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FOUNDATION INVESTIGATION
Green Valley Road at Tennessee Creek Bridge
El Dorado County, California

El Dorado County
Lead Agency

TRC
Design Engineer

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Introduction

A limited study of foundation conditions has been completed at the above site in accordance with the agreement between TRC and Taber Consultants. The purpose of this study is to evaluate geotechnical conditions and criteria for use in design and construction of a new bridge. This report discusses foundation materials and conditions and presents conclusions and recommendations for the subject project. This report supersedes our "Foundation Investigation – DRAFT" report dated April 10, 2007. Limitations of this work are discussed in the attached "General Conditions".

Site Description

The site is located approximately 4 miles north of Shingle Springs and US 50 on Green Valley Road. At the site, Green Valley Road runs southwest-northeast where it crosses Tennessee Creek. The existing bridge is a narrow two-lane crossing that cannot accommodate the projected increased traffic in the area.

The rural two-lane Green Valley Road crosses Tennessee Creek approximately 500-ft north of North Shingle Road at an approximate elevation of 1100 ft. Green Valley Road is oriented in a generally north-south direction at the bridge location where the road narrows to two 9.5±ft lanes (19±ft total between barriers) with no shoulder. From the south, the roadway descends down towards the creek around a blind curve and then ascends up to a hilltop with very limited sight distance near Oakvale Drive. South of the bridge between North Shingle Road and Peaceful Garden Way, Green Valley Road is in cut and transitions to fill approximately 50-ft east of Peaceful Garden Way. East of the bridge location, between Tennessee Creek and Oakvale Drive, the road continues in fill to within 50-ft of Oakvale Drive and on the north side of the road a cut slope is exposed.

Traffic is heavy with a posted 50 MPH speed limit. Large passenger trucks and commercial vehicles can occupy almost the entire width of the road when crossing the bridge; when larger vehicles are occupying the bridge it effectively reduces the bridge to one lane and dramatically slows oncoming traffic.

Tennessee Creek is aligned from southeast to the northwest with seasonal flow to the northwest. The channel bottom was observed to be typically composed of cobble sized material with no observed clasts larger than 12-inches in diameter.

The surrounding area is typified by oak and grass covered rolling hills. Slopes are generally gentle with outcrops of rock restricted to road cuts. Road cuts appear to be near vertical to 1¼h:1v. Rock cuts appear to have performed adequately at these slopes. The slopes descending to the flood plain of Tennessee Creek is approximately 5h:1V or flatter. Private residences are located on either side of the roadway/bridge and are known to have septic systems located near the county right-of-way.

Project Description

The proposed bridge is shown on a 95% submittal "Green Valley Road at Tennessee Creek Bridge Reconstruction Project – General Plan" drawing dated August 29, 2008, that was prepared by TRC for El Dorado County. The proposed bridge centerline is located near the east edge of the existing bridge. The proposed bridge will be constructed in stages to maintain traffic flow during construction; the existing bridge will be removed during Stage 2 of construction.

As shown on the plan sheet, approximate deck elevation is 1101±ft (4±ft higher than existing grade) and high water elevation is 1094±ft. The proposed bridge (single span) is 64±ft long and 55±ft wide consisting of two 12-ft wide travel lanes, 8-ft wide shoulders and 12-ft wide center median. The structure consists of a cast-in-place pre-stressed concrete slab deck with reinforced concrete seat-type abutments. Bottom of abutment footings are at elevation 1090±ft, approximately 4-ft below high water level for

Tennessee Creek. The structure is shown on the general plan to be supported on 24-inch diameter cast-in-drilled hole (CIDH) piles.

Approach fill embankments are shown on either side of the bridge; embankments are 7-10-ft in height with 2:1 slopes extending away from roadway edge.

It is assumed that the existing embankments will be left in-place after removal of the existing bridge and be re-used for the southbound lane.

The "General Plan" drawing shows the fill/cut slopes along the creek at this location to slope at about 1.5h:1v from above high water level down to the creek channel at about elevation 1086. Modifications to the channel appear to include minor cut slopes at Abutment-1 and fill slopes at Abutment-2; rock slope protection is indicated on the channel slopes. No restrictions have been indicated for typical bridge construction within the channel or for channel modifications.

Exploration and Testing

Office study included review of published geologic mapping, topographic data, site topography and planning study documents provided by TRC. Preliminary field work included site geologic reconnaissance and three double-ended seismic refraction profiles.

Information on the nature and distribution of subsurface materials and conditions at the bridge site for this study was obtained by this office from two drilled, sampled and logged test borings to a maximum depth of about 40-ft (to lowest elev. 1057±). Three short sampled borings were made along the existing roadway in evaluation of subgrade materials. Drilling methods included 4-inch diameter continuous flight auger and 3.8±inch diameter diamond core rotary drilling.

Soil samples were recovered from the borings by means of a 2.0-inch O.D. "standard penetration" sampler advanced with standard 350 ft-lb striking force (per ASTM D1586) using an automatic-trip hammer. Sampler penetration resistance was recorded and can be correlated to strength and bearing characteristics of the soils.

The borings were logged and earth materials field-classified by an engineer/engineering geologist as to consistency, color, texture and gradation on the bases of penetration resistance, examination of samples and observation of drill cuttings. Groundwater observations were made in the borings during drilling operations. The test borings were backfilled with neat cement grout at completion of the field study.

Selected portions of recovered samples were retained in moisture-proof containers for laboratory testing and reference. Moisture content-dry density and unconfined compressive strength tests were performed on selected samples in the laboratory to supplement field evaluation of earth material parameters. Corrosivity screening tests (minimum resistivity, pH, chloride, and sulfate) were also performed on selected samples. Asbestos testing (ARB 435) was performed on both rock cores and soil to evaluate the potential presence of naturally occurring asbestos (NOA). Three R-value tests were performed on bulk samples of near-surface soil to help evaluate soil subgrade characteristics.

Percent recovery and Rock Quality Designation (RQD) was recorded for recovered cores where diamond coring was used to advance the borings. Cores were photographed and stored in core boxes for reference.

Seismic profiles were completed using a Seistronix RS-100 Radioseis Wireless Seismic system with a 24-bit high resolution, single channel refraction seismograph. The energy source was by means of hand-actuated sledge-hammer blows.

Boring locations are referenced to project stationing and existing site features; elevations are referenced to the existing bridge with elevation datum provided by TRC. Locations, details of borings and results of tests are shown on the accompanying "Log of Test Borings" drawings, figures and appendices. Martin W. McIlroy, C.E.G., and Evan Hopson completed seismic profiles and were the field geologists for this study.

Geologic Setting

Published mapping (CDMG, Geologic Map of the Sacramento Quadrangle, Map 1A, 1982) indicates the site is located within Melange terrain and is underlain by Mesozoic ultramafic intrusive rocks. These ultramafic rocks are described as "Peridotite, pyroxenite, hornblendite that grades into gabbroic rock and is locally serpentinized." Pre-Cretaceous undifferentiated metamorphic (limestone or dolomite) rocks are mapped on either side of the mapped ultramafic rocks at the bridge site.

The Bear Mountain Fault Zone is shown (CDMG, Geologic Map of the Sacramento Quadrangle, Map 1A, 1982) to parallel North Shingle Road and is located west of the roadway alignment. Later mapping in 2000 (Reference 13) shows the Bear Mountain Fault trending in a similar manner to the 1982 mapping but located closer to Tennessee Creek and on the west side of North Shingle Road. Due to line weight of mapped fault and unfortunate placement of text on this map, it is difficult to discern whether the fault is mapped across the project site or to the west of the project site. An El Dorado County GIS Map (Reference 2), partially derived from the 2000 mapping, shows the Bear Mountain Fault parallel to and east of North Shingle Road and at the bridge site essentially crossing the project location. Caltrans Seismic Hazard Mapping shows the fault located less than 1-mile to the west of the existing bridge location but not crossing the bridge site.

Rock was observed in outcrops both south and north of the existing bridge location. Observed outcrops appeared consistent with descriptions in published mapping.

Refraction Seismic Profiles

Exploration included three double-ended seismic refraction profiles. Two were completed in the vicinity of Abutment-1 and one was completed in the vicinity of Abutment-2. Refraction seismic profiles are included as Figure-8.

The profiles exhibit seismic velocities ranging from 1000 to 6500 feet/sec (fps). Interpreted seismic velocities for each profile indicate that surface materials consist primarily of unsaturated and saturated, variably consolidated sediments.

Depth transitions to (high) seismic velocities characteristic of the typical range for rock are identified line S3 and line S5 and have interpreted velocities of 5800 to 6500 fps. At these locations, this transition to higher velocity materials is interpreted to be approximately 15-17±ft below ground surface (approximately elevation 1080±).

Lower seismic velocities in the range of 4000 to 5000 fps might be interpreted as a deeply weathered to decomposed rock surface or possibly saturated/unsaturated variably consolidated sediments.

Based on the interpreted seismic profiles and bridge plans, rock elevations are at least 20-25±ft below anticipated roadway grades. Poorly consolidated alluvial deposits are expected to be present at or just above footing elevations.

Earth Materials and Conditions

Earth materials encountered in the borings are generally consistent with descriptions in published mapping. Encountered materials can be divided into three units considered significant to the proposed project.

Embankment Materials/Fill

Surface soils include embankment and roadway fill. Embankment fill consists of loose to compact clayey sand with gravel and clayey sand/ soft sandy clay. In Boring-1, materials were encountered from ground surface to approximately 7.5±ft depth (elev. 1090±). In Boring-5, roadway and embankment fill was encountered from ground surface down to rock (elev. 1086±). Roadway fill materials were typically encountered in the borings within 1±ft of ground surface. These materials are not erosion resistant, are generally weak, and are not considered to be capable of supporting foundation loads.

Younger/Older Alluvium

In Boring-1, alluvial materials were encountered below embankment fill down to rock – between 7 and 17±ft depths (elevations 1090± and 1080±). These materials consist of semicompact sand with clay and fine gravel. Alluvial materials are generally granular with a clay matrix and appear to be variably consolidated.

These soils are not considered erosion resistant. Alluvial soils are considered moderately competent for supporting relatively light embankment loads without excessive settlement.

Rock

Metamorphic rock was encountered in all borings completed at the bridge for this study. The upper portions (upper 7-10±ft) of encountered rock is very intensely weathered to decomposed; sheared and weathered zones predominate possibly due to nearby fault along Tennessee Creek Valley with harder "Block in Matrix" rocks. Decomposed rock was encountered in the roadway holes (Borings-2, 3 and 4) below roadway fill down to total depth (typically 10±ft below ground surface). In Boring-5 (Abutment-1), decomposed rock was encountered from the top rock surface to 10±ft below (i.e., down to elev. 1075±). Core recovery was low within very intensely weathered and decomposed zones and ranged from 29-50%. Rock Quality Designation was zero for all diamond core runs in both borings at the bridge location.

More competent rock was encountered below elevation 1075 in Borings-1 and 5. Core recovery increased and ranged between 55-100% in Boring-1 and between 84-100% in Boring-5. Rock is sheared and very intensely fractured with some shear zones altered to clay. Some foliations were observed in Boring-1 with foliations dipping steeply at 70-85°. The rock can be characterized as mélange with "Block in Matrix" texture.

Metamorphic rock observed in roadcuts appears hard and intact. Roadcuts are typically steep (possibly 1¼h:1v) and appear to have performed adequately. Rock is considered capable of generating support for heavy, concentrated pile foundation loads.

Groundwater

At the time of the investigation (March 2006) the maximum depth of flow was on the order of 4 ft, but by September 2006 the channel was dry.

Free groundwater levels were measured in the sampled borings made near the channel between 8-10±ft below ground surface (between elev. 1087± and 1089±, approximate channel elevation). Free groundwater levels adjacent to the channels are expected to follow changes in channel water surface. All soils below these levels are expected to be saturated and, granular layers in particular, capable of transmitting substantial quantities of seepage to open excavations.

Site Seismic Conditions

In accordance with current Caltrans Division of Strutures site seismicity evaluation procedures (with reference to "California Seismic Hazard Map 1996," "Attenuation Curves" by Mualchin and Jones, 1992, and Caltrans "Seismic Design Criteria" (SDC) v.1.4), "peak rock acceleration" of 0.6 g is assigned to this site, associated with an event of 6.5 magnitude on the Big Bend-Wolf Creek-Maidu Bear Mountain/E Fault (BWM) located less than 1-mile from the site. Technical information accompanying the "California Seismic Hazards Map 1996" lists the fault type as "normal."

From initial data collected onsite, the site can be conservatively characterized by "Soil Profile Type D" per Caltrans "Seismic Design Criteria" (SDC, v.1.4) Table B.1. From seismic profile data obtained for this study, depth to "rock-like" material exceeds 20±ft. Caltrans procedures requires an increase in spectral acceleration to account for "near-field" directivity effects where structures are located within 15km (9.4± miles) of seismic sources. Typical Caltrans increases to peak bedrock acceleration for fault type (i.e. adjustments to Sadigh attenuation relationships for "reverse thrust" faults and "reverse oblique" faults) do not apply here.

Based on the guidelines as discussed above, the following SDC parameters should be considered:

- Big Bend-Wolf Creek-Maidu Bear Mountain/E Fault (BWM)
- Magnitude 6.5 + 0.25,
- Soil Profile Type D,
- Peak bedrock acceleration of 0.6g

ARS curve from SDC Figure B.7 modified as follows:

Structure Period	Increase in Spectral Acceleration
0-0.5 seconds	No Increase
0.5-1.0 seconds	0% to 20% Linear Increase
Over 1.0 second	20% Increase

The recommended ARS curve is attached (from SDC Figure B.7) as Figure-2.

From limited data, it appears that the risk of liquefaction is likely to be low at this site owing to the nature of encountered materials. Liquefaction effects on bridge structure elements are expected to be reduced/mitigated by foundations deriving support from rock. Potential secondary seismic effects may also include settlement, reduced shear strength, lateral spreading, and sloughing of any over-steep bank slopes.

Should there be important structural and/or economic considerations associated with more closely defining the above values or other site-seismicity characteristics, further study would be required.

Corrosivity

Caltrans Corrosion Guidelines defines corrosive soil as having a pH of 5.5 or less or concentrations of chlorides ≥ 500ppm or concentrations of sulfates ≥ 2000ppm. Soil resistivity is also tested but resistivity results are not used to define a corrosive area. Laboratory testing completed in accordance with CTM 643, CTM 422, and CTM 417 on selected soil and rock samples indicates a "non-corrosive" soils environment to both concrete and steel (per Caltrans "Corrosion Guidelines", September 2003). Laboratory test results are included in Appendix-A.

Conclusions and Discussion

No over-riding geologic hazards (e.g., faulting, landslides, severe erosion, subsidence, etc.), are identified at the bridge site. The Bear Mountain Fault is located nearby but does not appear to cross the site. While the project site is considered susceptible to strong ground shaking from events on nearby faults, the potential for surface fault-rupture on the site is considered low.

The ground appears adequately stable and capable of providing foundation support for the proposed bridge. Although access at both abutments would allow for spread footing foundation construction, this type of foundation is not recommended owing to the presence of shallow groundwater and the depth of excavations to provide foundation security.

Driven displacement piling would not achieve adequate penetration without pre-drilling. Owing to the presence of groundwater and granular soils, pre-drilled holes could be subject to caving. Driven H-piles are technically feasible at both abutments but penetration at Abutment-2 does not appear to be adequate for lateral stability and therefore driven H-piles are not recommended. Bored into place piles ("micropiles") are expected to be technically feasible alternative at both abutments but may be a more costly alternative to standard CIDH piles.

Owing to the presence of groundwater, the use of 16-inch diameter CIDH piling (per Caltrans "Standard Plans") is not appropriate at this site. However, use of 24-inch CIDH piling (per Caltrans "Standard Plans") constructed using "wet specifications" appears feasible and is recommended.

Corrosivity test results (for pH, minimum resistivity, chlorides and sulfates) indicate soils to be non-corrosive to steel and concrete foundations. Corrosion protection is not required for structure foundations.

