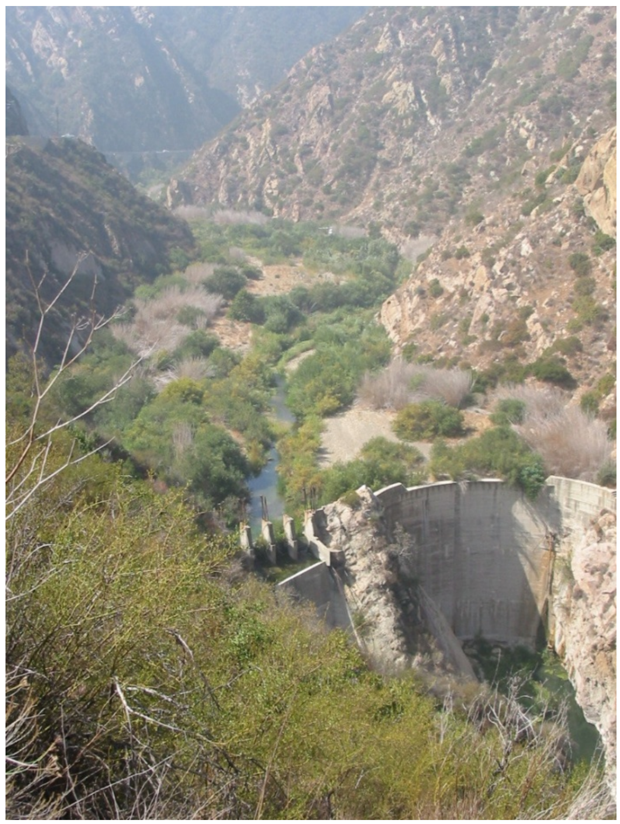
Malibu Creek Ecosystem Restoration Study

Los Angeles and Ventura Counties, California

Appendix C

Civil Design & Structural



U.S. Army Corps of Engineers
Los Angeles District

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1.0 INTRODUCTION

The overall goal of the Malibu Creek Environmental Restoration Feasibility Study is to restore the riparian and aquatic habitat, and the aquatic movement corridor along Malibu Creek. As covered in the Integrated Feasibility Report (IFR), the project objectives include the removal of existing Rindge Dam, including sediment removal for disposal and for beach nourishment, and establishment of fish passage.

Also reference the IFR for further explanation of Purpose of Study, Problems and Opportunities, Plan Objectives and Constraints, Key Assumptions, Local Sponsors Preferred Plan, Environmental Operating Principles, and Plan Implementation for the study procedural requirements,

This appendix summarizes the feasibility level methods developed for dam and sediment removal and preliminary design considerations for a fishway for Rindge Dam to support an array of project alternatives. Work included the investigation of various mechanical methods of sediment removal, including use of slurry, trucks and conveyors, design of various upland disposal sites and a beach disposal area. Access to the site for sediment and concrete dam removal were also evaluated. Geotechnical information was used for sediment quantity estimates, and design methods and quantities were used for cost estimates, plan formulation and environmental evaluations. Parametric design is used where applicable to support development for cost estimates for the alternative. A description of civil design & structural design assumptions and findings is summarized in the following sections.

In addition, the feasibility level design for the ramps and access roads is based on American Association of State Highway and Transportation Officials (AASHTO) manual on Design of Highways and Streets. Excavation and compacted fill slopes for the ramps and stockpile sites are based on Geotechnical recommendations documented in Appendix D. Dam demolition and sediment removal production rates are developed with the Project Delivery Team and Cost Engineering. Flood wall heights are based on Hydraulics and hydrology recommendations documented in Appendix B. The California Salmonid Stream Habitat Restoration Manual was used to investigate different fishway designs for this study.

The following are civil and structural design criteria:

- Develop and evaluate haul ramps layouts and vehicle accessibility.
- Develop and evaluate dam demolition feasibility.
- Develop and evaluate sediment haul route options and disposal area layout and capacities.
- Develop and evaluate sediment transport options.
- Develop parametric flood wall conceptual design for cost estimating purposes.

2.0 PROJECT ALTERNATIVES

As outlined in the Integrated Report, the project has been revised to include the following new Alternatives 2, 3, and 4:

2.1 <u>Alternative 1 – Take No Action</u>

Under this alternative, the dam would not be removed.

2.2 Alternative 2a –Dam Removal with Mechanical Transport

Under this alternative the project would extend for 5 years, assuming no local or regional restrictions to daily truck operating hours. The first year of the project would be dedicated to site preparation (clearing, diversion of surface flows, dewatering) and ramp construction. The dam and the sediment from behind the dam would be removed over the subsequent 4 year time span. Construction would only be performed outside the rainy season. Beach compatible sediment removed from behind the dam would be delivered to local beaches and all other sediment would be taken to the Calabasas Landfill to either be stockpiled or disposed of. All sediment would be removed with excavation equipment.

2.3 Alternative 2c- Dam Removal With Mechanical Transport, Limited Schedule

This alternative is the same as Alternative 2a, except that the available time frame for trucking material away from the dam site is limited to no more than 5-6 hours daily from 9 am to 3 pm per Los Angeles County imposed limits. Under this alternative the project would extend for 8 Years. The first year of the project would be dedicated to site preparation (clearing, diversion of surface flows, dewatering) and ramp construction. The dam and the sediment from behind the dam would be removed over the subsequent 7 year time span

2.4 Alternative 3a – Dam Removal with Natural Transport

Under this alternative the dam would be removed in 5 ft vertical increments over a span of 50 years and the sediment behind the dam would be removed over time by anticipated high flows down the creek. It is estimated that between 15,000 and 55,000 CY of sediment would be removed with each 5 ft vertical increment of dam removed. Over time, the sediment removed from behind the dam would be deposited downstream of the dam along the river bed between Cross Creek Road and Pacific Coast Highway. This deposited material would increase flood risks for nearby homes and commercial development and would require flood mitigation structures to be constructed along the river between Cross Creek Road and Pacific Coast Highway.

2.5 Alternative 4a – Dam Removal with Hybrid Mechanical and Natural Transport

This method is referred to as the “Hybrid” method because it uses both flows within the creek and excavation equipment to remove the sediment impounded behind the dam. Under this alternative the project would extend for 5 years, assuming no local or regional restrictions to daily truck operating hours as discussed before. The first year of the project would be dedicated to site preparation (clearing, diversion of surface flows, dewatering) and ramp construction. The dam and the sediment from behind the dam would be removed over the subsequent 4 year time span. At the end of each construction period an additional 5 ft of the dam would be removed so sediment could be naturally transported downstream by stream flow during the rainy season. It is estimated that between 15,000 and 55,000 CY of sediment would be removed from behind the dam each year by anticipated high flows. Construction would only be performed outside the rainy season. Beach compatible sediment removed from behind the dam would be delivered to local beaches and all other sediment would be taken to the Calabasas Landfill to either be stockpiled or disposed of. Any sediment removed from behind the dam by natural stream flows would be deposited downstream of the dam along the river bed between Cross Creek Road and Pacific Coast Highway. This deposited material would increase flood risks for nearby homes and commercial development and would require flood mitigation structures to be constructed along the river between Cross Creek Road and Pacific Coast Highway.

For each Alternative 2 through 4 there is an Alternative “b” variant where the removal of the dam and the impounded sediment is the same, but the “man made” barriers (such as culverts and bridges) upstream of Rindge Dam are either removed or modified to allow the Steelhead Trout to continue their upstream migration. Refer to the Appendix C1 for more information.

3.0 DAM REMOVAL

Built between April 1924 to January 1925, Rindge Dam is a 100-ft-high thin-arch concrete dam with a detached spillway cut into bedrock. The dam became State of California jurisdictional for safety following passage of legislation in August 1929. It was certified for safety operation on October 17, 1935. On May 2, 1967, the dam was declared nonjuridictional when it was found to store less than 15 acre-feet of water which is the threshold storage the State defines for a dam. **Figure 3.0-1** – Rindge Dam shows the dam viewed from above and looking upstream from Malibu Canyon Road.

The length of the dam at the crest is 175 ft and 95 ft at its base. The thickness of concrete at the base of the dam is 11.5 ft and tapers to a width of 2 ft at the top. The dam foundation extended 15 ft to below the original streambed to bedrock. Based on as-built drawings, the dam was reinforced with 30-foot long steel railroad rails weighing sixty lbs/yd. The rails were placed vertically near the abutments at the crest and at the base at the center of the arch on the upstream face.

The spillway has five concrete buttresses with four 8 ft wide by 11 ft high flow ways. The dam foundation extended 15 ft to below the original streambed to bedrock. A plan view of the spillway, taken from Google Earth, can be seen in **Figure 3.0-2**. The spillway would be removed with the dam.

Figure 3.0-1 Rindge Dam

Figure 3.0-2 Rindge Dam Spillway

3.1 Existing Condition of the Dam

Since de-certification in 1967, the dam was inspected by State Division of Safety of Dams (DSOD) on October 26, 1988 which documented no signs of incipient failure observed. Ownership of the dam was transferred to the State Department of Parks and Recreation in 1984. A subsequent inspection on May 28, 1992 by DSOD in response to their internal request concluded the dam and reservoir were not in danger of sudden failure. However, monitoring of the abutments and erosion downstream of the spillway and periodic evaluations were recommended. Currently, there is no obvious collapse or deterioration in the exposed downstream face of Rindge dam or along its abutments, but it should be recognized that no detailed, scientific inspection of the dam has occurred for many years.

The USACE in 2005 conducted an examination of the parts of the dam that could be reached readily on foot. This inspection area constituted only a small part of the dam, mostly in the upper few feet of the face and the uppermost abutments, and the width of the crest. The remainder of the dam downstream face was examined only to the degree that could be seen by binoculars. The upstream face of the dam is buried, mostly, and is an unknown. Concrete and abutment bedrock surfaces that were accessed were subjected to recoil instrument readings (Schmidt hammer) to estimate soundness of the rock and the concrete. Testing results are favorable.

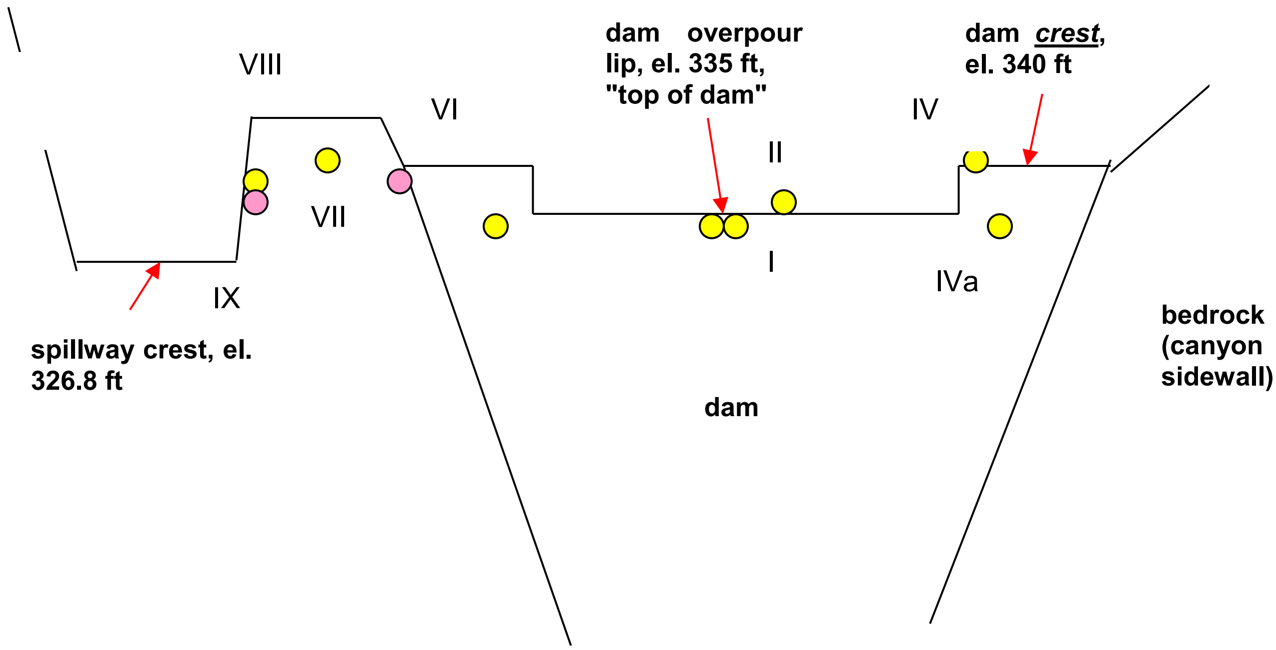
Schmidt hammer testing was conducted at Rindge Dam on 15 September 2005. The Schmidt hammer is an index apparatus for non-destructive testing of the compressive strength of concrete in situ. Because a number of factors such as testing methodology, hammer type, normalization of rebound values, surface smoothness, weathering and moisture content can affect results, the values obtained should be considered only as a general indication of concrete strength. The Schmidt hammer used for the tests on Rindge Dam was a Tecnotest (no model number given) instrument that gave correlated compressive strength readings between 10 and 70 Mpa (megapascals) from hammer rebound reading ranging from 20 to 55. The graphed correlations adjust for the angle of the Schmidt hammer with the horizon, the value of α being zero degrees when the instrument is horizontal, and -90 degrees when the instrument is pointed down vertically. In some cases, the Schmidt hammer was also used to assess the characteristics of the canyon bedrock, between the dam and spillway. **Table 3.1-1** and **Figure 3.1-1** show the test locations and rebound values on the dam and abutment bedrock.

Utilizing the conversion curves attached to the Tecnotest Schmidt hammer, the average rebound values have been converted into approximate compressive strengths, as shown in **Table 3.1-2**. Although there is a range of compressive strength values, the strengths do generally indicate that the concrete is in good condition. This assessment is in agreement with the visual inspection of the dam, which found that the concrete was free of significant spalling or other surficial distress.

Table 3.1-1 Schmidt Hammer Tests, Rindge Dam

Test location I	**Test location II**	**Test location III**	**Test location IV**	**Test location IVa**	**Test location V**
Values:	50	37	40	44	40
50	44	30	40	36	48
44	46	38	46	44	50
50	46	40	42	40	55
44	40	CONCRETE	28*	38	48
42	47		30*	44	36
43	42		20*	CONCRETE	44
44	40		26*		44
40	42		30*		38
40	40		CONCRETE		CONCRETE
40	42				
CONCRETE	42				
	44				
	38				
	44				
	CONCRETE				
Test location VI	**Test location VII**	**Test location VIII**	**Test location IX**		
52	43	37	51		
38	54	47	50		
47	46	46	47		
BEDROCK	CONCRETE	42	50		
		46	50		
		38	BEDROCK		
		CONCRETE			

* Test impacted by reinforcing iron at shallow depths below concrete surface.



Note: Sketch not to scale

Figure 3.1-1 Elevation Reference Points, General Features of Rindge Dam, Upstream View (West)

LEGEND:
I is on u/s face of dam, on the vertical, at overpour lip, instrument horizontal;
II is on the crest of the dam, overpour lip, instrument vertical;
III is on the d/s face of the dam on the vertical, overpour lip, instrument horizontal;
IV is on the horizontal and vertical faces of the "steps" leading to dam crest, N. of overpour lip;
IVa is on vertical face, u/s side of dam, about 6 ft below the crest;
V is on vertical face, u/s side of dam, about 10 ft below the crest;
VI is on Sespe conglomerate bedrock right abutment of dam, u/s side of dam;
VII is on vertical face, u/s side of dam at right abutment, about 10 ft below the crest
VIII is on vertical face, u/s side of spillway structure concrete, about 10 ft below a gate hoist;
IX is on Sespe conglomerate bedrock adjacent the SPILLWAY structure concrete, u/s side of dam.

Utilizing the conversion curves attached to the Tecnotest Schmidt hammer, the average rebound values have been converted into approximate compressive strengths, as shown in **Table 3.1-2**. Although there is a range of compressive strength values, the strengths do generally indicate that the concrete is in good condition. This assessment is in agreement with the visual inspection of the dam, which found that the concrete was free of significant spalling or other surficial distress.

Table 3.1-2 Compressive Strengths, Rindge Dam

Test Location	Material Tested	Maximum Rebound	Minimum Rebound	Average Rebound	Hammer Angle α	Average Compressive Strength Mpa	Average Compressive Strength PSI
I	Concrete	50	40	47	horizontal, 0°	58	8400
II	Concrete	50	38	43	vertical, -90°	56	8100
III	Concrete	40	30	36	horizontal, 0°	34	5000
IV	Concrete	--	--	--	--	--	--
IVa	Concrete	44	36	41	horizontal, 0°	45	6500
V	Concrete	36	55	45	horizontal, 0°	54	7800
VI	Bedrock	52	38	46	horizontal, 0°	56	8100
VII	Concrete	54	43	48	horizontal, 0°	60	8700
VIII	Concrete	47	37	43	horizontal, 0°	50	7250
IX	Bedrock	51	47	50	horizontal, 0°	64	9200

Additional concrete testing will be required prior to the dam removal to verify the integrity of the dam.

3.2 Demolition Methods

The anticipated method of demolition discussed below is based on Project Delivery Team discussions and preliminary research into demolition methods. The method presented is a conservative approach. The final plans for demolition methodologies would be determined with further investigation. Also see Risk Analysis in the Cost Engineering Appendix F on assessment of uncertainties and risks.

Dam stability during demolition will be addressed if the alternatives become favorable. Structural evaluation of the dam will need to be included during design for Preconstruction Engineering and Design (PED) phase. See further discussion in Geotechnical Appendix D, Section 5.5.

The dam would be demolished by the use of a combination of high impact breakers, blasting, and diamond wire saw cutting methods. The latter provides some advantages in that the diamond-wire sawcutting will provide smooth surfaces, facilitate excavation of notch portions of the arch dam section, improve control of the excavation grade, provide smooth working surfaces for excavation of each layer, and permit removal of the concrete in large blocks (rather than attempting to confine rubble to the working surface and removing the rubble by loaders). Large mobile cranes would be placed on pads and used to remove dam and spillway concrete.

The diamond wire cutting method was selected to remove the concrete arch portion of the dam for the following reasons:

- Ideal for cutting through composites – The dam is reinforced with 60 lbs/yd steel railroad rails embedded within the concrete arch of the dam. These rails have a total surface area of 5.93 square inches and are embedded 1 ft from the either face of the dam. The ties run along the 175 ft length of the dam and the 100 ft height of the dam and are space 10 ft on center in both directions. The diamond wire would be able to cut through the concrete and the steel rails simultaneously.

- Minimize debris – Removing the concrete arch in large blocks minimizes the amount of debris that would be generated during demolition and would reduce measures taken to prevent debris from entering the channel downstream of the dam.

The diamond wire saw method for demolition of the dam arch is used for the purposes of preparing cost estimates and demonstrates the technical feasibility of this method when combined with use of cranes, assumed blocks weighing less than 19 tons each, and methods to safely anchor and lift the blocks. It is not intended to preclude consideration of other methods for dam arch concrete removal considered during PED or Construction, as long as the consequences of other methods are clearly understood, evaluated, and coordinated with appropriate agencies.

The foundation of the dam would be removed using a combination of blasting and breakers. Any debris from the blasting would be contained with blasting mats and excavators would be used to load the debris onto trucks.

The concrete spillway would be demolished by first pre-splitting the concrete from the rock substratum than drilling and micro-blasting the surface to fracture the concrete and then manually breaking the concrete.

All the debris from the demolition would be taken to the Calabasas Landfill for disposal.

3.2.1 Quantities

Assuming the diamond-wire sawcutting method is used to produce large chunks of concrete with smooth surfaces, these large chucks of concrete are estimated at a total volume of 3,460 cubic yards, each weighing approximately 19 tons. These chunks would be loaded onto a flat bed truck using a crane located at the SRA. The full removal of the arch section will require about 325 truck trips. The dam foundation and spillway would be removed by blasting techniques. The dam foundation will generate about 540 cubic yards of concrete rubble while the spillway is expected to generate about 2,000 cubic yards of concrete rubble. The dam's foundation removal will require about 60 truck trips. The spillway removal will require about 154 truck trips. The concrete waste would be disposed of by hauling to the Calabasas Landfill located on Lost Hills Road in Agoura about 10.2 miles upstream of Rindge Dam.

3.3 <u>Demolition Time Frame</u>

The time frame for removing the dam are detailed as follows: See Appendix D and Integrated Feasibility Report for anticipated limitations of local and regional restrictions on daily truck operating hours that would limit productive transport time to no more than 5-6 hours, thereby yielding an estimated dam demolition and sediment removal period of 7 to 8 years.

3.3.1 Alternatives 2a, 2b, and 2c

For Alternatives 2a and 2b, the project would extend over five years based on assumption with no local or regional restrictions to daily truck operating hours. The first year would be used for preparatory work. The dam would be removed in four years during project years two through five. For Alternative 2c, the project would extend over 8 years, however the dam would be removed in 6 years during project years two through seven.

During each year of dam removal for the three alternatives, enough of the dam would be removed from the crest downward so the top of the remaining dam would coincide with the top of the sediment remaining behind the dam after excavation operations. The finished grade of the sediment at the end of each year would be gently sloping toward the top of the dam, it would be relatively uniform, and be free of any large depressions.
Demolition crews would start work one month after the sediment removal operations have began to ensure the sediment directly behind the dam has been removed and there is adequate room for demolition crews to work. Demolition operations would continue until no later than one month after the sediment removal operations have ended for each year. The estimated dam removal heights by project year are for each alternative summarized in the **Table 3.3-1** and **Table 3.3-2.**

Table 3.3-1 Dam Removal Heights - Alternatives 2a and 2b

Project Year	Height of the Dam to be Removed
2	38 ft
3	15 ft
4	20 ft
5	27 ft and the foundation

Table 3.3-2 Dam Removal Heights - Alternative 2c

Project Year	Height of the Dam to be Removed
2	29 ft
3	9 ft
4	10 ft
5	13 ft
6	15 ft
7	24 ft and the foundation

The dam removal values presented in **Table 3.3-1** and **Table 3.3-2** are based on the estimated depths of sediment removed during each project year. See the Appendix D for more information on sediment removal operations.

Saw cutting operations for each alternative would be limited to two crews removing the dam, one crane loading concrete blocks onto the truck, and one 20 cubic yard (cy) truck used to dispose of the concrete. Each crew would cut a block from the dam that would be 7ft long and 6 ft wide, with heights that vary from 2 ft to 6 ft. Block heights would be limited to 6 ft to ensure the blocks are less than 20 tons and can be transported along the proposed haul routes. Sawcutting crews would be able to cut 25 square feet of surface area every hour which means each crew would be able to remove one to three blocks per day, depending on the block height. This production is approximately equal to 9.5 cy of material removed by each crew in a day.

Sawcutting crews were limited to ensure the dam removal operations would not interfere with the sediment removal operations. The sawcutting rates could be increased by increasing the number of crews working on demolishing the dam, however the overall project time frame would not change. The height of the dam removed each year would ultimately be controlled by the amount of sediment that has been removed from behind the dam, not the rate at which the dam is removed.

3.3.2 Alternatives 3a and 3b

The entire width of the dam would be lowered in 5ft vertical increments to facilitate the natural transportation of the sediment by the creek. Once the dam height was initially reduced by 5 ft, the height of the dam would be decreased in 5ft increments every 2 to 3 years, or whenever the top of the impounded sediment matches the top of the dam. This removal process would continue until the dam is completely removed in approximately 50 years. No sediment would be mechanically removed from the dam site. Any material that is excavated from directly behind the dam would be placed upstream of the dam so it can be removed through natural sediment transport process.

There would be no limits to number of sawcutting crews since there would be no sediment removal operations taking place that may cause construction conflicts. Each crew would cut a block from the dam that would be 7ft long and 6 ft wide, with heights that vary from 2 ft to 6 ft. Block heights would be limited to 6 ft to ensure the blocks are less than 20 tons and can be transported along the proposed haul routes. Sawcutting crews would be able to cut 25 square feet of surface area every hour which means each crew would be able to remove one to three blocks per day, depending on the block height. This production is approximately equal to 9.5 cy of material removed by each crew in a day. 20 cy capacity trucks would be used to transport all material removed from the dam to reduce the total number of truck trips.

3.3.3 Alternatives 4a and 4b

The project would extend over five years based on assumption with no local or regional restrictions to daily truck operating hours. The first year would be used for preparatory work. The dam would be removed in four years during project years two through five. Each year, enough of the dam would be removed so the top of the dam is 5 ft below the top of the sediment. Crews would start work one month after sediment removal operations have began to ensure the sediment directly behind the dam has been removed and would continue operations until no later than one month after the sediment removal operations have ended. The estimated dam removal heights by project year are summarized in **Table 3.3-3.**

Table 3.3-3 Dam Removal Heights - Alternatives 4a and 4b

Project Year	Height of the Dam to be Removed
2	43 ft
3	20 ft
4	25 ft
5	12 ft and the foundation

The dam removal values presented in **Table 3.3-3.** are based on the estimated depths of sediment removed during each project year. See the Appendix D for more information on sediment removal operations.

Saw cutting operations would be limited to two crews removing the dam, one crane loading concrete blocks onto the truck, and one 20 cy capacity truck used to dispose of the concrete. Each crew would cut a block from the dam that would be 7ft long and 6 ft wide, with heights that vary from 2 ft to 6 ft. Block heights would be limited to 6 ft to ensure the blocks are less than 20 tons and can be transported along the proposed haul routes. Sawcutting crews would be able to cut 25 sq feet of surface area every hour which means each crew would be able to remove one to three blocks per day, depending on the block height. This production would be approximately equal to 9.5 cy of material removed by each crew in a day.

Sawcutting crews were limited to ensure the dam removal operations would not interfere with the sediment removal operations. The sawcutting rates could be increased by increasing the number of crews working on demolishing the dam, however the overall project time frame would not change. The height of the dam removed each year would be controlled by the amount of sediment that has been removed from behind the dam, not the rate at which the dam would be removed.

3.4 Calculation of Dam Height to be Removed

For each alternative, the rate at which the dam would be removed was determined by first estimating sediment removal rates per year and then using a model of the impounded sediment to determine excavation depths at the face of the dam. The depth of excavation in the model was manipulated until the total volume removed calculated by the model equaled the estimated total volume removed at the end each project year. Sediment removal is further discussed in the Appendix D.

3.5 Development of the Sediment Model

The sediment model was created to approximately estimate the volume of sediment removed from behind Rindge Dam and the quality of the material removed based on information provided in Geotechnical Appendix D (Attachment D from prior legacy F-4 Feasibilty Study Geotechnical Appendix). The information is now included in current feasibility study Appendix C2). Appendix C2 makes the following key assumptions to calculate the volume of sediment:

1) Malibu Creek is divided into 4 “Blocks”. Each block is a box shape with a constant width and depth along the creek. Each block width is based on the average canyon width along the creek and each blocks depth is based on borings collected within the creek. The blocks are defined by stationing along the creek, where station 0 is at the dam and station 2545 is 2,545 feet from the dam. **Table 3.5-1** summarizes each blocks characteristics:

Table 3.5-1 Block Characteristics

	Starting Station	End Station	Total Length (FT)	Block Width (FT)	Block Depth (FT)
Block 1	0	330	330	250	94.13
Block 2	330	1155	825	270	81.23
Block 3	1155	2045	890	175	44.47
Block 4	2045	2545	500	75	19.90

2) The canyon side slopes are accounted for with a reduction factor. Due to lack of reliable topographic information about the canyon, the reduction in canyon width as the creek is excavated is estimated by a volume reduction factor. These factors were developed based on typical geological characteristics for the region and reflect typical canyon side slopes. They represent a percent reduction in the total block volume to determine the final amount of material excavated from the canyon. **Table 3.5-2** shows the reduction factors used for each Block:

Table 3.5-2 Side Slope Reduction Factors

Block 1		30%
Block 2		30%
Block 3		50%
Block 4		50%

3) Each “Block” is divided into 3 “Units” to determine the type of material being excavated. Units represent like materials in each block and reflect general categories of material impounded behind the dam. The Unit composition is based on information from borings taken along the creek. Each unit is broken down into “Sand”,” Silt and Sand”, and “Other”. Attachment D defines “Other” as cobble and larger material. **Table 3.5-3** summarizes the characteristics of each Unit:

Table 3.5-3 Unit Characteristics

	Material Layer	Description	% Sand	%Silt and Clay	%Other
Unit 1	Fluvial deposition (i.e., not deposited in a reservoir pool)	Sand, gravel, cobbles, and larger rock	28	2	70
Unit 2	Shallow to intermediate depths reservoir pool deposition	Mainly silty sands with organic content; does contain silt layers, some gravel	73	22	5
Unit 3	Deeper depths of reservoir pool deposition	Sandy silts, lean clays, and silts (all with organic content); does contain some silty sand layers	22	78	0

4.) Unit depths vary between Blocks and are based on average depths of material found in borings along the creek. **Table 3.5-4** summarizes the depth of each unit by Block:

Table 3.5-4 Unit Depth, in Feet, By Block

	Unit 1	Unit 2	Unit 3
Block 1	13.75	29.28	51.1
Block 2	21.81	39.09	20.33
Block 3	17.27	27.20	0
Block 4	14.90	0	5

3.6 <u>Sediment Model Calculations</u>

The sediment model calculates volumes using cross sections that are based on the geometry of each Block, the material characteristics of each Unit, and the reduction factor for each Block. The depths of each Unit were taken directly from Attachment D while the cross section width was calculated with the following steps:

1. Find the volume of the entire Block
2. Reduce the volume of the Block by the appropriate factor
3. Divide the reduced volume by the length of the Block to get a cross sectional area
4. Divide the cross sectional area by the depth of the Block to get the width of the Block

Table 3.6-1 summarizes the cross sections widths used to calculate the volume of material at different depths:

Table 3.6-1 Cross Section Widths by Block

	Width (ft)
Block 1	175
Block 2	189
Block 3	71.25
Block 4	37.50

The sediment model develops a cross section at the start and end of each Block and every 100 ft along the creek, starting at Rindge Dam. To calculate volumes, the spread sheet takes two cross sections, averages the area of the two, and multiples the average area by the distance between the two cross sections. This is done between every cross section along the creek and the calculated volumes are added up to determine the total volume of material removed. As the excavation depth changes, the areas of the cross sections change and cross sections from two different blocks are not used in the same calculation.

Once the amount of material excavated is determined, the sediment model then converts the volumes into the amount of Sands, Silts and Clays, and Other based on the characteristics of each Unit.

The sediment model was developed to compare different creek bed profiles to help evaluate the amount of material removed over different time periods. The model considers three different profiles:

- Existing Slope – Establishes the existing elevations along the creek bed. These elevations should not change while using the sediment model since they are used to determine where each Unit begins and ends. The sediment model is formatted so the elevation of the creek does not have to be held constant.

- Notch 1 – This profile represents the creek at the beginning of a year of work. At Year 1 the elevations would match the Existing Slope Profile, at Year 2 the elevations should match the elevations along the creek after excavation. This profile is compared to the Existing Slope Profile to determine excavated quantities and represents total excavation year to date.

- Notch 2 – This profile represents the stream after a year of excavation work. This profile is compared to the Notch 1 profile to determine the amount of material excavated after a year of excavation.

Figure 3.6-1 is a graphical illustration of how Blocks and Units relate to the three profiles used in volume calculations.

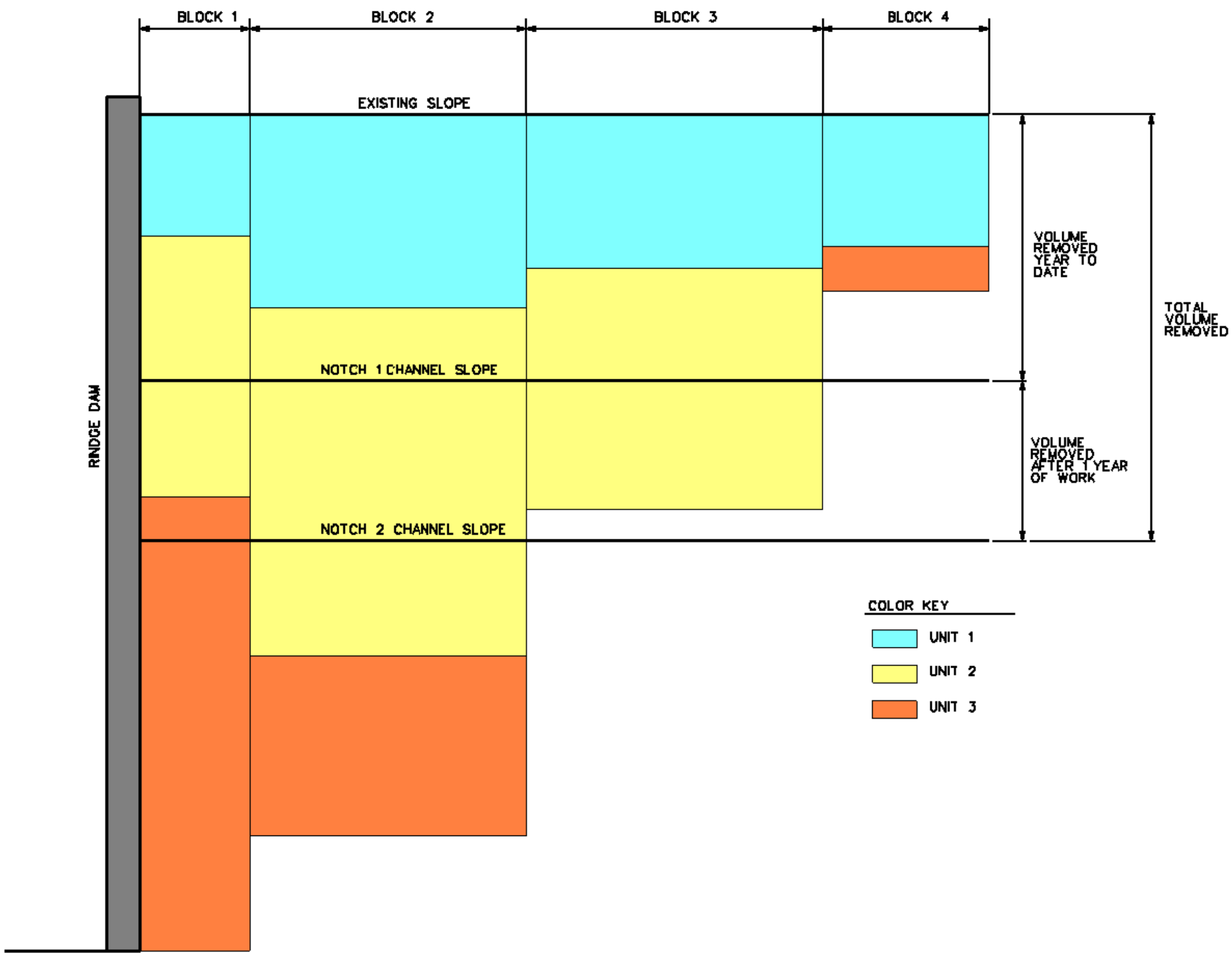


Figure 3.6-1 Sediment Model

3.6.1 Limitations of the Sediment Model

The blocks used for volume calculations transform the unknown trapezoidal geometry of the creek canyon into a simplified rectangular shape to calculate the amount of material removed at a given depth. Due to this transformation, the spread sheet underestimates the volume removed for excavation depths closer to the top of the dam while over estimating the volume removed for excavation depths closer to the creek bed.

Figure 3.6-2 below compares a basic trapezoidal representation of the canyon to the cross section of Block 1 used in the calculations in the sediment model. **Figure 3.6-2** uses Unit depths from the spread sheet for both the basic trapezoidal shape and the rectangular shape. Note, volume is the product of area multiplied by the length of the block; since the length of the block is constant, **Table 3.6-1** only compares areas of each unit.

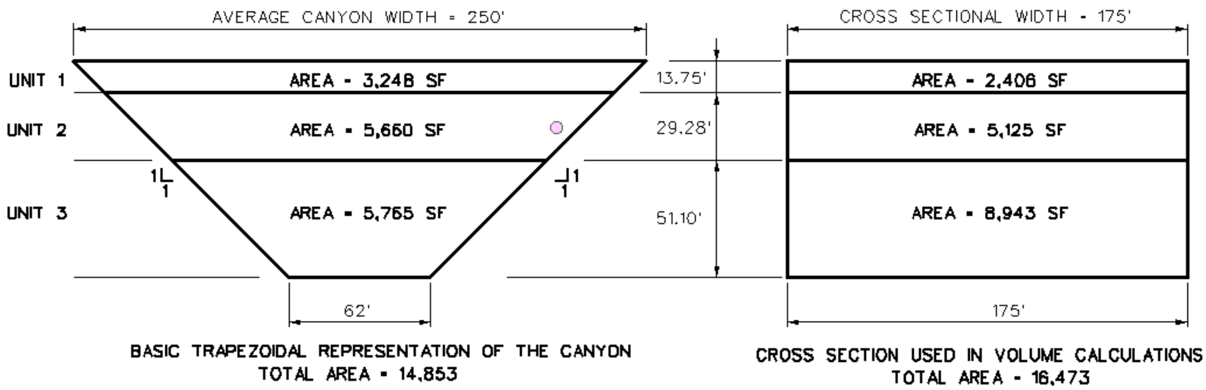


Figure 3.6-2 Model Cross Section

Table 3.6-1 shows that the sediment model could actually over estimate the total volume removed from behind the dam. This is not a problem because the side slopes and width of the canyon vary so much that the methods used in the spread sheet ultimately reflect average conditions and calculates a reasonable estimate for the total material removed from behind the dam.

