Final Geotechnical Investigation Report
Chinquapin Intersection
Wawona Road/Glacier Point Road
Yosemite National Park, California

Submitted to:

Jacobs Civil, Inc.
Tempe, Arizona

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Tempe, Arizona

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Transmittal No. 2

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Attn: Berwyn Wilbrink, P.E.

Re: Final Geotechnical Investigation Report
Chinquapin Intersection
Wawona Road/Glacier Point Road
Yosemite National Park, California

Our Final Geotechnical Report to support the redesign of the Chinquapin Intersection of Wawona and Glacier Point Roads, improvements to Glacier Point Road between the intersection and the Badger Pass ski area, and the ski area access roads and parking lots is presented herein.

The purpose of the investigation was to examine the geotechnical profile beneath the site and evaluate the engineering properties of the subsurface materials. This information was used to provide recommendations for earthwork, structure foundations, and slopes for proposed retaining walls and cut/fill slopes within the project limits.

Should any questions arise concerning this Final Geotechnical Report, please contact the undersigned at 480-940-2320.

Respectfully submitted,

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1.0 PROJECT DESCRIPTION

Glacier Point Road, also known as Yosemite National Park Route 15, starts at the Chinquapin Intersection along Wawona Road, about midway between Yosemite Valley and Wawona, and extends 16 miles to Glacier Point. The Badger Pass Ski Area is located along Glacier Point Road, approximately 5.1 miles from the Chinquapin Intersection. The existing roadway, which consists of a 21- to 24-foot wide, two-lane paved road with unpaved shoulders, was completed in 1936, replacing a wagon road that was first built in 1882. The project involves the rehabilitation, restoration and resurfacing of the roadway between the Chinquapin Intersection and the Badger Pass Ski Area, including realignment of the Chinquapin Intersection, addition of turn lanes and chain-up pullouts along Wawona Road, extension of a rock guardwall and safety and parking improvements at the El Portal Overlook, rehabilitation of the Badger Pass Ski Area access road, and drainage and pavement rehabilitation work at the Badger Pass Ski Area. The National Park Service Denver Service Center prepared preliminary design concept drawings for the El Portal Overlook and the Chinquapin Intersection in 1991. In 2005, Yosemite National Park (YNP) provided updated reference files for underground utilities in these areas. The design was completed to the 30 percent (%) level by Carter and Burgess in late 2005 (Carter & Burgess, 2005)[1].

The recent geotechnical investigation performed by AMEC Earth and Environmental (AMEC) was focused on the Chinquapin Intersection. Planned improvements at the intersection include minor realignment of the roadway to provide better sight distance, the addition of turn lanes on Wawona Road to improve safety, and the addition of winter chain-up/chain-off pullouts on both sides of Wawona Road. Several proposed retaining walls will also be constructed for the intersection improvements, and an "authorized vehicle only" pullout at the site of a former gas station will be added for YNP snow-plow vehicles. Additionally, the existing parking area will be reconstructed to provide accessible parking and access to existing restroom facilities.

2.0 INVESTIGATION

2.1 Previous Investigations

Previous geotechnical investigations were for this project performed at the Chinquapin Intersection, along Glacier Point Road and at the Badger Pass Ski Area. The investigations were performed by Kleinfelder, Inc. (Kleinfelder) in fall of 2005 and the geotechnical investigation reports were published in April, 2006.

The investigation work (Kleinfelder, 2006a) at the Chinquapin Intersection included three test borings drilled between about Sta. 16+80 and Sta. 18+70, left, at the location of a planned retaining wall.

A pavement investigation was performed along Glacier Point Road and at the Badger Pass Ski Area (Kleinfelder, 2006b). The investigation included a pavement condition survey, pavement coring. An appendix to the report presented the results of rock slope evaluations.

[1] References are listed at the end of this report.

2.2 Exploration Drilling

Four borings (C-1 through C-4) were advanced to approximate depths ranging from 20 to 30 feet below existing site grades at selected locations at the Chinquapin Intersection (Figure 1, attached). The locations and purposes of the borings are as follows:

- Boring C-1 – Advanced on the right side of Wawona Road, south of the intersection, at the site of an existing rock cut that may be impacted due to roadway widening.
- Borings C-2 and C-3 – Advanced on the left side of Wawona Road, on either side of the intersection with Glacier Point Road, to investigate subsurface conditions for proposed concrete retaining walls.
- Borings C-4 – Advanced on the right side of Wawona Road, north of the intersection, along a planned rockery wall.

All borings were drilled using a Burley 4500 drill rig owned and operated by CRUX Subsurface, Inc. Borings C-2 through C-4 were located along the shoulder of the road and were accessed using a rubber-tracked rig, whereas a crane was used to place the drill near the top of the rock cut at Boring C-1. Boring C-1 and the lower portion of C-2 were advanced using a HQ-size wireline, triple-tube and rock coring system. The HQ-size diamond rock bit produces 2.50-inch diameter core and a 3.78-inch diameter borehole. The upper portion of Boring C-2, and all of Borings C-3 and C-4, were advanced using a 4.5-inch diameter HWT tricone bit. Standard penetration testing (SPT) and sampling were performed in fill materials, soils, and decomposed or highly weathered rock at intervals of 5 feet or less.

Soil and rock encountered in the borings were inspected, visually classified in the field, and logged in accordance with ASTM and Central Federal Lands Highway Division (CFLHD) Project Development and Design Manual (PDDM) standards (FHWA, 1996). The boreholes were logged by Richard Bansberg, P.G., with AMEC. Logs of the test borings are presented in Appendix A.

Two of the borings were advanced to greater depths than originally planned due to low blow counts (n-values) at the planned termination depths. Boring C-3, which had a target depth of 20 feet, was deepened an additional 15 feet to 35 feet. SPT (N-value) results were 5 at 20 feet, 6 at 25 feet, 13 at 30 feet and 15 at 35 feet. Boring C-4, which had a target depth of 15 feet, was deepened an additional 5 feet to 20 feet. N-value results at 15 and 20 feet were 6 and 8, respectively.

All borings were backfilled with drill cuttings and fine-grained materials located along the roadway. Drilling fluids were re-circulated while drilling and used drilling fluids were removed from the Park for disposal.

2.3 Pavement Cores

Pavement cores (P-1 through P-4) were obtained at four locations as shown on Figure 1. A Burley 4500 drill rig using an HWT casing advancer was used to core through the pavement and aggregate base course (ABC) layer. Samples of subgrade materials were then obtained to a depth of 3 feet below existing grades. The thickness of the ABC was measured, the borings were backfilled to the bottom of the ABC layer with sand and gravel from a local stockpile of materials, and the borings were then backfilled with cold patch asphalt to match the existing roadway surface. Logs of the pavement core borings are presented in Appendix A.

2.4 Laboratory Analysis

Sieve analysis, Atterberg limits (plasticity index) and moisture content determinations were performed on selected samples collected from borings. Unconfined compressive strength testing was performed on selected segments of recovered rock core. The results of laboratory testing are included in Appendix B or are presented on the test boring logs. The results of laboratory testing are presented in Appendix B.

2.5 Review of Slope Inventory

As part of a geotechnical investigation performed by Kleinfelder (2006), field assessments and 'window' mapping of Glacier Point Road between the Chinquapin Intersection and Badger Pass Ski Area were performed to assess rock outcrops that are expected to be affected by proposed roadway improvements. Kleinfelder evaluated the rock slopes based on visual observations and conservative assumptions, and assigned point totals that were used to rank the performance of the slopes. They also provided rock scaling and associated recommendations for improving the rock slopes.

As part of AMEC's geotechnical investigation, the results of Kleinfelder's slope inventory were reviewed. AMEC briefly observed the conditions at each of the 16 rock slopes inventoried and reviewed the rating forms prepared by Kleinfelder for these slopes (Kleinfelder, 2006). The results of our review are discussed in Section 5.7.

2.6 Seismic Investigation

Seismic Line 6 was completed at Boring BA-1. The compression wave (p-wave) profile indicated that fill or surficial soil, with p-wave velocities of about 590 to 710 feet per second (ft/sec) to a depth of about 0 to 4 feet, and p-wave velocities of about 1,200 to 1,600 ft/sec is present from the surface to depths of about 11 to 23 feet along the profile. Below this horizon, the p-wave velocities increase to about 4,900 to 6,800 ft/sec to the p-wave depth of investigation of about 39 feet. These velocities are consistent with granodiorite where refusal was encountered in BA-1. Shear wave (s-wave) velocities below about 25 feet are interpreted to increase to 3,000 and eventually to 4,800 ft/sec, which is consistent with relatively competent bedrock with p-wave velocities of perhaps 6,000 to nearly 10,000 ft/sec.

Three 120-foot long refraction seismic surveys, Lines 1 and 2 at Wawona Road/Glacier Point Road interchange and Line 6 on Glacier Point Road were completed by Michael L. Rucker, P.E. with AMEC, with the assistance on Mr. Freiman at the upper Line 6. A Geometrics Smartseis 12-channel signal enhancement seismograph and geophone array consisting of 4.5 Hz vertical geophones deployed at 10-foot spacings were used in the evaluation. Both compression wave (p-wave) seismic refraction data and surface wave for s-wave refraction microtremor (Remi) data was collected at each of the three lines. Locations of the seismic lines are shown on the site plans in Appendix A.

A sledgehammer energy source was used to collect p-wave data for seismic refraction analysis. Jumping near one end of the geophone array was performed to generate surface wave energy for data to perform a one-dimensional Remi s-wave profile at each seismic line. The results of the refraction seismic surveys are presented in Appendix C at the end of this report, along with a brief description of the seismic refraction equipment and procedures used.

Due to the nature of the geophysical techniques utilized, all depths, locations and velocities presented on the interpretations attached are approximate. The maximum practical depth of p-wave investigation for 120-foot long seismic lines in about 30 to 40 feet below ground surface. However, actual depths of investigation vary according to the subsurface profile for each line. P-wave depth of investigation interpretations are included in the interpretations and range from about 18 to 40 feet. S-wave depths of investigation are typically deeper than p-wave depths of investigation. S-wave depths of investigation are also included in the interpretations and are estimated to be greater than 100 feet.

Velocity reversals, where softer, lower-velocity materials could underlie moderate- to higher-velocity materials, would not be detected using the p-wave seismic refraction technique. Significant, relatively large-scale velocity reversals might be detected in the s-wave profile obtained from the Remi technique. However, such reversals are interpreted as a one-dimensional s-wave profile without lateral variation across the seismic line. It must be understood that where subsurface profiles are sloping or dipping, interpreted s-wave depths will tend towards the deeper lower velocity portions of the profile. Interpreted subsurface material p-wave velocities from the seismic lines are average values obtained over distances of 10 to 20 feet. Discrete zones of material could have slower or faster velocities, and therefore be weaker or stronger that indicated by the average velocities interpreted from the seismic data.

Where p-wave results are not available to relevant depths, due to shallow depth of investigation or very low subsurface velocities similar to velocities of sound in air, s-wave results with deeper depth of investigation can be used to estimate corresponding deeper p-wave velocities. Given a typical soil Poisson's ratio of 0.33, a p-wave velocity can be estimated by doubling the corresponding s-wave velocity. Also, in subsurface profiles where the s-wave velocity is considerably less than one-half of the corresponding p-wave velocity, relatively thin horizontal-oriented cementation or the presence of a velocity reversal may be indicated.

Seismic Line 1 was completed at Boring C-2. The p-wave profile indicated that fill or surficial soil with p-wave velocities of about 500 to 900 ft/sec to a depth of about 3 to 5 feet, and p-wave velocities of about 1,200 to 1,300 ft/sec is present to depths of about 3 to 20 feet along the profile. Below this horizon, the p-wave velocities increase to about 2,400 ft/sec to the p-wave depth of investigation of about 18 feet. This velocity is consistent with decomposed to highly weathered granite found in C-2. Below an interpreted depth of about 40 feet, the s-wave velocity is interpreted to increase to 3,400 ft/sec, which is consistent with more competent granite with a p-wave velocity of perhaps 6,800 ft/sec.

Seismic Line 2 was completed at Boring C-3. The p-wave profile indicated that fill or surficial soil, with p-wave velocities of about 560 to 830 ft/sec to a depth of about 1 to 2 feet, and p-wave velocities of about 1,200 to 1,300 ft/sec is present to depths of about 11 to 26 feet along the profile. Below this horizon, the p-wave velocities increase to about 2,300 to 2,600 ft/sec to the p-wave depth of investigation of about 24 feet. This velocity is consistent with decomposed granite found in C-3. Below an interpreted depth of about 50 feet, the s-wave velocity is interpreted to increase to 4,900 ft/sec, which is consistent with more competent granite with a p-wave velocity of perhaps 9,800 ft/sec.

3.0 GEOLOGIC CONDITIONS AND GEOTECHNICAL PROFILE

3.1 Regional Geology

The project site is located in the Sierra Nevada Mountains, which combines with the Cascade Mountain range to the north to form the Sierra-Cascade Mountains physiographic province (Hunt, 1974). The Sierra Nevada range is a large block mountain, predominantly comprised of Jurassic and Cretaceous granite, which was raised by faulting on the east during the Pleistocene. The mountain range is tilted to the west and has been cut by a number of major rivers including the Merced River which flows through Yosemite Valley. The higher portions of the Sierra Nevada range were glaciated at least three times, deepening major river valleys and leaving tributaries as hanging valleys, like the many examples in Yosemite Valley.

The project site is located in Yosemite National Park on the western side of the Sierra Nevada batholith, in an area predominantly underlain by Cretaceous plutonic rocks, which typically range in composition from quartz diorite to granite and range in age from 120 million years (m.y.) to 88 m.y. (Peck, 2002). Dozens of separate plutons have been mapped in the Yosemite area, most of which were emplaced at depths of 3 to 6 miles below the ground surface as hot viscous magma and cooled slowly resulting in a crystalline texture. The Chinquapin Intersection is located near a roughly north-south trending contact between two large plutons: the Bass Lake Tonalite on the west and the El Capitan Granite on the east. The Bass Lake Tonalite typically consists of well-foliated, medium-grained, equigranular tonalite and granodiorite containing abundant mafic inclusions. Mafic minerals consist of equal parts biotite and hornblende. The El Capitan Granite typically consists of light-colored, coarse-grained biotite granodiorite and granite with 5 to 10% mafic minerals. Biotite dominates and hornblende is either absent or present in trace amounts.

Aerial photographs of the site available on Google-Earth™ were reviewed in an attempt to identify linear features which could be indicative of faults, shear zones or other geologic structures. An approximately east-west trending lineament represented by a linear trough or depression appears to pass through the site immediately to the north of the intersection of Wawona and Glacier Point Roads. This lineament may represent a zone of weaker and more readily erodable rock, although no geologic structures are identified on the geologic quadrangle map of the site prepared by the U.S. Geological Survey (Peck, 2002).

3.2 Geotechnical Profile

Boring C-1 was advanced on the right side of Wawona Road at the top of an existing rock cut that may be impacted due to planned roadway widening (Photograph 1 in Appendix A). The cut slope has a maximum vertical height of approximately 30 feet. Granite was encountered the full depth of the 30-foot deep boring advanced at this location. The granite is medium grained and light to medium gray, and dark gray mafic-rich xenoliths ranging from ½ inch to nearly 1 foot in diameter are common. The granite typically is hard and slightly weathered, although 1- to 2-foot thick zones of moderately soft to soft and medium to highly weathered rock were encountered throughout the boring. Discontinuity spacing typically is very close to close (0.2 to 3 feet), and locally wide (3 to 10 feet). Discontinuity orientations are variable, although near-horizontal and near-vertical sets appear to dominate. Discontinuity surfaces typically are slightly to moderately rough, and the discontinuities commonly are stained with iron-oxide and manganese oxide and are filled with silt. Core recovery ranged from 90 to 100% and rock quality designation (RQD) values ranged from 30 to 90. Drilling fluid recovery ranged from 25 to 50%.

Boring C-2 was advanced on the left shoulder of Wawona Road on the south side of the intersection at the proposed site of a concrete retaining wall (Wall 1). Embankment fill, which consists of nonplastic to low plasticity silt and fine-grained sand, was encountered to a depth of 8 feet below the ground surface (bgs) and is underlain by granite. N-values obtained in the fill at depths of 5 and 10 feet bgs were 7 and 5, respectively. The granite is decomposed to highly weathered to a depth 20 feet bgs and moderately weathered to a depth of 25 feet bgs, the final depth of the boring. The rock is soft from 8 to 13 feet bgs, and then moderately hard with occasional hard zones to a depth of 25 feet bgs. Discontinuity spacing is close to moderately close and discontinuity orientations are variable. Discontinuities commonly are stained with iron oxide and filled with silt. Core recovery was 100%, the RQD value for core from 8 to 20 feet was 0 and the RQD value for the core obtained below a depth of 20 feet was 65. Drilling fluid recovery was intermittent.

Kleinfelder (2006a) performed three test borings along the Wall 1 alignment. The borings encountered low and inconsistent density embankment fill to depths ranging from 9 to 30 feet, with the thickness of the fill increasing from about 9 feet near Sta. 16+80 to 31.5 feet near the existing culvert at Sta. 18+70. The material encountered in all three borings consists of silty sand with gravel and cobble sized granite fragments.

Boring C-3 also was advanced on the left shoulder of Wawona Road at the proposed site of a concrete retaining wall (Wall 2); however, the boring was advanced on the north side of the intersection and the subsurface conditions are markedly different from those encountered at Boring C-2 located on the south side of the intersection. Embankment fill, which consists of nonplastic to low plasticity silt and fine- to medium-grained sand, was encountered to an estimated depth of 17 feet bgs and is underlain by residual soil derived from the in-situ decomposition of granite bedrock. The contact between the fill and residual soil is estimated because the two materials appear very similar and have similar engineering properties (e.g., N-values). The actual contact between the embankment fill and residual soil may occur at a shallower depth, possibly as shallow as 10 feet bgs. The residual soil consists of nonplastic to low plasticity silt and fine- to medium-grained sand, and the soil appears to retain much of its original granitic fabric. N-values in both the fill and residual soil are low, with consistent values of 5 and 6 between depths of 5 and 25 feet bgs, and values of 13 and 15 at depths of 30 and 35 feet, respectively.