Asbestos testing (ARB 435) was performed on rock and soil samples only to identify the potential presence of asbestos at the project site. Results from tests performed on selected rock core samples did not show the presence of asbestos. Asbestos in trace amounts was indicated in test results from a bulk sample of soil and decomposed rock from Boring-3. Published mapping and asbestos testing would suggest that the potential presence of asbestos minerals within the project limits cannot be precluded. A dust mitigation plan should be considered for earthwork during construction; installation of CIDH piles is not expected to produce airborne asbestos particulates as construction will require use of "wet specifications" and asbestos testing did not indicate the presence of asbestos in rock encountered in test borings. It is understood that asbestos mitigation details and plans are being developed by others.

Recommendations

CIDH Piling

The presence of free groundwater and the probable need for casing and pumping for ground and water control preclude the use of Caltrans "standard" 16-inch diameter piling. On this basis, recommendations are presented based on the use of 24-inch diameter piling to allow for use of "wet specification" installation.

Design (service) loads to 100 tons are readily available for 24-inch diameter CIDH piling penetrating into rock. Although no tensile demands have been indicated, such piles are capable of developing substantial tensile resistance (if necessary, values can be provided from the data in hand). Piling at the abutments may be considered laterally "fixed" at elev. 1085 at both abutments (approximately 5±ft below bottom of footings).

Pile Data Table

Based on the foregoing, the following Pile Data Table has been developed. This office should be consulted in the event that changes in pile type, cut-off elevation and/or loading are required or if tensile loads need additional consideration.

Table 1: Pile Data Table

Location	Type	Design Loading	Nominal Resistance		Design Tip	Specified Tip
		(service load)	Compression	Tension	Elevations	Elevation
Abutment-1	24" CIDH	200 kips	400 kips	0	1065 (1, 2) 1065 (3)	1065
Abutment-2	24" CIDH	200 kips	400 kips	0	1060 (1, 2) 1060 (3)	1060

Pile tip elevation is controlled by the following demands: (1) compression, (2) tension, (3) lateral

Construction Conditions

Rock materials are expected to be typically "drillable" to anticipated depths with conventional drilling equipment including the use of air tools and/or diamond bits, if necessary. Although not encountered in the test borings within anticipated pile depth, zones of resistant, difficult to drill materials may be encountered in pile excavations. This office should be consulted before any modifications are made to pile tip elevations.

Significant groundwater seepage is expected to be present during foundation construction. The contractor should be prepared to place temporary casing to control caving and seepage. De-watering of uncased pile excavations by means of pumping is not expected to be feasible based on granular nature of the soils at the site. Tremie placement of concrete is anticipated. Good practice dictates pouring of drilled piles as soon as possible after completion of excavation.

Pavement Sections

"R"-value testing indicates subgrade "R"-values between 23 and 38. Design of new pavement sections is based on a basement "R"=23. Pavement section design is based on both Traffic Index (TI) and "R"-Value, but should also reflect local experience and practice, depth and nature of subgrade preparation, and acceptable level of maintenance.

Preliminary flexible pavement sections calculated in accordance with Caltrans design methods (Highway Design Manual, Chapter 600) at design "R"= 23 and for TI = 9.0 through 13.0 inclusive are shown in the following table.

Table 2: Pavement Sections

Preliminary Flexible Pavement Sections
"R"=23

T.I.	**Asphalt (ft)**	**AB Class 2 (ft)**	**AS Class 3 (ft)**	**Structural Section Thickness (ft)**
9.0	0.45	1.45	--	1.90
	0.45	1.00	0.50	1.95
10.0	0.50	1.65	--	2.15
	0.50	1.10	0.60	2.20
11.0	0.55	1.85	--	2.40
	0.55	1.25	0.65	2.45
12.0	0.60	2.05	--	2.65
	0.60	1.35	0.75	2.70
13.0	0.65	2.20	--	2.85
	0.65	1.45	0.85	2.95

Pavement sections shown in the table above incorporate a 0.20-ft gravel equivalent safety factor. Other flexible pavement structural sections for traffic index values other than those shown can be provided, if desired.

Design by the Caltrans method presumes materials and construction in accordance with Caltrans "Standard Specifications", including 95% relative compaction (CTM 216) on all materials within 30-inches of finished grade. Inability to achieve the required compaction on the scarified materials may be used as a field criterion to identify areas requiring additional removal and/or recompaction.

The subgrade soils should be field reviewed with respect to uniformity and suitability by the soils engineer. Any unsuitable material, including clay and loose or disturbed soils, should be removed to full depth and replaced with granular native soil or Class 2 aggregate base compacted to at least 90% relative compaction. Native granular soils – less debris, organic material and particles over 4-inches greatest dimension – are considered suitable for use as compacted fill.

The above pavement design assumes that free water will be absent from the structural section. Adequate surface drainage is of particular importance to limit subgrade saturation and excess free water.

At at-grade crossings, fill sections (excluding back of retaining wall drainage) should be comprised of Aggregate Base.

Existing pavement sections should be removed prior to placing the above recommended sections. Pavement and aggregate materials may be recycled and re-used for this project. Testing of recycled materials would be needed to determine proper use/placement during construction.

Earthwork

Preparation of ground for grading should include stripping and disposal of surficial vegetation and concentrated roots, debris and disturbed soils to at least 5-ft outside the limits of proposed cut and fill. Where trees are removed, all substantial roots should be excavated and removed. Debris, organic material, and otherwise unsuitable or excess materials should be disposed to an approved area/site. All site grading should be performed in accordance with Caltrans "Standard Specifications" (including Section 19, "Earthwork"). Existing facilities within the project limits (e.g., roadways, foundations, slabs, etc.) which will no longer be in service should be removed, along with soils disrupted as a result of facility removal.

Fill Foundations

Areas that are to receive fill should be excavated to a level, firm subgrade (affirmed by this office and expected at 2-3±ft below existing ground surface) and to at least 5-ft horizontal beyond structure limits. The surfaces exposed in these excavations should be reviewed by a representative of this office with respect to anticipated materials and conditions. After favorable field review, embankment fill foundations should be scarified and compacted to at least 90% relative compaction per CTM 216. For roadways, subgrade compaction should meet criteria stated above in "Pavement Sections." Inability to meet the compaction standard can be used as one field criterion for identifying weak soils which should be excavated and replaced as compacted fill.

Loose/soft materials may still be present where the leachfield was removed from the Freedom property. To help mitigate potential settlement, loose/soft materials should be removed down to firm subgrade. Lateral limits of removal should be determined, with consultation by this office, during construction. The base of overexcavation should be reviewed by the engineer with respect to suitability as fill foundation (i.e. no more than ½±ft of loose native soil remaining at bottom of excavation). After favorable review, the base of excavation should be scarified, moisture conditioned and compacted to at least 90% relative compaction (per CTM 216). Suitable and approved import materials should be placed as fill within the leach field excavations and should be compacted to 90% relative compaction (per CTM 216) to subgrade (fill foundation grade) elevation to provide a suitable foundation for embankment fill.

Embankment Fill

We expect that fill materials will be imported (i.e., local borrow source for embankment fill and aggregate base for roadway sections); a local source for borrow materials has not been indicated. Local borrow material for fill should be free of organic matter and debris. Fill slopes at 2h:1v are expected to be appropriate for embankments. Oversize materials (larger than 4-inches across) should be excluded from the fills. Materials should have an expansive index of "EI"< 50. Within 150-ft of the abutments, fill materials should conform to the expansive soils exclusion zone as indicated by Caltrans in "guidelines for Foundation Investigations and Reports," Version 2.0, March 2006.

All fill/embankment material should be thoroughly mixed and moisture-conditioned to 1-2% over optimum moisture (or as approved by this office), then placed in thin lifts (per Caltrans Standard Specifications) and compacted to at least 95% relative compaction (per CTM 216). Where fill is to be placed against existing embankments steeper than 5:1, it should be "bonded" to the slope by cutting vertical benches in the backslope face at 2±ft vertical intervals. If the embankment surface becomes dry due to delays in grading operations, the surface should be scarified, moisture conditioned and re-compacted before additional fill is placed.

Fill slopes should be compacted to a minimum relative compaction of 95% (per above) out to the finished face of the slopes. This can be achieved either by over-building and then trimming the slope, or by direct compaction of the outer face with suitable equipment as the fill is advanced. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Any settlement during embankment construction is expected to occur as fill (assuming granular import) is being placed and no waiting periods are considered necessary prior to construction of new roadway sections or before pile installation at bridge support locations.

Based on the absence of free groundwater at subgrade levels, no general uses of subdrainage elements are anticipated for project fills. Locally, however, seasonal seepage would not be unexpected in low areas; subdrainage should be considered in these low areas, especially if/where seepage is exposed at fill foundation subgrade at the time of construction.

Lateral Soil Pressures

With the use of Caltrans "Structural Backfill" and typical details, an active soil pressure of 36 pcf is considered appropriate for use in abutment wall design. The resultant of incremental lateral soil pressure due to seismic loading will act at 0.6 times the wall height above the base of the wall and the magnitude of resultant may be calculated on the basis of an equivalent fluid pressure of 25 pcf.

In accordance with Caltrans SDC v. 1.4, resistance to longitudinal loading for seismic conditions at abutments can be calculated as follows: 5.0 ksf maximum for effective abutment heights of 5.5 ft or more – which assumes compacted "Structure Backfill"; reduced pressure for shorter abutments is calculated as (h/5.5) X 5.0 ksf, where h is the effective abutment height (in feet).

Cut Slopes

Cut slopes within the project are expected to be stable at 2h:1v slopes. Changes to cut slope recommendations may be considered only upon further field review and affirmation by this office during construction.

Rock Slope Protection

The toe of Rock Slope Protection (RSP) should extend into firm materials below channel bottom for security. To establish the toe of RSP (per Typical Section as shown on Figure 873.3C of Caltrans "Highway Design Manual"), firm soils are expected to be present within the channel at/below elev. 1080. If loose/soft soils are encountered at toe depth, further excavation to a lower elevation may be required to provide adequate and stable support. RSP is expected to be placed a minimum of 3-ft above the top of abutment footings.

Differing Site Conditions

The conclusions presented in this report are based upon the indicated project criteria and the limited data as described. Variations in subsurface conditions may occur, and further, more detailed study could result in modification of conclusions and recommendations contained herein.

Should there be significant change in the project or should soil/rock conditions different from those described in this report be encountered during construction, this office should be notified for evaluation and supplemental recommendations as necessary or appropriate. Communication about site conditions between the Resident Engineer, Contractor and this office should be made as soon as differing conditions are recognized by any of the parties.

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GENERAL CONDITIONS

The conclusions and recommendations of this study are professional opinion based upon the indicated project criteria and the limited data described herein. It is recognized there is potential for variation in subsurface conditions and modification of conclusions and recommendations might emerge from further, more detailed study.

This report is intended only for the purpose, site location and project description indicated and assumes design and construction in accordance with Caltrans practice.

As changes in appropriate standards, site conditions and technical knowledge cannot be adequately predicted, review of recommendations by this office for use after a period of two years is a condition of this report.

A review by this office of any foundation and/or grading plans and specifications or other work product insofar as they rely upon or implement the content of this report, together with the opportunity to make supplemental recommendations as indicated therefrom is considered an integral part of this study and a condition of recommendations.

Subsequently defined construction observation procedures and/or agencies are an element of work, which may affect supplementary recommendations.

Should there be significant change in the project or should soils conditions different from those described in this report be encountered during construction, this office should be notified for evaluation and supplemental recommendations as necessary or appropriate.

Opinions and recommendations apply to current site conditions and those reasonably foreseeable for the described development--which includes appropriate operation and maintenance thereof. They cannot apply to site changes occurring, made, or induced, of which this office is not aware and has not had opportunity to evaluate.

The scope of this study specifically excluded sampling and/or testing for, or evaluation of the occurrence and distribution of, hazardous substances. No opinion is intended regarding the presence or distribution of any hazardous substances at this or nearby sites.

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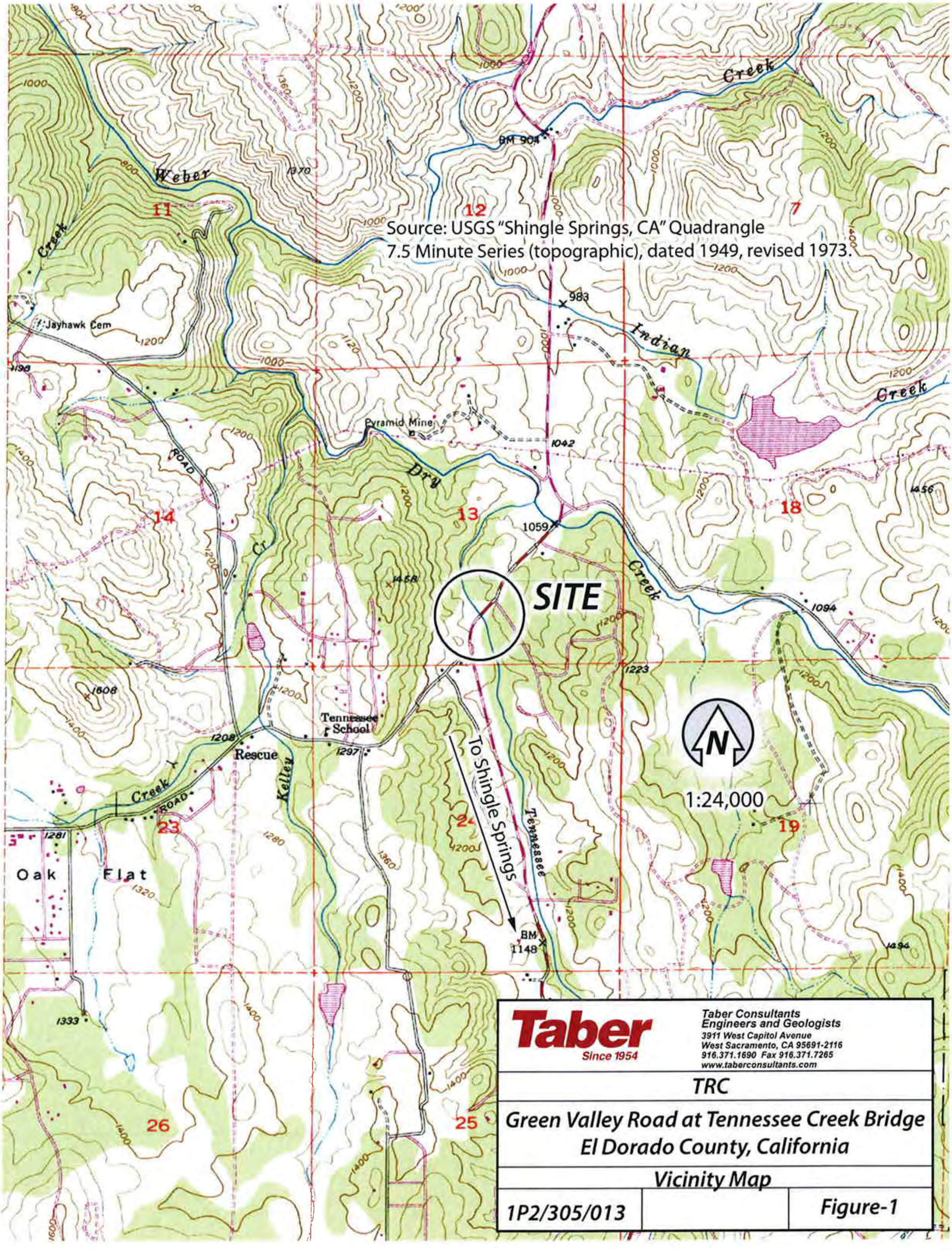

Source: USGS "Shingle Springs, CA" Quadrangle
7.5 Minute Series (topographic), dated 1949, revised 1973.

