

Federal Flood Control Channels in San Francisco Bay Region -
A Baseline Study to Inform Management Options for Aging Infrastructure

By

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Abstract

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This dissertation focused on flood control channels in urban areas built by the USACE between the 1950s and 1970s. These uniform-shaped earth or concrete lined channels were designed to “control” flooding and to make possible expanded floodplain developments. As many of these channels have been in service for over 50 years, it may be instructive to reassess them to examine how they were planned, designed, and maintained, and to identify the current conditions and issues. The assessment provides the basis to develop management strategies for this aging infrastructure. In this dissertation, I examined the flood control channels in the Federal scale and the local scale.

At the Federal scale, I reviewed the history and evolution of USACE policy, planning process, design criteria, and the 100-year flood standard. I also conducted a systematic evaluation of nine urban flood control channels built by USACE between the 1950s and 1970s in the San Francisco Bay Region. I found the commonality of problems in these flood control channels, driven by the USACE planning policies when these channels were built. The finding forms the justification to develop a channel safety program to manage all USACE flood control channels at the federal scale.

The policy review found that the planning process for these projects singly focused on national economic benefits, while environmental consequences were largely excluded. The land enhancement benefit in the cost benefit analysis encouraged floodplain and tidal marsh developments. However, habitat elimination resulting from the developments was not considered as a cost. The land enhancement benefit had significant impact on project justification. The San Francisco Bay Area case study found that six of the nine projects included the land enhancement benefit in the cost benefit analysis. If the land enhancement benefit were subtracted from the total project benefit, three of the six projects would not have been authorized due to low cost benefit ratio. This implies that if habitat protection had equal footing as flood risk reduction, as a planning criterion for these projects, many of the projects would not have been authorized in the form they were constructed.

The primary design criteria for these flood control channels were to maximize flow capacity

within a minimal footprint. The hydraulic design assumed a clear water condition, ignoring sediment effects. However, oversized thalwegs invite in-channel sediment deposition, and sediment transport increases channel roughness. Both effects reduce channel capacity. The clear water assumption led to questionable channel designs. The San Francisco Bay Area case study shows that six of the nine flood control channels have channel thalweg elevation at the outfall lower than the adjacent bay bed, created sediment sumps. The study also found that eight of the nine flood control channels had concrete channel reaches designed under supercritical flow. Such design demands the channel to function as designed under the clear water condition.

The San Francisco Bay Area case study found that the maintenance guidelines were ambiguous, but sediment deposition created significant operation and maintenance burden to local sponsors. On average the current sediment maintenance cost is 5 times higher than the original design estimate. Due to sediment deposition and channel deterioration, the average channel capacity reduced by 31%, as compared to the original design capacity. None of the channels in the study have the capacity to convey the existing 100 year flow. Even under the original design condition, five of the nine channels cannot provide existing 100 year flow capacity, considering two of the five channels were designed for the Standard Project Flood. This result is a matter of concern, since even if the channels were maintained to the original design specification and under the clear water condition, these channels still cannot provide 100-year flood protection.

The Federal scale analysis identified common issues among these flood control channels. These channels provide a false sense of flood protection security to the communities they serve, and the resultant floodplain developments further increase flood risk. At the Federal scale, the findings form the justification basis to develop a channel safety program to manage all USACE flood control channels. The proposed program would be similar to the existing dam and levee safety programs. The proposed program provides a management framework for the entire USACE flood control channel portfolio, for periodic inspection and risk assessment, condition review and classification, and critical improvement prioritization for flood risk reduction.

At the local scale, I conducted a detail review on the planning process of the Corte Madera Creek flood control project, to validate the findings from the USACE planning policies review. I also developed a hydraulic analysis to evaluate how sediment management schemes affect channel capacity.

The case study at Corte Madera Creek further illustrated the sediment management issue found in the Federal scale analysis. The local sponsor devised a more efficient sediment removal plan for 100 year flood protection, however it still costs 14 times more than the original maintenance budget estimate. Furthermore, the proposed flood protection projects in the upper watershed were designed based on the flood control channel providing 5,600 cfs capacity, about 2,000 cfs lower than the original design capacity. The analysis shows that even if the channel were maintained in accordance with the USACE operation and maintenance manual, the channel still would not provide the capacity needed. This finding validated the fundamental project design issues found at the Federal scale analysis.

At the local scale, my analysis identified a need to update the USACE operation and maintenance manual, to specify sediment removal frequency and volume based on the required

channel capacity. Local sponsors should develop a sediment deposition and channel capacity relationship, so the local sponsor can assess the need for sediment removal based on channel flow and stage monitoring data. The channel safety program provides the management tool to systematically prioritize and implement these updates to USACE projects.

USACE should update the engineering manual for flood control channel project planning and design. The revision should include methods to estimate channel roughness coefficient under various sedimentation and sediment transport conditions. To provide the technical basis to update the engineering manuals and project assessment methods, additional research is needed to (1) investigate how bedload sediment transport on smooth surface with no deposition affects channel roughness, and (2) improve existing risk analysis tools to quantify the stage discharge uncertainty.

In addition to managing the aging infrastructure to maintain its level of service, there is a need to develop intervention strategies to reinvent flood control channels to meet contemporary objectives in ecological restoration, floodplain management, recreation, and flood risk reduction. The intervention also needs to overcome the space limitation due to urban encroachments in the floodplain and often up to the channel banks.

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CHAPTER 1 – INTRODUCTION

1.1 Introduction

My professional and research interest is flood management in urban areas. Flood management is a long standing watershed planning topic. Valley floors are attractive areas for human settlement (Dunne and Leopold 1978), because they provide a convenient transportation corridor, fertile soil for agriculture, flat topography for urban development, and easy access to creeks and estuaries for water supply and recreation. For these reasons and increasing land development pressure, human societies often overlook the risk of flooding and urbanize these flood prone areas. Over time, urbanization alters watershed hydrology and confines natural floodplain corridors (Leopold 1997, Palhegyi and Bicknell 2004). Increased peak flow, and reduced flow conveyance and storage capacity due to sedimentation exacerbate flooding in urban streams, especially across flat alluvial floodplains (Kondolf and Keller 1991).

A primary goal of flood management is to minimize flood damage and loss of human life. Gilbert White (1945) outlined eight basic approaches for structural and non-structural flood management, the structural methods alter channels and floodplains to convey, confine, and detain natural water flow; the non-structural methods focus on land use planning, emergency warning and flood insurance. In the United States, the U.S. Army Corps of Engineers (USACE) and Federal Emergency Management Agency (FEMA) lead the nation's flood management efforts. The USACE flood management approach has historically been relied on structural methods. FEMA provides non-structural flood management, specifically through their National Flood Insurance Program (NFIP).

Structural methods have long been the preferred flood control approach. When originally constructed, these engineering solutions projected an image of certainty based on their predictable performance and reliable protection to filter out flooding up to the design storm. It have been a hallmark of engineering flood management especially in the USACE since the 1930s. There are three basic categories of structural methods:

- **Conveyance Methods.** The goal is to increase the flood flow conveyance capacity, to reduce the flood stage and ponding extent. Conveyance methods include channel expansion, diversion and bypass systems with pipes and canals, and mechanical pumping.
- **Detention Methods.** The goal is to attenuate flood flow by providing storage upstream of the area being protected from flooding. Detention methods include detention basins, reservoirs, and diversions to natural lakes and ponds.
- **Barrier Methods.** The goal is to confine flood flow within the channel corridor to protect the adjacent low-lying floodplain area. Barrier methods include small scale berms and mounds to protect a localized area, and large scale levees and floodwalls to provide regional flood protection. The protected areas require interior drainage systems to provide protection from flow originating within the protected areas. The interior drainage system may include combinations of structural and non-structural methods.

Barrier and conveyance methods, including levees and flood control channels, are widely used to contain creek flow within the channel corridor and prevent flood flow from entering adjacent floodplains during high frequency, low intensity flood events. Detention methods, including dams and detention basins, are typically implemented at the upper watersheds to capture and contain creek flow before it travels downstream. These detention methods also often provide water supply and recreational benefits.

The motivation to control nature, as manifested in the term “flood control”, drove early flood management towards a utilitarian approach that relied on structural methods. The approach often solely based on flood protection efficiency. Environmental considerations and other social factors were mostly ignored. As a result, natural streams were converted to straight uniform concrete or earth channels, valleys were dammed to create reservoirs, and levees were built to disconnect riparian floodplains from the creeks. These structural methods significantly altered the floodplain landscape, especially by flood control channel constructions in urban areas, encouraging development in the newly flood protected floodplains (USACE 2012).

When these projects were constructed, on paper, they typically fulfilled the immediate flood protection objectives they were designed to achieve. However, their long term sustainability is questionable (Williams 1990). Many of these projects were built decades ago, the project management, performance, operation and maintenance histories could provide valuable insight on how to manage these existing projects, and how to improve future project design, planning process and policy.

Motivated by the abundance of these utilitarian structural flood protection projects, and the need to identify options to manage these aging projects, this dissertation focuses on a subset of the structural flood management methods: The flood control channels in urban areas built by the USACE from the 1950s to 1970s. During this period the USACE actively constructed single purpose flood control channels throughout the United States. These channels have narrowly focused objectives to reduce flooding and increase developable land in the floodplains. To meet these objectives, uniform shaped earth or concrete lined channels were designed to straighten the creek alignment and disconnect the floodplain from creek overflow. As many of these channels have been in service for over 50 years, the economic life milestone (WRC 1983), it is valuable to reassess these channels to learn their performance and operation and maintenance histories. The assessment also provides an understanding of their existing condition and provides the justification and basis to develop options for managing these aging infrastructure.

1.2 USACE Civil Work Program

This section provides an introduction to the USACE, and the flood protection mission within its civil works program.

The USACE is a federal agency within the Department of Defense with military and civilian responsibilities. The military program provides engineering, construction, and environmental management services for Department of Defense agencies. Under its civil works program, at the direction of Congress, USACE plans, constructs, operates, and maintains a wide range of water

facilities. In addition, USACE established the Engineer Research and Development Center (ERDC), the Institute for Water Resources (IWR), and a collection of the Center of Expertise to provide research, advance planning and technical support (Carter 2005).

The civil works program is headed by a civilian Assistant Secretary of the Army for Civil Works. A military Chief of Engineers oversees USACEs' civil and military operations and reports on civil works matters to the Assistant Secretary for Civil Works. The Corps operates as a military organization with a largely civilian workforce, with 34,600 civilian and 650 military personnel (Carter and Coby, 2005). Eight divisions throughout the nation coordinate projects in 41 district offices in the United States and field offices worldwide. Projects are largely planned at the district level and approved at the division and headquarters levels (Carter and Coby, 2005).

The three primary missions of the USACE's civil works program are facilitating commercial navigation, reducing the risk of damage from floods and storms, and restoring aquatic ecosystems. This section provides an overview of important USACE legislations and flood control project planning policies. Detailed discussion is covered in Chapter 2.

The key Congressional legislation governing the USACE is the Water Resource Development Act (WRDA), and, before 1972, the Flood Control Act (FCA). These acts authorized the USACE to implement flood control projects and studies and to manage and maintain USACE assets. These acts also provided programmatic authorizations, under the Continuing Authorities Program, to implement smaller projects for flood control and ecosystem restoration (Carter and Coby 2005). The FCA of 1936 is a significant milestone in flood management (U.S. Congress 1936). The act officially established the USACE's commitment to provide flood protection (Arnold 1988), and marked the beginning of applying the cost benefit analysis to evaluate project feasibility. The 1939 FCA articulated the project authorization criteria based on the cost benefit ratio, which "the benefits to whomever they may accrue are in excess of the estimated cost." (U.S. Congress 1939). From 1936 through the 1960s the USACE constructed dams, levees, and channels for flood control. By the 1970s, national priorities were shifted away from flood control projects, and no major flood control project was authorized until 1985 (Carter and Stern 2010). The 1986 WRDA transformed the ground rules for USACE. The act established a new cost sharing formulas from local sponsors. The 1990 WRDA explicitly expanded the USACE mission to include aquatic ecosystem restoration and environmental protection.

Since the 1936 FCA, cost benefit analysis has been an important planning tool for USACE. The first federal guideline for the analysis is the Proposed Practices for Economic Analysis of River Basin Projects in 1950 (FIRBC 1950), commonly known as the Green Book. The Green Book defines the objective of the cost benefit analysis should be to maximize general economic welfare and economic efficiency. The evaluation should include market and non-market benefits and costs. In 1952, the Green Book was incorporated into Bureau of the Budget, Budget Circular A-47 (1952), and became a mandatory guideline.

In 1973, the Water Resources Council issued the Principles and Standards for Planning Water and Related Land Resources (WRC 1973). The standard included National Economic Development, Regional Development, Environmental Quality, and Social Factors as equal factors to evaluate the project's cost and benefit. These four accounts standards carried over to

the Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies (WRC 1983). However it established the National Economic Development as the only mandatory objective, and the other accounts as second tier objectives. This guideline governs the cost benefit analysis and the USACE planning process in use today.

1.3 Research Topic

USACE is a major contributor to the nation's flood risk management. USACE has constructed approximately 14,500 miles of levees, 400 miles of shoreline protection, 702 dams, and 13,000 miles of inland waterways (USACE 2013). Traditionally, USACE worked on "controlling" floods to "reduce flood damages". The USACE has changed its approach from "controlling" floods to "managing flood risks" with the understanding that flood risk management projects cannot completely eliminate all flood risk and that we cannot control floods (USACE 2012).

Despite the USACE's significant efforts and influence on the nation's flood protection infrastructure, it has received its share of criticism over the years. USACE is governed by over 200 public laws that date back to the late nineteenth century (NRC 2004). As this body of guidance has accreted over time, little attention has been paid as to whether new laws and directives were fully consistent with existing ones. As a result, internal inconsistencies within these guidelines often pose problems for USACE. Inadequate guidance on how to resolve these types of conflicts places USACE in the position of setting policy, often resulting in confused policy direction, inefficient operations, and criticism of the agency from all corners (NRC 2004).

The cost benefit analysis is a mechanical method to quantify the benefits and costs in monetary terms. A proposed project is viable only if its projected benefits exceed projected costs. This method is effective to evaluate the project's financial performance based on its economic impacts and commodity market values, and it imposes discipline on the planning process. However, it often pressures analysts to make questionable assumptions or to configure a study such that it produces a given cost benefit ratio (NRC 2004).

For example, before the 1970s, the two main project benefits are flood damage reduction and land enhancement. Land enhancement is the benefit resulting from changes in use of property made possible by flood control. This questionable justification improves the cost benefit ratio to be more favorable, but it encourages floodplain development which can devastate the riparian corridor (Kier 1981). As a result, the floodplain development encouraged by the land enhancement justification increases flood risk in the floodplain (Leopold 1997).

In addition, cost benefit analysis may not adequately consider uncertainties and relevant public policy considerations such as stakeholder opinions, non-market values, and equity. Although non-market benefits can be included in cost benefit analysis, because it is difficult to monetize, in reality the method is inflexible to consider social and environmental cost, so in practice it is usually left out from the cost benefit analysis (Powers 2003, NRC 2004, Goulder 2007). Therefore, in this dissertation I consider the cost benefit ratio from the cost benefit analysis only as a project evaluation metric in the USACE planning process. It is not a comprehensive representation of the true project value.

The USACE's historical approach on design uncertainties was based on a best estimate of the expected height of a design flood, with the addition of a standard increment of height called freeboard. Many USACE flood damage reduction projects used a standard of 3 feet of freeboard for levees and floodwalls (USACE 1970). Challenges to the standard freeboard concept emerged in the early 1990s. It was noted that standard freeboard did not provide consistent levels of flood protection across the nation. Procedures for calculating the economic benefits conferred by freeboard were also questioned. To address these issues, the USACE develops and applies risk analysis techniques to quantify and explicitly incorporate uncertainties in hydrologic, hydraulic, and geotechnical parameters into design. Although the techniques need further refinement, it was envisioned that risk analysis could replace the standard freeboard approach (NRC 2000).

The USACE civil works program has been criticized as an inflexible system favoring large scale structural projects (NRC 2004, Samet 2007). Efficient investment in USACE water infrastructure and operation and maintenance requires setting of priorities, but existing legislative processes for Corps funding and authorizations do not generally provide such guidance (NRC 2012). Each project is being considered by the congress individually. There is not an overarching program or master plan to classify and prioritize these projects.

Over 90% of the flood risk management structures in the USACE programs were built between 1936 and 1986, with more than half constructed prior to 1960 (USACE 2012). Many of these structures lack quality detailed maintenance records, despite the fact that many of the structures are either approaching or have already exceeded their design lives. As reported in the National Flood Risk Management Program, Program Management Plan (USACE 2012), managing these aging infrastructure are becoming a critical issue for flood risk management. One of the major challenges is a lack of baseline national inventory and assessment of the flood risk management projects (USACE 2012). Such baseline information is critical to systematically define priorities to invest and to maintain the existing infrastructure. Within the structural flood protection methods, the USACE dam and levee safety programs developed the National Inventory of Dams (USACE 2014a) and the National Levee Database (USACE 2014b). However, no such program or database existing for the flood control channel. This dissertation attempts to address this issue by providing reviews, case studies, and analysis to assemble an example inventory database and assessment of the 1950s to 1970s era USACE flood control channel in the San Francisco Bay Area, and articulate the issues presented by these aging flood control channels. In specific:

My dissertation examines the flood control channels built by the USACE between 1950s and 1970s in two scales. At the Federal scale, I review the USACE planning policies to trace back how these flood control channels were planned and designed, and how policies from FEMA and California State influenced project formulation. Then I systematically evaluate flood control channels in San Francisco Bay Area to identify planning, design, and operation and maintenance issues. My quantitative summaries show the commonality of problems in these flood control channels, and it forms the justification to develop a channel safety program to manage all USACE flood control channels at the federal scale. At the local scale, I conduct a detail review on the planning process of the Corte Madera Creek flood control project, to validate the findings from the USACE planning policies review. I also develop a hydraulic analysis to quantify the sensitivity of channel performance to hydraulic roughness, sediment transport, and channel

maintenance. My analysis identifies a need to update the USACE operation and maintenance manual and engineering manual to improve channel sediment management efficiency and uncertainty quantification. My dissertation concludes with my recommendations to manage, design, assess and reinvent aging flood control channels.

The following outlines the dissertation chapters and the study components.

In Chapter 2, I conduct a Federal policy review. I review the history and evolution of USACE policy, planning process, and design criteria since 1936, when the Flood Control Act authorized the USACE with the flood control mission. I also review the 100-year flood definition, and its significance to the flood protection work in United States. The review identifies planning issues with these flood control channels. In addition, I summarize the USACE dam safety program and the still developing levee safety program. Modeling after the dam and levee safety programs, I identify a number of program components that would be important to establish a channel safety program. In this Federal policy review, I address the question of how the flood control channels were planned and designed, and introduce policy examples for a channel safety program.

In Chapter 3, I conduct a systematic evaluation of the flood control projects in the San Francisco Bay Region. All projects in the evaluation are flood control channels built by USACE between 1950s and 1970s along densely developed urban areas, outflow to the bay. Due to urbanization in flat terrain with close proximity to the bay, these urban areas have high flood risks. In the systematic evaluation, I identify issues on the project justification, hydraulic performance, and operation and maintenance, specifically sediment management. I show that these issues are common among the flood control channels, and it justifies a need for a systematic management program at the Federal level.

In Chapter 4, I continue the flood control project review at the local scale. I evaluate the Corte Madera Creek Flood Control Project in detail. I review the project planning history to highlight the basic planning and design issues common to flood control channels. I also conduct a hydraulic analysis to test the sensitivities of the parameters influencing channel capacity, to assess various sediment management schemes, and to estimate the range and the probable concrete channel capacity. The study demonstrates how sediment changes effective flow area and hydraulic roughness, and its influence on flood control channel capacity and sediment management approaches. My analysis result identify needs to update the USACE operation and maintenance manual and engineering manual to improve channel sediment management efficiency and uncertainty quantification.

In Chapter 5, I summarize the reviews, evaluations, and analysis on the planning policy, design, and operation and maintenance of these flood control channels. I conclude this dissertation with recommendations to manage, design, assess and reinvent flood control channels.

The study components in each chapter are interconnected. The planning policy review in Chapter 2, plus the San Francisco Bay Region case study in Chapter 3, connects the planning policy, engineering design, and operation and maintenance issues, and show them as a set of problems common to flood control channels. The commonality and extent of these systematic problems form the justification to develop a channel safety program introduced in Chapter 2. The

management framework for the channel safety program is based on the dam and levee safety programs presented in Chapter 2. In addition, the San Francisco Bay Region case study in Chapter 3 identifies significant sediment management problem, and it directly impacts channel capacity. The finding motivates the Corte Madera Creek flood control project review and hydraulic analysis in Chapter 4. In Chapter 4, the project review and hydraulic analysis provide in depth explanation on the planning history and the technical details on how sediment management, sediment deposition, and sediment transport affect channel performance. The planning history review validates the issues identified in the policy review in Chapter 2. The hydraulic analysis indicates a need to update the operation and maintenance manual and the engineering manual. The channel safety program in Chapter 2 provides the management tool to systematically prioritize and implement these operation and maintenance manual updates to USACE projects. Hence the Corte Madera Creek flood control project review and hydraulic analysis in Chapter 4 provide both justification and sample application for the channel safety program.

CHAPTER 2 – A BRIEF OVERVIEW OF THE USACE FLOOD CONTROL PLANNING AND ENGINEERING POLICIES, DAM, LEVEE AND CHANNEL SAFETY PROGRAMS

2.1 Introduction

In Chapter 1 I laid out the theme of this dissertation on the urban flood control channels built by USACE before the 1970s, to evaluate the performance, identify issues, and explore management options for this aging infrastructures. This chapter focuses on the question of how these channels were planned and designed. In this chapter I summarize the history of the flood control policy and planning process, with emphasis on the cost benefit analysis and USACE local sponsor partnership. I discuss the engineering design criteria which form the basis of the flood control channel design. Since the current flood protection standard is based on the concept of the 100 year event, I also discuss the history, reasoning, and implications of the 100 year standard.

In addition, in this chapter I focus on the question of managing aging flood control infrastructure at the federal government. The USACE currently has safety programs for dams and levees. However, there is no parallel program for flood control channels. In this chapter I summarize the existing dam and levee safety programs, and identify components that could apply to a potential channel safety program. Such a program can provide a systematic means to manage the flood control channel inventory and prioritize resources for channel improvements, leading to improved flood protection reliability.

2.2 History of USACE and FEMA Flood Management Policy

This section briefly introduces the major flood management policies in the United States, with particular focus on USACE and FEMA policies since they are the leading flood management agencies in the United States. The key Congressional legislation governing the USACE is the Water Resource Development Act (WRDA), and its predecessor the Flood Control Act (FCA). The Congress passed 19 FCA between 1917 and 1972, and 12 WRDA between 1974 and 2014. These acts authorized flood control projects and studies, additional responsibilities to USACE, and categories of smaller projects for flood control and ecosystem restoration. The key policy for FEMA's involvement in flood management is the National Flood Insurance Program (NFIP) of 1968. The program provides Federal subsidized flood insurance for residents in flood prone areas (FEMA 2002). This flood management policy history discussion is organized in four periods: 1850s-1930s "Rising Federal Responsibility in Flood Control"; 1930s-1960s "The Golden Age of USACE Flood Control Projects"; 1960s-1980s "Flood Insurance Reshaping Floodplain use"; and 1980s-Today "A New Dawn in Ecosystem Restoration".

2.2.1 1850s to 1930s: Rising Federal Responsibility in Flood Control

The flood management policy as we know it today is highly influenced by the flood control efforts in the Mississippi River and Sacramento River. The 1850s was a milestone decade for both systems in flood management.

Flooding is a long standing problem on the Mississippi River. In 1543, a Spanish expedition team observed flooding along the Mississippi River. In those days, native Indians built their homes on high ground, or built berms around their settlements (Wright 2000). The informal berm systems evolved and expanded overtime to become a patch work levee systems. Federal involvement in flood control did not exist until the 1850s.

Major floods in 1849 and 1850 on the lower Mississippi River and delta moved the Congress to pass the Swamp Land Acts of 1849, 1850, and 1855. The Swamp Land Acts authorized the Federal government to transfer the ownership of swamps and wetlands to the local state governments. The local state government would then drain and develop the lands for agriculture. The development revenue funds local/state governments flood control projects (Wright 2000).

The subsequent Mississippi River Commission created in 1879 provided additional Federal support to fund and construct systematic flood control projects. The projects were almost exclusively levee constructions (Wright 2000).

In Sacramento River, gold discovery in 1850s changed the landscape in the Sacramento Delta. The population expansion, plus the Swamp Land Act encouraged human settlements along the floodplains. Devastating floods in 1850 and 1861, and the subsequent floods partly due to hydraulic mining debris prompted widespread levee construction in the Sacramento Delta (James and Singer 2008).

The FCA of 1917 was the first key congressional legislation governing and authorizing the USACE's flood control projects. The FCA of 1917 set a milestone in the nation's flood control policy. Before the FCA of 1917, USACE's water projects were focused on navigation improvements only. Even when a given project provided significant flood control benefits, the project would be packaged as a "navigation" project for congressional authorization and appropriation. The FCA of 1917 covers flood control projects in the Mississippi River and Sacramento River. It sets the precedent on Federal and USACE's involvement in flood control. The FCA of 1917 also introduced the concept of local sponsorship, in which the local agencies contribute (or "share") a portion of the project cost. The FCA of 1917 established the policy foundation for the subsequent FCA of 1936. Between FCAs of 1917 and 1936, the FCA of 1928 was in response to the Great Mississippi Flood of 1927 and authorized additional projects in the Mississippi and Sacramento river basins (Arnold 1988).

2.2.2 1930s to 1960s: The Golden Age of USACE Flood Control Projects

The FCA of 1936 was a landmark legislation that fundamentally reformed United States flood management policy. The FCA of 1936 was signed during the midst of Great Depression. The congress was desperately creating public work projects, but the local state governments could not afford such projects themselves. The FCA of 1936 allowed the congress to authorize over 250 projects (Arnold 1988). However, the significance of the FCA of 1936 was not merely on the volume of the authorized projects. The FCA of 1936 officially established the USACE's commitment to provide flood protection. When a local community experiences flooding

problems that are beyond their ability to solve, the local agencies can seek partnership with USACE to leverage their technical and financial assistance.

The FCA of 1936 also marks the beginning of applying the cost benefit analysis to evaluate project feasibility. As quoted from the Act (U.S. Congress 1936):

... the Federal Government should improve or participate in the improvement of navigable waters or their tributaries, including watersheds thereof, for flood-control purposes if the benefits to whomsoever they may accrue are in excess of the estimated costs, and if the lives and social security of people are otherwise adversely affected.

In a basic term, the cost benefit analysis is an economic based project evaluation method to weight the project's benefits, namely property damage reduction and economic output increase due to flood avoidance, against the project's life cycle life (Boardman et al 2001). The result of this analysis is a cost benefit ratio, defined as a ratio of annualized benefits over annualized cost (U.S. Bureau of the Budget 1952). Based on the cost benefit analysis, USACE would not approve a project with the cost benefit ratio less than unity (1.0/1).

However, there were no interdepartmental implementation guideline until 1950 (Hufschmidt 2011), when the Proposed Practices for Economic Analysis of River Basin Projects (FIARBC 1950) was published. This document and the 1958 revision (IACWR 1958) bearing the same title, are commonly known as the Green Book. It defines that the cost benefit analysis should be to maximize general economic welfare and economic efficiency. The evaluation should include market and non-market benefits and costs. While the Green Book is a significant technical reference for economic evaluation throughout the 1950s and 1960s, it was never formally accepted for implementation (Hufschmidt 2011). It is because the Bureau of Reclamation did not agree with the Green Book's approach on the secondary benefits. The Green Book approach considers secondary benefits only from the national economic efficiency standpoint, but the Bureau of Reclamation argued that the secondary benefits at the local and regional levels should be considered as the benefits in the economic analysis. The issue was never resolved, and in 1952, the Green Book was incorporated into the U.S. Bureau of the Budget, Budget Circular A-47 (U.S. Bureau of the Budget 1952). The circular coverage was much the same as the Green Book. Basically, it was a conservative document, which placed primary emphasis on economic-efficiency-oriented primary benefits as project justification. The use of secondary benefits was severely restricted. An opportunity-cost concept of interest or discount rate, tied to the interest rate of long-term government bonds, was adopted. A 50-year economic time horizon was established. It became the officially approved standards and procedures used by the executive office of the president in reviewing agency project proposals (Caulfield 2011).

Since the FCA of 1936, subsequent FCAs authorized numerous dam, levee, and channel projects for flood control. To streamline and expedite the project authorization process, Section 205 of the 1948 Flood Control Act, as amended, provides authority to USACE to plan and construct small flood damage reduction projects that have not already been specifically authorized by Congress. It is a part of the Continuing Authorities Program (CAP). The Federal cost limitation per project was \$100,000 (U.S. Congress 1948). The limit was raised by a number of amendments over time, up to the current per project limit of \$10,000,000 (U.S. Congress 2014).

USACE continued to construct major flood control projects through the 1960s. By 1970, the national priority was shifted away from flood control projects. There was no major flood control project authorized between 1970 and 1985 (Carter and Stern 2010).

2.2.3 1960s to 1980s: Flood Insurance Reshaping Floodplain Use

Since the FCA of 1936, the next revolutionary flood management legislation is the National Flood Insurance Act (NFIA) of 1968. The act created the Federal Insurance Administration (FIA) within the Department of Housing and Urban Development (HUD) to oversee the NFIP. The purpose of the program was to reduce flood damages by managing floodplain land use and to provide flood insurance (FEMA 2002). Although the flood insurance was federally subsidized, it did not generate sufficient incentive for voluntarily participation to the NFIP. As a result, the Flood Disaster Protection Act of 1973 was passed (Anderson 1974). The 1973 act prohibits Federal agencies providing financial assistance to any community that did not participate in the NFIP. In addition, the act's Mandatory Flood Insurance Purchase Requirement requires flood insurance for properties within 100-year floodplains in order to be eligible for federally insured or regulated grants and loans. As a result, there was a significant increase in flood insurance participation. The flood insurance program is currently managed by the Federal Emergency Management Agency (FEMA).

Another significance of the NFIP is that it established the 100-year flood as the national flood management standard. The program developed Flood Insurance Rate Maps (FIRMs) depicting the 100-year floodplain boundary and its base flood elevation across the nation (FEMA 2002). Through the flood insurance, the program legitimized floodplain developments, especially for the area outside the Special Flood Hazard Area (SFHA) defined in the FIRM. Within the SFHA, the Code of Federal Regulations (CFR) Title 44 Part 60 established the criteria for land management and use in the floodplain (Federal Register 1986). The spirit of the regulation is to ensure that any development in the floodplain should be protected from a 100-year flood, also the development should not adversely impact the 100-year floodplain extent, flow conveyance, and stage. Then based on President Carter's Executive Orders 11988 in 1977 and 11990 in 1978, the CFR Title 44 Part 9 established the criteria for floodplain management and wetland protection (FEMA 1987). The regulation tightened Federal agencies coordination on wetlands and other floodplains management. Local agencies must avoid long- and short-term adverse impacts on floodplains unless no practicable alternatives exist. If there is no practicable alternative, the Federal agencies must mitigate to ensure the action minimizes any loss of life, property and natural beneficial values (USACE 1984).

Aside from the NFIP, there were also two landmark environmental legislations during this era. The National Environment Policy Act (NEPA) of 1969 established the Environmental Impact Statement (EIS) process. The Act requires environmental evaluation on projects and assessment on the project impacts to the environment (Fabrick and Joseph 1982). The Clean Water Act (CWA) of 1972 established basic policy structure to regulate pollutant discharge into the waters of the United States (Alder et al 1993). Within the CWA, the EPA's National Pollutant Discharge Elimination System (NPDES) provides the permitting tool to enforce pollutant

discharge. The original NPDES focuses on wastewater discharge. Since 1990 NPDES expanded to permitting stormwater discharge, leading to State's stormwater discharge permitting programs for the Municipal Separate Storm Sewer System (MS4). The NPDES stormwater permits influence land development projects to incorporate stormwater Best Management Practice (BMP) and Low Impact Development (LID) into urban stormwater management (CASQA 2003).

In addition to the flood insurance and environmental regulations, the 1960s to 1980s period also has significant advancement in cost benefit analysis and planning guideline for Federal agencies. The Budget Circular A-47 was the officially approved standards and procedures for project proposal reviews in the 1950s (U.S. Bureau of the Budget 1952). However, there were growing criticisms on the circular especially in the early 1960s. In addition to the issues of regional/local secondary benefits, discount rate and economic life definitions, another criticisms for the circular is its implicit emphasis on market benefit. Although non-market benefit can be included in the cost benefit analysis, since it is difficult to monetize, in reality it is usually left out from the cost benefit analysis (Powers 2003). In the early 1960s, the Kennedy administration revised the cost benefit analysis standard. The revision is documented in the Senate Document 97 (WRC 1962). Although the National Economic Development is still the primary emphasis, for the first time the cost benefit analysis takes a multiple-objective approach to consider Preservation and Well-Being of People objectives as well (Caulfield 2011).

To elevate the status of water quality in project evaluation, in 1973 the Water Resources Council issued the Principles and Standards for Planning Water and Related Land Resources (WRC 1973). The standard specified National Economic Development and Environmental Quality as the co-equal objectives for formulating project plans. The Principles and Standards also added the regional development and social well-being objectives. While they could be displayed for consideration by decision makers, projects would not be formulated for these objectives (Hufschmidt 2011). This trend of multiple objectives project evaluation with environmental quality emphasis further enhanced in the new Principle and Standards in 1980. It called for (1) full integration of water conservation into project and program planning; (2) preparation of a primarily nonstructural water resources plan as an alternative to a structural project or program; and (3) uniform and consistent calculation of national economic development benefits and costs. The goal is to reduce the number of economically marginal and environmentally destructive water resources projects under taken by the Federal government (Hufschmidt 2011).

However, there is a change in policy direction with the Reagan administration, beginning in 1981 (Caulfield 2011). In 1983, after extensive review and revision to the 1980 Principles and Standards, the Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies (WRC 1983) was adapted as the new planning guideline (Hufschmidt 2011). The four evaluation accounts from the 1973 standard remained and were renamed as National Economic Development (NED), Regional Economic Development (RED), Environmental Quality (EQ), and Other Societal Effects (OSE). However, the guideline also stated that:

... the federal objective of water and related land resources project planning is to contribute to the national economic development...

Therefore, NED became the only mandatory objective, and the USACE should select the NED-maximizing alternative. The guideline lacks the flexibility to adapt to ecosystem restoration project planning, where EQ and OSE are likely the dominant project drivers over NED. It is the guideline still in use today, governing the cost benefit analysis and the USACE planning process (IWR 1996).

2.2.4 1980s to Today: A New Dawn in Ecosystem Restoration

The WRDA is the successor of the FCA. There are two relatively insignificant WRDAs in 1974 and 1976. Then the WRDA of 1986 transformed the ground rule for cost sharing at USACE. The act established a new cost sharing formula from local sponsor, as shown in Table 2.2.4.1 (U.S. Congress 1986). As I discussed in Chapter 1, the federal financial assistance is one of the main drivers for local agencies to partner with USACE on flood control projects. Since the Flood Control Act 1938, the federal government covered most of the flood control project cost. However, in WRDA of 1986, the cost sharing increased to up to 50% for feasibility study and 35% for project construction (Carter 2005).

	Planning Phase		Pre-construction Engineering Design Phase	Construction Phase	Operation and Maintenance Phase
	Reconnaissance	Feasibility			
Duration	6 - 12 months	2 - 3 years	2 years	Varies	As long as project remains authorized
Federal Funding	100%	50%	75%	65%	0%
Local Funding	0%	50%	25%	35%	100%

The Section 1135 of WRDA of 1986 authorized USACE to execute small scale repair and retrofit projects that are less than \$5 million to fix problems in existing USACE projects (U.S. Congress). Many of the Section 1135 projects were executed to restore ecosystem functions in the constructed flood control project. WRDA of 1986 Section 1135 provides an authorization vehicle for ecosystem restoration projects.

Then the WRDA of 1990 marked a new paradigm for USACE. The act explicitly expanded the USACE mission to include aquatic ecosystem restoration and environmental protection. This ecosystem mission plus the navigation and flood control missions formed the three core justification of USACE civilian water projects (Carter and Coby 2005). The WRDA of 1992 reframed the sediment management concept to beneficial use. The act authorized the USACE to use the “spoils” from dredging in implementing projects to protect, restore, and create aquatic and ecologically related habitats, including wetlands. The WRDA of 1996 Section 206 gave the USACE the authority to undertake aquatic ecosystem restoration projects. The WRDA of 2000 approved a restoration program for the Florida Everglades. It was the first multiyear, multibillion-dollar restoration program for USACE. The WRDA of 2007 authorized the USACE

to pursue more ecosystem restoration projects, including projects at coastal Louisiana and Upper Mississippi River (Carter and Coby 2005).

2.2.5 The Evolution of Flood Management Policy

The brief overview of the flood management policy history shows the USACE focus on the economically efficient engineering solution for flood control, and the importance of the 100-year flood standard stemmed from the FEMA NFIP.

The FCA of 1936 authorized USACE with its flood control mission. The 1950s era “Green Book” cost benefit analysis approach focused on the National Economic Development benefits as the principal project justification. These policies resulted in a surge of USACE led flood control structure constructions, mostly utilitarian designs which flood damage avoidance and floodplain urbanization trumps any other factors. There were progressive movements in the 1960s to liberate the standard to include secondary benefits in the regional and local level, and in the 1970s another step to increase emphasis on environmental quality and other local and regional social and recreational benefits. However, the 1983 Principles and Guidelines established National Economic Development as the de-facto primary cost benefit evaluation objective. Although the Congress authorized the ecosystem restoration mission to USACE in 1990, there are still fundamental questions on whether the existing organization and planning procedures can effectively support the mission (Tarlock 2004).

In addition, the introduction of the National Flood Insurance Program significantly elevated the importance of 100-year flood protection. The standard is deeply rooted in almost any flood control work and flood risk reduction planning. Therefore, in this dissertation, I used the 100-year flood standard extensively to measure and compare project performance.

2.3 USACE Civil Work Planning Process

Section 2.2 discussed how the history of USACE policy and cost benefit analysis influenced the 1950s to 1970s era flood control projects. In this section, I provide a brief overview of the USACE planning process. Each USACE project follows the following process to form a project partnership with the local sponsor (Carter and Stern 2010; Powers 2003). While the overall process is consistent since the 1970s, there were revisions to clarify the process, to incorporate environmental review and permitting requirements, and to define the project objectives in terms of the Federal interest.

2.3.1 Current Planning Process Based on 1983 Principle and Guideline

This section summarizes the current USACE civil work planning process. The objectives and procedures are based on the Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies (WRC 1983). Table 2.2.4.1 listed the main components of the current planning process.

Since the USACE is prohibited by Federal regulations from suggesting a project, responsibilities for project proposals fall to the local communities. When a local community cannot alleviate flooding problems by its own means, the community may engage the USACE to discuss Federal assistance options. Typically if the project is in the Federal interest, smaller projects, as defined by Section 205 of the FCA 1948 as amended, can be accomplished without Congressional authorization via the Continuing Authorities Program (CAP). Larger projects require project specific authorization from the Congress (Carter and Stern 2010).

The first step of the planning process is to obtain authorization either from WRDA or Congressional committee resolution for the USACE to conduct the project studies. Once the study authorization is established, study funding can be sought through the administration and Congress (Carter and Coby 2005). Typical USACE project undergo two phases of study, the reconnaissance study and then the feasibility study.

The reconnaissance study is used to make a preliminary determination of the federal interest to further study the problem and establishes the scope of the feasibility study. The planning activities undertaken during the reconnaissance phase also lay the groundwork for the feasibility study phase. There are six essential steps for the reconnaissance study (USACE 2000).

- Step 1 – Determine if the problem warrants federal participation in feasibility studies.
- Step 2 – Define the federal interest based on a preliminary appraisal consistent with USACE policies, costs, benefits, and environmental impacts of identified potential project alternatives.
- Step 3 – Complete a Reconnaissance Report as required by Section 905(b) of the WRDA 1986.
- Step 4 – Prepare a Review Plan and Project Management Plan (PMP). The PMP documents the federal and non-federal efforts required to conduct the feasibility study.
- Step 5 – Assess the level of interest and support of non-federal entities in the identified potential solutions and cost-sharing of feasibility phase and construction. A letter of intent from the local sponsor stating the willingness to pursue the cost shared feasibility study described in the PMP and to share in the costs of construction is required.
- Step 6 – Negotiate and execute a Feasibility Cost Sharing Agreement (FCSA) with a non-federal sponsor. Execution of the FCSA marks the beginning of the feasibility phase of study.

Presuming the reconnaissance study finds that project meets the Federal interests and the benefits will outweigh project costs, the study will advance to the feasibility study. The feasibility study produces the decision document that becomes the basis for seeking Federal project authorization. The Feasibility Report develops prospective project alternatives, and carries out a detailed analysis of all the relevant physical, biological and social economic impacts attributable to these project alternatives. During this phase, any project associated environmental impacts must be assessed and, depending on their significance, preparation of an Environmental Assessment or an Environmental Impact Statement may be required.

The feasibility study follows the following six step planning process, as outlined in the Principle and Guideline (WRC 1983).

Step 1 - Identifying problems and opportunities

The desire to alleviate problems and realize opportunities should be specified for the planning area in terms of the Federal objective and specific State and local concerns. The problems and opportunities should also be stated for both current and future conditions, with the desired future conditions explicitly stated. It should be defined so that their definition does not dictate a narrow range of alternatives. The problems and opportunities should reflect the specific effects that are desired by groups and individuals as well as the problems and opportunities declared to be in the national interest by the Congress or the Executive Branch. This identification and detailing of problems and opportunities is the process of making explicit the range of preferences and desires of those affected by resource development. It should be understood that the initial expressions of problems and opportunities may be modified during the planning process.

Step 2 - Inventorying and forecasting conditions

The potential for alleviating problems and realizing opportunities is determined during inventorying and forecasting. The inventory and forecast of resource conditions should be related to the problems and opportunities previously identified.

Step 3 - Formulating alternative plans

Alternative plans are to be formulated in a systematic manner to insure that all reasonable alternatives are evaluated. Usually, a number of alternative plans are identified early in the planning process and become more refined through additional development and through subsequent iterations. Additional alternative plans may be introduced at any time. Alternative plans may include National Economic Development, National Ecosystem Restoration, Non-Structural Plan, Locally Preferred Plan, and No Action. (Brian 2010).

Step 4 - Evaluating alternative plans

The evaluation of the effects of each alternative plan consists of assessment and appraisal. Assessment is the process of measuring or estimating the effects of an alternative plan. Assessment determines the difference between without-plan and with-plan conditions for each of the categories of effects. Appraisal is the process of assigning social values to the technical information gathered as part of the assessment process. Since the technical data concerning benefits and costs in the NED account are also expressed in monetary units for cost benefits analysis, appraisal is applicable only to the EQ, RED, and OSE evaluations.

Step 5 - Comparing alternative plans

The comparison of plans focuses on the differences among the alternative plans as determined in the evaluation phase. The differences should be organized on the basis of the effects in the four accounts or on a combination of the NED account and another appropriate format for other significant effects.

Step 6 - Selecting a plan

After consideration of the various alternative plans, their effects, and public comments, a plan is selected following the general guidance as follows.

- The National Economic Development plan with the greatest net economic benefit consistent with protecting the Nation's environment is to be selected unless the Secretary of a department or head of an independent agency grants an exception when there is some overriding reason for selecting another plan, based upon other Federal, State, local, and international concerns.
- The alternative of taking no action, i.e., selecting none of the alternative plans, should be fully considered.
- Plans should not be recommended for Federal development if there are non-Federal plans that would more effectively contribute to the Federal objective when comparably evaluated.

The draft Feasibility Report and Environmental Impact Statement (if needed) are submitted for agencies and public review. After the review, the final Feasibility Report and Environmental Impact Statement are submitted to the USACE Headquarters. The final Feasibility Report also include the Local Cooperation Agreement (LCA), which details the USACE and the local sponsor's responsibilities and cost sharing agreement (USACE 1989). Assuming the Feasibility Study concluded that the project merit Federal interest, the Feasibility Report is then summarized and repackaged as the proposed Report of the Chief of Engineers, or Chief's Report. The Chief's Report, along with any final Environmental Impact Statement, are distributed to relevant Federal agencies and to the Governors of all states affected by the proposed project, for their comments.

After all comments have been addressed, the Chief's Report is transmitted to Congress for comments as it relates to the President's programs. To be authorized for construction, Congress must include the projects in the next WRDA round of project proposals (Cater and Cody 2005).

Once the project is authorized, the project will proceed to the pre-construction engineering design and construction. The local district of the USACE provides the technical assistant, while the cost is shared between the USACE and the local sponsor. A key element of this final phase is to secure the Federal funding. The FCA and WRDA only authorize the projects. In order to receive funding appropriation for the project, USACE must purpose the projects for inclusion in the annual Energy and Water Development Appropriations Act, to be considered by the US House of Representatives Committee on Appropriations (Cater and Cody 2005). Fiscal priorities and public attitudes in recent decades have resulted in declining federal funding for water resources activities, thus increasing competition for funding among authorized activities. Moreover during the 1990s and in 2000, Congress authorized not only navigation and flood control projects, but also ecosystem restoration, environmental infrastructure assistance, and other non-traditional activities. The USACE now has an estimated "backlog" of more than 1,000 authorized projects (Carter and Stern 2010). Therefore, due to the backlog on project appropriations, not all authorized projects receive funding for construction. Funding appropriation is a time sensitive matter. If a project does not receive funding in five years, the project can be subject to be deauthorized.

If the project receives funding appropriation, USACE would complete the project design and construction. After the construction is completed, USACE and the local sponsor will inspect the project to confirm the construction meets the design specifications. USACE will also prepare an Operation and Maintenance Manual for the project. Then, the USACE will transfer the project ownership to the local sponsor per the LCA. After the ownership transfer, the local sponsor is responsible to operate and maintain the project (USACE 1989).

2.3.2 USACE Planning Process, Pre-1983 Principle and Guideline

This section steps back and provides a brief summary of the USACE planning procedures evolved between the FCA of 1936 and the Principle and Guideline of 1983. Key similarities and differences from the current planning process are highlighted in this overview.

Since the 1936 FCA, the USACE Civil Work planning follows the two steps study process (IWR 1996). The first step is a preliminary examination. Similar to the reconnaissance study, the preliminary examination is to determine if the proposed project is favorable for Federal participation. If it does, then the USACE would conduct a detail survey to formulate the project improvements, and evaluate if the improvements are economically justified. The survey report, similar to the feasibility report, documents the findings and recommend the project for Congressional authorization.

In the early 1950s, the Report on the Federal Civil Works Program as Administered by the Corps of Engineers U.S. Army: Appendix D Policies and Procedures for Investigating and Planning Civil Works (USACE 1951) outlined a 4-steps planning process:

- Step 1 – Establish the need for the project.
- Step 2 – Select the proper design scope, type, and details.
- Step 3 – Demonstrate the design’s economic value.
- Step 4 – Provide cost sharing allocation between various interests.

Compared to the current 6 step process, I found that this planning process misses the alternative analysis elements. As I discussed in previous sections, the current planning process requires evaluation in terms of four accounts and multiple alternatives. Based on the single National Economic Efficiency objective as stated in the Budget Circular A-47 (1952) and Green Book (1958), the project formulation process starts with developing a baseline improvement design, then considers alternatives with varying improvement scales from the baseline improvement design. The optimum scale is that which maximizes net benefits. The consideration of alternative plans concentrates on assuring that there is no cheaper means of accomplishing the same purpose. It is recognized that “in theory, the broadest range of alternatives...should be considered,” (IWR 1996), but the emphasis back in 1950s was clearly on a severely limited range of objectives.

In the late 1950s, the Laws and Procedures Governing Conduct of the Civil Works Program (USACE 1959b) outlined a revised 4 steps planning process.

Step 1 – Determination of the nature and scope of the problems for which solution is sought.

Step 2 – Identification of all alternative measures and combinations of measures which reasonably might be applied in the solution of these problems.

Step 3 – Determination of the benefits and costs or, more broadly, the determinate effects, beneficial or adverse, tangible or intangible, of the alternative projects and programs which have been identified.

Step 4 – Selection of the best solution from the array of alternative solutions which have been considered.

I found this planning process was a step forward, in that the revised planning process included alternative analysis elements. The evaluation was focused on weighing and comparing alternatives to determine their relative efficiency in achieving the desired improvement objectives.

In the early 1970s, the Principles and Standards for Planning Water and Related Land Resources (WRC 1973) outlined a 6 steps planning process similar to the current planning process.

Step 1 – Specify components of the objectives relevant to the planning setting.

Step 2 – Evaluate resource capabilities and expected conditions without any plan.

Step 3 – Formulate alternative plans to achieve varying levels of contributions to the specified components of the objectives.

Step 4 – Analyze the differences among alternative plans which reflect different emphasis among the specified components of the objectives.

Step 5 – Review and reconsider, if necessary, the specified components for the planning setting and formulate additional alternative plans as appropriate.

Step 6 – Select a recommended plan from among the alternative plans based upon an evaluation of the trade-offs between the objectives of national economic development and environmental quality and considering, where appropriate, the effects of the plans on regional development and social well-being.

In addition, the 1973 revision placed the environmental concerns on an equal basis with national economic development:

- National Economic Development: To enhance national economic development by increasing the value of the nation's output of goods and services and improving national economic efficiency.
- Environmental Quality: To enhance the quality of the environment by the management, conservation, preservation, creation, restoration, or improvement of the quality of certain natural and cultural resources and ecological systems.

Environmental quality and national economic development remained as a co-equal goals until the Principles and Guidelines adopted by the Reagan administration in 1983. Since then, the

USACE civil work planning only require that the project to meet the National Economic Development plan as the Federal interest (WRC 1983).

This section provided a brief overview of the USACE planning between the FCA of 1936 and Principles and Guidelines of 1983. I found the planning process evolution echoes the policy history overview in Section 2.2, highlighting the trend that the early planning process singly focused on the efficiency to meet the project objective with minimum cost. Before the environment regulation era starting late 1960s, there were few, if any, requirements to mandate environmental cost and benefits be included in project planning and evaluation.

2.3.3 Local Cooperation Agreement

As introduced in Section 2.3.1, the Local Cooperation Agreement is an agreement jointly developed between the USACE and the local sponsor. The agreement stipulated each party's project responsibilities and the project cost sharing arrangements.

Project Responsibilities

Examples of items in a local cooperation agreement that local sponsors must furnish include the following (USACE 1989):

1. The Local Sponsor shall hold and save the Government free from all damages arising from the construction, operation, and maintenance of the Project, except for damages due to the fault or negligence of the Government or its contractors.
2. The Government, subject to and using funds provided by the Local Sponsor and appropriated by the Congress of the United States, shall expeditiously construct the Project (including relocations of railroad bridges and approaches thereto), applying those procedures usually followed or applied in Federal projects, pursuant to Federal laws, regulations, and policies. The Government will consider the comments of the Local Sponsor, but award of contracts, modifications or change orders, and performance of all work on the Project shall be exclusively within the control of the Government.
3. The Local Sponsor shall provide all lands, easements, rights-of-way, and dredged material disposal areas, and perform all relocations (excluding railroad bridges and approaches thereto) determined by the Government to be necessary for construction of the Project. At its sole discretion, the Government may perform relocations in cases where it appears that the Local Sponsor's contributions will exceed the maximum non-Federal cost share.
4. When the Government determines that the Project or a functional portion of the Project is complete, the Government shall turn the completed Project or functional portion over to the Local Sponsor, which shall accept the Project or functional portion and be solely responsible for operating, repairing, maintaining, replacing, and rehabilitating the Project or functional portion.

5. The Local Sponsor agrees to participate in and comply with applicable Federal flood plain management and flood insurance programs.
6. No less than once each year the Local Sponsor shall inform affected interests of the limitations of the protection afforded by the Project.
7. The Local Sponsor shall publicize flood plain information in the area concerned and shall provide this information to zoning and other regulatory agencies for their guidance and leadership in preventing unwise future development in the flood plain and in adopting such regulations as may be necessary to prevent unwise future development and to ensure compatibility with protection levels provided by the Project.

I found that items 1 to 4 were routinely applied to most flood control projects since the 1940s. In essence, the USACE has full design and construction control, but is indemnified from claims. The local sponsor provides all lands, easements, rights-of-way, and relocations, plus takes on the lifetime operation and maintenance responsibility. I found such agreement poses significant disadvantage to the local sponsor. If a project was poorly designed, the local sponsor has little power to change the project, but must need to live with the project and be responsible for claims, and operation and maintenance. Therefore, local sponsor should carefully evaluate the project risk before entering the local cooperation agreement.

Project Cost Sharing

The financial assistance is attractive for local agencies to be a local sponsor partnering with USACE on flood control projects. Since the FCA 1936, the federal government covered a majority of the flood control project cost. Local sponsor mainly responsible for two items:

- Local sponsor responsible for lands, easements, rights-of-way, and relocations for construction. However, in California the cost could be reimbursed by California State as stipulated in the “The Flood Control Law of 1946” to the California Water Code (California State Legislature 1943).
- Since the Federal funding only covers the capital improvement cost. After the construction is completed, the project ownership and the responsibility to operate and maintain transfers back to the local sponsor (USACE 1989).

There was no unified formula on local sponsor cash contribution. On average, local sponsors paid about 17% of the total project cost (Shabman 1997, Water Resources Council 1975). Federal funding provided a significant financial incentive for the local sponsor to partner with the USACE on flood control projects

The WRDA of 1986 (U.S. Congress 1986) set a new cost sharing formula for studies and projects between the USACE and the local sponsor. The following list summarizes the requirements of the cost share (U.S. Congress 1986, Carter 2005):

- Reconnaissance study: USACE pay 100 percent of the study cost.

- Feasibility study: Local sponsor pay 50 percent of the study costs. Up to one-half of the local sponsor's share may consist of in-kind services, in which they actually undertake a portion of the study. Such services include surveying, soil investigations, schematic design, or public involvement. The remainder of the local share must be a cash contribution.
- Design and Construction: Local sponsor pay a minimum of 25 percent, but not more than 50 percent, of the total cost for flood control. The cost-sharing requirement for nonstructural projects is a flat 25 percent.
- Design and Construction: Local sponsor provide all lands, easements, rights-of-way, and relocations. The costs to obtain these items can be credited toward the local sponsor's share.
- Operation, Maintenance, Repair, Replacement, and Rehabilitation (OMRRR): Local sponsor pay 100 percent of the operation and maintenance cost.

As of the WRDA of 1986, the local sponsor cost sharing for flood control projects increased to up to 50%. As a result, the advantage of federal funding for local communities was significantly reduced.

2.4 100-Year Flood Standard

In Section 2.2.5, I discussed the National Flood Insurance Program (NFIP) set the 100-year flood as the de facto national flood protection standard. Although it was originally defined for setting the flood insurance coverage and rate, the 100-year flood is broadly applied to flood protection planning and improvement design. In this section I discuss the theoretical basis to set 100-year flood as the flood protection standard, and then contrast with the history of how the 100-year flood standard was established for the NFIP. The section concludes with a discussion of the significance of 100-year flood standard on flood management.

2.4.1 Theoretical Basis to Define 100-Year Flood

The flood probability describes how often creek water surface elevation (stage) exceeds the adjacent floodplain elevation. Stream stage is directly correlated with creek flow. Hence, the flood probability is usually defined in reference to flow and stage. The flood probability expresses either in percentage (e.g. 1% chance of flooding in a given year), or more commonly in return frequency (e.g. 100 year flood). The return frequency represents the average flood return period in a long term, and it is simply the reciprocal of flood probability in percentage (Dunne and Leopold 1978). The flood return period is a common standard to communicate the flood protection level of service.

In flood management and design theory, defining flood probability to evaluate flood protection solutions are commonly based on risk-based design method and reliability-based design method (Woodall & Lund 2009):

Risk method focuses on minimizing the future costs of flooding by taking preventative measures today. In a simplified term, the evaluation is based on cost benefit analysis. The analysis considers flood damage cost, improvement cost, operation and maintenance cost, and environmental and social cost. The analysis estimates cost benefit ratio for different improvement alternatives. Each alternative is designed for a certain level of flood probability. For example, Alternative A may design for 50-year flood, and Alternative B for 100-year flood. The goal is to select the alternative with a maximum cost benefit ratio. The selected alternative defines the appropriate level of protection. A benefit of the risk-based approach is that it allows choices based on comparison of expected outcomes and costs of solution to select the flood probability, instead of design based on a fixed flood probability.

Reliability method is based on a pre-established “acceptable” failure probability target. Often times the failure probability target is based on the risk method analysis. Legislation, insurance policies, or other parties may determine an acceptable failure probability based on different preferences regarding loss of life, infrastructure investment, or economic loss. Reliability method allows engineers and planners to develop a set of alternatives that provide the target level of protection and then choose the lowest-cost alternative.

In both risk and reliability methods, the analysis translate flood probability to expected flow rate, to assess the available capacity of the storm drainage and creek systems, to map the potential floodplain, and to estimate the flood damage cost. The flow rate and flood damage cost defines the flood damage / return frequency rating curve for cost benefit analysis. The outcome of the analysis specific the most cost effective improvement alternative.

Often times flood protection systems can incorporate both risk and reliability methods. For example, risk method can evaluate a sub-watershed within the study area. It defines the failure probability target, and then applies to the entire study area based on the reliability method. However, in this approach the reliability method can only apply to sub-watersheds where the hydrology, landuse, improvement cost and flood damages are similar to the sub-watershed analyzed by the risk method.

Reliability method requires less data and analysis than risk method. Therefore I found that reliability method based on a set level such as the 100-year flood is commonly used in municipal and regional scale flood protection projects, as well as in FEMA NFIP. However, in theory risk method is still needed to define this 100-year flood standard.

2.4.2 History of 100-Year Flood Standard

The concept of the probability based flood protection standard was first described by Weston D. Fuller in the 1914 Transactions of the American Society of Civil Engineers (Reuss 2002). Before 1950s, agencies apply various standards for their flood protection design. For examples, Tennessee Valley Authority (TVA) used the “maximum probable flood” standard; the USACE used the “standard project flood” standard (Robinson 2004). In 1971, FEMA NFIP defined the Base Flood Elevation based on 100-year flood (FEMA 2002). Since then the 100-year flood

became a standard for regional flood management. The USACE flood control design also moved from Standard Project Flood design standard to 100-year flood standard. I found that local counties and municipalities in San Francisco Bay Area commonly set their flood protection service goal at the 100-year design event.

Although there is a theoretical basis to define a flood standard, in reality, the 100-year flood protection standard in FEAM NFIP is adopted based on consensus and decision maker's level of comfort, rather than explicit theoretical evaluation (ASFPM Foundation, 2004).

The Chicago Seminar on December 16th to 18th in 1968 established the 100-year flood as the national flood protection design standard. The seminar was organized by the U.S. Department of Housing and Urban Development (HUD) which was directed by the National Flood Insurance Act of 1968 to establish flood management criteria for the NFIP. From the recollection of the seminar participants, the recommended standard is based on compromise to the participants' comfort level. Quoted from Sheaffer (2004):

“The 100-year flood emerged as the flood to be used in the flood insurance program. There was no data on 100-year floods, but it was stated that it had “a nice sound” to it and would give an illusion of safety.”

There was no economic analysis prepared to justify the 100-year flood selection due to the constraints of time. Nevertheless, the standard was adopted by the NFIP in its final rule on September 10, 1971.

The validity of the 100-year flood standard was reviewed over the years. Between 1981 and 1983, FEMA was directed by the Office of Management and Budget to review the use of the 100-year flood standard. Although the standard was not evaluated based on the cost versus its benefit, the study findings still support the 100-year flood standard. The 100-year flood standard continues to be the de Facto standard for flood management (FEMA 1983).

2.4.3 – Significance of the 100-Year Flood Protection Standard

As previously mentioned, the 100-year flood protection standard is important for the flood management in the United States (Carter 2005). FEMA NFIP used the Base Flood Elevation from the 100-year flood to delineate the floodplain (FEMA 2002). Developments within the FEMA NFIP floodplain are required to purchase flood insurance. Since most flood insurance policy holders are located in the floodplain, the risk is too high for private insurance companies to issue flood insurance. Therefore, the flood insurance is sponsored by the federal government. However, even with the federal sponsorship, the flood insurance requirement still presents a financial incentive to local agencies, in addition to public safety goal, to provide flood protection and remove urban settlement areas from the NFIP designated floodplains.

The 100-year flood protection standard from FEMA NFIP has profound influence on municipal flood management. For example, in San Francisco Bay Area, all nine counties use 100-year flood as their flood management goal. It dictates the minimum building pad and finish floor

elevation, grading for surface pavements, and the overland flow capacity of public streets and gutters. In new urban development projects, guarantee to provide 100-year flood protection is routinely a part of their condition of approval for development and building permits.

