FLOOD CAPACITY IMPROVEMENT OF SAN JOSE CREEK CHANNEL
USING HEC-RAS

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ABSTRACT

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The Santa Ynez Mountains of Santa Barbara County, California, have seen many major storm events during the past century. San Jose Creek, which runs out of these mountains, through the town of Goleta, and into the Pacific Ocean, has experienced several intense flood events as a result. The lower portion of the creek was diverted in 1960 to alleviate flooding through Old Town Goleta. However, flooding still occurred in the storms of 1995 and 1998. This study incorporates a hydraulic analysis component of a project aimed at re-designing this diverted portion of the channel. It presents an analysis of modifications to this reach in order to improve its capacity and reduce flooding during a 100-year event. As one of the most prominent software for hydraulic modeling for steady and unsteady state open channel flow, HEC-RAS is used to analyze multiple variations in channel geometry and combinations of lining materials. Of these modifications, the best configuration is suggested.

Keywords: HEC-RAS, hydraulic modeling, open channel flow, 100-year flood

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Chapter 1

Introduction

The Central Coast of California has had a long history of flood events, and Santa Barbara county is no exception. Ranging from Ventura up along the Gaviota Beach coast to Santa Maria, and extending inland into the Santa Ynez Mountains, Santa Barbara is subject to intense weather conditions from both inland and oceanic storms. As the area urbanized, development expanded and became increasingly prone to flooding damages. In the 1960, a portion at the end of San Jose Creek in Goleta, California was diverted to redirect the creeks flow away from Old Town Goleta using a concrete lined channel. In 1995 and 1998, two flood events proved that this channel did not prevent the creek from overflowing and the nearby commercial areas were inundated.

The City of Santa Barbara has since initiated the planning of a new channel to replace the diverted portion of San Jose Creek, and has partnered with Bengal Engineering in Goleta to modify the channel geometry in order to facilitate the design flow of a 100-year storm. Another component of the project is to provide adequate flow such that that channel can accommodate fish passage to promote the use of upper San Jose Creek as a spawning area for steelhead trout. However, the scope of this study excludes the concerns of fish passage and is limited to the determination of a channel geometry that will adequately convey a 100-year design flow.

This study incorporated the use of a hydraulic model generated using HEC-RAS, and provided by Bengal Engineering. Based on the past performances and implementations of HEC-RAS, it was chosen as a widely accepted hydraulic modeling tool for open channel flow, specifically when designing for flood conditions. HEC-RAS has continued to be the leading software preferred by

hydraulic engineers since its release in 1995, as it offers a simpler modeling approach yet offers accurate results comparable to those of other modeling software.

This thesis begins with a description of the region in which San Jose Creek is located, as well as flooding cases relevant to it and the surrounding areas, in Chapter 2. Chapter 3 discusses the basic principles of open channel hydraulics and the modeling software HEC-RAS as a tool for open channel hydraulics computations. Chapter 4 describes the modification parameters and process used for analysis. Chapter 5 explains and discusses the proposed modifications, and the most effective modifications are suggested. Finally, Chapter 6 offers a brief review of the study and the suggested channel modification, as well possible options for further study.

Chapter 2

Background of San Jose Creek and Flood Issues

The study area for this project is San Jose Creek in Goleta, California. Goleta is a coastal town located in Santa Barbara County in the Central Coast region of California, and is approximately eight miles west of the city of Santa Barbara. San Jose Creek begins and is fed by its watershed in the foothills of the Santa Ynez Mountains near San Marcos Pass of Highway 154, and is also fed by the Dennis Reservoir. The creek runs down through the foothills into a primarily residential section of Goleta and on into a main commercial district of Old Town Goleta. After this commercial section, the creek comes to an end at its confluence with San Pedro Creek, which converges with Atascadero Creek shortly thereafter, before feeding into the Pacific Ocean several hundred feet further (Figure 1). The model used for this study consists of the portion of San Jose Creek starting where it enters the residential section and ending where it merges with San Pedro Creek (Figure 2). The specific focus of this study (Figure 3) is on the commercial section of San Jose Creek, which consists of a man-made channel lined with concrete (Figure 4) constructed to divert the creek's natural flow path away from Old Town Goleta and runs parallel to Highway 217 towards the ocean. This segment will hereafter be referred to as the "study reach".

Figure 1: San Jose Creek, Tributaries, and Watershed - Generated in ArcMap

Figure 2: HEC-RAS Model Extent of San Jose Creek

Figure 3: Focus Area of San Jose Creek: Artificial Constructed Channel Segment Along Highway 217

Figure 4: Upstream Portion of Constructed Reach of San Jose Creek

2.1. Flood History

The South Central Coast region of California, including Goleta, Santa Barbara, and Carpinteria has a historic record of flooding events. Dating back as early as 1907, record flooding has caused major damages to much of Santa Barbara County. In 1914, a two-week storm delivered a peak of 16 inches of rainfall. Due to its proximity to the ocean, flooding poses an even greater threat from oceanic flooding. In 1946, ocean waves inundated much of the coast line. The decision to form a flood control district was made following a 1952 event, in which over 50 homes were damaged and evacuations occurred. Storm events in 1964 caused the destruction of six bridges, as well as homes and roadways, causing over \$300,000 in damages.

More devastating floods occurred as time continued. In 1969, a flood event caused over \$4.5 million in damages to agricultural, residential, public, and commercial properties, as well as highway, railroads, and bridges, and Santa Barbara County was declared a disaster area. It was the largest storm in 55 years, and classified as a 100-year storm (a storm of an intensity with only a 1% chance of occurring in any year, or only occurring once every 100 years) in the upper Santa Ynez watershed area and less intense (5- to 10-year storm intensity) in nearby areas. In 1971, the Montecito-Summerland area of the county was declared a disaster area as another severe storm occurred. In 1978, the northern coastline near Gaviota Beach was severely affected by exorbitant storm waves, intense winds, and high tide fluctuations causing inundation of the coastal region and landslides. Over 200-percent of normal rainfall fell in several areas throughout the southern county during the season of 1982-1983, and the 1992-1993 season resulted in a high of 209-percent of normal rainfall during which 1.25 inches fell in just fifteen minutes. The county was again declared a disaster area, and received \$1.4 million in damages.

Two severe floods occurred in 1995 and 1998, which rivaled the intensities of the 1907 and 1969 floods. Multiple floods occurred in 1995 in the major streams of Goleta and Montecito (Figure 5), causing over \$100 million in damages to 510 structures during an event in January, and all roadways in and out of the city were closed for several days. Goleta and Montecito were struck once again in March causing damage to more than 300 structures, many of which had been affected just months prior, amounting to \$30 million. Three major storm events occurred in February of 1998, causing \$15 million in damages and closing transportation throughout the county and the county was again declared a disaster area.

Runoff is a main contributor to flooding, as severe runoff from the steep slopes of the Santa Ynez foothills backing the coastal regions flows down into the impermeable areas of developed towns and cities. Runoff from the backing foothills can also pick up and carry debris, increasing the potential for damage. Though the intensity of the 1998 storms surpassed that of historic events, damages were greatly reduced because of flood control improvements enforced by the county such as increased channel and debris maintenance, which limited the effects of runoff. Over the course of 1960 to 1963, the lower portion of San Jose Creek was diverted and modified to include an approximately 0.7-mile concrete channel followed by an approximately 2,000-foot unlined channel before merging with San Pedro Creek. This concrete channel is the main focus of the current study. Due to the present capacity of the channel, overflow of the transition between the natural and the modified channel, which occurs at Hollister Avenue, is predicted for a 100-year flood, in which case the overflow is likely to follow the creeks original flow path as well as inundate much of Hollister and Kellogg Avenue and eventually merge with overflow from nearby creeks (FEMA, 2009). Figure 6, provided by the Federal Emergency Management Agency (FEMA), displays the floodplain for the lower portion of San Jose Creek. The regions highlighted in blue represent the 100-year flood plain.

Figure 5: San Jose Creek During 1995 Flood

Figure 6: Floodplain Map for San Jose Creek, San Pedro Creek, and Atascadero Creek

Chapter 3

Literature Review

This section covers an introduction to the terms, principles, concepts, and methods of analysis for open channel hydraulics. A history of hydraulic modeling is presented, and the development of HEC-RAS is discussed and briefly compared to other models.

3.1. Open Channel Hydraulics

Open channel flow (OCF) is defined as the flow of a fluid through any sort of conduit, or channel, that has a surface open to atmospheric pressure, called a *free surface*. Typical examples of OCF include streams, rivers, or partially full pipes such as sewers. The main parameters associated with OCF are

- Flow area, A
- Flow velocity, V
- Flow depth, y
- Discharge, Q , expressed by

$$Q = VA \tag{1}$$

- Wetted perimeter, P , a measurement of the surface that is in contact with the fluid
- Top width, T , a measure of the width of the free surface
- Hydraulic radius, R , expressed as

$$R = \frac{A}{P} \tag{2}$$

3.1.1. Basic Principles

OCF is governed by three main equations; the continuity, momentum, and energy equations.

The continuity equation satisfies the condition that the discharge is constant throughout a reach. Thus the relationship between A and V for adjacent cross-sections can be written as

$$Q = A_1 V_1 = A_2 V_2 \quad (3)$$

The momentum equation satisfies the condition that the forces acting on a volume of flow are balanced.

When analyzing a short reach and applying the momentum principle, assumptions can be made to formulate a simplified form of the momentum equation

$$\frac{Qw}{g}(V_2 - V_1) = P_1 - P_2 \quad (4)$$

Where w is the unit weight of the fluid, and P_1 and P_2 are the resultants of pressure acting on the end sections of the reach and are represented as

$$P = w\bar{y}A \quad (5)$$

where $\bar{y}$ is the distance to the centroid of the cross-sectional area from the channel bottom.

Equation (4) can thus be rewritten as

$$\frac{Q^2}{gA_1} + \bar{y}_1 A_1 = \frac{Q^2}{gA_2} + \bar{y}_2 A_2 \quad (6)$$

A new term can then be defined for each section as the *specific force*, written as

$$F = \frac{Q^2}{gA} + \bar{y}A \quad (7)$$

It follows then that between any adjacent sections of a reach, the specific forces must be equal.

