San Elijo Lagoon
Bridge Optimization Study

Final Report

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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.

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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)

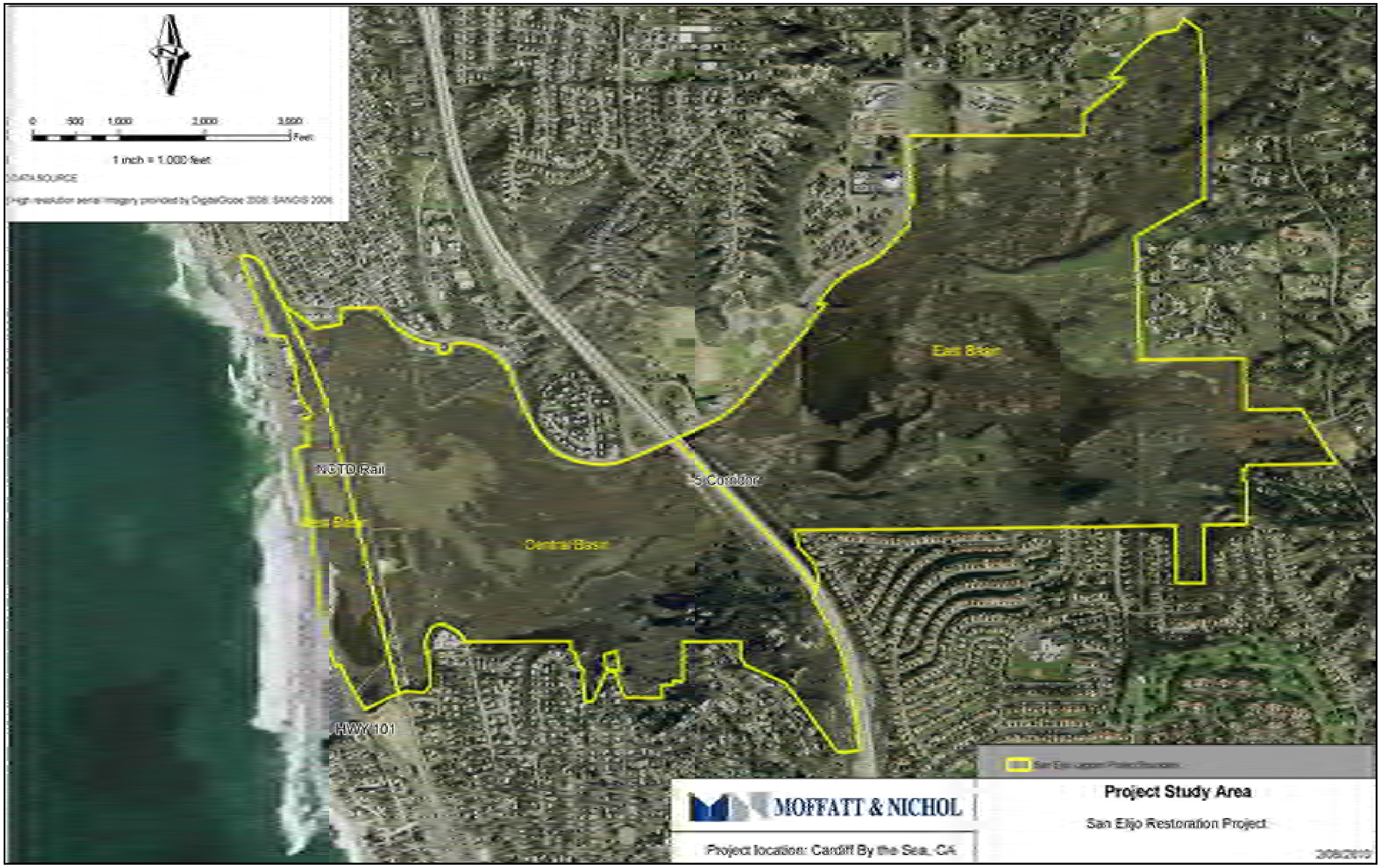


Figure 1-2: Project Study Area

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.

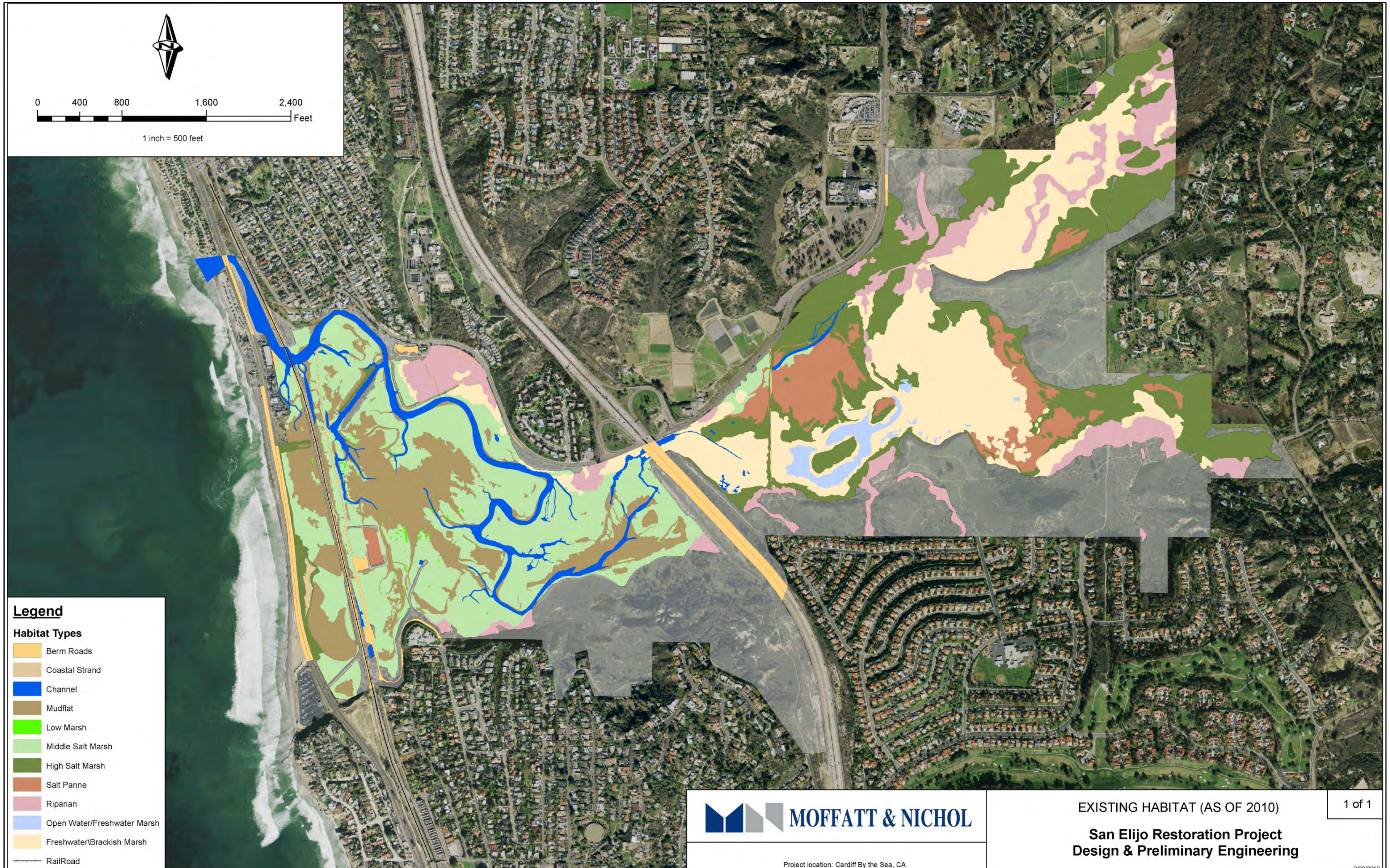


Figure 3-1: No Project - Existing Habitat

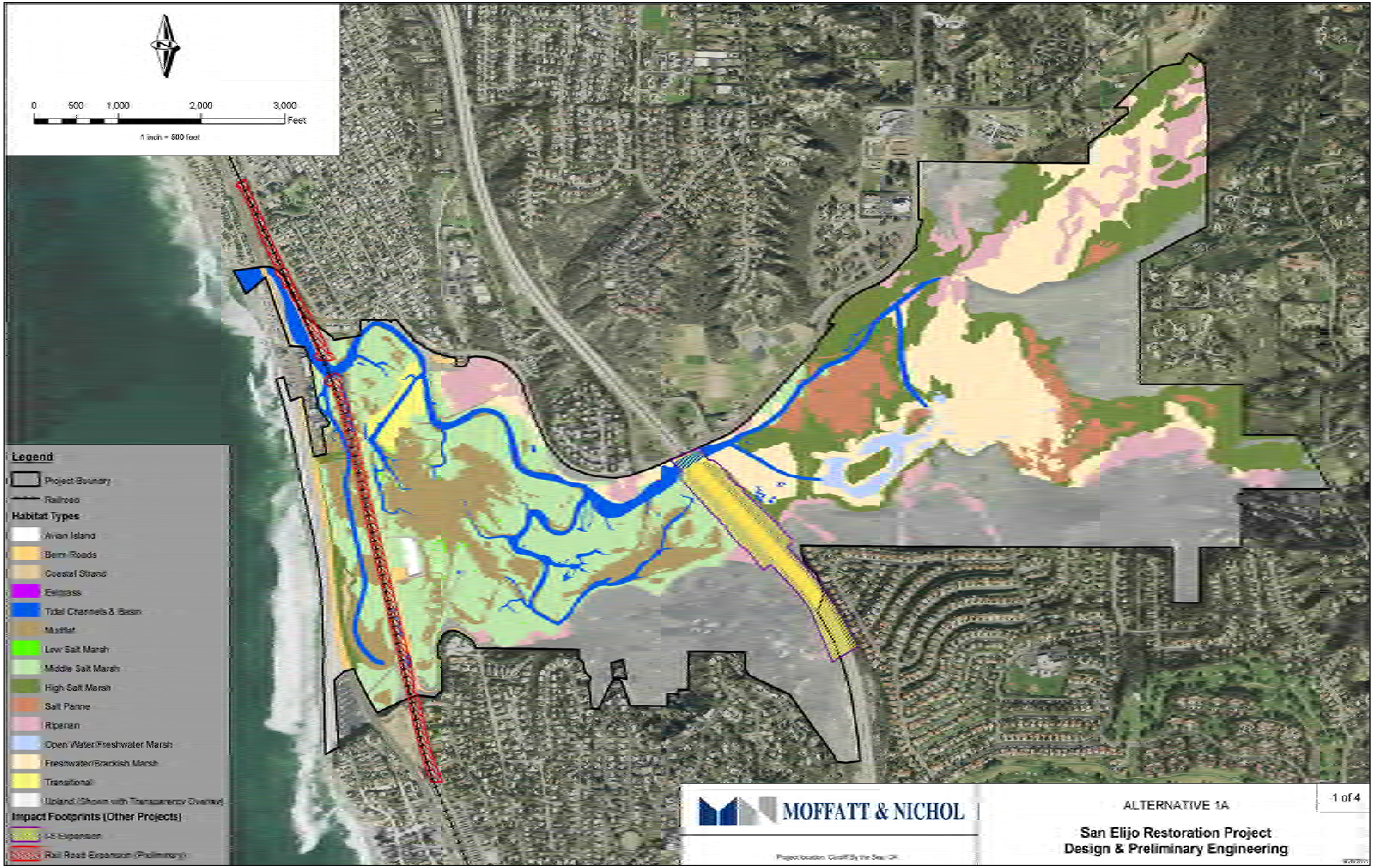


Figure 3-2: Alternative 1A

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.

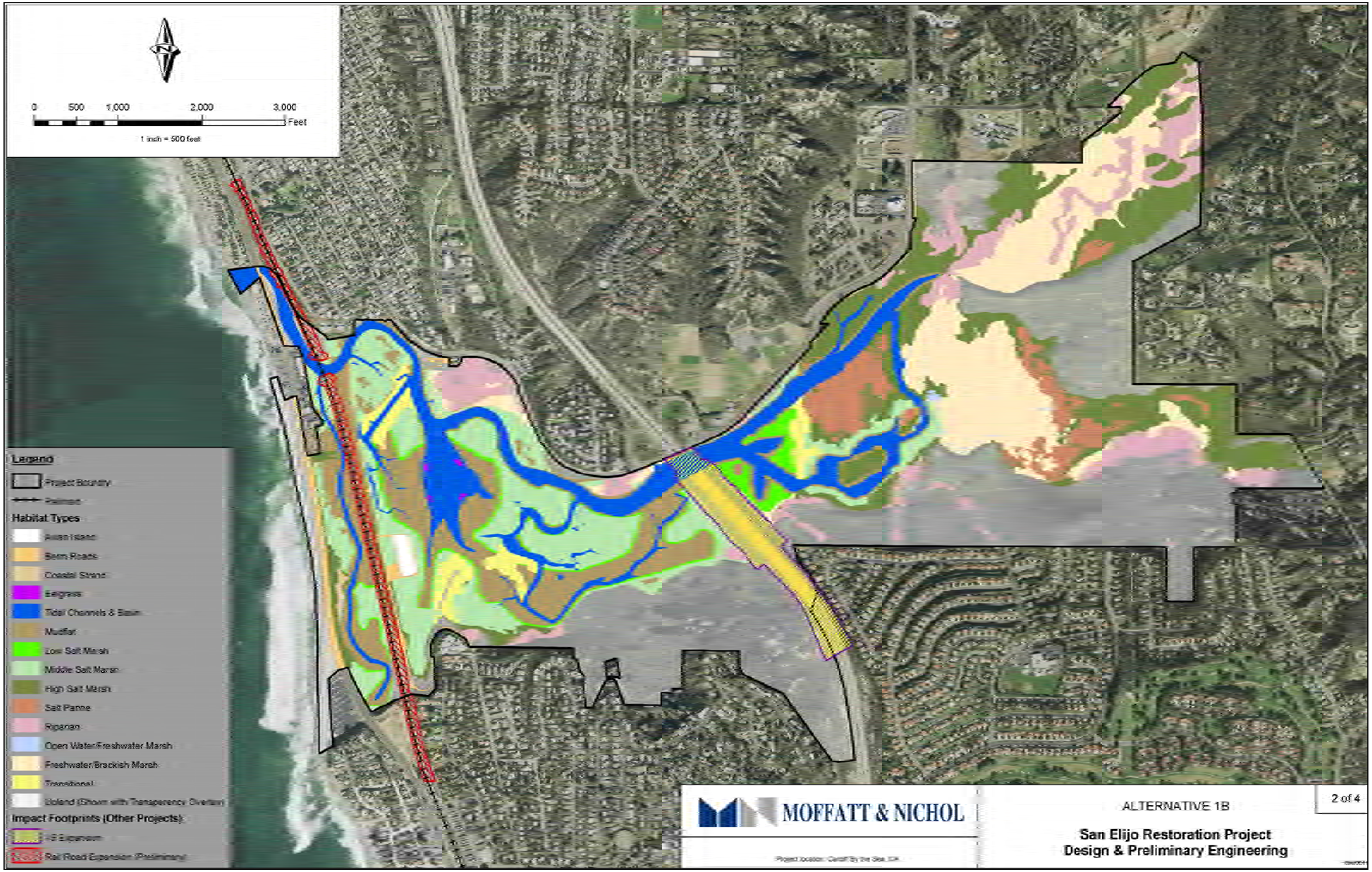


Figure 3-3: Alternative 1B

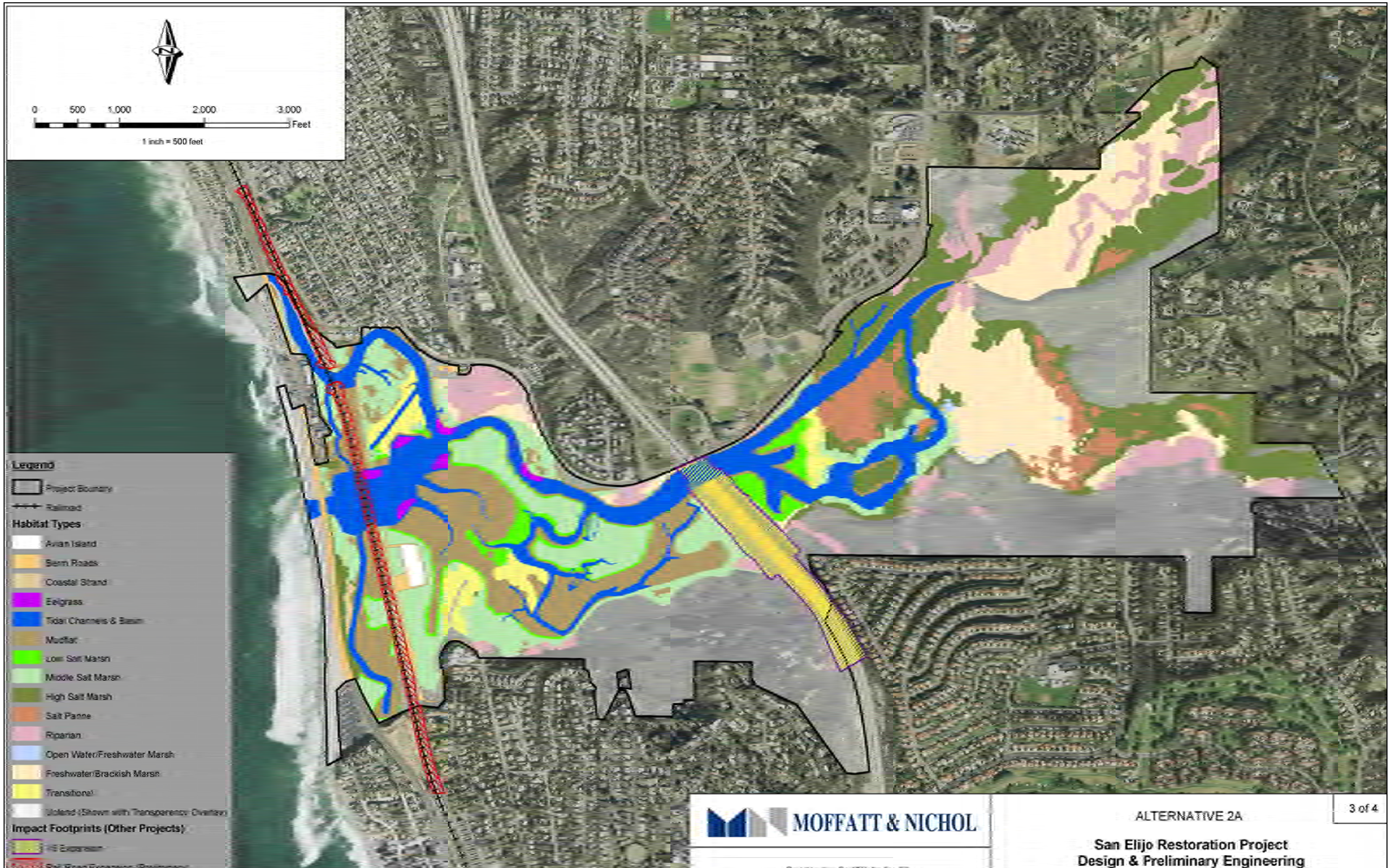


Figure 3-4: Alternative 2A

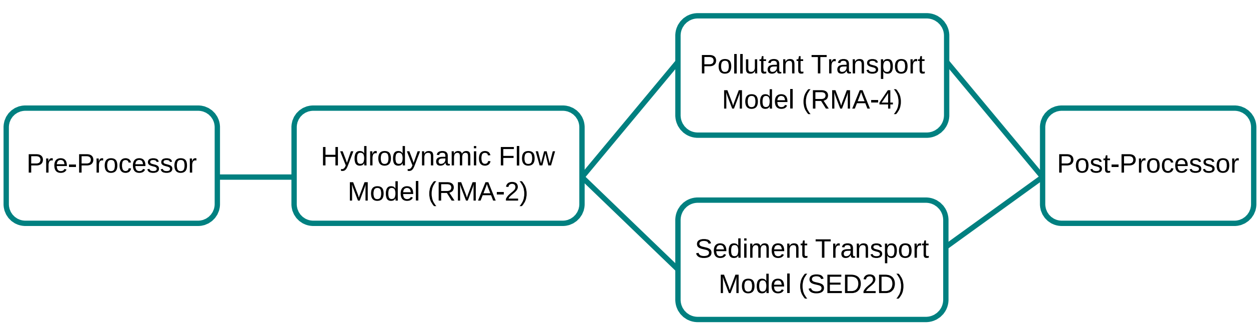
4.0 HYDRODYNAMIC MODEL DESCRIPTION

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.

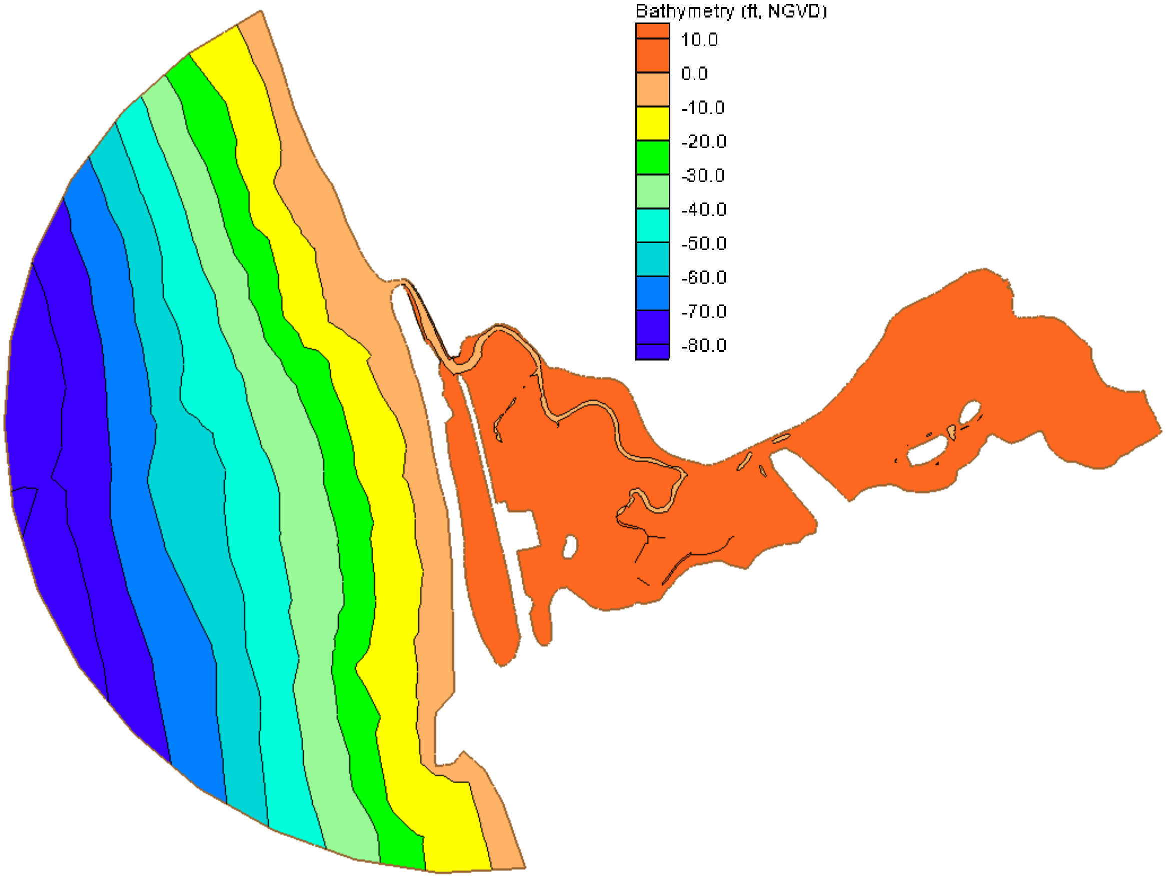


Figure 4-2: Existing Bathymetry for the Entire Modeling Area

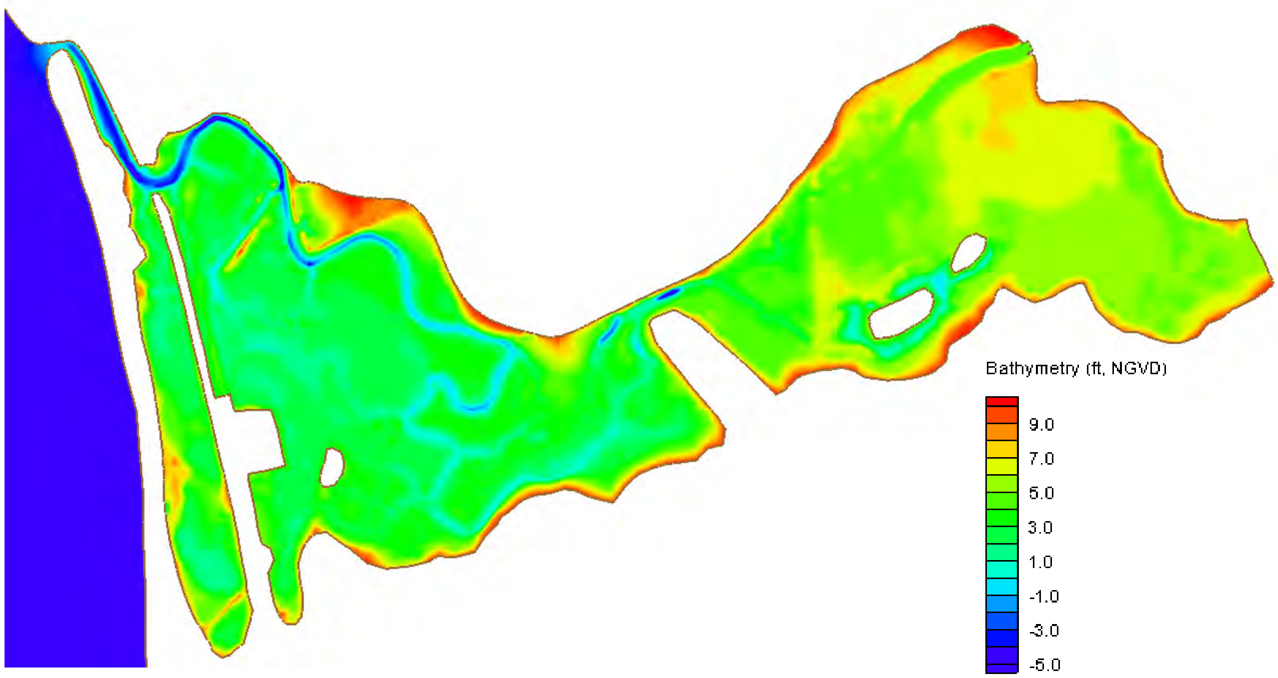


Figure 4-3: Existing Lagoon Bathymetry

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.

