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INTEGRATING EARTH & STRUCTURE

Geotechnical Investigation Report

Dry Creek Habitat Enhancement Demonstration Projects - Phase 3

Sonoma County, California

SAGE Project No. 07-082.02

Prepared for:

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October 13, 2011

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INTEGRATING EARTH & STRUCTURE

October 13, 2011
Project No. 07-082.02 Task 3.02

Mr. Michael Burke
Inter-Fluve, Inc.
1020 Wasco St., Ste. 1
Hood River, OR 97031

RE: Final Geotechnical Investigation Report
Dry Creek Habitat Enhancement Demonstration Projects
Station 325+00 to 383+00
Sonoma County, California

Dear Mr. Burke:

Sanders & Associates Geostructural Engineering, Inc. (SAGE) is pleased to submit this final report presenting the results of our geotechnical investigation for the proposed habitat enhancements in Dry Creek, a major tributary to the Russian River in Sonoma County, California. Specifically, this investigation was focused on the first phase of enhancements along an approximately 1.1 mile length of Dry Creek, referred to as the Demonstration Reach, which extends from the mouth of Grape Creek downstream to the mouth of Crane Creek (station 325+00 to 383+00).

We explored the subsurface conditions at selected off channel enhancement sites and a bank stabilization site by excavating eight (8) test pits and drilling two (2) small-diameter borings (Figure 2). In addition, NORCAL Geophysical Consultants performed a geophysical survey at an off channel enhancement site which could not be accessed by conventional mechanized equipment.

In general, we encountered alluvial soils consisting of mixtures of gravel and sand with interbedded layers of finer material. The material ranged from loose to dense, with the least dense materials generally near the ground surface. Saturated materials were very loose upon excavation and could not maintain excavation cuts or slopes.

Based on the results of the geophysical survey, the depth to bedrock at Off-Channel Enhancement Area D is estimated between 2 and 9 feet on the northwest and southeast ends of a seismic refraction line performed at this site, respectively. The recorded velocities suggest the rock is rippable to moderately rippable with a CAT 9L bulldozer. Bedrock was not encountered in any of the test pits excavated at the site.

The report submitted herewith contains recommendations regarding site grading, temporary and permanent slopes, slope stability, and bearing capacity. These recommendations are based on limited

subsurface exploration and laboratory testing. Consequently, variations between expected and actual soil conditions may be found during construction. SAGE should be retained to observe the earthwork to evaluate actual conditions encountered for conformance with the geotechnical aspects of the plans and specifications.

Please call us should you have questions.

Sincerely yours,
Sanders & Associates Geostructural Engineering, Inc.

Darren A. Mack
Geotechnical Engineer

Jerry S. Pascoe
Geotechnical Engineer

TABLE OF CONTENTS

1.0 INTRODUCTION 1
2.0 SCOPE OF SERVICES 2
3.0 SITE CONDITIONS 2
4.0 GEOLOGIC CONDITIONS 2
4.1 Regional Geologic Setting 2
4.2 Site Geology 3
5.0 SUBSURFACE CONDITIONS 3
6.0 SEISMICITY 5
6.1 Regional Seismicity 5
6.2 Seismic Hazards 6
6.3 Fault Rupture 6
7.0 RECOMMENDATIONS 6
7.1 Demolition & Clearing 6
7.2 Fill Material and Compaction Requirements 7
7.3 Aeration 8
7.4 Excavatability 8
7.5 Temporary Slopes 9
7.6 Permanent Slopes 10
7.7 Slope Stability 10
7.8 Bearing Capacity 10
7.9 Dewatering 11
8.0 SERVICES DURING CONSTRUCTION 11
9.0 LIMITATIONS 12
10.0 REFERENCES 13

FIGURES

Figure 1 Site Location Map
Figure 2 Subsurface Exploration Map

APPENDICES

Appendix A – Field Exploration Program

Figure A-1 Soil Classification Chart

Figures A-2 through A-3 Logs of Soil Borings B1 and B2

Figure A-4 Logs of Test Pits TP1 through TP8

Appendix B – Laboratory Test Results

Appendix C – Geophysical Survey Report

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INTEGRATING EARTH & STRUCTURE

FINAL GEOTECHNICAL INVESTIGATION REPORT
Dry Creek Habitat Enhancement Demonstration Projects
Station 325+00 to 385+00
Sonoma County, California

1.0 INTRODUCTION

Sanders & Associates Geostructural Engineering, Inc. (SAGE) is pleased to submit this final report presenting the results of our geotechnical investigation for habitat enhancements in Dry Creek, a major tributary to the Russian River in Sonoma County, California. Specifically, this investigation was focused on the first phase of enhancements along an approximately 1.1 mile length of Dry Creek, referred to as the Demonstration Reach, which extends from the mouth of Grape Creek downstream to the mouth of Crane Creek (STA 325+00 to 383+00).[1]

The purpose of the proposed habitat enhancements is to develop summer rearing and winter refugia habitat for local fish species, specifically coho salmon and steelhead trout. Based on our review of the 60% Dry Creek Habitat Enhancement Demonstration Projects plans, we understand this will be achieved using a combination of enhancement approaches, including backwater ponds and channels for the fish to inhabit. The backwater ponds and channels will require excavation in stream terraces adjacent to the active stream channel. Slope inclinations for channel regrading are expected to be on the order of 2H:1V or flatter, with cuts up to 15 vertical feet. In channel enhancement measures will include new riffle areas, deepening of existing pools, and construction of artificial log jams.

In addition, stabilization of the creek banks will be locally required to retain property and to enhance the habitat characteristics along the edge of Dry Creek. Anticipated bank stabilization measures will include: (1) flattening the existing slopes and covering with biodegradable fabrics; (2) bank reconstruction using log cribs with live willow cuttings; and (3) bank reconstruction using fabric encapsulated soil (FES) with live willow cuttings. The log cribs will have nominal widths (perpendicular to slope) of 10 to 17 feet, and will be underlain at the toe by 3-foot-wide by 5-foot-deep pads of 18-inch-minus rock. Backcuts for taller stabilization efforts are expected to consist of temporary slopes, although shoring may be required locally where layback space is limited by the presence of existing vineyards.

The approximate project location is shown on the Site Location Map (Figure 1). The important project features are shown on the Subsurface Exploration Map (Figure 2).

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1 Project stations (STA) are based on the 60% Dry Creek Habitat Enhancement Demonstration Projects plans prepared by Inter-Fluve, dated April 15, 2011.

2.0 SCOPE OF SERVICES

We performed a subsurface investigation in support of the proposed habitat enhancements for the Demonstration Reach. We have summarized the observations and results of our investigation in this geotechnical report, which provides recommendations and conclusions for developing the habitat enhancement design. Specifically, our investigation consisted of:

- Conducting a site reconnaissance to review selected locations for subsurface exploration;
- Obtaining the necessary drilling permits and coordinating our subsurface exploration program;
- Retaining the services of a private utility locator to clear investigation locations for possible underground utilities and/or buried objects;
- Performing a subsurface exploration program including eight (8) test pits and two (2) soil borings;
- Performing a geophysical survey at an off channel enhancement site which could not be accessed by conventional mechanized equipment;
- Collecting representative samples of the soil encountered in the test pits and soil borings;
- Performing laboratory testing on selected soil samples; and
- Preparing this geotechnical report.

3.0 SITE CONDITIONS

Dry Creek is an incised stream with flows regulated by the upstream Warm Springs Dam. Flow regulation has reduced the frequency and severity of major floods while providing a continuous baseflow during the summer months. This has resulted in the rapid growth of dense riparian vegetation and shrubs along the channel banks and formerly active bar surfaces since the dam was put into service in 1984. Where visible through the dense vegetation, the channel banks are generally steep to very steep and locally subject to erosion. Alluvial terraces are locally preserved along the Demonstration Reach, and are positioned above the active stream channel. These terrace surfaces, including a prominent terrace at Off Channel Enhancement Area C (Figure 2), are relatively flat benches with areas of dense vegetation to open grassy meadows.

4.0 GEOLOGIC CONDITIONS

4.1 Regional Geologic Setting

The Demonstration Reach is located in the Dry Creek drainage valley within the Coast Ranges geomorphic province of California. The Coast Ranges province is generally characterized by northwest-trending mountain ranges and intervening valleys that are controlled by right-lateral strike-slip faulting along the San Andreas fault system.

Review of available geologic mapping and literature sources indicate that the Dry Creek drainage valley is a structurally-controlled valley that generally lies on the boundary between sedimentary units

of the Great Valley Complex to the east and various fault bounded lenses of the Coast Range ophiolite and metamorphic rock units of the Franciscan Complex to the west (Blake, Graymer, and Stamski, 2002). However, sandstone, siltstone, and shale units belonging to the Great Valley Complex are also mapped along the western margin of the valley adjacent to the Demonstration Reach. The valley is filled with stream channel and floodplain deposits associated with Dry Creek and include up to three terrace deposits, the oldest of which appears to be approximately 1,000 years old (Harvey and Schumm, 1985).

4.2 Site Geology

Geologic conditions at the site are generally similar to those depicted by Huffman and Armstrong (1980) and Blake, Graymer, and Stamski (2002). In general, the Demonstration Reach is underlain by alluvial deposits of varying age. The deposits are comprised of clay, silt, sand, gravel, and cobble mixtures of varying rock types derived from tributaries extending into the adjacent Coast Range ophiolite, Great Valley Complex, and Franciscan Complex. The youngest alluvium is found within the active stream channel and low-lying gravel bars that are seasonally inundated.

Alluvial terraces are preserved along the length of the Demonstration Reach, and are comprised of older alluvial deposits. The position of these terraces relative to the active stream channel varies along the reach. In general, terraces positioned higher than the active stream channel are well vegetated, particularly the prominent terrace at Off Channel Enhancement Area C (Figure 2). Shallow slope failures are locally present along the active channel and terrace banks in areas where the banks are actively being undercut.

Bedrock outcrops observed along the active stream channel are generally limited to Grape and Crane Creeks near the confluence with Dry Creek, and within the Dry Creek channel below and immediately downstream of Lambert Bridge. The exposures are comprised of interbedded layers of weak siltstone and somewhat stronger, thicker beds of sandstone that appear to be consistent with descriptions of the siltstone, sandstone, and shale units of the Great Valley Complex. In general, the siltstone and sandstone exposures can easily be broken with a rock hammer, and are expected to be excavatable using conventional grading equipment. At Grape Creek, the bedrock is locally folded along a west-southwest plunging axis approximately parallel to the apparent syncline evident in the mapped Great Valley Complex units exposed on the western flank of the valley.

5.0 SUBSURFACE CONDITIONS

We explored the subsurface conditions at selected off channel enhancement sites and bank stabilization site by excavating eight (8) test pits and drilling two (2) small-diameter borings (Figure 2). In addition, NORCAL Geophysical Consultants (NORCAL) performed a geophysical survey at an off channel enhancement site which could not be accessed by conventional mechanized equipment. Table 1 summarizes the subsurface exploration performed. A description of our field exploration program, as well as the test pit and borings logs, is presented in Appendix A. The results of laboratory testing are presented in Appendix B.