Through manipulation of the momentum principles and another historical equation known as the Chezy equation, Robert Manning developed the most widely used equation in open channel hydraulics, which is written as

$$Q = \frac{c_m}{n} AR^{2/3} S_0^{1/2} \quad (8)$$

where c_m is a unit factor (1.486 US units, 1.0 SI units), n a measure of the roughness of the channel lining termed as the *Manning's roughness coefficient*, and S_0 is the bed slope of the channel and is generally limited to $S_0 < 0.1$ for OCF (Chin, 2006).

The energy equation presents the concept that the energy of any point along the length of the channel is comprised of the energy supplied by elevation and velocity in the form of the equation

$$H = y + \alpha \frac{V^2}{2g} + z_0 \quad (9)$$

where α is a kinetic energy correction factor, g is the acceleration due to gravity, and z_0 is the channel bed elevation with respect to a certain datum, The energy equation also satisfies the condition that the difference in energy at adjacent cross-sections is a result of losses caused by friction and other factors, and can be written as

$$y_1 + \frac{\alpha_1 V_1^2}{2g} + z_1 = y_2 + \frac{\alpha_2 V_2^2}{2g} + z_2 + h_L \quad (10)$$

where h_L is the total head loss between cross-sections. These parameters are illustrated in Figure 7. The concept of energy can be simplified as the specific energy, E , which represent the energy of a given cross-section with respect to the channel bottom, and represented as

$$E = y + \frac{\alpha V^2}{2g} \quad (11)$$

A plot of the specific energy along a reach is referred to as the *energy grade line* (EGL), which has as slope S_f , often referred to as the *friction slope*. The specific energy diagram is generated when depth, y , is plotted against E for a constant flow or velocity. From this relationship, it can be seen that at some depth, the specific energy reaches a minimum (Figure 8). This depth is called the *critical depth*, y_c . Under critical conditions, the following relationship can be formulated

$$\frac{Q^2}{g} = \frac{A_c^3}{T_c} \quad (12)$$

where the subscript c denotes the critical condition and can be rearranged to form a condition of unity such that

$$\frac{Q^2 T_c}{A_c^3 g} = 1 \quad (13)$$

Introducing a new term D , the *hydraulic depth*, where

$$D = \frac{A}{T} \quad (14)$$

a new parameter called the *Froude number* can be defined as

$$Fr^2 = \frac{Q^2 T}{A^3 g} = \frac{V^2}{gD} \quad (15)$$

And simplified to

$$Fr = \frac{V}{\sqrt{gD}} \quad (16)$$

Under critical flow conditions, it follows from equations (13) and (15) that

$$Fr_c = 1 \quad (17)$$

Figure 7: Energy Parameters for Gradually Varied Flow

Figure 8: Specific Energy Curve

3.1.2. Flow Classification

OCF has various classifications based on its behavior with respect to time and space. *Steady flow* describes the condition where the depth and velocity at a given location do not vary with time. Conversely, *unsteady flow* refers to flow in which depth and velocity change over a given time interval. In most cases, the assumption of steady state flow adequately represents natural flow behavior, however unsteady conditions may be required in cases where more precise or specific cases are assessed or results are required.

Uniformity refers to spatial variation at a given time, and flow can be either uniform or varied.

Uniform flow describes the condition where depth and velocity remain constant over a given reach at a given point in time, and is almost always at a steady state. One characteristic of uniform flow is that the slope of the channel bottom, water surface, and EGL are all parallel.

The depth of a uniform flow condition is referred to as the normal depth, y_n . *Varied flow*

describes the condition where depth and velocity change along the length of a channel reach, and has two sub-classifications. *Gradually varied flow* (GVF), occurs when changes in depth and velocity are small and transition smoothly. GVF is, in general, a very accurate representation of most open channel flows. For *rapidly varied flow* (RVF), changes in depth and velocity over a distance are abrupt and drastic. RVF usually occurs either at hydraulic structures such as spillways, or when there are sudden changes in channel geometry. One common instance of RVF is in a hydraulic jump, in which supercritical flow changes to subcritical. OCF classifications can be summarized as (Chow, 1959):

1. Steady flow
 - a. Uniform flow
 - b. Varied flow
 - i. Gradually varied flow
 - ii. Rapidly varied flow
2. Unsteady flow
 - a. Unsteady uniform flow (rare)
 - b. Unsteady flow
 - i. Gradually varied unsteady flow
 - ii. Rapidly varied unsteady flow

Recall that when the specific energy of flow is at a minimum, the flow is considered critical. The Froude number is a relation between inertial and gravitational forces, and the critical condition occurs when those forces are equal and result in a Froude number of unity. *Supercritical flow* occurs when the flow depth y is less than y_c , and is described as shallow, rapid, and turbulent. When depth y is greater than y_c , the flow is classified as *subcritical*, and is

described as deep, slow, and placid. It can be seen in Figure 8, E increases with any deviation from y_c , and in a small increase from $Fr = 1.0$ causes as drastic change in water depth. Thus when flow is at critical depth, it is very sensitive to water depth fluctuations since very slight variation of depth can make the flow subcritical or supercritical. As the water surface fluctuates between subcritical and supercritical flow, standing waves form due to the undulations in water depth. The US Army Corps of Engineers has shown that such fluctuation can be avoided if $Fr < 0.86$ or $Fr > 1.13$ (U.S. Army Corps of Engineers, 1995). Open channels are thus typically designed to maintain a Froude number outside this range.

The channel through which a fluid flows can also be classified, as either *prismatic* or *non-prismatic*. A prismatic channel is one in which the cross-section remains constant along its length and constant bed slope. A channel is otherwise referred to as non-prismatic.

3.1.3. Computational Methods

The two main procedures for OCF analysis and design are the *direct* and *standard step methods*. In general, a step method is a process in which portions of a reach are analyzed iteratively along its length. The direct step method is a procedure in which the water depth is known at two locations, and the distance between those locations is desired, and is solved by equating the energy at either end of a reach of length Δx such that

$$S_0\Delta x + y_1 + \alpha_1 \frac{V_1^2}{2g} = y_2 + \alpha_2 \frac{V_2^2}{2g} + S_f\Delta x \quad (18)$$

Solving for Δx results in

$$\Delta x = \frac{E_2 - E_1}{S_0 - S_f} \quad (19)$$

to solve for the distance between sections 1 and 2. S_f can be determined by rearranging Manning's equation such that

$$S_f = \frac{n^2 V^2}{c_m^2 R^{4/3}} \quad (20)$$

And in the case of the direct step method using equation (19), is taken as the average of the friction slopes for both cross-sections.

The standard step method is a much more common procedure, as it is used to determine an unknown water depth at a point that is a known distance upstream or downstream of a known water depth. By referencing a datum, the standard step method relates the total energy of adjacent cross-sections, where the water surface elevation (WSE) of each section are represented as Z_1 and Z_2 such that

$$Z_1 = y_1 + S_0 \Delta x + z_2 \quad (21)$$

$$Z_2 = y_2 + z_2 \quad (22)$$

The relation can thus be made that

$$Z_1 + \frac{\alpha_1 V_1^2}{2g} = Z_2 + \frac{\alpha_2 V_2^2}{2g} + h_L \quad (23)$$

The total head of each section can be written as

$$H_1 = Z_1 + \frac{\alpha_1 V_1^2}{2g} \quad (24)$$

$$H_2 = Z_2 + \frac{\alpha_2 V_2^2}{2g} \quad (25)$$

and equation (23) can be re-written

$$H_1 = H_2 + h_L \quad (26)$$

The total loss, h_L is a summation of *friction losses*, h_f , and energy losses due to *expansion* and *contraction*, h_0 . The friction loss is determined by

$$h_f = S_f \Delta x = \frac{1}{2} (S_1 + S_2) \Delta x \quad (27)$$

In a non-prismatic channel, the widening and narrowing of the channel will cause expansion or contraction of flow, respectively, both of which result in energy loss. These losses are determined by

$$h_0 = C_{e,c} \left| \frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g} \right| \quad (28)$$

where C_e and C_c are the expansion and contraction coefficients, respectively. Typical values for C_e and C_c are shown in Table 1 (HEC₁, 2010). It should be noted that the values in Table 1 are for subcritical flow. In the case of supercritical flow, C_e and C_c are typically 10% of the tabulated values, and do not exceed 0.1.

Table 1: Typical Values for Expansion and Contraction Coefficients

Type of Transition	Expansion	Contraction
None	0.0	0.0
Gradual	0.3	0.1
Typical bridge section	0.5	0.3
Abrupt	0.8	0.6

To analyze a channel using the standard step method, and iterative procedure is repeated by assuming a value for the unknown water depth and solving for equations (24) and (26) until the values for H_1 agree (Chow, 1959).

Another computation common for OCF, is the event of a *hydraulic jump*, in which flow transitions from supercritical to subcritical over a very short distance, such as at the bottom of a spillway. To analyze a hydraulic jump, the momentum principle is applied using the concept that the specific force at both ends of the jump must be equal. In any non-prismatic channel, equation (6) is applied to analyze a hydraulic jump when the conditions of either end of the jump are known. For a rectangular channel, however, a simplified equation is used to relate the water depth at either end directly, and is written as

$$\frac{y_2}{y_1} = \frac{1}{2} \left(-1 + \sqrt{1 + 8Fr_1^2} \right) \quad (29)$$

The two depths y_1 and y_2 are referred to as conjugate or sequent depths, and are defined as different depths the same cross-section at which the specific forces are equal.

3.1.4. Classification of Water Surface Profiles

For gradually varied flow, the water surface profile (WSP) can be classified based on criteria including slope and depth, and is dependent on how the WSP changes with respect to distance.

The WSP can be defined by the differential equation for gradually varied flow, written as

$$\frac{dy}{dx} = \frac{S_0 - S_f}{1 - Fr^2} \quad (30)$$

The typical slope types associated with WSP classification are *mild* (M), *steep* (S), *critical* (C), *horizontal* (H), and *adverse* (A). Each slope type is determined by the relationship between actual bed slope, S_0 and critical slope, S_c (critical slope is the slope that causes critical depth for given flow parameters). The slope in turn affects the relationship between y_c and y_n , the normal depth of flow. Normal depth occurs when the bed slope and friction slope are equal, and this depth remains constant along the channel length. Table 2 shows a summary of WSP classifications, and Figure 9 illustrates the behavior of each WSP (Chow, 1959). The *zone* is the region where the actual water depth y is, relative to y_c and y_n .