Boring C-4 was advanced on the right side of Wawona Road at the proposed site of a rockery wall, and the subsurface conditions encountered in this boring are similar to the conditions encountered in Boring C-3. Residual soil consisting of nonplastic to low plasticity silt and fine- to medium-grained sand was encountered from approximate depths of 1 to 20 feet bgs. Low N-values, ranging from 6 to 13, were obtained between depths of 5 and 20 feet bgs. As is the case with the residual soil in Boring C-3, the residual soil in Boring C-4 retains its relict granitic fabric, and the individual grains break down to silt using finger pressure.

Boring BA-1 was drilled at the planned El Portal Overlook, located near Glacier Point Road Sta. 97+00, right. The boring encountered loose fill soils, soft native soils and highly weathered rock to a depth of about 15 feet. Below 15 feet the rock was less weathered and the boring was refused on either a boulder or intact bedrock at 20 feet.

3.3 Groundwater and Soil Moisture Conditions

Water was introduced into the borings as part of the drilling operation, so the presence of natural groundwater could not be determined. A spring is located on the right side of Wawona Road at approximately Sta. 26+95, about 150 north (upstation) of Boring C-4. The spring was flowing at approximately 5 gallons per minute (gpm) at the time of our investigation. Water from the spring flows along the drainage ditch on the right side of the road in an upstation (northerly) direction for approximately 100 feet, flows beneath the road through a culvert, and then flows down the embankment on the left side of the road. Based on conversations with park personnel, it is understood that a groundwater well has recently been completed approximately 0.5 miles to the southwest of the project site. The depth to water in the well was reported to be approximately 800 feet bgs.

The soil samples obtained from the drill holes generally were moist, but the moisture may be a result of drilling operations, since all boring were drilled using water.

3.4 Site Seismicity

Recommended seismic response parameters for use in slope stability analyses are based on results of a probabilistic seismic hazard assessment published by the U.S. Geological Survey (USGS, 2004) and represent ground motion corresponding to an exceedance probability of 10 percent in 50 years (return period of 500 years), consistent with the AASHTO-recommended criterion in Division IA of AASHTO (2002). The USGS-published data consists of probabilistic horizontal spectral acceleration values for various return periods, and correspond to a specific peak ground acceleration (PGA) as reported by the USGS. A site location of 37.65 degrees north latitude and 119.70 degrees west longitude was used as input to the USGS seismic hazard website, with a resulting PGA value of 0.148g. This generally agrees with the acceleration coefficient (A) of 0.15g from Figure 1.5 in Division IA of AASHTO (2002).

4.0 PAVEMENT CORES

Pavement cores were obtained at four locations (P-1 through P-4), as shown on Figure 1. The thickness of the asphaltic concrete at these four locations ranged from 4.5 to 6 inches. Asphaltic concrete also was encountered at Borings C-2 through C-4 and had thicknesses ranging from 4 to 5 inches. The aggregate base course at core locations P-1 through P-4 consists of nonplastic sandy gravel ranging from 9.5 to 14 inches in thickness. The gravel typically ranges from ¼ to 1½ inches in diameter and is angular, whereas the sand is well graded and subangular to angular. The subgrade material, to a depth of 3 feet below existing grades, consists of low to medium plasticity silt and fine- to medium-grained sand.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 Discussion

The geotechnical conditions revealed at the retaining wall locations (Wall 1 – Sta. 17+00 to Sta. 19+75, left, and Wall 2 – Sta. 21+50 to Sta. 23+00, left) are challenging. As described in Section 2.3, very soft fill and native soils were encountered to depths of over 25 feet. While the currently planned cast-in-place (CIP) concrete retaining wall heights required to widen the intersection are relatively small, less than 8 feet in height, the variable density, the low strength and the high compressibility of the foundation materials would require extensive site grading, deep foundations or other site preparation activities, such as compaction grouting, to provide adequate foundation support.

Alternative approaches to CIP concrete retaining walls include mechanically stabilized earth (MSE) retaining walls, reinforced soil slopes (RSS) to over-steepen the existing slope, or permanent tied-back or soil nail walls. Each of these alternatives would need to be designed with consideration of the underlying low strength materials in order to meet overall global stability design requirements.

Each alternative described above is considered excessive for the relatively small lateral extension of the roadway. It is recommended that an assessment of the road geometry be undertaken to evaluate if an alternative to wall construction can be achieved.

If the CIP walls are the selected alternative for the necessary grade separations, they are recommended to be supported by micropiles extending to the underlying intact granite bedrock.

This recommendation is based on the following considerations:

- Micropiles can be installed using small sized equipment to minimize the disturbance to the road users during construction.
- The micropiles can be used with a conventional cantilever retaining wall section, with adjustments to account for the pile connection.
- Micropiles can be installed at various inclinations to resist lateral loads.
- Methods are available to counter potential corrosion of the piles.
- The micropiles will provide a more positive and uniform system for transferring the foundation loads through the soft soils, as compared to a grouting program.
- FHWA has extensive experience in the use of micropiles.

The depth to the bedrock at Wall 1 ranges from 10 to over 31 feet. The depth to bedrock at Wall 2 is in excess of 35 feet. Supplemental refraction seismic geophysics is planned to supplement the information on the profile of the underlying granite bedrock.

The soils exposed at the base of the CIP walls will not provide a suitable working surface for the forming of the concrete retaining wall footings/pile cap. A 6-inch thick layer of granular bedding material meeting Section 704.09 of the specifications is recommended to be placed below the base of the retaining wall foundations.

5.2 Micropile Design Criteria

Micropiles should be designed following the design methods described in the FHWA (2000) Micropile Design and Construction Guidelines Implementation Manual. A sample specification for micropiles is presented in Appendix D.

The final pile design should be completed by a specialty contractor who is qualified to perform micropile design and construction. A test pile should be constructed and load tested to confirm the design parameters.

For an estimation of the pile capacity, considering a Grade 75 No. 10 bar, each micropile is capable of resisting up to 56 kips axial load. Following a service load design approach, a grout to rock bond ultimate strength of 100 pounds per square inch and a safety factor of 2.5 and an 8-inch diameter micropile, the required bond lengths will be about 5 feet.

End bearing and skin friction contributions from the fill and residual soils overlying the bedrock are recommended to be neglected. A minimum rock bond length of 5 feet is recommended.

Since the existing grades and the proposed final grades along Wawona Road are the same at the retaining wall locations, settlement of the fill and residual soils and downdrag of the piles is not expected. Any raise in elevation of the planned roadway grade will need to be evaluated.

Lateral resistance is recommended to be provided by installing a number of the micropiles on an inclination. The lateral resistance of vertical micropiles should be evaluated using a method which estimates the lateral load-displacement behavior using a finite difference technique based on elastic beam column theory and p-y (soil reaction-displacement) curves. Computer programs such as LPILE or COM624P are available to assist the designer. Based on Reese and others (1984), the behavior of the soil surrounding the laterally loaded piles is described by lateral load-transfer functions referred to as p-y curves. The soil reaction (p) is related to the pile deflection (y) for various depths below the ground surface. In general, these curves are nonlinear and depend on several parameters, including depth, pile diameter and soil strength. Deflection, bending moment and shear profiles at specified intervals along the length of the pile are computed.

Should the project structural engineer prefer to perform LPILE or COM624P analyses in-house, recommended soil strength parameters for use in the analyses are provided in the following tables:

Soil Type (ft)	Average Effective Unit Weight (pcf)	Friction Angle (degrees)	Cohesion c (psf)	Soil Strain Ratio ε_{50}	Horizontal Subgrade Modulus K (pci)	Recommended Soil Type in LPILE
Fill and Residual Soils	105	30	---	---	25	Sand

Soil Type (ft)	Average Effective Unit Weight (pcf)	Unconfined Compressive Strength (psi)	Young's Modulus (psi)	Strain Parameter k_{rm}	Rock Quality Designation (RQD)	Recommended Soil Type in LPILE
Rock	160	4,000	375,000	0.0005	50	Weak Rock

As described in the design manual (FHWA, 2000), buckling of micropiles is a concern only in soils with the poorest mechanical properties, such as loose silts, peat and soft unconsolidated clay. A special design for buckling is not recommended. The recommended load testing program will provide a conservative check of the micropile bucking potential.

5.2.1 Special Contract Requirement Language

Special Contract Requirements for micropiles should be included as follows:

Section 208 of the Standard Specifications should be revised to read

> **(e) Foundations using micropiles.** Excavate to the foundation elevation and install the micropiles. Remove all loose and displaced material, and reshape the bottom of the excavation to the foundation elevation. Smooth and compact the bed to receive the footing.

Section 258 of the Standard Specifications should be amended as follows:

> **258.03 General.** Survey according to Section 152, and verify the limits of the wall installation. Prepare and submit forms and falsework drawings according to Section 562. Perform the work under Section 208.
>
> **258.07 Acceptance.** Reinforced concrete retaining wall material, construction, and services will be evaluated as follows:
>
> Structure excavation and backfill will be evaluated under Section 208.

5.3 Lateral Earth Pressure

The lateral earth pressure (earth load) acting on retaining walls is dependent on the following factors:

- Backfill properties, including soil type and gradation, unit weight, moisture content, drainage and creep behavior (if any)
- Soil-structure interaction (wall movement under load)
- Backfill drainage provisions
- Compaction-induced pressures and other surcharges
- Earthquake loads

5.3.1 Design Pressures

The lateral pressure against earth-retaining walls is dependent on the degree of restraint. Rigid walls are recommended to be designed considering the at-rest condition, as follows:

$$P_o = \frac{1}{2}\gamma H^2 K_o$$

where:

P_o = At-Rest Earth Pressure
H = Height of Wall
K_o = At-Rest Earth Pressure Coefficient $(1 - \sin\phi)$
Φ = Angle of Internal Friction, degrees
γ = Unit Weight of Fill/Granular Native Soil

Rigid, absolutely restrained walls will be subjected to earth pressures represented by a hydrostatic load diagram of about 55 pounds per square foot per foot of depth.

For walls capable of rotating at least 0.001 times the wall height, soil pressures will reduce from the at-rest to the active condition. The Rankine earth pressure theory is recommended for use in calculating lateral earth pressures on retaining walls. For smooth, vertical walls and horizontal backfill, the Rankine active earth pressure can be calculated as:

$$P_a = \frac{1}{2}\gamma H^2 K_a$$

where:

P_a = Active Earth Pressure
H = Height of Wall
K_a = Rankine Active Earth Pressure Coefficient
K_a = $\tan^2\left(45 - \frac{\phi}{2}\right)$
Φ = Angle of Internal Friction, degrees
γ = Unit Weight of Fill/Granular Native Soil

An equivalent hydrostatic load diagram of 35 pounds per square foot per foot of depth for level backfill can be used to model the earth pressures on walls that move sufficiently to develop active earth pressures.

A friction angle of 34 degrees and a unit weight of 125 pounds per cubic foot are recommended for use in these equations. Earth pressure coefficients for wall configurations other than for vertical, smooth walls with level backfill can be provided upon request. The earth pressures calculated from these equations are actual values and should be factored as appropriate to the design condition. Earth pressures will be significantly higher if free water is present in the backfill.

The Mononobe-Okabe (M-O) pseudo-static theory should be utilized to estimate the equivalent pseudo-static force on walls due to earthquake effects. The theory is applicable to walls which are free to yield to a sufficient degree to permit development of the active earth pressure. For fixed, rigid walls which cannot rotate, the M-O theory will significantly underestimate the seismic force.

Basic assumptions included in the development of the M-O theory include the following:

- The wall is sufficiently flexible, i.e., yields to a degree sufficient to produce minimum active stress conditions
- The wall backfill deforms to the extent that the full soil shear resistance is mobilized along the failure plane in an active sense, i.e., the soil wedge is at incipient failure
- The backfill is cohesionless and unsaturated
- Acceleration is constant throughout the failing wedge, i.e., the backfill behaves as a rigid body
- The location of the resultant force (dynamic component of thrust, estimated by M-O theory to be at 0.67H above the wall base) is based on the assumption that the backfill is uniform and elastic

The simplified form of the M-O equation is as follows:

$$P_{AE} = \frac{1}{2}\gamma H^2(1 - k_v)K_{AE}$$

where:

P_{AE}	=	active force on wall imposed by <u>both static and seismic loadings</u>
γ	=	unit weight of wall backfill = 125 pcf
H	=	height of wall (height of soil face, i.e., from finished grade of top of backfill to base of wall)
K_{AE}	=	seismic active pressure coefficient
	=	$K_A + \frac{3}{4}k_h$ (practical approximation suggested by Seed and Whitman, 1970)
K_A	=	Rankine active earth pressure coefficient = 0.26 for structural backfill
k_h	=	horizontal acceleration coefficient (g)
k_v	=	vertical acceleration coefficient (g)

The resultant of the seismic thrust should be located at a height of 0.6H above the wall base.

Seed and Whitman (1970) and Whitman (1990) indicate that, based on results of model (shaking table) tests and finite element analyses of rigid, non-yielding walls, the wall will move rigidly with its underlying base under dynamic loads, and it would be expected therefore that the dynamic earth pressure would be larger than that predicted by the M-O theory which assumes active stress conditions. The measured seismic earth pressure on the rigid wall may exceed the force predicted using M-O by a factor of 2 to 3. Therefore, it is recommended that the earth pressure on rigid retaining walls due to seismic loading be estimated using the procedure presented by Wood (1973). The steady-state dynamic thrust determined by Wood (1973), <u>exclusive of the static thrust</u>, can be approximated by the following (Whitman, 1990):

$$P_{AE} = \gamma H^2 a$$

where: P_{AE} = active force on wall imposed by seismic loading
a = horizontal base acceleration = k_h
γ and H as before

According to Wood (1973), the resultant of the seismic thrust should be located at 0.63H above the wall base, though use of 0.6H is deemed appropriate. Also, in the above expressions, k_h should be set at 0.15g (see Section 3.4) and k_v at 2/3 of 0.15g, or 0.10g.

5.3.2 Wall Backfill

Free-draining granular backfill, meeting Standard Specification 704.04, should be utilized behind walls.

5.4 Special Foundation Treatment – El Portal Overlook Wall

Considering the site soil conditions and soil characteristics at the planned guard wall located at the El Portal Overlook located along Glacier Point Road Stations 96+43 to 98+56, left, it is recommended that the upper 2.5 feet of soils measured from the bottom of the wall footing culvert be replaced with foundation fill materials. The intent of the soil replacement is to reduce the potential for settlement and differential settlement. However, the risk for settlement does exist for this limited removal of the soft soils. The excavation should be performed in accordance with Section 209 of the Standard Specifications.

The recommended soil improvement is to subexcavate and remove the native soils to a depth of 2.5 feet below the bottom of the wall footing, and extending downward and laterally outward from the bottom edges of the footing at a slope of 1:1 (V:H). The native soils should be replaced with material meeting Standard Specifications Section 704.01 – Foundation Fill. The backfill should be placed and compacted in accordance with the procedures of Section 208.10 and 208.11 of the Standard Specifications.

5.5 Earthwork Factors

The following excavation factors are recommended:

Station	Excavation Factor
Wawona Road	
Sta. 14+08 to Sta. 16+50	5% Swell (Intact Rock)
Sta. 16+50 to Sta. 30+55	20% Shrink (Fill and Residual Soils)
Glacier Point Road	
Sta. 1+03 to Sta. 268+08	5% Shrink (Subgrade Overexcavation)
Badger Pass Ski Area	
Sta. 64+67 to Sta. 69+98	5% Shrink (Subgrade Overexcavation)
Parking Areas	5% Shrink (Subgrade Overexcavation)

5.6 Excavation & Temporary Cut Slopes

Unprotected permanent cut slopes in soils should be constructed no steeper than 2.5H:1V (horizontal to vertical).

Temporary slopes of 1.5H:1V generally are recommended for excavation in the near-surface fill soils and residual highly weathered rock. These materials classify as Type C soils by the OSHA classification method.

Cuts in the intact weathered granite are recommended to be no steeper than 0.75H:1V.

Appropriate OSHA and California Division of Occupational Safety regulations concerning excavation safety must be followed.

5.7 Fill Slopes

Fill slopes are recommended to be no steeper than 2.5H:1V.

5.8 Glacier Road Slope Inventory Review Results

AMEC has reviewed the Rock Scaling Data Sheets prepared by Kleinfelder (2006), and observed the general conditions of the existing rock slopes along Glacier Point Road between the Chinquapin Intersection and the Badger Pass Ski Area. AMEC is in general agreement with the information and recommendations provided in the Kleinfelder report with the following additions:

- **Sta. 20+60 to Sta. 25+15, right.** The rock slope located along this segment of roadway is treated as three separate rock slopes (Sta. 20+60 to 21+15, 22+40 to 23+35 and 24+00 to 25+15). Rock outcrops also are present between the listed stations, and it is recommended that the entire length of roadway between Sta. 20+60 and 25+15 be treated as a single rock slope when performing remedial activities.
- **Sta. 28+70 to Sta. 30+30, right.** Hand scaling is recommended for this approximately 20-foot high slope, especially along the crown of the slope.
- **Sta. 84+00 to Sta. 86+00, right.** Scaling activities should be extended to include the portion of the rock slope between Sta. 84+00 to 84+70.
- **Sta. 111+00 to Sta. 112+70, right.** Hand scaling is recommended for this approximately 25-foot high slope, especially along the crown of the slope.
- **Sta. 227+20 to Sta. 229+00, right.** Minor hand scaling is recommended for this approximately 20-foot high slope.

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APPENDIX A

FIELD INVESTIGATION

Photograph 1. View looking northwest at drill rig situated at Boring C-1 located at top of existing road cut.

TEST DRILLING EQUIPMENT AND PROCEDURES

Description of Subsurface Exploration Methods

Auger Boring Drilling through overburden soils is performed with 6 5/8-inch O.D., 3 1/4-inch I.D. hollow stem auger or 4 1/2-inch solid stem continuous flight auger. Carbide insert teeth are normally used on bits so they can penetrate soft rock or very strongly cemented soils. A CME-75 truck-mounted drill rig is used to advance the auger. The drill rigs are powered with six-cylinder Cummins diesel engines capable of delivering about 11.4 kN-m torque to the drill spindle. The spindle is advanced with twin hydraulic rams capable of exerting 90 kN (20,000 pounds) downward force.

Generally, refusal to penetration of the auger is adopted as top of the SGC or "river-run" material or harder bedrock, which require other techniques for penetration. Grab samples or auger cuttings may be taken as necessary. Standard penetration tests or 2.42-inch diameter ring samples are taken in conjunction with the auger borings as needed, with the sampling interval and type being indicated on the boring logs.

