STONEBROOK CREEK SALMONID MIGRATION BARRIER REMOVAL PROJECT

December 2005

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American Rivers
and
California State Coastal Conservancy

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1.0 PROJECT OVERVIEW

1.1 Project Overview
The Stonybrook Creek watershed lies within Alameda County, east of Hayward. The watershed runs north to south and has a drainage area of 6.9 square miles. The creek joins Alameda Creek in Niles Canyon, approximately 13 river miles upstream from San Francisco Bay. The Alameda Creek watershed once supported a run of native steelhead trout. However, the placement of numerous dams, culverts, and other structures on Alameda Creek and its tributaries has resulted in the complete blockage of anadromous fish runs. In 2000 an assessment of the potential for restoring a viable steelhead trout population to the Alameda Creek watershed was completed (Gunther et. al, 2000). The report identified Stonybrook Creek as being the lowest tributary in the watershed containing suitable rearing and spawning habitat for steelhead.

In 2004 the Center for Ecosystem Management and Restoration (CEMAR) obtained funding to complete an alternatives analysis and develop conceptual designs for replacing two of the County maintained crossings on Palomares Road, at mile posts 8.60 and 8.75. These crossings were both identified as complete upstream migration barriers for steelhead and rainbow trout. The replacement of these crossings with structures that provide fish passage is part of the recommendations outlined in the Stonybrook Creek Fish Passage Assessment that was prepared for the Alameda County Public Works Agency (Love, 2001).

With support from CEMAR, Winzler & Kelly Consulting Engineers and Michael Love and Associates has developed conceptual designs for two new road crossings that improve fish passage conditions. In support of this design effort, Blackburn Consulting, Inc. conducted a preliminary foundation investigation and Marvin Smitherman conducted the topographic survey. Additionally, Michael Love and Associates assessed fish passage conditions at three downstream privately maintained stream crossings to determine if they are barriers to fish passage.

This report summarizes the conceptual designs developed for removal and replacement of two road-stream crossings on Palomares Road at post mile 8.60 and 8.75. The assessment of the downstream crossings is described in a separate report.

1.2 Design Overview
The Stonybrook Creek Salmonid Migration Barrier Removal Project included the completion of a conceptual design for the removal and replacement of two existing culvert barriers with two new road crossings that will improve access to upstream habitat for migrating steelhead/rainbow trout. Hydraulically, the new crossings must provide passage for various life stages of fish over a wide range of flows, allow for the natural movement of channel bed load, and provide enough capacity to accommodate a 100-year flood. An additional requirement of the project design is that it allow a minimum of one lane of traffic through the project site during construction. The conceptual design incorporates methods of construction and sequencing that are necessary to meet these project requirements.

The conceptual design includes a pre-cast bottomless arch culvert at post mile 8.60 on Palomares Road (hereafter, “upstream crossing”) and a pre-manufactured steel bridge at post mile 8.75 (hereafter, “downstream crossing”). The design also provides for one lane of traffic to pass
through the project site during construction activities. The channel bed at both locations will be
graded upstream and downstream of the new crossings to achieve the design slope, which closely
approximates the natural slope of Stonybrook Creek within the project reaches. The conceptual
designs were developed based on the assumption that both crossings could be constructed at the
same time. This assumption does not preclude the crossings being constructed at different times,
but this issue needs to be determined prior to the final design phase. We anticipate that
constructing the two crossings at different times will increase the overall cost for construction.

1.2.1 Upstream Crossing, Post mile 8.60
The conceptual design at the upstream crossing includes: the placement of a temporary bridge
during construction, removal of the existing concrete and masonry culvert, re-grading of the
channel in the vicinity of the crossing, the placement of a new pre-cast concrete arch culvert and
wing walls on cast-in-place strip footings, and the paving of the final roadway over the new
culvert. The conceptual plans are included in Appendix A of this report.

A temporary bridge is required to maintain one lane of traffic through the project site during
construction. The temporary bridge is a pre-manufactured steel bridge which would later be used
as part of the permanent downstream crossing (discussed in the following section). The
temporary bridge is approximately 14 feet wide and 80 feet long. The temporary bridge will be
set in place by cranes immediately to the north of the proposed culvert (as shown on sheet C-3.0)
to allow for one lane of traffic during the construction of the channel and new culvert stream
crossing. One side of the bridge will have a permanent guard rail attached, while a temporary
concrete barrier rail will be placed along the other side for traffic safety during construction.
There will be some minor grading and temporary fill required to allow the placement of the
temporary bridge. The temporary bridge is designed to be removed and relocated to the
downstream crossing after a minimum of five of the six pre-cast concrete culvert sections are
installed and backfilled with aggregate base to accept one lane of traffic.

The existing culvert at the upstream crossing will be removed after placement of the temporary
bridge. The concrete rubble from the demolished culvert will be removed from the site for proper
disposal. Suitable boulders from the existing masonry work may be incorporated into the channel
grading.

The channel grading at the upstream crossing will consist of re-grading the channel to a new
grade of approximately 8.9 percent. The design active channel is 24 foot wide with 8:1 side
slopes. From the active channel, the banks will be cut to 2:1 slopes, or steeper as needed, to
match the existing ground slope above the top of the banks. The regraded channel will have a
natural bottom consisting of native channel materials and is intended to resemble the natural
channel. Rather than being a straight grade, the channel section will be comprised of a series of
step-pools which will have a minimum depth of approximately one foot (as shown on Sheets C-
4.0 and C-5.0 of the conceptual plans). Pool spacing is based on the observed spacing in the
channel adjacent to the site and results from research into channel slope verses pool spacing.
These step pools are not intended to be permanent structures. Instead, the design channel is
expected to be further shaped by higher flows and adjust into a stable step-pool configuration
similar to the adjacent channels. Constructing the step-pools into the finished channel grade is
expected to provide suitable fish passage conditions immediately following construction while minimizing the amount of channel adjustment that may occur after construction.

The upstream culvert crossing will consist of a pre-cast concrete arch comprised of six pre-cast concrete units each with a 32 foot width and a 9 foot rise. The pre-cast sections would be hauled by truck to the site and set in place by crane or excavator. The pre-cast units would be set on cast-in-place reinforced concrete footings that run the length of the culvert. The footings would extend approximately two feet below the design channel, which is the presumed location of bedrock based on the limited geotechnical investigation performed as part of this conceptual design effort. The footings for the culvert will be keyed into the existing bedrock. The bedrock surface will have an irregular shaped surface either naturally or after excavation to the designed footing elevations. For added stability, rebar will also be drilled and epoxied into the bedrock prior to the footings being cast in place. The geotechnical report used for the design is included in Appendix B of this report.

One issue that was not specifically addressed in the geotechnical report was the potential for slaking of the exposed bedrock. Slaking is mostly a problem in clay shales, but much less of a problem in sandstone. The borings encountered predominately sandstone. The friable siltstone/shale beds noted on page two of the geotechnical report refer to very thin layers within hard, thick sandstone beds, exposed in existing canyon walls and road cut outcrops. The road cuts have stood for long periods of time without visual deterioration from slaking. There is no evidence that slaking is a significant problem at this site. It is possible that localized shale beds could be present along a downstream footing where exposure to air and water could cause deterioration of the rock. This potential issue should be addressed by Blackburn Consulting Inc. during final design and construction.