3.6.2 Dam Stability during Demolition

The stability of the dam during demolition will not be a critical issues because loading on the dam will be reduced as the sediment is removed from the impound area. Preliminary analysis shows that the dam will be able to support any 6 ft free standing segment which corresponds to the proposed 6'x6'x7' pieces the dam will be removed in.

3.6.3 Spillway Removal

The spillway would be removed in stages for each of the newly proposed Alternatives 2 through 4. The top half of the spillway would be removed when the top of the remaining sediment matches the top of the spill way. The second half of the spillway would be removed when the entire dam has been removed and crews would have direct access to the bottom portion of the spillway. All the debris from the spillway would be taken to the Calabasas Landfill for disposal.

4.0 IMPOUNDED SEDIMENT AND DAM SITE ACCESS

The dam and impounded sediment are located 100 ft below Malibu Canyon Road in a canyon with steep side slopes. There is no existing access ramp down to the dam site.

The access ramp conceptual designs discussed below is based on limited Geotechnical information on the slope stability of the canyon walls for construction and removal of the proposed access ramps. The design presented is a conservative approach. Final ramp design would be determined with further geotechnical investigations for canyon slope stability in the design phase. Also see Risk Analysis in the Cost Engineering Appendix F on assessment of uncertainties and risks for sediment removal including excavation, loading and hauling.

4.1 Truck Size

One key concern for the project is traffic impacts along the proposed haul routes and the project time frame. 20 cy capacity trucks were selected to transport all excavated sediment and debris out of the site and to either the Calabasas Landfill or the beach disposal sites because they would reduce the number of trucks leaving the site along with decreasing the project time frame by approximately 1 year. A detailed sediment removal plan for Alternative 2a can be found in the Appendix D.

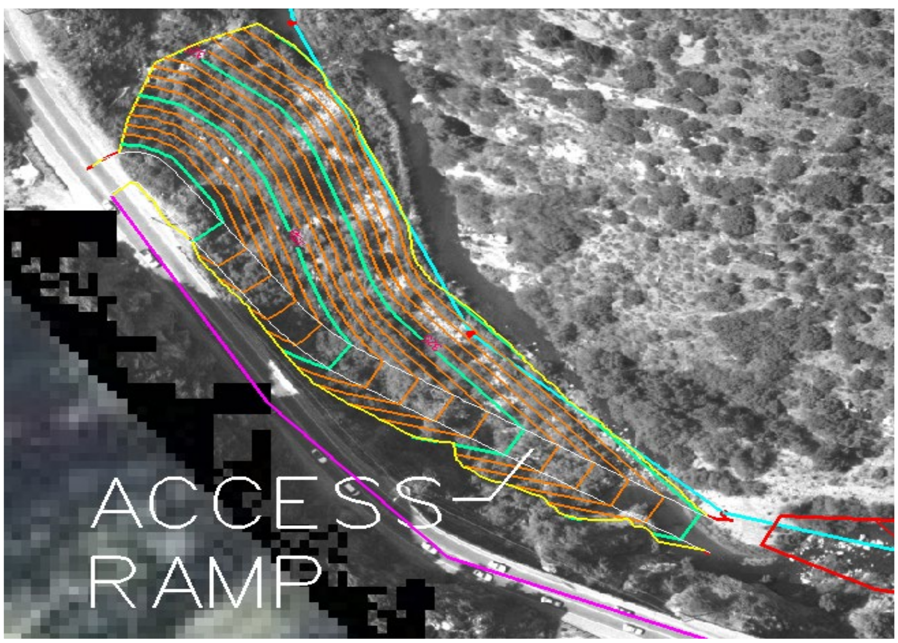


Figure 4.1-1 Access Ramp

After review, this ramp was determined to be inadequate for the current project requirements of the sediment removal plan for two reasons:

- Trucks would have insufficient space to turn from the ramp onto the southbound lane of Malibu Canyon Road without increasing traffic impacts
- The ramp design with 18% slope along the ramp is too steep of a slope for fully loaded 20 cy trucks to travel up.

For the revised sediment removal plan, 20 cy capacity trucks have to travel north along Malibu Canyon Road to reach the Calabasas landfill and they must also travel south along Malibu Canyon Road to reach the beach nourishment sites. Trucks must also be able to access the ramp when driving north when returning from the beaches and south when returning from the landfill.

Based on the "Geometric Design of Highways and Streets" by AASHTO, a double trailer truck has a minimum inner turning radius of 19.3 ft and a maximum outer turning radius of 45 ft. The template from Geometric Design of Highways and Streets is shown in **Figure 4.1-2**.

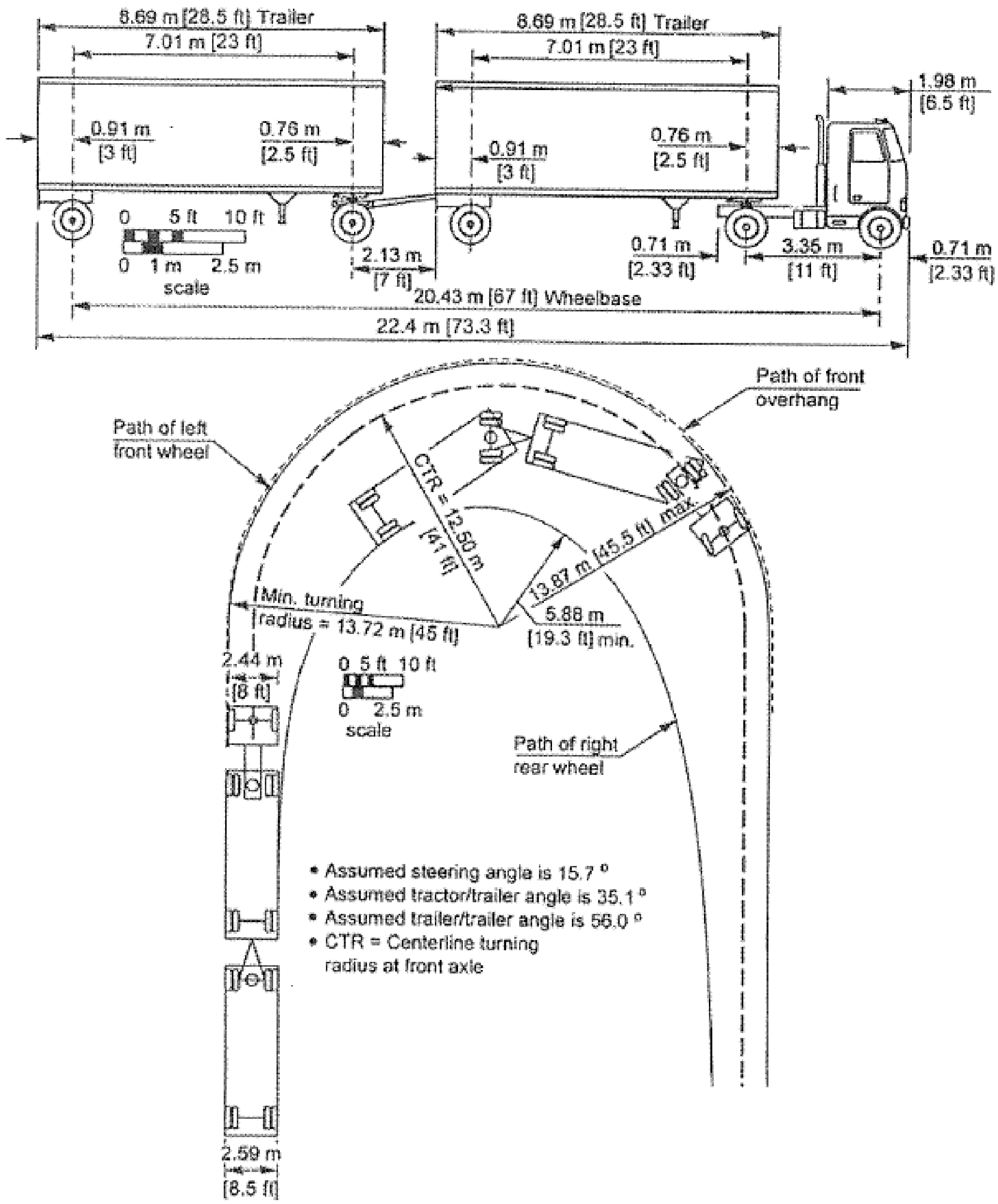


Figure 4.1-2 20 CY Capacity Truck Turning Radius

Following this template, trucks coming from the South cannot make the turn onto the design access ramp. Figure 4.1-3 illustrates the limitations in the tuck turning radiuses.

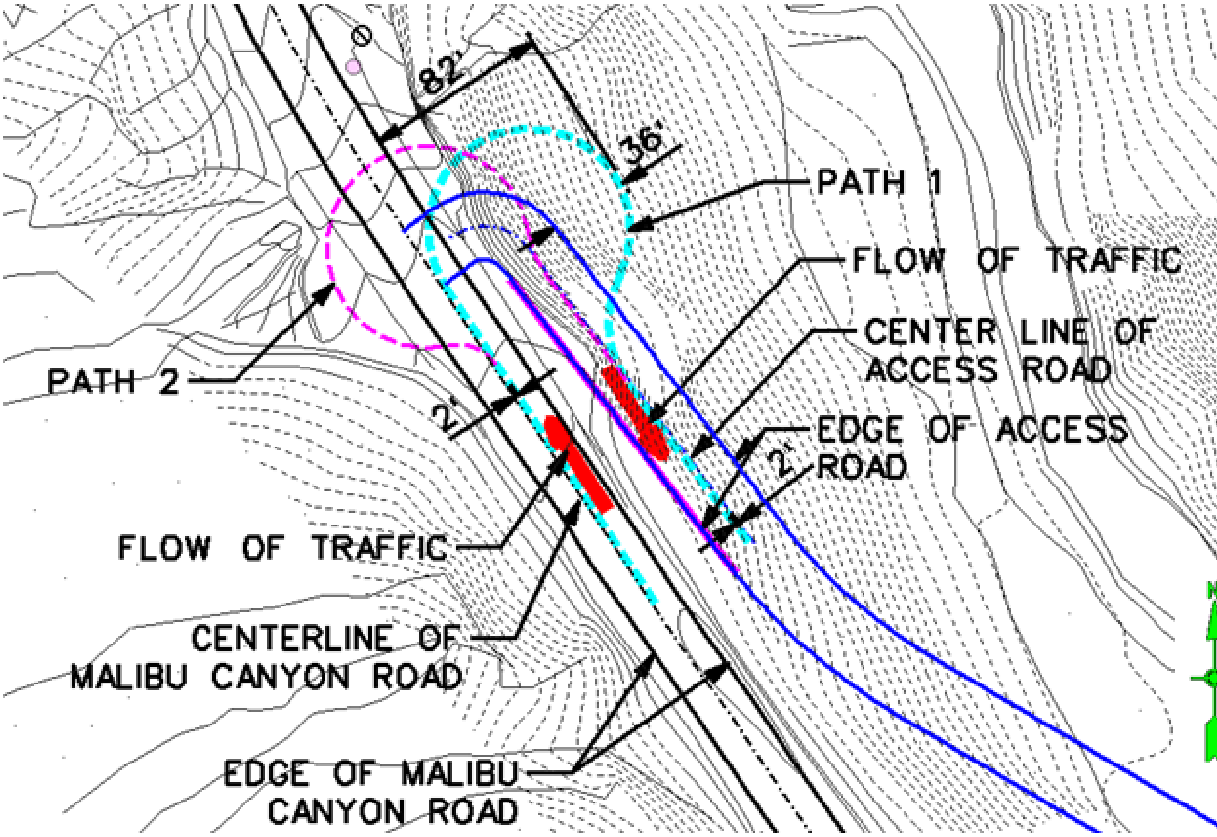


Figure 4.1-3 20 CY Capacity Truck Traffic Coming from the Beach Sites

Figure 4.1-3, Path 1 (The light blue, dashed line) represents the assumed path of the outer left wheel of the 20 cy capacity truck. Based on the minimum 45 ft outer turning radius, the truck would have to drive 36 ft past the proposed edge of the access ramp.

Figure 4.1-3, Path 2 represents a turning path that was determined not to be viable since it would increase traffic impacts of the project. Based on the estimated sediment removal rates presented in the Appendix D, during peak sediment removal operations of beach compatible material, approximately 16 trucks would be filled every hour. Based on this production rate, 16 trucks would be leaving the project site and 16 trucks would be entering the site every hour. If truck access was limited to the single ramp then all 32 trucks would block both lanes of traffic while making the required turns from either Malibu Canyon Road or the access ramp. Assuming it would take about a minute for a 20 cy capacity truck make the turn shown by Path 2 in **Figure 4.1-3** and about a minute for a full 20 cy capacity truck to turn onto the southbound lane of Malibu Canyon Road from the access ramp, than both lanes of Malibu Canyon would be blocked for at least 30 minutes out of every hour of sediment removal operations.

4.2 Revised Site Access – Alternatives 2 and 4

The site access ramps have been revised for Alternatives 2 through 4 in order to reduce traffic impacts along Malibu Canyon Road and facilitate quicker truck access.

4.2.1 Two Access Ramps

The preferred site access plan requires two ramps; one ramp for trucks heading to the landfill to the north and one ramp for trucks heading south to the beach disposal sites. **Figure 4.2-1** shows both ramps.

Figure 4.2-1 Proposed Access Ramps

Northbound Ramp

The ramp would start at the low point in Malibu Canyon road, would be 30 ft wide, 730 ft long, and descend into the canyon at maximum grade of 15%. This ramp would be built along the canyon side slopes and would leave an opening along the channel invert for flows during the winter. The proposed ramp is shown in **Figure 4.2-2** Northbound Ramp. This ramp would accommodate two-way 20 cy capacity truck traffic. Since truck traffic along the Northbound Ramp would be limited to trucks heading to the landfill, traffic along Malibu Canyon Road would only be blocked while trucks returning from the landfill turned left from Malibu Canyon Road onto the access ramp.

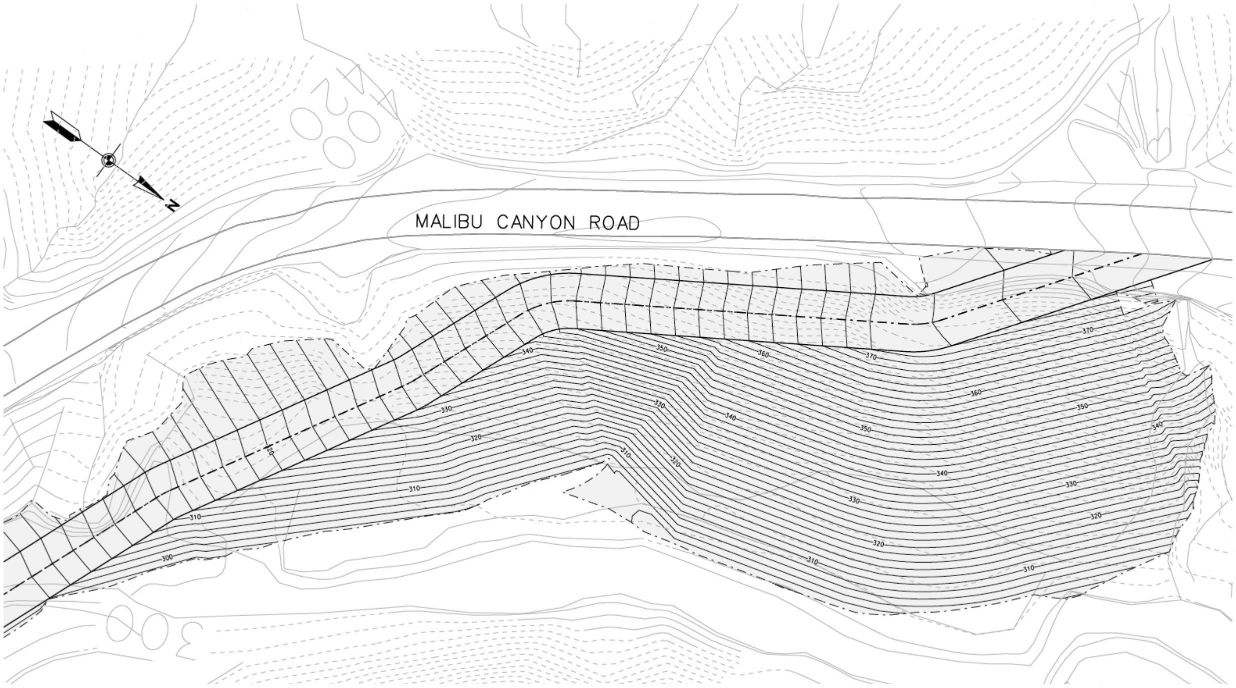


Figure 4.2-2 Northbound Ramp

<u>Southbound Ramp</u>

The ramp for trucks heading to the beach would be built on top of the existing invert access ramp that has had its toe washed away by creek flows.

The ramp would be only 15 ft wide, 1,000 ft long and descend into the canyon at a maximum grade 15%. This ramp leaves an opening along the channel invert for flows during the winter. The proposed ramp is pictured in **Figure 4.2-3**.

The switchback turn is based on the Minimum Turning Path for Double-Trailer Combination template found in AASHTO's "Geometric Design of Highways and Streets".

This ramp would only accommodate one-way 20 cy capacity truck traffic. If the road was designed wide enough to allow two-way 20 cy truck traffic, the fill required for the ramp would block the creek.

This ramp would minimize traffic impacts of truck traffic heading to the beach and returning from the beach by eliminating the requirement to make a 180 degree turn pictured in **Figure 4.1-3**. Trucks would only block traffic when they are turning left from the access ramp onto Malibu Canyon Road.

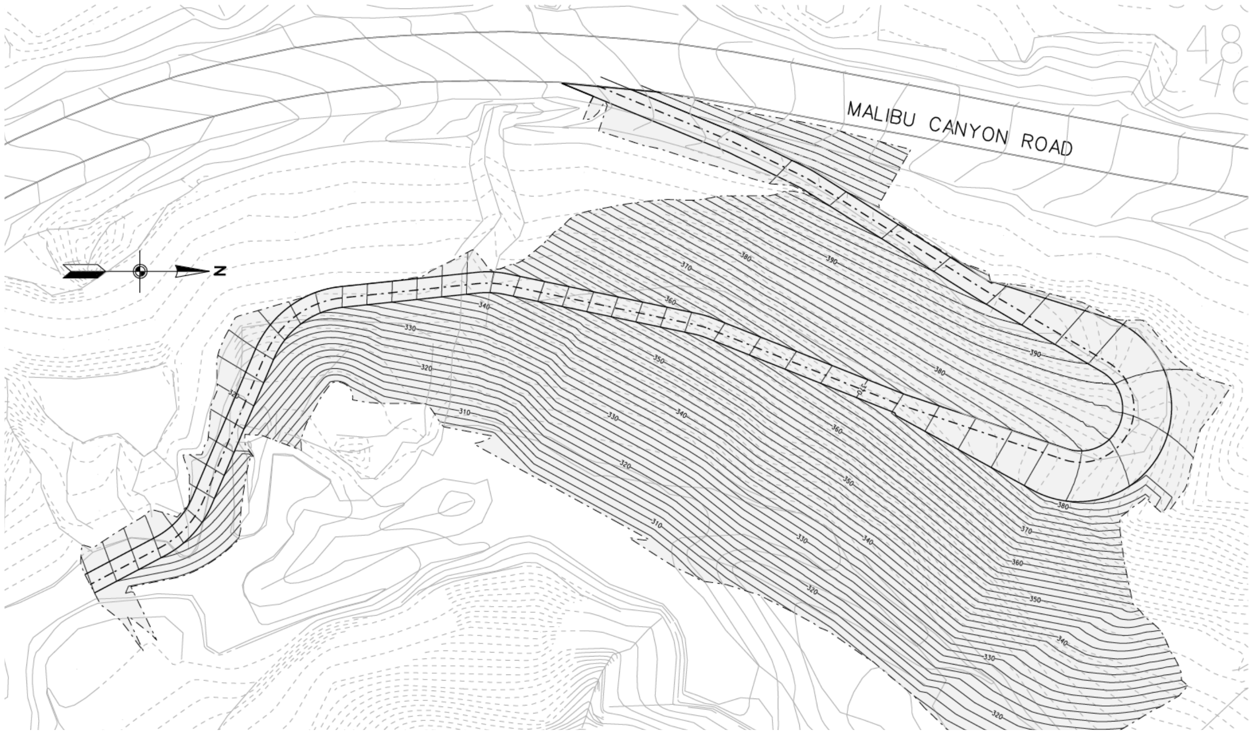


Figure 4.2-3 Southbound Ramp

4.2.2 Single Access Ramp

The southbound ramp could be eliminated if the either the top of the Northbound Ramp was widened or if a separate area north of the dam site was used for a truck turnaround.

Widen the top of the Northbound Ramp

For the Northbound ramp to allow truck traffic travel to the south, and minimize impacts to traffic along Malibu Canyon Road, the area at the top of the ramp would have to be widen to approximate 100 ft by 100 ft. This area would provide trucks enough room to make the 180 degree turn necessary to travel south from the ramp without blocking both lanes of traffic along Malibu Canyon Road.

This modification is not recommended because the fill slope for the ramp would extend to the adjacent canyon wall and create a barrier to the creek.

Trucks turn around north of the site

The single northbound ramp would accommodate southbound traffic if trucks used a separate turnaround area north of the project site. There are four sites that provide enough space for a 20 cy capacity truck to turn around.

Turnaround Area 1 - Area directly south of the tunnel

Area 1 is located about 0.30 mi north of the Northbound Ramp intersection with Malibu Canyon Road. This area was eliminated because both lanes of traffic would be blocked while the truck makes the turn from the northbound lane to the southbound lane. This would heavily impact traffic during construction hours, as outlined in the analysis of the **Figure 4.1-1** access ramp. There is also a safety concern due to the limited visibility of southbound traffic passing through the nearby tunnel.

Figure 4.2-4 Turnaround Area 1

Turnaround Area 2 - Existing pavement feature north of the tunnel

The entrance to Area 2 is located about approximately one mile north of the Northbound Ramp intersection with Malibu Canyon Road and can accommodate the truck traffic with some modifications. The northern driveway is not big enough to allow 20 cy capacity trucks to turn in but a new entrance could be created between the two existing driveways that could allow 20 y capacity trucks to turn around. Either a traffic signal would have to be installed to allow trucks to turn left into the turnaround or flagmen would need to be stationed in the area during construction hours to direct traffic while trucks are turning in and out of the area. Traffic would frequently be stopped in both directions along Malibu Canyon Road during construction hours. The area was rejected because it has the same traffic impacts that were outlined in the **Figure 4.1-1** access ramp analysis and it would add an additional 2 mi to the haul route for material going south.

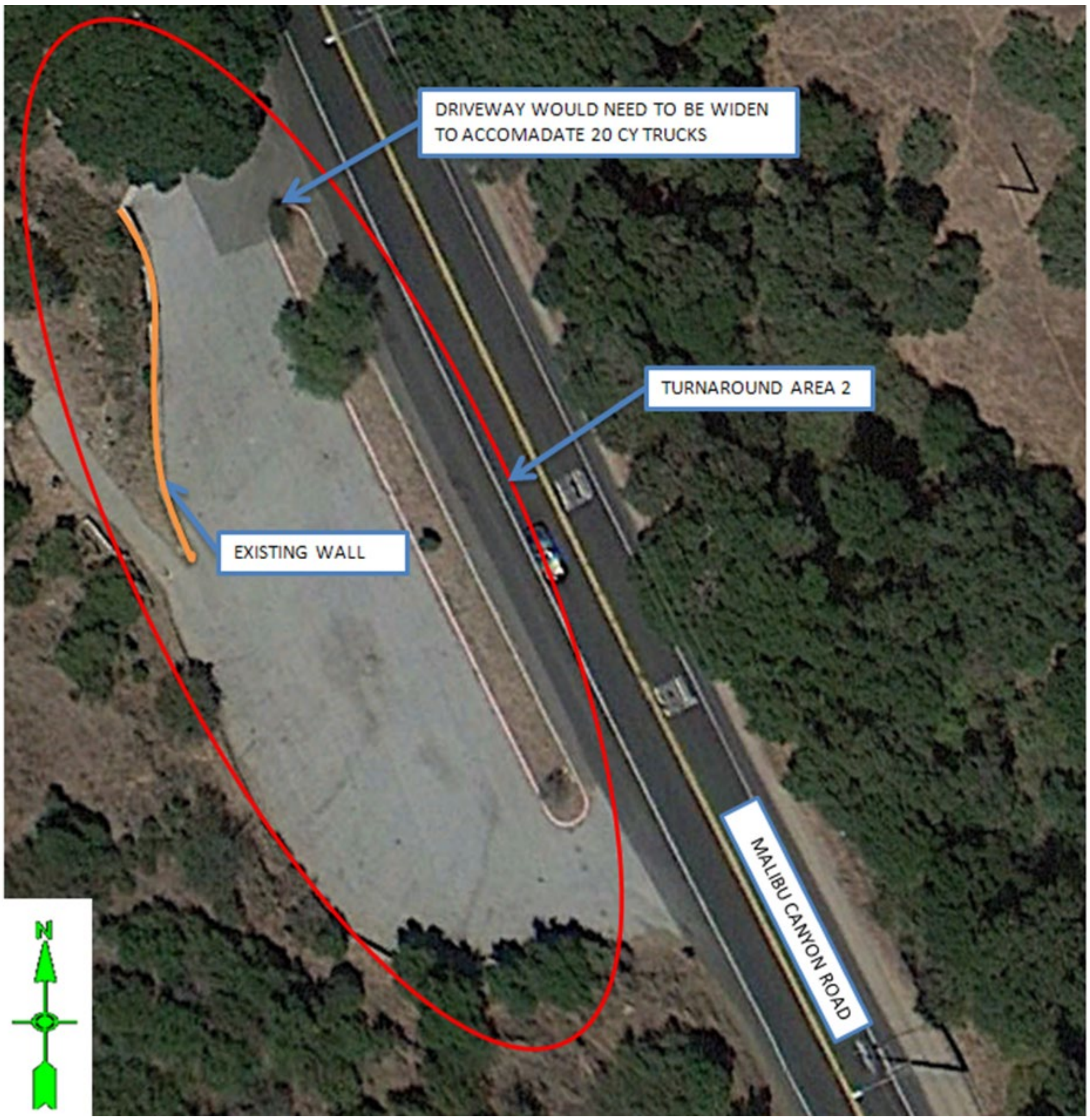


Figure 4.2-5 Turnaround Area 2

Turnaround Area 3 – Clearing to the east of Turnaround Area 2

The entrance to Area 3 is located 275 ft north of the entrance to Area 2, and approximately 1.05 mi north of the Northbound Ramp intersection with Malibu Canyon Road There is an existing intersection at the northern end of the site that can be modified to allow trucks to turn into the site and exit the site. There is plenty of room for multiple trucks to turn around at the same time and allow staging to get back onto Malibu Canyon Road. According to Google Maps this area is part of the Santa Monica Mountains National Recreation Area but the exact owner of the land is unknown at this time. This area was eliminated because it adds 2.1 mi to the haul route for material going south.

Figure 4.2-6 Turnaround Area 3

Turnaround Area 4 – Parking lot north of the tunnel

Area 4 is a parking lot located approximately 1.3 mi north of the Northbound Ramp intersection with Malibu Canyon Road tunnel. The first left turn onto Dorothy Road would need some modification to allow 20 cy capacity trucks through, but there is plenty of room in the parking lot for multiple trucks to pass through at the same time. There is already a traffic signal and a dedicated left turn lane along Las Virgenes Road to facilitate truck traffic and the parking lot area could also be used as a staging area or equipment storage site. This parking lot would have to be closed down during sediment removal operations. Traffic that would normally travel along Dorothy Road would have to be redirected so it does not interfere with truck traffic. The owner of this parking lot is unknown at this time. This site was eliminated because it added 2.6 mi the haul route for material going south.

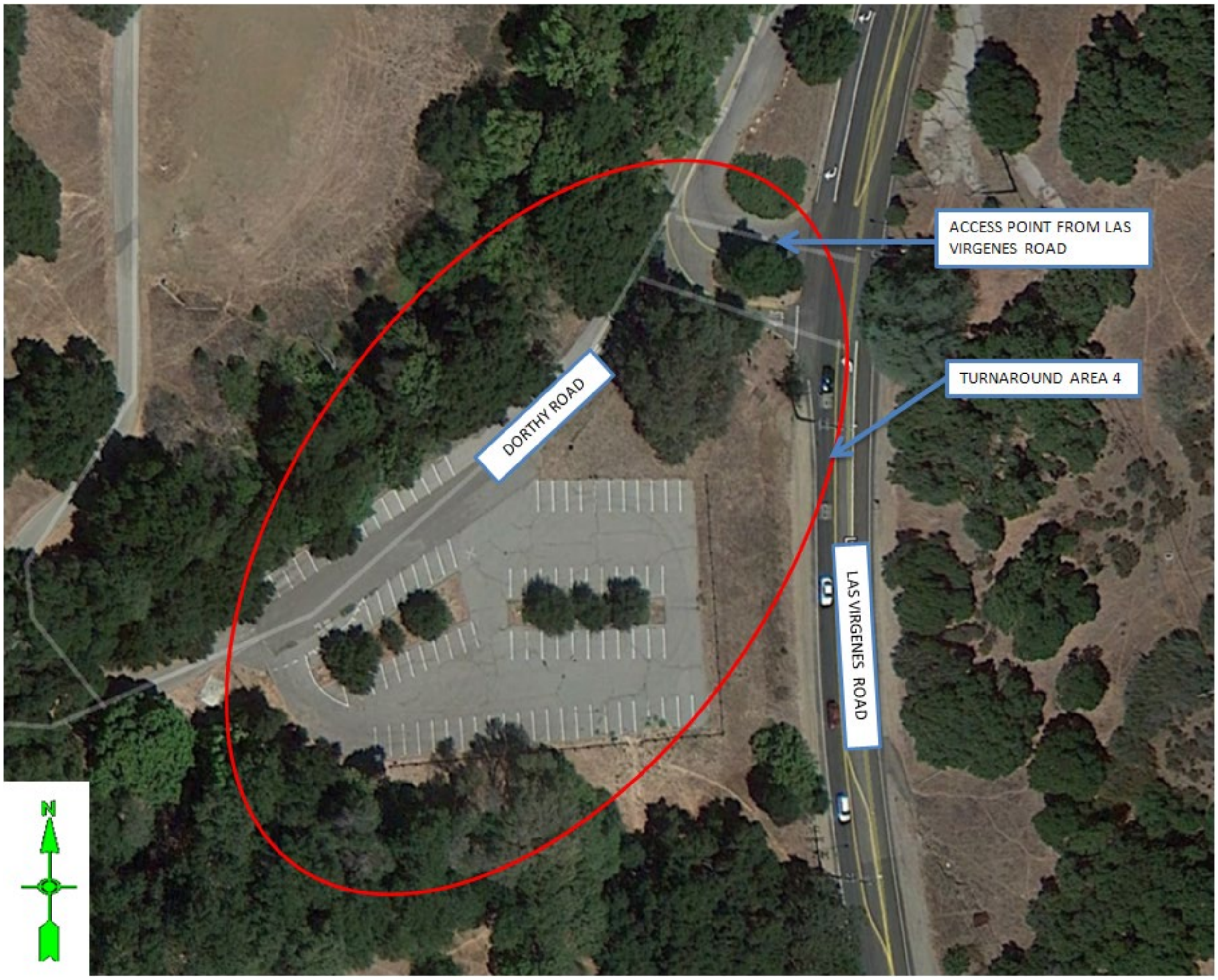


Figure 4.2-7 Turnaround Area 4

4.2.3 Ramp Construction and Quantities

The two ramps would have to be built in the follow stages:

Reconstruct the Existing Invert Ramp.

There is an existing ramp that at one time provided access to the creek invert, however the bottom half of the ramp has, over time, been washed away. The reconstructed ramp would only be 12 ft wide and have a maximum grade of 18%. Even though an 18% grade was determined to be too steep for the access ramp presented in **Figure 4.1-1**, it is an acceptable grade for the invert access ramp because the ramp was designed to allow construction equipment to get into the creek to begin clearing, dewatering, diverting the creek and excavation. The ramp would not accommodate full 20 cy capacity trucks. The ramp design is shown in **Figure 4.2-8**. This ramp would require 15,700 cy of imported material.

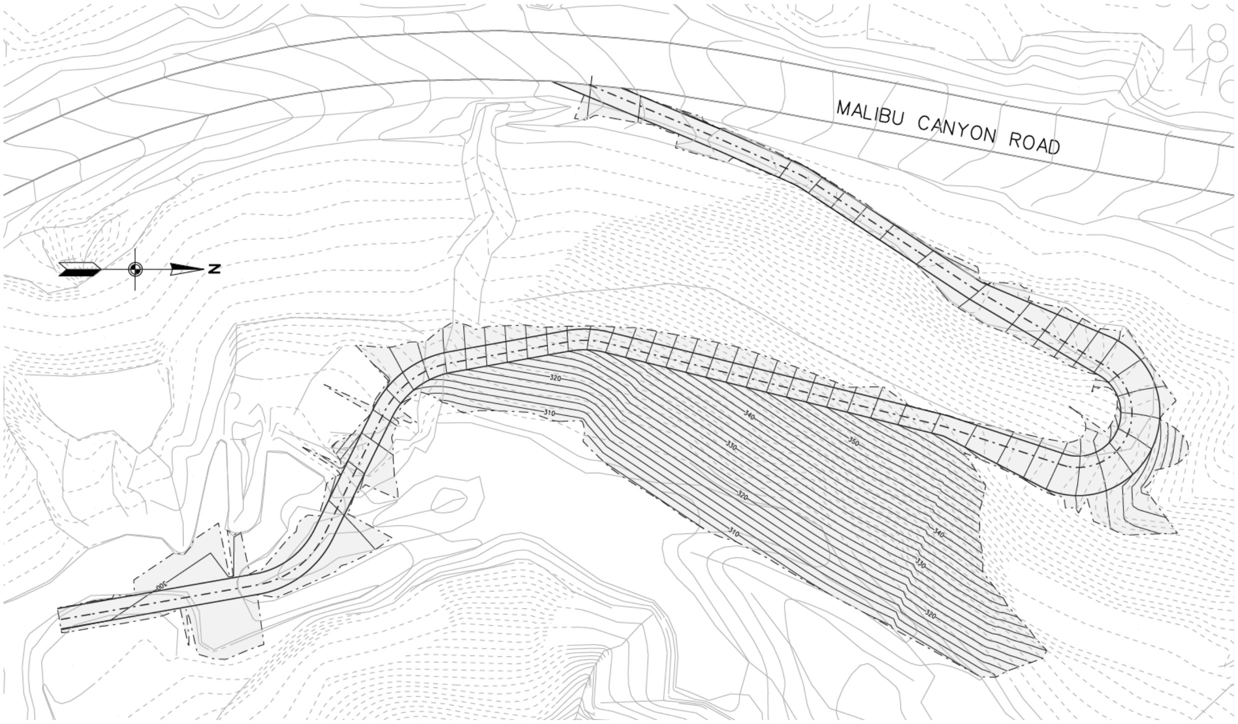


Figure 4.2-8 Reconstructed Invert Access Ramp

4.2.4 Construct Northbound Ramp

Once crews have access to the creek invert they would start building the Northbound Ramp. The Northbound Ramp would require 41,000 cy of material which would be excavated from the sediment behind the dam. Approximately 140 cy of 24 inch rock would be placed along the toe of the fill slope to protect the ramp against low flows during the winter. A 6 inch layer of crushed aggregate would be placed along the finished surface of the ramp to improve surface traction for fully loaded 20 cy capacity trucks. This ramp would be removed once the dam removal and sediment removal is completed.

4.2.5 Construct the Southbound Ramp

While crews are constructing the Northbound Ramp they could also be constructing the Southbound Ramp. To construct the Southbound Ramp, crews would have to place additional fill on top of the invert access ramp to reduce the grade of the ramp from 18% to 15% and widen the ramp to allow 20 cy capacity trucks to travel along the ramp. The Southbound Ramp would require an additional 55,000 cy of fill material which would be excavated from behind the dam. Approximately 160 cy of 24 inch rock would be placed along the toe of the fill slope to protect the ramp against low flows during the winter. 6 inch layer of crushed aggregate would be placed along the finished surface of the ramp to improve surface traction for fully loaded 20 cy capacity trucks. Once the sediment removal operation is complete, this ramp would be partially demolished the ramp that remained in place would be identical to the re-built invert access ramp.

