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FOUNDATION INVESTIGATION

Missouri Flat Road Overcrossing (Replace)
Bridge No. 25-0121
EA 03-370001
Phase 1
U.S. Route 50/Missouri Flat Road Interchange Project
El Dorado County, California
03-ED-50-23.2/25.4

El Dorado County
Lead Agency

Quincy Engineering, Inc.
Design Engineer

1P2/399/296-1.2M
38120-F7:314N:252W

March 2006

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Introduction

A limited study of foundation conditions has been completed at the above site in accordance with the agreement between Quincy Engineering, Inc. and Taber Consultants. The purpose of this investigation is to provide earth materials criteria for use in design of proposed new bridge foundations. This study specifically addresses Phase 1 project elements for the proposed overcrossing replacement. Additional field study and supplemental geotechnical recommendations may be required for future Phase 2 design.

Earth materials criteria for design of other Phase 1 project elements are to be addressed in separate reports prepared by this office. Limitations of study are discussed below and in the attached "General Conditions."

This project is the subject of our "Geologic/Geotechnical Review" (dated March 26, 2001), letter of "Memorandum – Site Seismic Conditions" (dated January 19, 2001) and "Addendum No. 1 to Geologic/Geotechnical Review" (dated July 24, 2001). Reference is also made to a Caltrans memorandum (dated January 29, 2001) discussing site seismic conditions specific to this project. Information from these documents is incorporated herein, as appropriate.

This report supercedes our draft foundation investigation report dated August 11, 2005 and has been modified to incorporate review comments from Caltrans Division of Engineering Services, Geotechnical Services, Office of Geotechnical Design – North as outlined in a letter dated September 26, 2005. A copy of the Caltrans review comments and our response are included as Appendix-E.

Site and Project Description

The existing Missouri Flat Road overcrossing (Br. No. 25-0077) is located in El Dorado County, California (see Figure-1). It was constructed in 1969 and is a three-span continuous reinforced concrete box girder structure of length 76±m (248 ft) and width 12±m (40 ft). Substructure consists of open-style end-diaphragm type abutments and two-column bents. The southerly abutment and bents are supported on spread footings, each established within bedrock. The northerly abutment is supported by 400 mm diameter cast-in-drilled-hole piles penetrating into bedrock.

At this location existing U.S. Route 50 is established in a cut section varying from 1.5±m to 4.0±m depth below original ground surface. In the vicinity of the Missouri Flat Road overcrossing, existing U.S. Route 50 grade slopes about 1.5-2.0% from west to east. With respect to the existing overcrossing, eastbound traffic is presently carried below the southerly span; westbound traffic below the center span; and westbound off-ramp traffic below the northerly span.

The proposed Missouri Flat Road overcrossing is shown on preliminary "General Plan" drawing for Phase 1 (dated June 21, 2005) prepared by Quincy Engineering, Incorporated. The currently proposed structure is shown to be 55.92 m long by 32.41 m wide, consisting of two cast-in-place prestressed concrete box girder spans (28.555 m on the north and 27.365 m on the south) between "MFRD" Sta. 13+02.093 (Begin Bridge) and "MFRD" Sta. 13+58.013 (End Bridge). New deck grade is shown on a vertical curve passing through elev. 539.479 at Abutment-1 (north) and elev. 539.679 at Abutment-3 (south).

Substructure is shown to be wall abutments to 8.5-9.4±m high and a five-column bent with spread footings. All supports are skewed 3 degrees to match existing US 50 alignment. Plans show a structure approach slab behind each abutment. Proposed Abutment-1 will be located about 13.2-13.8 m in front of existing; proposed Abutment-3 will be about 1.3-1.9 m in front of existing. At proposed Bent-2, the two most easterly column footings are shown to be partly within the footprint of existing spread footings. Base of all spread footings are tentatively shown 1.8±m below road grade (i.e., below elev. 532±).

Construction will consist of two stages to allow traffic to be maintained on the existing overcrossing. Stage 1 will consist of a 16.2±m wide bridge left (easterly) of the "MFRD" Station Line, with Stage 2 involving removal of the original structure and construction of an additional 16.2±m wide bridge with 1.5±m closure pour at completion of Stage 2.