Pre-cast concrete wing walls will abut the new pre-cast concrete culvert. The wing walls will have concrete anchors as well as pre-cast concrete footings to expedite placement. As with the cast-in-place footings design, the design utilizing bedrock at each of the wing wall locations was based on the limited geotechnical investigation and may need to be revised during the final design. If bedrock is found in the vicinity of the planned wing walls as a part of a focused geotechnical investigation suggested to be conducted as part of the final design, then the design may be changed to cast-in-place concrete wing walls that could be easily modified to match the bedrock surface. Sheet piling might also be considered as a construction method for the wing walls during final design.

The final roadway will be brought up to grade using structural backfill consistent with Caltrans Standards. The final surface will be asphalt concrete and match the existing roadway’s alignment and grade. New guard rails with appropriate terminal sections will replace the existing metal beam guard rails.

1.2.2 Downstream Crossing, Post Mile 8.75
The conceptual design at the downstream crossing includes: the placement of sheet pile with a cast-in-place concrete pile cap for the bridge footing, removal of the existing reinforced concrete box culvert and wing walls, installation of a pre-manufactured steel bridge, re-grading of the
channel in the vicinity of the crossing, and paving of the final roadway over the new bridge. The conceptual plans are included in Appendix A of this report.

The sheet pile with the cast-in-place pile cap will provide a foundation for the pre-manufactured bridge proposed for this site and is a key design feature that contributes to the constructability of the site. Sheet piling can be driven with minimal disturbance to the surrounding soils. Therefore, the sheet pile footing will be placed while the existing culvert is functioning to provide one lane of traffic during construction. The footing on the west side of the proposed bridge may not require sheet piling due to the expected presence of bedrock close to the ground surface. If this is the case, a cast in place footing on bedrock would be used on the west side similar to the strip footing described for the culvert at the upstream crossing.

The bridge will have an 80 foot span and a 28 foot width and will likely be comprised of four individual sections (each section seven feet wide) that will be pre-manufactured off site. The sections will be hauled to the site, set in place with one or more cranes and bolted together in place. These prefabricated steel bridges are normally installed quite rapidly, minimizing construction time and a greatly reducing road closure time. Two sections (14 feet width) of the new bridge will be set on the footings to provide one lane of traffic through the site to allow the demolition and removal of the existing culvert and excavation of the channel underneath the bridge (as shown on sheet C-6.0 of the conceptual plans). One side of the bridge will have the permanent guard rail attached and a temporary concrete barrier rail will be placed along the other side for traffic safety during construction. As previously discussed, the temporary bridge for the upstream crossing will utilize the other two pre-manufactured bridge sections (14 feet width). The remaining two bridge sections will be placed at this crossing once the upstream crossing progresses to the point where the temporary bridge crossing is no longer needed.

The roadway through the downstream crossing will match the alignment and grade of the existing roadway. The final roadway at the downstream crossing will be brought up to grade using structural backfill consistent with Caltrans Standards. The final surface for the new bridge will be asphalt concrete. The new roadway will have two 12 foot lanes and two foot shoulders for a total paved road width of 28 feet. New guard rails are planned only for the bridge but may be extended beyond the bridge as a part of the final design, if required.

1.3 Opinion of Probable Construction Costs and General Considerations

A planning level opinion of probable construction costs was completed based on the conceptual design. Since the designs are not fully developed the cost estimate relied on numerous assumptions. It is possible that during final design, site conditions may become apparent that could cause actual construction costs to vary. Our current overall opinion of probable construction costs for completing both crossings is $1,100,000.

The use of a pre-manufactured bridge at this site will have the benefit of being able to be used as a temporary bridge during construction of the upstream culvert. This offers significant savings since the cost of renting a temporary bridge is typically close to the cost of purchasing a pre-manufactured bridge. If the two crossings were to be completed independently, the cost of the upstream crossing alone would be substantially higher.
It was assumed that traffic control at the site could be one lane of controlled traffic consistent with a Caltrans low-volume road closure. This type of closure assumes good site distance and would consist of yield signs rather than a full signal system. A full signal system would be used if the road were considered to have a high traffic volume and would have significantly higher costs associated with it.

It was also assumed that the two project sites will be accessible by cranes. The cranes will be required to set the pre-cast culvert and pre-manufactured steel bridge, as well as for construction of the sheet pile foundation for the pre-manufactured steel bridge. If the road to the site is not accessible by cranes it is possible the design would need to be modified to reduce the size of the pre-manufactured culvert or bridge which could result in higher overall project costs. See Table 1.1 on the following page for an itemized list of probable costs.
# Stonybrook Creek Salmonid Barrier Removal Projects

**Engineers Opinion of Probable Construction Costs**

**October 25, 2005**

<table>
<thead>
<tr>
<th>Item No</th>
<th>Item Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Cost</th>
<th>Extended Cost</th>
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<tr>
<td>1</td>
<td>Mobilization and Demobilization</td>
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<td>$42,000</td>
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<td>$120,000</td>
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<td>$25,000</td>
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<td>10</td>
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<td>$60</td>
<td>$16,200</td>
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<td>11</td>
<td>Regrade Channel at Upstream Crossing</td>
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<td>Structure Backfill for Culvert</td>
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<td>Install Pre-Manufactured Steel Bridge</td>
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<td>Regrade Channel at Downstream Crossing</td>
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<td>111</td>
<td>$110</td>
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Subtotal: $841,995

Estimating Contingency@30%: $252,599

**Engineers Opinion of Probable Construction Costs:** $1,100,000

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1 Traffic Control System Assumes Low Volume Road with No Temporary Traffic Signals
2 Temporary Creek Crossings will be accomplished with pre-manufactured bridge for permanent installation at downstream crossing
2.0 DEVELOPMENT OF CONCEPTUAL DESIGNS

The following summarizes the numerous considerations, calculations, observations, and assumptions that lead to the development of the final conceptual designs.

2.1 Design Flows

The design process required estimating the peak design flows for the two crossings. The following watershed statistics were used as part of estimating flows (Table 1):

Table 2.1 - Watershed Statistics

<table>
<thead>
<tr>
<th></th>
<th>Downstream Crossing (MP 8.75)</th>
<th>Upstream Crossing (MP 8.60)</th>
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<tr>
<td>Drainage Area (DA):</td>
<td>5.72 mi²</td>
<td>5.68 mi²</td>
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<tr>
<td>Mean Annual Precipitation (MAP):¹</td>
<td>22.0 in/yr</td>
<td>22.0 in/yr</td>
</tr>
<tr>
<td>Mean Basin Elevation (E):</td>
<td>&lt; 1000 ft</td>
<td>&lt; 1000 ft</td>
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</table>

¹Estimated from isohyetal map of mean annual precipitation in California, developed by Daly and Taylor (1998).