4.2.6 Revised Site Access – Alternative 3

Alternative 3 would use the same ramp designs as Alternatives 2 and 4; however the Southbound Ramp would not need to be constructed. No sediment material would be removed from the site and placed at the beach in Alternative 3, but the concrete removed from the dam and spillway would still need to be disposed of at the Calabasas Landfill. Before the Northbound ramp can be constructed, the existing invert access ramp would have to be rebuilt, as outlined in the previous section, so crews can access the creek invert and construct the Northbound Ramp. The Northbound ramp would be removed once the dam removal and sediment removal is completed while the invert access ramp would remain in place to allow future invert access.

5.0 IMPOUNDED SEDIMENT REMOVAL

5.1 Impounded Sediment Area

The sediment area to be removed is confined to the narrow canyon along the creek bed from the dam to about 2,400 ft upstream of the dam. The top of the sediment is roughly 250 ft wide near the dam and maintains this width consistently to about 1,400 ft upstream of the dam, at which point it narrows to about 100 ft for the remainder 1,000 ft upstream. The approximate impounded sediment area is shown in yellow in **Figure 5.1-1**. **Figure 5.1-1** also depicts the Northbound Ramp in blue and the Southbound Ramp in green.

Figure 5.1-1 Sediment Impound Area

Heavy brush, large cobbles and ponding waters are prevalent throughout. The elevation of the top of the sediment basin area is roughly 294-300 ft above sea level. Along the sediment reach, the elevation of Malibu Canyon Road varies from elevation 536 at the overlook near the dam to elevation 372 near the upper most reach of the sediment. The existing canyon above the creek is extremely steep, sloping up to the Malibu Canyon Road at about 1V on 1H slope.

The proposed excavation would remove most of the sediment and restore the approximate gradient of the original channel invert. Some of the larger rocks in the creek would be utilized to stabilize the final invert. Excavation depth ranges from approximately 100 ft at the dam tapering to zero feet at the upstream end of the sediment area. Excavation of the sediment would expose rock canyon side-slopes. The width of the channel bottom upon final excavation would closely match the pre-dam conditions and be approximately 40 to 60 ft.

5.2 Estimation of Impounded Sediment Quantities

The total amount of sediment behind Rindge Dam is estimated at 780,000 cy. The analysis of the composition of the sediment is presented in the Appendix D. **Table 5.2-1** summarized the information from the Appendix D.

Table 5.2-1 Sediment Quantities

	Description	% Sand	%Silt and Clay	%Other	Approximate Volume (CY)
Unit 1	Sand, gravel, cobbles, and larger rock	28	2	70	210,000
Unit 2	Mainly silty sands with organic content; does contain silt layers, some gravel	73	22	5	340,000
Unit 3	Sandy silts, lean clays, and silts (all with organic content); does contain some silty sand layers	22	78	0	230,000

5.3 Sediment Removal Methods

The proposed three methods to remove the sediment from behind Rindge Dam are by slurry, by conveyor and by hauling the sediment away.

5.3.1 Removal of Sediment by Slurry

This measure consists of using a slurry pipe to transport sediment behind Rindge Dam to a combination of different disposal sites and transport of beach compatible material to Surfrider Beach. Water supply would be provided by Tapia Treatment Plant. A water supply pipeline would extend from the water treatment plant to the sediment removal area (SRA) a distance of about 10,510 ft. The length of the water supply pipeline would be slightly longer due to a 205 foot elevation difference between Tapia Treatment Plant and the sediment removal area. The length along the slurry line from the SRA to the beach nourishment site is approximately 16,790 ft (3.18 mi). The water supply line is assumed to be an 18" steel pipe. The slurry pipeline is an assumed to be 18" HDPE, SDR 11 pipe. It is assumed that we would need a maintenance road to maintain the slurry line. This road would be 20 ft wide, 15 ft for the road itself and an additional 5 ft for the slurry line, and would stretch for most of the 16,790 ft. The top of road alone would disturb approximately 335,800 sq ft of land. The maintenance road side slopes would disturb an additional 100,740 sq. ft. (Road 6 ft * 16,790 = 100,740 ft) for a total of 436,540 sq ft. This amounts to about 10 ac. The fill requirements for the slurry maintenance road can be around 61,000 cy based on the length, width and a height of about 4 to 4.5 ft.

If a maintenance road is needed for the water supply line, another 3.5 ac of land would be impacted. The water supply pipeline maintenance road can run along Malibu Canyon road from Tapia up to the tunnel entrance. From there the water supply line would jog in towards Malibu Creek. A maintenance road from Malibu Canyon Road servicing the supply line along the river would then be constructed up to the location of the temporary dike that would be used in the dredging and slurrying operations. The slurry line would operate immediately downstream of Rindge Dam and would run along Malibu Creek and along the east bank of Malibu Lagoon. The slurry line would then run parallel along the beach side of Pacific Coast Highway and to Surfrider Beach.

The maintenance road footprints would have to undergo clearing and grubbing. This material would have to be disposed of at a disposal site. Disposal of cleared vegetation from the water supply line maintenance road would be best done at DSA while disposal of cleared vegetation for the slurry line maintenance road would best be done at either DSB, DSC, or DSD. Fill material for these maintenance roads would come from selected Unit 1 material in the SRA. These maintenance roads would restrict the flow of the creek because the creek is very narrow. The maintenance roads cannot be cut into the creek walls because the canyon walls are steep. The canyon walls are about 1 on 1 slope. Cutting away from the slopes could result in the canyon walls becoming unstable. Access for the slurry maintenance road has not been determined. Access for the water supply maintenance road would come from the old access road to the SRA or from the proposed access road to the SRA.

Slurry Operations.

In order to operate the dredge and capture the water from the Tapia Treatment Plant a dike would need to be built. This dike would encompass the SRA and would be about 5,870 ft long, 20 ft high with a 15 foot crest width for equipment access. Therefore, the fill needed to construct this temporary dike would be about 239,150 cubic yards. This dike would probably be made up of select Unit 1 and Unit 2 materials. Unsuitable Unit 1 material for the construction of the dike would be hauled to a disposal site. The Unit 1 material is too large to be handled by the dredge and slurry line. The dike would then be filled with water from Tapia and the Unit 2 (sand) and Unit 3 (silt) soils would be dredged. The dredge used in this operation would be a small portable dredge that would operate 24 hours a day. The dredge would be connected with the slurry line in order to slurry the sand to Surfrider Beach. If the sand to silt ration is acceptable, the sand will be placed at the beach. Otherwise the sand/silt is either disposed of near shore by continuing the slurry line to the near shore or the silt is separated at the beach and the silt taken to a disposal site.

Sand to Beach (Slurry Method).

Assuming the sand-silt mix is compatible with the beach sand, it could be disposed right at Surfrider Beach. The amount of this sand could be about 250,000-323,000 cy of sand from the Unit 2 layer. There would need to be established limits to ensure that the sand is placed evenly throughout these limits and not just piled up into mounds. The slurry line would empty into a diffuser to place the sand on the beach. From there it would be placed by construction equipment to the design limits and grades.

If the sand-silt mix contained too much silt, the silt would have to be separated from the wet sand-silt mix by portable screening plant at Surfrider Beach. It can also be separated at one of the disposal sites, but it would be cost prohibitive compared to separation at the beach. Once enough silt is separated from the sand to meet beach requirements, the remaining silt would have to be disposed at one of the disposal sites.

Hauling

With the slurry alternative there would still be hauling by truck involved. Slurry lines cannot handle material that is larger than 10 mm. A large portion of the sediment, primarily the upper sediment layer, would have to be hauled to a disposal site. The amount of material available for slurry operations would be about 480,000 cubic yards. This is based on the amount of Unit 1 and Unit 2 sediments trapped behind the dam.

Slurry Line Constraints

Running a slurry line requires a maintenance road to maintain that slurry line. Running a slurry line along Malibu creek is difficult because the creek is very narrow. Placing a maintenance road in the creek would require heavy clearing of large boulders and shrubs. It would be very difficult to place a maintenance road adjacent to the creek. Access downstream of Rindge Dam for a maintenance road is difficult. The maintenance road would have a large footprint in the creek. Environmentally, the maintenance road would disturb the creek. Pipe and maintenance road would also have to be designed to handle flows in the creek.

The slurry method used a slurry pipe to transport the material behind the dam to various disposal sites and to the beach disposal site. The slurry line would be up to 3.18 mi long and run along the edge of Malibu Creek. The line could not run along Malibu Canyon Road due to space limitations. The water supply for the operation would come from the Tapia Treatment Plant. A 20 ft maintenance road would run parallel to both the slurry line and the water supply line from the treatment plant to allow easy access to both pipelines. The maintenance road for the slurry line would disturb about 10 ac of land while the maintenance road for the water supply line would disturb an additional 3.5 ac of land.

This method was not consider further due to the large foot print required for the slurry line and maintenance road.

5.3.2 Removal of Sediment by Conveyor.

The Conveyor belt alignment would run along the southern creek hillside between Malibu Creek and Malibu Canyon Road. It would then go through the Sheriff's Overlook area and up and over Malibu Canyon Road. The belt alignment would be about 2,064 ft long and would require the building of a 1,400 foot long, 15 foot wide maintenance road. The maintenance road and conveyor belts would move up the hillside at a 15% slope. The maintenance road would be used to maintain the belt and pick up sediment that falls off it. The belt would travel between the southern creek wall and Rindge Dam. Preliminary estimates of the fill requirements for this maintenance road indicate that about 85,000 cy would be needed to construct this maintenance road.

Conveyor belts can carry rocks up to 3 inches in diameter. Stones larger than 3 inches would have to be hauled out by truck. Stones larger than 3 inches can be disposed of at a disposal site, where they could be used to armor the disposal site, or left in place to armor the side slopes of the excavated SRA. In order to use the conveyor belt alternative, the contractor must have sufficient room to move and separate material at the SRA. Separation of material for the conveyor belt can be achieved by screening the material through a grizzly screen. Unit 1 material on the conveyor belt should be disposed of at DSA because it is the closest disposal site. If Unit 2 material is beach compatible and desired, it would either be hauled out of the SRA or conveyed to DSA via belts and hauled to the beach. If Unit 2 material is desired but requires processing, a portable screening plant would be installed in the SRA or in DSA.

Due to material size restrictions, the conveyor belt alternative will also require some hauling by dump trucks. The removal of the access ramp, cleared vegetation, and large size sediment in the SRA will have to be hauled to one of the disposal sites. The access ramp to the SRA is still necessary as well as the maintenance road for the conveyor belt. Its size and slopes would still be the same.

The conveyor alignment would run up to Sheriff's Overlook. If further separation of sediment is necessary, a portable screening plant can be installed at Sheriff's Overlook if space is limited in the SRA. A portable screening plant capable of separating sand from silt can be installed here if additional sand is desired from the Unit 3 region of the SRA. From this location beach compatible sand would be hauled through Malibu Canyon and non beach material disposed of at DSA by conveyor. Sheriff's Overlook would have to be cleared and grubbed. This area is about ½ acre in size. It would have to be regraded and the cut material be relocated to DSA. There are power poles along Malibu Canyon Road at this site. These power poles would be protected in place. The conveyor crossing Malibu Canyon Road would have to be a 20 ft minimum clearance from the surface of the road. If DSA is not available as a disposal site, the conveyor alignment would run along the east side of Malibu Canyon Road way to either DSB or DSC. Preliminary belt alignment to Disposal Sites A, B, and C are shown if **Figure 5.3-1** through **Figure 5.3-3**. Further analysis is necessary to determine the conveyor belt alignment. Bends in the conveyor alignment can be achieved by special sections of belt and/or the use of hoppers to redirect the conveyor alignment. Avoiding the power-poles along Malibu Canyon Road would be difficult. The road would probably have to be restriped and would require a reduction in traffic speed along Malibu Canyon Road for the duration of the construction.

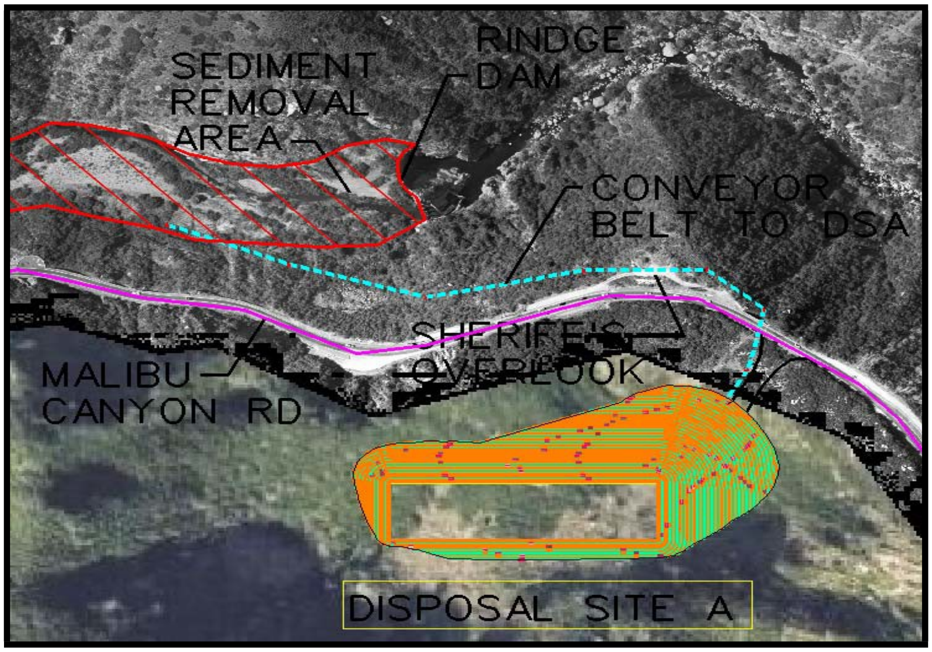


Figure 5.3-1 Conveyor Belt Alignment to Disposal Site A

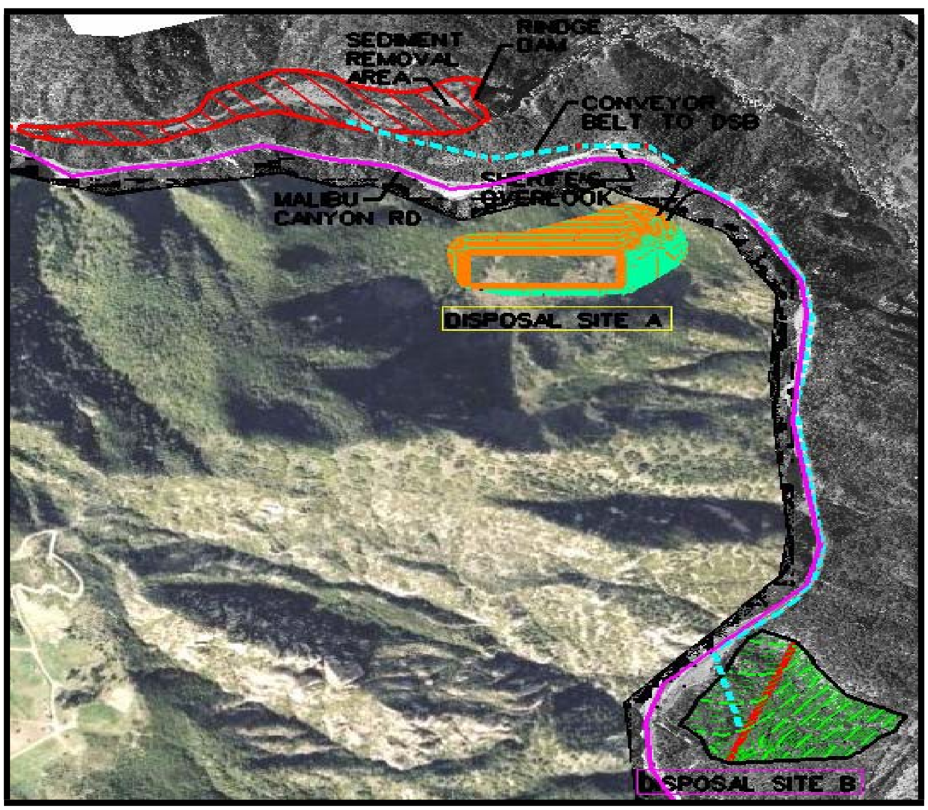


Figure 5.3-2 Conveyor Belt Alignment to Disposal Site B

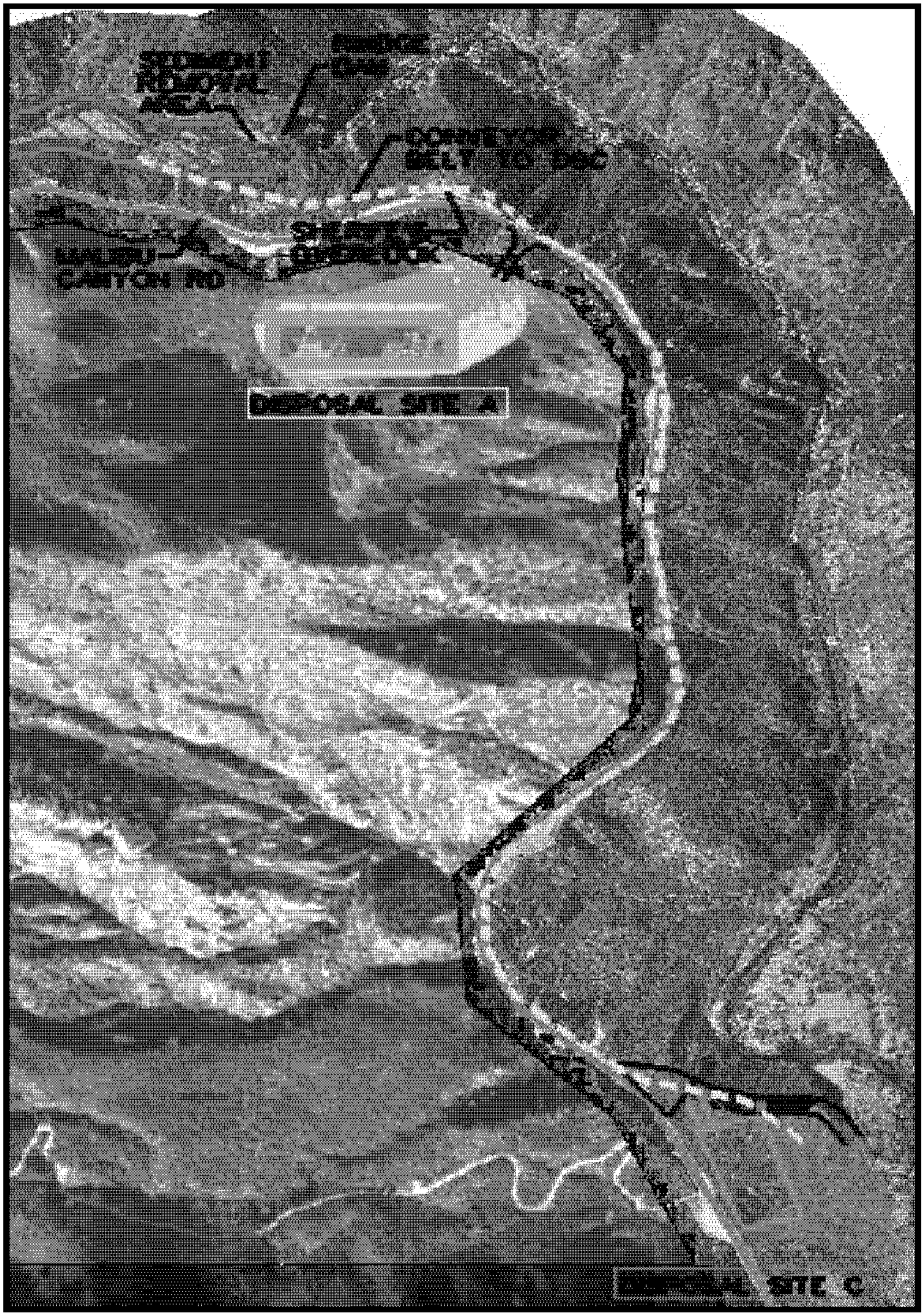


Figure 5.3-3 Conveyor Belt Alignment to Disposal Site C

Sand to Beach (Conveyor Method)

Conveying the sand to the beach via conveyor is not feasible due to the windiness of Malibu Canyon Road. To get sand to the beach, the conveyor would load the sand on to a truck at one of the disposal sites and it would be hauled to the beach.

The conveyor method used a conveyor belt to transport the excavated material up the canyon walls to disposal sites near the dam site. A maintenance road would have to be built along the conveyer to allow easy access during operation. This method was not consider further because the disposal sites close to the dam site were ultimately not viable options.

5.3.3 Removal of Sediment by Hauling

Hauling material to the disposal sites would be along Malibu Canyon Road. Malibu Canyon Road has two lanes, one for each way. There are several turnouts along this road between the SRA and DSA, DSB, and DSC. However, due to the narrowness of this road, it cannot support a dedicated lane for hauling operations. Hauling of disposal material would be done by 20 cubic yard dump trailers working 10 hour days, 5 days per week. Hauling operations are expected to occupy Malibu Canyon Road for a period of about 3.25 years with breaks during winter months. Loaded trucks traveling on Malibu Canyon Road cannot exceed 80,000 lbs. Flagmen would be needed at the ramp entrance to the SRA and at the entrance to the selected disposal site to control traffic along Malibu Canyon Road. Although Malibu Canyon Road is designed for standard truck traffic, road repairs are anticipated due to normal construction use. In addition, the daily dirt removal maintenance would be necessary on Malibu Canyon Road.

Sand to Beach (Haul Method)

The beach compatible Unit 2 layer in the SRA can be hauled to Surfrider Beach. For the location of Surfrider Beach see Figure 4. After negotiating the Unit 1 layer, the sand-silt mix can be hauled along Malibu Canyon Road and Pacific Coast Highway. The distance to Surfrider Beach is 27,000 ft (one way) and 55,200 ft (10.45 mi) roundtrip. The amount of sand-silt mix is about 323,000 cy.

It is preferable to dry out excavated sediment at the SRA if possible so that it will be easier to transport and will be able to be compacted more easily at the disposal site.

5.3.4 Conclusion

Since the slurry methods and conveyor belt method were not consider further, sediment removal methods are limited to mechanically excavating the material and hauling it out of the dam site or allowing the sediment material to wash downstream during anticipated high flow events. The alternatives use a different combination of these two methods.

- Alternative 2a and 2b - This alternative only uses mechanical methods to removal and dispose of all the sediment from behind the dam over a four year period based on assumption with no local or regional restrictions to daily truck operating hours. A detailed excavation plan can be found in the Appendix D.

- Alternative 3a and 3b – This alternative relies on anticipated natural sediment transport to remove all the sediment from behind the dam over a 50 year time period. Approximately 12,000 cy to 55,000 cy of sediment would be removed from the dam for each 5 ft vertical increment removed from the dam. This alternative eliminates the need for mechanical excavation, but it would require flood risk mitigation structures to be constructed downstream of the dam to mitigate the increase in flood risk generated by material being deposited along downstream portions of the creek.

- Alternative 4a and 4b – This alternative uses mechanical methods and anticipated natural sediment transport to remove all the sediment from behind the dam over a four year time period based on assumption with no local or regional restrictions to daily truck operating hours. The excavation plan would be similar to the detailed plan for Alternative 2a found in Appendix D, however Alternative 4a and 4b would require less truck trips than Alternative 2a, but it would require flood risk mitigation structures to be constructed downstream of the dam to mitigate the increase in flood risk generated by material being deposited along downstream portions of the creek.

6.0 IMPOUNDED SDIMENT DISPOSAL AND TEMPORARY STOCKPILE

Once the material is removed from behind the dam, beach compatible sand would be transported to pre selected beaches to be used for beach nourishment efforts. The remaining material would be taken off site to either be placed in a temporary stock pile or disposed of permanently.

6.1 Beach Nourishment

The project recommends the material would be transported to three separate beaches: Surfrider Beach, Topanga Beach, and Zuma Beach. An estimated total of 276,000 cy of material would be delivered to all the beaches with each beach getting the following volumes:

- Surfrider Beach – 32,400 cy
- Topanga Beach – 100,400 cy
- Zuma Beach – 143,200 cy

A more detailed analysis of the volumes of sediment delivered to each beach can be found in Appendix D. A discussion of how the material would be placed is covered in the Appendix O – Coastal Engineering.

6.2 Disposal Sites

Four neighboring sites were considered as permanent disposal sites for all the material excavated from behind the dam that was not delivered to beach sites. All disposal sites require an initial site preparation during and after clearing and grubbing operations and proof rolling to ensure a good foundation for the sediment. The disposal site would be proof rolled to ensure a good foundation for the sediment. This would ensure that clearing equipment can safely clear the disposal site of vegetation and also to prepare the site to receive material from the SRA.

6.2.1 Disposal Site A (DSA).

Disposal Site A is located south east of Rindge Dam and across Malibu Canyon Road. For the location and elevation of DSA see Figures 8 and 9. The distance from the center of SRA to DSA is about 6,300 ft (one way) and 12,600 ft (round trip) for hauling operations. The distance from the center of SRA to DSA is short but impractical for slurry operations because slurry cannot be pumped up to the elevations in DSA. In order to access DSA, a 350 ft road would have to be built from Malibu Canyon Road to DSA. This 350 ft pioneer road is included in one way and roundtrip distances from the SRA to the DSA. This pioneer road would be 50 ft wide. Permanent disposal of material at DSA is ideal because it is the closest disposal site to the SRA. The disposal site would have held 780,000 cy of soil at a 2H:1V slope as show in **Figure 6.2-1** and **Figure 6.2-2** The top elevation of DSA is at about 680 ft and the toe elevation at 520 ft. The top of the disposal material at DSA would form a 695' by 180' rectangle.

To make DSA suitable for disposal, the area would have to be cleared of trees and brush. This would then be moved to one side while non organic fill is used to build up the disposal site. The organic material cleared would then be placed near the upper layer or somewhere within an embankment zone such that it would not cause detrimental effects to the embankment. The upper soil removed just after clearing could be used to revegetate the disposal site slopes if it is required.

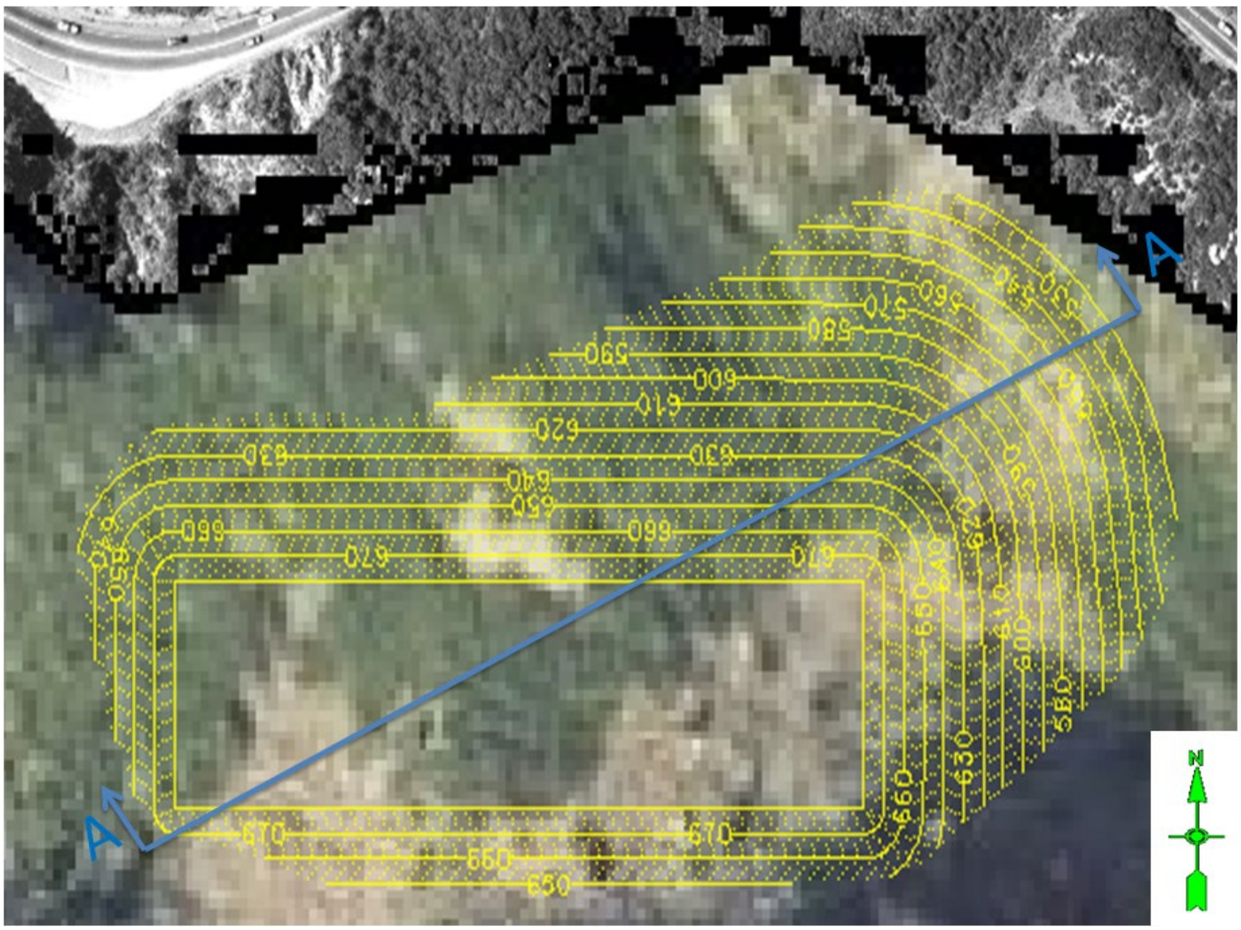


Figure 6.2-1 Disposal Site A - Plan View

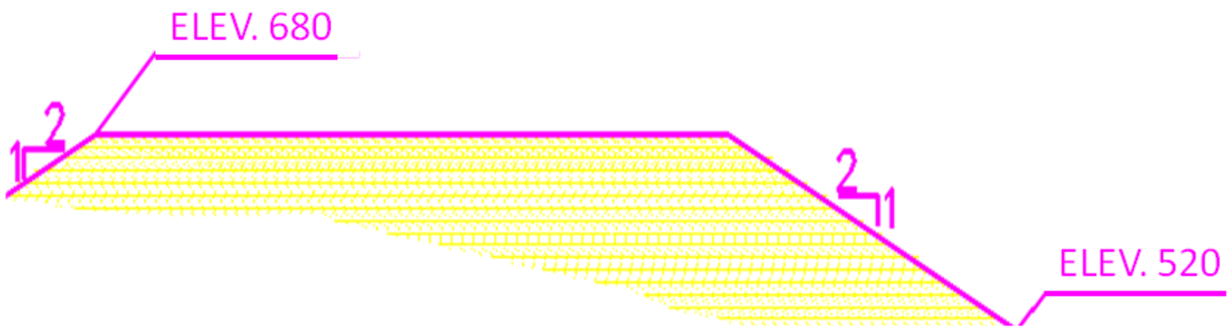


Figure 6.2-2 Disposal Site A - Section A-A

6.2.2 Disposal Site B (DSB).

Disposal site B is located between Malibu Canyon Drive and Malibu Creek about 5,000 ft south of Disposal Site A. The distance from center of SRA to DSB is about 10,600 ft (one-way) and 21,200 ft (round trip) for hauling operations. The distance from the center of SRA to DSB via slurry is 6,400 ft. The access road at DSB for hauling sediment would be longer due to haul trucks needing to access the bottom of the disposal site. This road will be about 1,650 ft long, 60' wide, with a 12-15% maximum grade. Excessive cut and fill for this road is not anticipated. The disposal site would have accommodated about 870,000 cy at about 3H:1V slope as show in **Figure 6.2-3** and **Figure 6.2-4**. The top elevation of DSB is 335 ft and the bottom elevation is at 85 ft. The top of DSB would be below the elevation of Malibu Canyon Road along this region. DSB is not expected to need scour protection because it is outside the existing 100 yr flood plain. To prepare DSB for sediment disposal, DSB would have to be cleared of trees and brush. The cleared vegetation would be pushed to one side to allow select Unit 1 fill to be placed to build up the disposal site. The sideslopes of DSB should be hydroseeded for aesthetics and slope stability. Top soil removed in during the clearing of DSB could be used as landscaping soil for the disposal site.

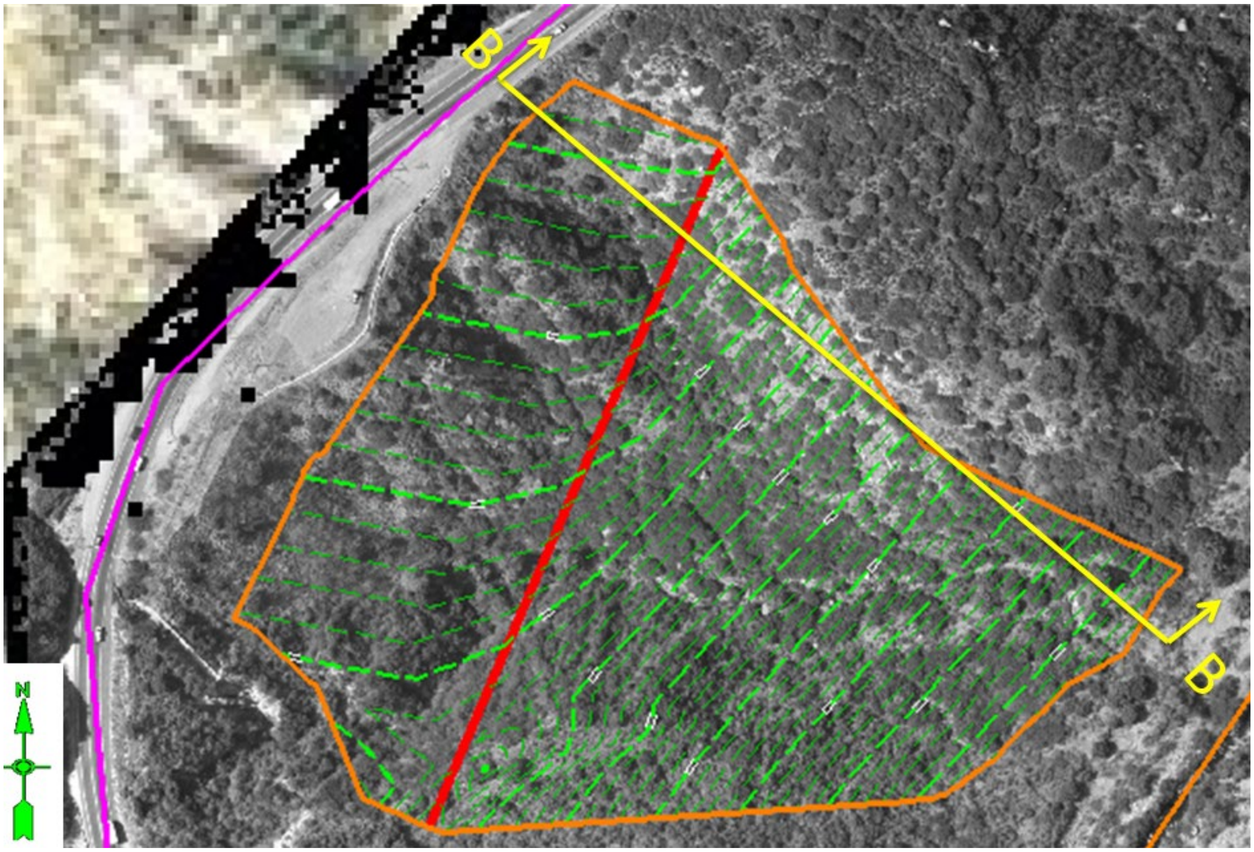


Figure 6.2-3 Disposal Site B - Plan View

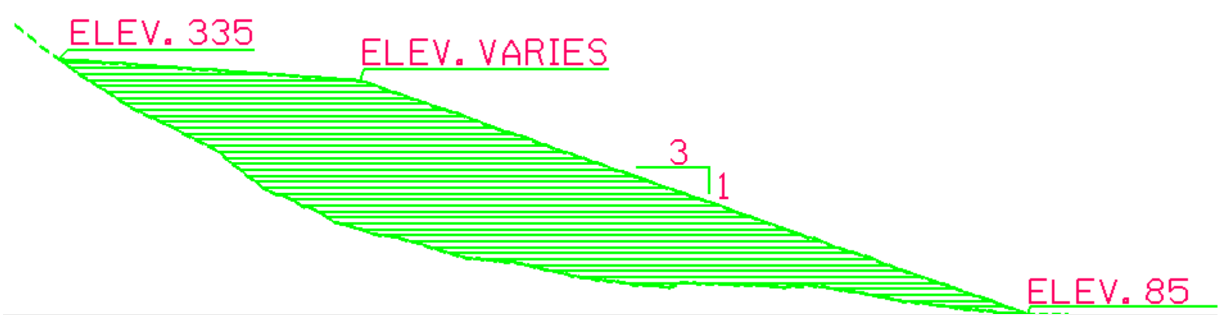


Figure 6.2-4 Disposal Site B- Section B-B

6.2.3 Disposal Site C (DSC).

Disposal site C is approximately 1,800 ft south of Disposal Site B and adjacent to Malibu Canyon Road. The distance from center of SRA to DSC is 12,150 ft (one-way) and 24,300 ft (round trip) for hauling operations. The distance from the center of SRA to DSC via slurry is 7,100 ft. In order to access Disposal Site C a 1,080 ft long, 60 ft wide, 15% slope access road would need to be built. Disposal site C would have accommodated up to 725,000 cy of disposal material at a 1 on 2 slope as show in **Figure 6.2-5** and **Figure 6.2-6**. Slopes that are 3H:1V at this site are not possible because the toe on this slope would extend into Malibu Creek. The toe of DSC will have to include erosion protection adjacent to Malibu Creek for this volume of deposition. Protection of the DSC toe would consist of using the Unit 1 layer in the SRA. The existing drainage utilities at DSC would need to be identified and analyzed and mitigated. The USGS stream gauge station located next to DSC would have to be protected in place. DSC would probably need to be revegetated to strengthen the slopes because these slopes would be 2H:1V.