In the regional and national scale projects, many of the new USACE projects not only are designed based on 100-year flood standard, but also set the post-project operation and maintenance (O&M) requirements based on 100-year flood protection goal. When a project failed to keep up with the O&M requirements, flooding within the project tributary will not be eligible for the USACE PL 84-99 disaster assistance program (Carter and Stern 2010). PL 84-99, per Section 5 of the FCA of 1941, authorized the USACE to emergency preparedness and disaster support for flood events, and provide financial assistance to repair the projects that were damaged by flood (U.S. Congress 1941).

Since the 100-year flood standard uniformly applied to the entire nation, while it may be sufficient or even conservative for rural low density communities, in large urban areas and river basins with high flood risk, the 100-year flood standard may not provide sufficient flood risk reduction (NRC 2000).

Perhaps the most significant issue of the 100-year flood standard is to the public's perception of flood safety. There is a misconception in public that the 100-year flood means the area will be safe in 99 years, and flood only once in 100 years. In reality, within a typical flood control project economic life of 50 years, there is close to 40% chance that a 100-year flood or higher would occur. Even assuming the flood control project will perform as designed to protect the area from the 100-year flood, and also assuming the 100-year flow is the same as the design estimate, the area will still subject to flooding if there is a "101-year" flow (Ludy and Kondolf 2011).

In real estate transactions, it is required to disclose if a property is within FEMA NFIP 100-year floodplain, as a way to communicate the risk of flooding and the associated flood insurance requirements (Troy and Romm 2004). In general, public defines 100-year floodplain as the clear boundary of flood risk. Outside of the 100-year floodplain means the area is free from flooding. This boundary encouraged urban development up to the edge of the 100-year floodplain. However, it does not stop floodplain development. Although a case study in California found that average floodplain home sold for 4.2% less than a comparable non-floodplain home (Troy and Romm 2004), economic and land pressures continue to drive developments in high flood risk areas.

2.5 USACE Flood Control Channel Design

In general flood control channel design is a two steps process: A hydrologic analysis to estimate the design flow, and then a hydraulic analysis to estimate the design water surface elevation based on different design alternatives. The analysis evolves over time. When most of the USACE flood control channel were designed before the 1970s, the hydraulic analysis assumed sediment and debris do not affect channel hydraulic, especially in concrete channel (Williams 1990). To capture the uncertainty, each channel design include a design safety factor in terms of the

freeboard, which is the additional vertical wall height above the design water elevation. To fully appreciate the design issues and their implications with the aging flood control channels, it is important to understand the basis of design back in the time when these flood control channels were built. This section summarizes the key engineering concepts on flood control channel design.

2.5.1 Hydrology Analysis

The first step of the flood control design is hydrology analysis. The hydrology analysis estimates the design flood flow. The flood flow is based on a number of factors. The most significant factors are precipitation, land cover, and stormwater routing path (Dunne and Leopold 1978). The following is a brief discussion of each parameter.

- Precipitation: While a 100-year storm does not necessarily produce a 100-year flood, a 100-year flood almost always coincides with a 100-year storm. It is especially true along the coastal area, since the 100-year flood may be due to a combination of high intensity storm, high tide and storm surge. 100-year storm is typically defined in one of the following two ways (Dunne and Leopold 1978):
 1. Review historical rain record to identify the storm with 100-year return interval. However, it requires long period of rain record, which is not available for most watersheds.
 2. Prepare regression analysis such as U.S. Geological Survey Bulletin 17B method based on limited historical rain record, typically 30 to 40 years of annual peak rainfall records (USGS 1982). The analysis assumed the relationship between rain intensity and return frequency follows a statistical probability pattern, such as Log Pearson Type III curve. The limited rain data can fit into the curve and the curve estimate the rain intensity from less frequent storms.

Both methods can define peak rainfall intensity, and in most cases the total rainfall volume as well. However for most watersheds historical data is not available. Therefore, flood protection design typically based on regional or synthetic design storm data such as in NOAA Atlas 14 (Bonnin et al 2004).

- Land cover: After rain water hits the ground, the land cover characterizes the rain water infiltration potential. Natural vegetated covers facilitate infiltration, so reduces stormwater volume and peak flow. Urbanized hard surface such as pavement and building roof increases stormwater volume and peak flow. It is because urbanized hard surface prevents infiltration and increase flow velocity (Leopold 1968). Therefore, land cover is an important parameter to determine the flood flow rate (Dunne and Leopold 1978).
- Stormwater routing path: Stormwater routing path affects the flood flow rate in two ways. First, flood flow rate depends on the relative position in the watershed. In most cases stormwater routing path along the downstream portion of watershed have higher flow rate. It

is simply because of its larger catchment area. The second aspect is the stormwater routing mechanism. Detention basins and other flow attenuation features reduce the flood flow rate. Conversely, hard surface drainage such as concrete channel increase flow velocity and likely increase the peak flood flow rate.

These information serves as input parameters for hydrology analysis. There is a wide range of analysis methods. One of the simplest and most common method is the Rational Method, which estimates peak flow rate in watershed up to 200 ac based on rainfall, runoff infiltration potential, and watershed area (Chow 1959). Extended period simulation such as the USACE computer modeling software HEC-HMS (USACE 2010a) can estimates the time series flow pattern or total flow volume.

All these methods are intended to estimate the channel flow rate based on a given rainfall. If the channel has gauge station recording flow rate and rainfall, it is probably the most accurate method to define the design flow. However, flow record at the design event is rarely available. In this case, flow record at a more frequent event can be used to calibrate the hydrology analysis. Generally, if there is a longer period of record available, it improves the accuracy of precipitation return frequency estimate, and return frequency flow rate estimate.

2.5.2 Hydraulic Analysis

The hydrology analysis estimates the design flow rate and flow hydrograph. These are the basic input for the hydraulic analysis. The hydraulic analysis estimates the design water surface elevation profile in the channel. At the USACE, the hydraulic analysis standard for the flood control channel is outlined in the USACE's Engineering Manual 1110-2-1601 (USACE 1959a, 1970, 1991, 1994). The manual has a number of iterations, but mostly follows the backwater calculation method as described in Chow (1959). Key concepts are summarized in this section.

The hydraulic analysis is based on steady state, non-uniform flow conditions. Steady state means flow rate at a location does not change in time. Non-uniform means at a given time flow rate is varying along the channel profile. There are four physical elements affecting channel hydraulic: channel slope, cross sectional area, wetted perimeter, and channel roughness (USACE 1994). Channel slope and cross sectional area are based on the proposed channel design. Wetted perimeter is defined as the channel cross section perimeter in contact with channel flow. It depends on the water surface elevation under the design flow. The channel roughness is based on Manning's equation. The Manning's roughness coefficient, or the n factor, characterize the channel friction loss potential. For example, for concrete channel the n factor is around 0.014 to 0.016, and for earth channel the n factor is around 0.025 (USACE 1970). Note that these n factors and the channel geometry are based on the clear water assumption. Under the clear water assumption, it is assumed bedload and suspended load sediment has insignificant impacts to the channel roughness, and sediment deposition is ineligible during the peak design flow, so the channel geometry will remain as designed.

Many concrete channel designs are based on supercritical flow hydraulic condition, because of its high conveyance efficiency with low water depth and high velocity. Supercritical flow occurs

in an ideal channel condition: steep channel profile slope, straight and uniform channel geometry, and smooth channel surface with low roughness. A more precise technically definition of supercritical flow is a flow condition with Froude numbers greater than 1 (Thompson and Kilgore 2006). Froude number is a dimensionless ratio of inertial and normal forces. Under the supercritical flow condition, channel flow is in high velocity, driving up the inertial force. At the same time, the water surface elevation is below the critical depth. Critical depth is the water surface elevation with the lowest specific energy of the flow. It is shown as Y_c in the Specific Energy Diagram in Figure 2.5.2.1 (Calvert 2007). If there is a change in channel geometry, flow rate, or channel roughness, the supercritical flow may convert to subcritical flow. Subcritical flow has the same specific energy as the supercritical flow, but subcritical flow is under lower velocity and higher water surface elevation above the critical depth. In most natural or man-made channels, channel flow is under subcritical flow condition.

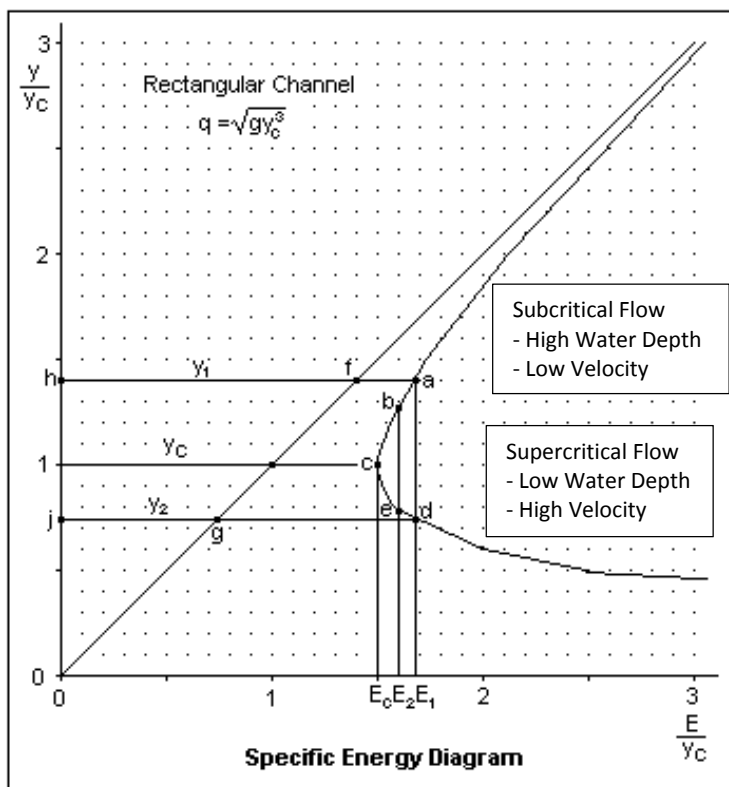


Figure 2.5.2.1: Specific Energy Diagram, shows the water depth-energy relationship. Each specific energy (for example E_1) has a pair of subcritical flow (Point a) and supercritical flow (Point d). The exception is Point c, it is the critical depth with the lowest specific energy. It divides the supercritical and the subcritical flow regimes. (Calvert 2007)

The hydraulic process of shifting supercritical flow to subcritical flow is the hydraulic jump. Figure 2.5.2.2 shows the hydraulic jump schematic (Thompson and Kilgore 2006). The hydraulic jump consists of an abrupt rise of the water surface in the region of impact between supercritical and subcritical flows. The zone of impact of the jump is accompanied by large-scale turbulence, surface waves, and energy dissipation. The area is unstable, hence it is important to identify the

hydraulic jump location in channel design, and reinforce the channel sections with potential hydraulic jump with higher bank height.

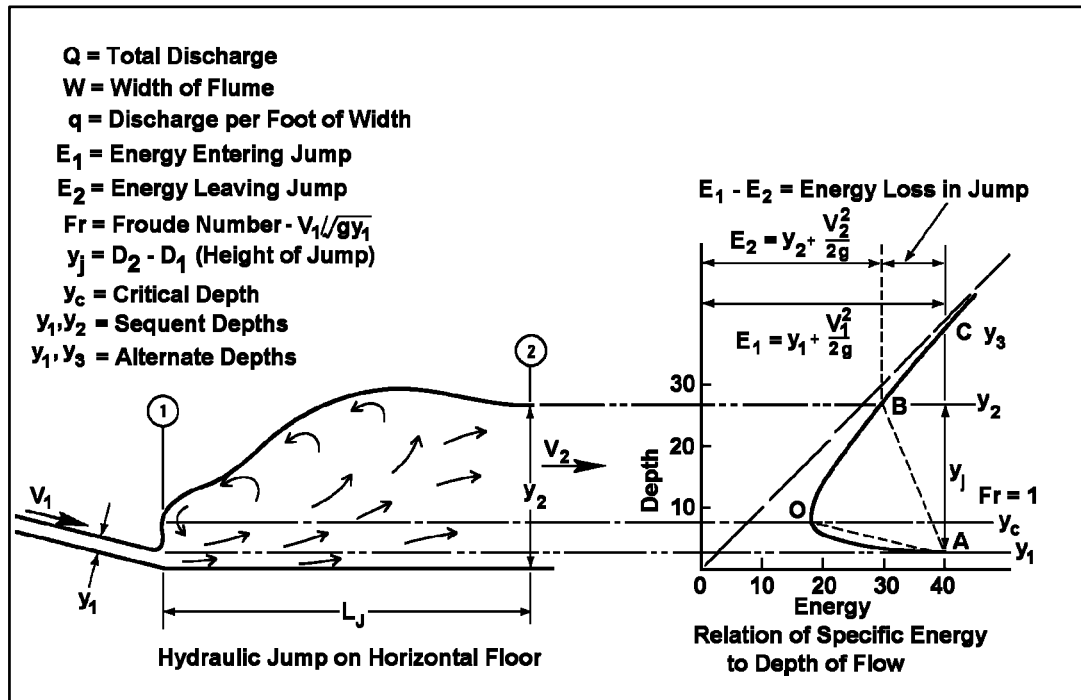


Figure 2.5.2.2: Hydraulic Jump, the transition from supercritical flow at location 1 to subcritical flow at location 2. The specific energy diagram on the right shows the water depth-energy relationship and the energy loss from hydraulic jump. (Thompson and Kilgore 2006)

The hydraulic analysis method assumed clear water condition. The USACE engineering manual in 1970 discussed the need for sediment transport analysis to evaluate potential change in earth channel geometry due to scouring and deposition. It was not until the 1994 revision that include a discussion on hydraulic roughness estimate influenced by sediment, namely the grain and bed roughness. Sediment transport and its hydraulic effects on concrete channel was mentioned in the last section, titled the “Unforeseen Factors”, of the manual. The discussion was based on the studies at Corte Madera Creek in California. However, the manual did not provide formal guideline on how to assess concrete channel roughness influence by sediment transport and sedimentation (USACE 1994).

In this review of the hydraulic analysis method, I illustrated that the flood control channel design is based on a set of idealistic assumptions and design criteria. The use of supercritical flow design in concrete channel demands the channel to function as design, with little margin of error. As I demonstrate in Chapters 3 and 4, hydraulic design based on the clear water assumption underestimates channel capacity especially the concrete channel reaches under supercritical flow. In Chapters 4, I discuss the Corte Madera Creek case study and the issues of sedimentation and sediment transport in detail.

2.5.3 Freeboard

Flood control channel, like any other engineering design, require a safety factor to migrate uncertainty. In flood control channels the safety factor is based on the concept of freeboard. The freeboard of a channel is the vertical distance measured from the design water surface to the top of the channel wall or levee. Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors. These might include erratic hydrologic phenomena; future development of urban areas; unforeseen embankment settlement; the accumulation of silt, trash, and debris; aquatic or other growth in the channels; and variation of resistance or other coefficients from those assumed in design.

The Engineering Manual 1110-2-1601 (USACE 1970) cited the following freeboard allowances guideline.

- 2 ft in rectangular cross sections for concrete-lined channels
- 2.5 ft in trapezoidal cross sections for concrete-lined channels
- 2.5 ft for riprap channels
- 3 ft for earth levees

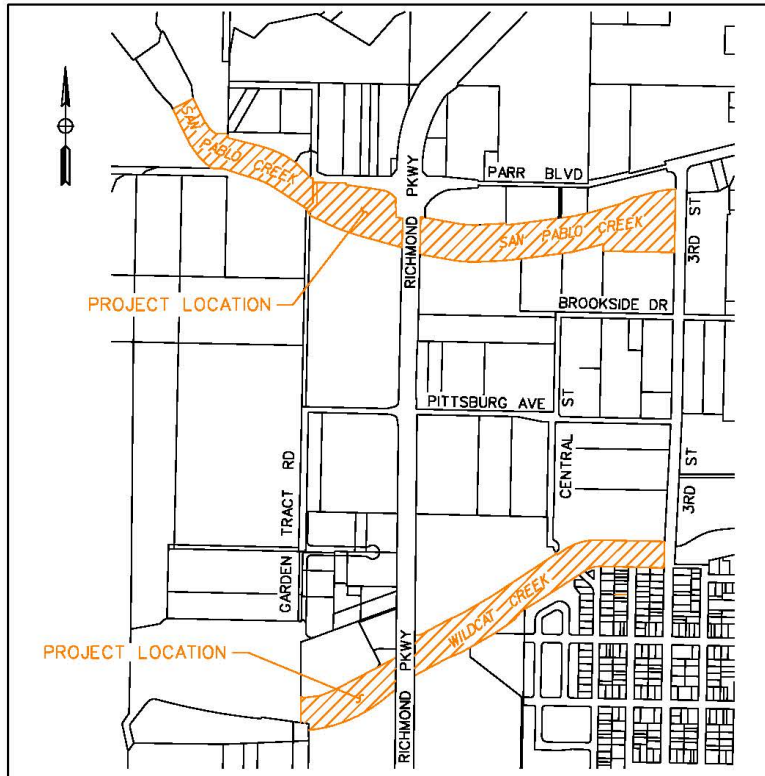
The manual noted that the freeboard for riprap and earth channels may be reduced somewhat because of the reduced hazard when the top of the riprap or earth channels are below natural ground levels. The manual also acknowledged that it is an approximated guideline, the actual freeboard definition should base on the site specific design conditions, and additional freeboard is needed at locations with special hydraulic features such as geometry changes, curves, and hydraulic jumps.

Similar to the USACE, the FEMA flood insurance program also establish a freeboard criteria for levees and floodwalls. Under the NFIP, the levee top height needs to be 3 ft to 4 ft above the 100-year flood elevation (Federal Register 1986). The freeboard is needed for a levee to be accredited as providing 100-year flood protection, so the areas protected behind the levees can be exempted from being a SFHA and subject to flood insurance (USACE 2010b). Comparing the USACE and FEMA levee freeboard requirements, I found that FEMA guideline requires higher freeboard height than USACE guideline. It means that even if the USACE levees were built to the guidance in their engineering manual, and the channel were performed as designed, the levee still could not meet the FEMA accreditation requirements. This conflicts of design criteria is potentially costly to the local agencies. For example, after the Hurricane Katrina in 2004, FEMA worked with the local agencies to re-evaluate the existing levee systems (USACE 2010b). Based on the result of the evaluation, the Contra Costa County Flood Control and Water Conservation District spent over \$1 million to raise the USACE levee sections at the San Pablo and Wildcat Creeks in order to meet the FEMA requirements, Figure 2.5.2.3 shows the project location (CCCFCWCD 2014).

CONTRA COSTA COUNTY
FLOOD CONTROL and WATER CONSERVATION DISTRICT
SAN PABLO AND WILDCAT CREEKS LEVEE REMEDIATION

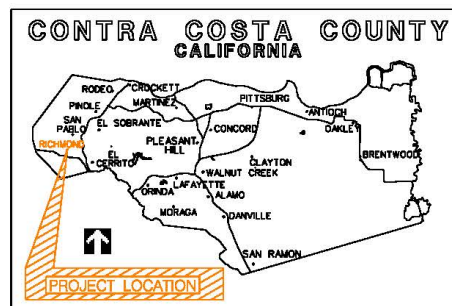
PROJECT NO. 7520-6F8740

TO BE SUPPLEMENTED BY THE 2010 STATE STANDARD PLANS AND THE CURRENT COUNTY STANDARD PLANS



VICINITY MAP

NO SCALE



LOCATION MAP

NO SCALE

Figure 2.5.2.3: Location Map for the San Pablo and Wildcat Creeks Levee Remediation Project (CCCFCWCD 2014)

2.6 USACE Dam and Levee Safety Programs

The previous sections discussed the history of the USACE civil works policy, planning process, and engineering design method for flood control channel. It also discussed the basis of the 100-year event as the national flood protection standard. Based on the Local Cooperation Agreement, operation and maintenance of these flood control channels are usually the responsibility of the local sponsor. While there is periodic inspection and reporting requirements to USACE, there is not a national level program to manage the flood control channels and to prioritize the resources needed to ensure these channels are providing the level of services as intended. For other structural flood control methods, USACE developed a dam safety program since the early efforts in the 1970s (USACE 2011), and is currently implementing a levee safety program as authorized in WRRDA of 2014 (US Congress 2014). These two safety programs provide valuable examples on formulating a possible channel safety program. This section provides an overview of the dam and levee safety programs. The overview highlights the components of these safety programs that could be applicable to a channel safety program.

2.6.1 USACE Dam Safety Program

The purposes of the dam safety program are to protect life, property, and the environment by ensuring that all dams are designed, constructed, operated, and maintained as safely and effectively as is reasonably practicable (USACE 2011). Although construction of dams dates back many years, the history of dam safety covers a much shorter time span (USACE 2011). Only a limited number of states had any laws regulating dam safety prior to 1900. The failure of the South Fork Dam in 1889 at Johnstown, Pennsylvania, resulting in 2,209 deaths, had limited influence on dam safety programs. California initiated a dam safety program following failure of the St. Francis Dam in 1928. Failure of the Buffalo Creek Dam in West Virginia and the Canyon Lake Dam in South Dakota in 1972 contributed to Congress passing The National Dam Inspection Act in 1972. The Reclamation Safety of Dams Act in 1977 followed failure of Teton Dam in Idaho in 1976. Failure of the Laurel Run Dam in Pennsylvania and the Kelly Barnes Dam in Georgia in 1977 set in motion the development of the Federal Guidelines for Dam Safety issued in 1979 by the Federal Coordinating Council for Science, Engineering, and Technology (FCCSET 1979). The key management practices outlined in the guidelines are:

- Establish a Dam Safety Officer and appropriate staff,
- Maintain an updated inventory of dams,
- Document design criteria and construction activities,
- Prepare initial reservoir filling plans and reservoir regulation criteria,
- Prepare operation and maintenance instructions and document activities,
- Maintain a training and awareness program,
- Prepare and maintain Emergency Action Plans (EAP's) for each dam,
- Establish a program of periodic inspections and evaluation of dams, and
- Monitor and evaluate the performance of each dam and appurtenant structure and provide remedial construction as necessary.

In addition, the Interagency Committee on Dam Safety (ICODS) be established to promote and monitor Federal and state dam safety programs. This overview focus on the dam safety program at the USACE. The current policy and procedures outlined in this section are based on the USACE Engineering Regulation 1110-2-1156 (USACE 2011).

The USACE dam safety program is based on the portfolio risk management approach, under the following guiding principles (USACE 2011).

- Life Safety is Paramount. A key mission of the USACE dam safety program is to achieve an equitable and reasonably low level of risk to the public from its dams.
- Do No Harm. The principle of ‘Do no harm’ should underpin all actions intended to reduce dam safety risk. Applying this principle will ensure that proposed IRRM implementation, emergency or permanent construction, or a temporary or compromised at any point in time or during measure implementation.
- Risk-Informed Corporate Approach. The USACE dam safety program will be managed from a risk-informed USACE-wide portfolio perspective applied to all features of all dams on a continuing basis.
- Urgency of Dam Safety Actions. The urgency of actions, including funding, to reduce risks in the short term (i.e., Interim Risk Reduction Measures) and in the long term (i.e., Dam Safety Modifications) will be commensurate with the level of risk based on current knowledge. This may require first addressing only those failure modes that contribute significantly to the overall risk.
- Risk Communications. USACE will provide risk information to internal and external stakeholders. An informed and engaged public is an empowered public that understands risk, can contribute to the evaluation of risk reduction options and can take some degree of responsibility for its safety.
- Prioritization of Studies and Investigations. Studies and investigations will be scoped with the goal of confirming dam safety issues and prioritized to reduce knowledge uncertainties and risk across the portfolio of dams in a cost effective and timely manner.
- Formulation and Prioritization of Risk Management Measures. Where practical, risk reduction measures will be formulated as separable measures and these will be prioritized to achieve tolerable risk as quickly as practicable and in a cost effective manner across the portfolio of dams.
- Level of Detail of Risk Assessments. The level of effort and scope of risk assessments will be scaled to provide an appropriate level of confidence considering the purpose of the risk management decision.
- Routine Dam Safety Activities. Execution of inspections, instrumentation, monitoring, Periodic Assessments, operations and maintenance, emergency action planning and other routine dam safety activities are an essential part of effective dam safety risk management for all USACE dams.
- Risk Reporting. The current level of risk for USACE dams will be documented and routinely reported. The basis for decisions will be documented.

Risk is a measure of the probability and severity of undesirable consequences. Evaluating and reducing risk requires a decision-making framework that explicitly evaluates the level of risk if no action is taken and recognizes the monetary and non-monetary costs and benefits of reducing

risks when making decisions. Under the dam safety program, risk analysis comprises three tasks: risk assessment, risk management, and risk communication.

Risk assessment is to quantify and describe the nature, likelihood and magnitude of risk under current and future conditions. Risk assessment involve the following four steps (NRC 1983):

- Hazard Identification (Risk Identification)
- Hazard Characterization (Risk Identification)
- Exposure Assessment (Risk Estimation)
- Risk Characterization (Risk Estimation)

An important aspect of risk assessment is to separate uncertainty from variability. Uncertainty is the result of imperfect knowledge about the system. Variability is an inherent property of the system. With better data and knowledge with the system, uncertainty can be reduced, but it can only better characterize variability. Variability cannot be reduced. Risk assessment need to clearly define uncertainty and variability, since they can influence risk management decisions.

Risk management is a process of problem finding and initiating action to identify, evaluate, select, implement, monitor and modify actions taken to alter levels of risk, as compared to taking no action. The process include assess risk management options, implement risk management decisions, and monitoring and review risk management implementations.

Risk communication ensures that the decision makers, other stakeholders and affected parties understand and appreciate the process of risk assessment and in so doing can be fully engaged in and responsible for risk management. Successful risk communication leads to a common recognition and understanding of the nature and uncertainties of risk, risk management options, and shared acceptance of the risk management decisions.

The metric for the risk analysis is based on the concept of tolerable risk. Tolerable refers to a willingness to live with a risk to secure certain benefits and with confidence that it is being properly managed. Tolerable risk is not regarded as negligible or something to ignore. It must, however be kept under review and reduced further if possible. In the dam safety program, tolerable risk guidelines are used in risk management to guide the process of examining and judging the significance of estimated risks obtained using risk assessment. There are four risk measures in the guidelines:

- Annual probability of failure
- Life safety risk
- Economic risk
- Environment and other non-monetary risk

When evaluating a dam and making risk management decisions, life safety risk and annual probability of failure will be given preference, with economic and environmental risk being given due consideration. For those projects where there is very low or no life safety risk, economic consequences and annual probability of failure will be the primary considerations along with environmental risk in making risk management decisions. The outcomes of risk

assessment are inputs to the risk management decision process along with other considerations. Meeting or achieving the tolerable risk guidelines is the goal for all risk reduction measures including permanent and interim measures.

The dam safety portfolio risk management process is a series of hierarchical activities that are used to assess, classify, manage, and monitor the risks associated with the USACE inventory of dams. Figure 2.6.1 outlined the management process. The process starts with an initial risk assessment to screen all dams within the USACE portfolio. The Screening Portfolio Risk Assessment is the one time rapid process to estimate the probability of failure, and the life and economic risk for each dam. The result classify a dam based on the Dam Safety Action Classification (DSAC) system. DSAC system provides consistent and systematic guidelines for appropriate actions to address the dam safety issues and deficiencies of USACE dams. USACE dams are placed into a DSAC class based on their probability of failure or the individual dam safety risk estimate considered as a combination of probability of failure and potential life safety, economic, environmental, or other consequences. The classification of a dam is dynamic over time, changing as project characteristics are modified or more refined information becomes available affecting the loading, probability of failure, or consequences of failure. There are five DSAC classes, depict the range of dams from those critically near failure to those considered adequately safe.

Class I (Urgent and Compelling). Class I is for those dams where progression toward failure is confirmed to be taking place under normal operations and the dam is almost certain to fail under normal operations within a time frame from immediately to within a few years without intervention; or, the combination of life or economic consequences with probability of failure is extremely high.

Class II (Urgent). Class II is for dams where failure could begin during normal operations or be initiated by an event. The likelihood of failure from one of these occurrences, prior to remediation, is too high to assure public safety; or, the combination of life or economic consequences with probability of failure is very high.

Class III (High Priority). Class III dams have issues where the dam is significantly inadequate or the combination of life, economic, or environmental consequences with probability of failure is moderate to high.

Class IV (Priority). Class IV dams are inadequate with low risk such that the combination of life, economic, or environmental consequences with a probability of failure is low and the dam may not meet all essential USACE guidelines.

Class V (Normal). Class V is for dams considered adequately safe, meeting all essential agency guidelines (see Appendix E) and the residual risk is considered tolerable.

If a dam classified as Classes I, II, and III, it is required to develop and implement an Interim Risk Reduction Measure (IRRM) Plan. The IRRM Plan frames operational decision making by establishing specific threshold events, decision points, and actions required. It provides a plan for which emergency measures such as rapid reservoir drawdown and recommendations on

evacuation of the reservoir storage must occur. Funding for IRRM plan preparation and implementation are from the program's operation and maintenance account at the USACE District level.

Classes II, III, and IV dams also required to develop an Issue Evaluation Studies (IES). IES are used to assist dam safety officials with making risk informed decisions by determine the nature of a safety issue or concern, and the degree of urgency for action within the context of the entire USACE inventory of dams. IES focus on risk evaluation on the significant potential failure modes, to determine if the issues of concern approach or exceed the tolerable risk limits. The potential failure mode analysis identify the initiators, the failure progression mechanisms, and the resulting impacts. Facilitation is a critical part of the process to develop credible risk estimates during an assessment of risk. Facilitators are assigned to teams and contribute to the process by bringing experience with risk assessments, consistency in approach, knowledge of latest technology in risk assessments, and serves as a resource to the risk assessment team for technical input and questions. Based on the potential failure mode analysis, IES verifies the current DSAC rating, guide the selection and gauge the effectiveness of interim risk reduction measures, and justify and prioritize Dam Safety Modification (DSM) studies within the context of the entire USACE inventory of dams.

For Class I dams, and based on the IES recommendation for Classes II, III, and IV dams, USACE proceed with the Dam Safety Modification (DSM) studies. The study objective is to identify and recommend an alternative risk management plan that supports the expeditious and cost effective reduction of risk within the overall USACE portfolio of dams. DSM studies undertaken the same six step planning process in civil works feasibility study, per the Engineering Regulation 1105-2-100 (USACE 2000) and summarized in Section 2.3.1. Recommended risk management alternatives are to be technically feasible and acceptable following current best practices, comply with applicable laws, and satisfy applicable tolerable risk and essential USACE guidelines for remediation of existing dams. The risk associated with each failure mode being addressed by a risk management alternative must be reduced to a level that satisfies the tolerable risk guidelines. The intent is to achieve a complete remediation of those individual failure modes being addressed by the plan to support the ultimate goal of having an adequately safe dam where the dam meets essential USACE guidelines and the total residual risk for the dam is considered tolerable (DSAC V). Each alternative risk management plan must be formulated to support effective and efficient risk reduction within the USACE portfolio of dams which may require a staged implementation approach. The principle of "Do No Harm" must be respected in development of the risk management plan.

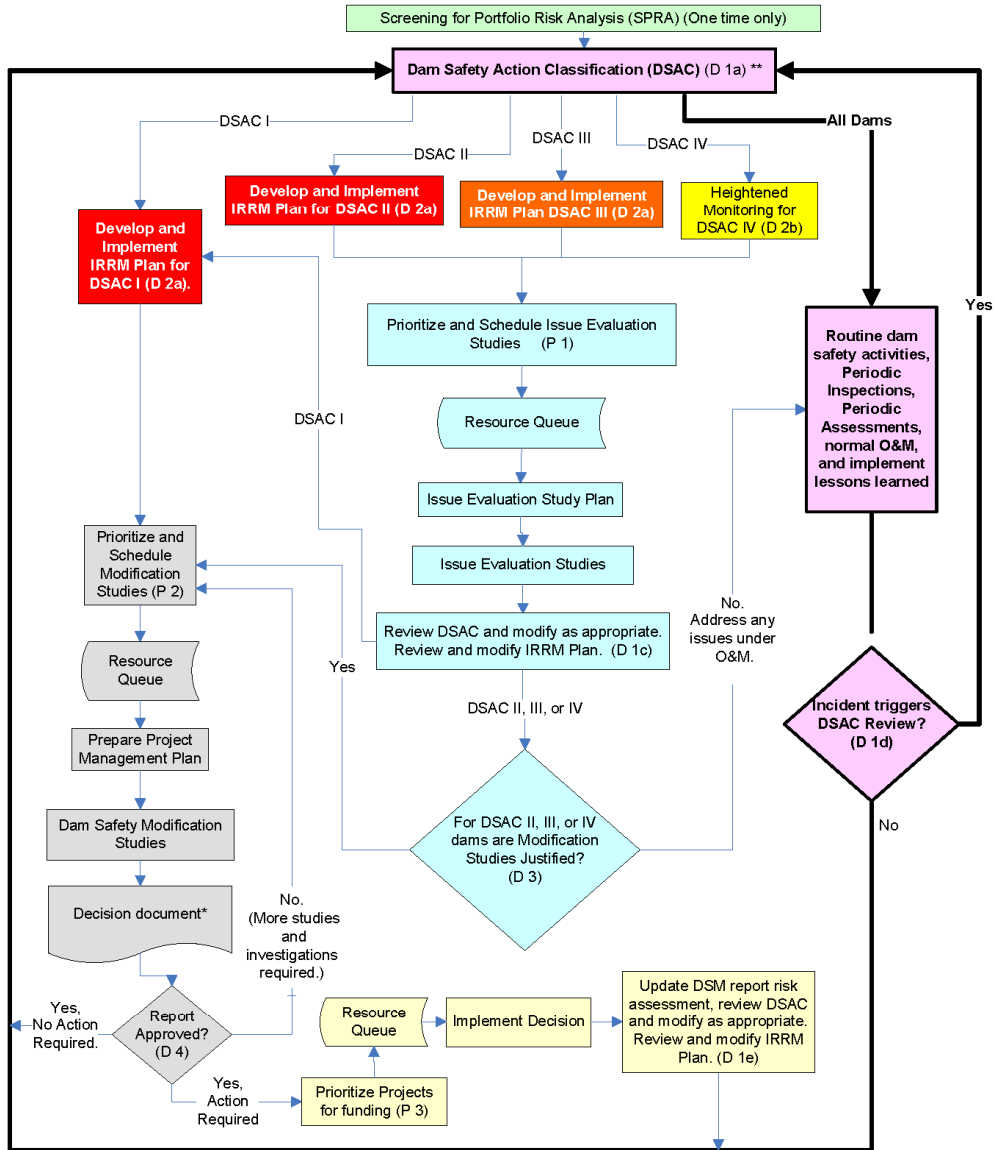
The cost of dam safety modifications needed to address new hydrologic or seismic data shall be cost shared with the local sponsor. WRDA 1986 Section 1203 authorization may be used to modify dams built by the USACE where local interests are responsible for operation, maintenance, repair, rehabilitation, and replacement, but only if Congress directs the Secretary of the Army to do so, in law, for a specifically named project. Without specific congressional direction, in law, non-Federal sponsors remain responsible for operation, maintenance, repair, rehabilitation and replacement of these projects, as required by their authorizations and the terms of the agreements, including cost sharing, under which they were constructed by the Federal government.

Another component of the dam safety program is inspection and assessment. USACE conducts two types of dam inspections. The first one is the Annual Inspection, which is performed on an annual basis to ensure the dam is being properly operated and maintained. The Periodic Inspection is the next level of inspection and is conducted by a multidisciplinary team led by a professional engineer. It includes a more detailed, comprehensive evaluation of the condition of the dam and will be conducted every five years. Components of the Periodic Inspection include evaluating annual inspection items; verifying proper operation and maintenance; evaluating operational adequacy, structural stability, and safety of the system; and comparing current design and construction criteria with those in place when the dam was built. Periodic Assessments consist of a periodic inspection, a potential failure modes analysis, and a risk assessment based on existing data and a minimum development of limited consequence data. Periodic Assessment of projects shall be conducted as determined by risk factors, but they shall not exceed a ten-year interval.

To support the dam safety program to provide better data management, USACE maintains and publishes the National Inventory of Dams (NID). NID contains information on approximately 79,000 dams throughout the U.S. that are more than 25 feet high, hold more than 50 acre-feet of water, or are considered a significant hazard if they fail. NID covers all 50 states, Puerto Rico, and 16 Federal agencies (USACE 2014a).

The USACE maintains a three-level decentralized organization, the Headquarter at the national level, the Major Subordinate Command at the regional level, and the District at the local level. Each organizational level have a Dam Safety Officer (DSO) with supporting organization to implement the dam safety program. National oversight is furnished by the Dam Safety Steering Committee and the Senior Oversight Group. Prioritization of all risk assessments, studies and remediation are managed on behalf of the Headquarters by the Risk Management Center (RMC), with oversight by the Senior Oversight Group and Special Assistant for Dam Safety. RMC has been established to provide technical expertise and advisory services to assist in managing and facilitating the USACE-wide dam safety program. In support of the Headquarter management of the dam safety program, the Modeling, Mapping, and Consequences (MMC) Production Center performs hydraulic modeling, mapping, and consequences analysis for USACE dams.

Figure 3.1 -- Corps of Engineers Dam Safety Portfolio Risk Management Process



Decision Points are label as (D 1a), Prioritization Points are labeled as (P 1), and the details for each point is explained in Chapter 3.
 * Independent External Peer Review requirements are to be addressed per guidance in the Dam Safety Modification chapter.
 ** Regardless of DSAC classification, dams with insignificant or no consequences should they fail are considered exceptions; will be so tagged, and are exempt from the dam safety portfolio management process depicted here in Figure 3.1.

Figure 2.6.1: USACE Dam Safety Portfolio Risk Management Process (USACE 2011)

2.6.2 USACE Levee Safety Program

In 2005, Hurricanes Katrina and Rita in the Gulf Coast caused more than \$200 billion economic damages and more than 1,800 deaths (NCLS 2009). The role of levees in providing for public safety and flood risk management was prominently thrust into the national spotlight. In 2006, the USACE created a levee safety program with the mission to assess the integrity and viability of levees and recommend courses of action to make sure that levee systems do not present unacceptable risks to the public, property, and environment. USACE subsequently launched a major effort to establish a levee safety organization, create the National Levee Database (NLD), develop a methodology for performing technical risk assessments of existing levee infrastructure, and review and revise current related policies and procedures associated with levees (USACE 2010b). In 2007, Congress passed the National Levee Safety Act (US Congress 2007). The act seeks to develop basic information on federal levees to quantify the risk of the existing levees, and develop a National Committee on Levee Safety to provide recommendations to the Congress for implementation of a levee safety program.

The levee safety program is still in the development stage. Many components in development shares similarities with the dam safety program, such as the inventory database and levee inspection. USACE has created a levee database to serve as a living, dynamic depository of information. The database includes attributes of levees design, construction, operations, maintenance, repair, and inspections (USACE 2014b). USACE has completed inventorying all levees included in the USACE levee safety program.

USACE updated the levee inspection checklist and currently conduct two types of inspections. The annual inspection is a visual inspection to verify and rate levee system operation and maintenance in three level: Acceptable, Minimally Acceptable, and Unacceptable (USACE 2008a). The periodic inspection is a comprehensive inspection conducted by a USACE multidisciplinary team that includes the levee sponsor and is led by a professional engineer. USACE typically conducts this inspection every five years (USACE 2008b). Periodic inspections include review existing data on operation and maintenance, previous inspections, emergency action plans and flood fighting records, perform additional field inspection, evaluate design criteria changes since the levee was constructed, and recommend further evaluations and improvements as needed.

As part of the FEMA levee accreditation process, USACE conduct levee condition evaluation for federally authorized levees and non-federal levee in the Levee Safety Program. The levee evaluation is based on the probabilistic approach as defined in the Engineering Circular 1110-2-6067 (USACE 2010b) and Engineering Manual 1110-2-1619 (USACE 1996). Risk-analysis methodologies for analyzing the full suite of engineering and operational elements of a levee system is under development for application to USACE levee safety assessments. One of the objectives of the levee safety program is to create a consistent risk-based method to evaluate levees nationally and to prioritize actions to maximize flood risk reduction to the public. USACE continue to develop and implement policies and procedures to assess, evaluate and communicate risks associated with levees.

In 2009, the National Committee on Levee Safety developed a set of recommendations for a national levee safety program. A summary of the recommendations is attached in the Section 2.8 (NCLS 2009). Key recommendations include establishing national leadership on levee safety program, developing safety standards including a hazard potential classification system and tolerable risk guidelines, updating the NFIP levee certification terminology and peer review requirements, harmonizing levee safety with environmental protection requirements, providing outreach, training and technical support, coordinating with states levee safety programs and FEMA flood insurance program, and providing funding for levee safety works. These recommendations are incorporated into WRRDA 2014 (U.S. Congress 2014).

2.6.3 Channel Safety Program

As discussed in this section, the dam and levee safety programs provided a systematic approach for the USACE to manage the risk of its dam and levee assets, and to prioritize its limited resources on its most critical improvement needs. A similar portfolio based management approach could apply to flood control channel. While most of the dam and levee safety program components are needed and translatable to a channel safety program, I recommend the following components that are most critical to the success of a possible channel safety program.

- Establish a national leadership on the channel safety program. The leadership should:
 - Ensure consistent approach for all flood control channels
 - The approach does not create conflict with other agencies' standards such as the FEMA NFIP
 - Enforce the periodic inspection and assessment will be in place and on schedule
 - Facilitate information exchanges between local sponsors and agencies, so all can leverage the collective expertise to improve flood control channels management.
- Develop a database to inventory all USACE flood control channels. Such a database development could begin with a review on the National Levee Database, to identify flood control channel reaches and import/link the data into the channel safety database. The database also provides a platform to systematically document channel performance observations and periodic inspection and assessment records.
- Provide an initial screening to all flood control channels to assess their current performance and risk level, and follow-up with periodic inspection and assessment. While the flood control channel currently require periodic inspection, there is not a requirement to assess channel performance and risk level in periodic basis. Therefore, the existing condition is unknown, and many flood control channels' rated capacity is based on the original design, especially in the FEMA Flood Insurance Study (FIS) that form the basis of the Flood Insurance Rated Maps (FIRM).
- Develop a risk analysis approach and tolerable risk guidelines for all flood control channels. Such approach should build upon the Engineering Manual 1110-2-1619, and improve the uncertainty quantification on the flow stage relationship. The approach should also incorporate structural, geomorphic, and other elements beyond hydrology and hydraulic

parameters in the potential failure mode analysis. The approach should be consistent with the dam and levee safety programs.

- Develop a channel safety action classification system, based on similar classification system for dams. However, the quantitative risk definition for each class should be based on a combination of the annual probability of failure, economic risk, and environment and other non-monetary risk. The life safety risk is omitted since in most channel overflows the life safety risk is not as significant as for dam breaks and levee breaches.

These recommended components focused on the initial efforts to assess and classify the existing flood control channel inventory. After these components are implemented and the initial assessment and classification efforts are underway, I recommend that the channel safety program leadership should continue to develop method on how to evaluate, select, and prioritize flood control channels that requires additional assessment and/or improvement evaluation studies. They are essential for decision making under incomplete portfolio information to prioritize resources for flood risk reduction. I also recommend that the methods and approaches in the channel safety program should be consistent with the dam and levee safety programs, such that the assessment results and improvement justifications can be compared across different technologies under the same basis.

2.7 Conclusion

In this chapter I reviewed the history of the USACE policy and planning process, the background of the 100-year flood, and the engineering standards for flood control channel design. My review highlighted the factors that shape the flood control channels built in the 1950s to 1970s. I found that the policy at the time, especially the cost benefit analysis, was singly focused on the national economic benefits, primarily to reduce flood damage and increase land value. The planning process was also singly focused on the efficiency to meet the project objective with minimum cost. The process did not have an explicit requirement to evaluate alternatives focusing on different objectives. The lack of environmental regulation at the time allowed projects to ignore their environmental consequences. While the USACE project cost sharing was an attractive financial incentive for local sponsor partnership, I found the Local Cooperation Agreement limits the local sponsor's influence on the project, while the local sponsor is responsible for the project operation and maintenance. My review of the flood control channel design standard shows that the channel was designed based on idealistic hydraulic conditions that are high risk and unrealistic. The concrete channel design assumed supercritical flow condition and did not account for sediment. As I discuss in Chapters 3 and 4, these design issues lead to significant channel capacity deficiency and costly channel maintenance requirements.

The USACE dam safety program and the work-in-progress levee safety program provided example frameworks on managing aging infrastructure in a national portfolio approach. I suggest a similar safety program for flood control channel to understand the risk of these channels, to have a collective and organized system to periodically inspect and assess the channels, and to prioritize management and the limited resources for improvement.

2.8 Attachment: Summary of Recommendations for a National Levee Safety Program (NCLS 2009)

Comprehensive and Consistent National Leadership

1. Establish a National Levee Safety Commission to provide national leadership and comprehensive and consistent approaches to levee safety including standards, research and development, technical materials and assistance, training, public involvement and education, facilitation of the alignment of federal programs and design, delegation and oversight of a delegated program to states.
2. Expand and Maintain the National Levee Database to include a one-time US Army Corps of Engineers inventory and inspection of all non-federal levees. Baseline information will be included and maintained in an expanded National Levee Database (NLD) in order that critical safety issues, true costs of good levee stewardship, and the state of individual levees can inform priorities and provide data for needed risk-informed assessments and decision-making.
3. Adopt a Hazard Potential Classification System as a first step in identifying and prioritizing hazard in leveed areas. Due to a lack of data regarding probability of failure, initial classifications should be based solely on consequences in order to assist in setting priorities, criteria, and requirements as the NSLP is being established.
4. Develop and Adopt National Levee Safety Standards that will assist in ensuring that the best engineering practices are available and implemented throughout the nation at all levels of government.
5. Develop Tolerable Risk Guidelines in order to facilitate an understanding of the options to reduce identified risks, how uncertainty affects this understanding, and to better inform levee construction/enhancement decisions and weigh nonstructural alternatives to flood risk management in a risk-informed context.
6. Change “Levee Certification” to “Compliance Determination” to better articulate the intent that “certification” under the National Flood Insurance Program (NFIP) requirements does not constitute a safety guarantee or warranty. The purpose of this change is to more clearly communicate residual risks of living and working in leveed areas.
7. Subject Levee Certifications (Compliance Determinations) under FEMA’s National Flood Insurance Program to Peer Review in order to increase confidence in technical determinations of compliance.
8. Swiftly Address Growing Concerns Regarding Liability for Damages Resulting from Levee Failures through exploration of a range of measures aimed at reducing the potential liability of engineering firms and/or government agencies that perform engineering services for levee systems (e.g. inspections, evaluations, design, construction administration, certification, or flood fighting). Congress should address this liability concern as a first priority in order to help ensure state and local interest in developing levee safety programs, and to prevent much needed levee repairs, rehabilitation and certification from coming to a halt.
9. Develop a Comprehensive National Public Involvement and Education/Awareness Campaign to Communicate Risk and Change Behavior in Leveed Areas as an essential element of levee safety by improving public understanding of the role of levees, associated risks, and individual responsibilities to empower people to make risk-informed choices.

10. Provide Comprehensive Technical Materials and Direct Technical Assistance crucial to the successful implementation of consistent national standards to states, local communities and owner/operators.

11. Develop a National Levee Safety Training Program including a combination of courses, materials, curricula, conferences, and direct assistance resulting in an increase in the level of expertise and knowledge in all aspects of levee safety. This would include the development of curricula and certification requirements for a Certified Levee Professional program.

12. Develop and Implement Measures to More Closely Harmonize Levee Safety Activities with Environmental Protection Requirements to ensure that critical levee operations and maintenance is not delayed and that, where possible without compromising human safety, environmentally-friendly practices and techniques are developed and used.

13. Conduct a Research and Development Program that will continually advance state-of-the-art technologies and practices for levee safety and conduct critical operations and maintenance activities in as cost-effective and environmentally-friendly manner as possible.

Building and Sustaining Levee Safety Programs in All States

14. Design and Delegate Program Responsibilities to States to assist states and local governments develop effective levee safety programs focused on continual and periodic inspections, emergency evacuation, mitigation, public involvement and risk communication/awareness, etc.

15. Establish a Levee Safety Grant Program to assist states and local communities develop and maintain the institutional capacity, necessary expertise, and program framework to quickly initiate and maintain levee safety program activities and requirements.

16. Establish the National Levee Rehabilitation, Improvement, and Flood Mitigation Fund to aid in the rehabilitation, improvement or removal of aging or deficient national levee infrastructure. Investment (cost-shared) is recommended to be applied to the combination of activities, both structural and non-structural, that combined, would maximize overall risk reduction and initially be focused in areas with the greatest risk to human safety.

Aligning Existing Federal Programs (Incentives and Disincentives)

17. Explore Potential Incentives and Disincentives for good levee behavior through alignment of existing federal programs.

18. Mandate Purchase of Risk-Based Flood Insurance in Leveed Areas to reduce financial flood damages and increase understanding of communities and individuals that levees do not eliminate risk from flooding.

19. Augment FEMA's Mapping Program to improve risk identification and communication in leveed areas and consolidate critical information about flood risk.

20. Align FEMA's Community Rating System (CRS) to Reward Development of State Levee Safety Programs by providing further incentives to communities to exceed minimum program requirements and benefit from lower risk-based flood insurance rates to individuals who live in leveed areas.

CHAPTER 3 – A SAN FRANCISCO BAY AREA CASE STUDY ON THE USACE FLOOD CONTROL PROJECTS

3.1 Introduction

The United States Army Corps of Engineers (USACE) historically involved in limited flood control work. The work mainly focused on Mississippi River and Sacramento River watersheds. In the 1936 Flood Control Act, the congress added flood control to the USACE's water resources responsibility. From the 1930s to 1960s, USACE constructed levees, dams, and channel throughout the nation (Arnold 1988). As discussed in Chapter 2, these projects were justified by the cost benefit analysis when the project was proposed, and the project planning and authorization processes draw critical criticisms. Many of these projects are now approaching the end of service life, at least based on the lifespan used in the economic analysis. It is important to evaluate these projects to assess whether they function as designed, and this evaluation provides justification to explore options to improve the planning, design, and management practices for these aging projects and future projects.

Most of the USACE flood control projects consist of structural approaches, namely dams, levees, and channels. USACE has a dam safety program and levee safety program in place to evaluate and maintain these infrastructures (USACE 2011, US Congress 2014), but none exist for flood control channel. In addition, most of the USACE project agreements require turning over project ownership to the local sponsor after the construction is completed. The local sponsor will be responsible for project operation and maintenance. Therefore, USACE does not have a direct management role to the flood control channels. As these flood control channels age over time, currently there is not a nation-wide program to systematically track their performance and prioritize improvement needs.

In addition, these flood control channels suffers planning and design issues that impair their performance (Williams 1990). I identified the three most significant issues:

- Disconnection from stream corridor. The primary design criteria for flood control channel is to maximize channel flow capacity in a minimal footprint. Therefore, the flood control channels typically have straightened channel alignment for two purposes:
 - Increase channel slope to increase flow velocity.
 - Reducing travel distance to decrease travel time.

In addition, the channel corridor is sized with uniform shape to contain the design flow, so for the single purpose flood flow conveyance, the adjacent floodplain is not needed. Therefore, the stream floodplain will not be flooded and it can be utilized for urban development. It is captured in the "land enhancement" benefit under the cost benefit analysis. The land enhancement creates two issues.

The first issue is the loss of riparian habitats from new urban developments in the floodplains. Even for areas that are undisturbed, the floodplain is hydrologically disconnected from the stream so the ecological function is severely compromised.

The second issue is the combination of economic pressure and poor landuse planning lead to urban development up to the banks of the flood control channels. The constrained channel corridor eliminated options for channel expansion or floodplain restoration. In many cases, the constraint also prohibited local sponsor to access the channel banks for maintenance operations.

- Lack of flexibility to adapt changing watershed hydrology. The flood control channel is designed as a high performance technology for efficient flow conveyance. With the typical uniform shape concrete or earth channel design, plus the limited channel corridor due to urban development encroachment as discussed above, flood control channel has little margin for change. It becomes problematic when the watershed hydrology change over time from natural hydrology pattern to high peak low travel time runoff, or when the flood frequency curve updated with new gauge data and it changes the design flow. Within the narrow creek corridor, it is difficult and expensive for meaningful stream restoration, or even just to increase channel conveyance capacity to meet the new design flow and flood protection level of service.
- Sediment impacts. One of the key technical design deficiency on these flood control channel is the design disregard the impacts of sediment, under the clear water assumption. For the clear water assumption, the project design assumed channel flow only consists of water with no sediment or debris. The hydraulic impacts of sediment within the flow, and sediment deposition in the channel are not accounted for in the channel design. As a result, the clear water assumption lead to the following questionable channel designs, with the channel capacities unrealistically overestimated.
 - Concrete channel designed under supercritical flow with roughness coefficient based on smooth concrete surface. In reality, sediment transport and deposition in the concrete channel increase channel roughness, lead to water surface elevation increase due to hydraulic jump to subcritical flow.
 - Channel outfall thalweg elevation was set lower than the adjacent bay bed to increase longitude slope for higher channel capacity. However, this sump created at the channel outfall trapping sediment, and it reduces the effective flow area, channel slope, and channel capacity.

The combination of these fundamental planning and design issues and the lack of a Federal management framework to operate and maintain the channels, and to identify and prioritize improvement needs created an uncertainty on how to manage these aging infrastructure. Since most of these flood control channels are within densely populated urban areas, there is a significant public health and safety consequences from the increased flood risk due to non-performing flood control channels.

Taking the San Francisco Bay Area (Bay Area) as an example, many of these flood control channels built in the 1950s to 1970s are now approaching their end of service life. These rigid structures are inflexible to adapt to the new urban hydrology, and many are plagued with costly

maintenance problems due to structural deterioration and in-channel sediment deposition. To better understand the scope of the problems, I interviewed flood control agencies in Bay Area. Most of the interview discussions covered the agencies specific issues, but many of them do not fully aware how widespread these issues are among other flood control channels in Bay Area. There is a knowledge gap to systematically evaluate and identify common planning and design issues in the USACE flood control channels.

To address this knowledge gap, and to provide justification on a need to develop a management framework for flood control channels, similar to levee and dam safety programs, I conducted a post project examination on the USACE flood control channels in San Francisco Bay Area. The goal of this case study is to systematically quantify the issues, and the commonalities of these issues in the flood control channels.

3.2 USACE Flood Control Projects in San Francisco Bay Area

The San Francisco Bay Area (Bay Area) metropolitan region includes nine counties surrounding the San Francisco and San Pablo Bays, which are hydraulically connected to the Pacific Ocean to the east and Sacramento/San Joaquin delta system to the west. The Sacramento/San Joaquin delta system is the main source of fresh water supply (Conomos et al 1985). It also provides significant sediment supply to the Bay Area, as a result of hydraulic mining in its tributaries in the nineteenth century during the gold rush (Krone 1979). The area is in the Mediterranean climate zone, which has dry summer and wet winter seasons. Most of the flooding in Bay Area occur during the wet winter seasons, between October and April. Average annual precipitation ranges from 15 inches in South Bay (WRCC 2014b), 21 inches in San Francisco (WRCC 2014a), to close to 60 inches in Marin County (USACE 2010). Recent wet years include 1982-1983, 1997-1998, and 2005-2006 (Stetson 2011).

Urban developments in Bay Area is concentrated around the low laying area adjacent to the Bay, mostly in the City/County of San Francisco, San Mateo County, Santa Clara County, Alameda County, and Contra Costa County. Due to the close proximity to the bay, the urban areas are vulnerable to a combination of fluvial flooding, coastal flooding, and potential sea level rise impacts (SFBCDC 2011). Flood control responsibility varies between counties. Typically flood control districts maintain streams and major storm drain infrastructures, interior drainage is local municipalities' responsibility. In San Francisco, the flood control responsibility shares between San Francisco Public Utility Commission and San Francisco Public Works Department. Although San Mateo County has a flood control district, the District only responsible for Colma Creek, San Bruno Channel, and San Francisquito Creek (County of San Mateo Public Works 2014).

In addition to the storm drainage and flood control infrastructures built and operate by the municipal and regional agencies, USACE is also heavily involved in flood control works in Bay Area. Table 3.2.1 listed the USACE flood control projects constructed in Bay Area. Figure 3.2.1 shows the locations of the projects. This list covers flood control projects in the urban areas around the Bay, and except the Fairfield Vicinity Streams project, all projects are partially within the tidal zone. The table also listed the authorization for each project. As discussed in Chapter 2,

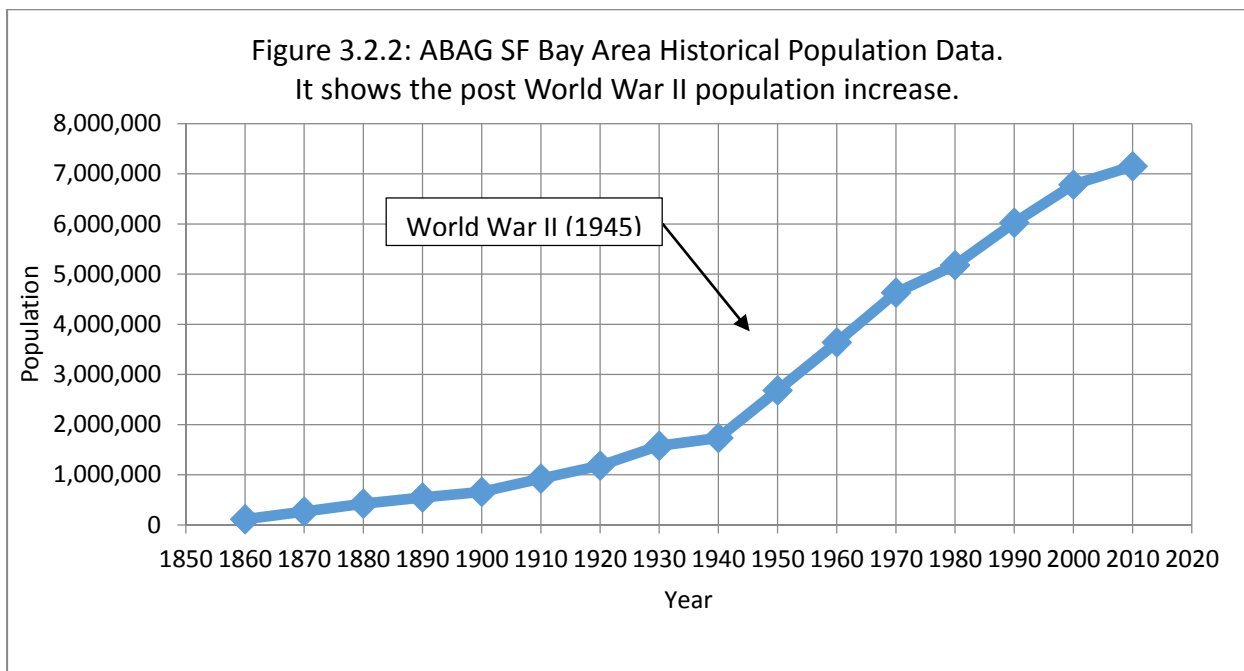
projects that are authorized under the Continuing Authorities Program (CAP) are smaller projects with specific proposes, such as Section 205 for flood control, and under the specific Federal spending limit at the time the project was authorized. These projects have faster turnaround time and do not need separate Congressional authorization for each project. Larger projects that exceed the authorization limit under the CAP are congressionally authorized in Flood Control Act (FCA) or Water Resources Development Act (WRDA). In addition to the project listed in Table 3.2.1, Table 3.2.2 listed the projects currently in progress. Most projects are still in study phase, Napa River and Petaluma River projects are currently in construction.

The project list in Table 3.2.1 shows that in Bay Area, there was a period of active flood control project implementations between 1960 and 1975, an era when the baby boom after World War II created a rapid population growth in the San Francisco Bay Area (Figure 3.2.2), and in response to floods in the 1950s and early 1960s, especially in 1955 and 1958. I found that most projects cited these two years had their worst flood in recent history, and provide strong supports to process with the flood control projects.

Creek Watershed Name	County	Case Study	Authorization	Authorized	Constructed
Rheem Creek	Contra Costa	X	CAP	1959	1960
San Lorenzo Creek	Alameda	X	FCA 37 54	1954	1962
Green Valley & Dan Wilson Creek	Solano		CAP	1960	1962
Walnut Creek	Contra Costa	X	FCA 50 60	1960	1965
Coyote Creek	Marin	X	CAP	1963	1965
Pinole Creek	Contra Costa	X	CAP	1963	1966
Rodeo Creek	Contra Costa	X	CAP	1963	1966
Corte Madera Creek	Marin	X	FCA 30 44 62	1962	1971
San Leandro Creek	Alameda	X	CAP	1970	1974
Alameda Creek	Alameda	X	FCA 37 62	1962	1975
Wildcat Creek	Contra Costa		FCA 60 76	1976	1990
San Pablo Creek	Contra Costa		FCA 60 76	1976	1991
Fairfield Vicinity Streams	Solano		CAP	1970	1992
Coyote Creek	Santa Clara		FCA 41 WRDA 86 90	1990	1996
Guadalupe River	Santa Clara		FCA 30 41 WRDA 86 92 99 01	1986	2004

Note: Projects authorized by the Flood Control Acts or Water Resources Development Acts are highlighted in yellow.