Table 2: Types of Flow Profiles

Channel Slope	Designation			Relation of y to y_c and y_n			General curve type	Type of Flow
	Zone 1	Zone 2	Zone 3	Zone 1	Zone 2	Zone 3		
Horizontal $S_0 = 0$	NA			$y > y_n > y_c$			NA	NA
		H2		$y_n > y > y_c$			Drawdown	Subcritical
			H3	$y_n > y_c > y$			Backwater	supercritical
Mild $0 < S_0 < S_c$	M1			$y > y_n > y_c$			Backwater	Subcritical
		M2		$y_n = y = y_c$			Drawdown	Subcritical
			M3	$y_n > y_c > y$			Backwater	supercritical
Critical $S_0 = S_c > 0$	C1			$y > y_c > y_n$			Backwater	Subcritical
		C2		$y_c = y = y_n$			Parallel to bottom	Uniform-critical
			C3	$y_c > y_n > y$			Backwater	supercritical
Steep $S_0 > S_c > 0$	S1			$y > y_c > y_n$			Backwater	Subcritical
		S2		$y_c > y > y_n$			Drawdown	supercritical
			S3	$y_c > y_n > y$			Backwater	supercritical
Adverse $S_0 < 0$	NA			$y > y_n > y_c$			NA	NA
		A2		$y_n > y > y_c$			Drawdown	Subcritical
			A3	$y_n > y_c > y$			backwater	supercritical

Figure 9: Water Surface Profiles in Gradually Varied Flow

3.1.5. Channel Lining

Two types of linings, rigid or flexible, can be used for open channels. Rigid linings are comprised of a fixed, impermeable material such as concrete. One characteristic of rigid linings is the inability to conform to deflections of the base material, and rigid linings therefore tend to crack or break due to any deformation. Due to their structural capacity, rigid linings typically are not affected by erosion and have no maximum permissible shear stress or velocity. Flexible linings, however, are capable of conforming to ground deflections and retain their structural integrity. Examples of flexible linings are riprap, gabions, and grass. Flexible linings are an environmentally preferred lining type, as they allow for infiltration and are similar to natural channels and habitats. Flexible linings also tend to be less expensive. Table 3 summarizes the advantages and disadvantages of both lining types (Director of BHD, 2007).

Table 3: Advantages and Disadvantages of Rigid and Flexible Linings

Advantages	Disadvantages
Rigid Linings	
Large capacity	Expensive to construct and maintain
Prevent erosion	Unnatural appearance
More applicable when right of way is limited	Prevent/reduce natural infiltration
Underlying material completely protected	Scour at downstream end
	Lining may be affected by undercutting, channel head cutting, or hydrostatic pressure
Flexible Lining	
Less expensive	Limited depth due to erosion
Self-healing	Low capacity
Permit infiltration and exfiltration	Requires more right of way
Natural appearance	Riprap may be unavailable
Provides filtering media for runoff contaminants	May not be able to establish vegetation
Lower velocity	

3.2. Hydraulic Modeling

The first step in the development of hydraulic floodplain modeling was the use of physical models. These scale models of real river systems were used to mimic different scenarios in order to predict the outcome of an event. By the 1940s, however, there arose a demand for a faster, easier method of hydraulic modeling. Hand computations were too time-consuming, and physical modeling was costly, impractical, and too case specific. The solution came in the early 1960s, when the first hydraulic modeling programs were developed. These models were simplistic and limited to only simple cross-sections; complicated sections such as those involving bridges still had to be analyzed by hand. The Hydrologic Engineering Center (HEC) of the United States Army Corps of Engineers (USACE) fronted the development of such software and incorporated the ability to analyze more complicated cross-sections with the release of their program HEC-2 in 1968 (Dyhouse, Hatchet, & Benn, 2003; Azagra, 1999).

Hydraulic modeling software was a huge step in the practice of floodplain modeling in several ways. It allowed for a faster analysis, and the methods and formulae used were widely accepted by most hydraulic engineers (Dyhouse, Hatchet, & Benn, 2003; Sui & Maggio, 1999; Pappenberger, Beven, Horritt, & Blazkova, 2005). As the modeling capabilities increased and improved, flood events could be predicted and then better analyzed using modeling software. Though solutions were only approximate, they accurately compared to other models or recorded events in predicting inundation, WSE, etc. (Hicks & Peacock, 2005). This created a great advantage by enabling engineers to create a range of flooding cases, specifically worst-case scenarios (i.e. 100- or 500-year events) rather than formulating guesses based on record events or waiting for an event to occur.

Though the practice of hydraulic modeling was greatly improved by the introduction of modeling software, it was still difficult to visually represent the results of the analysis in a detailed and digital manner. It became clear to hydraulic engineers that the best way to present the results of a floodplain analysis was through geographic representation.

3.2.1. Integration of GIS into Hydraulic Modeling

As the development of hydraulic modeling progressed, so did the development of geographical information systems (GIS). In the same way that hydraulic modeling needed to integrate a way of displaying the results of floodplain analyses, GIS researchers realized that hydraulic modeling capabilities would greatly improve their products. Both practices recognized a mutual need for the other's capabilities and began developing together in the 1980s, and various basic types of relationships between the two software were developed (Sui & Maggio, 1999; Tate, 1999). One type of relationship was one in which a GIS program has hydraulic modeling capabilities. The issue with these types of programs is that the hydraulic modeling is basic and lacks the detail in capabilities and calibration of other hydraulic modeling software (Sui & Maggio, 1999). A second type was one in which a hydraulic model uses GIS primarily as a simple mapping tool. Though it has poor graphical visualization, it caused a great advancement in the quality of the model output, and is the format of most leading modeling software today, including RiverCAD, HEC-RAS, and MODFLOW (a groundwater flow model). GIS became a fundamental tool for hydraulic modeling with the ability to import and export between GIS and modeling software by allowing for better river delineation capabilities provided by GIS to generate more accurate river profiles for model input, as well as better visualization of output by exporting results from a model into GIS.

3.2.2. Issues of Integrating GIS and Hydraulic Modeling Software

While the integration of GIS and hydraulic modeling software greatly improved the functionality of hydraulic modeling in floodplain analysis, several issues needed to be dealt with when communicating between the two different systems. One of the most common issues was how each technology represented spatial relation. From a flow standpoint, GIS focuses on a moving point (Lagrangian motion), whereas hydraulic models focus on a fixed point as fluid passes through it (Eulerian motion) (Sui & Maggio, 1999). From an orientation standpoint, GIS is based on actual map coordinates defined by geographic datums using northings and eastings, whereas spatial orientation in hydraulic models is relevant only to the model.

One of the more common formats of GIS data is in the form of a triangular irregular network (TIN), which is a digital representation of a geographic surface created using quasi-equilateral triangles whose corners represent the elevation of a given point. This differs greatly from hydraulic modeling software, in which a river is usually defined by a one dimensional centerline along which cross-sections are defined at arbitrary river station locations. Within each station, x- and y-coordinates are defined in a manner specific to that station, representing distance along that station's cross section and elevation. To remediate this issue, programs were developed to communicate between GIS and hydraulic models, such as ARC/HEC2, which was made specifically for conversion between ArcView and HEC-2 (Tate, 1999; Azagra, 1999). These packages were able to convert the file format of GIS data so that it was compatible with the hydraulic modeling software, which would then run the analysis. After the analysis was run, the mediating software would retrieve the output and produce a new GIS representation of the results.

3.2.3. HEC-RAS

When HEC-2 was created, it was a big advancement in the development of hydraulic modeling as it was one of the first models that could perform analyses quickly and had capabilities that other programs lacked, such as analyzing hydraulic structures such as bridges and culverts. However as software modeling enhanced, the MS-DOS based HEC-2 slowly fell behind. In 1991, HEC began developing a new Windows based software to replace the HEC-2 coded in FORTRAN (Yang, Townsend, & Daneshfar, 2006), and released the Hydrologic Engineering Center River Analysis System (HEC-RAS) in 1995 (Dyhouse, Hatchet, & Benn, 2003). Due to its wide applicability and accuracy, most government agencies today have generated HEC-RAS models of major flood risk areas within their jurisdiction. Hicks and Peacock (2005) used the HEC-RAS model to perform an unsteady analysis to show that HEC-RAS can be applied to obtain comparable results to those of more sophisticated hydraulic models, in this case SSARR (Streamflow Synthesis And Reservoir Regulation) and River1-D, used for the same analysis. These three models were applied to simulate a flood event in 1987 in the Peace River, Alberta, Canada.

HEC-RAS was the solution to several issues, regarding both the development of hydraulic modeling capabilities and integration of GIS with hydraulic modeling. New and improved analysis capabilities included: channel modification analysis, mixed-flow, hydraulic structures (weirs, gates, etc.), bridge analysis, sediment transport modeling, and modeling of changes in Manning's roughness coefficient (Dyhouse, Hatchet, & Benn, 2003; Yang, Townsend, & Daneshfar, 2006). HEC-RAS can also model subcritical and supercritical flow, can model either an entire network of rivers or a single river, and is one of few models that can run unsteady flow analyses. HEC-RAS's unsteady capabilities give it at great advantage over other models, and are

a big step in modeling development as unsteady modeling was avoided due to time and its complicated nature (Hicks & Peacock, 2005).

The other big improvement was the capability to import and export GIS data, thus eliminating the need for a program such as ARC/HEC2. A specific improvement in this area that was lacking in the HEC-2 software was that HEC-RAS now had a built-in compatibility for communicating with ArcView (Tate, 1999). HEC-GeoRAS is an ArcGIS extension designed to process geospatial data for a hydrologic area created in ArcGIS into a format specifically compatible with HEC-RAS. HEC-GeoRAS also has the capability to take exported HEC-RAS output data and use it to generate water surface profile maps in ArcGIS (Yang, Townsend, & Daneshfar, 2006; HEC2, 2009). Other expansions of HEC-RAS capability have since been created, such as RiverCAD, which implements the HEC-RAS program in conjunction with AutoCAD and Civil 3D (Boss International, 2008). Yang, Townsend, and Daneshfar (2006) investigated the applicability of using HEC-RAS model outputs to generate flood plain maps in GIS by transferring HEC-2 models for the South Nation River system, Ontario, Canada to HEC-RAS. A steady state analysis was used to generate WSE maps, which were overlain on GIS terrain maps to visualize the inundated areas as a result of the modeled flood. Tate (1999) performed a similar analysis in which a HEC-RAS model was used to explore the process of generating flood plain maps using HEC-RAS and ArcGIS in the case of a steady state simulation of an event in Waller Creek, Austin, Texas. The HEC-RAS model outputs were exported to ArcGIS and georeferenced (coordinates were spatially adjusted to match the GIS terrain model) in order to generate flood plain maps.