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TRC

Green Valley Road at Tennessee Creek Bridge
El Dorado County, California

Vicinity Map

1P2/305/013		Figure-1

Caltrans SDC: ARS Curve

Green Valley Road at Tennessee Creek Bridge

1P2/305/013
December 16, 2008

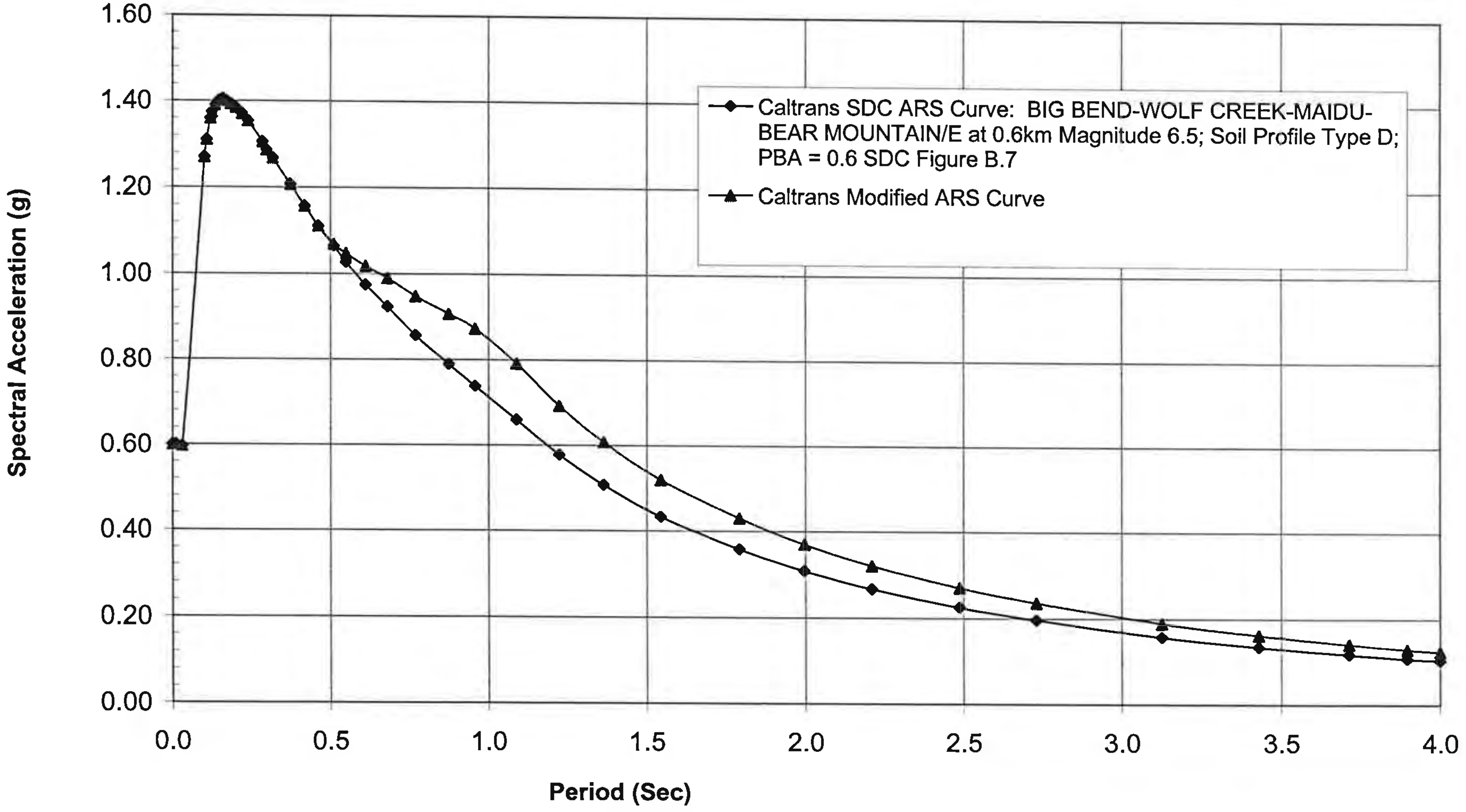


Figure-2

DIST.	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
03	ED	C.R.		48	48

McIlroy
CERTIFIED ENGINEERING GEOLOGIST

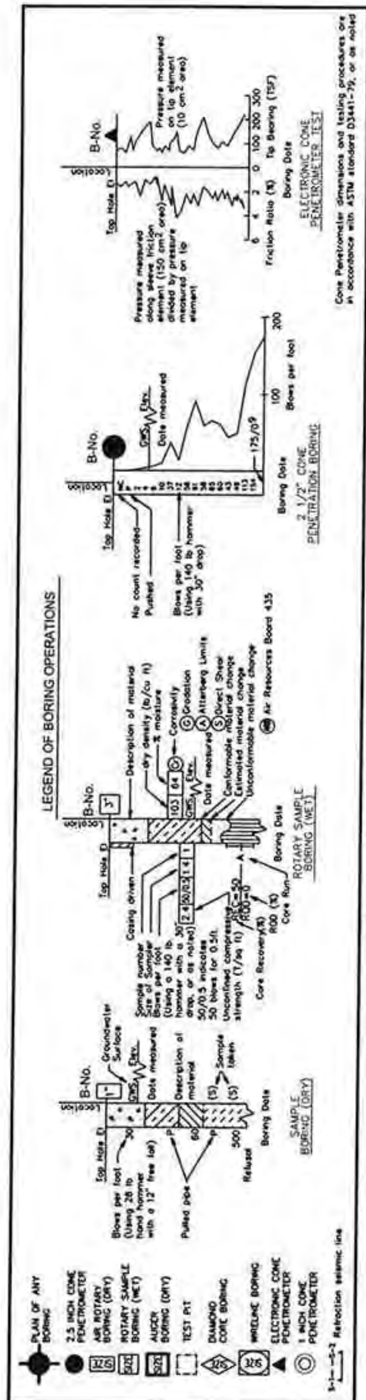
December 19, 2008
PLANS APPROVAL DATE

REGISTERED GEOLOGIST
MARTIN W McILROY
No. 2322
EXP. 03-31-09
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

TABER CONSULTANTS
3911 West Capitol Avenue
West Sacramento, CA 95691-2116

JOB No. 1P2/305/13 LOCATION: 38120-F8:328N:172W

TRC
10680 White Rock Road, Suite 100
Rancho Cordova, CA 95670



LEGEND OF EARTH MATERIALS

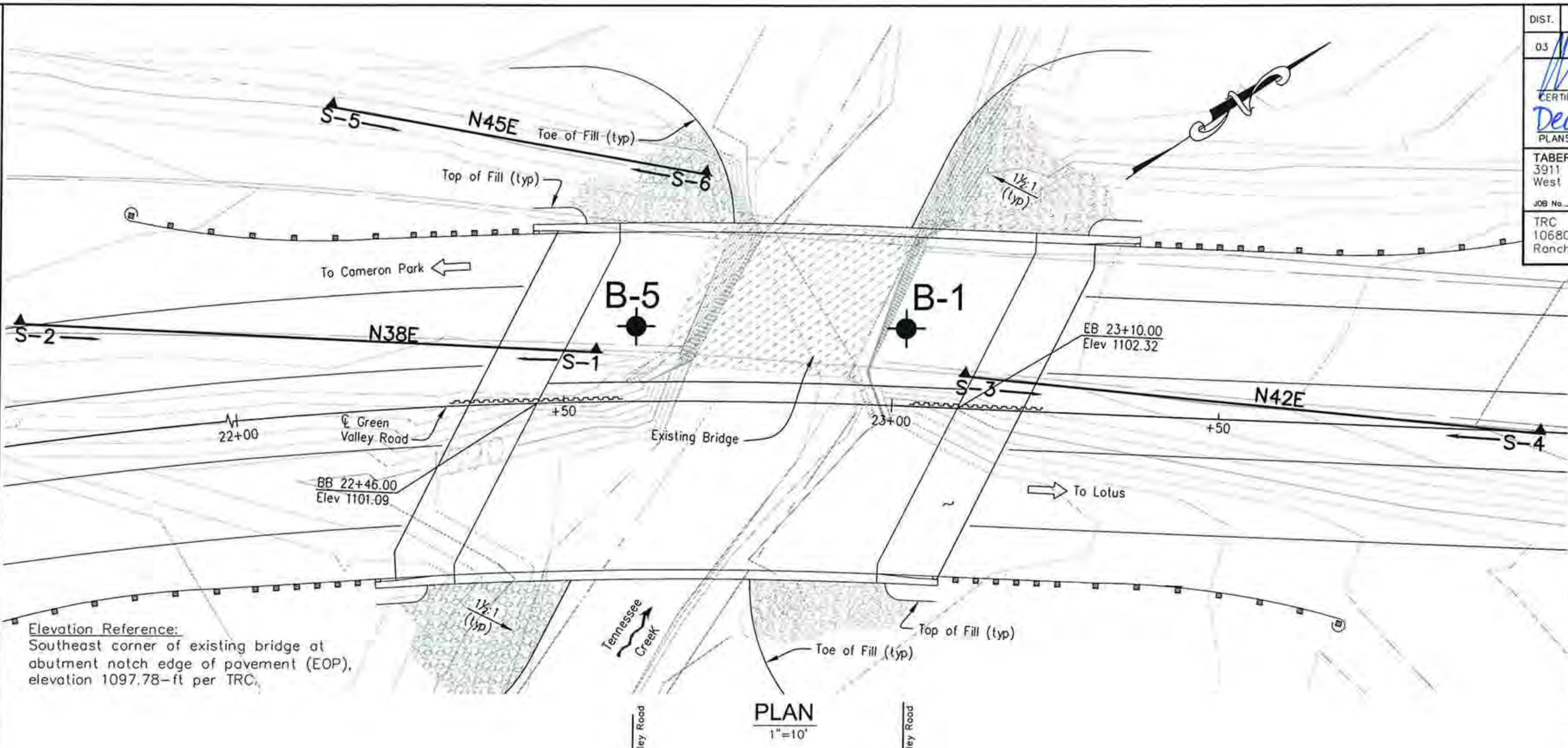
- GRAVEL
- SAND
- SILT
- CLAY
- SANDY CLAY or CLAYEY SAND
- SANDY SILT or SILTY SAND
- SILTY CLAY
- CLAYEY SILT
- PEAT and/or ORGANIC MATTER
- FILL MATERIAL
- IGNEOUS ROCK
- SEDIMENTARY ROCK
- METAMORPHIC ROCK

CONSISTENCY CLASSIFICATION FOR SOILS

According to the Standard Penetration Test

Standard Penetration "N"-Value	Granular	Cohesive
0-5	Very loose	Very soft
6-10	Loose	Soft
11-20	Semicompact	Stiff
21-35	Compact	Very stiff
36-70	Dense	Hard
>70	Very dense	Very hard

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.



Elevation Reference:
Southeast corner of existing bridge at abutment notch edge of pavement (EOP), elevation 1097.78-ft per TRC.

PLAN

1"=10'

NOTES:

1. Field classification of soils was in accordance with ASTM D 2488-00 "Description and Identification of Soils (Visual-Manual Procedure)".

2. Rock classification according to Bureau of Reclamation, U.S. Department of the Interior, "Engineering Geology Field Manual", and "Manual of Field Geology", (Compton, Robert R.).

3. Standard Penetration tests were performed in accordance with ASTM D 1586-99 using a hammer operated with an automated drop system. Drill rods were 1 5/8-inch diameter "A"-rods; sampler was driven with brass liners.

4. The length of each sampled interval is shown graphically on the boring log. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with ASTM D1586-99. Where less than 1 foot of penetration is achieved, the blow count shown is for that fraction of the "standard penetration resistance" interval actually penetrated.

5. Consistency of soils shown in ( ) where estimated.

6. Rock Quality Designation (RQD), Weathering, Rock Hardness/Strength, Bedding, and Fracture Density, as shown on this sheet, were used to describe all rock core. Descriptors were determined in the field.

7. REC = Core Recovery (percent).

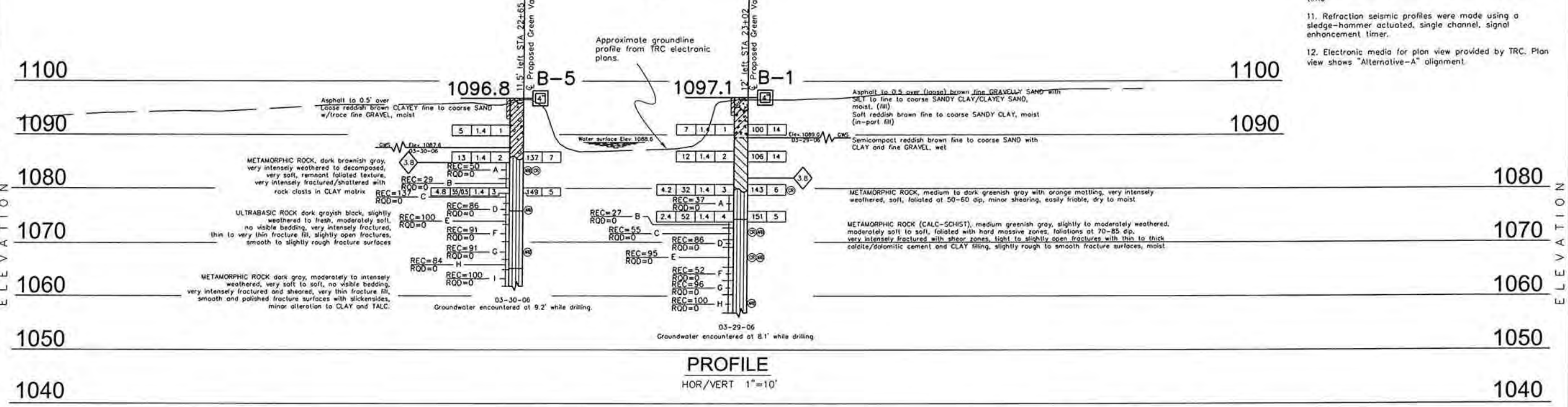
8. RQD = Rock Quality Designation (percent).

9. Groundwater surface (GWS) elevations in the borings indicated on the Log of Test Boring Sheets reflect the fluid level in the borings on the specified date.

10. Groundwater surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.

11. Refraction seismic profiles were made using a sledge-hammer actuated, single channel, signal enhancement timer.

12. Electronic media for plan view provided by TRC. Plan view shows "Alternative-A" alignment.



Asphalt to 0.5' over
Loose reddish brown CLAYEY fine to coarse SAND w/trace fine GRAVEL, moist

METAMORPHIC ROCK, dark brownish gray, very intensely weathered to decomposed, very soft, remnant foliated texture, very intensely fractured/shattered with rock clasts in CLAY matrix

ULTRABASIC ROCK dark grayish black, slightly weathered to fresh, moderately soft, no visible bedding, very intensely fractured, thin to very thin fracture fill, slightly open fractures, smooth to slightly rough fracture surfaces

METAMORPHIC ROCK dark gray, moderately to intensely weathered, very soft to soft, no visible bedding, very intensely fractured and sheared, very thin fracture fill, smooth and polished fracture surfaces with slickensides, minor alteration to CLAY and TALC.

03-30-06
Groundwater encountered at 9.2' while drilling.

