CORTE MADERA CREEK FLOOD CONTROL PROJECT UNIT 4 DESIGN ALTERNATIVES

Final Technical Memorandum

October 31, 2006

Prepared for
Friends of Corte Madera Creek Watershed
Marin County Flood Control and Water Conservation District
and the U.S. Army Corps of Engineers

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TECHNICAL MEMORANDUM

Date: **October 31, 2006**

To: **Sandra Guldman, Friends of Corte Madera Creek Watershed**

From: **Matt Smeltzer, Fluvial Geomorphology Consulting**
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Subject: **CORTE MADERA CREEK FLOOD CONTROL PROJECT OCTOBER 2006 UNIT 4 DESIGN ALTERNATIVES**

PART 1. BACKGROUND AND TECHNICAL INFORMATION

Purpose

This technical memorandum documents the development of four feasibility-level design alternatives for improving fish passage, bank stability, and/or flood flow capacity of Corte Madera Creek Flood Control Channel Unit 4. Unit 4 is an approx. 2,000-ft long still natural section of Corte Madera Creek extending from the upstream end of the existing Unit 3 concrete channel in Ross upstream to Sir Francis Drake Blvd at the border between Ross and San Anselmo.

The alternatives include a preliminary conceptual fish passage improvement design prepared by Michael Love & Associates and multiple feasibility-level bank stabilization flood flow capacity improvements for Unit 4, and associated hydraulic design recommendations for Lagunitas Road Bridge replacement, prepared by Fluvial Geomorphology Consulting and Stetson Engineers. This work was funded by the National Fish and Wildlife Foundation under a contract awarded to Friends of Corte Madera Creek Watershed.

This memorandum is intended for transmittal to the Marin County Flood Control and Water Conservation District and the Army Corps of Engineers with the associated HEC-RAS hydraulic model files for review by Corps technical staff, and incorporation into their environmental and public review procedures. Upon completion of technical and editorial review, the summaries and graphic materials contained in this memo are further intended to be presented to the Town of Ross for their consideration and public input. It is recommended that these technical materials be presented to the Town at the same time as architectural renderings of the alternatives to be prepared by MIG under contract to the County of Marin.

Historical Background

Corte Madera Creek Flood Control Channel Units 1-3 channel improvements were completed by the Army Corps of Engineers (Corps) in approximately 1971. The existing approx. 4 ft-high timber bulkhead grade control structure and wooden Denil fish ladder were installed at the upstream terminus of the Unit 3 concrete channel at that time, pending planned improvements to Unit 4, located between Unit 3 and the boundary between Ross and Anselmo. But planned continuation of the concrete channel was delayed by public opposition.

Since 1971, the Corps has conducted additional studies and prepared hydraulic and sediment transport models focused on evaluating the design performance of Units 1-3 following the 1982 flood, and prepared a suite of seven design alternatives documented in 1999-2000 screening documents and hydrology and hydraulics appendix documents (Alternatives I-VIIB). None of these received strong public support. Concerns regarding some of the alternatives that provided the desired increases in capacity included aesthetic and environmental impacts of constructing a proposed concrete-lined sedimentation basin in the channel and removing a large percentage of the existing canopy-forming riparian trees to install a new up to 16 ft-high vertical steel sheetpile retaining walls along the left (east) bank of Corte Madera Creek. There has also been a concern about the flood constricting effect of the potentially historically significant Lagunitas Road Bridge. Moreover, many of the Corps' larger capacity design alternatives were confounded by important concerns about flood flows leaving the channel upstream in San Anselmo and passing through Ross and into Kentfield on the residential floodplain rather than the to-be-improved Unit 4 channel and about the inability of Units 1-3 to accommodate local drainage at high flows.

Recent Project Developments and Technical Analyses

There have been a number of important developments and technical analyses since the 1999-2000 alternatives were presented, making it appropriate and timely to prepare and review new alternatives. First, the December 31, 2005 (2006 Water Year) flood renewed public interest in finding watershed-wide solutions to flooding in the Ross Valley. Using funds provided by the Coastal Conservancy, Marin County Department of Public Works Flood Control District 9 has contracted with Stetson Engineers Inc. to prepare a new hydraulic model of the Ross Valley including downtown San Anselmo and use the model to present to the public potential watershed-wide flood management improvements. These improvements will include an evaluation of the potential for reducing the frequency and amount of overbank flooding originating in Downtown San Anselmo, and thereby keeping more flood flow in the Corte Madera Creek channel passing through Unit 4 rather than on the floodplain in Ross and into Kentfield.

Second, the Town of Ross has declared that it plans to remove and replace Lagunitas Road Bridge with a new presumably less constrictive crossing structure. Third, hydraulic analysis of the 1982 flood and expert peer review of the Corps' proposed sedimentation basin raised serious concerns about the effectiveness and necessity of the basin as part of Unit 4 flood control improvements.

Fourth, the existing wooden Denil fish ladder partially failed during the December 31, 2005 flood and a conceptual permanent fish ladder replacement design was recently completed by Michael Love and Associates, with funding provided to Friends of Corte Madera Creek by the National Fish and Wildlife Foundation.

Fifth, the Marin County Flood Control District, Zone 9 collected detailed survey topography in the Units 3 and 4 reach beginning at the upstream end of tidal influence in Unit 3 concrete channel and extending upstream to the Ross gage behind the Ross Fire Department engine house. The County revised these data in Spring 2006 in the vicinity of the new rip-rap bank stabilization structure on the east bank just upstream from the Unit 3 concrete channel (21 Sir Francis Drake Blvd).

Sixth, the Corps contracted with Treadwell-Rollo to prepare an updated, more comprehensive geotechnical analysis report than had been available at the time the 1999-2000 alternatives were presented.

Pending Analyses

Work is currently underway by Stetson Engineers assisted by David Dawdy, Consulting Hydrologist, to update the stage-discharge rating curve for the Ross Gage using new field stage-discharge data collected by Stetson Engineers Inc. and Environmental Data Solutions in 2005-2006. The updated rating curve will allow the consultant team to convert stage data the County collected from 1996 to present to annual peak discharge data, and thereby revise the Ross gage flood frequency analysis using the longer peak flow record. Completion of these hydrologic analyses will allow more accurate recurrence intervals to be associated with the new design capacities of the five new alternatives.

Scope of New Modeling

The new October 2006 model design alternatives are intended to reduce overbank flood flows passing into Downtown Ross from the right (west) bank of Corte Madera Creek along an approximately 800-900 ft-long reach extending from the upstream end of the existing Unit 3 concrete channel to a location upstream from the private residence at 1 Sylvan Lane. Accordingly, the new design alternatives include channel modifications for fish passage, bank stabilization, and flood management improvement in this reach only. The new modeling work does not include channel modifications in Units 1-3. The Corps' 1999-2000 design alternatives included channel widening, floodwall parapet design, and channel dredging in Units 1-3. These design elements were carried forward unchanged in the new October 2006 alternatives.

The only exception is for Alternative I geometry files that include the preliminary concept design replacement fish ladder structure situated within the upstream end of the existing Unit 3 concrete channel. Michael Love & Associates proposed this location for the replacement permanent-type fish ladder to minimize potential impacts to the existing sanitary sewer siphon running beneath the creek about 20-25 ft upstream from Unit 3. See the fish ladder design report in Appendix A for more discussion on potential locations for the replacement fish ladder.

Again, only the downstream portion of the designated Unit 4 reach, up to the Ross gage, is modeled in detail by the new October 2006 design alternatives. Flood management improvements in Units 1-3 and the upstream portion of Unit 4 may be addressed by the ongoing Ross Valley modeling effort.

Adoption of Existing Corps Model Files and Parameters

The 1999-2000 modeling work generated nine individual recommended design alternatives, titled Alternatives I-VIIC. This memorandum titles the new set of five October 2006 design alternatives according to a similar convention: Baseline Alternative; Alternative I; Alternative IIA; Alternative IIB; and Alternative III.

At the initiation of this work, the Corps provided one HEC-RAS project file containing four HEC-RAS hydraulic model geometry files and four associated plan files (Plans 01-04) corresponding to four 1999-2000 design alternatives (4,100 cfs Minimal Plan, 3,200 Existing Conditions Plan, 5,400 cfs Plan with 5-yr dredging cycle, and 5,400 cfs plan with 10-yr dredging cycle).

The new October 2006 design alternatives and variants are documented in 21 new HEC-RAS model plan files (Plans 05-29). The 21 new October 2006 model plan files are incorporated in the original received Corps project file, with its original four 1999-2000 model plan files. The Corps project file contained one steady flow file (f01). This file was used throughout the new model file development process. A second steady flow file (f02) was created at end of this modeling work (Table 1). Flow file (f02) was created in order to model specific selected discharges and thereby more precisely document Unit 4 design capacity for the final recommended new design alternatives.

The 2005 existing conditions geometry file and the Baseline (Existing Conditions) Alternative geometry file were created by substituting 2005 surveyed topography data into the then-existing conditions (1999) geometry file. The 2005 survey data extend from the upstream end of the tidal influenced portion of the existing Unit 3 concrete channel upstream to the vicinity of the Ross flow gage. Therefore, geometry conditions in the portion of Unit 4 upstream from the gage location remain represented by the 1999 model geometry file data. The 2005 survey data were collected before the Town of Ross completed its annual dredging work beneath Lagunitas Road Bridge, before the 2006 flood deposited a new, apparently higher elevation gravel bar beneath the bridge, and before another dredging operation conducted in October 2006. All elevations are in the NGVD29 datum. If necessary, please refer to the Vertcon website to determine the conversion to NAVD88 datum.

Table 1. Flow files used in this model analysis.

Profile Number	Flow File f01	Flow File f02
PF1	2,000 cfs	3,200 cfs
PF2	3,250 cfs	5,000 cfs
PF3	4,100 cfs	5,100 cfs
PF4	4,700 cfs	5,200 cfs
PF5	5,950 cfs	5,400 cfs
PF6	6,600 cfs	5,500 cfs
PF7	7,400 cfs	5,600 cfs
PF8	8,100 cfs	6,000 cfs
PF9	na	6,100 cfs
PF10	na	6,200 cfs

Inspection of the original Corps model files revealed an inconsistency in the designated channel centerline stationing. Graphics and other model documentation in the January 2000 Screening Document variously represented the River Station at the upstream end of the existing Unit 3 concrete channel as 369+50 ft and 370+00 ft. This 50-ft discrepancy would produce a negligible difference in model calculated water surface profiles and design capacity estimates. For consistency, the value 369+70 ft was adopted and used in the new model files for the River Station at the upstream end of the concrete channel. This value was determined from pre-1999-2000 Corps' project documents, and confirmed by the Corps as the correct value (Bill Firth, pers. comm., April 2005). This resulted in a 29 ft river stationing discrepancy between received 1999-2000 model files and new October 2006 model files.

The original received 1999-2000 model generated a standard HEC-RAS summary of errors, warnings, and notes containing no errors but numerous warnings due to large differences in conveyance capacities (i.e., cross-section area) between successive cross-sections, and the large number of iterations therefore required to converge on an energy equation solution. It should be expected that the new design alternative models would generate a similar warning pattern, but with perhaps fewer warnings for the reach with effective flood management improvements. This is because flood management improvement design entails making the channel cross-section area more uniform through the reach, and thereby reducing differences in conveyance capacity between successive cross-sections.

Model parameters contained in the original received 1999-2000 model files were adopted by default and used universally in the 21 new plan files is described in this memorandum and contained in the final HEC-RAS model project file. These default model parameters and other physical constraints and definitions adopted by the new modeling work are documented below.

Summary of Physical Constraints and Modeling Assumptions

Manning's n roughness coefficient.

Both the original 1999-2000 Corps models and the new October 2006 model files use Manning's n roughness coefficients of 0.1 in overbank areas and 0.05 to 0.045 within the Unit 4 natural channel (Table 2). The in-channel values appear somewhat conservative (high) given the size of the substrate present on the bed. Stetson Engineers made a pebble count (Wolman 1954) on the exposed left bank gravel bar immediately downstream from the Ross gage on December 20, 2005 which gave a D84 of approx. 40 mm. The conservatively high values were retained for this model analysis both because of multiple friction losses present in the reach in addition to skin friction, and following on the results of test model runs showing that the recorded stages at the Ross gage were predicted well for discharges (up to approx. 1,500 cfs) measured at Lagunitas Road Bridge by Environmental Data Solutions in 2005-2006.

Table 2. Manning's n roughness coefficients adopted from original Corps 1999 existing conditions (with 2-year dredging cycle) model file for use in this modeling analysis.

From	To	Roughness Coefficient (Mannings' n)			
River Station	River Station	Left Overbank	In-Channel	Right Overbank	Notes
392+00	383+43	0.1	0.05	0.1	Unit 4 natural channel
382+97	370+00	0.1	0.045	0.1	Unit 4 natural channel
369+50	348+00	0.1	0.018	0.1	Unit 3 conc ch w/ sed deposit
347+00	331+60	0.1	0.028	0.1	Unit 3 conc ch w/ sed deposit
329+00	319+05	0.1	0.03	0.05	Unit 2 conc ch w/ sed deposit
318+75	318+10	0.1	0.028	0.05	Unit 1 earthen channel
317+10	317+10	0.05	0.031	0.05	Unit 1 earthen channel
310+10	166+40	0.05	0.035	0.05	Unit 1 earthen channel

Table 3. Contraction and expansion friction loss coefficients adopted from original Corps 1999 existing conditions model files for use in this modeling analysis, and revised only for new October 2006 design alternatives that included substantial channel straightening and smooth channel bank retaining walls.

		Friction Loss Coefficients		
From River Station	**To River Station**	**Contraction**	**Expansion**	**Notes**
392+00	369+71	0.3	0.5	Unit 4 natural channel existing and design conditions
392+00	369+71	0.1	0.3	Unit 4 improved conditions (Alternative III only)
369+70	16+640	0.1	0.3	Units 1-3 existing and design conditions

Expansion and contraction loss coefficients.

The original model files received from the Corps generally used contraction and expansion coefficients of 0.3 and 0.5, respectively, for natural unimproved existing channel conditions in Unit 4, and 0.1 and 0.3, respectively, for existing improved conditions in Unit 1-3. The new October 2006 model files adopt the same contraction and expansion coefficients for existing unimproved conditions in the Baseline (Existing Conditions) Alternative and design alternatives that include minimal or modest amounts of channel bank regrading at channel bends (Table 3). Sensitivity analysis test runs show that reducing the coefficients from 0.3-0.5 to 0.1-0.3 generally translates into a 0.3-0.7 ft difference in model-calculated water surface elevation, with the larger differences occurring in the upstream portion of Unit 4 where the overbank flood hazard is less. The lower 0.1-0.3 coefficients were only used for design alternatives that included substantial straightening of the lower Unit 4 channel (i.e., Design Alternative III).

Ineffective flow areas.

The original 1999-2000 model files received from the Corps designated all shallow overbank flow in Unit 4 as normal ineffective flow by placement of normal ineffective flow markers at or near the top of bank at approximately existing grade elevation. The new October 2006 model files designate ineffective flow areas the same way in Unit 4.

Deeper overbank flows are typically effective because they would exceed the ineffective flow marker elevation at or near top of bank grade. However, although effective, these overbank flows make up a very small fraction of total flow because the model contains only 20-40 ft-wide overbank areas. Moreover, the manner in which overbank flows are treated matters little to the accuracy to the original and new design models because modeling is intended to be accurate for in-channel flows only – flow up to but not exceeding the design channel capacity.

Bridge modeling approach.

The new October 2006 model files include planned removal and replacement of Lagunitas Road Bridge. The new model work assumes that Lagunitas Road Bridge would be replaced with a structure that has no more hydraulic effect on the design capacity flow than the existing Lagunitas Road Bridge vertical concrete abutments. That is, it is assumed that the bridge will be replaced with a structure designed according to the preferred design alternative, and, as such, would be a clear-span bridge deck structure (i.e., no piers) with its low chord elevation not less than the model-calculated water surface elevation for the design discharge. For this reason, none of the design alternatives contain bridge structures and no new bridge modeling was required.

Lagunitas Road Bridge replacement design.

Each of the recommended design alternatives resulting from the new modeling work will have a design capacity flow and a model-calculated water surface elevation at the Lagunitas Road Bridge location. It is assumed that the replacement structure will be designed to not constrict the design discharge and thereby have no piers or deck structure within the flow area up to the model-calculated water surface elevation. Model-simulated reconfiguration of the existing vertical concrete abutments is performed to determine for each of the recommended design alternatives whether or not reconfiguration of the abutments should be included in the hydraulic design for the replacement structure. The new model files do not include a sediment basin in the vicinity of the bridge. The new model files assume that the channel bed elevation in the bridge vicinity will be consistent with the undredged condition.

Existing sanitary sewer siphon protection.

The Ross Valley Sanitary District No. 1 (RVSD) maintains two parallel 24-inch diameter reinforced concrete pipe siphon sewer lines passing beneath the bed of Corte Madera Creek on a diagonal path immediately upstream from the existing wooden Denil fish ladder. Plan and profile views of the existing sewer line provided by RVSD are reproduced as appendices to the May 30, 2006 Michael Love and Associates report (see Appendix A).

It is assumed that design alternatives that include channel bed regrading in the vicinity of the sewer line must provide for a minimum 2.5-ft deep rip-rap lining above the top of the concrete encasement surrounding the two 24-inch diameter sewer pipes.

In addition, it is assumed that design alternatives that include bank regrading and/or installation of vertical retaining walls along the right (west) bank in the vicinity of the siphon must provide a similar minimum depth of rip-rap armor protection above the siphon. Retaining walls installed at depth beneath the bank must begin and end in plan view not more than 4 ft from the sewer line. The intervening approximately 10-12 ft-long reinforced concrete retaining wall section must be formed to provide collar protection for the sewer line.

The left (east) bank is heavily armored under existing conditions and is unlikely to be modified by the design alternatives given the negligible hydraulic effect of the armor materials in that vicinity.

Allowable left (east) bank modifications.

There are eight individual properties forming the left (east) bank along the lower Unit 4 reach downstream from the Ross gage. The most upstream property is the Town of Ross municipal property, where it is assumed that the channel banks may be regraded if it can be demonstrated that the regrading would create a flood benefit. Further, regrading to maximum 1.5(H):1(V) finished slope would be preferable on the Town property as it would allow biotechnical stabilization and vegetation establishment and thereby have lesser aesthetic and environmental impacts. The second property is within the Lagunitas Road Bridge right-of-way where it is assumed that the bridge removal and replacement project planned by the Town allows an opportunity to move the replacement bridge abutments farther to the west if it creates a substantial flood benefit. Accordingly, it is assumed that the Marin Art and Garden property encompassing approx. a 100-ft long section of the east bank immediately downstream from Lagunitas Road Bridge can be regraded if necessary to produce a substantial additional flood benefit, as would be balanced by the desire to preserve native ash trees at mid-bank and top-of-bank and the overall natural aesthetic characteristics immediately downstream from the bridge.

The remaining five properties forming the left (east) bank in lower Unit 4 are residential, three of which have existing permanent channel bank stabilization structures. It is assumed that no channel bank regrading would be acceptable to the landowners along the left (east) bank, particularly if existing top-of-bank property area or trees would be removed by the grading work. It is assumed that the two properties without existing permanent stabilization structures should be individually analyzed by this modeling work to determine if minimum impact biotechnical bank stabilization measures could be constructed at the sites in a manner that would improve long-term bank stability and provide a demonstrable flood benefit, while also not substantially reducing existing top-of-bank area.

It is also assumed that any of the design alternatives that include channel bed regrading and/or removal of the existing timber bulkhead grade control wooden Denil fish ladder structure will provide protection against channel bed incision that may destabilize any of the existing natural channel banks or permanent bank stabilization structures.

Recall there was general public opposition to the Corps' 1999-2000 5,400 cfs design alternative, for one because of the perceived aesthetic and environmental impacts of its up to 16-ft high vertical steel sheetpile retaining wall to be located along the left (east) bank downstream from the bridge. Both to reduce the aesthetic impacts of a new retaining wall in this alignment, and to avoid potentially unnecessary modifications to private property, the new modeling work assumes that high design capacity alternatives requiring new vertical wall to sufficiently increase channel width in constricted sections should be designed to locate the new wall along the Town property on the right (west) bank where it would be less visible from the pedestrian right-of-way near the Post Office.

<u>Allowable right (west) bank modifications</u>.

There are four individual properties forming the right (west) bank along the lower Unit 4 reach downstream from the Ross gage. The first is a residential property is directly across Corte Madera Creek from the Ross gage. It is assumed that bank regrading within this property may not be acceptable, nor would it likely create a substantial flood benefit due to the apparent flood flow constriction created by the existing encroached private residential vertical steel sheetpile retaining wall immediately downstream at 1 Sylvan Lane. It is further assumed that the existing retaining wall at 1 Sylvan Lane cannot be removed or modified. It is assumed that moving the right bank Lagunitas Road Bridge abutment and wingwalls back farther to the west as part of the bridge replacement design would be acceptable to the Town if it is part of a preferred, presumably relatively high-capacity design alternative. The right (west) bank downstream from the bridge is Town of Ross property containing numerous mature native riparian canopy-forming trees at low-, mid-, and upper bank location, and a pedestrian right-of-way and power and water utilities at the top of bank. The new modeling work assumes that a range of design alternatives should be developed to simulate regrading the right bank both minimally, as would preserve the majority of existing native riparian trees, and severely, as would demonstrate the maximum feasible design capacity of the channel.

Geotechnical constraints.

The Corps' contracted with Treadwell-Rollo to prepare a new more detailed borehole soil sampling and laboratory geotechnical analysis of soil samples than had been previously available. Review of the geotechnical analysis report shows, in general, that while soil strength is generally better on the right (west) bank than the left (east) bank, there do not appear to be significant geotechnical constraints that would abnormally limit the range of structurally feasible bank regrading and stabilization techniques. As is always the case in narrow, steep alluvial channels, the feasibility of biotechnical bank stabilization techniques is usually limited by the desire to maintain existing top of bank land uses, prohibiting the banks from being graded back to gradual enough slopes. The new model design work assumes that any biotechnical bank stabilization treatments proposed must be designed at a suitable maximum finished bank slope that is appropriate to the site according to the available soil strength data, and other geotechnical data contained in the geotechnical analysis report, as well as standard structural and earthquake loading, drainage, and hydraulic design assumptions.

Definition of Unit 4 channel capacity.

The new October 2006 modeling work assumes that the design alternatives should seek to eliminate flood flows that escape Corte Madera Creek to the southwest from the vicinity of the Lagunitas Road Bridge location and immediately upstream from the Unit 3 concrete channel. These flood flows would travel into Downtown Ross and then proceed into Kentfield. Accordingly, the new modeling work generally defines the Unit 4 channel capacity as the maximum flood flow discharge the channel can convey before the model calculated water surface elevation exceeds the right (west) top of bank elevation, as determined by comparing model-calculated water surface elevation profiles to the maximum elevation profile along the top of the right (west) bank.

To compile a right bank elevation profile, the 1966 Clair Hill and Associates 2-ft contour interval topographic map and the 2005 County of Marin topographic survey data were reviewed to determine and incorporate existing top of pavement elevations along Sylvan Lane, the Lagunitas Road centerline, and the pedestrian right of way running behind the Post Office. Where top of bank elevations at the existing chain link fence are higher than the sidewalk elevation, the higher elevation was incorporated in the top of bank profile. The resulting top of bank elevation profile was incorporated in the new model files as right levee (see Figure 1 at the beginning of Part 2). Figure 1 shows that the right bank elevation profile is not uniform, particularly upstream from Lagunitas Road Bridge (elevation 25 ft at 20,900 ft main channel distance) where the profile dips down about one vertical foot (to a minimum elevation of 24 ft (at 21,100 ft main channel distance).

Figure 1 also shows how the model calculated 3,250 cfs water surface elevation profile for 1999 existing conditions geometry compares to the estimated west top of bank profile. Note that a 3,250 cfs flow would very nearly exceed the Lagunitas Road Bridge deck elevation (25 ft NGVD29) at the upstream face of the bridge, and it would exceed the top of bank elevation profile upstream from the bridge by about 1 ft, and by about 0.5 ft at two locations downstream from the bridge. In 1999-2000, the Corps used the same model results as shown in Figure 1 to estimate the Unit 4 flood capacity under then-existing conditions to be 3,200 cfs (negligibly different than the model result for 3,250 cfs). In developing the 1999-2000 design alternatives, the Corps had specifically defined Unit 4 channel capacity as the maximum discharge that still does not overtop the 25 ft bridge deck elevation at the upstream face of the Lagunitas Road Bridge. This was an appropriate definition for then-existing conditions because the relatively minor amounts of overbank flow upstream and downstream from the bridge, as shown in Figure 1, could be mitigated by relatively inexpensive and low-impact measures, including raising the finished top of pavement elevation on Sylvan Lane and placing temporary seasonal sandbag levees along the existing chain link fence downstream from the bridge.

