North Coast Corridor

PUBLIC WORKS PLAN/TRANSPORTATION AND RESOURCE ENHANCEMENT PROGRAM

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Appendix K

Lagoon Optimization Studies

(Batiquitos, Buena Vista, San Elijo)

BATIQUITOS LAGOON BRIDGE OPTIMIZATION STUDY

Final Report

Interstate 5 North Coast Corridor Project
SAN DIEGO COUNTY, CALIFORNIA
DISTRICT 11-SD-5 (PM R28.4/R55.4)
EA 235800 (P ID 11-000-0159)

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Batiquitos Lagoon
Bridge Optimization Study

Final Report

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EXECUTIVE SUMMARY

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 Interstate 5 (I-5) North Coast Corridor (NCC) Project. Implementation of this project will require work within the major coastal lagoons of Northern San Diego County. The project will include new bridge structures across most of the lagoons, including Batiquitos Lagoon. The objective of this study is to evaluate a range of channel widths and depths under the I-5 and Railroad (RR) bridges. These evaluations will be used to determine bridge length and to identify a combination of channel widths and depths that provides the most favorable conditions for conveyance of tides and stormflows throughout the lagoon.

The TABS2 numerical modeling system, including the RMA-2 hydrodynamic model and the RMA-4 water quality model, was used for this study. A finite element numerical model grid was created based on a 2008 bathymetry survey of the lagoon. The RMA-2 model was calibrated and verified with tidal elevations recorded in the lagoon in July 2008. The calibrated and verified numerical model was then used in the channel dimensions optimization modeling. The RMA-4 model was used in this study to predict the residence time. The dispersion coefficients used in the RMA-4 model are based on modeling calibrations performed for other similar lagoons, as no data are available from Batiquitos Lagoon for calibration.

The selection of optimum channel widths and depths for bridge lengths was based on a sensitivity analysis conducted for each bridge crossing under: 1) typical dry weather tidal fluctuations and 2) extreme stormflow conditions (combined 100-year storm and 100-year water levels). Tidal range was used as the primary indicator for benefits to the wetland ecosystem. Extreme flood elevations were used to evaluate the high water surface elevations in the lagoon in comparison with bridge soffit elevations, although potential flooding of adjacent areas is not currently an issue at Batiquitos Lagoon. Using these indicators, the optimum channel width and depth at each bridge were identified as the point at which tidal range and flood conveyance are most favorable and further increases in channel width and depth result in only minimal benefit. The tidal inlet under the Carlsbad Boulevard bridges was originally sized and designed to achieve a stable tidal inlet as part of the Batiquitos Lagoon Restoration Project. The tidal inlet has been performing well since construction in 1995; therefore, no further optimization is required for that channel.

Table ES-1 presents the existing and optimum channel widths and depths for the I-5 and RR Bridges.

Table ES-1 Summary of Existing and Optimized Channel Dimensions

Infrastructure	Recommended Based on Optimization			Design Condition		
	Channel Invert (ft)		Bottom Width (ft)	Channel Invert (ft)		Bottom Width (ft)
	NGVD	MLLW		NGVD	MLLW	
Inlet	-8.0	-5.7	96	-8.0	-5.7	96
RR	-7.0	-4.7	202	-7.0	-4.7	162
I-5	-7.0	-4.7	134	-7.0	-4.7	66

Key findings from the optimization modeling study are summarized below:

- Dredging of the lagoon and channels under the bridges is an effective way to increase the tidal range and reduce tidal velocities under the bridges. Simply dredging the lagoon to its design condition will increase the tidal range by 0.4 feet in the Central Basin and by 0.7 feet in the East Basin, and will reduce the tidal velocity by more than 0.5 feet per second (fps). The tidal range will increase by an additional 0.17 feet in the Central Basin and by 0.21 feet in the East Basin with the optimized channel dimensions under both the I-5 and RR Bridges.

- With the optimized channel dimensions, the backwater effect created by the I-5 Bridge will be reduced and the flood elevation in the East Basin will be lowered. However, this will simply shift the backwater effect to downstream of the I-5 Bridge, resulting in an increase in flood elevations in the Central and West Basins compared to those under existing conditions.

- Tidal velocities at the bridge crossings, which are responsible for scour holes on both sides of the I-5 Bridge, will be reduced with the optimized channel dimensions. Reduced tidal flow velocities should significantly reduce the scour depth on both sides of the I-5 Bridge. Stormflow velocities will also be lowered at both the I-5 and RR Bridges; however, they will be slightly higher at the tidal inlet with channel optimizations. Fluvial sediment transport in the East Basin under the optimized condition should be slightly improved compared to existing conditions due to reduced backwater effects and the shortened flood travel time through the East Basin.

- Residence time is relatively short for Batiquitos Lagoon. In the West Basin the residence time is approximately 0.5 days, gradually increasing to approximately 1.5 days in the Central Basin and to approximately 5.5 days in the East Basin. A residence time of less than one week is considered relatively good for an estuary wetland system. While the tidal circulation in Batiquitos Lagoon is good, it can be further enhanced with maintenance dredging.

- The tidal inundation frequency curve under the optimized condition is very similar to that under existing conditions. The vertical range of the intertidal habitats would increase

slightly under the optimized channel dimensions condition. For Batiquitos Lagoon, the primary gain of intertidal habitat area will be mudflat.

- In the year 2100, with projected sea level rise (SLR), channels under both the existing and optimized I-5 and RR Bridges would pass the 100-year flood with more than 3-feet of freeboard. However, the soffit of the Carlsbad Boulevard Bridges will be below the 100-year flood water level. Flood velocities under the SLR scenario at all three bridge crossings will be lower than those under existing conditions.

TABLE OF CONTENTS

EXECUTIVE SUMMARY ..... **i**
1.0 INTRODUCTION **1**
1.1 Scope of Work 2
1.2 Modeling Bathymetry Conditions 2
2.0 NUMERICAL MODEL SETUP **5**
2.1 Model Selection and Description 5
2.2 Model Setup 7
2.2.1 Model Area 7
2.2.2 Bathymetry 8
2.2.3 Finite Element Mesh 9
2.2.4 Boundary Conditions 10
2.3 RMA-2 Model Calibration and Verification 14
2.3.1 Model Setup for Calibration 14
2.3.2 Calibration and Verification Results 16
3.0 ANALYSES TO ACHIEVE OPTIMAL TIDAL RANGE **18**
3.1 I-5 Channel Dimensions Optimization Results 20
3.2 RR Dimensions Optimization Results 23
3.3 Results of Combined I-5 and RR Dimensions Optimization 25
4.0 ANALYSES TO ACHIEVE OPTIMAL FLOOD CONVEYANCE **26**
4.1 I-5 Channel Dimensions Optimization Results 28
4.2 RR Dimensions Optimization Results 29
4.3 Results of Combined Channel Dimensions Optimization for I-5 and RR Bridges 30
4.4 Hydrodynamic Modeling Results of the 50-Year Storm Event 31
5.0 SUMMARY OF EXISTING AND OPTIMIZED CHANNEL DIMENSIONS UNDER BRIDGES **33**
5.1 Carlsbad Boulevard Bridges 33
5.2 Railroad Bridge 35
5.3 I-5 Bridge 37
5.4 Summary of Channel Dimensions 41
6.0 ANALYSES of VELOCITY AND SEDIMENTATION **42**
6.1 Analyses of Tidal Velocity Under Bridges 42
6.2 Analyses of Extreme Flood Velocities Under Bridges 43
6.3 Analyses of Sedimentation 45
6.3.1 Dry Weather Sedimentation 45
6.3.2 Extreme Storm Event Sedimentation 45
7.0 RESIDENCE TIME ANALYSES **47**
7.1 Methodology 47
7.2 Boundary Conditions 48
7.2.1 Hydraulic Input 48
7.2.2 Concentration Input 48
7.3 Residence Time Results 49
8.0 TIDAL INUNDATION FREQUENCY ANALYSES **50**
9.0 HYDRAULIC EFFECTS OF SEA LEVEL RISE **53**
10.0 FINDINGS AND RECOMMENDATIONS **56**
11.0 REFERENCES **58**

LIST OF FIGURES

Figure 1-1: Project Location Map 1
Figure 1-2: Central Basin Dredging Area 4
Figure 2-1: TABS2 Schematic 6
Figure 2-2: RMA-2 Modeling Area and Grid 8
Figure 2-3: Modeling Grid and Bathymetry of the Existing Shoaled Lagoon 9
Figure 2-4: Bathymetry of the Dredged Lagoon 10
Figure 2-5: TEA Modeling Tidal Series 12
Figure 2-6: 100-Year and 50-Year Hydrographs for San Marcos Creek 13
Figure 2-7: 100-Year and 50-Year Hydrographs for Encinitas Creek 13
Figure 2-8: Gage Locations with Recorded Tides and for Model Calibration 14
Figure 2-9: RMA-2 Model Calibration and Verification Results in the West Basin 16
Figure 2-10: RMA-2 Model Calibration and Verification Results in the Central Basin 17
Figure 2-11: RMA-2 Model Calibration and Verification Results in the East Basin 17
Figure 3-1: Spring High Tide Series for Tidal Optimization Modeling 18
Figure 3-2: Virtual Gage Locations for Tidal Range Comparison 19
Figure 3-3: Virtual Gage Locations for Tidal Range Calculations 20
Figure 3-4: I-5 Optimization Results with Different Channel Widths 21
Figure 3-5: I-5 Optimization Results with Different Channel Depths 22
Figure 3-6: RR Optimization Results with Different Channel Widths 23
Figure 3-7: RR Optimization Results with Different Channel Invert Elevations 24
Figure 4-1: Spring High Tidal Series for Flood Optimization Modeling 27
Figure 4-2: Virtual Gage Locations for Plotting Surface Water Profiles 27
Figure 4-3: Comparison of 100-Year Surface Profile for Different Lagoon Sedimentation Conditions 28
Figure 4-4: 100-Year Surface Profiles Under Different I-5 Channel Widths 29
Figure 4-5: 100-Year Surface Profiles Under Different RR Channel Widths 30
Figure 4-6: 100-Year Surface Profiles for Combined Channel Optimization Under I-5 and RR Bridges 31
Figure 4-7: 50-Year Water Surface Profiles 32
Figure 5-1: Image of Carlsbad Boulevard and Railroad Bridges (source: California Coastal Records Project, 2012) 33
Figure 5-2: East Carlsbad Boulevard Bridge As-Built Drawing (Looking from Lagoon to Ocean) 34
Figure 5-3: West Carlsbad Boulevard Bridge As-Built Drawing (Looking from Lagoon to Ocean) 34
Figure 5-4: Channel Cross Section Under East Carlsbad Boulevard Bridge 35
Figure 5-5: Image of the Existing Railroad Bridge 36
Figure 5-6: Channel Cross-Section Under the Railroad Bridge 37
Figure 5-7: Image of Existing I-5 Bridge 38
Figure 5-8: Channel Cross-Section Under I-5 Bridge 39
Figure 5-9: Proposed I-5 Bridge Exhibit (Looking from Lagoon to Ocean) 40
Figure 6-1: 100-Year Velocity Contours for Dredged Lagoon and Existing Bridge Dimension Condition 46
Figure 6-2: 100-Year Velocity Contours for Dredged Lagoon and Optimized Bridge Dimension Condition 46
Figure 7-1: Example of a Residence Time Plot 48
Figure 7-2: Gage Locations for Residence Time Calculations 49

Figure 8-1: Inundation Frequency for Shoaled Existing Condition....50
Figure 8-2: Inundation Frequency for the Dredged Existing Condition....51
Figure 8-3: Inundation Frequency for the Shoaled Optimized Condition....51
Figure 8-4: Inundation Frequency for the Dredged Optimized Condition....52
Figure 9-1: Spring High Tidal Series for Year 2100....53
Figure 9-2: 100-Year Surface Profile Comparison with Sea Level Rise....54

LIST OF TABLES

Table 1-1: Shoal Volume Estimates....3
Table 1-2: Channel Dimensions Shown on Record Drawings....3
Table 2-1 Datum Convertion Table at La Jolla (Based on 1983-3001 Tidal Epoch)....7
Table 2-2: Recorded Water Levels at La Jolla (1983-2001 Tidal Epoch)....11
Table 2-3: Setup Values For Model Calibration....15
Table 3-1: Comparison of Tidal Ranges (ft) in Each Basin....20
Table 3-2: Summary of I-5 Optimization Results....22
Table 3-3: Summary of RR Optimization Results....25
Table 3-4: Tidal Range (ft) in the Central and East Basins....25
Table 4-1: Summary of 100-Year Flood Levels in Each Basin....31
Table 5-1: Summary of Existing and Optimized Channel Dimensions....41
Table 6-1: Tidal Velocity (fps) at Bridge Crossings During the Dry Season....43
Table 6-2: 100-Year Peak Flood Velocity (fps) at Bridge Crossings....44
Table 6-3: 50-Year Peak Flood Velocity (fps) at Bridge Crossings....44
Table 6-4: Duration (Hour) of Stormflow Drainage Under a 100-Year Storm....46
Table 7-1: Summary of Residence Time (Days)....49
Table 9-1: Summary of Bridge Soffit and 100-Year Surface Water Elevations....55
Table 9-2: 100-Year Peak Flood Velocity (fps) at Bridge Crossings....55

LIST OF ACRONYMS

Caltrans	California Department of Transportation
CB	Central Basin (figure legends only)
cfs	Cubic Feet per Second
CSCC	California State Coastal Conservancy
EB	East Basin (figure legends only)
FEMA	Federal Emergency Management Agency
fps	Foot per Second
ft	Feet
GIS	Geologic Information System
GPS	Geological Position System
I-5	Interstate 5
ID	Identification
LOSSAN	Los Angeles – San Diego – San Luis Obispo Rail Corridor Agency
MHHW	Mean Higher High Water
MHW	Mean High Water
MLLW	Mean Lower Low Water
MLW	Mean Low Water
M&N	Moffatt & Nichol (Company)
MSL	Mean Sea Level
MTL	Mean Tidal Level
NAD	North American Datum
NAVD	North American Vertical Datum
NCC	North Coast Corridor
NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
RMA-2	Resource Management Associates (2D numerical model for surface flow)
RMA-4	Resource Management Associates (2D water quality model)
RR	Railroad
SANDAG	San Diego Association of Governments
SED2D	2 Dimensional Sedimentation Model
SLR	Sea Level Rise
TABS2	The Open Channel Flow and Sedimentation (Numerical Modeling System)
TEA	Tidal Epoch Analysis
USACE	United States Army Corps of Engineers
WB	West Basin (figure legends only)
WRA	Wetland Research Associates, Inc. (Company)
WSE	Water Surface Elevation
2D	Two-dimensional

1.0 INTRODUCTION

Batiquitos Lagoon, located approximately 30 miles north of the City of San Diego and shown in Figure 1-1, is a coastal wetland restored in 1995 with significant biological and ecological resources.

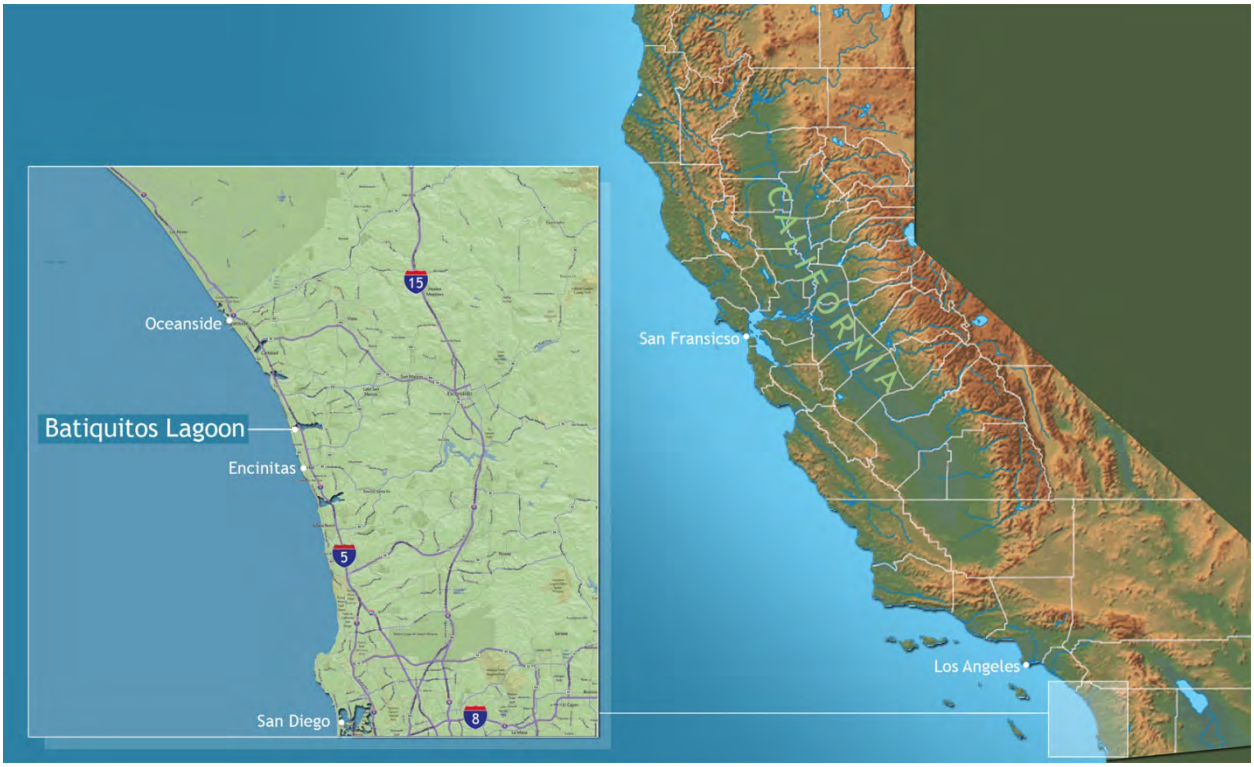

Figure 1-1: Project Location Map

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 Interstate 5 (I-5) North Coast Corridor (NCC) Project. Implementation of this project will require work within the major coastal lagoons of Northern San Diego County. The project will include new bridge structures across most of the lagoons, including Batiquitos Lagoon. The objective of this study was to evaluate a range of channel widths and depths under the I-5 and railroad (RR) bridges, which will be used to determine the bridge lengths, and identify a combination of channel widths and depths that provides the most favorable conditions for tidal range and flood conveyance throughout the lagoon. Channel dimensions, including both width and depth under the I-5 and RR Bridges, are optimized for both tidal and fluvial flows. The tidal inlet under the Carlsbad Boulevard East and West Bridges was sized and designed to achieve a stable inlet as part of the Batiquitos Lagoon Restoration Project. A stable tidal inlet is self-sustaining and remains open under most conditions. The tidal inlet has been performing well since construction in 1995. Therefore, no optimization is required for the tidal inlet.

However, the existing inlet is relatively narrow, resulting in tidal muting of 0.95 feet between the ocean and the east end of the inlet under existing conditions.

1.1 Scope of Work

The scope of work for this optimization study includes:

1. Gathering available data for model setup, calibration, verification, and alternative modeling runs.
2. Developing two dimensional numerical model (RMA-2) grids for both the post-construction condition and the existing condition (lagoon shoaled condition).
3. Calibrating and verifying the RMA-2 model by matching model-predicted tidal elevations to those measured in the lagoon in 2008 as part of long-term monitoring (Merkel & Associates 2009).
4. Performing RMA-2 hydrodynamic modeling runs to achieve the optimal tidal range in the lagoon and fluvial flood (stormflow) conveyance through the lagoon.
5. Performing RMA-4 water quality model runs to determine residence times in the lagoon.
6. Performing velocity and sedimentation pattern analyses based on RMA-2 hydrodynamic modeling results.
7. Preparing Draft and Final Reports of the methods, analyses, and results.

1.2 Modeling Bathymetry Conditions

Coastal lagoons with tidal connections to the ocean experience shoaling in the interior of the inlet as a result of tidal exchange between the ocean and the lagoon. Batiquitos Lagoon experiences such shoaling. Flood shoals have been gradually building up, first in the West Basin and then expanding into the Central Basin. To achieve the desired tidal exchange and maintain tidal inlet stability, the flood shoals have been partially removed on several occasions via maintenance dredging. Maintenance dredging will continue into the future. The bathymetric condition of Batiquitos lagoon has evolved between the dredged and shoaled lagoon conditions ever since. Therefore, for this study two bathymetric conditions were modeled: (1) the existing shoaled condition; and (2) the post-construction (dredged) lagoon condition. These two bathymetric conditions were selected to represent the two extreme tidal prism conditions.

(1) <u>Shoaled Lagoon Condition</u> - The bathymetry of the shoaled condition was based on the 2008 survey. As flood conveyance will be limited by shoals in the lagoon and channels under infrastructure crossings, this condition was the control condition in optimizing the channel widths and depths for lowering flood water levels in the lagoon.

(2) <u>Post-Construction (Dredged) Lagoon Condition</u> (Post-construction condition with material consolidation assumed to have occurred in the Central Basin) – Under this condition, the West and East Basins were assumed to be dredged to the post-construction condition. During original construction, the Central Basin was dredged deeper to create a disposal pit. The disposal pit was backfilled with silt/clay/fine sand

and topped with a 2-foot coarse sand cover to a finished elevation of -4.56 ft NGVD. According to the results of hydroprobing conducted in 2008 (Merkel & Associates 2009), an average of 2.2 feet of subsidence occurred in the disposal pit. Therefore, it is proposed to assume a lower shoal area in the Central Basin of -6.76 feet NGVD (-2.2 feet below the as-built elevation of -4.56 feet) for the modeling. The proposed shoal area is outlined in yellow in Figure 1-2. By lowering the shoal down to -6.76 feet, additional sediment storage will be created and the dredging interval may be extended. Based on the most recent survey conducted in 2008, the rest of the Central Basin is deeper than -4.56' NGVD and ranges from -5 to -7 feet due to consolidation. These depths were retained for modeling purposes. Table 1-1 shows shoals needing to be removed to return the lagoon to as-built conditions (Moffatt & Nichol, or M&N, 1997) except for the Central Basin. In the Central Basin the shoal volume estimate was limited to the dredging area shown in Figure 2-4, and the dredging area is assumed to be dredged down to -6.76 feet NGVD, as discussed earlier. For the West and East Basins, the shoal volume estimate was for the entire basin. All channels under infrastructure crossings were assumed to be dredged to dimensions shown on record design drawings. Table 1-2 shows the channel width and invert elevation for each bridge referenced to the NGVD vertical datum. Under this proposed dredged condition, the lagoon will have the largest tidal prism of any other scenario. Therefore, it will be the upper bound prism condition for the bridge optimization study to achieve the maximum tidal range.

Table 1-1: Shoal Volume Estimates

Basin	**West Basin**	**Central Basin (limited to the dredging area)**	**East Basin**
Volume (Cubic Yard)	59,000	125,000	464,000

Table 1-2: Channel Dimensions Shown on Record Drawings

West Carlsbad		**East Carlsbad**		**LOSSAN RR**		**I-5**	
Channel Invert (ft)	**Channel Width (ft)**	**Channel Invert (ft)**	**Channel Width (ft)**	**Channel Invert (ft)**	**Channel Width (ft)**	**Channel Invert (ft)**	**Channel Width (ft)**
-8	96	-8	109	-7	162	-7	66

Figure 1-2: Central Basin Dredging Area

2.0 NUMERICAL MODEL SETUP

This report section presents the model selection, description, set-up, and calibration/verification.

2.1 Model Selection and Description

The numerical modeling system used in this study is summarized herein. The TABS2 (McAnally and Thomas, 1985) modeling system was applied to this project because it realistically represents dynamic tidal and stormflow conditions to yield results most applicable to this evaluation.. TABS2 was developed by the U.S. Army Corps of Engineers (USACE), and consists of the following components:

1. Two-dimensional, vertically-averaged finite element hydrodynamics model (RMA-2);
2. Pollutant transport/water quality model (RMA-4); and
3. The sediment transport model (SED2D).

TABS2 is a collection of generalized computer programs and pre- and post-processor utility codes integrated into a numerical modeling system for studying 2D depth-averaged hydrodynamics, transport and sedimentation problems in rivers, reservoirs, bays, and estuaries. The finite element method provides a means of obtaining an approximate solution to a system of governing equations by dividing the area of interest into smaller sub-areas called elements. Time-varying partial differential equations are transformed into finite element form and then solved in a global matrix system for the modeled area of interest. The solution is smooth across each element and continuous over the computational area. This modeling system is capable of simulating tidal wetting and drying of marsh and intertidal areas of the estuarine system.

A schematic representation of the system is shown below. TABS2 can be used either as a stand-alone solution technique or as a step in the hybrid modeling approach. RMA-2 calculates water surface elevations and current patterns which are input to the pollutant transport and sediment transport models. The three models listed above are solved by the finite element method using Galerkin weighted residuals.

The hydraulic model RMA-2 and water quality model RMA-4 were applied to this study. The lagoon sedimentation models (SED2D) was not applied to this study due to limited input data, and the opportunity to use stormflow velocity as a surrogate for potential sedimentation..

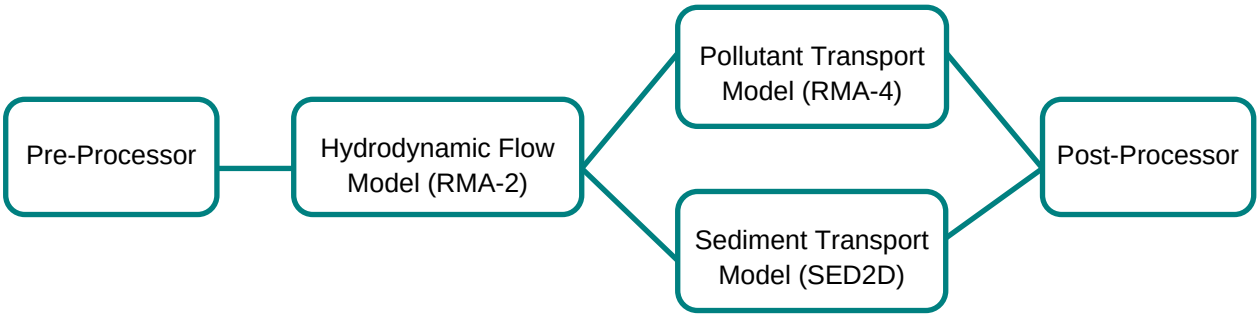

Figure 2-1: TABS2 Schematic

The hydrodynamic model simulates 2D flow in rivers and estuaries by solving the depth-averaged Navier Stokes equations for flow velocity and water depth. The equations account for friction losses, eddy viscosity, Coriolis forces and surface wind stresses. The general governing equations are:

$$h\frac{\partial u}{\partial t}+uh\frac{\partial u}{\partial x}+vh\frac{\partial u}{\partial y}+gh\frac{\partial a}{\partial x}+gh\frac{\partial h}{\partial x}-h\frac{\varepsilon_{xx}}{\rho}\frac{\partial^2 u}{\partial x^2}-h\frac{\varepsilon_{xy}}{\rho}\frac{\partial^2 u}{\partial y^2}+S_{f_x}+\tau_x=0$$

$$h\frac{\partial v}{\partial t}+uh\frac{\partial v}{\partial x}+vh\frac{\partial v}{\partial y}+gh\frac{\partial a}{\partial y}+gh\frac{\partial h}{\partial y}-h\frac{\varepsilon_{yx}}{\rho}\frac{\partial^2 v}{\partial x^2}-h\frac{\varepsilon_{yy}}{\rho}\frac{\partial^2 v}{\partial y^2}+S_{f_y}+\tau_y=0$$

$$\frac{\partial h}{\partial t}+\frac{\partial(hu)}{\partial x}+\frac{\partial(hv)}{\partial y}=0$$

where:

u,v = x and y velocity components, respectively
t = time
h = water depth
a = bottom elevation
Sf_x = bottom friction loss term in x-direction
Sf_y = bottom friction loss term in y-direction
τ_x = wind and Coriolis stresses in x-direction
τ_y = wind and Coriolis stresses in y-direction
ε_{xx} = normal eddy viscosity in the x-direction on x-axis plane
ε_{xy} = tangential eddy viscosity in the x-direction on y-axis plane
ε_{yx} = tangential eddy viscosity in the y-direction on x-axis plane
ε_{yy} = normal eddy viscosity in the y-direction on y-axis plane

Wind stress is computed using the following formula:

$$\tau_S = 3.8\cdot 10^{-6} W^2$$

Where:

τ_s = wind stress (lb/ft^2) on the water surface, and

W = the wind speed in miles per hour at 10 meters (33 feet) above the water surface.

2.2 Model Setup

The setup for the hydrodynamic model included determination of the model area, bathymetry, channel dimensions, mesh selection, and boundary conditions. For this study, a new RMA-2 model was created for this site. The RMA-2 model setup includes all areas of interest and potential tidal influence, and contains the most current topographic and bathymetry data.

The horizontal coordinate system for the modeling work is North American Datum (NAD) 83, California state plan zone 6, and the vertical datum is NGVD 1929, which is equivalent to Mean Sea Level (MSL) at that time. As sea level has risen since 1929, NGVD is lower than existing MSL by approximately 0.44 feet. The NGVD vertical datum was selected to maintain consistency with other lagoon bridge optimization studies. Both horizontal and vertical units are in feet. Table 2-1 shows conversions between different vertical datums based on the 1983 to 2001 tidal epoch for the La Jolla tidal station.

Table 2-1 Datum Convertion Table at La Jolla (Based on 1983-3001 Tidal Epoch)

Description	Elevation (ft, MLLW)	Elevation (ft, NGVD29)	Elevation (ft, MSL)	Elevation (ft, NAVD88)
Extreme High Water (11/13/1997)	7.65	5.36	4.92	7.47
Mean Higher High Water (MHHW)	5.33	3.04	2.60	5.15
Mean High Water (MHW)	4.60	2.31	1.87	4.42
Mean Tidal Level (MTL)	2.75	0.46	0.02	2.57
Mean Sea Level (MSL)	2.73	0.44	0.00	2.55
National Geodetic Vertical Datum 1929 (NGVD)	2.29	0.00	-0.44	2.11
Mean Low Water (MLW)	0.90	-1.39	-1.83	0.72
North America Vertical Datum 1988 (NAVD)	0.18	-2.11	-2.55	0.00
Mean Lower Low Water (MLLW)	0.00	-2.29	-2.73	-0.18
Extreme Low Water (12/17/33)	-2.87	-5.16	-5.60	-3.05

2.2.1 Model Area

The numerical model covers the nearshore ocean and the area below the +10 foot NGVD contour line of the site, as shown in Figure 2-2. The ocean boundary is approximately 1.5 miles offshore from the shoreline and the inlet location.

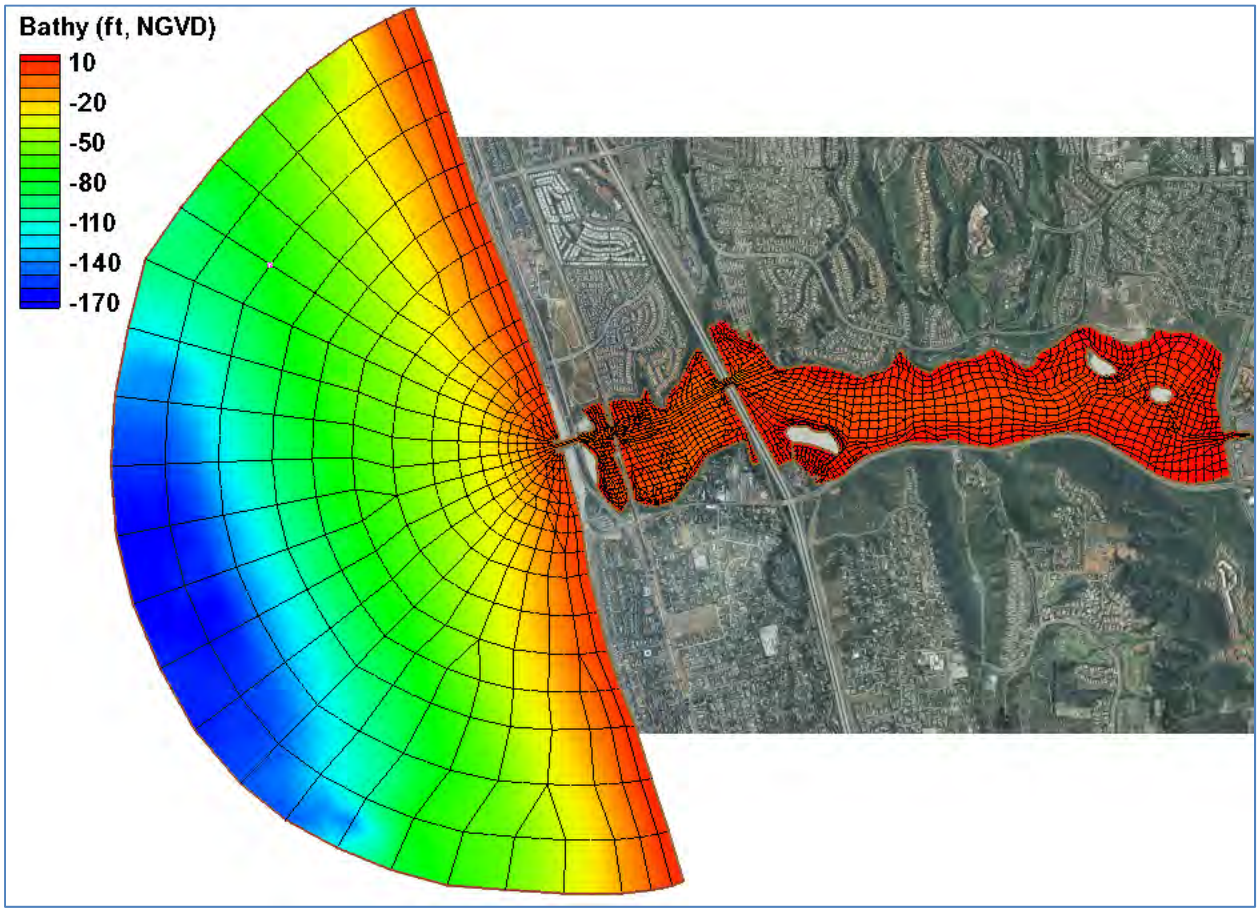

Figure 2-2: RMA-2 Modeling Area and Grid

2.2.2 Bathymetry

The ocean bathymetry used in the model is downloaded from the National Oceanic and Atmospheric Administration (NOAA) website (2005);it was originally compiled by the Pacific Marine Environmental Laboratory Center of NOAA for Tsunami Inundation Mapping Efforts. The bathymetry of the lagoon area for the existing shoaled condition was based on a GIS shape file provided by WRA, Inc. (2010). The GIS shapefile was based on three data sources: 1) a 2008 lagoon bathymetry survey (Merkel & Associates 2009) for areas below Mean High Water (MHW); 2) point elevations collected by WRA, Inc. (2010) with an Auto Level and a hand held Trimble GPS unit for a 2-foot elevation band above the MHW for the East basin, and 3) a topographic survey file provided by City of Carlsbad (provided by Caltrans through WRA in 2011). Figure 2-3 shows the modeling grid and existing bathymetry of the lagoon. The channel width under each bridge crossing was narrowed to account for flow constriction by bridge piers/columns together with growth of marine organisms.

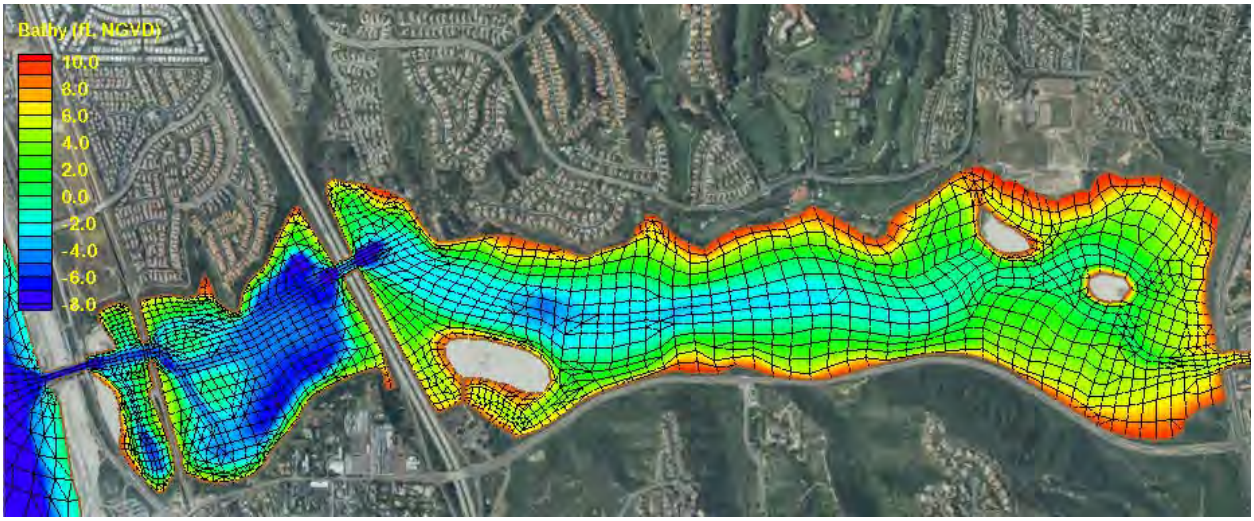

Figure 2-3: Modeling Grid and Bathymetry of the Existing Shoaled Lagoon

2.2.3 Finite Element Mesh

The RMA-2 modeling system requires that the estuarial system be represented by a network of nodal points and elements, points defined by coordinates in the horizontal plane and water depth, and areas made up by connecting these adjacent points, respectively. Nodes can be connected to form 1- and 2D elements, having from two to four nodes. The resulting nodal/element network is commonly called a finite element mesh and provides a computerized representation of the estuarial geometry and bathymetry.

It is noted that evaluations discussed herein correspond to 2D analyses. The two important aspects to consider when designing a finite element mesh are: (1) determining the level of detail necessary to adequately represent the estuary; and (2) determining the extent or coverage of the mesh. Accordingly, the bathymetric features of the estuary generally dictate the level of detail appropriate for each mesh. These concerns present trade-offs for the modeler to consider. Too much detail can lead the model to run slowly or even become unstable and “crash.” Too little detail renders the results less useful. For this project, a balance was achieved with a stable and efficient model that yields the level of detail required for the study. The model described in this section is numerically robust and capable of simulating tidal elevations, flows, and constituent transport with reasonable resolution.

There are several factors used to decide the aerial extent of each mesh. First, it is desirable to extend mesh open boundaries to areas which are sufficiently distant from the proposed areas of change so as to be unaffected by that change. Additionally, mesh boundaries must be located along sections where conditions can reasonably be measured and described to the model. Finally, mesh boundaries can be extended to an area where conditions have been previously collected to eliminate the need to interpolate between the boundary conditions from other locations.

The same finite element mesh is used for both the dredged and shoaled lagoon scenarios, but the bathymetry differs between the two conditions. The bathymetry for the shoaled existing condition is shown in Figure 2-3, and that for the dredged lagoon condition described in Section 1.2 is shown in Figure 2-4. The mesh contains an area of ocean sufficiently large to eliminate potential model boundary effects. The wetland portion of the mesh is bounded by the ocean and dry land is considered to be at the outermost extents of tidal influence. The entire modeling area, approximately 5.43 square miles, is represented as a finite element mesh consisting of 2,950 elements and 8,530 nodes.

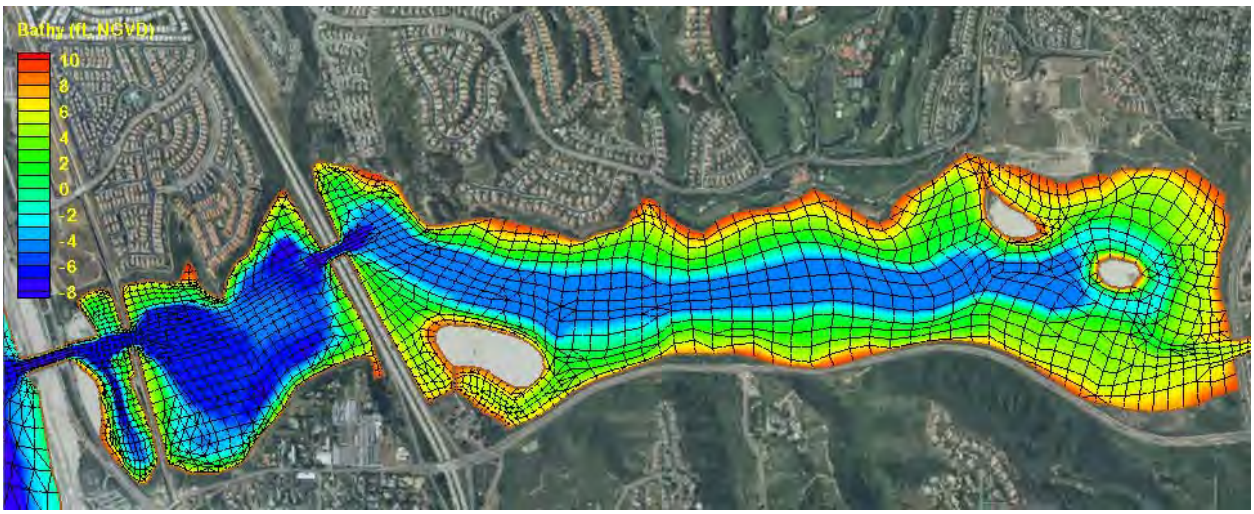

Figure 2-4: Bathymetry of the Dredged Lagoon

2.2.4 Boundary Conditions

Boundary conditions consist of the ocean driving tide and stormflows, both described below.

2.2.4.1 Ocean Tides

Since there are no tide stations at Carlsbad, the nearest La Jolla gage (NOAA Station ID: 9410230) was used to represent the ocean tide at the project site. As shown in Table 2-2, the diurnal tide range is approximately 5.33 feet MLLW to Mean Higher High Water (MHHW), and MSL is at +2.73 feet MLLW. Water level data records provide astronomical tides and other components including barometric pressure tide, wind setup, seiche, and the El Nino Southern Oscillation. Tidal variations can be resolved into a number of sinusoidal components having discrete periods. The longest significant periods, called tidal epochs, are approximately 19 years. In addition, seasonal variations in MSL can reach amplitudes of 0.5 feet in some areas. Superimposed on this cycle is a 4.4-year variation in the MSL that may increase the amplitude by as much as 0.25 feet. Water level gage records are typically analyzed over a tidal epoch to account for these variations and to obtain statistical water level information (e.g., MLLW and MHHW).

Table 2-2: Recorded Water Levels at La Jolla (1983-2001 Tidal Epoch)

Description	Elevation (feet, MLLW)	Elevation (feet, NGVD)
Extreme High Water (11/13/1997)	7.65	5.35
Mean Higher High Water (MHHW)	5.33	3.03
Mean High Water (MHW)	4.60	2.30
Mean Tidal Level (MTL)	2.75	0.46
Mean Sea Level (MSL)	2.73	0.44
National Geodetic Vertical Datum 1929 (NGVD)	2.30	0.00
Mean Low Water (MLW)	0.91	-1.39
North America Vertical Datum 1988 (NAVD)	0.19	-2.11
Mean Lower Low Water (MLLW)	0.00	-2.30
Extreme Low Water (12/17/33)	-2.87	-5.16

2.2.4.2 TEA Tidal Series

The tide series used for modeling was a representative period from June 7 to 21, 2011. Modeling long-term hydrologic conditions is typically done using a synthetic (artificially created) tide series that represents average spring tide conditions over the most recent 19-year tidal epoch, referred to as a Tidal Epoch Analysis (TEA) tide series. The benefit of using a statistical tide is that the long-term condition can be modeled over a shorter time period with less computation time.

Significant effort (beyond the scope of this study) is required to prepare a new TEA tide for this site. Therefore, a real tide series was used that matched average spring tide data available from National Oceanic and Atmospheric Administration (NOAA 2011).

Not using a statistical TEA tide for modeling does not create a serious information gap. To address this potential shortcoming, the modeler evaluated existing tide data from NOAA for San Diego at Scripps Pier (NOAA 2011). NOAA began publishing spring high and spring low tidal elevations of all tidal cycles in January of 2008. The modeler averaged the spring high and spring low tidal elevations of all tidal cycles from January of 2008 through July of 2011 (42 months), then examined the existing data to identify a real two-week tidal cycle that matched them. Tides during the period of June 7 through June 21, 2011 reached nearly the exact same spring high and spring low tidal elevations of NOAA's longer 42-month record. Also, the average tidal elevation of that June 7 through June 21, 2011 period compared with the average tidal elevation of the 19-year tidal epoch and was within 0.01 foot. Therefore, the modeler concluded that tides during the period of June 7 through June 21, 2011 sufficiently matched long-term tides at the site, and use of this record poses no implications for analyses. The modeling tide includes both spring and neap tidal ranges, as shown in Figure 2-5.

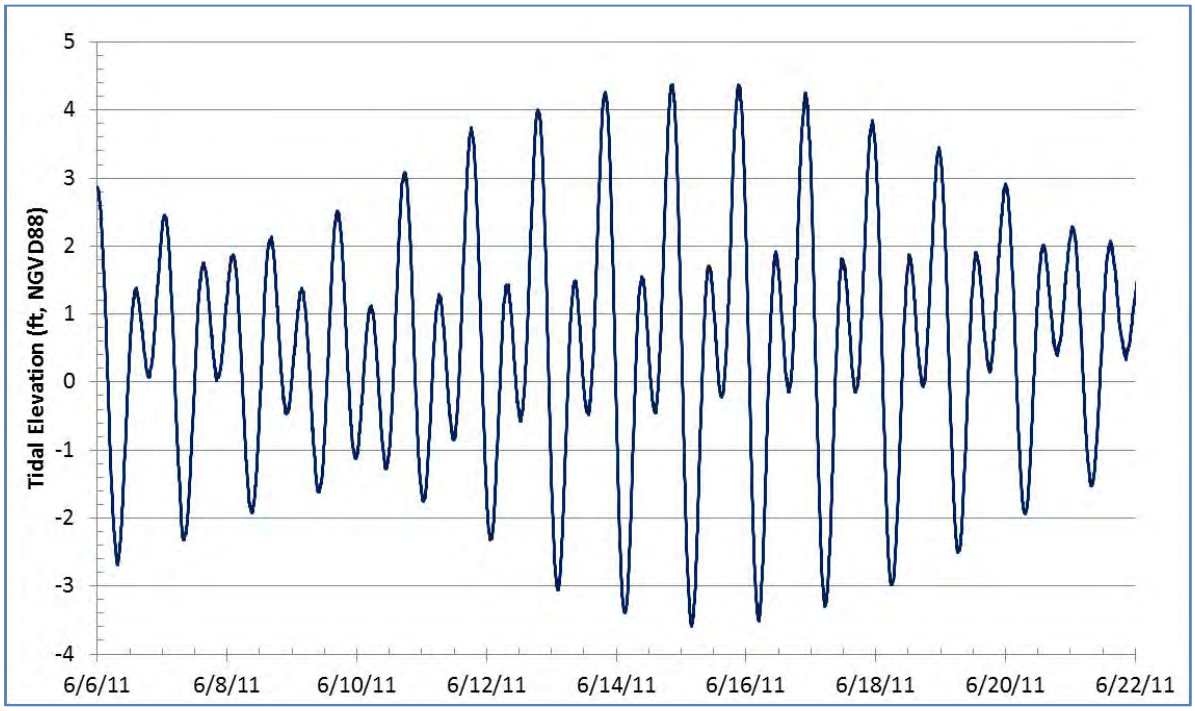

Figure 2-5: TEA Modeling Tidal Series

2.2.4.3 Flood Hydrographs

The watershed of Batiquitos Lagoon encompasses 52.3 square miles. The primary stream that drains into Batiquitos Lagoon is San Marcos Creek which begins in the mountains east of the lagoon and drains much of the watershed. A dam impounds San Marcos Creek about 5 miles upstream from its confluence with the lagoon and creates Lake San Marcos. The other primary stream in the watershed is Encinitas Creek which drains Green Valley and the Olivenhain Road area. Several other small creeks drain into the lagoon from its north and south shores. The 100-year and 50-year peak stormflow rates for San Marcos Creek are 12,050 and 6,707 cfs, respectively; and those for Encinitas Creek are 4,520 and 2,511 cfs, respectively (M&N, 1990). The hydrographs shown in Figure 2-6 and Figure 2-7 were based on the equations cited in Flood Plain Information (USACE 1971) and the Batiquitos Lagoon Watershed Sediment Control Plan prepared by the California State Coastal Conservancy (CSCC 1987).

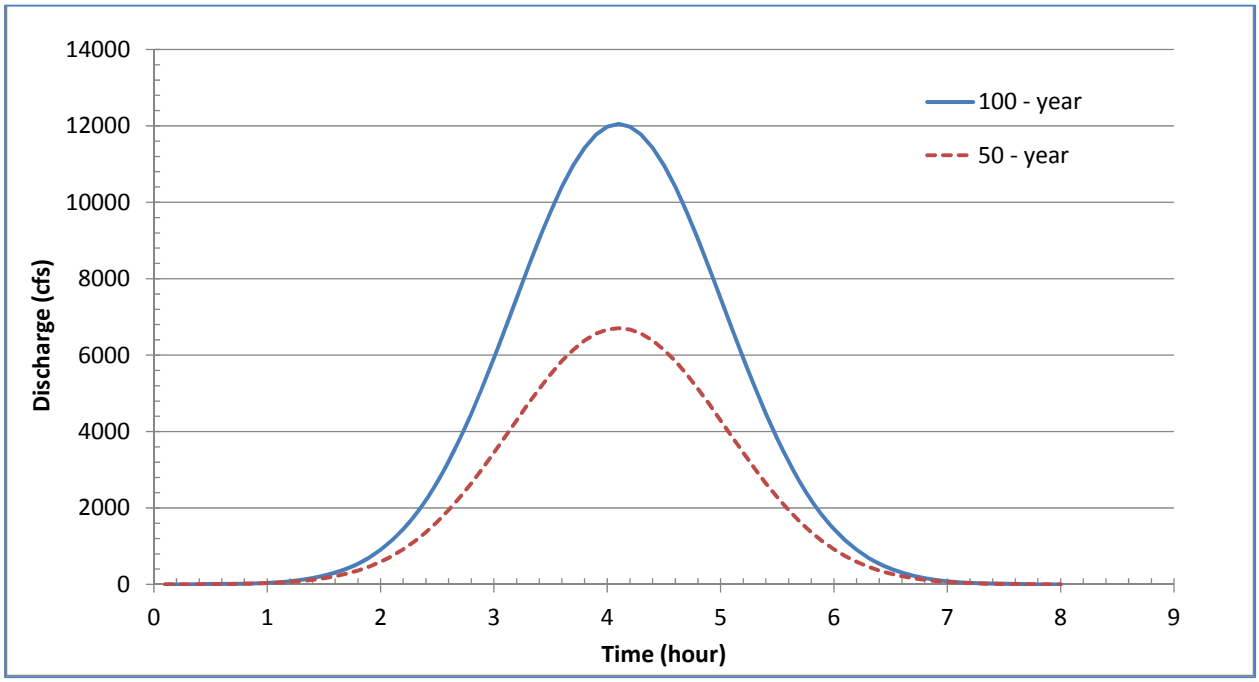

Figure 2-6: 100-Year and 50-Year Hydrographs for San Marcos Creek

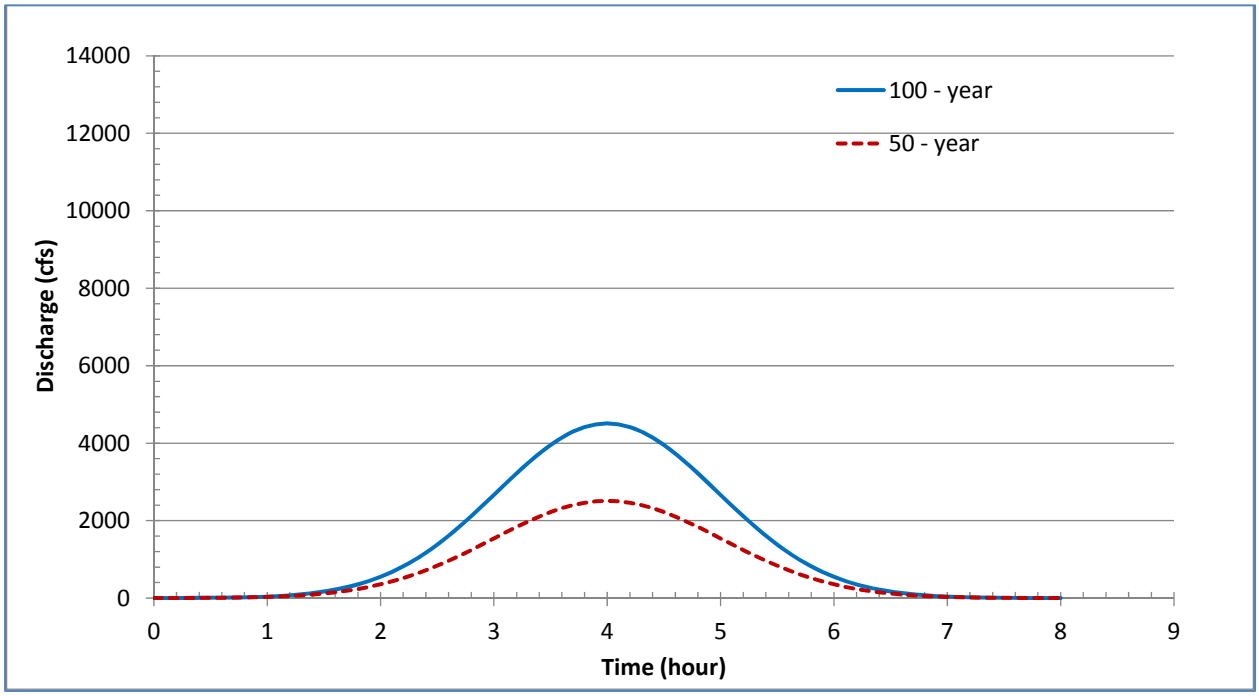

Figure 2-7: 100-Year and 50-Year Hydrographs for Encinitas Creek

2.3 RMA-2 Model Calibration and Verification

RMA-2 calibration involves matching model predictions with measured data by selecting appropriate variable input values (e.g., Manning's roughness coefficient - n, peclet numbers, and marsh porosity) to the model. A two-week period of measured tidal elevations in the lagoon that occurred very close in time period to the date of the lagoon bathymetry survey was selected for model calibration and verification. Tidal elevations were recorded by Merkel & Associates, Inc. (2009) at three gage locations shown in Figure 2-8. The two-week period covers both spring and neap tidal cycles. Instead of running the RMA-2 model separately for the spring and neap tidal cycles, the model was run continuously over the two-week period. Results for the first week served as model calibration and the second week results served as model verification.

Figure 2-8: Gage Locations with Recorded Tides and for Model Calibration

The tidal series used as the offshore model boundary input over the model calibration period was downloaded from the nearest La Jolla tide gage (NOAA Station ID: 9410230) as discussed in Section 2.2.4.1.

2.3.1 Model Setup for Calibration

The RMA-2 User's Manual recommends ranges of values for Manning's roughness coefficient (n) and eddy viscosity to be used in the model (USACE 2009). The value of Manning's roughness coefficient (n) is a function of the physics of the hydraulic system and represents the roughness of the channel bed. As discussed in Chaudhry (1993), values can range from 0.011 to 0.075 or higher for natural rivers and estuaries. Relatively high values (0.04 to 0.05) are specified for rough surfaces, such as channels with cobbles or large boulders. Mid-range values (0.03) represent clean and straight natural streams. Low values (0.013 to 0.02) are

specified for smooth surfaces, such as concrete, cement, wood, or gunite. The depth dependent method is used in assigning the Manning's roughness coefficient (n) for this analysis. The roughness coefficient is higher in areas with shallow water depths and lower for areas with deeper water.

The modeling grid size depends on and is limited by the Peclet number and eddy viscosity. The Peclet number is defined as,

$$\frac{\rho V \Delta X}{E_{ij}}$$

in which ρ, V, ΔX, and E_{ij} are the water density, velocity, grid size and eddy viscosity, respectively. In order for the solution to be stable, the Peclet number has to be less than 50. The Peclet number can be reduced by increasing the mesh density or by increasing the eddy viscosity. However, it is unrealistic and time-consuming to perform this modeling with a very fine grid. Eddy viscosity is another variable often specified in modeling. It represents the degree of turbulence in the flow. A higher value represents greater turbulence, while a low value suggests less turbulence. The modeling approach can either be based on use of the Peclet number or eddy viscosity. This modeling was based on specifying the Peclet number to maximize model stability and to minimize "crashing." Peclet numbers were adjusted until model results approximated field measurements. The resulting Peclet numbers for various areas are presented in Table 2-3.

Table 2-3: Setup Values For Model Calibration

Model Area	Peclet Number
Offshore Area	20
Nearshore Area	5
Tidal Inlet and Main Channels	5
Secondary Channels	5
Low Marsh	2
High Marsh	1
Lower Riparian Area	0.8
High Riparian Area	0.5

The time step is another important parameter in the modeling. Sensitivity tests were conducted and results showed that the RMA-2 model becomes unstable with increasing time steps, if tidal wetting and drying processes are considered in the model. Therefore, a relatively fine time step of 0.1 hour was used in order for the solution to be stable and to reflect the dynamic tidal series and flood flow hydrograph.

2.3.2 Calibration and Verification Results

Model calibration and verification were done over a two-week period for each basin from July 3 to 19, 2008. The first week of the model run serves as the model calibration and the second week of the model run serves as the model verification. Model predictions of tidal elevations were compared to measured tides at all three gage locations shown in Figure 2-8. The results are shown in Figure 2-9 through Figure 2-11. Tidal elevations simulated by the model correspond reasonably well with those measured in the field both in terms of tidal phase (timing) and range (elevation) for gages located in all three basins. The ocean tide is also included in the figures as reference. The tidal elevation differences between the ocean and those recorded in the lagoon especially during the low tide indicate tidal muting in the lagoon. The calibration and verification results indicate that the model can reasonably replicate (predict) the existing tidal conditions in all three basins of the lagoon as compared with measured values, and is, therefore, suitable for bridge optimization simulations for this study.

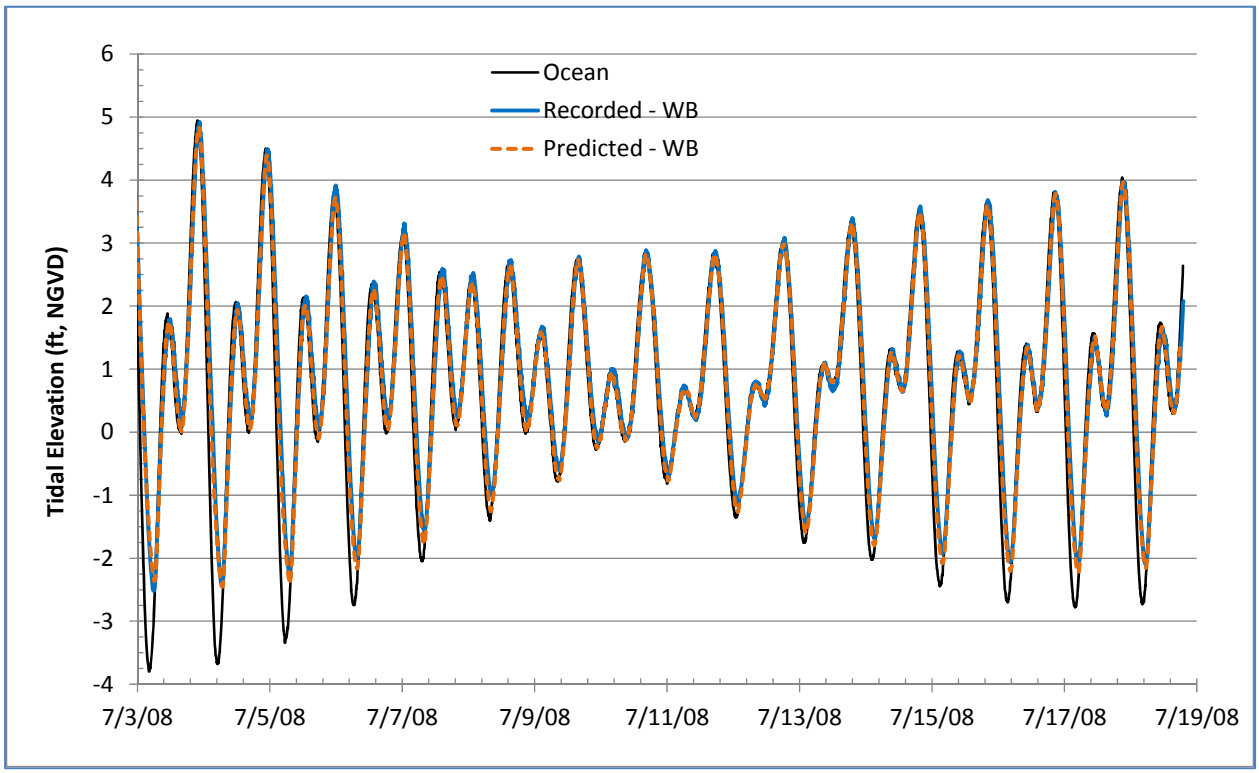

Figure 2-9: RMA-2 Model Calibration and Verification Results in the West Basin

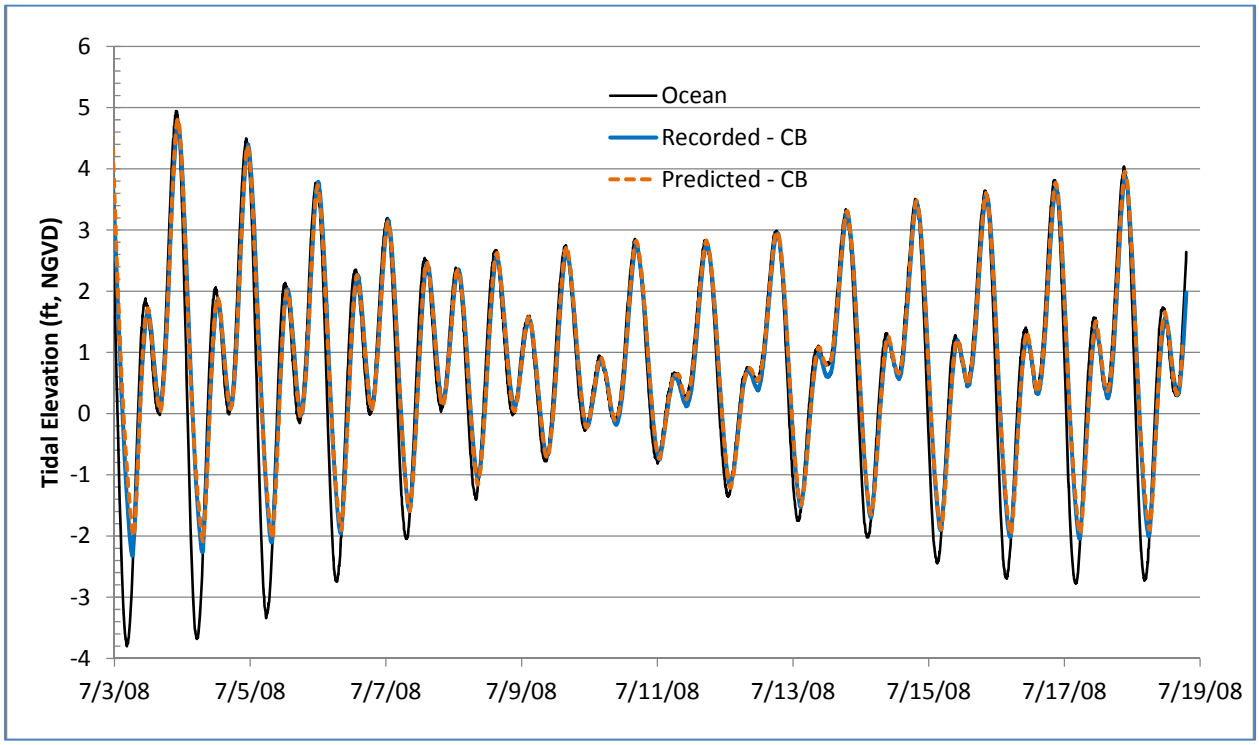

Figure 2-10: RMA-2 Model Calibration and Verification Results in the Central Basin

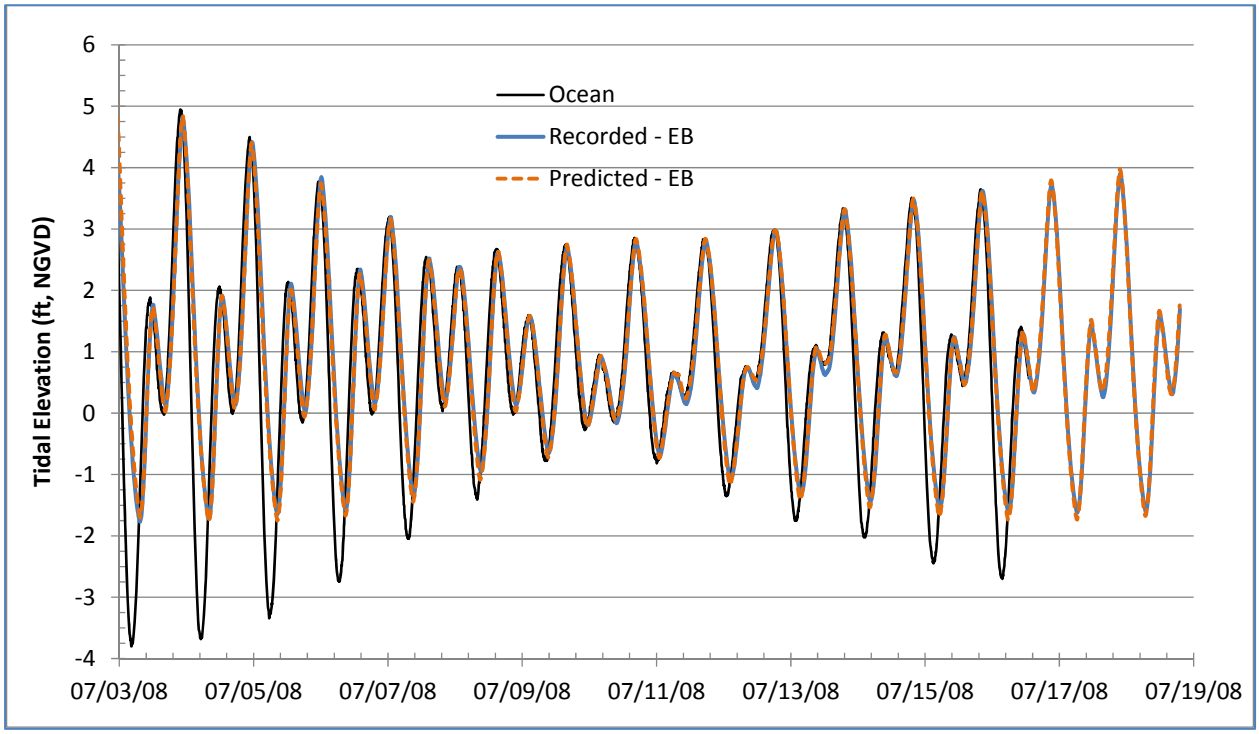

Figure 2-11: RMA-2 Model Calibration and Verification Results in the East Basin

3.0 ANALYSES TO ACHIEVE OPTIMAL TIDAL RANGE

The modeling parameters of roughness coefficients and Peclet numbers determined in model calibration and verification were assigned for the hydrodynamic modeling of Batiquitos Lagoon for both the shoaled and dredged lagoon bathymetry conditions. The two lagoon bathymetry conditions were described in Section 1.2.

The goal of this hydrodynamic modeling section is to determine the channel width and depth under the RR and I-5 Bridges required to achieve the optimal tidal range in both the Central and East Basins. The benefit of a larger tidal range is that it can support a broader vertical range of intertidal habitats.

The spring high tide has the largest tidal range and would experience the worst tidal muting if muting were to occur. Therefore, the spring high tidal series shown in Figure 3-1 is applied at the model offshore boundary in the tidal range optimization modeling. No storm flood flows were applied to the tidal modeling effort. Dry weather base flow has a negligible effect on the tidal range.

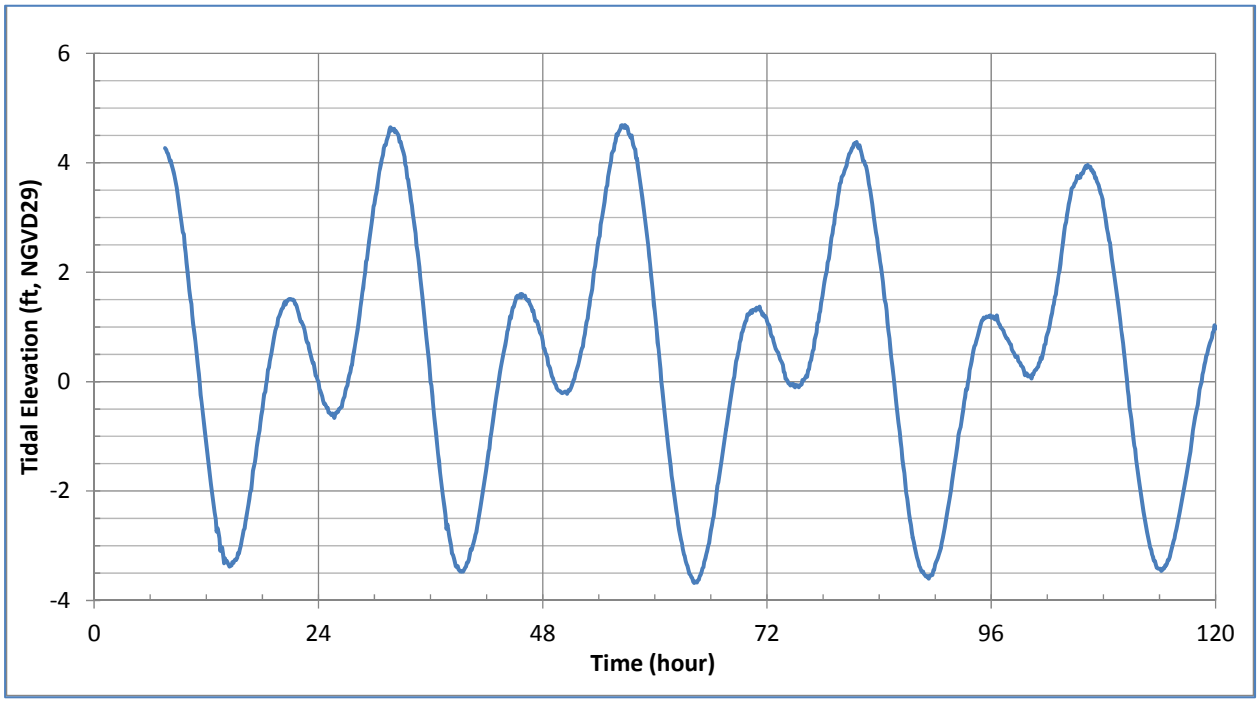

Figure 3-1: Spring High Tide Series for Tidal Optimization Modeling

A series of numerical modeling runs were performed to optimize channel dimensions under both RR and I-5 Bridges. The modeling approach was to begin upstream and work downstream. First, the channel dimensions under I-5 Bridge were optimized while artificially specifying that the channel under the RR Bridge as large enough to not pose a hydraulic constraint. Next, the channel dimensions under the RR Bridge were optimized with an artificially large channel cross-section under the I-5 Bridge. Then the model runs of nine combinations of different channel widths under the RR and I-5 Bridges were performed to determine the optimal channel

dimensions. Figure 3-2 shows locations where tidal ranges were calculated from the RMA-2 modeling results for comparison.

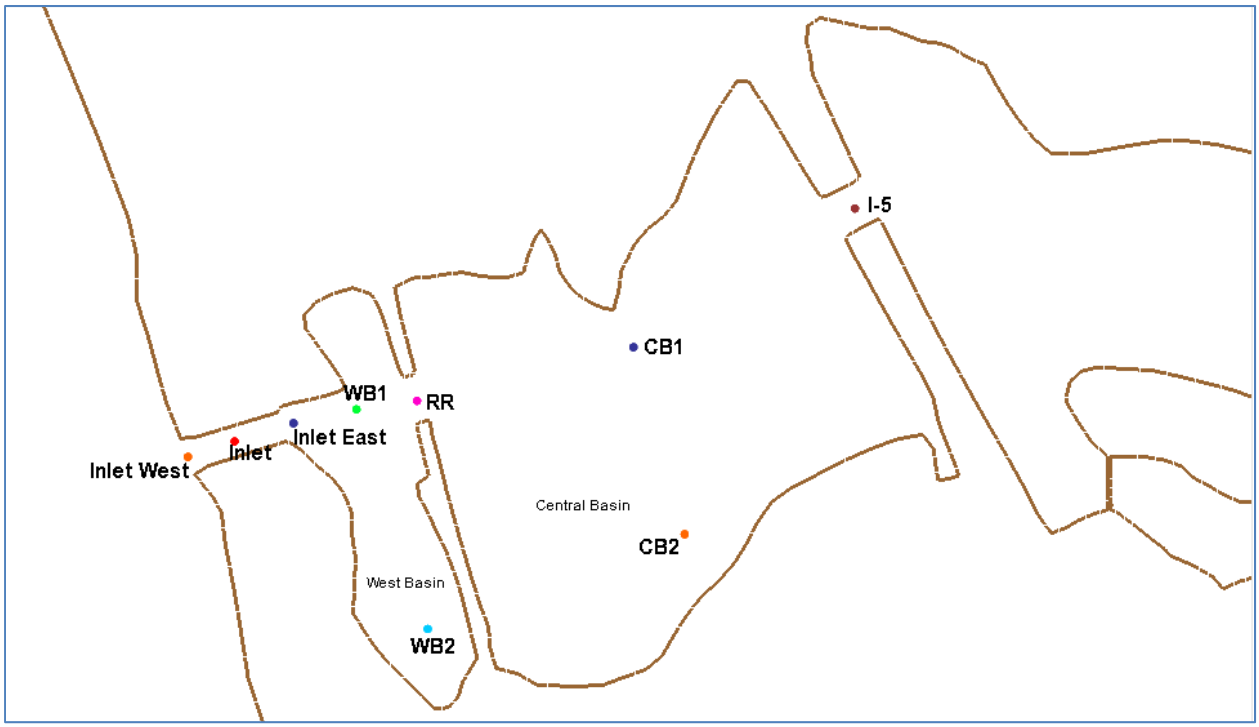

Figure 3-2: Virtual Gage Locations for Tidal Range Comparison

Table 3-1 shows tidal ranges from the ocean to the East Basin for existing conditions (assumed as post-construction or dredged) and the optimized bridge condition. An optimized channel width of 202 feet is specified under the RR Bridge and an optimized width of 134 feet is specified under the I-5 Bridge. The results indicate:

- Tides are muted through the relatively long and narrow tidal inlet, and the tidal range decreases from 8.37 feet in the ocean to 7.29 feet for the existing (post-construction dredged) condition and to 7.42 feet in the West Basin for the optimized condition.
- The tidal range at gage location WB1 is the same as that at WB2, and there is no muting from WB1 to WB2. Therefore, the tidal range may not vary throughout the Basin and gages WB1 and WB2 are representative of the West Basin.
- The tidal range at CB1 is the same as that at CB2. The tidal range may not vary throughout the Basin and gages CB1 and CB2 are representative of the Central Basin.
- The tidal range at EB1 shown in Figure 3-3 is slightly different from that at EB2. Therefore, the tidal range may vary throughout the Basin and results at both gages are calculated and reported.

Table 3-1: Comparison of Tidal Ranges (ft) in Each Basin

Bridge Condition	Ocean	Inlet West	Inlet	Inlet East	WB1	WB2	RR	CB1	CB2	I-5	EB1	EB2
Existing	8.37	8.31	7.78	7.41	7.29	7.29	7.26	7.23	7.23	7.17	7.12	7.14
Optimized	8.37	8.32	7.83	7.51	7.42	7.42	7.41	7.4	7.4	7.4	7.35	7.38

Figure 3-3: Virtual Gage Locations for Tidal Range Calculations

3.1 I-5 Channel Dimensions Optimization Results

An over-sized channel with a width of 600 feet and a depth of 7 feet was assumed under the RR Bridge for optimizing the channel dimensions under the I-5 Bridge. A lagoon with a larger tidal prism will typically experience more tidal muting than a lagoon with a smaller tidal prism if both lagoons have an identical tidal inlet that limits tidal exchange (as is the case at Batiquitos Lagoon). Therefore, the dredged lagoon bathymetry condition was modeled since the lagoon storage or tidal prism is bigger under the dredged condition than that under the shoaled condition with the same tidal inlet. Figure 3-4 shows the model predicted tidal ranges with various channel widths under the I-5 Bridge, while the lagoon condition and other bridge dimensions are kept the same. Figure 3-5 shows the model predicted tidal ranges with various

channel invert elevations under the I-5 Bridge, while all other parameters are held constant including the channel width under the I-5 Bridge. The results indicate the existing channel invert elevation of -7 feet under the dredged condition is appropriate.