The peak design flow was set at the peak flow having a 100-year recurrence interval, as specified by NMFS (2001) and California Department of Fish and Game (CDFG, 2002) stream crossing design guidelines.

Magnitudes of peak flows associated with varying recurrence intervals were estimated using three different methods: (1) rational method, (2) regional flood estimation regression equations by the USGS for the Central Coast region (Waananen and Crippen, 1977), and (3) probabilistic analysis of local streamflow records using standard procedures outlined in Bulletin 17B (USGS, 1982). The following table summarized the results (Table 2).

Table 2.2 - Estimated peak flows and recurrence intervals for the downstream crossing (MP 8.75). Peak flows at the upstream crossing are slightly lower.

<table>
<thead>
<tr>
<th>Estimated Peak Flow</th>
<th>2 year (cfs)</th>
<th>10 year (cfs)</th>
<th>25 year (cfs)</th>
<th>50 year (cfs)</th>
<th>100 year (cfs)</th>
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<tr>
<td>(1) Rational Method¹</td>
<td>403</td>
<td>681</td>
<td>1,018</td>
<td>1,370</td>
<td>1,771</td>
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<tr>
<td>(2) Waananen and Crippen, 1977</td>
<td>77</td>
<td>404</td>
<td>671</td>
<td>929</td>
<td>1,219</td>
</tr>
<tr>
<td>(4) Average of Local Streamflow Records</td>
<td>167</td>
<td>710</td>
<td>1,096</td>
<td>1,427</td>
<td>1,784</td>
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<tr>
<td>adjusted by drainage area²</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

¹ Used Intensity-Duration-Frequency curves from four rain gaging stations located between 3.8 miles and 11 miles from the project location. Estimated time of concentration was 55 min.

² Used records from seven nearby stream gages and adjusted peak flow estimates by drainage area. Stream gage drainage areas ranged between 0.28 mi² and 37.5 mi².

Estimates for the 100 year peak flow were very similar using the rational method and the average of stream flow records. For the design flow we conservatively selected a peak flow of 1,790 cfs for both stream crossings.
2.2 Channel Design

2.2.1 Existing Channel Grade
Surveying a longitudinal profile through the project site is essential for developing successful channel designs. Since the two crossings are less than 800 feet apart, we chose to survey one long continuous profile. It extended over 1,700 feet, beginning 455 feet below the downstream crossing to 400 feet above the upstream crossing. The average slope of the surveyed channel was 8.7 percent. However, the slope varied substantially above and below each of the crossings, ranging between 6.2 percent below the downstream crossing to 9.0 percent between the two crossings (Figure 1).

![Stonybrook Creek Existing Profile](image)

Figure 2.1 - Figure 1 – Longitudinal profile of Stonybrook Creek through the project area. Slopes were estimated using linear regression of surveyed thalweg points, excluding the culverts.

Upstream of the downstream crossing the channel appears severely aggraded with large boulders. The boulders have deposited at the culvert inlet due to the channel constriction caused by the undersized culvert. This creates a 5.5 foot drop in the channel bed immediately upstream from the culvert inlet which completely blocks fish passage and reduces the culvert’s capacity (Figure 2). The deposition above the culvert also appears to have starved the downstream channel of the larger bedload sizes.

2.2.2 Reference Reach
The project area is located within Stonybrook Canyon, a highly confined section of channel. The channel reach between the two stream crossings is approximately 800 feet in length. Throughout this reach the road bed encroaches on the stream’s right bank (looking downstream) and bedrock controls the form of the left bank. The bed shape and slope is predominately controlled by numerous large boulders.
Figure 2.2 - Inlet of the downstream crossing (MP 8.75), as flow cascades down a 5.5 foot drop over boulders deposited due to the channel constriction caused by the culvert.

Figure 2.3 - Location of the reference reach, 200 feet upstream of the downstream crossing (MP 8.75)
As part of the design process a reference channel reach was located roughly 200 feet upstream of the downstream crossing which appeared representative of the entire channel between the two crossings (Figure 3). Within the reference reach a Wolman pebble count was conducted to characterize the streambed material, a channel cross section was surveyed, and the water surface slope through the cross section was measured.

To characterize the streambed material and aid in estimating appropriate hydraulic roughness, a particle size distribution was created from the pebble count. It found the d84 (84 percent of particles less than the d84) to be approximately 2 feet and the d50 to be nearly 7 inches (Figure 4). These values correspond with large bedload characteristic of a steep stream such as Stonybrook Creek. The material size and distribution is considered suitable for reconstructing step-pools into the regraded channel without needing to import large rock.

Width at the 2-year flow and conveyance area at the 100-year flow were used in designing the shape of the regraded channel and selecting the appropriate culvert size for the upstream crossing. To estimate these values we conducted a hydraulic analysis of the reference cross section using the computer software, WinXSPro (USFS 2005). The analysis estimated the wetted channel width at the 2-year peak flow to be approximately 26 feet, with a mean water depth of 1.7 feet. The wetted area of the channel at the 100-year peak flow was 180 square feet and water velocities within the channel at the 2-year and 100-year peak flows were estimated to be 3.9 feet per second and 9.6 feet per second, respectively. Additional detail is provided in the attached representative cross section summary.

**Figure 2.4 - Particle size distribution of streambed material located within the reference reach approximately 200 feet upstream of the downstream crossing.**
2.3 Stream Channel Design

2.3.1 Design Channel for the Upstream Crossing (MP 8.60)
Determining stable design slopes at the two crossings required examining the channel grade upstream and downstream, and identifying hard features in the channel that control the stream’s grade. For the upstream crossing the stable channel slope is estimated to be 8.9 percent. The channel will be regraded from Station 22+35 to Station 23+72, a total of 137 feet to match the existing adjacent stream grades approximately 100 feet downstream and 200 feet upstream of the crossing (Figure 5). The survey also noted bedrock spanning the channel bottom at a location about two hundred feet downstream of the upstream crossing. Bedrock typically functions as a hard point in the channel, and is expected to prevent the streambed from incising below the upstream crossing following the replacement of the downstream crossing.

![Upper Crossing - Channel Profile](image)

Figure 2.5 - Design channel profile for the upstream crossing at Mile Post 8.60.

2.3.2 Design Stream Grade for Downstream Crossing (MP 8.75)
Determining a stable channel grade at the downstream crossing was more challenging than at the upstream crossing. The deposited boulders at the culvert inlet appear to have influenced the channel grade for more than 100 feet upstream. Additionally, the channel immediately downstream of the culvert has a much lower gradient than the rest of the channel. The discrepancy in slopes is likely due in part to encroachment of the road fill into the channel between the two crossings, whichstraightened the stream channel and caused it to become steeper.

The design of the downstream crossing incorporates two channel grades (Figure 6). The regrading begins just below the culvert outlet at Station 13+50 and continues upstream 140 feet.
at an average slope of 8.9 percent. At Station 14+90 there is a slope break and the design channel grade steepens to 11.8 percent. The regraded channel ends at Station 15+85, approximately 15 feet below a 4 foot high falls over an extremely large stable boulder that spans the channel. A slope break was needed to balance the amount of native streambed material available for grading.

