Stinson Beach Watershed Program Flood Study and Alternatives Assessment

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Marin County Flood Control and Water Conservation District

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EXECUTIVE SUMMARY

A County-wide Watershed Program was initiated in 2008 to provide a framework to integrate flood protection and environmental restoration with public and private partners. A watershed program was subsequently initiated for Stinson Beach to develop a suite of projects that address on-going flooding and sedimentation issues in the lower sections of the Easkoot Creek while maintaining and improving habitat for steelhead salmon. A series of technical studies were conducted to evaluate existing creek and flood plain conditions and to develop and analyze alternatives. Based on community input, the evaluation of alternatives focuses on flood protection, habitat restoration and emergency access. This report describes the alternatives that were identified and quantifies the benefits and cost of each. Studies to date have not evaluated coastal flooding caused by wave action or high tides. These impacts are significant and should be considered as flood protection alternatives are evaluated for their feasibility. Future sea level rise conditions at the mouth of Easkoot Creek in Bolinas Lagoon were evaluated. The complete version of this report is available online.

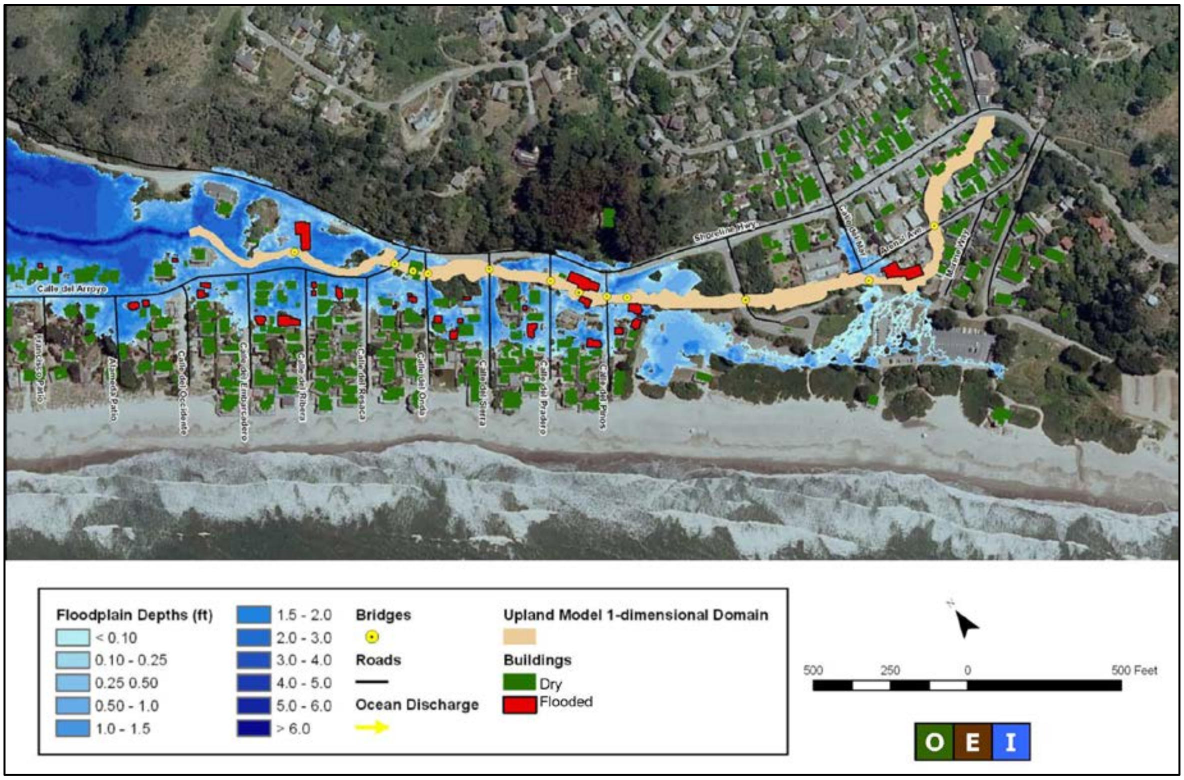
METHODOLOGY

In September 2011, O'Connor Environmental was awarded a contract to develop a hydrology and hydraulics model and complete an alternatives analysis. A computer model was developed for Easkoot Creek and its floodplain using Mike Flood 2D software. The model replicates the existing ground conditions across the watershed and shows the direction and velocity of the water in and out of the creek during floods. This information is used to determine what happens during a range of storms and to evaluate the change in flood conditions under various alternatives.

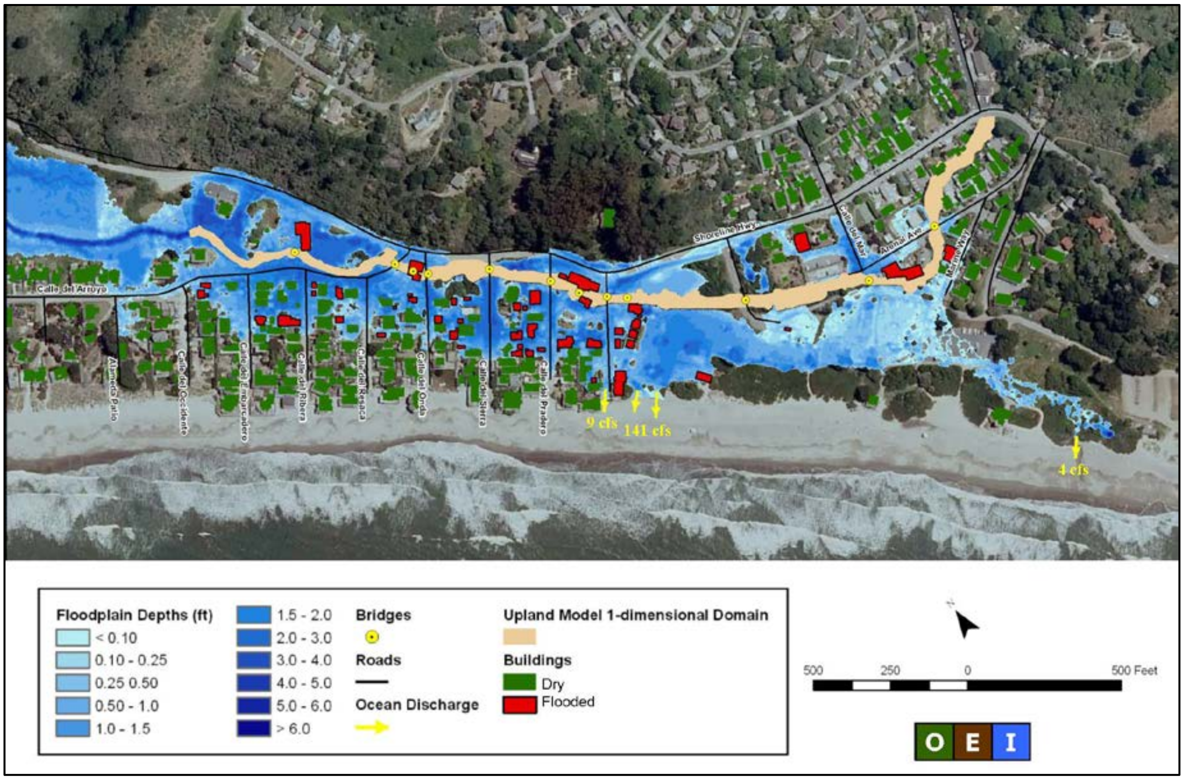
The December 31, 2005/06 storm was used as the reference storm to model baseline flood conditions. In addition, the 100-yr flood[1] was modeled when an alternative showed significant benefits for the 2005 flood. The impacts of sea level rise in Bolinas Lagoon on creek water surface elevations were simulated and evaluated for various flood protection and access alternatives. Projected sea level rise conditions were analyzed using the December 2005 flood with a tidal boundary condition of Mean Higher High Water (high tide) plus 18.2 inches of sea level rise.

[1] The term "100-year flood" refers to a probability that a storm of a certain size would occur in a given year. The 100-year storm has a 1% or 1 in 100 chance of occurring any given year,

The December 2005 flood as depicted by the computer model.



The 100-year flood in Stinson Beach as depicted by the computer model.



NOTE REGARDING TERMINOLOGY: Three alternatives, Alternatives 5, 6, and 9, have had their names changed from those used in previous drafts of this report. Alternative 5, formerly the 'North Bypass' alternative, is now 'Wetland Creation and Bypass to the National Park Service's North Parking Lot.' Alternative 6, formerly 'South Bypass,' is now 'Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.' Lastly, Alternative 9, formerly 'Combination Dredge and South Bypass,' is now 'Combined Dredge, Wetland Enhancement, and Bypass.' These new names were suggested by members of the community with the intent of greater precision and to emphasize the fact that wetland enhancement is a priority of the project.

ALTERNATIVES ANALYSIS

The Stinson Watershed Program has been developed to support a community decision-making process. The alternatives selected for evaluation were drawn from prior studies and from meetings with the TWG, and were presented at a public meeting in Stinson Beach in April 2012. Rather than identifying a 'preferred' alternative, the benefits and constraints of each alternative have been assessed and summarized to assist decision-making regarding future flood mitigation activities. Nevertheless, when the objective results of flood analyses indicated that a particular alternative did not substantially reduce flood impacts or that there were significant constraints bringing the feasibility of implementation into question this was noted and in some cases the effort to develop and evaluate additional details was curtailed.

Alternative 1-No Action is designed to represent a future "no action" condition. It assumes increased sediment accumulation in Easkoot Creek.

Alternative 2-Bridge Improvements considers modifications to or replacement of 12 existing bridges (5 public, 7 private) over Easkoot Creek.

Alternative 3-Vegetation Management investigates the potential for flood mitigation resulting from reducing roughness on the channel banks in Easkoot Creek through a program of vegetation management.

Alternative 4-Channel Dredge and Sediment Management consists of removing 3,100 cubic yards of sediment from a 2,300 foot reach of Easkoot Creek from upstream of Arenal Avenue to downstream of Calle del Arroyo.

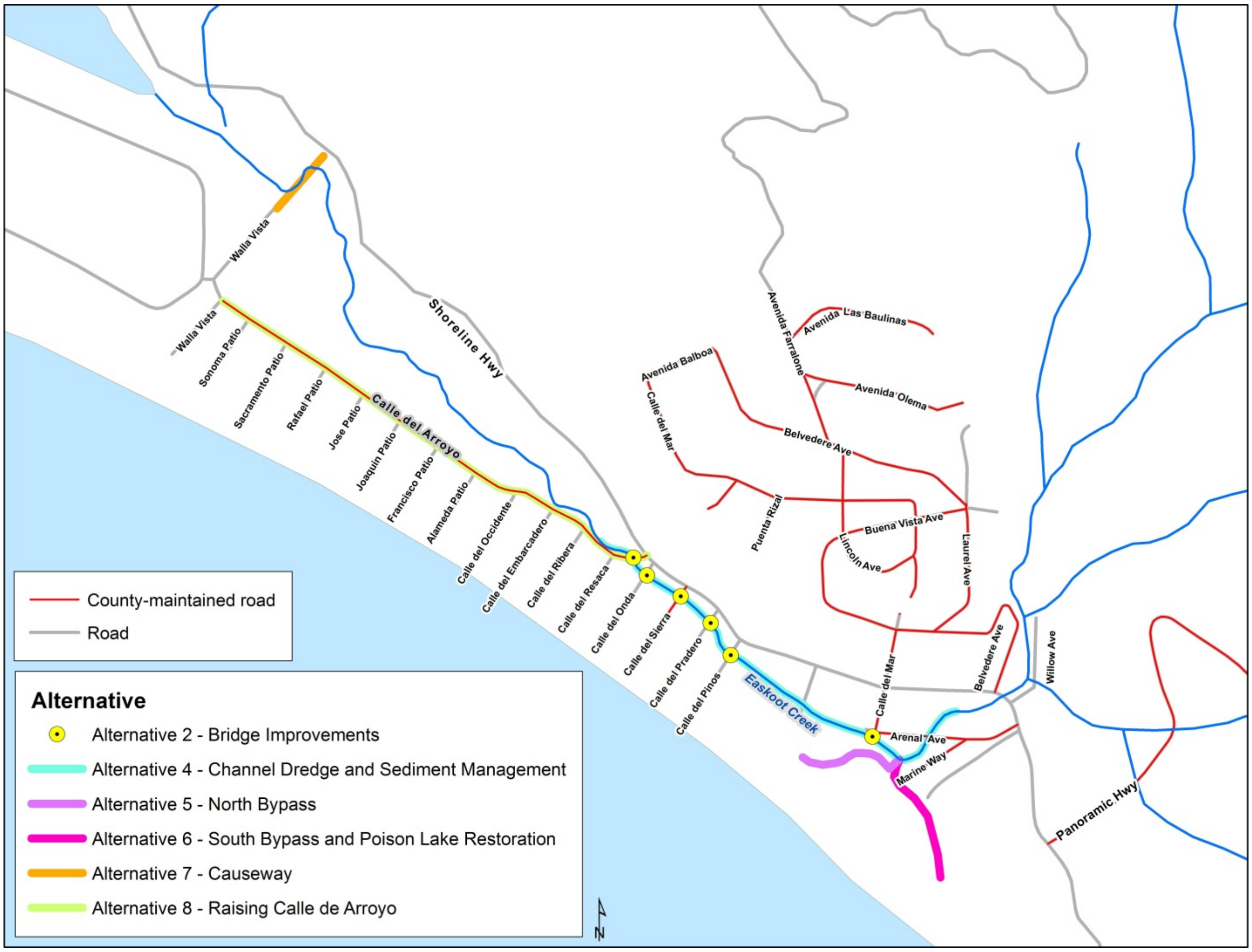
Alternative 5- Wetland Creation and Bypass to the National Park Service's North Parking Lot involves the construction of a bypass channel to the Park Service's north parking lot to divert a portion of the discharge of Easkoot Creek away from flood-prone lower reaches during high flow conditions.

Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot is similar to Alternative 5 except that it would also include the restoration of pond and wetland habitat in the vicinity of historical Poison Lake.

Alternative 7-Causeway construction over Bolinas Lagoon to connect State Highway 1 with Seadrift Road along the alignment of what is currently a gravel road named Walla Vista Road.

Alternative 8-Raising Calle del Arroyo involves elevating the entire length of Calle del Arroyo, a County maintained road, between State Highway 1 and Seadrift Road.

Overview of proposed flood mitigation alternatives.



Alternative 9-Combined Dredge, Wetlands Enhancement, and Bypass combines the features of Alternative 4-Dredge and Wetland Enhancement (near Poison Lake) and Alternative 6-Bypass to the National Park Service's South Parking Lot.

Alternative 10-Structure Elevation involves elevating buildings so that the ground floor is situated above flood elevation.

Other Alternatives Considered. Three alternatives had significant constraints that eliminated them from further consideration: direct ocean bypass, increased floodplain and off-channel habitat, and infiltration and storage of rainwater (for flood protection). The floodplain and infiltration alternatives were eliminated from further evaluation because they did not improve the level of flood protection. The bypass could not be engineered to protect steelhead trout and coho salmon while bypassing flood waters and therefore could be difficult or impossible to permit by the resource agencies.

KEY FINDINGS

The purpose of this study was to evaluate existing creek and floodplain conditions and alternatives that provide benefits for flood protection, habitat restoration and emergency access. Modeling shows the most effective alternative with respect to flood mitigation appears to be Alternative 9-Combined Dredge, Wetland Enhancement, and Bypass. Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot has similar flood benefits to the Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot, but existing infrastructure including nearby homes, Park Service septic system and a paved parking lot made the Alternative 5 site less feasible than that of Alternative 6. For full details, please see Chapters 12 and 13. Alternatives 5 and 6, as well as Alternative 2-Bridge Improvements and Alternative 4-Dredge, are less effective than Alternative 9-Combined, and are about equally effective compared with each other. Alternative 4 removes more buildings from the December 2005 floodplain but less than the others from the 100-yr floodplain, and Alternatives 5 and 6 result in improvements that extend downstream to the Lower Calles reach whereas Alternatives 2 and 4 do not. The remaining alternatives result in only minor improvements, and the Alternative 1-No Action is the only alternative that exacerbates flood hazards. Although they do not result in significant reductions in peak water levels or the number of flooded buildings, both Alternative 7-Causeway and Alternative 8-Raising Calle del Arroyo reduce flood hazards by improving access for residents of the lower watershed during flood conditions. Of these two options, Alternative 8 would improve access to the lower Calles, Patios, and Seadrift areas whereas Alternative 9 would only improve access to Seadrift when Calle del Arroyo is flooded.

Average change in peak water levels for the December 2005 flood.

Alternative	Average Change in Water Level (ft)		
	Parkside Café	Upper Calles	Lower Calles
1 No Action	+0.5	-0.1	-0.2
2 Bridge Improvements	-0.3	-0.2	0.0
3 Vegetation Management	0.0	0.0	-0.1
4 Channel Dredge and Sediment Management	-2.6	-1.1	0.0
5 Wetland Creation and Bypass to the National Park Service's North Parking Lot	-0.6	-0.4	-0.4
6 Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot	-0.6	-0.4	-0.6
7 Causeway	0.0	0.0	-0.1

8 Calle del Arroyo	0.0	0.0	0.0
9 Combined Dredge, Wetland Enhancement, and Bypass	-3.6	-2.2	-0.8

Number of buildings flooded in relation to the December 2005 flood.

Alternative	# of Flooded Buildings	# of Buildings No Longer Flooded
Existing Conditions	24	-
1 No Action	32	-8
2 Bridge Improvements	13	11
3 Vegetation Management	24	0
4 Channel Dredge and Sediment Management	6	18
5 Wetland Creation and Bypass to the National Park Service's North Parking Lot	13	11
6 Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot	13	11
7 Causeway	23	1
8 Calle del Arroyo	22	2
9 Combined Dredge, Wetland Enhancement, and Bypass	0	24

NEXT STEPS

Significant stakeholder and community input will be needed as the proposed alternatives include opportunities on private property and Federal lands and no funding is available to construct any of the proposed improvements. We plan to continue meeting with our stakeholders and the community to secure input and select alternatives to carry forward. Cost estimates for the final list of alternatives will be refined to include operations and maintenance requirements, environmental compliance, and easement acquisition. An analysis of funding strategies is being performed concurrent with the alternatives refinement. A public meeting to discuss the report and next steps will be scheduled early winter 2013. Information will also be posted online at www.marinwatersheds.org/stinson_beach.html

Contact Chris Choo at 415.473.7586 or cchoo@marincounty.org with questions or comments.

Comparison of alternatives.

Alternative	Description	Flood Benefit (For the modeled December 2005 flood)[2]	Cost to Construct[3]	Creek channel capacity (related to sediment and flow)	Fisheries concerns (impacts permit approvals)
1-No Action	No action assumes increased sediment accumulation in the creek.	Increased flooding to 8 homes for a total of 32 flooded homes.	None	Unmitigated sedimentation leads to significantly reduced channel capacity in < 10 years	Increased risk of stranding on floodplain; in-stream habitat degraded.
2-Bridge Improvements	Modifications to or replacement of 12 existing bridges (5 public and 7 private) over Easkoot Creek.	11 homes no longer flood.	Between $4-5 million. Operation and maintenance is minimal for the lifespan of the bridge.	Modest local change at modified and unmodified bridges possible	Somewhat reduced risk of stranding on floodplain; minimal change to in-stream habitat
3-Vegetation Management	Reduce 25% more vegetation along the creek channel except where structures exist.	Negligible benefit to flooding.	Between $5,000-7,000 per year.	Minimal change expected	Minimal change expected
4-Channel Dredge & Sediment Management (over entire channel length)	Remove 3,100 cubic yards of sediment from a 2,300-foot reach of Easkoot Creek between Arenal and Calle del Arroyo.	18 homes no longer flood.	Between $1.5-2.5 million. Anticipated to be needed once every ten years. $40-50,000 for annual operation and maintenance.	Reduced rate of sedimentation and reduced impact on conveyance due to near term future sedimentation; improvement temporary unless maintained by on-going sediment management including potential future dredging	Much reduced risk of stranding on floodplain; disturbed habitat may be improved by enhancement actions and implementation methods; habitat improvement in lower Easkoot temporary. Upstream sedimentation facilities could improve habitat.
5-Wetland Creation and Bypass to the National Park Service's North Parking Lot	Construct a bypass channel to the Park Service's north parking lot to divert a portion of the discharge of Easkoot Creek away from flood-prone lower reaches during high flow conditions.	11 homes no longer flood.	Between $1-2 million. Unknown costs for annual operation and maintenance.	Some redistribution of sedimentation expected-decreased potential near Arenal Avenue and increased potential near Calle del Mar; new sedimentation possible in bypass channel	Risks to fish lost to bypass are relatively high. An alternative path to the ocean is provided for fish. Reduced flooding lowers probability of stranding for other fish

[2] The computer model of the December 31, 2005 storm shows 24 flooded homes. Finished floor elevations surveyed by Flood Control staff provided elevations of the living area of homes. This column shows the number of homes that would flood after an alternative is implemented using the same December 31, 2005 storm. For homes without survey data, flooding was assumed when floodwaters adjacent to a home reached depths of 0.5-feet or greater. (Garages, sheds, yards, and utilities may still be flooded.)

[3] Cost estimates are extremely preliminary and do not include real estate acquisitions or easements, mitigation, or full permit preparation. The California Environmental Quality Act (CEQA, National Environmental Protection Act or NEPA for projects involving Federal jurisdiction) requires an evaluation of impacts, positive and negative, short- and long-term for projects. In addition to disclosure of all known impacts, this process serves to inform and involve the public in decision-making. An Environmental Impact Report (EIR, or Environmental Impact Statement at the Federal-level) is used to assess a project and its alternatives, mitigation to address impacts, and then identify the top alternative based on the evaluation. CEQA/NEPA has not been factored into the cost of any alternative. For a complex project, an EIR/EIS can take several years and several hundred thousand dollars to complete.

Alternative	Description	Flood Benefit (For the modeled December 2005 flood)[2]	Cost to Construct[3]	Creek channel capacity (related to sediment and flow)	Fisheries concerns (impacts permit approvals)
6- Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot	Similar to Alternative 5 except that it would also include the restoration of pond and wetland habitat in the vicinity of historical Poison Lake.	11 homes no longer flood.	Between $1-2 million. Substantial annual operation and maintenance for new flow and sediment management activities.	Some redistribution of sedimentation expected-decreased potential near Arenal Avenue and increased potential near Calle del Mar; new sedimentation possible in bypass channel and restored Poison Lake.	Risks to fish lost to bypass are relatively low, or beneficial due to potential high quality rearing habitat in restored Poison Lake; reduced flooding lowers probability of stranding of fish not entrained in bypass.
7-Causeway	Construction of causeway over Bolinas Lagoon to connect State Highway 1 with Seadrift Road along the alignment of Walla Vista Road.	N/A	Between $3-4 million to construct. Unknown costs for annual operation and maintenance.	No effect on sedimentation expected.	No effects expected.
8-Raising Calle del Arroyo	Elevate entire length of Calle del Arroyo between State Highway 1 and Seadrift Road.	N/A	Between $1-2 million to construct. Unknown costs for annual operation and maintenance.	No major effects expected.	Some potential reduction in floodplain stranding.
9-Combined Dredge, Wetland Enhancement, and Bypass	Combines Alternatives 4 and 6.	24 homes no longer flood.	Between $3.5-4.5 million. Substantial annual operation and maintenance for new flow and sediment management activities.	See above	See above
10-Structure Elevation	Elevate buildings so the ground floor is situated above flood elevation.	All homes are raised above the 100-year level of mapped flooding.	Between $50,000-100,000 per home. For all 24 homes the cost would total between $1.5-2.5 million. Unknown maintenance costs.	No major effects expected.	No effects expected.

1 INTRODUCTION

1.1 Overview

This study evaluates the existing creek and floodplain conditions of Easkoot Creek in the community of Stinson Beach, with respect to peak storm runoff and long-term sediment deposition. A range of conceptual alternatives is presented with the goals of providing protection from flooding, reducing sediment aggradation, and mitigating damage to salmon spawning habitat in the creek. The alternatives are described briefly in Section 4 of this Introduction and consist of raising bridges, homes, and Calle del Arroyo, dredging, bypasses for Easkoot Creek, altering creekside vegetation, and a causeway over Bolinas Lagoon. The alternatives are described in detail in Sections 8 through 17.

For each alternative, the study assesses probable benefits, as well as any issues likely to arise during the alternative's planning and implementation. The alternatives analysis, the results of which are summarized briefly in Section 6 (Summary of Results), is the culmination of an interdisciplinary study of the watershed begun in October 2011 under contract with the Marin County Department of Public Works. The contractor has worked closely with both the Department of Public Works and the Flood Control and Water Conservation District, and has participated in three project meetings with the Watershed Program's Technical Working Group (TWG) in Stinson Beach. The study area, its flood history, and the County-based flood control program are described below. Three sources contribute to the flooding in Stinson Beach: coastal storm surge, extreme tides, and Easkoot Creek. Notably, only riverine flooding from Easkoot Creek is evaluated here; coastal flooding caused by high sea level and storm surge is not the focus of this study.

The Community. Stinson Beach lies at the base of Mt. Tamalpais where the mountain meets the Pacific Ocean, southeast of Bolinas Lagoon, and comprises approximately 689 full and part time residences from the Seadrift neighborhood south to the residential area near the intersection of Shoreline Highway (State Highway 1) and Panoramic Highways. The community lies between several major landowners and special habitats. The National Park Service's Golden Gate National Recreation Area (GGNRA) includes a public beach and parking south of Shoreline Highway near the center of town. State Parks and GGNRA own the property above the community on the western slopes of Mt. Tamalpais. Steelhead trout and overwintering populations of Monarch butterflies are residents of the watershed; coho salmon have also reportedly been observed.

The District. The Marin County Flood Control and Water Conservation District (District) was formed in 1955 with the primary purpose of controlling flood and storm waters of streams which flow within and into the county. The District is staffed by the Department of Public Works and the Marin County Board of Supervisors serves as its overseeing body. The boundaries of the District are contiguous with those of the county and eight "zones" have been established to address specific issues related to flooding within individual watersheds. Zone No. 5, the location of the proposed study, includes the community of Stinson Beach, Easkoot Creek and its tributaries, and a small portion of Bolinas Lagoon.

Flood Control Zone No. 5. This zone was established in 1961 by the Board of Supervisors of the District and at the request of Stinson Beach residents to help address the reduction in flow capacity of Easkoot Creek due to the accumulation of sediment and debris within the waterway. To this effect, the District has dredged the creek on several occasions. The District has additionally commissioned a study of flooding concerns in Stinson Beach (Alternative Mitigation Measures for Storm and Flood Hazards – William Spangle & Associates, 1984), which included recommended options for mitigating coastal flooding and flooding of Easkoot Creek.

Marin County Watershed Program. The Board of Supervisors authorized the Department of Public Works to begin implementation of a County-wide watershed program on May 13, 2008. Staffing for this program is provided through the Flood Control and Water Resources division. The purpose of the watershed program is to provide a framework for flood protection and environmental restoration in Marin County's watersheds. The planning process will evaluate short and long term flood control needs and integrate these with environmental restoration opportunities where it makes sense to do so. The Easkoot Creek program will develop a suite of integrated projects that address flooding and sedimentation in the lower sections of the creek.

The watershed program includes extensive community outreach and public participation. Each watershed planning area has three committees to guide their planning process: the Policy Advisory and

Operations Committee, the Finance Committee, and the Technical Work Group. The Finance Committee is composed of the District County Supervisor and the Public Works Director, and the Policy Advisory and Operations Committee includes these two individuals as well as two members of the Flood Zone Five advisory board. The Technical Work Group is comprised of local watershed experts and technical staff of participating agencies such as the water and sanitary districts the National Park Service (NPS), NOAA fisheries, State of California Fish and Game, Marin County Open Space, research and science organizations and local watershed groups. This group will work with Federal and State regulatory staff that will provide input on program deliverables and watershed priorities. The group will also coordinate local business and homeowners groups within their respective watersheds. The Technical Work Group has met eight times during the study to review its progress. In addition to the committee process, the Watershed Program has a website with outreach and general watershed information at: www.marinwatersheds.org.

Flood History. Flooding that has caused significant damage to the community of Stinson Beach is documented as early as 1954. During the 1954 event, the Shoreline Highway Bridge over Easkoot Creek was heavily damaged leading to its reconstruction. Flood waters were diverted along streets parallel to the creek and the building housing the Parkside Café was damaged. Based on District records regarding dredging and knowledge of regional flood events, floods occurred in 1972/73, 1982/82, 1986, 1997, and 2005. The event that occurred in December 31, 2005, is relatively-well documented by hydrologic records. The event(s) that occurred in the El Niño winter of 1982/83 appear to have been the most severe since 1954. Landslides in the upper watershed have been documented in many of these same winters.

Existing Conditions. Easkoot Creek drains approximately 1.59 square miles of mostly undeveloped, steep and heavily forested watershed on the western escarpment of Mt. Tamalpais. Flow gauging of the creek has been conducted on GGNRA property by NPS since 2000. The Stinson Beach County Water District (SBCWD) conducted a stream gauging program on Easkoot Creek tributaries in 2004-05.

Figure 1-1 Stinson Beach location map.

Early maps depicting the proposed subdivision development in the 1900s locate the channel of Easkoot Creek near its present alignment to a point just downstream of the sharp turn at Arenal Avenue. The historic channel then branches off as the channel slope loses its grade and enters a willow thicket (located on the current GGNRA land). The map also shows an alternative alignment for Easkoot Creek breaching the sand dunes. With the development of the Stinson Beach and Seadrift communities and the public park at Stinson Beach, Easkoot Creek has been maintained in an alignment to Bolinas Lagoon. The essentially flat reach from Arenal Avenue to Calle del Arroyo near the southeastern edge of Bolinas Lagoon (Figure 1-1) creates a slower moving creek and a natural area for sediment to settle and deposit.

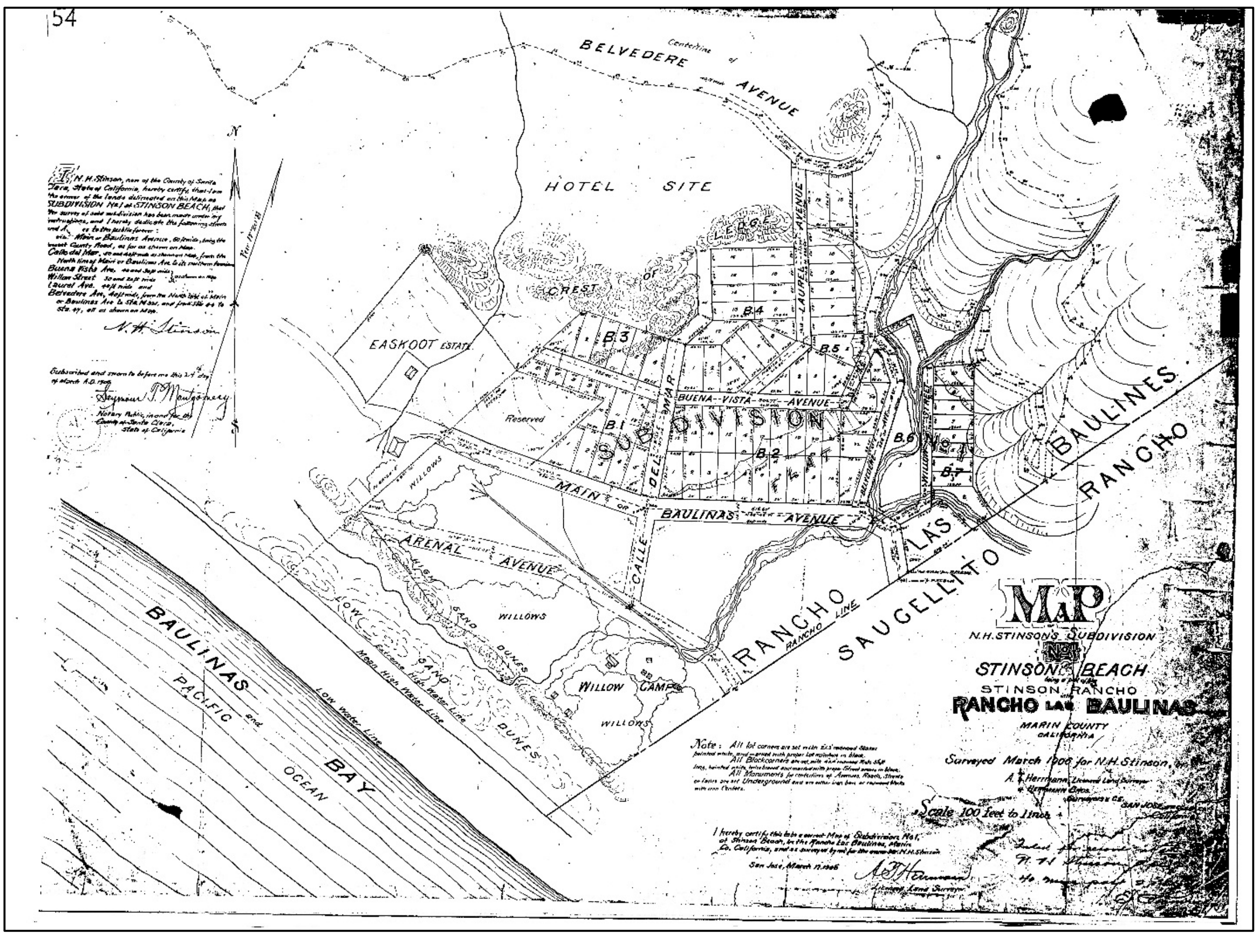


Figure 1-2 Early 1900s map of Stinson Beach.

Previous Flood Mitigation Concepts. Prior studies and work by the District and Watershed Program has identified several approaches that could be expected to mitigate flooding or its impacts. Private and County bridges across Easkoot Creek linking Shoreline Highway with the residential streets from Calle del Pinos to Calle del Arroyo (Figure 1-1) known collectively as the “Calles” have limited to no clearance (i.e., freeboard) above the creek during periods of peak storm runoff. This may contribute to the flooding of neighboring homes and preclude access to and from Shoreline Highway. These bridges are a substantial hydraulic constraint, but must be maintained to provide access to homes and to allow for emergency vehicles. Replacement of the bridges and elevating portions of Calle del Arroyo has been identified as potential flood improvement projects. A causeway across Bolinas Lagoon near the mouth of Easkoot Creek would provide access for residents as an alternate to Calle del Arroyo during emergencies. A causeway did exist historically at Walla Vista (across the arm of Bolinas Lagoon separating Shoreline Highway from the Seadrift community, but was removed and never rebuilt. The community has expressed interest in rebuilding the causeway as a future project. Additionally, vulnerable structures, such as private homes and businesses, could be elevated to reduce flood impact. The County’s Local Coastal Plan is currently being updated and contains draft language to ease the Coastal Zone requirements for such activities (LCP Policy C-EH-12 Floor Elevation Requirements for Existing Buildings in Flood Hazard Zones).

1.2 Scope of Technical Reports

This study is based on a series of seven technical memoranda and reports which are included in the Appendix. The scope of these studies was determined by the District’s Request for Proposals for the Easkoot Creek hydrology and hydraulics project. These individual reports describe the following analyses and assessments:

- Background Information and Data Acquisition Plan (Appendix pp. 1-9)
- Evaluation of Suitability of the Golden Gate LiDAR Data (pp. 10-18)
- Hydrologic Analysis and Modeling of Runoff and Peak Flow (pp. 19-48)
- Hydraulic Model and Flood Hazard Evaluation (pp. 49-91)
- Fish Habitat Existing Conditions and Enhancement Potential (pp. 92-98)
- Sediment Transport Evaluation (pp. 99-118)
- Geomorphic and Watershed Sediment Assessment (pp. 119-130)

These studies directly support the flood mitigation analysis and alternatives assessment and are occasionally referenced to support specific aspects of the flood mitigation analysis.

1.3 Primer on Hydrologic and Hydraulic Modeling

Floods are complex and episodic and developing solutions relies on computer models to simulate conditions before and during flood events. Describing flood phenomena for purposes of mitigating flood impacts cannot generally be based on direct measurements and observations, even if a stream gauge exists in the watershed. Instead, computer models are used for this purpose, and are capable of describing a range of floods, from relatively frequent events that have minor impacts to rare flood events capable of causing widespread property damage and harm to residents. The Hydrologic Analysis (Appendix, p. 19-48) develops estimates of peak flow for various flow recurrence intervals (e.g. the 100-yr flood; a flow event with a 1% probability of occurrence in any single years). This analysis took advantage of available stream gauging and rainfall data from various sources in the vicinity of Stinson Beach and Mt. Tamalpais, and was calibrated using specific storm events for which rainfall could be estimated and for which flow records were available. Although the use of local data is expected to improve the reliability of the hydrologic model, the model nevertheless requires certain assumptions and substantial uncertainties remain in estimation of the peak flows associated with flood events of specified recurrence intervals.

The Hydraulic Model and Flood Hazard Evaluation (Appendix, pp. 49-91) describes the implementation of a hydraulic model simulating in-stream flow as well as floodplain flows. This hydraulic model utilized available high-resolution topographic data of the Easkoot Creek study area from the Golden Gate LiDAR Project, as well as topographic data surveyed for this project in December 2011 (Appendix, pp. 10-18), to describe the channel and floodplain geometry. Field observations and aerial photography were used to evaluate flow roughness characteristics. Simulated flows were compared with observed historic flooding and with stream gauge data to evaluate model calibration. Although substantial uncertainty exists regarding the distribution and depth of flows in the channel and on the floodplain, the simulated flows and flood flows appear to be in reasonable agreement with observed instream flows and recent flood flows, particularly the event on December 31, 2005.

Having determined peak stream flows from the hydrologic model for the recurrence intervals of interest, a hydraulic model that routes those stream flows through a virtual stream channel network is used to identify flow rates and locations at which flooding may occur. The hydraulic simulation model requires topographic data to describe the shape and elevation of the stream channel and floodplain. Well-established algorithms and mathematical functions are used in the computer model to calculate the depth and velocity of stream flow at various locations. In addition to the controlling influence of shape and slope of the channel (referred to as channel geometry), the depth and velocity of flow is also controlled by the friction or roughness exerted by the channel on the flow. The material forming the channel bed and banks are primary determinants of flow resistance; for example, a channel with large caliber sediment such as cobbles and boulders and with brushy dense vegetation on the banks would have high roughness and a channel with sand and gravel and grassy banks would have much lower roughness. One of the most important elements of the hydraulic model development is assigning roughness values to the channel, its banks and the floodplain. The hydraulic model is calibrated by comparing simulated flows to observed flows and locations of flooding. Gauge data and observations describing a particular flood event can be invaluable in establishing confidence in simulation model predictions.

Although extremely valuable in predicting flood hazards, model simulations cannot readily incorporate the full range of dynamic conditions that may occur during a storm and flood event. For example, changes in stream channels during a storm (or more gradually over time) will alter its geometry and roughness characteristics. Depending on the magnitude of these changes, and whether or not large local changes occur, actual flows and flood phenomena may not conform to model predictions. Similarly, development on the floodplain, including construction of structures, fences, bridges, and roads, and changes in vegetation, can cause significant changes in conditions that diverge from conditions represented in the simulation model. This can cause substantial deviations between simulated flow/flood conditions and actual flood conditions. Nevertheless, hydraulic models remain the most reliable means of predicting flooding; model results indicate the likely distribution and approximate magnitude of flooding, but should not be expected to accurately predict the precise locations and depths of flooding in all situations and locations.

The hydrologic and hydraulic models developed for this evaluation of flooding in Stinson Beach are relatively detailed in relation to the size of the watershed. The hydraulic model results can be overlaid on aerial photo maps to portray the extent of floodplain inundation during floods. As discussed above, actual flooding may not occur as predicted by model simulations; the maps show well-founded estimates of likely flooding, but should not be considered to precisely represent actual flooding. Under no circumstances do these flood maps represent coastal flooding associated with storm surge from the Pacific Ocean.

2 WATERSHED PROCESSES AND SUSTAINABILITY OF FLOOD MITIGATION MEASURES

Stinson Beach has been subject to periodic flooding when large storms produce high rates of runoff from the upper watershed of Easkoot Creek. The severity and extent of flooding is strongly influenced by the channel capacity of lower Easkoot Creek, which has been significantly reduced by ongoing sedimentation. Watershed erosion processes have a strong influence on sedimentation in lower Easkoot Creek; storm events that cause flooding generate high stream flow, accelerated erosion and sediment supply to tributary channels, and high sediment transport rates in the watershed. The dominant watershed erosion process in upper Easkoot Creek is mass wasting (landslides) on the steep slopes adjacent to stream channels (Figure 2-1); landslide rates increase during intense, long-duration rainstorms. Sediment delivered to tributary channels may be stored for several years in and adjacent to the channel awaiting high stream flow events that are capable of transporting sediment through the channel network to lower Easkoot Creek. The flood prone portions of Stinson Beach are located along Easkoot Creek and the floodplain or lower elevation areas of the watershed. Portions of these areas are located atop an alluvial fan, which in geologic terms is a very dynamic setting from a sediment perspective and presents a range of challenges when evaluating flood protection alternatives.

Figure 2-1 Recent debris slide scarp on an Easkoot Creek tributary near Table Rock.

2.1 Alluvial Fan Processes

Alluvial fans are characterized by declining slope and sediment transport capacity, channel avulsions, extreme variations in erosion and sedimentation, and shifting channel positions and patterns of flooding. The alluvial fan is the deposit of rocks, sand, and other smaller materials in the watershed due to changes in velocities of water and elevation. The alluvial fan of Easkoot Creek extends to the back-beach environment a few feet above sea level such that water and sediment routed from the upper watershed across the fan encounters a relatively flat and broad floodplain. These conditions are portrayed prior to the development of Stinson Beach in Figure 2-3. Urbanization and development of Stinson Beach resulted in channelization of Easkoot Creek, perhaps establishing a defined channel draining towards Bolinas Lagoon where such a channel may not have previously existed. Given the uncertainty of this natural process of sediment deposit at the base of Mt. Tam in Stinson Beach, it is difficult to maintain a channel free of sediment, and therefore challenging to build projects in this area.

In the upper watershed, steep confined channels maintain continuity of flow and sediment transport. This continuity of flow and sediment transport extends across Easkoot Creek's upper alluvial fan to the vicinity of Arenal Avenue where declining channel slope reduces sediment transport capacity (Figure 2-2). Further downstream in Easkoot Creek on the toe of the alluvial fan below the Park Entrance Bridge, channel slope diminishes further and bank height declines to about three feet. Under these conditions of declining slope and channel confinement, channel sedimentation inevitably results.

Contemporary sedimentation in lower Easkoot Creek (below State Highway 1) was analyzed using data on historic dredging, modeling of sediment transport rates, and estimates of watershed erosion rates based on prior studies to assess likely future sedimentation and its impact on potential flood mitigation strategies, sustainability of salmonid fish habitat, and flood conveyance capacity in Easkoot Creek. The sedimentation analysis revealed that flood events with a recurrence interval of about ten years (ten percent probability of occurrence in any given year) are likely to cause significant sediment deposition in lower Easkoot Creek on the order of 1,000 cubic yards or more. Dredging of several thousands of yards

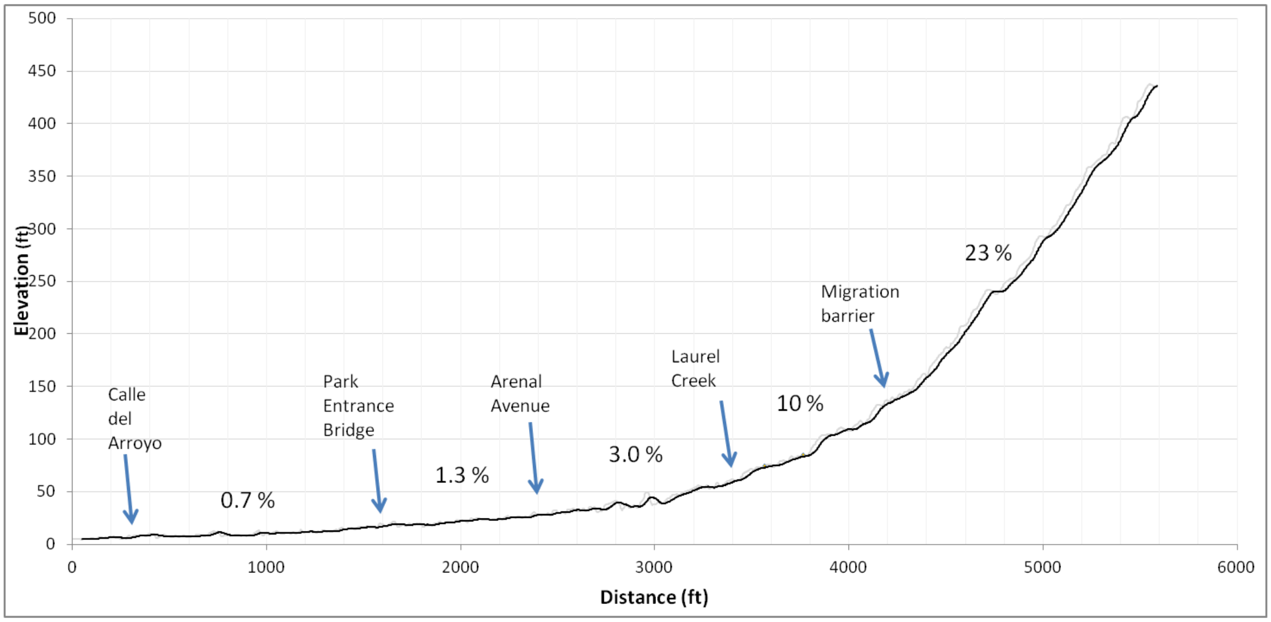


Figure 2-2 Easkoot Creek channel profile.

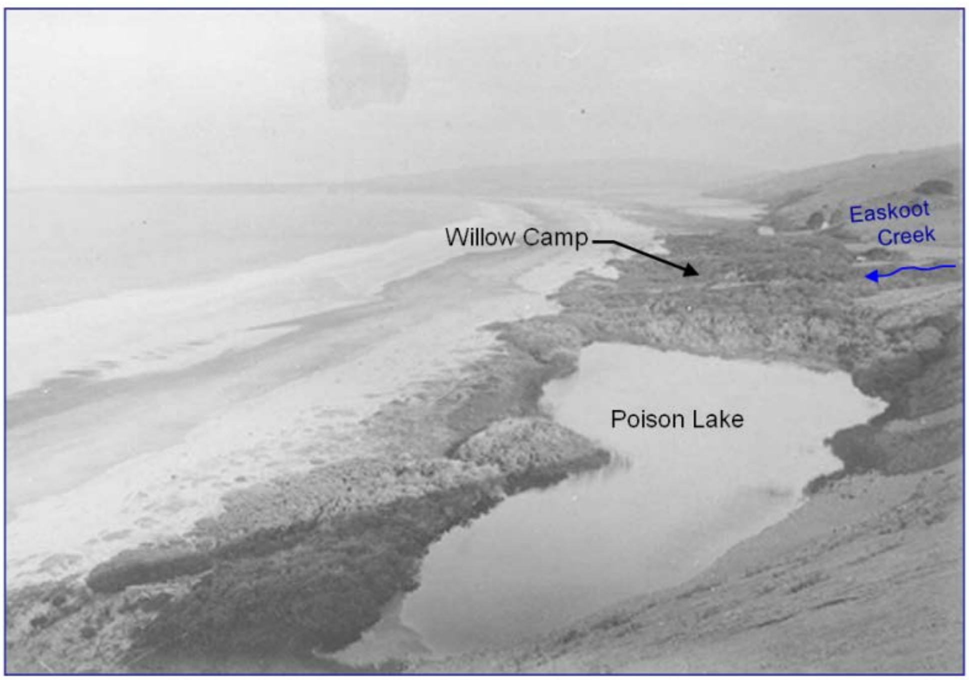


Figure 2-3 View to northwest of Stinson Beach c. 1904.

of sediment from Easkoot Creek to maintain the channel capacity after large storm events (e.g. winter 1982/83; Figure 2-4) was the historic response to these episodic flood events. In the absence of channel maintenance, Easkoot Creek would be expected to shift position periodically in response to decadal storms generating high runoff, sediment transport, and sedimentation. Typical annual sediment deposition does not significantly affect channel conveyance capacity. Expressed as an annual average, sedimentation rates are on the order of 125 to 160 cubic yards.

Figure 2-4 In-stream dredging of Easkoot Creek, c. 1982.

Habitat for endangered salmonids (steelhead trout and coho salmon) is also affected by sedimentation. A stream restoration project in 2004 on Golden Gate National Recreation Area (GGNRA) property on the lower fan downstream of Calle del Mar was affected by about two feet of sedimentation (Figure 2-5) resulting from the floods of December 2005. Habitat enhancement designed to create stable pools in this reach should only have been expected to provide desired habitat temporarily; based on the analysis of sedimentation processes and rates, sedimentation and channel aggradation in lower Easkoot Creek appears inevitable.

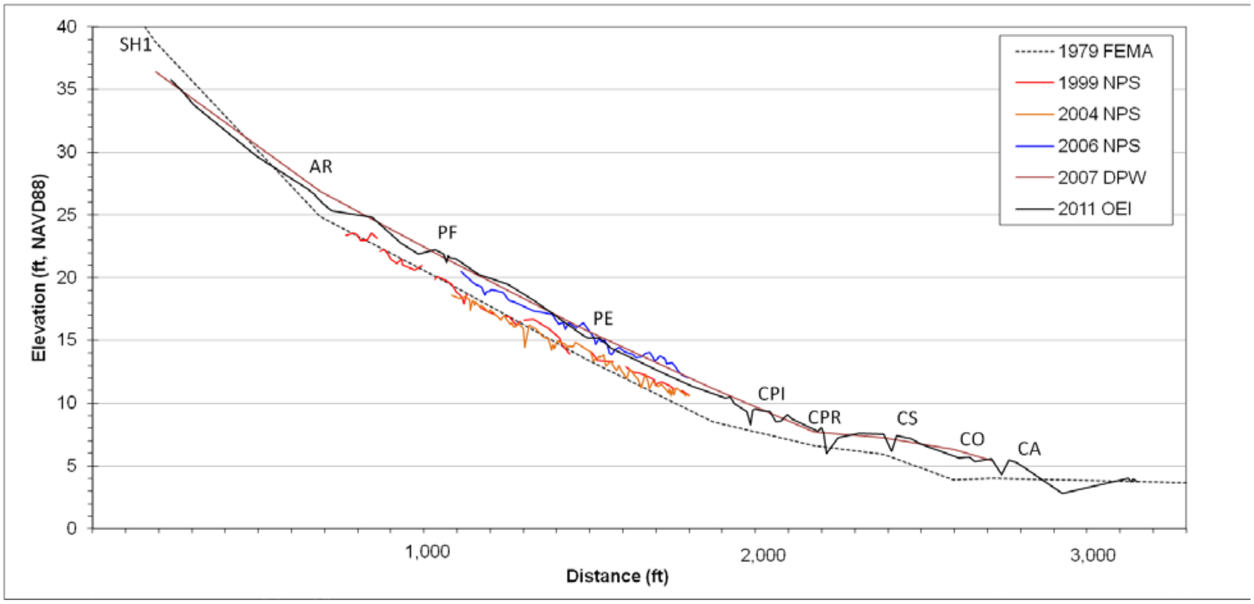


Figure 2-5 Longitudinal channel profile of Easkoot Creek.

Profile shows the channel from State Highway 1 (SH1) to Calle del Arroyo (CA); comparison between 2004 and 2006 shows the impact of the December 2005 event. AR (Arenal Avenue), PF (Park Footbridge at Calle del Mar), PE (Park Entrance Bridge), CPI (Calle del Pinos), CPR (Calle del Pradero), CS (Calle del Sierra), CO (Calle del Onda).

Upstream of Arenal Avenue, sediment transport capacity is sufficient to prevent long-term sedimentation and aggradation. In late summer, stream flow conditions and water quality (defined with respect to temperature and dissolved oxygen) decline in lower Easkoot Creek relative to locations higher on the fan upstream of Arenal Avenue. These spatial variations in sedimentation, stream flow, and water quality conditions suggest that efforts to restore or enhance salmonid habitat are more likely to be effective in the reach above Arenal Avenue.

Any expected improvements in flood conveyance and fish habitat derived from dredging, grading, or habitat enhancement on the lower portions of the alluvial fan are temporary, and ongoing sediment management (including periodic dredging) is expected to be unavoidable. Natural watershed processes will continue to produce sediment from erosion in the tributaries of Easkoot Creek, and declining sediment transport capacity on the alluvial fan will produce sedimentation and aggradation similar to that documented in Figure 2-6. This will force floodwater to spread onto the floodplain more frequently and degrade already marginal habitat for salmonids. Efforts to manage sediment by inducing deposition for managed removal should be considered as a means to better maintain channel conveyance and habitat in lower Easkoot Creek.

Figure 2-6 Long-term sedimentation at Calle del Sierra. Easkoot Creek c. 1960 (left) and 2011 (right).

3 FISH HABITAT

Easkoot Creek has recently supported a small population of steelhead trout (*Oncorhynchus mykiss*), and may also have historically supported a run of coho salmon *(Oncorhynchus kisutch*). Steelhead in Easkoot Creek are listed as threatened under the Endangered Species Act (ESA), and coho are listed as endangered under the ESA and the California Endangered Species Act (CESA). A proposed project that may affect these species or their habitat requires an assessment of potential impacts and permits will not likely be approved if the project is deemed to jeopardize their survival. The National Park Service (NPS) documented steelhead use of Easkoot Creek in 1999 and 2000 and evaluated the habitat conditions present at that time. The NPS also observed a few coho in Easkoot Creek in the early 2000s.

A natural barrier to upstream migration is located about 1,500 feet upstream of State Highway 1 (Figure 2-2). Juvenile steelhead have been found upstream and downstream of State Highway 1. All of the documented spawning sites are located in lower Easkoot Creek (below State Highway 1), however steelhead may spawn upstream of Highway 1 as well.

Pools providing complex cover (e.g., formed by and incorporating large woody debris) are important rearing habitats for steelhead and coho. Pools or other habitat features with complex cover to protect rearing fish are uncommon in Easkoot Creek, and thereby limit the suitable habitat for salmonids.

Sedimentation in the lower reaches of Easkoot Creek further limits and degrades habitat conditions for salmonids. Despite evidence of spawning in the lower reaches, relatively high concentrations of fine sediment (<1 mm in diameter) in the stream bed substantially reduces the quality of spawning habitat in the lower reaches. NPS's 2004 habitat restoration designed to create stable pools in the lower reach of Easkoot Creek had limited success because of subsequent sedimentation. Overbank flooding in the lower reaches, exacerbated by sedimentation, and may cause fish to become stranded on the urbanized floodplain.

Late summer stream flow and water quality (defined by temperature and dissolved oxygen) may also limit fish habitat in Easkoot Creek. Upstream of State Highway 1, water flow, water temperature, and dissolved oxygen conditions are stable and favorable for salmonids. Water quality in the lower reach is degraded, and the stream frequently goes dry in the reach between Arenal Avenue and Calle del Mar.

The proposed flood mitigation alternatives will affect fish habitat. For example, dredging the channel to increase conveyance could directly harm fish or disrupt known spawning and rearing habitat, and creating a managed overflow by routing flood waters to the ocean could strand juvenile fish or transport them to the ocean. The alternatives considered must protect individual fish during construction, and provide for suitable habitat conditions for the endangered fish to carry out all phases of their lives (i.e., spawning and rearing habitat). With care and foresight, flood control measures can be designed and constructed to protect, and possibly enhance, habitat conditions for salmonids in Easkoot Creek.

4 ALTERNATIVES SUMMARY

Based in part on the results of component analyses contributing to the overall study (provided in the Appendix), as well as previously identified flood mitigation concepts (as described in Section 1.1), a suite of alternatives were identified and evaluated in terms of their flood control benefits, preliminary design constraints, estimated construction costs, likely permitting issues, operation and maintenance requirements and estimated costs, sustainability, and overall feasibility. Alternatives are described in detail in subsequent sections of this report. Cost estimates are extremely preliminary and in some cases are limited due to lack of information. A rigorous cost analysis and peer review is recommended for any suite of alternatives identified as preferred through the stakeholder process.

In keeping with the philosophy of the community-based decision making process of the Marin County Watershed Program, the list of alternatives has been developed with significant input from the community. The alternatives selected for evaluation were drawn from prior studies and from meetings with the TWG, and were presented at a public meeting in Stinson Beach in April 2012. Rather than identifying a 'preferred' alternative, the benefits and constraints of each alternative have been assessed and summarized to assist decision-making regarding future flood mitigation activities. Nevertheless, when the objective results of flood analyses indicated that a particular alternative did not substantially reduce flood impacts or that there were significant constraints bringing the feasibility of implementation into question, this was noted and in some cases the effort to develop and evaluate additional details was curtailed. Such considerations affected Alternative 3-Vegetation Management, Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot and Alternative 7-Causeway. In addition, three previously identified potential flood mitigation alternatives were considered infeasible at the beginning of the alternatives analysis and were not included in the hydraulic analyses; these are described in Section 4.11. Whichever course of action is chosen, consideration must be given to the natural watershed processes that define the character of Easkoot Creek in Stinson Beach as well as the effects on habitat for endangered steelhead trout and coho salmon.

The suite of alternatives considered is summarized below; each is identified by name and alternative number. More detailed descriptions of Alternatives 1–10 are provided in the following sections, beginning at Section 8.

4.1 Alternative 1-No Action

This alternative is designed to represent the No Action (i.e. 'do nothing') alternative. To represent this alternative, hypothetical future sedimentation is assumed to further reduce the conveyance capacity of Easkoot Creek. Additional sediment deposition in Easkoot Creek is presumed to occur more or less uniformly from Arenal Avenue to Calle del Arroyo. The hypothetical sedimentation totals about 1,630 cubic yards with an average aggradation depth of 1.3 feet. This represents slightly more sediment than is estimated to have accumulated during the December 2005 flood and as such is a reasonable estimate of the future deposition that may be expected to occur over the next decade or so if no actions are taken.

4.2 Alternative 2-Bridge Improvements

This alternative considers modifications to or replacement of existing bridges over Easkoot Creek. Hydraulic modeling assumed the replacement of nine of the twelve bridges in the study area that were shown to significantly restrict flow in order to demonstrate potential flood mitigation benefits. Many options short of replacement of nine bridges are possible, however, and the alternative prioritizes the bridges in terms of their expected flood mitigation benefits and discusses the design constraints, permitting issues, and costs associated with modifying or replacing each of the various types of bridges in the study area.

4.3 Alternative 3-Vegetation Management

This alternative investigates the potential for flood mitigation resulting from reducing roughness on the channel banks through a program of vegetation management. A hypothetical reduction in bank roughness of 25% was assumed for all reaches where existing vegetation contributes to elevated roughness values.

4.4 Alternative 4-Channel Dredge and Sediment Management

This alternative consists of removing 3,100 cubic yards of sediment from a 2,300 foot reach of Easkoot Creek between Arenal Avenue and Calle del Arroyo. The average excavation depth was 2.4 feet, and the dredging plan was based on restoring the longitudinal profile of the channel as indicated by a survey from 1979. In combination with the dredging, sediment removal structures are proposed at two locations upstream of State Highway 1 to reduce future sedimentation in lower Easkoot Creek and maximize the effectiveness of dredging over a longer period of time. Habitat enhancement in the dredged reach is proposed, and potential habitat enhancement in two reaches upstream of Arenal Avenue is identified to provide for potential mitigation of potential impacts of dredging if necessary.

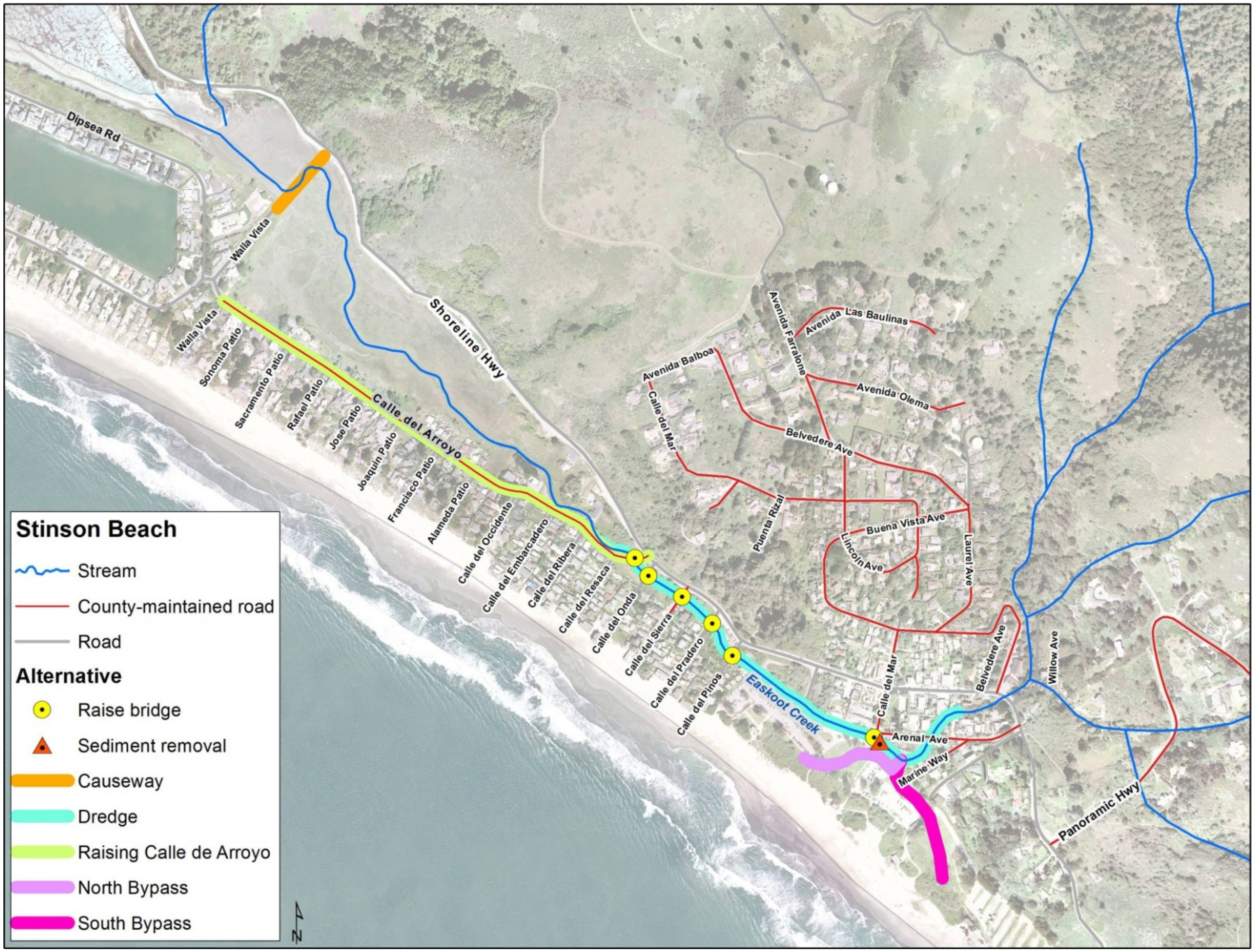


Figure 4-1 Overview of proposed flood mitigation.

4.5 Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot

This alternative involves the construction of a bypass channel to divert a portion of the discharge of Easkoot Creek away from flood-prone lower reaches during high flow conditions. The proposed diversion point is located on the left bank of the channel opposite the Parkside Café, and the diverted water flows through a 50-ft wide by 3-ft deep trapezoidal bypass channel, discharging to a detention basin located in the vicinity of the north GGNRA parking lot. Diversion would begin when flows exceeded approximately 40 cubic feet per second (cfs) meaning that the bypass would be active approximately one to four times a year. Diverted floodwater would drain to the ocean.

4.6 Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot

This alternative is similar to Alternative 5 except that it would also include the restoration of pond and wetland habitat in the vicinity of historical Poison Lake. High flows would be diverted to the restoration area located in the vicinity of the south GGNRA picnic area. The proposed restoration area covers approximately 2.4 acres and would create a range of habitat conditions ranging from a seasonal wetland to a perennial pond with depths on the order of two to four feet. Diverted floodwaters would drain to the ocean via an outlet from the restored Poison Lake.

4.7 Alternative 7-Causeway

This alternative involves the construction of a causeway over Bolinas Lagoon to connect State Highway 1 with Seadrift Road along the alignment of what is currently a gravel road named Walla Vista Road. The primary purpose of this alternative would be to improve access to the Seadrift community which relies on Calle del Arroyo as the only means of vehicular access; this route currently becomes submerged during moderate floods. The optional construction of a tide gate and pump station as part of the causeway design was also investigated in order to evaluate the potential flood mitigation benefits of these structures.

4.8 Alternative 8-Raising Calle del Arroyo

This alternative involves elevating the entire length of Calle del Arroyo between State Highway 1 and Seadrift Road, a distance of approximately 2,840 feet. The primary purpose of this alternative would be to improve access to the Lower Calles, Patios, and Seadrift community which all rely on Calle del Arroyo as the only means of vehicular access. Drainage beneath the roadway is also investigated to avoid exacerbating flooding by elevating the roadway and potentially reduce flooding impacts by arresting the downstream progress of floodplain flows.

4.9 Alternative 9-Combined Dredge, Wetland Enhancement, and Bypass

This alternative combines the features of Alternative 4-Dredge and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot, and represents a highly effective flood mitigation alternative with respect to reducing the extent and depth of flooding.

4.10 Alternative 10-Structure Elevation

This alternative involves elevating buildings so that the ground floor is situated above flood elevation rather than attempting to control the extent and depth of flooding by managing the channel, sediment and floodplain conditions.

4.11 Infeasible Alternatives

Three mitigation alternatives were considered that had significant constraints that were identified early in the study. For these alternatives, detailed evaluations were not developed owing to the low likelihood of identifying a feasible approach to the alternative that could accomplish substantial flood mitigation. A brief description of each of these infeasible alternatives is presented.

Direct Ocean Bypass. Construction of a bypass channel carrying flood flow directly to the ocean was suggested as early as 1971 by Marin County Department of Public Works, as well as in the 1984 study by Spangle and Associates. This alternative was also evaluated more recently by Michael Love & Associates (MLA 2009). The latter analysis acknowledged the high likelihood that steelhead trout would be entrained in flows routed through such a bypass channel and that a common engineering approach to this problem is installing a fish screen. The MLA report concluded that development of an effective fish screen for such a bypass presents significant difficulties owing to the required size of the screen and the high debris and sediment load in Easkoot Creek that render this approach impractical. The design flows considered in the MLA report relied on pre-existing estimates of peak flows from other studies; the present analysis indicates the peak flood discharges are substantially lower than assumed by MLA. The difficulties of design and maintenance of an effective fish screen would remain even if lower rates of flow were considered. Consequently, this alternative was not considered further. A direct bypass (without a fish screen) was considered infeasible because it would create a level of risk to endangered fish species that would be insurmountable with respect to project permitting. Two alternative bypass designs (Alternatives 5 & 6) were considered in greater detail; mitigating risks to endangered fish caused by entrainment in the bypass was among the principal design criteria for these bypass scenarios.

Increased Floodplain and Off-channel Habitat. A potential alternative involving creation of flood detention storage and off-channel habitat (e.g. side channels or sloughs) in available floodplain areas was rejected from detailed consideration because of insufficient potential for generating significant detention storage. Total runoff from the Easkoot Creek watershed during a 24-hr period representing the December 2005 flood event was about 310 acre-feet (an acre-foot is the volume of water that would cover an acre of land to a depth of one foot, equivalent to 43,560 cubic feet or about 326,000 gallons). While there is some potential to increase floodplain storage by dramatically re-configuring Easkoot Creek along Shoreline Highway and in GGNRA, the amount of land potentially available is realistically not more than several acres. Further, because of high water table in this area, particularly during winter storm events, the available storage during flood periods would be limited. By excavating extensively in the floodplain to create ponds and sloughs, flood water storage capacity of as much of 30 acre-feet might be possible, but this volume of storage would not be sufficient to provide significant flood mitigation potential.

Infiltration and Storage of Rainfall. Management of urban runoff by increasing local infiltration of rainfall and runoff was rejected from detailed evaluation as a flood mitigation alternative owing to insufficient potential to mitigate flooding. Most of the Easkoot Creek watershed is undeveloped forest and grassland in Mt. Tamalpais State Park and the GGNRA; less than twenty percent of the watershed drainage area has been developed. Consequently, the vast majority of storm runoff reaching Lower Easkoot Creek (over eighty percent) would not be affected by a program to manage urban runoff. In addition, runoff from many areas basins with significant urban land use (from which runoff could conceivably be managed) enter Easkoot Creek downstream of locations where substantial flooding is typically initiated. These facts strongly suggest that significant flood potential would remain even if runoff from the urbanized portion of the watershed could be completely infiltrated or retained. Finally, the potential for infiltration of runoff at the parcel scale is limited for purposes of flood mitigation because of the rainfall-runoff processes that cause flooding. Runoff rates increase significantly during intense, long-duration rainstorms because the soil becomes increasingly saturated and infiltration rates of rainfall to the soil are reduced. Thus the potential for infiltrating rainfall diminishes at the same time when it would need to be maximized in order to have any impact on runoff that could mitigate flood flows. During the December 2005 storm event, total runoff during the 24 hour period of peak rainfall and flooding from the urbanized sub-basins was modeled to be about 22 million gallons (about 68 acre-feet). Rainfall infiltration may be a beneficial and desirable strategy for managing the quality of urban runoff water; however, it is not a strategy that could be expected to provide significant flood mitigation.

5 METHODOLOGY FOR ALTERNATIVES ANALYSIS

Nine alternatives were evaluated with the hydraulic models for the historical December 2005 flood; Alternative 10-Structure Elevation was evaluated relative to existing condition hydraulic models described in the Appendix "Hydraulic Model and Flood Hazard Evaluation". The December 2005 event was selected as the primary evaluation event because it represents a recent historical flood, it was large enough to cause significant flood damage in the watershed, and it is small enough that the alternatives may be expected to provide significant mitigating effects. Four of the twelve alternatives were also evaluated for a 100-yr flood[4]. These alternatives were selected because they proved to have the greatest potential to mitigate flooding.

A tidal boundary condition equivalent to Mean Higher High Water (MHHW) was used for this analysis. It is important to note that while this analysis does include the effects of tidal forcing on riverine flooding it represents only riverine flood hazards. Given that the lower portions of Easkoot Creek are subject to flooding from extreme tides and coastal storm surge in addition to riverine flooding, final design of

[4] The term "100-year flood" refers to a probability that a storm of a certain size would occur in a given year. The 100-year storm has a 1% or 1 in 100 chance of occurring in a year, as opposed to a storm that occurs only once every 100 years.

many of the alternatives affecting the lower reaches of Easkoot Creek requires completion of a coastal flood hazard study which is beyond the scope of this analysis.

In order to evaluate the effects of sea level rise due to global climate change we also analyzed the December 2005 flood with a tidal boundary condition of Mean Higher High Water (MHHW) plus 18.2 inches of sea level rise. This is the value recommended for use in Marin County riverine flood studies by

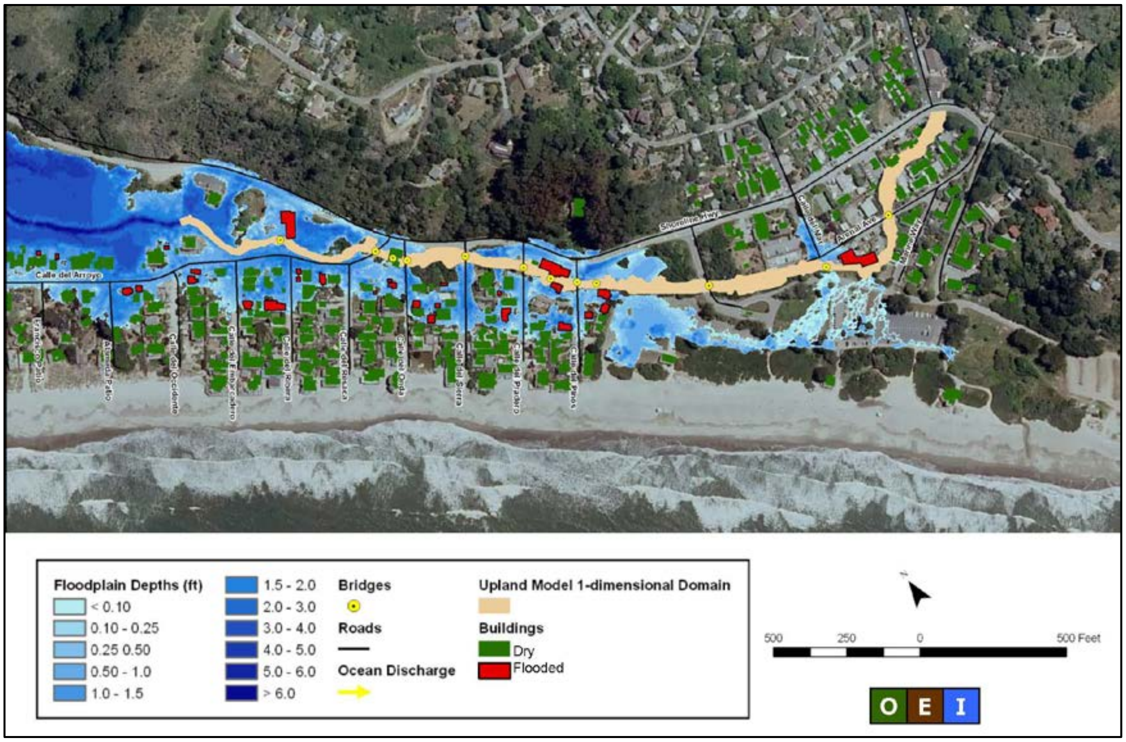


Figure 5-1 December 2005 flood as depicted from the computer model.

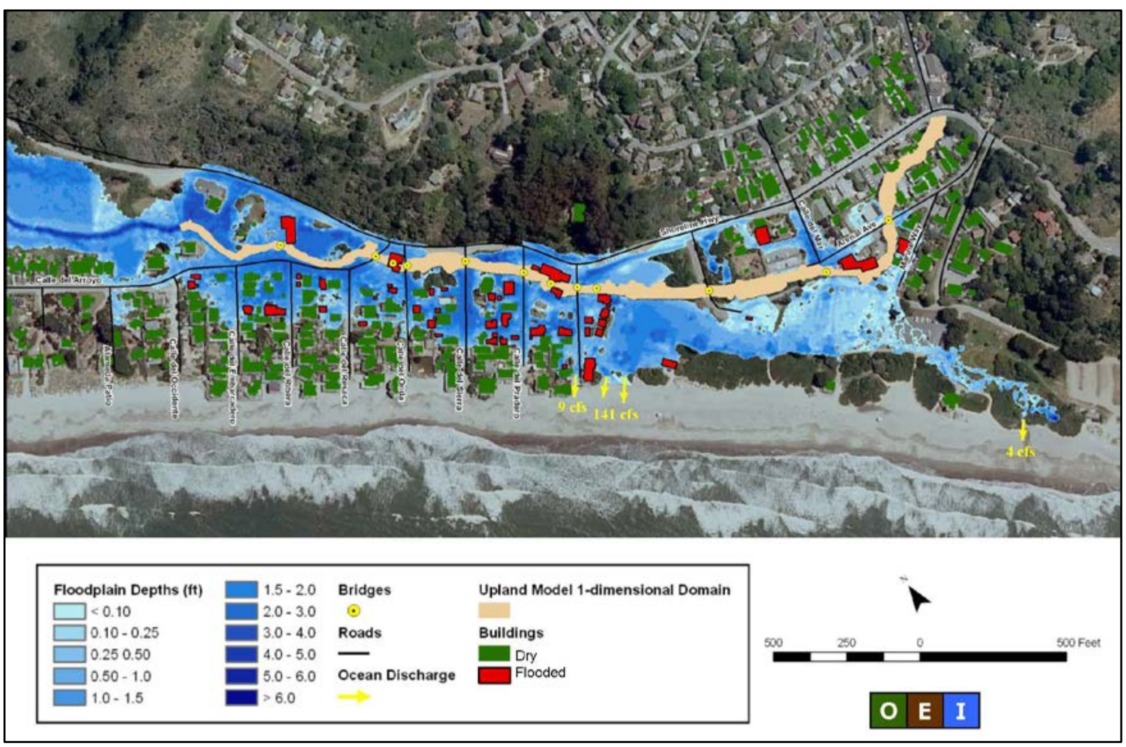


Figure 5-2 The 100-year flood in Stinson Beach as predicted from the computer model.

the August 2012 Technical Memorandum prepared by Marin County staff entitled *Recommended Sea Level Rise Modeling Methodology and Values to be used for Riverine and CIP Flood Studies*. It represents a 2050 sea level rise estimate and is based on a statistical analysis of the range of predicted values given in the 2012 National Research Council's report *Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future*.

This sea level rise condition (MHHW plus 18.2 inches) was evaluated for the one alternative that proved the most promising in terms of potential to reduce flood hazards. The sea level rise analyses performed for this alternative and for existing conditions provided a means of understanding the potential impacts of sea level rise on the effectiveness of the alternatives in general. Sea level rise may be expected to have a larger effect on flooding due to coastal flood processes; hence a full understanding of sea level rise impacts requires completion of a coastal flood hazard study.

Surveyed finished floor elevations (FFE) were available for a significant proportion of the flood-prone areas in the watershed. For these buildings it was possible to directly compare simulated water surface elevations with FFEs to more accurately determine which buildings would become flooded for a given flood event and alternative. For those buildings lacking FFE data, it was assumed that the building was flooded if floodwaters adjacent to any part of the building had depths of 0.5 feet or greater. It is important to note that the tabulation of the number of flooded buildings presented in Section 3 includes auxiliary buildings such as garages and sheds in addition to residential and commercial buildings.

In addition to tabulating the number of flooded buildings for each event and alternative, flood mitigation effects are presented in terms of the change in peak water levels within the active channel of Easkoot Creek for various reaches. These changes to flood extent and floodplain depths are depicted through a series of maps accompanying each alternative. The following notes may be helpful to assist in interpreting these flood maps:

- areas where flood waters no longer inundate homes are shown in light green
- areas where flood depths decrease are shown in light blue
- areas where flood depths do not change significantly are shown in dark blue
- areas where flood depths increase are shown in red

Alternatives were also evaluated with respect to design and construction options and constraints, estimated implementation and maintenance costs, and permitting considerations including a preliminary assessment of impacts to endangered fish (steelhead trout and coho salmon). These evaluations are in the form of narratives that attempt to identify and describe salient aspects of the different potential approaches to flood mitigation.

While we attempted to be thorough, the wide range of options and the uncertainty associated with the permitting process made it difficult to present salient considerations at consistent levels of breadth and depth. Nevertheless, these narratives provide a relatively detailed starting point for further analysis and preliminary design work for the most promising alternatives.

All alternative designs presented here should be considered conceptual (30%) designs and are provided for planning purposes only. All cost estimates presented here should be considered preliminary planning level estimates. More refined cost estimates for a given alternative can be developed following completion of more detailed design studies.

6 SUMMARY OF RESULTS

Flood mitigation results have been summarized in several ways including tables showing the change in peak water levels within Easkoot Creek (Tables 6-1 and 6-2), tables showing the number of buildings for which flood potential is significantly reduced (Tables 6-3 and 6-4), and maps indicating the simulated changes in flood extent and depths on the floodplain provided for each alternative in the following sections of this report. The water level change results have been summarized for three reaches where overbank flows are most prevalent, the reach adjacent to the Parkside Café, the reach between Calle del Pinos and Calle del Arroyo (Upper Calles), and the reach between Calle del Arroyo and Calle del Occidente (Lower Calles). Following is a summary of the effectiveness of alternatives with respect to flood mitigation; further discussion of the results is presented in Section 7-Key Findings.

<u>Alternative 9-Combined Dredge, Wetland Enhancement, and Bypass</u> provides the greatest flood mitigation benefits. Flooding is dramatically reduced for the December 2005 event with the exception of a small stretch of Calle del Arroyo near Calle del Ribera. All twenty four buildings affected by the

December 2005 flood have significantly reduced hazard (Table 6-3). For the 100-yr event, flooding above Calle del Pinos is significantly reduced. Only minimal reductions in flood extent occur within the Calles; however floodplain depths are reduced substantially throughout much of this area. Approximately twenty-three of fifty-nine buildings (39%) have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under 100-yr flood conditions (Table 6-4).

This alternative was also evaluated for the December 2005 event under a 2050 sea level rise condition. These results indicate that the mitigating effects of the alternative would remain above the Calle del Arroyo bridge but be somewhat diminished throughout the Lower Calles. Farther downstream the higher tidal condition results in overtopping of Calle del Arroyo in several reaches which increases the flood extent to a similar level as was simulated for existing conditions with sea level rise.

<u>Alternative 4-Channel Dredge Alternative and Sediment Management</u> significantly reduces flooding above Calle del Onda for the December 2005 event. Only limited changes in flood extent and floodplain depths occur in the Lower Calles. Approximately eighteen of twenty-four buildings (75%) experience significantly reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions (Table 6-3). Flood extents are significantly reduced for the 100-year flood and significant reductions in floodplain depths occur throughout the Upper Calles. In the Lower Calles only minimal changes occur. Approximately seven of fifty-nine buildings (12%) have significantly reduced flood hazard (decrease in flood extent and floodplain depths) for the 100-yr flood (Table 6-4).

<u>Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot</u> and <u>Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot</u> provide a similar level of flood protection. This alternative, however, has significant disadvantages regarding effects on fish, GGNRA facilities, and uncertainty regarding the ultimate effects on flood hazards. Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot has substantially greater potential to compensate for impacts on fish and GGNRA facilities, and floodwaters are routed to an area that has little potential to cause significant additional flood hazard. Flooding is significantly reduced above the Calles and flood extents and floodplain depths are reduced significantly throughout both the Upper and Lower Calles for the December 2005 flood. Both alternatives provide significantly reduced flood hazard (decrease in flood

extent and floodplain depths) for eleven of twenty-four buildings (46%, Table 6-3). Only minor decreases in flood extent are achieved during the 100-yr event, however floodplain depths decrease significantly throughout the study area and thirteen of fifty-nine buildings (22%) have significantly reduced flood hazard (decrease in flood extent and floodplain depths) in the 100-yr event (Table 6-4).

Alternative 2-Bridge Improvements appears to be relatively effective; however, this tentative conclusion results from modeling removal of all the bridges that constrain flood flows. Evaluating the effects of modifying individual bridges or groups of bridges was beyond the scope of this analysis; further hydraulic model runs should be conducted to evaluate these effects in a subsequent stage of project planning. Bridge modification as modeled eliminates flooding above the Calles with the exception of small overbank flows on the right bank adjacent to the Parkside Café during the December 2005 flood. Minor decreases in flood extent and significant decreases in floodplain depths occur throughout the Upper Calles, however only minor changes occur in the Lower Calles. Approximately eleven of twenty-four buildings (46%) have significant reductions in flood hazard (decrease in flood extent and floodplain depths) (Table 6-3). Only minor decreases in flood extent are achieved during the 100-yr event, however floodplain depths decrease significantly throughout the study area and seven of fifty-nine buildings (12%) have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under 100-yr flood conditions (Table 6-4).

Alternative 3-Vegetation Management does not substantially affect the extent or depth of floodplain flow. Alternative 7-Causeway and Alternative 8-Raising Calle del Arroyo do not significantly reduce peak water levels, flood extents, or floodplain depths (Tables 6-1 and 6-3); however, they would provide improved vehicular access to the lower Calles and Seadrift during flood periods. Alternative 1-No Action increases flood extents and floodplain depths significantly throughout the upper reaches of the creek above Calle del Onda. This results in significant increases in flood hazards to eight buildings, an increase of 33% in structures with significant flood hazard.

Additional considerations regarding the overall feasibility of flood mitigation alternatives relative to preliminary design constraints, estimated construction costs, likely permitting issues (including fish habitat impacts), operation and maintenance requirements and estimated costs, and sustainability are discussed in detail for each alternative in the following sections. These considerations are also summarized in Table 6-5.

Table 6-1 Average change in peak water levels for the December 2005 flood event for the various alternatives.

Alternative	Average Change in Water Level (feet)		
	Parkside Café	Upper Calles	Lower Calles
1 No Action	+0.5	-0.1	-0.2
2 Bridge Improvements	-0.3	-0.2	0.0
3 Vegetation Management	0.0	0.0	-0.1
4 Channel Dredge and Sediment Management	-2.6	-1.1	0.0
5 Wetland Creation and Bypass to the National Park Service's North Parking Lot	-0.6	-0.4	-0.4
6 Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot	-0.6	-0.4	-0.6
7 Causeway	0.0	0.0	-0.1
8 Calle del Arroyo	0.0	0.0	0.0
9 Combined Dredge, Wetland Enhancement, and Bypass	-3.6	-2.2	-0.8

Table 6-2 Average change in peak water levels for the 100-yr flood event for the various alternatives.

Missing values indicate that the alternative was not evaluated for this event.

Alternative	Average Change in Water Level (feet)		
	Parkside Café	Upper Calles	Lower Calles
1 No Action	-	-	-
2 Bridge Improvements	-0.1	-0.3	0.0
3 Vegetation Management	-	-	-
4 Channel Dredge and Sediment Management	-1.1	-0.2	-0.1
5 Wetland Creation and Bypass to the National Park Service's North Parking Lot	-	-	-
6 Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot	-0.5	-0.4	-0.3
7 Causeway	-	-	-
8 Calle del Arroyo	-	-	-
9 Combined Dredge, Wetland Enhancement, and Bypass	-3.4	-1.4	-0.1

Table 6-3 Number of buildings flooded or no longer flooded in relation to the December 2005 floodplain under the various alternatives.

Alternative	# of Flooded Buildings	# of Buildings No Longer Flooded
Existing Conditions	24	-
1 No Action	32	-8
2 Bridge Improvements	13	11
3 Vegetation Management	24	0
4 Channel Dredge and Sediment Management	6	18
5 Wetland Creation and Bypass to the National Park Service's North Parking Lot	13	11
6 Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot	13	11
7 Causeway	23	1
8 Calle del Arroyo	22	2
9 Combined Dredge, Wetland Enhancement, and Bypass	0	24

Table 6-4 Number of buildings flooded or not flooded in relation to the 100-yr floodplain under the various alternatives.

Missing values indicate that the alternative was not evaluated for this event.

Alternative	# of Flooded Buildings	# of Buildings No Longer Flooded
Existing Conditions	59	0
1 No Action	-	-
2 Bridge Improvements	52	7
3 Vegetation Management	-	-
4 Channel Dredge and Sediment Management	52	7
5 Wetland Creation and Bypass to the National Park Service's North Parking Lot	-	-
6 Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot	46	13
7 Causeway	-	-
8 Calle del Arroyo	-	-
9 Combined Dredge, Wetland Enhancement, and Bypass	36	23

Table 6-5 Comparison of alternatives.

Alternative	Description	Flood Benefit (For the modeled December 2005 flood)[5]	Cost to Construct[6]	Creek channel capacity (related to sediment and flow)	Fisheries concerns (impacts permit approvals)
1-No Action	No action assumes increased sediment accumulation in the creek.	Increased flooding to 8 homes for a total of 32 flooded homes.	None	Unmitigated sedimentation leads to significantly reduced channel capacity in < 10 years	Increased risk of stranding on floodplain; in-stream habitat degraded.
2-Bridge Improvements	Modifications to or replacement of 12 existing bridges (5 public and 7 private) over Easkoot Creek.	11 homes no longer flood.	Between $4-5 million. Operation and maintenance is minimal for the lifespan of the bridge.	Modest local change at modified and unmodified bridges possible	Somewhat reduced risk of stranding on floodplain; minimal change to in-stream habitat
3-Vegetation Management	Reduce 25% more vegetation along the creek channel except where structures exist.	Negligible benefit to flooding.	Between $5,000-7,000 per year.	Minimal change expected	Minimal change expected
4-Channel Dredge & Sediment Management (over entire channel length)	Remove 3,100 cubic yards of sediment from a 2,300-foot reach of Easkoot Creek between Arenal and Calle del Arroyo.	18 homes no longer flood.	Between $1.5-2.5 million. Anticipated to be needed once every ten years. $40-50,000 for annual operation and maintenance.	Reduced rate of sedimentation and reduced impact on conveyance due to near term future sedimentation; improvement temporary unless maintained by on-going sediment management including potential future dredging	Much reduced risk of stranding on floodplain; disturbed habitat may be improved by enhancement actions and implementation methods; habitat improvement in lower Easkoot temporary. Upstream sedimentation facilities could improve habitat.
5-Wetland Creation and Bypass to the National Park Service's North Parking Lot	Construct a bypass channel to the Park Service's north parking lot to divert a portion of the discharge of Easkoot Creek away from flood-prone lower reaches during high flow conditions.	11 homes no longer flood.	Between $1-2 million. Unknown costs for annual operation and maintenance.	Some redistribution of sedimentation expected-decreased potential near Arenal Avenue and increased potential near Calle del Mar; new sedimentation possible in bypass channel	Risks to fish lost to bypass are relatively high. An alternative path to the ocean is provided for fish. Reduced flooding lowers probability of stranding for other fish

[5] The computer model of the December 31, 2005 storm shows 24 flooded homes. Finished floor elevations surveyed by Flood Control staff provided elevations of the living area of homes. This column shows the number of homes that would flood after an alternative is implemented using the same December 31, 2005 storm. For homes without survey data, flooding was assumed when floodwaters adjacent to a home reached depths of 0.5-feet or greater. (Garages, sheds, yards, and utilities may still be flooded.)

[6] Cost estimates are extremely preliminary and do not include real estate acquisitions or easements, mitigation, or full permit preparation. The California Environmental Quality Act (CEQA, National Environmental Protection Act or NEPA for projects involving Federal jurisdiction) requires an evaluation of impacts, positive and negative, short- and long-term for projects. In addition to disclosure of all known impacts, this process serves to inform and involve the public in decision-making. An Environmental Impact Report (EIR, or Environmental Impact Statement at the Federal-level) is used to assess a project and its alternatives, mitigation to address impacts, and then identify the top alternative based on the evaluation. CEQA/NEPA has not been factored into the cost of any alternative. For a complex project, an EIR/EIS can take several years and several hundred thousand dollars to complete.

Alternative	Description	Flood Benefit (For the modeled December 2005 flood)[5]	Cost to Construct[6]	Creek channel capacity (related to sediment and flow)	Fisheries concerns (impacts permit approvals)
6- Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot	Similar to Alternative 5 except that it would also include the restoration of pond and wetland habitat in the vicinity of historical Poison Lake.	11 homes no longer flood.	Between $1-2 million. Substantial annual operation and maintenance for new flow and sediment management activities.	Some redistribution of sedimentation expected-decreased potential near Arenal Avenue and increased potential near Calle del Mar; new sedimentation possible in bypass channel and restored Poison Lake.	Risks to fish lost to bypass are relatively low, or beneficial due to potential high quality rearing habitat in restored Poison Lake; reduced flooding lowers probability of stranding of fish not entrained in bypass.
7-Causeway	Construction of causeway over Bolinas Lagoon to connect State Highway 1 with Seadrift Road along the alignment of Walla Vista Road.	N/A	Between $3-4 million to construct. Unknown costs for annual operation and maintenance.	No effect on sedimentation expected.	No effects expected.
8-Raising Calle del Arroyo	Elevate entire length of Calle del Arroyo between State Highway 1 and Seadrift Road.	N/A	Between $1-2 million to construct. Unknown costs for annual operation and maintenance.	No major effects expected.	Some potential reduction in floodplain stranding.
9-Combined Dredge, Wetland Enhancement, and Bypass	Combines Alternatives 4 and 6.	24 homes no longer flood.	Between $3.5-4.5 million. Substantial annual operation and maintenance for new flow and sediment management activities.	See above	See above
10-Structure Elevation	Elevate buildings so the ground floor is situated above flood elevation.	All homes are raised above the 100-year level of mapped flooding.	Between $50,000-100,000 per home. For all 24 homes the cost would total between $1.5-2.5 million. Unknown maintenance costs.	No major effects expected.	No effects expected.

7 KEY FINDINGS

Overall the most effective alternative with respect to flood mitigation appears to be Alternative 9-Combined Dredge, Wetland Enhancement, and Bypass. Alternative 2-Bridge Improvements, Alternative 4-Dredge, Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot, and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot are less effective than Alternative 9, and about equally effective compared with each other. Alternative 4 removes more buildings from the December 2005 floodplain but less than the others from the 100-yr floodplain, and Alternatives 5 and 6 result in improvements that extend downstream to the Lower Calles reach whereas the Bridge and Dredge alternatives do not. The remaining alternatives result in only minor improvements, and the No Action Alternative is the only alternative that exacerbates flood hazards.

Although they do not result in significant reductions in peak water levels or the number of flooded buildings, both Alternative 7-Causeway and Alternative 8-Raising Calle del Arroyo reduce flood hazards by improving access for residents of the lower watershed during flood conditions. Of these two options, Alternative 8 would improve access to the lower Calles, Patios, and Seadrift areas whereas Alternative 7 would only improve access to Seadrift when Calle del Arroyo is flooded.

While similarly effective from a flood control stand-point as modeled, Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot would provide significantly more fisheries benefits than Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot and would likely face fewer design and permitting constraints and have fewer impacts on existing GGNRA facilities and parking. In addition, the Alternative 5 routes flood water adjacent to the upper Calles neighborhood, which could potentially create more flooding if design capacity were to be exceeded. In contrast, the Alternative 6 routes flood waters away from residential and commercial areas and does not have the potential to create a secondary flood hazard.

After Alternative 9-Combined Dredge, Wetland Enhancement, and Bypass, Alternative 2-Bridge Improvements, Alternative 4-Channel Dredge and Sediment Management, and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot are the most effective at relieving flooding. These three alternatives represent very different flood control approaches; the Dredge alternative is effective because it increases conveyance in the creek, the Bridge Improvement alternative is effective primarily because it removes obstructions from the creek, and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot is effective because it reduces the discharge reaching the lower flood-prone reaches of the creek.

All three approaches were shown to be about equally effective; however, there are some distinct differences in terms of sustainability. Although sediment management activities can be implemented to reduce sediment inputs to the lower reaches of the creek, sedimentation is expected to be an ongoing problem at the base of the Easkoot Creek alluvial fan. In its present alignment, zones of sedimentation appear to occur at the northward bend of Easkoot Creek as it arrives at the GGNRA property and turns northwest towards the Parkside Café and Calle del Mar, and after it passes under the entrance road to the GGNRA beach parking lots. Channel capacity has been reduced dramatically by recent sedimentation, and dredging is an effective alternative to improve short-term flow capacity of the channel. It is anticipated that the increases in conveyance achieved through dredging will gradually decline over time, and that flood events with recurrence intervals of about ten years are likely to cause significant sedimentation. Provided that bridge decks are elevated sufficiently above the channel banks, much of the benefits achieved by removing these obstructions from the flow field should continue regardless of anticipated future sedimentation.

Alternatives for bypassing flood flows may be considered the most sustainable options. Bypass alternatives significantly reduce discharges in the lower flood-prone reaches which will continue to provide flood control benefits regardless of future changes that may occur in the lower system. Nevertheless, the bypass alternatives do not eliminate sedimentation issues. It is recommended that development of significant sedimentation facilities located upstream of State Highway 1 that can be routinely dredged be given serious consideration as one of the only means available to reduce long-term sedimentation and its contribution to flood hazards in lower Easkoot Creek.

Examination of the hydraulic modeling results for flooding under existing conditions reveals that as peak stream discharge and flooding increases, more water exits the channel along the left bank of the creek in the vicinity of the Parkside Cafe. When these overbank flows become large enough water begins to flow to the ocean in the vicinity of the northern GGNRA parking lot (approximately where the Alternative 5- Wetland Creation and Bypass to the National Park Service's North Parking Lot routes flow) and the wetland located in the vicinity of historical Poison Lake (where the Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot routes flow). Results for the No Action Alternative suggest that if channel capacity continues to decrease this process will be enhanced. These results suggest that the system has a natural tendency to bypass a substantial portion of flood flows away from the lower reaches of the creek as flows increase and/or channel capacity is reduced. Thus the bypass alternatives can be viewed as a way to manage this process and maintain better control over the fate of overbank flows.

Our analysis of 2050 sea level rise impacts suggests that the mitigating effects of the alternatives will remain approximately the same above Calle del Arroyo, but will be diminished below this point. Under the sea level rise condition, Calle del Arroyo becomes overtopped in several areas which results in significant increases in flooding. The only alternatives that are likely to mitigate against sea level rise impacts are Alternative 7-Causeway and Alternative 8-Raising Calle del Arroyo. It is important to note that the sea level rise analysis only considered Mean Higher High Water (MHHW) conditions and sea level rise is expected to have a larger impact on coastal flooding processes which have not been evaluated to date.

With respect to impacts on fish habitat, both the Dredge and Bypass alternatives appear to have the potential to substantially alter habitat conditions. Regulatory permits to implement these alternatives would require that potential negative impacts be avoided or mitigated. There appear to be feasible means of avoiding negative impacts to habitat as well as means of actively enhancing habitat conditions. Determining project designs and conditions that would meet regulatory requirements is difficult, particularly when multiple regulatory agencies have jurisdiction and where complex environmental conditions and impacts are involved. Further effort is required to map and plan the regulatory permitting process of selected alternatives at the outset of more detailed planning work.

8 Alternative 1-No Action

This alternative represents the "No Action" scenario in which significant additional sediment deposition is presumed further reducing channel capacity. This alternative did not consider any change to existing vegetation management practices. The amount of sediment deposition is comparable to what can occur as the result of one large flow event with a recurrence interval of approximately 10 years. Hypothetical deposition was presumed to occur from the upstream boundary of the study area at State Highway 1 through a point ~280 feet downstream of Calle del Arroyo below which the analysis of historical longitudinal profiles indicates relative stability of the channel bed. (Refer to description of Alternative 4 in Section 11 and the technical memorandum "Sediment Transport Evaluation" in the Appendix for a more comprehensive discussion of historical deposition and changes in bed elevations). The total reach length over which deposition was presumed to occur is approximately 2,800 feet. The average depth of deposition relative to the 2011 OEI profile is 1.3 feet with a maximum change of 3.0 feet (Figure 8-1). The total deposition volume represented by the alternative is approximately 2,100 cubic yards. This volume represents somewhat more sediment than is estimated to have accumulated during the December 2005 flood and as such is a reasonable estimate of the future deposition that may be expected to occur over the next decade or so if no sediment management actions are taken.

Hypothetical sedimentation was found to result in substantial increases in flood extent and floodplain depths throughout the reach extending from the Parkside Café through the upper Calles (Figure 8-1). A significant portion of the additional floodplain flow generated by the reduced conveyance through this reach inundates the GGNRA property, primarily in the vicinity of the north parking lot. Water breaches the sand dunes and overflows to the ocean at two locations: near the northwest corner of the north parking lot, and at the existing outfall of the remnant of historical Poison Lake north of the overflow parking lot (Figure 8-1). Decreases in flood extent and decreases in floodplain depths on the order of 0.1 to 0.25 feet occur downstream of Calle del Arroyo. These decreases in flooding can be attributed to increased diversion of flow through the GGNRA parking lots to the ocean, however, this also increases floodplain flow between the beach and Easkoot Creek in the upper Calles where flooding is more severe. These changes in flow and flood patterns ultimately result from reduced channel conveyance caused by hypothetical sedimentation. Peak water levels in the channel increase by as much as 1.6 feet in the upper reaches of the creek but show slight reductions on the order of 0.1 to 0.2 feet throughout the lower reaches (Table 6-1). Despite the mitigating effects of increased ocean outfalls, the overall effect is a significant increase in flood risk with an estimated eight additional buildings brought into the December 2005 floodplain raising the total from twenty-four to thirty-two (Table 6-3).

With respect to fish habitat, it is expected that additional sedimentation in lower Easkoot Creek would further degrade spawning and rearing habitat. Potential spawning habitat would be expected to be diminished by increased deposition of fine sediment as channel definition and flow confinement decreases. Surface flows during the summer base flow period would likely diminish, and the extent of dry channel would likely increase, thereby reducing the quantity and quality of rearing habitat available. Juvenile fish are also subject to moving into the floodplain, either actively (i.e., moving into the floodplain in search of foraging opportunities) or being passively carried into flooded areas by currents. Once in the floodplain, these fish are subject to being stranded, unable to return to the main channel. The potential loss of juvenile fish to the ocean during flood events is therefore proportional to the degree of flooding, and would likely increase with increasing floodplain flows and ocean outfall via the parking lots.

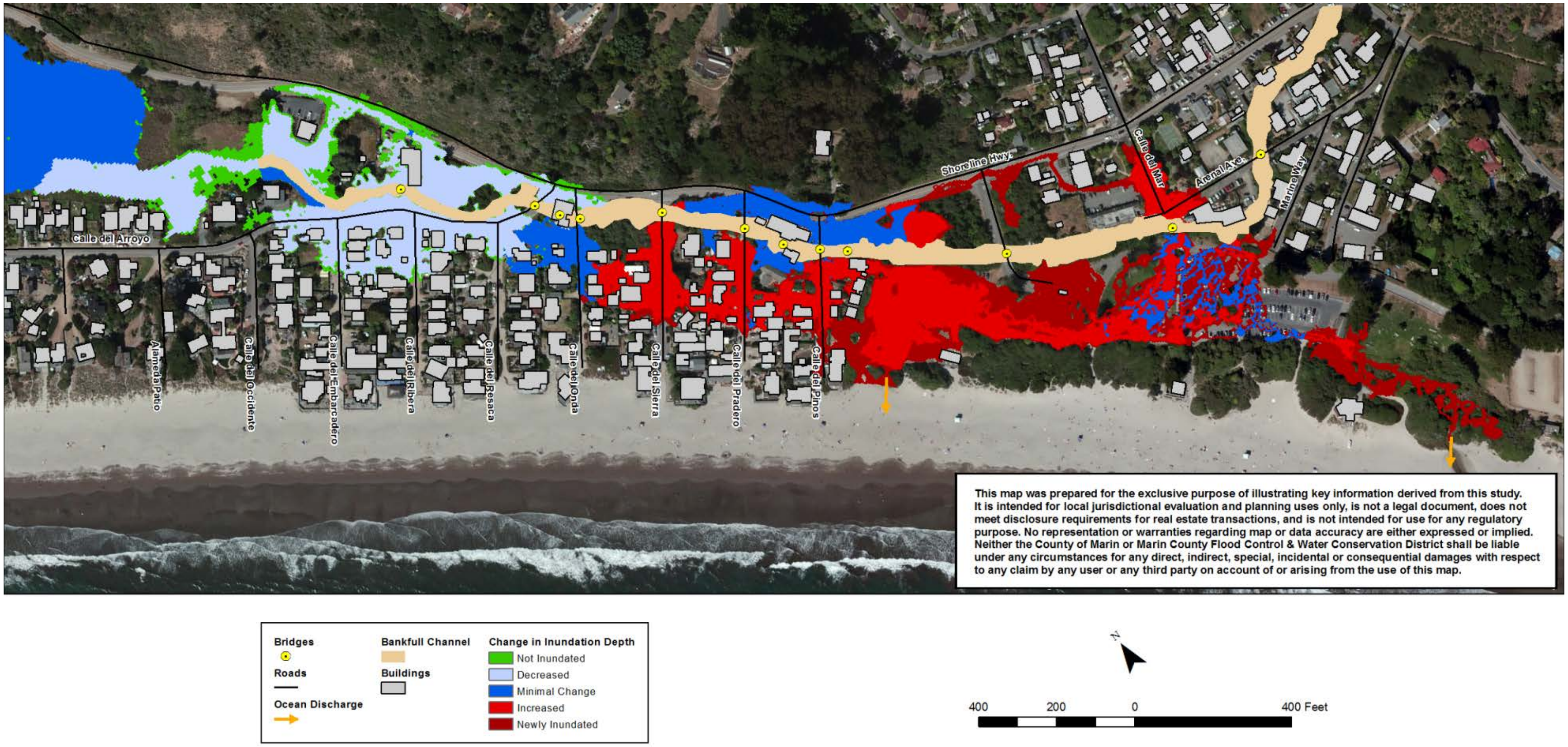


Figure 8-1 Flood extent and floodplain depths under the No Action alternative for the December 2005 flood.

9 Alternative 2-Bridge Improvements

9.1 Summary

The Easkoot Creek drainage contains twelve bridges in the area under study (Table 9-1). No two structures are exactly alike. The bridges primarily restrict flow via interaction of the water surface with the bridge decks; flow restriction due to decreases in channel width associated with bridge abutments is of only minor significance. Several of the bridges present a significant flow restriction in moderate flood events like December 2005 resulting in overbank flows occurring immediately upstream of the bridges which in turn results in downstream floodplain inundation. This effect is most pronounced at bridges 1, 3, 4, and 8 which become submerged by more than a foot during the December 2005 flood (Table 9-2). Hydraulic modeling results indicate that bridges 2, 5, 6, 10, and 12 have sufficient capacity for a December 2005 magnitude flood, but only bridge 10 has sufficient capacity for a 100-yr flood (Table 9-2).

Bridge improvements could potentially provide significant flood mitigation in some areas; this conclusion is based on a simplified hydraulic analysis in which all constraints on flow caused by <u>all</u> bridge decks are removed. Actual and model flood mitigation for <u>individual</u> bridges has not been determined; <u>it is possible that flood effects and flood mitigation benefits associated with individual bridges are not well represented by this simplified model analysis</u>. <u>Additional analysis of flood impacts/mitigation is recommended for individual bridges during future project planning and feasibility studies for bridge improvements</u>. In general, <u>it is expected that modifications to upstream bridges could cause increased flooding downstream, and that bridge improvements should proceed from downstream areas to upstream areas</u> to avoid this potential effect.

For the December 2005 event, flooding upstream of the North Lot is eliminated except for minor overbank flow near the Parkside Café (Figure 9-1). Minor decreases in flood extent occur throughout the Upper Calles; floodplain depths decrease by 0.1 to 0.5 feet in residential areas of the Upper Calles (Table 6-1). Approximately eleven of twenty-four buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions (Table 6-2). During the 100-yr flood, flood extent is slightly reduced and floodplain depths are reduced by 0.1 to 0.5 feet throughout the Upper Calles (Figure 9-2). Peak water level reductions are relatively minor overall

(Table 6-3), however significant reductions of 0.6 to 0.8 feet occur upstream of several of the bridges (Park Footbridge, Footbridge above Calle del Pinos, Calle del Pinos, Calle del Sierra, and Calle del Onda). Approximately six of fifty-nine buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under 100-yr flood conditions (Table 6-4).

Bridge impacts of flooding could be reduced by raising the existing supports for individual structures above the design high water levels (e.g. 100-yr water surface), which can be accomplished by raising bridge decks about two feet. The required incremental elevation has been determined through hydraulic modeling, and varies to an extent at individual bridges depending on channel width and depth and likelihood of future bed load deposition at the site. Bridges 1, 2, 3 and 4 are located low in the watershed within the zone of tidal influence, and as such are also subject to coastal flooding (any final design for modifications to these lower bridges should consider coastal flood hazards which were not addressed as part of this study).

Improvement of vehicle bridges is problematic; each bridge project poses unique design challenges. Most of the bridges serve dead-end streets with no alternative access unless temporary roads can cross the beach to adjacent roads, and necessary construction activity and would be disruptive to local traffic and State Highway 1. Permitting issues are substantial, but probably not a major obstacle.

Preliminary estimated cost for improvements to the bridges is significant, about $3.9 million. Cost estimates are summarized for each bridge in the table below.

Table 9-1 Summary cost estimate for bridge improvement.

Bridge Alternative - Planning-level Budget Summary	**Cost ($)**	**Percent**
Bridge 1 - Lower Footbridge	36,773	0.9
Bridge 2 - Calle del Arroyo	1,222,450	31.1
Bridge 3 - House - not evaluated	--	0.0
Bridge 4 - Calle del Onda	67,848	1.7
Bridge 5 - Calle del Sierra	614,305	15.6
Bridge 6 - Calle del Pradero	67,848	1.7
Bridge 7 - Gym Footbridge	37,273	0.9
Bridge 8 - Calle del Pinos	67,848	1.7
Bridge 9 - Footbridge ab. Calle d. Pinos	36,773	0.9
Bridge 10 - Park Entrance	--	0.0
Bridge 11 - Parkside Footbridge	41,773	1.1
Bridge 12 - Arenal Avenue	--	0.0
Subtotal Contractor Overhead	749,570	19.1
3,741,900 95.2		
Project Administration	187,100	4.8
3,929,000 100.0		

Table 9-2 Overview of the bridges in the study area listed from downstream to upstream.

Includes basic dimensions, construction materials, clearance from the bottom of the bridge deck to the channel bed, the December 2005 water surface, and the 100-yr water surface (negative clearances are shows in red and represent a submerged bridge deck), and prioritization in terms of potential to reduce flooding impacts.

ID	Name	Bridge Deck			Girder	Railing	Distance to Cross Street (ft)	Clearance (ft)		
		Width (ft)	Length (ft)	Material	Material			To Bed	To Dec. 2005 Water Surface	To 100-yr Water Surface
1	Footbridge below Calle del Arroyo	4	25	wood	wood	wood	-	3.0	-1.1	-1.8
2	Calle del Arroyo	30	17	paved	mono box	steel	36	3.8	0.4	-0.2
3	House Bridge	30	20	wood	wood	-	-	2.6	-1.0	-1.7
4	Calle del Onda	12	24	wood	wood	wood	27	2.5	-1.5	-2.3
5	Calle del Sierra	30	15	paved	mono box	steel	25	3.5	0.1	-1.1
6	Calle del Pradero	12	42	wood	steel	wood	45	3.7	0.6	-0.5
7	Gym Footbridge	4	15	wood	wood	wood	-	2.5	-0.2	-1.1
8	Calle del Pinos	12	25	wood	wood	wood	60	1.8	-1.9	-2.4
9	Footbridge above Calle del Pinos	4	20	wood	wood	wood	-	2.9	-0.2	-0.7
10	Park Entrance Road	30	33	paved	mono box	steel	150	4.3	1.3	1.1
11	Park Footbridge	6	15	wood	wood	fenced	10	2.6	-0.4	-1.2
12	Arenal Avenue	30	25	paved	mono box	steel	20	3.6	0.9	-1.3

Note: The Park Footbridge is also referred to as Calle del Mar, and provides access to the beach from central Stinson Beach at the Parkside Café.

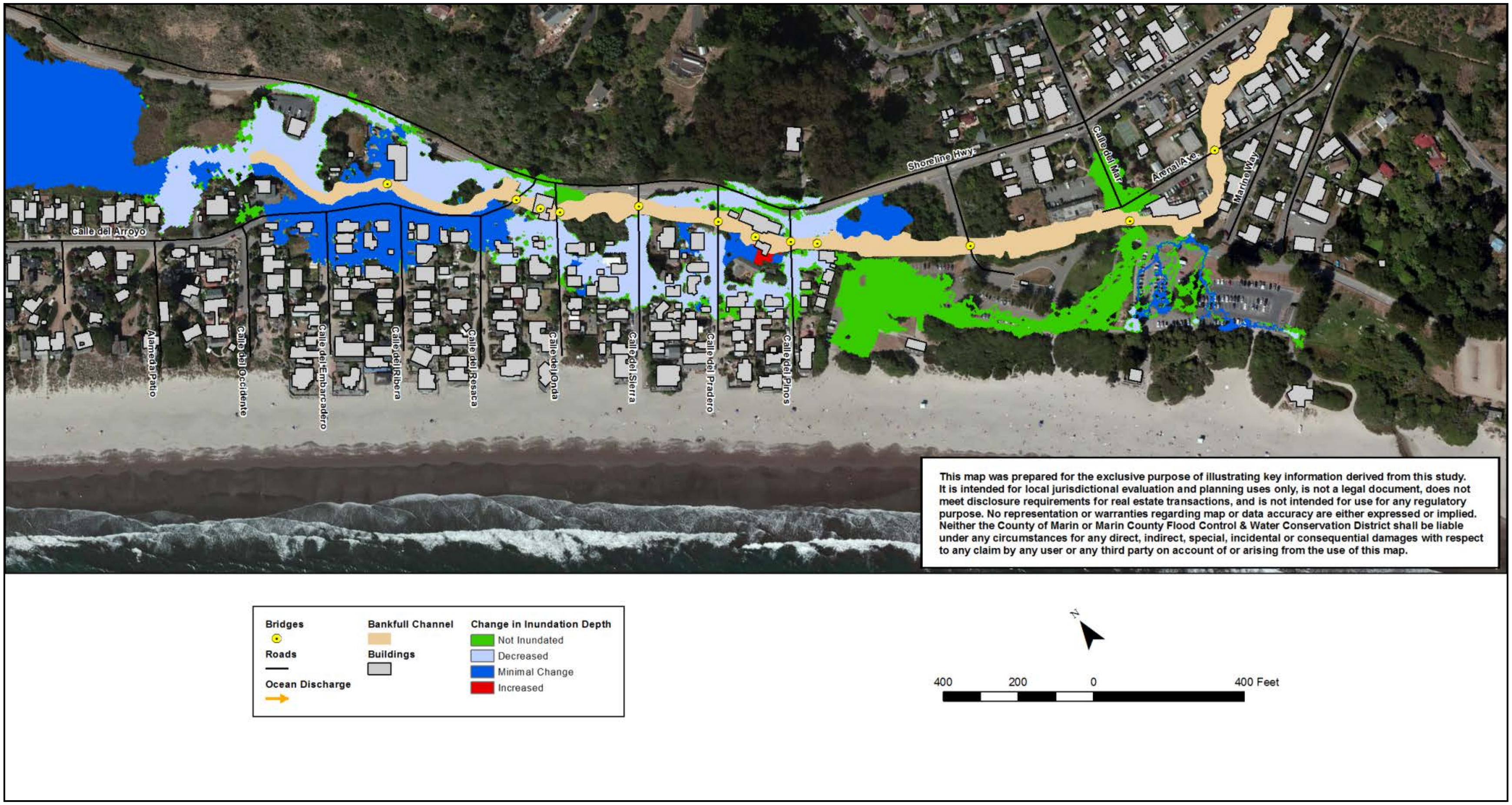


Figure 9-1 Decrease in flood extent and floodplain depths under the Bridge Improvement alternative for the December 2005 flood.