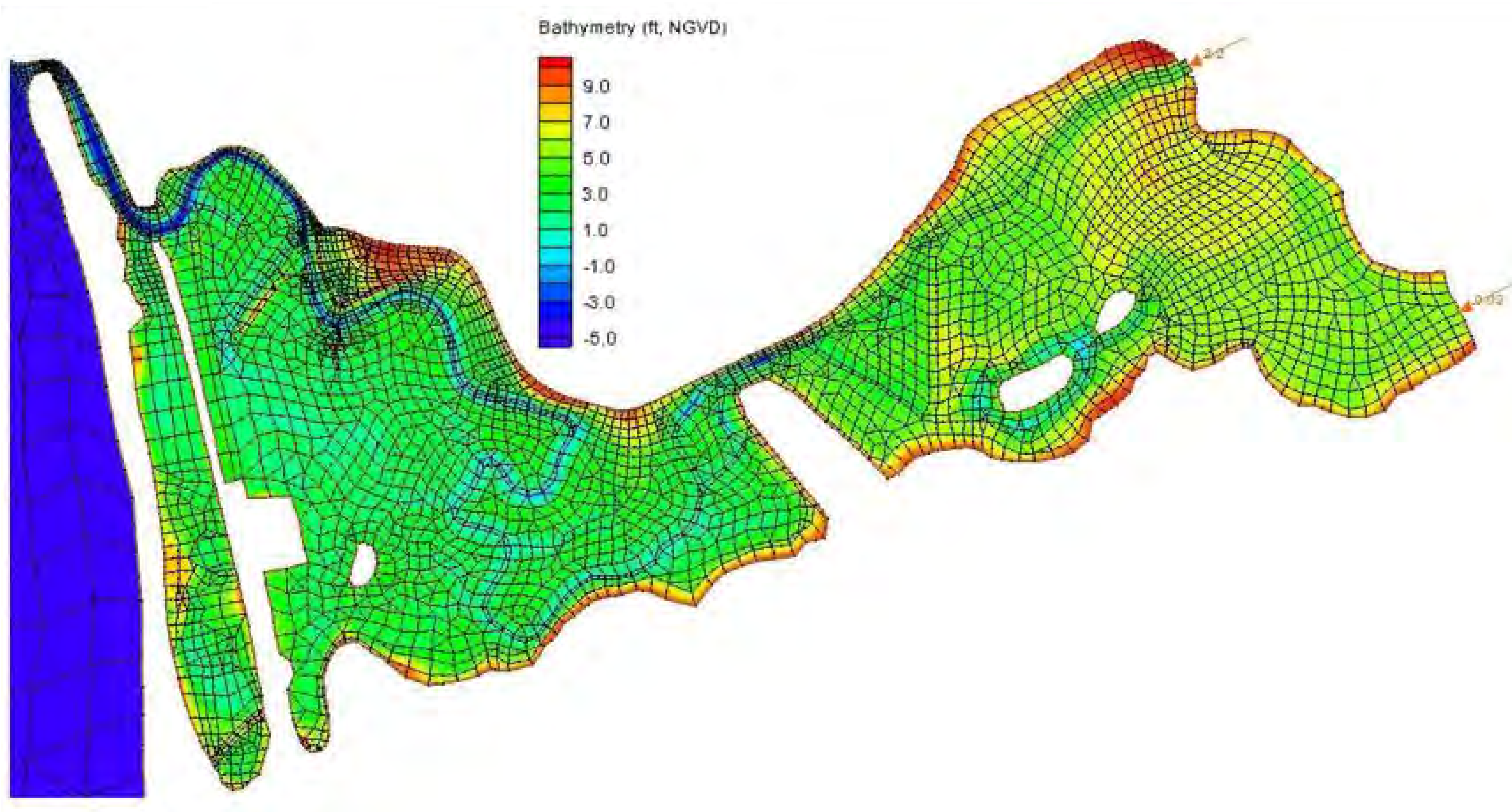


Figure 4-4: RMA-2 Model Mesh for Existing Conditions (No Project)

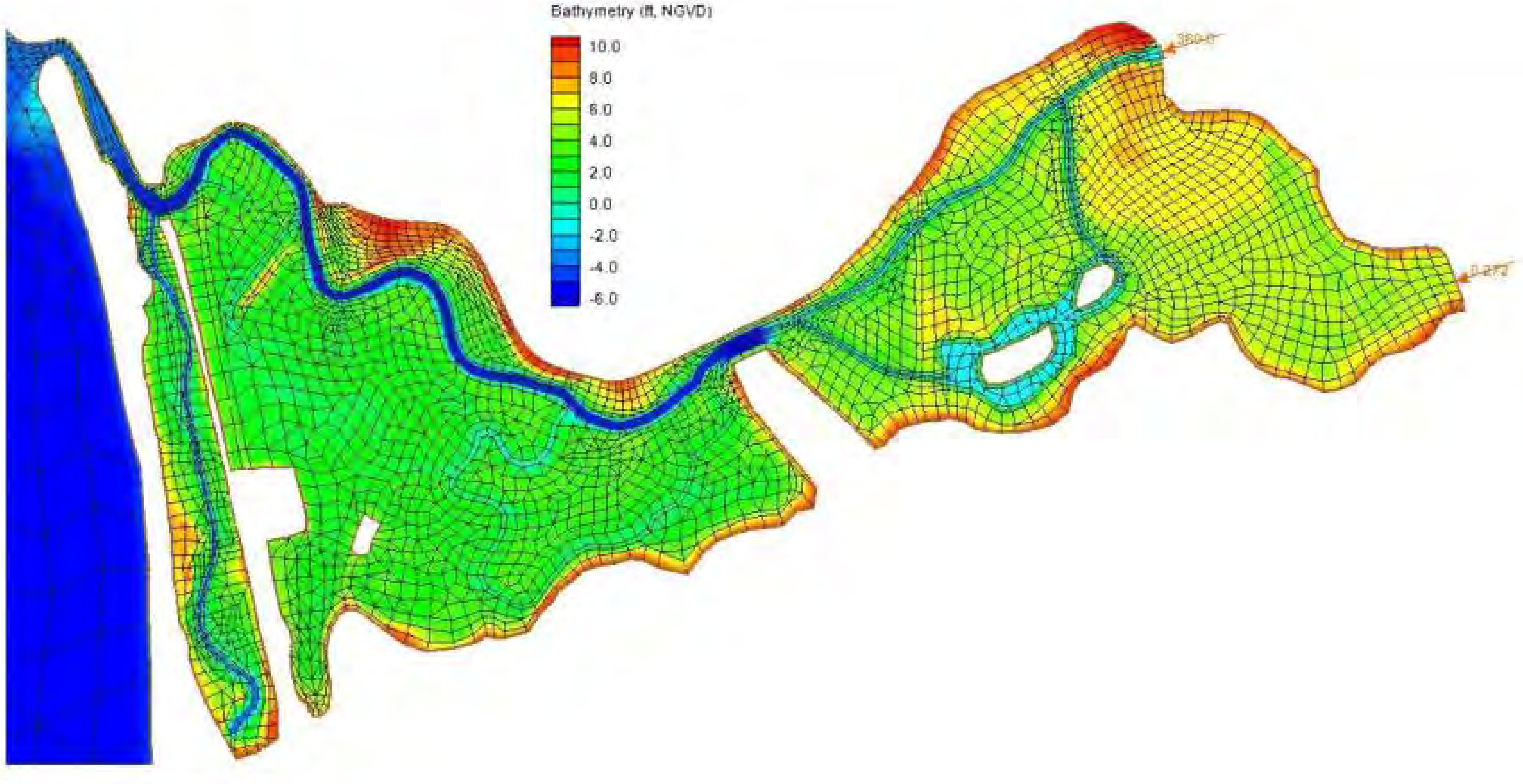


Figure 4-5: RMA-2 Modeling Mesh for Alternative 1A

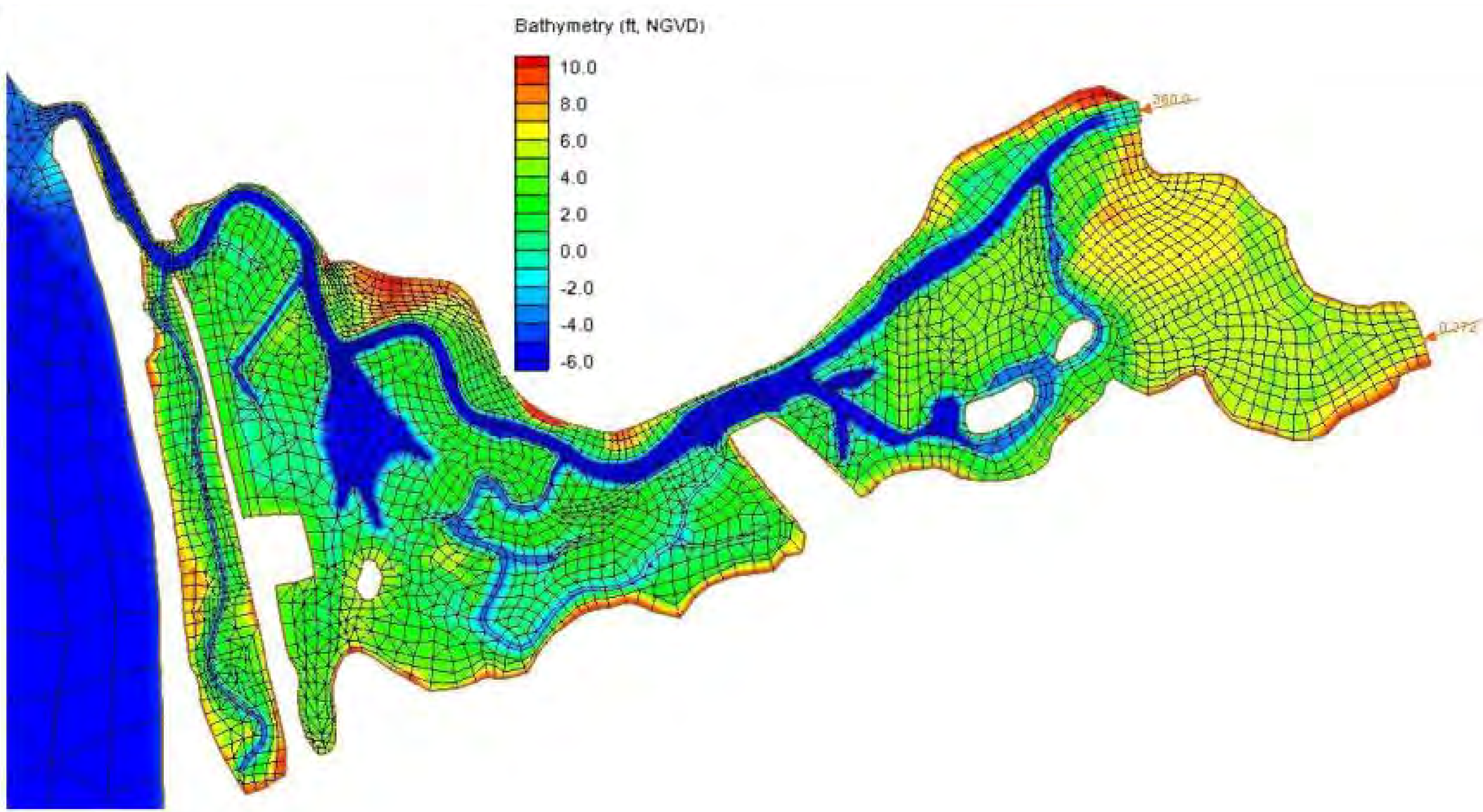


Figure 4-6: RMA-2 Modeling Mesh for Alternative 1B

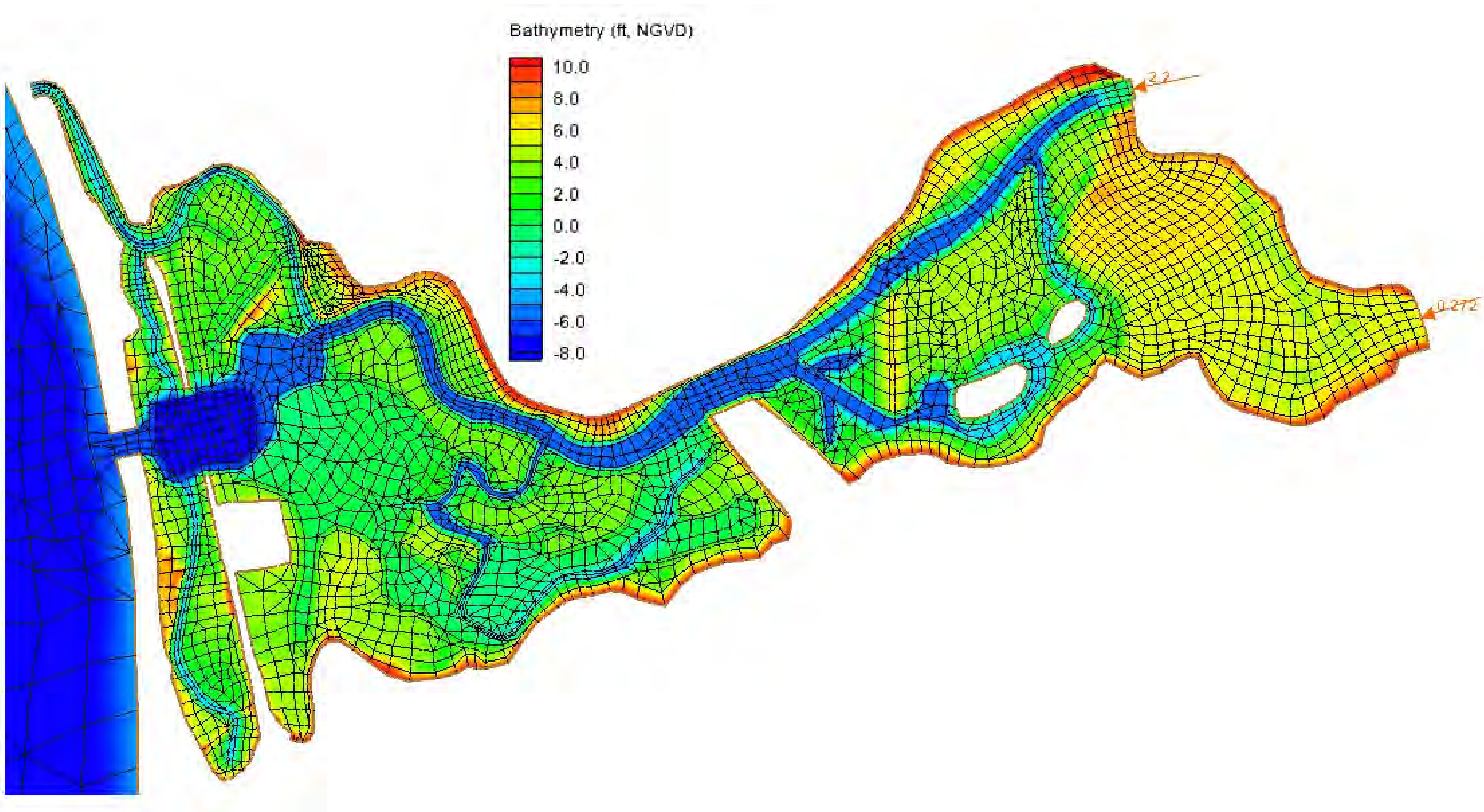


Figure 4-7: RMA-2 Modeling Mesh for Alternative 2A

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.

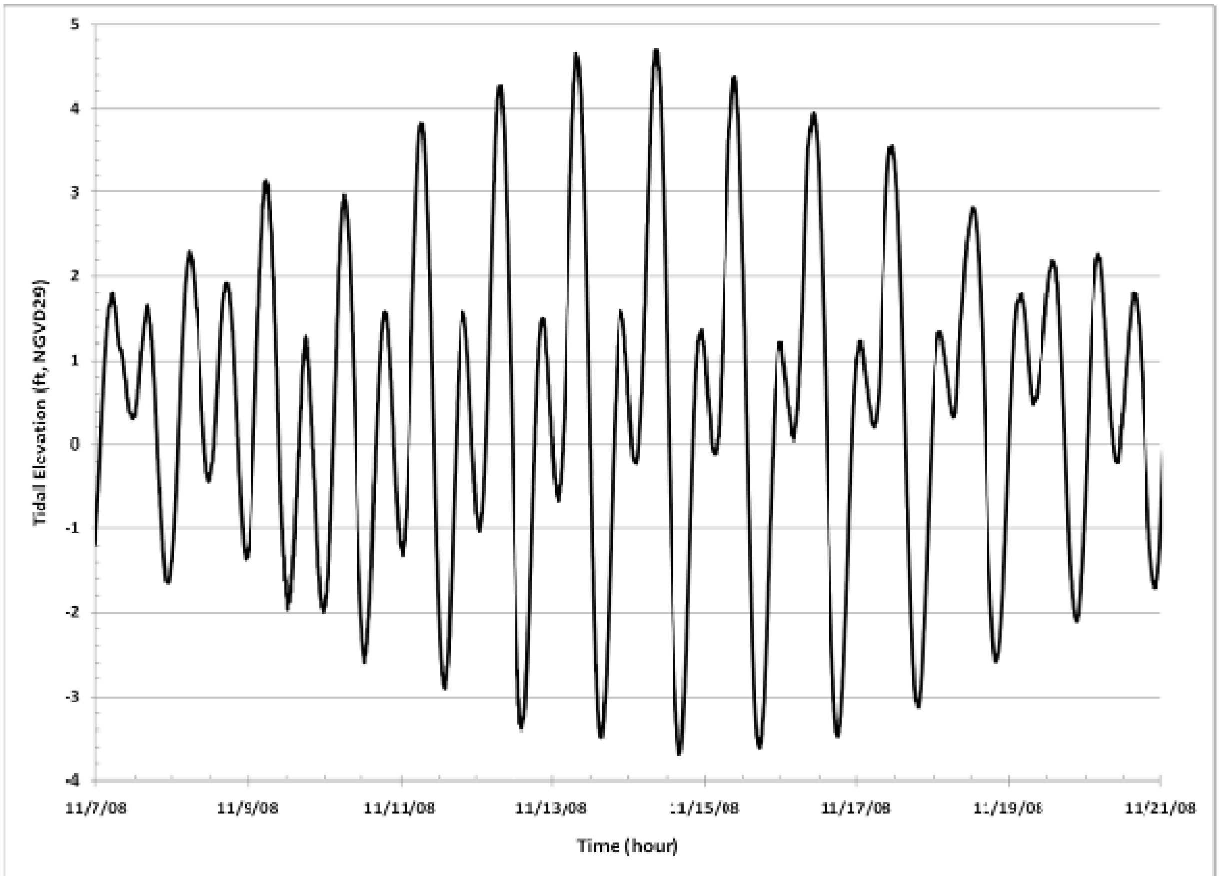


Figure 4-8: Modeling Tidal Series

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.

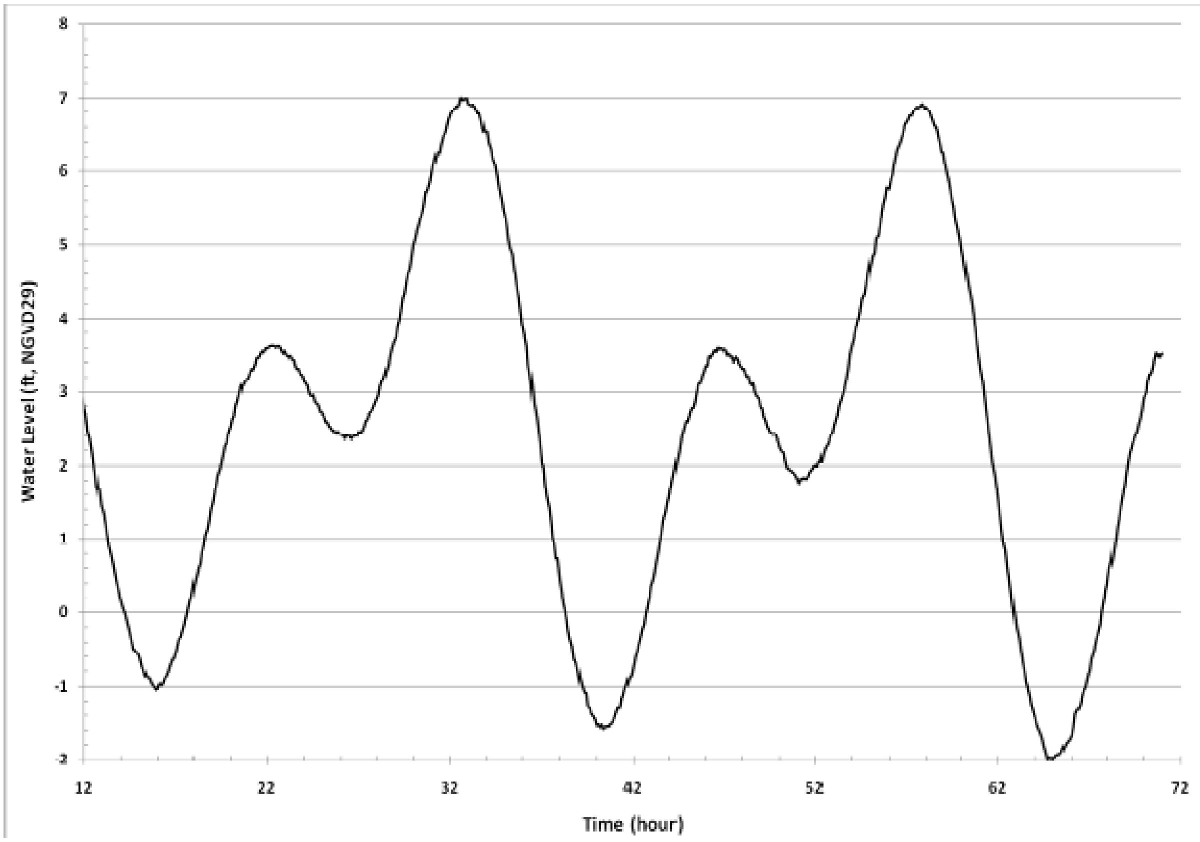


Figure 4-9: Extreme Tidal Series

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.

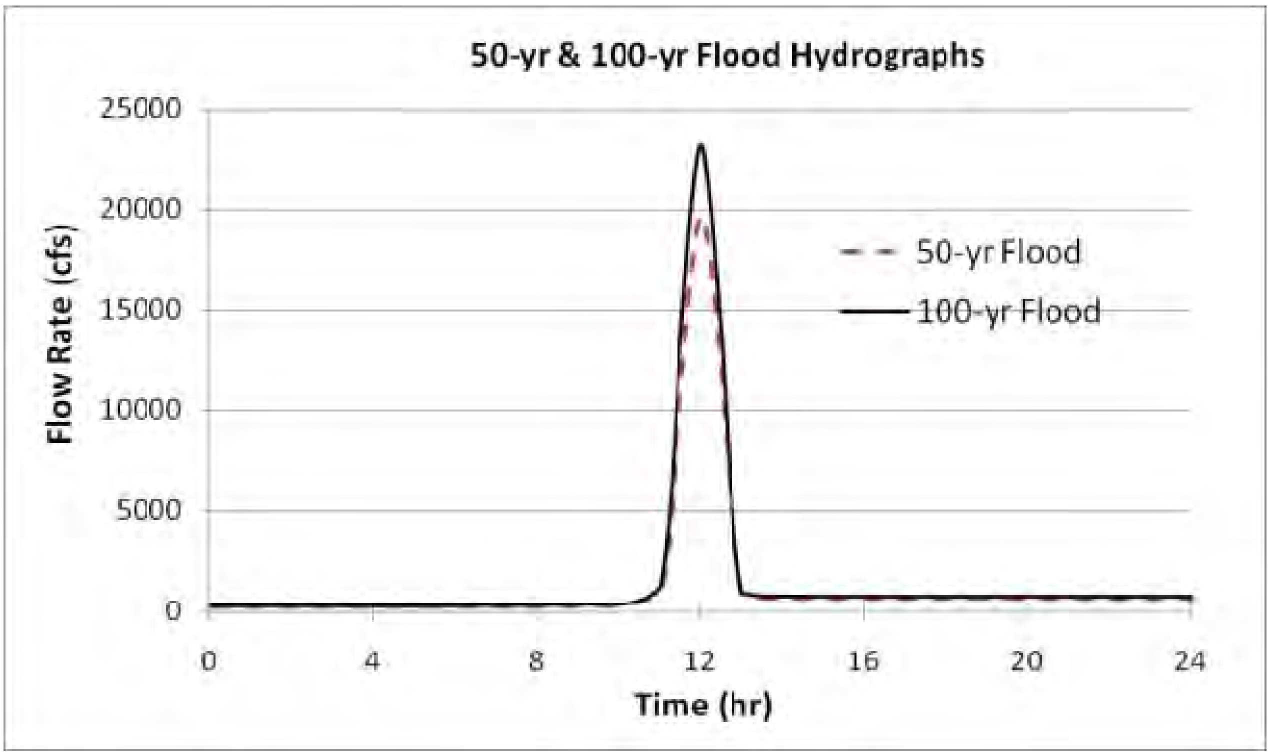


Figure 4-10: San Elijo Lagoon Flood Hydrographs

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.