Hammer Drill Drilling with the Hammer drill is accomplished with a Drill Systems AP-1000 drill rig advancing a double-walled drive casing with a link-belt 180 diesel pile driving hammer, having a rated energy of 8,100 foot-pounds per blow. Where noted on the boring log, the hammer is equipped with a supercharger which can boost the energy to approximately 12,000 foot-pounds per blow. The supercharger is used only in portions of the boring where blow counts are relatively high. Cuttings are removed with compressed air by a reverse circulation process, and are collected in a cyclone from which grab samples are obtained. The drive casing is either 9-inch O.D. by 6-inch I.D. or 6 5/8-inch O.D. by 4-inch I.D. and employs an expendable bit of slightly larger diameter than the O.D. of the casing. Hammer blows required to advance the drive casing are recorded in 1-foot increments, as noted on the boring logs. Standard penetration tests or 2.42-inch diameter ring samples taken are noted on the boring logs.

Core Boring Rock core samples are retrieved using a CME-75 drill rig, SAITECH GH 3 rig or Burley 2500, 4500 or 4000. The GH 3 is a portable hydraulic core drill. The GH 3 is powered by a Kohler two-cylinder 25-horsepower engine. The hydraulics motor which feeds a two-speed transmission and powers the BW spindle. This unit has a 3-foot stroke and is hand-fed with a 2,000 pound push-pull capability. The GH 3 has the capability of drilling with either B- or N-size core steel using standard or wireline systems. N-size core is the preferred size and it has a nominal O.D. of about 2 inches. The Burley 2500 and 4500 series are portable hydraulic core drills. The 4500 series is capable of a track-mounted or skid-type chassis. The Burley 2500 and 4500 series are powered by 44 and 75 HP power units, respectively, provide up to 2,000 foot-pounds (ft.-lbs.) of torque and in excess of 1,000 revolutions per minute (RPM) of spindle speed. Both rigs are capable of retrieving either N- or H-sized core using wireline systems. The N-size core has a nominal O.D. of about 2 inches and the H-size of about 2.4 inches. The Burley 4000 is a track-mounted core drill.

The CME-75 utilizes a wireline core drilling system that takes N-size cores. Using the NQ wireline system, core is recovered quickly by retrieving the core-laden inner tube through the drill string.

TEST DRILLING EQUIPMENT AND PROCEDURES (Cont.)

Sampling Procedures Dynamically driven tube samples are usually obtained at selected intervals in the borings by the ASTM D1586 test procedure. In many cases, 2-inch O.D., 1 3/8-inch I.D. samples are used to obtain the standard penetration resistance. "Undisturbed" samples of firmer soils are often obtained with 3-inch O.D. samples lined with 2.42-inch I.D. brass rings. The driving energy is generally recorded as the number of blows of a 140-pound, 30-inch free fall drop hammer required to advance the samples in 6-inch increments. However, in stratified soils, driving resistance is sometimes recorded in 2- or 3-inch increments so that soil changes and the presence of scattered gravel or cemented layers can be readily detected and the realistic penetration values obtained for consideration in design. These values are expressed in blows per 6 inches on the boring logs. "Undisturbed" sampling of softer soils is sometimes performed with thin walled Shelby tubes (ASTM D1587), pitcher samplers, Denison samplers or continuous CME samplers. Where samples of rock are required, they are obtained by NQ diamond core drilling (ASTM D2113). Tube samples are labeled and placed in watertight containers to maintain field moisture contents for testing. When necessary for testing, larger bulk samples are taken from auger cuttings. Also, representative samples are obtained from the cuttings from the hammer and Schramm drill rig.

Boring Records Drilling operations are directed by our field engineer or geologist who examines soil recovery and prepares the boring logs. Soils are visually classified in accordance with the Unified Soil Classification System (ASTM D2487), with appropriate group symbols being shown on the boring logs.

TERMINOLOGY USED TO DESCRIBE THE RELATIVE DENSITY, CONSISTENCY OR FIRMNESS OF SOILS

The terminology used on the boring logs to describe the relative density, consistency or firmness of soils relative to the standard penetration resistance is presented below. The standard penetration resistance (N) in blows per foot is obtained by the ASTM D1586 procedure using 2" O.D., 1 3/8" I.D. samplers.

1. **Relative Density.** Terms for description of relative density of cohesionless, uncemented sands and sand-gravel mixtures.

N	Relative Density
0-4	Very loose
5-10	Loose
11-30	Medium dense
31-50	Dense
50+	Very dense

2. **Relative Consistency.** Terms for description of clays which are saturated or near saturation.

N	Relative Consistency	Remarks
0-2	Very soft	Easily penetrated several inches with fist.
3-4	Soft	Easily penetrated several inches with thumb.
5-8	Medium stiff	Can be penetrated several inches with thumb with moderate effort.
9-15	Stiff	Readily indented with thumb, but penetrated only with great effort.
16-30	Very stiff	Readily indented with thumbnail.
30+	Hard	Indented only with difficulty by thumbnail.

3. **Relative Firmness.** Terms for description of partially saturated and/or cemented soils which commonly occur in the Southwest including clays, cemented granular materials, silts and silty and clayey granular soils.

N	Relative Firmness
0-4	Very soft
5-8	Soft
9-15	Moderately firm
16-30	Firm
31-50	Very firm
50+	Hard

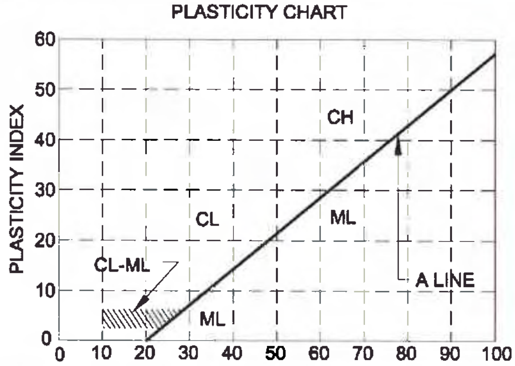
UNIFIED CLASSIFICATION SYSTEM FOR SOILS

Soils are visually classified by the United Soil Classification System on the boring logs presented in this report. Grain-size analysis and Atterberg Limits Tests are often performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. For a more detailed description of the system, see " The Unified Soil Classification System " ASTM Designation: D2487

MAJOR DIVISION				GRAPH SYMBOL	GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	**GRAVELS** (50% or less of coarse fraction passes No. 4 sieve)	**CLEAN GRAVELS** (Less than 5% passes No. 200 sieve)			GW	Well graded gravels, gravel-sized mixtures or sand-gravel-cobble mixture.
					GP	Poorly graded gravels, gravel-sized mixtures or sand-gravel-cobble mixture.
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		GM	Silty gravels, gravel-sand-silt mixture.
			Limits plot below "A" line & hatched zone on plasticity chart		GC	Clayey gravels, gravel-sand-clay mixture.
	SANDS (More than 50% of coarse fraction passes No. 4 sieve)	**CLEAN SANDS** (Less than 5% passes No. 200 sieve)			SW	Well graded sands, gravelly sands.
					SP	Poorly graded sands, gravelly sands.
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		SM	Silty sands, sand-silt mixtures.
			Limits plot below "A" line & hatched zone on plasticity chart		SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	**SILTS** LIMITS PLOT BELOW "A" LINE & HATCH ZONE ON PLASTICITY CHART	**SILTS OF LOW PLASTICITY** (Liquid limit less than 50)			ML	Inorganic silts, clayey silts with slight plasticity.
		SILTS OF HIGH PLASTICITY (Liquid limit more than 50)			MH	Inorganic silts of high plasticity, silty soils, elastic silts.
	CLAYS LIMITS PLOT BELOW "A" LINE & HATCH ZONE ON PLASTICITY CHART	**CLAYS OF LOW PLASTICITY** (Liquid limit less than 50)			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		CLAYS OF HIGH PLASTICITY (Liquid limit more than 50)			CH	Inorganic clays of high plasticity, fat clays, silty and sandy clays of high plasticity.

NOTE: Coarse-grained soils with between 5% to 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone on the plasticity chart to have dual symbol.

PLASTICITY CHART



DEFINITIONS OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Boulders	Above 300mm (12in.)
Cobbles	300mm to 75mm (12in. to 3in.)
Gravel	75mm (3in.) to No. 4 sieve
Coarse gravel	75mm to 19mm (3in to 3/4in.)
Fine gravel	19mm (3/4in.) to No. 4 sieve
Sand	No. 4 to No. 200
Coarse	No. 4 to No. 10
Medium	No. 10 to No. 40
Fine	No. 40 to No. 200
Fines (silt or clay)	Below No. 200 sieve

EXPLANATION OF CORE LOG PRESENTATION
AND TERMINOLOGY FOR THE DESCRIPTION OF ROCK

I. **ROCK QUALITY DESIGNATION (RQD).** Percentage of rock core per core run which is relatively sound and unfractured and which is longer than 0.33 feet in length. Rock which is soft or weathered, closely jointed, or rock from which the core recovery is low, will have poor to fair RQD.

II. **DISCONTINUITIES**

A. **Spacing of Joints**

Code	Spacing of Joints	Descriptive Term
1	Greater than 10 ft.	Very wide
2	3 ft. - 10 ft.	Wide
3	1 ft. - 3 ft.	Moderately close
4	0.2 ft. - 1 ft.	Close
5	Less than 0.2 ft.	Very close

B. **Orientation of Joints**

Measurements presented represent dip angles from horizontal.

Symbol	Description
Rdm	Random - preferred orientation cannot be determined.

C. **Condition of Joints**

1. Roughness

Symbol	Descriptive Term	Properties
Smth	Smooth	Appears smooth and is essentially smooth to the touch. May be slickensided.
Srgh	Slightly rough	Asperities on the fracture surfaces are visible and can be distinctly felt.
Mrgh	Medium rough	Asperities are clearly visible and fracture surface feels abrasive.
Rgh	Rough	Large angular asperities can be seen. Some ridge and high side angle steps evident.
VRgh	Very rough	Near-vertical steps and ridges occur on the fracture surface.

EXPLANATION OF CORE LOG PRESENTATION
AND TERMINOLOGY FOR THE DESCRIPTION OF ROCK

C. Condition of Joints (cont.)

2. Presence or Absence of Fracture Filling Material

Symbol	Descriptive Term	Definition
Cln	Clean	No fracture filling material.
Stn	Stained	Coloration of rock only. No recognizable filling material.
Fld	Filled	Fracture filled with recognizable filling material.

III. BEDDING

Symbol	Descriptive Term	Definition
TL	Thinly laminated	Less than 0.01 ft.
L	Laminated	0.01 ft. to 0.04 ft.
ThB	Thinly bedded	0.04 ft. to 0.20 ft.
MB	Medium bedded	0.20 ft. to 2.00 ft.
TkB	Thickly bedded	More than 2.00 ft.

IV. DEGREE OF WEATHERING

Symbol	Descriptive Term Properties
Dec	Decomposed, generally soil-like, can be crumbled by hand pressure.
HiW	Highly weathered, generally rock-like, can be broken easily, but crumbles with difficulty by hand.
MdW	Moderately weathered, fabric stained rusty brown, can be indented by steel nail, breaks only with difficulty.
SlW	Slightly weathered, open discontinuities are weathered, coated, but only slight weathering of rock mass, generally not indented by steel nail.
UnW Ex Jts	Unweathered except joints, weathering limited to the surface of discontinuities; fabric is fresh throughout but most joints show rusty stain and/or soil filling material.
UnW Inc Jts	Unweathered including joints, rock mass and discontinuities are unweathered; only occasional joints show rusty stain, practically no soil filling.
UnW	Unweathered, rock mass unweathered; no staining or infilling.

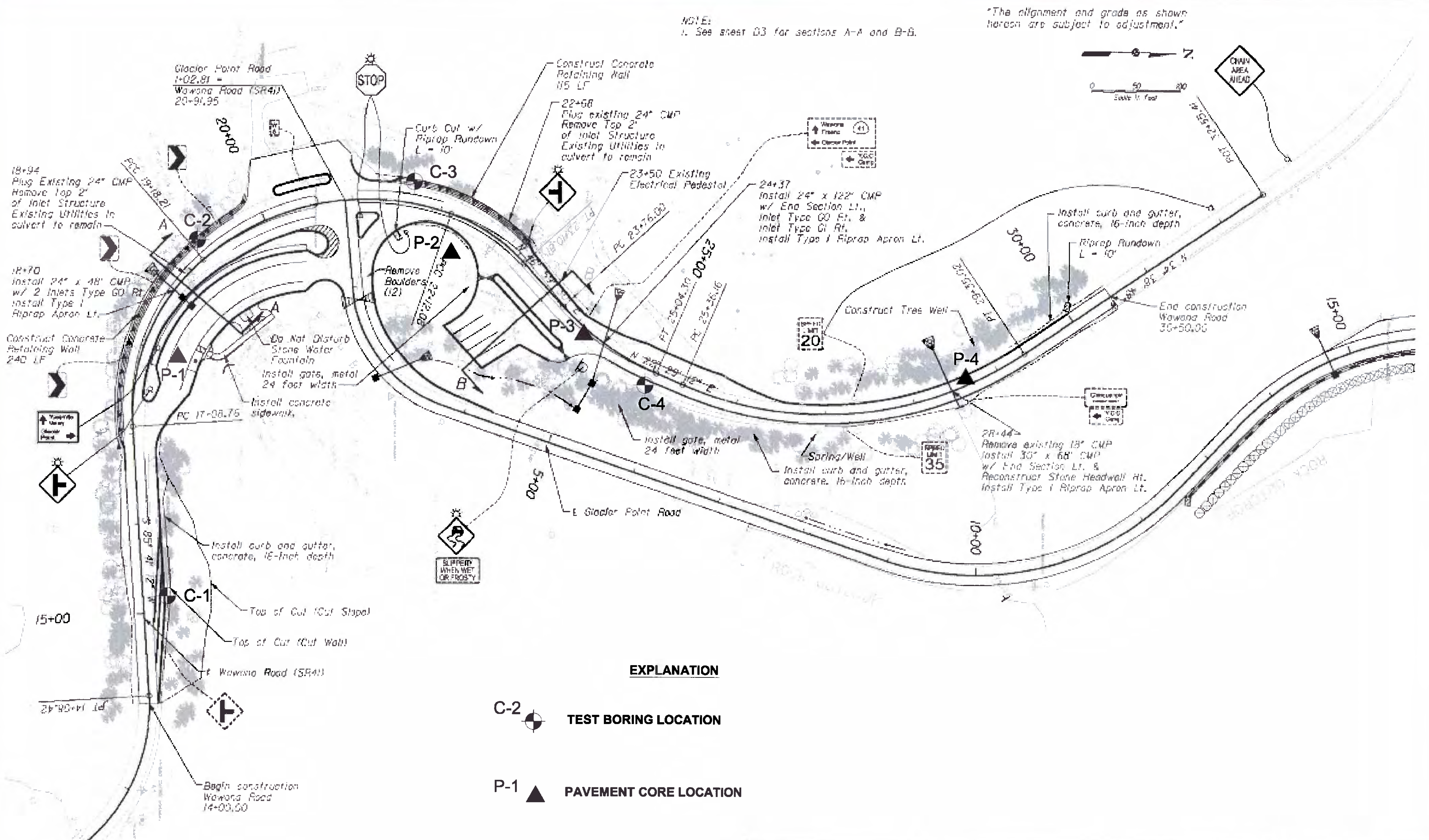
EXPLANATION OF CORE LOG PRESENTATION
AND TERMINOLOGY FOR THE DESCRIPTION OF ROCK

V. HARDNESS

Descriptive Term	Properties
Very hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately hard	Can be scratched with knife or pick. Gouges or grooves to 3 inch deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.
Moderately soft	Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1 inch maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1 inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

VI. MISCELLANEOUS ABBREVIATIONS

Symbol	Description	Symbol	Description
Bkn	Broken	Incl	Inclusions
Brc	Brecciated	Mgd	Medium-Grained
Band	Banded	Mod	Moderately
Qtz	Quartz	Wkly	Weakly
Calc	Calcite	Slicks	Slickensides
Cem	Cemented	Strong	Strongly
Frct	Fractured	SZ	Shear Zone
Fgd	Fine-Grained	Gog	Gouge



BASE MAP REFERENCE: SHEET D1, CARTER & BUGESS, 2005

JOB NO. 6-117-008002	SITE PLAN SHOWING BORING LOCATIONS CHINQUAPIN INTERSECTION		amec
DESIGN: TJF			
DRAWN: VF			
DATE: 05/2007	YOSEMITE NATIONAL PARK MARIPOSA COUNTY , CALIFORNIA	FIGURE 1	
SCALE: 1"=100'			

G:\Engineering Department\2006 Projects\6-117-008002 Glacier Point Road\CAD\SITE PLAN.dwg

PROJECT Glacier Point Road
Yosemite National Forest, CA

amec

JOB NO. 6-117-008002 **DATE** 6/13/07

LOCATION Glacier Point Road Station 97+00, L 25'

RIG TYPE Fraste
BORING TYPE 6" Core Barrel
SURFACE ELEV. +/-
DATUM

Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count (S) Per 6-inches (U) Per 12-inches	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification or Rock Unit	REMARKS	VISUAL CLASSIFICATION
0								GM	very soft	**FILL** **SILT WITH GRAVEL,** cobbles, uncemented, low plasticity, yellowish brown, decomposed granodiorite note: boulders to 7 ft. diameter nearby
5				S	3-2-2			ML	soft to very soft	**SILT,** low plasticity, dark brown note: roots in 5' sample
10				S	2-2-2				soft to very soft	**DECOMPOSED GRANODIORITE,** fine grained, very soft decomposes to soft, uncemented, low plasticity, brown note: wood fragments up to 1" in diameter
15				S	10-13-17					**DECOMPOSED GRANODIORITE,** fine to medium grained, very soft, yellowish brown note: possible boulder encountered from 18 to 20'
20				S	50/1"					Boring refused on boulder or bedrock at 20 feet Backfilled with cutting on 6-13-07
25										

GROUNDWATER

	DEPTH(ft)	HOUR	DATE
▽		none	
▼			
▼			
▼			

SAMPLE TYPE
A - Drill cuttings
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - Thin Walled Shelby tube sample
MC - Modified California ring sample
NR - No Recovery