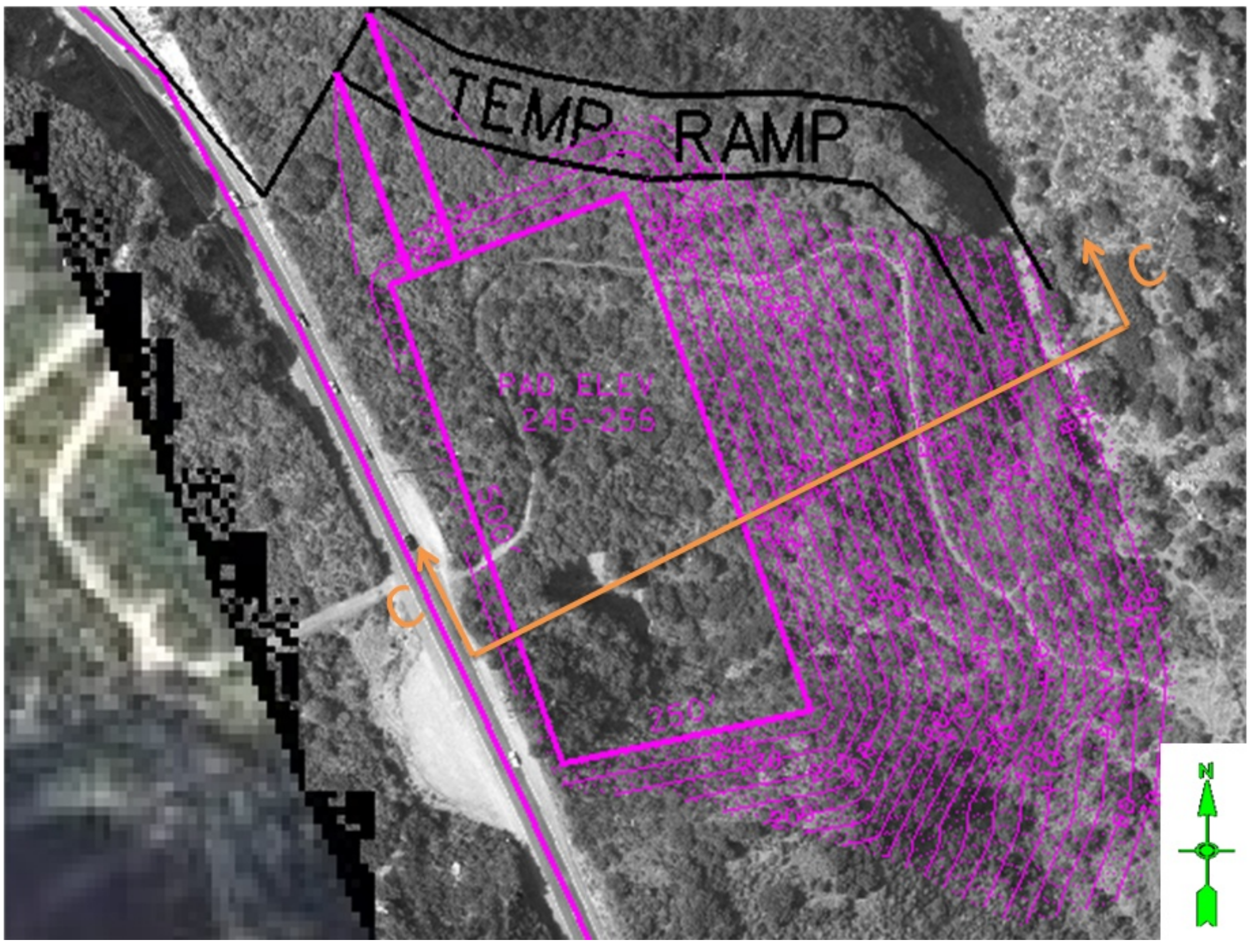


Figure 6.2-5 Disposal Site C - Plan View

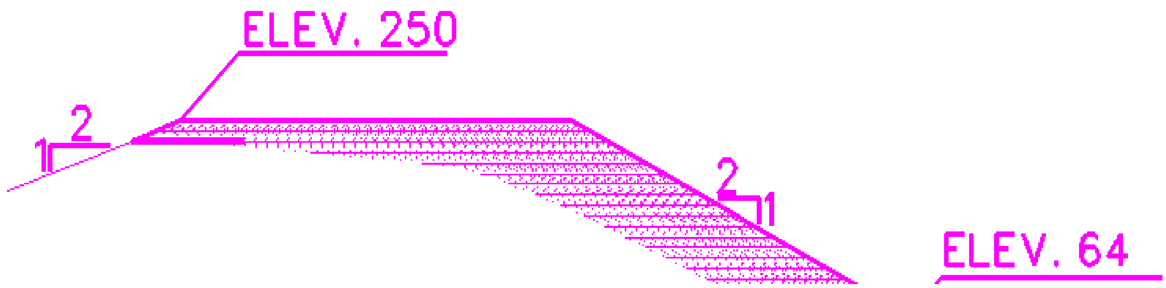


Figure 6.2-6 Disposal Site C - Section C-C

6.2.4 Disposal Site D (DSD).

Disposal site D is located between Disposal Sites B and C on the east side of Malibu Creek. In order to access DSD, a ramp crossing Malibu Creek would be built. The crossing of the creek would be an additional 600 ft extension of the access ramp for DSC. DSD can accommodate up to 870,000 cubic yards of disposal material with a 2H:1V side slope as shown in **Figure 6.2-7** and **Figure 6.2-8**. The distance from center of SRA to DSD is 12,750 ft (one-way) and 24,900 ft (round trip) for hauling operations. The distance from the center of SRA to DSD via slurry is 6,400 ft. DSD is located within the 25 yr floodplain and would cause the creek to become narrow. The use of DSD for sediment disposal is not practical due to extensive erosion protection and possible adverse environmental impacts and was eliminated from further consideration.

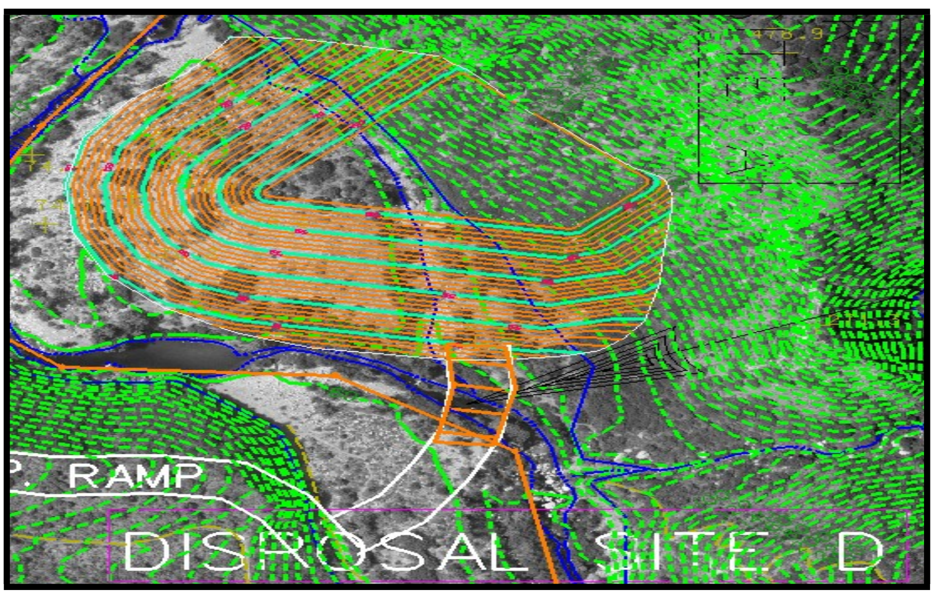


Figure 6.2-7 Disposal Site D Plan

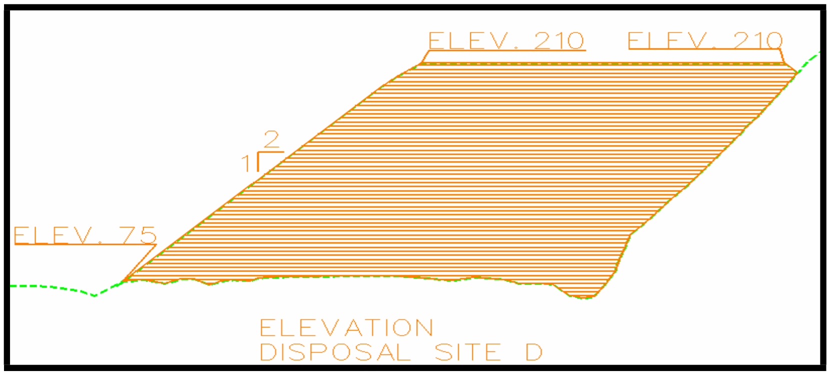


Figure 6.2-8 Disposal Site D Elevation

6.2.5 Conclusion

Disposal sites A, B and C were ultimately eliminated due to concerns over landslides as detailed in Appendix D. Disposal Site D was also eliminated due to the need for extensive erosion protection.

6.3 Disposal Sites and Temporary Stockpile Sites

The material excavated from behind Rindge Dam can be placed in one of four categories: material used for beach nourishment, material used for ramp construction, unused gravel rich material and unused silt and clay rich material. **Table 6.3-1** summarized the quantities of each category.

Table 6.3-1 Excavated Material Summary

	Estimated Volume
Material for Beach Nourishment	273,000 CY
Material Used for Ramp Construction	90,000 CY
Gravel Rich Material	187,000 CY
Silt and Clay Rich Material	230,000 CY
Total Material	780,000 CY

The material used for the ramp construction would either be left in place after construction or disposed of at the Calabasas Landfill. No beneficial use could be identified for the silt and clay rich material. Once excavated from behind the dam, this silt and clay rich material would have to be disposed of at the Calabasas Landfill. The gravel rich material could potentially be used in construction projects; however no groups have agreed to accept the material during the sediment removal operations. Once excavated from behind Rindge dam, this material would not be immediately disposed of, instead, this material would be temporarily stockpiled outside of the sediment impound site.

Ten different sites were investigated as potential temporary stockpile sites. **Figure 6.3-1** shows the ten sites that were located approximately 3 mi north of Rindge Dam.

Figure 6.3-1 Alternate Temporary Stockpile Sites North of Rindge Dam

When evaluating each site (**Figure 6.3-1**), the following assumptions were made about the layout of the stockpile for the gravel rich material:

- Material would be placed in cone shaped piles with 2H:1V side slopes
- Each pile would be 20 ft tall and contain 1,228 cy of material
- Each pile required 5,024 sq of land (0.11 Acres)
- 187,000 y of material would generate 152 piles
- 152 piles would require up to 16.7 ac of land

Each of the sites show in **Figure 6.3-1** were ultimately eliminated for further consideration for the following reasons:

- Temporary Stockpile Site E (**Figure 6.3-2**) was only 5 ac and was determined to be too small to use as a stockpile site.
- Temporary Stockpile Site F (**Figure 6.3-3**) was owned by State Parks and they felt any stockpile would interfere with their efforts to use the site for future mitigation efforts.
- Temporary Stockpile Sites G and H (**Figure 6.3-4**) were rejected because the land owners would not agree to let their property be used as a temporary stockpile site.

- Temporary Stockpile Site J (**Figure 6.3-5)**, was rejected because the land owner would not agree to let their property be used as a temporary stockpile site.

- Temporary Stockpile Sites K, L and M **(Figure 6.3-6**) were eliminated from further consideration because the land owners would not agree to let their property be used as a temporary stockpile site. The Mulholland Turnout site was only 1.2 ac and was determined to be too small to use as a stockpile site.

- The KGR Camp Site (**Figure 6.3-7**) was selected by State Parks and was 3.4 ac; however, this site was eliminated from further consideration due to the hilly topography of the site limiting the amount of material that could be stockpiled at the site.

With no other options for temporary stock piles sites, it was determined that all the material removed from the dam that is not suitable beach nourishment would have to be hauled to the Calabasas Landfill. The gravel rich material that may have additional beneficial uses would be stock piled separately at the landfill while material that cannot be reused, such as the silts and the fines, would be disposed of at the landfill.

Figure 6.3-2 Temporary Stockpile Site E

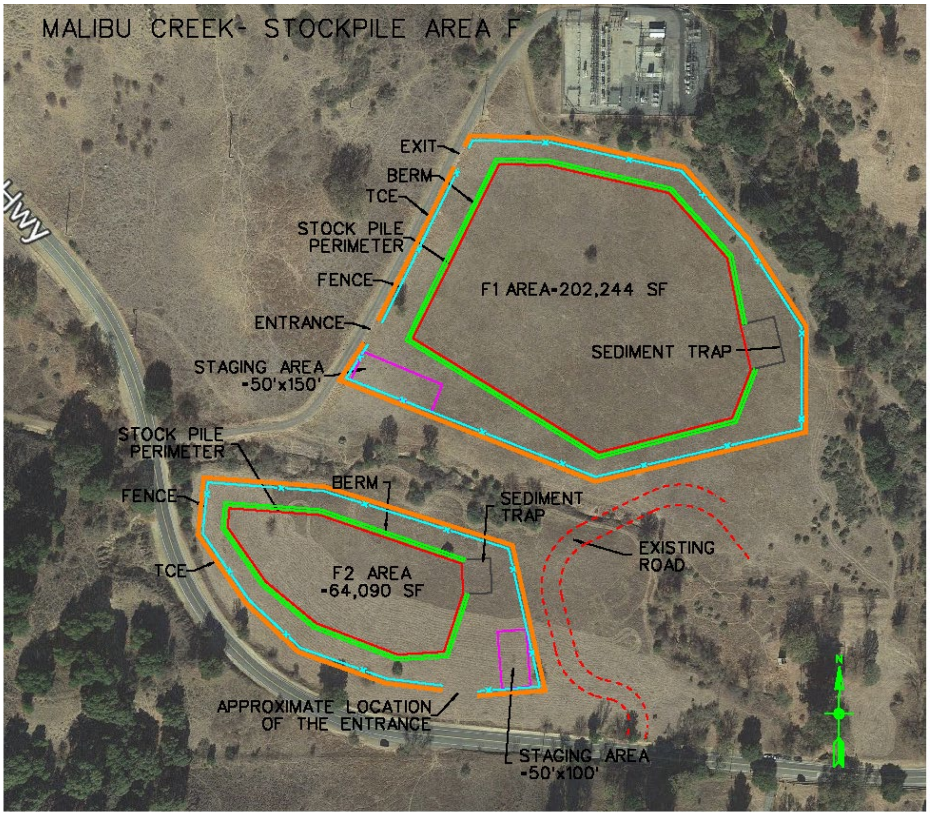


Figure 6.3-3 Temporary Stockpile Site F

Figure 6.3-4 Temporary Stockpile Sites G and H

Figure 6.3-5 Temporary Stockpile Site J

Figure 6.3-6 Temporary Stockpile Sites K, L, M and Mulholland Road Turnout

Figure 6.3-7 Temporary Stockpile KGR Camp Site

6.4 Proposed Haul Routes

All haul routes limit truck loading to 80,000 tons and even though the proposed routes are over roads designed for regular truck traffic, some road repair would be necessary when the project is complete due to the large amount of truck traffic wear generated by the project.

6.4.1 Haul Route to the Calabasis Landfill, Upland Storage Site F, and Malibu Pier Parking Lot

Calabasas Landfill - From the dam site, trucks would travel north 1.9 mi along Malibu Canyon Road and continue onto Las Virgenes Road. Malibu Canyon Road and Las Virgenes Road are two lane roads that pass through the hills and there is not enough room along either road for a dedicated haul lane. The trucks would then travel 3.9 mi north along Las Virgenes Road then turn onto Los Hills Road which is a four lane road that serves numerous residential communities. Once the trucks turn onto Lost Hills Road, they would travel 1.9 mi north to the Calabasas Landfill. The total distance, one way, is 7.1 mi.

Upland Storage Site F – From the dam site, trucks would travel north 1.9 mi along Malibu Canyon Road and continue onto Las Virgenes Road. The trucks would then travel approximately 2 mi north along Las Virgenes Road then turn onto Los Hills Road which is a four lane road that serves numerous residential communities. The total distance, one way, is approximately 4 mi.

Malibu Pier - From the dam site, trucks would travel south approximately 2 mi along Malibu Canyon Road and continue onto Pacific Coast Hwy. The trucks would then travel approximately 2 mi east along Pacific Coast Hwy. The total distance, one way, is approximately 4 mi.

Flagmen would need to be present at any points of entry along Malibu Canyon Road to control traffic while trucks enter and leave the dam site. Both lanes of traffic would have to be interrupted while trucks leave the site from the Southbound Ramp or enter the dam site from the Northbound Ramp. The proposed excavation operations would generate a substantial amount of truck traffic and preliminary estimates show a truck would leave the site approximately every 10 to 15 minutes. Additional flagmen would be necessary at each beach disposal site to direct truck traffic entering and leaving the sites.

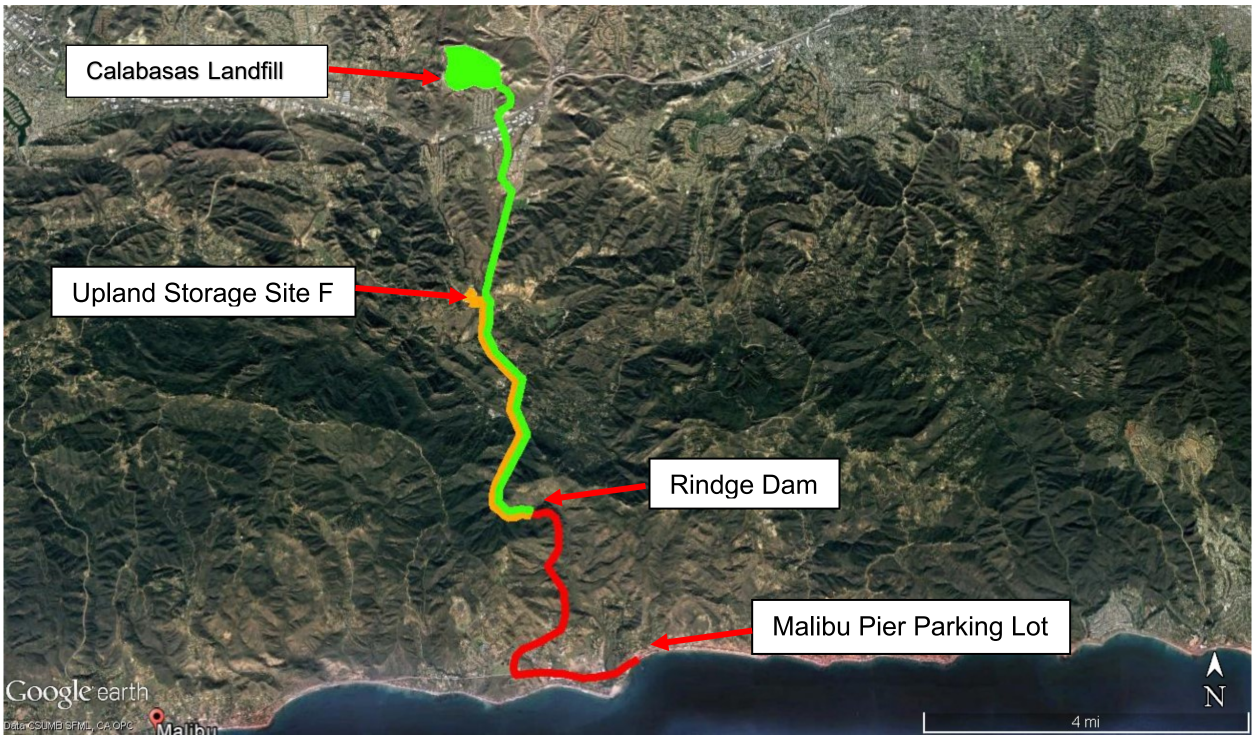


Figure 6.4-1 Haul Route to Calabasas Landfill, Upland Storage site F, and Malibu Pier

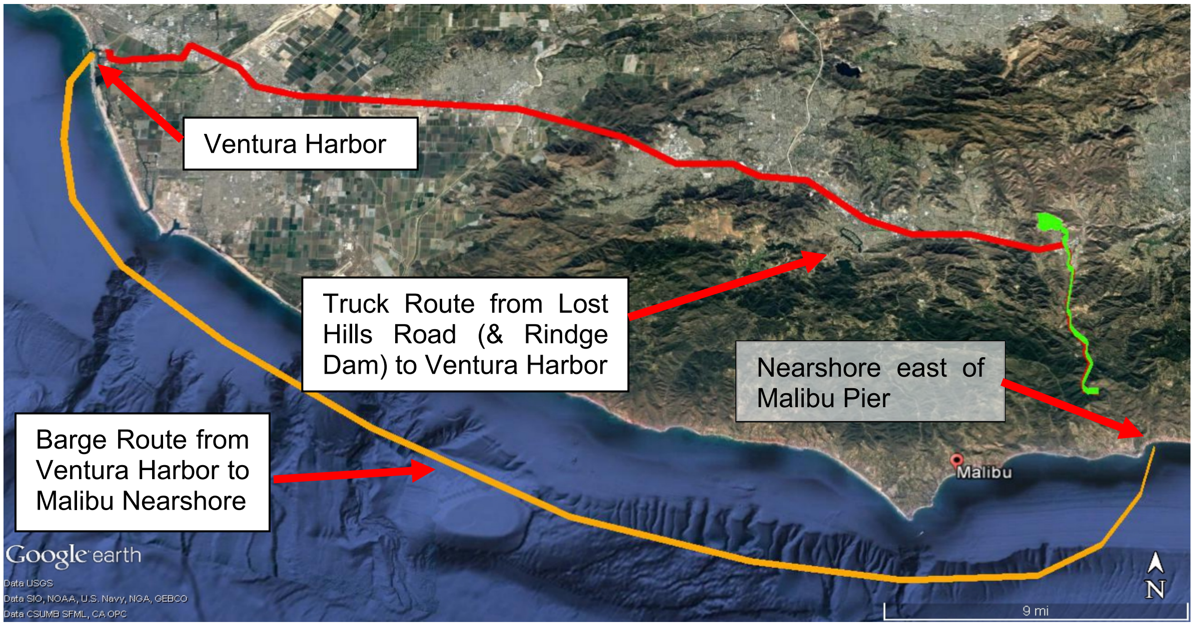


Figure 6.4-2 Ventura Harbor Haul Route to Barge

7.0 DEWATERING/DIVERSION AND CONTROL OF WATER

The stream would have to be diverted in Alternatives 2, 3 and 4. A 5 ft high coffer dam would be built upstream of the Southbound Ramp and it would direct water into a series of culverts. The culverts would convey the water across the impounded sediment to the dam. During the first year of the project, the culverts would discharge the water over the spillway. For project years two through five, the culverts would discharge the water over the top of the dam. The culverts would have to be periodically moved during sediment removal to make sure they do not get in the way of excavation equipment. The coffer dam, and the culverts, would be removed at the end of each sediment removal period to prevent them getting damaged by winter flows.

Dewatering would be necessary for Alternatives 2 and 4. The dewatering wells would be installed at the start of the project and left in place until the sediment has been completely removed. The lengths of the wells would be adjusted as the sediment is removed from behind the dam. If the wells were damaged during the rainy season they would have to be repaired prior to the start of the following year's excavation operations. More information about dewatering operations can be found in the Appendix D.

8.0 DOWNSTREAM FLOOD MITIGATION

8.1 ALTERNATIVES 3 AND 4

Alternatives 3a, 3b, 4a and 4b would use anticipated natural sediment transport to remove the impounded sediment from behind the dam. It is estimated that Alternatives 4a and 4b would release between 15,000 and 55,000 cy of sediment a year over 5 years while Alternatives 3a and 3b would use natural transport to remove all the sediment from behind the dam over 50 years. The sediment released from behind the dam would be deposited along the downstream portions of Malibu Canyon and eventually increase the flood risk for properties between Cross Creek Road and Pacific Coast Highway by increasing the water surface elevation of the floods. To mitigate for this flood risk, approximate 3,100 ft of flood wall would have to be built along the west bank and approximately 2,700 ft along the east banks of Malibu Creek for a total of approximately 5,800 ft of flood wall. **Figure 8.1-2** illustrates a preliminary alignment for the flood walls.

Alternatives 4a and 4b would require a 5 ft flood wall while Alternative 3a and 3b would require a 10 ft flood wall along each bank. A preliminary design concept for the flood wall was assumed for inclusion in the study to provide a cost estimate based on parametric design of similar applications of sheet pile flood walls from Project Delivery Team discussions,. It should be noted that no geotechnical investigation or assessment have been performed concerning the site for the proposed flood walls. If flood walls become a feature in the final alternative, appropriate geotechnical investigations and structural design would be necessary. See additional discussion in Appendix D, Section 5.7. For cost estimating purposes, the flood walls was assumed to be an I-Wall design with a reinforced concrete flood wall built above ground and sheet pile extended below the existing creek invert. The concrete flood wall would not require a footing because it would attach directly to the sheet pile concrete cap.

A typical cross-sectional detail of a flood wall is provided on the next page (**Figure 8.1-1**). Since this is only a conceptual design used for the evaluation of alternatives, additional field data and detailed design would have to conducted during PED to determine the actual dimensions of the flood walls, and only if the selected plan required flood walls were required for the selected plan.

A flood wall is the preferred flood mitigation measure because there is not enough space available for a levee in areas that protect the residential and commercial developments.

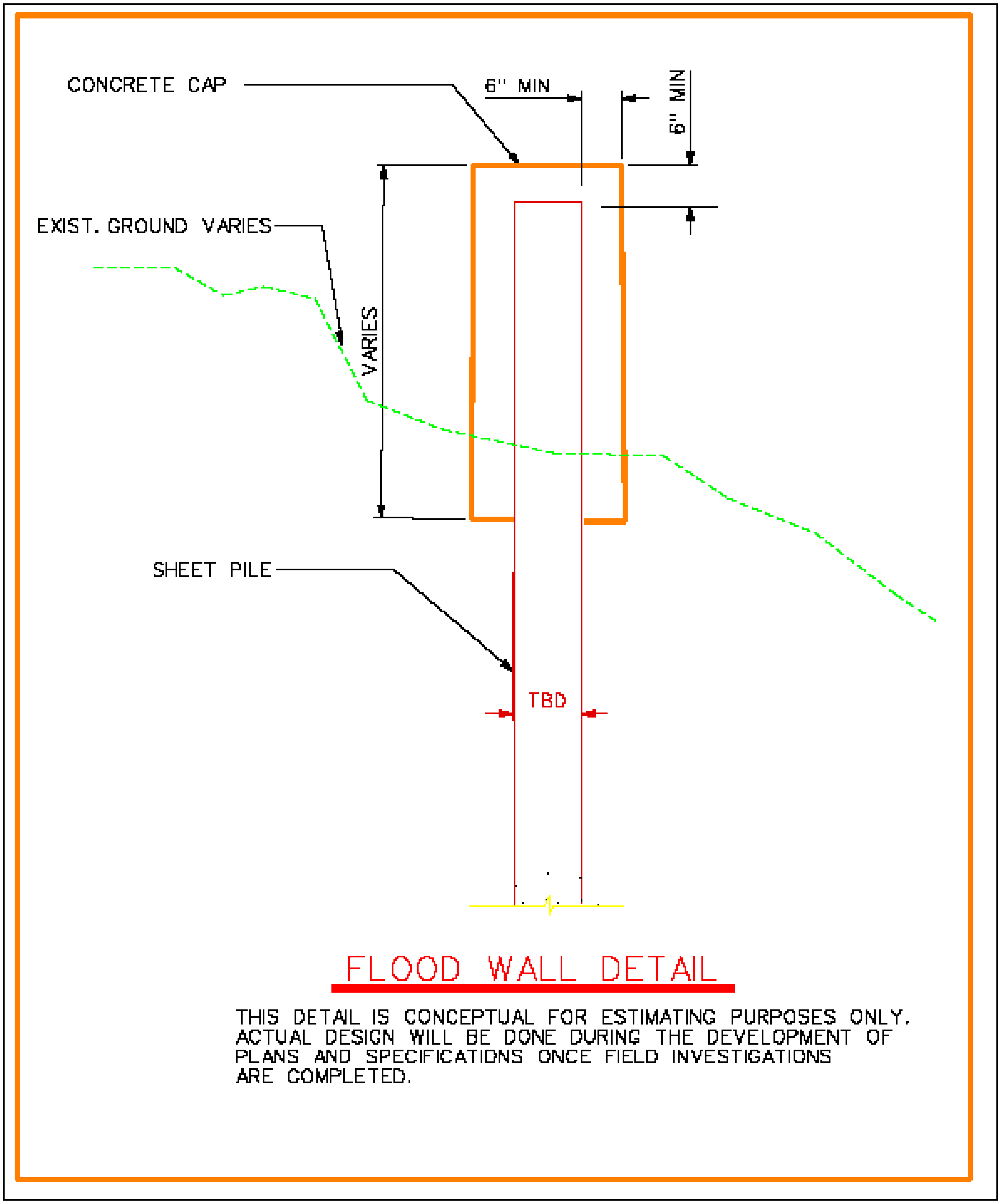


Figure 8.1-1 Flood Wall Tyoical Cross Section (Conceptual)

Figure 8.1-2 Downstream Flood Mitigation Alignment

9.0 FISHWAY DESIGNS

The California Salmonid Stream Habitat Restoration Manual was used to investigate different fishway designs for this study. The fishway ladders that were examined include the Alaskan Steep Pass, Denil, and the Step and Pool fishway. The Alaskan Steep Pass fishway has been used effectively to pass steelhead salmon but the entrance to fishway needs to be close to the obstruction with as few changes in direction as possible. Because of the difficulty in achieving this scenario, this fishway was not looked at further. The Denil fishway is easily blocked by debris and requires daily maintenance during the fish migration season so it too was not looked at further. The Step and Pool fishway has shown to be successful for this environment and as such, is the recommended plan.

9.1 <u>Step and Pool Fishway.</u>

The current alignment for the step and pool fishway goes along the southern bank of Malibu Creek. For the fishway alignment, see **Figure 9.1-1**.

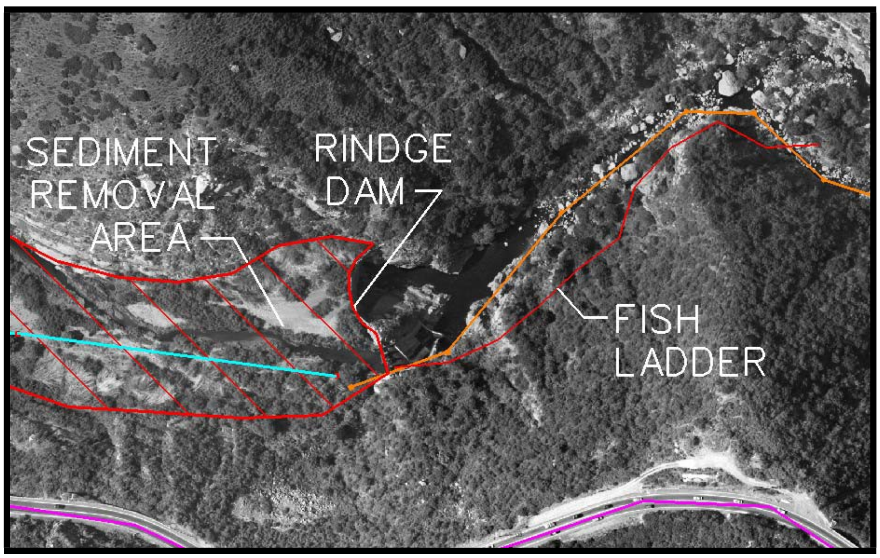


Figure 9.1-1 Fish Ladder Alignment

For the general plan and elevation see **Figure 9.1-2**. The design of the step and pool fishway requires a 1 foot drop every 10 ft. Because of the 100 foot difference in elevation from the top of the dam to the bottom of the dam, this fishway is very long. The fishway alignment shown in **Figure 9.1-1** extends well downstream of Rindge Dam. The fishway would then be brought into Malibu Creek. One requirement for a successful fishway is that the entrance to the fishway needs to be close to the vertical obstruction (Rindge Dam). Installing this fishway along the canyon walls would require many piers to hold the fishway in place. Another possible fishway alignment would cross back and forth across the downstream face of the dam. This would require massive piers to hold the fishway in place. Placing the fishway in such a way would make the entrance to the fishway closer to dam but would be difficult to construct because the downstream face of the dam is difficult to access. The intent of the fishway is to get the Steelhead salmon across the dam. The other types of fishways have not proven successful in passing the type of salmon species in the creek.

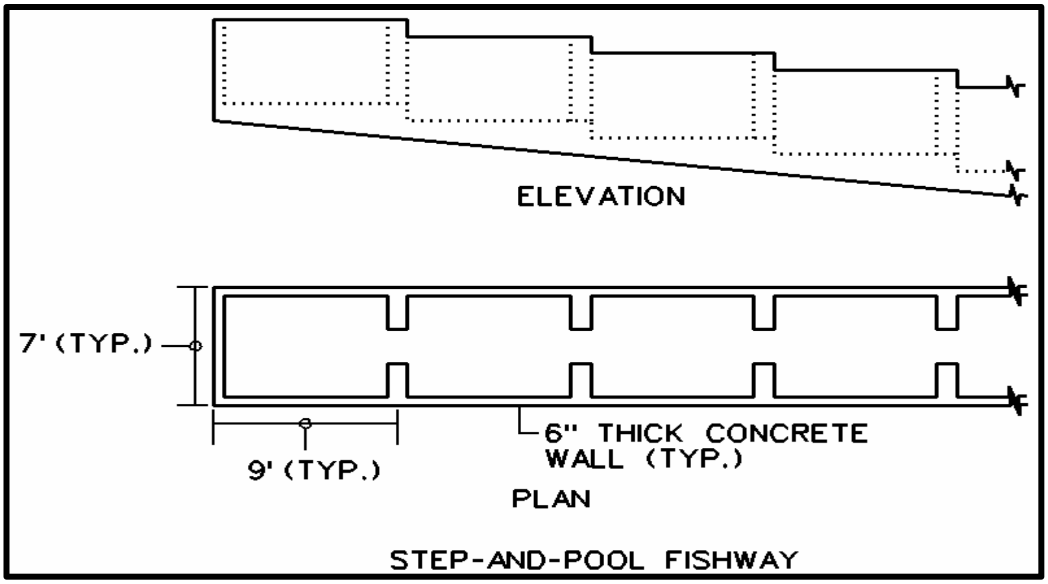


Figure 9.1-2 Step and Pool Fishway Plan and Elevation

Appendix C1
Upstream Barrier Removal

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Table of Contents

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LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

Rindge Dam removal will open up additional habitat for the project indicated species, the Steelhead Trout; however, additional barriers upstream of Rindge Dam have been identified that either limit or prevent upstream movements of the Steelhead. If some of these barriers could be altered or replaced to facilitate fish migration further upstream, then the project could generate additional habitat restoration.

2.0 UPSTREAM BARRIER SELECTION

There are a total of 38 barriers upstream of Rindge Dam. Of these 38, 29 are manmade structures and 9 are natural features. Manmade barriers included structures such as culverts, bridges and small agricultural dams while natural barriers include waterfalls and rock outcroppings. The fish can pass the natural features in at least one flow condition (usually under higher flow conditions) while most of the manmade barriers block fish migration during all flow conditions. A study conducted by Heal the Bay in 2005 evaluated the habitat upstream of each manmade barrier while a 2008 field survey conducted by the Corps of Engineers identified 10 priority structures for consideration in the Malibu Creek Environmental Restoration project. The barriers, and any proposed alterations, are summarized in **Table 2-1** and **Table 2-2**. The symbol names for each barrier are based on the 2005 Heal the Bay report. For each table, the stated width is the dimension parallel to the creek while the stated length is the dimension perpendicular to the creek.