HEC-RAS is a one dimensional (1D) model, making it much simpler to use than more sophisticated 2D models. Due to their simplicity, 1D models are often used in standard practice. Horritt and Bates (2002) applied HEC-RAS and TELEMAC-2D to model two flood events in the

River Severn, United Kingdom, during the summers of 1998 and 2000 to compare the results of steady state analyses of 1D (HEC-RAS) versus 2D (TELEMAC) models. Cook (2008) presented a similar study using HEC-RAS and the 2D model FESWMS (Finite Element Surface-Water Modeling System) to perform a steady state analysis of Strouds Creek, North Carolina and the Brazos River, Texas. The sensitivity of the models to terrain data, geometric configuration, and dimensionality was analyzed using different topographic data sets to generate the models.

A 1D model is one in which flow is simulated such that it travels in only one direction: the longitudinal, downstream direction. 2D models add another flow component in the lateral direction. Neither model incorporates flow in the vertical direction. 1D and 2D models also differ in the way in which spatial and elevation data are represented. In a 1D model, lateral cross-sections are defined at various points along a reach, each of which is defined by a series of x- and y- coordinates to represent the profile of the cross-section. Because of the added dimension, 2D models represent ground elevation using a mesh of interconnected and triangulated points, creating a continuous surface. While this creates a more continuous and representative model, it is much more complicated. Due to its simplification, a 1D model like HEC-RAS can be advantageous in that a simulation may take only a few seconds to run, while a 2D model may take up to a few hours (Cook, 2008). Also, point data for stage or discharge measurements is typically more applicable to 1D models (Horritt & Bates, 2002). Studies have shown that even in its lack of dimensionality, HEC-RAS is able to produce results comparable to those of 2D models such as FESWMS-2D and TELEMAC-2D. When compared to either model, HEC-RAS was able to comparably predict inundation areas (Cook, 2008; Horritt & Bates, 2002). The simplicity of HEC-RAS is due, in part, to the assumptions used by the program, which include (Pappenberger, Beven, Horritt, & Blazkova, 2005; Tate, 1999):

- There is an average, perpendicular velocity normal to each cross-section.
- Energy head is constant across each cross-section.
- Vertical acceleration can be neglected.
- The model does not account for erosion and deposition.

Since its release in 1995, HEC-RAS has been upgraded to include analyses for ice jams/breaks, bridge scour, analysis at river junctions, water quality, sediment transport, output visualization components including 3D plots and RAS Mapper, leakage, pump station and pump regulation, and others. HEC-RAS by default employs the standard step method to compute water surface elevations, but is also capable of applying the momentum balance, WSPRO (Water Surface Profile) method, and the Yarnell method (HEC, 2010). Depending on water level in a natural channel, the Manning's roughness has the potential to vary. HEC-RAS is still limited in its ability to account for varying roughness for any segment of a reach or cross-section with respect to water depth. Though HEC-RAS is capable of indicating the occurrence of a hydraulic jump, its representation and calculation is hindered by its single dimensionality and inability to process RVE scenarios. A hydraulic jump may be recognized by a drastic drop and spike in the WSE that crosses from supercritical to subcritical flow. Another inaccuracy pertaining to output representation is related to the extent of each cross-section. In the event that the model predicts water levels outside of the cross-section dimensions (i.e. flooding), HEC-RAS is only able to indicate the water surface within the channel cross-section up to the maximum of the dimensions specified (Figure 10). Also, as can be seen in Figure 11, HEC-RAS displays the water surface in the same level as the main cross-section in the areas separated by obstructing ground surface though not overflowing. With the capability to interpolate between two defined cross-

sections to generate intermittent cross-sections, proper care must be taken in order to ensure that the linearly interpolated cross-sections accurately represent the channel geometry (Yang, Townsend, & Daneshfar, 2006).

Figure 10: HEC-RAS Cross-Section Indicating Water Surface Up to the Maximum Dimension of Cross-Section in the Case of Flooding

Figure 11: HEC-RAS Output Indicating Water Surface Outside of Main Channel Without Overflow

The typical process for a hydraulic or model study is as follows. Terrain is processed through a GIS system, such as ArcGIS, in which it is represented by either a digital terrain model (DTM) or a TIN. A DTM is a raster grid made up of uniformly sized cells representing spatial data, typically elevation (digital elevation model, DEM), whereas a TIN is a mesh of points containing x-, y-, and z-coordinates representing lateral positioning (x and y) and elevation (z). The points are interconnected to generate a surface made of triangular shapes, which represent the ground surface and slope. A steeper surface will have a more complicated TIN structure (smaller triangles). A TIN is often preferred for representation as it is able to generate a three dimensional shape. Through various processing tools available in programs like ArcGIS, a river profile can be delineated. Extensions such as HEC-GeoRAS can then be used to create an import file for a hydraulic model, i.e. HEC-RAS. In the hydraulic model, data (cross-section, elevation, roughness, flow, etc.) can be modified. After running the model, an export file can then be generated to be converted back into the GIS program (ArcGIS) for geographical representation and enhanced visualization of the results.

Figure 12: TIN Land Surface

78	72	69	71	58	49
74	67	56	49	46	50
69	53	44	37	38	48
64	58	55	22	31	24
68	61	47	21	16	19
74	53	34	12	11	12

Figure 13: Raster DEM Indicating Elevation of Each Cell

DTMs are available primarily from the United States geological survey (USGS), and are available in multiple resolutions. As it would seem logical that different resolutions would return different results, studies by Brandt (2005), Omer (2003), and Casas et al. (2006) have assessed this concept and how it affects the results of a HEC-RAS model versus other models by analyzing the Eskilstuna River in Sweden, Leith Creek in North Carolina, and the Ter River in Spain, respectively. These studies show that for lower resolution DEMs, the predicted area and width of inundation is higher than for models generated using a higher resolution DEM. It has also been shown that the difference between HEC-RAS results and 2D model results were more similar when more cross-sections were used in HEC-RAS (Cook, 2008; Casas, Benito, Thorndycraft, & Rico, 2006; Sanders, 2007). Model calibration is another factor in sensitivity, and HEC-RAS was shown to have an advantage over other models in this context. When compared to SSARR, Hicks and Peacock (2005) showed that HEC-RAS returned results comparable to both SSARR and observed data, even without calibration. This is an advantageous attribute in the case of drastic flow analyses, since it is possible that calibration of the model for normal conditions may not represent an extreme event. As it is often the case that available data is limited, HEC-RAS was shown to be the best model to be used to achieve

accurate flow routing and determine water levels with limited or approximate data (Hicks & Peacock, 2005; Horritt & Bates, 2002).

Hydraulic modeling software has been widely accepted as the most efficient method for floodplain modeling. As it still implements the classic methods of hydraulic analyses, it has grown to allow hydraulic engineers a fast, reliable solution to predict hydraulic events and design hydraulic structures and floodplains. Overtaking its predecessor as the development of software modeling progressed, HEC-RAS has become one of the leading software programs, effectively incorporating GIS technology to produce clear, visual, and accurate results. Due to its sensitivity to input parameters such as topography and Manning's roughness coefficient, HEC-RAS is an excellent tool for investigating river behavior and how modifications would affect a river and its floodplain. The following sections describe how HEC-RAS was used to modify the study reach of San Jose Creek in order to improve the capacity to withstand a 100-year event.

Chapter 4

Methodology

This section discusses the tools and the process used for this study. The model provided by Bengal Engineering is described. The process for the analysis is discussed along with the modification components to establish the parameters of each modification.

4.1. Original Model

This study was based off a HEC-RAS model provided by Bengal Engineering in Santa Barbara, which included channel geometry, elevations, boundary conditions, and 100-year storm steady state data. The model consisted of the portion of San Jose Creek between its downstream most end where it converges with San Pedro Creek and extending up through the urban section of Goleta (Figure 2). However, only the constructed portion of the creek along Highway 217 was analyzed (Figure 3). The USGS maintains two stream gages along San Jose Creek; gage 111202500 is located in the mountainous portion of the creek and gage 111202510 is located along the realigned portion of the creek. The 100-year flow rates for these gages were reported to be 4,040 cfs and 5,210 cfs, respectively (NHC, 2010). The corresponding 100-year flow rates used in the HEC-RAS model were 4,200 cfs and 5,300 cfs. These model discharge values were close to the reported ones and used for the presented analysis since higher discharge values provided a conservative analysis of the flooding issue.

Figure 14: USGS Stream Gage Locations

4.2. Tidal Influence

Another possible source of flooding, specifically in the most downstream end of the creek where it converges with San Pedro and Atascadero Creek, could be inundation due to tidal influence. The sea water elevation for various storm intensities is shown in Figure 15 for the areas of San Diego and Los Angeles, indicating the trend before and after the 1982-1983 storms (Walker, Nathan, Seymore, & Strange, 1984). For the Santa Barbara and Goleta area, these values can be reduced by approximately 0.05 feet. These values are referenced to mean lower low water (MLLW), which is approximately 0.1 feet below the North American Vertical Datum -1988 (NAVD88), the HEC-RAS model datum (NOAA, 2001). Figure 16 shows a topographic map of the study area. As can be seen from this map, the area where the three creeks converge is below 10.0 feet, and may be subject to inundation during storm events. However, as this is outside the

focus of this hydraulic analysis of the realignment, and HEC-RAS would be incapable of modeling the tidal influence, backflow from tidal influence was not considered for this analysis.

Figure 15: Sea Water Elevation Versus Recurrence Interval

Figure 16: Topographic Map of San Jose Creek Study Area

4.3. Modification Components

For this study, the existing portion of San Jose Creek along Highway 217 is to be modified to generate a new channel geometry that will adequately facilitate the 100-year flow described above. Due to the intensity of the 100-year flow and past events, it was determined that the entire allowable width must be utilized. As this portion of the creek runs between the highway and developed area, the allowable width was determined to be limited to 50 feet. Therefore, all proposed modifications were set at 50 feet wide. The only other modification components were variations in the channel bed elevation, and the channel roughness. As the original channel section is trapezoidal, and still overflowing for a 100-year peak discharge, it is evident that a strictly trapezoidal channel would be inadequate. Further analysis and results lead to the same conclusion, as described in section 5.1.4. Thus, all modification plans incorporate vertical side walls. The geometry of each modification plan is described in section 5.1.

