I-5/SR78 Interchange Preliminary Engineering

I-5 Bridge Study at Buena Vista Lagoon

Fluvial Hydraulics and Residence Time Analysis

Final Report

Prepared For:

State of California, Department of Transportation

Prepared By:

Everest International Consultants, Inc.

EVEREST

Under Contract to:

Dokken Engineering

May 2012

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I-5 BRIDGE STUDY AT BUENA VISTA LAGOON

FLUVIAL HYDRAULICS AND RESIDENCE TIME ANALYSIS

FINAL REPORT

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LIST OF ACRONYMS

@	at
Alt	Alternative
BASINS	Better Assessment Science Integrating Point and Nonpoint Sources Model
BVLRP	Buena Vista Lagoon Restoration Project
Caltrans	California Department of Transportation
CDFG	California Department of Fish and Game
cfs	cubic feet per second
COPC	California Ocean Protection Council
e.g.	for example
EFDC	Environmental Fluid Dynamic Code Model
El.	Elevation
EPA	Environmental Protection Agency
Everest	Everest International Consultants, Inc.
Ex.	Existing
FEMA	Federal Emergency Management Agency
ft	feet
H:V	Horizontal to vertical
HEC-RAS	Hydrologic Engineering Centers River Analysis System Software
Hwy	Highway
I-5	Interstate Highway 5
Max	maximum
MHHW	mean higher high water
MLLW	Mean Lower Low Water
n	Manning's Coefficient
N	north
NAVD88	North American Datum of 1988
NCC	North Coast Corridor

NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NTDE	National Tidal Datum Epoch
RAG	Resource Agency Group
RR	Railroad
S	south
SANDAG	San Diego Association of Governments
SCC	California State Coastal Conservancy
SEP	Supplement Engineering Project
typ	typical
USACE	United States Army Corps of Engineers
USFWS	United States Fish and Wildlife Service
yr	year

1 INTRODUCTION

1.1 BACKGROUND

For the past several years, the California Department of Transportation (Caltrans) and the San Diego Association of Governments (SANDAG) have been working on the development and implementation of a large-scale transportation improvement project in northern San Diego County, known as the I-5 North Coast Corridor (NCC) Project. Implementation of the I-5 NCC Project will require work within the major coastal lagoons located throughout Northern San Diego County, including the Buena Vista Lagoon that is located on the Carlsbad and Oceanside border. Implementation of the I-5 NCC Project will include new bridge structures across most of the lagoons, including Buena Vista Lagoon.

Caltrans has been working with several state and federal resource and regulatory agencies to identify and resolve issues associated with implementation of the I-5 NCC Project. This group of agencies, known as the Resource Agency Group (RAG), has expressed concern to Caltrans regarding how the proposed bridge structures would impact tidal circulation, fluvial flows, and fluvial sedimentation. The RAG prepared a white paper that outlined their concerns and provided guidance on the analyses needed to address those concerns. Caltrans is moving forward with these analyses to address the concerns raised by the RAG for the bridges that will cross Batiquitos Lagoon and San Elijo Lagoon. Caltrans did not want to move forward with all the analyses suggested for Buena Vista Lagoon because work is not planned to begin in Buena Vista Lagoon for several years (5-10 years) and because restoration of Buena Vista Lagoon to a salt water regime is highly speculative given that the proposed location of a tidal inlet is controlled by private parties (i.e., private property). A compromise approach was developed to address the concerns raised by the RAG regarding impacts while achieving Caltrans's desire to minimize expenditures for work that will not be needed in the near term. This approach, which is based on the analysis of four previously developed restoration alternatives, is the focus of this study.

1.2 PURPOSE

The purpose of the work is to provide guidance for Caltrans to be used in the design of the three bridge structures (I-5 Bridge, Railroad Bridge, and Coast Highway Bridge) that cross the Buena Vista Lagoon. The guidance focuses on the required channel widths and channel depths under the bridges for use by Caltrans in determining the required bridge lengths and foundation requirements necessary to accommodate the desired channel dimensions for the range of restoration alternatives being considered. The guidance is based on consideration of flood impacts (flood water levels), tidal exchange (tide range), and water quality (residence time).

1.3 OBJECTIVES

The following objectives were established to fulfill the purpose summarized above. The work was based on alternatives developed previously for the Buena Vista Lagoon studies. One set of alternatives (one salt water alternative and one fresh water alternative) was taken from the 2004 Buena Vista Lagoon Feasibility Study (Everest 2004) and the 2008 Buena Vista Lagoon Restoration Project Fluvial Hydraulics, Sediment Transport, and Sedimentation Analysis (Everest 2008); and the other set of alternatives was the fresh water and salt water alternatives developed over the past two years (Everest 2011a).

- Conduct fluvial hydraulics modeling for the two salt water alternatives under existing sea level, focusing on the 100-year return period storm event.
- Conduct fluvial hydraulics modeling for the two salt water alternatives under future sea level, focusing on the 100-year return period storm event.
- Conduct fluvial hydraulics modeling for the two fresh water alternatives under existing sea level, focusing on the 100-year return period storm event.
- Conduct fluvial hydraulics modeling for the two fresh water alternatives under future sea level, focusing on the 100-year return period storm event.
- Conduct residence time analysis for the two salt water alternatives under existing sea level.
- Summarize and compare analysis results to establish desired channel dimensions.
- Develop design guidance for bridge dimensions needed to accommodate desired channel geometry.

2 ALTERNATIVES ANALYZED

2.1 OVERVIEW

The Buena Vista Lagoon is segmented into four basins by four hydraulic connections that include channels under two bridges (Railroad Bridge and Interstate 5), a culvert (under Coast Highway), and a weir (between lagoon and Pacific Ocean). The four basins are named according to the names of the downstream hydraulic connections. The names of the basins are: (i) Weir Basin, (ii) Railroad Basin, (iii) Coast Highway Basin, and (iv) Interstate 5 Basin. The four basins are shown in Figure 2.1.

A number of restoration alternatives were developed over the past few years under the direction of several federal and state agencies including, the California State Coastal Conservancy (SCC), U.S. Fish and Wildlife Service (USFWS), and California Department of Fish and Game (CDFG). Four of these alternatives were selected for the analyses conducted under the present study. These four alternatives were selected because the proposed grading and outlet/inlet configurations represent a reasonable range of potential restoration conditions for Buena Vista Lagoon. These alternatives were analyzed to evaluate the ranges of dimensions for the hydraulic connections in order to provide design guidance for the bridge structures under consideration by Caltrans. These four alternatives are listed below and described in the following sections.

- Saltwater Alternative: Alt 2-1
- Saltwater Alternative: Alt SW2-A
- Freshwater Alternative: Alt 1
- Freshwater Alternative: Alt FW-A

2.2 SALT WATER ALTERNATIVES

2.2.1 Alt 2-1

Alternative 2-1 represents the restoration configuration of a salt water hydrologic regime developed for the restoration project in 2008 (Everest 2008). This alternative achieved the restoration objectives primarily through elimination of the existing exotic vegetation, dredging to remove excess sediment, and establishment of continuous tidal exchange. The existing weir would be replaced with a tidal inlet to provide continuous tidal exchange between the

Lagoon and ocean. The tidal inlet would require stabilization with two jetties that would extend to the Mean Lower Low Water (MLLW) contour. The bottom elevation of the Railroad Basin and Weir Basin would be dredged to between -12 ft and -15 ft, NGVD to provide a sediment trap for sand entering the lagoon from the ocean. Prominent features of this alternative were described in the 2008 Hydraulic Study Report (Everest 2008). A plan view of this alternative is presented in Figure 2.2.

2.2.2 Alt SW2-A

Alternative SW2-A is the latest salt water restoration alternative developed for the Lagoon. In this alternative, a channel would run along the center of the I-5 Basin and Coast Highway Basin at -3.3 ft, NGVD, with the two banks of the channel being graded with a slope not greater than 1:8 (vertical: horizontal). Downstream of the Railroad Bridge, the channel would widen and form a basin with a uniform depth of -3.3 ft NGVD at the Railroad Basin and Weir Basin. The tidal inlet channel would be constructed with an initial bottom elevation of -2.0' NGVD and no jetties would be constructed to stabilize the inlet channel. Prominent features of this alternative were described in the 2011 technical memo (Everest 2011a). A plan view of this alternative is presented in Figure 2.3.

Figure 2.1 Buena Vista Lagoon

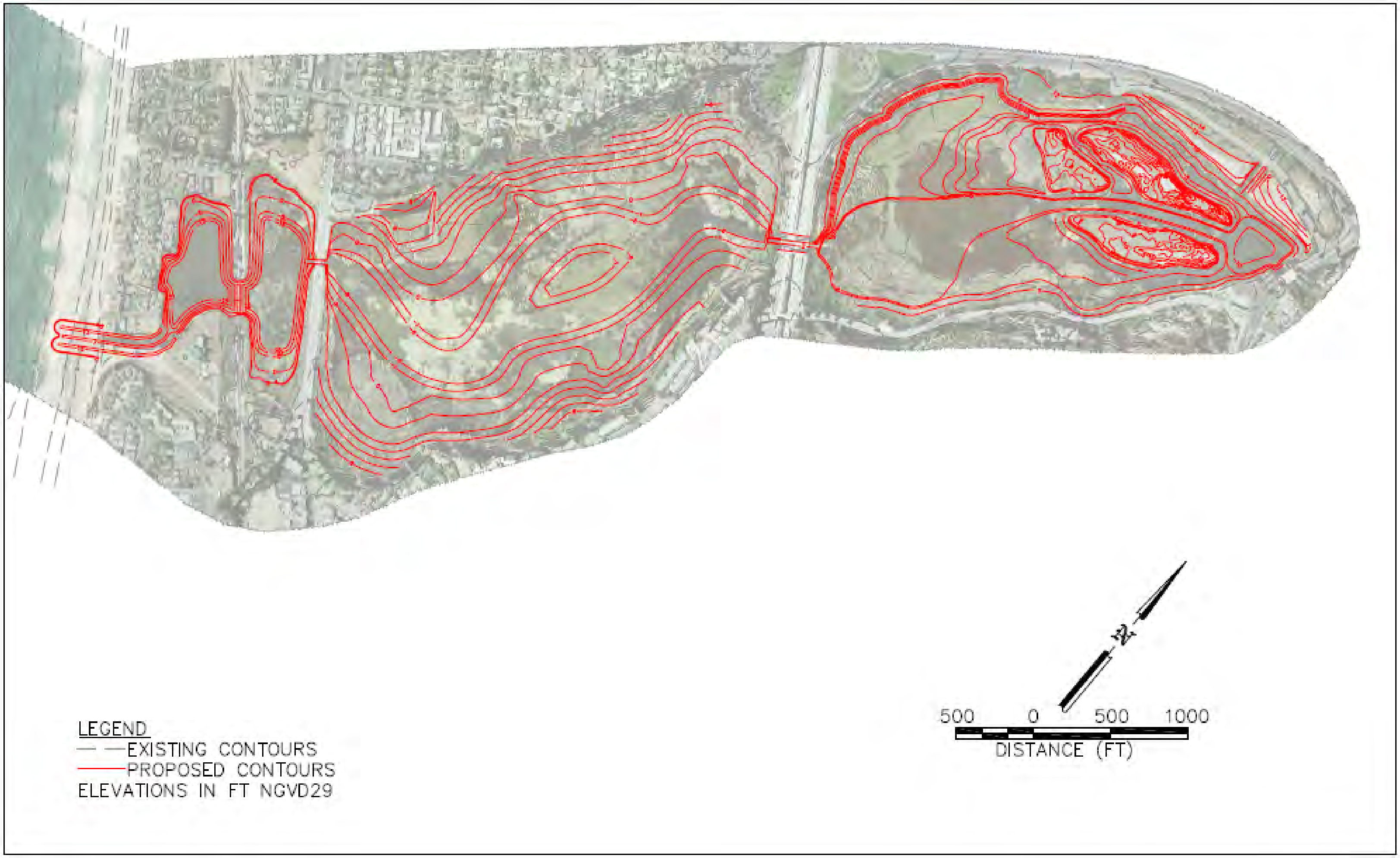


Figure 2.2 Alternative 2-1 Plan View

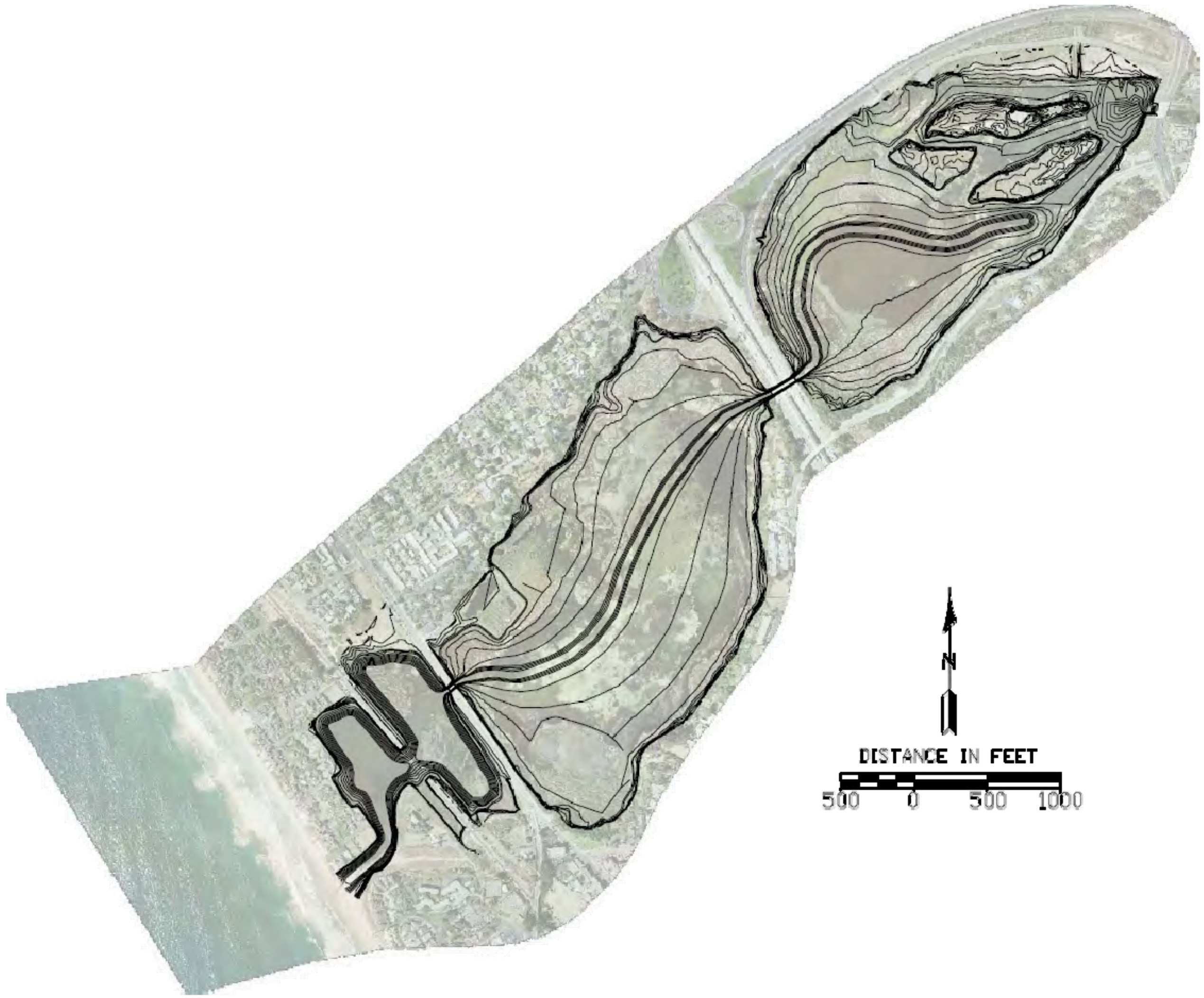


Figure 2.3 Alternative SW2-A Plan View

2.3 FRESH WATER ALTERNATIVES

2.3.1 Alt 1

Alternative 1 represents the restoration configuration that was used to analyze the fresh water hydrologic regime as part of the Buena Vista Lagoon Restoration Project. This alternative would achieve the restoration objectives primarily through elimination of the existing exotic vegetation and dredging to remove excess sediment. It was assumed that the existing ocean outlet weir would be replaced with an 80-foot (ft) wide ocean outlet weir in accordance with the weir widening project that was proposed by the City of Oceanside. The invert elevation of the weir would be kept at the invert elevation of the existing weir, which is 5.6 ft, NGVD. The bottom elevation of the Railroad Basin and Weir Basin would be dredged to between -12 ft and -15 ft, NGVD. Prominent features of this alternative were described in the 2008 fluvial hydraulics report (Everest 2008). A plan view of the alternative is presented in Figure 2.4. It should be noted that for the sea level rise analysis presented in this report, it was assumed that the invert elevation of the weir would be raised by the projected value of sea level rise (55 inches) in order to keep ocean water from entering the Lagoon. This assumption was necessary in order to preserve the fresh water condition of the Lagoon under this freshwater alternative.

2.3.2 Alt FW-A

Alternative FW-A is the latest freshwater alternative developed for the Lagoon. The central portions of each basin would be dredged to maintain a water depth of about six feet (bottom elevation of about 0 ft, NGVD) to minimize the future encroachment of reeds (cattails) throughout the Lagoon. Similar to Alt 1, it was assumed that the existing ocean outlet weir would be replaced with an 80-ft wide ocean outlet weir in accordance with the weir widening project that was proposed by the City of Oceanside. The invert elevation of the weir would be kept at the invert elevation of the existing weir, which is 5.6 ft, NGVD. Prominent features of this alternative were described in the 2011 technical memo (Everest 2011a). A plan view of this alternative is presented in Figure 2.5. It should be noted that for the sea level rise analysis presented in this report, it was assumed that the invert elevation of the weir would be raised by the projected value of sea level rise (55 inches) in order to keep ocean water from entering the Lagoon. This assumption was necessary in order to preserve the fresh water condition of the Lagoon under this freshwater alternative.

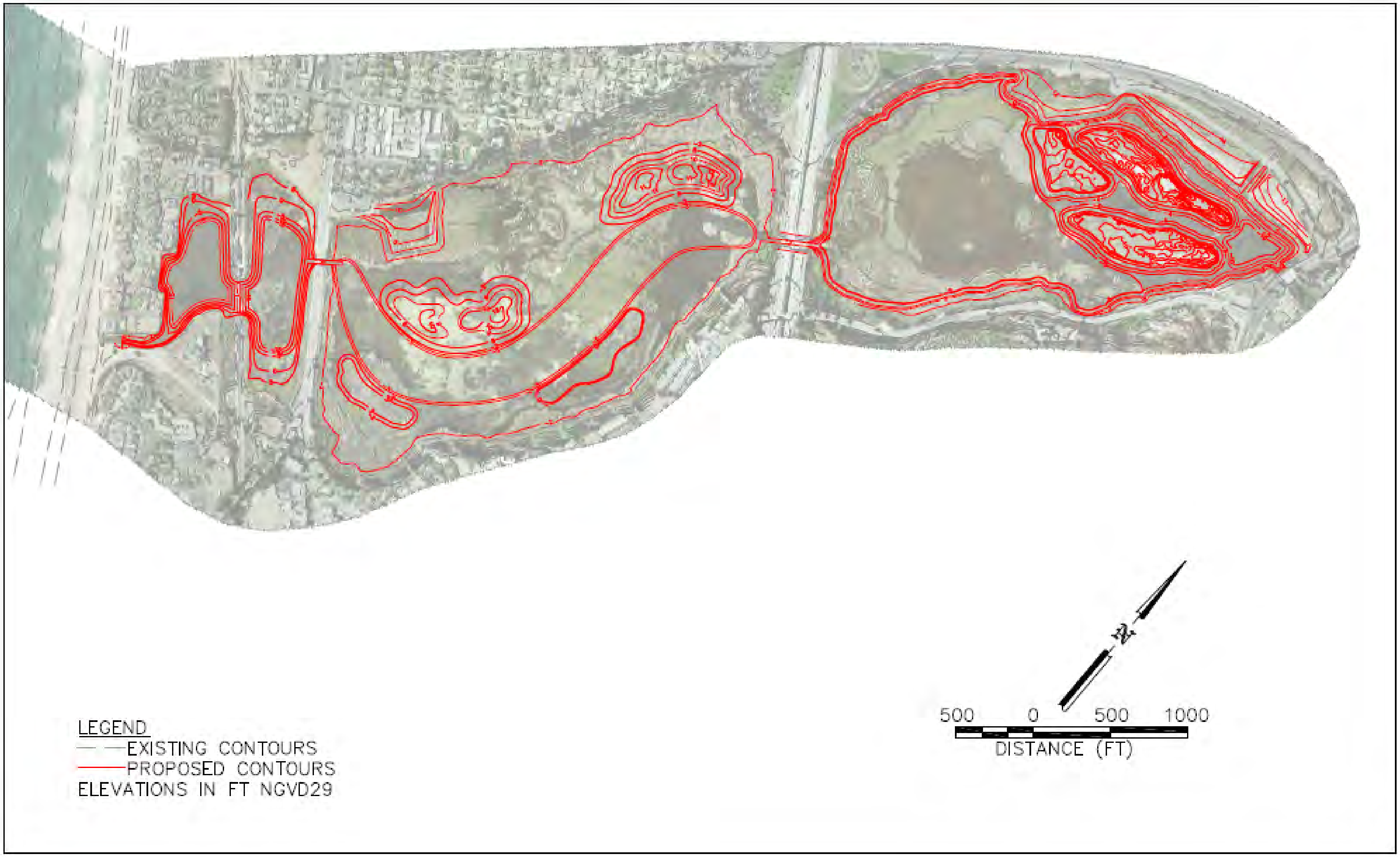


Figure 2.4 Alternative 1 Plan View

Figure 2.5 Alternative FW-A Plan View

3 FLUVIAL HYDRAULICS METHODOLOGIES

3.1 FLUVIAL HYDRAULICS ANALYSIS

The HEC-RAS one-dimensional fluvial hydraulics model developed by the U.S. Army Corps of Engineers (USACE 2006) was used to conduct the fluvial hydraulics analysis in the present study. HEC-RAS is capable of simulating unsteady flow through a network of open channels and can account for hydraulic structures such as bridges, culverts, and weirs. The model is approved by the Federal Emergency Management Agency (FEMA) for flood studies and is commonly used by the USACE and Caltrans for fluvial hydraulics analyses.

In the HEC-RAS model, the Lagoon is represented by cross-sections taken perpendicular to the primary direction of flow from the Buena Vista Creek to the ocean. These cross-sections reflect the area through which water flows. Each alternative was represented by a model domain that consisted of approximately 45 cross-sections. The bridges and weirs were simulated as hydraulic control structures within the model domain.

The fluvial hydraulic analysis focused on studying the impact of a 100-year return period storm from Buena Vista Creek. To evaluate impacts due to storms of lesser magnitudes, five other flood events (2-yr, 5-yr, 10-yr, 25-yr, and 50-yr) were included in the analysis of one of the salt water alternatives. The flood impact of storms coupled with high tides was assessed by using conditions during which the peak of the storm hydrograph was timed to match a tide elevation of mean higher high water (MHHW). In addition to evaluating impacts due to storms under current water levels, the storm impact coupled with high tides during the Year 2100 was conducted with a higher water level to evaluate the impact of anticipated sea level rise.

In the initial model run for each alternative, the hydraulic connections (e.g., bridges) were modeled using as-built dimensions. In subsequent simulations, the dimensions of the hydraulics connections were modified until the simulation results indicated that the storm flow through these hydraulic connections became unimpeded. This process was conducted for fluvial flow coupled with both the current tide level and 2100 tide level with sea level rise.