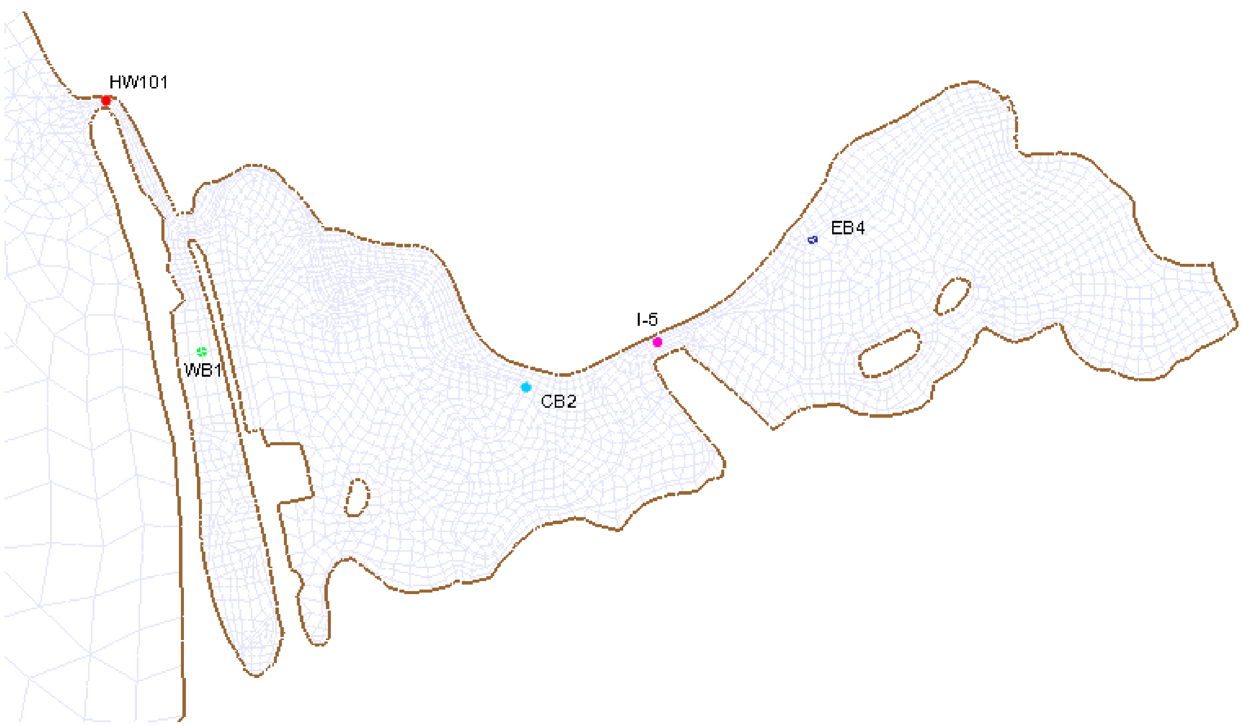


Figure 6-1: Tide Gage Locations (No Project, Alternatives 1A & 1B)

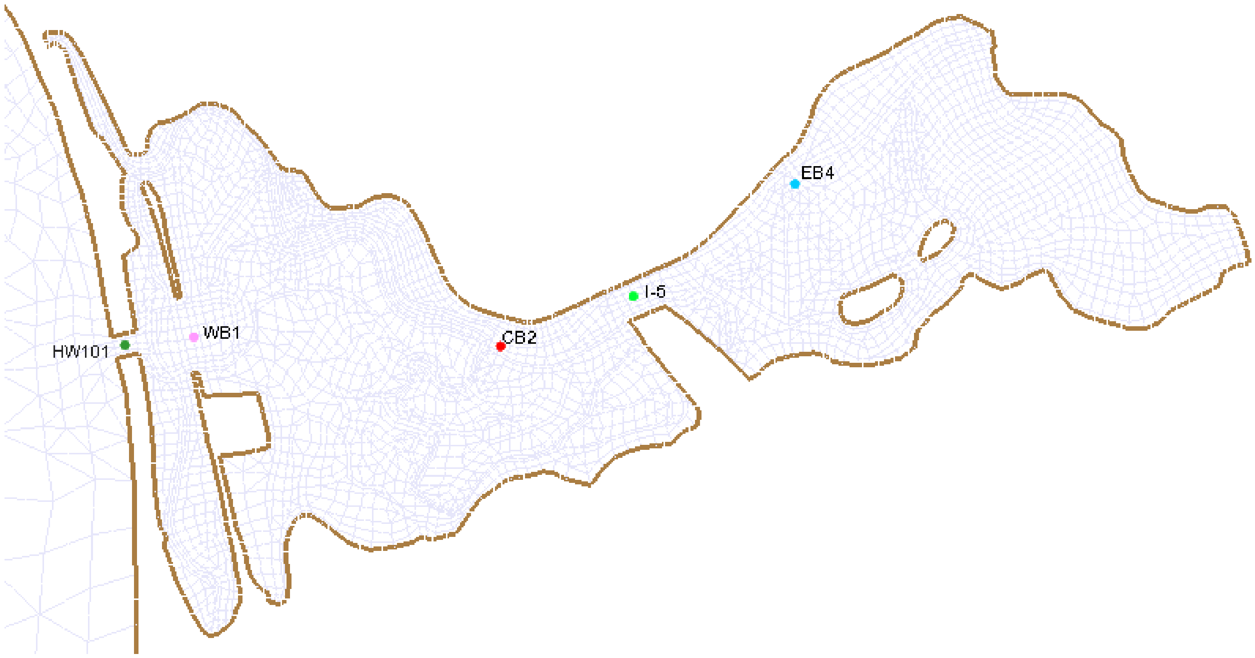


Figure 6-2: Tide Gage Locations (Alternative 2A)

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.

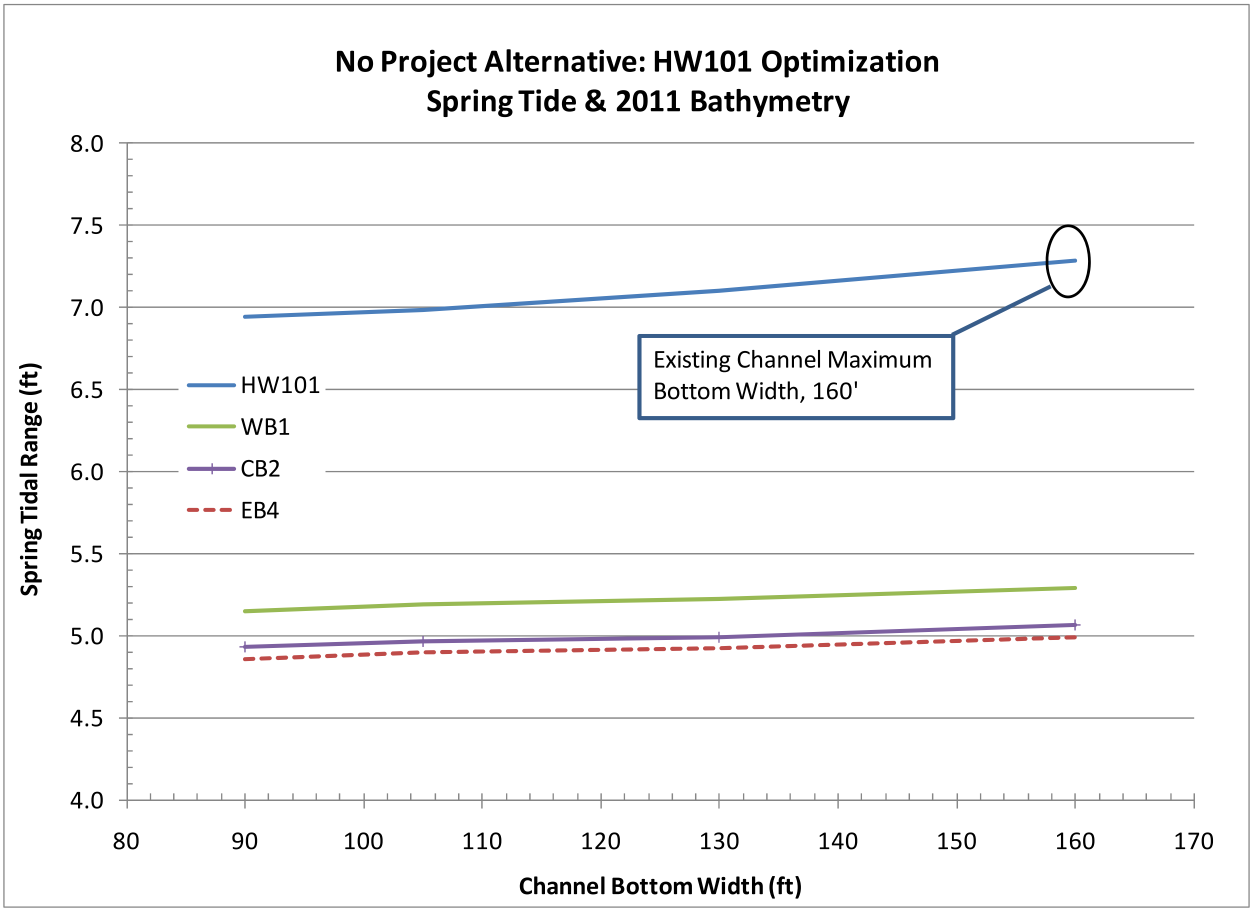


Figure 6-3: No Project Hwy 101 (Tidal Range vs Channel Width)

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.

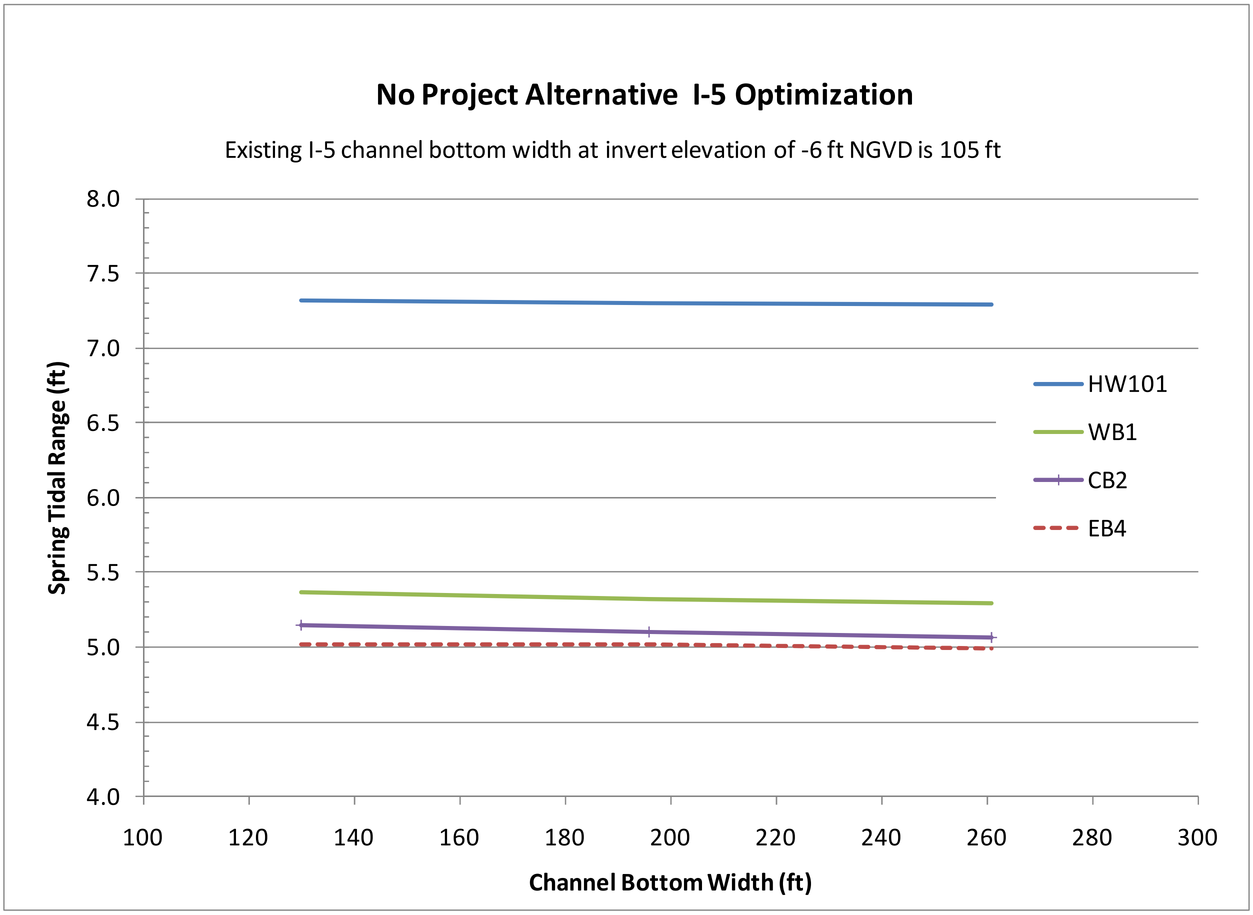


Figure 6-4: No Project I-5 Bridge (Tidal Range vs Channel Width)

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.

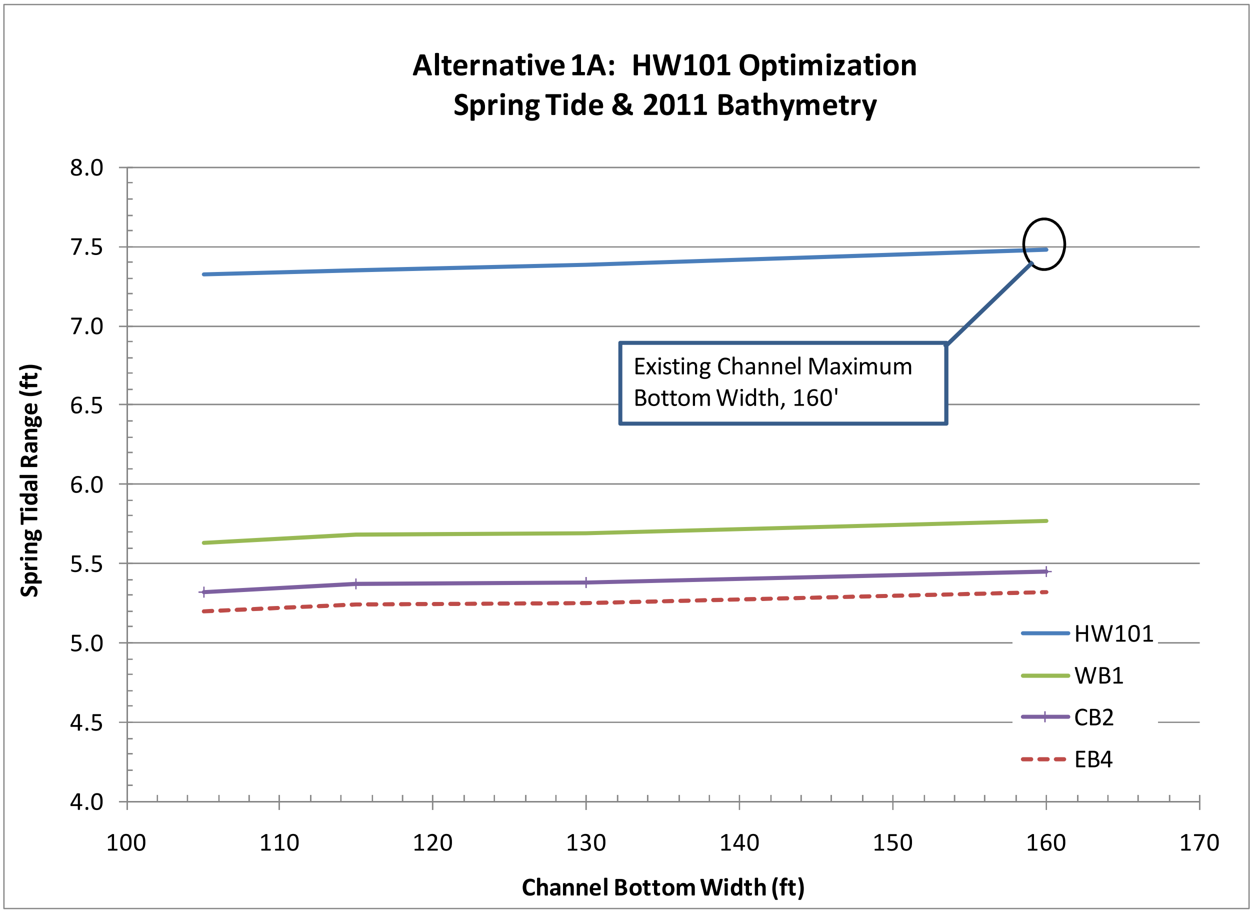


Figure 6-5: Alternative 1A Hwy 101 (Tidal Range vs Channel Width)

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

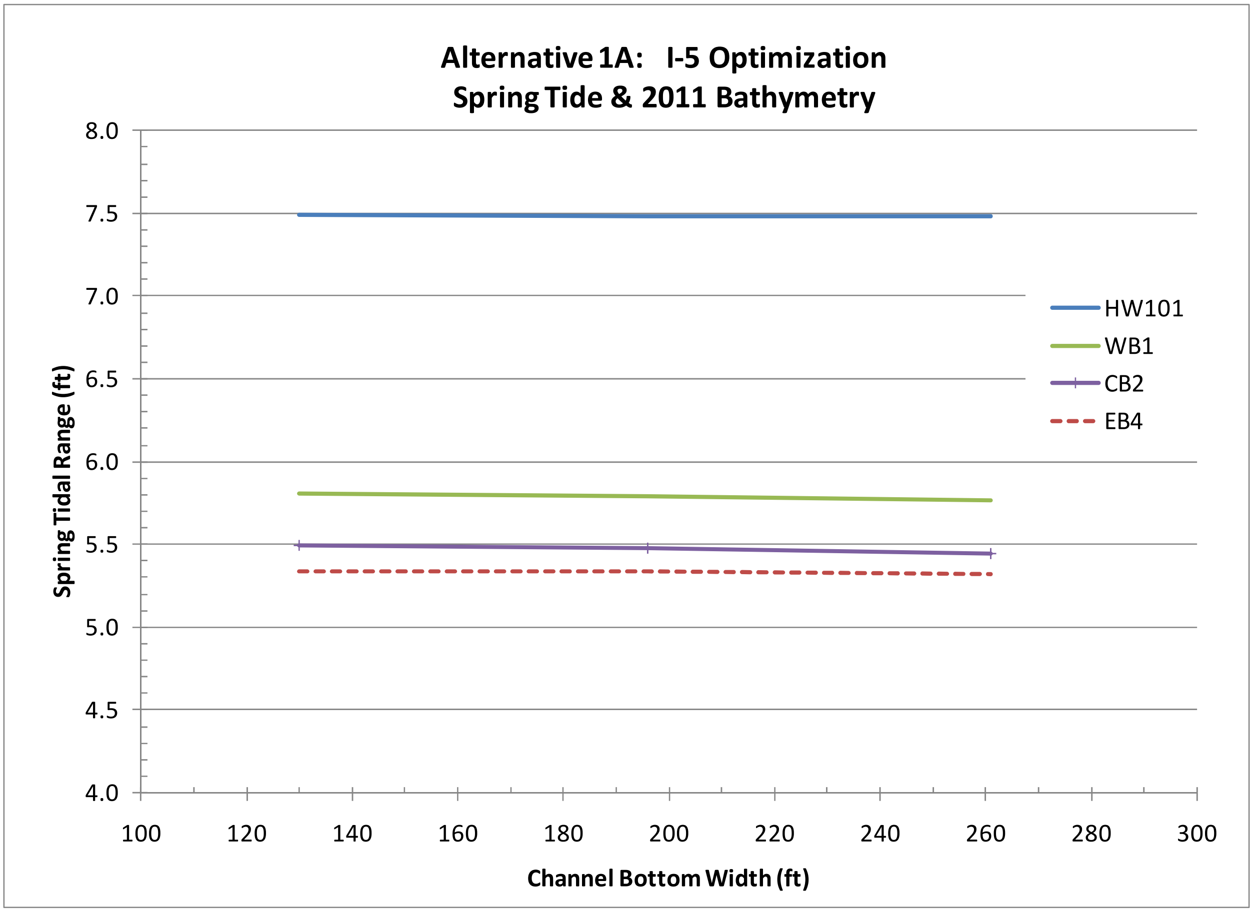


Figure 6-6: Alternative 1A I-5 Bridge (Tidal Range vs Channel Width)

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.

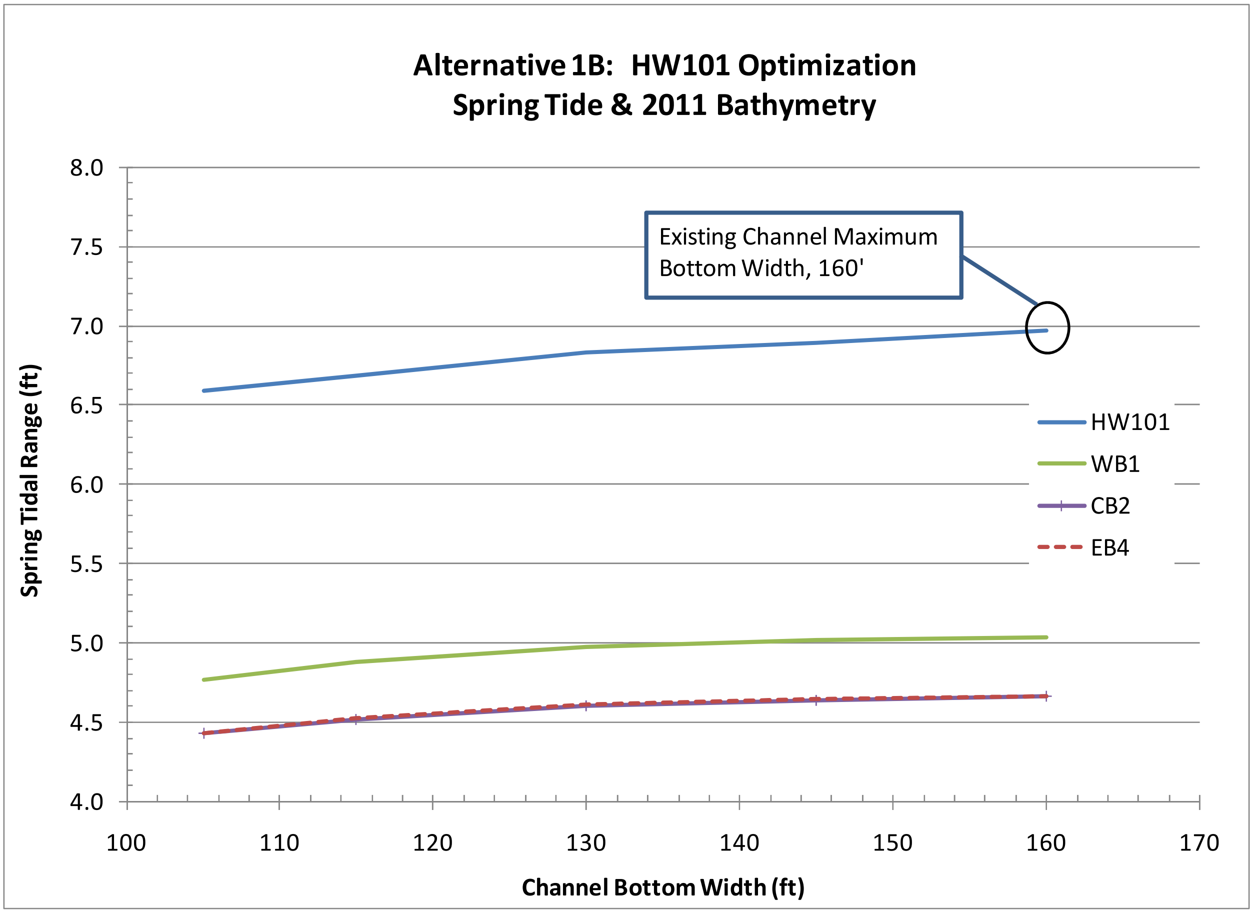


Figure 6-7: Alternative 1B Hwy 101 (Tidal Range vs Channel Width)