Three lining materials were considered for analysis representing low, medium, and high Manning's roughness values. As the original channel is already lined with concrete, concrete was considered as one of the options for channel lining. Since concrete lining is rigid, impermeable, and expensive, other lining materials were analyzed in various combinations based on flexibility, environmental friendliness (i.e. allowance for infiltration), and cost. One material proposed was riprap, while the other was a product made by Shoretech called shoreblocks, which was chosen based on the expressed interest of Santa Barbara County in articulated concrete block mats.

Articulated concrete blocks (ACBs), specifically shoreblocks, are a mat of pre-formed concrete blocks typically 16" x 16" x 6" (dimensions can vary), interlinked with steel cables. ACBs can be

open or closed cell units, as depicted in Figure 17. ACBs can be a very effective channel lining in that they provide flexibility as well as structural integrity and strength, and allow for vegetation and infiltration (Figure 18). Shoreblocks are very advantageous in that they typically require no maintenance.

Figure 17: Shoretech Shoreblocks - Closed Cell (left) and Open Cell (right)

Figure 18: Shoretech Shoreblocks in Barker Canal, Cameron Parish, Louisiana

As mentioned previously, each plan utilized the maximum 50-foot width available. Another commonality between plans was the necessity for vertical side walls, as it was apparent from the creeks current design that a trapezoidal channel does not adequately convey the design flow. It was assumed that the best lining material for the side walls would be concrete for structural stability. For the side walls and other sections of the channel bed that were lined with concrete, a Manning's roughness of $n = 0.013$ was used. In instances where shoreblocks were used as the channel lining, a roughness $n = 0.032$ (provided by Shoretech). Due to the potential for a high flow depth, the equation (31) below was used to determine the roughness of riprap as a function of depth and the size of riprap used (Blodgett, 1986).

$$n = \frac{\alpha d_a^{1/6}}{2.25 + 5.23 \log \frac{d_a}{D_{50}}} \quad (31)$$

In the above equation, d_a is the water depth (ft), D_{50} is the average diameter of the riprap (ft), and the term α is a unit factor and is equal to 0.319 (SI) or 0.262 (US). To determine the actual roughness, an initial roughness was assumed and analyzed, from which the resulting water depth was used to determine a new roughness value until there was little variation in input and output values for the depth and roughness. The parts of the channel geometry that were outside of the main channel, namely the regular ground surface, were assumed to have a roughness of 0.04 for all modifications.

One main advantage of rigid lining over flexible lining is maintenance requirements. While a riprap lining would provide flexibility, it would require a high amount of maintenance. Though the initial installation/construction may be more costly, implementing concrete as a lining could be advantageous overtime as it would have eliminated the maintenance cost. The shoreblock

product is very advantageous in that it provides a high amount of durability, and thus requires no maintenance, yet provides adequate flexibility. Another advantage is that since the blocks are connected, the installation process is much more rapid than that for other linings.

4.4. Analysis Process

For this study, the model provided by Bengal Engineering was modified to find alternate channel geometries that adequately convey the design flow. This was done by redefining the channel geometry and lining of the study reach. Various geometries were generated, each with various lining schemes combining concrete, shoreblocks, and riprap. The upper and lower limits of the study reach were selected as river stations 6100 and 2300, respectively. Modified channel geometry and linings were first defined for stations 6100 and 2300. As the channel was to be designed as a prismatic channel, cross-sections for the reach between the two end stations were generated using the interpolation tool in the geometric data editor at 100-foot intervals. A steady state analysis was then performed to determine the adequacy of each modification.

Channel geometry is defined using the geometric data editor in HEC-RAS (Figure 19). Using this feature, a linear river centerline can be defined to represent the pathway that a river will follow. Along this river line, *river stations* are created to define the cross-section of specific points along the river, denoted by a station number that represents, in general, the upstream distance from the river end to that river station. Within the geometric data editor, options are available to create junctions from inflowing tributaries, bridge structures (station 14085 in Figure 19), inline structures such as weirs, lateral structures such as culverts, storage areas, pump facilities, and cross-sections.

Figure 19: HEC-RAS Geometric Data Editor

Cross-sections are defined by lateral and elevation coordinates using the cross-section editor, as shown in Figure 20. Each cross-section has its own local lateral coordinate system, the numbering of which is referred to as the *station* and begins at an arbitrary starting number from the left side of the section, as each cross-section is viewed from upstream looking downstream. Other inputs in the cross-section editor used to define each cross-section are

- Reach length, representing the distance to adjacent downstream cross-section,
- Elevation, which represents the ground elevation at a given cross-section station,
- Manning's roughness coefficient,
- Expansion and contraction coefficients, and

- Left and right overbank (LOB, ROB) locations defining the boundaries of the main channel.

The cross-section editor also includes a graphical representation of the cross-section indicating features such as WSE, critical depth, EGL, and bank station locations.

Figure 20: HEC-RAS Cross-Section Editor

Each modification was comprised of variations in channel bed elevation and lining. Each modification with a similar geometry was denoted as a “plan” followed by a letter, while lining variations within each plan were denoted by a number. For example, Plans A-1 and A-2 had the same cross-sectional geometry, but had a different channel lining scheme. As HEC-RAS is able to contain multiple geometries for a given project, each modification variation was saved as its own geometry file. Similarly, when performing an analysis, HEC-RAS is capable of containing

multiple plans to be analyzed, each of which represents a specific geometric data set and flow data set.

For this study, each plan contained the same flow data and 100-year discharges. Flow data can be entered as either steady or unsteady. HEC-RAS allows for the definition of multiple flow profiles, and the flow rate at any river station can be defined for the reach downstream of that station. Figure 21 illustrates this, showing multiple flow rates for various sub-reaches within the model and different flow profiles (100-year, 2-year, etc.).

In order to perform an analysis, it is necessary to define boundary conditions. In HEC-RAS, this is done in the Steady and Unsteady State Flow Data Editors (Figure 21 and Figure 22, respectively). For a subcritical flow condition, only a downstream boundary condition is needed. Conversely, an upstream boundary condition is required in the case of supercritical flow. For a mixed flow analysis, both boundary conditions must be provided. Boundary conditions for steady state analyses include a known WSE, critical depth, normal depth, or a rating curve. Figure 22 shows the unsteady flow editor. Boundary conditions for unsteady flow include flow and/or stage hydrographs, normal depth, various control structures, and others (HEC₃, 2010).

Steady Flow Data - Proposed Floodway

File Options Help

Enter/Edit Number of Profiles (25000 max): Reach Boundary Conditions ...

Locations of Flow Data Changes

River:

Reach: River Sta.:

Flow Change Location			Profile Names and Flow Rates							
	River	Reach	RS	100-year	FW	2-year	Typical Winter Storm	2.3-year	50 cfs	300
1	San Jose Creek	Upper Reach	17067	4200	4200	600	150	675	50	300
2	San Jose Creek	Upper Reach	14665	5400	5400	600	150	675	50	300
3	San Jose Creek	Upper Reach	9011	5300	5300	600	150	675	50	300

Initial Flow Splits Specified

Edit Steady flow data for the profiles (cfs)

Figure 21: HEC-RAS Steady State Flow Data Editor

Unsteady Flow Data

File Options Help

Boundary Conditions | Initial Conditions |

Boundary Condition Types

Stage Hydrograph Flow Hydrograph Stage/Flow Hydr. Rating Curve

Normal Depth Lateral Inflow Hydr. Uniform Lateral Inflow Groundwater Interflow

T.S. Gate Openings Elev Controlled Gates Navigation Dams IB Stage/Flow

Rules

Add Boundary Condition Location

Select Location in table then select Boundary Condition Type

	River	Reach	RS	Boundary Condition
1	San Jose Creek	Upper Reach	17067	
2	San Jose Creek	Upper Reach	70	

Figure 22: HEC-RAS Unsteady Flow Data Editor

The study used a steady state analysis with the 100-year flood discharges. As it is a standard practice to simulate high flows using a peak flow rates at a steady state condition (Hicks & Peacock, 2005; Cook, 2008), this was the approach taken for this study. Analyzing the channel under steady state at 100-year peak flow conditions would adequately represent the worst-case flooding scenario and an unsteady analysis was deemed unnecessary. Therefore, the analyses performed in this study incorporated only a steady state analysis. The upstream boundary condition for this analysis was set as the critical depth, and the downstream boundary condition was set as a known WSE of 11.0 ft (as provided by Santa Barbara County). Because the creek was subject to both supercritical and subcritical flow, it was necessary to have both upstream and downstream boundary conditions.

From each analysis, HEC-RAS was able to calculate and display results in graphical and/or tabular form, including but not limited to,

- water surface elevation (WSE),
- energy grade elevation (EGL),
- energy grader slope,
- critical depth elevation,
- flow rate,
- minimum channel elevation (elevation of channel bottom),
- main channel velocity,
- area of flow,
- top width,
- shear,
- Froude number, and

- channel surface area.

The results of the final modification plans are described in the following section. The parameters considered for analysis consisted of flow depth, Froude number, shear stress, velocity, and cost. The main concern with flow depth was the risk of overflow at any point along the channel. A criterion for the Froude number was to maintain a value of $Fr < 0.86$ or $Fr > 1.13$ in order to prevent fluctuation from causing standing waves as discussed in section 3.1.2. Shear stress and velocity were both analyzed to assess erosion effects. Cost for each plan was determined to determine the most economic solution.

4.5. Modification Plans

Four basic cross-sectional geometries were used during this analysis. Channel cross-sections ranged from approximately 20 feet deep at the upstream end of the channel to 10 feet deep at the downstream end of the channel. The reduction in channel depth was primarily due to the elevation of the downstream end as the channel approaches the coast. Due to the constraints of the highway and development on either side of the channel, a maximum 50 foot width was assigned, as mentioned in section 4.2.

For each channel geometry considered, elevations were entered at stations 6100, 2300, and 2275 in order to align the upstream and downstream ends of the channel to the desired geometry, and cross-sections were interpolated between stations 2300 and 6100 at 100-foot intervals to obtain cross-sections for the rest of the channel.

