A-E)

The information provided to this office relative to the above project has been reviewed. On the basis of this analysis, Underground Classifications of "Potentially Gassy With Special Conditions" have been assigned to the tunnels identified on your submittal. Please retain the original Classifications for your records and deliver a true and correct copy of each Classification to the tunnel contractor for posting at the job site.

When the contractor who will be performing the work is selected, please advise them to notify this office to schedule the mandated Pre-Job Conference with the Division prior to commencing any activity associated with boring of the tunnels. A Pre-Job Request Form is enclosed.

Should you have another bore under construction that is not required to have an Underground Classification (i.e.: less than 30 inches in diameter), please contact the Mining and Tunneling Unit prior to any employee entry of such a space.

If you have any questions on this subject, please contact this office at your earliest convenience.

Sincerely,

Douglas Patterson
Senior Engineer

enc: Classifications
Pre-Job Request Form

cc: travis.gurney@dot.ca.gov
ballard@dir.ca.gov

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

15065A089CT
CORRECTION 1

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

of **R1D2 CALTRANS, 1031 BUTTE STREET; REDDING, CA 96001**

at **SOUTHBOUND ROUTE 5 SHASTA VIADUCT**

has been classified as ***** POTENTIALLY GASSY WITH SPECIAL CONDITIONS *****

as required by the California Labor Code § 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

*****SPECIAL CONDITIONS*****

1. **A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.**
2. **Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.**
3. **The Division shall be notified immediately if any Flammable Gas or Petroleum Vapor exceeds 5% of the Lower Explosive Limit.**
4. **All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.**

The eight 72-inch-diameter 27-to-42-foot-deep vertically bored shafts along southbound Interstate 5 located 2,100 feet north of the Turntable Bay Road overcrossing in Lakehead, Shasta County

This classification shall be conspicuously posted at the place of employment.

Douglas Patterson, Senior Engineer

March 2, 2015

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

15065B089CT
CORRECTION 1 **STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION**

of **R1D2 CALTRANS, 1031 BUTTE STREET; REDDING, CA 96001**

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*****SPECIAL CONDITIONS*****

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3. **The Division shall be notified immediately if any Flammable Gas or Petroleum Vapor exceeds 5% of the Lower Explosive Limit.**
4. **All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.**

The eight 72-inch-diameter 27.6-to-37.6-foot-deep vertically bored shafts along southbound Interstate 5 located 2,500 feet north of the Turntable Bay Road overcrossing in Lakehead, Shasta County

This classification shall be conspicuously posted at the place of employment.

Douglas Patterson, Senior Engineer

March 2, 2015

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

15065C089CT
CORRECTION 1

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

of R1D2 CALTRANS, 1031 BUTTE STREET; REDDING, CA 96001

at SOUTHBOUND ROUTE 5 SHASTA VIADUCT

has been classified as *** POTENTIALLY GASSY WITH SPECIAL CONDITIONS ***

as required by the California Labor Code § 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. **A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.**
2. **Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.**
3. **The Division shall be notified immediately if any Flammable Gas or Petroleum Vapor exceeds 5% of the Lower Explosive Limit.**
4. **All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.**

The two 96-inch-diameter 66 & 67-foot-deep vertically bored shafts along southbound Interstate 5 located 2,550 feet north of the Turntable Bay Road overcrossing in Lakehead, Shasta County

This classification shall be conspicuously posted at the place of employment.

Douglas Patterson, Senior Engineer

March 2, 2015

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

15065D089CT
CORRECTION 1

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

of R1D2 CALTRANS, 1031 BUTTE STREET; REDDING, CA 96001

at SOUTHBOUND ROUTE 5 SHASTA VIADUCT

has been classified as *** POTENTIALLY GASSY WITH SPECIAL CONDITIONS ***

as required by the California Labor Code § 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. **A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.**
2. **Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.**
3. **The Division shall be notified immediately if any Flammable Gas or Petroleum Vapor exceeds 5% of the Lower Explosive Limit.**
4. **All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.**

The two 96-inch-diameter 69-foot-deep vertically bored shafts along southbound Interstate 5 located 2,700 feet north of the Turntable Bay Road overcrossing in Lakehead, Shasta County

This classification shall be conspicuously posted at the place of employment.

Douglas Patterson, Senior Engineer

March 2, 2015

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

15065E089CT
CORRECTION 1

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

of R1D2 CALTRANS, 1031 BUTTE STREET; REDDING, CA 96001

at SOUTHBOUND ROUTE 5 SHASTA VIADUCT

has been classified as *** POTENTIALLY GASSY WITH SPECIAL CONDITIONS ***

as required by the California Labor Code § 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. **A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.**
2. **Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.**
3. **The Division shall be notified immediately if any Flammable Gas or Petroleum Vapor exceeds 5% of the Lower Explosive Limit.**
4. **All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.**

The 36-inch-diameter 22-foot-deep vertically bored shaft along southbound Interstate 5 located 3,730 feet north of the Turntable Bay Road overcrossing in Lakehead, Shasta County

This classification shall be conspicuously posted at the place of employment.