Asphalt to 0.5 over (loose) brown fine GRAVELLY SAND with SILT to fine to coarse SANDY CLAY/CLAYEY SAND, moist, (fill)

Soft reddish brown fine to coarse SANDY CLAY, moist (in-part fill)

Semicompact reddish brown fine to coarse SAND with CLAY and fine GRAVEL, wet

METAMORPHIC ROCK, medium to dark greenish gray with orange mottling, very intensely weathered, soft, foliated at 50-60 dip, minor shearing, easily friable, dry to moist

METAMORPHIC ROCK (CALC-SCHIST), medium greenish gray, slightly to moderately weathered, moderately soft to soft, foliated with hard massive zones, foliations at 70-85 dip, very intensely fractured with shear zones, tight to slightly open fractures with thin to thick calcite/dolomitic cement and CLAY filling, slightly rough to smooth fracture surfaces, moist

03-29-06
Groundwater encountered at 8.1' while drilling

PROFILE

HOR/VERT 1"=10'

DESIGN OVERSIGHT	DRAWN BY	M. W. McIlroy	M. W. McIlroy FIELD INVESTIGATOR
SIGN OFF DATE	CHECKED BY	M. W. McIlroy	DATE March 2006

PREPARED FOR
EL DORADO COUNTY
DEPARTMENT OF PUBLIC WORKS

PROJECT ENGINEER

BRIDGE NO.
25C0038
POST MILE

GREEN VALLEY ROAD AT TENNESSEE CREEK BRIDGE

LOG OF TEST BORINGS

12/18/08 1P230513.dwg 5.56 1=1

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU
EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 48 OF 48

WEATHERING DESCRIPTORS

Descriptors	Diagnostic features					General characteristic (strength, excavation, etc)§
	Chemical weathering–Discoloration and/or oxidation		Mechanical weathering–Grain boundary conditions (disaggregation) primarily for granitics and some coarse-grained sediments	Texture and solutioning		
	Body of rock	Fracture surfaces†		Texture	Solutioning	
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck. Almost always rock excavation except for naturally weak or weakly cemented rocks such as siltstones or shales.
Slightly weathered to fresh°						
Slightly weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck. Body of rock not weakened. With few exceptions, such as siltstones or shales, classified as rock excavation
Moderately to slightly weathered°						
Moderately weathered	Discoloration or oxidation extends from fractures usually throughout; Fe–Mg minerals are "rusty", feldspar crystals are "cloudy".	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened. Depending on fracturing, usually is rock excavation except in naturally weak rocks such as siltstones or shales.
Intensely to moderately weathered°						
Intensely weathered	Discoloration or oxidation throughout; all feldspars and Fe–Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, see grain boundary conditions.	All fracture surfaces are discolored or oxidized, surfaces friable	Partial separation, rock is friable; in semiarid conditions granitics are disaggregated	Texture altered by chemical disintegration (hydration, argillation)	Leaching of soluble minerals may be complete	Dull sound when struck with hammer, usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures, or veinlets. Rock is significantly weakened. Usually common excavation.
Very intensely weathered°						
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe–Mg minerals are completely altered to clay.		Complete separation of grain boundaries (disaggregated)	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Always common excavation. Resistant minerals such as quartz may be present as "stringers" or "dikes"

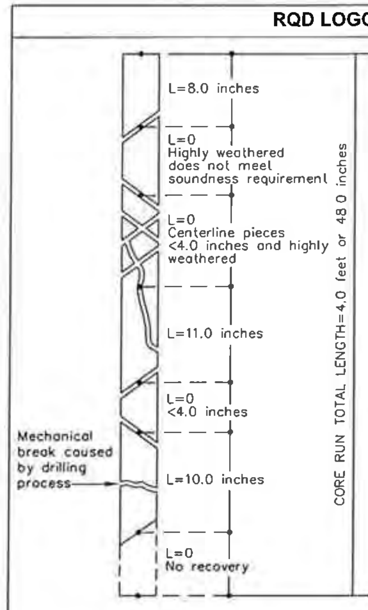
NOTE: This chart and its horizontal categories are more readily applied to rocks with feldspars and mafic minerals. Weathering in various sedimentary rocks, particularly limestones and poorly indurated sediments, will not always fit the categories established. This chart and weathering categories may have to be modified for particular site conditions or alterations such as hydrothermal effects; however, the basic framework and similar descriptors are to be used.

° Combination descriptors are permissible where equal distribution of both weathering characteristics are present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, dual descriptors should not be used where significant, identifiable zones can be delineated. When given as a range, only two adjacent terms may be combined (i.e., decomposed to slightly weathered or moderately weathered to fresh) are not acceptable.

† Does not include directional weathering along shears or faults and their associated features. For example, a shear zone that carried weathering to great depths into a fresh rock mass would not require the rock mass to be classified as weathered.

§ These are generalizations and should not be used as diagnostic features for weathering or excavation classification. These characteristics vary to a large extent based on naturally weak materials or cementation and type of excavation

RQD LOGGING



$$RQD=\frac{\sum \text{Length of sound core >4.0 pieces}}{\text{Total core run length}}$$

$$RQD=\frac{8.0+11.0+10.0}{48\ \text{inches}}\ X\ 100\%$$

RQD= 60% (fair)

RQD (ROCK QUALITY DESIGNATION)	DESCRIPTION OF ROCK QUALITY
0 – 25%	VERY POOR
25 – 50%	POOR
50 – 75%	FAIR
75 – 90%	GOOD
90 – 100%	EXCELLENT

After Deere & Deere, 1989

IGNEOUS AND METAMORPHIC ROCK GRAIN SIZE DESCRIPTORS

Descriptor	Average crystal diameter
Very coarse-grained or pegmatic	> 7 3/8 inch
Coarse-grained	3/16–3/8 inch
Medium-grained	1/32–3/16 inch
Fine-grained	0 04–1/32 inch
Aphanitic (cannot be seen with the unaided eye)	<0.04 inch

ROCK HARDNESS/STRENGTH DESCRIPTORS

Descriptor	Criteria
Extremely hard	Core, fragment, or exposure cannot be scratched with knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Can be scratched with knife or sharp pick. Core or fragment breaks with repeated heavy hammer blows
Hard	Can be scratched with knife or sharp pick with difficulty (heavy pressure). Heavy hammer blow required to break specimen
Moderately hard	Can be scratched with knife or sharp pick with light or moderate pressure. Core or fragment breaks with moderate hammer blow.
Moderately soft	Can be grooved 1/16 inch (2 mm) deep by knife or sharp pick with moderate or heavy pressure. Core or fragment breaks with light hammer blow or heavy manual pressure
Soft	Can be grooved or gouged easily by knife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure
Very soft	Can be readily indented, grooved or gouged with fingernail, or carved with a knife. Breaks with light manual pressure

Any bedrock unit softer than very soft, is to be described using USBR 5000 consistency descriptors.

NOTE: Although "sharp pick" is included in these definitions, descriptions of ability to be scratched, grooved or gouged by a knife is the preferred criteria

BEDDING, FOLIATION, OR FLOW TEXTURE DESCRIPTORS

Descriptor	Thickness/spacing
Massive	Greater than 10 ft (3 m)
Very thickly, (bedded, foliated, or banded)	3 to 10 ft (1 to 3 m)
Thickly	1 to 3 ft (300 mm to 1 m)
Moderately	0 3 to 1 ft (100 to 300 mm)
Thinly	0 1 to 0 3 ft (30 to 100 mm)
Very thinly	0 03 [3/8 in] to 0 1 ft (10 to 30 mm)
Laminated (intensely foliated or banded)	Less than 0 03 ft [3/8 in] (<10 mm)

FRACTURE DENSITY

FRACTURE DENSITY – Based on the spacing of all natural fractures in an exposure or core recovery lengths in drill holes; excludes mechanical breaks, shears, and shear zones; however, shear-distributed zones (fracturing outside the shear) are included. Descriptors for fracture density apply to all rock exposures such as tunnel walls, dozer trenches, outcrops, or foundation cut slopes and inverts, as well as boreholes. Descriptive criteria presented below are based on drill hole cores where lengths are measured along the core axis, for other exposures the criteria is distance measured between fractures (size of blocks).

UNFRACTURED: No/observed fractures

VERY SLIGHTLY FRACTURED: Core recovered mostly in lengths greater than 3 feet (1 m)

SLIGHTLY TO VERY SLIGHTLY FRACTURED

SLIGHTLY FRACTURED: Core recovered mostly in lengths from 1 to 3 feet (300 to 1000 mm) with few scattered lengths less than 1 foot (300 mm) or greater than 3 feet (1000 mm).

MODERATELY TO SLIGHTLY FRACTURED

MODERATELY FRACTURED: Core recovered mostly in 0 33 to 1.0 foot (100 to 300 mm) lengths with most lengths about 0 67 foot (200 mm)

INTENSELY TO MODERATELY FRACTURED

INTENSELY FRACTURED: Lengths average from 0.1 to 0.33 foot (30 to 100 mm) with scattered fragmented intervals. Core recovered mostly in lengths less than 0.33 foot (100 mm).

VERY INTENSELY TO INTENSELY FRACTURED

VERY INTENSELY FRACTURED: Core recovered mostly as chips and fragments with a few scattered short core lengths

* Combinations of fracture densities (e.g. very intensely to intensely fractured or moderately to slightly fractured) are used where equal distribution of both fracture density characteristics are present over a significant interval or exposure, or where characteristics are "in between" the descriptor definitions

Source: U.S. Department of Interior, Bureau of Reclamation "Engineering Geology Field Manual"

DESIGN OVERSIGHT
SIGN OFF DATE
DRAWN BY
CHECKED BY
FIELD INVESTIGATOR
DATE
PROJECT ENGINEER
BRIDGE NO.
POST MILE

GREEN VALLEY ROAD AT TENNESSEE CREEK BRIDGE

ENGINEERING GEOLOGY FIELD DESCRIPTORS

Figure - 3

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TEST BORING LOG

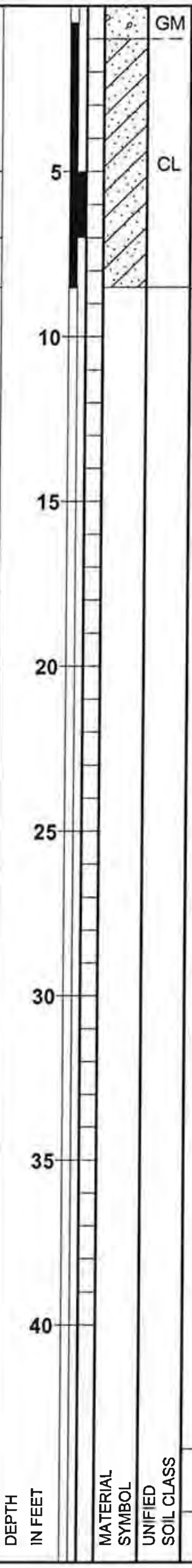
Job No. 1P2/305/13

TYPE: 4-INCH AUGER

SURFACE ELEVATION: 1098.2

BORING NO 2

UNCONFINED COMPRESSIVE STRENGT (tsf)	OTHER TESTS	DRY DENSITY (lbs/cu. ft.)	Moisture (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (inches)	SAMPLE No.
	R					A
0.4		108	22	4	1.4	1
						B



GM (Loose) reddish brown SILTY fine to coarse SAND with GRAVEL, (moist)

CL Very soft to (stiff) reddish brown fine to coarse SANDY CLAY, moist

Bottom of hole at 8.5 feet.

No groundwater encountered. Essential auger refusal at 8.5'

LOGGED BY: MWM/EMH

DATE: 03-29-2006

LOG OF BORING (SOILS ONLY QU) 1P2 305 13 GREEN VALLEY RD. XLN.GPJ LIBRARY.GLB DATATEMPLATE.GDT 12/18/08

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TEST BORING LOG

Job No. 1P2/305/13

TYPE: 4-INCH AUGER

SURFACE ELEVATION: 1105.3

BORING NO 3

UNCONFINED COMPRESSIVE STRENGT (tsf)	OTHER TESTS	DRY DENSITY (lbs/cu. ft.)	Moisture (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (inches)	SAMPLE No.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS	
										Asphalt concrete
									SM	(Loose) SILTY fine to coarse SAND with fine GRAVEL, moist
										METAMORPHIC ROCK, soft to very soft dark to medium brown/ reddish brown very intensely weathered to decomposed.
	R, ARB				BULK	C	5			
				89	1.4	1				
										Bottom of hole at 6.5 feet.
							10			No groundwater encountered.
							15			
							20			
							25			
							30			
							35			
							40			

THE BORING LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOGGED BY: MWM/EMH

DATE: 03-29-2006

LOG OF BORING (SOILS ONLY QU) 1P2 305 13 GREEN VALLEY RD. XLN.GPJ LIBRARY.GLB DATATEMPLATE.GDT 12/18/08

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TEST BORING LOG

Job No. 1P2/305/13

TYPE: 4-INCH AUGER

SURFACE ELEVATION: 1110.4

BORING NO 4

UNCONFINED COMPRESSIVE STRENGT (tsf)	OTHER TESTS	DRY DENSITY (lbs/cu. ft.)	Moisture (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (inches)	SAMPLE No.	DEPTH IN FEET	UNIFIED SOIL CLASS	
								GM	(Loose) brown SILTY fine GRAVELLY SAND, moist
								ML	Dense reddish brown / light brown, SILTY fine SAND with CLAY / CLAYEY fine SANDY SILT, moist
	R, ARB				BULK	D	5		
4.9		133	12	55	1.4	1			
									Bottom of hole at 6.9 feet.
							10		No groundwater encountered.
							15		
							20		
							25		
							30		
							35		
							40		

MATERIAL SYMBOL

THE BORING LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

LOGGED BY: MWM/EMH

DATE: 03-30-2006

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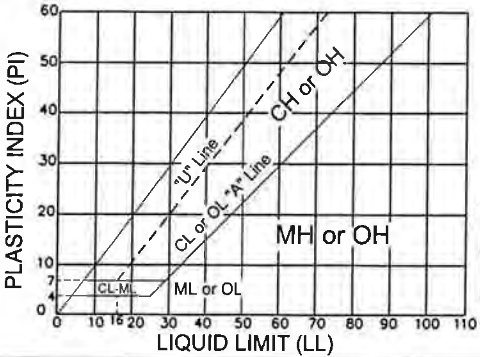
UNIFIED SOIL CLASSIFICATION SUMMARY

(ASTM D 2489-00)

Pt	OH	CH	MH	OL	CL	ML	SC	SM	SP	SW	GC	GM	GP	GW
Highly organic soils	Silts and clays Liquid limit 50 or more			Silts and clays Liquid limit less than 50			Sands with fines >15% fines		Clean sands ≤ 5% fines		Gravels with fines > 15% fines		Clean gravels ≤ 5% fines	
							Sands–50% or more of coarse fraction is smaller than No 4. Sieve				Gravels–more than 50% of coarse fraction is larger than No. 4 sieve			
Fine grained soils (50% or more is smaller than No. 200 sieve)							Coarse grained soils (More than 50% is larger than No. 200 sieve)							



LABORATORY CLASSIFICATION CRITERIA

GW and SW – Cu ≥ 4 for GW and 6 FOR SW; 1 ≤ Cc ≤ 3

GP and SP–Clean gravel or sand not meeting requirements for GW and SW.

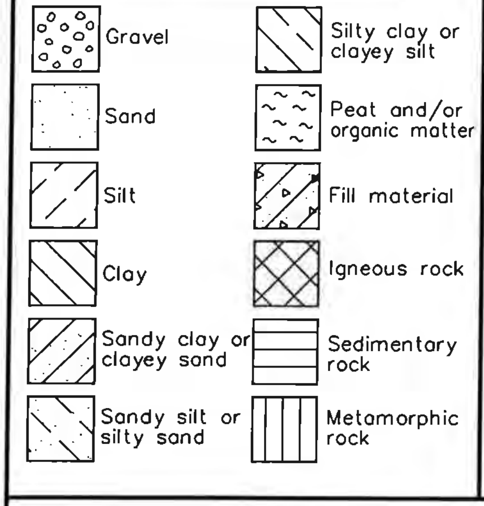
GM and SM–Atterberg limits of fines below "A" line or P.I. less than 4.

GC and SC–Atterberg limits of fines above "A" line with P.I. greater than 7.

Fines (silt or clay)	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		
Sieve sizes	200	40	10	4	3/4"	3"	10"

Classification of earth materials shown on the test boring logs is based on field inspection and should not be construed to imply laboratory analysis unless so stated.

MATERIAL SYMBOLS



CONSISTENCY CLASSIFICATION FOR SOILS

Standard Penetration "N"–Value*	Granular	Cohesive
0–5	Very loose	Very soft
6–10	Loose	Soft
11–20	Semicompact	Stiff
21–35	Compact	Very stiff
36–70	Dense	Hard
> 70	Very dense	Very hard

* According to the Standard Penetration Test (ASTM D 1586)

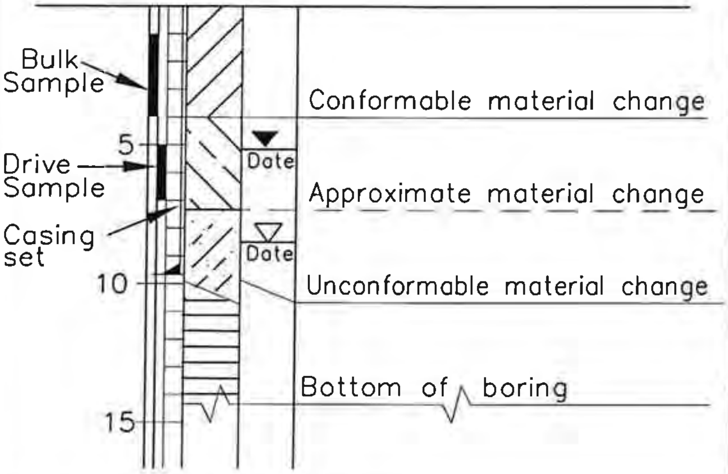
Blow count of 50/0.5 indicates 50 blows for 0.5 feet.