Table 3.2.2: USACE Channel Projects in Design or Construction in San Francisco Bay Area		
Creek / Project Name	County	Status
Arroyo De La Laguna	Alameda	Study
Berryessa Creek	Santa Clara	Design
Estudillo Canal	Alameda	Study
Gallinas Creek	Marin	Study
Laguna Creek	Alameda	Study, on hold
Napa River	Napa	Construction
Petaluma River	Sonoma	Construction
Pinole Creek	Contra Costa	Study
San Francisquito Creek	Santa Clara	Study
Upper Guadalupe River	Santa Clara	Design
Upper Penitencia Creek	Santa Clara	Study



As I discussed in Chapter 2 and Section 3.1, these flood control project designs were highly utilitarian and require significant operation and maintenance. However, often times the expensive maintenance efforts still may not be able to keep up the channel to its original design specification. Therefore, many of these projects are plagued by chronic problems on sedimentation and inadequate level of flood protection. It presents a challenge to the local agencies on how to manage these aging infrastructures.

When maintenance and project performance problems emerged from these pre-1975 projects, they have been treated as independent problems unique to the projects. In reality these projects share commonalities in planning and design approaches and subsequent maintenance problem. To identify these common issues, I prepared a case study on the flood control projects built

between 1950s and 1970s in the San Francisco Bay Region. I selected nine projects located in the urban areas with direct outfall to the San Francisco Bay. These projects have the highest vulnerability since they are located in the high flood risk urban areas with significant fluvial sediment influx and tidal flow influence. Table 3.2.1 identified the projects in this case study.

In each project, I document the watershed setting, project justification, cost benefit analysis, and project design data. Then I evaluate each's project's hydraulic performance and channel sediment management. The information used in the case study is based on the original USACE design documents, post-project study reports from the local sponsors, and the available operation and maintenance records for each channel. These data and analysis are synthesized to identify common problems among these flood control projects, and connecting these problems to the USACE planning and design issues and the broader issues with the flood control channel as a structural flood management technology.

In the following sections, I provide an introduction to each project in the case study, organized by the year the project is constructed. In Section 3.3, I summarize the findings on project issues including project justification, hydraulic performance, and sediment management. Tables 3.2a to 3.2i at the end of the chapter list the detail project data.

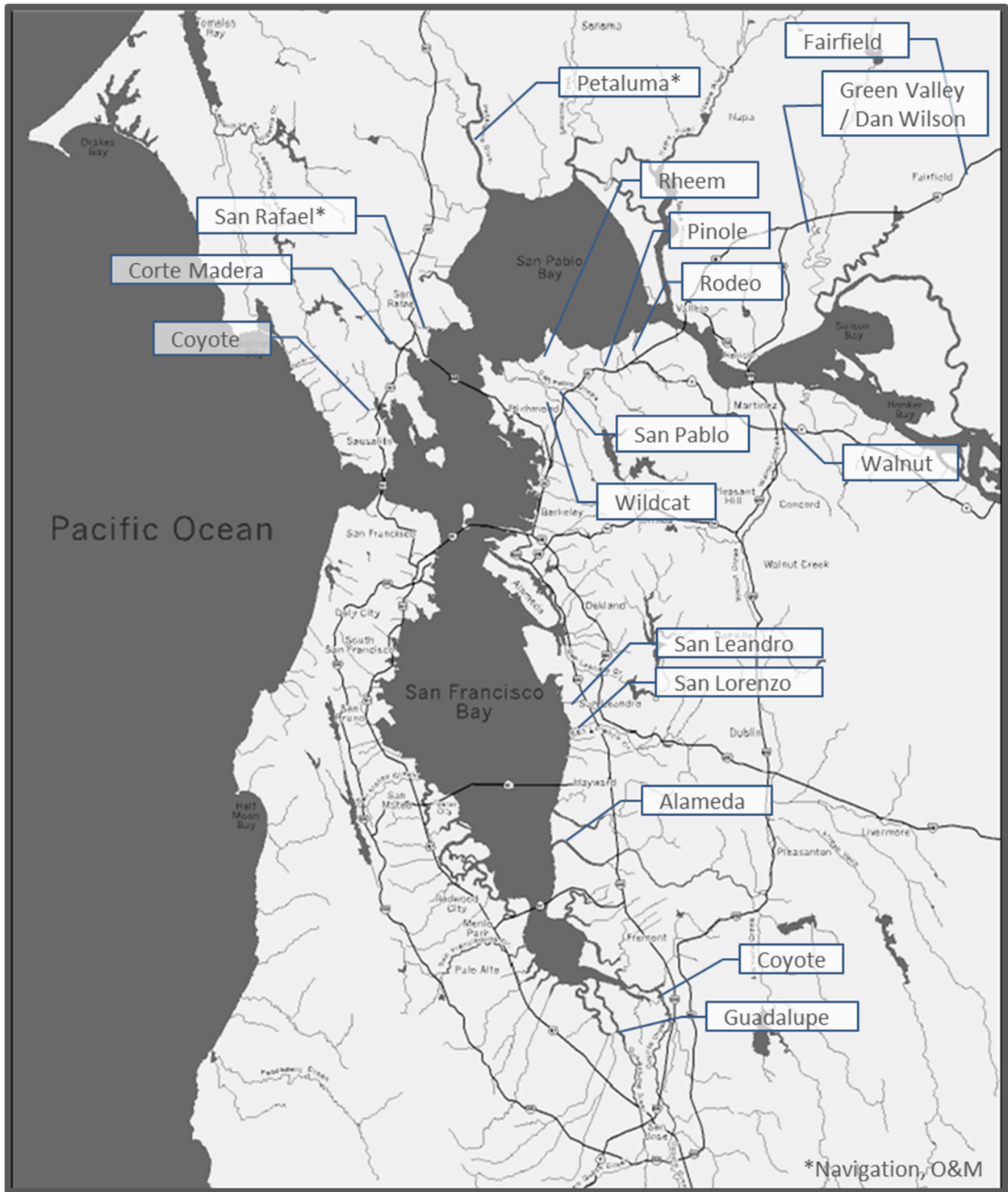


Figure 3.2.1 – Location Map for USACE Flood Control Channels Constructed in the San Francisco Bay Region

3.2.1 Rheem Creek, Contra Costa County (1960)

Watershed Setting

Rheem creek is located in the Cities of San Pablo and Richmond in Contra Costa County. The creek drains a 2 square miles watershed, from the headwater at 300 feet above mean sea level, flowing westward to San Pablo Bay (USACE 1958a). Figure 3.2.1.1 shows the watershed area. The mean annual precipitation is 22.5 inches, the maximum recorded mean annual precipitation is 47.5 inches in 1983 (NHI 2007).

Project Authorization

Contra Costa County Flood Control and Water Conservation District (Contra Costa County) is responsible for Rheem Creek watershed planning and flood management. Flooding in 22 December 1955 and 2 April 1958 prompted Rheem Creek local flood protection project, authorized under Section 205 of the 1948 Flood Control Act in 1959 (USACE 1962c). The project constructed in 1960, include 8000 feet of earth trapezoidal and concrete rectangle channel sections at the downstream section of Rheem Creek between San Pablo Avenue and the outfall at the Bay. Figure 3.2.1.2 shows the project alignment. The project cost \$587,600 in 1959 dollars, with 1.2 cost benefit ratio (USACE 1958a).

Channel Design Flow

The project was designed for 15 year flood protection, at 800 cfs at the Giant Highway. This design level is justified by economic analysis. It is because if the project were designed for higher flow capacity, the project would need to add a costly component to rebuild five bridge crossings, including the BNSF railroad bridge and Highway 40 (Highway 80) box culverts.

In the project General Design Memorandum, the 100 year flow is estimated at 1390 cfs. Since there were no stream or rain gauges in Rheem Creek watershed, the flow estimates were based on long term rainfall data in San Francisco, and the unit hydrograph were based on correlation with data for similar drainage basins in the Pasadena area of Southern California (USACE 1958a). After the project is completed, USGS maintained a stream gauge in Rheem Creek at Giant Highway. The gauge operated from 1960 to 1990. The peak flow recorded is 477 cfs in 1969 (NHI 2007). Contra Costa County prepared a frequency analysis based on the gauge data. The analysis shows that the current 25 year flow is 815 cfs, and the 100 year flow is 1060 cfs (CCCFCWCD). The result shows that the original 15 year design flow of 800 cfs is still valid, while the 100 year flow is reduced by about 300 cfs. Since the project was designed for 15 year flood protection, it does not meet the current 100 year flood protection need.

Sediment Deposition

There is no watershed sediment budget nor channel sediment removal record available. Field observations and aerial images shows in stream vegetation and sediment deposition especially along the downstream earth channel sections. Although there is no flood event reported since the project completed, the channel likely does not have the 800 cfs design capacity (Detjens 2014).

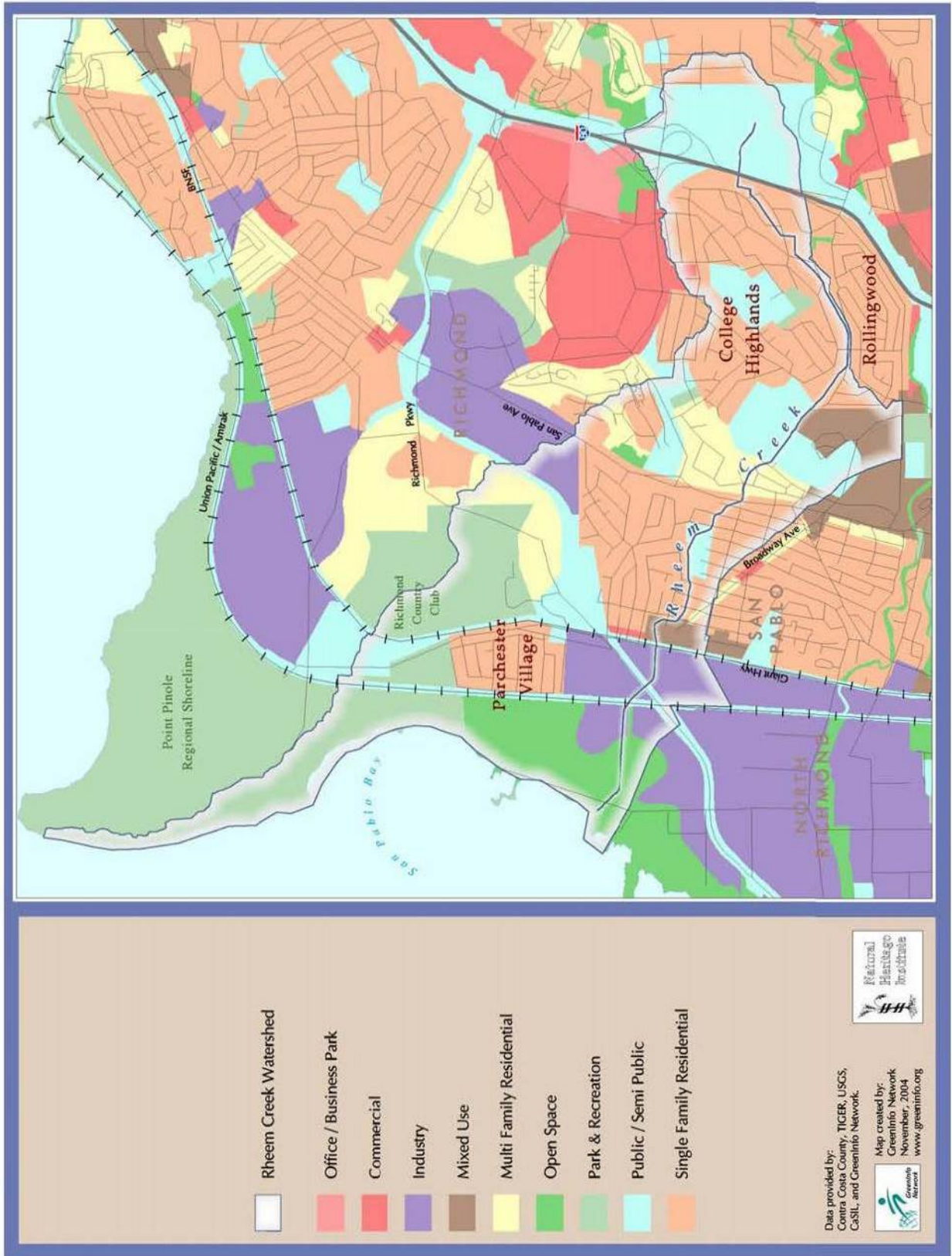


Figure 3.2.1.1: Rheem Creek Watershed (NHI 2007)

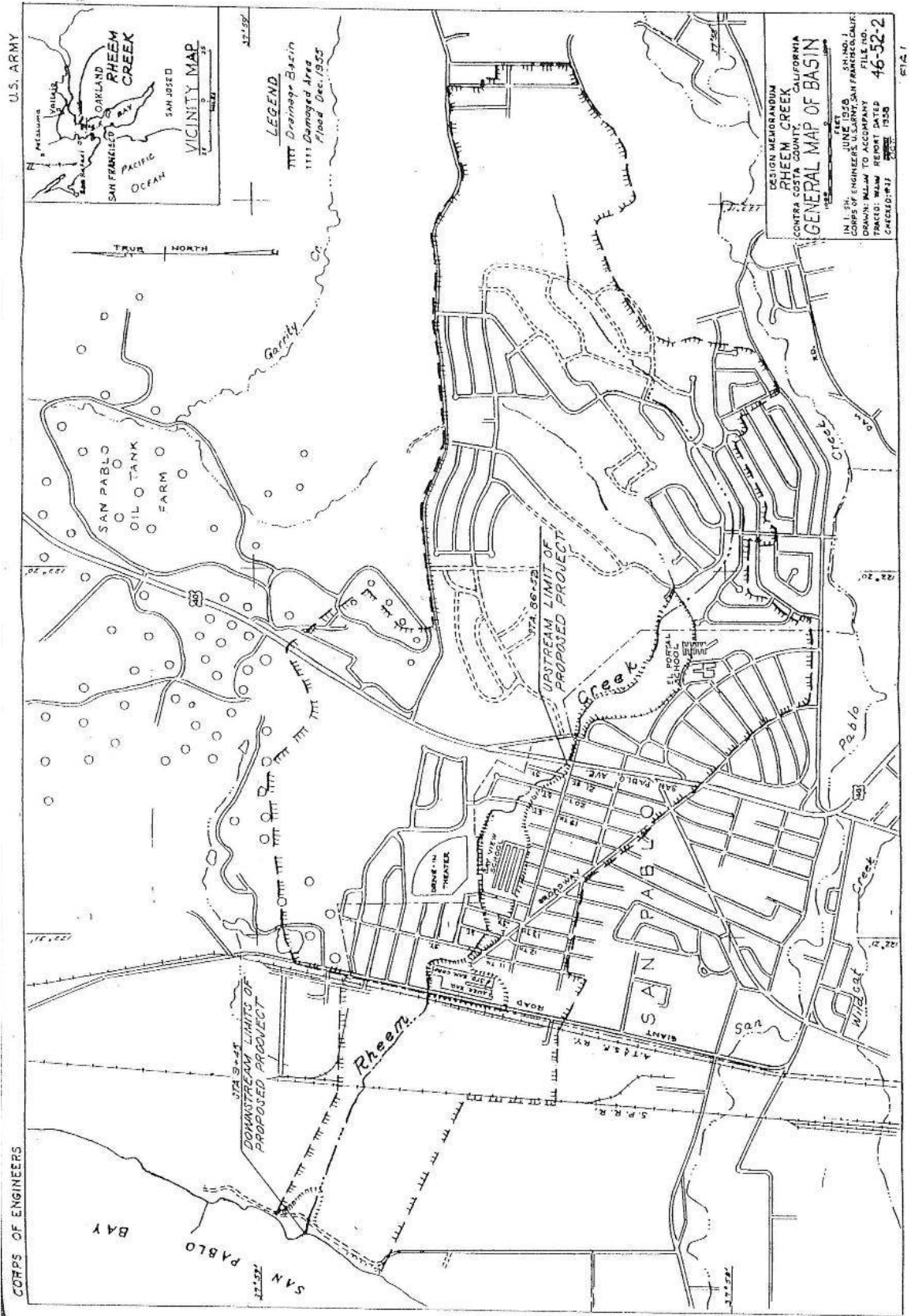


Figure 3.2.1.2a: Rheem Creek Local Flood Protection Project (USACE 1958a)



Figure 3.2.1.2b: Rheem Creek Local Flood Protection Project

Legend: Concrete Channel = Blue Earth Channel = Red

3.2.2 San Lorenzo Creek, Alameda County (1962)

Watershed Setting

San Lorenzo creek is located at the Cities of Castro Valley, Hayward, San Lorenzo and San Leandro in Contra Costa County. The creek drains a 45 square miles watershed, from the Bolinas Creek headwater at 1,850 feet above sea level, flowing westward to San Francisco Bay (USACE 1958b). There are three main tributary watersheds, Crow, Cull, and Palomares Creeks. The 8-mile long San Lorenzo Creek is considered to begin below the confluence of Palomares and Cull Creeks. Figure 3.2.2.1 shows the watershed area. The mean annual precipitation is 24.5 inches (Collins 2006b).

Project Authorization

Alameda County Flood Control and Water Conservation District (Alameda County) is responsible for San Lorenzo Creek watershed planning and flood management. There were 16 major storms between 1905 and 1958, results in 10 flood events. The December 1955 flood was the most severe, with the estimated peak flow of 4,790 cfs. These flood events prompted the San Lorenzo Creek Flood Control project. A flood control project survey was authorized in the 1936 Flood Control Act but the survey recommended no project. The 1943 Flood Control Act authorized USACE to review the survey. The new survey concluded that constructing new reservoir for flood control is not economically feasible, and the survey proposed a channelization only project. The proposed channel project was authorized under Section 203 of the 1954 Flood Control Act (USACE 1958b). The project constructed in 1962, include 27,670 feet of earth, riprap, and concrete channel sections between Foothill Blvd and the outfall at the bay. Figure 3.2.2.2 shows the project alignment. The project cost \$8,319,000 in 1959 dollars, with 1.2 cost benefit ratio (USACE 1958b).

Channel Design Flow

The project was designed for the Standard Project Flood, at 10,400 cfs at Bridge St. As a comparison, the estimated 100 year flow at Bridge Street was 8,000 cfs in the 1950s (USACE 1954), the current 100 year flow estimate is 16,000 cfs (ACFCWCD 2012). The flow increase was due to a combination of urban development and refined flow frequency analysis. After the 1998 wet year when the San Lorenzo Creek was close to overtopping, Alameda County questions the rated design frequency at San Lorenzo Creek. A review on the original flow estimate reveal that the analysis was only based on 14 years of record, hence there is a concern on the flow estimate accuracy. Alameda County updated the analysis with a more extensive data set, and it results in a higher 100 year flow estimate of 16,000 cfs (Saleh 2014). Due to the flow increase, currently the channel design capacity is below the existing 100-year flow.

Sediment Deposition

Around the same time the flood control project was constructed, Cull Canyon and Don Castro Reservoirs were constructed in San Lorenzo Creek watershed. Cull Canyon Reservoir, a 55 feet high earth dam built in 1963, provides 295 acre-feet of storage capacity (URS 2003a). The Don

Castro Reservoir was built in 1964 with a capacity of 380 acre-feet (URS 2003b). Both reservoirs were funded by the Davis Grunsky Act in 1960. The original intent for both reservoirs was to provide recreation areas, and also temporary water storage for groundwater recharge. Over time, groundwater recharge was proved to be infeasible, so currently both reservoirs only provide recreation benefits, and flood control function.

San Lorenzo Creek currently implementing a two-tier flood management strategy. While it was not the original design intent, the upstream reservoirs provide flow and sediment detention. The downstream creek channelization designed for efficient flow conveyance. The channel benefits from the upstream reservoir sediment trap, so the channel receives reduced watershed sediment influx. Although there were a number of sediment removal works since the reservoirs are constructed, a study in 2003 shows both reservoirs were sediment filled with limited available capacity (URS 2003a, URS 2003b), as shown in Table 3.2.2.1. The study estimated the sediment removal work to restore the reservoirs capacity would cost \$36 million in 2003 dollars. Alameda County is currently considering bypass systems to control sedimentation issues at the reservoirs. Sediment transport studies (DHI 2008) indicated that retrofitting the existing channel is difficult, if not impossible to provide self-scouring under a range of design flow events, and to maintain the design maximum water surface elevation in the creek.

Reservoir	Original Capacity	Capacity in 2003	Capacity Remain
	acre foot	acre foot	%
Cull	295	68	23%
Don Castro	380	72	19%

Cull Canyon and Don Castro Reservoirs disconnected most of the upper watershed from San Lorenzo Creek. Since both reservoirs trapped most of sediment supply from their tributaries except suspended load when the reservoirs are full, sediment supply to San Lorenzo Creek is mainly from Crow Creek watershed. The Crow Creek watershed consists of an 8-mile long alluvial valley, predominately agriculture land with some residential development at the southern ridge of the canyon. It covers about 23% of the San Lorenzo Creek watershed. Sediment influx at Crow Creek, 3,870 cy/mi², is much higher than the Cull Canyon Reservoir, 2,270 cy/mi², and Don Castro Reservoir, 590 cy/mi² (Collins 2006b). A recent hydraulic and sediment modeling analysis (DHI 2008) estimated 27,259 cy/yr sediment supply and 4370 cy/yr sediment trap in the flood control channel, at 16% trap rate. This estimate only accounts for fluvial sediment supply. Although tidal sediment data is not available, sediment deposition is observed at the earth channel in the tidal reach adjacent to the Bay. Alameda County has sediment removal activities in the flood control channel but maintenance record is not available (Saleh 2014).



Figure 3.2.2.1: San Lorenzo Creek and Tributary Watersheds (Alameda County Public Works Agency 2006)

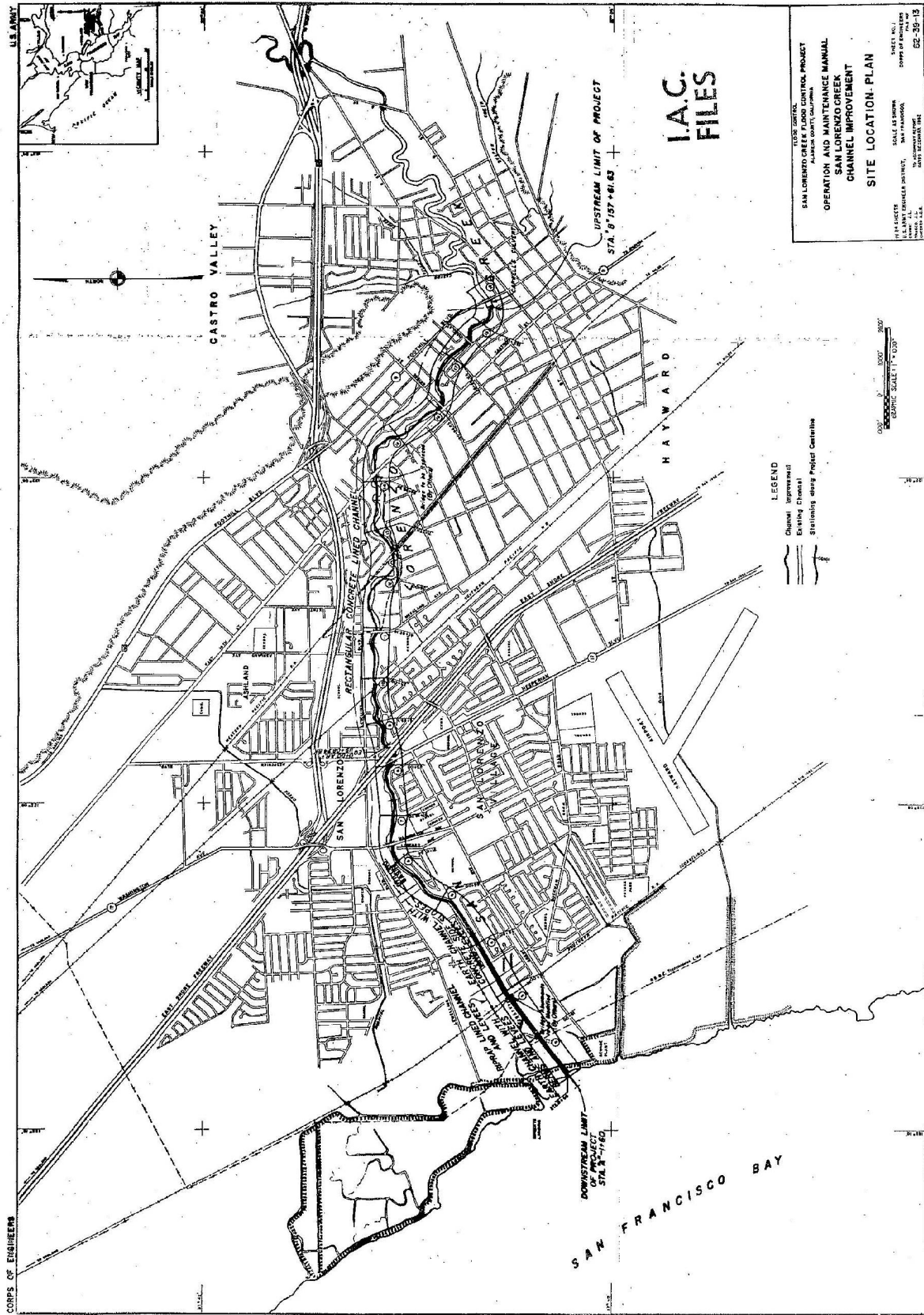


Figure 3.2.2.2a: San Lorenzo Creek Flood Control Project (USACE 1963b)

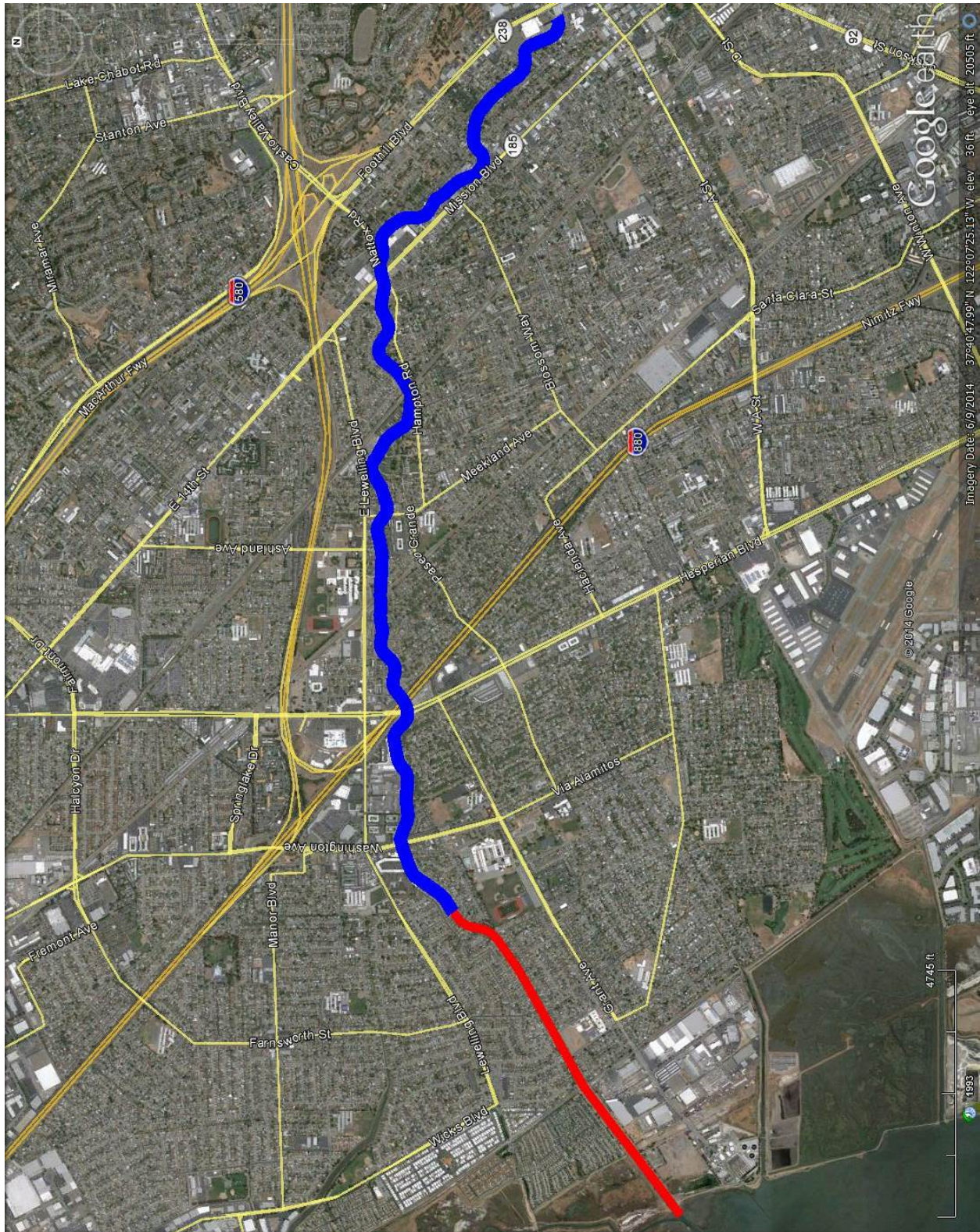


Figure 3.2.2.2b: San Lorenzo Creek Flood Control Project

Legend: Concrete Channel = Blue Earth Channel = Red

3.2.3 Walnut Creek, Contra Costa County (1965)

Watershed Setting

The drainage basin of Walnut Creek includes lands of eight cities, Walnut Creek, Lafayette, Pleasant Hill, Danville, and parts of Concord, Martinez, San Ramon, Moraga, and Orinda, all in Contra Costa County. The creek drains a 146 square miles watershed, from the Pine Creek headwater near the summit of Mount Diablo at 3,849 feet above mean sea level, flowing northward to San Pablo Bay (CCCFCWCD 2003). Walnut Creek is formed by the confluence of San Ramon Creek and Las Trampas Creek, and flows in a northerly direction toward Suisun Bay. San Ramon Creek tributaries include Sycamore, Green Valley, and Sans Crainte creeks. Tributaries to Las Trampas Creek include Lafayette, Happy Valley, Grizzly, Reliez, and Tice creeks. Downstream of the Las Trampas-San Ramon confluence, Walnut Creek is joined by Pine and Galindo creeks from the east and by Grayson-Murderer's and Pacheco creeks from the west. About 1.9 miles upstream of the bay, Walnut Creek joins with Pacheco Creek. While the official name of the stream flowing into the bay is Pacheco Creek, most of the drainage area belongs to the Walnut Creek basin (USACE 2008). Figure 3.2.3.1 shows the watershed area. The mean annual precipitation is 21 inches (CCCFCWCD 2003), but it varies from less than 15 inches at the mouth of Pacheco Creek, to about 30 inches in the Lafayette Creek headwaters in the Oakland Hills at the western end of the watershed (USACE 2008).

Project Authorization

Contra Costa County Flood Control and Water Conservation District (Contra Costa County) is responsible for Walnut Creek watershed planning and flood management. Pre-1950s flood events plus flooding in December 1955 and April 1958 prompted the Lower Walnut Creek Flood Control Project, authorized under the Flood Control Act in 1960 (USACE 1963a). The project constructed in 1965, include 14.1 miles of earth and concrete channel and levee sections at the downstream section of Walnut Creek between Rudgear Road and the outfall at the bay. Figure 3.2.3.2 shows the project alignment. The project cost \$31,500,000 in 1964 dollars, with 1.3 cost benefit ratio (USACE 1964a).

Channel Design Flow

The project was designed for the Standard Project Flood, equivalent to the 100 year flood, at 25,000 cfs at the Bay. Numerous small floods occurred between the project constructed in 1965 and 1981, and since then, major floods affected the area, in 1982, 1983, 1986, 1995, 1997, 1998, and 2002. The January 1997 flood, an estimated 15 year event, caused damage to about 100 homes in Pleasant Hill. Rainfall in February 1998 and December 2002 caused problems with mudslides and bank erosion (USACE 2008). In 2008 USACE conducted a general reevaluation on the Lower Walnut Creek Flood Control project. The updated hydrology analysis based on the latest gauge data, plus the updated landuse reflecting higher urbanization and impervious percentage. The analysis estimated the 100 year design flow at 31,200 cfs. The current channel flow capacity is estimated at 20,000 cfs (RDG 2013). The Lower Walnut Creek Flood Control project not only does not have 100 year flow capacity, but also could not maintain the original design capacity at 25,000 cfs.

Sediment Deposition

The Lower Walnut Creek Flood Control project was designed with a flat bottom and no low-flow channel. The project has been plagued with sediment issues. After construction was completed in 1965, there was a rapid sediment deposition in the channel. In response to Contra Costa County concerns, USGS completed a sediment study in 1972, concluded that sediment discharge between 1965 and 1970 was larger than the historic average due to larger peak flow and average stream flow (USGS 1972). In response to the USGS study, USACE revised the sediment deposition estimate. In the project General Design Memorandum, it was estimated the flood control channel trap 36,000 cy/yr of 180,000 cy/yr sediment supply, at 20% trap rate (USACE 1964a). The revision estimated the flood control channel trap 160,000 cy/yr of 250,000 cy/yr sediment supply, at 65% trap rate (USACE 1972).

In 1973, the Corps' contractor dredged the channel from the mouth to the BNSF Railroad Bridge. Approximately 850,000 cy of material was removed, with USACE share 1/3 of the sediment removal cost. Between 1986 and 1989, the District removed approximately 276,000 cy of sediment from the non-tidally influenced portions of the channel between Clayton Valley Drain and Drop Structure #1. In the early 1990s, the District estimated that 650,000 cy of sediment had accumulated in the area dredged by USACE in 1973. After significant effort to secure regulatory permits for sediment removal, Contra Costa County determined that the dredging work was unlikely to be permitted due to significant environmental impacts, with costly mitigation that far exceeded Contra Costa County's financial resources. Therefore, Contra Costa County focus shifted again to sediment removal farther up in the watershed, where habitat and species impacts were much less. In 1993 and 1995, over 76,000 cy of sediment was removed from Walnut Creek between Pine Creek and Drop Structure #1. And in the summer of 2006, an additional 25,500 cy of sediment was removed from Walnut Creek in selected upland areas between Concord Avenue and Drop Structure #1 (CCCFCWCD 2007).

In early 2007, the USACE released a nationwide evaluation of flood control systems and included Lower Walnut Creek in the deficient category. USACE required repairs within one year in order to remain eligible for the USACE PL 84-99 disaster assistance program (CCCFCWCD 2007). PL 84-99, per Section 5 of the FCA of 1941, authorized the USACE to emergency preparedness and disaster support for flood events, and provide financial assistance to repair the projects that were damaged by flood (U.S. Congress 1941).

As a result, Contra Costa County implemented the Interim Protection Measures Project and removed 200,000 cy of sediment between BNSF Railroad and Clayton Valley Drain in 2008. The project designated the lowest 2.5 miles of the channel as having the highest biological value, and USACE agreed to temporarily suspend oversight of this area while the USACE's General Reevaluation Report project proceeded to implement a long-term solution.

This suspension of the USACE oversight is important for the Lower Walnut Creek project to remain eligible for the PL 84-99. Contra Costa County cannot secure the permits to dredge the lower reach, so it was not meeting the USACE maintenance requirements. This non-compliance can lead to the entire project to be ineligible for the PL 84-99. Since the lowest reach was suspended from the USACE oversight, no dredging is immediately needed, the habitat in the

lowest reach can temporarily remain, while preserving the eligibility of the other 20 miles of channel for the PL 84-99 (CCCFCWCD 2013).

In the meantime, Contra Costa County continue to evaluate options to sustainably maintain the lowest 2.5 miles of Lower Walnut Creek, to balance ecological function and flood protection benefits, and meet the USACE maintenance requirements. The County concluded that there is a significant permitting hurdles to continue the needed dredging operation at the lower reach. In addition, since the lower reach is away from the urban areas, if the lower reach overtops during a flood event and floods the adjacent area, the residual risk is relatively low. Hence, this lowest 2.5 miles reach does not significantly benefits from the PL 84-99. As a result, the County selected the “Selective Deauthorization” approach. This approach is to remove the lowest 2.5 miles reach from the USACE Lower Walnut Creek flood control project authorization (CCCFCWCD 2012). In this case, the low reach will no longer subject to the USACE maintenance requirements. Contra Costa County can redesign the lower reach with different design frequency, as long as it does not impact the capacity of the upstream project, which still under the USACE authorization and eligible for the PL 84-99. The lower reach was deauthorized in The Water Resources Reform & Development Act of 2014 (US Congress 2014), and Contra Costa County is currently evaluate options for the lower reach improvements.

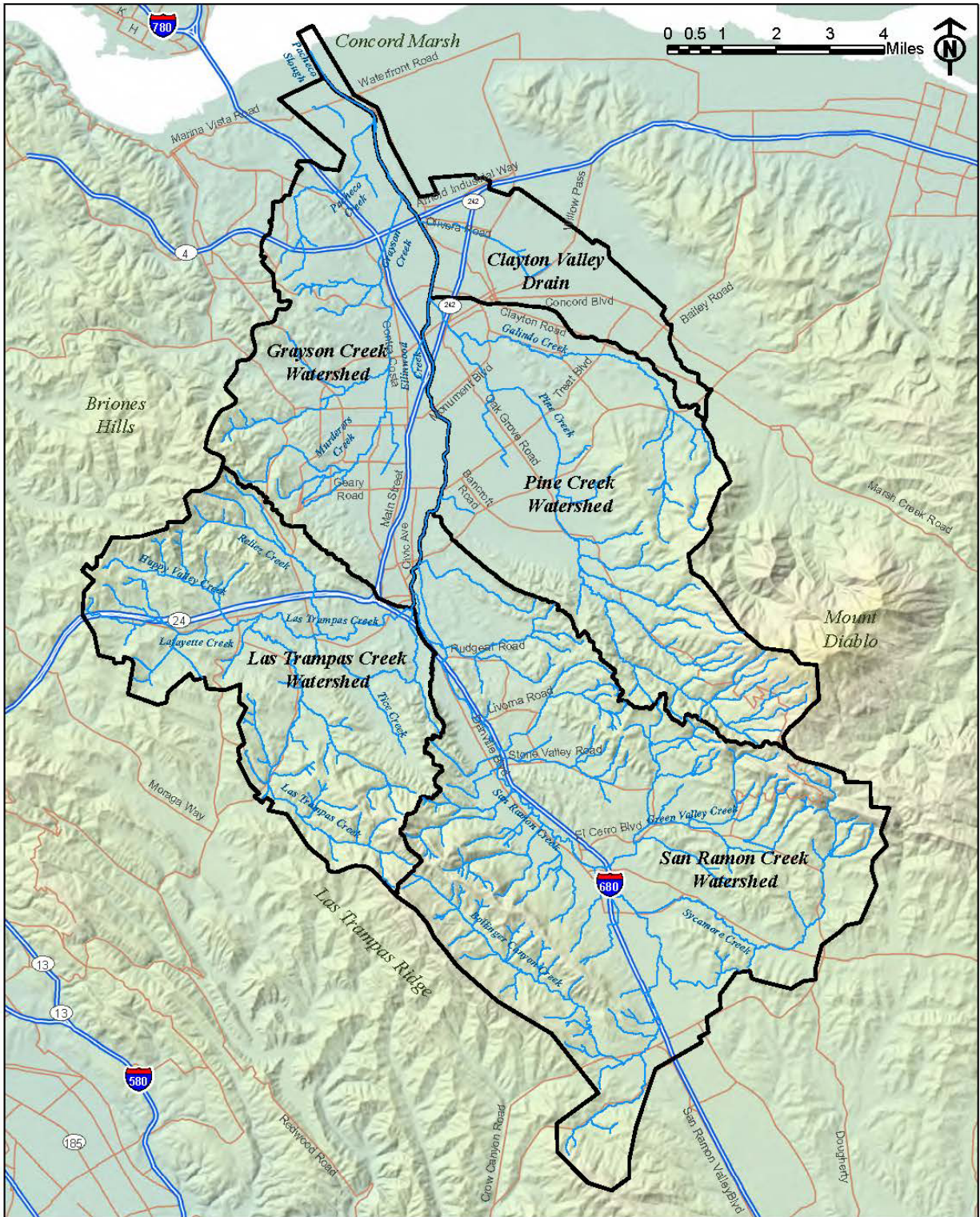


Figure 3.2.3.1: Walnut Creek Watershed (RDG 2013)

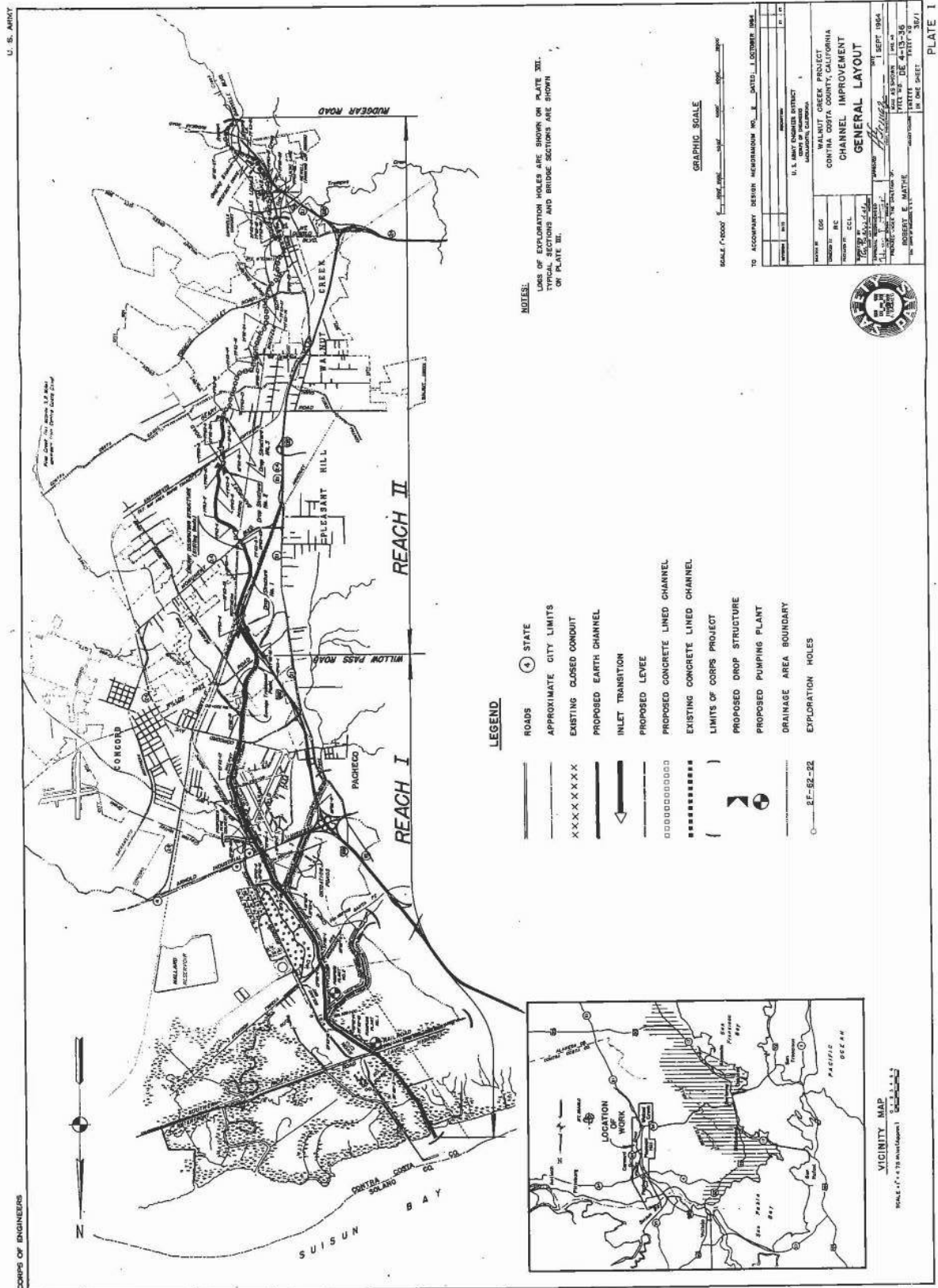


Figure 3.2.3.2a: Walnut Creek Flood Control Project (USACE 1964a)

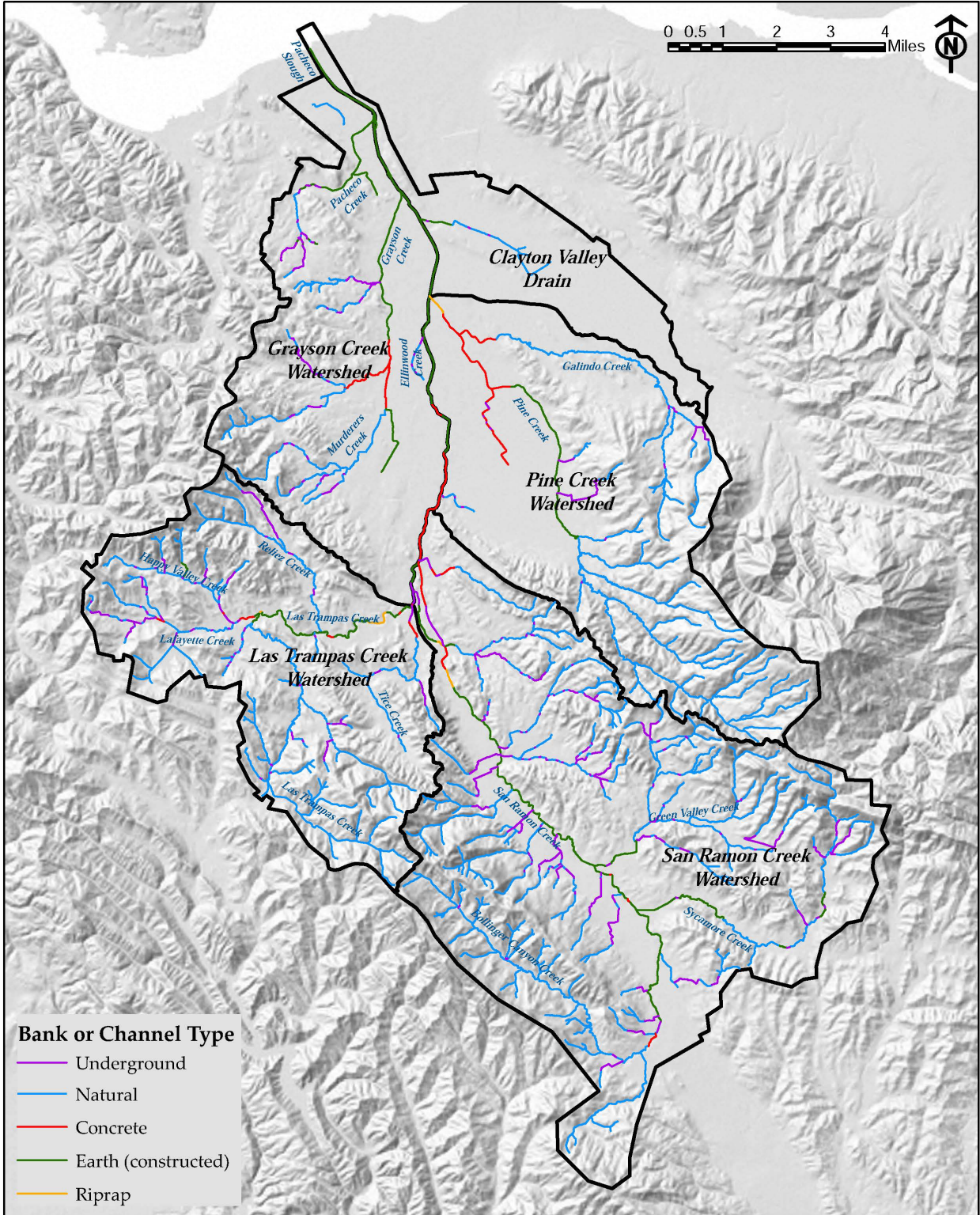


Figure 3.2.3.2b: Walnut Creek Flood Control Project (RDG 2013)

3.2.4 Coyote Creek, Marin County (1965)

Watershed Setting

Coyote creek is located in the Tamalpais Valley in Marin County. The creek drains a 3.5 square miles watershed, from the headwater in Oakwood Valley at 1000 feet above mean sea level, flowing northeastward to Richardson Bay (ASRP 1973). Figure 3.2.4.1 shows the watershed area. The mean annual precipitation is 35 inches, but it varies from 28 inches at the Bay to over 40 inches on the ridge separating Coyote Creek from the drainage to the west (USACE 1959).

Project Authorization

Marin County Flood Control and Water Conservation District (Marin County) is responsible for Coyote Creek watershed planning and flood management. Flooding occurred in January 1952, December 1955, January 1956, February 1958, and April 1958. The January 1956 flood has the highest magnitude, estimated at 800 cfs (USACE 1959). These flood events prompted the Coyote Creek local flood protection project, authorized under Section 205 of the 1948 Flood Control Act in 1963 (USACE 1965). The project was constructed in 1965, and included 4,500 feet of earth channel from Richardson Bay to Ross Drive, continue with 2,940 feet of concrete channel upstream to Maple Street. Figure 3.2.4.2 shows the project alignment. The project cost \$863,000 in 1959 dollars, with 1.2 cost benefit ratio (USACE 1959).

Channel Design Flow

The reconnaissance phase project design was based on Standard Project Flood, but the design required the concrete channel to extend further downstream into the area with deep Bay mud layer. The construction cost to provide stable concrete channel at the extension was considered not economically justifiable. Therefore, the project was designed for 20 year flood protection, at 1,750 cfs at Highway 1 plus 1 foot freeboard in fluvial zone and 0.5 foot freeboard in tidal zone (USACE 1959). The project Detailed Project Report noted that without the freeboard, the channel has close to 100 year flow capacity, estimated at 2,350 cfs. The current 20 year flow estimate is increased to 1,952 cfs at the Tennessee Valley confluence (PWA 2005). With no freeboard, the channel capacity is estimated at 1,830 cfs at the Tennessee Valley confluence. It shows that the channel no longer has 20 year flood protection capacity, nor its original design flow capacity.

Sediment Deposition

Marin County periodically surveys the channel and performs maintenance dredging of various reaches to remove accumulated sediment and restore the channel to its design condition. Marin County typically dredges the upper reaches more frequently than the lower Reach. The upper reaches of Coyote Creek, upstream of the Highway 1 Bridge, have been dredged every 7 to 10 years. The lower reach downstream of the Highway 1 Bridge has only been dredged twice since it was constructed in 1964. The upper and lower reaches of Coyote Creek were last dredged in 2003 and 1991 respectively. The outboard channel, from the mouth of the creek into Richardson Bay, has not been dredged since it was initially excavated by USACE in 1965. Table 3.2.4.1

summarizes the sediment deposition data (ESA PWA 2012). As demonstrated by dredging records, sediment deposition is a more significant problem upstream of Highway 1, and the need for dredging in that reach will continue (PWA 2012).

Table 3.2.4.1: Coyote Creek Sedimentation Volume Data				
Reach	Data Point	Years since prior dredging	Deposition Volume	
-	Yr	Yr	cy	cy/yr
Upper (concrete channel)	1998	33	1200	36
Upper (concrete channel)	2003	5	456	91
			Weighted Average	44
Middle (earth channel)	1974	9	4878	542
Middle (earth channel)	1991	17	5048	297
Middle (earth channel)	2003	12	7000	583
			Weighted Average	445
Lower	1974	9	2333	259
Lower	1991	17	4556	268
Lower	2006	15	4296	286
			Weighted Average	273
Outboard	2008	43	2781	65

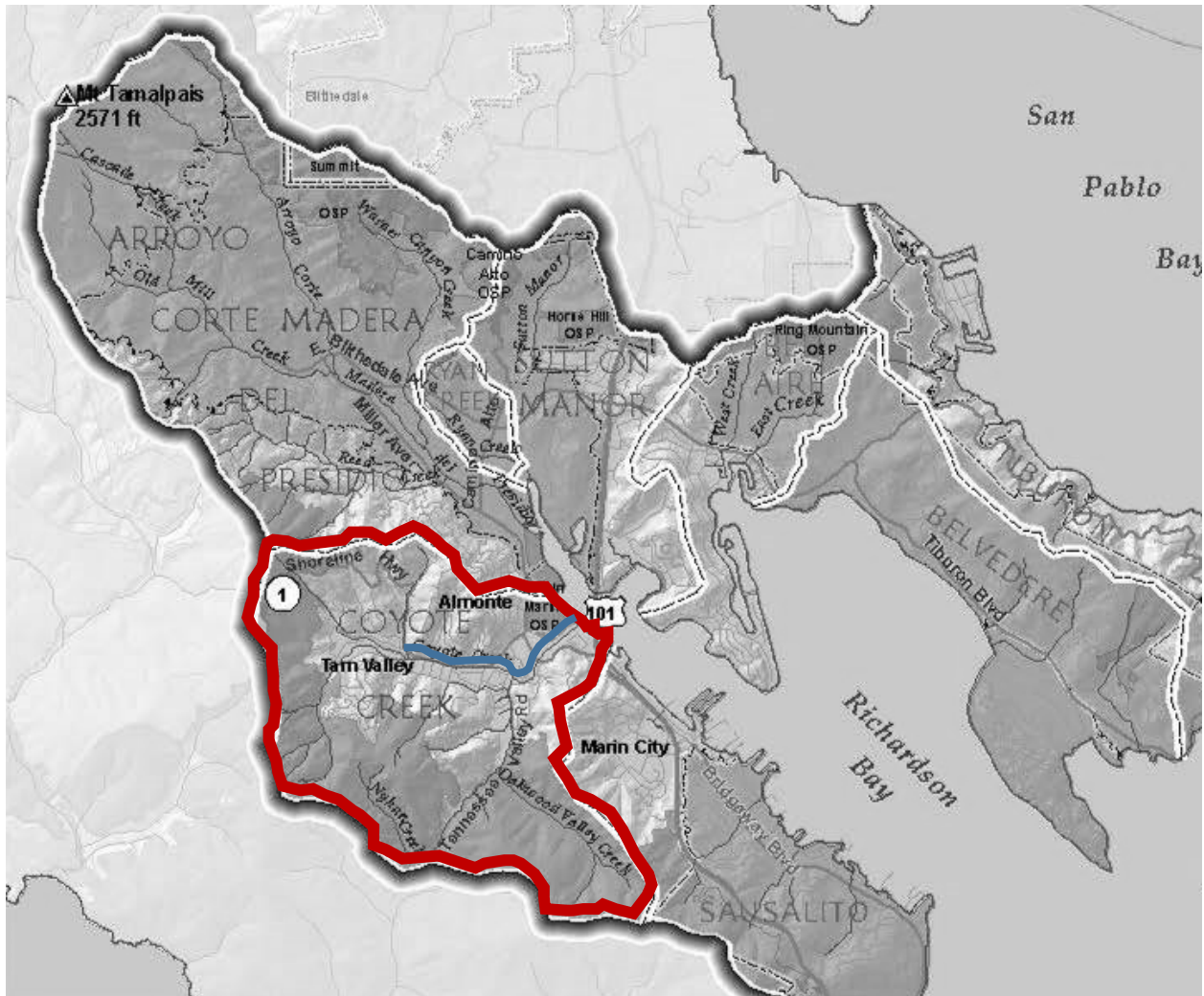


Figure 3.2.4.1: Coyote Creek Watershed (County of Marin Dept. of Public Works 2014a)

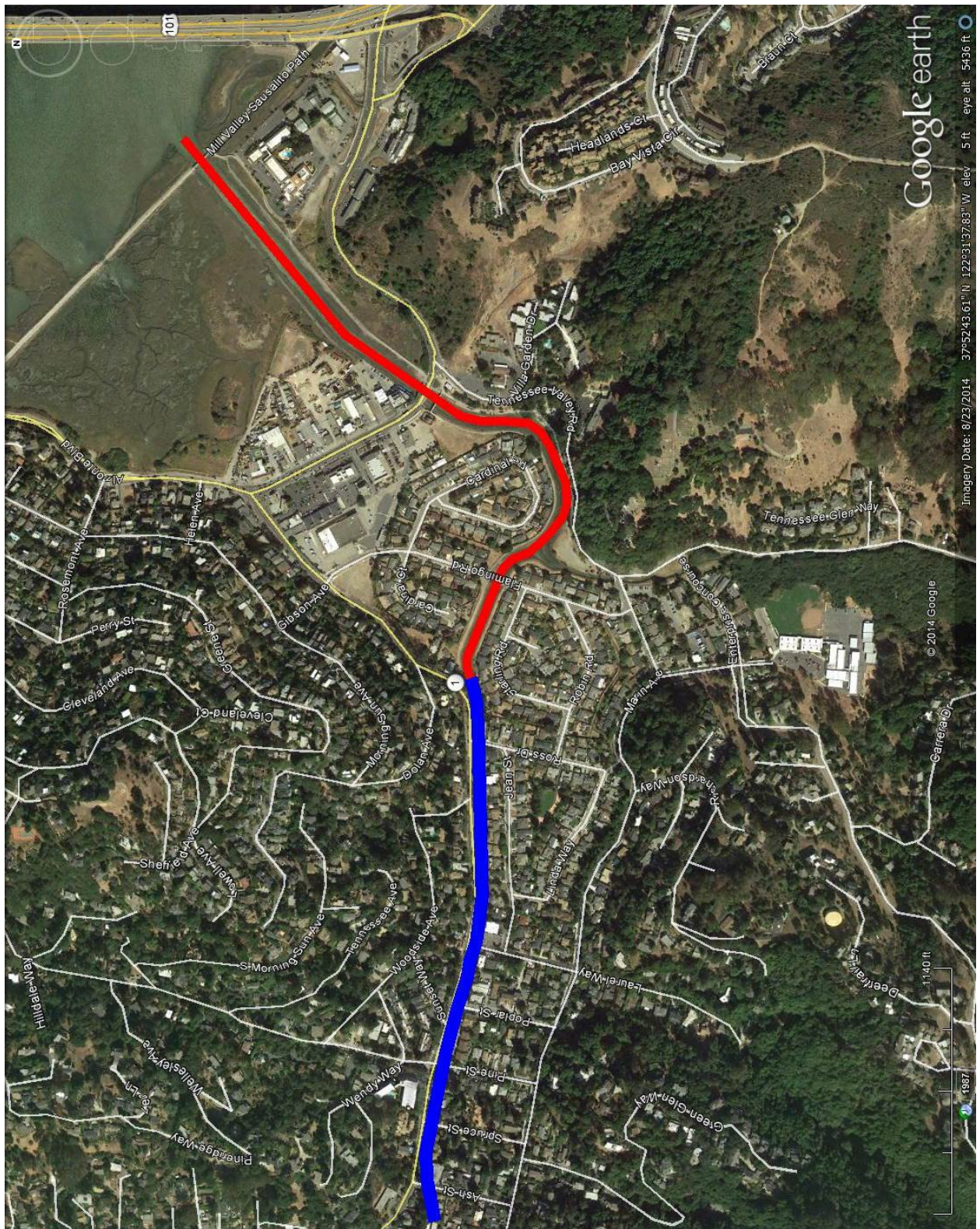


Figure 3.2.4.2b: Coyote Creek Local Flood Protection Project

Legend: Concrete Channel = Blue Earth Channel = Red

3.2.5 Pinole Creek, Contra Costa County (1966)

Watershed Setting

Pinole creek is located along the southwestern slopes of San Francisco East Bay Hills through the Cities of Pinole and Hercules in Contra Costa County. The creek drains a 20 square miles watershed, from the private ranchlands in the upper watershed with the highest elevation of 1,240 feet, thorough the valley owned by the East Bay Municipal Utility District (EBMUD), flowing northwestward crossing Highway 40 (Highway 80) to San Pablo Bay (CCCFCWCD 2003). Figure 3.2.5.1 shows the watershed area. The mean annual precipitation is 23 inches, but it varies from 19 inches at the Bay to over 25 inches in the hills (CCCFCWCD 1977).

Project Authorization

Contra Costa County Flood Control and Water Conservation District (Contra Costa County) is responsible for Pinole Creek watershed planning and flood management. Flooding has reported in 1940, 1942, 1952, 1955, and 1958. The April 1958 flooding was most severe. In the business district in the City of Pinole, overflow flooding reach 2.5 feet deep. These flood events prompted the Pinole Creek local flood protection project, authorized under Section 205 of the 1948 Flood Control Act in 1963 (USACE 1966b). The project constructed in 1966. The 8,000 feet improvements from Highway 40 (Highway 80) to the Bay outfall consists of mostly earth channel, except the two short sections of concrete channels. These two sections of concrete channels, about 300 feet and 500 feet in length, are constructed at Tennent Avenue and San Pablo Avenue bridges. The concrete channels construction was intended to increase the flow velocity to approximately 13 to 17 fps, under super critical flow condition, to eliminate the need for bridge replacements (USACE 1962a). Figure 3.2.5.2 shows the project alignment. The project cost \$585,000 in 1963 dollars, with 1.1 cost benefit ratio (USACE 1962a).