LOG OF TEST BORING NO. BA-1

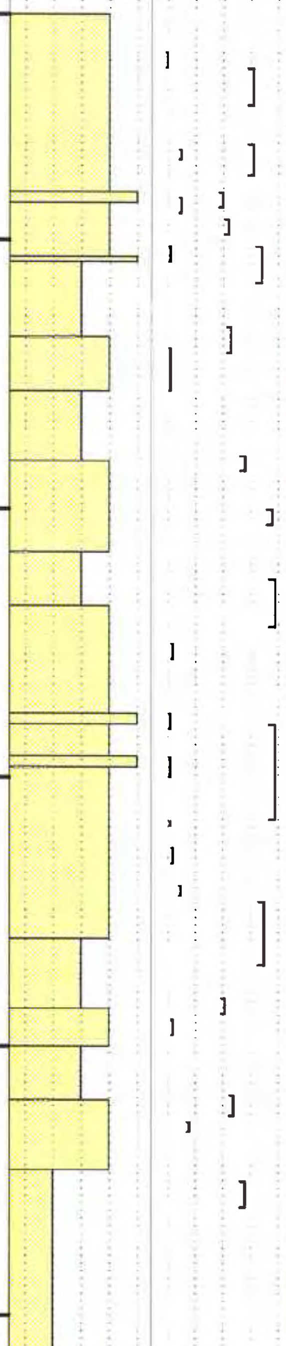
PROJECT	Chinquapin Intersection Wawona Road Yosemite National Park, California
JOB NO.	6-117-008002
DATE	11-9-06

amec

RIG TYPE	Burley 4500
METHOD	
OPERATOR	
ENGINEER	
LOCATION	Sta. 15+30, 35' R
STA/OFFSET	
ELEVATION	6065' ±
DATUM	
INCLINATION	

Boring Operation and Drill Rate (min/feet)	Depth in Feet	Sample	Sample Type	Unconfined Compression or Point Load IndexTest (psi)	% Core Recovery	% Drilling Fluid Rec.	Rock Quality Designation (RQD)
11/9 HQ3 2.9	0		HQ3		100	50	45
2.8	5		HQ3		90		55
2.4	10		HQ3		95	25	30
	15		HQ		100		70
2.9	20		HQ		100		90
2.2	25		HQ3		100		50

DISCONTINUITIES: Spacing (Wide - Close: 1 2 3 4 5); Orientation (Horiz - Vert: H 45 V)



Condition (top to bottom): MRgh Stn Fld; Smth Stn Fld; SRgh to MRgh Stn Fld; SRgh Stn; MRgh Stn Fld

Bedding and/or Fabric: Mgn

Weathering or USCS (Soils) (top to bottom): SlW; HiW; SlW-MdW; HiW; MdW; SlW

Rock Type & Remarks

GRANITE, hard, medium gray

note: fractures typically stained with orange iron-oxide & filled with brown fine grained silty sand

note: 1" diameter mafic-rich xenolith at 0.5'

note: several intersecting fractures at about 3.5'

note: occasional 1/2" to 1" diameter mafic-rich xenolith

note: 1" thick silt-filled fractures at about 4.7' & 6.5'

note: soft zones from 8.3' to 10.0' & from 11.2' to 12.7'

note: high-angle, silt-filled fracture from 10.5' to 11.3' & from 13.2' to 14.2'

note: 2 1/2" diameter mafic-rich xenolith at 12.2'

note: 1/4" to 1/2" thick, near horizontal, silt-filled fractures at 13.2', 13.4', 13.9' & 14.0'

note: fractures at 16.3' & near vertical fracture from 16.5' to 17.7' stained with manganese-oxide

note: 9" diameter mafic-rich xenolith from 17.7' to 18.6'

note: iron-oxide healed fracture at 18.1'

note: 1/2" thick high-angle fracture filled with silt at 20.3'

GROUNDWATER

DEPTH (ft)	HOUR	DATE
	*	

BORING OPERATION
A - Auger/Drill Cuttings
HQ3 - 3.8" O.D. Triple-tube Wireline Rock Coring
BWC - 2.9" O.D. Casing
GB - 3" or 6 1/8" Gear Bit
NV - No Value
NR - No Recovery

LOG OF TEST BORING NO. C-1

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

JOB NO. 6-117-008002 **DATE** 11-9-06

amec

RIG TYPE	Burley 4500
METHOD	
OPERATOR	
ENGINEER	
LOCATION	Sta. 15+30 35' R
STA/OFFSET	
ELEVATION	6065' ±
DATUM	
INCLINATION	

Boring Operation and Drill Rate (min/feet)	Depth in Feet	Sample	Sample Type	Unconfined Compression or Point Load IndexTest (psi)	% Core Recovery	% Drilling Fluid Rec.	Rock Quality Designation (RQD)	DISCONTINUITIES Spacing (Wide - Close, 1 2 3 4 5)	DISCONTINUITIES Orientation (Horiz - Vert, H 45 V)	Condition	Bedding and/ or Fabric	Weathering or USCS (Soils)	Rock Type & Remarks
11/9 HQ3 2.2	25					25				MRgh Stn Fld	Mgn	SlW	**GRANITE,** continued
												HiW	note: iron-oxide healed fracture at 25.2'
												SlW	note: soft zone from 25.8' to 26.5'; moderately soft below 27.3'
												MdW	note: series of low-angle 1/4" to 1/2" thick silt-filled fractures below 27.5'
	30												Stopped HQ3 Coring at 29.2'
													* Water was introduced into the boring as part of the drilling operation, so the presence of natural groundwater could not be determined. However it is likely that the boring was completed above the water table.
	35												
	40												
	45												
	50												

GROUNDWATER

DEPTH (ft)	HOUR	DATE
	*	

BORING OPERATION
A - Auger/Drill Cuttings
HQ3 - 3.8" O.D. Triple-tube Wireline Rock Coring
BWC - 2.9" O.D. Casing
GB - 3" or 6 1/8" Gear Bit
NV - No Value
NR - No Recovery

LOG OF TEST BORING NO. C-1

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

amec

JOB NO. 6-117-008002 DATE 11/7/06

LOCATION Sta. 19+20, 16' L
RIG TYPE Burley 4500
BORING TYPE 4 1/2" HWT Tricone
SURFACE ELEV. 6037' ±
DATUM

Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count (S) Per 6-inches (U) Per 12-inches	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification or Rock Unit	REMARKS	VISUAL CLASSIFICATION
0										4" Asphaltic Concrete
								ML	loose	**FILL** **SANDY SILT,** predominantly fine grained sand, uncemented, low plasticity, medium brown note: contains considerable mica note: increase in fine grained sand with depth
				S	6-3-4					
5								SM	loose very loose	**FILL** **SILTY SAND,** trace of gravel, predominantly fine to medium grained, angular to subangular, uncemented, nonplastic to low plasticity, dark brown note: some rootlets
				S	2-3-2		28			
										DECOMPOSED GRANITE, medium grained, very soft note: decomposes to silty, fine to medium grained sand, nonplastic, medium brown
10				S	50/3"					
										Stopped HWT Drilling at 10' Sampler refused at 10'3" Started HQ3 Coring at 10'3" * Water was introduced into the boring as part of the drilling operation, so the presence of natural groundwater could not be determined. However, it is likely the the boring was completed well above the water table.
15										
20										
25										

GROUNDWATER

	DEPTH(ft)	HOUR	DATE
∇		*	
▼			
∇			
∇			

SAMPLE TYPE
A - Drill cuttings
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - Thin Walled Shelby tube sample
MC - Modified California ring sample
NR - No Recovery

LOG OF TEST BORING NO. C-2

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

amec

JOB NO. 6-117-008002 **DATE** 11-7-06

RIG TYPE	Burley 4500
METHOD	
OPERATOR	
ENGINEER	
LOCATION	Sta. 19+20, 16' L
STA/OFFSET	
ELEVATION	6037' ±
DATUM	
INCLINATION	

Boring Operation and Drill Rate (min/feet)	Depth in Feet	Sample	Sample Type	Unconfined Compression or Point Load IndexTest (psi)	% Core Recovery	% Drilling Fluid Rec.	Rock Quality Designation (RQD)	DISCONTINUITIES Spacing Wide - Close 1 2 3 4 5	DISCONTINUITIES Orientation Horiz - Vert H 45 V	Condition	Bedding and/ or Fabric	Weathering or USCS (Soils)	Rock Type & Remarks
													Started HQ Coring at 10.2'
11/7 HQ3 2.5	10		HQ3		100	100	0			SRgh part Fld with silt	Mgn	Dec	**GRANITE,** soft from 10.2' to 13.0', then moderately hard with occasional hard zones to 25', light gray note: breaks down to silty fine to medium grained sand with firm hand pressure; individual crystals can be reduced to silt-sized material
												Dec to HiW	
3.6			HQ3		100		0						
4.0	15		HQ3		100		0						
3.8			HQ3		0		0						
												HiW	
						0							
						100							note: 5" thick silt-filled fracture at 18.5', some orange iron-oxide staining at both ends of fracture note: fractures typically stained with iron-oxide below 20.0'
4.4	20		HQ3		100		65			Stn Fld		MdW	note: moderately hard to hard with occasional soft zones below 20.0'
						~~0~~							
						100							note: 1/2" thick silt-filled fractures at 22.5' note: near vertical fracture filled with silt from 23.5' to 24.5'
						~~0~~						~~Dec~~	
						100						MdW	note: soft 2" thick zone at 23.5' note: 2" diameter mafic zenolith at 24.0'
	25												Stopped HQ Coring at 25.0' Backfilled boring with cuttings & roadside fill
	30												

GROUNDWATER

DEPTH (ft)	HOUR	DATE
	*	

BORING OPERATION
A - Auger/Drill Cuttings
HQ3 - 3.8" O.D. Triple-tube Wireline Rock Coring
BWC - 2.9" O.D. Casing
GB - 3" or 6 1/8" Gear Bit
NV - No Value
NR - No Recovery

LOG OF TEST BORING NO. C-2

APPENDIX B

LABORATORY ANALYSIS RESULTS

PROJECT: Chinquapin Intersection
LOCATION: Yosemite
SAMPLE SOURCE: SEE BELOW

JOB NO: 6-117-008002
WORK ORDER NO: 1
DATE ASSIGNED: 1/11/07

MECHANICAL SIEVE ANALYSIS
GROUP SYMBOL, USCS (ASTM D-2487)

PERCENT PASSING BY WEIGHT

				Silt or Clay	SAND								GRAVEL									COBBLES	
					Fine			Medium			Coarse		Fine				Coarse						
Location & Depth	USCS	LL	PI	#200	#100	#50	#40	#30	#16	#10	#8	#4	1/4"	3/8"	1/2"	3/4"	1"	1 1/4"	1 1/2"	2"	3"	6"	Lab #
C-2 @ 5'0" - 6'6"	SM	NV	NP	22	36	55	66	74	85	91	92	96	97	97	100	100	100	100	100	100	100	100	6
C-3 @ 5'0" - 6'6"	SM	NV	NP	27	40	57	67	74	86	94	95	99	100	100	100	100	100	100	100	100	100	100	11
C-3 @ 10'0" - 11'6"	SM	NV	NP	25	38	57	68	77	91	97	98	100	100	100	100	100	100	100	100	100	100	100	12
C-3 @ 20'0" - 21'6"	SM	NV	NP	21	34	52	62	72	89	97	98	100	100	100	100	100	100	100	100	100	100	100	14
C-4 @ 5'0" - 6'6"	SC	29	10	20	31	49	61	72	90	97	99	100	100	100	100	100	100	100	100	100	100	100	19
C-4 @ 15'0" - 16'6"	SM	NV	NP	44	59	72	79	85	95	98	99	100	100	100	100	100	100	100	100	100	100	100	21
P-1 @ 1'6" - 3'0"	SM	NV	NP	28	40	56	65	71	83	89	91	94	94	95	95	100	100	100	100	100	100	100	25
P-2 @ 0'4 1/2" - 0'9 1/2"	GP-GM	35	8	6.2	8	12	13	15	21	22	22	23	23	26	35	63	92	100	100	100	100	100	28
P-3 @ 0'10" - 1'6"	SM	31	5	23	32	42	48	52	58	59	60	64	66	70	78	92	94	100	100	100	100	100	33
P-4 @ 1'6" - 3'0"	SC-SM	29	7	21	28	36	42	47	59	70	73	84	87	91	96	100	100	100	100	100	100	100	36

AASHTO R18

REVIEWED BY

PROJECT:	Chinquapin Intersection	**JOB NO:**	6-117-008002
LOCATION:	Yosemite	**WORK ORDER NO:**	1
MATERIAL:	Soil	**LAB NO:**	See Below
SAMPLE SOURCE:	See Below	**DATE ASSIGNED:**	1/11/07

MOISTURE CONTENT OF SOIL (ASTM D2216)

LAB #	BORING & DEPTH	WET WT. (gram)	DRY WT. (gram)	MOISTURE CONTENT
6	C-2 @ 5'0" - 6'6"	206.8	162.1	27.6%
11	C-3 @ 5'0" - 6'6"	183.2	147.9	23.8%
12	C-3 @ 10'0" - 11'6"	288.4	233.3	23.6%
14	C-3 @ 20'0" - 21'6"	221.9	172.2	28.9%
19	C-4 @ 5'0" - 6'6"	264.8	220.8	19.9%
21	C-4 @ 15'0" - 16'6"	659.2	464.7	41.9%

AASHTO R18

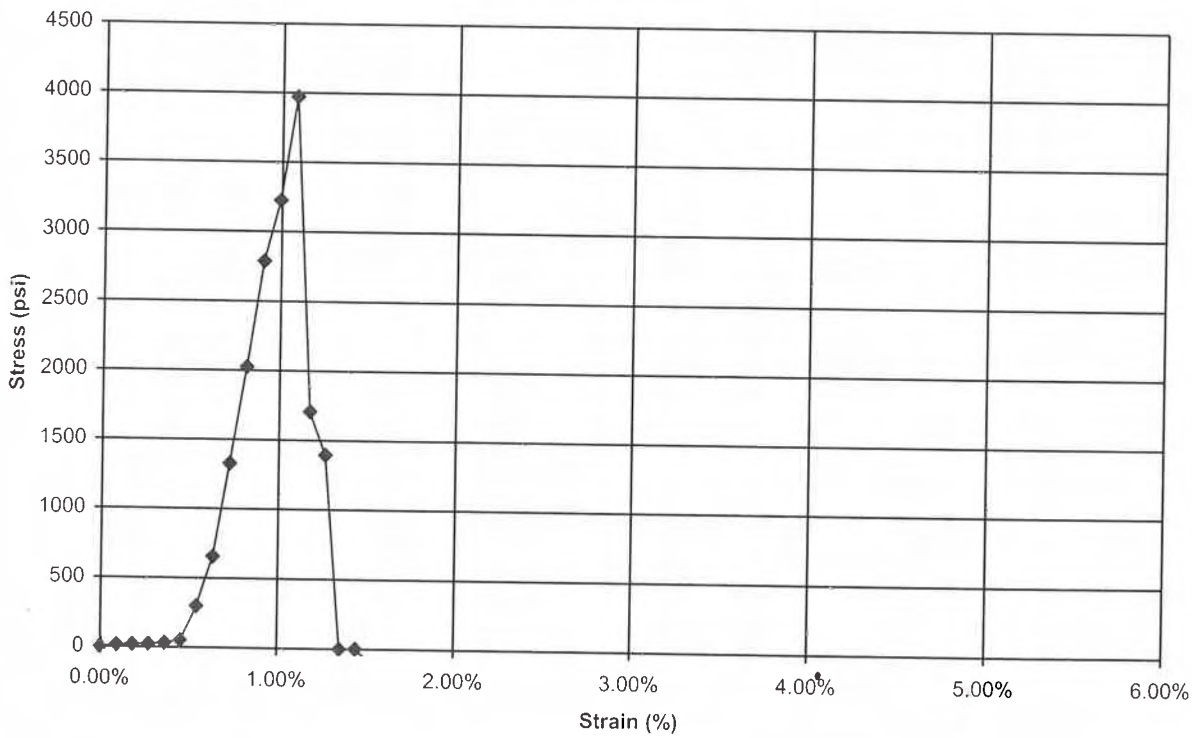
REVIEWED BY

PROJECT:	Chinquapin Intersection	**JOB NO:**	6-117-008002
LOCATION:	Yosemite	**WORK ORDER NO:**	1
MATERIAL:	Soil	**LAB NO:**	1A
SAMPLE SOURCE:	C-1 @ 0.0-8.8'	**DATE ASSIGNED:**	01/11/07
SAMPLE PREP:	INSITU		

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS
(ASTM D2938)

DIAMETER (D):	2.39 in	STRAIN RATE:	.18 inches/min.
HEIGHT (L):	5.54 in	TOTAL STRAIN:	1.08%
L/D (2.0-2.5 REQ.):	2.32		
DRY DENSITY:	161.2 lb/cu.ft	**UNCONFINED COMPRESSIVE STRENGTH:**	**3,970** (psi)
MOISTURE CONTENT:	1.1%		

SPECIMEN AIR DRIED UNTIL TIME OF TEST



AAP AASHTO R18

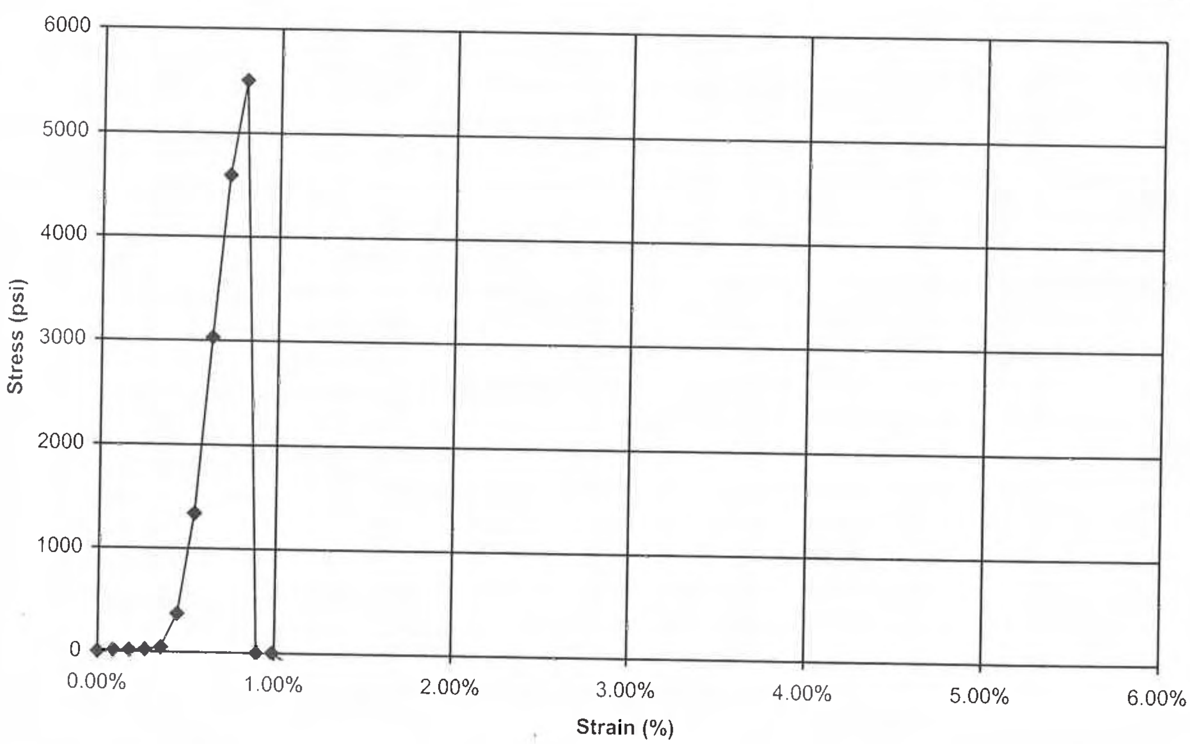
REVIEWED BY [signature]