Where standard penetration test has not been performed, consistencies shown (in parenthesis) on logs are estimated.

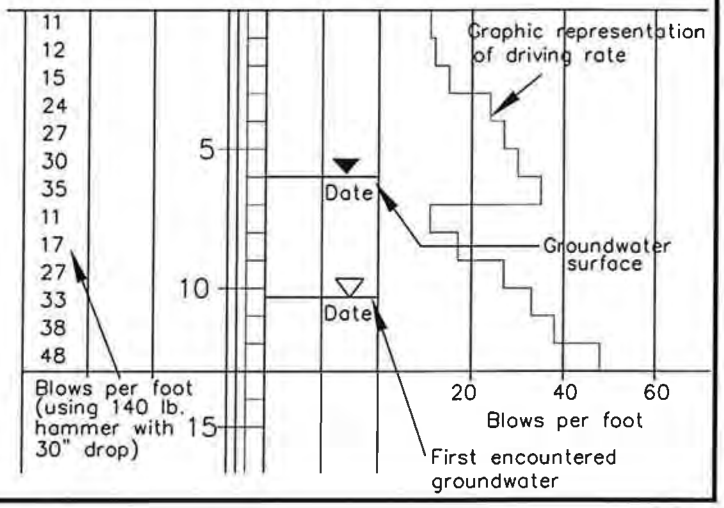
KEY TO "OTHER TESTS" LABORATORY

A – Atterberg Limits
ARB – California Air Resources Board 435
C – Consolidation
CR – Corrosivity
E – Expansion Index
G – Gradation
H – Hydrometer
M – Maximum Dry Density
P – Permeability
R – Resistance Value
S – Direct Shear
SE – Sand Equivalent
SG – Specific Gravity
T – Triaxial Shear

LEGEND OF BORING



LEGEND OF PENETRATION TEST



rev. 11/01 newboringlegend.dwg fit L

Boring Legend

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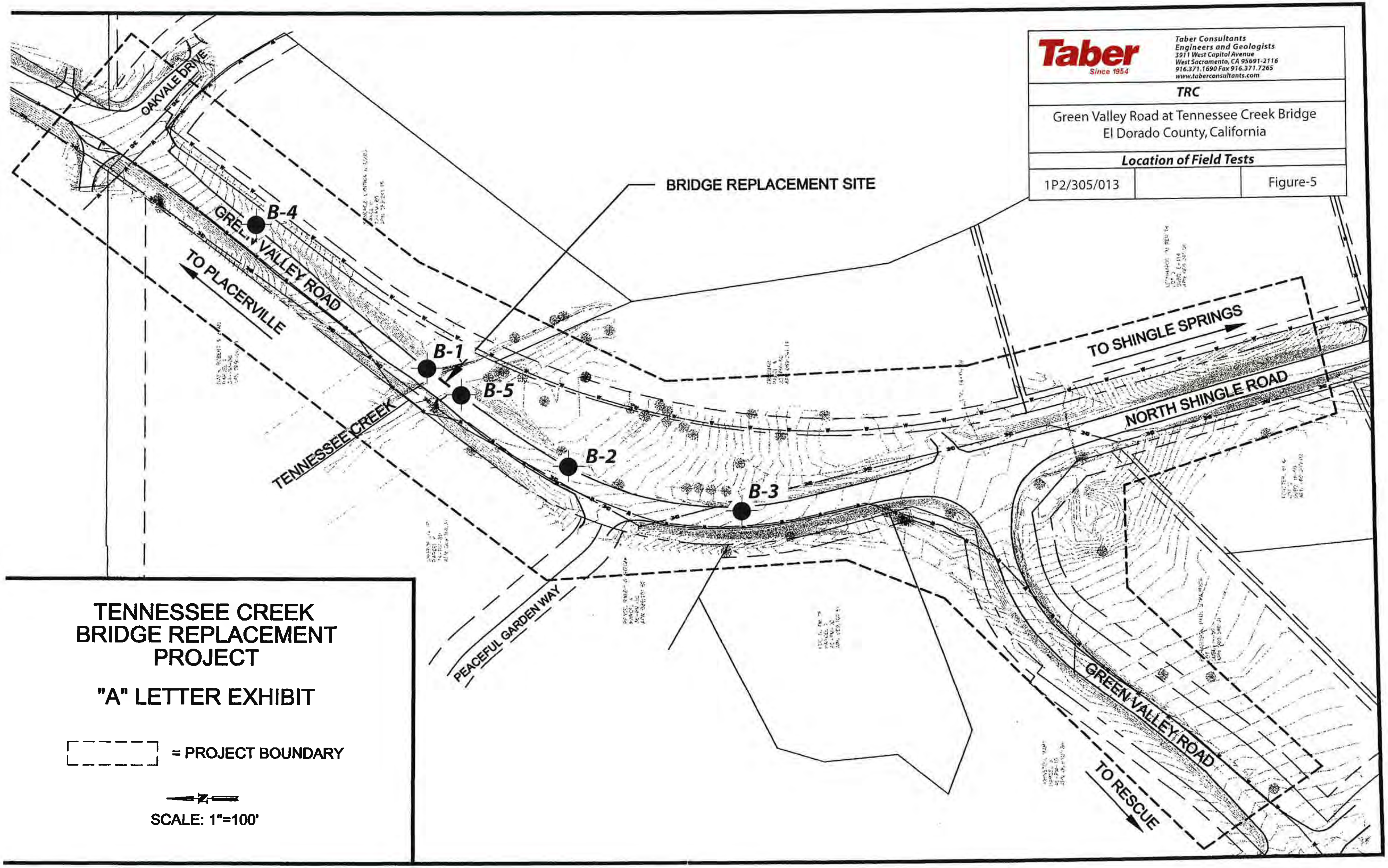
TRC

Green Valley Road at Tennessee Creek Bridge
El Dorado County, California

Location of Field Tests

1P2/305/013		Figure-5



TENNESSEE CREEK BRIDGE REPLACEMENT PROJECT

"A" LETTER EXHIBIT

= PROJECT BOUNDARY

SCALE: 1"=100'

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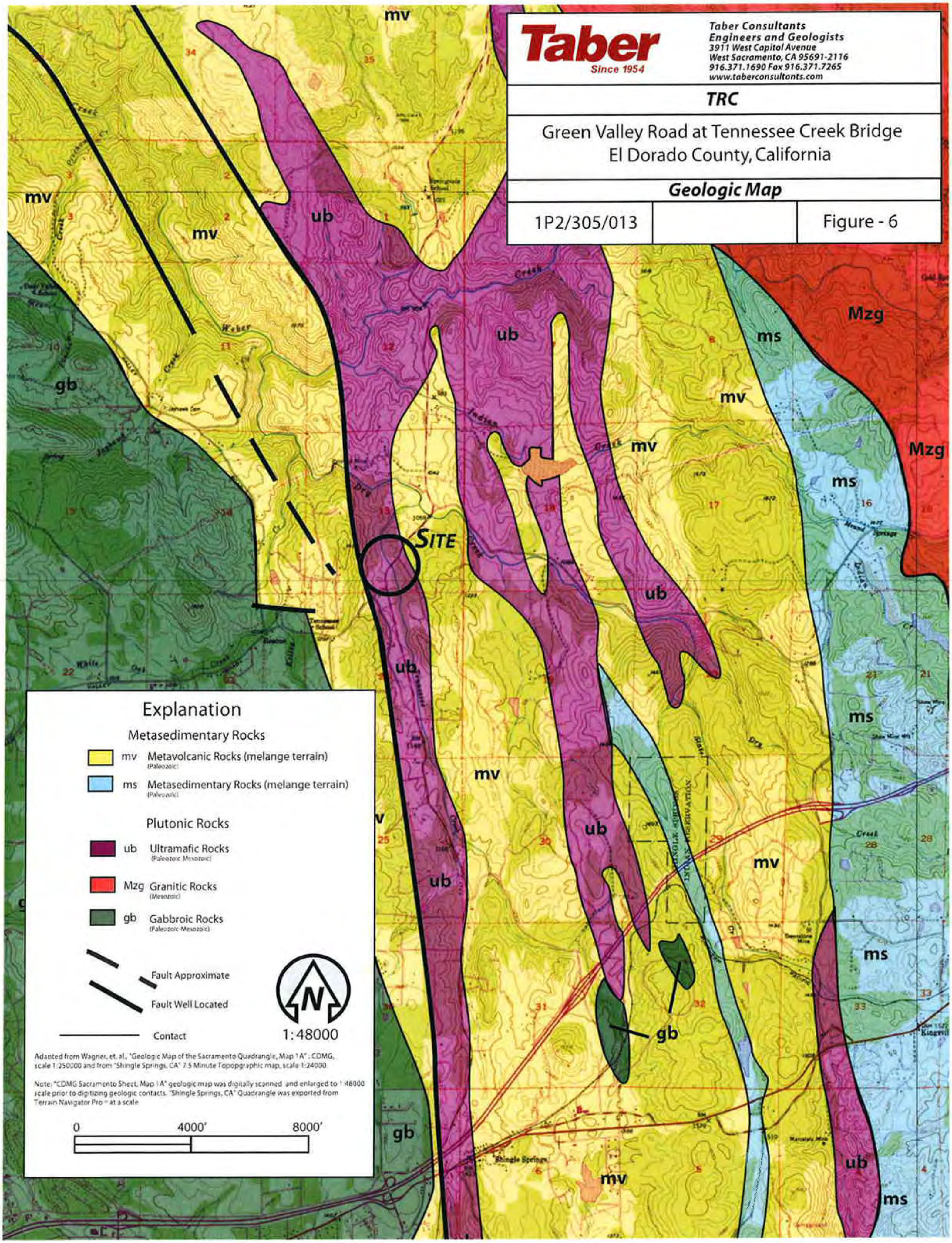
TRC

Green Valley Road at Tennessee Creek Bridge
El Dorado County, California

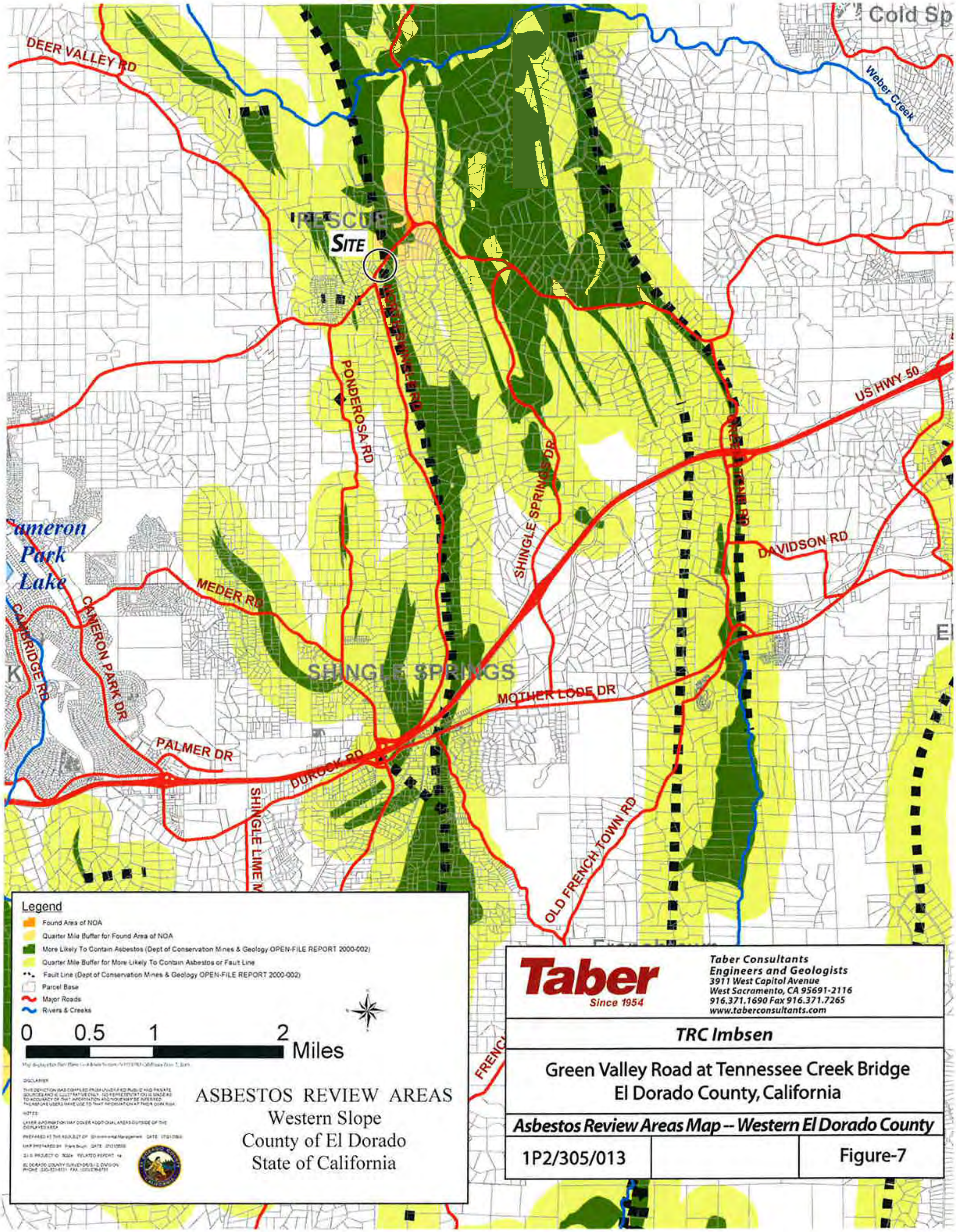
Geologic Map

1P2/305/013		Figure - 6



Adapted from Wagner, et. al., "Geologic Map of the Sacramento Quadrangle, Map 1A" : CDMG, scale 1:250000 and from "Shingle Springs, CA" 7.5 Minute Topopgraphic map, scale 1:24000

Note: "CDMG Sacramento Sheet, Map 1A" geologic map was digitally scanned and enlarged to 1:48000 scale prior to digitizing geologic contacts. "Shingle Springs, CA" Quadrangle was exported from Terrain Navigator Pro = at a scale



ASBESTOS REVIEW AREAS
Western Slope
County of El Dorado
State of California

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Green Valley Road at Tennessee Creek Bridge
El Dorado County, California