Accordingly, to be functionally equivalent to the 1999-2000 Corps definition, the new October 2006 modeling work specifically defines Unit 4 channel capacity as the maximum discharge that produces a model-calculated water surface elevation profile that is nowhere 1.0 ft higher than the above-defined right top of bank elevation profile downstream from the Lagunitas Road Bridge location, and nowhere 1.0 ft higher than the profile upstream from the bridge location.

The Corps' 1999-2000 design alternatives used a singular definition of channel capacity – the 25-ft Lagunitas Road Bridge deck elevation. The October 2006 design alternatives seek to apply a reach-scale definition of channel capacity, but it is possible that the design capacity (always a maximum) would be determined as limited by the either the lowest portion of the top of bank elevation profile, or the highest local portion of the water surface elevation profile, or both. That is, the reach-scale channel capacity may dissolve into a singular definition if it is revealed that there is a "weakest link" in the system – a location that is consistently overtopped by a lower discharge than the rest of the study reach.

Between the Ross gage and the bridge, the left (east) top of bank elevation profile is about the same or higher than the right (west) top of bank elevation profile. Downstream from Lagunitas Road Bridge, the left bank profile is non-uniform and generally 0.5 ft lower than the right bank. Therefore, this new modeling work assumes there will up to about 1.0-1.5 ft deep overbank flow along the left bank during the Unit 4 design capacity flow. It is further assumed that this amount of left bank flooding can be mitigated by either or both flood control project or private residential measures including, installing a low 1.5-2.0-ft high temporary seasonal (sandbags) or permanent concrete floodwalls along the top of the left bank, raising finished floor elevations of the five affected residential homes, or other structural floodproofing measures. Recall that the higher design capacity 1999-2000 design alternatives, including the 5,400 cfs plans, included installation of low permanent floodwalls along the top of left bank from Lagunitas Road Bridge to the Unit 3 concrete channel.

PART 2. UPDATED UNIT 4 EXISTING CONDITIONS CHANNEL CAPACITY

Part 2 Purpose

Part 2 of this memorandum is intended to determine how any changes in the channel bed geometry between 1999 and 2005 have increased or decreased existing conditions channel capacity. Recall that for 1999 existing conditions, the Corps estimated the Unit 4 channel capacity to be 3,200 cfs (Figure 1).

This is done by substituting 2005 topographic survey data into the 1999 existing conditions model geometry file received from the Corps, and comparing the water surface elevation profiles the model generates for the 1999 and the updated 2005 geometry files (Figure 2).

Instructions for Interpreting Water Surface Elevation Profile Charts.

This memorandum uses model calculated water surface elevation charts exported from the HEC-RAS model to display and explain incremental model results and thereby develop the technical rationale for the five recommended Design Alternatives. These charts compare model calculated water surface elevations along the lower Unit 4 reach to the estimated right (west) top of bank elevation profile to show where and how much various discharges would overtop the bank and begin to flow into Downtown Ross and Kentfield. The complete models extend from the downstream limit of the Unit 1 flood control channel at 16,640 ft main channel distance upstream to near the Ross Creek tributary confluence at 22,600 ft main channel distance. The charts only display the upstream 2,600-ft length of the model domain.

Note that main channel distance does not correspond directly to the River Station numbering convention for individual model cross-sections. For example, model cross-section at River Station 369+70 ft is at the upstream end of the existing Unit 3 concrete channel and this is shown as 20,330 ft main channel distance. Note that flow is from right to left. The existing Lagunitas Road Bridge centerline is at 20,900 ft main channel distance.

The right (west) top of bank profile is shown in the charts as the “Right Levee” line (purple). Per the adopted definition of Unit 4 channel capacity discussed in Part 1 above, upstream from Lagunitas Road the top of bank profile is taken as the elevation profile along Sylvan Lane. Downstream from Lagunitas Road it is taken as the elevation profile along the sidewalk pedestrian right of way.

All new modeling work includes 2005 survey data only from the upstream end of Unit 3 at 20,330 ft main channel distance upstream to the vicinity of the Ross gage near 21,400 ft main channel distance. Upstream from 21,400 ft main channel distance all geometry data are from the original 1999 models received from Corps. The water surface elevation charts show as “Ground” (black) the minimum channel bed elevation profile over the reach. These are profiles depicting the minimum elevation from each of the model cross-sections. For charts that compare model results for two plans, both ground profiles are shown (black and magenta) for comparing model geometry.

The model-calculated water surface elevation profiles (WS PF) are shown in blue, and the critical water surface elevation profiles (Crit PF) are shown in red. The energy grade lines (EG PF) are not plotted for the sake of clarity. The associated model discharges are according to profile number denoted. Table 1 shows the Unit 4 discharges according to each profile (PF) number and flow file. The chart header displays the geometry title, plan(s), and flow file(s). NGVD29 refers to the National Geodetic Vertical Datum of 1929, an approximation of mean sea level elevation.

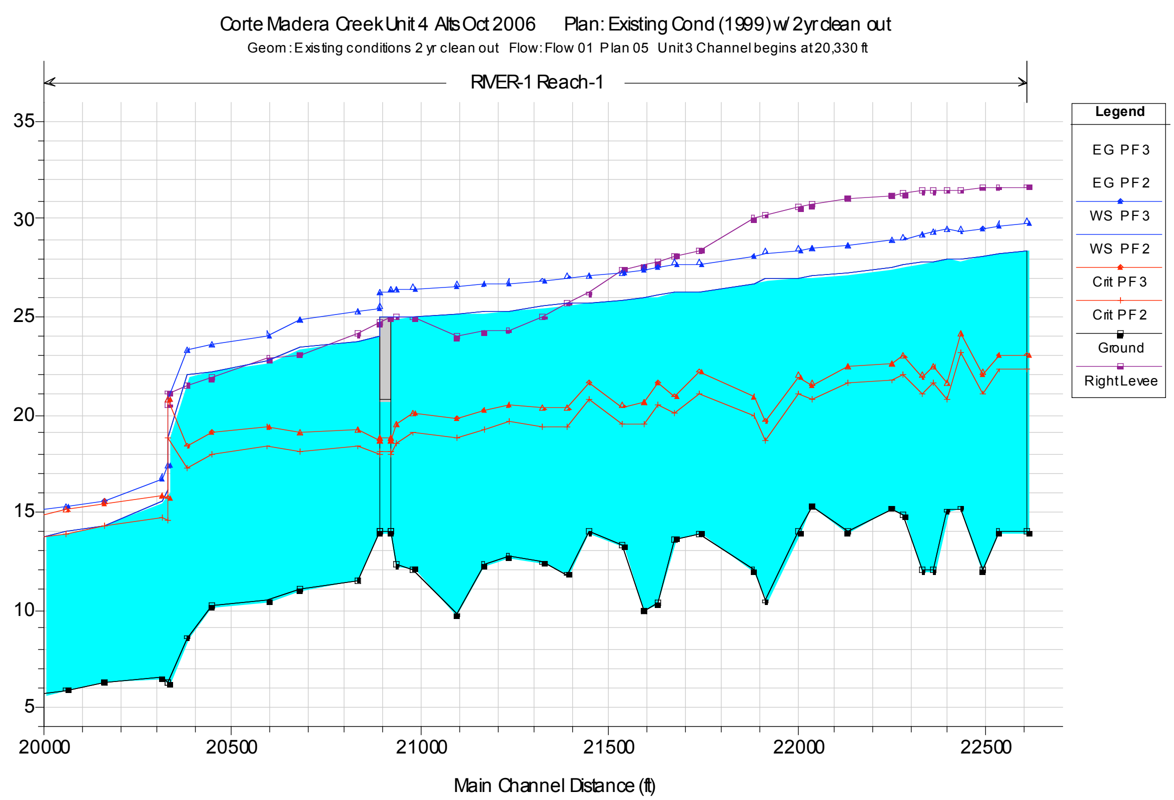

Figure 1. *Plan 05.* Comparison of calculated water surface profiles for PF2 (3,250 cfs) and PF3 (4,100 cfs) in the Unit 4 reach for 1999 existing conditions geometry (g04), including the existing Lagunitas Road Bridge configuration.

- *For 1999 existing conditions geometry, the Corps estimated Unit 4 channel capacity to be 3,200 cfs.*

- *In 1999, the Corps defined Unit 4 channel capacity as the maximum discharge just overtopping the existing Lagunitas Road Bridge deck at elevation 25 ft. Flows exceeding approx. 3,250 cfs just overtop the bridge deck. The same 3,250 cfs discharge also overtops Sylvan Lane by more than 1 ft, and overtop the west bank sidewalk about 0.5 ft in places between the bridge and the Unit 3 concrete channel.*

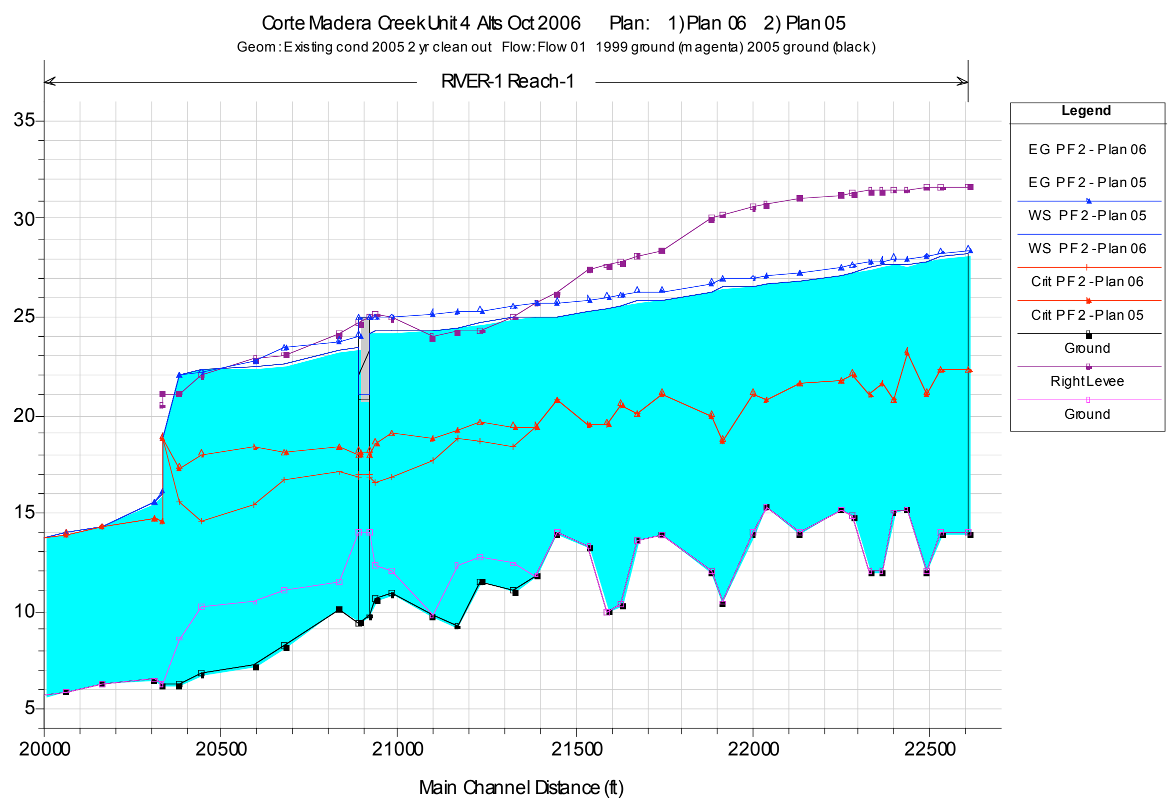

Figure 2. *Plan 05/06.* Comparison of calculated water surface profiles for PF2 (3,250 cfs) in the Unit 4 reach for 1999 existing conditions geometry (g04, Plan 05) and 2005 existing conditions geometry (g05, Plan 06). Both geometry files include the existing Lagunitas Road Bridge configuration. The 2005 minimum bed elevation profile is shown in black, compared to 1999 bed profile shown in magenta. The 2005 topography data extends from Unit 3 to approximately 21,400 ft main channel distance. Upstream, the 2005 geometry file contains a duplicate of the 1999 existing conditions geometry file.

- *According to survey data, from 1999 to 2005 the Unit 4 minimum bed elevation lowered 3-4 vertical feet downstream from Lagunitas Road Bridge, and 1-3 feet upstream from Lagunitas Road Bridge.*

- *Unit 4 channel bed lowering between 1999 and 2005 decreased the water surface elevation of a 3,250 cfs flood flow about one foot, except for in the downstream most 200-ft length of Unit 4, where flood overflow is predicted to occur.*

- *Channel bed lowering between 1999 and 2005 increased Unit 4 channel capacity upstream and downstream from Lagunitas Road Bridge but did not increase Unit 4 channel capacity immediately upstream from the Unit 3 concrete channel. Therefore the 2005 existing conditions capacity is essentially the same as the 1999 existing conditions capacity – 3,200 cfs.*

Part 2 Summary Discussion

Under existing conditions, the model calculated water surface elevation profile in Unit 4 remains strongly controlled by a backwater effect from the unsmooth hydraulic transition from the Unit 4 channel into the Unit 3 concrete channel, and particularly by the constriction the existing timber bulkhead grade control and wooden Denil fish ladder structure creates immediately upstream from Unit 3. Although the channel bed elevations reduced considerably from 1999 to 2005, the constriction caused by transition structure still exerts primary hydraulic control in Unit 4. Note in Figure 2 that the model-calculated critical water surface elevation at the structure location (20,330 ft main channel distance) is the same for 1999 and 2005 geometry conditions. From this analysis, it is predictable that removing the existing ladder structure will substantially reduce model-calculated water surface elevation profile in Unit 4, and is indeed the only way to increase design channel capacity of Unit 4 without constructing a floodwall in the downstream most 200-ft length of Unit 4.

PART 3. BASELINE EXISTING CONDITIONS MODEL DEVELOPMENT

Part 3 Purpose

Part 3 is intended to document the development of the appropriate geometry file for representing the “baseline” existing condition which model results for all final design alternatives can be uniformly compared to. The baseline existing condition is both technically and substantively different from the 2005 existing condition developed in Part 2 because it includes: assumed Lagunitas Road Bridge removal and replacement by hydraulic design; and additional 2005 survey cross-sections at key design locations.

Part 3 shows how hydraulically different the baseline existing condition is from the 2005 existing condition developed in Part 2, mainly because of the assumed removal and replacement of the Lagunitas Road Bridge structure. [It is specifically assumed that the replacement bridge structure will be designed according to the design capacity of the preferred design alternative, such that it will not interfere with or constrict Corte Madera Creek at flows less than the design discharge. In effect, the replacement bridge structure will be a clear-span bridge deck with the low chord elevation at or above the model-calculated water surface elevation for the design capacity discharge.]

In this way, the baseline existing conditions model development primarily demonstrates how much removing the existing Lagunitas Road Bridge structure would reduce water surface elevations in Unit 4, what the existing bridge's backwater effect is at the Ross gage, etc. This is done by substituting additional 2005 topographic survey data into the 2005 existing conditions model geometry file and completely removing bridge data, and then comparing the water surface elevation profiles the model generates for the 2005 existing conditions and updated 2005 baseline geometry files (Figure 3). Incremental development of the Baseline (Existing Conditions) Alternative is documented below, as shown in Figures 4-6.

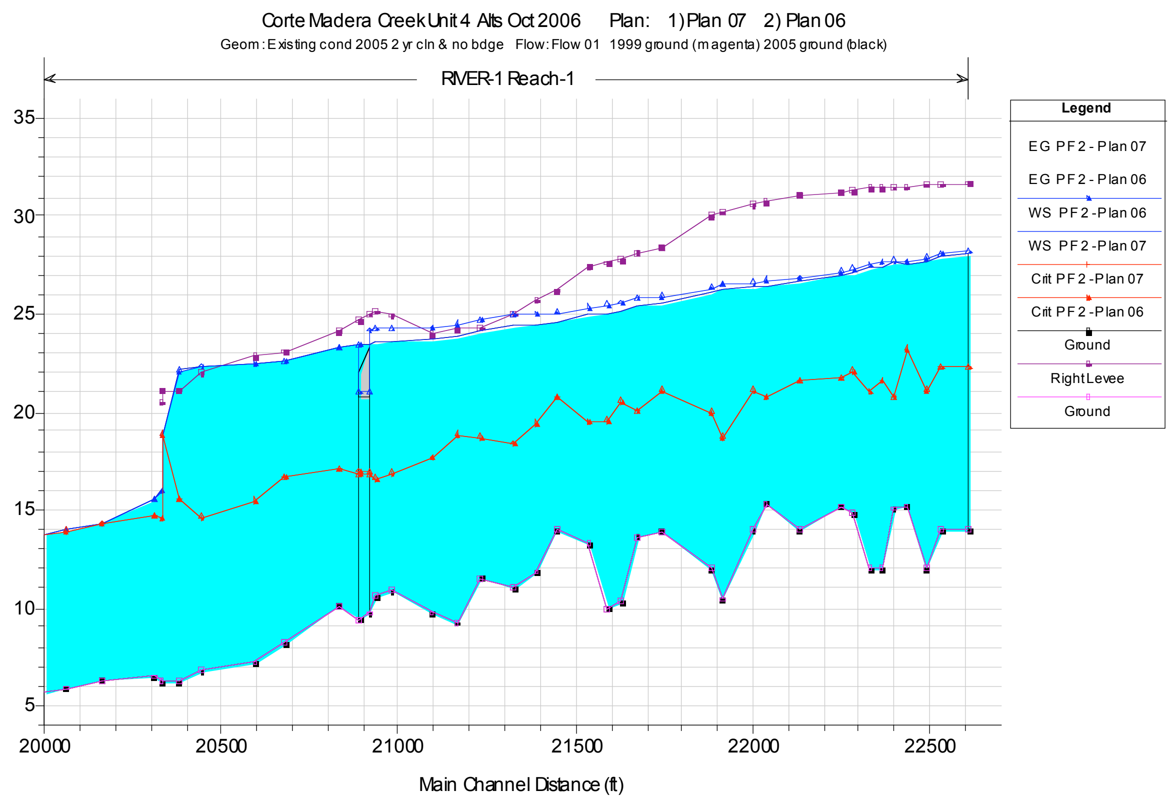

Figure 3. *Plan 06/07.* Comparison of calculated water surface profiles for PF2 (3,250 cfs) in the Unit 4 reach for 2005 existing conditions geometry with the existing Lagunitas Road Bridge intact (g05, Plan 06) and for 2005 existing conditions geometry with Lagunitas Road Bridge removed (g06, Plan 07).

- *Lagunitas Road Bridge removal reduces flood water surface elevations upstream but not downstream from the bridge location.*

- *For 2005 existing conditions, bridge removal reduces the flood water surface elevation of a 3,250 cfs flood flow approximately 0.8 feet immediately upstream from the bridge location, tapering off to approximately 0.1 feet at the Ross Ck tributary confluence (approx. 22,700 ft main channel distance).*

- *For 2005 existing conditions, the "backwater effect" of the existing bridge configuration for a 3,250 cfs discharge is approximately 0.5 feet at the Ross gage (approx. 21,320 ft main channel distance).*

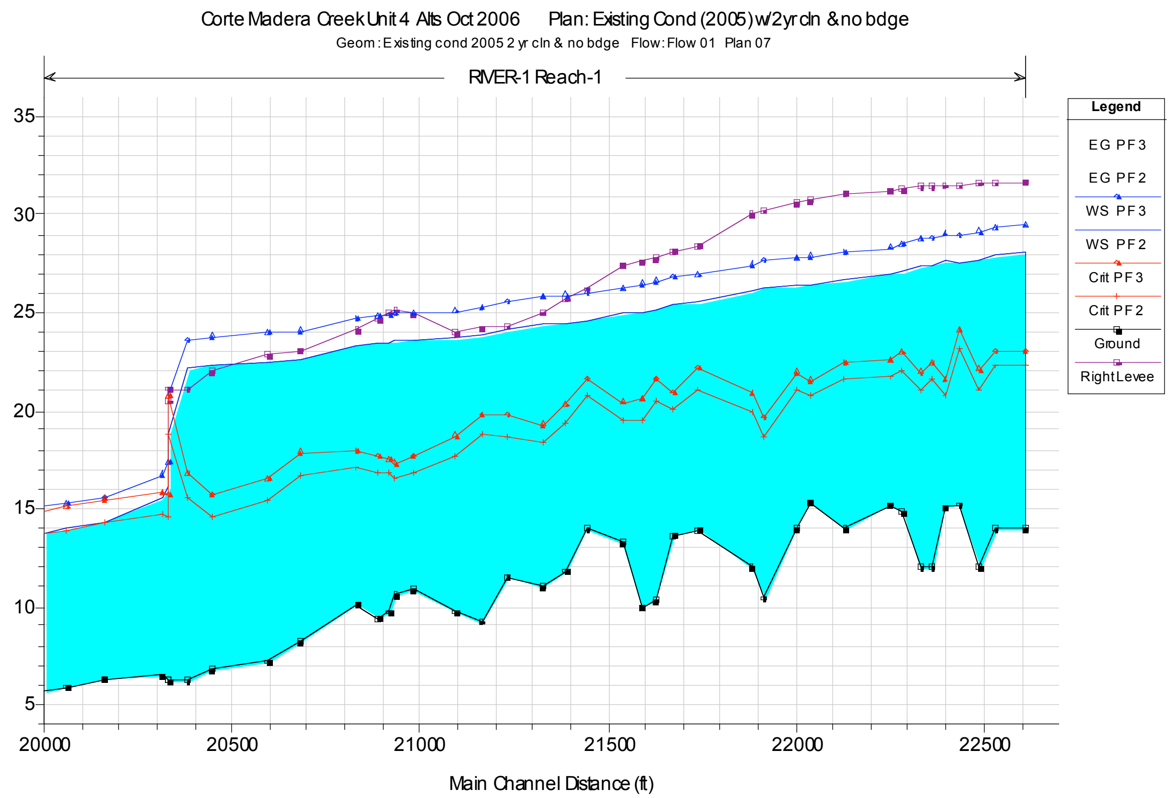

Figure 4. *Plan 07.* Comparison of calculated water surface profiles for PF2 (3,250 cfs) and PF3 (4,100 cfs) in the Unit 4 reach for 2005 existing conditions geometry with the Lagunitas Road Bridge structure removed (g06, Plan 07). The geometry file includes the existing wooden Denil fish ladder structure at the Unit 3/Unit 4 transition.