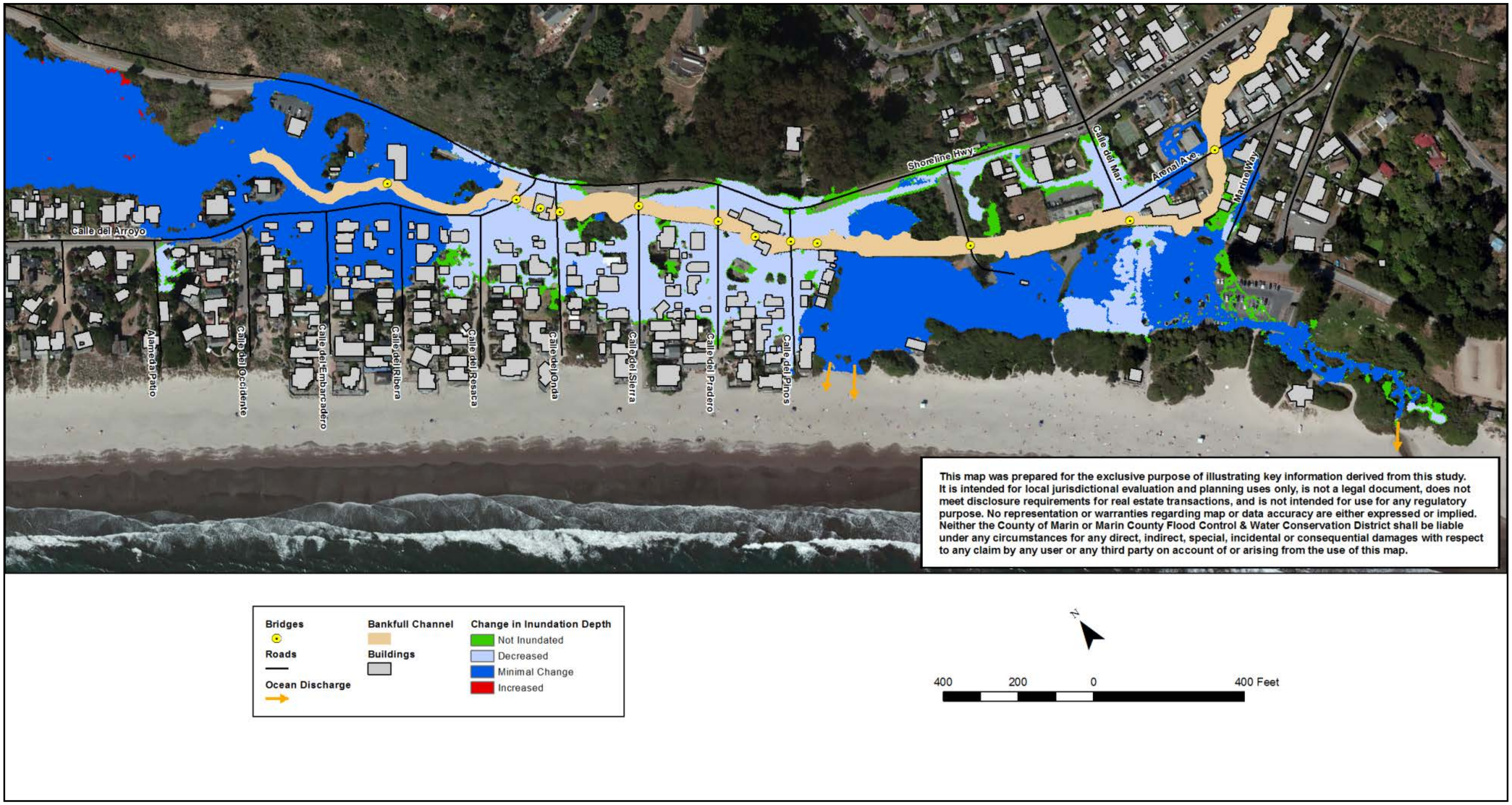


Figure 9-2 Decrease in flood extent and floodplain depths under the Bridge Improvement alternative for the 100-yr flood.

9.2 Description of Alternative

This alternative considers modifications of existing bridges over Easkoot Creek for flood mitigation. It is relatively easy to assess the effects of bridges on flooding and the likely reduction in flooding if bridge decks were high enough to remain above water level. It is more difficult to conceptualize potential bridge modifications and their costs. Following in this section is a general description of these bridges, including their construction, condition and relative impact on flooding.

The Easkoot Creek drainage contains twelve bridges in the area under study (Table 9-2). Bridges 1, 7, 9, and 11 are narrow wooden footbridges and appear to be located on private property. Bridges 4, 6, and 8 are structurally similar in nature; 4 and 8 employ wood girders, 6 at about twice the other's length uses steel I beam girders for deck support. Two are on concrete footings, and one appears to be supported on horizontal wood timbers. Bridges 2, 5, 10, and 12 are structurally similar in nature. They are monolithic concrete box structures oriented at various angles to the channel, support two-way traffic, and are paved.

No two bridges are exactly alike. All are unique designs and were constructed individually at different times. Given the age of the community, most were likely installed decades ago. However, the fairly new appearance of many of the bridges' structural elements indicates that they have been repaired and upgraded over time, possibly in response to flood damage.

County archives have not been consulted with regard to bridge permitting history or construction. Permitted construction either in the public or private sector should have resulted in archival documentation of construction drawings at a minimum, and perhaps would include relevant design materials as well. Most structures support or are immediately adjacent to one or more exposed utilities trunk lines that cross the channel. Water mains seem to be the most prevalent, although other improvements such as gas lines, underground power, or other utilities cannot be ruled out.

Significant bed load deposits are present adjacent to and under the decks of many of the bridges. Clearance between the bottom of the bridge deck and the streambed is less than 2 feet at bridge 8, between two and three feet at bridges 3, 4, 7, 9, and 11, between three and four feet at bridges 1, 2, 5, 6, and 12, and more than four feet at bridge 10 (Table 9-2). Bridge 10 has bed load deposits under the deck, and observed scour with exposed piling at the north east foundation corner. Gabion structures now in place were apparently retrofitted to reduce risk of additional scour. The gabions constrict cross sectional area of the channel under the bridge.

Hydraulic modeling results indicate that bridges 2, 5, 6, 10, and 12 have sufficient capacity for a December 2005 magnitude flood, but only bridge 10 has sufficient capacity for a 100-yr flood (Table 9-2). Bridges 1, 2, 3 and 4 are located low in the watershed within the zone of tidal influence, and as such are also subject to coastal flooding (any final design for modifications to these lower bridges should consider coastal flood hazards which were not addressed as part of this study).

The bridges primarily restrict flow via interaction of the water surface with the bridge decks; flow restriction due to decreases in channel width associated with bridge abutments is of only minor significance. Several of the bridges present a significant flow restriction in moderate flood events like December 2005, resulting in overbank flows occurring immediately upstream of the bridges which in turn result in downstream floodplain inundation. This effect is most pronounced at bridges 1, 3, 4, and 8, which were submerged by more than a foot during the December 2005 flood (Table 9-2).

9.3 Flood Control Benefits

In order to investigate the potential flood control benefits resulting from altering the bridges to reduce or eliminate interaction between the bridge decks and the water surface, a model scenario was evaluated with nine bridge decks completely removed. While complete removal of the bridges may not be feasible, the scenario is designed to represent a best case improvement of these bridges whereby the bridge decks are elevated sufficiently to avoid any interaction with the water surface, and sufficient drainage is provided to avoid restricting floodplain flows with any fill that may be required for bridge approaches. Additional model runs are necessary to examine the effects of different combinations of bridge improvements.

Improving the bridges leads to significant flood control benefits in some areas. During the December 2005 flood, flooding upstream of the North Lot is completely eliminated with the exception of minor overbank flow on the left bank in the vicinity of the Parkside Café (Figure 9-2). Minor decreases in flood extent occur throughout the Upper Calles and floodplain depths decrease by 0.1 to 0.5 feet throughout the residential areas of the Upper Calles. The average reduction in peak water levels in the channel is 0.3 feet in the reach adjacent to the Parkside Café, 0.2 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.0 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table 6-1). While these changes are relatively minor, water levels upstream of several of the bridges (Park Footbridge, Calle del Pinos, and Calle del Onda) are reduced by as much as 0.7 to 1.0 feet significantly reducing the volume of overbank flow associated with bridge constrictions (Table 6-1). These reductions in overbank flow result in approximately eleven of twenty-four buildings having significantly reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions under this alternative (Table 6-2).

During the 100-yr flood, the total flood extent is only slightly reduced from existing conditions; however floodplain depths are reduced by 0.1 to 0.5 feet throughout the Upper Calles (Figure 9-2). Similar to the results for the December 2005 flood, average peak water level reductions are relatively minor overall (Table 6-3), however significant reductions of 0.6 to 0.8 feet occur upstream of several of the bridges (Park Footbridge, Footbridge above Calle del Pinos, Calle del Pinos, Calle del Sierra, and Calle del Onda). The associated reductions in overbank flow above the bridges results in the removal of approximately six of fifty-nine buildings from the 100-yr floodplain (Table 6-4).

<u>It is important to keep in mind that the modeling results represent complete removal of nine of the bridges and as such may tend to over-state the expected flood control benefits of bridge improvement since it is unclear whether all nine bridges can be modified to eliminate flow restrictions</u>.

9.4 Preliminary Design and Estimated Construction Costs

Bridge impacts could be eliminated by complete removal of the bridges. This is not believed to be a feasible option in the case of vehicle bridges. In the case of footbridges, it may be somewhat feasible, depending on legality, ownership, and use factors of existing bridges. In the case of bridge 3 where an uninhabited house is situated over the creek, it may be a feasible if the structure is illegal and non-conforming, and creates a flooding hazard in the neighborhood.

Bridge impacts could be reduced by combining functionality of bridges to reduce total bridge count. This option would be feasible if the community was under initial development, because road and street design could be undertaken in consideration of the riparian corridor, channel, and floodplain. A small number of larger bridges could be used instead of a large number of smaller bridges. This option is not likely to be feasible due to the historic and fixed nature of development of streets and parcels.

Bridge impacts could be reduced by raising the existing supports for individual structures above the design high water levels (e.g. 100-yr water surface). The required incremental elevation has been determined through hydraulic modeling, and varies to an extent at individual bridges depending on channel width and depth and likelihood of future bed load deposition at the site. Raising individual bridges by about two feet above their present elevation would satisfy this objective.

Bridge impacts could be reduced by replacement of individual bridges with specialized structures providing greater separation between deck structural supports and the design water surface elevation.

- For conventional flat bridges, this would entail incorporation of low-profile, high-strength, low-deflection structural elements, and use of low-profile bridge decking (i.e. steel plate versus planks). The (likely modest) incremental height savings could be used to raise the structure without appreciably raising the existing deck elevation.
- Non-conventional approaches such as arched bridges might also be considered. These would be highly atypical of conventional construction, and would require special engineering for design. The individual support elements for an arched structure are inherently unstable and would need to be properly constrained to prevent torsion or tendency to rotate to a lower energy state. The arched configuration also transmits horizontal loads to the supporting foundation structure, so that re-engineering of the existing passive vertically-loaded foundations may be required as well.

Design Considerations. Bridge removal would not require design work per se. Some planning and permitting would be required as to the mechanics of removal. There may need to be redevelopment of the individual sites in terms of riparian stabilization or enhancement. Resource agency involvement would likely mirror the discussion provided in the Alternative 4 dredging scenario, although at a much reduced scale and on a site-specific basis.

Bridge raising would require design of the approaches, relocation of utilities, and assessment – repair – reuse of existing abutment structures. Structural engineering would be required to ensure adequate tiebacks and connections for re-use of existing supports. The option of installation of "legs" to the ends of the bridge structure versus modification of the foundation would need to be considered. Bridge raising is not feasible for bridges 2, 5, 10, and 12, all of monolithic concrete box tube design. Since deck and foundation are integrally cast, these units would require demolition and reconstruction in an alternative configuration.

Bridge approach planning would be part of the design consideration. Approaches would need to be configured so as to respect property limits. In some cases, engineered retaining walls would likely be required to stay within lateral constraints of the available property. Knowledge of underground utilities in the area is required for retaining wall design. High groundwater in the lower section of the work area may add to the complexity of design and construction.

The present discussion assumes simple reuse or replacement of existing structures. It may be that some structures are undersized, inadequately designed, or suffering from structural degradation and unsuitable for re-use. Part of the re-use effort should go towards evaluating adequacy of the existing structure(s). Reused or replaced structures should conform to current codes and current and projected traffic demands, both as a practical matter, and from a risk management standpoint.

Bridge redesign would require type selection and then detailed design of individual unit(s). Based on the variety of configurations now present in the field, it may not be possible to apply a single approach to redesign in which one design would serve for multiple installations. Design considerations would include selection of an appropriate load rating, design style, Fire Service constraints, individual components, connections, foundations, seismic considerations, hydrologic considerations, deck and wear surface, guard railings, pedestrian considerations, utilities relocations, esthetic considerations, costs, permitting constraints, and related items.

Prefabricated bridges with pre-engineered design could be considered, and would include proprietary designs available commercially or built up rail car units. Rail car bridges can be economical in certain situations, but do not appear to be appropriate in these applications. Single units are typically not wide enough to meet Code requirements, and require two units welded longitudinally in order to provide for vehicle and pedestrian needs. Foundation requirements would be similar to those required for stick-built girder-supported assemblies. The rail car girder section is typically 2 to 3 feet deep, so that the advantage of a low-profile configuration would be lost. Rail cars are a byproduct of another industry, and so may not be readily available in a preferred configuration.

Abutment and approach design would involve installation of a wedge of material in order to achieve the required elevation gain using a ramped surface. Per verbal County direction, a maximum slope of 15% would be allowed. This amounts to a 1.5 foot rise per 10 feet of length and a 13.3 foot approach for a 2 foot bridge height increment. The nearest cross street is close to that distance from the bridge in some cases (Table 9-2), so geometric constraints may limit allowable bridge height. A ramp at 15% slope is not ADA compliant. This may not be a factor for vehicular road design, but would need to be further investigated on a case-by-case basis.

Prudent design also utilizes vertical curves so that the transition between horizontal and ramp on the approach and exit is moderated. A 25 foot vertical curve is the normal minimum requirement for low-speed roads. Using that standard, the approach length would need to be nearly 50 feet long to account for the positive and negative inflections between existing ground and the elevated bridge. Since this distance is not available in most cases, a more sharply inflected section would be required. Such curves may deviate from local code requirements.

Raised abutments that are either earth-fill construction or structural members would need to be fitted with appropriate guard rails, particularly if a vertical wall is used at either side. New utilities connectors with flexible connectors would be required in cases where they are attached to the existing bridge.

Assumptions for preliminary cost estimates vary by bridge type. Costs of foot bridges, evaluated using standard procedures, were estimated assuming simple lift to higher elevation using hand labor methods. Cost for private vehicular wood-decked bridges, also evaluated using standard procedures, were estimated assuming simple lift to higher elevation using hand labor methods, and include new abutments, gravel ramps, extended railings, and paved approaches. Estimated costs for monolithic concrete box culvert bridges were estimated based on construction date and installed cost, which is brought to present-day values by applying a cost of living factor.

9.5 Permitting Issues

Impacts of bridge modifications on fish habitat and other ecological resources are expected to be modest. Although construction activity would occur in riparian areas, floodplains, and possibly in stream channels, impacts would be largely temporary and are not expected to significantly degrade fish habitat. In addition, reducing the frequency and magnitude of flooding would tend to prevent potential threats to aquatic species associated with urban flooding; endangered steelhead and coho salmon would be less likely to be flushed from the channel onto the floodplain. Furthermore, potential dredging at bridges that might occur in conjunction with bridge improvements could increase availability of pool habitat as has occurred after prior dredging. Mitigation for potential impacts of bridge improvement projects on habitat has not been specifically considered. However, as described in the dredging alternative, salmonid habitat enhancement could be more effective upstream of Arenal Avenue where sedimentation processes are not overwhelming.

Raising the approach on either side of a bridge requires installation of a triangular wedge of material so that vehicles may be positioned to cross the bridge. By definition, each bridge is located in the floodplain, so creation of the fill wedge using soil or gravel will violate the “No Net Fill” rule within the floodplain. Required fill volumes range from about 1.5 cubic yards for a 5 foot wide approach raised 1 foot to about 31 cubic yards for a 30 foot wide unit raised 3 feet. Incremental fill could be reduced by 0.4 to about 8 cubic yards for the range cited if vertical retaining walls were used for fill containment rather than using earth fill at 2H:1V along the approaches.

Bridge construction may be selectively limited to the top of the channel bank, particularly if the foundation is reused. In that case, work should be exempt from CDFW Stream Alteration Agreement permitting, because it takes place outside the jurisdictional area. Technically speaking, it would also remain exempt from jurisdiction of Army Corps, US Fish and Wildlife Service, Regional Water Board, and related agencies for the same reason. Practically speaking, it would make sense to remove accumulated bed load in under-bridge areas during construction. In that case, the work would become jurisdictional and permitting would be required with all of the resource agencies discussed in the Dredging Alternative scenario.

Replacement bridges would go through a typical design and permitting effort for the structural elements of the bridge. Design would be undertaken by a structural engineer in consultation with the client (public or private) in order to develop a value-engineered solution. Normal County requirements for bridge design would govern the development, and normal County permitting procedures would be utilized for the structural aspects of the construction. Bridge 11 is a footbridge between Arenal Ave. and the beach area. It does not presently meet ADA requirements, but could possibly be subject to such constraints if modified. It is not known if other vehicular bridges would be subject to ADA requirements.

The work of vehicular bridge relocation or replacement poses a substantial problem from a construction standpoint. Most of the bridges under consideration serve dead-end streets with no alternative access. The urban area is not configured to allow cross traffic between affected streets so that another route is available during bridge work. If a bridge is temporarily decommissioned, dead-end street access will be limited to foot traffic. Emergency vehicle access for fire, police, or ambulance service will not be available unless an alternate route can be made available, possibly through temporary roads on the beach linking to adjacent streets during construction.

Relocation of an existing bridge would be most efficiently accomplished by using a heavy lift crane to raise the structure in one move. The unit would then be lowered onto a new or upgraded foundation. This alternative is not likely available in the present circumstance, because work sequencing is a problem. The old foundation could not be adjusted until the existing bridge was lifted out of the work area.

A heavy-lift crane is expensive to operate and takes up the width of the roadway. It is not believed feasible or cost-effective for the crane to stand by for the extended time period required to do the foundation work between old and new bridge settings. Cranes are supported on outrigger foot pads. Work on secondary roads with unknown quality is likely to damage the roadway due to pad imprints. Excessively soft conditions could jeopardize crane operations. Cranes rely on counterbalance weights that are brought to the site on separate trucks and self-assembled by the crane operator. Offloading, assembly, and disassembly will require proximity between vehicles and would need to be worked out with operator's assistance. While likely feasible, at a minimum traffic restriction along Highway 1 would be likely during any work activity. Any crane work undertaken would also need to factor in local overhead power lines and the lift and extension constraints of the boom relative to field positioning at the end of the bridge.

The alternative to a crane lift of the bridge structure is to raise the bridge from below using jacking methods. This requires placement of materials in the channel, invoking permitting issues. Jacking would be relatively slow and labor intensive, and would require shoring for safety purposes. The bridge would remain in the way of any anticipated dredging below the structure during site upgrade. The bridge would remain elevated and unusable until the approaches were completed. The approach on the far side of the bridge would be hard to build, because the work area would remain inaccessible due to the incremental step between bridge and ground.

Development of the approach ramps is also problematic from a construction sequencing standpoint. Ramp construction prior to old bridge removal precludes bridge use for the duration of the construction effort. The foundation assembly supporting the new bridge would need to be a separate prefabricated drop-in unit that attaches to the old foundation to expedite final construction, because the foundation is covered by the old bridge until such time as removal occurs. Ramp construction on the far side of the bridge is problematic as well. If built first, construction access is assured, but the roadway becomes impassible. If built with the bridge out, access is a problem. If built once the bridge has been raised, access remains a problem due to the vertical drop off the raised bridge.

One way around the abutment conundrum would be to construct the approaches as a stand-alone drop-in trestle type arrangement. This would allow prefabricated construction prior to bridge raising or replacement. The trestle sections could be supported on a ground contact skid plate with columns, allowing flow of flood waters through the structure. The trestle could have built in vertical curves to facilitate vehicular traffic. Methods would need to be developed to assure adequate load bearing capacity and resistance to motion in the x and y directions. They would also need to be resistant to differential settlement or rotation so that they would not "walk out" from under the bridge due to cyclic loading or temperature induced expansions and contractions. Local streets would need to be reconstructed to meet strength and geometric constraints. Pinned joints or other kinds of attachments might be necessary to prevent rotation or separation at the abutments. The abutments could be placed relatively quickly once the original bridge was temporarily set aside. The bridge would then be dropped in place on the new abutments. Logistics of bridge movements might be problematic in tight spaces, because the old bridge would need to be out of the way of the new abutment placements. Dimensional tolerances for placements would need to be to the nearest ¼ inch or less, requiring a high degree of precision in assembly and placement. The abutments would likely need to be segmented due to trucking length constraints. They would be brought to the site by truck and staged for unloading and installation. For bridges with widths less than the channel or riparian width, it may be possible to park the bridge in the channel aligned upstream-downstream while the abutments are installed.

If existing bridges were considered unsuitable for reuse for any reason, the old units would be replaced with new structures. The old units would be disassembled on site for salvage or disposal. The footbridges constitute a special case and a general category, as they are narrow, long, and relatively lightly constructed. Each could be fairly easily raised by jacking. The individual approaches could be reconstructed into a stair stepped configuration.

9.6 Operation and Maintenance Requirements and Costs

There are no extraordinary Operation and Maintenance Costs associated with this alternative. Properly designed and installed bridges should have a reasonable 20-year design and economic life. Selection of materials that are resistant to corrosion and decay would be needed in this moist and corrosive coastal environment. A properly designed bridge should have low maintenance requirements. The approximate life of wood-based structures is perhaps 20 years, after which accumulated biological deterioration may require reconstruction or structural repairs. Concrete and steel members should have a longer service life of perhaps 50 years if properly designed and constructed. The bridge decking constitutes an expendable wear surface, whether of wood, concrete, asphalt, or other material. Periodic maintenance would be expected to be necessary to provide satisfactory long-term performance.

9.7 Sustainability (Short-term and Long-term)

Bridge improvements would remove flow obstructions from within the floodway and reduce diversion of flow onto the floodplain. Bridge replacement does not address the causes of channel capacity reduction due to sedimentation, however, when flood flows are affected by bridges, it is likely that sedimentation rates increase. Consequently, bridge improvements are likely to reduce local sedimentation observed at some bridges under existing conditions. Should bridge redevelopment be pursued without addressing sedimentation, it is likely that channel capacity will continue to be diminishing and the flood mitigation benefits of bridge improvements degraded. Long term flood mitigation is not likely to be achieved through bridge improvements alone.

9.8 Feasibility, Next Steps and Additional Information Needs

As described above, numerous property, design, and permitting issues and complex construction logistics must be resolved to implement bridge improvements. The hydraulic modeling performed for this alternative represented a 'best case' scenario of removal of nine of the twelve bridges. The effects of removal of individual bridges should be evaluated if a more refined plan for bridge improvements is developed to evaluate potential flood impacts downstream. If possible, the new bridge configurations should be evaluated with the hydraulic models in order to gain a clearer understanding of the expected flood mitigation effects and potential increases in flooding downstream of bridge improvements. This would likely be necessary for development of bridge designs. The effects of bridge improvements on sedimentation could also be assessed at that time.

10 Alternative 3-Vegetation Management

This alternative investigates the potential for flood mitigation resulting from reducing the density of shrubs and woody vegetation on the channel banks. Dense vegetation on stream banks can create high flow resistance that reduces water velocity and increases flow depth. A channel with more vegetation (higher flow resistance or "roughness") would reach flood stage at a lower rate of stream flow than the same channel with less vegetation. Routine vegetation management to reduce the density of shrubs and woody vegetation within a stream channel is expected to maintain higher channel conveyance (flow capacity). The District performs this type of vegetation management in Easkoot Creek on an annual basis, suggesting that it may be difficult to achieve further reductions in flow resistance associated with vegetation. In addition, this type of riparian vegetation may have significant habitat value for aquatic and terrestrial species, and more aggressive vegetation management might be inconsistent with habitat and aesthetic values.

To evaluate this alternative, it was assumed that bank roughness could be reduced by 25% in most reaches. All bank roughness values were reduced by 25% from the existing condition with the exception of the right bank reach adjacent to the Parkside Café where the existing bank is a concrete or gabion wall with little or no vegetation.

The hydraulic analysis indicates that reducing roughness through a program of vegetation management does not have the potential to significantly reduce flooding impacts. Average changes in water level for this alternative were 0.1 feet or less throughout the study reach for the December 2005 flood (Table 6-1). This did not result in significant changes in flood extent or floodplain depths and no buildings had reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions (Table 6-2).

Given the low degree of flood mitigation achieved by this alternative, no further assessment of implementation was conducted; similar to the three infeasible alternatives described previously (Section 4.11).

11 Alternative 4-Channel Dredge and Sediment Management

11.1 Summary

Proposed dredging (Figure 11-1) would remove about 3,100 cubic yards of sediment from a 2,300-foot reach from Arenal Avenue to Calle del Arroyo. The average depth of excavation would be 2.4 feet with a maximum of 3.4 feet (Figure 11-4 and Figure 11-5). Mean width of the dredged channel is about 15 ft.

Figure 11-1 Map overview of proposed dredging scenario.

The flood control benefits of dredging are substantial. In the December 2005 flood, flooding above Calle del Onda is completely eliminated (Figure 11-2). Below Calle del Onda flood extent and floodplain depths remain approximately the same as under existing conditions owning to the tidal control below Calle del Onda and the fact that the dredge terminates ~280-ft below this point. Approximately eighteen of twenty-four buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions under this alternative (Table 6-2). During the 100-yr flood, flooding is substantially reduced above Calle del Onda (Figure 11-3). Below Calle del Onda changes in floodplain depths are minor and even increase slightly in some areas. Flooding at the Arenal Avenue Bridge is eliminated and flooding on the right bank just downstream of the Parkside Café is reduced substantially such that only street flooding occurs in the vicinity of the intersection of Calle del Mar and Arenal Avenue. Approximately seven of fifty-nine buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under 100-yr flood conditions under this alternative (Table 6-4).

Significant dredging has been required at intervals of less than about ten years owing to episodes of very high stream flow and sediment transport from the upper watershed. The December 2005 flood event deposited about 1,000 to 1,500 cubic yards of sediment, and proposed dredging would remove about 3,100 cubic yards of sediment. Mean annual sedimentation is estimated to be about 122 to 160 cubic yards per year. To reduce future sedimentation that contributes to flooding and to extend the flood mitigation benefits of dredging as long as possible, supplemental sediment removal structures with a capacity of about 290 cubic yards are proposed upstream of State Highway 1. Spot dredging by the District between Arenal Avenue and Calle del Arroyo can remove at least 150 cubic yards. It is likely that a future storm event would cause significant sedimentation of the dredged channel even with a regime of annual sediment removal.

Dredging is feasible, but significant planning/permitting effort is needed because habitat of endangered species (steelhead and coho salmon) would be disturbed by dredging and construction of new sediment removal sites. Fish habitat mitigation efforts could be necessary, possibly including habitat restoration in the dredged reach and habitat enhancement upstream of Arenal Avenue in two reaches (Figure 11-1). Likely habitat restoration and enhancement objectives would be to create both spawning and rearing habitat. Mitigation for disturbance to riparian habitat is considered. Costs for CEQA compliance (e.g. preparation of an EIR, if necessary), are not included.

Table 11-1 Summary cost estimate for dredging and sediment management.

Alternative 4-Dredge - Planning-level Budget Summary	Cost ($)	Percent
Consultant Permitting Subtotal	28,720	3.4
Consultant Assessment & Design Subtotal	101,200	11.8
Construction Subtotal	457,275	53.4
Contractor Overhead Subtotal	124,970	14.6
Riparian Mitigation Subtotal	53,750	6.3
Construction Monitoring and Quality Control	49,200	5.7
Planning-level Cost Estimate (to nearest $100)	815,100	95.2
Project Administration	40,760	4.8
Installed Project Cost Estimate	**855,860**	**100.0**
Estimated Annual Maintenance (remove 150 cubic yards sediment)	18,600	
Estimated Maximum Maintenance (remove 440 cubic yards sediment)	32,800	

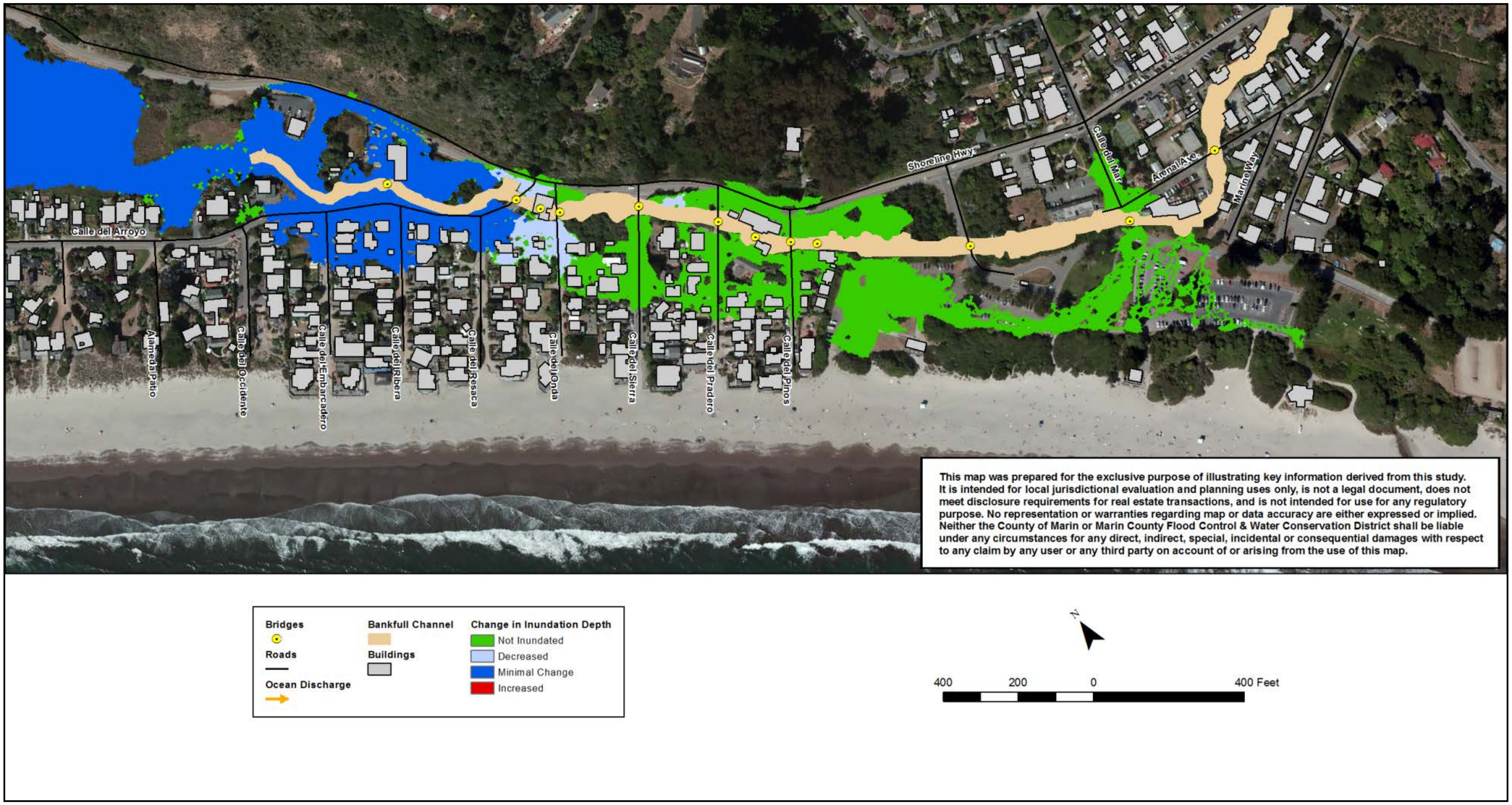


Figure 11-2 Decrease in flood extent and floodplain depths under Alternative 4- Channel Dredge and Sediment Management for the December 2005 flood.

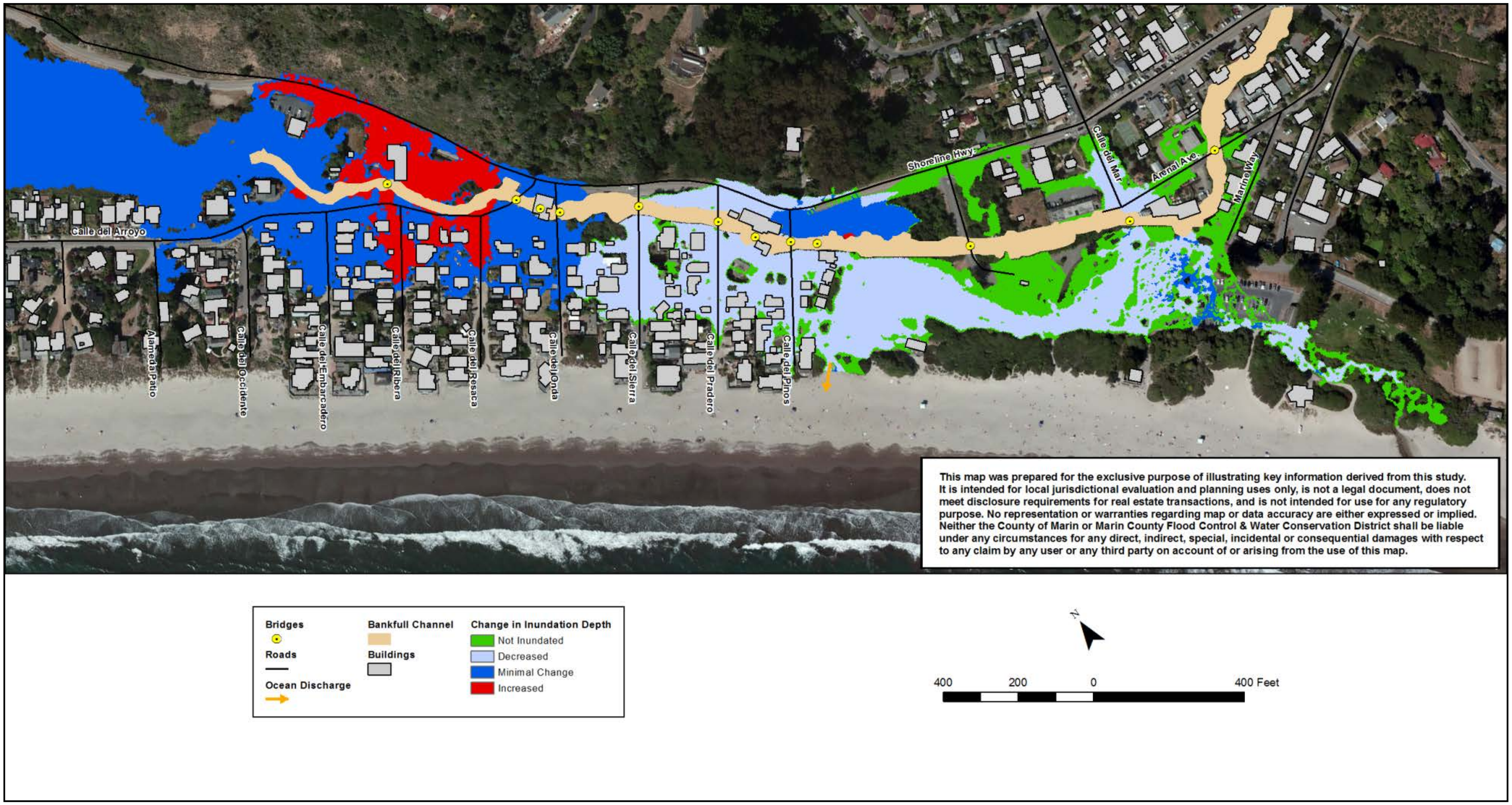


Figure 11-3 Decrease in flood extent and floodplain depths under Alternative 4- Channel Dredge and Sediment Management for the 100-yr flood.

11.2 Description of Alternative

This alternative combines dredging of the channel with installation and maintenance of sedimentation facilities along the creek.

Historic Dredging. Historic patterns of sedimentation indicate that significant dredging is required at intervals of less than about 10 years owing to episodes of very high stream flow and sediment transport from the upper watershed. The most recent event that caused significant sedimentation was the December 2005 flood which deposited about 1,000 to 1,500 cubic yards of sediment. Widespread channel dredging occurred in 1973, 1983, 1987, and 1997, and probably also occurred in 1955. In 1973, 4,000 cubic yards of sediment may have been excavated; about 1,500 cubic yards was removed in 1987. Elevated sediment transport rates associated with high flows and accelerated watershed erosion combine to cause high rates of sedimentation in reaches of Easkoot Creek in Stinson Beach beginning near Arenal Avenue. The mean annual sedimentation rate estimated for the period 1979-2011 is 122 cubic yards per year, and may be as much as 160 cubic yards per year if poorly-documented dredging in 1983 is incorporated in the estimate. Recent dredging by the District from 2007-2009 averaged about 100 cubic yards per year (Table 11-2). Subsequent sediment deposition through 2011 was insufficient to fill pools created by dredging at bridges along the Calles, indicating low sedimentation rates in recent years. Provided that peak flows are modest (approximately < 5 year recurrence interval) and watershed erosion rates are not accelerated by large-scale mass wasting, it appears that the existing dredging program, although limited, can remove sediment volumes approximately equal to the average sedimentation rate.

Table 11-2 Summary of District "spot" dredging volumes (cubic yards).

Year	Arenal Ave.	Calle del Mar	Calle del Pinos	Calle del Pradero	Calle del Sierra	Calle del Onda	Calle del Arroyo	Total
2007	0	0	37	26	0	26	0	100
2008	55	1	52	53	0	0	0	161
2009	35	0	0	0	0	0	0	35

Proposed Dredging. Proposed dredging (Figure 11-1) would remove about 3,100 cubic yards of sediment in a single effort. This mass dredging event would be supported by maintenance dredging at the locations currently spot dredged (see Table 11-2), at a proposed site upstream of Calle del Mar currently being developed by the District (Figure 1-1), and two potential supplemental sediment removal reaches proposed upstream of State Highway 1.

Channel topography for a dredged channel was based on the 1979 FEMA profile of Easkoot Creek, which was used as a template and modified.[7] The FEMA profile was blended into the existing bed elevations from the 2011 OEI Survey at the upstream and downstream ends of the dredged reach. The upstream boundary of the proposed dredging begins 260 feet downstream of the State Highway 1 Bridge and extends to a point 280 feet downstream of Calle del Arroyo. The total length of proposed dredging is approximately 2,300 feet. The average depth of excavation relative to the 2011 profile would be 2.4 feet with a maximum of 3.4 feet (Figure 11-4). Mean width of the dredged channel is about 15 ft. Dredging would produce about 3,100 yards of material, primarily sand and gravel.

11.3 Flood Control Benefits

The flood control benefits of dredging are substantial. During the December 2005 flood, flooding above Calle del Onda is completely eliminated (Figure 11-2). Below Calle del Onda flood extent and floodplain depths remain approximately the same as under existing conditions owning to the tidal control below Calle del Onda and the fact that the dredge terminates ~280-ft below this point. The average reduction in peak water levels in the channel is 2.6 feet in the reach adjacent to the Parkside Café, 1.1 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.0 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table 6-1). Approximately eighteen of twenty-four buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions under this alternative (Table 6-2).

[7]Surveys of the channel profile in subsequent years (1999, 2004) in the NPS reach showed little change relative to the 1979 profile. It is understood that dredging of the channel after the flood event in 1982 was substantial, and probably explains why the channel profiles were similar in 1979, 1999 and 2004. Surveys in 2006, 2007 and 2011 showed that the channel profile slope above 5 ft AMSL (approximately the elevation of high tides) was largely unchanged relative to earlier profiles, but that the bed aggraded by about 2 to 3 ft, apparently because of sediment deposition during the flood event on Dec. 31, 2005. We adopted the 1979 profile as our model for proposed dredging because of the evidence of relative stability (absent large sedimentation events) of this profile.

During the 100-yr flood, flooding is substantially reduced above Calle del Onda (Figure 11-3). Below Calle del Onda changes in floodplain depths are minor and even increase slightly in some areas. The small increases can be attributed to the effect that the increased conveyance in the upper dredged reach has in terms of more effectively moving water downstream to the lower reach and estuary. Flooding at the Arenal Ave. bridge is eliminated and flooding on the right bank just downstream of the Parkside Café is reduced substantially such that only street flooding occurs in the vicinity of the intersection of Calle del Mar and Arenal Avenue (Figure 11-3). Floodplain depths in the north Park Service parking lot (North Lot) are reduced by 0.5 to 1.0 feet. Flood extent and floodplain depths are reduced by 0.25 to 0.5-ft in the reach between the North Lot and a point just upstream of Calle del Onda (Figure 11-3). The average reduction in peak water levels in the channel is 1.1 feet in the Parkside Café reach, 0.2 feet in the Upper Calles, and 0.1 feet in the lower Calles (Table 6-3). Approximately seven of fifty-nine buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under 100-yr flood conditions under this alternative (Table 6-4).

The sedimentation basins are not expected to have significant direct flood control benefits and were not evaluated with the hydraulic models. The sedimentation basins are however considered necessary to extend the increased capacity of the dredged channel and the associated flood control benefits of the dredge as long as possible.

11.4 Preliminary Design and Estimated Construction Costs

Preliminary Design: Dredging of an open, unencumbered channel with no aquatic resources is a simple matter of accessing the site with earthmoving equipment, removing excess sediment accumulation, shaping banks, and providing necessary erosion controls. However, none of these conditions are present in the segment of Easkoot Creek now under consideration for dredging and supplemental sediment removal sites. It is anticipated that significant efforts in design, construction and maintenance of the dredging plan and sediment removal sites will be required to minimize potential adverse impacts on the aquatic ecosystem, particularly the habitat used by endangered anadromous salmonids.

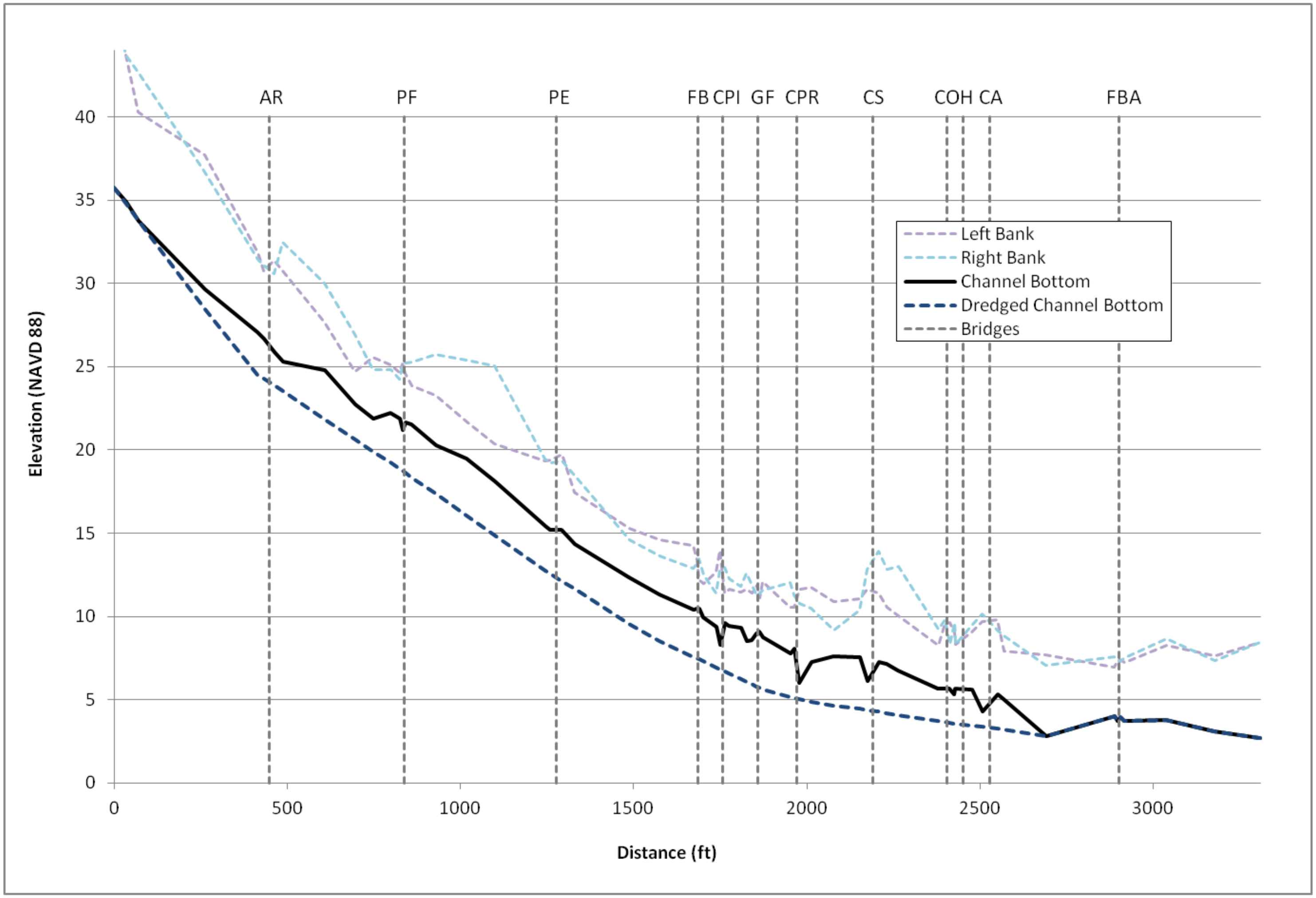


Figure 11-4 Longitudinal profile of the existing and dredged channel of Easkoot Creek.

Vertical dashed lines indicate the positions of the bridges for reference. Bridges are labeled as follows: AR (Arenal Ave.), PF (Park Footbridge), PE (Park Entrance Rd.), FB (Footbridge Above Calle del Pinos), CPI (Calle del Pinos), GF (Gym Footbridge), CPR (Calle del Pradero), CS (Calle del Sierra), CO (Calle del Onda), H (House Bridge), CA (Calle del Arroyo), and FBA (Footbridge below Calle del Arroyo).

The dredged channel begins about 260 feet downstream of the Highway 1 bridge and extends to a point about 280 feet downstream of Calle del Arroyo (Figure 11-1). This location corresponds to the approximate elevation of Mean Tidal Level (MTL), the transition from the fluvial sand and gravel deposits of Easkoot Creek to the estuarine silt and clay bed previously identified by the NPS[8] as well as the downstream extent of sedimentation (Figure 11-4). The total length of dredged channel is approximately 2,300 feet. The average depth of excavation was modeled at 2.4 feet with a maximum offset of 3.4 feet from existing conditions. Mean width of the dredged channel is 15 feet; representative cross-sections of the dredged channel as modeled are shown in Figure 11-5. Additional geotechnical analysis may be required to evaluate bank stability in the context of dredging. Dredging would remove about 3,100 cubic yards of silt, sand and gravel.

The initial design concept and installation methods are driven by regulatory constraints regarding endangered anadromous fish species. The proposal is to excavate to -3.0 feet and spoils would be off-hauled to a designated reprocessing or disposal site. Excavation methods are to be determined, based on access, permitting, and habitat impact issues. Traditional methods (mini-excavator, backhoe, dump truck) would be most cost-effective if access is readily available. Alternative methods (vacuum truck, hand methods, skyline yarder, etc.) may be considered for inaccessible reaches or where habitat values preclude entry or operation with traditional equipment. Methodology would be developed in consultation with affected landowners, resource consultants, regulatory agency staff and the District. Hand methods are likely required where headroom is constrained under some of the eight bridges within the work area.

It is assumed that the proposed depth of dredging will not grossly destabilize stream banks throughout the dredged reach; this assumption is based on matching the dredged channel profile to past channel bed profiles and that bank stability has not been reported to be a major problem in the past. Nevertheless, some bank revetments are present in the reach, suggesting that bank stability issues are locally significant. Bank heights are typically two to six feet under existing conditions, and bank heights will be two to three feet higher after dredging. Additional analysis of potential bank stability problems

[8] Fong, D. (2002) Fisheries Assessment for Bolinas Lagoon Tributaries within the Golden Gate National Recreation Area, 1995-2000. Prepared for the National Park Service, Golden Gate National Recreational Area, Division of Natural Resource Management and Research. Feb. 2002, p. 45.

that may result from dredging will be necessary during the permitting phase of a dredging program. Bank stabilization measures could be required in some areas, and existing revetments may affect site specific dredging activity. Previous surveys by NPS found about 540 feet of revetments in lower Easkoot Creek, most of which (354 feet) is in the concrete retaining wall and gabions (72 feet) located near Calle del Mar footbridge and the Parkside Café. An additional length of 66 feet of sacrete/sandbag and 53 feet of rip-rap revetments were also reported; these are believed to be located behind the commercial buildings between Calle del Pinos and Calle del Pradero.

Potential Habitat Restoration and Enhancement: After dredging, stream bed restoration in the dredged reach may be required to mitigate expected impacts on salmonid habitat. Habitat restoration in the dredged area could include shaping the dredged surface to create channel geometry and hydraulic conditions that would promote spawning and rearing habitat. Eight individual pool-riffle sequences distributed at intervals between Calle del Onda and Calle del Mar might be appropriate as restoration. The sustainability of these habitat features is limited by sedimentation effects as demonstrated by the NPS restoration work implemented in 2004. Adaptive management principles could be employed to ensure that adequate habitat conditions are maintained. Under adaptive management, habitat conditions would be evaluated and monitored. If after a period of channel adjustment to dredging the desired habitat conditions do not exist, additional enhancement and restoration measures could be implemented. Such measures are discussed in “Fish Habitat Existing Conditions and Enhancement Potential” (Appendix, pp. 92-98).

If necessary, potential habitat enhancement could occur in two reaches: the first (HE1) between Arenal Avenue and State Highway 1 and the other (HE2) extending from the confluence of Black Rock Creek (between the Community Center and the Fire Station) upstream to the Matt Davis Trail bridge (Figure 11-1). Each of the potential enhancement reaches is about 400 feet in length. These sites are believed to be much less vulnerable to sedimentation impacts compared to sites downstream of Arenal Avenue and are likely to be more sustainable.

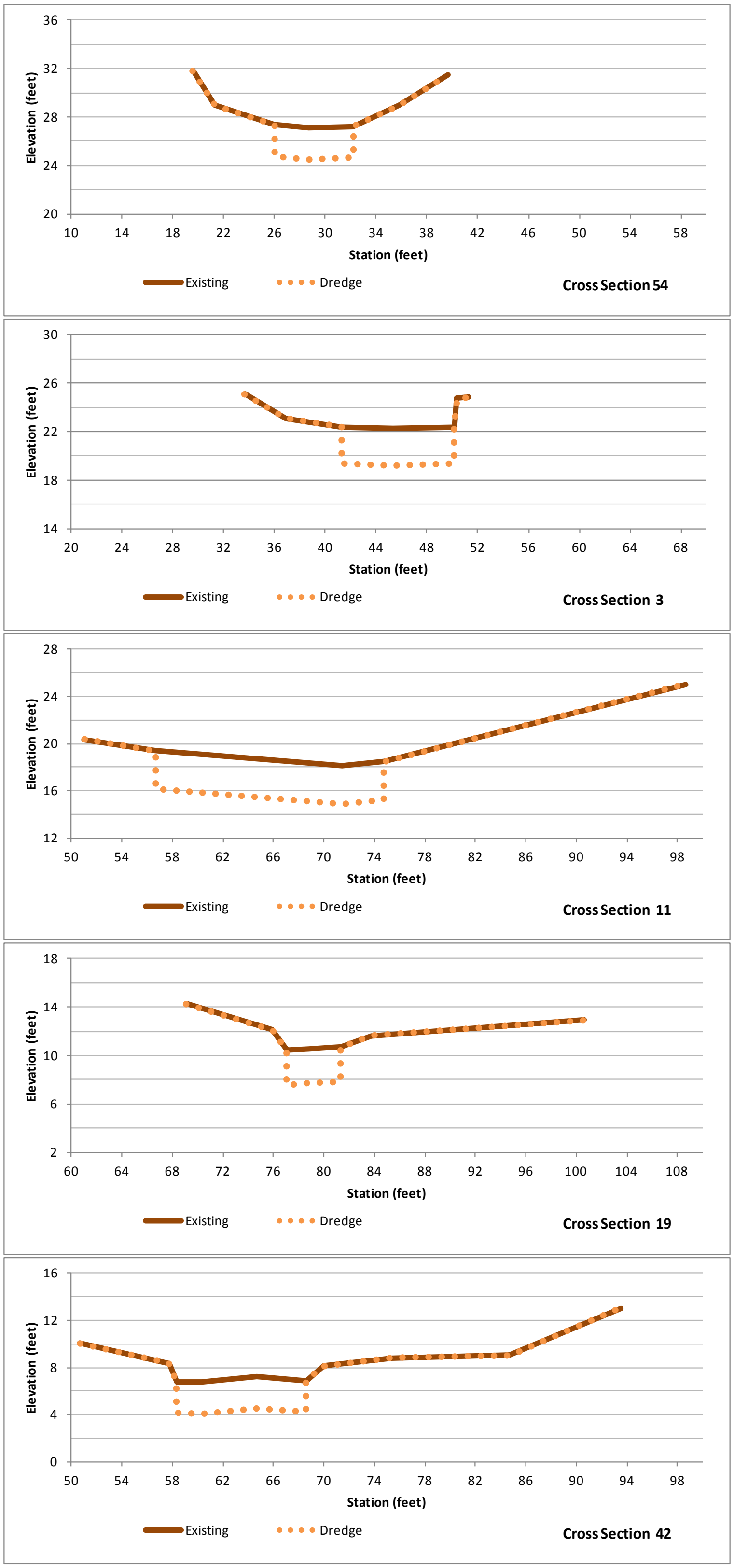


Figure 11-5 Representative channel cross-sections used in hydraulic model of proposed dredging scenario (locations shown in Figure 11-1).

Supplemental Sediment Removal: Channel elevation maintenance would be comprised of continuation of spot dredging practiced in recent years by the District, supplemented by new sedimentation structures. One new sedimentation structure being planned by the District is located upstream of the Calle del Mar pedestrian bridge. The District structure is expected to induce deposition primarily by widening the channel in a zone where historic sedimentation and sediment deposition potential is high. Two additional sets of proposed structures would be placed at two locations upstream of State Highway 1 (Figure 11-1). The first (SR1) would be immediately upstream of the State Highway 1 bridge adjacent to the fire station; the second, higher capacity site (SR2) would be located upstream of the Matt Davis Trail pedestrian bridge on Belvedere Avenue. Each of these would require a permanent access trail suitable for equipment (excavator or backhoe) needed to construct and maintain sediment removal structures. Access road development will require landowner approvals.

Historic patterns of sedimentation indicate that significant dredging has been required at intervals of less than about 10 years owing to infrequent episodes of very high stream flow and sediment transport from the upper watershed. The December 2005 flood event deposited about 1,000 to 1,500 cubic yards of sediment, and proposed dredging would remove about 3,100 cubic yards of sediment. Mean annual sedimentation is estimated to be about 122 to 160 cubic yards per year. Supplemental sediment removal sites are proposed to reduce future sedimentation that contributes to flooding and to extend the flood mitigation benefits of mass channel dredging as long as possible. With only existing "spot" dredging capacity and a new sedimentation basin near Parkside Cafe, it is unlikely that widespread channel sedimentation can be prevented during large storm events with recurrence intervals of about 10 years or greater, despite evidence that mean annual sedimentation rates are comparable to combined dredging capacity at "spot" locations with or without the proposed Parkside basin.

To avoid creating migration barriers affecting endangered salmonids and other aquatic organisms, designs will maintain a low flow channel consistent with the existing channel slope and profile, including considerations of channel morphology and habitat quality. At SR1 channel slope is about 1.2%, but increases to about 8% just upstream. At SR2 mean channel slope is about 5.5%. The channel morphology of the steeper portions of Easkoot Creek is a series of step pools and cascades[9], with generally shallow pools and few potential spawning beds. This channel morphology is typified as a stair-step profile, with short, steep drops over boulder "steps" or "dams" alternating with relatively flat channel segments with gravel-cobble substrate. Step-pool morphology has well-defined steps of boulders and/or wood debris and relatively uniform spacing with intervening zones of sediment deposition, whereas cascades have a more chaotic structure of boulder steps with smaller and irregular deposition zones.

Proposed sediment removal structures will be comprised of a series of partial weirs constructed of rock about 2.5-foot-high spanning the channel width, but maintaining a 2-foot-wide gap accommodating a low flow channel (see conceptual designs in Figures 11-6 and 11-7). The placement of the gap in successive structures at SR2 will be off-set from the channel centerline to lengthen the flow path and promote sediment deposition. Under most flow conditions, flow would be accommodated within the gap in the partial weir. During the largest flows, the weirs will carry flow across the broad crest of the rock structure. It is intended that these structures have maximum potential to induce deposition of sand and gravel during relatively rare periods of high (and deep) flow, approximately ≥ 5 year recurrence interval, when transport of bed load sediment from the upper watershed occurs at relatively high rates and when downstream sedimentation potential is greatest. The gaps in the partial weirs will also establish the location of the low-flow channel by maintaining a zone of high velocity flow capable of excavating a channel. The location and design of sediment removal structures must permit periodic (annual or nearly so) access by equipment to excavate accumulated sediment. Maintenance activity would attempt to avoid disturbance of the low flow channel, focusing on excavating accumulated sediment on the broader surfaces between the banks, the weirs, and the edge of the low flow channel except in the aftermath of major sedimentation events that fill available deposition zones.

SR1 (Figure 11-6) is proposed upstream of the State Highway 1 bridge (Figure 11-1) in a sixty foot reach adjacent to Fire Station #1 where the channel slope is about 1.2 %. Two weirs would be spaced at thirty foot intervals. Estimated sediment storage potential of thirteen cubic yards per weir adds a total estimated sediment storage potential of twenty-six cubic yards at SR1. Beginning just upstream of the

[9] Montgomery, D. and Buffington, J. (1997) Channel reach morphology in mountain drainage basins. GSA Bulletin 109(5)596-611.

Matt Davis Trail bridge, SR2 is proposed to extend 200 feet upstream with a series of ten partial weirs (Figure 11-7). Bed slope in this area is about 5.5%. To increase sediment capture potential, we propose to increase the width of the sedimentation design flow channel to about forty feet by excavating adjacent terraces (Figure 11-7). These weirs would be spaced at twenty foot intervals, and each weir has estimated sediment storage potential of twenty-seven cubic yards. As shown in Figure 11-7, the upper and lower weirs are narrower, representing the need to expand and contract the series of structures to conform with channel widths upstream and downstream. Total sediment storage potential at SR2 is about 260 cubic yards. Total potential sediment storage at SR1 and SR2 combined is about 290 cubic yards, nearly tripling the volume of potential maintenance dredging.

The partial rock weirs may also be expected to create channel morphology that is characteristic of these types of channels under natural conditions, with pools developing below the notch in each weir. Potential spawning sites would be expected at the downstream edge of the pool, which in this channel would probably be about halfway between the weirs. Hence, the weirs may be expected to preserve or improve the diversity of habitat conditions and provide additional spawning habitat.

Design Considerations: There are many significant constraints involved in the work, many of which have a bearing on dredging design, construction approach, methodology, and cost. Potential issues and constraints follow:

- Dredging Mechanics
    - Direction – upstream versus downstream; biological preference typically to move from downstream to upstream
    - Sequence – single project versus multi-year effort; biological preference likely to pursue as multi-year to limit impacts to critical fish habitat to acceptable level
    - Methodology – equipment, technique dependent on resource constraints
    - Water diversion – may be required for access, construction
- Access
    - Definitive surveys of property lines
    - Informal vs. formal approvals
    - Potential impacts/acquisition of easements
    - Avoidance of public utilities

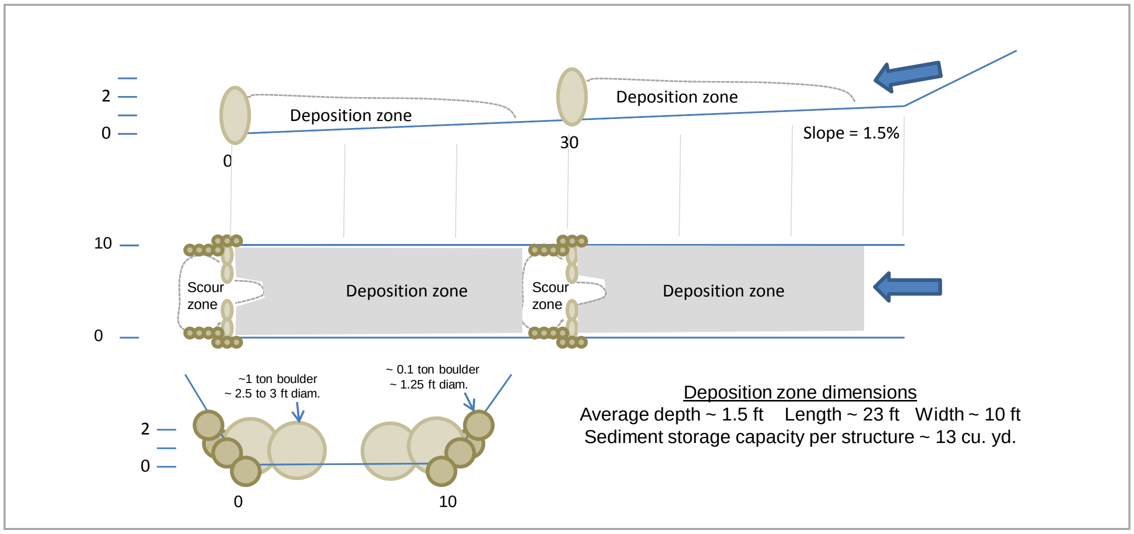


Figure 11-6 Conceptual design of two salmon habitat features at SR1 just upstream of the Shoreline Highway.

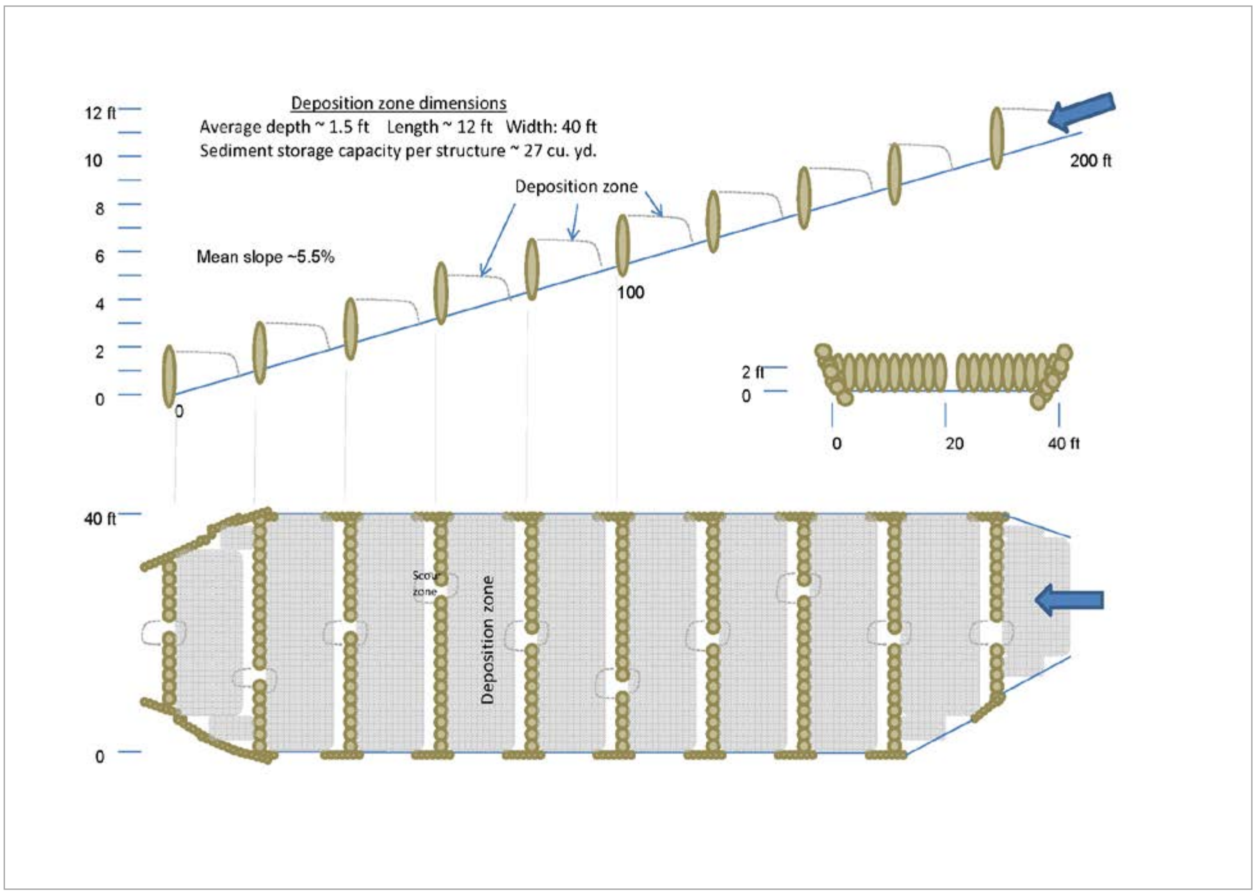


Figure 11-7 Conceptual design of ten salmon habitat features at SR1 upstream of the Matt Davis Trail Bridge at Belvedere Avenue.

- Infrastructure protection (public, private)
  - Fences
  - Bridges
  - Exposed utilities
  - Underground utilities – water, power, sewer, gas, phone, other
  - Retaining walls
  - Lateral culverts, drainage channels
  - Septic systems
- Outreach and education
  - Public and regulatory agency input on draft plans
  - Design adjustments based on comments and State/Federal law
- Risk Management
  - Bank stability – private and public property
  - Lateral bank scour after work installed
  - Over bank flooding due to boulders, log placements, etc.
  - Infrastructure and utilities protection
  - Stability of bridge piers, abutments, gabions, retaining wall (existing)
  - Aquatic species habitat quality
  - Inadvertent excavation of abandoned refuse dumps and/or hazardous materials
- Vegetation Management
  - Considered as stand-alone option, but also a subset of dredging
  - Protect high quality and/or native riparian vegetation
  - Selective trimming and tree removal for equipment access (equivalent to annual District maintenance)
  - Brush and invasive species management
- Acquisition of additional easements
- Permitting mitigations
  - Invasive species management
  - Canopy management
  - Native species enhancement
  - Lateral floodplain wetland enhancements

- Potential need for bypass of surface flow
- Potential need for groundwater management in lower reaches
- Channel diverges from roadways and not always readily accessible with equipment.
- Work under bridges with limited head space
- Sediment removal (dredging) equipment options
    - Traditional methods: Excavator, skid steer, other
    - Skyline yarder as model for inaccessible areas?
    - Manual labor under bridges
    - Manual labor option throughout
    - Vacuum truck – investigate potential
    - Dump truck access for spoils removal
- Spoils storage yard and spoils use/disposal
    - Determine location and ownership: CalTrans, County, or private
    - Haul distance
    - Clean streets constraints
    - Mud-sand-gravel-cobble segregation
    - Value engineering – native or quarried replacement material
    - Gravel-cobble replacement for fisheries mitigation
    - Removal-disposal of secondary material
- Pre-construction biological and archeological surveys
- Construction biological monitoring.
    - Observation – identification of vertebrates
    - Potential need to protect/move species
    - Species of concern
        - Steelhead
        - Coho salmon – state endangered
        - Red-legged frog
        - Other….
- Maintain-enhance variable thalweg
- Maintain existing surface roughness
- Maintain-enhance fisheries structures

  - Large woody debris
  - Boulder fields
  - Spawning gravel
  - Overhead canopy
  - Pool-riffle creation
- Bank stability
- Regulatory and permitting approvals
  - CDFW-Section 1602 Streambed Alteration Agreement
  - CEQA
  - NMFS– Threatened or Endangered Species
    - Critical Habitat for Steelhead
    - Essential Fish Habitat for Coho
  - National Park Service – access
  - Regional Water Quality Control Board
    - Section 401
    - SWPPP
    - Waste Discharge Requirements
  - USACOE
    - Section 404
    - Other Waters
    - Wetlands
  - State of California Coastal Commission

11.5 Permitting Issues

The National Marine Fisheries Service (NMFS) is responsible for protecting populations of steelhead and coho salmon listed under the federal Endangered Species Act (ESA). It is the responsibility of NMFS to ensure that any actions undertaken are not likely to jeopardize the continued existence of any threatened or endangered species or result in the destruction or adverse modification of habitat of such species (ESA 1973). Therefore, individual steelhead and coho salmon, plus the various habitats that they need to complete their life cycles, need to be maintained or improved during the course of any actions with the potential to affect the habitats in which they live (e.g., implementing flood control measures).

Similarly, the California Department of Fish and Game is responsible for protecting steelhead and coho salmon protected under state laws.

The dredging option presented here will need to have safeguards in place to protect both individual fishes present and to reasonably assure that the available habitat continues to meet the functional needs of those fishes, specifically by providing suitable spawning and rearing habitat. Potential habitat enhancement in two reaches (HE1 and HE2, Figure 11-1) upstream of Arenal Avenue has been considered should habitat conditions in the proposed dredge reach be deemed inadequate. The proposed dredging plan affects approximately 2,300 feet of Easkoot Creek, or approximately 60 percent of the stream available to steelhead and coho salmon. New sediment removal structures (SR1 and SR2) are also proposed as part of this alternative.

To protect individual fishes living within the affected areas, the project will need to be staged in discreet, manageable-sized units; a comprehensive water diversion plan will also need to be implemented to provide clean, well-oxygenated water to downstream reaches. However, in the event that large stretches of the work area are naturally dry (lacking surface flow), dredging those dry areas will lower the impacts that the overall dredging operation will have on individual fishes, instream habitats, and the overall ecology of Easkoot Creek. Resident fishes will need to be removed (probably via electro-fishing) and relocated to areas upstream of the dredging operation, and fish exclusions will need to be maintained to prevent re-colonization of work areas during dredging operations. During construction and maintenance activity, biologists will need to be on site to locate and remove fish and other aquatic organisms to an upstream location until the activity is complete and water quality conditions are acceptable.

The proposed dredging scenario can be implemented in such a way that habitat complexity (a key aspect for salmonid rearing) is maintained or enhanced. It is important that the resulting channel be allowed to function as a stream, and not be reduced to a homogenous channel. The dredged channel would be configured to several pool-riffle sequences to provide both spawning and rearing habitat. Proposed sediment management structures upstream of State Highway 1 are designed to maintain a low flow channel consistent with existing conditions to avoid creation of migration barriers.

Primary permitting issues are expected to be related to the potential impacts of channel disturbance during construction and maintenance activity on aquatic organisms, principally protected salmonids. The RWQCB, CDFG, and ACE are expected to have permit authority; NMFS is expected to have input through the ACE permit.

The proposed dredging, particularly the proposed annual or near-annual maintenance dredging, may represent potentially significant environmental impacts necessitating completion of an Environmental Impact Report (EIR) to comply with CEQA. Considering that maintenance dredging, habitat and fish use conditions may vary over time, and that repeated entries to remove sediment are likely, the scope of the EIR should be developed to accommodate changing conditions and changing needs.

11.6 Operation and Maintenance Requirements and Costs

The chief O & M concern for this scenario is removal of sediment on a routine, possibly annual, basis. Each new sediment removal site, including the District site being planned near Calle del Mar, must be designed to include an access road or trail suitable for heavy equipment such as an excavator and dump truck. The threshold for maintenance should be determined in advance, with the objective of preserving a pre-determined minimum sediment storage capacity in the system. In some years, winter flows may be insufficient to produce significant sedimentation at these sites, and there may be no need for excavation. However, it should be expected that annual maintenance will be necessary, and procedures and costs for annual permitting should be planned accordingly. In addition, it should be anticipated that occasional large sedimentation events will occur, and that the scope of sediment removal should expand to include the easement area near Arenal Avenue and the locations at bridges along the Calles. Potential volume of sediment removal at these sites is at least 150 cubic yards. Combined with the 290 cubic yards of sediment storage proposed at SR1 and SR2, maximum annual sediment removal would be about 440 cubic yards.

Costs for excavation and hauling, including contractor costs, are estimated below for both a maximum and average annual condition. Costs for professional supervision of maintenance excavation by a geomorphologist and biologist are included. Contractor and hauling costs are included. Disposal costs or aggregate value of excavated sand and gravel is not included. Annual permitting costs, if any are not included, but annual reporting is included.

Estimated Annual Maximum Maintenance (excavation of 440 cubic yard)

Annual Site Review	$ 3,500
Dredging	$ 17,600
Contractor Overhead	$ 4,700
Monitoring	$ 4,000
Cost Subtotal	$ 29,800
Project Administration @ 10%	$ 3,000
Total Cost Estimate	**$ 32,800**

Estimated Annual Average Maintenance (excavation of 150 cubic yards)

Annual Site Review	$ 3,500
Dredging	$ 7,200
Contractor Overhead	$ 2,200
Monitoring	$ 4,000
Cost Subtotal	$ 16,900
Project Administration @ 10%	$ 1,700
Total Cost Estimate	**$ 18,600**

11.7 Sustainability (Short-term and Long-term)

Watershed erosion processes will continue to produce sediment that will tend to be deposited in the lower reaches of Easkoot Creek. Sedimentation basins are expected to be effective, inducing deposition of gravel and sand transported as bed load, however, sediment not captured by sedimentation facilities is expected to be deposited in dredged areas. The rate of deposition will be substantially reduced by the sedimentation facilities. It is expected that during typical annual flood events extending up to 5 year recurrence interval (approximately), the rate of erosion and sediment transport in the watershed will be relatively low and proposed sediment removal upstream of State Highway 1 will be largely effective resulting in only incremental sedimentation in the dredged channel. Larger flood events (approximately > 5 year recurrence interval) are expected to produce significant erosion and sediment transport in the upper watershed that is likely to cause substantial sedimentation of the dredged channel, mitigated by the sedimentation facilities. The initial installation, if approved, is expected to provide flood control benefits in accordance with the results of recent modeling efforts. Flood control benefits will be degraded when sediment is deposited in the dredged reach of Easkoot Creek, emphasizing the need to maximize upstream sediment removal upstream.

The channel reach under consideration for dredging is located in an urbanized area with a history of disturbance by dredging and construction. As a consequence of channelization and stabilization efforts, the banks and channel elevation appear to be relatively stable and do not appear to be eroding significantly in most areas. Mobilized bed load is therefore delivered mostly from upstream sources, and cannot be effectively controlled at the source.

Effective lifetime and performance results of the proposed dredging cannot be predicted with much certainty. The bed load sediment volume that would completely eliminate flood control benefits of dredging equals the proposed removal volume (about 3100 cubic yards). A flood flow with about a 10-year return period may be capable of moving this volume into the treated area in a single season based on data available for the 2005 flood event. Average seasonal sedimentation is about 125 to 160 cubic yards based on analysis of the historic record. The last mass channel dredging was believed to have been undertaken in 1987, in response to 1986 flood sedimentation. The effective life of that work was on the order of 10 years owing to sedimentation during the floods of 1997 and/or 2005. The size, effectiveness and maintenance of sediment removal sites, as well as District spot dredging will determine the sustainability of flood benefits obtained by dredging. Incremental sedimentation of the dredged reach should be expected, and pulses of sedimentation associated with large storm events that exceed the sedimentation capacity of removal sites may diminish flood benefits more rapidly. It is also possible that in a large storm event, the upstream sediment removal sites may be more effective than has been assumed, and the incremental sedimentation of the dredged channel may proceed more slowly. The potential impact of a peak flow diversion (bypass) below Arenal Avenue may substantially reduce sediment transport capacity to the Calles, and is quantitatively evaluated in “Sediment Transport Evaluation” (Appendix, pp. 99-118).

11.8 Feasibility, Next Steps and Additional Information Needs

- Identification and survey of property lines for parcels adjacent to dredging area is needed to initiate the process of obtaining landowner consent and access for more detailed planning and design.
- Analysis of bank stability of dredged channel will be needed to refine concept plan for dredging, in particular determining proposed bank angles and the location and concept design to maintain bank stability.
- Investigate feasibility of sediment removal (SR2) with affected landowners and key regulatory agency staff. Significant upstream sedimentation capacity is necessary to extend the duration of flood mitigation benefit of dredging. Alternative designs with greater storage capacity should be considered.

12 Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot

NOTE REGARDING TERMINOLOGY: Three alternatives, Alternatives 5, 6, and 9, have had their names changed from those used in previous drafts of this report. Alternative 5, formerly the 'North Bypass' alternative, is now 'Wetland Creation and Bypass to the National Park Service's North Parking Lot.' Alternative 6, formerly 'South Bypass,' is now 'Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.' Lastly, Alternative 9, formerly 'Combination Dredge and South Bypass,' is now 'Combined Dredge, Wetland Enhancement, and Bypass.' These new names were suggested by members of the community with the intent of greater precision and to emphasize the fact that wetland enhancement is a priority of the project.

12.1 Summary

This alternative involves the construction of a bypass channel to divert a portion of the discharge of Easkoot Creek to the ocean during high flow conditions. The proposed diversion point is located on the left bank of the channel opposite the upstream portion of the Parkside Café, and the diverted water flows through a 50-ft wide by 3-ft deep trapezoidal bypass channel, discharging to the north GGNRA parking lot, thence to the ocean (Figure 12-1). The diversion structure is identical to that proposed for Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot (Figure 13-1) and the diversion channel evaluated is functionally similar. This alternative was evaluated to determine potential flood mitigation benefits; however, it became apparent that diverting flood waters to the northern GGNRA beach parking lot adjacent to the residential area on Calle del Pinos had substantial disadvantages with respect to impacts on GGNRA facilities, potential impacts on endangered fish, and uncertainty regarding potential flood impacts of diverted Easkoot Creek flows. These disadvantages relative to Alternative 6 suggest that Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot is substantially less feasible. Consequently, the level of detail developed to describe Alternative 5 was reduced relative to Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.

A detailed cost estimate was not prepared for Alternative 5; however, it shares many of the same cost elements included in Alternative 6. For purposes of comparison and evaluation of flood mitigation alternatives, the cost of Alternative 5 should be considered comparable to Alternative 6 (roughly $1.4 million).

12.2 Description of Alternative

This alternative provides for construction of a bypass channel to divert a portion of the discharge of Easkoot Creek to the ocean during high flow conditions. The proposed diversion would be identical to that developed for Alternative 6 (Figure 13-1) to divert water from Easkoot Creek in the same quantities and at the same times. The diverted flood water, however, would be routed to the northern GGNRA parking lot, where it would pond and flow out to the ocean at the opening in the dunes at the northwestern corner of the GGNRA property. Easkoot Creek floodwater is believed to flow out of the ocean at this location under existing conditions, and an outlet structure to control erosion would probably be necessary. To prevent diverted flood water from flowing back into Easkoot Creek along the northeast edge of the parking area, a berm or flood wall would need to be constructed. This is an essential element of flood mitigation under this alternative that would also prevent fish entrained in the diversion from the migrating back into Easkoot Creek. This alternative also includes a berm or flood wall along the northern edge of the parking lot to prevent potential flooding of residences in the Calles. Although this study does not attempt to address coastal flooding from storm surge, such flooding reportedly affects the GGNRA north parking lot under existing conditions. The effectiveness of flood mitigation as modeled for this alternative (Figure 12-2) could be diminished by coastal flooding that would prevent or inhibit drainage of floodwater diverted from Easkoot Creek; under these conditions it is possible that flooding in the vicinity of the parking lot could become more severe. The potential effect of coastal flooding on the effectiveness of this alternative should be analyzed if this alternative is given further consideration.

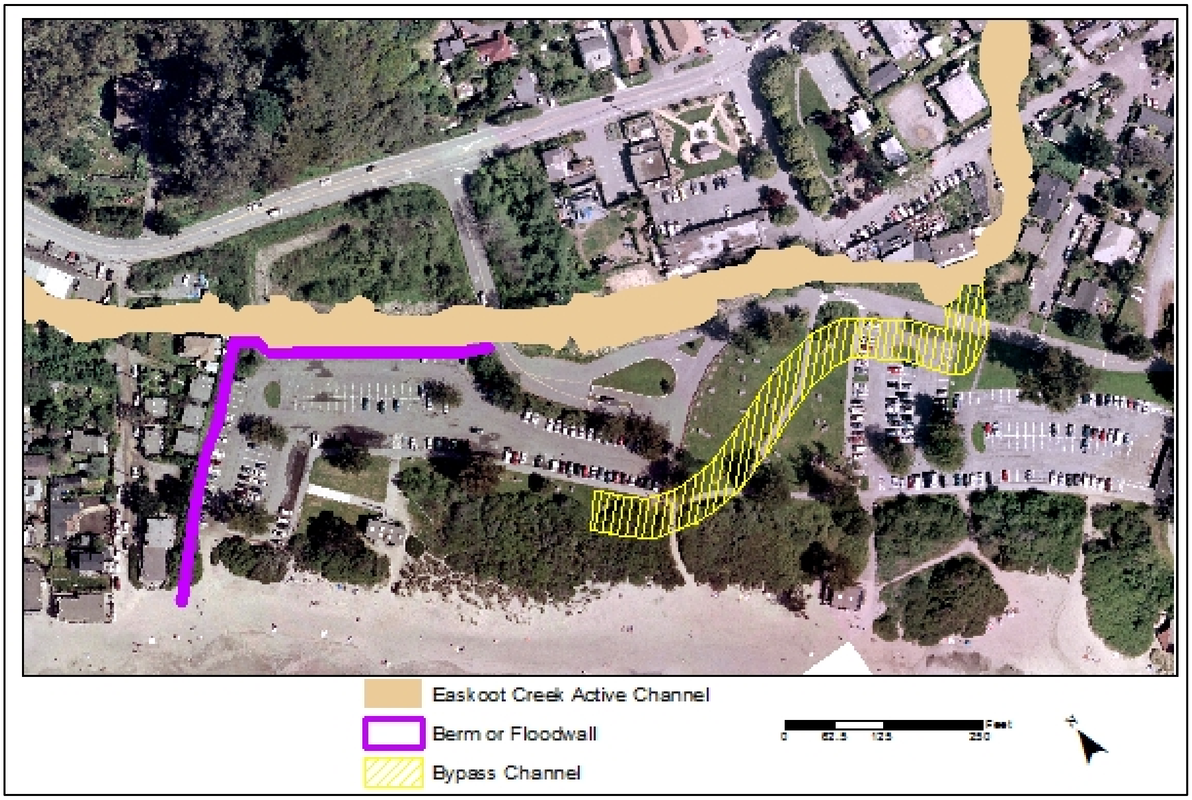


Figure 12-1 Overview map of Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot.

12.3 Flood Control Benefits

The potential flood control benefits of bypassing flows to the north parking lot are substantial. During a December 2005 magnitude flood, the bypass carries up to 72.6 cubic feet per second (cfs) or 42% of the total discharge of Easkoot Creek. Bypassing these flows completely eliminates flooding above the northern GGNRA parking lot (Figure 12-2). Flood extent is reduced somewhat and floodplain depths are reduced between 0.1 and 0.5 feet throughout the Upper Calles. Unlike most of the other alternatives, the bypass does reduce flood extent and floodplain depths significantly (>0.5 feet) throughout the Lower Calles (Figure 12-2). The average reduction in peak water levels in the channel is 0.6 feet in the reach adjacent to the Parkside Café, 0.4 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.4 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table 6-1). Approximately 11 of 24 buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions under this alternative (Table 6-2).

12.4 Preliminary Design and Estimated Construction Costs

The proposed diversion point is located on the left bank of the channel opposite the upstream portion of the Parkside Café. This location was selected because a) it represents the upstream-most location where diversion is practical, b) downstream of this point channel capacity, channel slope, and sediment transport capacity become significantly reduced, and c) it coincides with the location of overbank flows under existing conditions and likely also under historical conditions prior to the development of the GGNRA facilities. These considerations are equally applicable for Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot, which has substantial practical advantages and fewer disadvantages with respect to fisheries.

The proposed diversion structure is a 50-ft wide lateral weir with a crest elevation of 24.8-ft NAVD88. The bypass channel is a 50-ft wide by 3-ft deep trapezoidal channel with a 1:1 side slope. The channel flows south through dual 22-ft wide by 3-ft deep box culverts beneath the existing road leading to the south parking lot, bends to the west and flows through a second set of dual 22-ft wide by 3-ft deep box culverts beneath the existing road leading to the central parking lot, and terminates at the southern end of the north parking lot. The total channel length is approximately 636-ft and the total volume of material that would be excavated to construct the bypass channel is 3,320 cubic yards. The channel sizing is based on the capacity required to convey bypass flows during a 100-yr flood event. The alignment was selected so as to minimize the required modifications to existing GGNRA infrastructure and parking facilities.

In order for the parking lot to function as an effective detention basin, an approximately 350 foot long berm or flood wall would need to be constructed along the northwestern edge of the parking lot to prevent flood waters from moving into residential areas of the upper Calles as well as an approximately 330 foot long berm or flood wall along the bank of the creek at the northeastern edge of the parking lot to prevent flood waters from returning to the creek and exacerbating flooding downstream.

The elevation of the weir crest at the point of diversion of is arguably the most important design parameter as it will exert a strong control on a) the overall effectiveness of the bypass as a flood mitigation measure, b) the frequency that the bypass channel will be active and associated implications for fish movement, c) the flow above which downstream conditions will be altered with associated implications for fish passage, and d) the freeboard available to accommodate sediment deposition at the inlet. For conceptual design purposes, the diversion weir crest elevation was set to 1-ft above the existing channel thalweg (24.8-ft NAVD88). Under existing conditions, stream stage (flow depth) at this elevation occurs at flows of approximately 40 cubic feet per second (cfs). An examination of the flow record at stream gauge EK for seven water years with a nearly complete record from 2002 through 2010 indicates that flows exceeded this threshold between one and four times per year with an average frequency of two events per year.

There are potential limitations associated with design and construction of the necessary facilities that make Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot less feasible than Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot. Alternative 5 would involve impounding floodwaters in a location immediately adjacent to residential areas (the Calles), creating potential for an on-going risk of flooding. Additional design studies would be needed to ensure that containment berms/flood walls were properly sized and constructed to achieve the desired degree of flood risk. The height and design of these barriers might be aesthetically objectionable and inconsistent with GGNRA land-use objectives. Second, this alternative would likely interfere with existing uses (parking and possibly adjacent rest rooms) in the GGNRA; periodic use of the parking lot as a detention basin for floodwaters may require substantial maintenance.

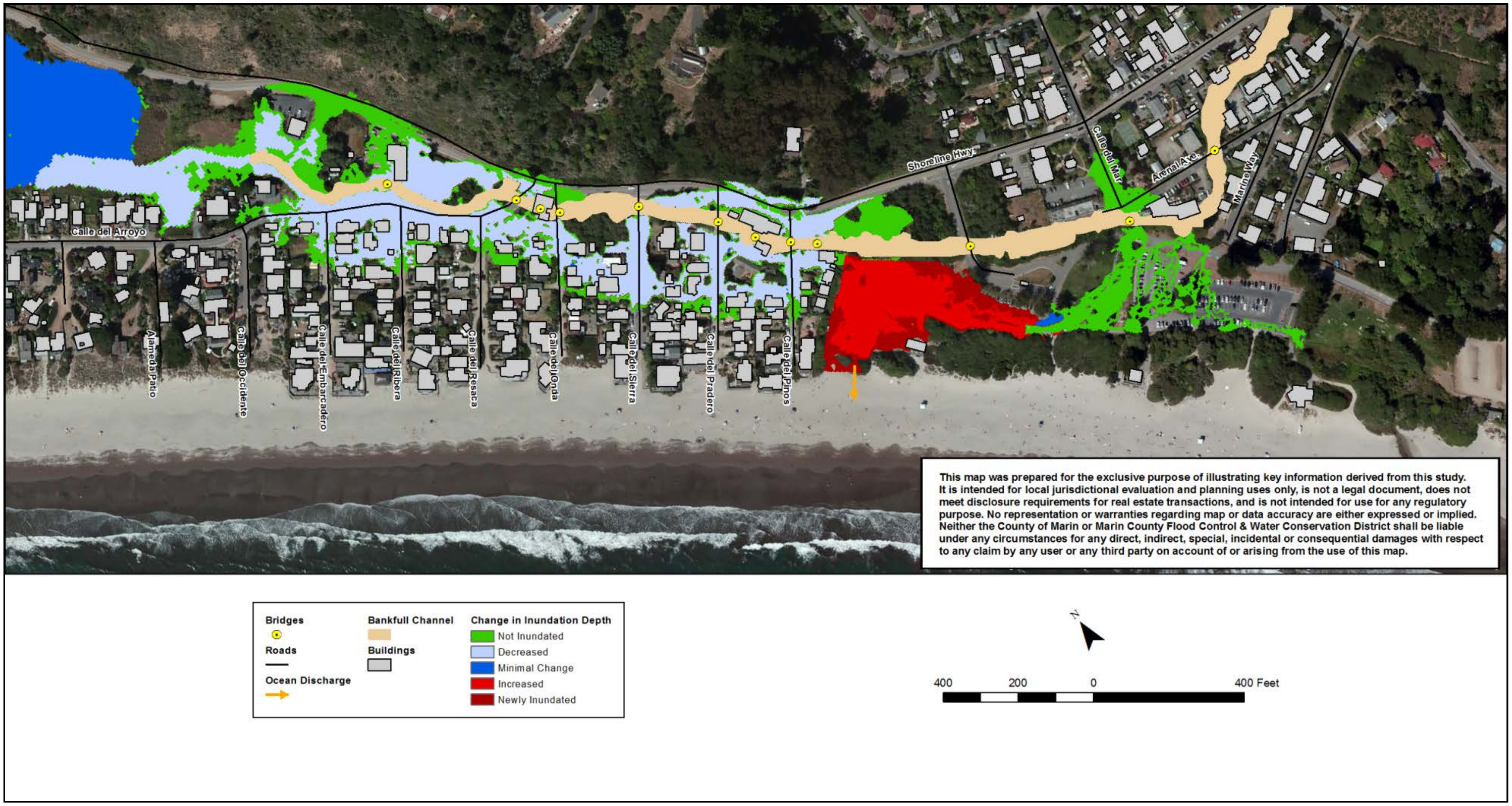


Figure 12-2 Decrease in flood extent and floodplain depths under Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot for the December 2005 flood.

12.5 Permitting Issues

Potential effects of Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot on endangered fish are substantially more difficult to mitigate than for Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot. Fish entrained in the bypass would be routed to an area that would not provide permanent habitat; the total amount of bypass flow would be routed to the ocean in a relatively short time period, so it would be expected that these fish would be exported to the ocean or stranded in the parking lot. In other words, the likelihood of harm to endangered fish is relatively high and difficult to mitigate. It might be possible to design a flow path back to Easkoot Creek, but it would require design elements that would operate at cross-purposes to flood mitigation by allowing diverted flood flows back to Easkoot Creek. Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot is less problematic from design/risk, parking impact, and fisheries perspectives; Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot as modeled has similar flood control benefits, but routes floodwater to an area where it could cause additional flooding, particularly if coastal flooding occurs.

12.6 Operation and Maintenance Requirements and Costs

The flow diversion and bypass system as planned will operate on a passive basis, without active management requirements, with the possible exception of occasional adjustment of a flashboard that may be incorporated in the design of the diversion structure. Periodic inspection by the District will be required, as will periodic maintenance of the inlet weir structure if debris or sediment accumulates at that point. The open channel or culvert conveyance system should have low maintenance requirements if properly designed and installed. Routine excavation of accumulated sediment is likely to be required. Some debris and vegetation management may be required in the GGNRA north parking lot.

The principal operations and maintenance costs of this alternative are expected to be associated with:

- maintaining water conveyance in Easkoot Creek, the diversion structure and the bypass channel at appropriate levels and frequency by monitoring conditions and removing debris and sediment as necessary,
- maintaining seasonal function of the north GGNRA parking lot by removing debris and sediment as necessary, and

- maintaining flood walls/berms and the outfall to the ocean from the parking lot at the northwest corner of the parking lot.

Maintenance activities would likely be required seasonally after the rainy season when sediment and debris deposition at the diversion structure would occur, and after bypass flows have occurred resulting in sediment deposition in the GGNRA parking lot. The diversion structure design would likely include operational parameters regarding the elevation of the stream bed in relation to the elevation of the diversion weir that would guide routine seasonal maintenance activity. In addition, there would need to be provisions for urgent maintenance during and after winter storms when high bypass flows occur and significant deposits of sediment and debris could compromise the function of the passive diversion structure. Permitting for maintenance activity should be incorporated in permitting for the construction phase of the project.

Marin County Public Works Department staff and/or NPS maintenance staff and equipment would likely be identified as the appropriate organization(s) to conduct these maintenance activities. Costs for the necessary personnel and equipment are best known by these organizations. For planning purposes, assuming about five working days for a small crew equipped with a loader/backhoe machine and a dump truck would likely be sufficient to accomplish routine and urgent maintenance. Personnel and equipment costs for this level of effort is estimated to be approximately $8,000 to $10,000 per year.

12.7 Sustainability (Short-term and Long-term)

For the proposed diversion, bypass and flood basin including floodwalls/berms, short-term sustainability is expected to be a function of deposition of sediment and debris from Easkoot Creek and diligent maintenance. There do not appear to be significant obstacles to positive sustainability in the short-term. Properly designed and installed channel/culvert and bridges should have a reasonable 20-year design and economic life. In the long-term, sustainability of the function of flood management structures could be threatened by severe storms causing extreme coastal flooding and/or Easkoot Creek flooding and potential damage to drainage structures by flood flows and/or deposition. Long-term sea-level rise would be expected to increase the frequency and severity of flooding over time. Damage caused by severe storms could require emergency funding as a supplement to annual maintenance budgets.

Efforts to manage sedimentation and maintain channel capacity must be maintained in conjunction with bypass installation.

12.8 Feasibility, Next Steps and Additional Information Needs

Further investigation of this alternative, if pursued, should initially address the following issues:

1. Identify and evaluate means by which fish entrained in the bypass and transported to the vicinity of the GGNRA north parking lot could return to Easkoot Creek.
2. Quantify and evaluate the coastal flooding hazard and analyze the interaction between coastal flooding and Easkoot Creek bypass flood flows.

13 Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot

NOTE REGARDING TERMINOLOGY: Three alternatives, Alternatives 5, 6, and 9, have had their names changed from those used in previous drafts of this report. Alternative 5, formerly the 'North Bypass' alternative, is now 'Wetland Creation and Bypass to the National Park Service's North Parking Lot.' Alternative 6, formerly 'South Bypass,' is now 'Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.' Lastly, Alternative 9, formerly 'Combination Dredge and South Bypass,' is now 'Combined Dredge, Wetland Enhancement, and Bypass.' These new names were suggested by members of the community with the intent of greater precision and to emphasize the fact that wetland enhancement is a priority of the project.

13.1 Summary

This alternative proposes construction of a bypass channel to divert peak flood flows from downstream portions of Easkoot Creek; the majority of flow would remain in the existing stream. Bypass flows of up to 73 cubic feet per second (cfs) (in a December 2005 magnitude event) are routed to a proposed 2.4 +/- acre wetland enhancement and restoration area that includes an existing wetland area of about 1 acre. The restored wetland, so-called Poison Lake, would provide refuge and rearing habitat for salmonids that could be conveyed by flood flows out of Easkoot Creek. This natural habitat refuge for fish entrained in bypass flows is a major advantage of this alternative relative to other bypass alternatives.

The proposed diversion point is located on the left bank (sea-ward) of the channel opposite the upstream portion of the Parkside Café (Figure 13-1). This location was selected because

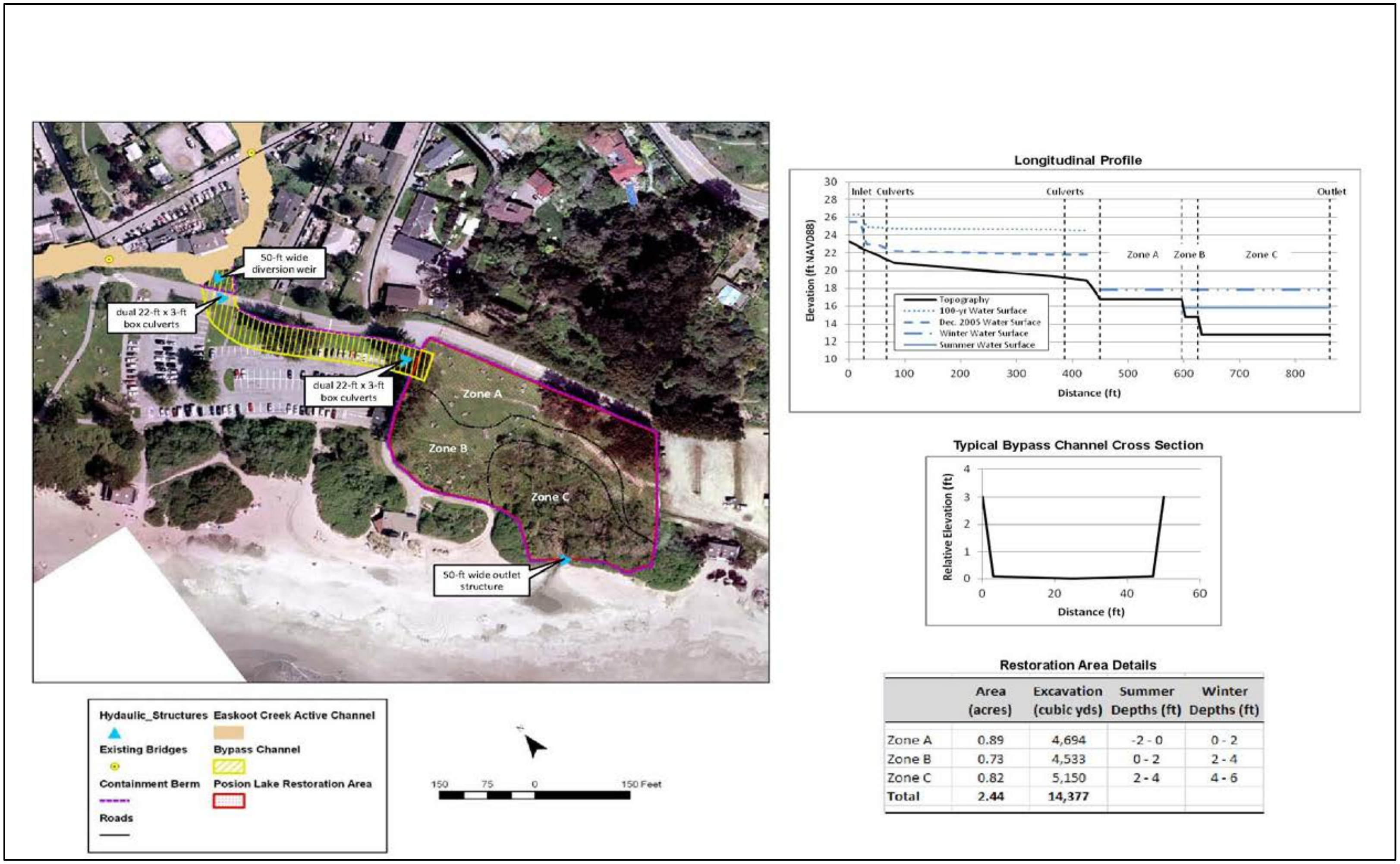
a) it is the upstream-most location where diversion is practical,
b) downstream of this point channel capacity, channel slope, and sediment transport capacity become significantly reduced, and
c) it coincides with the location of overbank flows under existing conditions and likely also under historical conditions prior to the development of the GGNRA facilities.

The proposed diversion structure is a 50-ft wide lateral weir with a crest elevation of 24.8-ft NAVD88; the bypass channel is a 50-ft wide by 3-ft deep trapezoidal channel with a 1:1 side slope (Figure 13-1). The channel flows south through dual 22-ft wide by 3-ft deep box culverts beneath the existing road leading to the south parking lot, bends and flows east along an alignment parallel to the road and north edge of the parking lot, flows through a second set of dual 22-ft by 3-ft culverts beneath the existing road and terminates in the restored wetland area (present-day picnic area). The total channel length is approximately 430 feet (Figure 13-1) and the total volume of material that would be excavated to construct the bypass channel is 2,230 cubic yards. The channel sizing is based on the capacity required to convey bypass flows during a 100-yr flood event. The alignment was selected so as to minimize the required modifications to existing GGNRA infrastructure and parking facilities. The extent to which the proposed diversion avoids GGNRA infrastructure relative to Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot is a major advantage of Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.

The flood control benefits of bypassing flows to a restored Poison Lake are substantial. A major advantage of this alternative is that floodwaters are diverted to an area where relatively little potential flood damage to private property could occur, unlike Alternative 5-Wetland Creation and Bypass to the National Park Service's North Parking Lot. During the December 2005 flood, the bypass carries up to 73 cubic feet per second (cfs) or 42% of the total discharge above the bypass of 172 cubic feet per second (cfs). Flood extent is reduced somewhat and floodplain depths are reduced throughout the Upper Calles; flood extent and floodplain depths are significantly reduced throughout the Lower Calles (Figure 13-2). Peak water levels in the channel are reduced 0.6 feet in the reach adjacent to the Parkside Café, 0.4 feet in the Upper Calles reach, and 0.6 feet in the Lowe Calles reach (Table 6-1). Approximately eleven of twenty-four buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions under this alternative (Table 6-2). During the 100-yr flood, the bypass carries up to 245 cubic feet per second (cfs) or 53% of the total discharge of 463 cubic feet per second (cfs) that reaches the bypass. Minor reductions in flood extent throughout the study area result from bypassing these flows. Floodplain depths are reduced in the vicinity of the Arenal Avenue bridge, Calle del Mar, and throughout the Upper and Lower Calles (Figure 13-3). The average reduction in peak water levels in the channel is 0.5 feet in the Parkside Café reach, 0.4 feet in the Upper Calles, and 0.3 feet in the Lower Calles (Table 6-3). Approximately thirteen of fifty-nine buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under 100-yr flood conditions (Table 6-4).

Table 13-1 Summary cost estimate for Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.

Bypass Alternative - Planning-level Budget Summary	Cost ($)	Percent
Consultant Planning, Permitting and Design Subtotal	176,470	12.7
Construction subtotal	838,430	60.4
Subtotal Contractor Overhead	211,100	15.2
Planning-level Cost Estimate (to nearest $1000)	1,226,000	88.4
Project Administration	61,300	4.4
Land acquisition (if necessary)	100,000	7.2
Installed Project Cost Estimate	**1,387,300**	**100.0**
O&M (not yet evaluated for cost)		



	Area (acres)	Excavation (cubic yds)	Summer Depths (ft)	Winter Depths (ft)
Zone A	0.89	4,694	-2 - 0	0 - 2
Zone B	0.73	4,533	0 - 2	2 - 4
Zone C	0.82	5,150	2 - 4	4 - 6
Total	2.44	14,377		

Figure 13-1 Overview map of Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.

Preliminary design profile and sample cross section for the bypass channel, and a summary table of the expected water depths and excavation volumes in the restoration area.

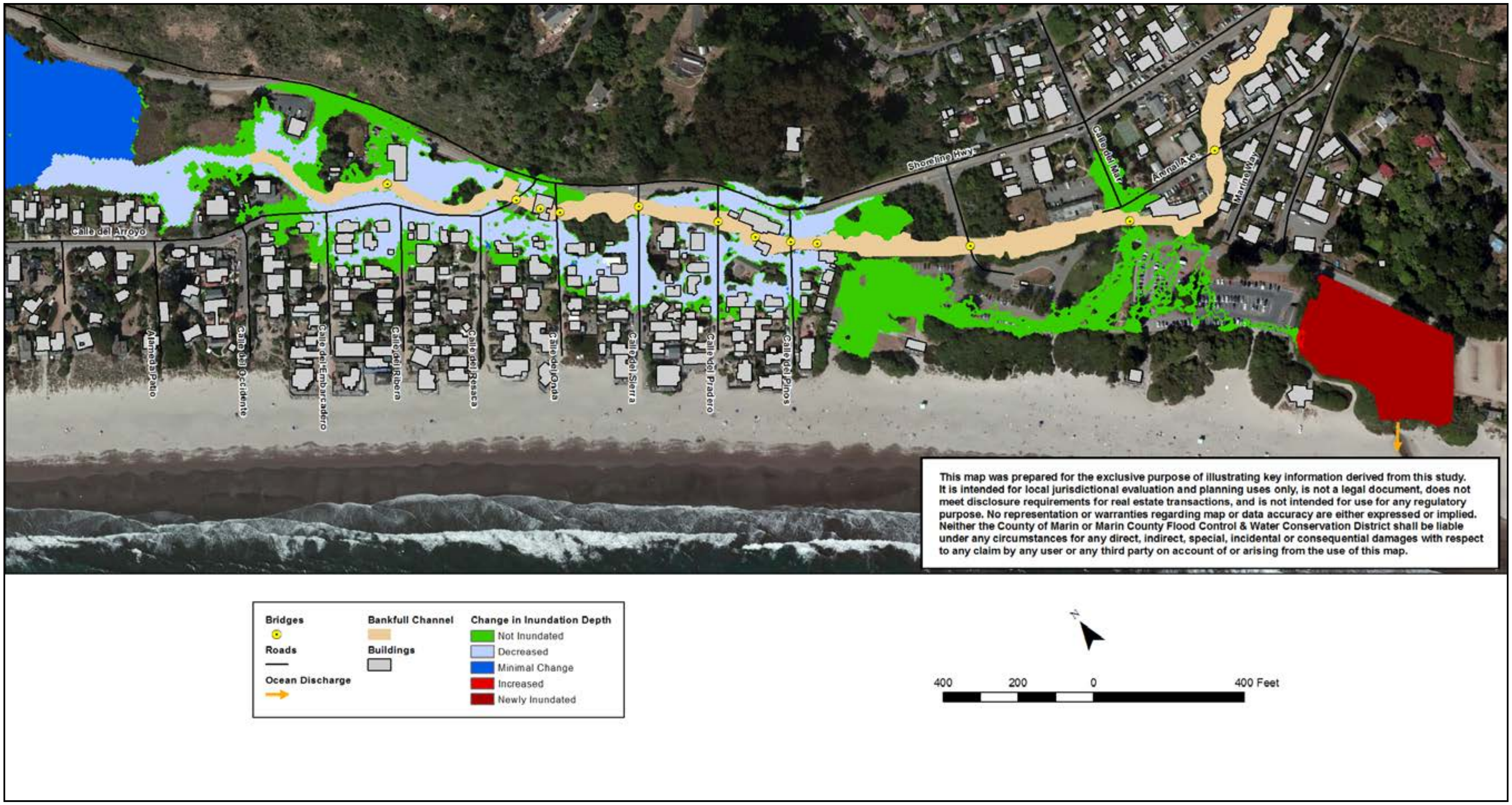


Figure 13-2 Decrease in flood extent and floodplain depths under the Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot for the December 2005 flood.

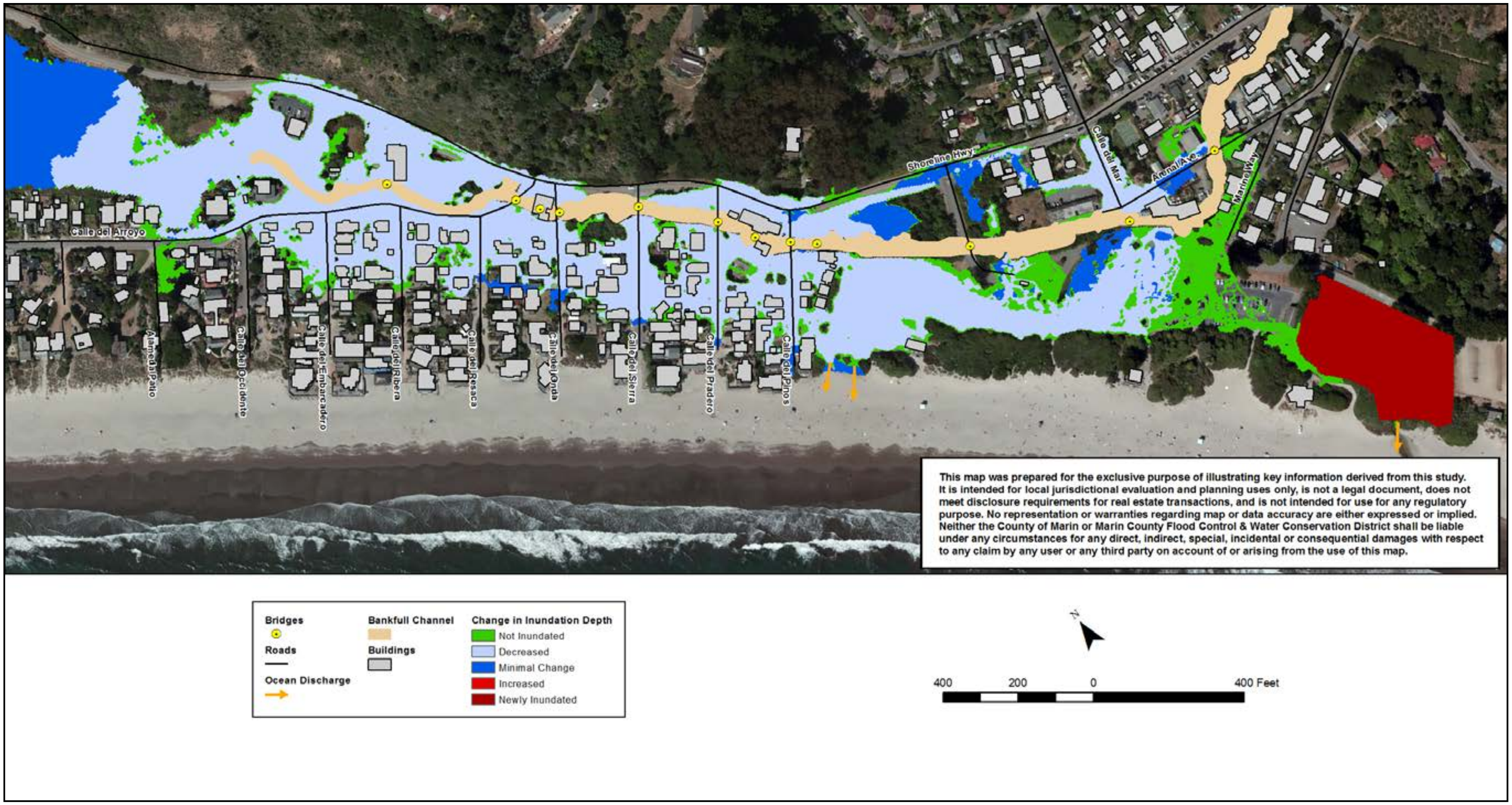


Figure 13-3 Decrease in flood extent and floodplain depths for the 100-yr flood under Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.