Asbestos Review Areas Map -- Western El Dorado County

1P2/305/013		Figure-7

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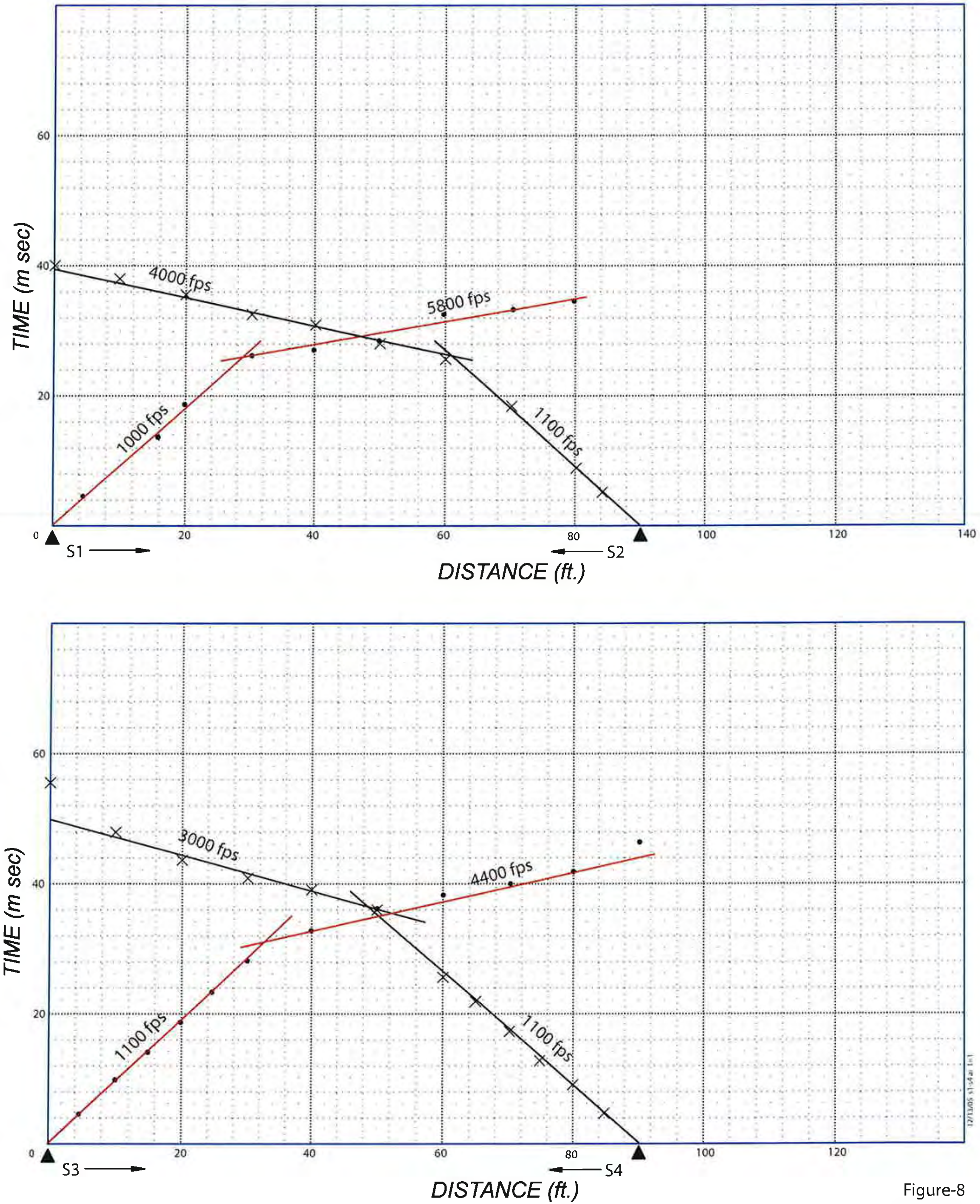
1P2/305/013

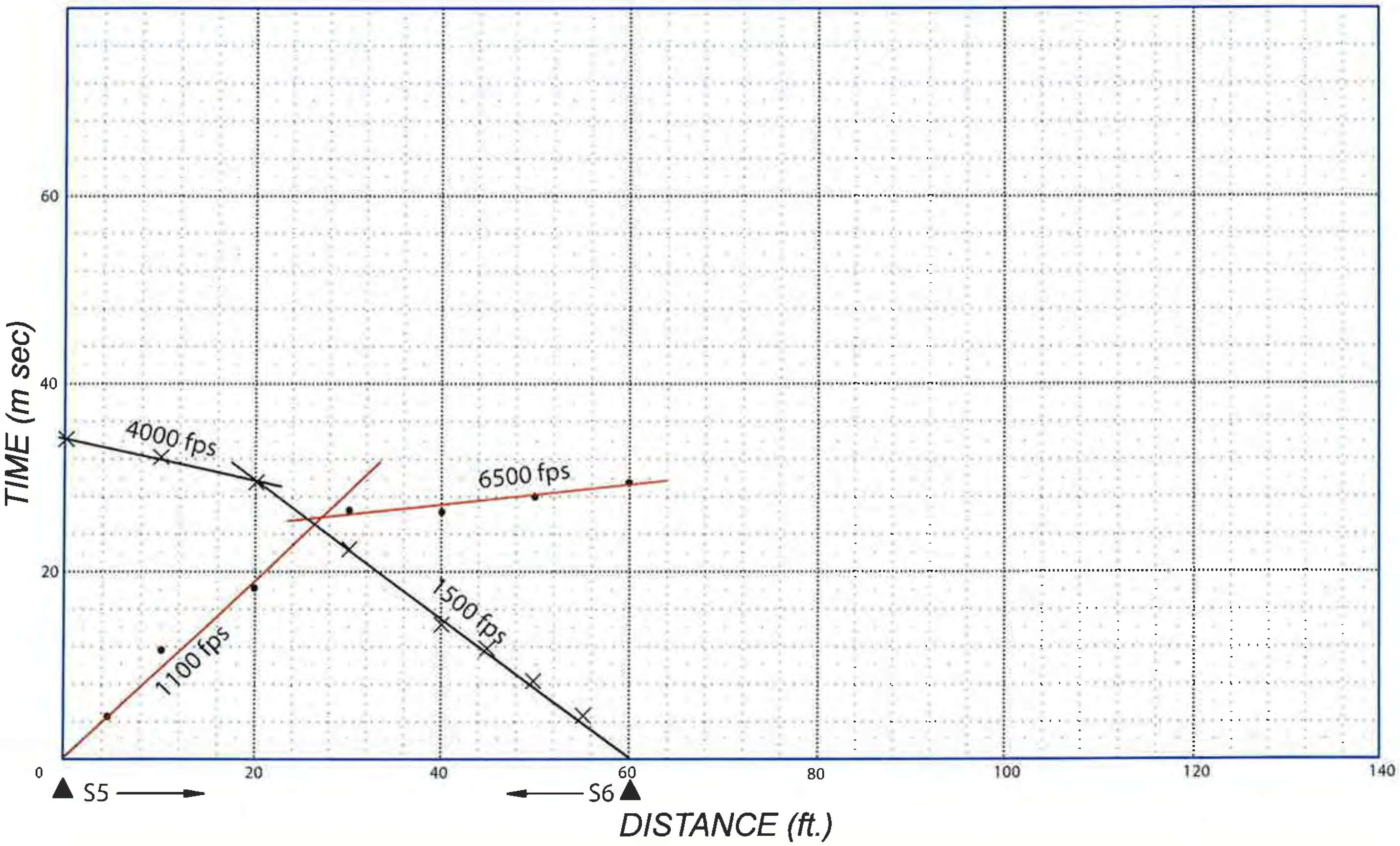


12/13/05 s1-s4.ai 1=1

REFRACTION SEISMIC RECORD

Figure-8
Page 1 of 2

1P2/305/013



REFRACTION SEISMIC RECORD

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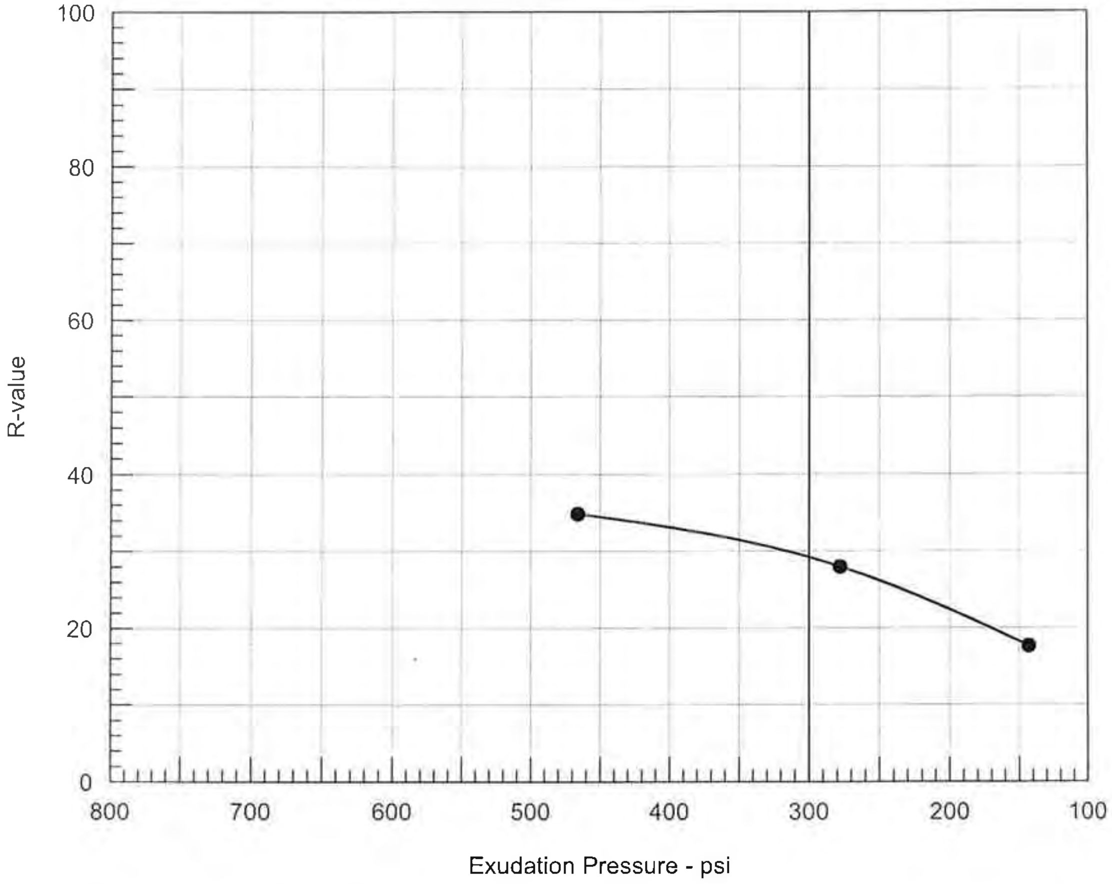
1P2/305/13

Laboratory Test Results

Chloride/ Sulfate & pH/Min. Resistivity

Boring/ Sample	Depth	Description	Chloride (ppm)	Sulfate (ppm)	pH	Minimum Resistivity (ohm-cm x 1000)
B-1/ 3	16.2-18.2	dark greenish gray clayey sand	14.3	4.8	7.53	2.57
B-1/ C	24.3-26.5	greenish gray metamorphic rock	24.2	3.3	8.80	2.09
B-1/ E	27.9-31.6	greenish gray metamorphic rock	4.5	3.7	9.02	1.66
B-5/ A	12.0-15.0	brown-brownish gray metamorphic rock	7.0	3.3	7.09	3.22

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	125.7	11.8	231	87	2.44	466	36	35
2	235	122.4	13.1	118	103	2.60	278	26	28
3	125	119.9	14.0	52	121	2.58	143	17	18

Test Results	Material Description
R-value at 300 psi exudation pressure = 29	

Project No.: 1P2/305/13

Project: Green Valley Road at Tennessee Creek Bridge Reconstruction Project

Location: Boring 2

Sample Number: Bag A **Depth:** 0.5'-5.0'

Date: 12/17/2008

Tested by: RJF

Checked by: MWM

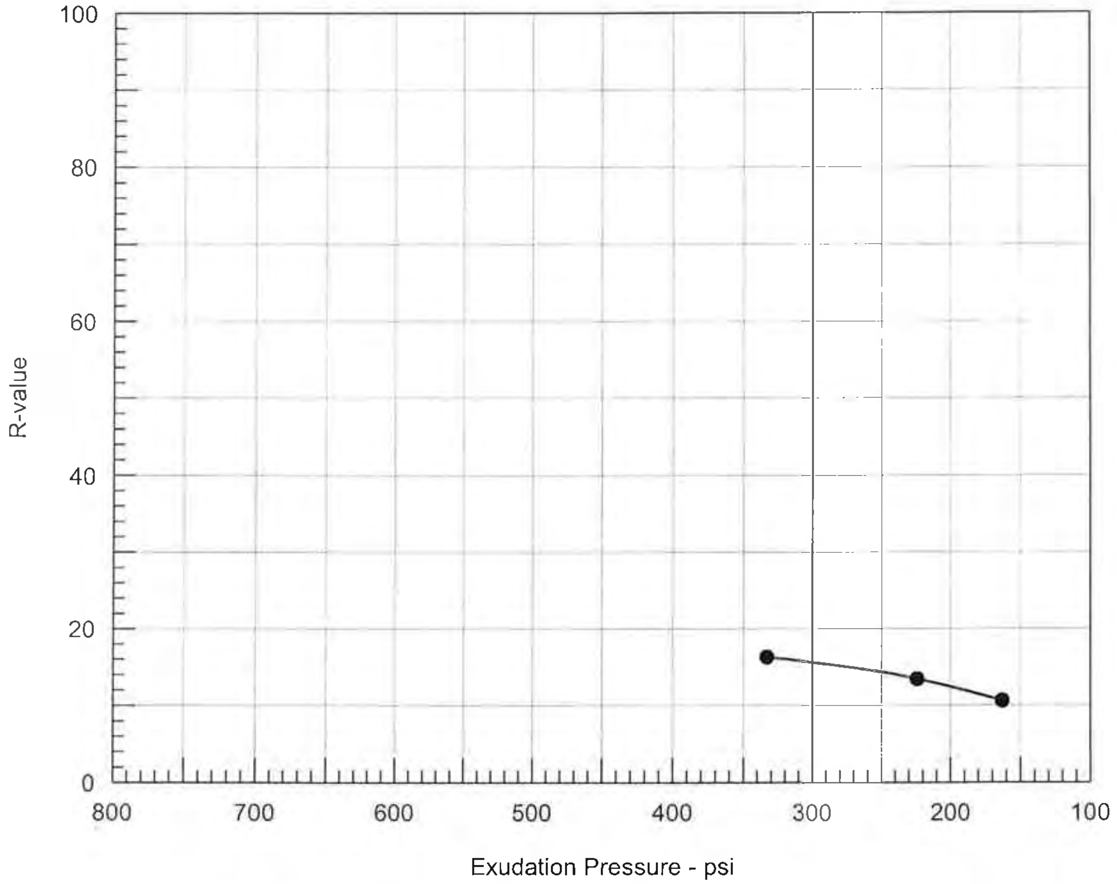
Remarks:

R-VALUE TEST REPORT

Taber Consultants

Appendix A

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	130	114.6	15.7	131	123	2.59	333	15	16
2	115	110.5	17.0	52	127	2.48	224	13	13
3	50	108.1	18.1	0	132	2.53	163	11	11

Test Results	Material Description
R-value at 300 psi exudation pressure = 16	

Project No.: 1P2/305/13
Project: Green Valley Road at Tennessee Creek Bridge Reconstruction Project
Location: Boring 3
Sample Number: Bag C **Depth:** 0.5'-5.0'
Date: 12/17/2008