Table 2-1 Summary of Upstream Barriers-Cold Creek

Barrier Symbol	Barrier Name	Existing Structure	Proposed Improvements Summary
CC1	Piuma Culvert	12 ft long, 46 ft wide CMP arch culvert with a concrete invert	Replace with a 12 ft long, 46 ft wide pre-cast arch culvert with a soft bottom
CC2	Malibu Meadows Road Crossing	28 ft wide, 40 ft long steel beam and wood deck bridge with a concrete invert	Replace with a 70 ft long, and 25 ft wide pre-manufactured bridge with a modified invert
CC3	Crater Camp Road Crossing	11 ft wide, 46 ft long steel beam and wood deck bridge with a concrete invert	Replace with a 70 ft long, and 12 ft wide pre-manufactured bridge with a modified invert
CC4	Cold Creek Barrier (Dam)	2.5ft high, 30ft long dam	Remove the dam
CC5	Cold Canyon Road Culvert	25ft diameter, 130 ft long concrete culvert	Construct a low flow channel through the culvert
CC8	Stunt Road Culvert	6ft diameter, 104 ft wide CMP culvert	Replace the culvert with an 8ft x 8ft precast concrete culvert with a low flow channel built along the bottom.

Table 2-2 Summary of Upstream Barriers-Las Virgenes Creek

Barrier Symbol	Barrier Name	Existing Structure	Proposed Improvements Summary
LV1	Crags Road Culvert Crossing	6 ft diameter, 31 ft long double barrel concrete culvert	Replace with a pre-manufactured 75 ft long, 20 ft wide clear span bridge
LV2	White Oak Dam	6 ft high, 87 ft long diversion dam	Lower the dam in stages
LV3	Lost Hills Road Culvert	61 ft wide,241 ft long, concrete box culvert with four 14ft x 14ft openings	Construct a low flow channel through the culvert
LV4	Meadow Creek Lane Crossing	61 ft wide, 82 ft long concrete box culvert with four 14ft x 14ft openings	Construct a low flow channel through the culvert

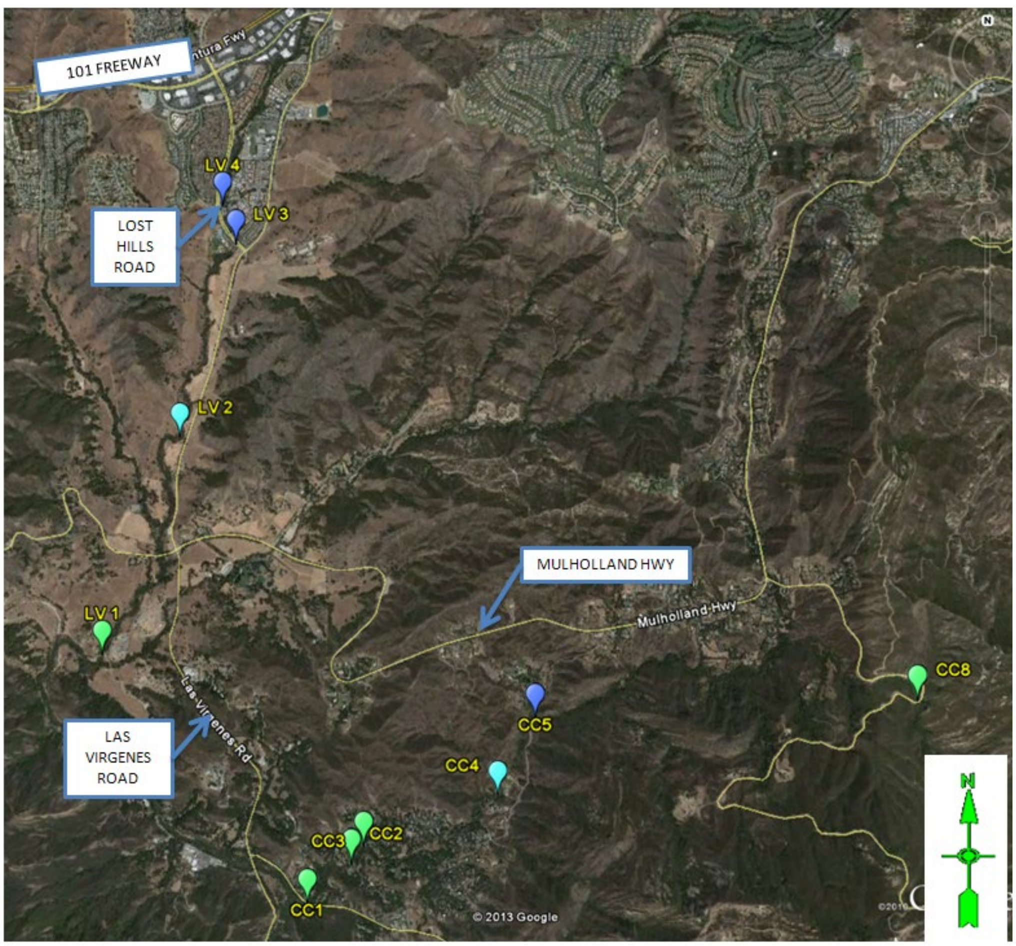

Figure 2.0-1 Location of the Upstream Barriers

Green barriers will be removed and replaced, light blue barriers will be removed, and dark blue barriers will be modified

3.0 UPSTREAM BARRIER ANALYSIS

The analysis for each of the following barriers is based on field visits, available photos of the structures and aerial photos in Google Earth. Each proposed solution was developed solely to facilitate fish passage to upstream habitat. The proposed replacement structures were designed to mimic the geometry of the existing barriers so they would not create additional flood risk. Reducing existing flood risk was not considered a goal for any barrier replacement. Additional hydraulic analysis, geotechnical investigations, existing utility research, and site surveys will be required during design phase for each proposed replacement structure.

3.1 LV1 – Crags Road Culvert

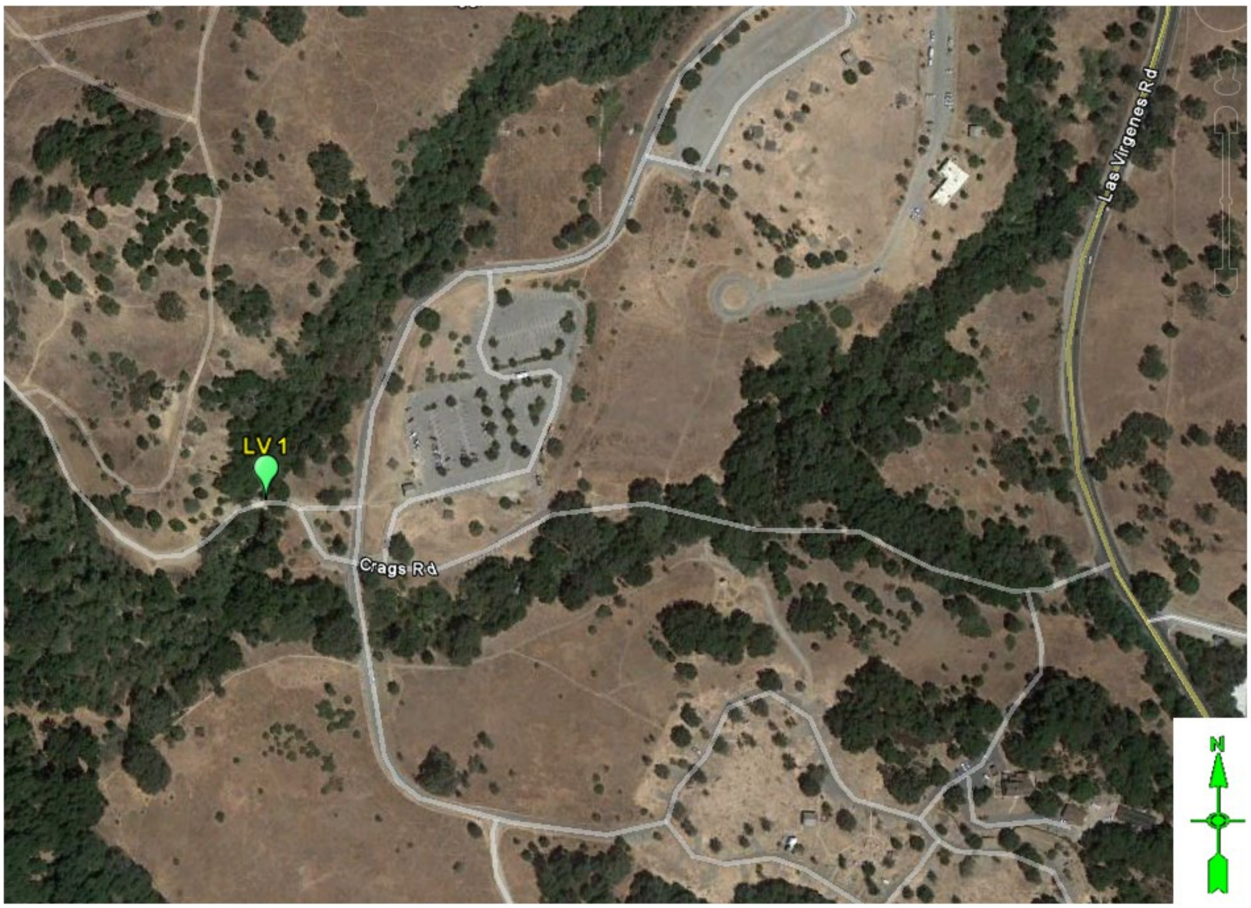

Figure 3.1-1 LV1, Crags Road Culvert (Aerial View)

3.1.1 Existing Structure

LV1, Crags Road Culvert is a 6 ft diameter concrete, double barrel culvert located on Malibu State Parks land along Las Virgenes Creek. It currently serves as a crossing for maintenance vehicles for Malibu State Park and fire trucks.

Figure 3.1-2 LV1, Crags Road Culvert (Picture taken while looking upstream toward the outlet of the culvert)

3.1.2 Design Considerations

This is culvert is considered an impassible barrier under all flow conditions. A low flow channel for fish passage cannot be carved into the invert of the structure without negatively affecting the structural integrity of the culverts and a low flow channel cannot be built on top of the existing invert without reducing the hydraulic capacity of the structure. This structure must be removed and replaced to allow fish to travel upstream.

This structure is in the Coastal Zone, governed by the Coastal Commission, and recent records show that only bridges have been approved as replacement structures for existing culverts. A 75ft span bridge would cross the creek and eliminate the exiting reduction in the creeks cross section. The creek is narrow enough that a clear span, pre-fabricated bridge can cross the entire creek. A pre-fabricated bridge is the preferred solution because of their affordability and quick installation times.

3.1.3 Proposed Improvements

The existing concrete box culvert, the existing concrete abutments, the existing concrete apron, and the existing concrete wing walls will be removed and replaced with a pre-manufactured 75 ft long, 20 ft wide clear span bridge. The new bridge deck elevation will match the top elevation of the existing structure.

Prior to installing the new bridge, the new wing walls, and the new bridge abutments will have to be constructed on both banks of the creek. The creek bed will have to be regraded to fill any voids left by the removal of the existing structures. Construction is estimated to take 15 days.

The creek flow will have to be diverted during removal of all the existing structures and construction of the new abutments and wing walls. Water diversion will also be necessary while any work is being performed within the creek. The creek will not need to be diverted while the pre manufactured bridge is being placed on the abutments. Dewatering will also be necessary during construction of the new bridge wing walls and the new bridge abutments.

Clearing will be required for the removal of the existing bridge wing walls and abutments along with construction of the new bridge wing walls and abutments. Additional clearing will be required at the designated staging area for the project. All areas that are cleared will be restored once construction is complete.

No traffic control measures will be required since this bridge is used for maintenance vehicles.

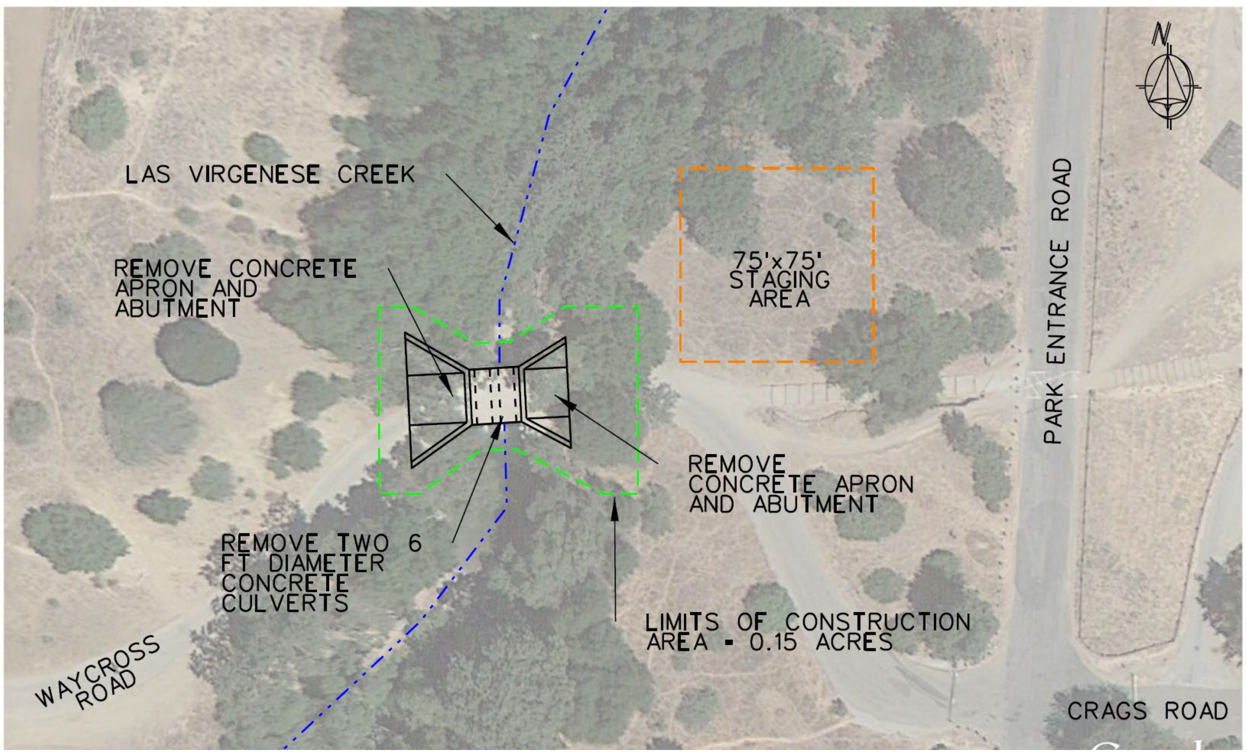

Figure 3.1-3 LV1, Crags Road Culvert Schematic Design Part A

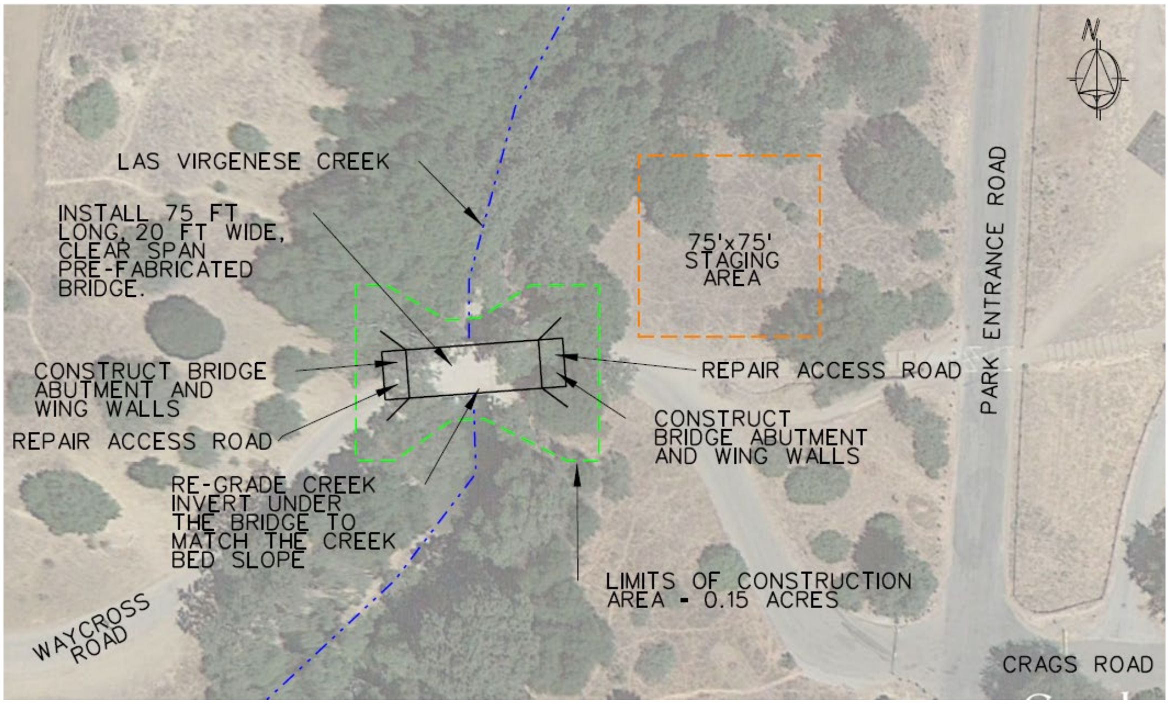

Figure 3.1-4 LV1, Crags Road Culvert Schematic Design Part B

3.2 LV2 – White Oak Dam

Figure 3.2-1 LV2, White Oak Cam (Aerial View)

3.2.1 Existing Structure

LV2, White Oak Dam is small diversion dam that is 6 ft high and spans 87 ft across Las Virgenes Creek. It was originally built to collect water for agricultural use.

3.2.2 Design Considerations

This structure is currently only passable under high flow conditions. For fish to pass under low flow conditions the dam must be removed. The dam is not a large structure, but removing it will alter the flows in the creek and may create erosion along the stream bed and banks.

3.2.3 Proposed Improvements

The existing 6 ft dam will be lowered by two feet a year over three years to reduce any erosion and scour problems. The foundation of the dam will be left in place. The creek will have to be diverted each year to protect any crews and equipment being used to remove the dam. However, work in the creek will be kept at a minimum since the dam will be removed by a backhoe stationed on the creek bank. Dewatering will not be required. Demolition is estimated to take 15 days each year. Clearing will be limited to a 40 ft by 40 ft area on either side of the cofferdam which will ensure the backhoe has adequate space to work. These areas will have to be cleared every year of dam removal. All areas that are cleared will be restored once the dam removal is completed. Once the dam is removed no further work will be done to restore the creek.

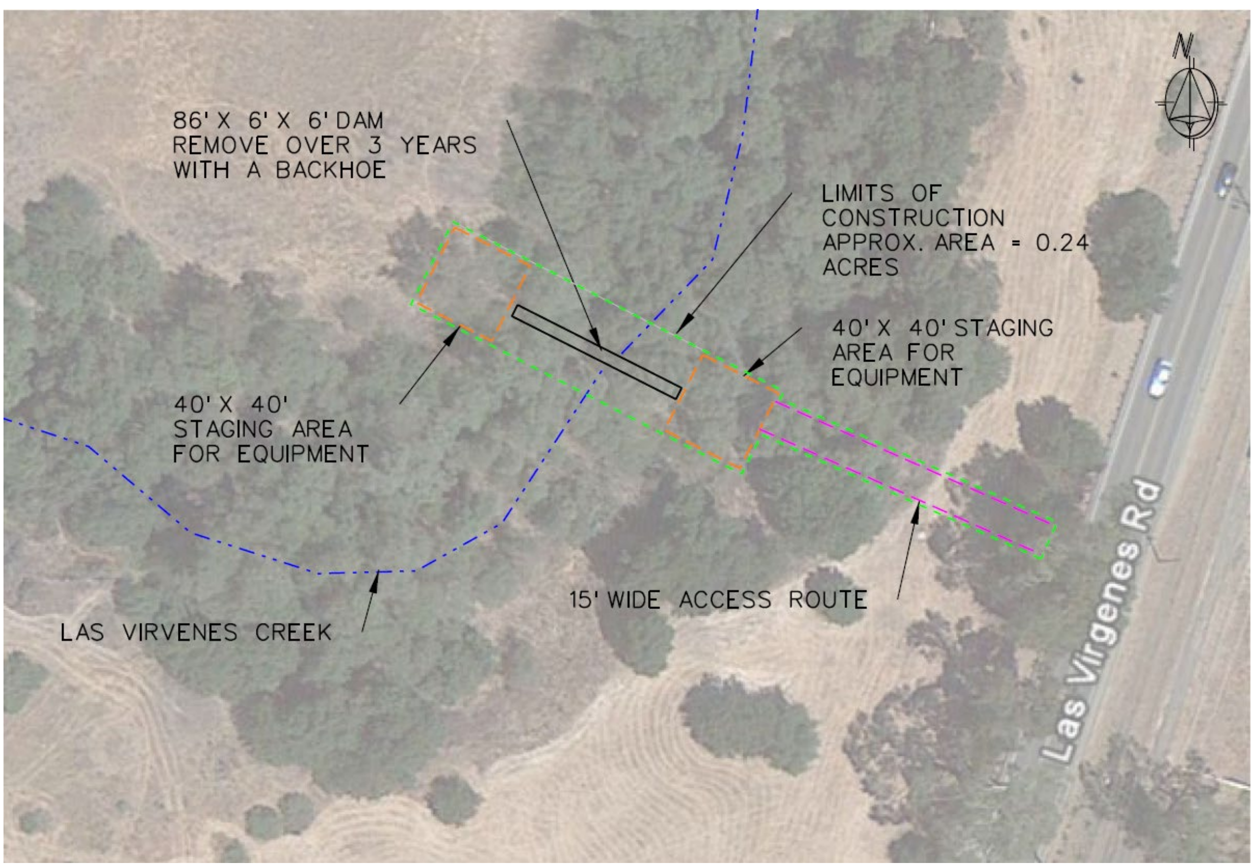

Figure 3.2-2 LV2, White Oak Dam Schematic Design

3.3 LV3 and LV4 – Lost Hills Road Culvert and Meadow Creek Lane Crossing

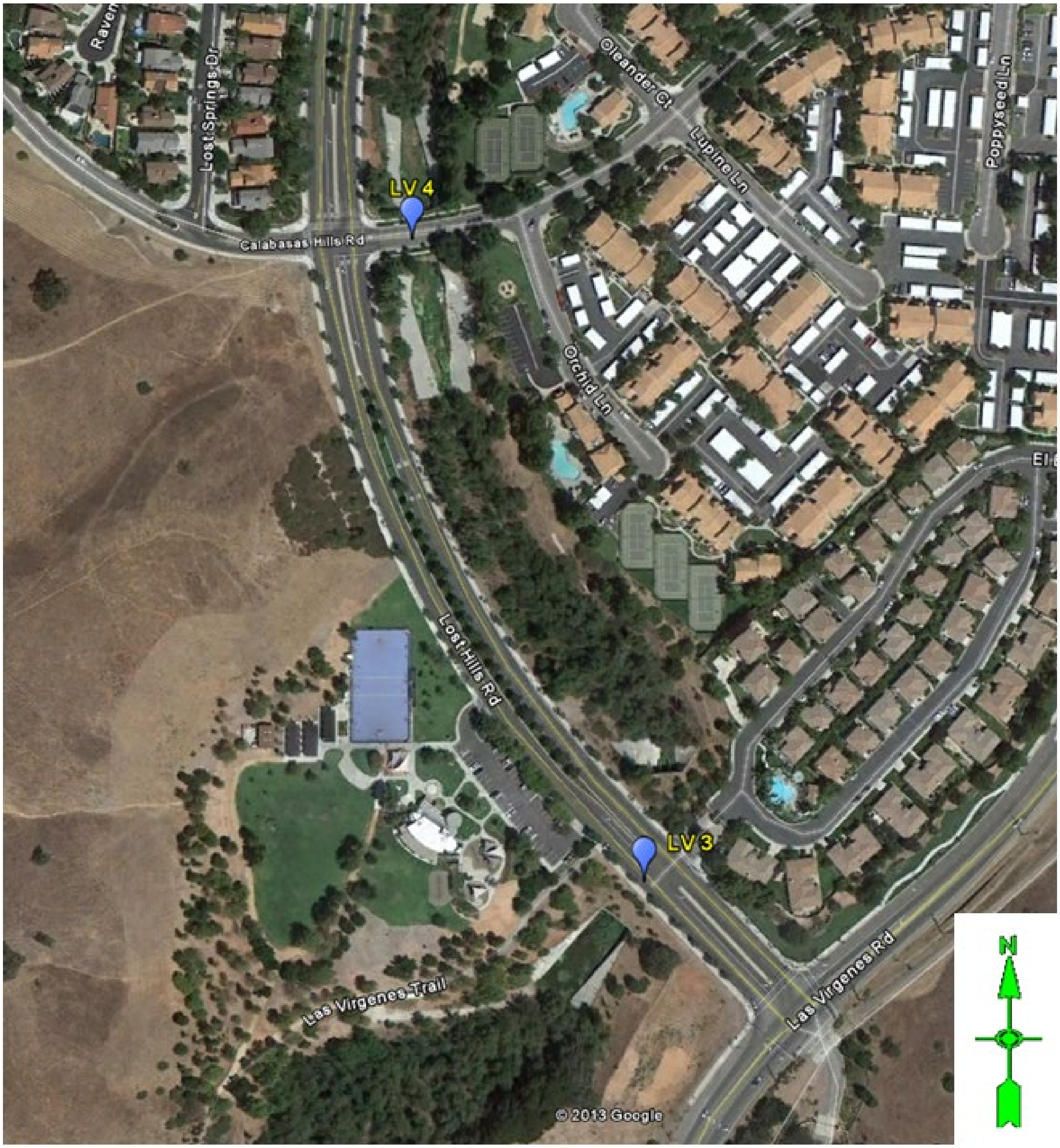

Figure 3.3-1 LV3 and LV4 Lost Hills Road Culvert and Meadow Creek Lance Crossing (Aerial View)

3.3.1 Existing Structure

LV3, Los Hills Road Culvert is a 61 ft wide, 241 ft long, concrete box culvert with four 14 ft x 14 ft openings. The culvert follows a curve in the creek. Los Hills Road is a four lane road that passes over the culvert and through a densely developed residential area. There are concrete aprons that extend 100 ft upstream of the inlet and 100 ft downstream of the outlet of the culvert. LV 4, Meadow Creek Lane Crossing, is 930 ft upstream of LV3 and consists of a 61 ft wide, 82 ft long concrete box culvert with four 14 ft x 14 ft openings. There are concrete aprons that extend 170 ft upstream of the inlet and 170 ft downstream of the outlet of the structure. Meadow Creek Lane is a two lane road that passes over the culvert and it serves as one of two points of entry into a densely developed residential neighborhood. Both structures are located on Las Virgenes Creek.

Figure 3.3-2 LV3, Lost Hills Road Culvert (Aerial View)

Figure 3.3-3 LV3, Lost Hills Road Culvert Inlet (Picture taken looking downstream)

Figure 3.3-4 LV4, Meadow Creek Crossing (Aerial View)

3.3.2 Design Considerations

Current stream flows through LV3 and LV4 are too shallow and fast moving to allow fish to swim through them. The two structures will have to be treated as a single project because fish have to pass through both barriers to reach the habitat areas upstream of LV 4. The stream between the two structures is not considered viable habitat. Due to the size of the structures, and the amount of residential traffic that passes over each one, they cannot easily be replaced with bridges.

These structures will not be removed, but instead a low flow channel will be constructed along the top of the invert of each structure. The channel cannot be carved into each structure without threatening the structural integrity of the culverts and the concrete aprons. The low flow channels passing through the culverts would be small relative to the overall size of the culvert and have a minimum impact on hydraulic capacity. Some form of low flow channel will be necessary along the creek segment between LV3 and LV4 to create a continuous low flow channel that can facilitate fish migration upstream of LV4.

3.3.3 Proposed Improvements

The low flow channel for LV3 will be built on top of the existing concrete invert. This channel will be six inches deep and start at the downstream end of the concrete apron and extend upstream, through the culvert structure and terminate at the end of the upstream concrete apron. This channel will be three feet wide and will ensure there is enough water traveling at low enough velocities for fish to pass through the structure. The drop at the downstream end of the concrete invert will not be modified. The low flow channel for LV4 will be similar to the channel passing through LV3 and allow fish to travel upstream to the designated habitat areas.

The invert of the creek between LV3 and LV4 will have to be cleared and regraded to provide a low flow channel which will connect the concrete channels along LV3 and LV4. This area will be restored once construction is complete.
The creek flow will have to be diverted during construction of both of the concrete low flow channels and while the creek invert between LV3 and LV4 is being regarded. Additional clearing will be required at the designated staging area for the project and along any invert access ramps. The staging area will be restored once construction is completed.

Limited dewatering will be necessary along the creek between LV3 and LV4 to ensure adequate working conditions for construction equipment.

Construction of the entire low flow channel, from the downstream of LV3 to upstream of LV4, is estimated to take 50 days.

Some traffic control measures may be required during construction hours to facilitate the movement of equipment from the staging are to the construction site.

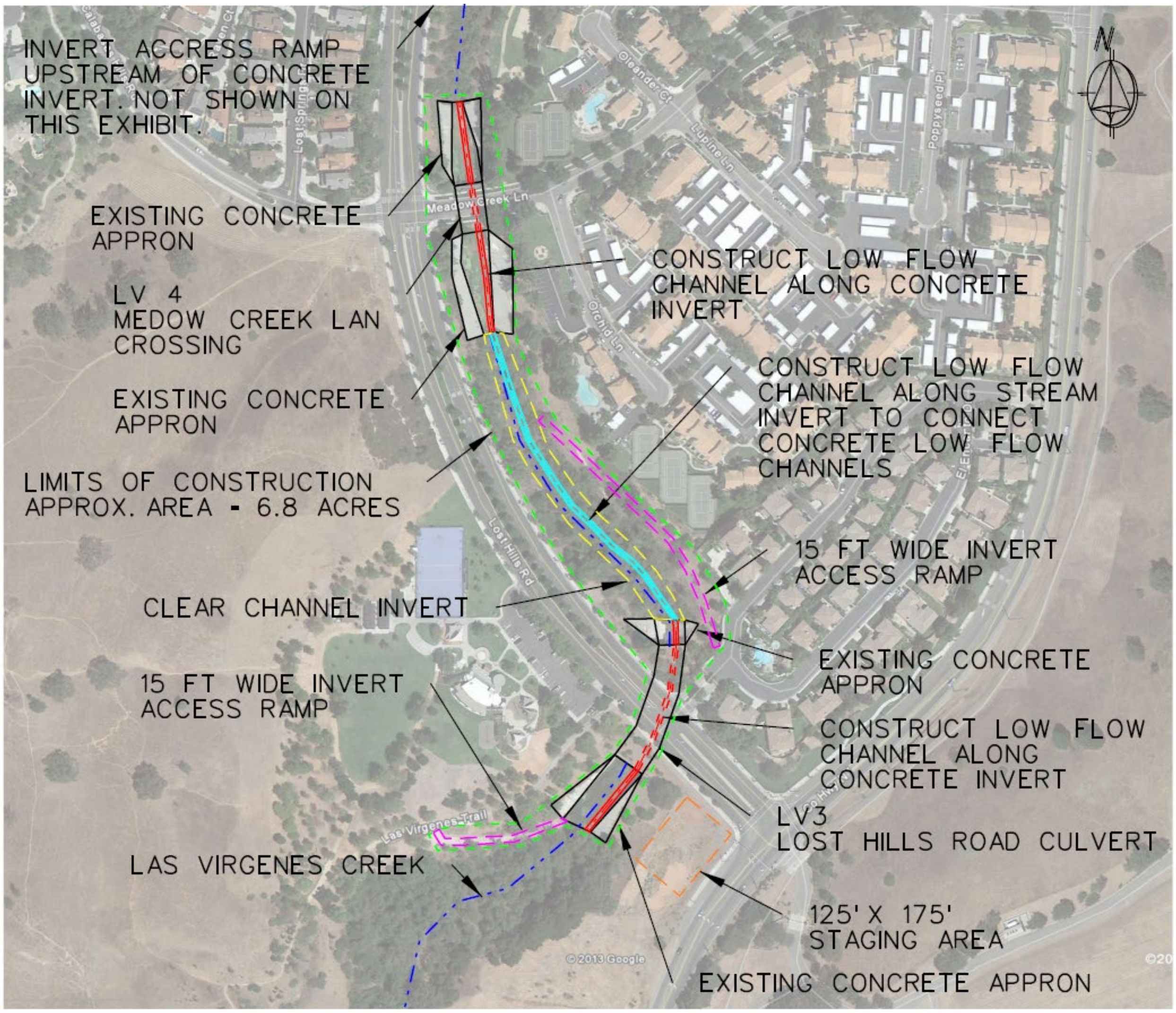

Figure 3.3-5 LV3 and LV4, Lost Hills Road Culvert and Meadow Creek Lane Crossing Schematic Design

3.4 CC1- Piuma Culvert

Figure 3.4-1 CC1, Piuma Culvert (Aerial Image)

3.4.1 Existing Structure

CC1, Piuma Culvert is a 12 ft long, 46 ft wide corrugated metal pipe (CMP) arch culvert with a concrete invert. The downstream end of the concrete invert ends abruptly and there is a two foot drop from the top of the concrete invert to the creek bed. Piuma road is a two lane rural road that passes over the structure and provides access to homes throughout the hills. CC1 is located along a curve in Piuma Road. There is a 90 degree bend in Cold Creek approximately 50 feet downstream of the culvert where bed rock is exposed.

Figure 3.4-2 CC1, Piuma Culvert (Picture taken looking upstream)

3.4.2 Design Considerations

Currently stream flows move to quickly through the culvert to allow fish to swim through. The concrete invert of the structure cannot safely be modified without compromising the structural integrity of the culvert. The culvert must be replaced with a soft bottom structure to accommodate fish migration.

The existing structure is small enough where it can be replaced with a pre-cast concrete arch culvert. A pre-fabricated culvert is the preferred solution because of their affordability and quick installation times.

3.4.3 Proposed Improvements

The existing CMP arch culvert, the concrete lining along the creek invert, and the stone head walls will be replaced with a 12 ft pre-cast arch culvert with new concrete footings and concrete head walls on both ends of the structure. The width and height of the new culvert will match the existing CMP culvert and the road elevations across the culvert will be the same as the existing roadway.

The existing CMP arch culvert, stone wing walls, and concrete invert will be removed in two stages. The first stage will be from the upstream inlet to the centerline of the road, the second state will be from the centerline of the road to the downstream outlet. The culvert must be removed in two parts so the traffic along the road can diverted into one lane across the creek. Traffic control measures will be required during and after construction hours to ensure traffic can safely be reduced down to one lane across the creek.

The pre cast culvert will reduce construction time since the culvert will be delivered to the site and placed on the footings with a crane. Prior to installing the new culvert sections, new headwalls, and the new footings will have to be constructed. Construction is estimated to take 30 days.

The concrete invert of the creek will be replaced with a natural channel. The creek bed under the culvert will have to be re-graded to compensate for the small elevation drop at the end of the existing concrete invert.

Temporary shoring will be required to preserve the road while the existing metal culvert and stone wing walls are being removed. The temporary shoring will be placed perpendicular to the centerline of the road and run parallel to the existing CMP culvert for 46 ft. The temporary shoring will be required on the east and west sides of the existing structure and will be removed once the new bridge abutments and wing walls are completed.

The creek flow will have to be diverted during removal of all the existing structures and construction of the new footings and headwalls. The creek will also have to be diverted while any work is being performed within the creek bed. Dewatering will also be necessary during construction of the new culvert footings and headwalls.

Clearing will be required for the removal of the existing culvert wing walls and abutments along with construction of the new culvert footings and headwalls. Additional clearing will be required at the designated staging area for the project. All areas that are cleared will be restored once construction is complete.