Douglas Patterson, Senior Engineer

March 2, 2015

State of California
Department of Industrial Relations
Division of Occupational Safety & Health

REQUEST FOR PRE-JOB (TUNNEL)

ATTACH COPY OF CLASSIFICATION AND DIESEL PERMIT

Company Name:____

Phone________________________ FAX:________________________

DATE FAXED: ____

PLEASE NOTE: THE BORING CONTRACTOR SHOULD SCHEDULE THE PREJOB AS FAR IN ADVANCE AS POSSIBLE - AT LEAST 3-4 DAYS IN ADVANCE. THE DIVISION REQUIRES THE JOB TO BE SET UP WHEN THE FIELD ENGINEER ARRIVES FOR THE PREJOB. THIS MEANS THAT THE BORE PIT HAS BEEN DUG AND PROPERLY GUARDED, THE CRANE IS IN PLACE AND READY TO LIFT, THE BORING MACHINE IS IN THE PIT AND READY TO GO, AND THE CREW IS READY TO BEGIN BORING THE TUNNEL. IF THERE IS A DELAY IN SETTING UP THE JOB, THE BORING CONTRACTOR SHOULD CONTACT THE DIVISION IMMEDIATELY.

PRE-JOB REQUEST DATE & TIME: ____________________________

ON-SITE SUPERVISOR & CELL NO.: ____________________________

CLASSIFICATION #: ________________ **DIESEL PERMIT #:** ________________

BORE DIAMETER AND LENGTH: ________________ (Diameter) ________________ (Length)

IS BORE ENTRY ANTICIPATED? **YES** **NO**
(Circle One)

You MUST contact the Division if entry is planned, REGARDLESS of the bore diameter.

MANNER OF EXCAVATION: ________________________________

____

JOB-SITE LOCATION AND DIRECTIONS: ____________________________

____

____

GENERAL CONTRACTOR: ________________________________

SUBMITTED BY: ______________________________________

REVIEWED BY: ______________________ **DATE:** ________________

☐ *Mining & Tunneling Unit, District 1*
2424 Arden Way, Suite 125
Sacramento, California 95825-2400
(916) 574-2540; ***FAX: (916) 574-2542***

☐ *Mining & Tunneling Unit, District 2*
6150 Van Nuys Blvd., Suite 310
Van Nuys, California 91401-3333
(818) 901-5420; ***FAX: (818) 901-5579***

☐ *Mining & Tunneling Unit, District 3*
464 West Fourth Street, Suite 354
San Bernardino, California 92401-1442
(909) 383-6782; ***FAX: (909) 388-7132***

MATERIALS INFORMATION

Optional Disposal Site Information

INFORMATIONAL HANDOUT

FOR A CONSTRUCTION CONTRACT
IN SHASTA COUNTY

Interstate 5, Post Mile R30.0 and 45.5
Bridge Replacement of Sidehill Viaduct and
Seismic Retrofit of Dog Creek Bridge

IN SHASTA COUNTY ABOUT 8 MILES NORTH OF SHASTA LAKE CITY AND NEAR LAKEHEAD FROM TURNTABLE BAY OVERCROSSING TO 0.1 MILE NORTH OF O'BRIEN SAFETY ROADSIDE REST AREA AND FROM 0.4 MILE SOUTH TO 0.3 MILE NORTH OF DOG CREEK BRIDGE

OPTIONAL DISPOSAL SITE

SWA @ Mountain Gate Quarry is a privately owned commercial quarry, located off of I-5 (Wonderland/Fawndale Rd Exit #689) on Radcliff Rd, approximately 3.5 miles south of the project location.

Note: The records from which this compilation was made may be inspected in the District Office at 1657 Riverside Drive Redding, CA 96001 or contact the Disposal Site Coordinator Russ Irvin: (530) 225-2084, e-mail: Russ_Irvin@dot.ca.gov

Facts stated herein are as known to the State of California, Caltrans, and are to be verified by the Contractor prior to bid in accordance with Section 2 “Bidding” of the 2010 Standard Specifications.

Table of Contents

General Information

This commercial site is provided by Caltrans, at the option of the contractor for the disposal of excavated materials generated from the Sidehill Viaduct and Seismic Retrofit of Dog Creek Bridge Improvement Project, in compliance to the following provisions.

SWA @ Mountain Gate Quarry has agreed to accept material from the project; however, no formal arrangement has been made for the disposal of material at this private site. The contractor is responsible for all liability, agreements, costs and fees associated with using this or any other site. The contractor shall make arrangements with the Manager of the Mountain Gate Quarry prior to use.

Contact information:
The commercial quarry is owned and operated by SWA @ Mountain Gate Quarry. As of April 3, 2015 this site was listed on the SMARA 3098 list, as an active SMARA approved site, indicating owner is compliant with all regulations and laws.