The approach embankments are shown to be about 6-8 m high with 1v:2h side-slopes. Retaining walls are also shown on the referenced plans located behind and on each side of abutments. They are expected to be Standard (Caltrans) Type-1 retaining walls, 12-14±m long and varying in height from 8 m to 10 m.

Future Phase 2 construction will expand the interchange to a Single Point Diamond Interchange (SPDI). This will include widening the Phase 1 structure on both sides for a total structure width to as much as 88±m.

Pertinent Structure/Site Information

Review of available structure/site information published by the State of California Bridge Department (Caltrans) pertinent to this project included the following:

- Foundation Investigation Report – Missouri Flat Road OC, dated March 23, 1965
- Foundation Review Memo – Missouri Flat Road OC, dated September 24, 1965
- As-Built "Log of Test Borings", Missouri Flat Road OC (Br. No. 25-77), dated January 29, 1969
- Post-construction "Foundation Report – Shingle Springs to Webber Creek", dated April 1969

The foundation report (March 1965) specified 250BP62 (10BP42) steel piles with design loads to 400 kN (45-tons) per pile at the abutments and spread footing support at the bents. Estimated pile tips were to elev. 532.80 m (elev. 1748 ft) at Abutment-1 and to elev. 529.75 m (elev. 1738 ft) at Abutment-4. At Bent-2 and Bent-3, spread footings with "design" bearing pressure specified to 383.1 kN/m^2 (4 tsf) were recommended to be established in bedrock at or below elev. 531.88 m (elev. 1745 ft) and elev. 530.97 m (elev. 1742 ft), respectively.

The foundation memo (September 1965) indicates that use of spread footings at each abutment with "design" bearing pressure of 239.4 kN/m^2 (2.5 tsf) was approved. Plan base of footing level was specified at elev. 533.71 m (elev. 1751 ft) at Abutment-1 and elev. 534.6 m (elev. 1754 ft) at Abutment-4.

The post-construction foundation letter (April 1969) and As-Built "Log of Test Borings" drawing by the State Bridge Department indicate the following:

- Abutment-1 (south); base of spread footing is shown at elev. 533.40 m (elev. 1750 ft).
- Bent-2 (south); base of spread footing is shown at elev. 530.21 m (elev. 1739.5 ft).
- Bent-3 (north); base of right footing was lowered 0.6-1.0 m (2-3 ft) to elev. 529.74 m (elev. 1738 ft) as a result of "over-blasting" the rock; base of left spread footing is shown at 530.36 m (elev. 1740 ft).
- Abutment-4 (north); material at planned base of footing was found to be unsatisfactory, and six, 16-inch (400 mm) diameter cast-in-drilled-hole piles with design loads to 625 kN (70 tons) per pile were drilled 0.6 m (2 ft) into "sound rock" with average pile tip shown at elev. 530.97 m (elev. 1742 ft).
- Some water was encountered in the footing excavations and successfully de-watered with a pump prior to casting the footings.

The as-built "Log of Test Borings" drawing attached to this report as "Log of Test Borings 2 of 2" shows added "MFRD" Line stationing for the current project. The locations of 1965 test borings are also shown on the "Log of Test Borings 1 of 2" prepared for this project (2005 test borings).

Exploration and Testing

State Bridge Department Study

Bridge foundation exploration performed by the State in 1965 consisted of three 57 mm (2¼-inch) cone penetration borings penetrating to lowest elev. 530.85±m (elev. 1742±). These borings were driven to effective refusal using a small compressed-air sheet-pile hammer.

Taber Study

Exploration to investigate the nature and distribution of earth materials and conditions for the proposed bridge included three drilled, sampled and logged test borings to a maximum depth of 9.8±m (lowest elev. 522±) supplemented by a short auger-identification boring to 1±m depth at proposed Abutment-3.

The borings were advanced by auger drilling through surficial unconsolidated soil and decomposed to very intensely weathered portions of the rock. Diamond-coring equipment was required to advance the borings through underlying, less weathered rock and to recover rock core for logging.

Drive samples of unconsolidated soil and decomposed to very intensely weathered rock were recovered from the borings by means of a 50 mm OD "standard penetration" sampler advanced with standard striking force (63.5 kg weight with 760 mm drop per ASTM D1586) to provide a field estimate of soils consistency. Sampler penetration resistance was recorded and, to some extent, can be correlated to strength and bearing characteristics of the foundation materials.