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

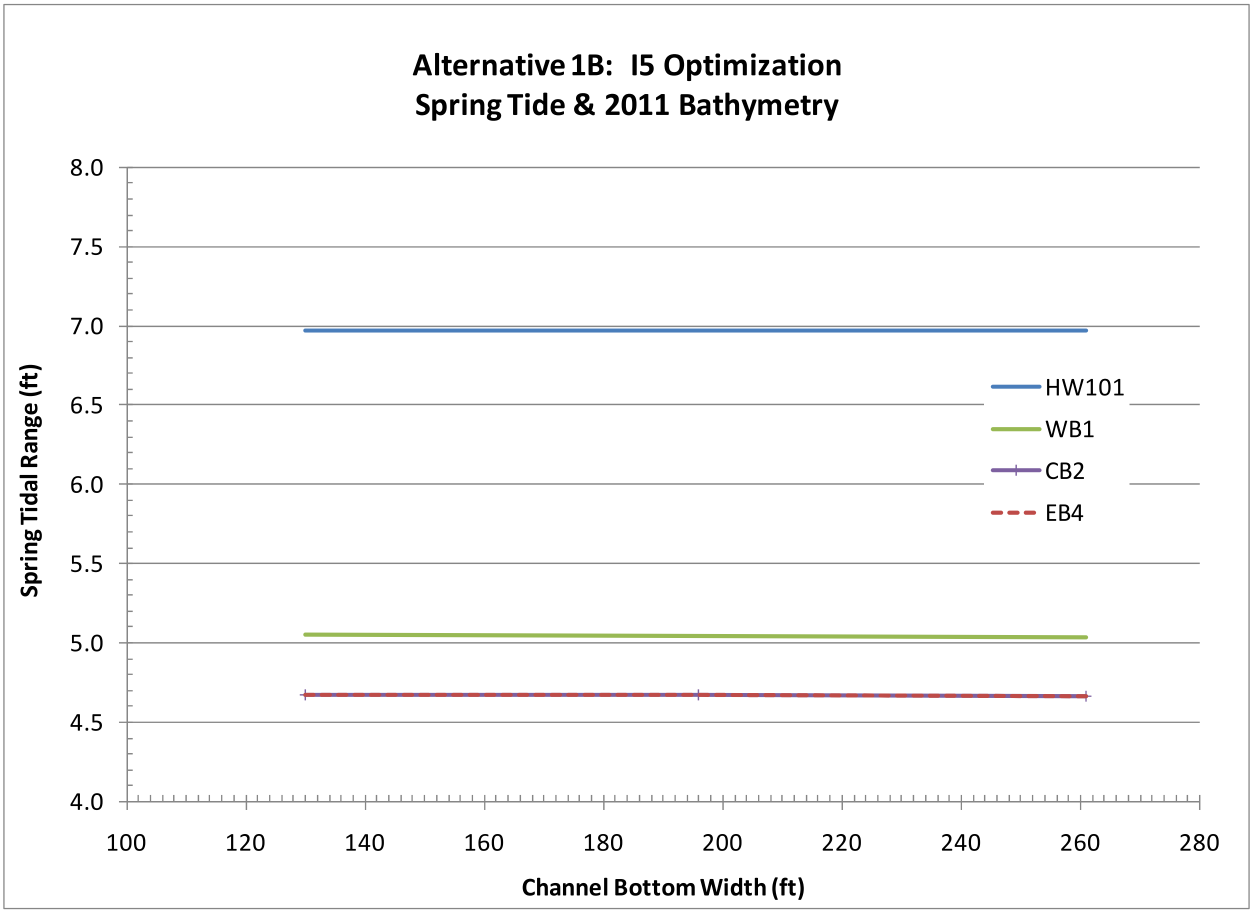


Figure 6-8: Alternative 1B I-5 Bridge (Tidal Range vs Channel Width)

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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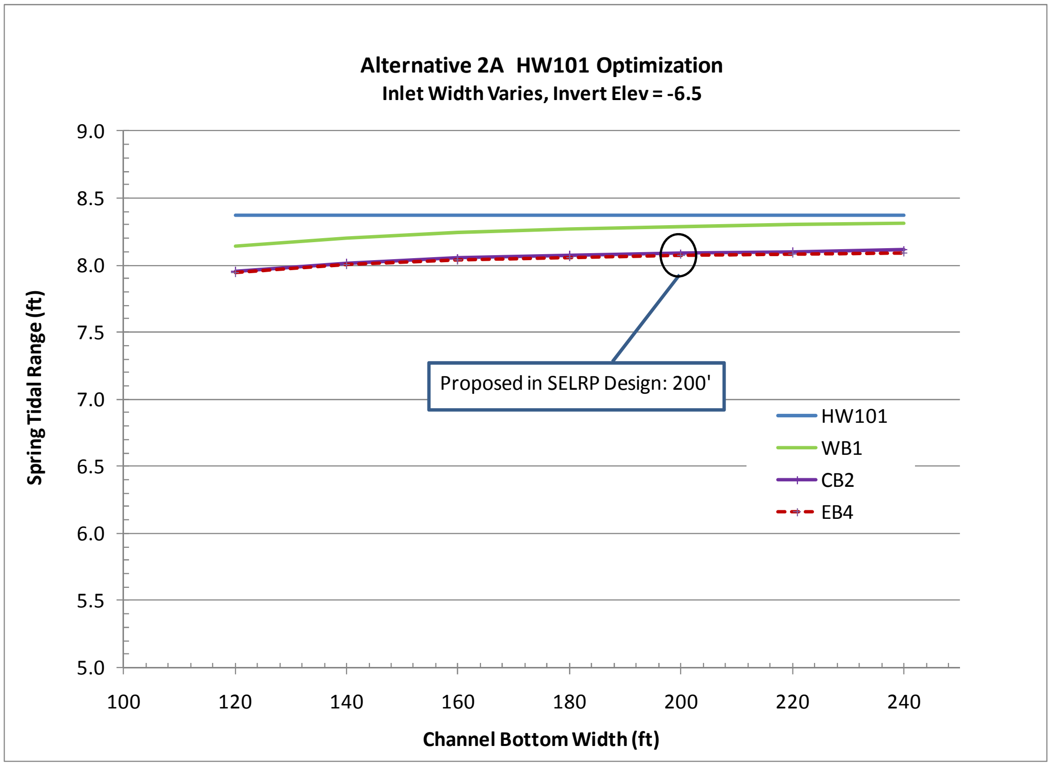

Figure 6-9: Alternative 2A Hwy 101 Tidal Range (Invert of -6.5 feet, NGVD)

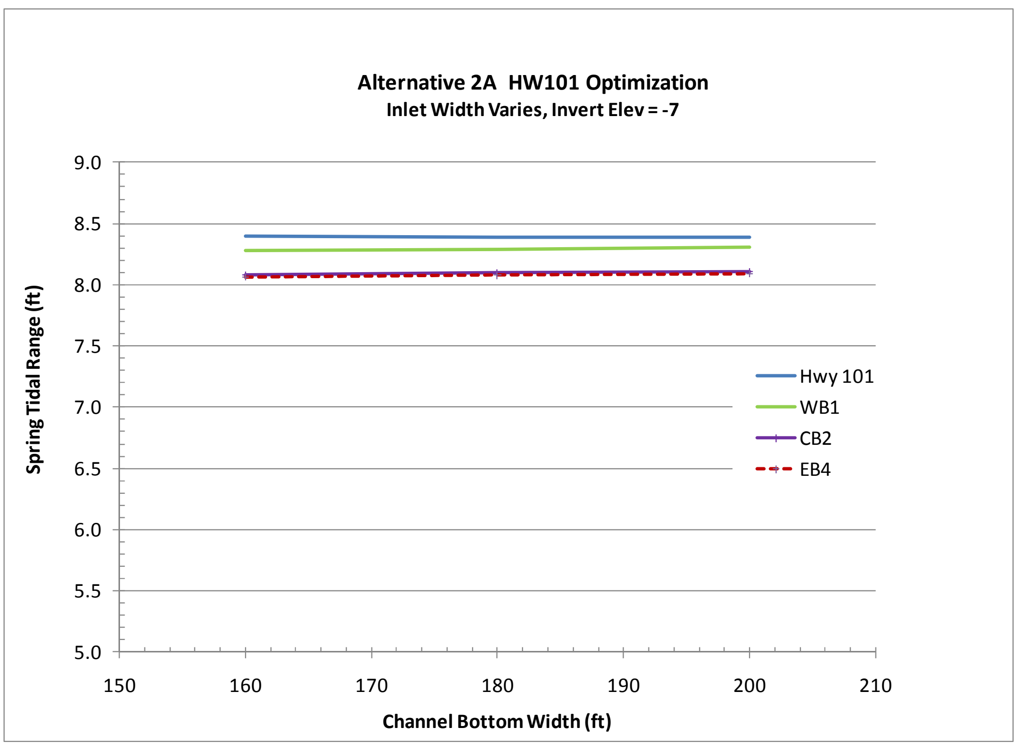

Figure 6-10: Alternative 2A Hwy 101 Tidal Range (Invert of -7 feet, NGVD)

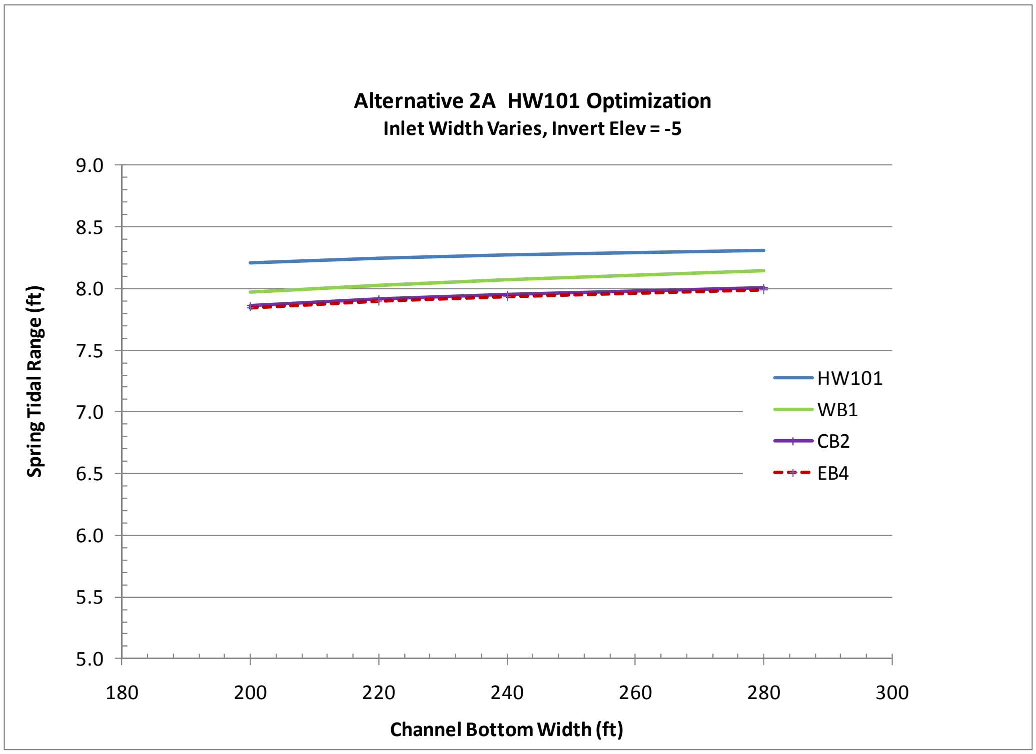

Figure 6-11: Alternative 2A Hwy 101 Tidal Range (Invert of -5 feet, NGVD)

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.

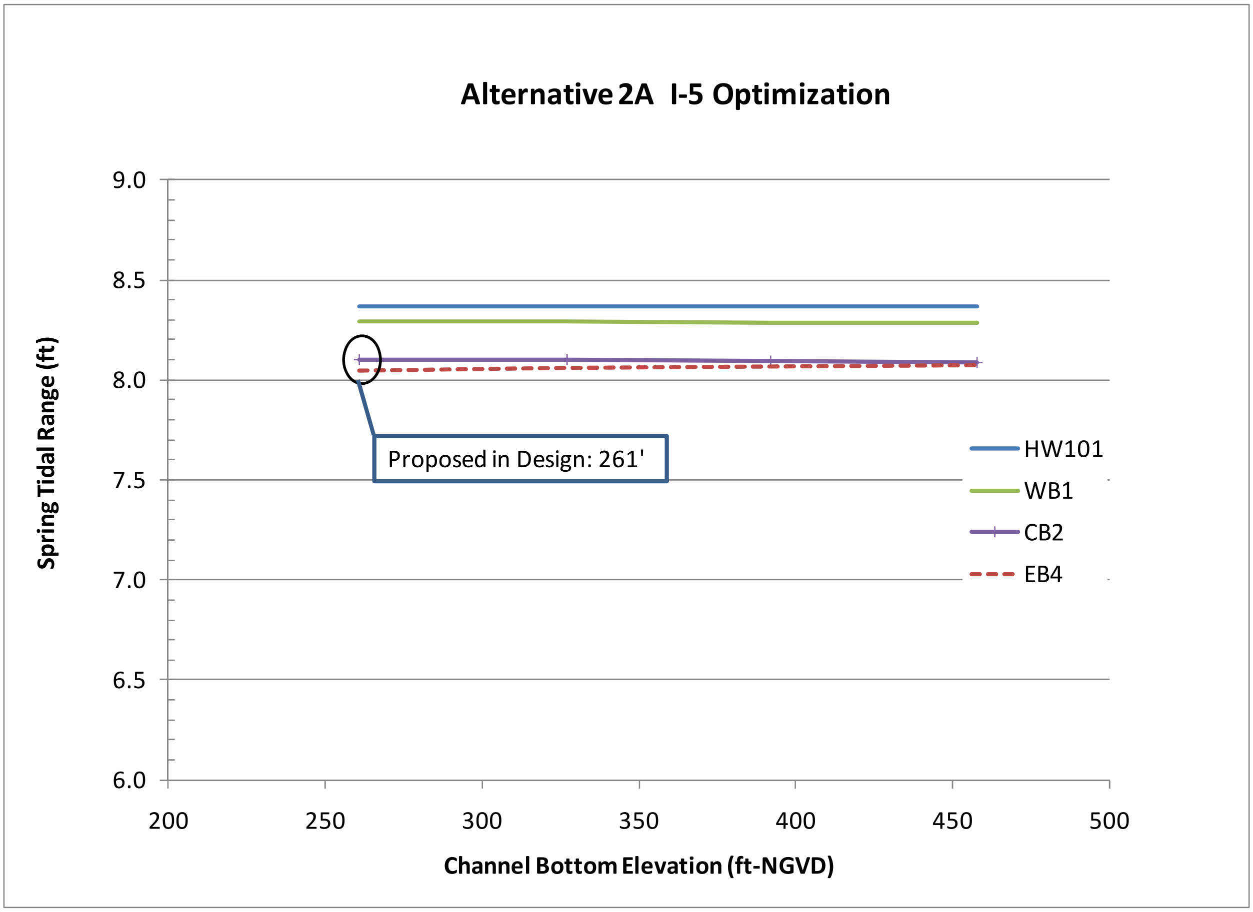

Figure 6-12: Alternative 2A I-5 Bridge (Tidal Range vs Channel Width)

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.

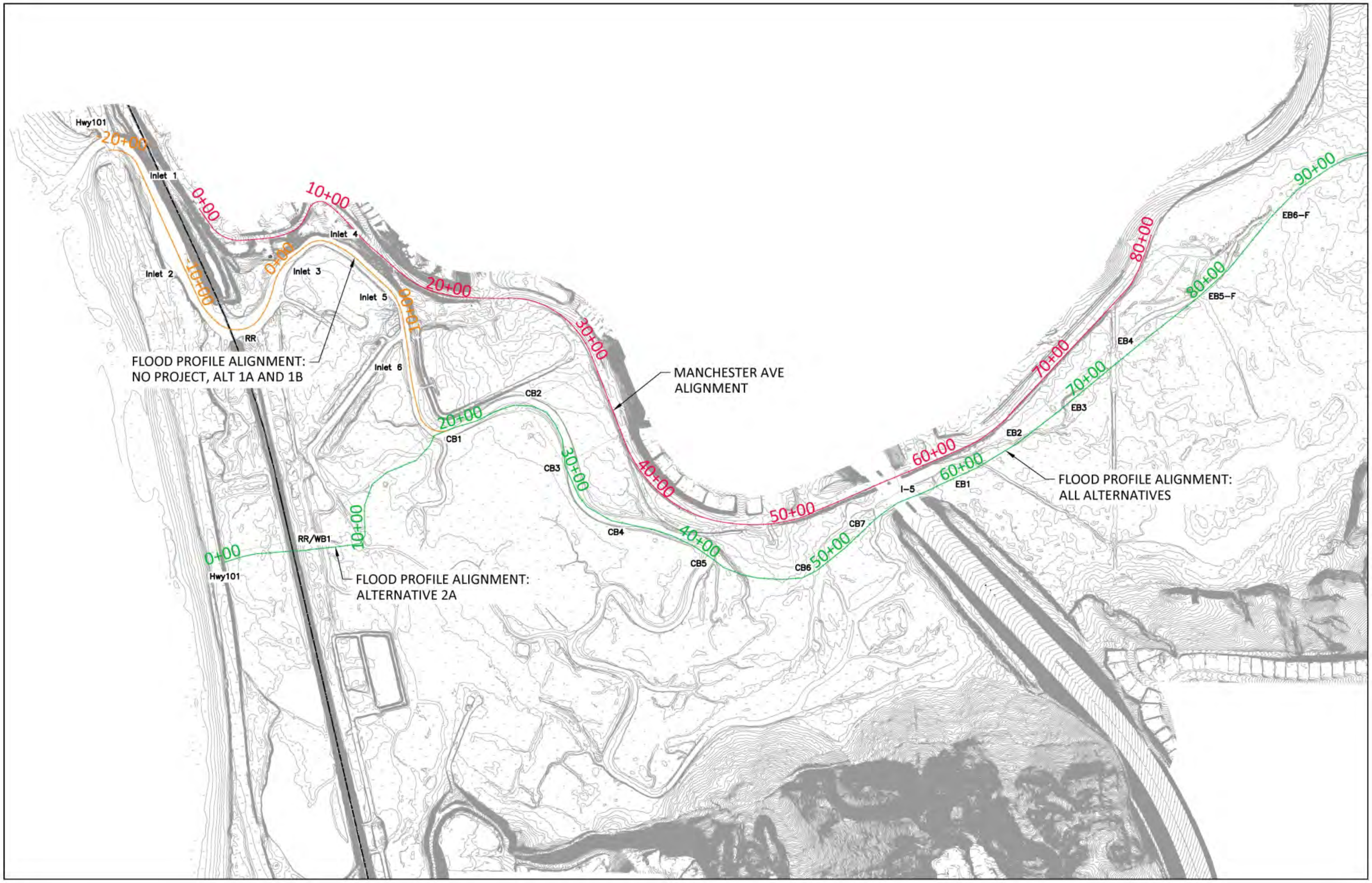

Figure 7-1: Flood Profile Alignments

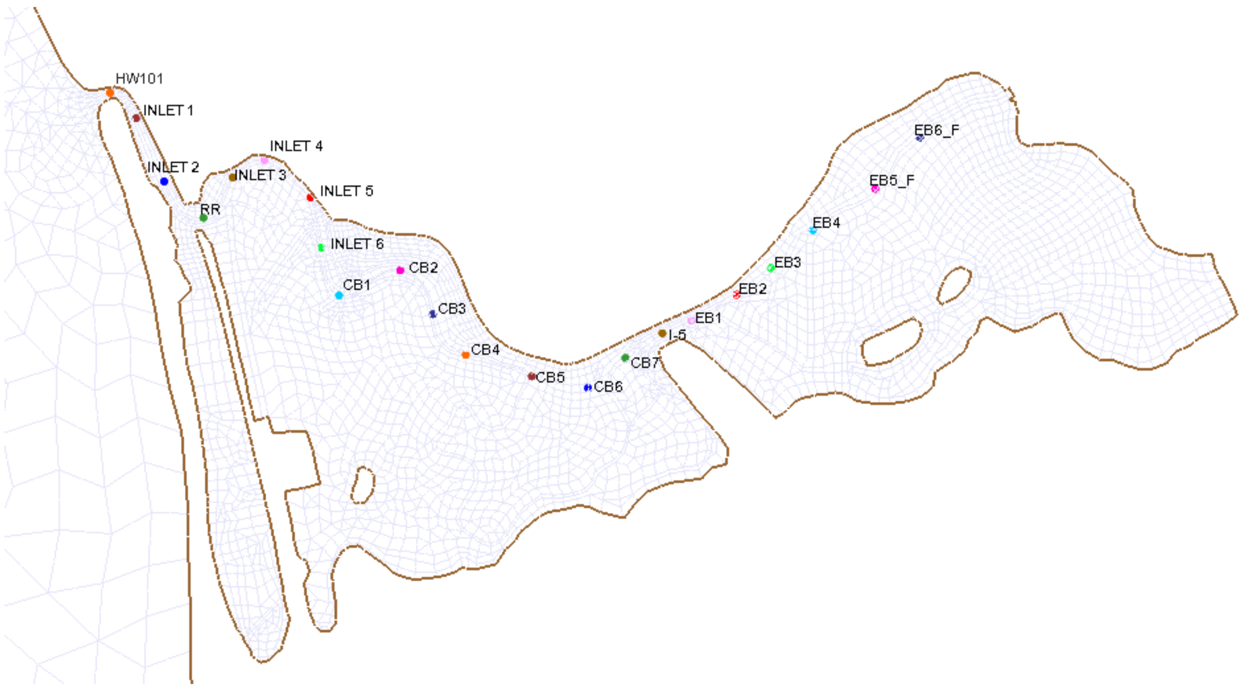

Figure 7-2: Flood Gage Locations on RMA-2 Mesh (No Project, Alternatives 1A & Alt 1B)

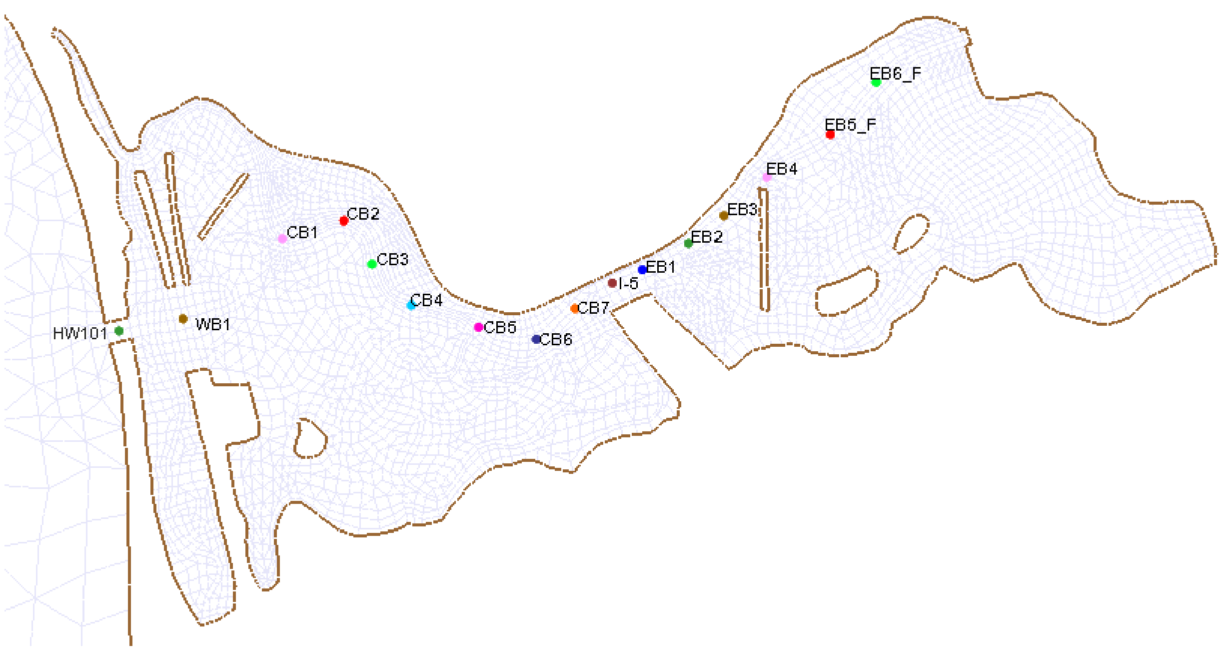

Figure 7-3: Flood Gage Locations on RMA-2 Mesh (Alternative 2A)