13.2 Description of Alternative

This alternative involves construction of a bypass channel to divert peak flood flows from downstream portions of Easkoot Creek; the majority of flow would remain in the existing stream. Bypass flows of up to 73 cubic feet per second (cfs) are routed to a proposed 2.4 +/- acre wetland enhancement and restoration area that includes an existing wetland area of about 1 acre. The present-day wetland is the remnant of the historic Poison Lake which was filled to accommodate park facilities c.1960. The flood routing design proposal seeks to restore a portion of the historic Poison Lake to its former open water habitat while retaining substantial components of vegetated and seasonal wetland. The proposed diversion point is located on the left bank (sea-ward) of the channel opposite the upstream portion of the Parkside Café (Figure 13-1). This location was selected because

a) it is the upstream-most location where diversion is practical,
b) downstream of this point channel capacity, channel slope, and sediment transport capacity become significantly reduced, and
c) it coincides with the location of overbank flows under existing conditions and likely also under historical conditions prior to the development of the GGNRA facilities.

The conceptual design of the bypass is summarized as follows:

- The proposed diversion structure is a 50-ft wide lateral weir with a crest elevation of 24.8-ft NAVD88 (Figure 13-1).
- The bypass channel is a 50-ft wide by 3-ft deep trapezoidal channel with 1:1 side slopes (Figure 13-1). It is sized to handle estimated 73 cubic feet per second (cfs) bypass flows during a 100-yr flood event.
- From the bypass structure, the channel is routed south through dual 22-ft wide by 3-ft deep box culverts beneath the existing road leading to the south parking lot.
- From the culvert, the bypass channel flows southeast along an alignment parallel to the road and north edge of the parking lot, requiring removal of some or all of the landscaping, and a small portion of the existing parking.
- It is then routed through a second set of dual 22-ft by 3-ft culverts beneath the existing road to the snack bar and main life guard tower, and through a line of trees requiring partial removal.

- The discharge point of the bypass channel is into a proposed wetland restoration area (present-day picnic grounds).
- The total channel length is approximately 430 feet (Figure 13-1). The construction excavation volume is about 2300 cubic yards.
- The main channel should include an inset lower-flow segment about 0.7 feet deep and 2 feet wide to facilitate fish passage during periods of flow initiation and flow termination.
- A perimeter berm would contain flood flows during bypass operation (Figure 13-1); the need for this containment and design parameters would depend in part on hydraulics of the outlet structure.
- An outlet structure controlling the flow of water from the wetland to the ocean and providing for emigration of fish would be constructed.

The draft alignment was selected so as to minimize the required modifications to existing GGNRA infrastructure and parking facilities. Value engineering or site utilization considerations may result in modification of the channel alignment or configuration, including relocation of the bypass to lie entirely on public property. The preliminary design used for the hydraulic model places the bypass on public property (GGNRA); however, it may ultimately be necessary to incorporate a portion of the adjacent privately-owned parcel in the bypass facility.

13.3 Flood Control Benefits

The flood control benefits of bypassing flows to a restored Poison Lake are substantial. During the December 2005 flood, the bypass carries up to 73 cubic feet per second (cfs) or 42% of the total discharge above the bypass of 172 cubic feet per second (cfs). Bypassing these flows completely eliminates flooding upstream of the GGNRA north parking lot (Figure 13-2). Flood extent is reduced somewhat and floodplain depths are reduced between 0.1 and 0.5 feet throughout the Upper Calles. Unlike most of the other alternatives, the bypass does reduce flood extent and floodplain depths significantly (>0.5 feet) throughout the Lower Calles (Figure 13-2). The average reduction in peak water levels in the channel is 0.6 feet in the reach adjacent to the Parkside Café, 0.4 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.6 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table 6-1). Approximately eleven of twenty-four buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions under this alternative (Table 6-2).

During the 100-yr flood, the bypass carries up to 245 cubic feet per second (cfs) or 53% of the total discharge of 463 cubic feet per second (cfs) that reaches the bypass. Minor reductions in flood extent throughout the study area result from bypassing these flows. Floodplain depths are reduced by 0.1 to 0.5 feet in the vicinity of the Arenal Ave. bridge and the intersection of Calle del Mar and Arenal Ave (Figure 13-3). Floodplain depths are reduced by 0.25 to 0.75 feet throughout the Upper Calles and by 0.25 to 0.50 feet throughout the Lower Calles (Figure 13-3). The average reduction in peak water levels in the channel is 0.5 feet in the Parkside Café reach, 0.4 feet in the Upper Calles, and 0.3 feet in the Lower Calles (Table 6-3). Approximately thirteen of fifty-nine buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under 100-yr flood conditions (Table 6-4).

13.4 Preliminary Design and Estimated Construction Costs

The essential elements of the preliminary proposed design are described above. Additional details and supplemental design and planning considerations are presented in this section.

Bypass Weir. The lateral weir crest elevation at point of discharge from the creek is a critical design parameter. It will strongly influence:

a) overall effectiveness of the bypass as a flood mitigation measure,
b) frequency of flows into the bypass channel and associated implications for fish movement and sediment transport,
c) the flow threshold at which downstream conditions will be altered and associated implications for fish movement and sediment transport, and
d) the freeboard available to accommodate potential sedimentation at the inlet.

For conceptual design purposes, the diversion weir crest elevation was set to 1-ft above the existing 2013 channel thalweg (24.8-ft NAVD88). Under existing conditions, stream discharge of approximately 40 cubic feet per second (cfs) is necessary to raise the stream stage to the point where flow to the bypass would occur. An examination of the flow record from the Park Service gauge on Easkoot Creek for the seven water years with a nearly complete record from 2002 through 2010 indicates that flows exceeded this threshold between one and four times per year with an average frequency of two events per year.

The diversion weir crest must remain at a constant elevation over time relative to the stream channel bed. This is necessary to ensure uniform bypass channel system performance. Measures must therefore be taken to stabilize the creek channel invert at this point, to prevent aggradation or degradation which would change the set point at which lateral flow can occur. The potential for sedimentation during storm events is substantial, and further analysis is required to evaluate performance and design of the bypass in relation to sedimentation (see additional discussion below). If dredging occurs in the main channel prior to bypass channel construction, an appropriate adjustment in weir elevation would need to be made.

One method for stabilizing the Easkoot Creek channel invert and stream stage in relation to the bypass weir elevation at this location would be to install a concrete or boulder weir across the channel just downstream of the lateral weir. This would serve to maintain a stable and fairly level channel invert and would ensure that the lateral bypass channel would function as intended. The structure could be configured as a roughened ramp suitable for fish passage. Confinement of flow in the section of Easkoot Creek just upstream of the bypass could prove critical to the performance of the bypass, and final designs may require some additional bank structures to contain stream flow as it approaches the bypass. Finally, the lateral weir could be fitted with flashboards that would allow for adjustment of the weir height to compensate for sedimentation adjacent to the weir.

Sedimentation of Bypass Facilities. Coarse bed load sediment in transport in Easkoot Creek at the bypass is not likely to be carried into the bypass channel given the 1 foot elevation difference (as modeled) between the bed of Easkoot Creek and the lateral bypass weir. Gravel and cobble would likely continue to move downstream, although at a reduced rate downstream of the bypass. The geometry of the channel adjacent to and immediately downstream of the bypass may need to be modified to better maintain sediment transport and reduce sedimentation in the bypass that could affect bypass performance. Provisions to accommodate anticipated sedimentation (potentially on the order of hundreds of yards); including additional or expanded sedimentation facilities may be required. Potential sedimentation at the bypass will require additional consideration in future feasibility and design studies. Suspended sediment load consisting of sand, silt, and clay would likely be transported by flow over the bypass weir. Some deposition would likely occur in the bypass channel owing to low gradient and relatively high width of the channel. Sedimentation in both the bypass channel and Easkoot Creek would need to be monitored and removed on a regular basis to ensure adequate capacity and satisfactory performance and conveyance and to prevent excessive sedimentation of the restored Poison Lake.

Bypass Channel. It would be possible to construct an open bypass channel that mimics a natural system, by lining it with cobble and boulders and/or grass cover. The gradient is low enough and flow depth shallow enough that channel erosion potential is limited thus permitting a variety of channel design options. A cobble/boulder configuration would have higher friction and greater trapping capability and so might need to be made a little wider or deeper in order to retain the desired capacity rating. It may be necessary to include sedimentation structures within the bypass channel to reduce the quantity that might reach the restored wetland area. A narrow low flow channel section inset on the floor of the bypass channel would likely be added to reduce potential for fish stranding. Construction costs would be increased with this configuration due to use of more materials that require greater labor for installation.

Outlet Structure from Restored Wetland. The area south and east of the proposed bypass channel discharge point consists of a picnic area lawn that transitions into a vegetated wetland area containing ponded water. Runoff from local area sheet flow as well as spring or groundwater flows exit the area through a gap in the sand dunes that contains an historic control structure (a hardened sill perforated by a 12 to 18 inch diameter CMP). The precise age, design and purpose of this structure is unknown, but may date to construction of the south overflow parking lot and the filling of Poison Lake. At present, it provides grade control, preventing seepage erosion of the sandy soil at the point of channel discharge to the beach. The outlet structure for the restored wetland would likely remain in this location; however, a new structure would be designed to provide some detention of bypass flows, to maintain ponded water at the desired elevation, and to provide suitable depth and velocity of flow for emigrating anadromous fish. The model design assumes a 50 foot wide broad crested weir; the ultimate configuration would be determined by subsequent design work. A notch in the weir that would contain lower flows would likely be incorporated in this outlet structure to better accommodate base flow and fish emigration.

Alternative Bypass Channel Configurations. The initial bypass configuration (Figure 13-1) maximizes vehicular access and parking by elimination of an existing landscaped area with mature trees and underground utilities. Alternatively, the landscaped area could be retained, and the 50' wide channel routed through the parking lot. It may be possible to retain some dual purpose functionality (at additional cost) by routing an enclosed channel under the parking lot. An alternative that would maximize parking and landscaping at the expense of local roadways would be to route the channel down the existing road at the north side of the parking lot. Since the channel is about twice the road width, some loss of landscaping would be inevitable. In addition, the preliminary design is conservative with respect to conveyance capacity providing 1 foot of freeboard for the 100-yr flow event, and it might be possible to modify the channel width to accommodate different objectives.

Consideration could be given to using multiple culvert bores in lieu of an open channel to route bypass flows to the wetland area. Multi-bore culverts may be less expensive and more aesthetic than an open channel bypass. Using culverts would allow cut and cover of the bypass waterway, minimizing conversion and disruption of parking and roadway areas. It may be possible to split tubular flows around the landscaped island, thus eliminating the 50' channel width constraint and preserving more of the existing landscaping. This design alternative would need to consider suitability for the anticipated fish use, expected to be limited to involuntary emigration from Easkoot Creek to the restored wetland during a flood bypass event. Culvert inlet conditions and head constraints would need to be considered as part of the design, to ensure design flows can be handled.

Poison Lake Restoration Design Factors. Many options are available in terms of the extent of the wetland restoration and enhancement area (so-called Poison Lake) and the desired wetland and habitat features. This conceptual design plan seeks to minimize the impacts to existing GGNRA parking facilities and infrastructure, although substantial infrastructure impacts would occur. The footprint of the Poison Lake restoration area follows the boundaries of the south picnic area and small existing wetland (Figure 13-1).

Open water habitat area would be restored at the lower end of the area by constructing a slightly elevated weir at the location of the present-day culvert outfall. The open water habitat would provide refuge and rearing habitat for any fish carried downstream into the bypass channel. The proposed weir crest elevation matches the local winter water table elevation. Upon lake-fill to the design elevation of 18.4' NAVD88, inflow and outflow volumes will be equivalent, allowing the lake to function as a flow-through basin with limited accumulation of water.

The conceptual design includes a 3-ft high berm surrounding the perimeter of the lake (Figure 13-1). This berm is not strictly necessary because both inflow and outflow structures are designed to accommodate the maximum flows expected from the bypass. The berm serves as a safeguard for residential areas in the unlikely event that water overtops the system and escapes via the parking lot access road to the north.

At flood stage conditions, lake water elevation will match that of the outlet weir plus surcharge necessary to achieve flood flows over the weir. Under no-flow conditions, the water surface elevation will fluctuate in accordance with the local shallow groundwater profile. Lake depth will be a function of the water surface elevation relative the degree of excavation that takes place within the restoration footprint.

Expected fluctuations in groundwater elevations are based on data from ten wells within the restoration footprint and an additional eleven wells in other areas of the park collected by NPS between November 2003 and May 2011. The number of water table elevation observations in individual wells ranged from nine to thirty-seven, but in all cases included both dry season and wet season measurements. Examination of these data indicates that in the northern portion of the restoration area, groundwater elevations range between three and four feet below ground during the late summer and fall and two to three feet below ground during winter and early spring. In the southern portion of the restoration area on the landward side of the dunes, groundwater elevations range from near land surface in the late summer and fall to one to two feet above ground in the winter and early spring.

Based on the existing topography and the spatial and temporal variations in groundwater elevations, we have delineated three zones within the restoration area. These zones are designed to provide a variety of habitat features and water depths in the restoration area while minimizing the required excavation. Based on the groundwater elevations, we have calculated design elevations for each zone that are expected to provide a seasonal range of desired water depths (Figure 13-1).

Zone A encompasses 0.89 acres in the higher northern portion of the restoration area and represents a zone of shallow water depths. The design elevation for this zone is 16.8-ft NAVD88. Under flood flow conditions, the design elevation would temporarily increase to 18.4 feet, with depth of inundation about 1.5 feet. Under non-flood conditions, it is expected to be a seasonal wetland area that is typically dry during the summer months.

Zone B encompasses 0.73 acres in the central portion of the restoration area and represents a perennial wetland zone with intermediate water depths. The design elevation for this zone is 14.8-ft NAVD88. In summer months water depths are expected to range from zero to two feet. Under flood flow conditions, the design elevation would temporarily increase to 18.4 feet with a water depth of about 3.6 feet.

Zone C is a 0.82 acre perennial pond in the lower restoration with a design elevation of 12.8 feet, summer water depths ranging from two to four feet, and winter depths ranging from four to six feet. Under flood flow conditions, the design elevation would temporarily increase to 18.4 feet.

The total restoration area covers about 2.44 acres, with average required excavation depths of about three to four feet in all zones. The total volume of required excavation is on the order of 14,400 cubic yards.

Construction-General. The bulk of construction work for this project involves standard grading and drainage activities. Standard earthwork activities are required for creation of the bypass channel. The work would occur in a developed semi-urban area, requiring relocation of substantial undergrounded utilities. Depending on the final configuration, partial or complete removal of selected trees, landscape, curb and gutter, roadway, and parking facilities will be required. Means for maintaining vehicular access during construction will be necessary.

A significant amount of concrete work is required for construction of the bypass weir and bridge-type box culvert crossings. The bypass weir may ultimately require additional elements to reliably accommodate flows, specifically sidewalls to contain flows approaching and passing the weir and to reduce sedimentation adjacent to the weir. Furthermore, the bypass structure will likely need to be integrated with sedimentation facilities immediately upstream and downstream. These more detailed design elements are beyond the scope of this conceptual design plan; should a more elaborate bypass facility with provisions for sedimentation be required, substantial additional cost would be expected.

Poison Lake restoration would require significant excavation with a large portion at or below local groundwater elevation. Special construction techniques will be required, and a suitable site for disposal of spoils will need to be identified. The need for the berm portrayed in Figure 13-1 has not been established, and there are many possible methods of construction. The stability of the dunes located seaward of the restored wetland when the system is operating at maximum flow through will need to be determined. This would be addressed by a geotechnical engineering study during a subsequent phase of design and feasibility.

13.5 Permitting Issues

The chief regulatory issues associated with this alternative pertain to listed salmonids (steelhead trout and coho salmon) and wetlands. The proposed Poison Lake restoration would require substantial modification of the existing wetland area that is a remnant of historical Poison Lake. A small pond supported by seepage flows (likely from the Easkoot Creek watershed) with some emergent wetland vegetation and dense woody riparian vegetation currently exists, which spills via a culvert to the beach into the high tide surf zone. Proposed excavation for Poison Lake restoration would likely impact the existing wetland area, however, little wetland fill is expected. Federal permits associated with wetlands would be handled by the US Army Corps of Engineers, and this will provide the nexus through which a Biological Opinion (BO) addressing fisheries impacts would be developed. The BO would fall under the purview of the National Marine Fisheries Service.

One of the primary objectives of this alternative is to mitigate risk to juvenile salmonids associated with bypass flows. Under existing conditions, as well as many of the alternatives considered, bypass flows occur in an uncontrolled fashion as flood flows spill from Easkoot Creek into the GGNRA parking lot. The Poison Lake diversion option would provide a flood bypass channel leaving Easkoot Creek near the Parkside Café and conveying water south, to a re-created wetland impoundment situated at the top of the beach, near the historical location of Poison Lake. The restored wetland habitat in Poison Lake is expected to be of higher quality than the existing wetland habitat currently present in lower Easkoot Creek.

The proposed bypass and restoration of Poison Lake is intended to provide suitable rearing habitat for juvenile salmonids that could be entrained by bypass flows. This alternative is not expected to affect the immigration of adult salmonids or spawning success. Juvenile salmonids are subject to entrainment by storm flows and subsequent displacement from Easkoot Creek into the restored area. Depending upon the timing, duration, and the magnitude of the flows captured during storm events, diverted fish may move directly through Poison Lake to the ocean, or they may reside in Poison Lake until subsequent storms provide suitable outflows for emigration. Because it is not likely that diverted fish will be able to swim from restored Poison Lake upstream through the bypass and back into Easkoot Creek, it is possible that some fish would remain in Poison Lake over the summer dry season.

Therefore, the Poison Lake diversion option presented here will need to have safeguards in place to both protect individual fishes present and reasonably assure that available habitat will continue to meet the functional needs of fish over time by providing adequate rearing habitat and a suitable emigration route to the ocean. Some degree of monitoring (e.g., water quality monitoring and surveys of the number of fish diverted), and possibly intervention (e.g., relocating trapped fish), may be required by resource agencies if this option is pursued.

Following restoration, Poison Lake is expected to range from 0-6 feet deep during the winter and from 0-4 feet deep in the summer, with a spillway allowing for discharge directly onto the beach. This will effectively create a partially closed lagoon system, similar to many small coastal lagoons along the California coast that could provide valuable rearing habitat for juvenile salmonids if water quality remains good (i.e., dissolved oxygen remains adequately high and temperature remains adequately low), predation does not decimate the diverted fish, and fish can emigrate to the ocean.

Per the proposed alternative, Poison Lake will essentially re-establish a historical permanent wetland feature and outflow channel. This small perennial lake is expected to maintain a maximum depth of about four feet deep during the summer months. Steady inputs of cool groundwater, shading from adjacent trees, moderate ambient temperatures, and persistent coastal fog during the summer months should keep the water temperatures suitable for juvenile steelhead that may end up rearing in Poison Lake through the summer. Aquatic vegetation is expected to be quickly established and colonized with aquatic insects (therefore forage for fish should not be a limiting factor). Aquatic vegetation will help keep the water well oxygenated during most of the year, but may contribute to the reduction of oxygen during some periods. Prolonged periods of coastal fog can reduce photosynthesis of aquatic vegetation to the point where the plants consume more oxygen via respiration than they produce by photosynthesis, thereby reducing the dissolved oxygen in the water to potentially stressful or lethal levels for fish. Also, inputs of salt water during storm surges can kill off aquatic vegetation and cause reduction of dissolved oxygen as the dead plant material decomposes.

Fish rearing in Poison Lake will also be subject to predation by birds. Because water depths are expected to be 2-4 feet deep during most of the year, and deeper during the winter, rearing salmonids should be able to escape large-scale predation from wading birds (e.g., herons and egrets), but may be vulnerable to predation by swimming birds (e.g., mergansers and cormorants).

Finally, creating conditions which allow diverted fish to continue their journey to the ocean will be essential for allowing them to successfully complete their life history. Poison Lake will be built with an outfall weir that discharges storm water directly onto the beach. The weir should be notched to concentrate the water flowing to and over the beach, giving out-migrating fish the best chance for crossing the beach at any flows. The actual length of beach that the fish will have to cross will depend upon the tidal stage during the storm, and may range from just a few feet to a couple hundred feet.

Significant additional feasibility and design studies would be necessary for the Poison Lake restoration effort. GGNRA has previously contemplated restoration of Poison Lake. Based on available information, it appears that sufficient water would be available (via seepage from the alluvial fan of Easkoot Creek), and that an outfall structure could be designed to accommodate fish passage (e.g. using California Department of Fish and Wildlife Salmonid Habitat Restoration Manual Part XII: Fish Passage Design and Implementation). Because the exact ecological conditions that will be created under this scenario are somewhat uncertain, a monitoring program should be established to measure the habitat conditions as well as the number and welfare of any fish diverted into restored habitats. This monitoring program should emphasize regular water quality parameters (i.e., dissolved oxygen, water temperature, and salinity) measured both near the surface and lower in the water column in deeper portions of the pool. Visual surveys (i.e., snorkeling) should be conducted after storms and into the summer in order to determine the number and species of fish diverted, and their fate. Observations on birds and other predators should also be made regularly. If conditions for the survival of salmonids are determined to be unsuitable, the resource agencies may require the capture and relocation of entrained salmonids (back to Easkoot Creek).

13.6 Operation and Maintenance Requirements and Costs

The flow diversion and bypass system as planned will operate on a passive basis, without active management requirements, with the possible exception of occasional adjustment to a flashboard that may be incorporated in the design of the diversion structure. Periodic inspection by the District will be required, as will periodic maintenance of the inlet weir structure if debris or sediment accumulates at that point. The open channel or culvert conveyance system should have low maintenance requirements if properly designed and installed. Routine excavation of accumulated sediment is likely to be required. Some debris and vegetation management may be required.

The principal operations and maintenance costs of this alternative are expected to be associated with:

- maintaining water conveyance in Easkoot Creek, the diversion structure and the bypass channel at appropriate levels and frequency by monitoring conditions and removing debris and sediment as necessary,
- maintaining seasonal function of GGNRA facilities in Zones A and B lot by removing debris and sediment as necessary, and
- maintaining flood walls/berms and the outfall to the ocean from Poison Lake.

Maintenance activities would likely be required seasonally after the rainy season when sediment and debris deposition at the diversion structure would occur, and after bypass flows have occurred resulting in sediment deposition in the portion of the GGNRA picnic area located within the flood detention basin (Zones A and B, Figure 13-1). The diversion structure design would likely include operational parameters regarding the elevation of the stream bed in relation to the elevation of the diversion weir that would guide routine seasonal maintenance activity. In addition, there would need to be provisions for urgent maintenance during and after winter storms when high bypass flows occur and significant deposits of sediment and debris could compromise the function of the passive diversion structure. Permitting for maintenance activity should be incorporated in permitting for the construction phase of the project.

Marin County Public Works Department staff and/or NPS maintenance staff and equipment would likely be identified as the appropriate organization(s) to conduct these maintenance activities. Costs for the necessary personnel and equipment are best known by these organizations. For planning purposes, assuming about five working days for a small crew equipped with a loader/backhoe machine and a dump truck would likely be sufficient to accomplish routine and urgent maintenance. Personnel and equipment costs for this level of effort is estimated to be approximately $8,000 to $10,000 per year.

13.7 Sustainability (Short-term and Long-term)

For the proposed diversion, bypass and flood basin including floodwalls/berms, short-term sustainability is expected to be a function of deposition of sediment and debris from Easkoot Creek and diligent maintenance. There do not appear to be significant obstacles to positive sustainability in the short-term. Properly designed and installed channel/culvert and bridges should have a reasonable 20-year design and economic life. In the long-term, sustainability of the function of flood management structures could be threatened by severe storms causing extreme coastal flooding and/or Easkoot Creek flooding and potential damage to drainage structures by flood flows and/or deposition. Long-term sea-level rise would be expected to increase the frequency and severity of flooding over time; the ongoing study of coastal flooding is expected to characterize this effect. Damage caused by severe storms could require emergency funding as a supplement to annual maintenance budgets.

Efforts to manage sedimentation and maintain channel capacity must be maintained in conjunction with bypass installation.

13.8 Feasibility, Next Steps and Additional Information Needs

- Confer with NPS regarding feasibility of reconfiguring road, parking and other affected facilities on GGNRA property.
- Confer with NPS, CDFW, NMFS, RWQCB, ACOE regarding grade control weir in Easkoot Creek specifically, and regarding the wetland restoration and enhancement project as a whole.
- Evaluate sediment transport and sedimentation characteristics of the bypass channel, the bypass weir, and Easkoot Creek in the vicinity of the bypass in greater detail to determine additional design constraints relating to sedimentation.
- Assess land requirements for bypass facility and sedimentation basins.
- Develop revised design plan (30% complete) and revised cost estimate.
    - Develop more detailed knowledge of undergrounded infrastructure in bypass route.
    - Geomorphological assessment of soils in bypass route and dunes seaward of restoration area.

- Conduct topographic survey of weir location, channel routes, and wetland restoration and enhancement area.

14 Alternative 7-Causeway

14.1 Summary

This alternative involves the construction of a ~400-ft long causeway over Bolinas Lagoon to connect State Highway 1 with Seadrift Road (Figure 14-1). Construction of the causeway would greatly improve vehicle access during flood events and result in improved safety for the Seadrift community. To investigate potential flood mitigation in the lower Calles by controlling tidal conditions in the upper estuary, a tide gate structure and pump station were included in hydraulic modeling of the causeway alternative. The concept is that the tide gate and pumps would operate to lower the downstream tidal condition during flood events on the creek in order to reduce backwater effects in the lower reaches of Easkoot Creek.

Water levels in the estuary adjacent to the proposed causeway are controlled by coastal storm surge, tides, and runoff from Easkoot Creek and other tributaries. A feasibility assessment and preliminary design for the causeway cannot be completed until a coastal flood hazard evaluation has been completed.

Construction of a causeway including a tide gate and a pump station reduced maximum water levels in the estuary immediately upstream of the causeway by two feet (by design). This effect diminishes moving up the estuary; at Francisco Patio the reduction is approximately one foot. Above this point the water elevation and flooding becomes increasingly dominated by flows from Easkoot Creek: at Calle del Occidente, the reduction is less than 0.5 feet and is less than 0.1 feet at Calle del Arroyo (Figure 14-2). These reductions only result in minor decreases in flood extent and one building having reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions (Table 6-2).

It is important to note that this analysis was only performed for Mean Higher High Water (MHHW) tidal conditions. Under more extreme tidal conditions such as occurred during the historical December 2005 flood, water overtops Calle del Arroyo and floods portions of the Patios. Under these conditions lowering water levels in the estuary via a tide gate and pump station would likely have significant flood control benefits. Evaluating potential mitigation of coastal flood hazards by regulating estuary water levels at the causeway appears to be warranted but is beyond the scope of this study.

Although access to Seadrift would be greatly improved under this alternative, construction of a causeway alone may not improve access for residents in the lower Calles and Patios depending on the extent of flooding on Calle del Arroyo.

The preliminary estimate of cost for implementation of this alternative is in Table 14-1 below.

Table 14-1 Summary cost estimate for Alternative 7-Causeway.

Causeway Alternative--Planning-level Budget Summary	Cost $	Percent
Permitting and Design Subtotal	196,470	6.5
Construction Subtotal	2,150,318	71.0
Contractor Overhead Subtotal	539,070	17.8
Planning-level Cost Estimate (to nearest $100)	2,885,900	95.2
Project Administration	144,300	4.8
Installed Project Cost Estimate	3,030,200	100.0

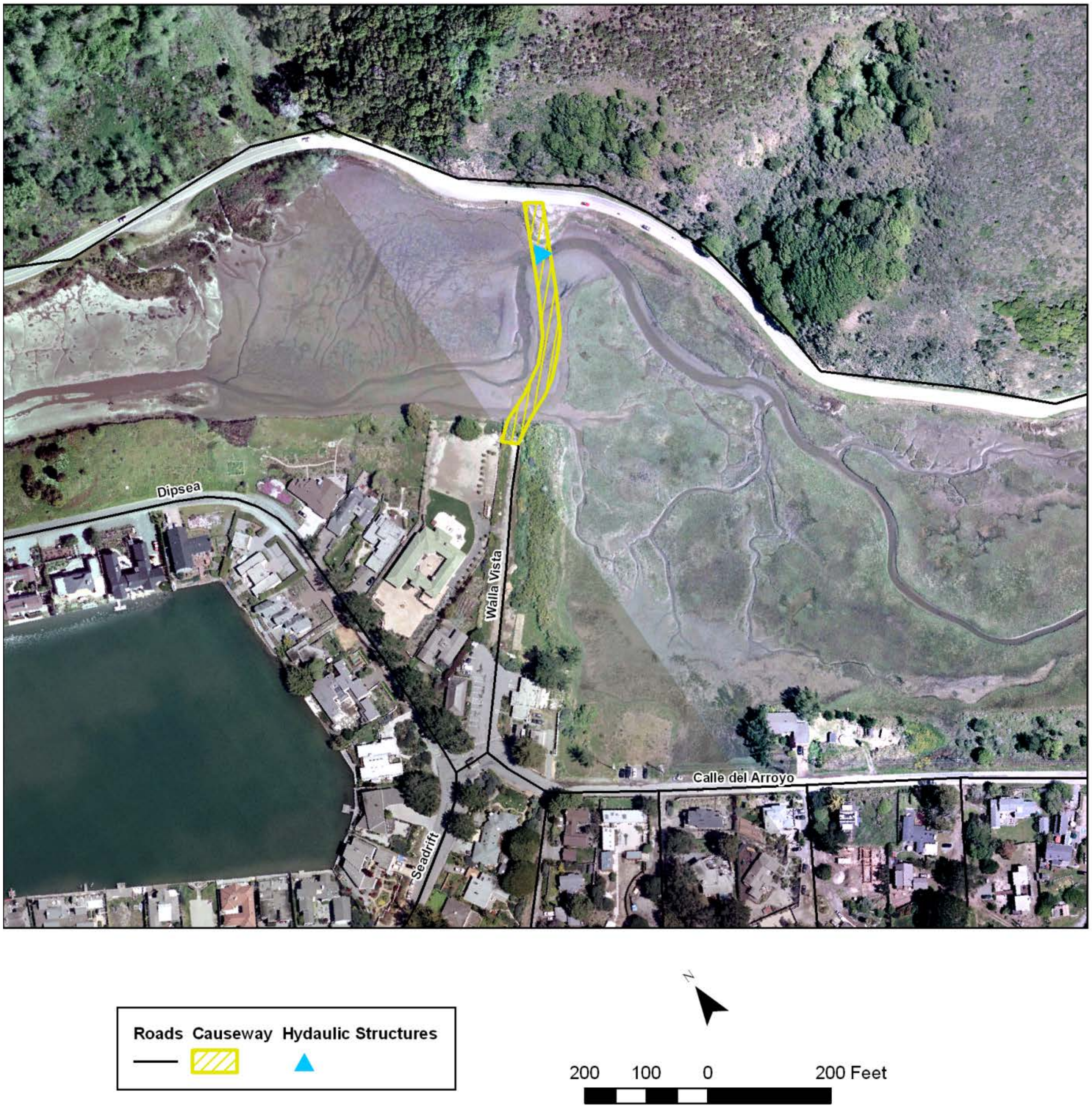


Figure 14-1 Overview map of Alternative 7-Causeway.

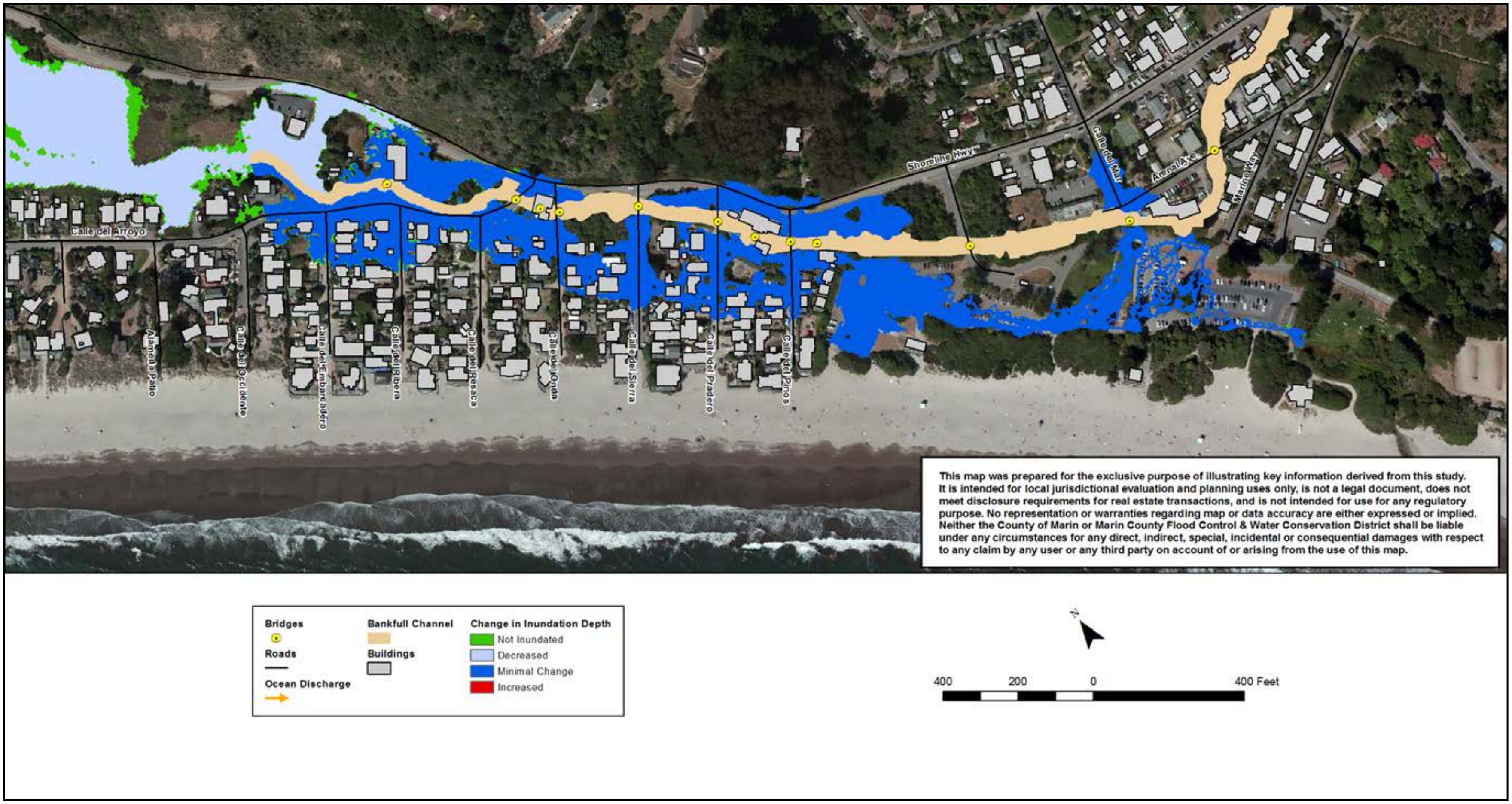


Figure 14-2 Decrease in flood extent and floodplain depths under Alternative 7-Causeway for the December 2005 flood.

14.2 Description of Alternative

This alternative involves the construction of a ~400-ft long causeway over Bolinas Lagoon to connect State Highway 1 with Seadrift Road along the alignment of what is currently a gravel road named Walla Vista Road (Figure 14-1). The primary purpose of this alternative would be to improve access to the Seadrift community which currently relies on Calle del Arroyo as the only means of vehicular access. Portions of the roadway currently become submerged in floods as small as a 2-yr event. Construction of the causeway would greatly improve vehicle access during flood events and result in improved safety for Seadrift residents, with limited benefit for residents of the “Patios” and “lower Calles”.

To investigate potential flood mitigation in the lower Calles by controlling tidal conditions in the upper estuary, a tide gate structure and pump station were included in hydraulic modeling of the causeway alternative. The concept is that the tide gate and pumps would operate to lower the downstream tidal condition during flood events on the creek in order to reduce backwater effects in the lower reaches of Easkoot Creek.

Water levels in the estuary adjacent to the proposed causeway are controlled by coastal storm surge, tides, and runoff from Easkoot Creek and other tributaries. A feasibility assessment and preliminary design for the causeway cannot be completed until a coastal flood hazard evaluation has been completed.

14.3 Flood Control Benefits

Construction of a causeway would result in improved vehicle access to the Seadrift community, and provided that it is constructed such that it does not restrict tidal action it is unlikely to have any significant effect on flooding. Inclusion of a tide gate and a pump station reduced maximum water levels in the estuary immediately upstream of the causeway by two feet (by design). This effect diminishes moving up the estuary; at Francisco Patio the reduction is approximately one foot. Above this point the flooding becomes increasingly dominated by flows from Easkoot Creek, and by Calle del Occidente, the reduction is less than 0.5 feet, and less than 0.1 feet by Calle del Arroyo (Figure 14-2). These reductions only result in minor decreases in flood extent and one building having reduced flood hazard (decrease in flood extent and floodplain depths) under December 2005 flood conditions (Table 6-2).

It is important to note that this analysis was only performed for Mean Higher High Water (MHHW) tidal conditions. Under more extreme tidal conditions such as occurred during the historical December 2005 flood, water overtops Calle del Arroyo and floods portions of the Patios. Under these conditions lowering water levels in the estuary via a tide gate and pump station would likely have significant flood control benefits. Also, it can be expected that the frequency with which estuary water levels overtop Calle del Arroyo will increase in the future due to sea level rise. Evaluating potential mitigation of coastal flood hazards by regulating estuary water levels at the causeway appears to be warranted but is beyond the scope of this study.

Although access to Seadrift would be greatly improved under this alternative, construction of a causeway alone may not improve access for residents in the lower Calles and Patios depending on the extent of flooding on Calle del Arroyo.

14.4 Preliminary Design and Estimated Construction Costs

The distance between potential causeway abutments is about 350 feet. In order for the causeway opening(s) to align roughly perpendicular to the primary flow direction in the main estuary channel, a causeway alignment with some curvature would be required and the total span of the causeway would be approximately 400 feet (Figure 14-1). The 340 feet gravel portion of Walla Vista Road would also likely need to be resurfaced in order to accommodate the increase in vehicle traffic using the causeway.

The highest water levels adjacent to the causeway that were simulated during this study occurred during the December 2005 flood event which coincided with a very high tidal condition. Maximum water levels adjacent to the causeway alignment during this event were on the order of 8.4-ft NAVD88. For the purposes of the preliminary conceptual design presented here we assume a design causeway elevation of 9.4-ft NAVD88 which represents 1-ft of freeboard above our highest simulated water levels. This elevation is approximately 0.5 feet higher than the existing ground elevations at the end of Walla Vista Road and approximately 1.2 feet higher than the existing ground elevations on Highway 1 where the causeway would connect.

A number of alternatives are possible for causeway construction. Although not essential from a hydraulic standpoint, providing multi-purpose capability of the system seems desirable. For planning purposes, we envision an earthen levee with a top width of about thirty feet supporting a two-lane paved road and shoulders. The levee would be of either imported fill, or of consolidated bay mud excavated and placed in sheet pile constraints. Depending on the ultimate intended use, the width could be reduced to that necessary for a one-lane roadway.

Tide Gates and Pump Station. Many options are available in terms of the number of openings in the causeway and their dimensions; for the purposes of this preliminary investigation of potential flood control benefits, a single 40 foot wide gated opening was assumed. Two concepts are possible for operating the gates for flood control purposes. One is to simply close the gates during low tidal conditions when flood flows are expected, thus isolating the upstream area from tidal influence. This would create a temporary detention basin in the portion of the estuary above the causeway. The water level in the 'detention basin' would rise as a function of inflow from the creek, and the gates would need to automatically open or overflow once the backwater elevation in the basin matched that of the external tidal elevation. A second concept would add a pump station to pump flows from Easkoot Creek past the causeway to Bolinas Lagoon and maintain the artificially lowered water level upstream of the causeway.

In this preliminary analysis, a water level of 3.8 feet NAVD88 was used as the threshold for closing the tide gate and activating the pump station. This elevation is 2 feet below the MHHW elevation of 5.8 feet NAVD88. This level was selected because it is low enough to significantly reduce water levels in the estuary and potentially reduce flooding impacts but not overly optimistic regarding the ability to anticipate flooding on Easkoot Creek in time to close the tide gates during low tidal condition.

A preliminary evaluation of the first concept revealed that the storage generated behind the causeway by artificially lowering water levels by two feet would represent only about 14% of the total December 2005 storm volume. Thus in order for this alternative to be effective, the second concept of adding a pump station is necessary. In order to maintain the 2-ft reduction in water level above the causeway, the pump station capacity would need to keep pace with Easkoot Creek discharges. This would mean maximum capacity on the order of 170 cubic feet per second (cfs) to mitigate against the December 2005 flood and 470 cubic feet per second (cfs) to mitigate against the 100-yr flood. These pumping rates are relatively large, and would require significant pumping capacity.

A preliminary cost estimate is summarized in Table 14-1. The estimated total cost to implement this alternative is about $3.0 million. This analysis suggests that the flood control benefits that could be achieved with this alternative would be minor. In addition, the effects of coastal flooding are likely to be substantial, and would need to be analyzed should this alternative be further considered.

14.5 Permitting Issues

Permitting is expected to be a substantial undertaking for this alternative. A California Coastal Commission permit will be required, as will permission from the U.S. Army Corps of Engineers, National Marine Fisheries Service, and any other resource agencies with jurisdiction over tidal waters. California Department of Fish and Wildlife and the Regional Water Quality Control Board will likely be involved. If dredged bay mud is used for levee construction, a detailed plan and permits will be required for the dredging alone. Impacts to flora and fauna will need to be documented. Given the limited flood mitigation benefits of regulating flows and water levels at the causeway, we have not provided detailed consideration of permitting issues.

14.6 Operation and Maintenance Requirements and Costs

Operation and Maintenance costs to maintain and operate the tide gates and pump station are likely to be significant. These systems would require active management for operation. Even if system operations are automated or remotely controlled, they would likely require trained personnel capable of operating the systems available on-call during the winter storm season. Mechanical elements would be designed appropriately for the environment, but the combination of tidal fluctuations and salt water should be expected to reduce the durability and operability of system components if not diligently maintained. Tide gates and pump station facilities would require seasonal maintenance and testing. The District operates pump systems elsewhere in the County, and is experienced with staffing and maintenance costs for these types of facilities. Detailed estimates of costs have not been prepared at this preliminary stage of project design; however, annual maintenance costs could lie in the range of $10,000 to $25,000 and fluctuate depending on the extent of repairs that could be required.

It should be noted that if the Causeway were constructed with the sole objective of providing emergency access to State Highway 1 from Calle del Arroyo and Seadrift, tide gates and pump stations would not be part of the project design and operation and maintenance would be significantly reduced to infrequent activities associated with the Causeway.

14.7 Sustainability (Short-term and Long-term)

Sedimentation originating from Easkoot Creek and other tributaries to the portion of Bolinas Lagoon affected by this alternative is unlikely to represent any constraints in the short-term. Because this alternative's facilities and effects lie primarily in the tidal zone, sustainability constraints are chiefly a function of coastal flooding and sea level rise. Analysis of coastal flooding would need to be completed to have an informed perspective on sustainability of this alternative.

14.8 Feasibility, Next Steps and Additional Information Needs

1. Completion of a coastal flood hazard analysis as it relates to the southeastern arm of Bolinas Lagoon where this alternative would be implemented; this should include consideration of sea-level rise to adequately address sustainability.

2. Following completion of the coastal flood hazard, re-evaluate the concept of regulating water levels and flows using the causeway and tide gates and/or a pump station as a possible means of mitigating coastal flooding. Consideration could also be given to an alternative causeway location farther south near the Stinson Beach County Water District office that might provide more effective flood mitigation in the lower Calles.

3. Re-evaluate the Causeway alternative considering the potential effects of Alternative 8-Raising Calle del Arroyo in combination with coastal flooding.

15 Alternative 8-Raising Calle del Arroyo

15.1 Summary

Portions of Calle del Arroyo become submerged in floods as small as a 2-yr event significantly restricting vehicular access to the Lower Calles, Patios, and Seadrift areas during flood conditions. This alternative is designed to improve access for residents of these areas by elevating the entire length of Calle del Arroyo between Highway 1 and Seadrift Road; a distance of approximately 2,840 feet (Figure 15-1). Elevating the roadway is also expected to restrict elevated water levels in the Easkoot Creek estuary from breaching the roadway and inundating residential areas. Given that elevating the roadway represents placement of fill within an active floodplain area, the design must include drainage features to prevent floodwaters from backing up behind the roadway potentially exacerbating flooding impacts.

This analysis suggests that elevating Calle del Arroyo can be accomplished without exacerbating riverine flooding provided that sufficient drainage is provided for flood flows to cross the roadway and return to the estuary. By preventing overtopping of Calle del Arroyo and providing a return flow pathway back to the estuary at Calle del Resaca, some flood mitigation is possible. During the December 2005 flood a significant reduction in flood extent was achieved in the Lower Calles and two of twenty-four buildings have significant reductions in flood hazard (Table 6-2). Additionally, Calle del Arroyo remained dry which would allow for vehicle access over the full length of the roadway (Figure 15-2 and Table 6-2). Some increases in floodplain depths do occur locally owning to water backing up behind the elevated roadway. This effect can likely be mitigated by developing a more refined design that includes additional drainage features designed to direct flows into culverts and back to the estuary, however more detailed drainage analysis is required.

Given that the area surrounding Calle del Arroyo is subject to flooding from a variety of sources including coastal storm surge, elevated tidal conditions, and riverine flooding, a design for the elevated roadway and associated drainage features cannot be fully developed until a coastal flood hazard evaluation has been completed (a task beyond the scope of this study which focuses only on riverine flooding). Assuming a preliminary design elevation for the roadway of 9.6-ft NAVD88 (1-ft of freeboard above our highest simulated water levels) yields a mean height increase of 2.3-ft requiring approximately 8,300 cubic yards of fill. Though complicated by the need to consider driveway access and existing utilities, design and construction of the elevated roadway should be relatively straightforward. Less straightforward though likely feasible would be the design of appropriate drainage features which would need to serve a variety of functions including preventing water from backing up behind the roadway, enhancing drainage of floodplain flows back to the estuary, and preventing backflows when estuary water levels are high.

Preliminary estimated cost for elevating the roadway and providing required drainage features is on the order of $1.0 million (Table 15-1).

Table 15-1 Summary cost estimate for Alternative 8-Raising Calle del Arroyo.

Bypass Alternative - Planning-level Budget Summary	**Cost ($)**	**Percent**
Consultant Planning, Permitting and Design Subtotal	68,470	6.8
Construction subtotal	710,776	70.6
Subtotal Contractor Overhead	179,200	17.8
Planning-level Cost Estimate (to nearest $1000)	958,400	95.2
Project Administration	47,920	4.8
Installed Project Cost Estimate	**1,006,320**	**100.0**

Figure 15-1 Overview map of Alternative 8- Raising Calle del Arroyo.

Note a single set of return flow culverts at Calle del Resaca were modeled but additional structures are likely necessary to accommodate coastal flooding.

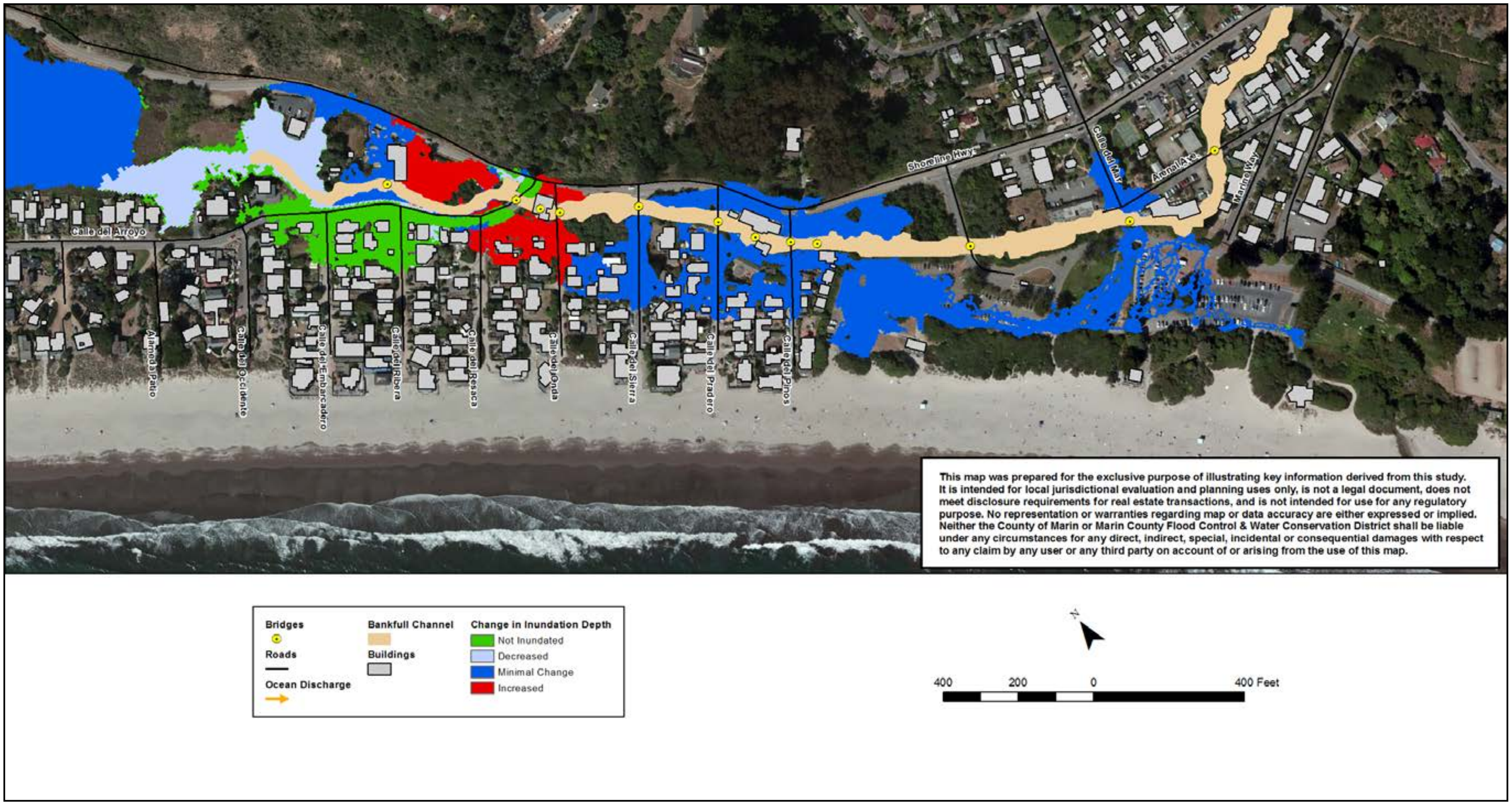


Figure 15-2 Decrease in flood extent and floodplain depths under Alterative 8-Raising Calle del Arroyo for the December 2005 flood.

15.2 Description of Alternative

This alternative involves elevating the entire length of Calle del Arroyo between Highway 1 and Seadrift Road; a distance of approximately 2,840 feet (Figure 15-1). The primary purpose of this alternative would be to improve access to the Lower Calles, Patios, and Seadrift community which rely on Calle del Arroyo as the only means of vehicular access. Portions of the roadway currently become submerged in floods as small as a 2-yr event. Elevating the roadway would greatly improve vehicle access during flood events and result in improved safety for these portions of the Stinson Beach community.

Given that water levels in the estuary adjacent to the roadway are subject to flooding from a variety of sources including coastal storm surge, elevated tidal conditions, and riverine flooding, a design for the elevated roadway cannot be fully developed until a coastal flood hazard evaluation has been completed (a task beyond the scope of this study which focuses only on riverine flooding). The highest water levels adjacent to the roadway that were simulated during this study occurred during the December 2005 flood event which coincided with a very high tidal condition. Maximum water levels adjacent to the roadway during this event were on the order of 8.6-ft NAVD88. For the purposes of the preliminary conceptual design presented here we assume a design road elevation of 9.6-ft NAVD88 which represents 1-ft of freeboard above our highest simulated water levels. Using this design elevation yields a mean height increase of 2.3-ft requiring approximately 8,300 cubic yards of fill.

Given that elevating the roadway represents placement of fill within an active floodplain area, it has the potential to exacerbate flooding conditions by backing up floodplain flows or coastal storm surge behind the roadway and/or preventing these flows from re-entering the estuary. In order to mitigate against this effect, a series of culverts beneath the roadway would be required. These culverts would need flap gates on the estuary side in order to prevent reverse flows from occurring when water levels in the estuary are high. For the purposes of this preliminary analysis a single set of three 36 inch circular culverts with downstream flap gates was evaluated at a location on the downstream side of Calle del Resaca (Figure 15-1). It is important to note that because of the potential for storm surge to carry water from the ocean up the Calles and Patios and towards the estuary, drainage beneath Calle del Arroyo would likely be needed at additional locations throughout the lower Calles and Patios.

15.3 Flood Control Benefits

Our analysis suggests that elevating Calle del Arroyo can be accomplished without exacerbating riverine flooding provided that sufficient drainage is provided for flood flows to cross the roadway and return to the estuary. Our analysis of flooding patterns under existing conditions revealed that flooding along the left bank within the lower Calles reach results both from overtopping of Calle del Arroyo as well as from a floodplain flow path that originates farther upstream. The hydraulic modeling results for the December 2005 flood demonstrate that by preventing overtopping of Calle del Arroyo and providing a return flow pathway back to the estuary at Calle del Resaca, some flood mitigation is possible with this alternative. Nearly all of the floodplain flow on the left bank was able to return to the estuary via the Calle del Resaca culverts and a substantial area of the floodplain downstream had significantly reduced flood hazard (decrease in flood extent and floodplain depths) (Figure 15-2). This resulted in the removal of two of the twenty-four buildings from the December 2005 floodplain in addition to allowing for vehicle access over the full length of Calle del Arroyo (Figure 15-2 and Table 6-2). The results do however show some increases in floodplain depths of as much as 0.6 feet between Calle del Onda and Calle del Resaca owning to water backing up behind the elevated roadway. This effect can likely be mitigated by developing a more refined design that includes additional drainage features designed to direct flows into culverts and back to the estuary.

15.4 Preliminary Design and Estimated Construction Costs

Calle del Arroyo is a relatively straight ~20 foot wide roadway oriented in a NW-SE direction and stretching some 2,840 feet from Highway 1 on the southeast to Seadrift Road on the northwest. In general, and although indistinct, the road surface is located at the high point of local topography. Local soils and drainage are such that little or no defined or developed drainage ditching is observed along the route, and no culverts are observed under the roadway. Over its length, twelve private roadways (mostly gravel) intersect the road, all entering from the south. There are two stop signs for traffic speed control, one at Calle del Occidente and one at Joaquin Patio.

An overhead power corridor traverses the length of the roadway with most poles located about 20 feet north of the edge of pavement. Power lines cross the road at a skew angle on the east end of the study area, with some poles within 5 feet of the pavement. An underground water main serving the Calles and Patios as well as Seadrift is likely located within the right-of-way. Residences are believed to be served by individual onsite septic systems rather than by a sanitary sewer system with force main within the right of way.

The north side of the road is relatively less developed than the south side and has a shoulder width of about twelve feet. An area of clustered houses is present on the north side between Calle del Occidente and Francisco Patio. A fire station with paved parking is located across from Calle del Occidente, and a thirty x fifty foot graveled parking lot is located across from Sonoma Patio. The south side has twelve access road intersections and several individual stand-alone driveways. In some locations, local fences and landscaping come to within a few feet of the roadway. Developed shoulder and parking is much less prevalent than on the north side.

A design for elevating the roadway cannot be fully developed until a coastal flood hazard evaluation has been completed which is beyond the scope of this study which focuses only on riverine flooding. Based on consideration of riverine flooding only, a preliminary design elevation of 9.6-ft NAVD88 is assumed. This elevation would provide 1-ft of freeboard above the highest water levels simulated for this study. Using this design elevation yields a mean height increase of 2.3-ft requiring approximately 8,300 cubic yards of fill.

A key design element will be locating and sizing return flow culverts so that floodwaters can pass the roadway and return to the estuary. The following design considerations pertain to the culverts:

- Hydraulic modeling for the December 2005 flood indicates that a single set of three 36 inch culverts located near Calle del Resaca would provide sufficient drainage to permit the ~50 cubic feet per second (cfs) of floodplain flow on the left bank to return to the estuary.
- Additional culvert locations would likely be needed to accommodate larger riverine floods and/or storm surge.
- Final locations and sizing should be determined based on consideration of both riverine and coastal flood hazards.
- Culvert inlets need to be placed at relative topographic low points. Such low points may require manufacture, swale creation, and routing to enhance drainage from low points within residential areas.

- Supplemental fill over the existing roadway of about 2.3' is proposed to create the flood control levee. Assuming a 12 inch minimum culvert cover allowance for development of load bearing capacity results in a maximum culvert diameter of about 12 inches (O.D. about 15 inches) if placed on local grade. If existing or created low swales are available for placement culvert diameter may be increased.
- Smoothbore culverts at 1% slope have an approximate pipe full capacity as noted below. Flow will be de-rated to about 60-70% of that shown due to entrance effects. Placement of a flap or rubber lipped valve at the outlet may further restrict flows. Flows shown are not developed unless the entrance is submerged enough to develop full pipe flow, which may not occur with a maximum available head of 12 inches above the entrance.

Diameter (inches)	Capacity (cfs)	Velocity (fps)	Culvert Count for 100 cfs
12	4	4.5	25
15	7	5.8	15
18	12	6.4	9
24	24	8.0	4
30	42	9.0	3
36	60	10.0	2

- Culvert banks in multiples providing (yet unknown) design return flow values will be required. Consideration of culvert inlet control as a flow limitation condition is necessary due to the low available head, further increasing culvert counts at flood return discharge points.
- At half depth, flows will be about half of full depth flows, resulting in backwater accumulation behind the culverts. Flooding may therefore not be totally mitigated by presence of flow relief culverts, because of the stage-discharge characteristics and the backwater elevation required to achieve design flows. In such a case flood elevations may not be significantly reduced, however flood durations may be reduced.
- Culvert discharge flows need to return to the creek or estuary in a non-erosive fashion. It may be necessary to provide armored discharge channel construction on/over private property in order to accomplish this goal.

- Culvert backflow is envisioned to be prevented by use of flap gates at the outlet end. Alternative devices may be commercially available. Units used should provide full flow at very low head, so as to provide the intended performance under flood flow conditions.
- Individual culvert performance and design should consider and use the minimum capacity as determined by inlet conditions, head constraints, pipe flow constraints, and outlet (flap valve) constraints.
- Post-flow flap valve maintenance may be required on an event-based schedule to ensure that debris or trash does not foul the apparatus or allow reverse flows.
- Some kind of risk management document may be appropriate to absolve the responsible agency from flood damage claims in the event that flow control devices fail and allow reverse flows and flooding where not already present.

The following considerations pertain to construction:

- The proposed work is considered technically feasible, and does not invoke any extra-ordinary construction methods or techniques.
- A detailed route survey is required to identify all ground features appropriate for engineering design of the project.
- A detailed engineering design is required in order to accommodate site-specific constraints on a case-by-case basis.
- Cooperative agreements, easements, or other formal agreements may be required in cases where the proposed work encroaches on private property. Eminent domain procedures may be required if recalcitrant owners are encountered because project integrity requires complete and seamless coverage of the route.
- Fill depth is not great and should be of imported base rock rather than soil, in order to preserve road sub-grade integrity.
- Lateral sloped fill prism at road shoulders would need to be 23 feet wide in order to maintain a 10% side slope. A steeper shoulder side slope would not be recommended due to vehicle safety and parking considerations. Lateral slopes of 10% may not be achievable in some areas.
- Installation of low retaining walls with guard railing may be required in some areas where lateral offset distance is not available for gravel prism creation.

- Lateral slope of 10% extending into the Fire Station parking lot may direct rainfall towards/into the building, requiring installation of secondary drainage facilities for mitigation. This might include placement of a slot drain at the toe of slope parallel to Calle del Arroyo.
- The right-of-way may contain underground utility access points including but not limited to manhole covers, inspection ports, junction boxes, and survey monumentation. Each will need to be identified and preserved during site work, extended about 2.3 feet in elevation.
- The old pavement should be ground up and recycled, so that new fill is not placed on a discontinuity or layer providing moisture detention.
- New pavement will be required, covering a minimum area of about 52,800 square feet; additional paving on the street approaches in the amount of 5,520 square feet is highly recommended.
- Public and emergency vehicle access over the roadway will be required at all times during demolition and reconstruction.

Costs to implement this alternative are summarized in Table 15-1, and are approximately $1.0 million.

15.5 Permitting Issues

Work on a public street will likely be undertaken by Marin County Department of Public Works as a capital improvement project. County grading, drainage, and/or floodplain permits may be required. A California Coastal Commission permit would likely be required. Given that the work area is below 100-yr flood elevations, a permit for placement of fill within the floodplain will be required from the U.S. Army Corps of Engineers. The project is likely exempt from CDFG, RWQCB oversight, since it is not conducted within those jurisdictional areas. If however, return flow channel construction occurs below the top of bank of Easkoot Creek, CDFG and other resource agency permitting may be required.

15.6 Operation and Maintenance Requirements and Costs

These facilities as planned will operate on a passive basis, without active management requirements, with the possible exception of occasional inspection of culverts and clearing obstructions as needed during the winter storm season. Routine excavation of accumulated sediment at culvert inlets and outlets will be required. Some debris and vegetation management may be required to maintain culvert function.

The principal operations and maintenance costs of this alternative are expected to be associated with maintaining water conveyance through culverts. Maintenance activities would likely be required seasonally after the rainy season when sediment and debris deposition would occur, and after or during exceptionally-high tides. Culverts fitted with flap valves may retain debris or trash, requiring regular maintenance to ensure satisfactory performance. In addition, there would need to be provisions for urgent maintenance during and after winter storms when floodplain flows or coastal flooding occurs. Significant deposits of sediment and debris could compromise the function of culverts passing flows under the elevated roadway, and flooding could occur if culverts fail to function properly. Permitting for maintenance activity should be incorporated in permitting for the construction phase of the project.

Marin County Public Works Department staff and/or NPS maintenance staff and equipment would likely be identified as the appropriate organization(s) to conduct these maintenance activities. Costs for the necessary personnel and equipment are best known by these organizations. For planning purposes, assuming about five working days for a small crew equipped with a loader/backhoe machine and a dump truck would likely be sufficient to accomplish routine and urgent maintenance. Personnel and equipment costs for this level of effort is estimated to be approximately $8,000 to $10,000 per year.

Additional maintenance activities for this alternative are associated with the roadway of Calle del Arroyo. A properly designed roadway should have low maintenance requirements. Depending on pavement section used and local environmental conditions, a service life of at least 20 years is anticipated. Periodic maintenance would be expected to be necessary to provide satisfactory long-term performance. Culvert life should match roadway life if properly installed. The proposed flood routing culverts would normally be dry and not subject to scour or wear.

15.7 Sustainability (Short-term and Long-term)

Sedimentation originating from Easkoot Creek and other tributaries to the portion of Bolinas Lagoon affected by this alternative is unlikely to represent any constraints in the short-term, provided that maintenance described above is performed. Because this alternative's facilities and effects lie primarily adjacent to the tidal zone, sustainability constraints are chiefly a function of coastal flooding and sea level rise. Analysis of coastal flooding would need to be completed to have an informed perspective on sustainability of this alternative.

A properly designed and installed, road surface should have a reasonable 20-year design and economic life. Selection of materials that are resistant to groundwater intrusion would be needed in this low elevation coastal environment. The impact of sea level rise and coastal flooding could affect the long-term sustainability of flood mitigation achieved by this alternative. Raising Calle del Arroyo could also help mitigate coastal flooding impacts and should be evaluated in that regard after the coastal flooding study is complete.

15.8 Feasibility, Next Steps and Additional Information Needs

- Commission detailed ground survey for design and planning purposes including:
    - Parcel and ROW limits
    - Overhead utilities infrastructure
    - Underground utilities infrastructure
    - Local drainage
    - Relative high and low points of roadway
    - Potential culvert locations for estuary return flows
- Develop a refined design based on survey results and consideration of both riverine and coastal flood hazard conditions.
- Perform additional hydraulic modeling to test the refined design, ensure appropriate culvert configurations, and evaluate the expected flood mitigation potential from both coastal and fluvial flood hazards, including sea-level rise.
- Obtain public comments on proposed alternative.
- Determine property ownership and parcel – Right of Way limits along Calle del Arroyo.
- Preliminary design by DPW or outside consultant in conformance with DPW requirements.

16 Alternative 9-Combined Dredge, Wetland Enhancement, and Bypass

NOTE REGARDING TERMINOLOGY: Three alternatives, Alternatives 5, 6, and 9, have had their names changed from those used in previous drafts of this report. Alternative 5, formerly the 'North Bypass' alternative, is now 'Wetland Creation and Bypass to the National Park Service's North Parking Lot.' Alternative 6, formerly 'South Bypass,' is now 'Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.' Lastly, Alternative 9, formerly 'Combination Dredge and South Bypass,' is now 'Combined Dredge, Wetland Enhancement, and Bypass.' These new names were suggested by members of the community with the intent of greater precision and to emphasize the fact that wetland enhancement is a priority of the project.

16.1 Summary

This alternative combines Alternative 4-Channel Dredge and Sediment Management with Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot. This alternative was conceived to evaluate the potential combined flood control benefits of the two most effective, independent alternatives considered. The combined dredge and bypass dramatically reduces flood potential for the December 2005 event for all area except a small portion of the lower Calles (Figure 16-1). During the 100-yr flood event, flooding above Calle del Pinos is dramatically reduced (Figure 16-2). Only minimal reductions in flood extent occur within the Calles, however floodplain depths are reduced significantly. Under 2050 sea level rise conditions, the mitigating effects of the alternative do not change significantly upstream of the Calle del Arroyo crossing (Figure 16-3). Below this point flood extent and floodplain depths are still reduced relative to existing conditions but the improvements are much less throughout the Lower Calles reach. Farther downstream water levels in the estuary overtop Calle del Arroyo in the vicinity of Alameda Patio, and between Walla Vista and Rafael Patio resulting in flooded areas that were dry under existing conditions with the lower MHHW tidal condition.

Considered as a single project, Alternative 9 might be designed, permitted, and implemented more efficiently than each alternative individually. There would likely be potential savings in the costs of planning, permitting and construction because the projects could be bundled together. Neglecting these potential savings, the estimated cost of Alternative 9 is about $2.25 million, the sum of estimated costs of Alternative 4 ($0.86 million) and Alternative 6 ($1.39 million).

16.2 Description of Alternative

This alternative combined Alternative 4-Dredge and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot/Poison Lake Restoration. Dredging involves removing 3,100 yards of material from 2,300 feet of Easkoot Creek, lowering the channel by 2.4 feet on average. Installation of a series of sedimentation structures is also proposed to help reduce deposition in the lower channel and extend the life of the dredged profile. The bypass concept involves diverting water during high flow conditions from a location adjacent to the Parkside Cafe and discharging it through a bypass channel to a restored wetland in the vicinity of historical Poison Lake.

16.3 Flood Control Benefits

The flood control benefits of dredging and bypassing flows to a restored Poison Lake are substantial. During the December 2005 flood, the bypass carries up to 97.8 cubic feet per second (cfs) or 57% of the total discharge above the bypass of 171.3 cubic feet per second (cfs). Note that lowering the elevation of the diversion weir crest (which is possible because of the lower dredged profile) results in an additional 25.2 cubic feet per second (cfs) entering the bypass compared to the stand-alone bypass alternative. The combined dredge and bypass completely eliminates flooding for the December 2005 event with the exception of a small stretch of Calle del Arroyo near Calle del Ribera (Figure 16-1), and all twenty-four buildings have significantly reduced flood hazard (decrease in flood extent and floodplain depths) (Table 6-3). The average reduction in peak water levels in the channel is 3.6 feet in the reach adjacent to the Parkside Café, 2.2 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.8 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table 6-1).

During the 100-yr flood, the bypass carries up to 329.5 cubic feet per second (cfs) or 68% of the total discharge above the bypass of 482.7 cubic feet per second (cfs). Flooding above Calle del Pinos is completely eliminated. Only minimal reductions in flood extent occur within the Calles, however floodplain depths are reduced significantly throughout the majority of the inundated area (Figure 16-2). The average reduction in peak water levels in the channel is 3.4 feet in the Parkside Café reach, 1.4 feet in the Upper Calles, and 0.1 feet in the Lower Calles (Table 6-2). Approximately twenty-three of fifty-nine buildings (39%) have significantly reduced flood hazard (decrease in flood extent and floodplain depths) under 100-yr flood conditions (Table 6-4).

Under 2050 sea level rise conditions, the mitigating effects of the alternative do not change significantly upstream of the Calle del Arroyo crossing (Figure 16-3). Below this point flood extent and floodplain depths are still reduced relative to existing conditions but the improvements are much less throughout the Lower Calles reach. Farther downstream water levels in the estuary overtop Calle del Arroyo in the vicinity of Alameda Patio, and between Walla Vista and Rafael Patio resulting in flooded areas that were dry under existing conditions with the lower MHHW tidal condition (Figure 16-3).

16.4 Preliminary Design and Estimated Construction Costs

This alternative combines the features of Alternative 4-Dredge and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot. The reader is referred to the preceding sections discussing these alternatives individually for details regarding the alternative designs. The only departure made from a simple combination of the individual alternatives is that the elevations of the weir crest and upper-most reach of the bypass channel were lowered to conform to the dredged channel profile and allow an even larger percentage of the flow in Easkoot Creek to enter the bypass channel. The weir crest elevation was lowered from 24.8 feet to 22.1 feet NAVD88 which maintains activation of the bypass channel when water depths in the creek reach approximately 1-ft. The slope of the upper ~40 feet of the bypass channel is reduced in order to conform to the lower weir crest; below this point the bypass channel remains as described in Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot.

16.5 Permitting Issues

The same permitting issues discussed for Alternative 4-Dredge and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot apply to this combined alternative and the reader is referred to these chapters for more details. Design and permitting costs for the combined alternative would be expected to be lower than costs for each alternative individually.

16.6 Operation and Maintenance Requirements and Costs

The same operation and maintenance requirements and costs discussed for Alternative 4-Dredge and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot apply to this combined alternative and the reader is referred to these chapters for more details.

16.7 Sustainability (Short-term and Long-term)

The same sustainability considerations discussed for Alternative 4-Dredge and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot apply to this combined alternative and the reader is referred to these chapters for more details. Results from the sea level rise analysis suggests that the mitigating effects of this alternative will likely be sustained under 2050 sea level rise conditions above the Calle del Arroyo bridge but will become diminished (though not eliminated) farther downstream.

16.8 Feasibility, Next Steps and Additional Information Needs

The same feasibility and next steps considerations discussed for Alternative 4-Dredge and Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot apply to this combined alternative and the reader is referred to these chapters for more details.

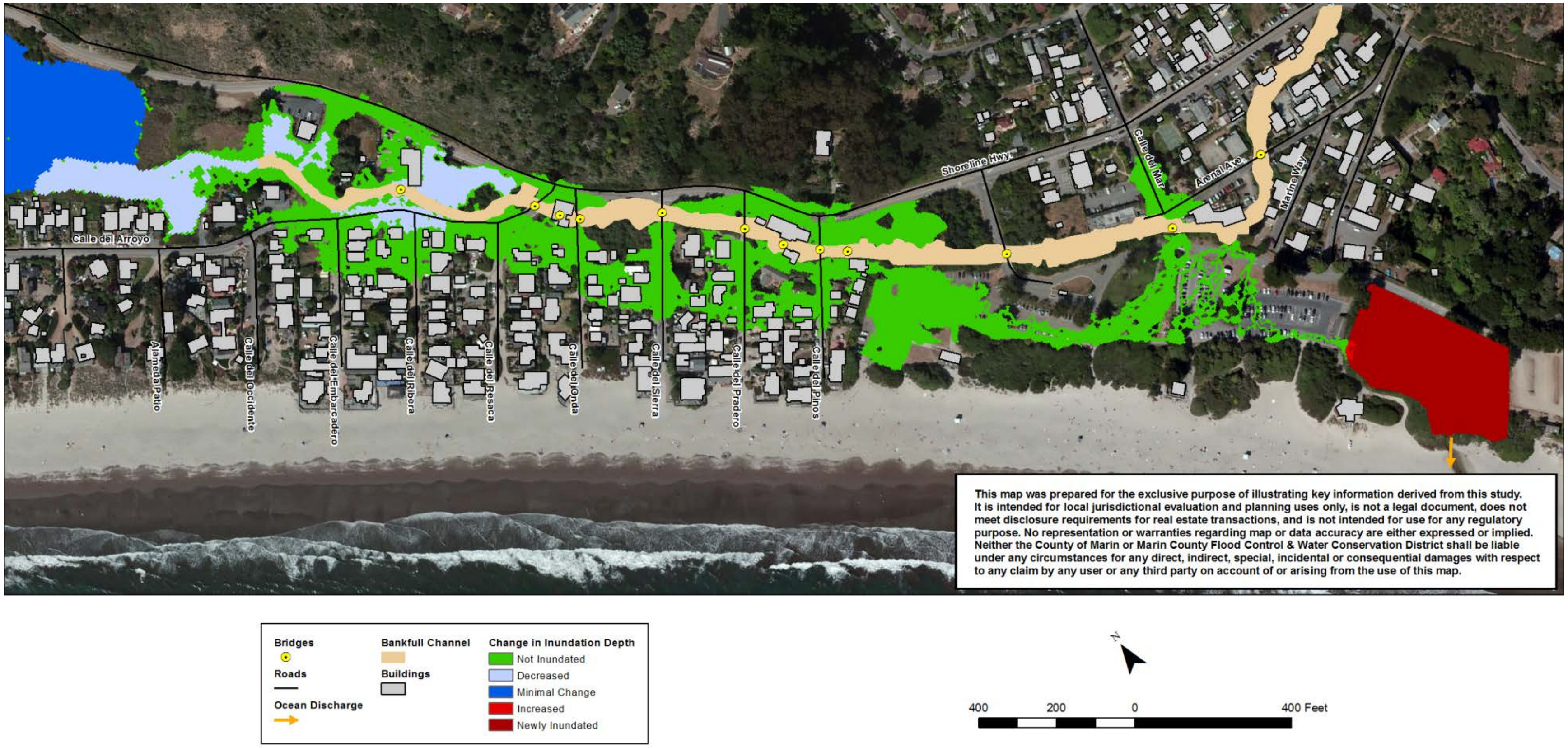


Figure 16-1 Decrease in flood extent and floodplain depths under Alternative 9-Combined Dredge, Wetland Enhancement, and Bypass for the December 2005 flood.

Figure 16-2 Decrease in flood extent and floodplain depths for the 100-yr flood under Alternative 9-Combined Dredge, Wetland Enhancement, and Bypass.

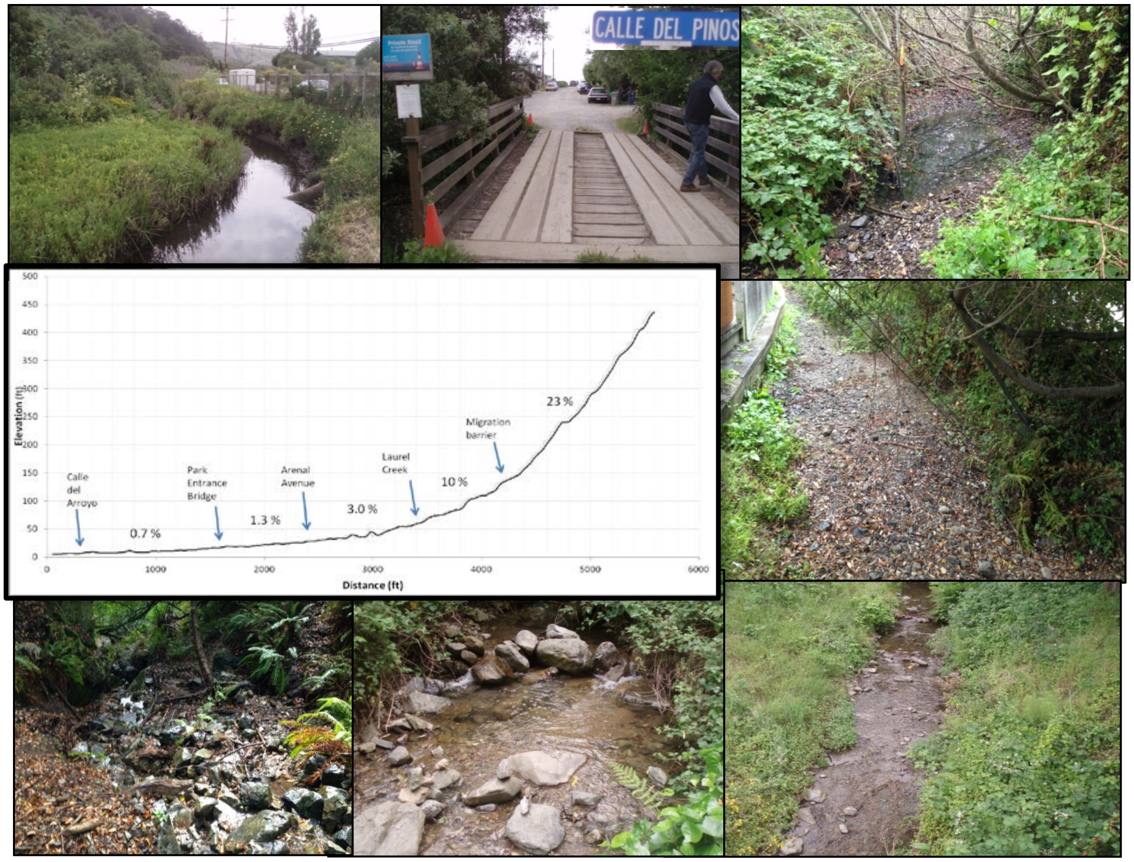
Figure 16-3 Decrease in flood extent and floodplain depths for the sea level rise scenario under Alternative 9-Combined Dredge, Wetland Enhancement, and Bypass.

17 Alternative 10-Structure Elevation

This alternative involves elevating buildings so that the ground floor is situated above flood elevation rather than attempting to control the extent and depth of flooding by managing the channel, sediment and floodplain conditions. The means by which this alternative would be implemented have not been determined. Homeowners typically undertake these significant structural modifications to their homes as a largely private project. In addition to the costs of structural modifications, there are costs for customary building permits from Marin County and significant costs for permitting construction projects in the Coastal Zone.

At least two homeowners in Stinson Beach have raised their homes in the past decade. Costs for any individual property would be expected to vary considerably depending on the character of the existing structure and specific conditions at each site. Based on the experience of one homeowner in the community, costs could be expected to range from $50,000 to $100,000 or more. Using this range of costs, it is possible to estimate the cost of raising structures throughout the community that are vulnerable to flooding from Easkoot Creek for comparison with other flood mitigation alternatives. Two cases have been considered, one that elevates the 24 buildings that lie within the December 2005 floodplain under existing conditions and one that elevates the 59 buildings that lie within the 100-yr floodplain under existing conditions. Details of these are provided in the Appendix “Hydraulic Model and Flood Hazard Evaluation”.

Costs to mitigate structure flooding for the December 2005 flood event given existing channel conditions assumes that the 24 structures are raised at a cost ranging from $50,000 to $100,000 each. Total cost to mitigate 2005 level flooding would be $1.2 million to $2.4 million. Costs to mitigate structure flooding for the 100-yr design flood event given existing channel conditions assumes that the 59 structures flooded are raised at a cost ranging from $50,000 to $100,000 each. Total cost to mitigate the 100-yr design flood would be $2.95 million to $5.9 million. The costs for this alternative could conceivably be reduced if elements of the process, particularly permitting, could be simplified or made routine. Given the potential for numerous individual projects in a small area over a relatively short time period should this alternative be pursued, there could be some potential for developing a special permit process to facilitate implementation of this alternative. In any event, since this alternative involves individual homeowners to undertaking improvements to their property, there are significant limitations on the District's ability to implement this alternative.



Easkoot Creek Hydrology and Hydraulics Study

Technical Appendices

Prepared for

Marin County Flood Control and Water Conservation District

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TO: Chris Choo
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Flood Control and Water Conservation District

FROM: Matt O'Connor
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SUBJECT: Easkoot Creek Hydrology & Hydraulics Study

Appendix A. Background Information and Data Acquisition Plan

This memorandum focuses on summarizing the most relevant available data and information pertaining to the hydrology and hydraulics study of Easkoot Creek. No significant data gaps that could substantially alter the proposed approach to modeling the hydrology and hydraulics of Easkoot Creek have been identified.

Prior studies and data sources are also identified in this memorandum. The discussion of particularly relevant aspects of these studies not directly related to the task of modeling watershed hydrology and the hydraulics of Easkoot Creek in Stinson Beach occurs in subsequent memoranda.

Other data and information and descriptions of existing conditions are provided in subsequent memoranda pertaining to Task 3, including:

- Watershed conditions related to flow and sediment transport
- Geomorphic assessment of the watershed
- Detailed reach scale conditions of Easkoot Creek related to flooding, sediment transport and habitat conditions. Identification of critical areas and concerns.
- Physical setting, overall land uses and utilities and park and public land uses

Overview

Information regarding Easkoot Creek has been compiled from previous reports and existing data in Marin County files, from studies of Easkoot Creek by Stetson Engineers for the Stinson Beach County Water District (SBCWD), from National Park Service (NPS) monitoring studies, from interviews with knowledgeable Stinson residents, from other publicly-available sources, and from field surveys conducted in late-November and early-December.

Data and information can be grouped in the following categories:

- stream flow
- precipitation
- topographic data
- tide data
- prior studies
    - flood-related

- ecosystem and watershed studies

The most relevant prior studies and data reviewed are compiled in this memorandum. Sources most relevant for the hydrologic and hydraulic analyses are discussed below and summarized in Table 1.

Stream Flow Data

The data available from stream flow gauging studies in Easkoot Creek are summarized in Table 1. Several gauging records have been developed beginning in 2000 with a National Park Service gauge near the Stinson Beach NRA parking lot. This gauge was moved in 2003 from a location upstream of the vehicle bridge accessing the parking lot to its current location about 25 feet downstream of the bridge. Data are available at 15 minute intervals for the period through September 30, 2010. A series of rating curves developed for the gauge by the NPS include flows up to approximately 100 cubic feet per second (cfs). Stream stage data are available for peak flow during the December 31, 2005 flood event; however, no valid rating curve exists for flows of that magnitude. In addition to the 2005 event, peak flows in December 2004 and January 2008 that approach or exceed 100 cubic feet per second (cfs) are expected to be useful for calibration of the runoff model.

Stetson Engineers installed and operated stream flow gauges on upper Easkoot Creek and its tributaries (Fitzhenry Creek and Black Rock Creek), and prepared a report summarizing flows for calendar year 2004. At the request of Marin County Flood Control District staff, Stetson provided detailed flow data for Fitzhenry and Black Rock Creeks for 2004, 2005 and January 2006. The rating curve for Fitzhenry Creek includes measured flow to about 14 cubic feet per second (cfs), but does not cover the range of stream stage reached during the December 31, 2005 peak flow event. The rating curve for Fitzhenry Creek is based on a small number of flow measurements. Flow data from these gauges may be utilized to evaluate hydrologic model results with respect to flow magnitude in model sub-basins (e.g. Fitzhenry Creek and Black Rock Creek).

In summary, the most useful data is from the Easkoot Creek gauge operated by the NPS, providing records of stream discharge for events up to about 100 cubic feet per second (cfs) and records of stream stage for flood events in December 2005 and January 2008. Stream gauge data obtained in Easkoot Creek tributaries by Stetson for the SBCWD are useful for evaluating hydrologic model simulations with respect to flow generation in tributary watersheds.

Estimates of stream discharge corresponding to 2-, 5-, 10-, 25-, 50-, 100- and 500-yr recurrence intervals can be obtained for Easkoot Creek using the US Geological Survey's National Streamflow Statistics program. These estimated flood discharges and frequencies are expected to provide a valuable supplement to previously-developed estimates.

Precipitation Data

Data available from precipitation gauges in the vicinity of Stinson Beach and in Marin County are summarized in Table 2. Precipitation gauging stations have been operated at Stinson Beach by SBCWD and NPS. SBCWD has maintained records of daily rainfall since July 1978. An automated rain gauge was operated for a two year period between June 2003 and June 2005. NPS operated an automated gauge adjacent to the stream gauge near the beach parking lot beginning in October 2000; dense riparian vegetation canopy compromised data from this site after 2006. NPS also operates an automated precipitation gauge near the town of Bolinas on the west edge of Bolinas Lagoon; the period of record begins in late-1998 at that site. Data from the automated gauges is most useful as input data to the hydrologic model because it provides rainfall observations at intervals of 15 or 30 minutes. Rainfall-runoff processes in small, steep watersheds such as Easkoot Creek may be rapid, so it is important to have precipitation data collected at short time intervals. Consequently, the period of record of the automated gauges is of primary interest.

Daily precipitation totals collected at various observations stations were available from the Marin Municipal Water Distinct (MMWD). Stations nearest Easkoot Creek are Kent Lake, Alpine Lake, Bon Tempe Lake, and Lake Lagunitas. Daily precipitation totals were also available from a station located on Middle Peak at an elevation of 2,400 feet west of the summit of Mt. Tamalpais. The Middle Peak station is part of a network of sites—the Remote Automated Weather Station (RAWS) network--operated cooperatively with a data repository maintained by the Western Region Climate Center in Reno, NV. The Middle Peak station is particularly valuable in delineating the spatial variability of precipitation across the watershed and because it provides a high-elevation record in close proximity to upper Easkoot Creek.

Annual rainfall isohyetal maps are available from Marin County's public GIS internet portal and from NOAA sources via the internet. Annual rainfall estimates on 800 m X 800 m grids for Marin County are available from PRISM via the internet. Extreme rainfall estimates (e.g. 100 year recurrence interval) can be obtained from the NOAA Precipitation Atlas via the internet.

In summary, the most valuable precipitation data are those recorded by automated gauges at intervals of 15- to 30-minutes that correspond to periods of high runoff and/or flooding in lower Easkoot Creek. Additionally, daily precipitation data from stations near Stinson Beach that correspond to episodes of high runoff and/or flooding are valuable in helping define the spatial distribution of rainfall over the watershed.

Topographic Data

The primary source of topographic data to be utilized for this study is the LiDAR-derived Digital Elevation Model (DEM) obtained from the Golden Gate LiDAR Project through Marin County Public Works Department. The DEM has a resolution of 1 square meter, and provides a detailed description of the topography of Stinson Beach and the floodplain adjacent to Easkoot Creek.

O'Connor Environmental, Inc. (OEI) conducted field surveys using a total station instrument in December 2011 to supplement the LiDAR data. The OEI survey focused primarily on the channel of Easkoot Creek and the dimensions of bridges over the creek. The OEI survey also covered representative locations on the floodplain, in the National Recreation Area, and on private streets and Highway 1.

Marin County Department of Public Works (DPW) has conducted surveys of Easkoot Creek in connection with dredging at bridge crossings, most recently in 2008. These data include records of the volumes of sediment dredged in recent operations and estimated for early dredging work. These surveys produced a thalweg profile for Easkoot Creek extending from Calle del Arroyo to the Stinson Beach Community Center, as well as cross-sections near bridge crossings where local dredging has been conducted. DPW has also surveyed floor elevations of residences at particular risk of flooding throughout Stinson Beach.

NPS topographic surveys have been conducted along the creek and its floodplain in connection with a stream restoration project in Easkoot Creek downstream of the "Parkside Cafe" pedestrian bridge (Calle del Mar). Of particular interest is an "as-built" topographic survey in 2004 of the channel restoration and habitat enhancement project area. Channel profiles in the restoration reach were surveyed in 1999, 2004 and 2006. Streambed elevation was surveyed by NPS Elevation data were also collected in connection with a study of the water table in an array of monitoring wells between Easkoot Creek and the sand dunes bordering the beach. NPS also periodically surveys the stream cross-section at their gauge station.

A stream channel (thalweg) profile of Easkoot Creek in Stinson Beach was published in 1979 in the Federal Emergency Management Administration (FEMA) Flood Insurance Study. The exact date of survey is not known, however it is assumed to represent conditions c. 1979. This thalweg profile is particularly useful in comparison with profiles surveyed by DPW in 2007 and NPS in 2006.

The critical topographic data for this study are the LiDAR DEM that describes the Easkoot Creek floodplain and the OEI survey data that described the stream channel of Easkoot Creek and its bridges. These data characterize current conditions in Easkoot Creek and will provide the topographic baseline for hydraulic simulations of flood flows.

Successive surveys of the Easkoot Creek thalweg, along with more recent cross-section surveys at dredging sites near bridges by DPW, and by NPS of its restoration project, is anticipated to provide an objective basis for estimating sedimentation rates.

Prior Studies

Several substantial studies of Easkoot Creek and the tributaries to Bolinas Lagoon have been prepared over the past few decades. Several pertain directly to flooding issues, while many others address broader watershed and ecosystem conditions, including fisheries. A recent study prepared for DPW in 2009 by Michael Love & Associates provides a good overview of past studies and the context for this study.

Flood-related Studies

- Stetson Engineers, Inc. (2010) Updated Flood Frequency Analysis of Corte Madera Creek at Ross Gage. Hydrologic analysis of probability of flood event of Dec. 31, 2005.
- MLA (2009) Review of Background Information and Flood Control Alternatives for Easkoot Creek, Stinson Beach, CA. July 17, 2009. Technical Memorandum prepared for Marin County DPW. Provides an overview of prior flood-related studies, and establishes the context for this study.
- FEMA (2009) Flood Insurance Study, Marin County, California, and Incorporated Areas. Effective date May 4, 2009. FIS Number 06041CV001A. This study contains the flood analyses and survey data completed in 1979 with updates in 1997 and as referenced by other studies. General characteristics of rainstorms and flood magnitudes throughout the county are described along with the results of hydraulic studies.
- William Spangle and Associates, Inc. (1984) Alternative Mitigation Measures for Storm and Flood Hazards, Stinson Beach, Marin County, California. This study, published in two volumes in December 1984, evaluated flood hazards from both ocean processes and Easkoot Creek and potential means of mitigating flood hazards. This study was motivated largely by significant flood damage at Stinson Beach during the winter of 1982-83.
- Sadjadi, M.M. (1971) Stinson Beach Drainage Study. This document is a hand-written engineer's preliminary analysis of alternatives for management of flood flows of Easkoot Creek.

Ecosystem and Watershed Studies

- NPS (undated-2003?) Environmental Assessment Easkoot Creek Restoration at Stinson Beach. Overview of Easkoot watershed environmental conditions.
- NPS (undated) Standard Operating Procedure (SOP) 3. Stream Gage Station Descriptions, Version 1.08. Hydrologic background of Easkoot watershed.
- Sanctuary Advisory Council, Gulf of Farallones National Marine Sanctuary (2008) Bolinas Lagoon Ecosystem Restoration Project. Provides recommendations for management of Easkoot Creek with respect to future management of Bolinas Lagoon.
- Garcia and Associates (2004) Channel Morphology of Easkoot Creek Following Lower Easkoot Creek Restoration Plan. Survey data documenting restoration project "as-built".
- Watershed Science (2003) Easkoot Creek Rehabilitation Plan. Geomorphic and hydrologic assessment of restoration plan. Includes pre-project cross-sections and estimates of stream flow and channel capacity, sediment characteristics and transport potential.
- Tetra Tech, Inc. (2002) Technical Appendices to the Bolinas Lagoon Ecosystem Restoration EIS and EIR. Prepared for US Army Corps of Engineers and Marin County Open Space District. Input sediment budget element of Bolinas Lagoon Watershed Study is of interest regarding sediment production rates estimated for watersheds near Easkoot Creek.
- A.A. Rich (1992) [reference incomplete; have pp. 8-47 plus Appendices]. Assessment of fisheries resources conditions in Easkoot Creek and development of a plan for rehabilitation of native fish habitat.
- Lehre, A.K. (1982) Sediment Budget of a Small Coast Range Drainage Basin in North-Central California IN Dietrich, W. E., T. Dunne, et al. (1982). Construction of sediment budgets for drainage basins. Portland, OR, Pacific Northwest Forest and Range Experiment Station: pp. 67-77. Study provides detailed erosion rates by process in headwater watershed of Lone Tree Creek located about 2 miles south of Easkoot Creek. Provides a basis for estimating watershed erosion rates in Easkoot Creek.

Tide Data

Tidal data are available at 6-minute intervals from July 2009 to present for the NOAA tide gauge located in Bolinas Lagoon near the end of Seadrift Road. Longer periods of record are available for Pt. Reyes (1975-present) and San Francisco Bay (1854-present).

Tide elevation analyses relevant for determining base level affecting flood hydraulics in Easkoot Creek are available in the FEMA FIS (2009). Similar analyses, including wave heights, are found in William Spangle and Associates (1984). Modeling of tides in Bolinas Lagoon has been conducted by Rachel Kamman, Kamman and Associates. Although potentially useful, availability of data and analyses from Kamman was found to be limited based on preliminary contacts with the author. A modeling study of tides affecting the project area is being conducted by Li Erickson, US Geological Survey. Contacts with Erickson (USGS) provided a significant conclusion regarding tidal phenomena in Bolinas Lagoon that might affect modeling tidal influence on flood base level in Easkoot Creek. USGS provided model runs for Bolinas Lagoon extending to the lower end of Easkoot Creek just beyond Calle del Arroyo and these data indicated that tide heights in the lagoon are essentially the same as those in lower Easkoot Creek.

Table A1 Streamflow Data Summary

STATION	LOCATION	OPERATOR	PERIOD OF RECORD	OBSERVATION FREQUENCY	UTILITY AND LIMITATIONS
EK (Easkoot Creek)	Vehicle bridge to Stinson Beach NPS parking lot	NPS	10-1-2000 to present	15 min. intervals	• Primary flow record for hydrologic model calibration • Flow rating curve extends to about 100 cubic feet per second (cfs) • Best event most for model calibration 12-27-2004 (peak near 100 cubic feet per second (cfs)) • Flood events 1-25-2008 and 12-31-2005 are not contained in channel at gauging station; secondary calibration with synthesized rating curve • Erosion/sedimentation of gauge cross-section affects rating curves and flow estimates for 2005 and 2008 events
FC-F (Fitzhenry Creek; principal tributary to Easkoot Creek)	Firehouse	Stetson Engineers (for SBCWD)	1-2004 through 12-2004	Mean daily	• Supplementary flow record for hydrologic model calibration • Flow rating curve extends to 14 cubic feet per second (cfs) • Data to evaluate model accuracy with respect to proportion of flow generation in sub-watersheds of Easkoot Creek
			12-6-2005 through 1-11-2006	30 min. intervals	• Raw stream stage extend far above range of rating curve • Data cover periods of high runoff and flood flows in December 2005; documents timing and relative magnitude of flow peaks • Potential use for model calibration with synthetic rating curve
FC-Ch	Church	Stetson (SBCWD)	1-2004 through 12-2004	Mean daily	• Supplementary flow record for hydrologic model calibration • Flow rating curve extends to 8 cubic feet per second (cfs) • Data to evaluate model accuracy with respect to proportion of flow generation in sub-watersheds of Easkoot Creek
FC-Ca	Catchment (lower Matt Davis Trail)	Stetson (SBCWD)	1-2004 through 12-2005	Mean daily	• Supplementary flow record for hydrologic model calibration • Flow rating curve extends to 6 cubic feet per second (cfs) • Data to evaluate model accuracy with respect to proportion of flow generation in sub-watersheds of Easkoot Creek
			12-6-2005 through 1-11-2006	30 min. intervals	• Raw stream stage extend far above range of rating curve • Data cover periods of high runoff and flood flows in December 2005; documents timing and relative magnitude of flow peaks
BR (Black Rock Creek; major tributary to Easkoot Creek)	Panoramic Hwy.	Stetson (SBCWD)	1-2004 through 12-2005	Mean daily	• Supplementary flow record for hydrological model calibration • Flow rating curve extends to ~ 2 cubic feet per second (cfs) • Data to evaluate model accuracy with respect to proportion of flow generation in sub-watersheds of Easkoot Creek
			12-6-2005 through 1-11-2006	30 min. intervals	• Raw stream stage extend far above range of rating curve • Data cover periods of high runoff and flood flows in December 2005; documents timing and relative magnitude of flow peaks
Hwy1	Highway 1 bridge across Easkoot Cr.	Environ-mental Data Solutions	11-12-2006 through 5-24-2007	20 min. intervals	• Flow rating curve defined by two observations to 14 cubic feet per second (cfs) • Limited period of record • Data not well-suited for use in hydrologic model calibration

Table A2 Precipitation Data Summary

STATION	LOCATION	OPERATOR	PERIOD OF RECORD	OBSERVATION FREQUENCY	UTILITY AND LIMITATIONS
EK (Easkoot Creek)	Vehicle bridge to Stinson Beach NPS parking lot	NPS	10-1-2000 through 9-30-2007	15 min. intervals	• Significant source of rainfall data for hydrologic model • After Water Year 2006, vegetation canopy encroaches on gauge and data becomes unreliable (per NPS personnel) •
PG (Pine Gulch)	West edge Bolinas Lagoon near Bolinas	NPS	10-31-1998 through 8-7-2011	15 min. intervals	• Significant source of rainfall data for hydrologic model • Data incomplete in some years; some inconsistency in data
LTP (Laurel Treatment Plant)	Laurel Water Treatment Plant, Stinson Beach	Stetson (SBCWD)	6-9-2003 through 6-30-2005	30 min. intervals	• Significant source of rainfall data for hydrologic model • No station maintenance notes
MP (Middle Peak)	Middle Peak, Mt. Tamalpais	RAWS Network; MesoWest	5-2004 through 2-2012	Daily	• Significant source of rainfall data for hydrologic model • Substantial data gaps • No station maintenance information; Station MDEC1
LTP (Laurel Treatment Plant)	Laurel Water Treatment Plant, Stinson Beach	SBCWD	7-1-1997 through 12-31-2005	Daily	• Supplementary rainfall data for spatial distribution pattern
MO (Main Office)	District Office, Hwy. 1, Stinson Beach	SBCWD	7-1-2007 through 6-30-2011	Daily	• Significant source of rainfall data for hydrologic model • Supplementary rainfall data for spatial distribution pattern • Monthly totals provided, daily data available • Some data available for Laurel Treatment Plant in this period
LTP (Laurel Treatment Plant)	Laurel Water Treatment Plant, Stinson Beach	SBCWD	7-1-1978 through 6-30-1996	Annual	• Supplementary rainfall data for spatial distribution pattern
MMWD Facilities	Throughout Marin County	MMWD	7-1-1999 through 6-30-2010; earlier data available	Daily	• Significant source of rainfall data for hydrologic model • Supplementary rainfall data for spatial distribution pattern • Period of record extends prior to 1999 Stations include Corte Madera, Nicasio Dam, Kent Lake, Alpine Lake, Lake Bon Tempe, Lake Lagunitas, Phoenix Lake, Soulajule, Nicasio (town), Tocaloma, Hicks Valley

First Draft February 24, 2012
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TO: Chris Choo
Roger Leventhal
Marin County Department of Public Works
Flood Control and Water Conservation District

FROM: Matt O'Connor
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SUBJECT: Easkoot Creek Hydrology & Hydraulics Study

Appendix B. Evaluation of Suitability of the Golden Gate LiDAR Data

The purpose of this review is to evaluate the accuracy of the LiDAR-based Digital Elevation Model (DEM) created by the Golden Gate LiDAR Project (GGLP, 2011) for use in supporting the development of hydrologic and hydraulic models of Easkoot Creek. These models are being developed for the Easkoot Creek Hydrology & Hydraulics Study being prepared by O'Connor Environmental Inc. (OEI) for the Marin County Flood Control and Water Conservation District (District).

GGLP LiDAR Overview

The LiDAR data is available in several forms including raw point cloud data, filtered point cloud data, and a processed 1-meter resolution Bare Earth DEM. This evaluation focused on the processed Bare Earth DEM which was generated as part of the Golden Gate LiDAR Project (GGLP, 2011) and obtained from MCFC. The LiDAR data was collected during April through July 2010 and processed over the following two years before being released late-2011. The data uses the NAD 1983 UTM Zone 10N projection and the NAVD 88 vertical datum. The horizontal accuracy of the LiDAR is stated as < 1-meter Root Mean Square Error (RMSE) and the vertical accuracy is stated as < 9.25 cm (0.3-ft) (Hines, 2011). The vertical accuracy was achieved through adjustment to a series of 47 ground control points located throughout the two county study areas. The closest control points to Easkoot Creek were near Bolinas, Muir Woods, and Bon Tempe Lake.

In order to generate a bare earth surface from the raw point cloud data, all returns associated with vegetation and buildings are filtered prior to interpolating the surface. Although advanced filtering techniques are available and the dataset underwent a thorough data quality review, this process is not perfect and some inaccuracies in the data are expected. Additionally, features with steep slopes that are small relative to the point density tend to be poorly represented with LiDAR. These features tend to get 'smoothed' in the LiDAR due to returns missing the precise locations of inflection points in the topography. LiDAR is also not capable of penetrating through water so areas below water at the time of the data acquisition are expected to be poorly represented.

OEI Survey Overview

In order to provide a means of evaluating the accuracy of the LiDAR and to provide topographic data for the active channel of Easkoot Creek including the details of all road crossings (bridges over Easkoot Creek that may affect flow hydraulics), OEI conducted a topographic survey of Lower Easkoot Creek using a Topcon Electronic Total Station. The survey was conducted on December 7, 8, 9 and 13, 2011. The surveyed reach began just above the Highway One bridge at the east end of the town of Stinson Beach and ended at the last foot bridge near Calle Ribera, approximately 280-feet upstream of the Stinson Beach Firehouse Number 2 (Figure 1). The survey focused on characterizing the active channel of Easkoot Creek, including the dimensions of all bridge crossings in the study reach.

Four cross sections were surveyed at each bridge crossings to represent the geometry of the channel above the contraction reach, below the expansion reach, and at the upstream and downstream faces of each bridge. Intervening cross sections between bridges were surveyed as needed in order to characterize changes in channel dimensions and/or slope and ensure a maximum cross section spacing of 100-feet; 66 cross sections were surveyed in total. Four longer cross sections extending onto the floodplain were also surveyed along with isolated floodplain points in order to support the LiDAR evaluation process (Figure 1). The survey data was horizontally geo-referenced using Control Point 18 and Control Point 52 from District survey data (MCFC, 2011). Intervening control points were also surveyed and were found to agree within 1-foot horizontally. The vertical datum was set using NGS Benchmark 1718 located within the study reach (Figure 1). The NAD 1983 UTM Zone 10N projection was used along with the NAVD 88 vertical datum to be consistent with the LIDAR.

LiDAR Evaluation

Prior to performing the topographic survey, OEI conducted an initial review of the LiDAR by comparing it to previous 2007 and 2008 topographic surveys of the active channel of Easkoot Creek completed by MCFC staff. This exercise was performed to inform planning of the December 2011 field survey. Cross sections were “cut” from the LiDAR DEM along the MCFC cross section lines using the ArcGIS 3D Analyst extension. The comparisons confirmed our expectation that the channel would not be well-represented in the LiDAR DEM and the channel details tend to be smoothed relative to the surveyed sections. Comparisons to the OEI-surveyed cross sections (Figure 2) confirm this, and provide evidence of the need to use surveyed cross sections to simulate the active channel geometry for hydraulic modeling. Some of the differences in the cross section topography may be attributable to erosion and sedimentation of the channel which may have occurred over the two winters between the LiDAR acquisition and the OEI survey.

The primary goal of the LiDAR evaluation was to verify the accuracy of the LiDAR in areas outside of the active channel of Easkoot Creek in order to evaluate the utility of using the LiDAR to represent these areas for hydraulic modeling. To accomplish this, all points representing the active channel were filtered out, and then the difference between the OEI-surveyed elevation and the elevation of the corresponding spatial element in the LiDAR DEM was subtracted to produce an estimate of the error at each point. Overall, there is reasonable correspondence between the LiDAR and survey elevations, except in areas of particularly dense vegetation. The majority of the LiDAR elevations (83%) agreed with the surveyed elevations to within 2-feet and 68% agreed to within 1-foot (Figure 3). The LiDAR DEM elevations relative to the field survey are, on average, higher by 0.9-feet with Root Mean Square Error (RMSE) of 1.5-feet (Table 1).

Table B1 Calculated Mean Error (ME) and Root Mean Square Error (RMSE) for LiDAR points classified based on open and densely-vegetated areas.

Error Summary	ME (ft)	RMSE (ft)
All Points Outside of Active Channel	0.9	1.5
Open Areas	0.5	0.7
Densely Vegetated Areas	1.2	2.1

Due to the difficulties of distinguishing between LiDAR ground returns and vegetation returns, areas of dense vegetation are known to cause inaccuracies in LiDAR datasets. In order to investigate the potential for vegetation interference, we classified the points into areas of dense vegetation and open areas of pavement or bare ground (Figure 1). This stratification of the data revealed that the LiDAR tends to over-estimate elevations in both areas but more significantly in densely vegetated areas compared to open areas: mean error of 1.2-feet versus 0.5-feet and RMSE of 2.1-feet versus 0.7-feet (Table 1 and Figure 4). These findings are consistent with the results of LiDAR evaluations from other areas. For example, Norheim et al., (2002) found that LiDAR data from western Washington State over-predicted elevations by an average of 1.2-ft in densely forested areas and by 0.4-ft in sparsely-vegetated urban areas.

Evaluating the ability of LiDAR to predict elevations of individual points is complicated by potential differences in horizontal positioning implicit in the comparisons. A qualitative evaluation of the horizontal accuracy of the LiDAR was performed by visually inspecting the positions of prominent features such as road crossings in both the survey and the LiDAR. Overall the positions agreed well however offsets on the order of several feet were noted in some areas; this is consistent with the 1-meter stated horizontal accuracy of the dataset (GGLP, 2011). The directions and magnitudes of the offsets did not appear to be consistent thus we attribute these differences to horizontal positioning errors in the LiDAR rather than to any problems with geo-referencing the survey relative to the LiDAR.

As a final means of evaluating the LiDAR, the four long cross sections that were surveyed were compared to LiDAR-derived cross sections through the same alignments (Figures 1 and 5). The overall shape of the floodplain topography is well described by the LiDAR and prominent floodplain features identified in the survey are readily-identified in the LiDAR as well. As was seen in the point comparisons, elevations in floodplain areas that are relatively free of vegetation agree more closely with the survey than do elevations in areas of dense vegetation, and overall the LiDAR tends to report higher elevations compared to ground-surveyed elevations.

Brian Quinn of the Marin County GIS Division of the Community Development Agency recently completed a preliminary evaluation of the GGLP data in the Novato area providing the opportunity to compare our findings with a separate analysis of the dataset. Quinn's findings are generally similar to ours in that he found that the LiDAR over-predicted elevations in some densely vegetated areas by as much as 1.6 m due to misclassification of ground returns and that the LiDAR was more accurate in areas with little or no vegetation (Brian Quinn, personal communication).

Conclusions

The results of our evaluation of the LiDAR-derived Bare Earth DEM of lower Easkoot Creek revealed relatively good agreement in both the horizontal positions and elevations of floodplain features with a tendency to over-estimate elevations on the order of 0.5-feet in open areas and 1.2-ft in densely vegetated areas. The active channel of Easkoot Creek is represented reasonably well in some areas but is poorly represented in the LiDAR for purposes of hydraulic modeling. These findings are consistent with LiDAR accuracy evaluations from other areas and while they do reveal some differences, these differences appear to be small enough to permit use of the LiDAR in representing the floodplain topography in the hydraulic model of Easkoot Creek. The tendency of the LiDAR to over-estimate elevation suggests that a vertical adjustment of the LiDAR may be justified and/or the ground classifications edited. OEI suggests that this decision be deferred until a preliminary hydraulic model has been developed that will allow examination of the inundation patterns predicted by the model for comparison with observed flooding during historic storm events. This comparison will provide an additional means of evaluating the accuracy of the LiDAR for purposes of hydraulic modeling. It is further suggested that a sensitivity analysis be performed whereby the LiDAR is adjusted vertically and the effect of the adjustment on the predicted inundation is evaluated with the model. This sensitivity analysis will provide a means of making a final decision regarding any vertical adjustments to the data and provide a means of quantifying the uncertainty associated with the modeling results due to potential bias in the LiDAR.

References

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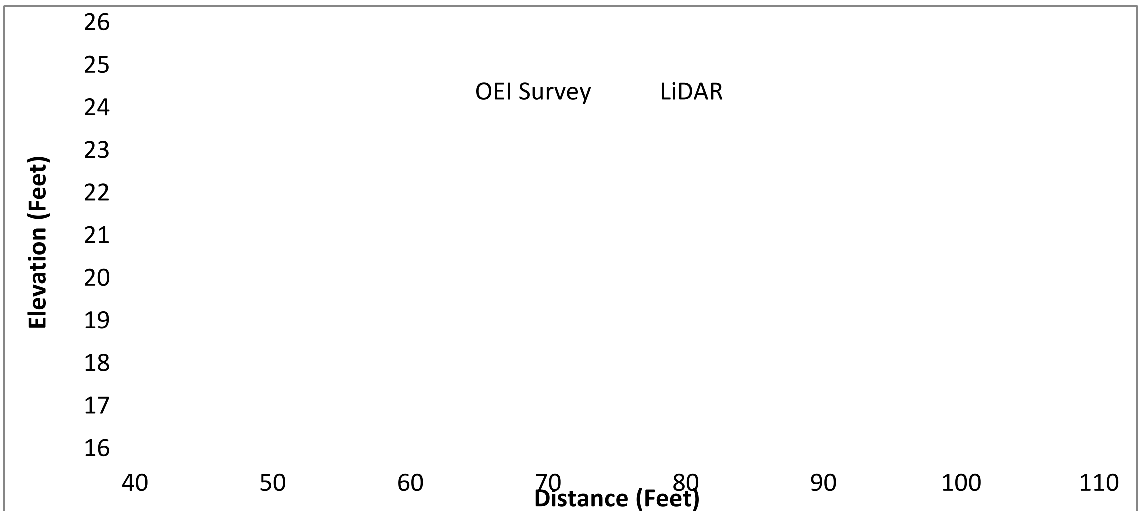
Stinson_Beach_Control_FFEs_10_12_11. (2011) Marin County Flood Control District. ArcGIS Point feature class of benchmark locations and elevations adjacent to and near Easkoot Creek by Stinson Beach**.**

Norheim, R. A., Queija, V. R., and Haugerud, R. A. (2002) Comparison of LiDAR and INSAR DEMs with Dense Ground Control. Proceedings of the 2002 Annual ESRI User's Conference, 11 pgs.
Quinn, Brian, personal email communication. "RE: GGLP - Golden Gate LiDAR Project data is in house!" E-mail to author. 23 Feb. 2012.

Figure B1 Overview map showing the locations of the OEI survey points, control points, and cross sections used in the LiDAR evaluation.



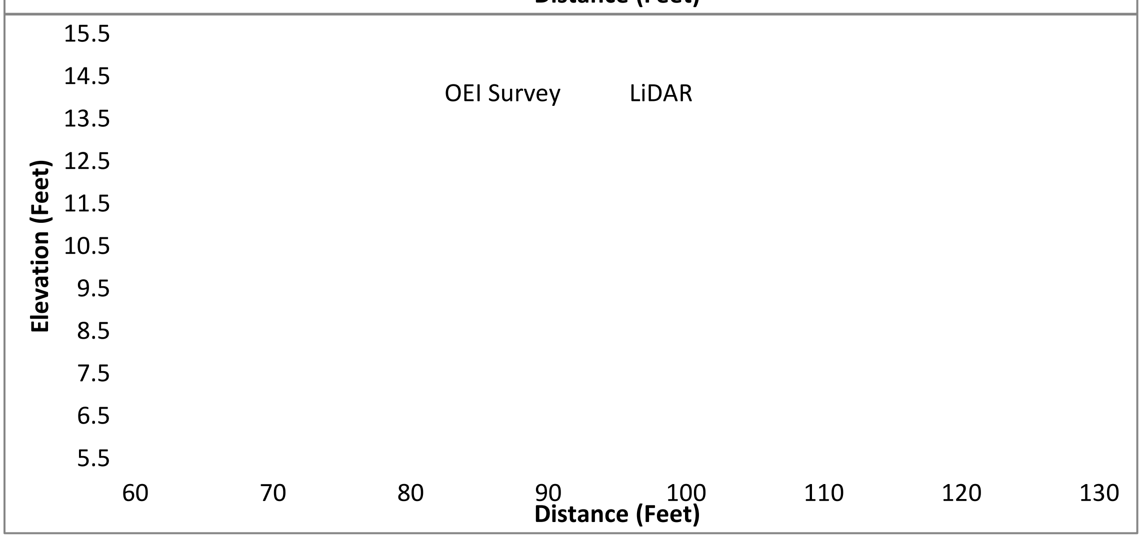


Figure B2 Example comparisons of active channel cross sections extracted from LiDAR with OEI-surveyed cross sections.