The four general cross-sectional geometries analyzed in this study were each designated as Plan A, B, C, and D (Figure 37, Figure 38, Figure 39, and Figure 40 in Appendix B show the general

geometry of Plans A, B, C, and D respectively). For all geometries, the lowermost portion of the cross-section will be referred to as the main channel, while the elevated portions will be referred to as shelves (Figure 23). Plan A incorporated a rectangular cross-section (Figure 37). The geometric shape of Plan B was a symmetrical trapezoidal main channel, with vertical sidewalls and a shelf on either side of the main channel (Figure 38). Plan C was a quasi-trapezoidal section, comprised of a main channel and a single shelf separated by a 4:1 slope (Figure 39). Plan D was also designed as a trapezoidal shape, but varied from Plan B in that the side slopes were steeper, and there was a shelf on only one side of the channel (Figure 40). The channel was approximately 19.0 feet deep at station 6100 for each plan, and 10.0 feet deep at station 2300. All geometry configurations had vertical sidewalls. The various geometries are further described in Section 5.1.

Figure 23: Diagram of General Cross-Section

Chapter 5

Results

Using the data and procedures described in the previous sections, an analysis was conducted to determine the WSE and flooding potential of the proposed geometries and various lining schemes. This section includes those results as well as a suggestion for the most effective modification plan.

5.1. Modification Plans

The results of the proposed plans for the channel modifications are described below. Width was a limiting factor for modifying the cross-sectional geometry since a restriction of a maximum 50 feet was imposed and it was found that utilizing the entire 50 feet was necessary. Therefore, modification in the bed elevation of each cross-section was the only adjusted parameter for all channel geometries, with a constant width of 50 feet. For each plan, multiple variations were analyzed, each of which deferred in channel lining and roughness while geometry remained constant. Concrete was used as the lining for the side walls of each plan. Based on the various materials considered for the channel bottom, concrete, riprap, and the Shoretec shoreblocks were analyzed in various configurations. The roughness used for concrete, shoreblocks, and riprap were 0.013, 0.032, and 0.042 respectively.

It may be noted that the depth of each plan decreases as the channel approaches the downstream end. Channel bottom elevation was another limiting factor in this study due to the creek's close proximity to the ocean. Since the creek is so close to sea level, a smaller channel depth was required in order to maintain a channel bottom elevation above sea level and above that of the rest of the channel downstream to prevent a dip in the channel bottom. This change

in depth also affected the side slope and depth of some trapezoidal main channel sections. Appendix A contains water surface profile plots of each plan, and Appendix B contains illustrations of all the channel geometries (excluding depth, as depth is variable) described below, as well as lining schemes. Table 4 provides a summary of the results for each plan.

5.1.1. Plan A

Plan A was a rectangular channel ranging from 18.5 feet deep at the upstream end of the channel to 10.0 feet deep at the downstream end of the channel and maintained a constant bed slope of 0.0046. The channel bottom of the Plan A geometry included a 1% grade from either side of the bottom, sloping towards the center in order to collect and funnel water during low flow conditions and prevent meandering (Figure 37).

Four different configurations were analyzed for Plan A. The purpose of the proposed configurations Plans A-1 and A-2 was to analyze the necessity of having a material other than concrete as the channel lining and how appropriate it would be to line the channel bed with concrete. An advantage of using concrete with a pervious material through the center of the channel is that percolation and vegetation growth would be promoted during more average flow conditions, and higher flow would, conceptually, be better facilitated by the concrete bed lining to reduce the WSE during flood conditions. Plan A-1 was configured with an 8.0 foot wide section lined with shoreblocks in the center of the channel, with the remaining 21.0 feet on either side lined with concrete. Plan A-2 was a similar configuration, the only difference being that the 8.0 foot section was increased to 20.0 feet wide.

One advantage to these configurations was that the lower roughness was able to facilitate faster flow, thus lowering the WSE and greatly reducing the risk of overflow. Plan A-1 resulted in an

average Froude Number of 1.29. Plan A-2 had a flow depth very close to critical, with an average Froude Number of 1.04, and was therefore considered unacceptable. Plan A-3 was configured with a channel bed comprised entirely of shoreblocks, while Plan A-4 was configured with a channel bed comprised entirely of riprap (both plans still maintained concrete lined walls). Plan A-3 resulted in a Froude Number of approximately 0.79 throughout the channel, which is adequately low enough to prevent fluctuation to a supercritical condition. Plan A-4 resulted in an average Froude of 0.62. The configurations of Plan A-3 and A-4 were able to facilitate the 100-year flow without any overtopping.

Plans A-1 and A-3 most adequately conveyed the flow of the design 100-year flood. The shoreblock lining provided a durable bed lining to reduce erosion, yet allowed for percolation and vegetation growth within the channel. By maintaining subcritical or subcritical flow, wave action and overtopping were prevented, keeping the flood water within the walls of the channel.

5.1.2. Plan B

Plan B incorporated a trapezoidal main channel section with a shelf on each side, and vertical side walls (Figure 38). The trapezoidal section was 3.0 feet deep, with a bottom width of 8.0 feet and side slope H:V ratio of 4:1. The shelves on either side of the main channel were each 9.0 feet wide and had a 1% grade sloping towards the center to collect and funnel water towards the center during low storm conditions, and extended to the vertical side walls. The channel ranged from 19.0 feet deep at the upstream end to 10.0 feet deep at the downstream end and maintained a constant bed slope of 0.0046.

Similar to Plan A, multiple variations of Plan B were also analyzed, all of which varied in channel lining configurations. Each of the Plans B-1 through B-4 were configured such that the portions of the channel bed (in various combinations of main channel, side slopes, and shelves) was lined with either shoreblocks or riprap, while the rest of the channel was lined with concrete. These essentially served a purpose similar to that of Plan A-1, by analyzing the applicability of having a central portion that allowed for percolation, particularly during normal flow, while providing durability during higher flow and utilizing the concrete lining to better convey higher flow rates. From the analysis of Plan D described in Section 5.1.4, it was determined that a deep trapezoidal section does not adequately convey the design flow. As stated above, it was more effective to utilize as much flow area as possible. In order to still analyze a trapezoidal channel similar to a rectangular channel however, the depth of the trapezoidal section of Plan B was set to 3.0 feet deep, the side slopes were set to 4:1, and the shelves were set to 9.0 feet wide, with a side slope of 1% to divert water to the lower portion of the channel during low flow.

Plan B-1 was configured such that the 8.0 foot bottom portion of the trapezoidal section was lined with shoreblocks, while the side slopes, shelves, and walls were lined with concrete. This configuration resulted in very low flow depth, greatly reducing the chance for overtopping. This flow depth resulted in an average Froude number of 1.30, which is adequately larger than 1.0 to prevent wave action. Plan B-2 was configured such that the 4:1 side slopes of the trapezoidal portion of the cross-section were also lined with shoreblocks. This configuration resulted in an average Froude Number of 0.88, which was considered to be too close to 1.0 to be acceptable due to the potential for wave action.

Plan B-3 investigated the feasibility of using riprap instead of shoreblocks in the same lining configuration as in Plan B-2. This resulted in an average Froude Number of 0.72, which was

acceptable and greatly reduces potential for wave action. The flood water was contained within the channel for most of its length. Overtopping did occur however, beginning at station 3121 and continuing through the end of the channel, a distance of approximately 800 feet. The maximum overtopping depth in this case was approximately 1.0 feet.

The configuration of Plan B-4 used shoreblocks as the channel lining once again, this time covering the entire width of the channel bed, including the center and 4:1 side slopes of the trapezoidal section, and the shelves on either side. The average Froude Number for this configuration was 0.74, which adequately reduces the likelihood of wave production. Similar to the other configurations of Plan B, overtopping did occur at the downstream end of the channel, exceeding the banks by approximately 1.0 foot.

5.1.3. Plan C

Plan C incorporated geometry that can be described as a quasi-trapezoidal shape (Figure 39). The leftmost (when viewing the cross-section from upstream to downstream) wall extended vertically to the channel bottom. The main channel bottom width was 10.00 feet, and the main channel depth ranged was 5.0 feet deep. The side slope of the trapezoidal section was approximately 4:1 and the shelf was 20.0 feet wide. Similar to Plans A and B, Plans C-1, C-2, and C-3 all varied in the width of the section lined by shoreblocks. Only the bottom most section was lined in Plan C-1, while Plan C-2 included the lining of the side slope. The entire channel bottom, side slope, and shelf were all lined with shoreblocks in Plan C-3. The channel ranged from 19.0 feet deep at the upstream end to 10.0 feet deep at the downstream end and maintained a constant bed slope of 0.0046.

Similar to Plans A-1 and B-1, the configuration of Plan C-1 resulted in supercritical flow with a Froude number of 1.21. Overtopping did not occur in any portion of the channel. For these reasons, Plan C-1 was considered a possible solution. The configurations of Plans C-2 and C-3 both resulted in subcritical flow regimes, with average Froude Numbers of 0.90 and 0.71, respectively. Overtopping occurred in both cases at the most downstream end of the channel, beginning approximately at station 2838. Plan C-2 was considered unacceptable due to the Froude number, and overtopping made both C-2 and C-3 ineligible.

One point of concern with Plan C is that though the channel geometry was constant, there was an increase in the critical depth at the downstream end of the channel. This had no affect on the WSE in Plan C-1, however there was some fluctuation in the WSE in Plans C-2 and C-3 causing overtopping. A cause for the change in the critical depth and the WSE could not be determined since the geometry remains constant through the length of the channel in which these irregularities occur. These irregularities can be seen in the profile plots for Plans C-1, C-2, and C-3 (Figure 32, Figure 33, Figure 34, respectively, in Appendix A).

5.1.4. Plan D

Plan D investigated the affect of using a trapezoidal channel bottom larger than those in Plans B and C. Though Plans B and C incorporated trapezoidal shapes, the side slopes of Plan D were steeper and the trapezoidal section was deeper. Plan D was set up such that trapezoidal section began on the left side of the channel with the vertical side wall meeting the side slope of the main channel (Figure 40). At station 6100, the side slop was at a 1.5:1 ratio and gradually changed to 3:1 at station 2300. The main channel bottom width was 5.0 feet, and the opposite side slope was 2:1 at station 6100 and gradually decreased to 4:1 at station 2300. The

remaining width of the channel was a 10.0 wide shelf. Due to the large flow area necessary to convey the design flow, a strictly trapezoidal channel was infeasible. This is obvious in that the channel geometry of San Jose Creek currently is strictly trapezoidal in shape, as can be seen in Figure 4. Therefore, vertical side walls were necessary to increase the flow area.