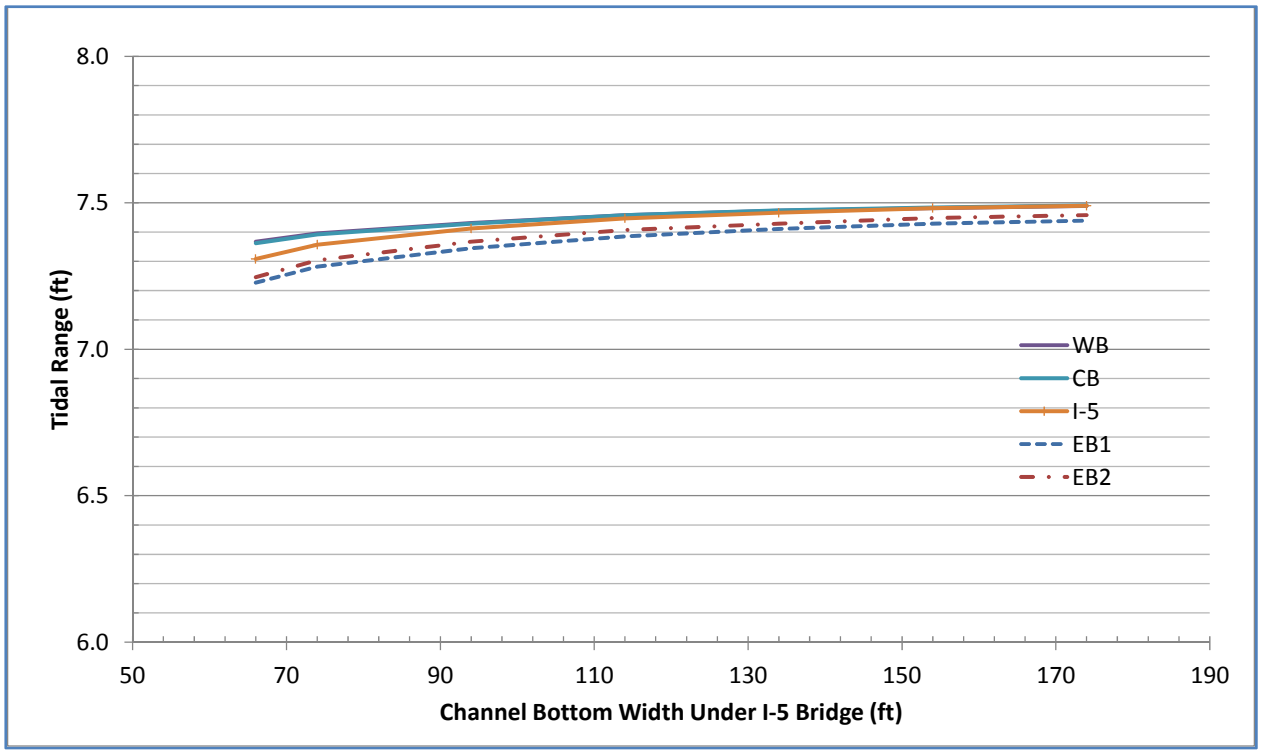

Figure 3-4: I-5 Optimization Results with Different Channel Widths

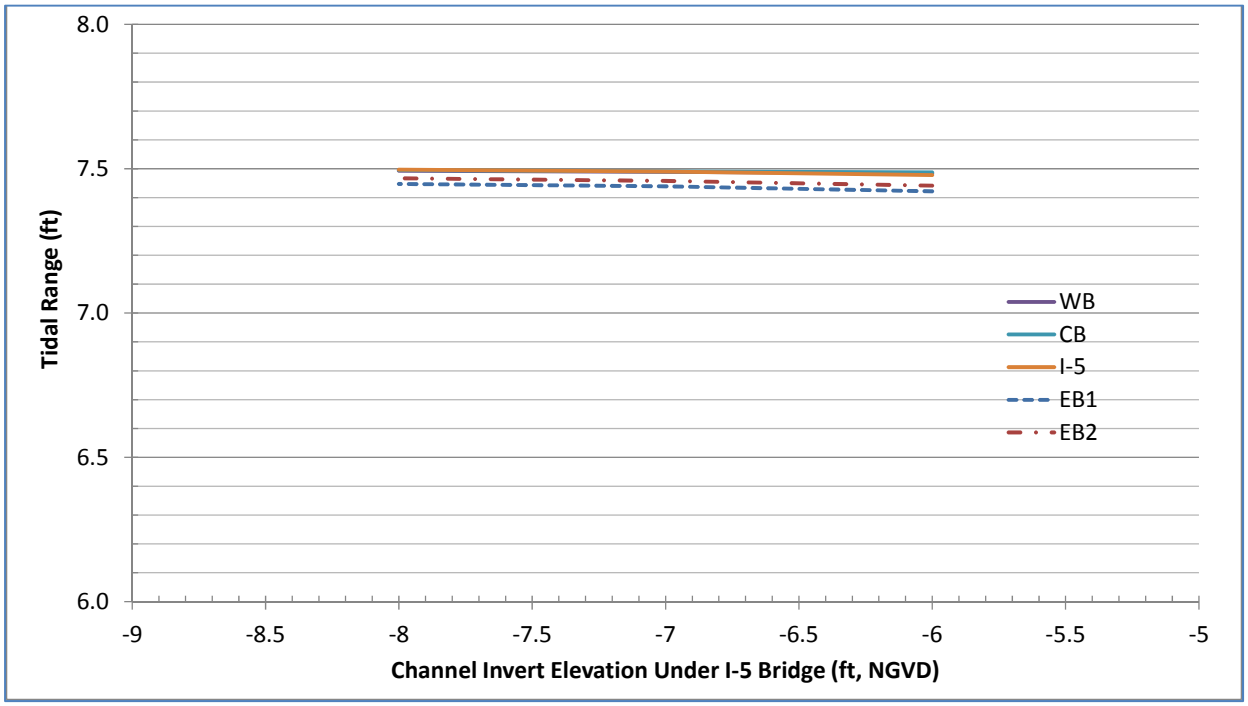

Figure 3-5: I-5 Optimization Results with Different Channel Depths

Table 3-2 summarizes the results of all dimension optimization modeling of the channel under the I-5 Bridge. The yellow highlighted row shows the tidal range under the existing baseline condition with a shoaled lagoon and channels under the bridges. The second row shows the result under the dredged condition of the existing lagoon and channels under the bridges. Dredging would increase the tidal range by approximately 0.4 feet in the Central Basin and 0.7 feet in the East Basin. Increasing the width of the channel has some benefit, but the benefit diminishes if the width is increased beyond 134 feet.

Table 3-2: Summary of I-5 Optimization Results

Run Description	I-5 Dimensions (ft)		Tidal Range (ft)							
	Width	Invert	Ocean	Inlet	WB	RR	CB	I-5	EB1	EB2
Existing shoaled condition	**66**	**-5.3**	**8.37**	**7.75**	**4.59**	**6.96**	**6.79**	**6.73**	**6.47**	**6.48**
Existing dredged condition	66	-7	8.37	7.78	7.29	7.26	7.23	7.17	7.12	7.14
RR Longer, I-5 Existing	66	-7	8.37	7.81	7.37	7.36	7.36	7.31	7.23	7.25
RR Longer, I-5 8 ft longer	74	-7	8.37	7.82	7.40	7.39	7.39	7.36	7.28	7.30
RR Longer, I-5 20 ft longer	94	-7	8.37	7.83	7.43	7.43	7.43	7.41	7.35	7.37
RR Longer, I-5 40 ft longer	114	-7	8.37	7.85	7.46	7.46	7.46	7.45	7.39	7.41
RR Longer, I-5 60 ft longer	134	-7	8.37	7.85	7.47	7.47	7.47	7.47	7.41	7.43
RR Longer, I-5 80 ft longer	154	-7	8.37	7.86	7.48	7.48	7.48	7.48	7.43	7.45
RR Longer, I-5 100 ft longer	174	-7	8.37	7.86	7.49	7.49	7.49	7.49	7.44	7.46
RR Longer, I-5 100 ft longer	174	-6	8.37	7.86	7.48	7.48	7.49	7.48	7.42	7.44
RR Longer, I-5 100 ft longer	174	-7	8.37	7.86	7.49	7.49	7.49	7.49	7.44	7.46
RR Longer, I-5 100 ft longer	174	-8	8.37	7.86	7.49	7.49	7.50	7.50	7.45	7.47

3.2 RR Dimensions Optimization Results

An over-sized channel with a width approximately 174 feet and a depth of 7 feet under the I-5 Bridge were assumed in optimizing the channel dimensions under the RR Bridge. The dredged lagoon bathymetry condition was modeled since the tidal prism is larger and will experience more muting due to tidal prism under the dredged condition than the shoaled condition. Figure 3-6 shows the model-predicted tidal ranges with various channel widths under the RR Bridge while the lagoon condition and other bridge dimensions are held constant. There is a relatively significant gain in the tidal range when the channel width increases from the existing 162 feet to 202 feet; however, the gain in tidal prism diminishes with furthering widening of the channel.

Figure 3-7 shows the model-predicted tidal ranges with various channel invert elevations under the RR Bridge while all other parameters are held constant, including the width of the channel under the RR Bridge. The results indicate the existing channel with an invert elevation of -7 feet under the dredged condition is appropriate. Further deepening of the channel invert does not provide much additional benefit.

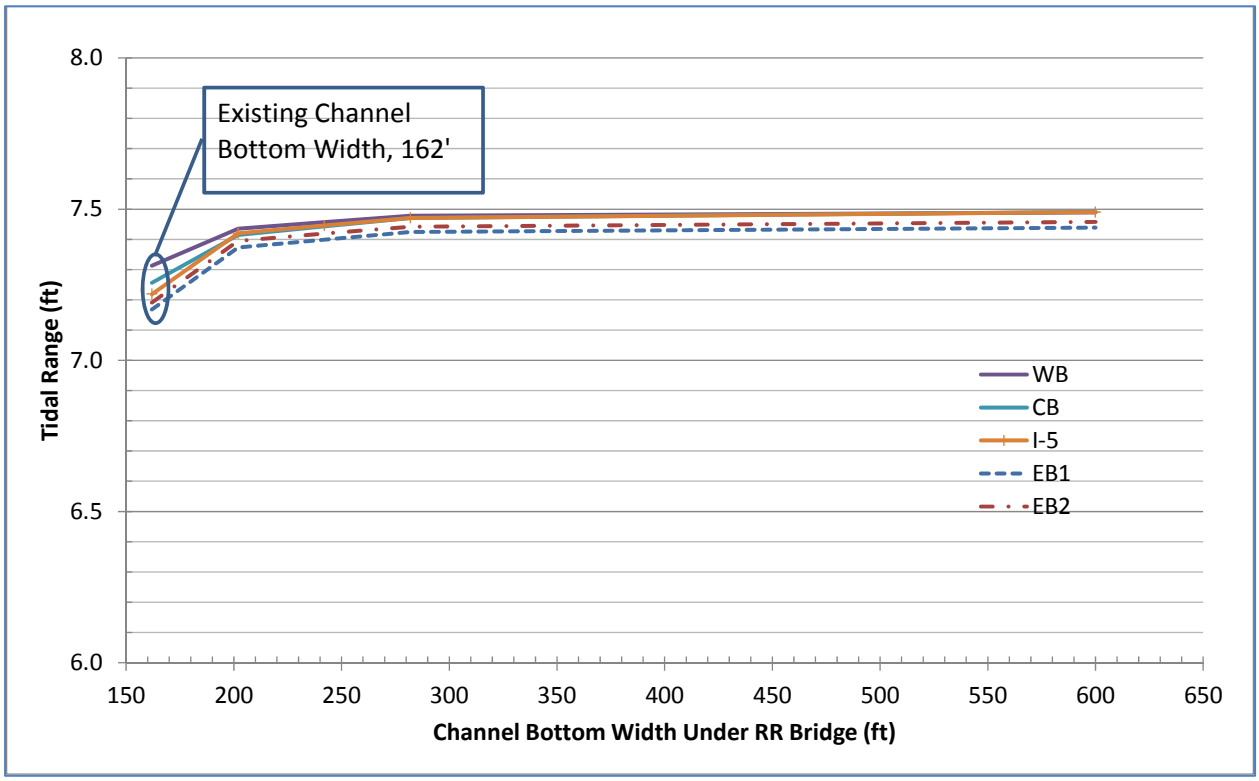

Figure 3-6: RR Optimization Results with Different Channel Widths

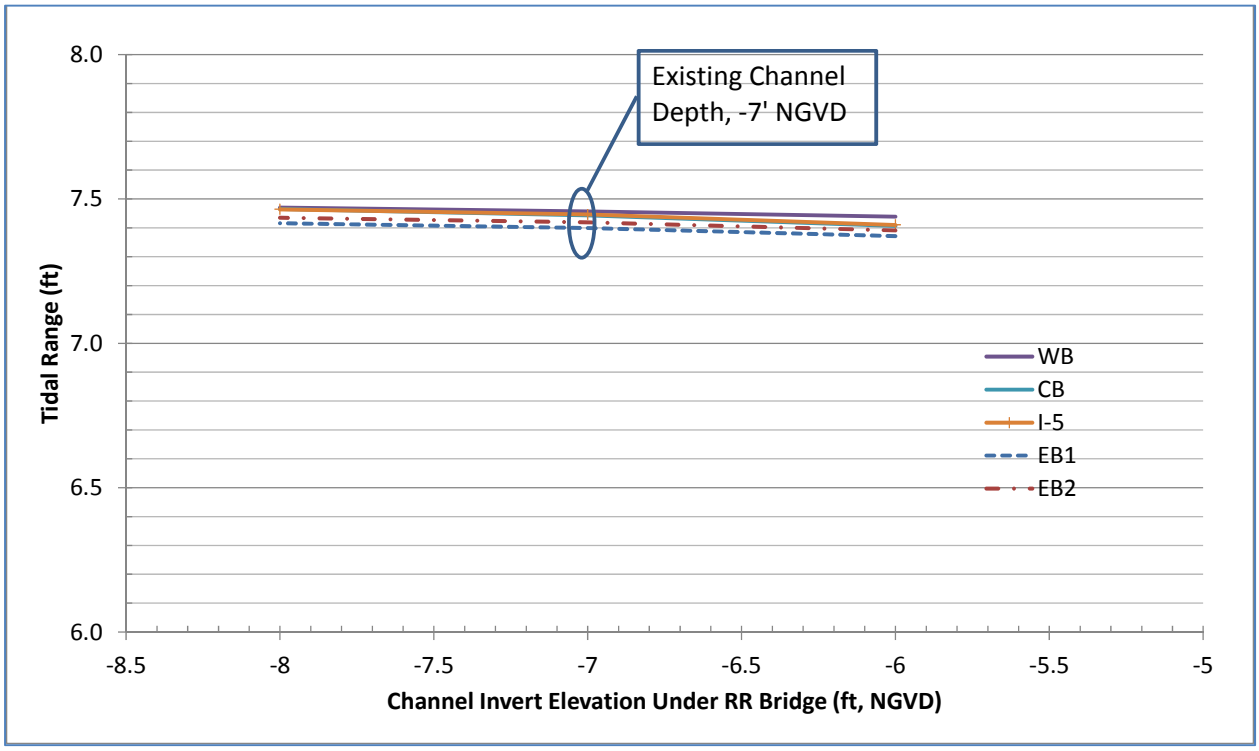

Figure 3-7: RR Optimization Results with Different Channel Invert Elevations

Table 3-3 summarizes the results of optimization modeling of the channel under the RR Bridge. The yellow highlighted row shows the tidal range under the existing baseline condition with a shoaled lagoon and channels under the bridges. The second row shows the result under the dredged condition of the existing lagoon and channels under the bridges. Dredging would increase the tidal range by approximately 0.5 feet in the Central Basin and 0.7 feet in the East Basin. Increasing the width of the channel has some benefit, but the benefit diminishes if the width is increased beyond 202 feet. The lower part of the table shows tidal ranges under different channel invert depths. The results indicate that modifying channel depths does not significantly increase tidal range and the current design invert elevation of -7 feet NGVD is appropriate.

Table 3-3: Summary of RR Optimization Results

Run Description	RR Dimensions (ft)		Tidal Range (ft)							
	Invert (NGVD)	Width	Ocean	Inlet	WB2	RR	CB2	I-5	EB1	EB2
Existing shoaled condition	-6.35	162	8.37	7.75	7.15	6.96	6.80	6.73	6.47	6.48
Existing dredged condition	-7	162	8.37	7.78	7.29	7.26	7.23	7.17	7.12	7.14
RR 40 ft Longer	-7	202	8.37	7.85	7.44	7.43	7.41	7.42	7.37	7.40
RR 80 ft Longer	-7	242	8.37	7.86	7.46	7.45	7.44	7.45	7.40	7.42
RR 120 ft Longer	-7	282	8.37	7.86	7.48	7.47	7.47	7.47	7.42	7.44
I-5 100 ft longer	-7	600	8.37	7.86	7.49	7.49	7.49	7.49	7.44	7.46
RR 80 ft Longer	-6	242	8.37	7.86	7.44	7.43	7.41	7.41	7.37	7.39
RR 80 ft Longer	-7	242	8.37	7.86	7.46	7.45	7.44	7.45	7.40	7.42
RR 80 ft Longer	-8	242	8.37	7.86	7.47	7.47	7.47	7.46	7.42	7.44

3.3 Results of Combined I-5 and RR Dimensions Optimization

Sections 3.1 and 3.2 presented the optimization results of one bridge at a time, while keeping the other bridge dimensions over-sized and constant. This section presents modeling results of a combination of different channel widths under the I-5 and RR Bridges. The modeling results in the previous sections indicate that the channel invert elevation of -7 feet is the optimized elevation. Therefore, the invert elevation of -7 feet is used for all remaining modeling runs.

Table 3-4 shows model-predicted tidal ranges in the Central and East Basins with various channel widths under the I-5 and RR Bridges. The dredged bathymetry condition was used for all modeling runs. Yellow highlighted cells show the tidal range with the existing channel dimensions with the channels dredged. The results indicate the optimal channel width is 134 feet under the I-5 Bridge and 202 feet under the RR Bridge. Green highlighted cells in Table 3-4 show the tidal range with the optimized bridge dimensions. The increase in tidal range is less than 0.05 feet if channels are widened beyond the recommended dimensions of 202 feet under the RR Bridge and 134 feet under the I-5 Bridge. The tidal range increase of 0.05 feet (or 0.6 inches) is insignificant when compared with the ocean tidal range of 8.37 feet.

Table 3-4: Tidal Range (ft) in the Central and East Basins

I-5 Channel Bottom Width (ft)	RR Channel Bottom Width (ft)							
	Central Basin				East Basin			
	162	202	242	282	162	202	242	282
66	***7.23***				***7.14***			
94		7.36	7.39	7.41		7.29	7.32	7.34
134	7.35	7.40	7.43	7.46	7.31	7.35	7.38	7.40
174		7.41	7.44	7.47		7.37	7.4	7.42

4.0 ANALYSES TO ACHIEVE OPTIMAL FLOOD CONVEYANCE

The goal of this hydrodynamic modeling section is to determine the optimal channel width and depth under the RR and I-5 Bridges for lowering the storm flood elevation in the lagoon. As discussed in the previous sections, the tidal inlet under Carlsbad Boulevard Bridges was sized and designed to achieve a stable inlet as part of the Batiquitos Lagoon Restoration Project. The tidal inlet has been performing well since construction in 1995. Therefore, no optimization was performed for the tidal inlet.

The calibrated and verified RMA-2 numerical model was used in predicting flood water surface elevations throughout the lagoon. The average spring high tide elevation is approximately 4.69 feet NGVD and the highest tidal elevation measured in this area is 5.35 feet NGVD. To be consistent with the optimization study of San Elijo Lagoon (M&N 2012), the spring high tidal series was raised vertically up such that the spring high tidal elevation is 7.00 feet. This 7-foot elevation is the FEMA base flood elevation along the shoreline of San Elijo Lagoon and it includes the water level rise due to wave runup. It is a very conservative elevation for the flood conveyance optimization. Using this value would affect the flood water level in the lagoon, but would not affect the optimization results when considering the head loss or the backwater effect through each bridge.

The resulting tidal series is shown in Figure 4-1 and applied at the model offshore boundary for the flood optimization modeling. The elevation of the tidal series would affect the flood water surface elevation in the lagoon, but it would not affect the optimized channel dimensions. The flood hydrographs of San Marcos and Encinitas Creeks discussed in Section 2.2.4.3 were superimposed to form one hydrograph and the resulting hydrograph was applied in the model upstream boundary.

The RMA-2 model is an unsteady hydrodynamic model. Both the offshore tidal boundary (downstream boundary) and the upstream boundary input are time varying. The time when the peak of flood hydrograph is superimposed on top of the high tide is important and affects the modeling result. Therefore, a series of modeling runs were performed by adjusting the phase of the flood hydrograph such that both the spring high tide and the peak of the flood occur simultaneously at the I-5 Bridge.

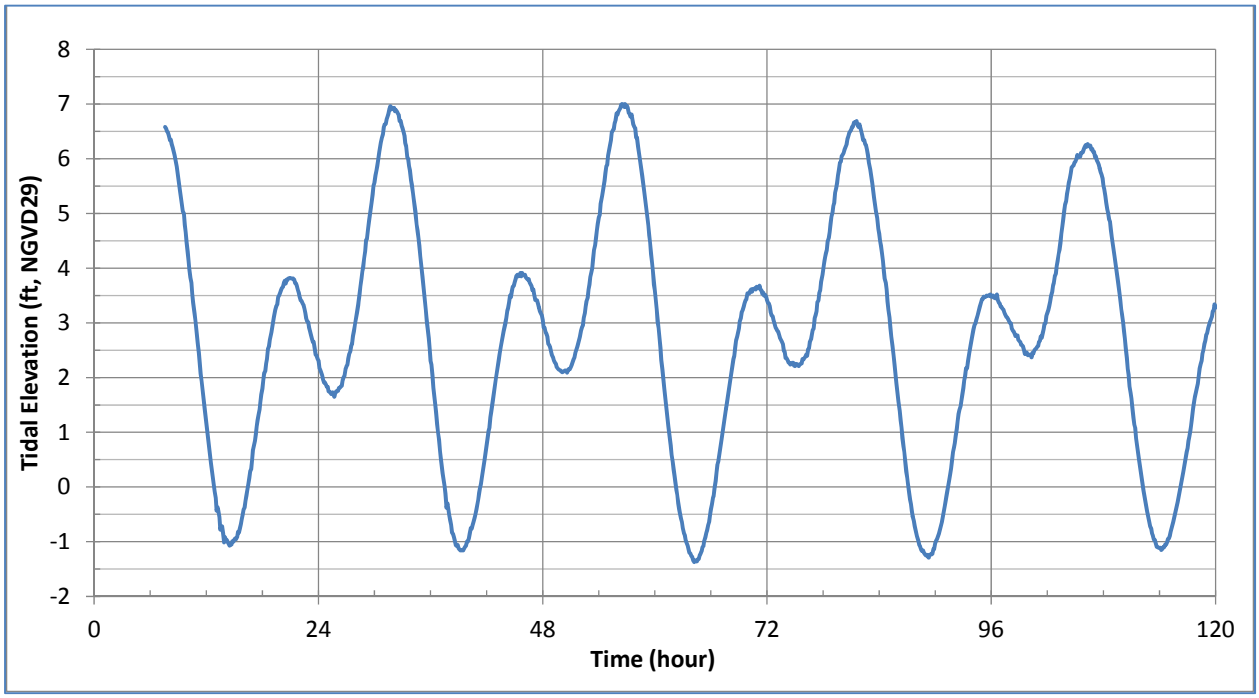

Figure 4-1: Spring High Tidal Series for Flood Optimization Modeling

The RMA-2 model predicts water surface elevation and velocity at every node point of the model grid. Figure 4-2 shows virtual gage locations where the maximum water surface elevation is extracted from the RMA-2 modeling results for plotting the maximum water surface profile. The maximum water surface profile is plotted for each model run. The maximum water surface elevation at different gage locations occurs at different times while the peak of the flood travels throughout the lagoon from east to west. The maximum water surface profile is not an instantaneous profile like those produced by a steady state model run. A steady state model run simplifies natural processes by assuming both the downstream tidal elevation and the flood elevation remain constant. The water surface profile from a steady state model run is an instantaneous profile.

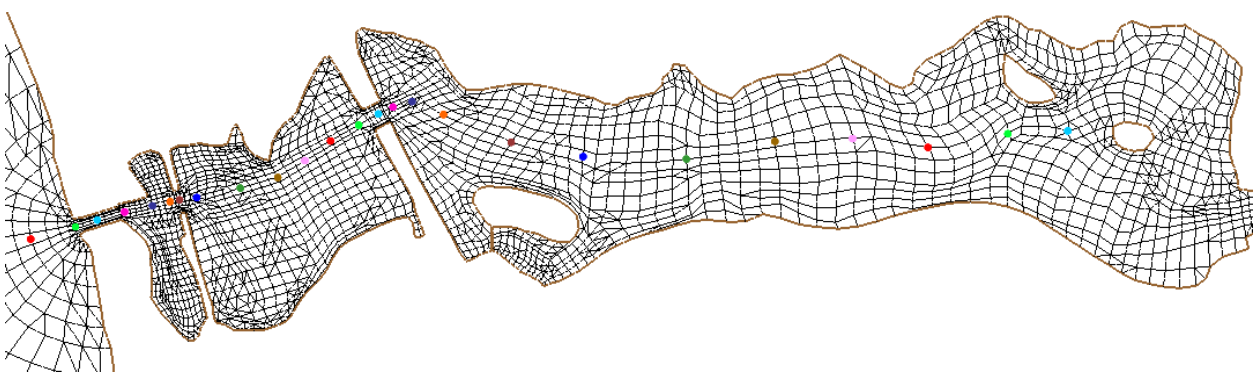

Figure 4-2: Virtual Gage Locations for Plotting Surface Water Profiles

For the channels under the I-5 and RR Bridges to achieve optimal flood conveyance throughout the lagoon, the 100-year stormflow event is modeled with the extreme spring tide. Figure 4-3 compares 100-year water surface profiles under different lagoon and channel conditions. The modeling results indicate that under the existing shoaled condition, the water surface elevation is backed up by approximately 0.8 feet in the East Basin by the I-5 Bridge, by approximately 0.4 feet in the Central Basin by the RR Bridge, and by approximately 0.7 feet in the West Basin by Carlsbad Boulevard Bridges. Clearing sedimentation from the channel under the bridge crossings will reduce the backwater effect at the I-5 and RR Bridges, and lower the water surface elevation in the East Basin, but will increase the water surface elevation in the Central and West Basins. The lagoon sedimentation condition (shoaled or dredged) assumed in the modeling has little effect on the water surface elevation. The water level is slightly lower under the dredged lagoon condition than that under the shoaled lagoon condition. Therefore, the shoaled lagoon condition is modeled for fluvial optimization modeling.

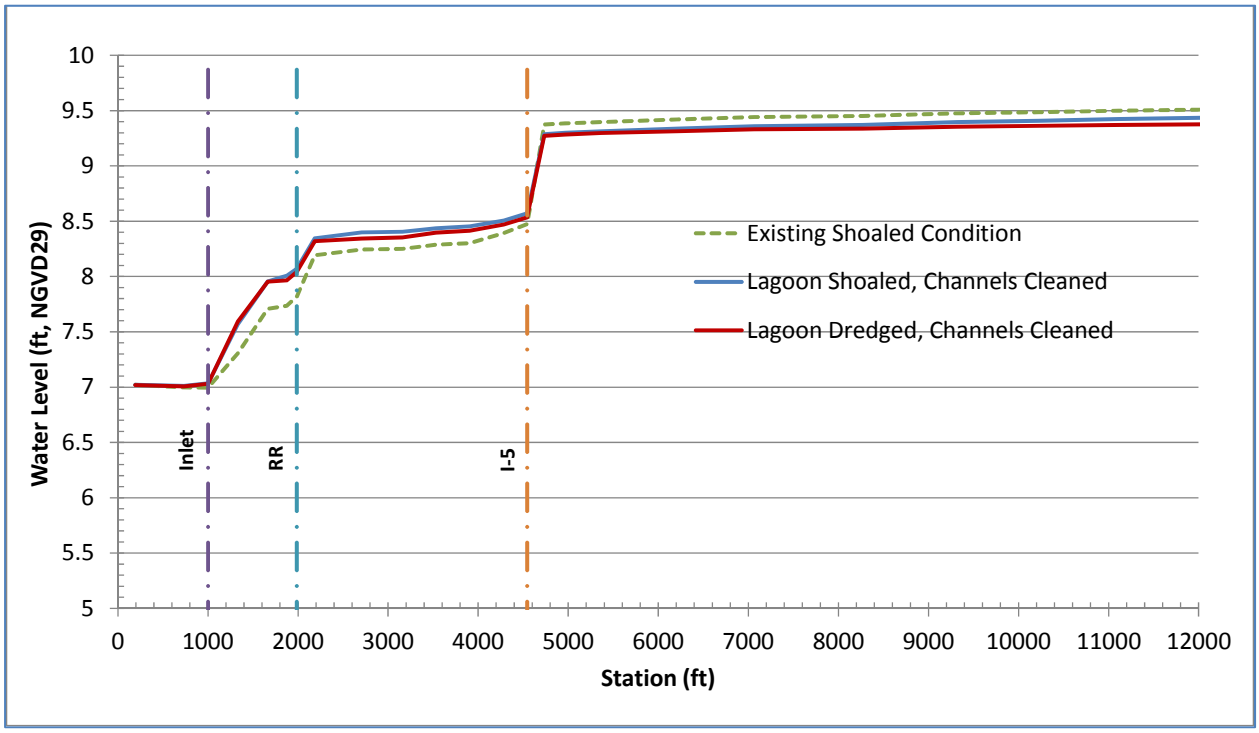

Figure 4-3: Comparison of 100-Year Surface Profile for Different Lagoon Sedimentation Conditions

4.1 I-5 Channel Dimensions Optimization Results

Figure 4-5 presents 100-year water surface profiles through the lagoon for different channel widths under I-5 Bridge while keeping the RR Bridge channel in its design condition. Modeling results indicate widening the channel under the I-5 Bridge will reduce the backwater effect created by the I-5 Bridge and lower the water surface elevation in the East Basin. However, it

will also shift the backwater effect to downstream of the I-5 Bridge and increase the water surface elevation in the Central and West Basins.

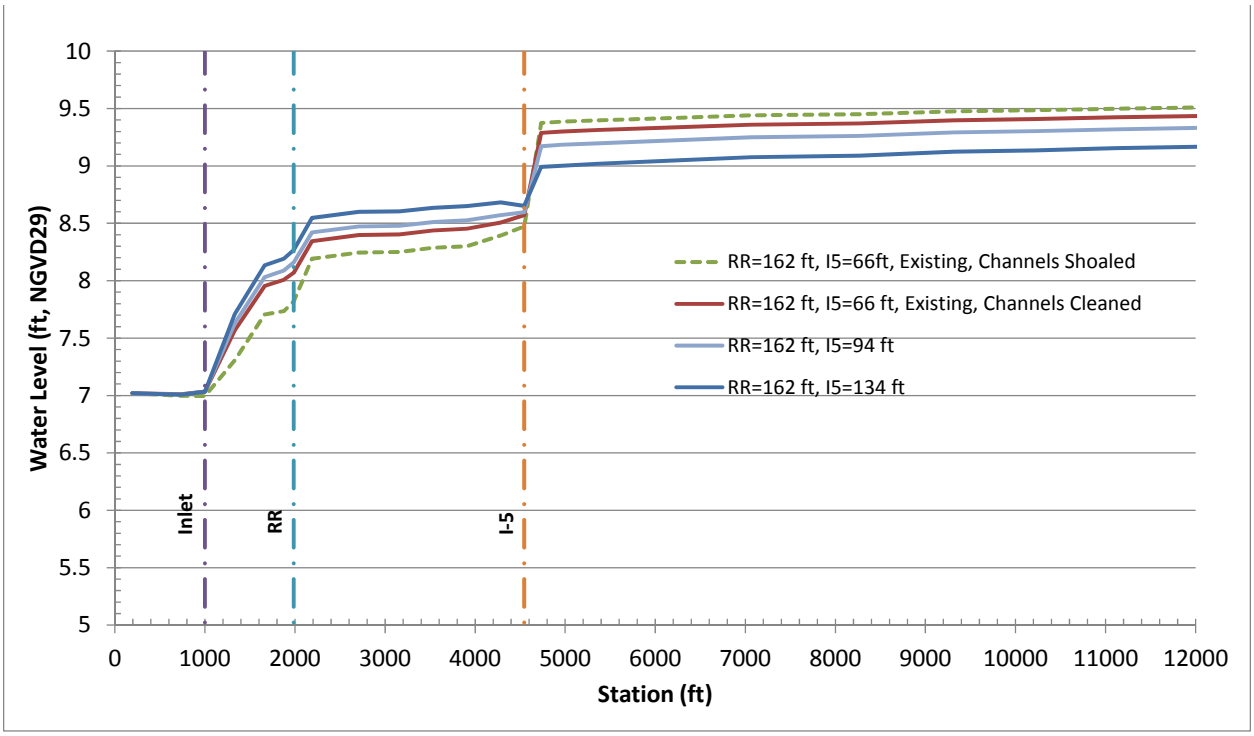

Figure 4-4: 100-Year Surface Profiles Under Different I-5 Channel Widths

4.2 RR Dimensions Optimization Results

Figure 4-6 illustrates 100-year water surface profiles through the lagoon under different RR Bridge channel widths while keeping the I-5 Bridge channel width under its optimized condition based on tidal range modeling. Modeling results indicate widening the channel under the RR Bridge will only slightly reduce the backwater effect created by the RR Bridge and will lower the water surface elevation in the Central and East Basins. However, this action will also shift the backwater effect to downstream of the RR Bridge and increase the water surface elevation in the West Basin.

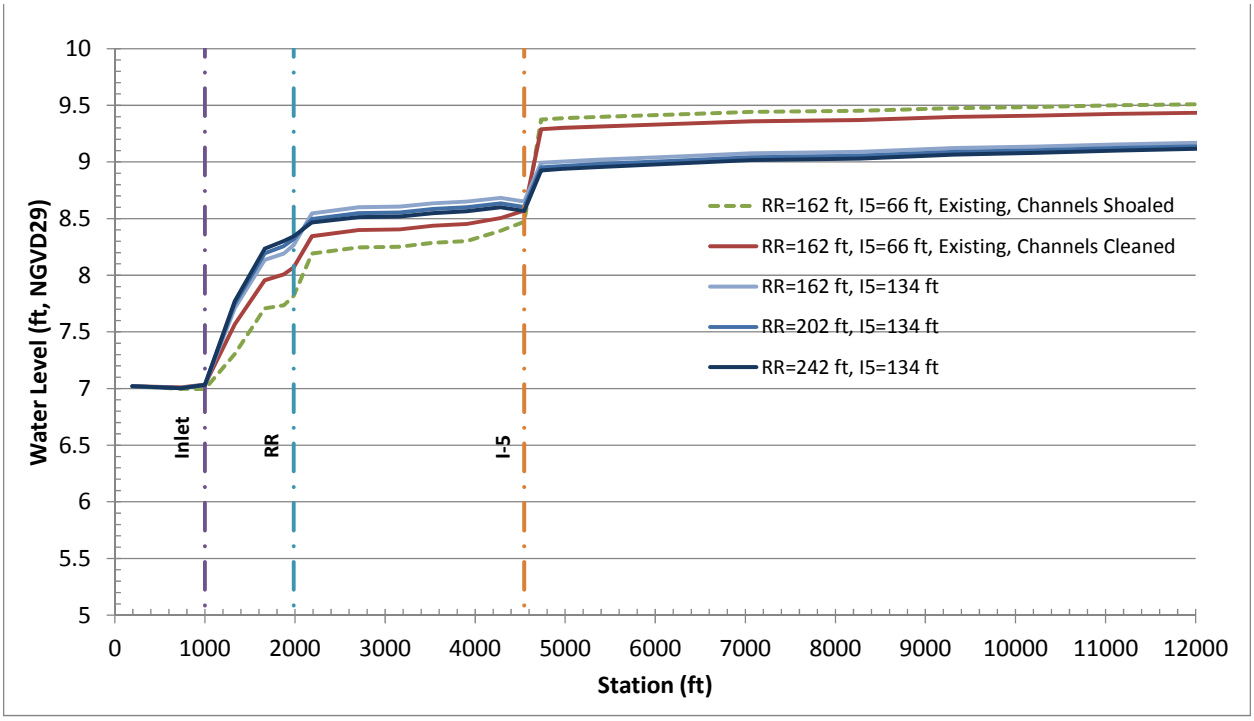

Figure 4-5: 100-Year Surface Profiles Under Different RR Channel Widths

4.3 Results of Combined Channel Dimensions Optimization for I-5 and RR Bridges

Figure 4-6 illustrates 100-year water surface profiles for various channel widths under the I-5 and RR Bridges. Table 4-1 summarizes the 100-year water surface elevations under different channel dimensions. In general, widening the channels under the I-5 and RR Bridges would reduce the backwater effects and slightly lower the water level in the Central and East Basins, but would shift the backwater effect to the West Basin upstream of the Carlsbad Boulevard Bridges and slightly increase water surface elevations in the West Basin. Since flooding is not currently an issue for Batiquitos Lagoon, the channel widths optimized for the tidal range also work for flood conveyance.

The green highlighted cells in Table 4-1 show the 100-year water levels under the optimized channel conditions. The yellow highlighted cells in the Table show the 100-year water levels under the existing channel conditions, assuming the channels are dredged to conditions shown in the design drawings.

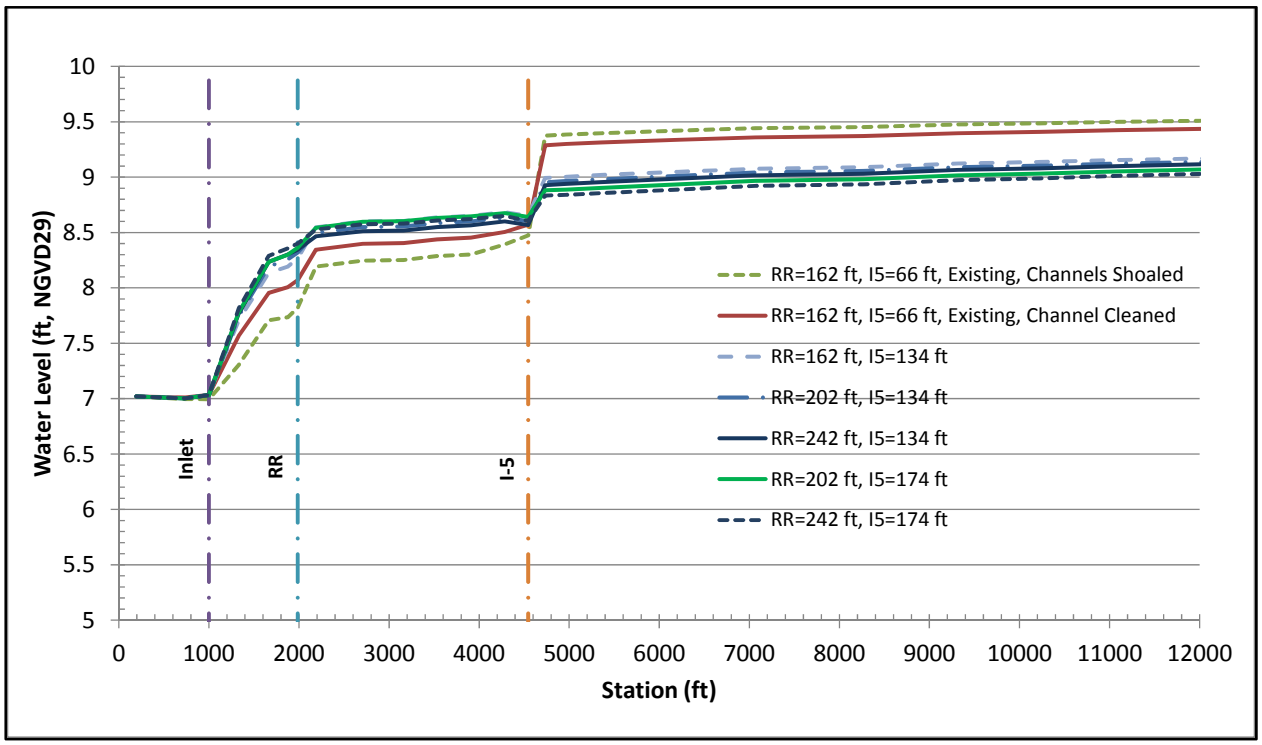

Figure 4-6: 100-Year Surface Profiles for Combined Channel Optimization Under I-5 and RR Bridges

Table 4-1: Summary of 100-Year Flood Levels in Each Basin

I-5 Channel Bottom Width (ft)	RR Channel Bottom Width (ft)								
	West Basin			Central Basin			East Basin		
	162	202	242	162	202	242	162	202	242
66	*7.9*			*8.4*			*9.4*		
94	8.0	8.1		8.5	8.4		9.3	9.2	
134	8.1	8.2	8.2	8.6	8.6	8.5	9.1	9.1	9.0
174		8.2	8.3		8.6	8.6		9.0	8.9

4.4 Hydrodynamic Modeling Results of the 50-Year Storm Event

Hydrodynamic modeling runs were also performed for the 50-year storm event. The same offshore tidal boundary used in the optimization of fluvial conveyance was used and is shown in Figure 4-1. The 50-year water surface profiles calculated from the RMA-2 modeling results are shown in Figure 4-7 in “warm” color lines. The 100-year water surface profiles shown in “cold” color lines are also included for relative comparison.

The water surface elevations for the 50-year flood are much lower than those under the 100-year storm event; however the pattern of change in water surface elevation is very similar to that under the 100-year storm event. The water surface elevations will be lower in the East Basin but higher in the Central and West Basins with optimized bridge dimensions.

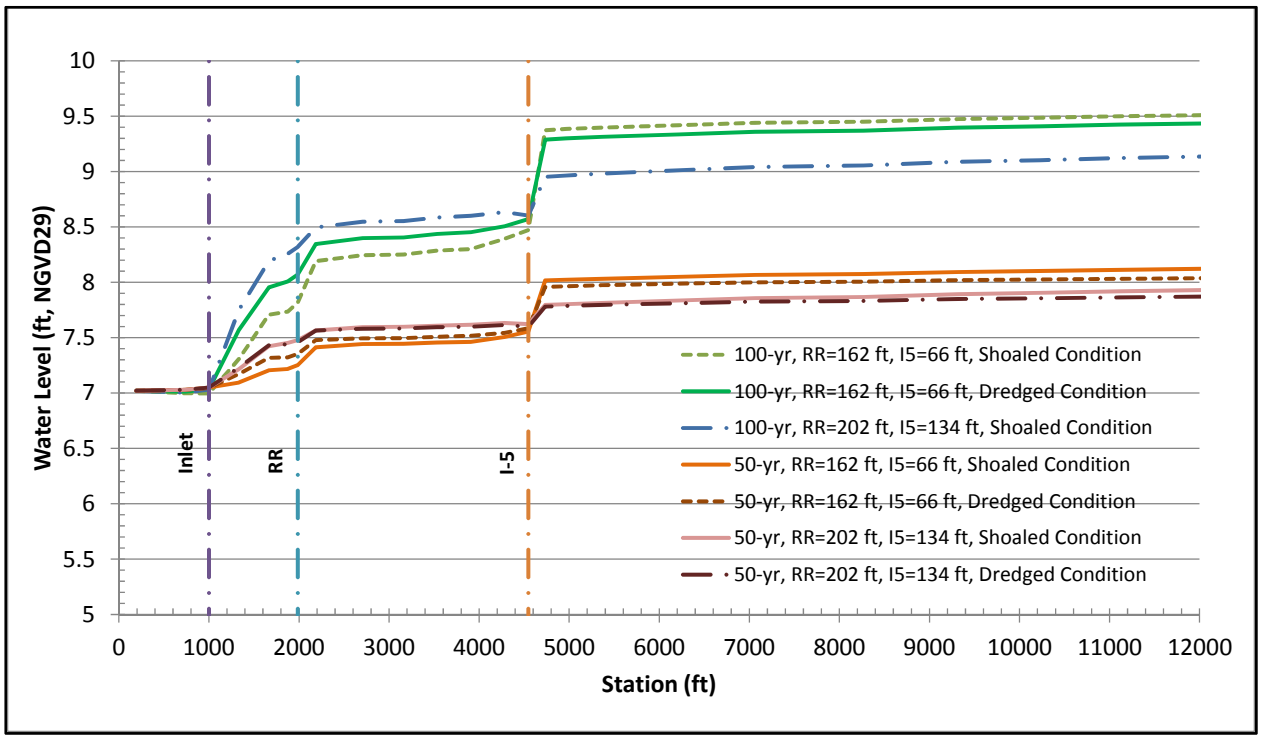

Figure 4-7: 50-Year Water Surface Profiles

5.0 SUMMARY OF EXISTING AND OPTIMIZED CHANNEL DIMENSIONS UNDER BRIDGES

The existing channel widths below the I-5 and RR Bridge crossings provided a starting point for the optimization modeling. The following section provides a brief description of each bridge and the range of channel dimensions evaluated in the optimization study.

5.1 Carlsbad Boulevard Bridges

The Carlsbad Boulevard Bridges, shown in Figure 5-1, cross over the Batiquitos tidal inlet. The existing tidal inlet under the Carlsbad Boulevard Bridges was sized and designed to achieve a stable inlet as part of the Batiquitos Lagoon restoration project. The tidal inlet has been performing well since construction in 1995. Therefore, no further optimization is required for the tidal inlet. As-built drawings (M&N 1993) indicate a channel bottom width of 96 feet at an invert elevation of -8 feet, NGVD. The channel under the East Carlsbad Boulevard Bridge is concrete lined as shown in Figure 5-2 and Figure 5-4, and the West Carlsbad Boulevard Bridge is lined with armor rocks as shown in Figure 5-3. Both bridges slope from north to south. The road surface profile grade of the East Carlsbad Boulevard Bridge is about 1.8 feet lower than that of West Carlsbad Boulevard Bridge. The soffit elevation of the East Carlsbad Boulevard Bridge is approximately +9.2 feet scaled from the as-built drawing.

Figure 5-1: Image of Carlsbad Boulevard and Railroad Bridges (source: California Coastal Records Project, 2012)

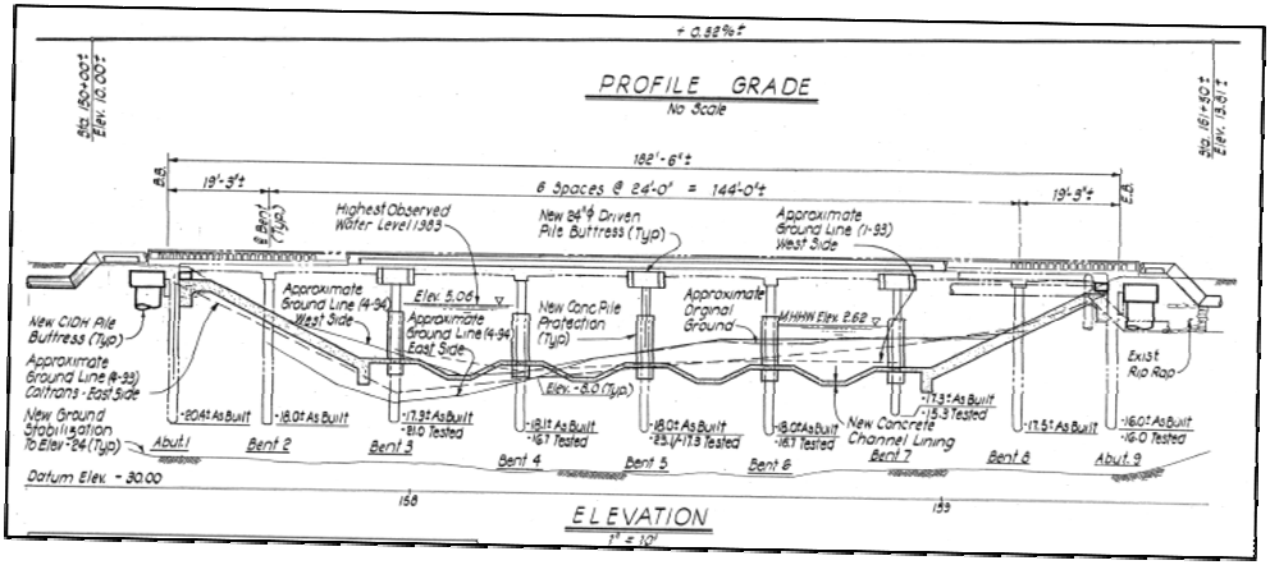

Figure 5-2: East Carlsbad Boulevard Bridge As-Built Drawing (Looking from Lagoon to Ocean)

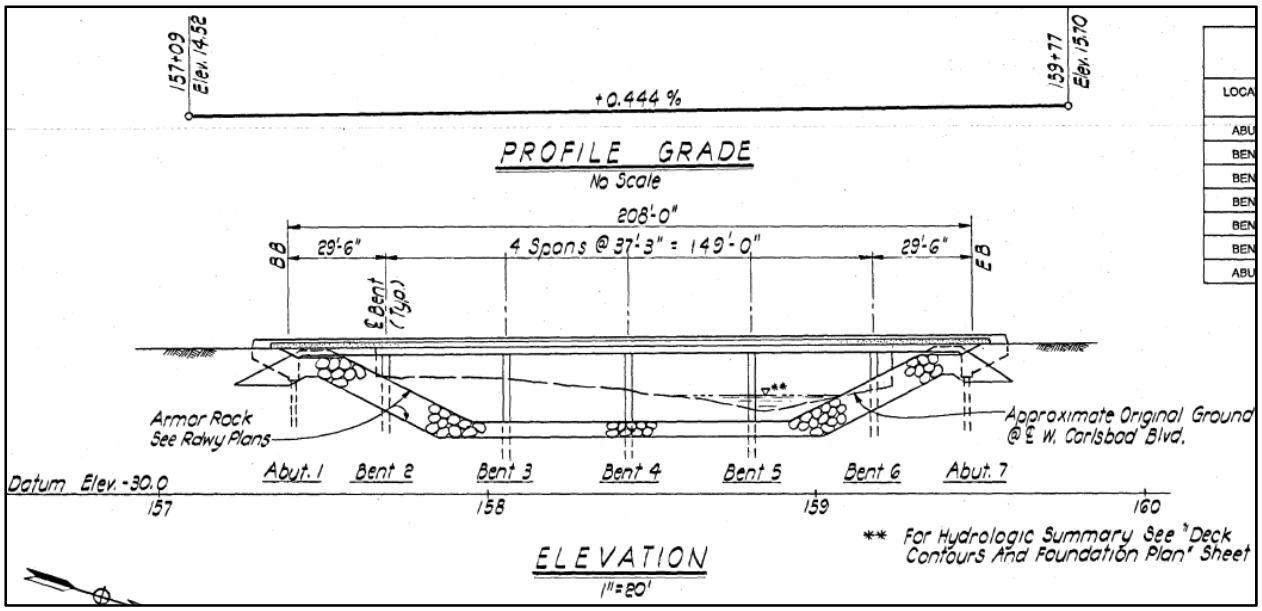

Figure 5-3: West Carlsbad Boulevard Bridge As-Built Drawing (Looking from Lagoon to Ocean)

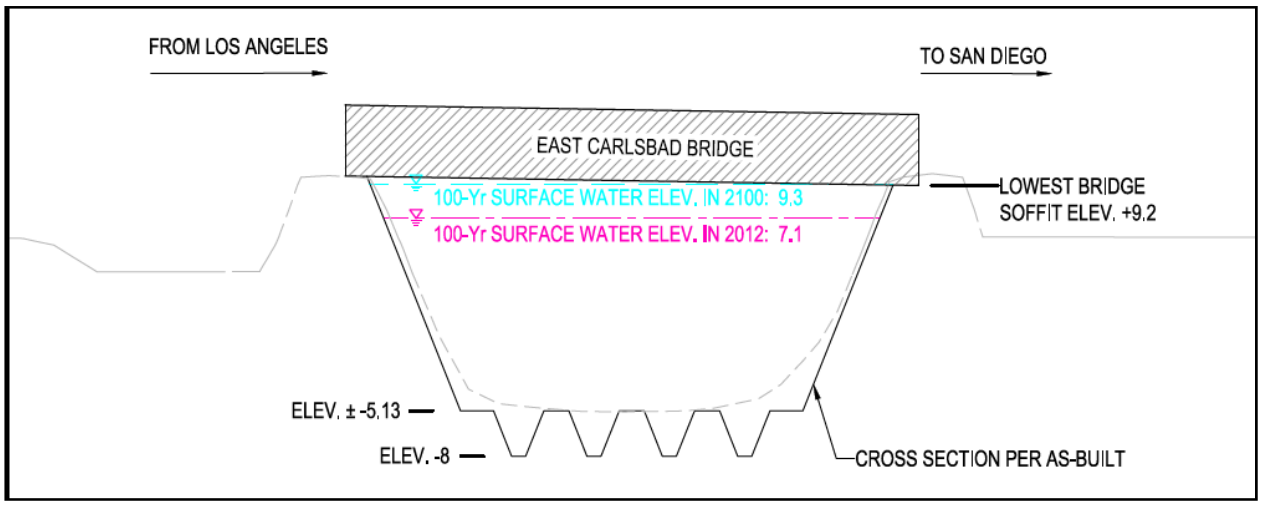

Figure 5-4: Channel Cross Section Under East Carlsbad Boulevard Bridge

Figure 5-4 also shows the model-predicted 100-year water surface elevations under the current horizon and for year 2100, taking into consideration 55-inches of SLR. The water surface elevation in year 2100 touches the bridge soffit in the east end of the bridge, but it does not create a flow situation that would require a specialized modeling approach called "pressure flow modeling." Existing modeling results should accurately portray future conditions.

5.2 Railroad Bridge

The RR Bridge, shown in both Figure 5-1 and Figure 5-5, runs just east of and parallel to Carlsbad Boulevard Bridges across the Batiquitos Lagoon. As-built drawings (M&N 1993) show the total bridge length is 308 feet and the channel bottom width is approximately 162 feet at an invert elevation of -7 feet, NGVD. The entire channel bottom is lined with a 2-feet thick layer of 400-lb riprap underlined with a 0.5-feet thick layer of bedding rock. The RR Bridge slopes from south to north. The top of rail elevation at the north end of the RR Bridge is 22.9 feet. The soffit elevation is approximately 17.3 feet scaled from the as-built drawing.

The optimization modeling focused on a range of channel bottom widths between 162 feet and 282 feet. The recommended optimal channel bottom width is 202 feet, which is 40 feet wider than its current bottom width of 162 feet. The increase in tidal range is less than 0.05 feet if the channel under the RR Bridge is widened to beyond the recommended width of 202 feet. For flood conveyance, widening the channel would only lower the water level in the Central Basin by approximately 0.1 feet, however, it would raise the water level in the West Basin by approximately the same amount. The optimization modeling also indicates that the channel invert elevation of -7 feet under the as-built (or dredged) condition is appropriate.

Figure 5-5: Image of the Existing Railroad Bridge

Figure 5-6 shows the channel cross-section under the Railroad Bridge for both existing and optimized channel dimension conditions. It also shows the model predicted water surface elevations under the current time horizon and for year 2100, taking into consideration 55-inches of SLR. Sufficient freeboard exists above the maximum flood elevations for the RR Bridge under both current and future SLR scenarios.

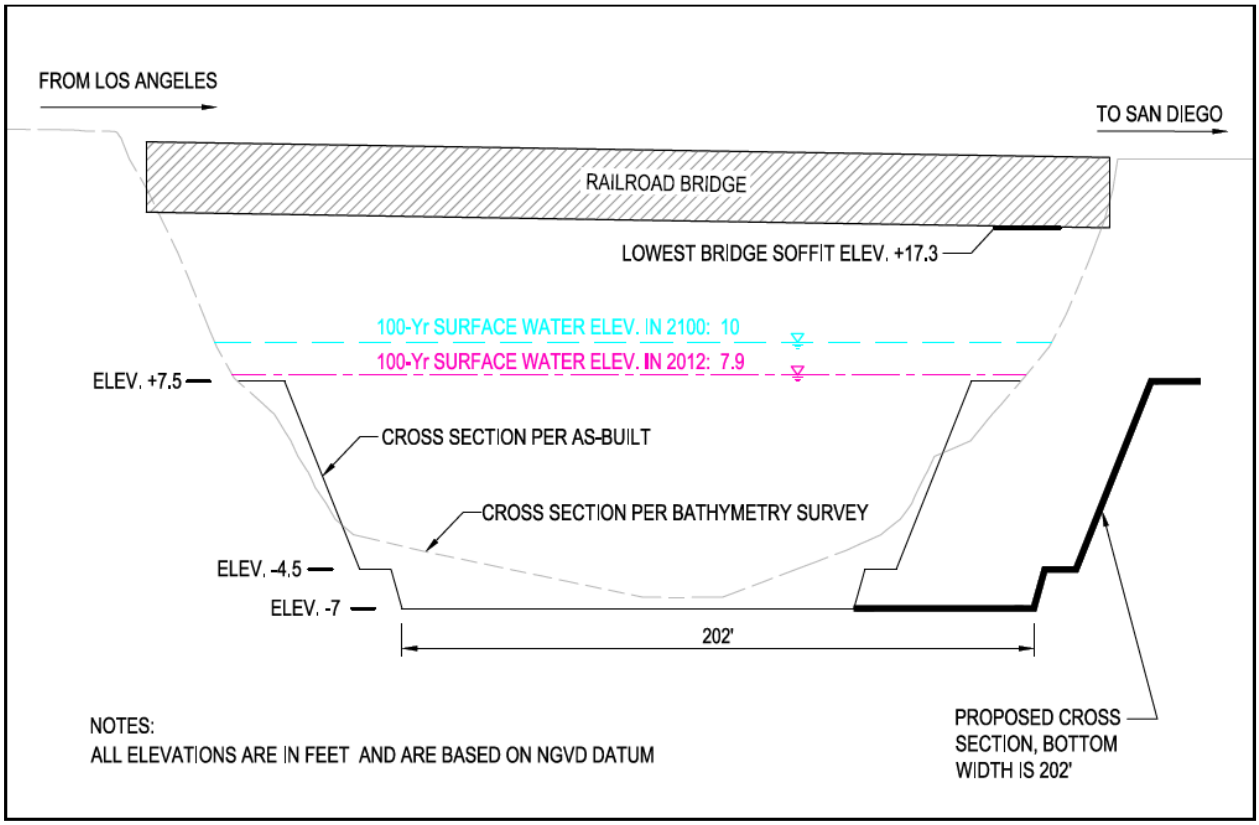

Figure 5-6: Channel Cross-Section Under the Railroad Bridge

5.3 I-5 Bridge

The I-5 freeway runs north to south across the Batiquitos Lagoon. The I-5 Bridge, shown in Figure 5-7, crosses near the middle of the lagoon serving as the boundary between the Central and East Basins. The as-built drawings indicate that the total bridge length is 219 feet and the channel bottom width under the I-5 Bridge is 66 feet at an invert elevation of -7 feet, NGVD. The entire channel is lined with a 3.5 feet thick layer of 400 lb riprap. The lowest bridge soffit elevation, indicated on the as-built drawings, is 16.14 feet, NGVD.

The I-5 Bridge optimization modeling focused on a range of channel widths between 66 feet and 174 feet. The recommended optimal channel bottom width is 134 feet, which is 68 wider than its current width of 66 feet. The increase in tidal range is less than 0.05 feet if the channel is widened to beyond the recommended width of 134 feet for the I-5 Bridge. With the recommended channel width, the 100-year flood water level in the East Basin will be lowered by 0.3 feet, however, the 100-year flood water level in the Central and West Basins will rise about 0.3 feet. The existing channel is very narrow and results a scour hole of more than 20 feet deep on both sides of the bridge. With the recommended channel width, the tidal velocity will be significantly reduced and therefore the scour depth will also diminish.

Figure 5-7: Image of Existing I-5 Bridge

Figure 5-8 shows the channel cross-section under the I-5 Bridge for both existing and optimized channel dimension conditions. It also shows the 100-year water surface elevations under the current time horizon and for year 2100, taking into consideration 55-inches of SLR. Sufficient freeboard exists above the maximum flood elevations for the I-5 Bridge under both current and future SLR scenarios. Figure 5-9 is an exhibit prepared by the Caltrans (2012) for the proposed I-5 Bridge.

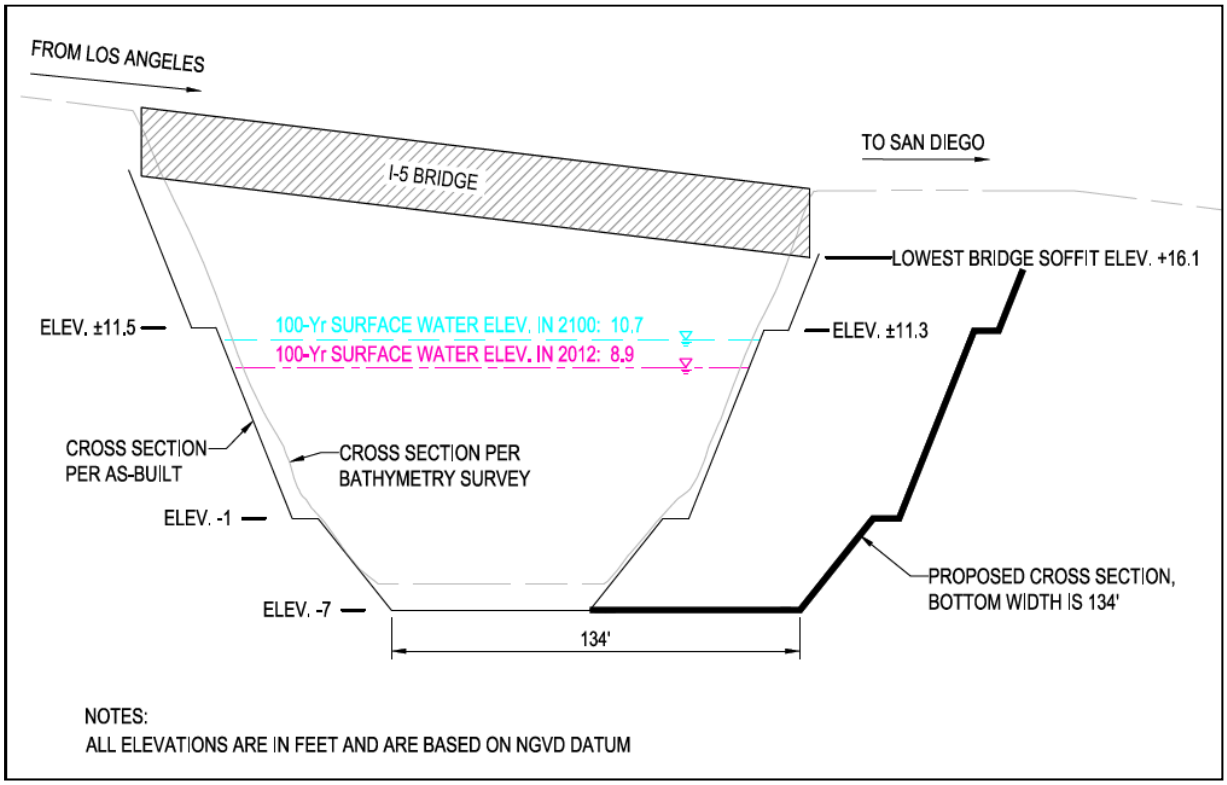

Figure 5-8: Channel Cross-Section Under I-5 Bridge

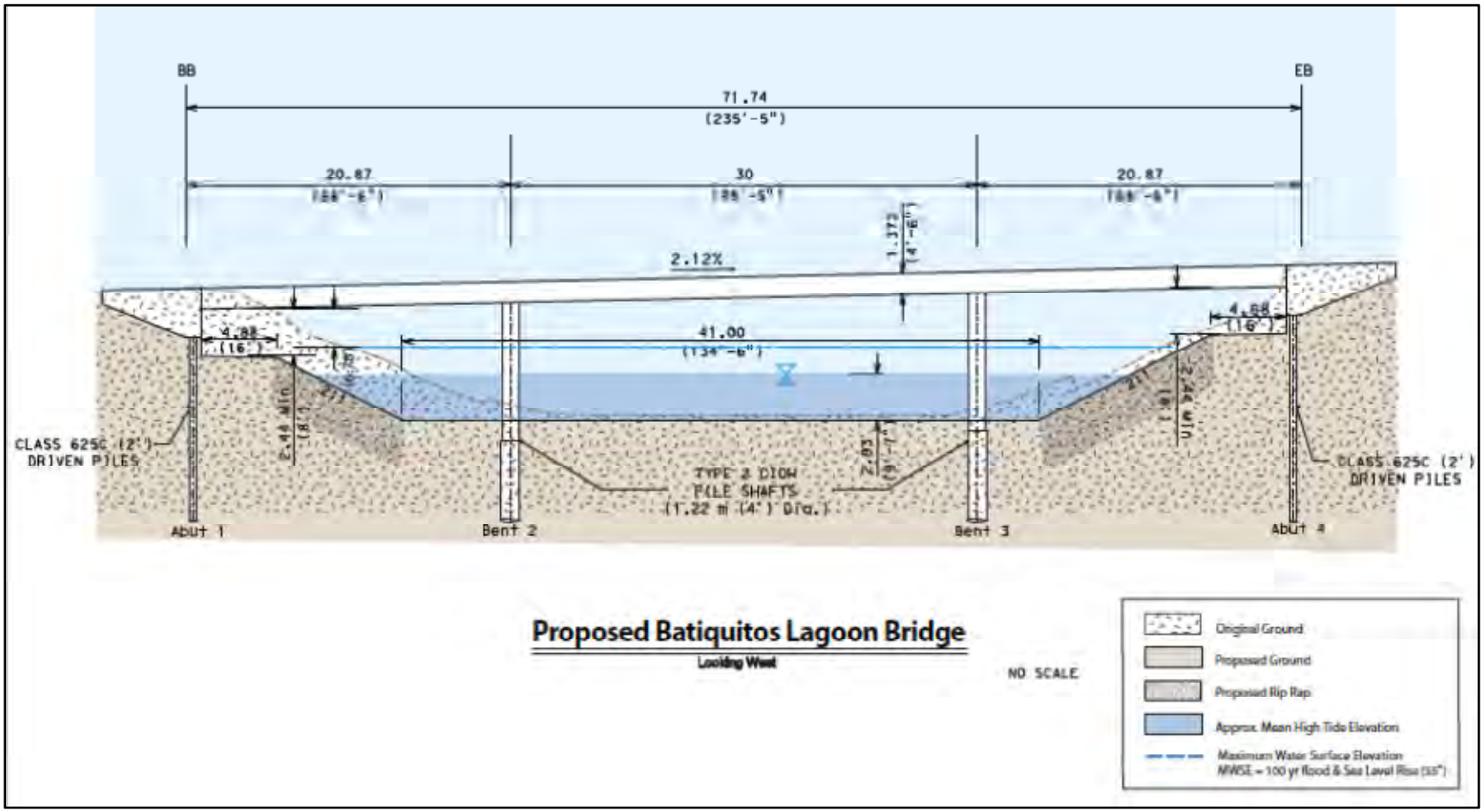

Figure 5-9: Proposed I-5 Bridge Exhibit (Looking from Lagoon to Ocean)

5.4 Summary of Channel Dimensions

Table 5-1 summarizes the recommended new channel dimensions (referred as the 'optimized channel dimensions') based on the tidal and flood optimization modeling results discussed in previous sections. The existing channel invert elevations are appropriate when they are dredged to the design condition. The dimensions of the tidal inlet channel are also included for reference although no optimization modeling is required or performed. The dimensions under the current design condition shown on record drawings are also included for each bridge.

The optimized channel dimensions are used in the following sections for analyses of flow velocity at bridge crossings, sedimentation patterns in the lagoon, tidal inundation frequency and residence time under the dry weather condition, and hydraulic impacts of predicted sea level rise.

Table 5-1: Summary of Existing and Optimized Channel Dimensions

Infrastructure	Recommended Based on Optimization			Design Condition		
	Channel Invert (ft)		Bottom Width (ft)	Channel Invert (ft)		Bottom Width (ft)
	NGVD	MLLW		NGVD	MLLW	
Inlet	-8.0	-5.7	96	-8.0	-5.7	96
RR	-7.0	-4.7	202	-7.0	-4.7	162
I-5	-7.0	-4.7	134	-7.0	-4.7	66

6.0 ANALYSES OF VELOCITY AND SEDIMENTATION

Analyses of velocity and sedimentation are performed for the following four lagoon bathymetry and bridge dimension conditions:

1. **Scenario1 - Shoaled Lagoon and Shoaled Existing Channels:** Both the lagoon and channels under existing bridge crossings are at the condition surveyed 2008.
2. **Scenario 2 - Dredged Lagoon and Dredged Existing Channels**: The lagoon is assumed to be dredged to the design condition, and assuming subsidence in the Central Basin as discussed in Section 1.2. The existing channels are dredged to their design conditions.
3. **Scenario 3 - Dredged Lagoon and Dredged Optimized Channels:** The lagoon is assumed to be dredged to the design condition assuming subsidence in the Central Basin as discussed in Section 1.2. The channels under the I-5 and RR Bridges with the optimized channel dimensions shown in Table 5-1 are at their design condition with no shoaling. The inlet entrance channel is also dredged to the design condition.
4. **Scenario 4 - Shoaled Lagoon and Dredged Optimized Channels:** The lagoon with the optimized channel dimensions under the I-5 and RR Bridges is assumed to be shoaled to the condition similar to that surveyed in 2008 since the shoaling pattern in the West and Central Basins will likely be similar to current conditions. Shoaling in the channels with optimized channel dimensions is expected to be different from that under the current condition. Because of largely unknown conditions, the channels under the I-5 and RR Bridges with the optimized channel dimensions shown in Table 5-1 are assumed to be at their design conditions with no shoaling. The inlet entrance channel is also assumed to be dredged to the design condition.

A comparison of Scenarios 1 and 2 would show the impact of shoaling in the lagoon. Comparison of Scenarios 2 and 3 would show the benefits of optimized channel dimensions when compared to the existing channel dimensions.

6.1 Analyses of Tidal Velocity Under Bridges

The section summarizes velocities under the bridges for both the dry weather (tidal only) and extreme storm (50-year and 100-year) conditions. Table 6-1 displays the spring high tide velocities at the infrastructure crossings during the dry weather condition under the four modeling scenarios, with their associated variations in lagoon bathymetry and bridge dimensions. The results indicate that the peak flood and ebb tidal flow velocities, especially under the I-5 Bridge, are generally lowered under the optimized bridge condition when compared to the existing condition. The peak velocities under the I-5 Bridge are lowered by at least 1 foot per second (fps) and are lower than those under the RR Bridge for the currently shoaled condition. Therefore, the lagoon bed erosion on both sides of the I-5 Bridge is expected to be significantly reduced compared to the existing condition. The velocities at the

RR Bridge are also reduced, but remain approximately the same at the tidal inlet. Maintaining a velocity at the tidal inlet which is similar to the existing condition is essential for inlet stability.

A comparison of modeling Scenarios 1 and 2 indicates that for existing bridge conditions lagoon dredging will also reduce tidal velocities at the I-5 and RR Bridge crossings as shown in Table 6-1. Therefore, more frequent channel dredging will also reduce erosion around the I-5 Bridge.

Scenarios 2 and 3 show effects of optimizing the channels under bridges. Both assume the lagoon is clear of sand (dredged). Tidal flow velocities decrease throughout the lagoon, with the exception of the tidal inlet.

Tidal velocities under Scenario 4 (shoaled condition) are lower than those under Scenario 3, the dredged lagoon bathymetry condition, since shoaling would reduce the tidal prism and consequently, the tidal flow velocities under bridges.

Table 6-1: Tidal Velocity (fps) at Bridge Crossings During the Dry Season

Modeling Scenario	Lagoon Bathymetry	Bridge Condition	Inlet		RR		I-5	
			Flood	Ebb	Flood	Ebb	Flood	Ebb
1	Shoaled	Existing Shoaled	4.4	5.6	3.7	4.3	4.3	3.9
2	Dredged	Existing Dredged	3.9	5.5	3.1	3.5	3.7	3.6
3	Dredged	Optimized Dredged	4.1	5.6	2.7	2.9	2.4	2.3
4	Shoaled	Optimized Dredged	3.8	5.1	2.6	2.7	2.2	2.0

6.2 Analyses of Extreme Flood Velocities Under Bridges

Table 6-2 summarizes the velocities at bridge crossings under the 100-year storm event superimposed on the 7-foot spring high tide scenario discussed in Section 4.0. The modeling runs were performed to determine the maximum water surface elevations for flood protection, but not to determine the potential maximum velocities at the bridge crossings. The maximum water surface elevation occurs at the high tidal elevation, while the maximum velocity occurs during the low tide. However, the velocities summarized in Table 6-2 still provide relative comparison between different lagoon and bridge conditions.

Velocities under both the I-5 and RR Bridges for the optimized bridge condition are lower than those under the existing bridge condition due to widening of the channels. However, the potential peak flood velocities will be higher than those shown if the peak flood arrives in a spring low tidal condition. Additional modeling analysis, which is beyond the scope of this study, is required to determine the maximum scour velocity and scour depth. With the magnitude of velocities at the bridge crossings, riprap protection of the bridge abutments and the entire

channel under both the RR and I-5 Bridges will still be required, similar to their current conditions. The storm flow velocity in the tidal inlet is expected to be higher under the optimized condition of the I-5 and RR Bridges since some backwater effects in the Central and East Basins are shifted to the West Basin. A slight increase in velocity in the inlet channel may not cause additional scour problems since the channel under the bridges is riprap protected.

Velocities under the dredged lagoon condition are lower than those under the shoaled condition. Frequent dredging of the lagoon will also improve flood conveyance and reduce peak flood velocity.

Table 6-2: 100-Year Peak Flood Velocity (fps) at Bridge Crossings

Modeling Scenario	Lagoon Bathymetry	Bridge Condition	Inlet	RR	I-5
1	**Shoaled**	**Existing Shoaled**	9.2	5.9	7.1
2	**Dredged**	**Existing Dredged**	7.6	5.0	6.6
3	**Dredged**	**Optimized Dredged**	8.0	4.5	4.8
4	**Shoaled**	**Optimized Dredged**	7.9	4.2	4.7

Table 6-3 summarizes velocities under a lesser frequency 50-year storm event. The conclusions are similar to those found for the 100-year storm event. Velocities under the I-5 and RR Bridges under the optimized bridge dimensions condition are lower than those under the existing bridge conditions. For the existing bridge dimensions condition, dredging of the lagoon and channels would also reduce the flood flow velocities. Under the 50-year storm event, the peak velocity in the tidal inlet also increases slightly.

Table 6-3: 50-Year Peak Flood Velocity (fps) at Bridge Crossings

Modeling Scenario	Lagoon Bathymetry	Bridge Condition	Inlet	RR	I-5
1	**Shoaled**	**Existing Shoaled**	7.6	5.3	5.2
2	**Dredged**	**Existing Dredged**	6.5	4.2	4.8
3	**Dredged**	**Optimized Dredged**	6.6	3.6	3.4
4	**Shoaled**	**Optimized Dredged**	6.4	3.3	3.3

6.3 Analyses of Sedimentation

Fluvial sedimentation has been gradually accumulating in the East Basin of Batiquitos Lagoon. With the current maintenance dredging program in place for the Lagoon, coastal sedimentation is not expected to pass beyond the Central Basin. On the other hand, coastal sedimentation is dominant in the Central and West Basins. The sediment accumulation in the Central and West Basins will be removed via maintenance dredging even if there is fluvial deposition in these areas. Analysis of coastal sedimentation is beyond the scope of this study. Another sedimentation-related issue for Batiquitos Lagoon is formation of deep scour holes present on both sides of the I-5 Bridge. This study therefore focuses discussion on fluvial sedimentation in the East Basin and sedimentation around the I-5 Bridge.

6.3.1 Dry Weather Sedimentation

The coastal sedimentation (shoaling) in the West and Central Basins is expected to be similar to that under existing conditions. The tidal inlet velocities under Scenario 3 (optimized channel dimensions) are slightly higher than those under Scenario 2 (existing channel dimensions). Therefore, the tidal inlet will be relatively more stable.

As shown in Table 6-1, both ebb and flood tidal velocities, which are believed to be responsible for the scour holes on both sides of the I-5 Bridge, are significantly reduced under Scenario 3 (optimized channel dimensions) compared to those under Scenario 2 (existing channel dimensions). Therefore, the erosion conditions are expected to be improved and scouring depths on both sides of the I-5 Bridge should be reduced under Scenario 3 (optimized channel dimensions).

As discussed previously, lagoon dredging will also result in lower tidal velocities at the I-5 and RR Bridge crossings. Consequently, more frequent channel dredging should also reduce erosion around the I-5 Bridge.

6.3.2 Extreme Storm Event Sedimentation

This section summarizes changes in the sedimentation pattern and potential sediment transport in the East Basin. Figure 6-1 and Figure 6-2 show the velocity contours in the lagoon during the 100-year stormflow event. The velocity increases slightly under the optimized bridge dimension condition. Therefore, the flood conveyance and sediment transport capacity will be slightly improved in the East Basin, which should reduce fluvial sedimentation in the East Basin. As shown in Table 6-4, the 100-year peak flood travel time through the East Basin is reduced from 0.7 hours to 0.6 hours with a widened channel under the I-5 Bridge, which would reduce the time for sediment to settle in the lagoon. Therefore, the flood conveyance and sediment transport capacity under the optimized bridge condition should be slightly improved compared to the existing condition, and may likely result in slightly reduced sedimentation in the East Basin. Dredging of the lagoon should also reduce flood travel time in the Central and West Basins

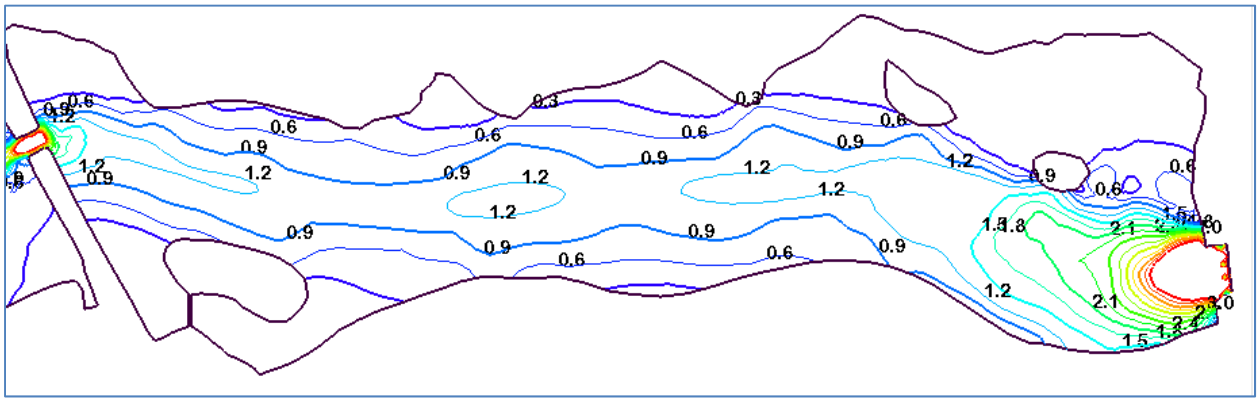

Figure 6-1: 100-Year Velocity Contours for Dredged Lagoon and Existing Bridge Dimension Condition

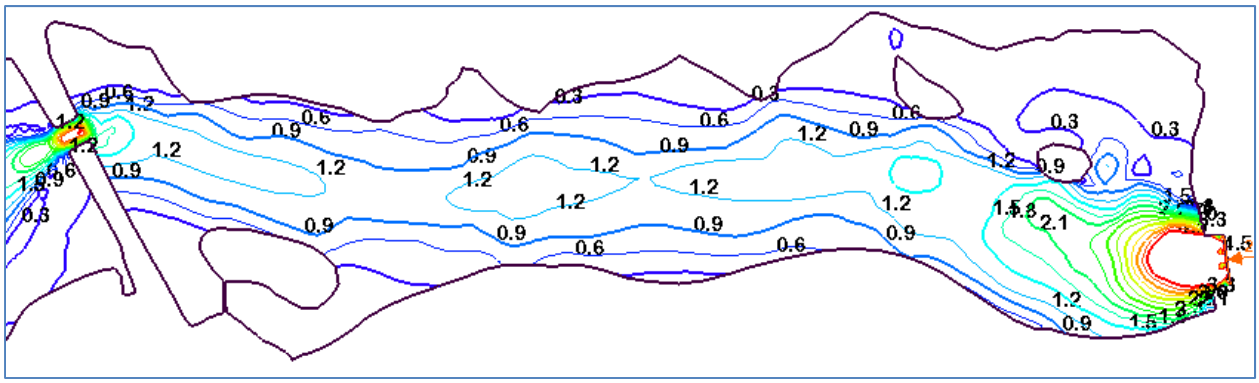

Figure 6-2: 100-Year Velocity Contours for Dredged Lagoon and Optimized Bridge Dimension Condition

Table 6-4: Duration (Hour) of Stormflow Drainage Under a 100-Year Storm

Modeling Scenario	Lagoon Bathymetry	Bridge Condition	East Basin	Central Basin	West Basin
1	Shoaled	Existing Shoaled	0.8	1.2	0.5
2	Dredged	Existing Dredged	0.7	0.8	0.4
3	Dredged	Optimized Dredged	0.6	0.6	0.2
4	Shoaled	Optimized Dredged	0.6	0.6	0.2

7.0 RESIDENCE TIME ANALYSES

The RMA-4 model is used in the study to calculate the residence time. The dispersion coefficients used in the RMA-4 model are based on modeling calibrations performed for other similar projects as no data are available for the model calibration. This is adequate for the purpose of comparison between existing and optimized project conditions.