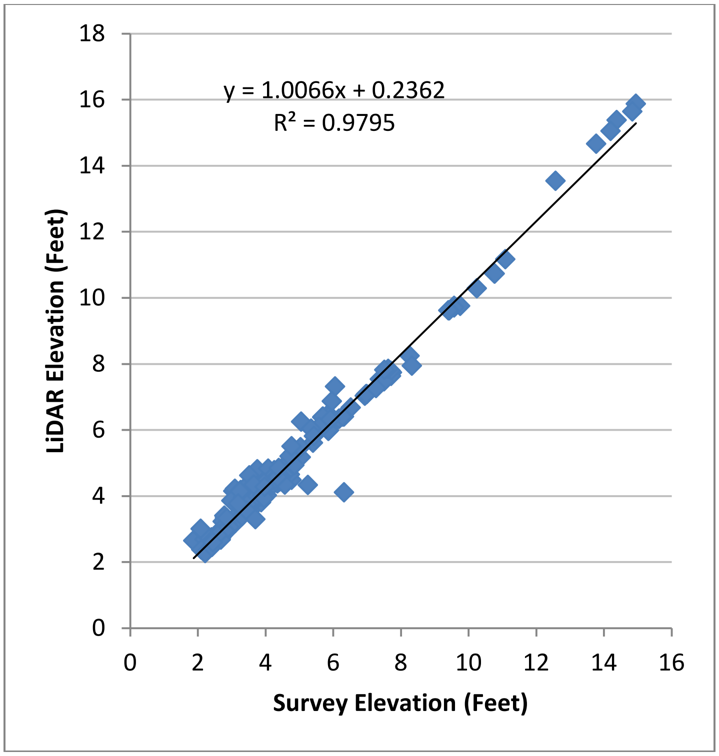
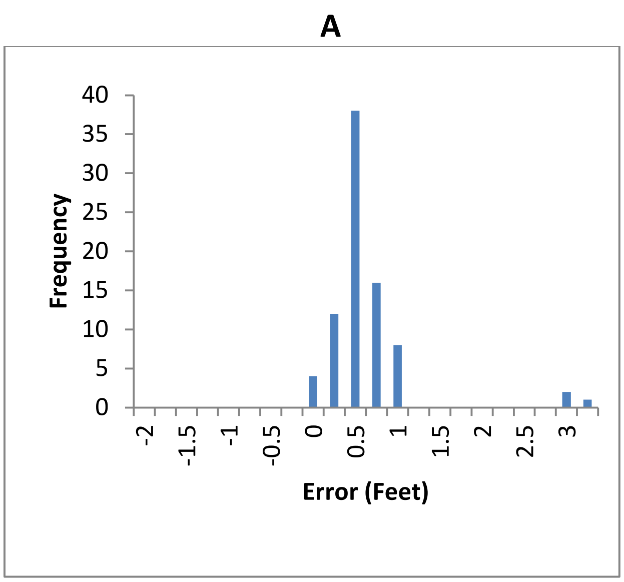


Figure B3 Relationship between OEI-surveyed elevations and LiDAR-derived elevations for areas outside of the active channel of Easkoot Creek (see Figure 1).



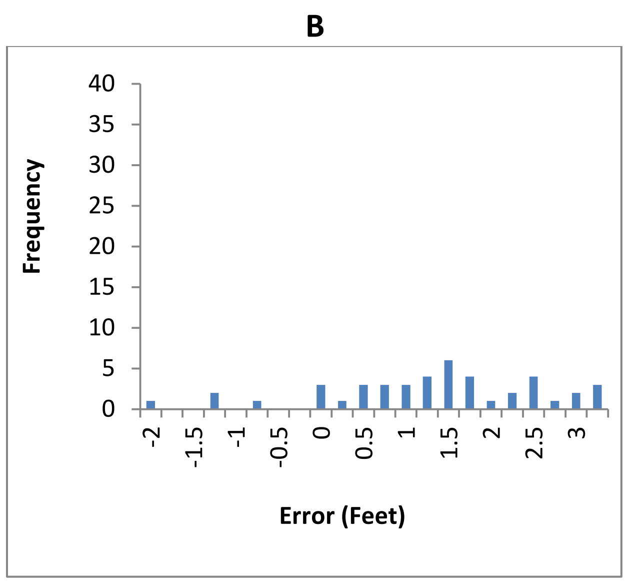


Figure B4 Distribution of errors in LiDAR elevations determined from comparison with surveyed data for open or sparsely-vegetated areas (A) and densely-vegetated areas (B).

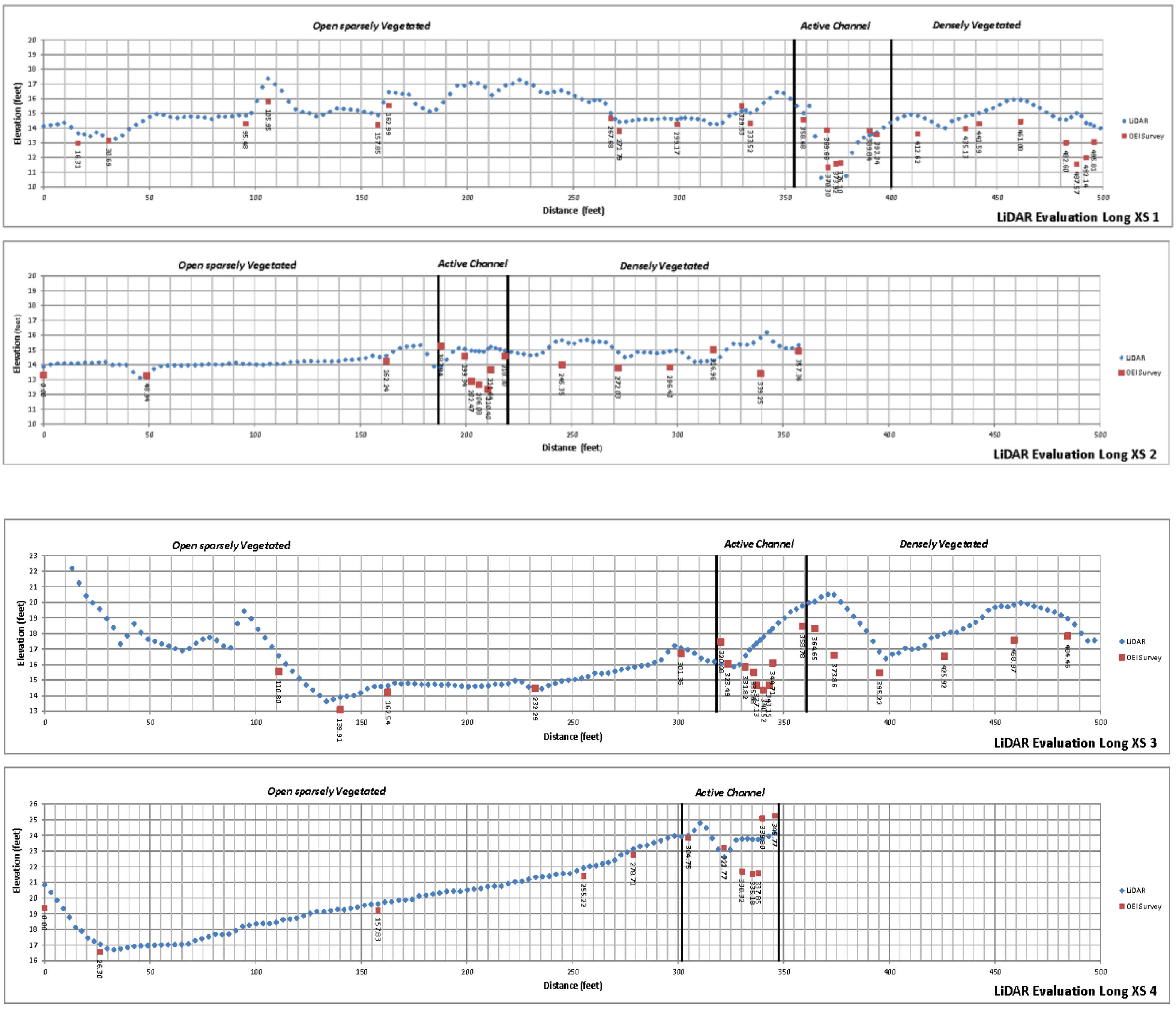


Figure B5 Comparison of floodplain cross sections extracted from LiDAR with OEI-surveyed cross sections.

Easkoot Creek Hydrology and Hydraulics Study

Appendix C. Hydrologic Analysis and Modeling of Runoff and Peak Flow

Prepared for

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Final Draft October 5, 2012
Revised December 6, 2012

Introduction and Purpose of Model

Development of a hydrologic model is a key step in the assessment of existing creek and floodplain conditions, and it is essential for the analysis of measures to provide flood protection, manage sedimentation and foster salmonid habitat and passage. All of these have been identified as management goals for Easkoot Creek.

Previous investigations of Easkoot Creek have produced estimates of peak flows at various return intervals, which are summarized in a recent report (MLA, 2009). These estimates vary widely and are based primarily on rational method calculations or regional regressions, although one source refers to an HEC-1 model without, however, supplying any documentation (Spangle, 1984). The present effort aims to reduce the uncertainty associated with the previous peak flow estimates and develop a model which makes comprehensive use of the available rainfall, flow and GIS data to provide more accurate estimates of peak flows in Easkoot Creek. The modeling software selected is the Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS), which includes a number of modules that permit analysis beyond the limitations of simple lumped and empirical methods.

Because a number of years of recent stream gage data are available for Easkoot Creek, along with synchronous rainfall data, the modeling strategy made use of historical data for specific high-flow events to calibrate an existing condition model, which was then applied to the study of various design storms and management alternatives. Implicit in this thinking was the notion that our goal is an *event* model of the creek, because our interest is in the effects of extreme hydrologic events on the creek. Although a continuous simulation of the creek would provide additional information of general interest, it was not judged appropriate in this case. The model of rainfall-runoff rates will be used to generate hydrographs for Easkoot Creek that will be used as input to a hydraulic model of lower Easkoot Creek.

Data Selection and Preparation

General

High-frequency rainfall data are required for the use of HEC-HMS as an event model. National Park Service rain gauges provided rainfall data for 15-minute intervals for two sites in or near the Easkoot Creek watershed[10]. One site, Station EK, is located in the lower Easkoot Creek watershed (see Figure 6); Station PG is located on the west shore of Bolinas Lagoon at the mouth of Pine Gulch. Data were available from one or both sites for water years 2001 through 2011. In addition, 30-minute data collected by Stetson Engineers, Inc. were obtained for parts of water years 2003 through 2005. To assist in estimating rainfall distributions across the watershed and the region, daily rainfall data were also obtained from gauges located at Marin Municipal Water District (MMWD) facilities: Nicasio Town, Nicasio Dam, Alpine Reservoir, Bon Tempe Reservoir, Kent Dam, Soulajule, Phoenix Lake, Tocaloma Town, Hicks Valley, Corte Madera, and Lake Lagunitas[11]. Daily rainfall data were also obtained from the

[10] Summarized in Background Information and Data Acquisition Plan.
[11] Data sets maintained by and obtained from Balance Hydrologics, Inc. (an MMWD contractor).

RAWS network station on Middle Peak (Mt. Tamalpais), and at Stinson Beach County Water District (SBCWD) facilities in the lower Easkoot Creek watershed[12].

Flow data were obtained from the National Park Service for Station EK at Stinson Beach for water years 2002 through 2010, along with supporting information regarding iterations of the stage-discharge rating curve, including short-term shifts in the rating and associated stream discharge measurements. For portions of water years 2004 through 2006, supplemental stream flow data were available for some Easkoot Creek tributaries. These data were collected by Stetson Engineers for SBCWD.

The flow data for Station EK were selected for use as the primary calibration dataset because the data are well supported by field discharge measurements and the site is well located for calibration of the Easkoot Creek watershed as a whole. The flow data obtained from Stetson were not as well supported by field discharge measurements, but the associated stage measurements offered a means to estimate flow at the sub-basin scale, as well as the timing and relative magnitude of runoff peaks, relative to data from Station EK and model predictions.

Detailed topographic data were available for the watershed from a recently acquired LiDAR-based Bare Earth Digital Elevation Model (DEM) which provided the basis for defining routing reaches, slopes, and sub-basin boundaries described in greater detail in Sections 3 and 6 (Hines, 2011). Soil information was available from the US Department of Agriculture internet portal for soil survey data[13]. A detailed spatial data base for vegetation was provided by the National Park Service, which was supplemented by reference to available aerial imagery and simplified for modeling purposes.

HEC's Geospatial Hydrologic Modeling extension HEC–GeoHMS was used to define the basic stream network and catchments for the model, on the basis of the LiDAR dataset mentioned above. This pre-processing step followed essentially Chapter 6 of the *HEC-GeoHMS User's Manual*. Subsequent development of the basin model was carried out using generic GIS tools to create and edit shape files for the network and sub-basins.

Selecting Flow Data and Filling Data Gaps

The flow records for Station EK were examined to identify likely historical storms for use in calibrating the HMS model. There were four peaks with a reported flow of around 100 cubic feet per second (cfs) or greater, as noted in Table 1. Stream discharge data during the period of peak runoff were missing for two of these four peak flow events. Although stage is reported continuously throughout these events, on both December 31, 2005 and January 25, 2008 there is a gap of several hours with no reported flow values. These gaps in the discharge record reflect the fact that stream stages exceeding the range of discharge measurements upon which the rating curve was based. In addition, these events caused changes to the stream bed that required adjustments to the rating curve, creating additional uncertainty regarding the relationship between stage and discharge. During each of these two storms the reported water level peaked during the gap in reported discharge. Consequently, it was necessary to estimate the actual peak discharge.

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[12] Summarized in Background Information and Data Acquisition Plan.
[13] http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx

Table C1 Peak discharge events at Station EK.

Date	Highest reported discharge (cubic feet per second (cfs))	Period of gap in reported discharge record	Estimated peak discharge (cubic feet per second (cfs))
12/27/2004	110		
12/22/2005	103		
12/31/2005	98.0	03:30 – 14:00	175
1/25/2008	102	19:45 – 23:45	142

Peak discharges during these two gaps in the reported discharge record were estimated using the conveyance-slope method of high-flow extrapolation as described by US Geological Survey (Rantz et al, 1982). The resulting estimated peak flows are given in Table 1. This method relies on Manning's equation for steady flow,

$Q = KS^{0.5}$

where the conveyance K equals $(1.486/n)AR^{2/3}$, when English units are used. In this equation A is cross-sectional area, R is hydraulic radius and n is channel roughness. Values of K are computed for a given water level h, using an estimate of Manning's n and values of A and R from a measured cross section, and the corresponding Q is calculated. To apply this in the present study, channel bed slope was used to estimate the energy slope S, usually a reasonable assumption at high flows; and Manning's n was estimated at 0.05 for December 2005 and 0.07 for January 2008, reflecting our understanding that channel roughness had increased over the intervening years owing to growth of riparian vegetation following a habitat restoration project completed in 2004.. This process effectively extended the rating curve for stream stages exceeding the range of discharge measurements.

Since there was a shift in the rating curve caused by channel erosion and sedimentation corresponding with the gaps in the discharge record, we used the best available channel cross section information for the period before *and* after the gap. This produced two alternate estimates of flow during the gap, one consistent with the preceding stage data and rating curve and one consistent with subsequent stage data and rating curve. Regarding these as upper and lower bounds of the true flow, we interpolated between the two so as to place the peak about 60% of the way up from the lower estimate to the higher one, with the curve connecting smoothly with the reported hydrograph before and after the gap. The 60% value was chosen as a somewhat conservative estimate of the central tendency indicated by the two bounding curves.

Figure 1 shows the development of the hydrograph for peak flow on December 31, 2005 during the gap in the discharge record. The same procedure was used for January 2008; however, the transition to the reported flow data is less smooth in that case, probably because the cross sections used for 2008 were actually from 2006 (representing the period *before* the gap) and 2011 (representing the period *after*). Figure 2 illustrates the cross sections used for the conveyance-slope calculations at Station EK. The cross-sections from 2004-2006 were extended toward the left bank (facing downstream) to conform to the surveyed topography from 2011

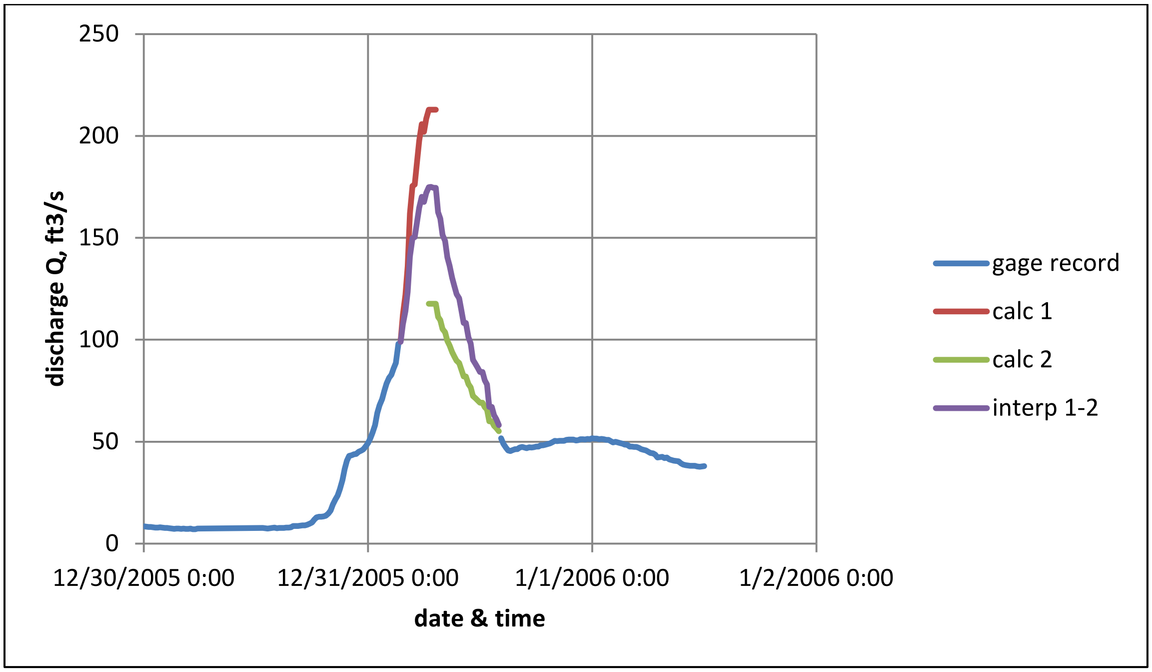

Figure C1 Estimates of missing flow, December 2005.

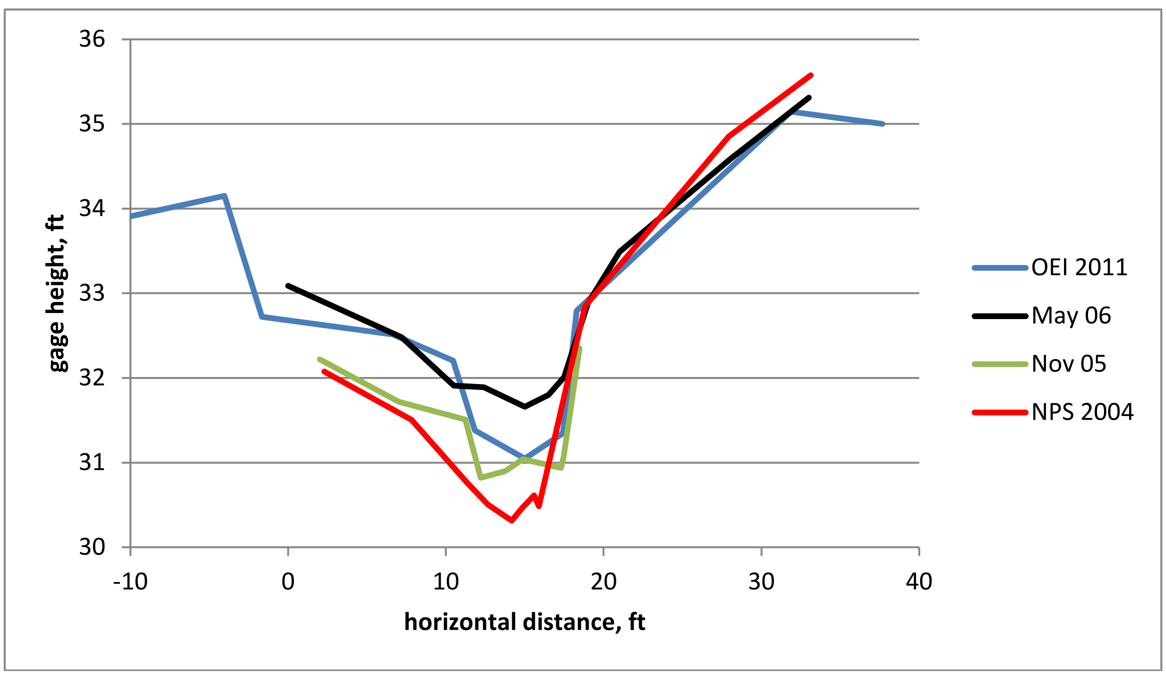

Figure C2 Variation in cross section at Station EK (view downstream).

Selection of Rainfall Gauge Stations and Estimation of Spatial Distribution of Rainfall

To represent the spatial variability of rainfall in the model, a network of multiple high-frequency rain gauges throughout the study watershed would be ideal. However, in the present case all the high-frequency data available were from sites relatively low in the watershed. To quantify the variability in rainfall over the watershed as a whole, particularly at higher elevation in portions of the watershed on the west slope of Mt. Tamalpais, we tabulated 3-day rainfall totals centered on the day of maximum rainfall at a number of rain gage locations with at least daily data for the December 2004 and January 2008 events. A spline technique in ESRI ArcEditor was then used to interpolate 0.5-inch rainfall isohyets for each event (Figures 3 and 4). A third interpolation was also developed using a 6-yr average annual precipitation to develop 2-inch rainfall isohyets for consideration in developing rainfall distributions for design storms (Figure 5). In all cases, it was necessary to extrapolate the isohyets towards the south, in order to cover the full extent of the Easkoot watershed. It was not possible to interpolate a distribution for the December 31, 2005 event owing to the lack of available data at a high elevation station near the watershed.

The various rainfall contours (isohyets) were then used to determine scaling factors which allowed us to estimate the rainfall in all model sub-basins (see Figure 6 for basin locations) on the basis of a primary rainfall record. For the modeled storms in December 2004 and December 2005, the rainfall record at Station EK was selected as the primary rainfall record since it is located within the Easkoot Creek watershed and provides rainfall data in 15-minute intervals. For the January 2008 event, rainfall data from Station EK were believed to be untrustworthy relative to early periods of record owing to growth of riparian forest canopy[14], so for that event, rainfall data in 15-minute intervals from Station PG were used. For December 2005, daily records were not available for both a high and a low elevation station, so two different sets of scaling factors were prepared for use in modeling the distribution of rainfall over the watershed in that storm. One developed a 2-event average based on the 2004 and 2008 events, and the other used the overall average of 6 years' data. Ultimately, the 2-event average was used for the December 2005 storm (see Section 7 below). All four sets of scaling factors are tabulated in Table 2.

We also considered using the PRISM dataset and/or the isohyets available on the Marin County web site[15] to assist in developing the rainfall distributions. The PRISM dataset was found to be too coarse to adequately represent the distribution across the watershed, as the entire watershed area is represented by only eight PRISM cells. The gradient indicated by the Marin County isohyets is quite similar to the gradient indicated by the 6-yr average annual isohyets. We relied on the Marin County isohyets as a check on our distribution; however, we chose not to use it directly owing to the fact that we have event-specific data for two of our events of interest and our 6-yr average distribution is based on a more complete set of stations in the vicinity of the watershed.

[14] Pers. comm., D. Fong, National Park Service
[15] www.marinmap.org

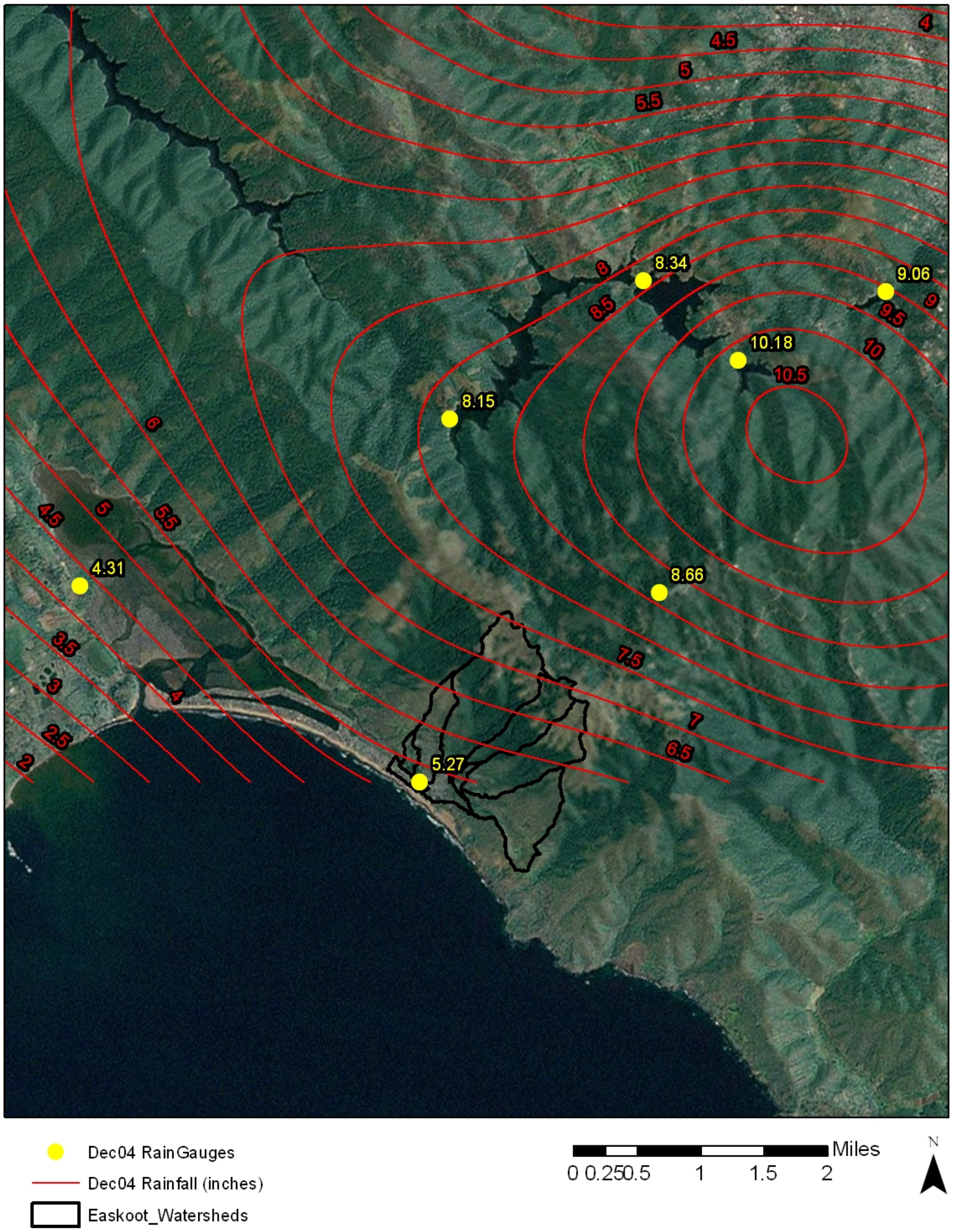

Figure C3 Isohyets for the December 2004 event.

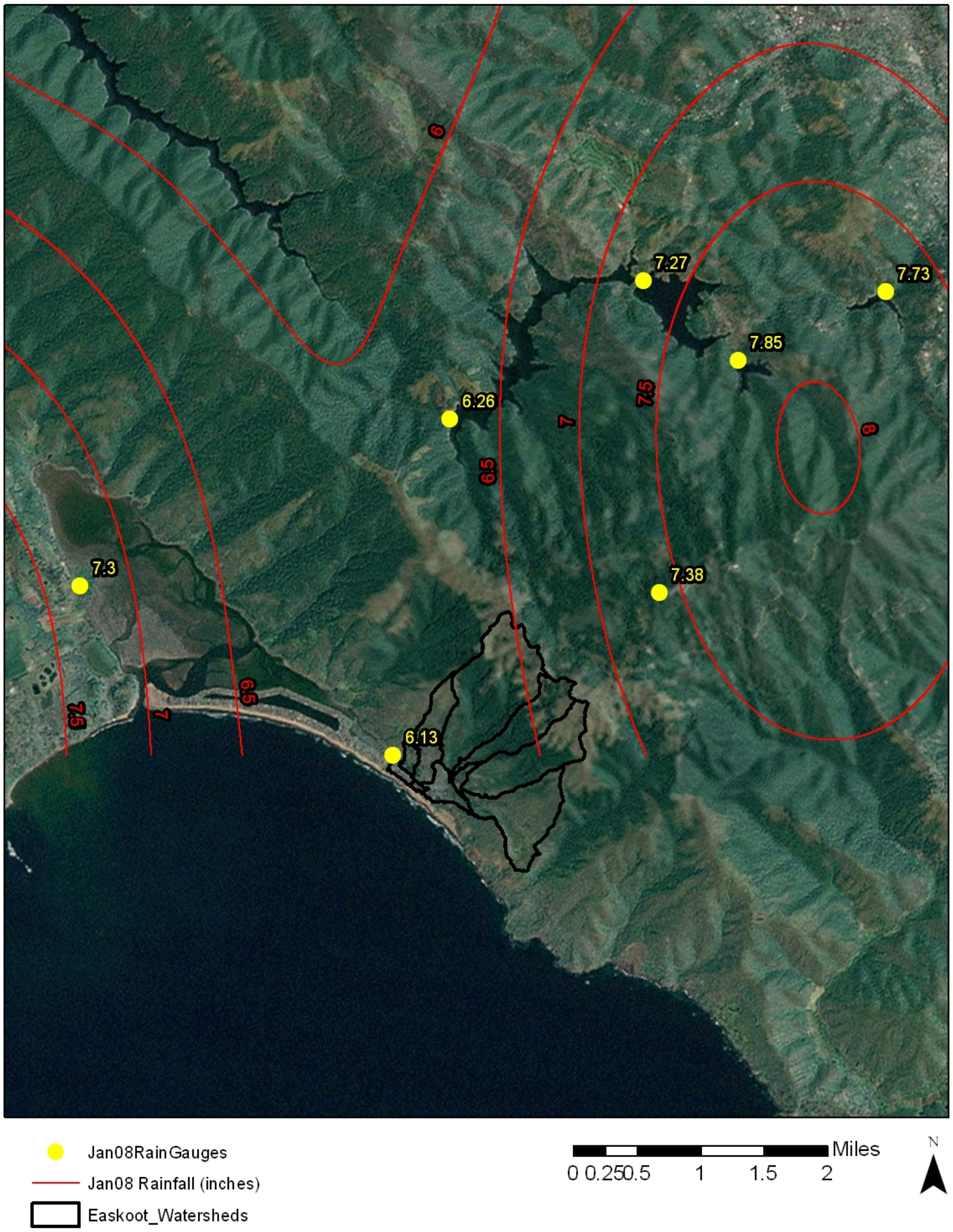

Figure C4 Isohyets for the January 2008 event.

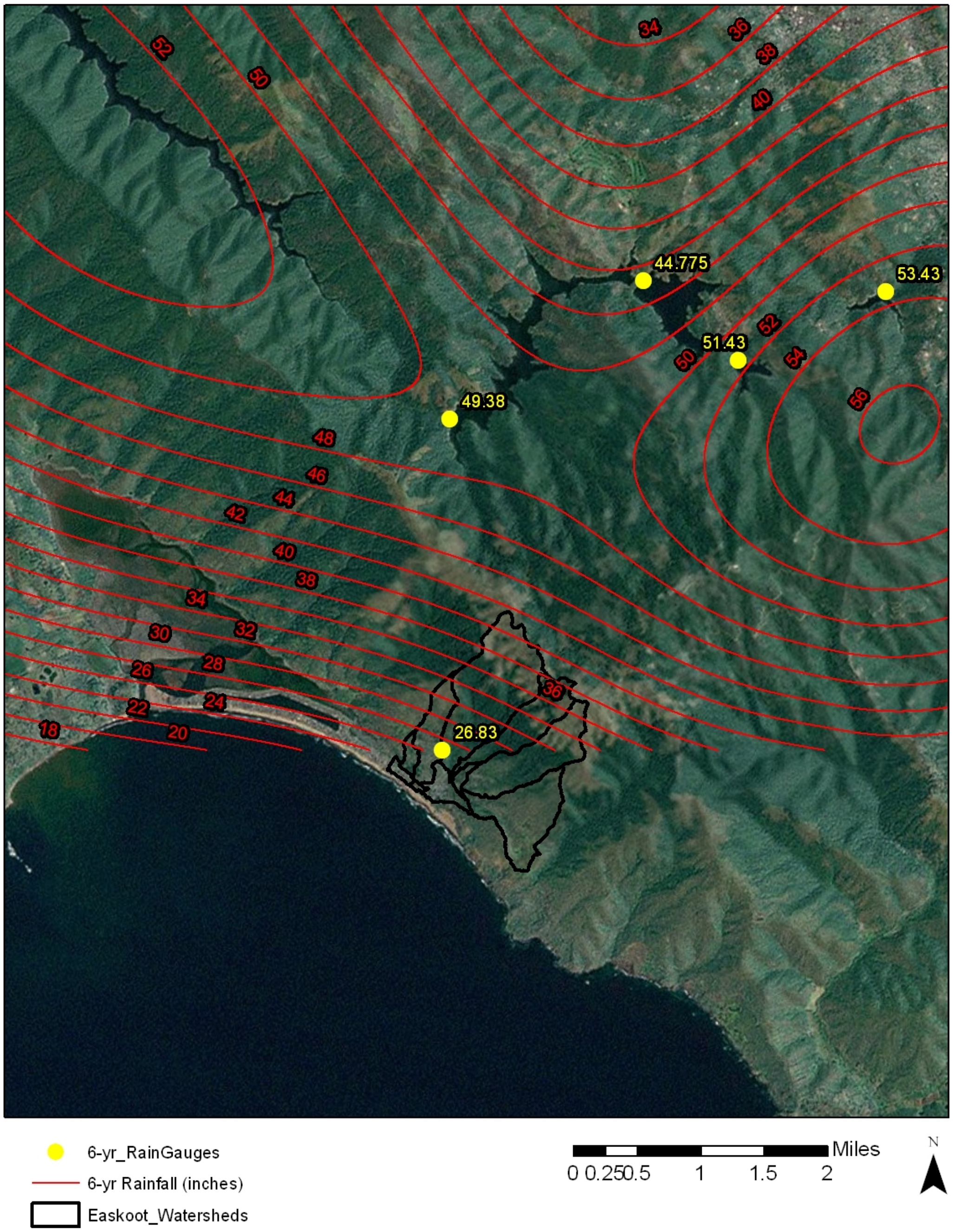

Figure C5 Average annual isohyets for available 6-year record.

Basin Model

A watershed model of Easkoot Creek – a basin model, in HEC terminology – was created in HEC-HMS, featuring four major sub-basins above the gauge Station EK, corresponding to the major tributaries, and three smaller transitional sub-basins. The drainage area contributing to the NPS gauge at Station EK is computed to be 1.43 mi^2. All these sub-basins together form a network leading to the outlet of Basin (sub-basin) G, where model predictions can be compared with the observed discharge record. An additional set of basins (H, J and K) downstream of the gage site were defined as well; although they played no role in calibration of the hydrologic model, runoff rates downstream of the gage were required for use in the hydraulic model. The sub-basins used in the basin model are illustrated in Figure 6.

Not shown explicitly in the figure, but essential to the calculations, are the modeled connections between the sub-basins shown in the figure. When two sub-basins like A and B meet, their runoff hydrographs are combined and routed through a downstream reach to the next stream (basin) junction. The calibrated model includes routing reaches that pass through sub-basins E, F and G.

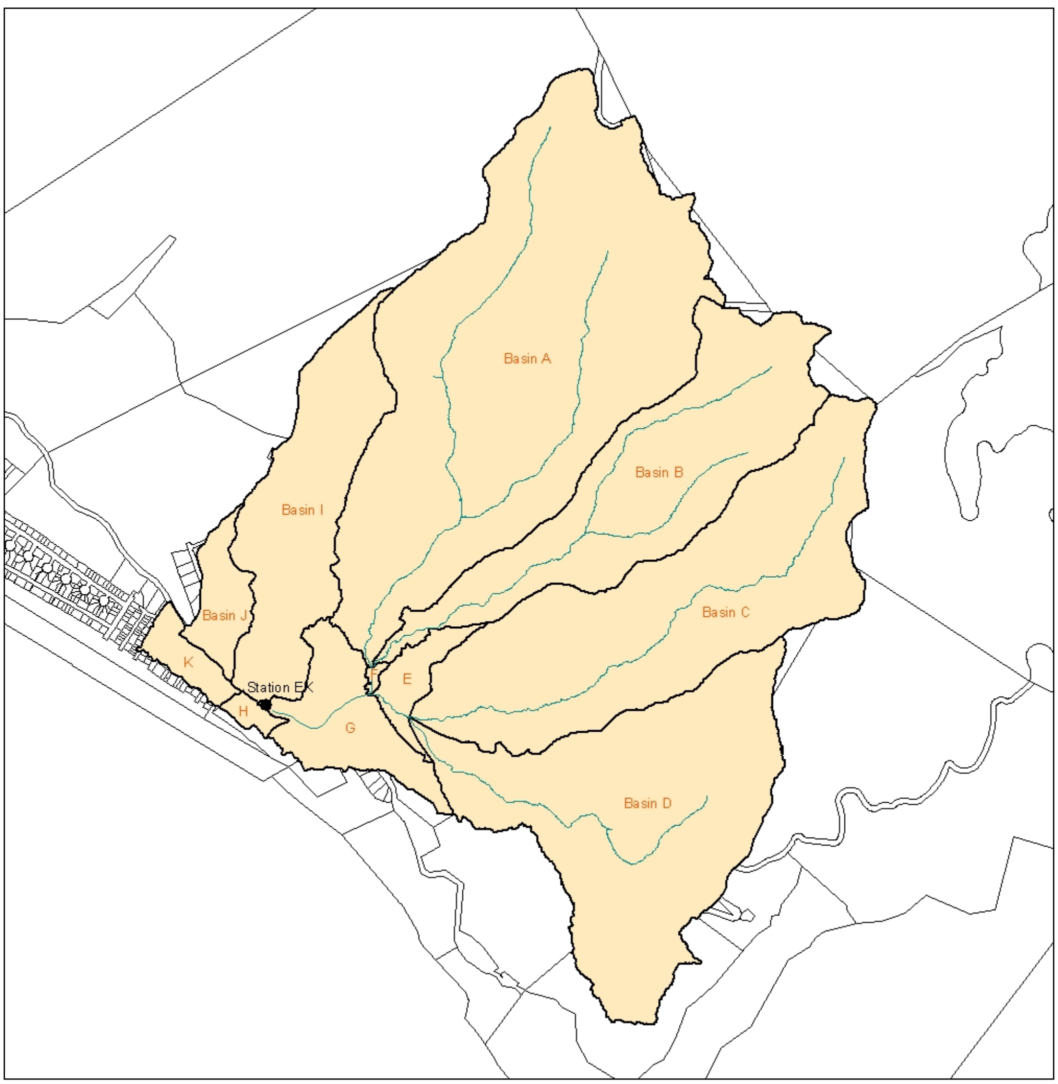

Figure C6 Easkoot Creek basin model.

Table C2 Scaling factors for rainfall distribution.

Period	Dec. 2004	Jan. 2008	2-event average	6-year average
Scaling Basis	Site EK	Site PG	Site EK	Site EK
Sub-basins				
A	1.264	0.872	1.22	1.430
B	1.229	0.891	1.21	1.381
C	1.154	0.891	1.17	1.254
D	1.037	0.879	1.11	1.044
E	1.069	0.853	1.10	1.039
F	1.061	0.849	1.10	1.018
G	1.031	0.847	1.08	0.967
H	1.008	0.843	1.07	0.931
I	1.132	0.845	1.13	1.175
J	1.064	0.841	1.09	1.035
K	1.020	0.841	1.07	0.958

SCS Loss Model

HEC-HMS conceptualizes the runoff process in a sub-basin in two separate simulation elements, a *loss* model and a *transform* model. The loss model has the function of partitioning the input rainfall into the portion that runs off during the modeled event and the portion that infiltrates to the soil (and is *lost* from the point of view of the model for the runoff event) while the transform model converts the runoff volume into a hydrograph on the basis of watershed characteristics. There are also methods of accounting separately for canopy and surface storage, but these are used primarily in continuous simulation applications and were not utilized for this model application.

Among loss models available in HEC-HMS, the well-known SCS (Soil Conservation Service) curve number method may be the most widely used. It has the useful advantage that curve numbers, which represent the infiltration capacity of the landscape, can be estimated on the basis of observed land cover and soil type, and it is widely used in hydrologic analyses in the North Bay region and elsewhere. We did not find local examples of the use of other HMS loss models, so we selected the SCS method, considering that it has the advantages of a familiar, oft-used approach. The availability of stream discharge data for model calibration was expected to substantially increase our confidence in this application of the SCS loss model. For this application, we chose the traditional model in HEC-HMS, which accepts input data which are lumped by sub-basin, rather than the Gridded SCS model. The latter might have been appropriate as well, but we judged that the subdivision of this thousand-acre watershed into 11 sub-basins provided a comparable result.

The SCS loss model requires distributed information on hydrologic soil groups and land cover. The vegetation data layer was clipped and intersected with spatially distributed soil data using standard GIS techniques. Figure 7 shows the breakdown of watershed areas by hydrologic soil group (HSG); the vegetation categories are too numerous to conveniently display. The intersected data were tabulated in a spreadsheet according to the SCS land cover categories of pasture, grassland, or range; brush; woods-grass; woods; and urban, and the results used to create a weighted curve number for each sub-basin.

For convenience, this was done using the graphic interface of the NRCS TR-55 model (USDA, 2009). For the publicly-owned watershed areas, we judged the hydrologic condition to be good. For urban areas, aerial photos were consulted to provide more specific information on the extent and intensity of urbanization.

The resulting initial curve number values (CN) are displayed in Table 3. Note that since the model was to be calibrated to measured flow, these values were subject to adjustment in the calibration process; the table also shows the calibrated values of CN, along with calculated values of time of concentration (see next section). Lower values of CN are associated with slower runoff rates.

In the SCS loss method, the initial condition of a sub-basin is normally represented by the assumption of an *initial abstraction* I_a that is defined as 20% of the *potential maximum retention* S (the latter is a measure of the overall ability of a sub-basin to abstract and retain storm precipitation, derived from the curve number). However, HEC-HMS allows the initial abstraction to vary, and we took advantage of this feature. This will be discussed under *Model Calibration* below.

Table C3 Curve Numbers (CN) and Time of Concentration, for all sub-basins.

Sub-basin	Initial CN	Calibrated CN	Time of Concentration, hr
A	65	58.5	0.516
B	67	60.3	0.573
C	69	62.1	0.535
D	70	63	0.570
E	76	68.4	0.237
F	76	68.4	0.168
G	85	76.5	0.272
H	92	82.8	0.566
I	79	71.1	0.498
J	77	69.3	0.359
K	90	81	0.328

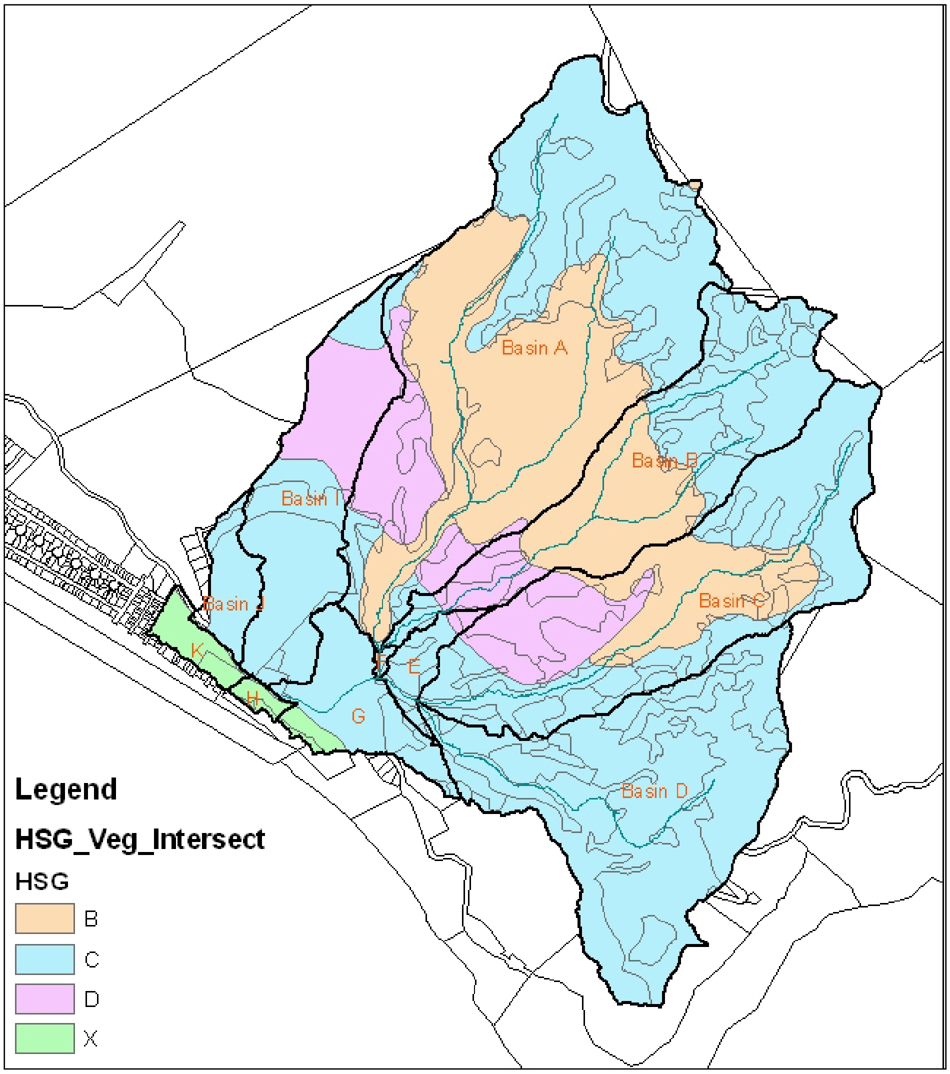

Figure C7 Hydrologic soil groups with vegetation polygons.

Clark's Unit Hydrograph Transform Model

HEC-HMS includes the SCS unit hydrograph transform model which derives the hydrograph for a sub-basin from a single parameter, the lag time. Lag time is defined as 60% of the *time of concentration* (the maximum travel time for flow in the sub-basin). This transform method is rather inflexible with respect to model calibration; the shape of the hydrograph is in our experience relatively insensitive to varying the time of concentration within reasonable limits. However, HEC-HMS offers another synthetic unit hydrograph method, the Clark Unit Hydrograph Transform, which uses the time of concentration as well but adds a second parameter, the *storage coefficient* R, which is a calibration parameter defining a linear reservoir to account for storage effects. This method was selected for the present study because it allowed for more flexibility during calibration.

The graphic interface of the NRCS TR-55 model was used to calculate the time of concentration for all sub-basins in the model (USDA, 2009). The resulting values are displayed in Table 3. As expected, peak discharge was found to be insensitive to changes in time of concentration (an increase of up to 30% in time of concentration changed peak discharge by less than 2%). Consequently, time of concentration was not considered a useful calibration parameter, and the Clark method was used. Initial values of R were estimated from the potential maximum retention S, and extensive use of this parameter was made during the calibration process (see Section 7).

Other Modeling Decisions

The three routing reaches E, F, and G (named for the sub-basins in which each lies) were modeled using the Muskingum-Cunge method in HEC-HMS. This method assumes that backwater effects are not important but is otherwise as versatile as any of the other HEC-HMS methods (USACE-HEC, 2000). The length and slope of each reach were obtained from LiDAR topographic data; channel bottom width, side slope and roughness (n) were estimated in the field. In general, the model was found to be insensitive to these parameters, probably because the routing reaches are relatively short. Variation of n over a range from 0.04 to 0.07 affected peak discharge by less than 1% and had a similarly minor effect on timing. Table 4 shows the routing reach parameters used in the model.

Table C4 Muskingum Cunge routing parameters.

Reach	Length, ft	Land slope	n	Bottom width, ft	Side slope
E	612	0.0229	0.05	8	1
F	342	0.0117	0.05	6	1
G	1433	0.00768	0.05	10	1.5

The HEC-HMS model time step was set at 1 minute, following the guidance that the time step should be no more than 0.29 times the lag time for the smallest sub-basin, in this case Basin F. Base flow was modeled using the exponential recession model, setting the parameters roughly by visual comparison with the recession measured in the aftermath of the historical storms modeled; this method was judged satisfactory since base flow is a relatively unimportant part of this study, which is focused on high flow events.

Model Calibration

Selection of Storm Events

Four storms with peaks over 100 cubic feet per second (cfs) were identified as candidate events to model (see Table 1). However, some of these peak events present special problems for calibration. The December 2004 storm is perhaps the least problematic event; it is preceded by a period of over two weeks of essentially dry weather, and the important rainfall occurs essentially within two days. The January 2008 storm is also a fairly concentrated single event, except that over an inch of rain occurred a couple of days before the main event. Both of these are acceptable candidates for modeling a large discrete storm on Easkoot Creek.

The two peaks in December 2005, however, are more problematic. The peak on December 22 was the highest point in a serial peak event, and this storm proved difficult to model and was not considered further. The peak flow on December 31 fits the definition of a unique event better than the earlier December (serial) peak, although it is clear that the watershed was unusually well-saturated before the event. The storm event the peaked on December 31, 2005 was selected for modeling, along with the 2004 and 2008 events, and will be referred to in the following sections as the December 2005 event. Having three different storms to model in three different years offered an opportunity to develop a robust calibration of the HEC-HMS model.

Initial Calibration to December 2004

We began with December 2004 (the smallest of the three) and calibrated the model essentially in two steps: first the runoff volume was calibrated by optimizing the initial abstraction I_a, and then the overall hydrograph shape and peak value were adjusted by varying R.

It was found that if the initial curve numbers were retained, I_a had to be about 0.6S, which seemed unreasonably high given that over eight inches of rain had already been recorded that season. It seemed most appropriate to adjust the curve numbers for all sub-basins proportionately, adopting the values shown above in Table 3, which were scaled down by 10% from the initial values. With these reduced curve numbers, the volume was satisfactorily calibrated at more appropriate I_a values. Table 5 in the next section shows the calibrated values of I_a for all basins and for all storms. The values for the December 2004 storm are retained for January 2008, while calibration of the December 2005 event required them to be considerably lower, consistent with high antecedent moisture conditions for the 2005 event.

Adjustment of the Clark storage coefficient R proved to be a flexible means to adjust the hydrograph shape and peak so that they resemble the observed record for the 2004 storm. An initial estimate of R for each sub-basin was made on the basis of potential maximum retention S. Although these numbers proved to be too low, their proportions were nevertheless retained during the calibration process, but they were scaled up by a common factor. The resulting values are shown in Table 6.

Model results for the 2004 event were also compared to water level records collected by Stetson at various tributary locations on 30-minute intervals during the storm. Peaks stream stages at around 6:00 and 9:00 AM on December 27, 2004 at all Stetson sites compares favorably with the model results. Peak values of discharge from the Stetson records were not used in the calibration because the rating curves for these gauge stations did not include observations of any significant flow peaks.

Calibration to January 2008 and December 2005

To represent the spatial distribution of rainfall throughout the watershed in the December 2005 event, both the 2-event average and the 6-year average (see Table 2) were tested using the initial calibration for 2004. The former was found to better match the observed results and was used for the calibration.

Application of the calibrated model for 2004 to the storms of 2008 and 2005 presented two issues. The values of I_a from 2004 worked well for 2008, but the December 2005 event required them to be drastically reduced, as was expected based on the documented characteristics of the storms in late December 2005. Nevertheless, the model as calibrated for December 2004 substantially over-predicted the peaks for both of the other events. It was necessary to increase values of R to compensate. The resulting calibration values of I_a and R were applied to all three storms and represent the final calibration of the model, with the incorporation of minor adjustments that were made to the base flow portion of the model. Figures 8-10 illustrate the final calibrations, and Table 7 compares modeled volumes and peaks with the observed values for each storm.

Table C5 Calibrated values of Initial Abstraction (I_a) for all modeled storms (inches).

Sub-basin	*December 2004*	*January 2008*	*December 2005*
A	2.81	2.81	0.54
B	2.57	2.57	0.50
C	2.35	2.35	0.45
D	2.24	2.24	0.43
E	1.65	1.65	0.32
F	1.65	1.65	0.32
G	0.92	0.92	0.18
H	0.76*	0.76*	0.15*
I	1.49*	1.49*	0.29*
J	1.62*	1.62*	0.31*
K	0.86*	0.86*	0.17*

*Values for sub-basins H, I, J, K were not part of the initial calibration

Table C6 Calibrated values of Storage Coefficient R (hours).

Sub-basin	*Initial estimate*	*Final calibrated value for all storms*
A	0.37	3.34
B	0.34	3.05
C	0.31	2.78
D	0.30	2.66
E	0.22	1.96
F	0.22	1.96
G	0.12	1.09
H	0.06	0.54
I	0.18	1.65
J	0.21	1.85
K	0.08	0.69

Table C7 Calibrated model results.

Storm	*Ratio of modeled to observed volume*	*Ratio of modeled to observed peak discharge at outlet*
December 2004	1.06	0.82
January 2008	0.91	1.03
December 2005	0.88	1.07
Mean	**0.95**	**0.97**

Calibration Discussion

The calibration presented here derives strength from two main sources. It uses inputs from the SCS curve number method, an empirical yet somewhat physically-based method with which we have experience; and it makes use of measured rain and flow data for three major storm events of recent years to make major adjustments to the initial parameter values in the model.

The quality of these data is an important consideration in evaluating the strength of the calibration. The primary data used were collected by NPS staff, and the flow data in particular were carefully and consistently handled. The rating curve was adjusted several times, generally in response to changes in the section control. Nevertheless, the peak flow values for the December 2005 and January 2008 events exceed the highest direct discharge measurements at the gage location and thus represent informed estimates of the actual discharge. Additional uncertainty arises from the assumptions regarding rainfall distributions across the watershed. While the distributions are based on interpolations of actual rainfall data, the model is driven by a single high-frequency rainfall record that was scaled to reflect the spatial variability in rainfall. While this approach should represent actual rainfall amounts with reasonable accuracy, local-scale variations in rainfall timing and intensity are likely not captured.

The SCS loss method as normally used (i.e. relying on the curve numbers tabulated for different combinations of land cover and hydrologic soil group) appears to be overly conservative in the sense of understating loss rates, at least in this particular application. To model the first storm we found it necessary both to reduce the initial curve numbers (which were obtained by the usual procedure) and to increase I_a. The final calibration for all three storms retained both these features, except for the storm of December 31, 2005, for which I_a required special consideration owning to the highly saturated antecedent soil moisture conditions associated with this event.

The final calibration represents our best effort to balance errors across each of the three events using a single set of parameter values (with the exception of I_a), and when averaged the modeled volumes are within 5% of the measured totals and the modeled peak discharges are within 3% of the measured values; when considered individually, the volumes and peak discharges are all within 20% of the measured values. Despite this relatively good agreement, the results should still be used with some caution because of the uncertainty associated with two of the three storm peaks, as was discussed in Section 2 above.

There are minor issues of timing and subsidiary peaks, particularly in the 2005 and 2008 events, which are likely associated with variations in the spatial distribution of rainfall. The scaling factors developed for this project, shown in Table 2, illustrate the general phenomenon well. Basin A receives over 40% higher rainfall than the lowest basins, according to the six-year average data, but the results are distinctly different for the two storms modeled for which data were available. In the 2008 event, Basin A shows almost no increase in rainfall, and in the 2004 event the increase is still much less than the six-year average.

A more difficult question concerns the value of initial abstraction to use, since this obviously varies by storm. It is interesting to note, however, that the storms calibrated well with a common set of I_a values, if the December 2005 event is left out of account.

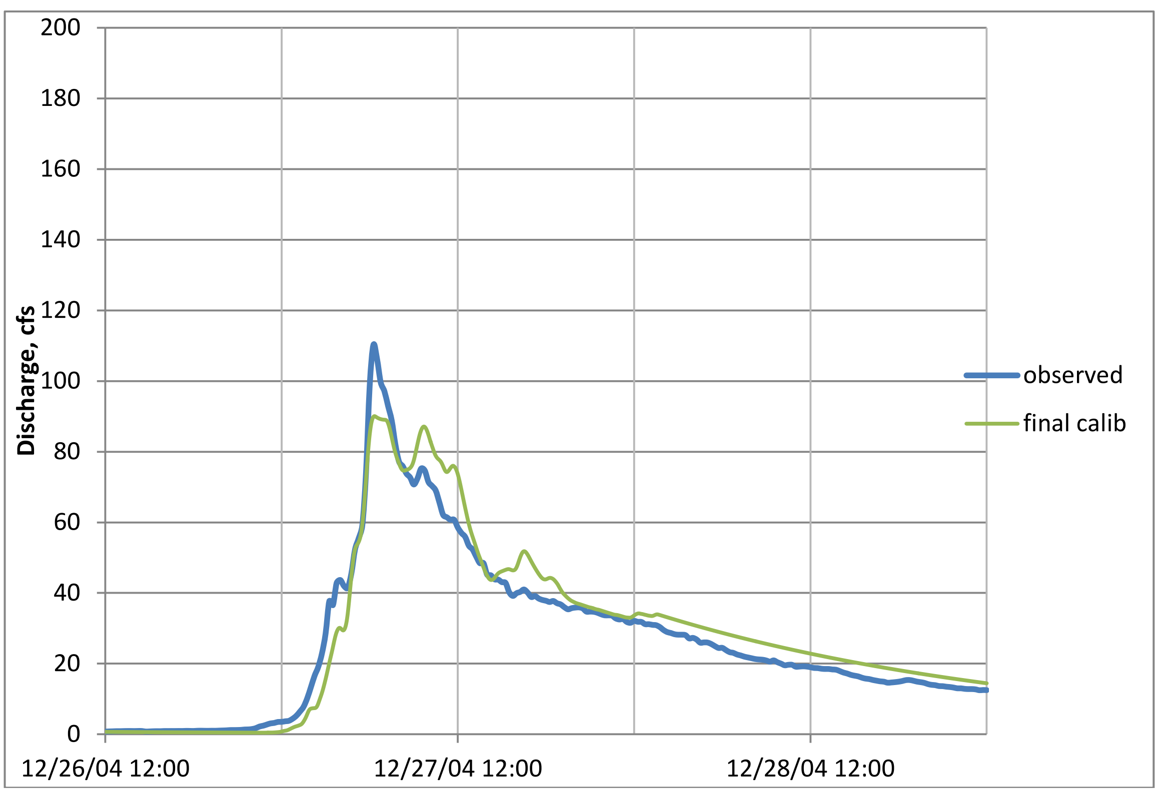

Figure C8 Calibration to storm of December 27, 2004.

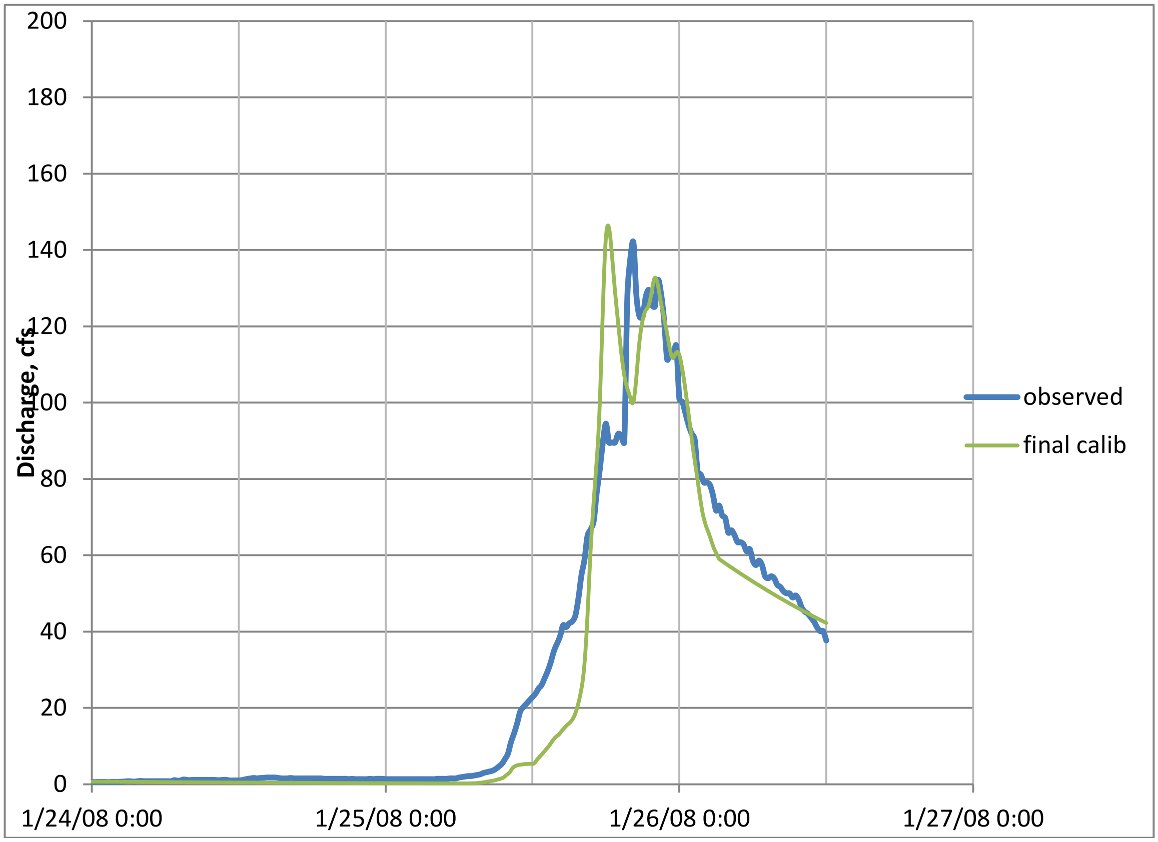

Figure C9 Calibration to storm of January 25, 2008.

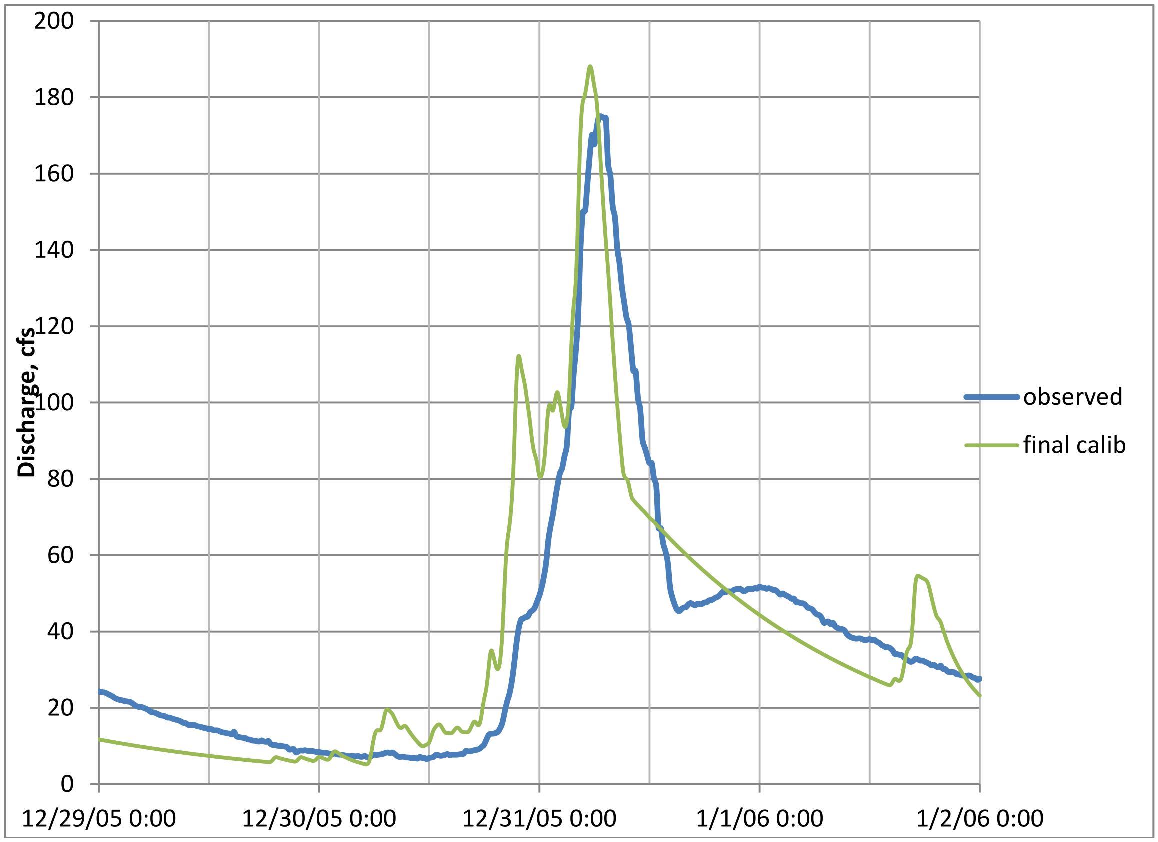

Figure C10 Calibration to storm of December 31, 2005.

Design Storms

In order to use the hydrologic model for analysis of flooding conditions in the Easkoot Creek basin, it is necessary to quantify the range of flows associated with various return intervals. These flows will then be used as input to the hydraulic model, which will in turn be used to quantify the phenomena of interest during flooding events. The desired flows are derived by running the calibrated model with input design storms that incorporate the total rainfall depth associated with the various return intervals. In that process, spatial variations over the model domain (mainly due to topography) must be accounted for, and both the duration of the design storm and the temporal distribution of the rainfall over that period must be specified and should be appropriate to the watershed. A further issue in our case arises from the fact that an important parameter for the hydrologic model, the initial abstraction I_a (i.e. antecedent soil moisture conditioned by storm events occurring prior to a hypothetical design storm), was not constant over all the calibration storms, so the appropriate set of values for use with the design storms needs to be determined.

The design rainfall depths used were derived from *NOAA Atlas 14 (Volume 6, Version 2.0)* that succeeded the previous NOAA Atlas for California in 2011, for which data are available from the convenient online *Precipitation Frequency Data Server* (http://hdsc.nws.noaa.gov/hdsc/pfds/). Atlas 14 is based on a considerably larger number of long-term stations than its predecessor Atlas 2, and its usefulness for work in Marin County was enhanced by the inclusion of data for a number of North Bay locations which were made available by Marin County staff. Data were obtained from the Precipitation Frequency Data Server for 24 hour duration storms with return intervals of 2, 10 and 100 years, which was comparable to the historical storms used to calibrate the hydrologic model.

A comparison of the range of values in the NOAA data over the Easkoot Creek watershed showed that, although the data are somewhat coarse, they do capture the spatial variation convincingly. A comparison between the Easkoot Creek gauge and the RAWS daily rainfall gauge on Middle Peak (Mt. Tamalpais) shows that Middle Peak gets about 1.5 times as much rainfall as Easkoot, according to the Atlas 14 data, which is quite consistent with the measured data for these two stations during the January 2008 event and the December 2004 event, the only two of our calibration storms for which this explicit comparison can be made. The NOAA raster data for the Easkoot Creek watershed were downloaded and re-sampled to produce data at a much finer resolution, and mean values of rainfall depth were computed for each model sub-basin.

The temporal distribution of design rainfall amounts over the 24-hour storm duration required careful consideration. In Appendix A.6, the NOAA Atlas includes a range of temporal distribution curves for several durations, including the 24-hour duration we used, for the various climate regions in California. We considered using one of these, but we found that the variation in rainfall intensity that was observed during calibration storms was much greater than that portrayed in the NOAA Atlas. There appears to be a smoothing effect in the calculation of the median distributions in the NOAA Atlas. Rainfall intensity data for all three of our calibration storms had ratios of maximum-to-mean intensity were around 5.0. For a small, steep watershed such as Easkoot Creek where time of concentration is relatively short, peak rainfall intensity is expected to have a substantial effect on peak stream discharge. Consequently, we used the ratio of maximum-to-mean rainfall intensity as a criterion in selecting an appropriate design storm.

The SCS IA 24-hour temporal distribution of rainfall intensity was found to have a ratio of maximum/mean intensity of 5.5, so this distribution was initially adopted as being more consistent with our observed storms. When evaluated in the HMS model however, we found that the shapes of the resulting hydrographs using the SCS 1A rainfall distribution were not very realistic when compared to the historical storms at gauge EK (Figure 11). In particular, the simulated hydrographs have steeper rising limbs and much more gradual recession limbs compared to the gauged hydrographs.

To investigate the relationship between rainfall intensity and runoff hydrographs further, we examined the intensities of the three historical storms in by computing intensities over a variety of durations and developing a balanced storm hyetograph by stacking the historical intensities for various durations around the highest intensity which was placed in the middle of the storm. We also followed a similar procedure using the NOAA Atlas 14 rainfall depths for various durations to develop a balanced storm hyetograph. The rainfall distributions are very similar for the two balanced storm approaches (Figure 12). Due to the fact that the historical rainfall intensities were calculated from 15-minute interval data rather than from raw tipping bucket data and the fact that it is unknown how representative these three events may be, we selected the balanced storm hyetograph that utilized the NOAA Atlas 14 rainfall depths and re-evaluated the design storms with the HMS model.

Using the balanced storm approach, the resulting hydrograph shapes are much more consistent with the historical events (Figure 13). Given this improved agreement with the historical storms and the consistency between the balanced storm hyetographs developed from the historical data and from the NOAA Atlas 14 data, we believe that the revised HMS results provide the most representative design storm hydrograph possible given the available data.

To resolve the issue of the appropriate antecedent moisture condition (represented by I_a) to be used with the design storms, we first ran each of the three design storms as a set of three cases with a *high*, *low* or *medium* assumption regarding the value of I_a. The definitions of each of these cases are shown in Table 8. The high and low values come from different calibrated storms, and the medium value is the default SCS value defined as 0.2S, where S is the maximum potential abstraction and is a function of the curve number.

In order to validate the HMS predictions and help decide which antecedent moisture condition is most appropriate for developing our design storm hydrographs, we utilized three additional flood frequency analysis techniques and compared the results to the HMS results. The first approach was to apply the USGS's regional regression equations using the NSS (National Streamflow Statistics) Program[16]. The second approach was to analyze the 8-year flow record at gauge site EK using the PKFQ Program, which is based on the methods of USGS Bulletin 17B[17] to calculate a flood frequency distribution. It is important to note that the EK gauge record is not of sufficient length to make the results highly reliable (a 10-year record is the recommended minimum for the USGS procedure; we used an 8-year record). Nevertheless we believe it is still useful for these purposes. The third method was to scale the results of the flood frequency analyses for Redwood Creek (National Park Service, 2010) and Corte Madera Creek (Stetson, 2010) on the basis of drainage area so that they predict peak flow for a basin with the drainage area of Easkoot Creek.

[16] http://water.usgs.gov/osw/programs/nss
[17] http://water.usgs.gov/software/PeakFQ

Table C8 Definitions of Abstraction/Antecedent Moisture Cases

Abstraction	*Typical I_a value (Sub-basin G)*	*Source*
High Abstraction, (Low Antecedent Moisture)	0.92	Calibrated model for December 04, January 08
Medium Abstraction	0.61	Default value for SCS method
Low Abstraction, (High Antecedent Moisture)	0.18	Calibrated model for December 05

The HMS results fall within the error bounds of the NSS and PKFQ results with the exception of the 2-yr event for the low and high antecedent moisture assumption which fall below and above the error bounds respectively (Figure 14). The HMS results are generally lower than the Redwood Creek and Corte Madera Creek results, particularly with the medium and low antecedent moisture assumptions, but they converge for the 100-year event with either high or medium antecedent moisture assumptions (Figure 14).

Perspective on the magnitude of peak flows in relation to individual storm events may be gained by reviewing the recurrence interval assigned to the December 2005 event for the studies of Corte Madera Creek and Redwood Creek. The Corte Madera Creek study assigns this event a 100-yr recurrence interval whereas the Redwood Creek study assigns it a 2.5-yr recurrence interval. Large differences in rainfall between the two sites are not unexpected given potentially complex orographic effects in the vicinity of Mt. Tamalpais. Based on mean annual rainfall distributions, the Corte Madera Creek watershed north and east of Mt. Tamalpais appears to receive additional rainfall relative to either Redwood Creek or Easkoot Creek (Figure 15). It is nevertheless surprising that such a large difference in recurrence interval is reported for the same storm event for two watersheds separated by less than 10 miles. Anecdotal accounts provided by residents in the Easkoot watershed regarding the severity of the December 2005 event suggest that it was substantially greater than a 2.5-yr event as is reported for Redwood Creek. An analysis of this storm event by USGS reported that most gauges in the North Bay region recorded peak flows in the 10 to 25 year recurrence interval range.[18]

For reference, our analyses suggest that the December 2005 event (peak discharge of about 175 cubic feet per second (cfs)) was between a 7- and 8-year recurrence interval event based on the HMS results for the medium antecedent moisture assumption scenario (Figure 14). This is reasonably consistent with USGS analyses of that storm event in northern California, as well as what was reported by residents in the Easkoot Creek watershed. We do not have an explanation for the apparently anomalous (low) flow recurrence interval for the event in Redwood Creek (2.5-yr).

We selected the HMS results using the medium antecedent moisture scenario for our design storm (middle black diamonds in Figure 14; Table 9). Although this scenario produces peak flow estimates that are substantially less than predictions for Easkoot Creek scaled from flood frequency curves for Redwood Creek and Corte Madera Creek, we believe they represent the best estimate due to the fact

[18] Parrett, Charles, and Hunrichs, R.A., 2006, Storms and flooding in California in December 2005 and January 2006—A preliminary assessment: U.S. Geological Survey Open-File Report 2006–1182, 8 p.

that they fall within the error bounds of both the NSS and PKFQ results for all recurrence intervals, and they converge for the 100-yr recurrence interval peak flow.

Given the uncertainties inherent in estimating the design storm magnitudes and hydrographs, we recommend utilizing the December 2005 event (the flood of record in available gauging records for Easkoot Creek) as the primary basis for evaluating flood control alternatives with the hydraulic model and relying on the design storms as a secondary means of evaluation. We believe that by considering both the documented 2005 event (estimated 8-year recurrence interval) and a simulated 100-year event, we are able to provide a balanced consideration of flood potential. This approach considers both a directly measured rainfall and runoff event in Easkoot Creek that produced substantial flooding in Stinson Beach and an empirically-derived hypothetical rainstorm to simulate a 100-year recurrence interval flood event.

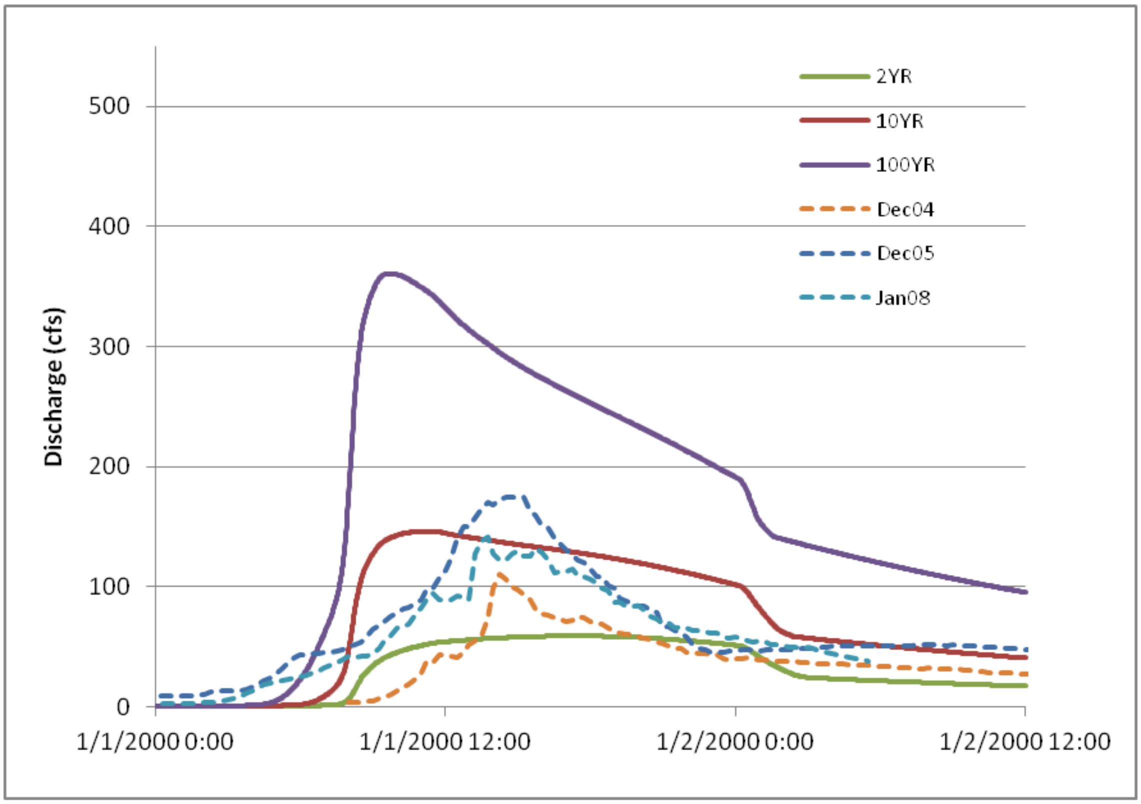

Figure C11 Comparison of gauged historical storms and preliminary hydrographs.

(Produced from the HMS model using an SCS 1A rainfall distribution.)

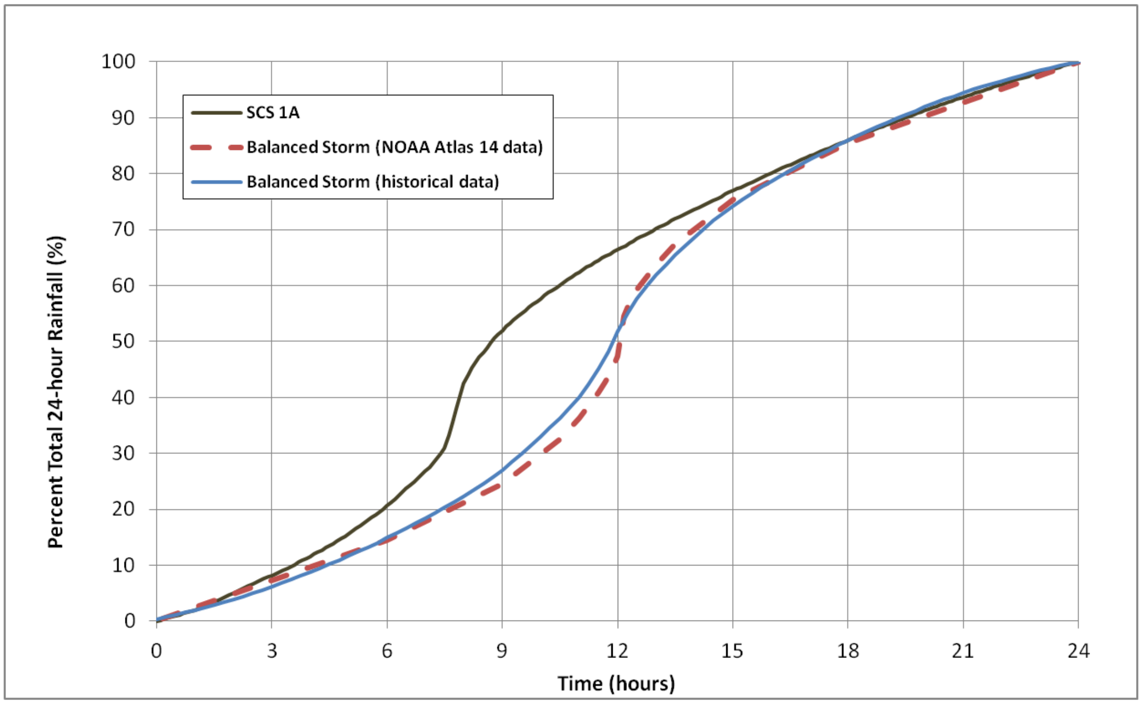

Figure C12 Comparison of rainfall distributions using SCS 1A.

(A balanced storm approach derived from historical rainfall data, and a balanced storm approach derived from NOAA Atlas 14 depths for various durations.)

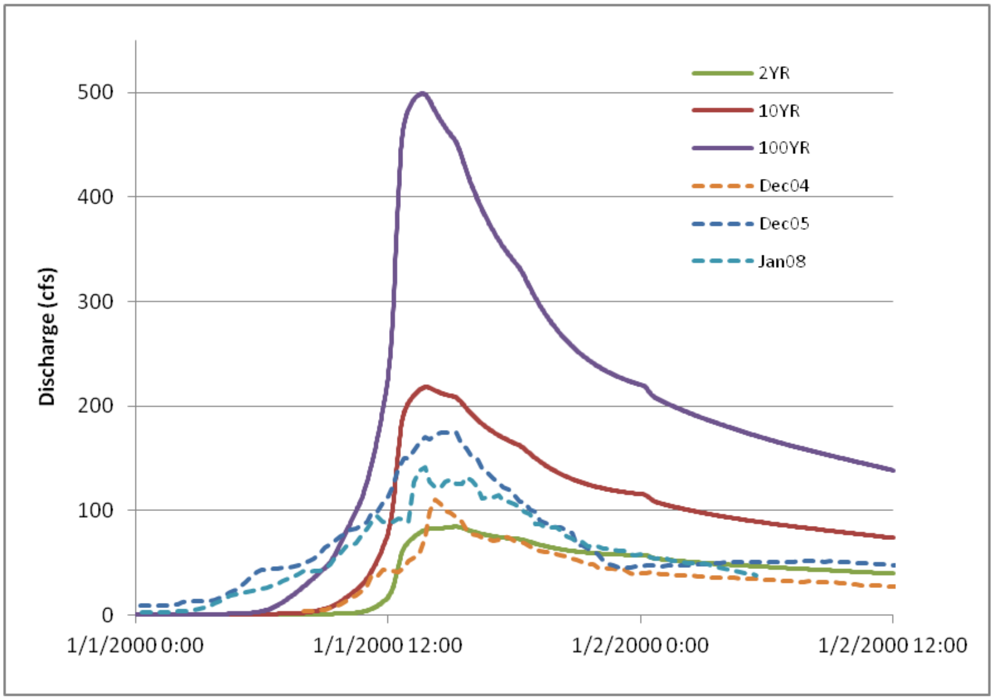

Figure C13 Comparison of gauged historical storms and final hydrographs.

(Produced from the HMS model using a balanced storm approach based on NOAA Atlas 14 depths for various durations.)

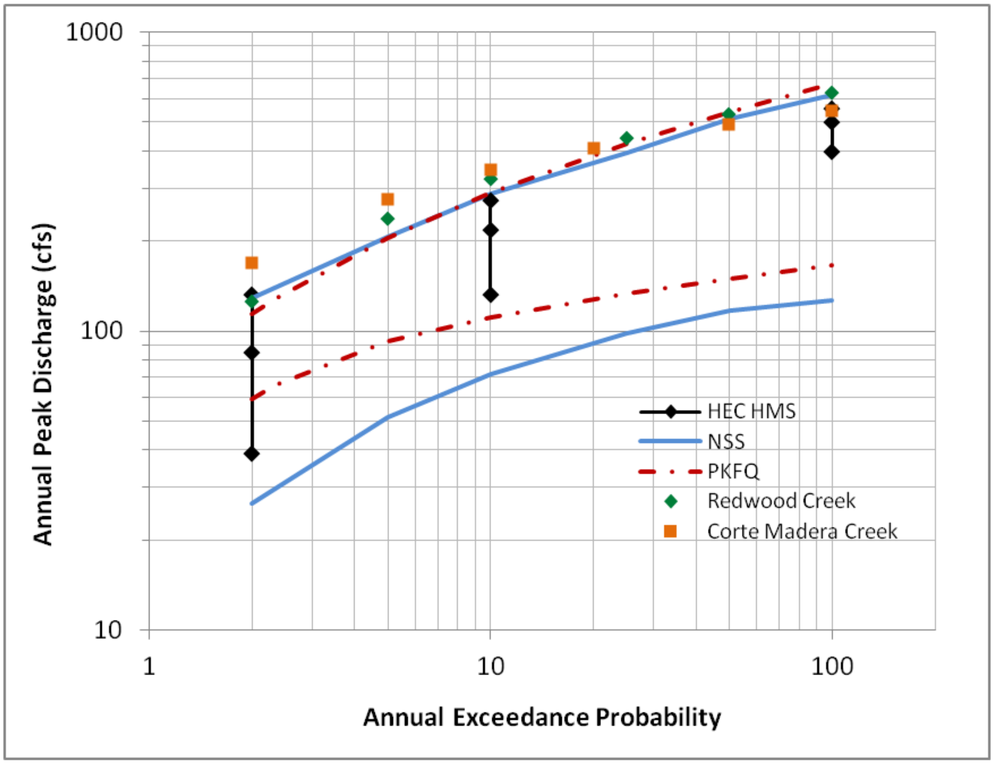

Figure C14 Flood frequency analysis for Easkoot Creek.

(Comparing the HMS model results for low, medium, and high I_a assumptions (black diamonds), with results obtained from the NSS and PKFQ analyses (shown as upper and lower bound estimates) and those obtained from studies of nearby watersheds adjusted for Easkoot Creek's drainage area.)

Table C9 Design storm peak flow magnitudes for Easkoot Creek at entrance bridge to NPS Stinson Beach parking lots.

Recurrence Interval (yrs)	*Annual Probability of Occurrence*	*Peak Discharge (cubic feet per second (cfs))*
2	0.5	85
10	0.1	218
100	0.01	499

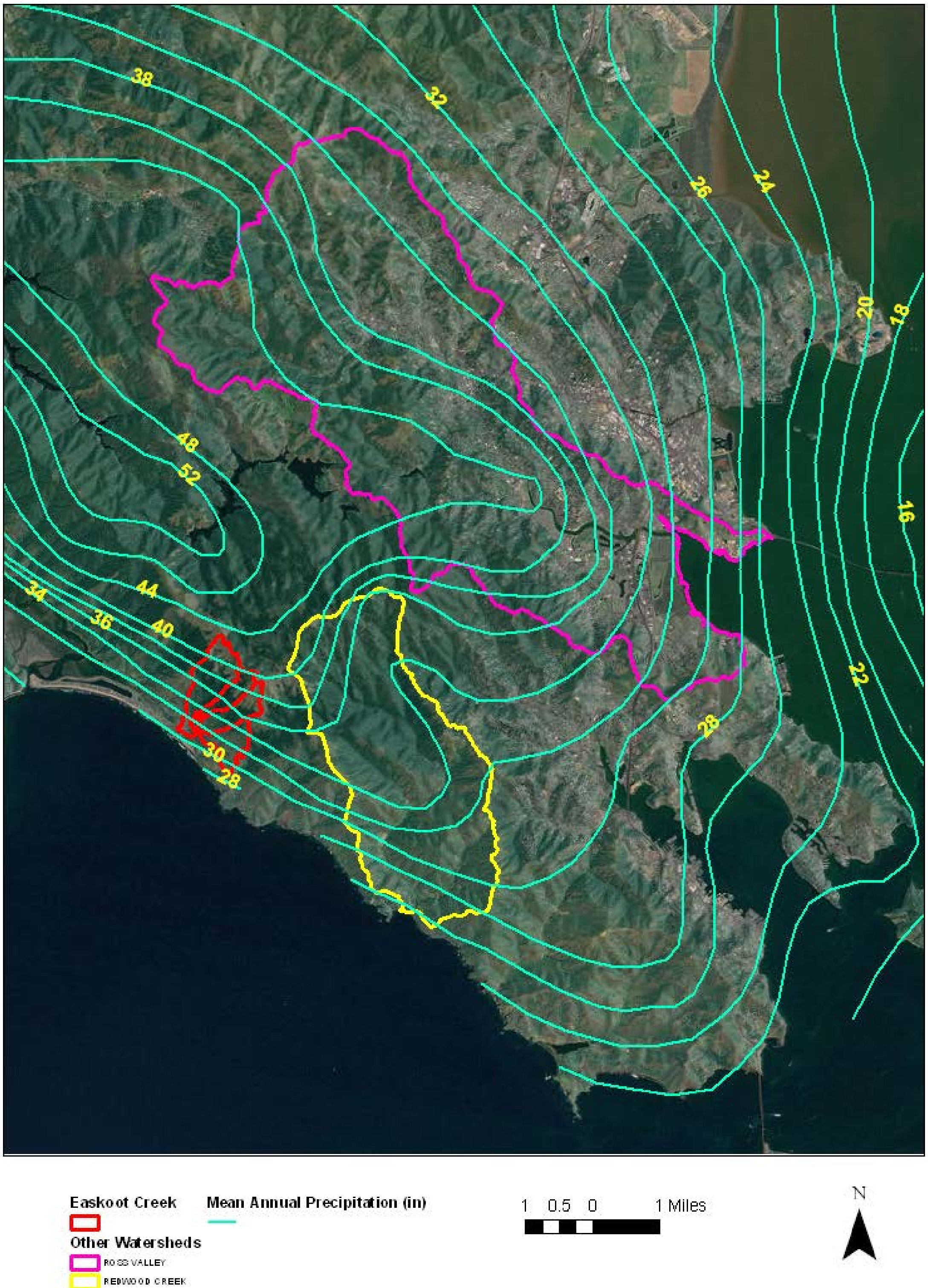

Figure C15 Isohyetal map of southern Marin County.

Note that Ross Valley watershed (shown in pink) is described as Corte Madera Creek watershed in the text.

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Easkoot Creek Hydrology & Hydraulics Study

Appendix D. Hydraulic Model and Flood Hazard Evaluation

Prepared for

Marin County Flood Control and Water Conservation District

Prepared by

O'Connor Environmental, Inc.

March 20, 2012

Introduction

This document describes the development of a set of coupled 1- and 2-dimensional hydraulic models of lower Easkoot Creek and the surrounding floodplains. The hydraulic models were developed to characterize existing flood hazard conditions for a series of historical and design storms and provide a framework for evaluating mitigation alternatives intended to reduce flooding and the impacts of sea-level rise. Additionally, the models provided detailed water depth, velocity, and shear-stress information to support the sediment transport and fisheries habitat assessments described in separate memorandums. The analysis covers the full area subject to flooding from Easkoot Creek downstream of the Highway 1 Bridge and includes the effects of tidal conditions on riverine flooding but does not evaluate the effects of storm surge and wave action and is not intended to provide an analysis of flooding from coastal hazards.

The remainder of the document gives an overview of the methods and data used in the construction and calibration of the model. Specifically, the memorandum includes:

- A brief description of the numerical model codes
- Summaries of the data and assumptions that went into the model setup
- A discussion of the model calibration
- Detailed information regarding existing condition flood extents and infrastructure impacts for a variety of historical and design storms
- A discussion of model uncertainty and summary of the key findings of the modeling effort

The sediment transport component of this task is described in a separate technical memorandum "Sediment Transport Evaluation".

Model Development

Model Description

The MIKE FLOOD model allows for the dynamic coupling of the 1-dimensional hydraulic model (MIKE 11) and the 2-dimensional hydraulic model (MIKE 21) developed by DHI, Inc. (DHI, 2011). Both models are designed to simulate free-surface flows and water-level variations in rivers, lakes, estuaries, and oceans using a finite-difference approximation to solve the equations of conservation of mass and momentum (Saint-Venant equations) for unsteady flows. Additionally, MIKE 11 includes the ability to simulate the hydraulic effects of bridges and other structures using an energy equation. The models are accepted by FEMA for use in the National Flood Insurance Program and they have been applied in numerous studies around the world to assess a wide variety of problems including flood hazard mitigation, restoration design analysis, sediment transport evaluations, and coastal circulation assessments.

Basic data requirements for the models include: channel and floodplain topographic data, boundary conditions, resistance distributions, and initial conditions. Calibration is generally performed by adjusting resistance or roughness values within a range of reasonable limits in order to provide the best match between simulated inundation patterns, discharges, and/or water levels and measured values of these parameters.

Model Domains

Two primary models were developed for Easkoot Creek: a Tidal Model and an Upland Model. The Tidal Model extends from a location in Bolinas Lagoon ~0.5 miles downstream of the western end of Calle del Arroyo upstream to where Calle del Arroyo crosses the creek (Figure 1). The Upland Model extends from a point ~800-ft downstream of the Calle del Arroyo crossing upstream to the Highway 1 crossing (Figure 1). Both models extend laterally onto the floodplain a sufficient distance to capture the full inundation extent expected during the largest event considered in this analysis (the 100-yr flood). Simulating the full area covered by both models in a high level of detail in a single model was not feasible owning to the computational limitations of the model codes. The Tidal Model was used to develop downstream water-level boundary conditions for the Upland Model reflecting the combined influences of riverine discharge and tidal forcing that characterize this location. Results from both models were combined to provide a comprehensive evaluation of flood hazard conditions for the entire area below Highway 1 that is potentially impacted by flooding from Easkoot Creek. It is important to note that while the models do simulate the effects of tidal conditions on riverine flooding, the models are intended to evaluate riverine flooding only and do not include the effects of storm surge or wave action that are important for evaluating flooding from the ocean caused by storm surge and wave run-up. These coastal flood hazards are being investigate in a separate study for the District and will be evaluated after the completion of this study. It will be possible to re-evaluate portions of this study in light of new information from the coastal flood study.

The Tidal Model is a stand-alone 2-dimenional MIKE 21 model with a 2-meter rectangular grid spacing. The model includes 189,000 active grid cells and covers a total area of 0.29 square miles. The Upland Model simulates the active channel of Easkoot Creek including twelve bridge crossings using a 1-dimensional model (MIKE 11) and a series of channel cross sections; it simulates the surrounding floodplains using a 2-dimensional MIKE 21 model with a 1-meter rectangular grid spacing (Figure 1). The MIKE 11 model covers ~3,300 linear feet of Easkoot Creek and included 66 cross sections and the MIKE 21 model includes 317,000 active grid cells and covers a total area of 0.12 square miles. The two component models in the Upland Model are dynamically linked via the MIKE FLOOD interface using a weir formula to exchange flows exiting or entering the active channel.

Topography

The topography of the Tidal Model and the 2-dimensional component of the Upland Model was developed from a 0.5-meter resolution Bare Earth Digital Elevation Model (DEM) developed from LiDAR collected as part of the Golden Gate LiDAR Project (Hines, 2011). In order to reduce artifacts in the data resulting from misclassified ground returns and improve the accuracy of the resulting DEM in heavily-vegetated areas, the LiDAR returns were re-classified and a new DEM was generated for the project area by Bill Kruse of Kruse Imaging. Based on a detailed evaluation of the LiDAR elevations in relation to surveyed point elevations (described in a separate memorandum for the project, *LiDAR Evaluation*), a constant value of 0.87-ft was added to the DEM elevations to reduce the vertical bias inherent in the dataset.

A separate DEM was interpolated from survey data for a small area north of the creek between Calle del Pinos and the National Park Service Entrance Road where survey data density was sufficient to make this possible. A mosaic procedure was used to combine this local DEM with the overall project DEM (Figure

2). The resulting composite DEM was re-sampled to 2-meter and 1-meter resolutions for use in the Tidal and Upland models respectively. A shape file of buildings footprints was used to block buildings out of the model topography in order to more realistically simulate the effects of these obstructions to flow using the Bare Earth DEM (Figure 2).

A topographic survey performed using an Electronic Total Station by OEI in 2011 provided the basis for developing 63 cross sections of the active channel (Figure 2). An additional 3 cross sections were extracted from the composite DEM in order to extend the 1-dimensional component of the Upland Model farther downstream. The topographic survey also included the details of bridge abutments and a minimum of four points on the bottom of each bridge deck and four points on the top of each bridge deck. These data provided the basis for simulating the hydraulic effects of the bridges in the MIKE 11 model using an energy equation formulation for flow through the bridge openings including the effects of bridge deck submergence and overtopping. A separate *LiDAR Evaluation* memorandum describes the survey procedures and data in greater detail.

Boundary Conditions

Riverine Boundaries

Two recent historical storm events were evaluated with the models; one event on 12/31/2005 with a peak discharge at stream gauge EK (see Figure 2) of 175 cubic feet per second (cfs) and a second event on 1/25/2008 with a peak discharge of 102 cubic feet per second (cfs). In addition to these two historical events, three design storm events with recurrence intervals of 2-, 10- and 100-yrs were evaluated with the models. Selection of these events, a discussion of peak flow estimation at the gauge site, and design storm development is described in greater detail in a technical memorandum (*Sub-task 3a Hydrologic Analysis)*

A total of six inflow boundaries are included in the 1-dimensional component of the upland model. Boundary #1 represents the majority of the inflow to the model and is a point-source boundary located at the upstream edge of the model at the Highway 1 crossing. The hydrograph for this boundary was developed by subtracting the results from the HEC-HMS model for Basin G (contributing area between Highway 1 and gauge EK) from the EK gauge record. The remaining boundaries #2 through #6 were all taken directly from the HEC-HMS model results for basins G through K respectively (see Figure 6, *Sub-task 3a Hydrologic Analysis*). Boundary #2 is a distributed-source boundary located between the Highway 1 crossing and gauge EK. Boundary #3 is a point-source boundary representing the contribution from the storm drain system discharging to the creek adjacent to the western National Recreation Area parking lot. Boundary #4 is a point-source boundary representing the tributary stream that discharges to the creek just upstream of Calle del Pinos. Boundary #5 is a point-source boundary representing the small tributary that discharges to the creek just downstream of the Calle del Arroyo crossing, and boundary #6 is a distributed-source boundary representing the residual watershed area on the south side of the creek between Calle del Pinos and the downstream edge of the model and on the north side of the creek between Calle del Arroyo and the downstream edge of the model. Figures 3 through 7 show the inflow boundary hydrographs for the December 2005, January 2008, 2-yr, 10-yr, and 100-yr events respectively.

Tidal Boundaries

Tidal data is available from NOAA for a tide gauge located near the mouth of Bolinas Lagoon (station #9414958) from July 2009 to present and at Point Reyes (station #9415020) from January 1975 to present. Because of the limited period of record at the Bolinas Lagoon station, it was necessary to perform a correlation between this station and the Point Reyes station. NOAA has not published the Bolinas Lagoon data in a universal vertical datum such as NAVD 88 because insufficient benchmark data is available for this purpose (Jena Kent, NOAA, personal communication). In order to overcome this limitation of the data, NOAA's VERTCON program was used to convert the Bolinas Lagoon record from the local Mean Lower Low Water (MLLW) datum to the universal NAVD 88 vertical datum. A comparison between daily high and low tides between the two stations indicated that the mean offset between the two stations was less than 0.1-ft. Higher high tides were on average 0.6-ft lower at Bolinas Lagoon than at Point Reyes and lower low tides were on average 0.8-ft higher at Bolinas Lagoon than at Point Reyes. A comparison of timing indicated that tides at Bolinas Lagoon are delayed by approximately 1-hr relative to Point Reyes.

Owing to the uncertainty associated with converting the Bolinas Lagoon record to NAVD88 due to insufficient benchmark data, the limited period of record for which to perform a correlation, and the relatively small computed offsets from the Point Reyes record, we decided to simply use the Point Reyes record directly for developing hydraulic model boundary conditions. This decision can be considered a 'conservative' assumption in that the correlation suggests high tides are generally lower at Bolinas Lagoon by 0.6-ft on average, thus using the Point Reyes record would tend to overstate the tide elevation by a small amount.

In order to investigate the potential for tidal amplification between the mouth of Bolinas Lagoon and the location of our model boundary, we utilized preliminary outputs from the USGS's ongoing modeling study of sea-level rise impacts which includes Bolinas Lagoon (Li Erickson, USGS, personal communication). The outputs from the USGS model showed virtually no amplification (<0.1-ft) between the mouth of the lagoon and the location of our model boundary. This supported our decision to use the Point Reyes data directly in our models adjusting the data 1-hr forward in time to account for the timing offset but without making any vertical adjustments. This procedure was followed for the December 2005 and January 2008 events (Figures 3 and 4). For the design storms, a tide record with a tidal range matching the Mean Tidal Range (MTR) of 3.1-ft at the Bolinas Lagoon station was selected from the historical record and scaled such that the maximum tide matched the Mean Higher High Water (MHHW) elevation of 5.8-ft at Point Reyes. The time series was shifted in time such that MHHW occurred coincident with the maximum discharge from the creek (Figures 5 through 7). This is a generally accepted approach for developing tidal boundaries for riverine flood studies in that it assumes a high but not extreme tidal condition.

In order to evaluate the effects of sea level rise due to global climate change we also analyzed the December 2005 flood with tidal boundary conditions of Mean Higher High Water (MHHW) and MHHW plus 18.2 inches of sea level rise. This is the sea level rise value recommended for use in Marin County riverine flood studies by the August 2012 Technical Memorandum prepared by Marin County staff entitled *Recommended Sea Level Rise Modeling Methodology and Values to be used for Riverine and CIP Flood Studies*. It represents a 2050 sea level rise estimate and is based on a statistical analysis of the range of predicted values given in the 2012 National Research Council's report *Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future*.

Model Exchanges
The two models were evaluated in an iterative fashion in order to balance the results across the over-lapping portions of the model domains (Figure 1). The Upland Model provided upstream inflow boundaries for the Tidal Model and the Tidal Model provided downstream water-level boundaries for the Upland Model that reflected the combined riverine and tidal influences at this location. The following procedure was followed for each flood event:

1. Run the Upland Model using an approximate downstream water-level boundary developed by taking the higher of the water-level calculated using Manning's equation at the downstream cross section of the model or the tidal time series for each time step.

2. Run the Tidal Model using channel and floodplain inflows extracted from the Upland Model and the MHHW tidal time series.

3. Re-run the Upland Model using a water-level boundary extracted from the Tidal Model.

4. Re-run the Tidal Model using updated channel and floodplain inflows extracted from the second run of the Upland Model.

5. Re-run the Upland Model using an updated water-level boundary extracted from the second run of the Tidal Model if different from the original model by more than 0.1-ft.

Resistance

Initial estimates of left bank, channel bottom, and right bank channel resistance (Manning's n) were performed in the field on a reach by reach basis and assigned to the appropriate portions of each model cross section in the 1-dimensional component of the Upland Model. Bank roughness values varied from 0.09 in heavily vegetated reaches to 0.02 in concrete lined reaches, while channel bottom roughness values ranged from 0.06 to 0.03 depending primarily on the channel substrate and vegetation conditions (Figure 8). Zones of similar roughness characteristics on the floodplain were mapped in the field and initial estimates of the associated roughness value were performed. This map was supplemented with additional aerial photography-based mapping in order to develop a complete roughness map for the Tidal Model and for the 2-dimensional component of the Upland Model. Floodplain roughness values ranged from 0.20 in areas of dense vegetation to 0.02 in paved areas (Figure 9).

Model Calibration

The primary quantitative calibration data available for the model are the measured water-levels at gauge EK for the December 2005 and January 2008 events. The simulated water-levels agreed quite closely with the measured water-levels for the both events (Figures 10 and 11). During the December 2005 event, water levels were slightly over-predicted on the rising limb and under-predicted on the recession limb, however at all times during the 22-hr simulation, the simulated water levels were within 0.4-ft of the observed levels. The simulated peak water level agreed with the observed water level to within 0.1-ft, and the overall Mean Error (ME) and Root Mean Square Error (RMSE) were -0.1-ft and 0.3-ft respectively. During the January 2008 event, water levels were slightly under-predicted throughout the event. The simulated peak water level agreed with the observed water level to within 0.5-ft and the overall ME and RMSE were -0.5-ft and 0.5-ft respectively.

There are several significant sources of uncertainty to bear in mind when interpreting the model calibration results. Repeated longitudinal profiles of the channel indicate that significant sediment deposition occurred during the December 2005 event resulting in bed aggradation of 2- to 3-ft (see *Sediment Transport Evaluation* for more details). The model is based on recent topographic information (LiDAR and ground surveys from the past 2 years) and it is unknown how representative the current topography at the gauge site is of conditions at the time of the peak December 2005 discharge. Additionally, the discharge record at the gauge site had to be estimated due to the fact that the measured water levels exceeded the rating curve for the gauge (see discussion in *Hydrologic Analysis* report). Lastly, though much effort was given to ensuring the quality of the LiDAR data which provides the basis for the model's representation of the floodplain topography, the LiDAR evaluation indicated that the accuracy of the LiDAR may be substantially lacking in some areas (see *Topographic Data Evaluation* memorandum). Given these various sources of uncertainty, the fact that the initial roughness values used in the model were based on detailed field observations, and the relatively good agreement between simulated and observed water levels using the initial roughness values, we decided that further adjustment of roughness values in order to achieve a closer match between the simulated and observed water levels was unwarranted.

In addition to the comparison between model results and measured water level data, a qualitative verification of the simulated inundation patterns and extent was possible owing to the high level of familiarity of local residents with the December 2005 flood. Detailed flood maps were presented at a Technical Working Group meeting in September of 2012 and were reviewed by long-time ranger at the Stinson Beach National Recreation Area, Pat Norton. There was general agreement that the flood maps were in close agreement with observations about historical flooding patterns on the creek. A final means of model verification is to compare the homes where flood insurance claims were filed following the event with those homes shown as flooded in the model.

Existing Conditions Results

January 2008

Results for the January 2008 flood indicate that flows are contained within the active channel of Easkoot Creek between Highway 1 and a point ~100-ft downstream of the Park Entrance Rd. bridge (Figure 12). Overbank flows occur at numerous locations downstream of this point and in many cases appear to be associated with bridge crossings. Some of the more prominent locations of overbank flow include the reaches immediately upstream of the footbridge above Calle del Pinos and the Calle del Onda bridge, as well as a location just upstream of Calle del Resaca. Complex split flow paths develop on the floodplain throughout the Calles as a result of these breakout flows. Floodplain inundation depths in the Calles reach as high as 2 to 3-ft locally but are more typically on the order of 0.25 to 1.25-ft (Figure 13A). Farther downstream, elevated water levels in the estuary result in floodplain inundation in the vicinity of Alameda Patio and in the area between Rafael Patio and Van Praag (Figures 13A and 13B).

Water levels exceed the bottom of the bridge decks at four of the twelve bridges including Calle del Pinos, Calle del Onda, the House bridge, and the Footbridge below Calle del Arroyo. Flooded streets include portions of all of the Calles except Calle del Occidente. Calle del Arroyo is flooded from the intersection of Highway 1 to Calle del Embarcadero and from Rafael Patio to Van Praag. Highway 1 is flooded between Calle del Pinos and Calle del Pradero, in the vicinity of the Calle del Arroyo intersection, and in the reach adjacent to Calle del Embarcadero (Figures 13A and 13B).

December 2005

Results for the December 2005 flood indicate that flows are contained within the active channel of Easkoot Creek between Highway 1 and the upstream side of the Parkside Cafe. Overbank flows occur along a reach stretching from the Parkside Cafe to just downstream of the park entrance pedestrian bridge (Figures 14A and 14B). Overbank flow on the right bank inundates an area near the intersection of Calle del Mar and Arenal Ave. including the Parkside Cafe. Overbank flow on the left bank results in the development of a distributed floodplain flow path carrying as much as 8 cubic feet per second (cfs) across the National Recreation Area property and flooding the western parking lot. Water levels within the western parking lot reach an approximate equilibrium with water levels in the adjacent reach of the creek such that only limited flow exchange occurs along this reach.

Flooding patterns in the Calles are similar to those described above for the 2008 event though the inundation extent and depths are larger. Depths in the Calles reach as high as 3-ft locally but are more typically on the order of 0.5 to 1.5-ft (Figure 14A). Water levels exceed the bottom of the bridge decks at seven of the twelve bridges in the study area including the Park Footbridge, the footbridge above Calle del Pinos, Calle del Pinos, the Gym footbridge, Calle del Onda, the House bridge, and the footbridge below Calle del Arroyo. Flooded streets include the intersection of Calle del Mar and Arenal Ave. and portions of all of the Calles. Calle del Arroyo is flooded throughout its entire length, and Highway 1 is flooded between Calle del Pinos and Calle del Pradero, between Calle del Arroyo and a point adjacent to Calle del Occidente, and farther downstream adjacent to the lower patios (Figures 14A and 14B).

2-yr Design Storm

Results for the 2-yr design storm indicate that flows are for the most part contained within the active channel of Easkoot Creek. Minor overbank flows occur upstream of Calle del Pinos and Calle del Resaca creating some floodplain inundation locally (Figures 15). Water levels exceed the bottom of the bridge decks at four of the twelve bridges in the study area including Calle del Pinos, Calle del Onda, the House bridge, and the footbridge below Calle del Arroyo. The only street flooding occurs along Calle del Arroyo between Calle del Resaca and Calle del Embarcadero (Figures 16A and 16B).

10-yr Design Storm

Inundation patterns for the 10-yr design storm are similar to those described above for the December 2005 flood with some exceptions. Additional overbank flow in the reach between the Parkside Café and the Park Footbridge results in larger discharges across the National Recreation Area property flooding the western parking lot and this overbank flow also begins to move eastward flooding portions of the eastern parking lot and adjacent picnic area (Figure 17A and 17B). Water levels in the western parking lot become elevated enough to overtop the berm separating the parking lot from the houses east of Calle del Pinos and overflow through the adjacent gap in the sand dunes provided for beach access. Maximum discharge breaching the dunes and flowing to the Pacific Ocean is 21 cubic feet per second (cfs). Flooding at the intersection of Calle del Mar and Arenal Ave. is extensive enough to result in development of a floodplain flow path that flows adjacent to Highway 1 and eventually merges with floodplain flow downstream of the Park Entrance Bridge (Figure 17A).

Water levels exceed the decks of the same seven bridges as during the December 2005 event and floodplain inundation and street flooding is generally similar though slightly more pronounced than the December 2005 street flooding above about Calle del Occidente. Below this point, however, the extent of floodplain inundation is much less than during the December 2005 flood due to the use of the Mean Higher High Water tidal boundary condition which is ~2.6-ft lower than the extremely high tide that occurred following the December 2005 event which was an approximately 25-yr tide (see Figures 3 and 6).

100-yr Design Storm

The patterns of inundation for the 100-yr design storm are similar to those described above for the 10-yr storm though the extent and depths of floodplain flows are significantly higher. Overbank flows occur at Arenal Ave. and merge with the floodplain flows resulting from overtopping in the vicinity of the Park Footbridge (Figures 15 and 18A). The floodplain flow moving across the National Recreation Area property is extensive enough to result in the development of new breach locations at Calle del Pinos and in the vicinity of the spring located in the eastern picnic area which discharge as much as 9 cubic feet per second (cfs) and 4 cubic feet per second (cfs) to the Pacific Ocean respectively (Figure 18A). The breach location that developed during the 10-yr flood becomes much more active and discharges as much as 141 cubic feet per second (cfs) to the Pacific Ocean.

Highway 1 is flooded along much of its length between a point downstream of Calle del Mar and a point adjacent to Calle del Occidente. Below about Alameda Patio, inundation results are very similar to those described above for the 10-yr event due to the fact that most floodplain flow returns to the channel and estuary upstream of this location (Figures 18A and 18B). Water levels exceed the bottoms of the bridge decks at all of the bridges with the exception of the Park Entrance Bridge and water levels overtop the bridge decks at the Park Footbridge, Calle del Pinos, the Gym Footbridge, Calle del Sierra, Calle del Onda, and the footbridge below Calle del Arroyo.

December 2005 Storm With 2050 Sea Level Rise

A comparison between the inundation extents for the December 2005 flood using the extreme tide elevation that coincided with the flood event (elevation 8.4-ft), MHHW (5.8-ft), and MHHW plus 18.2 inches of sea level rise (7.3-ft) provides some insight into the effects of sea level rise and tidally-induced flooding along lower Easkoot Creek. This comparison reveals that the effects of changes in tidal elevation diminish fairly rapidly as you move up the estuary and into lower Easkoot Creek (Figures 14A, 19A and 20A); upstream of the Calle del Arroyo crossing, peak water levels are virtually unchanged (<0.1-ft) over the range of tidal conditions that were considered.