PROJECT:	Chinquapin Intersection	**JOB NO:**	6-117-008002
LOCATION:	Yosemite	**WORK ORDER NO:**	1
MATERIAL:	Soil	**LAB NO:**	1B
SAMPLE SOURCE:	C-1 @ 0.0-8.8'	**DATE ASSIGNED:**	01/11/07
SAMPLE PREP:	INSITU		

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS
(ASTM D2938)

DIAMETER (D):	2.39 in	STRAIN RATE:	.28 inches/min.
HEIGHT (L):	5.57 in	TOTAL STRAIN:	0.81%
L/D (2.0-2.5 REQ.):	2.33		
DRY DENSITY:	162.2 lb/cu.ft	**UNCONFINED COMPRESSIVE STRENGTH:**	**5,503** (psi)
MOISTURE CONTENT:	1.0%		

SPECIMEN AIR DRIED UNTIL TIME OF TEST



AASHTO R18

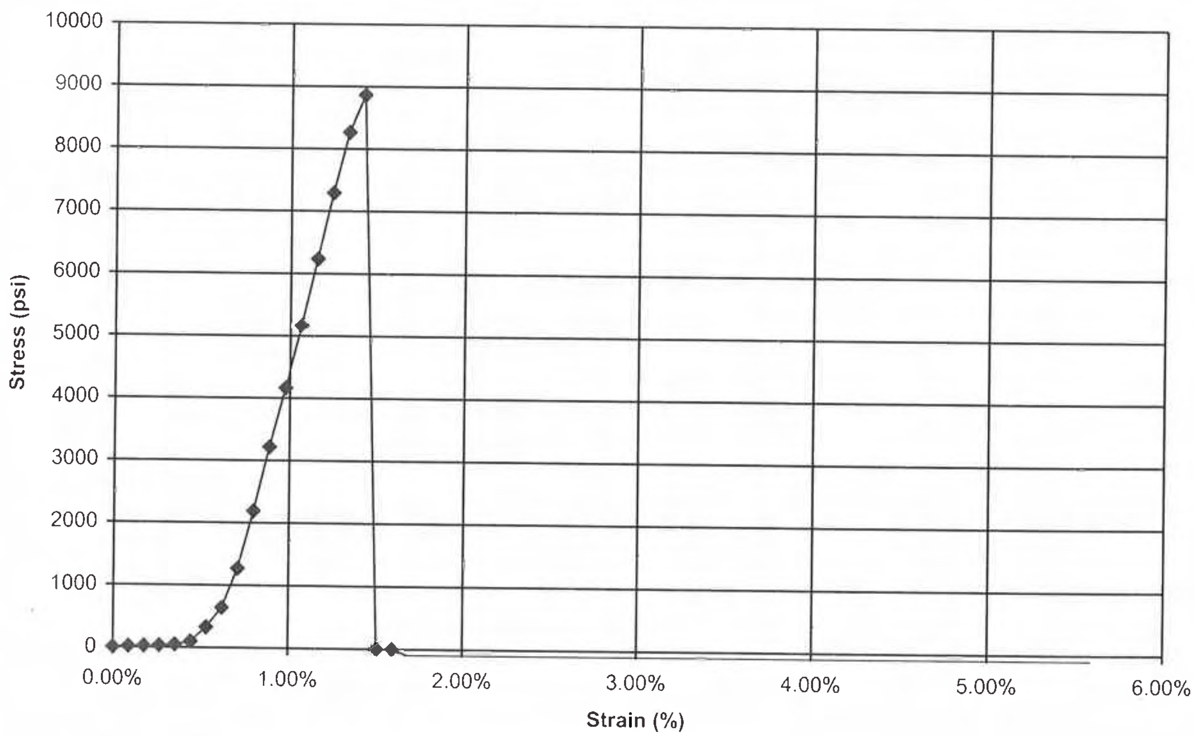
REVIEWED BY

PROJECT:	Chinquapin Intersection	**JOB NO:**	6-117-008002
LOCATION:	Yosemite	**WORK ORDER NO:**	1
MATERIAL:	Soil	**LAB NO:**	2a
SAMPLE SOURCE:	C-1 @ 8.8-16.5'	**DATE ASSIGNED:**	01/11/07
SAMPLE PREP:	INSITU		

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS (ASTM D2938)

DIAMETER (D):	2.39 in	STRAIN RATE:	.28 inches/min.
HEIGHT (L):	5.64 in	TOTAL STRAIN:	1.42%
L/D (2.0-2.5 REQ.):	2.36		
DRY DENSITY:	165.6 lb/cu.ft	**UNCONFINED COMPRESSIVE STRENGTH:**	**8,865** (psi)
MOISTURE CONTENT:	0.5%		

SPECIMEN AIR DRIED UNTIL TIME OF TEST



AASHTO R18

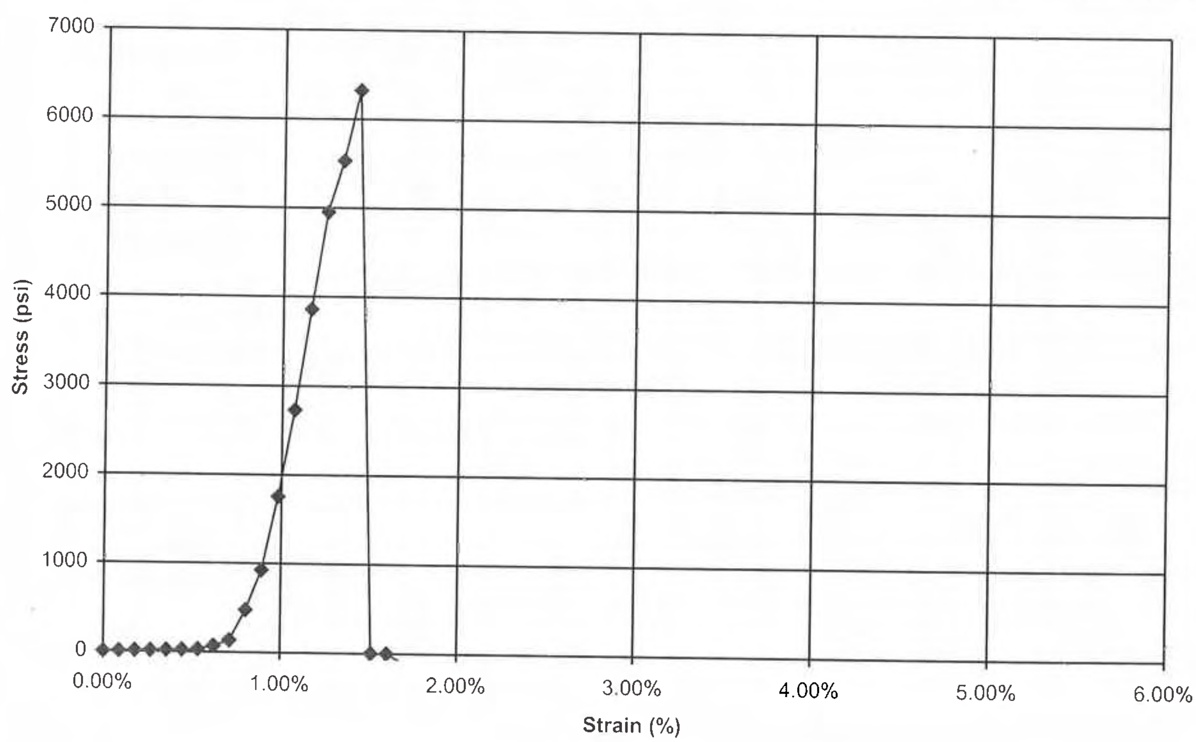
REVIEWED BY [signature]

PROJECT:	Chinquapin Intersection	**JOB NO:**	6-117-008002
LOCATION:	Yosemite	**WORK ORDER NO:**	1
MATERIAL:	Soil	**LAB NO:**	2b
SAMPLE SOURCE:	C-1 @ 8.8-16.5'	**DATE ASSIGNED:**	01/11/07
SAMPLE PREP:	INSITU		

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS
(ASTM D2938)

DIAMETER (D):	2.39 in	STRAIN RATE:	.22 inches/min.
HEIGHT (L):	5.61 in	TOTAL STRAIN:	1.43%
L/D (2.0-2.5 REQ.):	2.35		
DRY DENSITY:	165.1 lb/cu.ft	**UNCONFINED COMPRESSIVE STRENGTH:**	**6,323** (psi)
MOISTURE CONTENT:	0.9%		

SPECIMEN AIR DRIED UNTIL TIME OF TEST



AAP AASHTO R18

REVIEWED BY

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

amec

JOB NO. 6-117-008002 DATE 11/7/06

LOCATION Sta. 22+65, 19' L
RIG TYPE Burley 4500
BORING TYPE 4 1/2" HWT Tricone
SURFACE ELEV. 6036' ±
DATUM

Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count (S) Per 6-inches (U) Per 12-inches	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification or Rock Unit	REMARKS	VISUAL CLASSIFICATION
0										5" Asphaltic Concrete
								SM	moist to very moist loose	**FILL** **SILTY SAND,** trace of gravel, predominantly fine to medium grained, angular to subangular, uncemented, nonplastic, dark brown note: trace roots & pine needles note: denser zone above 3'
				S	12-5-5					
5				S	2-1-4		24			
								SP & SM	very moist loose	**FILL** **SAND,** considerable silt, trace of gravel, predominantly fine to medium grained, angular to subangular, uncemented, nonplastic, light brown note: grades locally to silty sand
10				S	3-3-3		24			
15				S	2-2-3					note: sample at 15' appears to have relict granitic fabric
									very moist loose	**Residual Soil** **SILTY SAND,** fine to medium grained, angular to subangular, nonplastic, medium brown note: some roots at 20'
20				S	2-2-3		29			
25										

GROUNDWATER

	DEPTH(ft)	HOUR	DATE
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▽			
▽			

SAMPLE TYPE
A - Drill cuttings
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - Thin Walled Shelby tube sample
MC - Modified California ring sample
NR - No Recovery

LOG OF TEST BORING NO. C-3

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

amec

JOB NO. 6-117-008002 **DATE** 11/7/06

LOCATION Sta. 22+65, 19' L

RIG TYPE Burley 4500
BORING TYPE 4 1/2" HWT Tricone
SURFACE ELEV. 6036' ±
DATUM

Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count (S) Per 6-inches (U) Per 12-inches	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification or Rock Unit	REMARKS	VISUAL CLASSIFICATION
25			X	S	3-3-3				loose to medium dense	**SILT,** with some fine grained sand, nonplastic, light gray
30			X	S	9-6-7					note: original granitic fabric well defined in sample at 30'
35			X	S	4-6-9					note: fine grained mafic-rich in sample at 35'
										Stopped HWT Drilling at 35' Stopped Sampler at 36'6" * Water was introduced into the boring as part of the drilling operation, so the presence of natural groundwater could not be determined. However, it is likely the the boring was completed well above the water table.
40										
45										
50										

GROUNDWATER

	DEPTH(ft)	HOUR	DATE
▽		*	
▼			
▽			
▽			

SAMPLE TYPE
A - Drill cuttings
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - Thin Walled Shelby tube sample
MC - Modified California ring sample
NR - No Recovery

LOG OF TEST BORING NO. C-3

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

amec

JOB NO. 6-117-008002 DATE 11/6/06

LOCATION Sta. 25+15, 20' R

RIG TYPE Burley 4500
BORING TYPE 4 1/2" HWT Tricone
SURFACE ELEV. 6033' ±
DATUM

Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count (S) Per 6-inches (U) Per 12-inches	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification or Rock Unit	REMARKS	VISUAL CLASSIFICATION
0										4" Asphaltic Concrete 6" Aggregate Base Course
								SM	medium dense	**SANDY GRAVEL,** predominantly fine to medium grained, angular, fine to medium grained sand, nonplastic, dark grayish-brown
			X	S	21-24-21				moist to very moist dense to loose	**Residual Soil** **SILTY SAND,** fine to medium grained, angular to subangular, nonplastic, light gray note: 3" thick harder layer at approximately 4' note: 1 1/2" thick mica-rich silt layer at 5'6"
5			X	S	2-7-6		20			
			X	S	3-4-5					
10			X	S	3-3-4					
			X	S	2-3-5				very moist loose	note: predominantly fine grained & brown below 12' note: samples readily disentegrate to mica-rich silt with greasey consistency
15			X	S	2-2-4		42			
20			X	S	1-3-5					
										Stopped HWT Drilling at 20' Stopped Sampler at 21'6" * Water was introduced into the boring as part of the drilling operation, so the presence of natural groundwater could not be determined. However, it is likely the the boring was completed well above the water table.
25										

GROUNDWATER

	DEPTH(ft)	HOUR	DATE
▽		*	
▼			
▽			
▽			

SAMPLE TYPE
A - Drill cuttings
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - Thin Walled Shelby tube sample
MC - Modified California ring sample
NR - No Recovery

LOG OF TEST BORING NO. C-4

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

JOB NO. 6-117-008002 DATE 11/8/06

LOCATION Parking Area
Sta. 17+80, 25' R
RIG TYPE Burley 4500
BORING TYPE HWT Casing Advancer
SURFACE ELEV.
DATUM

Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count (S) Per 6-inches (U) Per 12-inches	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification or Rock Unit	REMARKS	VISUAL CLASSIFICATION
0				P						6" Asphaltic Concrete
				G						14" Aggregate Base Course **SANDY GRAVEL,** predominantly 1/4" to 1 1/2" diameter, angular gravel, well graded, angular to subangular sand, nonplastic, medium brown
				U	14-16-12			SM		**SILTY SAND,** predominantly fine to medium grained, nonplastic to low plasticity, medium brown
										Stopped HWT Drilling at 1'6" Stopped Sampler at 3' * Water was introduced into the boring as part of the drilling operation, so the presence of natural groundwater could not be determined. However, it is likely the the boring was completed well above the water table.
5										
10										
15										
20										
25										

GROUNDWATER

	DEPTH(ft)	HOUR	DATE
∇		*	
▼			
∇			
∇			

SAMPLE TYPE
A - Drill cuttings
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - Thin Walled Shelby tube sample
MC - Modified California ring sample
NR - No Recovery

LOG OF TEST BORING NO. P-1

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

JOB NO. 6-117-008002 **DATE** 11/8/06

LOCATION Parking Area
Sta. 22+00, 40' R
RIG TYPE Burley 4500
BORING TYPE HWT Casing Advancer
SURFACE ELEV.
DATUM

Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count (S) Per 6-inches (U) Per 12-inches	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification or Rock Unit	REMARKS	VISUAL CLASSIFICATION
0				P						4 1/2" Asphaltic Concrete
				G						9 1/2" Aggregate Base Course
				G						**SANDY GRAVEL,** predominantly 1/4" to 1 1/2" diameter, angular gravel, predominantly fine to medium grained sand, nonplastic, dark grayish-brown
				U	5-7-7					**CLAYEY SILT,** considerable fine grained sand, low to medium plasticity, medium brown note: considerable mica
										SILTY SAND, predominantly fine grained, nonplastic to low plasticity, medium brown
5										Stopped HWT Drilling at 1'6" Stopped Sampler at 3' note: asphaltic concrete at first location did not remain intact; thickness varied from 3 1/2" to 4"; moved 30' south & tried again * Water was introduced into the boring as part of the drilling operation, so the presence of natural groundwater could not be determined. However, it is likely the the boring was completed well above the water table.
10										
15										
20										
25										

GROUNDWATER

	DEPTH(ft)	HOUR	DATE
▽		*	
▼			
▽			
▽			

SAMPLE TYPE
A - Drill cuttings
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - Thin Walled Shelby tube sample
MC - Modified California ring sample
NR - No Recovery

LOG OF TEST BORING NO. P-2

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

amec

JOB NO. 6-117-008002 DATE 11/8/06

LOCATION Sta. 24+20, 7' L

RIG TYPE Burley 4500
BORING TYPE HWT Casing Advancer
SURFACE ELEV.
DATUM

Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count (S) Per 6-inches (U) Per 12-inches	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification or Rock Unit	REMARKS	VISUAL CLASSIFICATION
0				P						5 1/2" Asphaltic Concrete
										10" Aggregate Base Course
				G						**SANDY GRAVEL,** predominantly 1/4" to 1" diameter, angular gravel, well graded, angular to subangular sand, nonplastic, dark grayish-brown
				S	10-7-7			SM		**CLAYEY SAND,** considerable to some fine grained gravel, predominantly fine grained, low to medium plasticity, medium brown note: decrease in gravel with depth
										SILTY SAND, predominantly fine to medium grained, angular to subangular, nonplastic to low plasticity, medium brown
5										Stopped HWT Drilling at 1'6" Stopped Sampler at 3'
10										
15										
20										
25										

GROUNDWATER

	DEPTH(ft)	HOUR	DATE
▽		•	
▼			
▽			
▽			

SAMPLE TYPE
A - Drill cuttings
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - Thin Walled Shelby tube sample
MC - Modified California ring sample
NR - No Recovery

LOG OF TEST BORING NO. P-3

PROJECT Chinquapin Intersection
Wawona Road
Yosemite National Park, California

amec

JOB NO. 6-117-008002 DATE 11/8/06

LOCATION Sta. 28+50, 7' L

RIG TYPE Burley 4500
BORING TYPE HWT Casing Advancer
SURFACE ELEV.
DATUM

Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count (S) Per 6-inches (U) Per 12-inches	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification or Rock Unit	REMARKS	VISUAL CLASSIFICATION
0			xxx	P						5" Asphaltic Concrete
										10" Aggregate Base Course **SANDY GRAVEL,** predominantly 1/4" to 1" diameter, angular, gravel, well graded, angular to subangular sand, nonplastic, brown
				U	8-20-12			SC-SM		**CLAYEY TO SITLY SAND,** trace fine grained gravel, well graded, angular to subangular, nonplastic, medium brown
										Stopped HWT Drilling at 1'6" Stopped Sampler at 3'
5										
10										
15										
20										
25										

GROUNDWATER

	DEPTH(ft)	HOUR	DATE
▽			
▼			
▽			
▽			

SAMPLE TYPE
A - Drill cuttings
S - 2" O.D. 1.38" I.D. tube sample
U - 3" O.D. 2.42" I.D. tube sample
T - Thin Walled Shelby tube sample
MC - Modified California ring sample
NR - No Recovery

LOG OF TEST BORING NO. P-4

APPENDIX C

GEOPHYSICAL INVESTIGATION

REFRACTION SEISMIC EQUIPMENT AND PROCEDURES

Refraction seismic surveys are performed in general conformance with the guidelines presented in ASTM D5777-95 Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation for refraction surveys using compression waves (p-waves). ASTM D5777 does not address shear wave (s-wave) surveys; standard practice is followed for refraction surveys using s-waves. In some investigations, such as seeking and tracing earth fissures or other significant discontinuities (Rucker and Keaton, 1998), non-standard procedures and analyses, such as signal amplitude analysis, are used as part of the investigation process.