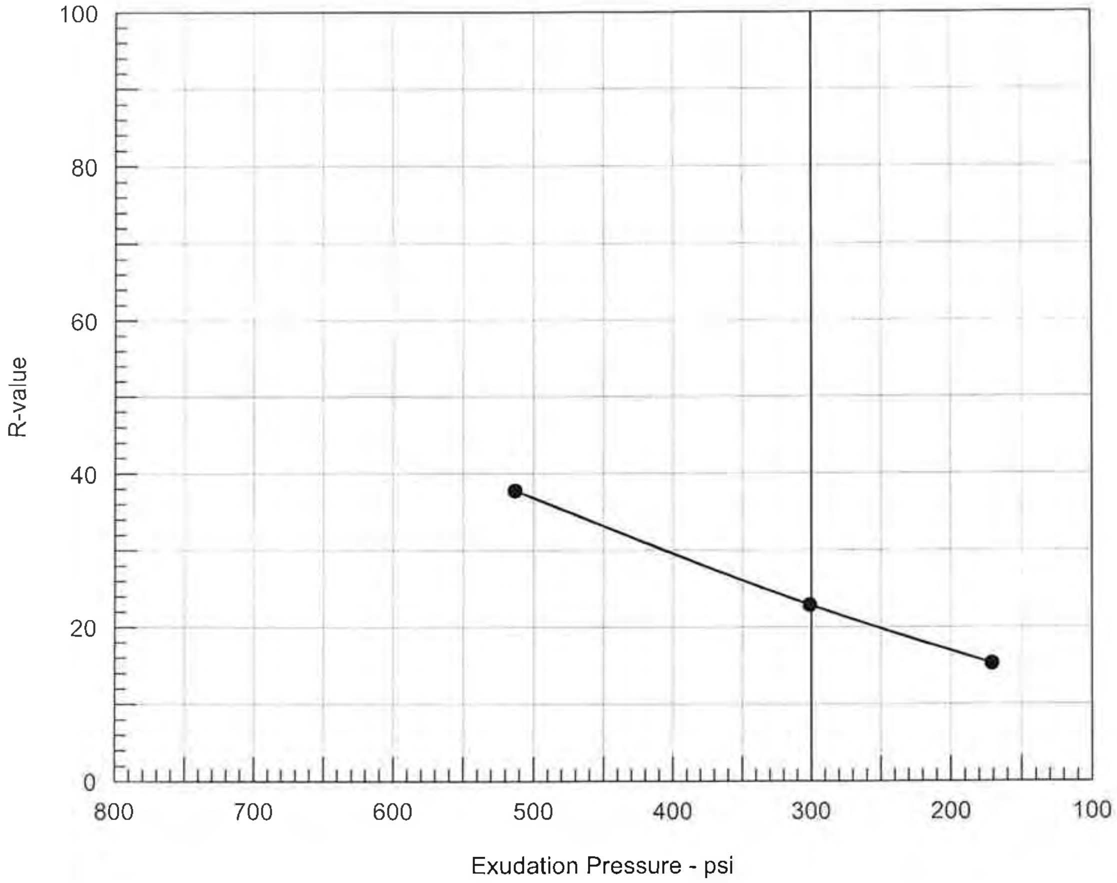
Tested by: RJF
Checked by: MWM
Remarks:

R-VALUE TEST REPORT

Taber Consultants

Appendix A

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	215	133.2	9.3	629	85	2.46	513	38	38
2	160	129.3	10.4	428	107	2.40	301	25	23
3	70	126.1	11.3	231	126	2.58	171	15	15

Test Results	Material Description
R-value at 300 psi exudation pressure = 23	

Project No.: 1P2/305/13

Project: Green Valley Road at Tennessee Creek Bridge Reconstruction Project

Location: Boring 4

Sample Number: Bag D **Depth:** 0.5'-5.0'

Date: 12/17/2008

Tested by: RJF

Checked by: MWM

Remarks:

R-VALUE TEST REPORT

Taber Consultants

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/28/2006
Date Submitted 04/25/2006

To: Ralph Fisher
Taber Consultants
3911 West Capital Avenue
W. Sacramento, CA 95691-2116

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 1P2/305/13 Site ID : 1/3.
Thank you for your business.

* For future reference to this analysis please use SUN # 47321-93747.

EVALUATION FOR SOIL CORROSION

Soil pH	7.53		
Minimum Resistivity	2.57	ohm-cm (x1000)	
Chloride	14.3 ppm	00.00143	%
Sulfate	4.8 ppm	00.00048	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/28/2006
Date Submitted 04/25/2006

To: Ralph Fisher
Taber Consultants
3911 West Capital Avenue
W. Sacramento, CA 95691-2116

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 1P2/305/13 Site ID : 1/C.
Thank you for your business.

* For future reference to this analysis please use SUN # 47321-93748.

EVALUATION FOR SOIL CORROSION

Soil pH	8.80		
Minimum Resistivity	2.09	ohm-cm (x1000)	
Chloride	24.2 ppm	00.00242	%
Sulfate	3.3 ppm	00.00033	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/28/2006
Date Submitted 04/25/2006

To: Ralph Fisher
Taber Consultants
3911 West Capital Avenue
W. Sacramento, CA 95691-2116

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 1P2/305/13 Site ID : 1/E.
Thank you for your business.

* For future reference to this analysis please use SUN # 47321-93749.

EVALUATION FOR SOIL CORROSION

Soil pH	9.02		
Minimum Resistivity	1.66	ohm-cm (x1000)	
Chloride	4.5 ppm		00.00045 %
Sulfate	3.7 ppm		00.00037 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/28/2006
Date Submitted 04/25/2006

To: Ralph Fisher
Taber Consultants
3911 West Capital Avenue
W. Sacramento, CA 95691-2116

From: Gene Oliphant, Ph.D. \ Randy Horney RH
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 1P2/305/13 Site ID : 5/A.
Thank you for your business.

* For future reference to this analysis please use SUN # 47321-93750.

EVALUATION FOR SOIL CORROSION

Soil pH	7.09		
Minimum Resistivity	3.22	ohm-cm (x1000)	
Chloride	7.0 ppm	00.00070	%
Sulfate	3.3 ppm	00.00033	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

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ENVIRONMENTAL TESTING & CONSULTING

Login #: 26033

Taber Consultants
3911 W. Capitol Avenue
W. Sacramento. CA 95691
Phone # (916) 371-1690
Fax #: (916) 371-7265

Attention: Ralph Fisher
Email:

Job Site:
Vacant Site
Green Valley Road
El Dorado Hills, CA

Date Samples Taken: 4/25/2006
Date Report Submitted: 4/27/2006
NAL ID # / Lot #: 3911 / 2
Lab Tracking #: 1505
Total Samples: 8

POLARIZED LIGHT MICROSCOPY ANALYTICAL REPORT (PLM)

Samples ID #	Asbestos Fibers		Non-Asbestos Fibers		Non-Fibrous Materials	
	%	Type	%	Type	%	Type
Sample ID #: B-1/Run C **NAL ID:** -2-1 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-1/Run E **NAL ID:** -2-2 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-1/Run H **NAL ID:** -2-3 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: 3/C **NAL ID:** -2-4 **Location:** Ground **Material:**	Trace	<1% Chrysotile	0	None Detected	100	Misc. Particles
Sample ID #: 4/D **NAL ID:** -2-5 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-5/RunA **NAL ID:** -2-6 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-5/Run D **NAL ID:** -2-7 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-5/Run G **NAL ID:** -2-8 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles

Comments: Results relate only to the items analyzed. For all obviously inhomogeneous samples easily separated into sub samples, and for layered samples, each component is analyzed separately. The Analylitical Uncertainty is plus or minus 1 percent

Key: Detection Limit = 1%
Trace Amount <1%
None Detected = 0%

Reviewed By: Draper

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National Analytical Laboratories, Inc. 10416 Investment Circle Ste. A, Rancho Cordova, CA 95670
Phone (916) 361-0555 Fax (916) 361-0540 Website www.NAL1.com

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ENVIRONMENTAL TESTING & CONSULTING

Login #: 26033

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W. Sacramento, CA 95691
Phone # (916) 371-1690
Fax #: (916) 371-7265

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Sample ID #: 3/C **NAL ID:** -2-4 **Location:** Ground **Material:**	Trace	<1% Chrysotile	0	None Detected	100	Misc. Particles
Sample ID #: 4/D **NAL ID:** -2-5 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-5/RunA **NAL ID:** -2-6 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-5/Run D **NAL ID:** -2-7 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-5/Run G **NAL ID:** -2-8 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles

Comments: Results relate only to the items analyzed. For all obviously inhomogeneous samples easily separated into sub samples, and for layered samples, each component is analyzed separately. The Analylitical Uncertainty is plus or minus 1 percent	**Key:** Detection Limit = 1% Trace Amount <1% None Detected = 0%

Reviewed By: DRaper

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Attention: Ralph Fisher
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Sample ID #: B-1/Run E **NAL ID:** -2-2 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
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Sample ID #: 3/C **NAL ID:** -2-4 **Location:** Ground **Material:**	Trace	<1% Chrysotile	0	None Detected	100	Misc. Particles
Sample ID #: 4/D **NAL ID:** -2-5 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-5/RunA **NAL ID:** -2-6 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
Sample ID #: B-5/Run D **NAL ID:** -2-7 **Location:** Ground **Material:**	0	None Detected	0	None Detected	100	Misc. Particles
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Key: Detection Limit = 1%
Trace Amount <1%
None Detected = 0%

Reviewed By:

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TABER
SINCE 1954

July 9, 1991

Bissell & Karn, Inc.
2200 "B" Douglas Boulevard, Suite 100
Roseville, California 95661-3800

Attention: Chris Rockway

Subject: Foundation Investigation
Dry Creek Bridge at Green Valley Road
El Dorado County, California

1P2/389/81

Gentlemen:

Transmitted herewith are five copies of our report of foundation investigation performed at the above site. The original transparency of the "Log of Test Borings" drawing is being forwarded under separate cover. Results of roadway subgrade evaluation will be submitted separately.

Should any questions arise concerning foundation conditions at the site, please do not hesitate to call on us. An opportunity to review and comment on plans and specifications insofar as they rely on this report is an integral part of our recommendations.

We appreciate this opportunity to be of service.

Very truly yours,

TABER CONSULTANTS

Franklin P. Taber
R.C.E. 30920
G.E. 816

FPT/ns

Accompanying: Reports (5)

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FOUNDATION INVESTIGATION
Dry Creek Bridge
at
Green Valley Road

El Dorado County
Owner

Bissell and Karn, Inc.
Design Engineer

1P2/389/81

July, 1991

TABER
SINCE 1954

FOUNDATION INVESTIGATION

Dry Creek Bridge at
Green Valley Road

1P2/389/81

Introduction

A limited study of foundation conditions has been completed at the above site in accordance with the agreement between Bissell and Karn, Inc. and Taber Consultants. The purpose of investigation is to provide earth materials criteria for design of proposed new bridge foundations. Limitations of study are discussed in the attached "General Conditions".

Site and Project Description

The existing bridge is about 50-ft long and 22-ft wide, consisting of two equal concrete spans supported by concrete wall abutments and a three-column bent. The structure has been widened on the west by 6-8±ft; widening appears to have included construction of replacement (or new?) bent columns supported on isolated concrete pads. The existing deck is at elev. 1059±, about 12±ft above low channel grade at the bridge, approach roadways are on 5-10±ft of fill.

It is proposed to replace the existing structure with a new bridge along a new alignment located westerly (downstream) from the existing structure. New centerline is about 30-ft west of the existing bridge at the south abutment and about 55-ft at the north. Roadway construction will include approximately 2200 lf of new and reconstructed roadway. Maximum cut of 6±ft (at centerline) and maximum fill to 12±ft (at bridge approach embankments) are proposed with cut and fill slopes constructed at 2:1 (horizontal:vertical distance).

The new bridge is expected to be 50±ft wide and 80±ft long of single-span cast-in-place pre-stressed concrete box-girder construction supported on skewed diaphragm abutments. New deck grade is indicated to be at elev. 1062±; base of abutment diaphragms is understood to be at about elev. 1051.9 at the south abutment and at from elev. 1050.5 to 1053.4 at the north.

An existing El Dorado County Irrigation District 20-inch diameter pipeline crosses Dry Creek at the proposed bridge-site. Based on "as-built" plans for the pipeline and a pot-hole survey by the County, the pipeline invert is at elev. 1040± in the channel and as much as 2-3±ft higher at and immediately behind proposed abutments. The pipeline centerline is located about 15-ft inside the southwest bridge corner and 3±ft inside the northeast corner; excavation/backfill conditions are not known.

It has been reported (by the current owner of the southern bank parcel) that Dry Creek originally flowed just south of its present course. A former owner apparently shifted the channel northward to its present location to accommodate a small reservoir that was impounded behind the existing check dam just downstream from the proposed bridge-site. The former channel was filled -- presumably with loose "non-engineered" fill which forms the existing south bank.

Exploration and Testing

Information on the nature and distribution of subsurface materials and conditions at the proposed bridge-site was obtained by means of two auger/rotary sampled test borings to maximum depth 35±ft (elev. 1018±ft) supplemented by hand probe tests made in the channel to investigate bedload materials, depth to bedrock and stream scour depth. Additional exploration included two short augered sampled borings along proposed roadway alignment in evaluation of approach roadway subgrade conditions (reported separately).

"Undisturbed" soil/rock samples were recovered from the sampled borings by means of a 2.0 inch OD "standard penetration" sampler advanced with standard 350 ft-lb striking force (ASTM D1557). Penetration resistance was recorded and can be correlated to soils strength and bearing characteristics.

The borings were logged and earth materials field-classified by a geologist as to consistency, color, texture, gradation, etc. on the bases of penetration resistance, examination of samples and observation of drill cuttings. Groundwater observations were made in the borings during drilling operations and after completion. Borings were backfilled with drill cuttings at completion of field study.

Selected portions of recovered samples were retained in moisture-proof containers for laboratory testing and reference. Moisture content-dry density and unconfined compressive strength determinations were performed on selected suitable samples in the laboratory to supplement field evaluation of earth materials parameters.

Boring locations were referenced to existing site features as shown on site topography prepared by Bissell and Karn, Inc.(scale 1"=50'; dated April, 1990); elevations to Bissell and Karn study datum. Locations, details of borings and results of tests are shown on accompanying "Log of Test Borings" drawing. R.T. Siegfried was field geologist for this project.

Earth Materials and Foundation Conditions

Exposures in the channel and boring encounters indicate the site to be underlain by highly to moderately weathered or nearly fresh meta-volcanic rock. Overlying soils include sandy gravelly colluvium/alluvium and/or old fill on the banks with a thin veneer of cobbly bedload debris in the creek channel. At the proposed bridge, the upper surface of rock was encountered at elevation 1043±ft on the southern bank (Boring-1), and at elev. 1047±ft at the northern bank (Boring-3) and, in the channel, is present at very shallow depth or exposed (elev. 1046±). The south bank encounter may represent the former stream thread and significant lateral variation in rock surface elevation is not unlikely.

The rock encountered in Boring-1 is "very dense" (per soils classification) and highly weathered meta-volcanic rock. The rock encountered in Boring-3 is "very dense" moderately weathered "greenstone". Rock appears to be relatively fresh below elev. 1031± in the south bank and elev. 1036± in the north. Both weathered and fresher rock is considered capable of developing support for heavy, concentrated foundation loads. The rock is expected to be typically erosion resistant where exposed in the channel, although locally more weathered zones may be erodible.

Overburden soils were penetrated to a maximum 10±ft depth (in the south bank) and consist of loose to dense clayey silt and sand with rock fragments (north bank) or semicompact to compact silty sandy gravel with some clay and cobbles (south). Overburden materials include loose fill from prior channel re-alignment and are considered potentially erodible if exposed to surface flow. They are considered capable of supporting anticipated embankment loads -- although likely to experience some settlement, particularly within the old fill -- but are not considered suitable for structure support. Channel bedload material observed at the proposed bridge-site is very thin, not considered suitable for structure support, and is subject to erosion/scour.

The free water level was measured at elevation 1048±ft in the boring on the southern bank (Boring-1), approximately 17 hours after completion of drilling. The channel water surface was at about elev. 1047± at the time of our field study (March 1991). The rock materials are generally relatively impermeable, although significant seepage may occur along local discontinuities, fractures and/or highly weathered zones. Seasonal infiltration and (downslope) seepage in soil overlying rock is not unexpected, particularly from the southern bank.

Site Seismic Conditions

In accordance with current Caltrans Office of Structure Design site seismicity evaluation procedures including Map Sheet-45, "maximum credible rock acceleration" 0.60g is assigned the site, associated with an event of 6.5 magnitude of the Foothills fault zone located less than ½ mile westerly. From on-site boring data obtained for this study, depth to "rock-like" material is 10-ft or less. Should there be important structural and/or economic considerations associated with more closely defining this value or other site-seismicity characteristics, further study would be required.

Conclusions-Recommendations

Suitable foundation support is available by means of spread footing and/or cast-in-drilled-hole pile foundations bearing in rock; footing support at one abutment is considered compatible with pile support at the other. Ground and water control conditions will be significant considerations in construction of such foundations. The presence of the pipeline may control required foundation penetration at some locations. Achieving adequate penetration for stability with driven piling is unlikely, owing to the limited clearance between the base of abutment diaphragms and rock surface and unlikelihood of significant pile penetration into rock.