Channel Design Flow

The project was designed for 50 year flood protection, at 2,600 cfs. Selection of 50 year flood protection was based on the cost benefit analysis, and the assumption that if the EBMUD proposed reservoir at the upstream of the project were constructed, it could reduce the Standard Project Flood at the bay from 3,700 cfs to 2,200 cfs, which is below the channel's design capacity. Since the proposed reservoir never been built, the project rated design level is still maintain at 50 year. Estimates for current channel capacity is not available. However, since the 100 year flow at the project is increased from 3,000 cfs (USACE 1962a) to 4,080 cfs (Michael Love & Associates 2012), it is likely that the project can no longer provide 50 year flood protection.

Sediment Deposition

Pinole Creek flood control channel has low sediment trap rate at around 2% (SFEI 2005). The watershed sediment supply estimate is 28,367 cy/year but only 445 cy/year is deposited in the channel. There were minimal sediment removed from the channels, estimated at around 7 cy/yr. The SFEI 2005 study notes that:

“In the flood control channel, higher flood velocities would scour out the channel, removing sediment accumulated during previous dry years, and might even cause net erosion of the estuarine basin. We suggest that during these wetter years, although the creek would likely be carrying a greater sediment load, the large flood flows would be sufficient to flush the sediment through the flood control channel and into the Bay. Given the capacity of the flood control channel, or the maximum volume of water it can contain, a 2-year return interval flood would completely flush channel water in about 13 minutes.” (SFEI 2005)

However, as shown in Figure 3.2.5.3, there is channel aggradation at the lower tidal reach. Since the channel was constructed with steep gradient to an outfall elevation below the bed elevation at the Bay, the channel profile adjusted overtime to reduce the gradient. While the gradient reduction and channel sediment deposition reduces channel capacity, dredging tidal reach is not a sustainable solution since sediment deposition will reoccur rapidly in the tidal reach.

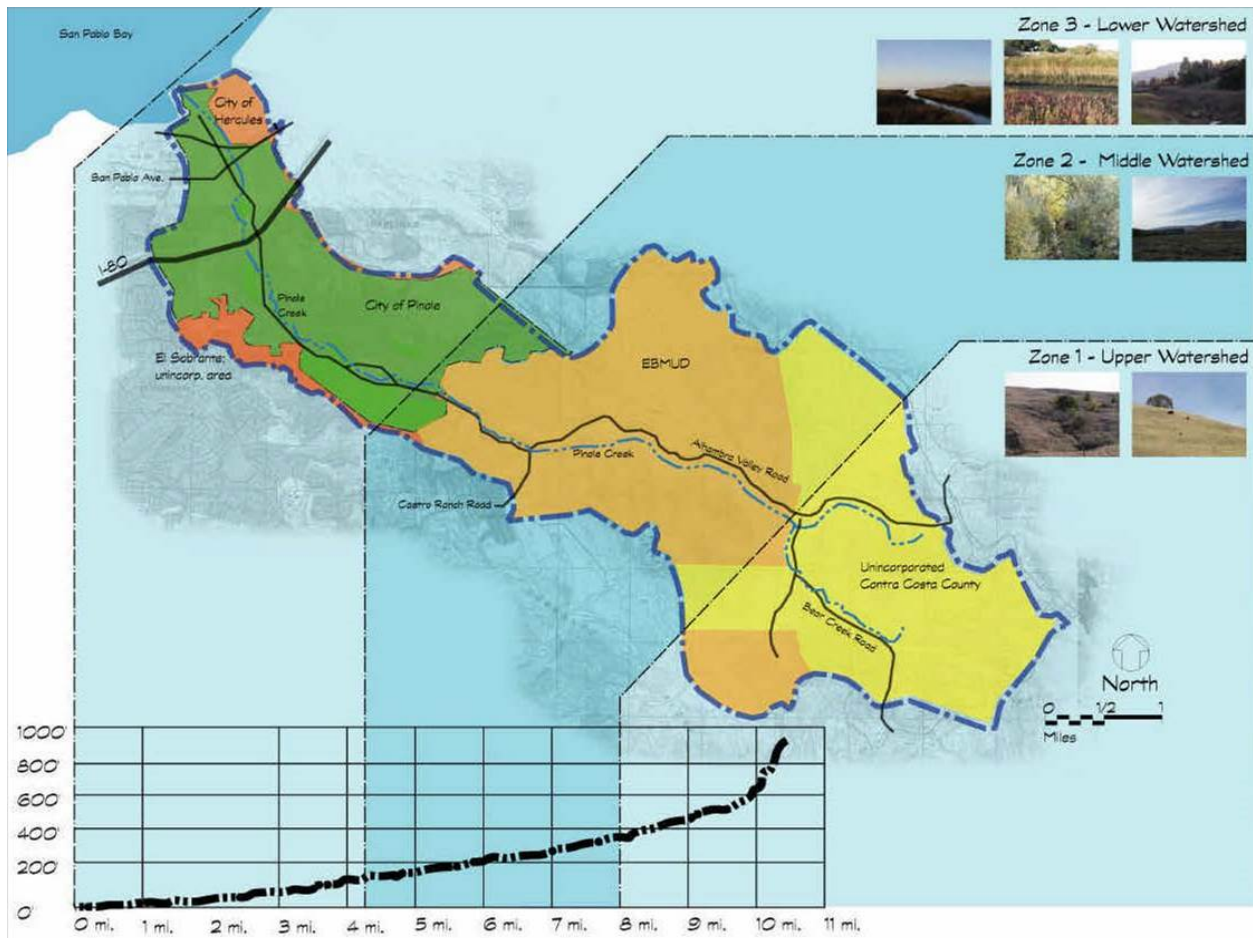


Figure 3.2.5.1: Pinole Creek Watershed (Urban Creeks Council 2004)

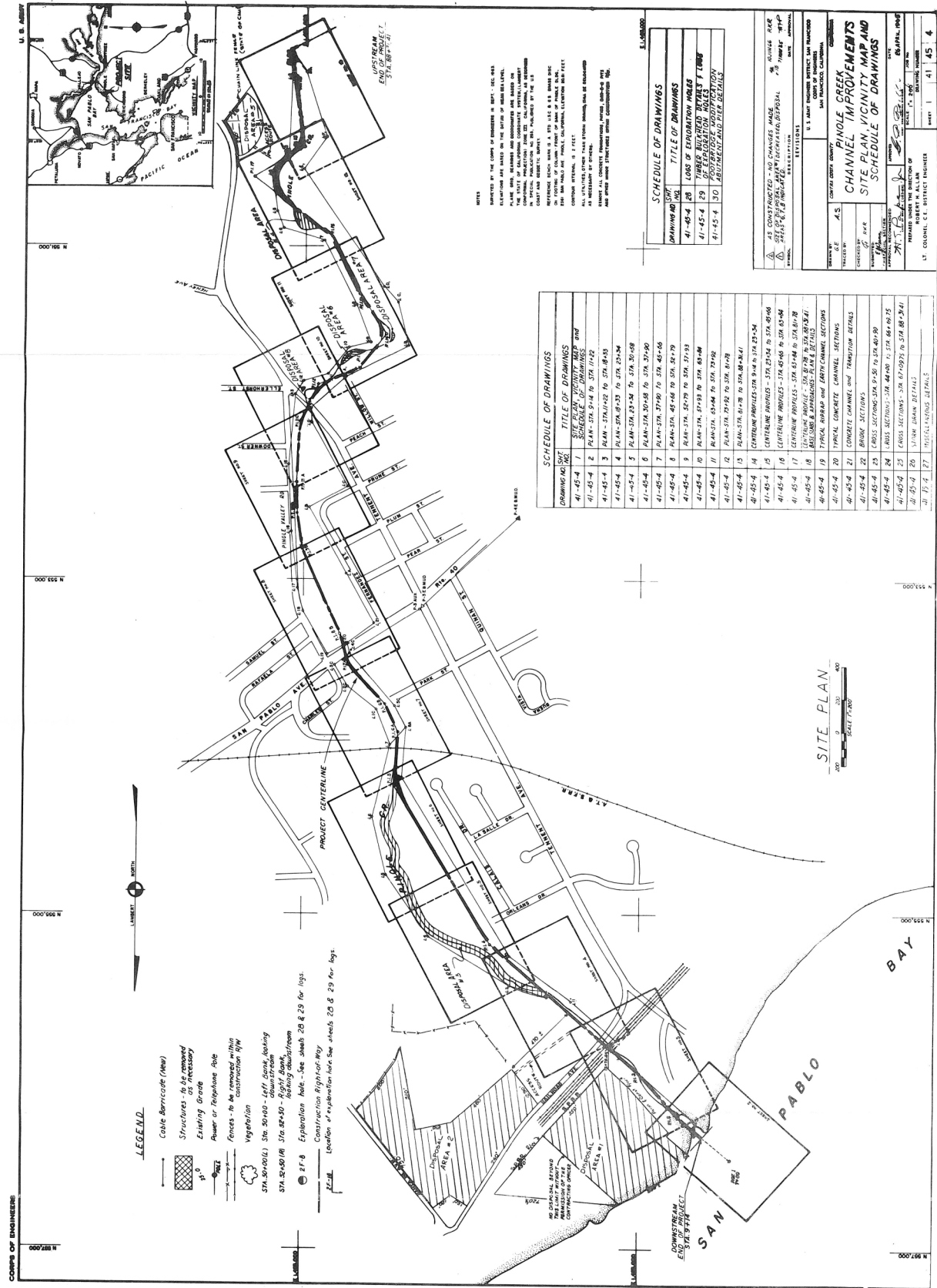


Figure 3.2.5.2a: Pinole Creek Local Flood Protection Project (USACE 1966b)

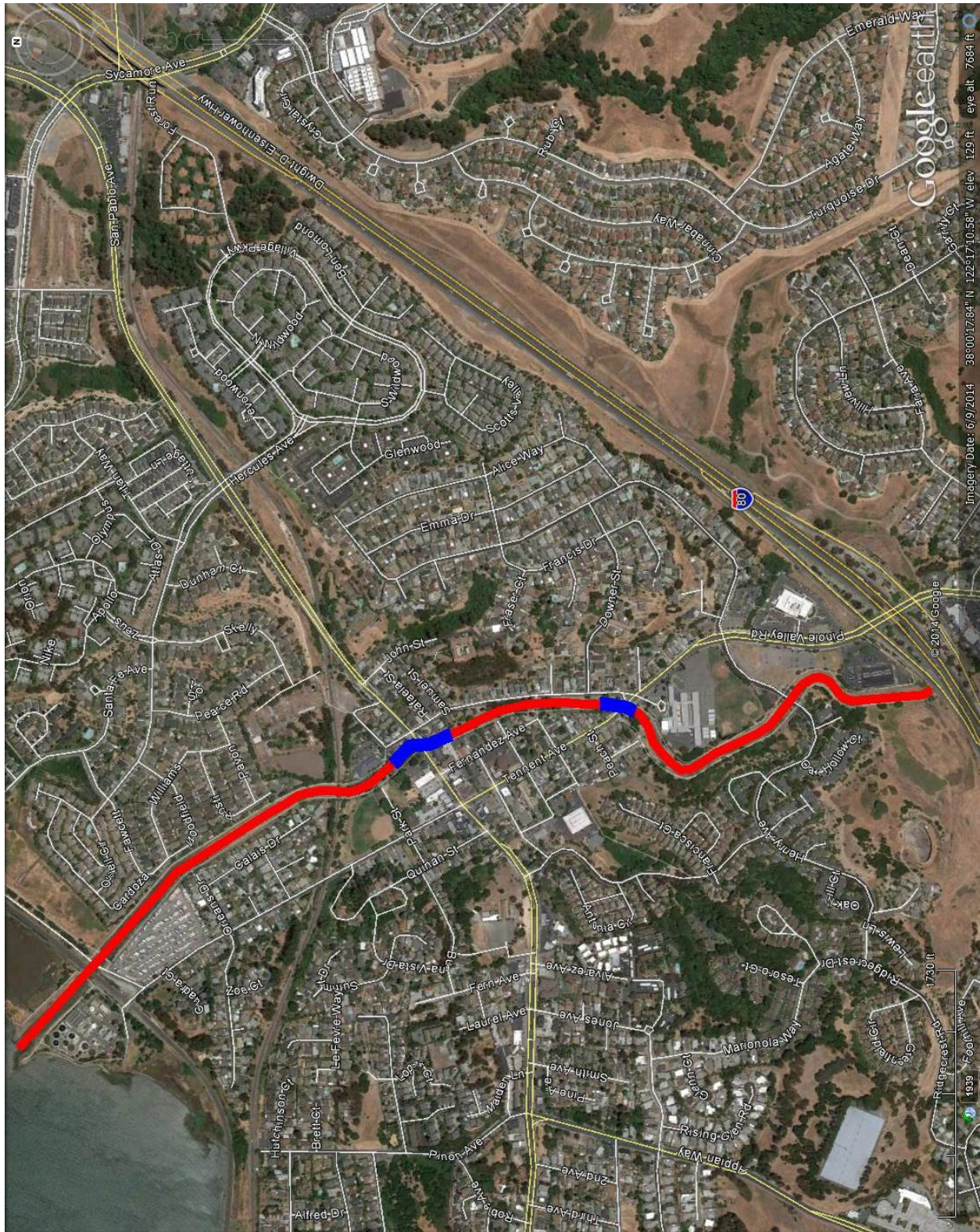


Figure 3.2.5.2b: Pinole Creek Local Flood Protection Project

Legend: Concrete Channel = Blue Earth Channel = Red

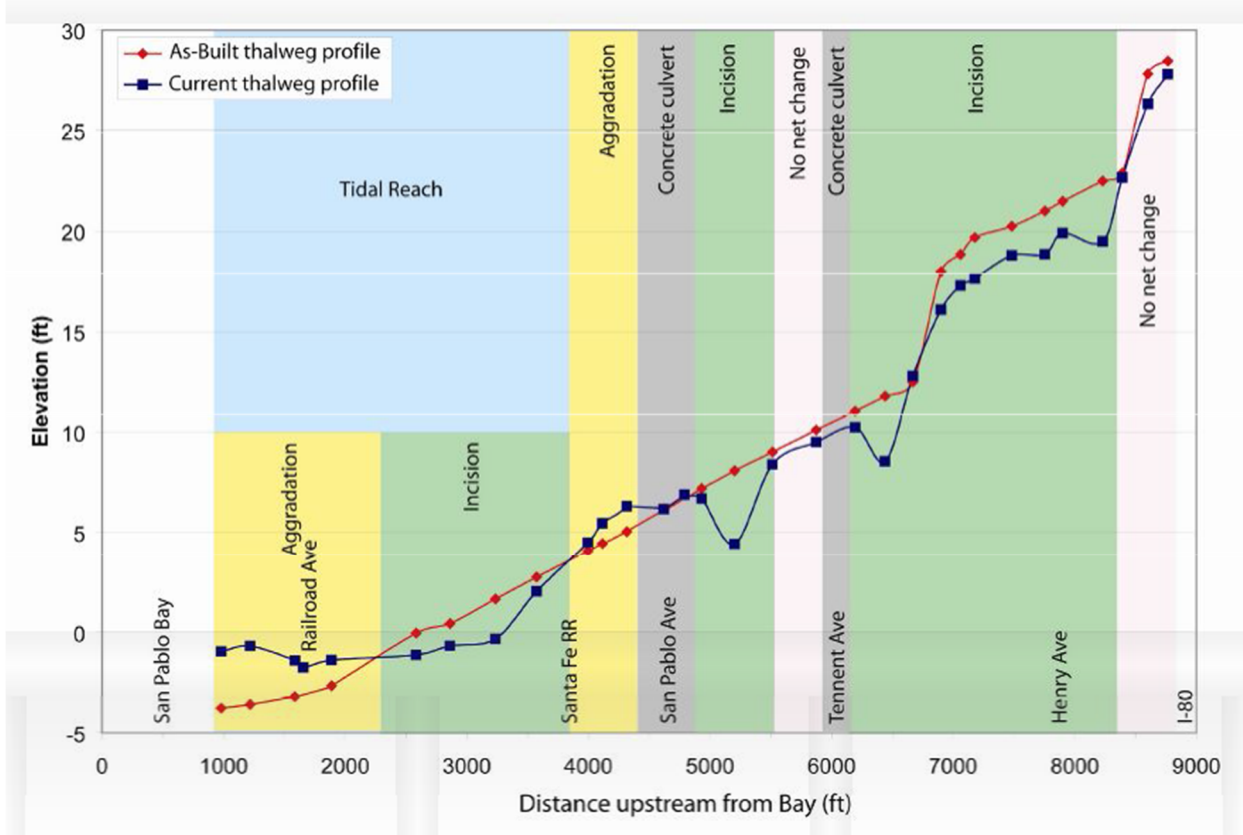


Figure 3.2.5.3: Thalweg longitudinal profile of the flood control channel. Red line is the “as-built” channel profile in 1965, blue line is the channel profile in 2003 (SFEI 2005)

3.2.6 Rodeo Creek, Contra Costa County (1966)

Watershed Setting

Rodeo creek is located along the southwestern slopes of San Francisco East Bay Hills through the City of Hercules and Town of Rodeo in Contra Costa County. The creek drains a 10.4 square miles watershed, from the private ranchlands and East Bay Municipal Utility District (EBMUD) property in the upper watershed with the highest elevation of 1,110 feet, flowing northeastward crossing Highway 80 and downtown Rodeo to San Pablo Bay (CCCFCWCD 2003). Figure 3.2.6.1 shows the watershed area. The mean annual precipitation is 21 inches, but it varies from 18 inches at the Bay to 24 inches in the hills (CCCFCWCD 1977).

Project Authorization

Contra Costa County Flood Control and Water Conservation District (Contra Costa County) is responsible for Rodeo Creek watershed planning and flood management. Historically the watershed was plagued with flooding problems. In 1890s, the Rodeo Reservoir was built for water supply and flood control. The 1000 feet wide 100 feet long reservoir dam was damaged in 1892 and 1895. By 1914, the dam was decommissioned. Since then, there were 10 flood events reported between 1938 and 1963. A major storm and resulting flood in 1955, 1957 and 1958 calls for action to protect downtown Rodeo from flood damage (Collins 2008). These flood events prompted the Rodeo Creek local flood protection project, authorized under Section 205 of the 1948 Flood Control Act in 1963 (USACE 1966c). The project constructed in 1966. The 5,800 feet improvements from Highway 80 to the bay outfall consists of 1,260 feet of concrete channel at the downstream and 4,370 of earth channel at the upstream (Collins 2008). Figure 3.2.6.2 shows the project alignment. The project cost \$929,000 in 1962 dollars, with 1.5 cost benefit ratio (USACE 1962b).

Channel Design Flow

The project was designed for the modified Standard Project Flood, at 2,500 cfs. The Standard Project Flood estimate at the upstream of BNSF railroad crossing is 3,000 cfs. However, due to the railroad crossing restriction and minimal upstream impact, the design flow was reduced to 2,500 cfs. The 100 year flow estimate was 2,100 cfs (USACE 1962b), which match the current 100 year flow estimate of 2,188 cfs (CCCFCWCD 2009).

Sediment Deposition

Reports of sedimentation mainly at the downstream tidal segment of the flood control channel, where 2 to 3 feet of sediment has accumulated. Long range operation and maintenance records are not available, but apparently dredging has not been conducted since 1993 and it was done infrequently before. A 1993 Public Notice posted by Contra Costa County proposed dredging of 6,000 cy of sediment that collected along the lower 1,600 feet of flood control channel. About 5,800 cy of sediment was proposed to be dredged from the concrete segment and 200 cy from the earthen channel. About 185 cubic yards was expected to be released into San Pablo Bay and 30 square feet of cord grass was to be removed near the mouth of the channel. Sediment deposits in

the channel were estimated to be 2 to 4 feet deep (Collins 2008). It is estimated by 2009, there was approximately 4,000 cy of sediment deposited in the concrete channel by the Bay outfall. The estimates was based on the limited silt survey conducted by Contra Costa County in 2009. Figure 3.2.6.3 shows the surveyed deposition profile. Based on this profile, a hydraulic modeling analysis in HEC-RAS shows that due to sediment deposition at the tidal reach, the current channel capacity is 1,300 cfs, assuming no channel scouring (CCCFCWCD 2009). The capacity is only about 50% of the original design capacity, and it is below the 2,188 cfs 100 year design flow. Therefore, the channel currently does not provide 100 year flood protection.

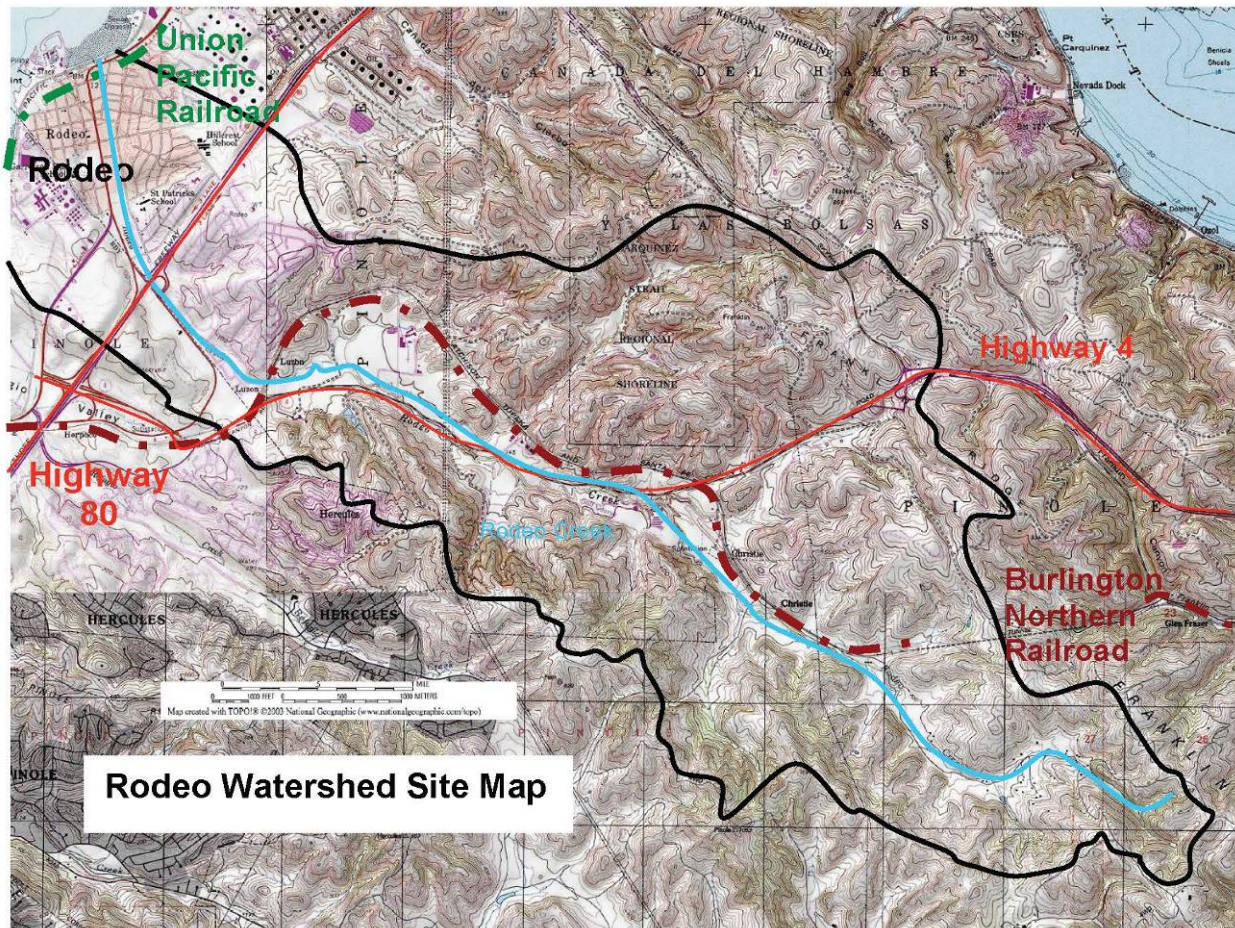


Figure 3.2.6.1: Rodeo Watershed (Collins 2008)

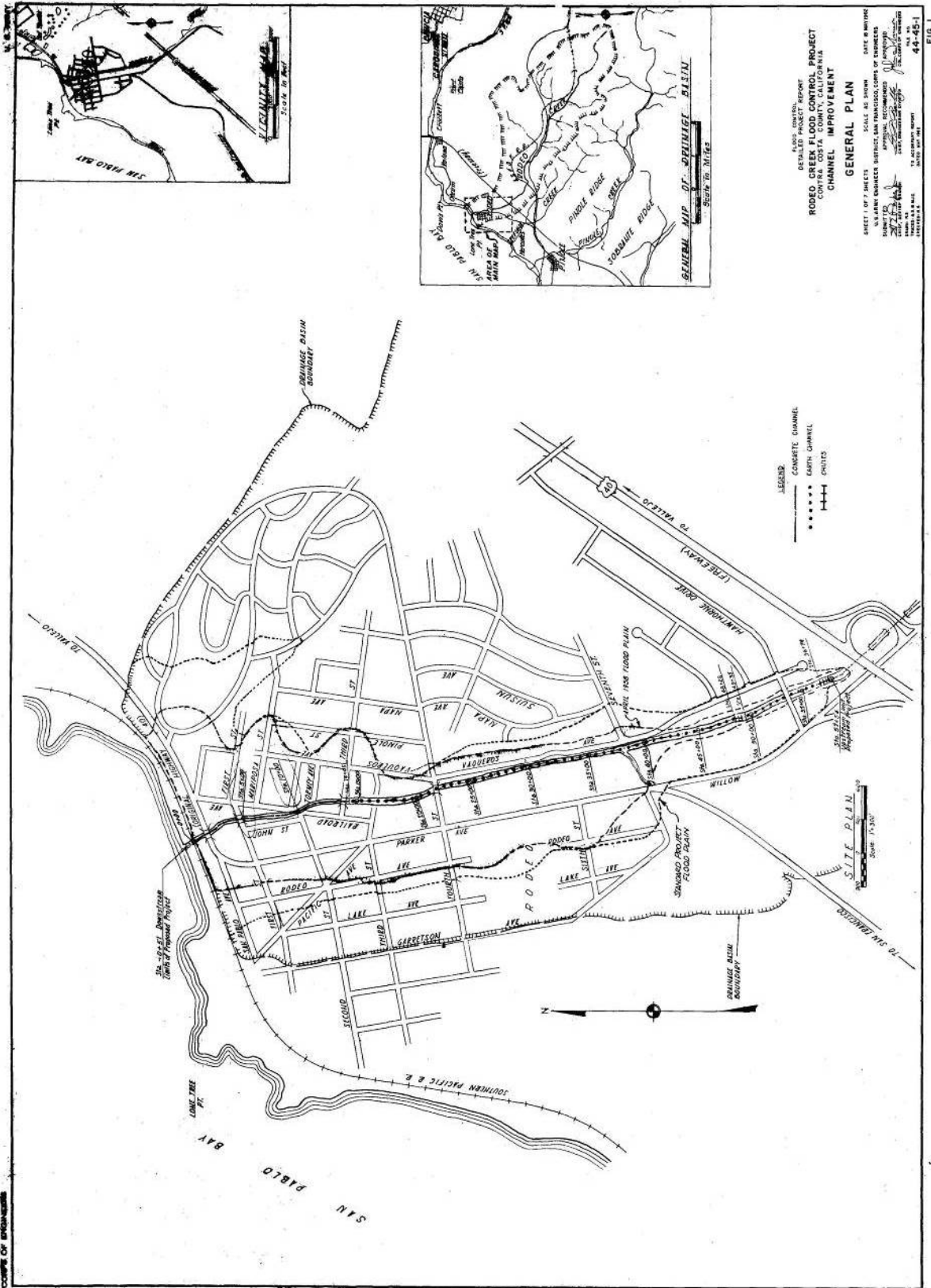


Figure 3.2.6.2a: Rodeo Creek Local Flood Protection Project (USACE 1962b)

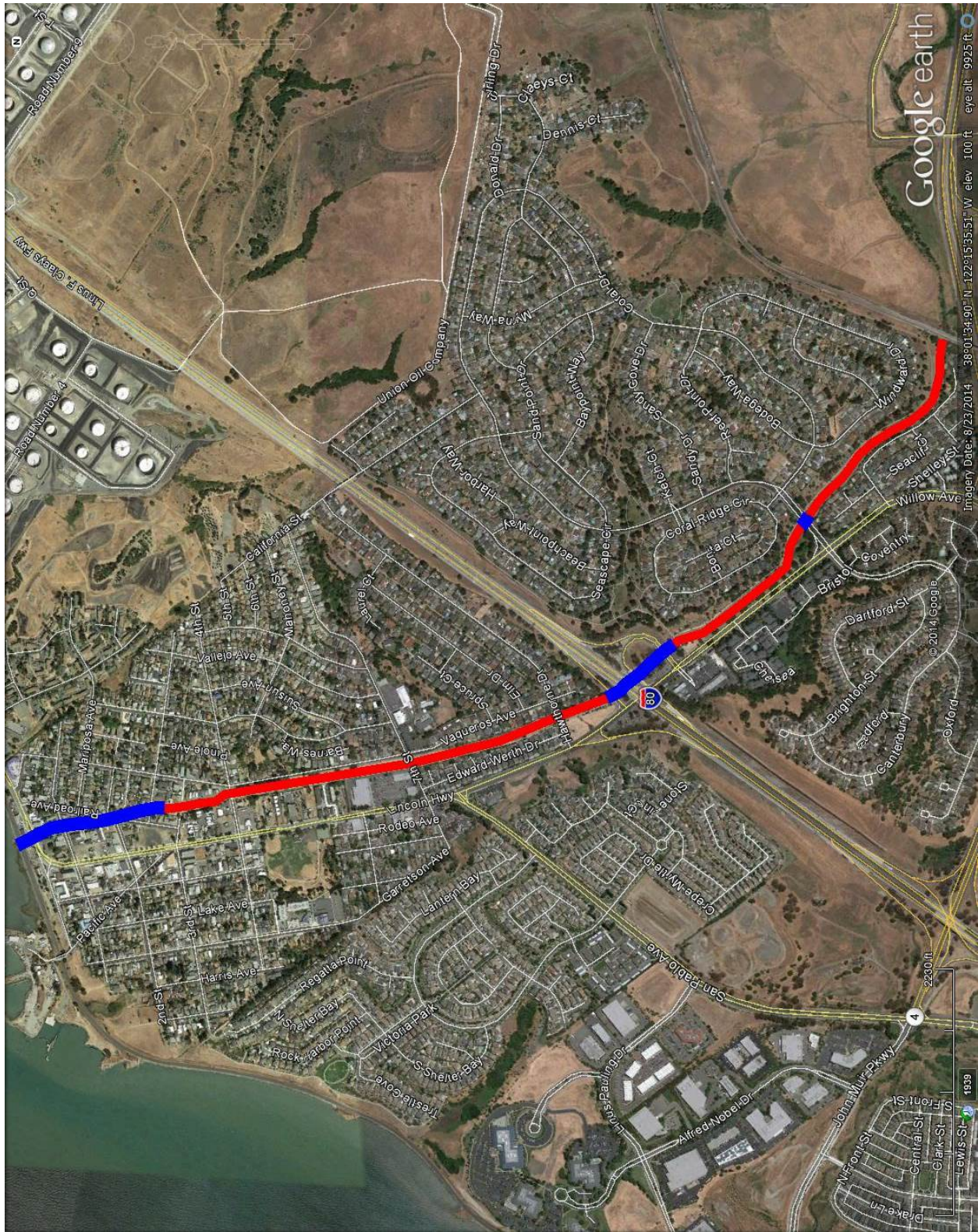


Figure 3.2.6.2b: Rodeo Creek Local Flood Protection Project

Legend: Concrete Channel = Blue Earth Channel = Red

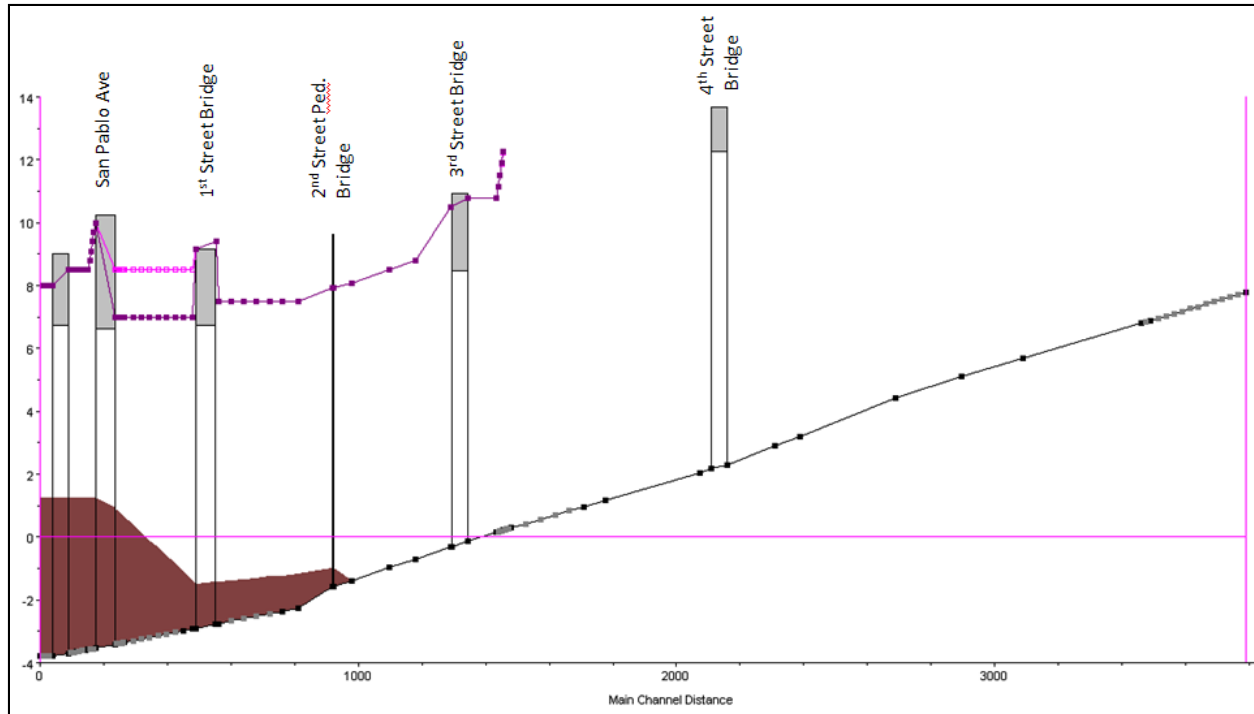


Figure 3.2.6.3: Sediment deposition profile in the flood control channel (CCCFCWCD 2009)

3.2.7 Corte Madera Creek, Marin County (Units 1 to 3: 1971)

Watershed Setting

Corte Madera creek is located in Ross Valley at Marin County. The creek drains a 28 square mile watershed, from the Sleepy Hollow Creek, Fairfax Creek, and San Anselmo Creek at the headwater, flowing eastward connecting with Ross Creek, Tamalpais Creek and Larkspur Creek to Richardson Bay (USACE 2010). Mount Tamalpais at the headwater of Ross Creek is the highest point in the watershed, at 2570 feet. Figure 3.2.7.1 shows the watershed area. The mean annual precipitation is 49 inches, recorded at Olive Avenue in Ross between October 1979 and September 1996. The highest record is 91 inches in water year 1983 (USACE 2000).

Project Authorization

Marin County Flood Control and Water Conservation District (Marin County) is responsible for Corte Madera Creek and Ross Valley watershed planning and flood management. The watershed experienced numerous flood events, including 1942, 1951, 1955, 1958, 1960, 1962, 1963, 1967, and 1969. The April 1958 flood resulted in one fatality. These flood problems prompted the Marin County to initiate the flood control project at Corte Madera Creek. In partnership with USACE, the Flood Control Act of 1962 authorized the Corte Madera Creek Flood Control Project. Three of the four authorized units of the project were constructed in 1971. The constructed project consisted of an earth trapezoidal channel through Unit 1 and most of Unit 2, extending three miles from San Francisco Bay to the Community of Kentfield, and a rectangular concrete channel in the upper 1,500 feet of Unit 2 and throughout Unit 3. Unit 4 was scheduled for construction in 1972, but was postponed due to litigation on the referendum vote requirement, and environmental concerns on the proposed concrete channel improvements from property owners along the creek. Several studies have been conducted to evaluate the improvement alternatives for Unit 4, but the planning process is still on-going. Figure 3.2.7.2 shows the project alignment. The project cost \$6,970,000 in 1967 dollars, with 1.1 cost benefit ratio (USACE 1967).

Channel Design Flow

The project was designed to carry runoff from a Standard Project Flood of 7,500 cfs at the Ross stream gage. The peak discharge vs frequency analysis for Corte Madera Creek estimated the 7,500 cfs is approximately equal to a 250 year event, and the 100 year flow is 6,600 cfs (USACE 2000). After the Units 1 to 3 projects were completed, flooding occurred in 1982, 1983, 1986, 1997, and 2005. The most extensive flood occurred in 1982, which the record peak flow of 7,200 cfs at the Ross stream gage exceeded the 100 year flow design estimate. In all of the flood events, since the Unit 4 project was not constructed and the conveyance capacity at Lagunitas Road Bridge was restricted at 3,500 cfs, Corte Madera creek overflows and floods Ross, Kentfield, and the College of Marin. In addition, the concrete channel in Unit 2 and Unit 3 does not provide the design capacity during the flood event. The concrete channel was designed for supercritical flow, with low friction loss under the clear water assumption. The sediment laden flow during the 1982 storm created a premature hydraulic jump along the concrete channel upstream of the stilling basin. Since the channel was designed with only 1 foot of freeboard

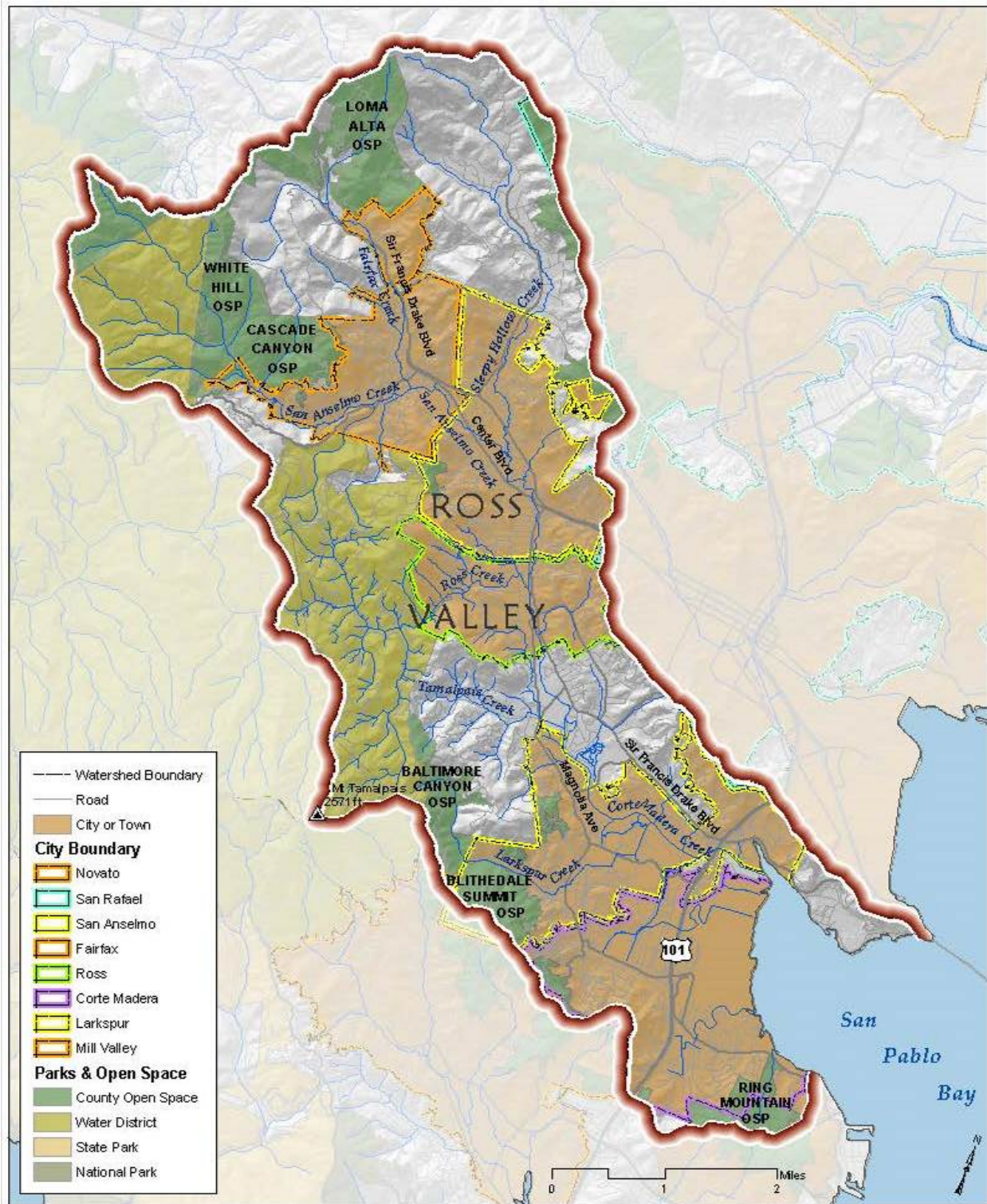
under the supercritical flow condition, the hydraulic jump and the resulting subcritical flow overtopped the channel and flooded the adjacent low-lying area along Kent Avenue. The total estimated flood damages amounted to \$2.25 million in 1983 dollars (USACE 2000, USACE 2010).

Sediment Deposition

Typical of most USACE flood control projects, Marin County (the local sponsor) is responsible for the operation and maintenance of the project on Corte Madera Creek. The operation and maintenance cost was estimated in 1961 at \$2,700 per year (USACE 1967). Based on the stated unit dredging cost of \$0.8 per cy (USACE 1966a), the annual dredging volume we can back estimate to have been 3,375 cy.

Since the earth channel in the Units 1 to 2 projects were dredged in 1966, Marin County dredged the three units in 1987 and 1998. In addition, Town of Ross extracted sediment annually in the vicinity of Lagunitas Road Bridge. The total volume of sediment removal through 1998 was 478,000 cy. Channel surveys in 2005 and 2010 estimated the total volume of sediment deposited in the channel as 460,000 cy. Based on the sediment volume and deposition data, the sediment deposition rate is 21,300 cy/yr. Consider the sediment supply rate is estimated at 54,000 cy/yr (Stetson 2000), the sediment trap rate at the flood control channel is 40%.

The total sedimentation of 938,000 divided by 44 years between 1966 and 2010 yields an annual deposition rate of 21,300 cy/yr, over 6 times the original design estimate. Note that if the channel dredged more frequently, the trap efficiency and sediment deposition volume would be even higher. The underestimated sedimentation rate and the flooding in the 1980s prompted a series of channel hydraulic and sediment studies from the local sponsor and USACE. In addition, Marin County developed a Capital Improvement Program in 2011 to address the flood protection needs in Ross Valley watershed (Stetson 2011). A detail case study on Corte Madera Creek sediment and sedimentation issues, Unit 4 and Ross Valley watershed planning process, and a sensitivity analysis on the channel hydraulic due to sediment influence are presented in Chapter 4.



Ross Valley Watershed

County of Marin
Department of Public Works

www.marinwatersheds.org



Map for general purposes only; not for site-specific planning purposes.

Figure 3.2.7.1: Corte Madera Creek Watershed (County of Marin Dept. of Public Works 2014b)

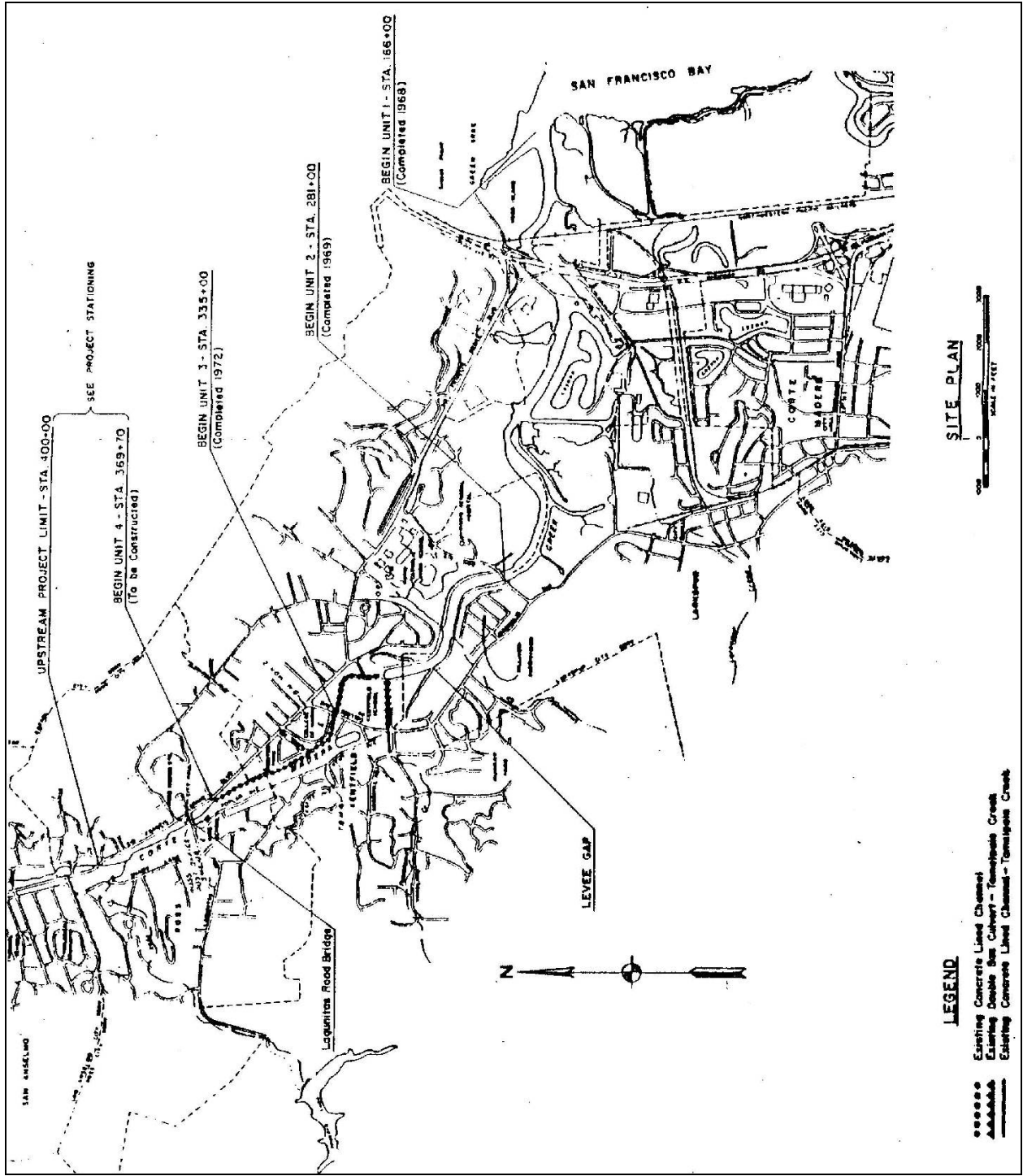


Figure 3.2.7.2a: Corte Madera Creek Flood Control Project (USACE 1988)

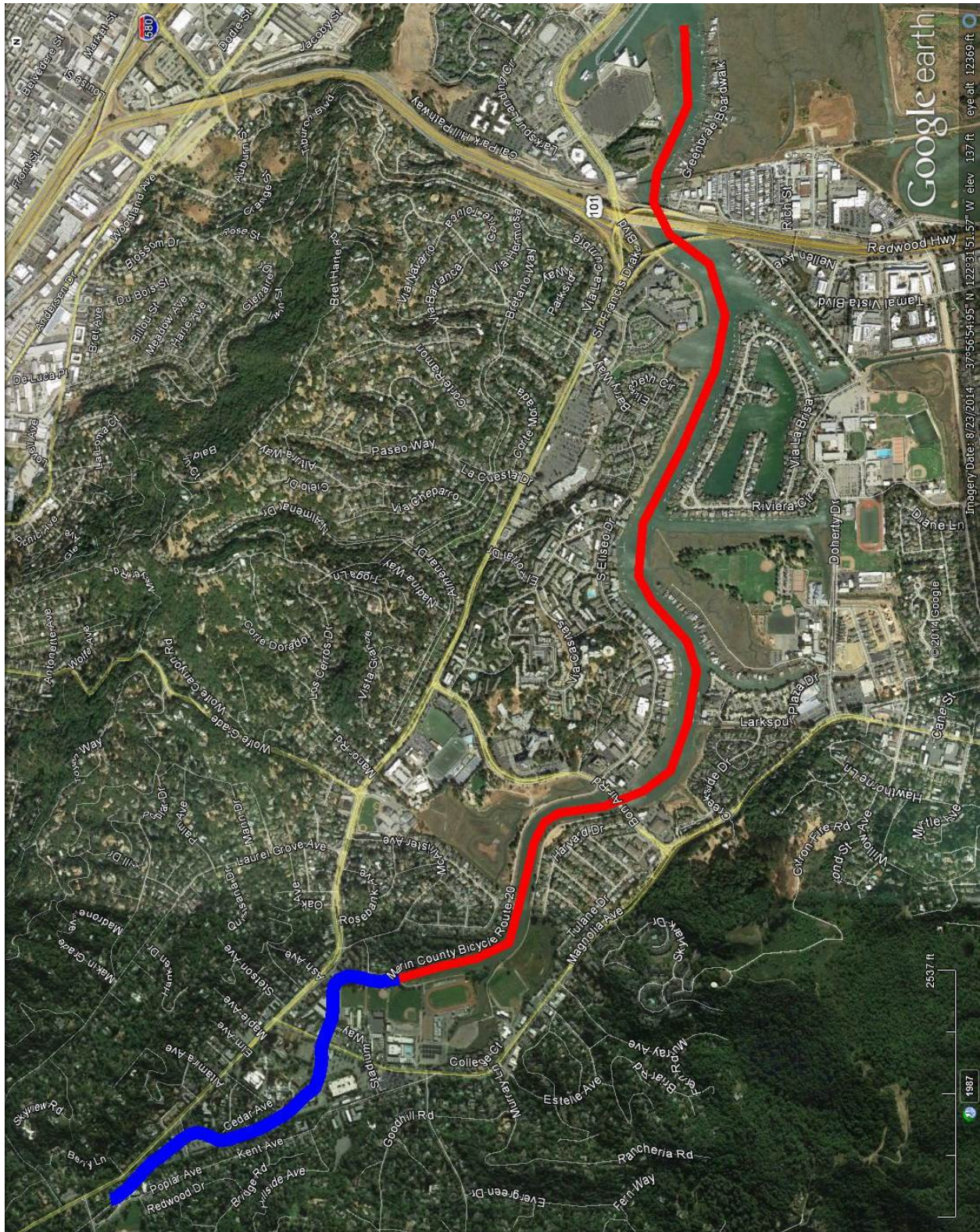


Figure 3.2.7.2b: Corte Madera Creek Flood Control Project

Legend: Concrete Channel = Blue Earth Channel = Red

3.2.8 San Leandro Creek, Alameda County (1974)

Watershed Setting

San Leandro creek is located along the San Francisco East Bay Hills through the City of San Leandro and Oakland in Alameda County. The creek drains a 48 square mile watershed, from Sibley Volcanic Park in Contra Costa County with the highest elevation of 1,700 feet, passes through Upper San Leandro and Chabot reservoirs, flowing eastward across Cities of San Leandro and Oakland to San Francisco Bay (San Leandro Creek Watershed Advisory Committee 1999). Chabot Reservoir was constructed in 1876. Upper San Leandro Reservoir, located upstream of Chabot Reservoir, was constructed in 1927. Both reservoirs are owned, operated, and maintained by the East Bay Municipal Utility District (EBMUD). The reservoirs provide domestic and emergency water supply, flood management by peak flow attenuation and sediment trap, and recreational feature for the region (EBMUD 2013, Eakin 1939). Figure 3.2.8.1 shows the watershed area. The mean annual precipitation is 25 inches, but it varies from 18 inches at the Bay to 28 inches in the hills (USACE 1970).

Project Authorization

Alameda County Flood Control and Water Conservation District (Alameda County) is responsible for San Leandro Creek watershed planning and flood management. The watershed was plagued with flooding problems. The April 1958 flood has 1,700 cfs peak flow, inundated 115 acres and isolated approximately 150 residences requiring emergency evacuation. There are other flood events in the watershed including the 1962 flood, but relevant data were not available. These flood events prompted the San Leandro Creek local flood protection project, authorized under Section 205 of the 1948 Flood Control Act in 1970 (USACE 1970). The project constructed in 1974 (USACE 1977). The 10,000 feet improvements from Highway 880 to the Bay outfall consists of 2,600 feet of concrete channel at the upstream and 7,400 of earth channel at the upstream (USACE 1977). Figure 3.2.8.2 shows the project alignment. The project cost \$1,720,000 in 1970 dollars, with 1.3 cost benefit ratio (USACE 1970).

Channel Design Flow

The project was designed for the 100 year flow, at 2,800 cfs. The Standard Project Flood estimate is 6,300 cfs. Note that neither Chabot nor Upper San Leandro Reservoirs were built with flood control as an objective. However, since the reservoirs control 90% of the watershed area, their flood protection effects were accounted into the flow estimate. The 6,300 cfs estimate was based on the assumption that both reservoirs are full. If only Chabot Reservoir is full but Upper San Leandro Reservoir still has flood storage capacity, the Standard Project Flood estimate could become 2,500 cfs, which is below the 100 year design flow adapted for this project. On the other hand, if the reservoirs do not exist in the watershed, the 100 year flow would increase to 8,500 cfs (USACE 1970). Updated flow estimate for with dam condition is not available, for the without dam condition, the 100 year flow estimate increased from 8,500 cfs to 12,437 cfs (AECOM 2013). The current channel capacity is estimated at 1,774 cfs, which is below the 2,800 cfs design flow (AECOM 2013). The FEMA Flood Insurance Study also found that the channel currently do not provide the 100 year flood protection (FEMA 2009).

If the project were to provide Standard Project Flood protection at 6,300 cfs, the construction cost will be doubled and the cost benefit ratio would be 0.8 (USACE 1970). Therefore Standard Project Flood protection is deemed economically infeasible. The project formulation also considered floodplain management option but dismissed, because the existing residential development were unlikely to change and the existing structures were not readily adaptable to flood proofing. In addition, if the industrial areas along the creek corridor were to be developed, it would require between 0.5 and 1 million cy of imported fill material to raise the ground elevation above the flood elevation. Therefore, the floodplain management option was deemed infeasible (USACE 1970).

Sediment Deposition

Sediment budget and deposition data is not available. Anecdotal information suggests that creek did not have significant fluvial sediment influx due to the upstream reservoirs, but sediment deposition in the tidal reach was observed. There have been no major sediment removal works in San Leandro Creek (Saleh 2014).