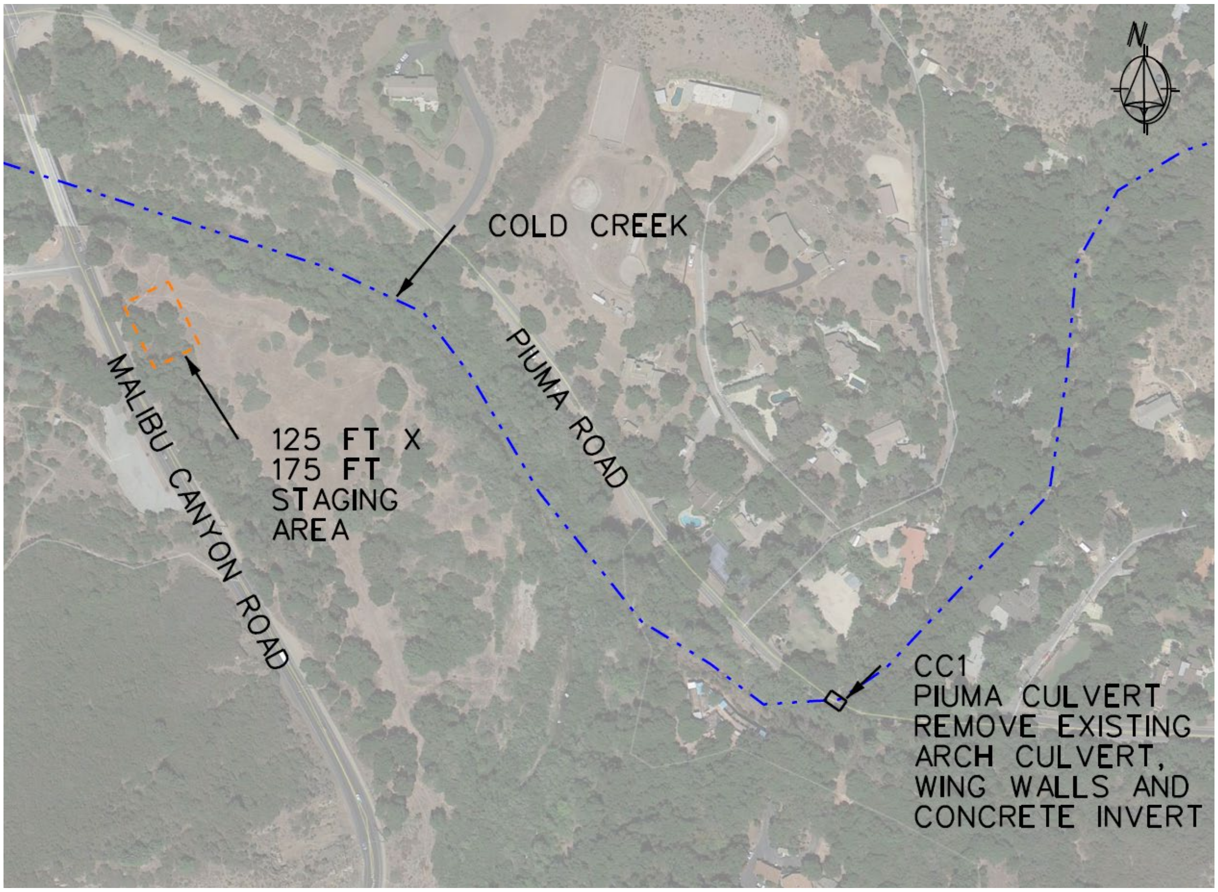

Figure 3.4-3 CC1, Piuma Culvert Schematic Design Part A

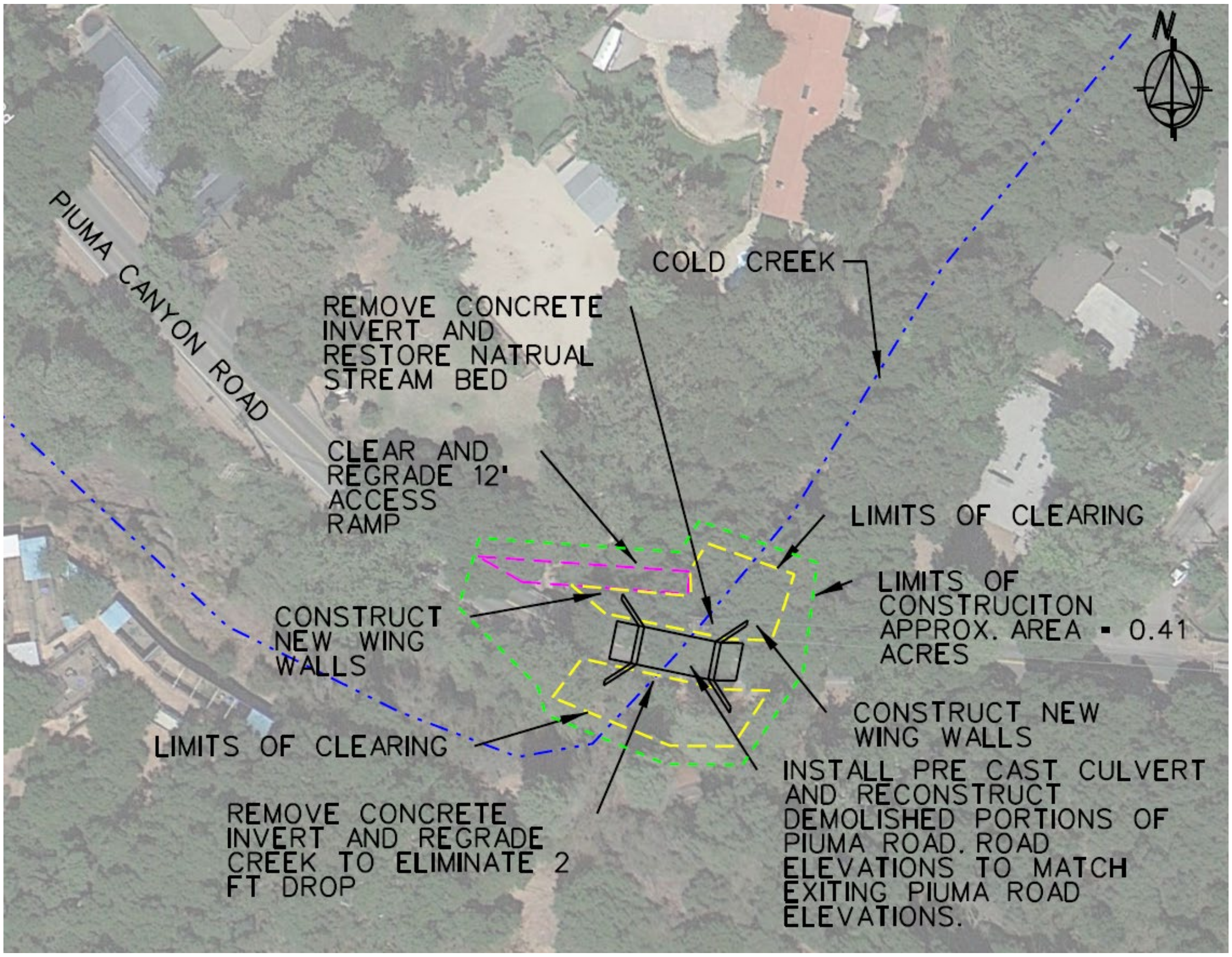

Figure 3.4-4 CC1, Piuma Culvert Schematic Design Part B

3.5 CC2 – Malibu Meadows Road Crossing

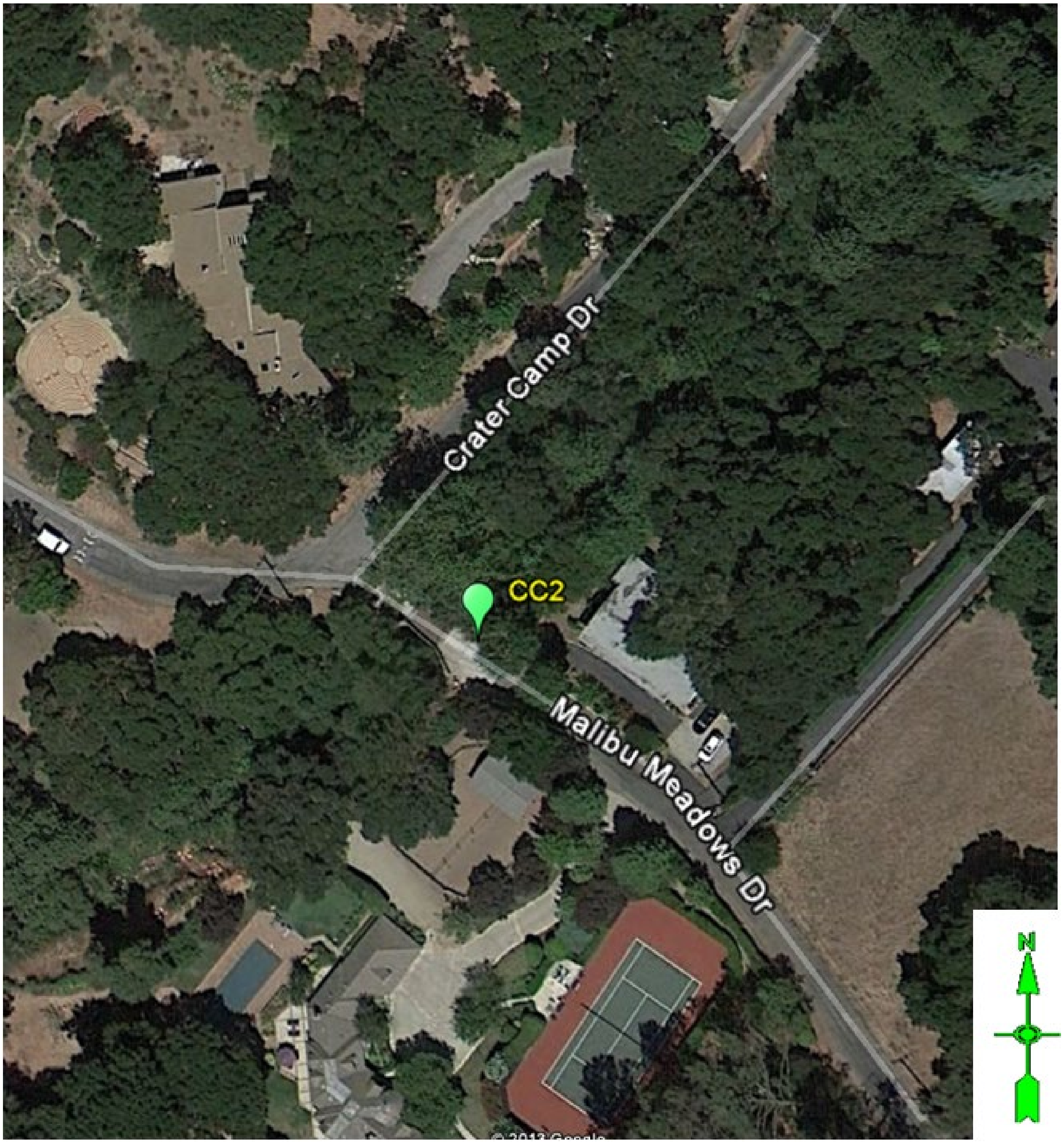

Figure 3.5-1 CC2, Malibu Meadows Road (Aerial Image)

3.5.1 Existing Structure

CC2, Malibu Meadows Road Crossing, is a 28 ft wide, 40 ft long steel beam bridge with a wood deck. A concrete slab that is at least 18 inches thick passes under the bridge and ends abruptly just downstream of the structure. There is a two foot to three foot drop from the top of the concrete slab to the creek invert. The bridge is part of Malibu Meadows Road which is a narrow two lane road that serves homes throughout the hills.

Figure 3.5-2 CC2, Malibu Meadows Road Crossing, Upstream End

Figure 3.5-3 CC2, Malibu Meadows Road Crossing, Downstream End

3.5.2 Design Considerations

CC2 is a physical barrier, the fish cannot overcome the drop along the invert, and a velocity barrier, flows through the structure are generally too fast for fish to swim through. A fish ladder could be constructed at the downstream end of the concrete invert, however flow velocities under the bridge would still prevent fish from swimming upstream. A low flow channel cannot be carved into the concrete invert because there would be fears of compromising the structural integrity of the slab. The slab appears to be incorporated into the bridge abutments and wing walls. If the bridge is replaced then there will have to be permanent improvements constructed along the creek bed to eliminate the existing drop.

The creek is narrow enough were a clear span pre-fabricated bridge can be installed. A pre-fabricated bridge is the preferred solution because of their affordability and quick installation times.

3.5.3 Proposed Improvements

The existing wood deck, steel beam bridge along with the concrete invert and concrete block abutments and wing walls will be removed and replaced with a 70 ft long, and 25 ft wide pre-manufactured bridge with concrete abutments and wing walls on both sides of the creek. The new bridge will have a longer span than the existing structure to eliminate the existing reduction of the creek cross section and the bridge deck elevation will match the existing bridge deck elevation.

The pre manufactured bridge will reduce construction time since the bridge will be delivered to the site and placed on the new abutments with a crane. Prior to installing the new bridge, new wing walls and the new bridge abutments will have to be constructed on both banks of the creek. Construction is estimated to take 30 days.

The existing concrete invert will be removed and replaced with a modified stream bed. The stream bed improvements will have to be designed to compensate for the three foot drop at the end of the existing concrete invert while still allowing fish to swim upstream. The stream bed improvements will have to prevent head cutting upstream of the new bridge.

The creek flow will have to be diverted during removal of all the existing structures and construction of the new abutments and wing walls. The creek flows will also have to be diverted while any work is being performed within the creek bed. The creek will not need to be diverted while the pre manufactured bridge is being installed. Dewatering will also be necessary during construction of the new bridge wing walls and the new bridge abutments.

Clearing will be required for the removal of the existing bridge wing walls and abutments along with construction of the new bridge abutments and wing walls. Additional clearing will be required at the designated staging area for the project. All areas that are cleared will be restored once construction is complete.

Traffic control measures will only be in place to warn drivers of a closed bridge and redirect them through neighboring streets.

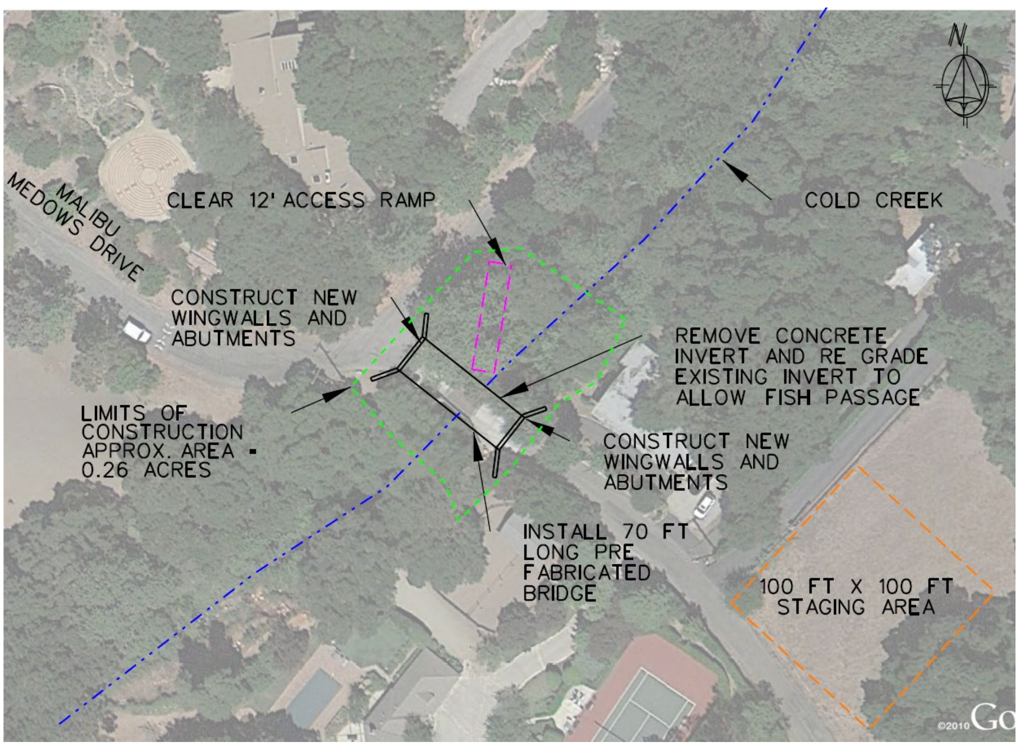

Figure 3.5-4 CC2, Malibu Meadows Road Crossing Schematic Design

3.6 CC3 – Crater Camp Road Crossing

Figure 3.6-1 CC3, Crater Camp Road Crossing (Aerial Image)

3.6.1 Existing Structure

CC3, Crater Camp Road Crossing, is 11 ft wide, 46 ft long steel beam bridge with a wood deck. A concrete slab that is at least 18 inches thick passes under the bridge and ends abruptly just downstream of the structure. There is a three foot drop from the top of the concrete slab to the creek invert. The bridge is part of Crater Camp Road which is a narrow road that serves homes throughout the hills.

Figure 3.6-2 CC3, Crater Camp Crossing, Upstream End

Figure 3.6-3 CC3, Crater Camp Crossing, Downstream End

3.6.2 Design Considerations

CC3 is a physical barrier, the fish cannot overcome the drop along the invert, and a velocity barrier, flows through the structure are generally too fast for fish to swim through. A fish ladder could be constructed at the downstream end of the concrete invert, however flow velocities under the bridge would still prevent fish from swimming upstream. A low flow channel cannot be carved into the concrete invert because there would be fears of compromising the structural integrity of the slab. The slab appears to be incorporated into the bridge abutments and wing walls. If the bridge is replaced then there will have to be permanent improvements constructed along the creek bed to eliminate the existing drop.

The creek is narrow enough were a clear span pre-fabricated bridge can be installed. A pre-fabricated bridge is the preferred solution because of their affordability and quick installation times.

3.6.3 Proposed Improvements

The existing wood deck, steel beam bridge along with the concrete invert and CMU abutments and wing walls will be removed and replaced with a 70 ft long, and 12 ft wide pre-manufactured bridge with concrete abutments and wing walls on both sides of the creek. The new bridge will have a longer span than the existing structure which will eliminate the existing reduction of the creek cross section and the bridge deck elevation will match the existing bridge deck elevation.

The pre-manufactured bridge will reduce construction time since the bridge will be delivered to the site and placed on the new abutments with a crane. Prior to installing the new bridge, new wing walls and the new bridge abutments will have to be constructed on both banks of the creek. Construction is estimated to take 30 days.

The existing concrete invert will be removed and replaced with a modified stream bed. The stream bed improvements will have to be designed to compensate for the three foot drop at the end of the existing concrete invert while still allowing fish to swim upstream. The stream bed improvements will have to prevent head cutting upstream of the new bridge.

The creek flow will have to be diverted during removal of all the existing structures and construction of the new abutments and wing walls. The creek flows will also have to be diverted while any work is being performed within the creek bed. The creek will not need to be diverted while the pre manufactured bridge is being installed. Dewatering will also be necessary during construction of the new bridge wing walls and the new bridge abutments.

Clearing will be required for the removal of the existing bridge wing walls and abutments along with construction of the new bridge abutments and wing walls. Additional clearing will be required at the designated staging area for the project. All areas that are cleared will be restored once construction is complete.

Traffic control measures will only be in place to warn drivers of a closed bridge and redirect them through neighboring streets.

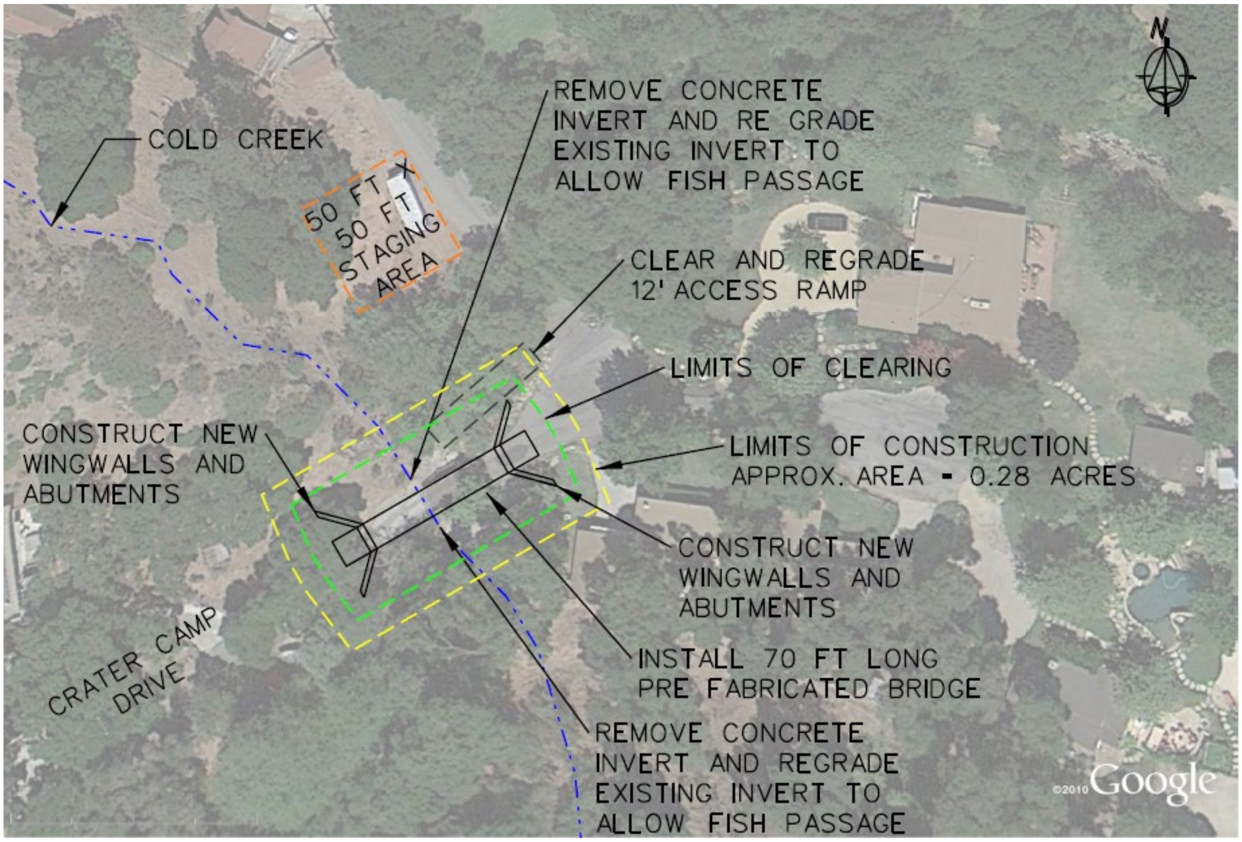

Figure 3.6-4 CC3, Crater Camp Road Crossing Schematic Design

3.7 CC4 – Cold Creek Barrier

3.7.1 Existing Structure

CC4, Cold Creek Barrier is small dam that is 2.5 ft high and spans 130 ft across Cold Creek.

3.7.2 Design Considerations

This structure is currently only passable under high flow conditions. For fish to pass under low flow conditions the dam must be removed. The dam is not a large structure, but removing it will alter the flows in the creek and may create erosion

3.7.3 Proposed Improvements

The existing 2.5 ft dam will be removed by a small crew in an estimated 15 days. The creek will have to be diverted to protect any crews and equipment being used to remove the dam. However, work in the creek will be kept at a minimum since the dam will be removed by a backhoe stationed on the creek bank. Clearing will be limited to a 40 ft by 40 ft area on either side of the cofferdam to ensure the backhoe has adequate space to work. All areas that are cleared will be restored once the dam removal is completed. No additional work will be done along the creek bed once the dam is removed.

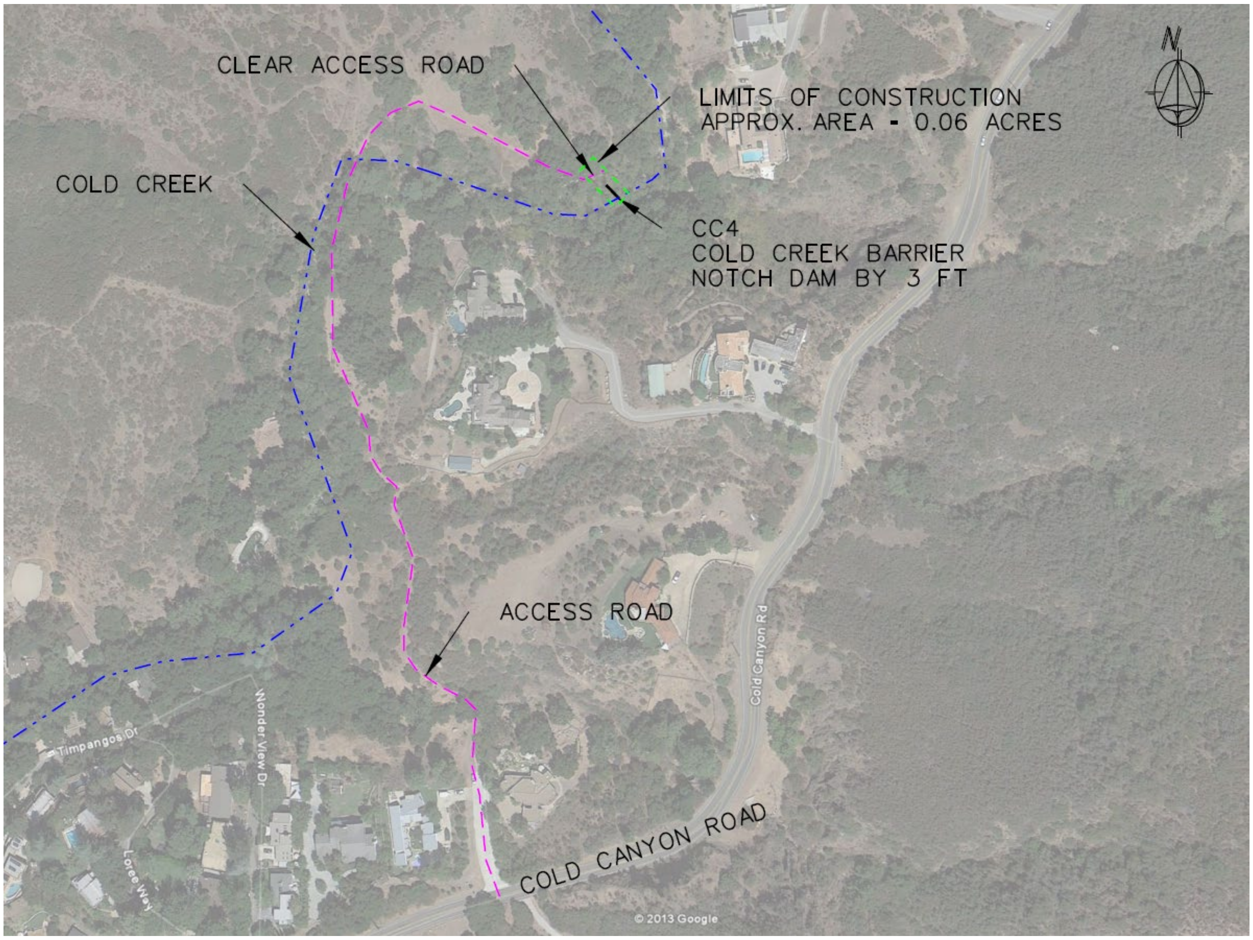

Figure 3.7-1 CC4, Cold Creek Barrier Schematic Design

3.8 CC5 – Cold Canyon Road Culvert

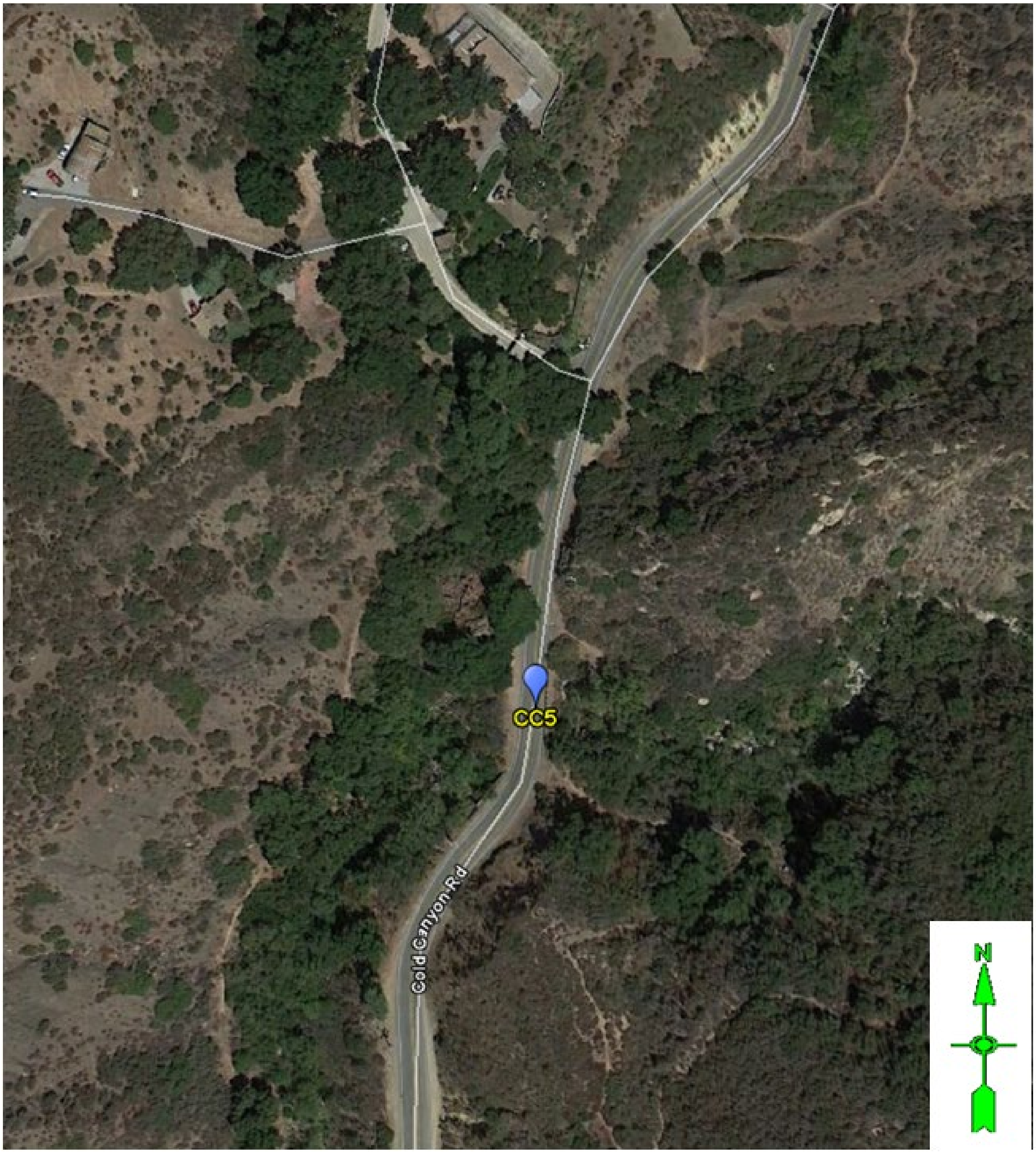

Figure 3.8-1 CC5, Cold Canyon Road Culvert (Aerial)

3.8.1 Existing Structure

CC5, Cold Canyon Road Culvert is a 25 ft diameter concrete culvert along Cold Creek. Cold Canyon Road is a two lane rural road that serves homes in the mountains.

Figure 3.8-2 CC5, Cold Canyon Road Culvert (Picture taken looking upstream)

3.8.2 Design Considerations

This culvert currently serves as a velocity barrier, flows pass through it too quickly for fish to swim upstream. It would be impractical to remove such a large structure but a low flow channel could be constructed along the invert of the culvert that could facilitate fish passage. The low flow channel would be small relative to the overall size of the culvert and have a minimum impact on hydraulic capacity.

3.8.3 Proposed Improvements

The existing 25 ft diameter concrete culvert cannot be removed so a low flow channel will be built along the culvert's invert to allow fish passage upstream. The channel would be six inches deep and thee feet wide and will ensure flows are slow enough and deep enough for fish to swim through during low flow conditions. The downstream portion of the culvert will not be modified because fish can use existing ponds to make their way into the low flow channel. The creek invert near the inlet of the culvert will have to be cleared and regraded to ensure flows can enter the low flow channel. Creek flows will need to be diverted during construction but no dewatering will be necessary. Construction is estimated to take 15 days. No traffic control will be necessary.

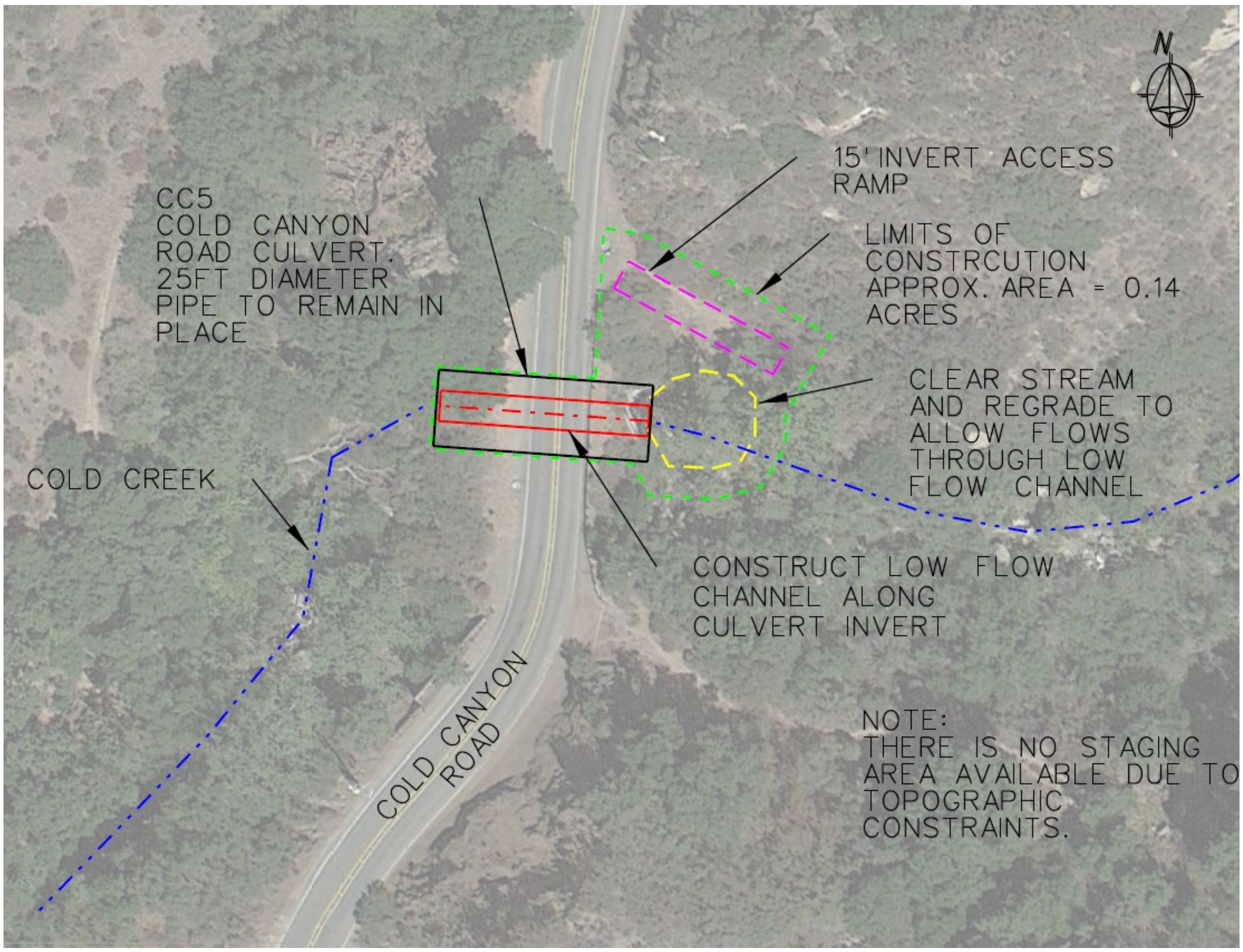

Figure 3.8-3 CC5, Cold Canyon Road Culvert Schematic Design

3.9 CC8- Stunt Road Crossing

Figure 3.9-1 CC8, Stunt Road Crossing (Aerial)

3.9.1 Existing Structure

CC8, Stunt Road Crossing is a six foot diameter CMP culvert that runs along the creek for 104 ft. The structure is covered with at least 20 ft of soil and has a stone headwall at the outlet that also acts as a retaining wall for the embankment of Stunt Road. Stunt Road is a two land rural road that serves traffic passing through the mountains. The culvert is located along a curve in Stunt Road on Cold Creek.

Figure 3.9-2 CC8, Stunt Road Crossing Picture (Picture taken looking upstream)

3.9.2 Design Considerations

This culvert currently serves as a velocity barrier for fish. The fish can use small ponds downstream of the culvert to reach the inlet, however they cannot pass through the culvert since flows are moving too fast. This culvert cannot easily be replaced with a bridge because the large amounts of fill on top of the structure. The existing culvert cannot be modified to accommodate fish passage without reducing the hydraulic capacity of the structure. This culvert must be replaced with a larger culvert that can accommodate fish passage.

3.9.3 Proposed Improvements

The existing six foot diameter CMP culvert will be removed and replaced with an 8 ft x 8 ft pre-cast concrete box culvert. The existing stone headwall will remain in place but be modified to accommodate the new concrete culvert. To ensure fish can pass through the new structure, a four foot wide, and six inch deep low flow channel will be built into the culvert. This low flow channel will help ensure that flow velocities are low enough, and the flow depths are high enough, that fish can pass through the structure during low flow events. The existing stone headwall must remain in place because it is a retaining wall for the road embankment.