- *Under 2005 existing conditions with Lagunitas Road Bridge removed, the channel capacity at and upstream from the Lagunitas Road Bridge location is approximately 4,100 cfs. This discharge is not contained in the channel* *downstream from the bridge location because of the backwater effect from the existing timber bulkhead grade control wooden Denil fish ladder structure.*

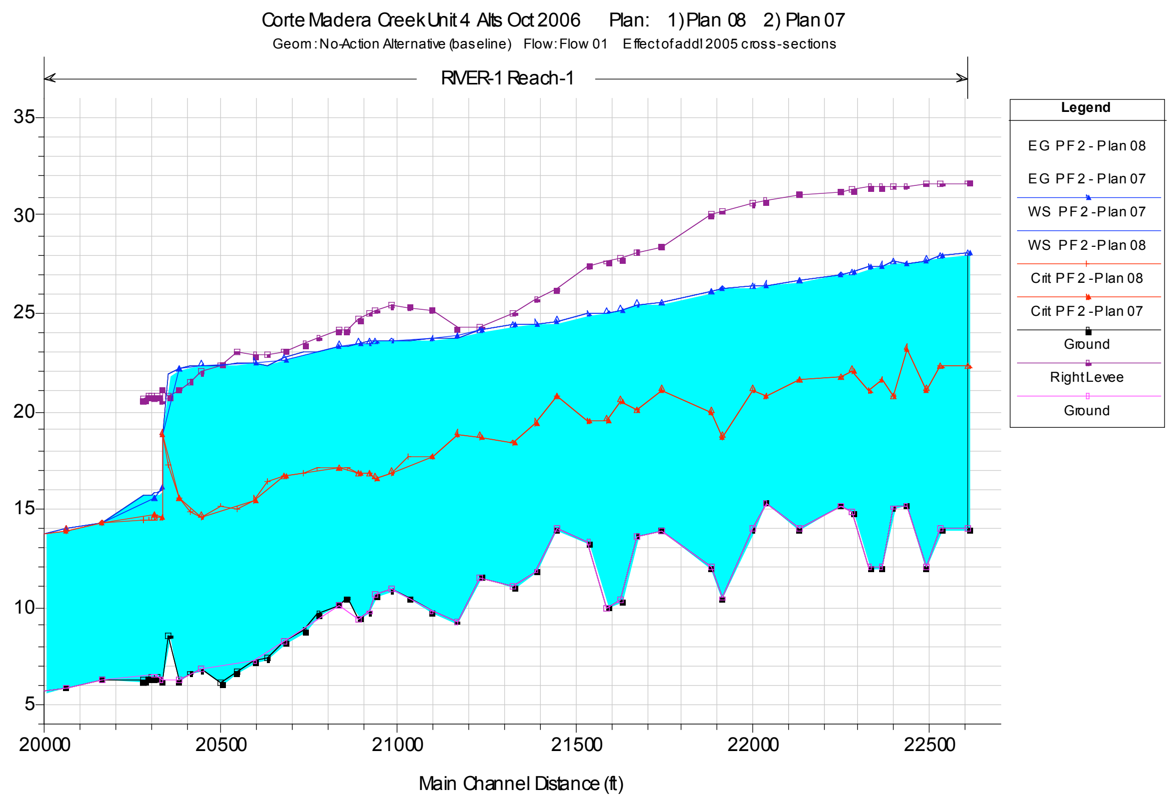

Figure 5. *Plan 07/08.* Comparison of calculated water surface profiles for PF2 (3,250 cfs) in the Unit 4 reach for two geometry conditions: 2005 existing conditions geometry with Lagunitas Road Bridge removed (g06, Plan 07); and 2005 existing conditions geometry with Lagunitas Road Bridge removed and additional 2005 cross-sections added to create a refined baseline existing conditions geometry file (g07, Plan 08). Plan 08 is the Baseline (Existing Conditions) Alternative. The 2005 minimum bed elevation profile for g07 (Plan 08) (black) shows the effect of adding cross-sections, compared to 2005 bed profile for g06 (Plan 07) shown in magenta. Note that the spike in the bed profile for g07 (Plan 08) (black) near 20,350 ft main channel distance is due to the addition of an intervening cross-section transecting the existing fish ladder structure. Despite its drastic appearance on the minimum channel bed elevation profile, the actual isolated hydraulic effect of adding this cross-section is negligible.

- *The minor changes in minimum bed elevation and channel geometry created by adding 2005 cross-sections to the existing conditions model produce negligible differences in calculated water surface elevation.*

- *Under the Baseline (Existing Conditions) Alternative (Plan 08), the Unit 4 channel capacity is the same as 2005 existing conditions – approximately 3,200 cfs.*

The bridge removal simulation in Plan 07 shown in Figure 3 did not include changes to channel geometry in the vicinity of the bridge location that may be necessary to maximize the design hydraulic capacity of the replacement bridge structure. Figure 6 below shows that reconfiguring the existing bridge abutments to make the channel wider in the vicinity of the bridge has a negligible hydraulic effect on channel capacity for the Baseline Existing Conditions Alternative.

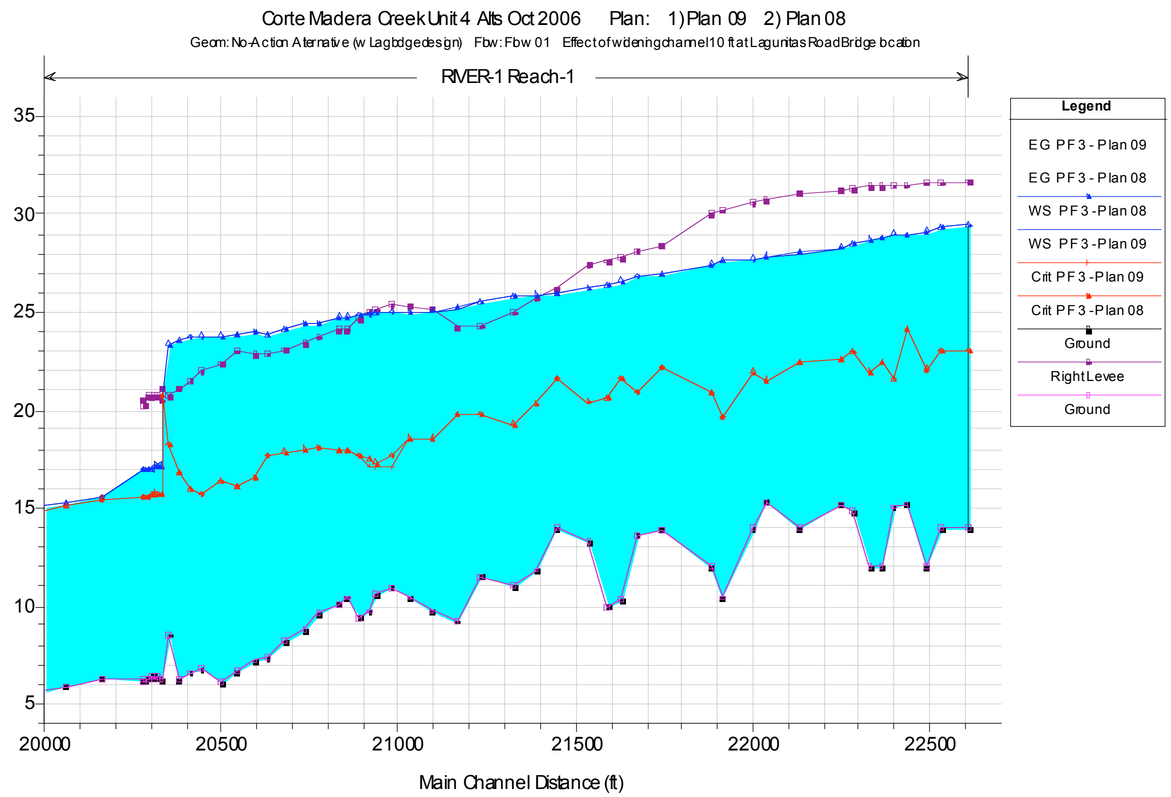

Figure 6. *Plan 08/09.* Comparison of calculated water surface profiles for PF3 (4,100 cfs) in the Unit 4 reach for the Baseline condition (g07, Plan 08) and the Baseline condition with 10-ft widened channel in the vicinity of the Lagunitas Road Bridge (g08, Plan 09). Plan 09 represents replacing the existing Lagunitas Road Bridge abutments with new vertical abutments set back 5 horizontal feet on both the left and right banks (10-ft overall widening between abutments).

- *Setting back both right bank and left bank vertical concrete abutment walls 5 horizontal feet creates a negligible difference in calculated water surface elevation in the immediate vicinity of the Lagunitas Road Bridge location. Additional setbacks would create no additional flood benefit.*

Part 3 Summary

Bridge removal reduced model-calculated water surface elevations and increased maximum channel capacity at and upstream from the bridge location but did not increase Unit 4 channel capacity immediately upstream from the Unit 3 concrete channel. Additional 2005 survey cross-sections had a negligible hydraulic effect. Reconfiguring the bridge abutments to widen the channel in the vicinity of the bridge (simulated as part of bridge replacement design) had a negligible hydraulic effect because the existing distance between abutments is sufficient for design channel capacities up to 4,100 cfs. Because these modifications did not reduce model-calculated water surface elevations immediately upstream from Unit 3 concrete channel, the Unit 4 channel design capacity for the Baseline (Existing Conditions) Alternative is essentially the same as the 2005 existing conditions capacity – 3,200 cfs. The recommended Baseline (Existing Conditions) Alternative model file (Plan 08) includes the existing Lagunitas Road Bridge abutments in their current configuration.

PART 4. DESIGN ALTERNATIVE MODEL DEVELOPMENT

Part 4 Purpose

With the Baseline (Existing Conditions) Alternative model (Plan 08) established in Part 3, Part 4 is intended to document model development and technical justification leading up to each of the recommended final Design Alternatives for fish passage improvement, bank stability, and flood management improvement in Unit 4.

Alternative I is the No-Action alternative intended to improve fish passage without also increasing flood flow capacity of Unit 4. Other alternatives are intended to increase flood flow capacity compared to existing conditions. Careful inspection of the model-calculated critical water surface elevation profile (red) for existing Unit 4 channel geometry (Figure 5) shows that there are at least three primary flood flow constraints in Unit 4 that need to be addressed through the design alternatives:

1. Unsmooth transition between Unit 4 and Unit 3 created in whole or in part by the existing timber bulkhead grade control structure and the wooden Denil fish ladder immediately upstream from the Unit 3 concrete channel River Station 369+71 (main channel distance 20,330 ft) and relatively narrow channel cross-sections 20 ft and 50 ft upstream at River Stations 369+91 ft and 370+21 ft, respectively.

    - *Strategy: Remove the existing fish ladder and modify the bed and banks in the downstream section of Unit 4 to make the transition to Unit 3 as hydraulically smooth as possible while still protecting the existing sanitary* *sewer siphon passing beneath the channel, providing improved fish passage at the Unit 3/Unit 4 transition, and not inducing upstream bed and bank erosion or destabilizing existing upstream stabilization structures.*

2. Narrow channel constriction created by the existing east bank vertical concrete retaining wall at 27 Sir Francis Drake Blvd in the vicinity of model river station 373+00 ft (main channel distance 20,650 ft).

    - *Strategy: Because it is assumed that the existing east bank retaining wall cannot be modified or destabilized by the design, the west bank must be graded back to a steeper slope in the vicinity of the wall, including removal of existing canopy-forming riparian trees.*

3. Narrow channel constricting created by the existing west bank vertical sheetpile retaining wall at 1 Sylvan Lane in the vicinity of model river station 378+00 ft (main channel distance 21,150 ft).

    - *Strategy: Because it is assumed that the existing west bank retaining wall cannot be modified or destabilized by the design, the east bank (Town of Ross property) must be graded back to a steeper slope in the vicinity of the wall, including removal of existing canopy-forming riparian trees.*

The physical nature of subcritical open channel flow hydraulics is such that flood flow constraints (i.e., narrow cross-sections) exert a hydraulic effect upstream but not downstream from the constraint. Therefore, in order to avoid modeling overlapping hydraulic effects of multiple channel modifications, downstream flood flow constraints must be addressed first, then the next upstream constraints can be addressed, etc. For example, the appropriate design cross-section width and shape at the east bank retaining wall constricted cross-section approx. 330 ft upstream from Unit 3 cannot be correctly determined through modeling until the (potentially overlapping) upstream hydraulic effect of removing the downstream existing grade control fish ladder structure is eliminated through model design.

In Part 4, design alternatives are developed by first maximizing the flood management benefits of addressing the downstream most hydraulic constraint (existing grade control-fish ladder structure). Second, the resulting model is evaluated to determine if any new hydraulic constraints are revealed by these actions, and each successive hydraulic constraint is mitigated by design (i.e., simulated streambank grading) proceeding upstream direction. This incremental design approach generates numerous design variants (individual model plan files) for each potential design alternative. The purpose, design, and results of each variant are presented and explained in this memorandum to document the range of design considerations leading up to the recommended Design Alternatives. The final recommended Design Alternatives are presented in Part 5, including new model runs to more precisely document the model-estimated design capacity of each.

The design analysis includes an emphasis on using biotechnical bank stabilization techniques to stabilize design-modified channel banks to the extent feasible. Biotechnical or bioengineered bank stabilization structures are generally defined as those that maximize use live native vegetation, live logs and woody debris, natural soil, and biodegradable fabric materials, and minimize use of traditional engineering bank stabilization materials such as rock rip-rap, and vertical concrete and steel retaining walls. Feasibility of biotechnical techniques is generally limited where existing or design bank slopes exceed approximately 45 degrees or 1(H):1(V). Most of the channel banks in Unit 4 approach or exceed 1(H):1(V) slope under existing conditions. Because project alternatives designed to increase channel capacity compared to existing conditions will necessarily require making the channel wider and the channel banks steeper than existing conditions, it follows that biotechnical techniques will not be feasible everywhere in Unit 4 for all alternatives. The recommended design alternatives will seek to include the maximum feasible amount of biotechnical bank stabilization structures. For aesthetic reasons, the recommended design alternatives will also seek to apply biotechnical techniques more universally on the west bank where they will be more visible from public areas including the pedestrian right-of-way.

Therefore, although biotechnical stabilization is feasible and environmentally preferable in and of itself, and feasible at most Unit 4 locations under existing conditions, requiring biotechnical stabilization measures for the design alternatives would severely constrain the design channel capacities. Part 4 documents an incremental model development procedure that demonstrates how much applying biotechnical bank stabilization measures limits design channel capacity. The procedure begins by modeling design variants applying only biotechnical measures, and proceeds to model design variants that include vertical retaining walls. This way design alternatives could be generated that would provide reasonable and worthwhile design capacities, in such a way that use of biotechnical measures is maximized, and aesthetic and environmental impacts are minimized to the extent feasible.

Alternative I (No-Action) – Permanent Replacement Fish Ladder

Alternative I is the No-Action alternative wherein the existing temporary and now partially failed Denil type fish ladder would be replaced with a permanent type fish ladder structure in roughly the same configuration and intended to have a similar grade control and hydraulic effect upstream in Unit 4. Michael Love & Associates (MLA) prepared a preliminary fish ladder concept design for replacing the existing, partially failed wooden Denil fish ladder at the transition between Unit 3 and Unit 4. The proposed permanent replacement fish ladder structure is situated entirely within the upstream end of the existing Unit 3 concrete channel, partly to avoid potential impacts to the existing sanitary sewer siphon passing beneath Corte Madera Creek about 20-25 ft upstream from Unit 3. The proposed replacement structure is explicitly designed to create the same grade control effect and at the same location as the existing timber bulkhead grade control structure. Therefore, by design, it should have virtually the same hydraulic effect as the existing structure. That is, the design channel capacity under Alternative I is not expected to be more than the Baseline (Existing Conditions) channel capacity. The proposed replacement fish ladder footprint is shown in Figure 33 (see Appendix B). For more detailed information on the proposed replacement fish ladder design concept, refer to Appendix A to review the fish ladder concept design report by MLA dated May 30, 2006.

To develop the initial Alternative I geometry file (g09), design cross-sections were developed corresponding to each of the 5 weir elements in the proposed replacement fish ladder structure and substituted into the Baseline (Existing Conditions) Alternative geometry file (g07).

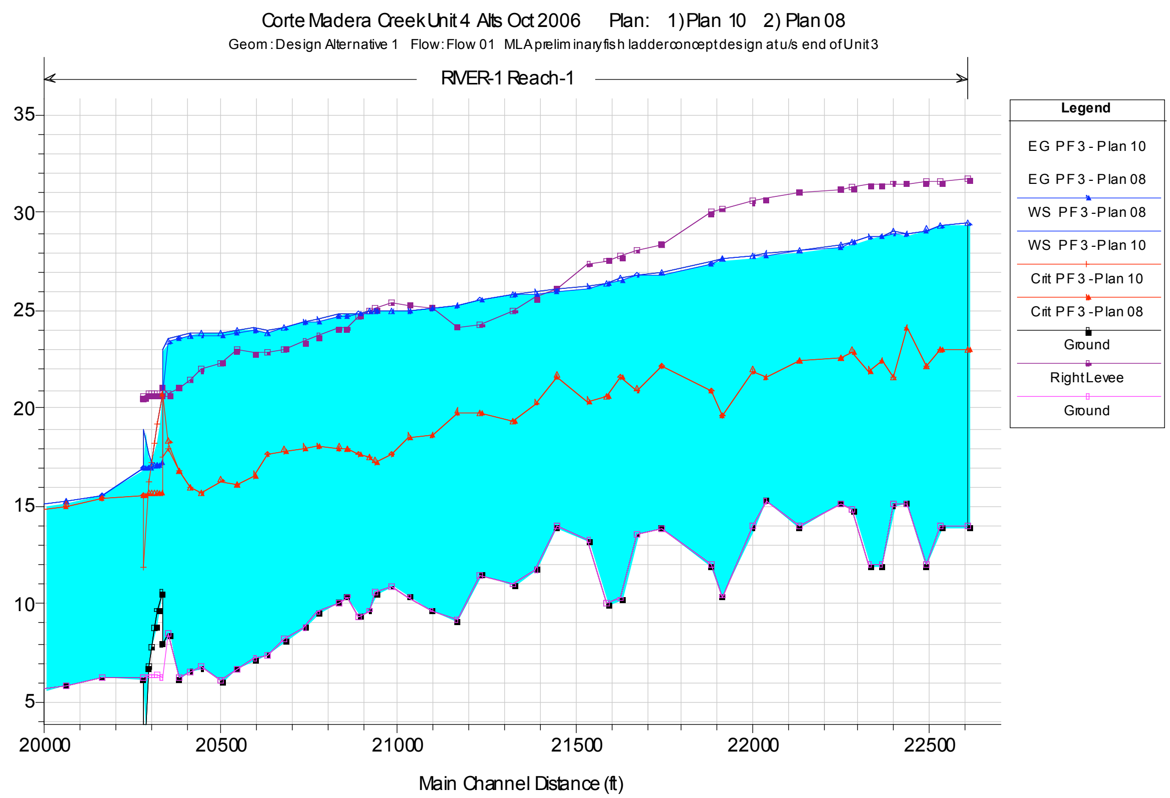

Figure 8. *Plan 08/10.* Comparison of calculated water surface profiles for PF3 (4,100 cfs) in the Unit 4 reach for the Baseline (Existing Conditions) Alternative (g07, Plan 08) and the Alternative I (No-Action) test run (g09, Plan 10). The Baseline (Existing Conditions) Alternative includes the existing timber bulkhead grade control and wooden Denil fish ladder structure, and Design Alternative I includes geometry for the preliminary replacement fish ladder concept design within the upstream end of Unit 3 concrete channel (MLA, May 30, 2006). Comparison of Plan 08 and Plan 10 demonstrates the isolated effect of replacing the existing grade control structure and partially failed wooden ladder with the MLA-proposed permanent fish ladder structure.

- *Replacing the existing wooden Denil fish ladder in the downstream end of Unit 4 with the MLA-proposed concrete fish ladder in the upstream end of Unit 3 produces a negligible change in model calculated water surface elevation for the 4,100 cfs discharge.*

- *Under Alternative I (No-Action), the Unit 4 channel capacity is approximately the same as under the Baseline condition (3,200 cfs).*

The MLA-proposed preliminary fish ladder concept produces a negligible difference in model-calculated water surface elevations in Unit 4 because the design cross-section of the most upstream replacement weir (Weir No. 5) at River Station 369+70 ft is designed to be approximately the same as the cross-section of the existing timber bulkhead forming the downstream end of the existing wooden Denil fish ladder structure (at River Station 369+71 ft). Therefore, the hydraulic influence of the existing and proposed replacement fish ladder structure on Unit 4 flood hydraulics and depths is both theoretically and practically the same.

The MLA-proposed replacement fish ladder therefore would have no impact on flood flow depths in Units 1-3, and a negligible or no impact on flood flow depths in Unit 4. Because the MLA-proposed replacement structure is entirely within the existing Unit 3 concrete channel, no bank stabilization work would be necessary upstream from the structure in addition to what was recently completed at the right bank immediately upstream (at 21 Sir Francis Drake Blvd.). Moreover, because the proposed replacement structure replicates the downstream most primary flood flow constraint in Unit 4 which exerts a backwater effect extending at least approx. 800-1,000 ft upstream from the Unit 3 concrete channel, the potential flood management benefits of including bank regrading and customizing the hydraulic design Lagunitas Road Bridge replacement in Design Alternative I are significantly reduced, but nonetheless explored and documented below.

Potential for Unit 4 Bank Regrading and Stabilization for Flood Management Improvements under Alternative I (No-Action)

The proposed concept permanent replacement fish ladder structure intentionally leaves unchanged the downstream most primary flood flow constraint in Unit 4. For this reason additional bank regrading work in Unit 4 is unlikely to substantially improve flood capacity. To verify this, a number of bank regrading actions targeting the remaining primary flood flow constraints are explored through model design, in the downstream to upstream direction:

1. Grading back the right (west) bank in the downstream 100 ft of Unit 4 to smooth the hydraulic transition between Unit 4 and Unit 3.

2. Grading back right (west) bank across from the encroached left bank private residential vertical concrete retaining wall structure at 27 Sir Francis Drake Blvd (near River Station 373+00 ft) to make the channel width and cross-sectional area in the vicinity of the encroached retaining wall more similar to upstream and downstream channel sections.

3. Including in the recommended hydraulic design of Lagunitas Road Bridge replacement reconfiguring the existing bridge abutments to widen the channel at the Lagunitas Road Bridge location.

4. Grading back left (east) bank on Town of Ross property across from the encroached right bank private residential vertical sheetpile retaining wall structure at 1 Sylvan Lane (near River Station 378+00 ft) to make the channel width and cross-sectional area in the vicinity of the encroached retaining wall more similar to upstream and downstream channel sections.

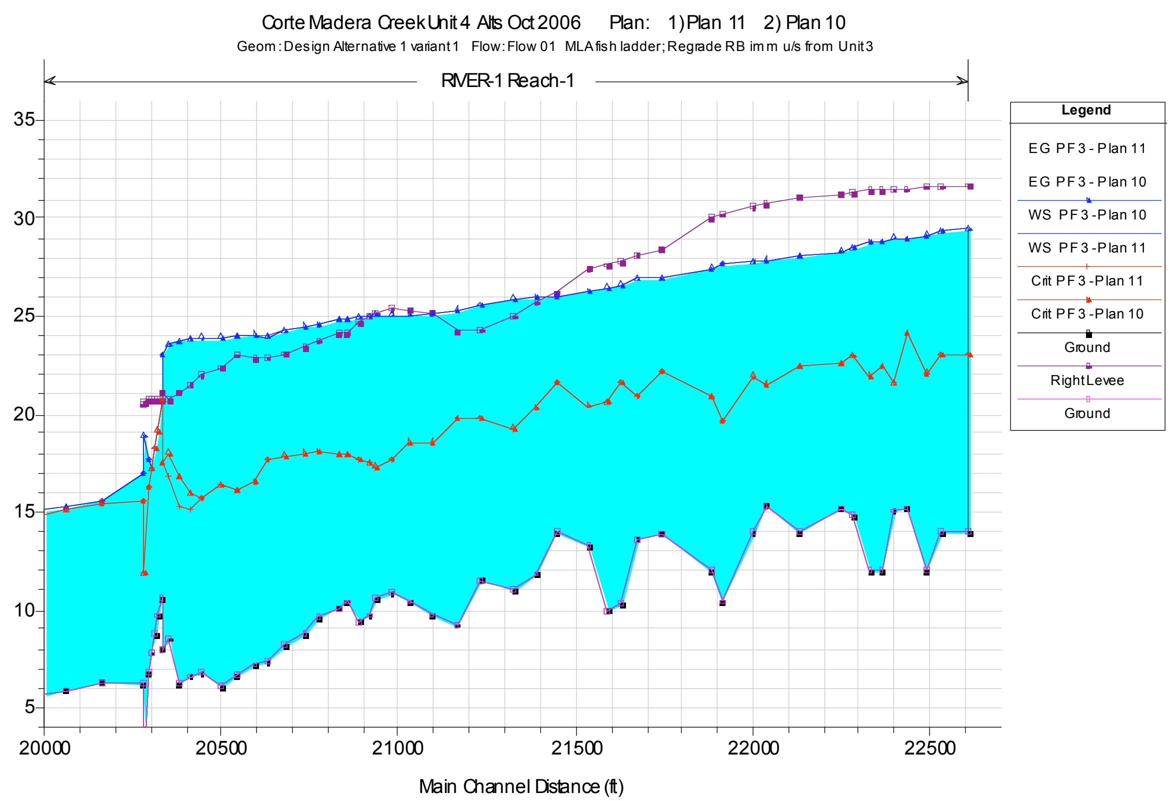

Figure 9. *Plan 10/11.* Comparison of calculated water surface profiles for PF3 (4,100 cfs) in the Unit 4 reach for the Design Alternative I test (g09, Plan 10) and the Design Alternative I Variant 1 (g10, Plan 11). Both geometry files include the preliminary replacement fish ladder concept design within the upstream end of Unit 3 concrete channel (MLA, May 30, 2006). Comparison of Plan 10 and Plan 11 demonstrates the

isolated effect of regrading the right bank in the downstream most 100-ft length of Unit 4 to smooth the transition from Unit 4 to Unit 3.