Portions of earth materials recovered with the drive sampler were retained in moisture-proof containers for laboratory testing and reference. Bulk samples were also obtained from auger drill cuttings. Rock cores were retained in core boxes for laboratory testing and reference and are available for inspection.

Borings were logged and earth materials field-classified by an engineer as to consistency, color, gradation and texture on the bases of sampler penetration resistance, and examination of samples, rock cores and drill cuttings. Subsequent to field investigation, rock cores were reviewed in the office by engineering geologists. Where diamond coring was used to advance the borings, the recovered cores were logged as to percent recovery, Rock Quality Designation (RQD[1]) degree of weathering, hardness and fracture density (see Drawing-1, "Engineering Geology Field Descriptors").

Laboratory tests performed on samples of both soil and decomposed rock materials to supplement field evaluation included moisture content-dry density tests. Testing on selected rock core was limited to Point-Load Index tests (utilizing a Soiltest Model RM-735 testing apparatus) in evaluation of the range of rock compressive strength. Laboratory testing on a bulk sample consisted of soils corrosivity screening (pH and minimum resistivity per CTM 643 – modified small cell, Sulfate per CTM 417 and Chloride per CTM 422). Results of laboratory testing are included in Appendix-A.

Groundwater observations were made in the borings during drilling operations. Borings 05-3, 05-4 and 05-5 were backfilled with cement-grout upon completion of drilling. Boring 05-27 was backfilled with drill cuttings.

The boring locations were referenced to project stationing as shown on the above referenced plans; elevations were referenced to project datum provided by Topographic Surveys, Incorporated. Locations, elevations, details of borings and results of tests are shown on the attached "Log of Test Borings 1 of 2" drawing and Appendix-A. Ron Loutzenhiser was field engineer for this study. Site reconnaissance and office review of rock cores was made by Martin McIlroy and Eric Nichols, both Certified Engineering Geologists.

Geologic Setting

The project site is located within the foothills of the Sierra Nevada geomorphic province of California. The Sierra Nevada has a general northwest topographic trend and is on the order of 690 km long and 64-129 km wide. The mountain ranges of the

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[1] RQD is the ratio of the total length of recovered core in pieces longer than 100 mm to the total length of boring cored, expressed as a percentage.

Sierra Nevada were created roughly 120 to 130 million years ago when sediments as thick as 9,200 m along with volcanic rocks were buckled and warped resulting in a series of low mountain ranges. The roots of these mountain ranges were then intruded by granitic rock.

The Sierra Nevada was tilted upward as a result of faulting along the east edge of the ranges. In the higher elevations of the Sierra Nevada, much of the sedimentary material has been eroded to extensively expose the granitic rock. Older rocks that remain have been metamorphosed and are exposed in the foothills of the Sierra Nevada.

Published geologic mapping (reference 4) shows surface materials within the project limits as Mesozoic granitic rock. Slate and metasedimentary rock of the Mariposa Formation and metavolcanic rock of the Logtown Ridge Formation are also shown nearby to the south and northeast of the project site.

Site reconnaissance made within project limits by personnel from this office indicates metamorphic rock exposed locally at/near each abutment and within a cut-slope along the north side of US 50 a few hundred meters east of the Missouri Flat Road Interchange. The rock is non-foliated with fine to medium grains contained within an aphanitic (i.e., grain size < 0.1 mm) matrix. Surface exposures are typically very intensely to moderately weathered.

Rock encountered in borings completed for the Weber Creek Bridge (located approximately 610 m east of the Missouri Flat Interchange; see Figure-1) is field-described similarly to rock encountered in borings completed for this project element (discussed below). Petrographic examination of two selected rock core samples from the Weber Creek site was made by personnel from Micro-Chem Laboratories (see Appendix-B). Based on petrographic examination, the two rock samples are generally classified as Hornfels – a non-foliated metamorphic rock typically formed by contact metamorphism.

At Missouri Flat Road, the existing cut-slope east of the interchange is at approximate 1:1 and has performed generally well with only minor sloughing in the more-weathered portions of the rock. The rock outcrop at existing Abutment-4 (northerly abutment) is randomly fractured with at least two prominent vertical joint/fracture sets with one set striking northwesterly and the other set striking northeasterly. Joints/fractures are spaced approximately 0.3-0.5±m.