Moving from the MMHW tide to MHHW plus sea level rise, results in modest increases in inundation extent between Calle del Embarcadero and Alameda Patio, and farther downstream a significant portion of Calle del Arroyo that was dry under a MHHW tidal condition becomes inundated with the increase in tidal level due to sea level rise (Figures 19B and 20B). Comparing the MHHW results with the results for the extreme tide that coincided with the historical flood event reveals large increases in inundation extent below the Calle del Arroyo crossing. Only the upstream 600-ft of Calle del Arroyo is flooded with the MHHW tidal elevation of 5.8-ft, but the entire length of the roadway and many buildings south of the roadway become flooded under the historical 8.4-ft tide (Figure 14B and 19B). It is important to note that much of the increase in inundation extent is attributable to overtopping of Calle del Arroyo due to the elevated tidal levels directly rather than from the effects of the tides on flooding originating from Easkoot Creek.

Building Impacts

Recent finished floor elevation (FFE) survey data was available for 163 buildings within the study area (Figures 21A and 21B) allowing us to tabulate the number of buildings that were flooded during each event based on a comparison with the simulated water surface elevations. For those buildings lacking FFE data, it was assumed that buildings with adjacent water depths in excess of 0.5-ft were flooded and that those buildings with adjacent water depths below 0.5-ft were not flooded. The analysis of building impacts indicated that approximately 16 buildings were flooded during the January 2008 event and 45 buildings were flooded during the December 2005 event. For the 2-yr, 10-yr, and 100-yr events, the number of flooded buildings was 2, 34, and 59 respectively (Table 1). Note that this includes all buildings included in the county building footprint shape file including garages and other auxiliary structures.

Above Calle del Arroyo, the number of flooded buildings during the December 2005 flood does not change under the range of tidal elevations that were considered (5.8 to 8.4-ft). Below Calle del Arroyo, moving from the existing condition MHHW elevation of 5.8-ft to the seal level rise estimate of MHHW (7.3-ft) adds an additional 3 buildings to the floodplain (Table 1). Moving higher still to the historical December 2005 tide elevation of 8.4-ft adds 17 more buildings to the floodplain (Table 1). Most of these increases are attributable to overtopping of Calle del Arroyo due to the elevated tidal levels directly rather than from the effects of the tides on flooding originating from Easkoot Creek.

Table D1 Summary of the number of flooded buildings for each simulated event.

Event	# of Flooded Buildings
2-yr	**2**
10-yr	**34**
100-yr	**59**
Jan. 2008	**16**
Dec. 2005	**45**
Dec. 2005 MHHW	**24**
Dec. 2005 MHHW + 2050 Sea Level Rise	**27**

Discussion and Conclusions

Results of the existing conditions modeling indicate that the current capacity of Easkoot Creek is slightly less than the 2-yr event or about 80 cubic feet per second (cfs) over the reach extending from a point ~100-ft above Calle del Pinos to the footbridge below Calle del Arroyo. Current capacity in the vicinity of the Parkside Café is approximately 140 cubic feet per second (cfs) above which flow begins to inundate the National Recreation Area property and the intersection of Calle del Mar and Arenal Ave. The western parking lot floods at flows in excess of approximately 160 cubic feet per second (cfs) and water begins to breach the dunes adjacent to the parking lot and discharge to the Pacific Ocean when flows exceed approximately 200 cubic feet per second (cfs) (slightly less than the 10-yr event). Above 200 cubic feet per second (cfs) floodplain flow originating from the vicinity of the Parkside Café begins to split, with a component flowing towards the picnic area east of the eastern parking lot (towards historic Poison Lake) and above 350 cubic feet per second (cfs) flow in this vicinity breaches the dunes and discharges to the Pacific Ocean. Capacity in the vicinity of the Arenal Ave. bridge is approximately 400 cubic feet per second (cfs) and capacity above Arenal Ave. and in the vicinity of the Park Entrance Bridge exceeds the 100-yr flow of approximately 500 cubic feet per second (cfs).

Prominent locations of breakout flows appear to be associated with bridge crossings and include locations just upstream of the Park Footbridge, Calle del Pinos, Calle del Onda, and Calle del Resaca at lower flows and Arenal Ave. at higher flows. Floodplain flows return to the channel in several locations as well including locations upstream and downstream of Calle del Pradero and the reach between Calle del Ribera and Alameda Patio. Most floodplain flow originating upstream has returned to the channel by Alameda Patio and flooding below this point is highly dependent on tidal conditions. No flooding impacts occur below Alameda Patio under Mean Higher High water (MHHW) tidal conditions (5.8-ft NAVD 88 maximum tide) even during flows as large as the 100-yr flow. When tidal levels are higher such as during the January 2008 and December 2005 events, water levels exceed the elevation of Calle del Arroyo in the lower reaches of the creek and estuary producing flooding in the reach between Rafael Patio and Van Praag during the January 2008 event (6.8-ft NAVD 88 maximum tide) and throughout the entire length of Calle del Arroyo during the December 2005 event (8.4-ft NAVD 88 maximum tide).

A comparison of the number of flooded buildings in the lower portion of the system highlights the effects of the tidal forcing and reveals that 24 buildings below Calle del Resaca flooded during the December 2005 event (~7.5-yr event with a 25-yr tide) and only 8 buildings flooded during the 100-yr event with a MHHW tide (Table 1). The majority of the highly flood-prone buildings occur within the Calles with the exception of the Parkside Café. Flooding within the Patios only occurs during tidal conditions above MHHW and flooding of additional buildings above the Calles only occurs during large floods like the 100-yr event. Street flooding is most prominent in the Calles as well. Other flood-prone streets include the intersection of Calle del Mar and Arenal Ave. and several reaches of Highway 1. The upper portions of Calle del Arroyo are highly flood-prone and during elevated tidal conditions the entire length of Calle del Arroyo is subject to flooding.

Comparing the results for the December 2005 flood over the range of tidal conditions considered in this analysis provides some insight into the potential effects of sea level rise due to climate change. These results indicate the that the effects of changes in tidal elevation diminish fairly rapidly as you move up the estuary and into lower Easkoot Creek, and are negligible above Calle del Arroyo. Flood extents increase dramatically downstream of this point as you move from a MHHW tide elevation of 5.8-ft to MHHW plus 2050 sea level rise (7.3-ft) to the extreme tide elevation of 8.4-ft). Most of these increases are attributable to overtopping of the estuary due to tide elevations exceeding the crest of the roadway rather than to tidally-induced increases in riverine flooding. Thus sea level rise does not appear to pose a major risk in terms of increasing flooding originating from Easkoot Creek but is does appear to have a substantial effect on tidal flooding. A coastal flood hazard analysis should be conducted to fully evaluate the expected impacts of sea level rise on coastal flood hazards which is beyond the scope of this project.

A significant finding of the analysis is that above about 200 cubic feet per second (cfs), water originating from Easkoot Creek begins to bypass the lower portion of the creek and discharge directly to the Pacific Ocean. Though these breach locations are not designed as flood control structures they provide significant mitigation against more severe flooding impacts in the Calles. During the 100-yr flood, as much as 154 cubic feet per second (cfs) or 31% of the total discharge above the Parkside Café flowed to the ocean; additional mitigation is also provided by floodplain storage in the parking lots and picnic areas.

While we believe this modeling analysis provides significant insight into flood processes and inundation patterns on Easkoot Creek, the results are subject to uncertainty arising from several key sources. The model calibration was limited to a single location with measured water level data and the discharges associated with the historical calibration storms had to be estimated owing to a lack of direct discharge measurements at flows of this magnitude. Additionally, the models are based on a representation of recent topographic conditions, and as such may not be representative of channel conditions during the historical calibration storms making it difficult to fully evaluate the calibration accuracy. Lastly, although significant effort was made to evaluate the accuracy of the LiDAR and improve/correct it where possible, inaccuracies in the LiDAR data (primarily where dense vegetation is present) remain a source of model uncertainty. Despite these uncertainties, relatively good agreement between simulated and measured water levels and between local knowledge of historical inundation patterns and inundation patterns simulated with the models suggests that the models are an accurate tool for characterizing flood hazard conditions and evaluating mitigation alternatives on Easkoot Creek.

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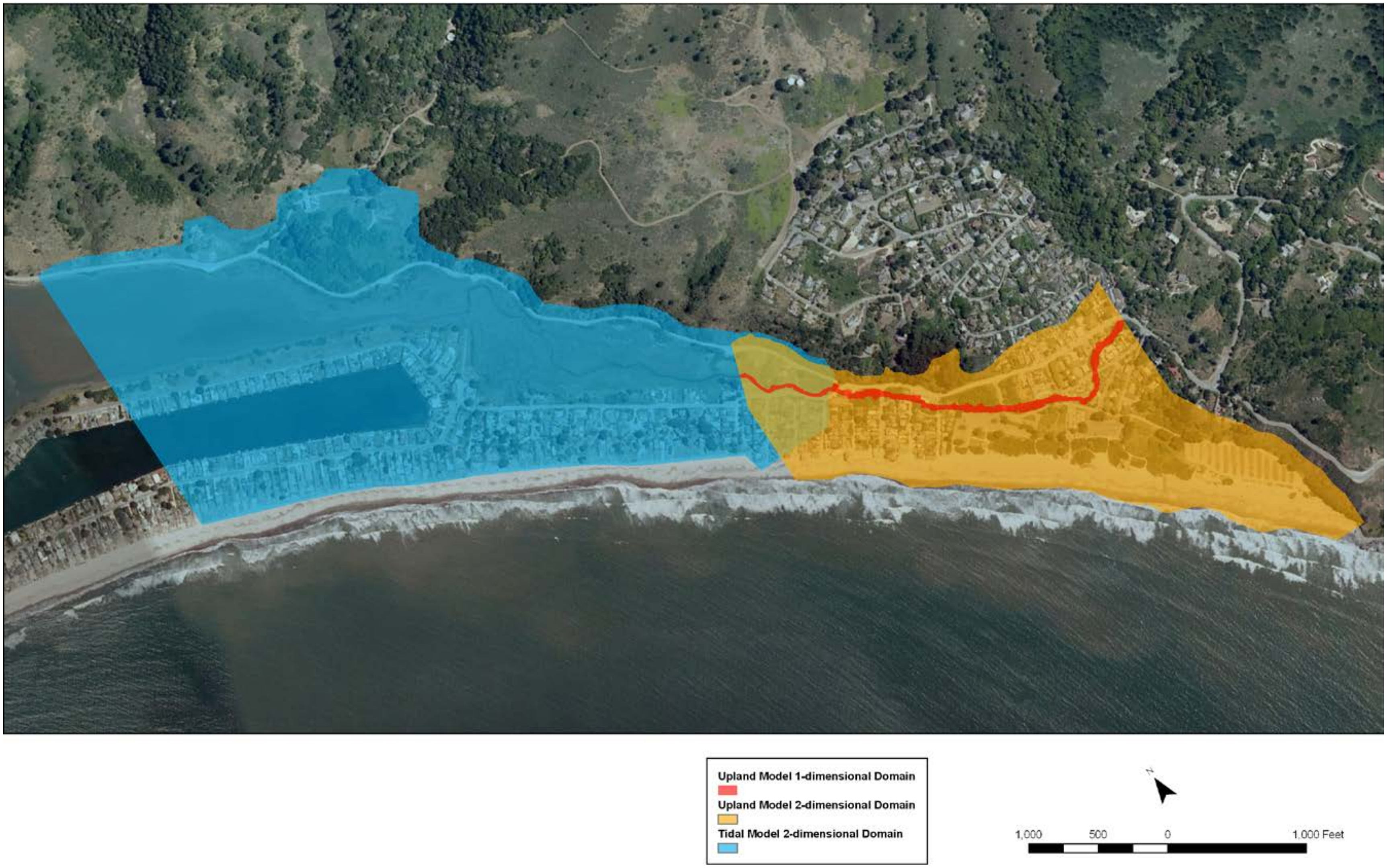

Figure D1 Extent of the model domains for the Tidal Upland models of Easkoot Creek.

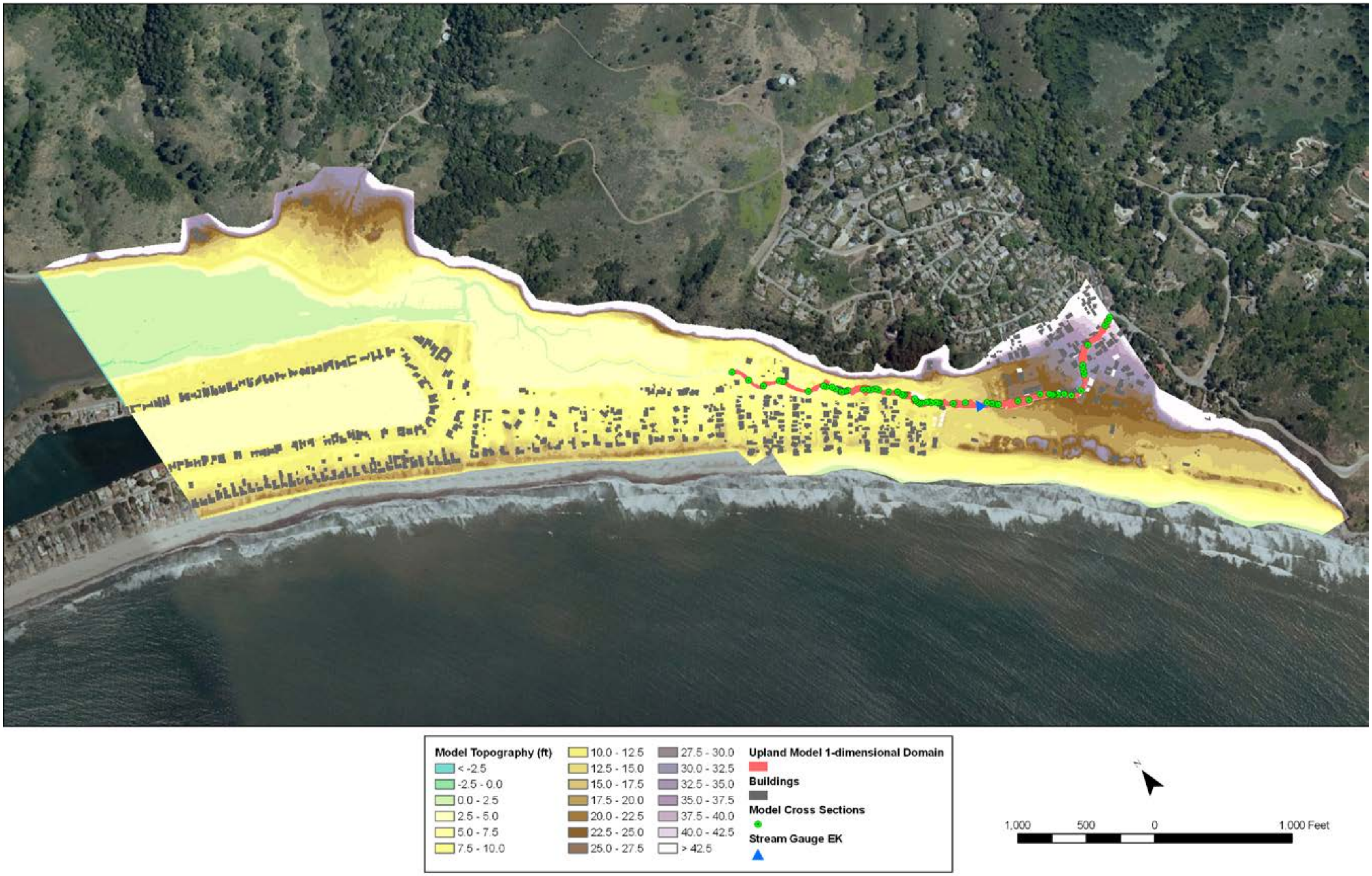

Figure D2 2-dimensional model topography, and locations of the 1-dimensional model cross sections and stream gauge EK.

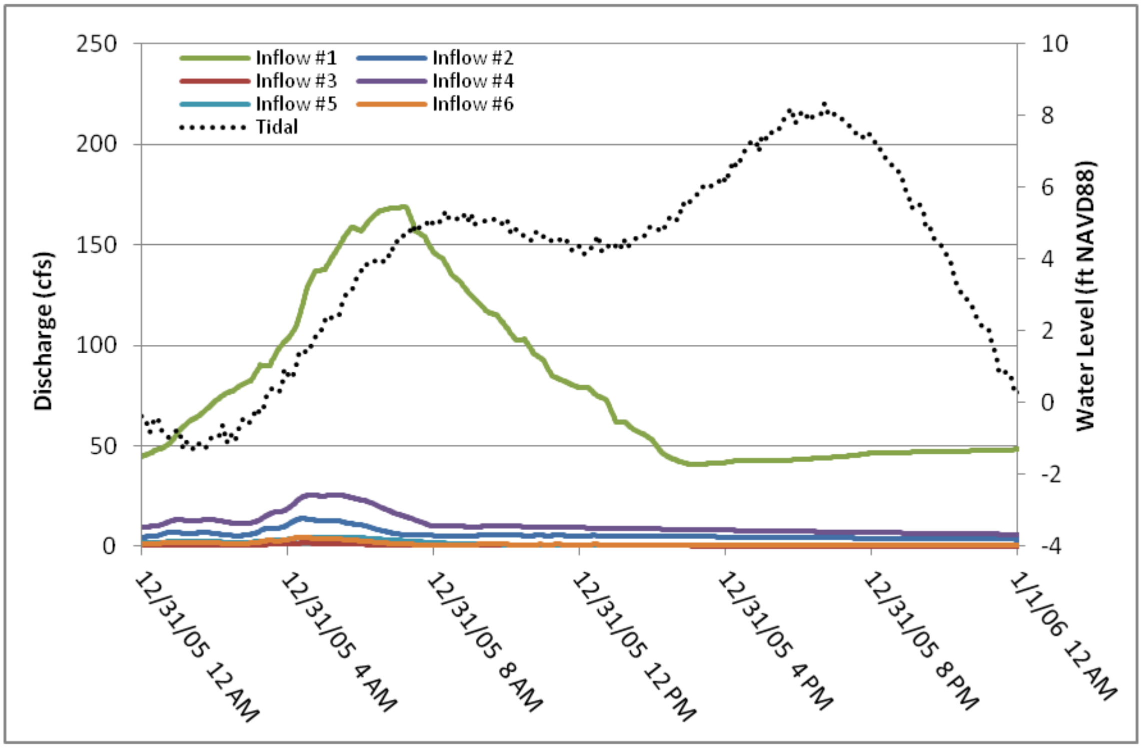

Figure D3 Boundary conditions for the December 2005 event.

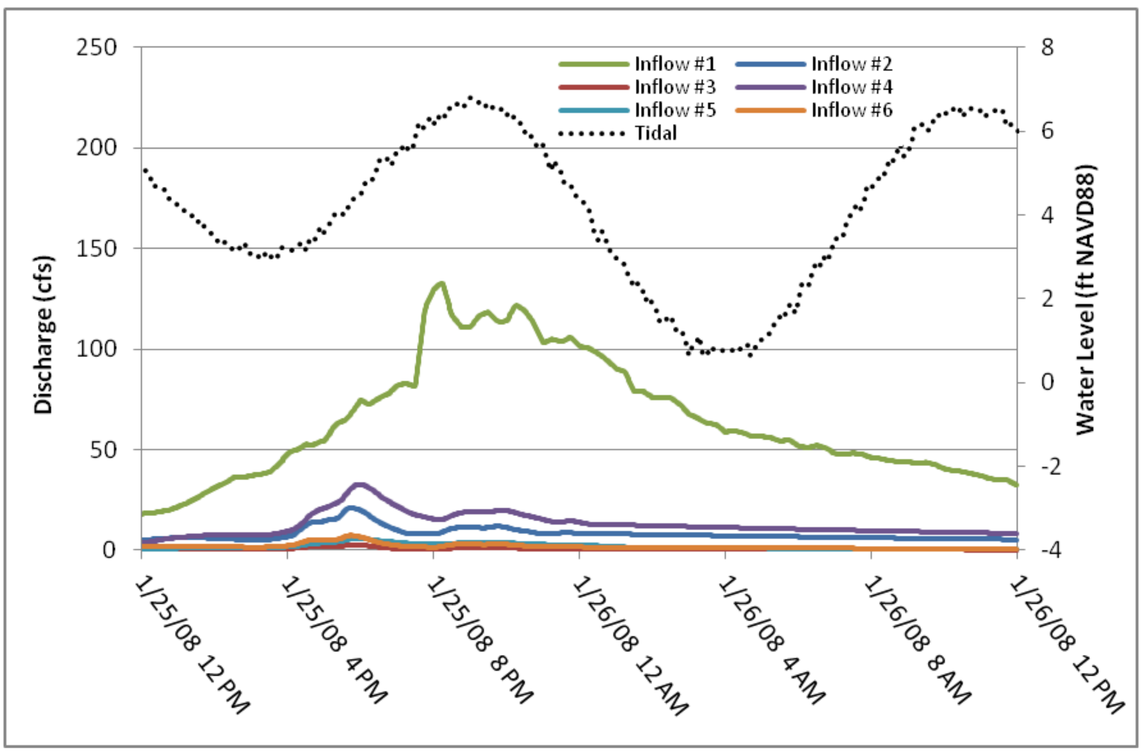

Figure D4 Boundary conditions for the January 2008 event.

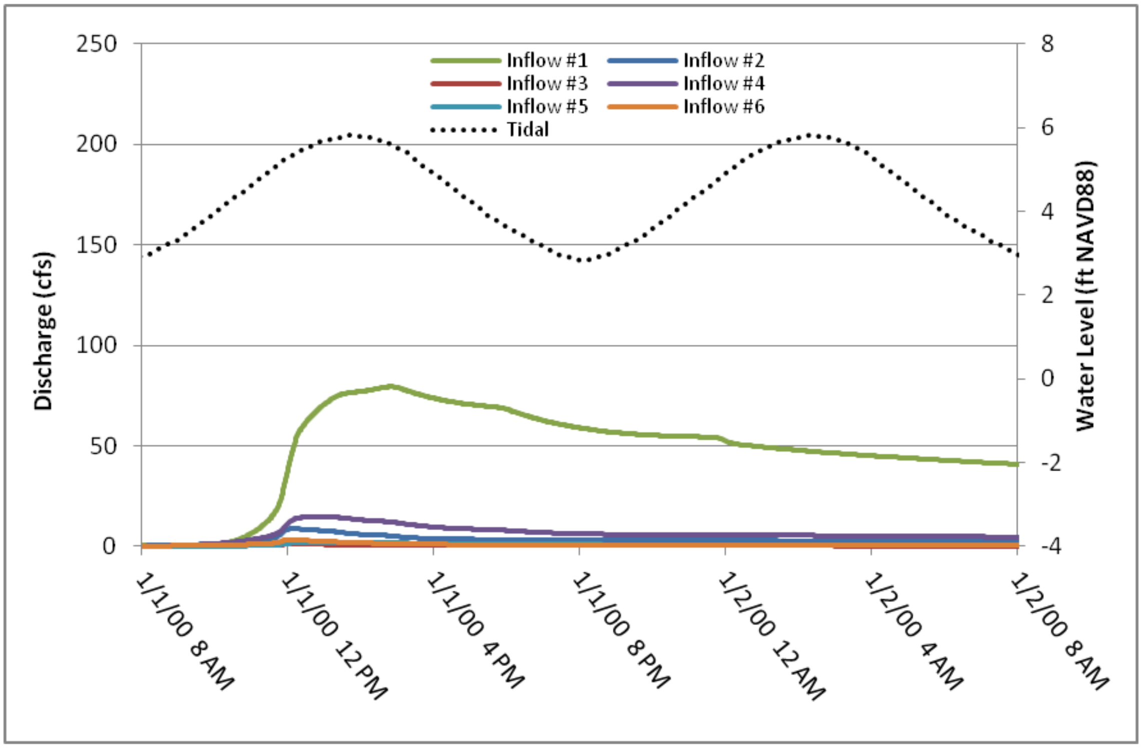

Figure D5 Boundary conditions for the 2 Year design storm.

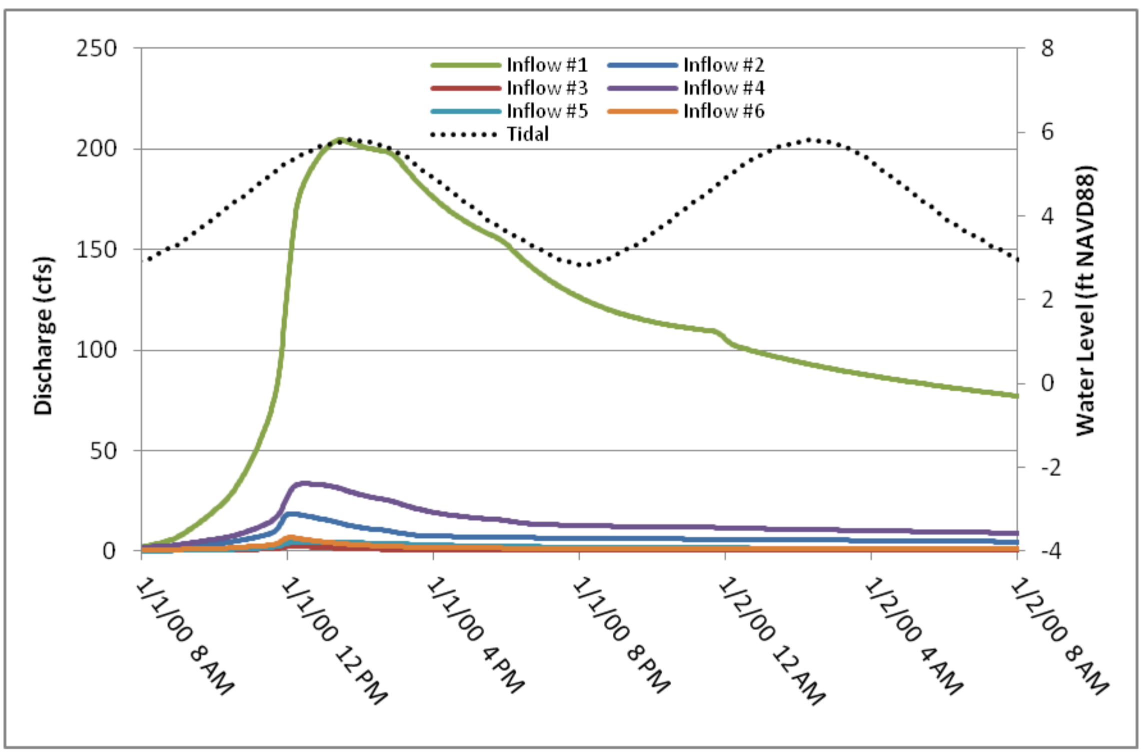

Figure D6 Boundary conditions for the 10 Year design storm.

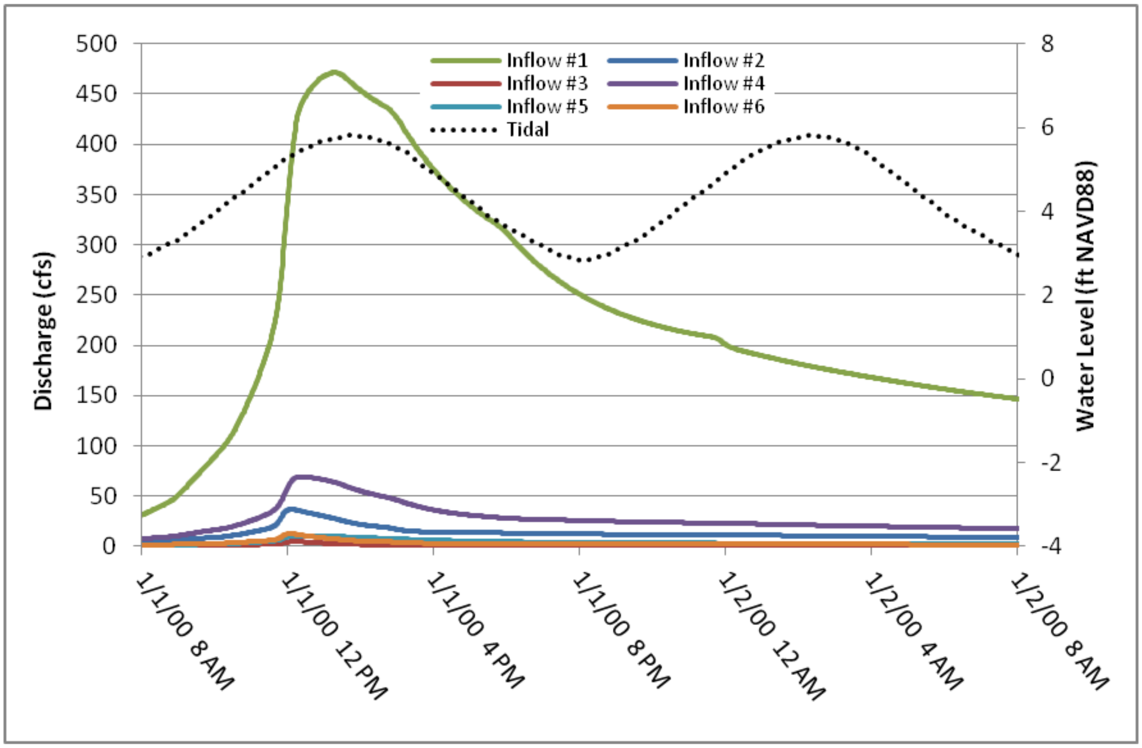

Figure D7 Boundary conditions for the 100 Year design storm.

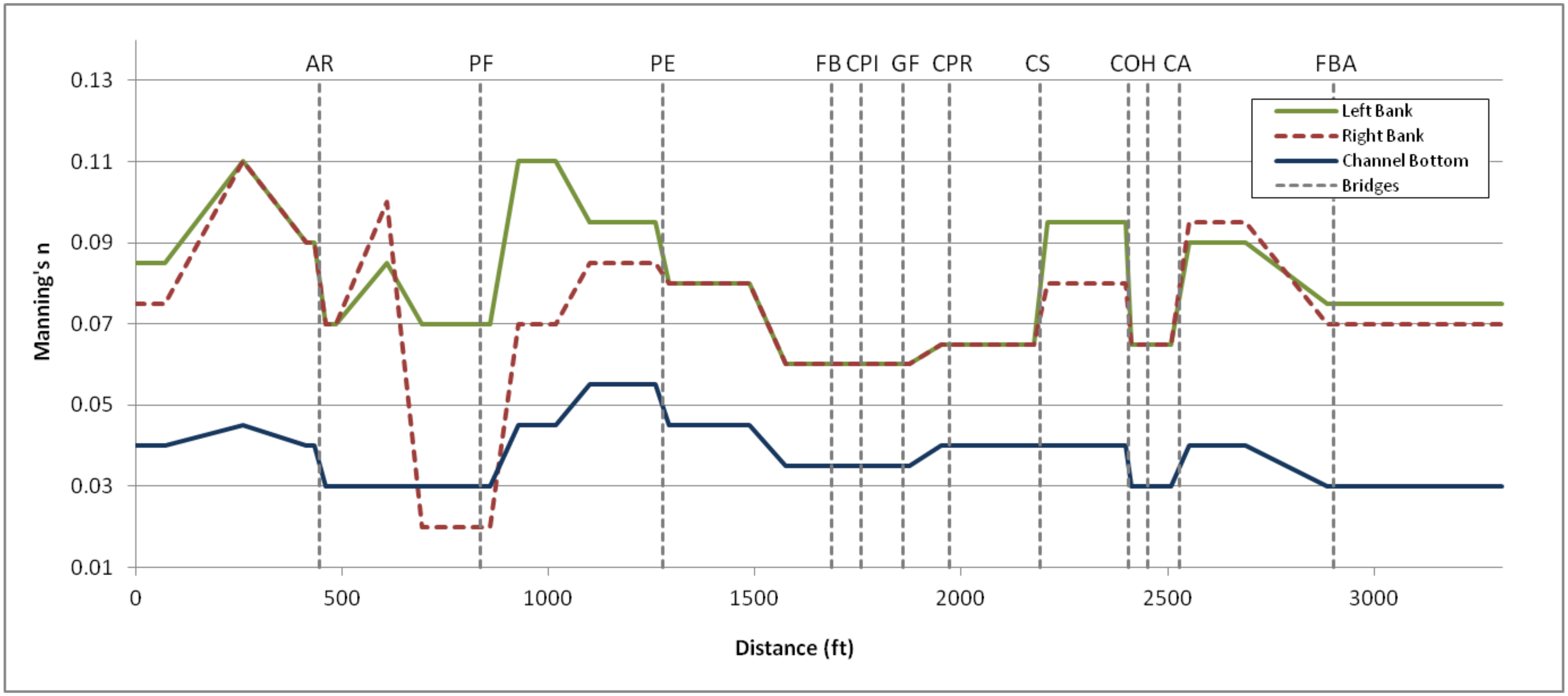

Figure D8 Longitudinal variation in Manning's n for the 1-dimensional component of the Upland Model.

Vertical dashed lines indicate the positions of the bridges for reference. Bridges are labeled as follows: AR (Arenal Ave.), PF (Park Footbridge), PE (Park Entrance Rd.), FB (Footbridge Above Calle del Pinos), CPI (Calle del Pinos), GF (Gym Footbridge), CPR (Calle del Pradero), CS (Calle del Sierra), CO (Calle del Onda), H (House Bridge), CA (Calle del Arroyo), and FBA (Footbridge below Calle del Arroyo).

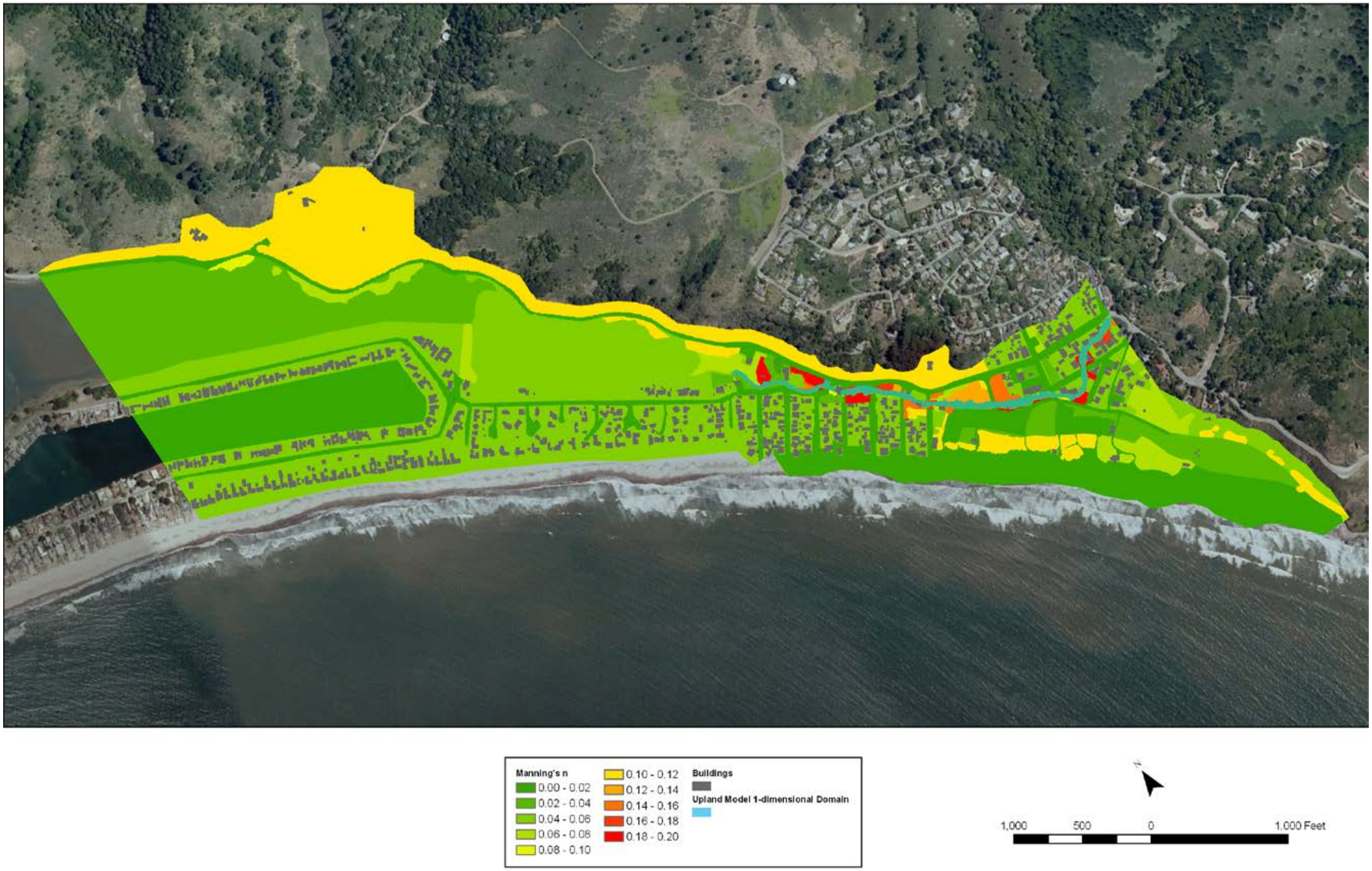

Figure D9 Distribution of Manning's n values used in the 2-dimensional models.

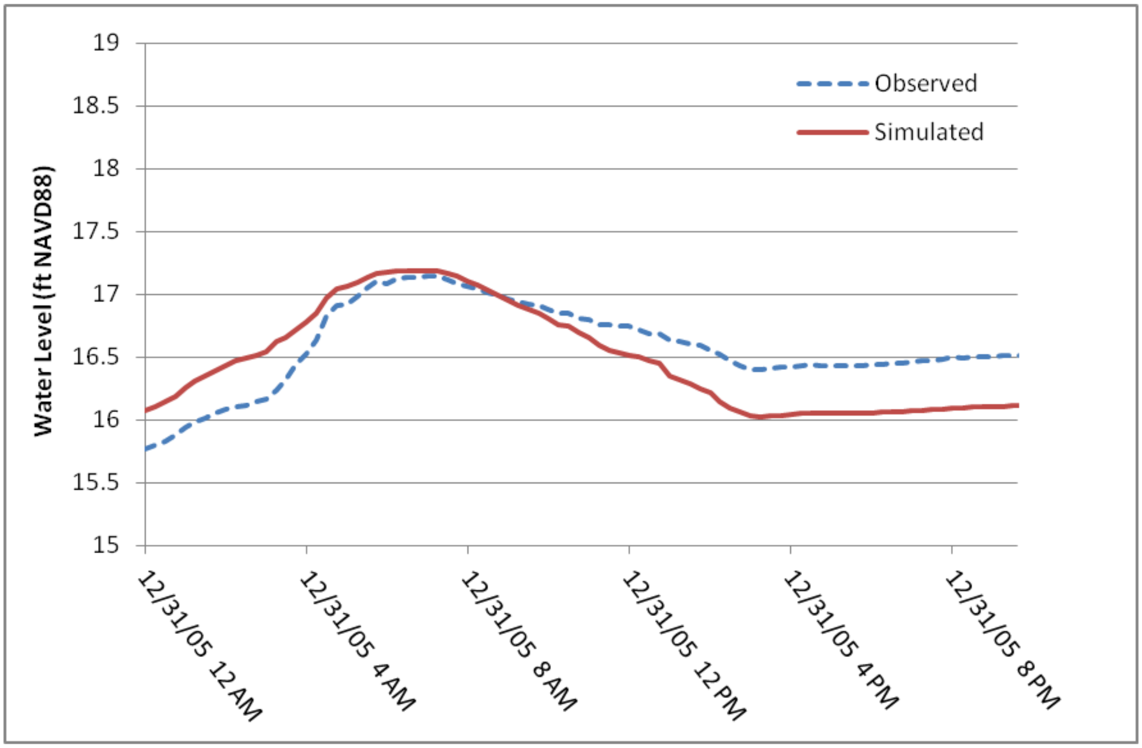

Figure D10 Comparison of simulated and observed water levels at gauge EK for the December 2005 flood event.

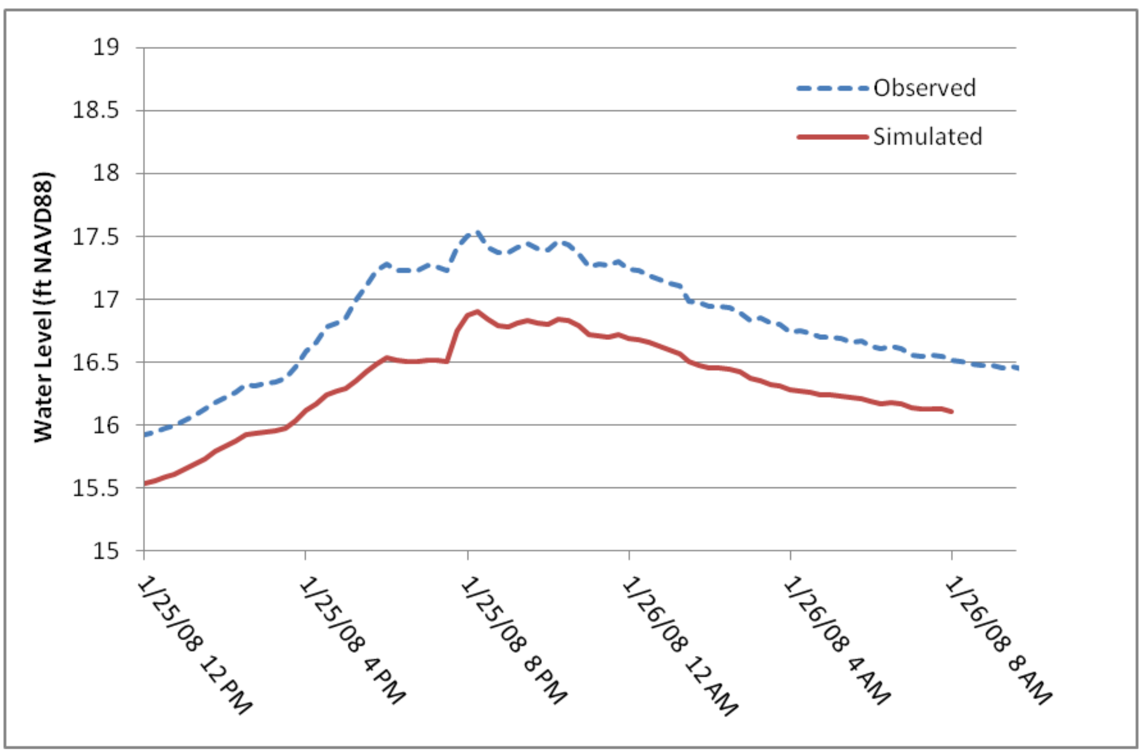

Figure D11 Simulated water-levels compared to measured water-levels for the Jan. 2008 event.

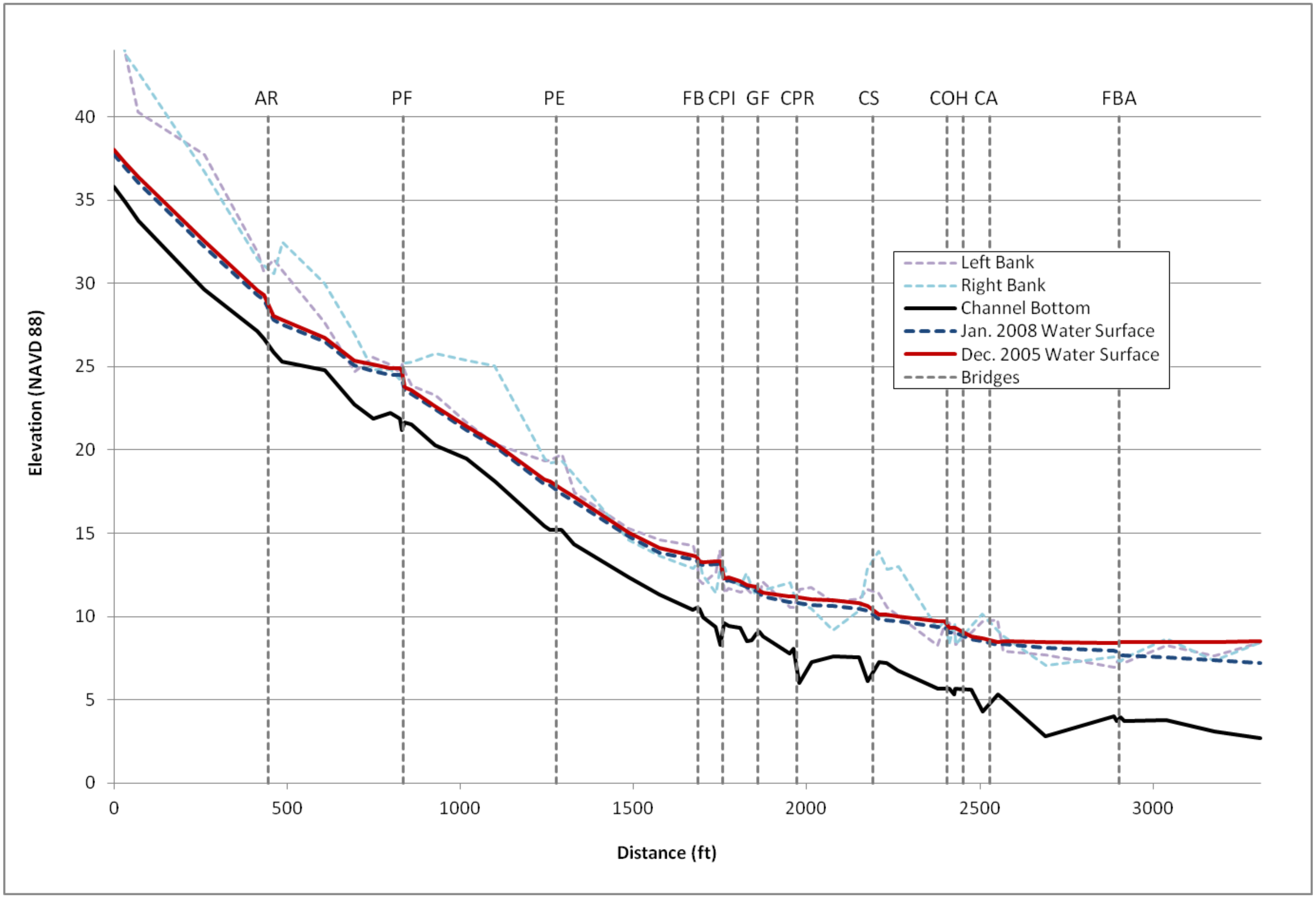

Figure D12 Water surface profiles for the Jan. 2008 and Dec. 2005 flood events.

(Upper reach of Easkoot Creek; see Figure 8 for explanation of bridge IDs).

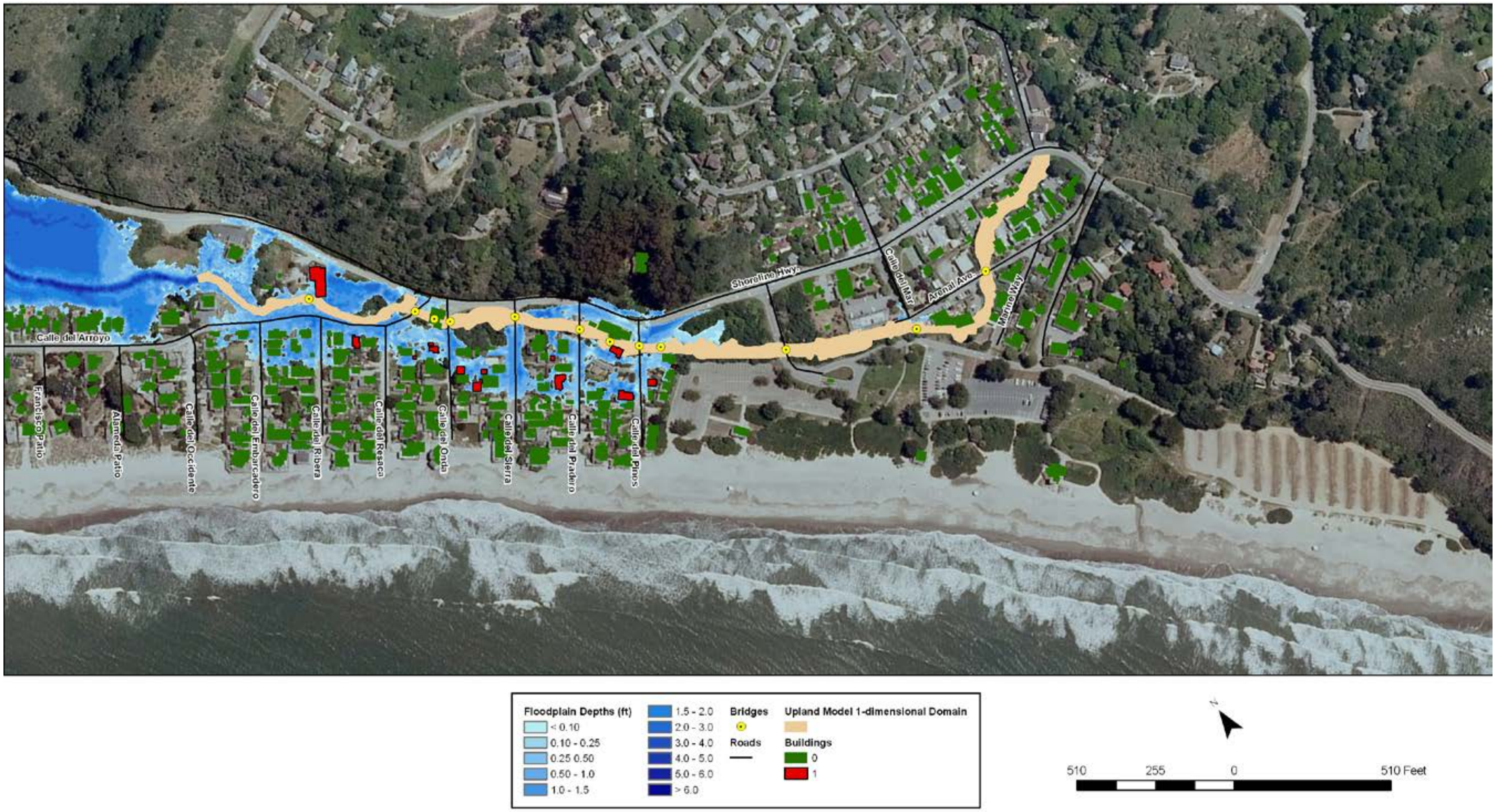

Figure D13A Maximum inundation depths during the January 2008 event (upper reach).

Figure D13B Maximum inundation depths during the January 2008 event (lower reach).

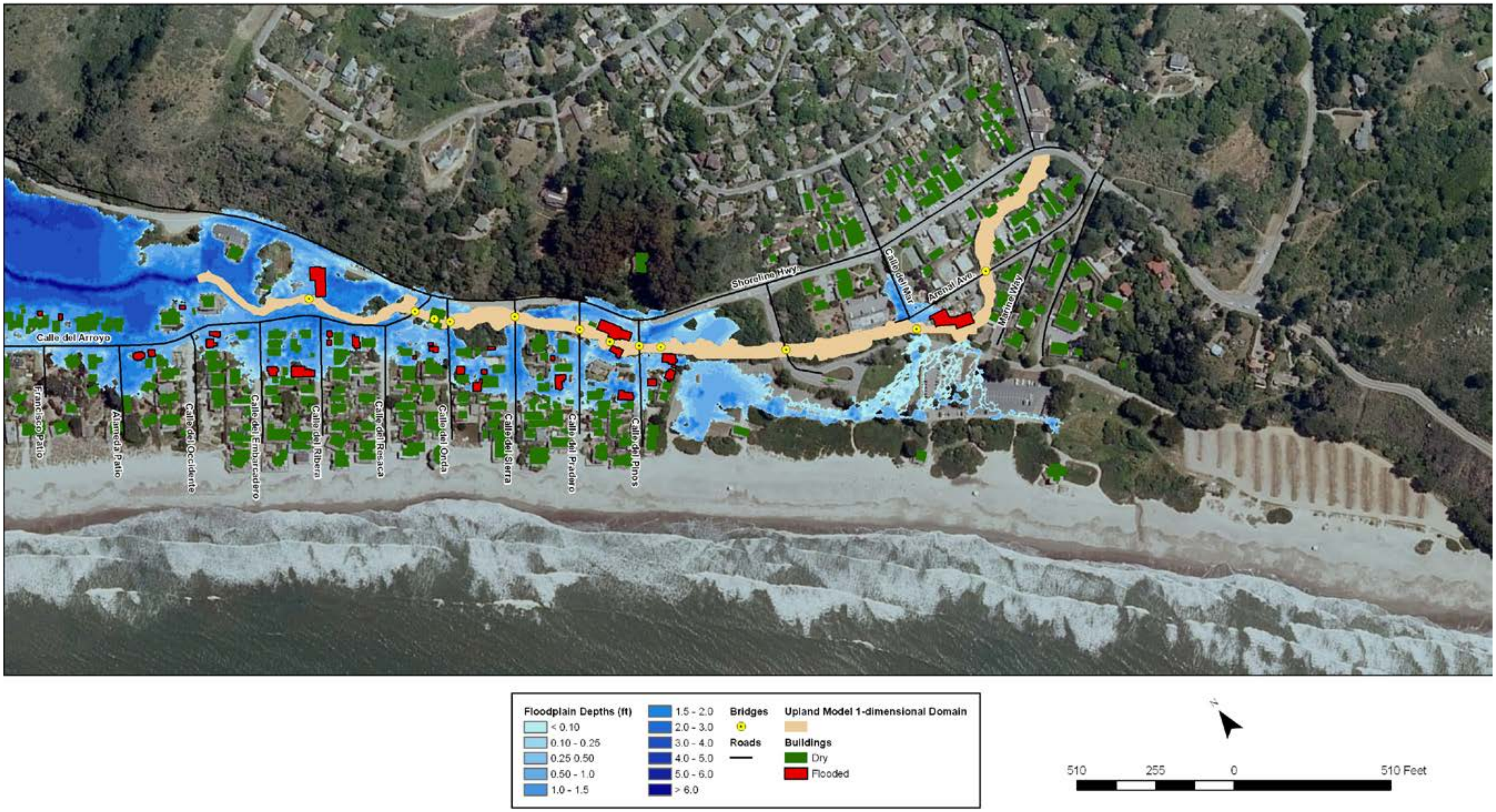

Figure D14A Maximum inundation depths during the December 2005 event (upper reach).

Figure D14B Maximum inundation depths during the December 2005 event (lower reach).

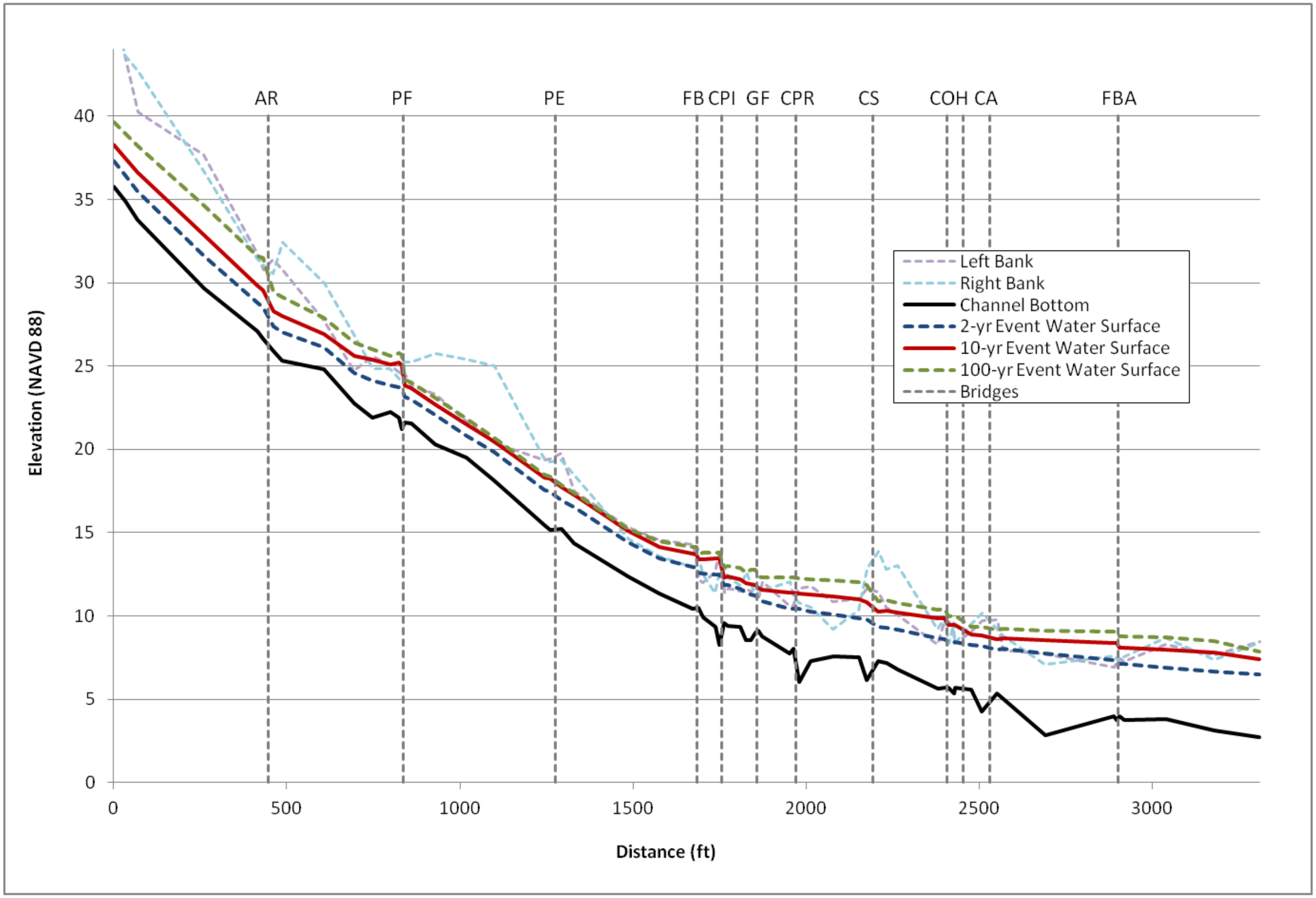

Figure D15 Water surface profiles for the 2-yr, 10-yr, and 100-yr flood events.

(Upper reach of Easkoot Creek; see Figure 8 for explanation of bridge IDs).

Figure D16A Maximum inundation depths during the 2-yr event (upper reach).

Figure D16B Maximum inundation depths during the 2-yr event (lower reach).

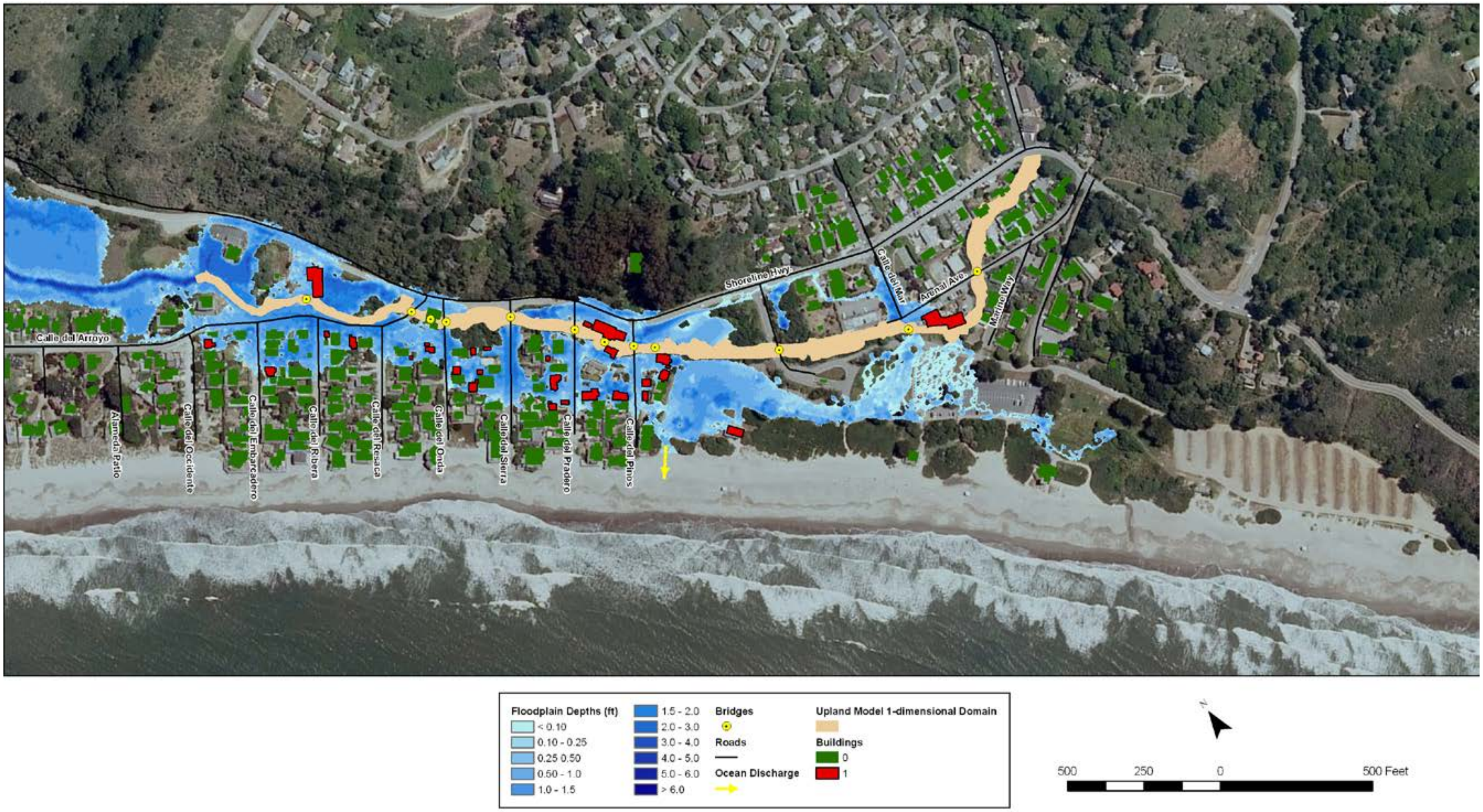

Figure D17A Maximum inundation depths during the 10-yr event (upper reach).

Figure D17B Maximum inundation depths during the 10-yr event (lower reach).

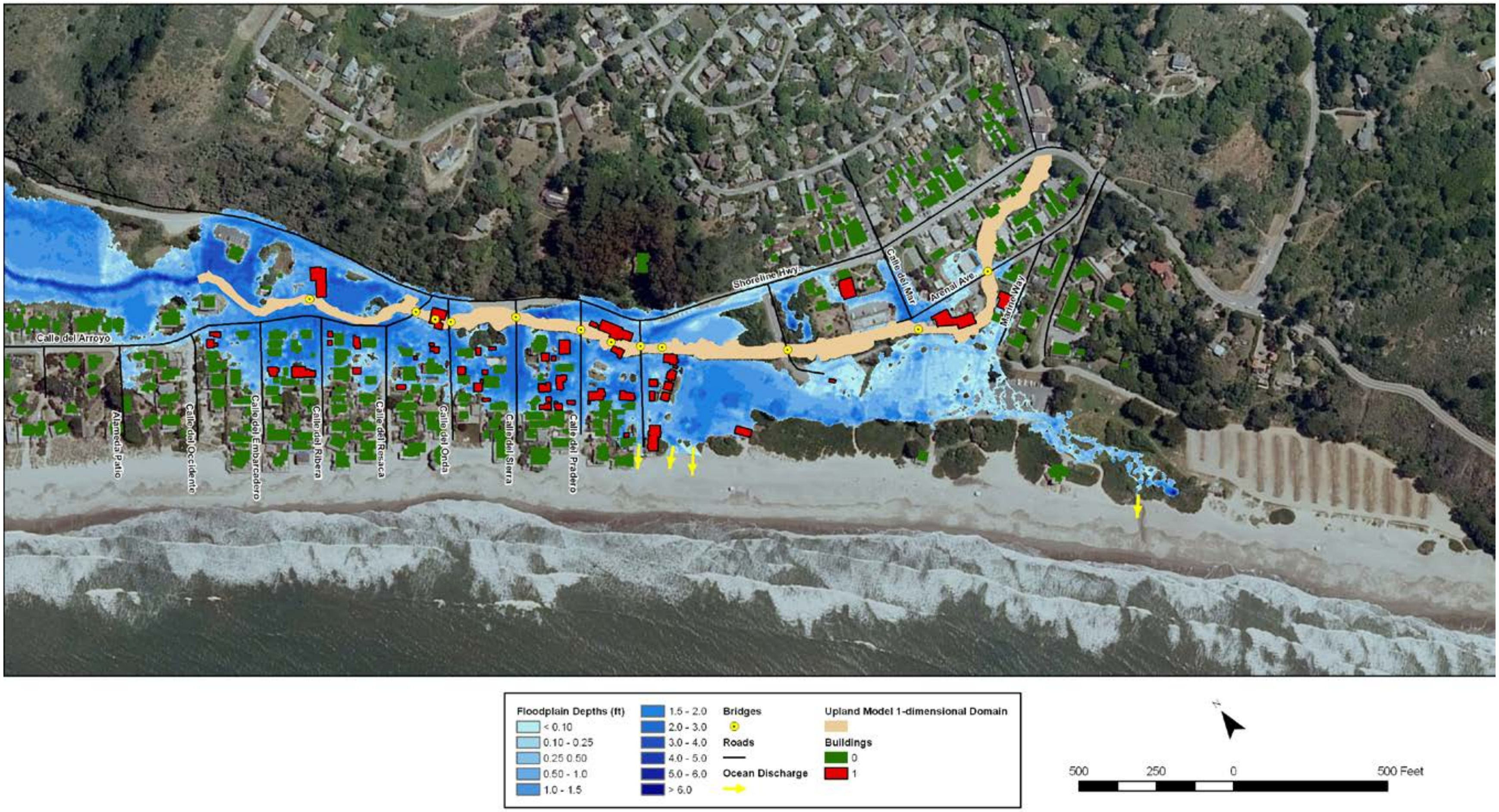

Figure D18A Maximum inundation depths during the 100-yr event (upper reach).

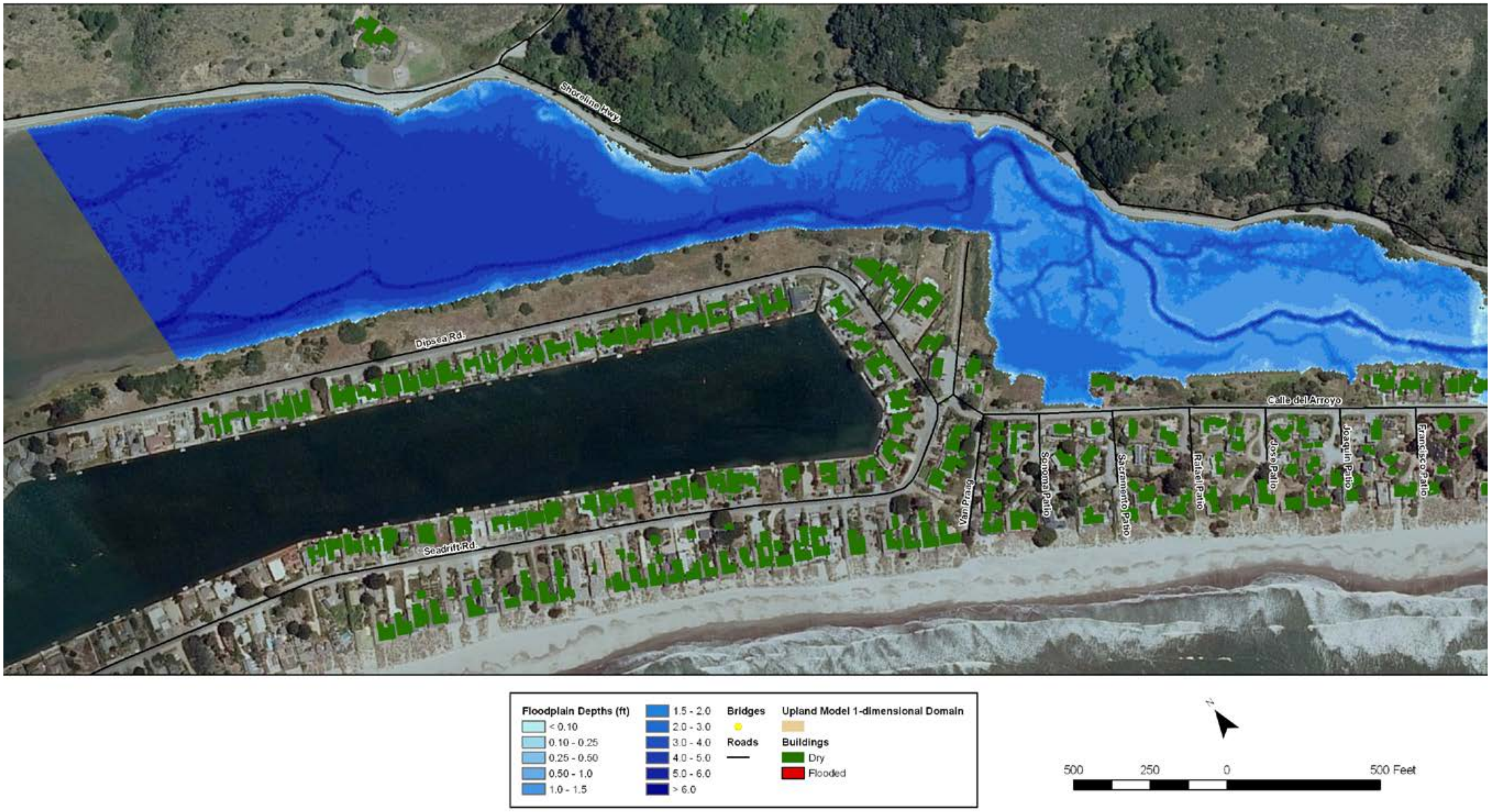

Figure D18B Maximum inundation depths during the 100-yr event (lower reach).

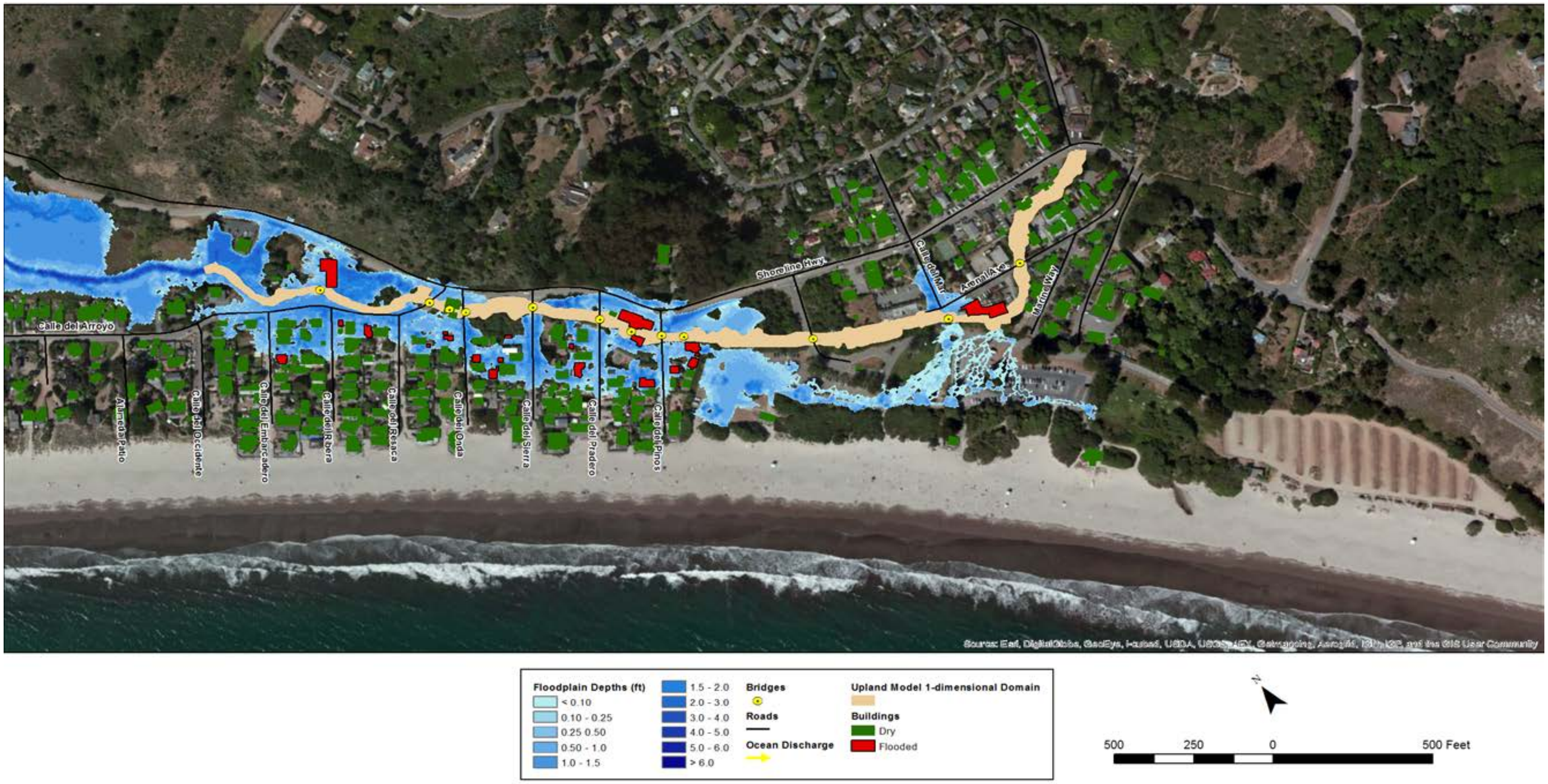

Figure D19A Maximum inundation depths during the December 2005 event using a MHHW tidal boundary condition (upper reach).

Figure D19B Maximum inundation depths during the December 2005 event using a MHHW tidal boundary condition (lower reach).

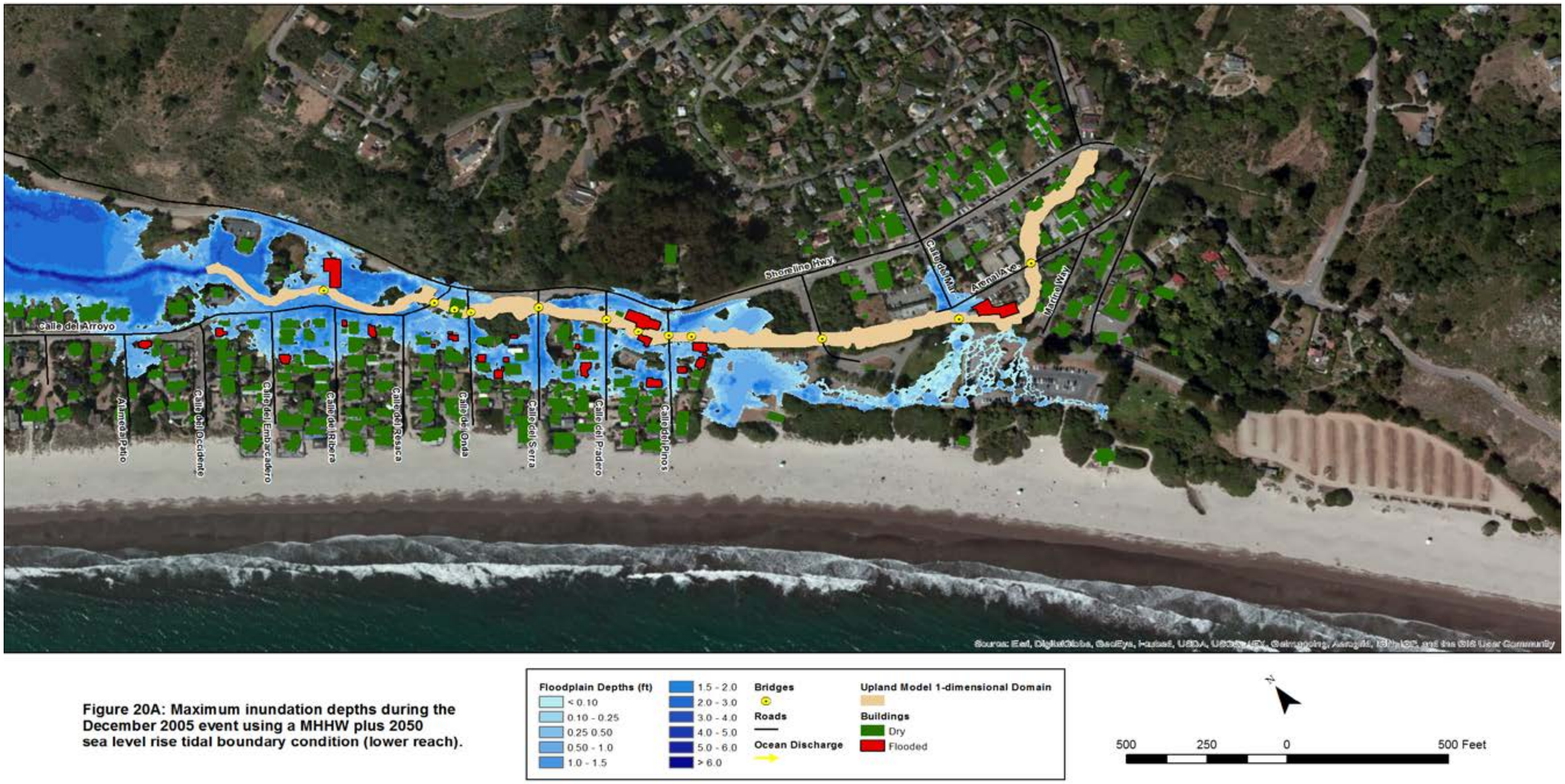

Figure D20A Maximum inundation depths during the December 2005 event using a MHHW plus 2050 sea level rise tidal boundary condition (upper reach).

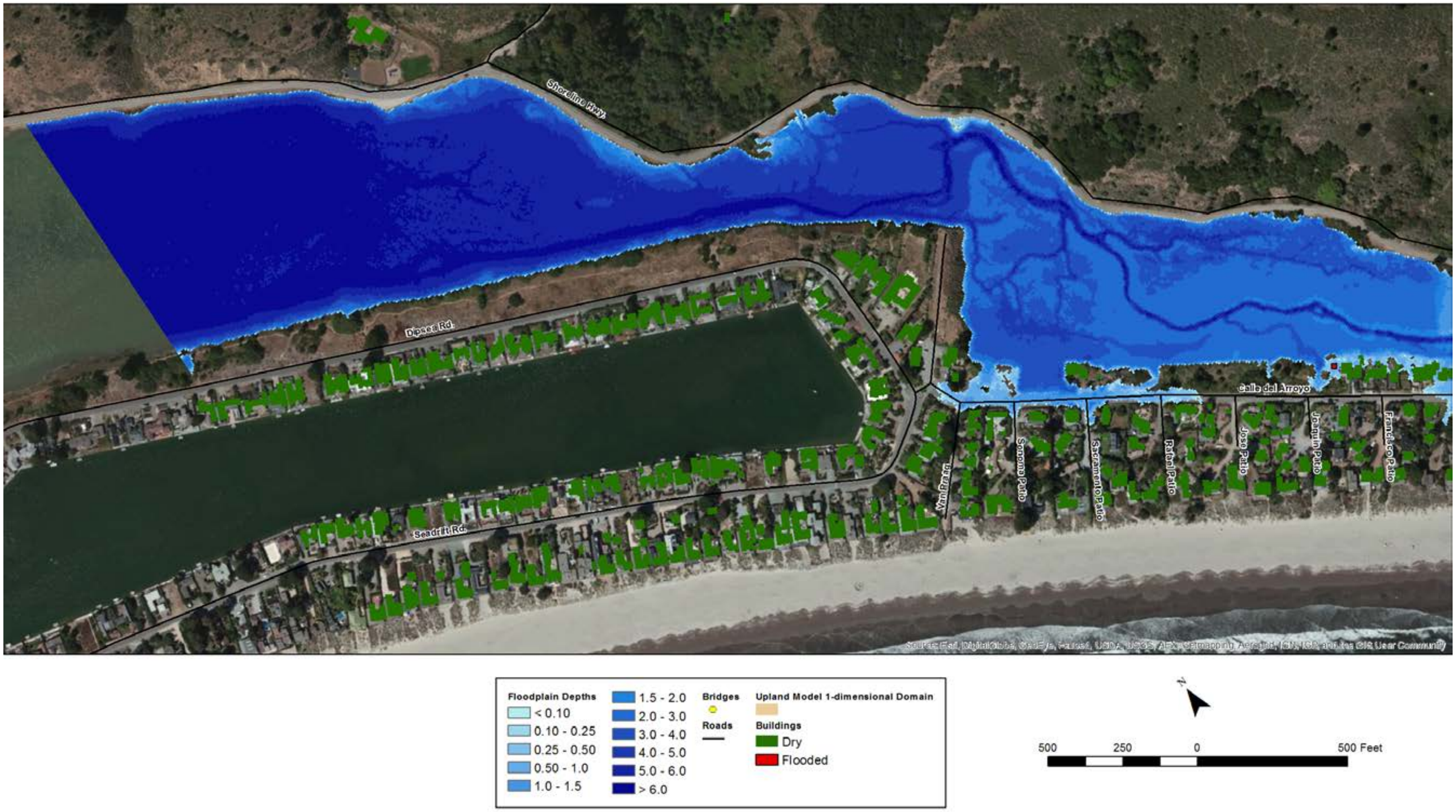

Figure D20B Maximum inundation depths during the December 2005 event using a MHHW plus 2050 sea level rise tidal boundary condition (lower reach).

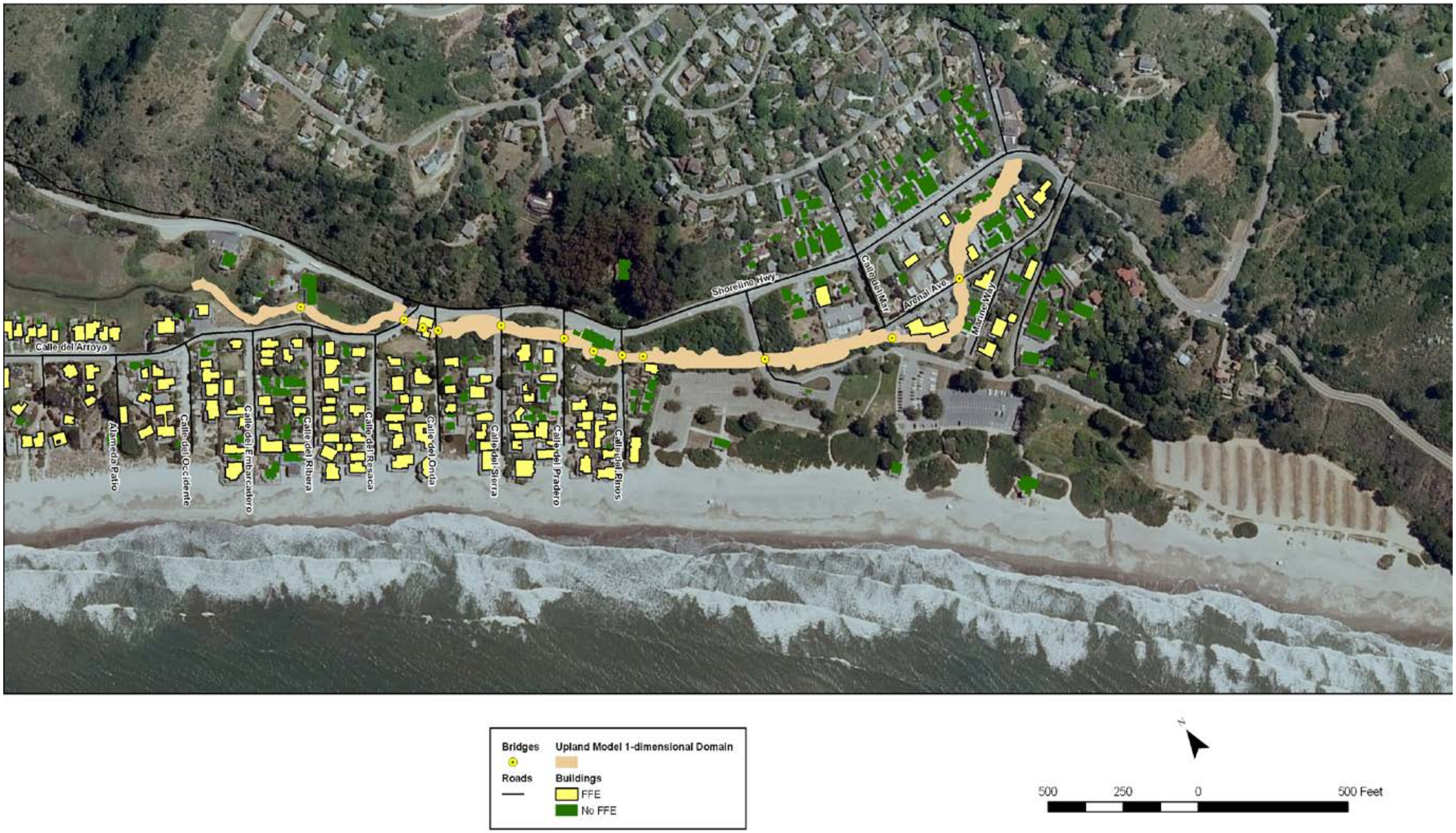

Figure D21A Buildings with surveyed finished floor elevations (upper reach).

Figure D21B Buildings with surveyed finished floor elevations (lower reach).

Original Draft December 31, 2012
Revised January 30, 2013
Revised February 28, 2013
Revised May 8, 2013

TO: Chris Choo
Roger Leventhal
Marin County Department of Public Works
Flood Control and Water Conservation District

FROM: Robert Aramayo, Garcia and Associates
Matt O'Connor, O'Connor Environmental, Inc.

SUBJECT: Easkoot Creek Hydrology & Hydraulics Study

Appendix F. Fish Habitat Existing Conditions and Enhancement Potential

Setting

Easkoot Creek flows out of a small, steep watershed of approximately 412 ha (1,018 acres) and is tributary to the southern end of Bolinas Lagoon. The upper portion of watershed is vegetated with annual grassland at the highest elevations, and conifer forest and a dense understory of ferns and vines are found in the middle elevations of the watershed. Alders are common along the lower reaches of the creek near Stinson Beach. The stream begins to lessen its grade and flatten out once it reaches the coastal plain between the steep upper watershed and the National Park Service (NPS) lands at Stinson Beach. Efforts to channelize the stream and stabilize the banks begin just below the Matt Davis Trail foot bridge and continue downstream to Bolinas Lagoon. Riparian vegetation in this lower section of Easkoot Creek transitions from the alder-fern-vine community common in the steeper uplands to a willow-blackberry community that dominates portions of the low gradient reach of the creek; exotic garden species also encroach from the adjacent houses in this lower section (Fong 2002). Downstream of the Arenal Avenue Bridge, Easkoot Creek has been straightened and flows north into Bolinas Lagoon. Upstream of tidal influence, the riparian community remains a dense tangle of willows and berry vines interspersed with many non-native species, along with a substantial component of native trees and shrubs; however, once the stream reaches tidewater, the channel flows through a pickleweed marsh.

Several species of fish are known to inhabit the lower reaches of Easkoot creek, up to the second crossing of the Matt Davis trail, approximately 1,300 feet (400 m) upstream of California State Route 1 (SR 1). Fish are presumed to be excluded from the upper watershed by the steep grade and small waterfalls, although the channel is too overgrown and steep to safely access and survey thoroughly. Regular residents of Easkoot Creek include steelhead (*Oncorhynchus mykiss;* Central California Coast Distinct Population Segment [DPS] - federally threatened [FT] status), three-spine stickleback (*Gasterosteus aculeatus*), coast range sculpin (*Cottus aleuticus*), prickly sculpin (*Cottus asper*), and staghorn sculpin (*Leptocottus armatus*) (Rich 1992). Coho salmon (*Oncorhynchus kisutch*; Central

California Coast Evolutionary Significant Unit [ESU] - state endangered [SE] and federally endangered [FE] status) have also been observed in Easkoot Creek (Darren Fong, *pers. comm*).

Staghorn sculpin and three-spine stickleback are primarily found in the lower, tidally influenced portion of the creek. Prickly sculpin, coast range sculpin, and steelhead can be found from tidewater upstream to near the second Matt Davis Trail Crossing (Rich 1992, Fong 2002). At this point, a steep cascade about 10 feet in height appears to form a barrier to upstream migration. Coho would have the same distribution as steelhead.

Steelhead Life History- Central California Coast DPS (FT)

Steelhead employ a variety of life history strategies that take advantage of the diversity of river systems and regional conditions to which they are adapted. Central California Coast steelhead have a typical "winter" immigration pattern and a "ocean-type" gamete development, which means that adults arrive at their spawning grounds with their eggs close to maturity, and are therefore ready to spawn within a short period of arriving (Moyle 2002). Steelhead typically choose steeper-gradient stream reaches, further upstream and further up tributaries than chinook or coho salmon. However, in Easkoot Creek, the distribution of steelhead is limited to the lower reaches, along the coastal plain and upstream to a migration barrier formed by a steep cascade approximately 1,300 feet above State Highway 1. Steelhead typically begin returning to their natal streams in winter, with most immigration occurring from December through February (Rich 1992, Fong 2002). Spawning takes place from January through April. Adults spawn in clean gravels and cobbles, typically at tail crests or riffles where surface waters are forced into the gravel, thereby keeping the gravel clean and the buried eggs well oxygenated. An abundance of fine sediment particles, less than one mm, can interfere with the delivery of oxygen to the eggs, the removal of waste products from the living eggs, and can interfere with the transfer of oxygen across the egg membrane. Similarly small gravel particles, 1 – 10 mm, can create a layer over the redd, making it difficult for alevins to emerge from the gravel. Generally, substrates composed of over 12-14 percent fine sediment particles of less than one mm; or contain over 30 percent particles smaller than six mm, have been shown to impair the spawning success of steelhead and other salmonids (Kondolf 2000).

Juvenile steelhead are found in all habitat types, and seasonal changes in stream conditions influence their habitat preferences. Steelhead require water temperatures below 20°C, and water temperatures between 20-23°C are considered stressful. Steelhead are typically excluded from streams where water temperatures exceed 23-27°C for extended periods of time. Steelhead and coho often co-exist, but will segregate by habitat preferences, with steelhead choosing the swifter, more mainstem habitats. In California, most juvenile steelhead remain in their natal streams for two years before emigrating to the ocean during the late spring or early summer, although strategies with one to four years of freshwater residence are also known from California. Age 0+, 1+, and 2+ steelhead have been documented in Easkoot Creek (Fong 2002). Estuaries are often important rearing areas for juvenile steelhead on their way to the ocean; however, Bolinas Lagoon does not appear to hold many juvenile steelhead (Rich 1992, Fong 2002).

Steelhead can remain in the ocean for one to four years before returning to freshwater to spawn for the first time (two years of ocean residence is the norm). However, unlike chinook and coho salmon, steelhead do not necessarily die after spawning.

The Central California Coast Steelhead DPS is listed as a federally threatened species. This DPS includes all naturally spawned populations of steelhead in coastal drainages from the Russian River basin south to Santa Cruz County. Critical habitat was previously designated for this DPS in all accessible stream reaches in its geographic range (NOAA 2000). This designation was rescinded by federal court decision in 2002. However, critical habitat was re-issued for the Central California Coast Steelhead DPS in 2006 (NOAA 2006) to include all known populations, including Easkoot Creek. There is no Essential Fish Habitat (EFH) for steelhead in California.

Coho Salmon Life History- Central California Coast ESU (FE, SE)

Coho salmon have a relatively fixed three-year life cycle. Adults typically return to their natal stream in the fall to spawn. In California, adult coho typically return to spawning areas between November and January, often moving upstream with the high water of winter storms. Most spawning occurs in December and January. Adults spawn in clean gravels and cobbles, typically at tail crests or riffles where surface waters are forced into the gravel, thereby keeping the gravels clean and buried eggs well oxygenated. Adult coho spawn in smaller waters and tributaries than Chinook salmon (*Oncorhynchus tshawytscha*), although there is some overlap with habitats chosen by steelhead. Juvenile coho are found in all habitat types, and habitat preferences change with seasonal changes in stream conditions. Coho usually segregate themselves from steelhead and other salmonids, often choosing deeper waters with complex cover including a mix of woody debris and undercut banks (habitats that are largely absent in Easkoot Creek). Juvenile coho remain in their natal streams for their full first year, and begin emigrating to the ocean during the spring of their second year. Coho require cool water temperatures, and are excluded from streams where summer water temperatures exceed 22-25°C for extended periods of time; however, some data suggests that the upper thermal limit may be closer to 18°C (Moyle 2002). Water temperatures from 18-22 °C are stressful for coho salmon.

In California, most coho remain in the ocean for the end of their second and third years, before returning as adults at the end of their third year. Some precocious males return as two year old ‘jacks.’ Adult coho salmon die after spawning.

The Central California Coast ESU encompasses all naturally-spawned populations in rivers and tributaries from the San Lorenzo River in Santa Cruz County north to Punta Gorda in Mendocino County. Critical habitat is designated for this ESU throughout its geographic range (NOAA 2000). Critical habitat and Essential Fish Habitat encompasses all accessible reaches of all rivers (including estuarine areas and tributaries) between Punta Gorda and the San Lorenzo River (inclusive) in California. The corresponding state-designated “Northern California population” of coho salmon is state-listed as endangered from San Francisco Bay north to Punta Gorda.

Instream Habitat

The lower reaches of Easkoot Creek that are inhabited by fish are relatively narrow (typically six feet or two meters wide), although wetted width varies from approximately three to ten feet wide (one to three meters). The mean water depths during the summer are typically less than 0.3 feet (0.1 meter), with few deeper pools. Maximum water depth at the sites electrofished by NPS was less than one meter (Fong 2002). Much of the woody debris that would otherwise provide instream cover for fish has been removed (Fong 2002). Consequently, there is relatively little instream cover provided for fish and other aquatic organisms, with the exception of some riparian vegetation reaching into the stream.

Gravels are the dominant substrate in the middle reach of Easkoot Creek from the Matt Davis Trail down to the tidally influenced section, with some cobbles and small boulders present along with significant fine sediment. Substrate in the tidally influenced section is primarily silt and clay (Fong 2002).

Water Quality Parameters

Stream flow in Easkoot Creek is seasonal, with low flows during the summer. NPS personnel report that portions of lower Easkoot Creek become intermittent when flows measured at the gage reach 0.3 cubic feet per second (cfs) (cubic feet per second (cfs)). The dry reach occurs from above the Arenal Bridge to the downstream end of the Parkside pedestrian bridge (Fong 2002). The lack of water during the late summer diminishes both the quantity and quality of the instream habitats, and is a serious limiting factor for steelhead and other aquatic organisms living in Easkoot Creek (Rich 1992).

In addition to the loss of habitat resulting from diminishing surface flows, water quality can deteriorate significantly during periods of low flow, making the stream unsuitable for steelhead and other cold water organisms living in the stream. Often, as stream temperatures increase, the amount of dissolved oxygen (DO) in the water decreases.

NPS measured the DO in Easkoot Creek from July through October, 2000, at two locations, one near the park entrance bridge and one near the Community Center upstream of State Highway 1. At the lower station, NPS found that the concentrations ranged from 2.4 to 8.1 mg/l during this period and approximately 30 percent of days during this period had minimum DO levels less than 5 mg/l (Fong 2002). During this period, large diel swings in the DO concentrations, often three-fold changes, were observed in Easkoot Creek. Such large daily changes in DO may be attributed to large amounts of aquatic vegetation and algae present in the stream, which add DO via photosynthesis during the day and removing DO through respiration at night. The invertebrate community measured in lower Easkoot Creek is consistent with habitats containing slow moving water and low DO levels (e.g., many species present are tolerant of low DO conditions such as the rat-tailed maggot *Eristalis sp.*) (Fong 2002).

Generally, water temperatures above 20°C are stressful to salmonids (Moyle 2002). Rich (1992) reported that water temperatures in Easkoot Creek exceeded 20°C (up to 23°C) in July, August, and September 1991. NPS did not record any water temperatures above 20°C during their monitoring efforts from July through October 2000 (Fong 2002), and found cooler temperatures at the upstream station.

Limiting factors for Steelhead and Coho Salmon

While Easkoot Creek supports a persistent steelhead population, and coho salmon have been recently observed in the stream, the anadromous salmonid population is limited by the quantity and quality of instream habitats available to steelhead and coho. Fish have historically been distributed in the lower reaches along the coastal plain and upstream approximately 1,300 feet above State Highway 1 to a natural migration barrier. In the most recent systematic surveys in 1999 and 2000 (Fong, 2002), steelhead redds were found as far upstream as State Highway 1, and juvenile steelhead (age 0 to 2+ years) were found in both lower Easkoot Creek and upper Easkoot Creek upstream of State Highway 1 in August 2000. The overall area available to fish is small, and the accessible area has few habitats suitable for spawning. The quality of the available habitats has been degraded by channelization and management actions that have reduced the complexity of instream habitats (e.g., removal of large woody debris). Furthermore, water quality within the available habitats is likely degraded during late summer as streamflow diminishes, temperatures rise, and DO levels fluctuate and drop to levels that are stressful to salmonids.

Potential spawning habitats in the freshwater section available to steelhead and coho are limited by both the overall length of stream and the quality of the gravels (i.e., an abundance of fine sediments that clog interstitial spaces between gravel particles). Salmonids typically spawn in gravel-dominated habitats at the head of riffles or the tail-outs of pools where oxygen rich water is hydraulically forced into the gravel such that it carries DO to the fish eggs, and can carry carbon dioxide and other waste away from the eggs. Bed samples in lower Easkoot Creek showed substrate compositions of 17, 16, and 8 percent of particles less than one mm in diameter, and compositions of 38, 33, and 29 percent of particles less than six mm in diameter, suggesting that both the incubation of the eggs and the emergence of the alevins are likely to be impaired.

Easkoot Creek is limited both in the number of hydraulically suitable habitats available and the relatively high proportion of fine sediment in the streambed, especially downstream of Arenal Avenue where long-term sedimentation effects are prevalent owing to declining stream gradient and diminishing channel confinement.

Available rearing areas are also limited by the relatively short length of stream available to salmonids, and the shallow depths of the water during the typically dry summers. This volume of suitable habitat is further reduced when the creek becomes intermittent in late summer, drying up substantial portions of habitat. Easkoot Creek surface flows routinely dissipate to the subsurface between Arenal Avenue and Calle del Mar; surface flows upstream of Arenal Avenue diminish in late summer and early fall, but typically do not disappear (Fong 2002). The limited number of pools in the channel reduces the extent of potential refugia for fish as stream flow declines. The remaining available habitat would be further impaired by potential low DO levels and stressfully high water temperatures during low-water years, particularly in lower Easkoot Creek below Arenal Avenue. The limited available habitat not only affects the water quality of Easkoot Creek, but also concentrates the fish, thereby making them more vulnerable to predators (e.g., great blue heron). The tidally influenced portion of the creek has no instream cover and most likely only serves as a migratory corridor for fish moving from the creek to Bolinas Lagoon and the Pacific Ocean.

The suitability of Easkoot Creek for coho salmon is further limited by the nature of the available habitats. While the spawning habitat preferences are similar for steelhead and coho, juvenile coho prefer more protected areas with high amounts of complex instream cover (e.g., backwater channels and undercut banks with extensive root wads). Easkoot Creek has very few areas with sufficient complex cover for coho to thrive.

Potential Habitat Enhancement

If necessary, potential habitat enhancement could occur in two reaches: the first (HE1) between Arenal Avenue and State Highway 1 and the other (HE2) extending from the confluence of Black Rock Creek (between the Community Center and the Fire Station) upstream to the Matt Davis Trail bridge. Each of these reaches is about 400 feet in length. The channel gradient in HE1 is about 1.7% and is about 5.9% in HE2. These reaches were selected because they are relatively accessible, they are located nearest to documented spawning sites (Fong 2002) in the proposed dredging reach, water availability and water quality is better: surface flow and dissolved oxygen conditions are significantly better in HE1 and HE2 relative to the dredging reach (Fong 2002), and the channel conditions are much less prone to sedimentation. These sites have also been disturbed by urban development and channelization (HE2 is lined with boulder rip-rap throughout its entire length), and are appropriate areas for habitat enhancement.

Detailed planning for the enhancement work is beyond the scope of this analysis, however, installation of about four boulder vortex weirs and/or upstream-v log weirs (CDFG 1998) in each reach would promote development of stable pool-riffle habitat sequences to provide both spawning and rearing habitat for anadromous and resident salmonids. In HE1, these installations would need to be designed to provide bank stability as well as habitat. In HE2, the existing rip-rap maintains bank stability. Utilization of redwood logs in enhancement structures is preferred, owing to the improved cover provided for rearing habitat, as well as the positive association between juvenile fish and large wood in Easkoot Creek (Fong 2002). Log structures would need to be designed for stability; installed logs should be buried and/or weighted to prevent flotation. Consideration should be given to completion of habitat enhancement work one year prior to dredging so that the improved habitat would be available to fish that might initially avoid the disturbed dredged reach. Completion of habitat enhancement in HE1 and HE2 prior to dredging might be necessary to allow dredging to occur in a single year in that the habitat enhancement work would be designed to ensure availability of suitable spawning habitat after completion of dredging.

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Revised May 15, 2013

TO: Chris Choo
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Marin County Department of Public Works
Flood Control and Water Conservation District

FROM: Matt O'Connor
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SUBJECT: Easkoot Creek Hydrology & Hydraulics Study

Appendix F. Sediment Transport Evaluation

Introduction

The purpose of this summary is to present analyses of sediment transport rates in Easkoot Creek from State Highway 1 to the tidal zone just downstream of Arenal Avenue where sedimentation and flooding occurs and within the domain of the hydraulic model. Sediment transport rates are relevant to the flood mitigation study because channel sedimentation reduces channel conveyance, thereby increasing the likelihood of overbank flow and associated flooding. Extensive channel dredging in lower Easkoot Creek downstream of State Highway 1 occurred as recently as the mid-1980's. Since that time, regulatory permitting requirements increased both the cost and mitigation requirements for in-stream channel dredging. The National Park Service (NPS) implemented a habitat restoration project in Easkoot Creek between Calle del Mar and the downstream boundary of the Golden Gate National Recreation Area (GGNRA) property in 2004 that redistributed sediment and probably increased channel conveyance capacity. Sedimentation resulting from storms in December 2005 deposited about two feet of sediment and significantly reduced channel conveyance capacity. Rates of aggradation (net sediment deposition in stream channels causing an increase in bed elevation) must be considered to assess the feasibility of dredging and/or installation of sedimentation basins to mitigate flood hazards.

This memorandum documents our analysis of sediment transport rates in Easkoot Creek. This analysis develops critical data necessary to answer the following questions which are addressed both in this document and in the subsequent assessment of watershed sediment production and geomorphology:

1. What is the range and sizes of sediment inflows into Lower Easkoot Creek? (discussed here and in Geomorphic and Watershed Sediment Assessment)

2. Is the existing creek channel able to transport the imposed bed load without excessive aggradation or erosion? i.e. is the creek channel is any state of equilibrium with the imposed water and sediment load? And is Easkoot creek in a geomorphic setting that can ever allow an equilibrium channel to develop? (discussed here and in Geomorphic and Watershed Sediment Assessment)

3. What is the likely rate of sediment build-up in Lower Easkoot Creek under current conditions? (discussed here and in Geomorphic and Watershed Sediment Assessment)

4. What is the impact of channel dredging alternatives on sediment conveyance? (discussed in this document)

5. How sustainable is channel dredging (i.e. how long until the channel requires re-dredging to meet flood control objectives?) (addressed in Geomorphic and Watershed Sediment Assessment)

6. Finally, what is the likely impact of sea level rise on the ability of the creek channel to convey sediment in the future? (qualitative assessment in conclusion of Geomorphic and Watershed Sediment Assessment)

The first element of this analysis estimates sedimentation rates in Easkoot Creek based on data from successive channel profile surveys and from past dredging projects for which records are available. The second element of the analysis estimates bed load sediment transport for the flood event that occurred on December 31, 2005 (estimated to be an 8-yr recurrence interval flow event) using the hydraulic simulation model to estimate bed shear stress (i.e. the force of the flowing water on the bed of the channel) as a function of stream discharge at representative locations. The third element of this analysis is a comparative evaluation of the two preceding analyses, culminating in a summary of likely sedimentation rates for consideration of flood mitigation alternatives. These sedimentation rates are also evaluated in a subsequent assessment of watershed erosion and sedimentation processes.

Easkoot Creek Topographic Profiles and Dredging History

Topographic survey data describing the elevation of the streambed of Easkoot Creek were available from three sources: the District, National Park Service (NPS) and OEI. These data are compiled in Figure 1. District data include the channel profile published in the 1979 FEMA Flood Insurance Study[19] and a 2007 profile surveyed by the District. These two profiles extend from the vicinity of Calle del Arroyo (CA) at the downstream end to the State Highway 1 Bridge (SH1) at the upstream end. NPS profiles from 1999, 2004 and 2006 cover a shorter length of Easkoot Creek extending from the NPS property boundary upstream of Calle del Pinos (CPI) to the Parkside Footbridge (PF). The 1999 NPS survey extends farther upstream to a point near the Arenal Avenue Bridge (AR). The most recent channel profile was surveyed by OEI for this study in December 2011. The source data were referenced to a common datum (NAVD 88), and were fit longitudinally by reference to bridges identified in individual data sets. This process provided reasonable confidence in the accuracy and comparability of the profiles. No quantitative assessment of the vertical accuracy of bed profiles in Figure 1 was developed; however, we believe the accuracy is on the order of about 0.1 feet. Inaccuracy in calculations of net aggradation or degradation would also be influenced by the longitudinal variability (i.e. the level of detail) of thalweg elevations within successive surveys. This potential inaccuracy is not likely to be large enough to alter the interpretation of the data.

[19] The FEMA FIS was originally published in 1979, but does not reference the year in which the topographic survey data were obtained. The data are shown in plates 23P, 24P and 25P of the FIS.

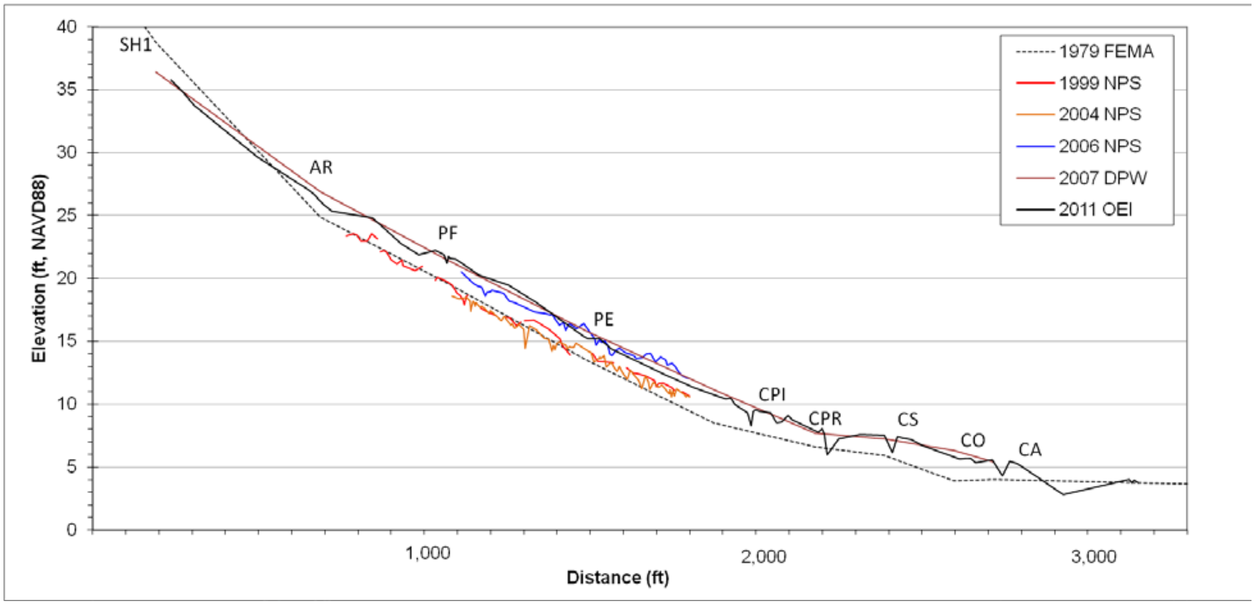


Figure F1 Easkoot Creek channel profiles, 1979-2011.

Comparison of the channel profiles in Figure 1, combined with data on dredging summarized in Table 1, suggest several inferences regarding sedimentation processes. First, comparison of the 1979 and 1999/2004 profiles shows that there was little net change in bed elevation over that time period. Dredging in 1982 (conducted on an emergency basis, estimated in this analysis to be not less than 1,000 cubic yards), 1987 (permitted for removal of about 1,500 cubic yards), and 1997 (permitted removal up to 500 yards) accounts for minimal net change observed for the period 1979-2004.

Second, comparison of the 2004 and 2006 profiles shows up to two feet of aggradation in the NPS reach resulting from the flood event December 31, 2005. The 2007 profile indicates aggradation of two to three feet throughout the length of Easkoot Creek between Arenal Avenue (AR) and Calle del Arroyo (CA) relative to 1979. The lower reach of Easkoot Creek through "the Calles" (beginning at Calle del Pinos, CPI) may have become aggraded prior to 2005, but no survey data are available to confirm this. Based on the profiles for 1999 and 2004 in Figure 1, it appears that substantial aggradation would have occurred throughout the Calles prior to 2005. The channel grade line for 2004 in Figure 1 extending downstream from the Park Entrance Bridge (PE) appears likely to blend at grade with the 2007 and 2011 profiles near CPI. Sediment transport simulations presented in this analysis suggest that the reach between the Park Entrance Bridge (PE) and Calle del Pinos (CPI) is prone to deposition under existing channel conditions, consistent with the preceding observation. The sediment transport simulations also suggest that the existing channel between Calle del Mar (PF in Figure 1) and the Park Entrance Bridge (PE) is competent to transport most of the sediment load supplied from upstream, as is the Calles reach downstream of Calle del Pradero (CPR in Figure 1). This does not indicate whether or not the Calles reach aggraded significantly prior to the 2005 event that caused significant upstream sedimentation.

Third, the 2011 profile shows that there has been little or no net change in channel elevation between 2007 and 2011, despite a substantial peak flow event in 2008 (about 140 cubic feet per second (cfs)). Sediment transport rates were not simulated for the 2008 event, but had it been, substantial bed load transport capacity would have been available. This suggests that relatively low sediment yield from the upper watershed and/or sediment removal (Table 1) at the bridge crossings and at the easement located downstream of Arenal Avenue resulted in low aggradation rates observed between 2007 and 2011. In the following assessment report, substantial evidence suggests that watershed sediment yield is significantly increased during runoff events with recurrence intervals greater than about ten years. This suggests that for smaller runoff events (e.g. 2008), the existing dredging program is sufficient to prevent significant bed aggradation.

The 2011 profile also clearly shows the location and depth of sediment removal by the District at bridges in 2007, 2008 and 2009 (Figure 1). It is apparent that sediment transport rates have been relatively low since that time as and sediment has not filled most of pools created by spot dredging at bridges along the Calles by 2011. This indicates that the modest rates of dredging that occurred beginning in 2007 are substantially effective at preventing significant short-term aggradation, with the important qualification that peak flows from storms in the period 2006-2011 have not been greater than about 140 cubic feet per second (cfs) (annual recurrence interval of four years) and associated sediment transport from the upper watershed appears to have been relatively low. Sediment transport of the magnitude that occurred during the December 31, 2005 storm event (175 cubic feet per second (cfs), annual recurrence interval of eight years) could be expected to be significantly greater than the volume of sediment dredged from the channel during the period 2007-2009. These qualitative observations are indicative of the spatial and temporal variation of sedimentation in Easkoot Creek. The following quantitative analysis of the channel profiles (Figure 1) and dredging data (Table 1) provides estimates of channel aggradation rates.

Estimation of Sedimentation Rates from Project Reach Survey and Dredge Records

Available data pertaining to changes in channel profile over time (Figure 1) and dredging rates over time (Tables 1 and 2) make possible quantitative estimates of sedimentation rates in lower Easkoot Creek. A subsequent analysis (Tasks 3D and 3E Geomorphic and Watershed Sediment Assessment) evaluates sediment loads delivered to lower Easkoot Creek from the upstream watershed. Watershed sediment loads are compared in the subsequent analysis with the estimates of sedimentation and sediment transport presented here.

The portion of the creek for which a sedimentation estimate has been developed in this analysis extends from just above the Arenal Avenue Bridge (AR, Figure 1) to the Calle del Arroyo Bridge (CA, Figure 1). This 2,000 feet long reach coincides with the areas where Easkoot Creek flooding is most common, and the history of this reach of the creek is relatively well-known since about 1979.