As in Plans A-C, Plan D-1 was configured such that the bottom portion of the trapezoidal section was lined with shoreblocks, while the side slopes, shelf, and walls were lined with concrete. This configuration reduced WSE well below the channel banks and resulted in a Froude Number of 1.46, causing highly supercritical flow. At the upstream end of the channel, the bridge at Hollister Ave. was subject to overtopping. Plan D-2 was configured such that only the shelf and side walls were lined with concrete while the bottom-most section and the side slopes were lined with shoreblocks. In this case, the Froude Number resulted as 0.86. Though this is the limit of the Froude Number range specified in section 3.1.2, it was decided that the potential for the Froude Number to fluctuate in and out of that range is undesirable. In addition, the bridge at Hollister Ave. was overtopped in this scenario, as well as the most downstream portion of the channel. As in Plan C, there was a increase in critical depth at the downstream end of the channel (Figure 35 and Figure 36 in Appendix A), as well as an irregular WSE in the same section for Plan D-2. Plans D-1 and D-2 were therefore considered unacceptable. Riprap linings were not analyzed for Plan D due to the tendency for the bridge to be overtopped. Using a high roughness would increase the overall WSE, increasing the chance of overtopping.

Table 4 on the following page summarizes the discussion in section 5.1.

Table 4. Summary of Proposed Plan Configurations

Plan	n ¹ (side/main)	Fr (average)	Overtopping		Velocity (ft/s)	Bed shear stress (psf)	Feasible?
			Channel	Hollister Ave. Bridge			
A-1	0.013/0.032	1.29	No	No	18.07	1.42	Yes
A-2	0.013/0.032	1.04	No	No	15.43	1.54	No
A-3	0.032	0.79	No	No	12.59	1.94	Yes
A-4	0.042	0.62	No	No	10.93	2.11	Yes
B-1	0.013/0.032	1.30	Yes ²	No	17.67	1.38	Yes
B-2	0.013/0.032	0.88	Yes ²	No	13.76	1.66	No
B-3	0.013/0.042	0.72	Yes ²	No	12.13	1.89	Yes ⁴
B-4	0.032	0.74	Yes ²	No	12.25	1.82	Yes ⁴
C-1	0.013/0.032	1.21	No	No	17.06	1.34	Yes
C-2	0.013/0.032	0.90	Yes ³	No	14.01	1.61	No
C-3	0.032	0.71	Yes ³	No	11.97	1.89	Yes ⁴
D-1	0.013/0.032	1.46	No	Yes	19.43	1.45	No
D-2	0.013/0.032	0.86	Yes ³	Yes	13.64	2.08	No

1. All vertical side walls n=0.013

2. Hydraulic jump at end of channel causes overtopping

3. Irregular WSE at downstream end of channel

4. Plan may be feasible with the addition of a wall at downstream end of channel

5.2. Other Ideas

One issue that was common for nearly all configurations described above was the risk of overtopping of the most downstream portion of the channel. Instances of this occurred in Plans B-1 through B-4, C-2, C-3, and D-2, and were most likely caused by a hydraulic jump. It is difficult to discern this solely by looking at the profile plots in Appendix A, as hydraulic jumps are improperly represented due to HEC-RAS's difficulty in displaying hydraulic jumps or other instances of RVF. Although the implementation of a wall would increase cost, it may be beneficial to include a wall in this section of the channel. Installing a wall at the downstream end would provide a way to decrease the chance of overtopping in that section of the channel. One other possible issue with the proposed configurations is inaccuracy of the ground surface and bank elevations in the model compared to the actual ground surface elevation. It is

possible that because the cross-sections in the model were interpolated based on the cross-sections at stations 6100 and 2300, the intermittent bank elevations in the model are higher than the actual elevations. Should either of these be the case, a possible remediation could be to use earth material removed during the construction of the channel to build up the ground surface and bank elevations. A wall, whether built using removed earth or another material, may serve the purpose of containing the water of a flow higher than the design 100-year flow. It should be noted that in several of the proposed plans, as well as the original model, the right bank elevation is approximately 1.0 foot higher than the adjacent ground level at the downstream end of the channel. In these instances, the geometry suggests that either a wall may be incorporated, or the actual ground level should be adjusted. A wall could be unnecessary in such sections by raising the ground level to match the bank elevation in the model.

In most cases, there was a drastic fluctuation in the WSE when transitioning from the study reach to the unlined remaining portion of the creek (which occurs at station 2300). Because HEC-RAS cannot calculate or properly display RVF, this is most likely due to a hydraulic jump as the channel geometry changes to an irregular, natural profile. Because of this, it is recommended that the transition be lined with a durable material to prevent scour in this section. In plans where shoreblocks are used, shoreblocks are the suggested material. Concrete is suggested otherwise, as riprap may be displaced due to the turbulence caused by the jump. Another possibility to dissipate the energy of the jump would be the use of a Type I stilling basin, which incorporates a paved, horizontal apron that spans the length of the jump. As the method for determining the dimensions are standardized, the design of a stilling basin would be left to the contractor (U.S. Bureau of Reclamation, 1987).

5.3. Cost Analysis

Of all the plans proposed in the above sections, only some of them were considered for further investigation, namely cost. Costs used for pricing are close approximate values gathered from multiple sources. Based on the various prices gathered for riprap, an average cost of \$125/yd³ was used for riprap in place. Using 6" riprap, and assuming a thickness of 12" (2 layers), this became approximately \$5.00/ft². The cost for transporting riprap was assumed to be \$4.50/yd³/mile, and was incorporated into the total cost based on the assumption that riprap would be provided by Bee Rock Quarry, the closest local quarry located approximately 25 miles from the site. The cost for concrete, with the inclusion of verticals walls, was estimated to be \$1000/yd³ for materials, transport, and placement. Concrete lining was assumed to have a thickness of 6", making the cost approximately \$18.50/ft². The cost for the shoreblocks, including transportation and installation, was reported as approximately \$10.00/ft². The material prices estimates are shown in Table 5. Though all plans were analyzed for cost, only some were further considered and are presented below. Table 6 includes the cost in millions of dollars for those plans deemed feasible in Table 4. The cost was determined based on the surface area to be covered by each lining for each cross-sectional plan. The costs displayed in Table 6 only represent channel lining costs. These values do not include costs for excavation and site preparation, etc., as such estimates would most likely be relatively the same for each case.

Table 5. Material Price Estimates

Material	Estimated price
Concrete	\$18.50/ft ²
Riprap	\$5.00/ft ²
Riprap shipping	\$4.50/CY/mi
Shoreblocks	\$10.00/ft ²

Table 6. Cost Estimates for Proposed Modification Plans

Plan	Area (1,000 ft ²)				Cost (\$M)
	Concrete	Shoreblock	Riprap	Total	
A1	262.70	29.60	0.00	292.30	5.16
A3	107.30	185.00	0.00	292.30	3.84
A4	107.30	0.00	185.00	292.30	3.30
B1	243.23	29.60	0.00	272.83	4.80
B3	151.70	0.00	121.13	272.83	3.72
B4	85.10	187.73	0.00	272.83	3.45
C1	239.08	37.00	0.00	276.08	4.80
C3	88.80	187.28	0.00	276.08	3.52

Since concrete is the most expensive of the three linings considered, the modification plans with the most concrete would be the most expensive. Thus Plans A-1, B-1, and C-1 (\$5.16M, \$4.80M, and \$4.80M, respectively) were considerably more expensive than other plans, and discarded.

As explained in section 5.2, some of the proposed plans may only be feasible with the incorporation of a wall or other structure, which would increase costs of material, labor, etc.

The cases in which this would be relevant are Plans B-3, B-4, and C-3. Another concern with Plans B and C in general are the shape of the main channel. Based on the confined space of the channel's location and the presence of the vertical side walls, it would be difficult to excavate complicated bed geometry such as those suggested by Plans B and C. In such cases, cost may increase due to increased time and labor required for excavation and site preparation. Based on this reasoning, Plans B-3, B-4, and C-3 were also discarded.

The remaining feasible plans to be considered are Plans A-3 and A-4, both of which incorporate a channel bottom completely lined with either shoreblocks or riprap. Though the riprap lining proposition is less expensive, it may be worth considering using shoreblocks for a few reasons.

One big advantage shoreblocks have over riprap is their maintenance requirements. Because of the interconnected and flat-laying nature of the ACB mats, and their durability as concrete units, shoreblocks are very resistant to damage due to both displacement and erosion. In the event that maintenance is needed, a shoreblock-lined channel would provide much easier access than a riprap channel. In addition, installation is streamlined because of the connectivity of the ACB mats, which would save considerable cost and time during installation. Another issue with the riprap is that the shear stress determined using HEC-RAS for the riprap lined channel (2.11 psf) is much higher than that for the shoreblock lined channel (1.94 psf) as shown in Table 4. The WSE is lower for shoreblock lining than that for riprap lining as well (Figure 26 and Figure 27, respectively). Since lower shear stress and WSE would be preferable, Plan A-3 would a better option in that regard. Though there was no potential for overtopping in the model for Plan A-4, a higher flow rate would have a greater risk for overtopping for Plan A-4 than for Plan A-3. Though the cost of maintenance has not been quantified and installation cost has not been further investigated, they should be considered when deciding the final modification.

Chapter 6

Conclusion

The objective of this study was to apply the HEC-RAS hydraulic modeling software to modify the downstream artificial reach of San Jose Creek in Goleta, California to improve the flood conveyance capacity of the reach. This study was in effort to suggest a channel geometry that can facilitate the flow of a 100-year flood better than the existing channel, thus preventing damages like those caused by historic flood events from affecting the surrounding area. The study reach was diverted in the 1960s to prevent the channel from running through the now-developed commercial area of Goleta, and is paralleled by this area and Highway 217 on either side. Both of these areas are subject to damage in the event that the channel overflows.