The site is within an area of high seismicity, but no active faults are mapped within the immediate site vicinity and the site is not located within an Alquist Priolo "Earthquake Fault Zone" for fault rupture hazard. The nearest active fault is indicated to be the Forest Hill-Melones fault (FHM) located approximately 5.7 km east of the project site. This fault is indicated (per Caltrans) to have a maximum credible earthquake magnitude of 6.5.

The published mapping (references 2 and 3) shows an isolated band of near surface (or exposed) ultramafic rocks about 2.8±km east of the Missouri Flat Road overcrossing. Such ultramafic rocks locally include serpentine (or serpentinite) and can, but do not always, contain naturally occurring asbestos. Ultramafic rock materials are not, however, mapped within the limits of this project, and none were observed during our site reconnaissance.

No landslides are shown on the published mapping within the project interval, and none were observed at time of site reconnaissance. No evidence of other geologic hazards (such as settlement, very soft soils, severe erosion, etc) was observed as part of this study.

Earth Materials and Conditions

State Bridge Department Study

The foundation report indicates that native materials encountered at the site consist of a mantel of soft gravelly clay underlain by bedrock described as "...light colored porphyritic rhyolite containing feldspar and quartz in a grayish or greenish ground mass of somewhat variable texture."

Taber Study

Earth materials encountered in the borings are divided into two units considered significant to the proposed project.

Unit I (Embankment/roadway fill and/or colluvium): In all borings, embankment/ roadway fill associated with the existing facilities and/or colluvium was encountered from ground surface to nominal depth (0.15 - 0.50±m). These materials are described as stiff and hard sandy and silty clay with gravel and dense-very dense silty sand. The fill/colluvium overburden materials are considered unreliable for direct support of new structure loads, but are stable and suitable for support of light-moderate superposed fill embankment loads. Locally, materials of this unit may also include residual soil.

Unit 2 (Weathered and Fractured Rock): This unit underlies Unit 1 soils and consists of metamorphic rock (Hornfels) consistent with outcrops at the project site. The rock unit was encountered in the borings at the following depths/elevations:

Boring	Support	Depth (m)	Elevation (m)
05-5	Abutment-1	0.15	531.57
05-4	Bent-2	0.30	531.53
05-3	Abutment-3	0.55	532.54
05-27	Abutment-3	0.46	532.14

The Unit 2 rock is divided into two sub-units, defined by an upper portion (Unit 2A) ranging from "decomposed" to "intensely weathered" (i.e., effectively "soil-like") and a lower portion (Unit 2B) ranging in condition from "moderately to slightly weathered" to locally "fresh." In general, the rock unit appears to become fresher with depth. However, the transition between Unit 2A and Unit 2B rock appears to be both abrupt and gradational, and depth of Unit 2A rock may vary significantly between borings.

Unit 2A rock materials were encountered to approximate elev. 529.7± in Boring 05-4 and Boring 05-5. In Boring 05-27, Unit 2A rock materials were penetrated below 0.4 m depth to terminal depth of boring at 1.0±m (elev. 531.9±). The rock in this interval was easily augered with 100 mm solid-stem continuous flight auger and at least nominal penetration was achieved with the "Standard Penetration" sampler; coring was not appropriate in these materials and achieved poor recovery where attempted (Boring 05-5). The rock mass of this subunit is estimated to classify as "very poor" to "poor" rock (see Appendix-C). Unit 2A rock materials were not encountered in Boring 05-3.

Unit 2B rock materials were encountered below Unit 2A in Boring 05-4, 05-5 and 05-27 and below elev. 532.3± in Boring 05-3. The rock of this sub-unit is less weathered and required diamond coring for drill advancement and is field-described (modified by office review of rock cores) as moderately hard to hard-very hard, non-foliated metamorphic rock (Hornfels – similar to rock examined in thin-section from core obtained at Weber Creek; see Appendix-B). The rock texture is typically composed of fine to medium grains within an aphanitic matrix. Degree of fracturing varies significantly from "very intensely" to "slightly." Based on boring encounter, this subunit is indicated to have a Rock Mass Rating (RMR) value of 56 to 68 and to classify as "fair" to "good" rock (see Appendix-C).