In this analysis we first estimate the change in bed elevation based on channel profile data, and then incorporate the volume of sediment removed from the channel by dredging operations to arrive at an estimated total volume of sediment deposition for the period 1979 to 2011.

Table F1 Historic dredging in Easkoot Creek as summarized by the District.

Prior to 1959	Pre-establishment of zone - periodic dredging performed by County DPW.
1965	No major evidence of sediment removal activity; however, fill permit issued by California Coastal Commission for spoils site at nearby lot. Extended to 1970.
1967	Funds in the amount of $744 transferred to zone to cover cost of maintenance.
1973	Emergency basis performed with right of entry forms. Marin County declared disaster area by both state and federal governments. Proposed to remove 4,000 cubic yards at a cost of $17,000. Zone relies on reimbursement to cover costs of sediment removal. Request for authorization made to California Coastal Commission.
1977	No evidence other than in staff (John Wooley) summary.
1982	Likely a large removal of sediment on an emergency basis
1983	Silt basin pools created on both sides of Arenal crossing. No evidence of any additional dredge work.
1986	Evidence of acquired easements from Community Center to foot bridge. Evidence of Fish & Game permit for dredging activities in 1986 in document dated 11-5-86. Activity likely minor, however, as indicated by large effort in 1987.
1987	Plan to remove estimated 1,500 cubic yards. Fish & Game permit secured. California Coastal Commission permit applied for.
1997	Letters to regulatory agencies states time of last dredging to be 1987. Unlike in 1987, dredging equipment was not to be placed in creek. Less than 500 yards of material was to be removed. Sediment removed at a cost of $24,680.00. Fish & Game permit issued, determination from California Coastal Commission and Corps supports that permits are not necessary for their agencies.
2006	Dredging performed at crossings but not easements. Emergency proclamation with CEQA exemption. Fish & Game permit acquired based on emergency proclamation.
2007	Dredging done at four crossings (Pinos, Pradero, Onda, Arroyo) but not at County easements in channel near Arenal Avenue. CEQA exemption. Fish & Game permit acquired based on emergency proclamation (previous year's permit had not expired). Applied for multi-year permit to cover 2007 and onward. F&G requested that there be evidence of positive impact dredging was having in the long-term and evidence of the County's commitment to a long-term solution other than dredging. Total of 100 cubic yards of sediment removed from four crossings.
2008	Dredging performed at three crossings (Arenal, Pinos, and Pradero) and the channel easement downstream of Arenal Ave. Previous CEQA Initial Study with Negative Declaration. New multi-year (through 2012) F&G permit issued allowing work to be conducted at low tide with minimal water present. Total of 160 cubic yards of sediment removed from three crossings. Continued monitoring might be necessary.
2009	35 cubic yards of materials removed from Arenal Rd. crossing and easement only (20 feet upstream and 100 feet downstream). Volume estimate based on material hauled by Roads crew. Water was present at Pinos and Pradero and no dredging was performed due to permit constraint.
2010	No removal due to minimal sediment accumulation.

The channel profiles from 1979, 2007 and 2011 provide the primary basis for estimating the volume of sediment deposits in the Easkoot Creek channel. In addition, profiles of the NPS reach from 1999, 2004, 2006 and 2011 can be analyzed to assess sedimentation in that area. To quantify changes in channel elevation, we used a spreadsheet to determine the area under the curves formed by the channel profiles in Figure 1, and determined the differences between successive profiles. This produced an area (units of square feet), which was then divided by the length of the reach to determine the mean change in bed elevation. The mean elevation change was then multiplied by the mean active bed width[20] (10 feet) to estimate the volume of sediment deposition (or erosion). Sediment volumes associated with changes in bed elevation are converted to units of cubic yards (cy).

The history of dredging in Easkoot Creek is described in Table 1. Quantities of sediment removed from the channel are summarized in Table 2, along with comments regarding the use of dredging data from particular years. The District has compiled detailed data for its dredging operations for the period 2007 to 2010, as shown in Table 2. Sediment removed from the channel by dredging is added to the change in bed elevation computed from channel profiles to yield the estimate of the channel sedimentation rate.

As noted above, channel profile data from the creek on NPS property (Fig. 1, 1999, 2004, 2006 and 2011) provides a means to estimate sedimentation in this sub-reach. Table 3 summarizes the analysis of sedimentation based on channel profile data where 1999 data serve as the baseline, and profiles from 2004, 2006 and 2011[21] are compared to the 1999 data. This analysis found that the aquatic habitat restoration work on Easkoot in late 2004 lowered the mean bed elevation by 0.12 feet, which can be equated to removal of 165 yards of sediment. This value is added to channel dredging totals entered in Table 2. The 2005 event deposited about 433 yards (165 yards + 268 yards).

The data and calculations used to estimate net deposition from Arenal Avenue to Calle del Arroyo for the period 1979 to 2011 are summarized in Table 4. Net bed elevation change is about 2 feet and mean annual bed elevation change is 0.06 to 0.07 ft. The change in bed elevation accounts for about 40% of the total estimated deposition; 60% of the total estimated deposition was dredged from the channel. As was noted above, it is likely that substantial dredging occurred during and after the winter of 1982/1983 that was not quantified. If it is assumed that 1,000 yards of sediment was removed by emergency dredging in 1983, then the total deposition rate would range from about 150 to 160 yards/yr.

Note that average annual deposition rate from this analysis is subject to high variability. Our analysis indicates that most years have a low sedimentation rate with decadal spikes during large storm events (e.g. 1982/3 and 2005/6).

[20] Mean channel width was computed using the bottom width from 62 channel cross-sections surveyed in 2011.
[21] 2011 data are used because they are relatively detailed; 2007 data are less detailed in this reach.

Table F2 Summary of quantities of sediment removed from Easkoot Creek by dredging 1973-2011.

<table>
<tr><th>Year</th><th colspan="14">Description of Dredging</th><th>Total (cubic yards)</th></tr>
<tr><td>1973</td><td colspan="14">Records suggest removal of 4,000 yards of sediment was planned following County disaster area declaration.</td><td>4,000(a)</td></tr>
<tr><td>1982</td><td colspan="14">Records indicate sediment removal conducted on emergency basis; no data on quantity.</td><td>Na</td></tr>
<tr><td>1983</td><td colspan="14">Dredging upstream and downstream of Arenal Road, but no data on quantity.</td><td>Na</td></tr>
<tr><td>1986</td><td colspan="14">Possible limited dredging, Community Center to Calle del Mar footbridge at Parkside; no data on quantity.</td><td>Na</td></tr>
<tr><td>1987</td><td colspan="14">Quantity specified in permits, but locations not specified in summary.</td><td>1500</td></tr>
<tr><td>1997</td><td colspan="14">Quantity specified in permits, but locations not specified in summary.</td><td>500</td></tr>
<tr><td>2004</td><td colspan="14">NPS habitat restoration project modified the channel bed and banks; net change in channel elevation determined by difference between 1999 and 2004 profiles (Fig. 1).</td><td>165(b)</td></tr>
<tr><td>2006</td><td colspan="14">Crossings dredged, but no quantity specified in summary.</td><td>Na</td></tr>
<tr><td rowspan="2"></td><th colspan="2">Arenal Rd.</th><th colspan="2">Calle del Mar</th><th colspan="2">Calle del Pinos</th><th colspan="2">Calle del Pradero</th><th colspan="2">Calle del Sierra</th><th colspan="2">Calle del Onda</th><th colspan="2">Calle del Arroyo</th><td rowspan="2"></td></tr>
<tr><th>US(c)</th><th>DS</th><th>US</th><th>DS</th><th>US</th><th>DS</th><th>US</th><th>DS</th><th>US</th><th>DS</th><th>US</th><th>DS</th><th>US</th><th>DS</th></tr>
<tr><td>2007</td><td>0</td><td>0</td><td>0</td><td>0</td><td colspan="2">37</td><td colspan="2">26</td><td>0</td><td>0</td><td colspan="2">26</td><td>0</td><td>11</td><td>100</td></tr>
<tr><td>2008</td><td>0</td><td>55</td><td>0</td><td>1</td><td>35</td><td>17</td><td>8</td><td>45</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>161</td></tr>
<tr><td>2009</td><td colspan="2">35</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>35</td></tr>
<tr><td>2010</td><td colspan="14">No dredging.</td><td>0</td></tr>
<tr><td>2011</td><td colspan="14">No dredging.</td><td>0</td></tr>
<tr><td></td><th colspan="14">Total known dredging after 1979</th><th>2,461</th></tr>
</table>

Notes: a) This quantity excluded from sedimentation rate analysis total because it is prior to the 1979 date assigned to the FEMA channel profile.
b) This quantity was determined from channel profile data for the NPS reach (see Table 3).
c) US = upstream side of bridge DS = downstream side of bridge

Table F3 Bed elevation changes in NPS reach and estimated sediment deposition rates since 1999; the reach length is 750 ft.

	2004	2006	2011
Change in area under channel ‘curve’ relative to 1999 (sq-ft)	-444	723	1045
Average change in bed elevation (ft)	-0.6	1.0	1.4
Rate of elevation change (ft/yr)	-0.12	0.14	0.12
Volume deposited (yds)	-165	268	387
Average annual Deposition rate (yds/yr)	**-6**	**8**	**12**

Table F4 Bed elevation changes in Easkoot Creek, Arenal Avenue to Calle del Arroyo and estimated sediment deposition rates since 1979; the reach length is 2,030 ft.

	2007	2011
Change in area under channel 'curve' relative to 1979 (sq-ft)	3979	4250
Average change in bed elevation (ft)	1.96	2.09
Rate of elevation change (ft/yr)	0.07	0.06
Volume deposited (yds)	1474	1574
Sediment removal (yds)	2165	2461
Deposition + sed. removed (yds)	3639	4035
Total Annual Average deposition rate (yds/yr)	**125**	**122**

Summary of Sedimentation Rate Analysis

While there is considerable uncertainty in the foregoing calculations, the estimate for annual average sedimentation rate is objective and quantitative. The comprehensive profile data over a relatively short reach of stream reduces the potential for gross errors in estimation that could arise from assumptions used to replace missing data. Sediment removal (dredging) volumes are likely to be quite accurate after 2007. Earlier data are derived from permitting records, so can be expected to be reasonably accurate estimates, but are not supported by data pertaining to actual sediment removal (e.g. number of dump truck loads). Overall, the estimated sedimentation rates should be considered reliable enough to estimate the magnitude of mean annual sedimentation rates in the future. The annual average deposition rate may be a poor representation of actual deposition likely to occur in any given year, and plans for management of sedimentation should acknowledge the likelihood of large deposition events at intervals of approximately ten years.

The mean annual sedimentation rate estimated for the period 1979-2011 is 122 yds/yr, and may be as much as 160 yds/yr if likely dredging in 1982/3 is incorporated in the estimate. Recent dredging by the District from 2007-2009 averaged about 100 yds/yr, and sediment deposition through 2011 was insufficient to fill pools created by dredging at bridges along the Calles. Provided that peak flows are modest (approximately less than five year recurrence interval) and watershed erosion rates are not accelerated by large-scale mass wasting, it appears that the existing dredging program, although limited, can remove sediment volumes approximately equal to the average sedimentation rate. It is also evident that during larger storm events (e.g. 1973, 1982/1983, 1997, 2005) that produce higher peak flows and/or accelerated watershed erosion rates, much greater sediment deposition in a single year (or storm event) may be expected. Sedimentation resulting from the 2005 event, which is relatively well-documented by channel profile data, was probably about 1,000 yds or more[22], several times the mean annual deposition rate and the sediment deposition capacity created by the current dredging program.

The foregoing clearly indicates that persistent channel sedimentation has the potential to contribute significantly to flood hazards in Stinson Beach. Although channel dredging would likely reduce flood

[22] From Arenal Ave. to Calle del Arroyo, channel length is 2,030 ft, width is 10 ft and deposition depth was about 2 ft through this reach relative to 1979, yielding 40,600 cubic feet of sediment, equivalent to 1,500 cubic yards; if it is assumed that aggradation had already filled "the Calles" reach to the 2007 elevation, then the 2005 event probably deposited about 1,000 yards.

hazards, the benefit is not likely to persist for more than ten years: large single storm events can deposit at least 2 ft of sediment (or more) in lower Easkoot Creek. Although efforts to maintain channel conveyance capacity through a dredging program are likely to be beneficial with respect to reduced flood hazard, dredging at the current rate probably cannot be expected to significantly reduce flood hazards associated with low-recurrence, high magnitude floods that transport much larger sediment loads. Development of sedimentation facilities, particularly upstream of Highway 1, and continued or expanded dredging in lower Easkoot Creek between Arenal Avenue and Calle del Arroyo, would probably be necessary to reduce sedimentation impacts. Unless large and efficient sedimentation basins can be developed, however, it is likely that sedimentation will continue to be a significant factor contributing to flooding along Easkoot Creek.

The sedimentation rates estimated above, particularly for the December 31, 2005 storm event, provides data that can be used to validate estimates of sediment transport derived from generalized sediment transport equations. The hydrologic and hydraulic models developed for this project provide a simulation of stream flow and hydraulic parameters of the flow at different locations along the channel for the December 31, 2005 flood event. In the following section, we describe our test of two bed load sediment transport equations for potential application in predicting future sedimentation as a function of stream discharge and corresponding simulated hydraulic conditions.

Motivation for Estimating Sediment Transport Rates

Future management of the stream to mitigate flood hazards and/or improve fish habitat may include channel dredging and installation of new sedimentation facilities, and may also include diversion of peak flow. The performance and effects of potential sediment and flow management can be better evaluated if a model relating sediment transport to stream flow at critical locations in Easkoot Creek can be developed to assess local changes in sediment transport/deposition resulting from proposed flood mitigation strategies.

Sediment transport processes in gravel bed streams are complex and have been the subject of numerous field studies and laboratory experiments that have produced a large body of scientific theory. Although considerable progress has been made, no single approach to analyzing and predicting sediment transport has emerged and significant uncertainty remains in any particular estimate of sediment transport rate. The chief factors controlling sediment transport rates in stream channels are the tractive force (bed shear stress) of stream flow on the stream bed (e.g. flow velocity, flow depth, channel slope), and characteristics of the sediment on the bed (e.g. sediment supply on the bed locally and from upstream sources, sediment size, sediment size distribution). Professional judgment must be exercised in selection of appropriate methods, including approaches that provide empirical data to evaluate models or formulas used to predict sediment transport rates. Estimates of sediment transport rates are within 50 to 100% of measured rates may be considered “good” estimates; estimates for a particular site using different equations can vary by a factor of ten[23]. Consequently, the empirical sedimentation data described in the preceding sections should be considered to be relatively reliable, while estimates derived from transport equations developed in this section might be considered with greater caution.

[23] Gomez, B. and M. Church (1989). "An assessment of bed load sediment transport formulae for gravel bed rivers." Water Resources Research **25**(6): 1161-1186.

Estimates of sedimentation rates in Easkoot Creek developed above predict the likely magnitude of potential channel bed aggradation for average conditions and in response to large storm events. However, these estimates are representative of the entire lower reach of Easkoot Creek between Arenal Avenue and Calle del Arroyo under existing channel conditions. The preceding analysis of stream profiles suggests differential rates of deposition in lower Easkoot Creek. Deposition rates estimated in the NPS reach since 2004 (0.12 to 0.14 ft/yr, Table 3) are about double the long-term average for the entire reach since 1979 (0.06 to 0.07 ft/yr, Table 4). This difference might be attributable to variation in sedimentation rates over time, perhaps attributable to differences in sediment supply from the upper watershed. Nevertheless, differential deposition rates of sediment would not be unexpected because the channel slope declines significantly throughout the length of the reach, which typically indicates a decline in sediment transport capacity. Changes in sediment size often correspond to changes in transport capacity, and as shown in Figure 2, sediment size on the channel bed also declines as the channel slope declines (Figure 2).

Analysis of transport and deposition processes as a function of location and stream flow is intended to provide better predictions of local sedimentation. The following analysis develops a sediment transport model based on 2011 channel conditions and simulated flow and hydraulic conditions over a 3-day period representing the flood event of December 31, 2005. This model of sediment transport can be used to estimate rates and locations of sedimentation, and will be used to help evaluate selected flood mitigation alternatives.

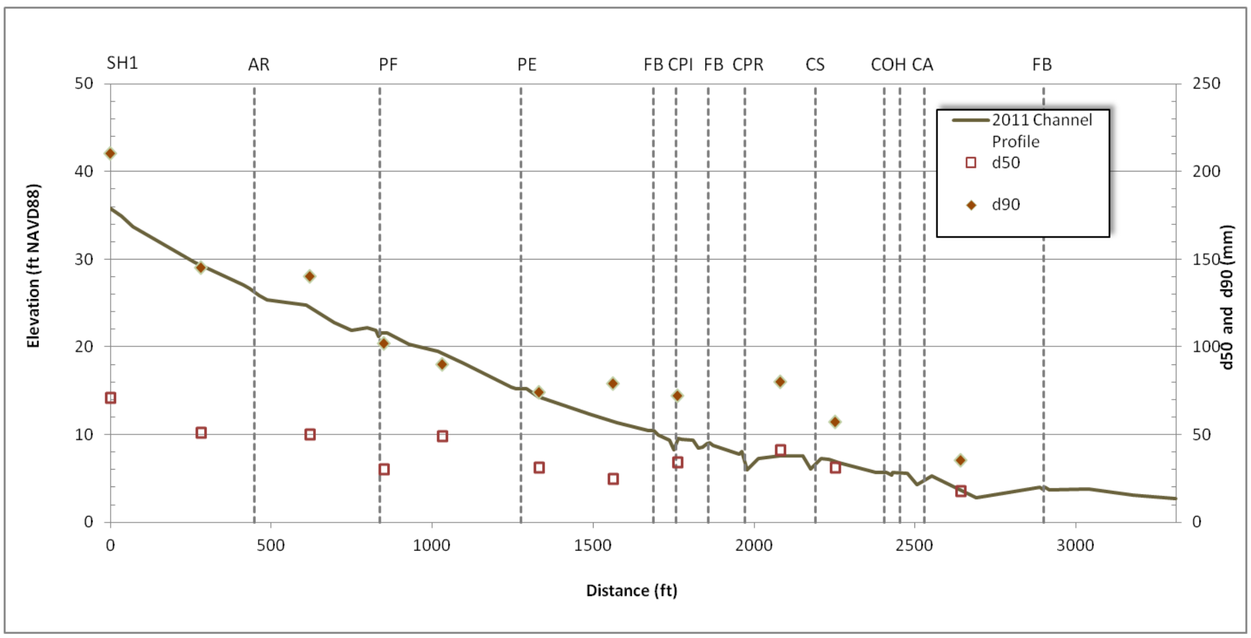


Figure F2 Surface sediment size and channel profile, December 2011.

Sediment d50 is the median size and d90 is the 90th percentile of the surface size distribution (90% of sediment particles are finer than the d90).

Sediment Transport Processes

Transport of sediment in most streams occurs only during periods of peak stream flow and occurs in Easkoot Creek during winter rainstorms. Sediment is transported either as suspended load or bed load. Smaller diameter sediment (typically fine sand, silt and clay) becomes suspended in turbulent flow, remains suspended in the water column above the stream bed, and moves at approximately the same velocity as the flow. Larger diameter sediment (coarse sand and gravel) skips or rolls along the stream bed (hence the term “bed load”) and moves at a fraction of the velocity of the flow. Higher flows that typically occur only a few times each year are required to entrain bed load. The transition between transport as suspended load and bed load occurs as stream flow changes through the course of a storm and as flow conditions change through the watershed. While suspended sediment typically accounts for the majority of sediment transport in coastal California watersheds, most of the suspended load is transported out of the gravel bed channel and deposited in on stream banks, floodplains, lakes, estuaries and/or the ocean. The bed load sediment moves slowly, and represents most of the material deposited on the channel bed; it is the bed load sediment that may cause the bed to aggrade. Consequently, for this analysis we focus on bed load transport processes and the equations used to predict bed load transport as a function of stream flow/hydraulic parameters.

Bed load sediment transport rates are sensitive to many factors including the diameter of sediment available for transport on the streambed. Generally, when the diameter of sediment is smaller the transport rate is higher. In addition, the supply of sediment from upstream sources is expected to have a strong influence on the diameter and availability of sediment on the stream bed. When sediment supplies from upstream are cut off (e.g. by an upstream reservoir), sediment size on the channel bed increases as the flow strips transportable sizes from the bed. Similarly, the size of sediment in a stream channel decreases when a large supply is available from upstream sources (e.g. in response to landslides) that cannot be quickly removed by the flow.

Modeling Sediment Transport

As discussed above, sediment transport processes are complex, and efforts to model and predict sediment transport are equally complex. In general, predicting sediment transport with reasonable accuracy requires a record of stream flow over a range of flows that includes periods of significant sediment transport, good estimates of stream hydraulics (i.e. the relationship between stream discharge and stream depth and velocity), data on the size distribution of sediment on the stream bed, and an appropriate sediment transport equation that can be expected to perform well chosen from among many available. Data and analyses discussed in the previous section provide a means to assess the appropriateness of the chosen bed load transport equation.

The necessary data are available for Easkoot Creek to develop reasonably accurate sediment transport predictions. First, the stream flow records developed and maintained by the NPS for its gauge station just downstream from the Park Entrance Bridge (PE, Figure 2) provide a continuous flow record[24], including the peak flow event centered on December 31, 2005 during which significant sediment transport occurred as documented above. Second, the hydraulic model used to simulate stream flow and floodplain flow, including flows returning from the floodplain to the channel, provides estimates of flow parameters needed as inputs to sediment transport equations (i.e. shear stress of the flow acting on the stream bed) at locations of interest continuously through the stream flow event. The hydraulic model provides estimates of flow parameters at locations where cross-sections were surveyed; there are 62 cross-sections incorporated in the hydraulic model. Third, characteristics of the stream bed sediment were obtained during surveys of Easkoot Creek in 2011 (described below).

[24] This stream gauging record was used to develop and calibrate the hydrologic and hydraulic models described in Task 3 technical memoranda; the gauge record is described in the Task 1 technical memorandum.

A relatively simple bed load transport equation was selected from a family of much-studied equations[25] based on a power function of bed shear stress in excess of the critical bed shear stress at the threshold of significant bed load transport. The family of equations takes the form

$$q_{s*} = c\,(\tau_* - \tau_{*cr})^{1.5}$$

where q_{s*} is the dimensionless instantaneous bed load transport rate, c is a constant, τ_* is dimensionless bed shear stress and τ_{*cr} is dimensionless critical bed shear stress[26]. Experiments have generally determined values of c and τ_{*cr} applicable under specified conditions of excess shear stress and the degree of sorting (uniformity) of sediment on the channel bed.

Sediment on the streambed of Easkoot Creek is relatively well sorted but not uniform, hence τ_{*cr} is set equal to 0.047, near the center of the range of commonly cited values (0.03 for poorly sorted sediment and 0.06 for uniform sediment). The reader should note that although τ_{*cr} may be considered constant; the critical shear stress is proportional to sediment size, such that the threshold of transport is greater for larger diameter sediment.

The value of the constant c has been found to vary systematically with the ratio τ_*/τ_{*cr} (referred to as "transport stage", or T_*). Values of T_* estimated from the maxima of bed shear stress (simulated in the hydraulic model) at representative locations in Easkoot Creek during the simulated December 2005 peak flow event range as high as 4, but are typically about 2. An appropriate value of c is 5.7 for T_* in this relatively low range[27]. Although laboratory experiments have tested these equations with T_* much greater than 2, hydraulic conditions where T_* exceeds 2 are uncommon in natural channels[28].

The remaining decisions regarding input to the Easkoot Creek bed load transport model regards the sediment size used at each of the eleven approximately evenly-spaced cross sections where bed load transport was computed. The median diameter (d50) of sediment size distributions is typically used to represent the behavior of mixtures of sediment in bed load transport equations.

Sediment Size Distributions

Sediment sizes in Easkoot Creek were sampled in December 2011, about 6 years after the large flow event that caused significant sedimentation as described above. Surface sediment size distributions were measured by systematic point counts of 100 sediment grains on the stream bed at locations shown in Figure 2. Values of d50 and d90 of the cumulative size distribution are shown at the sample locations. The median sediment diameter (d50) ranged from about 20 to 50 mm (0.8 to 2 inches), but was coarser at SH1 (d50 of about 70 mm).

The sediment sizes in Easkoot Creek in 2011 are believed to be representative of relatively low sediment supply; large inputs of sediment to lower Easkoot Creek have not occurred since Dec. 2005 and the

[25] Meyer-Peter, E. and Müller, R. (1948). "Formulas for bed-load transport." Proc. 2nd Meeting IAHR, Stockholm, 39-64.
[26] Richards, K. (1982). Rivers, Form and Process in Alluvial Channels. Methuen & Co., New York, pp. 113 and Julien, P. (2010). Erosion and Sedimentation. Cambridge University Press, Cambridge, UK. Pp. 197.
[27] Fernandez Luque, R., and Van Beek, R. (1976). "Erosion and transport of bed-load sediment." J. of Hydraulic Res. **14**(2), 127-144.
[28] Pers. comm., Professor J.D. Smith, Univ. of Washington, 1991.

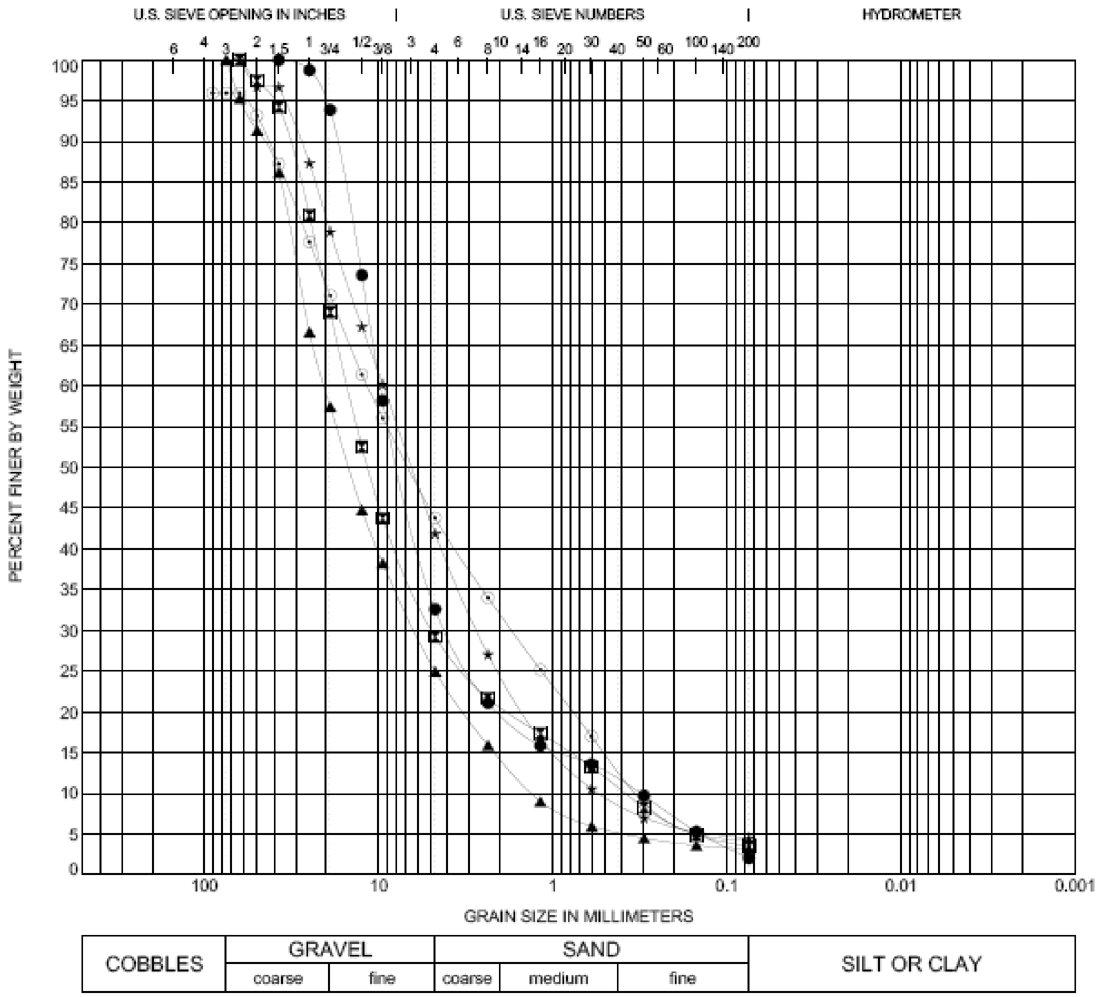
channel bed has been extensively worked by smaller magnitude flows that would likely have mobilized much of the readily-transportable sediment leaving a relatively coarsened bed surface. The size distribution of the bed surface, specifically the median diameter (d50), can be used in bed load transport equations to represent the bed load sediment available for transport.

In addition, five bulk samples of the bed, gravel bars and other deposits averaging about 18 kg dry weights were collected. A shovel was used to excavate sediment to a depth of about 20 cm (8 inches) at selected locations where a sample could be excavated without encroaching on the wetted channel. Sediment was placed in a 5 gallon bucket and transported to a geotechnical laboratory where the particle size distribution was determined.[29] The bulk sample data is summarized in Figure 3. The median diameter (d50) of these samples ranged from about 6 mm to 16 mm. With consideration of the deposit from which the samples are collected, the size distribution of these samples may also be used to represent conditions of higher bed load sediment availability (i.e. high sediment supply).

Two of the bulk sediment samples (ESK-2 and ESK-3, Figure 3) were collected from gravel bars deposited near Calle del Pradero and Calle del Pinos, respectively. These deposits were in locations that had been dredged in 2008, establishing that these samples represent bed load that has been recently transported and deposited. The average d50 of these two samples is about 14 mm. Sample ESK-5 was collected from a gravel bar deposited under the SH1 Bridge; the height of the bar relative to the existing bed suggested that this deposit is a remnant of the December 2005 flow event. The d50 of ESK-5 was about 6 mm, and may be considered representative of the bed load size during a large sedimentation event when relatively large volumes of bed load are available and transported. Sample ESK-4 was collected from a small, relatively fine-textured deposit between the Calle del Mar footbridge (PF in Figure 2) and the Park Entrance Bridge (PE). This bar appears to have been formed in relation to a local channel obstruction such as a small woody debris jam, and appeared to have been deposited during 2010. Sample ESK-1 was collected from a dry portion of the channel bed downstream of Calle del Arroyo (CA in Figure 2) and represents the size distribution of bed load that reaches the edge of the Bolinas Lagoon from Easkoot Creek. This sample is the most well-sorted of this group of samples, and has very few particles with a diameter > 20 mm. This suggests an upper limit on the diameter of sediment that is typically transported through Easkoot Creek to Bolinas Lagoon.

The sediment transport simulation was initially tested to gain perspective on sensitivity to sediment diameters, beginning with the surface sediment size distributions measured in December 2011. Perspective was gained by comparing sedimentation estimates from the bed load transport simulation with previously observed sedimentation from the December 2005 event. Ultimately, the sediment diameters selected to evaluate sedimentation potential include a range of diameters from large to small representing sediment supply from upstream ranging from low to very high as detailed in Table 5.

[29] Brunsing Associates, Inc., Santa Rosa, CA. ASTM C 117 and C 136.



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● ESK-1	0.0 ft	POORLY GRADED GRAVEL with SAND(GP)				5.33	31.22
⌧ ESK-2	0.0 ft	POORLY GRADED GRAVEL with SAND(GP)				4.21	39.59
▲ ESK-3	0.0 ft	WELL-GRADED GRAVEL with SAND(GW)				1.43	15.74
★ ESK-4	0.0 ft	WELL-GRADED GRAVEL with SAND(GW)				1.45	17.57
⊙ ESK-5	0.0 ft	POORLY GRADED GRAVEL with SAND(GP)				0.77	35.03

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● ESK-1	0.0 ft	37.5	9.812	4.053	0.314	67.4	30.5	2.1	
⌧ ESK-2	0.0 ft	63	15.106	4.927	0.382	70.8	25.7	3.5	
▲ ESK-3	0.0 ft	75	20.51	6.172	1.303	75.0	21.8	3.2	
★ ESK-4	0.0 ft	63	9.423	2.71	0.536	58.1	37.4	4.5	
⊙ ESK-5	0.0 ft	90	11.64	1.723	0.332	52.1	39.6	4.2	

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Appr.:
Date: 01/30/12

GRAIN SIZE DISTRIBUTION
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PLATE

Figure F3 Sediment size distribution of five bulk samples of Easkoot Creek bed material, December 2011.

Simulation Procedure

A spreadsheet was used to compute bed load transport using the Easkoot Creek simulated hydraulic results for eleven selected cross-sections (locations shown in Fig. 2 with d50 and d90 of sediment size distributions). Initially we computed bed load transport at five minute intervals for the 76 hour period beginning at 1200 hours on 12-30-2005 concluding at 1640 hrs. on 1-2-2006; based on those results, the final computations were conducted at five minute intervals for a 24 hour period during 12-31-2005 that was responsible for almost all bed load transport. Computations were made with a dimensionalized form of the bed load equation in SI units (kg-m-s). At each cross-section, instantaneous bed load transport rates were computed in units of m^3/s for each five minute interval, and then summed over the simulated hydrograph. The sediment sizes used for the bed load transport simulation are summarized in Table 5. Total bed load transport for the event was calculated and converted to units of cubic yards for ease of comparison to data from the dredging analysis (Figure 4).

Table F5 Summary of sediment supply and sediment size for bed load transport simulation.

Sediment Supply Condition Represented	Simulated Bed Load Entering Reach at State Highway 1 (cu yds)	Sediment Diameter for Simulation
Low (Large d50)	~360	Surface d50 measured 2011; specific to each simulation station
Moderate	~730	Surface d50 adjusted down ½ size class to simulate modestly higher sediment supply; specific to each simulation station
High	~1550	Uniform d50 = 14 mm at all stations representing relatively high sediment supply
Very High (Small d50)	~1760	Uniform d50 = 6 mm at all stations representing bed load at SH1 during Dec. 2005 event

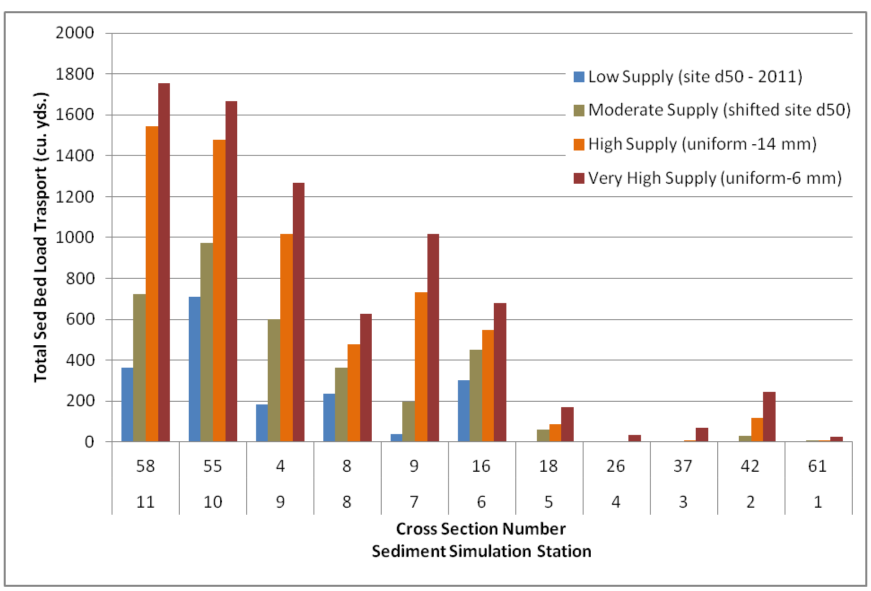

Figure F4 Simulated bed load sediment transport capacity at simulation stations.

(2011 channel conditions, Easkoot Creek)

As can be seen in Figure 4, there is a gradual decline in simulated bed load transport capacity downstream from Station 11 at SH1 to the vicinity of the Park Entrance Bridge (Station 6, Figure 4). The decline in simulated bed load transport capacity is precipitous beginning at Station 5, a short distance upstream of Calle del Pinos. This validates the model in that the bed load transport capacity in Easkoot Creek in the vicinity of "the Calles" is very low in comparison with the upper steeper reaches upstream, and that sedimentation predicted by the model conforms generally to historic observed sedimentation.

To evaluate potential sedimentation in greater detail, a simple mass balance was calculated from upstream to downstream using the data shown in Figure 4 (2011 channel conditions). Total bed load available to the simulated reach was assumed to be limited to the transport capacity at the uppermost section at SH1 (Table 5, Figure 4). At each station, the bed load transport capacity was compared to the capacity at the upstream station. If the upstream station had higher capacity than downstream, then the difference was assumed to be deposited at the downstream station. If the downstream station had higher capacity than the upstream station, then it was assumed that all of the sediment from upstream was transported through to the next station downstream with no deposition or bed erosion. The procedure was repeated in the downstream direction for each station until no excess sediment remained. Different hydraulic simulations were used to evaluate patterns of transport and deposition under existing conditions, proposed dredged conditions (described for Alternative 4- Channel Dredge and Sediment Management, Appendix A), and proposed flood bypass conditions (described for Alternative 6-Wetland Enhancement (near Poison Lake) and Bypass to the National Park Service's South Parking Lot, Appendix A). The resulting patterns of simulated sedimentation are shown in Figures 5, 6 and 7.

The transport and deposition calculated at each station and portrayed in Figures 5, 6 and 7 should be considered to represent sediment dynamics for a reach centered on each station. It should be noted that although this simplified sediment routing scheme provides a quantitative estimate of the likely location and magnitude of sediment deposition, it is an approximation that does not reflect the complex process of channel adjustment (e.g. bed aggradation and scour, shifts in sediment size distribution) that occurs with significant bed load transport over time.

Although the simulations show some potentially important differences in patterns of sediment deposition, the overall pattern of deposition does not change very much. The pattern of sedimentation for 2011 channel conditions (Figure 5) provides perspective on the management of sediment for purposes of reducing flooding potential. First, under any conditions, it appears likely that the majority of sedimentation takes place in the reach beginning upstream of Arenal Avenue (AR) and extending to the Calle del Mar footbridge (PF). The reach below the Park Entrance Bridge (PE) is also prone to sedimentation; however, the magnitude of deposition is diminished owing to deposition upstream reach. Deposition along the Calles is minimal except under conditions of high or very high sediment supply; under simulated conditions, most of the sediment entering at SH1 is deposited before it reaches the Calles. Finally, the bed load transport simulations indicate that all of the bed load sediment entering the reach at SH1 is deposited in lower Easkoot Creek, primarily upstream of the Calles.

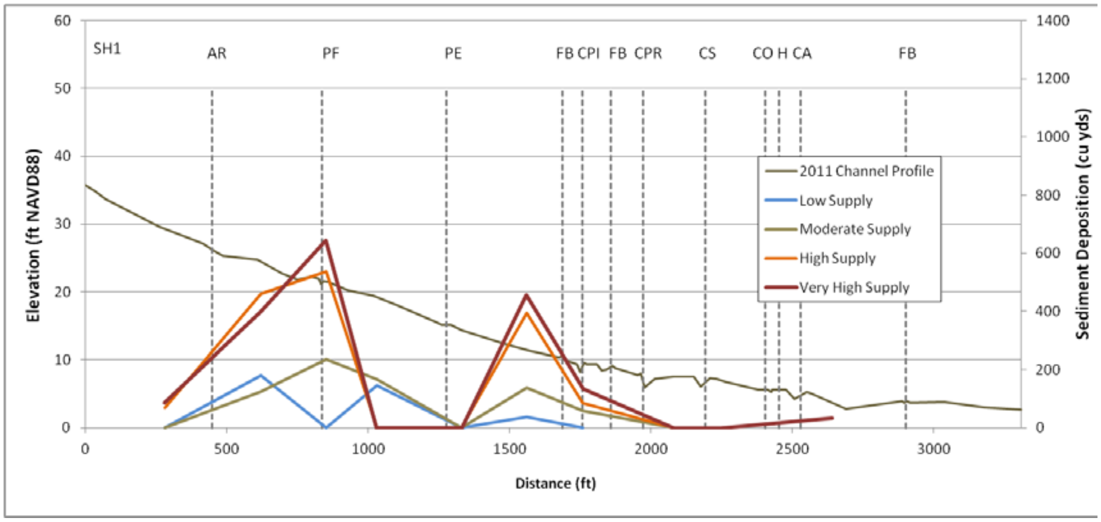

Figure F5 Simulated sedimentation for 2011 channel conditions, Easkoot Creek.

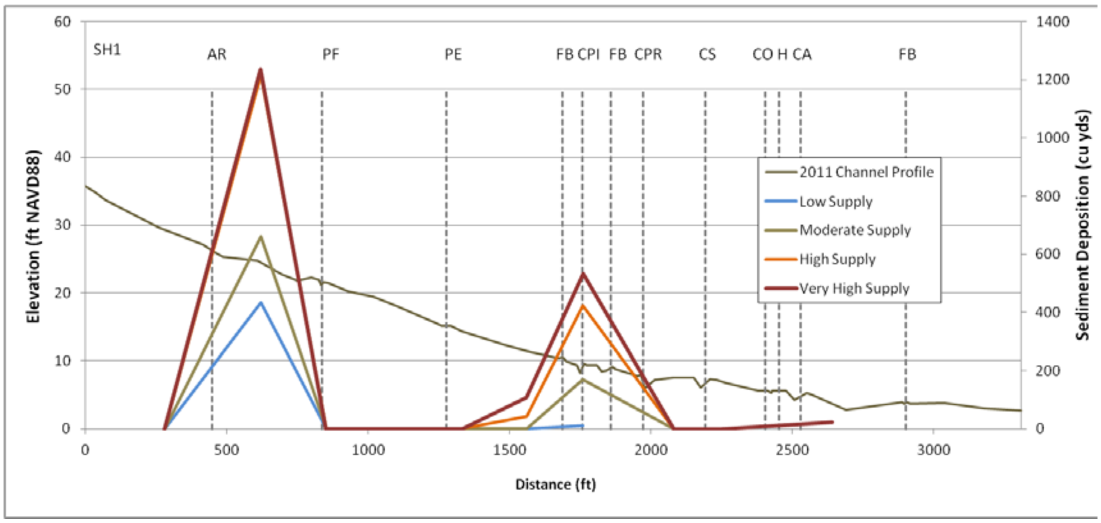

Figure F6 Simulated sedimentation for proposed channel dredge, Easkoot Creek.

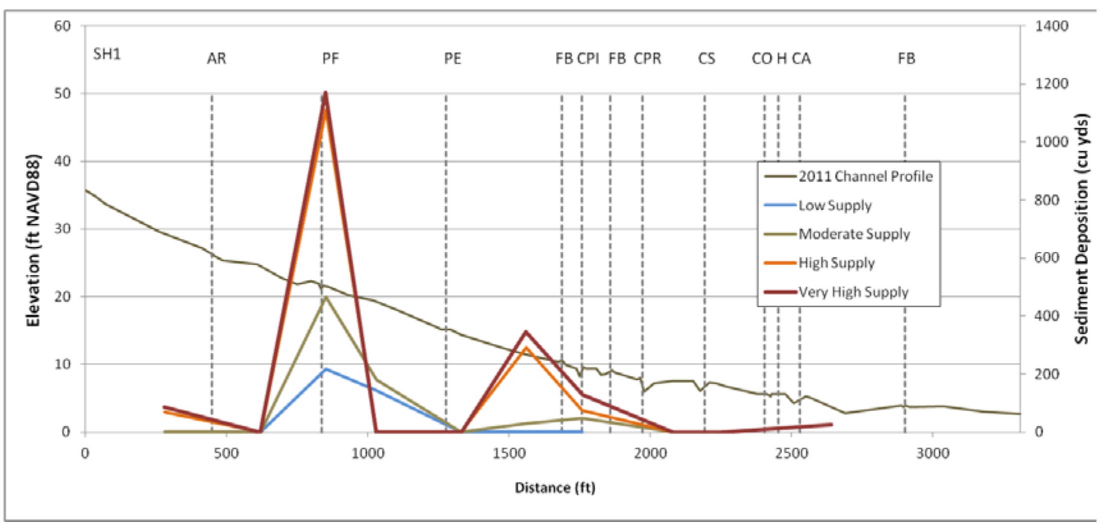

Figure F7 Simulated sedimentation for proposed flood bypass, Easkoot Creek.

Under conditions for the proposed channel dredge alternative, deposition is substantially increased and concentrated in the reach between Arenal Avenue (AR, Figure 6) and Calle del Mar (PF, Figure 6). In addition, sedimentation downstream shifts from a reach centered on GGNRA property below the Park Entrance Bridge (PE, Figure 6) to a reach centered on Calle del Pinos (CPI, Figure 6). Under conditions for the proposed flood bypass, the reach of most concentrated sedimentation shifts to the reach centered on the Calle del Mar footbridge (PF, Figure 7), which is the reach downstream of the bypass channel where sediment transport capacity is abruptly curtailed. The downstream locus of sedimentation under bypass conditions is the reach downstream of the Park Entrance Bridge (PE, Figure 7), similar to that predicted under 2011 channel conditions.

These simulations suggest that efforts to trap and remove sediment in Easkoot Creek are likely to be effective in the reaches upstream of the Park Entrance Bridge where sedimentation is most pronounced. In particular, this includes both the area above and below Arenal Avenue where the District routinely dredges, and the location of the District's proposed sedimentation facility just upstream of the Calle del Mar foot bridge (PF). Whether or not there is sufficient sedimentation capacity (i.e., sufficient space) to capture sediment at these locations is uncertain. Additional sedimentation capacity is probably necessary, and is proposed upstream of State Highway 1 in Alternative 4- Channel Dredge and Sediment Management.

The bed load simulation should not be understood to predict that sedimentation does not occur in the vicinity of the Calles. The simulation is based fundamentally on 2011 channel conditions and bed profiles, and it may well be the case that under different channel conditions, such as those that existed in 2005, that bed load sediment may have been more effectively transported to the Calles reach. In addition, sediment transported in suspension in the upper portion of the simulation reach and not explicitly included in the simulation of bed load sediment would be expected to add to actual sedimentation in the Calles reach.

Conclusion

This analysis estimated sedimentation rates in lower Easkoot Creek based on observed sedimentation and simulated bed load sediment transport and deposition. The most detailed, specific sedimentation estimates relate to the December 2005 storm event. Repeated channel profiles suggest that about 1,000 yds of sediment was deposited in lower Easkoot Creek during the winter of 2005, presumably most of which occurred in the December 31, 2005 flood event. Simulated bed load transport and deposition based on hydraulic simulation data for the 24-hour storm period December 31, 2005, and based on 2011 channel conditions indicated a range of sediment deposition from about 360 yds to 1,760 yds, depending on the size (diameter) of sediment (a surrogate for sediment supply) used in the simulation. Although imprecise, this range of simulated sedimentation brackets observed sedimentation and is of the same order of magnitude as observed sedimentation. Consequently, the simulation should be considered to be reasonably well supported by available data, and the simulation may be used, with due consideration of its limitations, to evaluate sedimentation aspects of selected flood mitigation scenarios. The significance of bed load transport with respect to flooding in Easkoot Creek is further evaluated in "Geomorphic and Watershed Sediment Assessment".

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Revised March 21, 2013
Revised May 15, 2013

TO: Chris Choo
Roger Leventhal
Marin County Department of Public Works
Flood Control and Water Conservation District

FROM: Matt O'Connor
O'Connor Environmental, Inc.

SUBJECT: Easkoot Creek Hydrology & Hydraulics Study

Appendix G. Geomorphic and Watershed Sediment Assessment

Introduction

The purpose of this memorandum is to present:

1. the sediment source assessment for the Easkoot Creek watershed and a qualitative geomorphic evaluation of the watershed, and
2. the assessment regarding whether a "stable" channel is possible within the natural variability of the system.

The findings from these assessments (identified in the project work plan as separate tasks) are presented in a single document because they are closely-related topics; presenting them together is more efficient and better conveys their significance to overall project objectives. In addition, this memorandum will draw on the Sediment Transport Evaluation, which documents estimated sedimentation rates in lower Easkoot Creek that are another significant element of the geomorphic assessment.

The first element of this memorandum summarizes the sediment source assessment conducted for the Easkoot Creek watershed. The sediment source assessment is based on prior studies of the Bolinas Lagoon watershed and of Lone Tree Creek, located about 2 miles southeast of Easkoot Creek, field reconnaissance of erosion processes in Easkoot Creek, review of historic aerial photography, and topography shown in imagery from the LiDAR-derived bare-earth digital elevation model (DEM).

The second element of this memorandum presents an assessment of geomorphic conditions in the Easkoot Creek watershed. The primary concern of the geomorphic assessment is evaluating long-term channel stability (and conveyance capacity) in lower Easkoot Creek given historic sedimentation patterns and expected future conditions. The geomorphic assessment summarizes watershed processes of erosion and sedimentation and their significance in relation to strategies to mitigate flooding.

Easkoot Creek Upper Watershed Sediment Source Assessment

The primary sources of information for this sediment source assessment are previous studies in the immediate vicinity of Easkoot Creek that provide erosion rate estimates applicable to the Easkoot watershed. The most recent study near to the project sites was a sediment source assessment of watersheds tributary to Bolinas Lagoon.[30] It was designed to obtain a representative sample of erosion processes and rates in the Bolinas Lagoon watershed area, and included field sites in Stinson Gulch, adjacent to and north of Easkoot Creek. The Bolinas Lagoon study also drew upon elements of a more intensive, field-based scientific study of erosion in Lone Tree Creek[31] spanning a three-year period from October 1971 through September 1974. The Lone Tree Creek study provides much more detailed data on erosion processes and rates than the Bolinas Lagoon study. In the latter study, bank erosion and soil creep rates estimated for Lone Tree Creek were directly applied as representative of the region. Although erosion processes and rates documented in Lone Tree Creek are likely to be generally representative of those in Easkoot Creek, the bedrock geology of the two watersheds differs to some degree as described below.

Bedrock Geology, Mass Wasting Processes and Influence on Erosion Rates.

As shown in Figure 1, Lone Tree Creek is located entirely within a single geologic unit (map symbol fsr, light blue), which is the mélange unit of the Franciscan Complex. Lower Easkoot Creek and an insignificant portion of upper Easkoot Creek are comprised of this unit. In Easkoot Creek the terrain in which erosion processes are most active is in two other geologic units within the Franciscan Complex: Kfs (Cretaceous sandstone and shale, shaded olive green) and Jfg (Jurassic greenstone, shaded green). The mélange unit is described as:

> *A tectonic mixture of variably sheared shale and sandstone containing (1) hard tectonic inclusions largely of greenstone, chert, graywacke, and their metamorphosed equivalents, plus exotic high-grade metamorphic rocks and serpentinite and (2) variably resistant masses of graywacke, greenstone, and serpentinite up to several miles in longest dimension…*[32]

The mélange contains significant quantities of greenstone (unit Jfg) and shale and sandstone (unit Kfs), hence it is comprised of substantially similar materials. Nevertheless, the mélange is distinguished by the extensive shearing of rocks and the presence of serpentinite and other highly metamorphosed rocks. From a geomorphic perspective, the mélange is known for extensive landslides of varying degrees of activity, and is expected to produce relatively high erosion rates. The mélange typically has relatively gentle slopes owing to the low strength of most of the materials it contains. In contrast, the sandstone and shale (Kfs) and the metamorphosed oceanic basalts comprising the greenstone (Jfg) are relatively strong, and can maintain relatively steep slopes. These characteristics are reflected by the steepness of Bolinas Ridge above Stinson Beach and contrast with the somewhat gentler slopes and rounded topography found along the Panoramic Highway and extending south along the coast towards Muir Beach including Lone Tree Creek, as well as the gently sloping terrain between Bolinas Ridge and the summit of Mt. Tamalpais (Figure 1).

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[30] Tetra Tech, Inc. (2001) Bolinas Lagoon Watershed Study Input Sediment Budget. Prepared for US Army Corps of Engineers, San Francisco, CA. November, 2001, 84 p.

[31] Lehre, A. K. (1982) Sediment budget of a small coast range drainage basin in north-central California. IN Swanson, F. J., R. J. Janda, et al. (Eds.) Sediment Budgets and Routing in Forested Drainage Basins. USDA Forest Service, Pacific Northwest Forest and Range Experiment Station, Portland, OR. General Technical Report 165, pp. 67-77.

[32] Blake, M.C. Jr., et al. (2000) Geologic map and map database of parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California. US Geological Survey, Miscellaneous Field Studies MF-2337, v. 1.0.

Based on differences in geology, erosion rates and processes in Lone Tree Creek might be somewhat different from those in Easkoot Creek. Nevertheless, the overall geologic setting and climate of the two watersheds are quite similar; the data from Lone Tree Creek are likely to be reasonably representative of Easkoot Creek. In the Bolinas Lagoon sediment source assessment, no distinction was made between different rock types within the Franciscan Complex, and no differences in erosion rates were inferred in the geologic units Kfs, Jgs and fsr described above. Data from the Bolinas Lagoon sediment source assessment, field reconnaissance, and a hill-shade image from the LiDAR DEM (Figure 2) indicates that a significant component of erosion in Easkoot Creek is comprised of landslides that occur in shallow soils on steep slopes adjacent to stream channels in canyons that extend from about 200 ft to about 1,200 ft elevation. These canyons are cut in rocks (Kfs and Jgs) that are strong enough to form relatively steep slopes; colluvial soil that mantles these slopes are inherently unstable. There are four main canyons with a few smaller tributary canyons: two that branch above Fitzhenry Creek, Laurel Creek (identified as Table Rock Creek on the State Park trail map), and Black Rock Creek (identified as Silva Gulch on the State Park trail map).

The steep canyon walls in the tributaries of Easkoot Creek generate landslides (primarily debris slides). In contrast, most landslides (79%) in Lone Tree Creek are described as occurring near the top of swales that branch off of Lone Tree Creek tributaries, while the remaining 21% occur at channel banks.[33] Field reconnaissance, review of historic aerial photographs, and interpretation of terrain shown in Figure 2 suggests that landslides near the top of swales are present in upper Easkoot Creek, but few of these features appear to have originated in the past 25 years. Landslides mapped from historic aerial photographs and field observations in 2012 (Figure 2) appear to be debris slides that deliver sediment to stream channels. It is likely that there are more debris slide scarps in Easkoot Creek tributary canyons that are hidden under forest canopy and virtually inaccessible to field geologists.

There was limited field evidence of debris flows, which often generate fast-moving slurries of water, rock and mud with a texture and density similar to wet concrete that typically erode steep stream channels before depositing in more gently-sloping canyon floors or alluvial fans. Debris flows are potentially more hazardous to urban areas near stream channels and on alluvial fans; although there are various references to 'debris flows' and 'debris flow deposits' in reports pertaining to Easkoot Creek, we have not seen any compelling evidence of debris flows presented in any of the documents that allude to them. During field reconnaissance in 2011, Dr. Matt O'Connor observed patterns of erosion and deposition typical of a small debris flow in an unnamed Easkoot Creek tributary at the end of Avenida Olema; however, there was no evidence of a debris flow deposit about 500 ft downstream where this unnamed tributary passes under a footbridge. No evidence of recent debris flows was found in Fitzhenry Creek, although debris slide scarps were observed on very steep slopes immediately adjacent to the channel. Debris flows in Lone Tree Creek were reported to be relatively small and not erosive, but capable of delivering significant quantities of sediment to nearby stream channels.[34]

[33] *Ibid*. 2, p. 71
[34] *Ibid.* 2. p. 71.

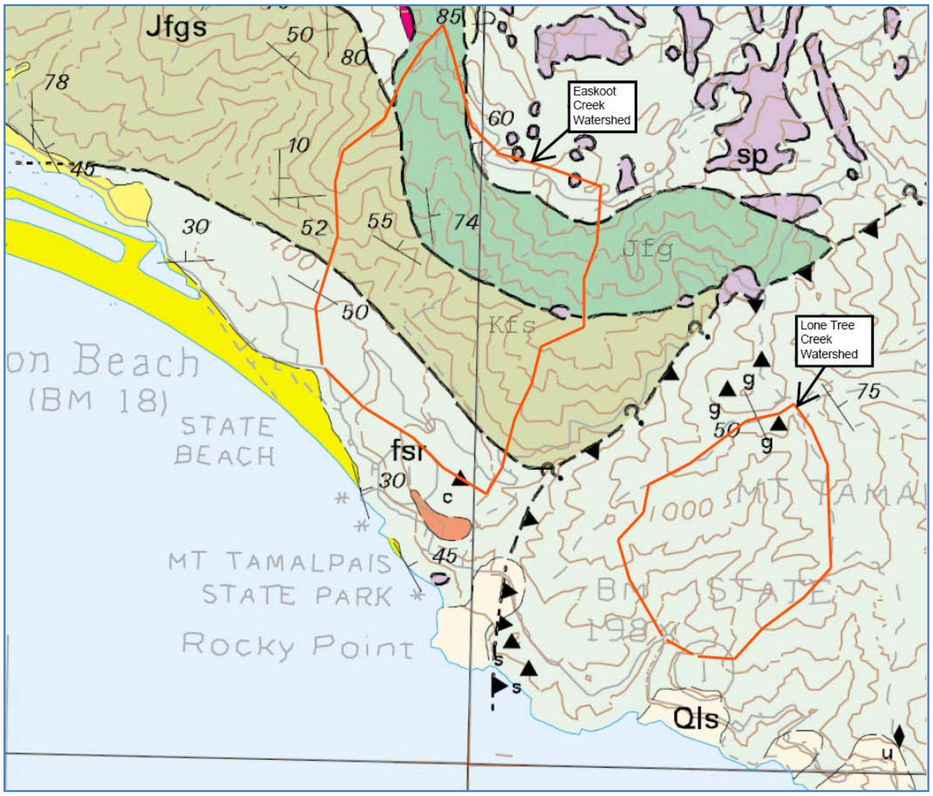

Figure G1 Bedrock geology of Easkoot Creek and Lone Tree Creek and vicinity.

The two watersheds are outlined in orange; the downstream boundaries for both are shown at the State Highway 1. Approximate scale as shown is 1:48,000; source map (USGS MF2337) scale is 1:75,000. See text for discussion of relevant geologic units and symbols.

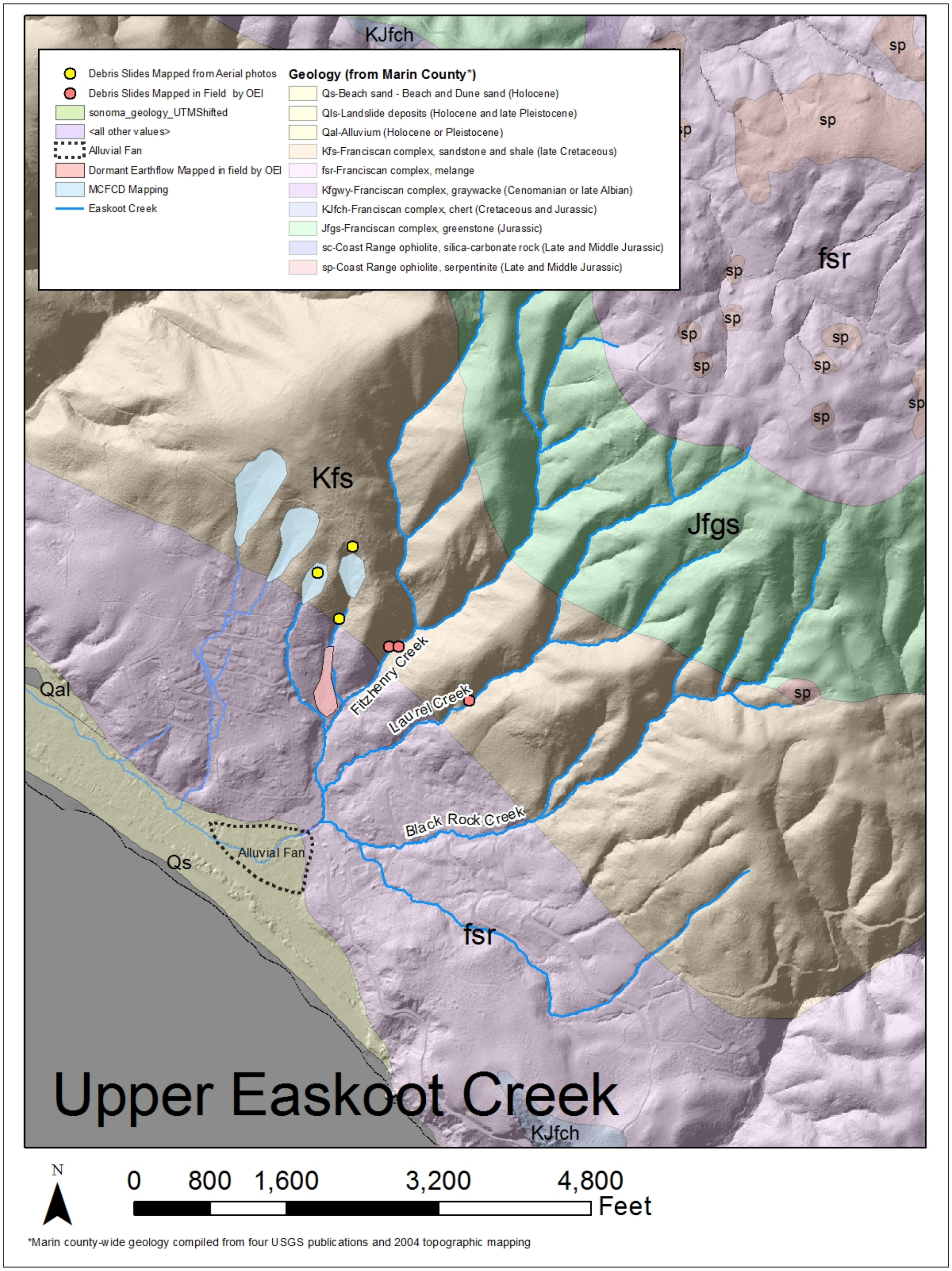

Figure G2 Hillshade image of LiDAR DEM.

Landslide features observed in the field and from aerial photography are shown, along with the alluvial fan formed by Easkoot Creek in Stinson Beach.

Based on field reconnaissance and interpretation of topographic data, a dormant earthflow was mapped on the west bank of Fitzhenry Creek near the second Matt Davis Trail bridge ascending from Stinson Beach. This landslide is slow moving, and sediment is gradually eroded from its toe from tall stream banks. The gradual movement of the earthflow has filled much of the width of the canyon formed by Fitzhenry Creek, and has pushed the channel against the eastern hillslope and likely delivered boulders to the channel that have formed steep cascades impassable by anadromous fish.

The types of landslides that occur in Easkoot Creek are primarily shallow, rapidly moving landslides (debris slides and small debris flows) and deep, slow moving landslides (earthflows and/or rock slides).[35] Debris flows can be particularly hazardous to life and property, and produce rapidly moving, highly viscous flows with the consistency of liquid concrete. Debris flows typically originate as debris slides; if the mass of soil, rock and water mobilized by a debris slide occurs in a steep narrow valley, a debris flow may develop. Large debris flows in Easkoot Creek have not, thus far, been documented to reach Stinson Beach. The tributaries of Easkoot Creek could produce debris flows in the future, and could represent a potential geologic hazard to life and property in Stinson Beach. In addition, landslide scarps on hillslopes above Avenida Las Baulinas demonstrate potential of future instability; we understand that one lot on the north (uphill) side of Avenida Las Baulinas was abandoned following landslides in the winter of 1982/83. Geologic structure documented in the geologic map (Figure 1) on the slope above Avenida Las Baulinas appears to contribute to landslide potential; the rock beds dip parallel to the hillslope in the area where large landslide scarps are visible. The proximity of the San Andreas Fault, located just offshore at Stinson Beach, and the potential for significant seismic shaking contributes further to landslide hazards. Analysis of those hazards is beyond the scope of this assessment.

Watershed Erosion and Sedimentation Processes.

Sedimentation in lower Easkoot Creek that reduces channel capacity and increases the likelihood of flooding results from erosion, transport, and deposition of sediment in the watershed. Long-term erosion rates in the upper watershed are determined by small-scale bank erosion and soil creep (including bioturbation[36]) from hillslopes into stream channels throughout the stream channel network and periodic landslides that occur on steep canyon walls and other unstable areas. When landslides occur, typically during wet winters and large rainstorms (e.g. winter 1973/4, 1982/3, 1996/97, 2005/6), they contribute significant quantities of sediment to stream channels. A portion of the landslide-derived sediment likely reaches the lower watershed during the same winter (or even the same storm event) while stream flow and sediment transport rates are high. Such “episodic” landslide events are likely responsible for observed high rates of sedimentation periodically observed in Easkoot Creek. Another portion of landslide-derived sediment is typically deposited in and adjacent to stream channels; these deposits are eroded in subsequent years during periods of peak storm runoff. Sediment deposits from small-scale bank erosion and soil creep also tend to accumulate in at channel margins. Consequently, the magnitude and duration of stream flow tends to determine the delivery rate of sediment from erosion sources in the watershed to lower Easkoot Creek in Stinson Beach, even in the absence of episodic landslides that tend to coincide with unusually intense rainstorms. The Lone Tree Creek study

[35] California Dept. of Conservation (1999). Factors Affecting Landslides in Forested Terrain, Division of Mines and Geology, Note 50. 5 p.
[36] Includes rodent burrows, coyote digs, and tree-throw.

concluded that about half of the sediment mobilized by landslides is stored in the watershed, and that "[s]ediment is removed from storage by storms with recurrence intervals greater than ten years."[37]

Sediment transport by streams through a watershed causes sediment to be sorted by size along the length of the watershed. Sediment entering the stream channel is transported by stream flow in a mode depending primarily on the size of sediment grains. Fine sediment (silt and clay less than about 0.1 mm diameter) are typically well-mixed (i.e. suspended) in the water column and are transported at about the velocity of flow; this is called suspended load or wash load. Suspended sediment is generally transported out of the channel system to the ocean or to the floodplain; silt and clay is therefore found in only small quantities on the streambed. Sand and gravel (from 0.1 mm to about 16 mm diameter) may become suspended in the water column during periods of unusually high stream flow (intermittent suspended load). Sand, gravel and cobbles are more frequently transported by rolling, tumbling or skipping along the streambed during periods of peak storm runoff; this is called bed load. The sediment stored on the streambed that can be transported through the watershed during periods of peak stream flow is mostly sand and gravel. Although coarser material on the bed is transported, the net rate of transport is much lower for cobbles and boulders, which more often comprise the framework of bed forms during long periods of immobility. In addition, the finer fraction of bed load (sand) is transported somewhat more rapidly than the coarser fraction of bed load (gravel). Consequently, sediment on the stream bed comprising the bed load near tidewater in Easkoot Creek is substantially finer than stream bed sediment in steep canyons nearer sediment sources.

The relationship between channel morphology and sediment transport processes in mountain streams can be expressed as a function of channel slope gradient..[38] In particular, where prevailing channel gradient is greater than about 0.03 (3%), sediment supplied to channels is generally transported downstream and the channel is said to be "supply limited". Channels were the slope gradient is less than about 0.03 have increasing potential for sediment deposition, and channels tend to be "transport limited" in that sediment is generally available for transport when there is sufficient stream flow. In general, Easkoot Creek conforms to these hypotheses.

Most of the channel network of Easkoot Creek is steep, with narrow, confined channels. Sediment entering these channels tends to be transported, and deposits of mobile sediment are only in temporary storage in gravel bars and on stream margins. In lower Easkoot Creek in the vicinity of State Highway 1 and Arenal Avenue in Stinson Beach, stream channel gradient begins to decline substantially (Figure 3) as Easkoot Creek crosses its alluvial fan, and sediment deposition becomes more likely due to declining stream energy. The stream channel remains confined within high banks until Easkoot Creek passes under the bridge linking the beach parking lot to State Highway 1 at the downstream margin of the alluvial fan. At this point, stream gradient declines further (Figure 3), stream banks are low, and high flows have the opportunity to spread laterally. Successive topographic surveys of the channel and records of channel dredging document this zone of sedimentation and have been used to estimate sedimentation rates (refer to Sediment Transport Evaluation for details). In the following section, watershed erosion rates in Easkoot Creek are estimated as an alternative means of predicting long-term sedimentation rates in lower Easkoot Creek. Considered together with the prior estimate of sedimentation rates, a robust prediction regarding likely future sedimentation rates can be determined.

[37] *Ibid*. 2, p. 67.
[38] Montgomery, D.R. and Buffington, J.M. (1997) Channel-reach morphology in mountain drainage basins. GSA Bulletin 109(5):596-611.

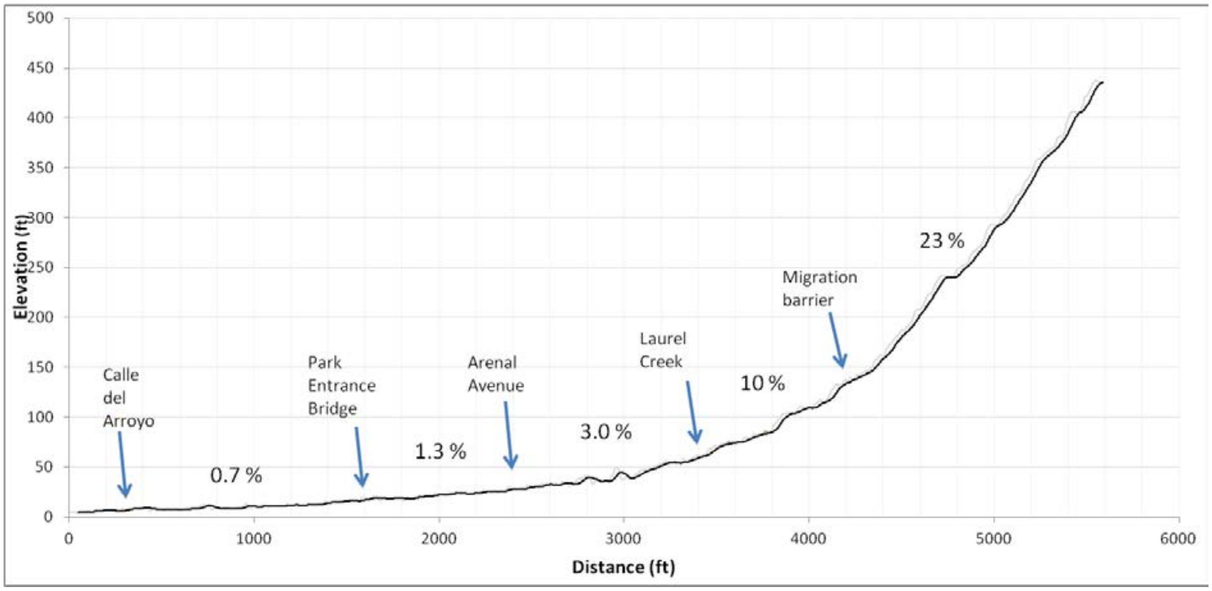

Figure G3 Easkoot Creek channel profile derived from LiDAR-derived digital elevation model.

The mean channel grade (%) is shown for reaches between selected landmarks. Laurel Creek is a tributary with its confluence just downstream of the first bridge on the Matt Davis Trail on Belvedere Avenue near the Community Center. The "migration barrier" is the limit of upstream migration for steelhead trout and coho salmon; it is located just downstream of the second bridge on the Matt Davis Trail. The channel profile shown ends at a major tributary confluence upstream.

<u>Watershed Erosion Rate Estimate for Easkoot Creek</u>. As described above, sediment eroded from the watershed and entering the stream channel network is transported either as suspended load or bed load. The suspended load is, for the most part, transported out of the channel network to the sea or to the floodplain while the bed load sediment remains in the channel bed and therefore comprises most of the material responsible for channel sedimentation. In most coastal watersheds in northern California, much more sediment is transported as suspended load than bed load. At Caspar Creek, a forested watershed in coastal Mendocino County, long term sediment yield studies indicate that about 70 percent of the total sediment yield is suspended load and the remaining 30% is bed load.[39] To estimate potential channel sedimentation rates in Easkoot Creek as a function of watershed erosion rates, it is necessary to estimate both the watershed erosion rate and the relative proportions of suspended load and bed load.

The Lone Tree Creek study discussed above estimated rates of various erosion processes, including debris slides and debris flows, bank erosion and soil creep. The most useful data from Lone Tree Creek, however, are measurement-based estimates of suspended sediment and bed load sediment yield. Data on stream flow, suspended sediment load and bed load were collected over a three year period from October 1971 to September 1974. In addition, an estimate of long-term average suspended load and bed load was synthesized from these data using regional hydrologic data. Sediment transport rates

[39] Cafferata, P. and Spittler, T (1998) Logging impacts of the 1970's vs. the 1990's in the Caspar Creek watershed. IN Ziemer, R. (tech. coord.) Proceedings of the Conference on Coastal Watersheds: The Caspar Creek Story. USDA Forest Service, PSW-GTR-168, p. 111.

from Lone Tree Creek are summarized in Table 1. These data indicate that about 84% of the total sediment yield is suspended load and 16% is bed load. The sediment yield data presented in units of weight per unit watershed area can be used to estimate sediment yield from Easkoot Creek simply by multiplying the rate and drainage area.

Table G1 Summary of sediment yield data from Lone Tree Creek (after Lehre, 1982).

Period	Suspended Sediment Yield (T/km2/yr)	Bed Load Sediment Yield (T/km2/yr)	% Suspended: % Bed Load
Estimated Long-term Mean	180	34	84 : 16
Three-year Mean (1971-1974)	607	85	88 : 12
Maximum Annual (Water Year 1973)	1,227	193	86 : 14

The Bolinas Lagoon study was based mostly on field observations of landslide scarps and roads, supplemented by estimates of bank erosion and soil creep rates from Lone Tree Creek which could not be as readily measured. Much of the Bolinas Lagoon study area is comprised of younger sedimentary rocks; landslide rates were differentiated among different bedrock formations, and landslide rates in the older Franciscan Complex bedrock tended to be substantially lower.[40] Landslide survey data collected in the field provided estimated rates of landslide sediment delivery per unit length of stream channel, which was about 61 T/km/yr over the Bolinas Lagoon study area, equivalent to about 233 T/km^2/yr. In the Franciscan Complex, landslide sediment delivery rates were about 40% of the average rate in the Bolinas Lagoon study area (about 25 T/km/yr). This re-calculated rate of sediment delivery from landslides (93 T/km^2/yr) was then added to the bank erosion rate (18.5 T/km^2/yr) and soil creep rate (2.6 T/km^2/yr) used for the Bolinas Lagoon study (extrapolated from the Lone Tree Creek study) to provide a revised estimate for sediment yield from Franciscan Complex watersheds (114 T/km^2/yr) such as Easkoot Creek.

For the current study, field reconnaissance in October 2012 provided an estimate of landslide rates from a survey of 0.5 km spread over the lower portions of two Easkoot Creek tributary canyons (see Figure 2). Using techniques comparable to the Bolinas Lagoon study; we estimated landslide delivery rates in this sample area to be about 29 T/km/yr. Although the sample size is small, this estimate agrees well with the rate estimated for Franciscan Complex streams from the Bolinas Lagoon study (25 T/km/yr). Reconnaissance in the upper watershed of Easkoot Creek indicated that the bank erosion and soil creep rates applied per the Bolinas Lagoon study were appropriate. Hence, we believe the estimated sediment yield for Easkoot Creek derived from the Bolinas Lagoon study is reasonably reliable.

Estimating Bed Load Proportion from Soil Texture Data.

The likely proportions of suspended load and bed load sediment can be estimated using soil texture data from the USDA Natural Resources Conservation Service soil data base.[41] Soil survey data includes representative textural analysis of inorganic material in soil, and the proportion of bed load size sediment in the soil column can be estimated based on the proportion of soil coarser than 2 mm diameter. For purposes of this analysis, material finer than 2 mm is presumed to be suspended load; this is a reasonable approximation for the steep, confined channels in the upper watershed of Easkoot Creek where sediment delivered to the lower watershed originates. In lower Easkoot Creek where

[40] *Ibid*. 1, p. 4-47.
[41] http://websoilsurvey.nrcs.usda.gov

stream energy declines, much of the sediment finer than 2 mm would be expected to drop from suspension and become part of the bed load.

In Easkoot Creek, two soils mantle the portion of the watershed where most erosion is believed to occur. These are the Centissima-Barnabe complex, 50 to 75 percent slopes, and the Saurin-Bonnydoon complex, 50 to 75 percent slopes. The Centissima-Barnabe complex has a typical soil depth of about thirty inches, and about twenty-one percent of the soil is coarser than 2 mm. The Saurin-Bonnydoon complex has a typical soil depth of about twenty-nine inches, and about ten percent of the soil is coarser than 2 mm. These soils cover roughly equal size areas in Easkoot Creek, so an estimate of the proportion of sediment input from erosion of the soil is average of ten and twenty-one percent, or about fifteen percent.

The foregoing estimate of fifteen percent bed load sizes in soil probably underestimates inputs of bed load size material from landslides because some larger rock fragments from bedrock underlying the landslides are generally incorporated in sediment delivered to streams. Nevertheless, the estimate of about fifteen percent of sediment inputs as bed load agrees well with the data from Lone Tree Creek (Table 1). This analysis provides independent evidence that it is reasonable to apply the long-term average proportion of sixteen percent bed load in the total sediment load from Lone Tree Creek to erosion rate estimates for Easkoot Creek to estimate bed load yield from Easkoot Creek. For a long-term average, it may be assumed that all sediment input to steep confined streams such as those in the upper watershed of Easkoot Creek is ultimately routed through the stream system with no net change in storage of sediment in the stream channel. Making this assumption allows for a direct approximation of bed load yield to lower Easkoot Creek from erosion rate estimates for upper Easkoot Creek.

Erosion Rate and Bed Load Rate Conversion to Volume.

Erosion rate estimates discussed above were summarized using metric units as reported in the original studies. The erosion rates reported in units of weight per unit drainage area must be adjusted for Easkoot Creek drainage area to produce the desired estimate of bed load yield per year. To facilitate comparison with the sediment transport and historic sedimentation estimates, the erosion rates are converted to units of volume in terms of cubic yards per year. Table 2 summarizes watershed erosion rate estimates applied to Easkoot Creek and the conversion to units of volume. The relevant drainage area for Easkoot Creek is that above State Highway 1, which is 3.53 km^2. The density of bed load sediment is assumed to be 1.2 short tons/cubic yard, consistent with the Bolinas Lagoon study.

Table G2 Summary of bed load yield estimates for Easkoot Creek.

Estimate Source	Rate (T/km^2/yr)	Easkoot Creek Rate (T/yr)	Units Conversion (short tons/yr)	Estimated Bed Load (t/yr)	Volume of Bed Load (cu. yd./yr)
Easkoot Creek Erosion Rate Adjusted for Bedrock Geology (after Bolinas Lagoon TMDL)	114	402	443	71	59
Lone Tree Creek Mean Annual Bed Load Yield	34	120	132		110
Lone Tree Creek Maximum Annual Bed Load Yield (WY 1973)	193	681	749		625

The estimated range of mean annual bed load yield from Easkoot Creek is 59 to 110 cubic yards per year. This is somewhat less than, but in general agreement with, the estimated range of mean annual sedimentation in lower Easkoot Creek of 122 to 160 cubic yards per year. The estimated maximum rate of bed load yield from Easkoot Creek, based on measurements in Lone Tree Creek during winter 1972/73, is 625 cubic yards per year. This is less than, but in general agreement with, maximum rates of sedimentation lower Easkoot Creek of about 1,000 cubic yards during the December 2005 flood event, and in the lower end of the range of estimated sedimentation (400 to 2,700 cubic yards per year) based on the bed load transport modeling.

Conclusion

Erosion rates in the Easkoot Creek watershed are not exceptionally high for a steep coastal watershed in northern California, and are probably lower than other areas tributary to Bolinas Lagoon with different underlying bedrock. Nevertheless, substantial erosion from landslides and bank erosion occurs at rates high enough to accumulate sediment in short-term storage in and near channels in the upper watershed. During most winters, peak stream flow is not high enough to mobilize and transport very much of this sediment stored in bars and on the upper stream banks in the upper watershed, so in "average" years, sediment yield from the upper watershed is modest and thus sediment deposition in lower Easkoot Creek is limited, and probably not more than about 125 cubic yards based on calculations of mean annual sedimentation from the sediment transport analysis.

In winters with more intense storms that produce high rates of runoff with recurrence intervals of about ten years (the estimated recurrence interval of the December 2005 event was about eight years), much higher rates of sediment transport occurs in the upper watershed because stored sediment is accessed by high flows and because landslides tend to occur during intense storm events. The high sediment supply and high transport capacity in these episodic events tend to overwhelm the channel in lower Easkoot Creek and result in significant sedimentation and associated loss of flood conveyance capacity.

Erosion rates estimates indicate that average annual rates of stream flow and erosion should be expected to deliver at least 60 to 110 cubic yards of bed load from the upper watershed to lower Easkoot Creek, similar to estimated mean annual sedimentation rate of about 125 cubic yards per year calculated in the prior analysis of sediment transport and historic sedimentation. The average annual rates of sediment supply and sedimentation are small enough that the existing program of dredging can be expected to maintain channel conditions and channel conveyance at or near the existing level

Relatively infrequent large storm events with annual recurrence intervals of about ten years produce several hundred to thousands of cubic yards of sedimentation. The most recent such event occurred in December 2005 and caused significant sedimentation and channel aggradation in lower Easkoot Creek. This pattern of episodic sedimentation is likely to persist and is in fact "natural" for this geomorphic setting. There are no feasible opportunities to reduce erosion rates from the upper watershed. Future sedimentation events are certain to occur, and are most likely to affect Easkoot Creek downstream of Arenal Avenue. It is unlikely that a "stable" channel can persist without significant intervention to manage sedimentation. It is likely than the sharp bend of Easkoot Creek below Arenal Avenue is a product of urban development of Stinson Beach, including placement of fill, development of infrastructure and commercial and residential buildings.

In a natural system subject to large episodic event loadings of sediment such as Easkoot, the channel could be expected to adjust by avulsing (forming new channel branches when existing channels are filled in) or adjusting its channel geometry (slope, width and/or depth). Given that dense riparian vegetation would likely be present, the channel's ability to adjust it width would be quite limited, and channel avulsion would be more likely. At the base of the fan adjacent to the beach, flows might also be so unconfined that water simply spreads out and no primary channel is maintained.

Finally, in natural systems subject to these episodic high sediment loadings events, in-stream habitat for fish and other aquatic species is often highly impacted by these high flow events. Natural systems naturally recover from episodic disturbances and in fact, there are environmental benefits from these periodic disturbances (i.e. removal of fines from the creek bed, reforming of creek habitat bed forms such as riffles, pools, and point bars), formation of new channels by avulsion, and recruitment and transport of natural woody debris that forms high quality habitat. Dredging in response to episodic events to re-establish creek channel dimensions that have been established over the past forty years in this urban setting and that have been demonstrated to be sustainable under average hydrologic conditions (i.e. in events smaller than about ten year recurrence interval) can provide a beneficial balance between habitat and flood control.

Future sea level rise does not appear likely to have a strong influence on bed load transport and sedimentation in Easkoot Creek because the geomorphic influence of declining channel slope gradient and channel confinement induces decline in transport capacity and sedimentation substantially upstream of the transition between freshwater and estuarine conditions.

SUBJECT: **Easkoot Creek Hydrology & Hydraulics Study**

Appendix H. Public Comments and Responses to the Final Draft Stinson Beach Watershed Program Flood Study and Alternatives Assessment

Gulf of the Farallones Comment Letter

UNITED STATES DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
NATIONAL OCEAN SERVICE

Gulf of the Farallones National Marine Sanctuary
991 Marine Drive, The Presidio
San Francisco, CA 94129

February 13, 2014

ATTN: Chris Choo
Marin County Flood Control District

RE: Stinson Beach Watershed Program Flood Study and Alternatives Assessment

Dear Ms. Choo:

Gulf of the Farallones National Marine Sanctuary (GFNMS) has completed a cursory review of the Stinson Beach Watershed Program Flood Study and Alternatives Assessment, dated August 2013 and prepared by O'Connor Environmental, Inc. (OEI) for the Marin County Flood Control and Water Conservation District (District). GFNMS, in coordination with other federal, state and local agencies and governments manages the waters and submerged lands of Bolinas Lagoon to the mean high tide as well as the coastal waters along Stinson Beach (also to the mean high tide). GFNMS also works closely with the Bolinas Lagoon Advisory Committee (BLAC), Marin County Open Space District (MCOSD), and other federal, state, and local partner agencies to coordinate activities within and adjacent to the lagoon. GFNMS would like to thank the District and OEI for the opportunity to provide comments. All comments provided herein discuss the proposed alternatives being considered as part of process to develop flood control strategies for the Easkoot Creek and surrounding community (within Zone 5 of the District).

The GFNMS was designated through the National Marine Sanctuary Act (NMSA) and protects an area of 966 square nautical miles off the northern and central California coast. Specifically, GFNMS manages the marine environment from Bodega Head (Sonoma County) to Point Año Nuevo (San Mateo County), including the tidal habitats of Tomales Bay, Estero Americano, Estero San Antonio, Bolinas Lagoon and Walker Creek. This is a place of special significance, and the GFNMS was designated to protect its ecological and cultural integrity for current and future generations. It is the intent of the National Marine Sanctuaries Act to protect certain areas of the marine environment which possess conservation, recreational, ecological, historical, research, educational, or esthetic qualities that give them special national, and in some instances, international, significance (National Marine Sanctuaries Act, 16 U.S.C. § 1431 et. seq., (NMSA) as amended by Public Law 104-283: § 301). Through regulation, GFNMS prohibits certain activities that are inconsistent with the goals, objectives, mandates and policies of the NMSA.

While most of the alternatives being considered are likely to be outside the boundaries of the Sanctuary, the District should consider the potential effects of each alternative on Sanctuary resources and water quality given the connectivity, close proximity, and tidal influence of Bolinas Lagoon to the Easkoot Creek and the nearby marine waters of Bolinas Bay. GFNMS currently has regulations to protect water quality within the Sanctuary up to the mean high tide line. Sanctuary regulations prohibit "discharging or depositing, from beyond the boundary of the Sanctuary, any material or other matter that subsequently enters the Sanctuary and injures a Sanctuary resource or quality" (15 CFR § 922.82). The National Marine Sanctuaries Act defines "injure" as "to change adversely, either in the short or long term, a chemical, biological or physical attribute of, or the viability of. This includes but is not limited to, to cause the loss of or destroy." "Sanctuary quality" is defined as "any of those ambient conditions, physical-chemical characteristics and natural processes, the maintenance of which is essential to the ecological health of the Sanctuary, including, but not limited to, water quality, sediment quality and air quality" (15 CFR § 922.3). These regulations may apply to activities proposed by many of the alternatives in the study given that activities in the creek and beach area could cause matter to be discharged and ultimately enter the Sanctuary and cause either short-term or long-term adverse effects.

None of the alternatives in the study address coastal flooding caused by wave action, storm surge, or extreme high tide events. GFNMS recommends that the study includes an analysis of this information as part of the evaluation of each alternative. While some of the alternatives may provide short-term benefits to mitigate flooding, impacts from climate change (such as coastal sea level rise, increasing storm surge, and higher wave run-up) may affect the longer-term viability of these options. "Our Coast, Our Future (OCOF)" is a collaborative, user-driven project focused on providing San Francisco Bay Area coastal resource and land use managers and planners with locally relevant, online maps and tools to help understand, visualize, and anticipate vulnerabilities to sea level rise. We recommend using their resources, especially storm models and flood maps, as a planning tool in the development of your study. You can find OCOF information here: http://data.prbo.org/apps/ocof/.

Comments on Specific Alternatives

Based on our initial review of the study, it appears that four alternatives may have a greater potential to affect sanctuary resources (i.e. the lagoon or ocean waters) whether during construction or thereafter.

Alternative 4

It's our understanding this alternative would involve the removal of 3,100 cubic yards of sediment from 2,300 feet of Easkoot Creek from upstream of Arenal Avenue to downstream of Calle del Arroyo. This action as proposed raises concerns about the potential effects that dredging may cause to established habitat and species both in the riparian zone (due to loss or alteration of existing habitat) and downstream in the tidal marsh of Bolinas Lagoon (due to changes in sediment input). The study should discuss how the project would mitigate for these potential impacts. In addition, if dredging is to be conducted annually, the project should address long-term cumulative impacts on habitat from repeated dredging episodes.

Alternatives 5 and 6

It's our understanding these alternatives would involve the construction of bypass channels on National Park Service land (either to the north parking lot or to the south parking and Poison Lake). The project should evaluate potential water quality impacts to the adjacent tidal waters of Bolinas Bay (including effects on recreational swimmers) due to possible reduced water quality in the detention basins. Further, the park's existing septic system and infrastructure should be assessed to determine the potential for impacts to water quality. Lastly, the study should evaluate potential effects on fish using the creek as a result of the periodic diversion of water into a bypass.

Alternative 7 - Causeway

It's our understanding that this alternative involves the construction of a causeway over Bolinas Lagoon to connect State Highway 1 with Seadrift Road along the alignment of what is currently a gravel road named Walla Vista Road. The primary purpose of this alternative would be to improve access to the Seadrift community, which relies on Calle del Arroyo as the only means of vehicular access; this route currently becomes submerged during moderate floods. As described in the document, the causeway would likely not be an elevated roadway structure like other causeways but rather a levee constructed from earthen fill and Bay mud with a 1-2 lane road on top. The causeway could include an optional construction of a tide gate and pump station to allow limited tidal influence to the inner portion of the marsh.

GFNMS regulations prohibit "…constructing any structure other than a navigation aid on or in the submerged lands of the Sanctuary; placing or abandoning any structure on or in the submerged lands of the Sanctuary; or drilling into, dredging, or otherwise altering the submerged lands of the Sanctuary in any way…" (15 CFR § 922.82). GFNMS can only issue a permit for a prohibited activity if we find that the activity will: (1) further research or monitoring related to Sanctuary resources and qualities; (2) further the educational value of the Sanctuary; (3) further salvage or recovery operations; or (4) assist in managing the Sanctuary. Thus, the construction of a new roadway in the Sanctuary would not be consistent with Sanctuary laws and policies. Further, the design of an earthen levee (even with a tide gate) would also result in adverse impacts to habitat and species in that area and thus undermine the ecological function of the Lagoon.

Alternative 9

It's our understanding that this alternative involves a combination of the Dredge option (Alternative 4) and the South Bypass (Alternative 6). Please see comments above for Alternatives 4, 5, and 6. Again, any proposals for on-going sediment management and potential future dredging activities should consider potential impacts or disturbances to the adjacent lagoon habitat and species.

Remaining Alternatives

In regards to Alternative 3, involving the proposed removal of vegetation within the creek to address flood impacts, if this alternative proposed the use of aerial application of herbicides adjacent to the sanctuary, the project would need to consider the potential impacts to the marine environment as a result of aerial treatments, including incidental drift of the compound into the sanctuary and the operation of low-flying aircraft within Sanctuary Overflight Restriction Zones. GFNMS regulations and management approaches can be found on our website: http://farallones.noaa.gov/ecosystemprotection/welcome.html. In addition, please note the use of construction machinery may cause harassment to federally protected species, including species protected by the Marine Mammal Protection Act (MMPA). The National Marine Fisheries Service is the lead federal agency on all MMPA permitting, and should be consulted before implementing any of the proposed options.

Lastly, GFNMS recommends for any of the alternatives that may be selected as preferred, mitigation measures include monitoring both physical and chemical characteristics at a site within the GFNMS boundaries to establish a baseline, and continued throughout the project duration and post-reclamation to ensure that there are no changes to water quality of the GFNMS.

GFNMS appreciates this opportunity to comment on the ICWMP and can provide additional information as needed. Please contact Max Delaney at 415-970-5255 or max.delaney@noaa.gov if you have any questions. Thank you.

Sincerely,

Maria Brown

Maria Brown
Sanctuary Superintendent

GGNRA Comments and Responses

Easkoot Creek Hydrology & Hydraulics Study

March 20, 2013 DRAFT

O'Connor Environmental

#	Commenter	Document	Reference (General or Sheet #, Station #s)	GOGA comment	Response
1	Fong	Executive Summary	Page 1, 1st paragraph	coastal flooding caused by high sea level not a focus--hmm, shouldn't analyses assume flood frequency assuming some high tide condition. DF-OK, see language on Page 9 about using MHHW-- perhaps should clarify statement on Page 1	Clarified on Pg 1 of the Ex. Summary
			Page 1, last paragraph and Page 4	threatened steelhead trout and endangered coho salmon. Note-- we only had one year where we had spawning for coho.	Edited (P2 of intro)
			Page 2, 1st paragraph	are the mass wasting processes natural or accelerated? E.g., should focus be on sediment reduction?	Sediment was not a focus of this study. Sediment reduction would be welcome.
			page 6, 2nd paragraph	Should be careful about generalizations without hard data (e.g., high concen. of fine sediment)	
			MAP and ALT FIGS	Need a map showing where all the Calles and other locations referenced in the document. Would also be helpful to bring a conceptual map for each alt. forward	Done
			page 7, No Project alt	Does the floodplain detention basin across from Parkside fit in here?	No, assumes increased sedimentation and no additional work by Flood Control
			page 7, Dredge and Sediment Mngt alt	It is unclear what the re-dredge trigger/threshold would be and whether the modeling and subsequent flooding benefits assumes a static bedlevel condition. DF-OK, Table ES5 references potential future dredging. Still seems to need a mgnt trigger threshold to evaluate	Will evaluate further in project development
			Table ES5 "Fish Habitat" column	There is a persistent philosophy in the Fish Habitat section re: access to floodplain and stranding risk. Using this philosophy, we should have confined channel for fisheries and no floodplain habitat. There is information for the Sacramento River basin and Yolo Bypass and other areas that juvenile salmonids use cues to minimize risk of stranding but allow foraging, etc.	We asked for evaluation of backwater/side channel habitat, but this wasn't developed due to the lack of flood benefits
		Appendix A	Page 25-preliminary design	The project description provides info on the fate of bedmaterials. What will be done with large woody debris that is in channel-- presumably removed?	Unknown. Will evaluate further in project development
		Appendix A	P27-Potential habitat restoration	How was 8 pool:riffle sequences derived? Would structures or LWD be installed to maintain pools? Doesn't seem like pool:riffle sequences would persist for more than a winter (or less)	Will evaluate further in project development
		Appendix A	P28-partial weirs	How would the weirs be accessed for sediment maintenance?	Unknown. Will evaluate further in project development
		Appendix A	P37-North Bypass Alt	Not sure what the logic is for replacing back the left bank berm in the north lot-- the north lot would still provide flood attentuation services without it. Just a guess-- but historically the North lot area likely flooded frequently under moderate events and drained back to creek as hydrograph receeded with possible discharge into ocean under extreme events. I'd be interested in seeing how a design which was closer to this model would have worked. The current North Bypass proposal appears to be counter to what we had wanted from our early restoration plan-- which was to remove berms that were placed along the channel banks that confined the channel during flood events.	Will evaluate further in project development
		Appendix A	P37-North Bypass Alt	What set the design height of the bypass weir structure at 40 cfs?	Included in P100
		Appendix A	P43-South Bypass Alt	It is unclear from the long. Profile graphich what the bottom invert elev. Of the culvert as it discharges to "Poison Lake" It looks like it coulbe be up to 7 ft (25 to 18 ft NAVD88)? Seems steep dropoff over short area and implications for fish passage and erosion	Will evaluate further in project development
		Appendix A	General-South Bypass Alt	It is unclear the justification for the proposed siting of the new "Poison Pond." As Joel Wagner notes, it is not exactly in the same location as the historic pond. The footprint of the open water habitat is further south in the South Parking Lot. However, the fringing wetlands definitely extend northward and of course around the perimeter of the pond. It would probably be best to analyze the location that best meets your project goals and comes closest to historic conditions. Then things can be tweaked to look at how the built environment (picnic area, parking) can fit in.	Agreed. This was fairly conceptual and we will evaluate it further in project development
		Appendix A	General-South Bypass Alt	Another thing to note is that there are enhancement/restoration opportunities for things other than salmonids assoc. with Poison Pond. Anecdotal info from long-time residents on turtles and frogs.	Agreed.

		Appendix A	Page 50-Zones A-C	It wasn't clear the basis for the water depths chosen for the zones. E.g., was the depth of Zone C driven by NPS well log data and gleyed materials or biotic basis (>3 ft summer depth to prevent encroachment of cattails/tules).	Some explanation provided on P104, but preliminary. Would need to be developed further.
		Appendix A	>Page 47-Bypass structures	Although the document is designed to allow one-way movement of fish from Easkoot Creek to the pond, it would be helpful to look at whether there were other locations where both inlet/outlet weirs and long culverts had been successfully modified to allow for up-down movement. It is possible that the creation of the lagoon type system might result in adult steelhead getting into Poison Pond after freshets.	
		Appendix A	Bypass Alternatives	Was wondering what the effect on the Easkoot channel might be of diverting all the channel forming flows.	Will evaluate further in project development. Some level of sediment maintenance is likely downstream, but overall, this would be evaluated further when project is defined.
		Appendix A	Combination Dredge/ South Bypass Alternative	The combined project included the lowering of the weir crest elevation by about 2-ft. If channel dredging were significantly delayed, could result in channel aggradation that would cause most of the flows to enter into bypass and Poison Pond. Loss of functionality in Easkoot?	Agreed. Would have to work out timing of both projects.
			General Comments:	1) I'm concerned about presenting the results of this study, which is very well done, to the public without additional study/information on how coastal storm hazards, and coastal storm hazards in combination with SLR would affect the feasibility of the most feasible alternatives presented in the report. Storms that increase streamflow are likely to occur coincident with increased coastal storm wave height and energy, and could also occur coincident with extreme high tides. While I think it makes sense to show the community what work is being done to study the issues and identify alternatives, it would be helpful to have some next steps in progress so that it's clear that we don't have answers yet. Hopefully there will be an effort to work with USGS and the OCOF project to start to analyze these additional factors.	Executive Summary addresses most these issues. We agree that more is needed before projects are implemented.
				2) It is also critical to have the additional information on the cost of raising homes in the floodplain. The long-term prognosis, with SLR and coastal storm hazards, for many of these homes is bleak, but the interim step of raising many of these homes addresses both flooding associated with Easkoot Creek and coastal hazards/SLR. It would be especially helpful to see the overlap of flooded homes from these two exercises.	Alternative 10 (P. 143)
				3) Putting the burden of protecting private property built in the flood zone on public lands, the purpose of which is to provide for the preservation of resources for future generations and enjoyment by hundreds of thousands of visitors, is going to be a tough hurdle.	Noted.
			South Bypass and Poison Lake Restoration alternative	1) Appendix A states that GGNRA has contemplated Poison Lake restoration. I think it would be more accurate to say that the NPS has conducted studies to determine the feasibility of restoring Poison Lake or a portion of it, but the park has not really contemplated doing the restoration although it was included as an alternative in the Draft General Management Plan but was not selected as the preferred alternative.	Noted.
				2) While we appreciate the sensitivity shown to impacts to parking at Stinson Beach, I think this may have led to the mistaken conclusion that picnic facilities were of lesser value than parking. Picnicking is a primary visitor activity at Stinson Beach and is one of the few large picnicking facilities within the entire park. This is an activity the park is looking to actively expand in other areas and its importance should not be undervalued. Given that Poison Pond was, in fact, located further south, this alternative may want to consider using a portion of the South Parking Lot rather than the picnic area and existing wetlands, even though it would impact parking on peak days.	Noted. We will have to evaluate specific uses if we pursue alternatives on NPS lands.

				3) The construction and long-term maintenance of a large bypass channel (essentially a long large ditch, along with perimeter berms and a 50 ft. wide outlet structure) in the midst of Stinson Beach park seems potentially incompatible with existing recreational use of the area; seems likely to introduce new safety hazards into this heavily used area; and may impact aesthetics and visual enjoyment of the 'natural' environment by visitors; as well as potentially being incompatible with park values. These are some of the issues that would need to be addressed in the EIR/EIS.	Noted. We will have to evaluate specific uses if we pursue alternatives on NPS lands..
				4) There is no discussion of potential reuse or disposal of the 16,500 cu. yds. of material that would need to be excavated for this alternative. It's unclear if costs associated with this volume of material was included in the cost estimate for this alternative. These would also be issues to address in the EIR/EIS.	Noted. Cost estimates assumed a local disposal site (within 30 minutes). We will have to evaluate specific uses if we pursue alternatives on NPS lands.
				5) The NPS has no information on the stability of the, at least partially manmade, dune system at Stinson Beach. Some portions of the dunes are being undermined and/or eroded. The park has no plans, no funds, and no excess sand with which to reconstruct the dunes to ensure they continue to provide protection from coastal hazards and SLR.	Noted. We will have to evaluate specific uses if we pursue alternatives on NPS lands..

Review of Easkoot Creek Hydraulics and Hydrology Study

Joel Wagner, NPS Water Resources Division (WRD)

April 24, 2013

Overall, the report is well-written and is presented in a clear and professional way. The authors have developed a good range of alternatives and presented their results in very understandable tables, figures and discussions that allow for meaningful comparison and evaluation. Gary Smillie (WRD) will provide technical review of the hydrology/hydraulics analyses in a separate response. My comments focus on the South Bypass and Poison Lake Restoration alternative, since it (in combination with the dredge alternative) provides the most effective flood protection and would have the greatest impact on park resources.

South Bypass and Poison Lake Restoration (starts on P. 41)

This alternative is a misnomer in that it does not actually "restore" Poison Lake. Historic photos and backhoe excavations in the south parking lot performed by WRD, Colorado State University (CSU) and GOGA staff in 2004 provide evidence that the open water portion of Poison Pond was within the south parking lot footprint. This alternative proposes to excavate a new pond in the picnic area and in the existing wetland, both of which are north of the historic pond location. Using standard federal definitions, excavating the existing wetland to create a pond is wetland "enhancement" (elevating wetland functions beyond their natural levels, often to the detriment of other functions) as opposed to true restoration (reestablishment of pre-disturbance conditions and functions). On the other hand, re-establishment of wetlands around the proposed pond may include restoration of buried toe slope wetlands, depending on the final design.

Comment [cc1]: Agree. Keeping name for now to be consistent with what's been published during scoping.

Logs from the 2004 excavations indicate that the picnic area and the northern part of the parking lot historically were groundwater-fed, vegetated wetlands at the toe of the Easkoot Creek alluvial fan that sloped toward the historic Poison Pond. They had either peat soils (histisols) or mineral soils with thick organic layers on the surface (histic epipedons). These wetland types are particularly valuable for carbon sequestration and water quality improvement, and they frequently provide specialized habitats for plants and animals. The picnic area was created by filling these toe slope wetlands, and the existing wetland between the picnic area and the beach is probably a remnant of these wetlands. So, the statement on P. 46 (Description, lines 4-6) that these wetlands are a "remnant of the historic Poison Lake" appears to be inaccurate. The statement on P. 47 (Flood Control Benefits, line 1) referencing a restored Poison Lake under this alternative is inaccurate as well.

A proposal to excavate and convert the wetland by the picnic area would be subject to NPS wetland protection policies (*NPS Management Policies (2006),* Section 4.6.5*),* and would trigger preparation of a Wetland Statement of Findings (WSOF) as required by *NPS Procedural Manual #77-1: Wetland Protection.* Since nearly all of the wetlands in this part of the watershed have been obliterated, the WSOF would somehow need to justify excavation of this last vestige of what was once a high-value wetland resource, and would have to show that there are no practicable alternatives to excavating the wetland that still meet project purposes. That would be a difficult task.

Comment [cc2]: Noted. Good things to keep in mind if this alternative is pursued. For now, this is purely feasibility.

On P. 49 (Poison Lake Restoration Design Factors), the study acknowledges that there are many options for the actual placement, size and configuration of the proposed wetland/pond enhancement project. But, for this study, the authors minimized impacts on existing parking in the south lot. I would be interested in working with the park to look at alternatives that might strike a better balance between parking and the restoration and preservation of wetlands while still assuring that the flood relief and fish habitat (open water) purposes are achieved. David Cooper (CSU) is also interested in participating in planning, design, and implementation since he worked with us on the initial investigations and he has the appropriate wetland restoration expertise, including peatland restoration.

I agree with the authors that the proposed 3-foot berm around the wetland/pond site may not be necessary, and that additional evaluation is needed during a more detailed design phase.

It is very clear that sediment control/removal would be necessary for this alternative to be sustainable because of the large sediment load of Easkoot Creek. It may not take long for the pond/wetlands to fill with sediment without these controls.

On P. 51 (Permitting Issues), I reiterate that a Wetland Statement of Findings would be needed for the project as proposed. WRD would have difficulty signing it if there are practicable alternatives that satisfy the project purpose but would have fewer impacts on wetlands.

On P. 51 (last sentence of the second paragraph), the statement that excavating the existing wetland will result in a "higher quality" wetland is not supported by an analysis of the existing wetland's functions or values. The 2004 excavations showed that the wetlands that were filled at the picnic area and the north end of the south parking lot had organic soils that provided important functions including carbon sequestration, water quality improvement and specialized habitat for plants and animals. Organic soils can take centuries or millennia to reestablish in a new location, and their "quality" or "values" should not be disregarded. I do think it is fair to say that new open water habitat would add important functions (e.g., fish habitat) that are not there now.

EXECUTIVE SUMMARY

Easkoot Creek Hydrology & Hydraulics Study

Prepared for

Marin County Flood Control and Water Conservation District

Prepared by

O'Connor Environmental, Inc.

In Association with

Garcia and Associates
Robert Zlomke, PE
Professor John Largier, UC Davis

March 20, 2013

DRAFT

TABLE OF CONTENTS

1. INTRODUCTION

This executive summary describes the results of the analysis of a suite of alternatives designed to reduce flooding in Stinson Beach caused by Easkoot Creek. Coastal flooding caused by high sea level and storm surge is not the focus of this study. Additional objectives to be addressed by viable alternatives include improved sediment management and habitat for salmonids in Easkoot Creek. The alternatives analysis is the culmination of an interdisciplinary study of the watershed conducted under contract with the Marin County Flood Control and Water Conservation District (the District) that began in October 2011. The contractor has worked closely with the District throughout the study and participated in three project meetings with the District's Technical Working Group (TWG) in Stinson Beach.

The various components of the study resulted in a series of technical reports which are included in Appendix B of this report. Individual reports describe the following efforts:

- Background information summary and development of a data acquisition plan
- Evaluation of the suitability of the Golden Gate LiDAR data
- Hydrologic modeling and runoff characterization
- Hydraulic modeling and flood hazard evaluation
- Assessment of fisheries conditions
- Evaluation of sediment transport rates
- Geomorphic and watershed sediment assessment

Based in part on the results of these investigations, a suite of alternatives were identified and evaluated in terms of their flood control benefits, preliminary design constraints, estimated construction costs, likely permitting issues, operation and maintenance requirements and estimated costs, sustainability, and overall feasibility. Alternatives are described in detail in Appendix A of this report.

In keeping with the philosophy of the community-based decision making process of the Marin County Watershed Program, the list of alternatives has been developed with significant input from the community. The alternatives selected for evaluation were drawn from prior studies and from meetings with the TWG, and were presented at a public meeting in Stinson Beach in April 2012. Rather than identifying a 'preferred' alternative, the pros and cons of each alternative have been assessed and summarized to assist decision-making regarding future flood mitigation activities. Nevertheless, when the objective results of flood analyses indicated that a particular alternative did not substantially reduce flood impacts, this was noted and in many cases the effort to develop and evaluate additional details was curtailed. Whichever course of action is chosen, consideration must be given to the natural watershed processes that determine much of the character of Easkoot Creek in Stinson Beach as well as the effects on habitat for endangered steelhead trout and Coho salmon.

2. WATERSHED GEOMORPHIC CONTEXT AND IMPLICATIONS FOR SUSTAINABILITY

Stinson Beach has been subject to periodic flooding when large storms produce high rates of runoff from the upper watershed of Easkoot Creek. The severity and extent of flooding is controlled in part by channel capacity in lower Easkoot Creek, which has been significantly reduced by ongoing sedimentation. Watershed erosion processes have a strong influence on sedimentation in lower Easkoot Creek; storm events that cause flooding generate high stream flow, sediment supply, and sediment transport rates in the watershed. The dominant watershed erosion process in upper Easkoot Creek is mass wasting (landslides) on the steep slopes adjacent to stream channels (Figure ES1); landslide rates increase during unusually intense rainstorms. Sediment delivered to tributary channels may be stored for several years in and adjacent to the channel awaiting high stream flow events that are capable of transporting sediment through the channel network to lower Easkoot Creek.

Figure ES1. Recent debris slide scarp on an Easkoot Creek tributary near Table Rock.

In the upper watershed, steep confined channels maintain continuity of flow and sediment transport. This continuity of flow and sediment transport extends across Easkoot Creek's upper alluvial fan to the vicinity of Arenal Avenue where declining channel slope reduces sediment transport capacity (Figure ES2). Further downstream in Easkoot Creek on the toe of the alluvial fan below the Park Entrance Bridge, channel slope diminishes further and bank height declines to about three feet. Under these conditions of declining slope and channel confinement, channel sedimentation inevitably results.

Alluvial fans are characterized by declining slope and sediment transport capacity, channel avulsions, extreme variations in erosion and sedimentation, and shifting channel positions and patterns of flooding. In addition, the alluvial fan of Easkoot Creek extends to the back-beach environment a few feet above sea level such that water and sediment routed from the upper watershed across the fan encounters a relatively flat and broad floodplain. These conditions are portrayed prior to the development of Stinson Beach in Figure ES3. Urbanization and development of Stinson Beach resulted in channelization of Easkoot Creek, perhaps establishing a defined channel draining towards Bolinas Lagoon where such a channel may not have previously existed.

Comment [T1]: How does Bolinas Lagoon affect the flow in the lower reaches of Easkoot Creek? Is it manipulated at the mouth of the creek?

Comment [COM2]: This is discussed in the model development. A second tidal area model was developed to model the connection between the tidal Bolinas Lagoon and Easkoot Creek.

Comment [COM3]: Word choice? Replace with "influenced"?

Comment [T4]: It's likely that there wouldn't have been a persistent channel; alluvial fan channels tend to form and migrate throughout the 180 degrees of the fan. Suggest acknowledging that a fixed channel is not a natural condition, but it's whatwe're dealing with in this developed setting. The challenge of Protecting development in a floodplain or alluvial fan setting should be articulated here.

Comment [COM5]: Good comment. We will add something to the section.

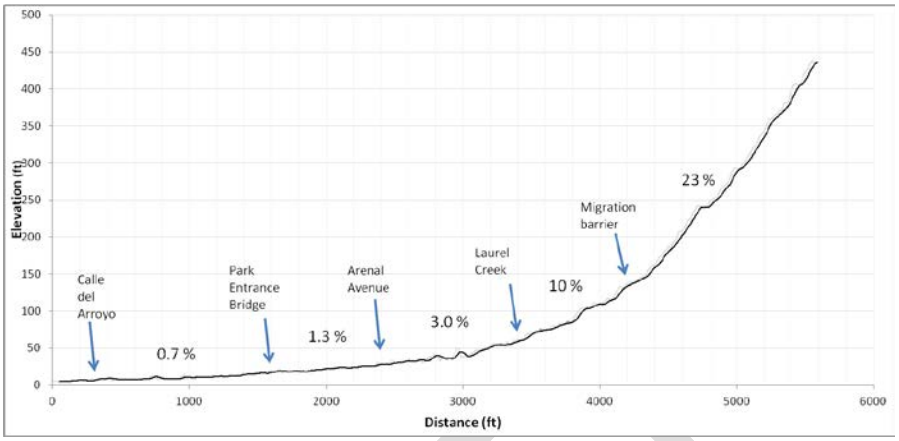

Figure ES2. Easkoot Creek channel profile.

Figure ES3. View to northwest of Stinson Beach c. 1904.

Contemporary sedimentation in lower Easkoot Creek (below State Highway 1) was analyzed using data on historic dredging, modeling of sediment transport rates, and estimates of watershed erosion rates based on prior studies to assess likely future sedimentation and its impact on potential flood mitigation strategies, sustainability of salmonid fish habitat, and flood conveyance capacity in Easkoot Creek. Typical annual sediment deposition does not significantly affect channel conveyance capacity. Expressed as an annual average, sedimentation rates are on the order of 125 to 160 cubic yards.

The sedimentation analysis revealed that flood events with a recurrence interval of about ten years (ten percent probability of occurrence in any given year) are likely to cause significant sediment deposition in lower Easkoot Creek on the order of 1,000 cubic yards or more. Dredging of several thousands of yards of sediment from Easkoot Creek to maintain the channel capacity after large storm events (e.g. winter 1982/83; Figure ES4) was the historic response to these episodic flood events. In the absence of channel maintenance, Easkoot Creek would be expected to shift position periodically in response to decadal storms generating high runoff, sediment transport, and sedimentation. ~~Typical annual sediment deposition does not significantly affect channel conveyance capacity. Expressed as an annual average, sedimentation rates are on the order of 125 to 160 cubic yards.~~

Figure ES4. In-stream dredging of Easkoot Creek, c. 1982.