- *Regrading the right bank in the downstream most 100-ft length of Unit 4 to smooth the transition from Unit 4 to Unit 3 slightly reduces the critical depth in the regraded reach but has no effect on model calculated water surface elevations in Unit 4.*

- *Under the Design Alternative I Variant 1, the Unit 4 channel capacity is approximately the same as under the Baseline and No-Action Alternatives (3,200 cfs).*

<u>Design Alternative I Variant 1</u>

Design Alternative I Variant 1 tests the isolated effect of taking the first bank regrading action in the downstream end of Unit 4. Specifically, the right (west) bank would be graded back from existing 1(H):1(V) slope to steeper 1(H):2(V) slope beginning at the upstream end of the existing Unit 3 concrete channel and replacement fish ladder structure extending approx. 100 ft upstream. The modified bank would need to be stabilized with vegetated rip-rap or live vegetated log crib-wall structure. The simulated regrading work is designed to avoid the existing 24 inch-diameter sewer siphon pipes passing beneath the right (west) bank within the grading area. The hydraulic effect of this isolated action is tested as shown in Figure 9 by modifying the Design Alternative I test geometry file (g09) to simulate the above regrading work and running the model using the resulting Design Alternative I Variant I geometry file (g10).

Figure 9 shows the negligible flood management improvement that biotechnically designed bank regrading and stabilization measures would have under Design Alternative I because Weir No. 5 of the MLA-proposed fish ladder structure would continue to exert primary hydraulic control on the reach.

<u>Design Alternative I Variant 2</u>

Design Alternative I Variant 2 tests the combined effect of taking the first two bank regrading actions. Specifically, the downstream most 100 ft of the right (west) bank of Unit 4 would be regraded as described and model simulated above. In addition, a 100-ft length of the right (west) bank of Unit 4 would be graded back in the vicinity of the encroached left (east) bank private residential retaining wall, specifically from River Station 372+50 ft to River Station 373+50 ft. In order for the grading action to achieve the objective of widening the channel in the vicinity of the existing retaining wall enough to be similar to upstream and downstream channel sections, and at the same time preserve existing mid-bank and upper bank native riparian trees, it would be necessary to install an approx. 5 ft-high (exposed) vertical concrete or steel retaining wall along the entire 100-ft long grading area. Two native willow trees (Trees No. 46-47) and two non-native willow trees (Trees No. 48-49) would be removed to accommodate the channel widening. Figure 10 shows the existing (black) and proposed design (magenta) cross-section at River Station 373+18. The hydraulic effect of this combined action is tested as shown in Figure 11 by modifying the Design Alternative I Variant 1 geometry file (g10) to simulate the above additional regrading work and running the model using the resulting Design Alternative I Variant I geometry file (g11).

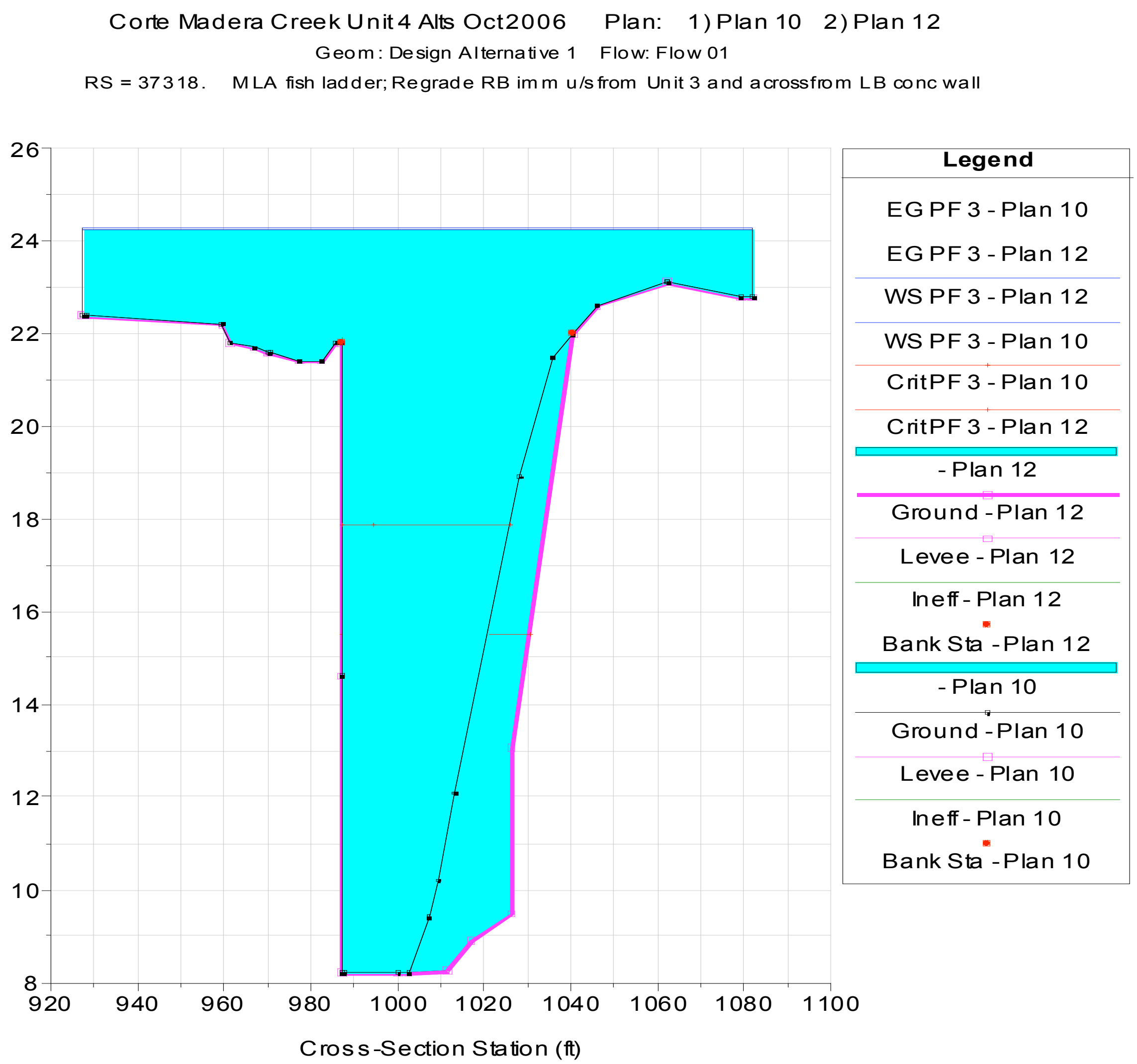

Figure 10. *Plan 10/12.* Comparison of existing cross-section geometry under Design Alternative I test (g09, Plan 10), and example proposed design cross-section geometry for Design Alternative I Variant 2 (g11, Plan 12) at River Station 37+318 ft (Main Channel Distance 20,678 ft). Under Design Alternative I Variant 2, the right bank is graded back approx. 20 horizontal ft near the toe of the bank. The toe of the new right bank profile would be stabilized with vertical concrete or steel sheetpile wall. Above the vertical wall, the upper bank would be sloped at maximum 1.5(H):1(V) and stabilized

with vegetated geoxtextile fabric cover. The existing chain-link fence at the top of the right bank is at Cross-Section Station 1050 ft. The left bank is an existing private residential vertical concrete retaining wall at 27 Sir Francis Drake Blvd.

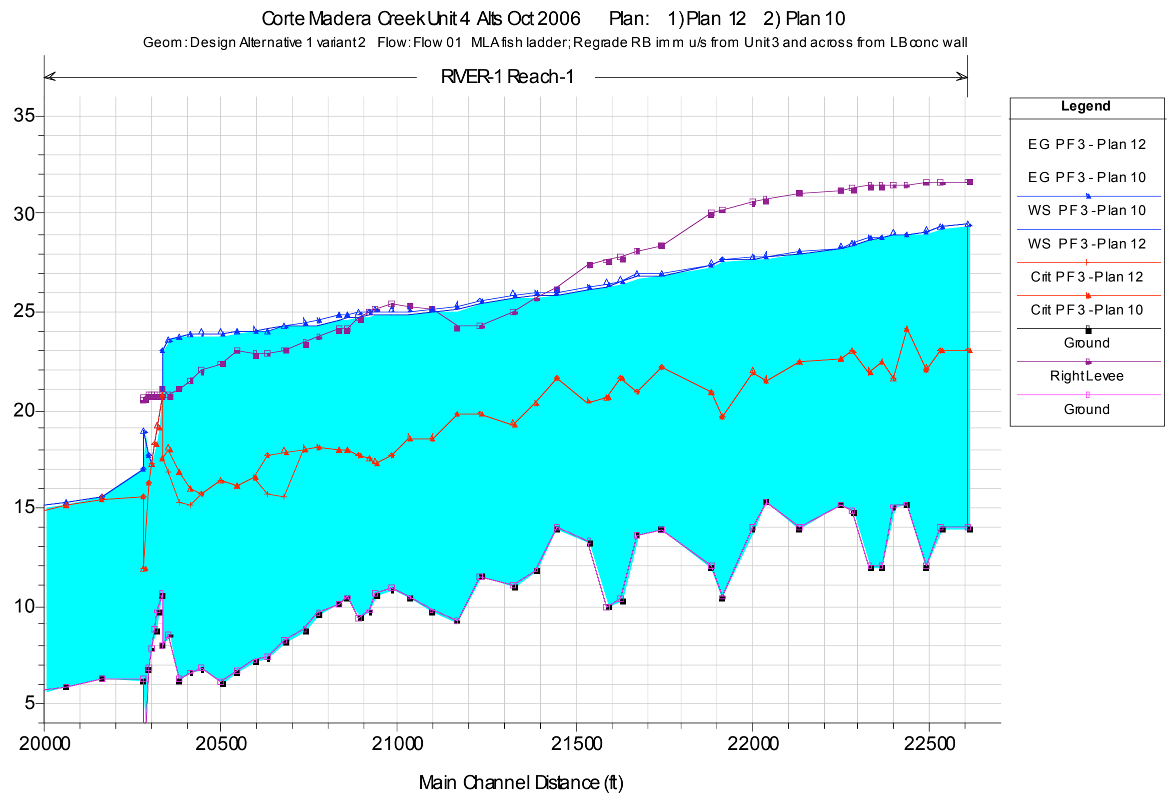

Figure 11. *Plan 10/12.* Comparison of calculated water surface profiles for PF3 (4,100 cfs) in the Unit 4 reach for the Design Alternative I test (g09, Plan 10) and the Design Alternative I Variant 2 (g11, Plan 12). Comparison of Plan 10 and Plan 12 demonstrates the combined effect of: (1) regrading the right bank in the downstream most 100-ft length of Unit 4 to smooth the transition from Unit 4 to Unit 3; and (2) regrading the right bank across from the existing left bank private residential vertical concrete retaining wall at 25 Sir Francis Drake Blvd.

- *Regrading the right (west) bank over a 100-ft length across from the left (east) bank existing private residential vertical retaining wall significantly reduces the critical depth in the regraded reach but only slightly reduces the flood water surface elevation in Unit 4.*

- *Under the Design Alternative I Variant 2, the Unit 4 channel capacity is approximately the same as under the Baseline and No-Action Alternatives (3,200 cfs).*

The above model design analyses confirm that combining relatively minor Unit 4 bank regrading actions with fish ladder replacement in Design Alternative I (No-Action Alternative) would produce a negligible flood capacity improvement. The analyses show that although the regrading actions significantly reduce the local critical water surface elevation the backwater effect from the proposed permanent replacement fish ladder structure continues to exert primary hydraulic control on the reach. No additional variants of Design Alternative I need to be model evaluated. In Part 5, the Design Alternative I test (g09, Plan 10) is adopted as the recommended Design Alternative I.

Design Alternative II – Natural Grade Alternative with Preserved West Bank Riparian Corridor

The hydraulic model design analysis documented above shows that Unit 4 flood flow capacity cannot be substantially increased under Design Alternative I (No-Action Alternative). Fish passage could also feasibly be improved by constructing an approx. natural grade, roughened rock channel type gradual grade control structure in the lower section of Unit 4. Restoring an approximation of the natural grade of Corte Madera Creek in the lowermost approx. 400-ft length of Unit 4 would provide for a smoother hydraulic transition from Unit 4 to Unit 3. Therefore, a natural grade passage structure would potentially provide both improved fish passage and substantially increased design flood flow capacity.

There are at least three major potential constraints for restoring a natural grade transition from Unit 4 to Unit 3: (1) protecting the existing sewer siphon running beneath the channel bed immediately upstream from the existing wooden Denil fish ladder; (2) concern over possible destabilizing by undercutting effect of reduced channel bed elevations on private residential property and existing structures along the lower reach of Unit 4; and, (3) providing suitable fish passage conditions.

<u>Protecting the existing sanitary sewer line</u>

The Ross Valley Sanitary District No. 1 (RVSD) maintains two 24-inch diameter reinforced concrete pipe siphon sewer lines running parallel, passing on a diagonal path beneath the bed of Corte Madera Creek immediately upstream from the existing wooden Denil fish ladder. Plan and profile views of the existing sewer line provided by RVSD are reproduced as appendices to the May 30, 2006 Michael Love and Associates report (see Appendix A).

The existing sewer lines are encased in concrete and the RVSD profile drawing in Appendix A shows that the top of concrete elevation is approx. 4.3 ft NGVD29 at the channel centerline. The bottom of new concrete elevation for the previously proposed Army Corps Unit 4 concrete channel extension was 5.6 ft – designed to provide 1.3 ft of clearance above the existing encasement. At a minimum, a natural grade design would include large rip-rap lining the channel bed in the vicinity of the existing siphon encasement. The minimum likely suitable rip-rap diameter for channel bed lining over the exposed top of siphon encasement is 2.5 ft. Accordingly, the minimum feasible channel bed invert at the channel centerline directly above the encasement is approx. 7.0 ft. Natural grade alternatives should provide a smooth hydraulic transition from the minimum design channel invert elevation of 7.0 ft at River Station 370+03 ft to Unit 3 inlet invert elevation 6.25 ft at River Station 369+70 ft (2.2 percent local slope).

Inspection of the existing conditions topography in this vicinity shows that the existing grades are approximately the same as the recommended design top of rip-rap lining grades. This means that the siphon protection could be accomplished by dredging about 1.5-2.0 ft from the width of the channel bed extending from the upstream end of the concrete channel about 60 ft, and lining the entire graded area with 1-2-ton and smaller (minimum 2.5-ft diameter) rock rip-rap.

<u>Possible bank destabilization under the natural grade alternative</u>

There is a concern that design alternatives that include removal of the existing timber bulkhead grade control wooden Denil fish ladder structure will cause channel bed incision upstream, which may destabilize existing natural channel banks and permanent bank stabilization structures. However, 2005 existing conditions minimum channel bed elevation profile indicates that the existing 4-ft high timber bulkhead is not acting as a particularly strong grade control. Grade control structures in gravel bed streams are typically completely filled with gravel up to the crest of the structure. The minimum and (by field inspection) average channel bed elevations upstream from the existing timber bulkhead structure are typically 3 ft lower than the crest elevation of the structure. The minimum bed elevations approach the crest elevation of the structure (approx. 10.5 ft elevation) about 500 ft upstream (at about 20,850 ft main channel distance). Still, it appears that the elevation of the downstream end of the gravel bar that annually forms beneath Lagunitas Road Bridge is influenced by the bulkhead crest elevation. Geomorphic reasoning would suggest that bulkhead removal without stabilization measures would cause the lower Unit 4 minimum channel bed elevation profile to adjust down 1-2 ft in places.

However, bulkhead removal would require, at a minimum, installation of a channel-spanning reinforced concrete cut-off wall immediately upstream from the Unit 3 channel, and channel-spanning heavy rip-rap channel bed lining extending 60-100 ft upstream to protect the sanitary sewer line. Extending the heavy rip-rap lining an additional 300 ft upstream would serve to fix the channel bed elevation at approx. 9.5 ft about 400 ft upstream from Unit 3 and completely protect the entire exposed length of the existing left (east) bank private residential vertical concrete retaining wall at 27 Sir Francis Drake Blvd. Additional upstream grade control structures do not appear necessary to replace the grade control effect of the timber bulkhead.

Providing for suitable fish passage

There does not appear to be a conflict between replacing the grade control effect of the timber bulkhead and providing suitable fish passage without a replacement permanent fish ladder structure. The minimum recommended channel bed lining described in the above section would establish a 400-ft long, 0.7 percent slope reach immediately upstream from Unit 3. The cross-section profile of the channel bed lining could be designed specifically for fish passage at low flows, similar to the design of a roughened-rock channel (refer to page 5 of Appendix A for a description of roughened rock channels). It should be expected that flood flows would deposit native sediment and natural gravel bars on a portion of the rip-rap lined bed.

Hydraulic effect of removing the existing timber bulkhead grade control structure and wooden Denil fish ladder – Design Alternative II test

To test the effect of simply removing the existing grade control and fish ladder structure and redistributing some of the existing approx. 18-inch diameter rip-rap upstream from the structure, the No-Action Alternative (baseline) geometry file (g07) was modified to subtract the structure at River Station 369+71 ft, and slightly flatten the channel bed grade at cross-sections 369+91 ft and 370+21 ft. The resulting initial Design Alternative II test plan (g12, Plan 13) was run against the Baseline (Existing Conditions) Alternative (Plan 08). Figure 12 shows that removing the existing grade control fish ladder structure significantly reduces the model-calculated water surface elevation in Unit 4.

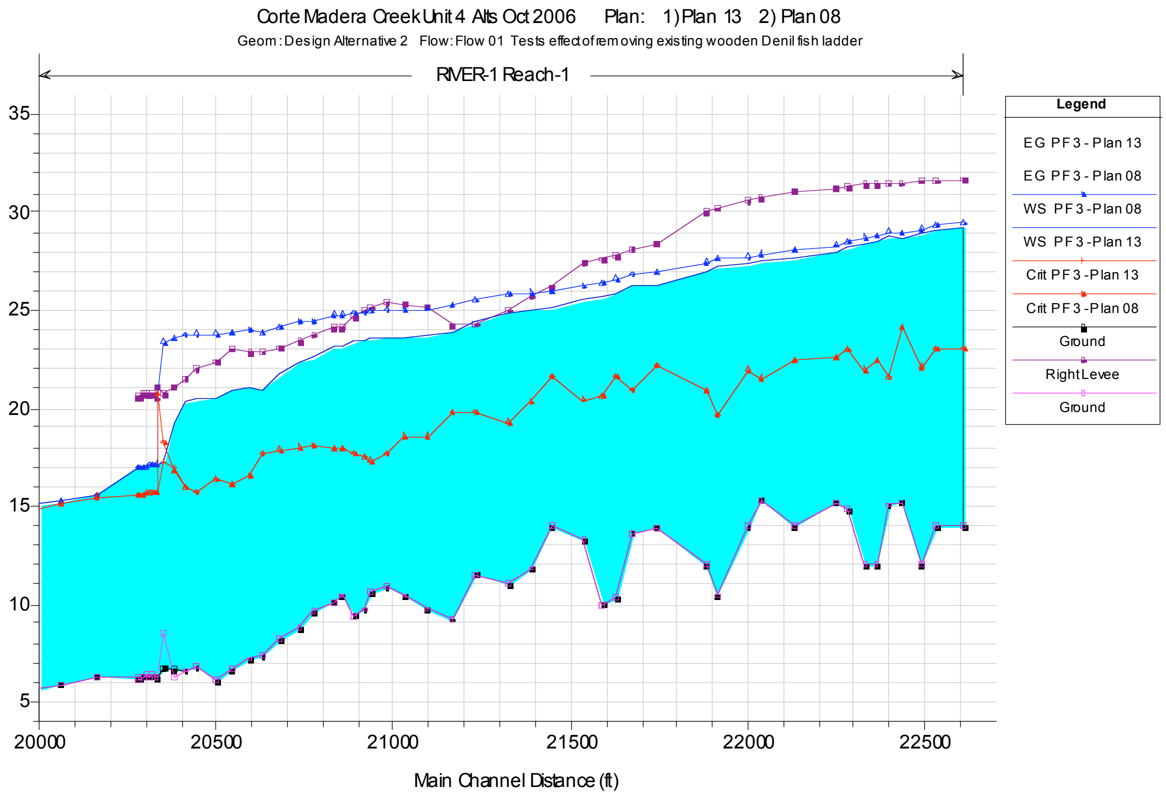

Figure 12. *Plan 08/13.* Comparison of calculated water surface profiles for PF3 (4,100 cfs) in the Unit 4 reach for the Baseline (Existing Conditions) Alternative (g07, Plan 08) and the Design Alternative II test (g12, Plan 13). Comparison of Plan 08 and Plan 13 demonstrates the isolated effect of only removing the existing grade control fish ladder structure and locally redistributing some of the existing rip-rap immediately upstream from the structure.

- *Simply removing the existing grade control fish ladder structure and locally redistributing some of the existing rip-rap (not including any channel widening, tree removal, or bank regrading in the Unit 4 reach) would reduce the 4,100 cfs water surface elevation approx. 6 vertical ft at the entrance to Unit 3, decreasing uniformly to an approx. 1.5 vertical ft reduction at the Lagunitas Road Bridge location, then to about 0.2 ft at the Ross Creek tributary confluence.*

- *Under this initial Design Alternative II test, the Unit 4 flood capacity increases from approx. 3,200 cfs for the Baseline and No-Action Alternatives to more than 4,100 cfs.*

The Alternative II test condition design capacity is determined more precisely below (Figure 13).

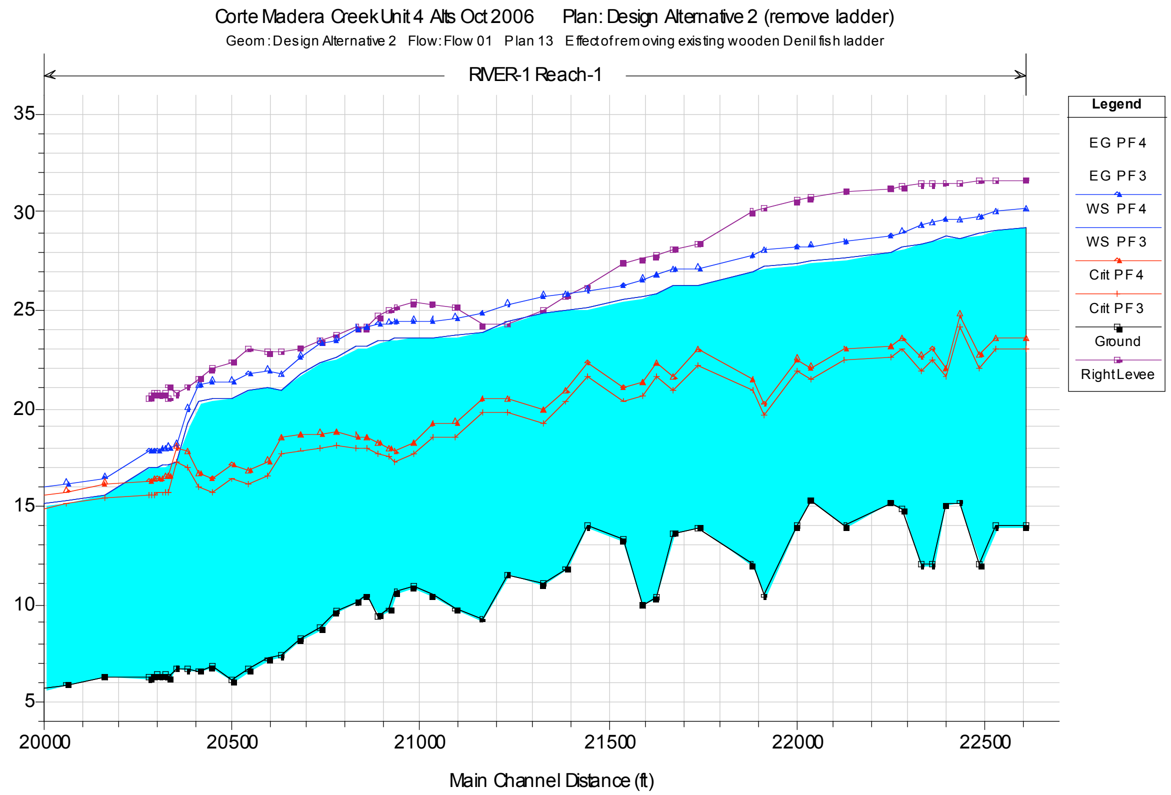

Figure 13. *Plan 13.* Comparison of calculated water surface profiles for PF3 (4,100 cfs) and PF4 (4,700 cfs) in the Unit 4 reach for the initial Design Alternative II test (g12, Plan 13).