The demolition and construction of the culvert will have to be done in phases. The first phase will consist of removing the culvert from the outlet to the edge of the existing road and replacing it with the new culvert. During this phase traffic will have to be redirected into a new lane built along the south side of the road. This new lane will require additional fill and impact the area directly upstream of the existing culvert. Once the first half of the culvert is installed, and the road across the culvert is re-built, traffic will be redirected over the new road. The temporary lane and the remaining existing culvert will be removed while the remaining part of the new culvert is constructed. Traffic control measures will be required during and after construction hours to ensure traffic can safely be reduced down to one lane for the duration of the project. It is estimated that construction will take 30 days. Once the culvert is completed the area of the creek that was impacted by the temporary traffic lane construction will be restored.

The creek flow will have to be diverted during all phases of construction. Limited dewatering will also be necessary during construction of the new culvert and roadway to ensure adequate working conditions for the construction equipment.

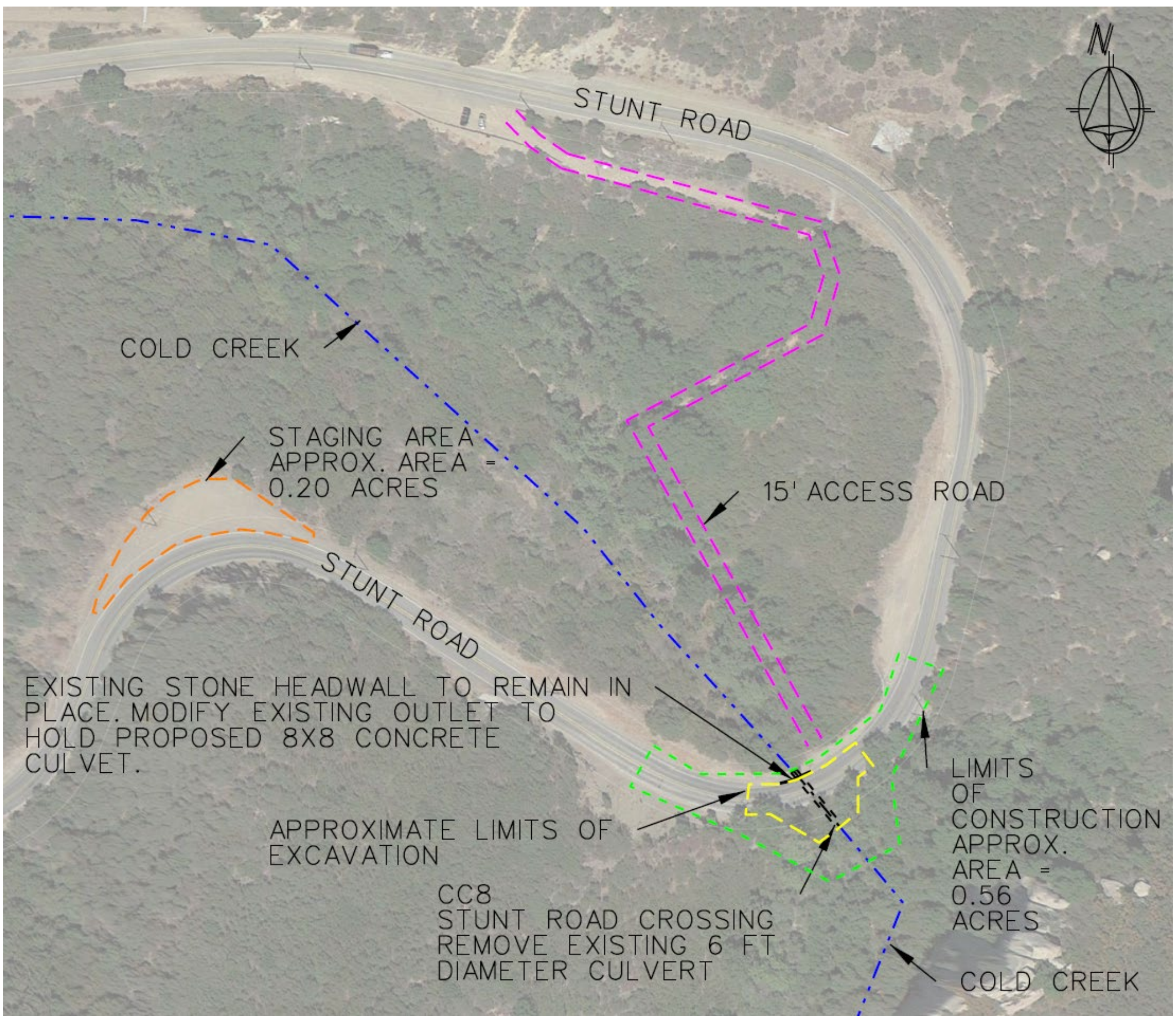

Figure 3.9-3 CC8, Stunt Road Crossing Schematic Design Part A

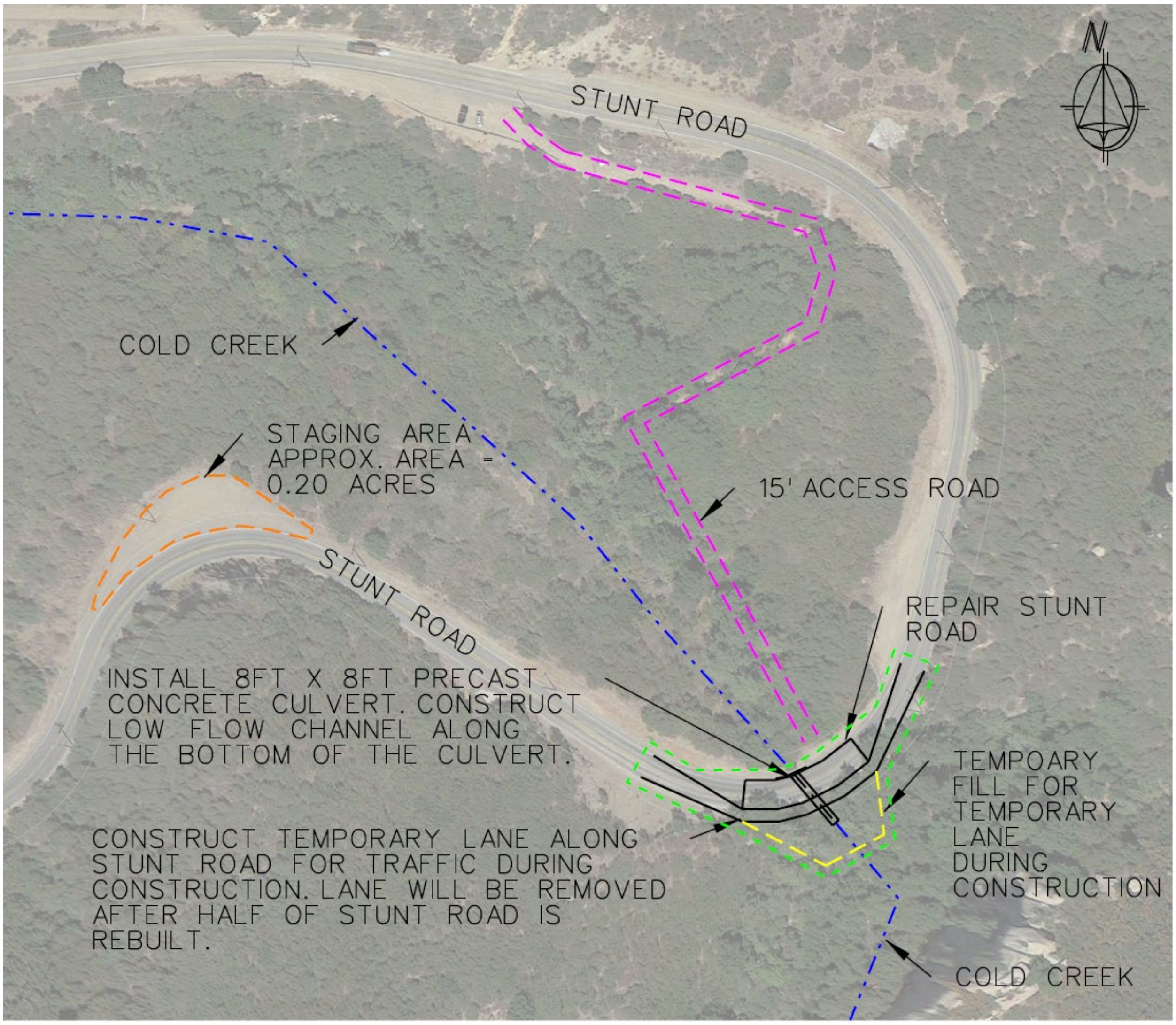

Figure 3.9-4 CC8, Stunt Road Crossing Schematic Design Part B

Appendix C2
Sediment Quantities Calculations

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LEGACY FEASIBILITY STUDY, F-4 GEOTECHNICAL REPORT

Attachment D.
Calculations, impounded sediment quantity estimates.

Estimates by USACE-Geotech, 3-10-03.

Several steps in calculations were undertaken in order to estimate quantities of impounded sediment of the former reservoir.

Step 1 was dividing the impounded sediment into 4 blocks (see figs. 19, 20, 21):

- Block one is defined by the logs of USACE-Geotech borings TH02-01, -04, and Law / Crandall's 1993 boring 2 and test pit 1. Using the aerial photo (fig. 19, its dimensions are 330 ft in the u/s - d/s direction, 250-ft-wide at the top of the basin (with an estimated 30% of the "block" volume at depth occupied by the rock of the canyon side walls), and an average thickness of 94 ft (based on the three borings).
- Block two is defined by the logs of USACE-Geotech borings TH02-02, -03, and -05, and Law / Crandall's 1993 boring 1 and test pit 2. Its dimensions are 825 ft in the u/s - d/s direction, 270-ft-wide at the top of the basin (with and estimated 30% of the "block" volume at depth occupied by the rock of the canyon side walls), and an average thickness of 80 ft (based on the four borings).
- Block three is defined by the logs of USACE-Geotech borings TH02-06, -07. Its dimensions are 890 ft in the u/s - d/s direction, 175-ft-wide at the top of the basin in the d/s half of the block (with an estimated 40% of the "block" volume at depth occupied by the rock of the canyon side walls), 75-ft-wide at the top of the basin in the u/s half of the block (with 50% of the "block" volume at depth occupied by the rock of the canyon side walls), and an average thickness of 44 ft (based on the three borings).
- Block four is defined by the logs of USACE-Geotech borings TH02-08, and Law / Crandall's 1993 test pits 3, and 4. Its dimensions are 500 ft in the u/s - d/s direction, 75- ft-wide at the top of the basin in the u/s half of the block (with an estimated 50% of the "block" volume at depth occupied by the rock of the canyon side walls), and a thickness of 20 ft (based on one boring).

Note that the estimating of the amount of encroachment of sloping canyon bedrock walls--essentially the angle of the slope--carries the greatest potential for imparting error into the estimated materials quantities. Encroachment at depth by the canyon sidewalls reduces amount of sediment that potentially could be contained in each block. Estimates of the volume of that encroachment are based on observing local geomorphology and topography at the surface of the reservoir. If these estimates of slope angle are not correct, the volume estimate could vary largely from the actual quantity, perhaps by 20 to 30 percent. The only way to relieve this uncertainty would be to drill holes along the reservoir periphery, but in 2002 USACE-Geotech was prevented from cutting vegetation in that same zone for make a foot traffic access path. The possibility of permitting drilling in that same area does not seem likely, nor could it be entertained financially at this time. The uncertainty will have to remain.

The results of the gross materials quantities estimates are in the chart below.

Block	Quantity of material (cu yds)	Quantity of material, rounded (cu yds)	Thickness of block, avg., ft
Block 1	201,055	200,000	94 (see note 1)
Block 2	462,000	470,000	80 (see note 1)
Block 3	103,338	100,000	44 (see note 1)
Block 4	13,889	10,000	20 (see note 1).
Total	780,282	780,000	
Note 1: See the beginning of this attachment for a listing of the specific borings used to estimate avg. total thickness of each specific block. The bottom-of-hole data from the eight USACE-Geotech borings from late 2002 and from the three Law / Crandall borings of 1993 were studies to determine these avg. total depths of each block..			

Some thought was given to making one volume estimate for the entire reservoir, based on a triangular wedge shape, rather than on the shapes of four blocks. This would have imparted more error rather than less, because the shape of the basin varies so greatly from block to block. The four-block method addresses those changes from block to block.

Step 2 was categorizing each logged layer of soil or rock into Units 1, 2, 3, 4, or 5. As described in text under section 6.1, Unit 1 is the uppermost layer of coarse material deposited as fluvial stream flows, and that is still being added to and reworked in storms flows. Unit 2 is a reservoir pool deposit, largely of silty sand, with a few layers of silty material and gravelly materials included. Unit 3 is a deep-water reservoir pool deposit consisting of silts and clays. Unit 4 is the pre-reservoir alluvium. Unit 5 is sandstone bedrock. The logs of Attachment A and the materials classification table of Attachment B both show the categorization of each logged material as one of these five Units.

Step 3 was determining an average percentage that each of Units 1, 2, and 3 occupy in each of Blocks 1, 2, 3, and 4[18]. Boring logs from each Block, including both the USACE-Geotech borings and those from Law / Crandall were used to determine upper and lower boundaries of Units 1, 2, and 3 in each boring. A thickness was calculated of each Unit in each boring. The *percentage* of the volume each block that is represented by each Unit was determined by averaging the thicknesses measured in each borehole for each Unit. The determined percentages (adjusted to an even 100%) were applied to the non-rounded total volumes for each block as in the in the chart above. The result was an estimated volume for each Unit in each Block. The calculations are in supporting Excel spreadsheet named ***pct thickness.xls***. Results are summarized in the chart below.

	Unit 1 (cu yds)	Unit 2 (cu yds)	Unit 3 (cu yds)	Per block totals	Per block totals, rounded
Block 1	29,280	63,861	107,894	201,035	200,000
Block 2	128,667	212,058	121,275	462,000	460,000
Block 3	39,992	63,346	0	103,338	100,000
Block 4	10,382	0	3,507	13,889	10,000
per Unit totals	208,320	339,265	232,676	780,262	780,000
per Unit totals, rounded	200,000	340,000	230,000		

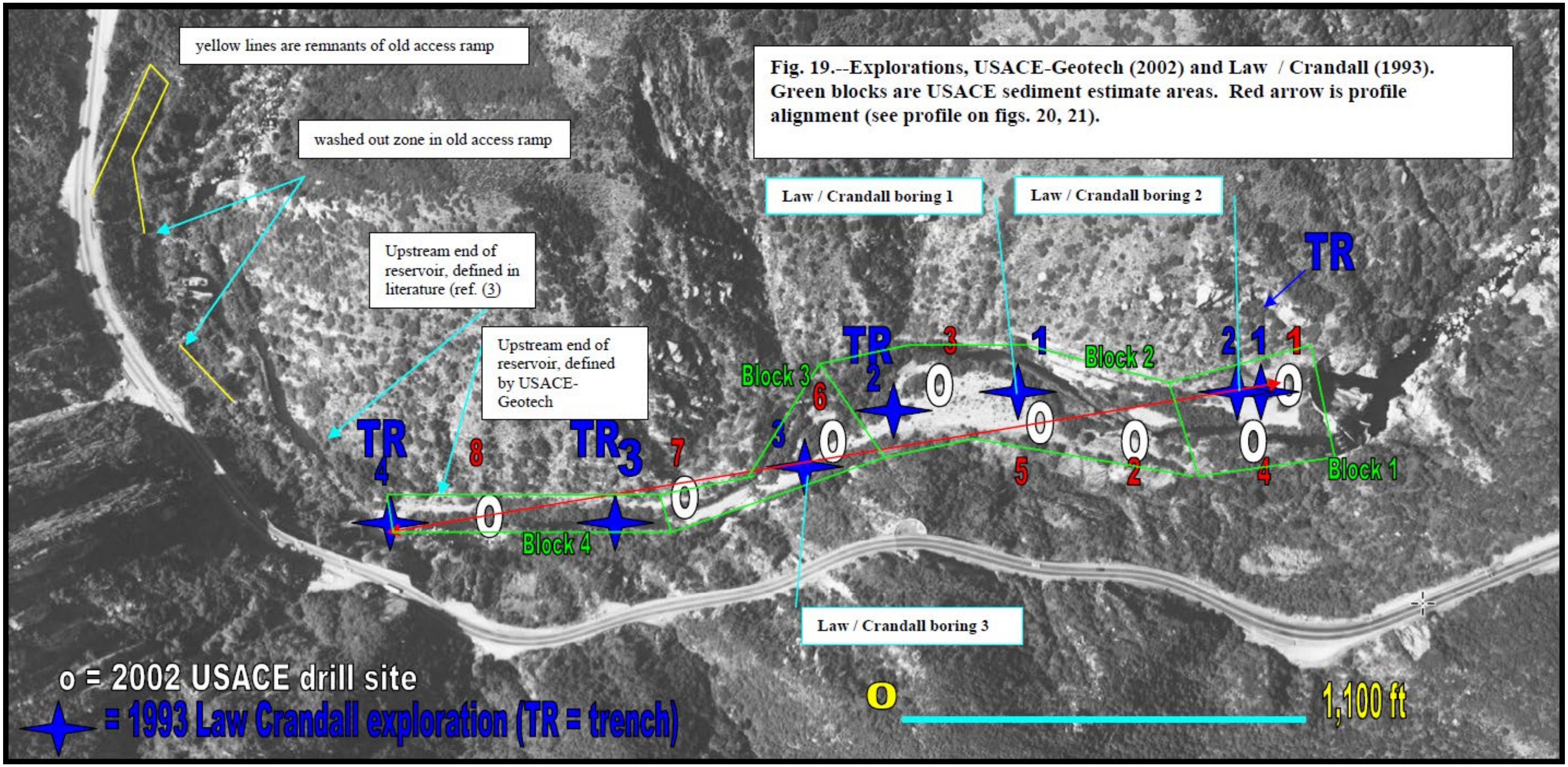
18 As mentioned in preceding text, Units 4 and 5 are considered as material that will and should be left in place. No excavation or movement of Unit 4 or Unit 5 materials is expected.

Step 4 was determining the *weighted average* percentage of sand, silt, etc., in each of Units 1, 2, and 3. For this step, only the classification results and logs of the eight USACE-Geotech borings were used. No detailed mechanical analysis classification results were available for the Law / Crandall work (their three borings). One average percentage composition, all blocks, (gravel- sand-silt-clay, etc.) was determined for Unit 1, 2, and 3. This percentage was averaged for all the blocks, rather than having a separate percentage of each material size for each block because the samples collected were small and represent a small percentage of the total mass. The averaging throughout all four blocks will serve to reduce the impacts of variance. The results of these calculations are in the supporting Excel spreadsheet named ***exploration summary.xls***. Results are summarized in the chart below.

	Unit 1	Unit 2	Unit 3
Percent sand	51% (see note 1)	73%	22%
Percent silt and clay	4% (see note 1)	22%	78%
Percent other material sizes	45% (see note 1)	5%	<1%

Symbol used: "<" = "less than".

Note 1: these percentages do not take into account the cobbles and larger stone in the Unit. While the laboratory mechanical analyses, averaged, suggest 45% gravel, the larger stone was excluded from the sampler, due to large size of the material and small size of the sampler. Some gravel also had to have been excluded from the samples due to the small sampler orifice. So, the 45% number for gravel content has to be considered too low. An estimate of average percentage cobbles and larger material, based on visual observations, is 25 to 30% in Unit 1. Thus, at a minimum, Unit 1 is 70% gravel and larger material and no more than 30% sand and smaller material.



yellow lines are remnants of old access ramp
washed out zone in old access ramp
Upstream end of reservoir, defined in literature (ref. (3)
Upstream end of reservoir, defined by USACE-Geotech
Fig. 19.--Explorations, USACE-Geotech (2002) and Law / Crandall (1993). Green blocks are USACE sediment estimate areas. Red arrow is profile alignment (see profile on figs. 20, 21).
Law / Crandall boring 1
Law / Crandall boring 2
Law / Crandall boring 3
TR
TR 2
TR 3
TR 4
Block 1
Block 2
Block 3
Block 4
o = 2002 USACE drill site
= 1993 Law Crandall exploration (TR = trench)
0
1,100 ft

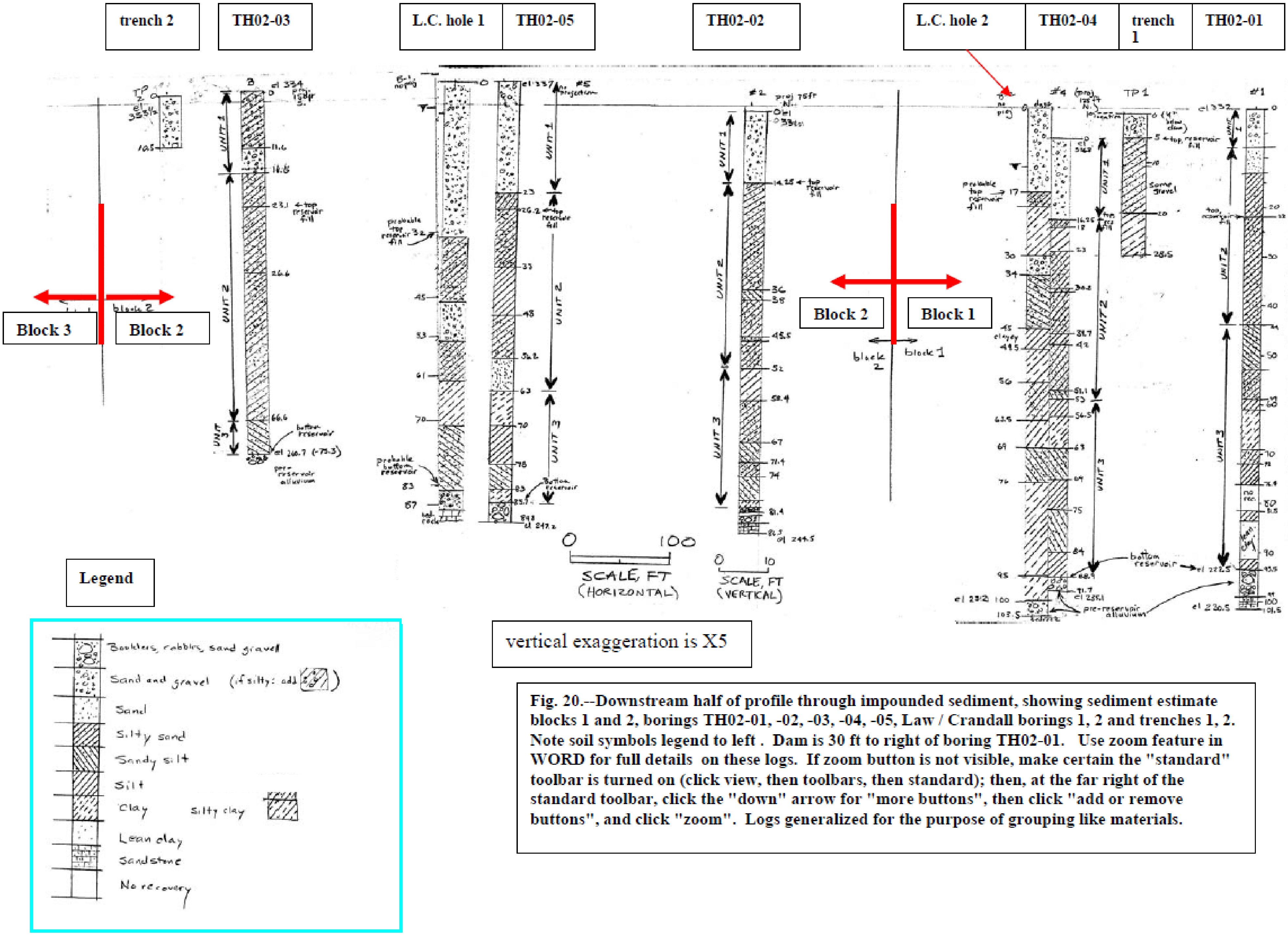
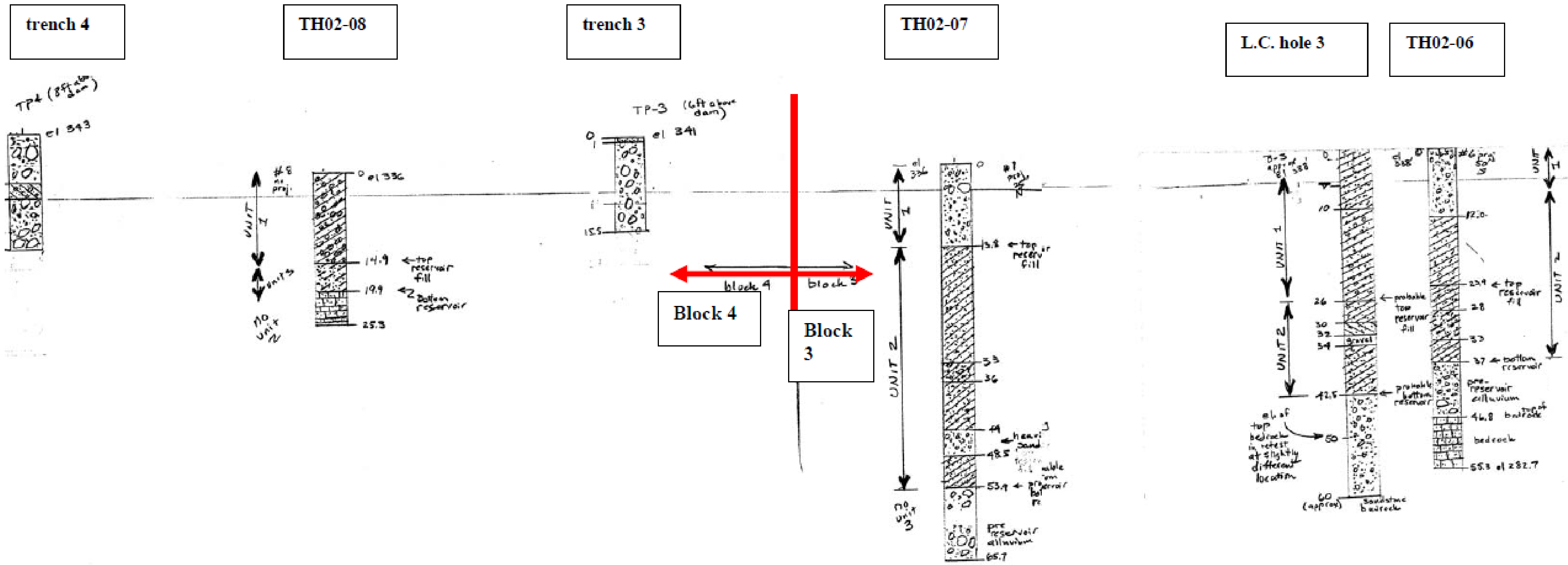
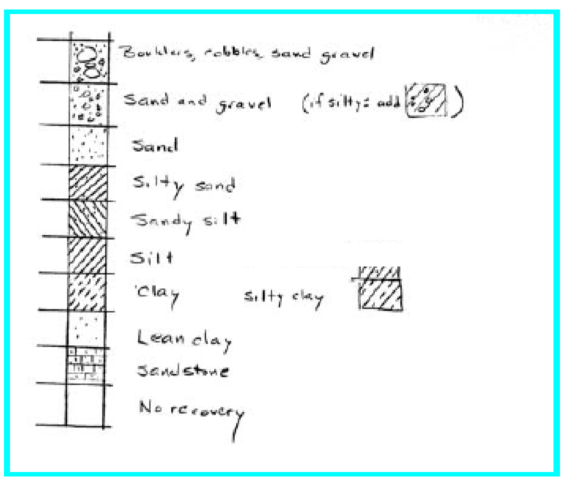


Fig. 20.--Downstream half of profile through impounded sediment, showing sediment estimate blocks 1 and 2, borings TH02-01, -02, -03, -04, -05, Law / Crandall borings 1, 2 and trenches 1, 2. Note soil symbols legend to left . Dam is 30 ft to right of boring TH02-01. Use zoom feature in WORD for full details on these logs. If zoom button is not visible, make certain the "standard" toolbar is turned on (click view, then toolbars, then standard); then, at the far right of the standard toolbar, click the "down" arrow for "more buttons", then click "add or remove buttons", and click "zoom". Logs generalized for the purpose of grouping like materials.



Legend



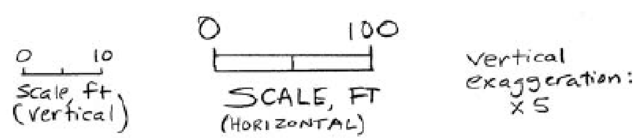


Fig. 21.--Upstream half of profile through impounded sediment, showing sediment estimate blocks 3 and 4, borings TH02-06, -07, -08, Law / Crandall boring 3 and trenches 3, 4. Note soil symbols legend to left . Use zoom feature in WORD for full details on these logs. If zoom button is not visible, make certain the "standard" toolbar is turned on (click view, then toolbars, then standard); then, at the far right of the standard toolbar, click the "down" arrow for "more buttons", then click "add or remove buttons", and click "zoom". Logs generalized for the purpose of grouping like materials. Full detail on log descriptions in Attachment A. Profile line on fig. 19.

CIVIL DESIGN (18 Jan 2013)

Development of the Spreadsheet

The spreadsheet was created to approximately estimate the volume of sediment removed from behind Rindge Dam and the quality of the material removed based on information provided in Attachment D of the Geotechnical Appendix for the f-4 Milestone report dated August 2008. Attachment D makes the following key assumptions to calculate the volume of sediment:

1.) Malibu Creek is divided into 4 "Blocks". Each block is a box shape with a constant width and depth along the creek. Each block width is based on the average canyon width along the creek and each blocks depth is based on borings collected within the creek. The blocks are defined by stationing along the creek, where station 0 is at the dam and station 2545 is 2,545 feet from the dam. The following table summarizes each blocks characteristics:

Block Characteristics

		Starting Station	End Station	Total Length (FT)		Block Width (FT)		Block Depth (FT)
Block 1		0	330	330		250		94.13
Block 2		330	1155	825		270		81.23
Block 3		1155	2045	890		175		44.47
Block 4		2045	2545	500		75		19.90

2.) The canyon side slopes are accounted for with a reduction factor. Due to lack of reliable topographic information about the canyon, the reduction in canyon width as the creek is excavated is estimated by a volume reduction factor. These factors were developed based on typical geological characteristics for the region and reflect typical canyon side slopes. They represent a percent reduction in the total block volume to determine the final amount of material excavated from the canyon. The following table shows the reduction factors used for each Block:

Side Slope Reduction Factors

Block 1		30%
Block 2		30%
Block 3		50%
Block 4		50%

3.) Each "Block" is divided into 3 "Units" to determine the type of material being excavated. Units represent like materials in each block and reflect general categories of material impounded behind the dam. The Unit composition is based on information from borings taken along the creek. Each unit is broken down into "Sand"," Silt and Sand", and "Other".
Attachment D defines "Other" as cobble and larger material. The following table summarizes the characteristics of each Unit:

Unit Characteristics

		Material Layer		Description		% Sand	%Silt and Clay	%Other
Unit 1		Fluvial deposition (i.e., not deposited in a reservoir pool)		Sand, gravel, cobbles, and larger rock		28	2	70
Unit 2		Shallow to intermediate depths reservoir pool deposition		Mainly silty sands with organic content; does contain silt layers, some gravel		73	22	5
Unit 3		Deeper depths of reservoir pool deposition		Sandy silts, lean clays, and silts (all with organic content); does contain some silty sand layers		72	78	0

4.) Unit depths vary between Blocks and are based on average depths of material found in borings along the creek. The following Table summarizes the depth of each unit by Block:

Unit Depth, in Feet, By Block

		Unit 1	Unit 2	Unit 3	
Block 1		13.75	29.28	51.1	
Block 2		21.81	39.09	20.33	
Block 3		17.27	27.20	0	
Block 4		14.90	0	5	

Spreadsheet Calculations

The spreadsheet calculates volumes using cross sections that are based on the geometry of each Block, the material characteristics of each Unit, and the reduction factor for each Block. The depths of each Unit were taken directly from Attachment D while the cross section width was calculated with the following steps:

1. Find the volume of the entire Block
2. Reduce the volume of the Block by the appropriate factor
3. Divide the reduced volume by the length of the Block to get a cross sectional area
4. Divide the cross sectional area by the depth of the Block to get the width of the Block

The following table summarizes the cross sections widths used to calculate the volume of material at different depths:

Cross Section Widths by Block

		Width (ft)
Block 1		175
Block 2		189
Block 3		71.25
Block 4		37.50

The spreadsheet develops a cross section at the start and end of each Block and every 100 feet along the creek, starting at Rindge Dam. To calculate volumes, the spread sheet takes two cross sections, averages the area of the two, and multiples the average area by the distance between the two cross sections. This is done between every cross section along the creek and the calculated volumes are added up to determine the total volume of material removed. This method of calculating volume is called the Average Ends Method. As the excavation depth changes, the areas of the cross sections change and cross sections from two different blocks are not used in the same calculation.

Once the amount of material excavated is determined, the spreadsheet then converts the volumes into the amount of Sands, Silts and Clays, and Other based on the characteristics of each Unit. This breakdown is reported in the Materials Summary Table within the spreadsheet.

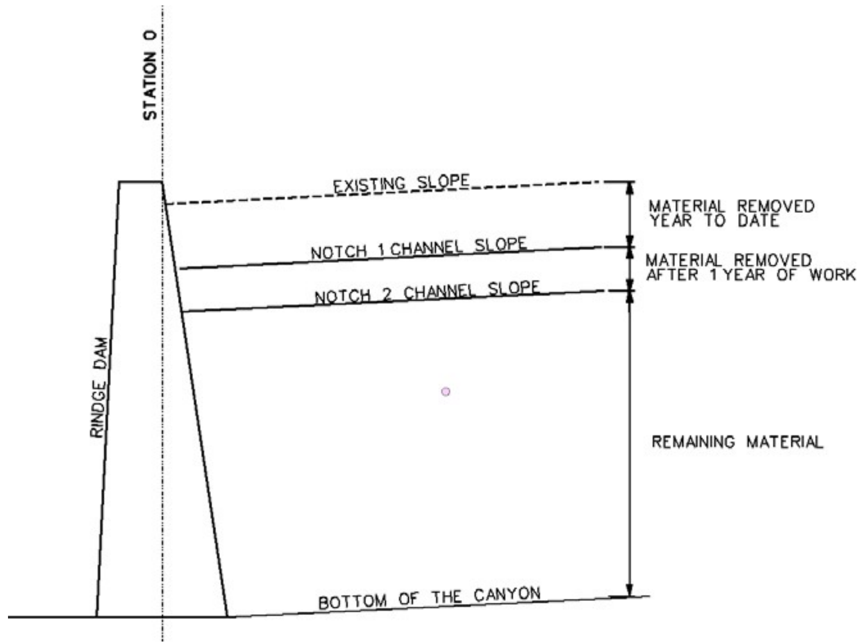
The spreadsheet was developed to compare different creek bed profiles to help evaluate the amount of material removed over different time periods. These profiles are represented in the Channel Profiles Table within the spread sheet. The Chanel Profiles Table includes the following three profiles:

- Existing Slope – Establishes the existing elevations along the creek bed. These elevations should not change while using the spreadsheet since they are used to determine where each Unit begins and ends. The spreadsheet is formatted so the elevation of the creek does not have to be held constant.