3.2 FLOOD HYDROGRAPH

As input to the fluvial hydraulic analysis, flood hydrographs were specified as a boundary condition at the upstream end of the Buena Vista Lagoon. The flood hydrographs used in the model were developed for the Buena Vista Lagoon Restoration Feasibility Analysis Project (Everest 2004). Flow conditions in the creek for various magnitudes were generated from watershed modeling using the Better Assessment Science Integrating Point and Nonpoint

Sources (BASINS) modeling system developed by the EPA (2001). The BASINS analysis included considerations for land uses, topography, soil characteristics, precipitation, and evaporation. Figure 3.1 shows the flood hydrographs for the 2-yr, 5-yr, 10-yr, 25-yr, 50-yr and 100-yr return period storms. It can be seen that the maximum flow for the 100-year return period storm is 8,500 cfs. For a 5-yr return period storm, the maximum flow is about 1,000 cfs.

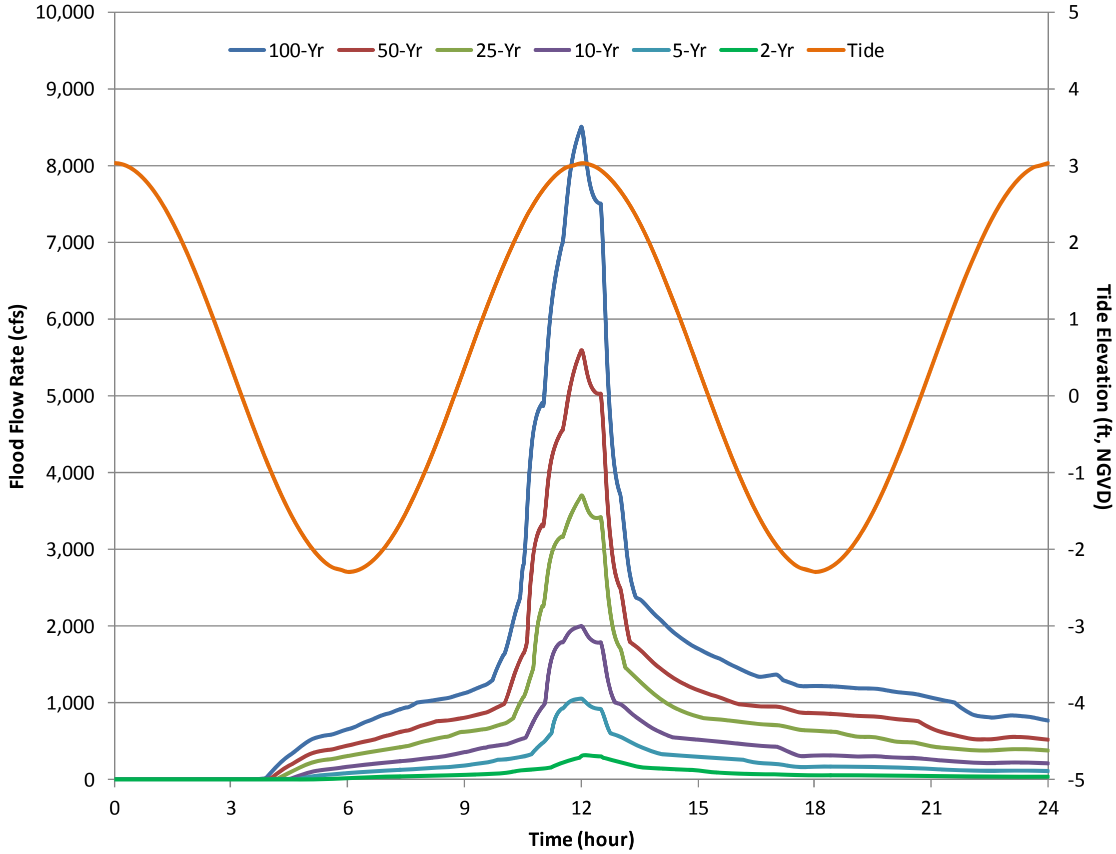


Figure 3.1 Flood and Diurnal Tide Hydrographs

3.3 TIDAL HYDROGRAPH

The tidal influence in the Lagoon was simulated with a mean tide specified as a boundary condition at the downstream end of the HEC-RAS model grid where the Lagoon meets the ocean. The tide data used for modeling was based on historical water level data collected at the NOAA Scripps Pier Station (Station 9410230) in La Jolla. The tidal benchmarks and tidal datum at this station are shown in Table 3.1. These tidal datums are based on the most recent 1983-2001 National Tidal Datum Epoch (NTDE) and are representative of tidal conditions near the Lagoon given the geographic proximity to the Lagoon. Figure 3.1 shows the diurnal tide hydrograph with tide elevations varying between MHHW and MLLW over a 12-hour period. To simulate a reasonably large flood impact, the peak storm flow entering the Lagoon from upstream was timed to enter the Lagoon at the same time that MHHW occurred at the ocean. This is depicted in Figure 3.1. In addition, to determine if there is any effect in flood level due to a time lag between the two peak occurrences, supplemental simulations were conducted for the salt water alternatives using time lags ranging from 15 minutes to 105 minutes.

Table 3.1 Tidal Benchmarks and Tidal Datum

TIDE	ELEVATION (FT, MLLW)	ELEVATION (FT, NGVD)
Highest Observed Water Level (11/13/1997)	7.64	5.35
Mean Higher High Water (MHHW)	5.33	3.04
Mean High Water (MHW)	4.60	2.31
Mean Sea Level (MSL)	2.73	0.44
Mean Low Water (MLW)	0.91	-1.39
North American Vertical Datum-1988 (NAVD 88)	0.19	-2.11
Mean Lower Low Water (MLLW)	0.00	-2.29
Lowest Observed Water Level (12/171933)	-2.87	-5.16

Source: NOAA, 2003

3.4 SEA LEVEL RISE ANALYSIS

One of the objectives of this study was to conduct a sea level impact analysis to provide guidance for the hydraulic connections under the bridges that would be hydraulically adequate to withstand impacts due to long-term sea level rise. This was done in the present study by evaluating the sea level rise impact at the Year 2100. In the sea level rise analysis for the Buena Vista Lagoon SEP study (Everest 2011b), the tide elevations in 2100 were predicted to be 55 inches higher than those of Year 2000. This projection was based on the guidance of the California Ocean Protection Council (COPC, 2011) as well as the value adopted by the California State Coastal Conservancy. The 55-inch increase was added to the tide elevations in Figure 3.1 and the result is shown in Figure 3.2 as Year 2100 Mean Tide. This tide was used as the downstream boundary condition in the HEC-RAS model for the sea level rise analysis. To simulate a reasonably large flood impact, the peak storm flow entering the Lagoon from upstream was timed to enter the Lagoon at the same time that MHHW occurred at the ocean. In addition, to determine if there is any effect in flood level due to a time lag between the two peak occurrences, supplemental simulations were conducted for the salt water alternatives using time lags ranging from 15 minutes to 105 minutes.

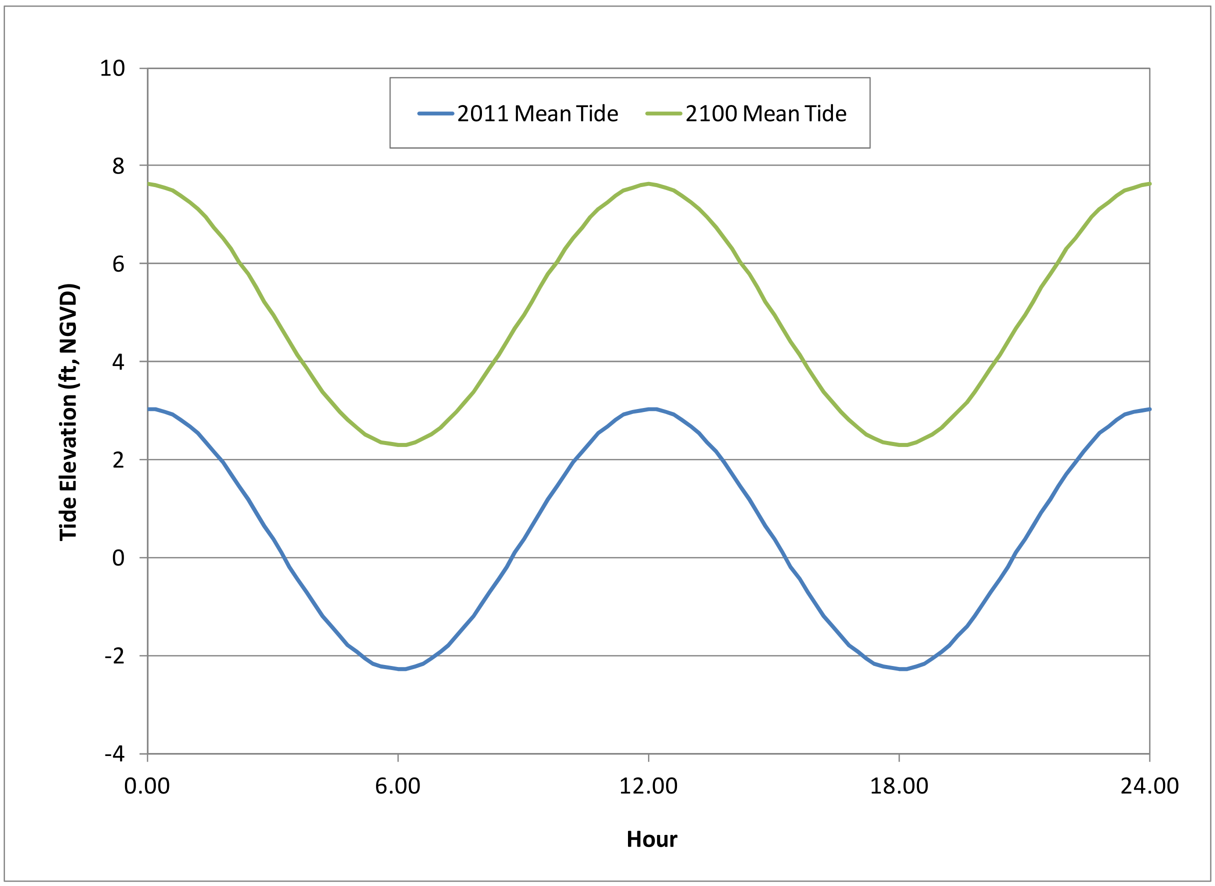


Figure 3.2 Diurnal Tide Hydrographs for Year 2011 and Year 2100

3.5 AS-BUILT HYDRAULIC CONNECTIONS

The existing hydraulic control structures in the Buena Vista Lagoon consist of a weir at the ocean boundary and a culvert under Coast Highway as well as the Railroad Bridge and I-5 Bridge. The existing weir, which was built in 1972, controls the minimum water levels within the Lagoon at 5.6 ft, NGVD. It was assumed that the existing 50-ft weir would be replaced with an 80-ft wide weir in accordance with the weir widening project that was proposed by the City of Oceanside. The NCTD Railroad Bridge spans 280 feet at a deck elevation of 15.2 ft, NGVD and is supported by nine piers. The Coast Highway Bridge (culvert) is the lowest and smallest bridge with a deck elevation of 9.7 ft, NGVD. The I-5 Bridge is supported with two abutments and two piers within the channel. It spans 99 ft across the Lagoon with a deck elevation of 25.0 ft, NGVD. These structure dimensions were based on available information such as as-built drawings, sketches, and field measurements. The structure dimensions used in the HEC-RAS model to simulate as-built conditions are summarized in Table 3.2.

Table 3.2 Approximate Structure Dimensions for As-Built Conditions

STRUCTURE	INVERT WIDTH (FT)	INVERT ELEVATION (FT, NGVD)	CHANNEL SIDE SLOPE (H:V)
Weir	80	5.6	N/A
NCTD Railroad Bridge	17	-2.5	17:1 (S), 12:1 (N)
Coast Highway Bridge	25/29	-6.0/-3.0	vertical
I-5 Bridge	24	-2.0	1.5:1

H:V = horizontal to vertical, S = south side, N = north side

3.6 MANNING'S COEFFICIENT

In the HEC-RAS models, the channel and lagoon bottom materials were assumed to be earth. The Manning's coefficient (n value) used in the models is 0.03. This value was based on Table 3-1 of the HEC-RAS User Manual (USACE 2006) and the recommendation from the USGS website (USGS 2012). While it is understood that channel linings may be used at the bridge connections where the n value may vary from about 0.013 for concrete lining to about 0.033 for riprap, the same manning's coefficient of 0.03 was used for convenience at all bridge connections in the HEC-RAS analyses since any local change in n value is expected to have negligible effect on the flood elevations in the lagoon. A supplemental analysis to assess the effect of a higher Manning's coefficient at the bridge crossings was conducted and the findings are presented in Section 4.4.3.

4 BRIDGE ANALYSIS FOR LAGOON RESTORATION

4.1 OVERVIEW

The HEC-RAS model domain was developed from the grading plans for each of the four alternatives. Initially, the hydraulic connections (e.g., bridges) were modeled as as-built conditions in the model. The HEC-RAS results showed that flood flow was restricted at the bridge connections for the as-built conditions with water backing up in the upstream area. The model domain for each alternative was modified by varying the invert elevation of the channel at each hydraulic connection and then by increasing the span length at the hydraulic connections. The simulation runs with modified bridge parameters tested the sensitivity of the effect of dimension changes on improving the flow restriction through the hydraulic connections. The combined effects of changing both the invert elevations and widths were then analyzed until the HEC-RAS results showed unrestricted flow through all the bridges. For all analyses, the 100-year return period storm flow was used as the boundary condition upstream and the diurnal tide was used as the boundary condition downstream. Scenarios with desirable results were also tested for the sea level rise scenario to determine if further adjustments would be required for future water level conditions. The set of bridge dimensions that yielded unimpeded flow results represent the minimum dimensions needed to accommodate the range of lagoon restoration alternatives considered at this time. It should be noted that the invert (bottom) elevations of the proposed bridge cross sections are assumed to be finished ground. If the channel is designed to be lined with constructed materials such as concrete or riprap, the proposed invert elevation should be the top of the lining material.

4.2 SALT WATER ALTERNATIVE ANALYSIS

HEC-RAS model simulations were conducted for the two salt water alternatives: Alt 2-1 and Alt SW2-A. The results of the analysis are discussed below.

4.2.1 Alt 2-1 Analysis

Alt 2-1 Fluvial Hydraulic Analysis for Bridge Channel Elevation Sensitivity Evaluation

Simulations for Alt 2-1 with as-built bridge connections were conducted initially and the results were examined to identify areas with flow restrictions in the model. These areas were mostly immediately upstream of the bridges. The model was then modified by deepening the invert elevations at the hydraulic connections by a few feet at a time, and the responses of the water elevations to these changes were evaluated. The five cases summarized in

Table 4.1 were analyzed. The maximum water elevation results for the five cases are presented in Figure 4.1. The structure dimension changes were made first to the I-5 Bridge, as in Cases 1 to 3. In these cases, the adjusted dimensions helped to improve the flow through the I-5 Bridge, but the flow at the Coast Highway Bridge downstream of the I-5 Bridge was still restricted. In Cases 4 and 5, in addition to modifying the I-5 Bridge, the Coast Highway Bridge channel was deepened. It was found that deepening the channels at the hydraulic connections improved the flow somewhat, but appeared not effective in eliminating the flow restriction completely. This is evident in Figure 4.1 for Cases 4 and 5, in which the flows are still restricted although the channel elevations were lowered by 6 to 10 feet from the as-built levels.

Table 4.1 Parameters used in Alt 2-1 Bridge Channel Elevation Sensitivity Analysis

Bridge	Parameters	As-Built	Case 1	Case 2	Case 3	Case 4	Case 5
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6/-3	-6/-3	-6/-3	-10	-12
	Width (ft)	25/29	25/29	25/29	25/29	29	29
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7	9.7	9.7
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2	8.2	8.2
	North side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-6	-10	-12	-12	-12
	Bottom Width @ Invert (ft)	24	24	24	24	24	24
	Width @ Ex. Soffit (ft)	99	99	99	99	99	99
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.3:1	1.1:1	1.1:1	1.1:1	1.1:1
	South side slope (H:V)	1.5:1	1.3:1	1.1:1	1.1:1	1.1:1	1.1:1

Red = different from as-built

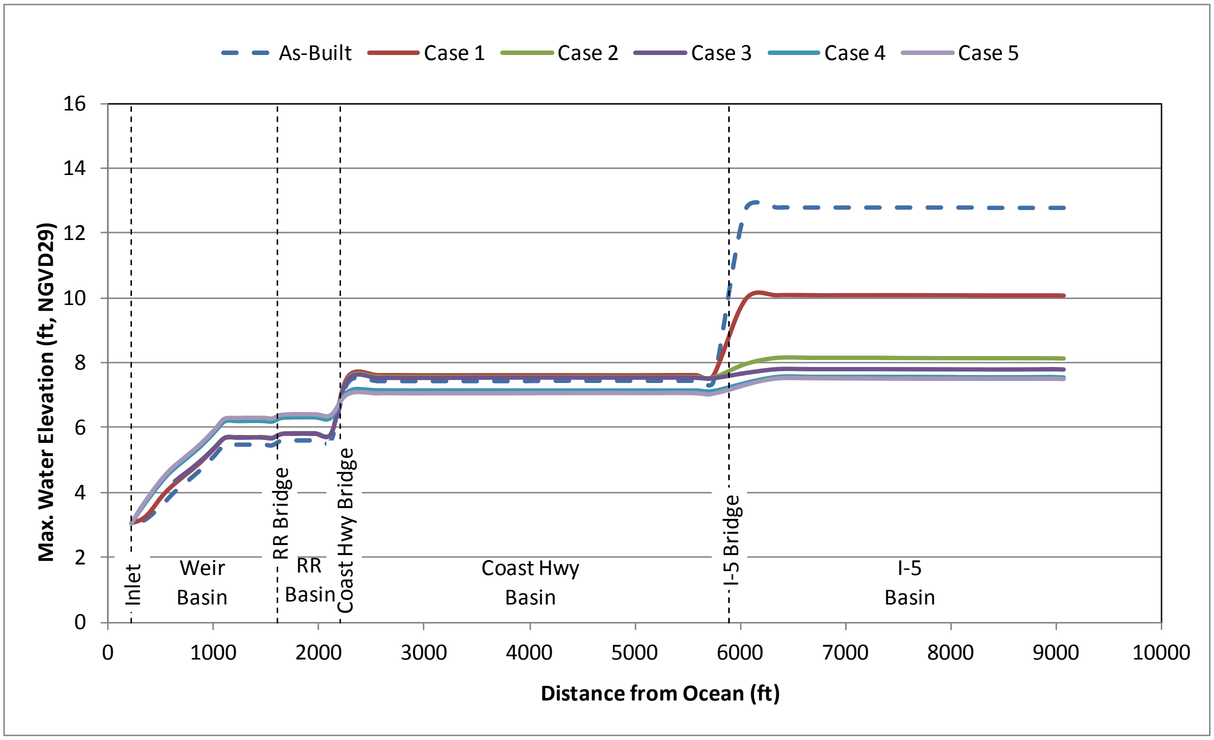


Figure 4.1 Alt 2-1 HEC-RAS Results for Cases with Various Channel Elevations

Alt 2-1 Fluvial Hydraulic Analysis for Bridge Span Length Sensitivity Evaluation

To test the effectiveness of increasing span length at improving water conveyance, the model was modified by increasing the span lengths at the hydraulic connections, while the channel elevations were kept at as-built conditions. A total of three cases were analyzed and the changes in the bridge dimensions are summarized in Table 4.2. The structure dimension changes were made first to the I-5 Bridge, as in Cases 1 to 2. When the adjusted dimensions produced results of unimpeded flow through the I-5 Bridge, the Coast Highway Bridge downstream of the I-5 Bridge was modified. Case 3 represented a scenario in which the span lengths of both the I-5 Bridge and Coast Highway Bridge were modified.

The maximum water elevation results for the five cases are presented in Figure 4.2. It can be seen that Case 3 would almost eliminate the flow restriction. For this case, the span length of the I-5 Bridge was increased by 80 feet and that of the Coast Highway Bridge was increased by 60 feet.

Table 4.2 Parameters used in Alt 2-1 Bridge Span Length Sensitivity Analysis

BRIDGE	PARAMETERS	AS-BUILT	CASE 1	CASE 2	CASE 3
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6/-3	-6/-3	-6
	Width (ft)	25/29	25/29	25/29	89
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2
	North side slope (H:V)	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-2	-2	-2
	Bottom Width @ Invert (ft)	24	104	84	104
	Width @ Ex. Soffit (ft)	99	179	159	179
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.5:1	1.5:1	1.5:1
	South side slope (H:V)	1.5:1	1.5:1	1.5:1	1.5:1

Red = different from as-built

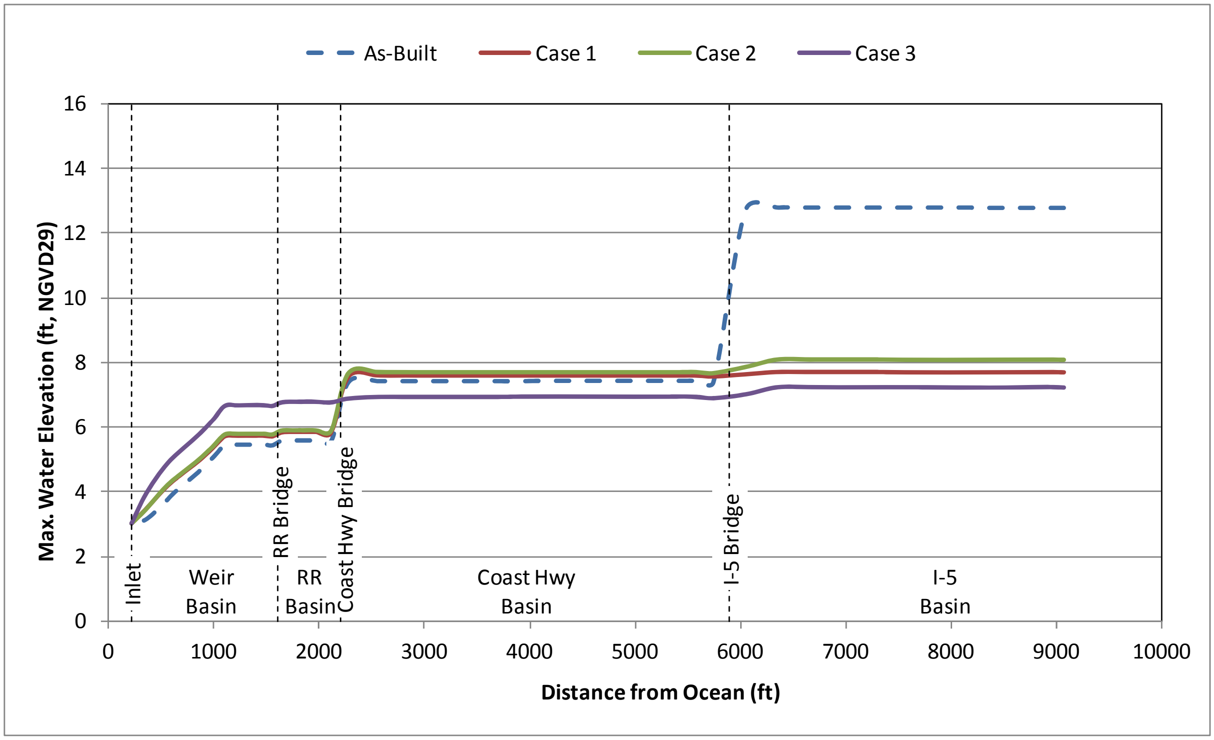


Figure 4.2 Alt 2-1 HEC-RAS Results for Cases with Various Span Lengths

Alt 2-1 Fluvial Hydraulic Analysis for Bridge Structure Analysis

It is expected that a modification of both the channel invert and span length would be more effective in improving the storm flow capacities of the hydraulic connections. Based on the outcome of the sensitivity analysis, the model was modified by both deepening the channels and increasing the span lengths at the hydraulic connections. The four cases summarized in Table 4.3 were analyzed. The structure dimensions change were made first to the I-5 Bridge, as in Case 1. When the adjusted dimensions produced results of unimpeded flow through the I-5 Bridge, the Coast Highway Bridge downstream of the I-5 Bridge was modified. Cases 2 to 4 represent the scenarios in which changes were made to both the I-5 Bridge and Coast Highway Bridge. The maximum water elevation results for the four cases are presented in Figure 4.3. While Cases 2 and 3 achieved large reductions in flow impedance, the results for Case 4 indicated unimpeded flow through the hydraulic connections. Therefore, Case 4 was selected as the best case for Alternative 2-1. It should be noted that the maximum water elevation at the Coast Highway Bridge is about 7 feet NGVD, which is only about a foot below the existing soffit. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris. The effect of the additional fill associated with raising the road by increasing the height and width of the embankment was included in one model simulation to verify the sensitivity of increasing the embankment height on the results. The results indicated that increasing the height would have little impact on the results.