Rock Quality Designation (RQD) of all cored rock ranges from 45% to 57% (average 51%) in Boring 05-5 at Abutment-1, from 0 to 75% (average 43%) in Boring 05-4 at Bent-2 and from 0 to 100% (average 62%) in Boring 05-3 at Abutment-3. Within the intervals cored, average recovery was 86% in Boring 05-5 (Abutment-1), 90% in Boring 05-4 (Bent-2) and 92% in Boring 05-3 (Abutment-3).

Point load tests were performed on selected core samples from Borings 05-3, 05-4 and 05-5 in evaluation of rock compressive strength. For this project element, a total of thirteen rock cores were broken using a basic diametral test procedure in which the core axis is oriented perpendicular to the applied load. Point load tensile-strength index values were used to estimate uniaxial compressive strength values based on correlations developed by Bieniawski (Reference 1). Factors accounting for the variability in point load strength include rock composition, fracturing, grain size and weathering characteristics.

Results of point-load tests are included with Appendix-A. Samples tested yielded ultimate compressive strength values ranging from 93.2 MPa to 421.7 MPa (13,512 psi to 61,158 psi) with a mean of 211.2 MPa (30,634 psi).

A sample earth material profile with engineering properties is shown on Figure-2.

Groundwater

State Bridge Department Study

No free groundwater was encountered at time of January 1965 exploration made by the State. As referenced above, State records indicate that some water was encountered in the footing excavations and that it was successfully de-watered with a pump prior to casting the footings.

Taber Study

At time of April/May/June 2005 field study, no seepage or groundwater was noted within the augered intervals (lowest elev. 529.7±) of Borings 05-3, -4, -5 and -27. Groundwater measurement was not made in Borings 05-3, -4 and -5 below the augered intervals due to the presence of residual drill fluid.

The soil overburden materials and decomposed rock are expected to be seasonally saturated and are considered capable of transmitting seepage to open excavations; the decomposed to very intensely weathered rock, somewhat less so then soil. Groundwater occurrences in the underlying less weathered/fractured rock are expected to be restricted to open fracture/joint planes and localized/limited in extent and quantity. Other occurrences of relatively shallow "perched" groundwater may be present, particularly during the wet season and/or wetter years.

Site Seismic Conditions

In accordance with current Caltrans Division of Structural Foundations site seismicity evaluation procedures (with reference to "Caltrans California Seismic Hazard

Map 1996" and "A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996"), "Peak Bedrock Acceleration" (PBA) of 0.40g can be assigned the site associated with a controlling event of 6.5 magnitude on the Forest Hill-Melones fault located approximately 5.7 km east. The calculated Geomatrix (1997) PBA is 0.44g. Reference 13 lists this fault type as "normal."

This site may conservatively be assigned a soil profile "Type C" per Table B.1, Caltrans "Seismic Design Criteria" (SDC) version 1.3. Based on boring encounter, a soil profile "Type B" could be considered for use in design.

Caltrans structure design practice requires certain increases in SDC response curves due to fault type and/or fault proximity. At this site, fault type is not a factor, however, the proximity of the site to the seismic source will require a staged increase in spectral accelerations depending upon structure period. Per Caltrans procedures, sites within 15 km of an active fault should receive an increase in design spectral accelerations as follows:

Structure Period (seconds)	**Increase in Spectral Acceleration (%)**
0-0.5	No Increase
0.5-1.0	0% to 20% Linear Increase
≥1.0	20% Increase

Based on the guidelines and published Caltrans criteria as discussed above, the following SDC seismic design parameters are recommended for this site.

- Forest Hill-Melones Fault
- Magnitude 6.5±0.25
- Soil Profile Type C
- PBA = 0.5 g
- ARS curve from SDC Figure B.4 (modified to show increases in spectral accelerations)

The modified ARS curve is attached as Figure-3.

Liquefaction

Liquefaction is a secondary effect associated with seismic loading. Other than possible distortion of remnant fill, no major soil defects with respect to seismic loading are identified in the borings and soils data do not suggest the likelihood of secondary seismic effects such as liquefaction or lurching adversely affecting bridge foundations supported in the underlying rock unit. No other significant site soils defects with respect to seismic loading (e.g., lateral spreading, ground lurching, etc.) were identified from the limited data obtained in this study.