We anticipate the stream shape and grade at the downstream crossing will adjust over time. Since both banks under the proposed bridge are bedrock, the channel adjustments will be confined and will not disturb the bridge abutments. Additionally, the bridge will be more than 20 feet above the channel bed, providing ample room for the bed to aggrade while maintaining adequate conveyance for the 100 year flow.

Figure 2.6 - Design channel profile for the downstream crossing at Mile Post 8.75.

2.3.3 Shaping of Streambed for Both Crossings
Since streams with slopes greater than 3 to 4 percent naturally either have a step-pool or cascade channel morphology, we propose to construct a rough step-pool shape into the finished channel bed. This will provide suitable fish passage conditions immediately after construction and assist the channel in setting up a stable step-pool configuration. However, we anticipate that substantial adjustment to the bed shape will still occur following construction.

The step-pool design is intended to be relatively easy to construct, provide suitable resting areas for fish, and be shaped in roughly the same configuration that is found in steeply sloping natural streams (i.e., 9 to 12 percent). Specified pool spacing for this project was determined through field observations of the adjacent channel and use of pool spacing relationships developed for other steep streams (Chin, 1999; Zimmermann and Church, 2001) (Figure 7 and Table 3).

Additionally, each step pool must have a residual pool depth of one foot to provide adequate...
resting area for fish. The steps will be constructed using native streambed material from the project site, which is comprised of large boulders as well as sands, gravels, and cobbles.

![Typical Design Profile with Step-Pools](image)

**Figure 2.7 - Typical of step-pool configuration for regraded channel sections.**

<table>
<thead>
<tr>
<th>Channel Reach</th>
<th>Average Design Grade</th>
<th>Reach Length</th>
<th>Average Pool Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream Crossing (MP 8.60)</td>
<td>8.9%</td>
<td>137 feet</td>
<td>22 feet</td>
</tr>
<tr>
<td>Downstream Crossing (MP 8.75):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower Section</td>
<td>8.9%</td>
<td>140 feet</td>
<td>22 feet</td>
</tr>
<tr>
<td>Upper Section</td>
<td>11.8%</td>
<td>95 feet</td>
<td>17 feet</td>
</tr>
</tbody>
</table>

### 2.3.4 Channel Cross Sectional Shape

For both crossings the bottom channel width was set at 24 feet, based on results from the hydraulic analysis of the reference cross section at the 2-year peak flow. The bottom has a side slope of 8(Horizontal):1(Vertical) and the banks have side slopes of 2(Horizontal):1(Vertical). Given the large size of material that will be graded, the channel will likely end up with many irregularities with numerous large rocks protruding from the bed and banks. This will be similar in nature to the natural channel and provide many quiet-water areas and dissipate energy at higher flows.

### 2.4 Fish Passage Improvements

When implemented, this project will result in the removal of two complete fish barriers that were identified in the Stonybrook Fish Passage Assessment (Love, 2001). These existing barriers will be replaced with new crossings that provide for a roadway over a natural stream channel. At the crossings, the proposed designs utilize a step-pool channel at slopes found in the adjacent natural channel. The channel will be shaped using only native streambed material found on site. The design is intended to allow the channel to adjust naturally in both shape and profile. The end result will be a channel that provides fish passage and fish habitat similar to the adjacent natural channels. Due to the type of work involved, it would be preferable to select a contractor with
experience completing successful in-stream channel work. When completed, the project will open up access to over 3,000 feet of previously blocked habitat for steelhead and rainbow trout.

2.5 References


CONCEPTUAL DESIGNS FOR TWO
STONYBROOK CREEK SALMONID MIGRATION
BARRIER REMOVAL PROJECTS
(PALOMARES ROAD AT POSTMILES 8.60 AND 8.75)

OCTOBER 2005

SHEET INDEX

G-1.0 COVER SHEET
C-1.0 GENERAL NOTES, SYMBOLS, ABBREVIATIONS, & INDEX SHEET
C-2.0 OVERALL CREEK PROFILE
C-3.0 POST MILE 8.60 PLAN VIEW
C-4.0 POST MILE 8.60 PROFILE
C-5.0 POST MILE 8.60 SECTIONS
C-6.0 POST MILE 8.75 PLAN VIEW
C-7.0 POST MILE 8.75 PROFILE
C-8.0 POST MILE 8.75 SECTIONS

VICINITY MAP

LOCATION MAP
CONSTRUCTION NOTES FOR DRAIN CHANNEL BENDS

1) EDDIES caused by moving water material to conform channel on plans
2) ELEVATION OF DRAIN BEND MUST MATCH DESIGN GRADE
3) USE LARGEBOARDS TO FORM DRAIN CURVES

TYPICAL SECTION AT GULFERT

SCALE: 1" = 24" REAL

TYPICAL BED SHAPE

SCALE: 1" = 24" REAL

TYPICAL CHANNEL SECTION

SCALE: 1" = 24" REAL
Appendix B
Preliminary Foundation Investigation
PRELIMINARY FOUNDATION INVESTIGATION
Stonybrook Creek Migration Barrier Removal Projects
Palomares Road M.P. 8.75 and M.P. 8.60
Alameda County, California
Mr. Steve Allen  
Winzler & Kelly, Consulting Engineers  
633 3rd Street  
Eureka, CA 95501-0417

Subject: Preliminary Foundation Investigation  
Stonybrook Creek Fish Passage Projects  
Palomares Road M.P. 8.75 and M.P. 8.60  
Alameda County, California

Dear Mr. Allen:

Blackburn Consulting, Inc. (BCI) prepared this Preliminary Foundation Investigation report for the subject Migration Barrier Removal Projects along Stonybrook Creek in accordance with our agreement dated May 28, 2004.

Please call if you have questions on this report or require additional information. We appreciate this opportunity to be of service.

Sincerely,

BLACKBURN CONSULTING, INC.

Rick Sowers, P.E., C.E.G.  
Senior Project Manager

Distribution: Client (4)

Reviewed by:

Patrick Fischer, C.E.G.  
Principal
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  Laboratory Test Results  
  Weathering Descriptors
INTRODUCTION

BCI prepared this preliminary foundation investigation for two migration barrier removal projects along Stonybrook Creek. This report provides preliminary geotechnical recommendations for design and construction of new structure foundations at each channel crossing. Our services are based on design for conceptual replacement structures as developed by Winzler & Kelly. Elevations used in this report are based on site topography developed by Marvin Smitherman.

We prepared this report for Winzler & Kelly, project engineer. Do not use or rely upon this report for different locations or improvements without the written consent of BCI. Additional investigation may be required for final design, depending on final structure type, layout and channel profile.

SCOPE OF SERVICES

To prepare this report, BCI:

- Conducted a site review, marked boring locations, notified USA and obtained an Alameda County encroachment permit.
- Reviewed published geologic and topographic mapping of the sites.
- Performed surface geologic reconnaissance of the site vicinities.
- Drilled and sampled two test borings at each project location.
- Performed laboratory testing and geotechnical engineering analysis in support of the recommendations contained herein.