TABLE 1
SUMMARY OF EXPLORATION LOCATIONS

Enhancement Site*	Property Owner(s)	Subsurface Exploration
Off Channel Enhancement Area A Bank Stabilization (STA 334+00 – 337+70)	Wallace & Farrow	Test pits (TP5 to TP8)
Off Channel Enhancement Area C	Van Alyea	Test pits (TP1 thru TP4)
Bank Stabilization (STA 360+00 – 363+55, STA 365+10 – 365+80)	Mascherini	Soil borings (B1, B2)
Off Channel Enhancement Area D	Seghesio	Geophysical survey

TP1 through TP4 were excavated at Off Channel Enhancement Area C. The upper 6 to 12 inches of TP1 through TP3 were composed of loose to medium dense gravelly silt and silty gravel with organic material. TP4 exposed four feet of medium stiff gravelly clay at the surface of the excavation. Below the surficial layer, we encountered easily excavatable loose to medium dense sand and gravel mixtures. Groundwater was encountered around Elevation 122 feet in each test pit.

Soils encountered in TP5 through TP9 generally comprised sandy gravel with trace fines and cobbles up to 10 inches in dimension. Localized layers of sand and clayey sand were also encountered. The subsurface material was loose to medium dense and could be easily excavated. Groundwater was encountered between Elevation 115 and 118 feet.

The soils encountered in B1 and B2 indicate that the upper 13 to 15 feet of the creek bank is variable. In B1, we encountered loose to medium dense silty sand and sand. In B2, we encountered medium stiff to stiff clay with some silty sand. Below 15 feet, we encountered sand and gravel with varying silt and clay content. Groundwater was between 21 and 23 feet below existing grade, which corresponds to elevations of 121 to 122 feet.

The water level in Dry Creek was measured adjacent to TP-1 and TP-5. At these two locations, the measured groundwater elevations in the test pits were approximately the same as the adjacent water surface elevation in Dry Creek. Although not measured in the field, we would expect similar results for the remaining test pits.

The test pit side slopes were marginally stable in dry to moist conditions. However, rapid caving or sloughing generally occurred below the water table, particularly where active seepage was encountered, which limited the depth of the test pits.

Although bedrock of the Great Valley Complex is visible in Grape and Crane Creeks near the confluence with Dry Creek, and within Dry Creek below and immediately downstream of Lambert Bridge, bedrock was not encountered in the test pits and borings. At Off Channel Enhancement Area D, the results of the NORCAL survey suggest the depth to sedimentary bedrock is between 2 and 9 feet below existing grades on the northwest and southeast ends of the seismic refraction line,

respectively. The recorded velocities suggest the rock is rippable to moderately rippable with a CAT 9L bulldozer. The approximate location of the seismic line is shown on Figure 2, and the full geophysical report is presented in Appendix C.

6.0 SEISMICITY

6.1 Regional Seismicity

Seismicity is defined as the geographical and historical distribution of earthquakes, or more simply, earthquake activity. The potential for ground shaking at the site is related to earthquake activity that might occur along nearby or distant faults. Based on historical earthquake activity and fault hazard mapping, the Sonoma County region is considered to have a relatively high potential for seismic activity related to the San Andreas fault system.

The 2002 Working Group on California Earthquake Probabilities (WGCEP) suggests the overall probability of one or more $M_W \geq 6.7$ earthquakes occurring in the San Francisco Bay region during the period from 2002 to 2032 is 62 percent (WGCEP, 2003). The highest probability of 27 percent was assigned to the Hayward/Rodgers Creek fault zone.

The closest active faults in this system are the Maacama and Rodgers Creek faults, which are mapped approximately 6 miles northeast and 8 miles southeast of the site, respectively. The San Andreas fault is mapped approximately 20 miles southwest of the site.

Regional fault maps and databases (Jennings et al., 2010; USGS, 2010) and a fault evaluation report (Bryant, 1982) show several strands of the Healdsburg fault within and adjacent to the Dry Creek drainage valley. No strands are mapped as crossing or projecting towards the Demonstration Reach. Seismically, the Healdsburg fault comprises an approximately one mile wide system of northwest trending, right-lateral strike-slip fault strands. These strands appear to be a northwest extension of the Rodgers Creek fault and define part of a complex seismic stepover with the Maacama fault to the north (McLaughlin and Sarna-Wojcicki, 2003). Both the Rodgers Creek and Maacama fault systems are zoned as active[2] under the State of California Alquist-Priolo (AP) Earthquake Fault Zoning Act (Bryant and Hart, 2007).

Although not currently zoned as active under the AP Act, workers mapping in the surrounding region considered some traces of the Healdsburg fault to be "recently active" (Huffman and Armstrong, 1980) or "Quaternary active" (Blake, Graymer, and Stamski, 2002). Based on available paleoseismic studies for the region and the structural relationship of the Healdsburg fault with the active Rodgers Creek and Maacama fault systems, the Healdsburg fault should be considered potentially active[3].

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2 Active faults are defined as those exhibiting either surface ruptures, topographic features created by faulting, surface displacements of Holocene (younger than about 11,000 years old) deposits, tectonic creep along fault lines, and/or close proximity to linear concentrations or trends of earthquake epicenters.

3 Potentially active faults displace geologic deposits of Pleistocene age (about 2 million to 11,000 years old).

6.2 Seismic Hazards

Based on the close proximity of the site to the Maacama, Rodgers Creek, and other major active faults in the area, there is a high potential for the site to experience moderate to very strong ground shaking during a major earthquake on one of these faults. The intensity of earthquake ground motion at the site will depend on the characteristics of the generating fault, the distance to the earthquake epicenter, the magnitude and duration of the earthquake, and specific site geologic conditions.

In addition, given the sandy nature of the materials and high elevation of the groundwater table encountered during the subsurface excavation, liquefaction may occur. It is possible that liquefaction or ground shaking may damage the bank stabilization structures due to lateral spreading. However, damage caused by lateral spreading should not cause a safety hazard for the local population since the improvements are for the remediation of an existing habitat and are not infrastructure related. Therefore, recommendations regarding liquefaction and liquefaction mitigation were not included in our scope of work.

6.3 Fault Rupture

Given the structural relationship of the Healdsburg fault with the active Rodgers Creek and Maacama faults, there is a reasonable chance of ground surface rupture along traces of the Healdsburg fault during a major earthquake on either of the active faults. Stereoscopic analysis of aerial photos and digital imagery suggests that one or more low sinuosity reaches of Dry Creek upstream/downstream of the Demonstration Reach may be structurally controlled along unmapped traces of the Healdsburg fault or other lineaments that may be associated with the fault. However, the Demonstration Reach is a higher sinuosity reach that does appear to be structural controlled. In addition, given the nature of the proposed habitat enhancements, any potential fault offset would be unlikely to have any significant impacts to the long term performance.

7.0 RECOMMENDATIONS

We believe the proposed construction is feasible from a geotechnical standpoint provided our geotechnical recommendations are incorporated into project design and construction. The primary geotechnical considerations for the site are the excavatability of the native subsurface material and stability of temporary and permanent slopes. In accordance with our scope of services, the following subsections present our recommendations for site grading, temporary and permanent slopes, and excavations.

7.1 Demolition & Clearing

Site demolition is expected to be minimal, but could include the removal of existing below-grade improvements, if any, that will interfere with the proposed construction. These could include utilities, culverts, and abandoned auto bodies.

Where utilities are to be abandoned and removed, they should be capped or plugged with grout at the Right-of-Way (ROW). Where it is feasible to abandon utilities in-place, utilities greater than three inches in diameter should be completely filled with flowable cement grout over their entire length. Where abandoned utilities are perpendicular to an excavation, they should be filled with grout to the nearest manhole or valve. It may be necessary to pothole utilities in several locations to facilitate and/or verify grouting. Utilities less than or equal to three inches in diameter can be plugged with concrete at the sides of the excavation. Existing utility lines, where encountered, should be addressed on a case-by-case basis.

Any demolition requiring excavation should be properly backfilled with engineered fill according to the recommendations provided later in this section.

7.2 Fill Material and Compaction Requirements

Fill is expected for grading along the Demonstration reach within creek bank stabilization construction areas. The areas of fill are anticipated to include the FES cells, area behind the FES cells, and within the rear portion of the log crib structure. On-site soil will be acceptable for use as general site fill for the FES cells and area behind them provided it is free of organic material and contains no rocks or lumps larger than four inches in greatest dimension. Rock fragments larger than four inches can be reused in the fill provided they are broken down to less than four inches in diameter.

Select fill should be used in the rear approximately two-thirds of the log crib system. Select fill should be free of organic matter or other deleterious material, contain no rocks or lumps larger than four inches in greatest dimension, and have a relatively low expansion potential (defined by liquid limit less than 40 and a plasticity index lower than 15). Acceptable select materials are sandy loam, loamy sand, or sand which generally corresponds to silty sand (SM), sand (SP), silty sandy gravel (GM), and gravelly silty sand (SM) as shown on the boring logs.

All fill material, including on-site fill, should be submitted to the Geotechnical Engineer for approval at least 72 hours before it is to be used on site. Where imported fill is required, the grading subcontractor should provide analytical test results or other suitable environmental documentation at least three days before use at the site indicating the proposed fill material is free of hazardous materials.

Where fill is required, the existing subgrade should be scarified to a depth of eight inches, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction.[4] Engineered fill should be placed in 8-inch thick loose lifts, moisture conditioned, and compacted as previously mentioned. However, 85 percent relative compaction is acceptable where vegetation or replanting is planned.

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[4] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by ASTM D1557-09 laboratory compaction procedure.

Prior to compaction, each layer should be spread evenly and mixed to obtain uniformity of material in each layer. The fill should be brought to a water content that will permit proper compaction by either (a) aerating the material if it is too wet, or (b) spraying the material with water if it is too dry. Compaction should be performed by footed rollers or other types of approved compaction equipment and methods. Backfill behind the log crib structures should be compacted using light (hand-operated) compaction equipment, unless larger equipment is approved by the designer. Compaction equipment should be of such design that they will be able to compact the fill to the specified density. Rolling of each layer should be continuous over its entire area and the equipment should make sufficient passes to ensure that the required density has been obtained. Flooding or jetting is not permitted with the exception of the 18-inch minus material within the crib structure.

The standard test used to define maximum densities and optimum moisture content of all compaction work shall be the Laboratory Test procedure ASTM D 1557.Field tests shall be expressed as a relative compaction in terms of the maximum dry density and optimum moisture content obtained in the laboratory by the foregoing standard procedure. Field density and moisture tests should be made in each compacted layer by the Geotechnical Engineer in accordance with Laboratory Test Procedure ASTM D 6938. When footed rollers are used for compaction, the density and moisture tests shall be taken in the compacted material below the surface disturbed by the roller. When these tests indicate that the compaction requirements on any layer of fill, or portion thereof, have not been met, the particular layer, or portion thereof, shall be reworked until the compaction requirements have been met.

7.3 Aeration

If wet subgrade conditions are encountered at the site, or the base of excavations or backfill areas become soft, unstable and/or disturbed by construction equipment, it may be necessary to stabilize the base of the excavation prior to fill placement. For granular soils, particularly gravels, installation of sumps to locally lower the water level will likely be sufficient to stabilize the material provided the pumps are large enough to keep up with infiltration. For clayey soils, the least costly stabilization measure typically consists of aeration (drying) of the wet soil to reduce its moisture content to a compactable level. However, depending on climatic conditions, several days to several weeks of relatively warm, dry weather may be required to dry the soil to an acceptable level. In addition, it is often necessary to turn the material several times a day to promote uniform drying. The soil will be deemed sufficiently aerated when the required degree of compaction can be achieved and/or the resulting subgrade surface is firm and unyielding.

7.4 Excavatability

Based on the results of our borings and test pits, we believe standard construction equipment, such as a hydraulic excavator, should be able to complete the excavations required for the proposed habitat improvements. The encountered materials were generally loose to medium dense and were easily excavated. No cemented soils or bedrock was encountered in our borings or test pits.