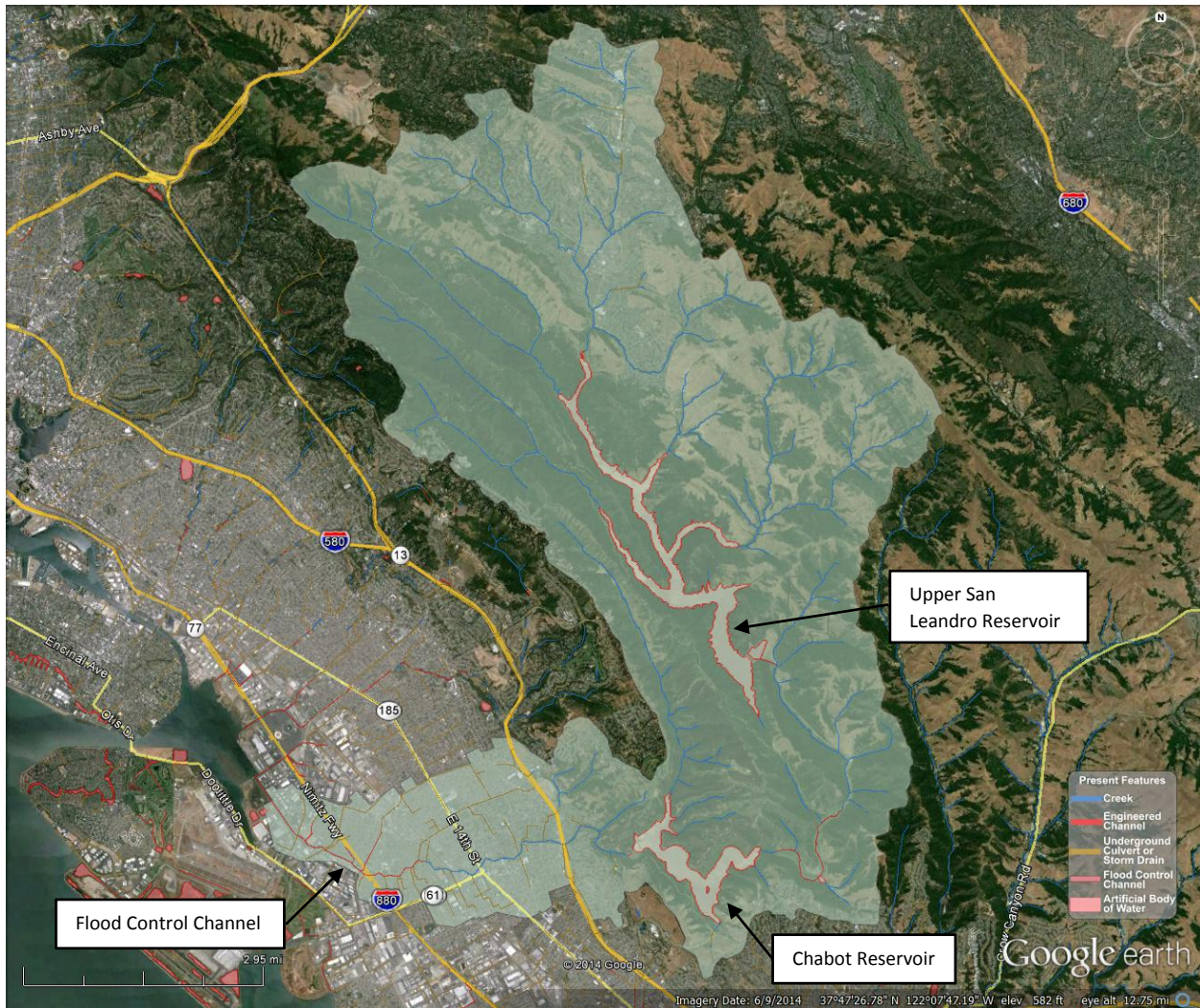


Figure 3.2.8.1: San Leandro Creek Watershed (Oakland Museum 2010)

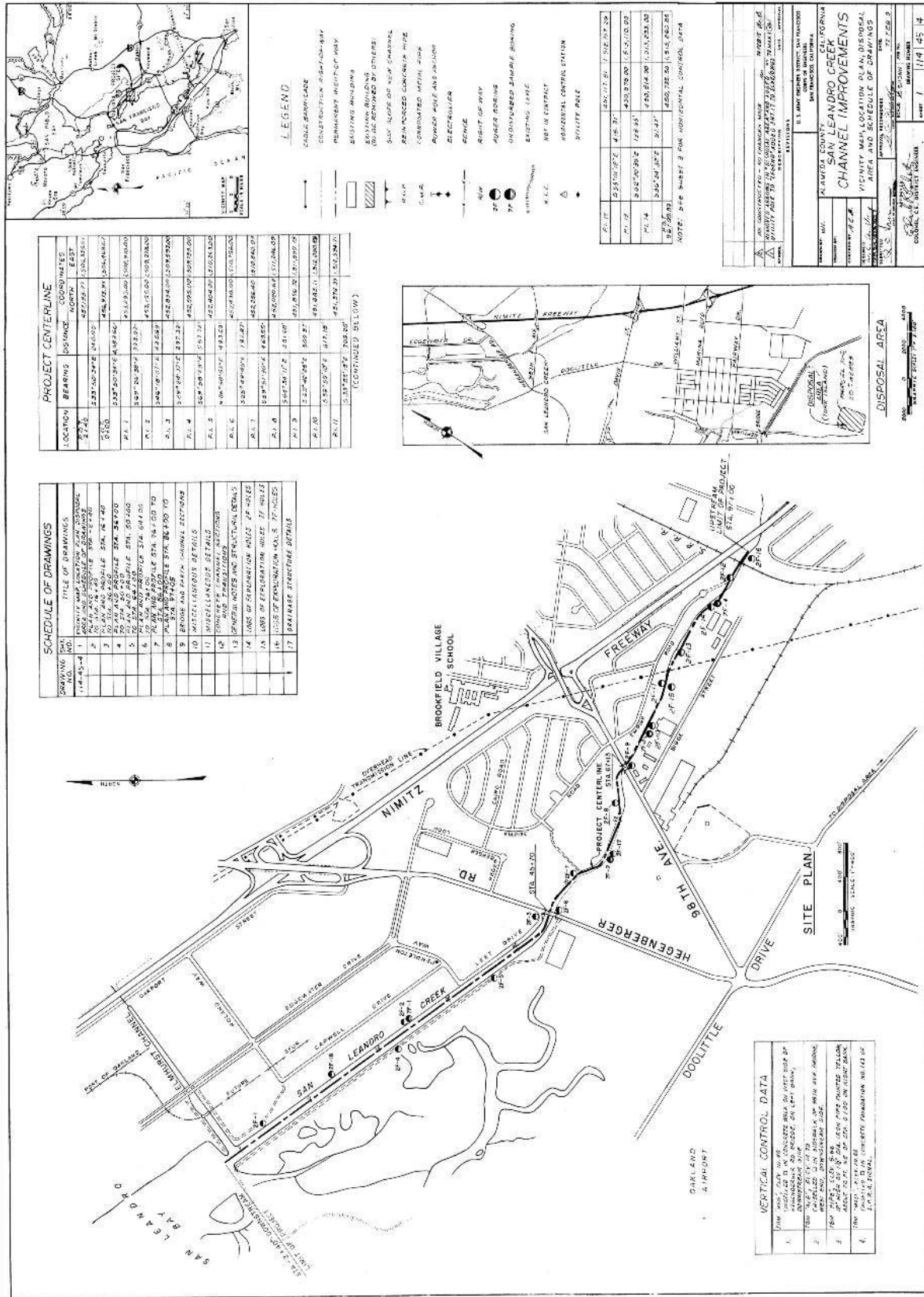


Figure 3.2.8.2a: San Leandro Creek Local Flood Protection Project (USACE 1977)

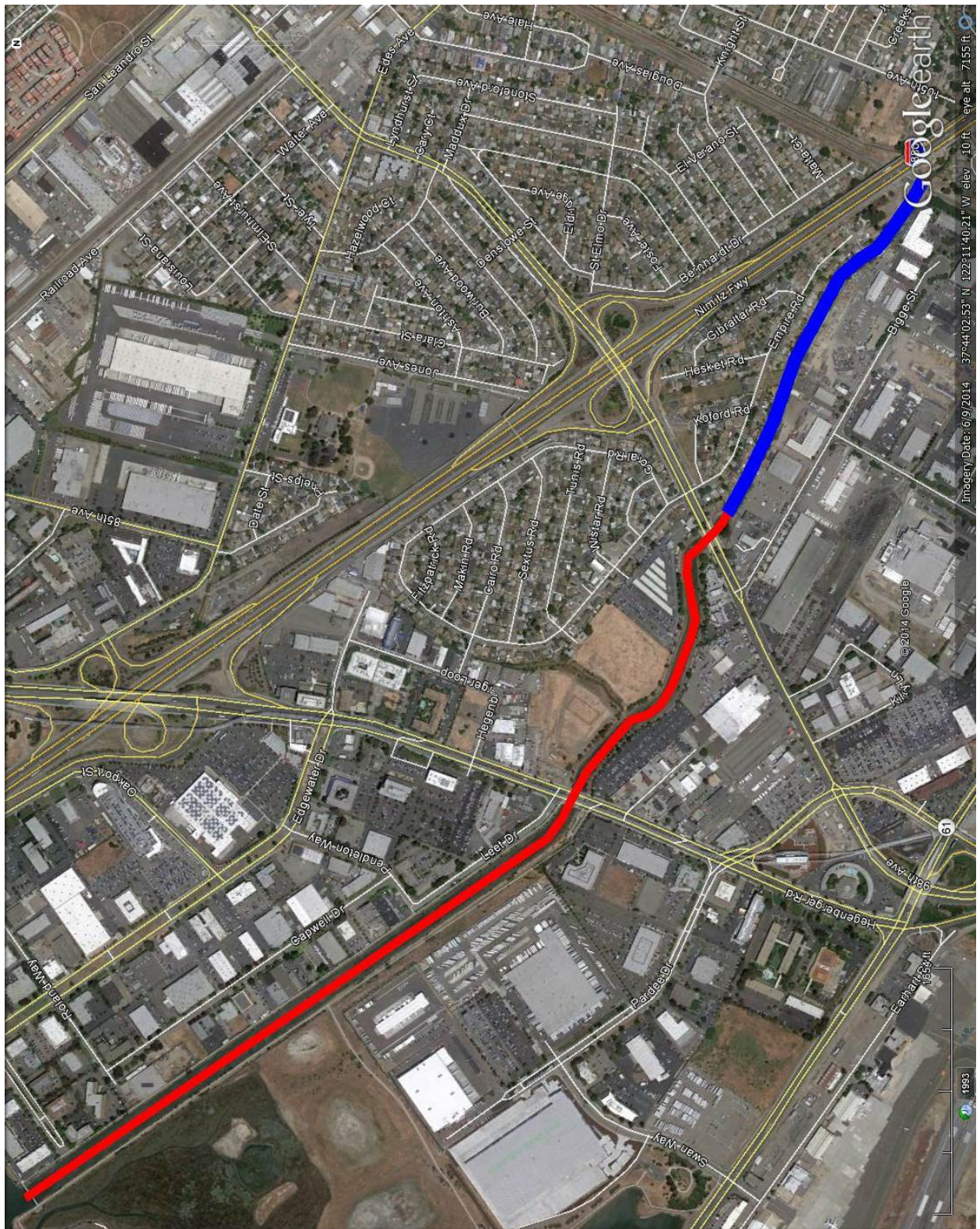


Figure 3.2.8.2b: San Leandro Creek Local Flood Protection Project
Legend: Concrete Channel = Blue Earth Channel = Red

3.2.9 Alameda Creek, Alameda County (1975)

Watershed Setting

Alameda creek watershed spans across Alameda County to the east, Santa Clara County to the south, and Contra Costa County to the north. At 695 square miles, it is the largest watershed draining to San Francisco Bay outside of the Sacramento-San Joaquin watershed. The creek drains from the Sunol Valley drainage unit, the Del Valle drainage unit, and the Cities of Livermore, Pleasanton, Dublin and San Ramon in the Livermore-Amador Valley drainage unit, flowing westward through the East Bay Hills via Niles Canyon, exiting at the town of Niles to the lower Alameda Creek watershed. Lower Alameda Creek watershed is the historic alluvial fan and floodplain complex between the Niles Canyon and San Francisco Bay. It consists of urban areas including Cities of Fremont, Hayward and Union City. The peak elevation in the watershed is 4,400 feet at Mount Hamilton. The watershed contains three large reservoirs: Calaveras Reservoir, San Antonio Reservoir, and Del Valle Reservoir. The reservoirs control 48 percent of the watershed area. Calaveras and San Antonio Reservoirs are for water supply to the San Francisco Public Utility Commission. The Del Valle Reservoir is a part of the Alameda Creek Flood Control Project, providing both flood control and water supply functions (SFPD 2008). Figure 3.2.9.1 shows the watershed area. Average annual precipitation is 22 inches (USACE 1961), but it ranges from about 20 to 25 inches per year. Precipitation is heaviest in the west at higher elevations, and lowest in the eastern part of the watershed and at lower elevations (SFPD 2008).

Project Authorization

Alameda County Flood Control and Water Conservation District (Alameda County) is the primary agency responsible for Alameda Creek watershed planning and flood management. The lower Alameda Creek watershed experienced flooding in 1952, 1955, 1958, 1962, and 1963. While these flood events highlighted the importance and urgency of the flood control project, leading the 1962 Flood Control Act authorization, the planning process actually started years before the consecutive flooding in the 1950s.

The 1937 Flood Control Act authorized a pre-project examination and survey to evaluate potential need for a flood control project in Alameda Creek. An unpublished report was completed by the USACE on July 21, 1942 (USACE 1961). The report concluded that improvements in the Coastal Plain (Alvarado) area were not economically justified at that time, and although improvements in Livermore Valley area appeared to be justified, local interests apparently were not willing to assume the required local cooperation. The Chief of Engineers recommended that no survey be made. In 1949, the Committee of Public Works of the United States Senate adopted a resolution to review the 1942 report, stipulate that the review report give full consideration to land enhancement, and to a multiple-purpose method of supplementing the deficient water supply, and alleviating salt water intrusion in the area near the shore of San Francisco Bay. The review resulted with a Review Report for Flood Control and Allied Purposes for Alameda Creek (USACE 1961), which recommended the flood control project. The Flood Control Act of 1962 authorized the Alameda Creek Flood Control Project. The project consists of 12 miles of earth flood control channel from Niles Canyon to San Francisco Bay Outfall, a

multi-purpose reservoir at Arroyo del Valle, rubber dams and other design components along the flood control channel to facilitate groundwater recharge and channel protection. The Del Valle Dam was constructed in 1968, and the flood control channel was constructed in 1975. Figure 3.2.9.2 shows the project alignment. The project cost \$22,400,000 in 1964 dollars, with 2.8 cost benefit ratio (USACE 1964b). Note that the project cost did not cover the full cost of the Del Valle Dam construction, the project cost is based on the pro-rated first cost for the flood protection benefits, and \$776,000 pre-payment for the future operation and maintenance. Since the reservoir is part of the state water project, it was designed and constructed by the State of California (USACE 1964b).

Channel Design Flow

The project was designed for the Standard Project Flood, at 52,000 cfs. If Del Valle Dam were not a part of the project, the Standard Project Flood would have been 71,000 cfs. The 100 year flow is estimated at 32,000 cfs. This 100 year design flow is used in the FEMA Flood Insurance Study, and various studies and projects up to now. Although since the project was authorized, the San Antonio reservoir constructed in 1965 would reduce flood flow, extensive urbanization in the Livermore Valley would likely modify the urban hydrology to increase the flow. While an updated hydrology study for the watershed is not available to verify the current 100 year design flow (Saleh 2014), Flood Frequency Analysis of peak flow data from the USGS gage at Niles (USGS 11179000) indicates the current 100 year flow is 26,420 cfs. In addition, the largest post-project peak flow was 17,900 cfs in 1998. These evidence suggest the current 100 year design flow may not exceed 32,000 cfs.

Sediment Deposition

Collins (2006a) estimated Alameda Creek total sediment budget between 1975 and 2004. The analysis was based on the dredging record and sediment deposition data in Alameda Creek Flood Control Channel, sediment records at the Niles Canyon Gage, and upstream reservoirs sedimentation data. The analysis estimated that 194,000 cy/yr total sediment trapped in the upstream reservoirs. At the Niles Canyon Gage, the total sediment supply rate is 125,300 cy/yr. About 50,300 cy/yr of this sediment (40% of total load) is stored in the flood control channel, and about 25,000 cy/yr (20% of the total) has been dredged to maintain flood capacity. This means that Alameda Creek trapped about 75,300 cy/yr of sediment deposition in the flood control channel, the trap rate is 60% (Collins 2006a).

Since the project was completed, sediment deposition in the flood control channel has required frequent sediment removal. The first dredging occurred in 1975 for the 15,000 feet section of channel constructed in 1968 at the downstream of Highway 880 crossing. Since then, there were sediment removal works in various reaches as shown in Figure 3.2.9.3 (Collins 2006a). There were close to 1 million cy of sediment removed from Alameda Creek, with close to 2 million cy still deposit in the channel, mostly at the downstream tidal reach. Since 1998, Alameda County already spent over \$3.2 million on sediment removal at Alameda Creek. Even with these efforts, sedimentation at the tidal reach formed floodplain benches within the flood control channel at elevation 6.5' (Saleh 2014). Consider the design thalweg elevation at the outfall is approximately -3', the 9.5' high benches formation restricted the channel flow. Alameda Creek is currently

analyzing the available channel capacity, and develop options to properly maintain the channel to meet the flood protection, habitat enhancement, and regulatory permitting objectives.

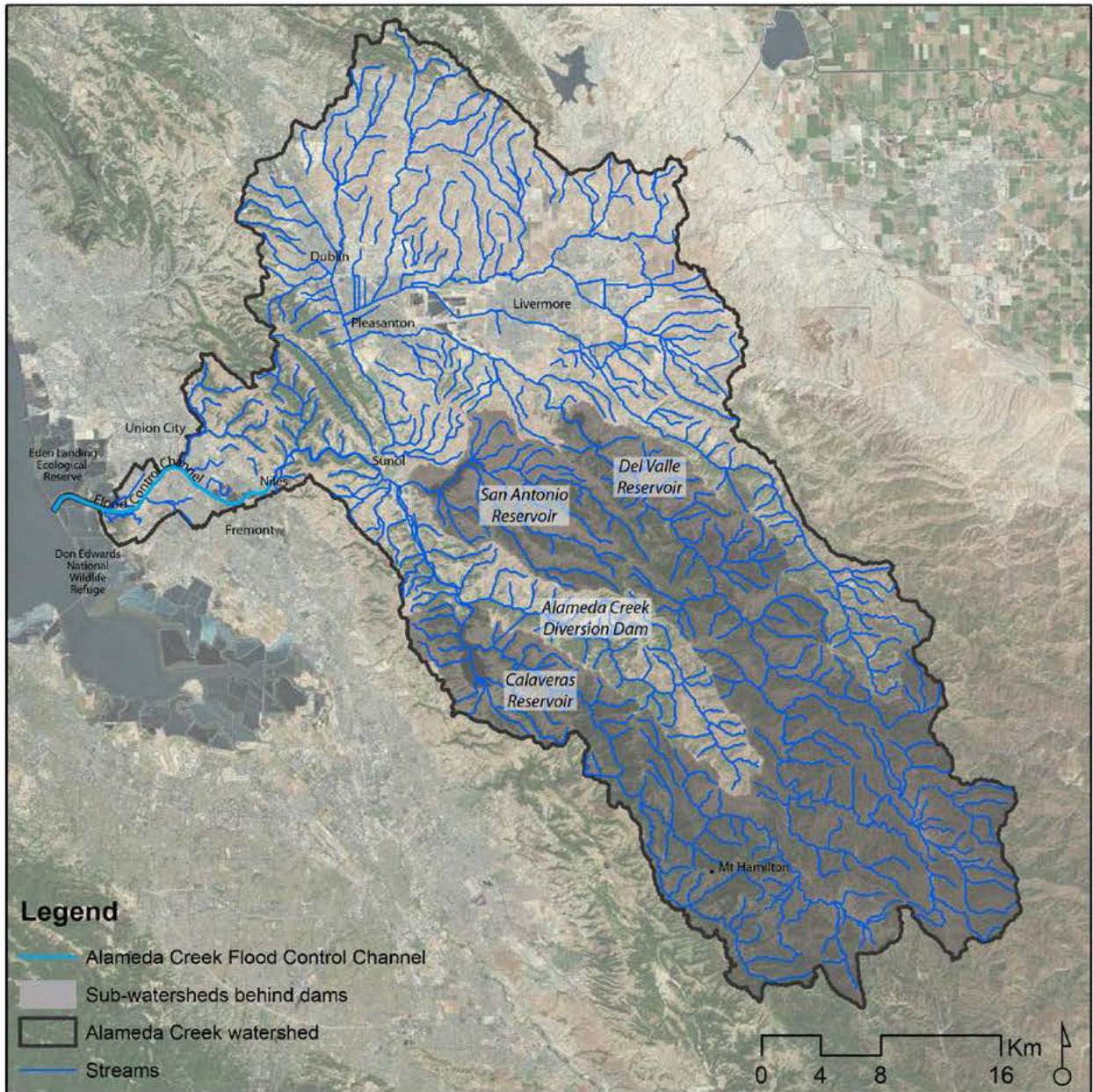


Figure 3.2.9.1: Alameda Creek Watershed (SFEI 2014)

Alameda Creek Flood Control Channel

Sedimentation Rates and Location of Dredging Activities along the Longitudinal Thalweg Profile

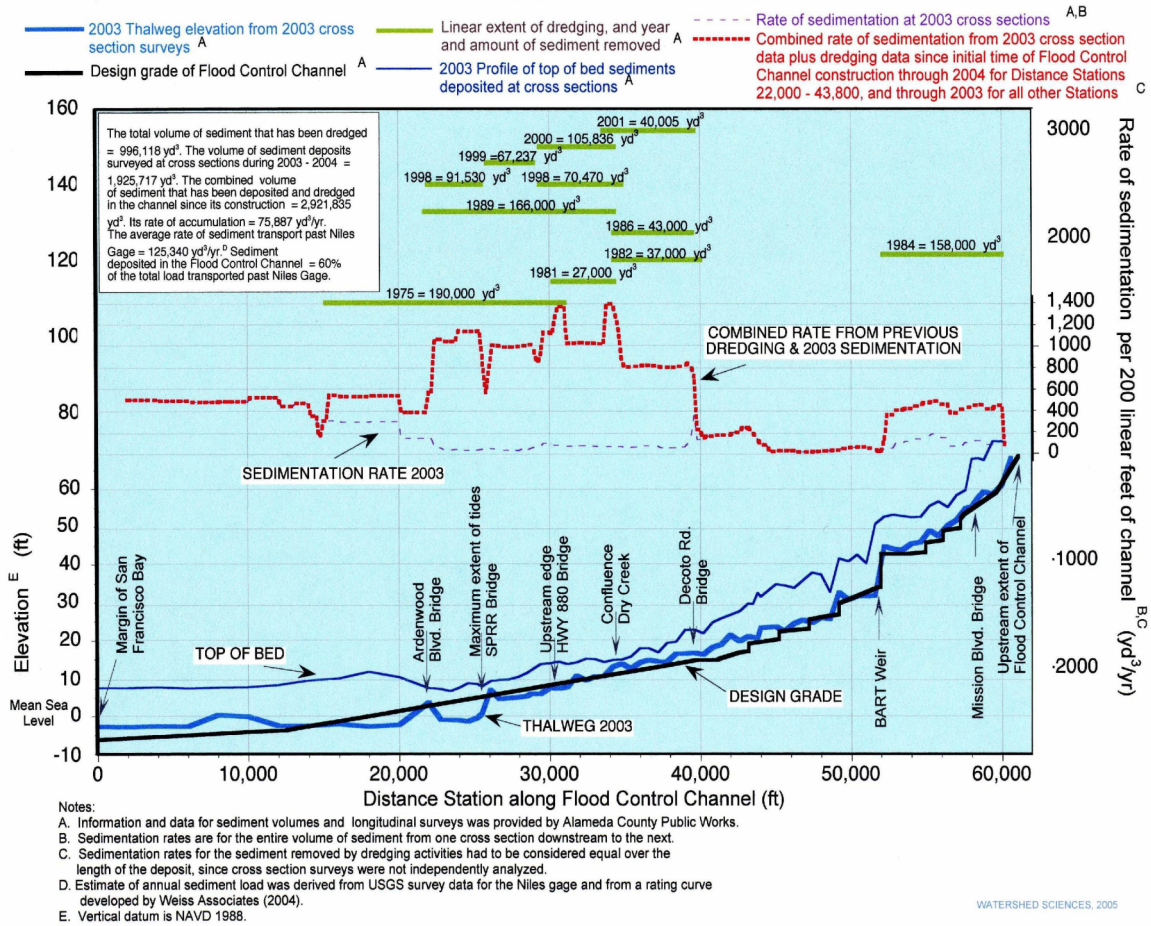


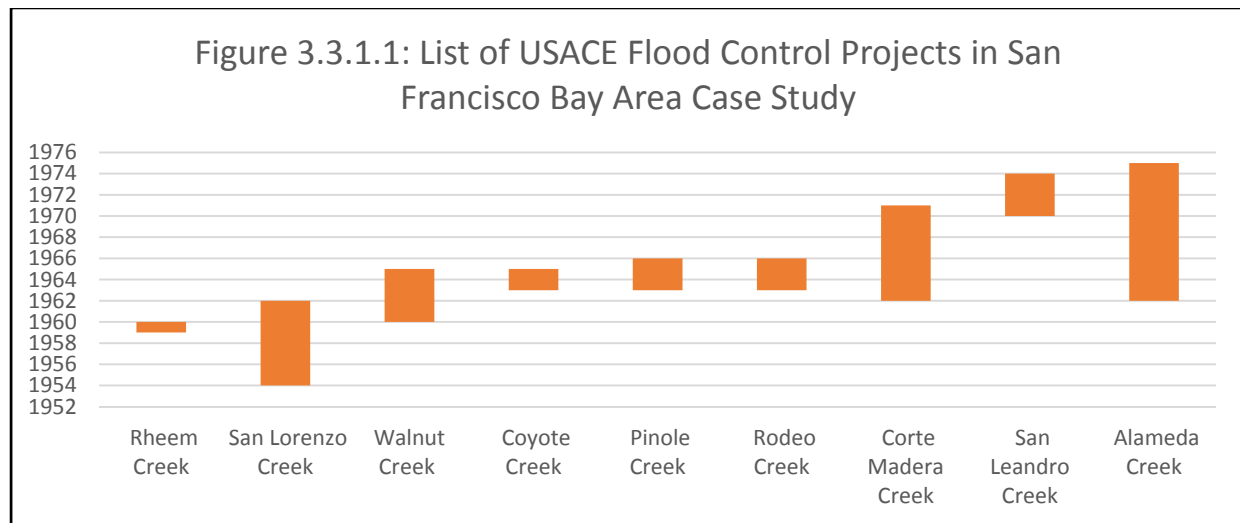
Figure 3.2.9.3: Sedimentation and removal history at the Alameda Creek Flood Control Channel (Collins 2011)

3.3 Case Study Findings

In this case study I reviewed the project justification, hydraulic performance and sediment management on 9 USACE flood control projects in San Francisco Bay Area. My goal is to identify commonalities among these projects in order to highlight the systematic planning and design issues. The following summarizes the case study findings. A data summary is listed in Tables 3.2a to 3.2i at the end of the chapter.

3.3.1 Project Justification – Cost Sharing

All of the USACE flood control projects in this case study are located around urban areas in San Francisco Bay Area. As shown in Figure 3.3.1.1, these projects were authorized and constructed in a 20 years span from mid-1950s to mid-1970s.



These projects were driven by the post World War II urban growth and flooding events in the 1950s. It is also the era after the “The Flood Control Law of 1946” written into the California Water Code (California State Legislature 1943). Per the California Water Code Section 12803:

The Legislature intends that from time to time by law allocations from the General Fund for flood control projects will be made to pay for the cost of all lands, easements, and rights-of-way necessary for the construction of flood control projects adopted and authorized by the Legislature prior to March 12, 1946, and also those adopted and authorized on or after that date by the Congress of the United States, recommended by the department, and approved by the Legislature, in the order in which the Congress makes available funds for their construction.

The definition of lands, easements, and rights-of-way per the California Water Code Section 12573:

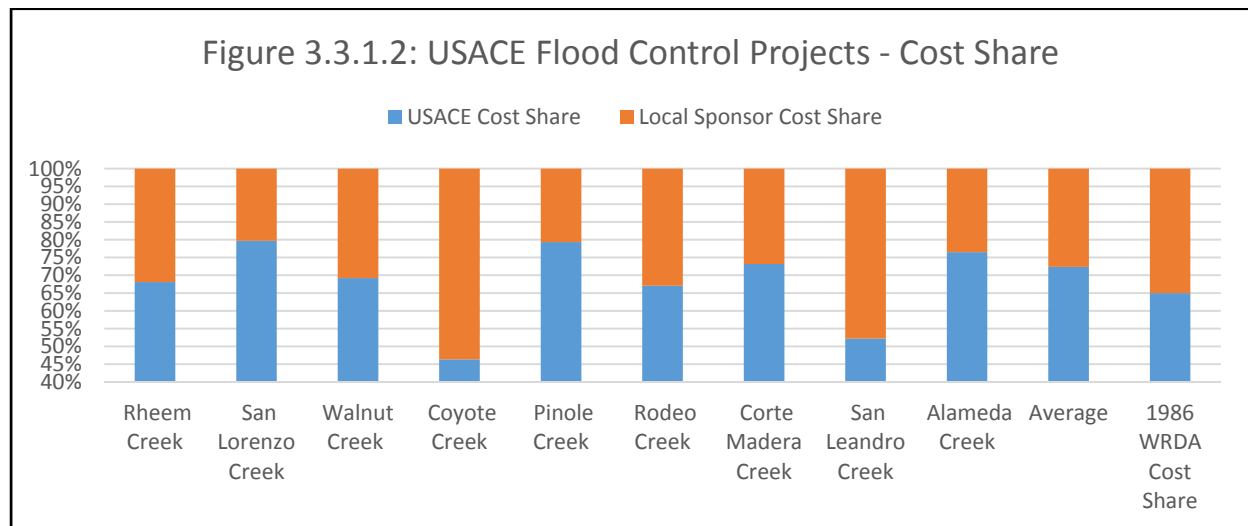
"Lands, easements and rights of way" includes lands and rights or interests in lands whereon channel improvements and channel rectifications are located; lands, rights, or interests in lands

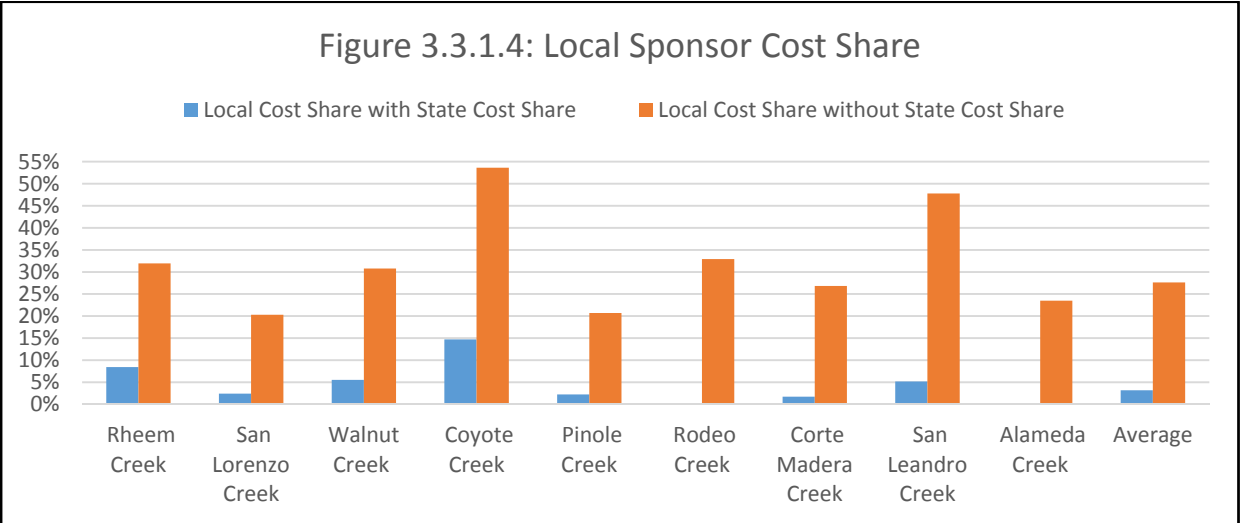
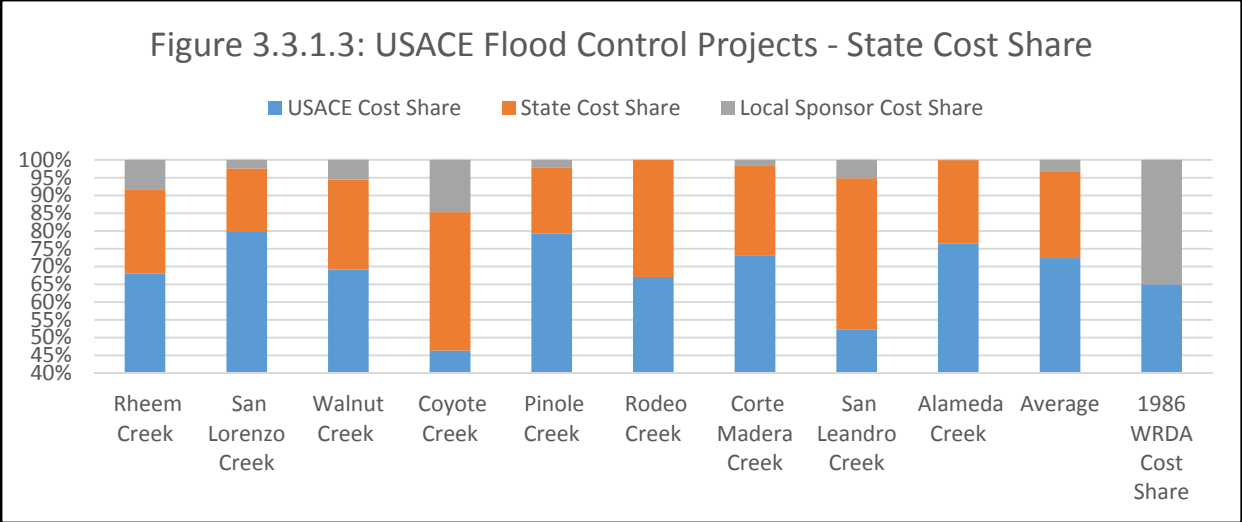
necessary in connection with the construction, operation, or maintenance of such channel improvements and rectifications, including those necessary for flowage purposes, spoil areas, borrow pits, or for access roads; and including the cost of the relocation, reconstruction, or replacement of existing improvements, structures, or utilities rendered necessary by such channel improvements and rectifications.

The state cost share program provided a significant financial incentive for the local sponsor on the USACE flood control projects. To quantify the effects of the state cost sharing, Figure 3.3.1.2 shows the USACE and local sponsor cost sharing. On average, I found the cost sharing division between USACE and local sponsor is about 72% to 28%, which is close to the 65% to 35% formula set in Water Resources Development Act of 1986 (US Congress 1986). However, in Figure 3.3.1.3, I found that with the California state cost sharing, the local sponsor average cost share reduced from 28% to 3%. I further illustrated the reduction in local cost share in Figure 3.3.1.4, by comparing the local sponsor cost with and without the state cost share.

I found that the state cost share program provided a strong justification for the local sponsors. The local sponsors require very low percentage cost share, and the resultant project facilitate profitable urbanization in the flood prone areas. Therefore, partnership with USACE on flood control project was an attractive option for the local sponsors.

However, in 1973, state statute was changed to one of state-local cost sharing, instead of 100% paid by the state. The benefits of state cost share diminished, and, as predicted, the new policy had a chilling effect on local flood control projects (Kier 1981). It may also partly explain the project gap between 1975 and 1990, as shown in Figure 3.2.1.



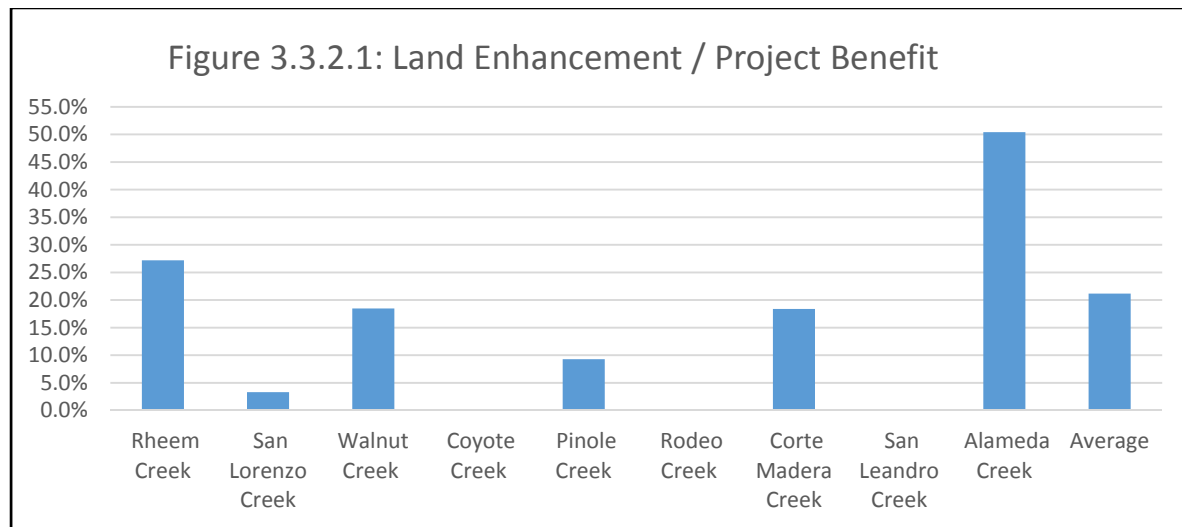


3.3.2 Project Justification – Land Enhancement

For all USACE flood control projects, the ability to demonstrate favorable cost benefit ratio is the key to justify the project for Congressional authorization. The cost portion covers the capital project construction cost, right of way and easement, any productivity and land use loss as a result of the project, and operation and maintenance cost. The benefit portion covers the flood damage avoidance and land enhancement. There are issues on the cost benefit analysis, from how to properly estimate flood damages to ignoring environmental benefits and cost. The most significant issue I identified in this case study is the land enhancement benefit. According to the Inter-Agency Committee on Water Resources’ Proposed Practices for Economic Analysis of River Basin Projects (IACWR 1958), a technical reference known as the Green Book, land enhancement is the benefit resulting from changes in use of property made possible by flood control. It should be measured as the increase, in excess of the estimated reduction of flood damage, in the net income and/or property value of the affected property under conditions expected with and without flood control.

Based on this definition, while the benefit can apply to the net income and property value increases in the existing developed urban areas, as shown in this case study, in most cases I found that the land enhancement benefit is applied to turning undeveloped floodplain into urban development. Developing these riparian floodplain posts ecological and flood risk problems. The developments eliminate existing riparian floodplains which maybe ecologically significant, and at the same time significantly increase the flood control project residual risk by increasing the number of at-risk floodplain developments.

Nevertheless, the land enhancement benefit was commonly applied to the flood control projects as a project justification. Of the nine projects reviewed, I found only three of them, Coyote Creek, Rodeo Creek, and San Leandro Creek, did not have the land enhancement benefit in the cost benefit analysis. Figure 3.3.2.1 shows that on average the land enhancement benefit accounted for over 20% of the total project benefits.



The following is a summary of the land enhancement benefit justification for each project, excerpted from each of the project’s design memorandum or project report.

Rheem Creek: Land enhancement benefits would accrue as a result of providing flood-control for the area. Higher type development would be made possible on 15 city lots and 200 acres of industrial property (USACE 1958a).

San Lorenzo Creek: A major portion of the increase in estimated project benefits results from urbanization of 1953 agricultural areas located within the flood plain (USACE 1958b). Extensive areas of land formerly used for agriculture are now occupied by residential and business properties. This development would extend to an additional area of about 139 acres but for the fact that local authorities have withheld permits until such time as an acceptable degree of flood protection shall have been provided. It is probable that the land would be used for residential building sites (USACE 1954).

Walnut Creek: The proposed plan of improvement would make possible a substantial improvement in land use on an area of about 1,300 acres along the lower reaches of Walnut Creek in the vicinity of State Highway 4, by controlling all floods having a frequency of about once in 100 years. The land north of State Highway 4 is now flooded every two or three years, on the average, and the land south of the highway every 4 to 10 years. As a consequence, present and prospective use of the area is limited to seasonal grazing land and some agricultural crop production. The area is zoned for light and heavy industrial use, and under proposed project conditions, this use would materialize with a substantial increase in land values (USACE 1963a).

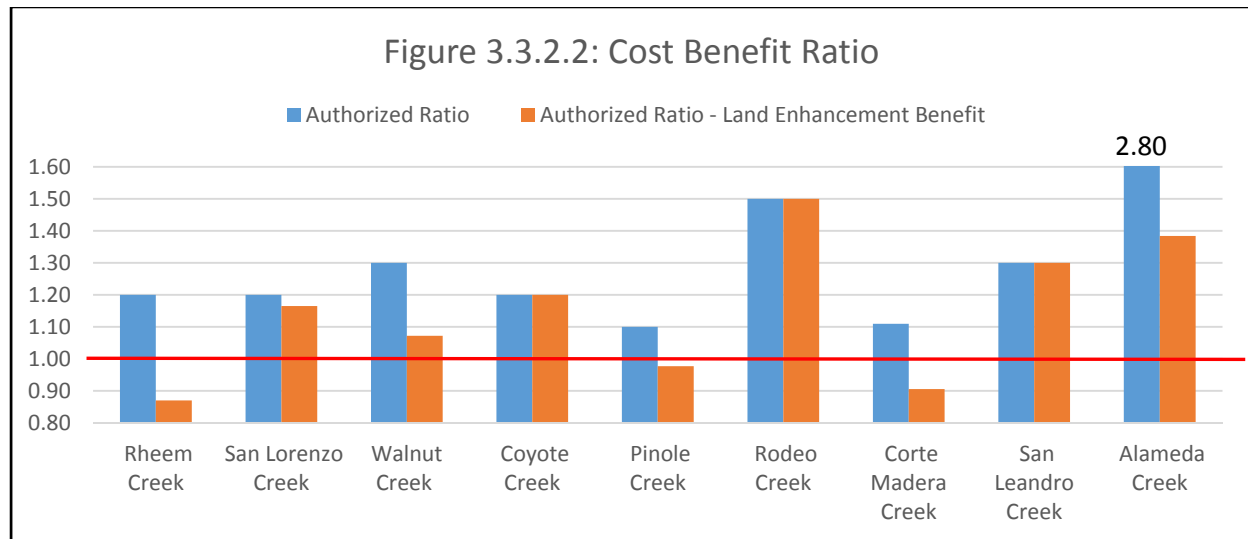
Pinole Creek: There are about 50 acres of marshland in the lower reaches of the basin adjacent to the creek which can undergo a change to higher land use if flood protection were provided. Frequency and severity of flooding currently is such that improvement of the area for higher land use is not contemplated under present conditions. With flood protection, however, the area is projected for light industrial uses (USACE 1962a).

Corte Madera Creek: Prospective benefits associated with the proposed channel improvements consist of reduction of inundation damages, reduction of bank erosion damages, reduction of local bank protection expense, and land enhancement created by reclamation of tidal marshlands (USACE 1966a).

Alameda Creek: With the construction of proposed improvements, it is expected that 2,800 acres of land will yield an increased net annual income of \$40 per acre about 15 years after project construction. It is also estimated that construction of the proposed project will result in conversion of about 5,130 acres of agricultural land to urban development. It is further estimated that 6,030 acres of agricultural land, salt ponds and undeveloped lands will be converted to urban use following construction of the proposed project (USACE 1961).

The land enhancement for these projects mean developing agricultural land to residential or light industrial/commercial land use. For Pinole, Corte Madera and Alameda Creeks, the land enhancement also include conversion of tidal marshlands into urban areas. Since land enhancement is considered as a justifiable project benefit, it implicitly reflects that ecological values in tidal marshland and riparian corridor were disregarded. None of these projects attempted to quantify habitat elimination as a cost entity in the cost benefit analysis. On the other hand, the land enhancement encouraged floodplain development. Floodplain developments increase the flood risk. The increased flood risk under the project design flood frequency level was turned around in the cost benefit analysis and considered as “benefit”, due to flood protection and the resulting flood avoidance provide by the project. At the same time, the residual flood risk above the project design flood frequency level was not included in the cost benefit analysis. Therefore, I found that floodplain and tidal marshland developments provided a significant favorable justification mean for these flood control projects. It is highlighted in the Alameda Creek case study. As discussed in Section 3.2.9, in 1949 the Committee of Public Works directed the USACE review the previous no project recommended, stipulate that the review give full consideration to land enhancement and other factors (USACE 1961). The intent was to increase the cost benefit ratio so a project can be justified.

In Figure 3.3.2.2, I show the impacts of land enhancement on the cost benefit ratio. For each project has land enhancement benefit, I subtracted the land enhancement benefit from the total benefit, and recalculate the cost benefit ratio is recalculated. My recalculation shows that of the six projects with land enhancement benefit, half of them fall below cost benefit ratio of 1.0. In order words, if developing tidal marshland, undeveloped floodplain, and agriculture land in the floodplain were not allowed, Rheem Creek, Pinole Creek, and Corte Madera Creek projects would not have had a favorable cost benefit ratio for project authorization. While Alameda Creek would still have favorable cost benefit ratio, without the land enhancement benefit the ratio is reduced by a half, from 2.8 to 1.4.



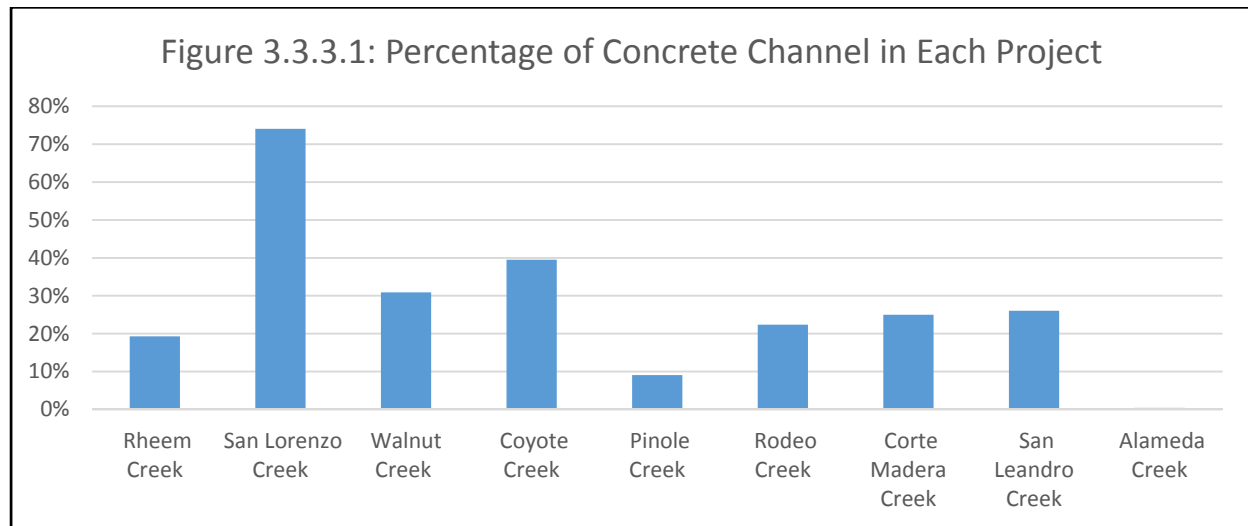
My analysis shows that land enhancement has significant impacts on the project justification. It also provided evidence that if riparian and marsh habitats have equal footing as flood risk reduction in these projects, many of the projects would not have had authorized to proceed in its current form. Note that this analysis did not deduct the flood control project benefits derived from the reduced flood risk in the land enhancement developments. Otherwise, it is likely that even more projects would have had cost benefit ratio below 1.0.

3.3.3 Hydraulic Performance

The hydraulic performance evaluation has two focuses. The first focus is to assess the change in channel capacity since the project was completed. The second focus is to assess if the channel can provide 100 year flood protection. The 100 year flood protection is the minimum standard for the Federal Emergency Management Agency (FEMA) Special Flood Hazard Area (SFHA) delineation for the Flood Insurance Rate Map (FIRM). Areas without 100 year flood protection would require mandatory flood insurance from the property owners.

As discussed in Section 3.2, most of these flood control channels are earthen channels with uniform shape. In locations where there are space constraints, flow restrictions such as bridge crossings, or high flow velocity, uniform shape concrete channel would be built. The nine projects reviewed in this case study have a combined total of 43 miles of flood control channels:

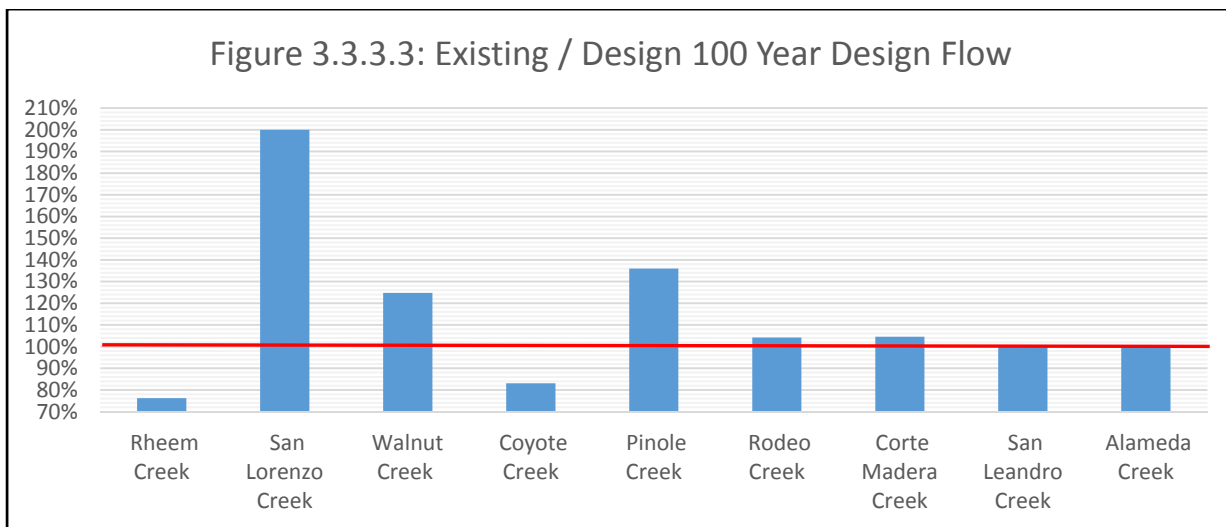
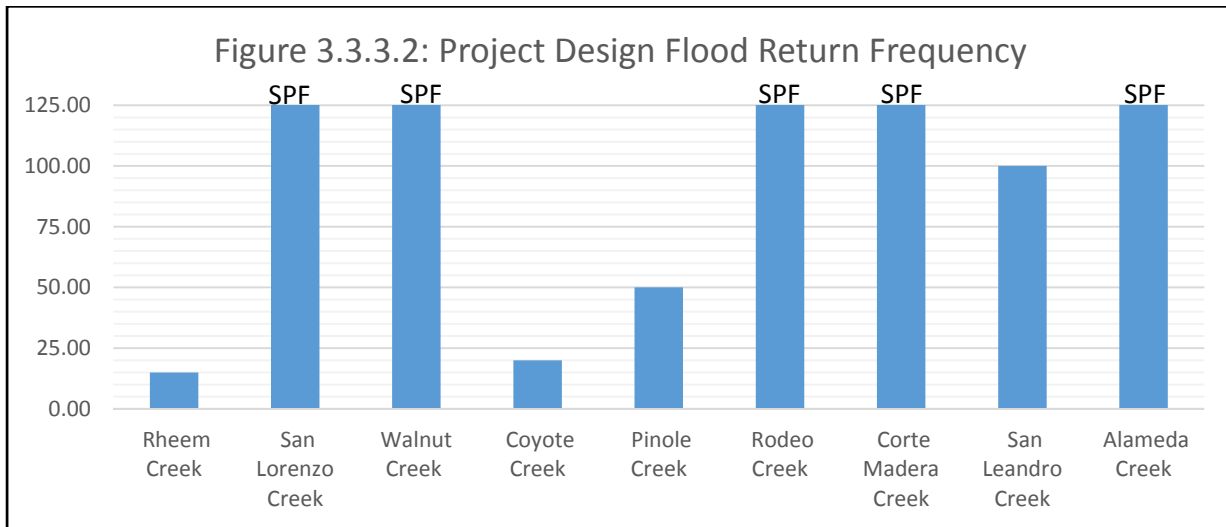
11 miles of concrete channel and 32 miles of earth channel. Figure 3.3.3.1 shows the percentage of concrete channel in each project.



Most of these concrete channels are located at the upstream reaches of the flood control project or at the bridge crossings. The notable exception is Rodeo Creek. Due to the creek corridor restriction at the downtown area, the concrete channel was built at the downstream reaches connected to the channel outfall at the Bay. Concrete channels have high risk of failure, defined as creek flow overtopping the banks and adjacent lands. It is because most of the concrete channel reaches in these flood control channels were designed under supercritical flow condition. The only exceptions is San Leandro creek, where the concrete channel design flow has high Froude number but still under subcritical flow condition (USACE 1970), and Alameda Creek, where the project has almost no concrete channel. For these supercritical flow channels, the channel design only has 1 foot of freeboard based on the supercritical flow water surface. If there is any hydraulic variable causes the concrete channel to flow under subcritical flow, there will be overflow flooding due to the hydraulic jump and the increased creek water surface elevation. Such high risk design highlighted the issue that these channels are precisely designed for efficient creek flow conveyance. The design is highly vulnerable to failure because it leaves little margin of safety. Therefore these flood control controls need significant maintenance to constantly keep the channels in its unrealistic design specification, namely smooth channel surface with no sediment and debris.

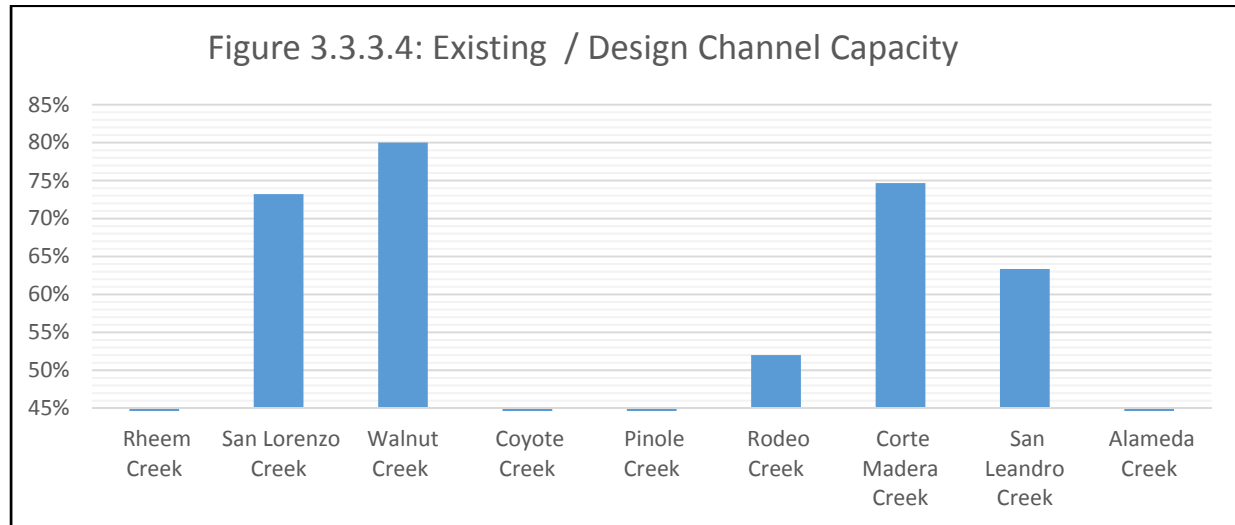
In figure 3.3.3.2, I summarize the design flood return frequency for each project. Except for Rheem, Coyote, and Pinole creeks, all projects were designed for 100 year flood protection or the Standard Project Flood (SPF). In figure 3.3.3.3, I compare the existing and design 100 year flow estimate, and show that five of the nine creeks have higher 100 year flow estimate than the original design estimate. The flow increase is partly due to the urban hydrology as a result of watershed developments. However, in Rheem and San Lorenzo creeks, the change in flow estimate also due to new rainfall-runoff data since the 1950s, when most of these projects were designed. The longer data record changes the probability distribution fitting in the flood frequency analysis, and redefines the probability flow curves (USGS 1982).

The 100 year flow estimates for San Leandro Creek and Alameda Creek have not been adjusted upward. In San Leandro Creek watershed, the majority of watershed at the upstream of the two reservoirs is undeveloped, and the downstream channel flow is regulated by the reservoirs. In Alameda Creek, no revised hydrology analysis is available, but in light of extensive development throughout the Alameda Creek watershed, it seems plausible that the runoff would have increased. Studies for San Leandro Creek and Alameda Creek are currently underway to provide information to assess the hydraulic performance of the projects (Saleh 2014).



In Figure 3.3.3.4, I plotted the percentage ratio of the existing channel capacity over the original channel design capacity for each creek. Current channel capacity data for Rheem, Coyote, Pinole, and Alameda creeks are not available so no ratio shown. For the other 5 creeks, the percentage ratios are all below 100%, meaning the existing channel capacity is lower than the original channel design capacity. The average capacity reduction is 31%. While it is possible that during high creek flow, the earth channel would scour and temporarily increase the channel capacity, as documented in the San Lorenzo River in Santa Cruz during the 1982 storm (Griggs et al, 1982), the capacity data analyzed did not include the scouring effects.

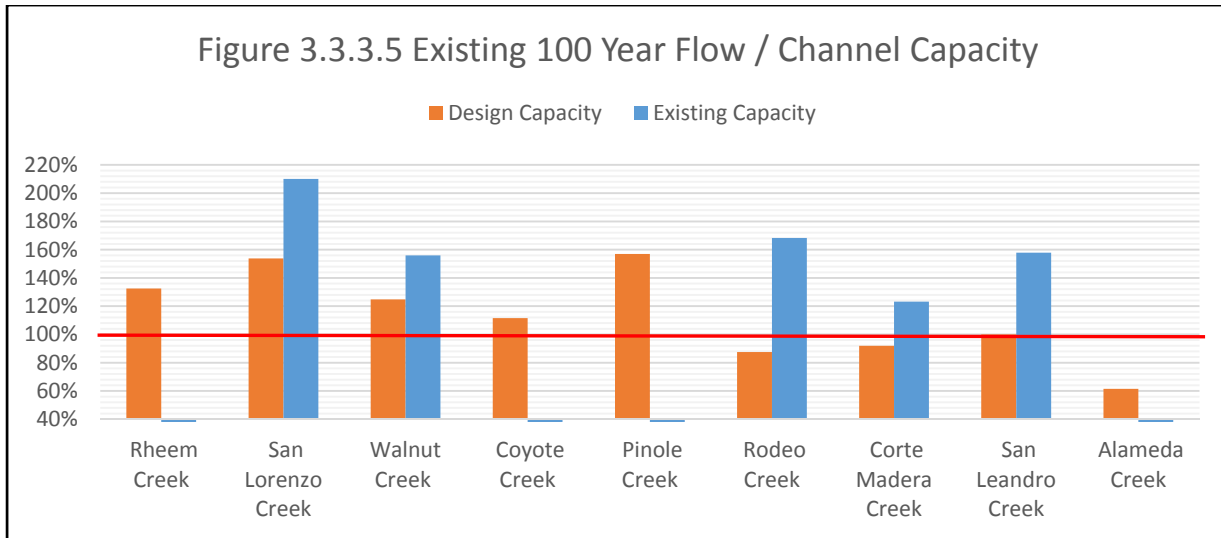
As I further discuss in Section 3.3.4, the capacity reduction is mainly due to sediment deposition reducing the effective flow area especially in the tidal zone. Taking Corte Madera creek as an example, in Chapter 4 I provide an in depth analysis on how sediment and sedimentation affects the hydraulic capacity, especially on the concrete channel.



In figure 3.3.3.5, I show the percentage ratio of the existing 100 year flow estimate to the original channel design capacity at each creek. The figure also shows the percentage ratio of the existing 100 year flow estimate to the existing channel capacity at each creek. Percentage ratio exceeds 100% means the channel does not have the 100 year flow capacity. Of the five creeks where existing channel capacity is available, none can provide the existing 100 year flow capacity. Even considering the original design capacity, five of the nine creeks cannot provide the existing 100 year flow capacity. While Rheem, Coyote and Pinole creeks were not designed for 100 year flood protection, San Lorenzo Creek and Walnut Creek were designed for Standard Project Flood. This channel capacity deficiency under the original design condition is a matter of concern. It is because the original design capacity was calculated assuming the channel was free of sediment, and assuming clear water flow with no sediment or debris. My finding means even if the channels were dredged frequently and so maintained to such unrealistic conditions, they still cannot provide 100-year flood protection.

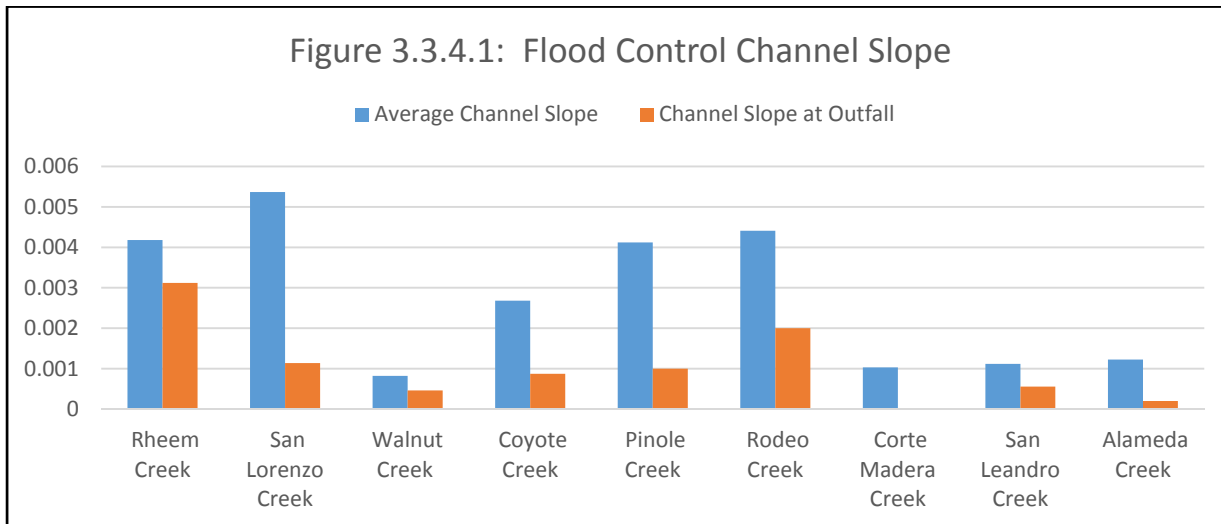
To further compounding the significance of the channel capacity deficiency impacts, all concrete channels except San Leandro Creek were designed for supercritical flow. When these supercritical flow channels overcapacity, the supercritical flow water surface elevation abruptly jumps to subcritical flow water surface elevation. As I show in the Corte Madera Creek example in Chapter 4, subcritical flow in supercritical flow concrete channel reaches create significant channel overtopping.

The project map for each channel shows that the urban floodplain developments encroach to the channel banks. Therefore, in-channel improvement options to adapt a higher design flow is limited, if possible at all.



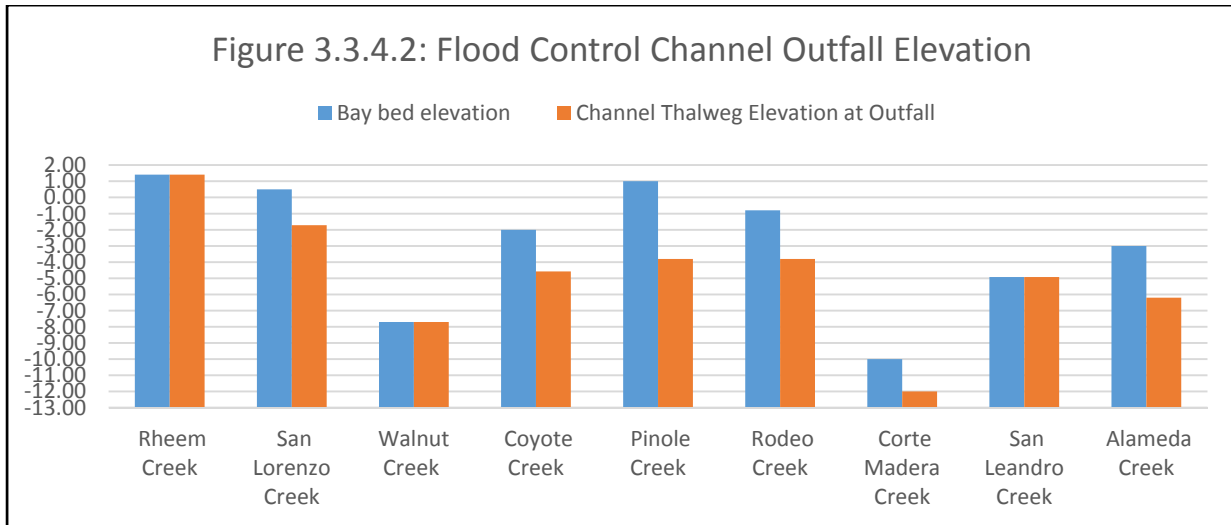
3.3.4 Sediment Management

The flood control projects in this case study are all located in urban areas in San Francisco Bay Region. These urban areas are built on the alluvial fans, with flat topography and at close proximity to the bay waterfront. Naturally, these areas are prime sediment deposition zones subject to fluvial sedimentation due to gentle longitudinal creek slope, and coastal sedimentation due to bay tidal flow. In Figure 3.3.4.1, I compiled the flood control channel slope. Note the extreme case at Corte Madera Creek, where the outfall slope is 0%, extended from the bay outfall for two miles upstream to the Bon Air Road Bridge.