Table 4.3 Parameters used in Alt 2-1 Bridge Analysis

BRIDGE	PARAMETERS	AS-BUILT	CASE 1	CASE 2	CASE 3	CASE 4 – BEST
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6/-3	-6	-6	-6
	Width (ft)	25/29	25/29	69	69	80
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7	
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2	*
	North side slope (H:V)	vertical	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-6	-6	-6	-6
	Bottom Width @ Invert (ft)	24	64	64	84	85
	Width @ Ex. Soffit (ft)	99	139	139	159	160
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.3:1	1.3:1	1.3:1	1.3:1
	South side slope (H:V)	1.5:1	1.3:1	1.3:1	1.3:1	1.3:1

Red = different from as-built
* Proposed soffit elevation should be max water elevation + value (such as freeboard) based on design criteria.

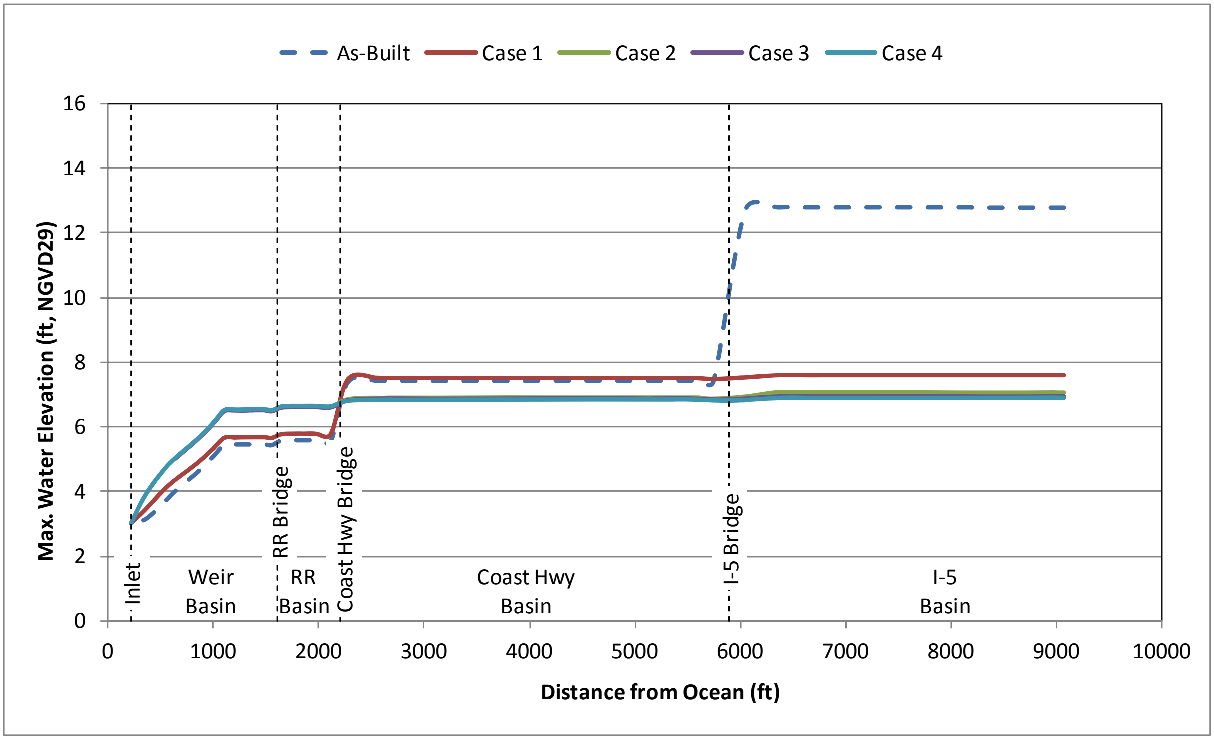


Figure 4.3 Alt 2-1 HEC-RAS Results for Analysis Cases without Sea Level Rise

Sea Level Rise Analysis for Alt 2-1

To determine the bridge dimensions which would be adequate for Alt 2-1 for the 2100 water level conditions, a fluvial hydraulic analysis was conducted with projected sea level rise in Year 2100. Table 4.4 shows the bridge parameters used in the HEC-RAS analysis. The model with the best hydraulic connections derived for 2011 water level conditions was used in Case 1. The maximum water elevation results are presented in Figure 4.4. It should be noted that the maximum water elevation at the Coast Highway Bridge is about 10 feet NGVD, which is above the existing soffit. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris. A higher soffit (e.g., 10 ft NGVD) for the Coast Highway Bridge was tested in Case 2. The water elevation plot shown in Figure 4.4 is the same as Case 1, indicating that the water elevation would not change with the higher soffit elevation.

Table 4.4 Parameters used in Alt 2-1 Sea Level Rise Analysis

BRIDGE	PARAMETERS	AS-BUILT	CASE 1	CASE 2 - BEST
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	
	Soffit Elevation (ft, NGVD)	11.1	11.1	*
	North side slope (H:V)	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6	-6
	Width (ft)	25/29	80	80
	Deck Elevation (ft, NGVD)	9.7	9.7	
	Soffit Elevation (ft, NGVD)	8.2	8.2	*
	North side slope (H:V)	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-6	-6
	Bottom Width @ Invert (ft)	24	85	85
	Width @ Ex. Soffit (ft)	99	160	160
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.3:1	1.3:1
	South side slope (H:V)	1.5:1	1.3:1	1.3:1

Red = different from as-built

* Proposed soffit elevation should be max water elevation + value (such as freeboard) based on design criteria.

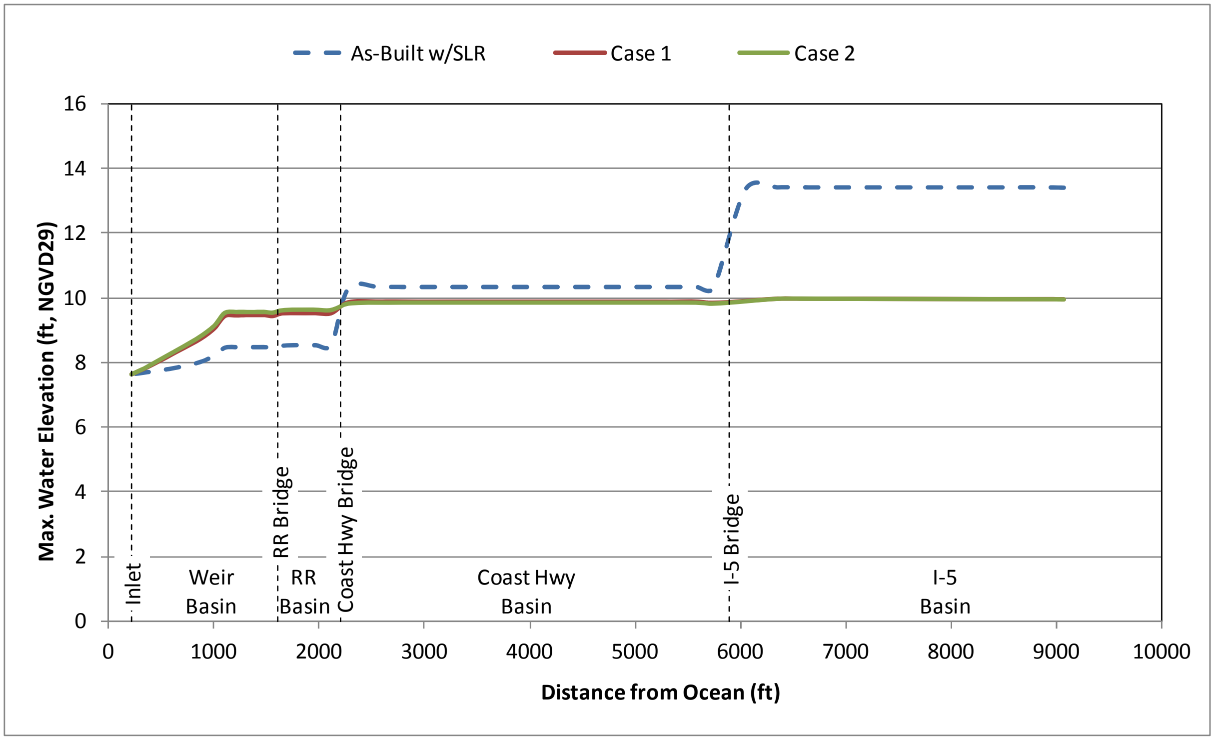


Figure 4.4 Alt 2-1 HEC-RAS Results for Sea Level Rise Analysis

4.2.2 Alt SW2-A Analysis

Alt SW2-A Fluvial Hydraulic Analysis for Bridge Channel Elevation Sensitivity Evaluation

Simulations for Alt SW2-A with as-built bridge connections were conducted initially and the results were examined to identify areas with flow restrictions in the model. Similar to Alt 2-1, the flow restrictions were mostly at locations immediately upstream of the bridges. The bridge dimensions of the four cases being analyzed are summarized in Table 4.5. The structure dimension changes were made first to the I-5 Bridge, as in Cases 1 and 2. In these two cases, the adjusted dimensions helped to improve flow through the I-5 Bridge, but the flow at the Coast Highway Bridge still backed up. Therefore, in addition to modifying the I-5 Bridge, the Coast Highway Bridge channel was deepened in the next two cases. The maximum water elevation results for the four cases are presented in Figure 4.5. Similar to Alt 2-1, deepening channels at the hydraulic connections improved the flow somewhat, but was not effective in eliminating the flow restriction completely. This is evident in Figure 4.5 for Cases 3 and 4, in which the channel elevation at the I-5 Bridge was lowered by 8 feet from the as-built level, and the Coast Highway Bridge was lowered by 2 to 4 feet. One reason that deepening channels at the hydraulic connections is not effective in eliminating flow restriction is that the proposed bottom elevation of the channel in the Lagoon for Alt SW2-A is -3.3' NGVD, therefore lowering the invert at the bridges much deeper than -3.3' NGVD without deepening the channel would not cause great improvement.

Table 4.5 Parameters used in Alt SW2-A Bridge Invert Elevation Sensitivity Analysis

Bridge	Parameters	As-Built	Case 1	Case 2	Case 3	Case 4
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6/-3	-6/-3	-10	-8
	Width (ft)	25/29	25/29	25/29	29	29
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7	9.7
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2	8.2
	North side slope (H:V)	vertical	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-4	-10	-10	-10
	Bottom Width @ Invert (ft)	24	24	24	24	24
	Width @ Ex. Soffit (ft)	99	99	99	99	99
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.4:1	1.1:1	1.1:1	1.1:1
	South side slope (H:V)	1.5:1	1.4:1	1.1:1	1.1:1	1.1:1

Red = different from as-built

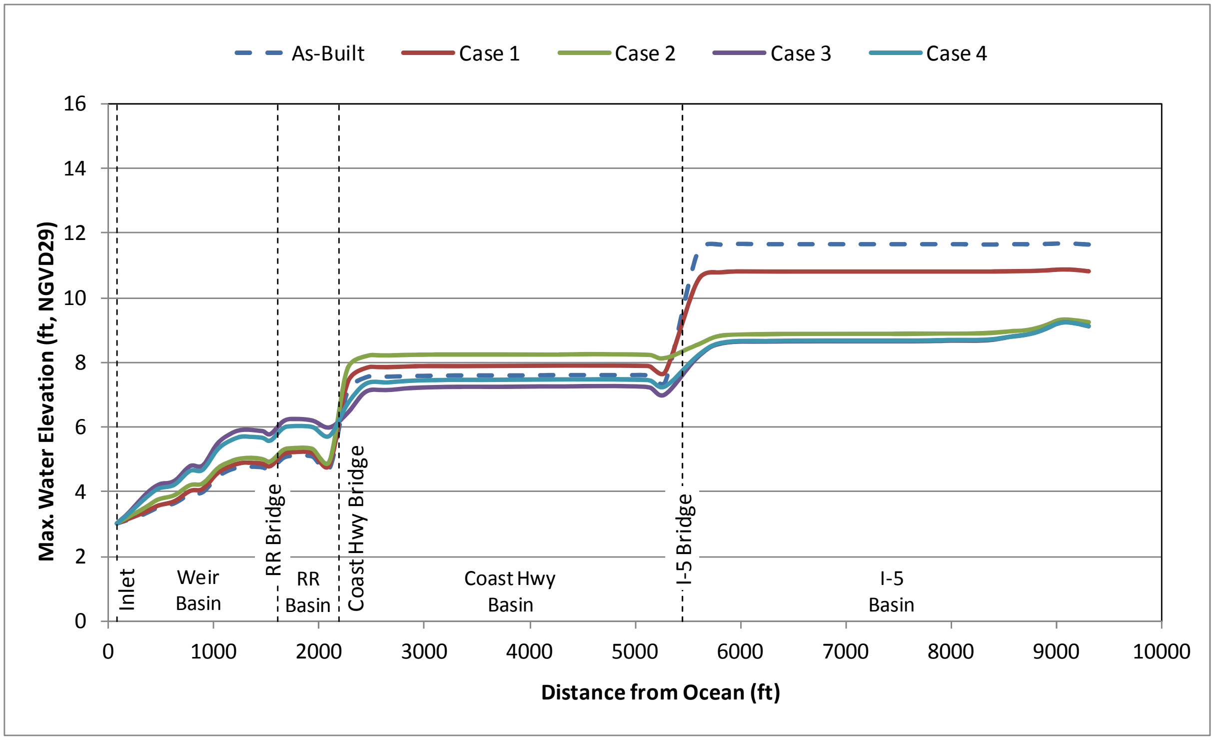


Figure 4.5 Alt SW2-A HEC-RAS Results for Cases with Various Bridge Channel Elevations

Alt SW2-A Fluvial Hydraulic Analysis for Bridge Span Length Sensitivity Evaluation

For the span length evaluation, the model was modified by increasing the span lengths at the hydraulic connections, while the channel elevations at the hydraulic connections were kept at as-built elevations. A total of six cases were analyzed and the bridge dimensions are summarized in Table 4.6. The span length change was made first to the I-5 Bridge, as in Cases 1 to 4. When the adjusted dimensions resulted in improved flow through the I-5 Bridge, the span of the Coast Highway Bridge was also lengthened. Cases 5 and 6 are two scenarios in which the span lengths of both the I-5 Bridge and Coast Highway Bridge were increased. The maximum water elevation results for the six cases are presented in Figure 4.6. The last two cases show substantial improvements in flow through the bridges. The spans of the I-5 Bridge and Coast Highway Bridge were extended by 80 feet in Case 6.

Table 4.6 Parameters used in Alt SW2-A Bridge Span Length Sensitivity Analysis

BRIDGE	PARAMETERS	AS-BUILT	CASE 1	CASE 2	CASE 3	CASE 4	CASE 5	CASE 6
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	12:1	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6/-3	-6/-3	-6/-3	-6/-3	-6	-6
	Width (ft)	25/29	25/29	25/29	25/29	25/29	89	109
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7	9.7	9.7	9.7
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2	8.2	8.2	8.2
	North side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-2	-2	-2	-2	-2	-2
	Bottom Width @ Invert (ft)	24	44	64	104	124	104	104
	Width @ Ex. Soffit (ft)	99	119	139	179	199	179	179
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1
	South side slope (H:V)	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1

Red = different from as-built

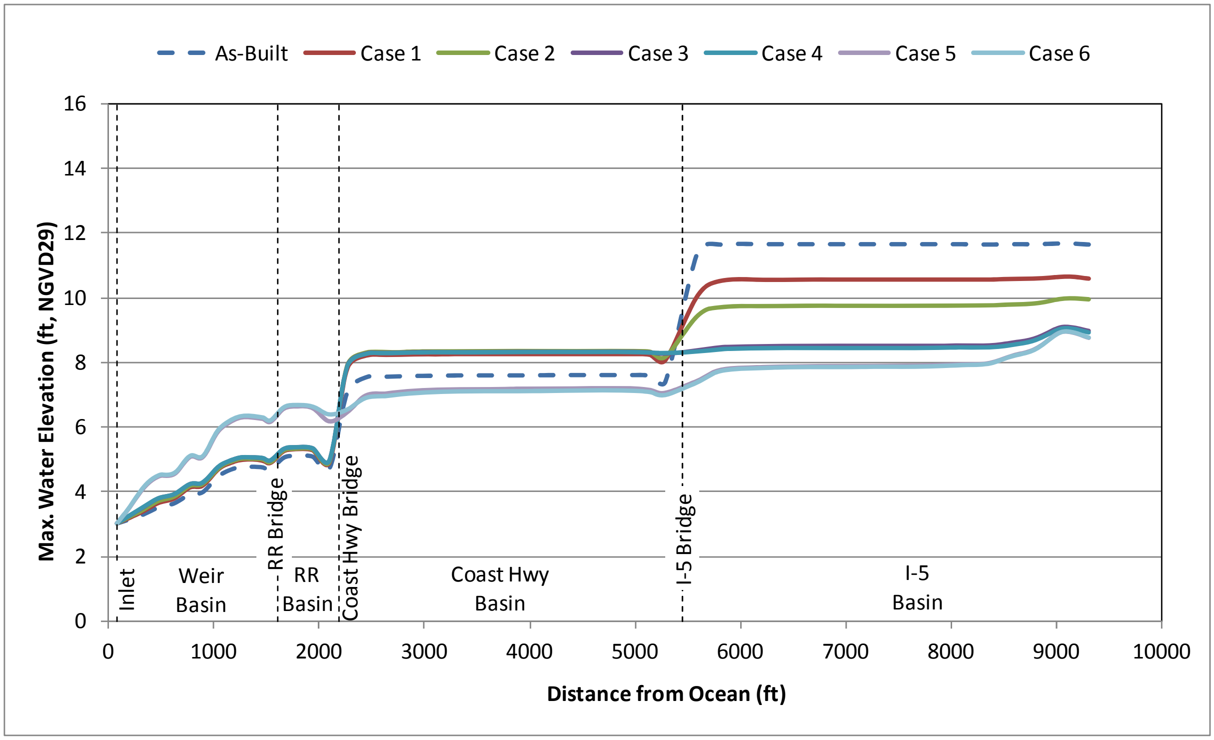


Figure 4.6 Alt SW2-A HEC-RAS Results for Cases with Various Span Lengths

Alt SW2-A Fluvial Hydraulic Analysis for Bridge Structure Analysis

The Alt SW2-A model was modified by both deepening the channels and increasing the span lengths at the hydraulic connections. The three cases summarized in Table 4.7 were analyzed. The structure dimension changes were made to both the I-5 Bridge and Coast Highway Bridge based on the outcome of previous simulations. The maximum water elevation results for the five cases are presented in Figure 4.7. Case 2 involves deepening the channel of the Railroad Bridge from -2' to -4' NGVD, which is about the depth of the proposed channel (-3.3' NGVD) in the basins on either side of the Railroad Bridge. While the three cases yielded very similar results, Case 2 was selected as the desirable scenario because the water elevations in this case were more uniform. It should be noted that the maximum water elevation at the Coast Highway Bridge is about 7 feet NGVD, which is only about a foot below the existing soffit. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.7 Parameters used in Alt SW2-A Bridge Analysis

Bridge	Parameters	As-Built	Case 1	Case 2 - Best	Case 3
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-4	-4
	Bottom Width @ Invert (ft)	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	11:1	11:1
	South side slope (H:V)	17:1	17:1	15:1	15:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-8	-6	-6
	Width (ft)	25/29	109	109	89
	Deck Elevation (ft, NGVD)	9.7	9.7		9.7
	Soffit Elevation (ft, NGVD)	8.2	8.2	*	8.2
	North side slope (H:V)	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-4	-4	-4
	Bottom Width @ Invert (ft)	24	104	104	104
	Width @ Ex. Soffit (ft)	99	179	180	180
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.4:1	1.4:1	1.4:1
	South side slope (H:V)	1.5:1	1.4:1	1.4:1	1.4:1

Red = different from as-built

* Proposed soffit elevation should be max water elevation + value (such as freeboard) based on design criteria.