Based on the preliminary results of the seismic refraction line performed at Area D, bedrock appears to be about 2 to 9 feet below existing grade. The reported seismic velocities range from about 1,000 feet per second in overburden materials to over 6,800 feet per second in bedrock, which suggests the bedrock is rippable to marginally rippable using a CAT D9L bulldozer. We expect bedrock encountered at Crane Creek, Grape Creek, and Lambert Bridge will be similar.

7.5 Temporary Slopes

Temporary slopes are expected to be cut for bank stability construction in areas B and C. All temporary slopes should be excavated in accordance with the latest edition of the CAL-OSHA excavation and trench safety standards as a minimum (CCR, 2005). We understand some top-of-cut setback limitations may exist in these areas due to the proximity of an existing vineyard.

Test borings B1 and B2 were drilled through the proposed backslope materials for Area C. In boring B1, the upper 20 feet of the materials encountered consist of loose to medium dense sand, silty sand, and silty sandy gravel. Because this is a layered system, the maximum slope inclination is controlled by the least stable layer, in this case, the sand. At this location, it is our opinion that the soil should be preliminarily classified as Type C according to the CAL-OSHA classification system. The maximum allowable slope for Type C soil is 1.5H:1V. Vertical benches should not be cut into the base of temporary excavations. Type C should also be assumed for Areas B, where access limitations did not allow for site specific exploration to be performed.

At the location of boring B2, however, the upper 11 feet of the embankment consists of medium stiff to stiff sandy clay, which in our opinion can be preliminarily classified as a Type B soil. The maximum allowable slope for Type B soil is 1H:1V. Below this depth, the soil is classified as silty sand and gravel, and a direct shear test in the silty gravel indicates the material has some apparent cohesion. The silty sand and gravel is transitional between Type B and C soil and will require on-site classification during excavation to determine the CAL-OSHA soil type. Because OSHA does not allow layered systems with upper slopes steeper than lower slopes, we recommend cuts taller than 11 feet in the vicinity of boring B2 have an assumed inclination of 1.5H:1V for preliminary planning purposes.

The contractor should be responsible for all temporary slopes excavated at the site, and should designate one of their on-site employees as a "competent person" who is responsible for trench and excavation safety. The competent person should be responsible for determination of the correct CAL-OSHA soil type and should direct the excavation crews to use shallower slopes than presented above if appropriate. The competent person should also be prepared to flatten slopes if seepage is observed within the excavation.

If there is insufficient space to construct temporary slopes, temporary shoring may be required. Given the medium dense nature of the sands and gravels encountered at the site, we anticipate steel sheet piles, installed using a vibratory hammer mounted to a hydraulic excavator, are the most likely method of shoring to be used at the site. For design of temporary shoring, and assuming granular slope deposits, we recommend using active pressures of 35 and 65 pounds per cubic foot (pcf) for

level backslope conditions and a maximum backslope of 1.5H:1V, respectively. Passive resistance should be computed using allowable passive pressures of 300 and 145 pcf above and below the groundwater table, respectively. These passive pressures include a factor of safety of 1.5 to limit sheet pile deflections.

7.6 Permanent Slopes

Permanent slopes are expected to be cut for channel regrading and also filled as part of FES bank stabilization construction. Cut slopes will generally be excavated in gravelly sands and sandy gravels with no appreciable cohesion. Fill slopes will likely consist of predominately granular soils or fine grained soils with little cohesion. Therefore, all permanent slopes should have a maximum finished slope of 2H:1V. Permanent slopes should be revegetated and/or be covered in biodegradable fabrics as shown in the final construction plan set.

7.7 Slope Stability

We understand bank stabilization will be performed at Off Channel Enhancement Area B and C. For slope stability evaluation at these locations, a cohesion of 250 psf and a friction angle of 24 degrees is applicable for intact native soils or recompacted native soils within the upper 15 feet of the creek bank slopes. Where imported soil meeting the requirements presented in section 7.2 is used, or for intact native soils below 15 feet, a friction angle of 32 degrees (no cohesion) can be used.

7.8 Bearing Capacity

The proposed log crib structures for the Type 3 and 4 bank construction systems may bear on underlying soils at two points. An average bearing pressure may be imposed over the width of the overall crib structure, which is estimated to be on the order of 10 to 17 feet. However, pressures may be imposed locally on the 18-inch-minus rock streambed substrate (toe rock) buried beneath the toe of the wall. If it is necessary to evaluate the bearing capacity at these two points, we recommend using the allowable dead load bearing capacities presented in Table 2.

TABLE 2
ALLOWABLE DEAD LOAD BEARING CAPACITY

Structure	Min. Width, ft	Min. Embedment*, ft	Allowable Dead Load Bearing Pressure, psf
Type 3 Log crib (overall structure)	10	None required	2,000
Type 4 Log crib (overall structure)	17	None required	3,600
Toe Rock	3	5	3,000

*Measured vertically from creek bed to bottom of improvement

These values assume fully saturated (submerged) soil conditions and a factor of safety of at least 3 for dead load conditions. The toe pressures are provided as a check to ensure that excessive toe pressures at not imposed, which could cause the bank stabilization system to settle and/or rotate toward the channel.

7.9 Dewatering

Many of the proposed improvements will be constructed within the active creek channel. While construction of improvements outside of and above the water line of the active creek is feasible with flow in the channel, many of the proposed improvements will require work below the water line. Working within areas of active creek flow has several limitations which include, but are not limited to:

- the presence of soft/saturated soils and an unstable working base;
- difficulty in obtaining proper compaction;
- increased turbidity in the water due to disturbance of saturated soils, especially silty sand; and
- the potential for introduction of hydrocarbons into the water from construction equipment.

Therefore, where fill is to be placed and/or where improvements are to be constructed below the water line, the work areas must be dewatered. The water level must be lowered to at least 2 feet below the bottom of the excavations. Due to the extent of the proposed improvements, it is suggested to consider diversion of water around the entire construction zone.

Where new side channels are to be excavated, it may be feasible to excavate the channels without dewatering provided it is not necessary to place new fill soils. However, if dewatering is not performed, it should be expected that disturbed granular soils may slump at the water line and flattening of slopes may be necessary. This behavior was observed in our test pit excavations where flowing sands were encountered at the water table. As a result, we recommend that dewatering be performed for all grading (cut or fill) performed below the water table.

8.0 SERVICES DURING CONSTRUCTION

The recommendations provided in this report are based on the assumption that SAGE will be retained to provide plan review and observation and testing services during construction in order to evaluate compliance with our recommendations. Prior to construction, we should review the excavation and/or shoring plans prepared by the contractor. During construction, we should periodically check the materials exposed due to excavation of temporary and permanent slopes. These observations will allow us compare the subsurface conditions observed during construction with those encountered during our investigation and allow us to assess the contractor's work with respect to the project plans and specifications and the recommendations presented herein. If SAGE

is not retained for these services, we cannot assume responsibility for any and all potential claims that may arise during or after construction as a result of misuse or misinterpretation of SAGE's report by others.

9.0 LIMITATIONS

This report has been prepared for the sole use of Sonoma County Water Agency and their agents specifically for the design of the Dry Creek Habitat Enhancement Demonstration Projects described herein. The opinions, conclusions, and recommendations contained in this report are based upon the information obtained from our site subsurface exploration, our engineering studies, experience, and engineering judgment, and have been formulated in accordance with generally accepted geotechnical engineering practices that exist at the time this report was prepared. No other warranty, expressed or implied, is made or should be inferred. In addition, the recommendations presented in this report are based on the subsurface conditions encountered in a limited number of test pits and borings. Actual conditions may vary. If subsurface conditions encountered in the field differ from those described in this report, we should be consulted to determine if changes to our conclusions or supplemental recommendations are required.

The opinions presented in this report are valid as of the date of this report for the property being evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man. If site conditions vary from those described herein, we should be consulted to evaluate the impact of the changes, if any. In addition, changes in applicable standard of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of SAGE's control. In any case, this report should not be relied upon after a period of three years without prior review and approval by SAGE.

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FIGURES

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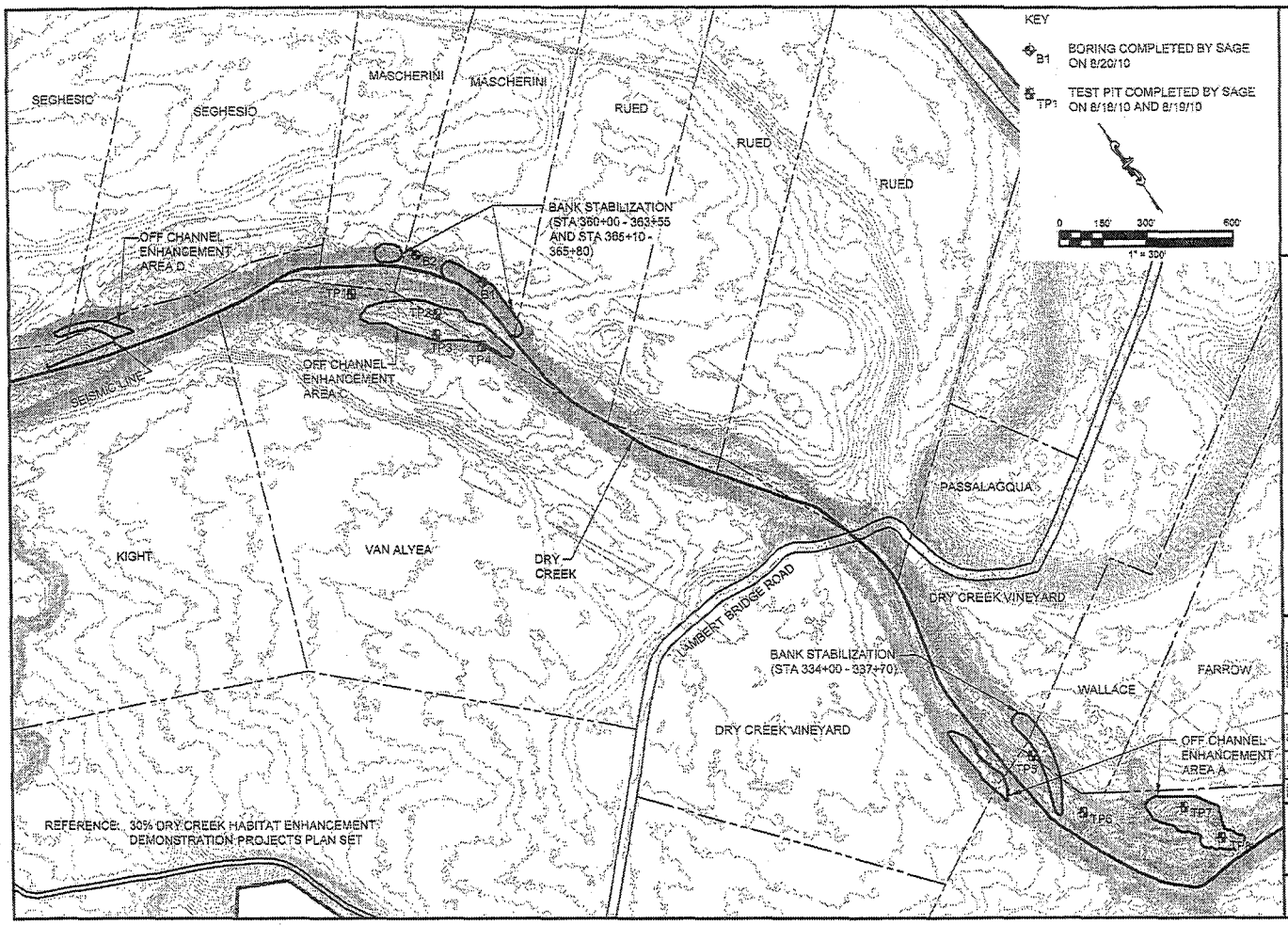
DRY CREEK HABITAT ENHANCEMENT DEMONSTRATION PROJECTS