Seismic Equipment - Refraction seismic surveys are performed using a Geometrics ES-1225 or Smartseis S-12 signal enhancement seismograph. These instruments have the capability to simultaneously record 12 channels of geophone data and produce hard copies of that data. The Smartseis also has the capability of digitally storing geophone data. Signal enhancement capability permits the use of a sledgehammer as the seismic energy source. A timing sensor is attached to the hammer, and for p-waves, a metal plate is set securely on the ground surface and struck. Generating horizontally polarized s-waves typically involves setting the plate against the end of a wooden plank or railroad tie oriented perpendicular to the axis of the geophone array and striking with a horizontal motion of the sledgehammer. A truck is usually driven onto the plank or tie to effectively couple the plank or tie to the ground.

Because of the signal enhancement capability, signals from several or many strikes can be added together to increase the total signal available relative to noise to obtain the seismic record. Although explosives can also be used as a p-wave seismic energy source, a sledgehammer does not require licenses or permits, or involve special limitations, regulations and liabilities. Explosive energy sources may be needed for long geophone arrays. Geophone cables with 12 geophone takeouts at 10-foot, 25-foot or 20-meter spacings are presently used. Vertical geophones are used to obtain p-wave data and horizontal geophones are used to obtain s-wave data. The seismograph system is extremely portable. In areas where vehicular access is not possible, the equipment can be mobilized by various means, including backpacking, packhorse, helicopter and canoe.

Field Procedures - The field operations are directed by our experienced engineer or geologist, who operates the equipment, prepares the records and examines the data in the field. Refraction seismic lines are generally laid out using the standard spacings on the geophone cables. A maximum depth of investigation of about 75 to 100 feet may be possible using a 300-foot array. For shorter lines with improved near-surface resolution, 10-foot spacings between geophones with a 120-foot array have a maximum depth of investigation of about 30 to 40 feet. Other geophone spacings can also be used. To improve the resolution of near-surface interfaces, energy source positions generally are set at 12.5 feet from the ends of a 25-foot spacing geophone array or at 5 feet from the ends of a 10-foot geophone spacing array. Several shots locations are utilized along the length of an array. When three shots are obtained, there is a foreshot and a backshot at the array ends and a midshot at the array center. The midshot is usually placed midway between the two centermost geophones. When five shots are obtained, the additional shotpoints are located midway between the foreshot-midshot and the midshot-backshot. These multiple shot points permit interpretation of near-surface interfaces at various locations along the array as well as near the endpoints for variable subsurface profiles, and permits more refined overall interpretations of shallow and mid-depth subsurface velocities and interfaces. In cases when both enhanced depth of investigation and improved shallow resolution are needed, multiple 12-geophone arrays are completed end to end and combined into longer composite 24- or more geophone arrays with greater depths of investigation. Additional energy shotpoints are then, at a minimum, performed at the midpoint and far endpoint of each adjacent 12-geophone array to provide seismic energy travel path coverage over the extended array.

REFRACTION SEISMIC EQUIPMENT AND PROCEDURES (Cont.)

P-wave data are recorded for general exploration work. S-wave data are also recorded when dynamic subsurface material properties are desired. An s-wave arrival is verified by obtained two sets of horizontal data that are 180 degrees out of phase. The phase reversal is obtained by either reversing the horizontal geophone orientation or reversing the hammer impact direction. Hard copy printouts of all field data are made and inspected as the information is collected. Field notes, including line number and orientation, topographic variations and other notes as appropriate are made on the hard copy printout. Locations and other notes are made on site maps and in notebooks as appropriate. Initial first arrival picks are made in the field and array endpoint arrival times are checked for immediate data adequacy verification as part of the quality control process.

Interpretation - Although preliminary or quality control initial refraction seismic data interpretations may sometimes be performed in the field, full interpretations are completed in the office. At the present time, two interpretation methods are being used; the intercept time method (ITM) and an optimization software routine based on finite difference optimization software. ITM breaks an interpretation into several distinct layers. It is simple, can be performed with a calculator, and can provide excellent interpretations of near surface layer depths and velocities. Optimization provides a continuously variable velocity interpretation through a discrete grid. Interpretations using optimization also indicate zones where interpretation has occurred, thus providing quality control on the depths to which the interpretation can be relied upon. However, the discrete grid used by optimization results in a low resolution near surface interpretation. The combination of both ITM and, when appropriate, optimization methods provides two separate interpretations with complimentary strengths and cross-checking capability. These interpretation methods are applied as appropriate to a particular project.

Refraction seismic data interpretation using the intercept time method is detailed by Mooney (1973). A personal computer spreadsheet is used to perform the necessary calculations to obtain depths and layer velocities, and print out time-distance plots and depth interpretations. This method is used for interpretations of up to three layers. It is considered that more than three layers cannot be effectively interpreted using twelve geophone data points. Interpretations are then completed manually to produce a final interpreted geologic profile and layer depths.

Refraction seismic data interpretation using optimization is performed using the SeisOpt2D software package by Optim, L.L.C., 1999, of Reno, Nevada. Energy source and geophone receiver locations and elevations, and first arrival times are entered into the software package, and first arrival travel times are optimized through a process of repeated (typically 10,000 to 100,000) iterations. Multiple seismic lines combined end to end into a longer composite line can be effectively interpreted using this software. Model grid dimensions and element sizes are selected, with larger grids containing smaller elements providing greater potential resolution. However, very large grids containing small elements may become unstable, and several runs may need to be made to obtain stable, robust interpretations. Once a robust interpretation has been obtained, the resulting seismic velocity profile is printed out with varying colors indicating the interpreted velocities.

References:

Mooney, H.M., 1973, Engineering Seismology Using Refraction Methods, Bison Instruments, Inc., Minneapolis, Minnesota.

Rucker, M.L. and Keaton, J.R., 1998, Tracing an Earth Fissure Using Seismic-Refraction Methods with Physical Verification, in Land Subsidence Case Studies and Current Research: Proceedings of the Dr. Joseph F. Poland Symposium on Land Subsidence, Edited by Borchers, J.W., Special Publication No. 8, Association of Engineering Geologists, Star Publishing Company, Belmont, California, p. 207-216.

REFRACTION MICROTREMOR (ReMi) SHEAR WAVE EQUIPMENT AND PROCEDURES

Refraction microtemor or ReMi surveys are performed in general accordance with the method described by Louie (2001) to develop vertical one-dimensional shear wave (s-wave) velocity profiles. The same equipment used for ReMi is also used for refraction seismic. When appropriate, both p-wave and s-wave data can be collected with the same physical seismic line setup.

ReMi Seismic Equipment - ReMi surveys are performed using a Geometrics S-12 Smartseis signal enhancement seismograph. This instrument has the capability to digitally record and store up to 12 channels of geophone data in SEG2 format. Up to 16,384 samples can be acquired for each geophone channel at sample intervals as long as 0.25, 0.5, 1 and 2 milliseconds. Sampling events to collect ReMi field data may typically last 6, 12 or 24 seconds. Geophone cables with 12 geophone takeouts at 10-foot or 20-meter spacings are presently used. Vertical geophones with resonant frequencies of 28 Hz and 4.5 Hz are used to obtain surface wave data for s-wave vertical profile analysis. High frequency geophones are used for shorter arrays with shallower depths of investigation, and low frequency geophones are used for longer arrays with greater depths of investigation. Broad band ambient site noise may be used as a surface wave energy source. Controlled surface wave energy sources include jogging alongside shorter geophone arrays and driving a field vehicle alongside longer geophone arrays. The seismograph system is extremely portable. In areas where vehicular access is not possible, the equipment can be mobilized by various means, including backpacking, packhorse, helicopter and canoe.

ReMi Field Procedures - The field operations are directed by our experienced engineer or geologist, who operates the equipment, prepares the records and examines the data in the field. ReMi seismic lines are generally laid out using the standard spacings on the geophone cables. A depth of investigation of about 100 meters or more may be possible using a 240 meter array. For shorter lines with improved near-surface resolution, 10-foot spacings between geophones with a 120-foot array have a depth of investigation of about 30 to 40 feet or more. Other geophone spacings can also be used.

Data collection consists of the system sampling the ambient or generated surface waves (a sampling event) at the geophone array for several to many seconds. Typical sampling times and intervals for a sampling event may be 6 seconds at 0.5 milliseconds, 12 seconds at 1 millisecond and 24 seconds at 2 milliseconds for array lengths of 60 feet, 120 feet and 240 meters, respectively. Several sampling events are collected at each ReMi setup. For shorter arrays where ReMi with surface wave energy generated by jogging is conducted in concert with seismic refraction data collection, four sampling events may typically be recorded. For longer arrays where urban ambient noise or a field vehicle generates the surface wave energy, six to ten sampling events may be recorded. Field notes, including line number and orientation, topographic variations and other notes as appropriate are made on hard copy of traces. Locations and other notes are made on site maps and in notebooks as appropriate. Sample data files may be transferred by 3.5-inch floppy to the laptop computer and preliminary interpretations made for immediate data adequacy verification as part of the quality control process.

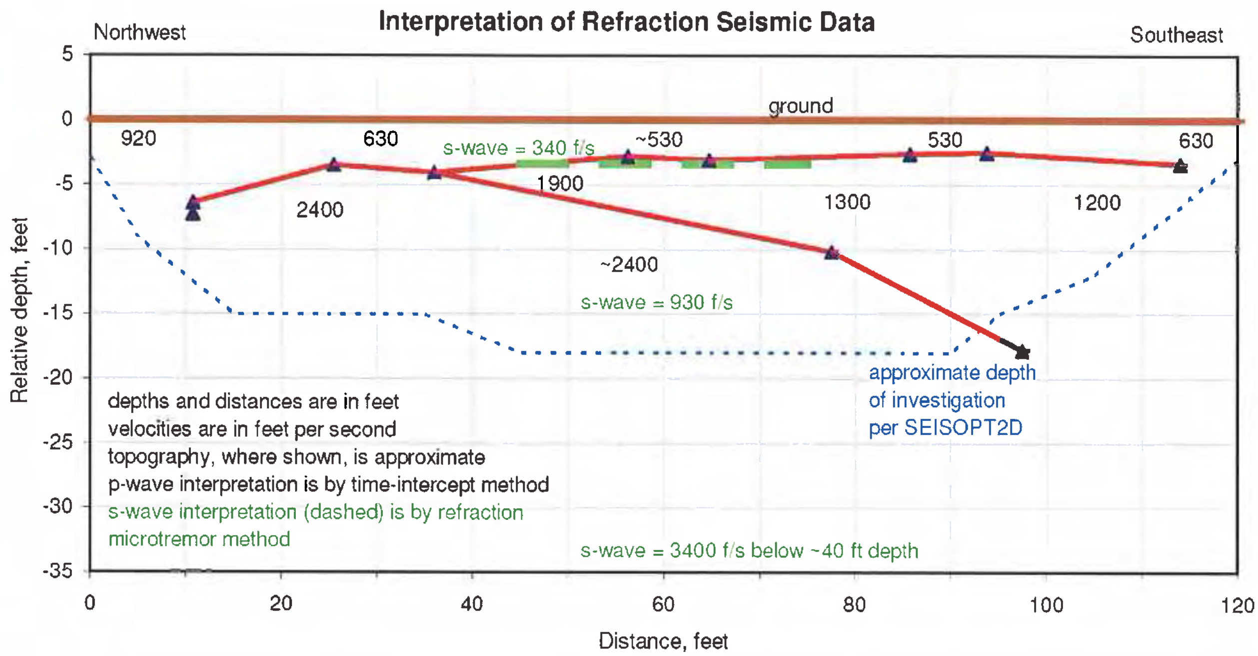
Interpretation - Although preliminary or quality control initial ReMi seismic data interpretations may sometimes be performed in the field, full interpretations are completed in the office. Data files, typically about 580kb each in size, are transferred from the seismograph to the laptop computer using 3.5-inch floppy disks. Interpretation is performed using the SeisOpt ReMi Version 3.0 (2004) software package by Optim, L.L.C., of Reno, Nevada. The software consists of two modules. The ReMiVsSpect module is used to convert the SEG2 files into a spectral energy shear wave frequency versus shear wave velocity presentation for a ReMi seismic setup. The interpreter then selects a dispersion curve consisting of the lower bound of the spectral energy shear wave velocity versus frequency trend, and that dispersion curve is saved to disk. Tracing the lower bound (slowest) of the shear wave velocity at each frequency selects the ambient energy propagating parallel to the geophone array, since energy propagating incident to the array will appear to have a faster propagating velocity. The second module, ReMiDisper, is then invoked. The interpreter models a dispersion curve with multiple layers and s-wave velocities to match the selected dispersion curve from the field data. An interpreted vertical s-wave profile is obtained through this process. It must be understood that this type of interpretation may not result in a unique solution.

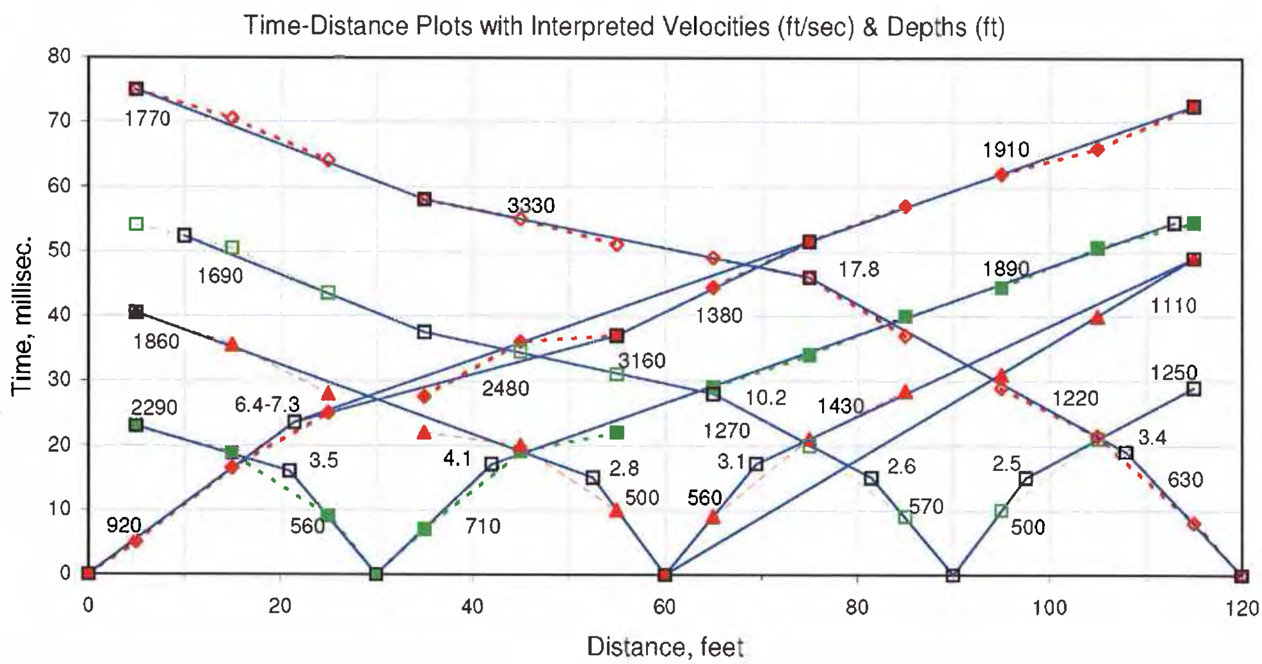
Louie, J.L., 2001, Faster, Better: Shear-wave velocity to 100 meters depth from refraction microtremor arrays, Bulletin of the Seismological Society of America, Vol. 91, 347-364.