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

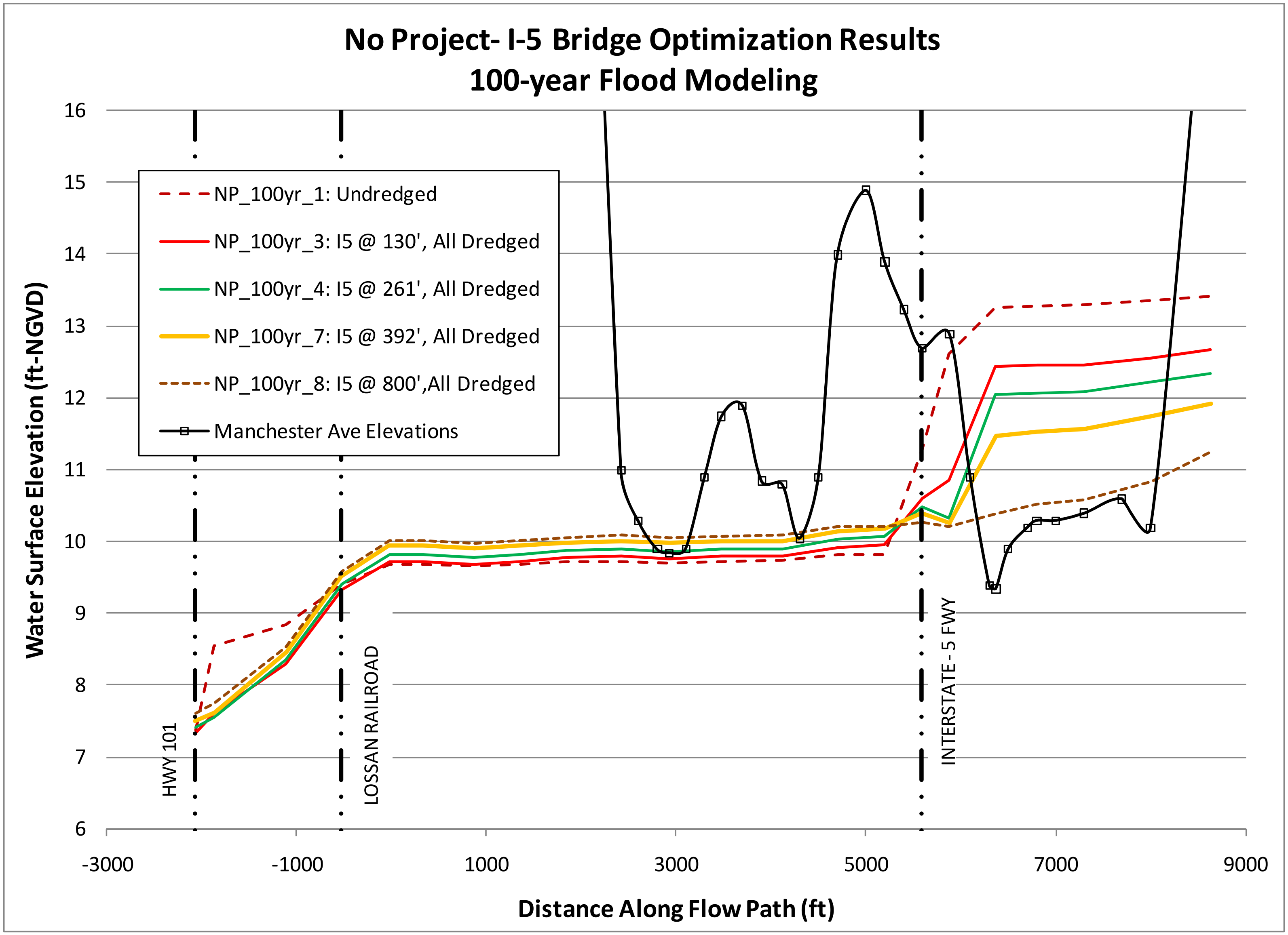

Figure 7-4: No Project Flood Optimization Profiles

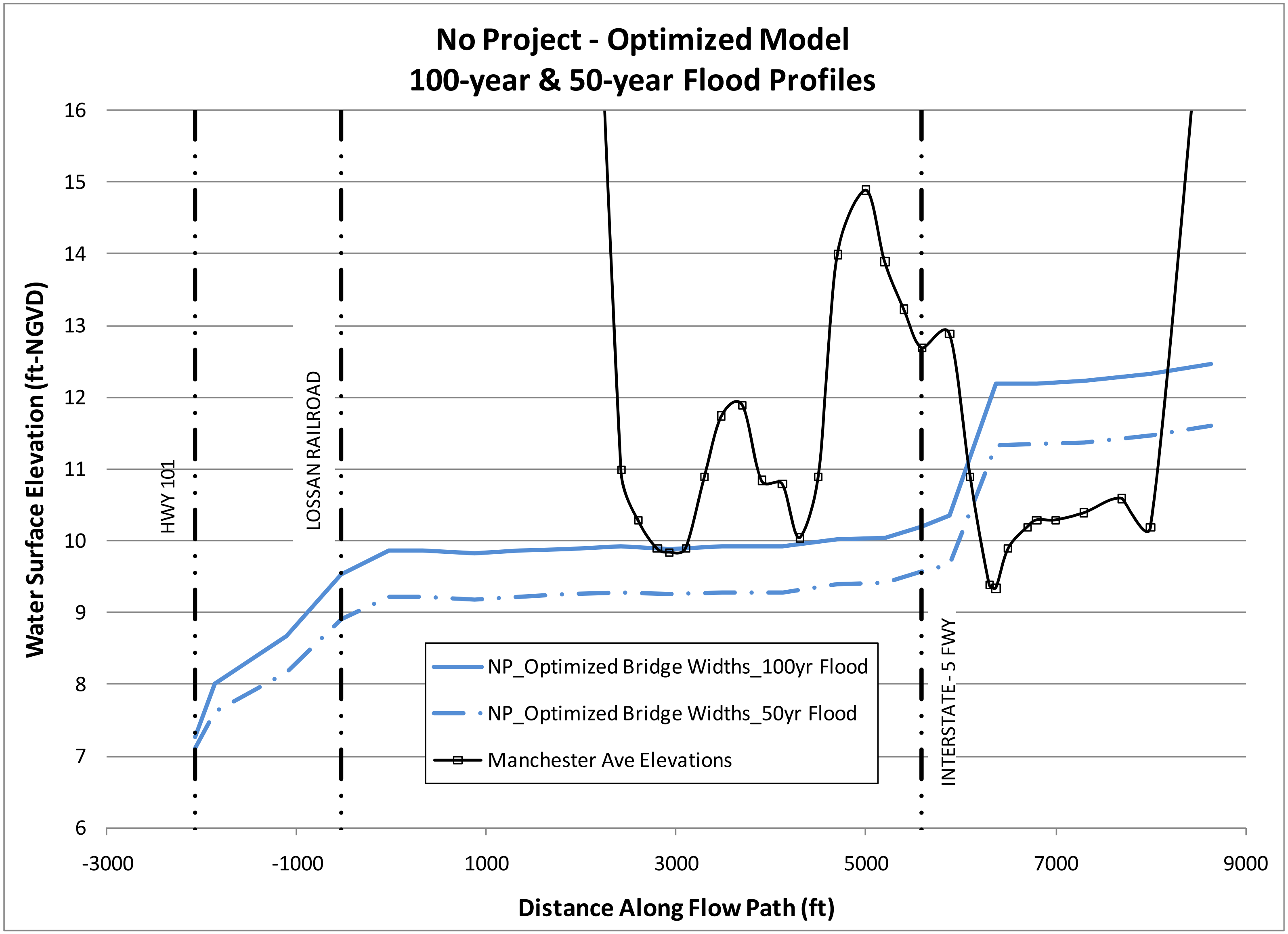

Figure 7-5: Optimized No Project Model – 50-year & 100-year Flood Profiles

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

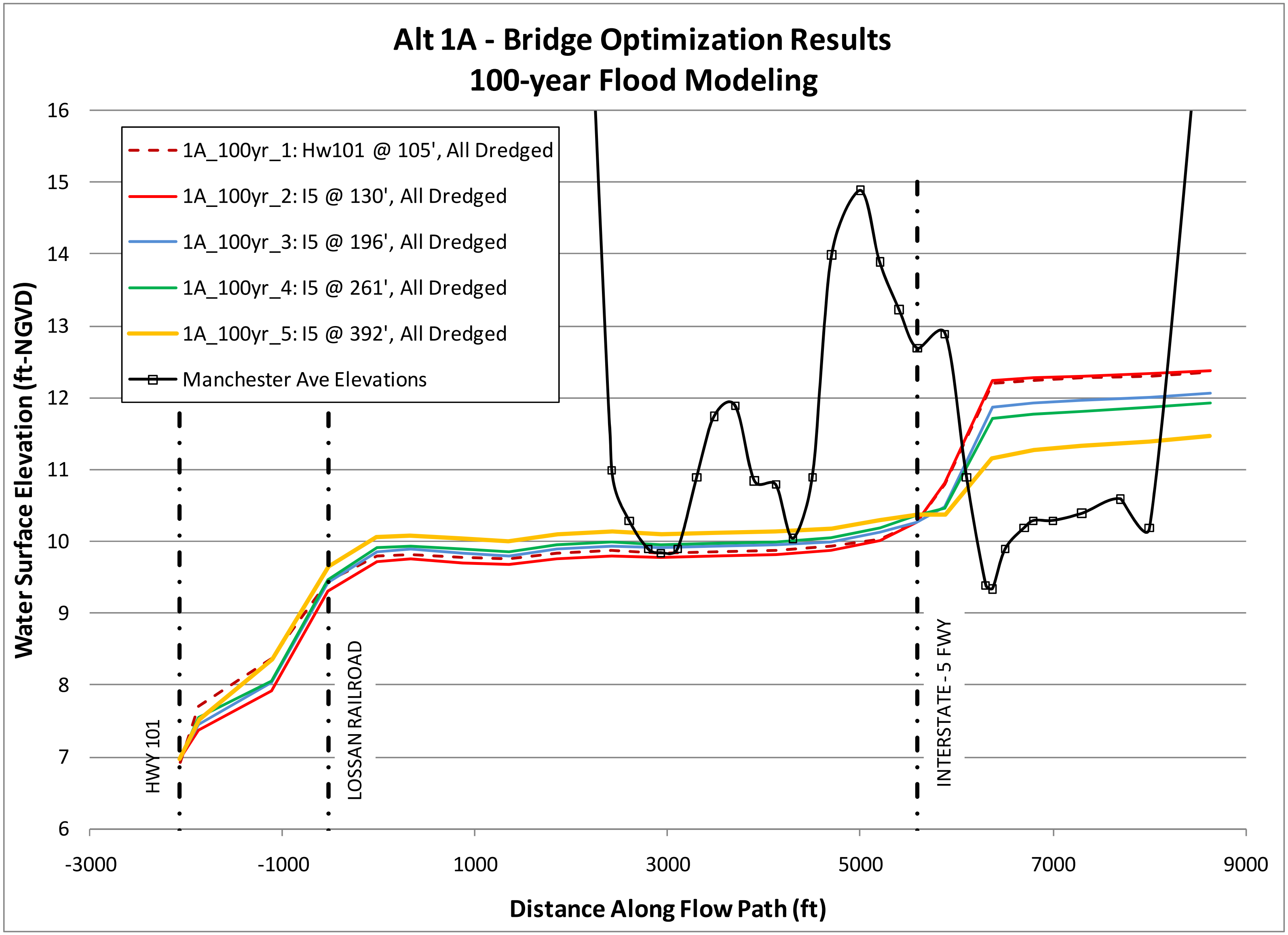

Figure 7-6: Alternative 1A Flood Optimization Profiles

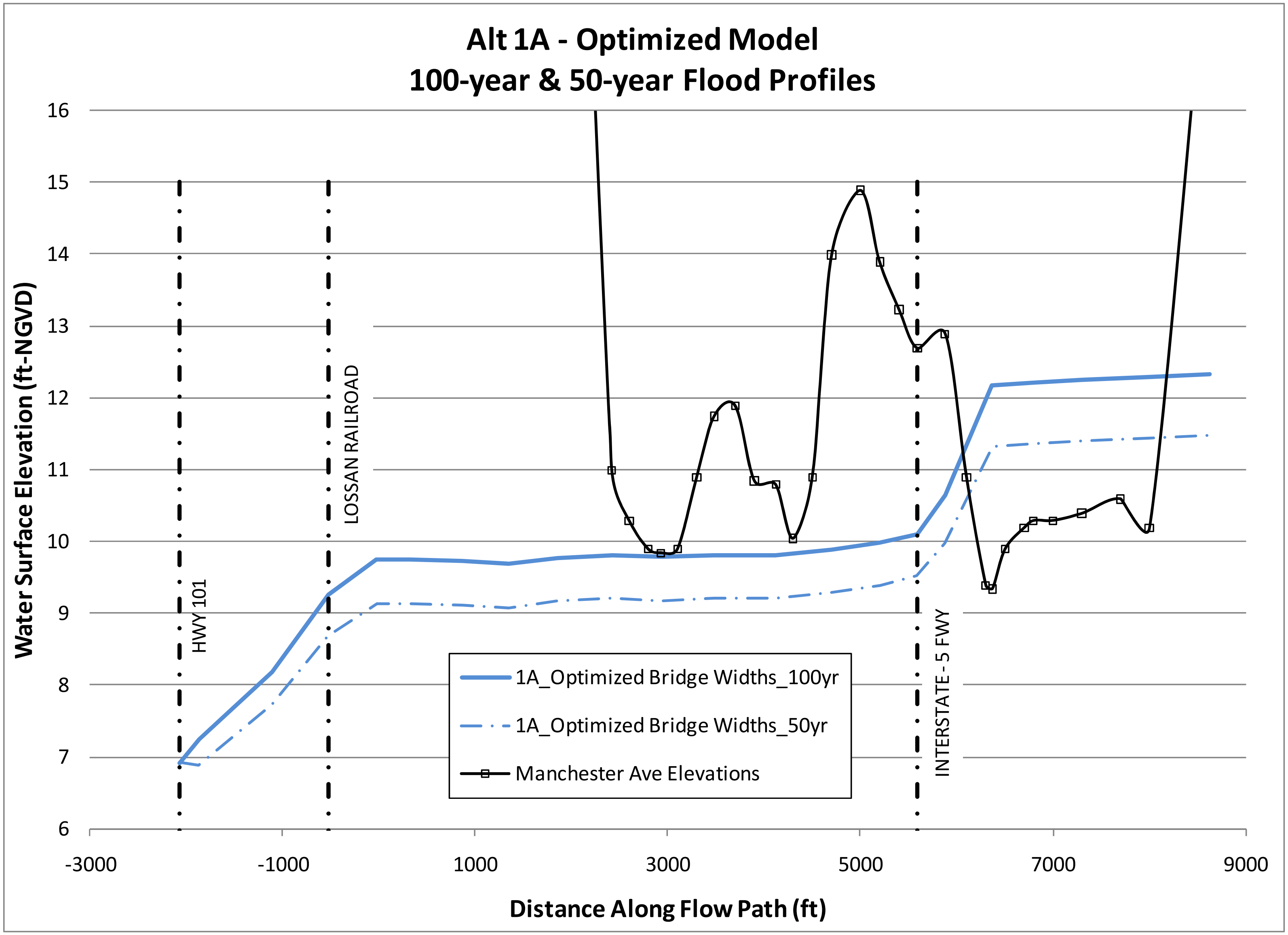

Figure 7-7: Optimized Alternative 1A Model – 50-year & 100-year Flood Profiles

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

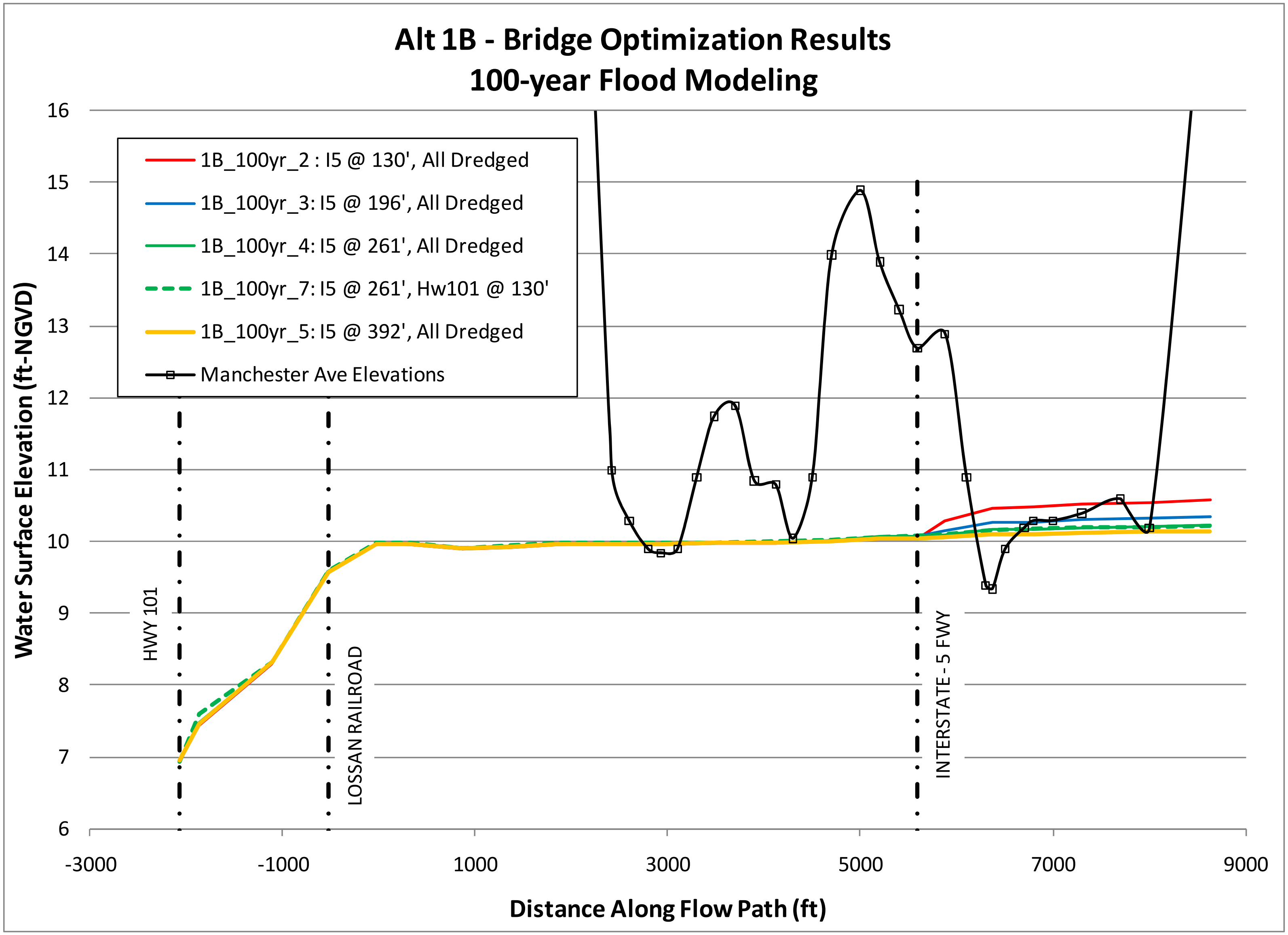

Figure 7-8: Alternative 1B Flood Optimization Profiles

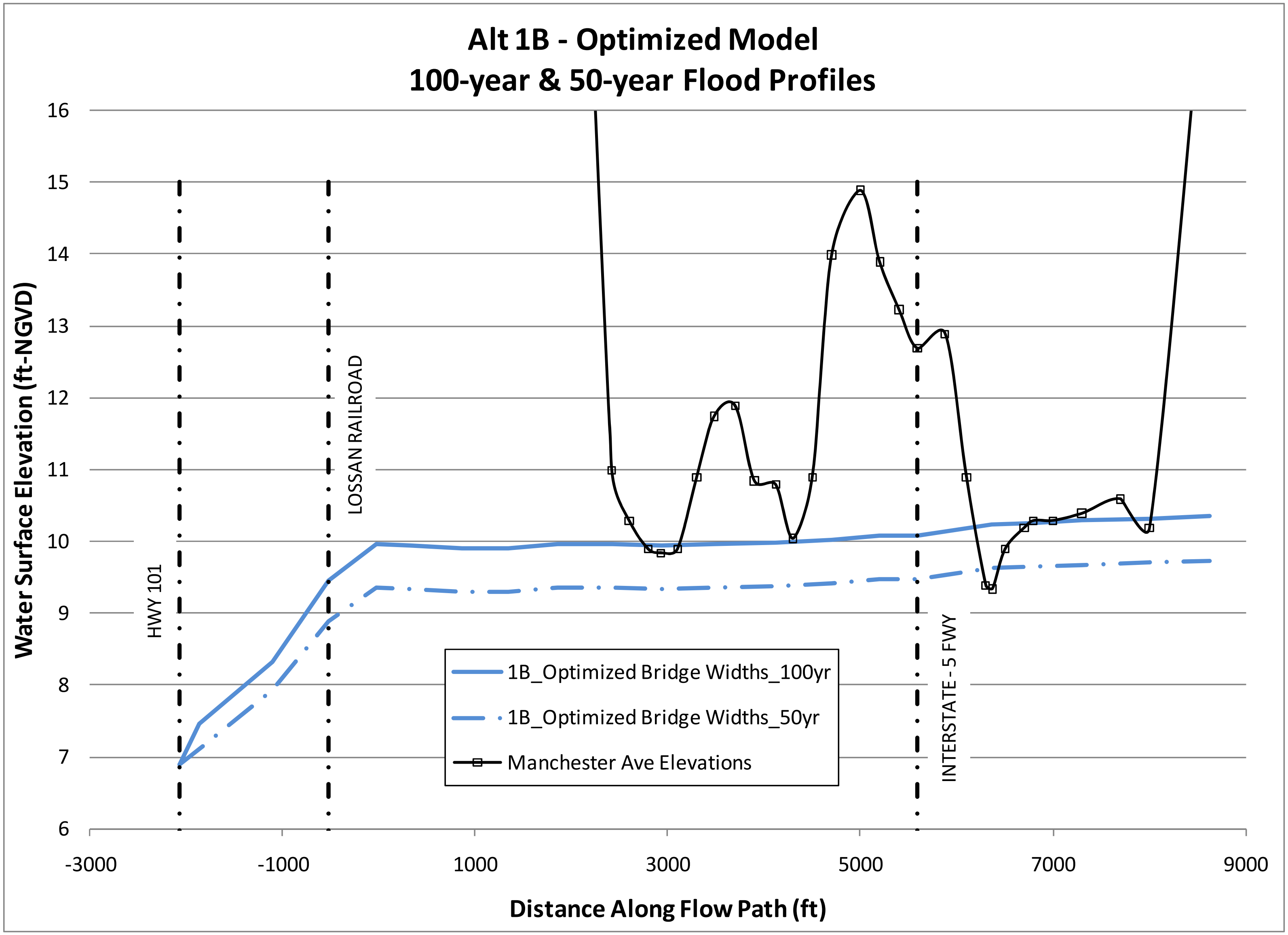

Figure 7-9: Optimized Alternative 1B Model – 50-year & 100-year Flood Profiles

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

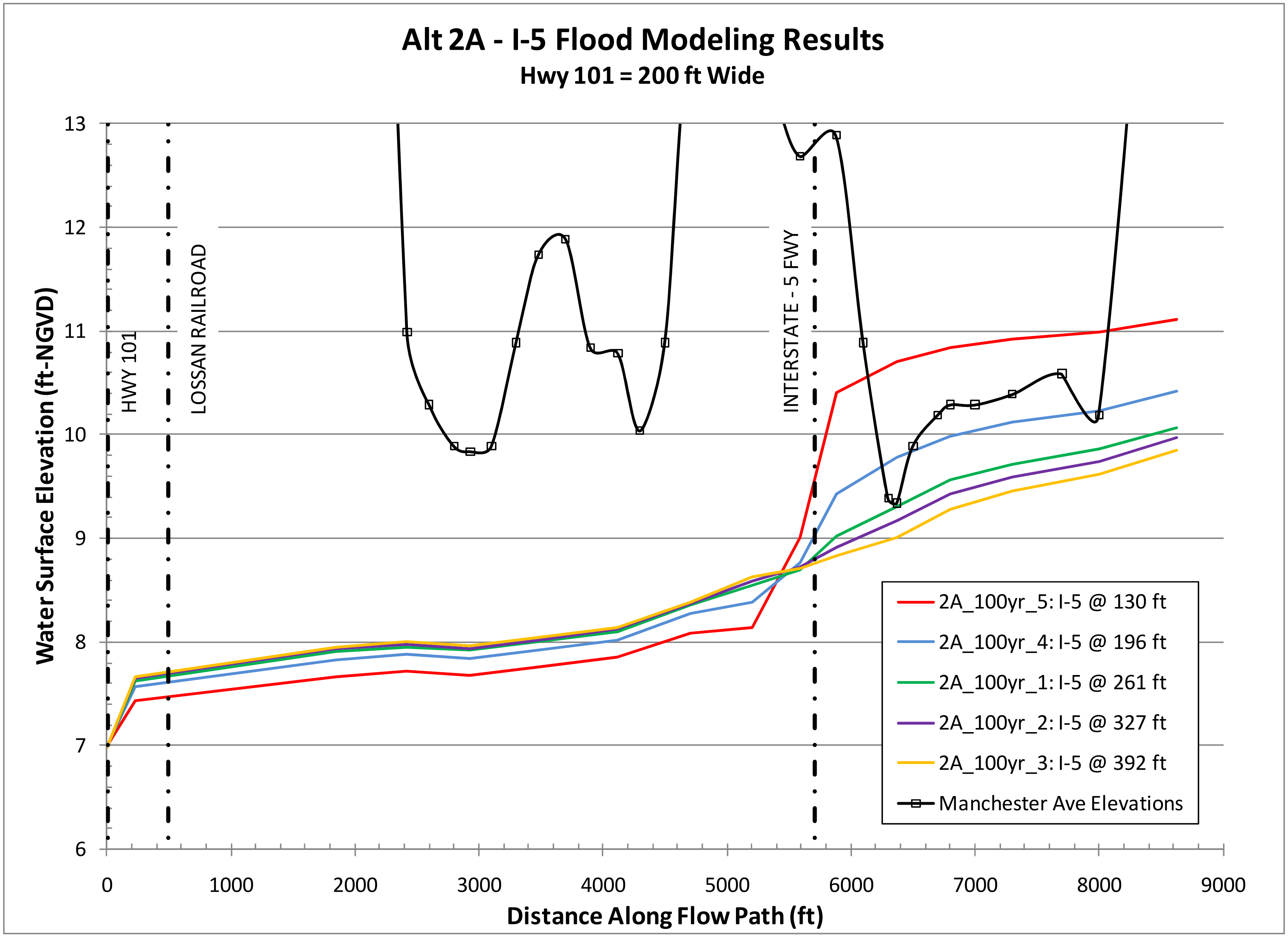

Figure 7-10: Alternative 2A Flood Optimization Profiles (Hwy101 = 200 feet)