SONOMA COUNTY — SITE LOCATION MAP — CALIFORNIA

FIG 1

DATE: 10/13/11	FILE: 07-082.02	SCALE: NONE	JOB NO.: 07-082.02


KEY
B1 BORING COMPLETED BY SAGE ON 8/20/10
TP1 TEST PIT COMPLETED BY SAGE ON 8/18/10 AND 8/19/10
0 150' 300' 600'
1" = 300'
SEGHESIO
SEGHESIO
MASCHERINI
MASCHERINI
RUED
RUED
RUED
BANK STABILIZATION (STA 360+00 - 363+55 AND STA 365+10 - 365+80)
OFF CHANNEL ENHANCEMENT AREA D
TP5
TP2
TP3
TP4
OFF CHANNEL ENHANCEMENT AREA C
SEISMIC LINE
KIGHT
VAN ALYEA
DRY CREEK
PASSALAGQUA
LAMBERT BRIDGE ROAD
DRY CREEK VINEYARD
BANK STABILIZATION (STA 334+00 - 337+70)
WALLACE
FARROW
DRY CREEK VINEYARD
OFF CHANNEL ENHANCEMENT AREA A
TP6
TP7
REFERENCE: 30% DRY CREEK HABITAT ENHANCEMENT DEMONSTRATION PROJECTS PLAN SET
sage
SUBSURFACE EXPLORATION MAP
DRY CREEK HABITAT ENHANCEMENT DEMONSTRATION PROJECTS
FIG 2

APPENDIX A
Field Exploration Program

A.1 Field Exploration Program

Our field exploration program consisted of excavating eight (8) test pits and drilling two (2) small-diameter soil borings. The approximate test pit and boring locations, designated TP1 through TP8 and B1 through B2, respectively, are presented on Figure 2.

Prior to the start of drilling, SAGE obtained a drilling permit from the Sonoma County Permit and Resource Management Department (PRMD) and notified Underground Service Alert (USA) at least 48 hours prior to the start of work. Furthermore, all borings and test pits were cleared by a private utility locator.

The test pits were excavated by Luce Backhoe Excavation of Santa Rosa, California. Test pits TP1 through TP4 were excavated on August 18, 2010 using a CAT 416C rubber-tire backhoe equipped with a 24-inch bucket. Test pits TP5 through TP8 were excavated on August 19, 2010 using a CAT 315L track-mounted excavator equipped with a 42-inch bucket. The test pit depths were limited to 13.5 feet or less due to caving conditions below groundwater.

Borings B1 and B2 were drilled on August 20, 2010 by Clear Heart Drilling of Santa Rosa, California. The borings were drilled using a truck-mounted DR5K1 drill rig equipped with seven-inch-diameter hollow stem augers. B1 and B2 were advanced to a depth of 41.5 feet below the existing ground surface, which corresponds to 21.3 and 23.2 feet below the existing channel bed invert, respectively.

During excavation of the test pits and drilling of the borings, our geologist logged the materials encountered and obtained representative samples for visual classification and laboratory testing. The materials encountered were classified in general accordance with the Unified Soil Classification System (USCS) as summarized on Figure A-1. Logs of the borings and test pits are presented as Figures A-2 through A-3 and Figure A-4, respectively.

Representative soil samples were recovered during drilling using the following sampler types:

- Modified California (MCA) split-barrel sampler with a 3.0-inch-outside diameter fitted with 2.43-inch-inside-diameter, six-inch-long brass or stainless steel liners;
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch-outside diameter, without liners; and

Both split-barrel samplers were driven with a 140-pound, safety (rope and cathead) hammer falling 30 inches. The blow counts required to drive the samplers over a standard 18-inch-drive were recorded in six-inch increments in the field. Where refusal was encountered, which is defined as greater than 50 blows over any six-inch increment, drive lengths less than 12 inches were recorded. The blow counts for the final 12-inches of the drive (less in the case of refusal) were added to develop the reported blow count. The blow counts for the MCA sampler were corrected for the effects of sampler size and converted to SPT N-values using a conversion factor of 0.6. The final, corrected values for each drive are presented on the boring logs and represent N_{60} values.

Due to the proximity of the borings to Dry Creek, grout migration through the coarse grained alluvial deposits and into the creek was a concern. To mitigate this, our geologist obtained verbal

approval from the on-site Sonoma County PRMD inspector to backfill the borings with soil cuttings to 15 feet, and top off with neat cement grout. The remaining cuttings were spread out on the ground surface adjacent to the boring.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > No. 200 sieve size)	Gravels (More than half of coarse fraction > No. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction > No. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils (more than half of soil < No. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No.4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
Silt and Clay	Below No. 200	Below 0.074

TYPES OF STRENGTH TESTS

PP	Pocket Penetrometer
TV	Field Torvane
LVS	Laboratory Vane Shear
UC	Unconfined Compression
TXUU	Triaxial, unconsolidated, undrained
DS	Direct Shear

▽ Unstabilized (initial) groundwater level

▼ Stabilized groundwater level

SAMPLER TYPE

C — Core barrel

O — Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT — Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

ST — Shelby tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

BULK — Disturbed grab sample

CA — California split-barrel sampler with 2.5-inch outside diameter and 1.93-inch inside diameter

MCA — Modified California split-barrel sampler with 3.0-inch outside diameter and 2.5-inch inside diameter

SPT — Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

o — Sampling attempted without recovery

DRY CREEK HABITAT ENHANCEMENT
Demonstration Reach
Sonoma County California

SOIL CLASSIFICATION CHART

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Project No. 07-082.02 Task 3.02

Date 10/13/11 | Figure A-1

PROJECT:	Dry Creek Demonstration Reach Sonoma County, CA	LOG OF BORING B1	Sheet 1 of 2
BORING LOCATION:	See Figure 2	DRILLING SUBCONTRACTOR:	Clear Heart Drilling
DATE STARTED: 8/20/2010	DATE FINISHED: 8/20/2010	DRILL RIG:	DR5KI Truck Mounted
LOGGED BY:	D. Kennedy	DRILLING METHOD:	7-inch hollow stem auger
ELEVATION (FT): 144.2	DATUM: NAVD 88		
GW DEPTH (FT): 22.7	GW DATE: 8/20/2010	HAMMER TYPE:	Rope and cathead (safety)
CASING NOTES:	N/A	HAMMER WT (LBS): 140	HAMMER DROP (IN): 30
BACKFILL MATERIAL:	Soil cuttings & neat cement	SAMPLERS:	MCA, SPT

DEPTH (FT)	ELEV. (FT)	SAMPLE TYPE	SAMPLE	SPT N60 VALUE	LITHOLOGY	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	FINES (%)	TYPE of TEST	UNCONFINED STRENGTH (tsf)	SHEAR STRENGTH (ksf)	PLASTICITY LL	PLASTICITY PI
						SILTY SAND (SM) yellow brown, loose, moist, fine grained sand								
1	143.2													
2	142.2													
3	141.2	MCA		7	SM									
4	140.2													
5	139.2													
6	138.2	MCA		12		SAND (SP) brown, medium dense, moist, fine grained sand, trace silt, locally grades to medium grained sand								
7	137.2													
8	136.2	MCA		8	SP	loose, with SILTY SAND (SM) interbeds								
9	135.2													
10	134.2													
11	133.2	MCA		12										
12	132.2				SM	SILTY SAND (SM) brown, medium dense, moist, fine grained sand, approximately 25% fines								
13	131.2					gravel at 13.5 based on change in drilling								
14	130.2					SILTY SANDY GRAVEL (GM) brown, medium dense, moist, fine gravel with some coarse gravel, medium to coarse sand, trace fines								
15	129.2													
16	128.2	SPT		25	GM									
17	127.2													
18	126.2													
19	125.2					GRAVEL WITH SILT AND SAND (GW-GM) brown, medium dense, moist, primarily fine gravel with coarse sand, medium grained sand lenses								

MCA and SPT blow counts converted to SPT N60 values using conversion factors of 0.6 and 1.0, respectively.

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Figure: A-2

LOG OF BORING 07-082 GINT BORING LOGS.GPJ SAGE.GDT 10/13/11

PROJECT: Dry Creek Demonstration Reach, Sonoma County, CA

LOG OF BORING B1

DEPTH (FT)	ELEV. (FT)	SAMPLE TYPE	SPT N60 VALUE	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	FINES (%)	TYPE of TEST	UNCONFINED STRENGTH (tsf)	SHEAR STRENGTH (ksf)	PLASTICITY LL	PLASTICITY PI
				locally								
21	123.2	MCA	18	**Approximate elevation of channel bed invert** GRAVEL WITH SILT AND SAND (GW-GM) (Con't)			6.6					
22	122.2			***Groundwater Encountered***								
23	121.2			softer drilling below 22.7', groundwater encountered								
24	120.2			GW-GM								
25	119.2			gragvel size increases, fine to coarse gravel, decreased sand content locally, wet								
26	118.2	MCA	17									
27	117.2											
28	116.2			GRAVELLY SILTY SAND (SW)								
29	115.2			brown gray, medium dense, wet, medium to coarse grained sand, fine to coarse gravel (1/4" to 1"), some silt								
30	114.2											
31	113.2	MCA	16									
32	112.2											
33	111.2											
34	110.2											
35	109.2			SW dense, coarse grained sand, gravels up to 2.5"								
36	108.2	MCA	31									
37	107.2											
38	106.2											
39	105.2											
40	104.2			medium to coarse grained sand, reduced gravel content, with some clay locally								
41	103.2	SPT	36									
42	102.2											
43	101.2											

Boring terminated at a depth of 41.5 feet below existing ground surface.
MCA and SPT blow counts converted to SPT N60 values using conversion factors of 0.6 and 1.0, respectively.

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Figure: A-2

PROJECT:	Dry Creek Demonstration Reach Sonoma County, CA	LOG OF BORING B2	Sheet 1 of 2
BORING LOCATION:	See Figure 2	DRILLING SUBCONTRACTOR:	Clear Heart Drilling
DATE STARTED: 8/20/2010	DATE FINISHED: 8/20/2010	DRILL RIG:	DR5KI Truck Mounted
LOGGED BY:	D. Kennedy	DRILLING METHOD:	7-inch hollow stem auger
ELEVATION (FT): 142.3	DATUM: NAVD 88		
GW DEPTH (FT): 20.0	GW DATE: 8/20/2010	HAMMER TYPE:	Rope and cathead (safety)
CASING NOTES:	N/A	HAMMER WT (LBS): 140	HAMMER DROP (IN): 30
BACKFILL MATERIAL:	Soil cuttings and neat cement	SAMPLERS:	MCA, SPT

DEPTH (FT)	ELEV. (FT)	SAMPLE TYPE	SAMPLE	SPT N60 VALUE	LITHOLOGY	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	FINES (%)	TYPE of TEST	UNCONFINED STRENGTH (tsf)	SHEAR STRENGTH (ksf)	PLASTICITY LL	PLASTICITY PI
						SANDY CLAY (CL) brown, medium stiff to stiff, moist, fine grained sand								
1	141.3													
2	140.3													
3	139.3													
4	138.3													
5	137.3													
6	136.3	MCA		6	CL	silty sand lense			64.6					
7	135.3													
8	134.3	MCA	○	6										
9	133.3													
10	132.3													
11	131.3	MCA		6		DSDC: Ø=24°, c=250psf; See Appendix B	20.8	88						
						SILTY SAND (SM) brown, loose, moist, fine grained sand								
12	130.3													
13	129.3				SM									
14	128.3													
15	127.3					GRAVELLY SILTY SAND (SM) brown, medium dense to dense, moist, medium to coarse sand, fine gravel, trace fines, weakly cemented locally								
16	126.3	MCA		26										
17	125.3													
18	124.3					**Approximate elevation of channel bed invert**								
19	123.3					8/20/2010 ***Groundwater Encountered***								

MCA and SPT blow counts converted to SPT N60 values using conversion factors of 0.6 and 1.0, respectively.