Refraction Seismic Interpretation

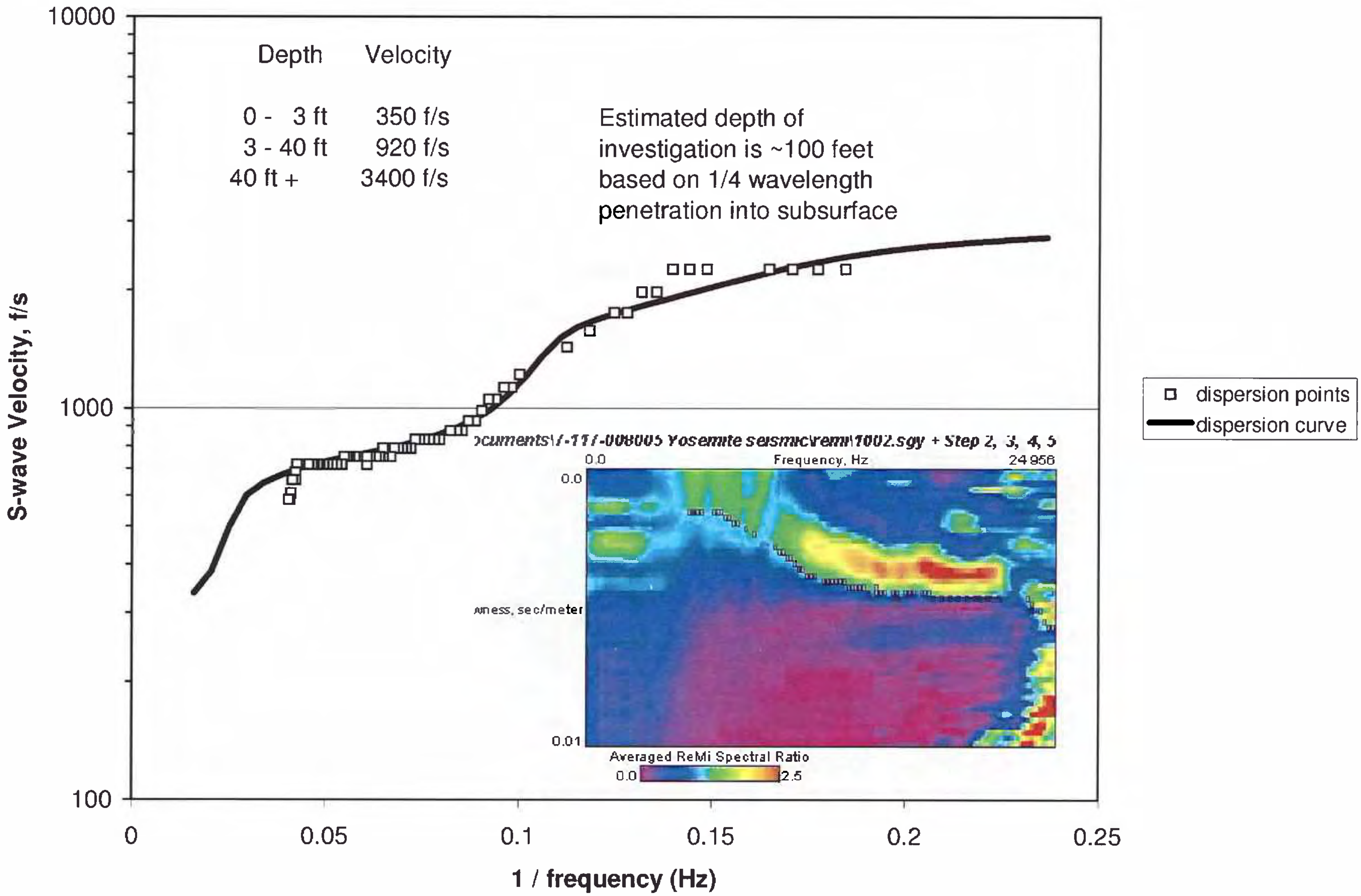
Line 1

AMEC Job No. 7-117-008002





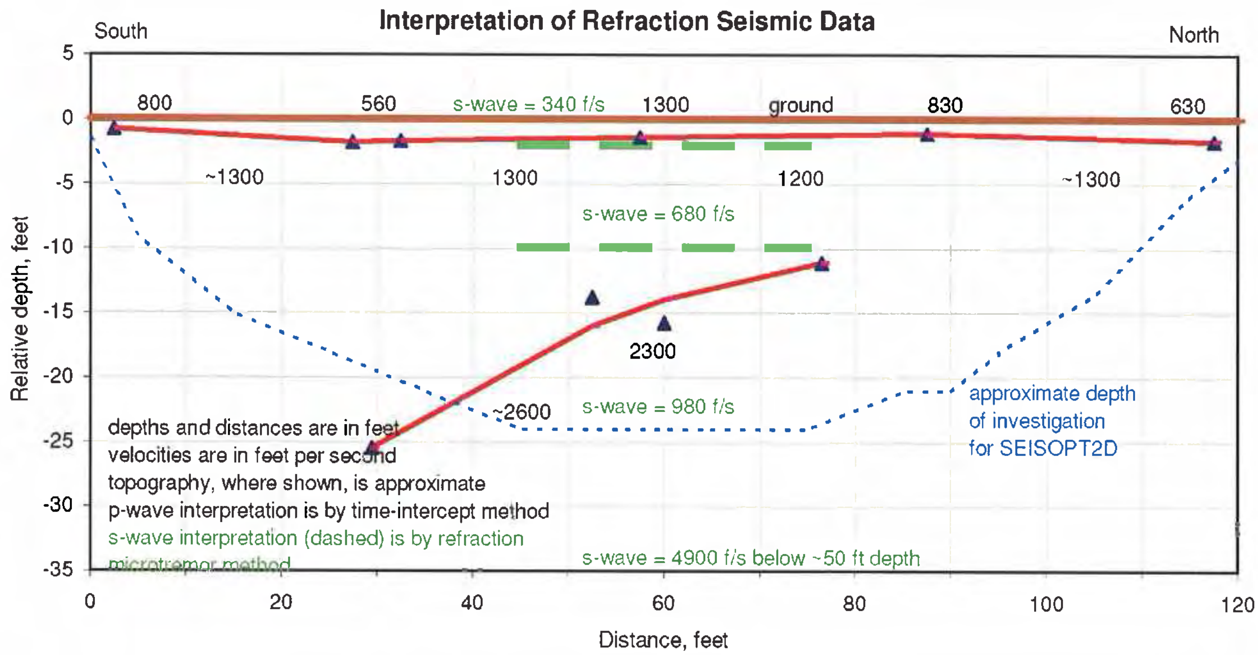
Refraction Microtremor S-wave Interpretation | Line 1 | AMEC Job No. 7-117-008002