HEC-RAS was chosen as the modeling software to be used for this study for several reasons. Today, HEC-RAS has become the most widely used hydraulic modeling program. As a one dimensional hydraulic model, HEC-RAS has proven to be a competitive and preferable tool when compared to more sophisticated, two dimensional models. Because of its simplicity, and its application of the most fundamental methods for open channel flow, HEC-RAS offers accurate results that are quickly and easily attainable. Throughout its development, it has surpassed several other models with its capability to perform unsteady analyses, computations at bridge structures, ice jam and release scenarios, sediment transport, and other advanced computations. With its built-in ability to communicate with ArcGIS software through HEC-GeoRAS, it is a very appropriate tool for modeling terrain data processed in ArcGIS and generating results that can then be displayed graphically either in its own visual representation or in ArcGIS. Also due to its dimensionality and the functionality of the interface, channel geometry is easily modified to perform analyses on multiple geometries and flow scenarios.

For this study, four main modification plans were analyzed (Figure 37, Figure 38, Figure 39, and Figure 40). A 50-foot width limit was imposed due to the confined location through which the study reach passes, and the entire 50-foot width was utilized for the plans considered. Also, due to this limitation, it was determined that vertical walls were necessary to allow adequate area to contain flooding. The modification variables of each plan were channel bed geometry, and the roughness of the channel lining. Three different lining materials were chosen representing low, medium, and high Manning's roughness value. The materials analyzed were concrete ($n = 0.013$), as it is a typical channel lining material, shoreblocks ($n = 0.032$), and riprap ($n = 0.042$), which was also chosen based on its common application as a durable flexible lining material. Concrete was chosen because of its common use as a lining material. Shoreblocks were selected based on interest expressed by Santa Barbara County in articulated concrete blocks. Riprap was also chosen due to its common application as a durable flexible lining material.

The results of each plan were analyzed based on the Froude number, water surface elevation and overtopping potential, velocity, bed shear, and cost. Based on these criteria, Plans A-3 and A-4 were the best modifications of all the plans analyzed. Both plans were rectangular in shape, which was advantageous over other plans with quasi-trapezoidal bed geometries due to the ease of construction and greater flow area, specifically at the downstream most end of the study reach where the channel depth was restricted due to its close proximity to the ocean. The vertical side walls of both plans were lined with concrete, and the channel bed was lined with shoreblocks for Plan A-3 and six-inch riprap for Plan A-4. The main difference between these plans is cost, with A-3 being more expensive. However, as Plan A-3 incorporates the shoreblocks as the bed lining, it is suggested that Plan A-3 be chosen over Plan A-4 based on the

multiple advantages of using shoreblocks instead of riprap. Shoreblocks are more easily installed and require much less maintenance, which will justify the higher price of Plan A-3.

6.1. Future Ideas

The issue of the channels conveyance capacity can be further investigated to find a potentially more desirable solution. The following are suggestions for further study:

- As this study was in part intended to specifically focus on hydraulics, other flood prevention practices were not assessed, and may be a more viable option than re-constructing the channel. Though the study area is located in a developed area with little open land around it, implementing a detention basin or other structure to which flood waters can be diverted and held for a period of time may be beneficial in reducing the flow through the channel in a major event. This may be an expensive solution, however, due to cost of site acquisition and preparation, and the construction of the basin or structure and the diversion to it.
- Further investigation of material costs may have an impact on the cost estimates present.
- Further investigation should be performed to determine the best explanation for the apparent hydraulic jump at the transition at the downstream end of the study reach.
- Because this study focused on the hydraulic aspect of the modification of San Jose Creek, fish passage was not addressed, though it is a consideration in the broad scope of the actually modification project. To do so, fish spawning habits and other data pertaining to steelhead trout would have to be assessed, as well as hydraulic and hydrologic conditions during pertinent seasons.

- Tidal influence may affect the wetland area where San Jose Creek merges with San Pedro Creek and Atascadero Creek. Further investigation would be necessary to determine the storm recurrence interval that would cause a 100-year flow through San Jose Creek, which could then be used to determine water elevation at that recurrence level and how the tidal backflow would behave.

Appendix A
Profile Plots of Proposed Modification
Plans

Figure 24: Profile Plot for Plan A-1

Figure 25: Profile Plot for Plan A-2

Figure 26: Profile Plot for Plan A-3

Figure 27: Profile Plot for Plan A-4

Figure 28: Profile Plot for Plan B-1

Figure 29: Profile Plot for Plan B-2

Figure 30: Profile Plot for Plan B-3

Figure 31: Profile Plot for Plan B-4

Figure 32: Profile Plot for Plan C-1

Figure 33: Profile Plot for Plan C-2

Figure 34: Profile Plot for Plan C-3

Figure 35: Profile Plot for Plan D-1

Figure 36: Profile Plot for Plan D-2

Appendix B
Cross-Sectional Geometry and Lining
Schemes of Proposed Modification
Plans

Figure 37: Plan A Channel Geometry

Figure 38: Plan B Channel Geometry

Figure 39: Plan C Channel Geometry

Figure 40: Plan D Channel Geometry

Figure 41: Plan A Lining Roughness Configurations

Figure 42: Plan B Lining Roughness Configurations

Figure 43: Plan C Lining Roughness Configurations

Figure 44: Plan D Lining Roughness Configurations

Appendix C
Calculation of Shear Stress on Riprap
Lined Channel

Plan A-3

Slope, $S = 0.00459$

Initial water depth, $y = 9.84$ ft

Bottom roughness, $n_1 = 0.042$

Side roughness, $n_2 = 0.013$

Step 1: Roughness ratios

$$\frac{n_2}{n_1} = \frac{0.013}{0.042} = 0.3095$$

$$\frac{P_L}{P} = \frac{50 \text{ ft}}{69.68 \text{ ft}} = 0.71757$$

Step 2: Effective roughness, n

$$n = n_1 \left[\frac{P_L}{P} + \left(1 - \frac{P_L}{P} \right) \left(\frac{n_2}{n_1} \right)^{3/2} \right]^{2/3}$$

$$n = 0.042 \left[0.71757 + (1 - 0.71757)(0.3095)^{3/2} \right]^{2/3}$$

$$n = 0.035$$

Step 3: Solve Manning's equation for y_n

$$Q = \frac{C_m}{n} A R^{2/3} S^{1/2} = \frac{C_m}{n} \frac{A^{5/3}}{P^{2/3}} S^{1/2}$$

$$5300 \text{ cfs} = \frac{1.486}{0.035} \frac{(50 y_n)^{5/3}}{(50 + 2 y_n)^{2/3}} (0.00459)^{1/2}$$

$$y_n = 9.97 \text{ ft}$$

Step 4: Repeat steps 1-3 until initial and calculated water depth are approximately equal

$$\frac{P_L}{P} = 0.71496$$

$$n = 0.035$$

$$y_n = 9.95 \text{ ft}$$

Step 5: Permissible shear stress

$$R = \frac{A}{P}$$

$$R = \frac{50 \text{ ft} \times y_n}{50 \text{ ft} + 2y_n}$$

$$R = \frac{50 \text{ ft} \times 9.95 \text{ ft}}{50 \text{ ft} + 2(9.95 \text{ ft})} = 7.12 \text{ ft}$$

$$\tau_b = \gamma RS$$

$$\tau_b = (62.4 \text{ pcf})(7.12 \text{ ft})(0.00459)$$

$$\tau_b = 2.04$$

Appendix D

Cost Estimate Sample Calculation

Plan A-3

Upstream channel depth (Station 6100), $d_u = 19.0$ ft

Downstream channel depth (station 2300), $d_d = 10.0$ ft

Channel width, $w = 50.0$ ft

Channel length, $L = 3,700$ ft

Concrete thickness, $t = 6.0$ in

Concrete price = \$1,000/yd³

Shoreblock price = \$10.00/ft²

Wall material: concrete

Bed material: Shoreblock

Surface Area of side wall

$$A_{wall} = (d_d \times L) + \left[\frac{1}{2} (d_u - d_d) \times L \right]$$

$$A_{wall} = (10.0 \text{ ft} \times 3,700 \text{ ft}) + \left[\frac{1}{2} (19.0 \text{ ft} - 10.0 \text{ ft}) \times 3,700 \text{ ft} \right]$$

$$A_{wall} = 37,000 \text{ ft}^2 + 16,650 \text{ ft}^2$$

$$A_{wall} = 53,650 \text{ ft}^2$$

Concrete surface area = $2 \times A_{wall}$

$$A_{conc} = 2 \times 53,650 \text{ ft}^2$$

$$A_{conc} = 107,300 \text{ ft}^2$$

Concrete cost

$$\frac{\$1,000}{yd^3} \times \frac{1 yd^3}{27 ft^3} \times 0.5 ft \cong \frac{\$18.50}{ft^2}$$

$$Cost_{conc} = \$18.50/ft^2 \times 107,300 \text{ ft}^2$$

$$Cost_{conc} = \$1,985,050$$

Surface Area of channel bed

$$A_{bed} = w \times L$$

$$A_{bed} = 50.0 \text{ ft} \times 3,700 \text{ ft}$$

$$A_{bed} = 185,000 \text{ ft}^2$$

Shoreblock surface area = A_{bed}

$$A_{sb} = 185,000 \text{ ft}^2$$

Shoreblock cost

$$Cost_{sb} = \$10.00/ft^2 \times 185,000 \text{ ft}^2$$

$$Cost_{sb} = \$1,850,000$$

Total Cost

$$Cost_{total} = Cost_{conc} + Cost_{sb}$$

$$Cost_{total} = \$1,985,050 + \$1,850,000$$

$$Cost_{total} = \$3,835,050$$

Table 7. Cost Estimates for All Plans

Plan	Area, 1,000 ft ²				Cost (\$M)
	Concrete	Shoreblock	Riprap	Total	
A1	262.70	29.60	-	292.30	5.16
A2	218.30	74.00	-	292.30	4.78
A3	107.30	185.00	-	292.30	3.84
A4	107.30	-	185.00	292.30	3.30
B1	243.23	29.60	-	272.83	4.80
B2	151.70	121.13	-	272.83	4.02
B3	151.70	-	121.13	272.83	3.72
B4	85.10	187.73	-	272.83	3.45
C1	239.08	37.00	-	276.08	4.80
C2	162.80	113.28	-	276.08	4.15
C3	88.80	187.28	-	276.08	3.52
D1	233.62	18.50	-	252.12	4.51
D2	92.50	159.62	-	252.12	3.31

Table 8. Trucking Cost Estimates for Riprap from Bee Rock Quarry

Plan	Area, ft ²	Vol, CY	Distance, mi	\$/CY/mile	Cost (\$M)
A4	185000	3426	25	4.50	0.385
B3	121133	2692	25	4.50	0.303

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