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Project No: 07-082.02

Figure: A-3

PROJECT: Dry Creek Demonstration Reach, Sonoma County, CA

LOG OF BORING B2

DEPTH (FT)	ELEV. (FT)	SAMPLE TYPE	SAMPLE	SPT N60 VALUE	LITHOLOGY	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	FINES (%)	TYPE of TEST	UNCONFINED STRENGTH (tsf)	SHEAR STRENGTH (ksf)	PLASTICITY LL	PLASTICITY PI
					SM	some clay GRAVELLY SILTY SAND (SM) (Con't)								
21	121.3	SPT		24										
22	120.3													
23	119.3													
24	118.3													
25	117.3													
26	116.3	MCA		6	SC	CLAYEY SAND (SC) gray, loose, wet, fine grained sand, marsh or overbank deposit								
27	115.3	SPT		28	SM	GRAVELLY SILTY SAND (SM) brown, medium dense to dense, wet, medium to coarse sand, fine gravel with some coarse gravel, with thin clay lenses								
28	114.3													
29	113.3				SP	SAND (SP) gray, loose to medium dense, wet, medium grained sand, trace fines								
30	112.3													
31	111.3	MCA		17		GRAVELLY SILTY SAND (SM) brown, medium dense, wet, medium to coarse grained sand, fine gravel with some coarse gravel								
32	110.3				SM									
33	109.3													
34	108.3													
35	107.3					SILTY SANDY GRAVEL (GM) brown to yellow brown, dense, wet, with locally clean gravel lenses								
36	106.3	MCA		30										
37	105.3													
38	104.3				GM									
39	103.3													
40	102.3					little to no coarse gravel								
41	101.3	SPT		25										
42	100.3													
43	99.3													

LABORATORY TEST DATA

Boring terminated at a depth of 41.5 feet below existing ground surface.
MCA and SPT blow counts converted to SPT N60 values using conversion factors of 0.6 and 1.0, respectively.

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Project No: 07-082.02

Figure: A-3

FIGURE A-4 – LOGS OF TEST PITS TP1 THROUGH TP8

Test Pit Number	Depth (feet)	Soil Classification	Soil Description
TP1 (El. 126.0)	0' – 1'	SILTY GRAVEL (GM)	brown, loose to medium dense, dry with some sand, fine to coarse gravel, with organics
	1' – 6'	SANDY GRAVEL (GW)	brown, loose, moist to 3.5', wet below 3.5', primarily fine gravel with some coarse gravel, medium to coarse sand, with fines; Laboratory Gradation: 64.3% gravel, 35.4% sand, 0.3% fines Groundwater encountered at El. 122.8'
TP2 (El. 128.3)	0' – 0.5'	SILTY GRAVEL (GM)	brown, loose to medium dense, dry, with some sand, fine to coarse gravel, with organics
	0.5' – 3.5'	SANDY GRAVEL (GW – GM)	brown, loose, dry to moist, primarily fine gravel with some coarse gravel, medium to coarse sand, trace fines, clean gravel interbeds locally, estimate 50% gravel (40% fine, 10% coarse), 45% sand, 5% fines
	3.5' – 10'	GRAVELLY SAND (SW)	brown gray, medium dense, moist to 6.5', wet below 6.5', medium to coarse grained sand, gravel primarily fine with some coarse, trace fines; Laboratory Gradation: 37.7% gravels, 61.7% sand, 0.6% fines Groundwater encountered at El. 121.8'
TP3 (El. 131.1)	0' – 0.5'	GRAVELLY SILT (ML)	brown, soft to medium stiff, dry, with some sand, fine to coarse gravel, with organics
	0' – 3.0'	SANDY GRAVEL (GW – GM)	brown, loose to medium dense, dry, primarily fine gravel with some coarse gravel, medium to coarse sand, trace fines, clean gravel interbeds locally ,estimate 60% gravel (50% fine, 10% coarse), 35% sand, 5% fines
	3.0' – 4.5'	SAND WITH GRAVEL (SP)	brown, loose, dry to moist, medium grained sand, with fine to coarse gravel, some organics/roots, estimate 70% sand, 30% gravel
	4.5' – 12'	GRAVELLY SAND (SW)	brown gray, medium dense, moist to 8.5', wet below 8.5', medium to coarse grained sand, fine to coarse gravel, trace fines, estimate 60% sand, 35% gravel (25% fine, 10% coarse), 5% fines Groundwater encountered at El. 122.6'

TP4 (El. 126.6)	0' – 4'	GRAVELLY CLAY (CL)	brown, medium stiff, dry to moist, with some sand, fine to coarse gravel, with organics
	4' – 11'	SANDY GRAVEL (GW – GM)	brown gray, medium dense, moist to 4.4', wet below 4.4', primarily coarse grained sand with medium grained sand, fine to coarse gravel, trace fines, estimate 35% sand, 55% gravel (35% fine, 20% coarse), 10% fines, trace cobble Groundwater encountered at El. 122.2'
TP5 (El. 119.2)	0' – 10'	SANDY GRAVEL (GW –GM)	brown gray, loose to medium dense, moist, wet below 2.5', fine to coarse gravel, medium to coarse sand, trace cobbles (up to 6"), traces fines, estimate 60% gravel (40% fine, 20% coarse), 30% sand, 5% cobbles, 5% fines Groundwater encountered at El. 117.6'
TP6 (El. 118.5)	0' – 10.5'	SANDY GRAVEL (GW – GM)	brown gray, loose to medium dense, moist, wet below 2.4', fine to coarse gravel, medium to coarse sand, trace cobbles (up to 10"), trace fines; Laboratory Gradation: 64.4% gravel, 35.1% sand, 0.5% fines CLAYEY SAND (SC) interbed at ~10', medium grained, increased clay content locally, estimate interbed is less than 0.5' thick Groundwater encountered at El. 115.9'
TP7 (El. 121.4)	0' – 3.8'	SANDY GRAVEL (GW)	brown gray, loose to medium dense, moist, fine to coarse gravel, medium to coarse sand, trace fines, estimate 60% gravel (40% fine, 20% coarse), 35% sand, 5% fines
	3.8' – 5.5'	SAND (SP)	olive brown, loose to medium dense, moist, medium grained sand, trace coarse grained sand and fine gravel
	5.5' – 13.5'	SANDY GRAVEL (GW – GM)	brown gray, loose to medium dense, moist, wet below 6', fine to coarse gravel, medium to coarse sand, trace fines, trace cobbles, cobbles up to 6", estimate: 50% gravel (30% fine, 20%), 40% sand, 5% cobbles, 5% fines Groundwater encountered at El. 115.5'
TP8 (El. 118.0)	0' – 9'	SANDY GRAVEL (GW)	brown gray, loose to medium dense, dry in upper 1.5', moist 1.5' – 3.1', wet below 3.1', fine to coarse gravel, medium to coarse sand, trace fines (<5%), no cobbles observed, thin (~3") clean gravel (fine) lenses visible in upper 3' where pit can be safely accessed, increased sand content locally, estimate 50% gravel (30% fine, 20% coarse), 45+% sand, <5% fines Groundwater encountered at El. 114.9'

APPENDIX B
Laboratory Test Results

B.1 Laboratory Testing

Representative soil samples obtained from the borings were reviewed in our office to confirm field classifications. Representative samples were selected and submitted for laboratory testing. Samples were selected based on how representative they were of surrounding materials. Laboratory testing was performed to determine the following properties:

- Percent Passing the No. 200 sieve (Fines Content) per ASTM D1140;
- Consolidated-Drained Direct Shear (DSCD) per ASTM D3080;
- Particle Size Analysis per ASTM D422;

LABORATORY TESTING SUMMARY

Boring/Test Pit	Laboratory Test	Approximate Samp Depth (ft)
B1	Percent Passing No. 200	21
B2	Percent Passing No. 200	5.5
B2	Direct Shear	11
TP1	Particle Size Analysis	3
TP2	Particle Size Analysis	5.5
TP6	Particle Size Analysis	6.5

The laboratory reporting sheets for the laboratory testing follow. Note that there are two Direct Shear result sheets because the test was run twice with two different normal loads on the sample. A minimum of two different normal loads are required in order to calculate cohesion and internal friction.

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GRAIN SIZE ANALYSIS – WASH

Project	Sage Project No. 07-082.02	Job No.	110033		
Project Name	Dry Creek Phase 3	Boring No.	B1	Sample No.	6
Tested By	RD	Depth of Sample	21.0 ft		
Reviewed By	PF	Date of Testing	9/10/10		

Before Wash		After Wash	
Wt. of dry sample + Container (g)	941.6	Wt. of dry sample + Container (g)	889.3
Wt. of Container (g)	143.6	Wt. of Container (g)	143.6
Wt. of dry sample (g)	798.0	Wt. of dry sample(g)	745.7

Sieve analysis and grain shape

Sieve No.	Diam. (mm)	Wt. Retained	% Retained	% Passing
#200	0.075	745.7	93.4	6.6

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GRAIN SIZE ANALYSIS – WASH

Project	Sage Project No. 07-082.02	Job No.	110033		
Project Name	Dry Creek Phase 3	Boring No.	B2	Sample No.	1
Tested By	RD	Depth of Sample	5.5 ft		
Reviewed By	PF	Date of Testing	9/13/10		

Before Wash		After Wash	
Wt. of dry sample + Container (g)	785.5	Wt. of dry sample + Container (g)	359.2
Wt. of Container (g)	125.2	Wt. of Container (g)	125.2
Wt. of dry sample (g)	660.3	Wt. of dry sample(g)	234.0

Sieve analysis and grain shape

Sieve No.	Diam. (mm)	Wt. Retained	% Retained	% Passing
#200	0.075	234.0	35.4	64.6

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DIRECT SHEAR TEST RESULT

(ASTM D3080)

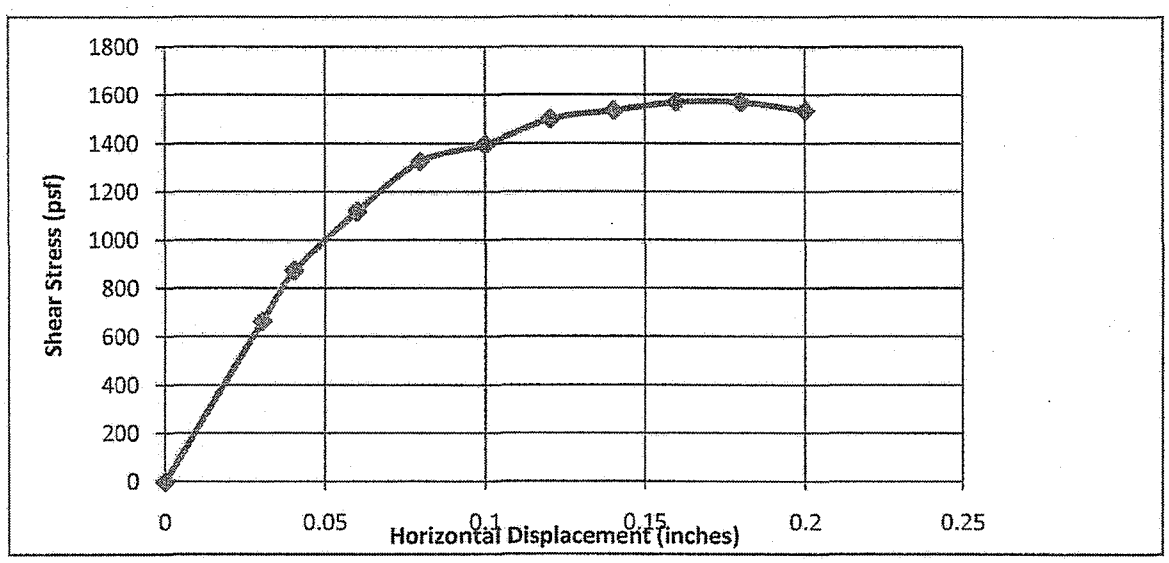
Horizontal Displacement (inches)	Shear Stress (psf)
0	0
0.03	663
0.04	873
0.06	1117
0.08	1326
0.1	1396
0.12	1501
0.14	1536
0.16	1571
0.18	1571
0.2	1536