- *Under the initial Design Alternative II test, the Unit 4 flood conveyance capacity increases from approx. 3,200 cfs for the baseline (existing condition) to slightly more than 4,700 cfs, approx. 5,000 cfs.*

The above model documentation shows that the Unit 4 flood conveyance capacity can be increased from 3,200 cfs to approx. 5,000 cfs simply by removing the existing grade control fish ladder structure and locally redistributing some of the existing rip-rap upstream from the structure. Inspection of the water surface and critical water surface profiles in Figure 13 shows that the water surface elevation remains strongly controlled by the constriction at the entrance to Unit 3 concrete channel, and numerous upstream constrictions. Additional, more comprehensive Design Alternative II variants are developed below to reduce these constraints the extent feasible considering the physical constraints discussed above: sewer line protection; grade control; and fish passage improvement.

Design Alternative II Variant 1

Design Alternative II Variant 1 is intended to: (1) improve fish passage with an approx. natural grade roughened rock channel type fish passage structure; (2) increase flood capacity; and, (3) preserve the entire right (west) bank riparian corridor.

With the existing grade control fish ladder structure removed, the next upstream flood flow constraint is in the downstream 100-110-ft length of Unit 4 where the channel is approx. 20-ft wide, substantially less than the 33-ft design width of the Unit 3 concrete channel. Note the resulting high critical water surface elevation between 20,350 ft and 20,400 ft main channel distance (Figure 13). Both the left and right banks are sloped approx. 1(H):1(V) or steeper under existing conditions. Widening the channel 10-15 ft would therefore require installation of an approx. 114-ft long maximum 9-ft high vertical retaining wall at mid-bank from River Station 369+80 ft to River Station 370+70 ft (Plates 1 and 2, in Appendix C).

Virtually all of the existing native riparian canopy-forming vegetation in the proposed grading area (native willow) was recently removed during emergency construction access to construct the left (east) bank rip-rap bank at 21 Sir Francis Drake Blvd immediately following the December 31, 2005 flood. The grading activities would therefore have negligible impacts on the existing riparian corridor.

To simulate the effect of channel widening and installation of a 114 ft-long vertical retaining wall, the initial Design Alternative II test geometry file (g12) was modified accordingly at River Stations 369+91 ft, 370+21 ft, 370+51 ft, and 370+83 ft. The configuration of the proposed retaining wall is shown in Figure 34 (see Appendix B). Figure 14 shows the typical modified cross-section profile at River Station 370+21 ft – the critical cross-section for evaluating potential impacts on the existing sewer line where it passes on a slope under the right bank before continuing under the channel bed.

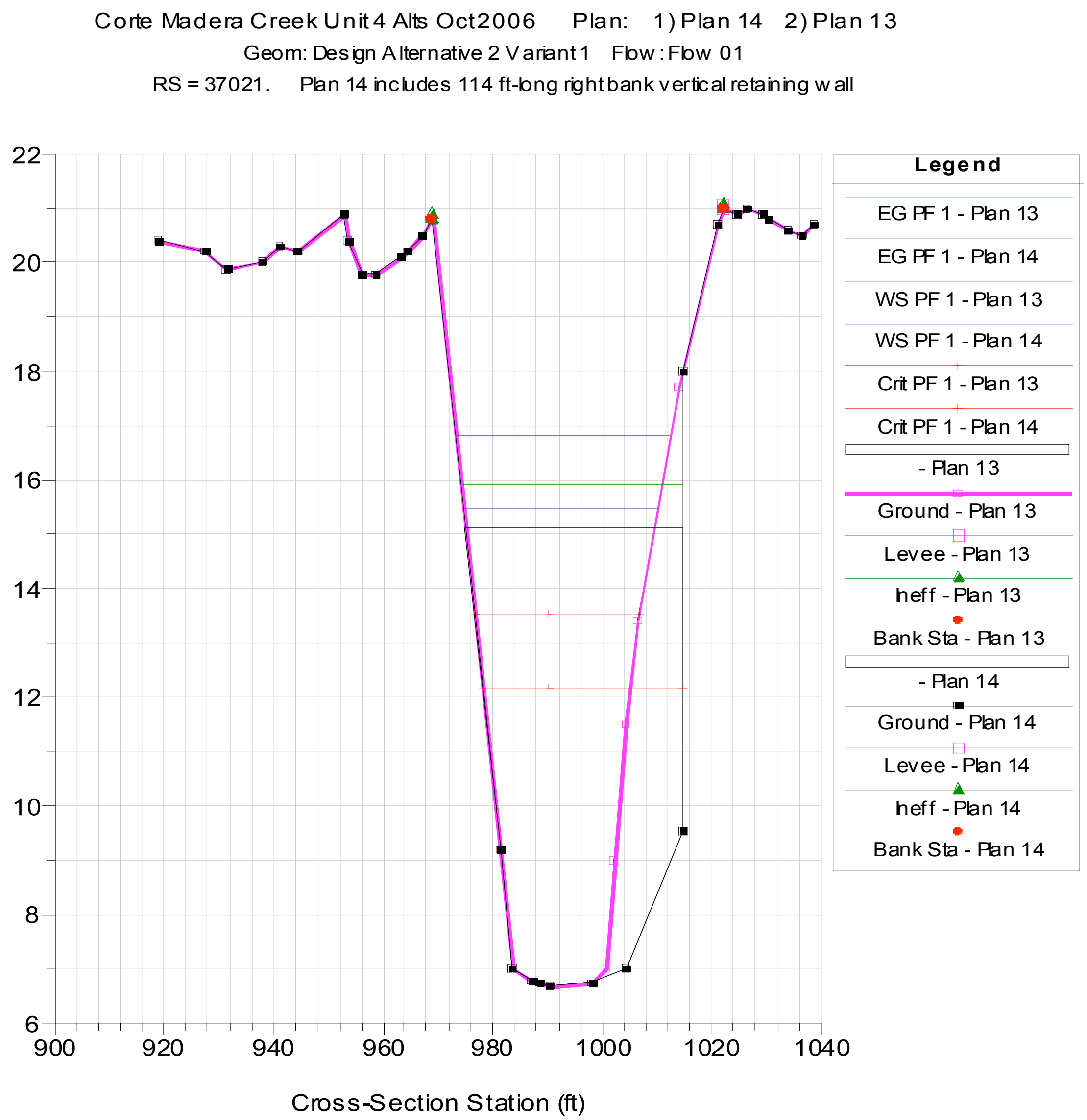

Figure 14. *Plan 13/14*. Comparison of cross-section geometry for existing conditions (Design Alternative II test) (g12, Plan 13) and Design Alternative II Variant 1 (g13, Plan 14) at River Station 370+21 ft. The top of the existing sewer siphon pipe concrete encasement at River Station 370+21 ft is at approx. elevation 5.6 ft at Cross-Section Station 1009 ft.

To simulate the effect of protecting the existing sewer line and improving fish passage the geometry file (g12) was modified to simulate lining the width of the channel with 42-inch diam rip-rap along the 114-ft length of proposed retaining wall, and with 18-24-inch rip-rap upstream from the upstream end of the proposed retaining wall up to River Station 373+74 ft (including the entire length of the existing left bank private residential vertical concrete retaining wall at 27 Sir Francis Drake Blvd). The proposed rip-rap lining would meet the fish passage, sewer line protection, and grade control objectives discussed above. Figure 15 shows how the proposed rip-rap lining changes the minimum channel bed elevation profile from the initial Design Alternative II test run (magenta) to the Design Alternative II Variant I (black).

To test the combined effects of installing the 114-ft long right bank retaining wall and installing a 400-ft long 0.7 percent sloped roughened rock channel type fish passage and grade control structure, the resulting Design Alternative II variant 1 plan (g13, Plan 14) was run against the initial Design Alternative II test plan (g12, Plan 13) (Figure 15).

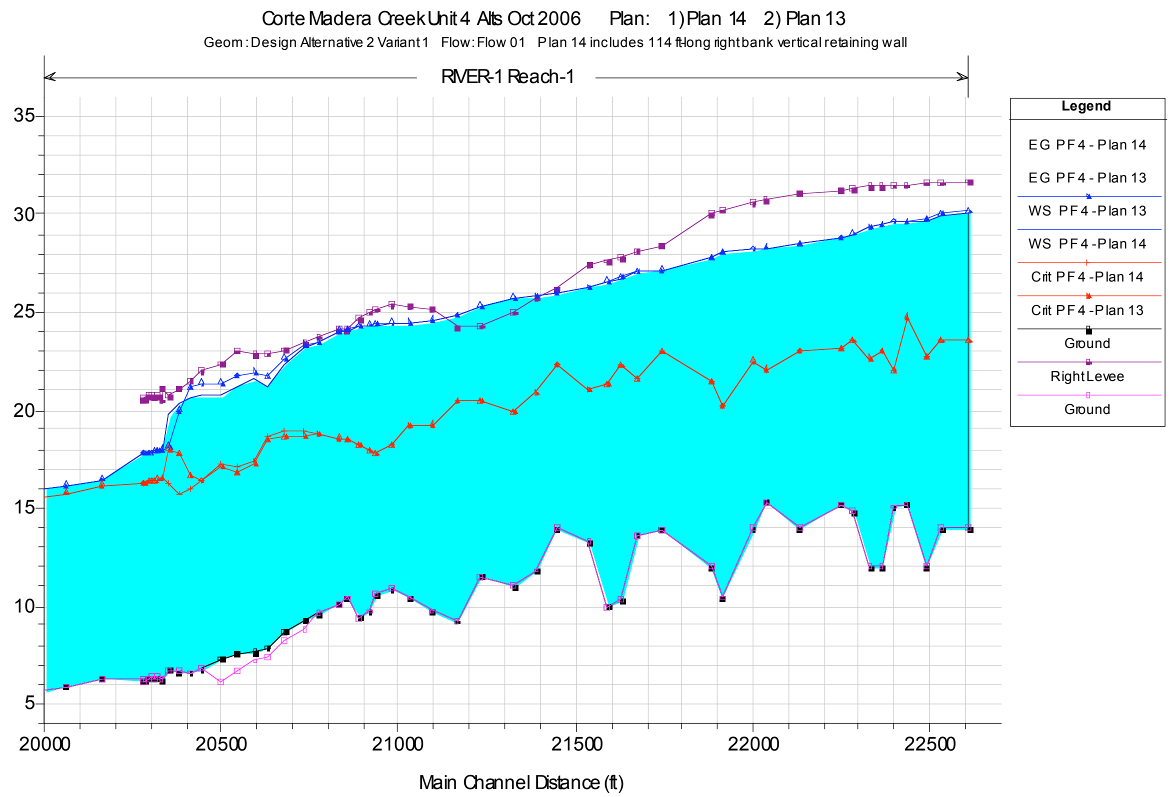

Figure 15. *Plan 13/14.* Comparison of calculated water surface profiles for PF4 (4,700 cfs) in the Unit 4 reach for the initial Design Alternative II test (g12, Plan 13) and the Design Alternative II Variant 1 (g13, Plan 14). Design Alternative II Variant I tests the isolated effect of replacing the existing grade control fish ladder structure with a 400-ft long 0.7 percent sloped approx. natural grade rock-lined channel for fish passage improvement, sewer line protection, and grade control replacement, *and* installing a 114-ft long mid-bank retaining wall immediately upstream from the Unit 3 channel.

- *The Design Alternative II Variant I plan would further reduce water surface elevations up to about 0.6 vertical ft in the lower 400-ft length of Unit 4.*

- *The Design Alternative II Variant I plan does not reduce water surface elevations upstream from the next upstream flow constraint where the downstream end of the encroached existing left (west) bank private residential vertical concrete retaining wall at 27 Sir Francis Drake Blvd narrowed the channel (at about 20,640 ft main channel distance). Therefore, the Design Alternative II Variant 1 plan does not increase the Unit 4 capacity compared to the initial Design Alternative II test plan (approx. 5,000 cfs).*

Design Alternative II Variant I is the first viable *natural grade* type alternative design developed by this incremental model analysis, because it substantially increases Unit 4 flood capacity, and at the same time provides for fish passage improvement, grade control, and sewer line protection without a permanent replacement fish ladder structure. Moreover, the required channel bed and bank modifications would have minimal, if any, detectable impacts on the existing native riparian canopy-forming trees or the natural aesthetics of the existing stream.

Inspection of critical water surface elevation profiles shown in Figure 15 shows that extending the right (west) bank retaining wall upstream to widen the channel in the vicinity of the encroached existing left (west) bank private residential vertical concrete retaining wall at 27 Sir Francis Drake Blvd would extend flood management improvements upstream and potentially substantially increase the Unit 4 design channel capacity. Additional Design Alternative II variants are developed below to maximize the Unit 4 design capacity while still preserving the majority of the existing native riparian trees.

First, because Design Alternative II Variant 1 is a viable plan that also demonstrably preserves the existing environmental and aesthetic values of the riparian corridor, it is selected as a recommended Design Alternative – Design Alternative IIA, as documented more completely in Part 5.

<u>Design Alternative II Variant 2</u>

As shown above, increasing the Unit 4 design capacity to more than 5,000 cfs would require widening the channel at all of the narrow channel cross-sections from Unit 3 concrete channel upstream to the vicinity of the Ross gage. Design Alternative II Variant 2 is intended to achieve this by reconfiguring and extending upstream the 114-ft long retaining wall simulated in Variant 1 to provide for a 40-ft minimum bankfull channel width from the inlet of the Unit 3 channel upstream to the existing encroached right bank private residential vertical steel sheetpile bank stabilization structure at 1 Sylvan Lane.

Design Alternative II Variant 2 includes an approx. 550-ft long mid-bank vertical concrete or steel sheetpile retaining wall extending from the Unit 3 channel upstream to near the existing Lagunitas Road Bridge right bank downstream concrete wingwall (Plates 1 and 2). By careful design of the retaining wall alignment, Variant 2 preserves the majority of existing native riparian canopy forming trees in the Unit 4 reach, and the overall character of the corridor and the aesthetic backdrop it creates for public space on the Ross Town commons. Figure 34 (see Appendix B) shows the configuration of the proposed retaining wall, and the trees to be removed under Variant 2, and the resulting reduction in the canopy cover.

First, widening the channel and installing the vertical wall some distance away from the existing toe of the channel bank allows the existing native alders to be saved and not removed. It is important to note that the existing alders are rooted at or within 1-2 vertical feet from the low-flow water surface, at the same elevation as natural free-forming gravel bars. *Therefore, removing the alder and grading the channel bed down would produce a negligible additional flood management benefit – the gravel bars will simply reform at their former, pre-excavated, elevation during the first water year.* It follows that preserving existing alders does not constrain maximum achievable design channel capacity. Furthermore, the existing alders have few horizontal branches and are rooted in lines parallel to the direction of flood flow, minimizing their hydraulic effect.

Second, the 550-ft long vertical retaining wall alignment was designed to substantially achieve the flood management objective while avoiding removal of the largest native riparian canopy forming trees near the top of bank (primarily maple and ash). By saving the native alders along the toe of the bank and the maple and ash near the top of bank, the majority of the canopy is preserved and the overall aesthetic character of the corridor would be substantially the same. Figure 39 (see Appendix B) shows, in concept, how Design Alternative II variants would save the toe of bank and top of bank trees, but necessarily remove the mid-bank trees. This way, the majority of the canopy cover is retained, as shown in Figure 34. Figure 16 shows an example cross-section profile at River Station 373+74 ft where an existing alder is preserved at the toe of the bank, and two existing ash are preserved near the top of bank. Figure 40 (see Appendix B) shows, in concept, the grading effects at other cross sections in the study reach.

Design Alternative II Variant II also includes two left (east) bank biotechnical bank stabilization projects made necessary at two locations where existing right (west) bank structures and utilities prevent achieving the 40-ft minimum channel width with additional right (west) bank grading. The first site is on private residential property at 23 Sir Francis Drake Blvd across from the existing vertical steel sheetpile wall at the back of the Ross Post Office (Figure 34). The site is situated between an existing rock-filled gabion mattress-wall structure at 25 Sir Francis Drake Blvd, and an existing approx. 1(H):1(V) sloped rip-rap covered bank at 21 Sir Francis Drake Blvd. The upstream approx. 68-ft long portion of the site would be graded back to maximum 1.25(H):1(V) slope to meet the minimum width requirement and not necessarily destabilize the existing garage foundation at the proposed design top of bank (Plates 1 and 2). The finished slope would be hydroseeded, covered with biodegradable geofabric, and vegetated with native riparian canopy forming trees (e.g., native CA willow and alder in the lower horizon and native CA ash, maple, and oak in the upper horizon). The bank section immediately below the existing garage foundation may also require lining with vegetated rip-rap. The entire graded area would be protected with 18-24-inch minimum diameter rip-rap placed along the toe of the existing bank to extend the hydraulic profile of the existing gabion basket mattress at 25 Sir Francis Drake Blvd (Plates 1 and 2). The approx. 68-ft long toe protection would be constructed to form a bench at the same elevation as the existing alders are rooted across the channel from the site, with sufficient void spaces on the top of bench and near the toe of the fabric and rock covered upper bank to allow backfill with sand-and-gravel and planting 1-gallon native alder container plants. Three existing native trees on the mid-bank would be removed.

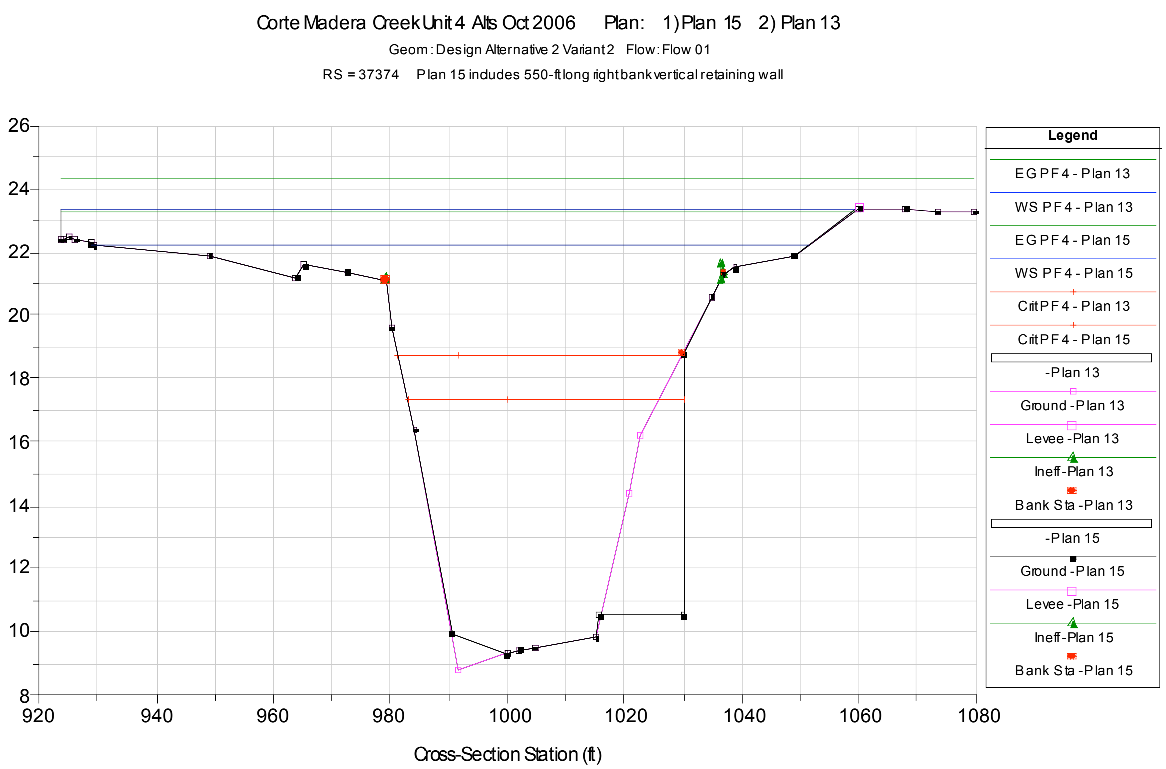

Figure 16. *Plan 13/15.* Comparison cross-section profiles for existing conditions (Design Alternative II test) (g12, Plan 13) (magenta) and for Design Alternative II Variant 2 (g14, Plan 15) (black) at River Station 373+74 ft, 404 ft upstream from Unit 3. The proposed 550-ft long mid-bank vertical retaining wall is about 9-ft high and increases the channel width from about 26 ft to the 40-ft minimum design width. There is an existing 10-inch diameter alder is rooted at elevation 11 ft, cross-section station 1016 ft, and an existing 35-inch diameter ash tree is rooted at elevation 20.6 at cross-section station 1036 ft. These trees would not be removed to accommodate the retaining wall as designed. All mid-bank vegetation would be removed and replaced with the 14-ft wide flat gravel bar feature at the foot of the wall, where it is expected that native alders would establish by natural processes. The gravel bar feature could be planted immediately after construction to provide a partial visual screen for the steel, concrete,

or decorative rock-covered concrete finished wall. The 400-ft long rip-rap channel bed lining feature terminates near this cross-section. A minor amount of vegetated rip-rap fill is shown along the left (east) bank toe at the upstream terminus of the structure. Also note the intentional v-shaped channel bed profile within the low-flow channel area, consistent with typical design criteria for roughened rock channel type fish passage structures.

According to this proposed design for the 23 Sir Francis Drake Blvd site, the new top of bank would be set back 10 horizontal feet from the existing top of bank (fence) at the upstream property boundary, and about 3.5 horizontal feet at the downstream corner of the existing garage (i.e., tight to the garage corner). (Laying back the channel bank tight to the corner of the existing garage may destabilize the structure without foundation improvements. Accordingly, the existing garage at 23 Sir Francis Drake Blvd may be a mild constraint for increasing the flood conveyance capacity of Unit 4.) Design Alternative II Variant 2 incorporates this proposed biotechnical stabilization project on private property subject to permission and cooperation of the landowner.

The second left (east) bank site is on Town of Ross property across the creek from the existing encroached private residential vertical steel sheetpile wall at 1 Sylvan Lane. The proposed left bank regrading work would be centered on the narrowest cross-section at about River Station 378+04 ft, directly across the creek from the sheetpile wall, and would extend upstream about 130 ft to approx. River Station 379+35 ft, and extend downstream about 218 ft to the existing upstream face of the Lagunitas Road Bridge location (Plates 1 and 2). The toe of the bank would be graded back to achieve the 40-ft minimum channel width along the entire 348-ft long site. The bank would be graded back to maximum 1.5(H):1(V) slope extending up to the preserve the existing edge of pavement on the Town property. The finished slope would then be hydroseeded, planted with native trees and shrubs, and drip-irrigated for 2-3 years until trees are established. Existing alders near the toe of the existing bank would be preserved, but at least 5 native riparian trees would be removed from the mid-bank area to accommodate necessary channel capacity.

The Design Alternative II Variant 2 plan (g14, Plan 15) was developed by incorporating the above-described approx. 550-ft long right (west) bank vertical retaining wall and two left (east) bank biotechnical bank stabilization projects into the geometry file for Design Alternative II Variant 1 (g13, Plan 14). Design Alternative II Variant 2 therefore also includes the 400-ft long 0.7 percent sloped rip-rap channel bed lining designed for grade control preservation, sewer line protection, and fish passage improvement. Model runs were made to test the hydraulic effect of the Design Alternative II Variant 2 plan compared to the initial Design Alternative II test run (Figure 17).