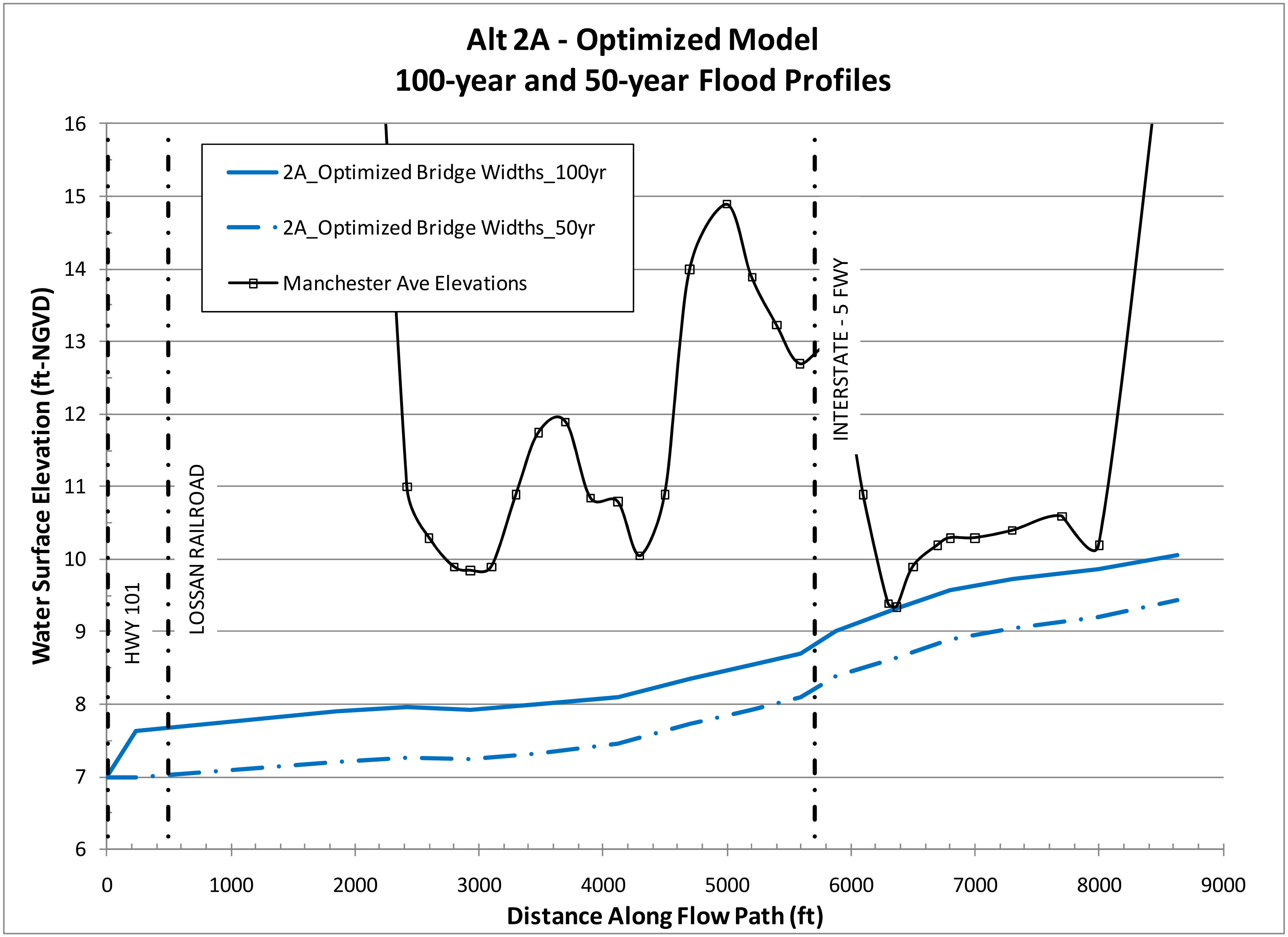

Figure 7-11: Optimized Alternative 2A Model – 50-year & 100-year Flood Profiles

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

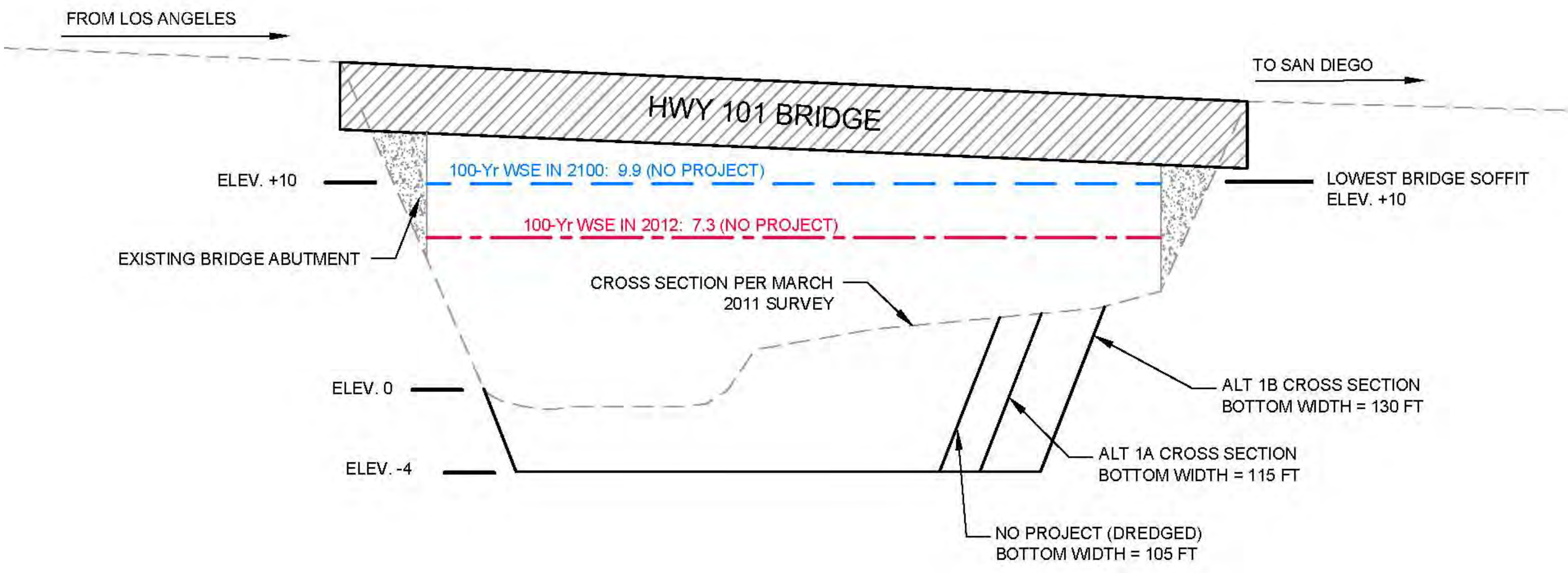

Figure 8-1: Existing Highway 101 Bridge Channel Cross-Section

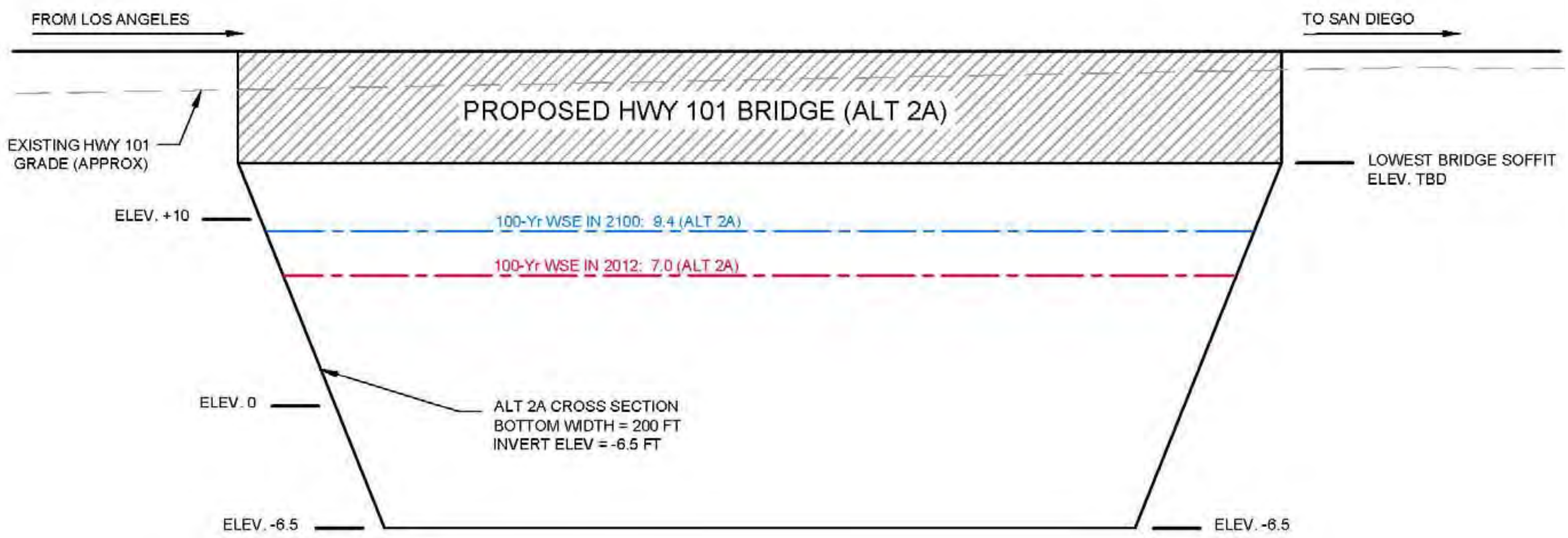

Figure 8-2: Proposed Alt 2A Highway 101 Bridge Channel Cross-Section

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

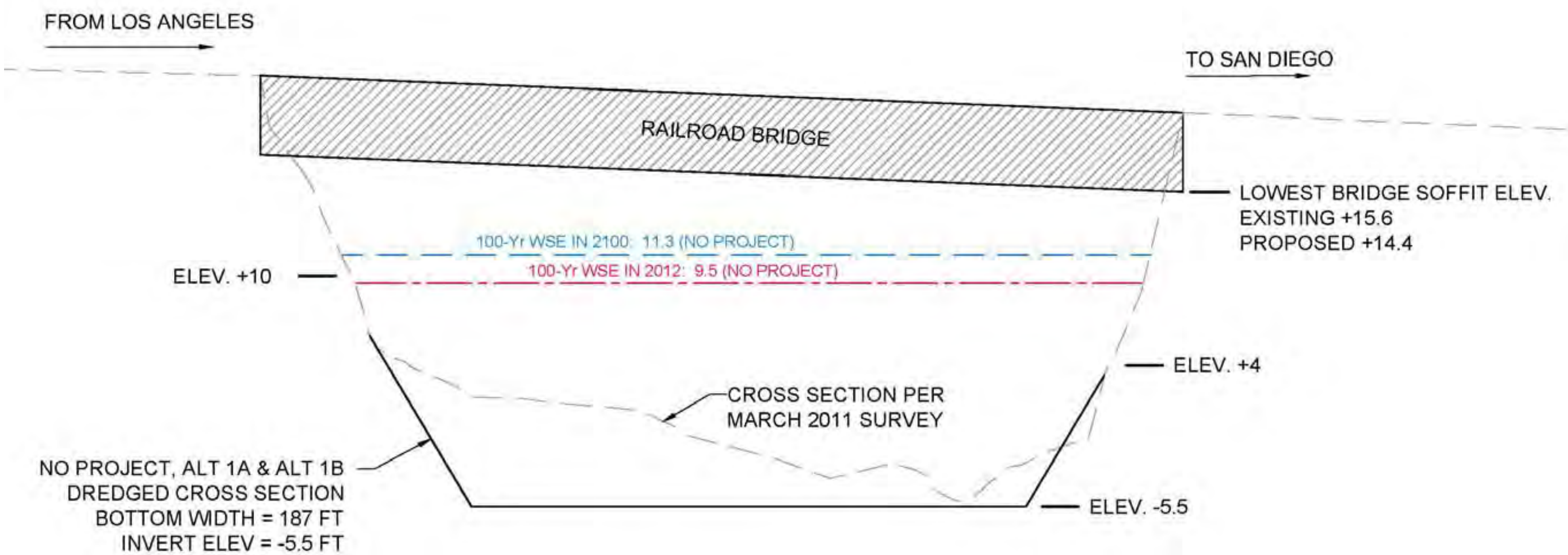

Figure 8-3: Existing Railroad Bridge Channel Cross-Section

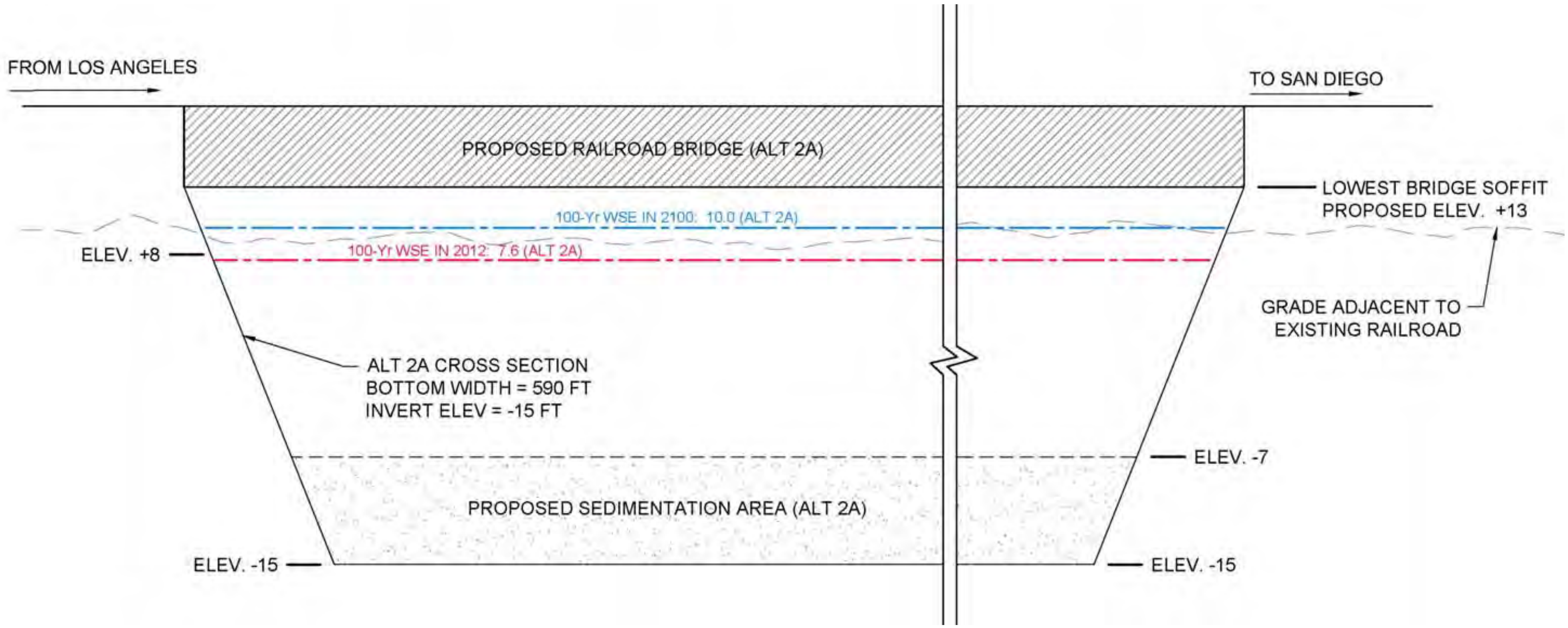

Figure 8-4: Proposed Alt 2A Railroad Bridge Channel Cross-Section

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

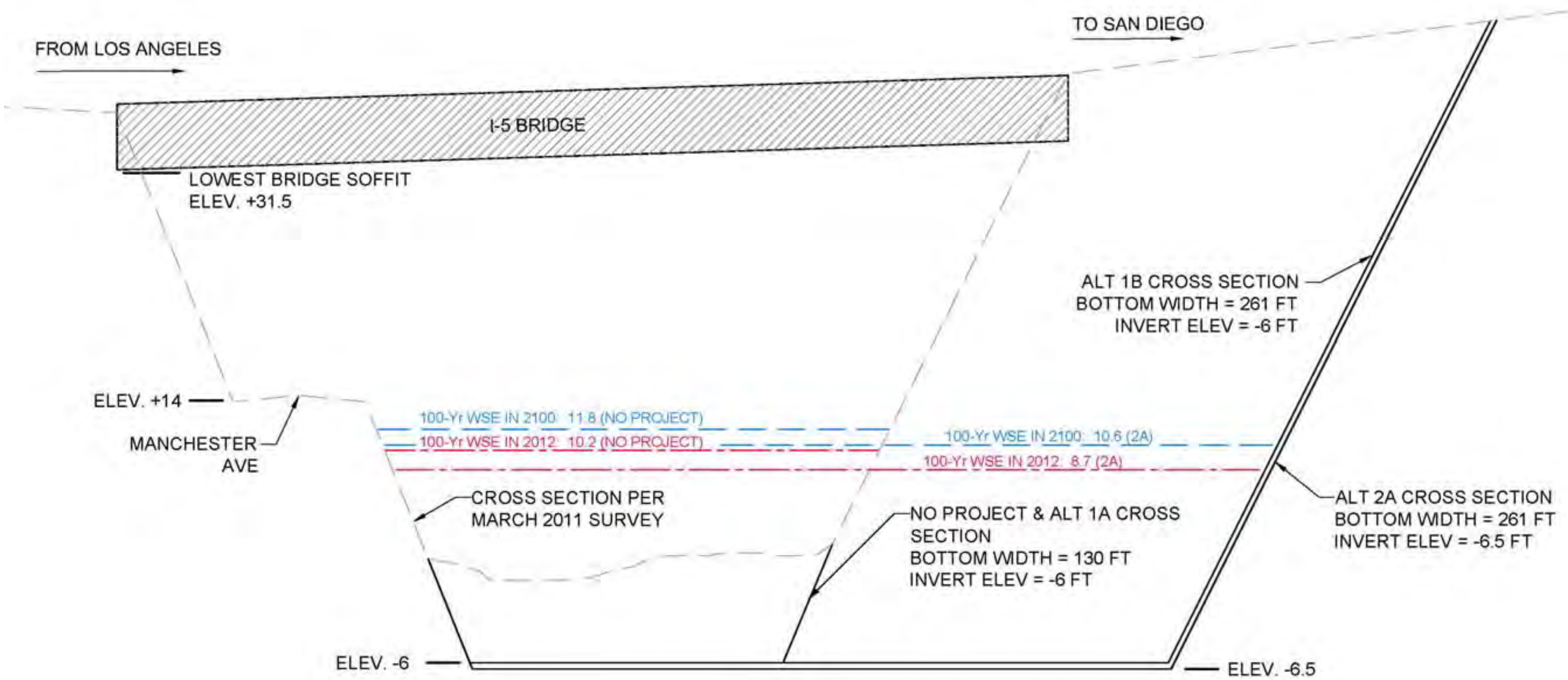

Figure 8-5: Existing I-5 Bridge Channel Cross-Section

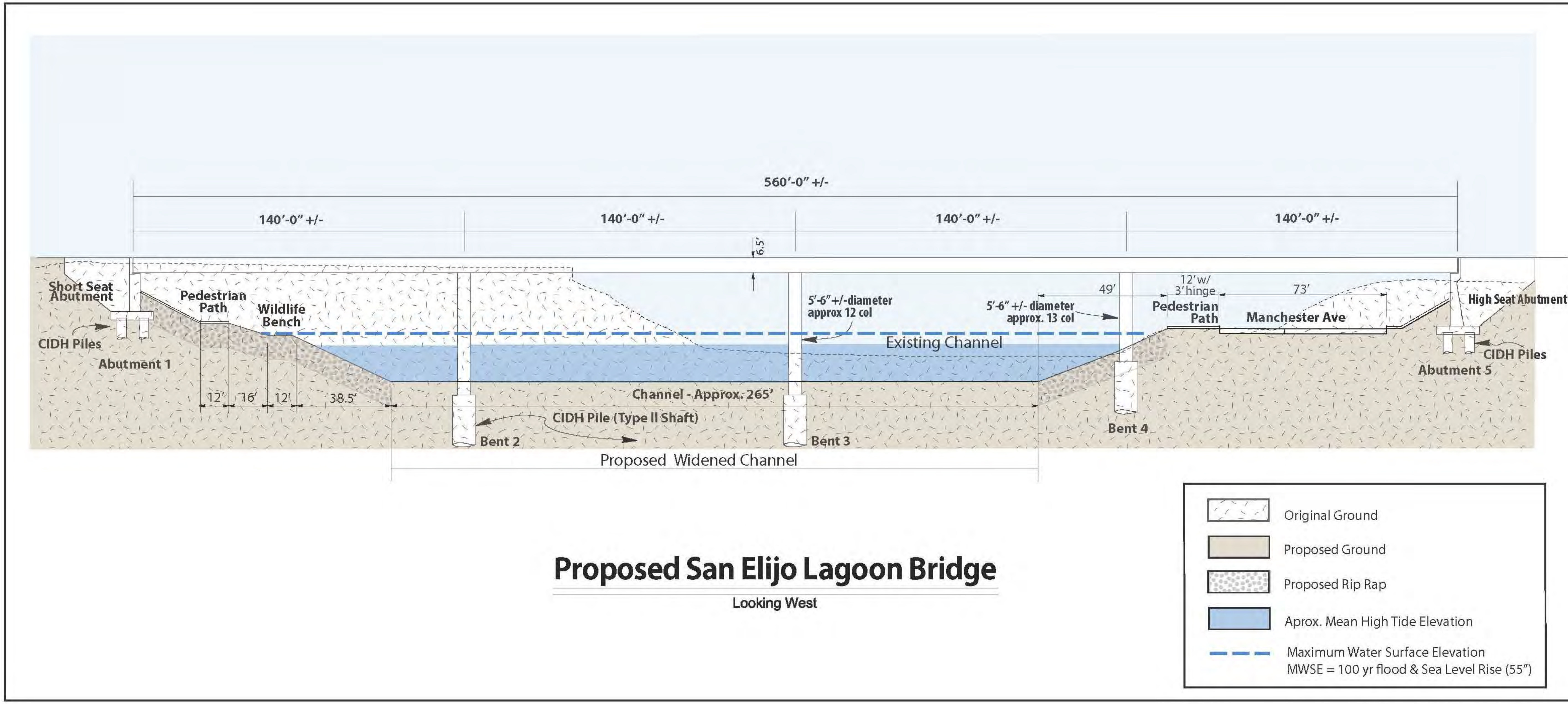

Figure 8-6: Proposed I-5 Bridge Channel Cross-Section

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]).

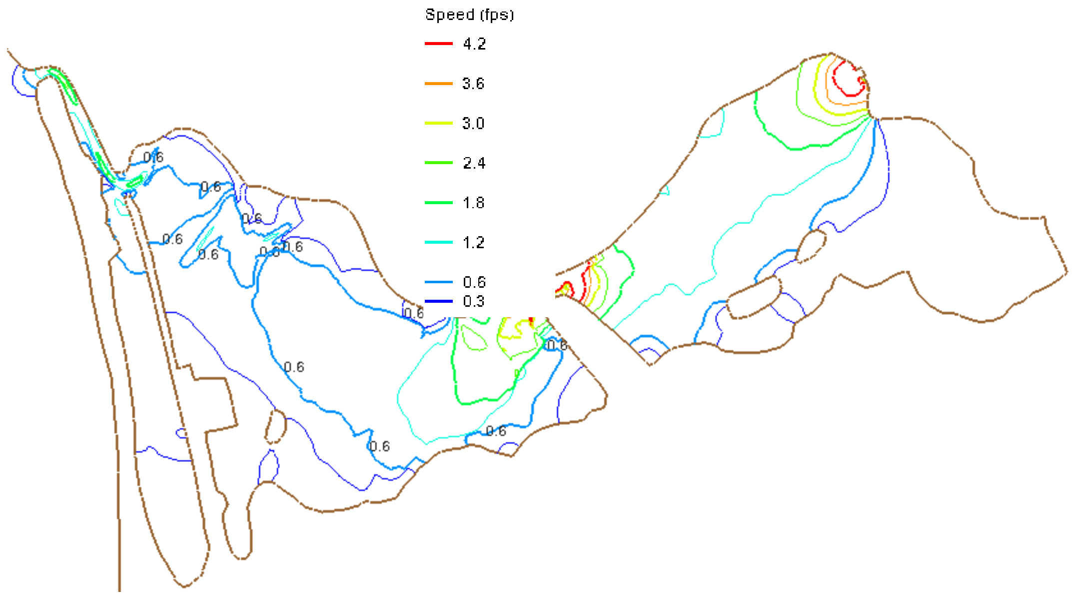

Figure 9-1: Peak Velocity Distribution During a 100-Year Event for No Project

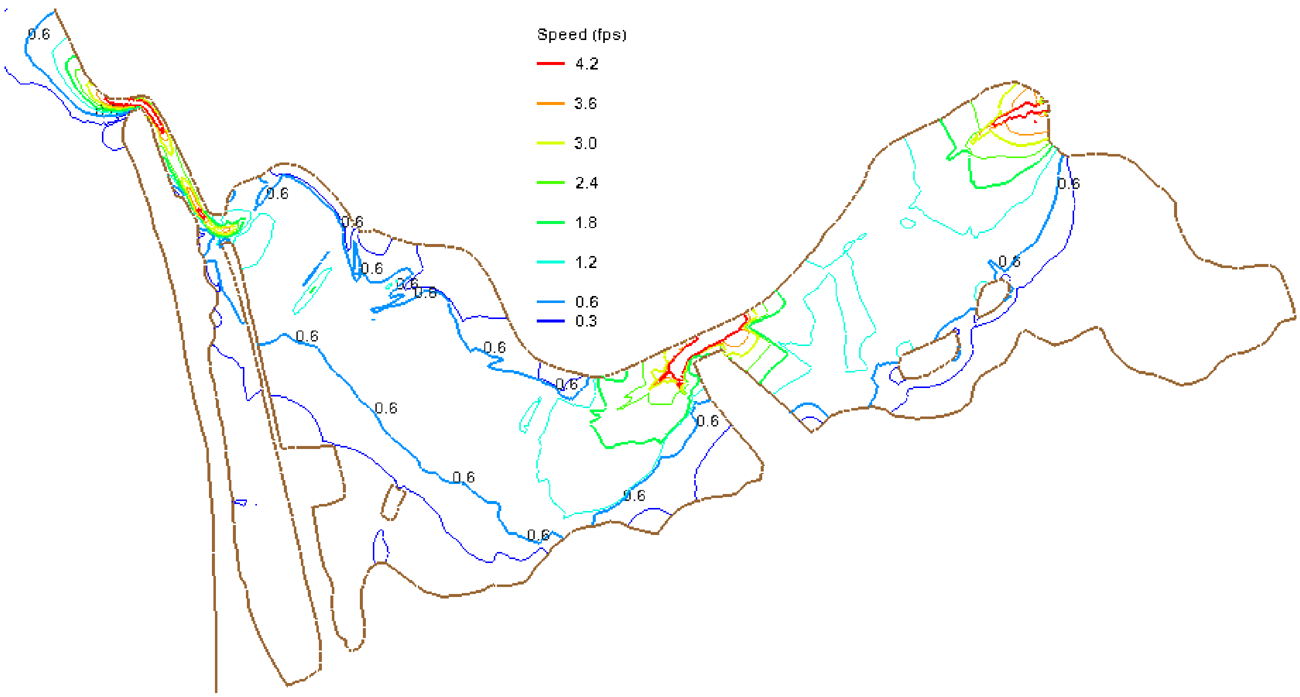

Figure 9-2: Peak Velocity Distribution During a 100-Year Event for Alternative 1A