Habitat for endangered salmonids (steelhead trout and Coho salmon) is also affected by sedimentation. A stream restoration project in 2004 on Golden Gate National Recreation Area (GGNRA) property ~~on the lower fan~~ downstream of Calle del Mar was affected by about two feet of sedimentation (Figure ES5) resulting from the floods of December 2005. Habitat enhancement designed to create stable pools in this reach should only have been expected to provide desired habitat temporarily; based on the analysis of sedimentation processes and rates, sedimentation and channel aggradation in lower Easkoot Creek appears inevitable.

Comment [T6]: Darren – Did we expect the pools to be stable? I thought we were focused on floodplain connection.

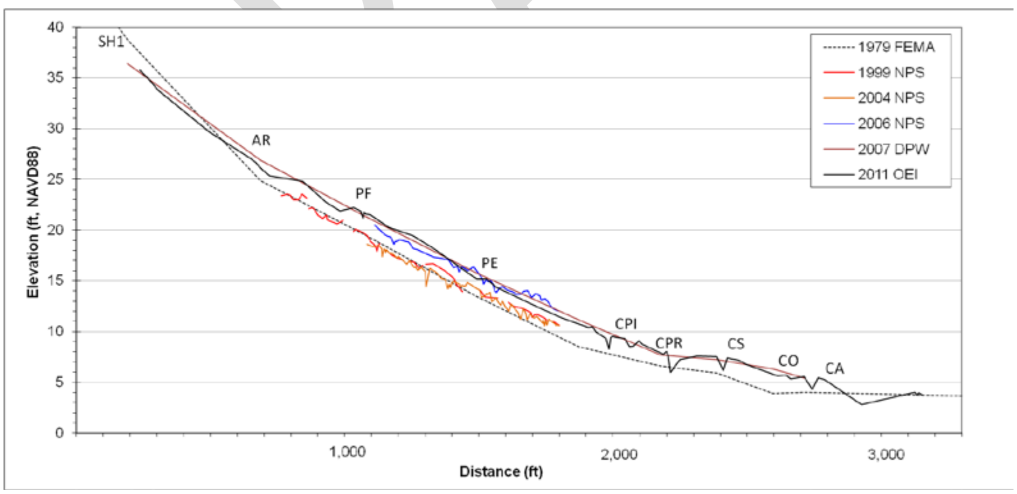

Figure ES5. Longitudinal channel profile of Easkoot Creek from State Highway 1 (SH1) to Calle del Arroyo (CA); comparison between 2004 and 2006 shows the impact of the December 2005 event.

Upstream of Arenal Avenue, sediment transport capacity is sufficient to prevent long-term sedimentation and aggradation. In late summer, stream flow conditions and water quality (defined with respect to temperature and dissolved oxygen) decline in lower Easkoot Creek relative to locations higher on the fan upstream of Arenal Avenue. These spatial variations in sedimentation, stream flow, and water quality conditions suggest that efforts to restore or enhance salmonid habitat are more likely to be effective in the reach above Arenal Avenue.

Comment [T7]: Is this relevant?

Any expected improvements in flood conveyance and fish habitat derived from dredging, grading, or habitat enhancement on the lower portions of the alluvial fan are temporary, and ongoing sediment management (including periodic dredging) is expected to be unavoidable. Natural watershed processes will continue to produce sediment from erosion in the tributaries of Easkoot Creek, and declining sediment transport capacity on the alluvial fan will produce sedimentation and aggradation similar to that documented in Figure ES6. This will force floodwater to spread onto the floodplain more frequently and degrade already marginal habitat for salmonids. Efforts to manage sediment by inducing deposition for managed removal should be considered as a means to better maintain channel conveyance and habitat in lower Easkoot Creek.

Figure ES6. Long-term sedimentation at Calle del Sierra can be seen by comparing conditions c. 1960 (left) and 2011 (right).

3. FISH HABITAT

Easkoot Creek has recently supported a small population of steelhead trout (*Oncorhynchus mykiss*), and may also have historically supported a run of Coho salmon *(Oncorhynchus kisutch*). Steelhead in Easkoot Creek are listed as threatened under the Endangered Species Act (ESA), and Coho are listed as endangered under the ESA and the California Endangered Species Act (CESA). A proposed project that may affect these species or their habitat requires an assessment of potential impacts and permits will not be granted if the project is deemed to jeopardize their survival. The National Park Service (NPS) documented steelhead use of Easkoot Creek in 1999 and 2000 and evaluated the habitat conditions present at that time. The NPS also found a few Coho in Easkoot Creek in the early 2000s.

A barrier to upstream migration is located about 1,500 feet upstream of State Highway 1 (Figure ES2). Juvenile steelhead have been found upstream and downstream of State Highway 1. All

Comment [T8]: Is this a natural or artificial barrier?

Comment [COM9]: Natural

of the documented spawning sites are located in lower Easkoot Creek (below State Highway 1), however steelhead may spawn upstream of Highway 1 as well.

Pools providing complex cover (e.g., formed by and incorporating large woody debris) are important rearing habitats for steelhead and Coho. Pools or other habitat features with complex cover to protect rearing fish are uncommon in Easkoot Creek, and thereby limit the suitable habitat for salmonids.

Sedimentation in the lower reaches of Easkoot Creek further limits and degrades habitat conditions for salmonids. Despite evidence of spawning in the lower reaches, relatively high concentrations of fine sediment (<1 mm in diameter) in the stream bed substantially reduces the quality of spawning habitat in the lower reaches. The NPS's 2004 habitat restoration designed to create stable pools in the lower reach of Easkoot Creek had limited success because of subsequent sedimentation. Overbank flooding in the lower reaches, exacerbated by sedimentation, may cause fish to become stranded on the urbanized floodplain.

Late summer stream flow and water quality (defined by temperature and dissolved oxygen) may also limit fish habitat in Easkoot Creek. Upstream of State Highway 1, water flow, water temperature, and dissolved oxygen conditions are stable and favorable for salmonids. Water quality in the lower reach is degraded, and the stream frequently goes dry in the reach between Arenal Avenue and Calle del Mar.

The proposed flood mitigation alternatives will affect fish habitat. For example, dredging the channel to increase conveyance could directly harm fish or disrupt known spawning and rearing habitat, and creating a managed overflow by routing flood waters to the ocean could strand juvenile fish or export them to the ocean. The alternatives considered must protect individual fish during construction, and provide for suitable habitat conditions for the endangered fish to carry out all phases of their lives (i.e., spawning and rearing habitat). With care and foresight, flood control measures can be designed and constructed to protect, and possibly enhance, habitat conditions for salmonids in Easkoot Creek.

4. ALTERNATIVES SUMMARY

The suite of alternatives that were investigated are summarized below. More detailed descriptions of these are provided in Section 8 of this report.

Infeasible Alternatives

One of the earliest potential alternatives identified to alleviate flooding along Easkoot Creek, the construction of a bypass channel carrying excess flow directly to the ocean, was rejected from detailed consideration because it would create a level of risk to endangered fish species that was deemed insurmountable. A potential alternative involving creation of flood detention storage and off-channel habitat (e.g. side channels or sloughs) in available floodplain areas was also rejected from detailed consideration because of insufficient potential for generating significant detention storage. Similarly, an alternative evaluating management of urban runoff by increasing local infiltration of rainfall and runoff was rejected from further consideration owing to insufficient potential to mitigate flooding.

Comment [T10]: The alternative of removing development subject to flooding should at least be mentioned here.

Comment [COM11]: This is intentionally out of the scope of this report.

Comment [T12]: The flooding associated with each alternative is compared to some base condition, but not the No Action Alternative. Suggest inserting a brief section describing that condition and how it was developed or modeled for comparison with the flood control alternatives.

Comment [COM13]: I believe Matt does this below.

No Project
This alternative is designed to represent the no project (i.e. 'do nothing') alternative. To represent this alternative, hypothetical future sedimentation is assumed to further reduce the conveyance capacity of Easkoot Creek. Additional sediment deposition in Easkoot Creek is presumed to occur more or less uniformly from Arenal Avenue to Calle del Arroyo. The hypothetical sedimentation totals about 1,630 cubic yards with an average aggradation depth of 1.3 feet. This represents slightly more sediment than is estimated to have accumulated during the December 2005 flood and as such is a reasonable estimate of the future deposition that may be expected to occur over the next decade or so if no actions are taken.

Bridge Improvements
This alternative considers modifications to or replacement of existing bridges over Easkoot Creek. Hydraulic modeling assumed the replacement of nine of the twelve bridges in the study area that were shown to significantly restrict flow in order to demonstrate potential flood mitigation benefits. Many options short of replacement of nine bridges are possible however, and the alternative prioritizes the bridges in terms of their expected flood mitigation benefits and discusses the design constraints, permitting issues, and costs associated with modifying or replacing each of the various types of bridges in the study area.

Vegetation Management
This alternative investigates the potential for flood mitigation resulting from reducing roughness on the channel banks through a program of vegetation management. A hypothetical reduction in bank roughness of 25% was assumed for all reaches where existing vegetation contributes to elevated roughness values.

Dredge and Sediment Management
This alternative consists of removing 3,100 cubic yards of sediment from a 2,300 foot reach of Easkoot Creek between Arenal Avenue and Calle del Arroyo. The average excavation depth was 2.4 feet, and the dredging plan was based on restoring the historical longitudinal profile of the channel as indicated by a survey from 1979. In combination with the dredging, sediment removal structures are proposed at two locations upstream of State Highway 1 to reduce future sedimentation in lower Easkoot Creek and maximize the effectiveness of dredging over a longer period of time. Habitat enhancement in the dredged reach is proposed, and potential habitat enhancement in two reaches upstream of Arenal Avenue is identified to provide for potential mitigation of potential impacts of dredging if necessary.

Comment [T14]: Does this include provisions for delivery of sediment to the littoral system?

Comment [COM15]: No, but it won't remove all the sediment in the system so sediment delivery will continue.

North Bypass
This alternative involves the construction of a bypass channel to divert a portion of the discharge of Easkoot Creek away from flood-prone lower reaches during high flow conditions. The proposed diversion point is located on the left bank of the channel opposite the Parkside Café, and the diverted water flows through a 50-ft wide by 3-ft deep trapezoidal bypass channel, discharging to a detention basin located in the vicinity of the north GGNRA parking lot.

Diversion would begin when flows exceeded approximately 40 cfs meaning that the bypass would be active approximately one to four times a year.

South Bypass and Poison Lake Restoration

This alternative is similar to the North Bypass except that it would also include the restoration of pond and wetland habitat in the vicinity of historical Poison Lake. High flows would be diverted to the restoration area located in the vicinity of the south GGNRA picnic area. The proposed restoration area covers approximately 2.4 acres and would create a range of habitat conditions ranging from a seasonal wetland to a perennial pond with depths on the order of two to four feet.

Causeway

This alternative involves the construction of a causeway over Bolinas Lagoon to connect State Highway 1 with Seadrift Road along the alignment of what is currently a gravel road named Walla Vista Road. The primary purpose of this alternative would be to improve access to the Seadrift community which relies on Calle del Arroyo as the only means of vehicular access; this route currently becomes submerged during moderate floods. The optional construction of a tide gate and pump station as part of the causeway design was also investigated in order to evaluate the potential flood mitigation benefits of these structures.

Calle del Arroyo

This alternative involves elevating the entire length of Calle del Arroyo between State Highway 1 and Seadrift Road, a distance of approximately 2,840 feet. The primary purpose of this alternative would be to improve access to the Lower Calles, Patios, and Seadrift community which all rely on Calle del Arroyo as the only means of vehicular access. Drainage beneath the roadway is also investigated to avoid exacerbating flooding by elevating the roadway and potentially reduce flooding impacts by arresting the downstream progress of floodplain flows.

Combination Dredge and South Bypass

This alternative combines the features of the Dredge and South Bypass/Poison Lake Restoration alternatives as discussed above.

Structure Elevation

This alternative involves elevating buildings to remove them from the floodplain. Two cases have been considered, one that elevates the 24 buildings that lie within the December 2005 floodplain and one that elevates the 59 buildings that lie within the 100-yr floodplain. This alternative is not presented in further detail in Appendix A, however it may be a cost-effective alternative to mitigating flood impacts.

Comment [T16]: What about removal of the structures?

Comment [COM17]: Again, this is not part of the scope.

Comment [A18]: Cost estimate to be developed with help from District

5. METHODOLOGY FOR ALTERNATIVES ANALYSIS

Nine alternatives were evaluated with the hydraulic models for the historical December 2005 flood. The December 2005 event was selected as the primary evaluation event because it represents a recent historical flood, it was large enough to cause significant flood damage in the watershed, and it is small enough that the alternatives may be expected to provide significant mitigating effects. Four of the twelve alternatives were also evaluated for a 100-yr flood. These alternatives were selected because they proved to have the greatest potential to mitigate flooding.

Comment [T19]: What is used for the base case for the 100-yr flood event alternatives evaluation.

Comment [COM20]: Current conditions are modeled for the 100-year event

A tidal boundary condition equivalent to Mean Higher High Water (MHHW) was used for this analysis. It is important to note that while this analysis does include the effects of tidal forcing on riverine flooding it represents only riverine flood hazards. Given that the lower portions of Easkoot Creek are subject to flooding from extreme tides and coastal storm surge in addition to riverine flooding, final design of many of the alternatives affecting the lower reaches of Easkoot Creek requires completion of a coastal flood hazard study which is beyond the scope of this analysis.

Comment [T21]: It seems that evaluation of the alternatives is compromised by not including the coastal flood hazard.

Comment [COM22]: True, but the FEMA and OCOF studies were running in parallel to this one.

In order to evaluate the effects of sea level rise due to global climate change we also analyzed the December 2005 flood with a tidal boundary condition of Mean Higher High Water (MHHW) plus 18.2 inches of sea level rise. This is the value recommended for use in Marin County riverine flood studies by the August 2012 Technical Memorandum prepared by Marin County staff entitled *Recommended Sea Level Rise Modeling Methodology and Values to be used for Riverine and CIP Flood Studies*. It represents a 2050 sea level rise estimate and is based on a statistical analysis of the range of predicted values given in the 2012 National Research Council's report *Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future*.

This sea level rise condition (MHHW plus 18.2 inches) was evaluated for the one alternative that proved the most promising in terms of potential to reduce flood hazards. The sea level rise analyses performed for this alternative and for existing conditions provided a means of understanding the potential impacts of sea level rise on the effectiveness of the alternatives in general. Sea level rise may be expected to have a larger effect on flooding due to coastal flood processes; hence a full understanding of sea level rise impacts requires completion of a coastal flood hazard study.

Surveyed finished floor elevations (FFE) were available for a significant proportion of the flood-prone areas in the watershed. For these buildings it was possible to directly compare simulated water surface elevations with FFEs to more accurately determine which buildings would become flooded for a given flood event and alternative. For those buildings lacking FFE data, it was assumed that the building was flooded if floodwaters adjacent to any part of the building had depths of 0.5 feet or greater. It is important to note that the tabulation of the number of flooded buildings presented in Section 3 includes auxiliary buildings such as garages and sheds in addition to residential and commercial buildings.

In addition to tabulating the number of flooded buildings for each event and alternative, flood mitigation effects are presented in terms of the change in peak water levels within the active channel of Easkoot Creek for various reaches as well as through a series of maps depicting the change in flood extent and floodplain depths. The following notes may be helpful to assist in interpreting these flood maps:

- areas where flooding would be completely eliminated are shown in light green
- areas where flood depths decrease are shown in light blue
- areas where flood depths do not change significantly are shown in dark blue
- areas where flood depths increase are shown in red

Alternatives were also evaluated with respect to design and construction options and constraints, estimated implementation and maintenance costs, and permitting considerations including a preliminary assessment of impacts to endangered fish (steelhead trout and Coho salmon). These evaluations are in the form of narratives that attempt to identify and describe salient aspects of the different potential approaches to flood mitigation.

While we attempted to be thorough, the wide range of options and the uncertainty associated with the permitting process made it difficult to present salient considerations at consistent levels of breadth and depth. Nevertheless, these narratives provide a relatively detailed starting point for further analysis and preliminary design work for the most promising alternatives.

All alternative designs presented here should be considered conceptual (30%) designs and are provided for planning purposes only. All cost estimates presented here should be considered planning level estimates. More refined cost estimates for a given alternative can be developed following completion of more detailed design studies.

6. SUMMARY OF RESULTS

Flood mitigation results have been summarized in several ways including tables showing the change in peak water levels within Easkoot Creek (Tables ES1 and ES2), tables showing the number of buildings removed from the floodplain (Tables ES3 and ES4), and maps indicating the simulated changes in flood extent and depths on the floodplain (provided for each alternative in Appendix A). The water level change results have been summarized for three reaches where overbank flows are most prevalent, the reach adjacent to the Parkside Café, the reach between Calle del Pinos and Calle del Arroyo (Upper Calles), and the reach between Calle del Arroyo and Calle del Occidente (Lower Calles). Following is a summary of the effectiveness of alternatives with respect to flood mitigation.

The Combination Dredge and South Bypass Alternative provides the greatest flood mitigation benefits. Flooding is completely eliminated for the December 2005 event with the exception of a small stretch of Calle del Arroyo near Calle del Ribera. All twenty four buildings are removed from the December 2005 floodplain (Table ES3). For the 100-yr event, flooding above Calle del Pinos is completely eliminated. Only minimal reductions in flood extent occur within the Calles, however floodplain depths are reduced substantially throughout much of this area. Approximately twenty-three of fifty-nine buildings (39%) are removed from the 100-yr floodplain (Table ES4).

This alternative was also evaluated for the December 2005 event under a 2050 sea level rise condition. These results indicate that the mitigating effects of the alternative would remain above the Calle del Arroyo bridge but be somewhat diminished throughout the Lower Calles. Farther downstream the higher tidal condition results in overtopping of Calle del Arroyo in several reaches which increases the flood extent to a similar level as was simulated for existing conditions with sea level rise.

The Dredge Alternative completely eliminates flooding above Calle del Onda for the December 2005 event. Only limited changes in flood extent and floodplain depths occur in the Lower Calles. Approximately eighteen of twenty-four buildings (75%) are removed from the December 2005 floodplain (Table ES3). Flood extents are significantly reduced for the 100-year flood and significant reductions in floodplain depths occur throughout the Upper Calles. In the Lower Calles only minimal changes occur. Approximately seven of fifty-nine buildings (12%) are removed from the 100-yr floodplain (Table ES4).

The North Bypass and South Bypass alternatives provide a similar level of flood protection. Flooding is completely eliminated above the Calles and flood extents and floodplain depths are reduced significantly throughout both the Upper and Lower Calles for the December 2005 flood. Both alternatives remove approximately eleven of twenty-four buildings (46%) from the floodplain (Table ES3). Only minor decreases in flood extent are achieved during the 100-yr event, however floodplain depths decrease significantly throughout the study area and thirteen of fifty-nine buildings (22%) are removed from the 100-yr floodplain (Table ES4).

The Bridge Improvements Alternative appears to be relatively effective, however, this tentative conclusion results from modeling removal of all the bridges that constrain flood flows. Evaluating the effects of modifying individual bridges or groups of bridges was beyond the scope of this analysis; further hydraulic model runs should be conducted to evaluate these effects in a subsequent stage of project planning. That said, bridge modification as modeled eliminates flooding above the Calles with the exception of small overbank flows on the right bank adjacent to the Parkside Café during the December 2005 flood. Minor decreases in flood extent and significant decreases in floodplain depths occur throughout the Upper Calles, however only minor changes occur in the Lower Calles. Approximately eleven of twenty-four buildings (46%) are removed from the floodplain (Table ES3). Only minor decreases in flood extent are achieved during the 100-yr event, however floodplain depths decrease significantly throughout the study area and seven of fifty-nine buildings (12%) are removed from the 100-yr floodplain (Table ES4).

The remaining alternatives do not significantly reduce peak water levels, flood extents, or floodplain depths and remove at most two buildings from the December 2005 floodplain (Tables ES1 and ES3). The No Project Alternative increases flood extents and floodplain depths significantly throughout the upper reaches of the creek above Calle del Onda. This results in the addition of eight buildings (increase of 33%) to the floodplain.

Additional considerations regarding the overall feasibility of flood mitigation alternatives relative to preliminary design constraints, estimated construction costs, likely permitting issues (including fish habitat impacts), operation and maintenance requirements and estimated costs, and sustainability are discussed in detail for each alternative in Appendix A. These considerations are also summarized in Table ES5.

Table ES1. Average change in peak water levels for the December 2005 flood event for the various alternatives.

Alternative	Average Change in Water Level (ft)		
	Parkside Café	Upper Calles	Lower Calles
No Project	+0.5	-0.1	-0.2
Bridge Improvements	-0.3	-0.2	0.0
Vegetation Management	0.0	0.0	-0.1
Dredge and Sediment Management	-2.6	-1.1	0.0
North Bypass	-0.6	-0.4	-0.4
South Bypass and Poison Lake Restoration	-0.6	-0.4	-0.6
Causeway	0.0	0.0	-0.1
Calle del Arroyo	0.0	0.0	0.0
Combination Dredge and South Bypass	-3.6	-2.2	-0.8

Table ES2. Average change in peak water levels for the 100-yr flood event for the various alternatives (missing values indicate that the alternative was not evaluated for this event).

Alternative	Average Change in Water Level (ft)		
	Parkside Café	Upper Calles	Lower Calles
No Project	-	-	-
Bridge Improvements	-0.1	-0.3	0.0
Vegetation Management	-	-	-
Dredge and Sediment Management	-1.1	-0.2	-0.1
North Bypass	-	-	-
South Bypass and Poison Lake Restoration	-0.5	-0.4	-0.3
Causeway	-	-	-
Calle del Arroyo	-	-	-
Combination Dredge and South Bypass	-3.4	-1.4	-0.1

Table ES3. Number of buildings removed from the December 2005 floodplain under the various alternatives.

Alternative	# of Flooded Buildings	# of Buildings Removed
Existing Conditions	24	-
No Project	32	-8
Bridge Improvements	13	11
Vegetation Management	24	0
Dredge and Sediment Management	6	18
North Bypass	13	11
South Bypass and Poison Lake Restoration	13	11
Causeway	23	1
Calle del Arroyo	22	2
Combination Dredge and South Bypass	0	24

Table ES4. Number of buildings removed from the 100-yr floodplain under the various alternatives (missing values indicate that the alternative was not evaluated for this event).

Alternative	# of Flooded Buildings	# of Buildings Removed
Existing Conditions	59	0
No Project	-	-
Bridge Improvements	52	7
Vegetation Management	-	-
Dredge and Sediment Management	52	7
North Bypass	-	-
South Bypass and Poison Lake Restoration	46	13
Causeway	-	-
Calle del Arroyo	-	-
Combination Dredge and South Bypass	36	23

Table ES5. Comparison of alternatives regarding overall feasibility.

Alternative	Sedimentation	Fish Habitat	Permitting	Cost
No Project	Unmitigated sedimentation leads to significantly reduced channel capacity in < 10 years	Increased risk of stranding on floodplain; in-stream habitat degraded.	None expected	Undetermined costs caused by future flood damage to private and public property
Bridge Improvement	Modest local change at modified and unmodified bridges possible	Somewhat reduced risk of stranding on floodplain; minimal change to in-stream habitat;	Modest requirements, localized project impacts	Varies with number and type of bridges modified; estimated total for 3 highest priority bridges is $173,000; estimate for 9 bridges in $3.93 million
Vegetation Management	Minimal change expected	Minimal change expected	Similar to existing District maintenance program (modest)	Similar to existing District maintenance program (modest)
Dredge & Sediment Management	Reduced rate of sedimentation and reduced impact on conveyance due to near term future sedimentation; improvement temporary unless maintained by on-going sediment management including potential future dredging	Much reduced risk of stranding on floodplain; disturbed habitat may be improved by enhancement actions and implementation methods; habitat improvement in lower Easkoot temporary. Upstream sedimentation facilities could improve habitat.	Major permitting involving Federal and State agencies because of direct impacts to stream with endangered species habitat; likely EIR/EIS and individual permits from CDFW, RWQCB, ACE; Biological Opinion from NMFS. Project permit should provide for future dredging and other sediment management	Construction and design costs not including EIR/EIS estimated at $830,000. Substantial annual operation and maintenance for new sediment management activities.
North Bypass	Some redistribution of sedimentation expected-decreased potential near Arenal Avenue and increased potential near Calle del Mar; new sedimentation possible in bypass channel	Risks to fish lost to bypass are relatively high. An alternative path to the ocean is provided for fish. Reduced flooding lowers probability of stranding for other fish	Major permitting involving Federal and State agencies because of direct impacts to stream with endangered species habitat; likely EIR/EIS and individual permits from CDFW, RWQCB, ACE; Biological Opinion from NMFS	Construction costs not developed in detail, but could be considered of similar magnitude to South Bypass & Poison Lake Restoration

Comment [T23]: Decommissioning costs should be accounted for in some way.

Comment [COM24]: For the no project alternative? This study definitely didn't address all components of each alternative. It is meant to be a preliminary assessment with some basis to help the community make decisions. Additional studies and refined cost estimates are needed to finalize any alternative.

Table ES5. Comparison of alternatives regarding overall feasibility (continued).

Alternative	Sedimentation	Fish Habitat	Permitting	Cost
South Bypass & Poison Lake Restoration	Some redistribution of sedimentation expected- decreased potential near Arenal Avenue and increased potential near Calle del Mar; new sedimentation possible in bypass channel and restored Poison Lake.	Risks to fish lost to bypass are relatively low, or beneficial due to potential high quality rearing habitat in restored Poison Lake; reduced flooding lowers probability of stranding of fish not entrained in bypass.	Major permitting involving Federal and State agencies because of direct impacts to stream with endangered species habitat; likely EIR/EIS and individual permits from CDFW, RWQCB, ACE; Biological Opinion from NMFS.	Construction and design costs not including EIR/EIS estimated at $1.39 million. Substantial annual operation and maintenance for new flow and sediment management activities. Unknown cost may be large; expect potentially significant additional cost for wetland restoration in GGNRA .
Causeway	No effect on sedimentation expected.	No effects expected.	Significant permitting involving Federal and State agencies because of direct local impacts to tidal wetlands.	Cost estimate not fully developed; minimal flood mitigation predicted.
Calle del Arroyo	No major effects expected.	Some potential reduction in floodplain stranding.	Impacts along existing right-of-way may affect estuarine fringe and wetlands or channel in some locations indicates substantial permitting but likely modest impacts.	Construction and design costs not including EIR/EIS estimated at $1.00 million.
Combination Dredge & South Bypass	See above	See above	See above	Combined cost of individual alternatives about $2.2 million.

7. DISCUSSION AND CONCLUSIONS

Overall the most effective alternative with respect to flood mitigation appears to be the Combination Dredge and South Bypass Alternative. The Bridge Improvements, Dredge, North Bypass, and South Bypass alternatives are about equally effective. The Dredge Alternative removes more buildings from the December 2005 floodplain but less than the others from the 100-yr floodplain, and the two bypass alternatives result in improvements that extend downstream to the Lower Calles reach whereas the Bridge and Dredge alternatives do not. The remaining alternatives result in only minor improvements, and the No Project Alternative is the only alternative that exacerbates flood hazards.

Although they do not result in significant reductions in peak water levels or the number of flooded buildings, both the Causeway and Calle del Arroyo alternatives reduce flood hazards by improving access for residents of the lower watershed during flood conditions. Of these two options, the Calle del Arroyo alternative would improve access to the lower Calles, Patios, and Seadrift areas whereas the Causeway Alternative will only improve access to Seadrift when Calle del Arroyo is flooded.

While similarly effective from a flood control stand-point as modeled, the South Bypass Alternative would provide significantly more fisheries benefits than the North Bypass Alternative and would likely face less design and permitting hurdles and have less impacts on existing GGNRA facilities and parking. In addition, the North Bypass Alternative routes flood water towards the Upper Calles neighborhood, creating more potential flooding in events exceeding design capacity. In contrast, the South Bypass routes flood waters away from residential and commercial areas and does not create a secondary flood hazard.

Comment [T25]: Is there a parking analysis?

Comment [COM26]: No, just recognition of the parking issue.

After the Combination Dredge and South Bypass alternative, the Bridge Improvements, Dredge, and South Bypass alternatives are the most effective at relieving flooding. These three alternatives represent very different flood control approaches, the Dredge Alternative is effective because it increases conveyance in the creek, the Bridge Improvement Alternative is effective primarily because it removes obstructions from the creek, and the South Bypass alternative is effective because it reduces the discharge reaching the lower flood-prone reaches of the creek.

All three approaches were shown to be about equally effective, however there are some distinct differences in terms of sustainability. Although sediment management activities can be implemented to reduce sediment inputs to the lower reaches of the creek, sedimentation is expected to be an ongoing problem at the base of the Easkoot Creek alluvial fan. In its present alignment, zones of sedimentation appear to occur at the northward bend of Easkoot Creek as it arrives at the GGNRA property and turns northwest towards the Parkside Café and Calle del Mar, and after it passes under the entrance road to the GGNRA beach parking lots. It is anticipated that the increases in conveyance achieved through dredging will gradually decline over time, and that flood events with recurrence intervals of about 10 years are likely to cause excessive sedimentation. That being said, current channel capacity has been reduced so dramatically by recent sedimentation that dredging is a viable alternative. Provided that bridge decks are elevated sufficiently above the channel banks, much of the benefits achieved by removing these obstructions from the flow field should continue regardless of anticipated future sedimentation.

Alternatives for bypassing flood flows may be considered the most sustainable options. Bypass alternatives significantly reduce discharges in the lower flood-prone reaches which will continue to provide flood control benefits regardless of future changes that may occur in the lower system. Nevertheless, the bypass alternatives do not eliminate sedimentation issues. It is recommended that development of significant sedimentation facilities located upstream of State Highway 1 that can be routinely dredged be given serious consideration as one of the only means available to reduce long-term sedimentation and its contribution to flood hazards in lower Easkoot Creek.

Comment [T27]: Do we have details on where and what these would be?

Comment [COM28]: Suggested locations are included in the alternatives.

Examination of the hydraulic modeling results for flooding under existing conditions reveals that as peak stream discharge and flooding increases, more water exits the channel along the left bank of the creek in the vicinity of the Parkside Cafe. When these overbank flows become large enough water begins to flow to the ocean in the vicinity of the northern GGNRA parking lot (approximately where the North Bypass Alternative routes flow) and the wetland located in the vicinity of historical Poison Lake (where the South Bypass Alternative routes flow). Results for the No Project Alternative suggest that if channel capacity continues to decrease this process will be enhanced. These results suggest that the current system has a ~~natural~~ tendency to bypass flows away from the lower reaches of the creek as flows increase and/or channel capacity is reduced. Thus the bypass alternatives can be viewed as a way to manage this process and maintain better control over the fate of overbank flows.

Our analysis of 2050 sea level rise impacts suggests that the mitigating effects of the alternatives will remain approximately the same above Calle del Arroyo, but will be diminished below this point. Under the sea level rise condition, Calle del Arroyo becomes overtopped in several areas which results in significant increases in flooding. The only alternatives that are likely to assist in mitigating against sea level rise impacts are the Causeway and Calle del Arroyo alternatives. It is important to note that the sea level rise analysis only considered Mean Higher High Water (MHHW) conditions and sea level rise is expected to have a larger impact on coastal flooding processes which have not been evaluated to date.

With respect to impacts on fish habitat, both the Dredge and Bypass Alternatives appear to have the potential to substantially alter habitat conditions. Regulatory permits to implement these alternatives would require that potential negative impacts be avoided or mitigated. There appear to be feasible means of avoiding negative impacts to habitat as well as means of actively enhancing habitat conditions. Determining project designs and conditions that would meet regulatory requirements is difficult, particularly when multiple regulatory agencies have jurisdiction and where complex environmental conditions and impacts are involved. Further effort is required to map and plan the regulatory permitting process of selected Alternatives at the outset of more detailed planning work.

APPENDIX A-ALTERNATIVES DESCRIPTIONS

Easkoot Creek Hydrology & Hydraulics Study

Prepared for

Marin County Flood Control and Water Conservation District

Prepared by

O'Connor Environmental, Inc.

In Association with

Garcia and Associates
Robert Zlomke, PE
Professor John Largier, UC Davis

March 20, 2013

DRAFT

No Project Alternative

This alternative represents the "no project" scenario in which the significant additional sediment deposition is presumed further reducing channel capacity. The amount of sediment deposition is comparable to what can occur as the result of one large flow event with a recurrence interval of approximately 10 years. Hypothetical deposition was presumed to occur from the upstream boundary of the study area at State Highway 1 through a point ~280 feet downstream of Calle del Arroyo below which the analysis of historical longitudinal profiles indicates relative stability of the channel bed. (Refer to the "Dredge" alternative or the technical memorandum "Task 3b-Sediment Transport Analysis" for a more comprehensive discussion of historical deposition and changes in bed elevations). The total reach length over which deposition was presumed to occur is approximately 2,800 feet. The average depth of deposition relative to the 2011 OEI profile is 1.3 feet with a maximum change of 3.0 feet (Figure A1). The total deposition volume represented by the alternative is approximately 2,100 cubic yards. This volume represents somewhat more sediment than is estimated to have accumulated during the December 2005 flood and as such is a reasonable estimate of the future deposition that may be expected to occur over the next decade or so if no sediment management actions are taken.

Hypothetical sedimentation was found to result in substantial increases in flood extent and floodplain depths throughout the reach extending from the Parkside Café through the upper Calles (Figure A1). A significant portion of the additional floodplain flow generated by the reduced conveyance through this reach inundates the GGNRA property, primarily in the vicinity of the north parking lot. Water breaches the sand dunes and overflows to the ocean at two locations: near the northwest corner of the north parking lot, and at the existing outfall of the remnant of historical Poison Lake north of the overflow parking lot (Figure A1). Decreases in flood extent and decreases in floodplain depths on the order of 0.1 to 0.25 feet occur downstream of Calle del Arroyo. These decreases in flooding can be attributed to increased diversion of flow through the GGNRA parking lots to the ocean, however, this also increases floodplain flow between the beach and Easkoot Creek in the upper Calles where flooding is more severe. These changes in flow and flood patterns ultimately result from reduced channel conveyance caused by hypothetical sedimentation. Peak water levels in the channel increase by as much as 1.6 feet in the upper reaches of the creek but show slight reductions on the order of 0.1 to 0.2 feet throughout the lower reaches (Table ES1). Despite the mitigating effects of increased ocean outfalls, the overall effect is a significant increase in flood risk with an estimated eight additional buildings brought into the December 2005 floodplain raising the total from twenty-four to thirty-two (Table ES3).

With respect to fish habitat, it is expected that additional sedimentation in lower Easkoot Creek would further degrade spawning and rearing habitat. Potential spawning habitat would be expected to be diminished by increased deposition of fine sediment as channel definition and flow confinement decreases. Surface flows during the summer base flow period would likely diminish, and the extent of dry channel would likely increase, thereby reducing the quantity and quality of rearing habitat available. Juvenile fish are also subject to moving into the floodplain, either actively (i.e., moving into the floodplain in search of foraging opportunities) or being passively carried into flooded areas by currents. Once in the floodplain, these fish are subject to being stranded, unable to return to the main channel. The potential loss of juvenile fish to the ocean during flood events is therefore proportional to the degree of flooding, and would likely increase with increasing floodplain flows and ocean outfall via the parking lots.

Comment [T1]: If the alternatives are being compared to the 2005 event, it would be helpful to show the modeled extend and depth of inundation for that event. It would also be helpful to include the topographic base map.

Comment [cc2]: Final version includes 2005.

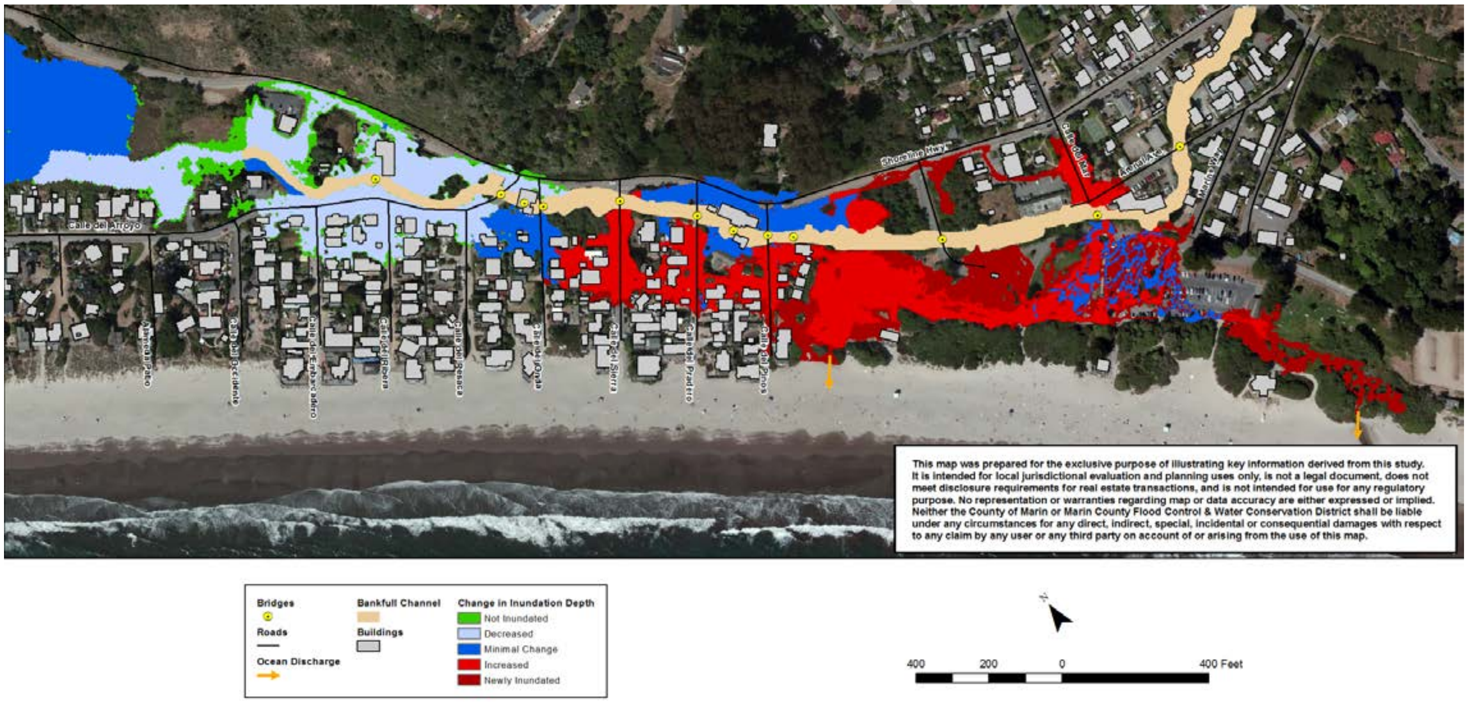

Figure A1. No Project Alternative -- Change in ~~F~~flood extent, floodplain depths, and number of buildings removed from the floodplain ~~under No Project alternative~~ compared to ~~for~~ the December 2005 flood.

Comment [T3]: Clarify – what are the base conditions for determining the Changes in Inundation Depth? December 2005 flood?

Comment [cc4]: Updated

Comment [T5]: GLOBAL -- Clarify titles of these figures.

Comment [T6]: Darren – Do we have the topo base that was used for these simulations?

Summary: Bridge Improvement Alternative

The Easkoot Creek drainage contains twelve bridges in the area under study (Table A2). No two structures are exactly alike. The bridges primarily restrict flow via interaction of the water surface with the bridge decks; flow restriction due to decreases in channel width associated with bridge abutments is of only minor significance. Several of the bridges present a significant flow restriction in moderate flood events like December 2005 resulting in overbank flows occurring immediately upstream of the bridges which in turn results in downstream floodplain inundation. This effect is most pronounced at bridges 1, 3, 4, and 8 which become submerged by more than a foot during the December 2005 flood (Table A2); these four bridges are therefore distinguished as “priority 1” for modification (Figure A2). Hydraulic modeling results indicate that bridges 2, 5, 6, 10, and 12 have sufficient capacity for a December 2005 magnitude flood, but only bridge 10 has sufficient capacity for a 100-yr flood (Table A2).

Bridge improvements provide significant flood mitigation in some areas. For the December 2005 event, flooding upstream of the North Lot is eliminated except for minor overbank flow near the Parkside Café (Figure A3). Minor decreases in flood extent occur throughout the Upper Calles; floodplain depths decrease by 0.1 to 0.5 feet in residential areas of the Upper Calles (Table ES1). Approximately 11 of 24 buildings are removed from the December 2005 floodplain (Table ES2). During the 100-yr flood, flood extent is slightly reduced and floodplain depths are reduced by 0.1 to 0.5 feet throughout the Upper Calles (Figure A4). Peak water level reductions are relatively minor overall (Table ES3), however significant reductions of 0.6 to 0.8 feet occur upstream of several of the bridges (Park Footbridge, Footbridge above Calle del Pinos, Calle del Pinos, Calle del Sierra, and Calle del Onda). Approximately six of fifty-nine buildings are removed from the 100-yr floodplain (Table ES4).

Bridge impacts of flooding could be reduced by raising the existing supports for individual structures above the design high water levels (e.g. 100-yr water surface), which can be accomplished by raising bridge decks about two feet. The required incremental elevation has been determined through hydraulic modeling, and varies to an extent at individual bridges depending on channel width and depth and likelihood of future bed load deposition at the site. Bridges 1, 2, 3 and 4 are located low in the watershed within the zone of tidal influence, and as such are also subject to coastal flooding (any final design for modifications to these lower bridges should consider coastal flood hazards which were not addressed as part of this study).

Improvement of vehicle bridges is problematic. Each bridge project presents some unique complications in design and construction sequence. Most of the bridges serve dead-end streets with no alternative access unless temporary roads can cross the beach to adjacent roads. Necessary construction activity and would be disruptive to local traffic and State Highway 1. Permitting issues are substantial, but probably not a major obstacle.

Preliminary estimated cost for improvements to the nine bridges ranked as priority 1 or 2 is significant, about $3.9 million. Improvement to priority 1 bridges alone, excluding the “house bridge” at Calle del Arroyo, has a much lower estimated cost of about $173,000. Cost estimates are summarized for each bridge in the table below. Detailed breakdowns of estimated costs are provided in Appendix C.

Table A1. Summary cost estimate for bridge improvement.

Bridge Alternative - Planning-level Budget Summary	Priority	Cost ($)	Percent
Unit 1 - Lower Footbridge	1	36,773	0.9
Unit 2 - Calle del Arroyo	2	1,222,450	31.1
Unit 3 - House - not evaluated	1	--	0.0
Unit 4 - Calle del Onda	1	67,848	1.7
Unit 5 - Calle del Sierra	2	614,305	15.6
Unit 6 - Calle del Pradero	3	67,848	1.7
Unit 7 - Gym Footbridge	2	37,273	0.9
Unit 8 - Calle del Pinos	1	67,848	1.7
Unit 9 - Footbridge ab. Calle d. Pinos	2	36,773	0.9
Unit 10 - Park Entrance	3	--	0.0
Unit 11 - Parkside Footbridge	2	41,773	1.1
Unit 12 - Arenal Avenue	3	--	0.0
Subtotal Contractor Overhead		749,570	19.1
Planning-level Cost Estimate (to nearest $100)		3,741,900	95.2
Project Administration		187,100	4.8
Installed Project Cost Estimate		**3,929,000**	100.0

Table A2. Overview of the twelve bridges in the study area listed from downstream to upstream including basic dimensions, construction materials, clearance from the bottom of the bridge deck to the channel bed, the December 2005 water surface, and the 100-yr water surface (negative clearances are shows in red and represent a submerged bridge deck), and prioritization in terms of potential to reduce flooding impacts.

ID	Name	Bridge Deck			Girder	Railing	Distance to Cross Street (ft)	Clearance (ft)			Priority
		Width (ft)	Length (ft)	Material	Material			To Bed	To Dec. 2005 Water Surface	To 100-yr Water Surface	
1	Footbridge below Calle del Arroyo	4	25	wood	wood	wood	-	3.0	-1.1	-1.8	1
2	Calle del Arroyo	30	17	paved	mono box	steel	36	3.8	0.4	-0.2	2
3	House Bridge	30	20	wood	wood	-	-	2.6	-1.0	-1.7	1
4	Calle del Onda	12	24	wood	wood	wood	27	2.5	-1.5	-2.3	1
5	Calle del Sierra	30	15	paved	mono box	steel	25	3.5	0.1	-1.1	2
6	Calle del Pradero	12	42	wood	steel	wood	45	3.7	0.6	-0.5	3
7	Gym Footbridge	4	15	wood	wood	wood	-	2.5	-0.2	-1.1	2
8	Calle del Pinos	12	25	wood	wood	wood	60	1.8	-1.9	-2.4	1
9	Footbridge above Calle del Pinos	4	20	wood	wood	wood	-	2.9	-0.2	-0.7	2
10	Park Entrance Road	30	33	paved	mono box	steel	150	4.3	1.3	1.1	3
11	Park Footbridge	6	15	wood	wood	fenced	10	2.6	-0.4	-1.2	2
12	Arenal Avenue	30	25	paved	mono box	steel	20	3.6	0.9	-1.3	3

Note: The Park Footbridge is also referred to as Calle del Mar, and provides access to the beach from central Stinson Beach at the Parkside Café.

Figure A2. Bridge improvement overview.

Figure A3. ~~Decrease~~ Bridge Improvement Alternative -- Change in flood extent, floodplain depths, and buildings removed from the floodplain compared to ~~under the Bridge Improvement alternative for~~ the December 2005 flood.

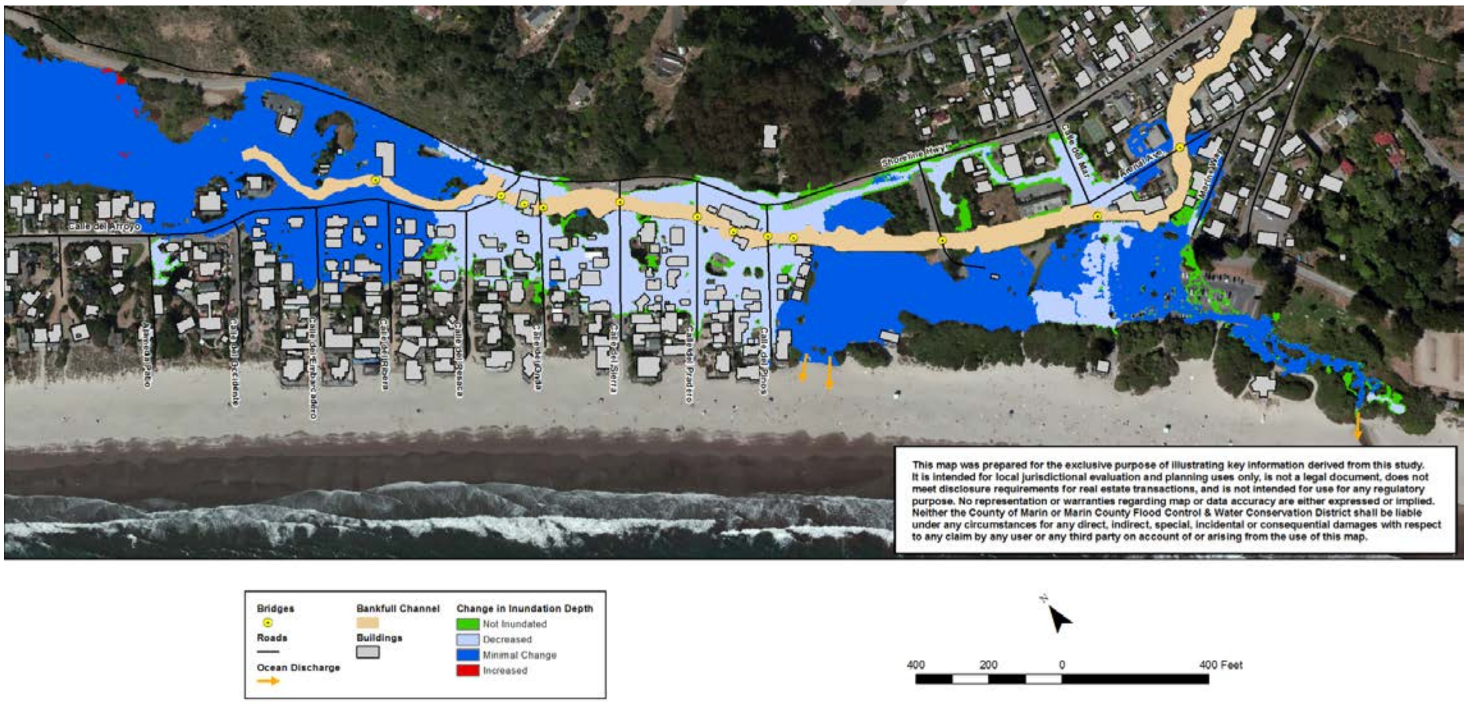

Figure A4. Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the Bridge Improvement alternative for the 100-yr flood.

Comment [cc7]: Updated.

Comment [T8]: Clarify what the base condition is for this 100-yr flood comparison.

ALTERNATIVE: Bridge Improvements

1. Description

This alternative considers modifications of existing bridges over Easkoot Creek for flood mitigation. It is relatively easy to assess the effects of bridges on flooding and the likely reduction in flooding if bridge decks were high enough to remain above water level. It is more difficult to conceptualize potential bridge modifications and their costs. Following in this section is a general description of these bridges, including their construction, condition and relative impact on flooding.

The Easkoot Creek drainage contains twelve bridges in the area under study (Table A2). Bridges 1, 7, 9, and 11 are narrow wooden footbridges and appear to be located on private property. Bridges 4, 6, and 8 are structurally similar in nature; 4 and 8 employ wood girders, 6 at about twice the other's length uses steel I beam girders for deck support. Two are on concrete footings, and one appears to be supported on horizontal wood timbers. Bridges 2, 5, 10, and 12 are structurally similar in nature. They are monolithic concrete box structures oriented at various angles to the channel, support two-way traffic, and are paved.

No two structures are exactly alike. All are one-off designs and were likely constructed individually over time. Due to the age of the community, most were likely installed decades ago. Based on the fairly new appearance of decking and structural materials on many of the wooden units, they are likely to have been individually upgraded over time, or repaired in response to flood damage.

County archives have not been consulted with regard to bridge permitting history or construction. Permitted construction either in the public or private sector should have resulted in archival documentation of construction drawings at a minimum, and perhaps would include relevant design materials as well. Most structures support or are immediately adjacent to one or more exposed utilities trunk lines that cross the channel. Water mains seem to be the most prevalent, although other improvements such as gas lines, underground power, or other utilities cannot be ruled out.

Significant bed load deposits are present adjacent to and under the decks of many of the bridges. Clearance between the bottom of the bridge deck and the streambed is less than 2 feet at bridge 8, between two and three feet at bridges 3, 4, 7, 9, and 11, between three and four feet at bridges 1, 2, 5, 6, and 12, and more than four feet at bridge 10 (Table A2). Bridge 10 has bed load deposits under the deck, and observed scour with exposed piling at the north east foundation corner. Gabion structures now in place were apparently retrofitted to reduce risk of additional scour. The gabions constrict cross sectional area of the channel under the bridge.

Hydraulic modeling results indicate that bridges 2, 5, 6, 10, and 12 have sufficient capacity for a December 2005 magnitude flood, but only bridge 10 has sufficient capacity for a 100-yr flood (Table A2). Bridges 1, 2, 3 and 4 are located low in the watershed within the zone of tidal influence, and as such are also subject to coastal flooding (any final design for modifications to these lower bridges should consider coastal flood hazards which were not addressed as part of this study).

The bridges primarily restrict flow via interaction of the water surface with the bridge decks; flow restriction due to decreases in channel width associated with bridge abutments is of only minor significance. Several of the bridges present a significant flow restriction in moderate flood events like December 2005 resulting in overbank flows occurring immediately upstream of the bridges which in turn results in downstream floodplain inundation. This effect is most pronounced at bridges 1, 3, 4, and 8 which become submerged by more than a foot during the December 2005 flood (Table A2); these four bridges are therefore distinguished as "priority 1" for modification.

2. Flood Control Benefits

In order to investigate the potential flood control benefits resulting from altering the bridges to reduce or eliminate interaction between the bridge decks and the water surface, we identified nine of the twelve bridges as priority (bridges 1, 2, 3, 4, 5, 7, 8, 9, and 11 shown as priority 1 or 2 in Table A2) based on the severity of the flow restriction. A model scenario was evaluated with these nine bridge decks completely removed from the model. While complete removal of the bridges may not be feasible, the scenario is designed to represent a best case improvement of these bridges whereby the bridge decks are elevated sufficiently to avoid any interaction with the water surface, and sufficient drainage is provided to avoid restricting floodplain flows with any fill that may be required for bridge approaches. Additional model runs are necessary to examine the effects of different combinations of bridge improvements.

Improving the bridges leads to significant flood control benefits in some areas. During the December 2005 flood, flooding upstream of the North Lot is completely eliminated with the exception of minor overbank flow on the left bank in the vicinity of the Parkside Café (Figure A3). Minor decreases in flood extent occur throughout the Upper Calles and floodplain depths decrease by 0.1 to 0.5 feet throughout the residential areas of the Upper Calles. The average reduction in peak water levels in the channel is 0.3 feet in the reach adjacent to the Parkside Café, 0.2 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.0 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table ES1). While these changes are relatively minor, water levels upstream of several of the bridges (Park Footbridge, Calle del Pinos, and Calle del Onda) are reduced by as much as 0.7 to 1.0 feet significantly reducing the volume of overbank flow associated with bridge constrictions (Table ES1). These reductions in overbank flow result in approximately 11 of 24 buildings being removed from the December 2005 floodplain under this alternative (Table ES2).

During the 100-yr flood, the total flood extent is only slightly reduced from existing conditions, however floodplain depths are reduced by 0.1 to 0.5 feet throughout the Upper Calles (Figure A4). Similar to the results for the December 2005 flood, average peak water level reductions are relatively minor overall (Table ES3), however significant reductions of 0.6 to 0.8 feet occur upstream of several of the bridges (Park Footbridge, Footbridge above Calle del Pinos, Calle del Pinos, Calle del Sierra, and Calle del Onda). The associated reductions in overbank flow above the bridges results in the removal of approximately six of fifty-nine buildings from the 100-yr floodplain (Table ES4).

It is important to keep in mind that the modeling results represent complete removal of nine of the bridges and as such may tend to over-state the expected flood control benefits of bridge improvement since it is whether all nine bridges can be modified to eliminate flow restrictions.

3. Preliminary Design and Estimated Construction Costs

Bridge impacts could be eliminated by complete removal of the bridges. This is not believed to be a feasible option in the case of vehicle bridges. In the case of foot bridges, it may be somewhat feasible, depending on legality, ownership, and use factors of existing bridges. In the case of bridge 3 where an uninhabited house is situated over the creek, it may be a feasible if the structure is illegal and non-conforming, and creates a flooding hazard in the neighborhood.

Bridge impacts could be reduced by combining functionality of bridges to reduce total bridge count. This option would be feasible if the community was under initial development, because road and street design could be undertaken in consideration of the riparian corridor, channel, and floodplain. A small number of larger bridges could be used instead of a large number of smaller bridges. This option is not likely to be feasible due to the historic and fixed nature of development of streets and parcels.

Bridge impacts could be reduced by raising the existing supports for individual structures above the design high water levels (e.g. 100-yr water surface). The required incremental elevation has been determined through hydraulic modeling, and varies to an extent at individual bridges depending on channel width and depth and likelihood of future bed load deposition at the site. Raising individual bridges by about two feet above their present elevation would satisfy this objective.

Bridge impacts could be reduced by replacement of individual bridges with specialized structures providing greater separation between deck structural supports and the design water surface elevation.

- For conventional flat bridges, this would entail incorporation of low-profile, high-strength, low-deflection structural elements, and use of low-profile bridge decking (i.e. steel plate versus planks). The (likely modest) incremental height savings could be used to raise the structure without appreciably raising the existing deck elevation.
- Non-conventional approaches such as arched bridges might also be considered. These would be highly atypical of conventional construction, and would require special engineering for design. The individual support elements for an arched structure are inherently unstable and would need to be properly constrained to prevent torsion or tendency to rotate to a lower energy state. The arched configuration also transmits horizontal loads to the supporting foundation structure, so that re-engineering of the existing passive vertically-loaded foundations may be required as well.

Design Considerations

Bridge removal would not require design work per se. Some planning and permitting would be required as to the mechanics of removal. There may need to be redevelopment of the individual sites in terms of riparian stabilization or enhancement. Resource agency involvement would likely mirror the discussion provided in the Dredging Alternative scenario, although at a much reduced scale and on a site-specific basis.

Bridge raising would require design of the approaches, relocation of utilities, and assessment – repair – reuse of existing abutment structures. Structural engineering would be required to ensure adequate tiebacks and connections for re-use of existing supports. The option of installation of "legs" to the ends of the bridge structure versus modification of the foundation would need to be considered. Bridge raising is not feasible for bridges 2, 5, 10, and 12, all of monolithic concrete box tube design. Since deck and foundation are integrally cast, these units would require demolition and reconstruction in an alternative configuration.

Bridge approach planning would be part of the design consideration. Approaches would need to be configured so as to respect property limits. In some cases, engineered retaining walls would likely be required to stay within lateral constraints of the available property. Knowledge of underground utilities in the area is required for retaining wall design. High groundwater in the lower section of the work area may add to the complexity of design and construction.

The present discussion assumes simple reuse or replacement of existing structures. It may be that some structures are undersized, inadequately designed, or suffering from structural degradation and unsuitable for re-use. Part of the re-use effort should go towards evaluating adequacy of the existing structure(s). Reused or replaced structures should conform to current codes and current and projected traffic demands, both from a practical matter, and from an Agency risk management standpoint.

Bridge redesign would require type selection and then detailed design of individual unit(s). Based on the variety of configurations now present in the field, it may not be possible to apply a single approach to redesign in which one design would serve for multiple installations. Design considerations would include selection of an appropriate load rating, design style, Fire Service constraints, individual components, connections, foundations, seismic considerations, hydrologic considerations, deck and wear surface, guard railings, pedestrian considerations, utilities relocations, esthetic considerations, costs, permitting constraints, and related items.

Prefabricated bridges with pre-engineered design could be considered, and would include proprietary designs available commercially or built up rail car units. Rail car bridges can be economical in certain situations, but do not appear to be appropriate in these applications. Single units are typically not wide enough to meet Code requirements, and require two units welded longitudinally in order to provide for vehicle and pedestrian needs. Foundation requirements would be similar to those required for stick-built girder-supported assemblies. The rail car girder section is typically 2 to 3 feet deep, so that the advantage of a low-profile configuration would be lost. Rail cars are a byproduct of another industry, and so may not be readily available in a preferred configuration.

Abutment and approach design would involve installation of a wedge of material in order to achieve the required elevation gain using a ramped surface. Per verbal County direction, a maximum slope of 15% would be allowed. This amounts to a 1.5 foot rise per 10 feet of length and a 13.3 foot approach for a 2 foot bridge height increment. The nearest cross street is close to that distance from the bridge in some cases (Table 1), so geometric constraints may limit allowable bridge height. A ramp at 15% slope is not ADA compliant. This may not be a factor for vehicular road design, but would need to be further investigated on a case-by-case basis.

Prudent design also utilizes vertical curves so that the transition between horizontal and ramp on the approach and exit is moderated. A 25 foot vertical curve is the normal minimum requirement for low-speed roads. Using that standard, the approach length would need to be nearly 50 feet long to account for the positive and negative inflections between existing ground and the elevated bridge. Since this distance is not available in most cases, a more sharply inflected section would be required. Such curves may deviate from local code requirements.

Raised abutments that are either earth-fill construction or structural members would need to be fitted with appropriate guard rails, particularly if a vertical wall is used at either side. New utilities connectors with flexible connectors would be required in cases where they are attached to the existing bridge.

Based on the results of the hydraulic modeling we have classified the bridges from priority 1 through priority 3 based on the flood control benefits expected from raising or replacing the bridge. Bridges 1, 3, 4, and 8 are considered highest priority and bridges 2, 5, 7, 9, and 11 are considered second priority (Table 1).

Assumptions for preliminary cost estimates vary by bridge type. Costs of foot bridges, evaluated using standard procedures, were estimated assuming simple lift to higher elevation using hand labor methods.
Cost for private vehicular wood-decked bridges, also evaluated using standard procedures, were estimated assuming simple lift to higher elevation using hand labor methods, and include new abutments, gravel ramps, extended railings, and paved approaches. Estimated costs for monolithic concrete box culvert bridges were estimated based on construction date and installed cost, which is brought to present-day values by applying a cost of living factor.

4. Permitting Issues

Impacts of bridge modifications on fish habitat and other ecological resources are expected to be modest. Although construction activity would occur in riparian areas, floodplains, and possibly in stream channels, impacts would be largely temporary and would not be expected to significantly degrade fish habitat. In addition, reducing the frequency and magnitude of flooding would tend to prevent potential threats to aquatic species associated with urban flooding; endangered steelhead and coho salmon would be less likely to be flushed from the channel onto the floodplain. Furthermore, potential dredging at bridges that might occur in conjunction with bridge improvements could increase availability of pool habitat as has occurred after prior dredging. Mitigation for potential impacts of bridge improvement projects on habitat has not been specifically considered, however, as described in the dredging alternative salmonid habitat enhancement could be more effective upstream of Arenal Avenue where sedimentation processes are not overwhelming.

Raising the approach on either side of a bridge requires installation of a triangular wedge of material so that vehicles may be positioned to cross the bridge. By definition, each bridge is located in the floodplain, so that creation of the fill wedge using soil or gravel will violate the “No Net Fill” rule within the floodplain. Required fill volumes range from about 1.5 cy for a 5 foot wide approach raised 1 foot to about 31 cy for a 30 foot wide unit raised 3 foot. Incremental fill could be reduced by 0.4 to about 8 cy for the range cited if vertical retaining walls were used for fill containment rather than using earthfill at 2H:1V along the approaches.

Bridge construction may be selectively limited to the top of channel bank, particularly if re-utilization on the same foundation takes place. In that case, work should be exempt from CDFW Stream Alteration Agreement permitting, because it takes place outside the jurisdictional area. Technically speaking, it would also remain exempt from jurisdiction of Army Corps, US Fish and Wildlife Service, Regional Water Board, and related agencies for the same reason. Practically speaking, it would make sense to remove accumulated bed load in under-bridge areas during construction. In that case, the work would become jurisdictional and permitting would be required with all of the resource agencies discussed in the Dredging Alternative scenario.

Replacement bridges would go through a typical design and permitting effort for the structural elements of the bridge. Design would be undertaken by a structural engineer in consultation with the client (public or private) in order to develop a value-engineered solution. Normal County requirements for bridge design would govern the development, and normal County permitting procedures would be utilized for the structural aspects of the construction. Bridge 11 is a footbridge between Arenal Ave. and the beach area. It does not presently meet ADA requirements, but could possibly be subject to such constraints if modified. It is not known if other vehicular bridges would be subject to ADA requirements.

The work of vehicular bridge relocation or replacement becomes a substantial problem from a construction standpoint. Most of the bridges under consideration serve dead-end streets with no alternative access. The urban area is not configured to allow cross traffic between affected streets so that another route is available during bridge work. If a bridge is temporarily decommissioned, dead-end street access will be limited to foot traffic. Emergency vehicle access for fire, police, or ambulance service will not be available unless an alternate route can be made available, possibly temporary roads on the beach linking to adjacent streets during construction.

Relocation of an existing bridge would be most efficiently accomplished by using a heavy lift crane to raise the structure in one move. The unit would then be lowered onto a new or upgraded foundation. This alternative is not likely available in the present circumstance, because work sequencing is a problem. The old foundation could not be adjusted until the existing bridge was lifted out of the work area.

A heavy-lift crane is expensive to operate and takes up the width of the roadway. It is not believed feasible or cost-effective for the crane to stand by for the extended time period required to do the foundation work between old and new bridge settings. Cranes are supported on outrigger foot pads. Work on secondary roads with unknown quality is likely to damage the roadway due to pad imprints. Excessively soft conditions could jeopardize crane operations. Cranes rely on counterbalance weights that are brought to the site on separate trucks and self-assembled by the crane operator. Offloading, assembly, and disassembly will require proximity between vehicles and would need to be worked out with operator's assistance. While likely feasible, at a minimum traffic restriction along Highway 1 would be likely during any work activity. Any crane work undertaken would also need to factor in local overhead power lines and the lift and extension constraints of the boom relative to field positioning at the end of the bridge.

The alternative to a crane lift of the bridge structure is to raise the bridge from below using jacking methods. This requires placement of materials in the channel, invoking permitting issues. Jacking would be relatively slow and labor intensive, and would require shoring for safety purposes. The bridge would remain in the way of any anticipated dredging below the structure during site upgrade. The bridge would remain elevated and unusable until the approaches were completed. The approach on the far side of the bridge would be hard to build, because the work area would remain inaccessible due to the incremental step between bridge and ground.

Development of the approach ramps is also problematic from a construction sequencing standpoint. Ramp construction prior to old bridge removal precludes bridge use for the duration of the construction effort. The foundation assembly supporting the new bridge would need to be a separate prefabricated drop-in unit that attaches to the old foundation to expedite final construction, because the foundation is covered by the old bridge until such time as removal occurs. Ramp construction on the far side of the bridge is problematic as well. If built first, construction access is assured, but the roadway becomes impassible. If built with the bridge out, access is a problem. If built once the bridge has been raised, access remains a problem due to the vertical drop off the raised bridge.

One way around the abutment conundrum would be to construct the approaches as a stand-alone drop-in trestle type arrangement. This would allow prefabricated construction prior to bridge raising or replacement. The trestle sections could be supported on a ground contact skid plate with columns, allowing flow of flood waters through the structure. The trestle could have built in vertical curves to facilitate vehicular traffic. Methods would need to be developed to assure adequate load bearing capacity and resistance to motion in the x and y directions. They would also need to be resistant to differential settlement or rotation so that they would not “walk out” from under the bridge due to cyclical loading or temperature induced expansions and contractions. Local streets would need to be reconstructed to meet strength and geometric constraints. Pinned joints or other kinds of attachments might be necessary to prevent rotation or separation at the abutments. The abutments could be placed relatively quickly once the original bridge was temporarily set aside. The bridge would then be dropped in place on the new abutments. Logistics of bridge movements might be problematic in tight spaces, because the old bridge would need to be out of the way of the new abutment placements. Dimensional tolerances for placements would need to be to the nearest ¼ inch or less, requiring a high degree of precision in assembly and placement. The abutments would likely need to be segmented due to trucking length constraints. They would be brought to the site by truck and staged for unloading and installation. For bridges with widths less than the channel or riparian width, it may be possible to park the bridge in the channel aligned upstream-downstream while the abutments are installed.

If existing bridges were considered unsuitable for reuse for any reason, the old units would be replaced with new structures. The old units would be disassembled on site for salvage or disposal. The footbridges constitute a special case and a general category, as they are narrow, long, and relatively lightly constructed. Each could be fairly easily raised by jacking. The individual approaches could be reconstructed into a stair stepped configuration.

5. Operation and Maintenance Requirements and Costs

A properly designed bridge should have low maintenance requirements. The approximate life of wood-based structures is perhaps 20 years, after which accumulated biological deterioration may require reconstruction or structural repairs. Concrete and steel members should have a longer service life of perhaps 50 years if properly designed and constructed. The bridge decking constitutes an expendable wear surface, whether of wood, concrete, asphalt, or other material. Periodic maintenance would be expected to be necessary to provide satisfactory long-term performance.

6. Sustainability (Short-term and Long-term)

Properly designed and installed bridges should have a reasonable 20-year design and economic life. Selection of materials that are resistant to corrosion and decay would be needed in this moist and corrosive coastal environment.

Bridge improvements would remove flow obstructions from within the floodway and reduces diversion of flow onto the floodplain. Bridge replacement does not address the causes of channel capacity reduction due to sedimentation, however, when flood flows are affected by bridges, it is likely that sedimentation rates increase. Consequently, bridge improvements are likely to reduce local sedimentation observed at some bridges under existing conditions. Should bridge redevelopment be pursued without addressing sedimentation, it is likely that channel capacity will continue to be diminish and the flood mitigation benefits of bridge improvements degraded. Long term flood mitigation is not likely to be achieved through bridge improvements alone.

7. Feasibility and Next Steps (Additional Information Needs)

The hydraulic modeling performed for this alternative represented a 'best case' scenario of removal of nine of the twelve bridges. If a more refined plan for bridge improvements is developed, the new bridge configurations should be evaluated with the hydraulic models in order to gain a clearer understanding of the expected flood mitigation effects. This would likely be necessary for development of bridge designs. The effects of bridge improvements on sedimentation could also be assessed at that time.

As described above, numerous property, design, and permitting issues and complex construction logistics must be resolved to implement bridge improvements. If bridge improvements are to be pursued, the prioritization presented above provides an approach to focus efforts on a sub-set of bridges to reduce complexity of the planning process. Bridges 1, 3 ("house"), 4, and 8 have been identified as the highest priority bridges for improvement. Excluding the "house" bridge, the estimated cost for improvements to the other three bridges is about $173,000.

Summary: Vegetation Management Alternative

1. Description

This alternative investigates the potential for flood mitigation resulting from reducing the density of shrubs and woody vegetation on the channel banks. Dense vegetation on stream banks can create high flow resistance that reduces water velocity and increases flow depth. A channel with more vegetation (higher flow resistance or "roughness") would reach flood stage at a lower rate of stream flow than the same channel with less vegetation. Routine vegetation management to reduce the density of shrubs and woody vegetation within a stream channel is expected to maintain higher channel conveyance (flow capacity). The District performs this type of vegetation management in Easkoot Creek on an annual basis, suggesting that it may be difficult to achieve further reductions in flow resistance associated with vegetation. In addition, this type of riparian vegetation may have significant habitat value for aquatic and terrestrial species, and more aggressive vegetation management might be inconsistent with habitat and aesthetic values.

To evaluate this alternative, it was assumed that bank roughness could be reduced by 25% in most reaches. All bank roughness values were reduced by 25% from the existing condition with the exception of the right bank reach adjacent to the Parkside Café where the existing bank is a concrete or gabion wall with little or no vegetation.

The hydraulic analysis indicates that reducing roughness through a program of vegetation management does not have the potential to significantly reduce flooding impacts. Average changes in water level for this alternative were 0.1 feet or less throughout the study reach for the December 2005 flood (Table ES1). This did not result in significant changes in flood extent or floodplain depths and no buildings were removed from the December 2005 floodplain (Table ES2).

Given the low degree of flood mitigation achieved by this alternative, no further assessment of implementation was conducted.

Summary: Channel Dredge and Sediment Management

Proposed dredging (Figure A5) would remove about 3,100 cubic yards of sediment from a 2,300 ft reach from Arenal Avenue to Calle del Arroyo. The average depth of excavation would be 2.4 feet with a maximum of 3.4 feet (Figure A8 & A9). Mean width of the dredged channel is about 15 ft.

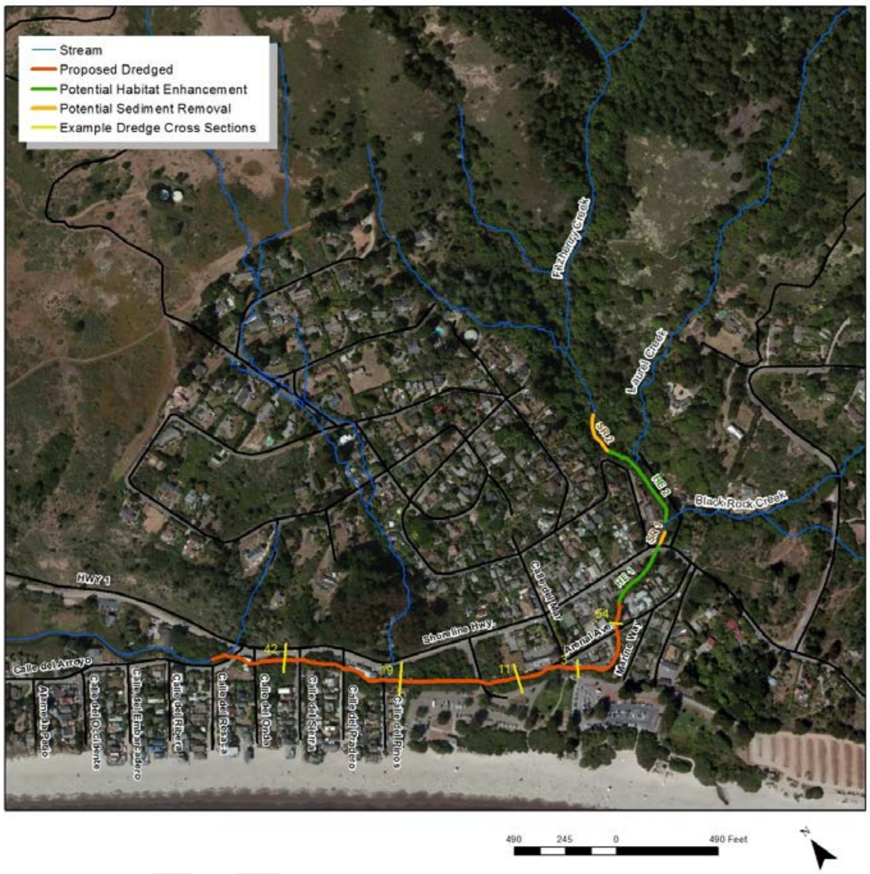

Figure A5. Map overview of proposed dredging scenario.

The flood control benefits of dredging are substantial. In the December 2005 flood, flooding above Calle del Onda is completely eliminated (Figure A6). Below Calle del Onda flood extent and floodplain depths remain approximately the same as under existing conditions owning to the tidal control below Calle del Onda and the fact that the dredge terminates ~280-ft below this point. Approximately eighteen of twenty-four buildings are removed from the December 2005 floodplain under this alternative (Table ES2). During the 100-yr flood, flooding is substantially reduced above Calle del Onda (Figure A7). Below Calle del Onda changes in floodplain depths are minor and even increase slightly in some areas. Flooding at the Arenal Avenue bridge is eliminated and flooding on the right bank just downstream of the Parkside Café is reduced

Comment [T9]: Does the analysis assume that the dredging occurs immediately before the modeled event?

Comment [cc10]: Yes.

substantially such that only street flooding occurs in the vicinity of the intersection of Calle del Mar and Arenal Avenue. Approximately seven of fifty-nine buildings are removed from the 100-yr floodplain under this alternative (Table ES4).

Significant dredging has been required at intervals of less than about ten years owing to episodes of very high stream flow and sediment transport from the upper watershed. The December 2005 flood event deposited about 1,000 to 1,500 cubic yards of sediment, and proposed dredging would remove about 3,100 cubic yards of sediment. Mean annual sedimentation is estimated to be about 122 to 160 cubic yards per year. To reduce future sedimentation that contributes to flooding and to extend the flood mitigation benefits of dredging as long as possible, supplemental sediment removal structures with a capacity of about 290 cubic yards are proposed upstream of State Highway 1. Spot dredging by the District between Arenal Avenue and Calle del Arroyo can remove at least 150 cubic yards. It is likely that a future storm event would cause significant sedimentation of the dredged channel even with a regime of annual sediment removal.

Dredging is feasible, but significant planning/permitting effort is needed because habitat of endangered species (steelhead and coho salmon) will be disturbed by dredging, construction of new sediment removal sites, and habitat enhancement. Substantial mitigation efforts are expected to be required, including proposed habitat restoration in the dredged reach and habitat enhancement upstream of Arenal Avenue in two reaches (Figure A5). Habitat restoration and enhancement objectives are to create both spawning and rearing habitat. Mitigation for disturbance to riparian habitat is also included. Costs for CEQA compliance (e.g. preparation of an EIR, if necessary), are not included.

Table A3. Summary cost estimate for dredging and sediment management.

Design Alternative - Planning-level Budget Summary	**Cost ($)**	**Percent**
Consultant Permitting Subtotal	28,720	3.4
Consultant Assessment & Design Subtotal	101,200	11.8
Construction Subtotal	457,275	53.4
Contractor Overhead Subtotal	124,970	14.6
Riparian Mitigation Subtotal	53,750	6.3
Construction Monitoring and Quality Control	49,200	5.7
Planning-level Cost Estimate (to nearest $100)	815,100	95.2
Project Administration	40,760	4.8
Installed Project Cost Estimate	**855,860**	**100.0**
Estimated Annual Maintenance (remove 150 cubic yards sediment)	18,600	
Estimated Maximum Maintenance (remove 440 cubic yards sediment)	32,800	

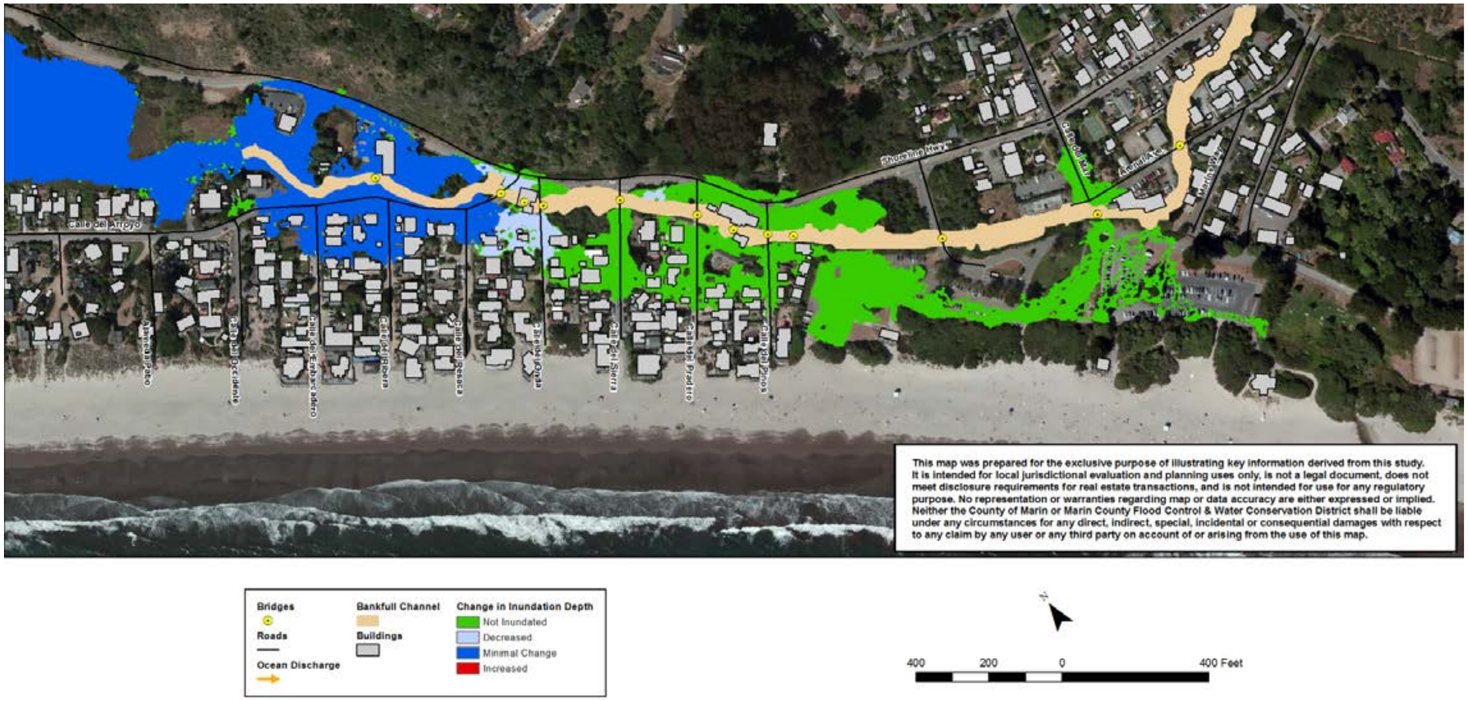

Figure A6. Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under Dredge and Sediment Management alternative for the December 2005 flood.

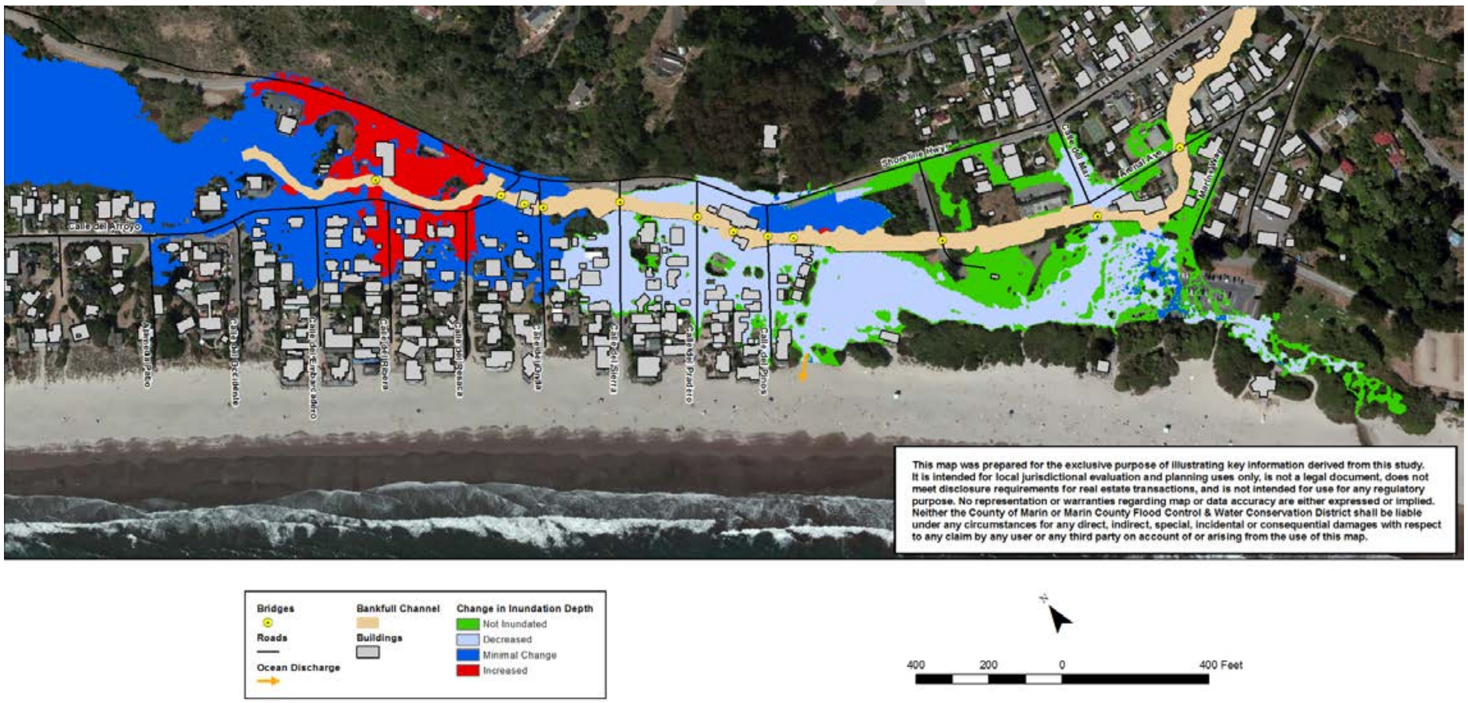

Figure A7. Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the Dredge and Sediment Management alternative for the 100-yr flood.

ALTERNATIVE: Channel Dredge and Sediment Management

1. Description

This alternative combines dredging of the channel with installation and maintenance of sedimentation facilities along the creek.

Historic Dredging

Historic patterns of sedimentation indicate that significant dredging is required at intervals of less than about 10 years owing to episodes of very high stream flow and sediment transport from the upper watershed. The most recent event that caused significant sedimentation was the December 2005 flood which deposited about 1,000 to 1,500 cubic yards of sediment. Widespread channel dredging occurred in 1973, 1983, 1987, and 1997, and probably also occurred in 1955. In 1973, 4,000 cubic yards of sediment may have been excavated; about 1,500 cubic yards was removed in 1987. Elevated sediment transport rates associated with high flows and accelerated watershed erosion combine to cause high rates of sedimentation in reaches of Easkoot Creek in Stinson Beach beginning near Arenal Avenue. The mean annual sedimentation rate estimated for the period 1979-2011 is 122 yds/yr, and may be as much as 160 yds/yr if poorly-documented dredging in 1983 is incorporated in the estimate. Recent dredging by the District from 2007-2009 averaged about 100 cu. yds./yr (Table A4). Subsequent sediment deposition through 2011 was insufficient to fill pools created by dredging at bridges along the Calles, indicating low sedimentation rates in recent years. Provided that peak flows are modest (approximately < 5 yr recurrence interval) and watershed erosion rates are not accelerated by large-scale mass wasting, it appears that the existing dredging program, although limited, can remove sediment volumes approximately equal to the average sedimentation rate.

Table A4. Summary of District "Spot" Dredging Volumes (cubic yards)

Year	Arenal Ave.	Calle del Mar	Calle del Pinos	Calle del Pradero	Calle del Sierra	Calle del Onda	Calle del Arroyo	Total
2007	0	0	37	26	0	26	0	100
2008	55	1	52	53	0	0	0	161
2009	35	0	0	0	0	0	0	35

Proposed Dredging

Proposed dredging (Figure A5) would remove about 3,100 cubic yards of sediment in a single effort. This mass dredging event would be supported by maintenance dredging at the locations currently spot dredged (see Table A4), at a proposed site upstream of Calle del Mar currently being developed by the District, and two potential supplemental sediment removal reaches proposed upstream of State Highway 1.

Channel topography for a dredged channel was based on the 1979 FEMA profile of Easkoot Creek, which was used as a template and modified.[1] The FEMA profile was blended into the existing bed elevations from the 2011 OEI Survey at the upstream and downstream ends of the dredged reach. The upstream boundary of the proposed dredging begins 260 feet downstream of the State Highway 1 Bridge and extends to a point 280 feet downstream of Calle del Arroyo. The total length of proposed dredging is approximately 2,300 feet. The average depth of excavation relative to the 2011 profile would be 2.4 feet with a maximum of 3.4 feet (Figure A8). Mean width of the dredged channel is about 15 ft. Dredging would produce about 3,100 yds of material, primarily sand and gravel.

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Comment [cc12]: Fixed

2. Flood Control Benefits

The flood control benefits of dredging are substantial. During the December 2005 flood, flooding above Calle del Onda is completely eliminated (Figure A6). Below Calle del Onda flood extent and floodplain depths remain approximately the same as under existing conditions owning to the tidal control below Calle del Onda and the fact that the dredge terminates ~280-ft below this point. The average reduction in peak water levels in the channel is 2.6 feet in the reach adjacent to the Parkside Café, 1.1 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.0 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table ES1). Approximately eighteen of twenty-four buildings are removed from the December 2005 floodplain under this alternative (Table ES2).

During the 100-yr flood, flooding is substantially reduced above Calle del Onda (Figure A7). Below Calle del Onda changes in floodplain depths are minor and even increase slightly in some areas. The small increases can be attributed to the effect that the increased conveyance in the upper dredged reach has in terms of more effectively moving water downstream to the lower reach and estuary. Flooding at the Arenal Ave. bridge is eliminated and flooding on the right bank just downstream of the Parkside Café is reduced substantially such that only street flooding occurs in the vicinity of the intersection of Calle del Mar and Arenal Avenue (Figure A7). Floodplain depths in the north Park Service parking lot (North Lot) are reduced by 0.5 to 1.0 feet. Flood extent and floodplain depths are reduced by 0.25 to 0.5-ft in the reach between the North Lot and a point just upstream of Calle del Onda (Figure 3). The average reduction in peak water levels in the channel is 1.1 feet in the Parkside Café reach, 0.2 feet in the Upper Calles, and 0.1 feet in the lower Calles (Table ES3). Approximately seven of fifty-nine buildings are removed from the 100-yr floodplain under this alternative (Table ES4).

The sedimentation basins are not expected to have significant direct flood control benefits and were not evaluated with the hydraulic models. The sedimentation basins are however considered necessary to extend the increased capacity of the dredged channel and the associated flood control benefits of the dredge as long as possible.

Comment [T13]: Is there an estimate of the volume of sediment that would be collected in the basins rather than removed from the channel? This seems important for the comparison of the sub-alternatives that involve initial dredging.

Comment [cc14]: Would have to be analyzed further if alternative is pursued.

[1]Surveys of the channel profile in subsequent years (1999, 2004) in the NPS reach showed little change relative to the 1979 profile. It is understood that dredging of the channel after the flood event in 1982 was substantial, and probably explains why the channel profiles were similar in 1979, 1999 and 2004. Surveys in 2006, 2007 and 2011 showed that the channel profile slope above 5 ft AMSL (approximately the elevation of high tides) was largely unchanged relative to earlier profiles, but that the bed aggraded by about 2 to 3 ft, apparently because of sediment deposition during the flood event on Dec. 31, 2005. We adopted the 1979 profile as our model for proposed dredging because of the evidence of relative stability (absent large sedimentation events) of this profile.

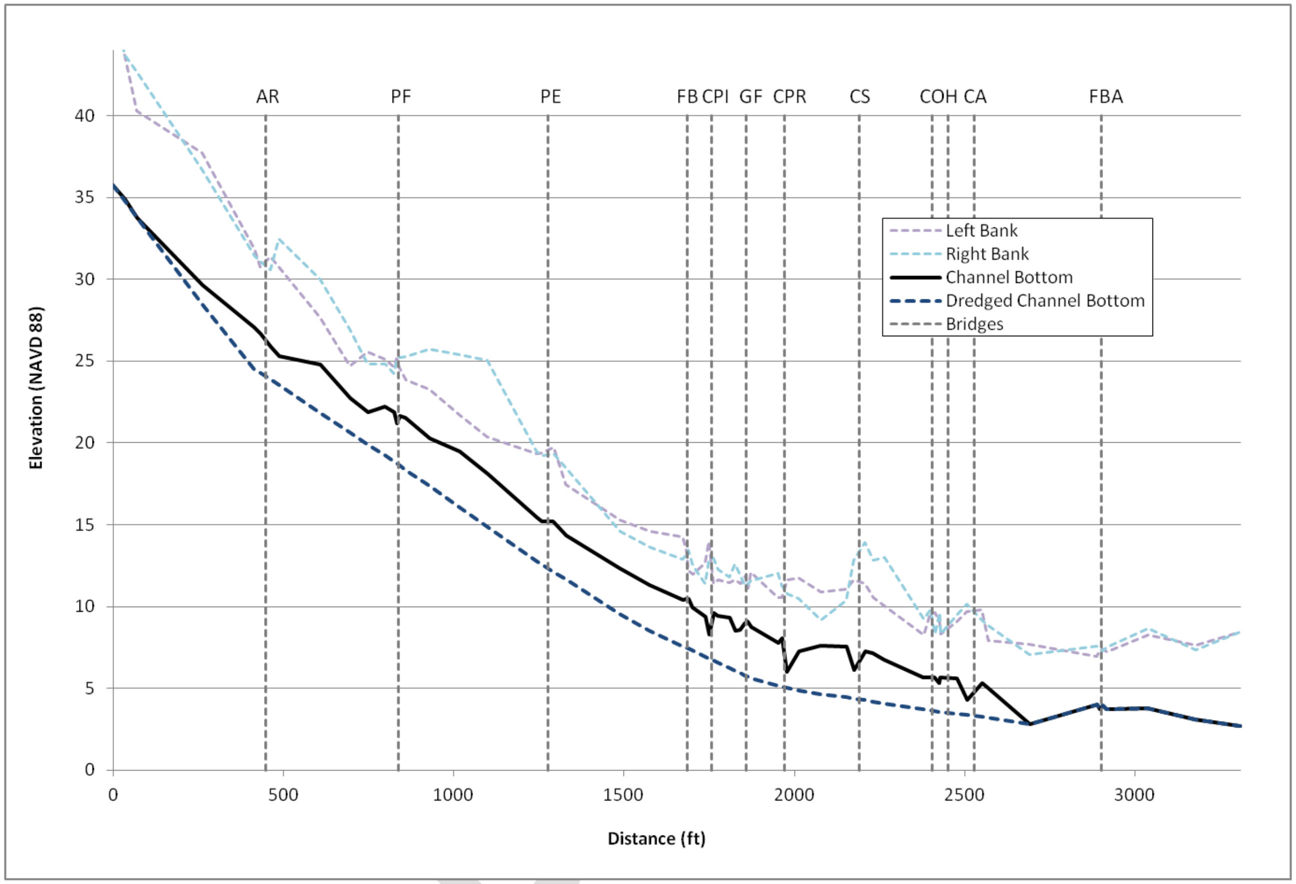


Figure A8. Longitudinal profile of the existing and dredged channel of Easkoot Creek. Vertical dashed lines indicate the positions of the bridges for reference. Bridges are labeled as follows: AR (Arenal Ave.), PF (Park Footbridge), PE (Park Entrance Rd.), FB (Footbridge Above Calle del Pinos), CPI (Calle del Pinos), GF (Gym Footbridge), CPR (Calle del Pradero), CS (Calle del Sierra), CO (Calle del Onda), H (House Bridge), CA (Calle del Arroyo), and FBA (Footbridge below Calle del Arroyo).

3. Preliminary Design and Estimated Construction Costs

Preliminary Design: Dredging of an open, unencumbered channel with no aquatic resources is a simple matter of accessing the site with earthmoving equipment, removing excess sediment accumulation, shaping banks, and providing necessary erosion controls. However, none of these conditions are present in the segment of Easkoot Creek now under consideration for dredging and supplemental sediment removal sites. It is anticipated that significant efforts in design, construction and maintenance of the dredging plan and sediment removal sites will be required to minimize potential adverse impacts on the aquatic ecosystem, particularly the habitat used by endangered anadromous salmonids.

The dredged channel begins about 260 feet downstream of the Highway 1 bridge and extends to a point about 280 feet downstream of Calle del Arroyo (Figure A5). This location corresponds to the transition from the fluvial sand and gravel deposits of Easkoot Creek to the estuarine silt and clay bed previously identified by the NPS[2] as well as the downstream extent of sedimentation (Figure A8). The total length of dredged channel is approximately 2,300 feet. The average depth of excavation was modeled at 2.4 feet with a maximum offset of 3.4 feet from existing conditions. Mean width of the dredged channel is 15 ft; representative cross-sections of the dredged channel as modeled are shown in Figure A9. Additional geotechnical analysis may be required to evaluate bank stability in the context of dredging. Dredging would remove about 3,100 cubic yards of silt, sand and gravel.

The initial design concept and installation methods are driven by regulatory constraints regarding endangered anadromous fish species. The proposal is to excavate to -3.0 ft and spoils would be off-hauled to a designated reprocessing or disposal site. Excavation methods are to be determined, based on access, permitting, and habitat impact issues. Traditional methods (mini-excavator, backhoe, dump truck) would be most cost-effective if access is readily available. Alternative methods (vacuum truck, hand methods, skyline yarder, etc.) may be considered for inaccessible reaches or where habitat values preclude entry or operation with traditional equipment. Methodology would be developed in consultation with affected landowners, resource consultants, regulatory agency staff and the District. Hand methods are likely required where headroom is constrained under some of the eight bridges within the work area. About 310 cubic yards of gravel and cobble would be returned to the channel for restoration of pool-riffle sequences. The replaced material would be screened and washed native material or new material, depending on value engineering considerations. Provisionally, cost estimates are based on importing new material to the site.

It is assumed that the proposed depth of dredging will not grossly destabilize stream banks throughout the dredged reach; this assumption is based on matching the dredged channel profile to past channel bed profiles and that bank stability has not been reported to be a major problem in the past. Nevertheless, some bank revetments are present in the reach, suggesting that bank stability issues are locally significant. Bank heights are typically two to six feet under existing conditions, and bank heights will be two to three feet higher after dredging. Additional analysis of potential bank stability problems that may result from dredging will be necessary during the permitting phase of a dredging program. Bank stabilization measures could be

[2] Fong, D. (2002) Fisheries Assessment for Bolinas Lagoon Tributaries within the Golden Gate National Recreation Area, 1995-2000. Prepared for the National Park Service, Golden Gate National Recreational Area, Division of Natural Resource Management and Research. Feb. 2002, p. 45.

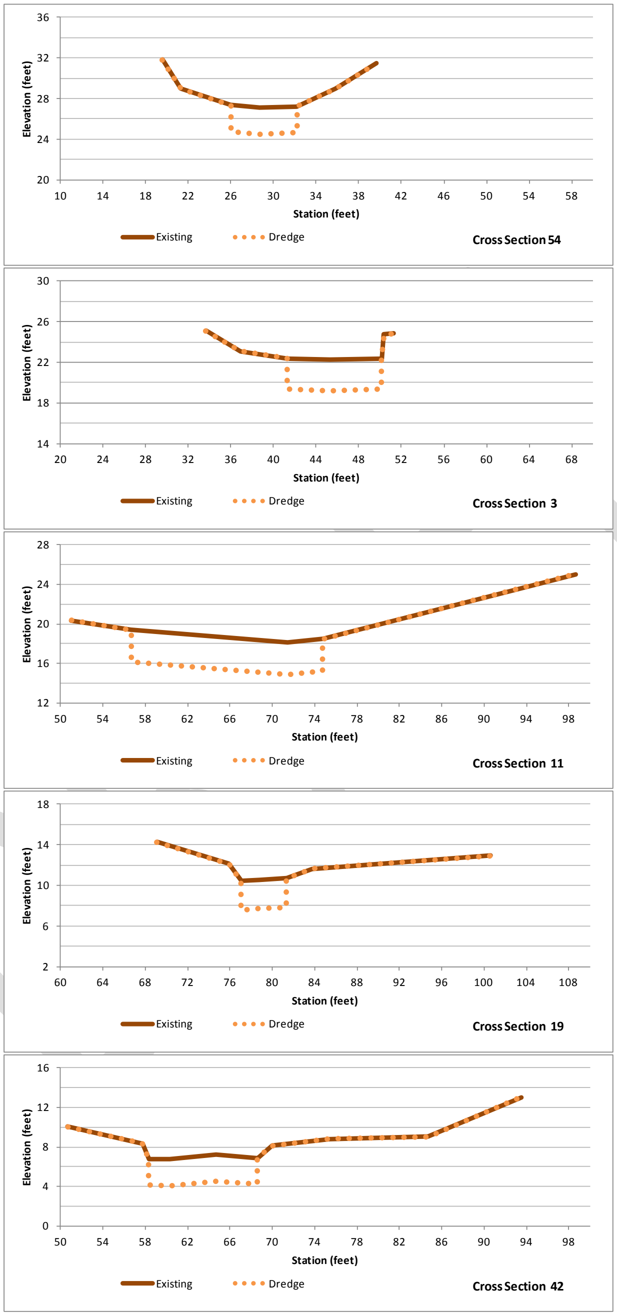


Figure A9. Representative channel cross-sections used in hydraulic model of proposed dredging scenario. Locations shown in Figure A5.

required in some areas, and existing revetments may affect site specific dredging activity. Previous surveys by NPS found about 540 ft of revetments in lower Easkoot Creek, most of which (354 ft) is in the concrete retaining wall and gabions (72 ft) located near Calle del Mar footbridge and the Parkside Café. An additional length of 66 ft of sacrete/sandbag and 53 ft of rip-rap revetments were also reported; these are believed to be located behind the commercial buildings between Calle del Pinos and Calle del Pradero.

Potential Habitat Restoration and Enhancement: After dredging, stream bed restoration in the dredged reach may be required to mitigate expected impacts on salmonid habitat. Habitat restoration in the dredged area could include replacement of a surface layer of channel substrate comprised of coarse gravel and cobbles at re-created pool-riffle sequences suitable for spawning and rearing. Eight individual pool-riffle sequences distributed at intervals between Calle del Onda and Calle del Mar might be appropriate as restoration. The restored channel would be expected to have comparable or better habitat for spawning because the proportion of finer sediment (sand and fine gravel) will be lower relative to coarse gravel and cobbles used to recreate the streambed after dredging is complete. Over a period of years, the habitat quality in the dredged reach would likely be degraded by deposition of finer sediment, including substantial quantities of sand < 1 mm diameter.

If necessary, potential habitat enhancement could occur in two reaches: the first (HE1) between Arenal Avenue and State Highway 1 and the other (HE2) extending from the confluence of Black Rock Creek (between the Community Center and the Fire Station) upstream to the Matt Davis Trail bridge (Figure A5). Each of the potential enhancement reaches is about 400 ft in length.

Supplemental Sediment Removal: Channel elevation maintenance would be comprised of continuation of spot dredging practiced in recent years by the District, supplemented by new sedimentation structures. One new sedimentation structure being planned by the District is located upstream of the Calle del Mar pedestrian bridge. The District structure is expected to induce deposition primarily by widening the channel in a zone where historic sedimentation and sediment deposition potential is high. Two additional sets of proposed structures would be placed at two locations upstream of State Highway 1 (Figure A5). The first (SR1) would be immediately upstream of the State Highway 1 bridge adjacent to the fire station; the second, higher capacity site (SR2) would be located upstream of the Matt Davis Trail pedestrian bridge on Belvedere Avenue. Each of these would require a permanent access trail suitable for equipment (excavator or backhoe) needed to construct and maintain sediment removal structures. Access road development will require landowner approvals.

Historic patterns of sedimentation indicate that significant dredging has been required at intervals of less than about 10 years owing to infrequent episodes of very high stream flow and sediment transport from the upper watershed. The December 2005 flood event deposited about 1,000 to 1,500 cubic yards of sediment, and proposed dredging would remove about 3,100 cubic yards of sediment. Mean annual sedimentation is estimated to be about 122 to 160 cubic yards per year. Supplemental sediment removal sites are proposed to reduce future sedimentation that contributes to flooding and to extend the flood mitigation benefits of mass channel dredging as long as possible. With only existing “spot” dredging capacity and a new sedimentation basin near Park Side Cafe, it is unlikely that widespread channel sedimentation can be prevented during large storm events with recurrence intervals of about 10 years, despite evidence that mean annual sedimentation rates are comparable to combined dredging capacity at “spot” locations with or without the proposed Park Side basin.

Comment [T15]: Is this in addition to the one proposed on NPS land near the Parkside?

Comment [cc16]: No, this is the one by Parkside.

Comment [T17]: Is the proposed sediment basin on NPS property a given for this alternative?

Comment [cc18]: No, but it is in the permit phase.

To avoid creating migration barriers affecting endangered salmonids and other aquatic organisms, designs will maintain a low flow channel consistent with the existing channel slope and profile, including considerations of channel morphology and habitat quality. At SR1 channel slope is about 1.2%, but increases to about 8% just upstream. At SR2 mean channel slope is about 5.5%. The channel morphology of the steeper portions of Easkoot Creek is a series of step pools and cascades[3], with generally shallow pools and few potential spawning beds. This channel morphology is typified as a stair-step profile, with short, steep drops over boulder "steps" or "dams" alternating with relatively flat channel segments with gravel-cobble substrate. Step-pool morphology has well-defined steps of boulders and/or wood debris and relatively uniform spacing with intervening zones of sediment deposition, whereas cascades have a more chaotic structure of boulder steps with smaller and irregular deposition zones.

Proposed sediment removal structures will be comprised of a series of partial weirs constructed of rock about 2.5 ft high spanning the channel width, but maintaining a 2 ft wide gap accommodating a low flow channel (see conceptual designs in Figures A10 and A11). The placement of the gap in successive structures at SR2 will be off-set from the channel centerline to lengthen the flow path and promote sediment deposition. Under most flow conditions, flow would be accommodated within the gap in the partial weir. During the largest flows, the weirs will carry flow across the broad crest of the rock structure. It is intended that these structures have maximum potential to induce deposition of sand and gravel during relatively rare periods of high (and deep) flow, approximately ≥ 5 year recurrence interval, when transport of bed load sediment from the upper watershed occurs at relatively high rates and when downstream sedimentation potential is greatest. The gaps in the partial weirs will also establish the location of the low-flow channel by maintaining a zone of high velocity flow capable of excavating a channel. The location and design of sediment removal structures must permit periodic (annual or nearly so) access by equipment to excavate accumulated sediment. Maintenance activity would attempt to avoid disturbance of the low flow channel, focusing on excavating accumulated sediment on the broader surfaces between the banks, the weirs and the edge of the low flow channel except in the aftermath of major sedimentation events that fill available deposition zones.

SR1 (Figure A10) is proposed upstream of the State Highway 1 bridge (Figure A5) in a sixty foot reach adjacent to Fire Station #1 where the channel slope is about 1.2 %. Two weirs would be spaced at thirty foot intervals. Estimated sediment storage potential of thirteen cubic yards per weir adds a total estimated sediment storage potential of twenty-six cubic yards at SR1. Beginning just upstream of the Matt Davis Trail bridge, SR2 is proposed to extend 200 ft upstream with a series of ten partial weirs (Figure A11). Bed slope in this area is about 5.5%. To increase sediment capture potential, we propose to increase the width of the sedimentation design flow channel to about forty feet by excavating adjacent terraces (Figure A11). These weirs would be spaced at twenty foot intervals, and each weir has estimated sediment storage potential of twenty-seven cubic yards. As shown in Figure A11, the upper and lower weirs are narrower, representing the need to expand and contract the series of structures to conform with channel widths upstream and downstream. Total sediment storage potential at SR2 is about 260 cubic yards. Total potential sediment storage at SR1 and SR2 combined is about 290 cubic yards, nearly tripling the volume of potential maintenance dredging.

[3] Montgomery, D. and Buffington, J. (1997) Channel reach morphology in mountain drainage basins. GSA Bulletin 109(5)596-611.

The partial rock weirs may also be expected to create channel morphology that is characteristic of these types of channels under natural conditions, with pools developing below the notch in each weir. Potential spawning sites would be expected at the downstream edge of the pool, which in this channel would probably be about halfway between the weirs. Hence, the weirs may be expected to preserve or improve the diversity of habitat conditions and provide additional spawning habitat.

Comment [T19]: Would we want fish to spawn here? If this refers to the configurations in Figures A10 and A11, it appears that the potential spawning sites would be smothered by sediment deposition.

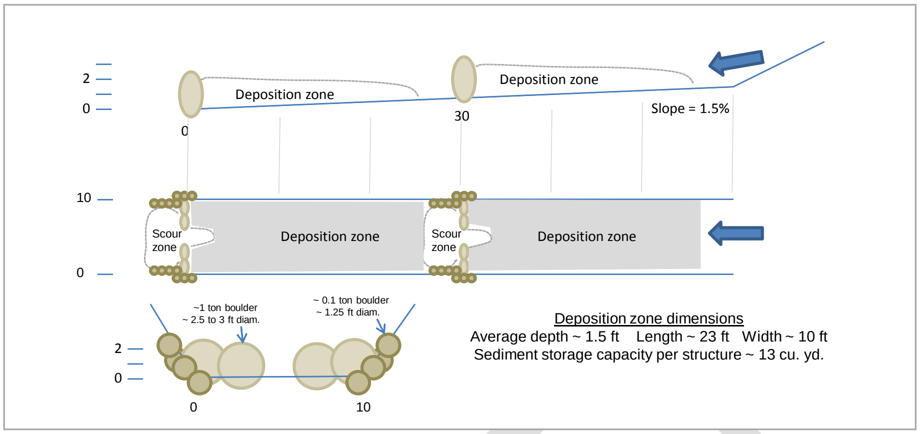


Figure A10. Conceptual design of two partial weirs at SR1.

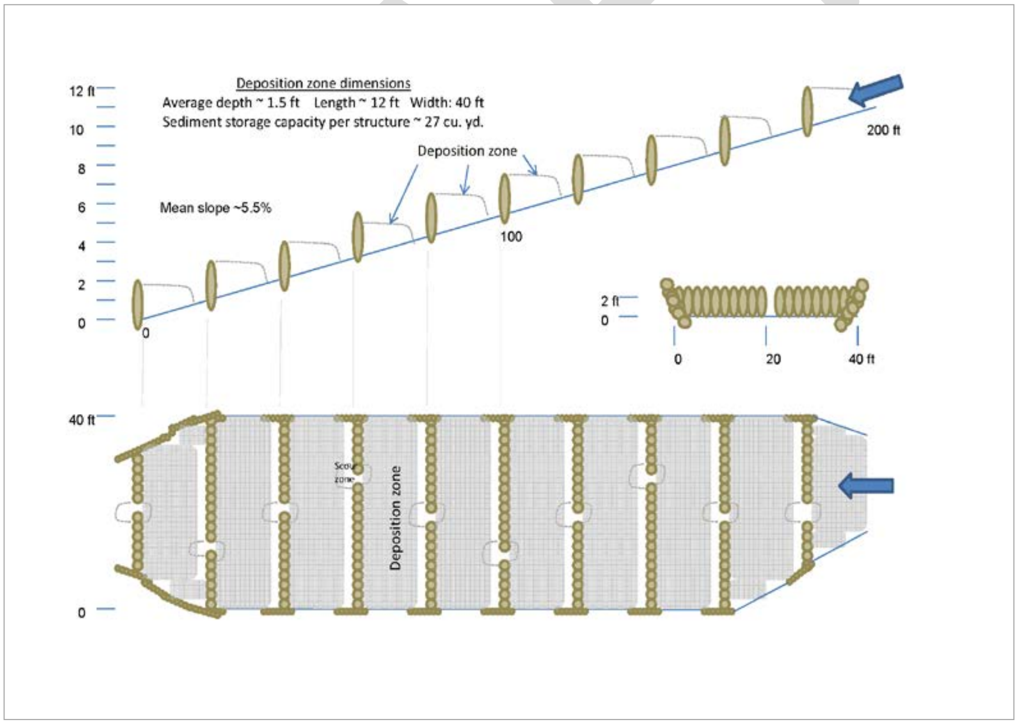


Figure A11. Conceptual design of ten partial weirs at SR2.

Design Considerations: There are many significant constraints involved in the work, many of which have a bearing on dredging design, construction approach, methodology, and cost. Potential issues and constraints follow:

- Dredging Mechanics
  - Direction – upstream versus downstream; biological preference typically to move from downstream to upstream
  - Sequence – single project versus multi-year effort; biological preference likely to pursue as multi-year to limit impacts to critical fish habitat to acceptable level
  - Methodology – Equipment, technique dependent on resource constraints
  - Water diversion – May be required for access, construction.
- Access
  - Definitive surveys of property lines
  - Informal vs. formal approvals
  - Potential impacts on existing easements
  - Avoidance of public utilities
- Infrastructure protection (public, private)
  - Fences
  - Bridges
  - Exposed utilities
  - Underground utilities – water, power, sewer, gas, phone, other
  - Retaining walls
  - Lateral culverts, drainage channels
  - Septic systems
- Outreach and education
  - Public and regulatory agency input on draft plans
  - Design adjustments based on comments and State/Federal law
- Risk Management
  - Bank stability – private and public property
  - Lateral bank scour after work installed
  - Over bank flooding due to boulders, log placements, etc.
  - Infrastructure and utilities protection
  - Stability of bridge piers, abutments, gabions, retaining wall (existing)
  - Aquatic species habitat quality
  - Inadvertent excavation of abandoned refuse dumps and/or hazardous materials
- Vegetation Management
  - Considered as stand-alone option, but also a subset of dredging
  - Protect high quality and/or native riparian vegetation
  - Selective trimming and tree removal for equipment access (equivalent to annual District maintenance)
  - Brush and invasive species management
  - Maintain hydraulic capacity of channel
- Permitting mitigations
  - Invasive species management
  - Canopy management
  - Native species enhancement
  - Lateral floodplain wetland enhancements
- Potential need for bypass of surface flow
- Potential need for groundwater management in lower reaches.
- Channel diverges from roadways and not always readily accessible with equipment.

Comment [T20]: Clarify what "brush management" is. Maybe include it in the previous bullet for access.

Comment [cc21]: Items here are intentionally vague. Would need to be developed further.

Comment [T22]: Suggest deleting or reframing this; according to the evaluation of veg management as a stand-alone option, this doesn't have much effect on flooding.

Comment [T23]: Clarify -- Does this mean planting of native riparian trees to provide shade for instream habitat?

Comment [T24]: Clarify. This sounds very challenging given the proximity to sea level – where and what type of management?