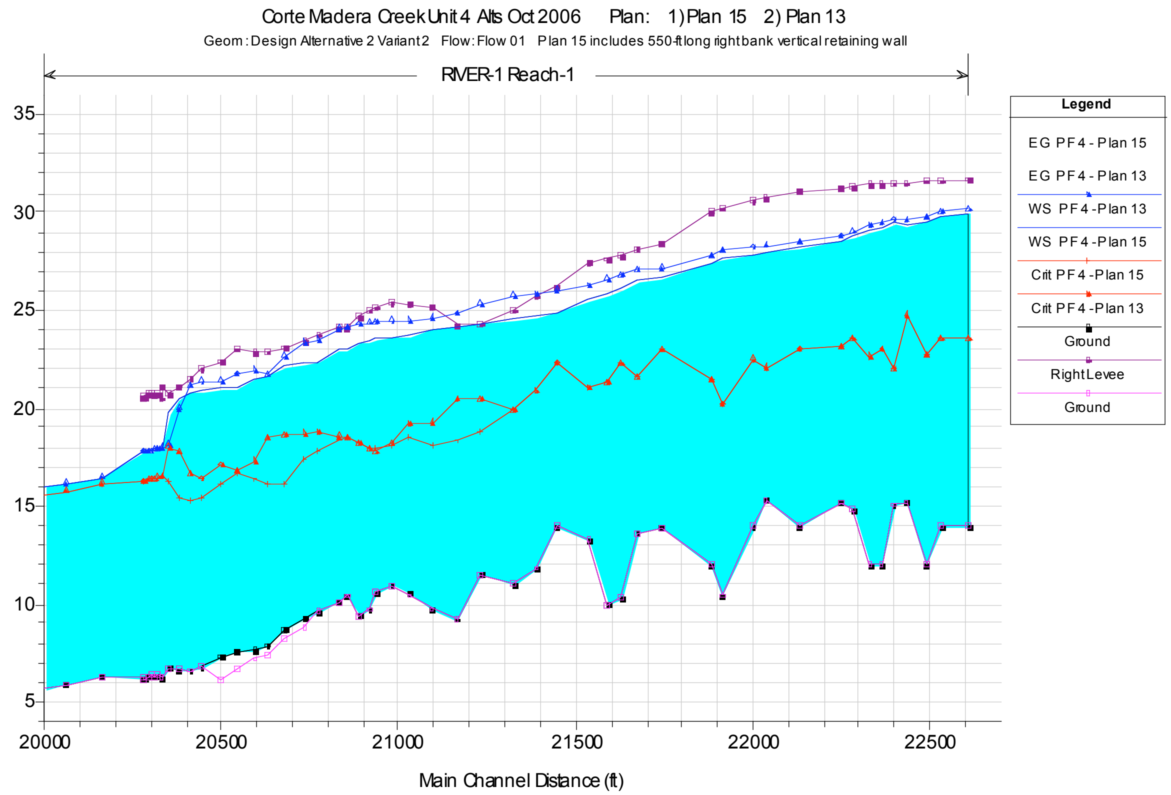

Figure 17. *Plan 13/15.* Comparison of calculated water surface profiles for PF4 (4,700 cfs) in the Unit 4 reach for the initial Design Alternative II test (g12, Plan 13) and the Design Alternative II Variant 2 plan (g14, Plan 15). Design Alternative II Variant II tests the isolated effect of replacing the existing grade control fish ladder structure with a 400-ft long 0.7 percent sloped approx. natural grade rock-lined channel for fish passage improvement, sewer line protection, and grade control replacement, *and* installing a 550-ft long mid-bank vertical retaining wall extending along the right (west) bank from the Unit 3 channel upstream to the Lagunitas Road Bridge location, as well as two biotechnical bank stabilization projects at left (east) bank properties.

- *The Design Alternative II Variant 2 plan would further reduce water surface elevations up to about 0.6-1.3 vertical ft in the lower 1,000-ft length of Unit 4, and substantially increase Unit 4 design capacity compared to Design Alternative II test plan and Alt II Variant 1.*

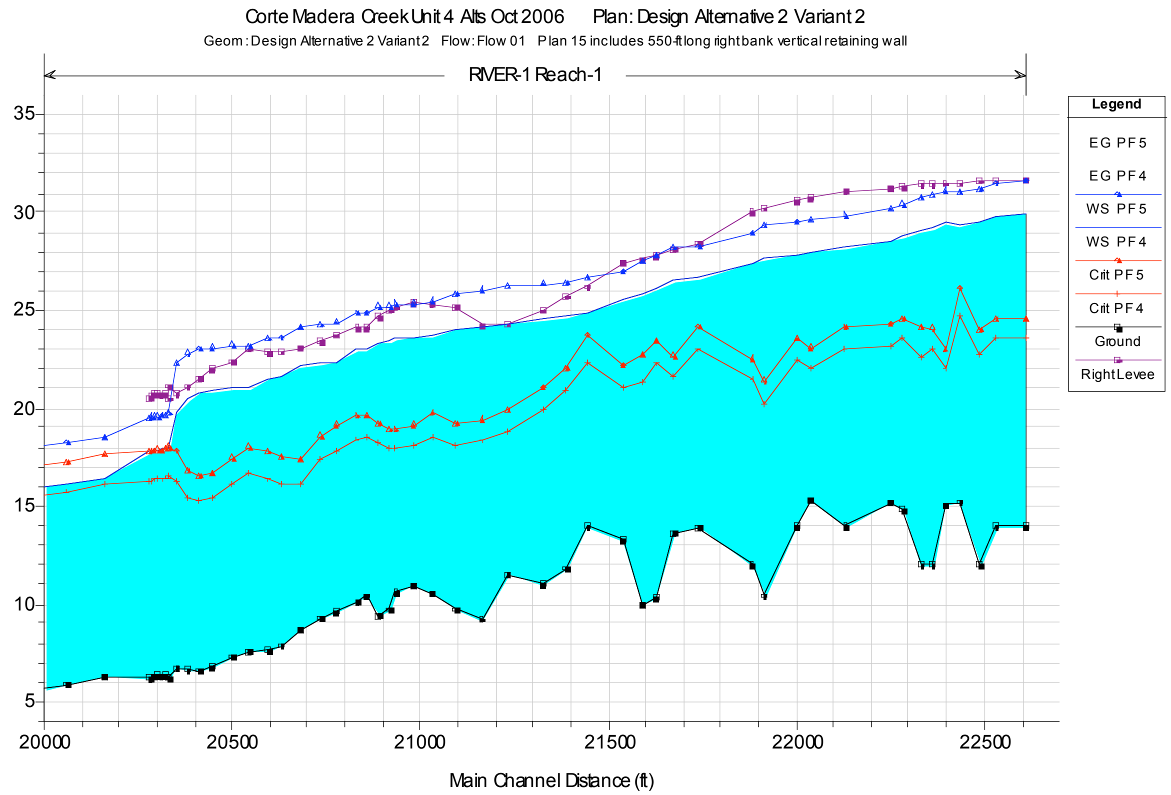

Figure 18. *Plan 15.* Comparison of water surface elevations under Design Alternative II Variant 2 conditions for FP4 (4,700 cfs) and PF5 (5,950 cfs). See Figure 17 for further description of Design Alternative II Variant 2.

- *Unit 4 design capacity under the Design Alternative II Variant 2 is more than 4,700 cfs and less than 5,950 cfs (approximately 5,400 cfs).*

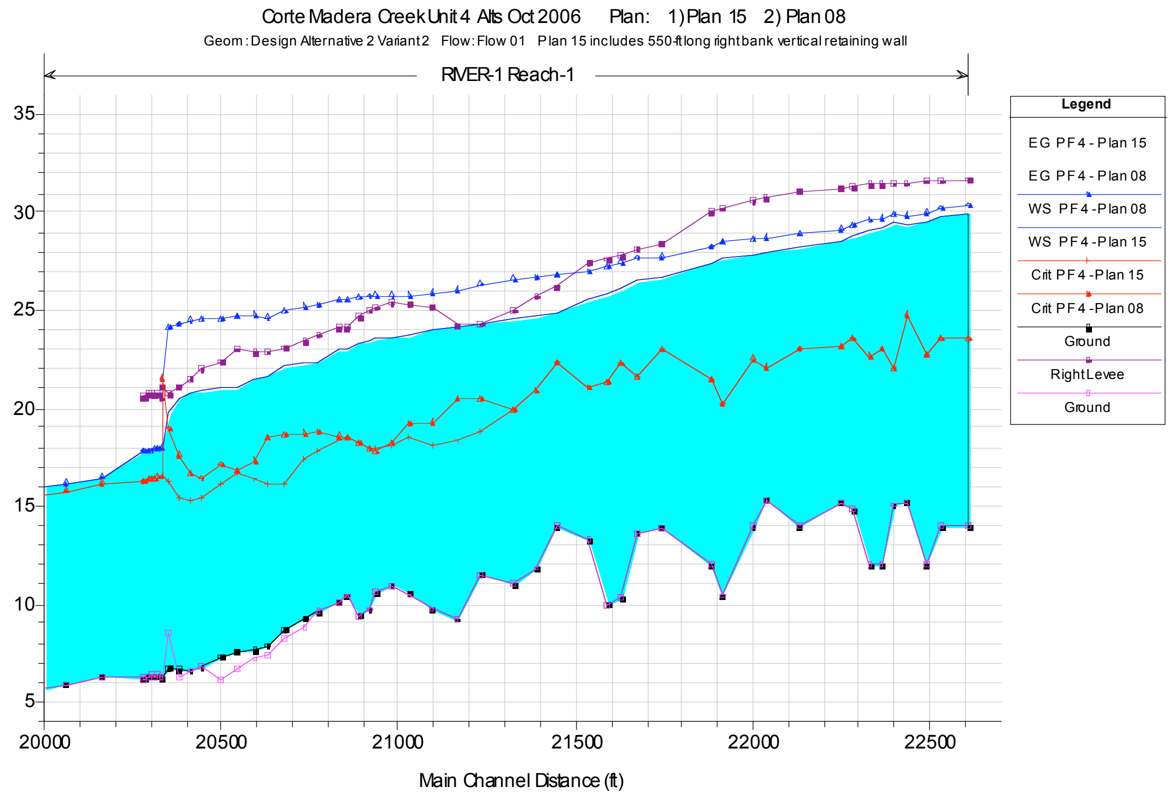

Figure 19. *Plan 08/15.* Comparison of calculated water surface profiles for PF4 (4,700 cfs) in the Unit 4 reach for the Baseline (Existing Conditions) Alternative (g07, Plan 08) and the Design Alternative II Variant 2 plan (g14, Plan 15). See Figure 17 for further description of Design Alternative II Variant 2.

- *Compared to the existing condition, the Design Alternative II Variant 2 plan would reduce water surface elevations about 4 vertical ft near the entrance to the Unit 3 concrete channel to as little as 0.3 ft near the Ross Creek tributary confluence.*

- *The Design Alternative II Variant 2 plan would increase unit 4 design capacity from approx. 3,200 cfs (existing conditions) to about 5,400 cfs. (Note: the capacity of the Design Alternative II Variant 2 plan is determined more precisely in Part 5).*

Inspection of the critical water surface elevations in Figure 19 shows that a slight hydraulic constraint remains in the vicinity of River Station 374+71 (20,830 ft main channel distance) and River Station 374+95 ft (20,854 ft main channel distance) where the Design Alternative II Variant 2 retaining wall alignment provides the target design 40-ft minimum channel width, but the resulting 40-ft section is immediately downstream from the wider section in the vicinity of the Lagunitas Road Bridge. Additional Design Alternative II Variants are developed below to test the effect of widening the channel to create a smoother hydraulic transition between River Station 375+59 ft and River

Station 373+74 ft. The additional variants will require removal of existing native trees along the top of bank immediately downstream from the existing right bank Lagunitas Road Bridge wing wall (Tree No. 73: 24-inch diameter bay tree clump, and Tree No. 76: 26-inch diameter oak tree). Design Alternative II Variant 3 would remove the bay tree clump and preserves the oak tree (Figure 20).

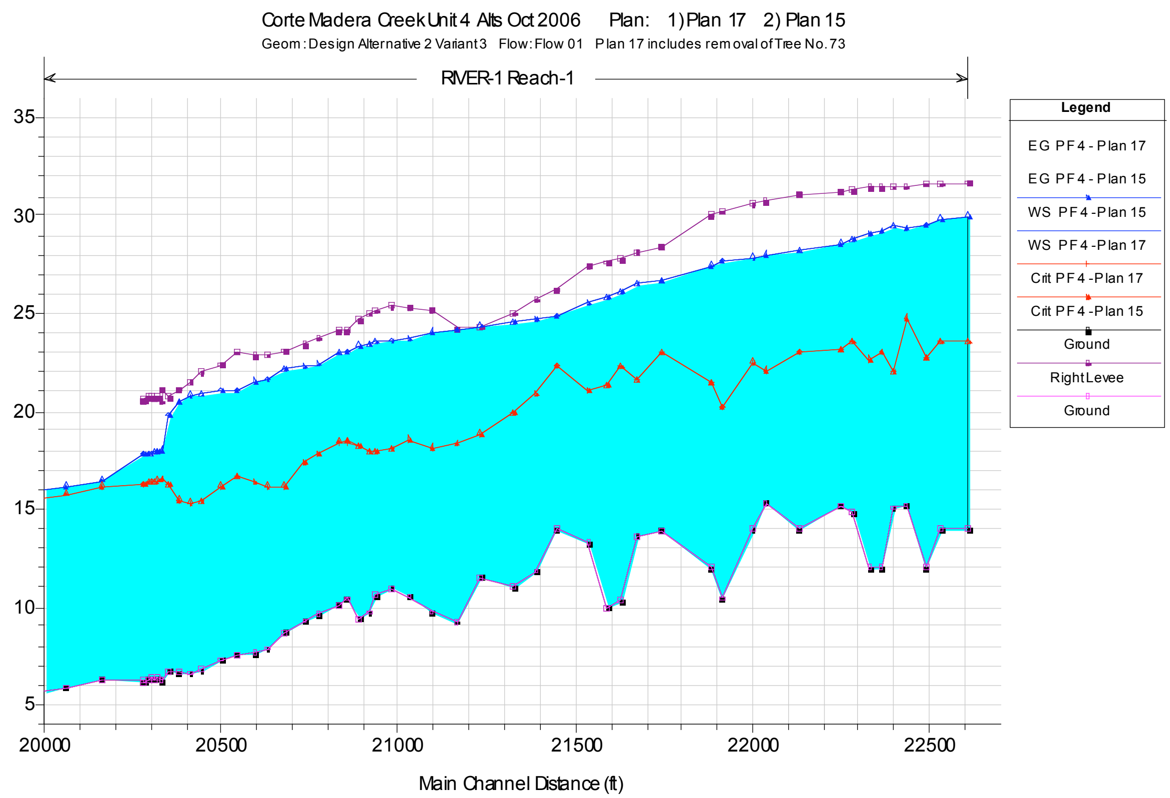

Figure 20. *Plan 17/15*. Comparison of calculated water surface profiles for PF4 (4,700 cfs) in the Unit 4 reach for the Design Alternative II Variant 2 plan (g14, Plan 15) and the Design Alternative II Variant 3 plan (g16, Plan 17).

- *The Design Alternative II Variant 3 plan has virtually the same hydraulic performance as the Design Alternative II Variant 2 plan, so there is no flood benefit to removing the existing bay tree clump (Tree No. 73).*

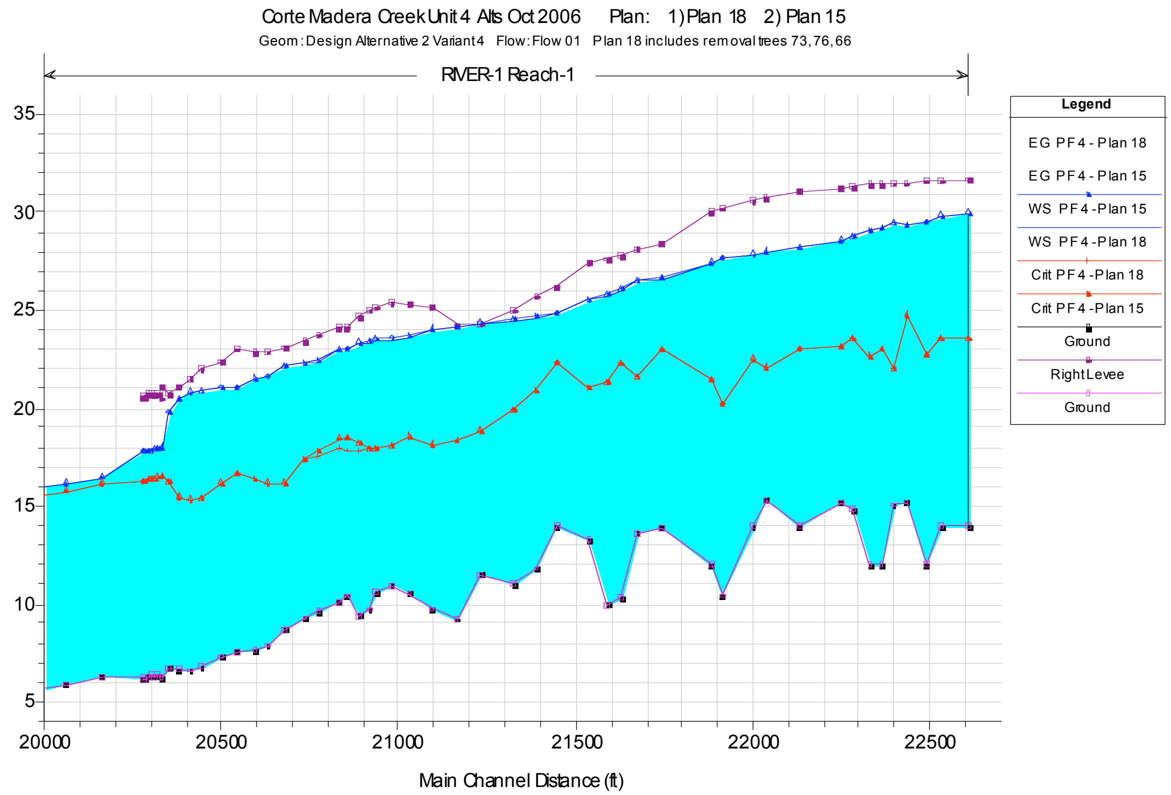

Figure 21. *Plan 18/15*. Comparison of calculated water surface profiles for PF4 (4,700 cfs) in the Unit 4 reach for the Design Alternative II Variant 2 plan (g14, Plan 15) and the Design Alternative II Variant 4 plan (g17, Plan 18).

- *The Design Alternative II Variant 4 plan has virtually the same hydraulic performance as the Design Alternative II Variant 2 plan, so there is no flood benefit to removing the existing top of bank oak tree downstream from Lagunitas Road Bridge (Tree No. 76).*

Figures 20 and 21 show that Variants 3 and 4 provide no flood benefits compared to Variant 2. To increase Unit 4 design capacity substantially more than the preliminary estimated 5,400 cfs capacity under Design Alternative II Variant II, it will be necessary to grade the banks farther back to provide for more than 40-ft wide channel through the entire reach, requiring a longer, higher retaining wall and removal of virtually all of the top of bank riparian trees along the pedestrian right of way. Additional design Alternatives are developed below to simulate more severe right bank grading to determine the maximum feasible design capacity for Unit 4.

First, because Design Alternative II Variant 2 is a viable plan that provides the same fish passage improvement but substantially more flood capacity than Design Alternative II Variant 1 (Recommended Alternative IIA), and at the same time demonstrably preserves the existing environmental and aesthetic values of the riparian corridor, it is selected as a recommended Design Alternative – Recommended Alternative IIB, as documented more completely in Part 5.

Design Alternative III – Natural Grade Alternative with Completely Regraded Right Bank

The Design Alternative III test plan and variants are intended to further reduce flow constraints in the lower Unit 4 reach by substantially widening the channel compared to the Alternative IIB plan, requiring a 785-ft long vertical concrete or steel sheetpile wall along the top of the right (west) bank, extending from the Unit 3 channel upstream to the downstream end of the existing vertical sheetpile retaining wall at 1 Sylvan Lane (River Station 377+55 ft). The Alt III test plan wall would follow the same alignment as the existing chain link fence along the pedestrian right of way, and incorporate the existing vertical sheetpile retaining wall behind the Post Office (Plates 1 and 2). The Alternative III wall would thereby preserve the existing pedestrian right of way (sidewalk) in its existing condition, but it would also require removing all the existing native riparian canopy forming trees along the top of the right bank (Figure 35, in Appendix B). The existing alders along the toe of the bank would not be removed. The new vertical wall would extend upstream to the existing steel sheetpile wall at 1 Sylvan Lane and require removal of the existing Lagunitas Road Bridge abutment, making the replacement bridge span about 6-7 ft wider. The Design Alternative III test plan and variants include the same biotechnical bank stabilization treatments at two left (east) bank sites, and the same 400-ft long 0.7-percent slope rock rip-rap channel bed lining to provide approx. natural grade roughened rock channel type fish passage improvement as Alternative IIB.

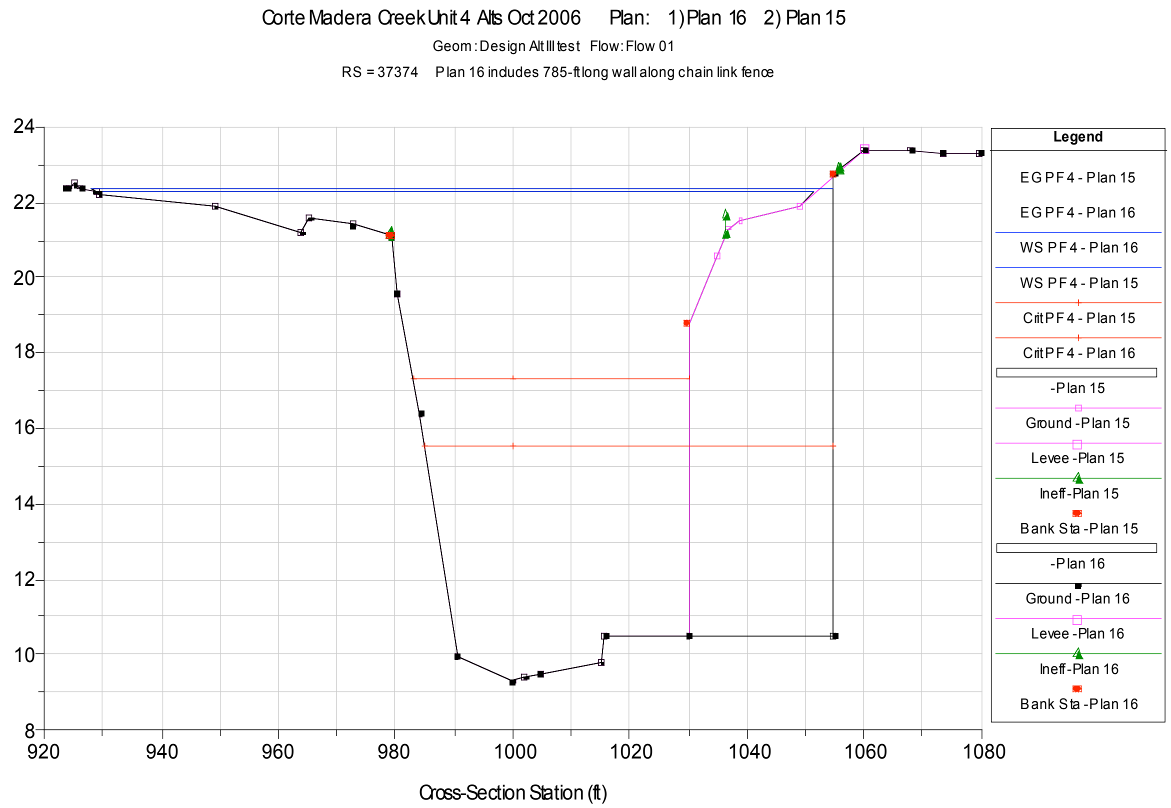

Figure 22. *Plan 16/15.* Comparison cross-section profiles for Design Alternative III test (g15, Plan 16) (black) and for Design Alternative II Variant 2 (g14, Plan 15) (magenta) at River Station 373+74 ft, 404 ft upstream from Unit 3. The Alt III test 785-ft long vertical top of bank retaining wall is about 13-ft high and increases the channel width from about 40 ft to about 65 ft. Figure 16 shows the same cross-section under its 26-ft wide existing condition and explains how the Design Alternative II Variant 2 retaining wall would save the existing 10-inch diameter alder is rooted at elevation 11 ft, cross-section station 1016 ft, and the existing 35-inch diameter ash tree is rooted at elevation 20.6 at cross-section station 1036 ft. The Design Alternative III test plan would save the alders at the toe of the bank but not the top of bank ash tree. The Design Alternative III test wall is configured on the same line as the existing chain-link fence bordering the pedestrian right of way downstream from Lagunitas Road Bridge. Note that although the channel is significantly widened at this cross-section under the Design Alternative III test plan, the critical water surface elevation reduces about 1.0 ft but the model calculated 4,700 cfs water surface elevation on reduces slightly.