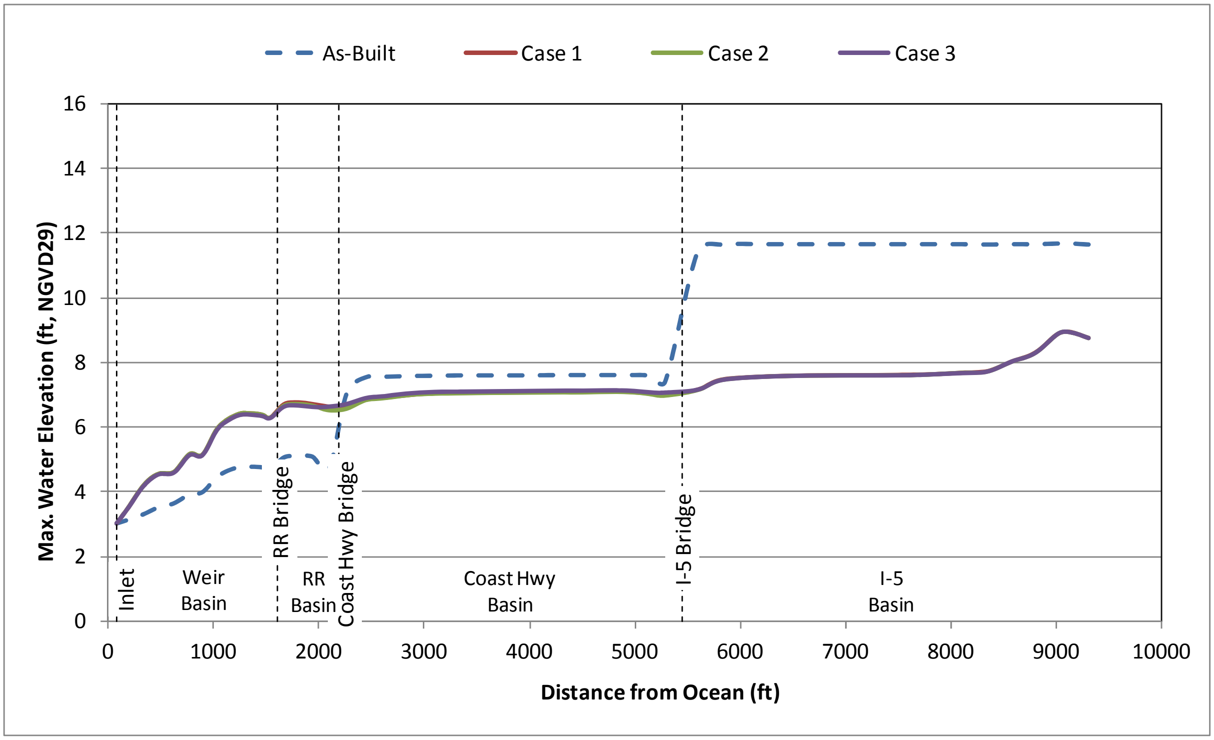


Figure 4.7 Alt SW2-A HEC-RAS Results for Analysis Cases without Sea Level Rise

Sea Level Rise Analysis for Alt SW2-A

A fluvial hydraulic analysis for Alt SW2-A with the projected sea level rise in 2100 was conducted to determine the bridge dimensions adequate for this condition. Table 4.8 shows the bridge parameters used in the HEC-RAS analysis. The model with the best hydraulic connections derived for the 2011 water level condition was used in Case 1. The maximum water elevation results are presented in Figure 4.8. It can be seen that the water flow in Case 1 is not impeded at the hydraulic connections, therefore the bridge parameters in Case 1 are adequate for the sea level rise scenario. It should be noted that the maximum water elevation of about 9.5 feet NGVD at the Coast Highway Bridge is above the soffit elevation. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.8 Parameters used in Alt SW2-A Sea Level Rise Analysis

Bridge	Parameters	As-Built	Case 1 - Best
Railroad	Invert Elevation (ft, NGVD)	-2.5	-4
	Bottom Width @ Invert (ft)	17	17
	Width @ Ex. Soffit (ft)	280	280
	Deck Elevation (ft, NGVD)	15.2	
	Soffit Elevation (ft, NGVD)	11.1	*
	North side slope (H:V)	12:1	11:1
	South side slope (H:V)	17:1	15:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6
	Width (ft)	25/29	109
	Deck Elevation (ft, NGVD)	9.7	
	Soffit Elevation (ft, NGVD)	8.2	*
	North side slope (H:V)	vertical	vertical
	South side slope (H:V)	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-4
	Bottom Width @ Invert (ft)	24	104
	Width @ Ex. Soffit (ft)	99	180
	Deck Elevation (ft, NGVD)	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1
	North side slope (H:V)	1.5:1	1.4:1
	South side slope (H:V)	1.5:1	1.4:1

Red = different from as-built

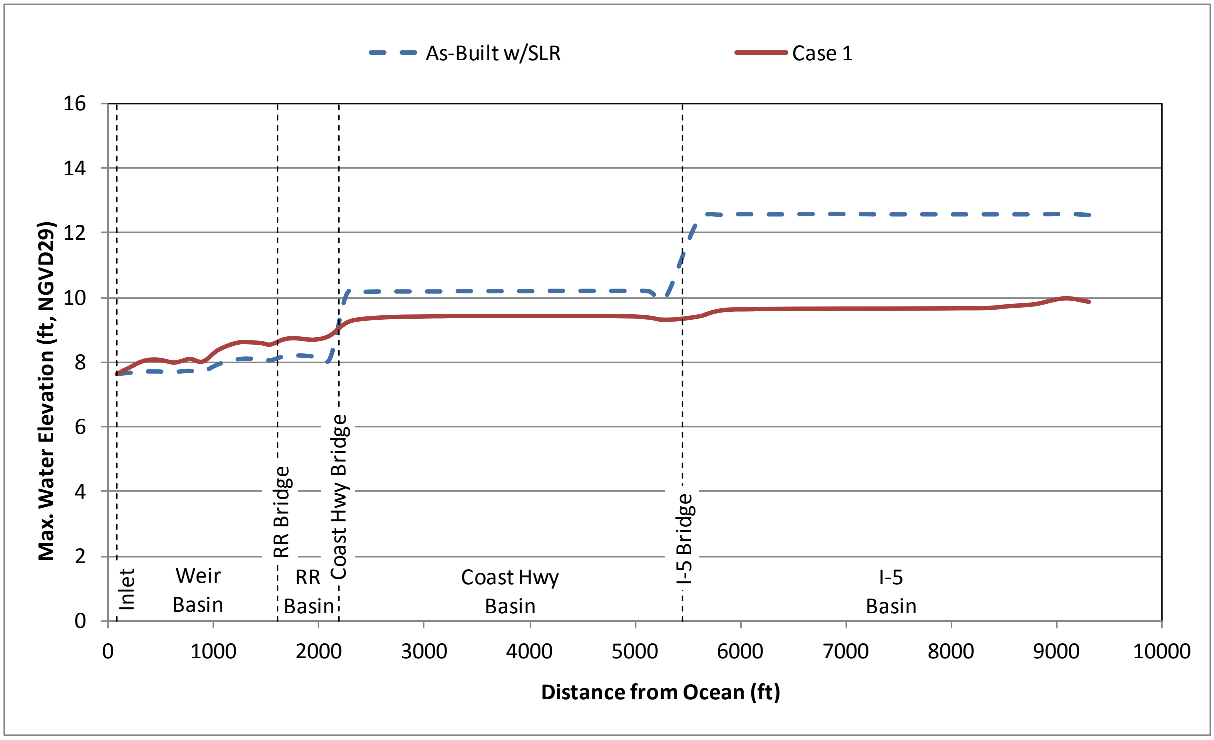


Figure 4.8 Alt SW2-A HEC-RAS Results for Sea Level Rise Analysis

4.3 FRESH WATER ALTERNATIVE ANALYSIS

HEC-RAS model simulations were conducted for the two fresh water alternatives: Alt 1 and FW-A. Alt 1 was evaluated using the same set of bridge parameters used for analyzing Alt 2-1, and Alt FW-A was evaluated with the parameters used in analyzing Alt SW2-A. The results of the analysis are presented below.

4.3.1 Alt 1 Optimization

Alt 1 Fluvial Hydraulic Analysis for Bridge Channel Elevation Sensitivity Evaluation

In order to test the effect of channel depth on flow performance for Alt 1, five cases were analyzed. The bridge dimensions for these cases are summarized in Table 4.9. The structure dimension changes were made first to the I-5 Bridge, such as in Cases 1 to 3. Cases 4 and 5 were the two scenarios in which both the I-5 Bridge and Coast Highway Bridge were modified. The maximum water elevation results for the five cases are presented in Figure 4.9. The maximum water elevations for Cases 2 to 5 are quite uniform throughout the Lagoon, indicating that modifying the channel invert for Alt 1 could effectively improve the flow capacity. It should be noted that the maximum water elevation at about 11 feet NGVD (see Figure 4.9) is above the existing deck elevation of the Coast Highway Bridge and the existing soffit of the Railroad Bridge. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.9 Parameters used in Alt 1 Bridge Channel Elevation Sensitivity Analysis

BRIDGE	PARAMETERS	AS-BUILT	CASE 1	CASE 2	CASE 3	CASE 4	CASE 5
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6/-3	-6/-3	-6/-3	-10	-12
	Width (ft)	25/29	25/29	25/29	25/29	29	29
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7	9.7	9.7
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2	8.2	8.2
	North side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-6	-10	-12	-12	-12
	Bottom Width @ Invert (ft)	24	24	24	24	24	24
	Width @ Ex. Soffit (ft)	99	99	99	99	99	99
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.3:1	1.1:1	1.1:1	1.1:1	1.1:1
	South side slope (H:V)	1.5:1	1.3:1	1.1:1	1.1:1	1.1:1	1.1:1

Red = different from as-built

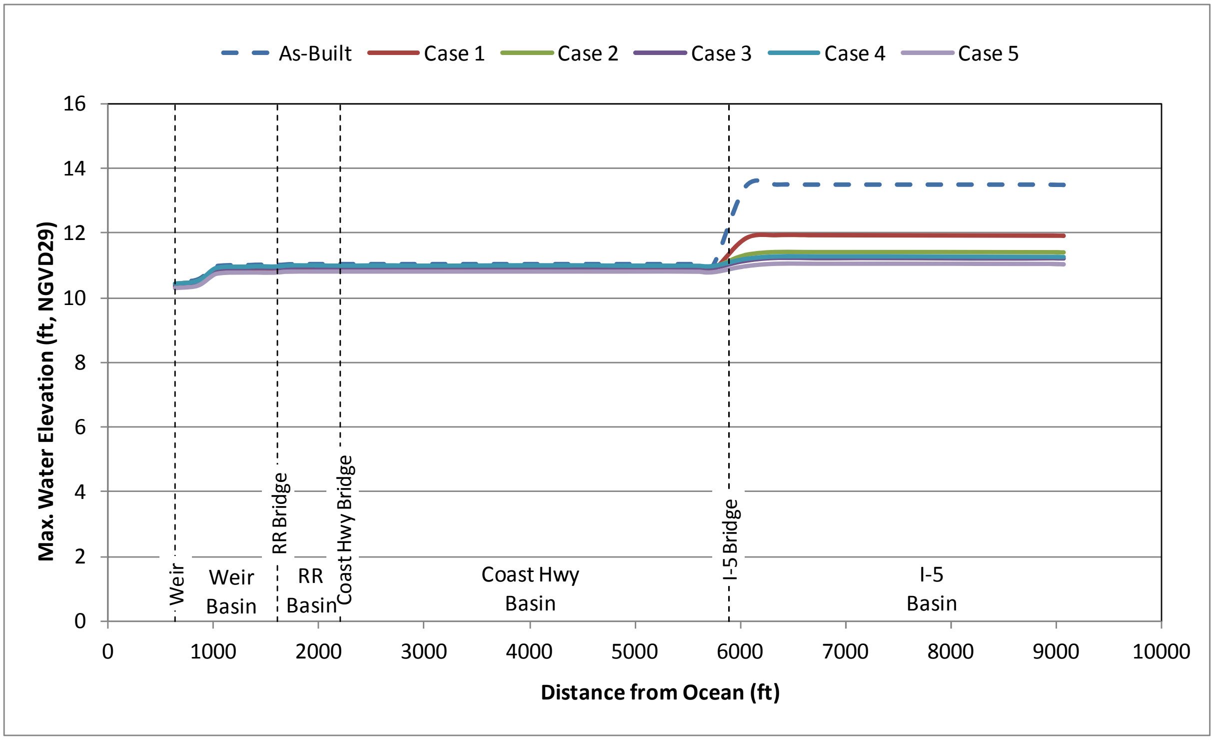


Figure 4.9 Alt 1 HEC-RAS Results for Cases with Various Bridge Channel Elevations

Alt 1 Fluvial Hydraulic Analysis for Bridge Span Length Sensitivity Evaluation

The bridge parameters of the three cases for evaluating span length sensitivity are summarized in Table 4.10. The maximum water elevation results for the three cases are presented in Figure 4.10. It can be seen that the maximum water elevations for the three cases are all uniform throughout the Lagoon. It should be noted that the maximum water elevations at about 11 feet NGVD is above the existing deck elevation of the Coast Highway Bridge and the existing soffit elevation of the Railroad Bridge. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.10 Parameters used in Alt 1 Bridge Span Length Sensitivity Analysis

BRIDGE	PARAMETERS	AS-BUILT	CASE 1	CASE 2	CASE 3
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6/-3	-6/-3	-6
	Width (ft)	25/29	25/29	25/29	89
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2
	North side slope (H:V)	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-2	-2	-2
	Bottom Width @ Invert (ft)	24	104	84	104
	Width @ Ex. Soffit (ft)	99	179	159	179
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.5:1	1.5:1	1.5:1
	South side slope (H:V)	1.5:1	1.5:1	1.5:1	1.5:1

Red = different from as-built

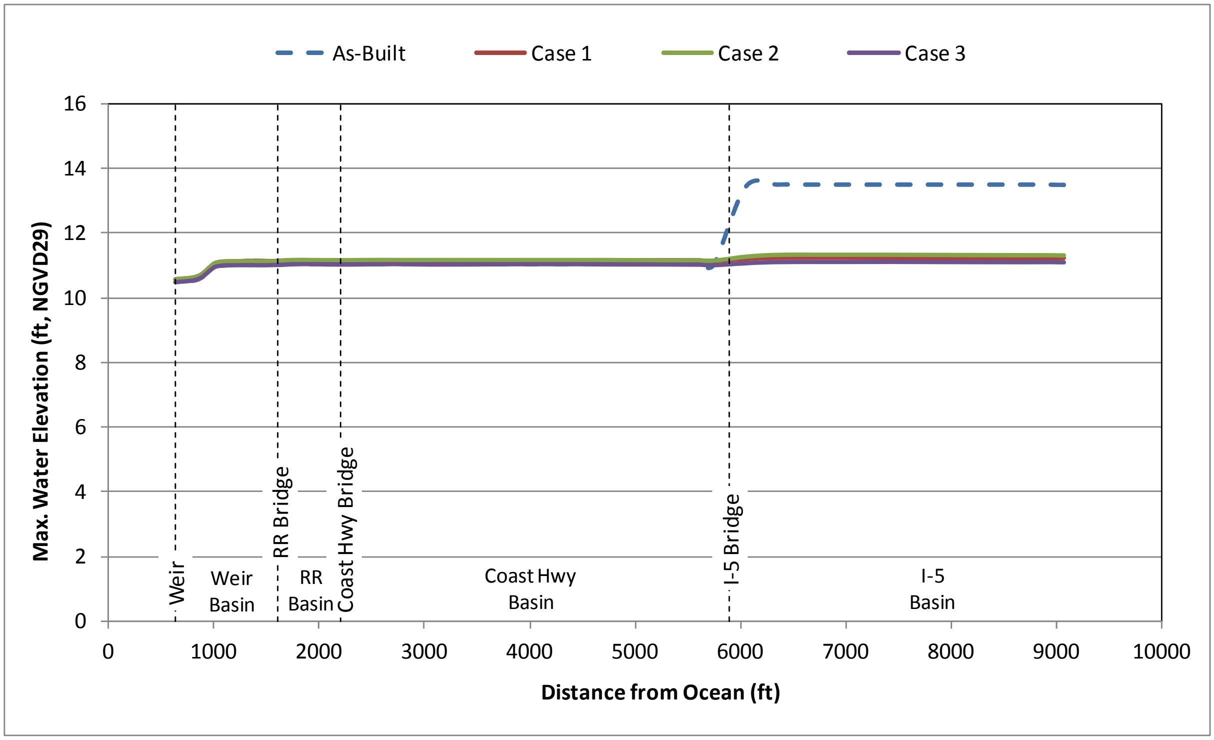


Figure 4.10 Alt 1 HEC-RAS Results for Cases with Various Span Lengths

Alt 1 Fluvial Hydraulic Analysis for Bridge Structure Analysis

The parameters of the four cases evaluated for the bridge analysis are summarized in Table 4.11. The maximum water elevation results are presented in Figure 4.11. It can be seen that the maximum water elevations for all four cases are uniform throughout the Lagoon. The best case for the salt water Alternative 2-1, which is Case 4, would be adequate for this fresh water alternative. It should be noted that the maximum water elevations at about 10.5 feet NGVD is above the deck elevation of the Coast Highway Bridge and near the soffit elevation of the Railroad Bridge. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.11 Parameters used in Alt 1 Bridge Analysis

Bridge	Parameters	As-Built	Case 1	Case 2	Case 3	Case 4	Best
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2	15.2	
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1	11.1	*
	North side slope (H:V)	12:1	12:1	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6	-6	-6	-6	-6
	Width (ft)	25/29	69	69	69	80	80
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7	9.7	
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2	8.2	*
	North side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-6	-6	-6	-6	-6
	Bottom Width @ Invert (ft)	24	64	64	84	85	85
	Width @ Ex. Soffit (ft)	99	139	139	159	160	160
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.3:1	1.3:1	1.3:1	1.3:1	1.3:1
	South side slope (H:V)	1.5:1	1.3:1	1.3:1	1.3:1	1.3:1	1.3:1

Red = different from as-built

* Proposed soffit elevation should be max water elevation + value (such as freeboard) based on design criteria.

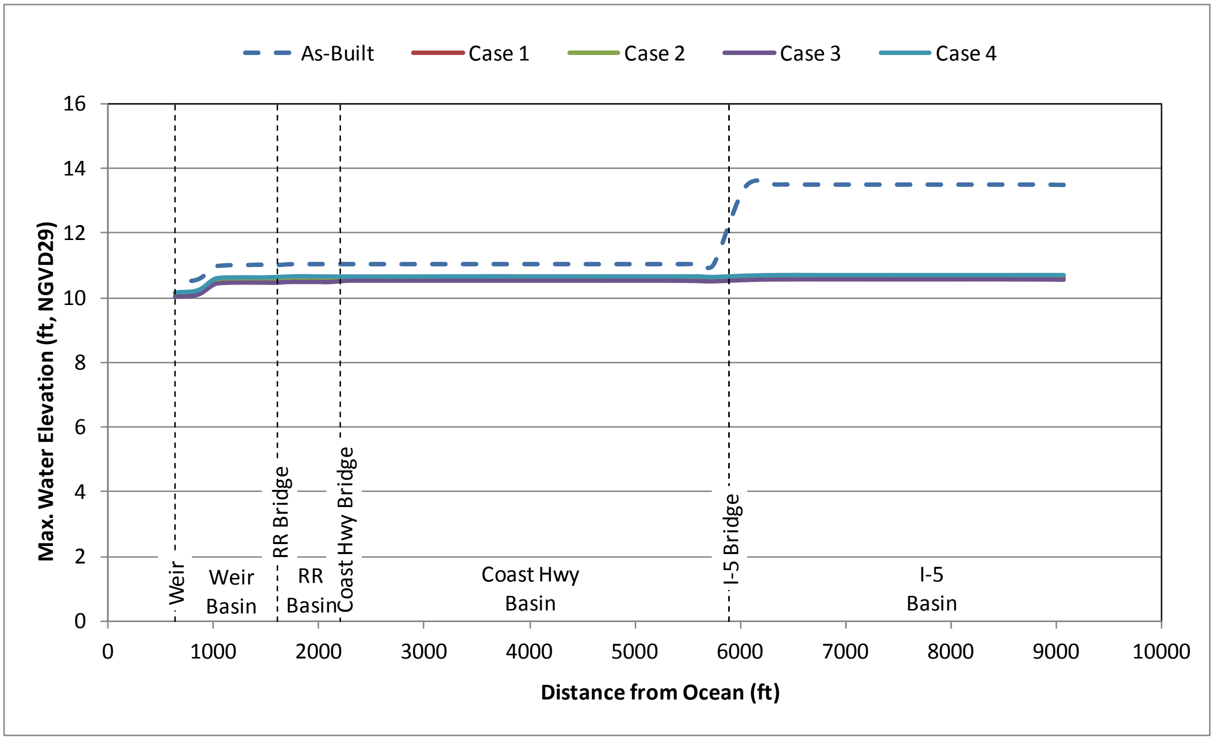


Figure 4.11 Alt 1 HEC-RAS Results for Analysis Cases without Sea Level Rise

Sea Level Rise Analysis for Alt 1

A fluvial hydraulic analysis for Alt 1 with projected sea level rise in Year 2100 was conducted to determine the bridge dimensions adequate for this condition. For the sea level rise scenario for Alt 1, the weir near the ocean inlet/outlet was assumed to be raised adequately to maintain a fresh water regime in the Lagoon. Table 4.12 shows the bridge parameters used for this analysis. The maximum water elevation results are presented in Figure 4.12. It should be noted that the maximum water elevations at about 12.5 feet NGVD is above the deck elevation of the Coast Highway Bridge and the soffit elevation of the Railroad Bridge. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.12 Parameters used in Alt 1 Sea Level Rise Analysis

BRIDGE	PARAMETERS	AS-BUILT	CASE 1 BEST
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17
	Width @ Ex. Soffit (ft)	280	280
	Deck Elevation (ft, NGVD)	15.2	
	Soffit Elevation (ft, NGVD)	11.1	*
	North side slope (H:V)	12:1	12:1
	South side slope (H:V)	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6
	Width (ft)	25/29	80
	Deck Elevation (ft, NGVD)	9.7	
	Soffit Elevation (ft, NGVD)	8.2	*
	North side slope (H:V)	vertical	vertical
	South side slope (H:V)	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-6
	Bottom Width @ Invert (ft)	24	85
	Width @ Ex. Soffit (ft)	99	160
	Deck Elevation (ft, NGVD)	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1
	North side slope (H:V)	1.5:1	1.3:1
	South side slope (H:V)	1.5:1	1.3:1

Red = different from as-built
* Proposed soffit elevation should be max water elevation + value (such as freeboard) based on design criteria.