7.1 Methodology

Changes in constituent concentrations in a water body reflect a balance between the rate of constituent supply and the rate of constituent removal by tidal flushing. Residence time (i.e., average time a particle resides in a hydraulic system) provides a useful measure of the rate at which waters in the hydraulic system are renewed. Accordingly, residence time provides a means for assessing the water quality of the hydraulic system.

Consider the reduction of a tracer concentration in a tidal embayment due to flushing after being released (Fisher et al., 1979), in which C_0 is initial concentration, K is a reduction coefficient and C(t) is the concentration at time t.

$$C(t) = C_0 e^{-Kt} \tag{7.1}$$

The residence time of the tracer in the embayment is determined as follows:

$$T_r = \frac{\int_0^\infty t\, C(t)\, dt}{\int_0^\infty C(t)\, dt} = \frac{1}{K}\,. \tag{7.2}$$

Since the concentration at t = T_r is

$$C(T_r) = C_0 e^{-1} = \frac{C_0}{e} \tag{7.3}$$

T_r can be calculated from a regression analysis of the tracer concentration time series computed by the numerical model RMA-4.

Based on the above methodology, the general procedure for computing residence times for different parts of a tidal embayment is as follows:

- Assign an initial constituent concentration of one over the entire embayment element mesh (wetlands for this study) and a value of zero at the open water boundaries to simulate an instantaneous release of a new constituent into an embayment.
- Run the numerical model RMA-4 for an adequate number of tidal cycles until substantial reduction of constituent concentrations have occurred due to tidal flushing at the locations of interest.
- Analyze the computed concentration results by regression analysis to obtain the constituent reduction distributions at the locations of interest.

- Find the residence times for the locations of interest from the distribution curves according to Equations 7.1 through 7.3.

Figure 7-1 shows an example of how the method works, where the zigzagging solid blue line shows the direct results from RMA-4 and the dashed green line shows the daily moving average results. An arrow points to the amount of time it takes for the moving average to fall below the threshold concentration of 1/e, which in this example represents a residence time of approximately 173 hours. This method was used in the project study for all scenarios.

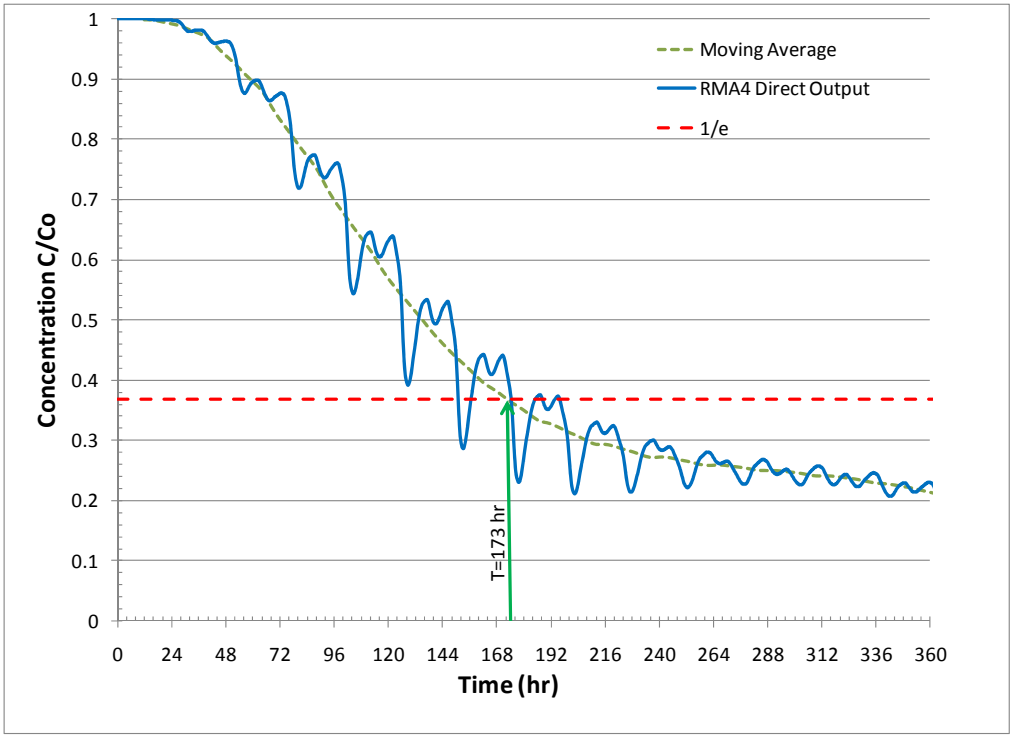

Figure 7-1: Example of a Residence Time Plot

7.2 Boundary Conditions

7.2.1 Hydraulic Input

The 15-day modeling tidal series, representing the average spring and neap tidal cycle, as described in Section 2.2.4.2 is applied as the offshore driving tide. No runoff from the fresh water boundary is considered, as the base flow of the creek is negligibly small.

7.2.2 Concentration Input

An initial constituent concentration of one is specified for the entire lagoon. No constituent concentration is assigned at the open water boundaries. Also, it is assumed that ocean water is clean and does not supply additional constituents, or “contaminants.”

7.3 Residence Time Results

Residence times are calculated at representative gage locations shown in Figure 7-2. The lagoon is well circulated in both the West and Central Basins. The difference in residence time between Gages WB1 and WB2 is very small and less than 0.1 day. Similarly, the residence time at Gage CB1 is very similar to that at CB2. Therefore, since the residence time value at each station is the same within a basin, only one residence time value is reported. Table 7-1 summarizes residence times under the four scenarios described in Section 6.0. The residence times are very similar for existing and optimized channel dimension conditions. The overall residence times are short, being less than one week, indicating that tidal waters within Batiquitos Lagoon circulate well. However, dredging of the lagoon will reduce residence times in the East Basin by one half of a day and further enhance lagoon circulation.

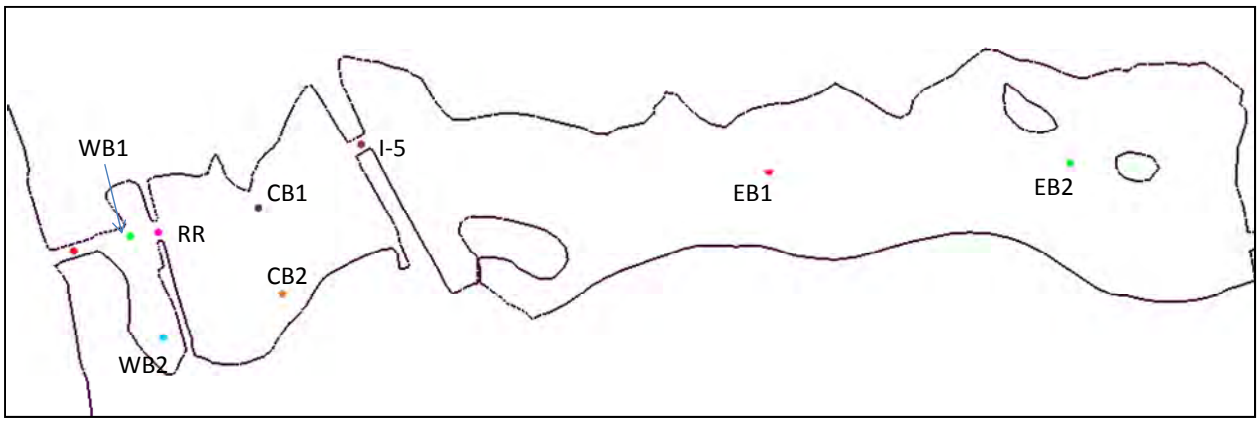

Figure 7-2: Gage Locations for Residence Time Calculations

Table 7-1: Summary of Residence Time (Days)

Modeling Scenario	Lagoon Bathymetry	Bridge Condition	West Basin	Central Basin	East Basin	
					EB1	EB2
1	Shoaled	Existing Shoaled	0.6	1.6	4.3	5.8
2	Dredged	Existing Dredged	0.6	1.6	3.8	5.4
3	Dredged	Optimized Dredged	0.5	1.6	3.8	5.4
4	Shoaled	Optimized Dredged	0.5	1.6	4.5	5.9

8.0 TIDAL INUNDATION FREQUENCY ANALYSES

Tidal inundation frequency is analyzed and plotted with tidal elevation data from the TEA tidal model runs. The tidal range difference within each basin is very small, so only one inundation frequency is plotted for each basin. Figure 8-1 through Figure 8-4 show the inundation frequency plots. There is no high tide muting in Batiquitos lagoon. However, the lagoon does experience low tidal muting, especially under the shoaled lagoon condition. Dredging would reduce muting by 0.4 feet in the Central Basin and by 0.7 feet in the East Basin, and increase the vertical range of the intertidal habitat zone. Optimizing the channel dimensions under the RR and I-5 Bridges would further reduce tidal muting by approximately 0.2 feet and add to the increase in vertical range of the intertidal habitat zone. For Batiquitos Lagoon, the primary gain of intertidal habitat area will be mudflat. Mudflat lies from an inundation frequency of approximately 100 to 40 percent.

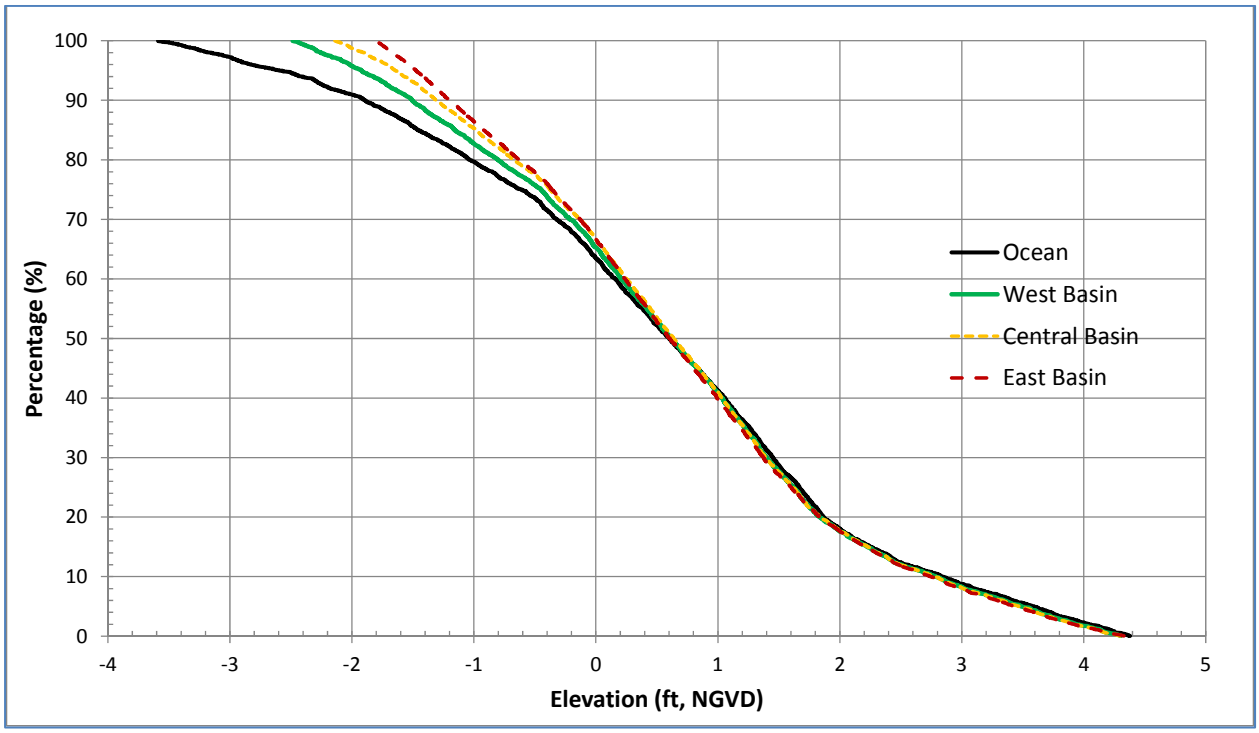

Figure 8-1: Inundation Frequency for Shoaled Existing Condition

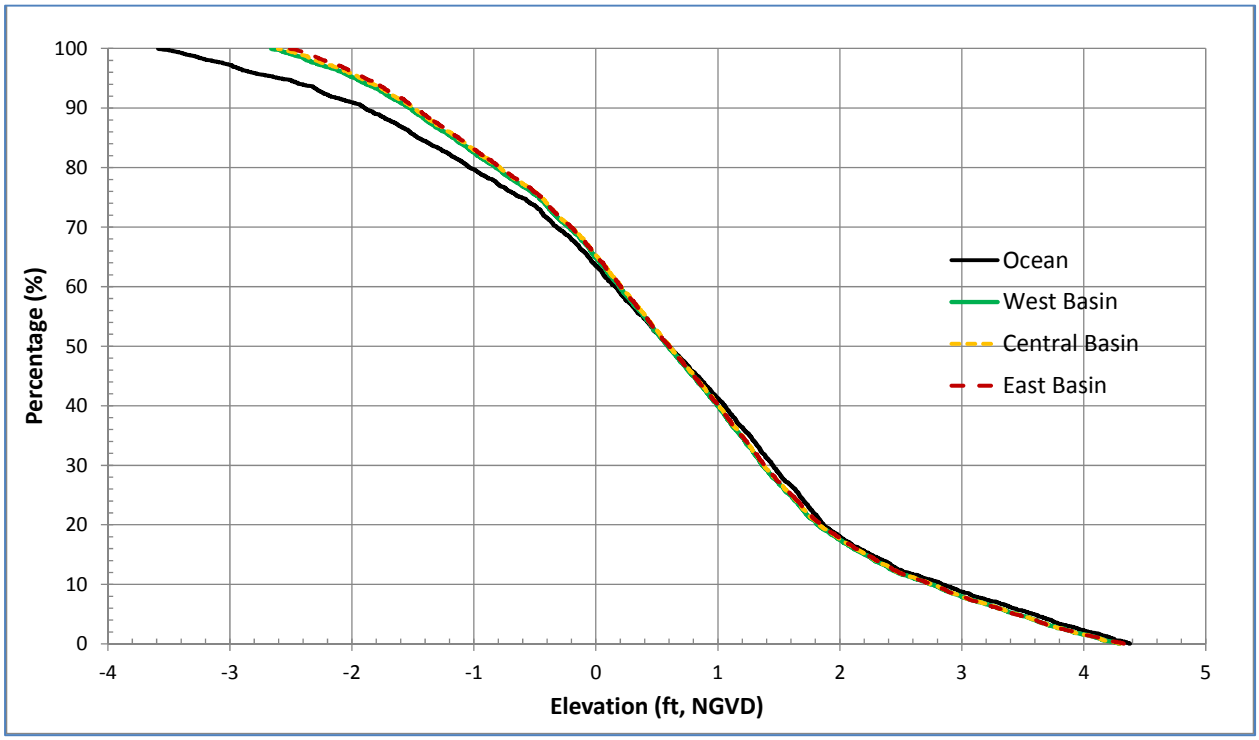

Figure 8-2: Inundation Frequency for the Dredged Existing Condition

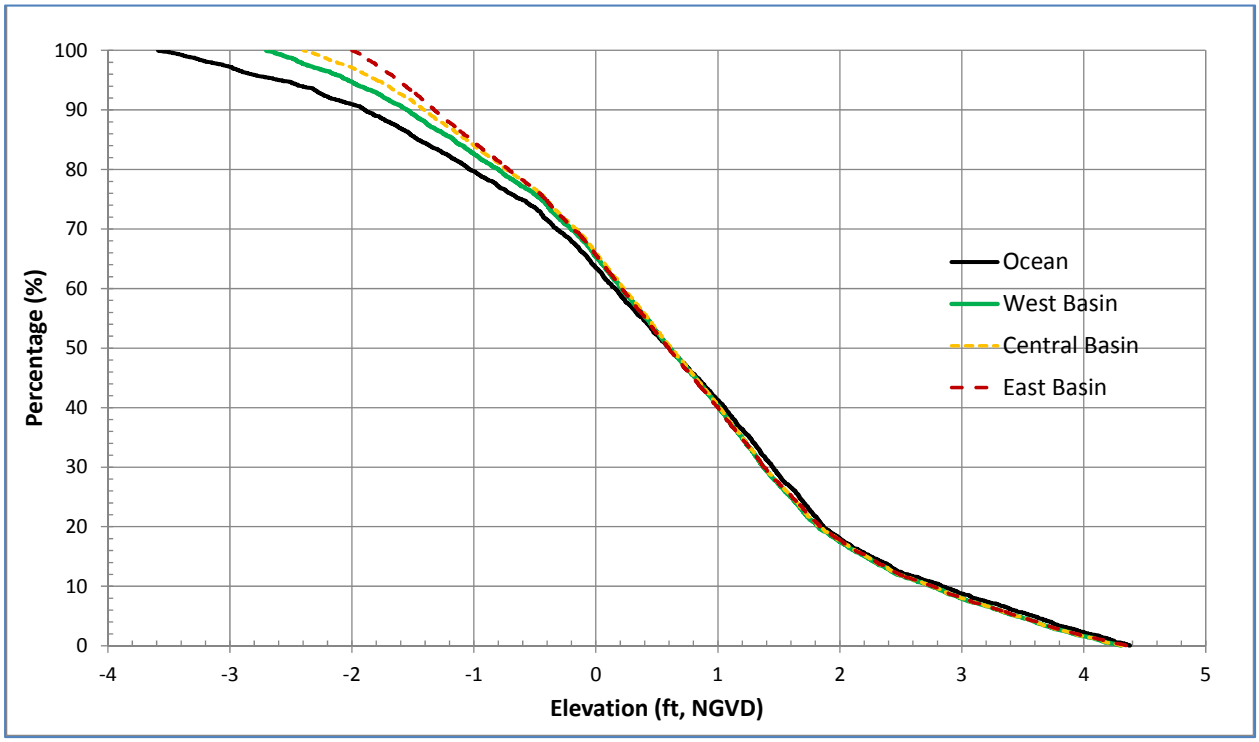

Figure 8-3: Inundation Frequency for the Shoaled Optimized Condition

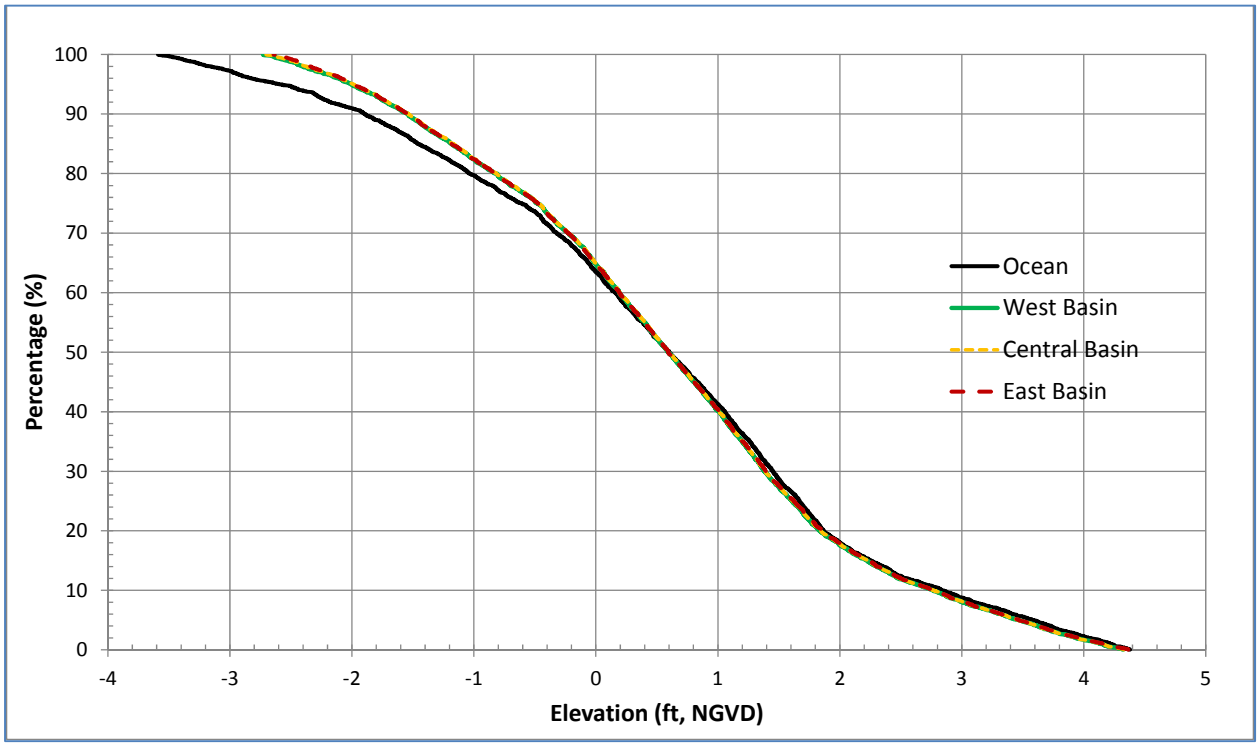

Figure 8-4: Inundation Frequency for the Dredged Optimized Condition

9.0 HYDRAULIC EFFECTS OF SEA LEVEL RISE

Hydrodynamic modeling runs were performed to consider sea level rise (SLR) predicted for the year 2100. A 55-inch SLR estimate was considered in the modeling study based on the guidance provided by Caltrans internal guidance (Caltrans 2011) and the California State Coastal Conservancy on its web site (CSCC 2012) for horizon year 2100. The offshore spring high tide series (with a high tide elevation of 4.69 feet NGVD) was raised linearly upward by 55 inches to form the spring high tide series in year 2100 (future high tide elevation of 9.27 feet NGVD).

The offshore high tide base level of 4.69 feet used for modeling of SLR compares to a base level of 7.0 feet used for stormflow modeling under existing conditions. The ocean base level for SLR modeling is therefore different, and 28 inches lower, than that assumed for existing conditions stormflow modeling. The difference is the omission of the value of wave run-up from the SLR modeling base level. Wave run-up is not included because it is too conservative to assume that breaking waves would exist at the Lagoon mouth during combined maximum high tide and SLR conditions based on engineering judgment. Water depths at the Lagoon mouth are estimated to be sufficient to preclude wave breaking within the tidal inlet channel.

The resulting tidal series is shown in Figure 9-1. It is also assumed that the 100-year stormflow condition, as shown in Figure 2-6, will be the same as it is today.

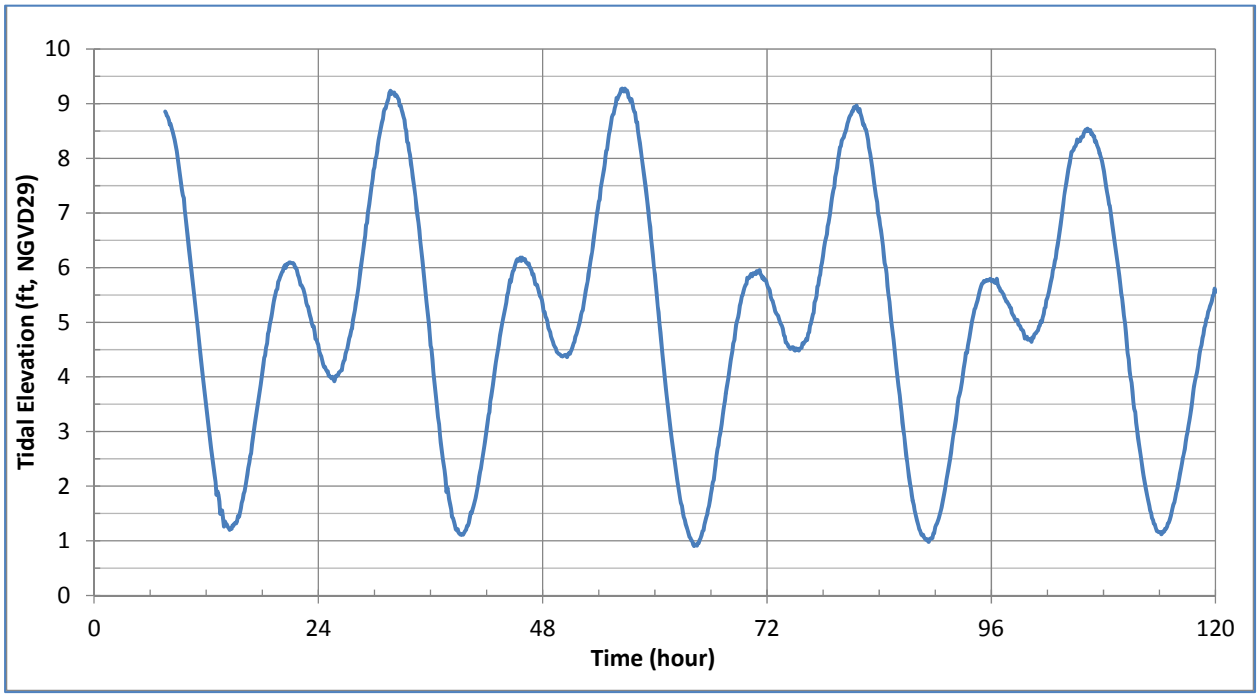

Figure 9-1: Spring High Tidal Series for Year 2100

Figure 9-2 compares 100-year water surface profiles shown in “warm” color lines in the year 2100 with predicted sea level rise. The 100-year water surface profiles in the year 2012, shown

in “cold” color lines, are also included for relative comparison. The water surface elevation will be higher, but head losses through each bridge will be less than those under the current condition. The water surface elevation in the East Basin will be lower with optimized bridge dimensions than with existing bridge dimensions. However, the water surface elevation in the Central and West Basins will be higher with the optimized bridge dimensions than with existing bridge dimensions.

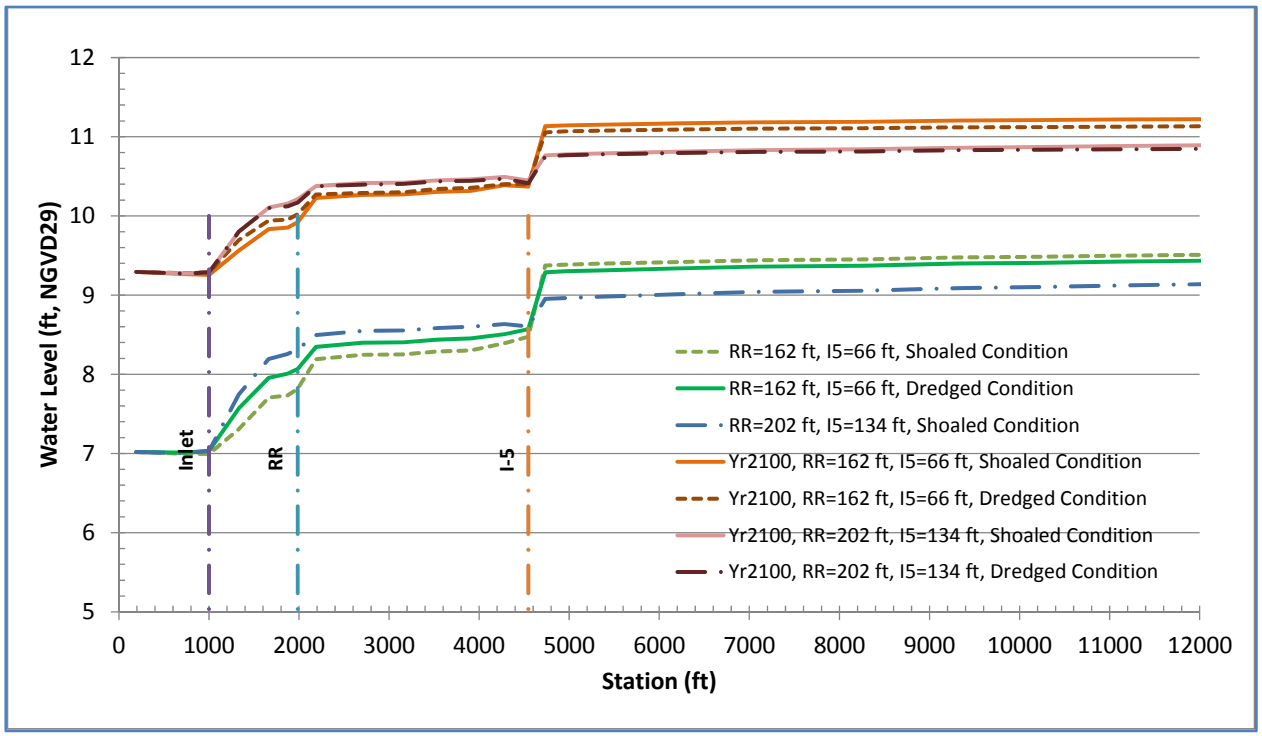

Figure 9-2: 100-Year Surface Profile Comparison with Sea Level Rise

In order to predict the maximum flood water elevation at the I-5 Bridge, a series of iterative modeling runs were performed by adjusting the phase of the flood peak to arrive simultaneous with the spring high tide. The same procedure was also repeated separately for both the RR Bridge and the Carlsbad Boulevard Bridges, such that the water levels at the RR and the East Carlsbad Boulevard Bridge are maximized since the flood travel time from the model upstream boundary to each bridge crossing is different. The maximum water surface elevations at the upstream side (eastside edge of the bridge) of the bridges were extracted from the modeling results and summarized in Table 9-1. The last row of the table summarizes the existing bridge soffit elevations (M&N 1993). The soffit elevation of the East Carlsbad Boulevard Bridge is used for the inlet constraint since the West Carlsbad Boulevard Bridge is about 1.8 feet higher in elevation. The 100-year water surface elevations in both year 2012 and 2100 are included in the Table. The water surface elevations in year 2100 take into consideration 55-inches of SLR. The water surface elevations upstream of the bridges are higher than those at the bridge crossings, as shown in the preceding profile Figures. The 100-year water surface elevation touches the east-end soffit of the East Carlsbad Boulevard Bridge as shown in Figure 5-4 in year 2100, but it does not create pressurized flow. Existing Carlsbad Boulevard Bridge has

approximately 1.5 feet of freeboard above the 100-year maximum water levels under current conditions. Sufficient freeboard exists above the maximum flood elevations for the I-5 and RR Bridges under both current and future SLR scenarios.

Table 9-1: Summary of Bridge Soffit and 100-Year Surface Water Elevations

Modeling Scenario	Lagoon Bathymetry	Bridge Condition	Inlet (ft, NGVD)		RR (ft, NGVD)		I-5 (ft, NGVD)	
			Year-2012	Year-2100	Year-2012	Year-2100	Year-2012	Year-2100
1	Shoaled	Existing Shoaled	7.1	9.3	7.9	10	8.9	10.7
2	Dredged	Existing Dredged	7.5	9.6	8.1	10.1	8.8	10.6
3	Dredged	Optimized Dredged	7.6	9.7	8.3	10.2	8.6	10.5
4	Shoaled	Optimized Dredged	7.6	9.7	8.4	10.3	8.7	10.5
Bridge Soffit Elevation (ft, NGVD)			9.2		17.3		16.1	

Table 9-2 summarizes 100-year peak flood velocities at bridge crossings under both current and future SLR scenarios. The velocities will be slightly lower under the future SLR scenario than under current conditions because the tidal inlet cross-sectional area is larger under the SLR condition than under existing conditions.

Table 9-2: 100-Year Peak Flood Velocity (fps) at Bridge Crossings

Modeling Scenario	Lagoon Bathymetry	Bridge Condition	Inlet		RR		I-5	
			Year-2012	Year-2100	Year-2012	Year-2100	Year-2012	Year-2100
1	Shoaled	Existing shoaled	9.2	8.5	5.9	5.5	7.1	6.5
2	Dredged	Existing dredged	7.6	7.2	5.0	4.9	6.6	6.3
3	Dredged	Optimized dredged	8.0	7.5	4.5	4.4	4.8	4.5
4	Shoaled	Optimized dredged	7.9	7.4	4.2	4.1	4.7	4.4

10.0 FINDINGS AND RECOMMENDATIONS

Channel dimensions (width and depth) under the I-5 and RR Bridges were optimized to achieve the optimal tidal range and flood conveyance in Batiquitos Lagoon in order to support optimal ecosystem, lagoon circulation and sediment transport conditions. The tidal inlet at Carlsbad Boulevard Bridges has been performing well since construction in 1995, so no further optimization is required for that channel. A summary of findings and recommendations is below.

1. Dredging of the lagoon and channels under the bridges is an effective way to increase the tidal range and reduce tidal velocities under the bridges. Simply dredging the lagoon to its design condition will increase the tidal range by 0.4 feet in the Central Basin and by 0.7 feet in the East Basin, and will reduce the tidal velocity by more than 0.5 fps.
2. The current channel invert elevation of -7 feet NGVD for both the RR and I-5 Bridges is appropriate and is the optimal channel invert elevation.
3. The recommended optimal channel bottom width under the I-5 Bridge is 134 feet, which is 68 feet wider than its current channel width.
4. The recommended optimal channel bottom width under the RR Bridge is 202 feet, which is 40 feet wider than the existing channel width.
5. The tidal range will increase by 0.2 feet in both the Central and East Basins with the optimized channel dimensions under both the I-5 and RR Bridges, compared to those under the existing dredged condition.
6. The flood water surface elevation with the optimized channel dimensions will be lowered in the East Basin, but will be raised in the Central and West Basins compared to those under the existing condition.
7. Tidal flow velocities at the bridge crossings with the optimized channel dimensions will be lowered, especially at the I-5 Bridge, compared to those under the existing condition. This should significantly reduce the scour depth on both sides of the I-5 Bridge. The storm flood velocities will also be lowered.
8. Fluvial sediment transport in the East Basin under the optimized condition should be slightly improved than under existing conditions due to reduced backwater effects and the shortened flood travel time through the East Basin.
9. Residence time, a measure of tidal circulation, is relatively short for Batiquitos Lagoon. In the West Basin the residence time is approximately one half of a day. It gradually increases to approximately 1.5 days in the Central Basin and to about 5.5 days in the East Basin. A residence time of less than one week is considered good for an estuary wetland system. The tidal circulation in Batiquitos Lagoon is good, but can be further enhanced with maintenance dredging.
10. Under the optimized channel dimensions condition the tidal inundation frequency curve is very similar to that under existing conditions. The vertical range of the intertidal habitats would increase slightly under the optimized channel dimensions condition. The

study shows that dredging would increase the vertical tidal range (therefore, the intertidal habitat) by approximately 0.5 feet in the Central Basin and approximately 0.7 feet in the East Basin.

11. In year 2100 with projected SLR, channels under both the existing and optimized I-5 and RR Bridges would pass the 100-year flood with a more than 3 feet of freeboard. However, the east-end soffit of the East Carlsbad Boulevard Bridge will be just below the 100-year flood water level. Flood velocities under the SLR scenario at all three bridge crossings will be lower than those under the current time horizon.

11.0 REFERENCES

California Coastal Records Project. 2012. http://www.large.images.californiacoastline.org /images/2002/large/8/9138.JPG.

California State Coastal Conservancy. 1987. Batiquitos Lagoon Watershed Sediment Control Plan.

California State Coastal Conservancy. 2012. http://scc.ca.gov/2009/01/21/coastal-conservancy-climate-change-policy-and-project-selection-criteria/.

Caltrans. 2011. Guidance on Incorporating Sea-level Rise. Prepared by the Caltrans Climate Change Workgroup and the HQ Divisions of Transportation Planning, Design, and Environmental Analysis. May 16, 2011.

Caltrans. 2012. Proposed Batiquitos Lagoon I-5 Bridge Section. Caltrans, District 11.

Chaudhry, M. Hanif. 1993. Open-Channel Flow. Prentice-Hall, Englewood Cliffs, New Jersey.

Fischer, H.B., List, E.J., et. al., 1979. "Mixing in Inland and Coastal Waters", Academic Press, Inc., 1979.

McAnally, W.H. and Thomas, W.A. 1985. User's Manual for the Generalized Computer Program System, Open Channel Flow and Sedimentation, TABS-2 Main Text. U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS.

Merkel & Associates. 2009. Batiquitos Lagoon Long-Term Biological Monitoring Program. Final Report. 2009.

Moffatt & Nichol. 1990. Batiquitos Lagoon Enhancement Plan, Phase I, Preliminary Study Summary Report.

Moffatt & Nichol. 1993. Batiquitos Lagoon Enhancement Plan Set, prepared for City of Carlsbad and Port of Los Angeles.

Moffatt & Nichol. 1997. Batiquitos Lagoon As-Built Plans, prepared for City of Carlsbad and Port of Los Angeles.

Moffatt & Nichol. 2012. San Elijo Lagoon Bridge Optimization Study, Final Report.April 2012.

National Oceanic and Atmospheric Administration. 2005. Oceanic and Atmospheric Research Pacific Marine Environmental Laboratory, Center for Tsunami Inundation Mapping Efforts, http://nctr.pmel.noaa.gov/index.html.

National Oceanic and Atmospheric Administration. 2011. Center for Operational Oceanographic Products and Services, website: http://tidesandcurrents.noaa.gov/.

U.S. Army Corps of Engineers, Los Angeles District. 1971. Flood Plain Information: San Marcos Creek, vicinity of San Marcos, San Diego County, California.

U.S. Army Corps of Engineers. 2009. Users Guide To RMA-2 WES Version 4.5. Engineer Research and Development Center, Waterways Experiment Station, Coastal and Hydraulics Laboratory.

WRA, Inc. 2010. Topographic and Vegetation Analysis of Batiquitos Lagoon and Agua Hedionda Lagoon, I-5 North Coast Corridor Project, San Diego County, California. Prepared for Caltrans, District 11 and San Diego Association of Governments. August 2010.

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

I-5/SR78 Interchange Preliminary Engineering

I-5 BRIDGE STUDY AT BUENA VISTA LAGOON

FLUVIAL HYDRAULICS AND RESIDENCE TIME ANALYSIS

FINAL REPORT

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TABLE OF CONTENTS

1 INTRODUCTION 1.1
1.1 Background 1.1
1.2 Purpose 1.1
1.3 Objectives 1.2
2 ALTERNATIVES ANALYZED 2.1
2.1 Overview 2.1
2.2 Salt Water Alternatives 2.1
2.2.1 Alt 2-1 ..... 2.1
2.2.2 Alt SW2-A 2.2
2.3 Fresh Water Alternatives 2.6
2.3.1 Alt 1 ..... 2.6
2.3.2 Alt FW-A 2.6
3 FLUVIAL HYDRAULICS METHODOLOGIES 3.1
3.1 Fluvial Hydraulics Analysis 3.1
3.2 Flood Hydrograph 3.1
3.3 Tidal Hydrograph 3.3
3.4 Sea Level Rise Analysis 3.3
3.5 As-Built Hydraulic Connections 3.5
3.6 Manning's Coefficient 3.5
4 BRIDGE ANALYSIS FOR LAGOON RESTORATION 4.1
4.1 Overview 4.1
4.2 Salt Water Alternative Analysis 4.1

4.2.1 Alt 2-1 Analysis 4.1

4.2.2 Alt SW2-A Analysis 4.9

4.3 Fresh Water Alternative Analysis 4.17

4.3.1 Alt 1 Optimization 4.17

4.3.2 Alt FW-A Analysis 4.25

4.4 Supplemental Analysis 4.33

4.4.1 Impact of Marine Organisms Colonizing Structural Members 4.33

4.4.2 Peak Fluvial and Tidal Flows Time Phasing Variation Analysis 4.34

4.4.3 Channel Lining at Bridge Connections 4.35

4.4.4 Width of the Weir at the Ocean Outlet 4.35

4.4.5 Maximum Water Elevation 4.35

4.5 Analysis for Minor Storms 4.35

4.6 Summary and Recommendations 4.36

5 RESIDENCE TIME ANAYSIS 5.1

5.1 Study Approach 5.1

5.2 Model Setup 5.2

5.3 Tidal Flushing 5.4

5.4 Summary 5.8

6 CONCLUSIONS 6.1

7 REFERENCES 7.1

LIST OF FIGURES

Figure 2.1 Buena Vista Lagoon 2.3

Figure 2.2 Alternative 2-1 Plan View 2.4

Figure 2.3 Alternative SW2-A Plan View 2.5

Figure 2.4 Alternative 1 Plan View 2.7

Figure 2.5 Alternative FW-A Plan View 2.8

Figure 3.1 Flood and Diurnal Tide Hydrographs 3.2

Figure 3.2 Diurnal Tide Hydrographs for Year 2011 and Year 2100 ..... 3.4

Figure 4.1 Alt 2-1 HEC-RAS Results for Cases with Various Channel Elevations 4.3

Figure 4.2 Alt 2-1 HEC-RAS Results for Cases with Various Span Lengths 4.5

Figure 4.3 Alt 2-1 HEC-RAS Results for Analysis Cases without Sea Level Rise 4.7

Figure 4.4 Alt 2-1 HEC-RAS Results for Sea Level Rise Analysis 4.9

Figure 4.5 Alt SW2-A HEC-RAS Results for Cases with Various Bridge Channel Elevations 4.11

Figure 4.6 Alt SW2-A HEC-RAS Results for Cases with Various Span Lengths 4.13

Figure 4.7 Alt SW2-A HEC-RAS Results for Analysis Cases without Sea Level Rise 4.15

Figure 4.8 Alt SW2-A HEC-RAS Results for Sea Level Rise Analysis 4.17

Figure 4.9 Alt 1 HEC-RAS Results for Cases with Various Bridge Channel Elevations 4.19

Figure 4.10 Alt 1 HEC-RAS Results for Cases with Various Span Lengths 4.21

Figure 4.11 Alt 1 HEC-RAS Results for Analysis Cases without Sea Level Rise 4.23

Figure 4.12 Alt 1 HEC-RAS Results for Sea Level Rise Analysis 4.25

Figure 4.13 Alt FW-A HEC-RAS Results for Cases with Various Channel Elevations 4.27

Figure 4.14 Alt FW-A HEC-RAS Results for Cases with Various Span Lengths 4.29

Figure 4.15 Alt FW-A HEC-RAS Results for Analysis Cases without Sea Level Rise . 4.31

Figure 4.16 Alt FW-A HEC-RAS Results for Sea Level Rise Analysis 4.33

Figure 4.17 Fluvial Hydraulic Analyses for Minor Storms for Alt SW2-A Best Case 4.36

Figure 4.18 Desired Channel Cross Section of Railroad Bridge at Buena Vista Lagoon 4.39

Figure 4.19 Desired Channel Cross Section of Coast Highway Bridge at Buena Vista Lagoon 4.40

Figure 4.20 Desired Channel Cross Section of I-5 Bridge at Buena Vista Lagoon 4.41

Figure 4.21 Design Guidance for Channel Cross Section of Railroad Bridge at Buena Vista Lagoon 4.44

Figure 4.22 Design Guidance for Channel Cross Section of Coast Highway Bridge at Buena Vista Lagoon 4.45

Figure 4.23 Design Guidance for Channel Cross Section of I-5 Bridge at Buena Vista Lagoon 4.46

Figure 5.1 Buena Vista Lagoon Bathymetry used in Model Grids for Salt Water Alternatives 5.3

Figure 5.2 Alternative 2-1 Tidal Response 5.5

Figure 5.3 Alternative SW2-A Tidal Response 5.6

Figure 5.4 Tracer Concentrations in Weir Basin 5.7

Figure 5.5 Residence Times for Salt Water Alternatives 5.9

LIST OF TABLES

Table 3.1 Tidal Benchmarks and Tidal Datum 3.3

Table 3.2 Approximate Structure Dimensions for As-Built Conditions 3.5

Table 4.1 Parameters used in Alt 2-1 Bridge Channel Elevation Sensitivity Analysis 4.2

Table 4.2 Parameters used in Alt 2-1 Bridge Span Length Sensitivity Analysis 4.4

Table 4.3 Parameters used in Alt 2-1 Bridge Analysis 4.6

Table 4.4 Parameters used in Alt 2-1 Sea Level Rise Analysis 4.8

Table 4.5 Parameters used in Alt SW2-A Bridge Invert Elevation Sensitivity Analysis 4.10

Table 4.6 Parameters used in Alt SW2-A Bridge Span Length Sensitivity Analysis. 4.12

Table 4.7 Parameters used in Alt SW2-A Bridge Analysis 4.14

Table 4.8 Parameters used in Alt SW2-A Sea Level Rise Analysis 4.16

Table 4.9 Parameters used in Alt 1 Bridge Channel Elevation Sensitivity Analysis . 4.18

Table 4.10 Parameters used in Alt 1 Bridge Span Length Sensitivity Analysis 4.20

Table 4.11 Parameters used in Alt 1 Bridge Analysis 4.22

Table 4.12 Parameters used in Alt 1 Sea Level Rise Analysis 4.24

Table 4.13 Parameters used in Alt FW-A Bridge Channel Elevation Sensitivity Analysis 4.26

Table 4.14 Parameters used in Alt FW-A Bridge Span Length Sensitivity Analysis ... 4.28

Table 4.15 Parameters used in Alt FW-A Bridge Analysis 4.30

Table 4.16 Parameters used in Alt FW-A Sea Level Rise Analysis 4.32

Table 4.17 Comparison of Maximum Water Elevations Resulted from Peak Flow Phasing Variations 4.34

Table 4.18 Desired Bridge Parameters and Maximum Water Elevations for Salt Water Alternatives 4.37

Table 4.19 Desired Bridge Parameters and Maximum Water Elevations for Fresh Water Alternatives 4.38

Table 4.20 Lagoon Restoration Design Guidance for Bridge Dimensions 4.43

Table 5.1 Bridge Structure Dimensions for Alt 2-1 ..... 5.2

Table 5.2 Bridge Structure Dimensions for Alt SW2-A 5.2

Table 5.3 Residence Time Analysis Results 5.8

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

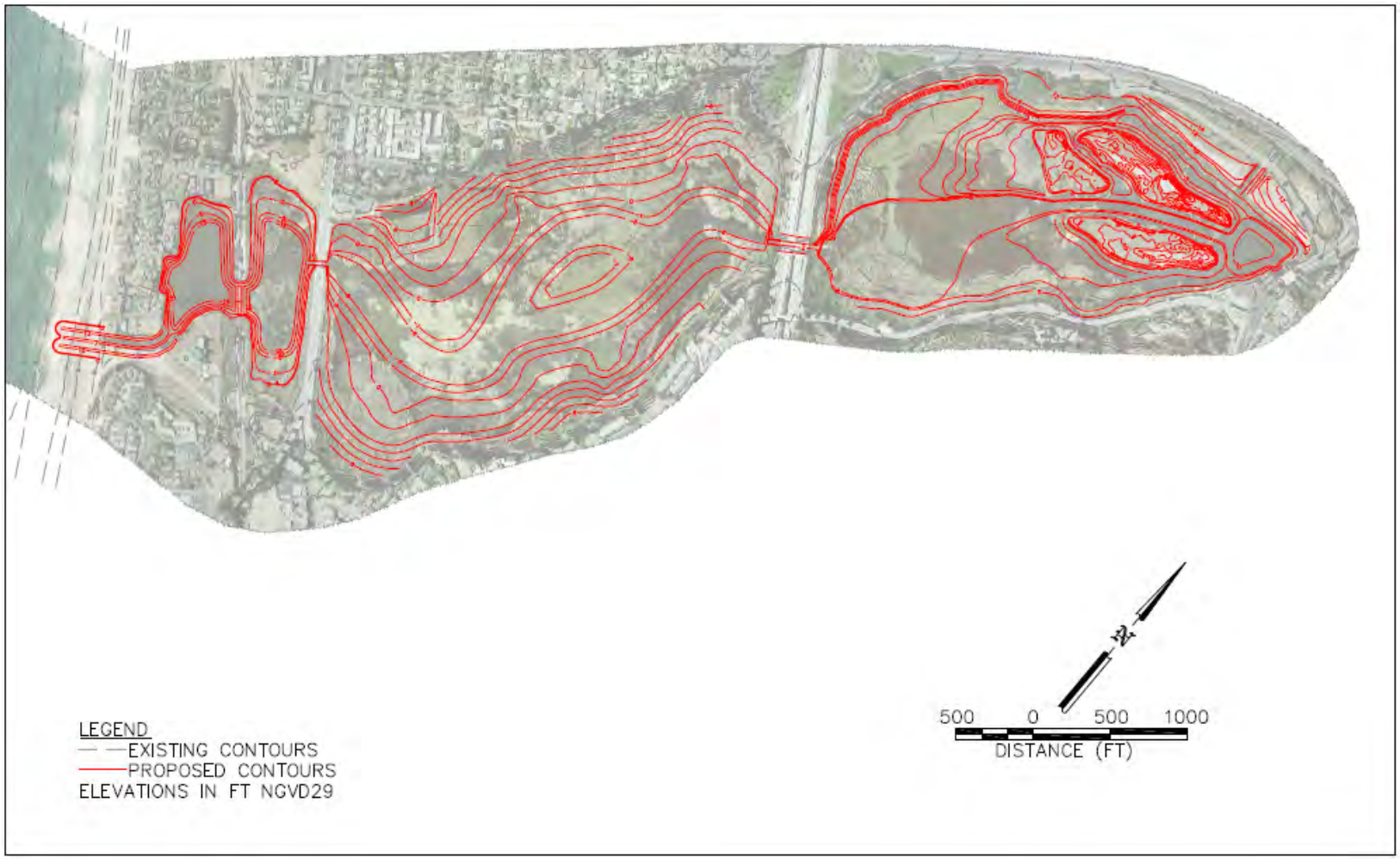


Figure 2.2 Alternative 2-1 Plan View

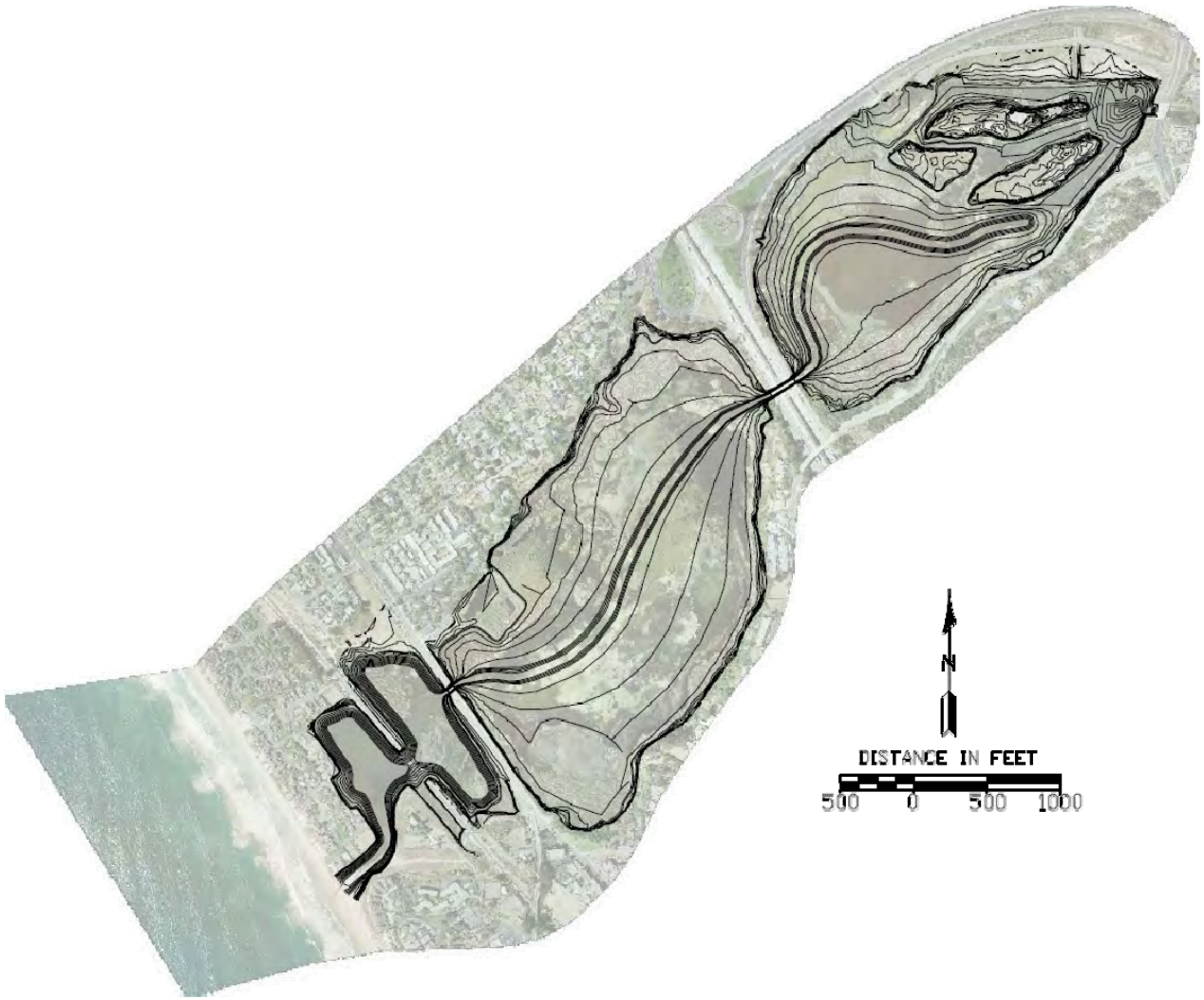


Figure 2.3 Alternative SW2-A Plan View

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.

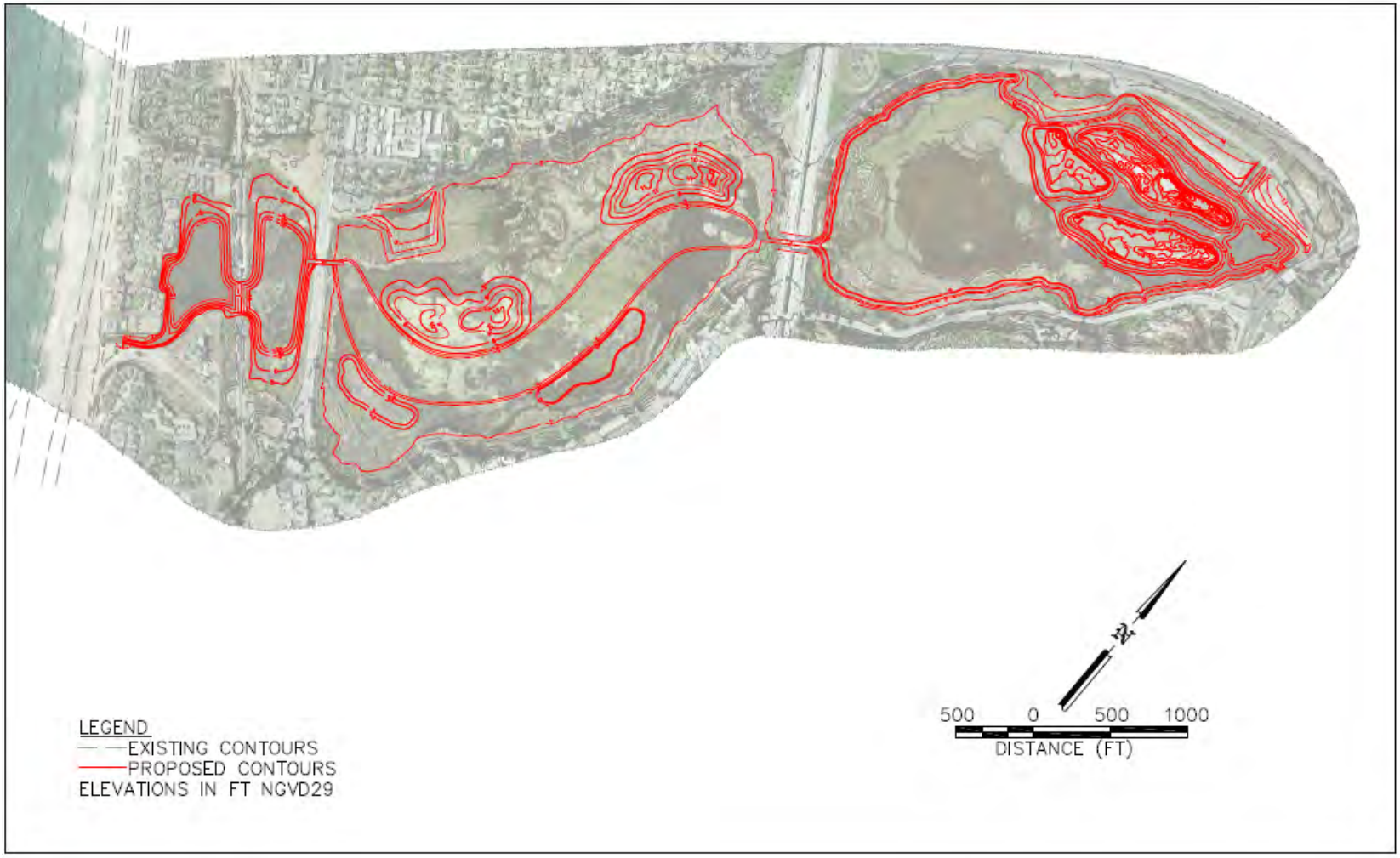


Figure 2.4 Alternative 1 Plan View

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.

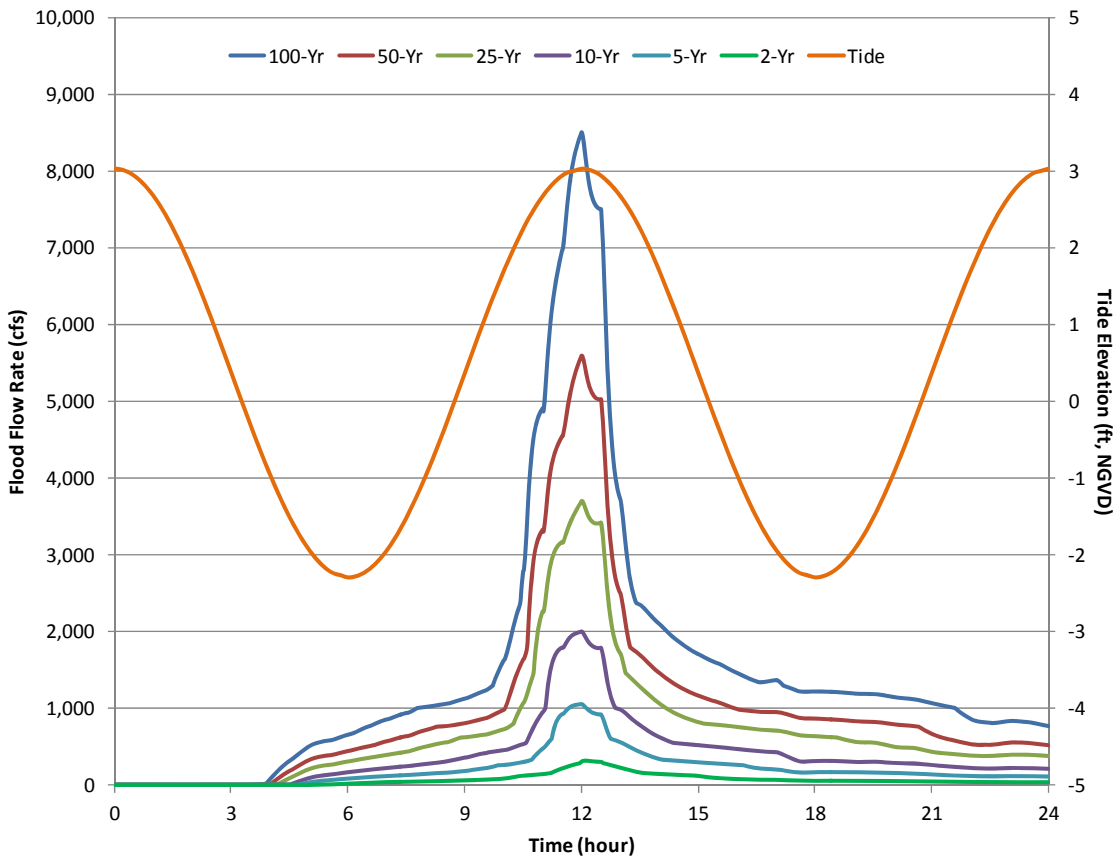


Figure 3.1 Flood and Diurnal Tide Hydrographs

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.

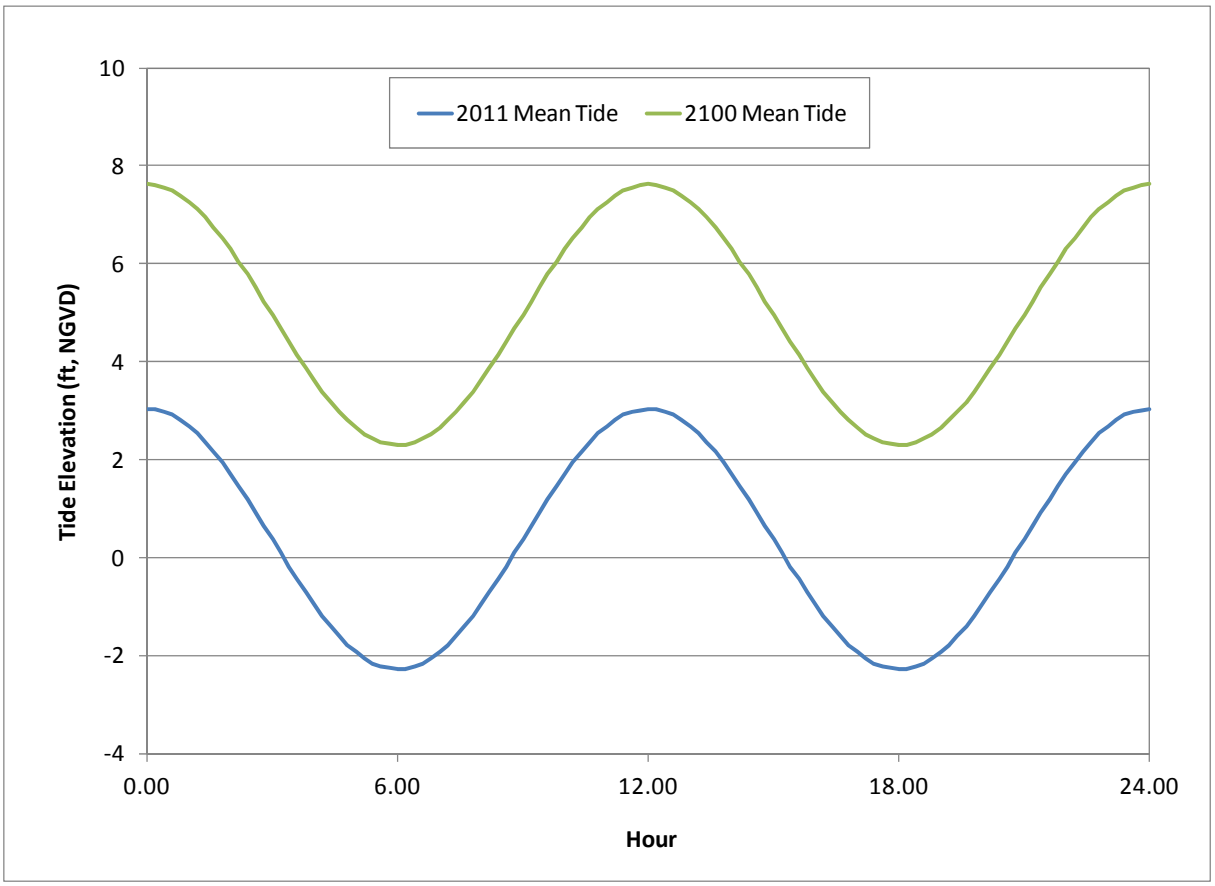


Figure 3.2 Diurnal Tide Hydrographs for Year 2011 and Year 2100

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

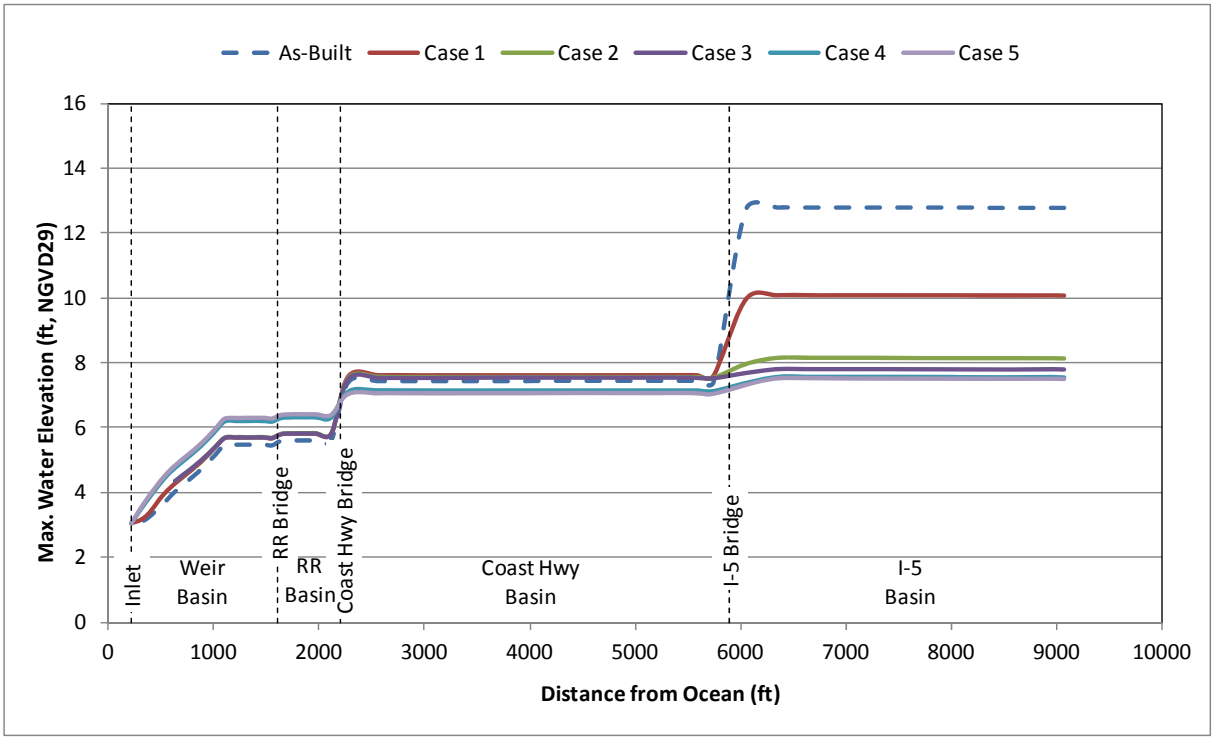


Figure 4.1 Alt 2-1 HEC-RAS Results for Cases with Various Channel Elevations

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

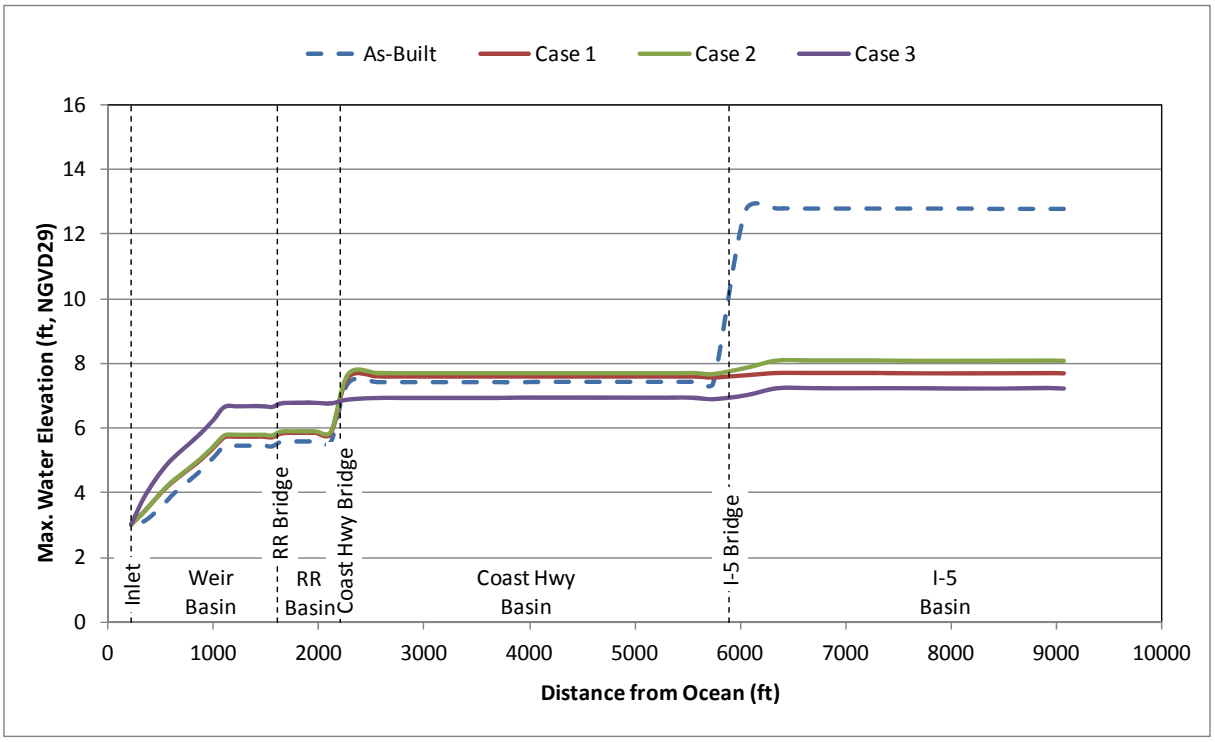


Figure 4.2 Alt 2-1 HEC-RAS Results for Cases with Various Span Lengths

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.

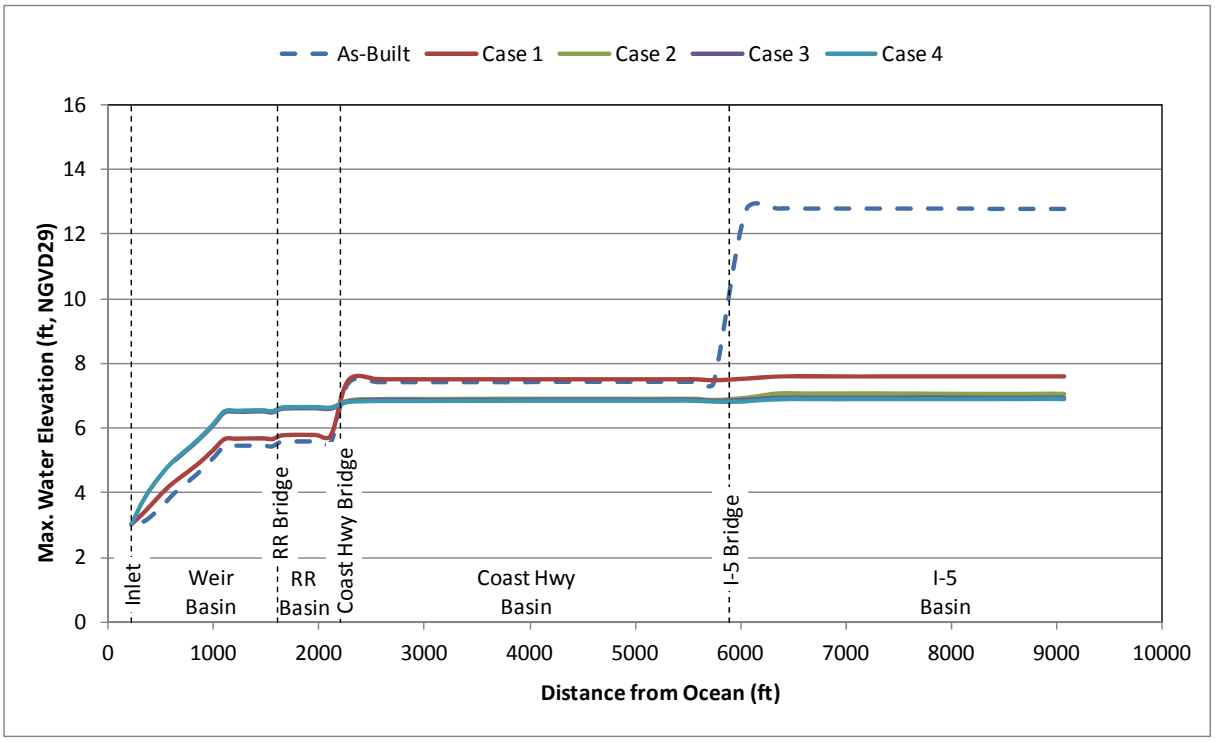


Figure 4.3 Alt 2-1 HEC-RAS Results for Analysis Cases without Sea Level Rise

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.

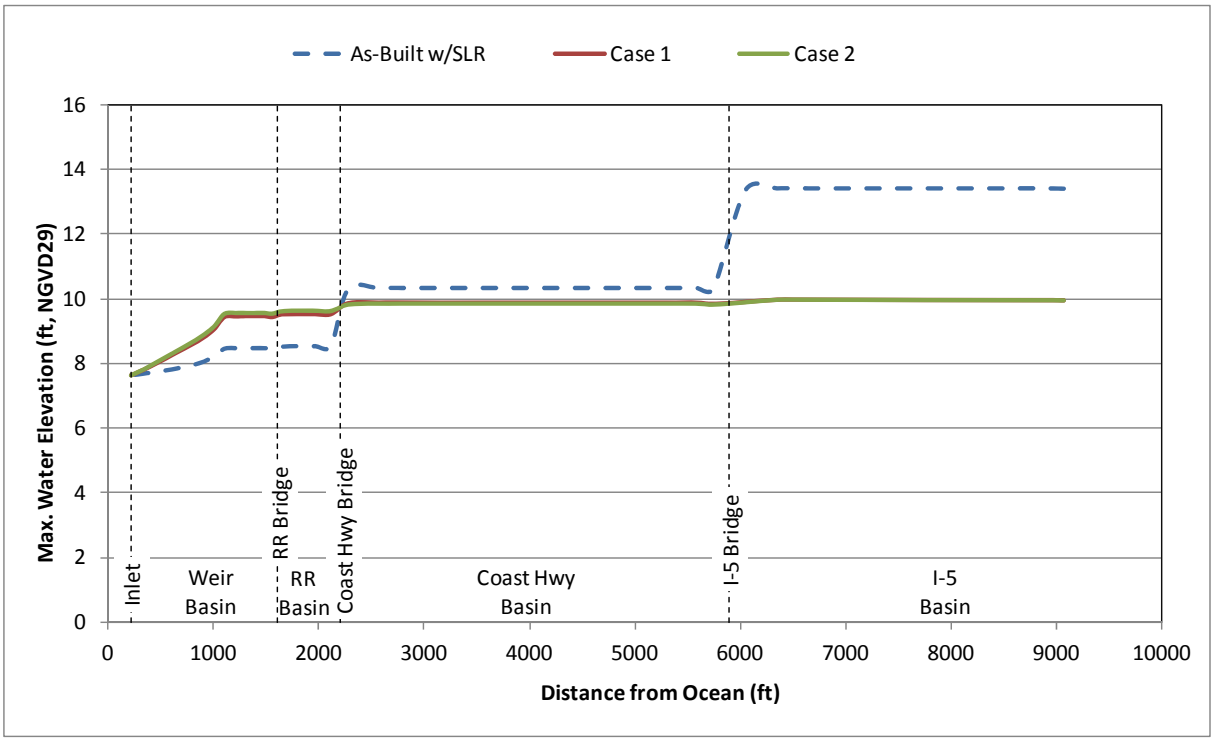


Figure 4.4 Alt 2-1 HEC-RAS Results for Sea Level Rise Analysis

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

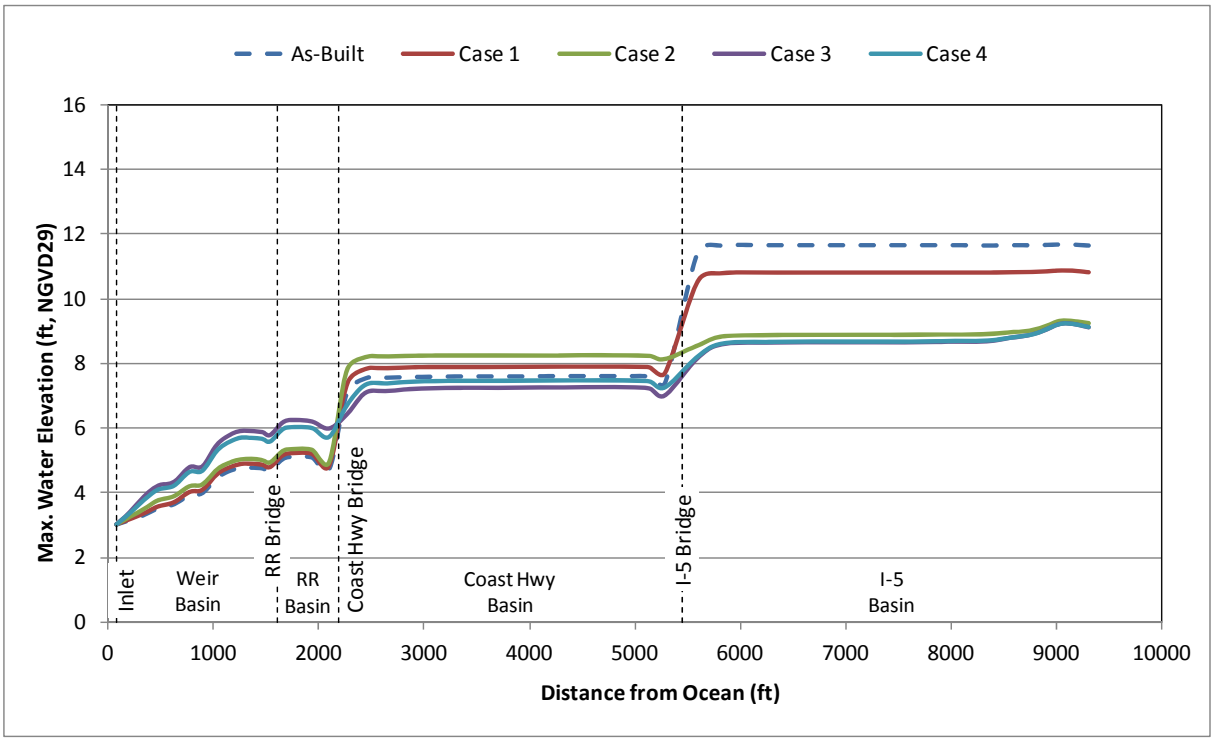


Figure 4.5 Alt SW2-A HEC-RAS Results for Cases with Various Bridge Channel Elevations

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

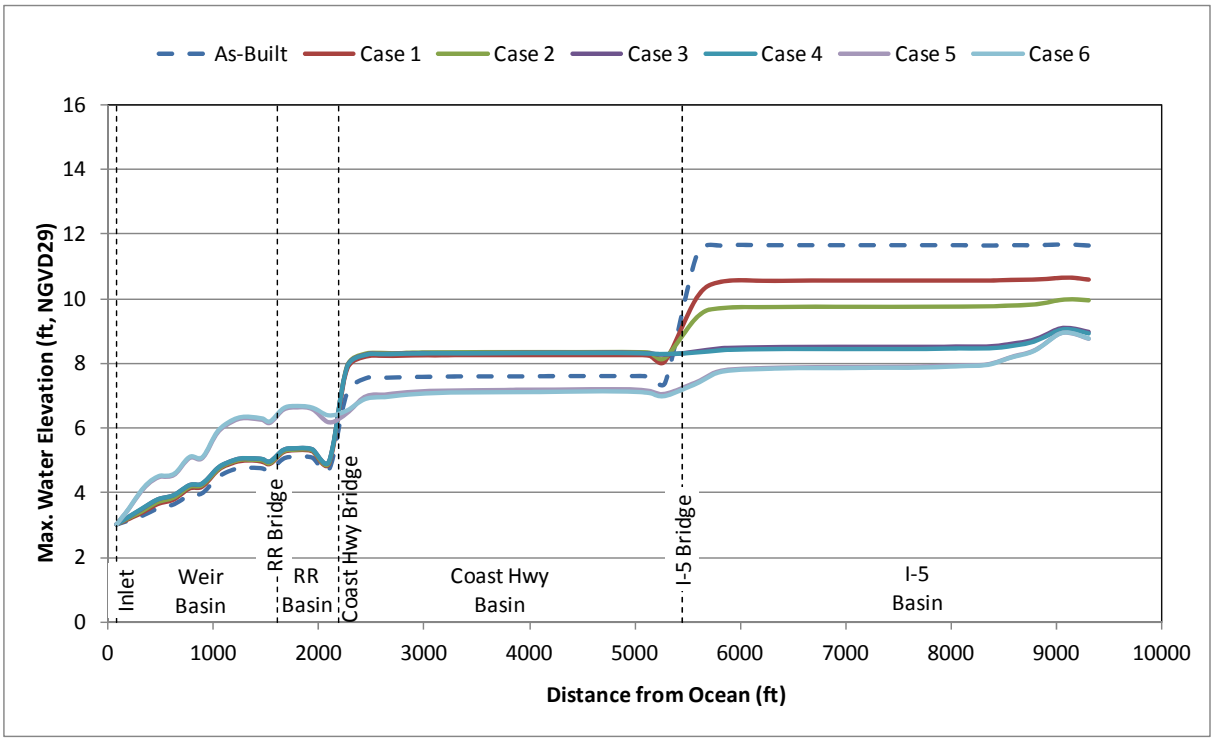


Figure 4.6 Alt SW2-A HEC-RAS Results for Cases with Various Span Lengths

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.