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

Corrosivity

Corrosivity tests (pH, minimum resistivity, chlorides and sulfates) were performed on a bulk sample of residual soil obtained from Boring 05-04. Test results indicate a "non-corrosive" soils environment as defined by the September 2003 Caltrans "Corrosion Guidelines" publication. No special corrosion considerations with respect to concrete/steel design are required for bridge foundations and substructure. Results of corrosivity tests are included with Appendix-A.

Conclusions and Discussion

Structure support is available and should be achieved within intact (Unit 2B) rock materials. The use of spread footing foundations appears to be the most appropriate foundation type and is recommended for both the new bridge and contiguous retaining walls. For spread footings, major site foundation characteristics/constraints affecting details of support level, bearing, etc. include location of support lines on irregular rock surfaces, excavation of hard rock to bearing levels, mechanical defects of the rock (fractures/joints) and local variation in rock depth/condition. Conditions are considered suitable for use of rock anchors, bolts, etc, if/as needed to provide uplift/overturning resistance.

The use of cast-in-drilled-hole (CIDH) pile foundations or large diameter drilled-shafts is also considered technically feasible, although this would require hard rock excavation. Tip elevations would depend on pile/shaft diameter and compressive, tensile and lateral loading requirements. Further details for such foundation can be provided based on data in-hand, if desired.

Driven (displacement) piles would not be expected to achieve adequate penetration for stability and are not recommended. Steel "H"-piling could be considered at some locations, but would be short (likely < 3.0 m) – achieving only very limited rock penetration – and would provide little lateral or tensile resistance.

The existing structure foundations are to be removed prior to construction of the new bridge. At Abutment-3, existing foundations are in very close proximity to proposed. At Bent-2, existing spread footings are indicated to be within the footprint of proposed foundation elements. While existing foundations are not expected to directly conflict with new spread footing foundations established lower in elevation, disruption from their removal might require increased footing depth or other consideration.

Recommendations

Bridge Structure

Spread footings should be at least 1.0 m wide and established with minimum penetration of at least 0.6 m into intact rock (Unit 2B) as affirmed by the personnel from this office. Such footings may very conservatively be assigned allowable (service load) bearing pressure of 478 kPa (5 tsf), net at ground line. Higher bearing pressures are readily available based on specific footing size and loading and/or with increased rock penetration, higher levels of preparation, etc. Settlement of such footings is expected to be nominal (<13 mm).

The metamorphic rock is expected to be moderately to slightly weathered with rock surface along footing lines typically variable on order of 0.3-0.6±m and containing open fractures; however, local irregularities of greater magnitude cannot be precluded. Based on boring encounters, highest plan footing levels meeting the above criteria are shown in the following Spread Footing Data Table.

Table 1

Spread Footing Data Table

Support Location	Minimum Footing Width	Bottom of Footing Elevation	Recommended Bearing Limits	
			WSD[1]	LFD[2]
			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Abut-1	1.0 m	529.00	478 kPa	N/A
Bent-2	1.0 m	529.00	N/A	1434 kPa
Abut-3	1.0 m	531.50	478 kPa	N/A

Notes: 1) Working Stress Design, (WSD), the Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}).

2) Load Factor Design, (LFD), The Maximum Contact Pressure (q_{max}) divided by the Strength Reduction Factor (ϕ) is not to exceed the Nominal Bearing Resistance (q_n).

Local surface irregularities along footing lines may be considered for field adjustment of rock penetration requirement upon review/approval of the foundation engineer.

Lateral load resistance of spread footings may be calculated as follows:

1. A base friction factor of 0.75 is recommended for intact rock.

2. Soil resistance against the face of footings can be based on passive pressure of 64.0 $kN/m^2/m$ (based on formed footings with compacted structure backfill or footings poured neat against intact rock).

3. Per Caltrans practice, the following guidelines should be used for the force/moment equilibrium analysis of the foundations:
   - Use 100% base friction and 0% passive resistance, or
   - Use 0% base friction and 100% passive resistance, or
   - Use 50% base friction and 50% passive resistance.

Footing concrete should be poured neat, without forming, against trimmed, intact bearing materials within clean and dry excavations. Any exposed open fractures or other discontinuities should be carefully evaluated by the soils engineer with respect to bearing/stability considerations and cleaned/surface-grouted, if necessary.