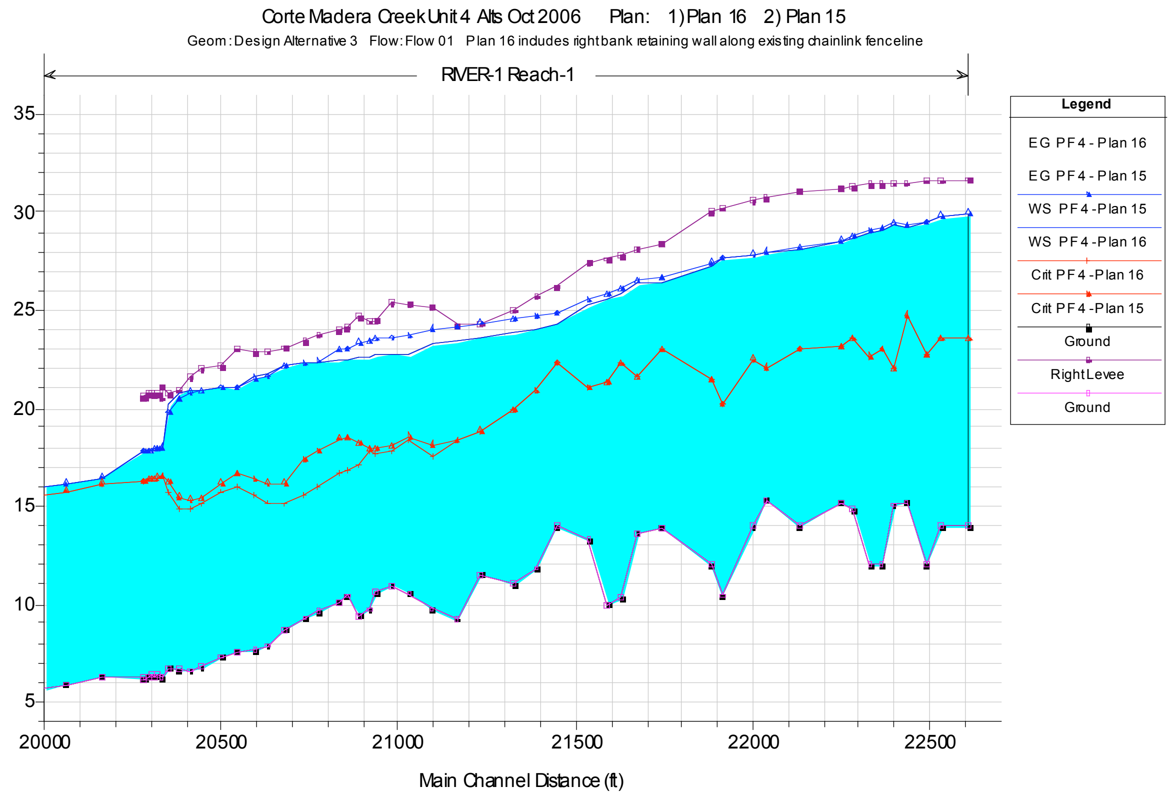

Figure 23. *Plan 16/15.* Comparison of calculated water surface profiles for PF4 (4,700 cfs) in the Unit 4 reach for the Design Alternative III test plan (g15, Plan 16).and the Design Alternative II Variant 2 plan (g14, Plan 15). This comparison tests the hydraulic effect of moving the right bank vertical retaining wall proposed under Design Alternative II Variant 2 back to the existing chain link fence so as to maximize flood flow capacity without modifying the existing pedestrian right of way and water supply pipe line and other utilities located beneath the sidewalk and along the existing top of bank.

- *The Design Alternative III test plan reduces the critical water surface elevation profile along the entire 785-ft length of the wall, but only reduces flood water surface elevations upstream from approx. River Station 374+15 ft (20,775 ft main channel distance).*

- *Because Design Alternative III test plan does not reduce flood water surface elevation in the downstream100-200-ft length of Unit 4, it does not increase the design flood flow capacity compared to the Design Alternative II Variant 2 plan (approx. 5,400 cfs).*

Although the Design Alternative III test plan would widen the channel more than the Design Alternative II Variant 2 wall, it would not increase the Unit 4 design flood capacity because it would not reduce the flood water surface elevation in the downstream 100-200-ft length of Unit 4. It appears from the Design Alternative III test run results that the transition from the Unit 4 natural channel to the 33-ft wide Unit 3 concrete channel may be a limiting hydraulic constraint. Note too, however, that the channel width obtained by placing the Design Alternative III test wall along the existing chain link fence line is not everywhere significantly greater than the 40-ft minimum design width achieved by the mid-bank Design Alternative II Variant 2 retaining wall (Plates 1 and 2). For example, under the Design Alternative III test plan, the channel is only 42-ft wide at River Station 371+40 ft (20,500 ft main channel distance). The width is constrained here by the existing left (east) bank private residential gabion basket retaining wall at 25 Sir Francis Drake Blvd an the existing vertical steel sheetpile retaining wall forming the foundation of the chain link fence behind the Post Office (Figure 35). Additional Design Alternative III variants are developed below that would widen the channel to a 60-ft wide minimum at River Station 371+40 ft, requiring a vertical retaining wall aligned tight to the back wall of the Post Office building and associated impacts to the existing parking area. However, the first variant will test the isolated hydraulic effect of removing the existing alders along the toe of the bank in Unit 4.

Design Alternative III Variant 1

The Design Alternative III Variant 1 geometry file (g18) was prepared by modifying the Design Alternative III test geometry (g15) to simulate removal of the existing toe of bank alders and the gravel bar benches they are rooted on, in two reaches: (1) between River Station 370+80 ft and River Station 372+15 ft, and (2) between River Station 373+74 ft and River Station 374+74 ft. The trees and the canopy to be removed specifically under Design Alternative III Variant 1 are shown in Figure 35. The canopy to be removed is delimited by a solid red line.

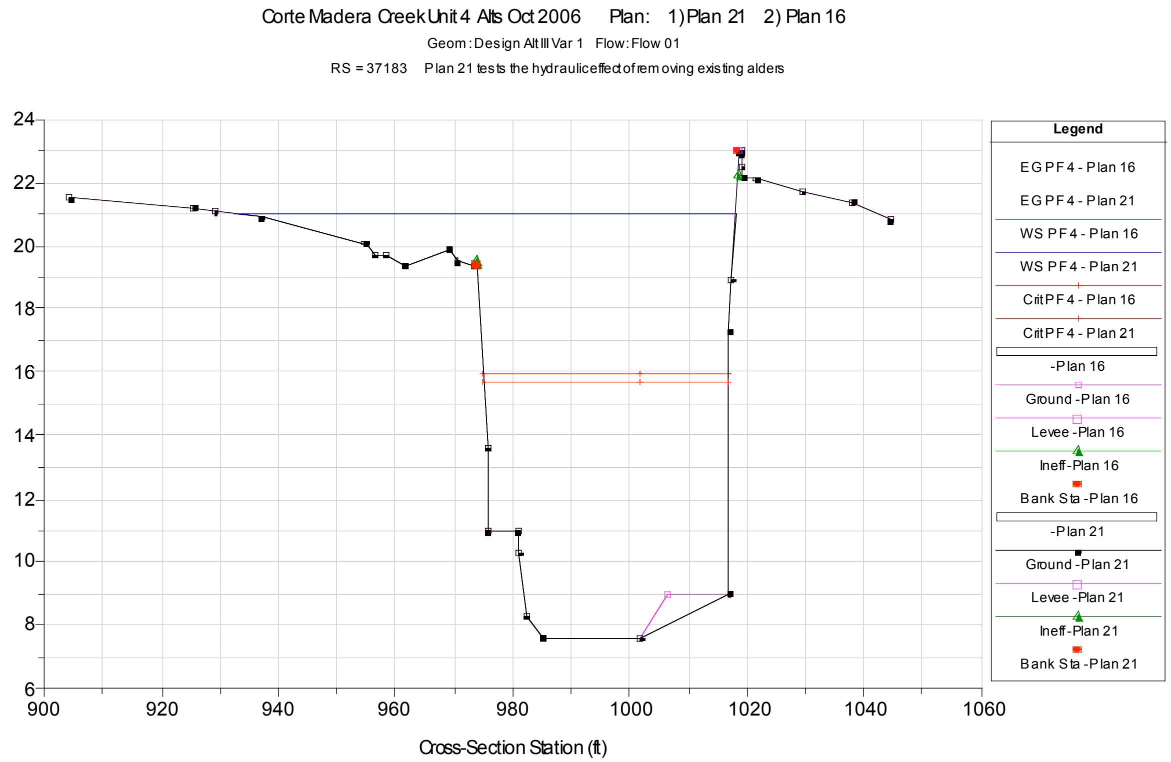

Figure 24. *Plan 16/21.* Comparison of cross-section profiles for the Design Alternative III Variant 1 (g18, Plan 21).and the Design Alternative III test plan (g15, Plan 16) at River Station 371+83 ft. The existing alders and the associated gravel bar bench preserved under the Design Alternative II plans and the Design Alternative III plan (magenta) is removed in Design Alternative III Variant 1 (black). At this cross-section, the left bank is an existing private residential gabion basket mattress and vertical retaining wall structure at 25 Sir Francis Drake Blvd. The right bank is the existing vertical steel sheetpile retaining wall forming the foundation of the chain-link fence behind the Post Office.

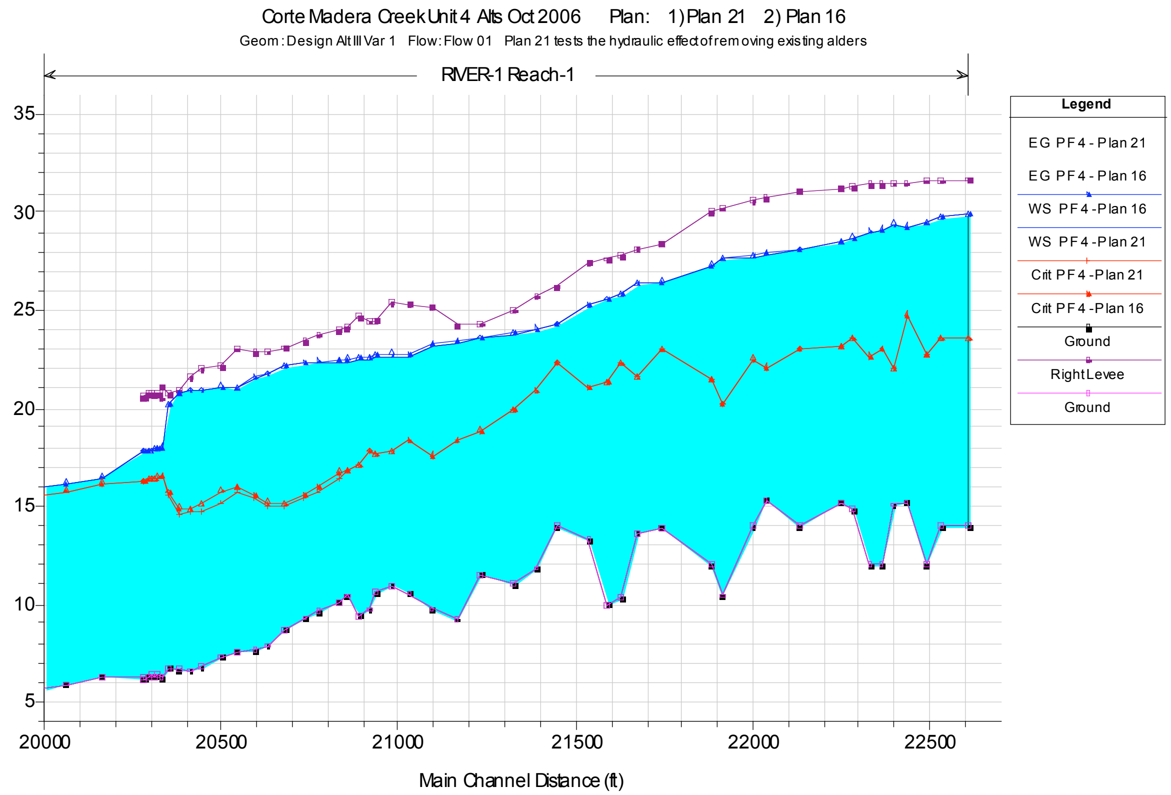

Figure 25. *Plan 16/21.* Comparison of calculated water surface profiles for PF4 (4,700 cfs) in the Unit 4 reach for the Design Alternative III Variant 1 (g18, Plan 21).and the Design Alternative III test plan (g15, Plan 16). This comparison tests the isolated hydraulic effect of removing the 27 existing alders along the toe of the bank along a total of 240 lineal ft of Unit 4, and grading down the associated gravel bar bench to the base of the alders.

- *Removing the existing alders rooted along the toe of the bank in the Unit 4 reach and grading down the associated gravel bar benches would slightly reduce critical water surface elevations in the affected portions of Unit 4, but not reduce flood water surface elevations or increase Unit 4 design capacity.*

Design Alternative III Variant 2

Alternative III Variant 2 is intended to determine if the Unit 4 design capacity can be increased compared to the Design Alternative II Variant 2 and Design Alternative III test plans. The Alternative III Variant 2 plan would require a 790-ft long vertical concrete or steel sheetpile retaining wall joining with the Unit 3 channel and the existing private residential steel sheetpile wall at 1 Sylvan Lane (Plates 1 and 2). The Variant 2 wall provides a 60-ft minimum design channel width at the Post Office, where the retaining wall would run along the back wall of the Post Office building receiving area. Accordingly, the Variant 2 wall would eliminate the existing pedestrian right of way and eliminate the vehicle parking area along the right of way. Figure 41 (see Appendix B) shows, in concept at selected cross sections, how the test plan would preserve the pedestrian right of way and parking area, and Variant 2 would require its removal. The plan would also require reconfiguration of the existing water line beneath the sidewalk, and electricity and telephone lines with poles located near the existing top of bank. The plan would preserve and protect the existing sanitary sewer siphon pipes, including the standpipe inlet in the parking area west of the channel (Figure 35).

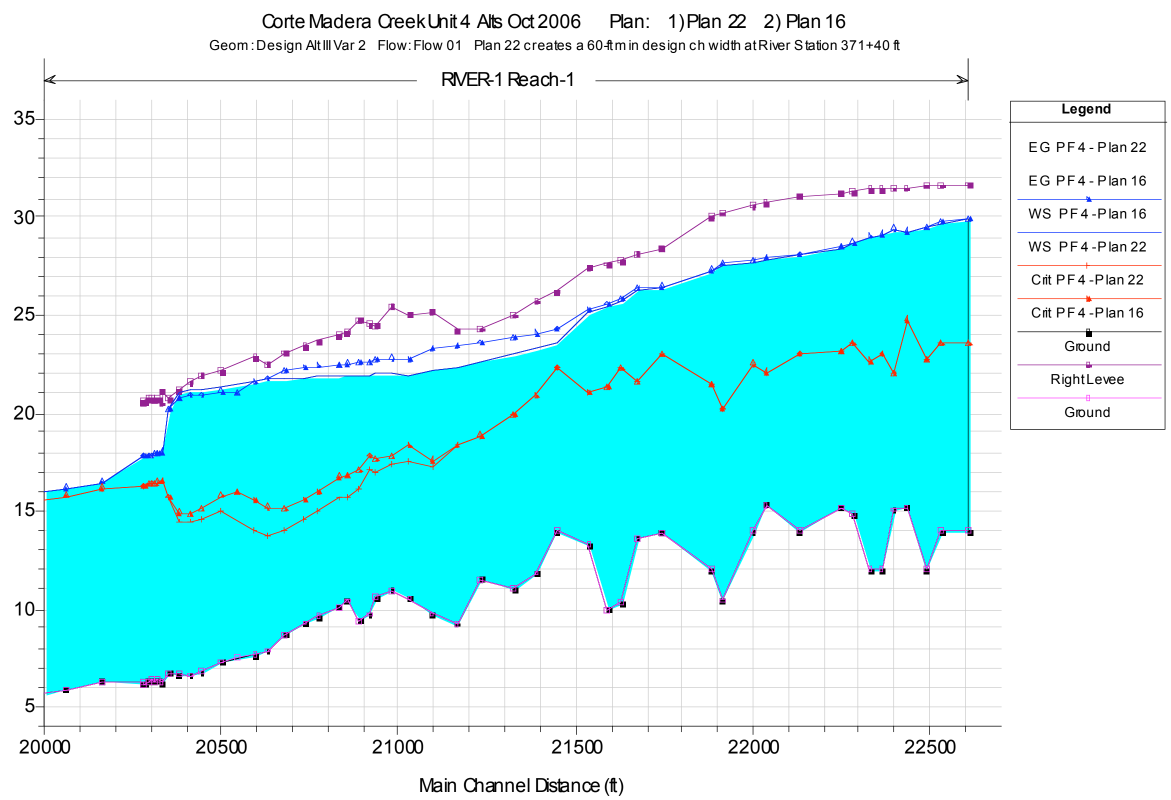

Figure 26. *Plan 16/22.* Comparison of model calculated water surface profiles for PF4 (4,700 cfs) in the Unit 4 reach for the Design Alternative III Variant 2 (g19, Plan 22).and the Design Alternative III test plan (g15, Plan 16). This comparison tests the effect of widening the channel to achieve a 60-ft minimum design channel width by moving the vertical top of bank retaining wall to the back wall of the Post Office building, consequently eliminating the throughway vehicle access, parking areas, and existing pedestrian right of way.

- *The Design Alternative III Variant 2 wall reduces the critical water surface elevation along the entire 790-ft long retaining wall, but slightly increases not decreases the flood water surface elevation in the downstream from River Station 372+35 ft (20,595 ft main channel distance).*

Although the channel is everywhere wider and the critical depth is everywhere lower under Design Alternative III Variant 2 plan than the Design Alternative III test condition, the flood water surface elevation *increases not decreases* in the lower 265-ft long section of Unit 4. Evidently, by widening the channel to the 60-ft wide minimum in this vicinity, the hydraulic transition to the 33-ft wide, low roughness Unit 3 concrete channel is made less efficient. Although the Design Alternative III Variant 2 plan reduces water surface elevations in part of Unit 4, it does not increase the design channel capacity compared to Alternative III test or Design Alternative II Variant 2 plan (Alternative IIB).

The above analyses demonstrate that design alternatives that include bank regrading more severe than Design Alternative II Variant 2 (Alternative IIB) do not increase the Unit 4 design capacity because the transition from the widened natural Unit 4 channel to the relatively narrow, low roughness Unit 3 concrete channel is an intractable hydraulic constraint.

Figure 26 shows that the Design Alternative III Variant 2 plan does produce a substantial flood benefit upstream from the Lagunitas Road Bridge location, but with relatively minor overall benefits, it does not appear to justify the required elimination of the existing parking areas.

Given the failure of Alternative III variants to produce a justifiable additional flood benefit, the Design Alternative III test plan is recommended as a final Design Alternative – Design Alternative III, and documented more completely in Part 5.

PART 5. RECOMMENDED OCTOBER 2006 DESIGN ALTERNATIVES

Part 5 Purpose

Part 5 is intended to document more precise channel capacity estimates for each of the five recommended final design alternatives. Part 5 also presents some general findings of the overall model analysis.

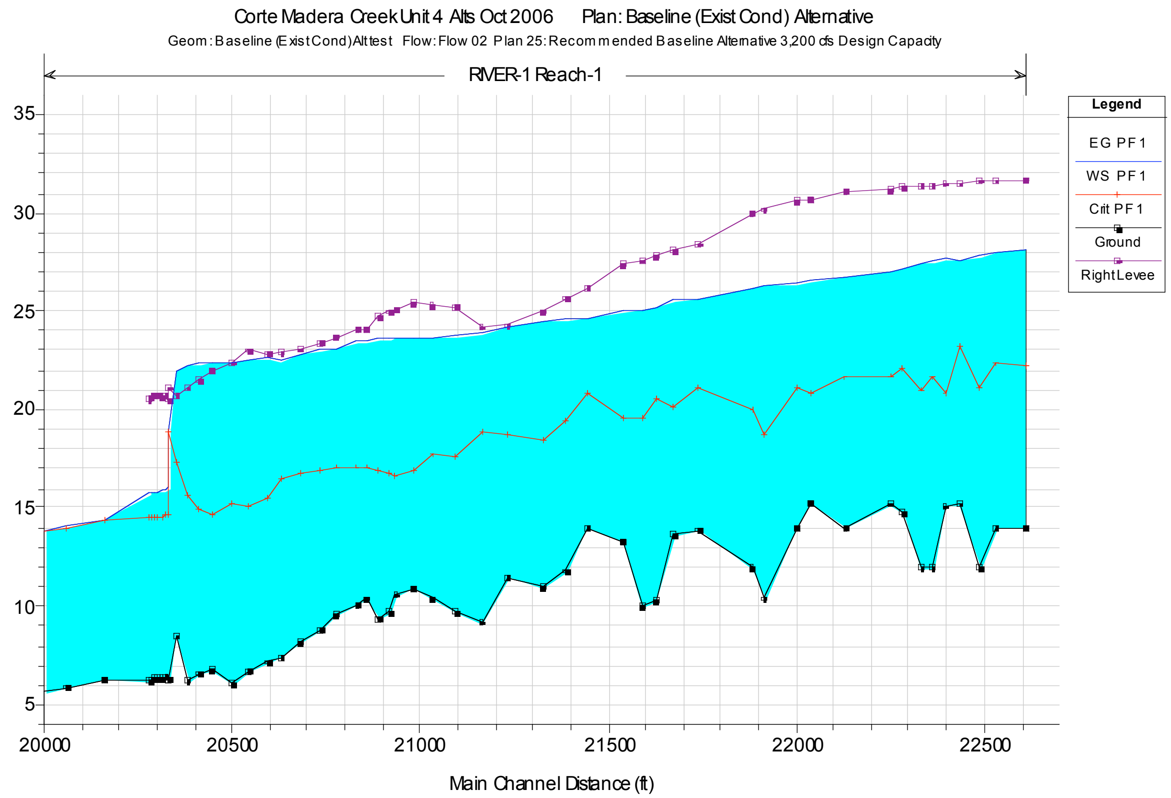

Figure 28. *Plan 25.* Flood water surface elevation profile for PF1 (3,200 cfs) under recommended Design (Existing Conditions) Alternative (g07, Plan 25).

- *A 3,200 cfs discharge produces maximum 1.1 ft overbank flow at the right (west) bank immediately upstream from the Unit 3 concrete channel entrance. It is assumed that this depth of overbank flow can be prevented with a temporary seasonal floodwall (e.g., sandbag wall placed along the existing chain-link fence).*

- *The estimated design capacity for the recommended Baseline (Existing Conditions) Alternative (g07; Plan 25) is 3,200 cfs.*

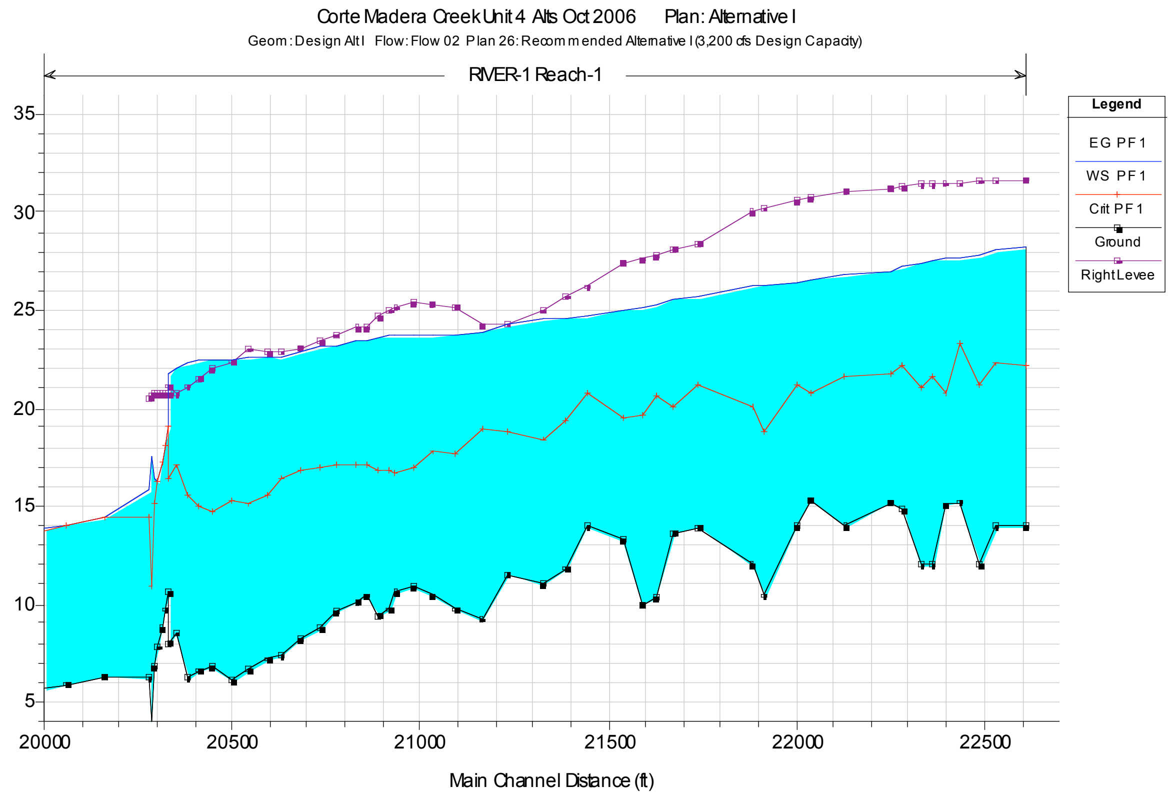

Figure 29. *Plan 26.* Flood water surface elevation profile for PF1 (3,200 cfs) under recommended Alternative I (g09, Plan 26).