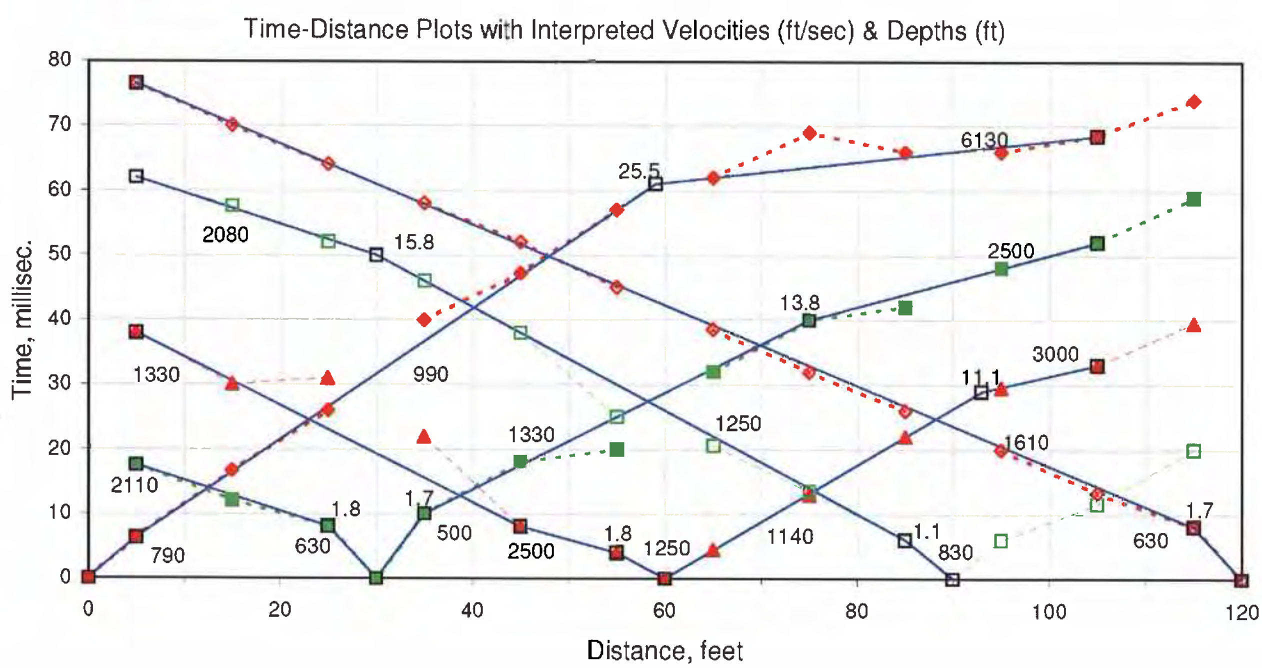


Refraction Seismic Interpretation

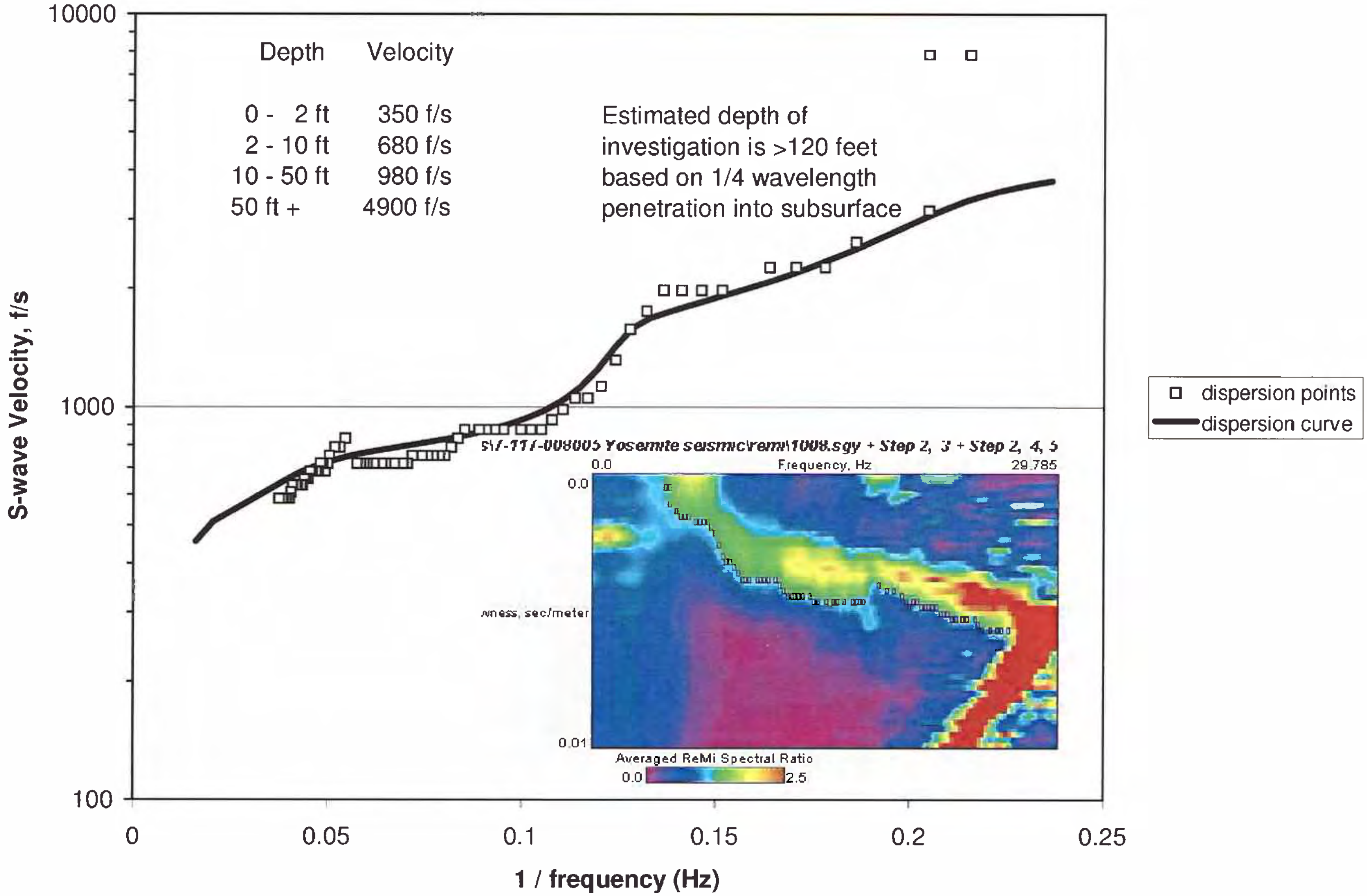
Line 2

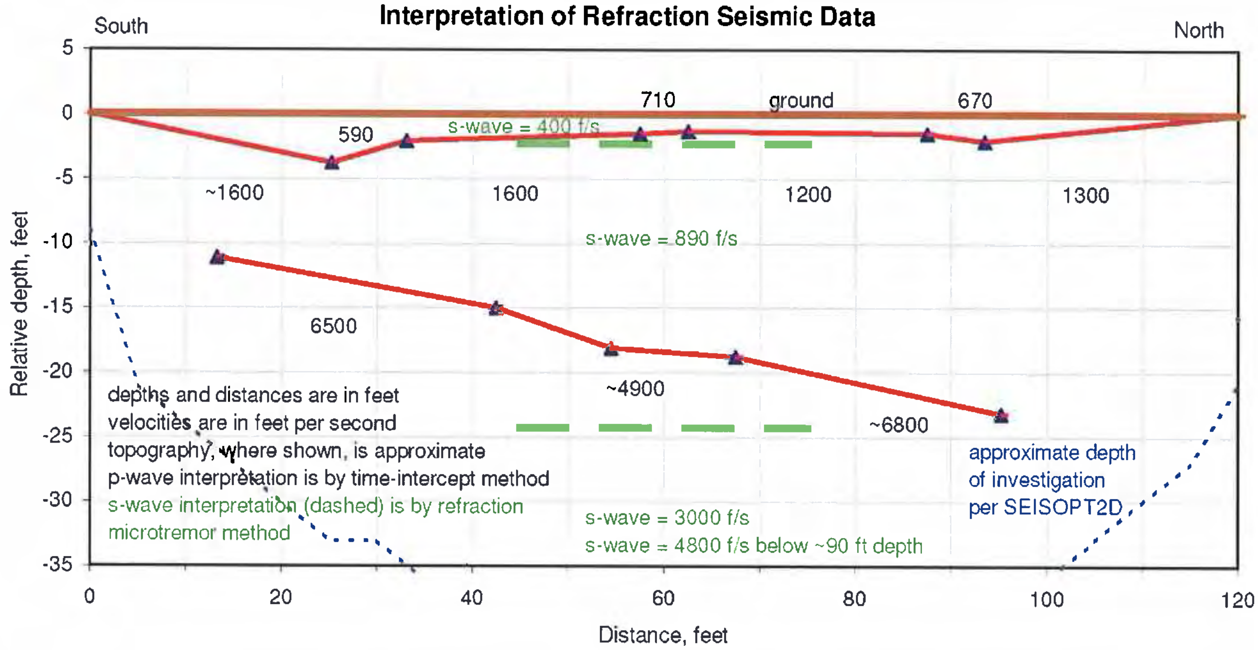
AMEC Job No. 7-117-008002

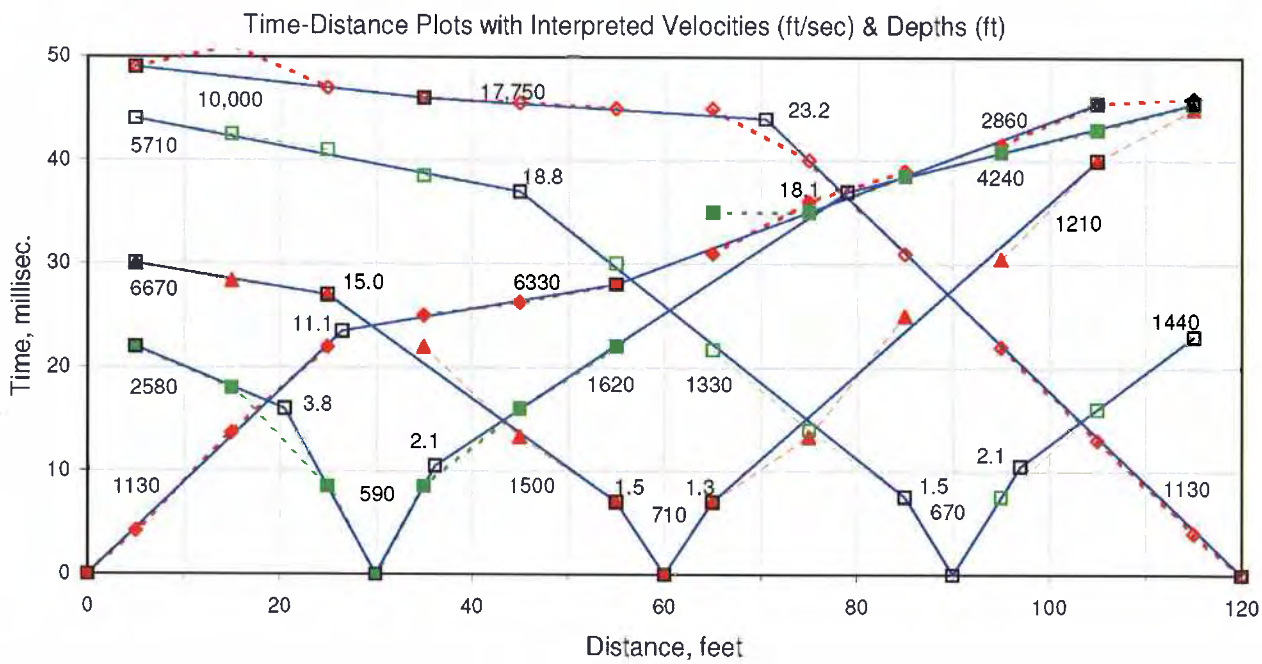




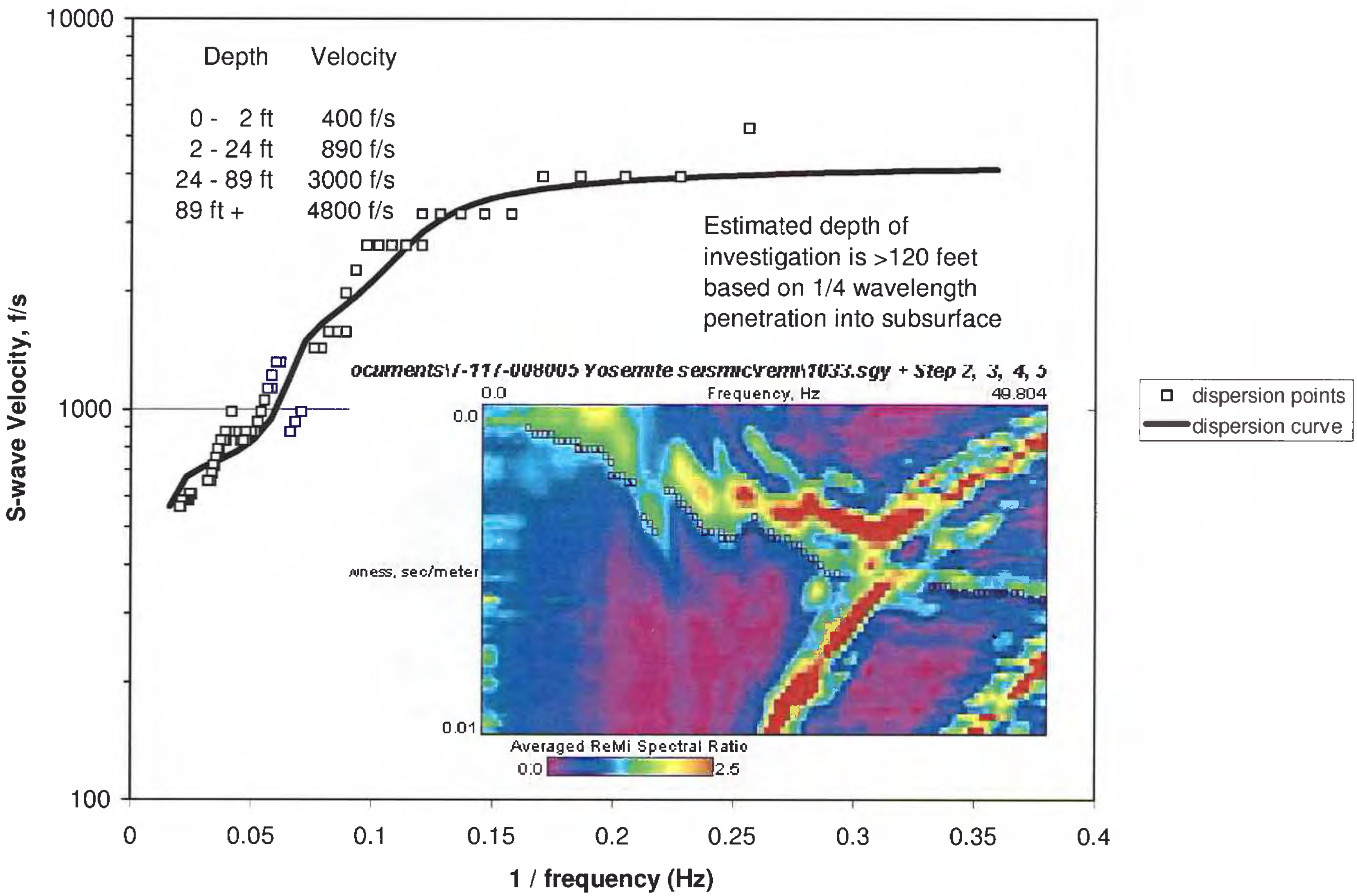
Refraction Microtremor S-wave Interpretation | **Line 2** | AMEC Job No. 7-117-008002




Interpretation of Refraction Seismic Data
South
North
ground
710
670
590
s-wave = 400 f/s
~1600
1600
1200
1300
s-wave = 890 f/s
6500
~4900
~6800
depths and distances are in feet
velocities are in feet per second
topography, where shown, is approximate
p-wave interpretation is by time-intercept method
s-wave interpretation (dashed) is by refraction microtremor method
s-wave = 3000 f/s
s-wave = 4800 f/s below ~90 ft depth
approximate depth of investigation per SEISOPT2D
Relative depth, feet
Distance, feet


Time-Distance Plots with Interpreted Velocities (ft/sec) & Depths (ft)
10,000
17,750
23.2
2860
5710
18.8
18.1
4240
1210
6670
15.0
6330
11.1
2580
3.8
1620
1330
1440
2.1
2.1
1130
590
1500
1.5
710
1.3
1.5
670
1130
Time, millisec.
Distance, feet

Refraction Microtremor S-wave Interpretation **Line 6** AMEC Job No. 7-117-008002



APPENDIX D

MICROPILE SAMPLE SPECIFICATION

Section 569. - MICROPILES

Description

569.01 This work consists of furnishing all necessary materials, services, supervision, labor, and equipment to install and test micropiles.

Material

569.02 Conform to the following Subsections:

Water	725.01
Admixtures	711.03.
Cement	701.01(a).
Centralizers	722.02(f)
Grout	725.22 (f).
Welding	551.09(a)
Bar reinforcement	709

Epoxy-coated bars. Furnish bars conforming to AASHTO M284, except that coating flexibility bending requirements are waived. Store, handle and repair epoxy-coated bars in accordance with Subsection 554.07.

Bar couplers. Use couplers with a strength that is at least 125 percent of the required yield strength of the reinforcing steel.

When a bearing plate and nut are required to be threaded onto the top end of reinforcing bars for the pile top to the anchorage, the threading may be continuous spiral deformed ribbing provided by the bar deformations or may be cut into a reinforcing bar. If threads are cut into a reinforcing bar, provide the next larger bar number designation from that shown on the plans.

Permanent casing. Provide permanent steel casing which meets the following:

(1) Tensile requirements of ASTM A252, Grade 3
(2) Minimum yield strength **as shown on the plans**
(3) Minimum elongation = 15%
(4) Minimum diameter and wall thickness **as shown on the plans**

Permanent casing may be new "Structural Grade" (a.k.a. "Mill Secondary") API Specification 5CT, Grade N80 steel pipe meeting (1) thru (4) above but without Mill Certification, free from defects (dents, cracks, tears)

Welding, if required is in accordance with Section 551.09(a)

Plates and shapes. ASTM A36M.

Construction Requirements

569.03 Qualifications. Provide a micropile contractor fully experienced in all aspects of micropile design and construction, and have successfully completed at least 5 micropile projects during the past 3 years of similar scope and size and demonstrated experience with micropile testing. At least 30 days prior to the start of construction, identify the engineer, on-site supervisor, and drill operators assigned to the project and submit a summary of each individual's experience. Also submit project reference list, and brief project description with the owners name phone number and load test reports for the projects listed

569.04 Pre-Installation Submittals. A pre-construction meeting will be held by the CO prior to the start of micropile installation to clarify construction requirements, coordinate the construction schedule and activities, and identify contractual relationships and delineation of responsibilities amongst the prime Contractor and the various Subcontractors.

At least 30 days prior to construction submit the following to the CO for approval:

- Proposed construction schedule. Schedule all installation techniques such that there will be no interconnection or damage to piles in which grout has not achieved final set.

- Proposed method(s) for constructing and load testing the micropiles. Include all necessary drawings and details to describe the load test method and equipment proposed. Calibration reports for each test jack, pressure gauge, and master pressure gauge to be used. An independent testing laboratory must have conducted the tests within 6 months of the date submitted. Include supporting calculations for all structural components of the micropile load test apparatus.

- Certified mill test reports for the reinforcing steel, properly marked and as the materials are delivered. Include the ultimate strength, yield strength, elongation, and composition. For steel pipe used as permanent casing submit a minimum of two representative coupon tests on each load delivered to the project. Submit mill certifications if available.

- Grout mix designs, including details of all materials to be incorporated and the procedure for mixing and placing the grout. The grout design strength is 4000 psi at 28 days and 2000 psi at 7 days. Include certified test results verifying the acceptability of the proposed mix designs whereby 4 inch by 8 inch cylinders are made and standard cured in accordance with AASHTO T23 and tested at 7 and 28 days in accordance with AASHTO T22. The measured grout density shall typically range from 112 pcf to 118 pcf with grout mixes generally conforming to Section 725.22(f) with fine aggregate.

569.05 Allowable Tolerances.

1) Construct centerline of piling no more than 3 in. from indicated pile spacing and plan location.
2) Construct the pile-hole alignment within 2% of design alignment.
3) Construct the top elevation of pile within +1 in. of the design vertical elevation.
4) Construct the centerline of reinforcing steel no more than ¾ in. from centerline of piling.

569.06 Installation Records. Record the following and provide a report within 24 hours after each pile installation is completed:

- Pile drilling duration and observations (e.g., flush return),
- Soil/rock/water encountered, included in drilling log format,
- Approximate final tip elevation,
- Cut-off elevation,
- Description of problems encountered,
- Grout pressures attained,
- Grout quantities pumped,
- Pile materials employed and dimensions,
- Installation methods/equipment employed, and
- Micropile test records, analyses, and details, as applicable.

569.07 Micropile Installation.

- Review geotechnical and environmental conditions and select the appropriate pile drilling method and the grouting procedures for the installation of the micropiles. Provide drilling equipment and methods suitable for drilling through the conditions to be encountered, with minimal disturbance to these conditions or any overlying or adjacent structure or service. Upon drilling completion remove drill cuttings and/or other loose debris from the bottom of the hole. The borehole must be open to the defined nominal diameter, full length, prior to placing grout and reinforcement. Develop methods of stabilizing borehole that do not have a deleterious effect on the geotechnical bond development of the grout.

- Attach centralizers and spacers to the reinforcement bars at 10 feet maximum vertical spacing and within 5 feet from the top and bottom of the reinforcement.

- Lower the central reinforcement steel with centralizers into the stabilized drill holes to the desired depth without difficulty. Provide reinforcement that is free of deleterious substances such as soil, mud, grease, or oil that might contaminate the grout or coat the reinforcement and impair bond.

- If, during installation of a pile, an obstruction is encountered that prevents the practical advancement of the pile, abandon the hole and fill with grout. Drill a new pile at a location to be determined by the CO; however, it must be acknowledged that in certain structures, relocation options may be severely limited, and further attempts at the original location with different methods may be required.

- Inject grout according to Section 569.09, below.

- Check pile top elevations and adjust all installed micropiles to the planned elevations.

- For a minimum of one micropiles at each of the foundation units, advance micropile borings a minimum of 12.5 feet below the assumed bedrock contact to verify top of bedrock and that piles are not founded on a boulder.

- Immediately suspend operations and notify the CO if adjacent structures are damaged from drilling or grouting. Take corrective actions necessary to stop the movement or perform repairs at no cost to the government as directed by the CO.

569.08 Pipe Casing and Reinforcing Bar Placement and Splicing. Secure lengths of casing and reinforcing bars to be spliced in proper alignment and in a manner to avoid eccentricity or angle between the axes of the two lengths to be spliced. Locate threaded pipe casing joints at least two casing diameters (OD) from a splice in any reinforcing bar. When multiple bars are used, stagger bar splices at least 1 foot. Construct all pile splices to develop the required design strength of the pile section.

569.09 Grouting. Grout micropiles the same day the load transfer bond length is drilled. Provide a grout that does not contain lumps or any other evidence of poor or incomplete mixing. Mix admixtures, if used, in accordance with manufacturer's recommendations. Equip the pump with a pressure gauge to monitor grout pressures. Provide a pressure gauge capable of measuring pressures of at least 150 psi or twice the actual grout pressures used by the Contractor, whichever is greater. Size the grouting equipment to enable the grout to be pumped in one continuous operation. Keep the grout in constant agitation prior to pumping.

Inject the grout from the lowest point of the drill hole by gravity fill (tremie method) until clean, pure grout flows from the top of the micropile. The tremie grout may be pumped through grout tubes, hollow stem augers, or drill rods. Subsequent to tremie grouting, all grouting operations associated with, for example, extraction of drill casing and pressure grouting, must ensure complete continuity of the grout column. The use of compressed air to directly pressurize the fluid grout is not permissible. Control the grout pressures and grout takes to prevent excessive heave in cohesive soils or fracturing of soil or rock formations. Grout the entire pile to the design cut-off level.

Upon completion of grouting, the grout tube may remain in the hole, but filled with grout. If the Contractor uses a post-grouting system, submit all relevant details including grouting pressure, volume, location and mix design, as part of Section 569.04. Grout within the micropiles shall be allowed to attain the minimum design strength prior to being loaded.

During production, micropile grout shall be regularly tested by the Contractor for compressive strength and consistency. Compressive strength of 4 inch by 8 inch cylinders shall be determined in accordance with AASHTO T22 at a frequency of no less than one set of three samples from each grout plant each day of operation, or per every 20 micropiles, whichever occurs more frequently. Cylinders shall be made and standard cured in accordance with AASHTO T23. The compressive strength shall be the average of the three samples tested, and shall meet or exceed the approved mix design strength requirements, as submitted under 569.04. Grout consistency, as measured by grout density, shall be determined in accordance with AASHTO T133 at a frequency of at least one test per every five piles. Provide grout compressive strength and density test results to the CO within 24 hours of testing.

Pile Load Tests

569.10 Micropile Verification Load Tests. Perform pre-production verification load tests to verify the design and construction methods proposed prior to installing any production piles. Construct sacrificial verification load test piles along each structure, or as specified on the plans

Submit the proposed micropile load testing procedure at least two weeks prior to starting the verification load testing. Provide the micropile verification load testing proposal in general conformance with ASTM D-1143 and D-3689, and indicate the minimum following information:

(1) Type and accuracy of apparatus for load measurement,
(2) Type and accuracy of apparatus for applying load,
(3) Type and apparatus for measuring pile deformation and displacement,
(4) Type and capacity of reaction load system, including sealed design drawings,
(5) Hydraulic jack calibration report.

Size the verification test micropile structural steel sections to safely resist the maximum test load. Do not exceed 80 percent of the structural capacity of the micropile structural elements, including steel yield in tension, steel yield or buckling in compression, or grout crushing in compression when the maximum verification test loads are applied.

Provide verification load test results for approval by CO. The test results will be reviewed and accepted by the CO prior to beginning production micropiles.

Load tested micropiles to 250% of the compression design load (DL) (i.e., 2.5 DL). The load tested piles must be of the same design as the production piles. Test piles under compression loads as applicable. An Alignment Load (AL), if required, may be applied to the pile prior to setting the movement recording devices. Provide an Alignment Load no greater than 10% of the Design Load (i.e., 0.1 DL). Zero dial gauges after the first setting of AL.

Conduct axial compressional pile load tests by loading the micropile and recording the pile head movement in the following cyclic load increments:

Table 569-1

Load	Hold Time (Minutes)
AL	1
0.25 DL	1
0.50 DL	1
AL	1
0.25 DL	1
0.50 DL	1
0.75 DL	1
AL	1
0.25 DL	1
0.50 DL	1
0.75 DL	1
1.00 DL	1
AL	1
0.25 DL	1
0.50 DL	1
0.75 DL	1
1.00 DL	1
1.33 DL	60*
1.75 DL	1
2.00 DL	1
2.25 DL	1
2.50 DL	10
AL	1

* Hold until pile meets acceptance criterion (2) below

AL = Alignment Load
DL = Design Load

Obtain measurement of pile movement at each loading increment. Start the load hold period immediately after the test load is applied, and measure and record the pile movement, with respect to a fixed reference. Also measure the pile movement during a creep test, and record, at 1, 2, 3, 4, 5, 6, 10, 20, 30, 50, and 60 minutes.

The acceptance criteria for micropile verification load tests are:

(1) Provide a pile to sustain the compression design loads (1.0 DL) with no more than 3/8 in. total vertical movement at the top of the pile as measured relative to the top of

the pile prior to the start of testing. If an Alignment Load is used, then the allowable movement will be reduced by multiplying by a factor of (DL-AL)/DL.

(2) Provide test piles that have a creep rate at the end of the 1.33 DL increment which is not greater than 1 mm/log cycle (0.040 in./log cycle) time from 1 to 10 minutes or 2 mm/log cycle (0.080 in./log cycle) time from 6 to 60 minutes and has a linear or decreasing creep rate.

(3) Failure does not occur at the 2.5 DL maximum compression and tension loads. Failure is defined as a slope of the load versus deflection (at end of increment) curve exceeding 0.635 mm/kip (0.025 inches/kip).

Provide the CO a written report confirming micropile geometry, construction, and testing details within seven working days following completion of the verification load tests.

If the micropile test fails to meet the acceptance criteria, establish the cause(s) and provide modifications to the design, the construction procedures, or both. Reconstruct a verification load test pile according to the new design and conduct a compression test at no additional costs to the government.

At the completion of verification testing, test piles shall be removed down to the elevation specified by the CO.

569.11 Production Pile Proof Testing. Perform proof load tests on production piles selected by the CO. Test piles designated for compression proof load testing to a maximum test load of 1.67 DL. Proof tests shall be made by incrementally loading the micropile in accordance with the following loading schedule:

Table 569-2

Load	Minimum Hold Time (Minutes)
AL	1
0.25 DL	1
0.50 DL	1
0.75 DL	1
1.00 DL	1
1.33 DL	10*
1.67 DL	1
AL	1

* Hold until pile meets acceptance criterion (2) of next paragraph
AL = Alignment Load
DL = Design Load

The acceptance criteria for micropile proof load tests are:

(1) Compression and tension design loads (1.0 DL) with no more than 13 mm (1/2 in.) total vertical movement at the top of the pile as measured relative to the pile prior to the start of testing. If an Alignment Load is used, then the allowable movement will be reduced by multiplying by a factor of (DL-AL)/DL.

(2) A creep rate at the end of the 1.33 DL increment which is not greater than 1 mm/log cycle (0.040 in./log cycle) time from 1 to 10 minutes or 2 mm/log cycle (0.080 in./log cycle) time from 6 to 60 minutes and has a linear or decreasing creep rate.

(3) Failure does not occur at the maximum compression and tension load increment. Failure is defined as a slope of the load versus deflection (at end of increment) curves exceeding 0.635 mm/kip (0.025 inches/kip).

If a proof-tested micropile fails to meet the acceptance criteria, proof test another micropile in the immediate vicinity and provide proposal for remedial measure for approval.

Measurement

569.13 Measure Section 569 items listed in the bid schedule according to Subsection 109.02 and the following as applicable.

Measure micropiles per length of installation

Measure Micropile VerificationLoad tests that indicate acceptable installations,

There will be no extra payment for grout overruns and proof tests.

Payment

569.14 The accepted quantities will be paid at the contract price per unit of measurement for Section 569 pay items listed in the bid schedule. Payment will be full compensation for the work prescribed in this Section. See Subsection 109.05.