PROJECT LOCATION AND DESCRIPTION

Project Locations

The sites are located on Palomares Road about one mile north of SR 84 (Niles Canyon Road) in Alameda County, California. Palomares Road generally parallels Stonybrook Creek along the base of Stonybrook Canyon and crosses the creek at several locations. The crossing at PM 8.75 is approximately 800 ft south of the PM 8.60 crossing. We show the project locations and site topography on Figure 1.

Site and Project Descriptions

M.P. 8.75

This crossing contains an existing 8 ft wide x 9 ft high, 89 ft long, concrete box culvert. The culvert invert is about 25-30 ft below road grade, with about 15-20 ft of earth embankment above the culvert and exterior fill slopes at a gradient of about 2H:1V. The creek alignment bends sharply (>90°) near both the inlet and outlet, and a large aggredation of boulders has accumulated near the inlet (resulting in a 10+ ft drop in channel profile at that location).
Upstream and downstream of the crossing, the Palomares Road alignment is generally parallel to the creek along a steep (6-8%) southwest gradient.

We understand this culvert will be replaced with either a pre-fabricated concrete arch or clear-span bridge. We expect a minimum 20 ft channel span to provide adequate width for bedload transport. The channel profile may be re-graded to eliminate the steep inlet drop and provide a uniform gradient for fish passage.

**M.P. 8.60**

The existing crossing is a 20-25 ft long concrete slab bridge over a trapezoidal-shaped channel with stone masonry side-slopes and a rough concrete floor. Base of channel is about 9 ft wide; top of channel is about 15 ft wide; and length of the trapezoidal channel about 77 ft. Channel erosion at the outlet has created a 5 ft drop in profile grade.

As at the M.P. 8.75 crossing, we understand this crossing will be replaced with either a pre-fabricated concrete arch or clear-span bridge. We expect a minimum 20 ft channel span to provide adequate width for bedload transport. The channel profile may be re-graded to eliminate the perched outlet and provide a uniform gradient for fish passage.

**GEOLOGIC SETTING**

Our site work and published geologic literature indicate both sites to be underlain by Cretaceous sedimentary rocks of the Panoche Formation. The Geologic Map of the San Francisco-San Jose Quadrangle (Wagner, et al., 1990) describes these rocks as marine sandstone, shale, siltstone and conglomerate. Mapping by Graymer, et al., (USGS Open File Map 94-132, 1994), shows the sites to be underlain by distinctly bedded and well lithified Cretaceous sandstone and shale. Rock bedding is shown to strike uniformly northwesterly and dip southwest at 70°. We attach a site geologic map as Figure 2.

We observed rock consistent with the above description at both sites, as well as throughout the canyon in this vicinity. The canyon slopes are very steep (gradients up to 1:1 and steeper) and expose mostly bedded sandstone. The sandstone is moderately to slightly weathered and hard, with generally thick beds (greater than one foot). The sandstone is interlayered with thin (less than one inch) laminated and intensely fractured ("frangible") siltstone/shale beds. Rock bedding at both sites strikes uniformly northwest and dips generally 60-80° to the southwest.

**SEISMIC SETTING**

Our review of published geologic mapping and preliminary site review did not reveal the presence of Late Quaternary (displacement within the last 700,000 years) or younger faults within or adjacent to the project site, and the sites are not within or adjacent to an Alquist-Priolo Earthquake Fault Zone for fault rupture hazard.
Based on published geologic maps and the State of California, Department of Transportation, California Seismic Hazard Map (1996), the closest recognized Late Quaternary or younger faults are the following:

1. Calaveras-Paicines-San Benito Fault, located approximately 2.8 miles (4.5 km) northeast of the site.

2. Verona-Williams Fault, located approximately 3.0 miles (4.8 km) northeast of the site.

3. Hayward Fault, located approximately 3.4 miles (5.5 km) southeast of the site.

The controlling fault for design is the Calaveras-Paicines-San Benito Fault, a strike-slip style fault with an estimated maximum magnitude of 7.5. The horizontal Peak Bedrock Acceleration (PBA) at the site is 0.55g based on the California Seismic Hazard Map. Caltrans’ Division of Geotechnical Services indicates the PBA must be rounded up to the nearest tenth (0.6g) for spectra construction. Based on our boring data, we classify the site soil profile as Type C (based on foundations established in weathered rock).

For seismic design, use the 0.6g peak horizontal rock acceleration curve from Figure B.5 (Soil Profile Type C for a Magnitude of 7.5±0.25) of the Caltrans Seismic Design Criteria (2004, Version 1.3).

SUBSURFACE CONDITIONS

To characterize the subsurface soil and rock conditions, BCI observed and logged two test borings at each site, drilled to a maximum depth of 29 ft. The borings were advanced using hollow stem auger and, in rock, by means of wireline coring. We obtained drive-samples from soil and highly weathered rock with 2.0 inch OD “standard penetration” and 3.0 inch OD “California Modified” samplers driven with standard 350 ft-lb striking force (per ASTM D1586). Samples of less weathered underlying rock were recovered by NQ core and retained in core boxes for reference.

The boring locations are shown on Figures 3 and 4. Detailed logs are included in the Appendix. The degree of weathering and hardness noted on the logs are keyed to the descriptors included in the Appendix.

M.P. 8.75 Site

The test borings at the M.P. 8.75 site encountered loose to medium dense, silty sand and sandy silt, with scattered sandstone fragments, to depth 27 ft in B-1 (southeast abutment, to elev. 405 ft) and depth 20 ft in B-2 (northwest abutment, to elev. 412 ft). We interpret these materials as culvert backfill and young colluvium/alluvium. These soils are unconsolidated, variable in strength, and subject to erosion where exposed.

We encountered bedrock below the surface soils, described as slightly to moderately weathered, moderately hard, thickly to thinly bedded sandstone, with interbedded, moderately weathered, soft, laminated siltstone. Bedding planes observed in the cores typically dip about 60-70° from
horizontal. Core recovery ranged from 30-100%, with RQD\(^1\) in the range of 20-40% (reflecting the variably weathered and bedded nature of the rock).

**M.P. 8.60 Site**

The test borings at the M.P. 8.60 site encountered medium dense, sandy silt and stiff, sandy clay with scattered sandstone fragments to depth 14 ft in B-3 (southeast abutment, to elev. 486 ft) and depth 4 ft in B-4 (northwest abutment, to elev. 490 ft). We interpret these materials as fill and young colluvium/alluvium, with variable strength and subject to erosion where exposed.

We encountered sandstone bedrock underlying the surficial soils. The rock is described as intensely to slightly weathered, moderately hard, sandstone. The rock at this site was drillable with power auger equipment and no rock cores were recovered.

**GROUNDWATER**

The augered portions of the test borings were dry during drilling (June, 2004). We expect that shallow groundwater at both sites may be seasonally “perched” over the rock unit at elevations slightly above channel bottom. Groundwater within the underlying rock is likely restricted to zones of intense weathering and within layers of laminated bedding.