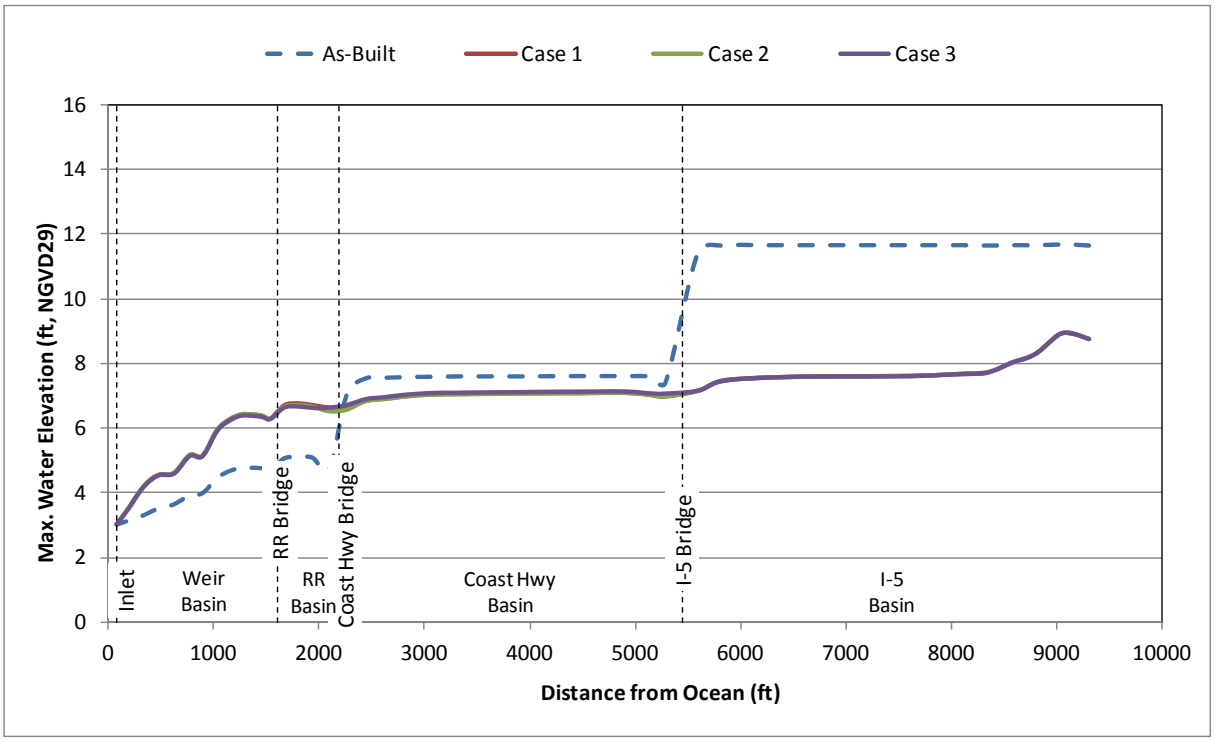


Figure 4.7 Alt SW2-A HEC-RAS Results for Analysis Cases without Sea Level Rise

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

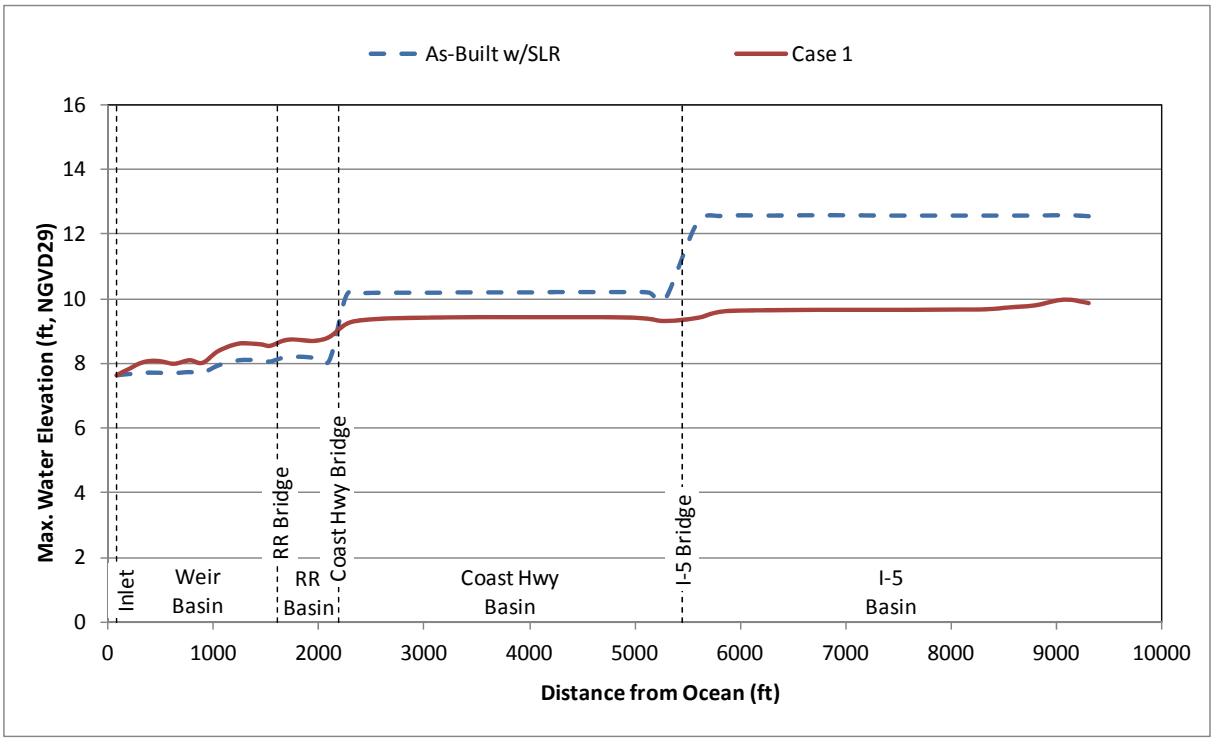


Figure 4.8 Alt SW2-A HEC-RAS Results for Sea Level Rise Analysis

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

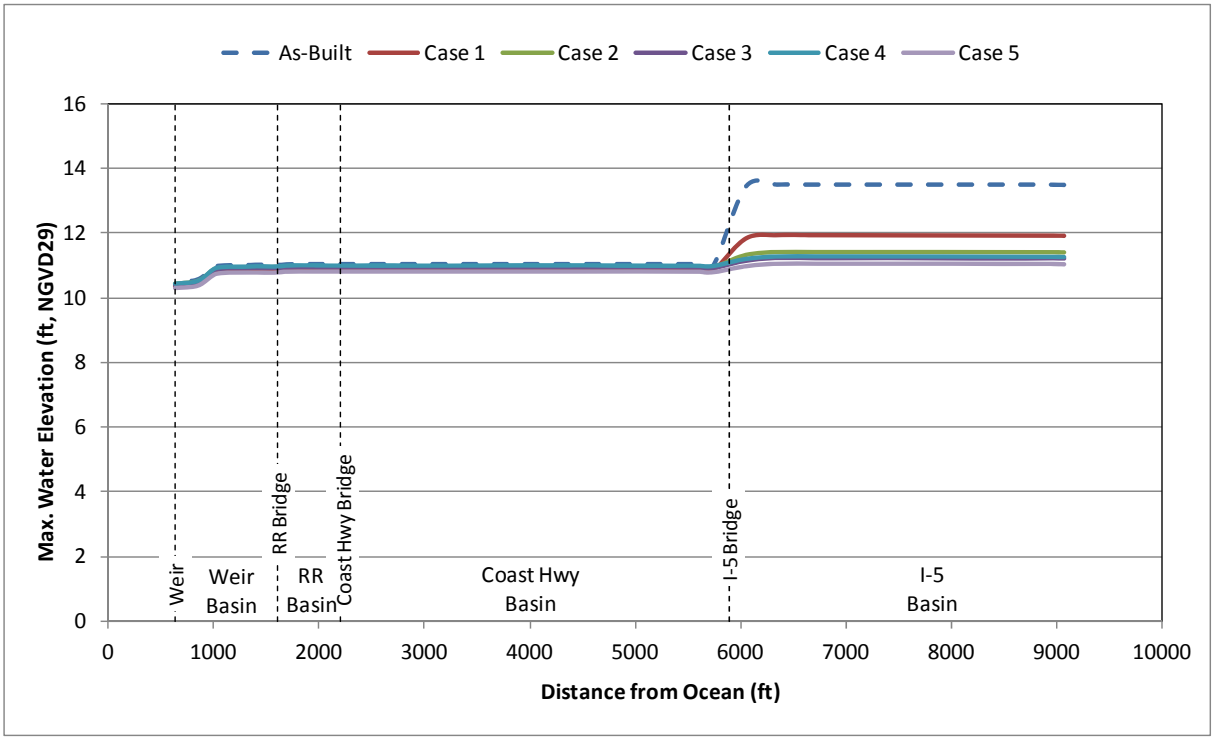


Figure 4.9 Alt 1 HEC-RAS Results for Cases with Various Bridge Channel Elevations

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

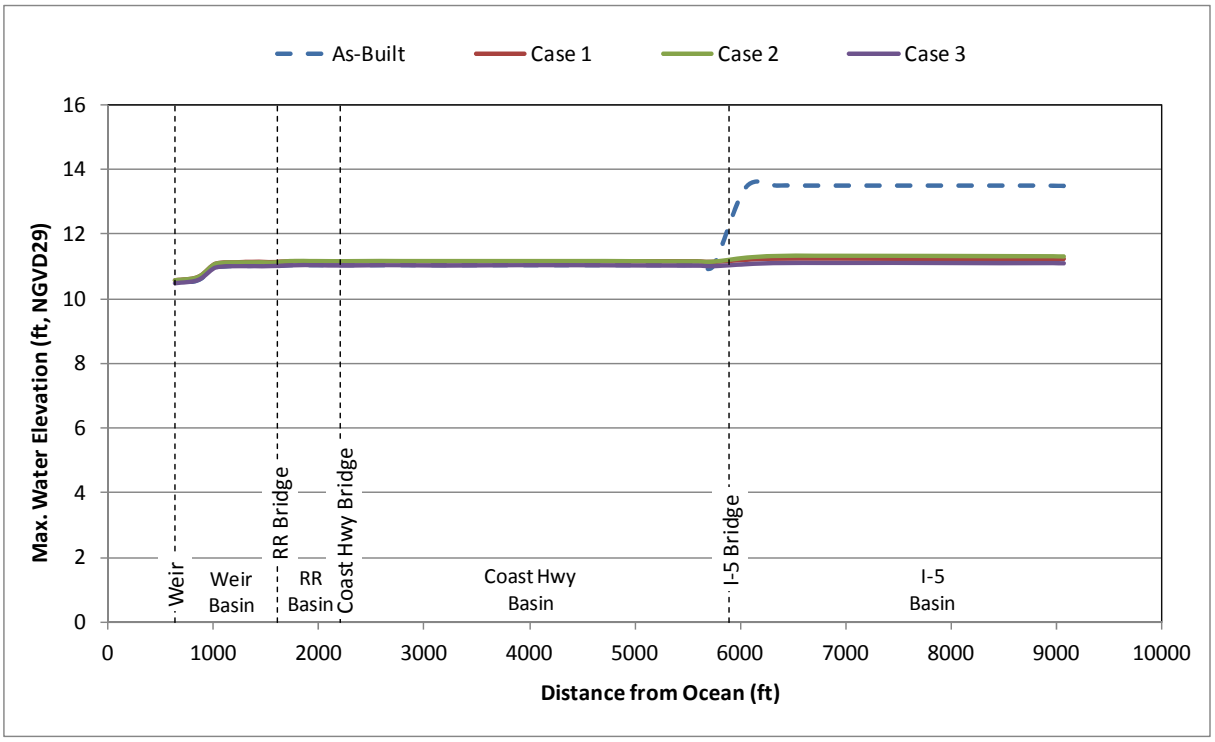


Figure 4.10 Alt 1 HEC-RAS Results for Cases with Various Span Lengths

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.

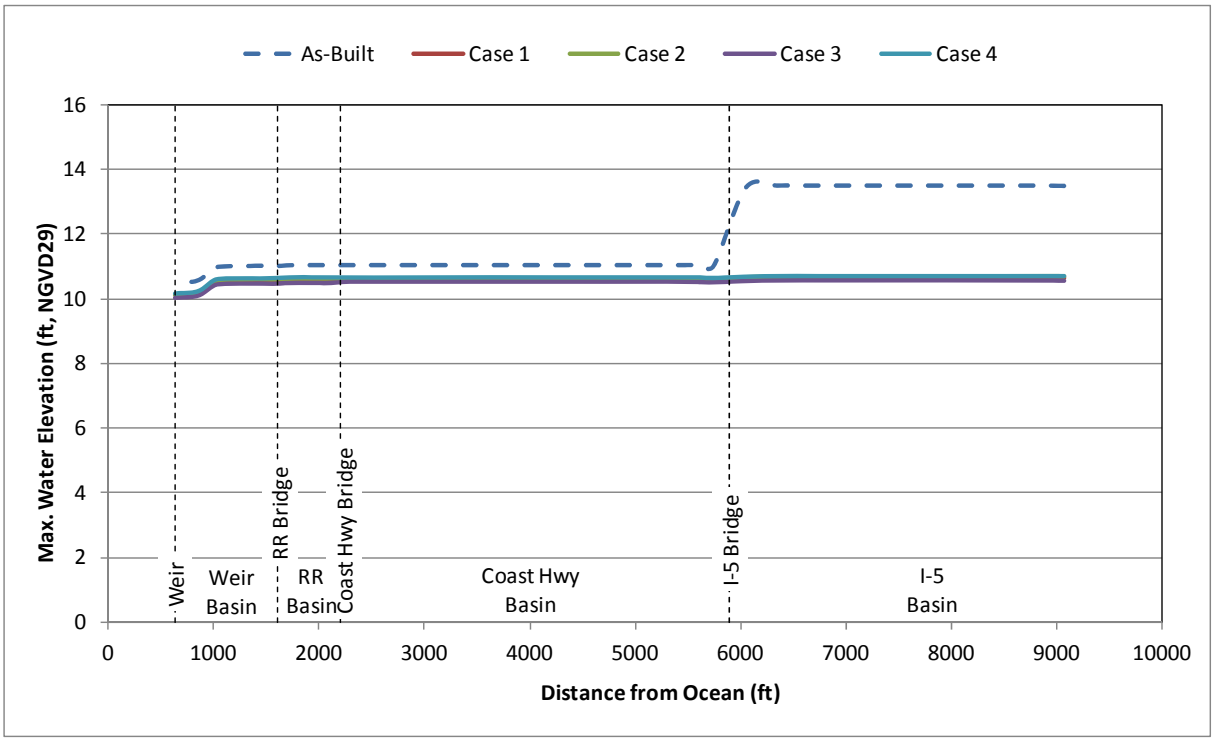


Figure 4.11 Alt 1 HEC-RAS Results for Analysis Cases without Sea Level Rise

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.

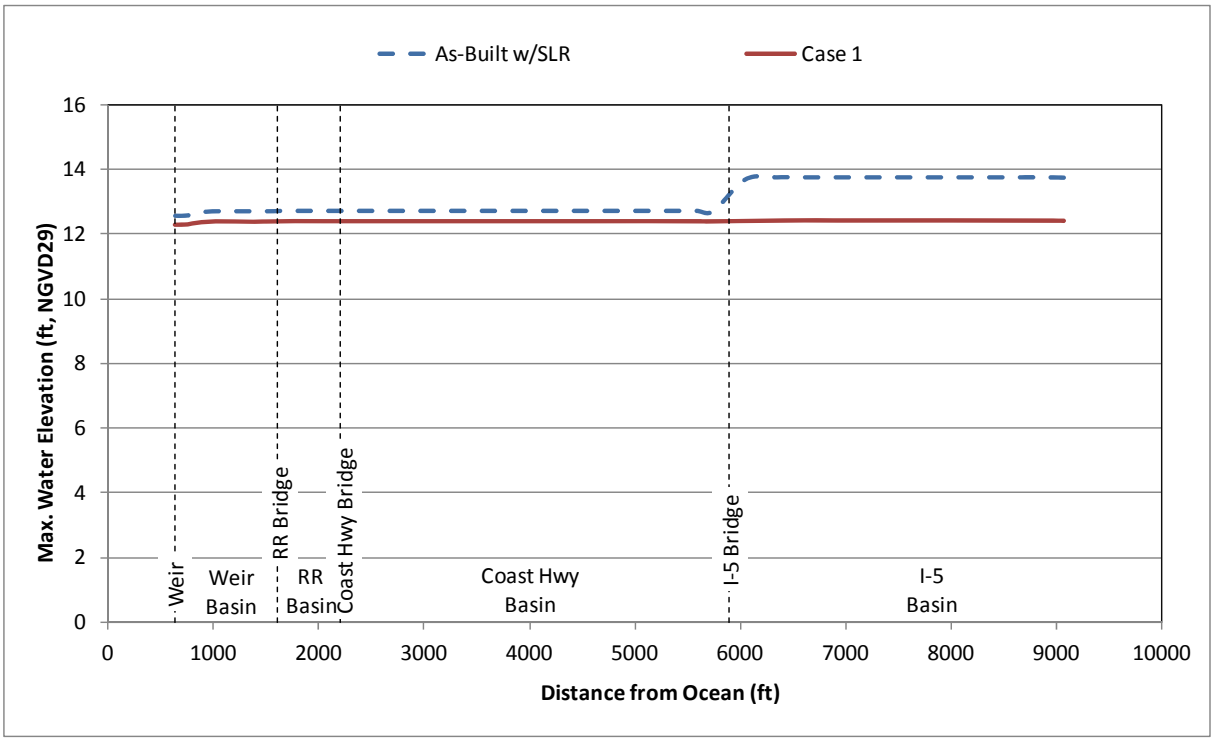


Figure 4.12 Alt 1 HEC-RAS Results for Sea Level Rise Analysis

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

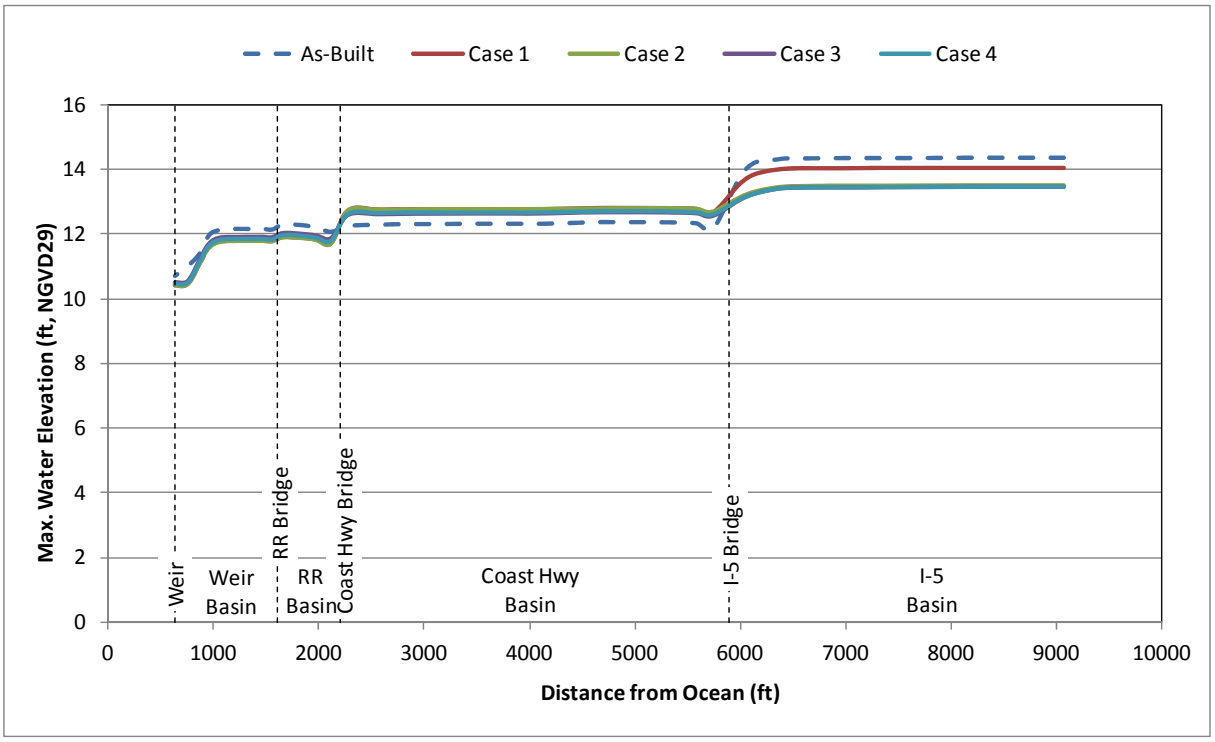


Figure 4.13 Alt FW-A HEC-RAS Results for Cases with Various Channel Elevations

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

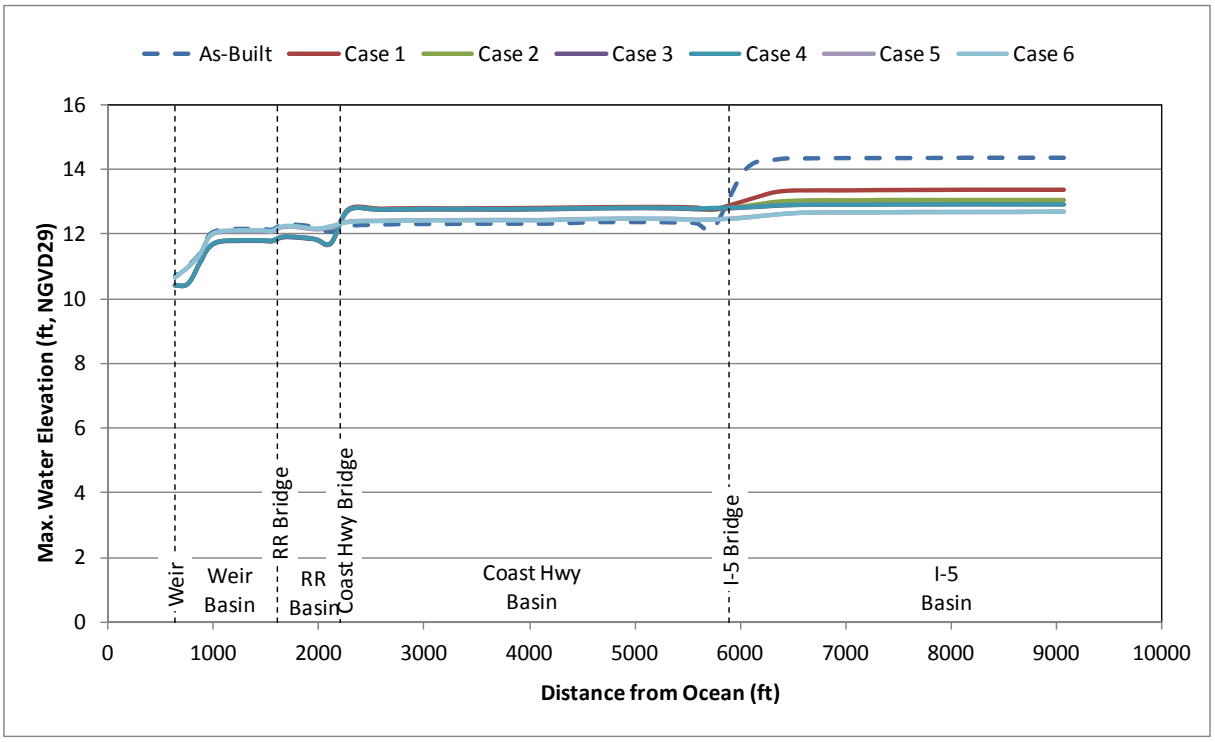


Figure 4.14 Alt FW-A HEC-RAS Results for Cases with Various Span Lengths

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.

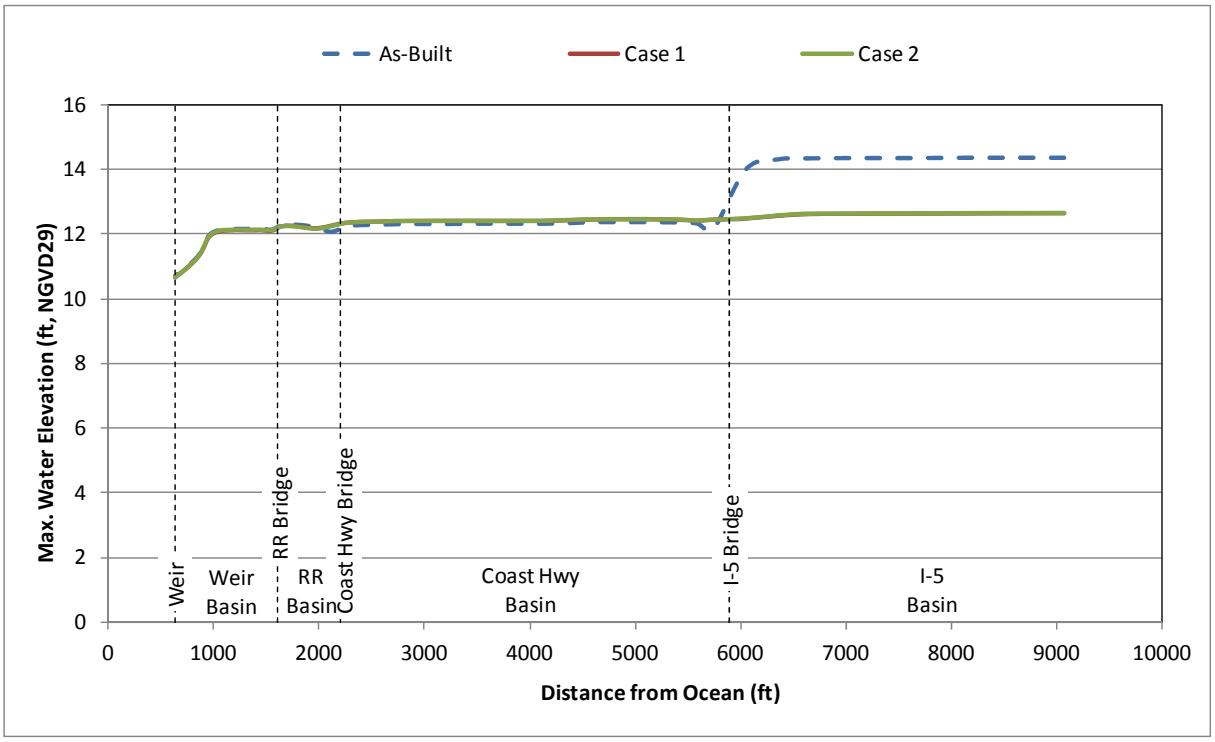


Figure 4.15 Alt FW-A HEC-RAS Results for Analysis Cases without Sea Level Rise

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.

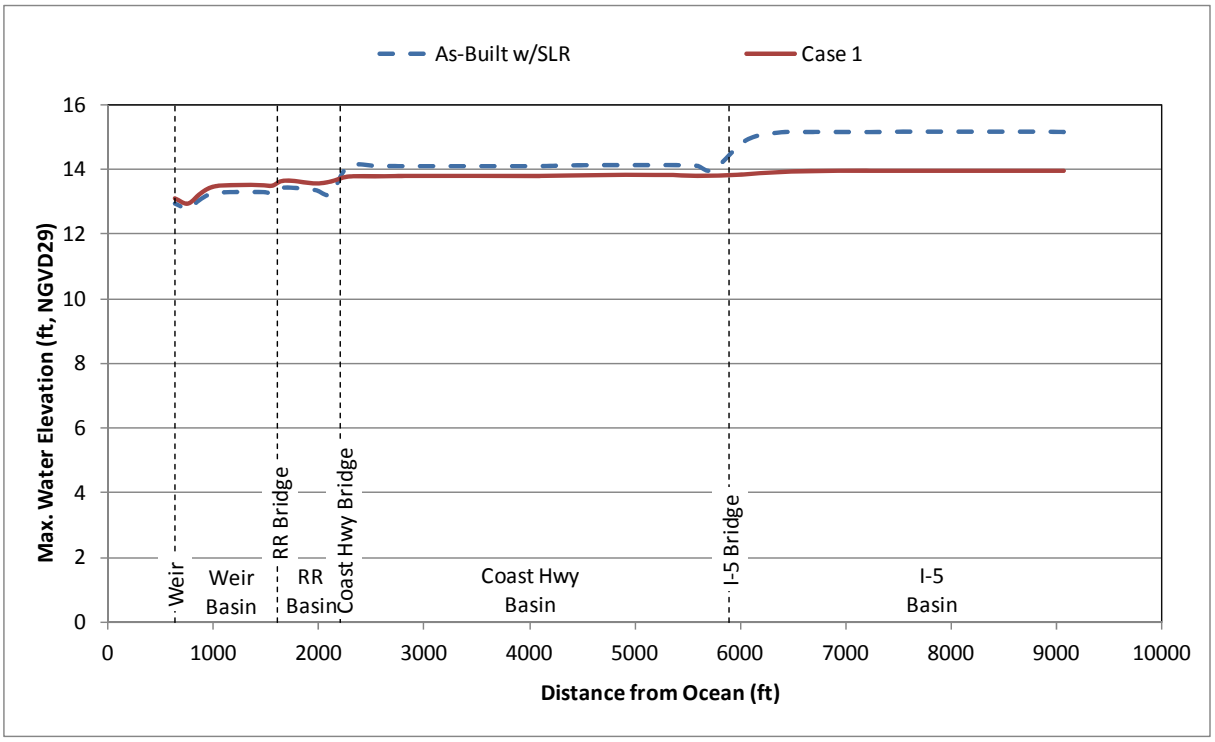


Figure 4.16 Alt FW-A HEC-RAS Results for Sea Level Rise Analysis

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.

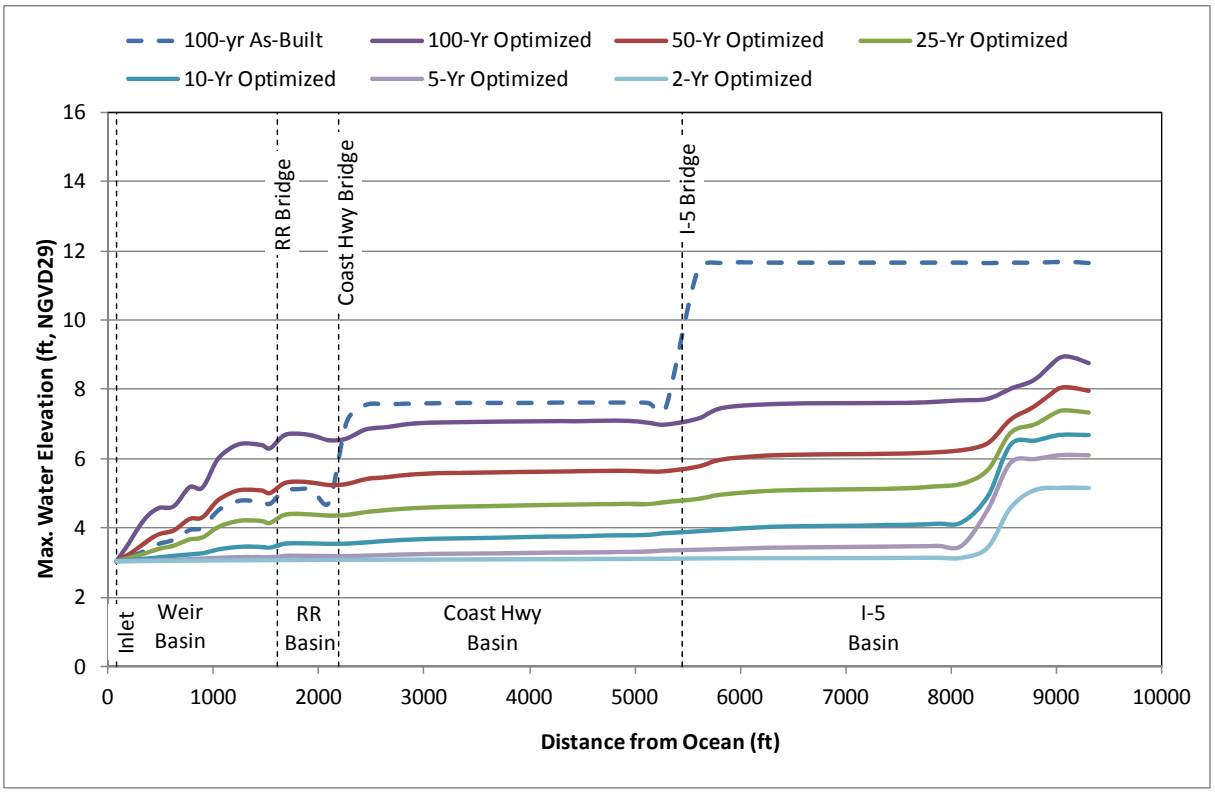


Figure 4.17 Fluvial Hydraulic Analyses for Minor Storms for Alt SW2-A Best Case

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

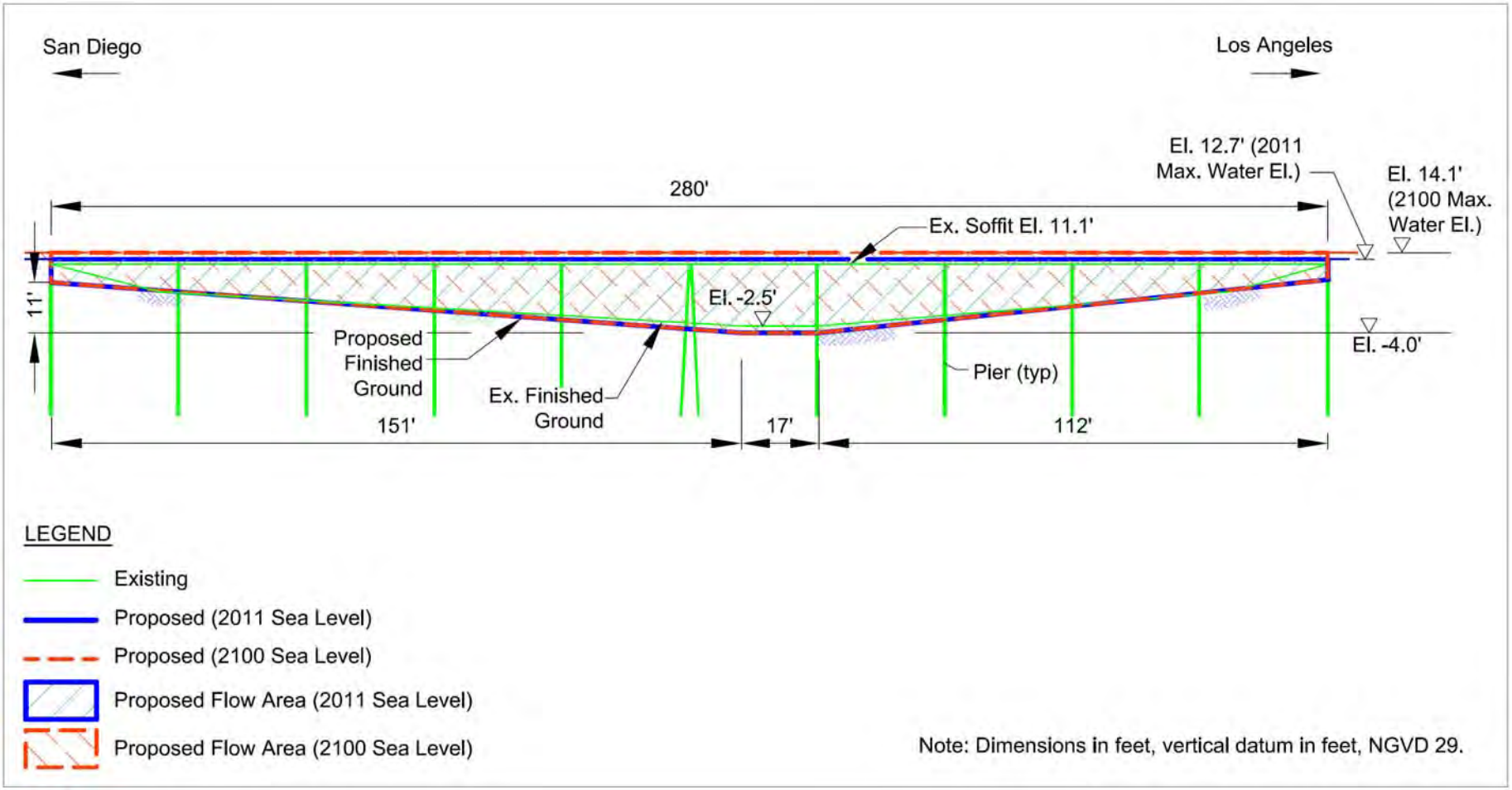


Figure 4.18 Desired Channel Cross Section of Railroad Bridge at Buena Vista Lagoon

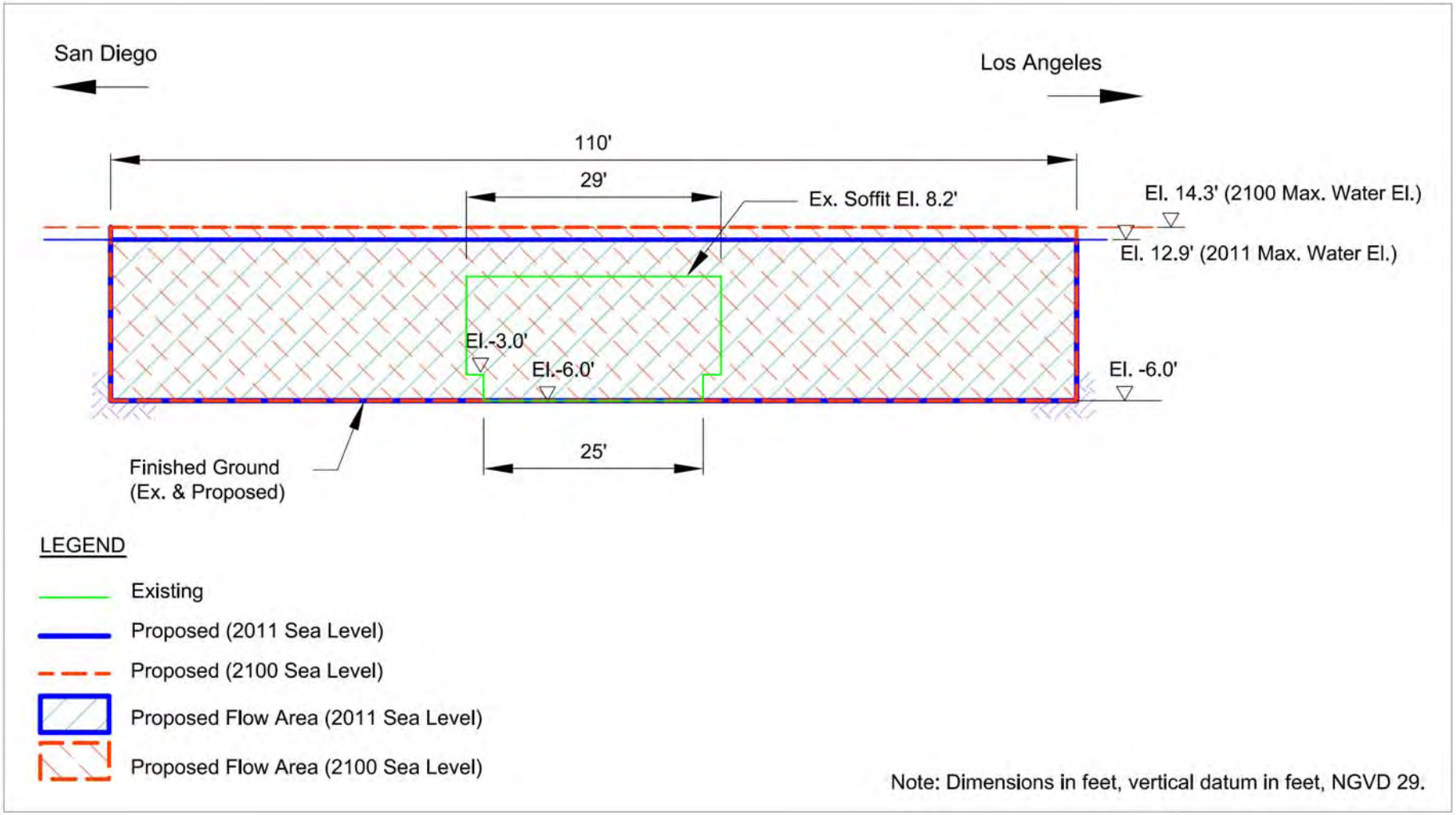


Figure 4.19 Desired Channel Cross Section of Coast Highway Bridge at Buena Vista Lagoon

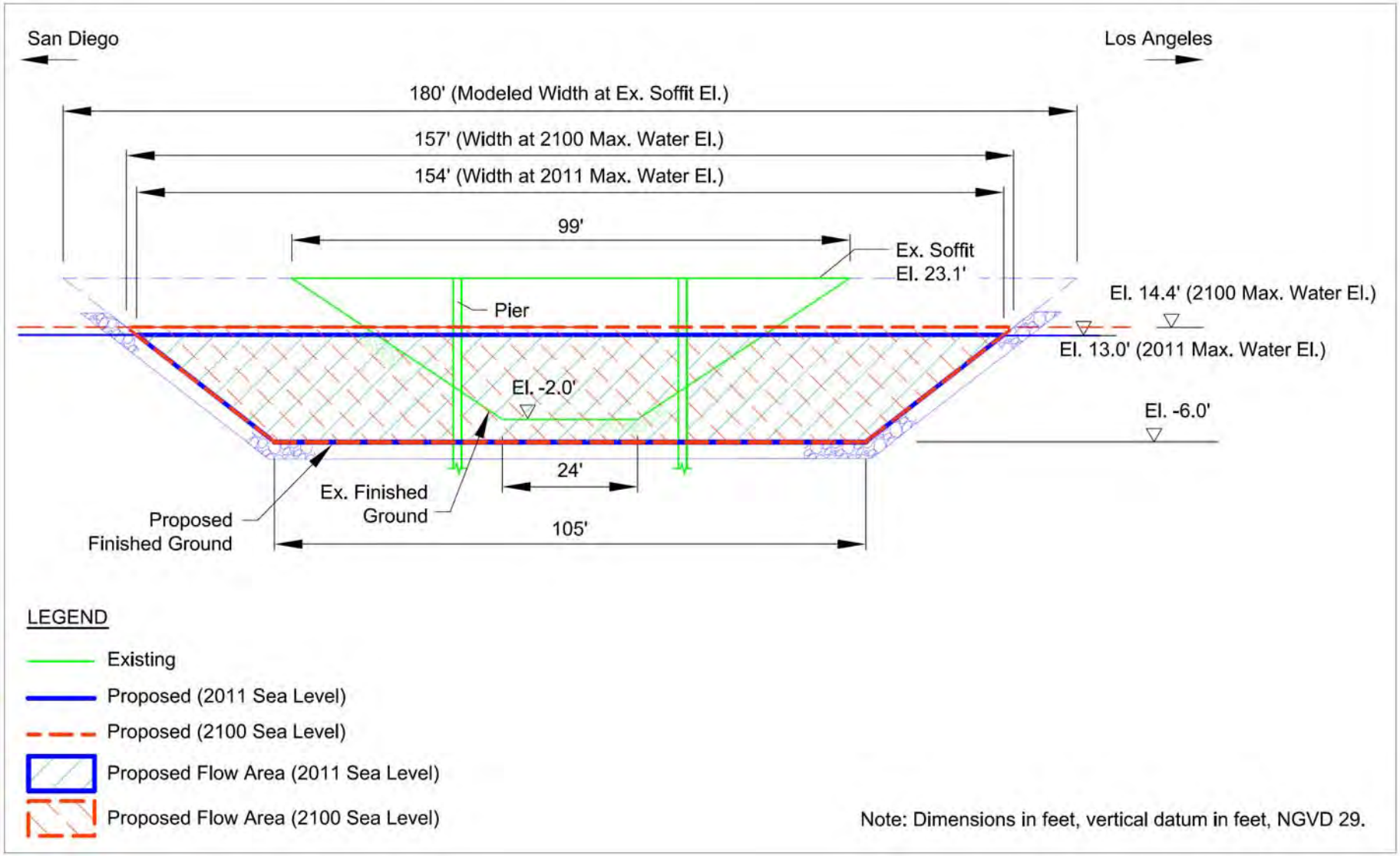


Figure 4.20 Desired Channel Cross Section of I-5 Bridge at Buena Vista Lagoon

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.

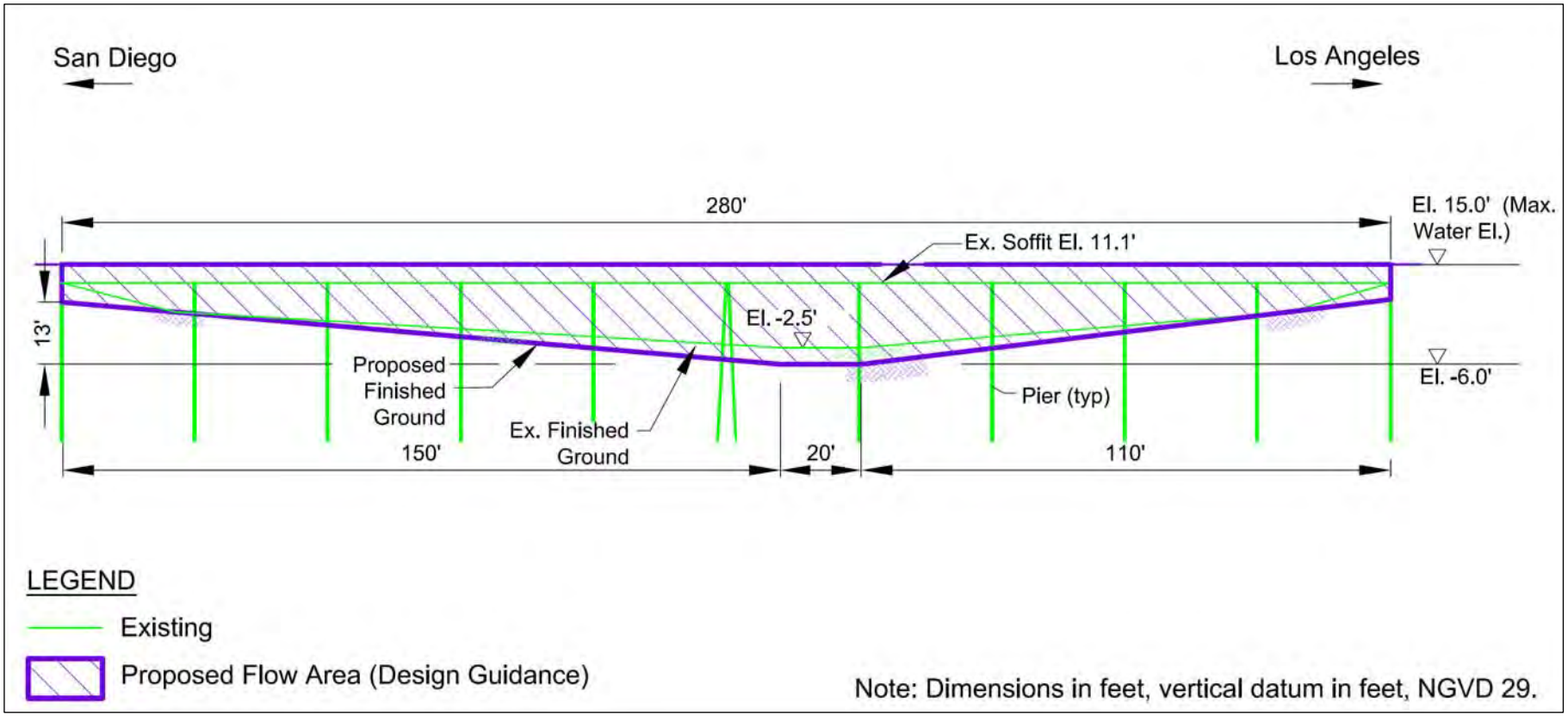


Figure 4.21 Design Guidance for Channel Cross Section of Railroad Bridge at Buena Vista Lagoon

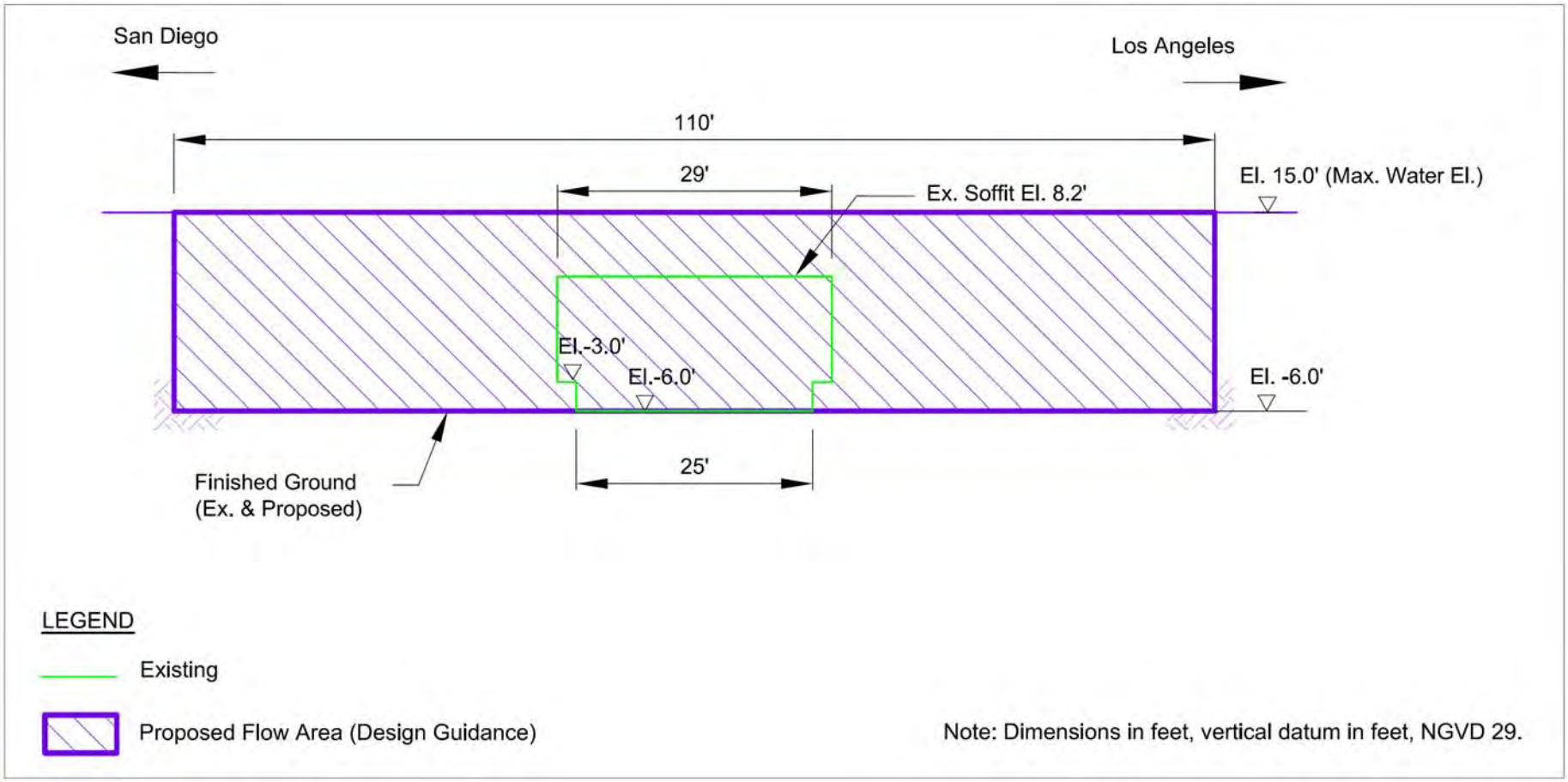


Figure 4.22 Design Guidance for Channel Cross Section of Coast Highway Bridge at Buena Vista Lagoon

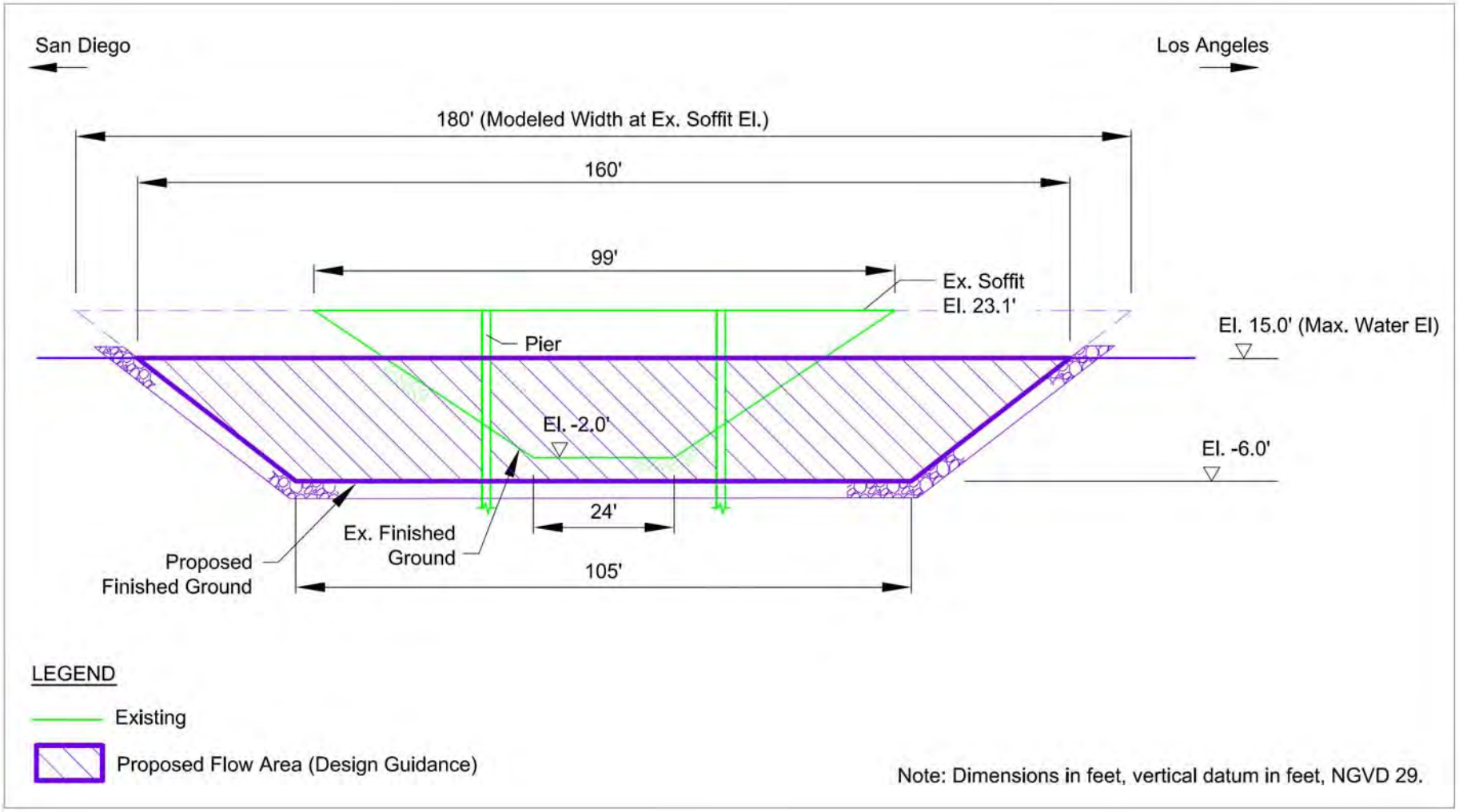


Figure 4.23 Design Guidance for Channel Cross Section of I-5 Bridge at Buena Vista Lagoon

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

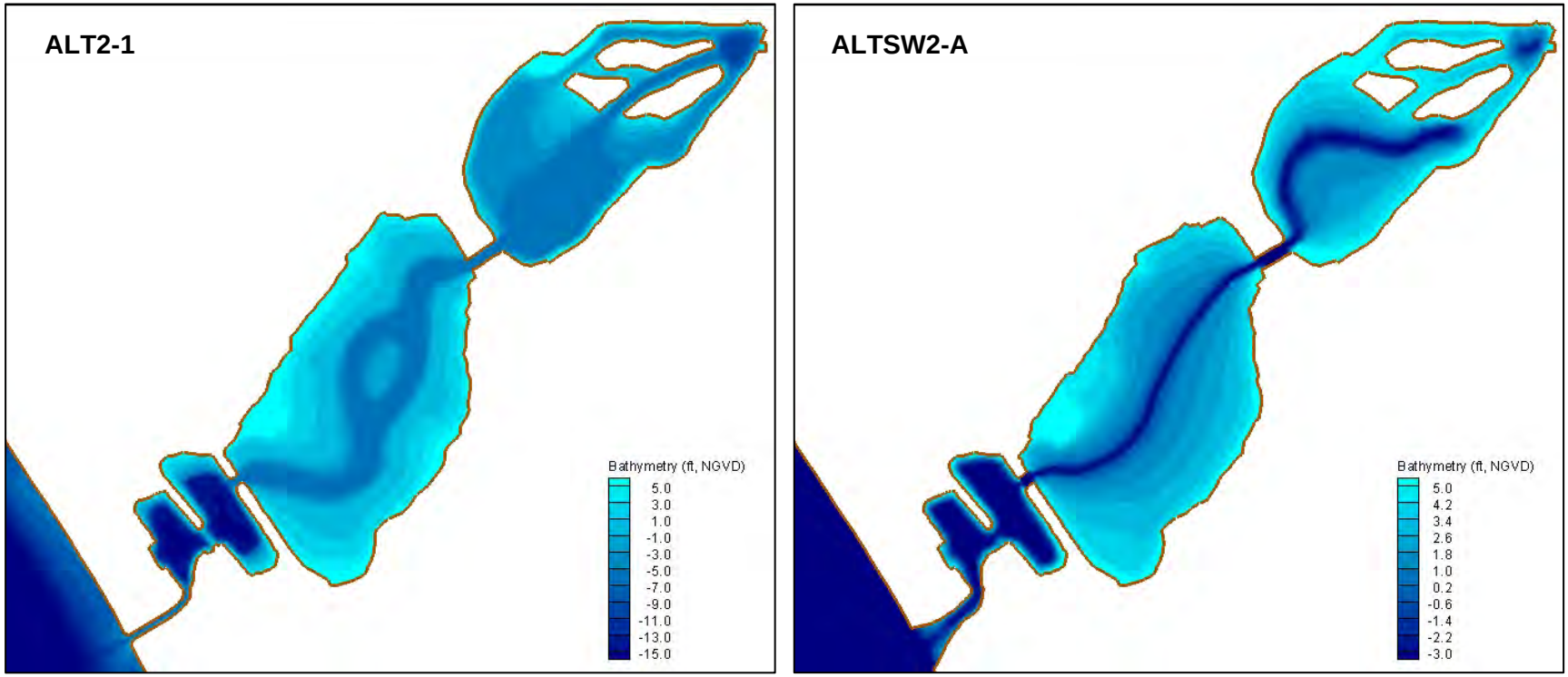


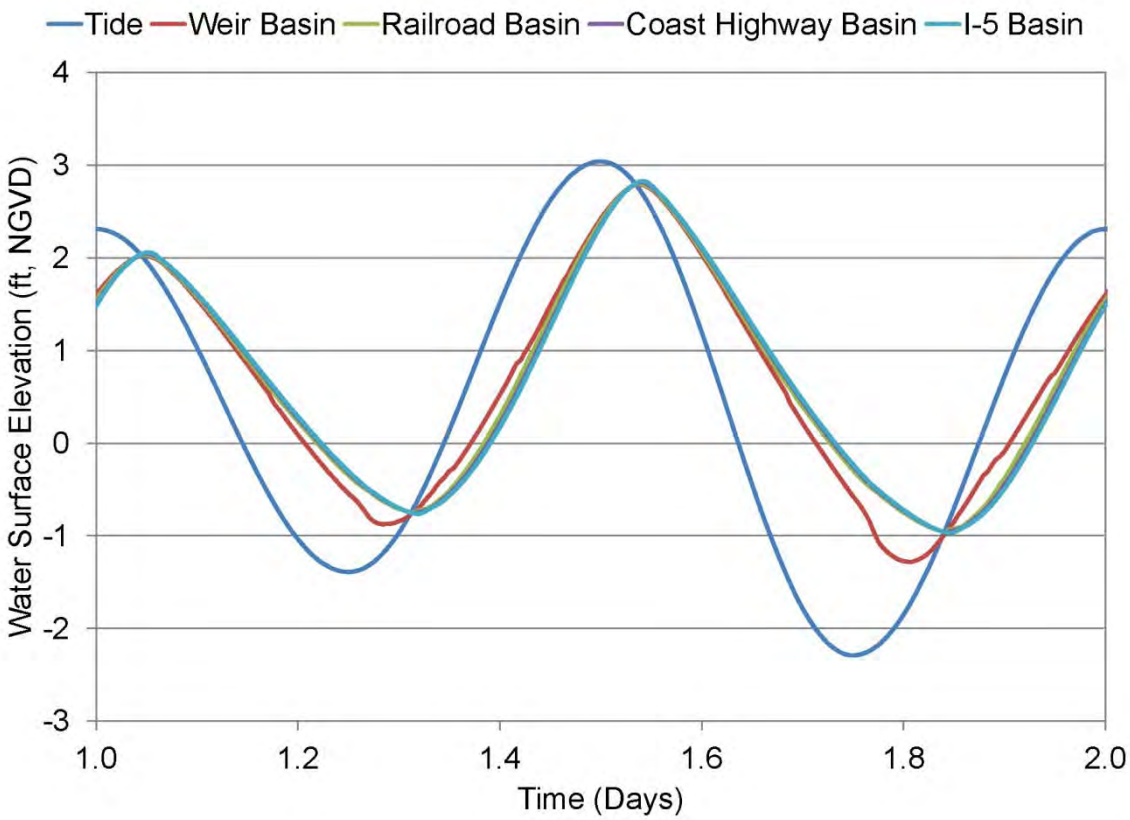
Figure 5.1 Buena Vista Lagoon Bathymetry used in Model Grids for Salt Water Alternatives

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.



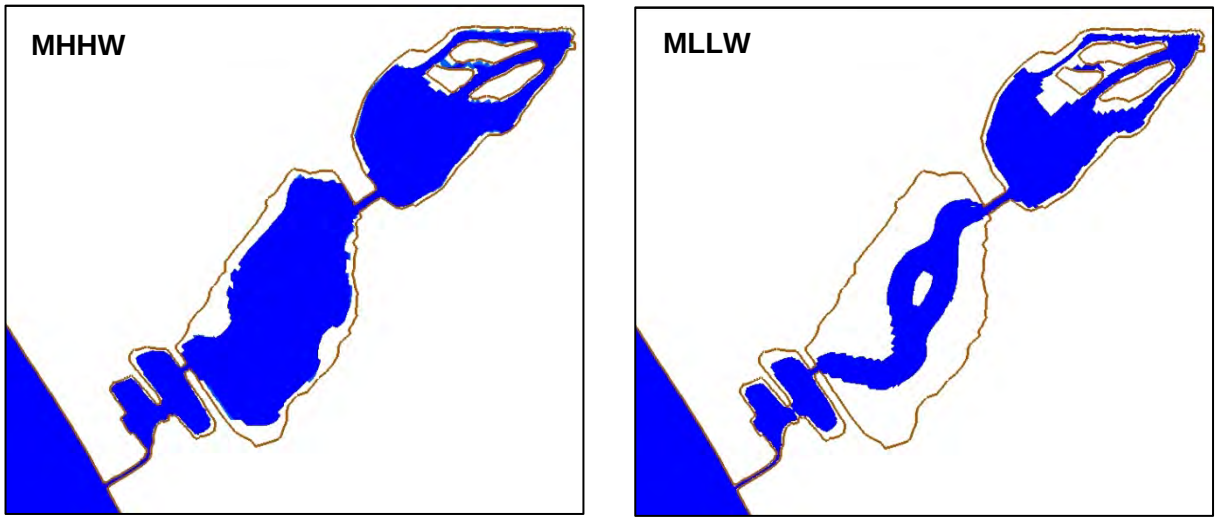
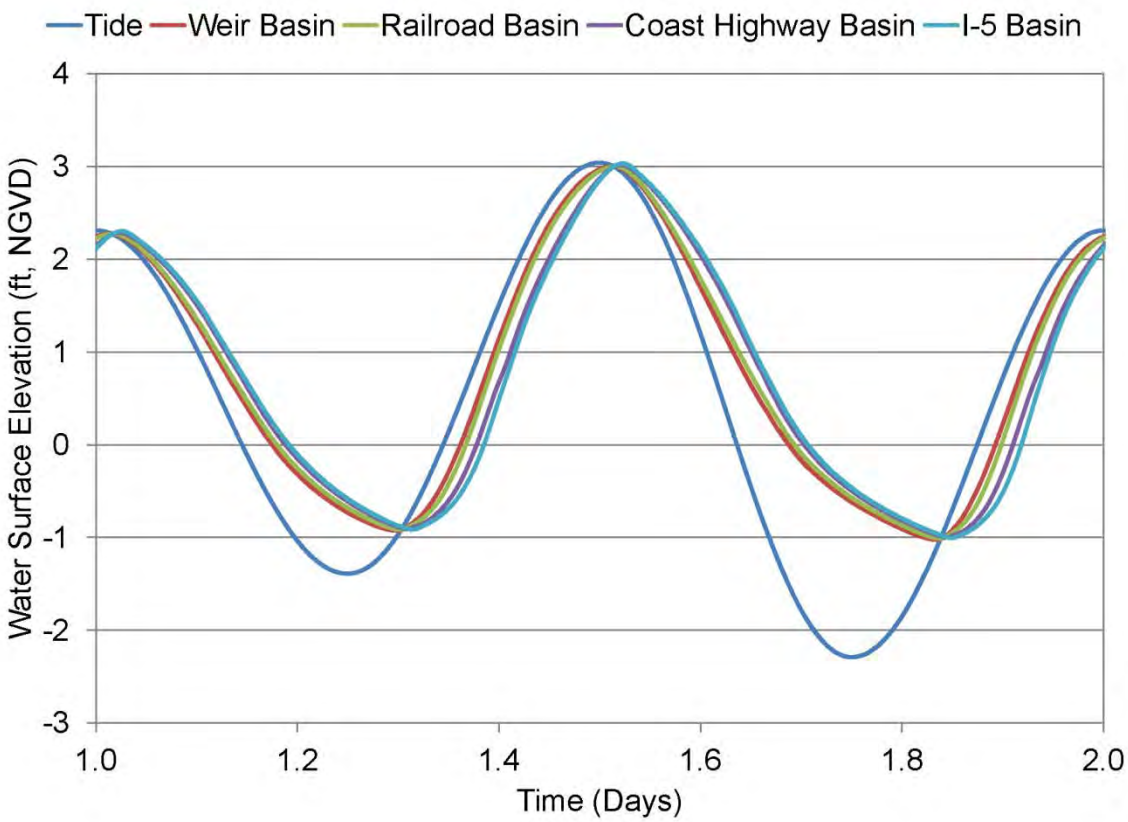


Figure 5.2 Alternative 2-1 Tidal Response



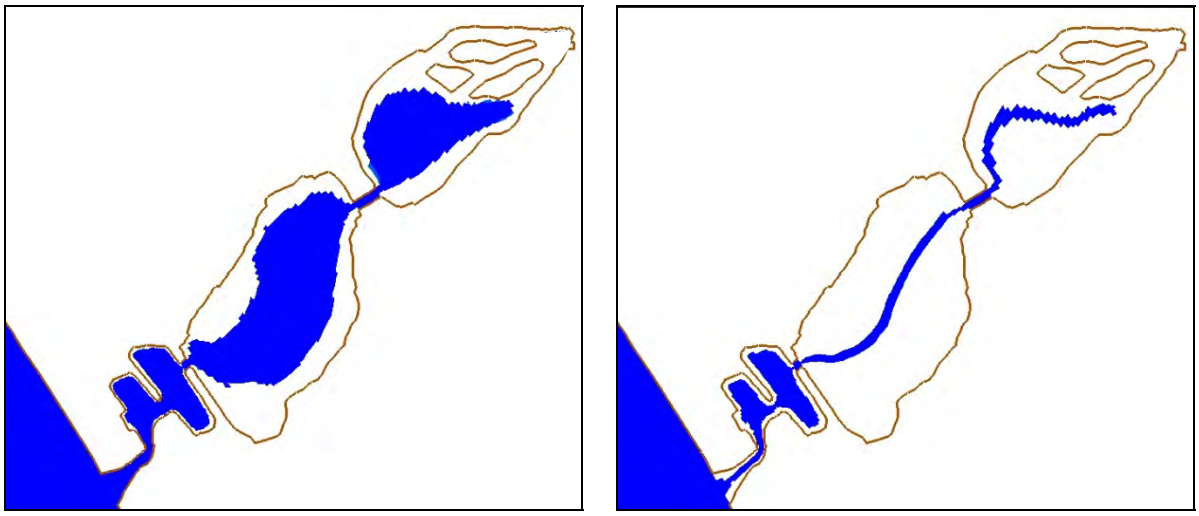
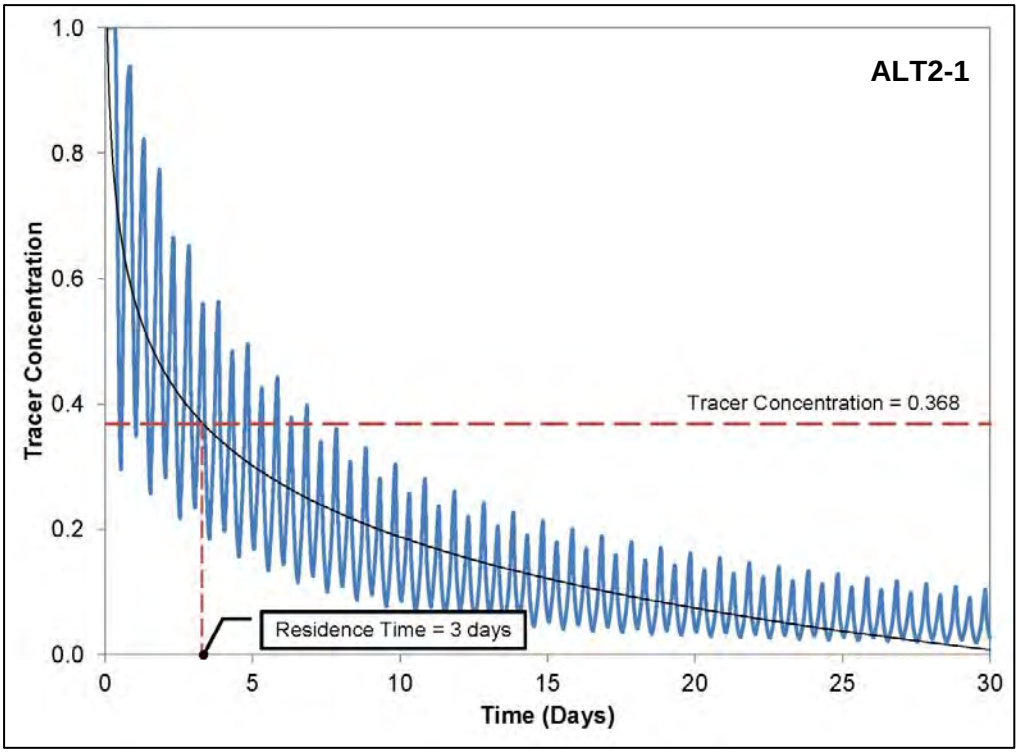


Figure 5.3 Alternative SW2-A Tidal Response



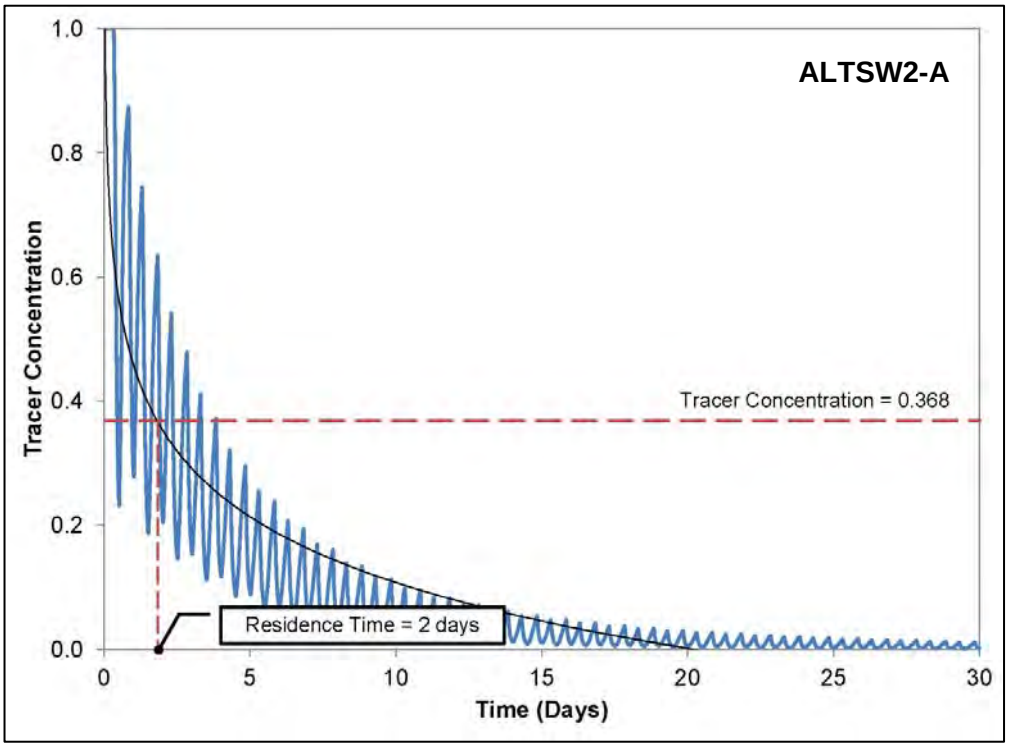


Figure 5.4 Tracer Concentrations in Weir Basin

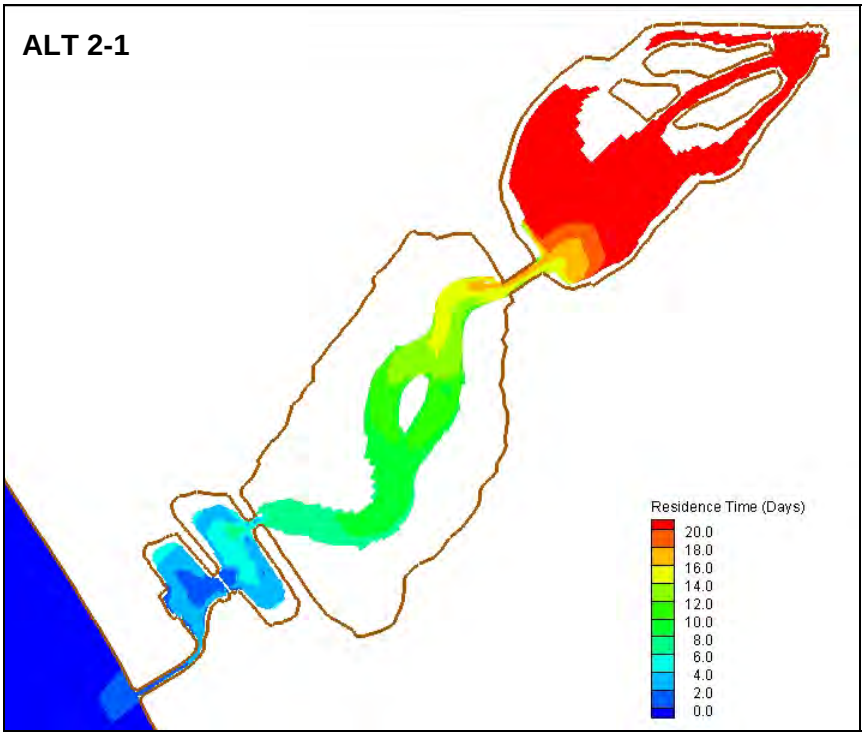
The calculated residence times were determined in the Lagoon for Alternatives 2-1 and SW2-A, as shown in Figure 5.5. Due to the differences in the range of residence times for each alternative, different color scales are used for the two alternatives. Both alternatives show that the residence times are 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.