To partly address the slope issue, flood control channels designed with channel thalweg elevation at the outfall below the adjacent bay bed elevation. In figure 3.3.4.2, I show that except Rheem, Walnut and San Leandro creeks, all flood control channels in this case study has this outfall sump design feature. Such design creates a sump at the outfall. Sediment naturally deposits at the sump, reduces the channel slope, the effective flow area, and the channel capacity. Since the

channel bank elevation and freeboard were designed based on the channel thalweg elevation, not bay bed elevation, sediment deposition created flood risk vulnerability at the downstream tidal reaches.

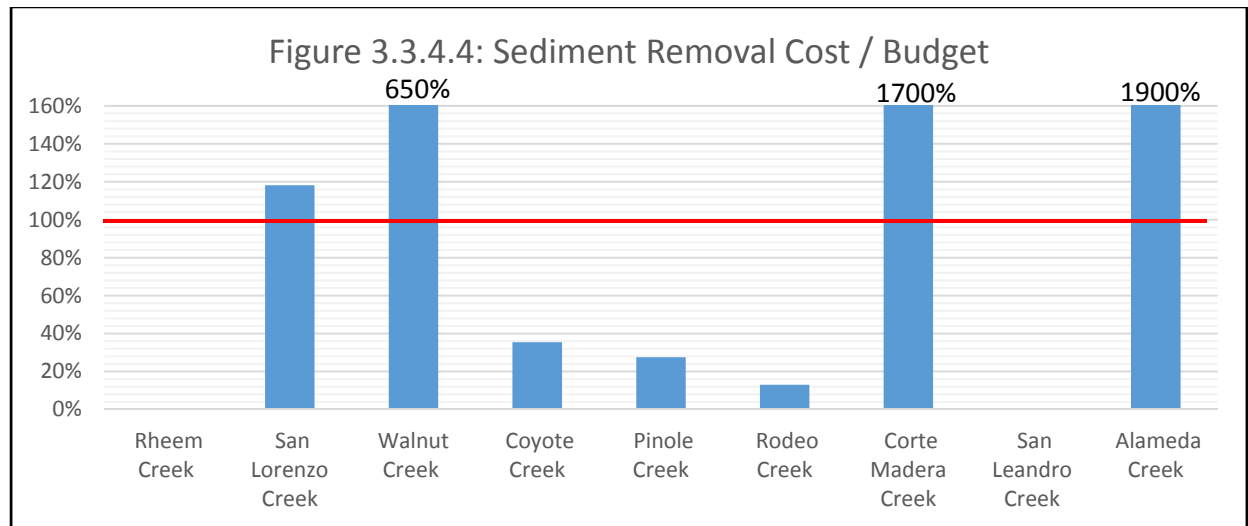
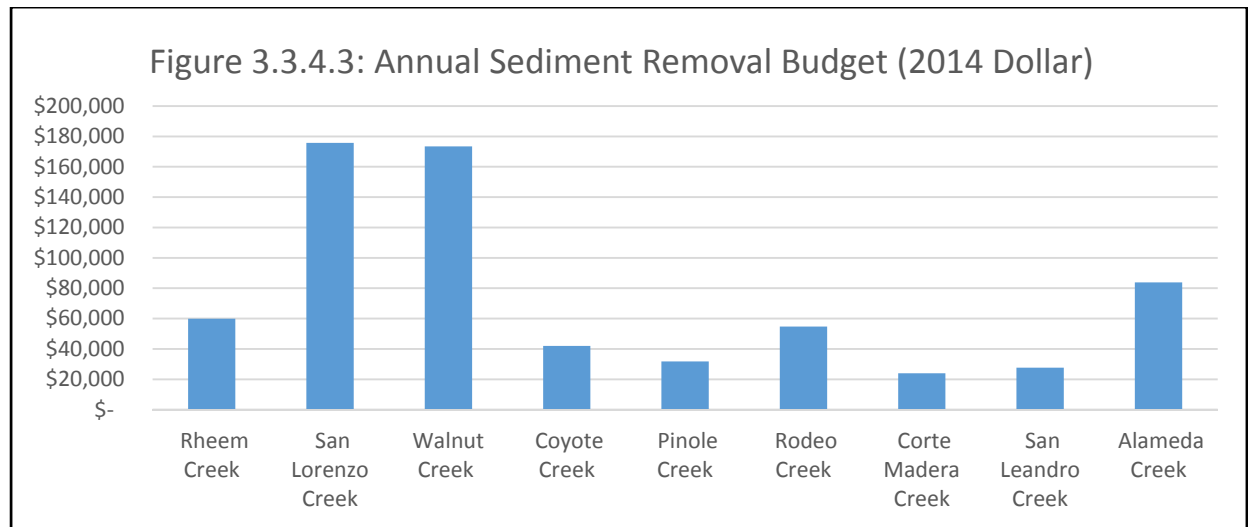


In addition to the slope and outfall design issues, floodplain disconnection is a built-in feature of flood control projects to meet the flood protection objectives. The resultant oversized channel does not have access to the floodplain for sediment deposition. At the same time the restricted tidal prism reduces channel sediment flashing in the tidal zone. As a result, the flood control channels often face with significant in channel sediment deposition.

Sediment deposition did recognized in project design as a maintenance line item. However, Walnut Creek was the only project that had sediment budget and deposition estimate. However, as discussed in Section 3.2.3, it was underestimated and the revision increased the estimate by over 4 times. In all other flood control channels, the sediment management requirements were vaguely noted in the design documents, as to desilt at least once a year. In the project Operation and Maintenance manuals, there is no explicit sediment management instruction, except for Corte Madera, San Leandro, and Alameda creeks, the latest projects in this case study. For these three projects, the maximum sediment deposition depths were specified as a maintenance criteria.

In each project, the economic analysis included the operation and maintenance cost estimate. Some projects had an explicit line item on sediment removal, otherwise it was lumped into the channel maintenance line item. The operation and maintenance cost estimate was typically based on a percentage of the channel construction cost. Such method detach the cost estimate from the reality of the actual operation and maintenance needs. Therefore, its legitimacy is questionable, but yet it was used in the cost benefit analysis to justify the project existence. In figure 3.3.4.3, I show the annual sediment removal budget estimated during the project planning and design, adjusted to 2014 dollar based on the Engineering News-Record Construction Cost Index (ENR CCI) for San Francisco (ENR 2014).

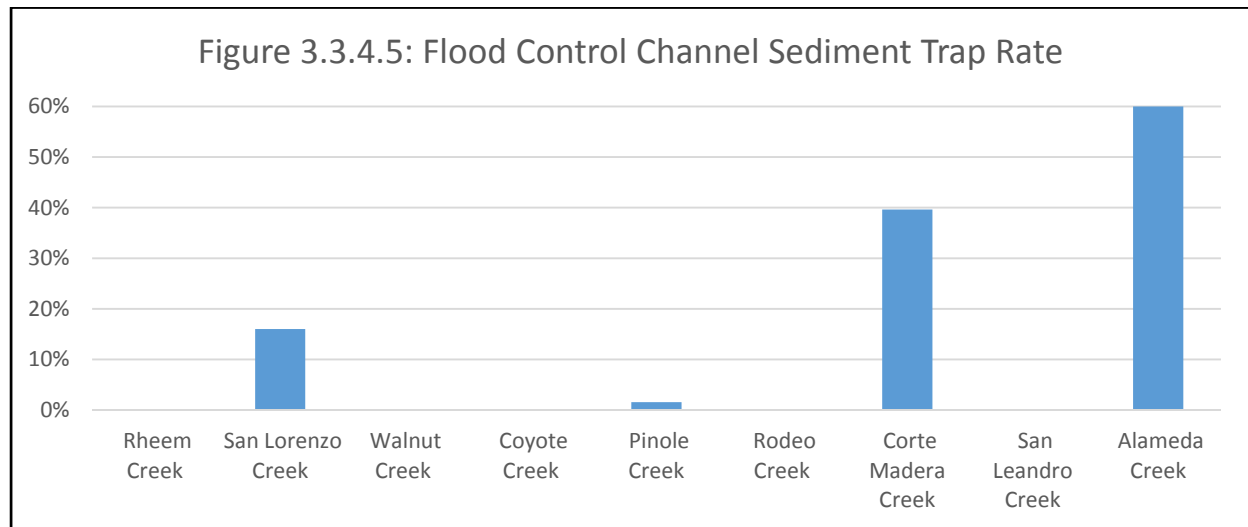
In figure 3.3.4.4, I show the percentage ratio of the current sediment removal annual cost estimate over the original estimate as shown in Figure 3.3.4.3. The current sediment removal annual cost estimate is derived based on the available sediment removal records and sediment deposition survey data. These records and data provide an annual total sediment deposit estimate. For San Lorenzo creek and Alameda creek, the estimate included both channel sedimentation and flood control reservoirs sedimentation. The cost estimate is based on the unit sediment removal cost at about \$20/cy. The unit sediment removal cost is based on the available sediment removal cost data since 1998.



Sediment data for Rheem creek and San Leandro creek were not available, so they are excluded from the analysis. The results show that four of the seven channels have higher sediment removal cost than originally planned. These four channels were the largest projects in this case study, each of them have their own authorization from the Flood Control Act. Sediment removal cost is lower for Coyote, Pinole, and Rodeo creeks. These three channels were smaller projects authorized under Section 205 of the Flood Control Act 1948. In addition, these three channels have smaller watersheds and design flood level. Coyote creek and Pinole creek only designed for

20 year and 50 year flood protection levels. Although Rodeo creek was designed for the Standard Project Flood, the design was based on the restricted flow rate at the railroad crossing. Therefore, the design flow was artificially lowered from 3000 cfs to 2500 cfs. Smaller design flow and watershed means the channel was not as oversized as other projects, so it could have higher shear stress for more efficient sediment transport thorough the flood control channel.

In figure 3.3.4.5, I show the estimated sediment trap rate for the flood control channels in San Lorenzo (DHI 2008), Pinole (SFEI 2005), Corte Madera (Stetson 2000, Stetson 2011), and Alameda creeks (Collins 2006a). The other creeks did not have sediment budget information, so I could not estimate the sediment trap rate. I estimate the sediment trap rate based on the ratio of flood control channel annual sediment supply to channel deposition. The result shows that San Lorenzo Creek and Pinole creek has low sediment trap rate. At Pinole creek, the low sediment trap rate validated that the channel is efficient to transport sediment. At San Lorenzo creek, the low sediment trap rate is due to Cull and Don Castro reservoirs trapped most of the low flow sediment supply from the upper watershed. Under peak flow condition, the reservoirs bypass sediment, but the channel has higher energy to pass sediment thorough the channel.



Note that this analysis is based on available sediment deposition and removal records. Local sponsors have limited resources to remove sediment annually per the operation and maintenance requirements from the project design documents. Channel sediment accumulates over time and gradually reduces the sediment trap rate and channel capacity. Therefore, infrequent sediment removal reduces channel sediment deposition volume. As a result, the current sediment removal cost as shown in Figure 3.3.4.4 is likely underestimated. However, the important point is even with these underestimated sediment removal costs, my estimate shows that local sponsors need to spend \$3.4 million annually to remove sediment from these seven channels. This estimate is five times higher than the original sediment removal cost estimate from the project design. My analysis conclude the fact that these flood control projects grossly underestimated the effects of sediment deposition, and the sediment removal needs.

3.4 Conclusion

In this San Francisco Bay Area case study, I highlighted the issues with the USACE flood control channel projects. I found that six of the nine projects were justified based on the land enhancement “benefits” by ignoring the environmental consequences to develop floodplain, agricultural land, and tidal marshland. I also found that five of the nine projects have higher 100 year flow than the design estimate, due to watershed urbanization and revised flood frequency analysis. Five of the nine projects have existing channel capacity data. All five projects have lower existing channel capacity than the design capacity, mainly due to sediment influencing channel capacity. None of them has 100 year flow capacity.

I found that sediment deposition created significant operation and maintenance burden to local sponsors. On average my current sediment removal cost estimate is 5 times higher than the original sediment removal cost estimate from the project design. In this case study, I justified that flood control channel projects as designed by USACE in the 1950s to 1970s are no longer a viable solution to flood risk management. My analysis identified common problems on project justification, hydraulic performance, and sediment management in my case study channels, many of these problems can trace back to the USACE planning policy issues I discussed in Chapter 2. Local sponsors need a new set of tools to manage these aging infrastructure, and identify options to reinvent these flood control channels.

Table 3.2a: Project Summary Timeline							
Creek	Detail Project Report	Authorized	Design Memo	Constructed	O&M Manual	Post Project Floods	Sediment Removal
Rheem	1958	1959	N/A	1960	1962	None	No Record
San Lorenzo	N/A	1954	1958	1962	1963	Close call in 1998	1975 2000 1975-2003
Walnut	N/A	1960	1963	1965	1968	1982 1983 1986 1995 1997 1998 2002	1973 1986-1989 1993 1995 2006 2007
Coyote	1959	1963	N/A	1965	1965	None	1974 1983 1990 1991 1998 2003
Pinole	1962	1963	N/A	1966	1966	None	2003
Rodeo	1962	1963	N/A	1966	1966	None	1998
Corte Madera	N/A	1962	1967	1971	1988	1982 1983 1986 1994 2005	1986 1987-1998
San Leandro	1970	1970	N/A	1974	1977	None	No Record
Alameda	N/A	1962	1964	1975	1977	None	1975 1981 1982 1984 1986 1989 1998-2001

Creek	FCA Authorized Cost	Cost Basis (yr)	Project Cost	USACE Cost	Local Cost	Land, Easement, ROW, Relocation Cost	Local Cash Cost
Rheem	N/A	1959	\$587.6	\$400.0	\$187.6	\$138.1	\$49.5
San Lorenzo	\$2,665.0	1959	\$8,319.0	\$6,630.0	\$1,689.0	\$1,489.0	\$200.0
Walnut	\$17,980.0	1964	\$31,500.0	\$21,800.0	\$9,700.0	\$7,960.0	\$1,740.0
Coyote	N/A	1959	\$863.0	\$400.0	\$463.0	\$336.0	\$127.0
Pinole	N/A	1963	\$585.0	\$464.0	\$121.0	\$108.0	\$0.0
Rodeo	N/A	1962	\$929.0	\$623.0	\$306.0	\$306.0	\$0.0
Corte Madera	\$5,534.0	1967	\$6,970.0	\$5,100.0	\$1,870.0	\$1,750.9	\$119.1
San Leandro	N/A	1970	\$1,720.0	\$898.0	\$822.0	\$733.0	\$22.0
Alameda	\$14,680.0	1964	\$22,400.0	\$17,140.0	\$5,260.0	\$5,240.0	\$0.0

Creek	Project Life	Discount Rate	Annual Cost	Local Share	O&M	Annual Benefit	Land Enhancement	Cost Benefit Ratio
-	yr	%	\$/yr	\$/yr	\$/yr	\$/yr	\$/yr	\$/ \$
Rheem	50	3.0%	\$27.7	\$13.6	\$5.0	\$33.1	\$9.0	1.2
San Lorenzo	50	2.5%	\$345.3	\$111.5	\$52.0	\$416.0	\$13.8	1.2
Walnut	50	2.9%	\$1,335.0	\$508.0	\$65.0	\$1,755.0	\$324.0	1.3
Coyote	50	2.5%	\$39.9	\$25.8	\$3.5	\$45.8	\$0.0	1.2
Pinole	50	3.0%	\$26.0	\$8.0	\$3.3	\$28.0	\$2.6	1.1
Rodeo	100	2.5%	\$35.3	\$16.3	\$5.5	\$52.8	\$0.0	1.5
Corte Madera	100	3.1%	\$245.2	\$78.2	\$14.6	\$272.0	\$50.0	1.1
San Leandro	50	3.3%	\$74.0	\$37.0	\$4.0	\$95.0	\$0.0	1.3
Alameda	100	3.0%	\$849.0	\$280.0	\$114.0	\$2,370.0	\$1,195.0	2.8

Creek	Watershed Size	Length	Earth Channel	Concrete Channel	U/S Detention
-	square mile	ft	ft	ft	-
Rheem	2	8000	6,394	1,530	None
San Lorenzo	45	27670	7,137	20,373	Cull Canyon; Don Castro
Walnut	146	80256	55,440	24,816	Viano; Kubicek; Pine; Rossmoor
Coyote	3	7522	4,500	2,940	None
Pinole	15	7917	7,145	712	None
Rodeo	10	5808	4,370	1,260	None
Corte Madera	28	20370	15,300	5,100	Phoenix Lake
San Leandro	48	9940	7,100	2,500	Chabot; USLR
Alameda	695	61110	61,668	262	Del Valle; San Antonio; Calaveras

Creek	Ave Slope	Outfall Slope	Bay Bed Elevation	Channel Thalweg Elevation at Outfall
-	ft/ft	ft/ft	ft	ft
Rheem	0.0042	0.0031	1.41	1.41
San Lorenzo	0.0054	0.0011	0.50	-1.72
Walnut	0.0008	0.0005	-7.70	-7.70
Coyote	0.0027	0.0009	-2.00	-4.58
Pinole	0.0041	0.0010	1.00	-3.80
Rodeo	0.0044	0.0020	-0.80	-3.80
Corte Madera	0.0010	0.0000	-10.00	-12.00
San Leandro	0.0011	0.0006	-4.92	-4.92
Alameda	0.0012	0.0002	-3.00	-6.20

Creek	Design Frequency	Qdesign	Location	Q100 design	Location
-	yr	cfs	-	cfs	-
Rheem	15	800	Giant Hwy	1390	Giant Hwy
San Lorenzo	SPF	10400	Bridge St	8000	Bridge St
Walnut	100 (SPF)	25000	SF Bay	25000	SF Bay
Coyote	20	1750	Hwy 1	2350	Hwy 1
Pinole	50	2600	SF Bay	3000	SF Bay
Rodeo	SPF	2500	BNSF RR Track	2100	BNSF RR Track
Corte Madera	SPF	7500	Ross Gage	6600	Ross Gage
San Leandro	100	2800 (8500 w/o dam)	SF Bay	2800 (8500 w/o dam)	SF Bay
Alameda	SPF	52000	SF Bay	32000	SF Bay

Creek	Q100 existing	Location	Qcapacity	Location
-	cfs	-	cfs	-
Rheem	1060	BNSF RR Track (Giant Rd)	N/A	N/A
San Lorenzo	16000	Bridge St	7615	SF Bay
Walnut	31200	Pacheco Ck	20000	SF Bay
Coyote	1952	Tennessee Ck	1830 (no fb)	Tennessee Ck
Pinole	4080	I-80	N/A	N/A
Rodeo	2188	BNSF RR Track	1300	downtown
Corte Madera	6900	Ross Gage (2006)	5600	Ross Ck
San Leandro	2800 (12478 w/o dam)	SF Bay	1774	sample XS
Alameda	32000	SF Bay	<52000	SF Bay

Creek	Sediment Supply Rate	Channel Sediment Deposition Rate	Trap Rate
-	cubic yard/year	cubic yard/year	%
Rheem	N/A	N/A	N/A
San Lorenzo	27259	4370	16%
Walnut	N/A	57481	N/A
Coyote	N/A	757	N/A
Pinole	28367	445	2%
Rodeo	N/A	364	N/A
Corte Madera	53806	21316	40%
San Leandro	N/A	N/A	N/A
Alameda	125300	75300	60% (75-04)

Creek	Design De-siltation Requirements	Max Deposition Depth	Design Sediment Removal Cost	
			\$/yr (design)	\$/yr (2014)
-	-	ft		
Rheem	desilt at least once a yr	N/A	\$5,000	\$59,925
San Lorenzo	desilt at the lower reach	N/A	\$14,665	\$175,759
Walnut	36000cy/yr (1963); 160000cy/yr (1972)	N/A	\$17,000	\$173,487
Coyote	desilt at least once a yr	N/A	\$3,500	\$41,947
Pinole	desilt at least once a yr	N/A	\$3,000	\$31,805
Rodeo	desilt at least once a yr	N/A	\$5,000	\$54,771
Corte Madera	\$2700/yr @ \$0.8/cy = 3375 cy	earth channel = 3.5-7.6, concrete channel = 0	\$2,700	\$24,013
San Leandro	desilt at least once a yr	1.5	\$4,000	\$27,667
Alameda	desilt at least once a yr	2.5	\$8,210	\$83,788

Creek	Sediment Deposition	Sediment Removed	Channel + Reservoir Sediment Deposition Rate	Estimated Sediment Removal Cost
-	cy	cy	cy/year	\$/yr (2014)
Rheem	N/A	N/A	N/A	N/A
San Lorenzo	863133	647233	10567	207658
Walnut	1086700	1327500	57481	1129616
Coyote	7078	25471	757	14875
Pinole	438	7	445	8740
Rodeo	4000	6000	364	7146
Corte Madera	460000	477910	24049	418905
San Leandro	N/A	N/A	N/A	N/A
Alameda	1925717	996078	79314	1558675

Creek	Year	Sediment Removed	Cost	Unit Cost	
-	-	cy	\$	\$/cy	\$/cy (2014)
Walnut Creek	1973	750000	\$940,000	\$1.25	\$6.32
San Lorenzo Creek (res)	1975	330000	\$900,000	\$2.73	\$11.80
Corte Madera Creek	1986	450000	\$2,900,000	\$6.44	\$14.33
Alameda Creek	1998	151,635	\$1,131,974	\$4.15	\$6.70
Corte Madera Creek	1998	22000	\$626,500	\$28.48	\$45.95
Alameda Creek	1999	67,196	\$441,666	\$4.91	\$7.74
Alameda Creek	2000	97,393	\$989,187	\$8.37	\$12.85
San Lorenzo Creek (res)	2000	11300	N/A	\$35.00	\$53.74
San Lorenzo Creek (res)	2000	16000	N/A	\$28.00	\$42.99
Alameda Creek	2001	33,266	\$681,575	\$12.75	\$19.20
San Lorenzo Creek (res)	2003	4033	\$50,000	\$12.40	\$17.69
Walnut Creek	2007	200000	\$5,040,000	\$25.20	\$30.22
Average Unit Cost (since 1998)					\$19.65

CHAPTER 4 – SEDIMENT TRANSPORT AND SEDIMENTATION, THE 5,600 CFS QUESTION AT THE CORTE MADERA CREEK

4.1 Introduction

The San Francisco Bay Area case study of the United States Army Corps of Engineers (USACE) flood control projects drew a number of important findings. I showed that the projects built between the 1950s and 1970s shared common issues with project justification, hydraulic performance, and long term sediment management. I found the projects were justified based on the economic benefits to develop floodplain and marshland without the consideration of their ecological significance. I found that many projects without this land enhancement benefit could not be authorized due to low cost benefits ratio. Due to urban development and new hydrologic data collected since the projects were constructed, I found the design frequency flows were revised and mostly increased, and the channels can no longer provide the design flood protection level. Furthermore, in order to eliminate floodplain overflow during large storm events, the uniform shaped flood control channels were oversized for flow conveyance. Wide thalweg with flat longitudinal slope invites sediment deposition. Sediment deposition reduces channel capacity, and it was underestimated in the project design. In-stream sediment transport increases channel roughness, and the clear water assumption neglected this hydraulic factor in the project design. Therefore the combination of sediment disposition and sediment transport reduce channel capacity below the design capacity. Local sponsors cannot maintain channel sedimentation due to funding and permitting issues, hence the local sponsors inherited aging flood control facilities that no longer provide the promised level of service, nor can the local sponsors afford the ongoing maintenance needed to restore the channel capacity. These non-properly maintained flood control channels provide a false sense of flood protection security to the locals, and the resultant floodplain developments further increase flood risk.

The San Francisco Bay Area case study highlighted the issue of channel sediment. The clear water design assumption underestimated sediment impacts on channel capacity and in channel sedimentation rate, leading to unaffordable long term sediment management needs and channel capacity reductions. Drawing upon this case study finding, a key technical information gap in managing these flood control channels is to have a better understanding of how sediment affects flood control channel performance. To address this knowledge gap, I examine one flood control project in detail to quantify the relative significance of various factors influencing sediment induced channel failure, and the uncertainty in the channel capacity estimate.

In this study, I focus on the sediment induced channel failure. I define channel failure as the channel water surface elevation is higher than the top of bank elevation (Stetson 2011). In this case, the stream overflows to adjacent areas. I define sediment impact as the effects of sediment transport and sedimentation on channel hydraulics, by decreasing effective flow area and increasing channel roughness. In this study, I define channel roughness as the Manning's friction coefficient, or the n factor. The n factor is derived from the Manning's equation for one dimensional water surface elevation estimate under normal flow condition. Manning's equation is commonly used for channel hydraulic analysis and modeling. It is applied in the HEC-RAS modeling software developed by the USACE (USACE 2010).

The sediment transport and hydraulic roughness relationship is partitioned into the following three factors (USACE 2000b, USACE 2000c, Garcia 2007, McKay and Fischenich 2011):

- **Grain Roughness:** In creek hydraulic analysis, flow resistance due to substrate shape and size is commonly quantified as grain or skin roughness. In many gravel bed streams where bed load is insignificant, the total hydraulic roughness due to sediment can be adequately estimated based on grain roughness alone. A number of analytical approaches exist to predict grain roughness from various channel properties. While these approaches are divided into those that account for flow dependency and those that do not, all of them uses a single characteristic grain size to represent bed roughness. Hence, it is assumed constant or insignificant particle size distribution. Flow dependent methods include Leopold and Wolman (1957), Limerinos (1970), Hey (1979), Griffiths (1981), Jarrett (1984), and Bathurst (1985). Flow independent methods include Strickler (1923), Maynard (1991), and Wong and Parker (2006). Among these methods, Limerinos (1970) is adopted in the USACE Engineering Manual for Hydraulic Design of Flood Control Channel, EM 1110-2-1601 (USACE 1994).
- **Form Roughness:** Depending on height, area, and distribution and constituent sediment characteristics, the influence of alluvial bedforms on channel flow patterns can be extremely pronounced. The ability to model the influence of bedforms on hydraulic roughness adds significant complexity to roughness calculations, but consideration of these features is often critical to successfully estimate hydraulic resistance, especially in sand bed channels or gravel bed channel where bedform height generally exceeds grain diameter. In gravel bed stream, form roughness is directly proportional to sediment transport. However in sand bed stream, form roughness is inversely proportional to sediment transport, since increase sand transport reduce bed form magnitude. Three commonly applied techniques includes Brownlie (1983), Engelund and Hansen (1967), and van Rijn (1984). Among these methods, Brownlie (1983) is adopted in the USACE Engineering Manual for Hydraulic Design of Flood Control Channel, EM 1110-2-1601 (USACE 1994).
- **Drag Roughness:** This roughness factor is due to the drag created by particles moving along a smooth channel bed. This hydraulic roughness mechanism was reported by Smith and Mclean (1977), Grant and Madsen (1982), and Wiberg and Rubin (1989). Wiberg and Smith (1991) and Nalluri and Kithsiri (1992) developed equations to estimate hydraulic roughness due to sand bedload transport. Copeland (USACE 2000c) developed an equation to estimate hydraulic roughness due to gravel transport on smooth boundary with no deposition. The study reported a 4% to 20% increase in hydraulic roughness depending on gravel bedload concentration.

Due to the clear water assumption, these factors were not considered in the flood control channel designs that I reviewed in the San Francisco Bay Area study.

To examine these sediment factors and in combination with sediment deposition, I take the Corte Madera Creek in Marin County as an example project for the analysis. As I discussed in Chapter 3, the Corte Madera Creek Flood Control Project shared many similarities on planning, design, and maintenance issues with other San Francisco Bay Area flood control channel projects.

Therefore, while this study is focused on Corte Madera Creek, the findings are relevant to other flood control channels project in the San Francisco Bay Area.

4.2 Corte Madera Creek Flood Control Project Overview

In Chapter 3, I provided a brief overview of the Corte Madera Creek Flood Control Project on project justification, hydraulic performance, and sediment management. This section provides a detailed overview of the project, focusing on the project planning and management history. This overview is mainly based on the baseline report for the project Unit 4 study (USACE 2010), supplemented with additional information from other sources.

The Corte Madera Creek watershed, also known as the Ross Valley watershed, is located in central eastern Marin County, California. The watershed contains 42 linear miles of stream channels, and covers approximately 28 square miles area, including areas of unincorporated Marin County and the towns of Corte Madera, Larkspur, Ross, San Anselmo, and Fairfax. The two major upstream branches of the Corte Madera Creek are Fairfax Creek and San Anselmo Creek. Beginning at their confluence, the stream is known as San Anselmo Creek until it reaches Ross Creek, where it is renamed Corte Madera Creek. The tidal portion of the creek extends several miles upstream from the Richardson Bay to the Kentfield Bridge at the Kentfield Rehabilitation Hospital. The lower ridges and valley areas of the watershed, including areas adjacent to Corte Madera Creek, are highly developed suburban residential and commercial areas. Less than 10% of the watershed remains available for development. Most future development is likely to be from infill and redevelopment. Figure 4.2.1 shows the watershed map with existing land use.

In response to numerous floods in Corte Madera Creek, including the 1942 flood that caused major damage to surrounding communities, Congress directed the USACE to evaluate possible solutions to flooding in the vicinity of Corte Madera Creek under Section 11 of the 1944 Flood Control Act. The USACE completed a preliminary examination report in 1946. Following another major flood in 1951, the California Legislature created the Marin County Flood Control and Water Conservation District (MCFCWCD) through the MCFCWCD Act of 1953. The Corte Madera Creek watershed is identified as the Flood Control Zone 9.

Three additional flood events occurred in 1955, 1958, and 1960, with the December 1955 flood documented as the most severe event prior to the flood control project (FEMA 2009). Following a study of the Corte Madera Creek watershed by the USACE (USACE 1962), Congress authorized the Corte Madera Creek Flood Control Project with the Flood Control Act of 1962:

“The project for Corte Madera Creek, Marin County, California, is hereby authorized substantially in accordance with the recommendations of the Secretary of the Army and the Chief of Engineers in House Document Numbered 545, Eighty- seventh Congress, at an estimated cost of \$5,534,000: Provided, That local interests shall contribute in cash 3 per centum of the Federal construction of the Rose [sic] Valley Unit with a contribution presently estimated at \$158,000.”

The project was originally conceived to consist of six units with a concrete-lined channel extending approximately 6.5 miles from the San Francisco Bay upstream into Fairfax. It was designed to carry the Standard Project Flood at 7,500 cfs, estimated at approximately a 250 year flood event. The USACE completed Design Memorandum No. 1 for improvements to Unit 1 of Corte Madera Creek in 1966, following two additional major flood events in 1962 and 1963. In addition, Congress amended the project under the Flood Control Act of 1966 to reduce the local cash contribution from 3 percent to 1.5 percent. In 1967, the Marin County Board of Supervisors adopted Resolution 92-61, which formally requested the upper project limit be set at the Sir Francis Drake Boulevard Bridge. This action was followed shortly by the USACE's completion of Design Memorandum No. 2 for Units 2, 3, and 4 in 1967.

Unit 4 is the most upstream unit of the Corte Madera Creek Flood Control Project. Unit 4 extends downstream from Sir Francis Drake Boulevard and continues approximately 600 feet downstream of the Lagunitas Road Bridge in the Town of Ross, before terminating at the fish ladder. Unit 3 begins at the fish ladder and continues to the College Avenue Bridge, where Unit 2 then extends downstream to the Bon Air Road Bridge. Unit 3 and portions of Unit 2 are comprised of 5,000 ft of concrete channel, extending for more than a mile downstream of Unit 4. The concrete channel opens to a 3 miles long earth channel in Unit 2, which continues through to Unit 1 where the creek joins Larkspur and Tamalpais Creeks. In Unit 1 the earth channel continues from the Bon Air Road Bridge into Richardson Bay, at the Corte Madera Marsh State Marine Park. Following the completion of flood control improvements to Unit 1 in 1968 and Unit 2 in 1969, the area experienced another flood event in 1969. Unit 3 was completed in 1971. Figure 4.2.2 shows the project alignment and stationing.

Construction of Unit 4 as a concrete channel was originally scheduled to begin in 1972, but it was delayed by litigation. In 1971, Towns of Fairfax and San Anselmo decided to drop out from the project, since both towns did not want to turn the natural creek to a concrete channel. However, the Town of Ross Council voted to proceed with the project to provide flood protection to the town. In December 1971, a Town of Ross resident Peter Valentine, filed a lawsuit to halt the project. The lawsuit insisted a referendum should be placed on the ballot to allow the Town of Ross voters to decide the project plan. In April 1972, a Marin Superior Court judge ruled in Valentine's favor. The decision was later overturned, however, after the Ross council appealed. The case was finally settled when the State Court of Appeals ruled in favor of the Town of Ross on August 15, 1974. At an interview with Peter Valentine by San Jose Mercury News, Valentine commented that USACE's refusal to consider a more environmentally friendly design doomed the original project. "I told them, 'What we want is a Chevrolet,'" Valentine said. "Their response was 'No. You can have a tank, or you can have a tank.'" (San Jose Mercury News 2006).

Construction was then further delayed due to environmental concerns of property owners whose residences/businesses were directly adjacent to the creek. In 1975, at the request of former Congressman John Burton, the USACE, in conjunction with a citizens advisory committee, restudied Unit 4 to develop an alternative to the concrete channel that would provide the 100 year flood protection instead of the Standard Project Flood, but be less damaging to the natural environment and include an extensive public participation process. This alternative consensus plan, the Royston Plan, was completed in 1977 (Royston 1977), and was evaluated in a Design

Memorandum that was completed in 1980. Despite the completion of the 1980 Design Memorandum for the Royston Plan, California Proposition 13 was passed in 1978, and it limited the ability for Marin County to raise the funding needed for the local share portion of the project. As a result, the balance of the authorized project for Units 4, 5, and 6 were halted pending support from the local sponsor.

In 1982, the record peak flow of 7,200 cfs at the Ross gauge caused considerable damage in San Anselmo, Ross, Kentfield, and Larkspur. The storm peak flow exceeded the 100-year recurrence interval but below the Standard Project Flood of 7,500 cfs. However, the concrete channel in Units 2 and 3 as well as the unimproved Unit 4 project reach overflowed and flood water flowed along Kent Avenue. Subsequent studies (PWA 1983, USACE 1989a, USACE 2000b, USACE 2000c) reveal that since the project was designed based on the clear water assumption, the project did not account for the effects of sediment on the hydraulic capacity: It reduces channel capacity, creates backwater upstream, and moves the hydraulic jump location upstream so subcritical flow overtops the concrete reach. The total estimated flood damages amounted to \$2.25 million in 1983 dollars. In December 1983, the Marin County Board of Supervisors requested the USACE to reinstate the Unit 4 project.

Following another major flood event in 1983 and then 1986, USACE dredged the channel back down to its original design level at -12 feet National Geodetic Vertical Datum of 1929 (NGVD 1929) to restore the flow capacity at the earth channel reaches in Unit 1 and Unit 2. In addition, the Congress authorized the USACE under the Water Resources Development Act (WRDA) of 1986 (US Congress 1986) to proceed with the Unit 4 project based on the Royston plan, and to eliminate Units 5 and 6 projects at the upstream of Sir Francis Drake Boulevard from further consideration. Consistent with the adopted Marin County Resolution 92-61, WRDA 1986 states:

“The project for flood control on Corte Madera Creek, Marin County, California, authorized by section 201 of the Flood Control Act of 1962 is modified to authorize and direct the Secretary to construct the project for Unit 4, from the vicinity of Lagunitas Road Bridge to Sir Francis Drake Boulevard, substantially in accordance with the plan dated February 1977 on file in the office of San Francisco District Engineer. The plan is further modified to authorize and direct the Secretary to construct such flood-proofing measures as may be necessary to individual properties and other necessary structural measures in the vicinity of Lagunitas Road Bridge to insure the proper functioning of the completed portions of the authorized project. The project is further modified to eliminate any channel modifications upstream of Sir Francis Drake Boulevard.”

Following the 1986 WRDA authorization, the USACE developed a draft EIS and several Design Memorandum that addressed community concerns about floodwall heights and alignments in Unit 4. Additional public comments on the documents resulted in further modifications to the proposed flood control improvements in Unit 4, which were documented in the final supplemental environmental impact statement (FSEIS) published in November 1987 (USACE 1987). At MCFCWCD’s request, a supplemental information paper titled Supplemental Information Paper I (SIP I) was developed and released to the public in 1988 in response to the public comment on the FSEIS (USACE 1988b). Supplemental Information Paper I identified the public concerns about the completion of Unit 4, as well as concerns about channel flow capacity

and the effects of sedimentation in Units 2 and 3.

Based on the public concerns raised in SIP I, the USACE Waterways Experiment Station (WES), now known as the USACE Engineer Research and Development Center (ERDC), conducted an extensive sedimentation study in 1989 (USACE 1989a). It determined the flow capacity in the existing concrete-lined channel to be significantly less than the 100-year level of protection (USACE 1989a). As a result of this study, the USACE and the MCFCWCD developed a report titled Supplemental Information Paper II (SIP II), which discussed alternatives for restoring flow capacity in Units 2 and 3 (USACE 1989b).

In 1992, the project was reclassified from active to deferred status pending endorsement of a new consensus plan by the local sponsor. Fully acknowledging the previous studies undertaken by the USACE, the Marin County Board of Supervisors determined that the various alternatives for both correction and completion of the project at a 100 year level of protection were environmentally unacceptable to the community. After several more years of discussion and consideration of alternatives, the MCFCWCD reached agreement on a project that, while providing less than a 100-year level of protection, would minimize impacts on the creek and surrounding lands. It called for a project that would provide up to 5,400 cfs flow capacity while retaining the then-historic Lagunitas Road Bridge and limiting the size of the sediment basin in Ross to the natural channel width.

On March 5, 1996, the Marin County Board of Supervisors adopted Resolution 96-26, which recognized the need to complete Unit 4, and redesign Units 2 and 3, under criteria established by the Zone Nine Advisory Board. The criteria included minimizing the use of concrete, retaining adjacent recreational facilities such as the creek side multi-use pathway, using native plants, enhancing riparian and fish spawning habitat, and maximizing the channel capacity while retaining the Lagunitas Road Bridge as is. The resolution also served as an official request that the USACE proceed with the project at an overall lower level of design protection, at a 40 year level or 5,400 cfs, which would meet the environmental concerns of the community.

Following another major flood event in 1997, the USACE approved the project study plan for Corte Madera Creek and reactivated the project in 1998. In 2005, following a flood event that damaged the fish ladder and indicated flow restrictions at the Lagunitas Road Bridge, the MCFCWCD initiated a reevaluation of flow restrictions and necessary flood risk management measures within the creek. In 2009, the USACE had initiated a General Reevaluation Report for the project.

In 2010, the Town of Ross replaced the Lagunitas Road Bridge out of seismic and traffic safety concerns, and to remove the in-channel hydraulic restriction (California Regional Water Quality Control Board 2009). The project increased the channel capacity at the bridge from 3,500 cfs to 5,400 cfs (USACE 2010). The new bridge's inside opening is about 53 ft wide. The bridge soffit is approximately 0.5 ft below the adjacent floodplain elevation. Depending on the final Unit 4 project plan, the new bridge is expected to have negligible backwater effect for creek flows up to about 5,500 cfs, and a minor backwater effect for flows up to the 2005 flood, or approximately 6,800 cfs (Stetson 2011).

In 2011, MCFCWCD completed a Capital Improvement Plan (CIP) study to address the flooding issues in Ross Valley (Stetson 2011). The CIP includes numerous in-stream improvement to increase upstream flow capacity and bank stability. The CIP also consists of five new detention basins for peak flow attenuation. In addition, the study programmed a 7 to 9 year dredging cycle in the tidal reach to maintain the 2010 sediment deposition level. These channel improvements, detention basins, dredging programs are designed to provide 100 year flood protection, with an assumption that the Unit 4 project will be constructed with 5,600 cfs capacity. The project design storm is based on the December 2005 event, with peak discharge of 6,834 cfs at Ross gage. It is comparable to the 100 year flow estimated from the flood frequency analysis, at approximately 6,900 cfs (USACE 2010). Note that the CIP study did not comprehensively address Corte Madera Creek and lower Ross Creek because this critical reach lies within the USACE's Unit 4 project. The CIP study offered general design criteria and recommendations intended to provide compatibility and continuity between the Unit 4 project and the CIP. The USACE is currently formulating and evaluating alternatives designed to contain floodwaters in Unit 4 reach. The total project cost is \$137 million, with the cost benefit ratio of 0.9 (Stetson 2011).

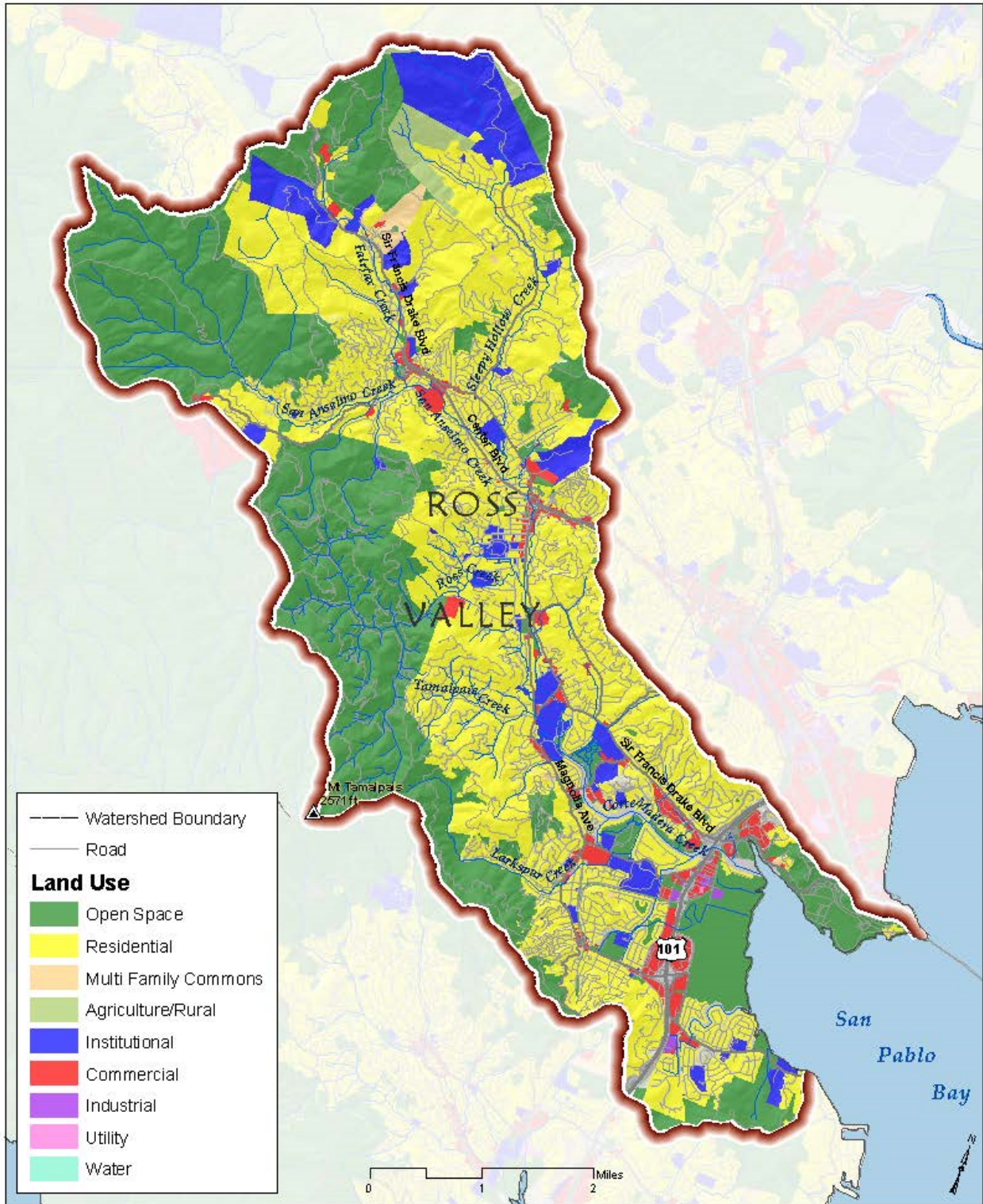
The history of the Corte Madera Creek Flood Control project highlighted the local sponsor's efforts to balance natural stream preservation and flood protection. On the one hand, local citizen groups pushing for ecological sensitive design, while recurring flooding especially along the Unit 4 reach triggered a renewed sense of urgency to move forward with project planning and implementation. However, the hydraulic capacity and sediment management issues in the completed Units 1 to 3 imposed restrictions on the available improvement options. As shown in the 2011 CIP study, the \$137 million watershed-wide program hinged on the assumed flood control channel capacity of 5,600 cfs and the completion of the Unit 4 project. MCFCWCD has an option to proceed with the Unit 4 project as a part of the CIP, but funding limitations and the fact that the Unit 4 project is still likely to qualify for the 1.5% cost sharing formula as grandfathered from the 1966 Flood Control Act provided a strong financial incentive for the local sponsor to continue the partnership with USACE.

Regardless of the partnerships and financing arrangements to proceed with the project, the fundamental question of the 5,600 cfs downstream capacity is critical to the success of the CIP. This design capacity not only hinged on the future Unit 4 project, but also on the existing Units 1 to 3 projects' ability to provide the design capacity. The CIP study concluded that the earth channel has sufficient capacity to convey 100 year flow with no freeboard, under channel sediment deposition bathymetry as surveyed in 2010. However, the study did not discuss whether the concrete channel can provide 5,600 cfs capacity. In addition, the channel capacity is hinged on the interactions of sediment transport and sedimentation along the upstream concrete channel and the downstream earth channel. Drawing upon the volume of sediment analysis and studies on the Corte Madera Creek Flood Control Project since the 1982 flood, the chapter will focus on the sensitivity analysis to quantify the magnitude of various sediment mechanisms influencing channel hydraulics, and a capacity analysis estimate the ability for Units 1 to 3 project to convey the 5,600 cfs flow.

Table 4.2.1 provides a summary of the project timeline. Figure 4.2.3 shows the historical annual peak discharge recorded at the Ross gauge.

Year	Table 4.2.1 – Corte Madera Creek Timeline
1942	Flood Event
1944	FCA Authorization, Sect 11, Flood Study
1946	Preliminary Examination Report
1951	Flood Event
1953	MCFCWCD Act
1955	Major Flooding (biggest pre-project flooding)
1958	Flood Event
1960	Flood Event
1961	Survey Report for Flood Control and Allied Purposes
1961	County Board of Supervisor Approval
1962	Chief of Engineers Report
1962	FCA Authorization, SF Bay to Fairfax Creek
1962	Flood Event
1963	Flood Event
1966	County Board of Supervisor Approval
1966	FCA Authorization, 1.5% cost share
1967	County Board of Supervisor Approval
1967	General Design Memorandum 1
1967	General Design Memorandum 2
1968	General Design Memorandum 5
1968	Unit 1 Construction
1969	General Design Memorandum 3
1969	Unit 2 Construction
1971	Unit 3 Construction
1972	Unit 4 Construction Halted
1974	EIS
1974	General Design Memorandum
1974	State Court of Appeals favor halt Unit 4
1975	Ownership Transfer
1977	Corte Madera Creek, A Flood Control Study
1980	General Design Memorandum 2, Supplement No.1
1982	Flood Event
1983	Flood Event
1984	Energy & Water Development Appropriation Act: Re-dreging
1984	Units 4, 5, 6 reclassified as "inactive"
1985	Cooperation Agreement: Re-dredging
1986	Flood Event
1986	WRDA: Keep Unit 4, delete Units 5, 6
1987	Re-dredging
1987	Ownership Transfer
1987	Unit 4 FSEIS
1987	General Design Memorandum 2, Supplement No.1

Year	Table 4.2.1 – Corte Madera Creek Timeline
1988	Supplemental Information Paper I
1988	Interim O&M Manual
1989	Supplemental Information Paper II
1992	Unit 4 reclassified as "deferred"
1996	County Board of Supervisor Approval
1997	Flood Event
1998	Sediment removal at stilling basin only
1998	Project Study Plan, reactivate project
2000	General Re-evaluation Report, H&H Appendix
2004	Hydrography Survey on Units 1 and 2
2005	Flood Event
2006	Ross Valley Flood Control and Watershed Program Established
2010	Baseline Study
2011	Ross Valley CIP Study: \$122 million CIP projects



Ross Valley Watershed
Land Use

Map for general purposes only; not for site-specific planning purposes.

County of Marin
Department of Public Works

www.marinwatersheds.org



Figure 4.2.1: Corte Madera Creek Watershed Land Use Map (County of Marin Dept. of Public Works 2014)

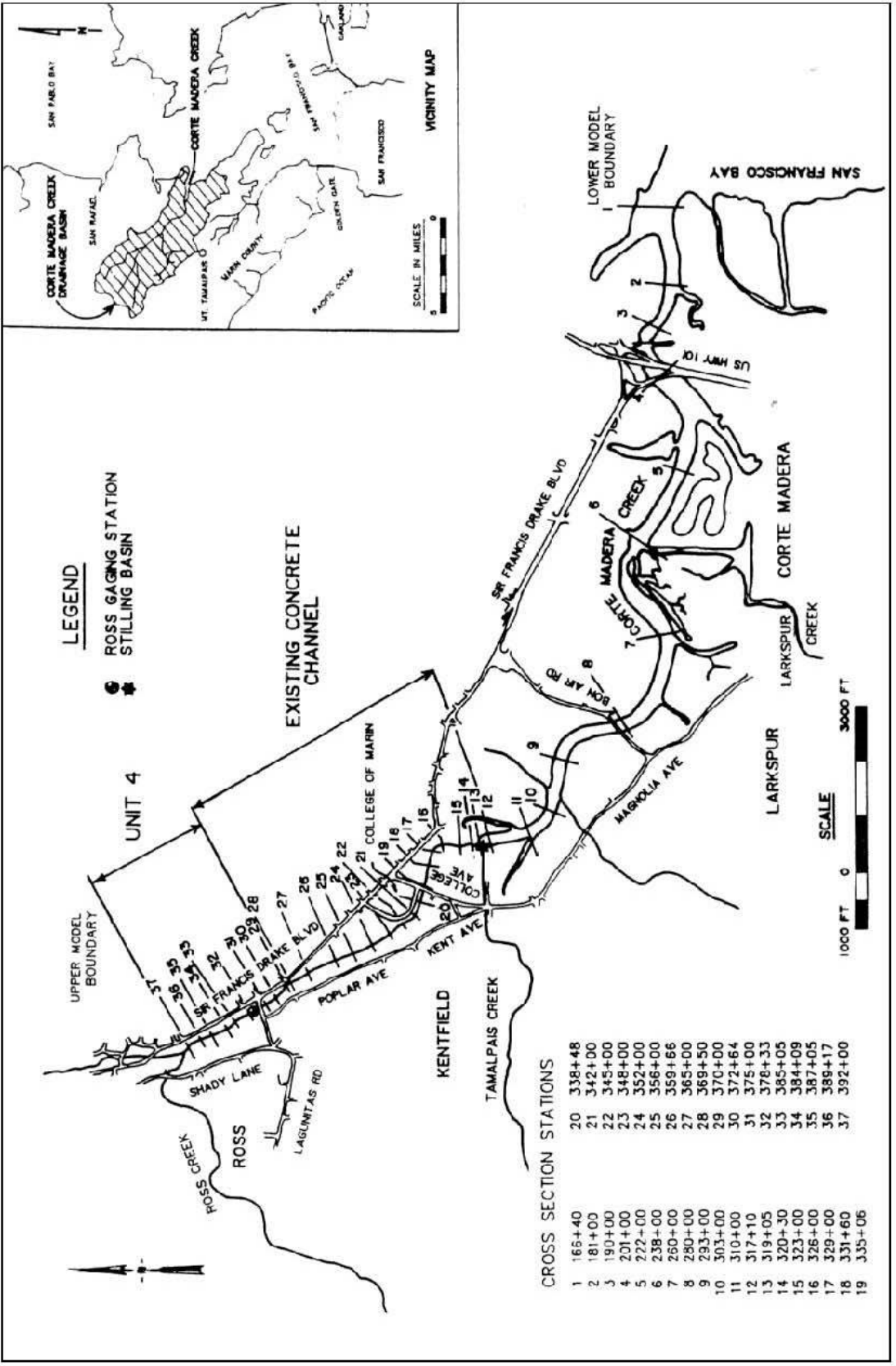


Figure 4.2.2a: Corte Madera Creek Flood Control Project (USACE 2000a)

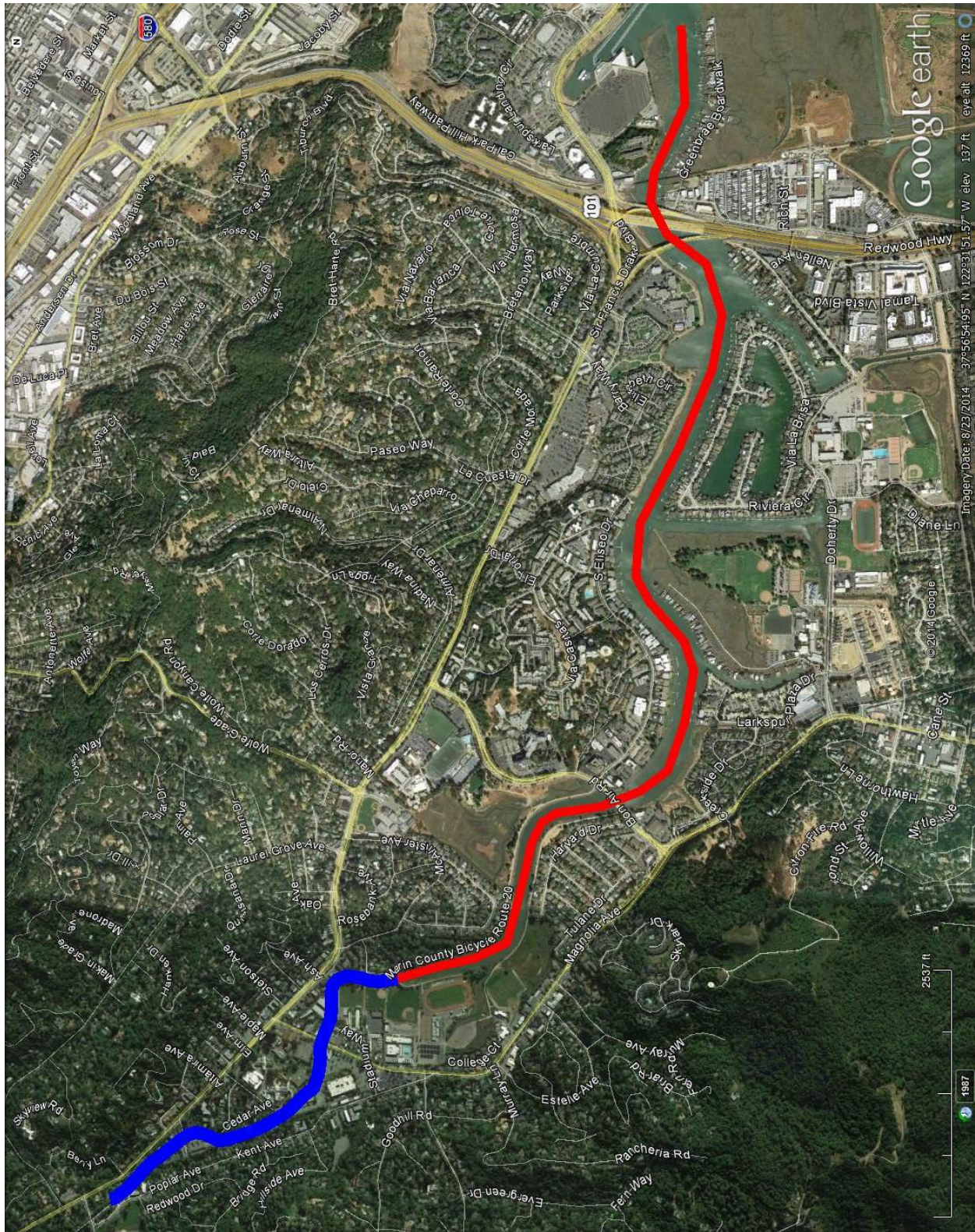


Figure 4.2.2b: Corte Madera Creek Flood Control Project
 Legend: Concrete Channel = Blue Earth Channel = Red

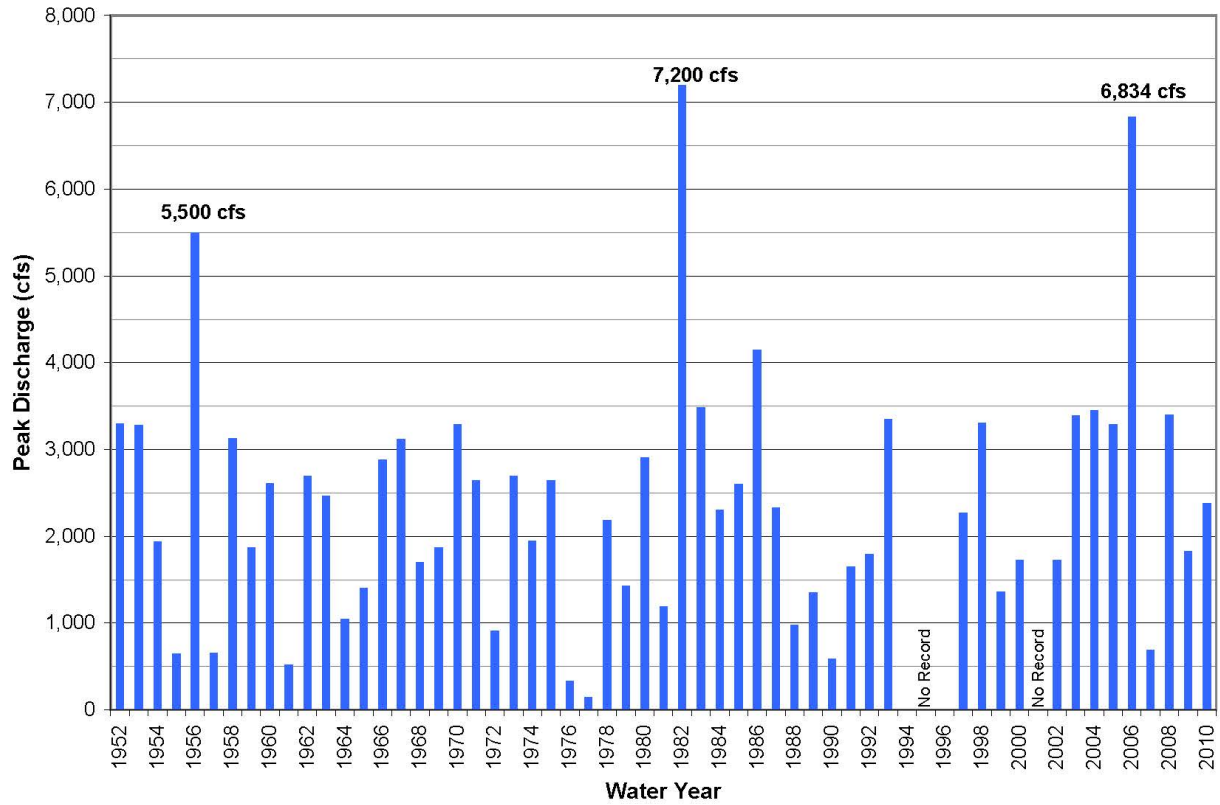


Figure 4.2.3: Corte Madera Creek Annual Peak Flow Record at the Ross Gage (Stetson 2011)

4.3 Sediment Management History

Sediment deposition plagued the flood control channel, especially along Units 1 and 2, where a part of the earth channel has flat slope ($S = 0.00\%$). As noted in the USACE (2000):

“After construction of portions of the project, the channel began to aggrade at rates much greater than anticipated and the District was unable to maintain the channel at its design depth.”

Common to most USACE flood control projects, Marin County Flood Control and Water Conservation District (MCFCWCD) is responsible for the operation and maintenance at Corte Madera Creek. The anticipated operation and maintenance cost (estimated in 1967) was \$14,600 per year (USACE 1967), of which \$2,700 (\$24,000 in 2014 dollar, based on ENR CCI San Francisco data) was devoted to sediment management, to remove approximately 3375 cy sediment per year. Because the Unit 4 project was not completed, the USACE issued an Interim Operation and Maintenance Manual in 1988. The manual did not provide information on sediment budget, deposition estimates, nor dredging frequency. It only specified the threshold sediment deposition depth that would trigger sediment removal, as shown in Table 4.3.1. The table also showed the channel sediment depth estimated from the 2010 hydrography survey (Stetson 2011). The data shows significant sediment deposition along the earthen reaches, but there is only one location, at Station 220+00, where sediment depth exceeded the maximum allowable limit.

Station	Channel	Design Invert	Maximum Allowable Sediment Depth	Sediment Depth in 2010
-	-	ft NGVD	ft	ft
190+00	Earth	-12	5.1	3.45
220+00	Earth	-12	3.5	4.09
260+00	Earth	-12	7.6	5.6
293+00	Earth	-11.9	6.9	6.12
318+10	Earth	-8.8	4.8	-0.88
323+25	Concrete	-9	0	N/A
335+00	Concrete	-6.6	0	N/A

Since the earth channel in Units 1 to 2 projects were dredged in 1966, there were a number of dredging episodes at the flood control channel and Unit 4 reach at the Town of Ross (USACE 2000a, Stetson 2000).