At the indicated levels, the existing water pipeline at the bridge site appears to be founded within weathered rock; as such, it is not expected to be subject to more than nominal settlement from incremental fill embankment loading. With structure foundations per below, additional loading applied to the pipeline will be limited to that resulting from superposed fill. The nature and condition of pipeline trench backfill may affect foundation construction.

Based on boring encounters, the existing overburden materials are suitable for proposed embankment placement. With nominal fill foundation preparation -- including stripping of topsoil and scarification and compaction of the exposed surface -- embankment settlement is expected to be limited and to occur as fill is placed. No specific "waiting period" is required prior to pile installation. Typical toe of Rock Slope Protection (per "Standard Plans") is expected to engage relatively firm, secure materials.

Continuous reinforced concrete footings at least 3-ft wide and with at least 1-ft penetration into rock as approved by this office may be assigned allowable (service load) bearing pressures to 5 tsf -- net at ground line. The bottom of footings should be below a 1½:1 (horizontal:vertical distance) plane projected upward from the invert of the pipeline. Based on boring encounters, the highest plan footing elevations meeting these criteria are elev. 1042 at the south abutment and elev. 1046 at the north. Base of footing elevations as low as elev. 1038 may be required for footings immediately-adjacent the pipeline. Isolated footings (drilled piers designed as footings) at least 3-ft wide and penetrating 3-5 ft into rock could be assigned allowable bearing pressures to 10-15 tsf.

Footings should be poured neat, without forming against trimmed, undisturbed bearing materials in clean and dry excavations. De-watering of excavations is expected to be achievable by means of diking/diversion of surface water and sump pumping. Foundation excavations should be made <u>without</u> blasting. This office should be consulted regarding identification and penetration of bearing materials. If local weak zones are encountered, they should be excavated and backfilled to plan footing grade with plain concrete.

Alternatively, standard (Caltrans) 16-inch diameter cast-in-drilled-hole piling <u>could</u> be installed and may be assigned design (service) loads to 70 tons per pile. They should be specified to penetrate to elev. 1023 at the south abutment and to elev. 1027 at the north and should penetrate at least 12-ft into intact rock bearing materials as affirmed by this office. The specified tip elevations should be 4-ft lower for such piles within 6-ft of the existing pipeline; a minimum clearance of 2-ft should be maintained between piling and the pipeline.

This office should be consulted during construction regarding questions of bearing materials identification and penetration. The use of casing and pumping is expected to be required for ground and water control -- particularly within the overburden material. The contractor should be prepared to encounter local bodies of hard, resistant rock within the typically weathered rock mass.

* * * * * *

The bridge foundations recommended above are considered appropriate for the assignment of typical service-load combinations. For "load factor" design, "ultimate" (downward) bearing capacities are expected to be much greater than those derived from typical Caltrans design procedures and uplift resistance of cast-in-drilled-hole piling may be much greater.

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EXP. 3-31-92
GEOTECHNICAL
STATE OF CALIFORNIA

July 9, 1991

Attachments: "General Conditions"
"Log of Test Borings"

GENERAL CONDITIONS

The conclusions and recommendations of this study are professional opinion based upon the indicated project criteria and the limited data described herein. It is recognized there is potential for sufficient variation in subsurface conditions that modification of conclusions and recommendations might be emergent from further, more detailed study.

This report is intended only for the purpose, site location and project description indicated and assumes design and construction in accordance with Caltrans practice.

As changes in appropriate standards, site conditions and technical knowledge cannot be adequately predicted, review of recommendations by this office for use after a period of two years is a condition of this report.

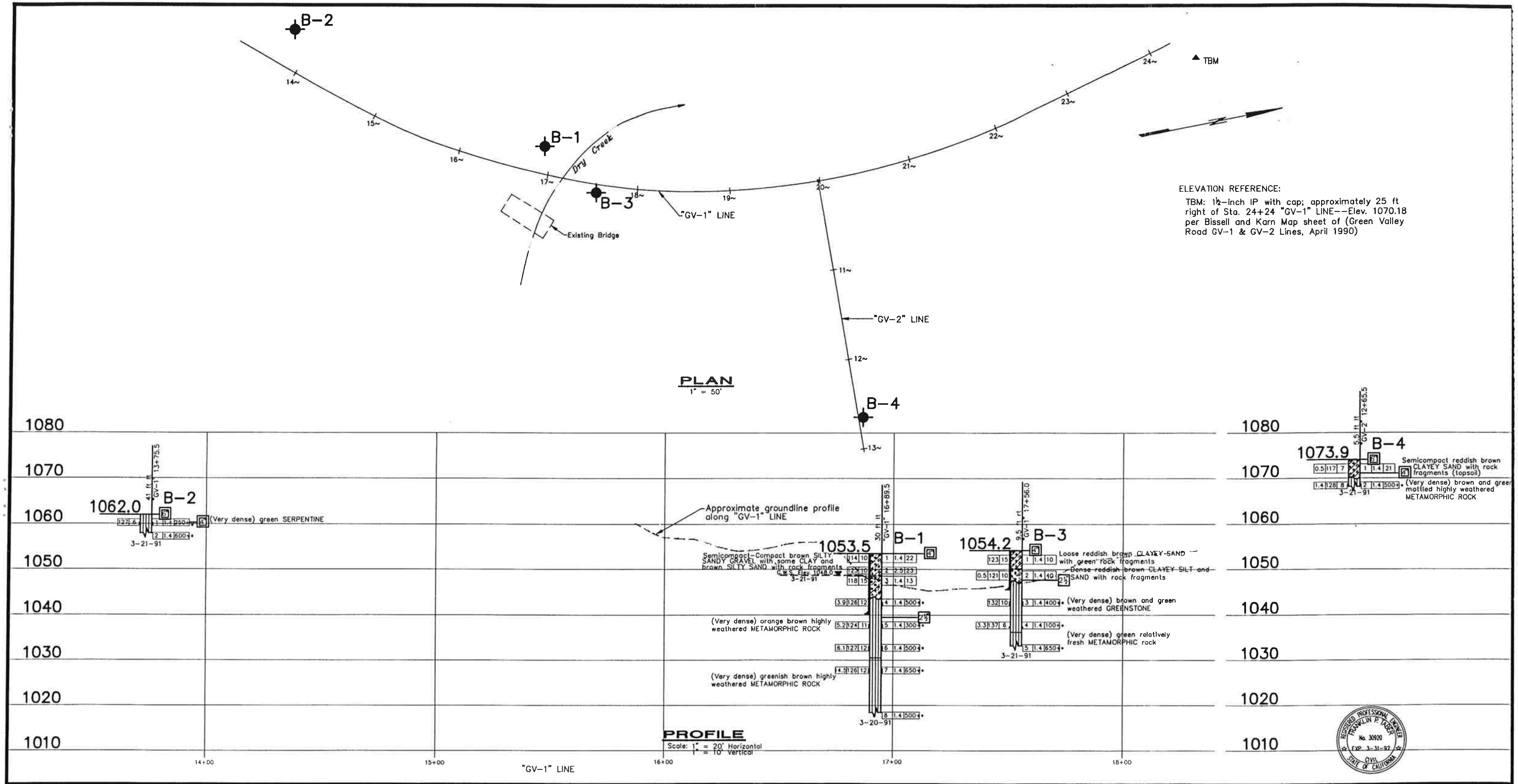
A review by this office of any foundation and/or grading plans and specifications or other work product insofar as they rely upon or implement the content of this report, together with the opportunity to make supplemental recommendations as indicated therefrom is considered an integral part of this study and a condition of recommendations.

Subsequently defined construction observation procedures and/or agencies are an element of work which may affect supplementary recommendations.

Should there be significant change in the project or should soils conditions different from those described in this report be encountered during construction, this office should be notified for evaluation and supplemental recommendations as necessary or appropriate.

Opinions and recommendations apply to current site conditions and those reasonably foreseeable for the described development -- which includes appropriate operation and maintenance thereof. They cannot apply to site changes occurring, made, or induced, of which this office is not aware and has not had opportunity to evaluate.

The scope of this study specifically excluded sampling and/or testing for, or evaluation of the occurrence and distribution of, hazardous substances. No opinion is intended regarding the presence or distribution of any hazardous substances at this or nearby sites.



PLAN
1" = 50'

PROFILE
Scale: 1" = 20' Horizontal
1" = 10' Vertical

ELEVATION REFERENCE:
TBM: 1½-inch IP with cap; approximately 25 ft right of Sta. 24+24 "GV-1" LINE--Elev. 1070.18 per Bissell and Karn Map sheet of (Green Valley Road GV-1 & GV-2 Lines, April 1990)

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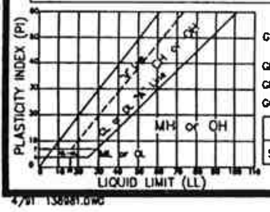
CIVIL

LEGEND OF EARTH MATERIALS

UNIFIED SOIL CLASSIFICATION SUMMARY

Pt	OH	CH	MH	OL	CL	ML	SC	SM	SP	SW	GC	GM	GP	GW
Highly organic soils	Silts and clays Liquid limit 50 or more			Silts and clays Liquid limit less than 50			Sands with fines >12% fines		Clean sands < 5% fines		Gravels with fines > 12% fines		Clean gravels < 5% fines	
							Sands—50% or more of coarse fraction is smaller than No. 4 Sieve				Gravels—more than 50% of coarse fraction is larger than No. 4 sieve			
Fine grained soils (50% or more is smaller than No. 200 sieve)							Coarse grained soils (More than 50% is larger than No. 200 sieve)							



LABORATORY CLASSIFICATION CRITERIA

GW and SW—$Cu=\frac{D_{60}}{D_{10}}$ greater than 4 for GW & 6 for SW; $Cc=\frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1&3.

GP and SP—Clean gravel or sand not meeting requirements for GW and SW.

GM and SM—Atterberg limits below "A" line or P.I. less than 4.

GC and SC—Atterberg limits above "A" line with P.I. greater than 7.

Fines (silt or clay)	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		
Sieve sizes	200	40	10	4	3/4"	3"	10"

Classification of earth materials shown on test boring logs is based on field inspection and should not be construed to imply laboratory analysis unless so stated.

MATERIAL SYMBOLS

▨	Gravel	▨	Peat or organic matter
▨	Sand	▨	Fill material
▨	Silt	▨	Shale
▨	Clay	▨	Sandstone
▨	Sandy clay or clayey sand	▨	Limestone
▨	Sandy silt or silty sand	▨	Metamorphic rock
▨	Silty clay or clayey silt	▨	Igneous rock

CONSISTENCY CLASSIFICATION FOR SOILS

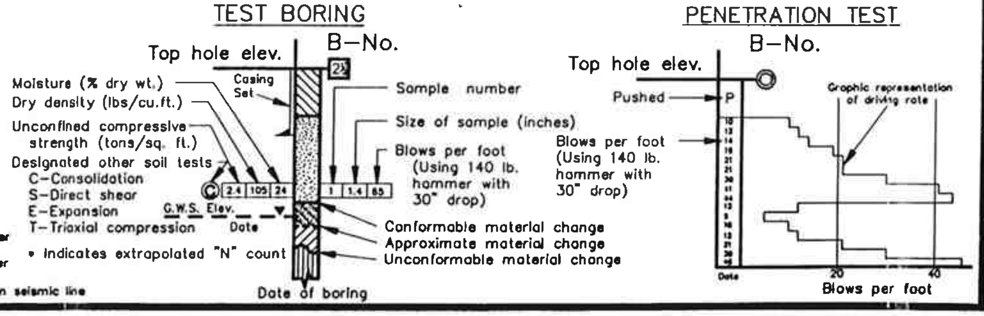
According to the Standard Penetration Test

Nº of blows	Granular	Cohesive
0–5	Very loose	Very soft
6–10	Loose	Soft
11–20	Semicompact	Stiff
21–35	Compact	Very stiff
36–70	Dense	Hard
> 70	Very dense	Very hard

Where standard penetration test has not been performed consistencies shown on logs are estimated.

LEGEND OF BORING OPERATIONS

- ● Plan of any boring
- ▢ Rotary boring
- ◇ Diamond core boring
- ▢ Auger boring
- ▯ Sample boring
- ▯ Jet boring
- ▢ Test pit
- ◉ 2¼" Cone penetrometer
- ○ 2½" Cone penetrometer
- S-1 → ← S-2 Refraction seismic line



* Indicates extrapolated "N" count

TABER CONSULTANTS Engineers & Geologists

SINCE 1954

APPROVED 7-9-91 JOB No. 1P2/389/81
License No. RCE 30920, GE 816

Bissell & Karn
Green Valley Road Bridge
Across
Dry Creek
LOG OF TEST BORINGS

Scale: As Shown	Date: April 1991	By: M.D.R. Check By: R.T.S.	Drawing:

4/91 138981.DWG

copy

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SINCE 1954

TABER CONSULTANTS • Engineers & Geologists
536 Galveston Street • West Sacramento, California 95691
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July 9, 1991

Bissell & Karn, Inc.
2200 "B" Douglas Boulevard, Suite 100
Roseville, California 95661-3800

Attention: Chris Rockway

Subject: Approach Roadway Subgrade Evaluation
Green Valley Road at Dry Creek
El Dorado County, California

1P2/389/81

Gentlemen:

In accordance with our agreement, two sampled test borings were completed in the proposed roadway approaches to the proposed bridge structure. The logs of these borings and the results of field and other tests are shown on the "Log of Test Borings" drawing forwarded with our report of bridge foundation investigation. The results of two Stabilometer Resistance value tests (CTM 301) performed on bulk samples of anticipated subgrade soils are attached.

Approximately 2200 feet of new and re-constructed approach roadway is proposed for Lotus and Green Valley Roads in association with the new bridge construction. Maximum cut of 6±ft (at centerline) and maximum fill to 12±ft at bridge approach embankments are proposed with cut and fill slopes constructed at 2:1 (horizontal:vertical distance).

Soils encountered in the borings typically consist of very shallow (less than 0.5-ft) topsoil and very dense rock --Boring-2-- and semicompact clayey sand with rock fragments (topsoil) -- Boring-4. No free groundwater was encountered within the depth of drilling. Bag-A was collected from Boring-2 in natural ground located approximately 300-ft southwest of the proposed southern bridge abutment; Bag-B was collected from Boring-4 in natural ground within the existing triangular area at the intersection of Lotus and Green Valley Roads. A full study of materials for pavement section recommendations was not authorized or performed.

Test results from Bag-A indicate R-value = 51; results from Bag-B indicate R-value = 61. R = 51 is considered appropriate for use in pavement structural section design with native soils at subgrade level based on assumption these data are typical. Minimum thicknesses of asphalt and base may apply at low Traffic Indexes.

Subgrade per Caltrans "Standard Specifications" requires at least 95% relative compaction or equivalent quality on materials to 30-inches below finished pavement grade. Excavation and re-compaction of native soils and/or rock below the base/subbase subgrade does not appear to be generally necessary along this alignment, but is recommended for existing embankment fill. No requirement for subdrainage of subgrade is indicated by boring data.

Please call if you have any questions on the above or as we may by of further assistance. We appreciate this opportunity to be of service.

Very truly yours,

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STATE OF CALIFORNIA

FPT/ns

Attachment: "Resistance Value Tests"

Distribution: Client (5)

RESISTANCE VALUE TESTS
(CTM 301)

Specimen No.	Dry Density (PCF)	Moisture (%)	Exudation Pressure (PSI)	Expansion Pressure (PSF)	"R" Value
Sample: Boring-2/Bag-A - 0-1 ft; Brown and pale green clayey silty sand					
1	121.9	12.7	183	30	40
2	123.7	11.3	303	56	52
3	123.8	10.3	637	247	63

"R" Value by Stabilometer (300 psi exudation) = 51

Specimen No.	Dry Density (PCF)	Moisture (%)	Exudation Pressure (PSI)	Expansion Pressure (PSF)	"R" Value
Sample: Boring-4/Bag-B - 0-1 ft; Reddish-brown clayey sand with rock fragments					
1	130.7	9.4	143	9	32
2	132.1	8.4	295	43	60
3	133.0	7.4	637	147	75

"R" Value by Stabilometer (300 psi exudation) = 61