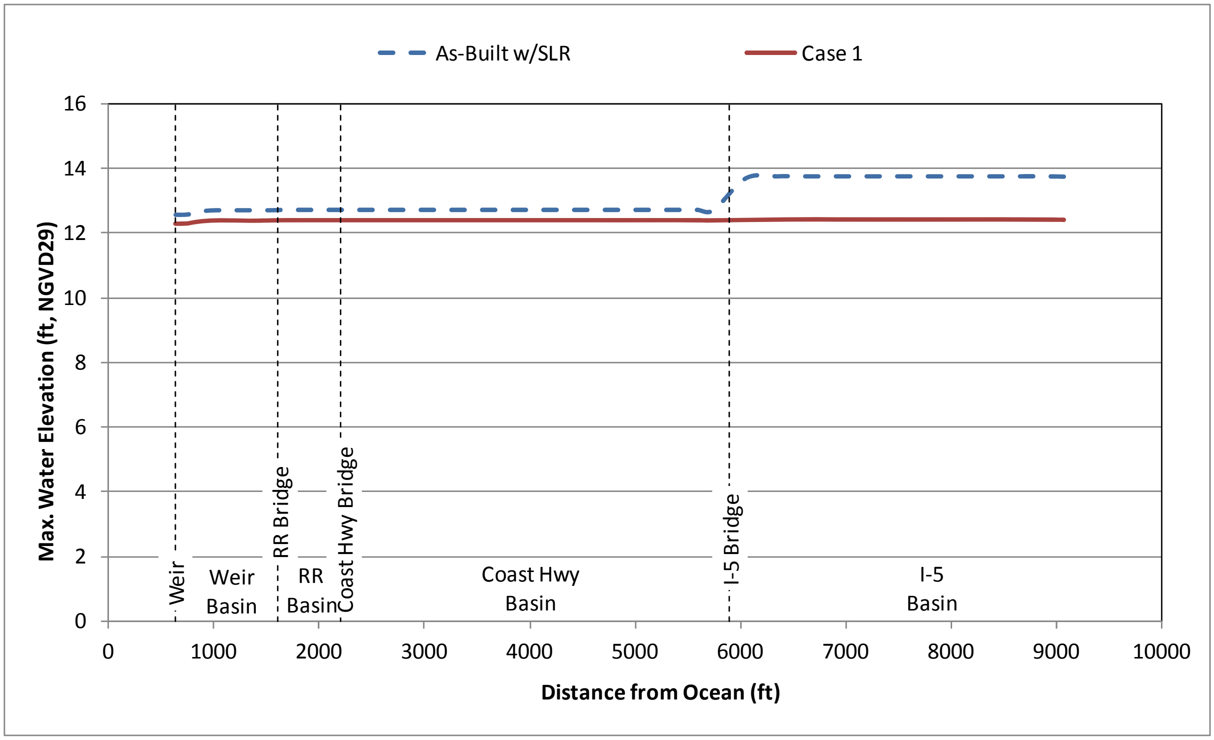


Figure 4.12 Alt 1 HEC-RAS Results for Sea Level Rise Analysis

4.3.2 Alt FW-A Analysis

Alt FW-A Fluvial Hydraulic Analysis for Bridge Channel Elevation Sensitivity Evaluation

In addition to the case with as-built hydraulic connections, four other cases were analyzed for Alt FW-A to evaluate the effect of channel depth variations. The bridge dimensions for these cases are summarized in Table 4.13. The maximum water elevation results for the five cases are presented in Figure 4.13. The maximum water elevations for Cases 2 to 4 are quite uniform throughout the Lagoon. It should be noted that the maximum water elevation at about 13 feet NGVD (see Figure 4.13) is above the existing deck elevation of the Coast Highway Bridge and the existing soffit of the Railroad Bridge. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.13 Parameters used in Alt FW-A Bridge Channel Elevation Sensitivity Analysis

BRIDGE	PARAMETERS	AS-BUILT	CASE 1	CASE 2	CASE 3	CASE 4
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6/-3	-6/-3	-10	-8
	Width (ft)	25/29	25/29	25/29	29	29
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7	9.7
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2	8.2
	North side slope (H:V)	vertical	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-4	-10	-10	-10
	Bottom Width @ Invert (ft)	24	24	24	24	24
	Width @ Ex. Soffit (ft)	99	99	99	99	99
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.4:1	1.1:1	1.1:1	1.1:1
	South side slope (H:V)	1.5:1	1.4:1	1.1:1	1.1:1	1.1:1

Red = different from as-built

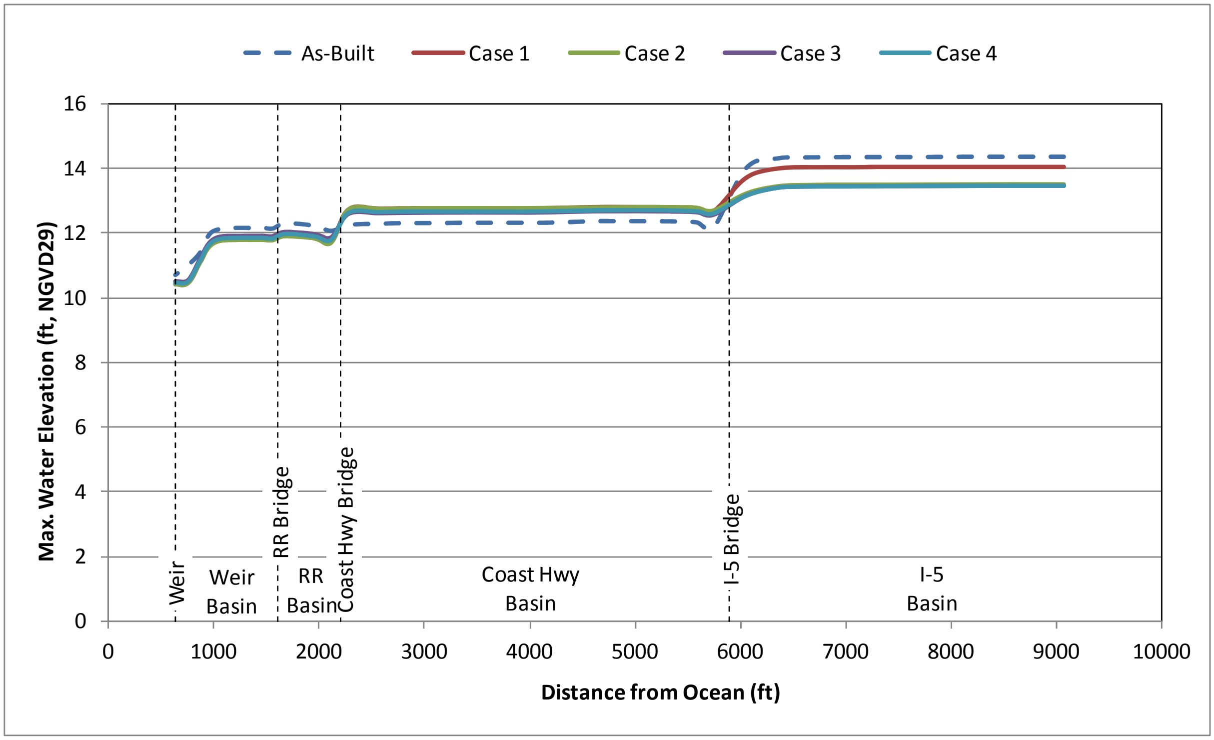


Figure 4.13 Alt FW-A HEC-RAS Results for Cases with Various Channel Elevations

Alt FW-A Fluvial Hydraulic Analysis for Bridge Span Length Sensitivity Evaluation

The bridge parameters of the six cases analyzed for evaluating span length sensitivity are summarized in Table 4.14. The maximum water elevation results are presented in Figure 4.14. It can be seen in Case 2 that increasing the span lengths of I-5 Bridge by about 40 feet would almost eliminate the flow restriction at the I-5 Bridge. Further span increases at the I-5 Bridge as in Cases 3 and 4 would not yield substantial improvement. Cases 5 and 6 helped to improve the flow both at the I-5 Bridge and Coast Highway Bridge, as the maximum water elevations become uniform when the span length of the Coast Highway Bridge was increased by 60 feet. It should be noted that the maximum water elevations at about 12.5 feet NGVD for Cases 5 and 6 are above the existing deck elevation of the Coast Highway Bridge and the existing soffit elevation of the Railroad Bridge. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.14 Parameters used in Alt FW-A Bridge Span Length Sensitivity Analysis

Bridge	Parameters	As-Built	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5
	Bottom Width @ Invert (ft)	17	17	17	17	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280	280	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	15.2	15.2	15.2	15.2	15.2
	Soffit Elevation (ft, NGVD)	11.1	11.1	11.1	11.1	11.1	11.1	11.1
	North side slope (H:V)	12:1	12:1	12:1	12:1	12:1	12:1	12:1
	South side slope (H:V)	17:1	17:1	17:1	17:1	17:1	17:1	17:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6/-3	-6/-3	-6/-3	-6/-3	-6	-6
	Width (ft)	25/29	25/29	25/29	25/29	25/29	89	109
	Deck Elevation (ft, NGVD)	9.7	9.7	9.7	9.7	9.7	9.7	9.7
	Soffit Elevation (ft, NGVD)	8.2	8.2	8.2	8.2	8.2	8.2	8.2
	North side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical	vertical	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-2	-2	-2	-2	-2	-2
	Bottom Width @ Invert (ft)	24	44	64	104	124	104	104
	Width @ Ex. Soffit (ft)	99	119	139	179	199	179	179
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0	25.0	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1
	South side slope (H:V)	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1	1.5:1

Red = different from as-built

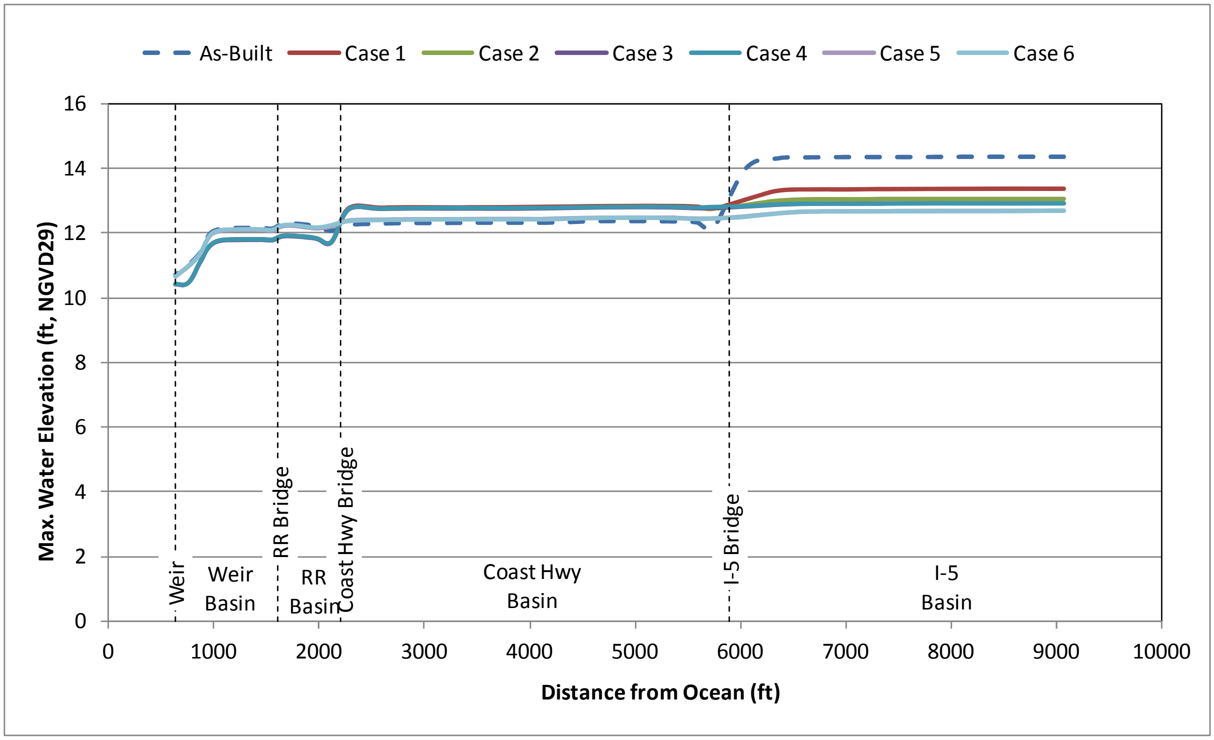


Figure 4.14 Alt FW-A HEC-RAS Results for Cases with Various Span Lengths

Alt FW-A Fluvial Hydraulic Analysis for Bridge Structure Analysis

The parameters of the two cases used in evaluating the bridge structures are summarized in Table 4.15. The maximum water elevation results are presented in Figure 4.15. It can be seen that the maximum water elevations for both cases are uniform throughout the Lagoon. The best case for the salt water alternative Alt 1, which is Case 2, would be adequate for this fresh water alternative. It should be noted that the maximum water elevations at about 12.5 feet NGVD is above the deck elevation of the Coast Highway Bridge and near the soffit elevation of the Railroad Bridge. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.15 Parameters used in Alt FW-A Bridge Analysis

Bridge	Parameters	As-Built	Case 1	Case 2 - Best
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-4
	Bottom Width @ Invert (ft)	17	17	17
	Width @ Ex. Soffit (ft)	280	280	280
	Deck Elevation (ft, NGVD)	15.2	15.2	
	Soffit Elevation (ft, NGVD)	11.1	11.1	*
	North side slope (H:V)	12:1	12:1	11:1
	South side slope (H:V)	17:1	17:1	15:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-8	-6
	Width (ft)	25/29	109	109
	Deck Elevation (ft, NGVD)	9.7	9.7	
	Soffit Elevation (ft, NGVD)	8.2	8.2	*
	North side slope (H:V)	vertical	vertical	vertical
	South side slope (H:V)	vertical	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-4	-4
	Bottom Width @ Invert (ft)	24	104	104
	Width @ Ex. Soffit (ft)	99	179	180
	Deck Elevation (ft, NGVD)	25.0	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1
	North side slope (H:V)	1.5:1	1.4:1	1.4:1
	South side slope (H:V)	1.5:1	1.4:1	1.4:1

Red = different from as-built

* Proposed soffit elevation should be max water elevation + value (such as freeboard) based on design criteria.

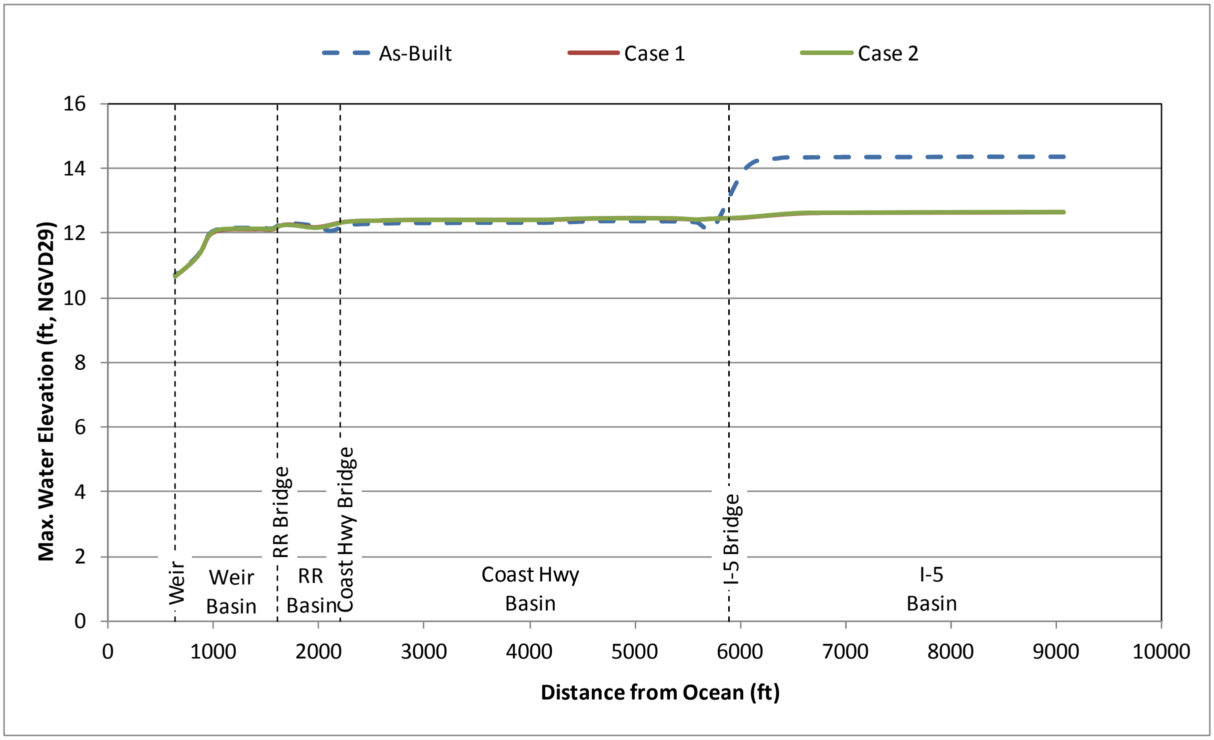


Figure 4.15 Alt FW-A HEC-RAS Results for Analysis Cases without Sea Level Rise

Sea Level Rise Analysis for Alt FW-A

A fluvial hydraulic analysis for Alt FW-A with projected sea level rise in Year 2100 was conducted to determine the bridge dimensions adequate for this condition. For the sea level rise scenario for Alt FW-A, the weir near the ocean inlet/outlet was assumed to be raised adequately to maintain a fresh water regime in the Lagoon. Table 4.16 shows the bridge parameters input for this analysis. The model with the best hydraulic connections derived for the 2011 water level conditions was used in Case 1. The maximum water elevation results for the five cases are presented in Figure 4.16. It should be noted that the maximum water elevations at about 14 feet NGVD is above the deck elevation of the Coast Highway Bridge and the soffit elevation of the Railroad Bridge. The future design of bridges should take into consideration such maximum water elevations when determining the soffit elevations and when estimating bridge loadings due to fluvial flows, including impact loading associated with debris.

Table 4.16 Parameters used in Alt FW-A Sea Level Rise Analysis

BRIDGE	PARAMETERS	AS-BUILT	CASE 1 BEST
Railroad	Invert Elevation (ft, NGVD)	-2.5	-4
	Bottom Width @ Invert (ft)	17	17
	Width @ Ex. Soffit (ft)	280	280
	Deck Elevation (ft, NGVD)	15.2	
	Soffit Elevation (ft, NGVD)	11.1	*
	North side slope (H:V)	12:1	11:1
	South side slope (H:V)	17:1	15:1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6
	Width (ft)	25/29	109
	Deck Elevation (ft, NGVD)	9.7	
	Soffit Elevation (ft, NGVD)	8.2	*
	North side slope (H:V)	vertical	vertical
	South side slope (H:V)	vertical	vertical
I-5	Invert Elevation (ft, NGVD)	-2	-4
	Bottom Width @ Invert (ft)	24	104
	Width @ Ex. Soffit (ft)	99	180
	Deck Elevation (ft, NGVD)	25.0	25.0
	Soffit Elevation (ft, NGVD)	23.1	23.1
	North side slope (H:V)	1.5:1	1.4:1
	South side slope (H:V)	1.5:1	1.4:1

Red = different from as-built
* Proposed soffit elevation should be max water elevation + value (such as freeboard) based on design criteria.

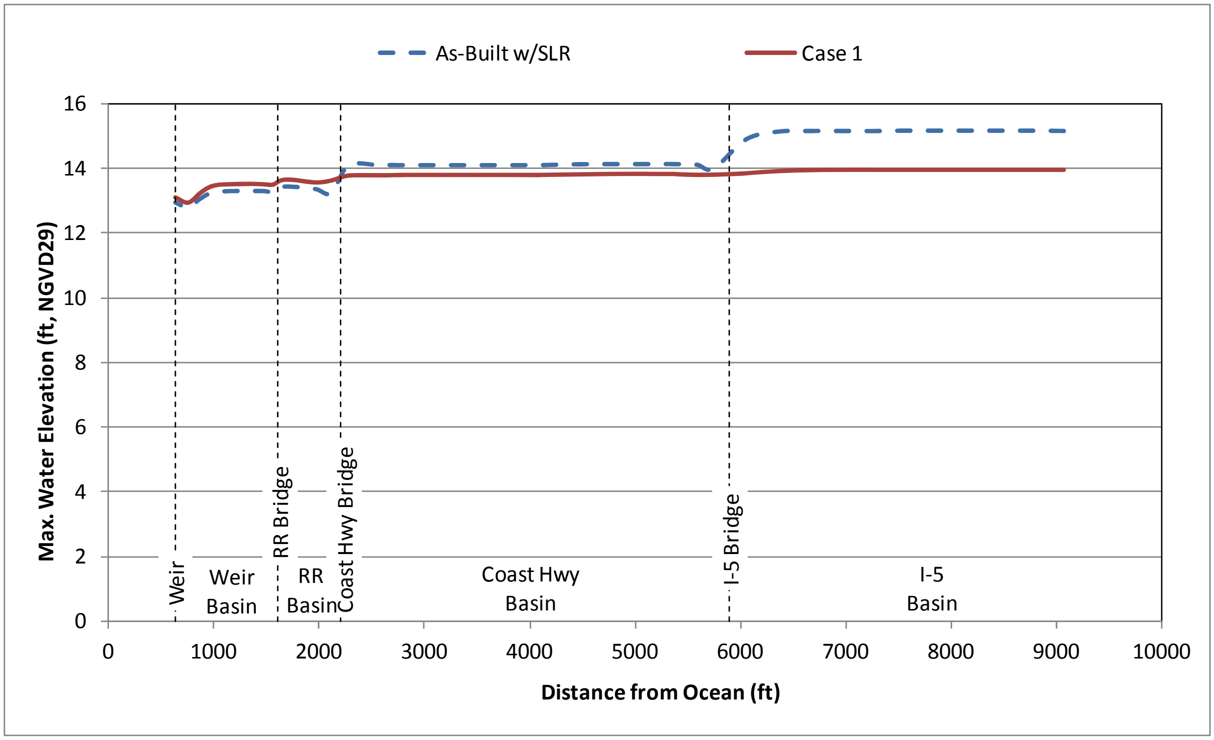


Figure 4.16 Alt FW-A HEC-RAS Results for Sea Level Rise Analysis

4.4 SUPPLEMENTAL ANALYSIS

4.4.1 Impact of Marine Organisms Colonizing Structural Members

Under the salt water alternatives, the Buena Vista Lagoon would be open to tidal exchange and the structural members of the bridges would likely be colonized with marine organisms. In order to evaluate the effect of the reduction in flow cross-sectional area and increase in roughness due to this marine growth, the HEC-RAS models were tested with bridge configurations having reduced cross sectional areas and increased roughness. It was assumed that the walls and piers of the bridge structures would be ultimately covered with six inches of marine growth; therefore, the sectional width of each pier was increased by one foot in the models, and the distance between abutment walls was reduced by one foot. The Manning's coefficient was also increased from 0.03 to 0.5 at the elevation range between MLLW and MHHW to account for the higher roughness of the marine organisms. Simulations were conducted for the 2011 and 2100 best case scenarios for Alt 2-1 and Alt SW2-A. It was found that in all cases the maximum water elevations changed by less than 0.1 feet when compared with the cases without marine growth.

Although the marine growth on the bridge vertical members do not exacerbate the impact of flooding, the bridges in the lagoon should be designed to withstand conditions of a marine environment if the lagoon is restored to salt water, including the effect of salt water corrosion to the structural members.

4.4.2 Peak Fluvial and Tidal Flows Time Phasing Variation Analysis

As discussed in Section 3.3, the peak storm flow entering the Lagoon from upstream was timed to enter the Lagoon at the same time that MHHW occurred at the ocean in order to simulate a reasonably large flood impact in the HEC-RAS models for the proposed alternatives. Depending on the distance from the ocean and from Buena Vista Creek, the highest water elevation due to flood flow may not necessarily occur when the peak fluvial flow and peak tidal flow happen simultaneously. To assess the effect of a time lag between the two peak flow occurrences, several simulations were conducted for the best case scenarios for each salt water alternative using time lags ranging from 15 minutes to 105 minutes. The results of these simulations indicated that there were no differences in the maximum water elevations for the Alt SW2-A for both 2011 and 2100 SLR scenarios. For Alt 2-1, the water elevations at all the bridges were higher. The maximum water elevation when there is a lag between the peak fluvial flow and MHHW, was found to be 0.4 feet higher at the I-5 Bridge for the 2011 scenario and 0.1 feet higher for the 2100 SLR scenario, when compared with those of the synchronized peak flow simulations. The results for different scenarios are shown in Table 4.17.

Table 4.17 Comparison of Maximum Water Elevations Resulted from Peak Flow Phasing Variations

BRIDGE	PEAK FLOWS TIMING	MAXIMUM WATER ELEVATION (FT, NGVD)			
		ALT 2-1		ALT SW2-A	
		2011	2100	2011	2100
Railroad	Synchronized	6.6	9.6	6.6	8.7
	Lag Phasing	6.9	9.7	6.6	8.7
Coast Hwy	Synchronized	6.8	9.8	6.7	9.2
	Lag Phasing	7.1	9.9	6.7	9.2
I-5 Bridge	Synchronized	6.8	9.9	7.2	9.4
	Lag Phasing	7.2	10.0	7.2	9.4

4.4.3 Channel Lining at Bridge Connections

As discussed in Chapter 3, the Manning coefficient used in the HEC-RAS models is 0.03. This is typical for natural channels consisting of soil and small pebbles. It is understood that the channel at the bridge connections may be lined with erosion resistant materials, such as riprap and concrete. The Manning coefficient for concrete lined channel ranges from 0.013 to 0.022, and the Manning coefficient for riprap is about 0.033(USACE 2006). The salt water alternatives were tested with a Manning's coefficient of 0.033 at the bridge connections. It was found that the maximum water elevation increased by less than 0.1 feet at all bridge connections for the tested scenarios. It should be noted that if the channel at the bridge connections is to be lined, the finished ground line of the material should be at the recommended invert elevation.