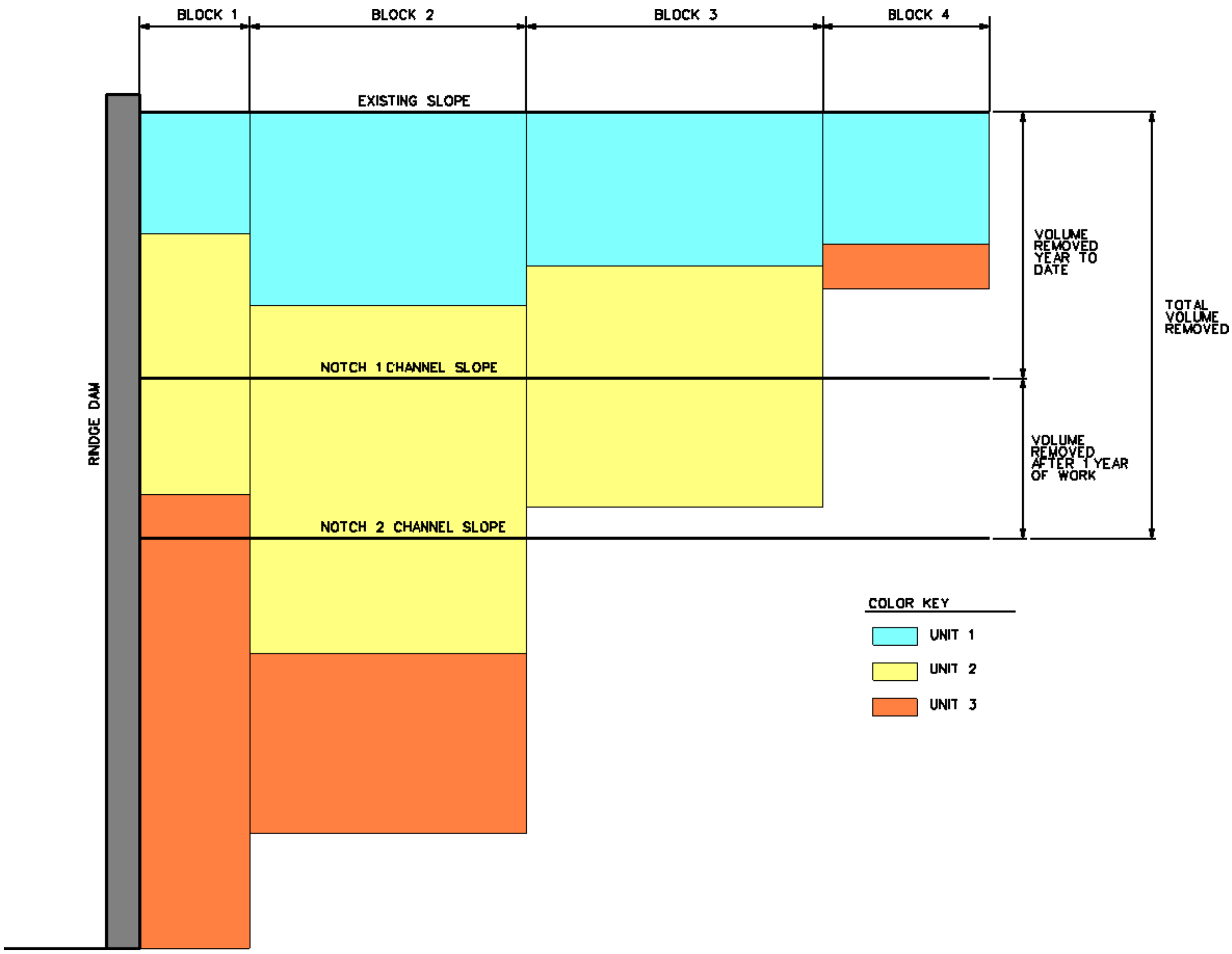
- Notch 1 – This profile represents the creek at the beginning of a year of work. At Year 1 the elevations will match the Existing Slope Profile, at Year 2 the elevations should match the elevations along the creek after excavation. This profile is compared to the Existing Slope Profile to determine excavated quantities and represents total excavation year to date.

- Notch 2 – This profile represents the stream after a year of excavation work. This profile is compared to the Notch 1 profile to determine the amount of material excavated after a year of excavation.

The image below illustrates the three profiles used in the spreadsheet.



The image below is a graphical illustration of how Blocks and Units relate to the three profiles used in volume calculations.



The profiles are manipulated by changing the invert elevation at Station 0 (the face of the dam) and the creek slope every 100 ft moving upstream from the dam.

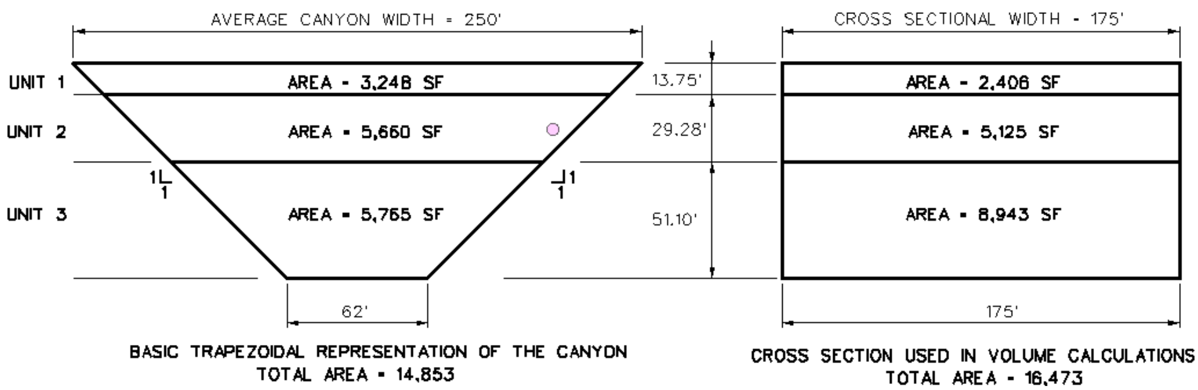
The final results of the volume calculations are reported in the Materials Summary Table in the spreadsheet. This table states the total volume removed from behind the dam and breaks the volume down into how much of that material is Sands, Silts and Clays, and Other, and also states how much is remaining behind the dam breaks the remaining volume down into how much of that material is Sands, Silts and Clays, and Other. The Table below is an example of the results from the spreadsheet after a excavating 35 feet below the Existing Slope.

Materials Summary					
	Volume Removed	% Removed	Volume Remaining	% Remaining	Estimated Total
Sands	272,987	74.68	92,542	25.32	365,529
Silts and Clays	82,102	31.44	179,032	68.56	261,134
Other	158,858	98.06	3,147	1.94	162,006
Total	513,948	65.17	274,721	34.83	788,669

Limitations of the Spread Sheet

The blocks used for volume calculations transform the unknown trapezoidal geometry of the creek canyon into a simplified rectangular shape to calculate the amount of material removed at a given depth. Due to this transformation, the spread sheet underestimates the volume removed for excavation depths closer to the top of the dam while over estimating the volume removed for excavation depths closer to the creek bed.

This figure below compares a basic trapezoidal representation of the canyon to the cross section of Block 1 used in the calculations in the spreadsheet. This figure uses Unit depths from the spread sheet for both the basic trapezoidal shape and the rectangular shape. Note, volume is the product of area multiplied by the length of the block; since the length of the block is constant, the figure below only compares areas of each unit.



The figure above shows that the spreadsheet could actually over estimate the total volume removed from behind the dam. This is not a problem because the side slopes and width of the canyon vary so much that the methods used in the spread sheet ultimately reflect average conditions and calculates a reasonable estimate for the total material removed from behind the dam.

The depths and volumes of material calculated by the spreadsheet are estimations based on limited available data. Actual depths will vary and ultimately be controlled by conditions at the site. Volumes may change as more information about the material behind the dam becomes available a

INPUT:
Channel Profiles

Channel Profiles							
Station	Distance Upstream from the Dam (FT)	Existing Slope (%)	Existing Invert Elevation	Notch 1 Channel Slope (%)	Notch 1 Channel Invert	Notch 2 Channel Slope (%)	Notch 2 Channel Invert
0	0	0	94	0	33	0	0
100	100	0	94	0	33	0	0
200	200	0	94	0	33	0	0
300	300	0	94	0	33	0	0
330	330	0	94	0	33	0	0
400	400	0	94	0	33	0	0
500	500	0	94	0	33	0	0
600	600	0	94	0	33	0	0
700	700	0	94	0	33	0	0
800	800	0	94	0	33	0	0
900	900	0	94	0	33	0	0
1000	1000	0	94	0	33	0	0
1100	1100	0	94	0	33	0	0
1155	1155	0	94	0	33	0	0
1200	1200	0	94	0	33	0	0
1300	1300	0	94	0	33	0	0
1400	1400	0	94	0	33	0	0
1500	1500	0	94	0	33	0	0
1600	1600	0	94	0	33	0	0
1700	1700	0	94	0	33	0	0
1800	1800	0	94	0	33	0	0
1900	1900	0	94	0	33	0	0
2000	2000	0	94	0	33	0	0
2045	2045	0	94	0	33	0	0
2100	2100	0	94	0	33	0	0
2200	2200	0	94	0	33	0	0
2300	2300	0	94	0	33	0	0
2400	2400	0	94	0	33	0	0
2500	2500	0	94	0	33	0	0
2545	2545	0	94	0	33	0	0

NOTE: ONLY CHANGE VALUES IN SHADED CELLS

Materials Summary

Materials Summary					
	Volume Removed	% Removed	Volume Remainin	% Remaining	Estimated Total
Sands	365,529	100.00	0	0.00	365,529
Silts and Clays	261,134	100.00	0	0.00	261,134
Other	162,006	100.00	0	0.00	162,006
Total	788,669	100.00	0	0.00	788,669

About this Spreadsheet

This spread sheet calculates the volume of material removed based on 3 separate profiles:
an existing profile, a profile for one dam notch and a profile for a second dam notch below the first.
The methodology used to calculate all volumes is based on the methods outlined in Attachment D
of the Geotechnical Appendix for the F-4 Milestone dated August 2008.
Block and Unit Definitions are taken from the same appendix.
All Sections are based on a reduced area determined using values
for volume reduction taken from the Geotechnical Appendix which account for the canyon side slopes.
Slopes are taken going upstream of the dam.
All Volumes are in Cubic Yards
Please do not change any values on the Volume Calculations Tab.
If you have any questions, or there is a problem with the calculations, contact Larry Walsh at x3634 or lawrence.f.walsh@usace.army.mil

Calculations Check
Good

Volumes Removed

Volumes Removed			Unit 1				Unit 2				Unit 3			
Station Range			Total Volume Removed	Volume of Sands	Volume of Silts and Clays	Volume of Other	Total Volume Removed	Volume of Sands	Volume of Silts and Clays	Volume of Other	Total Volume Removed	Volume of Sands	Volume of Silts and Clays	Volume of Other
Between Existing Channel and Notch 1														
Block 1	0	330	29,410	8,235	588	20,587	62,634	45,723	13,779	0	38,429	8,454	29,974	0
Block 2	330	1155	125,967	35,271	2,519	88,177	225,730	164,783	49,661	3,132	578	127	450	0
Block 3	1155	2045	40,552	11,355	811	28,387	63,882	46,634	14,054	11,287	0	0	0	0
Block 4	2045	2545	10,347	2,897	207	7,243	0	0	0	3,194	3,472	764	2,708	0
Between Notch 1 and Notch 2														
Block 1	0	330	0	0	0	0	0	0	0	0	70,869	15,591	55,277	0
Block 2	330	1155	0	0	0	0	0	0	0	0	116,799	25,696	91,104	0
Block 3	1155	2045	0	0	0	0	0	0	0	0	0	0	0	0
Block 4	2045	2545	0	0	0	0	0	0	0	0	0	0	0	0
Total Removed-Between Existing and Notch 2														
Block 1	0	330	29,410	8,235	588	20,587	62,634	45,723	13,779	0	109,297	24,045	85,252	0
Block 2	330	1155	125,967	35,271	2,519	88,177	225,730	164,783	49,661	3,132	117,377	25,823	91,554	0
Block 3	1155	2045	40,552	11,355	811	28,387	63,882	46,634	14,054	11,287	0	0	0	0
Block 4	2045	2545	10,347	2,897	207	7,243	0	0	0	3,194	3,472	764	2,708	0
		Unit Total	206,276	57,757	4,126	144,394	352,246	257,139	77,494	17,612	230,146	50,632	179,514	0

Volumes Remaining			Unit 1				Unit 2				Unit 3			
Station Range			Total Volume Remaining	Volume of Sands	Volume of Silts and Clays	Volume of Other	Total Volume Remaining	Volume of Sands	Volume of Silts and Clays	Volume of Other	Total Volume Remaining	Volume of Sands	Volume of Silts and Clays	Volume of Other
Total Remaining After Notch 1														
Block 1	0	330	0	0	0	0	0	0	0	0	70,869	15,591	55,277	0
Block 2	330	1155	0	0	0	0	0	0	0	0	116,799	25,696	91,104	0
Block 3	1155	2045	0	0	0	0	0	0	0	0	0	0	0	0
Block 4	2045	2545	0	0	0	0	0	0	0	0	0	0	0	0
Total Remaining After Notch 2														
Block 1	0	330	0	0	0	0	0	0	0	0	0	0	0	0
Block 2	330	1155	0	0	0	0	0	0	0	0	0	0	0	0
Block 3	1155	2045	0	0	0	0	0	0	0	0	0	0	0	0
Block 4	2045	2545	0	0	0	0	0	0	0	0	0	0	0	0
Total Remaining			0	0	0	0	0	0	0	0	0	0	0	0

Volumes Remaining

Unit Definition

Unit Definition		
	Material Layer	Description
Unit 1	Fluvial deposition (i.e., not deposited in a reservoir pool)	Sand, gravel, cobbles, and larger rock
Unit 2	Shallow to intermediate depths reservoir pool deposition	Mainly silty sands with organic content; does contain silt layers, some gravel
Unit 3	Deeper depths of reservoir pool deposition	Sandy silts, lean clays, and silts (all with organic content); does contain some silty sand layers

Unit Composition			
	Unit 1	Unit 2	Unit 3
% Sand	28	73	22
% Silt and Clay	2	22	78
% Other	70	5	0

Unit Composition

Typical Sections Used for Calculations

Typical Sections Used for Calculations			
Block 1		Width	Depth
	UNIT 1	175.00	13.75
	UNIT 2	175.00	29.28
	UNIT 3	175.00	51.10
Block 2		Width	Depth
	UNIT 1	189.00	21.81
	UNIT 2	189.00	39.09
	UNIT 3	189.00	20.33
Block 3		Width	Depth
	UNIT 1	71.25	17.27
	UNIT 2	71.25	27.20
	UNIT 3	0.00	0.00
Block 4		Width	Depth
	UNIT 1	37.50	14.90
	UNIT 2	0.00	0.00
	UNIT 3	37.50	5.00

PROFILES:

Existing Channel Profile

	Existing Channel Profile									
	Distance Upstream from the Dam	Existing Slope	Existing Invert Elevation	Top of Unit 1 Elevation	Bottom of Unit 1 Elevation	Top of Unit 2 Elevation	Bottom of Unit 2 Elevation	Top of Unit 3 Elevation	Bottom of Unit 3 Elevation
Block 1	0	0	94	94	80.25	80.25	50.97	50.97	-0.13
Block 1	100	0	94	94	80.25	80.25	50.97	50.97	-0.13
Block 1	200	0	94	94	80.25	80.25	50.97	50.97	-0.13
Block 1	300	0	94	94	80.25	80.25	50.97	50.97	-0.13
Block 1	330	0	94	94	80.25	80.25	50.97	50.97	-0.13
Block 2	330	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 2	400	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 2	500	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 2	600	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 2	700	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 2	800	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 2	900	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 2	1000	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 2	1100	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 2	1155	0	94	94	72.19	72.19	33.10	33.10	12.78
Block 3	1155	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	1200	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	1300	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	1400	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	1500	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	1600	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	1700	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	1800	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	1900	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	2000	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 3	2045	0	94	94	76.73	76.73	49.53	49.53	49.53
Block 4	2045	0	94	94	79.10	79.10	79.10	79.10	74.10
Block 4	2100	0	94	94	79.10	79.10	79.10	79.10	74.10
Block 4	2200	0	94	94	79.10	79.10	79.10	79.10	74.10
Block 4	2300	0	94	94	79.10	79.10	79.10	79.10	74.10
Block 4	2400	0	94	94	79.10	79.10	79.10	79.10	74.10
Block 4	2500	0	94	94	79.10	79.10	79.10	79.10	74.10
Block 4	2545	0	94	94	79.10	79.10	79.10	79.10	74.10

Notch 1 Profile

Notch 1 Profile							
Notch 1 Channel Slope	Notch 1 Channel Invert	Top of Unit 1 Elevation	Bottom of Unit 1 Elevation	Top of Unit 2 Elevation	Bottom of Unit 2 Elevation	Top of Unit 3 Elevation	Bottom of Unit 3 Elevation
0	33	0	0.00	0.00	0.00	33.00	-0.13
0	33	0	0.00	0.00	0.00	33.00	-0.13
0	33	0	0.00	0.00	0.00	33.00	-0.13
0	33	0	0.00	0.00	0.00	33.00	-0.13
0	33	0	0.00	0.00	0.00	33.00	-0.13
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	33.00	12.78
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00
0	33	0	0.00	0.00	0.00	0.00	0.00

Notch 2 Profile

Notch 2 Profile							
Notch 2 Channel Slope	Notch 2 Channel Invert	Top of Unit 1 Elevation	Bottom of Unit 1 Elevation	Top of Unit 2 Elevation	Bottom of Unit 2 Elevation	Top of Unit 3 Elevation	Bottom of Unit 3 Elevation
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00
0	0	0	0.00	0.00	0.00	0.00	0.00

UNIT INFORMATION:

Unit Information													
	Length	Assumed Depth	Unit 1 Ave. Depth	Unit 2 Ave. Depth	Unit 3 Ave. Depth	Average Width	Volume Reduction	Volume (CY)	Volume (CF)	Unit 1 Volume (CY)	Unit 2 Volume (CY)	Unit 3 Volume (CY)
Block 1	330	94	13.75	29.28	51.10	250	30	201,055	5,428,485	29,410	62,633.80	109,297.22
Block 2	825	80	21.81	39.09	20.33	270	30	462,000	12,474,000	125,967	225,730.31	117,376.88
Block 3	890	44	17.27	27.20	0.00	125	50	103,338	2,790,126	40,552	63,881.67	0.00
Block 4	500	20	14.90	0.00	5.00	75	50	13,889	375,003	10,347	0.00	3,472.22
									Unit Totals	206,276	352,246	230,146

VOLUME CALCULATIONS:

Existing Channel vs Notch 1 Calculations

Existing Channel vs. Notch 1 Calculations

	Distance Upstream from the Dam	(Existing Invert Elevation)-(Notch 1 Channel Invert)	Unit 1 X-Sec Area	Unit 1 Width	Unit 1 Depth Remaining	Unit 1 Area Removed	Unit 1 Area Remaining	Unit 2 X-Sec Area	Unit 2 Width	Unit 2 Depth Remaining	Unit 2 Area Removed	Unit 2 Area Remaining	Unit 3 X-Sec Area	Unit 3 Width	Unit 3 Depth Remaining	Unit 3 Area Removed	Unit 3 Area Remaining
Block 1	0	61	2,406.25	175.00	0.00	2,406.25	0.00	5,124.58	175.00	0.00	5,124.58	0.00	8,942.50	175.00	33.13	3,144.17	5,798.33
Block 1	100	61	2,406.25	175.00	0.00	2,406.25	0.00	5,124.58	175.00	0.00	5,124.58	0.00	8,942.50	175.00	33.13	3,144.17	5,798.33
Block 1	200	61	2,406.25	175.00	0.00	2,406.25	0.00	5,124.58	175.00	0.00	5,124.58	0.00	8,942.50	175.00	33.13	3,144.17	5,798.33
Block 1	300	61	2,406.25	175.00	0.00	2,406.25	0.00	5,124.58	175.00	0.00	5,124.58	0.00	8,942.50	175.00	33.13	3,144.17	5,798.33
Block 1	330	61	2,406.25	175.00	0.00	2,406.25	0.00	5,124.58	175.00	0.00	5,124.58	0.00	8,942.50	175.00	33.13	3,144.17	5,798.33
Block 2	330	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 2	400	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 2	500	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 2	600	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 2	700	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 2	800	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 2	900	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 2	1000	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 2	1100	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 2	1155	61	4,122.56	189.00	0.00	4,122.56	0.00	7,387.54	189.00	0.00	7,387.54	0.00	3,841.43	189.00	20.23	18.90	3,822.53
Block 3	1155	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1200	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1300	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1400	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1500	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1600	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1700	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1800	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1900	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	2000	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	2045	61	1,230.24	71.25	0.00	1,230.24	0.00	1,937.98	71.25	0.00	1,937.98	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 4	2045	61	558.75	37.50	0.00	558.75	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!	187.50	37.50	0.00	187.50	0.00
Block 4	2100	61	558.75	37.50	0.00	558.75	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!	187.50	37.50	0.00	187.50	0.00
Block 4	2200	61	558.75	37.50	0.00	558.75	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!	187.50	37.50	0.00	187.50	0.00
Block 4	2300	61	558.75	37.50	0.00	558.75	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!	187.50	37.50	0.00	187.50	0.00
Block 4	2400	61	558.75	37.50	0.00	558.75	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!	187.50	37.50	0.00	187.50	0.00
Block 4	2500	61	558.75	37.50	0.00	558.75	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!	187.50	37.50	0.00	187.50	0.00
Block 4	2545	61	558.75	37.50	0.00	558.75	0.00	0.00	#DIV/0!	0.00	#DIV/0!	#DIV/0!	187.50	37.50	0.00	187.50	0.00

Notch 1 vs Notch 2 Calculations

Notch 1 vs. Notch 2 Calculations

	Distance Upstream from the Dam	(Notch 1 Channel Invert)-(Notch 2 Channel Invert)	Unit 1 X-Sec Area	Unit 1 Width	Unit 1 Depth Remaining	Unit 1 Area Removed	Unit 1 Area Remaining	Unit 2 X-Sec Area	Unit 2 Width	Unit 2 Depth Remaining	Unit 2 Area Removed	Unit 2 Area Remaining	Unit 3 X-Sec Area	Unit 3 Width	Unit 3 Depth Remaining	Unit 3 Area Removed	Unit 3 Area Remaining
Block 1	0	33	0.00	175.00	0.00	0.00	0.00	0.00	175.00	0.00	0.00	0.00	5,798.33	175.00	0.00	5,798.33	0.00
Block 1	100	33	0.00	175.00	0.00	0.00	0.00	0.00	175.00	0.00	0.00	0.00	5,798.33	175.00	0.00	5,798.33	0.00
Block 1	200	33	0.00	175.00	0.00	0.00	0.00	0.00	175.00	0.00	0.00	0.00	5,798.33	175.00	0.00	5,798.33	0.00
Block 1	300	33	0.00	175.00	0.00	0.00	0.00	0.00	175.00	0.00	0.00	0.00	5,798.33	175.00	0.00	5,798.33	0.00
Block 1	330	33	0.00	175.00	0.00	0.00	0.00	0.00	175.00	0.00	0.00	0.00	5,798.33	175.00	0.00	5,798.33	0.00
Block 2	330	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 2	400	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 2	500	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 2	600	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 2	700	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 2	800	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 2	900	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 2	1000	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 2	1100	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 2	1155	33	0.00	189.00	0.00	0.00	0.00	0.00	189.00	0.00	0.00	0.00	3,822.53	189.00	0.00	3,822.53	0.00
Block 3	1155	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1200	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1300	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1400	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1500	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1600	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1700	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1800	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	1900	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	2000	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 3	2045	33	0.00	71.25	0.00	0.00	0.00	0.00	71.25	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!
Block 4	2045	33	0.00	37.50	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!	0.00	37.50	0.00	0.00	0.00
Block 4	2100	33	0.00	37.50	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!	0.00	37.50	0.00	0.00	0.00
Block 4	2200	33	0.00	37.50	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!	0.00	37.50	0.00	0.00	0.00
Block 4	2300	33	0.00	37.50	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!	0.00	37.50	0.00	0.00	0.00
Block 4	2400	33	0.00	37.50	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!	0.00	37.50	0.00	0.00	0.00
Block 4	2500	33	0.00	37.50	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!	0.00	37.50	0.00	0.00	0.00
Block 4	2545	33	0.00	37.50	0.00	0.00	0.00	#DIV/0!	#DIV/0!	0.00	#DIV/0!	#DIV/0!	0.00	37.50	0.00	0.00	0.00

Note: All Lengths are in Ft and all Areas are in SQFT

Existing Channel vs Notch 1 Volume Calculations

Existing Channel vs. Notch 1 Volume Calculations	Unit 1 Volume Removed (CF)	Unit 1 Volume Removed (CY)	Unit 1 Volume Remaining (CF)	Unit 1 Volume Remaining(CY)	Unit 2 Volume Removed (CF)	Unit 2 Volume Removed (CY)	Unit 2 Volume Remaining (CF)	Unit 2 Volume Remaining (CY)	Unit 3 Volume Removed (CF)	Unit 3 Volume Removed (CY)	Unit 3 Volume Remaining (CF)	Unit 3 Volume Remaining (CY)
Block 1	240,625.00	8,912.04	0.00	0	512,458.33	18,979.94	0.00	0.00	314,416.67	11,645.06	579,833.33	21,475.31
Block 1	240,625.00	8,912.04	0.00	0	512,458.33	18,979.94	0.00	0.00	314,416.67	11,645.06	579,833.33	21,475.31
Block 1	240,625.00	8,912.04	0.00	0	512,458.33	18,979.94	0.00	0.00	314,416.67	11,645.06	579,833.33	21,475.31
Block 1	72,187.50	2,673.61	0.00	0	153,737.50	5,693.98	0.00	0.00	94,325.00	3,493.52	173,950.00	6,442.59
								0.00				
Block 2	288,579.38	10,688.13	0.00	0	517,127.63	19,152.88	0.00	0.00	1,323.00	49.00	267,576.75	9,910.25
Block 2	412,256.25	15,268.75	0.00	0	738,753.75	27,361.25	0.00	0.00	1,890.00	70.00	382,252.50	14,157.50
Block 2	412,256.25	15,268.75	0.00	0	738,753.75	27,361.25	0.00	0.00	1,890.00	70.00	382,252.50	14,157.50
Block 2	412,256.25	15,268.75	0.00	0	738,753.75	27,361.25	0.00	0.00	1,890.00	70.00	382,252.50	14,157.50
Block 2	412,256.25	15,268.75	0.00	0	738,753.75	27,361.25	0.00	0.00	1,890.00	70.00	382,252.50	14,157.50
Block 2	412,256.25	15,268.75	0.00	0	738,753.75	27,361.25	0.00	0.00	1,890.00	70.00	382,252.50	14,157.50
Block 2	412,256.25	15,268.75	0.00	0	738,753.75	27,361.25	0.00	0.00	1,890.00	70.00	382,252.50	14,157.50
Block 2	412,256.25	15,268.75	0.00	0	738,753.75	27,361.25	0.00	0.00	1,890.00	70.00	382,252.50	14,157.50
Block 2	226,740.94	8,397.81	0.00	0	406,314.56	15,048.69	0.00	0.00	1,039.50	38.50	210,238.88	7,786.63
								0.00				
Block 3	55,360.77	2,050.40	0.00	0	87,209.25	3,229.97	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	123,023.94	4,556.44	0.00	0	193,798.33	7,177.72	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	123,023.94	4,556.44	0.00	0	193,798.33	7,177.72	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	123,023.94	4,556.44	0.00	0	193,798.33	7,177.72	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	123,023.94	4,556.44	0.00	0	193,798.33	7,177.72	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	123,023.94	4,556.44	0.00	0	193,798.33	7,177.72	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	123,023.94	4,556.44	0.00	0	193,798.33	7,177.72	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	123,023.94	4,556.44	0.00	0	193,798.33	7,177.72	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	123,023.94	4,556.44	0.00	0	193,798.33	7,177.72	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	55,360.77	2,050.40	0.00	0	87,209.25	3,229.97	0.00	0.00	0.00	0.00	0.00	0.00
								0.00				
Block 4	30,731.25	1,138.19	0.00	0	0.00	0.00	0.00	0.00	10,312.50	381.94	0.00	0.00
Block 4	55,875.00	2,069.44	0.00	0	0.00	0.00	0.00	0.00	18,750.00	694.44	0.00	0.00
Block 4	55,875.00	2,069.44	0.00	0	0.00	0.00	0.00	0.00	18,750.00	694.44	0.00	0.00
Block 4	55,875.00	2,069.44	0.00	0	0.00	0.00	0.00	0.00	18,750.00	694.44	0.00	0.00
Block 4	55,875.00	2,069.44	0.00	0	0.00	0.00	0.00	0.00	18,750.00	694.44	0.00	0.00
Block 4	25,143.75	931.25	0.00	0	0.00	0.00	0.00	0.00	8,437.50	312.50	0.00	0.00

Notch 1 vs Notch 2 Volume Calculations

Notch 1 vs. Notch 2 Volume Calculations																
	Unit 1 Volume Removed (CF)	Unit 1 Volume Removed (CY)	Unit 1 Volume Remaining (CF)	Unit 1 Volume Remaining(CY)	Unit 2 Volume Removed (CF)	Unit 2 Volume Removed (CY)	Unit 2 Volume Remaining (CF)	Unit 2 Volume Remaining (CY)	Unit 3 Volume Removed (CF)	Unit 3 Volume Removed (CY)	Unit 3 Volume Remaining (CF)	Unit 3 Volume Remaining (CY)
Block 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	579,833.33	21,475.31	0.00	0.00
Block 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	579,833.33	21,475.31	0.00	0.00
Block 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	579,833.33	21,475.31	0.00	0.00
Block 1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	173,950.00	6,442.59	0.00	0.00
Block 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	267,576.75	9,910.25	0.00	0.00
Block 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	382,252.50	14,157.50	0.00	0.00
Block 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	382,252.50	14,157.50	0.00	0.00
Block 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	382,252.50	14,157.50	0.00	0.00
Block 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	382,252.50	14,157.50	0.00	0.00
Block 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	382,252.50	14,157.50	0.00	0.00
Block 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	382,252.50	14,157.50	0.00	0.00
Block 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	382,252.50	14,157.50	0.00	0.00
Block 2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	210,238.88	7,786.63	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Block 4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

VOLUME TOTAL & CHECKS:

Volume Total By Unit

Volume Totals By Unit

	Total Unit 1 Removed (CY)	Total Unit 1 Remaining (CY)	Total Unit 2 Removed (CY)	Total Unit 2 Remaining(CY)	Total Unit 3 Removed (CY)	Total Unit 3 Remaining (CY)
Notch 1 Block 1	29,410	0	62,634	0	38,429	70,869
Notch 1 Block 2	125,967	0	225,730	0	578	116,799
Notch 1 Block 3	40,552	0	63,882	0	0	0
Notch 1 Block 4	10,347	0	0	0	3,472	0
Notch 2 Block 1	0	0	0	0	70,869	0
Notch 2 Block 2	0	0	0	0	116,799	0
Notch 2 Block 3	0	0	0	0	0	0
Notch 2 Block 4	0	0	0	0	0	0
Notch Change Block 1	29,410	0	62,634	0	109,297	0
Notch Change Block 2	125,967	0	225,730	0	117,377	0
Notch Change Block 3	40,552	0	63,882	0	0	0
Notch Change Block 4	10,347	0	0	0	3,472	0

Material Percentage By Unit

	Unit 1	Unit 2	Unit 3
% Sand	28	73	22
% Silt and Clay	2	22	78
% Other	70	5	0

Unit 1 Adjustment

	Original	Adjusted
% Sand	51	28
% Silt and Clay	4	2
% Other	45	70

Calculation Checks

Calculation Checks												
										0	0	0
	Unit 1 Calculated	Unit 1 Actual	Check	Unit 2 Calculated	Unit 2 Actual	Check	Unit 3 Calculated	Unit 3 Actual	Check	0	0	0
Notch 1	206,276	206,276	Good	352,246	352,246	Good	230,146	230,146	Good	0	0	0
Notch 2	206,276	206,276	Good	352,246	352,246	Good	230,146	230,146	Good			0
Total	206,276	206,276	Good	352,246	352,246	Good	230,146	230,146	Good			

Note

Orange Values Taken from Geotechnical Appendix

Values Adjusted to reflect the follow, found on page 143 Note1 of the table in the Geotechnical Appendix," Note 1: these percentages do not take into account the cobbles and larger stone in the Unit. W hile the laboratory mechanical analyses, averaged, suggest 45% gravel, the larger stone was excluded from the sampler, due to large size of the material and small size of the sampler. Some gravel also had to have been excluded from the samples due to the small sampler orifice. So, the 45% number for gravel content has to be considered too low. An estimate of average percentage cobbles and larger material, based on visual observations, is 25 to 30% in Unit 1. Thus, at a minimum, Unit 1 is 70% gravel and larger material and no more than 30% sand and smaller material. "

Yellow Values Calculated. Assumed a constant ratio between sand and silt. Adjusted original values based on ratio and ensured the adjusted summed up to 30

Volume Break Down by Material

	Unit 1-Total	Unit 1-Sand	Unit 1-Silt and Clay	Unit 1-Other Material	Unit 2-Total	Unit 2-Sand	Unit 2-Silt and Clay	Unit 2-Other Material	Unit 3-Total	Unit 3-Sand	Unit 3-Silt and Clay	Unit 3-Other Material
Material Lost between Existing and Notch 1	206,276	57,757	4,126	144,394	352,246	257,139	77,494	17,612	42,478	9,345	33,133	0
Material Lost between Notch 1 and Notch 2	0	0	0	0	0	0	0	0	187,668	41,287	146,381	0
Total Material Removed	206,276	57,757	4,126	144,394	352,246	257,139	77,494	17,612	230,146	50,632	179,514	0
Total Material Remaining	0	0	0	0	0	0	0	0	0	0	0	0

Misc. Checks

Unit Total	206,276	57,757	4,126	144,394	352,246	257,139	77,494	17,612	230,146	50,632	179,514	0
Check	206,276	57,757	4,126	144,394	352,246	257,139	77,494	17,612	230,146	50,632	179,514	0
Actual	208,320	58,330	4,166	145,824	339,265	247,663	74,638	16,963	232,676	51,189	181,487	0
% Difference	-0.98	-0.98	-0.98	-0.98	3.83	3.83	3.83	3.83	-1.09	-1.09	-1.09	0.00

Misc Checks

F4

Boring Log Information

Boring Log Information Taken from Geotechnical Appendix

		Unit 1			Unit 2			Unit 3		
		Top	Bottom	Depth	Top	Bottom	Depth	Top	Bottom	Depth
Block 4	Trench 4	0	15.5	15.5	0	0	0	0	0	0
	TH02-08	0	14.9	14.9	0	0	0	14.9	19.9	5
	Trench 3	0	1	1	0	0	0	0	0	0
Block 3	TH02-07	0	13.8	13.8	13.8	53.9	40.1	0	0	0
	L.C. Hole 3	0	26	26	26	42.5	16.5	0	0	0
	TH02-06	0	12	12	12	37	25	0	0	0
Block 2	Trench 2	0	10.5	10.5	0	0	0	0	0	0
	TH02-03	0	18	18	18	66.6	48.6	66.6	75.3	8.7
	L.C. Hole 1	0	32	32	32	62	30	62	84	22
	TH02-05	0	23	23	23	63	40	63	83.7	20.7
	TH02-02	0	14.25	14.25	14.25	52	37.75	52	81.9	29.9
Block 1	L.C. Hole 2	0	17	17	17	34	17	34	100	66
	TH02-04	0	16.25	16.25	16.25	51.1	34.85	51.1	88.9	37.8
	Trench 1	5	20	15	0	0	0	20	28.5	8.5
	TH02-01	0	8	8	8	44	36	44	93.5	49.5

Note:
Values highlighted in blue are referenced in volume calculations
The methodology used to calculate all volumes and depths is based on the methods outlined in Legacy Geot Atch D of the Geotechnical Appendix for the F-4 Milestone dated August 2008.

Depth Breakdown

Depth breakdown by percent

	Assumed Depth	Unit 1 Ave. Depth	Unit 1 %	Unit 2 Ave. Depth	Unit 2 %	Unit 3 Ave. Depth	Unit 3 %
Block 4	20	14.9	0.745	0	0.000	5	0.250
Block 3	44	17.27	0.392	27.2	0.618	0	0.000
Block 2	80	21.81	0.273	39.09	0.489	20.33	0.254
Block 1	94	13.75	0.146	29.28	0.312	51.10	0.544

Volume Checks

					Calced	Actual	Difference	%
Block 4					Calced	Actual	Difference	%
Length	500	Top Area	37,500	Total Volume(CY)	13,889	13,889	0	0.001
Width	75	Block Volume	750,000	Unit 1	10,347	10,382	35	0.335
Canyon Wall	50	Adjusted Volume	375,000	Unit 2	0	0	0	0.000
Depth	20			Unit 3	3,472	3,507	35	0.992
Block 3					Calced	Actual		
Length	890	Top Area	126,824	Total Volume(CY)	103,338	103,338	0	0.000
Width-u/s	75	Block Volume	5,580,252	Unit 1	40,552	39,992	-560	-1.401
Width-d/s	175	Adjusted Volume	2,790,126	Unit 2	63,882	63,346	-536	-0.846
Canyon Wall	50			Unit 3	0	0	0	0.000
Depth	44							
Block 2					Calced	Actual	Difference	%
Length	825	Top Area	222,750	Total Volume(CY)	462,000	462,000	0	0.000
Width	270	Block Volume	17,820,000	Unit 1	125,967	128,667	2,700	2.098
Canyon Wall	30	Adjusted Volume	12,474,000	Unit 2	225,730	212,058	-13,672	-6.447
Depth	80			Unit 3	117,377	121,275	3,898	3.214
Block 1					Calced	Actual	Difference	%
Length	330	Top Area	82,500	Total Volume(CY)	201,056	201,055	-1	0.000
Width	250	Block Volume	7,755,000	Unit 1	29,410	29,280	-130	-0.443
Canyon Wall	30	Adjusted Volume	5,428,500	Unit 2	62,634	63,861	1,227	1.922
Depth	94			Unit 3	109,297	107,894	-1,403	-1.301

Volume Check