**LABORATORY TESTS**

We performed the following laboratory tests on representative soil samples from the test borings:

- Soil Gradation (per ASTM D 422)
- Moisture/Density (per ASTM D-2216, D-2937)
- Soil Corrosivity (pH, Resistivity, Sulfates/Chlorides)

The gradation tests indicate the surface soil to be silty sand with gravel with 29% passing 200-mesh sieve (“SM” per Unified Soil Classification). In-place moisture contents range from 6-16%, and dry densities from 103-122 pcf.

Results of our corrosivity testing on a representative sample from B-3 (P.M. 8.60 Site) are as follows:

- pH: 7.4
- Min. Resistivity: 2250 ohm-cm
- Chloride: 13.3 ppm
- Sulfate: 61.9 ppm

---

\(^1\) RQD = Rock Quality Designation, defined as the sum of length of solid core pieces greater than 4 inches long divided by the total length of core run.
CONCLUSIONS AND RECOMMENDATIONS

We recommend new structure support at both sites to be achieved within intact rock underlying the surficial soil unit. Based on the boring data, we expect the top of rock in the near vicinity of both existing culverts to be at or slightly above channel bottom, and to slope up in directions away from the channel. In general, the upper portion of the rock is intensely weathered and relatively soft, especially at levels above channel bottom, and becomes less weathered and harder with depth.

Construction backslopes will need to be reviewed, based on the selected structure type, length, depth to rock and construction sequencing. For preliminary design, use 1.5:1 (H:V) gradient within fill material and 1:1 gradient within rock.

Structure Foundations – Bridge

For new bridge foundations established within existing fill, we recommend steel H-section piling penetrating the underlying sandstone. For new foundations established within rock (i.e., longer bridges with abutment lines located away from the channel), spread footings or cast-in-drilled-hole (CIDH) piling may be suitable, depending on the depth of fill and shape of the rock surface at those locations. Further exploration may be required for these alternatives.

Spread footing foundations established within fill or alluvial soils above the rock are not recommended due to variable bearing conditions, settlement potential, and security with respect to long-term bank erosion (and possible lower stream profile). Driven concrete piles are also not recommended due to the potential for shallow rock and limited penetration. The suitability of CIDH piling will depend on the depth of fill at the abutment locations, as these excavations may experience significant caving if drilled through the existing fill and/or alluvial soils.

For steel H-piles, we recommend a minimum of HP 10"x57 lb sections to enable pile advancement through the coarse alluvium and achieving a minimum 5 ft penetration into the underlying rock unit. For preliminary design, assign allowable design (service) loading to 45 tons per pile for these piles; pile loading to 70 tons per pile can be achieved with greater penetration into the rock unit.

Design tip elevations for the piles will depend on structure type and length. We can provide tip elevations based on further bridge details, when available. For preliminary design of short span bridges near the existing culverts, assume pile tips to extend 5 ft below channel bottom at each site (to approximately elevation 400 at M.P. 8.75, and elevation 480 at M.P. 8.60).

Structure Foundations – Arch Culverts

Footage support is available at both sites for open-bottom arch culverts with bases established near existing channel bottoms. For preliminary design, we consider 20 ft wide open-bottom culverts (e.g., pre-cast segmental units placed on cast-in-place concrete strip footings), centered at or near the existing culverts. Support for these structures is available on reinforced concrete strip footings of minimum width 3 ft, established about 2 ft below existing invert elevation and within intact rock, as identified by a BCI representative upon construction exposure.
Use an allowable bearing capacity of 5 tsf, net at groundline, for footings with minimum 3 ft width. Higher bearing pressures are available with increased rock penetration and/or positive field review by this office.

For preliminary design, and assuming footing lines near existing culvert inverts, use bottom of footing grades as shown in Tables 1 and 2. In general, we expect the rock surface to be higher as the footing lines move away from the channel, and the elevations provided in Tables 1 and 2 must be reviewed, and modified as necessary, once the structure types and locations are established.

<table>
<thead>
<tr>
<th>TABLE 1</th>
<th>TABLE 2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>P.M. 8.75</strong></td>
<td><strong>P.M. 8.60</strong></td>
</tr>
<tr>
<td>Plan Footing Grade</td>
<td>Plan Footing Grade</td>
</tr>
<tr>
<td>(Elev.- Ft.)</td>
<td>(Elev.- Ft.)</td>
</tr>
<tr>
<td>Inlet</td>
<td>Inlet</td>
</tr>
<tr>
<td>403</td>
<td>485</td>
</tr>
<tr>
<td>Outlet</td>
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</tr>
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<td>396</td>
<td>483</td>
</tr>
</tbody>
</table>

We expect rock excavation to be locally difficult but achievable by use of air tools without blasting. Rock blasting would likely disrupt/ degrade integrity of the surrounding rock and should be avoided. If the rock surface is variable along the footing lines, we consider the use of plain concrete appropriate to provide positive contact between the footing elements and suitable rock, as needed.

Use a coefficient of friction to resist sliding of 0.40 between the footing and weathered rock. Passive pressures in the rock unit can be based on an equivalent fluid weight of 500 pcf. Use passive pressures of 300 pcf (EFP) for soil above the rock. For additional security against sliding (if necessary), or if hard rock precludes reasonable rock excavation, doweling can be utilized to establish a positive connection between the footing and rock (e.g., #8 bars grouted in drilled holes extending 5±ft into rock).

Place footing concrete neat, without forming, against trimmed, intact bearing material in clean and dry excavations. Assuming dry season (low flow) construction, we expect de-watering to be achievable by means of diking/diversion of surface water and sump pumping.

The alluvial soils overlying the rock are subject to scour and bank erosion. If rock slope protection is placed at the inlet/outlet, or along the banks, construct in accordance with Caltrans Standard Plans. Depending on structure location, construction backslopes within the overlying fill may require slopes at gradients flatter than 1:1 for stability.
Culvert Corrosion

Based on our pH, sulfate and chloride testing, and Table 854.1A of the Caltrans Highway Design Manual, there are no restrictions on cementitious materials with respect to soil corrosivity. Table 3 presents metal corrugated pipe culvert material and minimum unprotected thicknesses for a 50-year maintenance free service life with respect to soil corrosivity. The recommendations are based on our pH and resistivity testing, and Table 854.3B of the CHDM.

<table>
<thead>
<tr>
<th>Recommended Metal Corrugated Pipe Culvert Material</th>
<th>Minimum 50-year Design Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Galvanized Steel-Metal</td>
<td>2.0 mm</td>
</tr>
<tr>
<td>Aluminum</td>
<td>1.5 mm</td>
</tr>
<tr>
<td>Aluminized Steel</td>
<td>1.6 mm</td>
</tr>
</tbody>
</table>

The above minimum thicknesses do not take pipe abrasion resistance and overfill height into consideration.