4.4.4 Width of the Weir at the Ocean Outlet

Similar to the existing conditions, a weir is assumed to be installed at the ocean outlet for the proposed fresh water alternatives Alt 1 and Alt FW-A. While the existing weir is 50 feet wide at the crest, the proposed weir is assumed to be 80 feet wide, which is the configuration of a design previously developed by the City of Oceanside. In order to evaluate the adequacy of the proposed bridge configurations for the existing 50-foot weir at the ocean outlet, additional HEC-RAS analyses were conducted for the two fresh water alternatives for the 2011 scenarios. The HEC-RAS results for the fresh alternatives with the 50-foot wide weir were slightly different. For Alt 1, the maximum water elevation at all the bridges was 11.1 feet, NGVD for an increase of 0.4 feet. For Alt FW-2, the maximum water elevation was 12.5 feet, NGVD at the I-5 Bridge, 12.4 feet, NGVD at the Coast Highway culvert, and 12.3 feet, NGVD at the Railroad Bridge for an increase of 0.1 feet at each location.

4.4.5 Maximum Water Elevation

Based on the results of the above sensitivity analyses, it was found that the maximum water elevation may increase by 0.5 feet when the combined effects of peak flow time phasing, channel lining friction coefficient, and marine growth are considered for the salt water alternatives. For the fresh water alternatives, the combined effects due to channel lining friction coefficient and weir width would increase the maximum water elevation by 0.5 feet.

4.5 Analysis for Minor Storms

While the 100-year storm event was used in the analysis of bridge parameters, an analysis for minor storm events was conducted for one restoration alternative to verify that the pattern of impacts (e.g., distribution of flooding among basins and timing of flooding with respect to peak of the hydrograph) of minor storms would not be different than that predicted for the

100-year event. The flood impacts for storms of 2-, 5-, 10-, 25- and 50-year return periods were simulated in HEC-RAS for the optimized case of Alt SW2-A. The maximum water elevation results for various storm events are shown in Figure 4.17. The results indicate that the pattern of impacts for these minor storm events is similar to the pattern of impacts predicted for the 100-year storm.

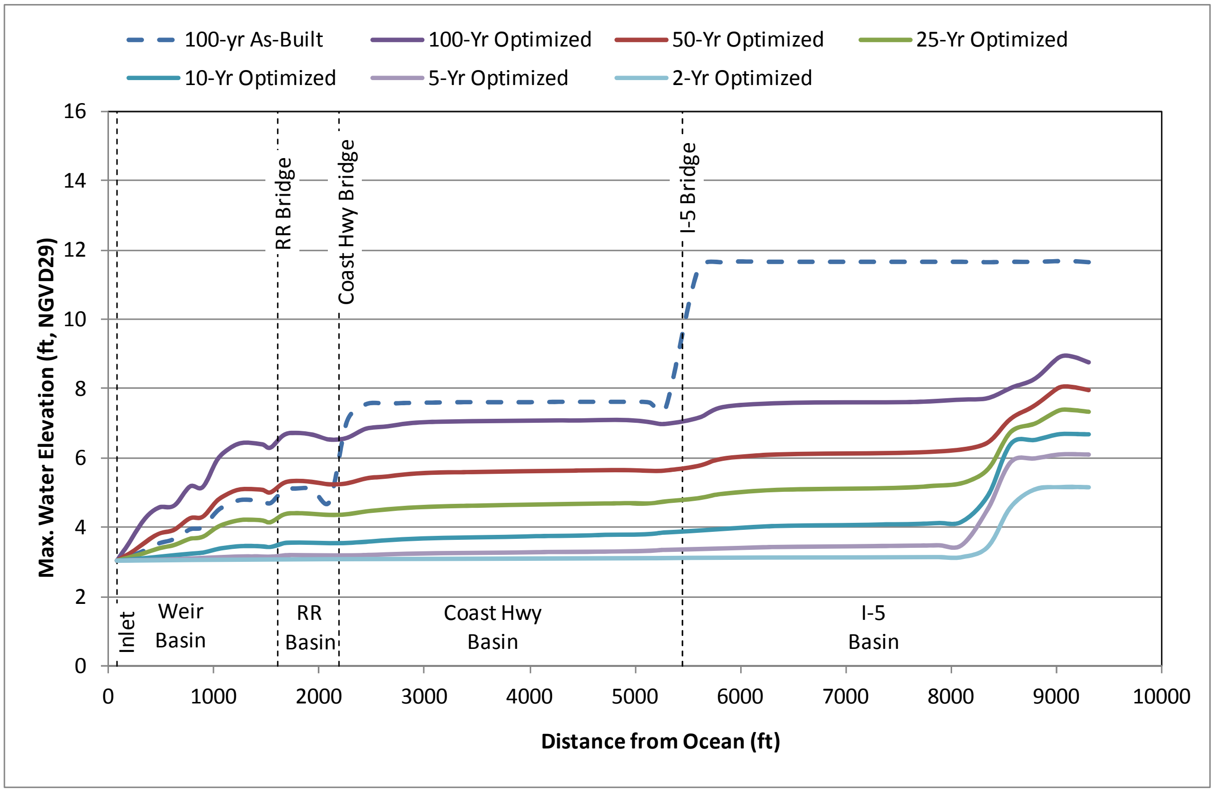


Figure 4.17 Fluvial Hydraulic Analyses for Minor Storms for Alt SW2-A Best Case

4.6 SUMMARY AND RECOMMENDATIONS

The best cases for the salt water alternatives are summarized in Table 4.18 and those for the fresh water alternatives are summarized in Table 4.19. To incorporate the effects of time peak flow time phasing, marine growth, weir width, and increased friction resulted from lined bridge channel bottom as discussed in the previous section, the maximum water elevations in the tables have been adjusted by adding 0.5 feet to the values from the HEC-RAS output. The columns on the right side of the tables show the highest value for each parameter among the alternatives and scenarios. Figures 4.18 to 4.20 show the desired cross sections of the Railroad Bridge, Coast Highway Bridge, and I-5 Bridge, respectively. In these figures,

Table 4.18 Desired Bridge Parameters and Maximum Water Elevations for Salt Water Alternatives

Bridge	Parameters	As-Built	Alt 2-1		Alt SW2-A		Desired
			2011	2100 SLR	2011	2100 SLR	
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-4	-4	-4
	Bottom Width @ Invert (ft)	17	17	17	17	17	17
	Width @ Existing Soffit (ft)	280	280	280	280	280	280
	Soffit Elevation (ft, NGVD)	11.1	*	*	*	*	*
	Max Water Elevation (ft, NGVD)		7.1	10.1	7.1	9.2	10.1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6	-6	-6	-6	-6
	Width (ft)	25/29	80	80	109	109	110
	Soffit Elevation (ft, NGVD)	8.2	*	*	*	*	*
	Max Water Elevation (ft, NGVD)		7.3	10.3	7.2	9.7	10.3
I-5	Invert Elevation (ft, NGVD)	-2	-6	-6	-4	-4	-6
	Bottom Width @ Invert (ft)	24	85	85	104	104	105
	Width @ Existing Soffit (ft)	99	160	160	180	180	180
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1	23.1
	Max Water Elevation (ft, NGVD)		7.3	10.4	7.7	9.9	10.4

Red = different from as-built

* Proposed soffit elevation should be max water elevation + value (such as freeboard) based on design criteria.

Table 4.19 Desired Bridge Parameters and Maximum Water Elevations for Fresh Water Alternatives

Bridge	Parameters	As-Built	Alt 1		Alt FW-A		Desired
			2011	2100 SLR	2011	2100 SLR	
Railroad	Invert Elevation (ft, NGVD)	-2.5	-2.5	-2.5	-4	-4	-4
	Bottom Width @ Invert (ft)	17	17	17	17	17	17
	Width @ Existing Soffit (ft)	280	280	280	280	280	280
	Soffit Elevation (ft, NGVD)	11.1	*	*	*	*	*
	Max Water Elevation (ft, NGVD)		11.2	12.9	12.7	14.1	14.1
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6	-6	-6	-6	-6
	Width (ft)	25/29	80	80	109	109	110
	Soffit Elevation (ft, NGVD)	8.2	*	*	*	*	*
	Max Water Elevation (ft, NGVD)		11.2	12.9	12.9	14.3	14.3
I-5	Invert Elevation (ft, NGVD)	-2	-6	-6	-4	-4	-6
	Bottom Width @ Invert (ft)	24	85	85	104	104	105
	Width @ Existing Soffit (ft)	99	160	160	180	180	180
	Soffit Elevation (ft, NGVD)	23.1	23.1	23.1	23.1	23.1	23.1
	Max Water Elevation (ft, NGVD)		11.2	12.9	13.0	14.4	14.4

Red = different from as-built
* proposed soffit elevation should be max water elevation + value (such as freeboard) based on design criteria

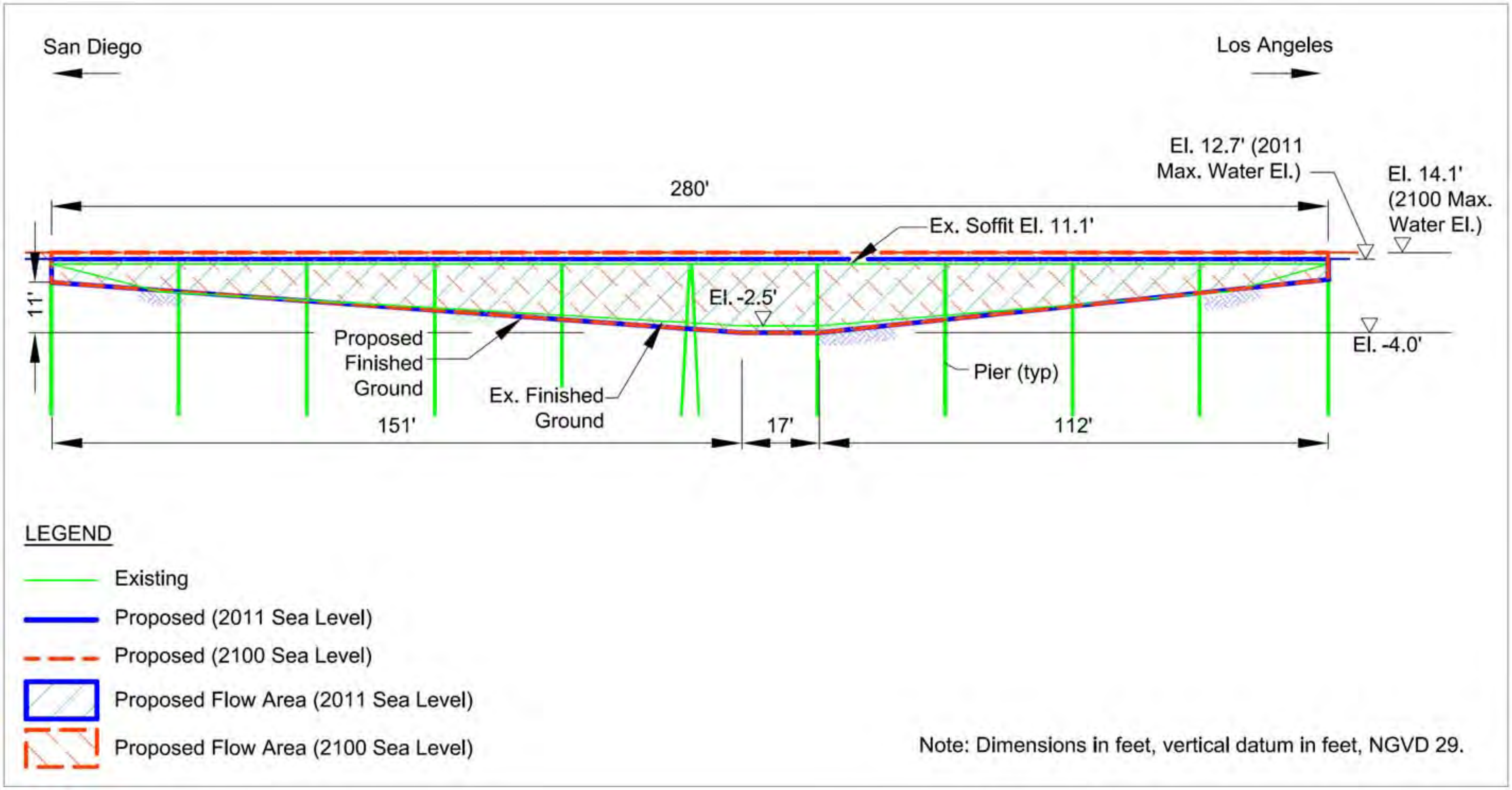


Figure 4.18 Desired Channel Cross Section of Railroad Bridge at Buena Vista Lagoon

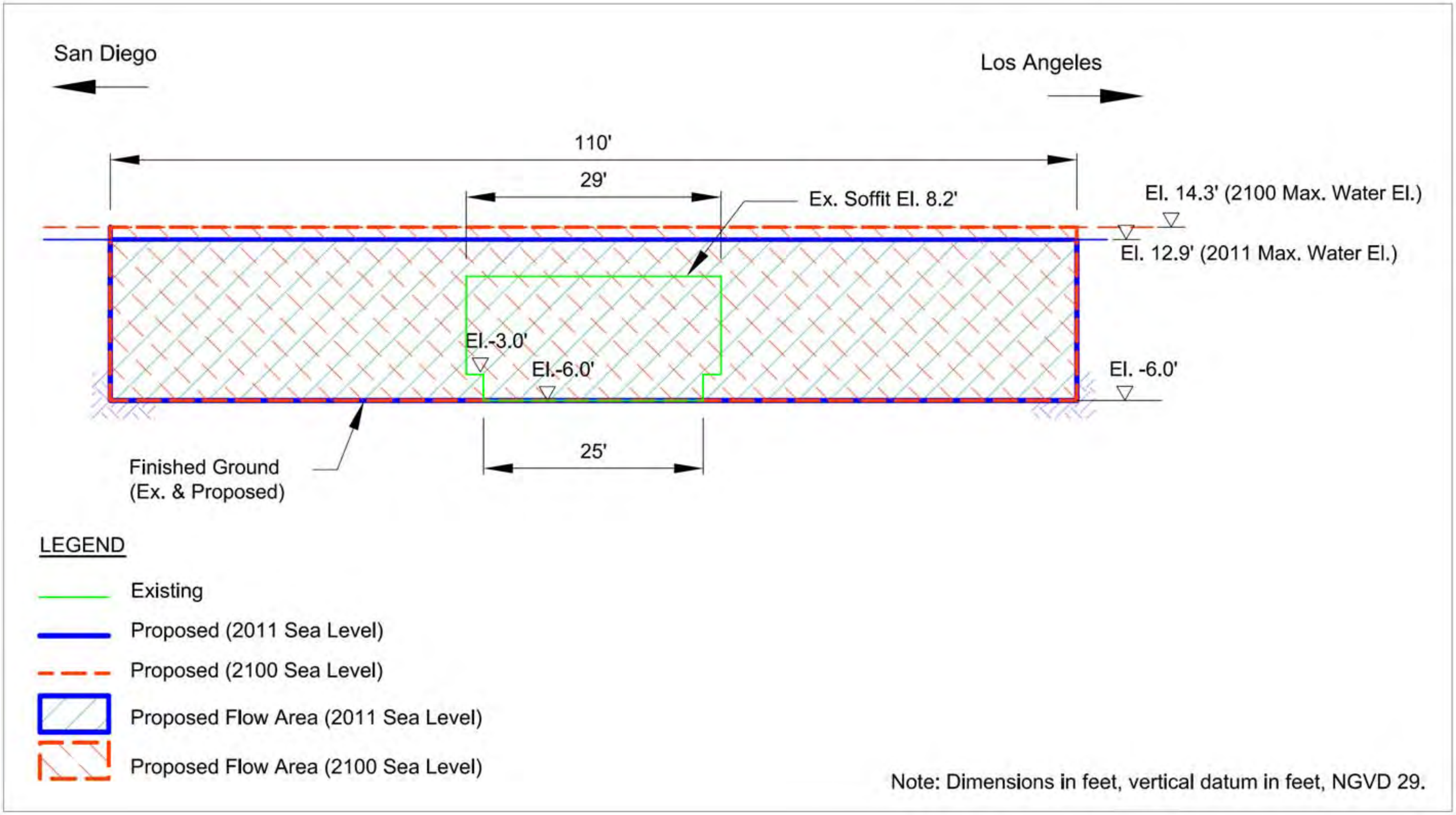


Figure 4.19 Desired Channel Cross Section of Coast Highway Bridge at Buena Vista Lagoon

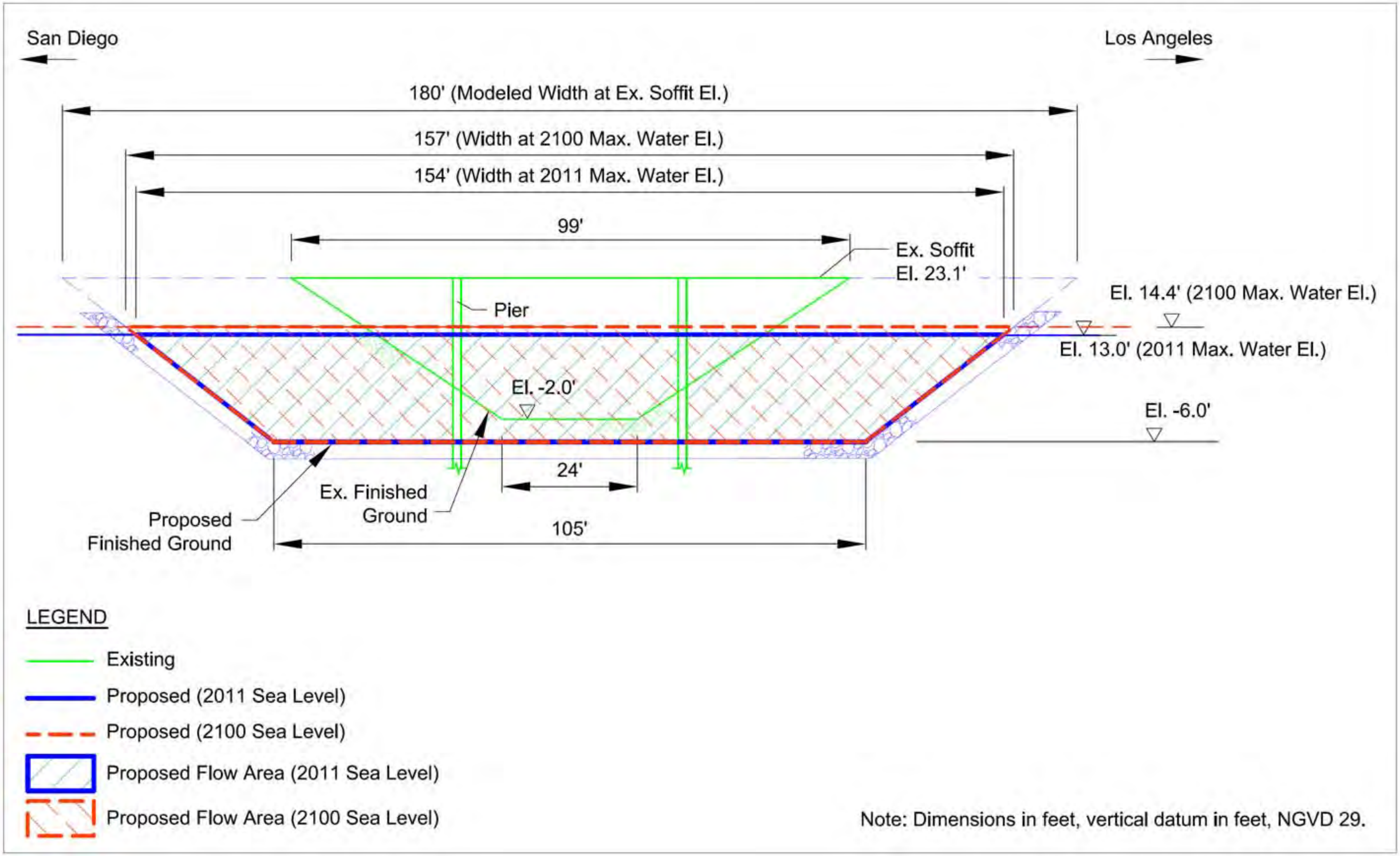


Figure 4.20 Desired Channel Cross Section of I-5 Bridge at Buena Vista Lagoon

the required flow area for each bridge is shown for the 2011 sea level and 2100 sea level conditions.

Table 4.20 compares the desired bridge parameters for both the salt water and fresh water alternatives as well as presents the highest value of each parameter in the right column. The desired dimensions for the bridges are the set of values listed in the right column of the table. Figures 4.21 to 4.23 show the proposed cross sections of the Railroad Bridge, Coast Highway Bridge, and I-5 Bridge, respectively. It should be noted that references to channel widths in the tables are based on the assumption that the cross sectional sizes of vertical members (e.g. columns and piers) of the future bridge structures in the channel will be equal to or less than those of the existing structures. In general, additional analyses may be needed if substantially larger vertical members are stipulated in the design since such an increase could subtantially decrease the cross sectional flow area utilized in the analyses conducted for this study, thereby potentially resulting in higher water elevations. For the I-5 Bridge, the vertical members were simulated in HEC-RAS as two bents of columns each at 1.5 feet in diameter, which was based on the as-built plans. The configuration currently designed for the vertical members of the proposed I-5 Bridge is similar to that of the as-built, except that the columns would be 3 feet in diameter instead of 1.5 feet. The larger columns would reduce the overall channel width by 3 feet. Since this channel area reduction is negligible when compared with the overall channel width, the proposed column sizes should not result in substantial changes in water elevation.

The desired soffit elevation is not provided but is recommended to be based on the maximum water elevation as predicted by the fluvial hydraulic analyses and listed in the same table, as well as other design criteria such as recommended freeboard value. If the desired soffit elevations cannot be achieved due to other design limitations, strategies to reduce the maximum water elevations may be evaluated in future phases. For fresh water alternatives, one such strategy would be to increase the flow area by widening the width of the weir, which was assumed to be 80 feet wide in this study.

For the Railroad Bridge, the fluvial hydraulics analysis concluded that the invert of -4’ NGVD would be adequate for the fluvial flow. Nevertheless, it is recommended that the invert at the Railroad Bridge be deepened to -6’ NGVD. Based on previous restoration studies, this lower channel elevation would be needed to accommodate a near full tide range for a salt water regime in the lagoon.

In addition to achieving the desired bridge parameters as recommended in this study, the other factors that the bridge designers should consider in designing new bridges at Buena Vista Lagoon include foundation structures in salt water, foundation structures in deep lagoon basins (which can be as deep as -15’ NGVD as in Alt 1 and Alt 2-1), and forces from storm water and tidal flows.

Table 4.20 Lagoon Restoration Design Guidance for Bridge Dimensions

BRIDGE	PARAMETERS	AS-BUILT	SALT WATER ALTS	FRESH WATER ALTS	DESIGN GUIDANCE**
Railroad	Invert Elevation (ft, NGVD)	-2.5	-4	-4	-6***
	Bottom Width (ft) @ Invert	17	17	17	20
	Width (ft) @ Existing Soffit	280	280	280	280
	Channel Width (ft) @ MWE		280	280	280
	Soffit Elevation (ft, NGVD)	11.1	*	*	*
	MWE (ft, NGVD)		10.1	14.1	15
Coast Hwy	Invert Elevation (ft, NGVD)	-6/-3	-6	-6	-6
	Bottom/Top Width (ft)	25/29	110	110	110
	Channel Width (ft) @MWE		110	110	110
	Soffit Elevation (ft, NGVD)	8.2	*	*	*
	MWE (ft, NGVD)		10.3	14.3	15
I-5	Invert Elevation (ft, NGVD)	-2	-6	-6	-6
	Bottom Width (ft) @ Invert	24	105	105	105
	Width (ft) @ Existing Soffit	99	180	180	180
	Channel Width (ft) @ MWE		147	157	160
	Soffit Elevation (ft, NGVD)	23.1	*	*	*
	MWE (ft, NGVD)		10.4	14.4	15

Red = Different from as-built
MWE = Maximum Water Elevation
* Soffit elevation should be determined during bridge design.
** Elevations are rounded up to the nearest whole number and top and bottom widths are rounded up to the nearest five.
*** 2 feet is added to the desired invert elevation for fluvial flows to accommodate near full tide range.