Client: SAGE P.N. 110033
Date: 9-15-10
Sample No: B2-2 @ 11 ft.
Initial water content: 20.8 %
Final water content: 24.6 %
Dry density: 87 pcf

Sample sheared at strain rate = 0.031 in/min
Sample submerged, unconsolidated

Maximum Vert. Displacement: 0.173 inches

Normal Load = 3000 psf



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DIRECT SHEAR TEST RESULTS

(ASTM D3080)

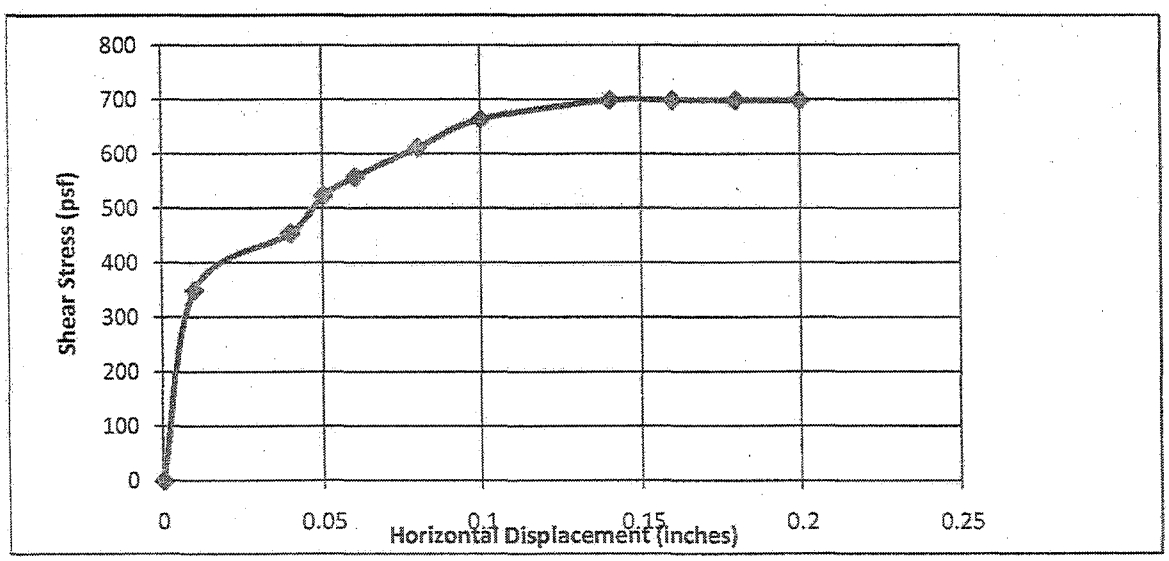
Horizontal Displacement (inches)	Shear Stress (psf)
0	0
0.01	349
0.04	454
0.05	523
0.06	558
0.08	611
0.1	663
0.14	698
0.16	698
0.18	698
0.2	698

Client: SAGE P.N. 110033
Date: 9-15-10
Sample No: B2-2 @ 11 ft.
Initial water content: 20.8 %
Final water content: 27.7%
Dry density: 88 pcf

Sample sheared at strain rate = 0.031 in/min
Sample submerged, unconsolidated

Maximum Vert. Displacement: 0.12 inches

Normal Load = 1000 psf



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GRAIN SIZE ANALYSIS – MECHANICAL

Project	Sage Project No. 07-082.02	Job No.	110033		
Project Name	Dry Creek Phase 3	Boring No.	TP1	Sample No.	1
Tested By	RD	Depth of Sample	3 ft		
Reviewed By	PF	Date of Testing	9/9/10		

Wt. of dry sample + Container (g)	2329.0
Wt. of Container (g)	363.0
Wt. of dry sample (g)	1966.0

Sieve analysis and grain shape

Sieve No.	Diam. (mm)	Wt. Retained	% Retained	% Passing
3"	75.0	0	0	100.0
2"	50.0	0	0	100.0
1 ½"	37.5	0	0	100.0
1"	25.0	103	5.2	94.8
¾"	19.0	356	18.1	81.9
½"	12.7	645	32.8	67.2
3/8"	9.5	838	42.6	57.4
#4	4.75	1265	64.3	35.7
#10	2.0	1628	82.8	17.2
#20	0.850	1836	93.4	6.6
#40	0.425	1896	96.4	3.6
#60	0.250	1934	98.4	1.6
#140	0.106	1957	99.5	0.5
#200	0.075	1960	99.7	0.3
PAN		1966	100	0.0

% passing = 100 - ∑% retained

% Cobbles >3"	**% Gravel** <3" to >#4	**% Sand** <#4 to >#200	**% Silt & Clay** <#200
0	64.3	35.4	0.3

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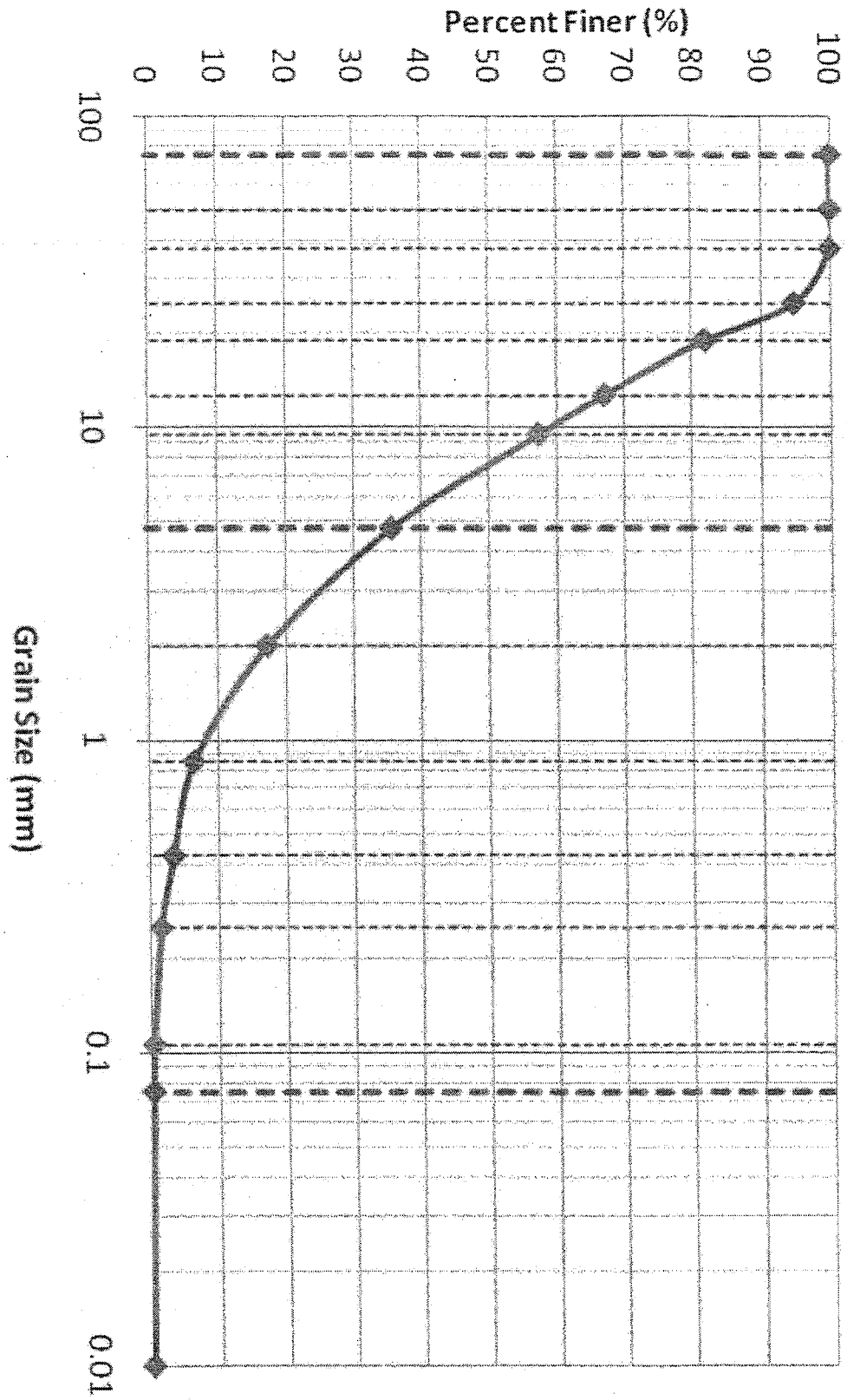

110033 TP1-1-3ft

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GRAIN SIZE ANALYSIS – MECHANICAL

Project	Sage Project No. 07-082.02	Job No.	110033		
Project Name	Dry Creek Phase 3	Boring No.	TP2	Sample No.	1
Tested By	RD	Depth of Sample	5.5 ft		
Reviewed By	PF	Date of Testing	9/9/10		

Wt. of dry sample + Container (g)	3156.0
Wt. of Container (g)	363.0
Wt. of dry sample (g)	2793.0

Sieve analysis and grain shape

Sieve No.	Diam. (mm)	Wt. Retained	% Retained	% Passing
3"	75.0	0	0	100.0
2"	50.0	0	0	100.0
1 ½"	37.5	0	0	100.0
1"	25.0	0	0	100.0
¾"	19.0	145	5.2	94.8
½"	12.7	320	11.4	88.5
3/8"	9.5	519	18.6	81.4
#4	4.75	1052	37.7	62.3
#10	2.0	1743	62.8	37.2
#20	0.850	2225	79.7	20.3
#40	0.425	2458	88.0	12.0
#60	0.250	2617	93.7	6.3
#140	0.106	2758	98.7	1.3
#200	0.075	2776	99.4	0.6
PAN		2792	100.0	0.0

% passing = 100 - ∑% retained

% Cobbles >3"	**% Gravel** <3" to >#4	**% Sand** <#4 to >#200	**% Silt & Clay** <#200
0	37.7	61.7	0.6