1986 – 1987: USACE and MCFCWCD dredged 450,000 cy at earth channel.
Cost = \$2,900,000

1987 – 1998: Town of Ross dredged 5,910 cy total at Lagunitas Road Bridge.
Total cost = \$63,244

1998: MCFCWCD dredged 13,000 cy at earth channel and 9,000 cy at concrete channel
Cost = \$626,500

Note that the College of Marin Bridge was also dredged in 1972, but volume and cost records are not available (Stetson 2000).

MCFCWCD conducted two hydrography surveys at the earth channel in 2004 and 2010. Section 4.7, excerpted from Noble (2011), contains the cross section survey data from the 1966 original design, 1986 dredging, 2004 and 2010 hydrography surveys. Figure 4.2.2 shows the stationing locations. Approximately 4' to 8' deep of sediment deposited along the earth channel. The upstream sections have higher sediment deposition than the downstream sections. Cross sections 251+33, 236+43 and 222+46 show thalweg alignments shifting from the 1986 dredged position to the pre-dredge alignments in 1966. Based on the survey data, the earth channel has 400,000 cy deposition in 2004. The channel accumulated 60,000 cy sediment between 2004 and 2010, with total deposition of 460,000 cy in 2010.

Based on the combined sediment dredging and deposition data, the annual sediment deposition rate is estimated at 21,300 cy per year. Stetson (2000) provided two sediment budget estimates, with the average total sediment transport rate of 53,800 cy per year. Therefore, the historical sediment trap rate is 45% at the flood control channel. Based on \$20/cy unit dredging cost (based on available dredging cost data since 1998), the annual sediment removal cost is \$419,000. The current sediment deposition estimate is 6 times higher than the design estimate, while the sediment management cost is 17 times higher than the original project budget.

4.4 Sediment Transport and Sedimentation Studies

As mentioned in Section 4.2, the 1982 storm provided an important test to the flood control channel, and it failed to function as intended. Shortly after the resulting flood, MCFCWCD realized the significance of channel sediment and its adverse impacts to the flood control channel capacity. In 1983, MCFCWCD completed a flood analysis to identify the cause of flooding (PWA 1983). The study found that flooding in Larkspur and Ross were mainly due to insufficient capacity at Lagunitas Road Bridge and the Unit 4 reach. In addition, the concrete channel also overflowed, due to increased hydraulic roughness from the presence of sand, gravel, and boulder transported along the channel bed during peak flow. However, the study found that sediment deposition in the concrete channel and the sand and gravel bar at the downstream end of the concrete channel had insignificant influence to the peak flow hydraulic, since the antecedent deposit likely consisted of fine non-cohesive sediments which would wash out before the storm peak. Even if the deposit remained in the concrete channel, water surface elevation change due to sediment deposition alone would not had cause channel overflow. The study also noted that sediment deposition downstream of the concrete reach only increased water surface elevation up to the College Avenue Bridge.

Based on the maximum debris elevations measured along the channel fences after the flood, the peak flow longitudinal water surface profile is estimated and used to calibrate the channel hydraulic model HEC-2, developed by USACE Hydraulic Engineering Center (HEC). The

modeling runs showed that the hydraulic jump occurred in the concrete channel at the downstream of the Kentfield Bridge (Station 35900), instead of at the channel slope change point (Station 32900) per the original design (USACE 1967). The analysis estimated the peak flow of 4000 cfs to 6000 cfs, and the channel roughness value, Manning's n factor, of 0.025 to 0.035 would most closely reproduce observed flood marks. Note that the roughness value is significantly higher than the 0.014 value used for the design under the clear water assumption.

The finding of the 1983 study is significant. It is because the study determined that the concrete channel overtopped because bedload sediment transport increase hydraulic roughness, not because sediment deposition reduce effective flow area in the concrete channel. As discussed in Chapter 2, it is the local sponsor's responsibility to maintain the concrete channel to be free of sediment. Since the study found that sediment deposition was not the primary failure mode, it implied that the concrete channel overflow and the resultant flooding was not the local sponsor's responsibility. Rather, it was the clear water assumption in the channel design that lead to the capacity deficiency.

After the 1983 study, USACE and the local sponsor reactivated the project and developed a new Unit 4 concept design. This design included channel improvements and a new sediment basin at the upstream of the Lagunitas Road Bridge. In addition, the design increased channel wall heights along the concrete channel in Units 2 and 3 to meet the capacity requirements. To test the effectiveness of the sediment basin and the Units 2 and 3 concrete channel floodwall design, USACE prepared a sedimentation study for Corte Madera Creek in 1989 (USACE 1989a). A TABS-1 sediment transport model was developed in the 1989 study. This model did not have the refinements of HEC-2 for water surface elevation analysis. For example, at the supercritical flow locations, water surface elevations were assumed at critical depth, and hydraulic parameters were estimated based on normal depth. However, the model was useful to estimate sediment deposition and its effects on channel hydraulic roughness.

The analysis was based on the 100 year storm, with 6,900 cfs at Ross gage. The concrete channel hydraulic roughness estimate included both grain roughness and form roughness. These estimates were based on sediment gradation data collected in April 1984 at the downstream of the concrete channel. The analysis validated that channel roughness had significant effects on water surface elevation. The analysis also linked form roughness to concrete channel sediment deposition. At the concrete channel below Station 34200, located at the College of Marin upstream of the College Avenue, a roughness coefficient of 0.022 was calculated for a channel free from sediment deposits. This value increased to 0.028 when there was sufficient gravel in the movable bed layer to increase the channel roughness, either by increasing the antecedent flow or by removing the sediment trap. When sediment deposits were not removed prior to the design flood, a value of 0.030 was calculated due to a thicker and coarser deposit. Peak water surface elevation in the concrete channel at the downstream of Station 34200 was directly influenced by the sediment deposition in the downstream earth channel. When deposition in the earth channel was higher than 1.6 times the 1984 surveyed deposition, the deposition increased hydraulic roughness and peak water surface elevation. The study claimed that the findings emphasized the importance of removing gravel deposits on an annual basis to maintain the channel capacity.

The 1989 study triggered a series of comments from the local sponsor (MCFCWCD 1989). The most prominent comment is while the USACE agrees that the channel does not have the design capacity in the 1982 flood due to high hydraulic roughness, the cause of this hydraulic roughness increase was mainly due to sediment deposition in the channel. It implied that the capacity deficiency was largely a maintenance issue. In other words, flooding originated from the concrete channel because the local sponsor did not properly remove sediment from the flood control channel. This is an opposite conclusion from the 1983 study, and it sparks a debate of the dominant failure mode.

Subsequent to the 1989 study, USACE conducted a flume experiment to better understand the effect of sediment transport on channel hydraulic roughness (USACE 2000c). The focus of this study was to quantify the effect of hydraulic roughness increase due to the drag effect caused by gravel bed load moving along the smooth channel. The flume experiment found that the hydraulic roughness increase by 4% to 21% depending on the flow rate and bed load concentration. The study also developed an equation to estimate hydraulic roughness increase due to gravel bed load transport, as a function of bed load concentration. This relationship was applied to an updated hydraulic analysis on the flood control channel (USACE 2000b). The focuses of the hydraulic analysis update were to test the concrete channel hydraulic performance and to assess the Unit 4 project improvement alternatives, to provide channel capacity to contain and convey 5,400 cfs flow to the concrete channel. The 5,400 cfs design flow, approximately 30 year flow, was the proposed design discharge for the Unit 4 project at the upstream of the concrete channel.

In this study, a HEC-6W one dimensional sediment transport model was developed based on the 1989 study. Similar to the TABS-1 model, this model only considers sub-critical flow condition. If the backwater calculations indicate transition to supercritical flow, then the program assigns critical depth for water-surface elevation, but assigns supercritical normal depth for determining hydraulic parameters for sediment transport.

The study found that based on the 1989 “locally preferred plan”, which proposed a floodwall along the Unit 4 project to contain the 5,400 cfs flow within the channel, the contained channel peak flow increase channel scouring and significantly reduce sediment deposition in concrete channel from 879 cy 58 cy. It also translates to lower hydraulic roughness at the concrete channel. The analysis estimated the concrete channel hydraulic roughness n factor, ranges from 0.019 at the upstream end of the concrete channel with no sediment deposition, to 0.031 at the downstream end of the concrete channel with sediment deposition during peak flow. The analysis considered various scheme for sediment removal scheduling, and it showed that frequent sediment removals reduce the hydraulic roughness especially along the mid-reach of the concrete channel (USACE 2000b).

Recognizing the flood control channel cannot provide the original design capacity, and the restriction in Unit 4 at Lagunitas Road Bridge limited most alternative options to 5,400 cfs capacity, USACE and MCFCWCD investigated various sediment maintenance options to avoid the highly ineffective sediment removal scheme to maintain the original design invert elevation. In the Ross Valley Capital Improvement Plan study (Stetson 2011), a 7 to 9 year dredging maintenance cycle was proposed to maintain the 2010 channel deposition profile. The study note

that while this profile does not provide the Standard Project Flood capacity, the earth channel can provide 100 year flow capacity with little to no freeboard.

Figure 4.3.1, from the Ross Valley Capital Improvement Plan study (Stetson 2011), shows the projected reduction in sediment deposition rate over time. It also shows the cumulative sedimentation volume of 390,000 cy at the beginning and 460,000 cy at the end of each dredging period. These two volumes are approximately coincide with the 2004 and 2010 estimated sediment deposition volume in the earth channel. Dredging to the 2004 deposition level was proven to provide the channel with 100 year flow capacity, since 100 year storm in 2005 did not cause channel overtopping. If the capital improvement plan includes tidal prism enhancement options, it could reduce the dredging cycles from 7 years to 9 years.

The sediment management history at Corte Madera Creek highlighted the magnitude and complexity of sediment management in the flood control channel. The local sponsor devised a more efficient sediment removal scheme than the prescription in the Interim Operation and Maintenance manual (USACE 1988a). However, even under the proposed 7 year dredging schedule to provide 100 year flood protection, it still costs the local sponsor approximately \$332,000 per year (or \$6.1 million over 50 years in 5% discount rate). It is estimated based on 72,400 cy dredging volume, at \$25/cy, with \$50,000 mobilization and 25% contingency. The cost is 14 times higher than the original project design estimate on the annual sediment management budget.

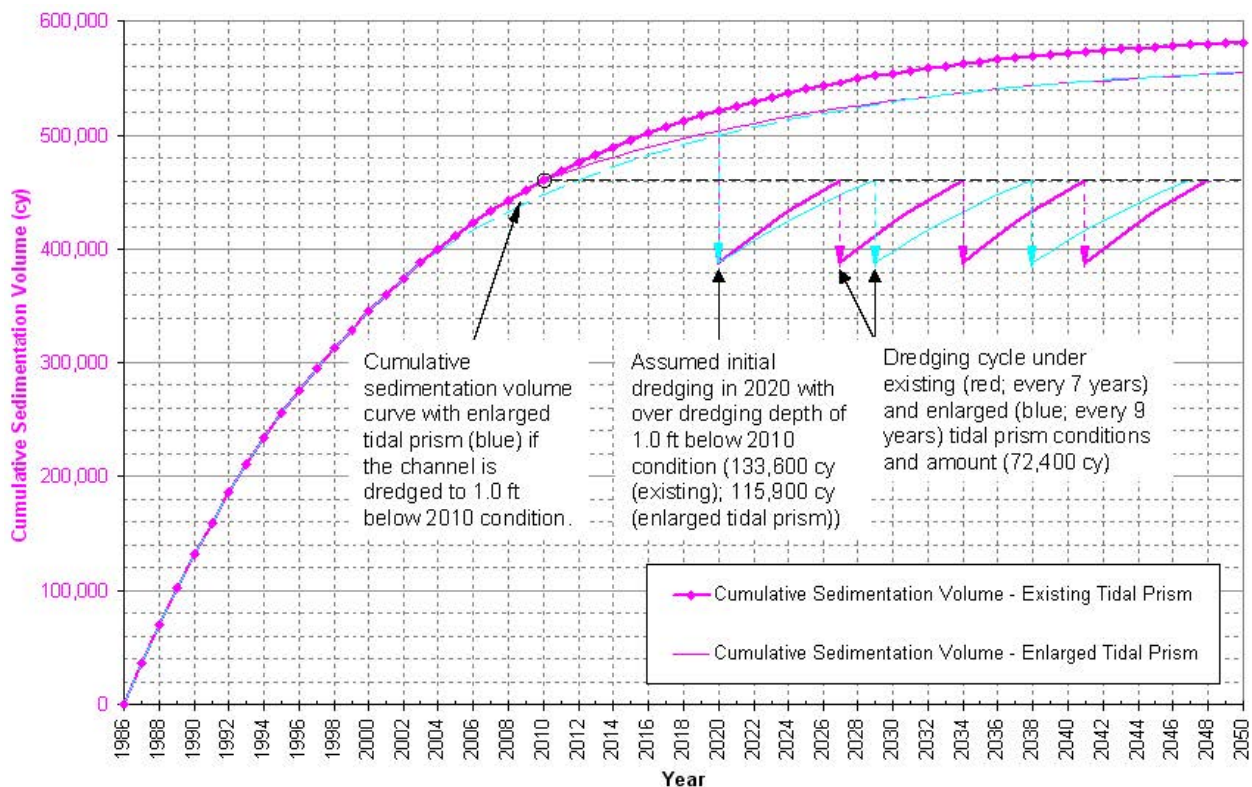


Figure 4.3.1: Proposed Flood Control Channel Dredging Schedule (Stetson 2011)

4.5 Sensitivity Analysis on Sediment Induced Channel Failure

In this section, I document the sensitivity analysis on the Corte Madera Creek flood control channel Units 1 to 3. The purpose of this analysis is to evaluate how various hydraulic and sediment factors affect the channel capacity. In the Ross Valley Capital Improvement Plan Study (Stetson 2011), a hydraulic analysis was prepared to evaluate the earth channel hydraulic capacity based on various sediment removal scheme. However, the modeling effort did not evaluate the Units 2 and 3 concrete channel capacity. Therefore, there is a need to analyze the concrete channel, under the 5,600 cfs minimum required capacity, which is the basis for the Ross Valley Capital Improvement Plan.

4.5.1 Analysis Setup

As I discussed in Section 4.4, numerous studies were completed for the Corte Madera creek on sediment and hydraulic capacity interactions in the concrete channel. The sedimentation study on Corte Madera Creek (USACE 2000b) focused on sediment transport and deposition prediction under existing condition and various upstream flow containment scenarios to convey the 5,400 cfs Unit 4 design flow to the concrete channel. In addition, based on the concrete channel sediment removal time intervals (1, 2, 5, and 10 year), the analysis developed a number of sediment deposition profiles and roughness coefficient estimates during and after the peak of the 5,400 cfs flow. In this analysis, I leverage the modeling data from prior studies to develop a new sensitivity modeling analysis.

The analysis used a USACE HEC-RAS hydraulic model developed by Noble Consultants as part of the Ross Valley Capital Improvements Plan study (Noble 2011). To prepare for this analysis, the HEC-RAS model was revised. The following is a list of the revisions:

- Added Kentfield Bridge at the Kentfield Rehabilitation Hospital, at Station 35969.5.
- Changed the contraction and expansion coefficients at the Kentfield Bridge, College of Marin Bridge, College Avenue Bridge, and Stadium Avenue Bridge to 0.1 and 0.3 respectively. The lowered coefficients prevent hydraulic loss overestimation under supercritical flow condition, and it is validated by field observations that these bridge structures do not create significant obstruction to affect the concrete channel hydraulics.
- Under the original USACE design condition, the n factor changed to 0.014 for concrete channel and 0.025 for earth channel, per the project Design Memorandum (USACE 1967).
- Ineffective flow areas are added to all cross sections in the concrete channel. It is to ensure the analysis only considers the conveyance capacity of the concrete channel, not including the adjacent floodplain area.
- This sensitivity analysis is based on steady state hydraulic runs. Therefore, storage areas and the connected lateral structures are removed. These features do not affect the modeling results.

The following factors are tested in the analysis:

- Channel design flow as upstream boundary condition. In this analysis, I included four design flow scenarios: the original Standard Project Flood flow (USACE 1967), the 1982 storm flow, the 100 year flow and the 5,600 cfs flow from the Ross Valley Capital Improvement Plan study (Stetson 2011). Table 4.5.1 lists the design flow under these four scenarios.

Station	Location	Standard Project Flood	1982 Storm	100 year flow	5600 cfs
36970	Unit 3 Limit	7800	4700	5352	5600
34900	Concrete Channel	7800	4700	5352	5600
33506	College Avenue	7800	5000	5352	5600
32150	Concrete Channel	7800	5000	5753	5600
30590	Earth Channel	8700	8400	8653	6740
28128	Bon Air Bridge	9000	8700	8653	6740
24095	Earth Channel	9000	8700	9680	7767
20196	Earth Channel	9000	8700	11065	9152

- Tide level as downstream boundary condition. The original project design was based on the Mean Higher High Water (MHHW) of 2.9 feet, NGVD 1929. In the USACE Baseline Study for the Unit 4 project (USACE 2010), the MHHW was 3.06 feet. In the Ross Valley Capital Improvement Plan Study, the MHHW estimate was 3.21 feet. In this analysis, I use the MHHW elevation at 3.21 feet. In addition, I use the 10 year tide of 5.81 feet and the 100 year tide of 6.31 feet (FEMA 2009) to test the sensitivity of the tide levels on the channel hydraulics.
- Earth channel sediment deposition and sediment removal maintenance alternatives. In this study, I model three earth channel sediment deposit patterns.
 - No sediment deposition, the channel flowline matches the original design.
 - Sediment deposition based on the maximum deposition level that triggers earth channel sediment removal as per the Interim Operation and Maintenance Manual (USACE 1988a).
 - Sediment deposition based on 2010 hydrography survey profile. It is also the maximum deposition level that triggers earth channel sediment removal as per the Ross Valley Capital Improvement Plan Study. The study found that under this deposition level, the earth channel can provide 100 year flow capacity under MHHW tide level.
- Earth channel hydraulic roughness. The original design specified 0.025 n factor for the channel roughness. Subsequent studies revised the n factor to 0.035. In the 1989 and 2000 sedimentation studies by USACE, the n factor for earth channel was 0.045. The sediment models used 0.045 instead of 0.035 because a higher roughness coefficient is needed to

account for headloss due to silt and clay sediment, which was not included in the sediment transport calculation. In this study, I use the n factor of 0.035 for the earth channel.

- Concrete channel sediment deposition and sediment removal maintenance alternatives. The deposition pattern is based on the findings from the sedimentation study for Corte Madera Creek (USACE 2000b). In this analysis, I use the deposition and roughness coefficient estimates under the existing condition scenario, except for the proposed improvement condition analysis which I will further discuss in the Test 4 section. I use the 1 year and 10 year sediment deposition patterns in the concrete channel during the peak flow. As expected, longer interval has higher sediment deposition. The 1 year pattern represents annual sediment removal, which is required per the Interim Operation and Maintenance Manual to maintain the concrete channel sediment free (USACE 1988a). The 10 year pattern represents the upper end of the maintenance program proposed by the Ross Valley Capital Improvement Plan (Stetson 2011), which calls for maintenance dredging every 7 to 9 years.
- Concrete channel hydraulic roughness. This study modeled four concrete channel hydraulic roughness conditions.
 - Original design condition with n factor of 0.014.
 - 1 year sediment removal interval from Corte Madera Creek sedimentation study (USACE 2000b).
 - 10 year sediment removal interval from Corte Madera Creek sedimentation study (USACE 2000b).
 - Design condition as applied in the Ross Valley Capital Improvement Plan study.

In the Corte Madera Creek sedimentation study, the roughness coefficient was partitioned into three zones:

- Concrete channel upper wall, assumed to be smooth surface with the n factor of 0.014.
- Concrete channel lower wall, with tubeworms and barnacles increase wall roughness, the n factor increases to 0.021.
- Concrete channel bed, with n factor of 0.017 to account for increase roughness from the fish nets and abrasion along the bed, based on the clear water assumption. In addition, there are two scenarios for hydraulic roughness estimate due to sediment impacts. The total concrete channel bed n factor is 0.017, plus the n factor from either one of the following scenarios:
 - Without sediment deposition in channel bed: The roughness coefficient increase due to drag created by sediment moving along the smooth channel bed. The sedimentation study (USACE 2000c) documented an equation on this roughness coefficient component based on the gravel bedload concentration. Flume experiment suggested this bedload drag

factor increase the roughness coefficient by 4% to 21%, depending on the gravel bedload concentration (USACE 2000c).

- With sediment deposition in channel bed: The roughness coefficient increase by a combination of grain and form roughness. Limerinos (1970) method was used to estimate grain roughness. Brownlie (1983) method was used to estimate form roughness. Note that form roughness only apply to the roughness calculation if the channel shear stress is above the critical shear stress based on the Shield number of 0.03. The maximum form roughness apply to the channel is 0.01, when the channel shear stress equal to the critical shear stress based on the Shield number of 0.06.

The composite roughness coefficient is calculated based on the average of these three roughness coefficients, weighted by the perimeter length (USACE 1989a). Based on this method, the channel roughness ranges from 0.018 at the upstream concrete channel sections where there is no deposition, to 0.031 at the downstream concrete channel sections where sediment deposition during the peak flow could be as high as 4.6'.

4.5.2 Analysis Findings

I conducted around 300 model scenario runs under different combinations of tide level, creek flow, roughness coefficients, and sediment deposition on concrete channel and earth channel. In this section, I summarize the hydraulic modeling analysis findings.

Test 1: Original Design Condition

In this first analysis, I rerun the original project design condition under the Standard Project Flood. It is based on the clear water assumption with low n factor and no sediment deposition. Figure 4.5.2.1 shows the water surface profile from the original USACE design (USACE 1967), Figure 4.5.2.2 shows the modeled water surface profile. Both profiles match well on the concrete channel upstream of the College of Marin Bridge, with around 1 foot of freeboard. Between College of Marin Bridge and the channel slope change point at Station 32900, the model shows instability in water surface profile. While it is still under supercritical flow, the Froude number is less than 1.1, meaning this area is at high risk of having hydraulic jump to subcritical flow. Both the model and the design profile shows the hydraulic jump occur at the slope change point at Station 32900, and shows subcritical flow at the downstream of Station of 32900. Note that the critical depth at the upstream of College of Marin Bridge is actually above the channel banks. In theory, it is possible that the channel flow can exceed the top of bank elevation while still under supercritical flow. However, in reality it is highly unlikely since as soon as the creek overtops the bank the flow area spreads out to the overbank and become subcritical flow. This observation on the critical depth level highlighted the high risk concrete channel design which allows little margin of error.

The earth channel roughness coefficient is 0.025 in the original design. Since then, the roughness coefficient has been updated to 0.035. I prepared an additional model run to evaluate the effects of the earth channel hydraulic roughness coefficient on the water surface elevation. Figure 4.5.2.3 shows that the 100 year flow water surface elevation increased from the bay outfall to the College of Marin Bridge. In addition, the hydraulic jump location moved to the upstream of the College of Marin Bridge. However, the change in roughness coefficient does not affect the concrete channel water surface elevation at the upstream of the College of Marin Bridge.

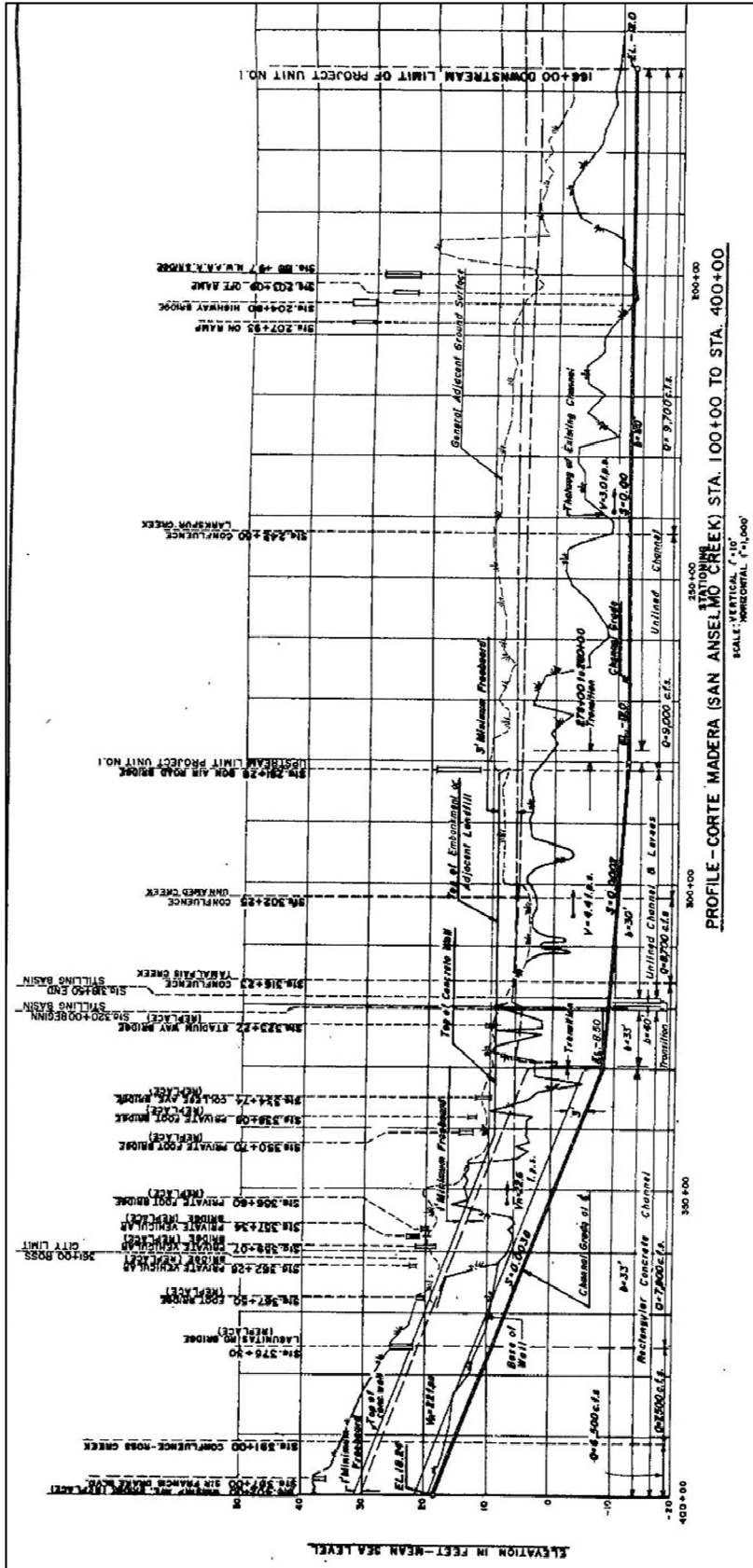


Figure 4.5.2.1: Water surface profile, Standard Project Flood, original design (USACE 1967)

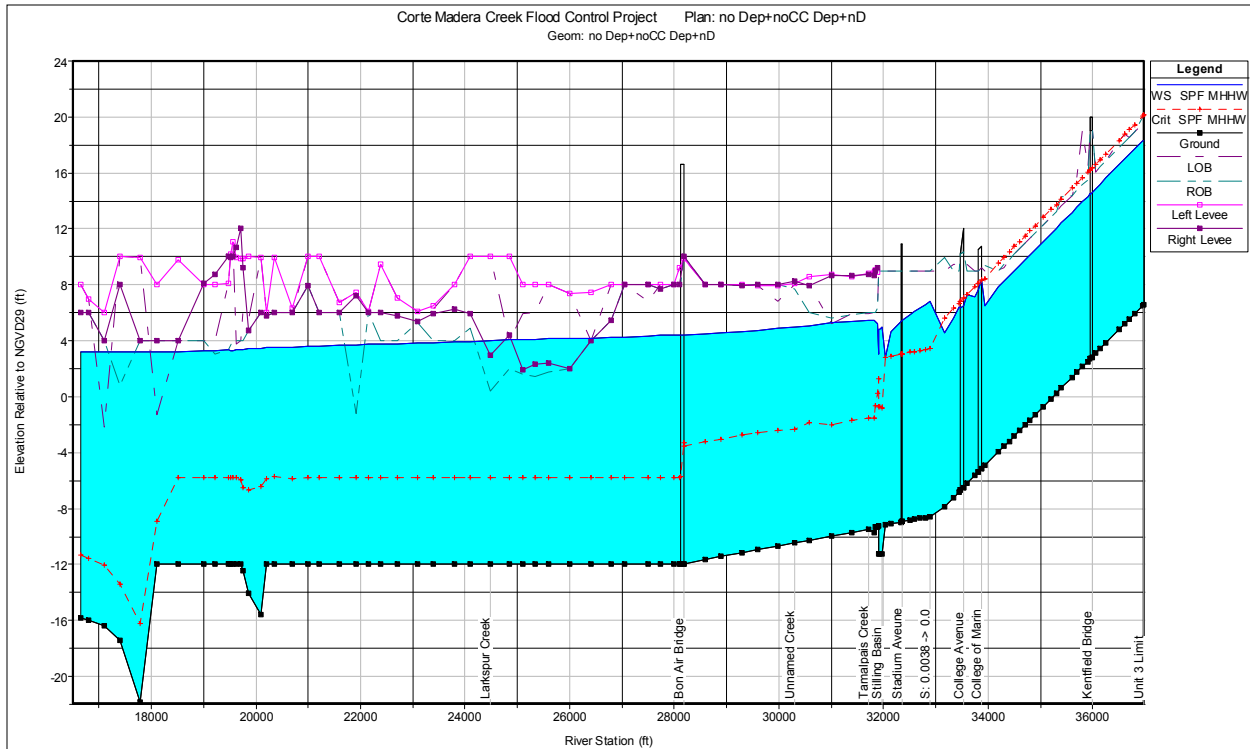


Figure 4.5.2.2: Water surface profile, Standard Project Flood, original design

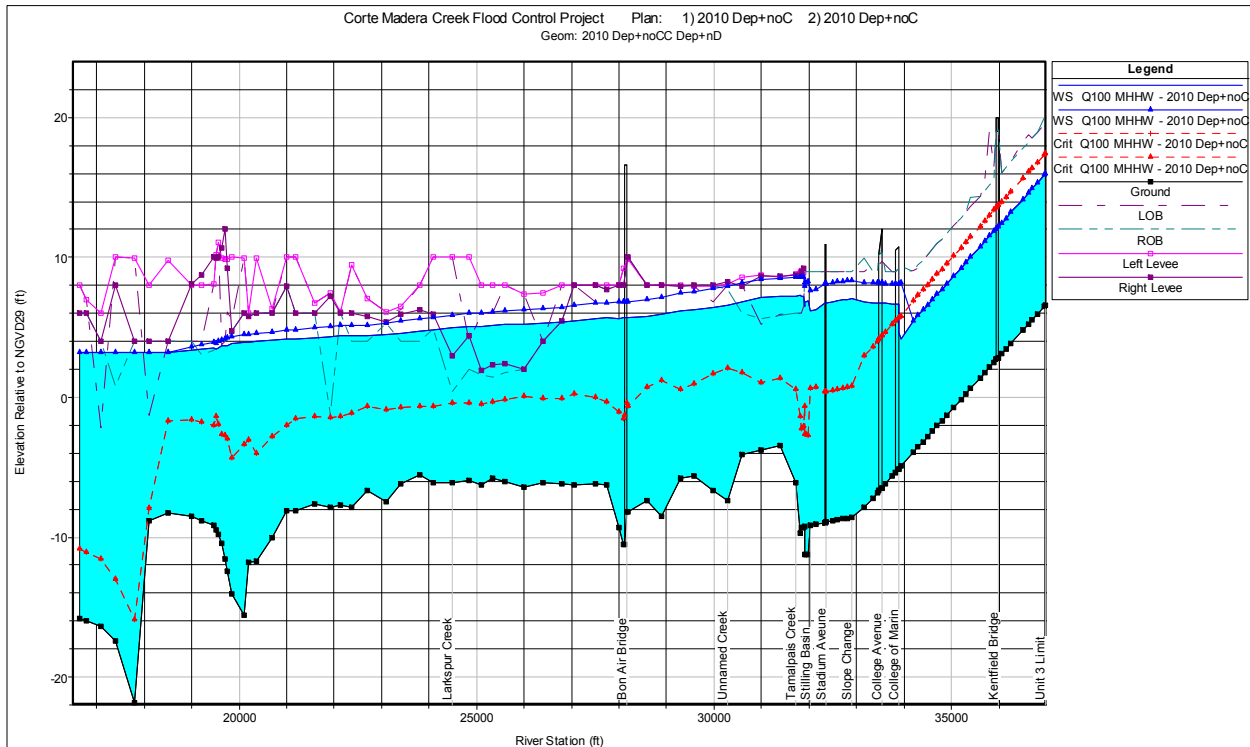


Figure 4.5.2.3: Water surface profile, 100 year flow, earth channel with 0.025 and 0.035 n factors, concrete channel in original design, 2010 earth channel deposition

Test 2: 1982 Storm

In this test, I compare the modeling result with the high water mark data collected after the 1982 storm (PWA 1983). The model run is based on the scenario that the concrete channel carried 4,700 cfs (USACE 2000b) out of 7,200 cfs flow during the peak of the 1982 event, and there was not a significant overland flow return to the concrete channel.

The concrete channel was completed in 1971. There was no sediment removal at the concrete channel since it was constructed. Therefore, it is reasonable to apply the 10 year concrete channel sediment deposition profile (USACE 2000b) to the 1982 storm flow. The model has MHHW tide level, and the Interim Operation & Maintenance Manual earth channel sediment deposition profile. Figure 4.5.2.4 shows the resultant profile. The modeled water surface profile matches the 1982 high water marks.

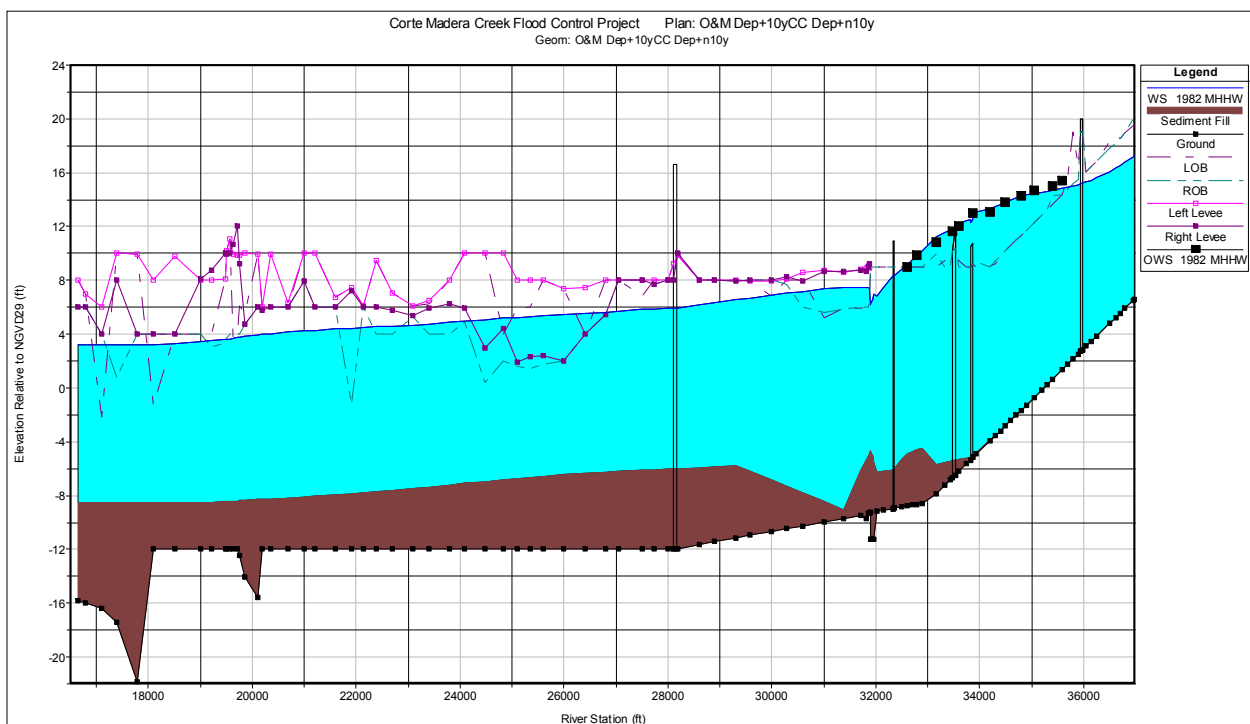


Figure 4.5.2.4: 1982 storm flow water surface profile, with concrete channel sediment deposit

Test 3: Tide Level and Sediment Deposition

In this test, I evaluate the sensitivity of tide level and channel sediment deposition on the water surface elevation, especially along the concrete channel. In this test, I use the 100 year flow as defined in the Ross Valley Capital Improvement Plan hydraulic model. The concrete channel sediment deposition patterns and roughness coefficients are based on the Corte Madera Creek Sedimentation Study (USACE 2000b), Existing Condition scenario. I use the 1 year and 10 year maintenance intervals for sediment deposition patterns during peak flow. In this test, I include three tide levels, MHHW, 10 year tide, and 100 year tide. Table 4.5.2.1 lists the modeling results from various scenarios. The results are summarized in Figures 4.5.2.5, 4.5.2.6, and 4.5.2.7.

I selected two stations to compare the water surface elevations across different scenarios. Station 31810 is located at the most upstream end of the earth channel adjacent to the stilling basin. Station 34300 is located at the upstream of the College of Marin Bridge. It is a low point of the channel bank and typically has the highest overbank water surface depth. The concrete channel flow is subcritical in all scenarios presented in Table 4.5.2.1.

The data shows that by increasing the tide level by 3.1 feet, from MHHW to 100 year tide, the earth channel water surface elevation increases by as much as 1.2 feet at Station 31810. Further upstream at Station 34300, the maximum water surface elevation increase is reduced by half to 0.7 feet. The data also shows that increasing earth channel and concrete channel sediment deposition levels reduce tide level impacts on the water surface elevation. Under 10 year deposition interval in concrete channel, the water surface elevation change due to tide level increase is less than 0.15 feet. Therefore, under the 7 year dredging cycle scenario to maintain the 2010 earth channel deposition level, the tidal influence to the concrete channel water surface elevation is small.

On the other hand, earth channel sediment deposition has a larger impact to the concrete channel water surface elevation than the tide level. At Station 31810, under MHHW tide level, the water surface elevation with the 2010 earth channel deposition level is 1.75 feet higher than the original design profile with no earth channel deposition. At the concrete channel Station 34300, earth channel sediment deposition increases the water surface elevation by as much as 1 foot. Sediment deposition in concrete channel reduces the effects of earth channel sediment deposition. Under 10 year deposition interval in concrete channel, the water surface elevation change due to earth channel sediment deposition increase is only about 0.15 feet.

As I discussed in Section 4.5.1, concrete channel deposition affects the roughness coefficient estimate. Figure 4.5.2.8 plots the roughness coefficient along the concrete channel under 1 year and 10 year deposition. Figure 4.5.2.9 shows the water surface elevation profile with 1 year and 10 year concrete channel sediment deposition, with 2010 earth channel sediment deposition, and MHHW tide. The sediment deposition between the stilling basin and the College of Marin Bridge increases the water surface elevation of the entire concrete channel reach.

The following list the range of concrete channel water surface elevation increase at Station 34300 by each factor:

Tide Level:	0.15 feet to 0.7 feet
Earth Channel Sediment Deposition:	0.15 feet to 1 foot
Concrete Channel Sediment Deposition:	2.6 feet to 3.5 feet

While the tide level and earth channel sediment deposition increase the concrete channel water surface elevation, concrete channel sediment deposition has much larger impact to the concrete channel water surface elevation.

Table 4.5.2.1: 100 Year Flow Modeling Results

Tide Level		Earth Channel	Concrete Channel	Earth Channel Station 31810	Concrete Channel Failure			
		Deposition	Deposition & Roughness	Water Surface Elevation (ft)	Upstream Station	Downstream Station	Length	Station 34300 Flood Depth
1	MHHW	No Deposition	1 Year Interval	6.84	34920	32950	1970	1.91
2	10 YR Tide	No Deposition	1 Year Interval	7.78	35100	32690	2410	2.46
3	100 YR Tide	No Deposition	1 Year Interval	8.05	35140	32610	2530	2.6
4	MHHW	O&M Limit	1 Year Interval	7.79	35080	32730	2350	2.39
5	10 YR Tide	O&M Limit	1 Year Interval	8.33	35170	32550	2620	2.69
6	100 YR Tide	O&M Limit	1 Year Interval	8.54	35210	32470	2740	2.81
7	MHHW	2010 Deposition	1 Year Interval	8.6	35240	32355	2885	2.92
8	10 YR Tide	2010 Deposition	1 Year Interval	8.94	35300	32320	2980	3.12
9	100 YR Tide	2010 Deposition	1 Year Interval	9.08	35320	32280	3040	3.21
10	MHHW	No Deposition	10 Year Interval	6.84	36190	32330	3860	5.44
11	10 YR Tide	No Deposition	10 Year Interval	7.77	36210	32300	3910	5.53
12	100 YR Tide	No Deposition	10 Year Interval	8.04	36220	32280	3940	5.57
13	MHHW	O&M Limit	10 Year Interval	7.78	36210	32300	3910	5.53
14	10 YR Tide	O&M Limit	10 Year Interval	8.33	36230	32260	3970	5.63
15	100 YR Tide	O&M Limit	10 Year Interval	8.53	36240	32240	4000	5.68
16	MHHW	2010 Deposition	10 Year Interval	8.6	36280	32230	4050	5.7
17	10 YR Tide	2010 Deposition	10 Year Interval	8.94	36250	32190	4060	5.78
18	100 YR Tide	2010 Deposition	10 Year Interval	9.08	36270	32170	4100	5.83

Figure 4.5.2.5: Station 31810 Water Surface Elevation

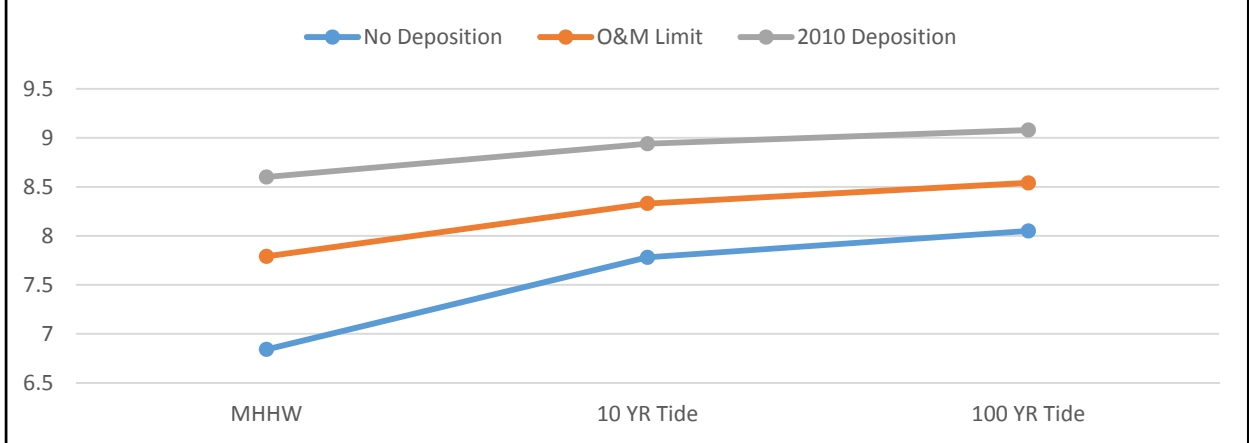


Figure 4.5.2.6: Station 34300 Flood Depth, 1 Year Deposition in Concrete Channel

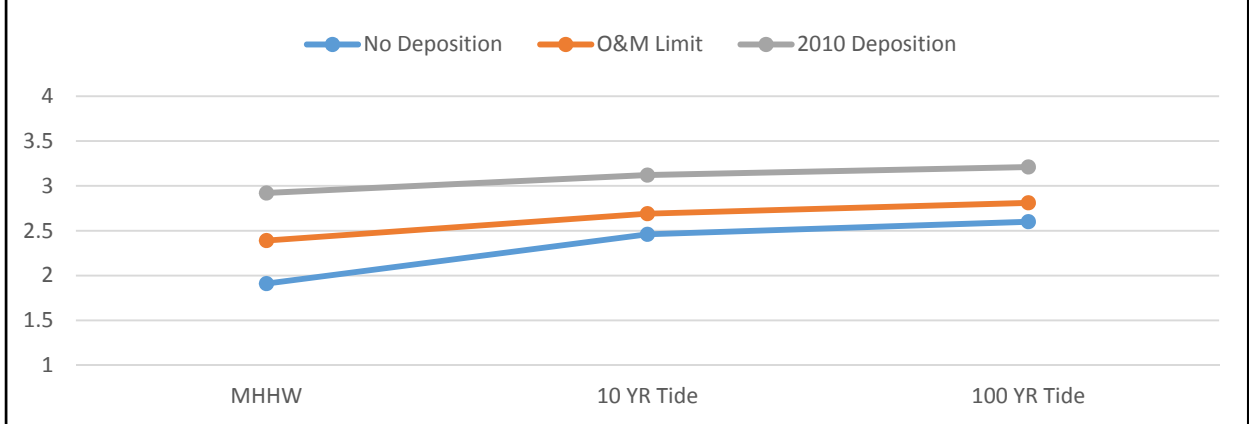
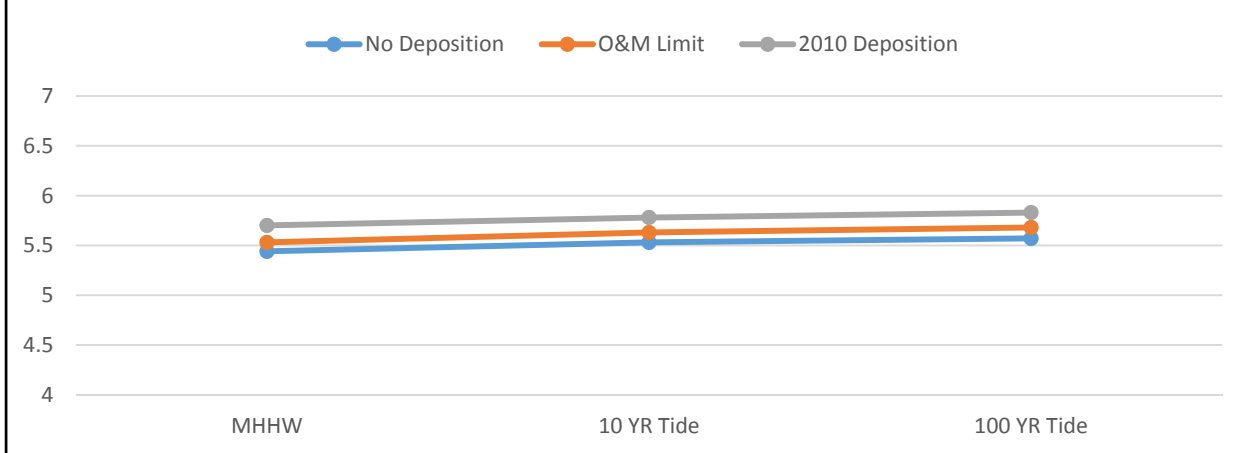


Figure 4.5.2.7: Station 34300 Flood Depth, 10 Year Deposition in Concrete Channel



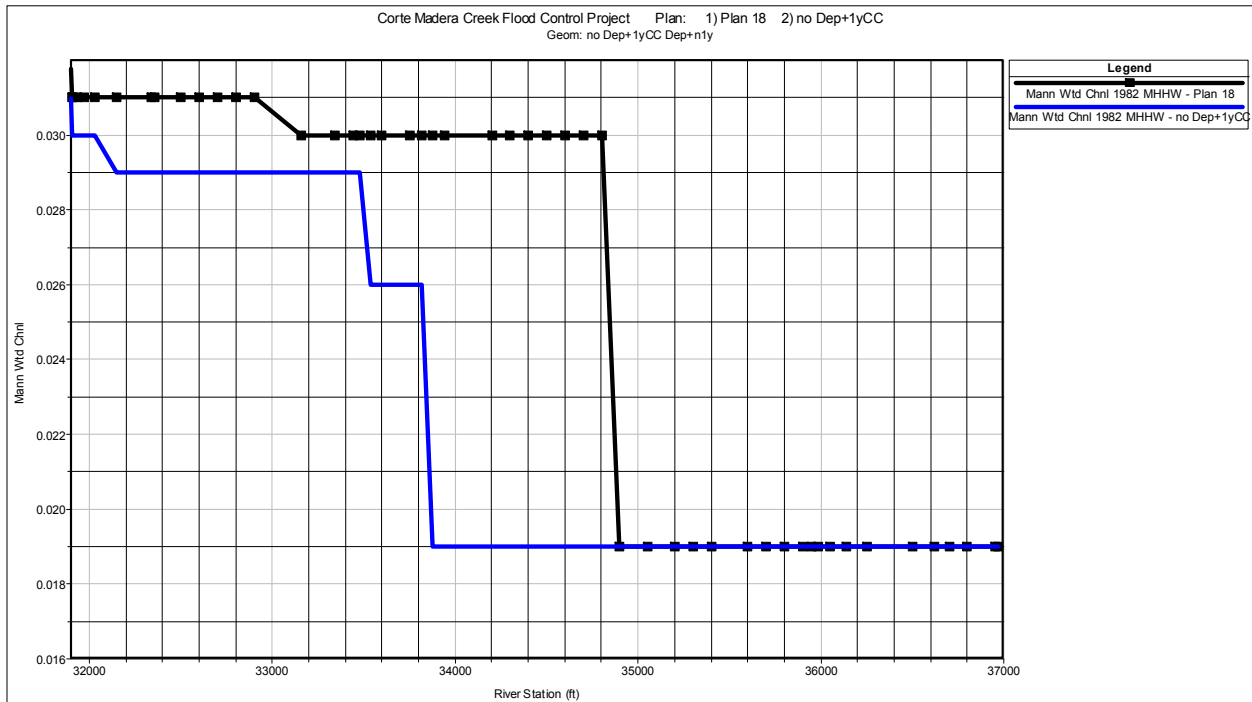


Figure 4.5.2.8: Hydraulic Roughness Coefficient (n Factor) for 1 year concrete channel deposition (blue line), and 10 year concrete channel deposition (black line with square dot)

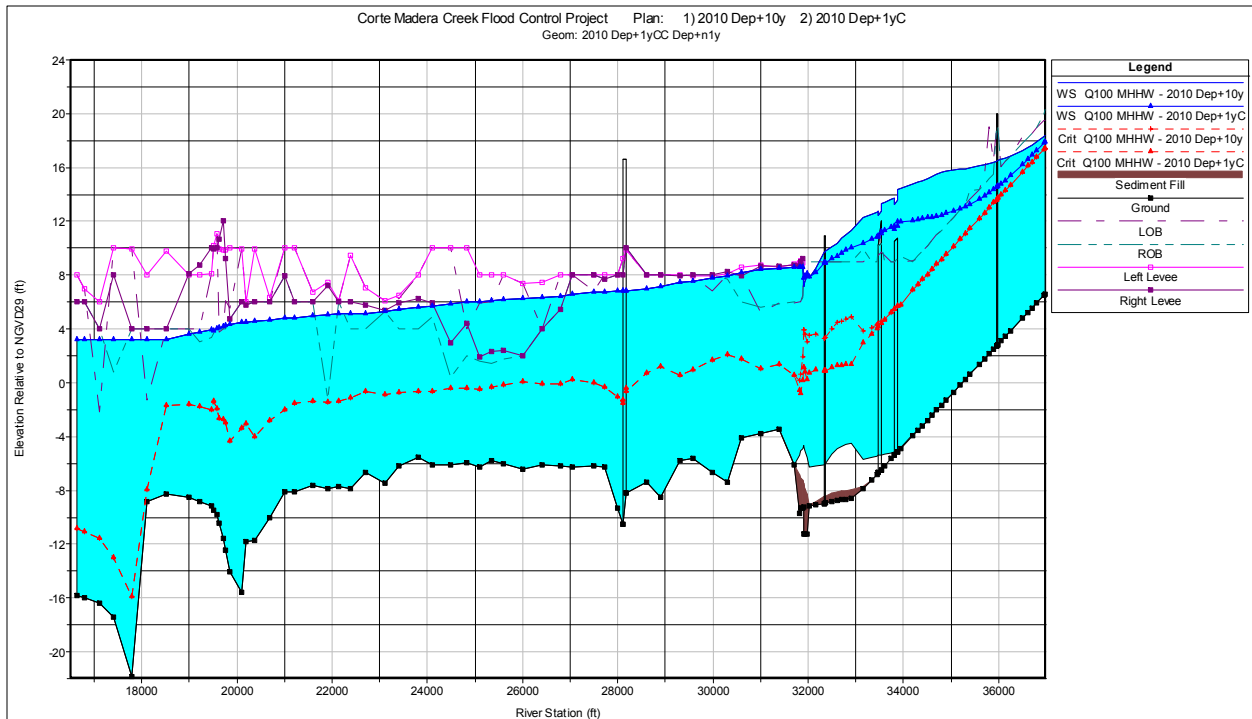


Figure 4.5.2.9: 100 year flow water surface profile, 10 year and 1 year concrete channel deposition

Test 4: 5,600 cfs Design Flow

In this test, I estimate if the concrete channel has sufficient capacity for the 5,600 cfs design flow. Under this scenario, a version of the Unit 4 project would have been constructed to provide the 5,600 cfs flow capacity, so there would be no creek overflow at the upstream of the concrete channel. In the Corte Madera Creek Sedimentation Study (USACE 2000b), since the existing condition scenario assumed creek overflow, this scenario is not applicable for this test. In this test, the concrete channel sediment deposition and roughness coefficient are based on the proposed Unit 4 Type 19 design scenario in the Corte Madera Creek Sedimentation Study (USACE 2000b). The Unit 4 Type 19 design consists of channel improvements and a new sediment basin, so Unit 4 can provide flow containment at 5,400 cfs capacity, and reduce sediment load to the concrete channel.

This test consists of two scenarios: Annual and 10 year concrete channel sediment removal intervals. The earth channel has 2010 deposition profile and MHHW tide level. Figure 4.5.2.10 shows that the concrete channel overtops in both scenarios:

- Scenario 1: Under annual sediment removal interval, concrete channel overtops between Stations 34580 and 32850. Along this 1,730 foot section the maximum floodwall height without freeboard is about 1 foot.
- Scenario 2: Under 10 year sediment removal interval, concrete channel overtops between Stations 36860 and 32520. Along this 4,340 foot section the maximum floodwall height without freeboard is about 5 foot.

The results indicate that even if the MCFCWCD follows the proposed sediment removal plan with 7- to 9- year intervals, the concrete channel will not have sufficient capacity for the 5,600 cfs flow. The results also indicate that increase the sediment removal frequency can reduce the floodwall extend, but the existing concrete channel still cannot provide the 5,600 cfs capacity. To take it one step further. Figure 4.5.2.11 shows the 5,600 cfs flow water surface profile with the deposition limit defined in the Interim Operation & Maintenance Manual (USACE 1988a) and 1 year concrete channel sediment deposition. I show that even if the MCFCWCD follows the maintenance schedule as prescribed in the Interim Operation & Maintenance Manual, the concrete channel still does not have sufficient capacity at the vicinity of the College of Marin Bridge, circled in red in Figure 4.5.2.11.

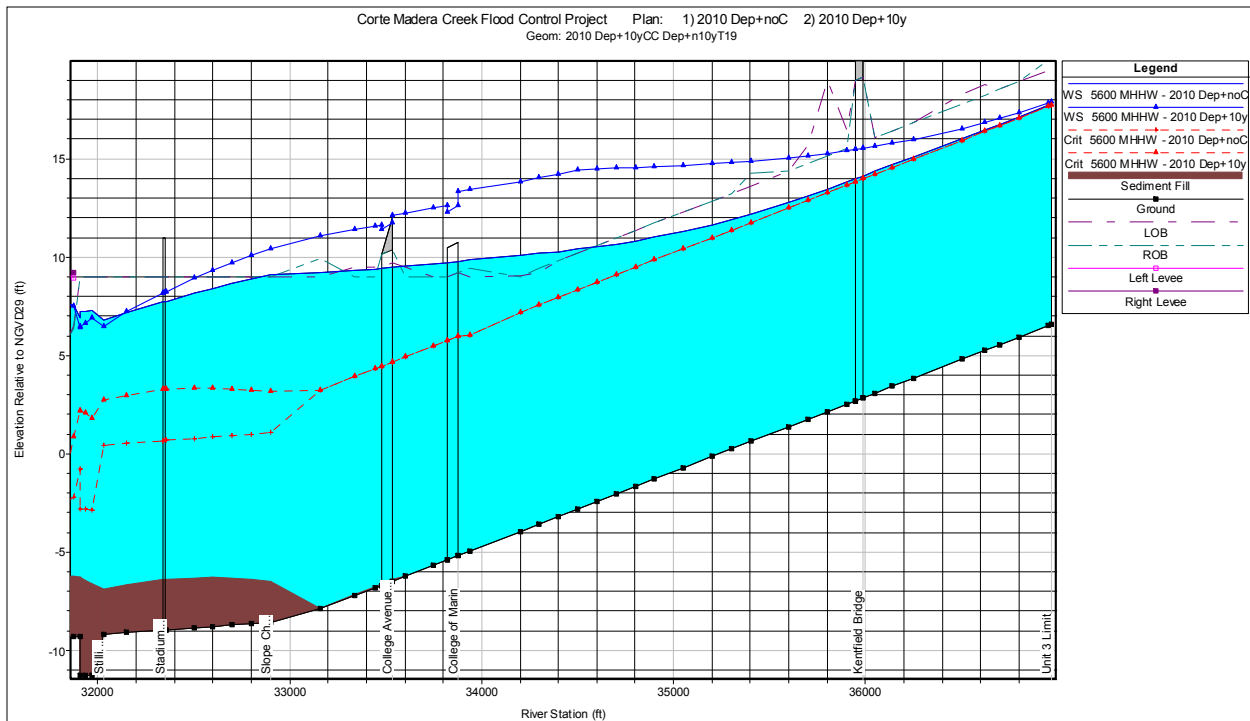


Figure 4.5.2.10: Water surface profile for the 5,600 cfs flow, with 2010 earth channel sediment deposition profile and annual and 10 year concrete channel sediment removal intervals

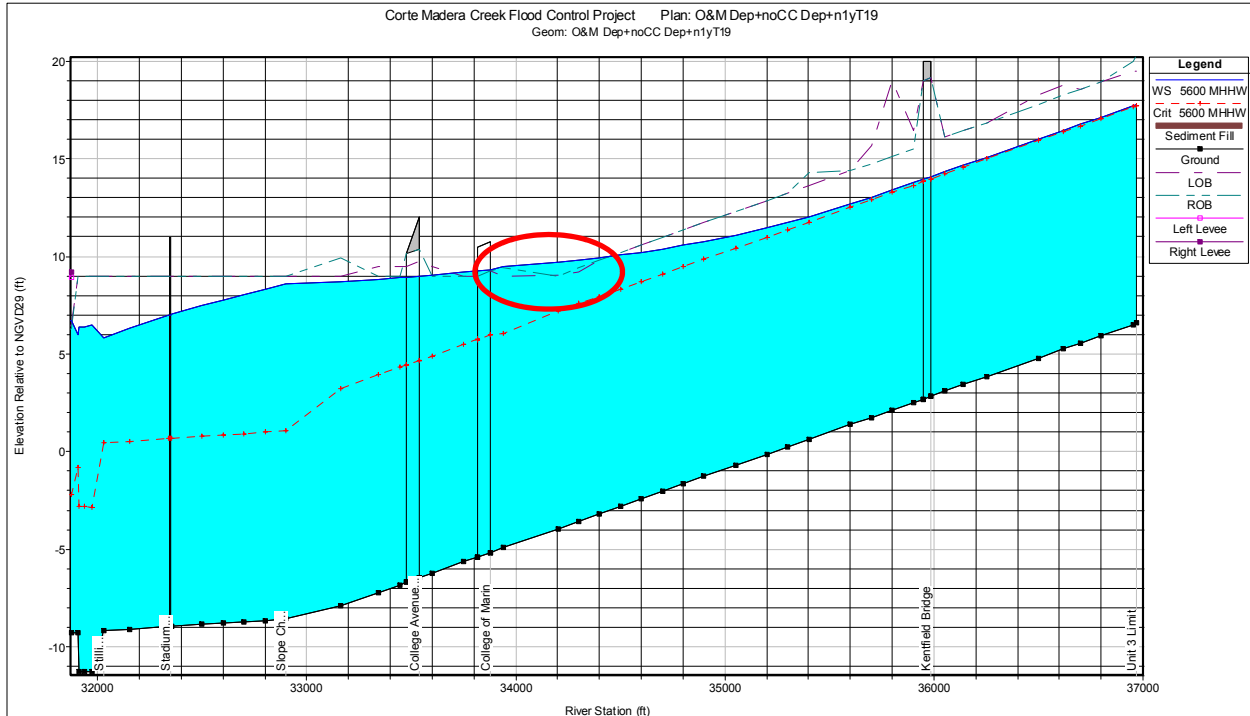
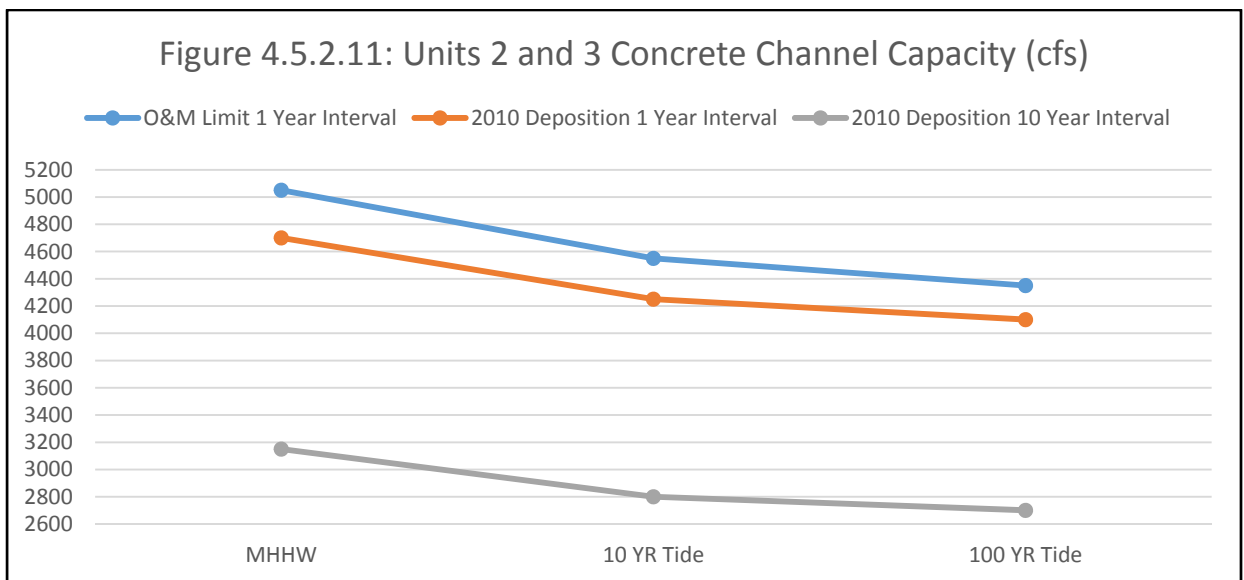


Figure 4.5.2.11: Water surface profile for the 5,600 cfs flow, with Interim O&M Manual earth channel sediment deposition profile and annual concrete channel sediment removal interval. The red circle indicates the overtopping location.