- *A 3,200 cfs discharge produces maximum 1.1 ft overbank flow at the right (west) bank immediately upstream from the Unit 3 concrete channel entrance. It is assumed that this depth of overbank flow can be prevented with a temporary seasonal floodwall (e.g., sandbag wall placed along the existing chain-link fence).*

- *The estimated design capacity for the recommended Alternative I plan (g09; Plan 26) is 3,200 cfs.*

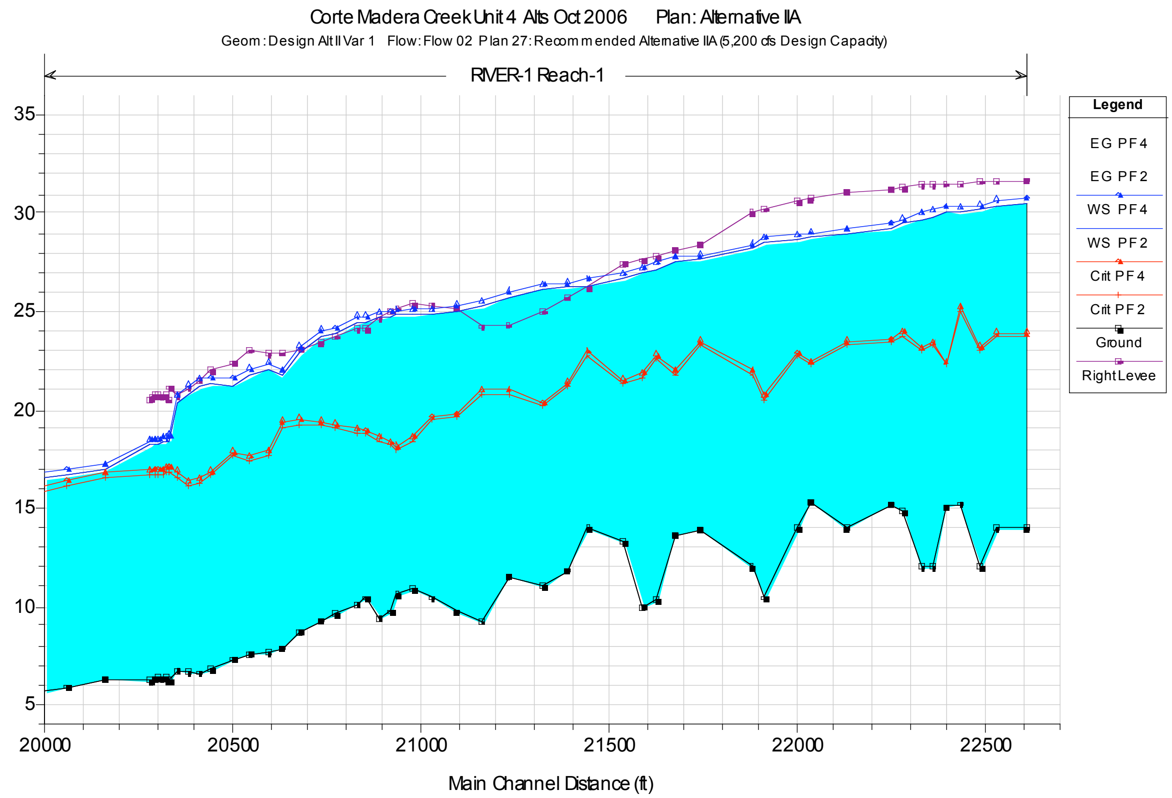

Figure 30. *Plan 27.* Comparison of flood water surface elevation profiles for PF2 (5,000 cfs) and PF4 (5,200 cfs) under recommended Alternative IIA (g13, Plan 27).

- *Under Alternative IIA, a 5,200 cfs discharge produces maximum 0.6 ft overbank flow just downstream from the Lagunitas Road Bridge location at River Station 374+95 ft (20,850 ft main channel distance). It is assumed that this depth of overbank flow can be prevented with a temporary seasonal floodwall (e.g., sandbag wall placed along the existing chain-link fence).*

- *A 5,200 cfs discharge also produces maximum 1.8 ft right overbank flow upstream from Lagunitas Road Bridge in the vicinity of 1 Sylvan Lane. It is assumed that this depth of overbank flow can be prevented with a temporary seasonal floodwall (e.g., sandbag wall placed along the existing chain-link fence). Flood depths in this vicinity could be reduced an additional approx.0.6 ft by including in the Alternative IIA plan the left (east) bank biotechnical bank stabilization project on Town property that is included in Alternative IIB. Raising and/or floodproofing the residential structures at 1 Sylvan Lane, and along the left (east) bank downstream from Lagunitas Road Bridge may also be required.*

- *The estimated design capacity for the recommended Alternative IIA plan (g13; Plan 27) is 5,200 cfs.*

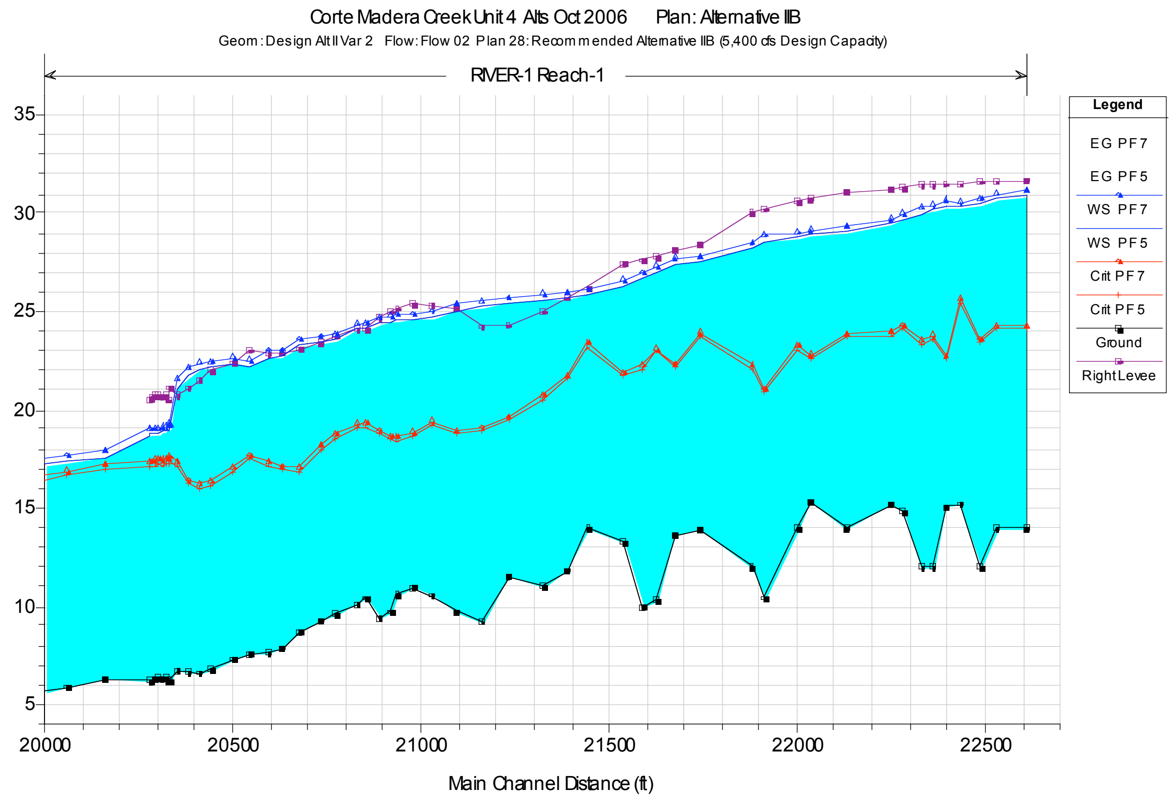

Figure 31. *Plan 28.* Comparison of flood water surface elevation profiles for PF5 (5,400 cfs) and PF7 (5,600 cfs) under recommended Alternative IIB (g14, Plan 28).

- *Under Alternative IIB, a 5,400 cfs discharge produces maximum 1.1 ft right overbank flow upstream from the Lagunitas Road Bridge and in the vicinity of 1 Sylvan Lane. It is assumed that this depth of overbank flow can be prevented with either a temporary seasonal floodwall (e.g., sandbag wall placed along the edge of Sylvan Lane), or with permanent modification to the Sylvan Lane top of pavement profile in coordination with or part of the planned Lagunitas Road Bridge removal and replacement project. Raising and/or floodproofing the residential structures at 1 Sylvan Lane, and along the left (east) bank downstream from Lagunitas Road Bridge may also be required.*

- *The estimated design capacity for the recommended Alternative IIB plan (g14; Plan 28) is 5,400 cfs.*

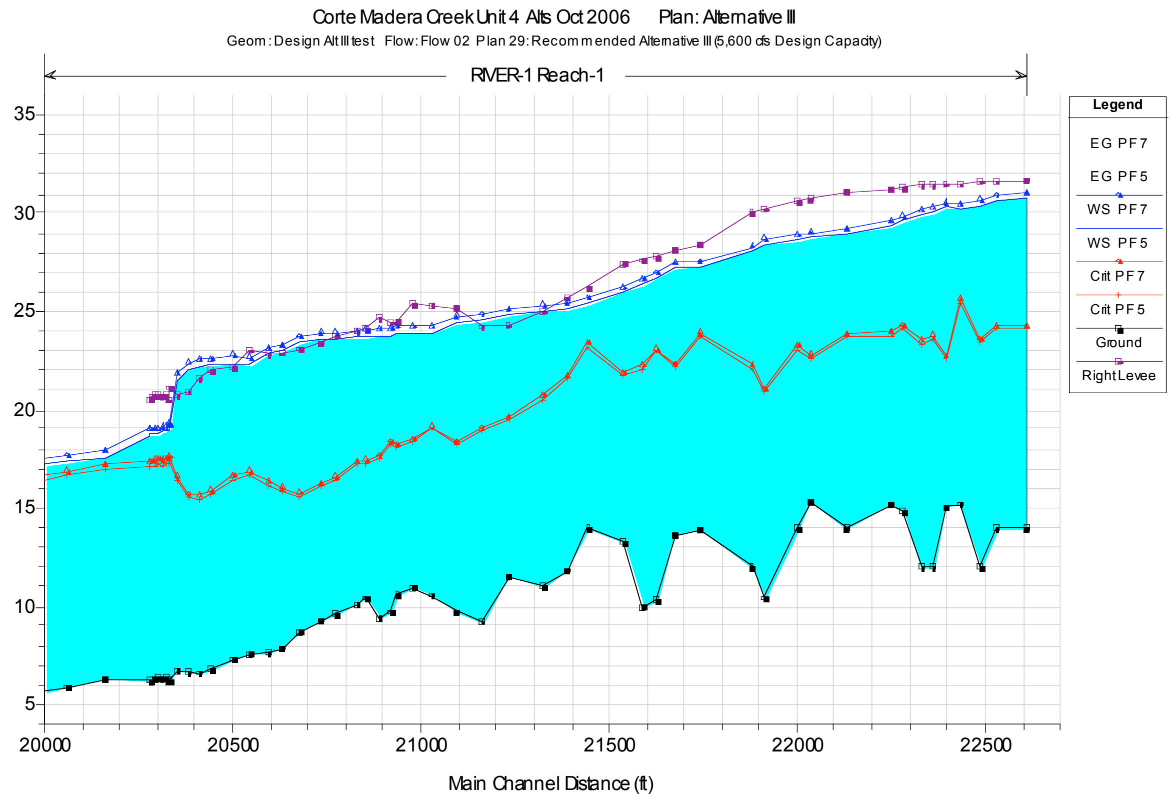

Figure 32. *Plan 29.* Comparison of flood water surface elevation profiles for PF5 (5,400 cfs) and PF7 (5,600 cfs) under recommended Alternative III (g15, Plan 29).

- *Under Alternative III, a 5,600 cfs discharge produces maximum 0.8 ft right overbank flow upstream from the Lagunitas Road Bridge and in the vicinity of 1 Sylvan Lane. It is assumed that this depth of overbank flow can be prevented with either a temporary seasonal floodwall (e.g., sandbag wall placed along the edge of Sylvan Lane), or with permanent modification to the Sylvan Lane top of pavement profile in coordination with or part of the planned Lagunitas Road Bridge removal and replacement project. Raising and/or floodproofing the residential structures at 1 Sylvan Lane, and along the left (east) bank downstream from Lagunitas Road Bridge may also be required.*

- *A 5,600 cfs discharge also produces about 1.5 ft right overbank flow immediately upstream from the Unit 3 concrete channel, an amount roughly commensurate with a 3,200 cfs flow under existing conditions, and therefore considered within the design capacity. It is assumed that this depth of overbank flow can be prevented with detailed design for the top of the Alternative III flood wall, nowhere to exceed 2 ft above the existing adjacent sidewalk elevation.*

- *The estimated design capacity for the recommended Alternative III plan (g15; Plan 29) is 5,600 cfs.*

Alternative III produces a 200-cfs increase the Unit 4 design capacity compared to Alternative IIB, but it requires the retaining wall to act as a low floodwall, rising no more than 2 ft above the adjacent sidewalk elevation from the existing right bank vertical steel sheetpile wall behind the Post Office to the Unit 3 channel entrance. However, while this wall would prevent higher flows from escaping the channel to the west, it would serve to exacerbate the depth of flood flows escaping the channel to the east. Therefore, Alternative III may also require a low 2-3-ft high floodwall along the top of the left (east) bank.

Part 5 Summary

Five new Unit 4 design alternatives are presented for the Corte Madera Creek Flood Control Project (Table 4). The designs are intended to address multiple objectives, including:

- fish passage improvement in the Unit 3/4 transition;
- increased flood flow capacity;
- preservation and restoration of native riparian canopy forming trees;
- preservation of the natural aesthetic character of the stream;
- grade control maintenance and bank stability;
- existing sanitary sewer line protection; and,
- hydraulic design for Lagunitas Road Bridge replacement.

Table 4. Summary of recommended new October 2006 design alternatives.

Plan No.	Plan Title	Design Capacity (cfs)	
Plan 25	Baseline (Existing Conditions) Alternative	3,200	
Plan 26	Alternative I	3,200	Figure 33
Plan 27	Alternative IIA	5,200	Figure 36
Plan 28	Alternative IIB	5,400	Figure 37
Plan 29	Alternative III	5,600	Figure 38

More detailed descriptions of the 21 new model plan files developed in this study are presented in Tables 5 and 6. Table 7 provides a list of trees identified by species and diameter, as measured and marked with numbered blue metallic shiners. Tables 5, 6, and 7 are in Appendix D.

Definition of Unit 4 Design Channel Capacity

The definition of Unit 4 design channel capacity is critical to the comparative evaluation of the recommended design alternatives. Recall that the Corps' 1999-2000 design alternatives used a singular definition of channel capacity – the 25-ft Lagunitas Road Bridge deck elevation. Although the October 2006 design alternatives sought to apply a reach-scale definition of channel capacity (discussed in Part 1), the hydraulic effect of the transition to the narrow, smooth-surfaced Unit 3 concrete channel causes the downstream most 200-ft length of Unit 4 to overtop its banks at a significantly lower discharge than the remainder of the study reach. For this reason, the constraint effectively limits, and thereby defines or quantifies the Unit 4 design channel capacity. Still, careful evaluation of the model-calculated water surface elevation profiles shows that although the flood water elevations for all of the recommended design alternatives are similar in the downstream-most 200-ft length of Unit 4, there are differences in the upstream remainder of the reach that would have an impact on, among other things, the design feasibility and construction cost of Lagunitas Road Bridge replacement. Moreover, with consideration of low floodwalls in the worst performing, or "weakest link", downstream portion of Unit 4, its design channel capacity may be increased over the capacities reported in this memorandum.

Consideration of Temporary or Permanent Low Floodwalls in Unit 4

Although not explicitly designed in this study, temporary or permanent low floodwalls could be used to increase the design channel capacity of Unit 4. As discussed above, the Unit 3 transition exerts hydraulic control on the reach which causes the downstream most 200-ft length of Unit 4 to overtop at a lower discharge (the design capacity discharge). Using low (2-3 ft high) floodwalls, either temporary (e.g., sandbags), or permanent (e.g., concrete) would thereby increase the design channel capacity of the reach. Permanent low concrete flood walls could be architecturally designed to appear and function as bench seating along the pedestrian right of way.

Note that the left (east) bank residential properties near the creek corridor are generally 0.5-1.0 ft lower than the right (west) bank. Therefore, placement of floodwalls (either temporary or permanent) on the right (west) bank would exacerbate flooding affecting the residential properties. The residential properties should either be floodproofed according to the design right (west) bank floodwall elevation profile, or there should be a similar floodwall installed along the left (east) bank.

As an alternative to low floodwalls, the right (west) overbank flows could be contained within the existing parking areas and conveyed back to the Unit 4 channel immediately downstream from the transition where the water surface elevations are much lower than the top of the concrete channel wall. This could be done by recontouring the existing parking lot surface and installing permanent low concrete walls along the western edge of the parking lot (away from the creek) in a configuration that directs the flow back to the Unit 3 channel. The "second stage" floodwall would need to be discontinuous at automobile, bicycle, and pedestrian throughways. Temporary sandbag walls would need to be installed during early flood warnings to make the second stage flood wall continuous.

High water marks from the December 31, 2006 flood were rather uniformly between 1.0 and 1.5-ft high along the existing 625-ft long chain link fence at the top of the right (west) bank. The model-calculated water surface profile for 2005 existing conditions (e.g., Figure 2) does not match this profile well, in that it shows greater overbank depths in the lower 200-ft length of Unit 4 and no overtopping upstream. This is partially a model artifact. The model does not contain a wide floodplain area on the right bank. Therefore, the model, in effect, keeps all of the overflow in the channel – it does not calculate the lower in-channel discharge that would result for overbank flow leaving the channel and flowing into Downtown Ross. That is to say, the modeling documented in this memorandum is in-channel modeling for the purposes of determining in-channel design channel capacity. For modeling higher discharges or for calibrating to measured high water mark profiles, such as the December 31, 2005 flood profile, it would be more correct to model the study reach as split-flow, designating the existing right bank top of bank profile as a parallel overflow weir.

The detailed design, feasibility and performance of Design Alternatives with design channel capacities higher than existing conditions (3,200 cfs) also depend on results of the overall Ross Valley Watershed Management Initiative, including successful identification, design, funding, and implementation of flood management improvement projects upstream on San Anselmo Creek to, among other things: increase channel capacity in Downtown San Anselmo; increase Unit 2 and Unit 3 channel capacity with dredging, low floodwall parapets and/or channel widening; and routing floodplain and Murphy Creek flows originating in San Anselmo and Ross through Kentfield to return them back to Corte Madera Creek. Selection of the preferred alternative design and design capacity for Unit 4 should depend both on what design capacity is achieved upstream (and considering correct hydraulic modeling of peak tributary inflows from Ross Creek), and how flood flows entering Kentfield are to be handled.

Unit 3 Transition appears an Intractable Hydraulic Constraint

The October 2006 modeling and design work has shown that the transition from the natural, somewhat hydraulically rough Unit 4 channel to the narrower and hydraulically smoother (i.e., less friction) Unit 3 concrete channel is an intractable hydraulic constraint. Under all of the design alternatives and variants considered, it is this transition – the hydraulic jump that occurs there, and the rapidly varied flow immediately upstream in Unit 4 – that raises the local water surface elevation, and, in effect, "pushes" the floodwaters overbank to the east and west. The design alternatives and variants considered included longer and more gradual (plan view) transitions into Unit 3 – all of which made slightly worse, not better the hydraulic effect of the transition compared to existing conditions. A practically infinite number of design alternatives and variants could be tested with the model, including a wide range of approach angles and transition lengths, but none of the small number of combinations selected over a wide range for testing in this study proved the hydraulic constraint could be significantly mitigated, without, for example dismantling and reconstructing the upstream most several hundred feet of the Unit 3 concrete channel to provide for a very gradual transition from the Unit 4 design minimum channel width to the 33-ft Unit 3 channel width. Even these drastic modifications, if modeled, may show that the hydraulic constraint can't be mitigated, in so far that the constraint may be entirely the result of the supercritical flow regime in the upstream end of the Unit 3 channel. Recall that the Unit 3 channel was designed to flow under supercritical flow conditions. (Note: only the upstream, steeper portion of the Unit 3 channel does.)

Feasibility of Biotechnical Bank Stabilization

In general, biotechnical bank stabilization techniques are feasible but only with limited application in Unit 4. This modeling work has shown that application of necessarily gradually sloped biotechnical treatments along the right (west) bank would not increase flood capacity of Unit 4 enough to warrant deeper exploration or more detailed design. The right (west) streambanks are already very steep under existing conditions, such that they cannot be made less steep without also removing virtually all of the existing riparian canopy-forming trees along the reach as well as the existing top of bank infrastructure (pedestrian right of way, parking areas, water and power utilities). Vertical retaining walls are required to provide for measurable increases in channel capacity while saving a portion of the existing canopy-forming trees. The October 2006 design alternatives minimize the aesthetic impacts of vertical walls by limiting their application to the right (west) bank where they can only be seen from Lagunitas Road Bridge and the private residential properties on the opposite bank. Other aesthetic improvements can be made during the detailed design phase, including options for naturalist or architectural rock masonry facing. It is recommended that professional architectural renderings be prepared to facilitate public review of these recommended alternatives.

Alternatives IIB and III include purely biotechnical bank stabilization treatments at two left (east) bank sites where it was necessary to widen the channel and the right (west) bank could not be widened further without removing existing public and private infrastructure. These biotechnical stabilization projects could also be incorporated in Alternative IIA.

Although the application of biotechnical stabilization techniques is indeed limited by the narrow channel and steep bank conditions in Unit 4, it's important to note the geomorphic and environmental benefits of the traditional engineering stabilization techniques more universally applied by the recommended alternatives. Alternatives IIB and III include more traditionally engineered vertical retaining wall structures that are not considered biotechnical. However, they also substantially widen the channel and thereby create more room for the creek to perform its natural processes of channel meandering and floodplain sediment deposition – the very processes that support healthy aquatic habitats and self-sustaining riparian plant communities. Viewed this way, Alternatives IIA, IIB, and III can be seen to provide for long-term benefits to both the environmental and natural aesthetic attributes of the stream corridor.

Hydraulic Design for Lagunitas Road Bridge Replacement

The detailed hydraulic modeling work completed in this study showed that the channel is sufficiently wide in the vicinity of the existing Lagunitas Road Bridge. Widening the channel as part of the planned replacement of the bridge would provide negligible, if any, hydraulic benefits. This modeling work assumed that the bridge would be replaced according to a hydraulic design specifically intended to exert no hydraulic effect on creek flows up to the design channel capacity. This is the same thing as assuming the replacement bridge structure would have: practically the same vertical concrete abutments as existing; no piers; and a clear-span bridge deck with a minimum (low-chord) elevation equal to or greater than the model-calculated design channel capacity flow water surface elevation at the bridge location. Do note, however, that Alternative III includes a new vertical retaining wall that would replace the existing right bank concrete bridge abutment and widen the necessary bridge span about 6-7 ft. Model results displayed in Part 5 of this memorandum are sufficient for determining the design low-chord elevation for each of the five recommended design alternatives.