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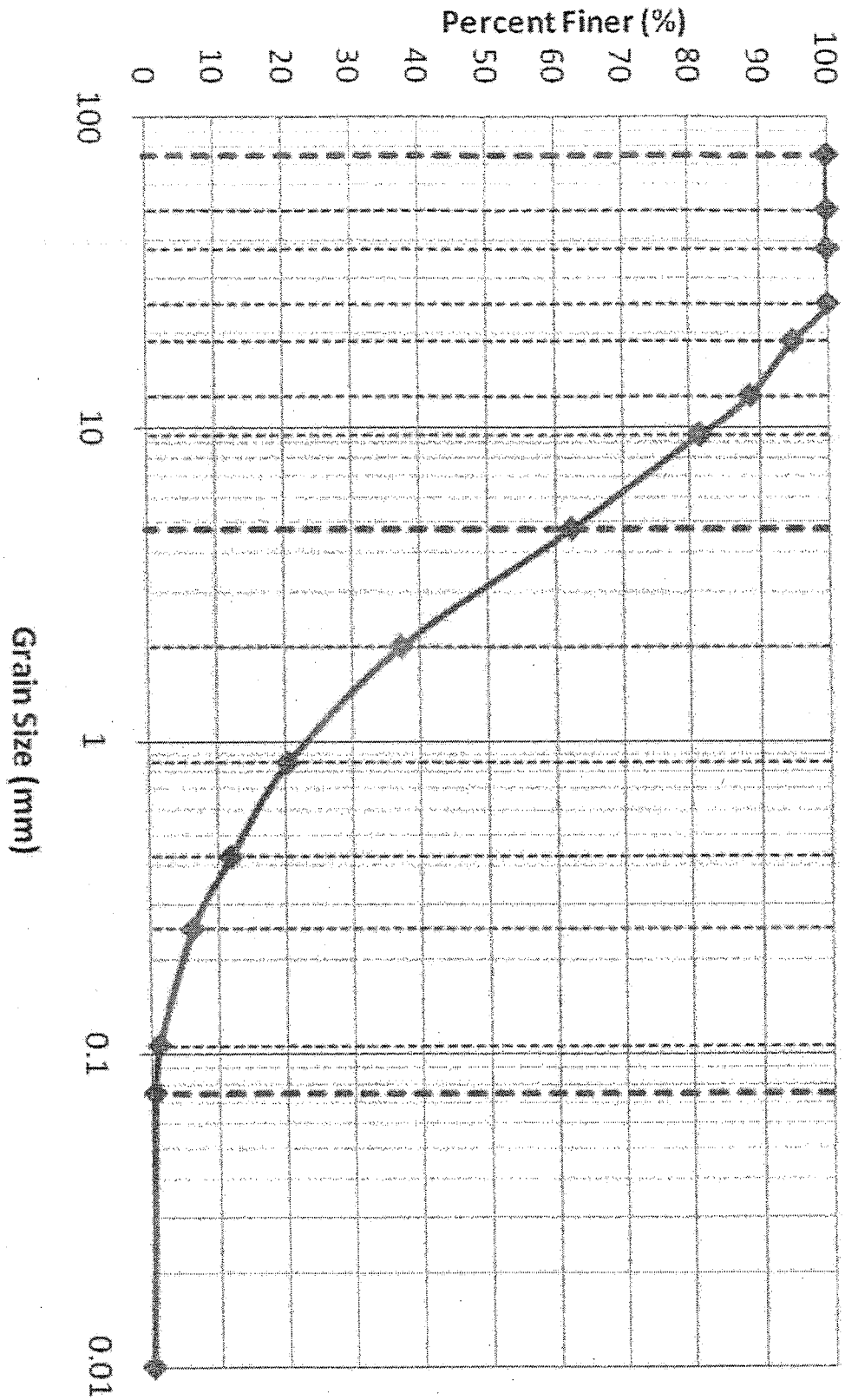

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GRAIN SIZE ANALYSIS – MECHANICAL

Project	Sage Project No. 07-082.02	Job No.	110033		
Project Name	Dry Creek Phase 3	Boring No.	TP6	Sample No.	1
Tested By	RD	Depth of Sample	6.5 ft		
Reviewed By	PF	Date of Testing	9/8/10		

Wt. of dry sample + Container (g)	3339.0
Wt. of Container (g)	947.0
Wt. of dry sample (g)	2392.0

Sieve analysis and grain shape

Sieve No.	Diam. (mm)	Wt. Retained	% Retained	% Passing
3"	75.0	0	0.0	100.0
2"	50.0	0	0.0	100.0
1 ½"	37.5	159	6.6	93.4
1"	25.0	303	12.7	87.3
¾"	19.0	497	20.8	79.2
½"	12.7	807	33.7	66.3
3/8"	9.5	1044	43.6	56.4
#4	4.75	1540	64.4	35.6
#10	2.0	1957	81.8	18.2
#20	0.850	2174	90.9	9.1
#40	0.425	2285	95.5	4.5
#60	0.250	2342	97.9	2.1
#140	0.106	2372	99.2	0.8
#200	0.075	2379	99.5	0.5
PAN		2392	100.0	0.0

% passing = 100 - ∑% retained

% Cobbles >3"	**% Gravel** <3" to >#4	**% Sand** <#4 to >#200	**% Silt & Clay** <#200
0	64.4	35.1	0.5

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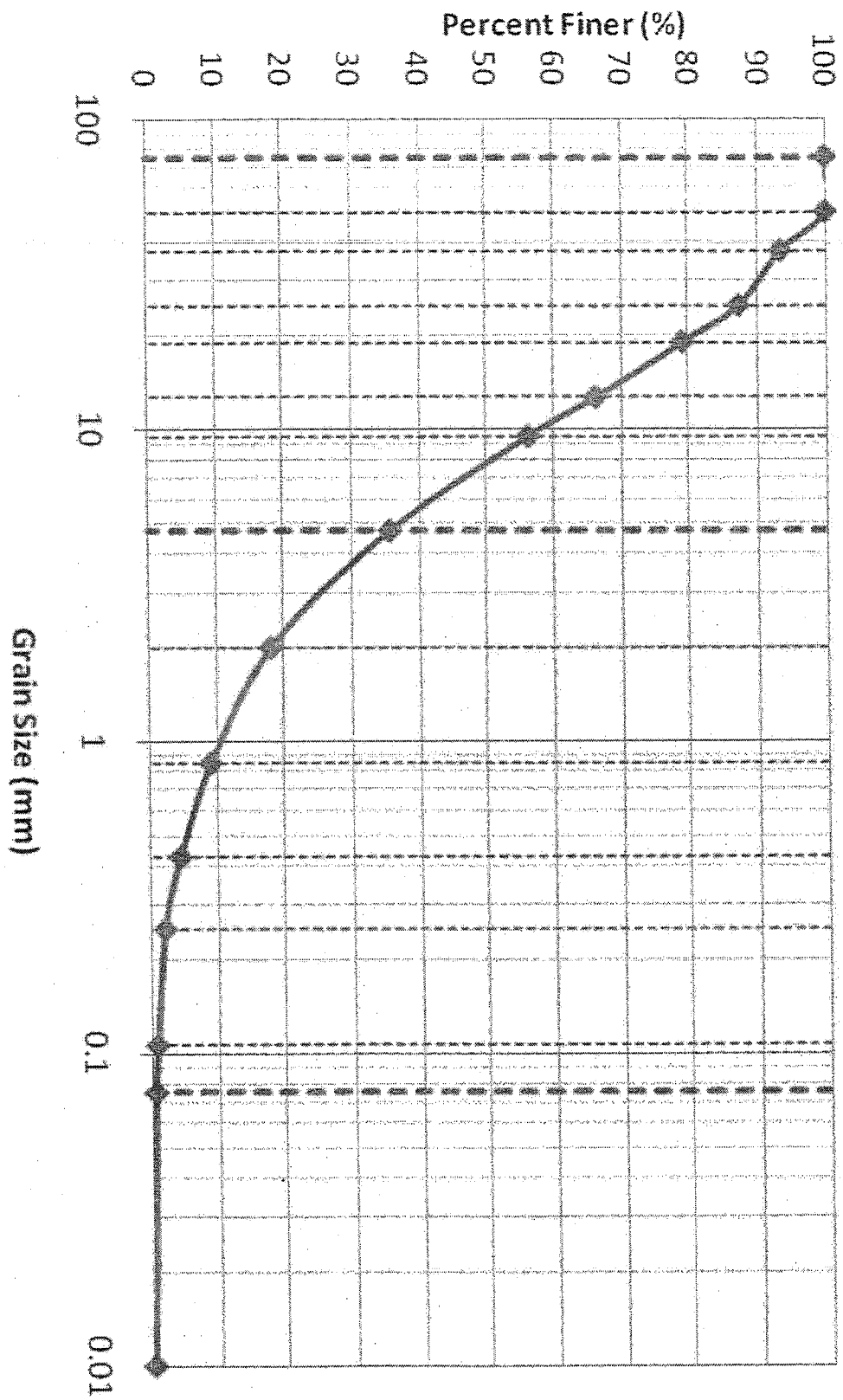

110033 TP6-1-6.5ft

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APPENDIX C
Geophysical Survey Report

SANDERS & ASSOCIATES GEOSTRUCTURAL ENGINEERING

sage

INTEGRATING EARTH & STRUCTURE

NORCAL GEOPHYSICAL CONSULTANTS, INC.

November 09, 2010

Mr. Drew G. Kennedy
Sanders & Associates Geostructural Engineering
4180 Douglas Blvd., Ste. 100
Granite Bay, CA 95746

Subject: Seismic Refraction Survey
Dry Creek Habitat Enhancement Demonstration Project
Off Channel Enhancement Area D, Demonstration Reach
Sonoma County, California
NORCAL Job # 10-916.04

Dear Mr. Kennedy:

This report presents the findings of a seismic refraction (SR) survey performed by NORCAL Geophysical Consultants, Inc. along Dry Creek in Sonoma County, CA. The survey was performed on October 20, 2010 by NORCAL Professional Geophysicists William E. Black and Donald J. Kirker, and geophysical technician David Spaulding. Logistical support was provided by Drew Kennedy of Sanders & Associates Geostructural Engineering (SAGE).

1.0 SITE DESCRIPTION AND PURPOSE

The geophysical survey was conducted in the "Off Channel Enhancement Area D" of the Demonstration Reach of Dry Creek. It is located approximately 6 miles downstream of the Warm Springs Dam on the Seghesio Parcel. The site comprises a relatively flat river cut terrace that is heavily vegetated. The parcel is accessed by a gravel/dirt road from an adjacent vineyard north of the creek.

The local geology, as indicated by SAGE, consists of alluvium (interbedded clays, silt, sand, and gravel) over Great Valley Complex sedimentary bedrock (sandstone, siltstone, and shale).

The seismic refraction survey was conducted along one line, as shown on Plate 1. It is designated as Line 1-1 and trends along the north bank of Dry Creek. Surface elevations along Line 1-1 range from 127- to 131-ft above mean sea level (msl).

The purpose of the SR survey was to obtain seismic refraction data to aid in evaluating the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock. We understand that this information will be used in conjunction with other geotechnical investigations to plan for the construction of backwater ponds and channels associated with habitat enhancements along the creek.

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www.norcalgeophysical.com

2.0 METHODOLOGY

The SR method is used to determine the compressional velocity of subsurface materials. The seismic velocity of fill, sediments, and rock are dependent on physical properties such as compaction, density, hardness, and induration. However, other factors such as bedding, fracturing, and saturation also affect seismic velocity. Typically, low velocities are indicative of loose soil, poorly compacted fill material, poorly to semi-consolidated sediments, and deeply weathered and highly fractured rock. Moderate velocities are usually indicative of dense and highly compacted sediments and fill, and/or moderately weathered and moderately fractured rock. High velocities are indicative of slightly weathered to unweathered rock with little fracturing. It should be noted that apparent velocities can be affected by the orientation of bedding planes with respect to the direction of the seismic profile. Apparent velocities of rock are typically slower when measured along lines oriented perpendicular to bedding planes of steeply dipping rock, than those measured along lines oriented parallel. A more detailed description of the SR methodology is provided in Appendix A.