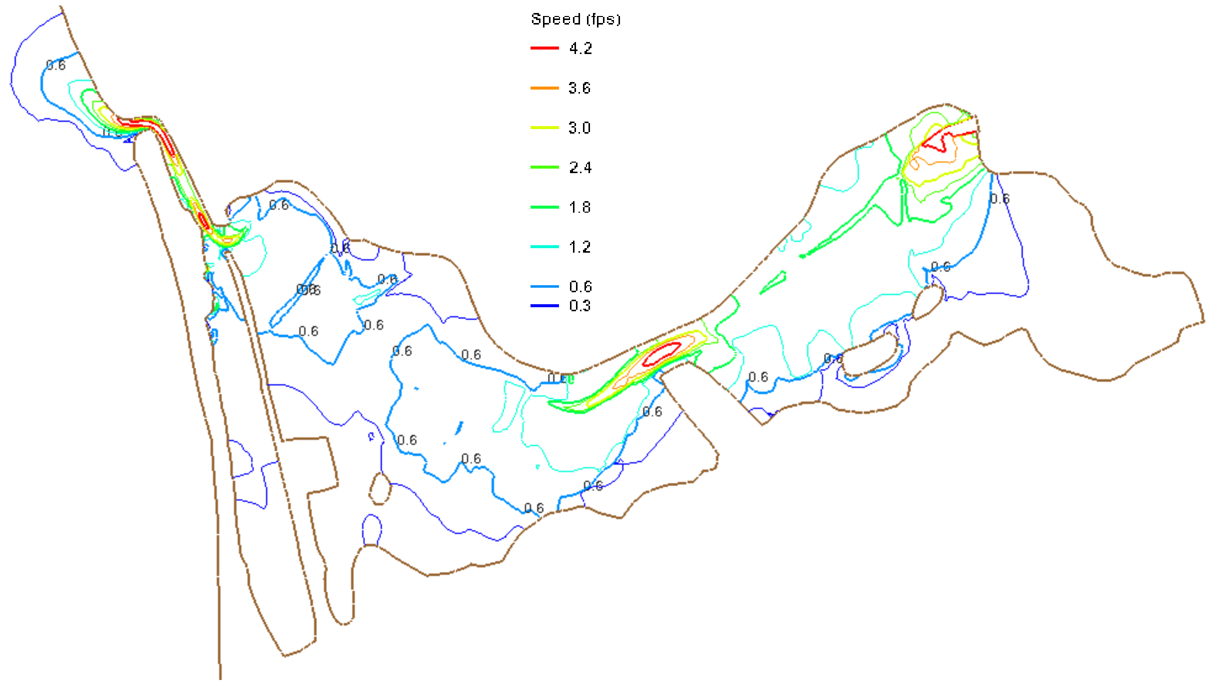

Figure 9-3: Peak Velocity Distribution During a 100-Year Event for Alternative 1B

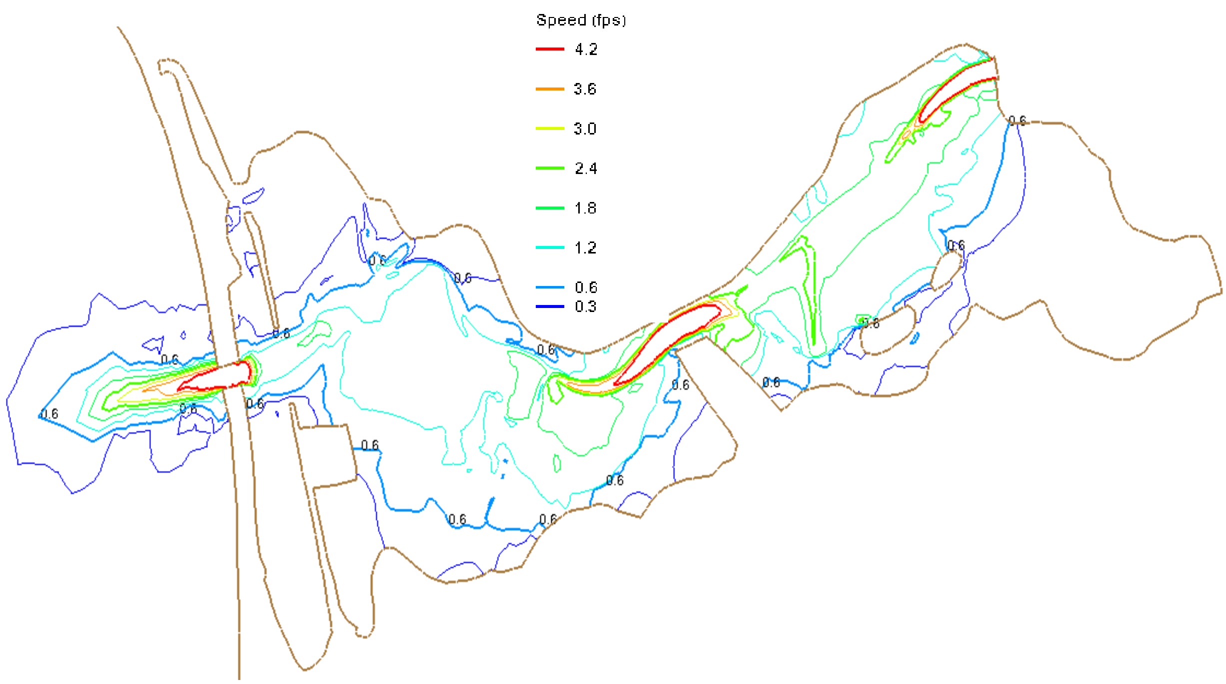

Figure 9-4: Peak Velocity Distribution During a 100-Year Event for Alternative 2A

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.

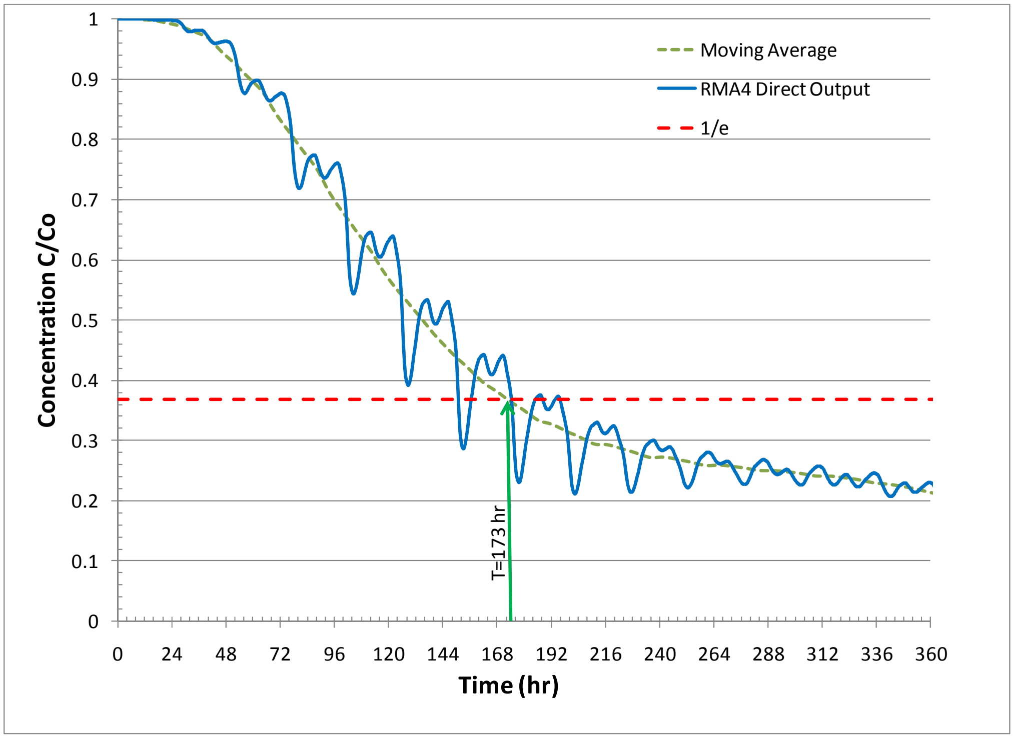

Figure 10-1: Example of a Residence Time Plot

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

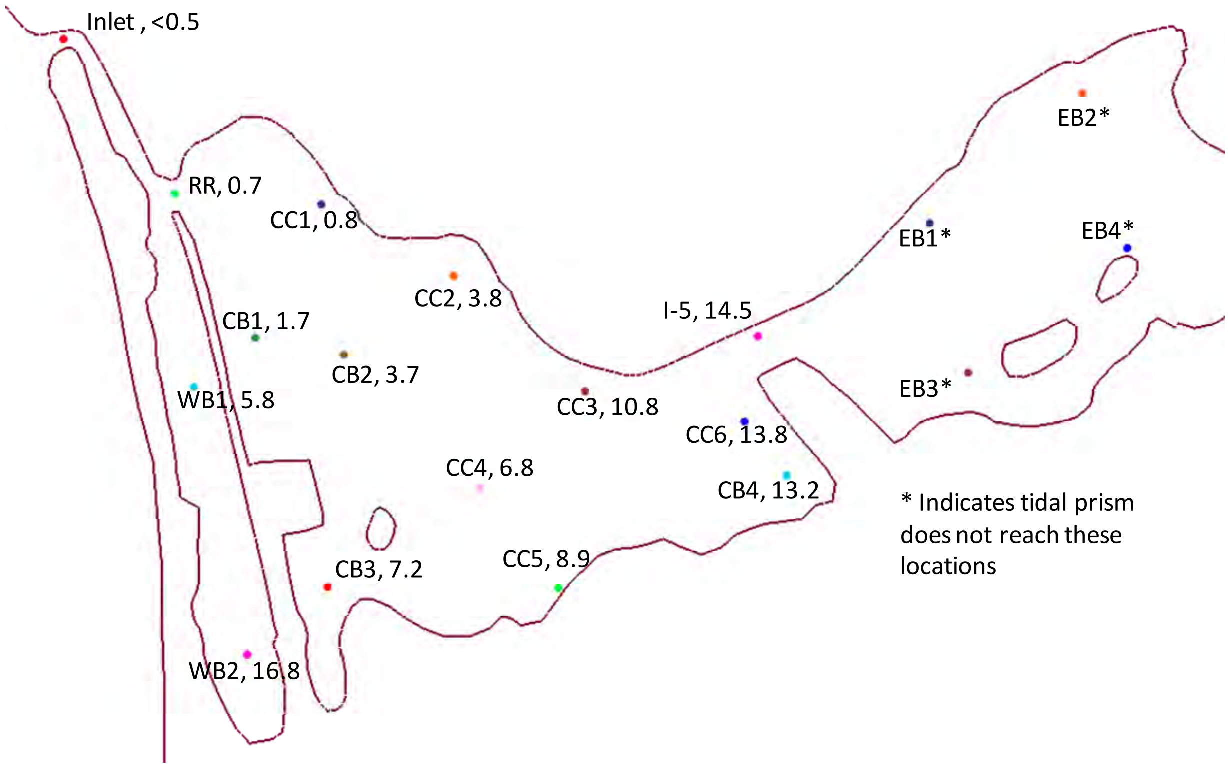

Figure 10-2: Residence Time (Days) for No Project Conditions

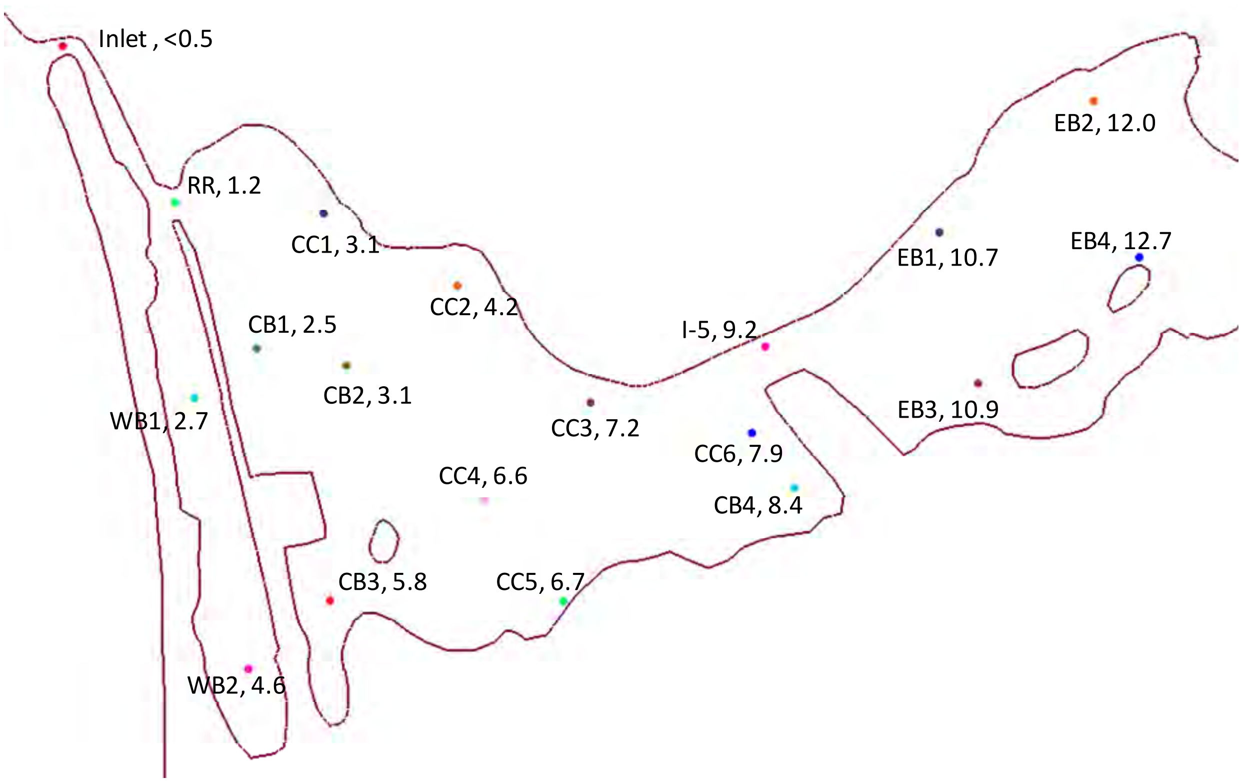

Figure 10-3: Residence Time (Days) for Alternative 1A

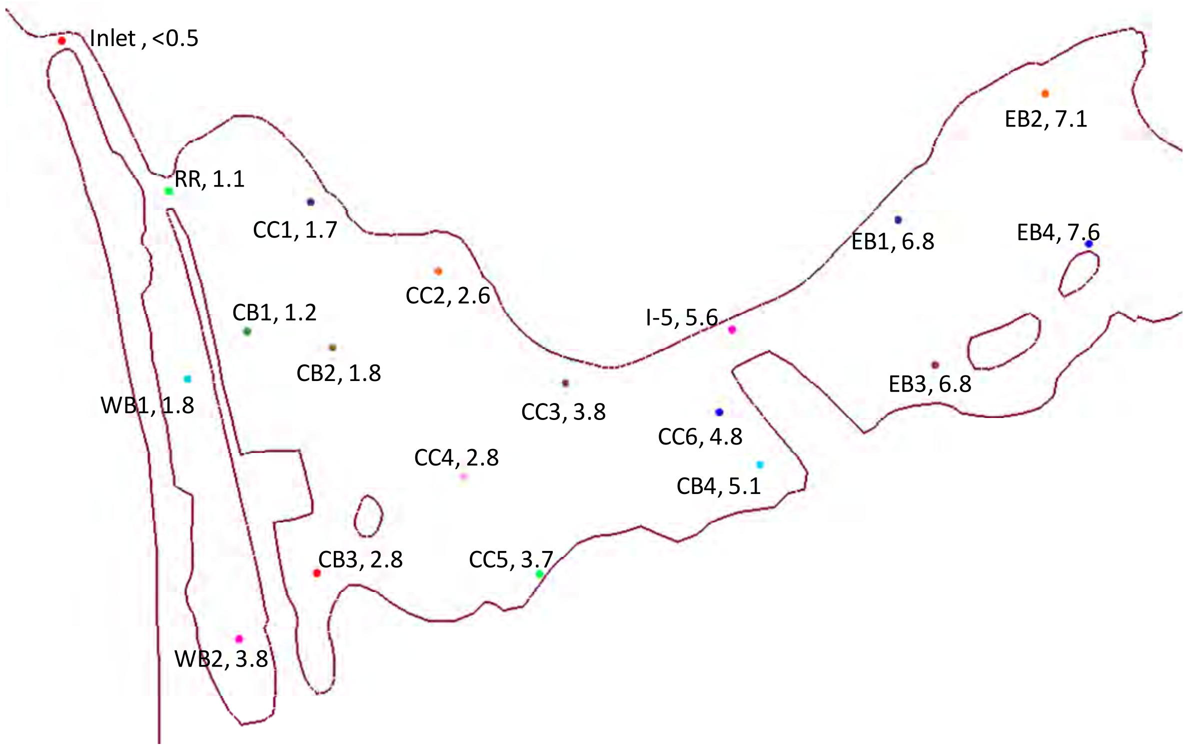

Figure 10-4: Residence Time (Days) for Alternative 1B

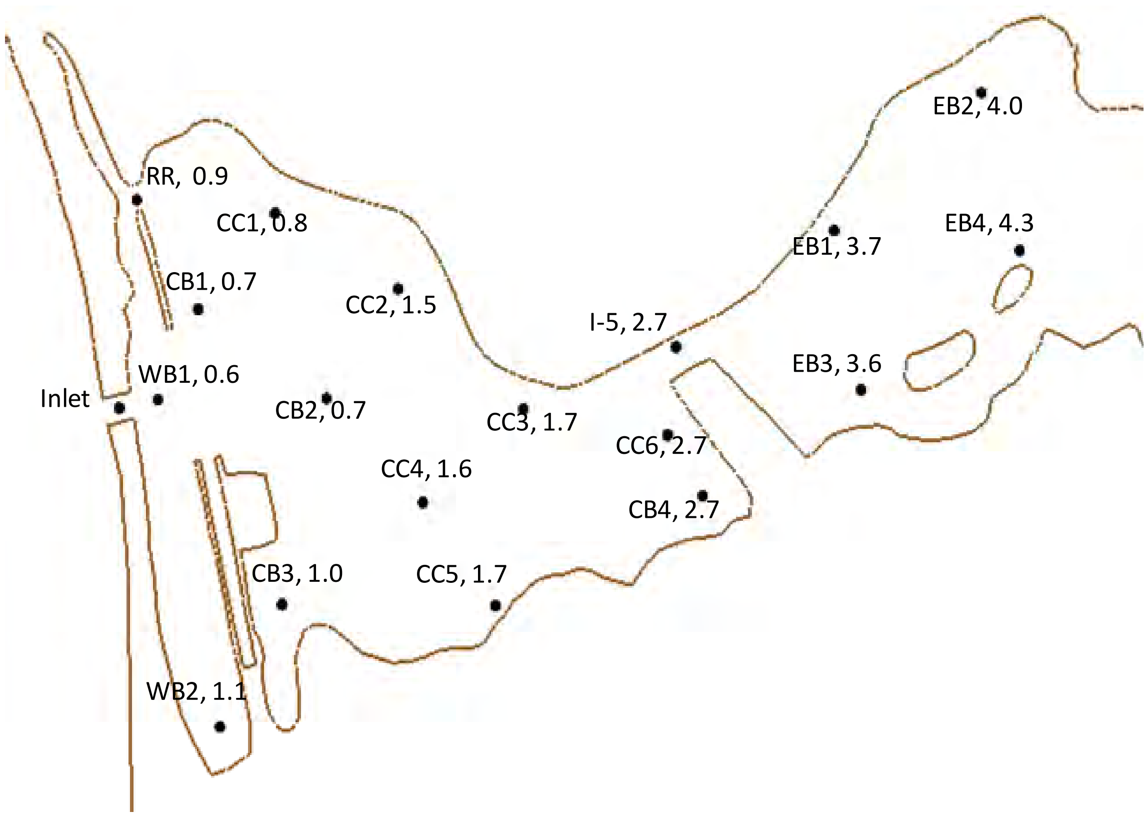

Figure 10-5: Residence Time (Days) for Alternative 2A

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.

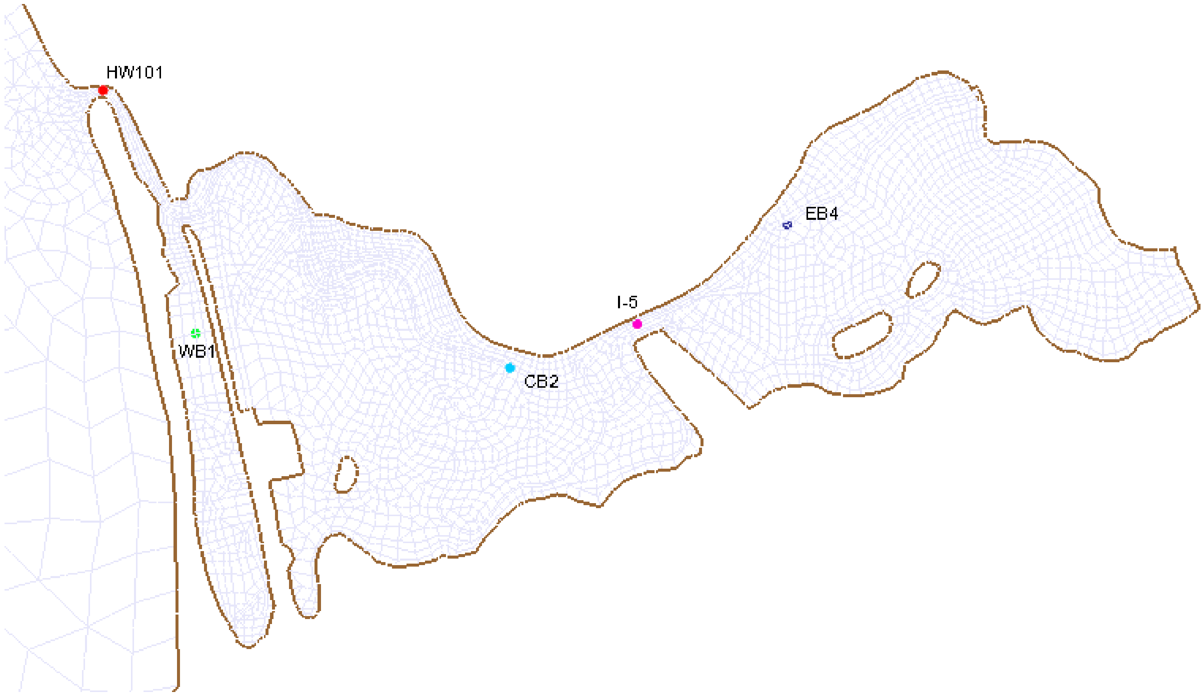

Figure 11-1: Gage Locations for Inundation Frequency Plots (No Project, Alt 1A & 1B)

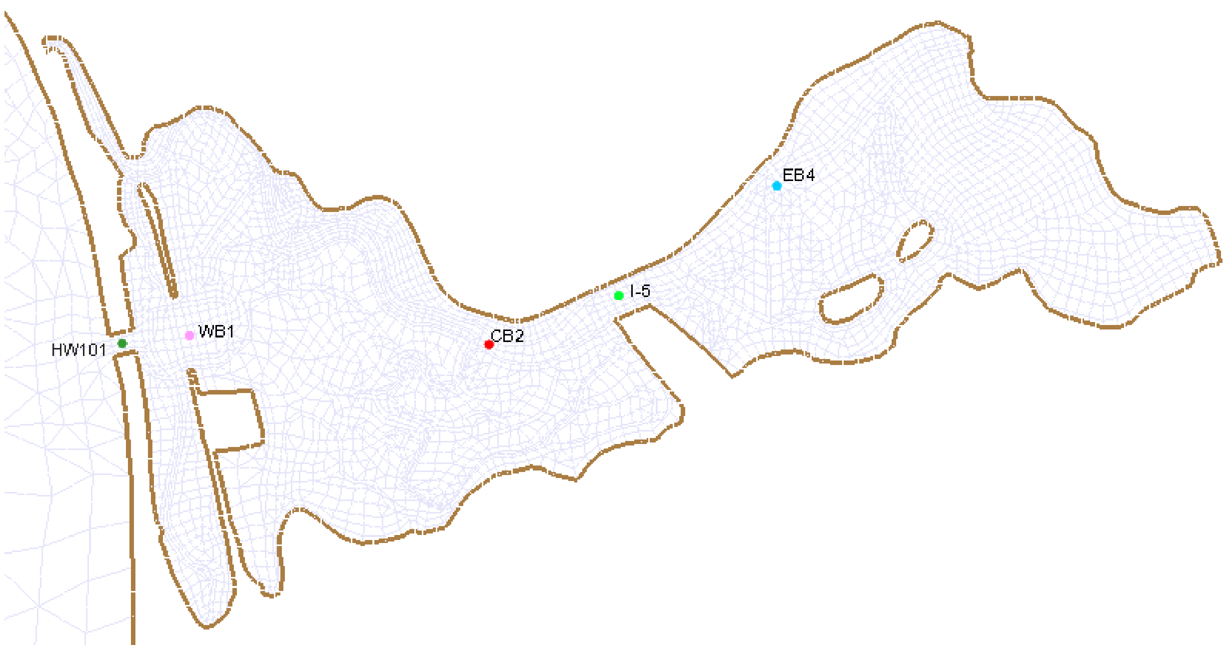

Figure 11-2: Gage Locations for Inundation Frequency Plots (Alt 2A)