Test 5: Concrete Channel Capacity

In Test 4, I show the existing concrete channel does not have sufficient capacity to convey the 5,600 cfs flow needed for the Ross Valley Capital Improvement Plan, unless the USACE improves the Units 2 and 3 projects with new floodwalls. In this test, I estimate the available concrete channel capacity, based on the three sediment deposition scenarios in Test 4 that bracketed the upper and lower capacity limits. In each deposition scenario, the analysis estimated the capacity under MHHW, 10 year tide, and 100 year tide levels. Table 4.5.2.2 and Figure 4.5.2.11 show the analysis results.

Table 4.5.2.2 - Units 2 and 3 Concrete Channel Capacity (cfs)				
Earth Channel	Concrete Channel	Tide Level (ft)		
Deposition	Deposition	MHHW	10 YR Tide	100 YR Tide
O&M Limit	1 Year Interval	5050	4550	4350
2010 Deposition	1 Year Interval	4700	4250	4100
2010 Deposition	10 Year Interval	3150	2800	2700



The following is the current design boundary conditions for the flood control project:

- Maintain the earth channel with the maximum sediment deposition to not exceed the 2010 level
- Remove sediment in earth and concrete channel in 7 to 9 year intervals
- Provide flood conveyance capacity under MHHW tide level, matching the FEMA design criteria (FEMA 2009)

Based on these criteria, the probable channel capacity is 3,150 cfs. However, my analysis shows that the capacity can range from 2,700 cfs to 5,050 cfs. In addition, there are numerous uncertainties on these capacity estimates, including the actual sediment deposition depth, coincident tide level, channel physical surface conditions, and hydraulic roughness due to sediment transport and sedimentation. Although the channel capacity estimates are under subcritical flow condition, the fact that the estimates did not include any freeboard highlighted the importance of understanding the uncertainty of the hydraulic performance in these high risk channel design. As I discussed in Chapter 2, USACE has a risk based analysis procedure documented in the engineering manual EM1110-2-1619 (USACE 1996). Due to computation and data limitations, the methods to estimate the stage discharge uncertainty are mostly empirically generalized. The USACE Hydraulic Engineering Center is currently developing a watershed analysis tool (HEC-WAT) for stochastic analysis using Monte Carlo method. In this analysis scheme, a user can define the flow and hydraulic roughness probability distributions to conduct long term random simulation. Although there is still a question on how to define the probability distribution curves for flow and hydraulic roughness, it is a significant step forward in developing the analytical methods to define uncertainty, and it would be especially applicable to Corte Madera Creek, where a clear understanding of the uncertainty is critical for the Ross Valley flood improvements planning, and extensive data is available for the analysis. As a next step to follow-on from this dissertation, an uncertainty analysis using HEC-WAT should be conducted to estimate the channel capacity and to quantify the confidence limits.

4.6 Conclusion

In the Corte Madera Creek Flood Control Project review, I presented a number of problems commonly encountered in USACE flood control channels in that era. The project design was based on the clear water assumption, which did not consider the impacts of sediment transport and sedimentation on flow conveyance capacity. Unrealistic operation and maintenance requirements created significant financial and management burdens to Marin County. Lack of environmental sensitivity generated public opposition to suspend the Unit 4 project, putting the channel design back to the drawing board since 1972. The unfortunate timing on the Unit 4 project delay contributed to post-project flooding including a major flood event in 1982, causing significant financial and property loss to Marin County. This project review validated my policy review finding in Chapter 2.

My hydraulic analysis found that even under the original design condition with the clear water assumption, the supercritical flow in the concrete channel is highly unstable. The concrete channel has a high risk of unintended hydraulic jump due to the low Froude number, especially at the downstream of the College of Marin Bridge. My analysis validated previous studies that the increased roughness due to sediment transport contributed to the 1982 flood. In the sensitivity analysis, I quantified the relative significance of tide level, earth channel sediment deposition, and concrete channel sediment deposition to the concrete channel hydraulics.

My analysis tested if the concrete channel can provide 5,600 cfs capacity needed for the Ross Valley Capital Improvement Plan. I found that unless the project constructs new floodwalls along the concrete channel, it does not have the 5,600 cfs capacity. My analysis also bracketed

the current channel capacity to be between 2,700 cfs and 5,050 cfs, with the probable capacity of 3,150 cfs based on the current design and management criteria. An important finding from my analysis is even if MCFCWCD maintains channel sediment as per the USACE Interim Operation and Maintenance Manual (USACE 1988a), the channel still cannot provide 5,600 cfs capacity, let alone the 100 year flow and the Standard Project Flood at 6,900 cfs and 7,500 cfs, respectively.

My analysis results show that due to the channel design deficiency, MCFCWCD need to revise the operation and maintenance guideline for the Corte Madera Creek. Otherwise, MCFCWCD need to construct a new floodwall along the concrete channel to prevent channel overtops. As discussed in Chapter 3, the existing operation and maintenance manuals in many flood control channel projects do not provide specific parameters on the frequency and volume of channel sediment removal. Although the Corte Madera Creek operation and maintenance manual listed deposition depth triggers for sediment removal, it is difficult to monitor. Therefore, as MCFCWCD revises the operation and maintenance guideline, the concept of performance based operation and maintenance should be considered. Project specific deterioration curves are needed to tie in channel performance parameters as triggers to channel maintenance and sediment removal. MCFCWCD should develop a sediment deposition and channel capacity relationship, so the MCFCWCD can assess the need for sediment removal based on channel performance. Such a system requires monitoring data including flow and stage, in addition to the typical channel inspection data such as channel bank stability and failing structures. The hydraulic analysis I developed in this chapter can be used to develop the deterioration curves. This information allows MCFCWCD to formulate performance based operation and maintenance, for targeting and prioritizing the maintenance components that have most impacts to flood control channel operation and channel capacity.

In Chapter 3, I show the commonality of the sediment management problem in flood control channels. The recommendation to update the operation and maintenance manual for the Corte Madera Creek is applicable to other flood control channels. The channel safety program in Chapter 2 provides the management tool to systematically prioritize and implement these updates to USACE projects, as part of the Interim Risk Reduction Measure Plan.

As I discussed in Section 4.5.1, under the no sediment deposition condition, the channel roughness coefficient is 0.019. It consists of concrete bed roughness for fish nets and surface abrasion, and the bedload drag. The grain and form roughness that bring the roughness coefficient to 0.029 to 0.031 only apply to the reaches with sediment deposition. Previous studies suggested that the channel roughness under no sediment deposition condition could be underestimated. The Corte Madera Creek study shows that in the with sediment deposition concrete channel reaches, the resultant water surface profile matches the 1982 flood profile. However, since there is insufficient data to calibrate or validate the no sediment deposition condition to confirm the roughness coefficient estimate of 0.019, it presents a significant uncertainty on the channel capacity estimate. Therefore, this presents a research need to further investigate how bedload sediment transport on smooth surface with no deposition affects channel roughness, under subcritical and supercritical flow. In addition, USACE should update the Engineering Manual Hydraulic Design of Flood Control Channels manual (USACE 1994). The

revision should include methods to estimate channel roughness coefficient under various sedimentation and sediment transport conditions.

My analysis highlighted the issues MCFCWCD faces to maintain the channel operation, and to live with the project and develop a watershed based flood protection plan around it. Due to the critical significance of the firm channel capacity to the Ross Valley Capital Improvement Plan, and the uncertainties surrounding the sediment, roughness coefficient, and channel capacity estimates, I recommend a follow-on analysis to quantify the uncertainty of the channel hydraulics.

MCFCWCD is actively implementing the Ross Valley Capital Improvement Plan. The Ross Valley Capital Improvement Plan estimated the cost benefit ratio of 0.9. MCFCWCD considers it a favorable ratio to proceed with the project. It gained public support, and is partly funded with outside grants and the \$40 million storm drainage fee voted by the watershed property owners in 2007. Reflecting the USACE planning process as discussed in Chapter 2, if it were a USACE project, the Ross Valley Capital Improvement Plan would not be able to proceed at the USACE since the cost benefit ratio is less than 1.0. This project is a significant step from a local agency to move forward, to confront the long history of flooding problems in the watershed.

4.7 Attachment: Corte Madera Creek Channel Cross Section Survey Summary (Excerpt from Noble 2011)

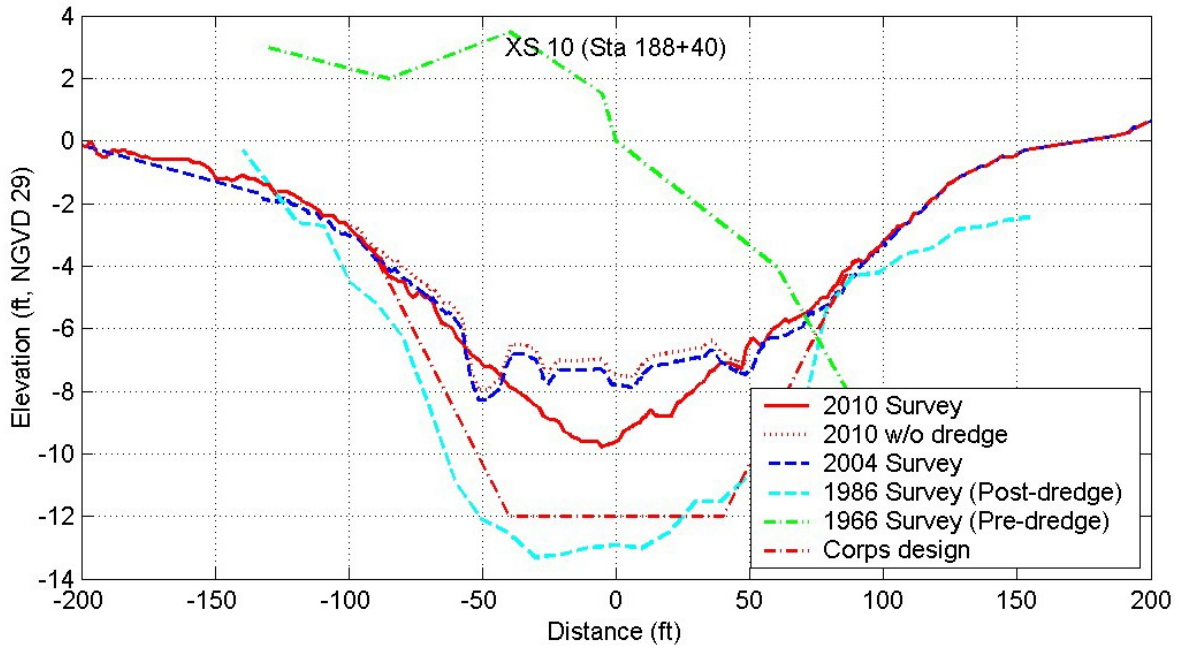


Figure 4.7.1: Cross Sections at Section 10 (Sta 188+40)

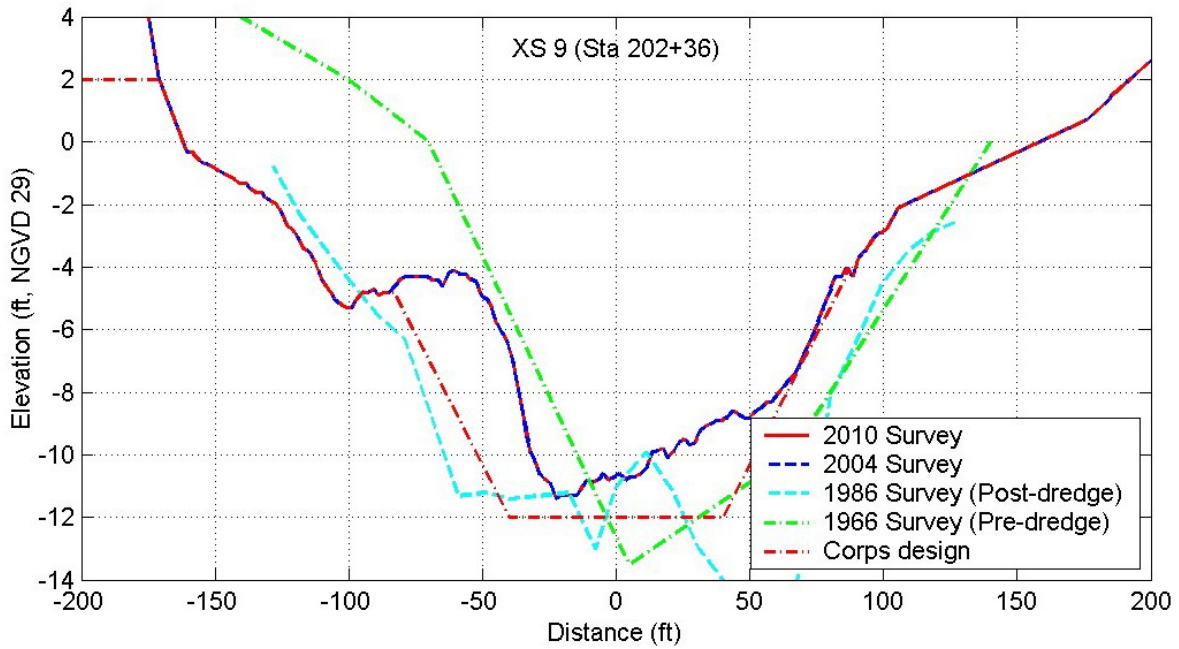


Figure 4.7.2: Cross Sections at Section 9 (Sta 202+36)

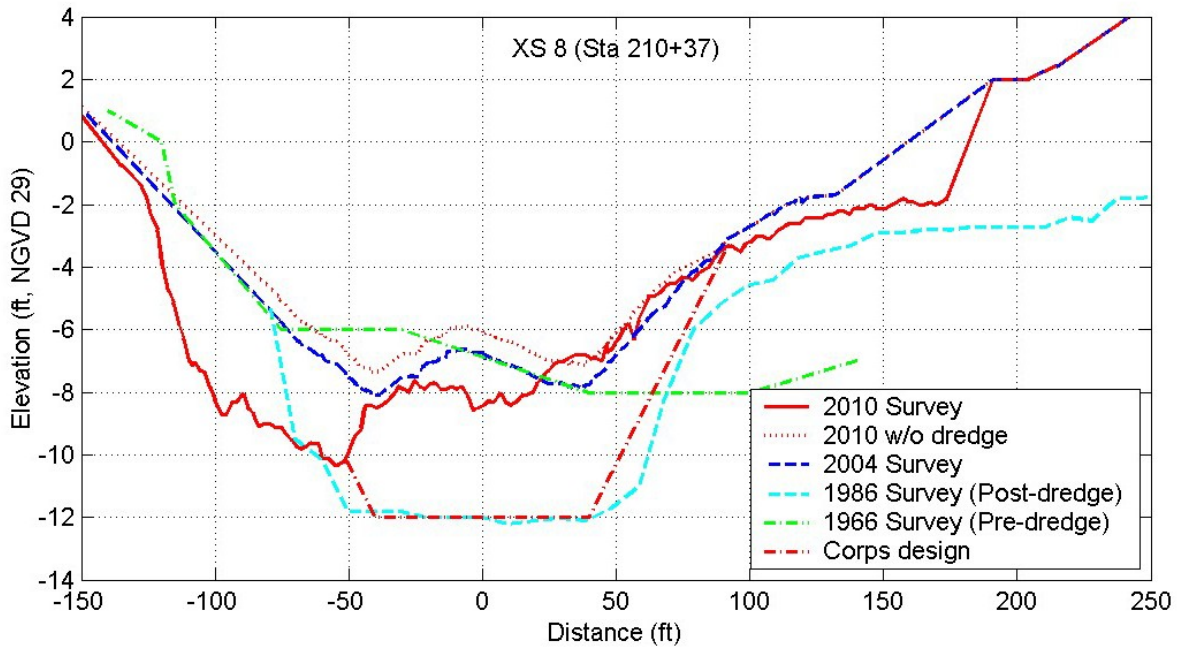


Figure 4.7.3: Cross Sections at Section 8 (Sta 210+37)

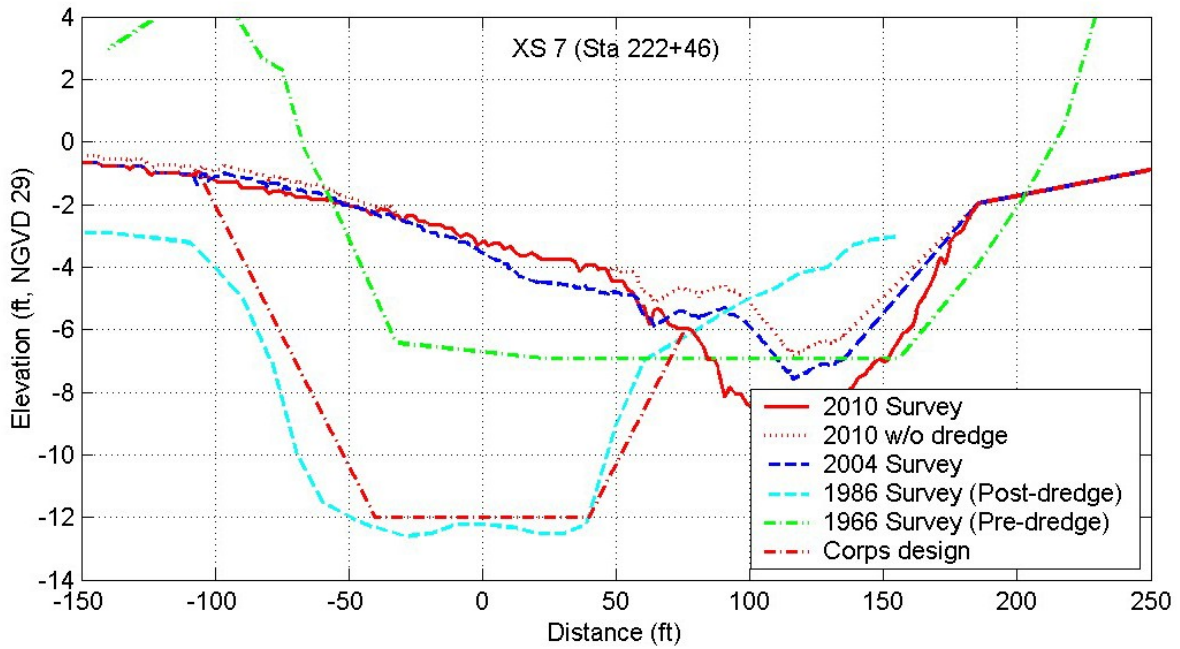


Figure 4.7.4: Cross Sections at Section 7 (Sta 222+46)

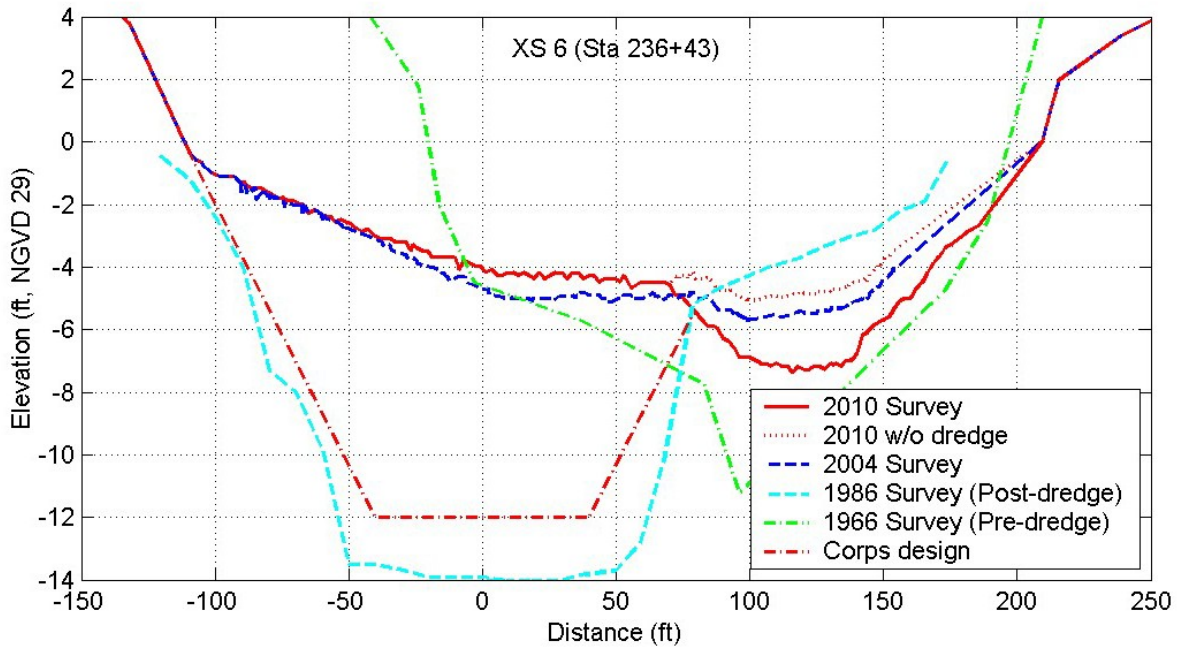


Figure 4.7.5: Cross Sections at Section 6 (Sta 236+43)

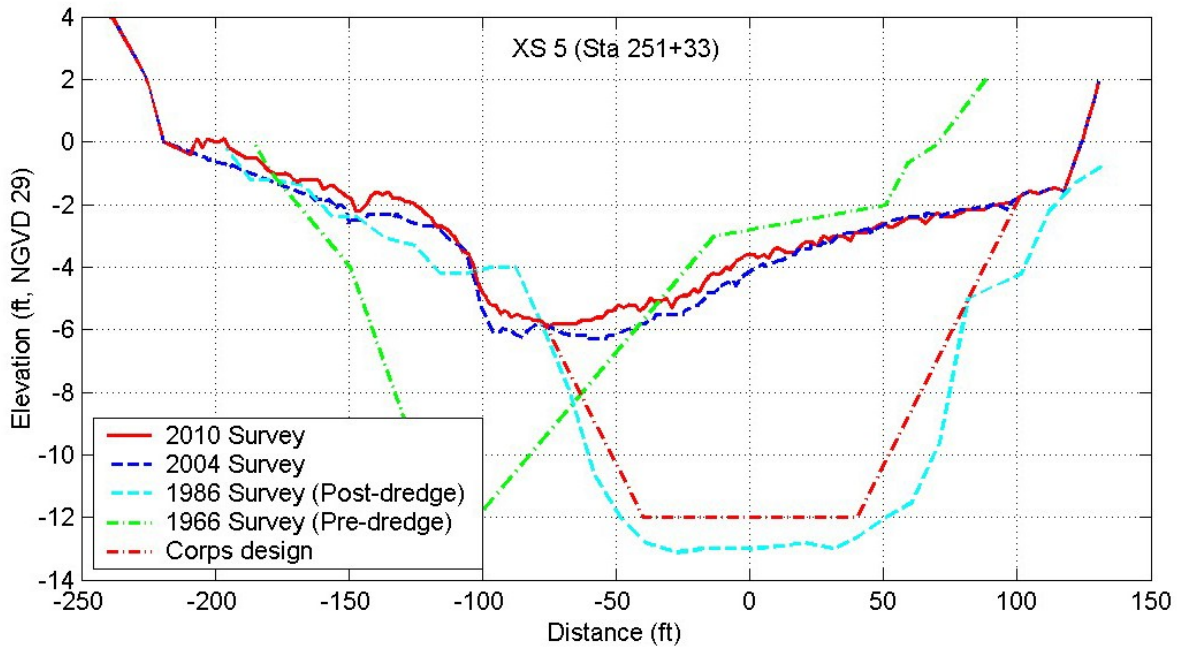


Figure 4.7.6: Cross Sections at Section 5 (Sta 251+33)

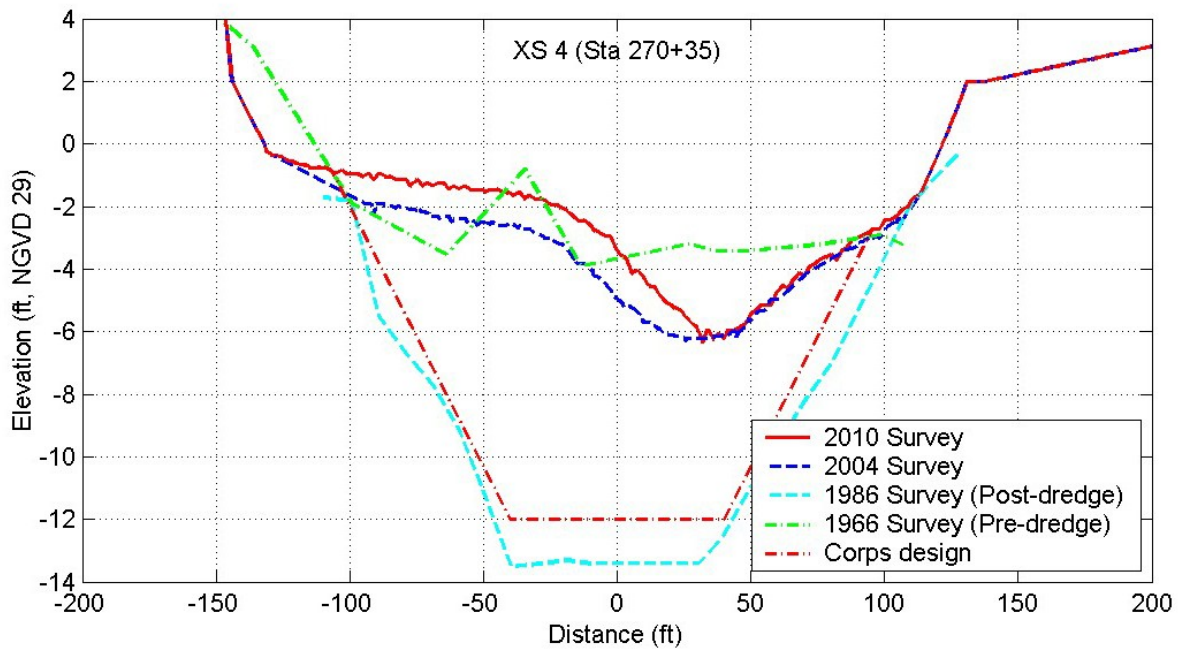


Figure 4.7.7: Cross Sections at Section 4 (Sta 270+35)

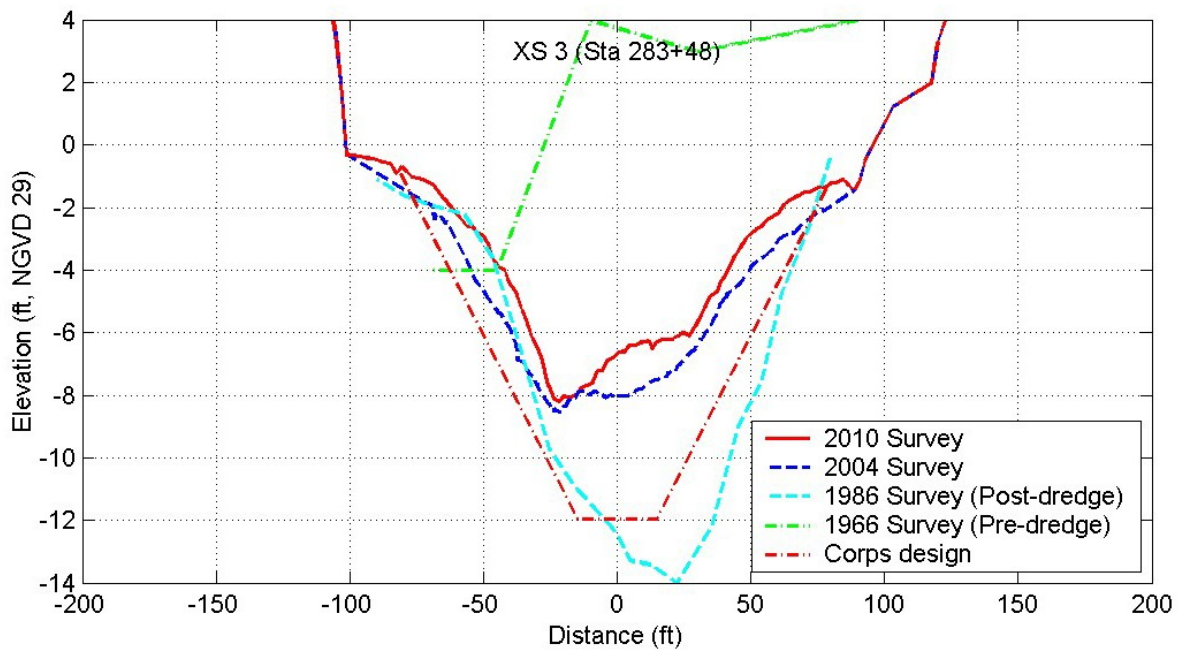


Figure 4.7.8: Cross Sections at Section 3 (Sta 283+48)

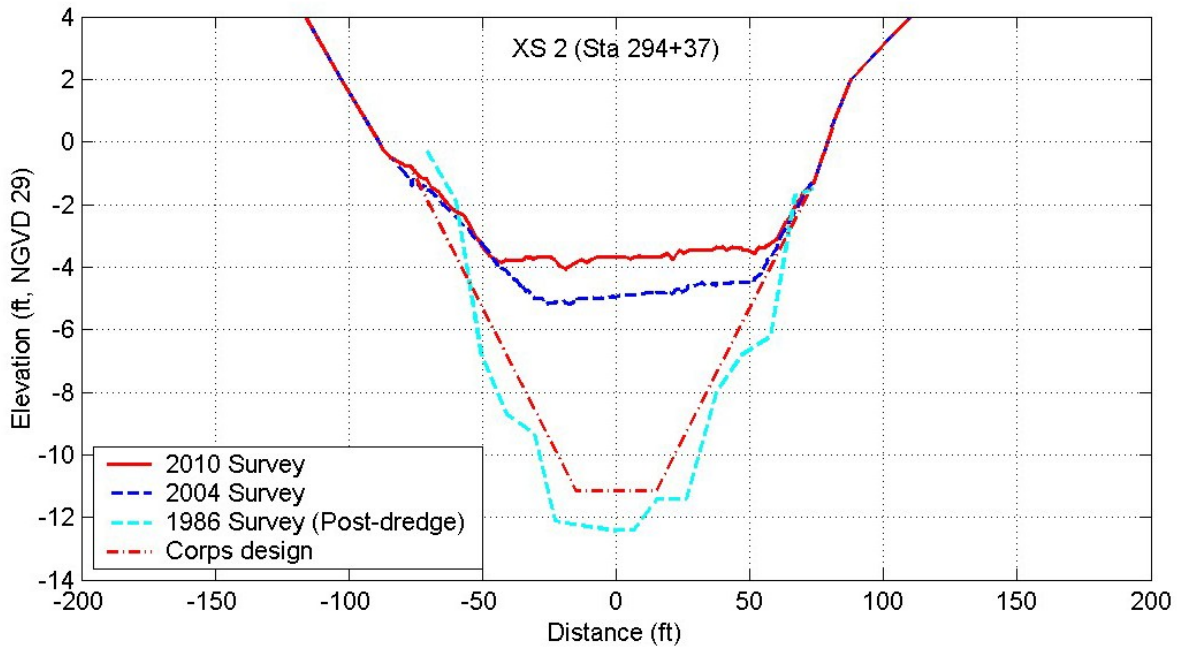


Figure 4.7.9: Cross Sections at Section 2 (Sta 294+37)

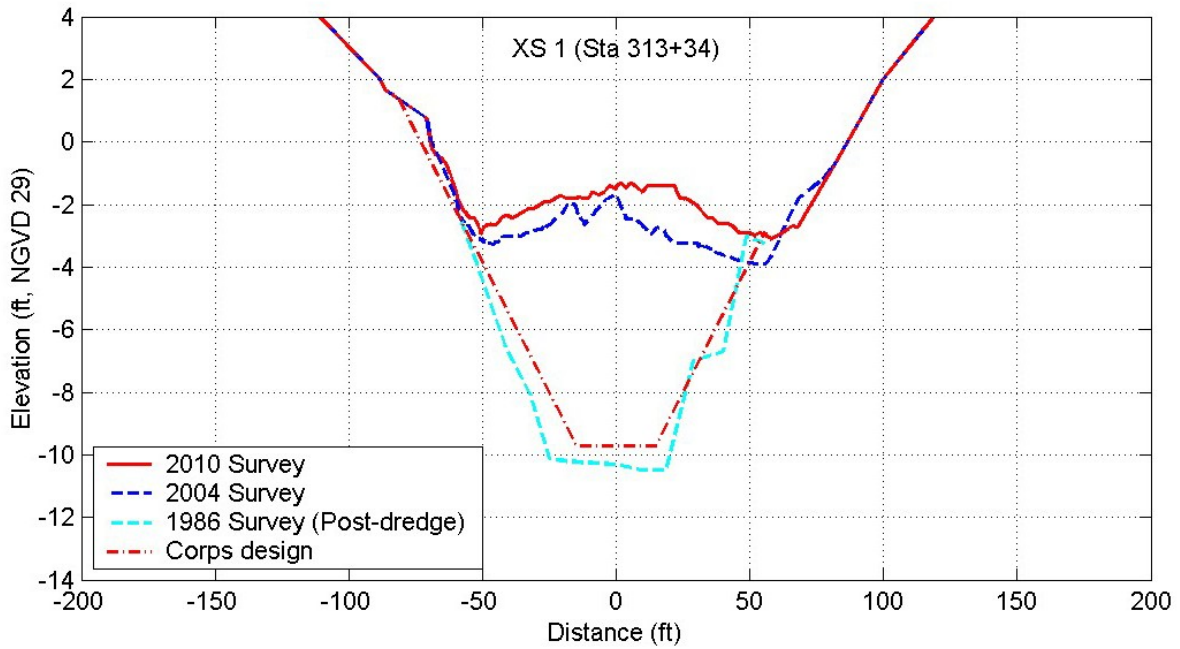


Figure 4.7.10: Cross Sections at Section 1 (Sta 313+34)

CHAPTER 5 – CONCLUSION

5.1 Study Summary

My dissertation examined the flood control channels built by the USACE between 1950s and 1970s in two scales. At the Federal scale, I reviewed the USACE planning policies to trace back how these flood control channels were planned and designed, and how policies from FEMA and California State influenced project formulation. Then I systematically evaluated flood control channels in San Francisco Bay Area to identify planning, design, and operation and maintenance issues. My quantitative summaries showed the commonality of problems in these flood control channels, and it forms the justification to develop a channel safety program to manage all USACE flood control channels at the Federal scale. At the local scale, I conducted a detail review on the planning process of the Corte Madera Creek flood control project, to validate the findings from the USACE planning policies review. I also developed a hydraulic analysis to quantify the sensitivity of channel performance to hydraulic roughness, sediment transport, and channel maintenance. My analysis identified a need to update the USACE operation and maintenance manual and engineering manual to improve channel sediment management efficiency and uncertainty quantification.

This chapter synthesizes my findings in this study. Sections 5.2 to 5.4 summarizes the planning policy, design, and sediment management issues common to the flood control channels. Section 5.5 summarizes suggestions to manage these aging infrastructures.

5.2 Planning Policy Issues

The cost sharing element of the USACE flood control program provides an attractive financial incentive for local sponsor partnership. The USACE covers the majority of the project cost. Local sponsor is responsible for lands, easements, rights-of-way, and relocation costs. However, the state of California covers these local sponsor costs, as authorized by the California Flood Control Law of 1946. To quantify the state cost share benefits, the San Francisco Bay Area case study shows that the average cost sharing division between USACE and the local sponsor is about 72% to 28%, which is close to the 65% to 35% formula set in the Water Resources Development Act of 1986 (U.S. Congress 1986). However, with the California state cost sharing, the local sponsor average cost share is reduced from 28% to 3%. The state cost share provided a significant financial incentive for the local sponsor on the USACE flood control projects. As a result, local sponsor requires very low percentage cost share, for a project that provide flood reduction benefits and facilitate profitable urbanization in the flood prone areas.

The Local Cooperation Agreement (LCA) for the USACE and local sponsor partnership limited the local sponsor's influence on the project, but at the same time indemnified USACE from claims. In addition, the LCA stated the local sponsor's responsibilities on lands, easements, rights-of-way, relocations, and lifetime operation and maintenance. Under the LCA, if a project was poorly designed, the local sponsor has little power to change the project design, but need to live with the project and responsible for the liability, and operation and maintenance. The LCA poses a significant disadvantage to the local sponsor, especially if the local sponsor has a

different approach to the project design. I demonstrated this issue in the case of the Corte Madera Creek Unit 4 project, where disagreements over the concrete channel design versus a more environmentally sensitive design delayed the project for over 40 years.

My review highlighted the policies that shape the flood control channels built from the 1950s to 1970s. The cost benefit analysis policy at the time singly focused on the national economic benefits. The national economic benefits were defined as flood damage reduction and land enhancement. The planning process also narrowly focused on the efficiency to meet the project objectives with minimum cost. The process did not have an explicit requirement to evaluate alternatives focusing on different objectives.

The current planning process requires evaluating multiple alternatives under four objectives, National Economic Development, Regional Economic Development, Environmental Quality, and Other Societal Effects. Based on the single National Economic Efficiency objective as stated in the Budget Circular A-47 (1952) and Green Book (1958), the project formulation and alternative evaluation process in the 1950s concentrated on assuring that there was no cheaper means to accomplish the same purpose. It is recognized that “in theory, the broadest range of alternatives...should be considered,” (IWR 1996), but the emphasis of each alternative was clearly on a severely limited range of objectives.

Before the environmental regulation era starting in the late 1960s, there were few, if any, requirements to mandate environmental costs and benefits be included in project planning and evaluation. This allowed projects to ignore their ecological consequences along the riparian corridor. The lack of environmental sensitivity in flood control channel design generated public opposition. As shown in the history of the Corte Madera Creek Flood Control project, local dissatisfaction with the proposed concrete channel at the Unit 4 project reach caused Marin County to suspend the Unit 4 project in 1972, sending the channel design back to the drawing board. The unfortunate timing of the Unit 4 project delay contributed to post-project flooding including a major flood event in 1982, causing significant property loss.

For the single purpose of flow conveyance, a flood control channel is sized to contain the design flow, so the adjacent floodplain is not needed for flow conveyance. Therefore, the floodplain will not be flooded and it can be utilized for urban development. This creates two problems.

The first issue is the combination of economic pressure and poor landuse planning that lead urban development up to the banks of the flood control channels. The constrained channel corridor becomes problematic when the hydrology of the watershed changes over time. Within the narrow creek corridor, it is difficult and expensive to increase channel conveyance capacity to meet the new design flow and flood protection level of service, not to mention any meaningful stream restoration. In many cases, the constraint even prohibits local sponsor from access to the channel banks for maintenance operations.

The second, and perhaps more important, issue is the loss of riparian habitat from new urban developments in the floodplains. Even for areas that are undisturbed, the floodplain is hydrologically disconnected from the stream so the ecological function is severely compromised.

Urban development as a result of the flood control channel project is captured as the land enhancement justification in the cost benefit analysis. The land enhancement for these projects means developing agricultural land to residential or light industrial/commercial land use. In the San Francisco Bay Area case study, for Pinole, Corte Madera and Alameda Creeks, the land enhancement also includes conversion of tidal marshlands into urban areas. Since land enhancement is considered as a justifiable project benefit, it implicitly results in disregard for ecological values in tidal marshland and riparian corridor. None of these projects attempted to quantify habitat elimination as a cost entity in the cost benefit analysis. On the other hand, the land enhancement encouraged floodplain development. Floodplain development increases flood risk. The increased flood risk under the project design flood frequency level was turned around in the cost benefit analysis as “benefit”, due to flood protection and the resulting flood avoidance provided by the project. At the same time, the residual flood risk above the project design flood frequency level was not included in the cost benefit analysis. Therefore, floodplain and tidal marshland developments provided a significant favorable justification for these flood control projects. This is highlighted in the Alameda Creek case study. As discussed in Section 3.2.9, in 1949 the Committee of Public Works directed the USACE to review the previous no project recommendation. The committee stipulated the review to give full consideration to land enhancement and other factors (USACE 1961) to increase the cost benefit ratio and justify the project.

In the San Francisco Bay Area case study, I analyzed 9 flood control channel projects. My study found that 6 of the 9 projects were justified based on the land enhancement “benefits” by ignoring the environmental consequences to develop floodplain, agricultural land, and tidal marshland. I show that if the land enhancement benefit is subtracted from the total benefit for each project, 3 of the 6 projects, on Rheem Creek, Pinole Creek, and Corte Madera Creek, would have had a cost benefit ratio below 1.0. Thus if developing tidal marshland, undeveloped floodplain, and agriculture land in the floodplain were not allowed to be counted as a benefit, these projects would not have the cost benefit ratio to justify authorization. The Alameda Creek project would have had a cost benefit ratio of 1.4, or approximately half the ratio of 2.8 that the project was authorized.

My analysis shows that land enhancement had significant impact on project justification. The finding provides evidence that if riparian and marsh habitat protection had equal footing as flood risk reduction in these projects, many of these projects would not have been authorized in the form they were constructed. Note that this analysis did not subtract the flood control project benefits derived from the reduced flood risk in the land enhancement developments. Otherwise, more projects would have had a cost benefit ratio below 1.0.

Aside from the USACE policies, the National Flood Insurance Program (NFIP) is another significant influence on the nation’s flood protection policy. Flood insurance buys down flood risk and legitimizes development in the flood plain. The program popularized the 100-year flood standard. Under the NFIP the 100-year flood standard is uniformly applied to the entire nation. However, while the 100-year flood may be sufficient or even over-conservative for rural low density communities, it could be insufficient in large urban areas and river basins with high flood risk (NRC 2000).

The 100-year flood redefined the public's perception of flood safety. There is a misconception in the public that the 100-year flood means the area will be safe in 99 years, and flood only once in 100 years. In reality, within a typical flood control project economic life of 50 years, there is close to a 40% chance that a 100-year flood or higher would occur. Even assuming a flood control project will perform as designed to protect the area from the 100-year flood, and also assuming the existing 100-year flow is actually the same as the design estimate, the area will still be subject to flooding if there is a "101-year" flow (Ludy and Kondolf 2011).

In real estate transactions, it is required to disclose if a property is within FEMA NFIP 100-year floodplain, as a way to communicate the risk of flooding and the associated flood insurance requirements (Troy and Romm 2004). In general, public defines 100-year floodplain as the clear boundary of flood risk. Outside of the 100-year floodplain means the area is considered to be free from flooding. This boundary encouraged urban development up to the edge of the 100-year floodplain. However, it does not stop floodplain development. Although a case study in California found that average floodplain home sold for 4.2% less than a comparable non-floodplain home (Troy and Romm 2004), economic and land pressures continue to drive developments in high flood risk areas.

5.3 Design Issues

The primary design criteria for flood control channels is to maximize channel flow capacity in a minimal footprint. Therefore, the flood control channels typically have straightened channel alignment for two purposes:

- Increase channel slope to increase flow velocity.
- Reducing travel distance to decrease travel time.

The hydraulic design assumes clear water condition. Under the clear water assumption, it is assumed the bedload and suspended load sediments have insignificant impacts to the channel roughness, and sediment deposition is ineligible during the peak design flow, so the channel geometry will remain as designed.

In order to eliminate floodplain overflow during large storm events, the uniform shaped flood control channels were oversized for flow conveyance. Wide thalweg with flat longitudinal slope invites sediment deposition. In addition, floodplain disconnection is a built-in feature of the flood control channels to meet the flood protection objectives. The resultant oversized channel does not have access to the floodplain for sediment deposition. At the same time the restricted tidal prism reduces channel sediment flashing in the tidal zone. As a result, the flood control channels are often faced with significant in-channel sediment deposition.

Sediment deposition reduces channel capacity, and it was underestimated in the project design. In addition, in-stream sediment transport increases channel roughness, and the clear water assumption neglected this hydraulic factor in the project design. Therefore the combination of sediment deposition and sediment transport reduce channel capacity below the design capacity.

The clear water assumption is a key technical design deficiency on these flood control channels. The clear water assumption led to the following questionable channel designs, with the channel capacities unrealistically overestimated.

- Channel outfall thalweg elevation was set lower than the adjacent bay bed to increase longitude slope for higher channel capacity. However, this created a hole below sea level at the channel mouth, which filled with sediment. The San Francisco Bay Area case study shows that 6 of the 9 flood control projects reviewed in this case study have this outfall design feature. Since the channel bank elevation and freeboard were designed based on the channel thalweg elevation, not bay bed elevation. Sediment deposition created flood risk vulnerability at the downstream tidal reaches.
- Concrete channels were designed under supercritical flow assumptions with roughness coefficients based on smooth concrete surface. In reality, sediment transport and deposition in the concrete channel increase channel roughness, leading to rapid water surface elevation increase due to hydraulic jumps from supercritical flow to subcritical flow.

My review of the flood control channel design standard shows that the channel was designed based on idealistic hydraulic conditions that are high risk and unrealistic. The use of supercritical flow design in concrete channel demands the channel to function as designed, with little margin of error and assumes channel hydraulics are not affected by sediment. The Corte Madera Creek analysis shows that even under clear water assumption, the supercritical flow water surface profile along the concrete channel has only 1 foot of freeboard. The model also shows instability in water surface profile. While it is still under supercritical flow, there are sections of the concrete channel with Froude number less than 1.1. This means these areas are at high risk of having unexpected hydraulic jump to subcritical flow with a corresponding increase water surface elevation. Note that the critical depth along the concrete channel is actually above the channel banks. In theory, it is possible that the channel flow can exceed the top of bank elevation while still under super critical flow. If sediment deposition and/or sediment induced channel roughness are introduced into the hydraulic analysis, the concrete channel would be under subcritical flow and overtop the bank in all sensitivity analysis scenarios.

The USACE engineering manual in 1970 discussed the need for a sediment transport analysis to evaluate the potential change in earth channel geometry due to scouring and deposition. It was not until the 1994 revision that a discussion on the hydraulic roughness estimate influenced by sediment, namely the grain and form bed roughness, were included. Sediment transport and its hydraulic effects on concrete channel were mentioned in the last section, titled the “Unforeseen Factors”, of the manual. The discussion is based on the studies at Corte Madera Creek in California. However, the manual did not provide formal guidelines on how to assess concrete channel roughness influenced by sediment transport and sedimentation (USACE 1994).

In addition to the 100-year standard as summarized in Section 5.3, the other issue with the NFIP is the freeboard criteria for levees and floodwalls. Under the NFIP, the levee top height needs to be 3 ft to 4 ft above the 100-year flood elevation. The freeboard is needed for a levee to be accredited as providing 100-year flood protection, so the areas protected behind the levees can be exempted from being defined as flood zones and subject to flood insurance. USACE has a

similar levee freeboard requirements but the guideline is 3 ft. Since FEMA requires higher freeboard height, even if the USACE levees were built to the guidance in their engineering manual, and the channel performs as designed, the levee still does not meet the FEMA accreditation requirements. This design standard conflict can be costly to local communities.

5.4 Operation and Maintenance Issues

The San Francisco Bay Area case study found that many local agencies cannot maintain sedimentation in flood control channels as prescribed in the original project design nor the operation and maintenance manual. While sediment deposition is recognized in project design as a maintenance line item, Walnut Creek was the only project that had a sediment budget and deposition estimate. However, the original sediment deposit estimate was underestimated and the revision increased the estimate by over 4 times. For all other projects, the sediment management requirements were vaguely noted in the design documents, as to desilt at least once a year. In the project Operation and Maintenance manuals, there are no explicit sediment management instruction, except for Corte Madera, San Leandro, and Alameda creeks, the latest projects in this case study. For these three projects, the maximum sediment deposition depths were specified as a maintenance criteria.

In each project, there is an operation and maintenance cost estimate for the economic analysis. Some projects have an explicit line item on sediment removal, otherwise it was lumped into the channel maintenance line item. The operation and maintenance cost estimate was typically based on a percentage of the channel construction cost. Such method detaches the cost estimate from the reality of the actual operation and maintenance needs. Therefore, its legitimacy is questionable, but yet it was used in the cost benefit analysis to justify the project existence.

The San Francisco Bay Area case study found that sediment deposition created significant operation and maintenance burden to local sponsors. On average the current sediment removal maintenance cost estimate is 5 times higher than the original sediment removal cost estimate from the project design. In the study, seven of the nine project have channel sedimentation data. Of the seven projects, four projects require higher sediment removal maintenance budget than the design estimate. The three projects with lower sediment removal maintenance cost than the design estimate are smaller watersheds with lower design flow rate and frequency. This means these channels are not as oversized as other projects, so it could have higher shear stress for more efficient sediment transport through the flood control channel.

Compounding on the sediment management issues, in the San Francisco Bay Area case study, five of the nine creeks have higher 100 year flow estimate than the design estimate. The flow increase is partly due to the urban hydrology as a result of watershed developments. However, as highlighted in Rheem and San Lorenzo creeks, the change in flow estimate is also due to additional rainfall-runoff data since the 1950s, when most of the projects were planned.

Five of the nine creeks in the San Francisco Bay Area case study have existing channel capacity data. In these creeks the existing channel capacities are all reduced by as much as 50% from the original design capacity. The capacity reduction is mainly due to sediment deposition resulting in

smaller effective flow area, especially in the tidal zone. In addition, none of them has the capacity to convey the existing 100 year flow. If we compare the current and original design capacities, five of the nine creeks cannot provide existing 100 year flow capacity, even though some were designed for Standard Project Flood. This result of capacity deficiency under original design condition is a matter of concern. It means even if the channels were maintained to the original design specification and under the clear water condition, these channels still cannot provide the 100-year flood protection.

The case study at Corte Madera Creek further illustrated the issue of sediment management in the flood control channel. The local sponsor devised a more efficient sediment removal scheme than the prescription in the Interim Operation and Maintenance manual (USACE 1988). However, even under the proposed 7 year dredging cycles to provide 100 year flood protection, it still costs the local sponsor approximately \$332,000 per year, 14 times higher than the original project design estimate on the annual sediment management budget, after adjusted for inflation

The proposed flood protection improvements at the upstream end of Corte Madera Creek are based on the assumption that the flood control channel can provide 5,600 cfs capacity. An important finding from the analysis is that even if the channel were maintained as per the USACE Interim Operation and Maintenance Manual (USACE 1988), the channel cannot provide 5,600 cfs capacity, let alone the 100 year and the Standard Project Flood capacity at 6,900 cfs and 7,500 cfs respectively. Additional analysis shows that the probable no-freeboard channel capacity is 3,150 cfs, with capacity range from 2,700 cfs to 5,050 cfs. This reduction in capacity is mainly due to the increased roughness to account for sediment effects. The sensitivity analysis showed that the concrete channel sediment deposition has a significant impact on the concrete channel water surface elevation. The analysis further highlighted the issues Marin County faced to maintain the channel operation, to live with the project and to develop a watershed based flood protection plan around it.

Aside from the operation and maintenance issues identified in this study, there is a fundamental question of channel monitoring. USACE requires periodic inspections on flood control channels. The inspection is mostly visual based, with limited scope to address physical conditions such as structure deterioration and channel erosion (USACE 2008). Therefore, important information such as channel performance, sediment budget, and stage discharge relationships are not monitored. This creates difficulties for the local sponsor and USACE to track the channel conditions for any meaningful management evaluation.

In conclusion, this study highlighted the issue of channel sediment. The clear water design assumption underestimated sediment impacts on channel capacity and in-channel sedimentation rate, leading to expensive and unexpected long term sediment management needs and channel capacity reductions. Local sponsors cannot maintain channel sedimentation due to funding and permitting issues. Hence the local sponsors inherited flood control channels that no longer provide the promised level of service, nor can the local sponsors afford the ongoing maintenance needed to restore the channel capacity. These non-properly maintained flood control channels provide a false sense of flood protection security to the locals, and the resultant floodplain developments further increase flood risk.

5.5 Manage Aging Infrastructure

This dissertation established that flood control channel projects as designed by USACE in the 1950s to 1970s are no longer a viable solution to flood risk management, especially taking this model as a template for future flood protection improvements. However USACE and the project owners still need to manage these aging infrastructures, until these channels are improved or restored in the future. The following outlines how that can be accomplished across different scales:

Federal Scale Management: Although the local sponsors are required to operate and maintain these channels, they are not required to reassess the channel performance and flood risk. Therefore existing conditions such as channel capacity, flood frequency protection level and sediment budget may be unavailable or outdated. Although currently there are policy mechanisms such as the Continuing Authorities Program (CAP) that allows USACE to revisit completed projects for structural improvements and environmental restoration, they must be initialized by individual local sponsors who feel the need for a project. There is not a federal program to systematically manage all USACE flood control channels in the nation. A management framework for the entire USACE flood control channel portfolio is needed, for periodic inspection and risk assessment, condition review and classification, and critical improvement prioritization for flood risk reduction.

Drawing upon the dam and levee safety programs, USACE should consider a similar safety program as a management framework for flood control channels. The program should be integrated as a part of the USACE Flood Risk Management Program, which includes the dam and levee safety programs. Chapter 2 proposed a number of critical components that have the following immediate needs:

- Establish national leadership on the channel safety program.
- Develop a risk analysis approach and tolerable risk guidelines for all flood control channels.
- Develop a channel safety action classification system.
- Develop a database to inventory all USACE flood control channels.
- Provide an initial screening for all flood control channels to assess their current performance and risk level, and follow-up with periodic inspection and assessment.

A new channel safety program requires clear justification and strong political support. Consider that the dam safety program was driven by dam breaks in the 1970s, and the levee safety program was driven by Hurricane Katrina in 2005, the formation of a channel safety program will likely take time, until the next major flood event motivates support and funding for a channel safety program.

Local Scale Management: Local sponsors should revise the channel operation and maintenance procedures. As discussed in Section 5.4, the existing operation and maintenance manuals in many flood control channel projects do not provide specific parameters related to the frequency and volume of channel sediment removal. The concept of performance based operation and maintenance should be considered. A project specific deterioration curve is needed to tie in the

channel performance parameters as triggers to channel maintenance and sediment removal. Local sponsor should develop a sediment deposition and channel capacity relationship, so the local sponsor can assess the need for sediment removal based on channel performance. Such a system requires monitoring data including flow and stage, in addition to the typical channel inspection data such as channel bank stability and failing structures. This information will allow local agencies to formulate performance based operation and maintenance, for targeting and prioritizing the maintenance components that have most impacts to flood control channel operation and channel capacity. The channel safety program provides the management tool to systematically prioritize and implement these updates to USACE projects, as part of the Interim Risk Reduction Measure Plan.

Engineering Manual: Additional research is needed on how sediment affects concrete channel hydraulics. In the Corte Madera Creek study, the USACE study (USACE 2000) found that under the no sediment deposition condition, the channel roughness coefficient is 0.019. It consists of concrete bed roughness for fish nets and surface abrasion, and the bedload drag. The grain and form roughness that bring the roughness coefficient to 0.029 to 0.031 only applies to the reaches where there is sediment deposition. The Corte Madera Creek study shows that in the with sediment deposition concrete channel reaches, the resultant water surface profile matches the 1982 flood profile. However, there is insufficient data to calibrate or validate the no sediment deposition concrete channel reaches to confirm the roughness coefficient estimate of 0.019. Therefore, this presents a research need to further investigate how bedload sediment transport on smooth surface with no deposition affects channel roughness, under subcritical and supercritical flow. In addition, USACE should update the Engineering Manual Hydraulic Design of Flood Control Channels manual (USACE 1994). The revision should include methods to estimate channel roughness coefficient under various sedimentation and sediment transport conditions.

Project Assessment: As discussed in Chapter 2, the deterministic safety freeboard method is slowly replaced by the stochastic risk and uncertainty analysis. In this approach, the analysis defines the confidence level of a design element, and the range defines the upper and lower limits of the design criteria. USACE has risk based analysis procedures documented in the engineering manual EM1110-2-1619 (USACE 1996). Due to computational and data limitations, the methods for the stage discharge uncertainty estimate are mostly empirically generalized. The USACE Hydraulic Engineering Center is currently developing a watershed analysis tool (HEC-WAT) for stochastic analysis using Monte Carlo method. In this scheme, user can define the flow and Manning's roughness probabilistic distribution to conduct long term random simulation. Although there is still a question on how to define the flow and Manning's roughness distribution curves, it is a significant step forward on the analytical methods to define the uncertainty, and it would be especially applicable to Corte Madera Creek, where a clear understanding of the uncertainty is critical for the Ross Valley flood reduction planning, and extensive data is available for the analysis. As a next step to follow-on from this dissertation, an uncertainty analysis using HEC-WAT would be conducted to estimate the channel capacity and to quantify the confidence limits.

Reinvention: The advancement in hydraulic engineering, the expanding volumes of hydrologic data, and the experience learned from the existing channels provide valuable insights on how to better design and manage these channels. As a next step, there is a need to develop intervention

strategies to reinvent these flood control channels to meet contemporary objectives in ecological restoration, floodplain management, recreation, and flood risk reduction. Such intervention is difficult since urban encroachments limit available space for restoration. USACE is currently working with the City of Los Angeles to restore the 8-mile long Glendale Reach of the Los Angeles River flood control project. The proposed project includes riparian floodplain restoration and concrete channel removal. The project attempts to reestablish hydrologic connectivity between floodplain and channel, and habitat connectivity between the channel and the adjacent riparian and terrestrial habitats (USACE 2013). It is a major undertaking from both USACE and the local sponsor, with widespread public support to an alternative with an estimated price over \$1 billion. The USACE independent external peer review (Battelle 2013) and our public review comments (Kondolf and Wong, 2013) both points to the critical issue that, “Flood risk management has not been effectively integrated with the objectives of the ecological restoration project, yet is a primary purpose and function of the Los Angeles River.” (Battelle 2013). The comments highlighted the difficulties to balance multiple objectives in urban flood control channel restoration.

5.6 Closing Remarks

My reviews, case studies and analysis in my dissertation provided evidence that federal flood control channels built by the USACE in the 1950s to 1970s are not performing as intended. This portfolio of aging infrastructure requires a systematic program to manage, monitor, and maintain its functionality. My dissertation also provided justification that flood control channels should no longer be a silver bullet solution for flood risk reduction. I hope this dissertation provides the basis and motivation to follow-on this important inquiry on how to manage aging flood control channel infrastructure and provide flood protection while integrating environmental and community benefits.

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