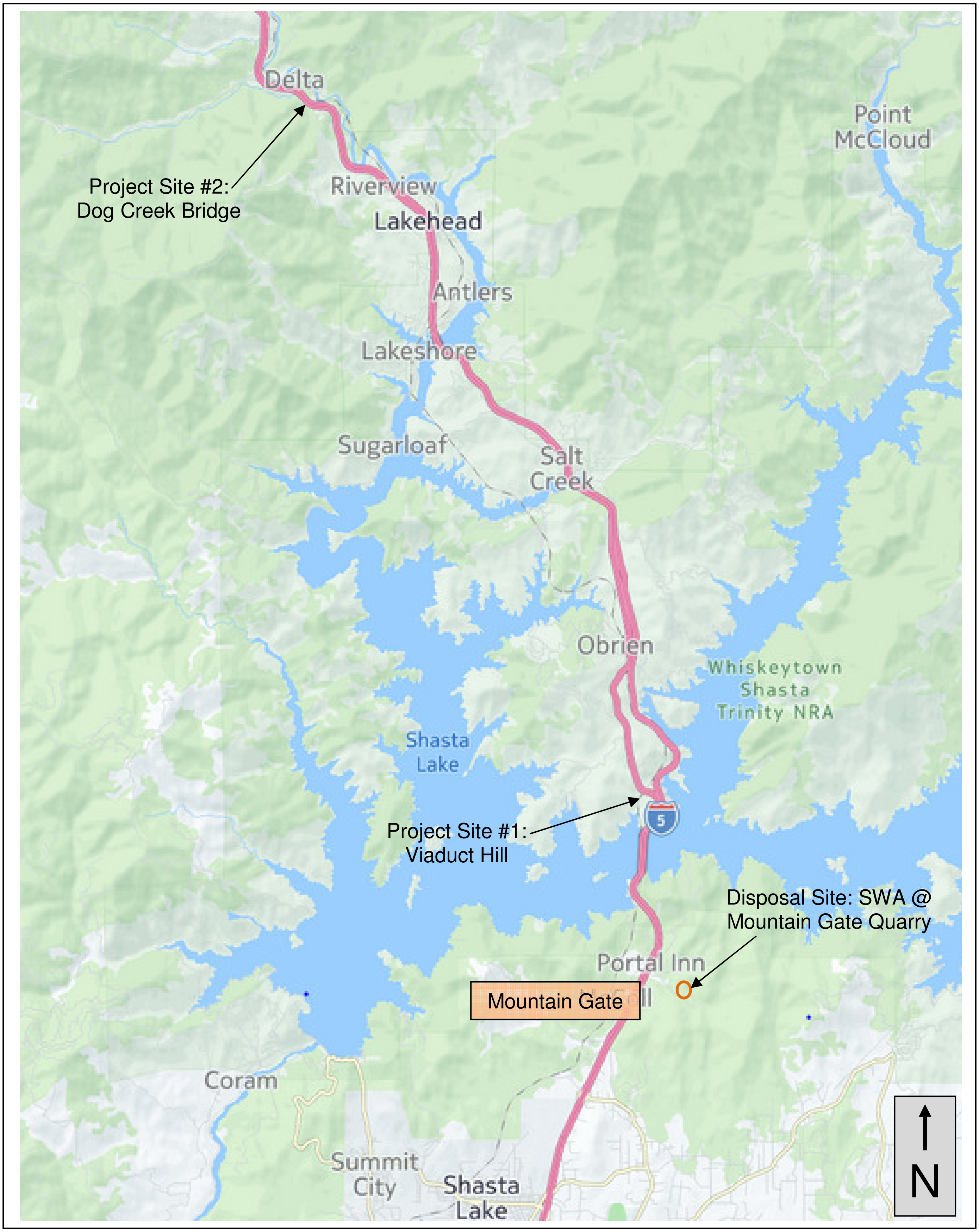
Phone (530) 223-6605; ask for the Manager Corkey Harmon.

Quarry Address:
SWA @ Mountain Gate Quarry
20285 Radcliff Rd.
Redding, CA 96003

Provisions that apply:

- The financial charge to dispose of excess material from this project shall be negotiated between the manager of the quarry and the contractor.
- Delivery of material shall be coordinated with the quarry manager.
- Disposal at this site is intended only for concrete chunks, asphalt chunks or grindings, treated cement base, or earthen material with no steel or Naturally Occurring Asbestos. Estimated quantity is 25,000 CY.
- The contractor bears all liability for damage to haul vehicles and any facility or equipment damaged by the contractor's use of the site. The State assumes no liability for damage to contractor's equipment.
- Disposal or reuse of savaged materials will be in accordance with Section 14 and Section 15 of the 2010 Standard Specifications.
- Existing facilities at the disposal site shall be protected from damage by the Contractor in accordance with Section 5-1.36 "Property and Facility Preservation", of the 2010 Standard Specifications.
- The contractor is encouraged to recycle loose steel at either the local landfill or commercial recyclers. In any case reinforcing steel becomes the property of the contractor.

Location Map: Project and Disposal Sites



Aerial Map: Disposal Site - SWA Quarry at Mountain Gate

MATERIALS INFORMATION

Water Source Information

NONPOTABLE WATER SOURCE

Stillwater Wastewater Treatment Plant

6475 Airport Road, Anderson, California 96007

Supervisor - Dave Johnston - (530) 378-6702