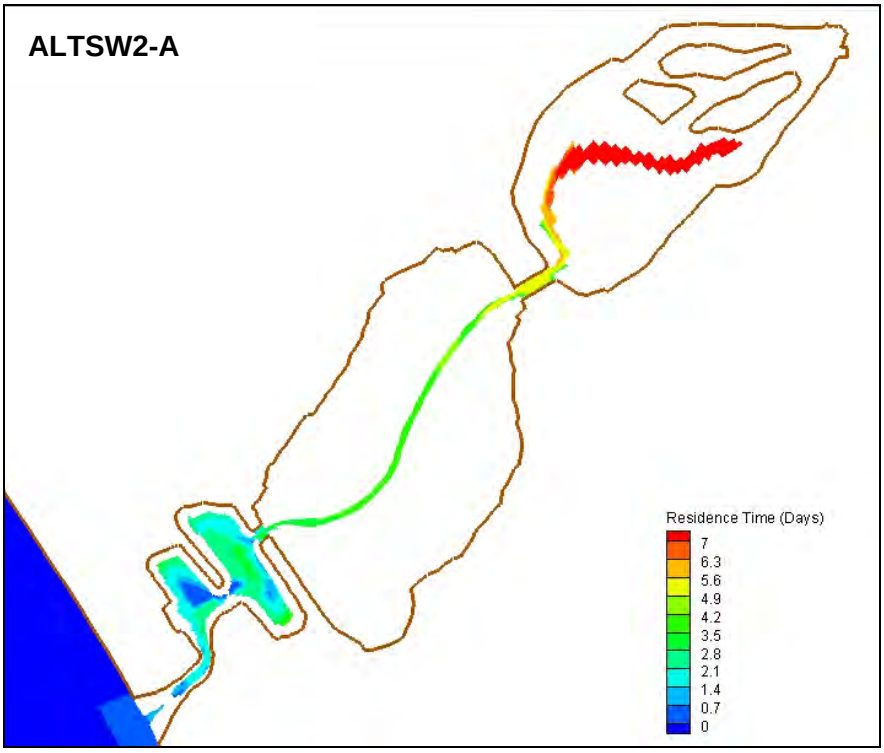


Figure 5.5 Residence Times for Salt Water Alternatives

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.

7 REFERENCES

Applegate, J. 1985. Buena Vista Lagoon Watershed Sediment Control Plan. Prepared for California State Coastal Conservancy. Prepared by June Applegate and Philip William & Associates.

COPC (California Ocean Protection Council). 2011. Resolution of the California Ocean Protection Council on Sea-Level Rise, Adopted on March 11, 2011.

Chang, H. H. 1986. Sediment Study for Buena Vista Creek. Prepared for City of Vista.

EPA. 2001. Better Assessment Science Integrating Point and Nonpoint Sources. U.S. Environmental Protection Agency Office of Water. EPA-823-B-01-001. June 2001.

Everest. 2004. Buena Vista Lagoon Restoration Feasibility Analysis Final Report. Prepared for Buena Vista Lagoon Foundation, California State Coastal Conservancy, U.S. Fish and Wildlife Service. Prepared by Everest International Consultants, Inc. June 2004.

Everest. 2008. Buena Vista Lagoon Restoration Project Fluvial Hydraulics, Sediment Transport, and Sedimentation Analysis Final Report. Prepared for California State Coastal Conversancy. Prepared by Everest International Consultants, Inc. under contract to Science Applications International Corporation. October 2008.

Everest. 2009. Buena Vista Lagoon Restoration Project Inlet Stability Analysis and Tidal Hydraulics Modeling Final Report. Prepared for U.S. Fish and Wildlife Service. Prepared by Everest International Consultants, Inc. June 2009.

Everest. 2011a. Buena Vista Lagoon Restoration Project – Restoration Alternatives, Draft Technical Memorandum. Prepared for California Department of Fish and Game. Prepared by Everest International Consultants, Inc. December 2011.

Everest. 2011b. Buena Vista Lagoon Restoration Project – Sea Level Rise Assessment, Draft Technical Memorandum. Prepared for California Department of Fish and Game. Prepared by Everest International Consultants, Inc. December 2011.

NOAA. 2003. Bench Mark Sheets. National Ocean Service. National Oceanic and Atmospheric Administration.

Tetra Tech. 2007. The Environmental Fluid Dynamics Code User Manual US EPA Version 1.01. Prepared by Tetra Tech, Inc. Prepared for U.S. Environmental Protection Agency. June 2007.

USACE. 2006. HEC-RAS River Analysis System User's Manual Version 4.0 Beta. U.S. Army Corps of Engineers Hydrologic Engineering Center. November 2006.

USGS. 2012. http://wwwrcamnl.wr.usgs.gov/sws/fieldmethods/Indirects/nvalues/index.htm. Verified Roughness Characteristics of Natural Channels. U.S. Geological Survey website. 2012.

SAN ELIJO LAGOON BRIDGE OPTIMIZATION STUDY

FINAL REPORT

Interstate 5 North Coast Corridor Project
SAN DIEGO COUNTY, CALIFORNIA
DISTRICT 11-SD-5 (PM R28.4/R55.4)
EA 235800 (P ID 11-000-0159)

APRIL 2012

San Elijo Lagoon
Bridge Optimization Study

Final Report

April 2012

Prepared for:

Dokken Engineering and the

California Department of Transportation

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EXECUTIVE SUMMARY

For the past several years, Caltrans and 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 Project. Implementation of this project will include new bridge structures across most of the lagoons of northern San Diego County, including the San Elijo Lagoon. The objective of this study was to evaluate a range of channel widths and depths under bridges at Interstate 5 Freeway (I-5), the North County Transit District Railroad (Railroad), and Highway (Hwy) 101 and identify a combination of optimized channel widths that provide the most favorable conditions for tidal range, flood conveyance and other environmental benefits throughout the lagoon.

Concurrent with the North Coast Corridor Project, the San Elijo Lagoon Restoration Project (SELRP) is pursuing an effort to restore the Lagoon functions and values given the constraints placed on it by surrounding current and historic development activities. The overarching goal of the SELRP is to protect, restore, and then maintain, via adaptive management, the San Elijo Lagoon ecosystem. The SELRP aims to enhance the tidal prism of the Lagoon by proposing modifications to known infrastructure “choke points” such as Hwy 101, the Railroad, and the I-5 freeway. The four alternatives under consideration for the SELRP and evaluated in this study are listed below:

1. **No Project** proposes no grading within the lagoon except for on-going maintenance dredging of the inlet channel.
2. **Alternative 1A** provides minimal physical changes to the site, with the exception of enlarging the main feeder channel throughout the site and redirecting its course just west of I-5.
3. **Alternative 1B** provides a more substantial change to the existing site to create a greater diversity of habitats while using the existing lagoon inlet. A new subtidal basin off the main channel is created in the Central Basin. The channel in the East Basin is significantly enlarged in cross-sectional area to promote more tidal exchange east of I-5.
4. **Alternative 2A** also provides changes to the existing site to create a greater diversity of habitats than presently exist. Seawater would enter the Lagoon via a new tidal inlet located south of the existing inlet and a new subtidal basin would be created just landward of the new inlet in the West and Central Basins. The existing tidal inlet would no longer function. The channel in the East Basin is identical to that for Alternative 1B.

The selection of optimum channel widths (for bridge lengths) and channel depths were based on a sensitivity analysis conducted for each bridge crossing under typical dry weather tidal fluctuations and extreme stormflow conditions (100-year storm and 100-year combined water levels). Tidal range was used as the primary indicator for benefits to the wetland ecosystem, and extreme flood elevations were used to evaluate the potential for flooding of Manchester Avenue. Using these indicators, the optimum channel width and depth were identified as the point at which tidal range and flood conveyance are most favorable and further increases in channel width and depth result in only minimal benefit. For the case of Hwy 101, the presence

of hard bottom reef and bedrock limits the channel depth to an elevation of -4 feet, NGVD. Table 1 presents the optimum channel widths and depths for each bridge and each SELRP alternative.

Table 1: Summary of Optimized Bridge Dimensions for Each Alternative

Alternative	Hwy 101 Bridge		Railroad Bridge		I-5 Bridge	
	Bottom Width (ft)	Channel Invert (ft,NGVD)	Bottom Width (ft)	Channel Invert (ft,NGVD)	Bottom Width (ft)	Channel Invert (ft,NGVD)
No Project	105	-4	187	-5.5	130	-6
1A	115	-4	187	-5.5	130	-6
1B	130	-4	187	-5.5	261	-6
2A	200	-6.5	590	-15	261	-6.5

Key findings from the optimization modeling study are summarized below:

- For alternatives which rely on the existing inlet channel (No Project, Alternative 1A, and Alternative 1B), the existing Hwy 101 Bridge structure and the Railroad Bridge structure have sufficient spans and are not limiting factors for tidal range or flood conveyance. The limiting factor for these alternatives is the long and narrow inlet channel between Hwy 101 and the Railroad Bridge. The main channel through the Central Basin is also narrow, shallow, and sinuous in planform resulting in additional energy losses during normal tidal fluctuations and extreme flood events.
- There is no benefit to tidal flows and storm flow convenayce from increasing the existing I-5 Bridge channel dimension for No Project and Alternative 1A conditions. Regardless of the I-5 Bridge channel dimension, Manchester Avenue will experience flooding in the East Basin during a 100-year event. The existing I-5 Bridge channel dimension actually helps prevent additional flooding of Manchester Avenue in the Central Basin by attenuating peak flows in the East Basin. This attenuation results in higher flood levels in the East Basin, but little or no flooding in the Central Basin. If the I-5 Bridge channel is widened, flood elevations are lowered in the East Basin, but raised in the Central Basin causing flooding of Manchester Avenue in both basins.
- Bridge optimization modeling of Alternative 1B suggested that increasing the I-5 Bridge channel width to 261 feet would relieve some flooding of Manchester Avenue in the East Basin. Portions of the roadway will still experience flooding, however, an increased bridge channel width would reduce flood levels below a significant length of roadway in the East Basin.

- For Alternative 2A, the optimization modeling study supported the recommended bridge channel dimensions identified in the SELRP Feasibility Studies (M&N 2010[a,b]). A Hwy 101 inlet channel width of 200 feet, a railroad channel width of 590 feet and an I-5 channel width of 261 feet were found to provide optimum tidal range and flood conveyance.

TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 SCOPE OF WORK 5

3.0 SAN ELIJO LAGOON RESTORATION PROJECT ALTERNATIVES 7

3.1 No Project - Existing Conditions 7
3.2 Alternative 1A – Minimum Changes 7
3.3 Alternative 1B – Maximum Habitat Diversity, Existing Inlet location 10
3.4 Alternative 2A – Maximum Habitat Diversity, New Inlet location 10

4.0 HYDRODYNAMIC MODEL DESCRIPTION 13

4.1 Model Setup 14
4.2 Model Area 15
4.3 Bathymetry 16
4.4 Finite Element Mesh 18
4.5 Boundary Conditions 24
4.5.1 Tides 24
4.5.2 Modeling Tidal Series 25
4.5.2.1 Extreme Tidal Series 26
4.5.2.2 Flood Flows from Creeks 27

5.0 DESCRIPTION OF EXISTING BRIDGE CHANNEL DIMENSIONS 29

5.1 Hwy 101 Bridge 29
5.2 Railroad Bridge 30
5.3 Interstate 5 Freeway Bridge 31

6.0 ANALYSES TO ACHIEVE OPTIMAL TIDAL RANGE 32

6.1 No Project Tidal Range Optimization 34
6.2 Alternative 1A Tidal Range Optimization 37
6.3 Alternative 1B Tidal Range Optimization 40
6.4 Alternative 2A Tidal Range Optimization 43

7.0 ANALYSES TO ACHIEVE OPTIMAL FLOOD ELEVATIONS 48

7.1 No Project Flood Optimization 51
7.2 Alternative 1A Flood Optimization 54
7.3 Alternative 1B Flood Optimization 57
7.4 Alternative 2A Flood Optimization 60

8.0 SUMMARY OF EXISTING AND OPTIMIZED BRIDGE CHANNEL DIMENSIONS 63

8.1 Highway 101 63
8.2 Railroad 67
8.3 I-5 70

9.0 ANALYSES OF VELOCITY AND SEDIMENTATION 73

9.1 Analyses of Tidal Velocities Under Bridges 73
9.2 Analyses of Extreme Flood Velocities Under Bridges 73
9.3 Analyses of Sedimentation During the 100-Year Flood 74

10.0 ANALYSIS OF RESIDENCE TIME 79

10.1 Methodology 79
10.2 Boundary Conditions 80
10.2.1 Hydraulic Input 80
10.2.2 Concentration Input 80
10.3 Residence Time Results 80

11.0 TIDAL INUNDATION FREQUENCY ANALYSIS 84

11.1 Existing Inlet Location Alternatives 84
11.2 New Inlet Location Alternative 84

12.0 HYDRAULIC EFFECTS OF SEA LEVEL RISE 88

12.1 Water Surface Elevations for Existing Inlet Location Alternatives 88
12.2 Water Surface Elevations for the New Inlet Location Alternative 88
12.3 Peak Flow Velocities for All Alternatives 88
12.4 Bridge Freeboards During High Water Conditions 94

13.0 FINDINGS AND RECOMMENDATIONS 95

14.0 REFERENCES 98

LIST OF FIGURES

Figure 1-1: Project Vicinity Map 3
Figure 1-2: Project Study Area 4
Figure 3-1: No Project - Existing Habitat 8
Figure 3-2: Alternative 1A 9
Figure 3-3: Alternative 1B 11
Figure 3-4: Alternative 2A 12
Figure 4-1: Numerical Modeling Area 16
Figure 4-2: Existing Bathymetry for the Entire Modeling Area 17
Figure 4-3: Existing Lagoon Bathymetry 18
Figure 4-4: RMA-2 Model Mesh for Existing Conditions (No Project) 20
Figure 4-5: RMA-2 Modeling Mesh for Alternative 1A 21
Figure 4-6: RMA-2 Modeling Mesh for Alternative 1B 22
Figure 4-7: RMA-2 Modeling Mesh for Alternative 2A 23
Figure 4-8: Modeling Tidal Series 26
Figure 4-9: Extreme Tidal Series 27
Figure 4-10: San Elijo Lagoon Flood Hydrographs 28
Figure 5-1: Hwy 101 Bridge (Existing Lagoon Inlet) 29
Figure 5-2: Railroad Bridge 30
Figure 5-3: Interstate-5 Bridge 31
Figure 6-1: Tide Gage Locations (No Project, Alternatives 1A & 1B) 32
Figure 6-2: Tide Gage Locations (Alternative 2A) 33
Figure 6-3: No Project Hwy 101 (Tidal Range vs Channel Width) 35
Figure 6-4: No Project I-5 Bridge (Tidal Range vs Channel Width) 36
Figure 6-5: Alternative 1A Hwy 101 (Tidal Range vs Channel Width) 38
Figure 6-6: Alternative 1A I-5 Bridge (Tidal Range vs Channel Width) 39
Figure 6-7: Alternative 1B Hwy 101 (Tidal Range vs Channel Width) 41
Figure 6-8: Alternative 1B I-5 Bridge (Tidal Range vs Channel Width) 42
Figure 6-9: Alternative 2A Hwy 101 Tidal Range (Invert of -6.5 feet, NGVD) 44
Figure 6-10: Alternative 2A Hwy 101 Tidal Range (Invert of -7 feet, NGVD) 44
Figure 6-11: Alternative 2A Hwy 101 Tidal Range (Invert of -5 feet, NGVD) 45
Figure 6-12: Alternative 2A I-5 Bridge (Tidal Range vs Channel Width) 46
Figure 7-1: Flood Profile Alignments 49
Figure 7-2: Flood Gage Locations on RMA-2 Mesh (No Project, Alternatives 1A & Alt 1B) 50
Figure 7-3: Flood Gage Locations on RMA-2 Mesh (Alternative 2A) 50
Figure 7-4: No Project Flood Optimization Profiles 52
Figure 7-5: Optimized No Project Model – 50-year & 100-year Flood Profiles 53
Figure 7-6: Alternative 1A Flood Optimization Profiles 55
Figure 7-7: Optimized Alternative 1A Model – 50-year & 100-year Flood Profiles 56
Figure 7-8: Alternative 1B Flood Optimization Profiles 58
Figure 7-9: Optimized Alternative 1B Model – 50-year & 100-year Flood Profiles 59
Figure 7-10: Alternative 2A Flood Optimization Profiles (Hwy101 = 200 feet) 61
Figure 7-11: Optimized Alternative 2A Model – 50-year & 100-year Flood Profiles 62
Figure 8-1: Existing Highway 101 Bridge Channel Cross-Section 65
Figure 8-2: Proposed Alt 2A Highway 101 Bridge Channel Cross-Section 66
Figure 8-3: Existing Railroad Bridge Channel Cross-Section 68
Figure 8-4: Proposed Alt 2A Railroad Bridge Channel Cross-Section 69
Figure 8-5: Existing I-5 Bridge Channel Cross-Section 71
Figure 8-6: Proposed I-5 Bridge Channel Cross-Section 72

Figure 9-1: Peak Velocity Distribution During a 100-Year Event for No Project 75
Figure 9-2: Peak Velocity Distribution During a 100-Year Event for Alternative 1A 75
Figure 9-3: Peak Velocity Distribution During a 100-Year Event for Alternative 1B 76
Figure 9-4: Peak Velocity Distribution During a 100-Year Event for Alternative 2A 76
Figure 10-1: Example of a Residence Time Plot 80
Figure 10-2: Residence Time (Days) for No Project Conditions 82
Figure 10-3: Residence Time (Days) for Alternative 1A 82
Figure 10-4: Residence Time (Days) for Alternative 1B 83
Figure 10-5: Residence Time (Days) for Alternative 2A 83
Figure 11-1: Gage Locations for Inundation Frequency Plots (No Project, Alt 1A & 1B) 85
Figure 11-2: Gage Locations for Inundation Frequency Plots (Alt 2A) 85
Figure 11-3: No Project Tidal Inundation Frequency 86
Figure 11-4: Alternative 1A Tidal Inundation Frequency 86
Figure 11-5: Alternative 1B Tidal Inundation Frequency 87
Figure 11-6: Alternative 2A Tidal Inundation Frequency 87
Figure 12-1: No Project 100-Year Flood Profile Comparison With Sea Level Rise 90
Figure 12-2: Alternative 1A 100-Year Flood Profile Comparison With Sea Level Rise 91
Figure 12-3: Alternative 1B 100-Year Flood Profile Comparison With Sea Level Rise 92
Figure 12-4: Alternative 2A 100-Year Flood Profile Comparison With Sea Level Rise 93

LIST OF TABLES

Table 4-1: Model Mesh Elements and Nodes 24
Table 4-2: Recorded Water Levels at La Jolla (1983-2001 Tidal Epoch) 24
Table 6-1: No Project Tide Optimization Models 34
Table 6-2: No Project Hwy 101 Tidal Range Results 35
Table 6-3: No Project I-5 Tidal Range Results 36
Table 6-4: Alternative 1A Tide Optimization Models 37
Table 6-5: Alternative 1A Hwy101 Tidal Range Results 38
Table 6-6: Alternative 1A I-5 Tidal Range Results 39
Table 6-7: Alternative 1B Tide Optimization Models 40
Table 6-8: Alternative 1B Hwy101 Tidal Range Results 41
Table 6-9: Alternative 1B I-5 Tidal Range Results 42
Table 6-10: Alternative 2A Tide Optimization Models 43
Table 6-11: Alternative 2A Hwy 101 Tidal Range Results 45
Table 6-12: Alternative 2A I-5 Tidal Range Results 47
Table 7-1: No Project Flood Optimization Models 51
Table 7-2: Alternative 1A Flood Optimization Models 54
Table 7-3: Alternative 1B Flood Optimization Models 57
Table 7-4: Alternative 2A Flood Optimization Models 60
Table 8-1: Highway 101 Optimized Bridge Channel Dimensions 64
Table 8-2: Railroad Optimized Bridge Channel Dimensions 67
Table 8-3: I-5 Optimized Bridge Channel Dimensions 70
Table 9-1: Tidal Velocity (fps) at Bridge Crossings During the Dry Season 73
Table 9-2: Peak Velocity (fps) at Bridge Crossings During a 100-Year Storm 74
Table 9-3: Duration (Hours) of Stormflow Drainage Under a 100-Year Storm 77
Table 10-1: Summary of Residence Time (Days) 81
Table 12-1: 100-Year Peak Flood Velocity (fps) at Bridge Crossings in Year 2100 89
Table 12-2: Summary of Bridge Soffit and 100-Year Flood Elevations 94
Table 13-1: Summary of Optimized Bridge Dimensions for Each Alternative 95

LIST OF ACRONYMS

CALTRANS	California Department of Transportation
CB	Central Basin (figure legends only)
cfs	Cubic Feet per Second
CSCC	California State Coastal Conservancy
EB	East Basin (figure legends only)
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
fps	Feet per Second
Hwy	Highway
HDR	HDR, Inc. (Company)
I-5	Interstate 5
KDM	KDM Meridian (Company)
LOSSAN	Los Angeles – San Diego – San Luis Obispo Rail Corridor Agency
MHHW	Mean Higher High Water
MLLW	Mean Lower Low Water
M&N	Moffatt & Nichol
MSL	Mean Sea Level
NAVD	North American Vertical Datum
NCTD	North Coast Transit District
NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NP	No Project (figure legends only)
PDC	Project Design Consultants (Company)
RMA-2	Resource Management Associates (2D numerical model for surface flow)
RMA-4	Resource Management Associates (2D numerical model for water quality)
RR	Railroad
SANDAG	San Diego Association of Governments
SDRWQCB	San Diego Regional Water Quality Control Board
SELC	San Elijo Lagoon Conservancy
SELRP	San Elijo Lagoon Restoration Project
SLR	Sea Level Rise
TABS2	The Open Channel Flow and Sedimentation (Numerical Modeling System)
TEA	Tidal Epoch Analysis
USACE	United States Army Corps of Engineers
WB	West Basin (figure legends only)
WSE	Water Surface Elevation
2D	Two-dimensional

1.0 INTRODUCTION

For the past several years, Caltrans and 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 Project. Implementation of this project will require work within the major coastal lagoons of northern San Diego County. The project will include new bridge structures across most of the lagoons, including the San Elijo Lagoon. The objective of this study was to evaluate a range of channel dimensions under bridges at Interstate 5 Freeway (I-5), North County Transit District Railroad (Railroad), and Highway (Hwy) 101 and identify a combination of optimized channel widths that provide the most favorable conditions for tidal range, flood conveyance and other environmental benefits throughout the lagoon.

The San Elijo Lagoon is a coastal wetland with significant biological and ecological resources located approximately 20 miles north of the City of San Diego, between the Cities of Solana Beach and Encinitas, as shown in Figure 1-1. The California Department of Fish and Game generally owns the San Elijo Lagoon west of Interstate 5 (I-5), the County of San Diego generally owns the Lagoon east of I-5, and the San Elijo Lagoon Conservancy (SELC) owns smaller areas west of I-5. The study area boundary is illustrated in Figure 1-2.

Concurrent with the North Coast Corridor Project, the San Elijo Lagoon Restoration Project (SELRP) is pursuing an effort to restore the Lagoon functions and values given the constraints placed on it by surrounding current and historic development activities. The overarching goal of the SELRP is to protect, restore, and then maintain, via adaptive management, the San Elijo Lagoon ecosystem. The SELRP aims to enhance the tidal prism of the Lagoon by proposing modifications to known infrastructure “choke points” such as Hwy 101, the Railroad, and the I-5 freeway. The four alternatives under consideration for the SELRP and evaluated in this study are listed below:

1. **No Project** proposes no grading within the lagoon except for on-going maintenance dredging of the inlet channel.
2. **Alternative 1A** provides minimal physical changes to the site, with the exception of enlarging the main feeder channel throughout the site and redirecting its course just west of I-5.
3. **Alternative 1B** provides a more substantial change to the existing site to create a greater diversity of habitats while using the existing lagoon inlet. A new subtidal basin off the main channel is created in the Central Basin. The channel in the East Basin is significantly enlarged in cross-sectional area to promote more tidal exchange east of I-5.
4. **Alternative 2A** also provides changes to the existing site to create a greater diversity of habitats than presently exists. Seawater would enter the Lagoon via a new tidal inlet located south of the existing inlet and a new subtidal basin would be created just landward of the new inlet in the West and Central Basins. The existing tidal inlet would no longer function. The channel in the East Basin is identical to that for Alternative 1B.

The selection of optimum channel dimensions under bridges were based on a sensitivity analysis conducted for each bridge crossing under typical dry weather tidal fluctuations and extreme storm conditions. Predicted tidal range during a dry weather spring tide cycle was used as the primary indicator for benefits to the wetland ecosystem. The extreme stormflow condition simulated the 100-year hydrograph concurrent with an extreme tidal series adjusted to match FEMA's base flood elevation at the lagoon inlet. Extreme flood elevations used to evaluate the potential for flooding of Manchester Avenue. Using these indicators, the optimum channel width and depth under bridges at each location were identified as the point at which tidal range and flood conveyance are most favorable and further increases in channel width and depth under bridges result in only minimal benefit.

Once the optimized channel dimensions under Hwy 101, the Railroad, and I-5 Bridges were identified for each of the four alternatives, additional analyses were conducted to evaluate the impacts on sedimentation, water quality, and tidal inundation frequency throughout the Lagoon. Additional flood modeling was performed to determine maximum flood elevations during a 50-year event and the influence of projected sea level rise (SLR) on the 100-year flood elevations.

Figure 1-1: Project Vicinity Map

(Source: EDAW 2009)

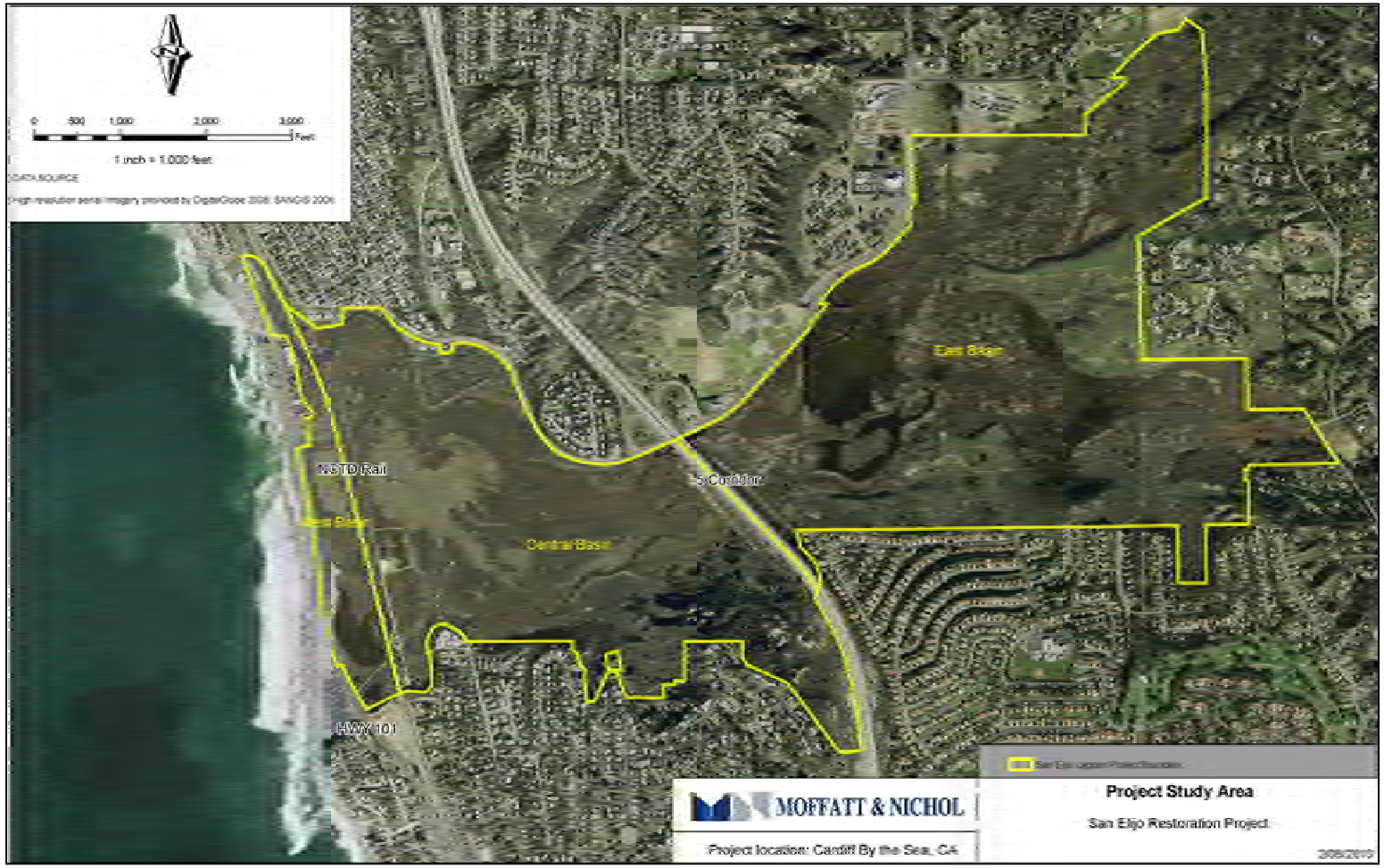


Figure 1-2: Project Study Area

2.0 SCOPE OF WORK

The scope of work for this study consisted of analyzing tidal and storm flood hydraulics for a range of channel dimensions under bridges using the calibrated RMA-2 and RMA-4 models developed for the SELRP (Moffatt & Nichol [M&N] 2010[a,b]) as a starting point. The RMA models were then updated with topography and bathymetry data from a 2011 survey and some minor modifications to the proposed grading for the SELRP alternatives.

Multiple RMA-2 models were developed to simulate changes to channel dimensions under bridges at Hwy 101, the Railroad, and I-5 for each of the four SELRP alternatives. The results of these simulations were analyzed to determine the "optimum" channel dimensions of each bridge crossing for each alternative. The scope of this effort involved a substantial number of model simulations considering there were four alternatives modeled, three bridge crossings of each alternative, multiple channel widths at each bridge crossing, and evaluation of each model under dry weather conditions (tide only), extreme stormflow conditions (100-year and 50-year), and potential SLR projections for the year 2100. Approximately 100 different models and over 200 simulations were used to generate the final results presented in this report.

Specific modeling tasks included:

1. Modifying the modeling domain (mesh grid) to optimally represent existing bathymetry based on the 2011 survey data.

2. Refining the modeling domain (mesh grid) to optimally represent proposed grading limits and depths based on most recent updates to the SELRP alternatives.

3. Performing RMA-2 tidal hydrodynamic modeling runs using a spring tide series and assuming dry weather flow conditions. Determining the maximum tidal range for different channel widths under each bridge crossing and for each alternative.

4. Performing RMA-2 flood modeling using the 50-year and 100-year flood hydrographs and a modified spring tide series elevated to match Federal Emergency Management Agency's (FEMA's) projected extreme water levels at the lagoon inlet. Plotting the maximum flood elevation throughout the lagoon relative to the elevation of Manchester Avenue. The 100-year flood modeling was performed for all the models developed for different channel widths (translated into bridge lengths) tested at each bridge crossing and for each alternative. The 50-year flood modeling was performed only for the "optimized" model of each alternative.

5. Evaluating the potential for fluvial sedimentation throughout the lagoon based on predicted flood velocities for modeling performed under task #4 above. Qualitatively comparing the results against different channel widths and determining if there is potential for sedimentation to adversely impact habitat.

6. Evaluating impacts to water quality by performing RMA-4 simulations to estimate residence times in the Lagoon using the optimized channel widths for each alternative.

7. Developing tidal inundation frequency curves and analyzing the range of elevations for each habitat type.

8. Performing RMA-2 flood modeling using the 100-year hydrograph and a SLR projection of 55 inches in the year 2100 added to the spring tide series.

3.0 SAN ELIJO LAGOON RESTORATION PROJECT ALTERNATIVES

The bridge optimization analyses were conducted for the four SELRP alternatives that have been identified by the Stakeholder Committee as likely to be included in the environmental document. They include:

- No Project – Existing Conditions;
- Alternative 1A – Minimum Changes;
- Alternative 1B – Maximum Habitat Diversity, Existing Inlet Location; and
- Alternative 2A – Maximum Habitat Diversity, New Inlet Location.

The RMA-2 models were based on the most recent conceptual design of each alternative. Brief descriptions of the alternatives are provided below, and habitat graphics of all alternatives are provided in this section.

3.1 No Project - Existing Conditions

No Project assumes no changes are made to the project site and existing conditions remain into perpetuity. The Lagoon presently experiences mouth constriction and manual re-opening annually, and sometimes more frequently. Tidal flushing is restricted, and water quality conditions are impaired for nutrients and sediment. Habitat is distributed at elevations and locations that are related to relic closed mouth conditions, and are progressively transitioning to distributions more reflective of managed mouth conditions. For example, mudflat habitat is located too high for a full tidal lagoon because it formed when the mouth was closed and lagoon water levels were higher from impoundment. Now that the mouth is managed to be remain open, the mudflat is converting to vegetated marsh because hydrologic conditions are favorable for salt marsh plant growth. Figure 3-1 shows existing conditions.

3.2 Alternative 1A – Minimum Changes

Alternative 1A provides minimal physical changes to the site, with the exception of enlarging the main feeder channel throughout the site and redirecting its course just west of I-5. The main tidal channel is also extended farther into the East Basin and existing constricted channel connections are cleared and enlarged. Existing habitat areas will essentially remain intact. The tidal prism of Alternative 1 will slightly increase compared to existing conditions. A relatively small area of transitional habitat above tidal elevations will be placed in the northwest portion of the Central Basin. Figure 3-2 shows Alternative 1A.

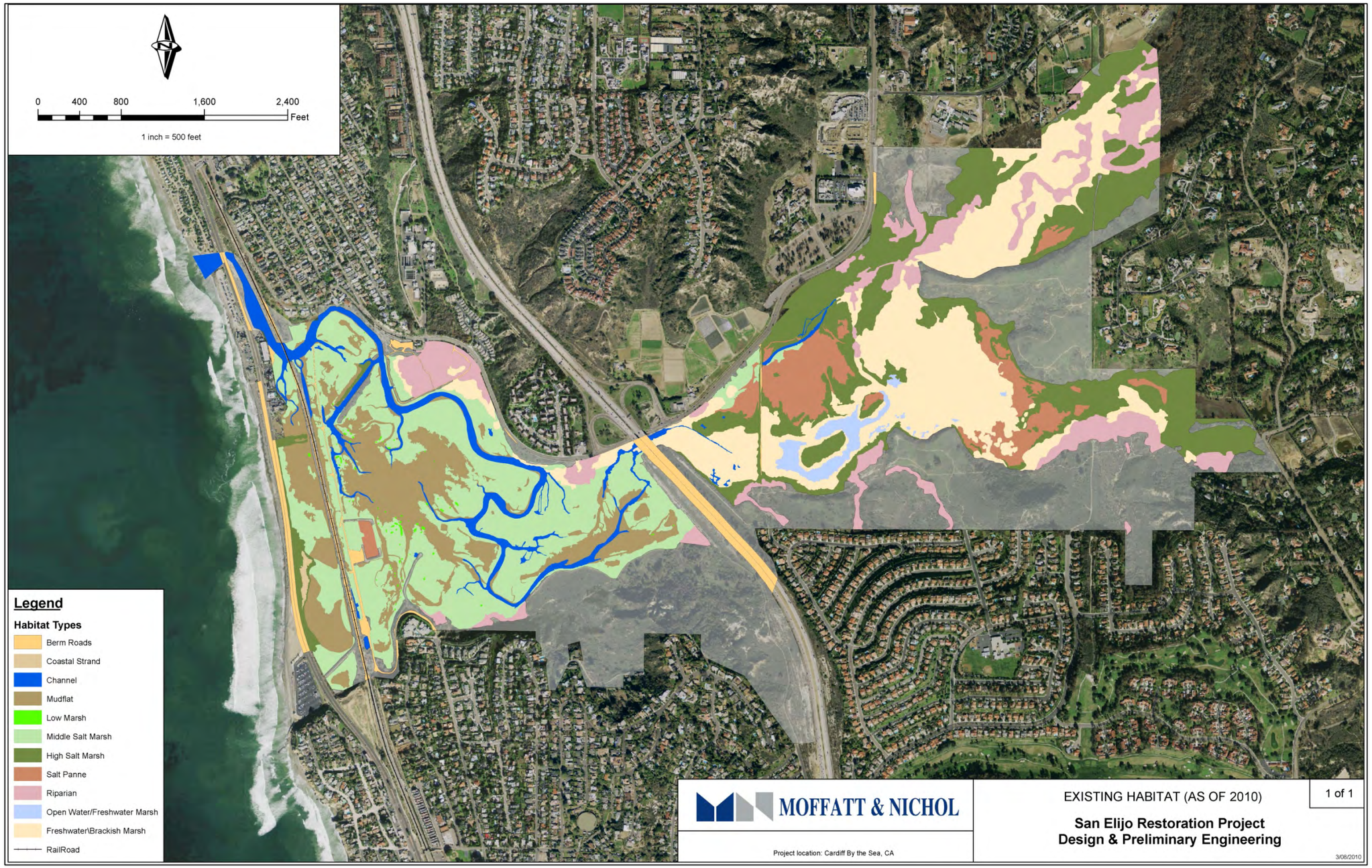


Figure 3-1: No Project - Existing Habitat

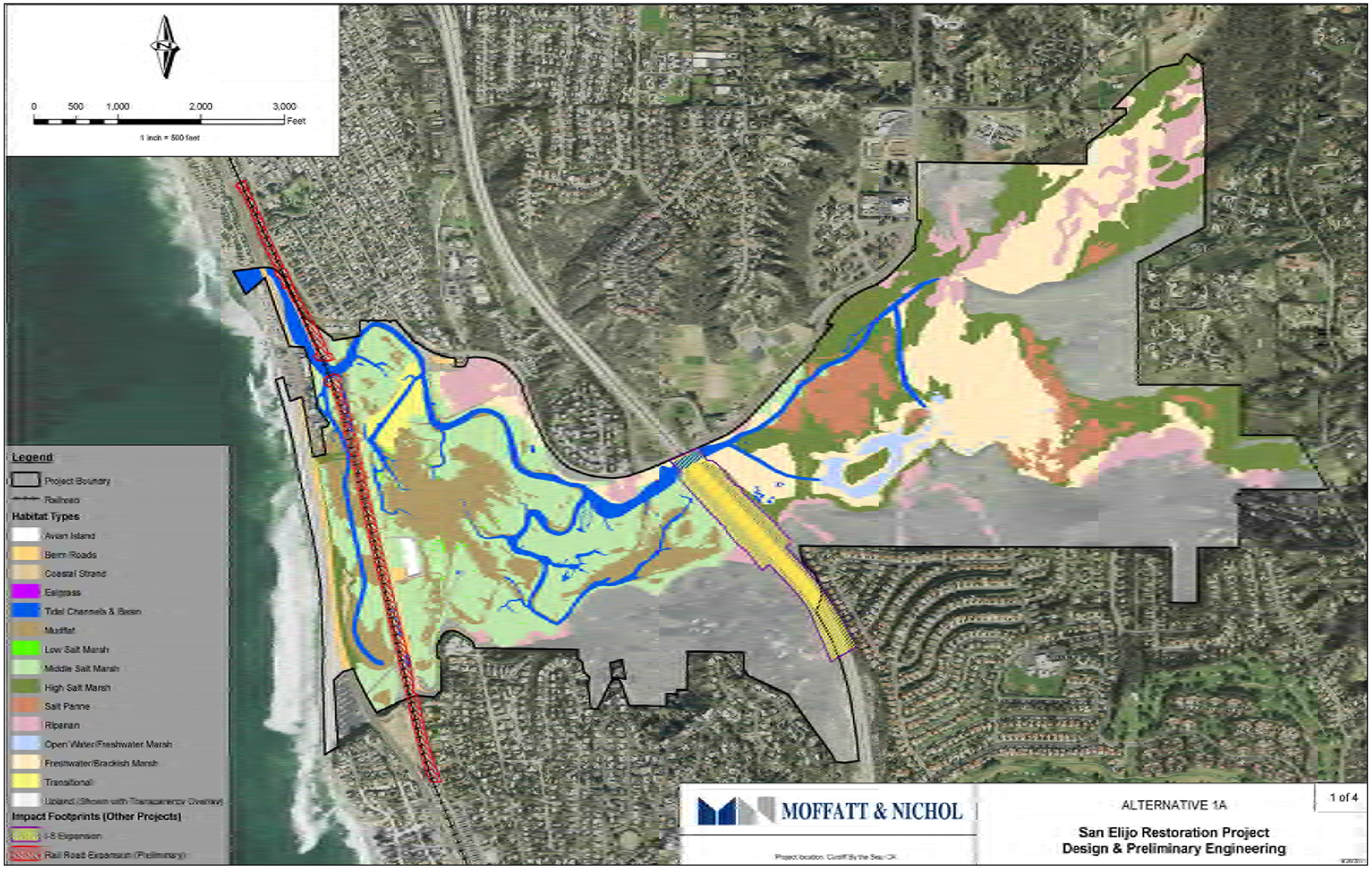


Figure 3-2: Alternative 1A

3.3 Alternative 1B – Maximum Habitat Diversity, Existing Inlet location

Alternative 1B provides a more substantial change to the existing site to create a greater diversity of habitats than currently exists. The existing tidal inlet remains the source of seawater, and the main tidal channel extends throughout the Lagoon. A new subtidal basin off the main channel is created in the Central Basin. The main feeder channel is redirected just west of I-5, and extended farther into the East Basin. The channel in the East Basin is significantly enlarged in cross-sectional area to promote more tidal exchange east of I-5. The tidal prism of Alternative 1B will significantly increase compared to Alternative 1A. Non-tidal habitat areas will still exist in the East Basin. Several areas of transitional habitat above tidal elevations will be placed in the western portion of the Central Basin. Figure 3-3 shows Alternative 1B.

3.4 Alternative 2A – Maximum Habitat Diversity, New Inlet location

Alternative 2A also provides changes to the existing site to create a greater diversity of habitats than presently exists. Seawater would enter the Lagoon via a new tidal inlet located south of the existing inlet and a new subtidal basin would be created just landward of the new inlet in the West and Central Basins. The main tidal channel would extend throughout the Lagoon and be redirected just west of I-5, and extend into the East Basin. The channel in the East Basin is identical to that for Alternative 1B. The tidal prism of Alternative 2A will increase compared to Alternative 1B. Non-tidal habitat areas remain in the East Basin. Transitional habitat areas above tidal elevations will also be included in the Central Basin. Figure 3-4 shows Alternative 2A.

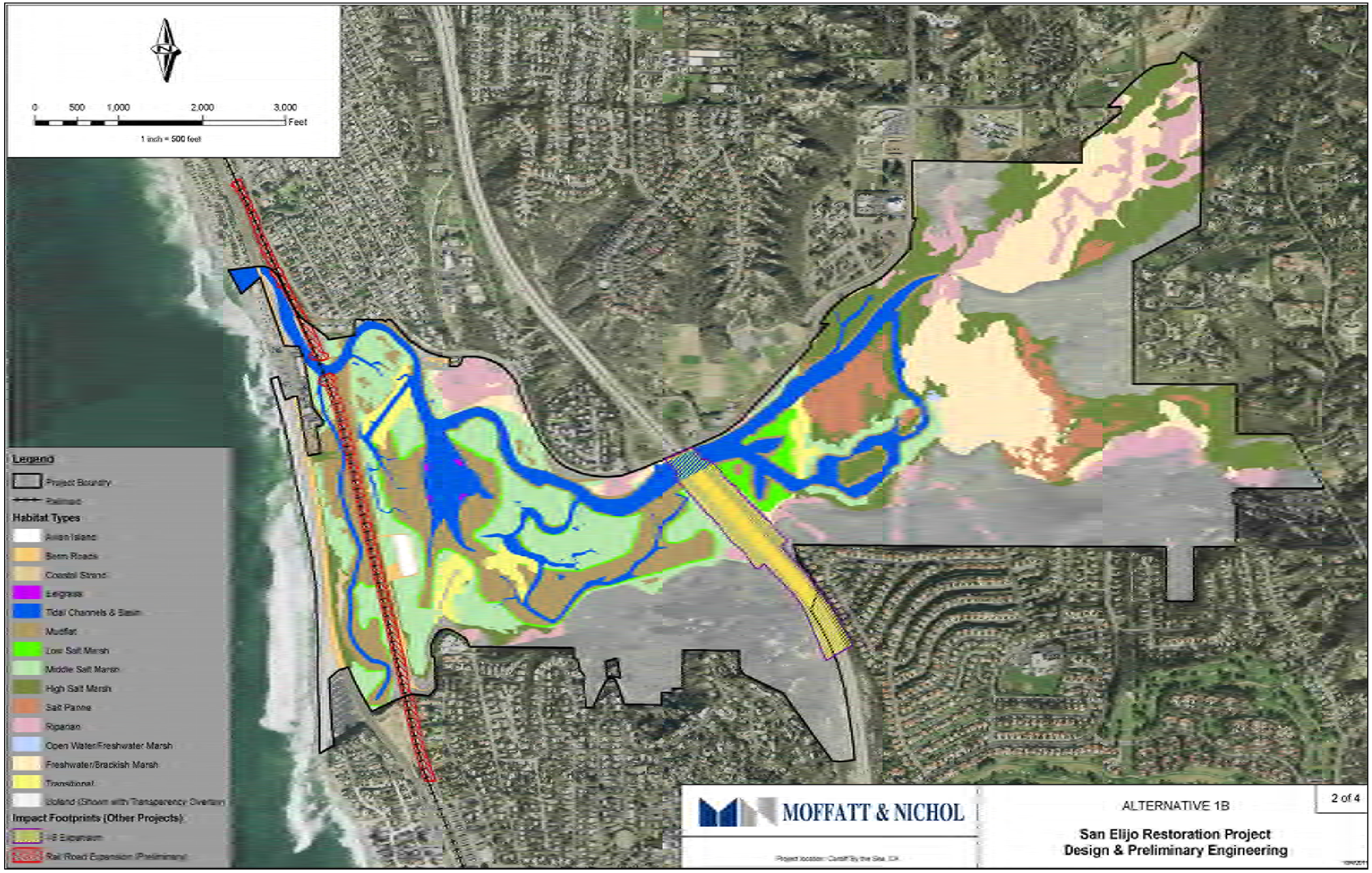


Figure 3-3: Alternative 1B

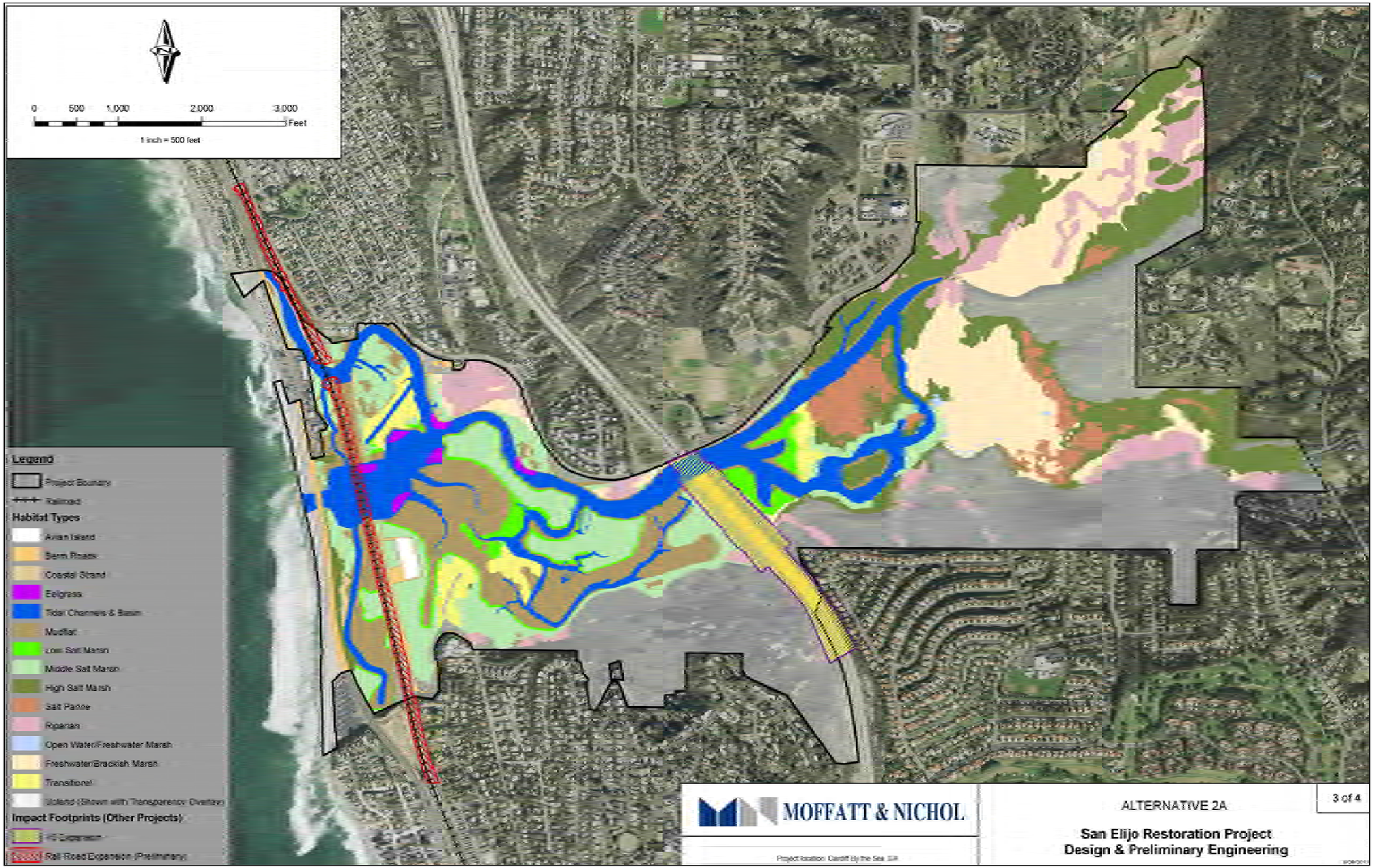


Figure 3-4: Alternative 2A

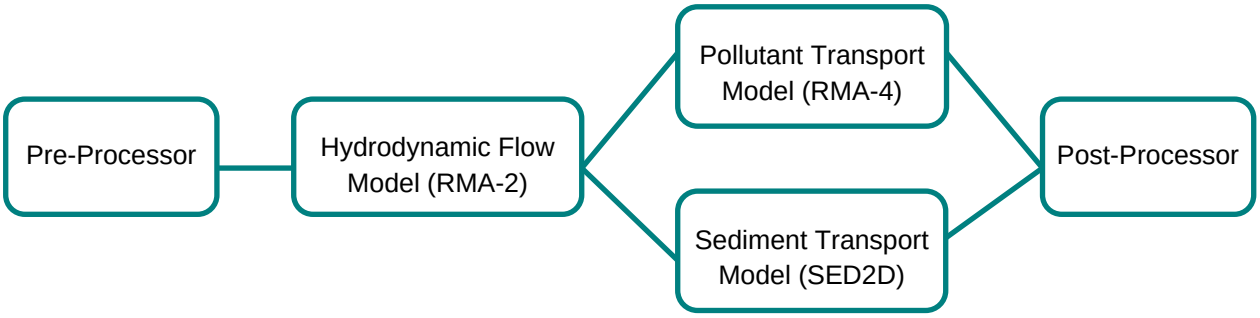
4.0 HYDRODYNAMIC MODEL DESCRIPTION

Numerical modeling of tidal and flood hydraulics was performed for each alternative to simulate wetland hydraulics under both dry weather tidal fluctuations and extreme storm conditions. The numerical modeling system used in this study is summarized in the following sections. The TABS2 (McAnally and Thomas 1985) modeling system was applied to this project. TABS2 was developed by the U.S. Army Corps of Engineers (USACE), and consists of the following components:

1. Two-dimensional, vertically-averaged finite element hydrodynamics model (RMA-2);
2. Pollutant transport/water quality model (RMA-4); and
3. The sediment transport model (SED2D-WES).

TABS2 is a collection of generalized computer programs and pre- and post-processor utility codes integrated into a numerical modeling system for studying 2-D depth-averaged hydrodynamics, transport and sedimentation problems in rivers, reservoirs, bays, and estuaries. The finite element method provides a means of obtaining an approximate solution to a system of governing equations by dividing the area of interest into smaller sub-areas called elements. Time-varying partial differential equations are transformed into finite element form and then solved in a global matrix system for the modeled area of interest. The solution is smooth across each element and continuous over the computational area. This modeling system is capable of simulating tidal wetting and drying of marsh and intertidal areas of the estuarine system.

A schematic representation of the system is shown below. TABS2 can be used either as a stand-alone solution technique or as a step in the hybrid modeling approach. RMA-2 calculates water surface elevations and current patterns which are input to the pollutant transport and sediment transport models. Existing and proposed wetland geometry can be analyzed to determine the impact of project designs on flow, circulation (this study), salinity and water quality and sedimentation on the estuarial system. The three models listed above are solved by the finite element method using Galerkin weighted residuals.



TABS2 Schematic

The hydrodynamic model simulates 2-D flow in rivers and estuaries by solving the depth-averaged Navier Stokes equations for flow velocity and water depth. The equations account for

friction losses, eddy viscosity, Coriolis forces and surface wind stresses. The general governing equations are:

$$\frac{\partial h}{\partial t}+\frac{\partial(hu)}{\partial x}+\frac{\partial(hv)}{\partial y}=0$$

$$h\frac{\partial u}{\partial t}+uh\frac{\partial u}{\partial x}+vh\frac{\partial u}{\partial y}+gh\frac{\partial a}{\partial x}+gh\frac{\partial h}{\partial x}-h\frac{\varepsilon_{xx}}{\rho}\frac{\partial^2 u}{\partial x^2}-h\frac{\varepsilon_{xy}}{\rho}\frac{\partial^2 u}{\partial y^2}+S_{f_x}+\tau_x=0$$

$$h\frac{\partial v}{\partial t}+uh\frac{\partial v}{\partial x}+vh\frac{\partial v}{\partial y}+gh\frac{\partial a}{\partial y}+gh\frac{\partial h}{\partial y}-h\frac{\varepsilon_{yx}}{\rho}\frac{\partial^2 v}{\partial x^2}-h\frac{\varepsilon_{yy}}{\rho}\frac{\partial^2 v}{\partial y^2}+S_{f_y}+\tau_y=0$$

where:

u,v = x and y velocity components

t = time

h = water depth

a = bottom elevation

Sf_x = bottom friction loss term in x-direction

Sf_y = bottom friction loss term in y-direction

τ_x = wind and Coriolis stresses in x-direction

τ_y = wind and Coriolis stresses in y-direction

ε_{xx} = normal eddy viscosity in the x-direction on x-axis plane

ε_{xy} = tangential eddy viscosity in the x-direction on y-axis plane

ε_{yx} = tangential eddy viscosity in the y-direction on x-axis plane

ε_{yy} = normal eddy viscosity in the y-direction on y-axis plane

Wind stress is computed using the following formula:

$$\tau_S = 3.8\cdot 10^{-6} W^2$$

where

τs is wind stress (lb/ft/sec2) on the water surface, and

W is the wind speed in miles per hour at 10 meters (33 feet) above the water surface.

4.1 Model Setup

The setup for the tidal and flood hydraulic models for existing conditions (No Project) and Alternatives 1A through 2A included updates to bathymetry, proposed wetland habitat area, mesh selection, and boundary conditions. An RMA-2 model previously created by the USACE (2006) and calibrated as part of the SELRP Hydrology & Hydraulics Study (M&N 2010[a]) was used as the baseline model for this study. That RMA-2 model setup was modified to represent

the most current topographic and bathymetry data, surveyed in March 2011, and also incorporate recent updates to the proposed grading associated with each project alternative.

The horizontal coordinate system for the modeling work is North American Datum (NAD) 83, California state plane zone 6, and the vertical datum is National Geodetic Vertical Datum (NGVD) 1929, which was equivalent to Mean Sea Level (MSL) at that time. As sea level has risen since 1929, NGVD is lower than existing MSL by approximately 0.44 feet. Both horizontal and vertical units are in feet.

4.2 Model Area

The numerical model covers the nearshore ocean and the area below the +10 foot NGVD contour line of West Basin, Central Basin, and East Basin, as shown in Figure 4-1. The original USACE model, which only covers the tidally-influenced area approximately below the +6.5 foot contour line, was raised to the +10 foot contour line to contain water levels during the 100-year flood condition.

The ocean boundary is approximately one mile from the shoreline. The side boundaries of the offshore area are approximately one mile north and two miles south from the existing inlet location, so the offshore ocean area will remain the same for all alternatives, regardless of the location of the tidal inlet.

Figure 4-1: Numerical Modeling Area

4.3 Bathymetry

The ocean bathymetry used in the model is the same data used in model meshes created by the USACE (2006) for this location. The USACE created the mesh of the lagoon area based on the 1990 topographic survey (Towill Inc. 1990). A recent survey of the San Elijo Lagoon was conducted by KDM Meridian in March 2011 and included aerial photogrammetric mapping augmented with a bathymetric survey of the main channel and tributaries. This data set provides the existing surface both above and below the water level for the entire model area east of Hwy 101. All project alternatives were updated to represent the 2011 survey and bathymetry data within the Lagoon. The ocean bathymetry was beyond the limits of the KDM Meridian survey and therefore was not updated. The ocean bathymetry data used in the model will not affect results for two reasons. One reason is that changes in ocean bathymetry mostly are confined to nearshore areas with little or no change in offshore bathymetry. A second reason is that modeling results are not sensitive to small changes in bathymetry because the relatively large ocean depths result in little or no energy loss during fluctuating sea levels. The No Project and Alternative 1A models are most sensitive to the updated bathymetry since there is little or no grading proposed for these alternatives. The 2011 survey indicates the ground surface throughout most of the lagoon is about 0.5 to 1 foot higher when compared to the 1990 topography. For this reason, there will be some disparity between the results presented herein

and previous results reported in the SELRP feasibility studies (M&N 2010[a,b]). The disparity will be larger for No Project and Alternative 1A models since little or no change to the existing bathymetry is proposed for these alternatives.

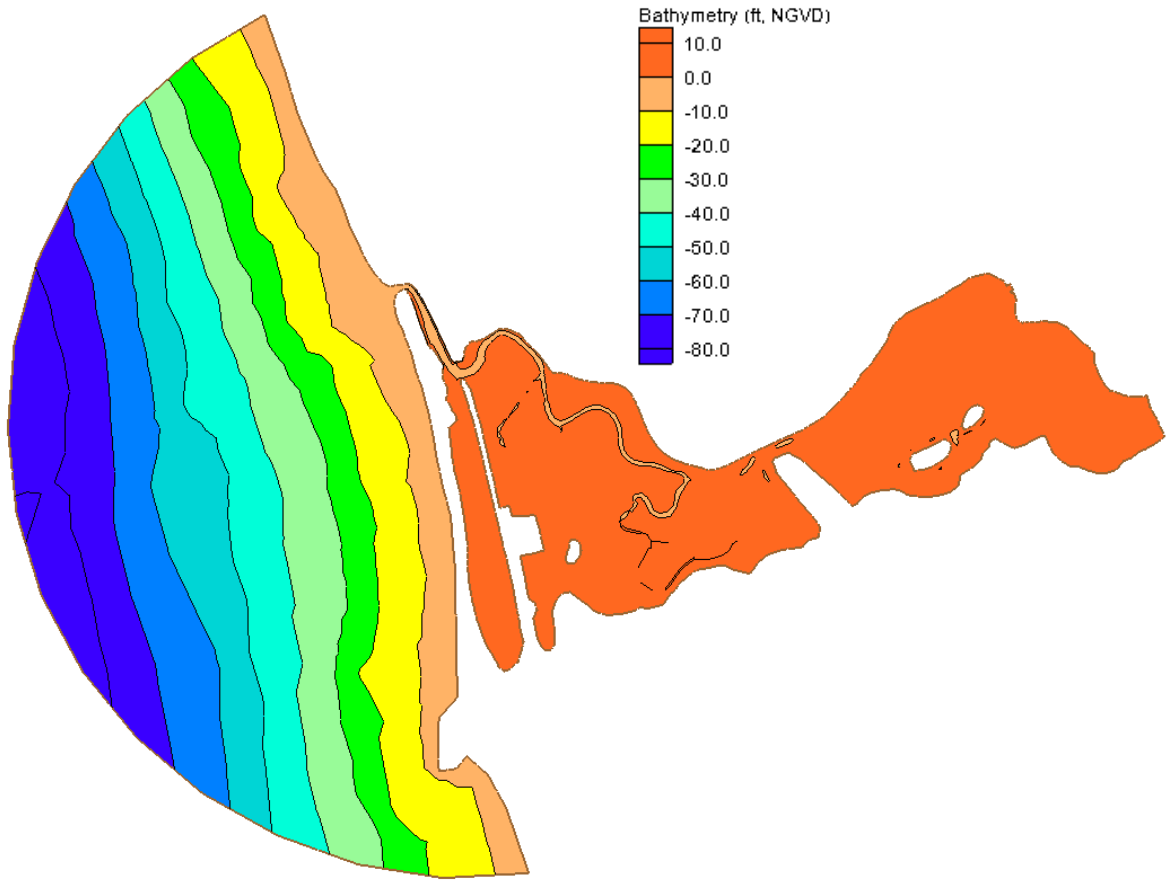


Figure 4-2: Existing Bathymetry for the Entire Modeling Area

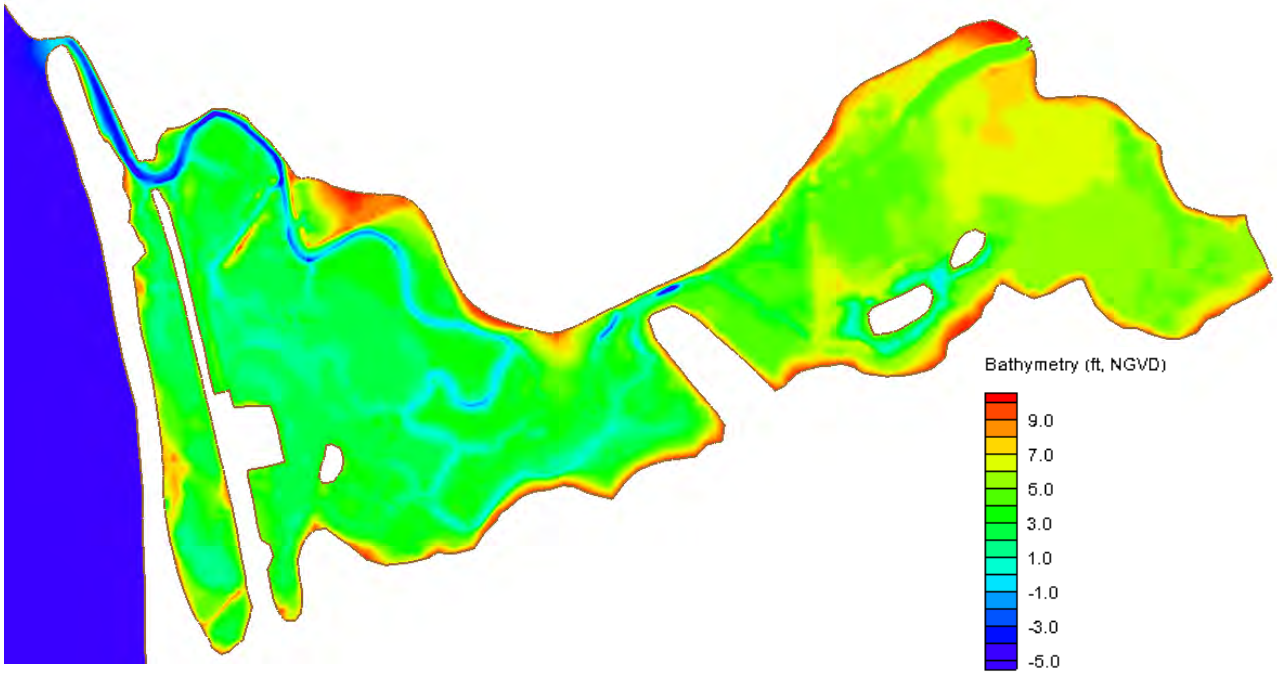


Figure 4-3: Existing Lagoon Bathymetry

4.4 Finite Element Mesh

The RMA-2 modeling system requires that the estuarial system be represented by a network of nodal points and elements, points defined by coordinates in the horizontal plane and water depth, and areas made up by connecting these adjacent points, respectively. Nodes can be connected to form 1- and 2-D elements, having from two to four nodes. The resulting nodal/element network is commonly called a finite element mesh and provides a computerized representation of the estuarial geometry and bathymetry.

It is noted that evaluations discussed herein correspond to 2-D analyses. Each alternative was sufficiently dissimilar that a unique finite element mesh was developed to reflect the bathymetry and wetland boundaries for each alternative considered.

The two important aspects to consider when designing a finite element mesh are: (1) determining the level of detail necessary to adequately represent the estuary; and (2) determining the extent or coverage of the mesh. Accordingly, the bathymetric features of the estuary generally dictate the level of detail appropriate for each mesh. These concerns present trade-offs for the modeler to consider. Too much detail can lead the model to run slowly or even become unstable and “crash”. Too little detail renders the results less useful. For this project, a balance was achieved with a stable and efficient model that yields the level of detail required for planning. The model described in this section is numerically robust and capable of simulating tidal elevations, flows, and constituent transport with reasonable resolution.

There are several factors used to decide the aerial extent of each mesh. First, it is desirable to extend mesh open boundaries to areas which are sufficiently distant from the proposed areas of change so as to be unaffected by that change. Additionally, mesh boundaries must be located along sections where conditions can reasonably be measured and described to the model. Finally, mesh boundaries can be extended to an area where conditions have been previously collected to eliminate the need to interpolate between the boundary conditions from other locations.

The finite element meshes for a representative model of each alternative are shown in Figure 4-4 through Figure 4-7. Each mesh contains a section of ocean sufficiently large enough to eliminate potential model boundary effects. The wetland portion of the mesh is bounded by Hwy 101, Manchester Avenue and dry land considered to be at the outermost extents of the flood influence. The nearshore mesh is the same for each alternative.

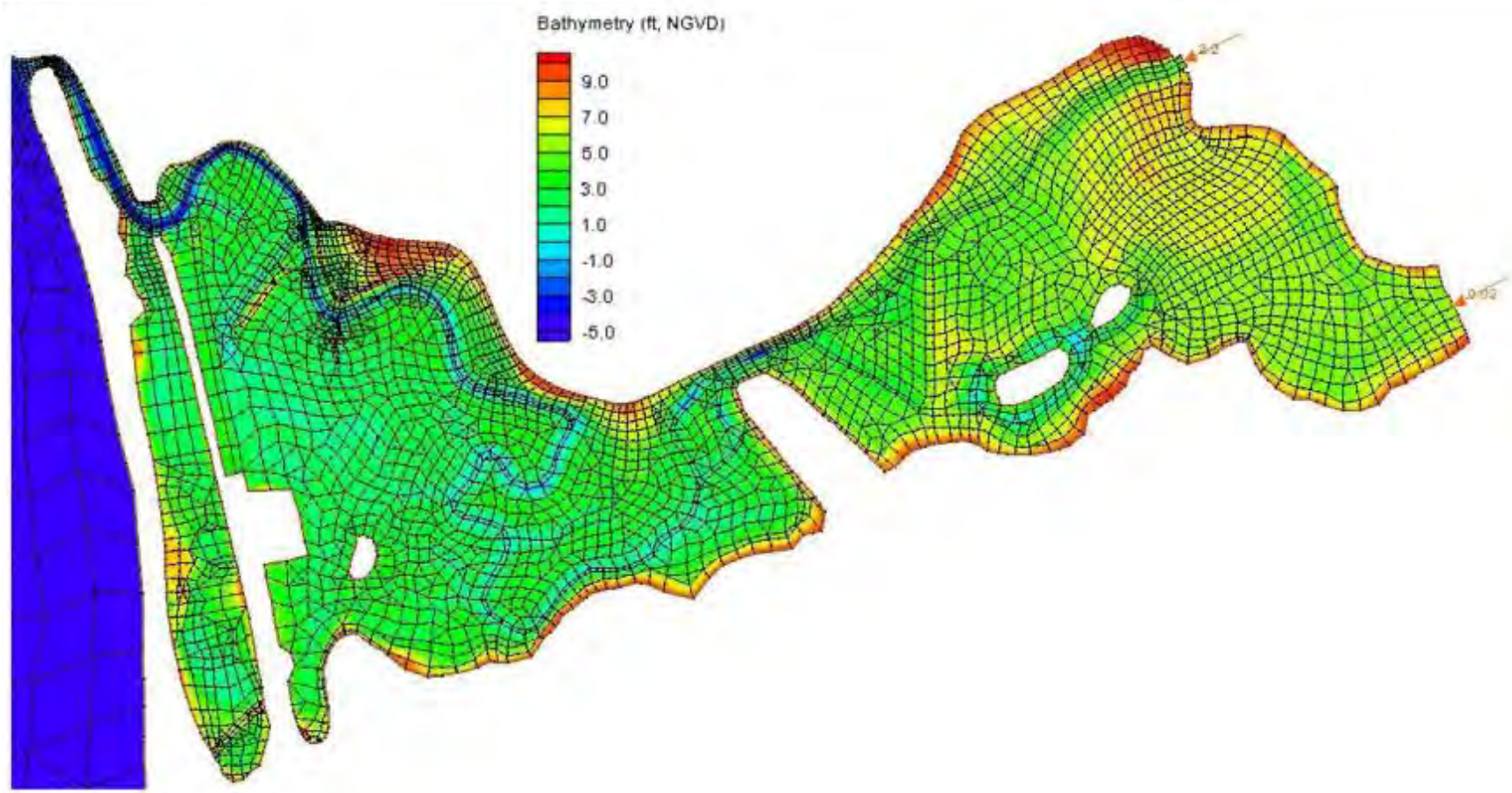


Figure 4-4: RMA-2 Model Mesh for Existing Conditions (No Project)

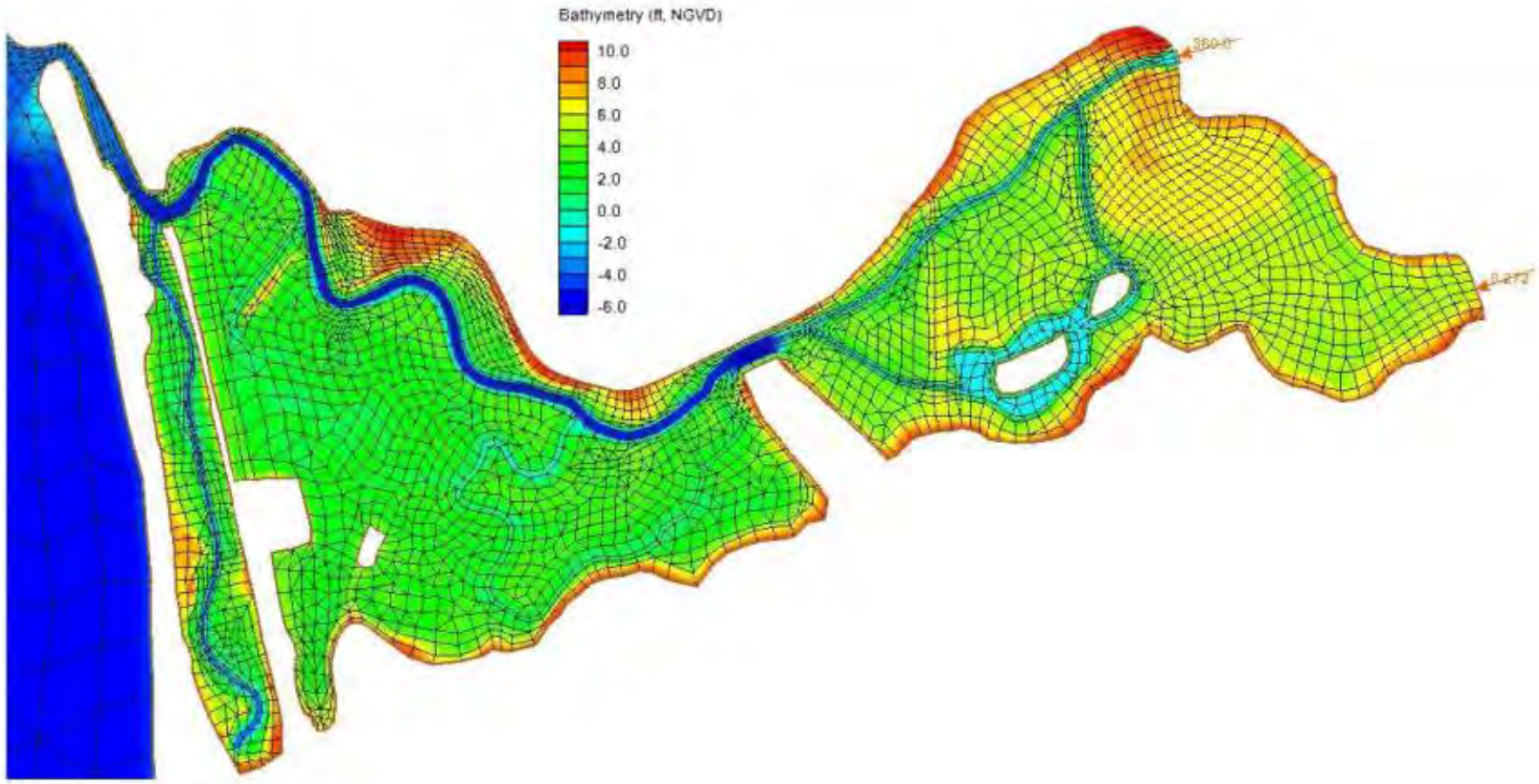


Figure 4-5: RMA-2 Modeling Mesh for Alternative 1A

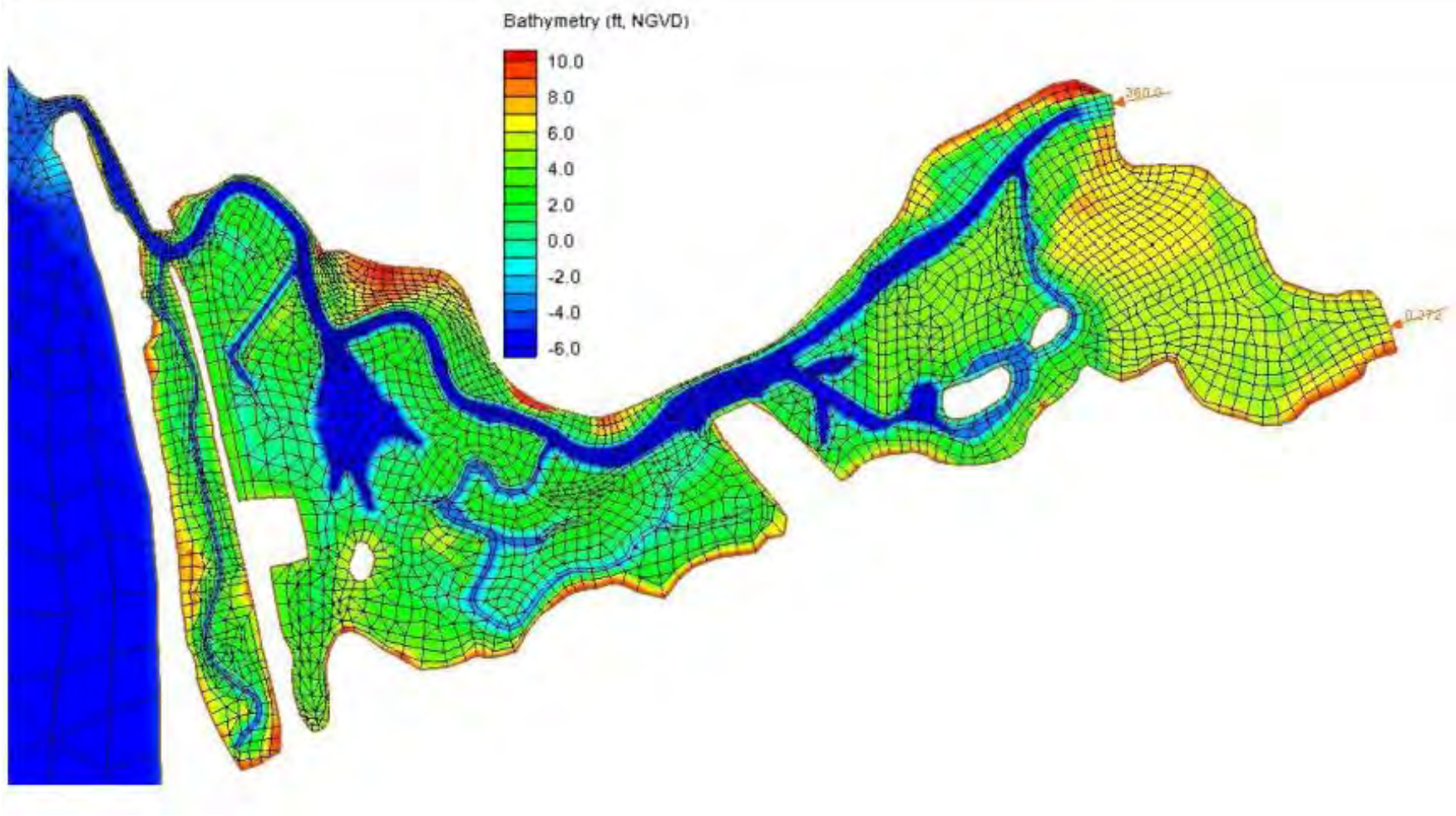


Figure 4-6: RMA-2 Modeling Mesh for Alternative 1B

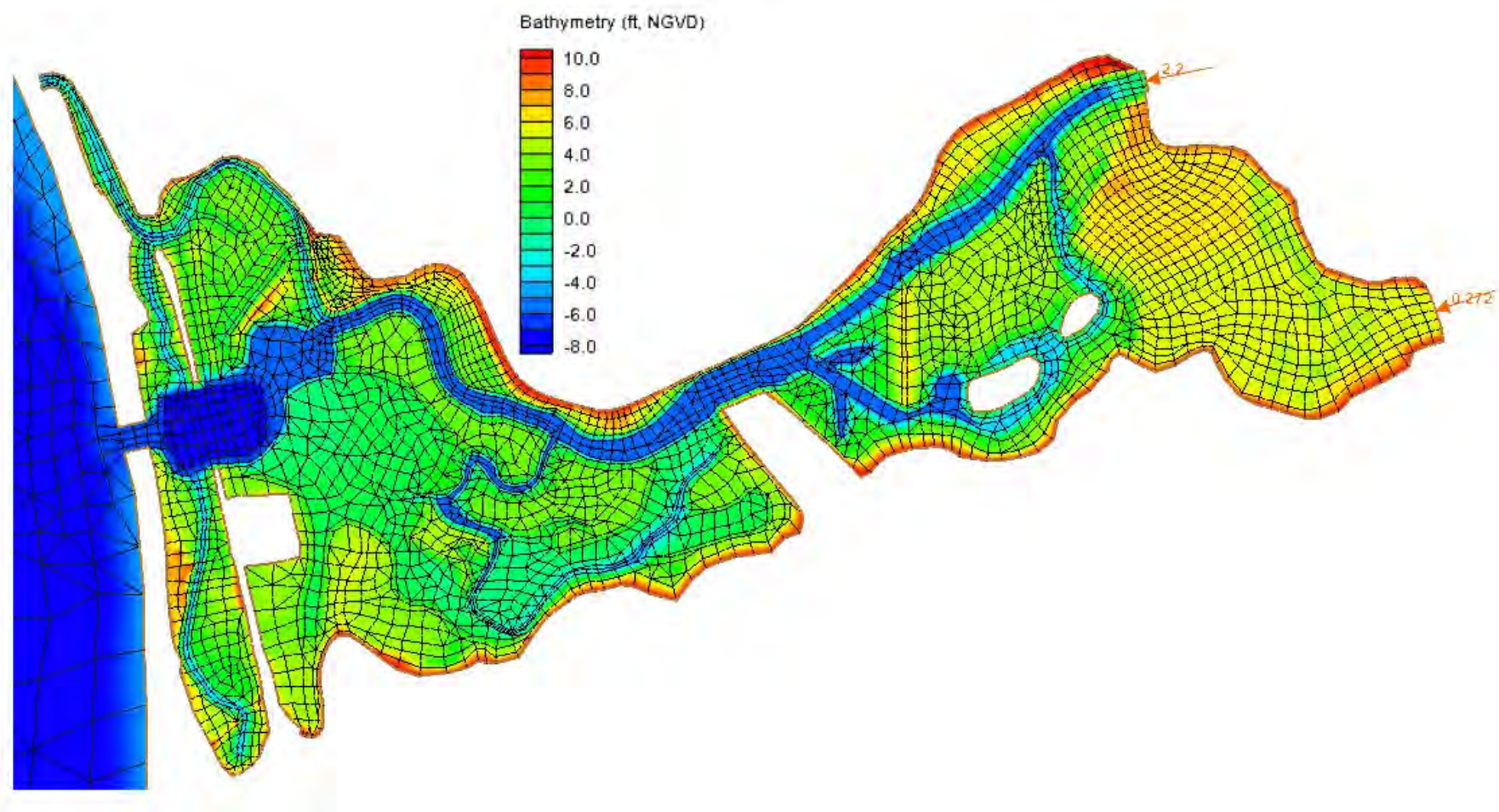


Figure 4-7: RMA-2 Modeling Mesh for Alternative 2A

The entire modeling area, approximately 2.54 square miles, is represented as a finite element mesh consisting of elements and nodes detailed in Table 4-1.