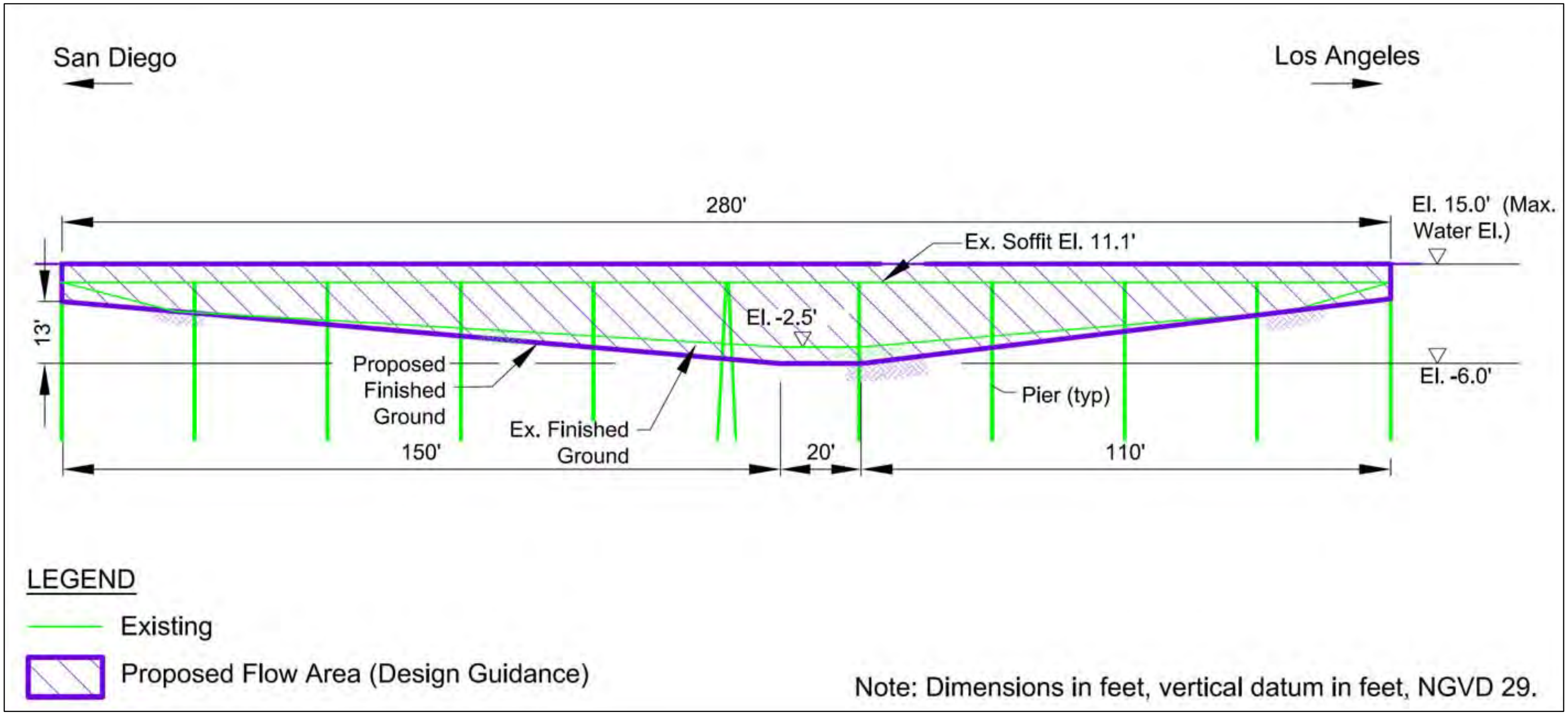


Figure 4.21 Design Guidance for Channel Cross Section of Railroad Bridge at Buena Vista Lagoon

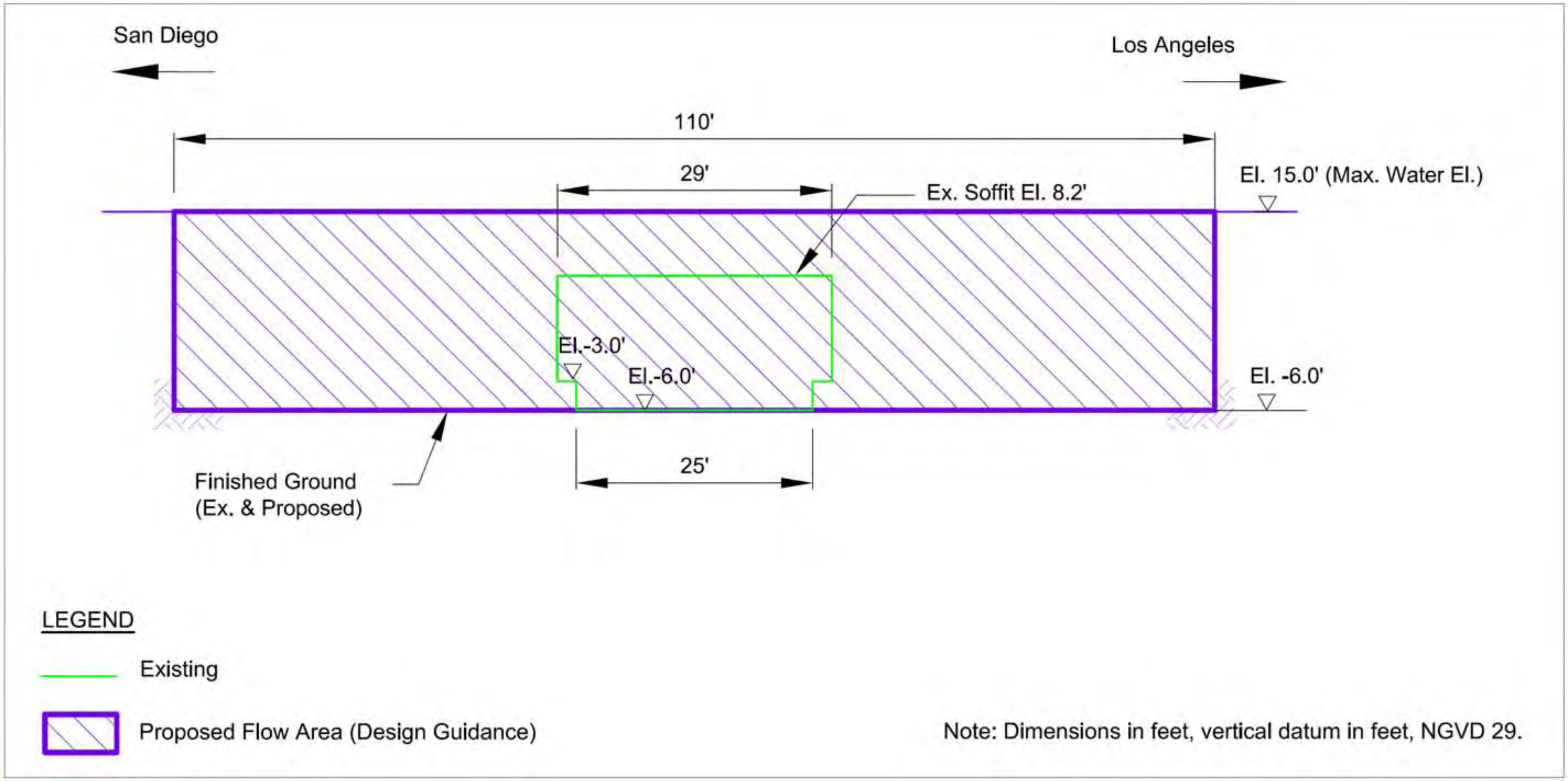


Figure 4.22 Design Guidance for Channel Cross Section of Coast Highway Bridge at Buena Vista Lagoon

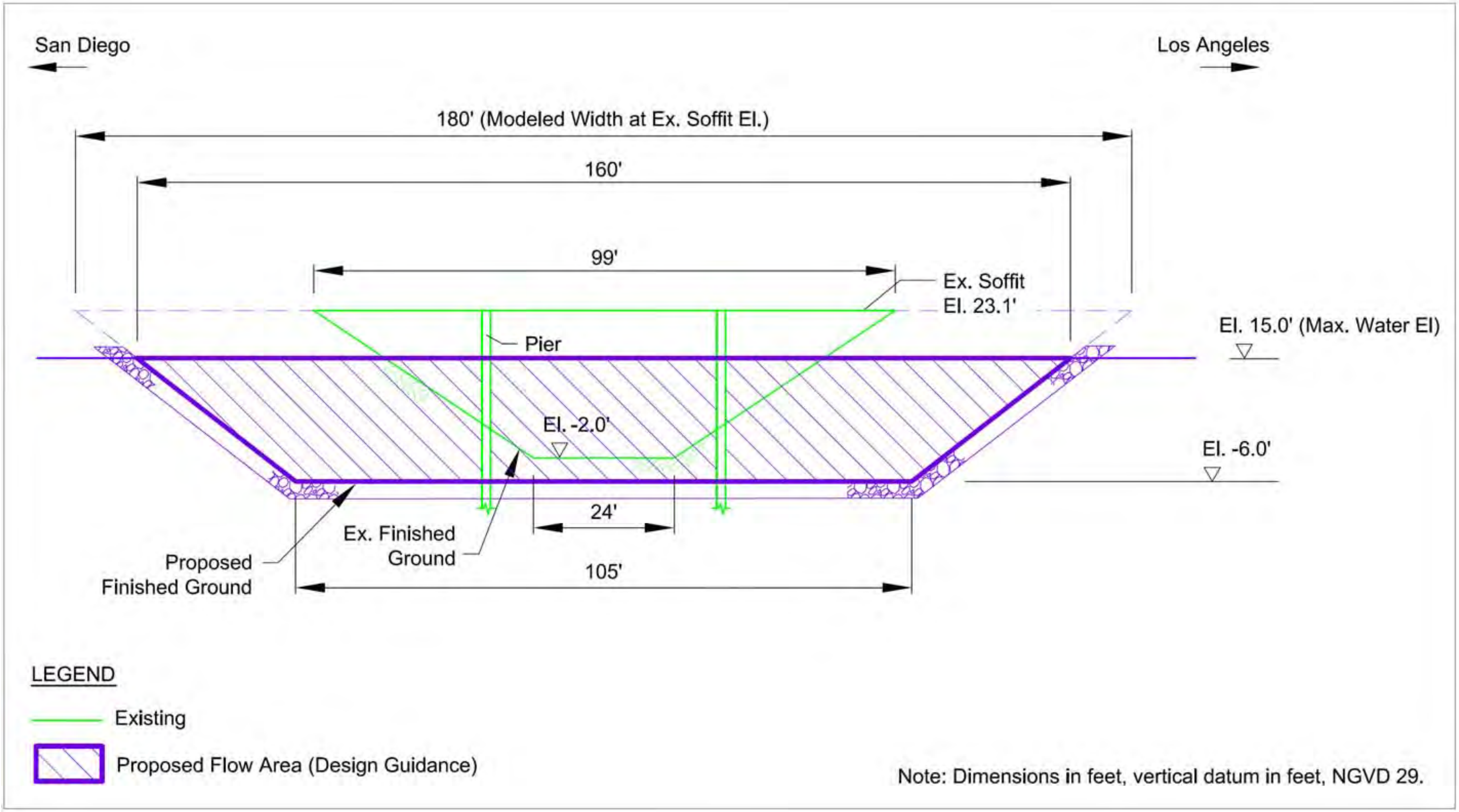


Figure 4.23 Design Guidance for Channel Cross Section of I-5 Bridge at Buena Vista Lagoon

5 RESIDENCE TIME ANAYSIS

This chapter summarizes the residence time analysis that was performed for the two salt water alternatives presented in Chapter 2. The main objective of the analysis was to analyze the tidal flushing capacity of the proposed alternatives as an indicator of potential water quality. The approach, results, and conclusions of the residence time analysis are presented below.

Residence time is commonly used as a surrogate for water quality. The potential for water quality issues is greater for areas with long residence times such as the back ends of enclosed water bodies farther away from the coastline. These areas are generally characterized by poor flushing with low net flow exchange. Long residence times are indicative of stagnant water with poor flushing while short residence times are indicative of good water circulation and flushing. For a given level of pollutant loading, better flushing usually indicates better water quality in a water body.

5.1 STUDY APPROACH

A two-dimensional (2-D) hydrodynamic and water quality model (EFDC) was used to estimate residence times by simulating average hydrodynamic and mixing conditions within the Lagoon. The 2-D model used for the residence time analysis is the Environmental Fluid Dynamic Code (EFDC) hydrodynamic and water quality model developed by the U.S. Environmental Protection Agency (EPA) (Tetra Tech 2007). EFDC is capable of simulating the hydrodynamic conditions of subcritical flows in estuarine systems with dynamic coupling to sediment/toxic transport and water quality (eutrophication) components. In addition, EFDC can simulate the wetting/drying effects that occur in estuarine systems due to the rise and fall of water elevations associated with tides. The EPA, San Diego Regional Water Quality Control Board, and watershed stakeholders have selected EFDC to be used in the development and implementation of TMDLs in the region.

EFDC was used to simulate tidal exchange between the ocean and Lagoon under the two salt water alternatives. Tidal water elevations, currents, and the transport of a conservative tracer were simulated within the Lagoon. Residence times within the Lagoon were determined based on the transport and dilution of the tracer due to tidal exchange. An initial amount of tracer was simulated over a 30-day period. Over time the initial tracer concentration decreased as “clean” water from the ocean replaces the water within the Lagoon, commonly referred to as tidal flushing. Residence times in the Lagoon were determined as the time required for the tracer concentration to drop to e^{-1} of the initial concentrations (i.e., time it takes for an initial concentration to drop from 1 to 0.368).

5.2 MODEL SETUP

The hydrodynamic and mixing characteristics of the two salt water alternatives (Alternatives 2-1 and SW2-A) were simulated. The bathymetry of these alternatives is shown in Figure 5.1. The bathymetry was used to prepare a model grid that covers the Lagoon and extends into the ocean. The Lagoon grading for Alt 2-1 contains deeper tidal channels and basins compared to Alt SW2-A. Bathymetry in the ocean was obtained from the National Oceanic and Atmospheric Administration (NOAA) Electronic Navigation Charts. Bridge dimensions correspond to the recommended channel widths and depths determined from the fluvial hydraulics analysis. Bridge structure dimensions determined based on the fluvial hydraulics analysis for Alt 2-1 and Alt SW2-A are summarized in Tables 5.1 and 5.2, respectively.

Table 5.1 Bridge Structure Dimensions for Alt 2-1

STRUCTURE	INVERT WIDTH (FT)	INVERT ELEVATION (FT, NGVD)	CHANNEL SIDE SLOPE (H:V)
Railroad Bridge	17	-2.5	17:1 (S), 12:1 (N)
Coast Highway Bridge	80	-6.0	Vertical
I-5 Bridge	85	-6.0	1.3:1

H:V = horizontal to vertical, S = south side, N = north side

Table 5.2 Bridge Structure Dimensions for Alt SW2-A

STRUCTURE	INVERT WIDTH (FT)	INVERT ELEVATION (FT, NGVD)	CHANNEL SIDE SLOPE (H:V)
Railroad Bridge	17	-4.0	15:1 (S), 11:1 (N)
Coast Highway Bridge	109	-6.0	Vertical
I-5 Bridge	104	-4.0	1.4:1

H:V = horizontal to vertical, S = south side, N = north side

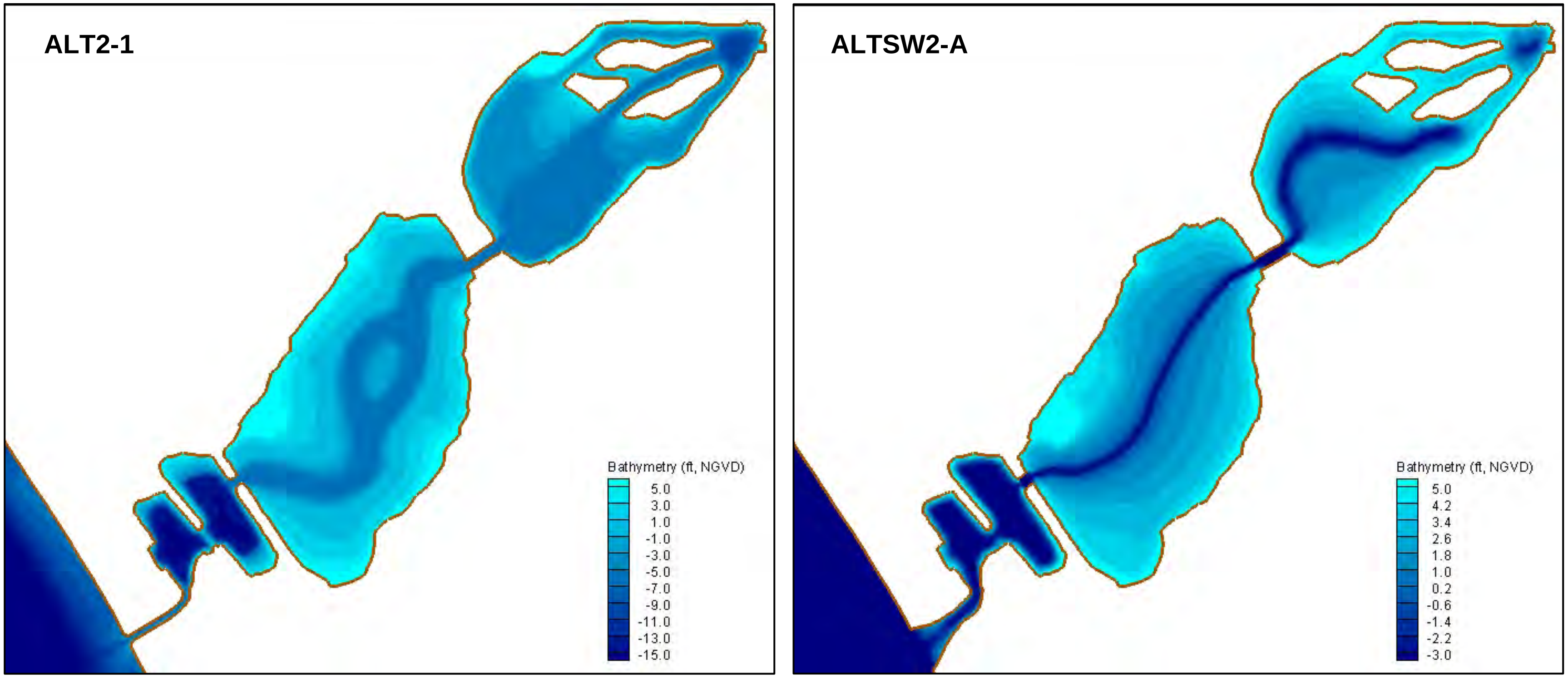


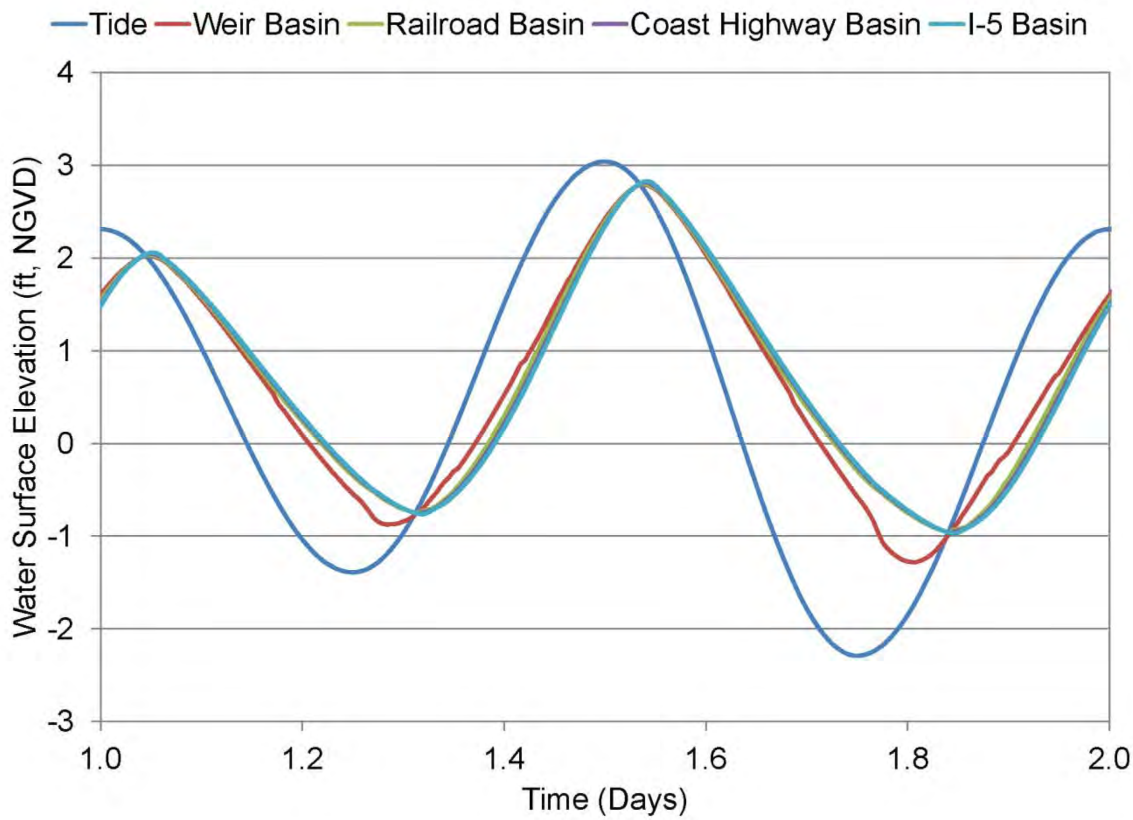
Figure 5.1 Buena Vista Lagoon Bathymetry used in Model Grids for Salt Water Alternatives

Tidal exchange between the Lagoon and ocean were simulated based on diurnal tide conditions to reflect typical tidal conditions. Tides off the coast of the lagoon are mixed, semidiurnal with two daily highs and lows. Tidal datums at the NOAA Scripps Pier station (9410230) is La Jolla are provided in Table 3.1.

5.3 TIDAL FLUSHING

The EFDC model was used to simulate the tidal response within the alternatives. Water levels within the Lagoon under Alternatives 2-1 and SW2-A are illustrated in Figures 5.2 and 5.3, respectively. The top panel in each figure compares the ocean tide range to the water surface elevations within each Lagoon basin. The bottom two panels in the figures compare the inundated or wetted area of the Lagoon at approximately MHHW and MLLW. Water covers most of the Lagoon during high tide (MHHW). At low tide (MLLW), water drains out of the intertidal area, leaving water only in the tidal channels such as the Coast Highway Basin. Hence, during each tidal cycle, most of the water in the intertidal areas will be flushed out during low tide. In other words, the residence time within the intertidal area would be less than one day. Longer residence times are expected at locations within the deeper portions of the tidal channels that are continuously inundated with water.

The transport and mixing of a conservative tracer was simulated to develop a quantitative estimate of the residence time. An example of using the simulated tracer concentrations to estimate the residence time in the Weir Basin for both salt water alternatives is shown in Figure 5.4. As shown in the figure, the tracer concentration oscillates with the tidal cycle. Over time the overall tracer concentration decreases exponentially with time (indicated by the black line). The residence time is determined as the time for the tracer concentration to be reduced to 0.368 (e^{-1}) of the initial condition (indicated by the red line in the figure). For Alt 2-1, the residence time in the Weir Basin is approximately 3 days, while the residence time in the Weir Basin is about 2 days for Alt SW2-A.