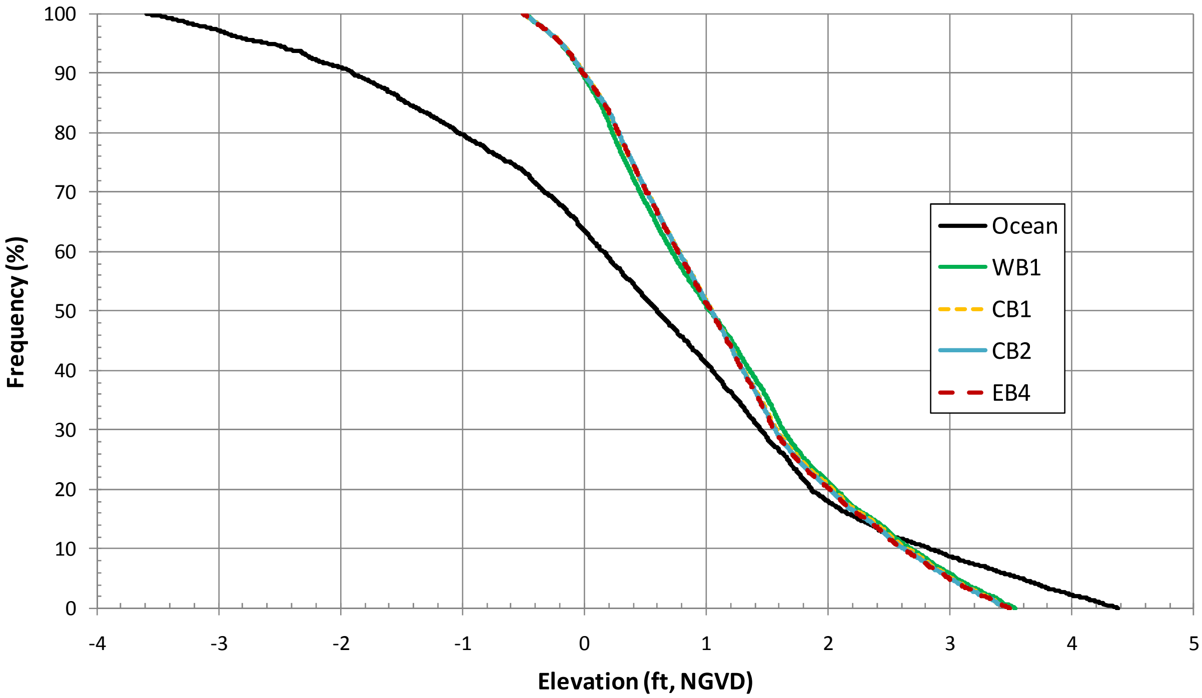

Figure 11-3: No Project Tidal Inundation Frequency

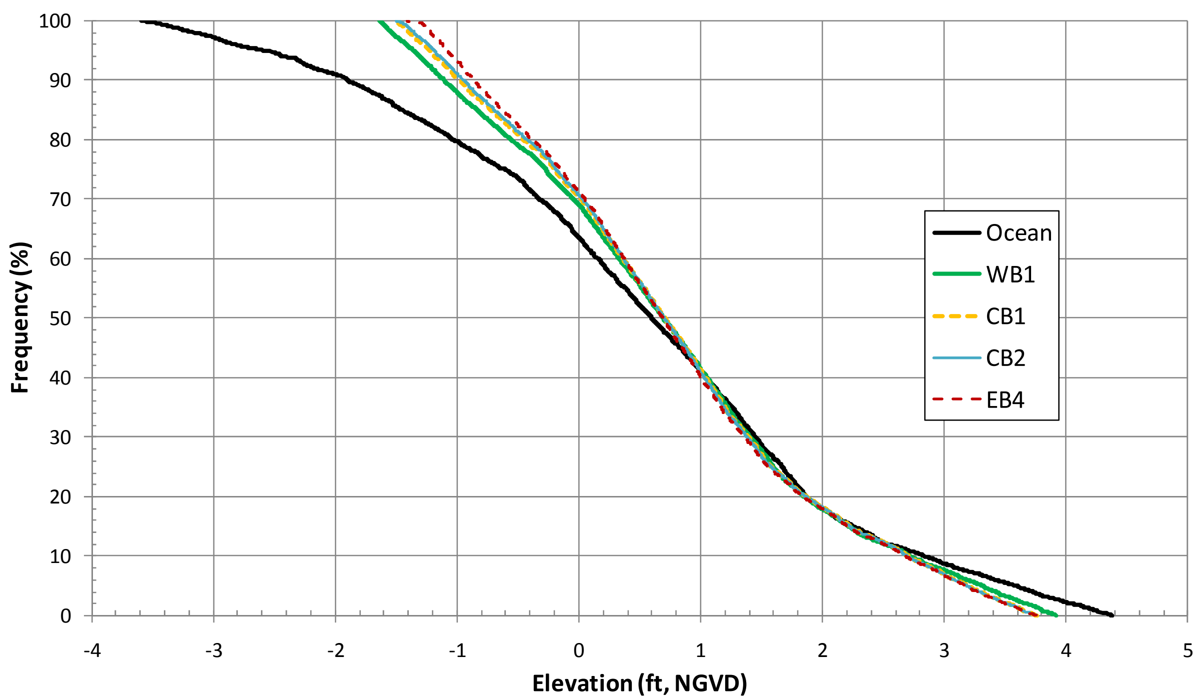

Figure 11-4: Alternative 1A Tidal Inundation Frequency

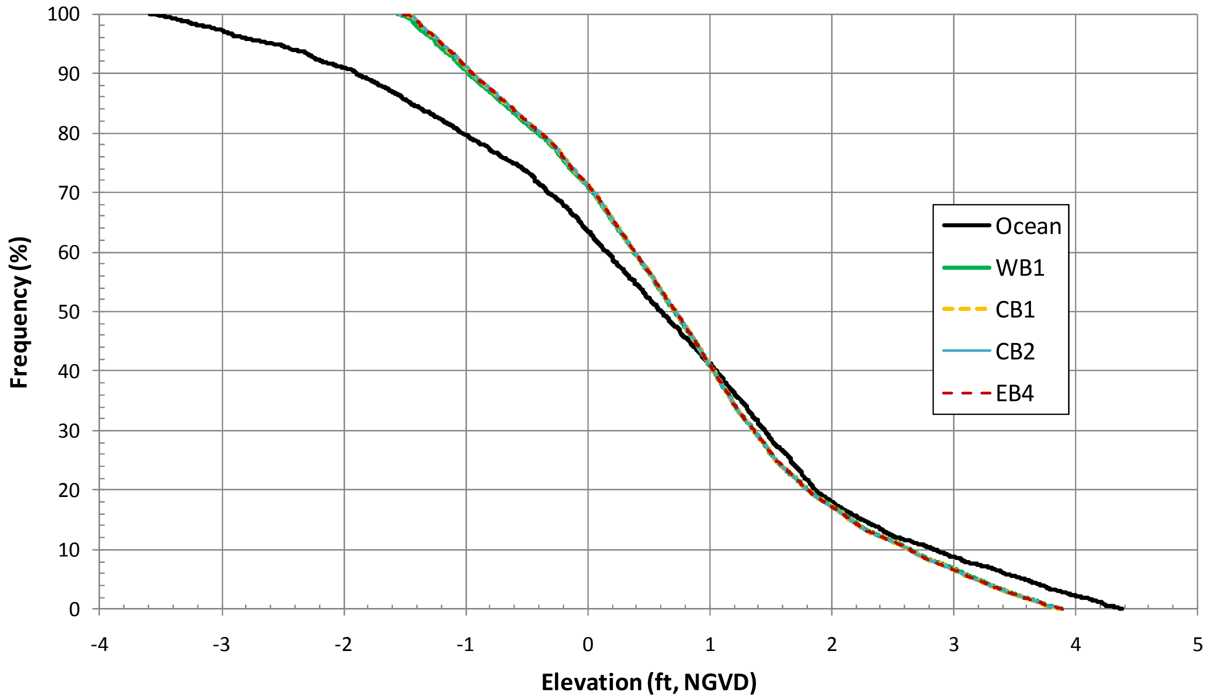

Figure 11-5: Alternative 1B Tidal Inundation Frequency

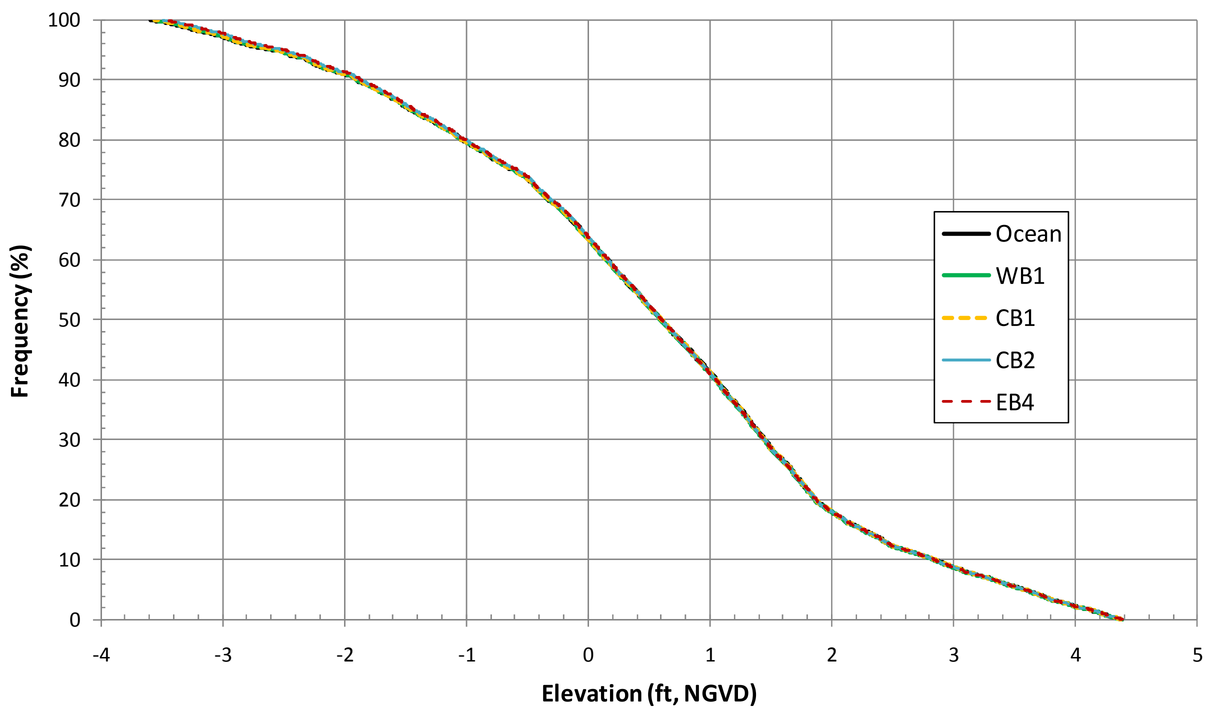

Figure 11-6: Alternative 2A Tidal Inundation Frequency

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

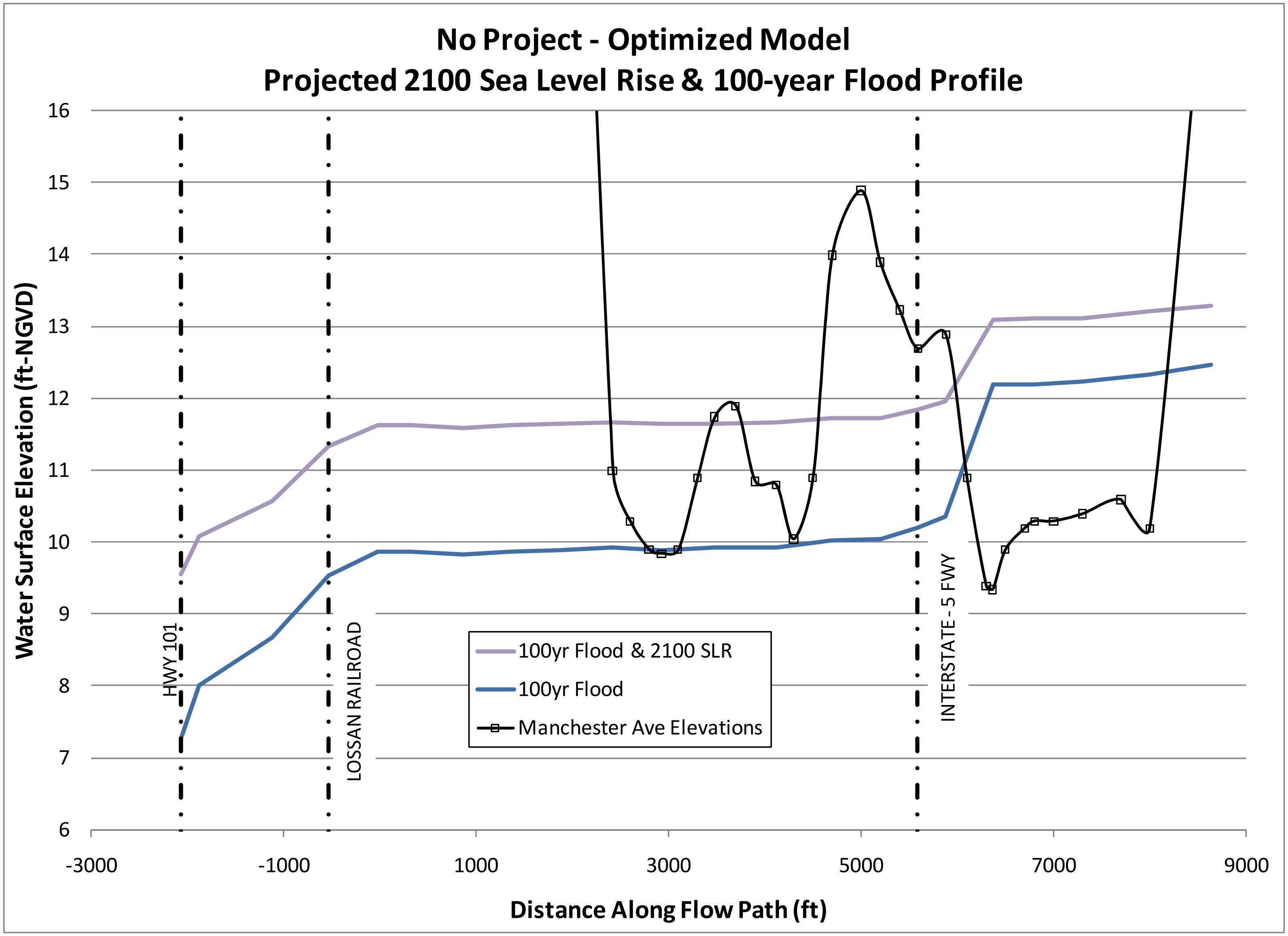

Figure 12-1: No Project 100-Year Flood Profile Comparison With Sea Level Rise

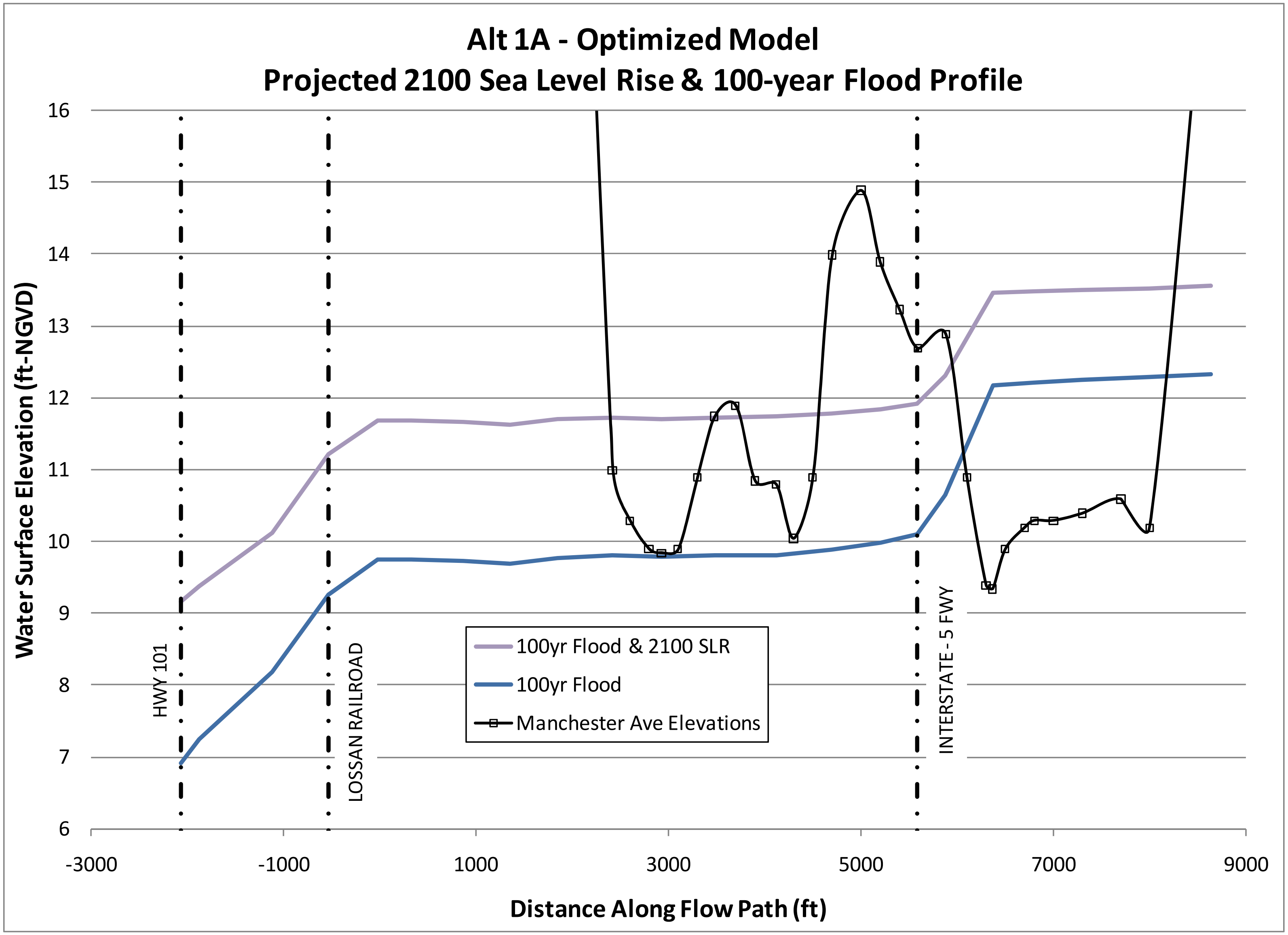

Figure 12-2: Alternative 1A 100-Year Flood Profile Comparison With Sea Level Rise

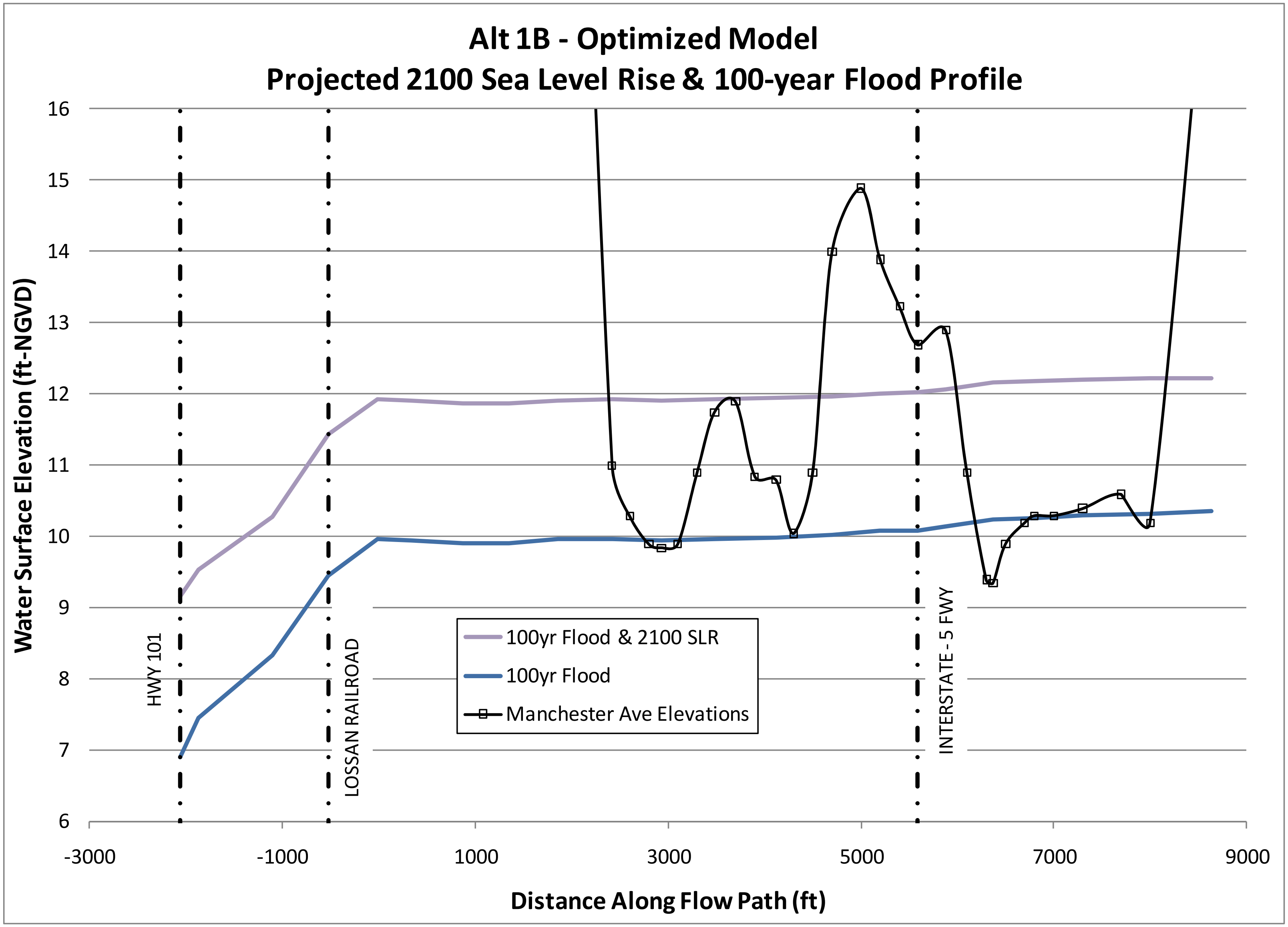

Figure 12-3: Alternative 1B 100-Year Flood Profile Comparison With Sea Level Rise

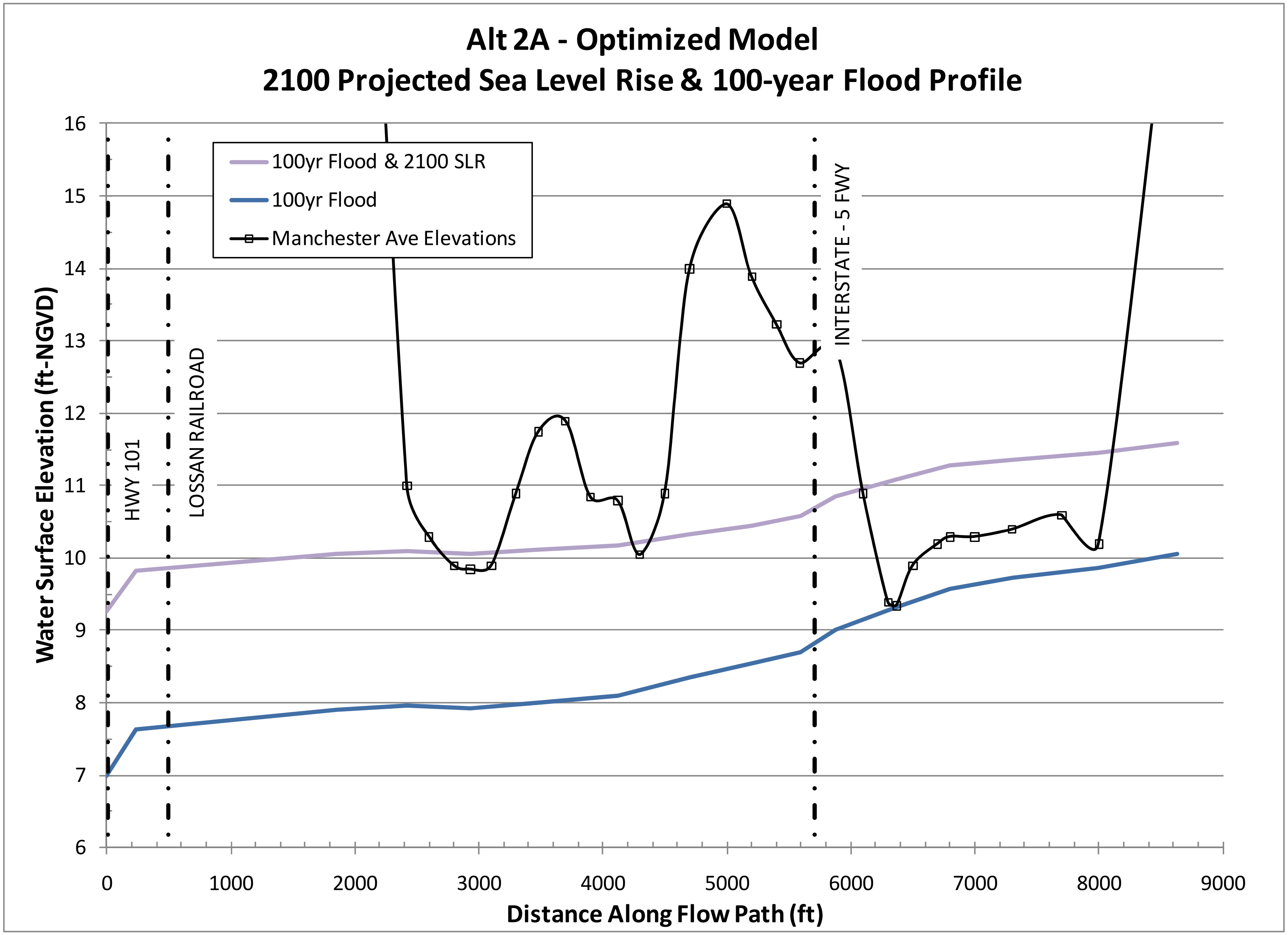

Figure 12-4: Alternative 2A 100-Year Flood Profile Comparison With Sea Level Rise

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

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