Table 4-1: Model Mesh Elements and Nodes

Alternative	Number of Elements	Number of Nodes
No Project	3,790	11,114
Alternative 1A	4,654	13,280
Alternative 1B	4,663	13,017
Alternative 2A	4,339	12,379

4.5 Boundary Conditions

4.5.1 Tides

Since there are no tide stations at San Elijo Lagoon, the nearest La Jolla gage (National Oceanic and Atmospheric Administration Station ID: 9410230) was used to represent the ocean tide at the project site, as shown in Table 4-2. The diurnal tide range is approximately 5.33 feet Mean Lower Low Water (MLLW) to Mean Higher High Water (MHHW), and Mean Sea Level (MSL) is at +2.73 feet MLLW. Water level data records provide astronomical tides and other components including barometric pressure tide, wind setup, seiche, and the El Nino Southern Oscillation. Tidal variations can be resolved into a number of sinusoidal components having discrete periods. The longest significant periods, called tidal epochs, are approximately 19 years. In addition, seasonal variations in MSL can reach amplitudes of 0.5 feet in some areas. Superimposed on this cycle is a 4.4-year variation in the MSL that may increase the amplitude by as much as 0.25 feet. Water level gage records are typically analyzed over a tidal epoch to account for these variations and to obtain statistical water level information (e.g., MLLW and MHHW).

Table 4-2: Recorded Water Levels at La Jolla (1983-2001 Tidal Epoch)

Description	Elevation (feet, MLLW)	Elevation (feet, NGVD)
Extreme High Water (11/13/1997)	7.65	5.35
Mean Higher High Water (MHHW)	5.33	3.03
Mean High Water (MHW)	4.60	2.30
Mean Tidal Level (MTL)	2.75	0.46
Mean Sea Level (MSL)	2.73	0.44
National Geodetic Vertical Datum 1929 (NGVD)	2.30	0.00
Mean Low Water (MLW)	0.91	-1.39
North America Vertical Datum 1988 (NAVD)	0.19	-2.11
Mean Lower Low Water (MLLW)	0.00	-2.30
Extreme Low Water (12/17/33)	-2.87	-5.16

4.5.2 Modeling Tidal Series

The tide series used for modeling was a representative period from November 7-21, 2008. Modeling long-term hydrologic conditions is typically done using a synthetic (artificially-created) tide series that represents average spring tide conditions over the most recent 19-year tidal epoch, referred to as a Tidal Epoch Analysis (TEA) tide series. The benefit of using a statistical tide is that the long-term condition can be modeled over a shorter time period with less computation time.

The most recent previous modeling of this site was done by the USACE without the benefit of preparing a Tidal Epoch Analysis (TEA) tide, and significant effort (beyond the scope of this study) is required to prepare a new TEA tide for this site. Therefore, a real tide series was used that matched average spring tide data available from National Oceanic and Atmospheric Administration (NOAA) (2009).

Not using a statistical TEA tide for modeling is not a serious information gap. To address this potential shortcoming, the modeler evaluated existing tide data from NOAA for San Diego at Scripp's Pier (NOAA 2009). NOAA began publishing spring high and spring low tidal elevations of all tidal cycles in January 2008. The modeler averaged the spring high and spring low tidal elevations of all tidal cycles from January 2008 through October 2009 (22 months), then examined the existing data to identify a real two-week tidal cycle that matched them. Tides during the period of November 7 through November 21, 2008 reached nearly the exact same spring high and spring low tidal elevations of NOAA's longer 22-month record. Also, the average tidal elevation of that November 7 through November 21, 2008 period compared with the average tidal elevation of the 19-year tidal epoch and was within 0.01 foot. Therefore, the modeler concluded that tides during the period of November 7 through November 21, 2008 sufficiently matched long-term tides at the site and use of this record poses no implications on habitat designs and analyses. The modeling tide includes both spring and neap tidal ranges as shown in Figure 4-8.

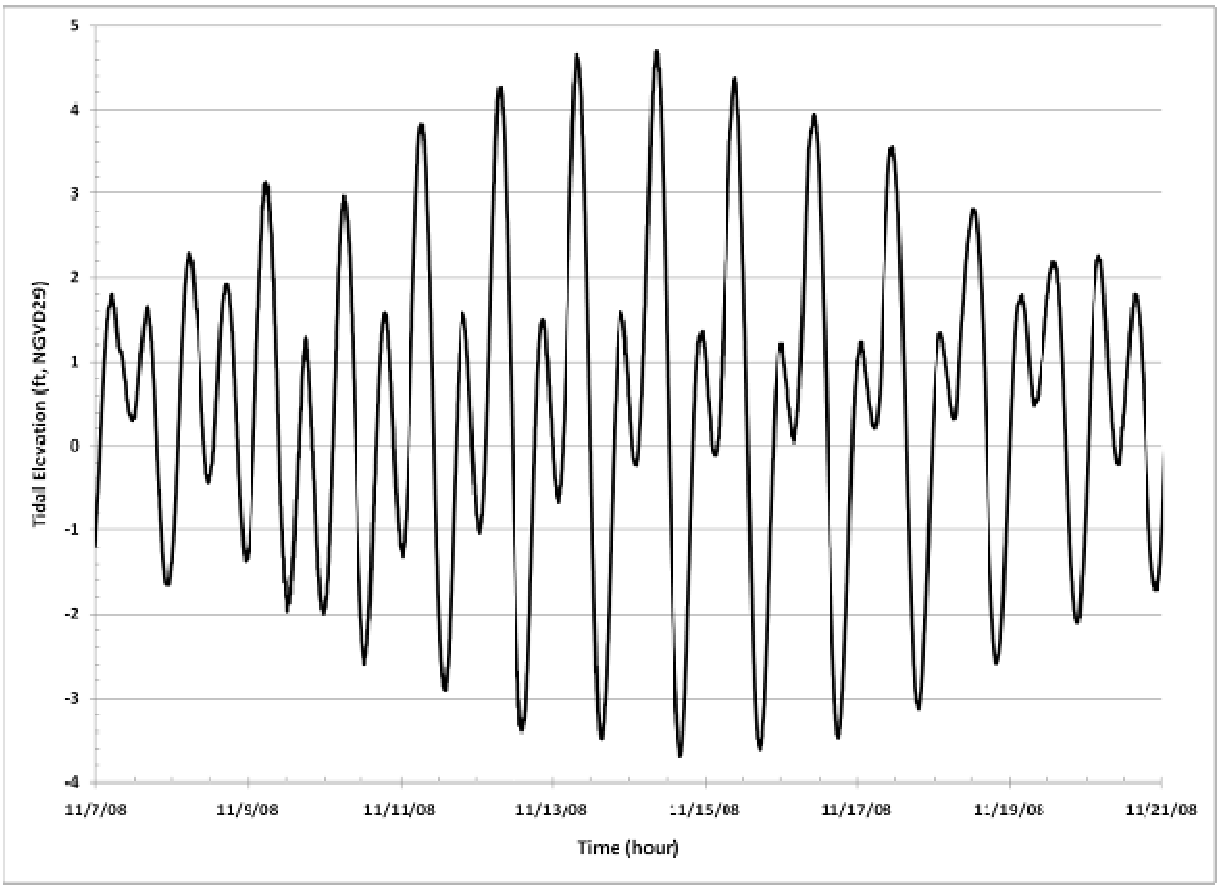


Figure 4-8: Modeling Tidal Series

4.5.2.1 Extreme Tidal Series

The estimated extreme water level in the wetlands is an important design parameter for determining the potential for flooding. Extreme high tides are superimposed on 100-year storm floods to predict water levels throughout the lagoon during an extreme flood event. The spring tidal series was shifted upward to match the FEMA base flood elevation of +7 feet NGVD (FEMA FIRM Panel No. 06073C1044F) along the coast seaward of San Elijo Lagoon. The base flood elevation is the floodwater level anticipated to be reached during the base flood, which has a one percent chance of being equaled or exceeded in any given year (100-year event). Base Flood Elevations are shown on Flood Insurance Rate Maps (FIRMs) and on the flood profiles. The resulting tidal series used for these analyses is shown in Figure 4-9.

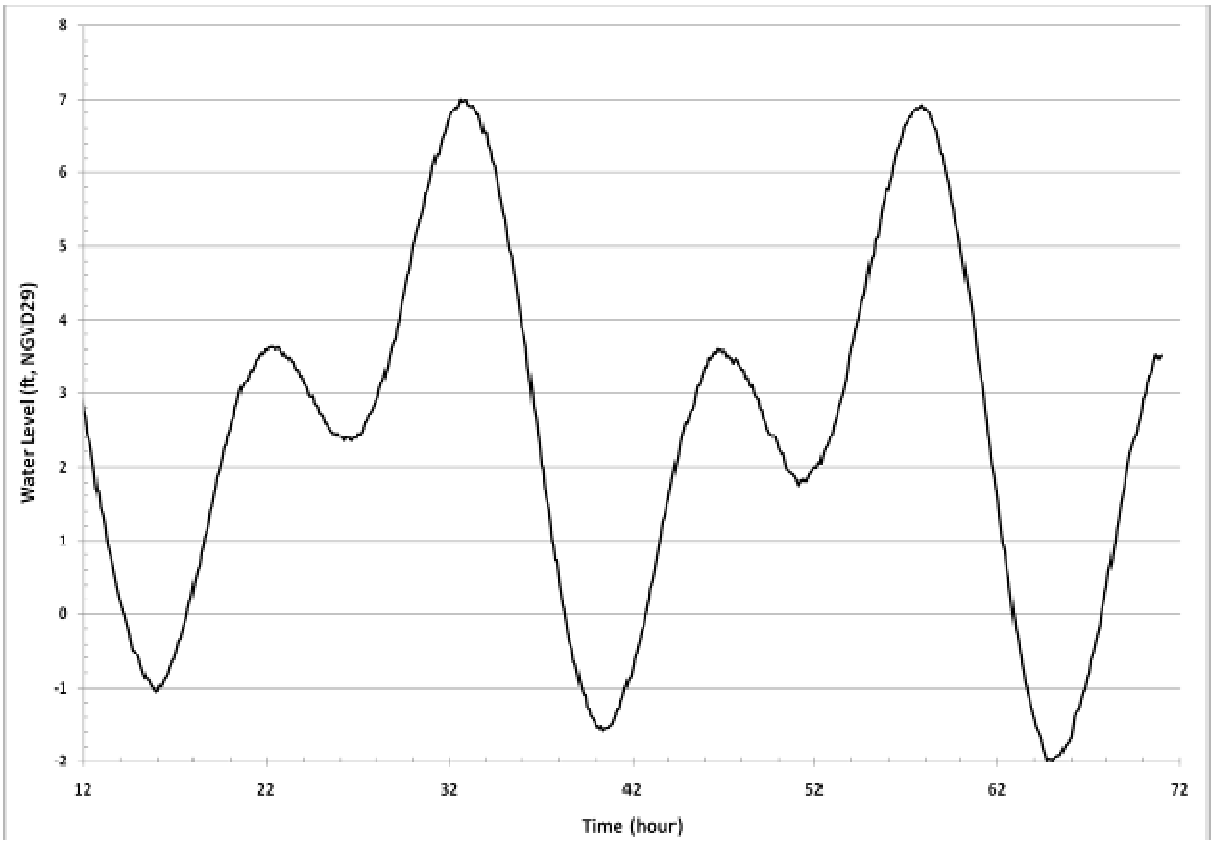


Figure 4-9: Extreme Tidal Series

4.5.2.2 Flood Flows from Creeks

San Elijo Lagoon is the estuary of both Escondido and La Orilla Creeks. The Escondido Creek watershed extends approximately 28 miles from its headwaters in Bear Valley to the San Elijo Lagoon before discharging into the Pacific Ocean. The watershed covers approximately 54,112 acres in area and is long and narrow. La Orilla Creek is a very short stream that has only a marginal contribution of flood and sediment discharges compared to Escondido Creek. In the past, these creeks were considered to be ephemeral, but in the last few decades low flows from urbanization are present all year long.

The SELC installed and has managed a network of stream gauges in the Carlsbad Hydrology Unit since 2004, which is partially supported by the San Diego Regional Water Quality Control Board (SDRWQCB) and the California State Coastal Conservancy (CSCC). The gage on Escondido Creek is located at Camino del Norte Bridge. However, the period of the recorded flows is insufficient for statistically generating 50-year and 100-year return period flood flows for storm flow modeling.

A statistical analysis was performed by Exponent Inc. (2000) to determine 50-year and 100-year flood flows based on data from a neighboring stream gage on Las Flores Creek near Oceanside Harbor, as the stream gage record for Escondido Creek is too short to generate statistics and

no stream gage exists on La Orilla Creek. The 100-year peak flood was determined to be 21,000 cubic feet per second (cfs) for Escondido Creek, which is the same as the flood flow rate used by FEMA for the National Flood Insurance Program. The watershed area of La Orilla Creek is about 10 percent of that for Escondido Creek; therefore the combined peak flow from both creeks is estimated to be 23,255 cfs. This value was used by Dokken Engineering (2007) in their location hydraulic study for the I-5 Bridge. A daily hydrograph was developed by Exponent (2000) and was raised to the peak flow rate of 23,255 cfs to represent storm flood flows into the Lagoon. The flood hydrographs modeled for the 50-year and 100-year events are shown in Figure 4-10.

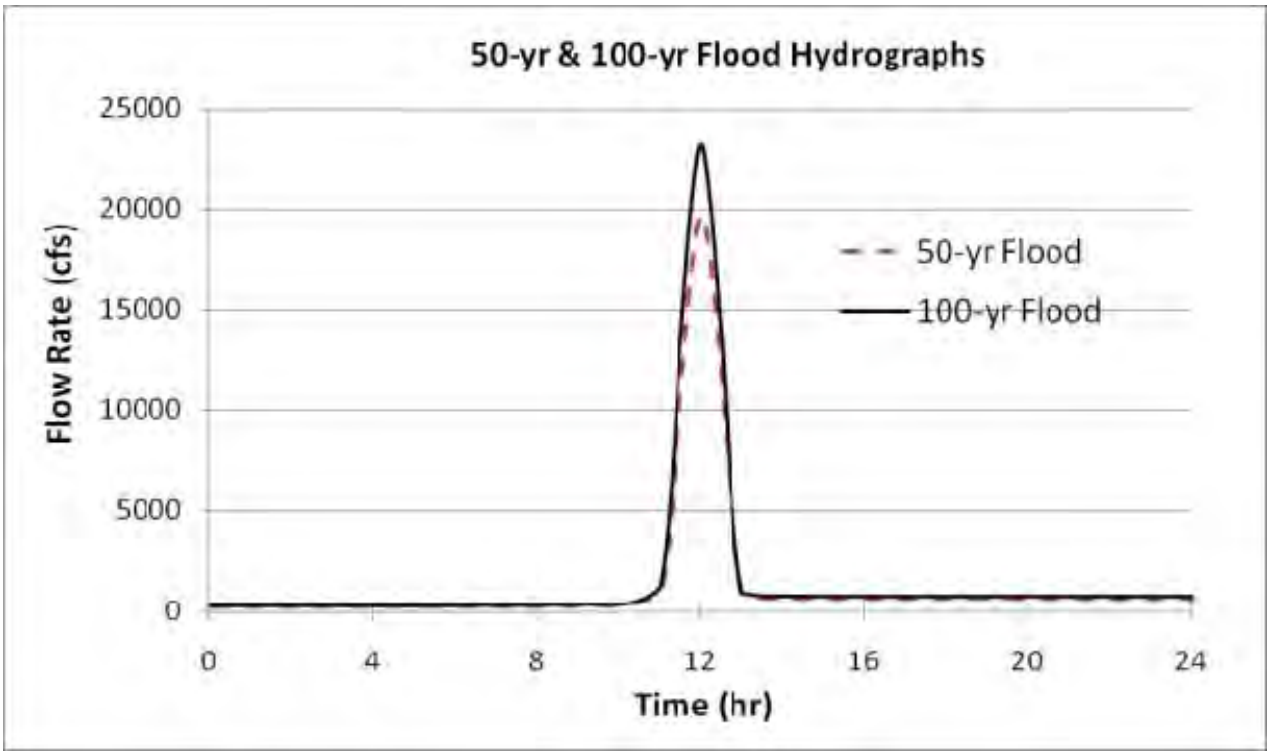


Figure 4-10: San Elijo Lagoon Flood Hydrographs

5.0 DESCRIPTION OF EXISTING BRIDGE CHANNEL DIMENSIONS

The existing channel widths below each bridge crossing provided a starting point for the optimization modeling of each alternative. The following section provides a brief description of each bridge and the range of channel dimensions evaluated in the optimization study.

5.1 Hwy 101 Bridge

The Hwy 101 Bridge, shown in Figure 5-1, crosses over the existing inlet of the lagoon. The existing inlet is unstable and subject to siltation and possibly closure if not dredged on a regular basis. Although the current active inlet channel is approximately 70 feet as-built drawings and field measurements indicate the maximum opening of the Hwy 101, from abutment to abutment, is approximately180 feet. The lowest possible invert at the current inlet is about -4 feet NGVD due to hard bottom reef and bedrock. Accounting for sideslopes and pier width, the maximum effective width of Hwy 101 is about 160 feet at an invert elevation of -4 feet NGVD. The minimum bridge soffit elevation, indicated on the as-built drawings, is +10 feet NGVD.

The optimization modeling for Hwy 101 focused on inlet widths between 70 feet and 160 feet for alternatives utilizing the existing inlet. A few models were run with widths of 180 feet and 200 feet, but there was little benefit to increasing the channel width beyond 160 feet. The long and narrow inlet channel, between Hwy 101 and the Railroad, seems to be the limiting feature in terms of tidal exchange and flood conveyance. Alternative 2A proposes a new inlet located south of the existing inlet. An inlet stability study performed for the SELRP had found that an inlet width of 200 feet would likely be sustainable for the tidal prism of Alternative 2A (M&N 2011 Draft). The optimization of the new inlet channel and the Hwy 101 Bridge lengths focused on channel widths between 160 feet and 240 feet.

Figure 5-1: Hwy 101 Bridge (Existing Lagoon Inlet)

5.2 Railroad Bridge

The Railroad, shown in Figure 5-2, runs just east of and parallel to Hwy 101 across the San Elijo Lagoon. Survey data of the existing railroad bridge indicate a channel width of approximately 250 feet from abutment to abutment at an elevation of +5 feet, NGVD. Assuming 3:1 (H:V) sideslopes, the maximum bottom width of the existing channel under the bridge is approximately 187 feet at the dredge depth. The bridge is supported by 23 piers spaced at approximately 14 feet on center. The piers consist of round piles about 16 inches in diameter. Subtracting for pier widths, the effective channel width at an elevation of -5.5 feet, NGVD is approximately 161 feet. The minimum bridge soffit elevation, according to a 2007 PDC survey provided by HDR, is about +15.6 feet NGVD.

This width is significantly larger than the tidal inlet channel and most of the main channel east of the railroad bridge. For this reason the railroad bridge is not expected to restrict tidal exchange or flood conveyance. Despite this expectation, tidal and flood optimization models for No Project, Alternatives 1A and 1B were set up to evaluate whether increasing the railroad bridge length to 210 feet and 230 feet would increase tidal range or flood conveyance. The results confirmed expectations and showed there is no benefit to increasing the channel width below the existing railroad bridge.

Alternative 2A proposes a new railroad bridge over a wide subtidal basin. The channel width under the bridge would be approximately 590 feet wide and significantly wider than the inlet channel and main channel throughout the lagoon. Model simulations were run for a slightly wider channel below the railroad bridge and confirmed that this width does not restrict tidal exchange or flood conveyance.

Figure 5-2: Railroad Bridge

5.3 Interstate 5 Freeway Bridge

The I-5 freeway runs north to south across the San Elijo Lagoon. The I-5 Bridge crosses near the middle of the lagoon serving as the boundary between the Central and East basins of the lagoon. The I-5 Bridge also spans Manchester Avenue, as shown in Figure 5-3. The as-built plans and survey data indicate the existing channel width below the Bridge, from abutment to abutment, is approximately 155 feet wide at an elevation of +5 feet, NGVD. Assuming 2:1 sideslopes, the existing channel bottom width in a dredged condition would be approximately 130 feet. The effective channel width modeled in RMA-2 further reduced the channel width to account for the 4-foot diameter piers supporting the bridge. The minimum bridge soffit elevation, indicated on the as-built drawings, is +31.5 feet NGVD.

The I-5 Bridge optimization modeling focused on a range of channel widths from between 130 feet and 400 feet for most alternatives. Some alternatives were modeled with an I-5 channel as wide as 800 feet. Based on findings from the SELRP feasibility studies (M&N 2010[a,b]), the minimum feasible width for the channel under the I-5 Bridge under Alternative 2A conditions was 261 feet. This was the minimum width modeled for tidal and flood optimization of Alternative 2A.

Figure 5-3: Interstate-5 Bridge

6.0 ANALYSES TO ACHIEVE OPTIMAL TIDAL RANGE

The selection of optimum channel dimensions under bridges was based on a sensitivity analysis conducted for each bridge crossing under typical dry weather tidal fluctuations. A 3-day spring tide series was used for the ocean boundary condition. The dry weather flow from the Escondido and La Orilla Creeks was modeled as 2.2 cfs and 0.2 cfs respectively. Tidal range was used as the primary indicator for benefits to the wetland ecosystem. A larger tidal range provides for greater wetland area, more habitat diversity, improved tidal exchange, and better water quality. The optimum channel dimensions were identified as the point at which tidal range was most favorable and further increases in channel width result in only minimal benefit. Representative tide gage locations are shown in Figure 6-1 and Figure 6-2.

This section focuses on the optimization of channel dimensions under bridges to improve the tidal range of each SELRP alternative. The various optimization models focused on widening and dredging the channels below the Hwy 101 and I-5 Bridges to improve tidal exchange and flood conveyance. Modeling results for both the existing Railroad Bridge (No Project, Alternatives 1A and 1B) and the proposed Railroad Bridge (Alternative 2A) suggest they have sufficient channel widths and do not restrict tidal range or flood conveyance.

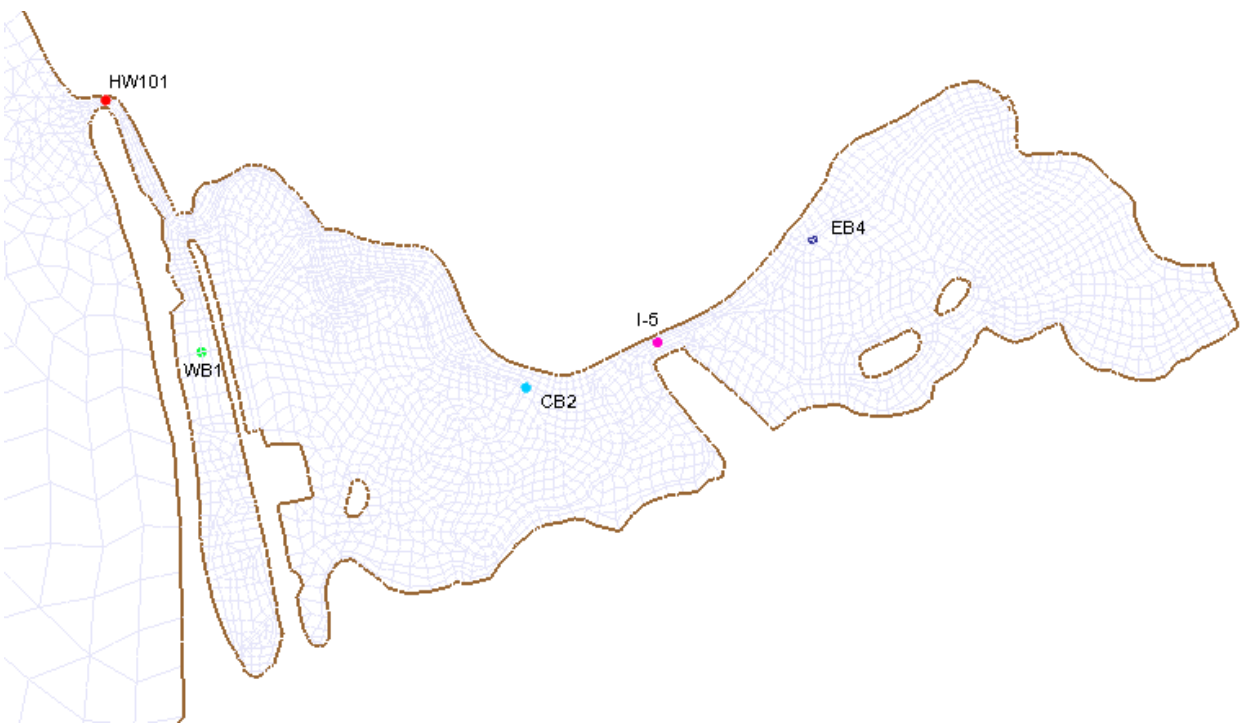


Figure 6-1: Tide Gage Locations (No Project, Alternatives 1A & 1B)

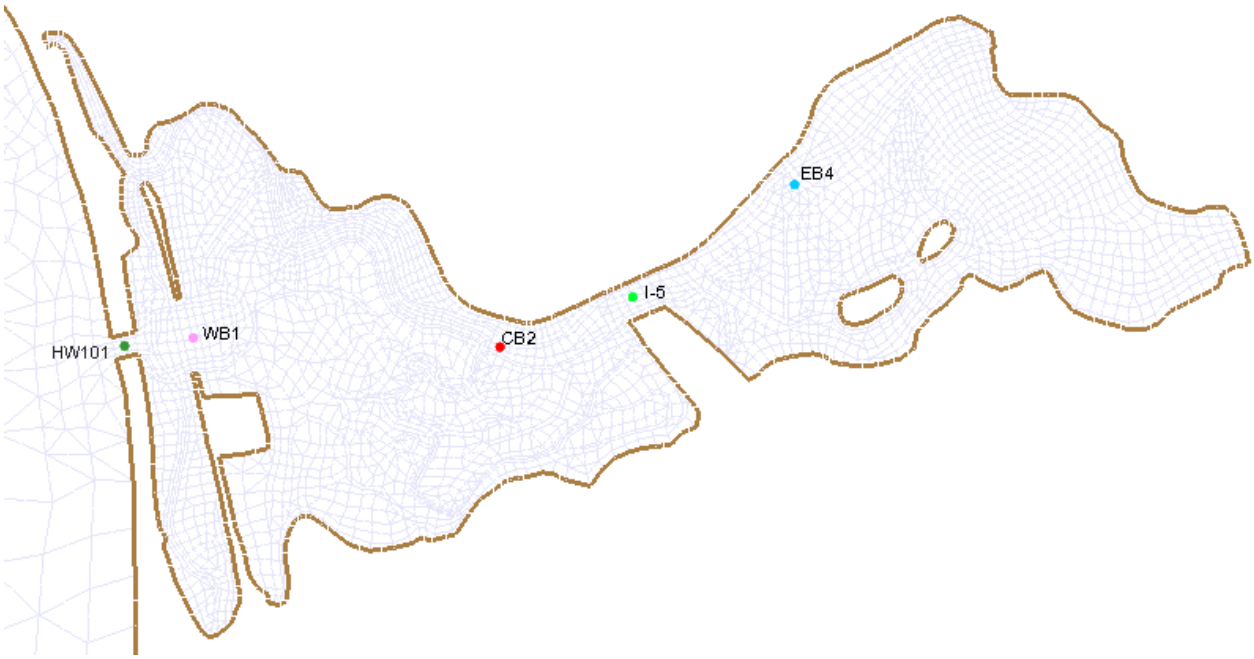


Figure 6-2: Tide Gage Locations (Alternative 2A)

6.1 No Project Tidal Range Optimization

The list of optimization models developed and used to generate results for the No Project alternative are presented in Table 6-1. Channel widths shown in the Table are the total bottom width, not excluding bridge piles (e.g. LOSSAN RR = 187 feet). Modeling was done, however, for the channel width minus the space occupied by bridge piles (e.g. LOSSAN RR = 161 feet). This is was also done for project alternatives, so subsequent Tables of channel widths show the same type of information. Under No Project conditions, the results for tide optimization suggest there is no significant benefit achieved by increasing the channel width beyond its current dimensions. The existing bathymetry of the lagoon, not the bridge dimensions, is the limiting factor for tidal exchange. In addition to the narrow inlet channel, the main channel through the Central Basin is also narrow, shallow, and sinuous in planform resulting in substantial energy losses during normal tidal fluctuations and extreme storm flood events.

Table 6-1: No Project Tide Optimization Models

San Elijo Lagoon Bridge Optimization Modeling							
No Project - Tide Modeling							
		Bridge Modeling Parameters					
		Hwy 101		LOSSAN RR		I - 5	
Model No.	File Name	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)
1	NP_Hwy101_1_new	-0.87	70	Varies	187	0.74	130
2	NP_Hwy101_5_new	-4	90	-5.5	187	-6	261
3	NP_Hwy101_2_new	-4	105	-5.5	187	-6	261
4	NP_Hwy101_3_new	-4	130	-5.5	187	-6	261
5	NP_Hwy101_4_new	-4	160	-5.5	187	-6	261
6	NP_I5_1_new	-4	160	-5.5	187	-6	130
7	NP_I5_2_new	-4	160	-5.5	187	-6	196
8	NP_I5_3_new	-4	160	-5.5	187	-6	261
9	NP_I5_4_new	-4	105	-5.5	187	-6	130

The tidal range optimization results for Hwy 101 are presented in Figure 6-3 and Table 6-2. The results indicate there is some benefit to a wider Hwy 101 channel but this benefit is relatively small and the tidal prism of this existing lagoon cannot support a wider inlet channel. The highlighted rows in each table represent the predicted tidal range for the optimized channel width. In some cases the optimized channel width does not result in the maximum tidal range. The optimized width was identified as the point at which further increases to channel width provide only minimal benefit.

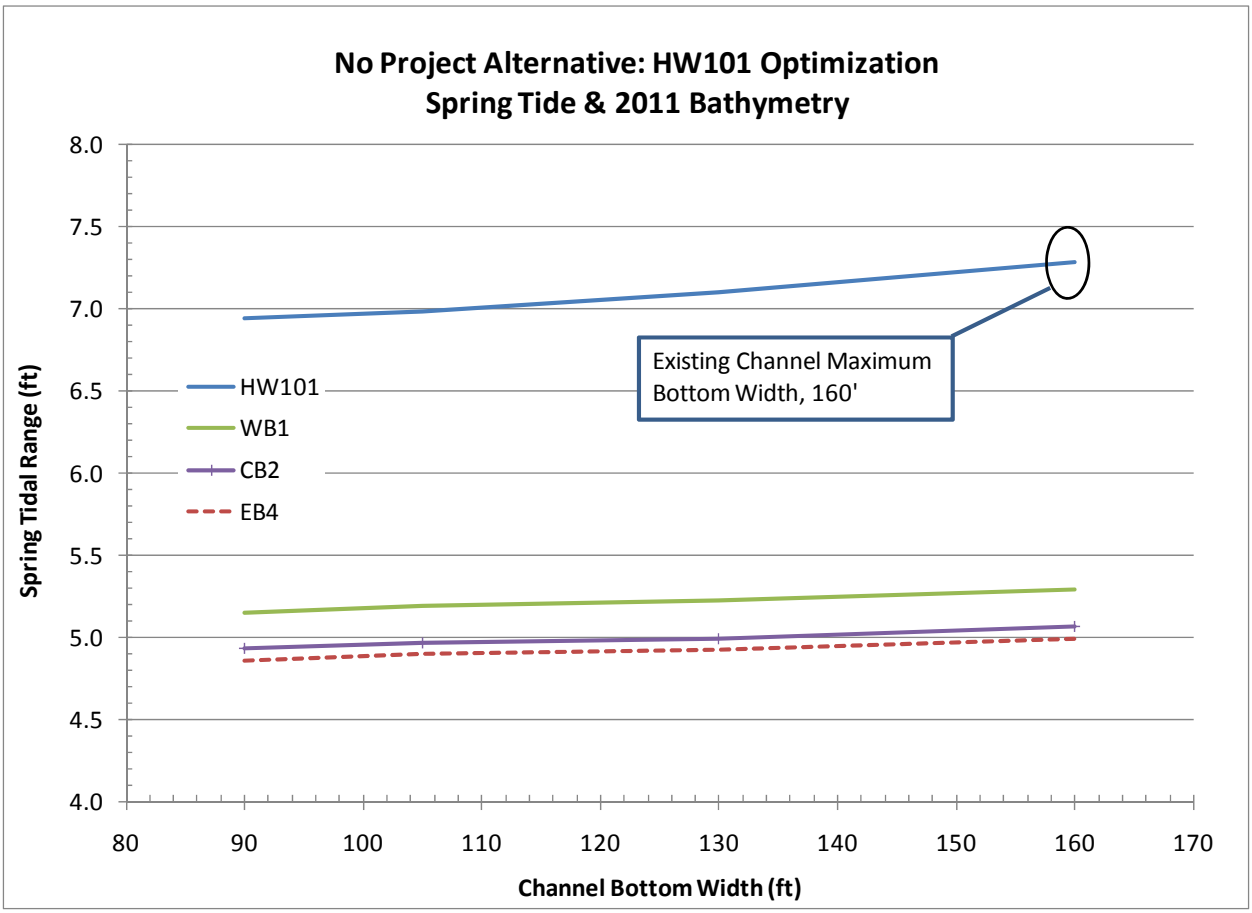


Figure 6-3: No Project Hwy 101 (Tidal Range vs Channel Width)

Table 6-2: No Project Hwy 101 Tidal Range Results

Channel Dimensions		Tidal Range (ft)					
Width (ft)	Invert (NGVD)	Ocean	HW101	WB1	CB2	I-5	EB4
70	-0.87	8.37	4.08	3.58	3.45	3.37	3.37
90	-4	8.37	6.94	5.15	4.93	4.89	4.86
105	-4	8.37	6.99	5.19	4.97	4.92	4.90
130	-4	8.37	7.10	5.22	5.00	4.95	4.92
160	-4	8.37	7.29	5.30	5.06	5.02	4.99

The tidal range optimization results for the I-5 Bridge are presented in Figure 6-4 and Table 6-3**.** The results indicate the width of the I-5 Bridge has no influence on tidal ranges under the No Project conditions. This is due to minimal tidal exchange through the I-5 Bridge caused by limitations of the existing lagoon bathymetry.

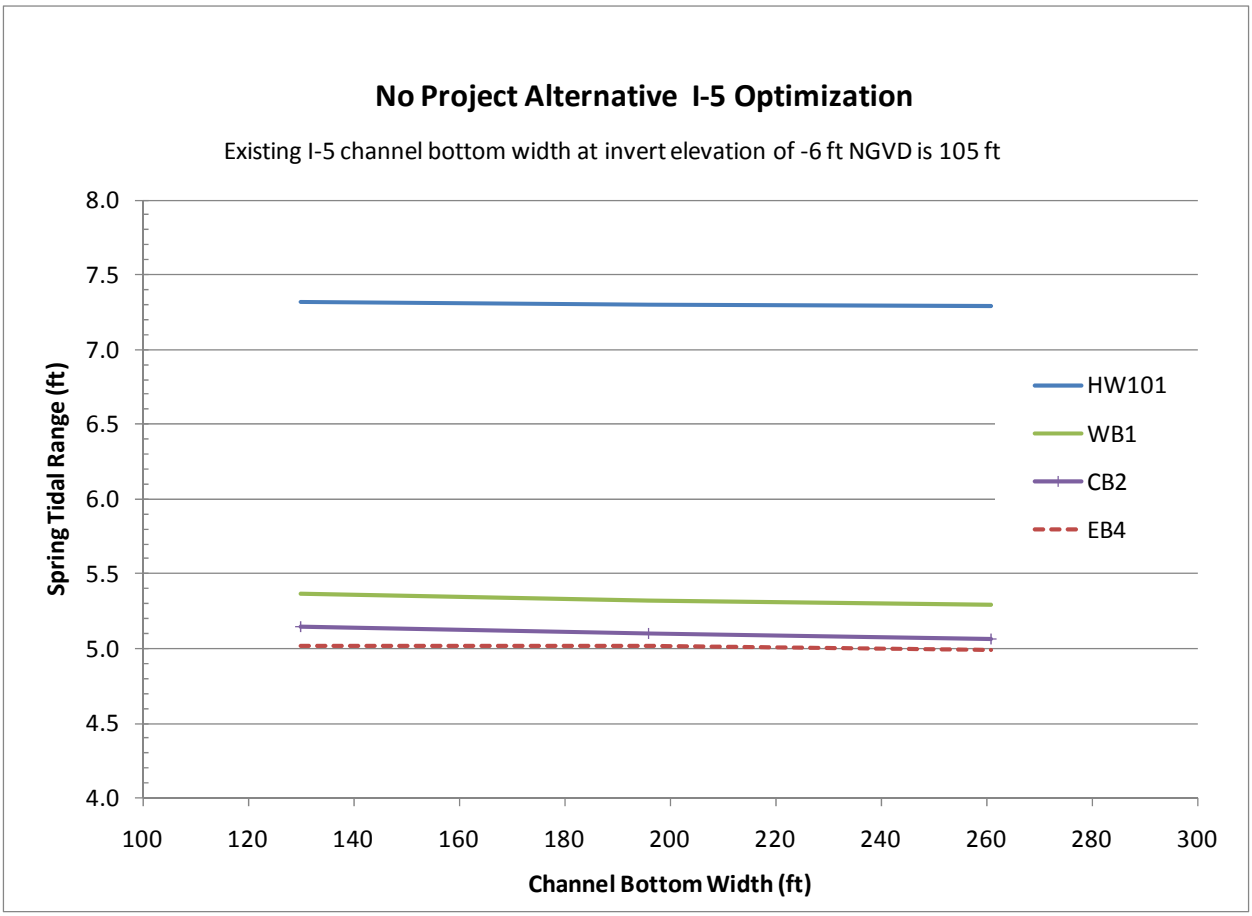


Figure 6-4: No Project I-5 Bridge (Tidal Range vs Channel Width)

Table 6-3: No Project I-5 Tidal Range Results

Channel Dimensions (ft)		Spring Tidal Range (ft)					
Width	Invert (NGVD)	Ocean	HW101	WB1	CB2	I-5	EB4
130	-6	8.37	7.31	5.36	5.14	5.06	5.01
196	-6	8.37	7.30	5.32	5.10	5.05	5.02
261	-6	8.37	7.29	5.30	5.06	5.02	4.99

6.2 Alternative 1A Tidal Range Optimization

The list of optimization models developed and used to generate results for Alternative 1A are presented in Table 6-4. In comparison to the tide range for the No Project alternative, there is an overall increase in tidal range of about 0.3 feet throughout the lagoon for Alternative 1A due to expanding channels on the site without significantly increasing the tidal prism, thereby improving hydraulic efficiency of the system. This will promote better opportunities for wetland habitat and the increase in tidal prism will provide some water quality and inlet stability benefits.

Table 6-4: Alternative 1A Tide Optimization Models

San Elijo Lagoon Bridge Optimization Modeling							
Alt 1A - Tide Modeling							
		Bridge Modeling Parameters					
		Hwy 101		LOSSAN RR		I - 5	
Run No.	File Name	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)
1	Alt1A_HW101_1_new	-4	105	-5.5	187	-6	261
2	Alt1A_HW101_2_new	-4	130	-5.5	187	-6	261
3	Alt1A_HW101_3_new	-4	160	-5.5	187	-6	261
4	Alt1A_HW101_4_new	-4	105	-5.5	187	-6	130
9	Alt1A_HW101_5_new	-4	115	-5.5	187	-6	261
5	Alt1A_I5_1_new	-4	160	-5.5	187	-6	261
6	Alt1A_I5_2_new	-4	160	-5.5	187	-6	196
7	Alt1A_I5_3_new	-4	160	-5.5	187	-6	130
8	Alt1A_I5_4_new	-4	115	-5.5	187	-6	130

The Hwy 101 tidal range optimization results for Alternative 1A are presented in Figure 6-5 and Table 6-5. Similar to the results for the No Project alternative, there was minimal improvement to tidal range by increasing the channel width below Hwy 101. Due to a slightly larger tidal prism, it may be possible to sustain a wider inlet channel and, therefore, 115 feet was identified as the optimum width for this alternative.

The I-5 tidal range optimization results for Alternative 1A are presented in Figure 6-6 and Table 6-6. Tidal range through the I-5 was not sensitive to changes in the channel width and, therefore, the existing width was identified as the optimum dimension. Highlighted rows in each table represent the predicted tidal range for the optimized channel width. In some cases the optimized channel width does not result in the maximum tidal range. The optimized width was identified as the point at which further increases to channel width provide only minimal benefit.

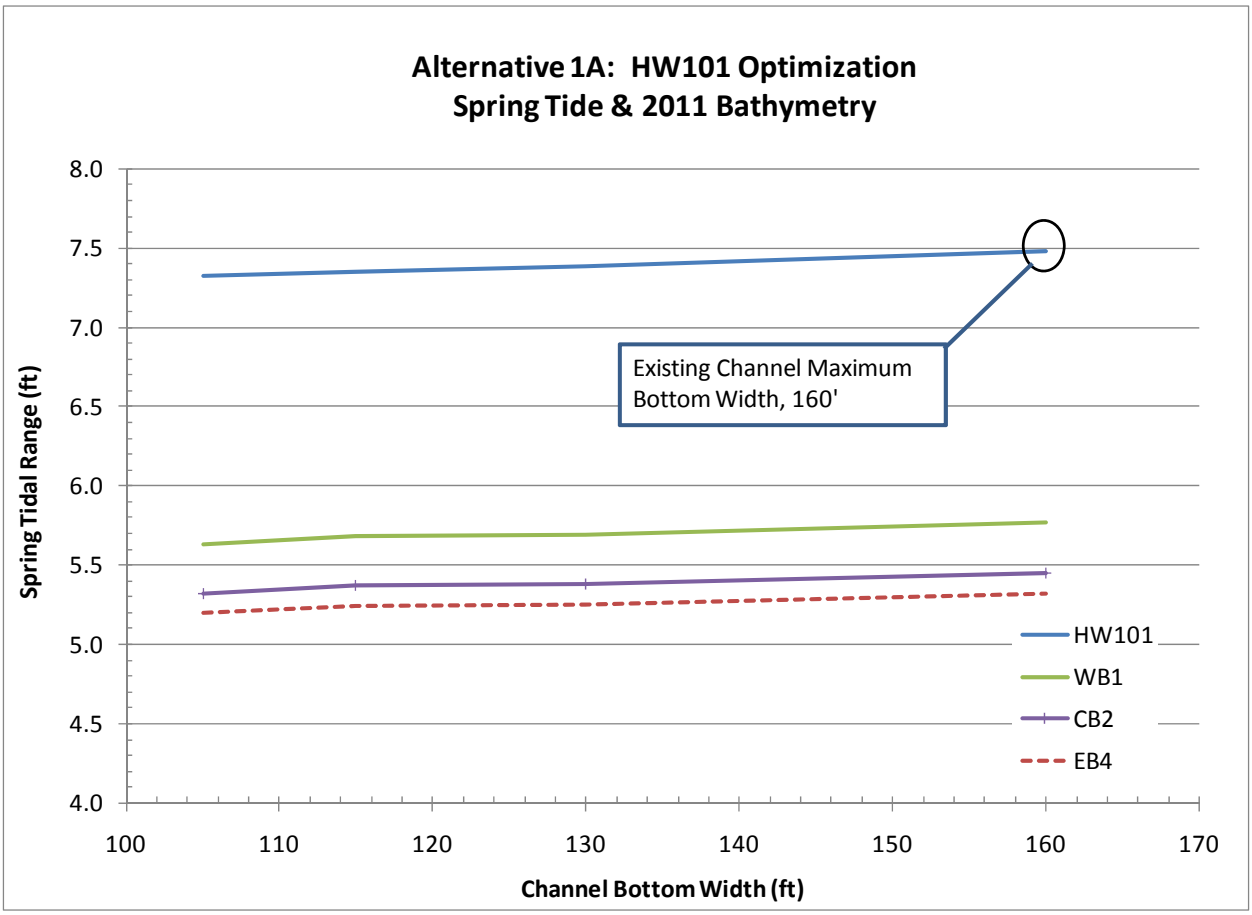


Figure 6-5: Alternative 1A Hwy 101 (Tidal Range vs Channel Width)

Table 6-5: Alternative 1A Hwy101 Tidal Range Results

Channel Dimensions (ft)		Tidal Range (ft)					
Width	Invert (NGVD)	Ocean	HW101	WB1	CB2	I-5	EB4
105	-4	8.37	7.32	5.63	5.32	5.27	5.20
115	-4	8.37	7.35	5.68	5.37	5.31	5.24
130	-4	8.37	7.39	5.70	5.38	5.32	5.25
160	-4	8.37	7.48	5.77	5.45	5.39	5.32

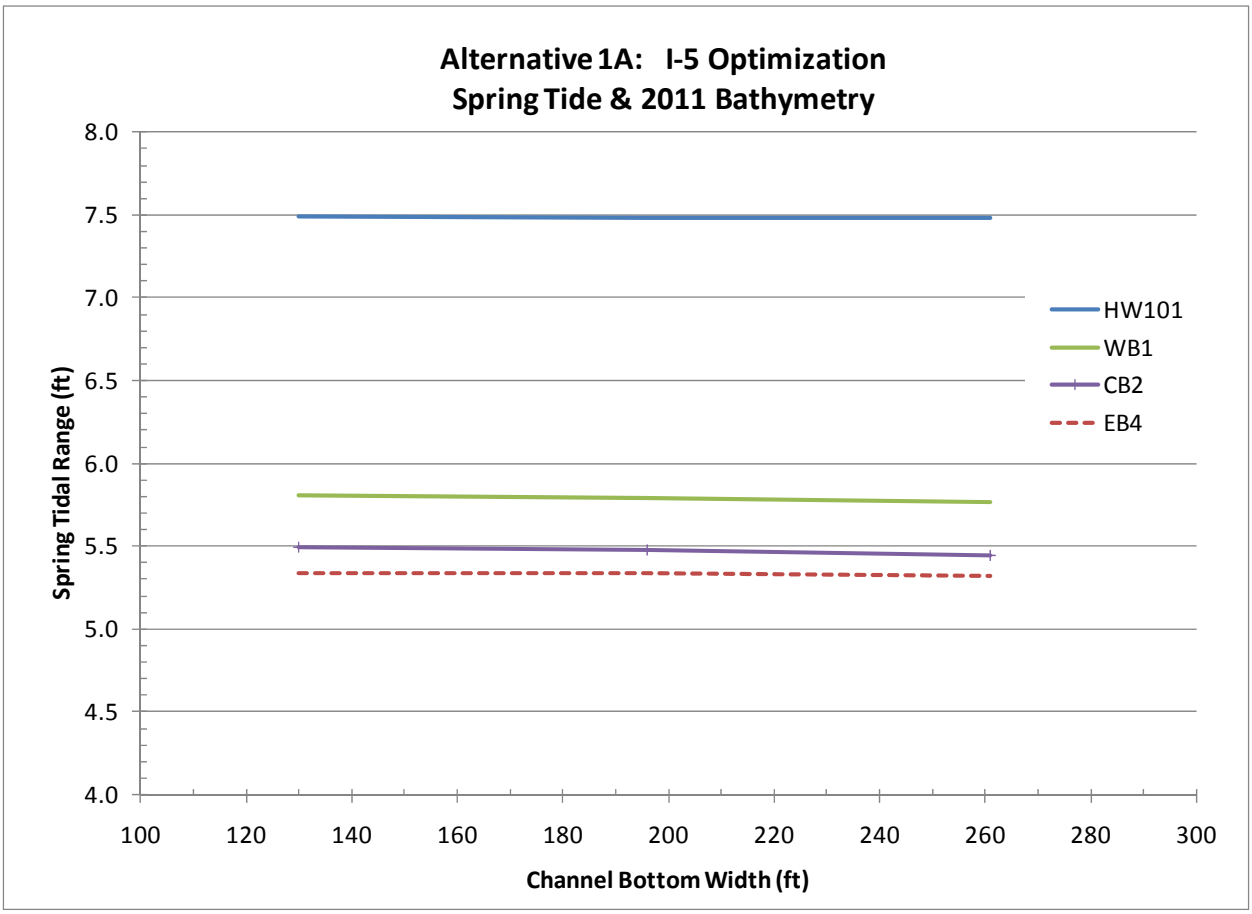


Figure 6-6: Alternative 1A I-5 Bridge (Tidal Range vs Channel Width)

Table 6-6: Alternative 1A I-5 Tidal Range Results

Channel Dimensions (ft)		Tidal Range (ft)					
Width	Invert (NGVD)	Ocean	HW101	WB1	CB2	I-5	EB4
130	-6	8.37	7.49	5.81	5.49	5.43	5.34
196	-6	8.37	7.49	5.79	5.47	5.42	5.34
261	-6	8.37	7.48	5.77	5.45	5.39	5.32

6.3 Alternative 1B Tidal Range Optimization

The list of optimization models developed and used to generate results for Alternative 1B are presented in Table 6-7. The tidal range of this alternative is smaller than that for Alternative 1A because the tidal prism created by restoration is larger for Alternative 1B. The larger tidal prism is difficult for the existing narrow and sinuous inlet channel system to effectively convey, resulting in a restricted tidal range.

Table 6-7: Alternative 1B Tide Optimization Models

San Elijo Lagoon Bridge Optimization Modeling							
Alt 1B - Tide Modeling							
		Bridge Modeling Parameters					
		Hwy 101		LOSSAN RR		I - 5	
Run No.	File Name	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)
1	Alt1B_HW101_1	-4	105	-5.5	187	-6	261
2	Alt1B_HW101_2	-4	130	-5.5	187	-6	261
3	Alt1B_HW101_3	-4	160	-5.5	187	-6	261
4	Alt1B_HW101_4	-4	115	-5.5	187	-6	261
5	Alt1B_HW101_5	-4	145	-5.5	187	-6	261
6	Alt1B_I5_1	-4	160	-5.5	187	-6	130
7	Alt1B_I5_2	-4	160	-5.5	187	-6	196
8	Alt1B_I5_3	-4	160	-5.5	187	-6	261

Hwy 101 tidal range optimization results for Alternative 1B are presented in Figure 6-7 and Table 6-8. The tidal optimization results show that the channel width of Hwy 101 has a slight influence on tidal range throughout the lagoon for Alternative 1B. A channel width of 130 feet provides optimal tidal range and further increases to channel width produced only minimal benefits. An inlet channel width of 130 feet will require regular maintenance dredging. However, the increase in tidal prism under Alternative 1B will improve the stability of the inlet channel compared to Alternative 1A, even with a slightly more muted tidal range.

The I-5 tidal range optimization results for Alternative 1B are presented in Figure 6-8 and Table 6-9. The I-5 tide optimization modeling suggests that increasing the channel width beyond 130 feet would not improve the tidal range in the East Basin. The existing channel width of 130 feet, provided it's dredged and maintained, can provide the optimum tidal range in the East Basin. The highlighted rows in each table represent the predicted tidal range for the optimized channel width. In some cases the optimized channel width does not result in the maximum tidal range. The optimized width was identified as the point at which further increases to channel width provide little or no benefit.

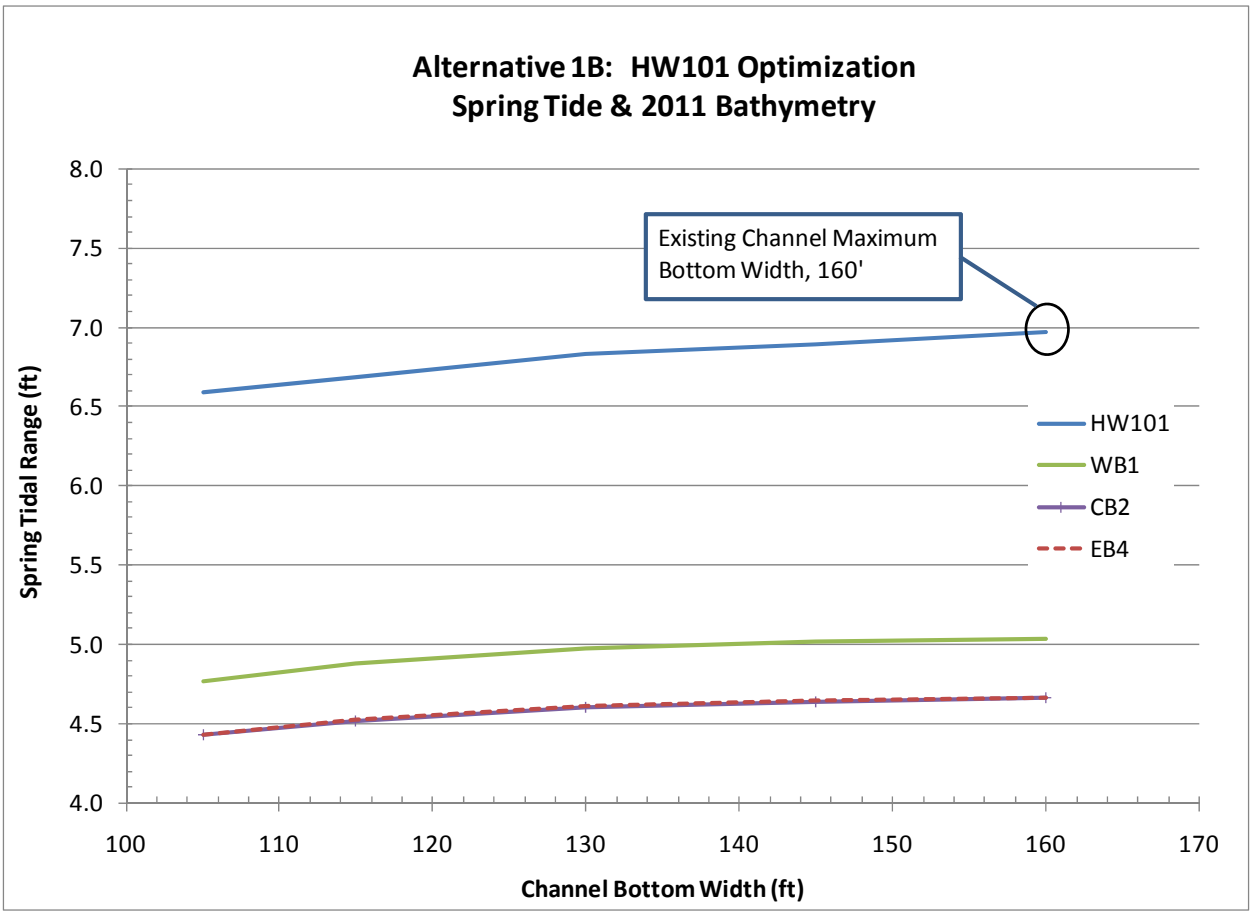


Figure 6-7: Alternative 1B Hwy 101 (Tidal Range vs Channel Width)

Table 6-8: Alternative 1B Hwy101 Tidal Range Results

Channel Dimensions (ft)		Tidal Range (ft)					
Width	Invert (NGVD)	Ocean	HW101	WB1	CB2	I-5	EB4
105	-4	8.37	6.59	4.77	4.43	4.43	4.43
115	-4	8.37	6.69	4.88	4.52	4.52	4.53
130	-4	8.37	6.83	4.98	4.61	4.61	4.61
145	-4	8.37	6.90	5.01	4.64	4.64	4.64
160	-4	8.37	6.97	5.04	4.66	4.66	4.66

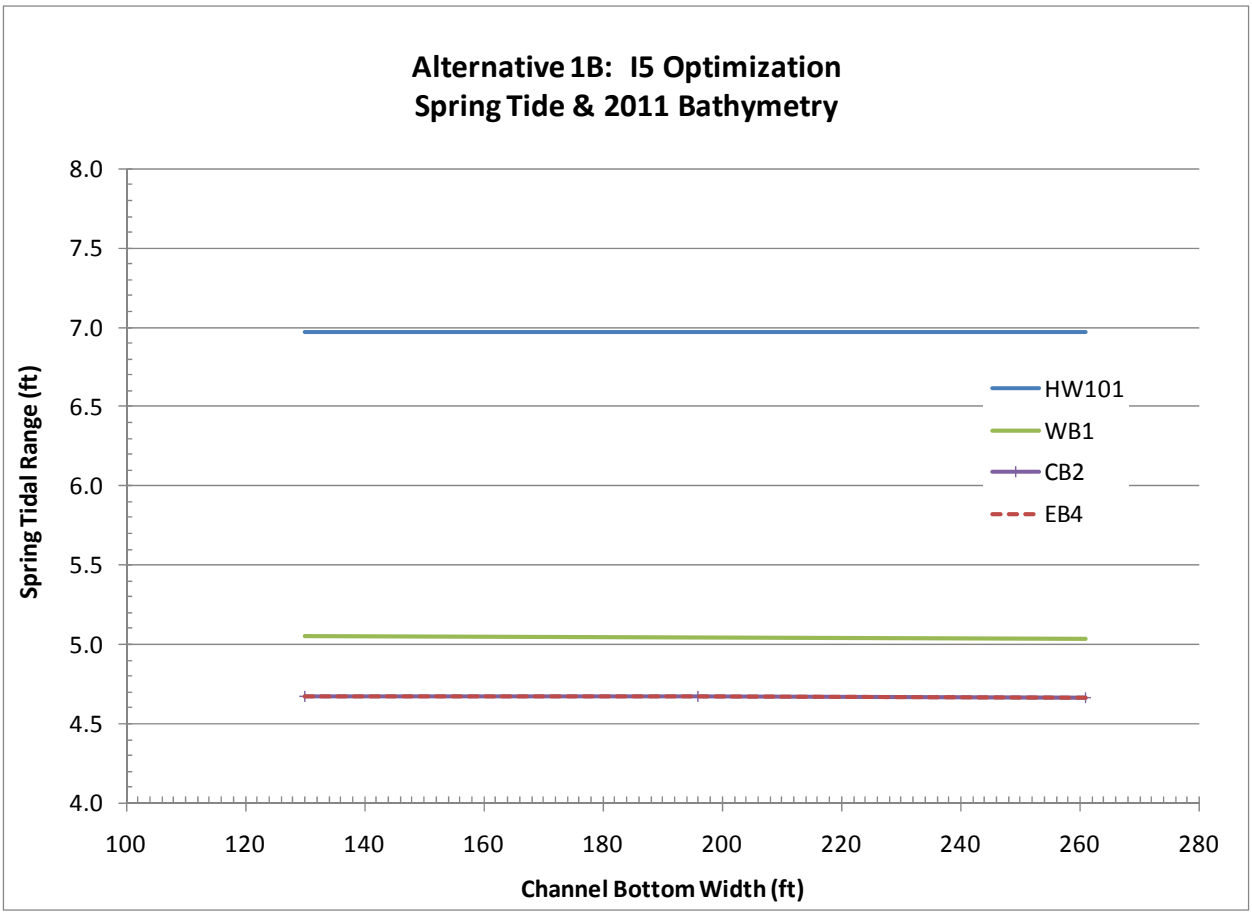


Figure 6-8: Alternative 1B I-5 Bridge (Tidal Range vs Channel Width)

Table 6-9: Alternative 1B I-5 Tidal Range Results

Channel Dimensions (ft)		Tidal Range (ft)					
Width	Invert (NGVD)	Ocean	HW101	WB1	CB2	I-5	EB4
130	-4	8.37	6.97	5.05	4.68	4.67	4.67
196	-4	8.37	6.97	5.05	4.67	4.67	4.67
261	-4	8.37	6.97	5.04	4.66	4.66	4.66

6.4 Alternative 2A Tidal Range Optimization

The tide optimization models for the new inlet channel below Hwy 101 Bridge considered changes to both width and invert elevation. The range of widths and invert elevations were centered on the design dimensions established in the SELRP Feasibility Study of 200 feet and -6.5 feet, NGVD. A list of optimization models developed and used to generate results for Alternative 2A is presented in Table 6-10.

Table 6-10: Alternative 2A Tide Optimization Models

San Elijo Lagoon Bridge Optimization Modeling							
Alt 2A - Tide Modeling							
		Bridge Modeling Parameters					
		Hwy 101		LOSSAN RR		I - 5	
Run No.	File Name	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)
1	Alt2A_HW101_1	-5	200	-15	590	-6.5	410
2	Alt2A_HW101_2	-6.5	200	-15	590	-6.5	410
3	Alt2A_HW101_3	-7	200	-15	590	-6.5	410
4	Alt2A_HW101_4	-6.5	220	-15	590	-6.5	410
5	Alt2A_HW101_5	-6.5	240	-15	590	-6.5	410
6	Alt2A_HW101_6	-6.5	180	-15	590	-6.5	410
7	Alt2A_HW101_7	-5	220	-15	590	-6.5	410
8	Alt2A_HW101_8	-5	240	-15	590	-6.5	410
9	Alt2A_HW101_9	-7	160	-15	590	-6.5	410
10	Alt2A_HW101_10	-7	180	-15	590	-6.5	410
11	Alt2A_HW101_11	-6.5	160	-15	590	-6.5	410
12	Alt2A_HW101_12	-6.5	120	-15	590	-6.5	410
13	Alt2A_HW101_13	-6.5	140	-15	590	-6.5	410
14	Alt2A_HW101_14	-5	280	-15	590	-6.5	410
15	Alt2A_I5_1	-6.5	200	-15	590	-6.5	261
16	Alt2A_I5_2	-6.5	200	-15	590	-6.5	327
17	Alt2A_I5_3	-6.5	200	-15	590	-6.5	392
18	Alt2A_I5_4	-6.5	200	-15	590	-6.5	458

In general, the Hwy 101 optimization results supported the recommendation of the feasibility study and showed that an invert elevation of -6.5 feet and channel width of 200 feet will provide the optimum tidal range. The modeling suggests that variations of inlet channel width and invert elevation will result in relatively small changes in tidal range. In general, a narrower inlet width of 160 feet required a deeper invert elevation of -7 feet, NGVD to provide a tidal range equal to optimized conditions (therefore requiring jetties to maintain inlet depth). Model simulations using a shallower invert elevation of -5 feet, NGVD showed a slightly smaller tidal range, even if the channel was widened to 280 feet. The Hwy 101 tidal range optimization results for Alternative 2A are presented in Figure 6-9 through Figure 6-11 and in Table 6-11.

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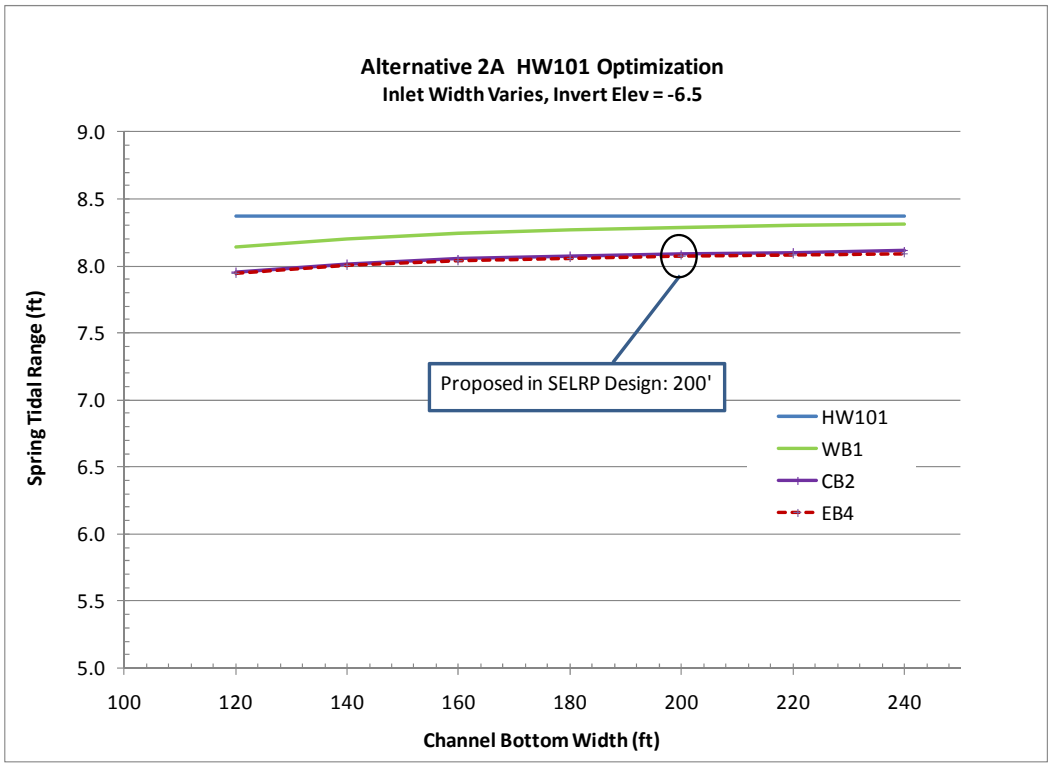

Figure 6-9: Alternative 2A Hwy 101 Tidal Range (Invert of -6.5 feet, NGVD)

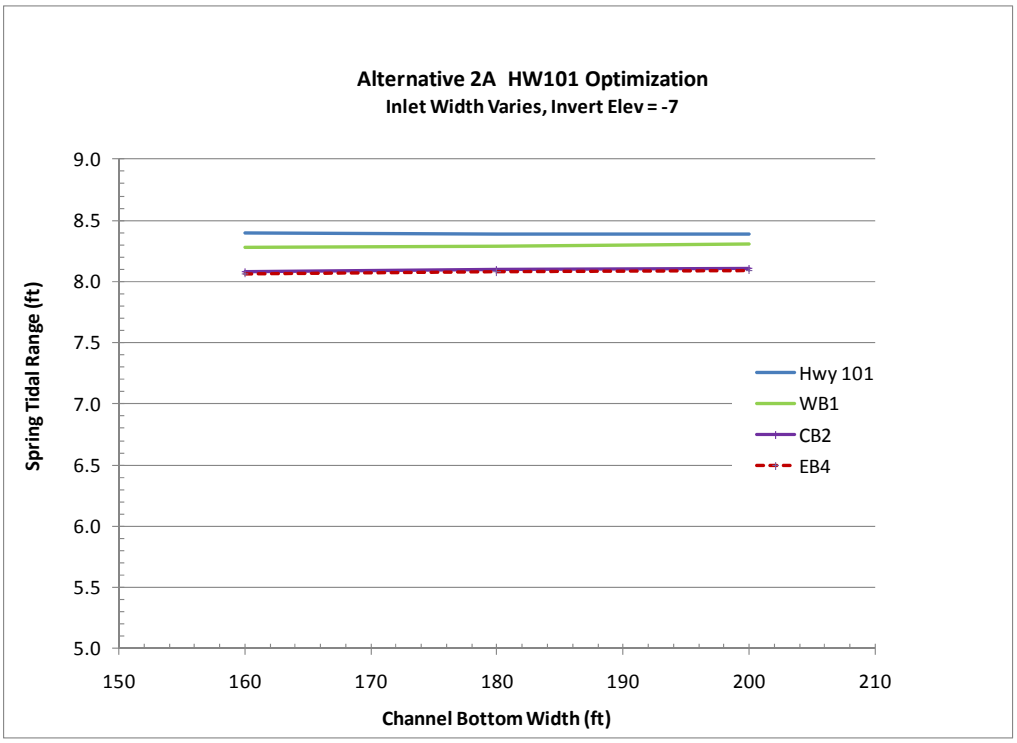

Figure 6-10: Alternative 2A Hwy 101 Tidal Range (Invert of -7 feet, NGVD)

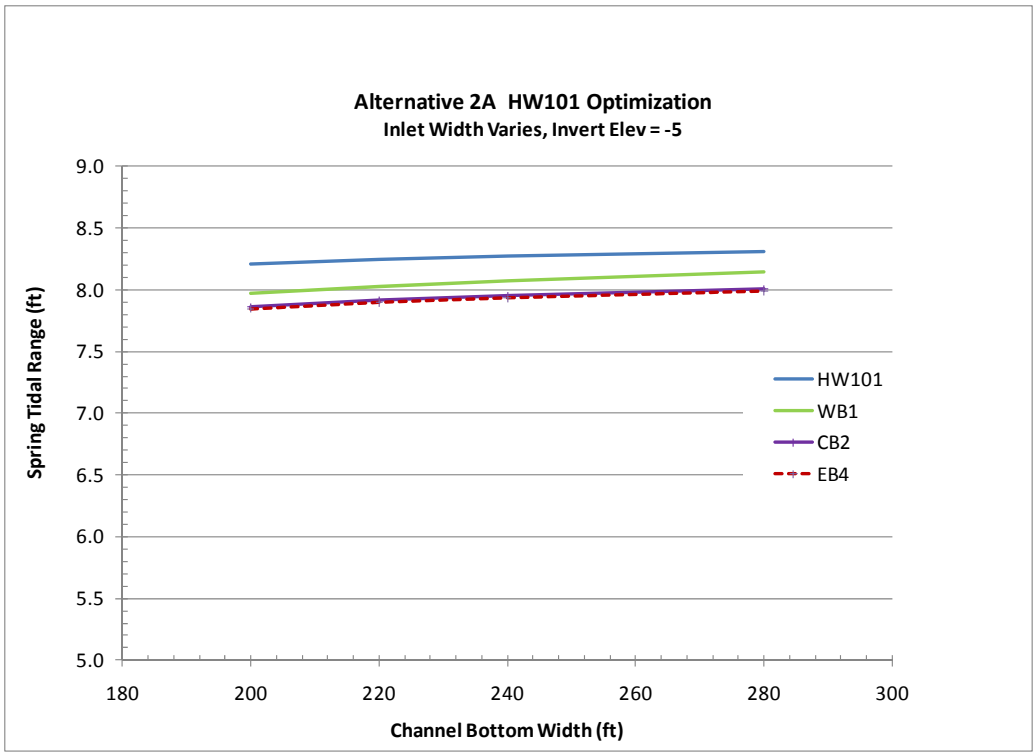

Figure 6-11: Alternative 2A Hwy 101 Tidal Range (Invert of -5 feet, NGVD)

Table 6-11: Alternative 2A Hwy 101 Tidal Range Results

Channel Dimensions (ft)		Tidal Range (ft)					
Width	Invert (NGVD)	Ocean	HW101	WB1	CB2	I-5	EB4
200	-5	8.37	8.21	7.97	7.86	7.85	7.85
220	-5	8.37	8.25	8.03	7.92	7.90	7.90
240	-5	8.37	8.28	8.08	7.95	7.94	7.93
280	-5	8.37	8.31	8.14	8.01	7.99	7.99
120	-6.5	8.37	8.37	8.14	7.96	7.95	7.94
140	-6.5	8.37	8.37	8.20	8.02	8.01	8.00
160	-6.5	8.37	8.37	8.25	8.05	8.04	8.04
180	-6.5	8.37	8.37	8.27	8.08	8.06	8.06
200	-6.5	8.37	8.37	8.29	8.09	8.08	8.07
220	-6.5	8.37	8.37	8.30	8.10	8.09	8.08
240	-6.5	8.37	8.37	8.31	8.11	8.09	8.09
160	-7	8.37	8.39	8.28	8.08	8.07	8.06
180	-7	8.37	8.39	8.29	8.10	8.08	8.08
200	-7	8.37	8.39	8.30	8.11	8.09	8.09

The I-5 tidal range optimization results for Alternative 2A are presented in Figure 6-12 and Table 6-12. The I-5 tide optimization modeling suggests that the tidal range in the East Basin is not

restricted provided a minimum channel width of 261 feet is maintained below the I-5. Feasibility studies prepared for SELRP established a minimum feasible width of 261 feet under the I-5 for Alternative 2A. For this reason, tide optimization models were not run for narrower widths of the I-5. The highlighted rows in each table represent the predicted tidal range for the optimized channel width. In some cases the optimized channel width may not correspond with the maximum tidal range. The optimized width was identified as the point at which further increases to channel width provide little or no benefit.

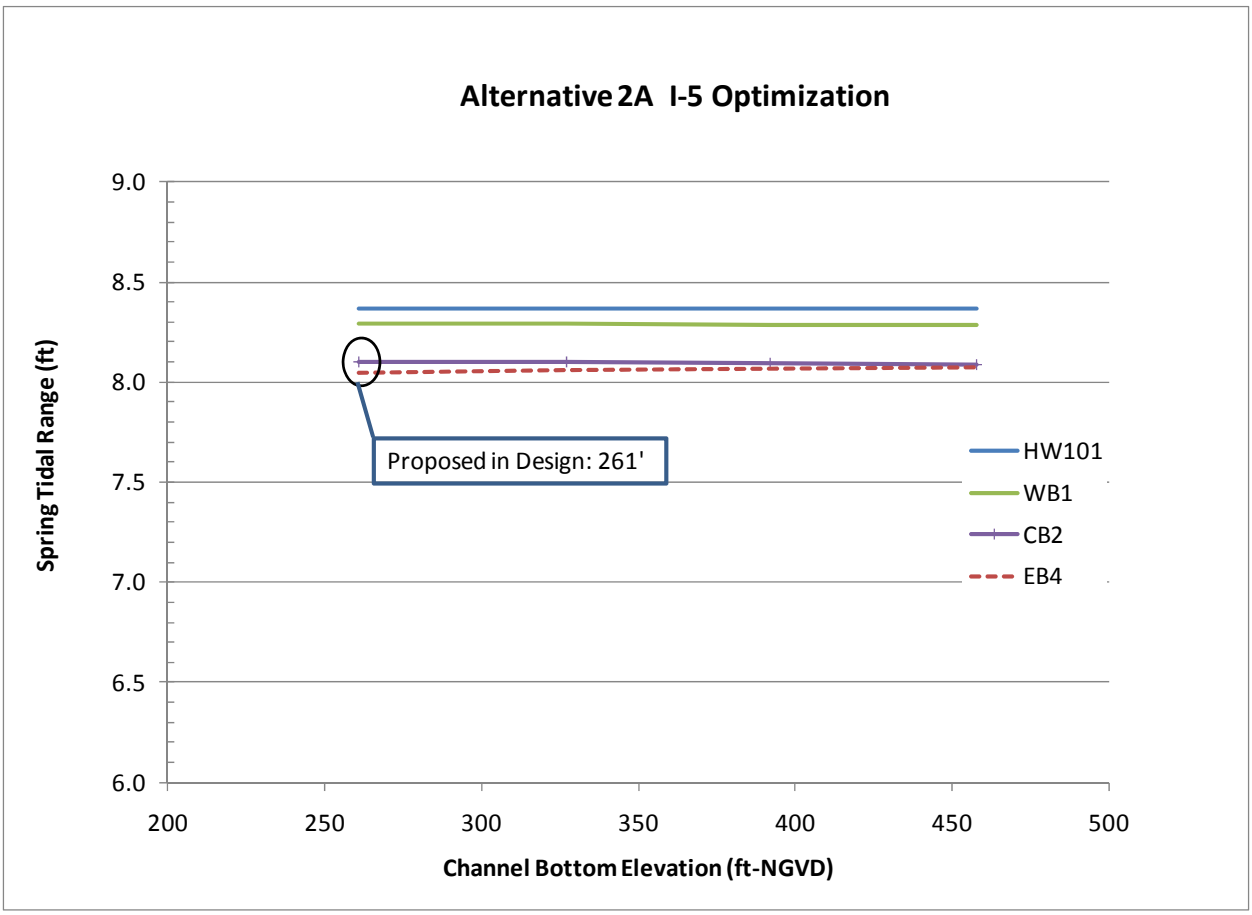

Figure 6-12: Alternative 2A I-5 Bridge (Tidal Range vs Channel Width)

Table 6-12: Alternative 2A I-5 Tidal Range Results

Channel Dimension (ft)		Tidal Range (ft)					
Width	Invert (NGVD)	Ocean	HW101	WB1	CB2	I-5	EB4
261	-6.5	8.37	8.37	8.29	8.10	8.06	8.05
372	-6.5	8.37	8.37	8.29	8.10	8.07	8.06
392	-6.5	8.37	8.37	8.29	8.09	8.07	8.07
458	-6.5	8.37	8.37	8.29	8.09	8.08	8.07

7.0 ANALYSES TO ACHIEVE OPTIMAL FLOOD ELEVATIONS

In terms of potential flooding, the main roadway of concern is Manchester Avenue that extends along the entire northern boundary of the Lagoon. Other important features are the three bridges at Hwy 101, the Railroad, and the I-5. These features have been the focus of flood studies over time by Caltrans. The flood optimization modeling assumes a 100-year event concurrent with a spring high tide equal to the FEMA 1 percent annual chance water surface elevation at the inlet of +7 feet, NGVD. Results indicate that for each alternative with the existing tidal inlet location, a reach of Manchester Avenue in both the Central and East Basins will be flooded during a 100-year event. Figure 7-1 provides a plan view showing the alignments used to generate flood profiles for each alternative. Figure 7-2 and Figure 7-3 show the RMA-2 virtual gage locations where maximum flood elevations were reported for each alternative.