- Work under bridges with limited head space
- Sediment removal (dredging) equipment options
  - Traditional methods: Excavator, skid steer, other
  - Skyline yarder as model for inaccessible areas?
  - Manual labor under bridges
  - Manual labor option throughout
  - Vacuum truck – investigate potential
  - Dump truck access for spoils removal
- Spoils storage yard and spoils use/disposal
  - Determine location and ownership: CalTrans, County, or private
  - Haul distance
  - Clean streets constraints
  - Mud-sand-gravel-cobble segregation
  - Value engineering – native or quarried replacement material
  - Gravel-cobble replacement for fisheries mitigation
  - Removal-disposal of secondary material
- Pre-construction biological survey
- Construction biological monitoring.
  - Observation – identification of vertebrates
  - Potential need to protect/move species
  - Species of concern
    - Steelhead
    - Coho salmon – state endangered
    - Red-legged frog
    - Other….
- Maintain-enhance variable thalweg
- Maintain existing surface roughness
- Maintain-enhance fisheries structures
  - Large woody debris
  - Boulder fields
  - Spawning gravel
  - Overhead canopy
  - Pool-riffle creation
- Bank stability
- Regulatory and permitting approvals
  - CDFW-Section 1602 Streambed Alteration Agreement
  - CEQA
  - NMFS– Threatened or Endangered Species
    - Critical Habitat for Steelhead
    - Essential Fish Habitat for Coho
  - National Park Service – access
  - Regional Water Quality Control Board
    - Section 401
    - SWPPP
    - Waste Discharge Requirements
  - USACOE
    - Section 404
    - Other Waters
    - Wetlands

Comment [cc25]: Agree

Comment [T26]: Include scheduling of vegetation management to avoid violations or constraints of Migratory Bird Treaty Act during nesting season.

Comment [T27]: NEPA for actions on federal land?

Comment [cc28]: Agree

4. Permitting Issues

The National Marine Fisheries Service (NMFS) is responsible for protecting populations of steelhead and coho salmon listed under the federal Endangered Species Act (ESA). It is the responsibility of NMFS to ensure that any actions undertaken are not likely to jeopardize the continued existence of any threatened or endangered species or result in the destruction or adverse modification of habitat of such species (ESA 1973). Therefore, individual steelhead and coho salmon, plus the various habitats that they need to complete their life cycles, need to be maintained or improved during the course of any actions with the potential to affect the habitats in which they live (e.g., implementing flood control measures). Similarly, the California Department of Fish and Game is responsible for protecting steelhead and coho salmon protected under state laws.

The dredging option presented here will need to have safeguards in place to protect both individual fishes present and to reasonably assure that the available habitat continues to meet the functional needs of those fishes, specifically by providing suitable spawning and rearing habitat. Significant habitat enhancement is proposed in two reaches (HE1 and HE2, Figure 1) upstream of Arenal Avenue in this alternative. The proposed dredging plan affects approximately 2,300 feet of Easkoot Creek, or approximately 60 percent of the stream available to steelhead and coho salmon. New sediment removal structures (SR1 and SR2) are also proposed as part of this alternative.

To protect individual fishes living within the affected areas, the project will need to be staged in discreet, manageable-sized units; a comprehensive water diversion plan will also need to be implemented to provide clean, well-oxygenated water to downstream reaches. However, in the event that large stretches of the work area are naturally dry (lacking surface flow), dredging those dry areas will lower the impacts that the overall dredging operation will have on individual fishes, instream habitats, and the overall ecology of Easkoot Creek. Resident fishes will need to be removed (probably via electro-fishing) and relocated to areas upstream of the dredging operation, and fish exclusions will need to be maintained to prevent re-colonization of work areas during dredging operations.

The proposed dredging scenario can be implemented in such a way that habitat complexity (a key aspect for salmonid rearing) is maintained or enhanced. It is important that the resulting channel be allowed to function as a stream, and not be reduced to a homogenous channel. The restoration concept described above includes reconstruction of the stream bed with a layer of clean gravel and cobble, as well as construction of riffle-pool units for spawning and rearing habitat. During construction and maintenance activity, biologists will need to be on site to locate and remove fish and other aquatic organisms to an upstream location until the activity is complete and water quality conditions are acceptable.

Primary permitting issues are expected to be related to the potential impacts of channel disturbance during construction and maintenance activity on aquatic organisms, principally protected salmonids. The RWQCB, CDFG, and ACE are expected to have permit authority; NMFS is expected to have input through the ACE permit. The proposed design is intended to provide significant sediment detention capacity while maintaining a low flow channel consistent with existing conditions to avoid creation of migration barriers.

The proposed dredging, particularly the proposed annual or near-annual maintenance dredging, may represent potentially significant environmental impacts necessitating completion of an

Environmental Impact Report (EIR) to comply with CEQA. Considering that maintenance dredging, habitat and fish use conditions may vary over time, and that repeated entries to remove sediment are likely, the scope of the EIR should be developed to accommodate changing conditions and changing needs.

Comment [T29]: And EIS to comply with NEPA for actions on NPS land?

Comment [cc30]: Agree

5. Operation and Maintenance Requirements and Costs

The chief O & M concern for this scenario is removal of sediment on a routine, possibly annual, basis. Each new sediment removal site, including the District site being planned near Calle del Mar, must be designed to include an access road or trail suitable for heavy equipment such as an excavator and dump truck. The threshold for maintenance should be determined in advance, with the objective of preserving a pre-determined minimum sediment storage capacity in the system. In some years, winter flows may be insufficient to produce significant sedimentation at these sites, and there may be no need for excavation. However, it should be expected that annual maintenance will be necessary, and procedures and costs for annual permitting should be planned accordingly. In addition, it should be anticipated that occasional large sedimentation events will occur, and that the scope of sediment removal should expand to include the easement area near Arenal Avenue and the locations at bridges along the Calles. Potential volume of sediment removal at these sites is at least 150 cu. yds. Combined with the 290 cubic yards of sediment storage proposed at SR1 and SR2, maximum annual sediment removal would be about 440 cubic yards.

Costs for excavation and hauling, including contractor costs, are estimated below for both a maximum and average annual condition. Costs for professional supervision of maintenance excavation by a geomorphologist and biologist are included. Contractor and hauling costs are included. Disposal costs or aggregate value of excavated sand and gravel is not included. Annual permitting costs, if any are not included, but annual reporting is included.

Estimated Annual Maximum Maintenance (excavation of 440 cubic yard)

Annual Site Review	$ 3,500
Dredging	$ 17,600
Contractor Overhead	$ 4,700
Monitoring	$ 4,000
Cost Subtotal	$ 29,800
Project Administration @ 10%	$ 3,000
Total Cost Estimate	**$ 32,800**

Estimated Annual Average Maintenance (excavation of 150 cubic yards)

Annual Site Review	$ 3,500
Dredging	$ 7,200
Contractor Overhead	$ 2,200
Monitoring	$ 4,000
Cost Subtotal	$ 16,900
Project Administration @ 10%	$ 1,700
Total Cost Estimate	**$ 18,600**

6. Sustainability (Short-term and Long-term)

Watershed erosion processes will continue to produce sediment that will tend to be deposited in the lower reaches of Easkoot Creek. Sedimentation basins are expected to be effective, inducing deposition of gravel and sand transported as bed load, however, significant quantities of sediment not captured by sedimentation facilities are expected to be deposited in dredged areas. The rate of deposition will be substantially reduced by the sedimentation facilities. It is expected that during typical annual flood events extending up to 5 yr recurrence interval (approximately), the rate of erosion and sediment transport in the watershed will be relatively low and proposed sediment removal upstream of State Highway 1 will be largely effective resulting in only incremental sedimentation in the dredged channel. Larger flood events (approximately > 5 yr recurrence interval) are expected to produce significant erosion and sediment transport in the upper watershed that is likely to cause substantial sedimentation of the dredged channel, mitigated by the sedimentation facilities. The initial installation, if approved, is expected to provide flood control benefits in accordance with the results of recent modeling efforts. Flood control benefits will be degraded when sediment is deposited in the dredged reach of Easkoot Creek, emphasizing the need to maximize upstream sediment removal upstream.

Comment [T31]: Have the sediment transport and deposition dynamics been modeled for various storm events?

Comment [cc32]: Yes, but sediment information is variable.

The channel reach under consideration for dredging is located in an urbanized area with a history of disturbance by dredging and construction. As a consequence of channelization and stabilization efforts, the banks and channel elevation appear to be relatively stable and do not appear to be eroding significantly in most areas. Mobilized bed load is therefore delivered mostly from upstream sources, and cannot be effectively controlled at the source.

Effective lifetime and performance results of the proposed dredging cannot be predicted with much certainty. The bed load sediment volume that would completely eliminate flood control benefits of dredging equals the proposed removal volume (about 2800 cubic yards of net removal). A flood flow with about a 10-year return period may be capable of moving this volume into the treated area in a single season based on data available for the 2005 flood event. Average seasonal sedimentation amounts to about 125 to 160 cubic yards based on analysis of the historic record. The last mass channel dredging was believed to have been undertaken in 1987, in response to 1986 flood sedimentation. The effective life of that work was on the order of 10 years owing to sedimentation during the floods of 1997 and/or 2005. The size, effectiveness and maintenance of sediment removal sites, as well as District spot dredging will determine the sustainability of flood benefits obtained by dredging. Incremental sedimentation of the dredged reach should be expected, and pulses of sedimentation associated with large storm events that exceed the sedimentation capacity of removal sites may diminish flood benefits more rapidly. It is also possible that in a large storm event, the upstream sediment removal sites may be more effective in than has been assumed, and the incremental sedimentation of the dredged channel may proceed more slowly. The potential impact of a peak flow diversion below Arenal Avenue may substantially reduced sediment transport capacity to the Calles, and will be quantitatively evaluated in a separate scenario.

7. Feasibility and Next Steps (Additional Information Needs)

- Identification and survey of property lines for parcels adjacent to dredging area is needed to initiate the process of obtaining landowner consent and access for more detailed planning and design.

- Analysis of bank stability of dredged channel will be needed to refine concept plan for dredging, in particular determining proposed bank angles and the location and concept design to maintain bank stability.
- Investigate feasibility of SR2 with affected landowners and key regulatory agency staff. Significant upstream sedimentation capacity is necessary to extend the duration of flood mitigation benefit of dredging. Alternative designs with greater storage capacity should be considered.

North Bypass Alternative

1. Description

This alternative involves the construction of a bypass channel to divert a portion of the discharge of Easkoot Creek to the ocean during high flow conditions. The proposed diversion point is located on the left bank of the channel opposite the upstream portion of the Parkside Café, and the diverted water flows through a 50-ft wide by 3-ft deep trapezoidal bypass channel, discharging to a detention basin located in the vicinity of the north GGNRA parking lot, thence to the ocean.

2. Flood Control Benefits

The potential flood control benefits of bypass flows to the north parking lot are substantial. During a December 2005 flood, the bypass carries up to 72.6 cfs or 42% of the total discharge of Easkoot Creek. Bypassing these flows completely eliminates flooding above the northern GGNRA parking lot (Figure A12). Flood extent is reduced somewhat and floodplain depths are reduced between 0.1 and 0.5 feet throughout the Upper Calles. Unlike most of the other alternatives, the bypass does reduce flood extent and floodplain depths significantly (>0.5 feet) throughout the Lower Calles (Figure A12). The average reduction in peak water levels in the channel is 0.6 feet in the reach adjacent to the Parkside Café, 0.4 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.4 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table ES1). Approximately 11 of 24 buildings are removed from the December 2005 floodplain under this alternative (Table ES2).

3. Preliminary Design and Estimated Construction Costs

The proposed diversion point is located on the left bank of the channel opposite the upstream portion of the Parkside Café. This location was selected because a) it represents the upstream-most location where diversion is practical, b) downstream of this point channel capacity, channel slope, and sediment transport capacity become significantly reduced, and c) it coincides with the location of overbank flows under existing conditions and likely also under historical conditions prior to the development of the park~~GGNRA~~ facilities. These considerations are equally applicable to another scenario, the "South Bypass", which has substantial practical advantages and fewer disadvantages with respect to fisheries.

Comment [T33]: It is likely that there wasn't a single historic location of overbank flows.

Comment [cc34]: True, but this statement is vague.

The proposed diversion structure is a 50-ft wide lateral weir with a crest elevation of 24.8-ft NAVD88. The bypass channel is a 50-ft wide by 3-ft deep trapezoidal channel with a 1:1 side slope. The channel flows south through dual 22-ft wide by 3-ft deep box culverts beneath the existing road leading to the south parking lot, bends to the west and flows through a second set of dual 22-ft wide by 3-ft deep box culverts beneath the existing road leading to the central parking lot, and terminates at the southern end of the north parking lot. The total channel length is approximately 636-ft and the total volume of material that would be excavated to construct the bypass channel is 3,320 cubic yards. The channel sizing is based on the capacity required to convey bypass flows during a 100-yr flood event. The alignment was selected so as to minimize the required modifications to existing GGNRA infrastructure and parking facilities.

In order for the parking lot to function as an effective detention basin, an approximately 350 foot long berm or flood wall would need to be constructed along the northwestern edge of the parking lot to prevent flood waters from moving into residential areas of the upper Calles as well as an approximately 330 foot long berm or flood wall along the left bank (sea-ward) of the creek at the northeastern edge of the parking lot to prevent flood waters from returning to the creek and exacerbating flooding downstream.

The elevation of the weir crest at the point of diversion of is arguably the most important design parameter as it will exert a strong control on a) the overall effectiveness of the bypass as a flood mitigation measure, b) the frequency that the bypass channel will be active and associated implications for fish movement, c) the flow above which downstream conditions will be altered and associated implications for fish passage, and d) the freeboard available to accommodate sediment deposition at the inlet. For conceptual design purposes, the diversion weir crest elevation was set to 1-ft above the existing channel thalweg (24.8-ft NAVD88). Under existing conditions, stream stage (flow depth) at this elevation is approximately 40 cfs. An examination of the flow record at stream gauge EK for seven water years with a nearly complete record from 2002 through 2010 indicates that flows exceeded this threshold between one and four times per year with an average frequency of two events per year.

There are potential limitations associated with design and construction of the necessary facilities that make the North Bypass less feasible than the South Bypass alternative. This alternative would be much more difficult to design and permit than the south bypass alternative for a variety of reasons. First, it would involve impounding floodwaters in a location immediately adjacent to residential areas (the Calles), creating potential for an on-going risk of flooding. Additional design studies would be needed to ensure that containment berms/flood walls were properly sized and constructed to achieve the desired degree of flood risk. The height and design of these barriers might be aesthetically objectionable and inconsistent with GGNRA land-use objectives. Second, this alternative would likely interfere with existing uses (parking and possibly adjacent rest rooms) in the GGNRA; periodic use of the parking lot as a detention basin for floodwaters may require substantial maintenance.

4. Permitting Issues

Potential effects of the North Bypass alternative on endangered fish appear to be substantially more difficult to mitigate than for the South Bypass alternative. Fish entrained in the bypass would be routed to a detention basin that would not provide permanent habitat; the total amount of bypass flow would be routed to the ocean in a relatively short time period, so it would be expected that these fish would be exported to the ocean. It might be possible to design a flow path back to Easkoot Creek, but it would require design elements that would operate at cross-purposes to flood mitigation. Given that the south bypass alternative is much less problematic from both design/risk, parking impact, and fisheries perspectives and the expected flood control benefits are similar to those for the north bypass, we believe that the south bypass is a much more feasible alternative.

5. Operation and Maintenance Requirements and Costs

Given our recommendation of pursuing the south bypass alternative over this alternative, no operation and maintenance requirements or costs were developed.

6. Sustainability (Short-term and Long-term)

Given our recommendation of pursuing the south bypass alternative over this alternative, the sustainability of this alternative was not considered.

7. Feasibility and Next Steps (Additional Information Needs)

Given our recommendation of pursuing the south bypass alternative over this alternative, no feasibility study or next steps are suggested

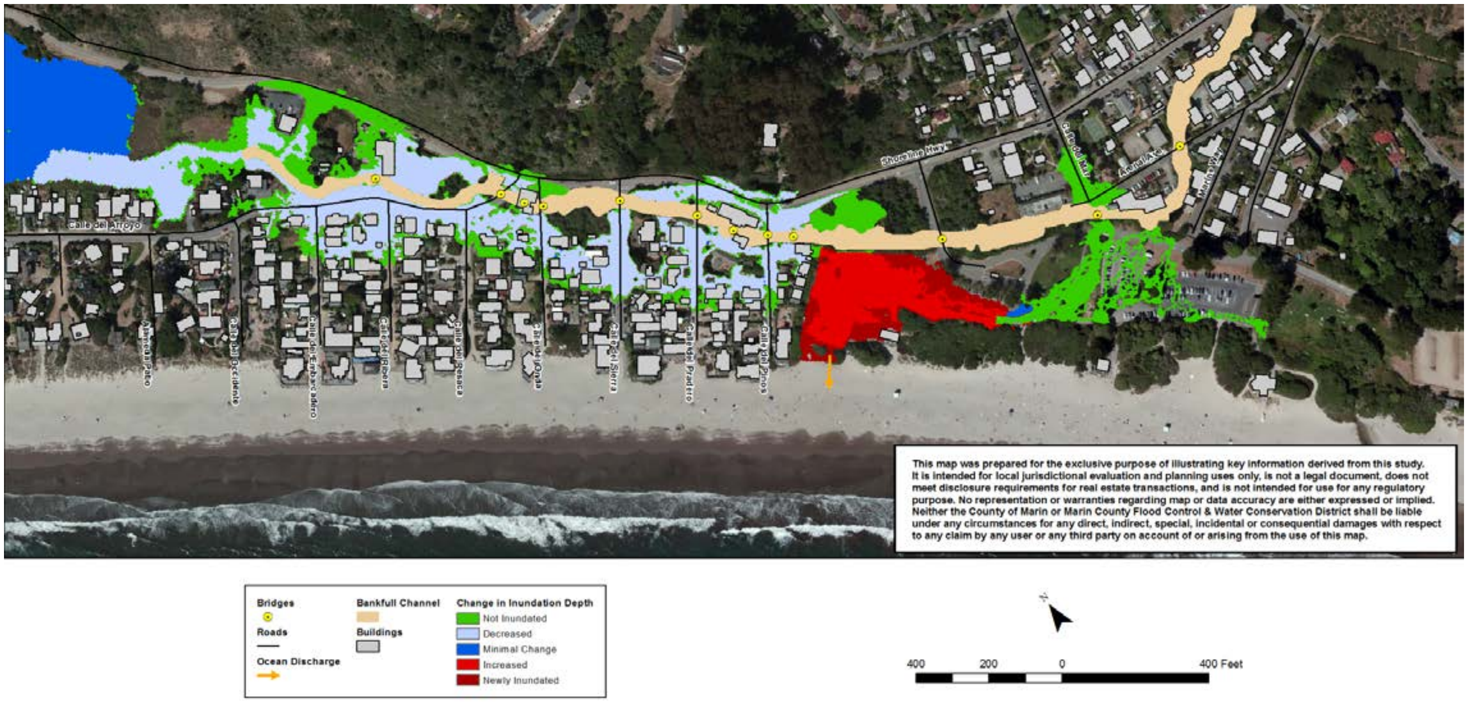


Figure A12: Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the North Bypass alternative for the December 2005 flood.

South Bypass and Poison Lake Restoration

This alternative proposes construction of a bypass channel to divert peak flood flows from downstream portions of Easkoot Creek; the majority of flow would remain in the existing stream. Bypass flows of up to 70 cfs are routed to a proposed 2.4 +/- acre wetland enhancement and restoration area that includes an existing wetland area of about 1 acre. The restored wetland, so-called Poison Lake, would provide refugia and rearing habitat for salmonids that could be conveyed by flood flows out of Easkoot Creek. The proposed diversion point is located on the left bank (sea-ward) of the channel opposite the upstream portion of the Parkside Café (Figure A13). This location was selected because

a) it is the upstream-most location where diversion is practical,
b) downstream of this point channel capacity, channel slope, and sediment transport capacity become significantly reduced, and
c) it coincides with the location of overbank flows under existing conditions and likely also under historical conditions prior to the development of the park ~~GGNRA~~ facilities.

The proposed diversion structure is a 50-ft wide lateral weir with a crest elevation of 24.8-ft NAVD88; the bypass channel is a 50-ft wide by 3-ft deep trapezoidal channel with a 1:1 side slope (Figure 1). The channel flows south through dual 22-ft wide by 3-ft deep box culverts beneath the existing road leading to the south parking lot, bends and flows east along an alignment parallel to the road and north edge of the parking lot, flows through a second set of dual 22-ft by 3-ft culverts beneath the existing road and terminates in the restored wetland area (present-day picnic area). The total channel length is approximately 430 ft (Figure 1) and the total volume of material that would be excavated to construct the bypass channel is 2,230 cubic yards. The channel sizing is based on the capacity required to convey bypass flows during a 100-yr flood event. The alignment was selected so as to minimize the required modifications to existing GGNRA infrastructure and parking facilities.

The flood control benefits of bypassing flows to a restored Poison Lake are substantial. During the December 2005 flood, the bypass carries up to 73 cfs or 42% of the total discharge above the bypass of 172 cfs. Flood extent is reduced somewhat and floodplain depths are reduced throughout the Upper Calles; flood extent and floodplain depths are significantly reduced throughout the Lower Calles (Figure A14). Peak water levels in the channel are reduced 0.6 feet in the reach adjacent to the Parkside Café, 0.4 feet in the Upper Calles reach, and 0.6 feet in the Lowe Calles reach (Table ES1). Approximately eleven of twenty-four buildings are removed from the December 2005 floodplain under this alternative (Table ES2). During the 100-yr flood, the bypass carries up to 245 cfs or 53% of the total discharge of 463 cfs that reaches the bypass. Minor reductions in flood extent throughout the study area result from bypassing these flows. Floodplain depths are reduced in the vicinity of the Arenal Avenue bridge, Calle del Mar, and throughout the Upper and Lower Calles (Figure A15). The average reduction in peak water levels in the channel is 0.5 feet in the Parkside Café reach, 0.4 feet in the Upper Calles, and 0.3 feet in the Lower Calles (Table ES3). Approximately thirteen of fifty-nine buildings are removed from the 100-yr floodplain (Table ES4).

Comment [T35]: Darren – Do we want to suggest an alternative name?

Comment [T36]: During the 2005 event, but not during the 100-yr event, when 47% remains in the existing stream.

Comment [T37]: Clarify. Next paragraph says up to 73 cfs for 2005 event, and up to 243 cfs for 100-yr event.

Comment [cc38]: Clarified.

Comment [T39]: It is likely that there wasn't a single historic location of overbank flows.

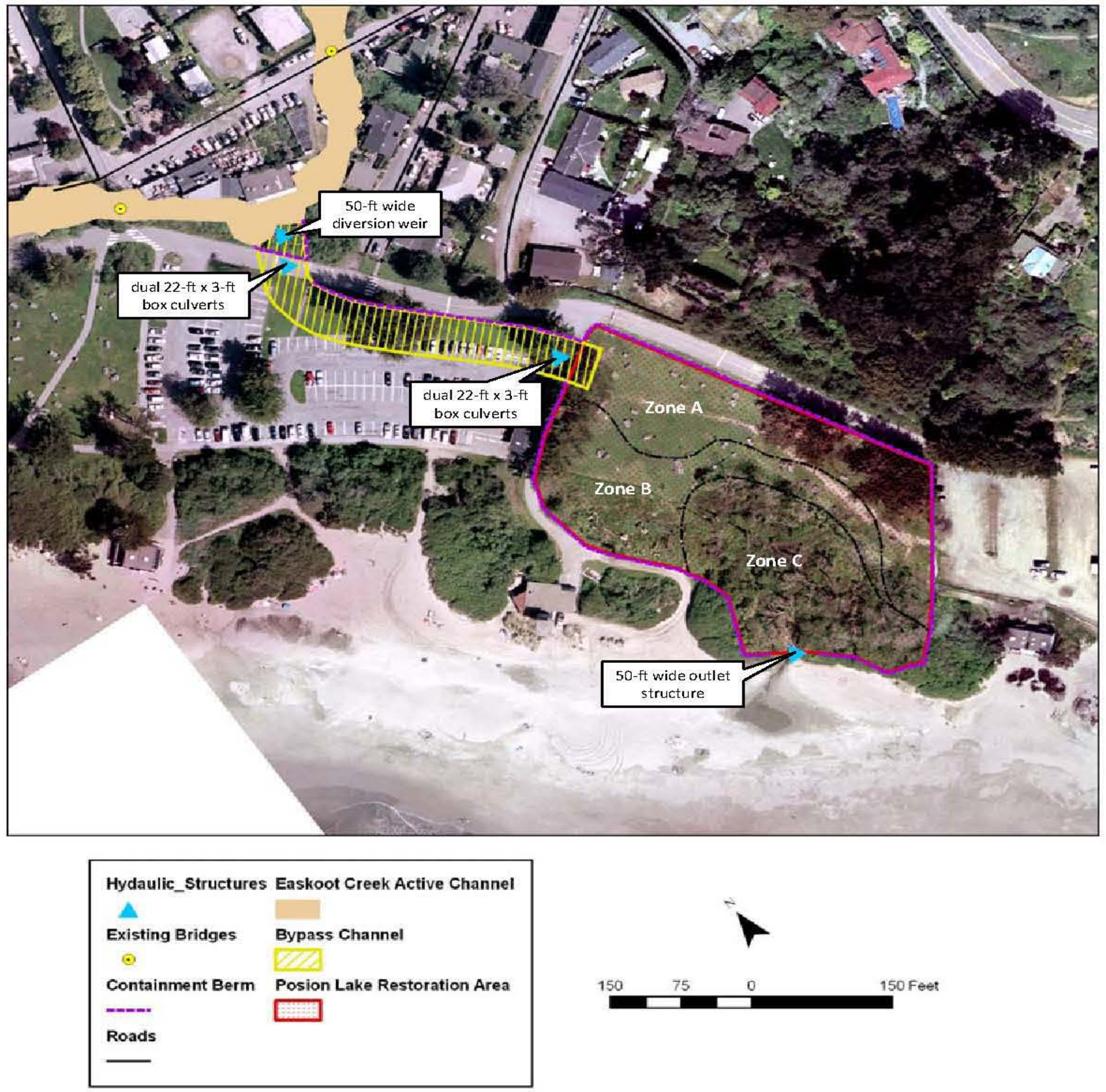
Comment [T40]: Previous paragraph says up to 70 cfs.

Comment [T41]: Previous paragraph says up to 70 cfs.

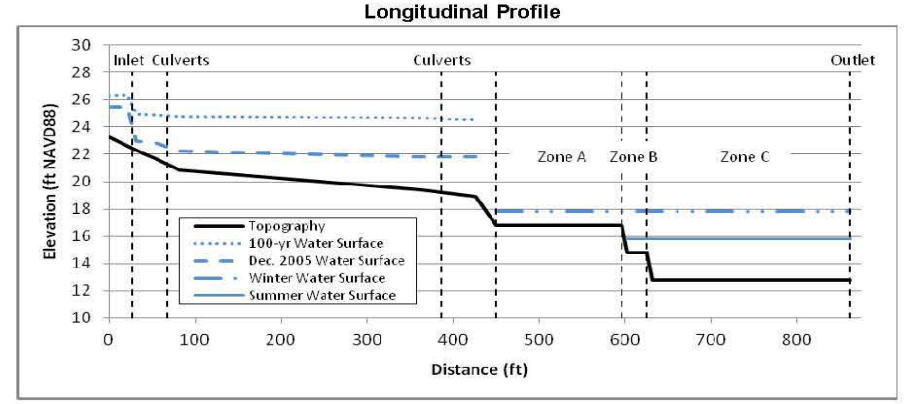
Table A5. Summary cost estimate for South Bypass and Poison Lake Restoration.

Bypass Alternative - Planning-level Budget Summary	Cost ($)	Percent
Consultant Planning, Permitting and Design Subtotal	176,470	12.7
Construction subtotal	838,430	60.4
Subtotal Contractor Overhead	211,100	15.2
Planning-level Cost Estimate (to nearest $1000)	1,226,000	88.4
Project Administration	61300	4.4
Land acquisition (if necessary)	100,000	7.2
Installed Project Cost Estimate	**1,387,300**	**100.0**
O&M (not yet evaluated for cost)		

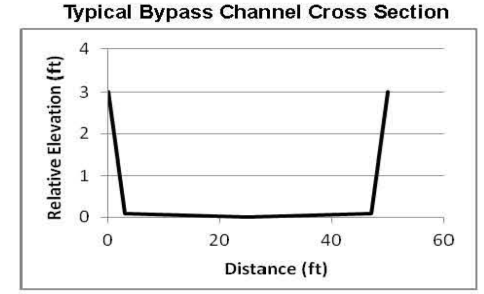
Comment [MO42]: Does not specifically include EIR but does provide significant time for an interdisciplinary design team to develop more detailed assessment and plans



Longitudinal Profile



Typical Bypass Channel Cross Section



Restoration Area Details

	Area (acres)	Excavation (cubic yds)	Summer Depths (ft)	Winter Depths (ft)
Zone A	0.89	4,694	-2 - 0	0 - 2
Zone B	0.73	4,533	0 - 2	2 - 4
Zone C	0.82	5,150	2 - 4	4 - 6
Total	2.44	14,377		

Figure A13. Overview map of the south bypass and Poison Lake restoration alternative including a preliminary design profile and sample cross section for the bypass channel, and a summary table of the expected water depths and excavation volumes in the restoration area.

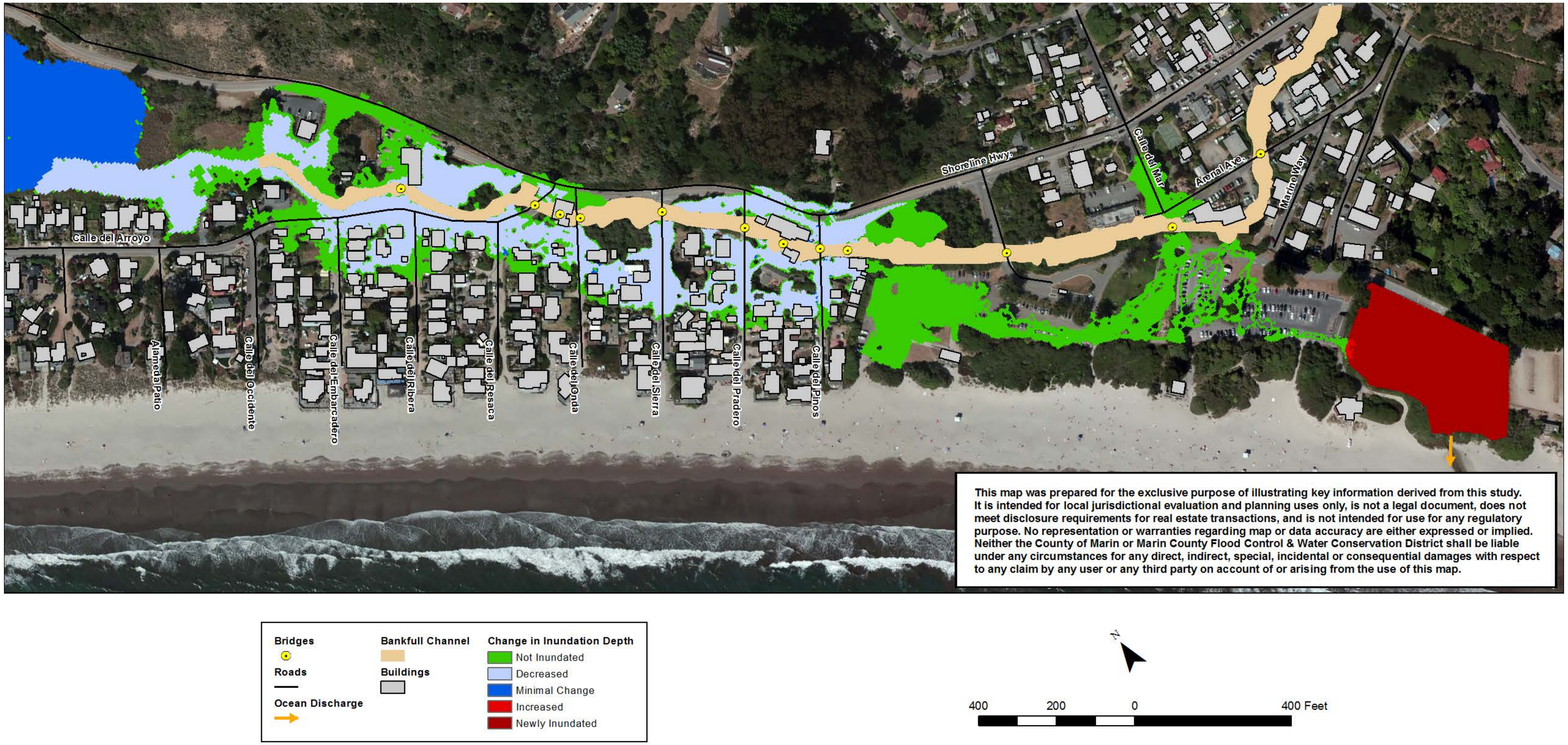


Figure A14: Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the South Bypass alternative for the December 2005 flood.

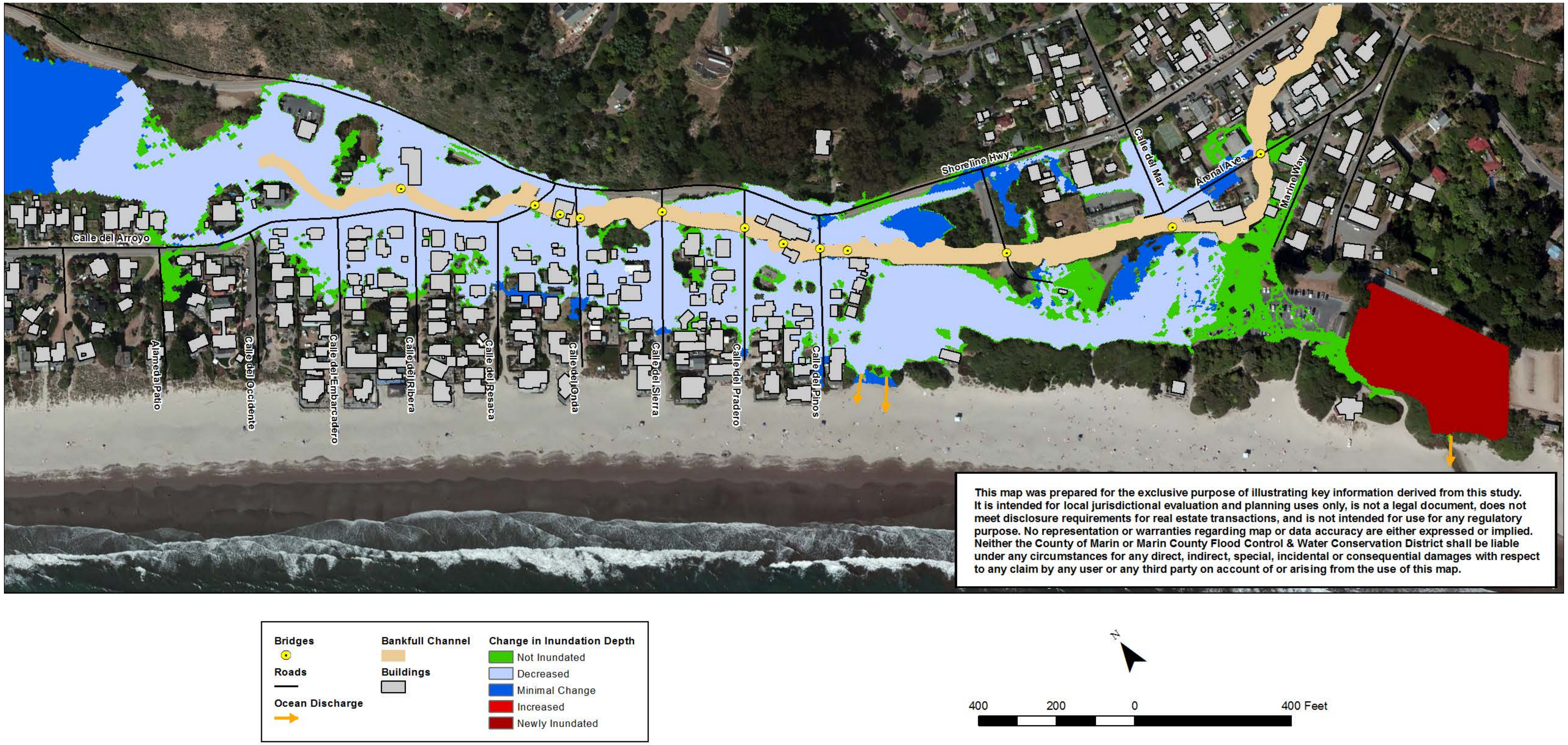


Figure A15. Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the South Bypass alternative for the 100-yr flood.

ALTERNATIVE: South Bypass and Poison Lake Restoration

1. Description

This alternative involves construction of a bypass channel to divert peak flood flows from downstream portions of Easkoot Creek; the majority of flow would remain in the existing stream. Bypass flows of up to 70 cfs are routed to a proposed 2.4 +/- acre wetland enhancement and restoration area that includes an existing wetland area of about 1 acre. The present-day wetland is the remnant of the historic Poison Lake which was filled to accommodate park facilities c.1960. The flood routing design proposal seeks to restore a portion of the historic Poison Lake to its former open water habitat while retaining substantial components of vegetated and seasonal wetland. The proposed diversion point is located on the left bank (sea-ward) of the channel opposite the upstream portion of the Parkside Café (Figure A13). This location was selected because

a) it is the upstream-most location where diversion is practical,
b) downstream of this point channel capacity, channel slope, and sediment transport capacity become significantly reduced, and
c) it coincides with the location of overbank flows under existing conditions and likely also under historical conditions prior to the development of the park ~~GGNRA~~ facilities.

Comment [T43]: See previous comment.

The conceptual design of the bypass is summarized as follows:

- The proposed diversion structure is a 50-ft wide lateral weir with a crest elevation of 24.8-ft NAVD88 (Figure A13).
- The bypass channel is a 50-ft wide by 3-ft deep trapezoidal channel with 1:1 side slopes (Figure A13). It is sized to handle estimated 73 cfs bypass flows during a 100-yr flood event.
- From the bypass structure, the channel is routed south through dual 22-ft wide by 3-ft deep box culverts beneath the existing road leading to the south parking lot.
- From the culvert, the bypass channel flows southeast along an alignment parallel to the road and north edge of the parking lot, requiring removal of some or all of the landscaping, and a small portion of the existing parking.
- It is then routed through a second set of dual 22-ft by 3-ft culverts beneath the existing road to the snack bar and main life guard tower, and through a line of trees requiring partial removal.
- The discharge point of the bypass channel is into a proposed wetland restoration area (present-day picnic grounds).
- The total channel length is approximately 430 ft (Figure A13). The construction excavation volume is about 2300 cubic yards.
- The main channel should include an inset lower-flow segment about 0.7 ft deep and 2 ft wide to facilitate fish passage during periods of flow initiation and flow termination.
- A perimeter berm would contain flood flows during bypass operation (Figure A13); the need for this containment and design parameters would depend in part on hydraulics of the outlet structure.
- An outlet structure controlling the flow of water from the wetland to the ocean and providing for emigration of fish would be constructed.

Comment [cc44]: Clarified

Comment [T45]: During the 2005 event? 245 cfs during a 100-yr event?

Comment [T46]: How would this be operated?

Comment [cc47]: To be determined.

The draft alignment was selected so as to minimize the required modifications to existing GGNRA infrastructure and parking facilities. Value engineering or site utilization considerations may result in modification of the channel alignment or configuration, including relocation of the bypass to lie entirely on public property. The preliminary design used for the hydraulic model places the bypass on public property (GGNRA); however, it may ultimately be necessary to incorporate a portion of the adjacent privately-owned parcel in the bypass facility.

Comment [T48]: These statements are confusing.

Comment [cc49]: I think there might be a small piece on private property.

2. Flood Control Benefits

The flood control benefits of bypassing flows to a restored Poison Lake are substantial. During the December 2005 flood, the bypass carries up to 73 cfs or 42% of the total discharge above the bypass of 172 cfs. Bypassing these flows completely eliminates flooding above the GGNRA north parking lot (Figure A14). Flood extent is reduced somewhat and floodplain depths are reduced between 0.1 and 0.5 feet throughout the Upper Calles. Unlike most of the other alternatives, the bypass does reduce flood extent and floodplain depths significantly (>0.5 feet) throughout the Lower Calles (Figure A14). The average reduction in peak water levels in the channel is 0.6 feet in the reach adjacent to the Parkside Café, 0.4 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.6 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table ES1). Approximately eleven of twenty-four buildings are removed from the December 2005 floodplain under this alternative (Table ES2).

During the 100-yr flood, the bypass carries up to 245 cfs or 53% of the total discharge of 463 cfs that reaches the bypass. Minor reductions in flood extent throughout the study area result from bypassing these flows. Floodplain depths are reduced by 0.1 to 0.5 feet in the vicinity of the Arenal Ave. bridge and the intersection of Calle del Mar and Arenal Ave (Figure A15). Floodplain depths are reduced by 0.25 to 0.75 feet throughout the Upper Calles and by 0.25 to 0.50 feet throughout the Lower Calles (Figure A15). The average reduction in peak water levels in the channel is 0.5 feet in the Parkside Café reach, 0.4 feet in the Upper Calles, and 0.3 feet in the Lower Calles (Table ES3). Approximately thirteen of fifty-nine buildings are removed from the 100-yr floodplain (Table ES4).

3. Preliminary Design and Estimated Construction Costs

The essential elements of the preliminary proposed design are described in Section 1 above. Additional details and supplemental design and planning considerations are presented in this section.

Bypass Weir. The lateral weir crest elevation at point of discharge from the creek is a critical design parameter. It will strongly influence:

a) overall effectiveness of the bypass as a flood mitigation measure,
b) frequency of flows into the bypass channel and associated implications for fish movement and sediment transport,
c) the flow threshold at which downstream conditions will be altered and associated implications for fish movement and sediment transport, and
d) the freeboard available to accommodate potential sedimentation at the inlet.

For conceptual design purposes, the diversion weir crest elevation was set to 1-ft above the existing 2013 channel thalweg (24.8-ft NAVD88). Under existing conditions, stream discharge of approximately 40 cfs is necessary to raise the stream stage to the point where flow to the bypass would occur. An examination of the flow record from the Park Service gauge on Easkoot Creek for the seven water years with a nearly complete record from 2002 through 2010 indicates that flows exceeded this threshold between one and four times per year with an average frequency of two events per year.

The diversion weir crest must remain at a constant elevation over time relative to the stream channel bed. This is necessary to ensure uniform bypass channel system performance. Measures must therefore be taken to stabilize the creek channel invert at this point, to prevent aggradation or degradation which would change the set point at which lateral flow can occur. The potential for sedimentation during storm events is substantial, and further analysis is required to evaluate performance and design of the bypass in relation to sedimentation (see additional discussion below). If dredging occurs in the main channel prior to bypass channel construction, an appropriate adjustment in weir elevation would need to be made.

One method for stabilizing the Easkoot Creek channel invert at this location would be to install a concrete or boulder weir across the channel just downstream of the lateral weir. This would serve to maintain a stable and fairly level channel invert and would ensure that the lateral bypass channel would function as intended. The structure could be configured as a roughened ramp suitable for fish passage. Confinement of flow in the section of Easkoot Creek just upstream of the bypass could prove critical to the performance of the bypass, and final designs ~~w~~may require some additional bank structures to contain stream flow as it approaches the bypass. Finally, the lateral weir could be fitted with flashboards that would allow for adjustment of the weir height to compensate for sedimentation adjacent to the weir.

Comment [T50]: Wouldn't this tend to cause aggradation at the diversion weir?

Comment [cc51]: To be developed further.

Sedimentation of Bypass Facilities. Coarse bed load sediment in transport in Easkoot Creek at the bypass is not likely to be carried into the bypass channel given the 1 ft elevation difference (as modeled) between the bed of Easkoot Creek and the lateral bypass weir. Gravel and cobble would likely continue to move downstream, although at a reduced rate downstream of the bypass. The geometry of the channel adjacent to and immediately downstream of the bypass may need to be modified to better maintain sediment transport and reduce sedimentation in the bypass that could affect bypass performance. Provisions to accommodate anticipated sedimentation (potentially on the order of hundreds of yards), including additional or expanded sedimentation facilities may be required. Potential sedimentation at the bypass will require additional consideration in future feasibility and design studies. Suspended sediment load consisting of sand, silt, and clay would likely be transported by flow over the bypass weir. Some deposition would likely occur in the bypass channel owing to low gradient and relatively high width of the channel. Sedimentation in both the bypass channel and Easkoot Creek would need to be monitored and removed on a regular basis to ensure adequate capacity and satisfactory performance and conveyance and to prevent excessive sedimentation of the restored Poison Lake.

Comment [T52]: Seems like this wouldn't occur until the proposed weir across the channel was overtopped with accumulated bed load.

Bypass Channel. It would be possible to construct an open bypass channel that mimics a natural system, by lining it with cobble and boulders and/or grass cover. The gradient is low enough and flow depth shallow enough that channel erosion potential is limited and that design options for the channel would include a variety of options. A cobble/boulder configuration would have higher friction and greater trapping capability and so might need to be made a little wider or deeper in order to retain the desired capacity rating. It may be necessary to include sedimentation structures within the bypass channel to reduce the quantity that might reach the restored wetland area. A narrow low flow channel section inset on the floor of the bypass channel would likely be added to reduce potential for fish stranding. Construction costs would be increased with this configuration due to use of more materials that require greater labor for installation.

Comment [T53]: This would create challenges for removal of accumulated sediment; the cobbles could become buried pretty quickly.

Outlet Structure from Restored Wetland. The area south and east of the proposed bypass channel discharge point consists of a picnic area lawn that transitions into a vegetated wetland area containing ponded water. Runoff from local area sheet flow as well as spring or groundwater flows exit the area through a gap in the sand dunes that contains an historic control structure (a hardened sill perforated by a 12 to 18 inch diameter CMP). The precise age, design and purpose of this structure is unknown, but may date to construction of the south overflow parking lot and the filling of Poison Lake. At present, it provides grade control, preventing seepage erosion of the sandy soil at the point of channel discharge to the beach. The outlet structure for the restored wetland would likely remain in this location; however, a new structure would be designed to provide some detention of bypass flows, to maintain ponded water at the desired elevation, and to provide suitable depth and velocity of flow for emigrating anadromous fish. The model design assumes a 50 ft wide broad crested weir; the ultimate configuration would be determined by subsequent design work. A notch in the weir that would contain lower flows would likely be incorporated in this outlet structure to better accommodate base flow and fish emigration.

Alternative Bypass Channel Configurations. The initial bypass configuration (Figure A13) maximizes vehicular access and parking by elimination of an existing landscaped area with mature trees and underground utilities. Alternatively, the landscaped area could be retained, and the 50' wide channel routed through the parking lot. It may be possible to retain some dual purpose functionality (at additional cost) by routing an enclosed channel under the parking lot. An alternative that would maximize parking and landscaping at the expense of local roadways would be to route the channel down the existing road at the north side of the parking lot. Since the channel is about twice the road width, some loss of landscaping would be inevitable. In addition, the preliminary design is conservative with respect to conveyance capacity providing 1 ft of freeboard for the 100-yr flow event, and it might be possible to modify the channel width to accommodate different objectives.

Consideration could be given to using multiple culvert bores in lieu of an open channel to route bypass flows to the wetland area. Multi-bore culverts may be less expensive and more aesthetic than an open channel bypass. Using culverts would allow cut and cover of the bypass waterway, minimizing conversion and disruption of parking and roadway areas. It may be possible to split tubular flows around the landscaped island, thus eliminating the 50' channel width constraint and preserving more of the existing landscaping. This design alternative would need to consider suitability for the anticipated fish use, expected to be limited to involuntary emigration from Easkoot Creek to the restored wetland during a flood bypass event. Culvert inlet conditions and head constraints would need to be considered as part of the design, to ensure design flows can be handled.

Poison Lake Restoration Design Factors. Many options are available in terms of the extent of the wetland restoration and enhancement area (so-called Poison Lake) and the desired wetland and habitat features. This conceptual design plan seeks to minimize the impacts to existing GGNRA parking facilities and infrastructure, although substantial infrastructure impacts would occur. The footprint of the Poison Lake restoration area follows the boundaries of the south picnic area and small existing wetland (Figure A13).

Open water habitat area would be restored at the lower end of the area by constructing a slightly elevated weir at the location of the present-day culvert outfall. The open water habitat would provide refugia and rearing habitat for any fish carried downstream into the bypass channel. The proposed weir crest elevation matches the local winter water table elevation. Upon lake-fill to the design elevation of 18.4' NAVD88, inflow and outflow volumes will be equivalent, allowing the lake to function as a flow-through basin with limited accumulation of water.

The conceptual design includes a 3-ft high berm surrounding the perimeter of the lake (Figure A13). This berm is not strictly necessary because both inflow and outflow structures are designed to accommodate the maximum flows expected from the bypass. The berm serves as a safeguard for residential areas in the unlikely event that water overtops the system and escapes via the parking lot access road to the north.

Comment [T54]: It isn't clear how this berm would function.

At flood stage conditions, lake water elevation will match that of the outlet weir plus surcharge necessary to achieve flood flows over the weir. Under no-flow conditions, the water surface elevation will fluctuate in accordance with the local shallow groundwater profile. Lake depth will be a function of the water surface elevation relative the degree of excavation that takes place within the restoration footprint.

Comment [T55]: Would dune sand accumulation and ocean storms compromise the function of the outlet structure?

Expected fluctuations in groundwater elevations are based on data from ten wells within the restoration footprint and an additional eleven wells in other areas of the park collected by NPS between November 2003 and May 2011. The number of water table elevation observations in individual wells ranged from nine to thirty-seven, but in all cases included both dry season and wet season measurements. Examination of these data indicates that in the northern portion of the restoration area, groundwater elevations range between three and four feet below ground during the late summer and fall and two to three feet below ground during winter and early spring. In the southern portion of the restoration area on the landward side of the dunes, groundwater elevations range from near land surface in the late summer and fall to one to two feet above ground in the winter and early spring.

Based on the existing topography and the spatial and temporal variations in groundwater elevations, we have delineated three zones within the restoration area. These zones are designed to provide a variety of habitat features and water depths in the restoration area while minimizing the required excavation. Based on the groundwater elevations, we have calculated design elevations for each zone that are expected to provide a seasonal range of desired water depths (Figure A13).

Zone A encompasses 0.89 acres in the higher northern portion of the restoration area and represents a zone of shallow water depths. The design elevation for this zone is 16.8-ft NAVD88. Under flood flow conditions, the design elevation would temporarily increase to 18.4 feet, with depth of inundation about 1.5 feet. Under non-flood conditions, it is expected to be a seasonal wetland area that is typically dry during the summer months.

Zone B encompasses 0.73 acres in the central portion of the restoration area and represents a perennial wetland zone with intermediate water depths. The design elevation for this zone is 14.8-ft NAVD88. In summer months water depths are expected to range from zero to two feet. Under flood flow conditions, the design elevation would temporarily increase to 18.4 feet with a water depth of about 3.6 feet.

Zone C is a 0.82 acre perennial pond in the lower restoration with a design elevation of 12.8 ft, summer water depths ranging from two to four feet, and winter depths ranging from four to six feet. Under flood flow conditions, the design elevation would temporarily increase to 18.4 feet.

Comment [T56]: Public safety considerations for these ponds?

The total restoration area covers about 2.44 acres, with an average required excavation depths of about three to four feet in all zones. The total volume of required excavation is on the order of 14,400 cubic yards.

Construction-General. The bulk of construction work for this project involves standard grading and drainage activities. Standard earthwork activities are required for creation of the bypass channel. The work would occur in a developed semi-urban area, requiring relocation of substantial undergrounded utilities. Depending on the final configuration, partial or complete removal of selected trees, landscape, curb and gutter, roadway, and parking facilities will be required. Means for maintaining vehicular access during construction will be necessary.

A significant amount of concrete work is required for construction of the bypass weir and bridge-type box culvert crossings. The bypass weir may ultimately require additional elements to reliably accommodate flows, specifically sidewalls to contain flows approaching and passing the weir and to reduce sedimentation adjacent to the weir. Furthermore, the bypass structure will likely need to be integrated with sedimentation facilities immediately upstream and downstream. These more detailed design elements are beyond the scope of this conceptual design plan; should a more elaborate bypass facility with provisions for sedimentation be required, substantial additional cost would be expected.

Poison Lake restoration would require significant excavation with a large portion at or below local groundwater elevation. Special construction techniques will be required, and a suitable site for disposal of spoils will need to be identified. The need for the berm portrayed in Figure 1 has not been established, and there are many possible methods of construction. The stability of the dunes located to seaward of the restored wetland when the system is operating at maximum flow through will need to be determined. This would be addressed by a geotechnical engineering study during a subsequent phase of design and feasibility.

Comment [T57]: Would this lake be subject to dune sand infilling?

Comment [T58]: Which Figure?

Comment [T59]: Suggest coastal geomorpologist /engineer involved in addressing this.

4. Permitting Issues

Comment [T60]: Might want to mention conformance with NPS policy.

The chief regulatory issues associated with this alternative pertain to listed salmonids (steelhead trout and coho salmon) and wetlands. The proposed Poison Lake restoration would require substantial modification of the existing wetland area that is a remnant of historical Poison Lake. A small pond supported by seepage flows (likely from the Easkoot Creek watershed) with some emergent wetland vegetation and dense woody riparian vegetation currently exists, which spills via a culvert to the beach into the high tide surf zone. Proposed excavation for Poison Lake restoration would likely impact the existing wetland area, however, little wetland fill is expected. Federal permits associated with wetlands would be handled by the US Army Corps of Engineers, and this will provide the nexus through which a Biological Opinion (BO) addressing fisheries impacts would be developed. The BO would fall under the purview of the National Marine Fisheries Service.

One of the primary objectives of this alternative is to mitigate risk to juvenile salmonids associated with bypass flows. Under existing conditions, as well as many of the alternatives considered, bypass flows occur in an uncontrolled fashion as flood flows spill from Easkoot Creek into the GGNRA parking lot. The Poison Lake diversion option would provide a flood bypass channel leaving Easkoot Creek near the Parkside Café and conveying water south, to a re-created wetland impoundment situated at the top of the beach, near the historical location of Poison Lake. The restored wetland habitat in Poison Lake is expected to be of higher quality than the existing wetland habitat currently present in lower Easkoot Creek.

Comment [T61]: Where is the lower Easkoot Creek wetland habitat?

The proposed bypass and restoration of Poison Lake is intended to provide suitable rearing habitat for juvenile salmonids that could be entrained by bypass flows. This alternative is not expected to affect the immigration of adult salmonids or spawning success. Juvenile salmonids are subject to entrainment by storm flows and subsequent displacement from Easkoot Creek into the restored area. Depending upon the timing, duration, and the magnitude of the flows captured during storm events, diverted fish may move directly through Poison Lake to the ocean, or they may reside in Poison Lake until subsequent storms provide suitable outflows for emigration. Because it is not likely that diverted fish will be able to swim from restored Poison Lake upstream through the bypass and back into Easkoot Creek, it is possible that some fish would remain in Poison Lake over the summer dry season.

Therefore, the Poison Lake diversion option presented here will need to have safeguards in place to both protect individual fishes present and reasonably assure that available habitat will continue to meet the functional needs of fish over time by providing adequate rearing habitat and a suitable emigration route to the ocean. Some degree of monitoring (e.g., water quality monitoring and surveys of the number of fish diverted), and possibly intervention (e.g., relocating trapped fish), may be required by resource agencies if this option is pursued.

Following restoration, Poison Lake is expected to range from 0-6 feet deep during the winter and from 0-4 feet deep in the summer, with a spillway allowing for discharge directly onto the beach. This will effectively create a partially closed lagoon system, similar to many small coastal lagoons along the California coast, that could provide valuable rearing habitat for juvenile salmonids if water quality remains good (i.e., dissolved oxygen remains adequately high and temperature remains adequately low), predation does not decimate the diverted fish, and fish can emigrate to the ocean.

Comment [T62]: Darren – your thoughts on these limitations?

Per the proposed alternative, Poison Lake will essentially re-establish a historical permanent wetland feature and outflow channel. This small perennial lake is expected to maintain a maximum depth of about four feet deep during the summer months. Steady inputs of cool groundwater, shading from adjacent trees, moderate ambient temperatures, and persistent coastal fog during the summer months should keep the water temperatures suitable for juvenile steelhead that may end up rearing in Poison Lake through the summer. Aquatic vegetation is expected to be quickly established and colonized with aquatic insects (therefore forage for fish should not be a limiting factor). Aquatic vegetation will help keep the water well oxygenated during most of the year, but may contribute to the reduction of oxygen during some periods. Prolonged periods of coastal fog can reduce photosynthesis of aquatic vegetation to the point where the plants consume more oxygen via respiration than they produce by photosynthesis, thereby reducing the dissolved oxygen in the water to potentially stressful or lethal levels for fish. Also, inputs of salt water during storm surges can kill off aquatic vegetation and cause reduction of dissolved oxygen as the dead plant material decomposes.

Comment [T63]: Is there evidence that there was a permanent outflow channel at this location?

Comment [T64]: Can mosquitoes be easily managed if there are salmonids there?

Fish rearing in Poison Lake will also be subject to predation by birds. Because water depths are expected to be 2-4 feet deep during most of the year, and deeper during the winter, rearing salmonids should be able to escape large-scale predation from wading birds (e.g., herons and egrets), but may be vulnerable to predation by swimming birds (e.g., mergansers and cormorants).

Comment [T65]: What about other animals?

Finally, creating conditions which allow diverted fish to continue their journey to the ocean will be essential for allowing them to successfully complete their life history. Poison Lake will be built with an outfall weir that discharges storm water directly onto the beach. The weir should be notched to concentrate the water flowing to and over the beach, giving out-migrating fish the best chance for crossing the beach at any flows. The actual length of beach that the fish will have to cross will depend upon the tidal stage and beach profile? during the storm, and may range from just a few feet to a couple hundred feet.

Significant additional feasibility and design studies would be necessary for the Poison Lake restoration effort. GGNRA has previously contemplated restoration of Poison Lake. Based on available information, it appears that sufficient water would be available (via seepage from the alluvial fan of Easkoot Creek), and that an outfall structure could be designed to accommodate fish passage (e.g. using California Department of Fish and Wildlife Salmonid Habitat Restoration Manual Part XII: Fish Passage Design and Implementation). Because the exact ecological conditions that will be created under this scenario are somewhat uncertain, a monitoring program should be established to measure the habitat conditions as well as the number and welfare of any fish diverted into restored habitats. This monitoring program should emphasize regular water quality parameters (i.e., dissolved oxygen, water temperature, and salinity) measured both near the surface and lower in in the water column in deeper portions of the pool. Visual surveys (i.e., snorkeling) should be conducted after storms and into the summer in order to determine the number and species of fish diverted, and their fate. Observations on birds and other predators should also be made regularly. If conditions for the survival of salmonids are determined to be unsuitable, the resource agencies may require the capture and relocation of entrained salmonids (back to Easkoot Creek).

5. Operation and Maintenance Requirements and Costs

The system as planned will operate on a passive basis, without active management requirements. Periodic inspection by the District will be required, as will periodic maintenance of the inlet weir structure if debris or sediment accumulates at that point. The open channel or culvert conveyance system should have minimal maintenance requirements if properly designed and installed. Some debris and vegetation management may be required in the restored lake area. Routine excavation of accumulated sediment is also likely to be required. Biological and ecological monitoring of Poison Lake is likely to required.

Comment [MO66]: Costs associated with these activities have not yet been estimated

6. Sustainability (Short-term and Long-term)

Properly designed and installed channel/culvert and bridges should have a reasonable 20-year design and economic life.

Bypass construction treats the symptom of inadequate downstream channel capacity by rerouting flood flows. The fundamental problem of channel capacity is caused by sedimentation processes. Efforts to manage sedimentation and maintain channel capacity must be maintained in conjunction with bypass installation.

7. Feasibility and Next Steps (Additional Information Needs)

- Confer with NPS regarding feasibility of reconfiguring road, parking and other affected facilities on GGNRA property.
- Confer with CDFW, NMFS, RWQCB, ACOE regarding grade control weir in Easkoot Creek specifically, and regarding the wetland restoration and enhancement project as a whole.

Comment [T67]: Include NPS in this?

- Evaluate sediment transport and sedimentation characteristics of the bypass channel, the bypass weir, and Easkoot Creek in the vicinity of the bypass in greater detail to determine additional design constraints relating to sedimentation.

- Assess land requirements for bypass facility and sedimentation basins.
- Develop revised design plan (30% complete) and revised cost estimate.
    - Develop more detailed knowledge of undergrounded infrastructure in bypass route.
    - Geotechnical assessment of soils in bypass route and dunes to seaward of restoration area.
    - Conduct topographic survey of weir location, channel routes, and wetland restoration and enhancement area.

Comment [T68]: Geomorphological

Summary: Causeway Alternative

This alternative involves the construction of a ~400-ft long causeway over Bolinas Lagoon to connect State Highway 1 with Seadrift Road (Figure A16). Construction of the causeway would greatly improve vehicle access during flood events and result in improved safety for the Seadrift community. To investigate potential flood mitigation in the lower Calles by controlling tidal conditions in the upper estuary, a tide gate structure and pump station were included in hydraulic modeling of the causeway alternative. The concept is that the tide gate and pumps would operate to lower the downstream tidal condition during flood events on the creek in order to reduce backwater effects in the lower reaches of Easkoot Creek.

Water levels in the estuary adjacent to the proposed causeway are controlled by coastal storm surge, tides, and runoff from Easkoot Creek and other tributaries. A feasibility assessment and preliminary design for the causeway cannot be completed until a coastal flood hazard evaluation has been completed.

Construction of a causeway including a tide gate and a pump station reduced maximum water levels in the estuary immediately upstream of the causeway by two feet (by design). This effect diminishes moving up the estuary; at Francisco Patio the reduction is approximately one foot. Above this point the flooding becomes increasingly dominated by flows from Easkoot Creek, and by Calle del Occidente, the reduction is less than 0.5 feet, and less than 0.1 feet by Calle del Arroyo (Figure A17). These reductions only result in minor decreases in flood extent and one building being removed from the December 2005 floodplain (Table ES2).

It is important to note that this analysis was only performed for Mean Higher High Water (MHHW) tidal conditions. Under more extreme tidal conditions such as occurred during the historical December 2005 flood, water overtops Calle del Arroyo and floods portions of the Patios. Under these conditions lowering water levels in the estuary via a tide gate and pump station would likely have significant flood control benefits. Evaluation of the potential for mitigating against coastal flood hazards by regulating estuary water levels at the causeway appears to be warranted but is beyond the scope of this study.

Comment [T69]: This raises the question of how the 2005 flood was modeled. Do the modeled results reflect actual flood condtions?

Comment [cc70]: Model behaved like the storm, according to TWG. It isn't quite the same, current topo with 2005 storm hydro.

Although access to Seadrift would be greatly improved under this alternative, construction of a causeway alone may not improve access for residents in the lower Calles and Patios depending on the extent of flooding on Calle del Arroyo.

Because this alternative does not provide substantial mitigation of flooding from Easkoot Creek, analyses of costs and permitting for this alternative are not provided.

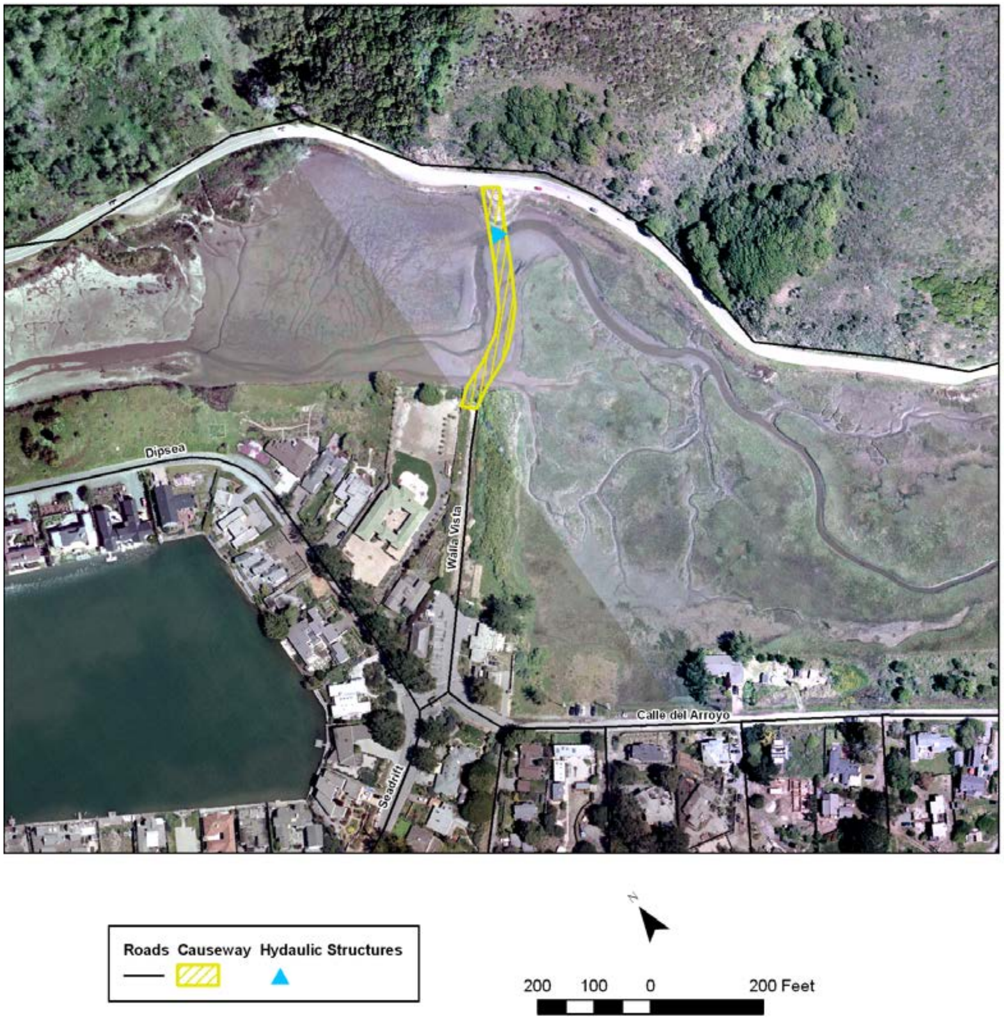


Figure A16. Overview map of the causeway alternative.

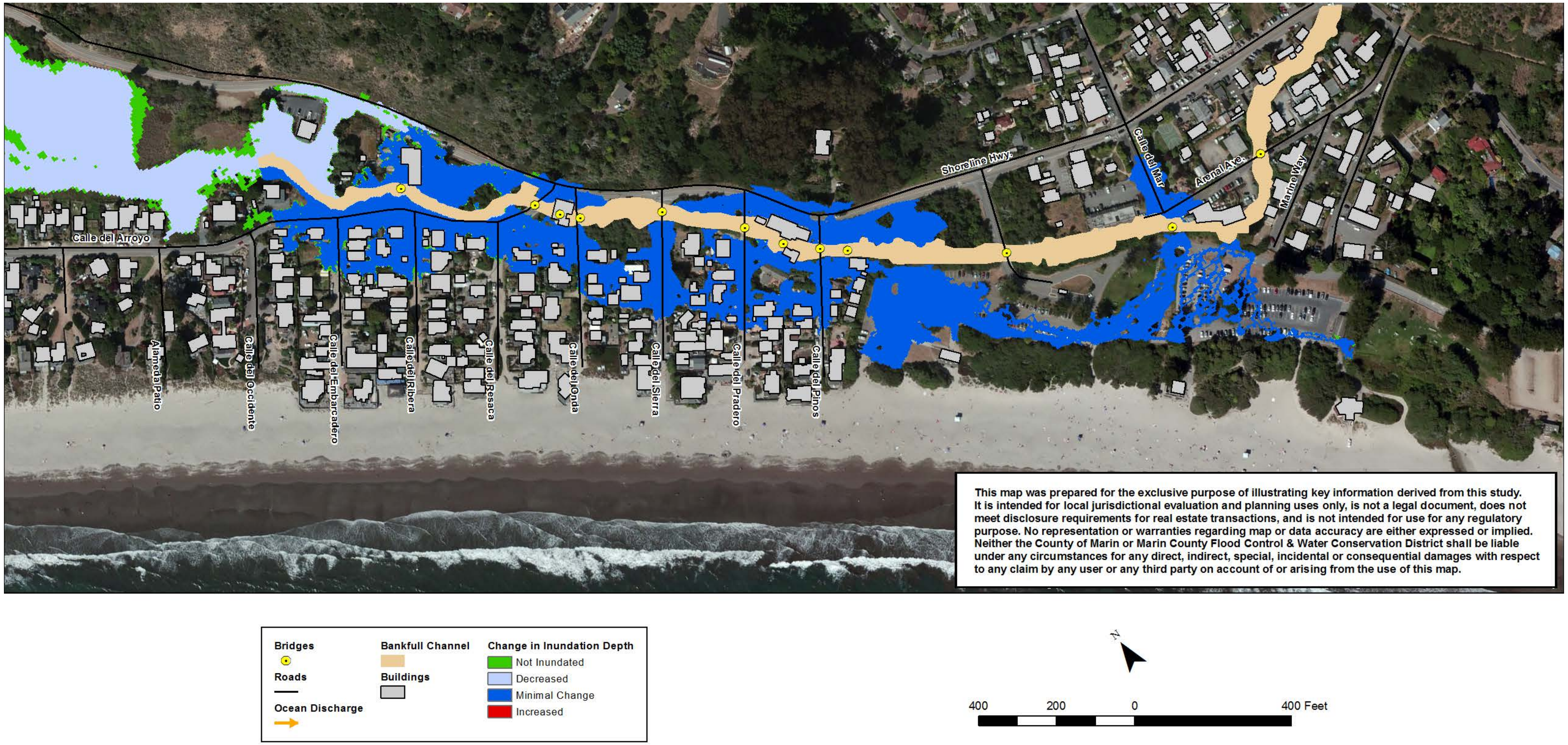


Figure A17. Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the causeway alternative for the December 2005 flood.

ALTERNATIVE: Causeway

1. Description

This alternative involves the construction of a ~400-ft long causeway over Bolinas Lagoon to connect State Highway 1 with Seadrift Road along the alignment of what is currently a gravel road named Walla Vista Road (Figure A16). The primary purpose of this alternative would be to improve access to the Seadrift community which currently relies on Calle del Arroyo as the only means of vehicular access. Portions of the roadway currently become submerged in floods as small as a 2-yr event. Construction of the causeway would greatly improve vehicle access during flood events and result in improved safety for the Seadrift, with limited benefit for the "Patios" and "lower Calles" community.

To investigate potential flood mitigation in the lower Calles by controlling tidal conditions in the upper estuary, a tide gate structure and pump station were included in hydraulic modeling of the causeway alternative. The concept is that the tide gate and pumps would operate to lower the downstream tidal condition during flood events on the creek in order to reduce backwater effects in the lower reaches of Easkoot Creek.

Water levels in the estuary adjacent to the proposed causeway are controlled by coastal storm surge, tides, and runoff from Easkoot Creek and other tributaries. A feasibility assessment and preliminary design for the causeway cannot be completed until a coastal flood hazard evaluation has been completed.

2. Flood Control Benefits

Construction of a causeway would result in improved vehicle access to the Seadrift community, and provided that it is constructed such that it does not restrict tidal action it is unlikely to have any significant effect on flooding. Inclusion of a tide gate and a pump station reduced maximum water levels in the estuary immediately upstream of the causeway by two feet (by design). This effect diminishes moving up the estuary; at Francisco Patio the reduction is approximately one foot. Above this point the flooding becomes increasingly dominated by flows from Easkoot Creek, and by Calle del Occidente, the reduction is less than 0.5 feet, and less than 0.1 feet by Calle del Arroyo (Figure A17). These reductions only result in minor decreases in flood extent and one building being removed from the December 2005 floodplain (Table ES2).

It is important to note that this analysis was only performed for Mean Higher High Water (MHHW) tidal conditions. Under more extreme tidal conditions such as occurred during the historical December 2005 flood, water overtops Calle del Arroyo and floods portions of the Patios. Under these conditions lowering water levels in the estuary via a tide gate and pump station would likely have significant flood control benefits. Also, it can be expected that the frequency with which estuary water levels overtop Calle del Arroyo will increase in the future due to sea level rise. Evaluation of the potential for mitigating against coastal flood hazards by regulating estuary water levels at the causeway appears to be warranted but is beyond the scope of this study.

Although access to Seadrift would be greatly improved under this alternative, construction of a causeway alone may not improve access for residents in the lower Calles and Patios depending on the extent of flooding on Calle del Arroyo.

3. Preliminary Design and Estimated Construction Costs

Causeway. The distance between potential causeway abutments is about 350 feet. In order for the causeway opening(s) to align roughly perpendicular to the primary flow direction in the main estuary channel, a causeway alignment with some curvature would be required and the total span of the causeway would be approximately 400 feet (Figure A16). The 340 feet gravel portion of Walla Vista Road would also likely need to be resurfaced in order to accommodate the increase in vehicle traffic using the causeway.

The highest water levels adjacent to the causeway that were simulated during this study occurred during the December 2005 flood event which coincided with a very high tidal condition. Maximum water levels adjacent to the causeway alignment during this event were on the order of 8.4-ft NAVD88. For the purposes of the preliminary conceptual design presented here we assume a design causeway elevation of 9.4-ft NAVD88 which represents 1-ft of freeboard above our highest simulated water levels. This elevation is approximately 0.5 feet higher than the existing ground elevations at the end of Walla Vista Road and approximately 1.2 feet higher than the existing ground elevations on Highway 1 where the causeway would connect.

A number of alternatives are possible for causeway construction. Although not essential from a hydraulic standpoint, providing multi-purpose capability of the system seems desirable. For planning purposes, we envision an earthen levee with a top width of about thirty feet supporting a two-lane paved road and shoulders. The levee would be of either imported fill, or of consolidated bay mud excavated and placed in sheet pile constraints. Depending on the ultimate intended use, the width could be reduced to that necessary for a one-lane roadway.

Tide Gates and Pump Station. Many options are available in terms of the number of openings in the causeway and their dimensions; for the purposes of this preliminary investigation of potential flood control benefits, a single 40 foot wide gated opening was assumed. Two concepts are possible for operating the gates for flood control purposes. One is to simply close the gates during low tidal conditions when flood flows are expected, thus isolating the upstream area from tidal influence. This would create a temporary detention basin in the portion of the estuary above the causeway. The water level in the 'detention basin' would rise as a function of inflow from the creek, and the gates would need to automatically open or overflow once the backwater elevation in the basin matched that of the external tidal elevation. A second concept would add a pump station to pump flows from Easkoot Creek past the causeway to Bolinas Lagoon and maintain the artificially lowered water level upstream of the causeway.

In this preliminary analysis, a water level of 3.8 feet NAVD88 was used as the threshold for closing the tide gate and activating the pump station. This elevation is 2 feet below the MHHW elevation of 5.8 feet NAVD88. This level was selected because it is low enough to significantly reduce water levels in the estuary and potentially reduce flooding impacts but not overly optimistic regarding the ability to anticipate flooding on Easkoot Creek in time to close the tide gates during low tidal condition.

A preliminary evaluation of the first concept revealed that the storage generated behind the causeway by artificially lowering water levels by two feet would represent only about 14% of the total December 2005 storm volume. Thus in order for this alternative to be effective, the second concept of adding a pump station is necessary. In order to maintain the 2-ft reduction in water level above the causeway, the pump station capacity would need to keep pace with Easkoot Creek discharges. This would mean maximum capacity on the order of 170 cfs to mitigate against the December 2005 flood and 470 cfs to mitigate against the 100-yr flood. These pumping rates are relatively large, and would require significant pumping capacity. We have not provided detailed cost estimates because it is clear from our preliminary investigation that the flood control benefits that could be achieved with this alternative would be minor.

Comment [MO71]: We aren't incorporating them here, but we have developed some cost estimates that can be included if desired.

4. Permitting Issues

Permitting is expected to be a substantial undertaking for this option. A California Coastal Commission permit will be required, as will permission from the U.S. Army Corps of Engineers, National Marine Fisheries Service, and any other resource agency with jurisdiction over tidal waters. California Department of Fish and Wildlife and the Regional Water Quality Control Board will likely be involved. If dredged bay mud is used for levee construction, a detailed plan and permits will be required for the dredging alone. Impacts to flora and fauna will need to be documented. Given the limited flood mitigation benefits of regulating flows and water levels at the causeway, we have not provided detailed consideration of permitting issues.

5. Operation and Maintenance Requirements and Costs

Operation and maintenance requirements or costs have not been developed.

6. Sustainability (Short-term and Long-term)

The sustainability of this alternative has not been considered.

7. Feasibility and Next Steps (Additional Information Needs)

Following the completion of a coastal flood hazard evaluation, we recommend re-visiting the concept of regulating water levels and flows via a causeway with tide gates and/or a pump station as a possible means of mitigating against coastal flooding. Consideration should be given to an alternative causeway location farther south near the Stinson Beach County Water District office that might provide more effective flood mitigation in the lower Calles.

Summary: Calle del Arroyo Improvements Alternative

Portions of Calle del Arroyo become submerged in floods as small as a 2-yr event significantly restricting vehicular access to the Lower Calles, Patios, and Seadrift areas during flood conditions. This alternative is designed to improve access for residents of these areas by elevating the entire length of Calle del Arroyo between Highway 1 and Seadrift Road; a distance of approximately 2,840 feet (Figure A18). Elevating the roadway is also expected to restrict elevated water levels in the Easkoot Creek estuary from breaching the roadway and inundating residential areas. Given that elevating the roadway represents placement of fill within an active floodplain area, the design must include drainage features to prevent floodwaters from backing up behind the roadway potentially exacerbating flooding impacts.

Our analysis suggests that elevating Calle del Arroyo can be accomplished without exacerbating riverine flooding provided that sufficient drainage is provided for flood flows to cross the roadway and return to the estuary. By preventing overtopping of Calle del Arroyo and providing a return flow pathway back to the estuary at Calle del Resaca, some flood mitigation is possible. During the December 2005 flood a significant reduction in flood extent was achieved in the Lower Calles and two of twenty-four buildings were removed from the floodplain (Table ES2). Additionally, Calle del Arroyo remained dry which would allow for vehicle access over the full length of the roadway (Figure A19 and Table ES2). Some increases in floodplain depths do occur locally owning to water backing up behind the elevated roadway. This effect can likely be mitigated by developing a more refined design that includes additional drainage features designed to direct flows into culverts and back to the estuary, however more detailed drainage analysis is required.

Given that the area surrounding Calle del Arroyo is subject to flooding from a variety of sources including coastal storm surge, elevated tidal conditions, and riverine flooding, a design for the elevated roadway and associated drainage features cannot be fully developed until a coastal flood hazard evaluation has been completed (a task beyond the scope of this study which focuses only on riverine flooding). Assuming a preliminary design elevation for the roadway of 9.6-ft NAVD88 (1-ft of freeboard above our highest simulated water levels) yields a mean height increase of 2.3-ft requiring approximately 8,300 cubic yards of fill. Though complicated by the need to consider driveway access and existing utilities, design and construction of the elevated roadway should be relatively straightforward. Less straightforward though likely feasible would be the design of appropriate drainage features which would need to serve a variety of functions including preventing water from backing up behind the roadway, enhancing drainage of floodplain flows back to the estuary, and preventing backflows when estuary water levels are high.

Preliminary estimated cost for elevating the roadway and providing required drainage features is on the order of $1.0 million (Table A6). The cost estimate is summarized in the table below and detailed breakdowns of estimated costs are provided in Appendix C

Table A6. Summary cost estimate for Calle del Arroyo.

Bypass Alternative - Planning-level Budget Summary	**Cost ($)**	**Percent**
Consultant Planning, Permitting and Design Subtotal	68,470	6.8
Construction subtotal	710,776	70.6
Subtotal Contractor Overhead	179,200	17.8
Planning-level Cost Estimate (to nearest $1000)	958,400	95.2
Project Administration	47,920	4.8
Installed Project Cost Estimate	**1,006,320**	**100.0**

Figure A19. Overview map of the Calle del Arroyo alternative. Note a single set of return flow culverts at Calle del Resaca were modeled but additional structures are likely necessary to accommodate coastal flooding.

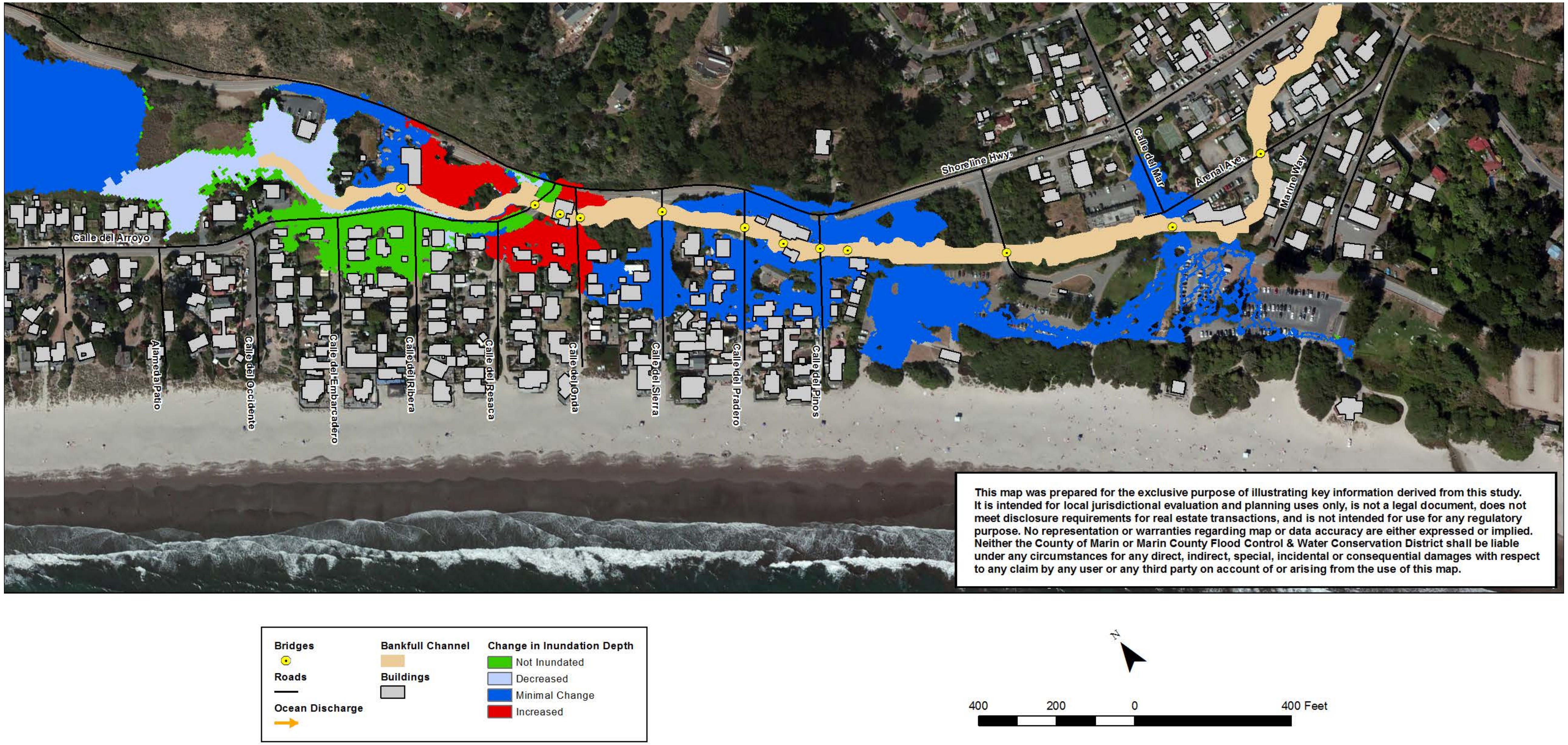


Figure A20. Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the Calle del Arroyo alternative for the December 2005 flood.

ALTERNATIVE: Calle del Arroyo Improvements

1. Description

This alternative involves elevating the entire length of Calle del Arroyo between Highway 1 and Seadrift Road; a distance of approximately 2,840 feet (Figure A18). The primary purpose of this alternative would be to improve access to the Lower Calles, Patios, and Seadrift community which rely on Calle del Arroyo as the only means of vehicular access. Portions of the roadway currently become submerged in floods as small as a 2-yr event. Elevating the roadway would greatly improve vehicle access during flood events and result in improved safety for these portions of the Stinson Beach community.

Given that water levels in the estuary adjacent to the roadway are subject to flooding from a variety of sources including coastal storm surge, elevated tidal conditions, and riverine flooding, a design for the elevated roadway cannot be fully developed until a coastal flood hazard evaluation has been completed (a task beyond the scope of this study which focuses only on riverine flooding). The highest water levels adjacent to the roadway that were simulated during this study occurred during the December 2005 flood event which coincided with a very high tidal condition. Maximum water levels adjacent to the roadway during this event were on the order of 8.6-ft NAVD88. For the purposes of the preliminary conceptual design presented here we assume a design road elevation of 9.6-ft NAVD88 which represents 1-ft of freeboard above our highest simulated water levels. Using this design elevation yields a mean height increase of 2.3-ft requiring approximately 8,300 cubic yards of fill.

Given that elevating the roadway represents placement of fill within an active floodplain area, it has the potential to exacerbate flooding conditions by backing up floodplain flows or coastal storm surge behind the roadway and/or preventing these flows from re-entering the estuary. In order to mitigate against this effect, a series of culverts beneath the roadway would be required. These culverts would need flapgates on the estuary side in order to prevent reverse flows from occurring when water levels in the estuary are high. For the purposes of this preliminary analysis a single set of three 36 inch circular culverts with downstream flapgates was evaluated at a location on the downstream side of Calle del Resaca (Figure A18). It is important to note that because of the potential for storm surge to carry water from the ocean up the Calles and Patios and towards the estuary, drainage beneath Calle del Arroyo would likely be needed at additional locations throughout the lower Calles and Patios.

2. Flood Control Benefits

Our analysis suggests that elevating Calle del Arroyo can be accomplished without exacerbating riverine flooding provided that sufficient drainage is provided for flood flows to cross the roadway and return to the estuary. Our analysis of flooding patterns under existing conditions revealed that flooding along the left bank within the lower Calles reach results both from overtopping of Calle del Arroyo as well as from a floodplain flow path that originates farther upstream. The hydraulic modeling results for the December 2005 flood demonstrate that by preventing overtopping of Calle del Arroyo and providing a return flow pathway back to the estuary at Calle del Resaca, some flood mitigation is possible with this alternative. Nearly all of the floodplain flow on the left bank was able to return to the estuary via the Calle del Resaca culverts and a substantial area was removed from the floodplain downstream (Figure A19). This resulted in the removal of two of the twenty-four buildings from the December 2005 floodplain in addition to allowing for vehicle access over the full length of Calle del Arroyo (Figure A19 and Table ES2). The results do however show some increases in floodplain depths of as much as 0.6 feet between Calle del Onda and Calle del Resaca owning to water backing up behind the elevated roadway. This effect can likely be mitigated by developing a more refined design that includes additional drainage features designed to direct flows into culverts and back to the estuary.

3. Preliminary Design and Estimated Construction Costs

Calle del Arroyo is a relatively straight ~20 foot wide roadway oriented in a NW-SE direction and stretching some 2,840 feet from Highway 1 on the southeast to Seadrift Road on the northwest. In general, and although indistinct, the road surface is located at the high point of local topography. Local soils and drainage are such that little or no defined or developed drainage ditching is observed along the route, and no culverts are observed under the roadway. Over its length, twelve private roadways (mostly gravel) intersect the road, all entering from the south. There are two stop signs for traffic speed control, one at Calle del Occidente and one at Joaquin Patio.

An overhead power corridor traverses the length of the roadway with most poles located about 20 feet north of the edge of pavement. Power lines cross the road at a skew angle on the east end of the study area, with some poles within 5 feet of the pavement. An underground water main serving the Calles and Patios as well as Seadrift is likely located within the right-of-way. Residences are believed to be served by individual onsite septic systems rather than by a sanitary sewer system with force main within the right of way.

The north side of the road is relatively less developed than the south side and has a shoulder width of about twelve feet. An area of clustered houses is present on the north side between Calle del Occidente and Francisco Patio. A fire station with paved parking is located across from Calle del Occidente, and a thirty x fifty foot graveled parking lot is located across from Sonoma Patio. The south side has twelve access road intersections and several individual stand-alone driveways. In some locations, local fences and landscaping come to within a few feet of the roadway. Developed shoulder and parking is much less prevalent than on the north side.

A design for elevating the roadway cannot be fully developed until a coastal flood hazard evaluation has been completed which is beyond the scope of this study which focuses only on riverine flooding. Based on consideration of riverine flooding only, a preliminary design elevation of 9.6-ft NAVD88 is assumed. This elevation would provide 1-ft of freeboard above the highest water levels simulated for this study. Using this design elevation yields a mean height increase of 2.3-ft requiring approximately 8,300 cubic yards of fill.

A key design element will be locating and sizing return flow culverts so that floodwaters can pass the roadway and return to the estuary. The following design considerations pertain to the culverts:

- Hydraulic modeling for the December 2005 flood indicates that a single set of three 36 inch culverts located near Calle del Resaca would provide sufficient drainage to permit the ~50 cfs of floodplain flow on the left bank to return to the estuary.
- Additional culvert locations would likely be needed to accommodate larger riverine floods and/or storm surge.
- Final locations and sizing should be determined based on consideration of both riverine and coastal flood hazards.

- Culvert inlets need to be placed at relative topographic low points. Such low points may require manufacture, swale creation, and routing to enhance drainage from low points within residential areas.
- Supplemental fill over the existing roadway of about 2.3' is proposed to create the flood control levee. Assuming a 12 inch minimum culvert cover allowance for development of load bearing capacity results in a maximum culvert diameter of about 12 inches (O.D. about 15 inches) if placed on local grade. If existing or created low swales are available for placement culvert diameter may be increased.
- Smoothbore culverts at 1% slope have an approximate pipe full capacity as noted below. Flow will be de-rated to about 60-70% of that shown due to entrance effects. Placement of a flap or rubber lipped valve at the outlet may further restrict flows. Flows shown are not developed unless the entrance is submerged enough to develop full pipe flow, which may not occur with a maximum available head of 12 inches above the entrance.

Diameter (inches)	Capacity (cfs)	Velocity (fps)	Culvert Count for 100 cfs
12	4	4.5	25
15	7	5.8	15
18	12	6.4	9
24	24	8.0	4
30	42	9.0	3
36	60	10.0	2

- Culvert banks in multiples providing (yet unknown) design return flow values will be required. Consideration of culvert inlet control as a flow limitation condition is necessary due to the low available head, further increasing culvert counts at flood return discharge points.
- At half depth, flows will be about half of full depth flows, resulting in backwater accumulation behind the culverts. Flooding will therefore not be totally mitigated by presence of flow relief culverts, because of the stage-discharge characteristics and the backwater elevation required to achieve design flows. In such a case flood elevations may not be significantly reduced, however flood durations may be reduced.
- Culvert discharge flows need to return to the creek or estuary in a non-erosive fashion. It may be necessary to provide armored discharge channel construction on/over private property in order to accomplish this goal.
- Culvert backflow is envisioned to be prevented by use of flap gates at the outlet end. Alternative devices may be commercially available. Units used should provide full flow at very low head, so as to provide the intended performance under flood flow conditions.
- Individual culvert performance and design should consider and use the minimum capacity as determined by inlet conditions, head constraints, pipe flow constraints, and outlet (flap valve) constraints.
- Post-flow flap valve maintenance may be required on an event-based schedule to ensure that debris or trash does not foul the apparatus or allow reverse flows.

- Some kind of risk management document may be appropriate to absolve the responsible agency from flood damage claims in the event that flow control devices fail and allow reverse flows and flooding where not already present.

The following considerations pertain to construction:

- The proposed work is considered technically feasible, and does not invoke any extra-ordinary construction methods or techniques.
- A detailed route survey is required to identify all ground features appropriate for engineering design of the project.
- A detailed engineering design is required in order to accommodate site-specific constraints on a case-by-case basis.
- Cooperative agreements, easements, or other formal agreements may be required in cases where the proposed work encroaches on private property. Eminent domain procedures may be required if recalcitrant owners are encountered because project integrity requires complete and seamless coverage of the route.
- Fill depth is not great and should be of imported base rock rather than soil, in order to preserve road subgrade integrity.
- Lateral sloped fill prism at road shoulders would need to be 23 feet wide in order to maintain a 10% side slope. A steeper shoulder side slope would not be recommended due to vehicle safety and parking considerations. Lateral slopes of 10% may not be achievable in some areas.
- Installation of low retaining walls with guard railing may be required in some areas where lateral offset distance is not available for gravel prism creation.
- Lateral slope of 10% extending into the Fire Station parking lot may direct rainfall towards/into the building, requiring installation of secondary drainage facilities for mitigation. This might include placement of a slot drain at the toe of slope parallel to Calle del Arroyo.
- The right-of-way may contain underground utility access points including but not limited to manhole covers, inspection ports, junction boxes, and survey monumentation. Each will need to be identified and preserved during site work, extended about 2.3 feet in elevation.
- The old pavement should be ground up and recycled, so that new fill is not placed on a discontinuity or layer providing moisture detention.
- New pavement will be required, covering a minimum area of about 52,800 square feet; additional paving on the street approaches in the amount of 5,520 square feet is highly recommended.
- Public and emergency vehicle access over the roadway will be required at all times during demolition and reconstruction.

4. Permitting Issues

Work on a public street will likely be undertaken by Marin County Department of Public Works as a capital improvement project. County grading, drainage, and/or floodplain permits may be required. A California Coastal Commission permit would likely be required. Given that the work area is below 100-yr flood elevations, a permit for placement of fill within the floodplain will be required from the U.S. Army Corps of Engineers. The project is likely exempt from CDFG, RWQCB oversight, since it is not conducted within those jurisdictional areas. If however, return flow channel construction occurs below the top of bank of Easkoot Creek, CDFG and other resource agency permitting may be required.

5. Operation and Maintenance Requirements and Costs

A properly designed roadway should have low maintenance requirements. Depending on pavement section used and local environmental conditions, a service life of at least 20 years is anticipated. Periodic maintenance would be expected to be necessary to provide satisfactory long-term performance. Culvert life should match roadway life if properly installed. The proposed flood routing culverts would normally be dry and not subject to scour or wear. Depending on the nature of flood flows, silt deposition in low-slope low-flow culverts may occur over time. Culverts fitted with flap valves may retain debris or trash, requiring regular maintenance to ensure satisfactory performance. Risk of flooding by unanticipated reverse flows may increase if event-based maintenance is not practiced.

6. Sustainability (Short-term and Long-term)

A properly designed and installed, road surface should have a reasonable 20-year design and economic life. Selection of materials that are resistant to groundwater intrusion would be needed in this low elevation coastal environment.

7. Feasibility and Next Steps (Additional Information Needs)

- Commission detailed ground survey for design and planning purposes including:
  - Parcel and ROW limits
  - Overhead utilities infrastructure
  - Underground utilities infrastructure
  - Local drainage
  - Relative high and low points of roadway
  - Potential culvert locations for estuary return flows
- Develop a refined design based on survey results and consideration of both riverine and coastal flood hazard conditions.
- Perform additional hydraulic modeling to test the refined design, ensure appropriate culvert configurations, and evaluate the expected flood mitigation potential from both coastal and fluvial flood hazards.
- Obtain public comments on proposed alternative.
- Determine property ownership and parcel – Right of Way limits along Calle del Arroyo.
- Preliminary design by DPW or outside consultant in conformance with DPW requirements.

Combination Dredge and South Bypass Alternative

1. Description

This alternative combined the Dredge and South Bypass/Poison Lake Restoration alternatives. Dredging involves removing 3,100 yards of material from 2,300 feet of Easkoot Creek, lowering the channel by 2.4 feet on average. Installation of a series of sedimentation structures is also proposed to help reduce deposition in the lower channel and extend the life of the dredged profile. The bypass concept involves diverting water during high flow conditions from a location adjacent to the Parkside Cafe and discharging it through a bypass channel to a restored wetland in the vicinity of historical Poison Lake.

2. Flood Control Benefits

The flood control benefits of dredging and bypassing flows to a restored Poison Lake are substantial. During the December 2005 flood, the bypass carries up to 97.8 cfs or 57% of the total discharge above the bypass of 171.3 cfs. Note that lowering the elevation of the diversion weir crest (which is possible because of the lower dredged profile) results in an additional 25.2 cfs entering the bypass compared to the stand-alone bypass alternative. The combined dredge and bypass completely eliminates flooding for the December 2005 event with the exception of a small stretch of Calle del Arroyo near Calle del Ribera (Figure A20), and all twenty-four buildings are removed from the floodplain (Table ES3). The average reduction in peak water levels in the channel is 3.6 feet in the reach adjacent to the Parkside Café, 2.2 feet in the reach extending from Calle del Pinos to Calle del Arroyo (Upper Calles), and 0.8 feet in the reach between Calle del Arroyo and Calle del Occidente (Lower Calles) (Table ES1).

During the 100-yr flood, the bypass carries up to 329.5 cfs or 68% of the total discharge above the bypass of 482.7 cfs. Flooding above Calle del Pinos is completely eliminated. Only minimal reductions in flood extent occur within the Calles, however floodplain depths are reduced significantly throughout the majority of the inundated area (Figure A21). The average reduction in peak water levels in the channel is 3.4 feet in the Parkside Café reach, 1.4 feet in the Upper Calles, and 0.1 feet in the Lower Calles (Table ES2). Approximately twenty-three of fifty-nine buildings (39%) are removed from the 100-yr floodplain (Table ES4).

Under 2050 sea level rise conditions, the mitigating effects of the alternative do not change significantly upstream of the Calle del Arroyo crossing (Figure A22). Below this point flood extent and floodplain depths are still reduced relative to existing conditions but the improvements are much less throughout the Lower Calles reach. Farther downstream water levels in the estuary overtop Calle del Arroyo in the vicinity of Alameda Patio, and between Walla Vista and Rafael Patio resulting in flooded areas that were dry under existing conditions with the lower MHHW tidal condition (Figure A22).

Comment [T72]: Was this evaluated for the other alternatives?

Comment [cc73]: Yes, for those that were promising. If they showed benefits for the 100-year, the SLR was also modeled.

3. Preliminary Design and Estimated Construction Costs

This alternative combines the features of the Dredge and South Bypass/Poison Lake Restoration alternatives. The reader is referred to the chapters discussing these alternatives individually for details regarding the alternative designs. The only departure made from a simple combination of the individual alternatives is that the elevations of the weir crest and upper-most reach of the bypass channel were lowered to conform to the dredged channel profile and allow an even larger percentage of the flow in Easkoot Creek to enter the bypass channel. The weir crest elevation was lowered from 24.8 feet to 22.1 feet NAVD88 which maintains activation of the bypass channel when water depths in the creek reach approximately 1-ft. The slope of the upper ~40 feet of the bypass channel is reduced in order to conform to the lower weir crest; below this point the bypass channel remains as described in the stand-alone South Bypass alternative.

4. Permitting Issues

The same permitting issues discussed for the individual Dredge and South Bypass alternatives apply to this combined alternative and the reader is referred to these chapters for more details.

5. Operation and Maintenance Requirements and Costs

The same operation and maintenance requirements and costs discussed for the individual Dredge and South Bypass alternatives apply to this combined alternative and the reader is referred to these chapters for more details.

6. Sustainability (Short-term and Long-term)

The same sustainability considerations discussed for the individual Dredge and South Bypass alternatives apply to this combined alternative and the reader is referred to these chapters for more details. Results from the sea level rise analysis suggests that the mitigating effects of this alternative will likely be sustained under 2050 sea level rise conditions above the Calle del Arroyo bridge but will become diminished (though not eliminated) farther downstream.

7. Feasibility and Next Steps (Additional Information Needs)

The same feasibility and next steps considerations discussed for the individual Dredge and South Bypass alternatives apply to this combined alternative and the reader is referred to these chapters for more details.

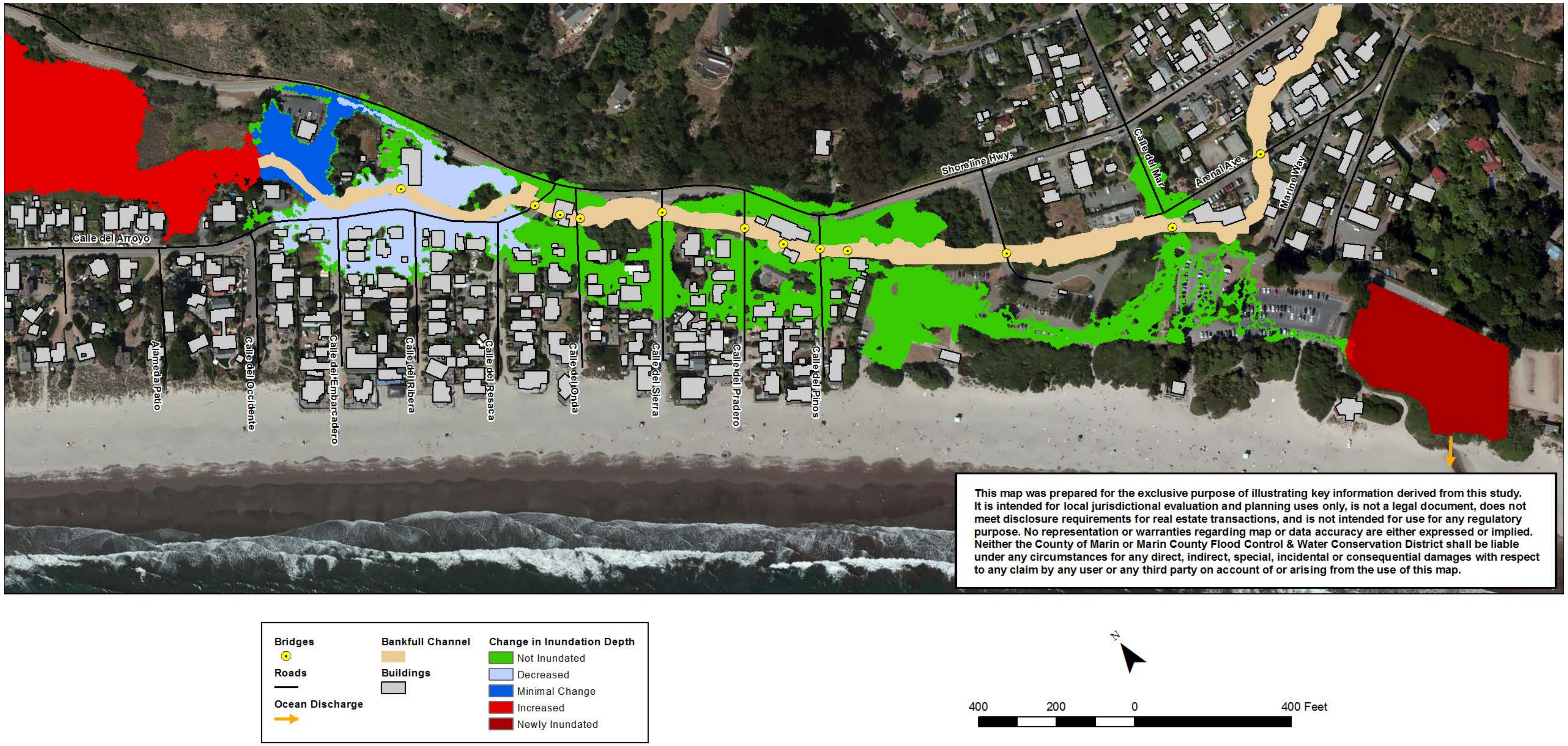


Figure A20. Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the Combined Dredge and South Bypass alternative for the December 2005 flood.

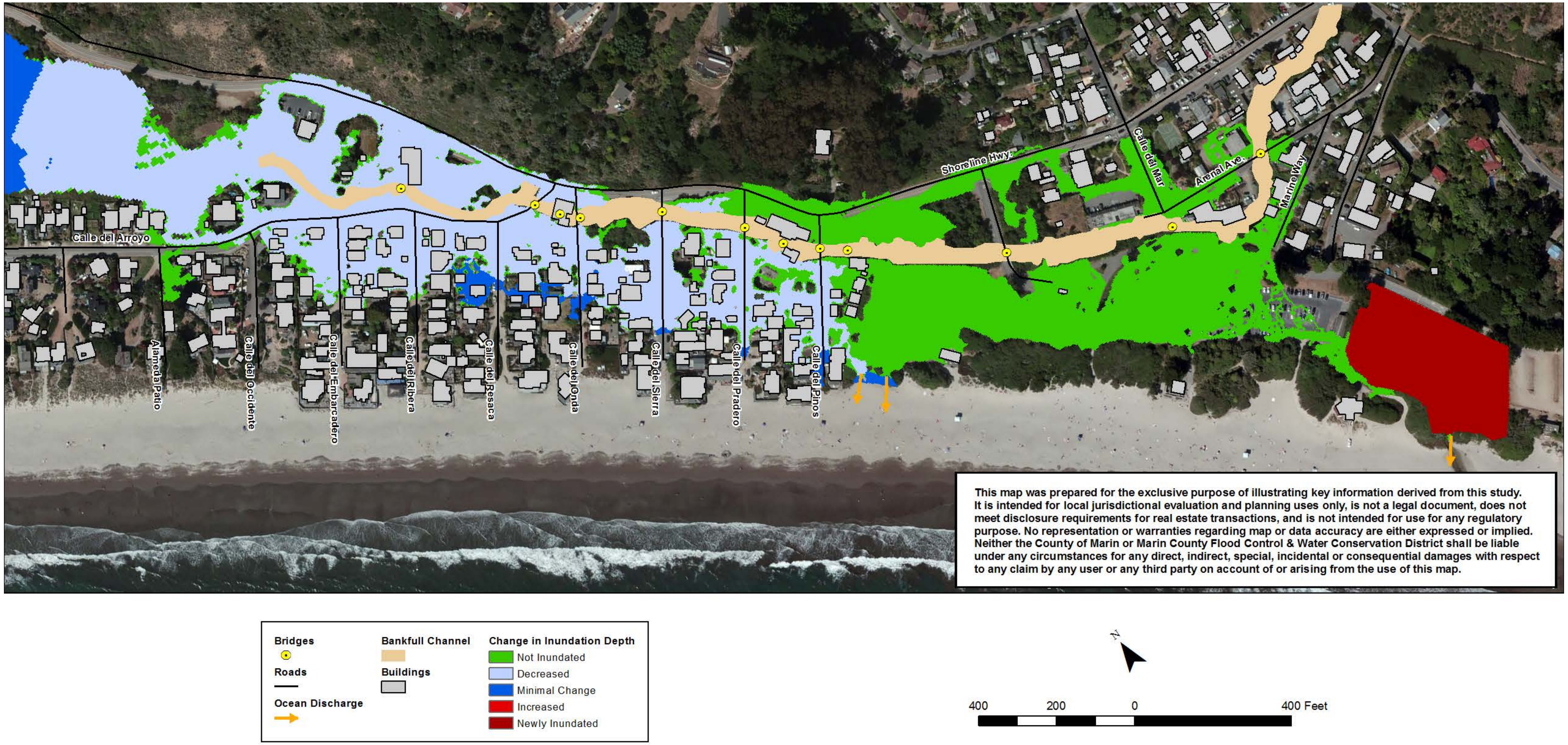


Figure A21. Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the Combined Dredge and South Bypass alternative for the 100-yr flood.

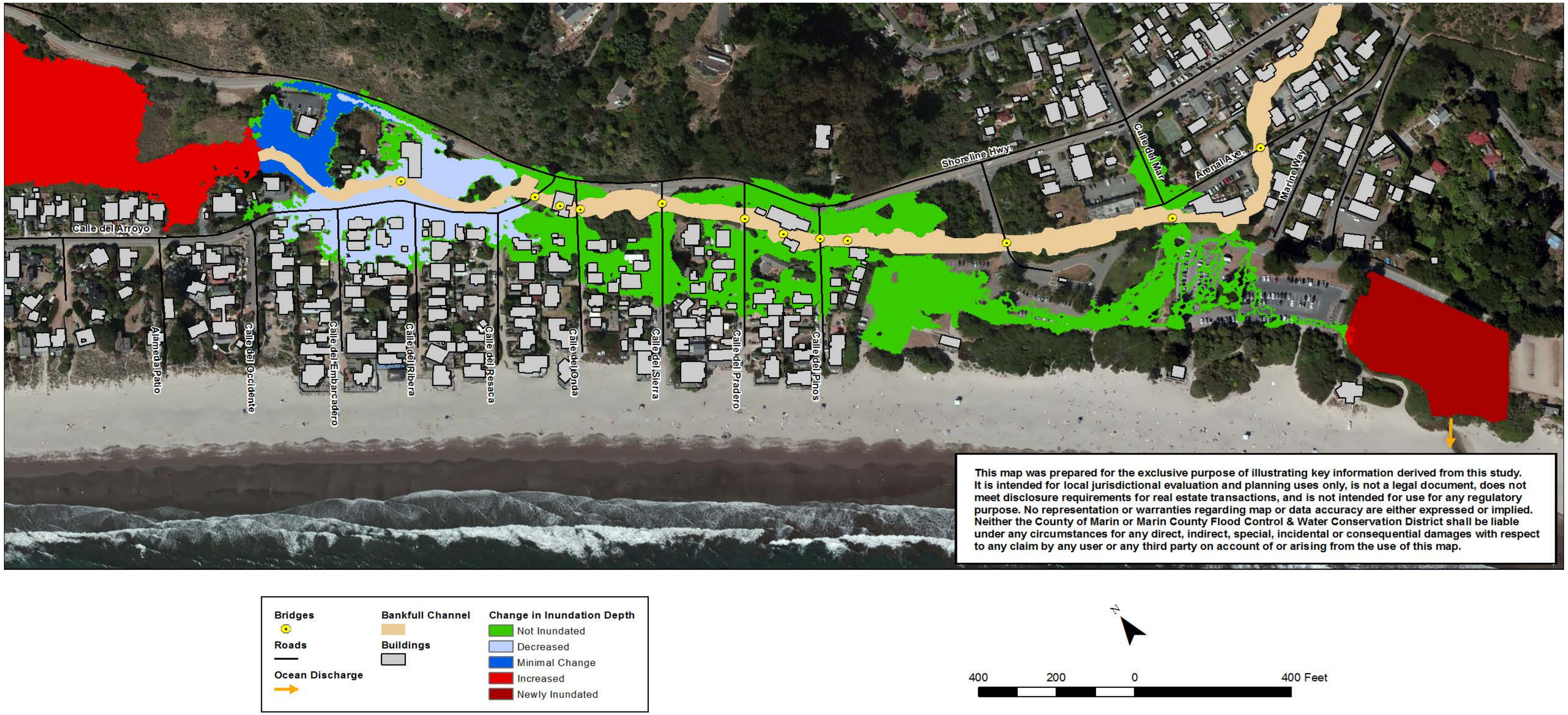


Figure A22. Decrease in flood extent, floodplain depths, and buildings removed from the floodplain under the Combined Dredge and South Bypass alternative for the sea level rise scenario.

From:	Fong, Darren
To:	Choo, Chris
Cc:	Lewis, Liz; Daphne Hatch
Subject:	Fwd: Stinson Flood Study
Date:	Tuesday, May 14, 2013 8:46:33 AM

Hi Chris, I'm forwarding you a set of emails from our Water Resources Division staff. Ideally, I would have collated the comments, but wanted to get them to you before you leave. Gary has a hydraulics background and Joel (next email) is the NPS lead wetland person. However, I can chat with you at a later point about some of Joel's comments. Also, I've uploaded Tamara's comments to a Google drive. You should be getting a separate invite to access that. My comments will be coming a bit later today. Sorry for the delay. Darren

Darren Fong
Aquatic Ecologist
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---------- Forwarded message ----------
From: **Smillie, Gary** <gary_smillie@nps.gov>
Date: Fri, May 10, 2013 at 3:19 PM
Subject: Re: Stinson Flood Study
To: "Fong, Darren" <darren_fong@nps.gov>
Cc: Tamara Williams <tamara_williams@nps.gov>

Darren -

I read through the hydrology/hydraulics report today and believe that the work was well done. The report is well written and demonstrates rigor in the work performed. I think the contractor put a good effort into the work and properly indicated the strengths and weaknesses of the effort by looking at things often ovelooked like the quality of lidar information, the difficulty in really nailing down the flood frequency info, etc. I am not extremely familiar with this creek or the setting so I was not able to quibble with the detail of some of the assumptions made by the consultants, but saw nothing in the write-up that caused me concern. In summary, I believe you have a credible hydrology/hydraulics study to help guide this effort.

Let me know if you have any further questions,

Gary

Gary M. Smillie
Hydrology Program Lead
National Park Service, Water Resources Division

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On Thu, May 9, 2013 at 11:52 AM, Fong, Darren <darren_fong@nps.gov> wrote:

> Hi Gary, I was just wondering if you had a chance to look at the Stinson report and when you might have comments available, especially for some of the hydraulics modeling. We don't have the expertise to determine whether their analyses are adequate for the alternatives that they are proposing. Thanks!
> Darren
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