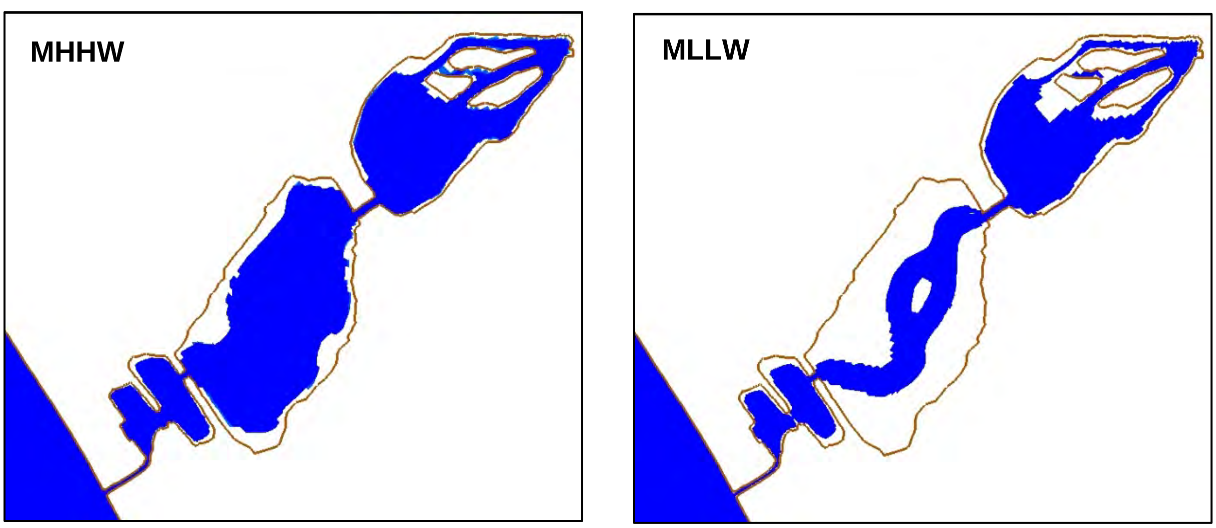
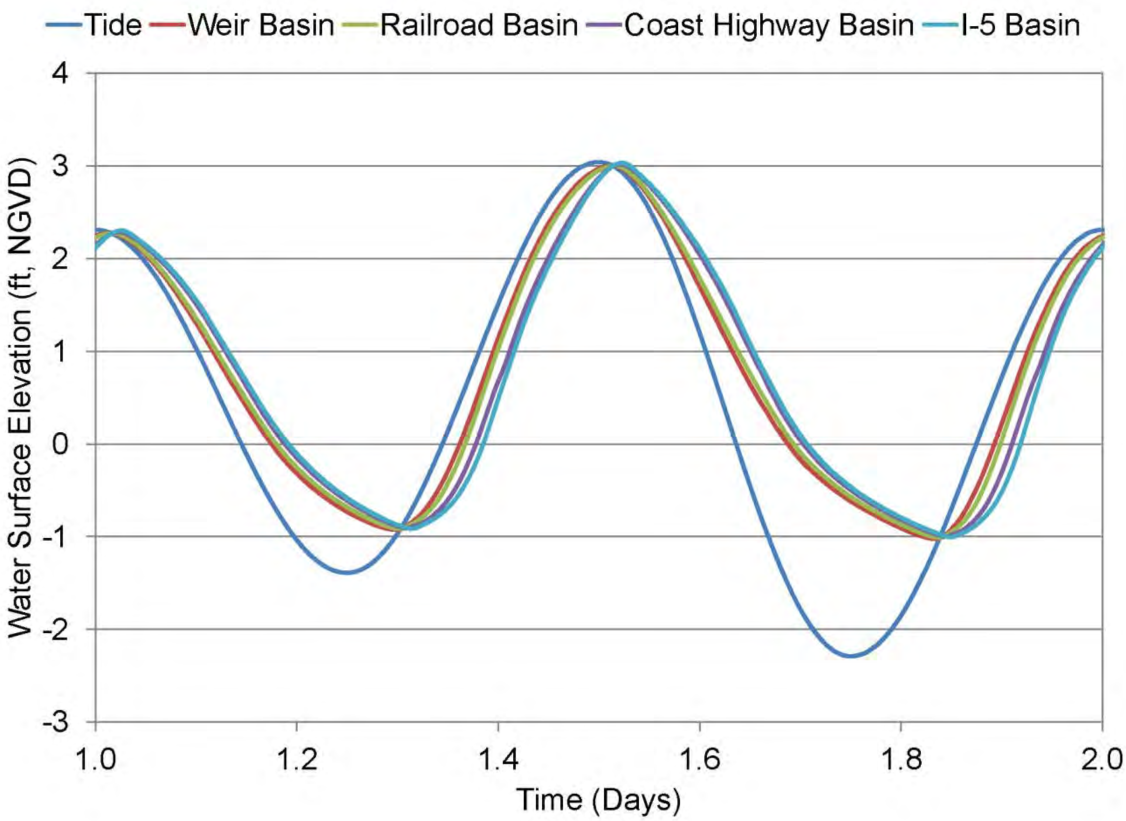
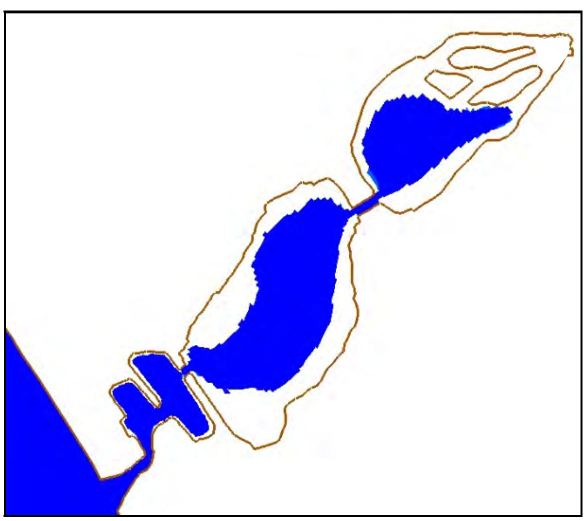


Figure 5.2 Alternative 2-1 Tidal Response





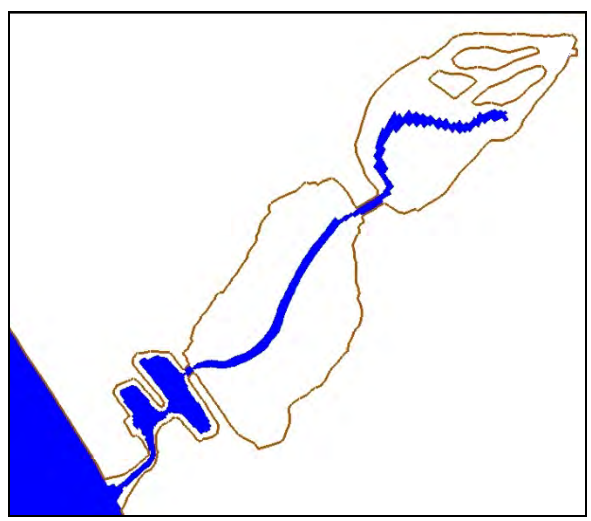
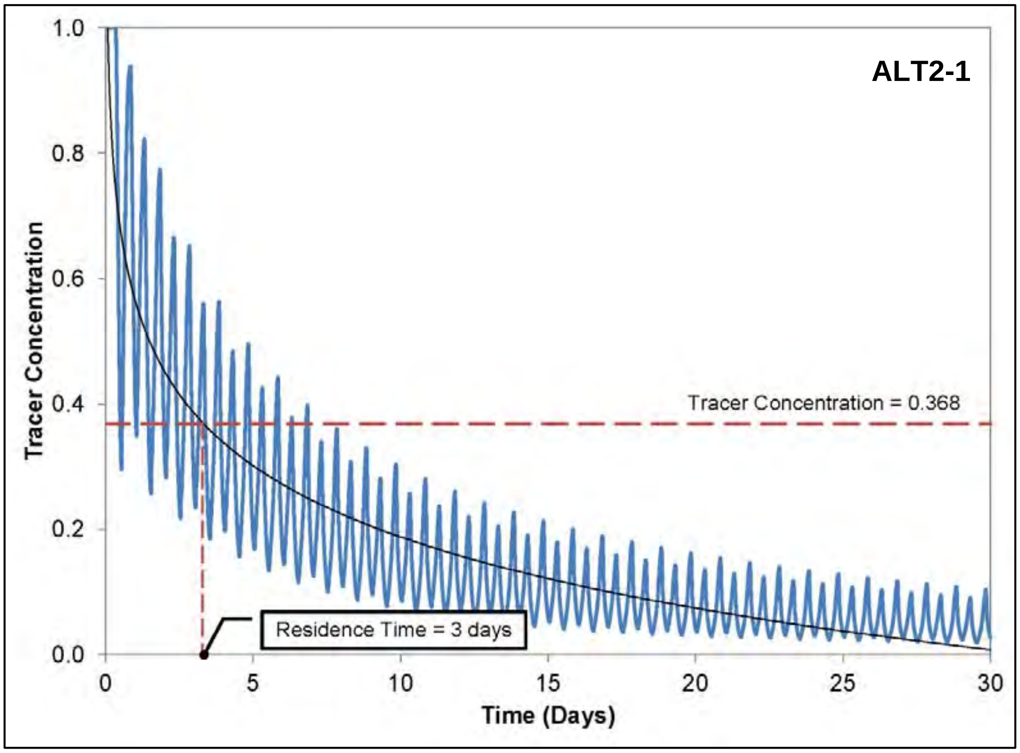


Figure 5.3 Alternative SW2-A Tidal Response



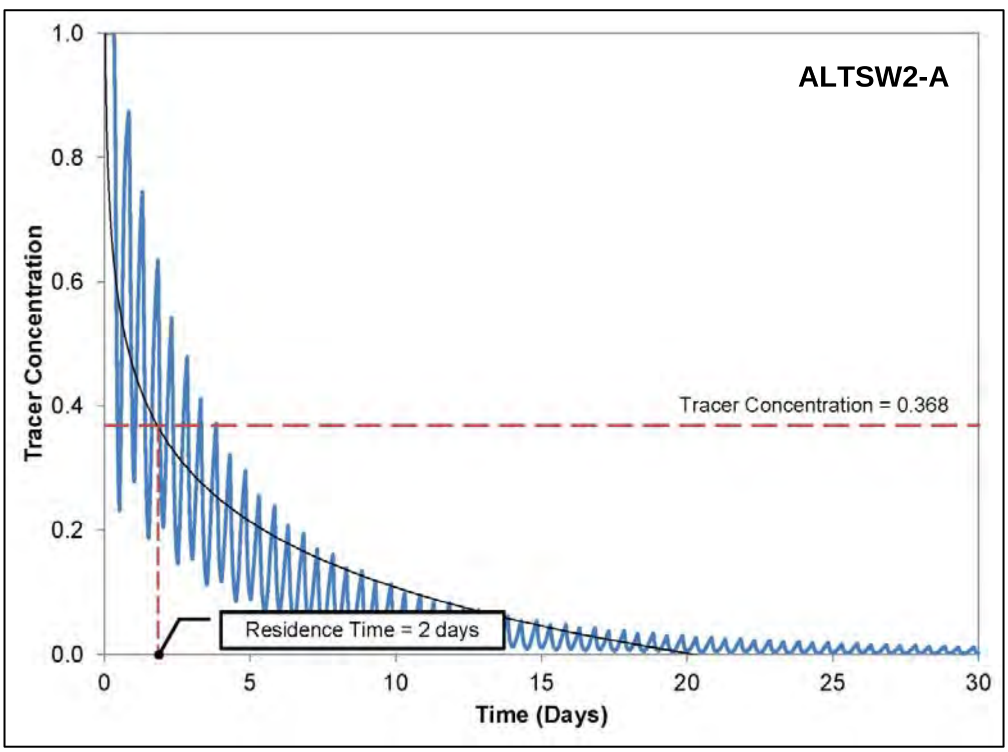


Figure 5.4 Tracer Concentrations in Weir Basin

The calculated residence times were determined in the Lagoon for Alternatives 2-1 and SW2-A, as shown in Figure 5.5. Due to the differences in the range of residence times for each alternative, different color scales are used for the two alternatives. Both alternatives show that the residence times are shortest near the tidal inlet and increase away from the tidal inlet. Residence times in each basin are summarized in Table 5.3. As expected, the shortest residence time is located nearest the tidal inlet in the Weir Basin. The residence time increases towards the back ends of the Lagoon in the I-5 Basin.

Table 5.3 Residence Time Analysis Results

LAGOON BASIN	RESIDENCE TIME (DAYS)	
	ALT 2-1 (2008)	ALT SW2-A (2011)
Weir Basin	3	2
Railroad Basin	6	3
Coast Highway Basin	11	4
I-5 Basin	26	6

5.4 SUMMARY

Potential water quality conditions of the two saltwater alternatives were evaluated based on a residence time analysis to assess tidal flushing. A numerical model was used to simulate tidal exchange in and out of the Lagoon. The results of the tidal hydraulic model were used, in conjunction with a conservative tracer transport model, to estimate residence times within the Lagoon. The results of the analysis indicated that the intertidal areas of the Lagoon would be flushed out with each tidal cycle since water completely drains from these areas during low tides (i.e., residence time less than one day). The residence times within the tidal channels of the Lagoon basins were found to vary throughout the Lagoon with the shortest residence times closest to the tidal inlet.

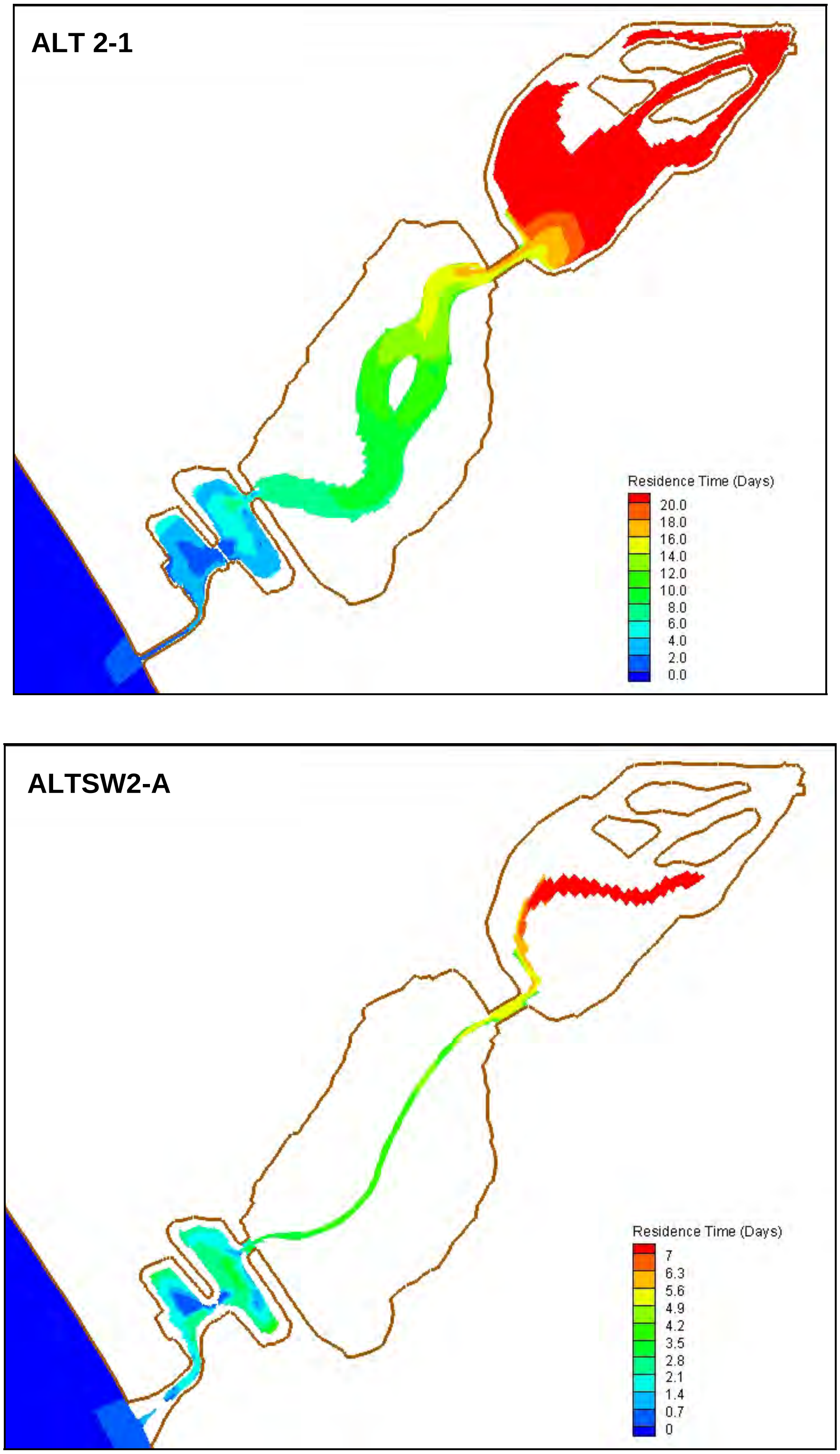


Figure 5.5 Residence Times for Salt Water Alternatives

6 CONCLUSIONS

A fluvial hydraulics analysis was conducted to provide guidance for the three bridge/culvert structures (Interstate 5 Bridge, Coast Highway Culvert/Bridge, and Railroad Bridge) located within the Buena Vista Lagoon. The purpose of the analysis was to establish the minimum channel width and minimum channel depth that would need to be accommodated by the three bridge/culvert structures such that future implementation of a salt water or fresh water restoration alternative will not be restricted by the existing and future bridges/culverts. The fluvial hydraulics analysis was conducted under the current mean sea level and the projected mean sea level in the Year 2100 based on the current guidance provided by the California Ocean Protection Council (COPC 2011). Based on the results of the analysis, the conclusions presented below were drawn for each of the three bridge channels. It should be noted that references to channel widths below are based on the assumption that the cross sectional sizes of vertical members (e.g., columns and piers) of the future bridge structures in the channel will be equal to or less than those of the existing structures. In general, additional analyses may be needed if substantially larger vertical members are stipulated in the design since such an increase could subtantially decrease the cross sectional flow area utilized in the analyses conducted for this study, thereby potentially resulting in higher water elevations. For the I-5 Bridge, the vertical members were simulated in HEC-RAS as two bents of columns each at 1.5 feet in diameter, which was based on the as-built plans. The configuration currently designed for the vertical members of the proposed I-5 Bridge is similar to that of the as-built, except that the columns would be 3 feet in diameter instead of 1.5 feet. The larger columns would reduce the overall channel width by 3 feet. Since this channel area reduction is negligible when compared with the overall channel width, the proposed column sizes should not result in substantial changes in water elevation.

1. The channel under the existing Interstate 5 Bridge is not sufficient to accommodate a near full tide range nor is it sufficient to convey the fluvial flows analyzed in this study. The new Interstate 5 Bridge should be designed to accommodate a channel with a bottom width of 105 feet (at -6 ft, NGVD) and top width of 160 ft (at 15 ft, NGVD). The proposed invert elevation is the elevation of the finished ground. If channel lining is installed, the invert elevation should be the top of the lining material. The soffit of the existing Interstate 5 Bridge (23.1 ft, NGVD) is eight feet above the predicted flood water elevation for a 100-year flood event occurring with the projected mean sea level for Year 2100.

2. The channel under the existing Coast Highway Culvert/Bridge is not sufficient to accommodate a near full tide range nor is it sufficient to convey the fluvial flows analyzed in this study. The new Coast Highway Culvert/Bridge should be designed to accommodate a vertically-walled channel with a width of 110 feet and bottom

elevation of -6 ft, NGVD. This is the elevation of the finished ground. If channel lining is installed, the invert elevation should be the top of the lining material. The soffit of the existing Coast Highway Culvert/Bridge (8.2 ft, NGVD) is about 2 feet (salt water alternative) to 6 feet (freshwater alternative) below the predicted flood water elevation for a 100-year flood event occurring with the projected mean sea level for Year 2100 thereby indicating that flooding of the structure would occur. This should be taken into account during design of the new Coast Highway Culvert/Bridge.

3. The channel under the existing Railroad Bridge is not sufficient to accommodate a near full tide range nor is it sufficient to convey the fluvial flows analyzed in this study. The width of the channel under the existing Railroad Bridge would be adequate to accommodate the fluvial flows analyzed in this study; however, the bottom elevation would need to be deepened from –2.5 ft, NGVD to -4 ft, NGVD. To accommodate a near full tide range the bridge would need to accommodate a channel with a bottom elevation of -6 ft, NGVD. This is the elevation of the finished ground. If channel lining is installed, the invert elevation should be the top of the lining material. If the existing bridge structure and foundation are capable of accommodating this increase in channel depth and the forces from higher flood levels, then the existing structural configuration would not need to be changed and would still convey the fluvial flows analyzed in this study and accommodate the implementation of a near full tidal salt water restoration project in the future (Everest 2008). This should be taken into account during design of a new Railroad Bridge when such work is undertaken. The soffit of the existing Railroad Bridge (11.1 ft, NGVD) is about 3 feet below the predicted flood water elevation for a 100-year flood event occurring with the projected mean sea level for Year 2100; therefore, flooding of that structure would occur under the fluvial flows analyzed in this study. This should be taken into account during design of a new Railroad Bridge when such work is undertaken.

4. The results of the fluvial modeling indicated that improvements to the Interstate 5 Bridge and Coast Highway Culvert/Bridge would result in higher flood levels within the Coast Highway Basin and Weir Basin because the flood flow is conveyed more efficiently to these lower basins from the Interstate 5 Basin. While representing an improvement in the overall flood hydraulics, an increase in flood levels within these two basins under the two freshwater alternatives could result in impacts to private property and infrastructure, especially in the Weir Basin where the St. Malo community is located. Everest (Everest 2004 and 2008) reported a similar finding for existing conditions, which is a freshwater system controlled by the 50 ft wide weir. The proposed freshwater alternatives feature an 80 ft wide weir which does help to alleviate the problem compared to the existing 50 ft wide weir; however, the 80 ft weir is still not large enough to convey increased rate of flow resulting from the improvements in the Interstate 5 Bridge and Coast Highway Culvert/Bridge. Consequently, this issue should be addressed as part of the implementation process

associated with future widening of channel connections and/or future restoration of the Lagoon. It is envisioned that this would include further analysis to determine if flooding of property and infrastructure would actually occur as well as the development of mitigation measures to reduce such flooding to levels of insignificance. For example, the weir could be widened to convey the increased rate of flow resulting from the improvements in the Interstate 5 Bridge and Coast Highway Culvert/Bridge. Alternatively, the berm surrounding the St. Malo community could be raised to reduce the risk to property from any increased flooding.

The I-5 North Coast Corridor (NCC) Project will likely be implemented before the Buena Vista Lagoon Restoration Project. In order to keep flood levels the same as the existing conditions, the channel under the new I-5 Bridge could be backfilled to match the geometry of the existing I-5 bridge until the Lagoon is restored in the future and/or until the downstream flood impacts are addressed.

5. The existing bridges in the Buena Vista Lagoon are not currently exposed to marine conditions. Restoring the Lagoon to wetlands may involve tidal exchange that allows ocean salt water to enter the Lagoon. Therefore, bridges and other structures in the restored Lagoon should be designed to withstand marine conditions, including the effect of salt water corrosion to the structural members.

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