3.0 FIELD INVESTIGATIONS

We obtained seismic refraction data along a single transect, designated as Line 1-1 on Plate 1. The line is 246 feet long and is located along the north bank of Dry Creek. It consists of two overlapping spreads that each comprises 24 geophones and three shot points distributed in a collinear array. The geophones were coupled to the ground surface at 6 foot intervals. Two of the shot points were located 6 feet beyond the end geophones of each spread. The third shot point was positioned in the center of the spread.

The SR data were recorded using a Geometrics ***Geode***, 24-bit digital seismic recording system and Oyo Geospace digital-grade geophones with a natural frequency of 10-Hz. We produced seismic energy at each shot point by striking an aluminum plate, placed on the ground surface, with a 16-pound sledge hammer. An accelerometer attached to the hammer transmitted a triggering pulse to the seismograph each time the plate was struck. The resulting travel time data were recorded on a seismograph and processed to generate seismic velocity cross-sections. A more detailed description of data acquisition and analysis procedures are also provided in Appendix A.

4.0 RESULTS

The results of the seismic refraction survey are illustrated by the seismic velocity profile shown on Plate 2. The vertical axis represents elevation (above mean sea level) and the horizontal axis represents distance. The solid line along the top of the profile depicts the ground surface. The color contours represent seismic velocities according to the color scale shown at the bottom of the section.

The profile shown on Plate 2 indicates seismic velocities that range from about 1,000- to over 6,800-ft/s. Since ground truth from borings is not available for comparison to the detected seismic

velocities shown along Line 1, our interpretation of these velocities is based on our observations of local geological conditions evident from nearby rock outcrops and our experience from past seismic surveys. Therefore, we interpret velocities ranging from 1,000 to about 3,000 ft/s (purple to dark blue) as representing surficial soils and unconsolidated sediments. Velocities ranging from 3,000 to 5,000 ft/s (green) are consistent with semi-consolidated sediments, saturated alluvium, and/or highly weathered/fractured bedrock. Velocities of over 5,000 ft/s represent moderately weathered and/or fractured rock. Plate 2 shows that the bedrock is very shallow at the northwest end (less than 2 ft deep) and increases to a depth of about 9-ft at the southeast end.

5.0 EXCAVATION CHARACTERISTICS (Rippability)

The interpreted bedrock exhibits velocities that range from 5,000 to over 6,800 ft/s. Seismic velocity charts relating seismic velocity and excavation characteristics have been developed from field tests by others. These charts list the seismic velocity of various types of rock and their relative ease of excavation using different types of rippers. Caterpillar Tractor Company publishes a performance manual that lists ripper performance charts for the D8L, D9L, and D11L tractors. The following information in Table A was obtained from a performance chart for a D9L Ripper:

Table A: D9L Ripper Performance Chart

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

According to the D9L Ripper Performance chart above, velocities of 5,000 to 6,800 ft/s are consistent with rock that is rippable to marginally rippable. This information should only be used as a general guide, however, as many other factors should also be considered. These factors include rock jointing and fracture patterns, the experience of the equipment operator, and the equipment and excavation methods selected. Also, the computed velocities measured along each profile are an average for each layer, and that the data analysis routine assumes that the velocity of subsurface

materials increase with depth. Therefore, there may be localized zones within each layer where the velocities may be higher or lower than indicated. This is especially true in areas where bedrock is highly bedded and steeply dipping. Also, if a layer has velocities that are slower than those of the material above it, the slower layer will not be resolved. Since the accuracy of our findings is subject to these limitations, it should be noted that subsurface conditions may vary slightly from those depicted in the final results. A more detailed discussion of the limitations with regard to the seismic refraction method is presented in Appendix A.

6.0 STANDARD OF CARE

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate having the opportunity to provide you with this information.

Respectfully,

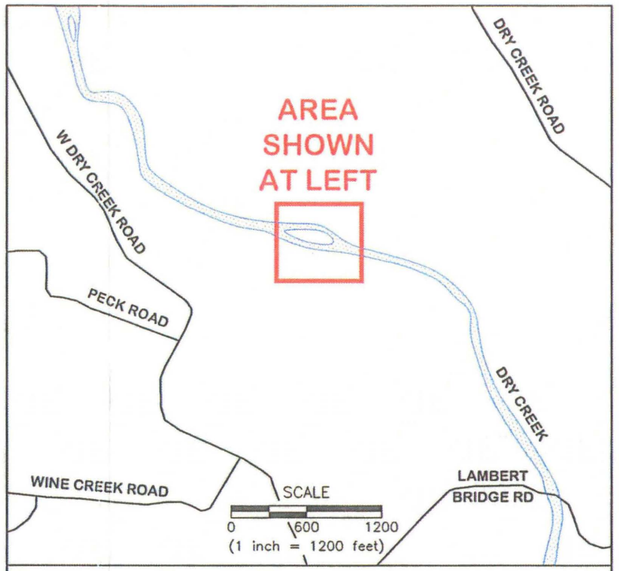
NORCAL Geophysical Consultants, Inc.

Donald J. Kirker

Donald J. Kirker
Professional Geophysicist, PGp-997

DJK/tt

Enclosures: Plates 1 and 2
Appendix A Seismic Refraction Survey



VICINITY MAP

SCALE
0 30 60 120
(1 inch = 60 feet)

LEGEND	
	SEISMIC REFRACTION PROFILE LOCATION

NORCAL

SITE LOCATION MAP
OFF CHANNEL ENHANCEMENT AREA D
DEMONSTRATION REACH – SEGHESIO PARCEL

LOCATION: DRY CREEK, HEALDSBURG, CALIFORNIA

CLIENT: SAGE

JOB #: 10-916.04

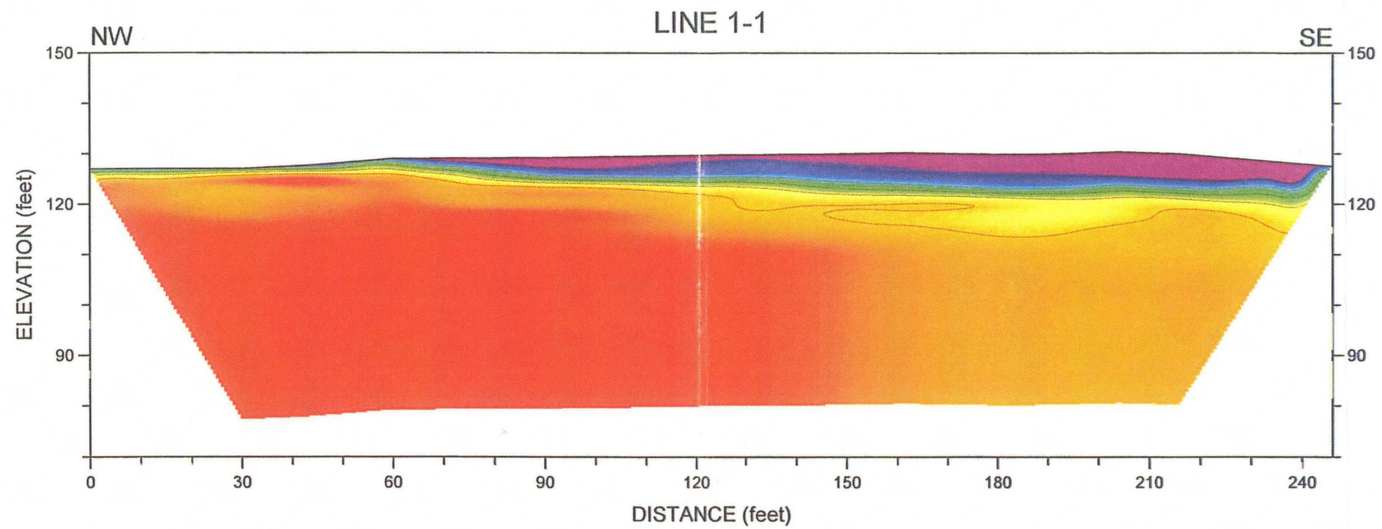
NORCAL GEOPHYSICAL CONSULTANTS INC.

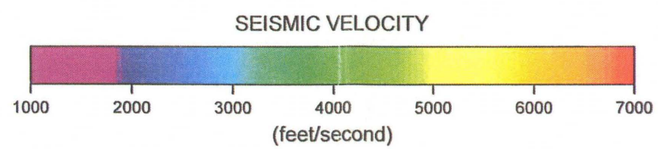
DATE: NOV. 2010

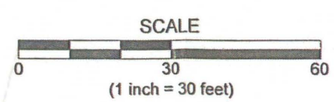
DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 1







NORCAL

SEISMIC REFRACTION PROFILE - LINE 1-1
OFF CHANNEL ENHANCEMENT AREA D
DEMONSTRATION REACH - SEGHESIO PARCEL

LOCATION: DRY CREEK, HEALDSBURG, CALIFORNIA

CLIENT: SAGE

JOB #: 10-916.04

NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: NOV. 2010

DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 2

Appendix A

SEISMIC REFRACTION SURVEY

Appendix A

SEISMIC REFRACTION (SR)

Methodology

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

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

1) that seismic velocity increases with depth, and,
2) that the velocity of each seismic layer is uniform over the length of the given spread.

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

Instrumentation

Data acquisition is initiated along each SR line by producing seismic energy using a mechanical source. Mechanical sources produce energy by impacting a metal strike plate on the ground surface with either a 12-16 pound sledge hammer or an elastic-band driven weight drop. The resulting seismic wave forms are recorded using a Geometrics 24-channel engineering seismograph and Mark Products geophones with a natural frequency of 10 Hz. The data are recorded on hard copy records (seismograms) as well as on computer disks for future processing. The seismograms display the amount of time it takes for a compression (P) wave to travel from a given shot point to each geophone in a spread.

Data Analysis

The seismic data are downloaded to a computer and processed using the program ***Seisimager*** by Geometrics, Inc. This is an interactive program that is used to determine the shot point to geophone travel times, and to compute a 2D model based on those times. Once the travel times for a given line are determined, the programs time-term algorithm is used to compute a preliminary 2D seismic model. This model is then used as input for the programs tomographic routine. Using this procedure, the program divides the starting model into a network of cells and assigns velocities to those cells based on the starting model. The program then traces the refracted seismic travel paths through

those cells and computes the associated travel times. It then compares the computed travel times with the measured times and adjusts the velocities of the appropriate cells to improve the fit. The software is programmed to continue this procedure for twenty iterations. Typically, at the end of the twenty iterations the travel times associated with the computed model match the observed travel times to an accuracy of one milli-second (mS) or better. Once a satisfactory model is computed, the software contours the model velocities to produce seismic velocity vs. depth and distance cross-sections (profiles).

Limitations

In general, there are limitations unique to the SR method. These limitations are primarily based on assumptions that are made by the data analysis routine. First, the data analysis routine assumes that the velocities along the length of each spread are uniform. If there are localized zones within each layer where the velocities are higher or lower than indicated, the analysis routine will interpret these zones as changes in the surface topography of the underlying layer. A zone of higher velocity material would be interpreted as a low in the surface of the underlying layer. Zones of lower velocity material would be interpreted as a high in the underlying layer.

Second, the data analysis routine assumes that the velocity of subsurface materials increase with depth. Therefore, if a layer exhibits velocities that are slower than those of the material above it, the slower layer will not be resolved. Also, a velocity layer may simply be too thin to be detected. Due to these and other limitations inherent to the SR method, the results of the SR survey should be considered only as approximations of the subsurface conditions. The actual conditions may vary locally.