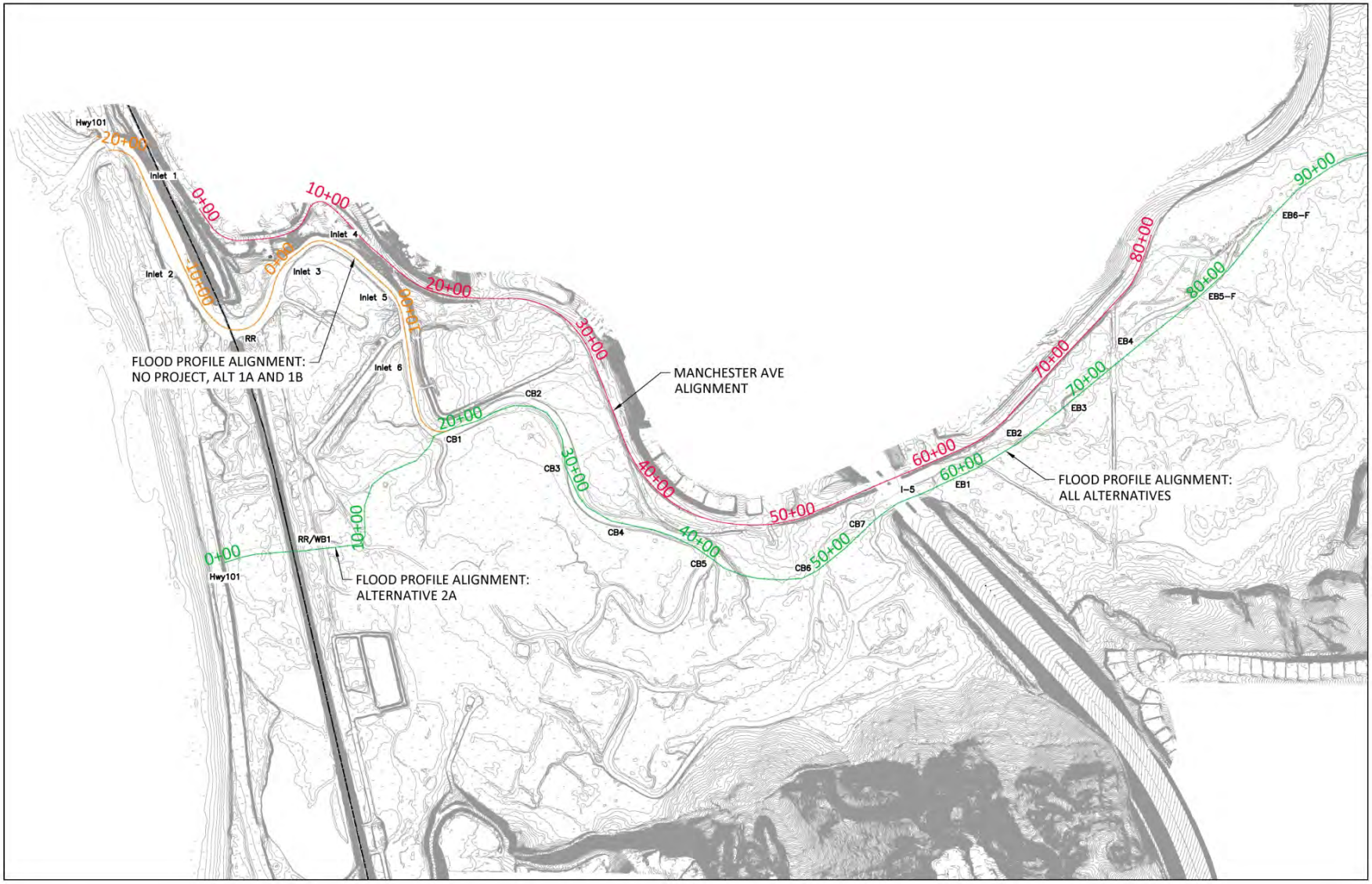

Figure 7-1: Flood Profile Alignments

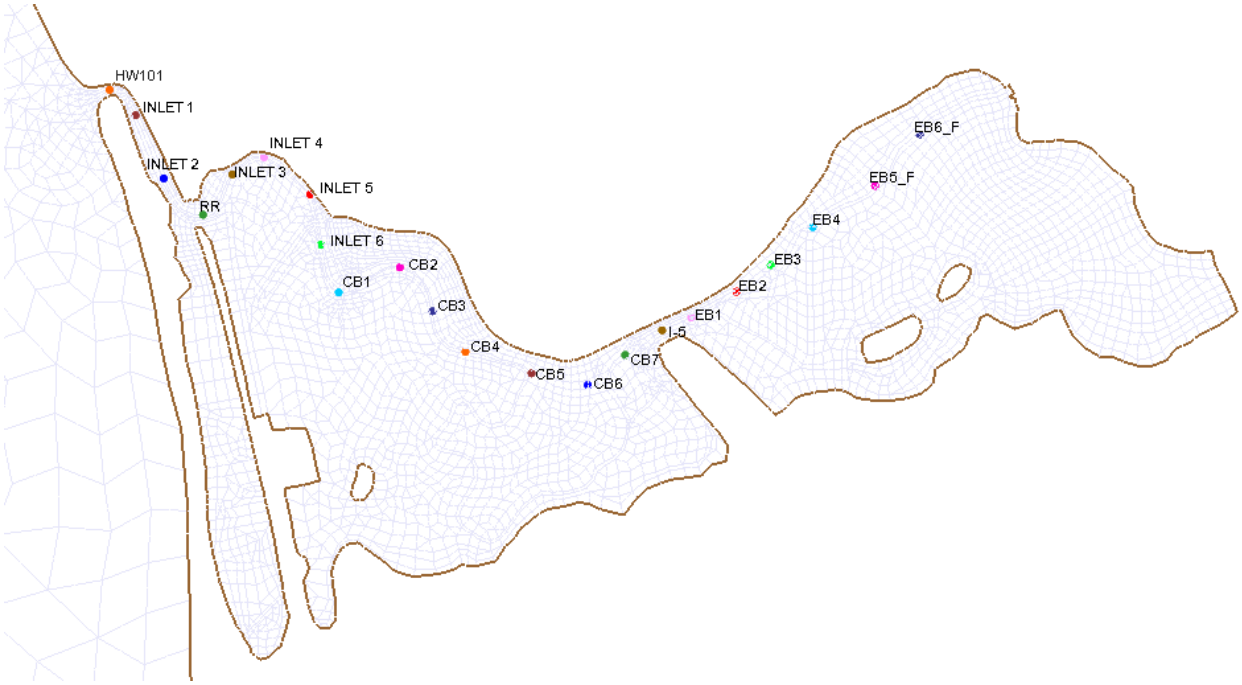

Figure 7-2: Flood Gage Locations on RMA-2 Mesh (No Project, Alternatives 1A & Alt 1B)

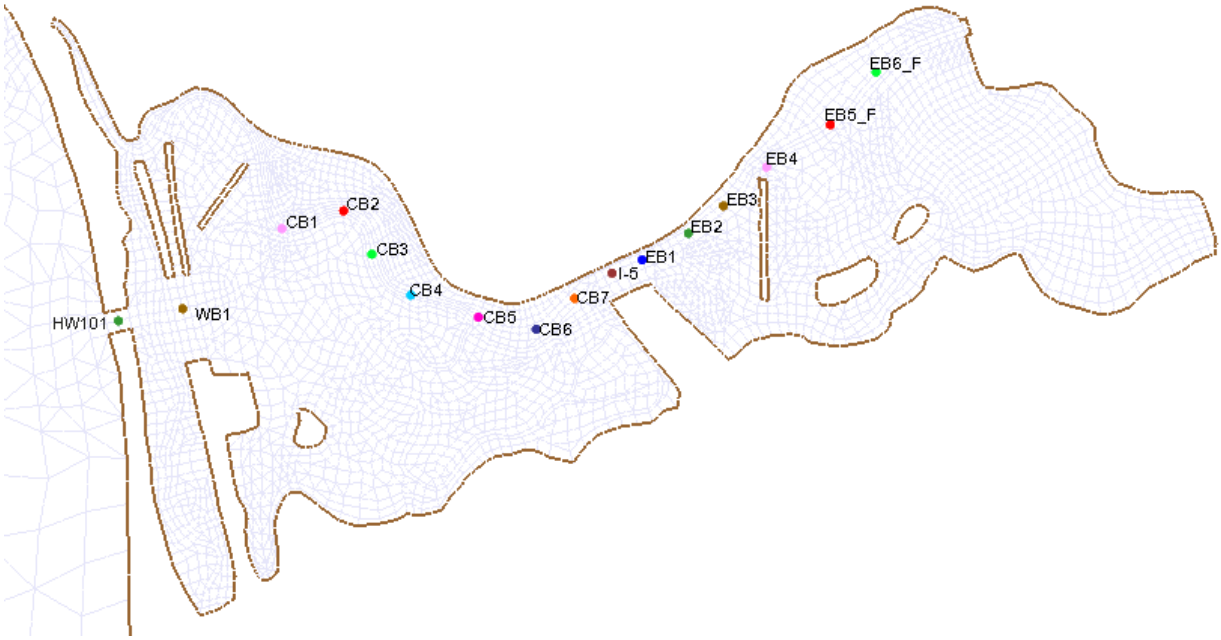

Figure 7-3: Flood Gage Locations on RMA-2 Mesh (Alternative 2A)

7.1 No Project Flood Optimization

The results for flood optimization at channels of Hwy 101 and the Railroad bridges were similar in that no significant benefit was achieved by increasing the channel width beyond its current dimensions. The narrow inlet channel and the sinuous main channel just east of the railroad seem to be limiting the flood conveyance in this location. Flood modeling results indicate that the I-5 channel does limit flood conveyance from the East Basin to the Central Basin. Widening the I-5 Bridge channel lowers the flood level in the East Basin but raises flood levels throughout the Central Basin. One simulation modeled a very wide channel under the I-5 Bridge (800 feet) under which condition Manchester Avenue would still experience flooding during a 100-year event. Since there is no increase in bridge width that would eliminate flooding in both Central and East Basins the existing width of 130 feet is considered the optimum width for this alternative. The flood profile for the optimized width predicts flooding of Manchester Avenue in the East Basin but not in the Central Basin. Table 7-1 provides a list of the flood optimization models developed for this alternative. Flood optimization modeling results for the No Project alternative are presented in Figure 7-4. The optimized 50-year and 100-year flood profiles are presented in Figure 7-5.

Table 7-1: No Project Flood Optimization Models

San Elijo Lagoon Bridge Optimization Modeling							
No Project - Flood Modeling							
		Bridge Modeling Parameters					
		Hwy 101		LOSSAN RR		I - 5	
Run No.	File Name	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)
1	NP_100yr_1_new	-0.87	70	Varies	187	0.74	130
2	NP_100yr_2_new	-0.87	70	Varies	187	-6	261
3	NP_100yr_3_new	-4	160	-5.5	187	-6	130
4	NP_100yr_4_new	-4	160	-5.5	187	-6	261
5	NP_100yr_5_new	-4	200	-5.5	187	-6	261
6	NP_100yr_6_new	-4	160	-5.5	230	-6	261
7	NP_100yr_7_new	-4	160	-5.5	187	-6	392
8	NP_100yr_8_new	-4	160	-5.5	187	-6	800
9	NP_100yr_9_new	-4	130	-5.5	187	-6	130
10	NP_100yr_10_new	-4	105	-5.5	187	-6	130
11	NP_50yr_1	-4	105	-5.5	187	-6	130
12	NP_SLR&100yr_1	-4	105	-5.5	187	-6	130

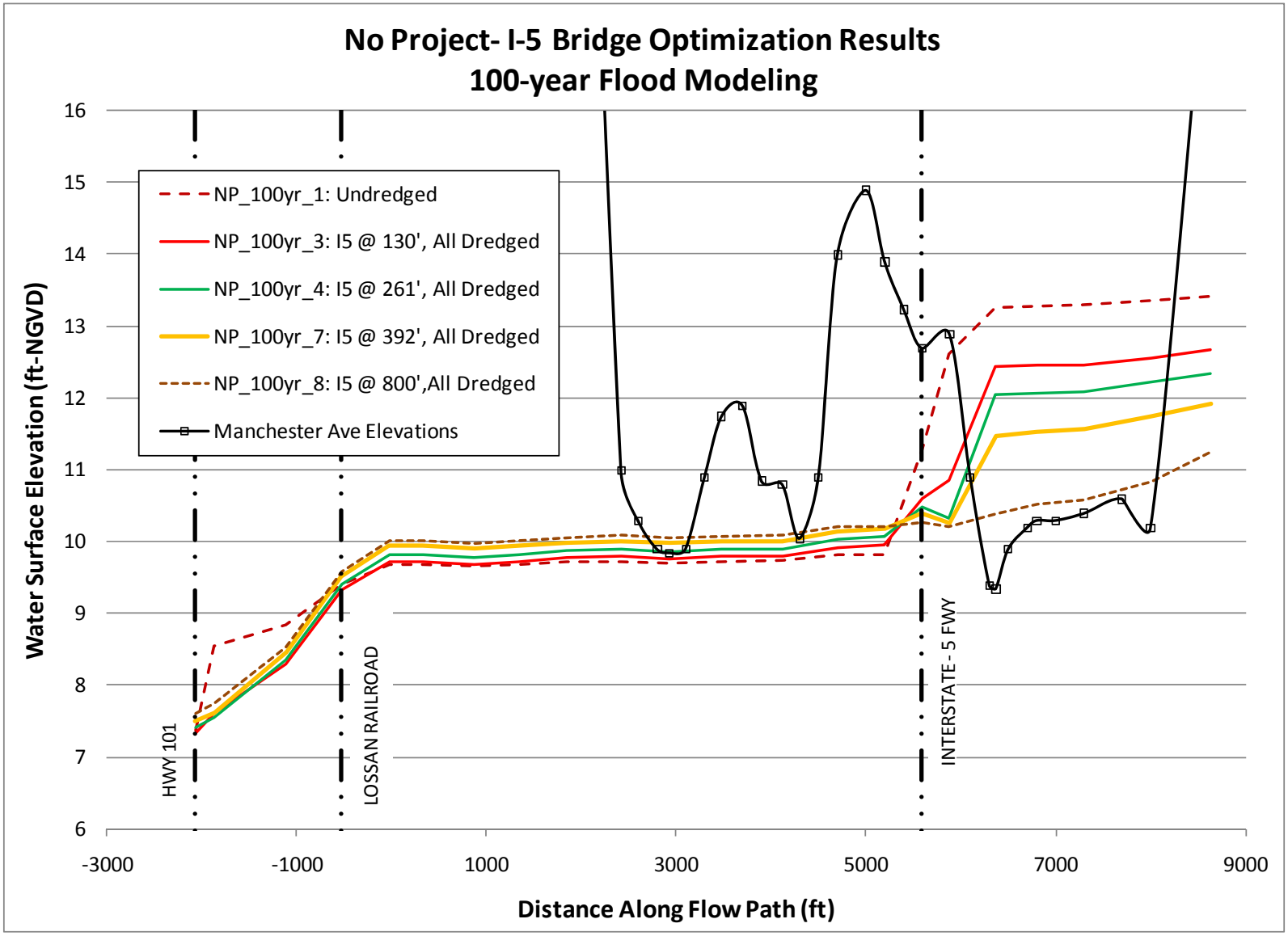

Figure 7-4: No Project Flood Optimization Profiles

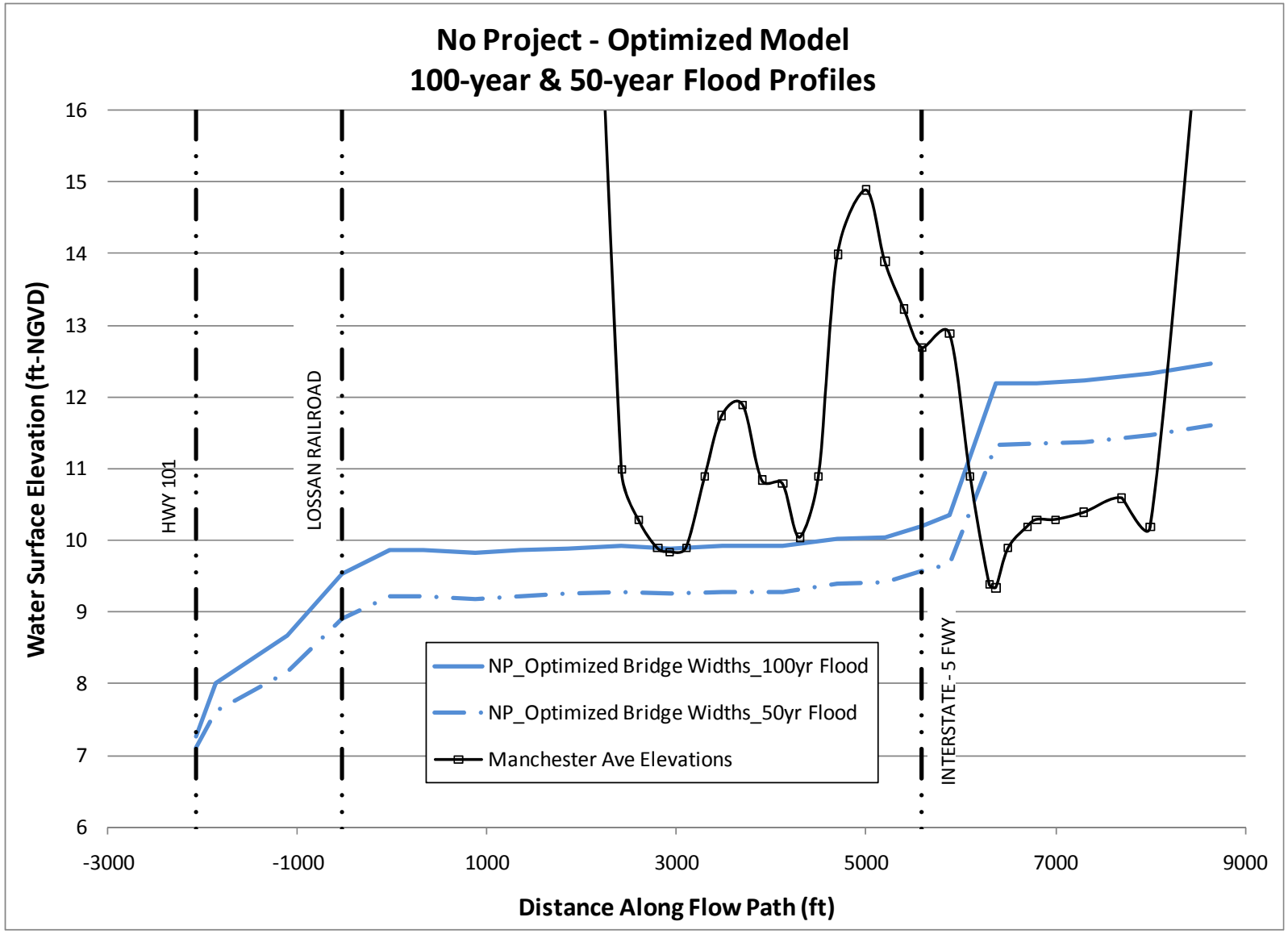

Figure 7-5: Optimized No Project Model – 50-year & 100-year Flood Profiles

7.2 Alternative 1A Flood Optimization

Similar to the results for the No Project alternative, no significant benefit was achieved by increasing the channel width below Hwy 101 and the Railroad Bridges for Alternative 1A. The existing bathymetry of the lagoon inlet, primarily the narrow and sinuous main channel, is limiting flood conveyance in this location. The I-5 optimization models suggest the existing channel width restricts flood conveyance creating a backwater effect in the East Basin. This attenuation provided by the East Basin is somewhat beneficial for flood conditions in the Central Basin and results in lower flood elevations. For example, the existing I-5 channel width of 130 feet causes flooding of Manchester Avenue in the East Basin but little or no flooding in the Central Basin. If the I-5 Bridge channel is widened, the flood wave reaches the Central Basin sooner, raising the flood levels above Manchester Avenue. Since there is no significant benefit to widening the I-5 channel, the existing width of 130 feet is considered optimum for this alternative. Table 7-2 provides a list of the flood optimization models developed for this alternative. Flood optimization modeling results for Alternative 1A are presented in Figure 7-6. The optimized 50-year and 100-year flood profiles are presented in Figure 7-7.

Table 7-2: Alternative 1A Flood Optimization Models

San Elijo Lagoon Bridge Optimization Modeling							
Alt 1A - Flood Modeling							
		Bridge Modeling Parameters					
		Hwy 101		LOSSAN RR		I - 5	
Run No.	File Name	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)
1	Alt1A_100yr_1_new	-4	105	-5.5	187	-6	130
2	Alt1A_100yr_2_new	-4	160	-5.5	187	-6	130
3	Alt1A_100yr_3_new	-4	160	-5.5	187	-6	196
4	Alt1A_100yr_4_new	-4	160	-5.5	187	-6	261
5	Alt1A_100yr_5_new	-4	160	-5.5	187	-6	392
6	Alt1A_100yr_6_new	-4	160	-5.5	230	-6	261
7	Alt1A_100yr_7_new	-4	115	-5.5	187	-6	130
8	Alt1A_50yr_1	-4	115	-5.5	187	-6	130
9	Alt1A_SLR&100yr_1	-4	115	-5.5	187	-6	130

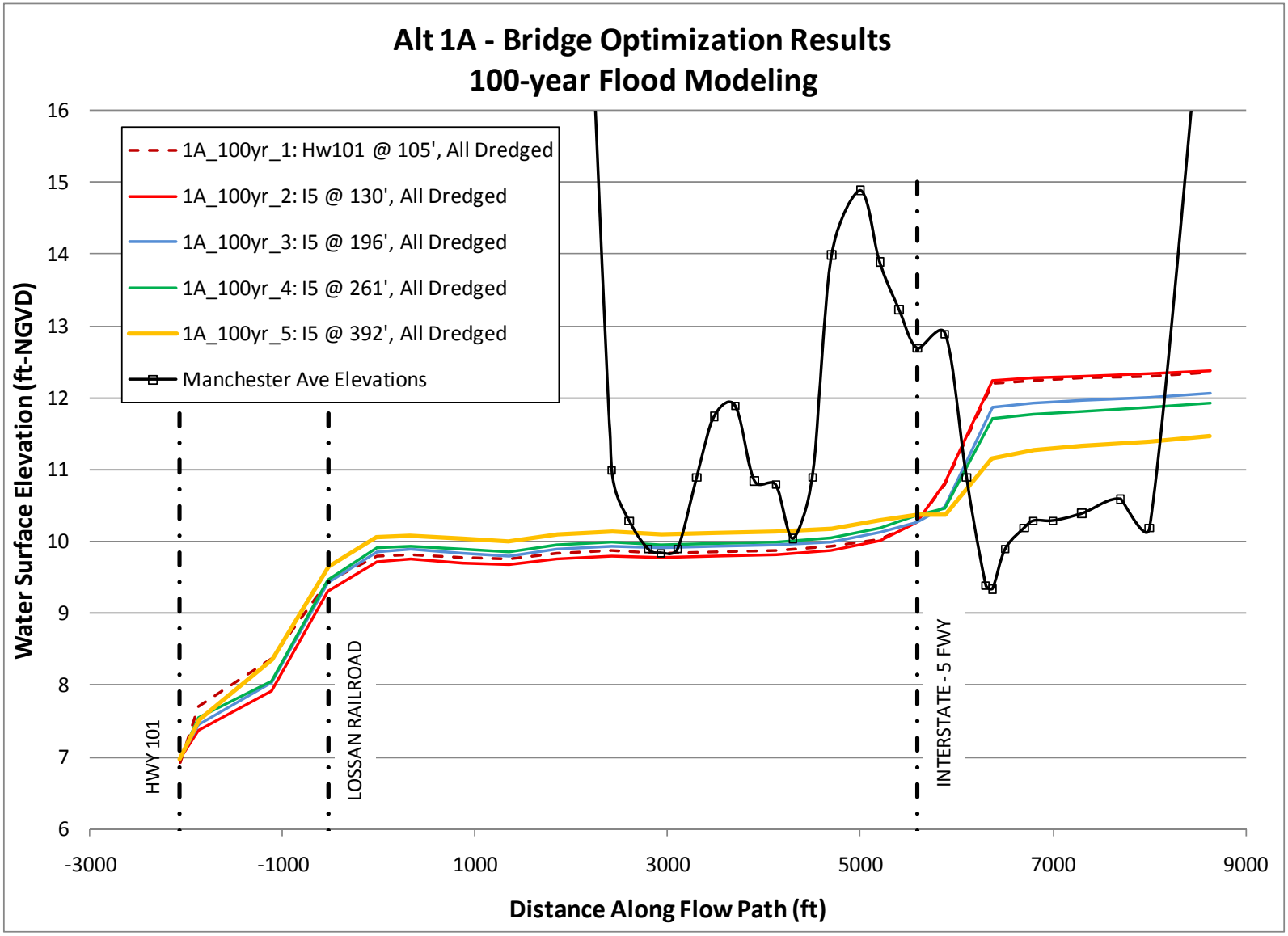

Figure 7-6: Alternative 1A Flood Optimization Profiles

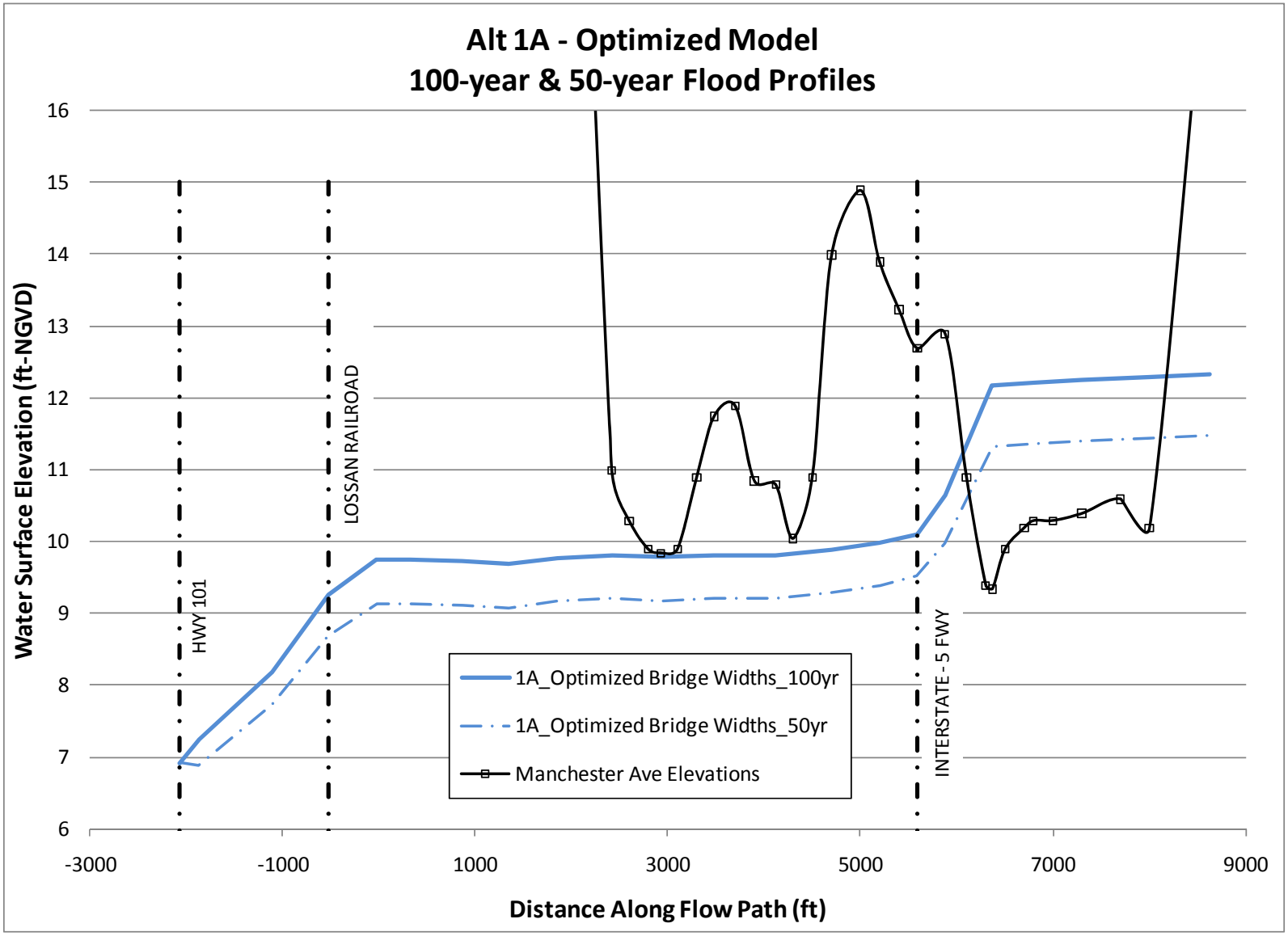

Figure 7-7: Optimized Alternative 1A Model – 50-year & 100-year Flood Profiles

7.3 Alternative 1B Flood Optimization

Similar to previous results, flood optimization of Hwy 101 and the Railroad Bridges showed there was no significant benefit achieved by increasing the channel width beyond its current dimensions. The narrow inlet channel and the sinuous main channel just east of the railroad seem to be limiting the flood conveyance in this location. The flood profiles for Alternative 1B indicate the existing I-5 Bridge channel width creates only a slight backwater effect. The optimum I-5 channel width specified for this alternative is 261 feet. Increasing the channel width to 261 feet will lower flood elevations enough to eliminate flooding for almost 1,000 feet of Manchester Avenue in the East Basin. Overall, the flood profiles in the Central and East Basins are noticeably flatter indicating very little head loss throughout the lagoon. This is due to the increased cross-sectional area of the main channel under Alternative 1B conditions. The flood conveyance capacity of the lagoon is mostly restricted by the existing inlet configuration where almost 3 feet of head loss occurs over a relatively short distance. Table 7-3 provides a list of the flood optimization models developed for this alternative. Flood optimization modeling results for Alternative 1B are presented in Figure 7-8. The optimized 50-year and 100-year flood profiles are presented in Figure 7-9.

Table 7-3: Alternative 1B Flood Optimization Models

San Elijo Lagoon Bridge Optimization Modeling							
Alt 1B - Flood Modeling							
		Bridge Modeling Parameters					
		Hwy 101		LOSSAN RR		I - 5	
Run No.	File Name	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)
1	Alt1B_100yr_1	-4	120	-5.5	187	-6	130
2	Alt1B_100yr_2	-4	160	-5.5	187	-6	130
3	Alt1B_100yr_3	-4	160	-5.5	187	-6	196
4	Alt1B_100yr_4	-4	160	-5.5	187	-6	261
5	Alt1B_100yr_5	-4	160	-5.5	187	-6	392
6	Alt1B_100yr_6	-4	160	-5.5	230	-6	261
7	Alt1B_100yr_7	-4	130	-5.5	187	-6	261
8	Alt1B_100yr_8	-4	130	-5.5	187	-6	196
9	Alt1B_50yr_1	-4	130	-5.5	187	-6	261
10	Alt1B_SLR&100yr_1	-4	130	-5.5	187	-6	261

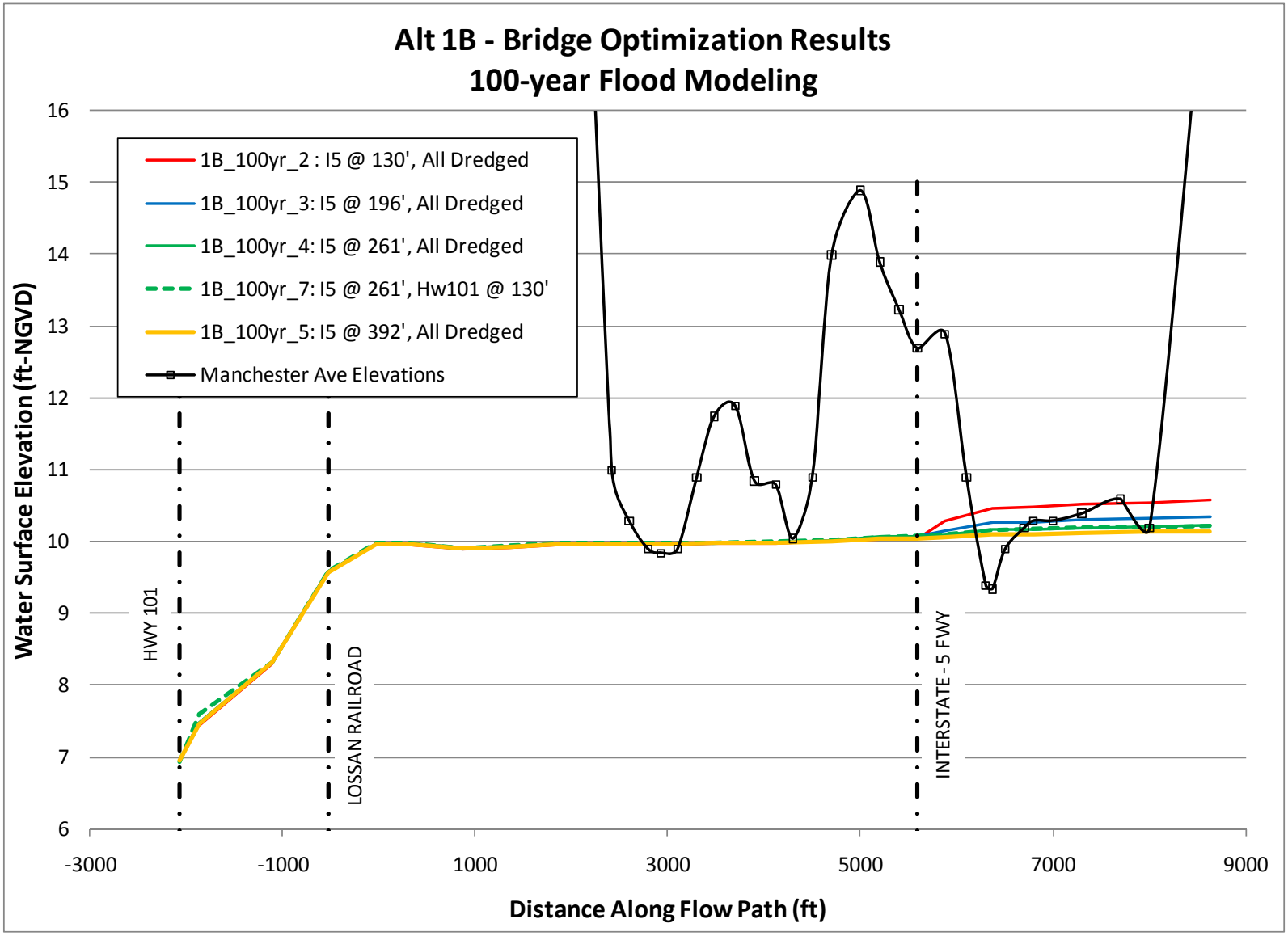

Figure 7-8: Alternative 1B Flood Optimization Profiles

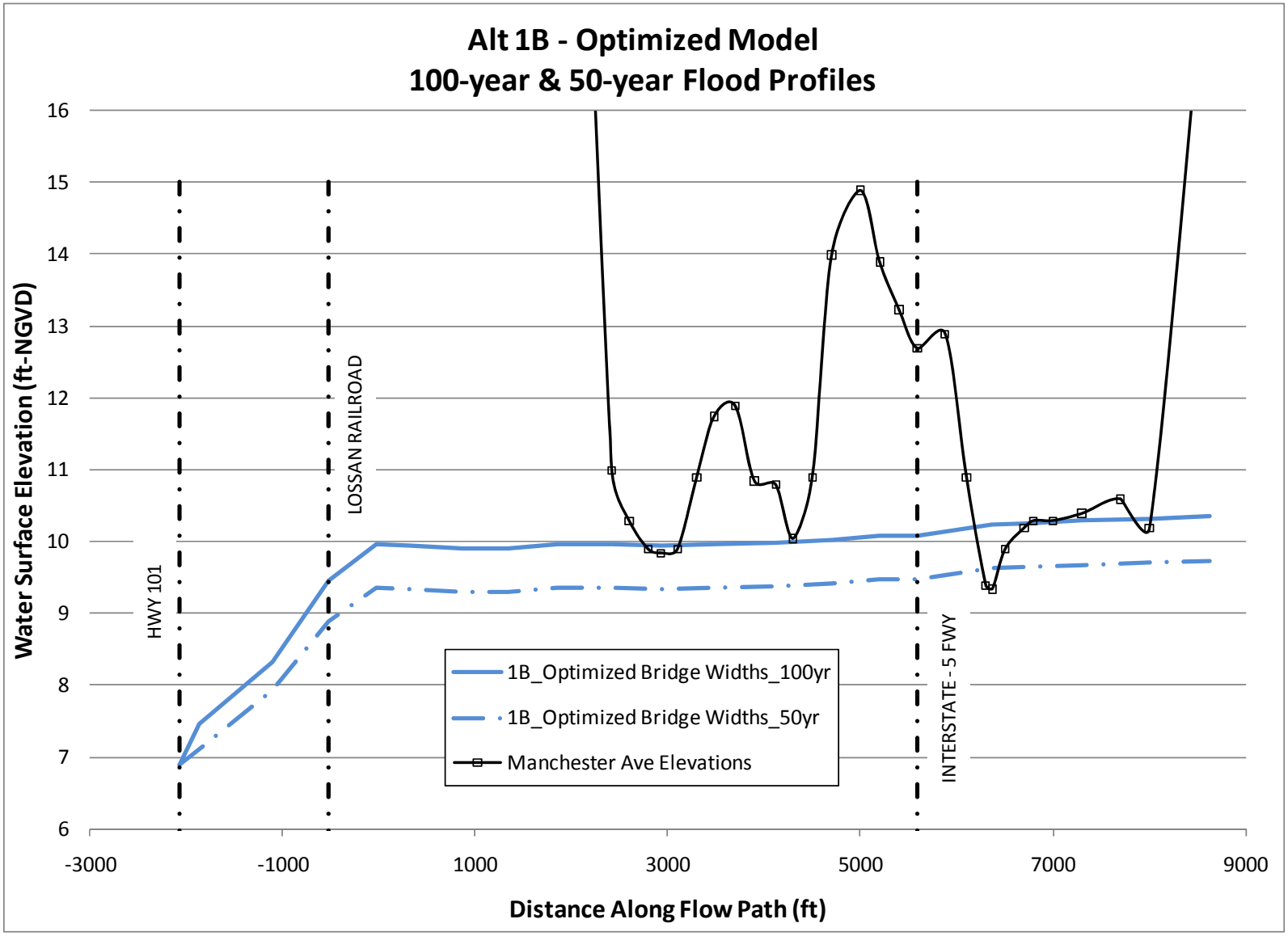

Figure 7-9: Optimized Alternative 1B Model – 50-year & 100-year Flood Profiles

7.4 Alternative 2A Flood Optimization

This new inlet configuration proposed for Alternative 2A will result in a significant improvement to the flood conveyance capacity of the San Elijo Lagoon. The new inlet configuration provides a more direct path for flood flows to travel through the lagoon as well as a wider inlet channel. The flood elevations throughout the Central Basin are 2 feet lower compared to alternatives which rely on the existing inlet.

The flood optimization models, listed in Table 7-4, focused on channel width combinations below Hwy 101 and the I-5 Bridges. The proposed channel width under the railroad is sufficiently wide (590 feet) to convey flood flows. These results are presented in Figure 7-10. If the inlet under Hwy 101 is 200 feet wide and the I-5 channel is widened to 261 feet, modeling results indicate the 100-year flood elevations would remain below Manchester Avenue. This combination of bridge channel widths was considered optimum for flood conveyance. The 50-year and 100-year flood profiles for this optimized condition are shown in Figure 7-11.

Table 7-4: Alternative 2A Flood Optimization Models

San Elijo Lagoon Bridge Optimization Modeling							
Alt 2A - Flood Modeling							
		Bridge Modeling Parameters					
		Hwy 101		LOSSAN RR		I - 5	
Run No.	File Name	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)	Invert Elev (ft, NGVD)	Width (ft)
1	Alt2A_100yr_I5_1	-6.5	200	-15	590	-6.5	234
2	Alt2A_100yr_I5_1a	-6.5	200	-15	590	-6.5	261
6	Alt2A_100yr_I5_2	-6.5	200	-15	590	-6.5	327
7	Alt2A_100yr_I5_3	-6.5	200	-15	590	-6.5	392
8	Alt2A_100yr_I5_4	-6.5	200	-15	590	-6.5	196
9	Alt2A_100yr_I5_5	-6.5	200	-15	590	-6.5	130
15	Alt2A_50yr_I5_1a	-6.5	200	-15	590	-6.5	261
16	Alt2A_SLR&100yr_1	-6.5	200	-15	590	-6.5	261

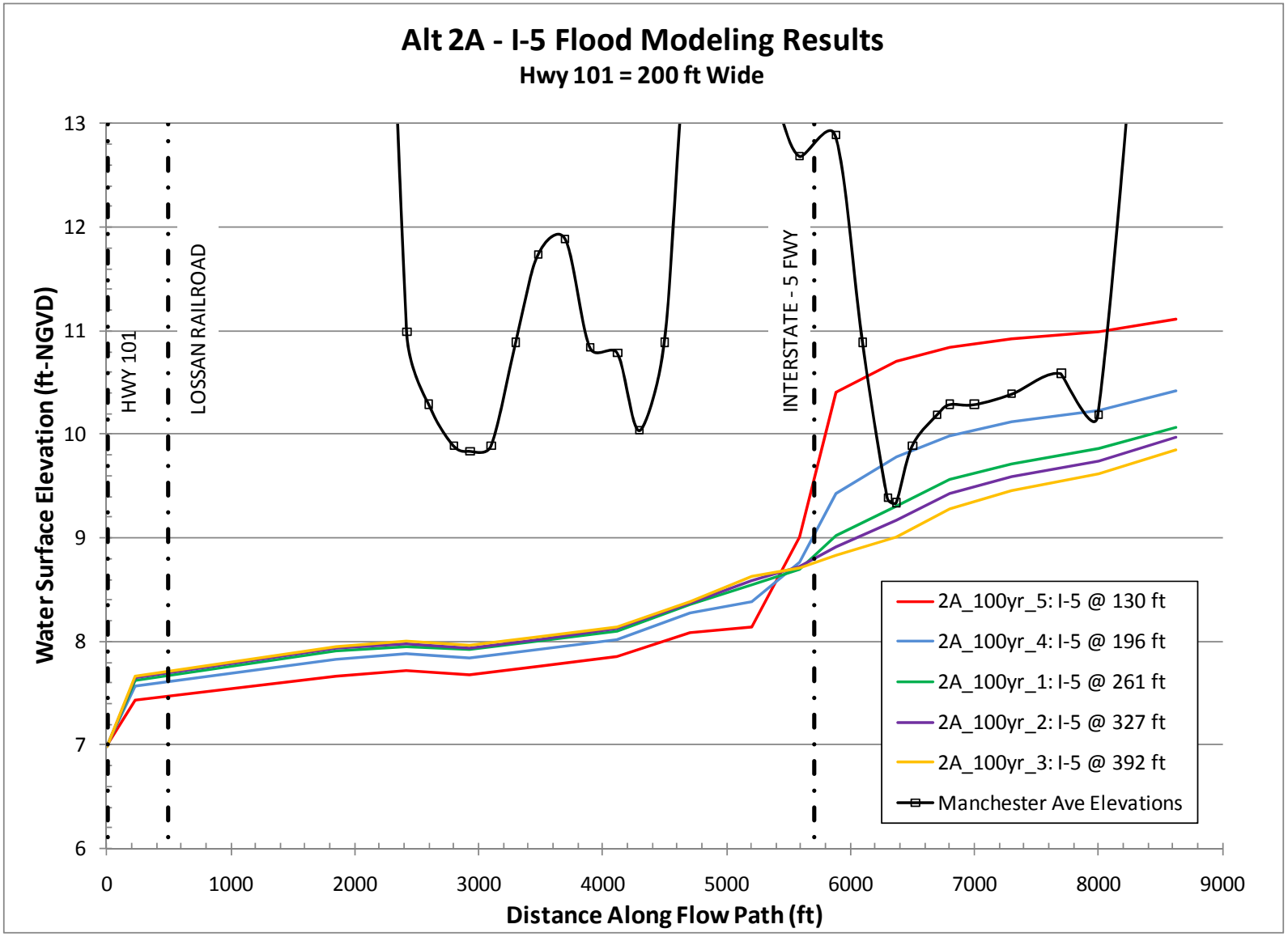

Figure 7-10: Alternative 2A Flood Optimization Profiles (Hwy101 = 200 feet)

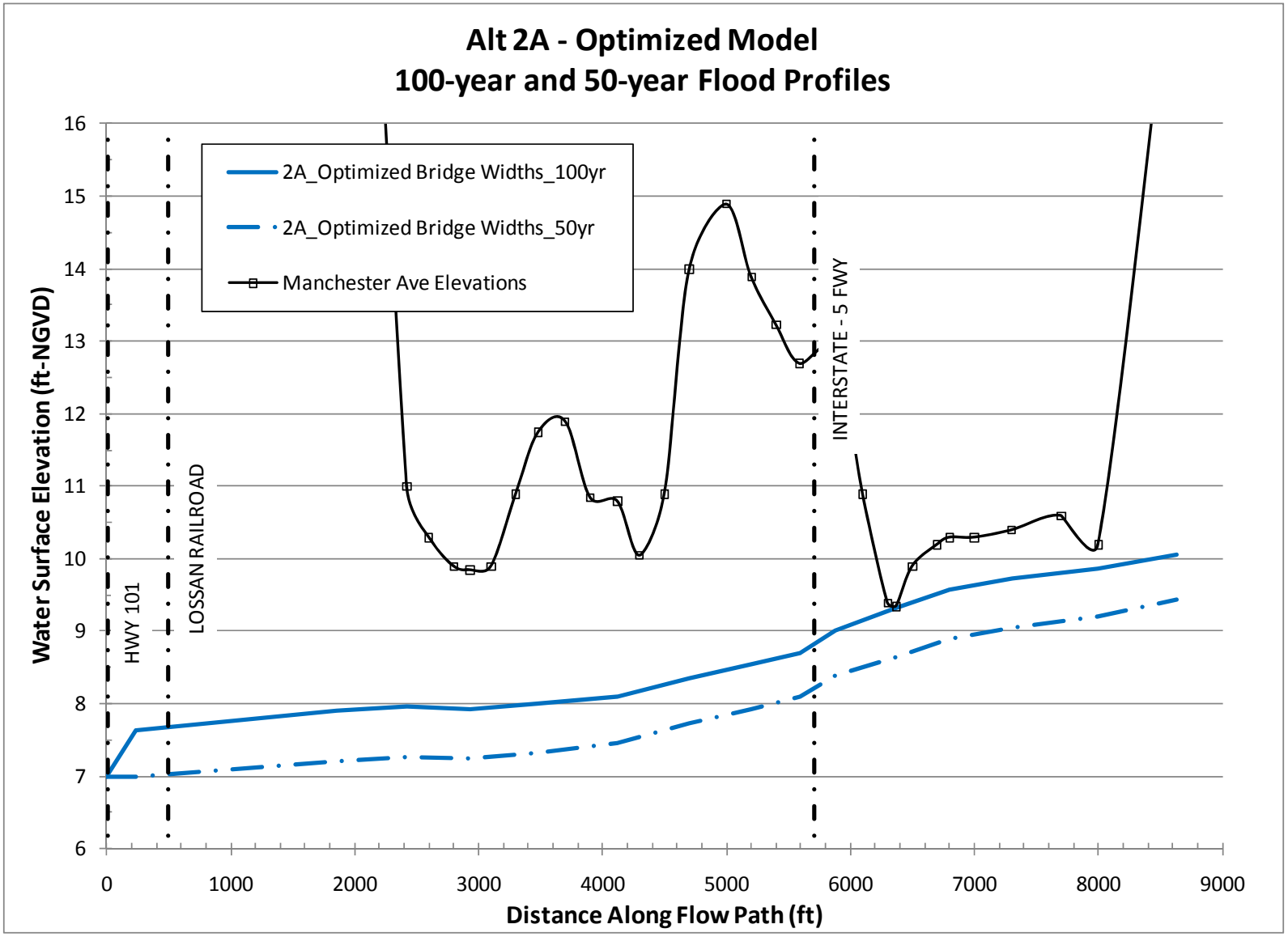

Figure 7-11: Optimized Alternative 2A Model – 50-year & 100-year Flood Profiles

8.0 SUMMARY OF EXISTING AND OPTIMIZED BRIDGE CHANNEL DIMENSIONS

Table 8-1 through Table 8-3 summarize the recommended new channel dimensions (referred as the Optimized Channel Dimensions) based on the tidal and flood optimization modeling results discussed in previous sections. The existing invert elevations are appropriate when the channels are dredged to the design condition. The existing channel bottom widths for each bridge are based on survey data and/or record drawings. The optimized channel dimensions were used in the following sections for analyses of velocity at bridge crossings and sedimentation patterns in the lagoon, and tidal inundation frequency and residence time under the dry weather condition, and hydraulic impacts of predicted SLR.

8.1 Highway 101

For No Project, Alternative 1A, and Alternative 1B, the existing Hwy 101 Bridge structure is not a limiting factor for tidal range or flood conveyance. The limiting factor is the long and narrow inlet channel between Hwy 101 and the Railroad Bridge. The instability of the existing inlet is also a problem. The maximum channel bottom width under the Hwy 101 Bridge is 160 feet. However, the unstable inlet cannot support a channel that wide. Each of these alternatives would require a maintenance dredging plan to maintain a minimum inlet width. The optimized channel dimension slightly varies for each alternative, depending on the tidal prism, but for all cases this width is less than the maximum existing bottom width of 160 feet. A cross-section of the existing Hwy 101 Bridge channel and the optimized channel widths for existing conditions and Alternatives 1A and 1B is shown in Figure 8-1.

For Alternative 2A, a 200-foot inlet channel width is considered optimized if the I-5 channel is 261 feet wide. A cross-section of the proposed Hwy 101 Bridge channel for Alternative 2A is shown in Figure 8-2. Flood conveyance is the controlling factor for the optimized Hwy 101 inlet channel width. Variations to the inlet channel width and invert elevation result in relatively small changes in tidal range. Preliminary optimization results had indicated a 160-feet-wide tidal inlet channel (with jetties) may be feasible since it had the potential to be more stable due to the higher inlet flow velocity. However, any inlet narrower than 200-feet wide would also require a deeper tidal inlet (to -7 feet NGVD) to achieve an equivalent tidal range in the lagoon. A deeper tidal inlet would require jetties for the inlet to be stable. A higher velocity in the tidal inlet would also jet the flood shoal further into the Central Basin similar to what occurs at Batiquitos Lagoon. The 200-foot-wide tidal inlet channel for Alternative 2A is considered superior to a 160-foot-wide channel because it was sized appropriately to function without jetties. It was optimized in size to provide the tidal flow velocities required to sustain scour and sediment removal under typical conditions.

However, the proposed 200-foot-wide inlet channel without jetties is still considered only “conditionally stable” due to unknowns about potential vulnerabilities during severe coastal storms. Any narrower channel such as a 160-foot-wide inlet without jetties is even more vulnerable to channel closure under severe coastal storm conditions than the 200-foot-wide inlet channel. A narrower channel at a non-jettied inlet is also more vulnerable to gradual closure during typical conditions than the 200-foot-wide inlet channel. Finally, the 160-foot-wide inlet

leads to higher 100-year storm flood elevations throughout the lagoon in year 2100 than the 200-foot-wide inlet. Constraining the inlet width for Alternative 2A too much will likely be a disadvantage to the project over the long-term. For these reasons, it is not recommended to further consideration of the 160-foot-wide tidal inlet channel for Alternative 2A.

Table 8-1: Highway 101 Optimized Bridge Channel Dimensions

Alternative	Existing Condition		Optimized for Tides		Optimized for Flood Level	
	Channel Invert (ft, NGVD)	Bottom Width (ft)	Channel Invert (ft, NGVD)	Bottom Width (ft)	Channel Invert (ft, NGVD)	Bottom Width (ft)
No Project	-4.0	160	-4.0	105	-4.0	105
1A	-4.0	160	-4.0	115	-4.0	115
1B	-4.0	160	-4.0	130	-4.0	130
2A	N/A	N/A	-6.5	200	-6.5	200

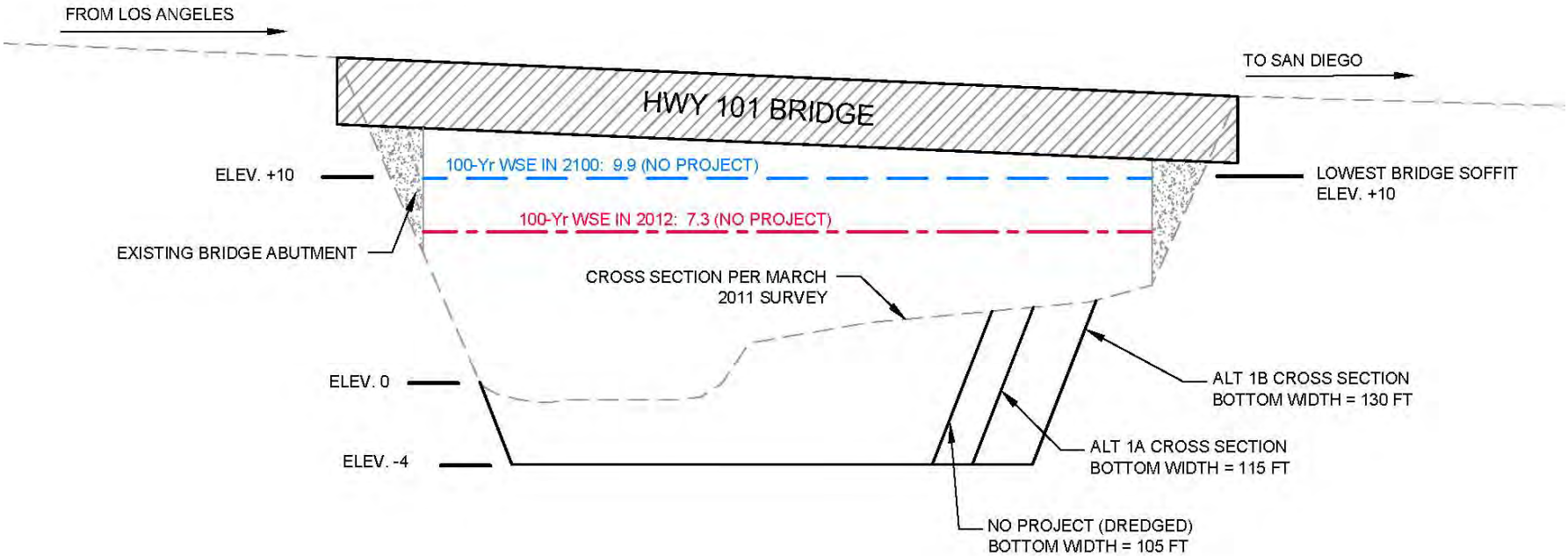

Figure 8-1: Existing Highway 101 Bridge Channel Cross-Section

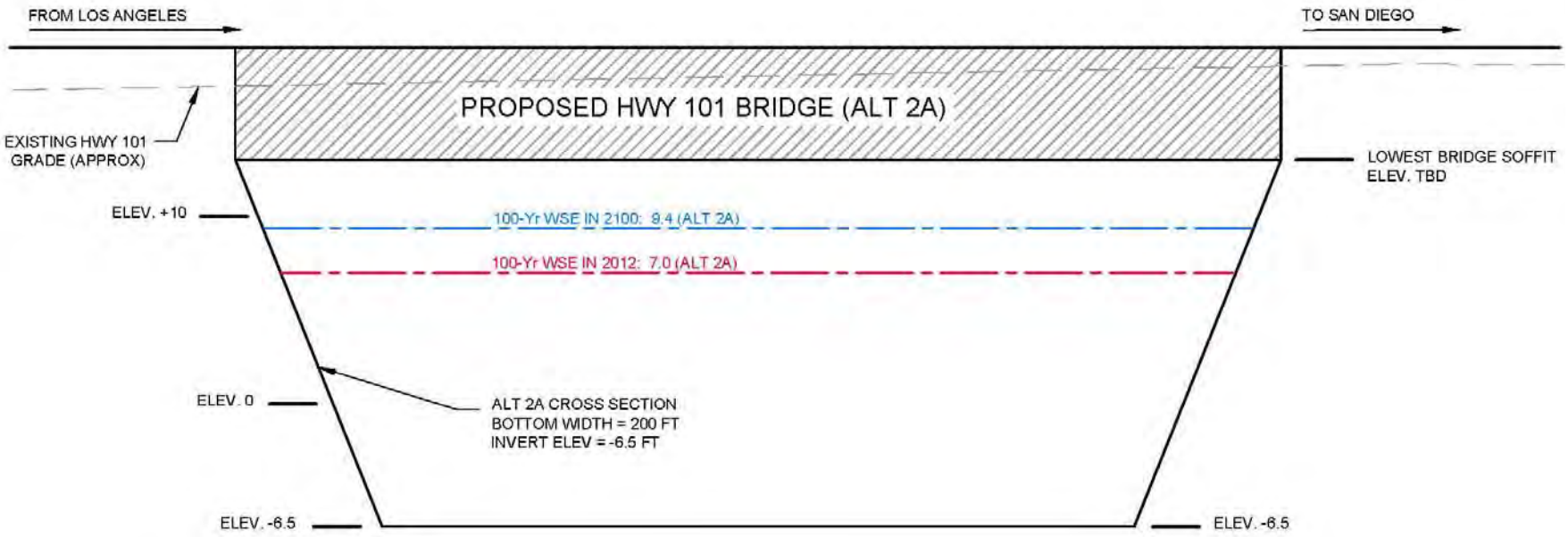

Figure 8-2: Proposed Alt 2A Highway 101 Bridge Channel Cross-Section

8.2 Railroad

The existing railroad bridge structure is sufficient for tidal exchange and flood conveyance for each alternative which utilizes the existing inlet (No Project, Alternative 1A, and Alternative 1B). The optimized bridge dimension is equal to the existing railroad bridge channel dimension for these alternatives. A cross-section of the existing railroad bridge channel is shown in Figure 8-3. For Alternative 2A, the proposed railroad bridge is also long enough to not influence tidal range or flood conveyance and is considered the optimized bridge channel dimension. A cross-section of the proposed railroad bridge channel for Alternative 2A is shown in Figure 8-4.

Table 8-2: Railroad Optimized Bridge Channel Dimensions

Alternative	Existing Condition		Optimized for Tides		Optimized for Flood Level	
	Channel Invert (ft, NGVD)	Bottom Width (ft)	Channel Invert (ft, NGVD)	Bottom Width (ft)	Channel Invert (ft, NGVD)	Bottom Width (ft)
No Project	-5.5	187	-5.5	187	-5.5	187
1A	-5.5	187	-5.5	187	-5.5	187
1B	-5.5	187	-5.5	187	-5.5	187
2A	N/A	N/A	-15.0	590	-15.0	590

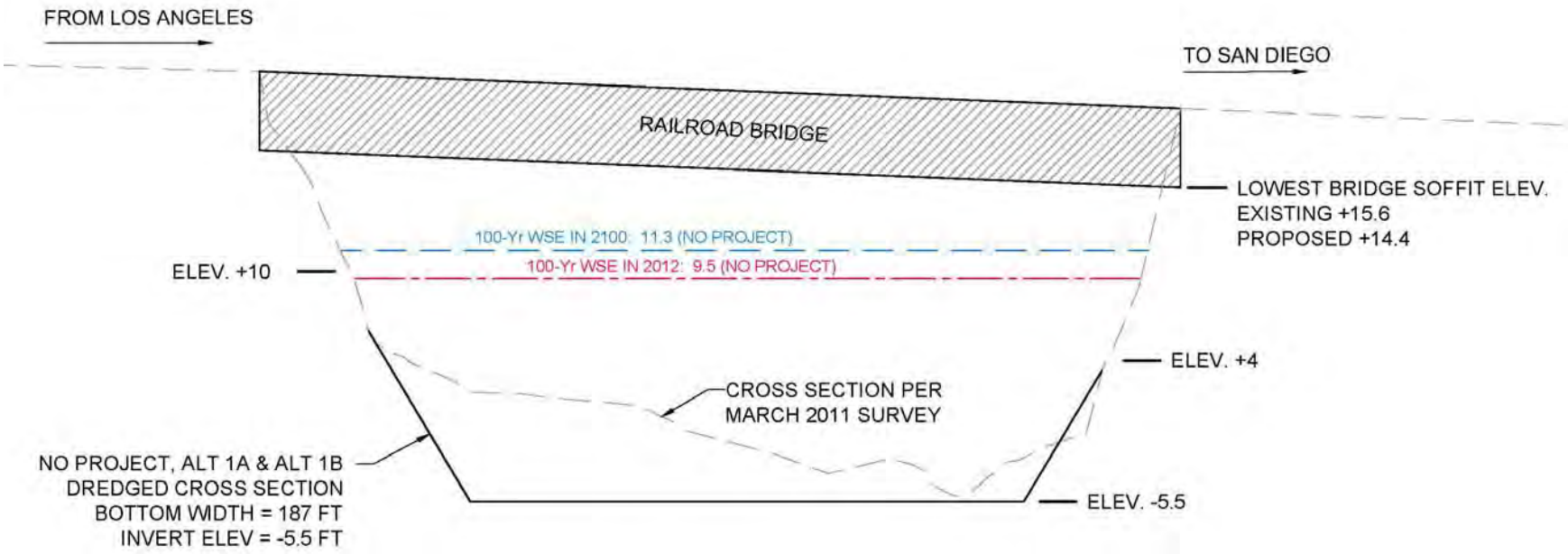

Figure 8-3: Existing Railroad Bridge Channel Cross-Section

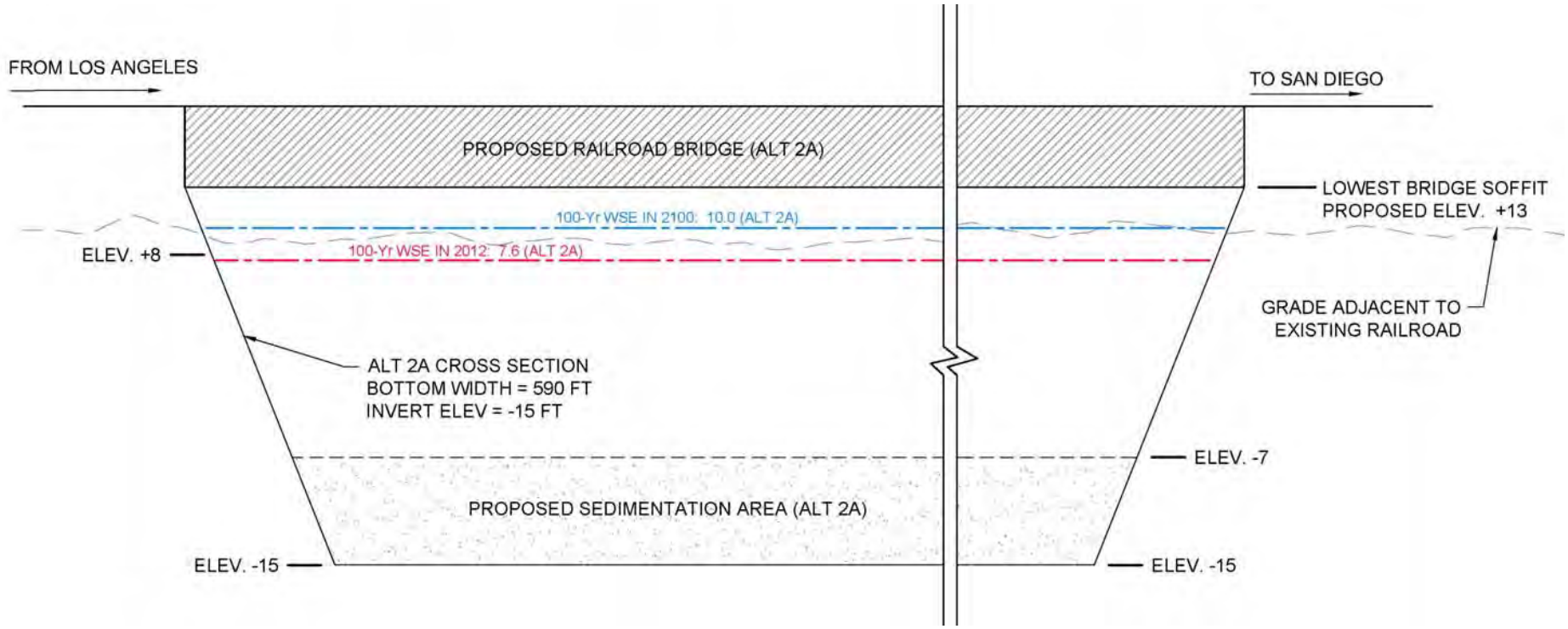

Figure 8-4: Proposed Alt 2A Railroad Bridge Channel Cross-Section

8.3 I-5

The existing I-5 Bridge channel dimension is considered optimum for No Project and Alternative 1A conditions. The I-5 Bridge channel does not influence tidal range for these alternatives but it does influence flood conveyance. Regardless of the I-5 Bridge channel dimension, Manchester Avenue will experience flooding in the East Basin during a 100-year event. The existing I-5 Bridge channel dimension actually helps prevent additional flooding of Manchester Avenue in the Central Basin by attenuating peak flows in the East Basin. The result is higher flood elevations in the East Basin, but little or no flooding in the Central Basin. If the I-5 Bridge channel is widened, flood elevations are lowered in the East Basin, but raised in the Central Basin causing flooding of Manchester Avenue in both basins.

For Alternative 1B and 2A, a minimum I-5 Bridge channel dimension of 261 feet was identified as the optimum width. The increased channel width helps lower flood elevations in the East Basin. The magnitude of Manchester Ave flooding is significantly reduced for Alternative 1B and eliminated for Alternative 2A. A cross-section of the existing I-5 Bridge channel and the optimized channel widths for each alternative is shown in Figure 8-5. A conceptual cross-section of the lengthened I-5 Bridge structure is shown in Figure 8-6.

Table 8-3: I-5 Optimized Bridge Channel Dimensions

Alternative	Existing Condition		Optimized for Tides		Optimized for Flood Level	
	Channel Invert (ft, NGVD)	Bottom Width (ft)	Channel Invert (ft, NGVD)	Bottom Width (ft)	Channel Invert (ft, NGVD)	Bottom Width (ft)
No Project	-6.0	130	-6.0	130	-6.0	130
1A	-6.0	130	-6.0	130	-6.0	130
1B	-6.0	130	-6.0	130	-6.0	261
2A	N/A	N/A	-6.5	261	-6.5	261

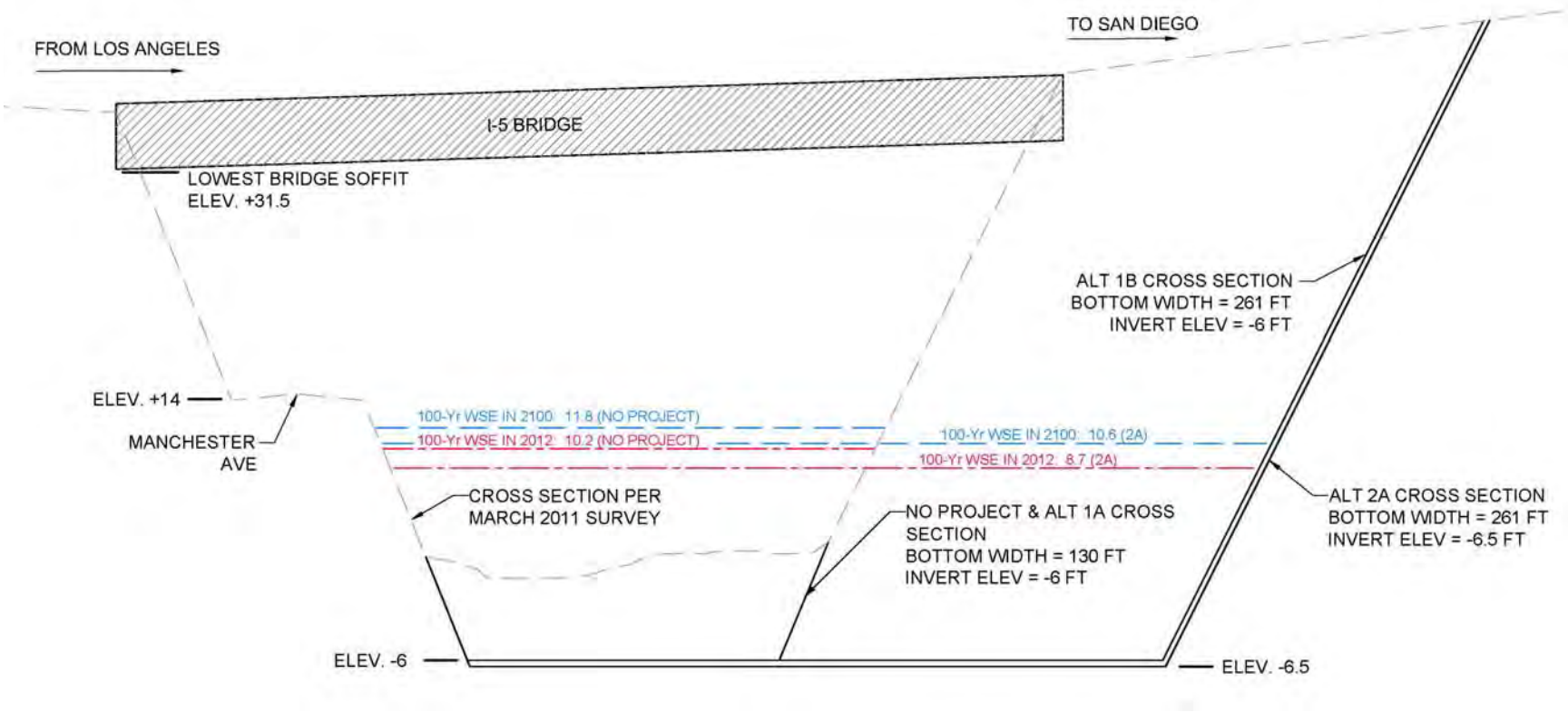

Figure 8-5: Existing I-5 Bridge Channel Cross-Section

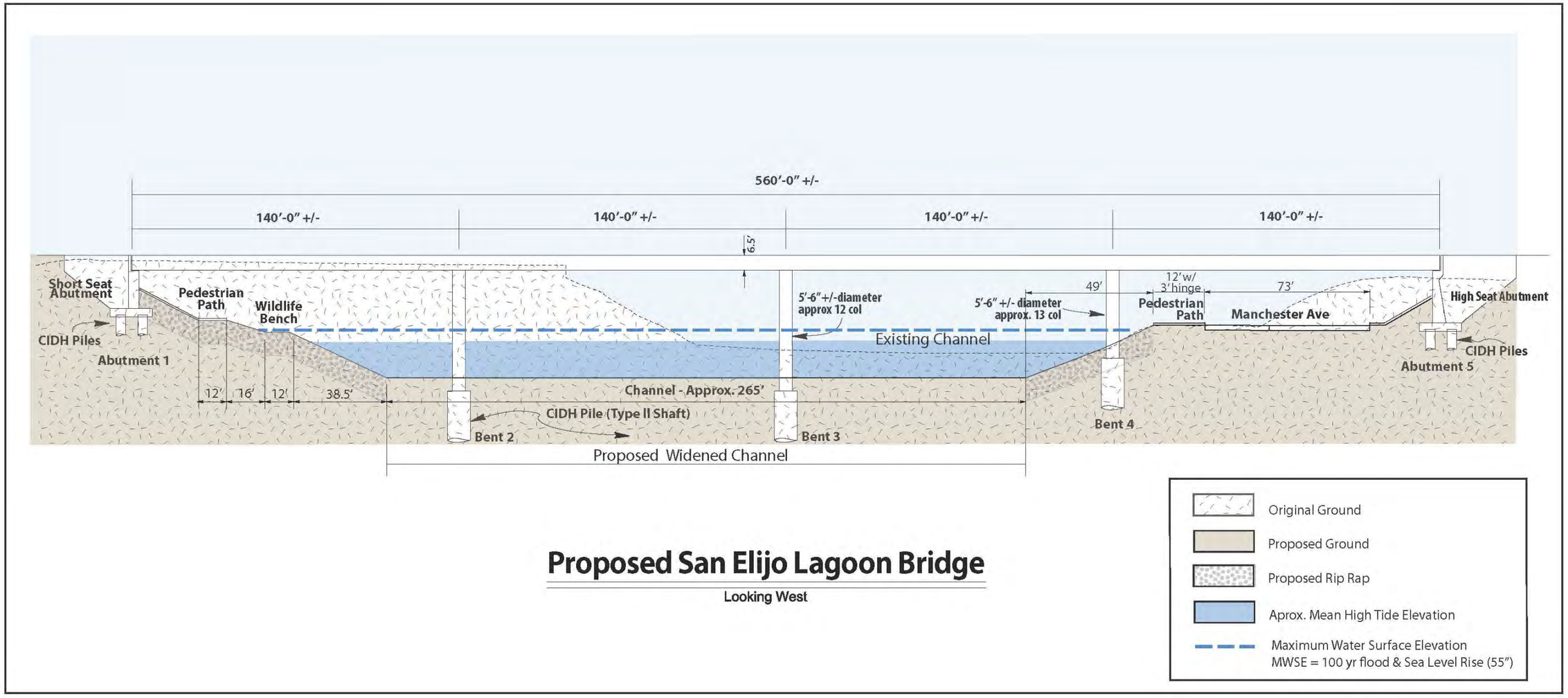

Figure 8-6: Proposed I-5 Bridge Channel Cross-Section

9.0 ANALYSES OF VELOCITY AND SEDIMENTATION

The section summarizes velocities under infrastructure crossings under dry weather conditions and also under extreme wet weather conditions (100-year stormflow event). Sedimentation patterns are also evaluated for each alternative based on flow velocities during the 100-year flood simulation.

9.1 Analyses of Tidal Velocities Under Bridges

The peak tidal velocities at the infrastructure crossings under each alternative are listed for the optimized bridge dimensions in Table 9-1. At the lagoon inlet, below Hwy 101, tidal velocity provides an indicator for inlet stability. Ebb tidal flow velocity at Hwy 101 was lowest for the No Project alternative due to the smaller tidal prism limited by the existing bathymetry of the Lagoon's main channel and inlet channel. The results indicate that ebb velocities are higher for Alternatives 1A and 1B due to the increase in tidal prism associated with these alternatives. In comparison to No Project conditions the higher ebb velocities for Alternatives 1A and 1B will help maintain a wider channel and reduce the frequency of maintenance dredging.

Ebb and flood velocities under the Railroad Bridge slightly increase for Alternatives 1A and 1B due to the increase in tidal prism when compared to No Project conditions. However, the small increases in velocity are probably not enough to influence the sediment transport patterns in this location. Alternative 2A proposes a wide and deep sedimentation basin below the Railroad Bridge to intentionally lower velocity and facilitate deposition.

Tidal velocities below the I-5 Bridge are higher for alternatives that increase the tidal prism in the East Basin, especially for Alternative 2A. Currently, there is little or no tidal exchange under the I-5 Bridge. Each project alternative proposes to increase the tidal prism of the East Basin and this will likely influence sedimentation patterns. There is a potential for areas of local scour and deposition to develop due to the daily fluctuation of tidal flow in and out the East Basin. The magnitude of these sedimentation patterns will be greater for Alternative 2A due to the increase in tidal prism and velocity, however they may be minor due to relatively low tidal flow velocities.

Table 9-1: Tidal Velocity (fps) at Bridge Crossings During the Dry Season

		Hwy 101		Railroad		I - 5	
Alternative	Bridge Dimensions	Flood	Ebb	Flood	Ebb	Flood	Ebb
No Project	Optimized	1.6	-3.0	1.0	-1.0	0.1	-0.1
1A	Optimized	1.5	-5.7	1.4	-1.6	0.3	-0.4
1B	Optimized	2.0	-6.2	2.0	-1.9	0.4	-0.3
2A	Optimized	3.9	-5.0	0.6	-0.6	0.9	-0.7

9.2 Analyses of Extreme Flood Velocities Under Bridges

Table 9-2 summarizes the velocities at infrastructure crossings under the 100-year storm event. The modeling runs were performed to determine the maximum water surface elevations for flood protection and therefore do not reflect the potential maximum velocities that may occur at each bridge crossing. The maximum water surface elevation occurs at the high tidal elevation

and the maximum velocity occurs at low tide. However, the 100-year flood velocities do provide a relative comparison between each alternative.

The peak velocity under Hwy 101 increases to nearly 14 feet per second (fps) for Alternatives 1A and 1B and to nearly 10 fps for Alternative 2A. This occurs because there is less attenuation throughout the lagoon for these alternatives and the inlet channel is dredged to the maximum width and depth. These high velocities will cause significant scour but the presence of hard bottom reef and bedrock should limit the scour depth below the existing inlet. A more detailed study of potential maximum scour depths will be necessary for design of the bridge structure and inlet channel for Alternative 2A.

The peak velocity under the railroad bridge is relatively low for each alternative and there is no significant scour expected. Peak velocities below the I-5 Bridge are slightly lower for Alternatives 1B and 2A but are in the same range as other alternatives. Further analysis of maximum velocity at I-5 would be necessary to estimate the potential scour depths and determine if there is a need for scour protection.

Table 9-2: Peak Velocity (fps) at Bridge Crossings During a 100-Year Storm

Alternative	Bridge Dimensions	Hwy 101	Railroad	I - 5
No Project	Optimized	-6.6	-2.5	-6.1
1A	Optimized	-13.8	-4.0	-6.6
1B	Optimized	-13.8	-3.6	-4.5
2A	Optimized	-9.6	-2.0	-5.6

9.3 Analyses of Sedimentation During the 100-Year Flood

San Elijo Lagoon is 303d listed for fine sediments. The optimized models for each alternative were analyzed to determine the impacts to sedimentation throughout the lagoon. Analyses were based on estimating flow velocities through the lagoon and the resulting sediment transport and patterns of deposition during stormflows. As sediments are mainly transported during the highest velocity condition, peak velocity distributions during the storm were plotted over the entire lagoon for sedimentation analyses. A threshold flow velocity of 0.6 feet per second (fps) was used to represent the velocity required to maintain sediment transport of sand-sized material and unconsolidated clay and silt materials (Hjulstrom 1935). Velocities below this threshold may result in sedimentation of sand sized material. Velocities below 0.3 fps may result in sedimentation of finer materials such as unconsolidated clay and silt. Peak velocity distributions during the 100-year event simulation are plotted in the following figures for each alternative. For sedimentation analyses of a more frequent storm event please refer to the SELRP Water Quality Study (M&N 2010[b]).