RISK MANAGEMENT

Our experience and that of our profession clearly indicates that the risks of costly design, construction, and maintenance problems can be significantly lowered by retaining the geotechnical engineer of record to provide additional services during design and construction. For this project, BCI should be retained to:

- Update the preliminary foundation recommendations upon selection of a preferred design.
- Review and provide comments on the civil plans, foundation plans, and specifications prior to construction, especially with respect to foundation support away from the channels. Additional data may be required for some structure locations.
- Monitor construction to check and document our report assumptions. At a minimum, BCI should monitor site grading, footing excavations, pile driving and RSP key (if used).
- Update this report if design changes occur, 2 years or more lapses between this report and construction, and/or site conditions have changed.

If we are not retained to perform the above applicable services, we are not responsible for any other party’s interpretation of our report, and subsequent addendums, letters, and discussions.
LIMITATIONS

BCI performed services in accordance with generally accepted geotechnical engineering principles and practices currently used in this area. We do not warrant our services.

BCI based this report on the current site conditions. We assumed the soil, rock and groundwater conditions observed in our borings are representative of the subsurface conditions on the site. Actual conditions between borings could be different. For culvert design, footing levels will vary depending on final structure locations, and the recommendations contained in this report are preliminary and conditioned on BCI's review of structure plans and the opportunity to modify the recommendations, as necessary. For bridge design, BCI will provide supplemental pile recommendations based on further design details.

Our scope was limited to support for new structure foundations and did not include evaluation of on-site hazardous materials, scour potential, abutment design or flooding. This study also did not include evaluation of approach roadway sections or future channel modifications.

Modern design and construction are complex, with many regulatory sources/restrictions, involved parties, construction alternatives, etc. It is common to experience changes and delays. The owner should set aside a reasonable contingency fund based on complexities and cost estimates to cover changes and delays.

Logs of our test borings are presented in the Appendix. The lines designating the interface between soil and rock types are approximate. The transition between soil/rock types may be abrupt or gradual. Our recommendations are based on the final logs, which represent our interpretation of the field logs and general knowledge of the site and geological conditions.
FIGURES

Figure 1 – Site Location and Topography
Figure 2 – Geologic Map
Figure 3 – Site Plan, PM 8.75
Figure 4 – Site Plan, PM 8.60
SITE PLAN

Palomares Road at PM 8.75

Legend

B-1
Approximate Boring Locations

77
Strike and dip of bedding

SS
Sandstone

Electronic media for plan view provided by Marvin Smitherman, plan dated 11-03-04.

Interbedded hard ss (beds 1-3ft. thick) and friable ss/shale (beds <\(\frac{1}{2}\)" thick)

Large boulders in channel

Estimated limits of fill

Scale: 1" = 20'
APPENDIX

Boring Logs

Laboratory Test Results

Weathering Descriptors
Silty sand (SM), medium dense, light brown, with subangular rock fragments, non-plastic, dry. (Fill and Slope Colluvium)

Sandstone, light gray, moderately to slightly weathered, moderately hard, thinly bedded with interbeds of soft laminated siltstone.
Run A: 100% rec; 33% RQD
Run B: 30% rec; 20% RQD

No free groundwater encountered.
Boring backfilled with native cuttings 6-1-04.
<table>
<thead>
<tr>
<th>FIELD</th>
<th>DESCRIPTION</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Silty sand (SM), loose to medium dense, light brown, with subangular rock fragments, non-plastic, dry. (Fill and Slope Colluvium)</td>
</tr>
<tr>
<td>5</td>
<td>1 24</td>
</tr>
<tr>
<td>10</td>
<td>2 6</td>
</tr>
<tr>
<td>15</td>
<td>3 30</td>
</tr>
<tr>
<td>20</td>
<td>4 50/0.2</td>
</tr>
<tr>
<td>25</td>
<td>5 50/0.1</td>
</tr>
<tr>
<td>Run A</td>
<td></td>
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</tbody>
</table>

No free groundwater encountered. Boring backfilled with native cuttings 6-1-04.
### Field

<table>
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<tr>
<th>Depth (Feet)</th>
<th>Sample</th>
<th>Sample No.</th>
<th>N-Value</th>
<th>Pocket Pen (TSF)</th>
<th>Graphic Log</th>
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<td>5</td>
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<td>1</td>
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<tr>
<td>10</td>
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<td>25</td>
<td></td>
<td>5</td>
<td>50/0.4</td>
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</tbody>
</table>

**Description**

- Silty sand (SM), medium dense, light brown, with subangular rock fragments, low plasticity, dry.
- Sandy clay (CL), stiff, mottled dark brown and black, with angular sandstone fragments, dry, medium plasticity.
- Sandstone/siltstone, dark brown-gray, decomposed, soft, moist.
- Sandstone, light gray, intensely weathered, moderately hard, becoming moderately to slightly weathered with depth, dry. Essential auger refusal at depth 29 ft.

---

**Laboratory**

<table>
<thead>
<tr>
<th>Dry Density (pcf)</th>
<th>Moisture Content (%)</th>
<th>% &lt;200 Sieve</th>
<th>Plasticity Index</th>
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<th>Direct Shear φ</th>
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<tbody>
<tr>
<td>102.7</td>
<td>16.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Grad; Corros.</td>
</tr>
</tbody>
</table>

**Additional Tests**

- No free groundwater encountered.
- Boring backfilled with native cuttings 6-2-04.
<table>
<thead>
<tr>
<th>FIELD</th>
<th>LABORATORY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>DEPTH (FEET)</td>
<td>SAMPLE NO.</td>
</tr>
<tr>
<td>1</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
</tr>
</tbody>
</table>

No free groundwater encountered.
Boring backfilled with native cuttings 6-2-04.
# Unified Soil Classification (ASTM D 2487-98)

<table>
<thead>
<tr>
<th>Material Types</th>
<th>Criteria for Assigning Soil Group Names</th>
<th>Group Symbol</th>
<th>Soil Group Names</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Coarse-grained Soils</strong></td>
<td><strong>Gravels</strong>&lt;br&gt; &gt;50% of coarse fraction retained on No. 4 sieve</td>
<td>Clean gravels &lt;5% fines&lt;br&gt; Cu &gt; 4 AND 1 &lt; Cc &lt; 3&lt;br&gt; Cu &lt; 4 AND/OR 1 &gt; Cc &gt; 3</td>
<td>GW</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gravels with fines &gt;12% fines&lt;br&gt; Fines classify as ML or MH&lt;br&gt; Fines classify as CL or CH</td>
<td>GP</td>
</tr>
<tr>
<td></td>
<td><strong>Sands</strong>&lt;br&gt; &lt;50% of coarse fraction retained on No. 4 sieve</td>
<td>Clean sands &lt;5% fines&lt;br&gt; Cu &gt; 6 AND 1 &lt; Cc &lt; 3&lt;br&gt; Cu &lt; 6 AND/OR 1 &gt; Cc &gt; 3</td>
<td>SW</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sands with fines &gt;12% fines&lt;br&gt; Fines classify as ML or MH&lt;br&gt; Fines classify as CL or CH</td>
<td>SM</td>
</tr>
<tr>
<td><strong>Fine-grained Soils</strong></td>
<td><strong>Sands and clays</strong>&lt;br&gt; Liquid limit &lt;50</td>
<td>Inorganic&lt;br&gt; PI &gt; 7 AND PLOTS &gt; &quot;A&quot; line&lt;br&gt; PI &gt; 4 AND PLOTS &lt; &quot;A&quot; line</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Organic&lt;br&gt; LL (oven dried)/LL (not dried) &lt;0.75</td>
<td>OL</td>
</tr>
<tr>
<td></td>
<td><strong>Sands and clays</strong>&lt;br&gt; Liquid limit &gt;50</td>
<td>Inorganic&lt;br&gt; PI PLOTS &gt; &quot;A&quot; line&lt;br&gt; PI PLOTS &lt; &quot;A&quot; line</td>
<td>CH</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Organic&lt;br&gt; LL (oven dried)/LL (not dried) &lt;0.75</td>
<td>OH</td>
</tr>
</tbody>
</table>