Some modification of footing level may be necessary if/as disruption of bearing material occurs due to removal of existing footings, conditions differ from those anticipated and/or if previous excavation disrupted the rock to levels near proposed footing elevation. If necessary, additional excavation (up to 1 m) can be backfilled with plain (Class-C) concrete, with doweling utilized to provide positive contact between the structural footing element and plain concrete.

Retaining Walls

For Type-1 retaining walls with level backfill (Case 1) condition, Caltrans "Standard Plans" indicate a maximum toe pressure of 275 kPa (5.7 ksf) and 325 kPa (6.8 ksf) for retaining wall height 8500 mm and 10300 mm, respectively. Base of retaining wall footings established within intact rock at the same levels shown in the Spread Footing Data Table for abutment footings are considered suitable for allowable design bearing pressures up to 325 kPa (6.8 ksf), net at ground line. "Ultimate" bearing pressures are to at least 3 times allowable values.

Materials exposed at footing grades should be reviewed by the soils engineer to affirm uniformity and suitability for support of retaining wall foundations. If the rock is found to be weak or disturbed, use of plain concrete would be considered appropriate to engage suitable rock below base of structural footing, if/as necessary. Any disturbed areas along footing grade (e.g., associated with existing footing construction) should be removed to full depth and replaced with plain concrete.

Conversely, stepping of individual footings would also be considered appropriate in hard rock to achieve required penetration of bearing materials without excessive excavation.

Lateral Soil Pressures

With use of Caltrans "Structure Backfill" or equivalent, an active soil pressure of 5.6 kN/m^2/m (36 pcf) is considered appropriate for use in abutment and retaining wall design with level backfill. Back of wall drainage should be established per Caltrans "Standard Plan" details (B3-8).

Seismic loading will apply additional soil pressure to abutment/retaining walls. 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 9.1 kN/m^2/m (58 pcf).

For free standing walls, expected to be capable of significant "yield" and displacement under seismic loading, it is appropriate to reduce the incremental soil loading from seismic forces by as much as 50% for evaluating wall stability with respect to sliding and overturning.

For seismic loading into abutments, passive soil resistance of up to 239 kPa is available (to be reduced for effective wall height less than 1.7 m in accordance with Caltrans SDC v.1.3).

Embankment

Embankment construction and any new fill placement should be in accordance with Caltrans "Standard Specifications", including at least 95% relative compaction on all fill within 50 m of bridge abutments. Where new fill is to be placed onto existing embankment slopes, it should be fully-bonded into the existing fill by placing on discrete horizontal benches cut fully into the slope and below any loose/soft or otherwise unsuitable materials (per Section 19 of Caltrans "Standard Specifications").

Excavation Conditions

Groundwater is not anticipated during dry season construction. However, the presence of seepage from surface infiltration cannot be precluded. Such seepage, if encountered, is expected to be readily controllable by pumping.

Existing fills and residual soils are expected to be readily excavated using typical earth moving equipment. Excavation of rock within bridge and retaining wall footing limits to depths indicated above is expected to be locally difficult (e.g., retaining wall footings), but generally achievable by use of air tools without blasting. Rock blasting may disrupt/degrade integrity of the surrounding rock and other facilities and should be performed only under carefully controlled conditions and with prior written approval of the engineer.

If required, blasting should be performed in accordance with Caltrans "Standard Specifications" (including Sections 7-1.10 and 19-2.03). The specifications and special provisions developed for blasting should address safety issues and avoidance of damage to existing pavement, utilities, structures and other natural and man-made features. Such procedures and specifications should be reviewed by this office.

Temporary (construction) backslope in rock is expected to be appropriately stable at 1:1 or flatter; lower cut sections (in less-weathered rock) may be stable at construction slopes of 2v:1h, upon positive review by the engineering geologist. Consideration for shoring will be required for local areas of weak rock, remnant embankment and/or any areas exhibiting potential for failure along daylighting fracture planes, and/or where existing supports may be jeopardized (particularly at new Abutment-3).

Excavation and shoring should conform with CalOSHA requirements and the Caltrans "Trenching and Shoring Manual."

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March 23, 2006

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Since 1954

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.