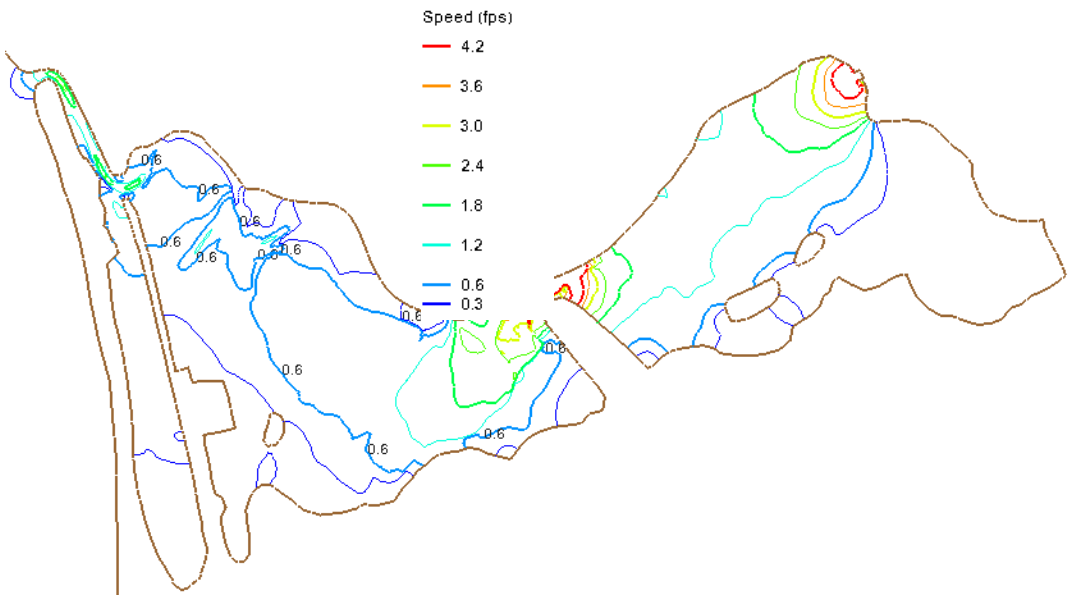

Figure 9-1: Peak Velocity Distribution During a 100-Year Event for No Project

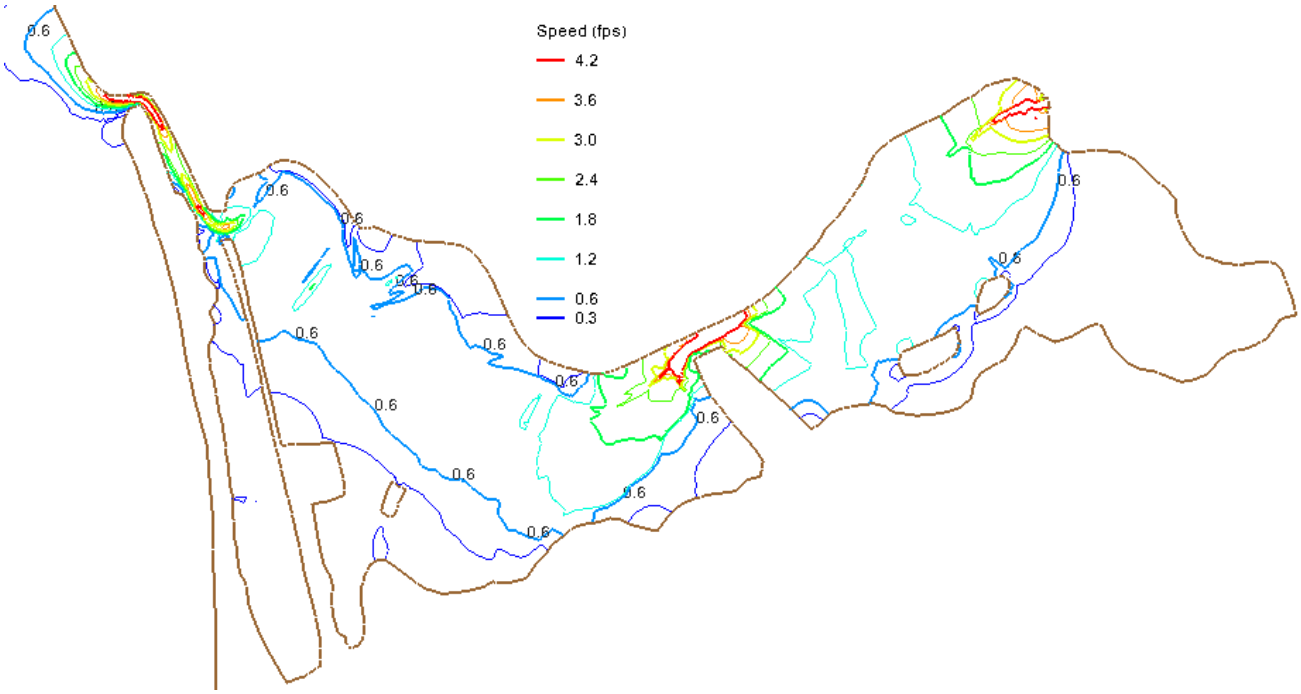

Figure 9-2: Peak Velocity Distribution During a 100-Year Event for Alternative 1A

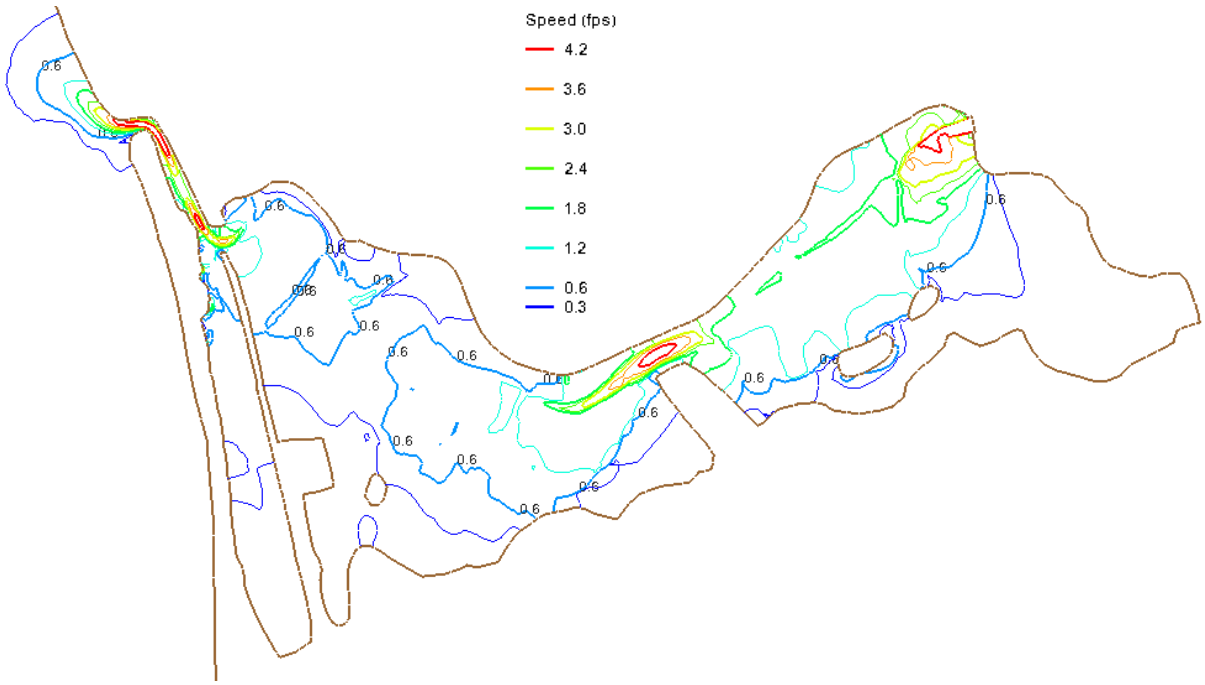

Figure 9-3: Peak Velocity Distribution During a 100-Year Event for Alternative 1B

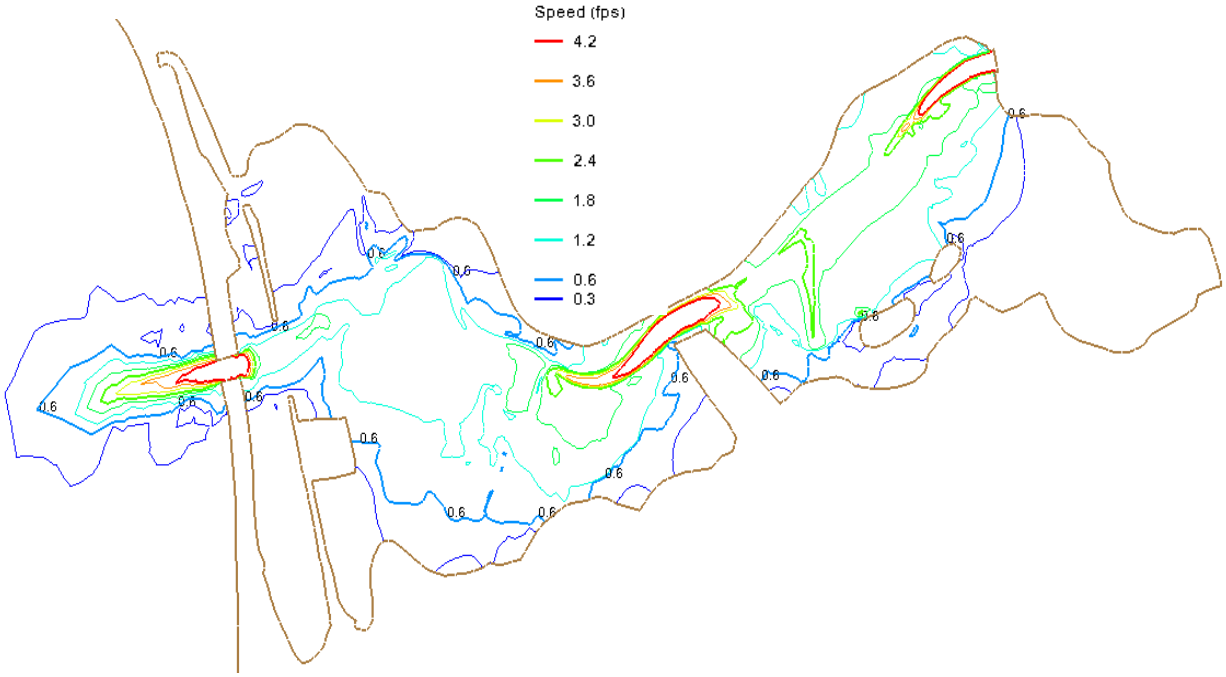

Figure 9-4: Peak Velocity Distribution During a 100-Year Event for Alternative 2A

Figure 9-1 and Figure 9-2 illustrate flow velocities in the San Elijo Lagoon for No Project and Alternative 1A conditions respectively. The results are similar and suggest velocities along the main flow path remain above 0.6 fps and will transport most of the fine grain sediment to the ocean during a 100-year event. Areas outside of the main flow path will likely experience some

sedimentation during a flood event of this magnitude. The results indicate that Alternative 1A will maintain a velocity of 0.6 fps over a larger portion of the lagoon and will probably experience less sedimentation than under No Project conditions.

Figure 9-3 demonstrates the velocity distribution pattern near the peak of the 100-year flood under Alternative 1B conditions. Alternative 1B proposes a much larger main channel and large subtidal area in the Central Basin while relying on the existing inlet to relieve flooding in the lagoon. Velocity patterns are similar to Alternative 1A for the East Basin and most of the Central Basin. However, at the location of the new subtidal basin in the Central Basin, velocities are below 0.6 fps for the duration of the 100-year flood. This is due to the large increase in cross-sectional area and corresponding drop in flow velocity across the subtidal basin. There will likely be some deposition of sand-sized particles in this area, but most of the fine material (silts and clays) will be transported to the ocean.

For Alternative 2A, a new tidal inlet is proposed that is wider and deeper than the existing inlet. The flow path to the new inlet is also shorter and its planform is straighter compared with alternatives using the existing inlet, thereby providing for improved downstream hydraulics. Flood drainage from the lagoon occurs much more rapidly with the new inlet than with the existing inlet due to significantly improved hydraulics, and less time is available for sedimentation. This is illustrated in Figure 9-4 by higher velocities throughout the lagoon near the peak of the 100-year flood simulation. There may be some sedimentation of finer materials near the edges of the lagoon, but the majority of the sediment will be transported through the lagoon to the ocean during a 100-year event.

The duration of the 100-year flood wave for each alternative are presented in Table 9-3. The timing of the 100-year hydrograph through the lagoon is very similar for No Project and Alternative 1A. In each case the flood duration is longer in the East Basin than the Central Basin . The primary reason is that the peak of the 100-year hydrograph coincides with the peak spring tide elevation in the East Basin to model a worst case scenario for flooding of Manchester Avenue. The second reason is that the existing channel width under the I-5 Bridge of 130 feet creates a backwater effect and helps attenuate the flood wave.

Table 9-3: Duration (Hours) of Stormflow Drainage Under a 100-Year Storm

Alternative	Bridge Dimensions	East Basin	Central Basin
No Project	Optimized	1.1	0.3
1A	Optimized	1.1	0.3
1B	Optimized	0.8	0.1
2A	Optimized	0.3	0.1

The flood duration for Alternative 1B is similar to that for No Project and Alternative 1A. Since the 100-year hydrograph and the peak high tide coincide in the East Basin, there is a significantly longer duration for the flood wave to pass through the East Basin. This represents a worst case scenario for Manchester Avenue. Once the flood wave passes through the East Basin, the tide has receded and there is less resistance to the flood flows as they pass through the Central Basin. The flood duration for Alternative 2A demonstrates the effectiveness of the new inlet as the duration of the flood is significantly less through the entire lagoon compared to Alternatives which rely on the existing inlet.

10.0 ANALYSIS OF RESIDENCE TIME

Residence time (i.e. average time a particle resides in a hydraulic system) provides a useful measure of the rate at which waters in the hydraulic system are renewed. Accordingly, residence time provides an indirect means for assessing the water quality of the hydraulic system, assuming contaminant inputs are held constant.

10.1 Methodology

Changes in constituent concentrations in a water body reflect a balance between the rate of constituent supply and the rate of constituent removal by tidal flushing. Residence time (i.e., average time a particle resides in a hydraulic system) provides a useful measure of the rate at which waters in the hydraulic system are renewed. Accordingly, residence time provides a means for assessing the water quality of the hydraulic system.

Consider the reduction of a tracer concentration in a tidal embayment due to flushing after being released (Fisher et al. 1979), in which C_0 is initial concentration, K is a reduction coefficient and C(t) is the concentration at time t.

$$C(t) = C_0 e^{-Kt} \tag{10.1}$$

The residence time of the tracer in the embayment is determined from

$$T_r = \frac{\int_0^\infty t\, C(t)\, dt}{\int_0^\infty C(t)\, dt} = \frac{1}{K}\,. \tag{10.2}$$

Since the concentration at t = T_r is

$$C(T_r) = C_0 e^{-1} = \frac{C_0}{e} \tag{10.3}$$

T_r can be calculated from a regression analysis of the tracer concentration time series computed by the numerical model RMA-4.

Based on the above methodology, the general procedure of computing the residence times for different parts of a tidal embayment is as follows:

- Assign an initial constituent concentration of one over the entire embayment element mesh (wetlands for this study) and a value of zero at the open water boundaries to simulate an instantaneous release of a new constituent into an embayment.
- Run the numerical model RMA-4 for an adequate number of tidal cycles until substantial reduction of constituent concentrations have occurred due to tidal flushing at the locations of interest.
- Analyze the computed concentration results by regression analysis to obtain the constituent reduction distributions at the locations of interest.
- Find the residence times for the locations of interest from the distribution curves according to Equations 10.1 through 10.3.

Figure 10-1 shows an example of how the method works, where the zigzag solid blue line shows the direct results from RMA-4 and the green dash line shows the daily moving average results. Arrows show the path of finding the residence time, which is approximately 173 hours for this case. This method was used in the project study for all scenarios.

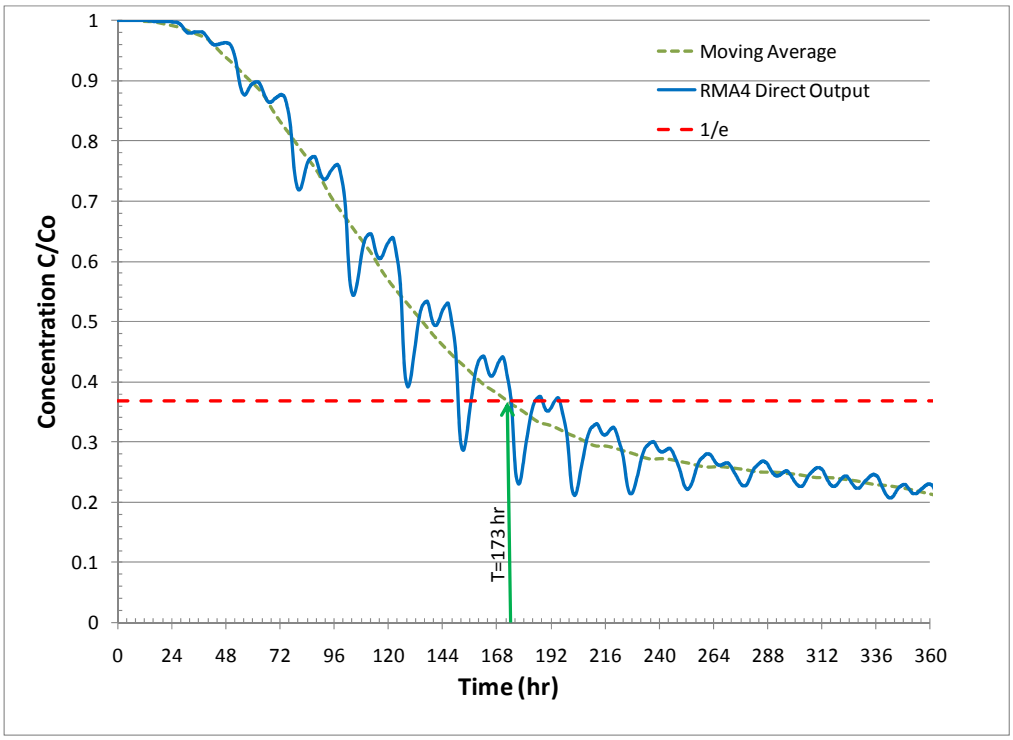

Figure 10-1: Example of a Residence Time Plot

10.2 Boundary Conditions

10.2.1 Hydraulic Input

The 15-day modeling tidal series, representing the average spring and neap tidal cycle, as described in Section 4.5.2 was applied as the offshore driving tide. No runoff from the fresh water boundary was considered, as the base flow of the creek is negligibly small.

10.2.2 Concentration Input

An initial constituent concentration of one was specified for the entire lagoon. No constituent concentration was assigned at the open water boundaries. Also, it is assumed that ocean water is clean and does not supply additional constituents, or “contaminants.”

10.3 Residence Time Results

RMA-4 water quality models were used to predict residence times in the lagoon for each alternative. Table 10-1 lists the predicted range of residence times for each basin and each proposed project condition. The results indicate that dredging of the main channel in each basin

will significantly shorten residence time and result in more frequent exchange of clean water into the basins.

There is no residence time reported in the East Basin for No Project conditions because the tidal inundation does not extend east of the I-5 Bridge. The residence times in the Central and West basins significantly increase in proportion to the distance from the inlet channel indicating poor tidal exchange throughout each basin.

Alternative 1A includes a widened main channel throughout the lagoon which helps reduce residence times. However, residence times in the East Basin are still greater than 12 days and parts of the Central Basin are greater than 7 days.

Alternative 1B includes additional subtidal areas and feeder channels throughout the Central and East Basins. These features reduce residence times and improve water circulation to the remote areas of the lagoon. Residence times in the East Basin are less than 8 days showing a significant improvement compared to Alternative 1A.

Alternative 2A also includes new subtidal areas and feeder channels throughout the Central and East Basins in addition to a new tidal inlet. This alternative provides improved residence times of less than 3 days in the Central Basin and about 4 days in the East Basin. By comparing Alternative 1B and 2A, the benefits of the new tidal inlet can be quantified. Both alternatives involve similar grading plans and the primary difference is the inlet location. The Alternative 2A inlet is positioned in a more central location allowing for better water circulation and reduced residence times for the entire lagoon.

Table 10-1: Summary of Residence Time (Days)

Alternative	Bridge Dimensions	West Basin (WB2)	Central Basin (CC4)	East Basin (EB4)
No Project	**Optimized**	0.7 – 16.8	0.8 – 14.5	N/A
1A	**Optimized**	1.2 - 4.6	1.2 – 9.2	9.2 - 12.7
1B	**Optimized**	1.1 - 3.8	1.1 – 5.6	5.6 - 7.6
2A	**Optimized**	0.6 - 1.1	0.7 – 2.7	2.7 - 4.3

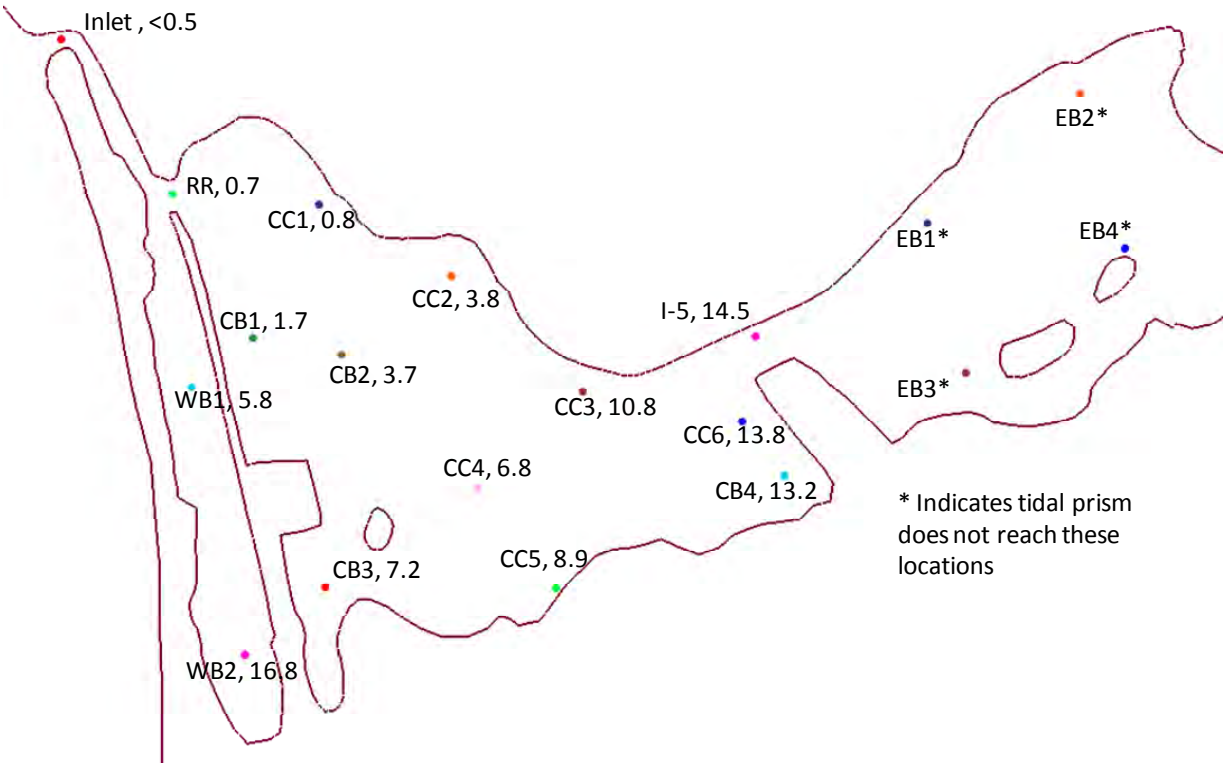

Figure 10-2: Residence Time (Days) for No Project Conditions

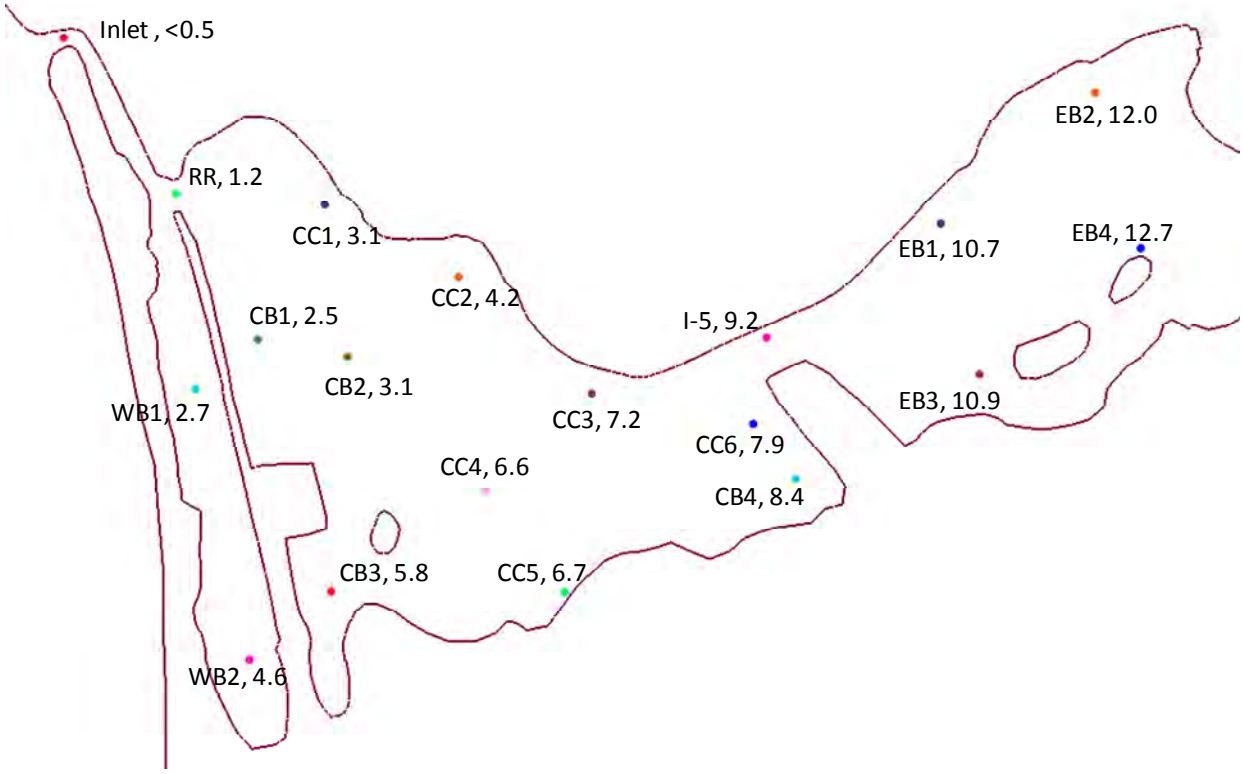

Figure 10-3: Residence Time (Days) for Alternative 1A

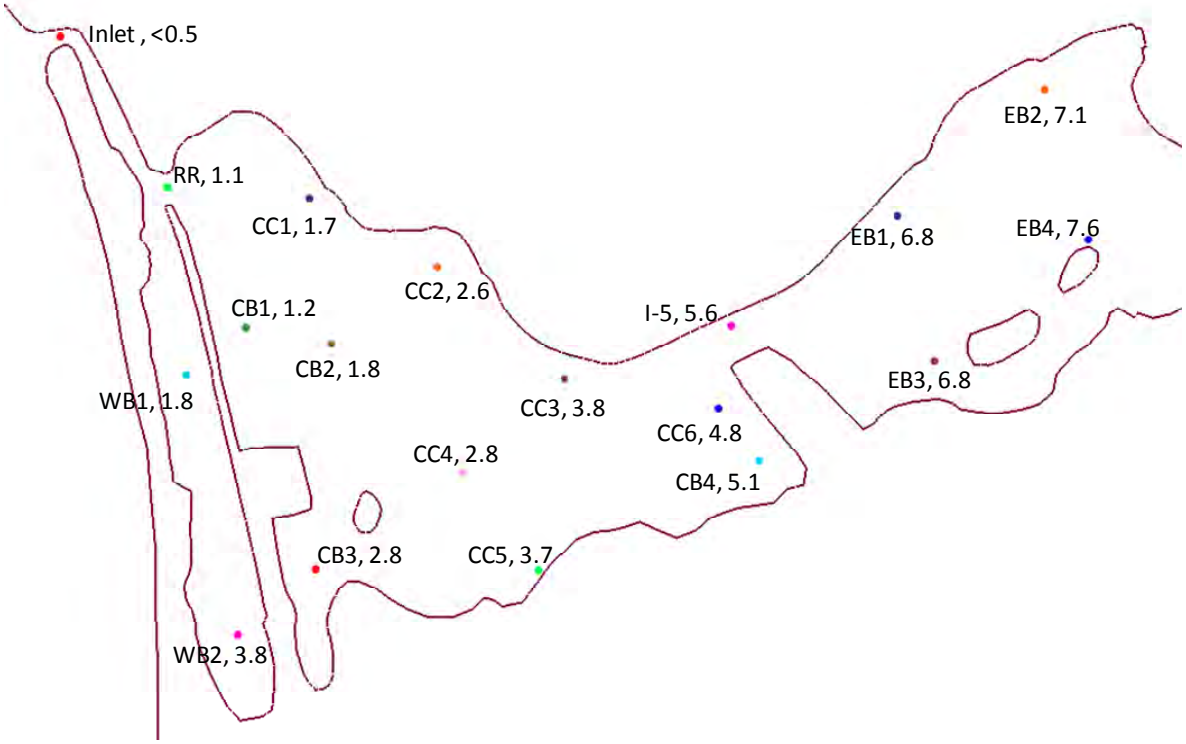

Figure 10-4: Residence Time (Days) for Alternative 1B

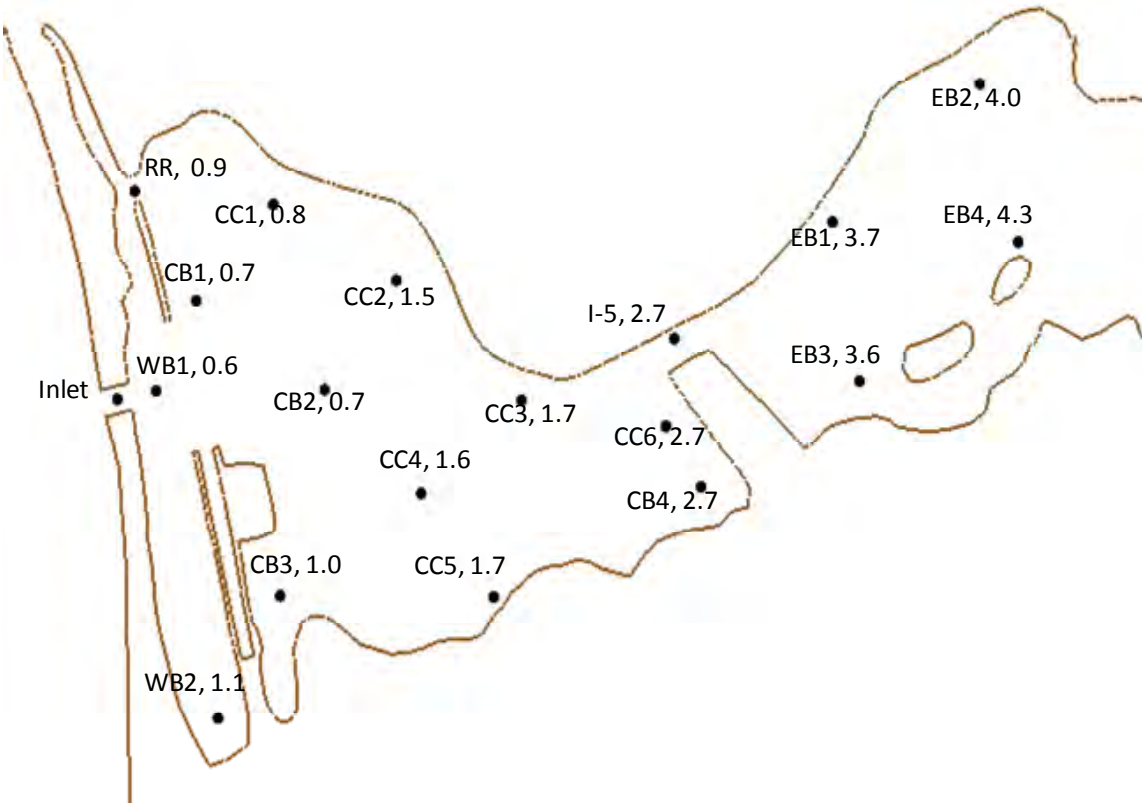

Figure 10-5: Residence Time (Days) for Alternative 2A

11.0 TIDAL INUNDATION FREQUENCY ANALYSIS

The inundation frequency was analyzed and plotted with tidal elevation data from the TEA tidal model runs. Figure 11-1 shows locations where the inundation frequency was analyzed and plotted. Figure 11-3 through Figure 11-6 show the inundation frequency plots for each project alternative. Curves that most closely approximate the ocean tidal curve reflect lagoon conditions that would sustain the broadest range of habitat in the vertical dimension. Curves that are truncated from the ocean tidal curve represent lagoon conditions with a narrower vertical range of habitats.

11.1 Existing Inlet Location Alternatives

For alternatives with the tidal inlet at its current location, both high and low tides are muted. Compared to the No Project conditions, Alternative 1A has a larger tidal range in the West Basin; therefore, it would increase the vertical range of intertidal habitat. The primary gain of the intertidal habitat will be the mudflat area and the mid to high marsh area.

Alternative 1B has a smaller tidal range compared to the No Project and Alternative 1A since Alternative 1B has the same narrow entrance channel, but a much larger tidal prism. However, compared to No Project and Alternative 1A, Alternative 1B will provide more intertidal habitat resulting from grading/dredging of the West, Central and East Basins.

11.2 New Inlet Location Alternative

Alternative 2A will provide almost a full tidal exchange with the ocean. With similar grading plans to Alternative 1B in the West, Central and East Basins, Alternative 2A will have a much larger tidal range and will support a much broader vertical range of intertidal habitat than that Alternative.

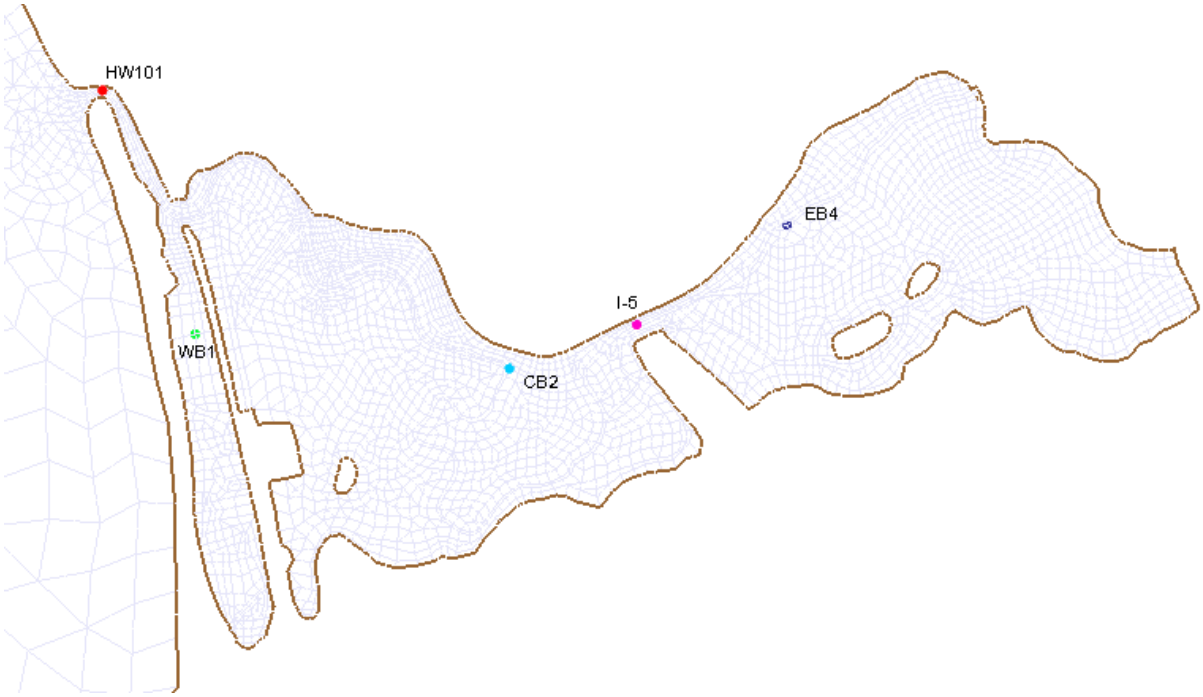

Figure 11-1: Gage Locations for Inundation Frequency Plots (No Project, Alt 1A & 1B)

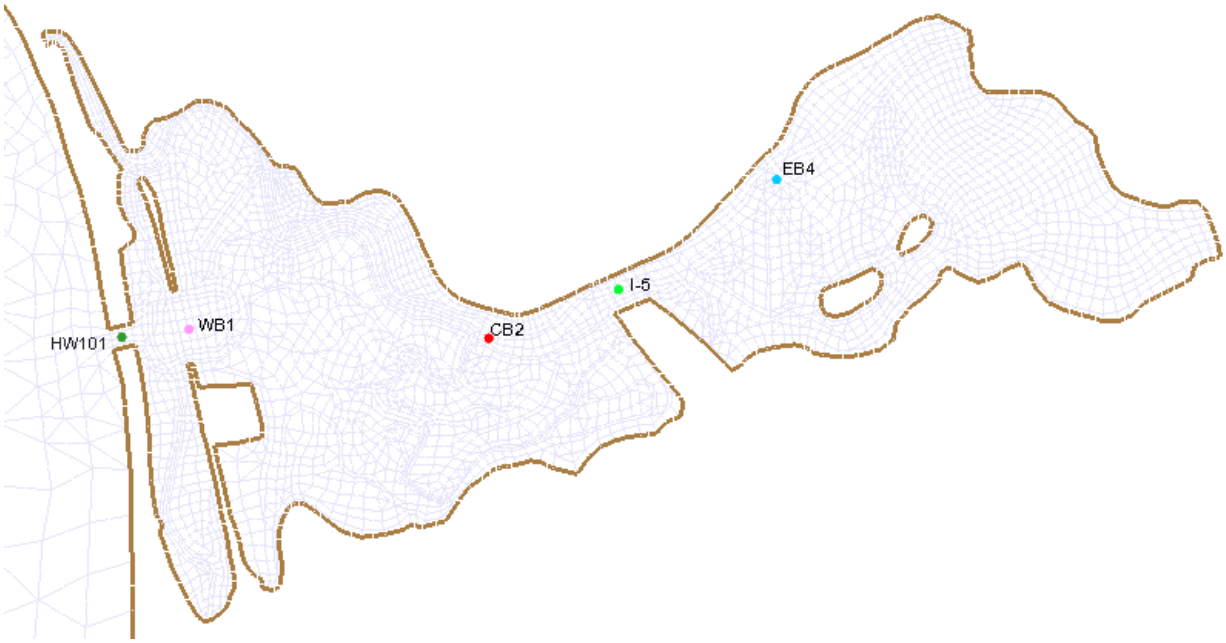

Figure 11-2: Gage Locations for Inundation Frequency Plots (Alt 2A)

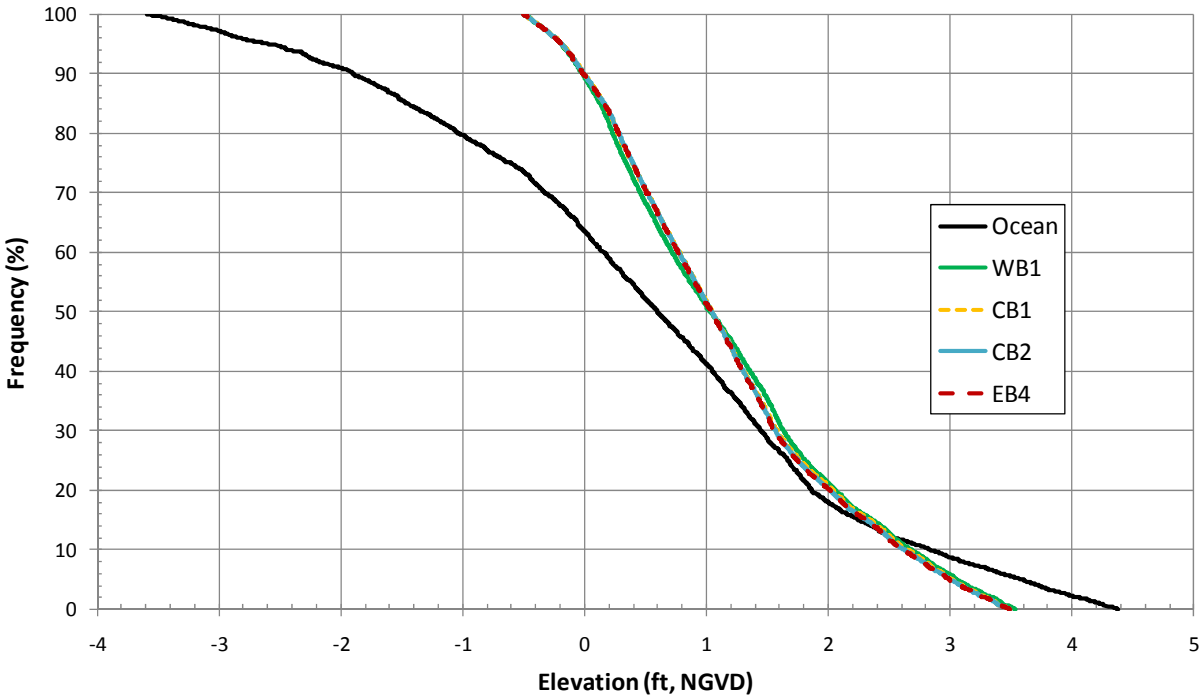

Figure 11-3: No Project Tidal Inundation Frequency

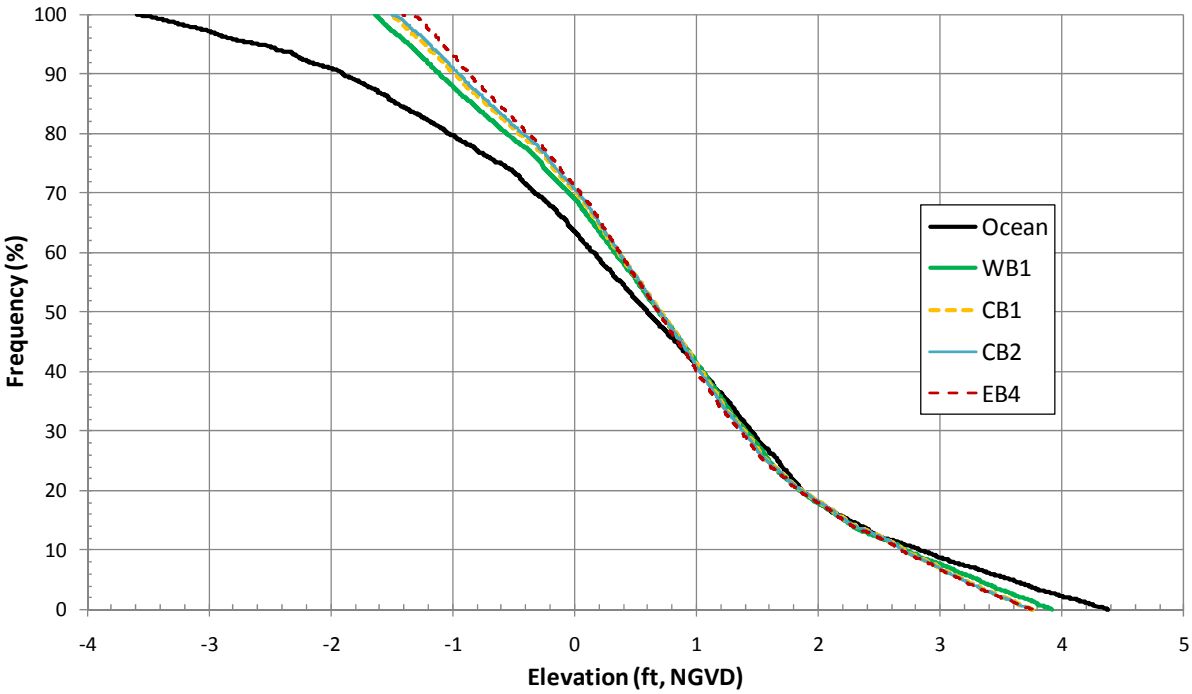

Figure 11-4: Alternative 1A Tidal Inundation Frequency

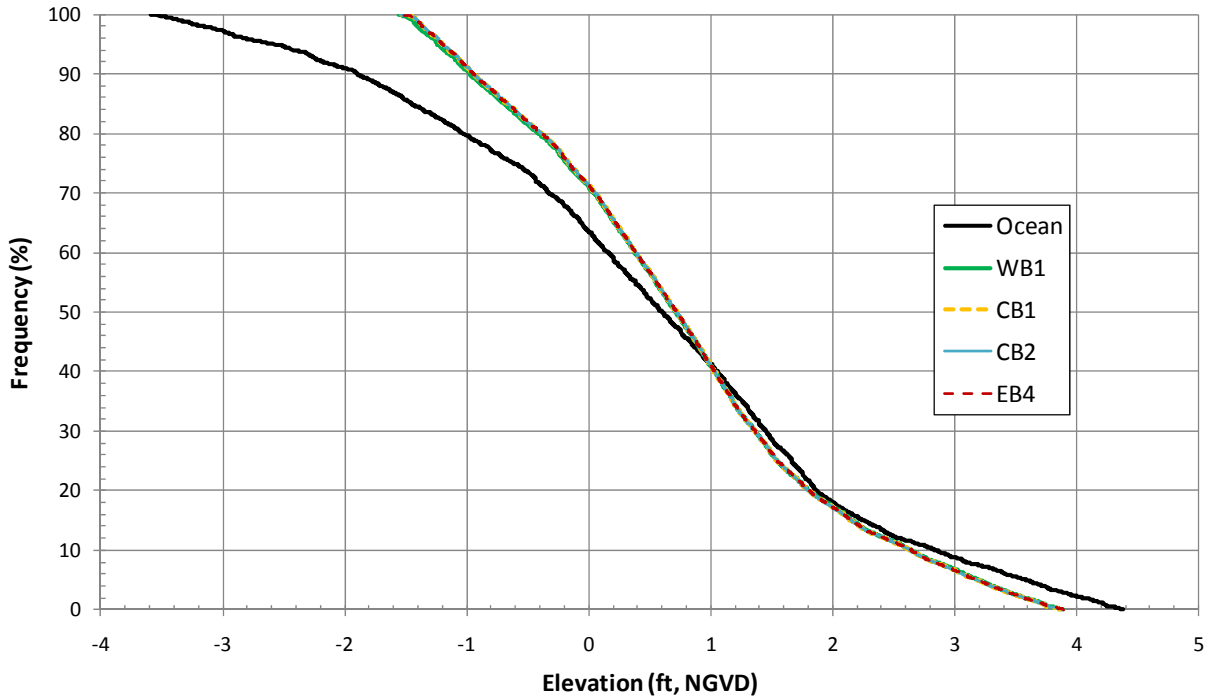

Figure 11-5: Alternative 1B Tidal Inundation Frequency

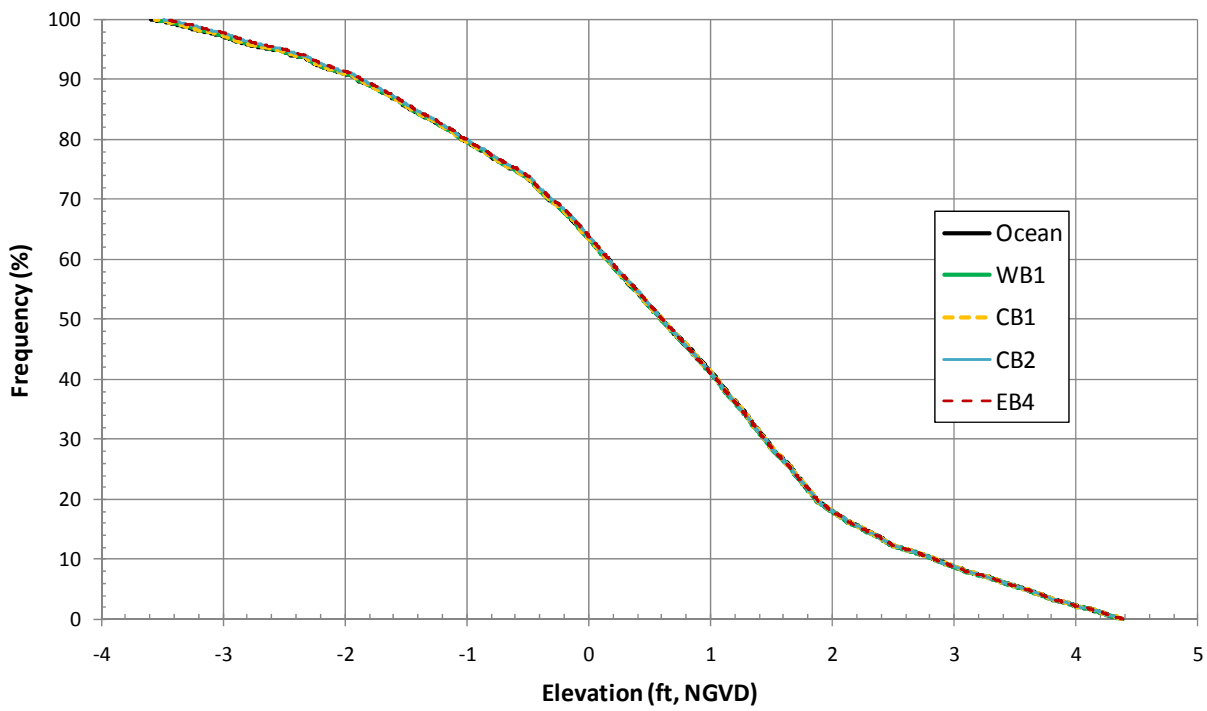

Figure 11-6: Alternative 2A Tidal Inundation Frequency

12.0 HYDRAULIC EFFECTS OF SEA LEVEL RISE

Hydrodynamic modeling runs were performed considering projections for SLR in the year 2100. A 55-inch SLR estimate was assumed for the modeling study based on the guidance provided by Caltrans internal guidance (Caltrans 2011) and the California State Coastal Conservancy on its web site (CSCC 2012) for horizon year 2100. The offshore spring high tidal series (with a high tide elevation of 4.69 feet NGVD) was raised linearly upward by 55 inches to form the spring high tide series in year 2100 (future high tide elevation of 9.27 feet NGVD). It was assumed that the 100-year storm condition will be the same as it is today, with no changes accounted for from possible climate change.

The offshore high tide base level of 4.69 feet used for modeling of SLR compares to a base level of 7.0 feet used for stormflow modeling under existing conditions. The ocean base level for SLR modeling is therefore different, and 28 inches lower, than that assumed for existing conditions stormflow modeling. The difference is the omission of the value of wave run-up from the SLR modeling base level. The reason that wave run-up is not included is that breaking waves should not exist at the Lagoon mouth during combined maximum high tide and SLR conditions. Water depths at the Lagoon mouth are estimated to be sufficient to preclude wave breaking within the tidal inlet channel.

Figure 12-1 through Figure 12-4 compare the 100-year water surface profiles today and in the year 2100 for each project alternative. The results indicate the flood elevations through the Central Basin will rise by approximately 2 feet.

12.1 Water Surface Elevations for Existing Inlet Location Alternatives

For No Project, Alternatives 1A and 1B, the Central Basin flood elevations will increase from about +10 feet, NGVD to +12 feet, NGVD and nearly 2,000 linear feet of Manchester Avenue will become flooded due to the projected SLR.

12.2 Water Surface Elevations for the New Inlet Location Alternative

For Alternative 2A, Central Basin flood elevations will increase from about +8 feet, NGVD to +10 feet, NGVD and Manchester Avenue may experience shallow flooding at low points in the roadway profile. Alternative 2A flood elevations increase sufficiently from the projected 2100 SLR to inundate nearly 2,000 linear feet of Manchester Avenue in the East Basin.

12.3 Peak Flow Velocities for All Alternatives

The peak velocities for a 100-year flood and projected 2100 SLR, shown in Table 12-1, will be slightly lower compared to the current 100-year flood velocities due to the increased cross-sectional flow area below each bridge.

Table 12-1: 100-Year Peak Flood Velocity (fps) at Bridge Crossings in Year 2100

Alternative	Bridge Dimensions	Hwy 101	Railroad	I - 5
No Project	Optimized	-5.9	-2.3	-5.4
1A	Optimized	-11.4	-4.0	-6.1
1B	Optimized	-8.8	-3.5	-4.1
2A	Optimized	-8.8	-1.9	-5.2

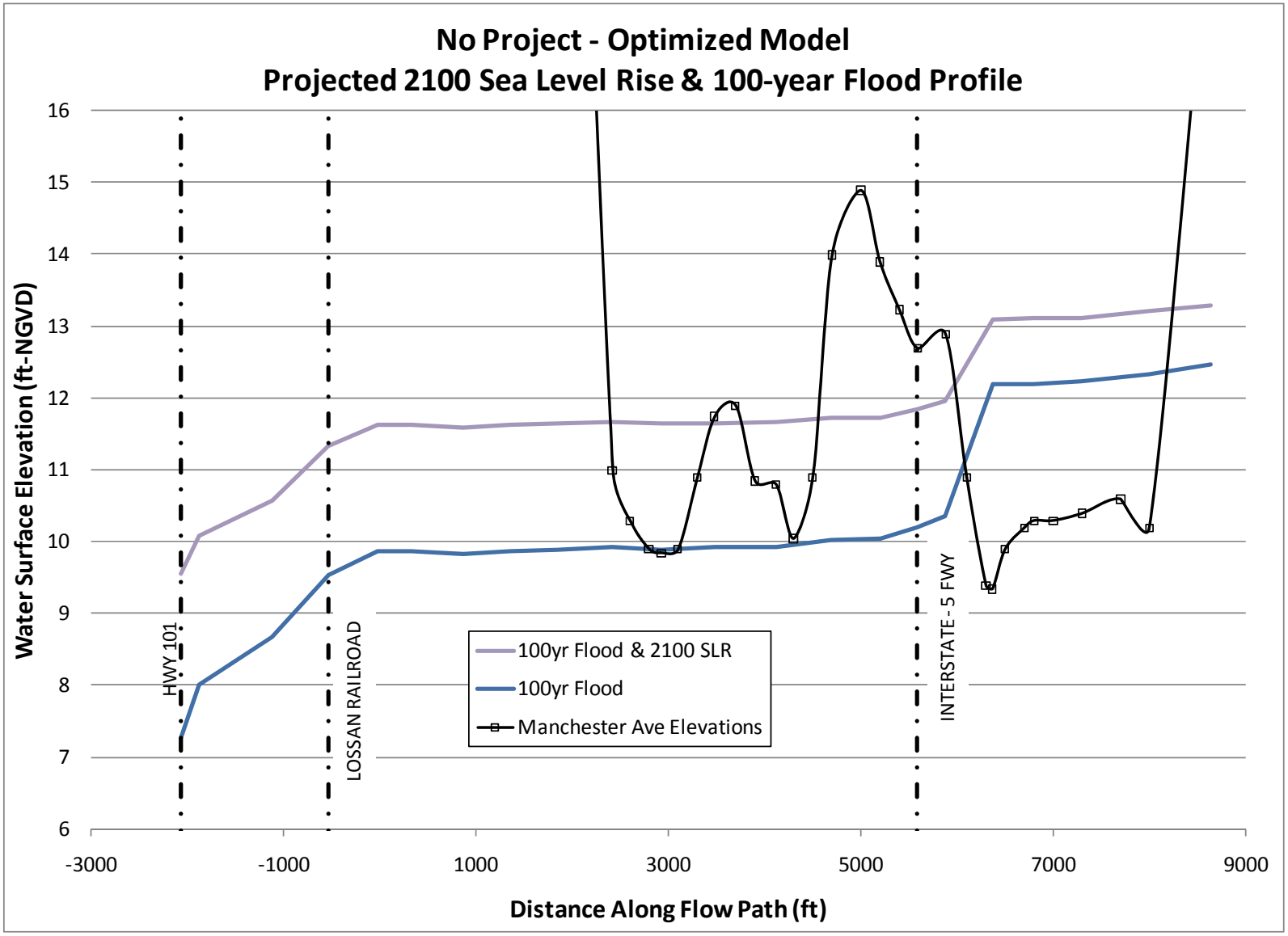

Figure 12-1: No Project 100-Year Flood Profile Comparison With Sea Level Rise

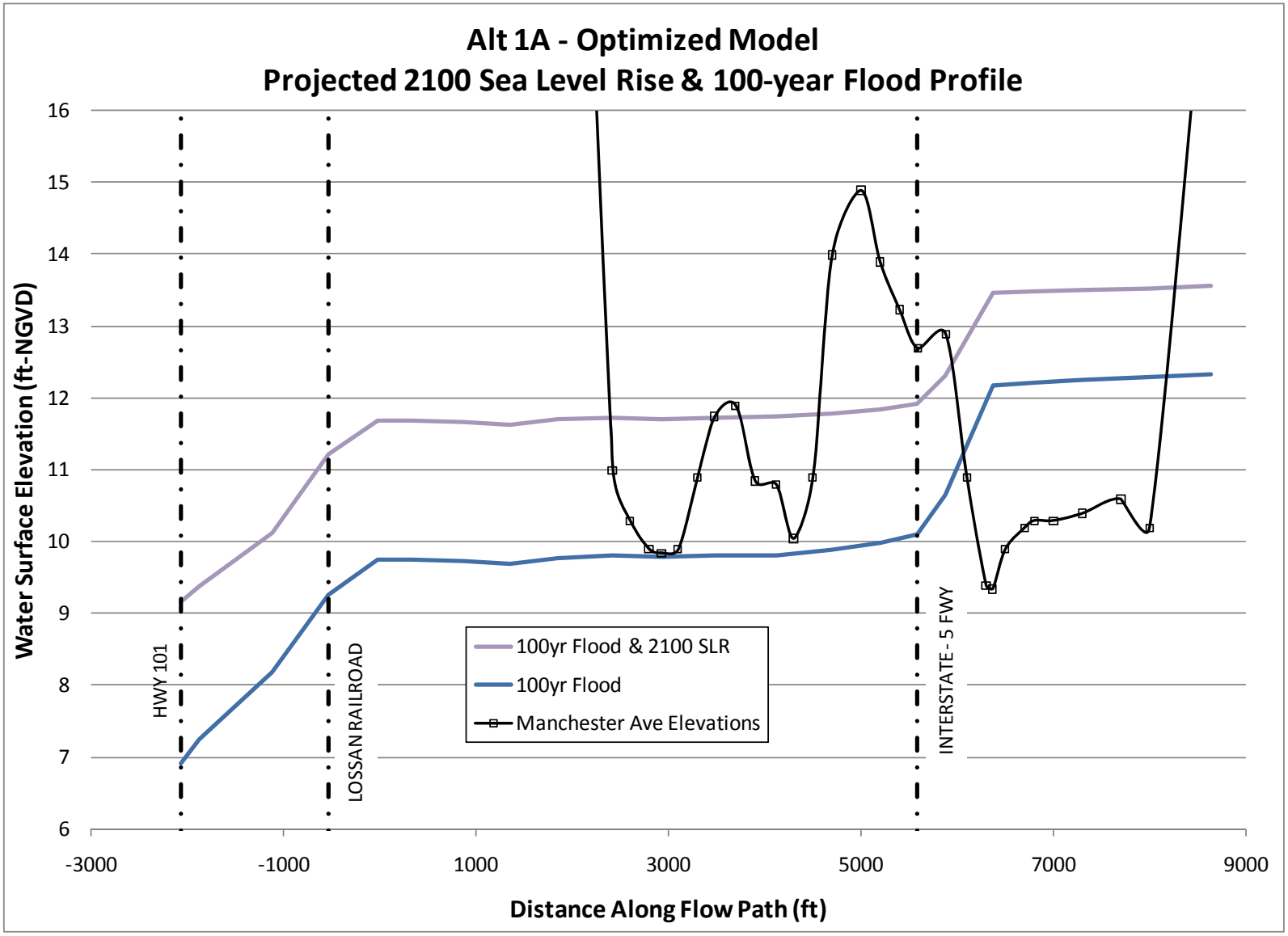

Figure 12-2: Alternative 1A 100-Year Flood Profile Comparison With Sea Level Rise

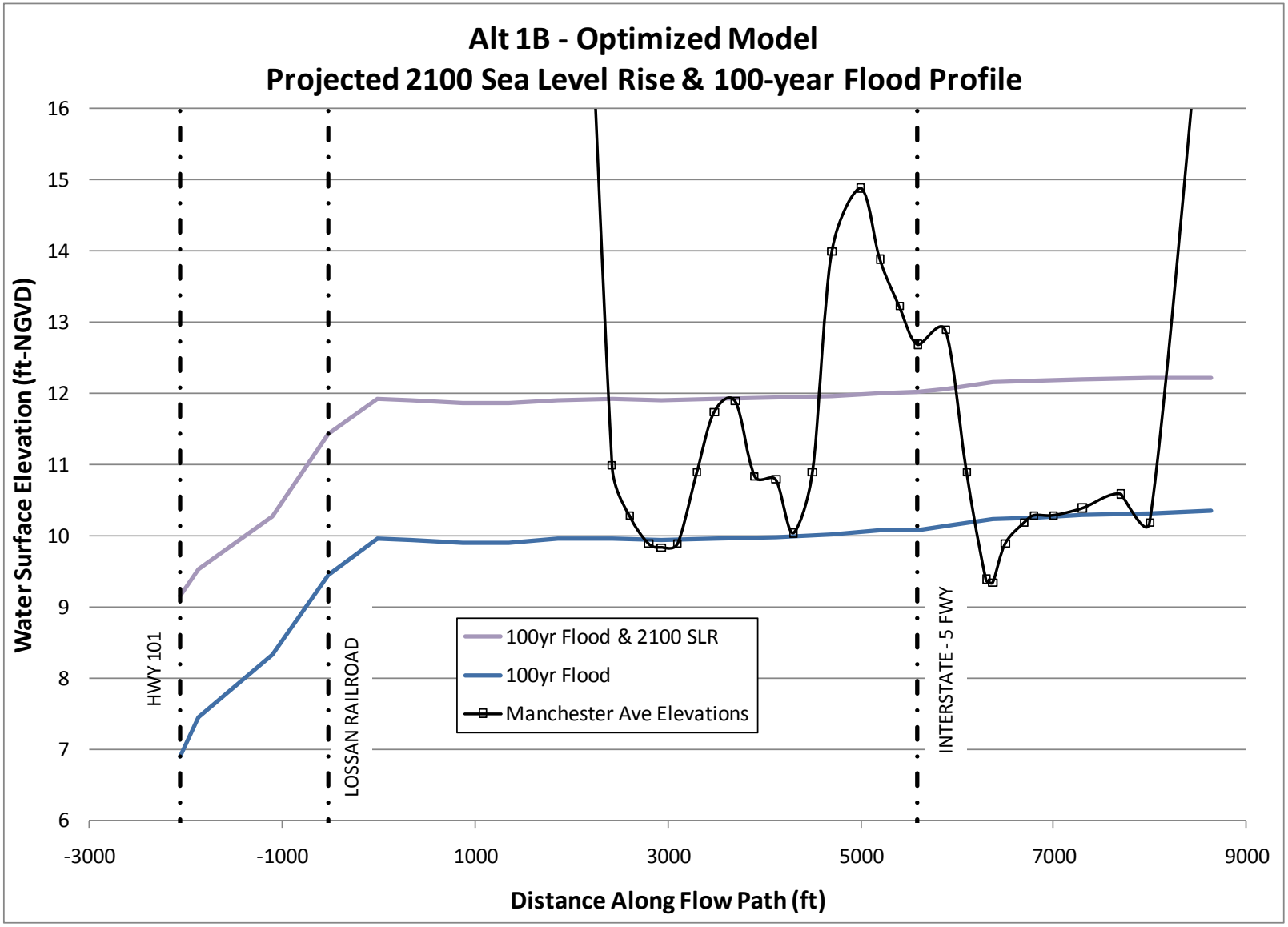

Figure 12-3: Alternative 1B 100-Year Flood Profile Comparison With Sea Level Rise

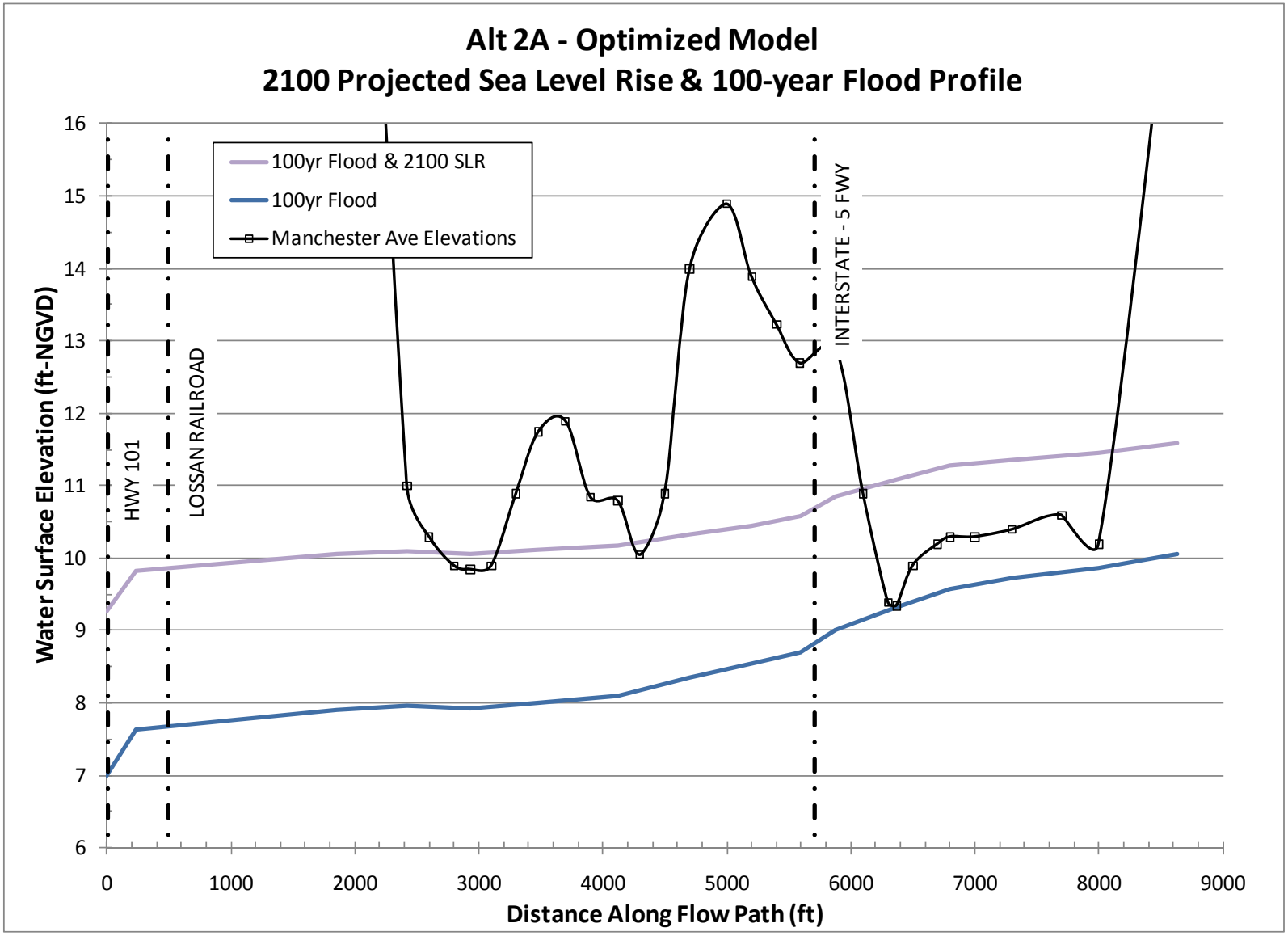

Figure 12-4: Alternative 2A 100-Year Flood Profile Comparison With Sea Level Rise

12.4 Bridge Freeboards During High Water Conditions

Table 12-2 presents the maximum predicted flood elevations at each bridge crossing for the 100-year flood, and the 100-year flood coupled with projected SLR in the year 2100. There is sufficient freeboard above the maximum flood elevations for the I-5 and Railroad Bridges under both current and future SLR scenarios. The existing Hwy 101 Bridge has about 3 feet of freeboard above the 100-year maximum water levels in 2012. In the year 2100 with sea level rise taken into account, freeboard is very limited under the existing Hwy 101 Bridge.

The maximum flood elevations at the existing Hwy 101 Bridge are sensitive to timing of the flood hydrograph and peak high tide. The worst case flood elevation occurs when the peak of the flood wave coincides with the peak tide at a specific location. An iterative modeling analysis was performed to determine maximum 100-year flood elevations in the year 2100 for each alternative. The results suggest little or no freeboard would be available under Hwy 101 for No Project conditions in the year 2100. About half a foot of freeboard would be available for Alternatives 1A and 1B. Alternative 2A has maximum flood elevations similar to that of Alternative 1A and 1B except that a new inlet is proposed for 2A and will likely include additional freeboard.

This iterative modeling analysis was not performed for year 2012 but onecan surmise from the 2100 analysis that worst case flood timing will result in a 0.1 to 0.3 feet increase above the maximum ocean tide elevation. When applied to the predicted flood elevations at Hwy 101 in the current year these results suggest there may be slightly less than 3 feet of freeboard available below the existing Hwy 101 Bridge for No Project and Alternatives 1A and 1B.

Table 12-2: Summary of Bridge Soffit and 100-Year Flood Elevations

Bridge:	**Hwy 101 Bridge**		**Railroad Bridge**		**I-5 Bridge**	
Soffit Elevation:	**+10 (ft, NGVD)[a]**		**+15.6 (ft, NGVD)[b]**		**+31.5 (ft, NGVD)[c]**	
Alternative	Year 2012	Year 2100	Year 2012	Year 2100	Year 2012	Year 2100
No Project	7.3	9.9	9.5	11.3	10.2	11.8
1A	7.0	9.4	9.3	11.2	10.1	11.9
1B	7.0	9.4	9.5	11.4	10.1	12
2A	7.0	9.4	7.6	10.0	8.7	10.6

Notes: a) Soffit elevation from Hwy 101 Bridge as-built drawings (NGVD datum assumed)
b) Soffit elevation from 2007 PDC survey, provided by HDR
c) Soffit elevation from I-5 Bridge as-built drawings (NGVD datum assumed)

13.0 FINDINGS AND RECOMMENDATIONS

The selection of optimum channel widths (for bridge lengths) and channel depths were based on a sensitivity analysis conducted for each bridge crossing under typical dry weather tidal fluctuations and extreme storm conditions (100-year storm and 100-year combined water levels). Tidal range was used as the primary indicator for benefits to the wetland ecosystem, and extreme flood elevations were used to evaluate the potential for flooding of Manchester Avenue. Using these indicators, the optimum channel width and depth were identified as the point at which tidal range and flood conveyance are most favorable and further increases in channel width and depth result in only minimal benefit. For the case of Hwy 101, the presence of hard bottom reef and bedrock limits the channel depth to an elevation of -4 feet, NGVD. Table 13-1 presents the optimum channel widths and depths for each bridge and each SELRP alternative (inclusive of all bridge bents and piles).

Table 13-1: Summary of Optimized Bridge Dimensions for Each Alternative

Alternative	Hwy 101 Bridge		Railroad Bridge		I-5 Bridge	
	Bottom Width (ft)	Channel Invert (ft,NGVD)	Bottom Width (ft)	Channel Invert (ft,NGVD)	Bottom Width (ft)	Channel Invert (ft,NGVD)
No Project	105	-4	187	-5.5	130	-6
1A	115	-4	187	-5.5	130	-6
1B	130	-4	187	-5.5	261	-6
2A	200	-6.5	590	-15	261	-6.5

Key findings from the optimization modeling study are summarized below:

- For alternatives which rely on the existing inlet channel (No Project, Alternative 1A, and Alternative 1B), the existing Hwy 101 Bridge structure and the Railroad Bridge structure have sufficient spans and are not limiting factors for tidal range or flood conveyance. The limiting factor for these alternatives is the long and narrow inlet channel between Hwy 101 and the Railroad Bridge. The main channel through the Central Basin is also narrow, shallow, and sinuous in planform resulting in additional energy losses during normal tidal fluctuations and extreme flood events.
- There is no benefit to tidal flows and storm flow convenayce from increasing the existing I-5 Bridge channel dimension for No Project and Alternative 1A conditions. Regardless of the I-5 Bridge channel dimension, Manchester Avenue will experience flooding in the East Basin during a 100-year event. The existing I-5 Bridge channel dimension actually helps prevent additional flooding of Manchester Avenue in the Central Basin by attenuating peak flows in the East Basin. This attenuation results in higher flood levels

in the East Basin, but little or no flooding in the Central Basin. If the I-5 Bridge channel is widened, flood elevations are lowered in the East Basin, but raised in the Central Basin causing flooding of Manchester Avenue in both basins.

- Bridge optimization modeling of Alternative 1B suggested that increasing the I-5 Bridge channel width to 261 feet would relieve some flooding of Manchester Avenue in the East Basin. Portions of the roadway will still experience flooding, however, an increased bridge channel width would reduce flood levels below a significant length of roadway in the East Basin.

- For Alternative 2A, the optimization modeling study supported the recommended bridge channel dimensions identified in the SELRP Feasibility Studies (M&N 2010[a,b]). A Hwy 101 inlet channel width of 200 feet, a railroad channel width of 590 feet and an I-5 channel width of 261 feet were found to provide optimum tidal range and flood conveyance.

A summary of the findings from additional modeling analyses performed for the optimized condition of each alternative are listed below.

1. Tidal Velocity at Infrastructure Crossings

    a. Ebb tidal flow velocity at Hwy 101 was lowest for the No Project alternative due to the smaller tidal prism limited by the existing bathymetry of the lagoon's main channel and inlet channel.

    b. Alternatives 1A and 1B show increased ebb tidal flow velocity at Hwy 101 suggesting these alternatives may be able to maintain a wider inlet channel. The increased tidal prism of these alternatives resulted in higher flood and ebb velocities at each bridge.

    c. Alternative 2A will have higher flood and ebb tidal flow velocities at Hwy 101 and I-5 when compared to the No Project conditions due to the significant increase in tidal prism associated with this alternative. Velocities under the railroad bridge are lower than other alternatives to facilitate deposition in this area.

2. The 100-year flood velocity at infrastructure crossings was reported for each optimized model. It should be noted that the model was not set up to estimate maximum velocities but the results are useful for comparison purposes.

    a. The peak velocities under No Project conditions are all near 6 fps for Hwy 101 and the I-5 Bridge channels. Some minor scour can be expected at these locations. The velocity under the railroad bridge is relatively low (below 3 fps) and will probably not result in any significant scour.

    b. Alternatives 1A and 1B models indicate increased velocities at Hwy 101 to almost 14 fps. These high velocities will cause significant scour but the presence of hard

bottom reef and bedrock should limit the scour depth. Velocities at the railroad and I-5 Bridges are relatively low for these alternatives.

c. Alternative 2A model results indicate velocities under the Hwy 101 Bridge of almost 10 fps at the peak of the 100-year flood. Significant scour can be expected and a more detailed study of potential maximum scour depths will be necessary for design of the bridge structure. Lower flood flow velocities exist at the other bridges and are not as much of a potential concern.

3. Sedimentation of each alternative was evaluated based on peak velocity during the 100-year flood simulation. The results indicate velocities throughout the lagoon remain mostly above the threshold of 0.6 fps along the main flow path for each alternative during a flood event of this magnitude. Suspended sediment in stormflows should largely remain in suspension and be transported to the ocean, rather than being deposited in the lagoon. These results are similar to the conclusions reached in the SELRP Water Quality Study (M&N 2010[b]).

4. The residence time analyses, used to assess the relative lagoon water quality of each alternative, indicates a significant reduction in residence time for alternatives which include dredging of the main channel through each basin. Alternatives 1B and 2A have the shortest predicted residence times and will achieve much higher turnover rates and should maintain better water quality than the other alternatives if other variables (contaminant inputs) remain unchanged.

5. Hydrodynamic modeling runs were performed considering projections for sea level rise (SLR) in the year 2100. A 55-inch SLR estimate was assumed for the modeling study, and the offshore spring tidal series was raised by this amount (although wave run-up was subtracted from the water level due to increased depths at the inlet under these conditions). The results indicate the flood elevations through the Central Basin will rise by approximately 2 feet.

   a. For No Project, and Alternatives 1A and 1B, the Central Basin flood elevations will increase from approximately +10 to +12 (feet, NGVD) and nearly 2,000 linear feet of Manchester Avenue will become flooded under these sea level rise projections. Freeboard below the existing Hwy 101 Bridge also becomes very limited for these alternatives. The Railroad bridge and I-5 Bridge structures both provide adequate freeboard above the 100-year maximum flood elevations.

   b. For Alternative 2A, Central Basin flood elevations will increase from about +8 to +10 (feet, NGVD) and Manchester Avenue may experience some shallow flooding in this area. In the East Basin, Alternative 2A flood elevations increase enough to inundate nearly 2,000 linear feet of Manchester Avenue. This alternative includes new bridge structures at each crossing that will account for sea level rise projections in the design and provide adequate freboard above maximum predicted water levels.

14.0 REFERENCES

Dokken Engineering. 2007. Location Hydraulic Study, San Elijo Lagoon. PM 38.6, EA 11-235800, Prepared for Caltrans.

EDAW. 2009. Final San Elijo Lagoon Restoration Project Alternatives Development Report.

Exponent Inc. 2000. Feasibility Study and Conceptual Plan for the Relocation of the San Elijo Lagoon, Progress Report #6 - Hydraulic and Sedimentation Study. Prepared for City of Encinitas.

Fischer, H.B., List, E.J., et. al., "Mixing in Inland and Coastal Waters", Academic Press, Inc., 1979.

Hjulstrom, F., "Studies of the Morphological Activity of Rivers as Illustrated by the River Fyris," *Bulletin,* Geological Institute of Upsala, Vol. XXV, Upsala, Sweden, 1935.

McAnally, W.H. and Thomas, W.A. 1985. User's Manual for the Generalized Computer Program System, Open Channel Flow and Sedimentation, TABS-2 Main Text. U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS.

Moffatt & Nichol. 2010a. San Elijo Lagoon Restoration Project, Hydrology/Hydraulic Study, Draft Report. Prepared for the San Elijo Lagoon Conservancy.

Moffatt & Nichol. 2010b. San Elijo Lagoon Restoration Project, Water Quality Study, Draft Report. Prepared for the San Elijo Lagoon Conservancy.

Moffatt & Nichol. 2011. San Elijo Lagoon Restoration Project, Tidal Inlet Stability Study, Draft Report. Prepared for the San Elijo Lagoon Conservancy.

National Oceanic and Atmospheric Administration. 2009. Center for Operational Oceanographic Products and Services, website: http://tidesandcurrents.noaa.gov/.

Towill Inc. 1990. Topographic Map of San Elijo Lagoon. Prepared for Philip Williams & Associates.

U.S. Army Corps of Engineers, Los Angeles District. 2006. San Elijo Lagoon Optimization Study, Final Report. Prepared for Caltrans and City of Encinitas.