**Highly Organic Soils**

- Primarily organic matter, dark color, organic odor

**Sample Types**

- Auger cuttings
- Shelby tube
- Standard penetration (SPT)
- Modified California
- Rock core

**Additional Tests**

- CN - Consolidation
- CP - Compaction
- CT - Corrosivity testing
- DS - Direct shear
- PM - Permeability
- RV - R-value
- SA - Grain size analysis
- SW - Swell test
- TV - Torvane shear
- UC - Unconfined compression
- WA - Wash analysis

**Ground Water Levels**

- Water level at time of drilling
- Later water level after drilling

---

**Legend to Boring Logs and Soil Descriptions**

**Note:** Cu=D_{40}/D_{10}, Cc=(D_{30})^2/(D_{10} + D_{60})

**Blow Count**

The number of blows of a 140-pound hammer falling 30" required to drive the sampler the last 12 inches of an 18-inch drive. The notation 50/4 indicates 4 inches of penetration achieved in 50 blows.
Particle Size Distribution Report

<table>
<thead>
<tr>
<th>% COBBLES</th>
<th>% GRAVEL</th>
<th>% SAND</th>
<th>% SILT</th>
<th>% CLAY</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>32.0</td>
<td>39.1</td>
<td>28.9</td>
<td></td>
</tr>
</tbody>
</table>

### Soil Description
Dark grayish brown silty Sand with gravel (SM)

### Atterberg Limits
- **PL** =
- **LL** =
- **Pl** =

### Coefficients
- **D_{85}** = 14.1
- **D_{60}** = 2.70
- **D_{50}** = 1.13
- **D_{15}** =
- **C_u** = 0.0842
- **C_c** =

### Classification
USCS = SM
AASHTO =

### Remarks

---

Sample No.: Bag A  
Source of Sample:  
Date: 6-15-04

---

Client: Winzler & Kelly  
Project: Stonybrook Creek

Project No: 562.1
**Sunland Analytical**

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

Date Reported 06/13/2004  
Date Submitted 06/14/2004

To: Craig Newport  
Blackburn Consulting Inc.  
3265 Fortune Ct.  
auburn, CA  95602

From: Gene Oliphant, Ph.D.  
General Manager

The following is the report of analysis requested on SUN Order 42107.  
Your purchase order number is  .  
Thank you for your business.

<table>
<thead>
<tr>
<th>SUN #</th>
<th>Sample</th>
<th>Sample #</th>
<th>Chloride as ppm Cl /Dry Wt.</th>
<th>Sulfate as ppm SO4 /Dry Wt.</th>
</tr>
</thead>
<tbody>
<tr>
<td>82202</td>
<td>562.1\STONYBROOK CRK</td>
<td>BAG A</td>
<td>13.3</td>
<td>61.9</td>
</tr>
</tbody>
</table>

Methods: Sulfate-Cal Trans #417, Chloride-Cal Trans #422
## WEATHERING DESCRIPTORS

<table>
<thead>
<tr>
<th>Descriptors</th>
<th>Chemical weathering-Discoloration And/or oxidation</th>
<th>Mechanical weathering-Grain boundary conditions (disaggregation) primarily for granitics and some coarse-grained sediments</th>
<th>Texture and solutioning</th>
<th>General characteristics (strength, excavation, etc.) §</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Body of rock</td>
<td>Fracture surfaces †</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fresh</td>
<td>No discoloration, not oxidized</td>
<td>No discoloration or oxidation</td>
<td>No change</td>
<td>Hammer rings when crystalline rocks are struck. Almost always rock excavation except for naturally weak or weakly cemented rocks such as siltstones or shales.</td>
</tr>
<tr>
<td>Slightly weathered to fresh*</td>
<td>Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull</td>
<td>Minor to complete discoloration or oxidation of most surfaces</td>
<td>Preserved</td>
<td>Hammer rings when crystalline rocks are struck. Body or rock not weakened. With few exceptions, such as siltstones or shales, classified as rock excavation.</td>
</tr>
<tr>
<td>Slightly weathered</td>
<td>All fracture surfaces are discolored or oxidized</td>
<td>Partial separation of boundaries visible</td>
<td>Soluble minerals may be mostly leached</td>
<td></td>
</tr>
<tr>
<td>Moderately to slightly weathered*</td>
<td>Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are &quot;rusty&quot;, feldspar crystals are &quot;cloudy&quot;.</td>
<td>All fracture surfaces are discolored or oxidized</td>
<td>Generally preserved</td>
<td>Hammer does not ring when rock is struck. Body of rock is slightly weakened. Depending on fracturing, usually is rock excavation except in naturally weak rocks such as siltstones or shales.</td>
</tr>
<tr>
<td>Intensely to moderately weathered*</td>
<td>Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, see grain boundary conditions</td>
<td>All fracture surfaces are discolored or oxidized, surfaces friable</td>
<td>Texture altered by chemical disintegration (hydration, argillation)</td>
<td>Leaching of soluble minerals may be complete</td>
</tr>
<tr>
<td>Intensely weathered</td>
<td>Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, see grain boundary conditions</td>
<td>All fracture surfaces are discolored or oxidized, surfaces friable</td>
<td>Texture altered by chemical disintegration (hydration, argillation)</td>
<td>Leaching of soluble minerals may be complete</td>
</tr>
<tr>
<td>Very Intensely weathered</td>
<td>Discorlred or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay</td>
<td>Complete separation of grain boundaries (disaggregated)</td>
<td>Resembles a soil, partial or complete remnant rock structure may be preserved; teaching of soluble minerals usually complete</td>
<td>Can be granulated by hand. Always common excavation. Resistant minerals such as quartz may be present as &quot;stringers&quot; or &quot;dikes.&quot;</td>
</tr>
<tr>
<td>Decomposed</td>
<td>Compelete separation of grain boundaries (disaggregated)</td>
<td>Resembles a soil, partial or complete remnant rock structure may be preserved; teaching of soluble minerals usually complete</td>
<td>Can be granulated by hand. Always common excavation. Resistant minerals such as quartz may be present as &quot;stringers&quot; or &quot;dikes.&quot;</td>
<td></td>
</tr>
</tbody>
</table>

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

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